Seismic Detailing for Reinforced Concrete Buildings in Australia

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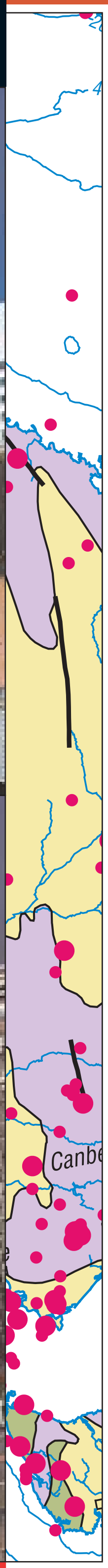

Sun Alliance Tower,Chatswood, NSW

Better Built with Concrete

STEEL REINFORCEMENT INSTITUTE OF AUSTRALIA

SRIA

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Foreword

I AM PLEASED to introduce this new publication on behalf of the Steel Reinforcement Institute of Australia. As a practicing design engineer with over 25 years general design experience and 12 years seismic design experience, the importance of correct detailing cannot be overemphasised in earthquake design.

Earthquake engineering, as we know it today, is at best a very imprecise art and much of our knowledge comes from the performance of buildings under actual earthquake loads. This has repeatedly shown that properly detailed buildings with confinement reinforcement perform far better than normally reinforced buildings detailed only for vertical loads.

Of all of the design considerations the designer must allow for in earthquake design, detailing and connection are by far the most important as an earthquake will always expose the weakest link in the structure when it strikes.

I welcome and commend this publication to you.

John Woodside

Principal Connell Wagner Pty Ltd and
Chairman BD6/4 responsible for the preparation of AS 1170.4 on behalf of Standards Australia.

Acknowledgments

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Seismic Detailing for Reinforced Concrete Buildings in Australia

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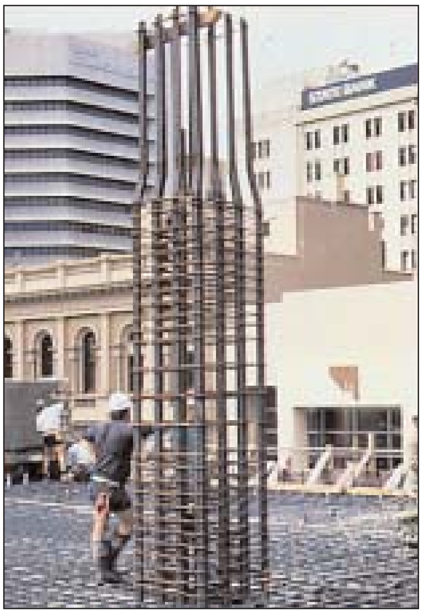


Rivers Corporation Building, Adelaide, 1992. Prefabricated column cage incorporating seismic detailing. (Note close spacing of ties adjacent to floor and ceiling).

Introduction

This publication discusses detailing requirements of AS 3600 *Concrete Structures*[1] which were amended in 1994 in response to the release of the new Australian Earthquake Loading Code AS 1170.4[2]. Following a brief overview of seismicity in Australia and relevant code requirements, the discussion focuses on reinforcement detailing aspects for building structures in regions of lower seismicity where seismic forces are still likely to have a significant impact on design. This is usually on structures from 5 to 15 storeys, as for higher structures wind loads will generally govern. For buildings lower than 5 storeys, earthquake loads may not govern due to the design category; however, seismic design might be required if the building is irregular. Wind and earthquake loadings are fundamentally different due to the cyclical nature of seismic action, while although wind loads may govern, limited detailing for seismic load will still be required. In particular, this publication shows how the requirements of the new loading code can be met through the use of predominantly simple 'seismic' details and by following general good detailing practice. Further, it will be seen how an appreciation of structural performance under seismic conditions will enable the structure to satisfactorily withstand the anticipated earthquake loads. The ease of use of these details is illustrated by commentary from Consulting Engineers and Builders involved in one of Australia's most recent seismically designed building structures. It is interesting to note that similar details have been used in Adelaide for reinforced concrete buildings over 12 metres in height since 1982. Examples of relevant overseas structures are also used to illustrate preferred construction practice. This publication is not intended to be a comprehensive design guide, and readers are referred to a number of excellent texts currently available, especially *Seismic Design of Reinforced Concrete and Masonry Buildings*[3] and *Concrete Structures in Earthquake Regions: Design and Analysis*[4].

Seismicity in Australia

The Newcastle earthquake of 1989 was one of Australia's most expensive natural disasters, costing upwards of $1.5 billion, causing 13 deaths and 120 injuries. This toll would have been much greater but for chance. The Newcastle Workers Club for instance, which collapsed during the earthquake, regularly held major events at which hundreds of people would be present. At the time, research into earthquake engineering was underway at academic institutions, and the Australian Standards Code Committee for a new earthquake loading code to replace AS 2121 was already formed. However, the event threw into sharp relief the need for the building industry to place greater emphasis on seismicity in the design and construction of our building structures. The philosophy now implicit in the seismic design of building structures is to:

- minimise loss of life, structural collapse and damage; and
- improve post-disaster recovery.

Whilst the Newcastle earthquake was a major natural disaster and the most expensive insurance loss in Australian history, it should be placed into perspective.

Australia is the world's sixth largest country (almost 8 million square kilometres), but its population of approximately 17 million is predominantly located in a coastal strip between Brisbane and Adelaide and concentrated in relatively few major urban centres. Australian earthquakes tend to be shallow, of short duration, and although they may be felt over a great distance, have a relatively small area of influence in contrast with those experienced in New Zealand, Japan and California. These considerations result, therefore, in Australia being classified as a 'low-risk' but 'high-consequence' area in terms of earthquake damage, ie the likelihood of an earthquake occurring in a major urban area is low but the consequences, should one occur, are likely to be severe. Furthermore, on average, Australia experiences earthquakes of Richter magnitude 6 or greater every five years, ie with amplitudes of ground motion of 2.5 times and released energy at least 4 times as great as those experienced in the Newcastle earthquake which measured magnitude 5.6 on the Richter scale. However, in intraplate areas such as Australia, it is not possible to accurately predict the time, location and intensity of earthquakes.

Both AS 1170.4 and Appendix A of AS 3600 (which are discussed below) are based on US experience modified for Australian seismological conditions and building practices. The requirements of these two codes allow for the possible seismological forces which a building structure situated in Australia may reasonably be expected to undergo at some point during its life.

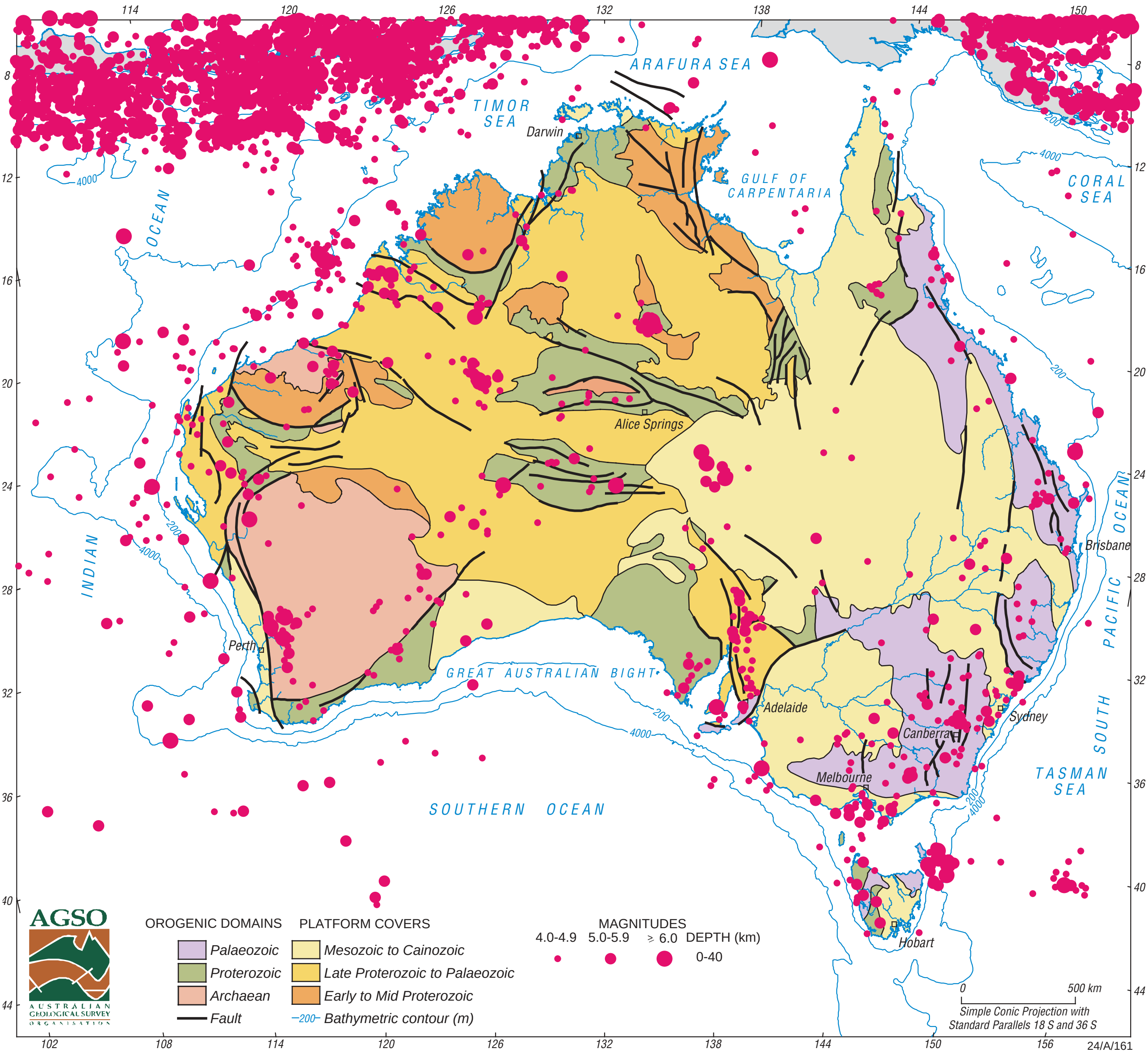


***Figure 1** Epicentres of Australian earthquakes in the period 1859 to 1992 with magnitudes of ML4 or greater. (Illustration courtesy AGSO)*

Reinforced Concrete and Earthquakes

As discussed above, due to the relatively short history of seismological measurement in Australia, it is not possible to predict the response spectra, time or location of a seismic event with any degree of accuracy. Even overseas in countries more culturally adapted to earthquakes, surprises can occur which cause major catastrophes.

