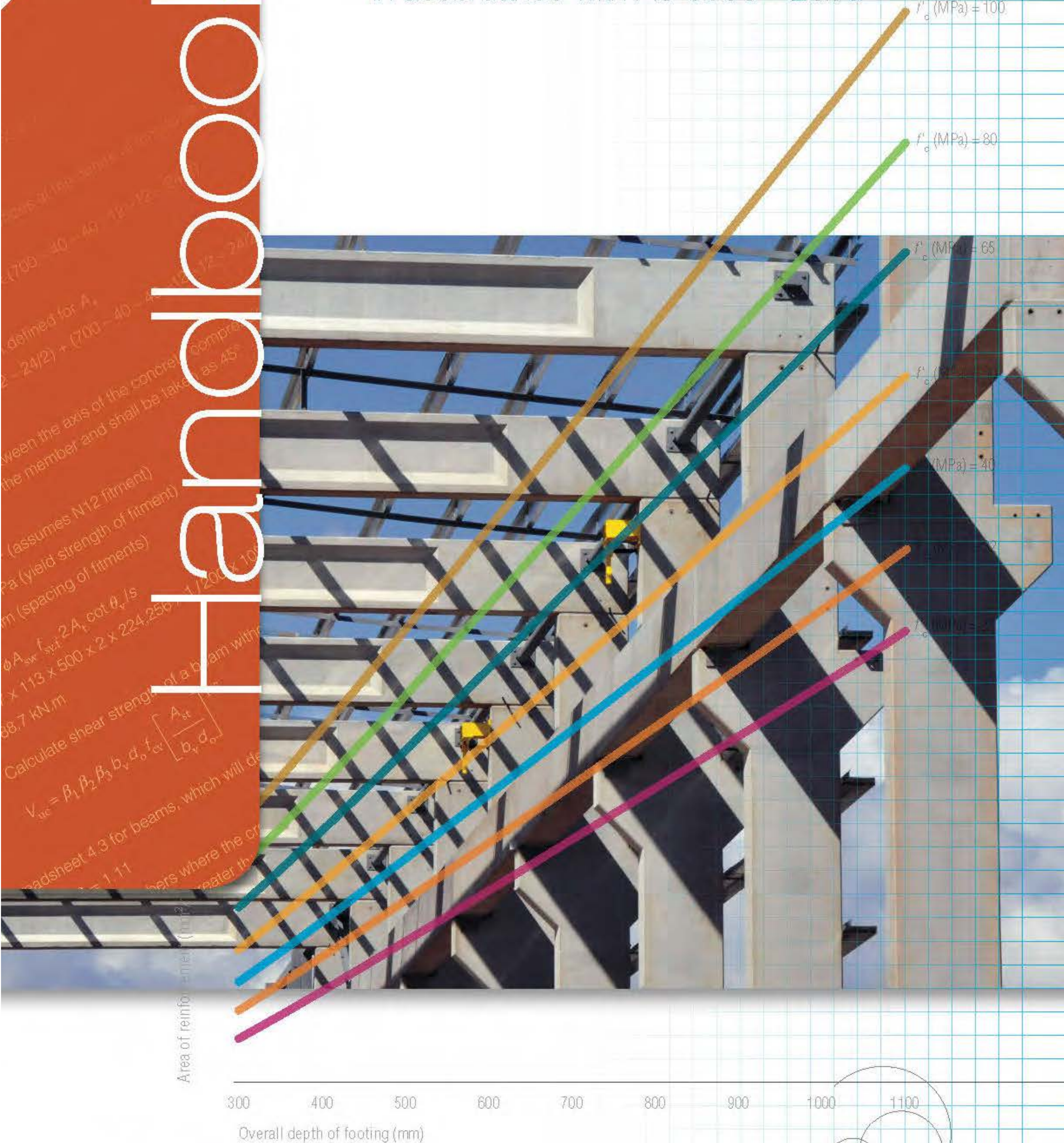
CCAA T38

HB71-2011 (Standards Australia)

Handbook

Reinforced Concrete Design

in accordance with AS 3600—2009



CEMENT CONCRETE & AGGREGATES AUSTRALIA

STANDARDS Australia

Handbook

Reinforced Concrete Design
in accordance with AS 3600—2009

A joint publication of

Cement Concrete & Aggregates Australia and

Standards Australia

Fifth edition July 2011

Fourth edition February 2002

Third edition May 1995

Second edition July 1991

First published September 1989

CCAA T38

HB71–2011 (**Standards Australia**)

Helen Rix Design

ISBN 978-1-877023-28-6

Standards Australia develops Australian Standards® and other documents of public benefit and national interest. These Standards are developed through an open process of consultation and consensus, in which all interested parties are invited to participate. Through a Memorandum of Understanding with the Commonwealth Government, Standards Australia is recognised as Australia's peak non-government national standards body.

For further information visit www.standards.org.au

Australian Standards®

Committees of experts from industry, governments, consumers and other relevant sectors prepare Australian Standards. The requirements or recommendations contained in published Standards are a consensus of the views of representative interests and also take account of comments received from other sources. They reflect the latest scientific and industry experience. Australian Standards are kept under continuous review after publication and are updated regularly to take account of changing technology.

International involvement

Standards Australia is responsible for ensuring the Australian viewpoint is considered in the formulation of International Standards and that the latest international experience is incorporated in national Standards. This role is vital in assisting local industry to compete in international markets. Standards Australia represents Australia at both the International Organization for Standardization (ISO) and the International Electrotechnical Commission (IEC).

Sales and distribution

Australian Standards®, Handbooks and other documents developed by Standards Australia are printed and distributed under license by SAI Global Limited.

CONTACT DETAILS:

For information regarding the development of Standards contact:

Standards Australia Limited
GPO Box 476 Sydney NSW 2001
phone: 02 9237 6000
fax: 02 9237 6010
email: mail@standards.org.au
internet: www.standards.org.au

For information regarding the sale and distribution of Standards contact:

SAI Global Limited
phone: 13 12 42
fax: 1300 65 49 49
email: sales@saiglobal.com

CEMENT CONCRETE
& AGGREGATES AUSTRALIA

Cement Concrete & Aggregates Australia is a not-for-profit organisation established in 1928 and committed to serving the Australian construction community.

CCAA is acknowledged nationally and internationally as Australia's foremost cement and concrete information body – taking a leading role in education and training, research and development, technical information and advisory services, and being a significant contributor to the preparation of Codes and Standards affecting building and building materials.

CCAA's principal aims are to protect and extend the uses of cement, concrete and aggregates by advancing knowledge, skill and professionalism in Australian concrete construction and by promoting continual awareness of products, their energy-efficient properties and their uses, and of the contribution the industry makes towards a better environment.

Cement Concrete & Aggregates Australia
ABN 34 000 020 486

CCAA OFFICES

SYDNEY OFFICE:
Level 6, 504 Pacific Highway
St Leonards NSW Australia 2065
POSTAL ADDRESS:
Locked Bag 2010
St Leonards NSW 1590
TELEPHONE: (61 2) 9437 9711
FACSIMILE: (61 2) 9437 9470

BRISBANE OFFICE:
Suite 2, Level 2, 485 Ipswich Road
Annerley QLD 4103
TELEPHONE: (61 7) 3227 5200
FACSIMILE: (61 7) 3892 5655

MELBOURNE OFFICE:
2nd Floor, 1 Hobson Street
South Yarra VIC 3141
TELEPHONE: (61 3) 9825 0200
FACSIMILE: (61 3) 9825 0222

PERTH OFFICE:
45 Ventnor Avenue
West Perth WA 6005
TELEPHONE: (61 8) 9389 4452
FACSIMILE: (61 8) 9389 4451

ADELAIDE OFFICE:
PO Box 229
Fullarton SA 5063
TELEPHONE: (61 8) 8274 3758

TASMANIAN OFFICE:
PO Box 246
Sheffield TAS 7306
TELEPHONE: (61 3) 6491 1509
FACSIMILE: (61 3) 6491 2529

WEBSITE: www.ccaa.com.au

EMAIL: info@ccaa.com.au

PREFACE TO FIFTH EDITION

This fifth edition is a complete rewrite of the *Reinforced Concrete Design Handbook* and brings it into line with the 2009 edition of AS 3600 *Concrete Structures* and *Amendment No. 1–2010*. It also takes into account changes in other Australian Standards that have occurred since the fourth edition was published, including AS/NZS 1170 *Structural Design Actions,* Part 0 *General principles* and Part 4 *Earthquake actions in Australia*.

The 2009 edition of AS 3600 includes significant changes to:

- The maximum concrete strength covered (now includes 100 MPa)
- Development lengths and lap lengths of reinforcement
- Use of Ductility Classes N and L reinforcement
- Durability and fire requirements.

The opportunity has been taken to review many of the charts and their relevance in the modern design office. In many cases, the previous charts were nomograms from an era when these were a common design tool. These have now been largely replaced by design software or, as in this Handbook, by spreadsheets.

The spreadsheets are used to illustrate the design principles of reinforced concrete, the requirements of AS 3600 and the recommendations of this Handbook. They are in keeping with current design technology.

The spreadsheets can be downloaded from CCAA website www.ccaa.com.au/publicationextras/.

There is a new chapter covering the strut-and-tie design method to reflect the new section in AS 3600.

There are also revised rules for crack control in beams and slabs but no charts or tables are provided. However, the Design Example in Chapter 10 includes calculations showing how these requirements can be checked.

By-and-large the order in which material is presented follows that of relevant sections in AS 3600, although not all the sections in the standard are covered.

The contribution of J Woodside FIEAUST FASCE FISTRUCTE MICE, Director, *J Woodside Consulting,* in reviewing the Handbook and in the preparation of the design charts and spreadsheets is gratefully acknowledged.

CONTENTS

Chapter 1_General

1.1 INTRODUCTION

1.1.1 Codes and regulations

Designers need to understand the framework of regulations and standards within which the design of the building or structure is designed and constructed. For most buildings in Australia, the Building Code of Australia (BCA)[1.1] sets out the regulatory requirements for the building*.

The Building Code of Australia sets out Objectives, Functional Statements, Performance Requirements and Building Solutions for the various aspects of design, eg structural, and health and amenity. The first two ('Objectives' and 'Functional Statements') are broad descriptors and are used mainly to interpret the latter two ('Performance Requirements' and 'Building Solutions').

The Performance Requirements are qualitative statements, eg that under structural provisions says:

A building or structure, to the degree necessary, must:

i Remain stable and not collapse; and

ii Prevent progressive collapse; and

iii Minimise local damage and loss of amenity through excessive deformation, vibration or degradation; and

iv Avoid damage to other properties,

by resisting the actions to which it may reasonably be subject...'

* *The terms 'building' and 'structure' are used to signify the same entity by administrations in Australia and in New Zealand. This may lead to some ambiguity where the terms are used interchangeably in some joint AS/NZS standards. In general, in Australia the term 'building' is used to refer to buildings—ranging from houses to multi-storey buildings—and 'structures' to refer to structures other than buildings whereas in New Zealand the term 'structure' is used inclusively to cover buildings and other structures.*

How the Performance Requirements are to be satisfied is spelt out in the Building Solutions. There are two broad approaches:

[a] Deemed-to-satisfy (DTS) solutions; and

[b] Alternative solutions.

The DTS approach involves designing the members, buildings and structures in accordance with the relevant Australian standards, eg for concrete in accordance with AS/NZS 1170[1.2] and AS 3600[1.3].

The Alternative Solution approach allows the designer to use other codes or standards, fire test data, etc. (The 2009 edition of AS 3600 omits a number of clauses from previous editions which gave rise to conflicts of interpretation with the BCA in this area, eg those that provided rules on interpretation of test data. Designers should be aware that their omission in AS 3600 does not imply that the approach is invalid but that the rules under which it is done now lie within the BCA under Alternative Solutions, not the DTS approach using the relevant Standard, eg AS 3600.)

As will be discussed later, AS 3600 provides for a number of different analysis and strength check approaches. However, the BCA and AS/NZS 1170.0 are written around a linear elastic analysis/limit states approach using characteristic strengths of the materials and factored loads.

Designers should be aware that AS 3600 provides minimum solutions, ie compliance is necessary for all buildings but particular buildings may require the application of more-stringent provisions or additional considerations/criteria to meet the client's requirements.

However, AS 3600 represents best practice even when it is not called up in the BCA and it cannot be ignored, especially where its requirements are more stringent than those in earlier editions of the standard.

1.1.2 Responsibility of designers and supervisors

The division of responsibility between the parties involved in the design and those in the construction of a building should be clearly understood and fully expressed in the terms of engagement between the owner and the designer, and in the contract for construction between the owner and the builder or contractor. 'Design' here includes the architectural and structural design of the building and the preparation of the drawings, specification, and sometimes the conditions of contract and preliminaries. Most projects will involve a number of other disciplines, eg mechanical, electrical and service engineering. Developing and documenting the final design solution will normally involve a design team covering all the required design disciplines.

The designer responsible for the structural design should be a practising civil or structural engineer eligible for Chartered Status of Engineers Australia or equivalent and experienced in the design of concrete structures of comparable importance. Architects and building graduates should not be expected to have the appropriate skills to undertake, nor should they assume responsibility for, the design of a concrete structure. Graduate engineers while working under guidance can design parts of concrete structures but they should not take responsibly for the overall design of the structure.

When designers assign the detail design of any elements to a manufacturer or supplier, they should ensure that this work is fully specified and controlled by way of detailed performance standards, and that these elements are coordinated with the structure as a whole. Examples of this are the detailed design of precast concrete elements and post-tensioned slabs.

The supervision of construction is the responsibility of the builder. All structures should be supervised by a suitably qualified person. If the structure is complex or incorporates prestressed concrete, a qualified and experienced engineer employed by the builder should be responsible for the supervision of construction.

Periodic inspection of construction on behalf of the owner or client is often undertaken by the designer, or by an experienced person employed by the owner or client but under the technical direction of the designer. Where the project is complex or unusual, a more-detailed inspection regime may be required. This arrangement facilitates the resolution, by the designer, of any queries that may arise as to the interpretation of the design documents.

Site records should be kept during construction to show the dates of concrete placing, test results, stressing details and any significant departures from the design drawings. These provide the owner with a useful record of the structure as built, should any modification be required in the future.

1.2 DESIGN PROCESS AND PROCEDURES

1.2.1 Broad structural design aims

[a] General The aim of structural design is to produce safe, serviceable, durable, aesthetic, economical and sustainable structures. Designers should always strive for simplicity, clarity and excellence in their design. Simple design does not mean elementary design but rather well conceived and quality design. As noted above, mere compliance with the appropriate codes and standards will not guarantee a satisfactory design for all buildings as they provide only a set of minimum requirements. The designer is responsible for ascertaining the appropriate criteria for the particular building and seeing that these are satisfied.

[b] Safety In service, a structure must be able to safely resist the actions (loads) expected to be imposed on it throughout its intended life. Safety must also be considered during the construction period, particularly while the concrete has not reached its design strength. Loads imposed on it during that period should be analysed as required. The design should also consider unusual load cases arising from any processes to be carried on in the structure. Some thought should also be given to the ultimate demolition of the structure.

Designers usually start with a framing plan, which is logical and sensible, and proceed to examine how that structure behaves when subjected to the various actions. In particular, they should review all possible failure modes to ensure that nothing important has been overlooked. This topic is discussed in more detail in Section 1.2.2 *The design process.*

A structure should be robust and possess structural integrity so that it is not unreasonably susceptible to the effects of accidental loads. Damage to a small area of a structure or the failure of a single element should not lead to the collapse of a large part of the structure, eg by progressive collapse. This topic is discussed in more detail in Section 1.4.6 *Structural integrity and robustness*.

The accidental hazard arising from fire is covered in building regulations, eg the Building Code of Australia. The particular requirements for different structural elements for fire resistance, eg Fire Resistance Levels to guard against structural collapse (structural adequacy), flame penetration (integrity) and heat transmission (insulation), are discussed in Chapter 3 *Durability and Fire Resistance*.

Designers must be alert to prevent gross errors during design, as these, along with those that may arise during construction, are probably the most common cause of failures. An independent check should be made of the design, including a review of the drawings and specification to ensure that the assumptions made in the design are valid.

[c] Serviceability Over its design life, during service under normal operating and load conditions, a structure must behave satisfactorily. The structure and its elements should not deflect or deform excessively or vibrate to cause discomfort to the occupants.

Any cracking or apparent distress of the concrete should not impair the structure's functionality or spoil its appearance. While some clients consider concrete to be indestructible, some maintenance and repairs of the concrete structure will normally be required during the life of the building, but this should be minimal.

[d] Durability A durable structure is one that performs its intended function over its design life in its environment without excessive degradation or unusual maintenance expense. There have been examples of inadequate durability, such as premature rusting reinforcement, spalling concrete, extensive wear and badly weathered concrete surfaces. The procedures necessary to ensure durable concrete structures are discussed in Chapter 3 *Durability and Fire Resistance*.

[e] Aesthetics An integral part of the design of any structure is consideration of its appearance. Buildings and structures such as bridges should be designed and detailed to present an attractive and well-proportioned appearance to suit their surroundings and environment. Architects rather than structural engineers are usually responsible for the appearance of buildings. However, there are many cases where the engineer can provide a significant input by the selection of appropriate framing systems and the proportioning of members to meet functional, load capacity and any aesthetic requirements.

[f] Sustainability In recent years, sustainability has become a design consideration for all structures. Sustainable design requires that social, environmental and economic outcomes are balanced. For example, a project is not sustainable if it damages the environment, or if it results in negative social outcomes such as loss of jobs or health problems, or if it results in financial loss.

Concrete is an important contributor to sustainable design. Concrete, like all products, has environmental impacts arising from the acquisition of raw materials, processing, transport and recycling at the end of its life. These are, however, significantly outweighed by the benefits that concrete delivers. Designers are referred to the CCAA's Briefings 11[1.4] and 13[1.5] and its website, www.ccaa.com.au, for further information on this topic.

Sustainable design also requires the designer to design an economical structure. Thus, the adoption of a simplistic, conservative design approach and poor detailing to minimise design costs—but resulting in an overdesigned structure—is not acceptable.

[g] Economy An economical structure contributes to limiting the overall cost of the project. This can be measured in terms of the initial cost, the construction time and the life cycle or overall cost. The low cost of concrete and reinforcement alone does not necessarily produce the most economical structure; construction and time-related costs must also be considered. Ease of construction must therefore be taken into account at the design stage.

1.2.2 The design process

The design process typically comprises three phases—conceptual design, preliminary design and final design. These are described briefly below and in more detail in Appendix A.

Since conceptual design will often be based on limited information, any structural design should be simple, quick and conservative without being heavy-handed. It is not the time for extensive computer modelling. Designers, however, need to carry out sufficient structural design to ensure that concepts are feasible.

The preliminary design phase is where the client's requirements for the project are developed in more detail. On major projects, more than one preliminary design may be produced.

The final design stage is where the design data is checked and the chosen optimum design is developed and detailed. This will include the preparation of project documentation and specifications. It is important for the designer to remember that the documentation is the means of communicating the design intentions to the contractor/builder and subcontractors. The documentation should be reviewed from this viewpoint before being issued.

There are a number of overseas manuals[1.6,1.7,1.8] on the design of reinforced concrete buildings to which the designer can refer for further information and guidance.

1.2.3 Order of design

The building should be designed in a logical order for analysis and drafting. For an office building the order of design might be as follows:

- Establish the design loads (AS/NZS 1170)
- Confirm the design data such as: survey, geotechnical, environmental, etc
- The occupancy of the structure, required fire ratings, sound transmissions, etc from the BCA, (normally provided by the architect)
- Establish exposure classification and durability requirements including concrete strength(s), cover(s) and axis distances, deflection criteria (AS 3600)
- Establish any other special design requirements
- Lateral stability for wind and earthquake loadings and general stability in two orthogonal directions
- Roof framing including slabs and beams
- Plant room slabs and beams
- Typical floor slabs and beams
- First floor and non-typical slabs and beams
- Ground floor slabs and beams

- Basement floor slabs and retaining walls
- Stairs and lift cores including lift motor rooms
- Column and wall load rundowns
- Column and wall designs
- Footings and foundation capacity
- Precast or external walling
- Robustness check and detailing
- Other architectural elements that may require structural design
- Checking and review of drawings and specifications.

1.2.4 Structural framing

Finding the best structural framing solution for a building is not straightforward and there will typically be alternative solutions. The framing must consider how all the loads find their way through the structure, both horizontally and vertically, to the footings. Framing is a trial-and-error process and adjustment will need to be made as the design proceeds. The process is neither taught nor covered in textbooks and requires a good appreciation of architectural and engineering constraints.

Concrete structural frames are commonly used in Australia and have the advantage of good performance in fire. They can be cast in situ, precast or both. The frame for larger projects usually needs to be modelled for input into the computer for analysis. Which members are pinned and which are continuous also need to be established.

Certain buildings lend themselves to standard solutions, eg an industrial building or shopping centre. Local conditions will sometimes favour different solutions depending on the local building industry capability, etc. Column/wall locations are often dictated by the intended use of space. For example, for a car parking building the column spacing must suit parking bay sizes; for an office building a column-free space may be required or there may be other spatial requirements developed by the architect from the client's needs.

The floor-to-floor height also needs to be considered and the space required for building services, particularly in the space under the floor and above the ceiling. Concrete allows efficient floor solutions, minimising the overall height of a building or maximising the number of floors in a given height.

Designers also need to define how lateral loads are resisted, suitable systems can include one or more of shear walls, moment-resisting (space) frames and cantilever columns or walls.

Assessing, apportioning loads and understanding the load paths can be difficult to appreciate. The assessment of all loads is one of the fundamental design considerations before commencing final analysis and design. If the loads are wrong or apportioned incorrectly, they will affect the design of all members, and extensive rework and extra time will be involved—assuming the errors are found—or, if the errors are not found, possibly an unsafe structure.

1.2.5 Initial estimation of member dimensions

The initial estimation of member sizes is generally based on past experience, some quick trial designs or other design information. Design offices may have design guides based on experience of successful designs and recommendations where problems have arisen.

The depth of flexural members is usually controlled by deflection considerations. The minimum thickness of walls tends to be governed by construction and cover considerations, and this is also true for column dimensions. The axial load capacity of columns can be significantly increased by increasing the concrete strength and/or increasing the longitudinal reinforcement percentage. Neither of these necessarily increases the column dimensions. However, lateral load bending moments and limiting sway movements may dictate some minimum dimensions.

1.3 DESIGN CHECKS AND METHODS OF ANALYSIS

In a real structure, the behaviour under load of individual elements can be complex, depending on the materials used and many other factors. Generally, idealised models of the frame or structure are developed to analyse how the real structure may behave.

The analysis that is carried out to validate a design is generally a two-step process although some computer programs may combine the two steps:

- Structural analysis of the frame or structure
- Strength check and other design checks at critical cross-sections of members.

The first step of analysis is aimed at determining the action effects such as bending moment, shear force, torsion and axial force at critical sections of members necessary for strength design or determining deformations of the structure. The second step is concerned with the strength check of these critical sections along with other design checks such as deflections.

AS 3600 makes provision for a variety of methods to be used for strength checks, viz:

[a] Procedure for use with linear elastic analysis methods of analysis with simplified analysis methods and for statically determinate structures (see AS 3600 Clause 2.2.2).

[b] Procedure for use with linear elastic stress analysis methods (see AS 3600 Clause 2.2.3).

[c] Procedure for use with strut-and-tie analysis (see AS 3600 Clause 2.2.4).

[d] Procedure for use with non-linear analysis of framed structures (see AS 3600 Clause 2.2.5).

[e] Procedure for use with non-linear stress analysis (see AS 3600 Clause 2.2.6).

In addition, it is permissible to carry out design checks for strength and serviceability by testing a structure or component member in accordance with the requirements of Appendix B in AS 3600 (see AS 3600 Clause 2.1.1).

The first of these procedures, (a), is the one which will be familiar to most designers and was in earlier editions of AS 3600. The other methods have been introduced into the 2009 edition of AS 3600 to permit the use of more-sophisticated computer-based analysis programs, eg Finite Element Analysis. Foster[1.9], while Foster et al[1.10] give a summary of the other methods, (b) to (e), and indicate where each may be applicable and the provisos associated with their use.

Designers should be aware that there is conflict between these latter methods, (b) to (e), and the requirements in the BCA and AS/NZS 1170.0. For example, BCA (Volume 1) BP1.2 mandates use of 5% characteristic material properties and this would preclude the use of some structural check procedures in AS 3600, eg non-linear analysis of framed structures which uses mean values of material properties. AS/NZS 1170.0 called up by the BCA is written around the linear elastic method of analysis and ultimate limit states approach. For example, see Section 2 in that Standard. This may or may not be a problem. However, strut–and–tie analysis may be the only appropriate method of design for non-flexural members.

This Handbook is written around the method in (a) which is compatible with both the BCA and AS/NZS 1170.0. No conflict is therefore foreseen with the following discussions, except perhaps for Chapter 9 *Strut-and-tie modelling*.

1.4 LIMIT-STATES DESIGN AND DESIGN CHECKS USING LINEAR ELASTIC METHODS OF ANALYSIS

1.4.1 General

A limit-states approach to design is assumed by the BCA and AS/NZS 1170.0. The procedure for use with linear elastic analysis methods and for statically determinate structures given in AS 3600 Clause 2.2.2 is compatible with this approach.

Limit-states design assumes there will be an acceptable probability that a structure designed and built in accordance with the Standard will not reach any limit state during its design life. That is to say, it will not fail by collapse or instability (ultimate limit states), or become unfit for service by deformation, vibration or cracking (serviceability limit states). In addition, the structure should not deteriorate unduly during its design life and should not be damaged by events such as fire, explosions and impact to an extent disproportionate to the cause. A checklist of design requirements includes:

- Stability
- Strength
- Serviceability
  - Deflection
  - Lateral drift (eg under wind or earthquake)
  - Cracking
  - Vibration
- Durability
- Fire resistance
- Structural Integrity/robustness (prevention of progressive collapse)
- Other limit states as required.

Limit-states design analyses the structure or part for the relevant combination of factored actions (the action effect). It then confirms that the design capacity, ie the nominal capacity multiplied by the capacity factor (capacity reduction factor), ϕ, exceeds the action effect. (The use of a global factor rather than partial safety factors, as adopted in European standards, follows the practice established in ACI 318[1.8] and that used in earlier editions of AS 3600.)

In essence, following this approach, the steps in design for the ultimate limit state are (the design for serviceability limit states is similar):

- Determine the actions on the structure
- Determine the appropriate combinations of actions
- Analyse the structure for the applied combinations of actions
- Design and detail the structure for robustness and earthquake

- Determine the design resistance of the structure using AS 3600
- Confirm the design resistance exceeds the action effects.

AS/NZS 1170.0 Section 5 provides broad guidelines for appropriate methods of analysis. The general nature of the rules allows for the wide variety of structural materials covered by the Standard, while reference is made to the appropriate material standard for specific guidance for that material.

1.4.2 Stability and strength

The structure as a whole and its parts are designed to prevent instability due to overturning, uplift and sliding. Generally, the design capacity of a member is calculated as the ultimate strength of the section, using a mathematical model to represent the failure condition, multiplied by the capacity reduction factor.

The capacity reduction factor, ϕ, accounts for variations between the basis of the calculation and the likely actual condition, viz:

- Variations in the strength of concrete and reinforcement
- Variations in the dimensions of the member and the location of reinforcement and in the relative position of members, eg eccentricities in columns
- Inaccuracies in the design equations for calculating internal actions and the strength of the section
- Mode of failure, eg ductile or brittle, and the resulting warning of failure
- Importance of the member and its effect on the structure.

For example, compare a beam and a column. The column is less ductile and more sensitive to concrete strength variations than the beam; the column usually supports a larger area than the beam, making the consequences of failure likely to be more serious. For these reasons, the capacity reduction factor, ϕ, for pure bending is larger than that for axial compression, eg 0.8 to 0.6.

Note that the design aids, spreadsheets and charts prepared to be compatible with one standard such as AS 3600 must be used only with the load factors, load combinations and capacity reduction factors applicable to that standard.

1.4.3 Serviceability – deflection control

AS 3600 Clause 2.3.1 requires that *Design checks shall be carried out for all appropriate service conditions to ensure the structure will perform in a manner appropriate for its intended function and purpose.*

Under working loads, the deflection of slabs and beams must be controlled to meet two general criteria:

- The total deflection should not adversely affect the appearance or efficiency of the structure. AS 3600 limits this value to span/250.
- The incremental deflection should not adversely affect other elements such as finishes, services, partitions, glazing and cladding. Where partitions are detailed to minimise the effect of movement, this deflection should be limited to span/500. If not (eg masonry partitions without closely spaced joints), this limit should be reduced to span/1000.

In addition, the following requirements as appropriate must be met:

- The deflection for imposed action (live load and dynamic impact) for members subjected to vehicular or pedestrian traffic should not exceed span/800.
- For transfer members the total deflection should not exceed span/500 where provision is made to minimise the effect of deflection of the transfer member on the supported structure. Otherwise, span/1000.

For cantilevers, the deflections are generally half of those noted above when rotation at the support can occur.

AS 3600 also states *Design limits given or implied in Clauses 2.3.2 and 2.3.3 are based on previous design experience, and reflect requirements for normal structures. In special situations other limits may be appropriate. For further guidance refer to Appendix C of AS/NZS 1170.0.*

Design for the serviceability limit states involves reliable predictions of the instantaneous and time-dependent deformation of the structure. This is complicated by the non-linear material behaviour of concrete caused mainly by cracking, tension stiffening, creep and shrinkage. Designers can refer to the notes on the CIA Seminar series[1.11] for further information.

The calculation of deflection comprises two stages – an elastic or immediate component and an inelastic or creep component that occurs over a long period as shown in Figure 1.1. These are considered as

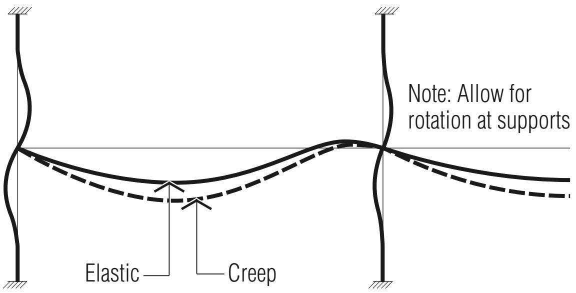


Figure 1.1 Elastic and creep deflections

short-term and long-term effects with the appropriate combinations of actions (loads) acting in each case. The long-term loading comprises permanent actions (eg self-weight) and the quasi-permanent component of the imposed action (eg live load), while the short-term loading includes the probable peak value of the imposed action. Typical values for these are given in Section 1.5 *Actions and Combinations of Actions*.

The calculated deflection is measured from a theoretical line diagram representing the member in its as-cast position. The limit on total deflection of span/250 below an assumed horizontal line may not be sufficient to prevent a slab being unsatisfactory for non-structural reasons, eg water ponding on a roof. These problems may be overcome by cambering the formwork (usually not preferred) or by stressing the floor or roof to counter the long-term deflection. In the case of long spans, these methods are used frequently; the designer should, however, be careful to check that the reduced stiffness of the floor does not result in excessive incremental deflection or vibration under live load, or large rotations and distress at supports.

There is also a visual limit of about 25–40 mm even in long-span floors where long-term deflections become noticeable. Building owners will often not accept deflection over these limits and may perceive a large deflection as a failure.

In assessing the practical effect of deflections, the designer should allow also for realistic construction tolerances. The limits in AS 3600, discussed in Section 1.7 *Material and Construction Requirements*, are based on the requirements of structural adequacy and strength. Tighter construction tolerances usually need to be specified or special details developed to meet the serviceability requirement.

1.4.4 Serviceability – cracking

The designer and the building owner tend to view cracking differently. Engineers generally regard some cracking as inevitable; owners on the other hand tend to regard any cracking as a major defect.

Most cracking in concrete structures is due to shrinkage of concrete. The structure and the steel reinforcement have to be designed and detailed to control the effects of this shrinkage. This will involve first determining whether or not cracks are allowed to occur and, if so, where they can occur in respect of structural integrity and aesthetics. The size of cracks must be limited so as not to cause future durability problems. In buildings, stiff vertical elements such as cores, basement and retaining walls can result in unsightly cracking in slabs unless steps are taken to minimise such cracking by methods such as the provision of construction joints or delayed pour strips.

AS 3600 sets out guidelines for the amount of reinforcement to control cracking, eg Clause 2.3.3 states that cracking of beams and slabs under service conditions shall be controlled in accordance with the requirements of Clause 8.6 or 9.4, as appropriate.

In a small percentage of cases, cracks are a symptom of structural or durability distress, eg spalling of concrete due to reinforcement corrosion. In these cases, the cause of cracking needs to be diagnosed and appropriate remedial measures taken.

For a more detailed discussion on the types of cracking and practices to minimise its occurrence, see *Guide to Concrete Construction*[1.12] and *Movement, restraint and cracking in concrete structures*[1.13].

1.4.5 Serviceability – vibration

Design for vibration is outside the scope of this Handbook. Designers should consult a specialist text or seek expert advice if vibration is likely to be a problem. The chapter on vibration in the *Precast Concrete Handbook*[1.14] is recommended.

1.4.6 Structural integrity and robustness

There are currently no specific requirements for design for structural integrity (the prevention of progressive collapse) or robustness in the BCA or AS 3600. The BCA mentions progressive collapse, implying that design for it is covered under the requirement of sustaining at an acceptable level of safety and serviceability the most adverse combination of loads. Section 6 in AS/NZS 1170.0 includes minimum structural robustness requirements. However, this still does not fully address the issue.

The spectacular 1968 failure in the UK of *Ronan Point*, a block of flats constructed of large precast concrete panels, focused attention on the susceptibility of this form of construction to accidental or other loading such as gas explosions, as shown in Figure 1.2. Because of this accident, the British Standard for concrete structures[1.15] was revised and included specific detailing requirements to provide continuity and ductility.

Other high profile cases of progressive failure include the 1995 bomb attack on the Alfred P. Murrah Federal Building in the US and the collapses of the towers at the World Trade Centre in New York in 2001. Other national codes now also include provisions to prevent progressive collapse.

A continuously reinforced, cast-in-place, concrete structure is less likely to be at risk of progressive collapse than a precast one because of its inherent ability to redistribute unusual loads and span over possible local failures, assuming the detailing allows for continuity. Normally, only a general review of such a structure would be required to check its possible failure modes.

Figure 1.2 *Ronan Point*, UK

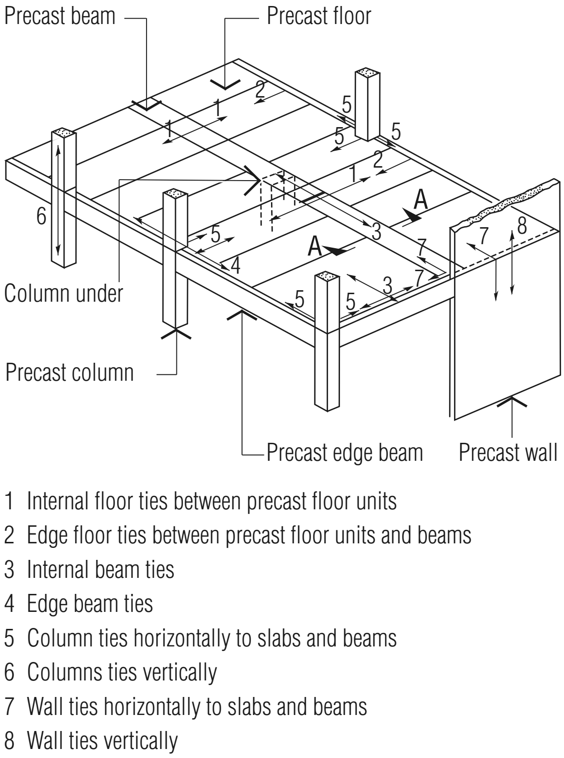
However, further investigation should be carried out for:

- precast concrete structures;
- unusual structural systems or mixed construction using different materials;
- structures subject to special risks, such as vehicle collision and explosion (eg a chemical factory).

If an abnormal load can be identified, then it is possible to design directly for this condition. Usually, however, this is not the case, so other methods must be adopted to control the extent of damage.

One method commonly used in Europe from Eurocode 2[1.16] is to design for specified forces at each level of the structure and to provide a system of horizontal and vertical ties, properly anchored, to resist these forces. Eurocode 2 replaced BS 8110 in the UK in 2010. For precast-panel buildings, this results in longitudinal, transverse and peripheral ties at each floor level interconnected with sufficient continuous vertical ties to restrain the walls at each level as shown in Figure 1.3.

Another method is to provide alternative load paths so that the structure can bridge over the gap formed if a part of a floor or wall or column is accidentally removed. For precast-concrete-panel buildings, this method also results in a system of horizontal and vertical ties. By notionally removing a part of each wall in turn, the floor over is designed to act as a catenary,



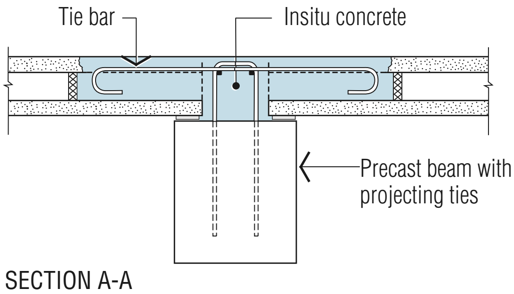


Figure 1.3 Integrity and robustness with 3-dimensional tying system for precast concrete

which can support a large load although it may sag 300 mm or more. In an insitu-concrete building, floors or beams can act in a similar way provided they are detailed correctly and can span twice their actual span even when sagging excessively in catenary action. Examples of cantilever and beam action for precast buildings are shown in Figure 1.4.

Useful references on methods of design for structural integrity are: Mitchell and Cook[1.17], FIP[1.18], Elliott and Tovey[1.19] and the ACI-ASCE[1.20].

1.4.7 Durability and fire resistance

These aspects are covered in Chapter 3 *Durability and fire resistance*.

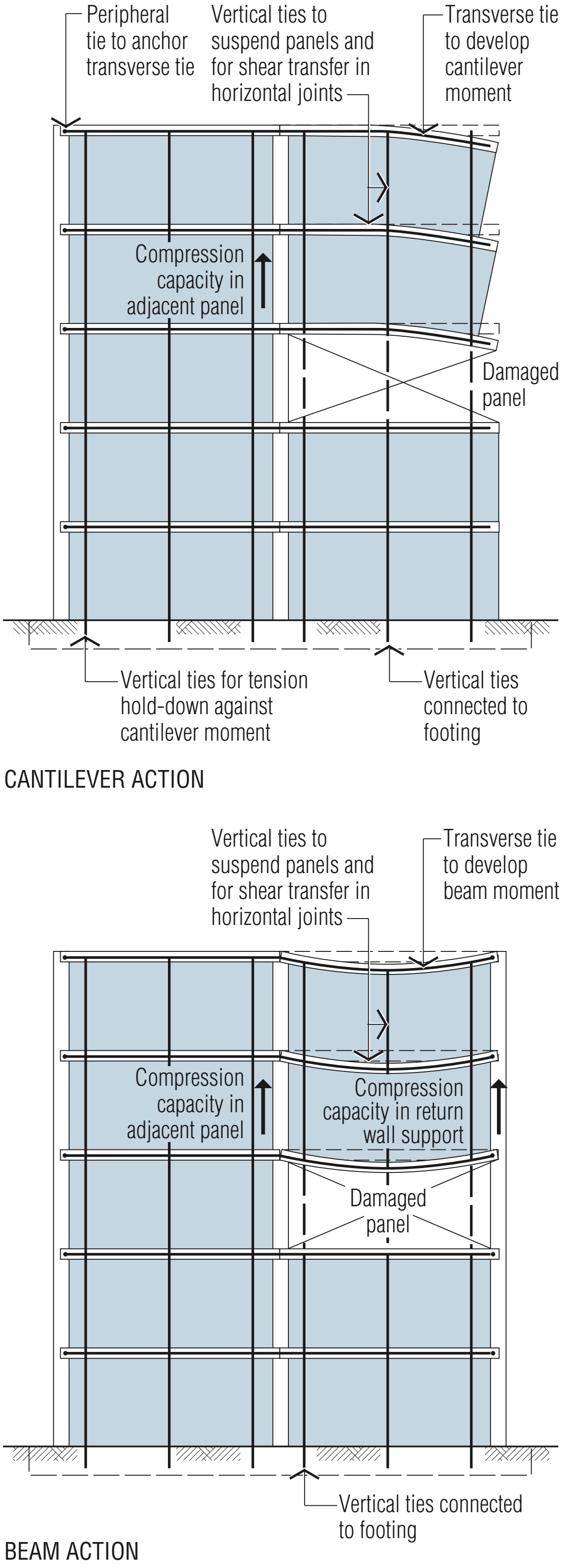


Figure 1.4 Structural integrity – alternative load

1.5 ACTIONS AND COMBINATIONS OF ACTIONS

1.5.1 General

AS/NZS 1170.0 sets out the various actions (loads) and the combinations of actions (load combinations) to be considered in design for ultimate and serviceability limit states. For combinations particular to prestressed concrete refer to AS 3600.

1.5.2 Permanent, imposed and other actions

A permanent action (dead load) is defined as 'an action that is likely to act continuously and for which variations in magnitude with time are small compared with the mean value'. Generally, it is taken to comprise the self-weight of the member plus the weight of all materials of permanent construction – walls, floors and ceilings (including finishes), services, permanent partitions and fixed machinery supported by the member.

An imposed action (live load) is defined as 'a variable action resulting from the intended use or occupancy of the structure' and is taken to include uniformly-distributed, concentrated, impact and inertia actions as applicable. Wind, snow and earthquake actions are considered separately. Many imposed actions are short-term relative to the life of the structure; however, some may be of long duration, eg storage loads, and thus have an effect similar to a permanent action.

AS/NZS 1170.1 specifies minimum values for imposed actions on floors (floor live loads) for various classified occupancies; these are typically in the range of 2 to 5 kPa, except for storage areas where stacked material results in larger values. The specified uniformly distributed loads are blanket values to cover the expected effect of the occupancy for both small and large areas. Surveys of actual loadings in offices indicate that the statutory loads are reasonable for small areas but tend to be conservative for larger areas. Live load reductions are permitted for certain floors and supporting columns, walls and footings. A reduction of up to 50% is allowed according to a formula, which depends on the loaded area (see AS/NZS 1170 Part 1 Clause 3.4.2). If this reduction is used, then the design drawings should state both the nominal live load and that the reduction for area has been applied.

During construction, special actions (loading) may occur and may control the design of some members. Staged construction and composite concrete members usually require a specific design check unless fully propped. Precast members such as floor units supporting wet concrete must be designed for construction loads in accordance with AS 3610[1.21]. Construction loads from concrete, formwork, falsework and equipment such as forklifts, cranes and hoists may be greater than the imposed actions (live load) and thus may require strengthening of the structure or the provision of temporary supports during construction. Note that a significant proportion of all structural failures occur during construction, often because a critical loading condition is overlooked or the concrete strength at the time of loading is over-estimated. Structures incorporating flat-plate floors are susceptible to progressive collapse during construction for this reason – when failure of an upper floor due to early stripping leads to failure of those below.

Other actions such as concrete shrinkage and creep, shortening due to prestress, temperature effects and foundation movements, cause deformations of the structure and, if resisted by the structure, result in internal forces, which are in equilibrium. A ductile structure is able to redistribute these loads so that the capacity of the member to carry the ultimate-strength loads is not affected. However, the deformation may be a significant factor in the serviceability check.

1.5.3 Combinations of actions – strength design

Combination factors for actions (load factors) for strength design take into account:

- the possibility of unfavourable deviations of the actions from the characteristic values;
- the possible inaccurate assessment of the action effects and their significance for safety;
- variations in dimensional accuracy in so far as they affect estimation of the action effects; and
- the reduced probability of combinations of actions occurring, all at their characteristic values.

The value of the load factor depends on the degree of uncertainty, while the combination factor depends on the probability of that combination of loads occurring. The nominal value of a permanent action (dead load), *G*, is its mean value. The factor 1.2 applied to it assumes that it can be assessed to within 10%. If circumstances arise where this assumption is not warranted, then a conservative estimate of the permanent action should be made, or part of it treated as an imposed action (live load). For the case of load reversal and where the permanent action is beneficial, the factor is taken as 0.9.

The nominal value of the imposed action (live load), *Q*, is intended to be the peak value for a 50-year life with a probability of exceedence of 5%. This is a characteristic value with a probability similar to the definition of characteristic concrete strength, f'_c. Imposed actions vary and usually comprise two components: a sustained relatively constant value for a particular occupancy, and a superimposed extreme value arising from a crowd of people or a concentration of objects. The factor of 1.5 reflects this greater variability compared with permanent actions, and the combination factor, ψ, of 0.4 to 0.6 reflects the probable lower action likely to occur at the same time as another peak-action effect. See Table 1.1.

Wind loads are determined in accordance with AS/NZS 1170 Part 2 either by a simplified procedure, applicable only to small buildings, or by a more detailed procedure using either a static or dynamic analysis. For significant structures, consideration should be given to a model tested in a wind tunnel to determine the wind forces more accurately, including effects on surrounding buildings.

For earth-retaining structures in accordance with AS 4678[1.22], the nominal earth load, F_{eu}, should take account of the likely earth and groundwater pressure. The load factor to be applied is 1.0 if the earth pressure is determined using a limit-states method, or 1.5 if earth pressures are determined using working loads.

Liquid-retaining structures are usually constructed so that there is an upper limit to the height of the retained liquid and its density is defined. The accuracy of determination is similar to that of a permanent action, so a factor of 1.2 is used for static liquid pressure, F_{lp}. However, if the density is not well defined or the height is not limited, then a value of 1.5 should be used. If dynamic conditions are possible, these should be considered separately and a factor similar to that for imposed actions applied.

Earthquake loads and load combinations are specified in AS 1170 Part 4[1.23] while additional design and detailing requirements for earthquake resistance are given in AS 3600 Appendix C.

The prestressing force, *P*, is limited by the breaking load of the tendons. In checking the ultimate strength of an anchorage or the possibility of a concrete compressive failure at transfer, a factor of 1.15 is specified by AS 3600. In this case, the prestressing force is similar to an external load and is taken as the maximum jacking force at the anchorage. A different situation arises where secondary moments and shears are being calculated in an indeterminate structure. Because these are caused by a prestress force, which is internal to the cross section, a load factor of 1.0 is sufficient.

In selecting combinations of actions, the principle adopted is to consider each imposed action at its maximum value taken in turn with other imposed actions at their probable values at any time. The load combinations (combinations of actions) given in AS/NZS 1170.0 and those specified in AS 3600 are shown in Table 1.1.

TABLE 1.1 Load combinations

Strength	
$1.35G$	Permanent action only (does not apply to prestressing forces)
$1.2G + 1.5Q$	Permanent action and imposed actions
$1.2G + \psi_c Q + W_u$	Permanent action and arbitrary-point-in-time imposed and wind actions
$0.9G + W_u$	Permanent action and wind action reversal
$G + \psi_c Q + E_u$	Permanent action and arbitrary-point-in-time imposed and earthquake action (given in AS 1170.4)
$1.2G + \psi_c Q + S_u$	Permanent action and arbitrary-point-in-time imposed action and the appropriate one of the following actions: *snow, liquid pressure, rainwater ponding, ground water and earth pressure*
$1.15G + 1.15P$	Permanent action and prestressing force (acting in same direction from AS 3600)
$0.9G + 1.15P$	Permanent action and prestressing force (acting in opposite directions from AS 3600)
$G + \psi_l Q$ + thermal action for fire	Permanent action and arbitrary point-in-time imposed action and thermal action for fire
Stability	
$0.9G$	For combinations that produce net stabilising effects
$1.35G$	For combinations that produce net destabilising effects
$1.2G + 1.5Q$	
$1.2G + \psi_c Q + W_u$	
$G + \psi_c Q + E_u$	
$1.2G + \psi_c Q + S_u$	
Serviceability	Use appropriate combinations of G, $\psi_s Q$, $\psi_l Q$, W_s, E_s, P and other actions
$G + W_s + P$	eg short-term serviceability
$G + \psi_s Q + P$	
$G + P$	eg long-term serviceability
$G + \psi_l Q + P$	

TABLE 1.2 Short-term, long-term and combination factors ψ_s, ψ_l and ψ_c (after AS/NZS 1170.0)

Imposed action	**Short-term factor** (ψ_s)	**Long-term factor** (ψ_l)	**Combination factor** (ψ_c)
Distributed imposed actions, Q			
Residential and domestic structures	0.7	0.4	0.4
Offices	0.7	0.4	0.4
Parking	0.7	0.4	0.4
Retail	0.7	0.4	0.4
Storage	1.0	0.6	0.6
Other	1.0	0.6	0.6
Roof actions			
Roofs used for floor-type activities	0.7	0.4	0.4
Other roofs	0.7	0.0	0.0
Concentrated imposed actions			
Floors	1.0	0.6	as for distributed floor actions
Floors of domestic housing	1.0	0.4	as for distributed floor actions
Roofs used for floor-type activities	1.0	0.6	as for distributed floor actions
Other roofs	1.0	0.0	0.0
Balustrades	1.0	0.0	0.0

1.5.4 Combinations of actions – stability

In checking for stability, the loads and actions are divided into components tending to cause instability and those tending to stabilise the structure. Where the strength of a member is used to provide stability, then the design strength, ϕR_u, should be used.

1.5.5 Combinations of actions – serviceability

For serviceability checks, both short-term and long-term effects should be considered. For wind loading, only short-term effects need to be considered. Values for ψ_c, ψ_l and ψ_s are shown in Table 1.2.

1.6 LINEAR ELASTIC METHODS OF ANALYSIS

1.6.1 General

AS 3600 Clause 2.2.2 sets out the strength limit state for linear elastic methods of analysis with simplified analysis methods, and for statically determinate structures where the design capacity must be greater than or equal to design load effect, ie:

$R_d \geq E_d$

where

$R_d = \phi R_u$ is the design capacity

ϕ is a capacity reduction factor given in AS 3600 Table 2.2.2

R_u is the calculated capacity determined in accordance with the relevant sections of AS 3600

E_d is the design action effect or the 'design action' or the ultimate load condition. Where E_d is determined by one of the following methods of analysis:

— Linear elastic analysis in accordance with Clause 6.2

— Linear elastic analysis incorporating secondary bending moments due to lateral joint displacement in accordance with Clause 6.3

— One of the simplified methods of analysis in accordance with Clauses 6.9 and 6.10

— Equilibrium analysis of a statically determinate structure.

1.6.2 Linear elastic analysis (AS 3600 Clause 6.2)

Concrete structures behave only in a linear elastic manner under small, short-term loads while the sections are uncracked. As loads increase, the sections crack and the behaviour becomes non-linear and moments are distributed from the peak-moment regions to less highly stressed sections of the members. Despite this, linear elastic analysis may be used to determine the action effects in structures for both the serviceability and the strength limit states. If the structure is ductile, this procedure has been shown by experience to be safe for strength design.

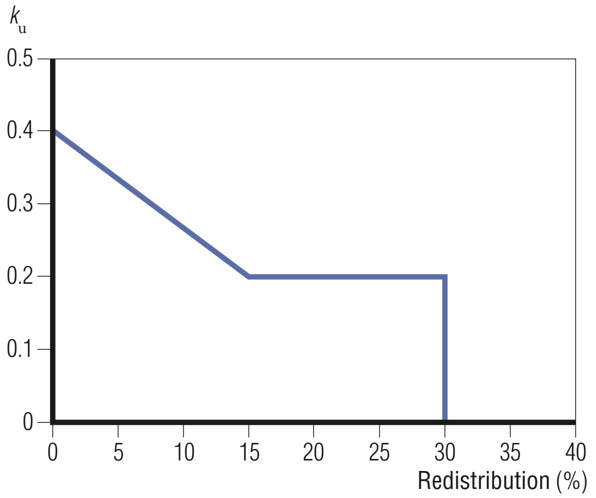


Figure 1.5 Moment redistribution versus k_u

Design moments and shears are calculated by applying factored loads to the structure, which is analysed assuming linear elastic behaviour. The critical sections are then checked for strength, assuming local inelastic action. Some redistribution of moments has to be relied upon when this model is used. This redistribution depends on ductile behaviour, which is ensured by limiting the neutral axis parameter, k_u, to 0.36 and placing limits on the amount of redistribution depending on the Ductility Class of the reinforcement. AS 3600 Clause 6.2.7 gives guidance on the amount of moment redistribution that can be assumed in design.

If Ductility Class N reinforcement is used, then the amount of redistribution permitted may be calculated using a deemed-to-satisfy approach based on the value of the neutral axis parameter, k_u. Up to 30% is permitted provided that static equilibrium is maintained and k_u does not exceed 0.2. For values of k_u between 0.2 and 0.4, the permissible distribution is obtained by interpolation as shown in Figure 1.5.

If Ductility Class L reinforcement is used, no redistribution is permitted unless an analysis is undertaken to show that there is adequate rotation capacity available at the critical cross sections to allow such redistribution to occur.

Figure 1.6 illustrates the use of moment redistribution to reduce the maximum values of bending moment to be used in design for Ductility Class N reinforcement.

A linear elastic analysis may be used for buildings with floor slabs and for framed structures without slabs. For reasons of equilibrium and static compatibility, the span of flexural members is taken as the distance centre-to-centre of supports. However, the size of these supports is taken into account by the defined critical sections for negative moment and shear force.

In prestressed concrete members, the restraint at supports usually induces parasitic reactions and so-called secondary bending moments and shears that are determined by applying the prestress to an assumed unloaded, uncracked structure. As these are internally induced, a load factor of 1.0 is sufficient to obtain the design values that are added to the elastically determined moments and shears for factored dead and live load. These total moments may be redistributed in the same way as for reinforced concrete members.

1.6.3 Linear elastic analysis incorporating secondary bending moments (AS 3600 Clause 6.3)

This method applies to frames that are not restrained by shear walls or bracing or both and for which the relative displacement at the ends of compression members is less than $L_u/250$ under the design load for strength. It is similar to linear elastic analysis except that additional bending moments are calculated to take account of the lateral displacement.

1.6.4 Simplified methods of analysis (AS 3600 Clauses 6.9 and 6.10)

[a] Idealised frame method (Clause 6.9)

This method may be used for framed structures incorporating reinforced or prestressed two-way slab systems and is not subject to the restrictions on geometry and loading applicable to other simplified methods.

The idealised frame is taken as one of a series of idealised full-height frames running longitudinally through the building and a second series taken transversely. A linear elastic analysis is carried out for each frame using one of the standard frame programs or similar and using a number of practical simplifications:

- For vertical loads, a simple frame may be taken as comprising one floor and the columns above and below, with these columns fixed at their far ends Figure 1.7.
- The width of the frame may be taken as the width of the design strip for flat slabs, or the effective width for T-beams and L-beams using the equations in AS 3600 Clause 8.8.2 and as shown in Figure 1.8.
- The relative stiffness of the members may be calculated using the gross concrete section or the transformed section if the same basis is used throughout.
- The fully idealised frame must be considered in the analysis for horizontal loads unless it is restrained by shear walls or similar.

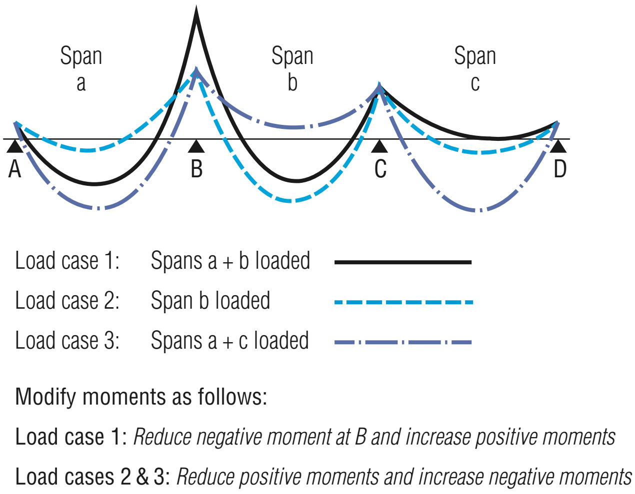


Figure 1.6 Examples of moment redistribution

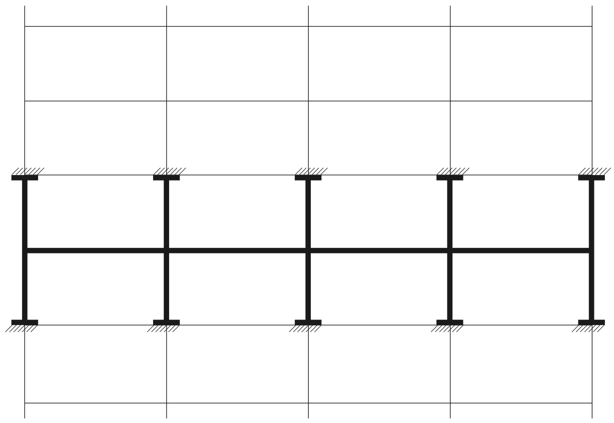


Figure 1.7 Idealised frame

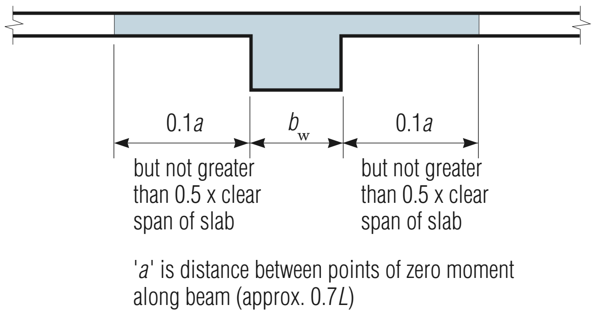


Figure 1.8 Effective width of T-beam

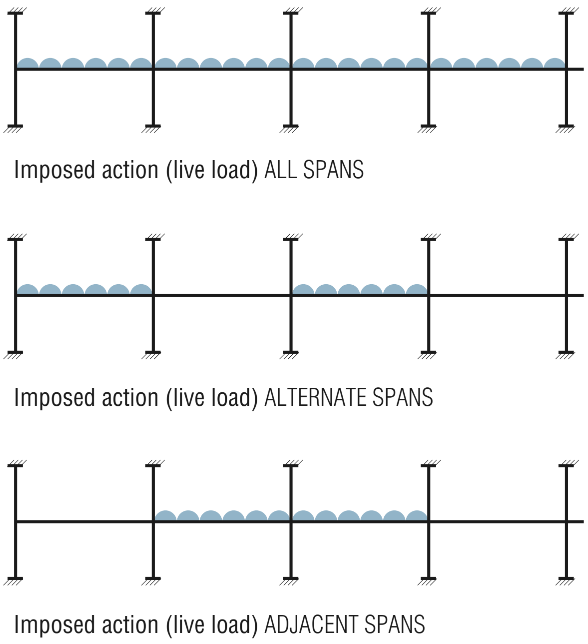


Figure 1.9 Examples of pattern actions (loading)

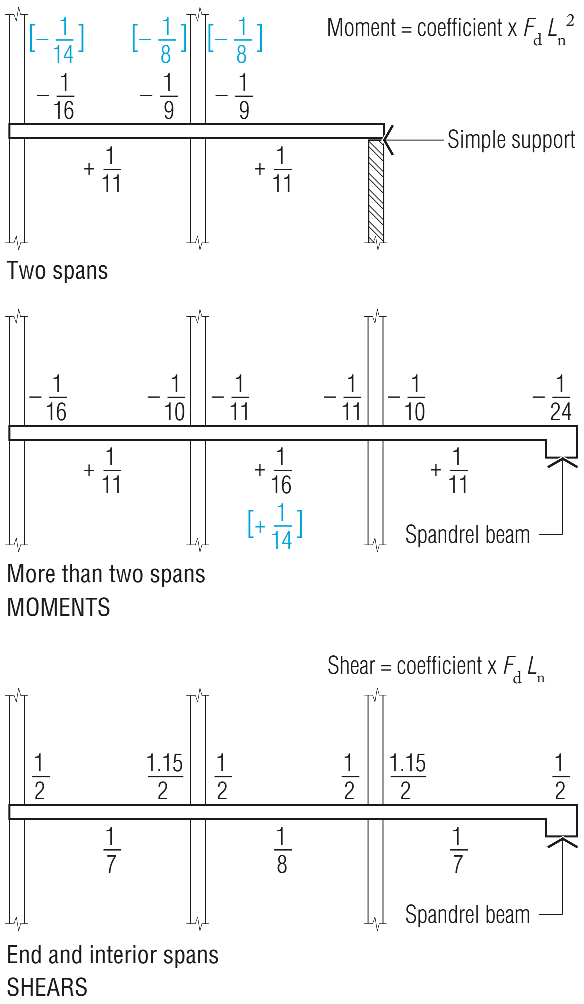


Figure 1.10 Approximate moments and shears – one-way members. Bracketed blue figures are for Ductility Class L reinforcement

- Ductility Class L reinforcement must not be used for the main flexural reinforcement.
- Openings must comply with AS 3600 Clause 6.9.5.5.

The arrangement of vertical action (load) to determine the critical moments and shears may be restricted to only a few combinations as set out in AS 3600 Clause 2.4.4.

- If the imposed action (live load) pattern is fixed
  - — the factored imposed action (live load).
- If the imposed action (live load), Q, does not exceed three-quarters of the permanent action (dead load), G,
  - — the factored imposed action (live load) on all spans.
- If the imposed action (live load), Q, exceeds three-quarters of the permanent action (dead load), G,
  - — the factored imposed action (live load) on alternate spans
  - — the factored imposed action (live load) on two adjacent spans
  - — the factored imposed action (live load) on all spans.

  as shown in Figure 1.9.

[b] Simplified method for reinforced continuous beams and one-way slabs (Clause 6.10.2)

This method provides a simple, approximate and conservative evaluation of the bending moments and shears in continuous reinforced concrete beams and one-way slabs where:

- the spans are approximately equal, with the longer of two adjacent spans not greater than the shorter by more than 20%;
- the loads are essentially uniformly distributed and the imposed actions (live load) do not exceed twice the permanent actions (dead load);
- the members are of uniform cross section and the reinforcement is arranged in a specific way.

This method is normally used only on simple structures and slabs, usually supported by loadbearing walls or similar.

The coefficients are shown diagrammatically in Figure 1.10. Because these values are not statically compatible, they should not be used for deflection calculations. If moment reversals occur during construction, these should be investigated separately. Note the higher moments (bracketed blue figures) to be used with Class L reinforcement.