The Kobe, or Great Hanshin, earthquake of 17 January 1995 and the Northridge, Los Angeles, earthquake of 17 January 1994 were just two examples where this has been the case. Both of these events highlighted the strengths and weaknesses of reinforced concrete in terms of design methodologies and as a structural material.

It is widely recognised that reinforced concrete does suffer from a number of drawbacks when compared to steel or timber structures since concrete:

- has an unfavourable mass to strength ratio;
- exhibits brittle behaviour when failing in shear – particularly for low levels of shear reinforcement;
- possesses a lack of ductility in compression when inadequately confined.

The mass to strength ratio is important because earthquake loads arise from inertial effects, and so are proportional to mass. Clearly, concrete is at a distinct disadvantage here 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 transverse confining steel greatly increases the ductility of concrete in compression.

Reinforced concrete does, however, possess a number of attributes which enable it to be successfully employed in structures resisting seismic loads:

- Properly conceived and detailed concrete structures possess excellent ductility in bending, which can equal that of structural steel.
- Well-confined concrete can possess good ductility under flexure and axial compression, with a lower tendency for buckling failure compared with an equivalent steel structure.
- Properly detailed concrete construction provides a monolithic structure, which contributes to good overall continuity, in itself a good earthquake-resistant feature.
- Shear walls can be an economical means of providing high lateral strength and stiffness, while still retaining significant ductility. (Well-designed 'shear' walls do not fail in shear, despite their name.)
- Internal damping before yielding is likely to be greater than in steel structures (approximately 6% compared with 3%). This is important for serviceability considerations during moderate earthquakes.
- In many countries, concrete is the building material of choice; the technology is familiar and at least some of the materials are locally available and cheap, while the finished structure can possess good sound and thermal insulation properties.

It should be noted that despite the commonly held view, modern steel structures are not immune from collapse or significant damage during seismic events (eg the Automobile Club of Southern California built in 1992 and demolished after the Northridge earthquake), while many well-designed concrete structures have survived major earthquakes undamaged as seen in Kobe and Northridge (see *The California Experience* below). There is a wealth of experimental and theoretical evidence to support this potential for good seismic performance from structural concrete in readily available literature[3,4].

Australian Standards Requirements

AS 1170.4 sets out a number of earthquake design categories. These provide the particular degree of design and detailing consideration required for the level of seismicity expected. As structures will also be designed for wind forces, the relative effects of both must be considered for Australian conditions. Wind strength requirements may be more onerous than those for earthquake loading. It should be noted, however, that even if this is the case, additional earthquake requirements for detailing must still be considered due to:

- the significant degree of crudeness by which earthquake forces are determined;
- the fact that even in columns designed to be elastic (ie with a Structural Response Factor of 1.0), if the earthquake is larger than expected and the concrete cover spalls, the compression reinforcement will buckle, resulting in the loss of core confinement and shear transfer mechanism (see below).

Appendix A forms a normative part of AS 3600. It sets out additional minimum requirements for the design and detailing of reinforced concrete structures under earthquake actions, as defined in AS 1170.4. Many of these provisions are based on Californian practice as codified in ACI 318-89 (amended 1992)[5] and modified for Australian conditions. The Appendix sets out and defines the construction systems and categories in relation to the earthquake design categories considered in AS 1170.4.

The requirements of Appendix A are considered with particular regard to structural systems set out in AS 1170.4, ie:

- Bearing wall systems Structural systems with loadbearing walls providing support for all or most of the vertical loads and shear walls or braced frames providing the horizontal earthquake resistance
- Building frame systems Structural systems in which an essentially complete space frame supports the vertical loads and shear walls or braced frames provide the horizontal earthquake resistance
- Moment resisting frame systems (MRFs) Structural systems in which an essentially complete space frame supports the vertical loads and the total prescribed horizontal earthquake forces by the flexural action of members.

Three types of MRF are defined in the Code:

- Ordinary Moment Resisting Frames (OMRF) Defined as moment resisting frames not more than 50 m in overall height above structural base, complying with the requirements of AS 3600, but not being required to satisfy the additional detailing requirements of Appendix A of AS 3600 or of 1170.4. The author recommends, however, that certain minimum seismic details be adopted to prevent the possibility of brittle failure, with catastrophic consequences, should the design forces be exceeded by an unexpectedly large seismic event (see below).
- Intermediate Moment Resisting Frames (IMRF) Defined as moment resisting frames of ductile construction, complying with the requirements of AS 3600, together with the additional requirements of Appendix A of AS 3600 and AS 1170.4.
- Special Moment Resisting Frames (SMRF) Concrete space frames designed in accordance with AS 3600, in which members and joints are capable of resisting forces by flexure as well as axial forces along the axis of the members with special ductility requirements. Few structures that designers will come across will fall within this category in Australia.

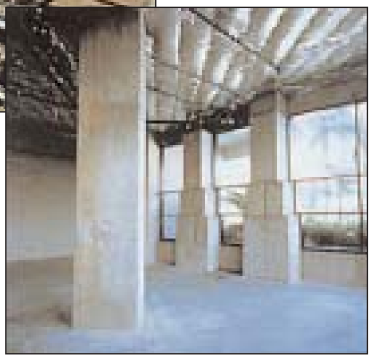
  Detailing requirements for the more onerous conditions of Special Moment Resisting Frames (SMRF) are not covered in Appendix A. Designers requiring guidance for these conditions are referred to ACI 318-89(92).
- Dual systems Structural systems in which an essentially complete space frame provides support for the vertical loads and at least a quarter of the prescribed horizontal forces are resisted by a combination of the moment frame, shear walls or braced frames, in proportion to their relative rigidities.

Once selected, it is imperative that the structural system is designed and detailed to ensure that the system will behave in the way intended.

The Californian Experience 1: 3900 West Alameda Tower, Burbank, CA[17]

The Tower, Burbank, CA.

Below: Post earth-quake photo showing undamaged first floor interior and exterior columns in unfinished tenant space.



Structural excellence at competitive cost The tallest reinforced concrete building in southern California at 32 storeys was completed in 1992. It survived the magnitude 6.8 Northridge earthquake on 17 January 1994 completely undamaged, even though it was less than 16 km from the epicentre.

Most office buildings in southern California have historically been low-rise rather than 'skyscrapers' due to the probability of seismic action. This building shows how properly reinforced concrete construction in tall buildings provides excellent performance in seismic zones. This structure was designed as a ductile frame meeting the requirements for the highest seismic zone rating (Zone IV) in the Uniform Building Code.

The tower is positioned diagonally on a nearly square lower building structure which covers a full city block. Its unique perimeter shape is a geometric abstract that is neither rectangular nor trapezoidal. The building has 102 000 square metres of rentable space with five above- and four below-ground parking levels.

Even so, insitu reinforced concrete framing showed substantial cost savings over alternative framing materials. Post-earthquake inspection showed even brittle, non-structural elements such as masonry and partitions to be damage free.

Photographs and data courtesy Concrete Reinforcing Steel Institute III, USA.

Design 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 break first?

The answer is that it is not possible to predict which will fail. The design structural engineer is in a similar predicament regarding the seismic performance of a building structure. The point of failure could be at any one of a number of locations, affected not just by such considerations as design strength and stiffness, but also by such matters as the provision, for fixing purposes, of a greater number of bars than required, strain hardening of reinforcement during cyclical action, and the overstrength of the reinforcement material above its nominal yield strength.

Paulay[3] uses this analogy to explain the concept of 'Capacity Design' for building structures. Structural failure occurs when the flexural 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 essentially involves the designer picking the positions of failure in the structure and ensuring that these failures are ductile and controlled in manner rather than brittle and catastrophic. Further, by choosing the location of the formation of these ductile failures or 'plastic hinges', the designer ensures that failure does not occur at critical points in the structure. Failure in the non-ductile elements can be prevented if their strength is in excess of the maximum strength (including overstrength) of the ductile elements.

The formation of plastic hinges uses up energy. Once all plastic hinges have formed, the structure will be unable to take any additional seismic load – effectively acting as a 'fuse' to prevent overload and collapse.

Most multi-storey reinforced concrete buildings constructed in Australia will be in either Earthquake Design Categories B or C, ie they will be over four storeys but below 50 m in height, be founded on medium-density clays, sands, gravels or rock, and fall within Structure Classification Type II as defined in AS 1170.4 (eg offices or residential buildings of more than four storeys, hotels and motels, most hospital facilities – excluding those essential for post-disaster recovery).

For Categories B and higher, AS 1170.4 specifies that all parts of a structure shall be interconnected, in both the horizontal and vertical directions. The connections are to be capable of transmitting the calculated horizontal earthquake force (F_p) in order to provide load paths from all parts of the structure. This enables earthquake forces to be carried to the foundation.

For Categories C and higher, diaphragm action is to be considered in the design in the same manner as wall anchorage to provide connection between the walls and the roof and floors. Openings in shear walls and diaphragms require additional reinforcement at the edges and corners to resist local stresses. Footings supported on piles, caissons, or spread footings, located in soils with a maximum ultimate bearing capacity of less than 250 kPa, must be restrained in the horizontal direction to limit differential movement during an earthquake.

The decision as to which design route to take is left largely to the engineer, eg Ordinary Moment Resisting Frames are deemed to require no further detailing consideration from those required in the body of AS 3600 (but see comments on minimum detailing requirements below). However, the Structural Response Factor, R_f, (a function of the ductility provided in the frame and an 'overstrength' factor), may be increased by 50% if the designer opts for an Intermediate Moment Resisting Frame, thus considerably reducing the horizontal earthquake base shear force. The extra design and detailing required for an IMRF may well prove economical by permitting reduced member sizes.

Another point to note is that excess strength provided above that notionally required by the design – for instance due to the provision of additional reinforcement for tying, or extra thickness or depth of section for fire requirements – means less ductility is required for the element. Therefore, less detailing for seismic resistance is required and there may be a resulting increase in buildability. Tilt-up and loadbearing precast panels often fall into this category, for example.