[c] Simplified method for reinforced two-way slabs supported on four sides (Clause 6.10.3)

For reinforced two-way slabs supported on four sides by beams or walls (having corners that are prevented from lifting) and subject to uniformly distributed loads, approximate bending-moment coefficients for a range of edge conditions are tabulated in AS 3600 Tables 6.10.3.2 (A) and (B) depending on whether Class N or Class L reinforcement is used. Detailing must be in accordance with AS 3600 Clause 9.1.3.3 and there must be no openings or penetrations through the thickness of the slab adversely affecting its strength or stiffness. Slabs with Class L reinforcement must be continuously supported on walls.

Moments are calculated as:

$$\text{Moment} = \text{coefficient } \beta \times F_d L_o^2$$

where

$$L_o = L - [0.7 \times \Sigma(a_{sup} \text{ at each end of the span})]$$

The coefficients β_x and β_y are given (in decimal values) in AS 3600 Tables 6.10.3.2 (A) and (B).

The shear forces in the slab and the reactions of the supporting beams may be determined by allocating the load using 45° lines emanating from the corners of the slabs as shown in AS 3600 Figure 6.10.3.4. Although not stated in AS 3600, the shear force at the face of the first interior support of an end span should be taken as 1.15 times the simple span value, similar to that for one-way slabs.

[d] Simplified method for reinforced two-way slab systems having multiple spans (Clause 6.10.4)

The simplified method provides a quick and direct method of design for slabs that meet the restrictions of geometry and live loading (imposed actions) as set out in AS 3600 Clause 6.10.4.1. The total static moment is calculated for each span of the design strips taken in two directions at right angles using the effective span, L_o. This is consistent with the idealised-frame method as the critical section for negative bending moment is the same in each case.

The design positive and negative moments are then determined by applying the factors tabulated for interior spans and end spans. In the latter case, the distribution is varied to suit the end restraint provided by the exterior support. Where adjacent spans differ, the designer may either use the larger negative moment or distribute the unbalanced moment to the adjoining members to obtain the design negative moment. In addition, a redistribution of the design moments of up to 10% is permitted, if the total static moment is not reduced. Only Class N reinforcement can be used for this design method.

TABLE 1.3 Distribution of design moment to column strip (from AS 3600 Table 6.9.5.3)

Location	Column strip moment factor for: Strength limit state	Column strip moment factor for: Serviceability limit state
Negative moment		
interior support	0.6 to 1.0	0.75
exterior support	0.75 to 1.0	0.75 to 1.0*
Positive moment	0.5 to 0.7	0.6

* *Depending on whether there is a spandrel beam*

The transverse distribution of these design moments into the column strip and the middle strip is carried out using tabulated factors that are the same for both the simplified method and the idealised-frame method and are shown in Table 1.3. The values for these factors give the designer a considerable range in which to work.

Unequal spans or patterned live loads cause unbalanced moments to be transferred from the slab to the column. A minimum value is specified for this unbalanced moment and this is obtained by taking half the design live load as acting on the longer span and no live load on the shorter span. This moment is included in the shear design at the column-slab junction.

1.7 MATERIAL AND CONSTRUCTION REQUIREMENTS

1.7.1 General

The designer is obliged to set out in the drawings and specification all the requirements for the construction of the structure so that it can be built to meet the intent of the design. AS 3600 Clause 1.4 sets out the design data that should be shown on the drawings. Note that AS 3600 does not contain specification-type clauses relating to construction, it has only general performance-type clauses. The project specification thus needs to spell out the specific requirements for the project's construction.

The project specification should include those items of good practice that the designer considers necessary. A useful document is the national building industry specification system, NATSPEC, which is a master specification containing a library of clauses from which designers can select those suitable for their project and which they can supplement with specific clauses as required.

1.7.2 Concrete materials and manufacture

The constituent materials of concrete and its manufacture are covered by a series of standard specifications, most of which are sufficiently comprehensive to require only a citing in the project specification. Where an Australian Standard specification is not available, reference may be made to an overseas standard such as ASTM, or a performance requirement may be set out in the specification. The manufacture of concrete is covered by AS 1379[1.24], which includes all methods of manufacture: site-mixed, transit-mixed and factory-mixed. It covers the handling, storage and batching of materials, equipment such as mixers and agitators, mixing and delivery of the plastic concrete.

1.7.3 Specification of concrete

AS 3600 specifies concrete by referencing AS 1379. This latter standard defines two classes of concrete: normal-class and special-class. Normal-class concrete is satisfactory for the majority of projects, while special-class concrete is specified only where particular performance criteria are needed or as required by AS 3600, such as for B2, C1 and C2 exposure. The classifications are used to avoid misunderstandings between the builder and the concrete supplier, and the possibility of concrete being ordered only in terms of strength when special requirements are called up in the specification.

Normal-class concrete provides for the standard strength grades N20, N25, N32, N40 and N50 with slump of 20, 30, 40, 50, 60, 70, 80, 90, 100, or 120 mm and maximum nominal size of aggregate of 10, 14 or 20 mm. The particular value of each together with the intended method of placement (if project assessment is required) and air entrainment (if required) should be specified.

Generally, normal-class concrete with strengths in the range of 25 to 50 MPa is used on low- and medium-rise building structures. Higher strength concretes are used typically in walls and columns carrying high loads in taller structures, or in special conditions. Designers should be aware that S class concrete with strengths greater than 50 MPa might be difficult to supply to some sites (eg in country areas), while very high strength concrete (>65 MPa) may not be available in some cities. Before specifying high performance concrete with strengths greater than 50 MPa, designers should check on its availability with their local suppliers.

Special-class concrete is specified when there are any different or additional requirements, and only after careful consideration for its need. It will be needed when the concrete is to have: compressive strength, slump or aggregate other than those available in normal-class concrete; any limit on ingredients or mix proportions; or any special performance requirement such as a particular limit on shrinkage. Special-class concrete is designated as S class and can have different prefixes depending on its specific requirement.

Where concrete is specified as special class and one of the exposure classifications B1, B2, C1, C2 or U is specified in AS 3600, prefixes to the strength grade shall be SB for concrete in exposure classification B2, SC for concrete in exposure classification C1 or C2 and SU for concrete in exposure classification U. For SB, SC or SU class concrete, the properties specified shall include the aggregate durability class in accordance with AS 2758.1 and the relevant requirements of AS 3600.

To avoid misunderstanding when the concrete is specified and ordered, the class must be stated.

1.7.4 Quality control of concrete

AS 3600 requires that all concrete for structures designed in accordance with it shall be assessed in accordance with AS 1379 for the specified parameters. All concrete must be tested and subjected to production assessment by the supplier to ensure that the appropriate quality is being maintained.

Project assessment is optional for normal-class concrete but is mandatory for special-class concrete. If project assessment is specified, then the concrete delivered to that project is subject to additional testing. In this case, the specification should nominate the responsibility for carrying out this extra testing and who will bear the cost. Note that for specified parameters other than strength, the specification has to set out the method of production control, eg test method, the frequency of testing and acceptance criteria.

For a large project, project assessment will usually give sufficient samples to obtain a statistically reliable assessment of the concrete supplied to that project and at an acceptable cost. However, for small or medium sized projects, the cost of obtaining sufficient samples for a reliable assessment is usually prohibitive. Production assessment, as specified in AS 1379, will provide a reasonable level of quality assurance for the majority of small structures.

1.7.5 Tolerances for construction

Tolerances for structures and members are specified for two reasons. The first is concerned with structural adequacy and strength, ie to ensure that the design assumptions, in particular the ϕ factors used in the strength calculations, are met. The second relates to serviceability and appearance and will normally overrule the requirements of the first.

AS 3600 specifies only tolerances that are necessary for the first reason, ie strength and safety. The designer should examine these carefully, and should generally specify tighter tolerances for construction. Limited guidance on this latter type is provided in AS 3610 where tolerances are specified for different classes of surface finish.

Experience has shown that concrete structures can typically be built to tolerances of about ±10 mm to ± 20 mm. Where such tolerances are not achieved, the non-structural elements such as walls, windows, etc will often not fit properly, or extra costs will be incurred in achieving flat faces, etc. This topic is also covered in Section 4.5 of the *Precast Concrete Handbook*.

The tolerances specified in AS 3600 reflect design practice and should be easy to achieve. For exposed-to-view concrete, the classes of surface finish in AS 3610 cover a range of work from the highest quality, suitable for monumental structures, to average quality, suitable for many structures. The designer should balance the cost of formwork and related construction to produce a given standard of finish against the overall appearance of the project and/or part being considered and the distance from which it will be viewed.

The practical difficulty of accurately measuring concrete surfaces and of achieving tight tolerances should not be overlooked. Note that AS 3610 limits the areas where the Class 1 finish can be specified. Its use requires a pragmatic approach.

For practical convenience, tolerances in AS 3600 are measured to the surface and not to the centreline of members. Any point on a surface should lie within a tolerance envelope from its theoretical position. For columns and walls in the first 20 storeys of a building, an absolute limit of 40 mm horizontally is specified to control the overall location of the building. For columns and walls, the deviation from plumb, floor-to-floor, must not exceed the greater of the specified dimension divided by 200 or 10 mm. For other members, the deviation from a specified dimension must not exceed the greater of the specified dimension divided by 200 or 5 mm.

In checking these tolerances, an allowance must be made for possible movement of members, such as the deflection of floors, axial shortening both vertically and horizontally or thermal movements in slender structures.

The acceptable tolerance on location of reinforcement and tendons depends on the effect of any variation on the strength of the member and also on the possible reduction in cover and its effect on durability. Negative tolerances are permitted on cover and have been allowed for in the covers specified for durability and axis distance for fire resistance in AS 3600. Where durability is considered a significant factor, consideration should be given to using a larger cover than the minimum requirement of AS 3600.

1.7.6 Formwork

The general requirements for formwork are covered in AS 3610. The particular requirements for formwork that affect the safety and serviceability of the concrete structure are specified in AS 3600. These relate essentially to two conditions:

- The removal of formwork and the strength of the recently cast member
- The loads imposed on the structure by the plastic concrete and its supporting formwork.

The builder is usually responsible for the design, erection, stripping and safety of the formwork. This responsibility should be stated clearly in the project specification. In addition, if the designer requires particular constraints on the method of construction or wishes to oversee and review details of the proposed formwork then this should be specified.

The removal of formwork from a vertical surface is controlled by the time necessary for the concrete to gain sufficient strength and to avoid damage during stripping the exposed surface. In addition, where the colour of the off-form concrete needs to be consistent, the time of removal should be similar for all elements. Typically such surfaces are stripped at 1–2 days.

The removal of soffit formwork from reinforced beams and slabs at an early age is limited by the need for safety, to control cracking in the concrete and to limit the deflection. In the case of a slab with undisturbed shores kept in place, the slab is analysed as a plain concrete member subject to its self-weight plus a construction load of 2 kPa Figure 1.11. The design moment induced by this load must be less than the ultimate strength of the section calculated using characteristic flexural tensile strength of the concrete at the time of form removal. If control samples are taken and the concrete strength is obtained by

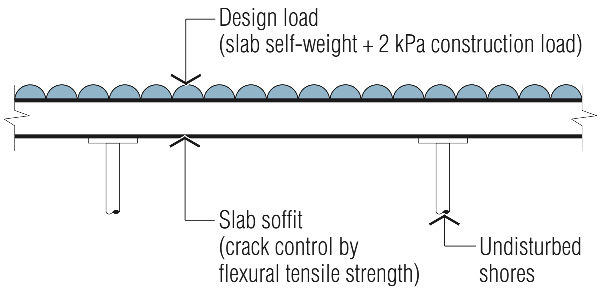


Figure 1.11 Form removal from soffit

testing, then that strength can be used to determine the characteristic flexural strength. As an alternative to testing of concrete, the mean concrete strength at 7 days may be taken as the values specified in AS 1379 Table 2.

In lieu of performing the above calculation, the designer may use the minimum stripping times for soffit forms specified in AS 3600 Tables 17.6.2.4 and 17.6.2.5 but formwork supports must stay in place for longer. These tables cover two cases: reinforced slabs continuous over formwork supports and of normal-class concrete cured at various temperatures; and removal of formwork supports from beams and slabs not supporting structure above.

In multi-storey structures, the early removal of formwork and its supports is desirable for speed of construction and economical reuse of forms. The floor being cast at any time is supported by the floors below. Depending on the number of supporting floors (usually a minimum of 3 or 4) and the relative stiffness of these floors, the load is shared between them with a series of closely spaced props. The floor directly beneath that being cast is less mature than, and is being supported also by, the floors below it.

A simplified elastic analysis of these floors results in quite high values for the maximum load. Depending on the number of sets of forms, this maximum load varies between 2.25 and 2.40 times the self-weight of the floor, with a converged value of 2.0. If the live load is less than the dead load of the floor, clearly there is a danger of overloading the slabs during construction. Further, if the temperature and curing conditions are not as expected there is a danger of a slab failure.

Various methods such as prop release and re-shoring have been devised to reduce these apparently very high loads, and measurements have been taken to check if the simplifications of analysis are reasonable. The measured loads generally comply with the simplified analysis; the variation apparently being due to creep and shrinkage warping of the concrete frame. Further guidance on this subject may be obtained from AS 3610 and literature referenced in it.

Prestressed floors are usually designed for staged stressing so that the prestress is applied progressively as the concrete gains strength, the floor becomes largely self-supporting, and the forms may be removed in stages, usually at about 7 days when the floor is fully stressed. For multi-storey buildings and several levels of formwork, the sharing of load and analysis is as for reinforced concrete.

REFERENCES

1.1 *Building Code of Australia* Australian Building Codes Board, 2010.

1.2 AS/NZS 1170 *Structural design actions,* Standards Australia,
Part 0: *General principles*
Part 1: *Permanent, imposed and other actions*
Part 2: *Wind actions*
Part 4: *Earthquake actions in Australia.*

1.3 AS 3600 *Concrete structures* Standards Australia, 2009.

1.4 *Sustainable Concrete Materials* Briefing 11, Cement Concrete & Aggregates Australia, 2010.

1.5 *Sustainable Concrete Buildings* Briefing 13, Cement Concrete & Aggregates Australia, 2010.

1.6 *Manual for the design of reinforced concrete building structures,* 2nd Ed, Institution of Structural Engineers (jointly with Institution of Civil Engineers), 2002.

1.7 *Manual for the design of concrete building structures to Eurocode 2* Institution of Structural Engineers, 2006.

1.8 ACI 318 *Building code requirements for structural concrete* American Concrete Institute, 2008.

1.9 Foster SJ 'Design and analysis procedures', paper presented at *AS 3600 – 2009 National Seminar Series: What's New? What's Different? Improvements and Developments – what are the implications and what do they mean for you,* Concrete Institute of Australia, 2009.

1.10 Foster SJ, Kilpatrick AE and Warner R *Reinforced Concrete Basics 2E: Analysis and design of reinforced concrete structures* Pearson Education, Australia, 2010.

1.11 *Serviceability – Design for Deflection and Crack Control* Concrete Institute of Australia Seminar Series, 2010.

1.12 *Guide to Concrete Construction* (T41/HB64) 2nd Ed, Cement Concrete & Aggregates Australia/Standards Australia, 2002.

1.13 *Movement, restraint and cracking in concrete structures*, The Concrete Society TR67, 2008.

1.14 *Precast Concrete Handbook* 2nd Ed, National Precast Concrete Association Australia and Concrete Institute of Australia, 2009.

1.15 BS 8110 *Structural use of concrete* Part 1: *Code of practice for design and construction* British Standards Institution, 1997.

1.16 BS EN 1992, Eurocode 2: *Design of concrete structures* British Standards Institution, 2004.

1.17 Mitchell D and Cook WD 'Progressive collapse of slab structures' *Concrete Framed Structures: Stability and Strength* Narayanan R (ed), Elsevier Applied Science, 1986.

1.18 FIP Commission on Prefabrication *Planning and Design Handbook on Precast Building Structures,* Thomas Telford Ltd, 1994.

1.19 Elliott KS and Tovey AK *Precast concrete framed buildings: Design guide* British Cement Association, 1992.

1.20 ACI-ASCE Committee 550 'Design Recommendations for Precast Concrete Structures', *ACI Structural Journal* January-February 1993, pp 115–121.

1.21 AS 3610 *Formwork for concrete* 1995 and AS 3610.1 *Formwork for concrete* Part 1: *Documentation and surface finish* Standards Australia, 2010.

1.22 AS 4678 *Earth-retaining structures* Standards Australia, 2002.

1.23 AS 1170 *Structural design actions* Part 4: *Earthquake actions in Australia* Standards Australia, 2007.

1.24 AS1379 *Specification and supply of concrete* Standards Australia, 2007.

BLANK PAGE

Chapter 2_ Design properties for concrete and reinforcement

2.1 CONCRETE

2.1.1 General

The performance of a concrete structure depends on a number of factors ranging from the design to its loading history. Not the least of these factors is the insitu quality of the concrete in it. This in turn is affected by two major factors, the quality of the concrete supplied to the project and the construction process employed.

Considering first the quality of the concrete supplied to the project, the major factors influencing this are the type and quality of the constituent materials and the proportions in which they are mixed. However, for design, it is usual to adopt values for the various design properties, eg compressive strength, on the basis of what is a reasonable design value from a member-behaviour perspective and what can be achieved in the geographical location using local materials.

In general, designers should specify the values of only the concrete properties they require and not specify limitations on how the supplier should produce the concrete, except to require that materials and manufacture comply with the relevant Australian standards.

Equally important to ensure that the design quality is achieved for the concrete in its final place, is specifying that the concrete is appropriately transported, placed, compacted, finished and cured, not overlooking the fact that to facilitate this, appropriate properties of the fresh concrete need to be specified. However, the importance of actually ensuring that the provisions of the specification are complied with on site must not be overlooked. Ensuring that proper curing is undertaken and that unauthorised addition of water is not allowed are also important[2.1].

Design values for concrete are specified in AS 3600[2.2]. In general, these are characteristic values, eg f'_c, as they provide appropriate values for strength design accommodating the variation inherent in concrete production and the subsequent construction processes. However, average values are preferred for some properties, eg E_c, as they give a better prediction of the in-service behaviour of the member or structure.

Generally, AS 3600 provides for design properties to be taken as either a prescribed value or to be determined from tests carried out on concrete made from the proposed materials. Alternatively, values may be derived from historical records of similar concrete. If records are not available and tests are required, the designer should allow for the time and possible delays to obtain the results.

If test results and testing of various properties are to be specified, the designer needs to understand the precision of the test, ie the repeatability and reproducibility. The choice of the design value and the value included in the specification need to take these factors into account.

2.1.2 Strength

General AS 3600 specifies concrete with a characteristic 28-day compressive strength in the range 20 to 100 MPa.

Although most pre-mixed concrete suppliers can supply concrete in the range 20 to 50 MPa, designers should be aware that concretes above 50 MPa are deemed to be special-class concretes and may not be readily available in some regions.

More importantly, while the design methods in AS 3600 cover high strength concretes, additional design and detailing will be required for them than for lower strength concretes.

Compressive strength AS 3600 Clause 3.1.1.1 specifies standard strengths of 20, 25, 32, 40, 50, 65, 80 and 100 MPa. In this series, the strength of each grade is about 25% greater than that of the preceding grade. In practice, for members such as slabs and beams, the choice of strength grade will frequently be determined by durability and serviceability considerations rather than the structural requirements. However, for columns and walls it may be determined by load-carrying capacity, ie strength.

Non-standard strength grades may be specified but these are deemed to be special-class concretes. Associated properties, eg shrinkage, may need to be specifically determined and project testing is required.

Tensile strength The uniaxial tensile strength of concrete is determined from either:

- Tests. The flexural tensile strength obtained by testing plain concrete beam specimens and calculating the extreme fibre stress at failure in accordance with AS 1012.11[2.3], or the principal tensile strength obtained using the split-cylinder test method in accordance with AS 1012.10[2.3]. In these cases the uniaxial tensile strength, f_{ct}, is taken as:

  $f_{ct} = 0.6 f_{ct.f}$ or $f_{ct} = 0.9 f_{ct.sp}$ as appropriate.

- Alternatively, in the absence of more-accurate data the characteristic uniaxial tensile strength may be taken as $0.36\sqrt{f'_c}$, and the characteristic flexural tensile strength as $0.6\sqrt{f'_c}$.

The uniaxial tensile strength is used in calculations limiting cracking of concrete such as web shear cracking in prestressed beams, while the flexural strength is used in designing plain concrete members such as pavements, and in calculations to control flexural cracking.

2.1.3 Modulus of elasticity

The modulus of elasticity of concrete, E_c, is taken as the secant modulus of the non-linear stress-strain relationship as shown in Figure 2.1 and is used in the calculation of deformations.

In most cases, the value of E_c can be taken as the value given by the empirical formulae given in the Standard:

$E_c = \rho^{1.5} \times 0.043\sqrt{f_{cmi}}$ (when $f_{cmi} \leq 40$ MPa); and

$E_c = \rho^{1.5} \times (0.0243\sqrt{f_{cmi}} + 0.12)$ (when $f_{cmi} > 40$ MPa)

For normal-weight concrete and up to f_{cmi} = 40 MPa, the formula reduces to $E_c = 5055\sqrt{f_{cmi}}$. Note that the formula uses f_{cmi}, the mean value of the in situ compressive strength at the age of the concrete being considered, not f'_c. In the absence of more-accurate data f_{cmi} can be taken as 90% of the mean value of the cylinder strength, f_{cm}. The precision of the formula is noted as ±20%. If a higher precision on the calculation of immediate deflection is required, then values from trial mixes or similar concretes should be used.

Properties of standard concrete grades using equations given in the Standard are shown in Table 2.1.

2.1.4 Density

To comply with AS 3600 the saturated surface-dry density of the concrete has to be in the range of 1800 to 2800 kg/m^3. The density of plain concrete depends on the density of the coarse aggregate and the water-cement ratio. AS 3600 suggests using a density of 2400 kg/m^3 for normal-weight unreinforced concrete.

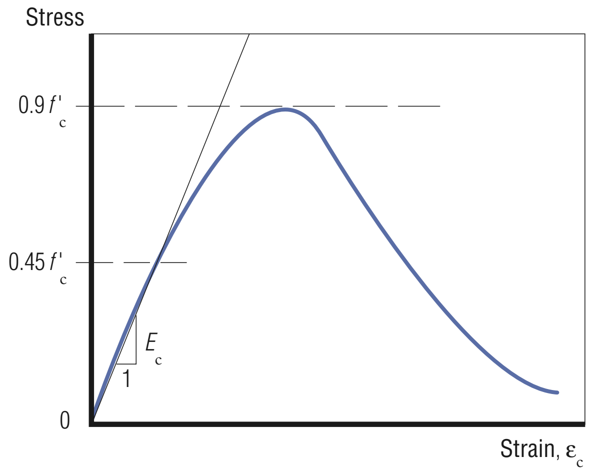


Figure 2.1 Modulus of elasticity of concrete

For reinforced concrete, an allowance should be made for the reinforcement. For most structures, a conservative value of 2500 kg/m^3 (25 kN/m^3) for the unit weight of reinforced or prestressed concrete is satisfactory for design purposes. AS 1170.1[2.4] suggests that the density of reinforced concrete is 24 kN/m^3 plus 0.6 kN/m^3 for each 1% of reinforcement.

2.1.5 Stress-strain curves

The Standard does not prescribe a stress-strain curve for concrete but allows the use of a curvilinear form defined by recognised equations, eg CEB[2.5], or determined from test data. For design, the shape of the in situ uniaxial compressive stress-strain curve is taken as that for a maximum stress of $0.9 f'_c$ for strength and f_{cmi} for serviceability limit states, respectively.

Under uniaxial stress, for concrete with characteristic compressive strengths in the range 20 to 100 MPa, the peak stress occurs at a strain of approximately 0.0025, but varies with mix. The shape of the curve changes

TABLE 2.1 Properties of standard concrete grades

Grade or characteristic compressive strength, f'_c (MPa)	Characteristic flexural tensile strength, $f'_{ct.f} = 0.6\sqrt{f'_c}$ (MPa)	Uniaxial tensile strength, $f'_{ct} = 0.36\sqrt{f'_c}$ (MPa)	Modulus of elasticity, $E_{c.28}$ (MPa)
20	2.68	1.61	24 000
25	3.00	1.80	26 700
32	3.39	2.04	30 100
40	3.79	2.28	32 800
50	4.24	2.55	34 800
65	4.84	2.90	37 400
80	5.37	3.22	39 600
100	6.00	3.60	42 200

with strength, with the ascending and descending branches becoming steeper as the strength increases.

In the stress-strain curve, the maximum stress is modified from that of the standard cylinder test to account for the differences in the size, environmental/curing and testing conditions between that of the cylinder test to that of the insitu concrete under local conditions.

2.1.6 Poisson's ratio

The Standard provides a value of 0.2 for concrete. This assumes the concrete is uncracked in tension.

2.1.7 Coefficient of thermal expansion

The Standard provides a value of 10 x 10^{-6}/°C. This should be sufficient for most calculations even though the actual value can vary by ±20% depending on aggregate type, volume of cement paste and degree of saturation. In other standards, eg Eurocode 2[2.6], the values of the coefficients of thermal expansion for concrete and steel are taken as equivalent, whereas AS 3600 suggests different values for each.

2.1.8 Shrinkage

Shrinkage is the decrease in the volume of hardened concrete caused mainly by the loss of moisture as a result of drying, and also by chemical changes in the cement hydration products. It is independent of the load applied to the concrete; it depends chiefly on the relative humidity and temperature of the environment, the size of the member and the constituent materials of the concrete.

The basic shrinkage strain is measured by taking standard test specimens wet-cured for 7 days and then stored in air at 23°C at a relative humidity of 50% for 56 days. Tests have shown that the aggregate type has a significant influence on the shrinkage of concrete. The range of shrinkage values for normal concrete in major cities is given in Table 2.2.

TABLE 2.2 Value of final drying basic shrinkage strains for major cities (after AS 3600)

City	**Value of final drying basic shrinkage strain** (mm/mm x 10^{-6})
Brisbane	800
Sydney	800
Melbourne	900
Adelaide	1000
Perth	1000
Hobart	1000

The figures reflect the best estimate of the value for design purposes of the whole range of normal-class concretes available in Australia. If designers are concerned about shrinkage, a better estimate for design purposes can be obtained by using measurements on similar local concrete. If shrinkage is a significant design parameter, then special-class concrete should be specified and the desired basic shrinkage strain nominated (remembering that for such concrete extra project testing will be required). Designers should also check that such concrete can be supplied since suitable aggregates may not be available locally to achieve such limits.

The Standard gives a method to calculate the design shrinkage strain, ie the sum of the autogenous shrinkage strain and the drying shrinkage strain, of concrete at any time. It also provides a table of typical final design shrinkage strains after 30 years based on a value of 1000 x 10^{-6} for the final drying basic shrinkage strain.

The design shrinkage strain for various environments and size of member may be obtained directly from the series of curves given in the Standard. The four environments covered may be assumed to reflect conditions of increasing humidity. The description 'interior environments' reflects the situation inside non-air-conditioned buildings, while the others reflect the environments defined for durability considerations given in Clause 4.3 of the Standard. For practical design conditions, these general classifications are considered to provide a more useful guide to designers than would an attempt to provide absolute values for the effects of temperature and relative humidity.

The suggested accuracy for the calculation of ε_{cs} using the nominated figures is ± 30%. The benefit of obtaining more-accurate results should be assessed before embarking on time-consuming and costly methods of data collection and calculation.

2.1.9 Creep of concrete

Creep of concrete is defined as the time-dependent increase in strain under sustained loading. The basic creep coefficient is expressed as the ratio of the ultimate creep strain to the elastic strain of a standard specimen initially loaded at 28 days and maintained under a constant stress of 0.4 f'_c. For the practical calculation of the creep of a member, the basic creep coefficient is modified for the effects of member size, exposure environment and the maturity of the concrete at the time of loading.

In the absence of specific data for local concrete, the designer may use the average values for the basic creep coefficient and modification factors given in the tables and graphs in the Standard. The suggested accuracy of creep calculations based on this average data is ±30%.

The creep under constant stress as determined above is known as pure creep. Practical examples of pure creep include creep due to prestress and sustained or dead load on uncracked concrete, such as axial shortening of concrete columns and loadbearing walls of buildings.

However, where stresses are induced by movements such as settlement or shrinkage, the initial stress caused by the induced strain is reduced by creep. This loss of stress is known as relaxation.

2.2 REINFORCEMENT

2.2.1 General

Reinforcement (reinforcing steel) is defined by the Standard in Clause 1.6.3.68 as *'steel bar, wire or mesh but not tendons'*. This definition precludes the use of fibres (steel and other types), non-metallic reinforcement and non-tensioned prestressing strand, bars and wires if the structure is to comply with the Standard.

The Standard also requires (see Clause 17.2.1.1) that reinforcement be deformed bars or mesh (welded wire fabric of either plain or deformed wire) although plain bars or wire may be used for fitments.

The following observations on reinforcement relate to the requirements set out in AS/NZS 4671 [2.7].

2.2.2 Shape

Reinforcing bars can be either plain, deformed ribbed or deformed indented. The shapes are designated by the letters R (Round), D (Deformed ribbed) and I (deformed Indented) respectively.

Generally, only deformed ribbed bars will meet the intention of the requirement in AS 3600 that reinforcement be deformed. However, AS/NZS 4671 contains provisions outlining a test method to measure the bond performance of indented bars or ribbed bars with ribs not meeting the specification set out in that standard.

2.2.3 Strength

Strength grade is represented by the numerical value of the lower characteristic yield stress, 250, 300, and 500 MPa. Reinforcing steel with a strength grade above 250 MPa is also required to comply with the specification of an upper characteristic yield stress.

2.2.4 Ductility class

The three classes of ductility are designated L, N and E for low, normal and earthquake respectively. Ductility Class E has been especially formulated for New Zealand and is not manufactured or available in Australia.

AS 3600 imposes a number of limitations on the use of Ductility Class L reinforcement. These reinforcing materials may be used as main or secondary reinforcement in the form of welded wire mesh, or as wire, bar and mesh in fitments; but are not permitted '*in any situation where the reinforcement is required to undergo large plastic deformation under strength limit state conditions*' (see Clause 1.1.2(c)(ii)). Importantly it also states (Clause 17.2.1.1) that '*Ductility Class L reinforcement shall not be substituted for Ductility Class N reinforcement unless the structure is redesigned*'.

The use of Ductility Class L reinforcement is further limited by other clauses in AS 3600. For example, where Ductility Class L reinforcement is used and where the design incorporates moment redistribution, then the designer has to undertake an analysis to show that there is adequate rotation capacity in critical moment regions to allow the assumed redistribution to take place. There is also a different value for the capacity reduction factor ϕ throughout the Standard where Ductility Class L reinforcement is used.

For further background as to the reasons for the restrictions on the use of Ductility Class L reinforcement, refer to the national seminars on AS 3600—2009 [2.8].

2.2.5 Size

The common sizes of bar available in Australia of the various grades and classes are shown in Tables 2.3 and 2.4.

2.2.6 Weldability

Reinforcement conforming to AS/NZS 4671 is weldable. Depending on the manufacturing process used and the chemical composition of the steel, the requirements for welding may differ and may be more or less stringent than requirements for other reinforcement complying with that Standard. Designers should consult the steel producer's literature for specific advice. Any structural welding of reinforcing steel should comply with AS/NZS 1554.3 [2.9] and be carried out by qualified operators. More-detailed information and guidance is provided in the WTIA Technical Note 1 [2.10]. Welding of galvanised reinforcement needs care and should be avoided if possible due to possible damage to the coating.

Locational tack-welds can be used for pre-assembly of reinforcement cages in lieu of tying at bar intersections. They may be smaller than tack welds as defined in AS/NZS 1554.4 and are (currently) not covered by it. They should be performed by trained personnel and should be executed in a manner that does not cause notching or reduce the cross sectional area of the intersecting bars. Where reinforcement cages are to be lifted care is required to ensure the welds are adequate to support the weight of the cages.

TABLE 2.3 Nominal values for hot-rolled deformed bars of grade D500N

Size	Cross-sectional area (mm²)	Mass per metre length (kg/m)
N10	78.5	0.617
N12	113	0.890
N16	201	1.580
N20	314	2.470
N24	452	3.550
N28	616	4.830
N32	804	6.310
N36	1020	7.990
N40	1260	9.860

Notes:

— These normal-ductility bars are used typically in beams, slabs as flexural reinforcement and in columns and walls as compression reinforcement.

— This Table includes sizes outside AS/NZS 4671.

— N10 bars are not available in all States and Territories.

— N40 bars may be available only on special order for larger quantities.

— Larger diameter fitments are made with N12, N16 and N20 bar as required.

TABLE 2.4 Nominal values for high-strength deformed bars of grade D500L

Size	Cross-sectional area (mm²)	Mass per metre length (kg/m)
L4	12.6	0.099
L5	17.7	0.139
L6	28.3	0.222
L7	35.8	0.281
L8	45.4	0.356
L9	57.4	0.451
L10	70.9	0.556
L11	89.1	0.699
L12	111.2	0.873

Notes:

— These low-ductility bars (sometimes known as wires) are used commonly as fitments in beams and columns generally using L6, L8, and L10 sizes. The other sizes may not be readily available.

— Larger size fitments are usually made from N bar as in Table 2.3.

TABLE 2.5 Nominal values for hot-rolled round bars of grade R250N

Size	Cross-sectional area (mm²)	Mass per metre length (kg/m)
R6.5	30	0.267
R10	80	0.632
R12	110	0.910
R16	200	1.619
R20	310	2.528
R24	450	3.640

Notes:

— The shaded bars, R6.5 and R10, are used for fitments.

— R12 to R24 bars are generally used only for dowel bars.

— For dowel bars larger than R24, check with supplier.

Note that reinforcement manufactured overseas may not conform to AS/NZS 4671, as it may have a higher carbon equivalent content. Some overseas sources, however, can supply complying reinforcement. Designers should consult the Australian Certification Authority for Reinforcing (ACRS) for details of those suppliers.

Generally, welding of reinforcement to comply with AS/NZS 4671 will require the use of:

- hydrogen-controlled electrodes;
- special precautions in adverse conditions, eg wet weather, temperatures ≤ 0°C;
- preheating when bars over 25 mm diameter are being welded.

Note the limitation on the location of welds in a bar that has been bent and re-straightened specified in AS 3600 (Clause 13.2.1(f)), ie it shall not be welded closer than $3d_b$ to the area that has been bent and re-straightened.

2.2.7 Bending and re-bending reinforcement

AS/NZS 4671 specifies for bars of diameter ≤16 mm a 90° bend and rebend test and for bars ≥ 20 mm a 180° bend test. It is thought that these requirements will ensure that bars likely to be restraightened in the field, ie with d≤16 mm, can be safely re-bent.

2.2.8 Mesh

Meshes commonly available in Australia are shown in Table 2.6.

TABLE 2.6 Meshes commonly available in Australia

Mesh type and reference number	Longitudinal bars No. x dia. (mm)	Longitudinal bars Pitch (mm)	Cross bars No. x dia. (mm)	Cross bars Pitch (mm)	Mass of 6-m x 2.4-m sheet (kg)	Cross-sectional area/m width Longitudinal bars (mm²/m)	Cross-sectional area/m width Cross bars (mm²/m)
	Rectangular						
RL 1218	25 x 11.90	100	30 x 7.60	200	157	1112	227
RL 1118	25 x 10.65	100	30 x 7.60	200	131	891	227
RL 1018	25 x 9.50	100	30 x 7.60	200	109	709	227
RL 918	25 x 8.55	100	30 x 7.60	200	93	574	227
RL 818	25 x 7.60	100	30 x 7.60	200	79	454	227
RL 718	25 x 6.75	100	30 x 7.60	200	68	358	227
	Square, with edge side-lapping bars						
SL 102	10 x 9.5 + 4 x 6.75	200 100	30 x 9.5	200	80	354	354
SL 92	10 x 8.6 + 4 x 6.0	200 100	30 x 8.6	200	66	290	290
SL 82	10 x 7.6 + 4 x 6.0	200 100	30 x 7.6	200	52	227	227
SL 72	10 x 6.75 + 4 x 5.0	200 100	30 x 6.75	200	41	179	179
SL 62	10 x 6.0 + 4 x 5.0	200 100	30 x 6.0	200	33	141	141
	Square, without edge side-lapping bars						
SL 81	25 x 7.6	100	60 x 7.6	100	105	454	454
	Trench meshes						
L12TM	*n* x 11.9	100	20 x 5.0	300	NA	1112	65
L11TM	*n* x 10.7	100	20 x 5.0	300	NA	899	65
L8TM	*n* x 7.6	100	20 x 5.0	300	NA	454	65

Notes:

— The edge bar on SL meshes may be replaced by smaller diameter edge bars of equal or greater total cross-sectional area provided the smaller bars meet the minimum ductility requirements of the bar to be replaced.

— Purpose-made mesh can be specified for large projects but designers should first check its availability from reinforcement suppliers.

— SL 52 is also usually available along with SL 53 and SL 63 which are available only in WA.

— Currently most meshes are made from Ductility Class L wire although normal ductility meshes may be available on special order.

2.3 STRESS DEVELOPMENT

2.3.1 General

The rules for stress development are given in AS 3600 Clause 13.1, the data and tables following are based on that information.

Development lengths and lapped splice lengths differ depending on whether the reinforcement is in tension or compression.

2.3.2 Development length for bars in tension

AS 3600 gives the option of a two-tier approach for determining the development length in tension. Either it can be taken as the *Basic development length* or, if desired, that length can be reduced as in the procedure given for the *Refined development length*.

For most designs, the basic development length will be used.

For bars in tension, the basic development length, $L_{sy.tb}$, and the refined development length, is multiplied by:

1.5 for epoxy-coated bars;

1.5 for all plain bars;

1.3 when lightweight concrete is used;

1.3 for all structural elements built with slip forms.

Tables 2.7 and 2.8 give development lengths for deformed bars in the various situations as detailed in each table. The lengths are based on the formula provided in AS 3600 Clause 13.1.2.2, ie:

$$L_{sy.tb} = 0.5\,k_1 k_3 f_{sy} d_b / (k_2 \sqrt{f'_c}) \geq 29 k_1 d_b$$

The values are rounded off to the nearest 10 mm. The values used for the factors k_1, k_2 and k_3 are shown at the top of each table.

The factor k_1 is to allow for the settlement of wet concrete and accumulation of bleed water under the bar in deep sections, which reduces the value of the bond strength. The factor k_2 accounts for the reduction in the average ultimate bond strength as the bar diameter increases.

The factor k_3 depends on the confining effects of the concrete surrounding the bar. The value of c_d used to calculate the factor k_3, and to produce Tables 2.7 and 2.8, is a dimension (mm) derived from the clear spacing between adjacent parallel bars (horizontally) and critical covers to the bar under consideration as shown in Figure 2.2. (Note the cover is to the main bar, ie the bar being anchored.)

Development lengths for bars developing a tensile stress, σ_{st}, less than f_{sy} can be calculated from the formula:

$$L_{st} = L_{sy.t}\,\sigma_{st} / f_{sy} \geq 12 d_b$$

Except that for slabs, the minimum lengths given in AS 3600 Clause 9.1.3.1(a) (ii) may be used where appropriate.

For wide elements such as band beams, slabs, walls and blade columns, where the bars being lapped are in the plane of the element or member, the tensile lap lengths for either contact or non-contact splices can be determined by multiplying the development lengths by 1.25. A lower value of 1.0 is possible where the area of steel provided is at least twice that required and less than 50% of bars are lapped. Refer to AS 3600 Clause 13.2.2.

For narrow elements such as the webs of beams or columns where the bars are in contact or where there is less than $3d_b$ between the bars, the tensile lap length is the development length multiplied by 1.25. Where the bars are further apart than $3d_b$ then additional calculations will be required. Refer to AS 3600 Clause 13.2.2.

For splices in tension tie members, only welded or mechanical splices are allowed.

2.3.3 Reducing tensile development length by standard hooks and cogs

By definition (AS 3600 Clause 13.1.2.7), the term cog is a 90° bend in a bar while a standard hook can be either a 135° or a 180° bend. The length of bar required to physically make each standard hook (which should be specified) is given in Table 2.9. The overall dimensions of hooks and cogs are given in Table 2.10.

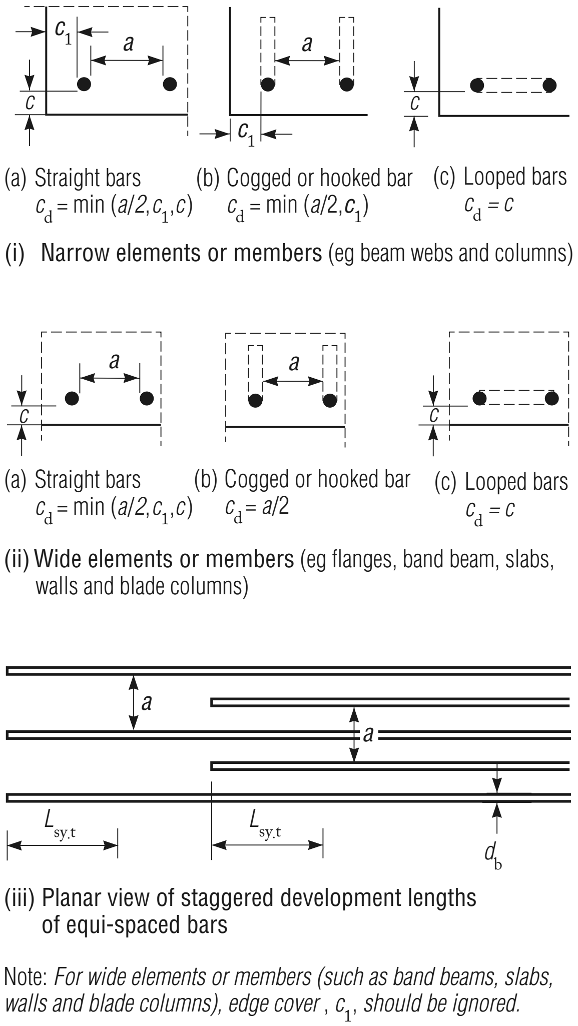
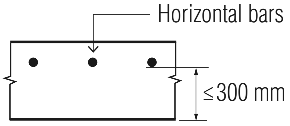


Figure 2.2 Values of c_d (after AS 3600)

Although hooks and cogs reduce the tensile development length as shown in Figure 2.3, they cause congestion of reinforcement in critical areas such as beam/column joints and ends of simply-supported beams. They can also become a source of corrosion if they are allowed to encroach into the required cover. Straight bars are easier to fix and ensure that the required cover is maintained. Where a short development length is required, an alternative to using standard hooks is to use smaller diameter bars and/or higher strength concrete.

Tensile stresses are also generated in the concrete in the plane of hooks because of bearing on the concrete on the inside of the hook when the bar is fully stressed under load. Hooks should not be used in sections thinner than about 12 bar diameters or as top bars in slabs, to avoid splitting or spalling of the concrete cover.

TABLE 2.7 Basic development lengths $L_{sy.t}$ for Grade D500N bars in tension and where there is ≤300 mm of concrete under the bar



$k_1 = 1.0$
$k_2 = (132 - d_b)/100$
$k_3 = 1.0 - 0.15(c_d - d_b)/d_b$ (within limits $0.7 \leq k_3 \leq 1.0$)
$f_{sy} = 500$ MPa

Concrete strength f'_c (MPa)	c_d	N12	N16	N20	N24	N28	N32	N36
20	30	430	670	920	1200	1490	1790	2100
	35	400	630	890	1160	1450	1760	2100
	40	390	600	850	1120	1410	1720	2060
	45	390	560	810	1080	1370	1680	2020
	50	390	540	770	1040	1330	1640	1970
	55	390	540	740	1000	1290	1600	1930
	60	390	540	700	960	1250	1550	1890
	65	390	540	700	920	1210	1510	1840
	70	390	540	700	890	1170	1470	1800
	75	390	540	700	870	1130	1430	1760
	80	390	540	700	870	1090	1390	1710
	85	390	540	700	870	1050	1340	1670
	90	390	540	700	870	1050	1300	1620
	95	390	540	700	870	1050	1260	1580
	100	390	540	700	870	1050	1250	1540
25	30	390	600	830	1070	1330	1600	1880
	35	360	570	790	1030	1300	1580	1880
	40	350	530	760	1000	1260	1540	1840
	45	350	500	730	970	1220	1500	1800
	50	350	480	690	930	1190	1470	1770
	55	350	480	660	900	1150	1430	1730
	60	350	480	630	860	1120	1390	1690
	65	350	480	630	830	1080	1350	1650
	70	350	480	630	790	1040	1320	1610
	75	350	480	630	780	1010	1280	1570
	80	350	480	630	780	970	1240	1530
	85	350	480	630	780	940	1200	1490
	90	350	480	630	780	940	1170	1450
	95	350	480	630	780	940	1130	1410
	100	350	480	630	780	940	1120	1380
32	30	350	530	730	950	1180	1410	1660
	35	350	500	700	910	1150	1390	1660
	40	350	470	670	880	1110	1360	1630
	45	350	460	640	850	1080	1330	1600
	50	350	460	610	820	1050	1290	1560
	55	350	460	580	790	1020	1260	1530
	60	350	460	580	760	990	1230	1490
	65	350	460	580	730	950	1200	1460
	70	350	460	580	700	920	1160	1420
	75	350	460	580	700	890	1130	1390
	80	350	460	580	700	860	1100	1350
	85	350	460	580	700	830	1060	1320
	90	350	460	580	700	830	1030	1280
	95	350	460	580	700	830	1000	1250
	100	350	460	580	700	830	990	1220
40	30	350	470	650	850	1050	1260	1480
	35	350	460	630	820	1020	1250	1480
	40	350	460	600	790	1000	1220	1460
	45	350	460	580	760	970	1190	1430
	50	350	460	580	740	940	1160	1400
	55	350	460	580	710	910	1130	1360
	60	350	460	580	700	880	1100	1330
	65	350	460	580	700	850	1070	1300
	70	350	460	580	700	820	1040	1270
	75	350	460	580	700	810	1010	1240
	80	350	460	580	700	810	980	1210
	85	350	460	580	700	810	950	1180
	90	350	460	580	700	810	930	1150
	95	350	460	580	700	810	930	1120
	100	350	460	580	700	810	930	1090
50	30	350	460	580	760	940	1130	1330
	35	350	460	580	730	920	1120	1330
	40	350	460	580	710	890	1090	1300
	45	350	460	580	700	870	1060	1280
	50	350	460	580	700	840	1040	1250
	55	350	460	580	700	810	1010	1220
	60	350	460	580	700	810	980	1190
	65	350	460	580	700	810	960	1170
	70	350	460	580	700	810	930	1140
	75	350	460	580	700	810	930	1110
	80	350	460	580	700	810	930	1080
	85	350	460	580	700	810	930	1060
	90	350	460	580	700	810	930	1040
	95	350	460	580	700	810	930	1040
	100	350	460	580	700	810	930	1040
≥ 65	30	350	460	580	700	830	990	1160
	35	350	460	580	700	810	980	1160
	40	350	460	580	700	810	960	1140
	45	350	460	580	700	810	930	1120
	50	350	460	580	700	810	930	1090
	55	350	460	580	700	810	930	1070
	60	350	460	580	700	810	930	1050
	65	350	460	580	700	810	930	1040
	70	350	460	580	700	810	930	1040
	75	350	460	580	700	810	930	1040
	80	350	460	580	700	810	930	1040
	85	350	460	580	700	810	930	1040
	90	350	460	580	700	810	930	1040
	95	350	460	580	700	810	930	1040
	100	350	460	580	700	810	930	1040

Notes:

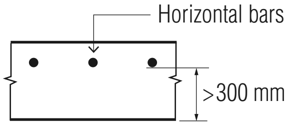
—$k_1 = 1.0$

—The basic development lengths have been calculated using the nominal areas as per AS/NZS 4761 and have been rounded (generally to the nearest 10 mm) within the accuracy of normal design limits.

—c_d = smaller of the cover to the deformed bar or 1/2 clear distance to next parallel bar.

—For concrete strength greater than 65 MPa use figures for 65 MPa.

TABLE 2.8 Basic development lengths $L_{sy.t}$ for Grade D500N bars in tension and where there is >300 mm of concrete under the bar



k_1 = 1.3
$k_2 = (132 - d_b)/100$
$k_3 = 1.0 - 0.15(c_d - d_b)/d_b$ (within limits $0.7 \le k_3 \le 1.0$)
f_{sy} = 500 MPa

Concrete strength f'_c (MPa)	c_d	Bar size N12	N16	N20	N24	N28	N32	N36
20	30	560	870	1200	1550	1940	2330	2730
	35	520	820	1150	1500	1880	2290	2730
	40	510	780	1100	1450	1830	2240	2680
	45	510	730	1050	1400	1780	2180	2620
	50	510	700	1010	1350	1730	2130	2570
	55	510	700	960	1300	1670	2070	2510
	60	510	700	910	1250	1620	2020	2450
	65	510	700	910	1200	1570	1970	2400
	70	510	700	910	1150	1520	1910	2340
	75	510	700	910	1130	1460	1860	2280
	80	510	700	910	1130	1410	1800	2230
	85	510	700	910	1130	1370	1750	2170
	90	510	700	910	1130	1370	1690	2110
	95	510	700	910	1130	1370	1640	2060
	100	510	700	910	1130	1370	1630	2000
25	30	500	780	1070	1390	1730	2080	2440
	35	460	740	1030	1350	1680	2050	2440
	40	460	690	990	1300	1640	2000	2400
	45	460	650	940	1250	1590	1950	2350
	50	460	630	900	1210	1540	1900	2300
	55	460	630	860	1160	1500	1860	2240
	60	460	630	810	1120	1450	1810	2190
	65	460	630	810	1070	1400	1760	2140
	70	460	630	810	1030	1360	1710	2090
	75	460	630	810	1010	1310	1660	2040
	80	460	630	810	1010	1260	1610	1990
	85	460	630	810	1010	1230	1560	1940
	90	460	630	810	1010	1230	1510	1890
	95	460	630	810	1010	1230	1470	1840
	100	460	630	810	1010	1230	1460	1790
32	30	450	690	950	1230	1530	1840	2150
	35	450	650	910	1190	1490	1810	2150
	40	450	610	870	1150	1450	1770	2120
	45	450	600	830	1110	1410	1730	2070
	50	450	600	800	1070	1360	1680	2030
	55	450	600	760	1030	1320	1640	1980
	60	450	600	750	990	1280	1600	1940
	65	450	600	750	950	1240	1550	1890
	70	450	600	750	910	1200	1510	1850
	75	450	600	750	900	1160	1470	1800
	80	450	600	750	900	1120	1420	1760
	85	450	600	750	900	1080	1380	1710
	90	450	600	750	900	1080	1340	1670
	95	450	600	750	900	1080	1300	1620
	100	450	600	750	900	1080	1290	1580
40	30	450	620	850	1100	1370	1640	1930
	35	450	600	810	1060	1330	1620	1930
	40	450	600	780	1030	1290	1580	1890
	45	450	600	750	990	1260	1540	1850
	50	450	600	750	960	1220	1510	1810
	55	450	600	750	920	1180	1470	1770
	60	450	600	750	900	1150	1430	1730
	65	450	600	750	900	1110	1390	1690
	70	450	600	750	900	1070	1350	1650
	75	450	600	750	900	1060	1310	1610
	80	450	600	750	900	1060	1270	1570
	85	450	600	750	900	1060	1240	1530
	90	450	600	750	900	1060	1210	1490
	95	450	600	750	900	1060	1210	1450
	100	450	600	750	900	1060	1210	1410
50	30	450	600	760	980	1220	1470	1720
	35	450	600	750	950	1190	1450	1720
	40	450	600	750	920	1160	1420	1690
	45	450	600	750	900	1120	1380	1660
	50	450	600	750	900	1090	1350	1620
	55	450	600	750	900	1060	1310	1590
	60	450	600	750	900	1060	1280	1550
	65	450	600	750	900	1060	1240	1520
	70	450	600	750	900	1060	1210	1480
	75	450	600	750	900	1060	1210	1440
	80	450	600	750	900	1060	1210	1410
	85	450	600	750	900	1060	1210	1370
	90	450	600	750	900	1060	1210	1360
	95	450	600	750	900	1060	1210	1360
	100	450	600	750	900	1060	1210	1360
≥ 65	30	450	600	750	900	1070	1290	1510
	35	450	600	750	900	1060	1270	1510
	40	450	600	750	900	1060	1240	1490
	45	450	600	750	900	1060	1210	1450
	50	450	600	750	900	1060	1210	1420
	55	450	600	750	900	1060	1210	1390
	60	450	600	750	900	1060	1210	1360
	65	450	600	750	900	1060	1210	1360
	70	450	600	750	900	1060	1210	1360
	75	450	600	750	900	1060	1210	1360
	80	450	600	750	900	1060	1210	1360
	85	450	600	750	900	1060	1210	1360
	90	450	600	750	900	1060	1210	1360
	95	450	600	750	900	1060	1210	1360
	100	450	600	750	900	1060	1210	1360

Notes:

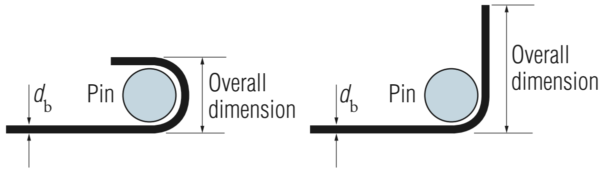
—k_1 = 1.3

—The basic development lengths have been calculated using the nominal areas as per AS/NZS 4761 and have been rounded (generally to the nearest 10 mm) within the accuracy of normal design limits.

—c_d = smaller of the cover to the deformed bar or 1/2 clear distance to next parallel bar.

—For concrete strength greater than 65 MPa use figures for 65 MPa.

TABLE 2.9 Overall dimensions (mm) of 180° hooks and 90° cogs



Pin diameter (mm)	Bar diameter, d_b (mm) 6	10	12	16	20	24	28	32	36
180° hooks									
$3d_b$	30	50	60	*	*	*	*	*	*
$4d_b$	40	60	70	100	120	140	170	190	220
$5d_b$	40	70	80	110	140	170	200	220	250
$6d_b$	50	80	100	130	160	190	220	260	290
$8d_b$	60	100	120	160	200	240	280	320	360
90° cogs									
$3d_b$	120	140	160	*	*	*	*	*	*
$4d_b$	130	150	170	200	240	280	330	370	420
$5d_b$	130	160	180	210	260	310	360	400	450
$6d_b$	140	180	200	240	290	340	400	450	510
$8d_b$	160	200	230	280	340	400	470	530	600

* Not to be used

Notes:

— $5d_b$ pin is the one most commonly used.

Cogs are commonly used with top reinforcement in slabs where the slab sits on beams and the cogged bars sit over the beam bars as shown in Figure 2.4.

For fitments with cogs, acting as shear reinforcement, AS 3600 Clause 8.2.12.4 requires that there is 50 mm or more of concrete cover over the cog.

AS 3600 also covers the development lengths of plain bars and headed reinforcement in tension (see Clauses 13.1.3 and 13.1.4 respectively).

2.3.4 Development length for bars in compression

Development lengths for bars in compression are less than those for bars in tension because the detrimental effects of tensile cracking are less and the end bearing of the bar is beneficial. Again, AS 3600 allows a two-tier approach with a *Basic development length*, which can be modified as in the *Refined development length*. For most designs, the basic development length will be used.

While no specific comment is made about the effect of cover, bar spacing and confinement by fitments, the general rules for cover and bar spacing (for placing and compacting concrete) given in Sections 4 and 17 of the Standard should be followed. The importance of confinement by fitments is highlighted by the

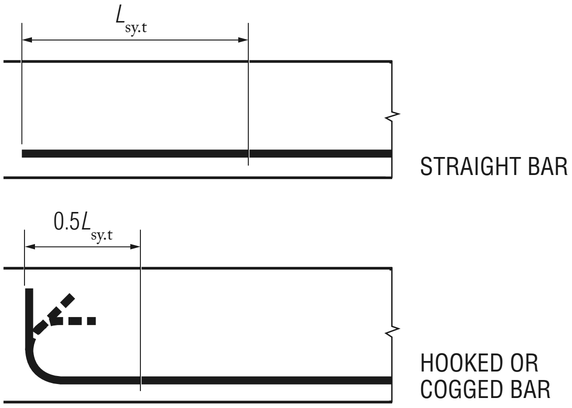


Figure 2.3 Reduced development length using hooks or cogs

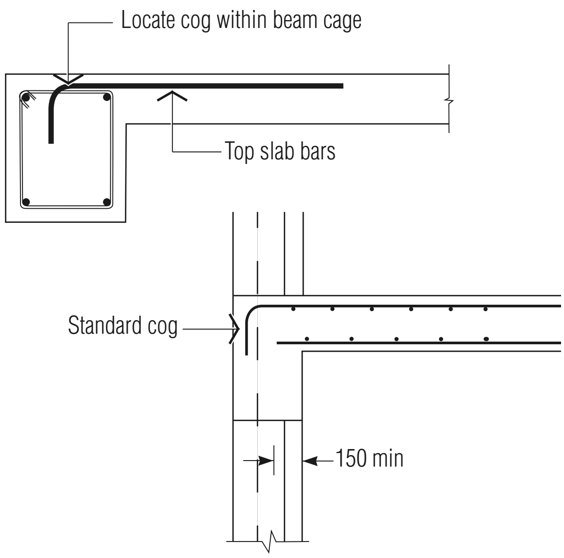


Figure 2.4 Cogs with slabs and beams

requirements for fitments around compression lap splices in AS 3600 Clause 13.2.4.

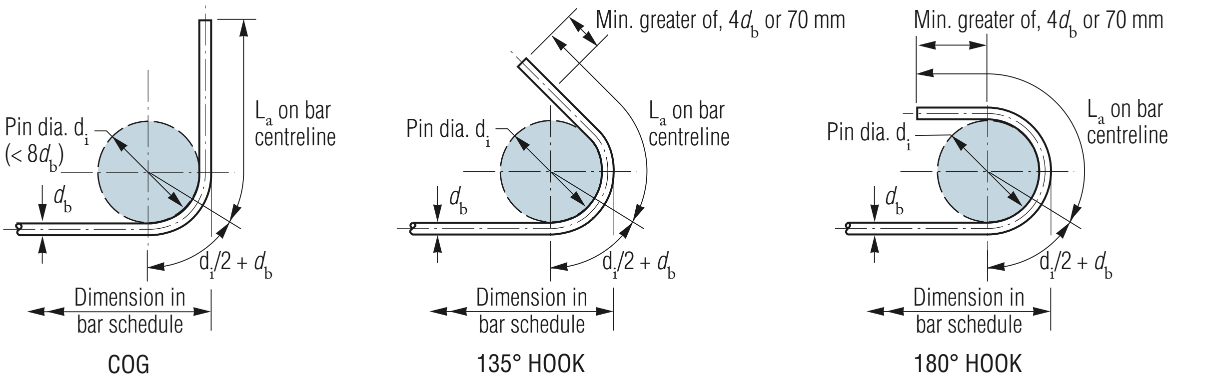
The development length, $L_{sy.c}$, shall be taken as the basic development length of a deformed bar in compression, $L_{sy.cb}$, calculated from:

$$L_{sy.cb} = \frac{0.22\, f_{sy}}{\sqrt{f'_c}}\, d_b \geq 0.0435\, f_{sy} d_b$$

or 200 mm, whichever is the greater.

In compression, the basic development length in the above equation is largely independent of the concrete strength as, generally, the minimum length will apply. However, all values of the basic development length for different concrete grades are shown in Table 2.10. A refined development length equal to 0.75 of the basic development length can be used subject to complying with AS 3600 Clause 13.1.5.3 but is not shown in Table 2.10.

TABLE 2.10 Minimum lengths for hooks and cogs (mm)



Type of bar	**Min. pin diameter** (mm)	**Bar diameter**, d_b (mm) 6	10	12	16	20	24	28	32	36
Fitments: D500L and R250N bars	$3d_b$	100	110	120	*	*	*	*	*	*
D500N bars	$4d_b$	110	130	140	170	200	230	270	300	340
Reinforcement other than those below	$5d_b$	120	140	160	180	220	260	300	340	380
Bends designed to be straightened or subsequently rebent	$4d_b$	110	130	140	170	*	*	*	*	*
	$5d_b$	*	*	*	*	220	260	*	*	*
	$6d_b$	*	*	*	*	*	*	330	380	430
Bends in reinforcement epoxy-coated or galvanised either before or after bending	$5d_b$	120	140	160	180	*	*	*	*	*
	$8d_b$	*	*	*	*	290	340	390	440	500

* Not to be used

Notes:

— $5d_b$ pin is the one most commonly used.

— The overall sizes are nominal. No allowance for spring-back is included, nor is the real oversize diameter of a deformed bar taken into account.

— 135° on fitments is the most common hook used, which has the same internal diameter and length as 180° hook.

Hooks and cogs cannot be used to reduce the development length in compression. For example, with the use of cogged starter bars in a footing, the overall depth of the footing must allow for the development length of the starter bar in compression, the cog, the bottom reinforcement and the bottom cover as shown in Figure 2.5.

AS 3600, also has rules for the development lengths of plain bars and bundled bars in compression (see Clauses 13.1.6 and 13.1.7).

2.3.6 Splicing of reinforcement

[a] General As reinforcing bars come in lengths up to about 12 m maximum, splicing is required for most concrete elements during construction, including across construction joints. This is a necessary part of the detailing of the reinforcement for any project.

AS 3600 Clause 13.2.1 requires that splices are to be made only as permitted in the drawings or specification. Therefore, the designer has the

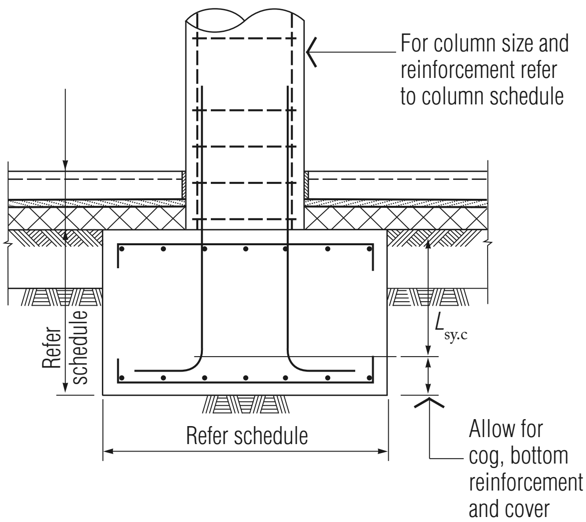


Figure 2.5 Development length of column starter bars in compression

responsibility to detail where and how splices are or can be made. It is important to specify which splices are full strength splices and which are not; otherwise full strength splices may be detailed by the reinforcement scheduler for all locations. The designer, knowing how the structure works, should do this detailing, eg crack control reinforcement may not need full strength splices.