Drift It should be noted that even if a structure or part of a structure is not designed specifically to withstand seismic forces, frames must be designed for the full drift (deflection) of the whole structure (eg in dual system or with combined insitu/precast construction). ACI 318-92 stipulates that a frame must be designed for twice the deflection of the building as a whole. This is regarded as too onerous for expected conditions in Australia. Instead, it is considered sufficient for the designer to allow for the gravity-supporting elements or frames to deflect to the maximum calculated for the most ductile frame or element, ie the earthquake-resisting frame or shear wall, without failure.

Pounding Pounding, or the impact of adjoining buildings due to differing amplitudes of motion, can be a serious problem and one not often considered by designers. This can be especially acute in large cities due to the close proximity of numbers of tall buildings of differing periods of oscillation. Analytical studies have not been validated experimentally and separation to avoid pounding has usually been based on design equations. If the motion experienced is higher than specified, significant damage can occur.

Detailing of Structural Elements for Earthquake Resistance

Detailing of the structure is an integral and important part of the seismic design process. For reinforced concrete, structural detailing centres around arrangement of the reinforcing bars. There must be sufficient transverse steel to suppress brittle shear or crushing failures and to prevent buckling of the main compression steel, once the cover concrete has been lost. The main steel bars must not lose their anchorage into the surrounding concrete during the repeated reversing loading cycles to which they would be subjected during a major earthquake.

Appendix A of AS 3600 sets out detailing criteria for general, regular and irregular structures, as defined in AS 1170.4. Regular structures in Design Categories A and B do not need to be specifically designed or detailed for resistance to earthquake loads. For irregular structures in Design Category B and structures in Design Categories C to E, the design action effects determined in AS 1170.4 are dependent upon both the type of structural system adopted and the type of member being considered. The relevant level of ductility is to be met by following the detailing requirements for the particular structural system concerned.

It should perhaps be restated here that design and detailing are inseparable. Proper detailing is required to ensure that the structure will respond under seismic loading in the manner for which it has been designed.

Shear Walls or Braced Frames

As AS 1170.4 assigns a low structural response factor (R_f) to reinforced or prestressed concrete shear walls or braced frames in a bearing wall system, these elements attract higher earthquake design forces. They are therefore required to be comparatively heavily reinforced and often will have a reasonable excess of strength above that notionally required. Appendix A allows elements in these systems to be designed and detailed in accordance with the main body of the code without further consideration. It must be noted by the designer, however, that the use of any R_f of greater than 1.0 results in a design earthquake force of less than the anticipated actual loading. Reinforcement will then yield once the *design* earthquake force is reached and plastic hinges form. Detailing must be provided to reflect this.

Building Frame Systems

As building frame systems are generally more ductile than bearing wall systems, they are assigned a correspondingly higher R_f value in AS 1170.4. The earthquake design forces are therefore lower. This may in turn result in less longitudinal and shear reinforcement. To maintain the required level of ductility, however, additional detailing is necessary.

The ductility provision requirements are as follows:

- The reinforcement ratio, $P_w \geq 0.0025$ both horizontally and vertically (ie an increase from 0.0015 in the vertical direction over cl 11.6.1).
- The reinforcement is to be divided between the two faces, if:

  $t_w > 200$ mm; or $\phi V_u > (A_g.f'_c)/6$

  where

  t_w = thickness of the wall
  ϕV_u = design shear strength
  A_g = gross cross-sectional area
  f'_c = characteristic 28-day compressive cylinder strength of concrete.

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

- Boundary elements must be provided at discontinuous edges of shear walls and around openings where:
  - vertical reinforcement is not restrained; and
  - the extreme fibre compressive stress in the wall exceeds 0.15 f'_c.

    Note: This stress may not be the actual stress developed, but is the 'trigger value' for determining when a boundary element is required.

Restraint of the longitudinal reinforcement in boundary elements is to comply with Clause 10.7.3 or, if the extreme fibre compressive strength exceeds 0.2 f'_c, with the requirements for Reinforced Braced Frames.

It should be noted that the above requirements do not necessarily result in an increase in wall thickness for a boundary element, only that the areas concerned are designed and detailed to resist specified axial forces **Figure 2**.

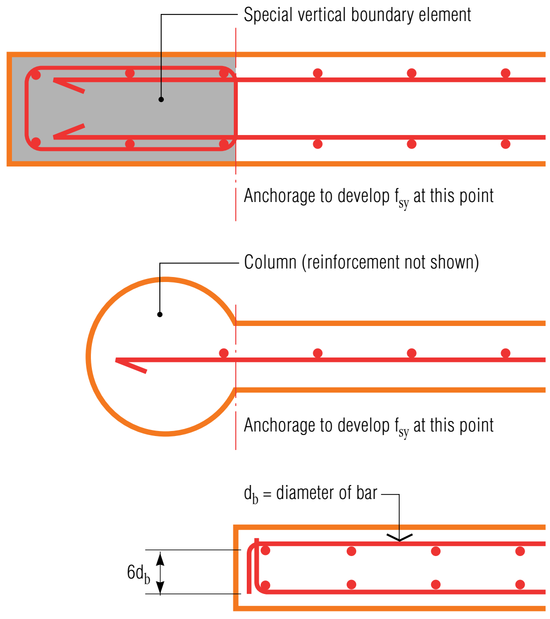


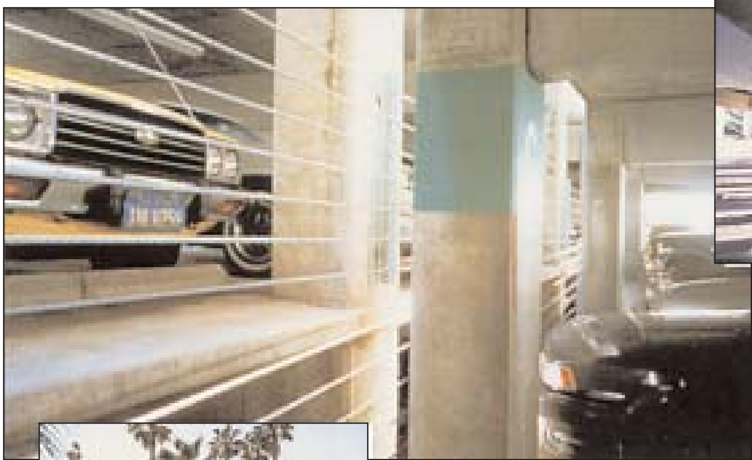
Figure 2

Typical boundary element details (from AS 3600[2])

The Californian Experience 2: Parking Structures

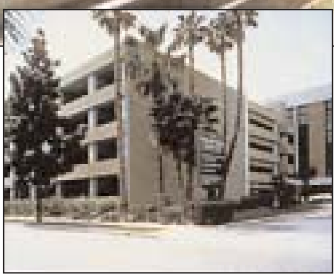
Parking structure on California State University Northridge Campus A testament to ductile reinforced concrete? Collapse occurred due to the failure of the internal precast concrete frame. The exterior elements – the primary seismic resisting system – were constructed as an insitu ductile frame. A telling testament to the ability of properly detailed reinforced concrete to perform in a ductile fashion.

(Photo courtesy EERC/University of California)



Different structural systems must be compatible Structural collapse occurred due to a lack of drift compatibility between the primary seismic resisting system and the precast gravity resisting structure. Seating of precast units must allow for frame growth under lateral deflection (see Figure 11).

(Photo courtesy CSIRO)



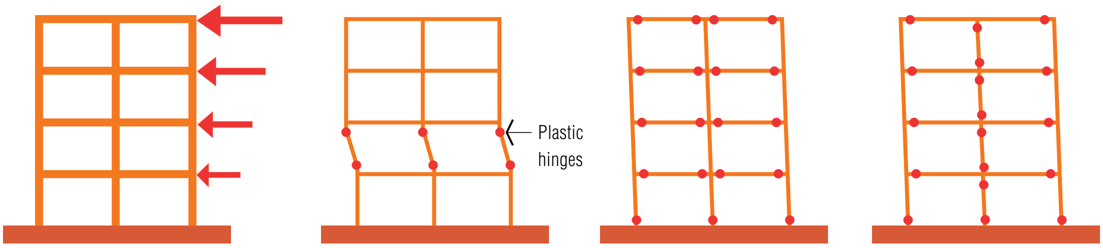
Top and above: Exteriors of the Northridge Medical Centre Parking Garage.

Interior: Post earthquake photo showing undamaged interior columns, beam and slab.

(Photos courtesy CRSI)

The Northridge Medical Centre Parking Garage was completed in 1989. Located at the epicentre of the Northridge earthquake it came through without damage. It is one of over 40 such parking structures built by one contractor, Sy Art, in the immediate vicinity of the earthquake – all of which were undamaged.

This record is all the more remarkable as a number of other parking structures fared particularly badly. There were a number of causes of these failures. Predominantly these were weaknesses in design philosophy. Such issues as a lack or irregular siting of stiffening elements, large wall openings and inadequate attention to detailing, especially where changes of construction type occurred.



Frame

a) Column sidesway mechanism
- soft storey
(strong beam/weak column)
- non-preferred arrangement

b) Beam sidesway mechanism
- plastic hinges in beams only
(strong column/weak beam)
- preferred arrangement

c) Mixed sidesway mechanism
- interior columns form plastic hinges
- acceptable with adequate design and detailing for ductility

Figure 3

Three possible mechanisms of post-elastic deformation of moment-resisting frames during severe seismic loading (after Goldsworthy[9])

Reinforced Braced Frames

Bracing members of braced frames are to be designed as struts or ties, as they will be subject to alternating compression and tension, and connections between members are to have greater strength than each connected member.