Splices in tension members can be only welded or mechanical splices.

[b] Lapped splices in tension For webs of beams where spliced bars are in contact or spaced less than $3d_b$ apart, the lap splice length is the development length multiplied by the factor k_7 (determined from AS 3600 Clause 13.2.2) and which is generally 1.25 times the development length $L_{sy.t}$.

Bars spliced by non-contact lap splices in flexural members, eg slabs and flanges of beams, spaced transversely further apart than $3d_b$ shall have a splice length not less than the larger of $k_7 L_{sy.t}$ (generally $1.25L_{sy.t}$) and $L_{sy.t} + 1.5s_b$, where s_b is the clear distance between bars of the non-contact lapped splice (mm) as shown in Figure 2.6. The length of the lapped splice can be calculated by multiplying the development lengths in Tables 2.7 and 2.8 by the factor k_7 (usually 1.25). However, if s_b does not exceed $3d_b$, then s_b may be taken as zero for calculating $L_{sy.t.lap}$.

Designers should remember that in most designs for bars in tension, bars should not be lapped at the point of maximum tension and good design practice will minimise bars being lapped in high stress areas. An example is top bars in a cantilever beam or slab, which are usually spliced at about the quarter points in the back span, depending on the length of the cantilever span and back span. AS 3600 allows a pro rata reduced development length (and lap splice) where the stress in the bar is less than the yield stress both in tension and compression. For the situation where the stress in the bars is less than $0.5f_{sy}$ and only half the bars are being spliced at the location, k_7 can be taken as 1.0 (see AS 3600 Clause 13.2.2). For tension, there is a minimum development length of $12d_b$ or D, whichever is the greater, for slabs as permitted by AS 3600 Clause 9.1.3.1 (a) (ii).

A lapped splice for welded mesh in tension shall be made so the two outermost cross-bars (spaced at not less than 100 mm or 50 mm apart for plain or deformed bars respectively) of one sheet of mesh overlap the two outermost cross-bars of the sheet being lapped as shown in AS 3600 Figure 13.2.3. The minimum length of the overlap shall be 100 mm. A lapped splice for welded deformed and plain meshes, with no cross-bars within the splice length shall be determined in accordance with AS 3600 Clause 13.2.2.

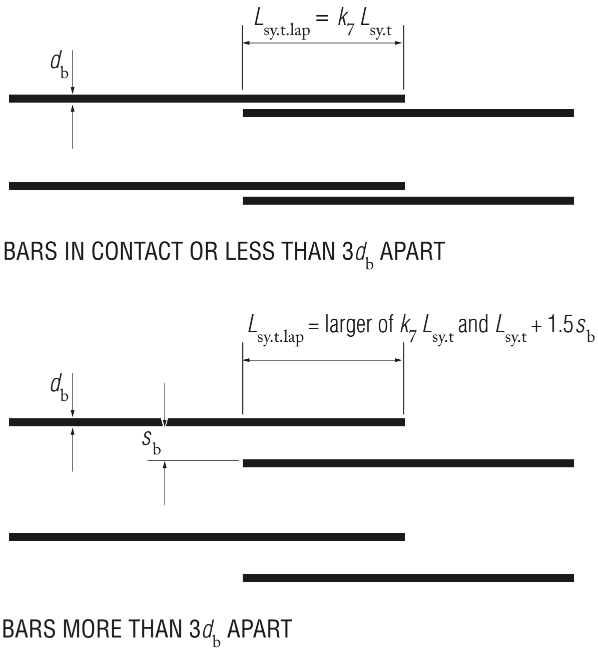


Figure 2.6 The lap-splice length of adjacent bars in tension in webs of beams and in columns (narrow elements)

TABLE 2.11 Basic development lengths $L_{sy.cb}$ and lap-splice lengths (mm) for Grade D500N bars in compression

Concrete strength f'_c (MPa)	**Bar size** N12	N16	N20	N24	N28	N32	N36
20	300	390	490	590	690	790	890
25	260	350	440	530	620	700	790
32	260	350	440	520	610	700	780
40	260	350	440	520	610	700	780
50	260	350	440	520	610	700	780
65	260	350	440	520	610	700	780
80	260	350	440	520	610	700	780
100	260	350	440	520	610	700	780
	Lap-splice length for bars in contact or spaced at less than $3d_b$ apart – development length or $40d_b$ (see AS 3600 Clause 13.2.4 (a))						
	480	640	800	960	1120	1280	1440
	0.8 Concessional value*						
	380	510	640	770	900	1020	1150

* If certain conditions are met (see AS 3600 Clause 13.2.4 (b) and (c) for details).

[c] Lapped splices in compression AS 3600 Clause 13.2.4 (a) requires that the lap-splice length for deformed bars in compression be a minimum of 300 mm and not less than $40d_b$ which are independent of the concrete strength. However, there are two conditions in AS 3600 Clause 13.2.4 which allow the lap splice length to be reduced to 0.8 of the $40d_b$ value. This reduced value of 0.8 has also been included in Table 2.11.

2.4 DETAILING

Structural analysis is only one part of the design process. Good detailing is equally important and requires an understanding of what each bar, fitment or piece of mesh is doing in the structure and what forces it is resisting.

Detailing of reinforcement is the interface between the theoretical design and what can be built in practice on site. There is no point in having the most refined analysis and design, if it cannot be constructed. Detailing also has an impact on durability as poor placement of reinforcement leads to insufficient cover and premature failure. Designers need to be practical in terms of what is readily achievable on site and clear in their drawings and their details to allow their concrete structure to be properly built.

It is important not to expect reinforcement to end up in precisely the position nominated in the documentation. As with all site work, some tolerance must be allowed. In general, ±10 mm is realistic, while ±5 mm is the tightest that should ever be specified. Note that AS 3600 Clause 17.5.3 allows –5 mm +10 mm deviation from the specified position controlled by cover for beams, slabs, columns and walls.

Generally, reinforcement fixing is a three dimensional problem. Lines, dots, cogs, hooks and laps on drawings have real sizes and location in the formwork. Some adjustments of the reinforcement in formwork will be necessary to make it all fit, especially with larger bars. Reinforcement cannot be shifted quickly from its position without bends and cranks. Figure 2.7 illustrates this point.

To gain an understanding of how reinforcement is fixed and the practicalities of work on site, designers should be encouraged to inspect their design work in the field. It should not be assumed that the builder and/or the scheduler will 'work it out', nor that the detailing will just 'happen'. Comprehensive detailing (particularly of complex reinforcement) is vital.

Some examples of poor detailing and suggested improvements are shown in Figure 2.8, 2.9 and 2.10.

Guidance on detailing of reinforcement is provided in *Reinforcement Detailing Handbook*[2.11].

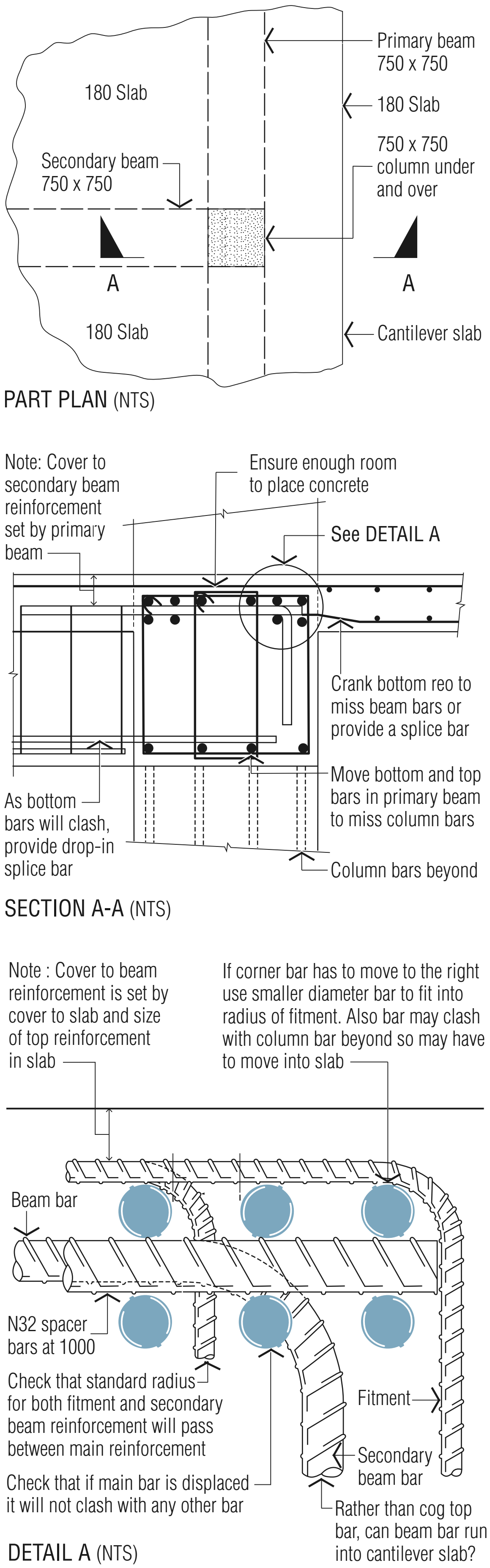
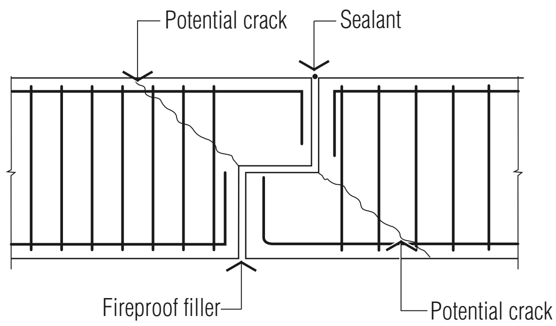
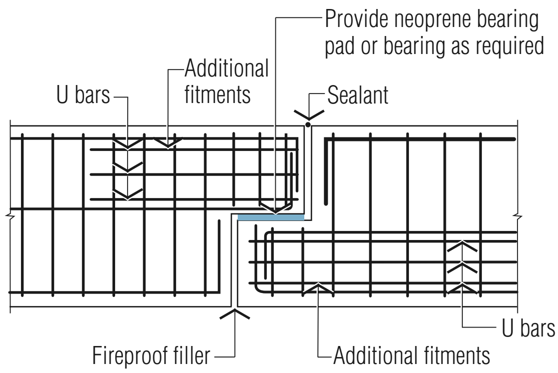


Figure 2.7 What was shown on the drawings and how it fits differently on site



POOR DETAILING

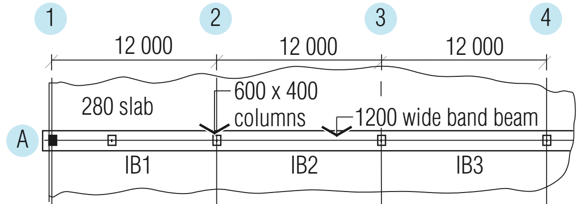
Joint is likely to fail due to diagonal cracking



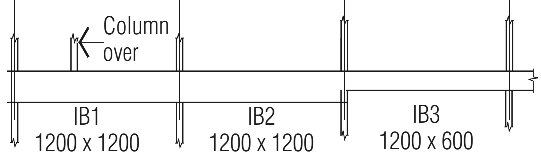
GOOD DETAILING

Note details are indicative only and are subject to final design

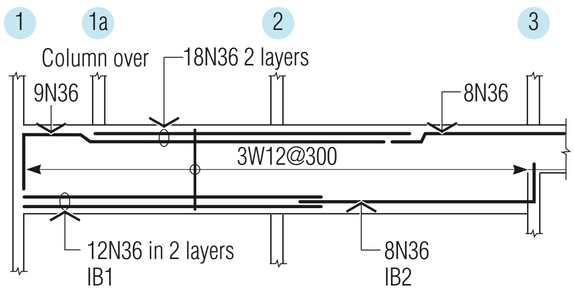
Figure 2.8 Detailing of corbel



PART PLAN

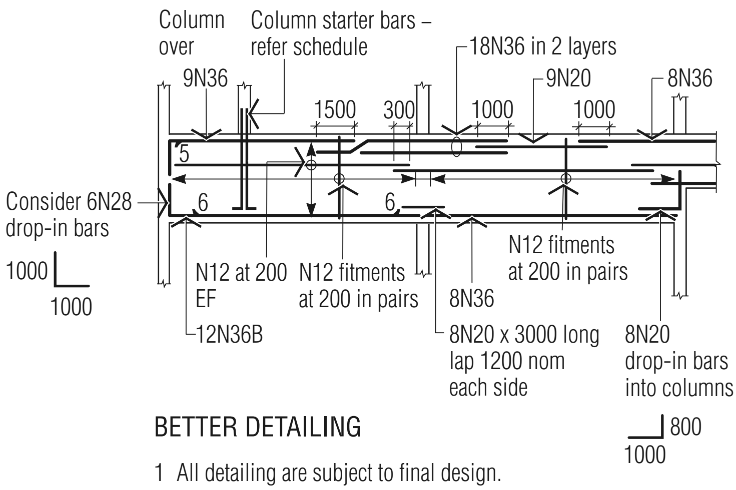


ELEVATION



POOR DETAILING

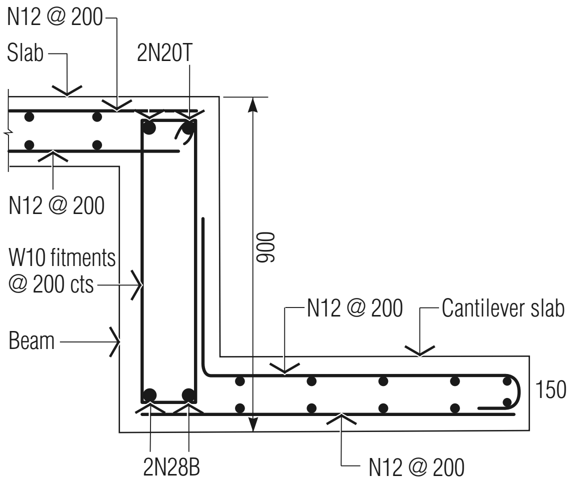
1 Many bars exceed 12 m length in IB1 and IB2.
2 9N36 cogged into column or edge beam and will not fit at grid 1. Consider drop-in bars to match moment into column or reduce number and size of bars.
3 12N36 in 2 layers in bottom of IB1 could be in 1 layer.
4 No side face reinforcement shown.
5 W12 is the wrong designation. Consider N12 fitments at 200 centres in pairs to reduce fixing and to comply with transverse spacing.
6 Starter bars for column at 1a not shown.
7 Cogging 8N36 bottom at grid 3 not required. Suggest 8N20 bars with 4 bars cogged into column.
8 As splice lengths not shown, the scheduler will assume a full splice.



BETTER DETAILING

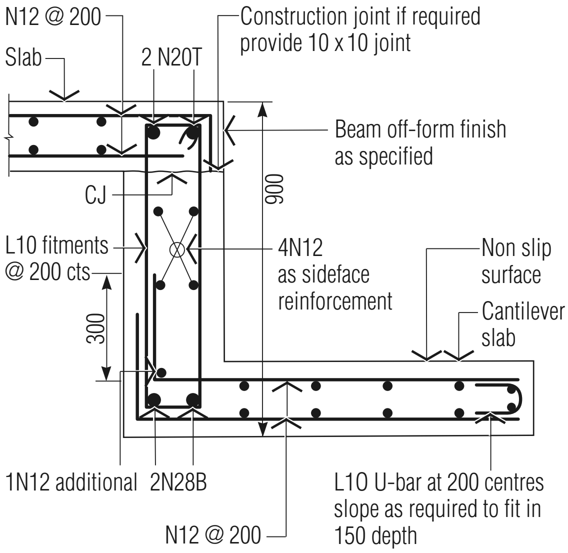
1 All detailing are subject to final design.
2 Splices generally shown so scheduler will not use full strength splices.

Figure 2.9 Detailing of beam



POOR DETAILING

1 N12 top bars to cantilever slab are not properly anchored. Also hooks in 150 cantilever slab difficult to fit in depth.
2 N12 top bars to slab at top of beam not properly anchored.
3 No side face reinforcement to beam.
4 Designation of fitments is incorrect.
5 Can it be cast in one pour?



GOOD DETAILING

1 Note details are indicative only and are subject to final design.

Figure 2.10 Detailing of cantilever slab

REFERENCES

2.1 Sirivivatnanon V *Fitness for Purpose of Residential Slab-on-Ground* Proceedings of Concrete 07, 18–20 October 2007, Adelaide, Concrete Institute of Australia.

2.2 AS 3600 *Concrete structures* Standards Australia, 2009.

2.3 AS 1012 *Methods of testing concrete* Standards Australia.

2.4 AS/NZS 1170 *Structural design actions,* Standards Australia,

Part 1: *Permanent, imposed and other actions*

2.5 *Deformability of concrete structures – basic assumptions* Bulletin D'Information No. 90, Comité Européen du Béton (CEB), 1973.

2.6 BS EN 1992, Eurocode 2 *Design of concrete structures* British Standards Institution, 2004.

2.7 AS/NZS 4671 *Steel reinforcing materials* Standards Australia, 2001.

2.8 Lecture 8, *Steel reinforcement* National Seminar Series AS 3600—2009, CIA, EA and CCAA.

2.9 AS/NZS 1554 *Structural steel welding* Standards Australia

Part 3: *Welding of reinforcing steel*, 2008.

Part 4: *Welding of high strength quenched and tempered steels*, 2010.

2.10 Welding Technology Institute of Australia (WTIA), Technical Note 1, 1996.

2.11 *Reinforcement Detailing Handbook* (Z06), 2nd Ed, Concrete Institute of Australia, 2010.

BLANK PAGE

Chapter 3_Durability and fire resistance

3.1 DESIGN FOR DURABILITY

3.1.1 General

Concrete is one of the most widely used materials in structures and durability is one of its key advantages.

Durability can be defined as the *ability of a concrete structure to resist during its design life the effects of weathering, chemical attack, abrasion and other deteriorating influences (acting on the structure or its members) arising within the concrete, from the environment or from processes being carried out inside the structure*. Although not specifically mentioned in this definition, deterioration due to the corrosion of reinforcement, tendons or other inserts cast into the concrete is an important aspect of durability.

In designing for durability the environment in which the structure is to be built, including micro-climates generated by the structure itself, has to be evaluated. The following also need to be taken into account:

- Aggressive agencies and actions of the processes to be carried out in or around the structure
- The expected wear and deterioration through the intended service life of the structure
- The amount of periodic inspection and maintenance the structure is likely to receive during its working life (particularly external parts exposed to the environment)
- The length of time the concrete structure is expected to be operational without repair
- The difficulty of carrying out repairs and their economic impact.

Durability of concrete is covered in Section 4 of AS 3600 *Concrete structures*[3.1]. The Standard applies to plain, reinforced and prestressed concrete structures and members with a design life of 50 years ± 20%, but notes that:

- *More stringent requirements would be appropriate for structures with a design life in excess of 50 years (eg monumental structures), while some relaxation of the requirements may be acceptable for structures with a design life less than 50 years (eg temporary structures).*
- *Durability is a complex topic and compliance with these requirements may not be sufficient to ensure a durable structure.*

The second point should alert designers to the issues involved in the design and construction of concrete structures for durability and to think about these issues, rather than just meeting the minimum requirements of the Standard.

Nevetheless, structures designed to AS 3600 have generally performed satisfactorily in normal environments and properly designed, proportioned, inspected, placed, finished and cured concrete is capable of providing many years of durable service. The information in this chapter is based on that given in AS 3600, unless noted otherwise.

Guidance in AS 3600 is, however, given for only a limited number of aspects of durability – corrosion of reinforcement (based on the concrete strength and cover for various exposure classifications), aggressive soils, freeze-thaw and abrasion – and is restricted to a limited number of exposure conditions. For specific durability-related issues reference to publications relevant to the specific issue in question is recommended (eg abrasion resistance, acid attack, or corrosion of embedded steel in concrete). Documents such as *Durable Concrete Structures*[3.2], *Performance Criteria for Concrete in Marine Environments*[3.3] and *Guide to Durable Concrete*[3.4] provide sound guidance.

3.1.2 AS 3600 requirements

The 2009 edition of AS 3600 includes some important changes to exposure classifications. Specifically for surfaces of maritime structures in seawater the exposure classification C has been split into:

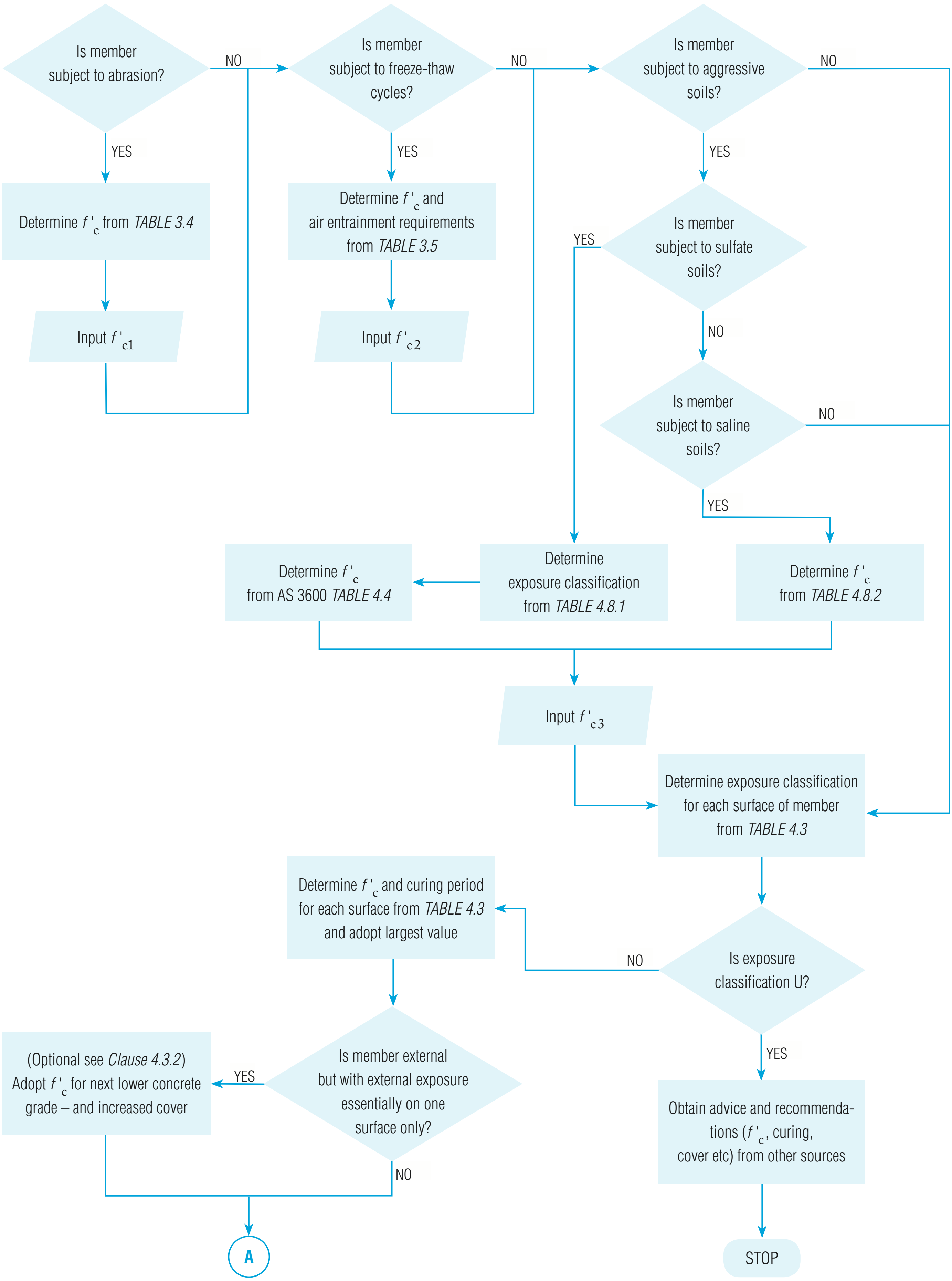
- C1 (spray zone) and
- C2 (tidal/splash zone).

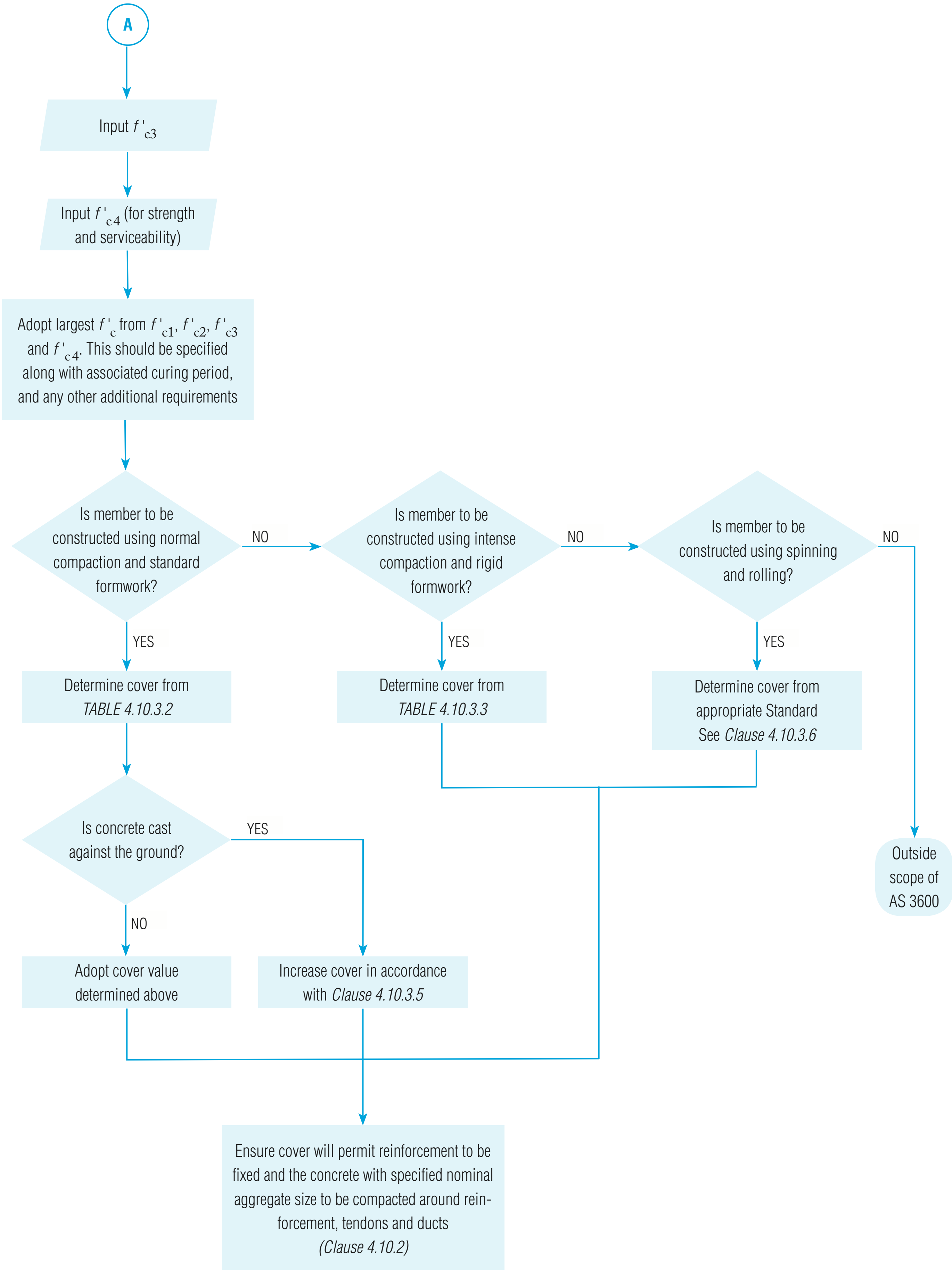
It should be noted that there is now a separate standard guide on maritime structures, AS 4997 *Guidelines for the design of maritime structures*[3.5]. Also, CCAA has published a document on *Chloride Resistance of Concrete*[3.6].

AS 3600 now includes specific guidance for sulfate soils and saline soils. There is considerable background information on aggressive soil conditions. CCAA's Technical Note *Sulfate-resisting Concrete*[3.7] and *Guide to Residential Slabs and Footings in Saline Environments*[3.8] provide background information in this complex area.

Importantly, while durability is a complex topic, for the criteria discussed, AS 3600 essentially manages issues of durability from a compressive strength and cover specification perspective for the particular exposure classification. Concrete mix designs

FLOWCHART 3.1 Designing for durability in accordance with AS 3600




A
Input f'_{c3}
Input f'_{c4} (for strength and serviceability)
Adopt largest f'_{c} from f'_{c1}, f'_{c2}, f'_{c3} and f'_{c4}. This should be specified along with associated curing period, and any other additional requirements
Is member to be constructed using normal compaction and standard formwork?
NO
Is member to be constructed using intense compaction and rigid formwork?
NO
Is member to be constructed using spinning and rolling?
NO
YES
Determine cover from *TABLE 4.10.3.2*
YES
Determine cover from *TABLE 4.10.3.3*
YES
Determine cover from appropriate Standard See *Clause 4.10.3.6*
Is concrete cast against the ground?
YES
NO
Outside scope of AS 3600
Adopt cover value determined above
Increase cover in accordance with *Clause 4.10.3.5*
Ensure cover will permit reinforcement to be fixed and the concrete with specified nominal aggregate size to be compacted around reinforcement, tendons and ducts *(Clause 4.10.2)*

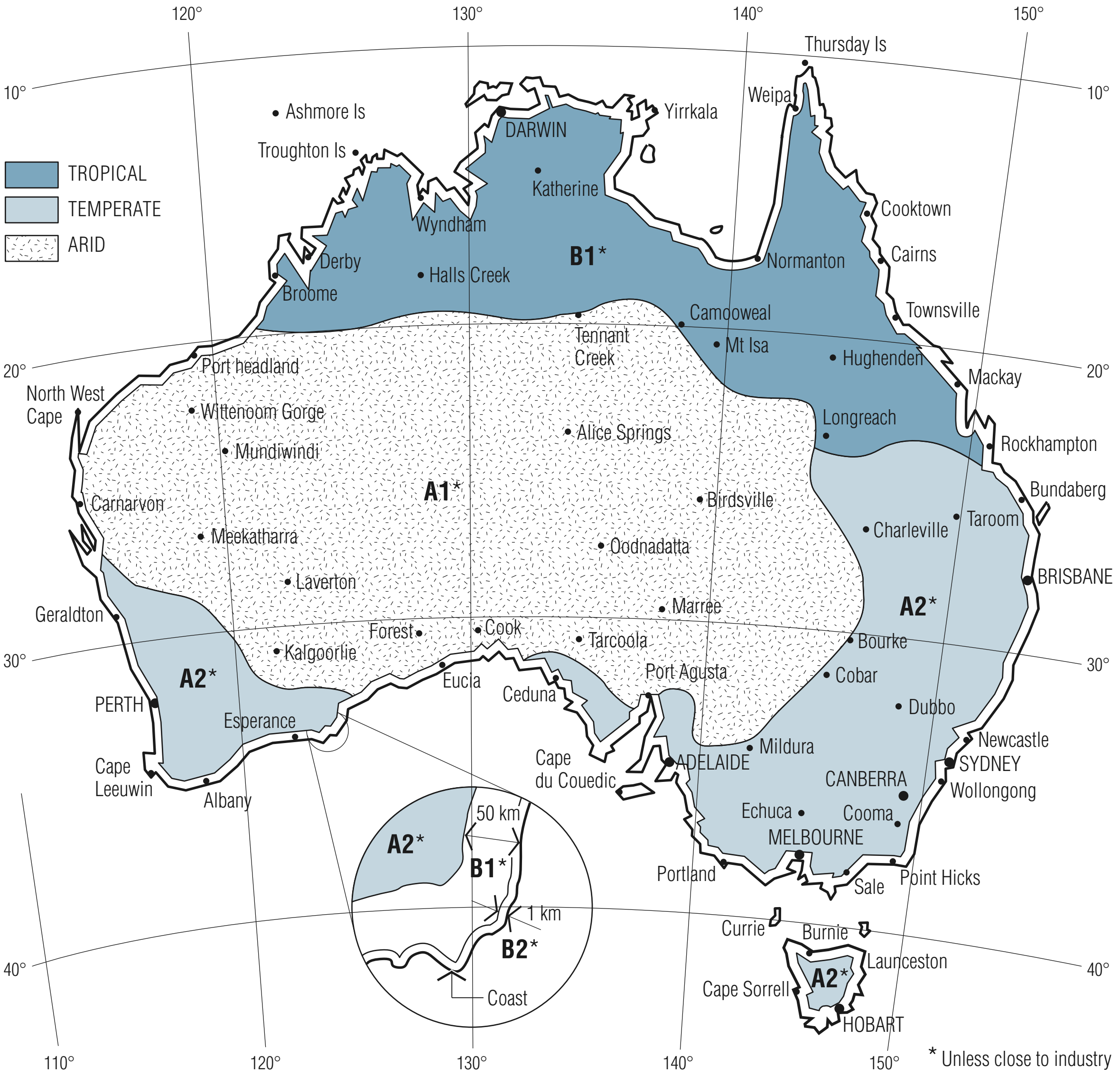


Figure 3.1 Climatic zones and exposure classifications (after AS 3600)

using supplementary cementitious materials (SCM), maximum water-cement ratios, cementitious binder types, etc are not discussed in any detail in AS 3600 except that for exposure classification B2, C1 and C2 special class concrete must be used.