In terms of detailing, it is important to provide adequate lateral restraint along the whole length of the longitudinal reinforcement when it is subject to compression in the form of:

- helices: the volume of steel divided by the volume of concrete, per unit length of member must be greater than $0.12(f'_c/f_{sy.f})$; or
- closed ties:

  $A_{sv} \geq 0.30\, sy_1, (A_g/A_c\text{-}1)\,(f'_c/f_{sy.f})$ (unless $\phi N_{uo} > N^*$)
  or
  $\geq 0.09\, sy_1, (f'_c/f_{sy.f})$; whichever is greater.

  where

  s = centre to centre spacing of the ties

  y_1 = the larger core dimension

  A_g = the gross cross-sectional area of the column

  A_c = the cross-sectional area of the core measured over the outside of the ties

  f'_c = the characteristic compressive cylinder strength of concrete at 28 days

  $f_{sy.f}$ = the yield strength of the ties

  ϕ = a strength reduction factor

  N_{uo} = the ultimate strength in compression of an axially loaded cross-section without eccentricity

  N^* = the axial compressive or tensile force on a cross-section.

Moment Resisting Frame Systems

As discussed above, there are three types of moment resisting frames:

- **Ordinary Moment Resisting Frames** These require no specific detailing for seismic resistance. Standard detailing as set out in the body of AS 3600 is considered to provide structural adequacy to reinforced concrete structures when coupled with the higher earthquake design forces consequent from the use of lower R_f values (ie reduce the 'ductility demand', or likely joint rotation, on the frame and assume they are essentially elastic).

  The designer should note that for OMRFs, normal detailing to AS 3600 will result in only limited frame ductility, primarily as a result of poor joint performance. Joint failure will result in collapse (Kobe, Northridge, Mexico City). In order to achieve the required R_f value of 4, the designer needs to ensure that an excess value of this magnitude (or higher) is available (see Reference 9), or that detailing is provided such that plastic hinges may form. (See *Intermediate Moment Resisting Frames*.)

  As noted in Design Methodology above, it is important to ensure, however, that the non-seismically-designed frames are sufficiently ductile to cater for forces they will attract if the earthquake is bigger than that assumed in the model. The designer must detail with care to ensure that plastic hinges, if any, form at the locations specified **Figure 3**. It is important to remember that AS 3600, Appendix A, does not specifically direct the designer to provide a weak beam/strong column mechanism (see below) so any of the three indicated modes could occur during a seismic event of sufficient magnitude to cause yielding of reinforcement.

- **Special Moment Resisting Frames** These will rarely be required in Australia as this will be economically viable only for Design Category E structures. As such, AS 3600 refers readers to ACI 318 and references (3, 6 and 7) for detailing concrete structures for more-onerous seismic conditions.

- **Intermediate Moment Resisting Frames** Attention will be concentrated on the detailing requirements for these systems as they will more commonly be seen, especially in dual systems where for instance shear walls are provided only in one direction. Further, some of the provisions must be considered as good detailing practice in OMRF systems as well.

Detailing Requirements for Intermediate Moment Resisting Frames

Beams Under the effects of earthquake action, flexural members are subjected to a number of reversals of bending moment. To ensure adequate ductility potential in IMRFs, beams are always doubly, and continuously, reinforced **Figure** 4.

If yield occurs, the Young's Modulus of the reinforcement will not remain within the elastic part of the stress-strain curve, and that 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.) The stable hysteretic 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, for instance.

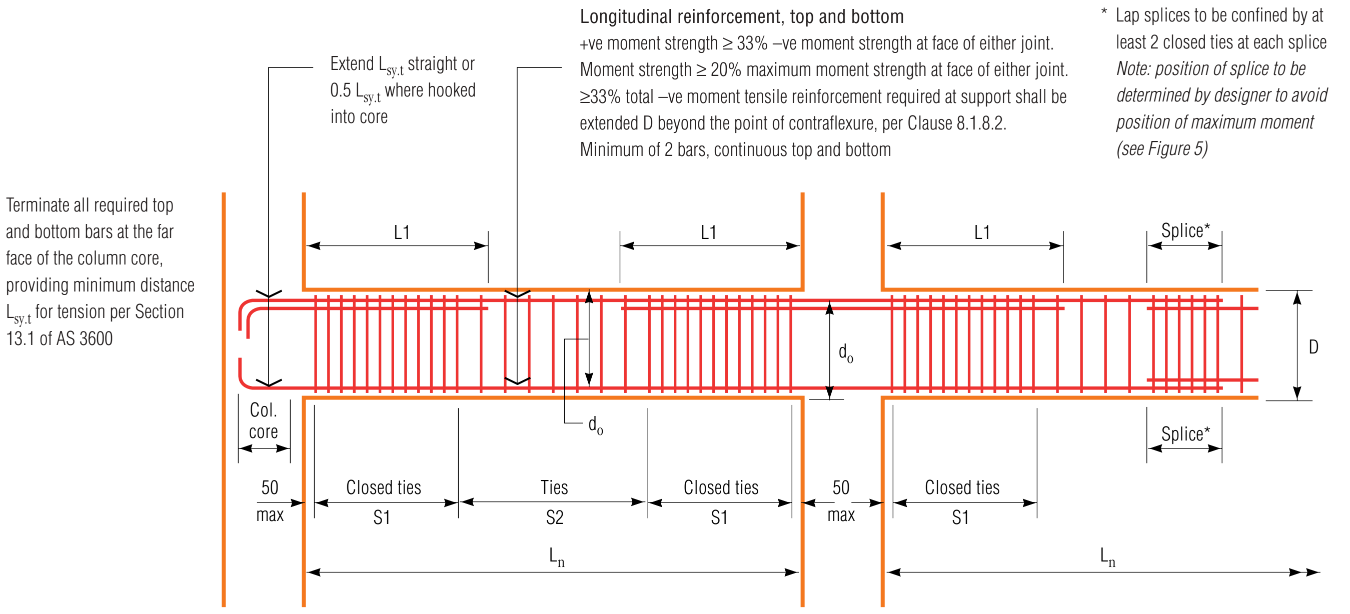
The effect of reversing moments is generally concentrated at the junctions between beam and column. The Appendix therefore, stipulates that in a span:

- the positive moment strength at a support face is to be not less than one-third 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 one-fifth of the maximum moment strength provided at the face of the support.

All longitudinal reinforcement must be 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 and is fully anchored.

Lapped splices in longitudinal reinforcement, located in a region of tension or reversing stress, are to be confined by a minimum of two closed ties at each splice to inhibit the possibility of non-ductile failure at this point. The position of maximum moment under seismic load will be dependent upon the magnitude of the earthquake **Figure** 5. The position of the splice should therefore be located at a position of known moment, perhaps in the middle third of the span, unless the designer is confident that the splice is sufficiently confined to safely locate it elsewhere in the span.



Engineer must provide dimensions L1, S1, S2, tie and closed tie spacing, anchorage length, cut-off points of discontinuous bars and $L_{sy.t}$

Maximum tie spacings
In length S1, spacing for closed ties ≤ 0.25 d_o; $8d_b$; $24d_f$; or 300 mm, whichever least.
In length S2, spacing of tie ≤ d/2 or 300 mm, whichever least.

L_n ≥ 4D (Clause 12.1.1.1)
L1 = distance required by design for moment plus anchorage length (= $L_{sy.t}$ + D)
d_b = diameter of smallest longitudinal bar enclosed by tie
d_f = diameter of bar forming tie
d_o = design depth for –M and +M
S1 ≥ 2D

Figure 4

Typical beam restraint details for IMRF structures

Shear type failures tend to be brittle. Also, as mentioned above, maintaining a stable hysteretic response of plastic hinge regions requires that the compression bars be prevented from buckling. It must therefore be assumed that major spalling of concrete cover will occur and that the compression bars must rely solely upon transverse support provided by the ties. Limitations on maximum tie spacing are required to ensure that the effective buckling length of the compression bars is not excessive and that concrete within the stirrup ties has reasonable confinement. Furthermore, due to the possible occurrence of the Bauschinger effect and the reduced tangent modulus of elasticity of the steel, a smaller effective length must be considered for bars subject to flexural compression, rather than compression alone. The appendix specifies a minimum area of shear reinforcement:

$A_{sv} \geq 0.5\ b_w s/f_{sy.f}$ (ie 50% greater than stipulated in the body of the Code) with closed ties provided over a minimum distance of 2D from the face of the support. The first placed 50 mm from the support face, and the remainder spaced at 0.25 d_o, 8d_b, 24 d_f or 300 mm, whichever is least,

where:

b_w = width of web.

s = centre to centre spacing of ties.

$f_{sy.f}$ = yield strength of ties.

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 that 0.8D.

d_b = the diameter of the smallest longitudinal bar enclosed by the tie, and

d_f = the diameter of the bar forming the tie.

Since tension in vertical tie legs acts simultaneously to restrict longitudinal bar buckling and to transfer shear force across diagonal cracks, it is considered that the tie areas are sufficient to satisfy both the requirements for bar buckling and those for shear resistance **Figure** 4.

(Note: these requirements do not preclude efficient fabrication techniques such as loose bar detailing as recommended in the CIA detailing manual [8].)

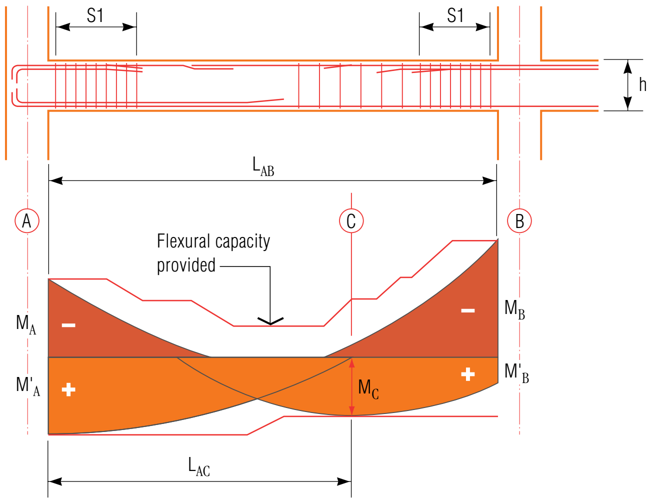


Figure 5

Localities of plastic hinges when stirrup-ties are required (after NZS 3101:Part 2:1995) Note: plastic hinges will form when the flexural capacity envelope and the actual moment coincide

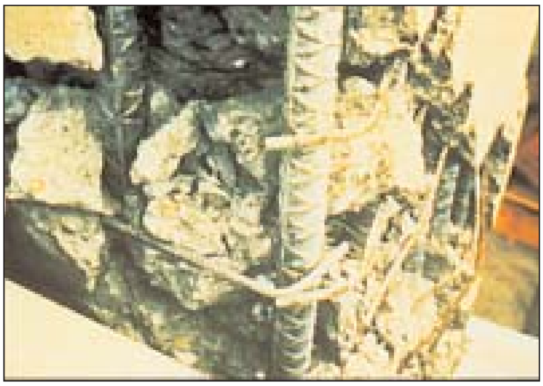
Columns As discussed in Moment Resisting Frame Systems above, it is desirable to ensure that any plastic hinges that may form should do so in the beam elements rather than the columns by ensuring that the flexural capacity of the column is higher than that of the beam by a significant margin to allow for any 'overstrength' due to design or materials (see Design Methodology above.) This is known as the 'weak beam/strong column' philosophy. Although it may not always be possible to achieve this, especially with such forms of construction as band beams (see below), care should be taken that catastrophic collapse, especially due to brittle shear failure in the column will not occur.

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: this may occur even where the design is based upon weak beam/strong column philosophy, such as at the base of all columns **Figure 3b** and **c**.)

Confinement Close-spaced transverse reinforcement acting in conjunction with longitudinal reinforcement restrains the lateral expansion of the concrete. This enables the concrete to withstand higher levels of compression. Circular or helical ties, due to their shape, are placed in hoop tension by the expanding concrete and provide confinement to the enclosed concrete **Figure** 6a. Rectangular ties 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 cross-ties or interconnected

Loma Prieta, USA, 1989. Shear failure of column and lack of confinement of core concrete. Note failure of reinforcement to confine core concrete due to inadequate detailing and wide spacing of transverse hoops. AS 3600 specifies 135° hooks rather than 90° as shown (and previously stipulated in AS 1480).

(Photos courtesy EERI)



closed ties. 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 ties will also significantly aid confinement **Figure** 6b, c, d and e.

The confinement of concrete is addressed in Appendix A by the provision of closed ties, where required, over a distance of either:

- the maximum dimension of the column cross-section, or
- one-sixth of the least clear distance between consecutive flexural members framing into it. Further, the spacing of the closed ties is to be the least of $0.25d_o$, $8d_b$, $24d_f$ or 300 mm with the first tie located at 50 mm from the support face. (Note: d_o in this case is taken from the larger column dimension.) The overall cross-sectional area of the ties must obviously be sufficient to satisfy the shear requirements of the column **Figure** 7.

Lapped splices It is inevitable that splices will occur in the column reinforcement of multi-storey buildings. It is important therefore to ensure that these are detailed and located such that failure will not occur under earthquake action. Splicing is usually achieved by the use of overlapping parallel bars. In this method, force transmission occurs due to the bond between the bars and the surrounding concrete, as well as due to the response of concrete between adjacent bars.

Under severe cyclical loading, column splices tend to progressively 'unzip'. Further, where large steel forces are to be transmitted by bond, splitting of the concrete can occur. To prevent these occurrences, ties are required to provide a

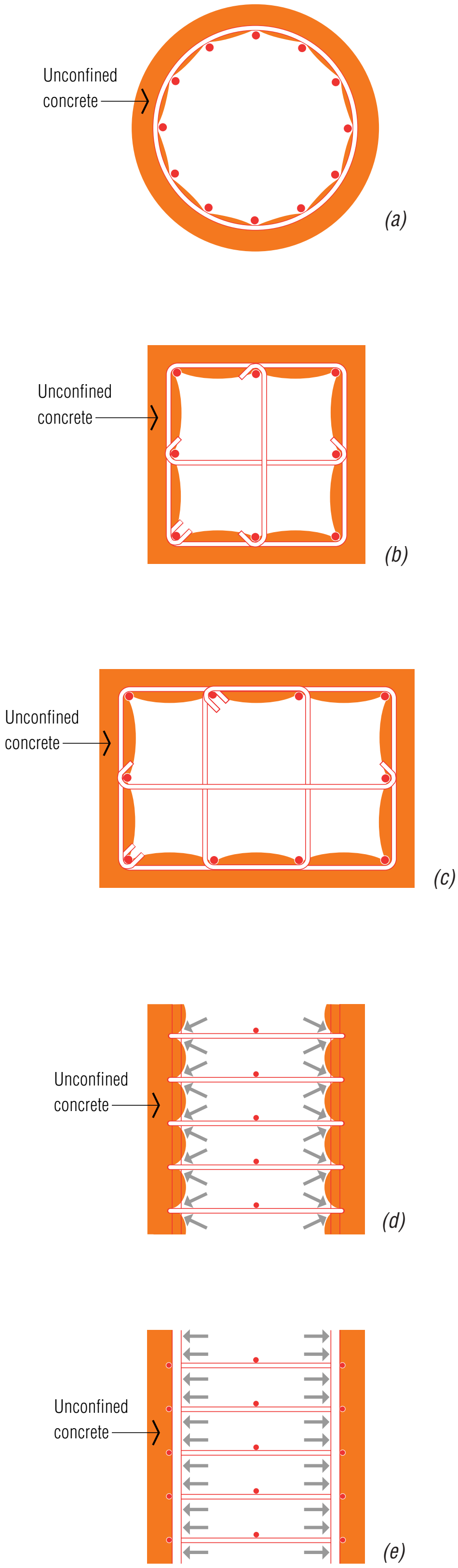
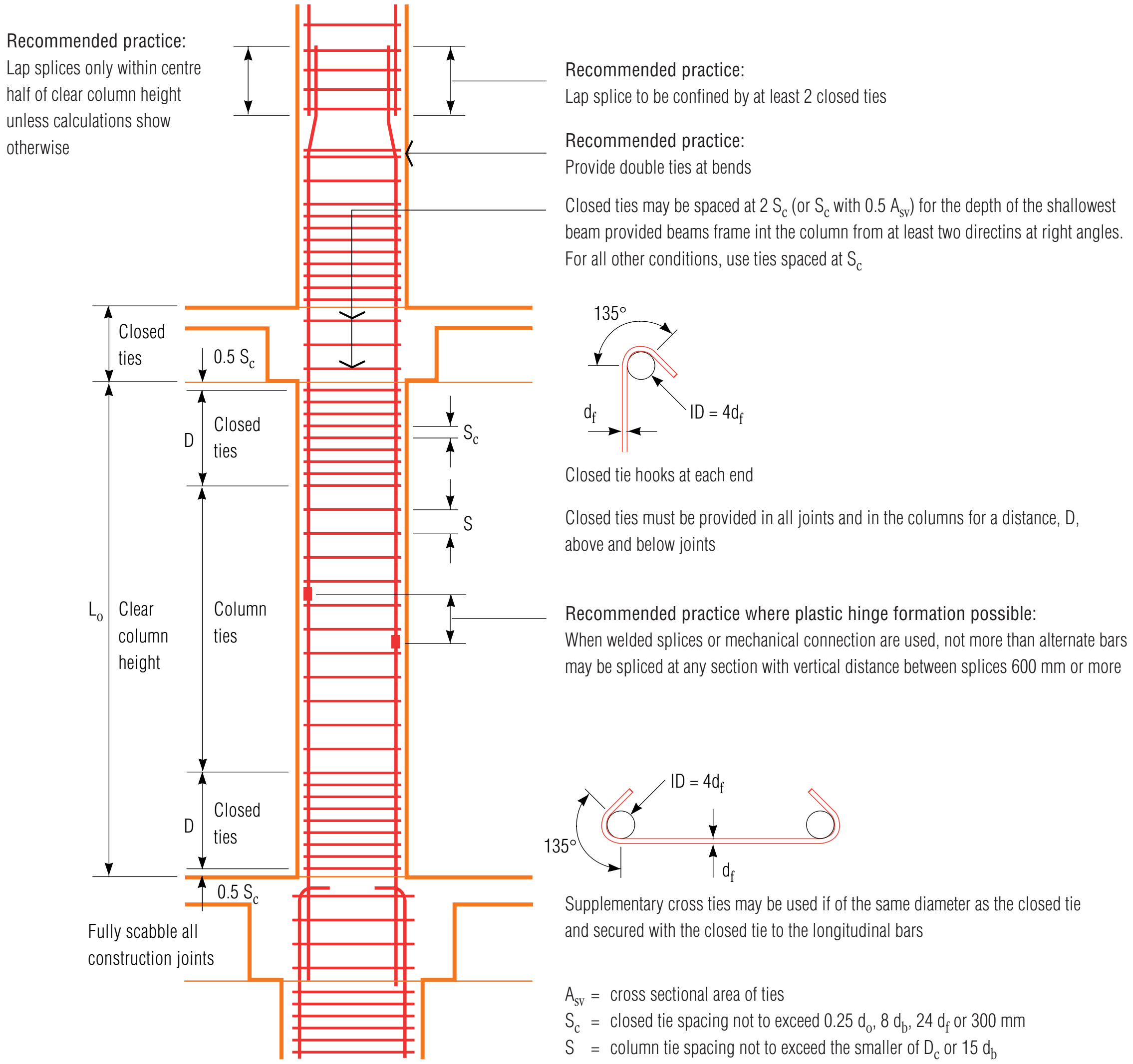


Figure 6

Confinement of column sections by transverse and longitudinal reinforcement (from Paulay and Priestley[3])



A_{sv} = cross sectional area of ties
S_c = closed tie spacing not to exceed 0.25 d_o, 8 d_b, 24 d_f or 300 mm
S = column tie spacing not to exceed the smaller of D_c or 15 d_b
D_c = smaller dimension of column cross-section
D = largest column dimension, but not less than one-sixth clear height
d_o = effective depth of member $\geq$ 0.8 D
d_b = diameter of smallest longitudinal bar enclosed by the tie
d_f = diameter of tie bar