Appendix B of AS 1379 *Specification and supply of concrete*[3.9], lists the various criteria for special class concrete which are additional to or different from those for normal class concrete. As concrete mix design is a specialist area, designers are recommended to seek specialist advice if special mix designs are required for durability.

There are number of important issues that designers should discuss with their clients on the durability of concrete at the beginning of any project, including:

- The design life for the concrete structure and whether Clause 4.1 of AS 3600 with an implied design life of 50 years ± 20% is appropriate (eg an iconic building such as a church might have a required design life of 100 years or more)
- The need for maintenance and repairs during the life of the building
- Any specific durability requirements for individual concrete members, as the durability issues may not be immediately apparent
- The level of inspection during construction to ensure that cover requirements are achieved on site

The above should be part of a durability plan and durability report for the building being designed which is accepted by all parties as part of the project risk management.

Such durability issues may require a re-assessment of covers and concrete strengths and the need for special concrete mixes.

TABLE 3.1 Required concrete properties

Surface and exposure environment	Exposure classification	Concrete properties f'_c (MPa)	Curing period[6] (days)
External surfaces above ground			
Within 1 km of coastline[1]	B2	40[5]	7
Within 1 to 50 km of coastline	B1	32	7
Further than 50 km from coastline and			
– within 3 km of industrial polluting area[2]	B1	32	7
– in tropical zone[3]	B1	32	7
– in temperate zone[3]	A2	25	3
– in arid zone[3]	A1	20	3
Internal surfaces			
In industrial building subject to repeated wetting and drying	B1	32	7
Non-residential	A2	25	3
Residential	A1	20	3
Surfaces in contact with the ground			
In contact with aggressive soils[4]			
– Sulfate bearing (magnesium content <1g/L)	A1 – C2 *Refer AS 3600 Table 4.8.1*	25–50	3–7
– Sulfate bearing (magnesium content >1g/L)	U	Designer to assess[7]	
Protected by a dpm	A1	20	3
in contact with non-aggressive soils			
– residential buildings	A1	20	3
– other members	A2	25	3
Surfaces in contact with water[8]			
In soft or running water	U	Designer to assess[7]	
In fresh water	B1	32	7
In seawater			
– permanently submerged	B2	40[5]	7
– in spray zone	C1	50[5]	7
– in tidal/splash zone	C2	50[5]	7
Other situations	U	Designer to assess[7] [8]	

Notes:

(1) See Figure 3.1. AS 3600 states that the coastal zone includes locations within 1 km of the shoreline of large expanses of salt water (eg Port Phillip Bay, Sydney Harbour east of the Spit and Harbour Bridges, Swan River west of the Narrows Bridge). Where there are strong prevailing onshore winds or vigorous surf, the distance should be increased beyond 1 km and higher levels of protection should be considered.

(2) Industrial polluting areas are defined in AS 3600 as areas where there are industries that discharge atmospheric pollutants. The 3-km distance should be increased if there are strong prevailing winds in one direction.

(3) See Figure 3.1.

(4) Severity of sulfate attack depends on the type of sulfate, which must be in solution. For example, magnesium and ammonium sulfates are more aggressive than sodium sulfate. The use of sulfate-resisting cement would be adequate for sodium sulfate conditions. For the magnesium and ammonium sulfates conditions, specific consideration should be given to the cement and the concrete that are likely to resist this type of sulfate.

(5) Special-class concrete is required for B2, C1 and C2 exposure classifications and this may require items such as the minimum cement content, the cement type, SCM and water-cement ratios to be specified by the designer.

(6) AS 3600 makes provision for accelerated curing regimes to be used by specifying average compressive strengths at the completion of the curing period in column 4 of AS 3600 Table 4.4.

Exposure classification	f'_c (MPa)	f'_{cm} at end of accelerated curing (MPa)
A1	20	≥15
A2	25	≥15
B1	32	≥20
B2	40	≥25
C1, C2	50	≥32

(7) Classification U represents an exposure environment not specified in this table but for which a degree of severity of exposure should be appropriately assessed and will involve special class concrete. Protective surface coatings may be taken into account in such an assessment. Further guidance on measures appropriate in exposure classification U may be obtained from AS 3735 *Concrete structures for retaining liquids*[3.10].

(8) For water-retaining structures, designers should consult AS 3735 as its requirements supplement and take precedence over those of AS 3600. It provides more-detailed advice for particular situations and sets out more-stringent requirements for concrete quality and cover to reinforcement and tendons.

AS 3600 sets out the minimum requirements for the design of concrete structures for durability. As noted earlier these will be adequate in many situations, but in others (particularly when there are unknowns) it will be prudent to exceed these requirements.

A sequence of steps in designing for durability in accordance with AS 3600 is provided in Flowchart 3.1.

Details of the Standard's requirements for particular concrete members are provided in Tables 3.1 to 3.6, while information for the achievement of appropriate durability in specific circumstances is provided in Sections 3.1.5, 3.1.6 and 3.1.7.

In Table 3.1, classifications A1, A2, B1, B2, C1 and C2 represent increasing degrees of severity of exposure. For most capital cities in Australia, external surfaces above ground will be B1 or B2 exposure classification as a minimum. Exposure classification B2 (within 1 km of the coastline) requires the use of special class concrete.

3.1.3 Concrete properties

Frequently, the concrete properties (including strength) required to meet the requirements for durability will control that for the design, while the requirements for cover can influence the member size.

3.1.4 Cover to reinforcement

AS 3600 sets out the minimum cover required to protect the reinforcement and tendons from fire and long-term corrosion. Since cover has a very significant influence on concrete's durability, greater cover should be specified when there are any durability concerns.

Inadequate and inappropriate cover to the reinforcement has been an ongoing durability problem, particularly with concrete exposed to the elements, and especially that in marine or other aggressive environments. Marosszeky and Gamble[3.11] have reported that on a number of building sites, where corrosion occurred the cover was as low as 5 mm. In a similar paper, Clarke et al[3.12] recognized the difficulty of achieving the right cover on site. Problems with cover were also discussed in a series of papers in *Concrete in Australia*[3.13].

Reinforcement can be complicated and congested. AS 3600 Clause 4.10.2 requires the designer to consider the cover depending on the the size and shape of the member, the size, type and configuration of the reinforcement (and, if present, the tendons or ducts), the aggregate size, the workability of the concrete and the direction of concrete placement.

It is important to recognise that reinforcement cannot be expected to end up in precisely the position shown on the drawings. An accuracy of ±5 mm is the best that can be expected. AS 3600 Clause 17.5.3 allows –5 or +10 mm deviation from the specified position for beams, slabs, columns and walls (–10 or +20 mm for slabs on ground and –10 or +40 mm for footings) where cover is critical. These tolerances are, however, sometimes difficult to achieve on site, especially with larger bars.

If, for example, 30 mm cover for a wall or beam is the absolute minimum required, then 40 mm should be specified (35-mm bar chairs are not available) as the reinforcement will then end up between 35 to 50 mm from the face of the wall or beam, based on the tolerances in AS 3600 (or between 30 and 50 mm for ±10 mm tolerances).

TABLE 3.2 Required cover (mm) – Standard formwork and standard compaction

Concrete strength f'_c (MPa)	Exposure classification A1	A2	B1	B2	C1	C2
20	20	[50]				
25	20	30	[60]			
32	20	25	40	[65]		
40	20	20	30	45	[70]	
≥ 50	20	20	25	35	50	65

Use of figures in brackets to the right of the zigzag line (with the related characteristic strength) is limited to when essentially only one surface of a member is subject to the particular exposure classification.

Where concrete is cast on or against ground, then the figures in this table should be increased:

Where protected by dpm – add 10 mm

Where not protected by dpm – add 20 mm

TABLE 3.3 Required cover (mm) – Rigid formwork with repetitive procedure and intense compaction or self-compacting concrete

Concrete strength f'_c (MPa)	Exposure classification A1	A2	B1	B2	C1	C2
20	20	[45]				
25	20	30	[45]			
32	20	20	30	[50]		
40	20	20	25	35	[60]	
≥ 50	20	20	20	25	45	60

Use of figures in brackets to the right of the zigzag line (with the related characteristic strength) is limited to when essentially only one surface of a member is subject to the particular exposure classification.

TABLE 3.4 Concrete strength for abrasion resistance

Member and type of traffic	Minimum f'_c (MPa)
Footpaths and residential driveways	20
Commercial and industrial floors not subject to vehicular traffic	25
Floors and pavements in public car parks, driveways and parking areas, subject only to light traffic (vehicles ≤3 t gross)	25
Floors and pavements in warehouses, factories, driveways and hard standings subject to:	
– medium or heavy pneumatic-tyred traffic (>3 t gross)	32
– non-pneumatic-tyred traffic	40
– steel-wheeled traffic	(to be assessed but ≥40)

TABLE 3.5 Freeze-thaw resistance

Exposure condition (cycles per annum)	Minimum f'_c (MPa)	Entrained air for nominal aggregate size (mm) 10–20	40
<25	32	8–4%	6–3%
≥25	40	8–4%	6–3%

TABLE 3.6 Exposure classification for concrete in sulfate soils (after AS 3600)

Exposure conditions: Sulfates (expressed as SO_4)* In soil (ppm)	In groundwater (ppm)	pH	Exposure classification: Soil conditions A**	Soil conditions B***
<5000	<1000	>5.5	A2	A1
5000–10 000	1000–3000	4.5–5.5	B1	A2
10 000–20 000	3000–10 000	4–4.5	B2	B1
>20 000	>10 000	<4	C2	B2

* Approximately 100 ppm SO_4 = 80 ppm SO_3
** Soil conditions A – high permeability soils (eg sands and gravels) which are in groundwater
*** Soil conditions B – low permeability soils (eg silts and clays) or all soils above groundwater

TABLE 3.7 Strength and cover requirements for saline soils (after AS 3600)

Soil electrical conductivity, EC_e*	Exposure classification	Minimum f'_c	Minimum cover (mm)
4–8	A2	25	45
8–16	B1	32	50
>16	B2	40	55

* EC_e is saturated electrical conductivity in deciSiemens per metre

TABLE 3.8 Effect of chemicals on concrete (after ACI 515.1R[3.17])

Material — Effect

Acetone — Liquid loss by penetration. May contain acetic acid as impurity.

Acid

- acetic — Disintegrates concrete slowly.
- carbonic — Disintegrates concrete slowly.
- formic — Disintegrates concrete slowly.
- humic — Disintegrates concrete slowly.
- hydrochloric — Disintegrates concrete and steel rapidly.
- hydrofluroic — Disintegrates concrete and steel rapidly.
- lactic — Disintegrates concrete slowly.
- nitric — Disintegrates concrete and steel rapidly.
- oxalic — Not harmful. Protects tanks against acetic acid, carbon dioxide and salt water. Poisonous. Should not be used with food or drinking water.
- phosphoric — Disintegrates concrete slowly.
- sulfuric — Disintegrates concrete and steel rapidly.
- sulfurous — Disintegrates concrete and steel rapidly.
- tannic — Disintegrates concrete slowly.

Acid water (pH of ≤ 6.5)[(1)] — Disintegrates concrete slowly. Attacks steel in porous or cracked concrete.

Alcohol (ethyl, methyl) — Liquid loss by penetration.

Alum (potassium aluminium sulfate) — Disintegrates concrete of inadequate sulfate resistance.

Aluminium chloride — Disintegrates concrete rapidly. Attacks steel in porous or cracked concrete.

Aluminium sulfate — Disintegrates concrete. Attacks steel in porous or cracked concrete.

Ammonia, liquid — Harmful only if it contains harmful ammonium salts.

Ammonia vapours — May slowly disintegrate moist concrete or attack steel in porous or cracked moist concrete.

Ammonium chloride — Disintegrates concrete slowly. Attacks steel in porous or cracked concrete.

Ammonium hydroxide — Not harmful.

Ammonium

- nitrate — Disintegrates concrete. Attacks steel in porous or cracked concrete.
- sulfate — as above
- superphosphate — as above

Automobile and diesel exhaust gases[(2)] — May disintegrate moist concrete by action of carbonic, nitric or sulfurous acid.

Beef fat — Solid fat disintegrates concrete slowly, melted fat more rapidly.

Beer — May contain (as fermentation products) acetic, carbonic, lactic or tannic acids.

Calcium chloride — Attacks steel in porous or cracked concrete. Steel corrosion may cause concrete to spall.

Calcium sulfate — Disintegrates concrete of inadequate sulfate resistance.

Carbon dioxide[(3)] — Gas may cause permanent shrinkage (see also *carbonic acid*).

Material — Effect

Castor oil — Disintegrates concrete, especially in presence of air.

Cinders — Harmful if wet, when sulfides and sulfates leach out (eg, see *sodium sulfate*).

Coke — Sulfides leaching from damp coke may oxidize to sulfurous or sulfuric acid.

Copper sulfate — Disintegrates concrete of inadequate sulfate resistance.

Creosote — Phenol present disintegrates concrete slowly.

Ethylene glycol[(4)] — Disintegrates concrete slowly.

Fermenting fruit, grains, vegetables or extracts[(5)] — Industrial fermentation processes produce lactic acid. Disintegrates concrete slowly (see also *fruit juices*).

Ferric sulfate — Disintegrates concrete of inadequate quality.

Ferrous sulfate — Disintegrates concrete of inadequate sulfate resistance.

Fertilizer — See *ammonium sulfate*, *ammonium superphosphate*, *manure*, *potassium nitrate* and *sodium nitrate*.

Fish liquor[(6)] — Disintegrates concrete.

Fish oil — Disintegrates concrete slowly.

Flue gases — Hot gases (200–600°C) cause thermal stresses. Cooled, condensed sulfurous and hydrochloric acids disintegrate slowly.

Fruit juices — Hydrofluoric, other acids, and sugar cause disintegration (see also *fermenting fruits, grains, vegetables or extracts*).

Hydrogen sulfide — Not harmful, but in moist, oxidizing environments converts to sulfurous acid, disintegrates concrete slowly.

Kerosene — Liquid loss by penetration.

Linseed oils — Liquid disintegrates concrete slowly. Dried or drying films are harmless.

Lubricating oil, machine oil — Fatty oils, if present, disintegrate concrete slowly.

Magnesium chloride — Disintegrates concrete slowly. Attacks steel in porous or cracked concrete.

Magnesium sulfate — Disintegrates concrete of inadequate sulfate resistance.

Manure — Disintegrates concrete slowly.

Margarine — Solid margarine disintegrates concrete slowly, melted margarine more rapidly.

Milk

- fresh — Not harmful.
- sour — Disintegrates concrete slowly.

Mine water, waste — Sulfides, sulfates, or acids present disintegrate concrete and attack steel in porous or cracked concrete.

Ores — Sulfides leaching from damp ores may oxidize to sulfuric acid or ferrous sulfate.

continues

TABLE 3.8 *continued* Effect of chemicals on concrete (after ACI 515.1R[3.17])

Material — Effect

Paraffin — Shallow penetration not harmful, but should not be used on highly-porous surfaces like concrete masonry[(7)]

Petroleum oils — Liquid loss by penetration. Fatty oils, if present, disintegrate concrete slowly.

Pickling brine — Attacks steel in porous or cracked concrete.

Potassium nitrate — Disintegrates concrete slowly.

Seawater — Disintegrates concrete of inadequate sulfate resistance. Attacks steel in porous or cracked concrete.

Silage — Acetic, lactic acids (and sometimes fermenting agents of hydrochloric or sulfuric acids) disintegrate slowly.

Sodium chloride — Magnesium chloride, if present attacks, steel in porous or cracked concrete. Steel corrosion may cause concrete to spall.

Sodium hydroxide

1–10% — Not harmful[(8)].

20% or over — Disintegrates concrete.

Sodium nitrate — Disintegrates concrete slowly.

Sodium sulfate — Disintegrates concrete of inadequate sulfate resistance.

Sugar — Disintegrates concrete slowly.

Turpentine — Mild attack. Liquid loss by penetration.

Urea — Not harmful.

Urine — Attacks steel in porous or cracked concrete.

(1) Waters of pH higher than 6.5 may be aggressive if they also contain bicarbonates. (Natural waters are usually of pH higher than 7.0 and seldom lower than 6.0, though pH values as low as 0.4 have been reported. For pH values below 3, protect as for dilute acid.)
(2) Composed of nitrogen, oxygen, carbon dioxide, carbon monoxide, and water vapour. Also contains unburned hydrocarbons, partially burned hydrocarbons, oxides of nitrogen, and oxides of sulfur.
(3) Carbon dioxide dissolves in natural waters to form carbonic acid solutions. When it dissolves to extent of 0.9 to 3 parts per million it is destructive to concrete.
(4) Used as deicer for airplanes overseas. Heavy spillage on runway pavements containing too-little entrained air may cause surface scaling.
(5) In addition to the intentional fermentation of many raw materials, much unwanted fermentation occurs in the spoiling of foods and food wastes, also producing lactic acid.
(6) Contains carbonic acid, fish oils, hydrogen sulfide, methyl amine, brine and other potentially reactive materials.
(7) Porous concrete which has absorbed considerable molten paraffin and then been immersed in water after the paraffin has solidified has been known to disintegrate from sorptive forces.
(8) However, in the limited areas where concrete is made with reactive aggregates, disruptive expansion may be produced.

3.1.5 Abrasion

The abrasion resistance of concrete is the ability of a surface to resist being worn away by rubbing or friction. Abrasion can be caused by foot or vehicular traffic or other sources such as mechanical equipment. The resistance is generally proportional to concrete strength. The use of mixes with a low water-cement ratio improve strength and thus the wear resistance of the surface.

Table 3.4 sets out requirements for abrasion resistance for foot and vehicular traffic after AS 3600. For other forms of abrasion, specialist advice may be required.

3.1.6 Freezing and thawing

Freezing and thawing of concrete is generally not a major problem in Australia except in sub-alpine and alpine areas and in specialist facilities such as cold stores. Table 3.5 sets out the requirements in AS 3600.

The entrained air content is determined in accordance with AS 1012.4[3.14].

3.1.7 Soil conditions

Acid sulfate soils The National Strategy for the Management of Coastal Acid Sulfate Soils[3.15], indicates that substantial low-lying coastal areas of the Northern Territory, Queensland and New South Wales are affected by acid sulfate soils (ASS). It also notes that there are similar conditions along the northern coastline of Western Australia, and around Perth, Adelaide and Westernport Bay near Melbourne as shown on Figure 3.2. In Australia, the acid sulfate soils of most concern are those which formed within the past 10,000 years, after the last major sea level rise.

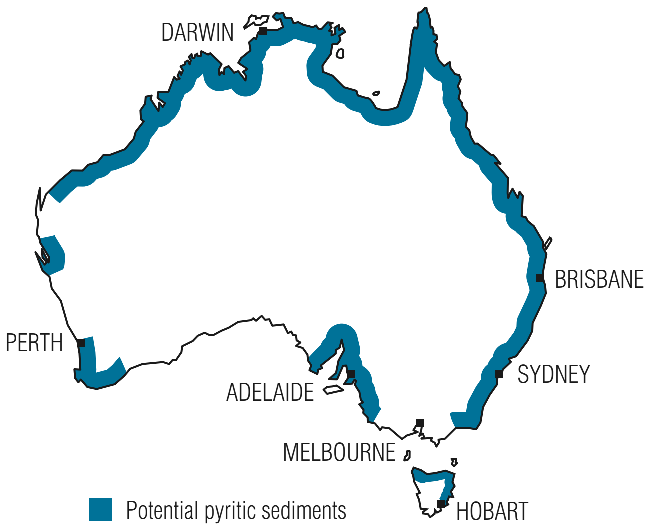


Figure 3.2 Indicative distribution of coastal acid sulfate soils in Australia. (from *National Strategy for the Management of Coastal Acid Sulfate Soil*)

TABLE 3.9 Protective barrier systems (after ACI 201.2R – 01[3.18])

Severity of chemical environment	Total nominal thickness range	Typical protective barrier systems	Typical but not exclusive uses of protective systems in order of severity
Mild	Under 1 mm	Polyvinyl butyral, polyurethane, epoxy, acrylic, chlorinated rubber, styrene-acrylic copolymer Asphalt, coal tar, chlorinated rubber, epoxy, polyurethane, vinyl, neoprene, coal-tar epoxy, coal-tar urethane	– Improve freeze-thaw resistance – Prevent staining of concrete – Protect concrete in contact with chemical solutions having a pH as low as 4, depending on the chemical
Intermediate	3 to 9 mm	Sand-filled epoxy, sand-filled polyester, sand-filled polyurethane, bituminous materials	– Protect concrete from abrasion and intermittent exposure to dilute acids in chemical, dairy, and food processing plants
Severe	0.5 to 6 mm	Glass-reinforced epoxy, glass-reinforced polyester, precured neoprene sheet, plasticised PVC sheet	– Protect concrete tanks and floors during continuous exposure to dilute mineral, organic acids (pH is below 3), salt solutions, strong alkalies
Severe	0.5 to 7 mm Over 6 mm	Composite systems: (a) Sand-filled epoxy system topcoated with a pigmented but unfilled epoxy (b) Asphalt membrane covered with acid-proof brick using a chemical-resistant mortar	– Protect concrete tanks during continuous or intermittent immersion, exposure to water, dilute acids, strong alkalies, and salt solutions – Protect concrete from concentrated acids or combinations of acids and solvents

TABLE 3.10 Recommended surface finishes (after *Guide to Industrial Floors and Pavements*[3.19])

Typical applications	Anticipated traffic	Exposure/service conditions	Finish
Office and administration areas, laboratories	Pedestrian or light trolleys	Pavements to receive carpet, tiles, parquetry, etc	Steel float
		Pavements with skid-resistant requirements	Wooden float or broomed tined (light texture)
Light to medium industrial premises, light engineering workshops, stores, warehouses, garages	Light to heavy forklift trucks or other industrial vehicles with pneumatic tyres	Smooth pavements	Steel trowel
		Dry pavements with skid-resistant requirements	Steel trowel (carborundum dust or silicon carbide incorporated into concrete surface)
		Wet and external pavements	Broomed/tined (hessian drag light to medium texture) or grooved
Sloping floors or ramps or high-speed traffic areas			Broomed/tined (coarse texture) or grooved
Heavy industrial premises, heavy engineering works, repair workshops, stores and warehouses	Heavy solid wheel vehicles or steel wheeled trolleys	Pavements subject to severe abrasion	Steel trowel/burnished finish (use of special aggregate monolithic toppings)

In themselves, acid sulfate soils are not necessarily a problem. The problem is that the coastal regions are home to the majority of the Australian population and to various industry and production activities including utility supply, agriculture, aquaculture as well as sand and gravel extraction, which can disturb acid sulfate soils. Once disturbed and exposed to oxygen, these soils oxidise and produce sulfuric acid. The economic consequences of exposure to acid sulfate soils have the potential to be significant and can impact on agriculture, fishing, industry, urban development and infrastructure.

The requirements for exposure classifications for concrete in sulfate soils provided in AS 3600 Table 4.8.1 are reproduced in Table 3.6. The Standard includes extensive notes to Table 4.8.1. See also CCAA's Technical Note *Sulfate-resisting Concrete*[3.7].

Saline soils In saline conditions, or where they are likely to develop over time, the requirements for concrete in contact with the ground need to be assessed to ensure its durability and satisfactory performance over the design life of the structure. Table 3.7 sets out AS 3600 requirements for saline soils.

Designers can also refer to *Building with concrete in saline soils*[3.16] and the CCAA *Guide to Residential Slabs and Footings in Saline Environments*[3.8] for further information.

3.1.8 Effect of chemicals

The effects of a comprehensive range of chemicals are shown in Table 3.8, while recommended barrier systems are shown in Table 3.9.

3.1.9 Floor finishes

The surface finish always needs careful consideration; the recommended finishes are set out in Table 3.10.

3.2 DESIGN FOR FIRE RESISTANCE

3.2.1 General

Both engineers and regulators consider concrete structures to be inherently fire resistant and that high levels of fire resistance can be achieved by adopting certain axis distances and member dimensions. The reason for this is that concrete has both low thermal conductivity and high heat capacity; concrete elements are therefore naturally resistant to temperature rise due to fire exposure. In addition, experience in real fires has shown that concrete structures generally perform well. CCAA's *Fire Safety of Concrete Buildings*[3.20] covers these issues including advice on the performance of high-strength concrete.

The deemed-to-satisfy provisions for Design for Fire Resistance are set out in Section 5 of AS 3600, which should be read in conjunction with the requirements given in the Building Code of Australia (BCA)[3.21]. The BCA sets out the requirements for fire resistance for elements of a building. These depend on the building classification, rise in storeys and exposure to a fire-source feature, taking into account applicable concessions and other general requirements relating to the performance in fires. The rules for design for fire resistance in Section 5 of AS 3600 are based on Eurocode 2[3.22].

Designing for fire resistance using the deemed-to-satisfy approach largely involves the specification for the various members of the required Fire Resistance Level (FRL). This is a composite number of the grading periods in minutes for each of structural adequacy/integrity/insulation, as appropriate. The grading periods are specified in Specification A2.3 of the BCA. It is implied that the grading periods are intended to reflect the Fire Resistance Periods (FRP) for the respective criteria when the element is tested in a Standard Fire Test.

The BCA provides procedures to meet those performance requirements. There are three basic approaches:

- Deemed-to-satisfy constructions specified in terms of required Fire Resistance Levels (FRL) for various elements of construction
- Alternative solutions
- By fire tests.

The data in AS 3600 is provided to complement the first of these three approaches. The Fire Resistance Level (FRL) is the Fire Resistance Periods (FRP) for structural adequacy, integrity and insulation, expressed in that order.

Section 5 of AS 3600 sets out deemed-to-satisfy data to determine Fire Resistance Periods for the various member types. While this approach is used for the great majority of buildings, a small but increasing number of buildings are being designed using the other two approaches, involving fire engineering. These approaches involve neither the use of AS 3600 in general nor Section 5 in particular.

The BCA clearly states that in the event of conflict between it and clauses in referenced standards, then the rules in the BCA shall take precedence.

The fire resistance requirements in AS 3600 nominate 'axis distances' for longitudinal reinforcement (see Figure 3.3), not 'cover' (to any reinforcement, including fitments) as is the case for durability.

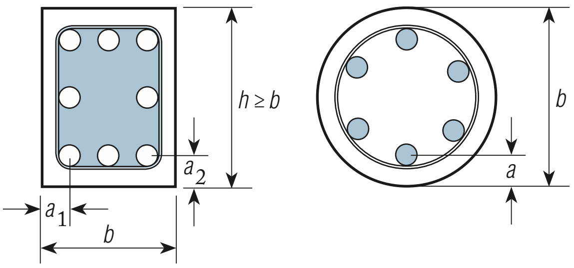


Figure 3.3 Axis distance, *a* (after AS 3600)
Note: Axis distances are nominal values and no allowance for tolerance need be added

Interpolation is permitted between adjacent values in the tables. It should also be noted that in many cases the axis distance will not control the design. For example, in a beam with a single layer of reinforcement with a main bar diameter of 20 mm, fitment size of 10 mm and a cover of 30 mm (for durability), if the axis distance required is less than 50 mm it will not control the design.

In cases where small axis distances imply small or negative covers, the minimum cover for durability and concrete placement will determine the axis distance actually provided for the member.

Definitions for Fire Resistance Level (FRL), Fire Resistance Period (FRP), structural adequacy, integrity and insulation are given in AS 3600 Clause 5.2. Generally, there are no specific rules for integrity; it is assumed to be satisfied when the requirements for structural adequacy and insulation are met.

AS 3600 Clause 5.3.1 states that *the FRP for a member shall be established by either one of the following methods:*

(a) Determined from the tabulated data and figures given in this Section. Unless stated otherwise within this Section, when using the tabulated data or figures no further checks are required concerning shear and torsion capacity or anchorage details.

(b) Predicted by methods of calculation. In these cases, checks shall be made for bending, and where appropriate, shear, torsion and anchorage capacities.

NOTE: Eurocode 2, Part 1.2 provides a method of calculation to predict the FRP of a member.

3.2.2 Spalling of floors, beams and columns

Spalling is defined as the breaking off of pieces of concrete from the surface of a structural element when it is heated in a fire. Malhotra[3.23] defines three types of spalling: surface pitting, corner break-off and explosive. Surface pitting is when pieces of aggregate fly off from the surface. This usually occurs during the early part of the heating. Corner break-off occurs during the latter part of the heating after cracks have developed in the member and pieces fall off from the corners and arrises of beams and columns. Explosive spalling occurs during the early part of heating when large pieces of concrete, up to 300-mm long, forcibly burst from the member. This is regarded as the most important type from a practical point of view.

BS 8110 Part 2[3.24] Clauses 4.1.6 and 4.1.7 suggest that with covers exceeding 40 to 50 mm, rapid rates of heating, large compressive stresses and moisture contents over 2% to 3% by mass of dense concrete there is a high risk of spalling. Forrest[3.25], reporting the findings of the committee that formulated the BS 8110 requirements, notes that fitments act to retain concrete around the main bars and that this limitation on cover should be applied to the surface of the fitment and not the main reinforcement. This is the basis of the shaded area on Charts 3.1 and 3.2, suggesting where spalling may need to be considered.

When spalling has to be considered, the alternative strategies for its prevention are:

- the use of polypropylene fibres;
- the use of an applied finish of plaster or vermiculite, etc or lost formwork;
- the provision of a false ceiling as a fire barrier;
- the use of lightweight aggregate or (for columns) limestone aggregate;
- the use of sacrificial tensile steel reinforcement;
- if it does not conflict with durability requirements, the use of a supplementary mesh in the cover zone of concrete 20 mm from the concrete face.