Figure 7

Typical column details for IMRF structures

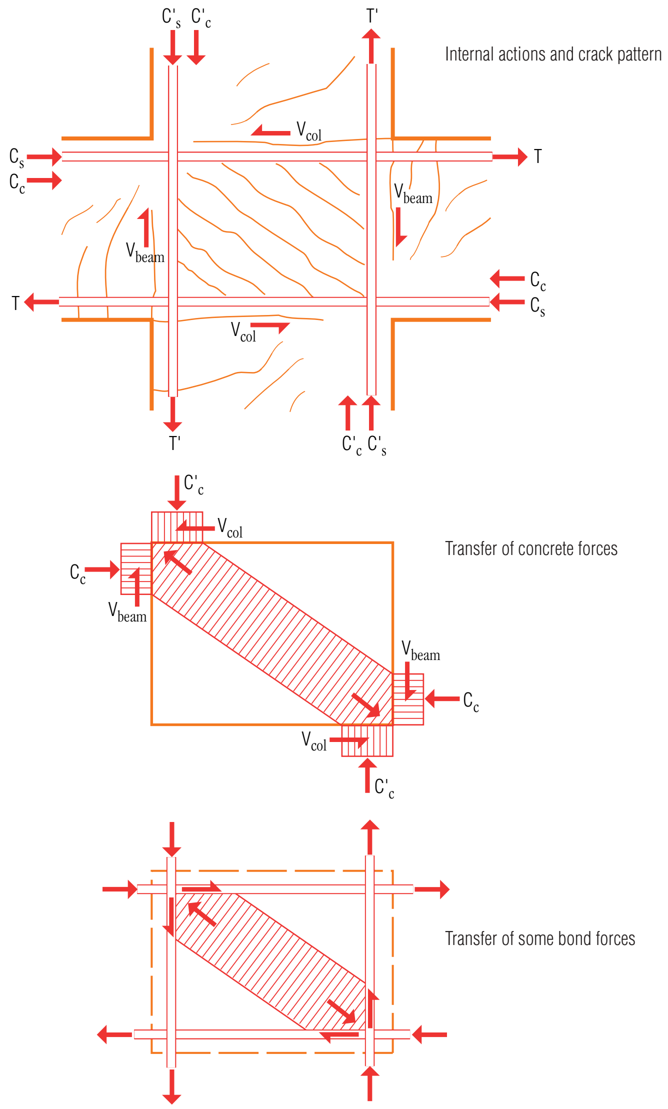
'clamping force' to the longitudinal reinforcement against the core concrete. In circular columns the clamping force is provided by helical or circular ties. This form of reinforcement has been shown to be very efficient at resisting the radial cracks that can develop. Further, these ties can restrain an unlimited number of splices.

Unless the capacity has been checked by design, it is recommended that splices should not be placed in potential plastic hinge regions. Whilst transverse ties may ensure strength development of the splice under cyclical loading at up to but still below 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 bars (see Reference 3). For example, a plastic hinge would normally be expected to occur at the base of first-storey columns. (Note: this is true for *all* frame types.) Consideration should therefore 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.

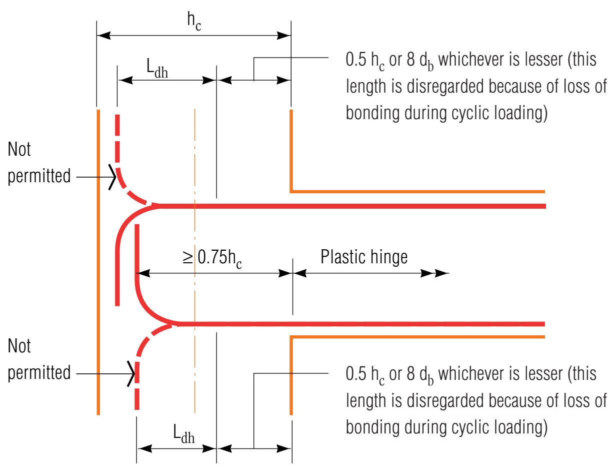
New Zealand practice allows that columns that have greater than 1.25–1.4 times the flexural strength of the adjoining beams are unlikely to yield and form plastic hinges – providing the column shear strength is similarly higher, ie matching the column flexural capacity. If the formation of plastic hinges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

Splicing by welding or the use of mechanical couplers (eg Alpha or Lenton) is often done where bar congestion may prove problematic. It is recommended that under no circumstances should these be situated in a potential plastic hinge region, in order to help ensure a strong column/weak beam failure.

Site welding of bar splices requires special consideration and care during execution. It is recommended that lap welding should be avoided. Butt welding is acceptable, provided it is carried out using a proper procedure but, again, it is recommended that welded splices are never used in a potential plastic hinge region **Figure** 7.



(a) Shear transfer of concrete compression forces and some bond forces from longitudinal bars by diagonal compression strut mechanism (from Goldsworthy[9])



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

Figure 8

Shear transfer at beam/column connections and suggested reinforcement anchorage at external column position

Beam/Column Joints Under seismic loading, the reversing moments induced above and below the column joint, 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. However low the calculated shear force in a joint resisting earthquake-induced forces, transverse reinforcement must be provided through the joint to prevent the occurrence of brittle joint shear failure, rather than obtaining the desired flexural beam hinges (see References 3, 5 and 6). This transverse reinforcement is provided by continuing the closed ties required for columns adjacent to the joint.

The area required A_{sv} is to be a minimum of:

$0.3\ sy_1,\ (A_g/A_c - 1)\ (f'_c/f_{sy.f})$, (unless $\phi N_{uo} > N^*$);

or

$0.09\ sy_1,\ (f'_c/f_{sy.f})$ whichever is the greater.

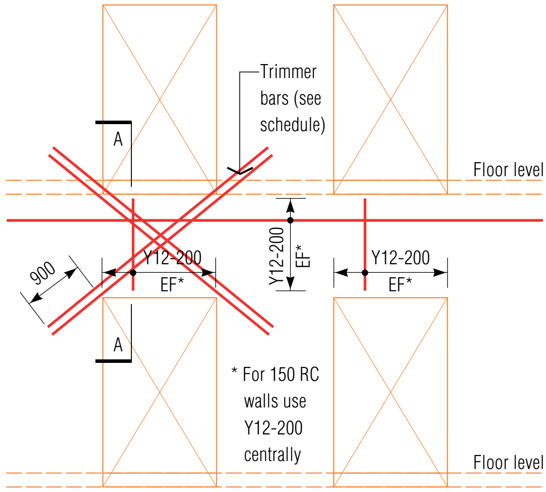
where

- s = centre to centre spacing of the ties
- y_1 = the larger core dimension
- A_g = the gross cross-sectional area of the column
- A_c = the cross-sectional area of the core measured over the outside of the ties
- f'_c = the characteristic compressive cylinder strength of concrete at 28 days
- $f_{sy.f}$ = the yield strength of the ties
- ϕ = a strength reduction factor
- N_{uo} = the ultimate strength in compression of an axially loaded cross-section without eccentricity
- N^* = the axial compressive or tensile force on a cross-section.

The area of reinforcement required may, however, be reduced by half where equal resistance to joint rotation is provided in at least two directions at right angles, but only over the depth of the shallowest of the framing members **Figure** 7.

Figure 8 shows the effects of shear transfer of concrete compression forces and some bond forces which, especially at external beam/column connections, require special consideration with regard to reinforcement anchorage. A considerable length of the top bars is ignored when calculating the development length because of expected bond deterioration under cyclic load reversal. It should also be noted that the bottom beam bars are bent upwards at the end **Figure 8b**. If they are not, this will result in poor behaviour of the joint in the direction of loading. In addition, proper anchorage of the bottom beam bar is necessary in order to transfer shear through the joint via a strut mechanism. However, tests have shown that U-bars are not as effective as top and bottom bars anchored separately (Reference NZS 3101:1995).

Sun Alliance Tower, Chatswood
Seismic detailing practice



Typical doorway detail (except for 150 thick wall as noted)

Trimming Bars Size Schedule

Marking	Bar size
A	4Y28
B	4Y20
C	4Y28
D	2Y36
E	3Y24
F	2Y24

This high-profile building, destined to be Sun Alliance's head office was specified from the outset to provide CBD-quality construction with first-class services and finishes.

The 15-level tower will occupy a 3200-m^2 site directly across the road from the Chatswood railway station and bus interchange. Typical floors have 1138 m^2 gross floor area providing a total net lettable area of 15 000 m^2, including ground- and first-floor retail space and professional suites.

The facade of the building will be a combination of pre-finished metal cladding and glazed elements. The ground-level retail-space finishes will be natural stone, polished precast walls and masonry units. There will be an extensive landscaped area incorporating seating and a pedestrian bridge to the bus/rail interchange.

Three basement levels of carparking will provide spaces for 235 cars with feature lobbies on each level to provide a pleasant exit from carparking areas.

This is the first multi-storey building with a reinforced concrete frame to be designed to meet the requirements of AS 1170.4 and AS 3600–1994 which necessitated significant detailing for seismic performance. ■

The Engineer's Perspective

Paul Sancandi
Director, Meinhardt (NSW) Pty Ltd

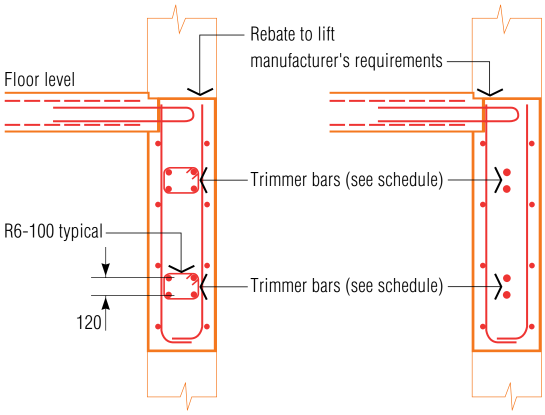
The lift shafts and stairwells were combined to form a Y-shaped core structure which has been constructed in reinforced concrete using a climb-form system. Because of the stiffness of the core, it was designed to provide all the lateral resistance to bulk earthquake and wind loads. The analysis showed that earthquake loading was the critical case. Wall thicknesses and reinforcement requirements were all calculated in accordance with AS 3600 and AS 1170.4.

An important consideration with respect to the seismic design was to ensure proper anchorage and detailing of the reinforcement. The main concern was adequate reinforcement detailing with respect to the coupling beams, over the heads of the lift shafts, connecting the main wall elements. It was recommended[16] that whenever the normal shear stress exceeds $V_i = 0.1 (l_n/h)\sqrt{f'_c}$ MPa, diagonal reinforcement, to resist both shear and bending moments, should be used (where l_n and h are the clear span and overall depth of the beam respectively).

Specific requirements for the detailing of the reinforcement for coupling beams included:

- the provision of transverse ties around the cage of the diagonal bars, spaced no further apart than 100 mm;
- increasing the development length specified for individual bars by 50% and extending into the adjacent walls;
- basketing a nominal grid of reinforcement in the remainder of the beam.

It should be noted that a minimum wall/beam thickness of 350 mm was preferred in order to accommodate the reinforcement properly. For thinner



Section A-A
Typical for type A, B and C bars

Section A-A
Typical for type D bars

walls a single line of reinforcement may be acceptable as long as the stresses in the compression 'strut' are within code limits.

This detail ensured a ductile coupling beam. Other detailing requirements related to ensuring adequate anchorage of horizontal reinforcement at the ends of walls (boundary elements). This was mainly achieved by providing cogs at the end of the bars or by the use of closed ties at the end.

With respect to the connection of the slabs to the corewalls, conventional blockouts with 'pull-out' reinforcement bars were utilised. This allowed the cores to progress ahead of the floors. The amount of reinforced required was checked against the Code requirements. ■

Sun Alliance Tower

Location
461–471 Victoria Avenue Chatswood, NSW, Australia

Client
Vickbrow Pty Ltd

Structural Engineer
Meinhardt (NSW) Pty Ltd

Contractor
Concrete Constructions Group Pty Ltd

Date Commenced
February 1994

Estimated Completion
July 1995

The Contractor's Perspective

Mark Zvirblis
Public Relations Manager
Concrete Constructions Group Pty Ltd

Concrete Construction's involvement with the Sun Alliance Project began in mid-1991, commencing with assistance in cost planning and a buildability analysis.

This developed into a contract fully documented by the client, giving the client a guaranteed maximum price and time. This will be Sun Alliance's head office and thus their brief was to provide a CBD-quality building with first-class services and finishes.

Construction commenced in February 1994 and is scheduled for completion in mid-1995. The building was 'topped-off' in February 1995, within two days of its commencement 12 months earlier.

- Rock Anchors with a 350-tonne capacity were installed through the core base slab. There were ten anchors with lengths varying from 8 to 10 metres. They were drilled after the jump form had been taken up approximately five levels in order to clear the drilling rig, as core construction was moved forward to save time on the construction programme.
- Door heads and sills received extra reinforcement to cater for seismic loading, placed diagonally between the door heads and sills on each floor. Limited access and slender walls required some modifications to the original design, with the use of shorter lengths of bar and more-frequent splices. This was the only area where seismic considerations had a major influence on reinforcement quantities and placement.
- Core walls required alpha splices to bars to minimise congestion due to limited wall thickness and required reinforcement quantities.

Final typical reinforcement ratios on the project for selected elements were:

Suspended prestressed tower slabs
30 kg/m^3

Tower beams
45 kg/m^3

Columns
225–260 kg/m^3

Core walls
280 kg/m^3

Typical sizes of structural elements are as follows:

Tower slabs
160 mm thick

Tower edge beams
400 mm deep x 900 mm wide

Tower main beams
295/320 mm deep x 2400 mm wide

Columns (B3 to underside ground)
1500 x 450 mm

Columns (ground to underside Level 1)
800 mm diameter

Columns (Level 1 to 16)
620 x 620 mm

Core walls (B3 to underside Level 1)
350 mm thick

Core walls (Level 1 to underside Level 5)
230 mm thick

Core walls (Level 5 to Level 16)
150 mm thick

■

Floor Slabs It has generally been found that insitu floor slabs spanning in either one or both directions and acting monolithically with the supporting beams are more than capable of acting as a diaphragm unless the number of large openings is excessive. The detailing requirements for slab reinforcement for moment resisting frame systems in Appendix A are essentially the same as for beams (eg provisions of reinforcement, continuity, anchorage, lapping).

Flat-slab construction has additional requirements due to the need to ensure ductility and continuity conditions are met at column and middle strips along the line of support.

Appendix A 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 beyond the face of the column or capital.

  This proportion is the greater of 0.5

  or

  $1 / 1 + {}^{2}/_{3} \sqrt{[(b_l + d_o)/(b_t + d_o)]}$

  where

  b_l = the size of rectangular (or equivalent) 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) column, capital or bracket, measured transversely to the direction of the span for which moments are being determined.
- At least 25% of the top reinforcement at the support in the column strip is to be 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 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 9**.

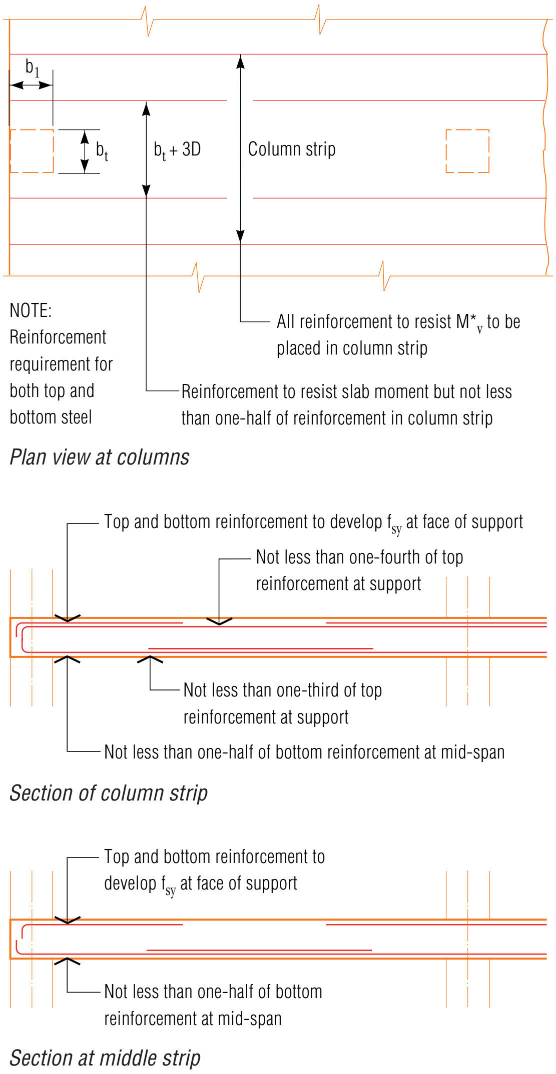


Figure 9

Reinforcement detailing for flat slabs (from AS 3600[2])

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. Booth[4] reports that failure occurs in the slab close to the column rather than in the joint zone. 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. This failure can be brittle in character, leading to the possibility of progressive collapse.

To prevent this, secondary reinforcement should be placed in the bottom of the slab at the column/slab intersection to resist the gravity loads in a tensile membrane action **Figure 10**.

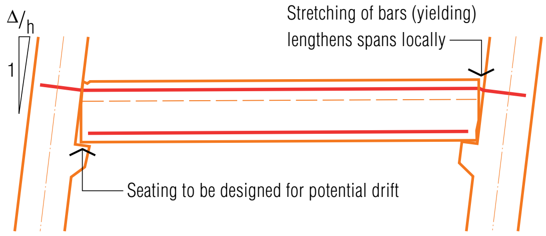


Figure 11

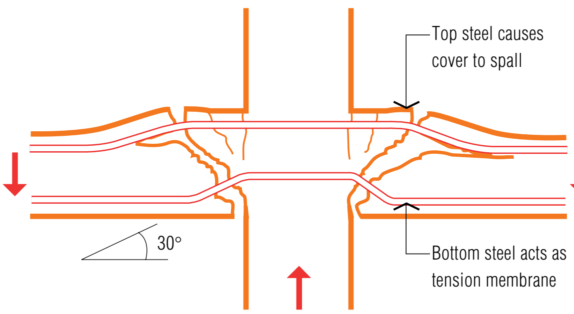
Drift effects on precast concrete flooring

Band Beams Shallow beams with large principal reinforcement ratios and correspondingly large joint shears will often give rise to problems in placing all the required joint ties. Irvine and Hutchinson[6] report that in fully ductile frames, the joint ties would usually be placed with one tie 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_s/bd) be restricted to 0.02 or less, so as to reduce the problems of placing the beam/column joint ties. This needs to be considered especially at band beam/external column joint connections to ensure sufficient ductility in the column to prevent a plastic hinge and potential collapse mechanism forming in the column. If this proves impracticable, the mechanism shown in **Figure 3c** may be considered, provided rigorous analysis and careful detailing are employed.

Precast Concrete

General Whilst loadbearing precast structures have not had the same degree of field and laboratory testing as cast-insitu structures, experience in both the USA[10,11,12] and New Zealand[3,7,10 and 13] has shown that precast elements can be used successfully if sufficient attention is paid to connection details. This has been the major source of concern in certain overseas locations (eg the Spitak earthquake, Armenia, 1988 where inadequate detailing led to catastrophic collapses).

In New Zealand especially, innovative ideas from both the academic and practising fraternities has resulted in the development of capacity design procedures, and precast elements are now used for both low- and high-rise construction.



$A_{sb} \geq 2V/øf_y$

A_{sb} = total area of bottom steel in slab passing through the column perimeter

Figure 10

Tensile membrane steel at column-slab intersection (from Booth[4])

Seismic Design Concepts The main criteria to consider obviously are:

- Buildability – the designing and detailing of structural elements such that they may be produced economically and erected easily and quickly whilst providing structural adequacy. Consideration of tolerances is especially important in detailing, together with the maintenance of the seismic performance of the structure.
- Ductility – the level of ductility assumed in the design must be achieved in practice both by the precast elements and their connections.
- Continuity – continuity is used in precast flooring systems to provide a number of benefits including fire resistance and deflection control. It may also be used to resist diaphragm forces. Flexural continuity may be achieved by placing reinforcement in the topping concrete at the ends of the precast flooring unit. However, the engineer needs to exercise care to ensure that the topping thickness is adequate to enable full anchorage of the reinforcement to be obtained.
- 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 a premature failure of a loadbearing element is well recognised. This need is especially important in precast construction where numbers of discreet elements are connected together to form the whole. The designer must therefore consider detailing as an integral part of the design in order to achieve a structure that will behave in a monolithic fashion during a seismic event, eg P-delta effects due to plastic hinge formation, or inter-storey drift require consideration of minimum seating dimensions to prevent loss of support of precast floors or stair sections **Figure 11**.