The use of 500-MPa reinforcement of itself will not tend to influence the spalling tendency, though consequential detailing practices may. However, it has been reported[3.26] that the use of high strength concrete ($f'_c > 50$ MPa), increased the spalling tendency.

Plank[3.27] states that for reinforced concrete structures in fire and in particular for slabs, two important phenomena, spalling and diaphragm action are not accounted for in the current simple code approaches. Ignoring spalling in slabs is unconservative but in contrast, tensile membrane action, which is also ignored in simple approaches, can significantly improve the performance of a fire-exposed structure.

3.2.3 Joints

AS 3600 Clause 5.3.5 requires that joints between members or adjoining parts be constructed so that the FRL of the whole assembly is not less than that required for the member. Data on the performance of various generic joint and sealant types is limited and information on specific proprietary sealants (including limitations on joint geometry) should be obtained from the manufacturers. The CIA's *Design of Joints in Concrete Buildings*[3.28] includes charts for calculating the extent of non-combustible fibre blanket needed in a butt joint to walls to provide the required fire-resistance periods.

3.2.4 Chases and openings

AS 3600 Clause 5.3.6 requires that chases in concrete members subject to fire be kept to a minimum. Openings will normally require a fire rated infill to meet the same FRP as the wall, eg fire rated doors and fire rated dampers for services.

3.2.5 Increasing the FRP by addition of insulating material

AS 3600 Clause 5.3.7 provides guidance on how to increase the FRP of an element by various techniques. More information is given in Section 3.2.10.

3.2.6 Beams

Charts 3.1 and 3.2 reflect the information in AS 3600 Figures 5.4.2(A) and (B). See the discussion in Section 3.2.2 for the basis of the shaded area where spalling may need to be considered. They can also be used for beams exposed on all four sides (see AS 3600 Clause 5.4.6).

3.2.7 Slabs

Tables 3.11 to 3.15 reflect the information in AS 3600 Clause 5.5 and Tables 5.5.1 and 5.5.2(A), (B), (C) and (D).

AS 3600 Clause 5.5.1 states that for insulation the effective thickness of slab shall be taken as: for solid slabs, the actual thickness; for hollowcore slabs, the net cross-sectional area divided by the width of the cross section; for ribbed slabs, the thickness of the solid slab between the webs of adjacent ribs.

For structural adequacy for slabs, AS 3600 requires the following:

- For solid or hollow-core slabs supported on beams or walls the average axis distance is not less than the value shown in AS 3600 Table 5.5.2(A).
- For flat slabs, including flat plates, the average axis distance is is not less than that shown in AS 3600 Table 5.5.2(B); and at least 20% of the total top reinforcement in each direction over intermediate supports is continuous over the full span and placed in the column strip.
- For one-way ribbed slabs, for the appropriate support conditions, the slab is proportioned so that the width of the ribs and the axis distance to the lowest layer of the longitudinal bottom reinforcement in the slab complies with the requirements for beams given in AS 3600 Clause 5.4.1, and the axis distance to the bottom reinforcement in the slab between the ribs is not less than that given in AS 3600 Table 5.5.2(A).
- For two-way ribbed slabs the slabs shall be proportioned so that the width and the average axis distance to the longitudinal bottom reinforcement in the ribs is not less than that given in AS 3600 Tables 5.5.2(C) and (D) as appropriate, and the axis distance to the bottom reinforcement in the slab between the ribs, and the axis distance of the corner bar to the side face of the rib, is not less than that value plus 10 mm.

A slab shall be considered continuous if, under imposed actions, it is designed as continuous at one or both ends.

Table 3.11 Minimum effective thickness of slabs for insulation (after AS 3600)

Fire-resistance period (min)	**Effective thickness** (mm)
30	60
60	80
90	100
120	120
180	150
240	175

3.2.8 Columns

Insulation and integrity of columns is required only where they are part of a wall with a fire-separating function. In this case they must comply with AS 3600 Clause 5.7.1 for walls.

The structural adequacy for columns can be determined from Table 3.16 (axis distance and smaller column cross-sectional dimension are not less than the tabulated value for the desired FRP). Where the column is a wall (ie the longer cross section dimension is more than four times the shorter dimension), Table 3.18 can be used. AS 3600 Clause 5.6.2 also provides for an alternative method for columns in a braced structure.

Generally, the value of the load level, N^*_f/N_u, will be taken as 0.7 but designers can calculate the value if they wish.

When A_s is greater than 2% and the required FRP is greater than 90 min, then bars need to be distributed along all faces with a minimum of two bars in any face.

The effective length of columns shall not exceed 3 m, and eccentricity shall be limited to 0.15*b*. Braced columns can be up to 6 m long. For longer columns designers will have to use the Eurocode 2, Part 1.2 method of calculation to predict the FRP of the column.

CHART 3.1 Simply-supported reinforced concrete beams exposed to fire on three or four sides

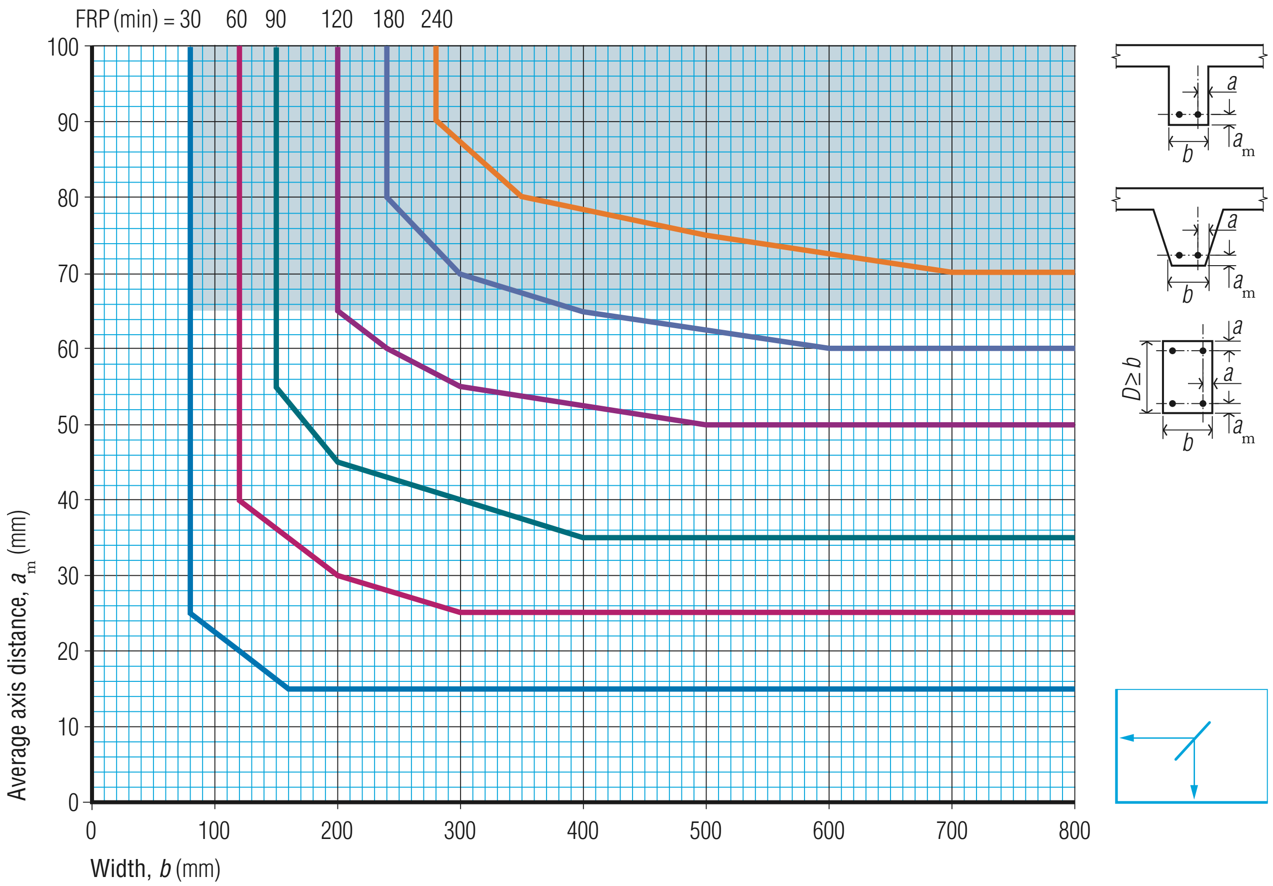


CHART 3.2 Continuous reinforced concrete beams exposed to fire on three or four sides

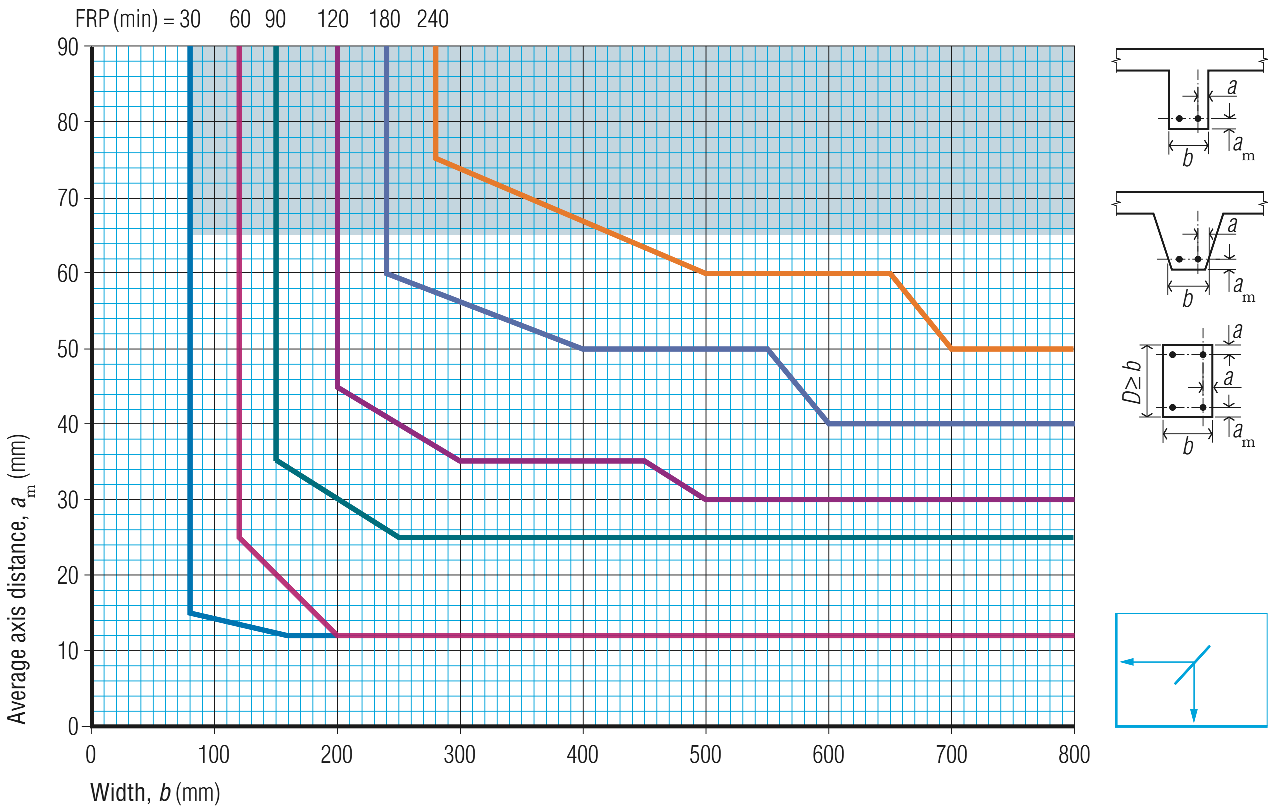


TABLE 3.12 Structural adequacy of solid and hollowcore slabs supported on beams or walls and for one-way ribbed slabs (after AS 3600)

Fire-resistance period for structural adequacy (min)	Axis distance to lower layer of bottom reinforcement (mm)			
	Simply-supported slabs			Continuous slabs
	One-way	Two-way $l_y/l_x \le 1.5$	Two-way $1.5 < L_y/l_x \le 1.5 \le 2$	One- and two-way
30	10	10	10	10
60	20	10	15	10
90	30	15	20	15
120	40	20	25	20
180	55	30	40	30
240	65	40	50	40

Notes:

1 l_y is the longer span and l_x the short span for two-way slabs.

2 The axis distance assumes slabs are supported on four sides, otherwise they are treated as one-way slabs.

TABLE 3.13 Structural adequacy of flat slabs including flat plates (after AS 3600)

Fire-resistance period for structural adequacy (min)	Minimum dimensions (mm) Slab thickness	Minimum dimensions (mm) Axis distance to lower layer of bottom reinforcement
30	150	10
60	180	15
90	200	25
120	200	35
180	200	45
240	200	50

TABLE 3.14 Structural adequacy of two-way simply supported ribbed slabs (after AS 3600)

Fire-resistance period for structural adequacy (min)	Minimum dimensions (mm)							
	Possible combinations of axis distance, a_s, and width of ribs, b						Flange thickness, h_s and axis distance, a_s in flange	
	Combination 1		Combination 2		Combination 3			
	a_s	b	a_s	b	a_s	b	a_s	h_s
30	15	80	—	—	—	—	10	80
60	35	100	25	120	15	≥200	10	80
90	45	120	40	160	30	≥250	15	100
120	60	160	55	190	40	≥300	20	120
180	75	220	70	260	60	≥410	30	150
240	90	280	75	350	70	≥500	40	175

Note:

1 The axis distance is measured to the lowest layer of the longitudinal reinforcement.

TABLE 3.15 Structural adequacy of two-way continuous supported ribbed slabs (after AS 3600)

Fire-resistance period for structural adequacy (min)	**Minimum dimensions** (mm)							
	Possible combinations of axis distance, a_s, and width of ribs, b						Flange thickness, h_s and axis distance, a_s in flange	
	Combination 1		Combination 2		Combination 3			
	a_s	b	a_s	b	a_s	b	a_s	h_s
30	10	80	—	—	—	—	10	80
60	25	100	15	120	10	≥200	10	80
90	35	120	25	160	15	≥250	15	100
120	45	160	40	190	30	≥300	20	120
180	60	310	50	600	—	—	30	150
240	70	450	60	700	—	—	40	175

Notes:

1 The axis distance is measured to the lowest layer of the longitudinal reinforcement.

2 For prestressing tendons, the axis distance shall be increased as given in Clause 5.3.3.

TABLE 3.16 Fire resistance periods for structural adequacy of columns

Fire-resistance period for structural adequacy (min)	**Minimum dimensions** (mm)							
	Combinations for column exposed on more than one side						Column exposed on one side	
	$N^*_f/N_u = 0.2$		$N^*_f/N_u = 0.5$		$N^*_f/N_u = 0.7$		$N^*_f/N_u = 0.7$	
	a_s	b	a_s	b	a_s	b	a_s	h_s
30	25	200	25	200	32 27	200 300	25	155
60	25	200	36 31	200 300	46 40	250 350	25	155
90	31 25	200 300	45 38	300 400	53 40[2]	350 450[2]	25	155
120	40 35	250 350	45[2] 40[2]	350[2] 450[2]	57[2] 51[2]	350[2] 450[2]	35	175
180	45[2]	350[2]	63[2]	350[2]	70[2]	450[2]	55	230
240	61[2]	350[2]	75[2]	450[2]	—	—	70	295

Notes:

1 a_s = axis distance

b = smaller cross-sectional dimension of a rectangular column or the diameter of a circular column.

2 These combinations for columns with a minimum of 8 bars.

TABLE 3.17 Minimum effective thickness for insulation for walls (after AS 3600)

Fire-resistance period (FRP) **for insulation** (min)	**Effective thickness** (mm)
30	60
60	80
90	100
120	120
180	150
240	175

TABLE 3.18 Fire resistance periods (FRP) for structural adequacy for walls

Fire-resistance period for structural adequacy (min)	**Minimum dimensions** (mm) **combinations of** a_s **and** b							
	$N^*_f/N_u = 0.35$				$N^*_f/N_u = 0.7$			
	Wall exposed on one side		Wall exposed on two sides		Wall exposed on one side		Wall exposed on two sides	
	a_s	b	a_s	b	a_s	b	a_s	b
30	10	100	10	120	10	120	10	120
60	10	110	10	120	10	130	10	140
90	20	120	10	140	25	140	25	170
120	25	150	25	160	35	160	35	220
180	40	180	45	200	50	210	55	270
240	55	230	55	250	60	270	60	350

Legend:
a_s = axis distance
b = wall thickness

TABLE 3.19 Thickness of vermiculite/perlite concrete or gypsum-vermiculite/gypsum-perlite plaster to provide increased cover

Increased cover required (mm)	**Plaster thickness** (mm)
5	4
10	8
15	12
20	15
25	19
30	23

TABLE 3.20 Thickness of plaster to increase the insulation value of slabs

Increase in thickness required (mm)	**Nominal thickness of topping to be added** (mm)		
	Plain concrete	Vermiculite/perlite	Gypsum
10	20	18	16
20	30	26	22
30	40	34	28
40	50	42	34
50	60	50	40

3.2.9 Walls

Tables 3.17 and 3.18 reflect the information in AS 3600 Clause 5.7 and Tables 5.7.1 and 5.7.2.

The FRP for insulation depends on the effective thickness as shown in Table 3.17. The effective thickness of the wall to be used in Table 3.17 shall be: for solid walls, the actual thickness; for hollowcore walls (and sandwich walls or similar), the net cross-sectional area divided by the length of the cross-section.

AS 3600 requires that for walls that have an FRL, the ratio of the effective height to thickness shall not exceed 40, where the effective height is determined from AS 3600 Clause 11.4. This latter restriction does not apply to walls where the lateral support at the top of the wall is provided by an element not required by the relevant authority to have an FRL. AS 3600 Clause 11.1.(b) (ii) limits the slenderness ratio to 50 assuming the wall is designed as a slab.

The FRP for structural adequacy for a wall given in Table 3.18 shall be used, provided the axis distance to the vertical reinforcement and the effective thickness of the wall is not less than the corresponding values given in the Table.

For walls where the lateral support at the top of the wall is provided on one side only by a member not required by the relevant authority to have an FRL, structural adequacy will be considered to be achieved by satisfying Table 3.17. This would apply to single-storey buildings with precast or tilt-up walls.

AS 3600 Clause 5.7.4 covers recesses and chases in walls under various conditions.

3.2.10 Increasing FRPs by use of insulating materials

The information given in Tables 3.19 and 3.20 is derived from AS 3600 Clause 5.8

REFERENCES

3.1 AS 3600 *Concrete structures* Standards Australia, 2009.

3.2 *Durable Concrete Structures* (Z07), 2nd Ed, Concrete Institute of Australia, 2001.

3.3 *Performance Criteria for Concrete in Marine Environments* (Z13), Concrete Institute of Australia, 2001.

3.4 ACI 201.2R-08 *Guide to Durable Concrete*, ACI Manual of Concrete Practice, 2008.

3.5 AS 4997 *Guidelines for the Design of Maritime Structures* Standards Australia, 2005.

3.6 *Chloride Resistance of Concrete*, Technical Report, Cement Concrete & Aggregates Australia, 2009.

3.7 *Sulfate-resisting Concrete* (TN68), Cement Concrete & Aggregates Australia, 2007.

3.8 *Guide to Residential Slabs and Footings in Saline Environments* (T57), Cement Concrete & Aggregates Australia, 2005.

3.9 AS 1379 *Specification and supply of concrete* Standards Australia, 2007.

3.10 AS 3735 *Concrete structures for retaining liquids* Standards Australia, 2001.

3.11 Marosszeky M and Gamble J *Design, Detailing and Construction of Reinforcement for Durable Concrete Building*, Research Centre, The University of New South Wales, 1987.

3.12 Clark LA, Shamas-Toma MGK, Seymour DA, Pallet PF and Marsh BK 'How can we get the cover we need', *The Structural Engineer*, Volume 75, No. 17, September 1997.

3.13 'Concrete Cover', *Concrete in Australia*, Vol 36, No. 1 March 2010.

3.14 AS 1012.4.1 *Methods of testing concrete* Part 4: *Methods for the determination of air content of freshly mix concrete* Standards Australia, 1999.

3.15 http://www.ozcoasts.org.au/indicators/econ_cons_acid_sulfate_soils.jsp.

3.16 Lume E and Sirivivatnanon V *Building with Concrete In Saline Soils*, Proceedings of UrbanSalt 2007 Conference, 22–23 May 2007.

3.17 ACI *515 A Guide to the Use of Waterproofing, Damp proofing, Protective and Decorative Barrier Systems for Concrete*, ACI Manual of Concrete Practice, 2000.

3.18 ACI 201.2R-08 *Guide to Durable Concrete*, ACI Manual of Concrete Practice, 2009.

3.19 *Guide to Industrial Floors and Pavements* (T48), Cement Concrete & Aggregates Australia, 2009.

3.20 *Fire Safety of Concrete Buildings* (T61), Cement Concrete & Aggregates Australia, 2010.

3.21 *Building Code of Australia* Australian Building Codes Board, 2010.

3.22 European Committee for Standardisation (CEN) (2004), – Eurocode 2: Design of concrete structures Part 1-1: General rules for buildings, The European Standard EN 1992-1-1:2004.

3.23 Malhotra HL *Spalling of concrete in fires* Technical Note 118, Construction Industry Research and Information Association, 1984.

3.24 BS 8110 *Structural use of concrete* Part 2: *Code of practice for special circumstances* British Standards Institution, 1985.

3.25 Forrest JCM 'New Fire-Resistance Data for Concrete', *Concrete*, UK, Vol. 18, No. 11, November 1984.

3.26 Phan LT *Fire performance of high strength concrete: a report of the state-of-the-art*, NISTIR 5934, US Department of Commerce, December 1996.

3.27 Plank R *The fire resistance of reinforced concrete structures*, Concrete Institute of Australia, Biennial Conference, 2007.

3.28 *Design of Joints in Concrete Buildings* (CPN 24), Concrete Institute of Australia, 2005.

Chapter 4_Beams

4.1 APPLICABILITY TO DUCTILITY CLASSES OF REINFORCING STEEL

The Charts, Tables and Spreadsheets in this Handbook are dependent on the Ductility Class of the reinforcement.

AS/NZS 4671[4.1] covers three Ductility Classes of reinforcement: N, L and E. Only two of these are available in Australia, ie N and L. Designers should note that they must specify that reinforcement complies with the requirements of AS/NZS 4671 for building projects where AS 3600[4.2] is used in conjunction with the BCA[4.3].

AS 3600 imposes limitations on the use of reinforcing steel of Ductility Class L, eg AS 3600 Clause 1.1.2 states that Ductility Class L reinforcement:

- *may be used as main or secondary reinforcement in the form of welded wire mesh, or as wire, bar and mesh in fitments; but*
- *shall not be used in any situation where the reinforcement is required to undergo large plastic deformation under strength limit state conditions.*

These limitations on Ductility Class L bar preclude its use as longitudinal tensile reinforcement in beams. As a result, it is not considered in any of the charts and spreadsheets in this Chapter. In addition, the use of Ductility Class L reinforcement is further limited by other clauses in AS 3600.

Reinforcing steel of Ductility Class N may be used, without restriction, in all applications referred to in AS 3600 and the Charts and Spreadsheets herein are based on it.

It is the designer's responsibility to ensure that the Ductility Class of the reinforcement specified and used on site: reflects the assumptions in the analysis methods, and is appropriate to the situation and the member being designed. The capacity reduction factor, ϕ, for a strength check using a linear elastic analysis for Ductility Class L reinforcement is lower than that for Ductility Class N reinforcement, see AS 3600 Table 2.2.2 to allow for its lower ductility, compared to that of Ductility Class N reinforcement.

4.2 RECTANGULAR BEAMS IN BENDING

4.2.1 General

To ensure a beam section has adequate ductility (at ultimate strength under bending and/or compression), AS 3600 Clause 8.1.5 states that k_o (the ratio, at ultimate strength, without axial force of the depth to the neutral axis from the extreme compression fibre to d) should not exceed 0.36 and M^* should not exceed 0.6 M_u unless specific requirements are met.

It should be noted that k_{uo} is not the balanced design condition as balanced sections are not ductile. The basic outline of a rectangular beam in bending is shown in Figure 4.1.

The general theory of bending in reinforced concrete members is discussed in more detail in textbooks such as *Concrete Structures*[4.4] and *Reinforced Concrete Basics*[4.5].

Cross-sections where k_{uo} is greater than 0.36 are referred to as 'over-reinforced' or 'non-ductile' and have limited ductility. Over-reinforced members may have a number of unfavourable characteristics such as:

- susceptibility to sudden, brittle failure with little warning;
- reduced ability to redistribute moments due to unexpected loads or settlement; and
- limited energy-absorption capacity under seismic or blast loading.

When the structural analysis has been carried out in accordance with AS 3600 Clauses 6.2 to 6.6 and an over-reinforced cross-section cannot be avoided, then a minimum amount of compression reinforcement has to be provided, viz 1% of the area of concrete in compression. The design strength in bending of an over-reinforced section is not to be taken as more than the ultimate strength in bending, ϕM_{uo}, when k_u = 0.36, with the force in the tensile reinforcement reduced to balance the reduced compressive force in the concrete. An over-reinforced cross-section is therefore not an economical or preferred design solution.

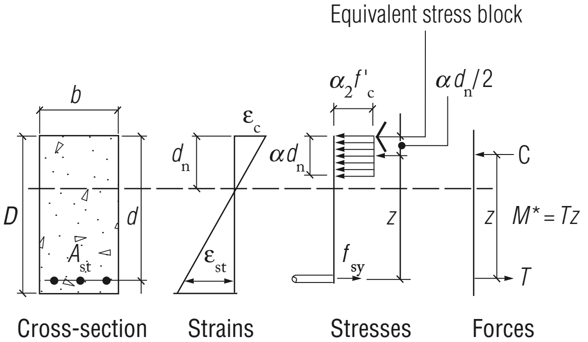


Figure 4.1 Basic sections, strains, stresses and forces

The charts, tables and spreadsheets in this chapter for the strength of beams in bending are based on the principles set out in AS 3600 Clauses 8.1.2 and 8.1.3. The rectangular stress block assumes a maximum strain in the extreme compression fibre of the concrete of 0.003 and a uniform compressive stress of $\alpha_2 f'_c$ acting on an area bounded by the edges of the section and a line parallel to the neutral axis under the loading concerned and located at a distance $\gamma k_u d$ from the extreme compressive fibre.

To calculate the equivalent rectangular stress block the factors α_2 and γ are taken as:

$\alpha_2 = 1.0 - 0.003\, f'_c$ (within the limits $0.67 \le \alpha_2 \le 0.85$),

and

$\gamma = 1.05 - 0.007\, f'_c$ (within the limits $0.67 \le \gamma \le 0.85$).

The values of α_2 and γ are shown in Table 4.1 and graphically in Figure 4.2.

TABLE 4.1 Value of γ and α_2 for various concrete strengths, f'_c

	Concrete strength f'_c (MPa)							
Factor	20	25	32	40	50	65	80	100
γ	0.85	0.85	0.826	0.77	0.7	0.67	0.67	0.67
α_2	0.85	0.85	0.85	0.85	0.85	0.805	0.76	0.7

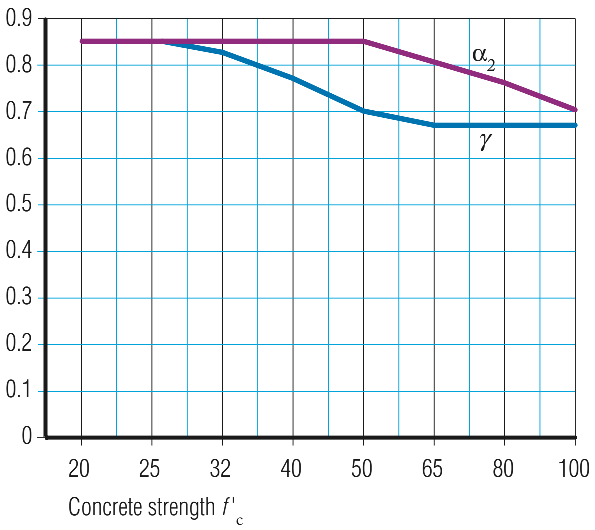


Figure 4.2 Relationship of γ and α_2 with the concrete strength, f'_c

4.2.2 Basis of Chart 4.1

The non-dimensional curves in Chart 4.1 are useful for initial sizing and are derived from the following basic equations:

$$M^*/bd^2 = \phi f'_c\, q\,(1 - q/1.7)$$

This equation assumes $\alpha_2 = 0.85$, the value given in Table 4.1 for concrete strengths from 20 to 50 MPa. The chart is therefore limited to that range of concrete strengths. This will cover most design situations for beams in bending.

For the above equation:

$\phi = 0.6 \le (1.19 - 13k_{uo}/12) \le 0.8$ from AS 3600 Table 2.2.2. (When $k_u \le 0.36$ as set out in AS 3600 Clause 8.1.5, then $\phi = 0.8$.)

$q = A_{st} f_{sy} / bd\, f'_c$ and

f_{sy} = 500 MPa.

The maximum design strength in bending, ϕM_{uo} allowed by AS 3600 occurs at the ductile limit, ie k_{uo} = 0.36. At the ductile limit:

$p_{max} = 0.85\, \gamma f'_c / f_{sy}\, k_{uo} = 0.306\, \gamma f'_c / f_{sy}$ and

$\gamma = 1.05 - 0.007\, f'_c$ (within the limits $0.67 \le \gamma \le 0.85$) and γ varies from 0.85 for f'_c = 20 MPa to 0.67 for $f'_c \ge 65$ MPa and

$M_{ud}/bd^2 = 0.306\, \gamma f'_c\, (1 - 0.18\gamma)$.