Slabs – Toppings for Diaphragm Action

Where precast flooring elements are used, an adequately reinforced insitu topping of at least 50 mm in thickness should be placed in order to provide suitable diaphragm action. It is essential to ensure that this topping is adequately bonded to the precast elements, if composite action is required, by the use of mechanical connectors or chemical (eg epoxy) bonding 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. This was graphically illustrated by the extremely poor performance of precast framed buildings in the Armenian earthquake of 1988. A major factor in these failures was the lack of positive connection between precast floor-slab elements, and also between these and their supporting elements.

Concern as to the adequacy of untopped precast floors using mechanical connections for diaphragms in seismic conditions have been expressed by several American engineers. Clough[12] has stated that:

Untopped diaphragms in which inter-element connection is made by grouting or mechanical connectors, have relatively low in-plane shear strength and ductility and are most suitable when seismic equilibrium and compatibility forces are small. In zones of high seismic intensity, or with structural configurations which impose large in-plane compatibility forces under lateral load, diaphragms joined by cast-in-place reinforced concrete, either as pour strips or as a topping, usually are more satisfactory.

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 that connections are detailed such that they adequately transfer the anticipated loads.

The strut and tie method may be used for the design of these details.

Tilt-Up

Tilt-up or loadbearing precast buildings have been a popular form of construction in both Australia and the United States for more than 20 years, especially for low-rise industrial and commercial buildings, and since the mid-1980s also for medium-density residential in Australia.

In the Northridge earthquake, it was reported that over 400 of the approximately 1200 tilt-up buildings were significantly damaged, mainly with partial roof collapses but also in some cases the collapse of perimeter wall panels[14].

However, some significant differences exist in the construction of tilt-up buildings between the USA and Australia/New Zealand. Primarily the differences centre around the diaphragms. American practice commonly is to use flexible (typically wooden) floors and roofs. The failures at Northridge were predominantly due to failure of the out-of-plane connections between the perimeter walls and the roof. Most Australian buildings use concrete floors and either steel truss or concrete roofs. This should give greater rigidity to these structures under seismic loading due to improved diaphragm capacity in conjunction with robust diaphragm-to-wall connections.

However, many buildings in Northridge also used expansion anchors to connect diaphragms to walls which pulled out under repeated cyclic loading. It was observed that many of these failures occurred adjacent to the short walls of long rectangular buildings, rather than the long walls as might be expected. It is considered[15] that this is

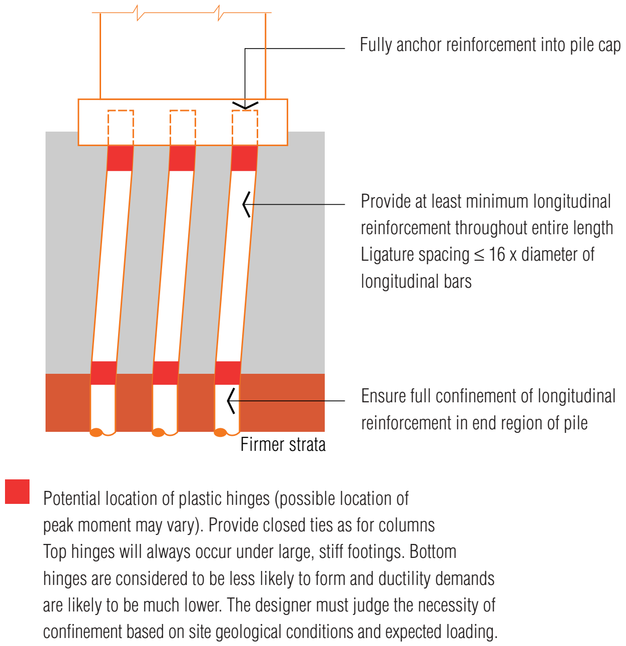


Figure 12

Piled foundations

because diaphragm capacity is selected on the basis of the maximum shear demand from the design earthquake loads. Therefore the capacity of the diaphragm across the shorter direction often greatly exceeds the demand based on the design earthquake. This additional capacity in turn leads to an additional demand on the connections when the earthquake forces greatly exceed the design loads.

Failures were also observed to be concentrated at re-entrant corners and discontinuities in the diaphragms where forces were required to be distributed between different members.

It is therefore imperative that designers consider diaphragm/wall connection design and detailing such that they might satisfactorily transfer these forces. AS 1170.4 Sections 4.2 to 4.4 specify the design forces for which diaphragms and their connections should be able to withstand. These conditions are not especially onerous and, as comparison with typical detailing in New Zealand shows, due to the inherent rigidity of the structure, tilt-up buildings may be designed and detailed for limited ductility provided sufficient attention is paid to force transfer between panels and between vertical and horizontal elements.

Foundation Systems

General Adequate foundation design is critical for ensuring that a building structure will be able to resist both the gravity loads and seismic forces calculated.

Where there is no possibility for inelastic deformations to develop under earthquake conditions, it is considered that standard detailing of reinforcement as for gravity loads and wind forces will be adequate. This will be the situation in the majority of buildings constructed in Australia.

However, where design indicates the occurrence, or possible occurrence, of reinforcement yielding during seismic action, the foundation structure, like the superstructure, must be detailed accordingly. As already mentioned, as a result of code loading requirements or design decision, the seismic response of the structure may be elastic.

Paulay and Priestly[3] suggest foundation systems that may support elastic superstructures. Two of these will be relevant to Australian designers;

- **Elastic Foundation Systems** In regions of low seismicity (as is generally the case in Australia) or for low buildings with structural walls it will be possible to design and detail the entire structure to respond within elastic limits.
- **Ductile Foundation Systems** In certain cases, the potential strength of the superstructure with respect to the specified seismic forces may be excessive (eg large shear wall structures). The designer might therefore consider that it will be preferable for the foundation system rather than the superstructure to be the principal source of energy dissipation during inelastic response. A potential drawback for this system is that damage may occur during moderately strong earthquakes. Large cracks may form if yielding of reinforcement has occurred. Further, repairs to foundations may be difficult and costly if required below the water table.

Foundation Structures for Frames As discussed in Design Methodology, AS 3600 provides some limited guidance regarding footing design and detailing. Although the code stipulates that for foundations located in soils with a maximum bearing capacity of less than 250 kPa, restraint must be provided in the horizontal direction to limit differential movement during an earthquake. It should be noted that reports from Kobe indicate that although liquefaction is a problem in poor soils, the water in the saturated reclaimed areas acted as a dampener, restricting damage to significantly less than that experienced in the adjoining 'dry soil' areas.

The Code considers that there is no possibility for inelastic deformations to develop under earthquake loading, and that standard detailing of footings for gravity and wind-induced loads only will be sufficient. However, the author considers that certain additional precautions can be warranted.

Isolated Footings These can prevent a problem with rocking or tipping if a plastic hinge forms in 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. The detailing of the column/footing joint must be carefully considered.

Combined Footings It may prove more feasible to absorb large moments transmitted by plastic hinges at column bases by using stiff tie beams between footings, whereby a high degree of elastic restraint against column rotations can be provided. In fact this detail is such that reinforcement yielding is unlikely to occur and it is considered that no special detailing requirements for ductility need be provided. It would, however, be necessary for the tie beams to have sufficient reserve strength over that of the hinging columns – see Design Methodology – Capacity Design.

If it is required to reduce the bearing pressure under the footing pads, they may be joined to provide one continuous footing.

Stub columns do require special consideration if inelastic deformations and shear failure are to be avoided. Paulay and Priestley[3] consider that plastic hinges should therefore be restricted to the column section immediately above the beam.

Piled Foundations Piled systems supporting structural walls may be subject to large concentrated forces due to overturning moments and shear forces. Careful design is therefore required.

Detailing of reinforced concrete piles should follow the recommendations set out above for columns. The end region of a pile under the foundation structure should be detailed to ensure full confinement of the longitudinal reinforcement using closed or helical ties. The locations of peak moments in the pile may necessitate the length confinement being considerably extended. Further, even if calculations indicate no tension loads, it is recommended that minimum longitudinal reinforcement be provided. The arrangement of longitudinal reinforcement should be as for columns, and the reinforcement should be fully anchored within the pile cap. In non-critical regions, nominal transverse ties or spiral hoops should be provided. Paulay and Priestley[3] recommend that vertical spacing not exceed 16 times the diameter of longitudinal bars **Figure 12**.

Conclusions

Australia is an area of moderate seismicity and low risk in comparison to California, Japan and New Zealand. The provisions for both the design and detailing of reinforced concrete structures in Australian codes reflect this.

The detailing requirements therefore are not onerous and loose-bar detailing and efficient fabrication techniques currently used (as recommended in the CIA detailing manual[8]) are still adequate and with little additional consideration will provide the levels of ductility and continuity of reinforcement to enable the structure to satisfactorily weather anticipated earthquake loading in Australia.

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 steel (even if not beam steel as there is a certain level of redundancy due to the presence of slab reinforcement) regardless of the type of structural system employed.

It is strongly recommended that columns in, for instance, shear wall or ordinary moment resisting frame systems that are not deemed to require detailing to Appendix A, should still be detailed so that buckling of bars will not occur. As noted above, magnitudes of earthquakes are difficult to predict with accuracy. Should an earthquake occur of significantly greater magnitude than that which has been designed for (at least a statistical possibility), brittle failure and collapse could occur.

With a limited additional quantity of properly detailed extra ligatures 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 transfer of the larger forces).

The choice for the designer/detailer is clear. A fully elastic response by the structure, whilst allowed by the Code, cannot be guaranteed. Therefore, to prevent catastrophic collapse and probable loss of life under a greater than designed-for event, a ductile failure must be ensured. This minimum required level of ductility can be readily achieved by judicious detailing in selected areas.

Precast concrete construction requires some special care in detailing to ensure that floors, especially, will act as diaphragms (if so designed) in order to properly transfer horizontal forces. Further consideration is required with mixed precast and insitu construction to ensure monolithic behaviour.

Compatible overseas experience has shown reinforced concrete, both insitu and precast, to be an eminently suitable and cost-effective solution for building structures in low to medium seismic zones such as Australia. Designers and specifiers can remain confident of reinforced concrete's ability to functionally and elegantly meet the needs of today's construction industry.

■

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