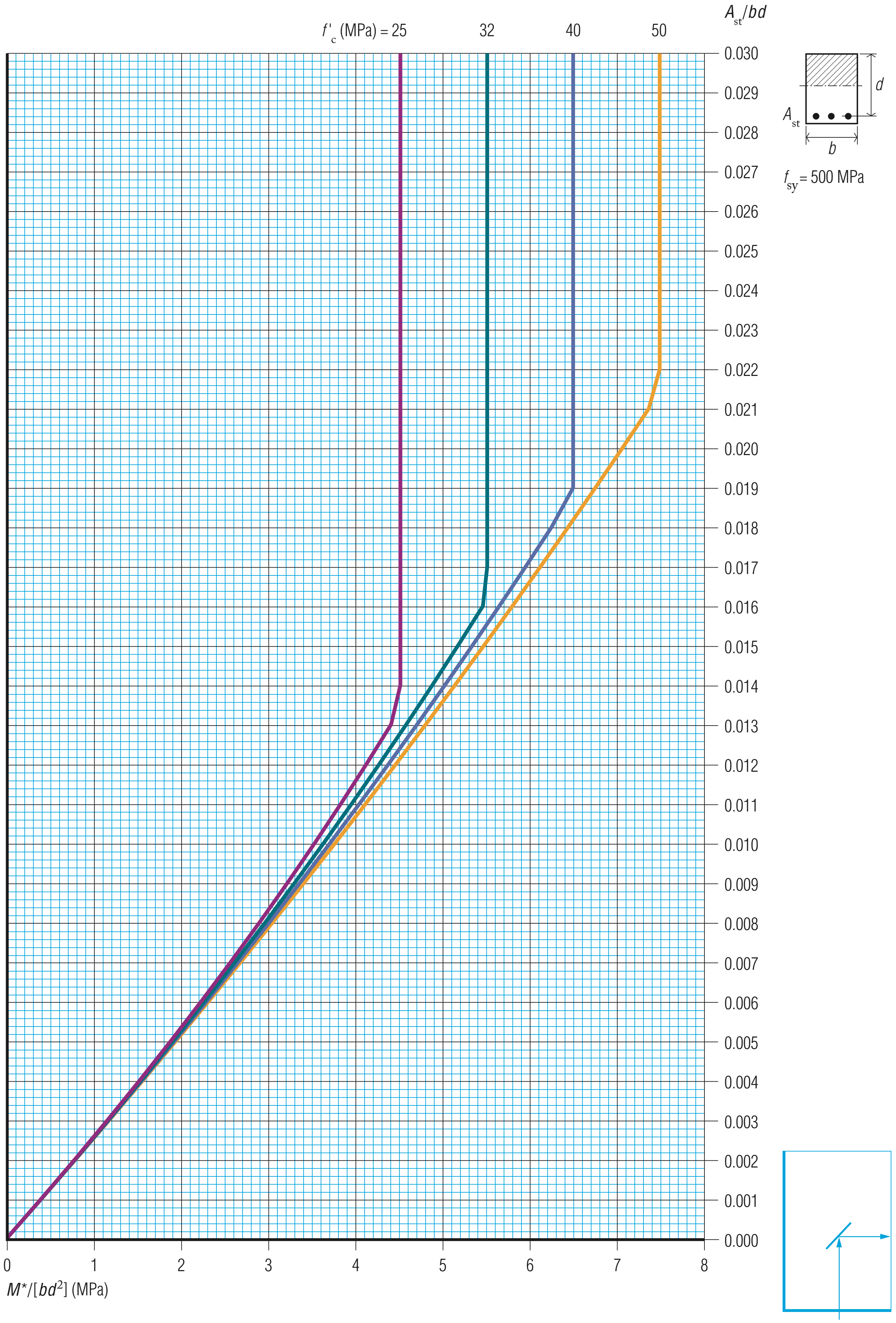
4.2.3 Design of rectangular beams

The bending moments in a beam are determined from the structural analysis. Generally they vary along the beam, eg from maximum moment at the middle of a simply-supported beam reducing to nil at the ends. They vary from positive to negative across a span depending on the continuity and spans, as determined by the analysis. It is the responsibility of the designer to establish the critical section(s) for bending and the design ultimate moments.

Spreadsheet 4.1 can be used to calculate the reinforcement requirements for a reinforced rectangular concrete beam cross-section in flexure in accordance with Flowchart 4.1. It uses the requirements of AS 3600 and the standard design principles for ultimate strength design. It checks the minimum reinforcement and assumes that $k_u \le k_{uo}$ but it does not check cover, spacing requirements or detailing requirements, nor does it cover cross-sections with compression reinforcement.

As with many design calculations, some initial design parameters are assumed and then checked and adjusted as required. For the design of concrete beams, once the size and concrete strengths are chosen, then an initial area of reinforcing steel is required to be input.

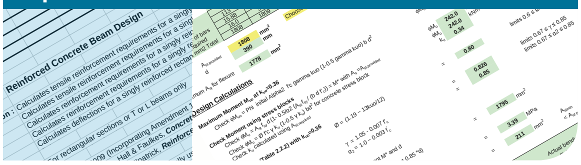
CHART 4.1 Rectangular beam without compression reinforcement



As noted in *Reinforced Concrete Basics*[4.5], for an under reinforced beam, the ultimate moment capacity, $M_u \approx 0.85\ A_{st}\ f_{sy}\ d$ (within 10% of a more accurate calculation), ie it is independent of the concrete strength and width of the beam. This approximation can be used to make an initial estimate of the area of reinforcement required.

The designer then enters an area of reinforcement approximating to this estimate (usually a number of bars of one size, eg 3N24). The spreadsheet then calculates the actual moment capacity for the chosen area of reinforcement and compares it with the design moment. If the calculated moment is less than or significantly greater than the design moment, it will be necessary to repeat the calculations with a revised area of reinforcement.

Spreadsheet 4.1 is available at www.ccaa.com.au



4.3 T-BEAMS AND L-BEAMS IN BENDING

4.3.1 General

Concrete beams will often be part of the floor or roof structure; in these cases they will be T- or L-beams for part of their span, usually at the centre portion of the span where the flange is in compression. A T- or L-beam is significantly stiffer than a rectangular beam of the same depth and web width. AS 3600 Clause 8.8 sets out the width of flange that can be adopted for such beams. However, over a support, a T- or L-beam will normally be considered as a rectangular beam.

The determination of the flexural strength of a T- or L-beam depends on whether the depth of the assumed rectangular compressive stress block lies within the flange thickness, t, only or if it lies both in the flange and in part of the lower section of web.

If $t \geq A_{st}\ f_{sy} / (\alpha_2\ f'_c\ b_{ef})$, the area of concrete in compression is rectangular and is within the flange. The beam may then be designed as a rectangular beam with the web width equal to flange width, b_{ef}.

If $t < A_{st}\ f_{sy} / (\alpha_2\ f'_c\ b_{ef})$, the area in compression is T- or L-shaped and includes the full depth of the flange and extends down into the web of the beam. The strength in bending has then to be calculated using two components: the compression stress in the flange outstands beyond the web and the compression stress in part of the upper section of the web including the portion in the flange.

4.3.2 Design of T- and L-beams

Spreadsheet 4.2 can be used to calculate the reinforcement requirements for a singly reinforced rectangular T- or L-beam in flexure in accordance with Flowchart 4.1. It checks minimum reinforcement, but it does not check cover, spacing requirements or detailing requirements.

The spreadsheet assumes the T- or L-beam is not over reinforced and that $k_u \leq k_{uo}$ and calculates an initial approximation of the reinforcement required as noted above for rectangular beams.

The spreadsheet then checks the flange thickness to see if $t \geq A_{st}\ f_{sy} / (\alpha_2\ f'_c\ b_{ef})$.

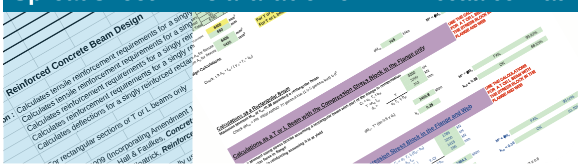
If $t \geq A_{st}\ f_{sy} / (\alpha_2\ f'_c\ b_{ef})$, the area of concrete in compression is rectangular and within the flange width. The strength of the section in bending can be obtained from the first part of Spreadsheet 4.2 for beams with the stress block within the flange, using the width of the compression flange as b_{ef}.

If $t < A_{st}\ f_{sy} / (\alpha_2\ f'_c\ b_{ef})$, then the area in compression is T- or L-shaped, ie the flange and part of the stem resist compression. The strength in bending can then be calculated from the second part of Spreadsheet 4.2 for a T- or L-beams with the stress block in the flange and part of the web. This uses the outstand of the compression flange, $b_{ef} - b_w$, times the full depth of the flange, t_f, and the dimensions of the web resisting compression is the width of the web times the effective depth, ie $b_w\,\alpha\,d_n$.

Based on the initial area of tensile reinforcement calculated and depending on whether the flange or the flange and web are in compression, the designer then enters an area of reinforcement approximating to this estimate (usually a number of bars of one size). The spreadsheet then calculates the actual moment capacity for the chosen reinforcement and for the chosen design case and compares it with the design moment.

For both design cases, the spreadsheet checks that the actual moment capacity exceeds the design moment capacity. If the actual moment capacity is less than or significantly greater than the design moment, it will be necessary to repeat the calculations with a revised area of reinforcement.

Spreadsheet 4.2 is available from www.ccaa.com.au



4.4 BEAMS IN SHEAR

4.4.1 General

The vertical shear forces in a beam are determined from the structural analysis. They vary from maximum shear at the supports at each end of the beam reducing to nil in the middle of the beam for uniformly loaded beams, to nearly uniform shear for a heavy point load in the middle of a beam. The designer needs to determine which are the critical sections for shear design. (Note that the critical sections for shear will usually differ from those for bending.)

Shear reinforcement in the form of fitments (sometimes referred to as ties, ligatures, stirrups or links) is generally provided in beams for shear (and sometimes torsion). They also support the longitudinal reinforcement within the beam. The fitments are usually vertical due to practical fixing considerations. In the past, fitments could be inclined from the vertical, while bent-up bars from the bottom reinforcement to the top reinforcement were sometimes used as shear bars. These methods are not used in modern construction because they are difficult to fix and AS 3600 does not cover bent-up bars. See Figure 4.3 for a typical arrangement of shear reinforcement.

Fitments need to anchored at each end, and for practical reasons, are typically closed. For band beams, however, open fitments are used to allow fixing of the beam reinforcement, with the slab reinforcement supporting the top bars. AS 3600 Clause 8.2.12.4 has specific requirements for anchorage with hooks and cogs, and specifies that cogs must not be located within 50 mm of any concrete surface.

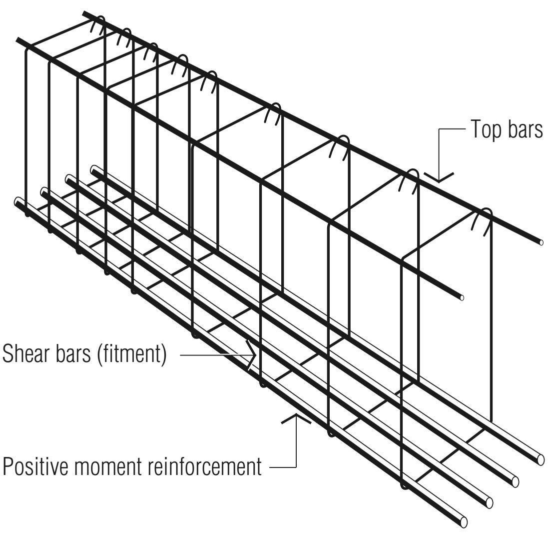


Figure 4.3 Typical arrangement of beam reinforcement

Mesh or bar (or bar made from coil) can be used for fitments. Fitments can be made from either round bar R250N, round bar R500L, deformed bar D500N or deformed bar D500L. Generally, only six sizes of fitments are available, from 6 mm to 20 mm as noted below.

Fitments are normally made from either round bar grade 500 low ductility (R500L) or deformed bar grade 500 low ductility (D500L) for the 6-, 8- or 10-mm diameter sizes or from deformed bar grade 500 normal ductility (D500N) for the 12-, 16- and 20-mm diameter sizes. Mesh is generally not used.

Designers need to specify the type and spacing of fitment they require for the beam, eg L8 fitments at 200 centres or N16 fitments in threes at 150 centres.

Note that an L8 fitment is actually 7.6 mm in diameter and it can be made from either low ductility grade 500 round or deformed bar. The smaller diameters 6, 8 and 10 L bars were previously known as wire. As a result, the designation W6, W8 and W10 is still often used for L6, L8 and L10 fitments indicating grade 500L bar (wire). Grade 250 normal ductility R6N and R10N fitments are only available in some locations.

As noted above, fitments generally have two legs and are usually of the closed type with two 135° hooks. A combination of fitments is sometimes used for beams (which might not be rectangular, eg a beam with an upstand section) or for wide shallow beams, especially if the shear is high and a considerable area of shear reinforcement is required, eg one overall fitment and two smaller fitments to form a triple fitment. Detailing of fitments is discussed in more detail in Section 13 of the *Reinforcement Detailing Handbook*[4.6].

Logical models for shear are very complex, so, like most design codes, AS 3600 Clause 8.2 uses an empirical approach to the design for shear.

The first check is to ensure the applied shear does not exceed the web-crushing limit, otherwise the beam size and or concrete strength will need to be increased.

For the determination of the shear strength of a beam without shear reinforcement, AS 3600 Clause 8.2.7 provides an empirical formula relating the shear strength to the amount of tensile reinforcement and the concrete strength. Modifying factors are included to allow for the effect of the overall beam depth, axial tension or compression in the beam, and for concentrated loads near the face of a support.

The determination of the contribution to shear strength by the shear reinforcement is based on the truss analogy method, using an angle of the concrete compression strut to the horizontal that varies from 30° when only the minimum shear reinforcement is required to 45° when the shear approaches the upper limit of web-crushing failure. *Concrete Structures*[4.4] and *Reinforced Concrete Basics*[4.5] provide more information on the concept of the truss analogy.

4.4.2 Basis of Spreadsheet for design for shear

Spreadsheet 4.3 is based on Flowchart 4.2 for reinforced concrete beams in accordance with AS 3600 Clause 8.2.

The designer has to determine the ultimate shear forces, V^*, in a beam from the analysis and determine which sections need to be designed for shear.

Analysis for shear requires the input of the material properties of the concrete and fitments, the cover and the section properties of the beam along with the area of tension reinforcement used. For T- and L-beams the flange is ignored for shear design, so they become rectangular beams.

The maximum shear (web crushing), $V_{u.max} = 0.2\, f'_c\, b_v\, d_o$, is then calculated along with the shear strength of the beam without shear reinforcement and the shear strength of the beam with minimum shear reinforcement. If the ultimate shear forces exceed the maximum shear then the concrete strength and/or the section size of the beam need to be increased.

The Standard then requires a series of design steps with various outcomes depending on which path has to be taken. The spreadsheet takes the designer through the various design steps. The first is to determine the shear strength of a beam, V_{uc}, assuming no shear reinforcement, ie:

$$V_{uc} = \beta_1\, \beta_2\, \beta_3\, b_v\, d_o\, f_{cv} \left[\frac{A_{st}}{b_v\, d_o}\right]^{1/3}$$

Then it checks the shear strength of a beam with minimum shear reinforcement where:

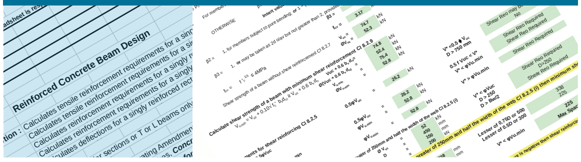
$$V_{u.min} = V_{uc} + 0.10\sqrt{f'_c}\,(b_v\, d_o) \geq V_{uc} + 0.6\, b_v\, d_o$$

The need for shear reinforcement is then determined depending on whether $V^* \leq 0.5\phi V_{uc}$ or $0.5\phi V_{uc} \leq V^* \leq \phi V_{u.min}$. For shallow beams if $V^* < \phi V_{uc}$ shear reinforcement may not be required. For beams greater than 750 mm in depth (even if $V^* \leq 0.5\phi V_{uc}$), minimum reinforcement, $A_{sv.min}$, shall be provided in accordance with AS 3600 Clause 8.2.8.

For the case where $V^* > \phi V_{u.min}$, the spreadsheet also gives fitment sizes and spacing to provide the minimum shear reinforcement of $A_{sv}/s_{min} = 0.35\, b_v / f_{sy.f}$.

The component of shear $V_{us} = V^*/\phi - V_{uc}$ is then determined along with the spacing of fitments and is assumed to be 45°, ie $\cot\theta_v = 1$, which is conservative.

Spreadsheet 4.3 is available at www.ccaa.com.au



4.5 BEAMS IN TORSION

4.5.1 General

Although torsion occurs in most concrete beams, it becomes important only in a few cases: eg a spandrel beam supporting significant offset loads (especially if it is not cast with an adjoining slab), a curved beam (again if it is not cast with an adjoining slab) or a beam where loads are offset and cause significant torsion in the beam.

AS 3600 Clause 8.3.2 states *Where torsional strength is not required for the equilibrium of the structure and the torsion in a member is induced solely by the angular rotation of adjoining members, it shall be permissible to disregard the torsional stiffness in the analysis and torsion in the member, if the torsion reinforcement requirements of Clauses 8.3.7 and the detailing requirements of Clause 8.3.8 are satisfied.*

In discussing combined bending, shear and torsion, Warner et al[4.4] note that the approach of AS 3600 is to determine if the torsion is large enough to require special reinforcement. If it is, then the reinforcement for a beam is designed separately for flexure, shear and torsion and the results combined.

For beams subject to torsion combined with bending and shear, AS 3600 Clause 8.3.3 requires the strength of a section to be determined for torsion and shear acting separately and compared to their respective factored web-crushing limits. The combined action must not exceed the following simple interaction equation:

$$\frac{T^*}{\phi T_{u.max}} + \frac{V^*}{\phi V_{u.max}} \leq 1$$

Torsional reinforcement is not required if:

$$T^* < 0.25\,\phi\, T_{uc} \quad \text{or} \quad \frac{T^*}{\phi T_{uc}} + \frac{V^*}{\phi V_{uc}} \leq 0.5$$

or where the overall depth does not exceed the greater of 250 mm and half the width of the web and

$$\frac{T^*}{\phi T_{uc}} + \frac{V^*}{\phi V_{uc}} \leq 1$$

Torsional reinforcement consists of both closed fitments and longitudinal bars and is designed using a simple truss analogy equation.

4.5.2 Basis of Spreadsheet for design for torsion

Spreadsheet 4.4 is based on Flowchart 4.3 for reinforced concrete beams in accordance with AS 3600 Clause 8.3.

The designer has to determine the ultimate torsional moment, T^*, along with the associated shear forces, V^*, in a beam from the structural analysis and determine which sections need to be designed for torsion.

Analysis of the cross-sections for torsion requires the input of the material properties of the concrete and fitments, the cover and the section properties of the beam. For T- and L-beams the flange is ignored for torsion design, so it normally becomes a rectangular beam. In addition, the longitudinal tensile reinforcement is required along with fitment diameter and a notional spacing.

The spreadsheet can be used to calculate the torsional modulus, J_t.

It then calculates $T_{u.max} = 0.2\, J_t\, f'_c$ and the maximum shear (web crushing) $V_{u.max} = 0.2\, f'_c\, b_v\, d_o$

Then it checks $\dfrac{T^*}{\phi T_{u.max}} + \dfrac{V^*}{\phi V_{u.max}} \leq 1$

If it exceeds unity, the member size and or the concrete strength will need to be increased.

The torsional strength of a beam in accordance with Clause 8.3.5 is calculated with:

$T_{uc} = 0.3 J_t \sqrt{f'_c}$ without fitments and

$T_{us} = A_{sw}\, f_{sy.f}\, 2A_t \cot \theta_t / s$ for a given fitment size and spacing.

The spreadsheet then determines the shear strength of a beam, V_{uc}, excluding shear reinforcement, ie:

$$V_{uc} = \beta_1 \beta_2 \beta_3\, b_v\, d_o\, f_{cv} \left[\frac{A_{st}}{b_v\, d_o}\right]^{1/3}$$

It then checks the requirements for torsional reinforcement in accordance with Clause 8.3.4 (a).

This includes if $T^* < 0.25\,\phi T_{uc}$ or if $\dfrac{T^*}{\phi T_{uc}} + \dfrac{V^*}{\phi V_{uc}} \leq 0.5$

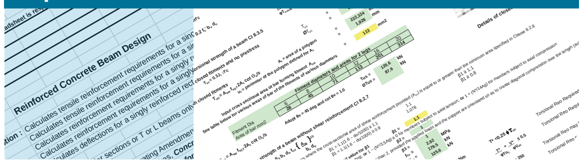
or where the overall depth does not exceed the greater of 250 mm and half the width of the web and

$$\frac{T^*}{\phi T_{uc}} + \frac{V^*}{\phi V_{uc}} \leq 1$$

If this equation is not satisfied torsional reinforcement is required.

The spreadsheet then calculates the requirements for both additional tensile and compression reinforcement and fitments as required. The total shear reinforcement and tensile and compression reinforcement can then be calculated by adding all the reinforcement required for flexure, shear and torsion.

Spreadsheet 4.4 is available at www.ccaa.com.au



4.6 DEFLECTION OF BEAMS

4.6.1 General

Reinforced concrete structures deflect or move in response to loading, changes in the environment and as the concrete dries out. It is important to ensure that the concrete structure, including the beams, performs satisfactorily during its life as discussed in Chapter 1.

Assessment of deflections of beams involves the prediction of the time-dependent behaviour of concrete; this is complicated by the fact that concrete is a non-linear material. Deflection is mainly due to cracking, tension stiffening, creep and shrinkage. This is discussed in more detail elsewhere[4.4, 4.7, 4.8].

AS 3600 Clause 8.5 gives a three-tier approach to the design for deflection as follows:

- Refined calculations
- Simplified calculations
- Deemed-to-comply span-to-depth ratios for reinforced beams. Flowchart 4.4 and Spreadsheet 4.5 set out this approach.

Refined calculations The calculation of the deflection of a beam by refined calculation needs to make allowance for cracking and tension stiffening, shrinkage and creep properties of the concrete, the expected load history, the expected construction procedure and the deflection of formwork or settlement of props during construction (particularly when the beam formwork is supported on suspended floors or beams below). This method is too complicated for most designs; specialist advice would be required if it was to be used.

Simplified calculations This is commonly known as the Branson formula and involves the calculation of a short-term and long-term component.

It was noted[4.8] that for lightly reinforced beams the Branson formula can overestimate the stiffness of a beam after cracking; the Eurocode 2 may be a better design model.

The short-term deflections due to external loads, which occur immediately on their application, are calculated using the value of E_{cj} determined in accordance with AS 3600 Clause 3.1.2 and the value of the effective second moment of area of the member, I_{ef}.

This value of I_{ef} may be determined from the values of I_{ef} at nominated cross-sections as follows:

$$I_{ef} = I_{cr} + (I - I_{cr})\,(M_{cr}/M^*_s)^3 \le I_{cf.max}$$

For beams, AS 3600 allows a simplified calculation of I_{ef} using equations 8.5.3.1 (2) and (3), ie:

$$I_{ef} = [(5 - 0.04\,f'_c)\,p + 0.002]\,b_{ef}\,d^3 \le [0.1/\beta^{2/3}]$$
$$b_{cf}\,d^3 \text{ for } p \ge 0.001\,(f'_c)^{1/3}/\beta^{2/3}$$

or

$$I_{ef} = [0.055(f'_c)^{1/3}/\beta^{2/3} - 50\,p]\,b_{cf}\,d^3 \le [0.06/\beta^{2/3}]$$
$$b_{cf}\,d^3 \text{ for } p < 0.001\,(f'_c)^{1/3}/\beta^{2/3}.$$

In the absence of more-accurate calculations, the long-term deflection due to shrinkage and creep is calculated by multiplying the short-term deflection caused by the sustained loads by a multiplier, k_{cs}, given by:

$$k_{cs} = (2 - 1.2\,A_{sc}/A_{st}) \ge 0.8$$

It should be noted that compression reinforcement will reduce k_{cs} and normally some reinforcement is provided in the compression face of a beam to support the fitments, etc that should be included.

Deemed-to-comply span-to-depth ratios for reinforced beams This method (see AS 3600 Clause 8.3.4) involves a simple calculation but is limited to beams that are

- of uniform section;
- fully propped during construction;
- subject to uniformly distributed loads only and where the imposed action (live load), q, does not exceed the permanent action (dead load), g.

Deemed-to-comply span-to-depth ratios are likely to result in conservative beam depths and the long-term deflections will be small. The simplified calculations are likely to give a more appropriate depth.

Beam deflections are deemed to comply with the requirements of AS 3600 Clause 2.3.2 if the ratio of effective span to effective depth satisfies the following equation:

$$L_{ef}/d \le \left[\frac{k_1(\Delta/L_{ef})\,b_{ef}\,E_c}{k_2\,F_{d.ef}}\right]^{1/3}$$

The serviceability load factors in accordance with AS 1170.0 Table 4.1, combined with the long-term deflection multiplier as defined in AS 3600 Clause 8.5.4, are:

$1 + k_{cs}$ for permanent actions (dead loads) for total deflection, and k_{cs} for permanent action (dead loads) for deflection that occurs after the addition or attachment of brittle partitions or finishes.

and

$\psi_s + k_{cs}\psi_1$ for imposed actions (live loads) and both total deflection and deflection that occurs after the addition or attachment of brittle partitions or finishes.

Where $k_{cs} = (2 - 1.2\,A_{sc}/A_{st}) \ge 0.8$

Again, it should be noted that compression reinforcement will reduce k_{cs} and normally some reinforcement is provided in the compression face of a beam to support the fitments, etc that should be included.

The effective design action (load), $F_{d.ef}$, is then calculated as follows:

$F_{d.ef} = (1 + k_{cs})\,g + (\psi_s + k_{cs}\psi_1)\,q$ for total deflection

or

$F_{d.ef} = k_{cs}\,g + (\psi_s + k_{cs}\psi_1)\,q$ for the deflection that occurs after the addition or attachment of brittle partitions or finishes

The formulae given for $k_1 = I_{ef}/b_{ef}d^2$ in AS 3600 Clause 8.5.4 for rectangular sections assume that $\beta = b_{ef}/b_w \ge 1.0$ and $p = A_{st}/b_{ef}\,d$ at midspan, viz:

$$k_1 = (5 - 0.04\,f'_c)\,p + 0.002 \le 0.1/\beta^{2/3}$$
$$\text{for } p \ge 0.001(f'_c)^{1/3}/\beta^{2/3}$$

or

$$k_1 = 0.055\,(f'_c)^{1/3}/\beta^{2/3} - 50\,p \le 0.06/\beta^{2/3}$$
$$\text{for } p < 0.001\,(f'_c)^{1/3}/\beta^{2/3}$$

The factor k_2 for various end-restraint conditions is:

For simply supported beams $k_2 = 5/384$

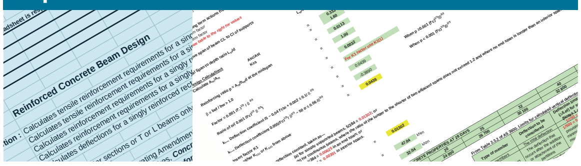
For continuous beams where the ratio of the longer to the shorter of the two adjacent spans does not exceed 1.2 and where no end span is longer than an interior span

$k_2 = 2.4/384$ in an end span

$k_2 = 1.5/384$ in an interior span

Spreadsheet 4.5 calculates the above deemed-to-comply span-to-depth ratios.

Spreadsheet 4.5 is available at www.ccaa.com.au



4.7 LONGITUDINAL SHEAR IN COMPOSITE AND MONOLITHIC BEAMS

AS 3600 Clause 8.4 applies to the transfer of longitudinal shear forces, across the interface shear planes through webs and flanges of composite and monolithic beams. Generally, for insitu monolithic beams, this is not a critical design case but designers should always satisfy themselves that their designs do comply.

The purpose of composite construction is to form a single flexural element. For concrete beams, this requires the transfer of longitudinal shear across the interface between the abutting concrete surfaces. It is important to check the shear at the interface between concrete elements that are cast at different times. Examples of this could be precast concrete beams with insitu concrete slabs on top, or beams constructed in two or more sections for particular design or construction reasons. The latter case could be an upstand beam cast after the bottom section of the slab is cast; the amount of shear to be designed for will depend on whether the lower section is propped or unpropped.

The design procedure assumes a degree of roughness of the hardened surface as set out in AS 3600 Table 8.4.3. With such composite members, the shear stress at the interface can be high and vertical reinforcement additional to any shear reinforcement is sometimes required across the interface to increase the longitudinal shear strength. See Figure 4.4 which shows a precast pile cap and roughened surface for a future section to be cast on top.

Designers should refer to AS 3600 for specific design requirements when design for longitudinal shear is required.

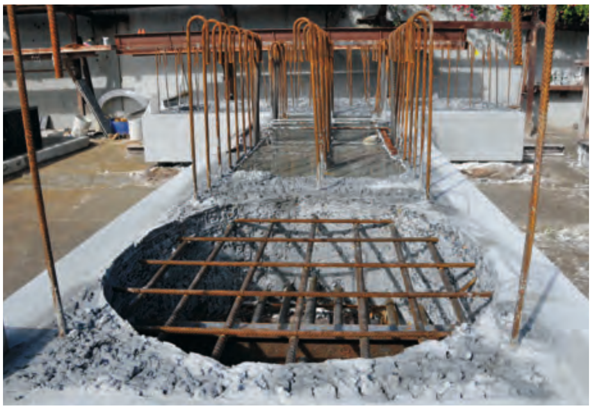


Figure 4.4 Precast pile cap with roughened surface

4.8 CRACK CONTROL IN BEAMS

It is important to understand that all reinforced concrete beams crack when subjected to design loading. Such cracking is inevitable, as it is needed to allow the tensile reinforcement to act. Uncracked beams will occur only in members which cannot flex and/or are limited in span, are significantly overdesigned or are in areas of low stress, eg the ends of simply supported beams.

For beams, the crack width should be limited to about 0.3 mm to reduce the risk of long-term corrosion. Unlike some overseas codes, AS 3600 does not provide methods for calculating crack widths but rather provides an approach in Clause 8.6 where it gives minimum requirements to limit the spacing and stress in the bars.

AS 3600 Clause 8.6.1 specifies that cracking in reinforced beams subjected to tension, flexure with tension or flexure shall be deemed to be controlled if the appropriate requirements in Items (a) and (b), and either Item (c) for beams primarily in tension or Item (d) for beams primarily in flexure are satisfied. For regions of beams fully enclosed within a building (except for a brief period during construction), and where it is assessed that crack control is not required, only items (a) and (b) need be satisfied.

For crack control in the side face of beams, AS 3600 Clause 8.6.3 requires that where the overall depth of the beam exceeds 750 mm, longitudinal reinforcement, consisting of 12-mm bars at 200-mm centres or 16-mm bars at 300-mm centres, shall be placed in each side face.

4.9 DETAILING OF BEAMS

For all reinforced concrete beams, appropriate detailing must be shown on the drawings.

AS 3600 Clause 8.1.10 sets out general rules for the detailing of flexural reinforcement for beams. It requires the tensile reinforcement to be extended from the theoretical shape of the bending moment by $D + L_{sy.t}$ past the cut-off point. Clause 8.1.10.6 sets out a deemed-to-comply arrangement for continuous beams that must be used for beams which have been designed using the simplified method of analysis.

Detailing for shear and torsion reinforcement is covered in AS 3600 Clauses 8.2.12 and 8.3.8 respectively.

Designers should also refer to Chapter 13 of the *Reinforcement Detailing Handbook*[4.6] for further guidance.

Areas that designers need to consider in detailing of reinforcement for beams include:

- The top cover to the flexural bars in beams will often be controlled by the cover to the top reinforcement in the slab and its size. This often results in covers to top beam bars being of the order of 65–100 mm rather than the 30–50 mm cover to the side or bottom reinforcement.
- Beams that intersect will have different effective depths (because of the bars being in layers) and different covers.
- Where beams are heavily reinforced, bars in layers may be required to allow placing of concrete. It is usual to use an N32 spacer bar at about 600- to 900-mm centres. This of course will reduce the effective depth for flexure which needs to be taken into account.

In addition, the following are recommended:

- Avoid lapping bars in tension in high-stress zones and ensure that all laps and splices are adequately detailed on the drawings. Typically, bottom bars are lapped at the points of support and top bars in the middle third of the beam.
- Allow adequate space (typically 70–100 mm) between top bars in beams to facilitate the placing of concrete and the use of vibrators. AS 3600 Section 17 requires the reinforcement to be placed so as to allow the concrete to closely surround it. Generally, this means a minimum clear spacing between parallel bars of the greater of 25 mm, d_b or 1.5 times the maximum nominal aggregate size.
- Try to use the same size bars in each section of the top and bottom faces as the use of multiple bar-sizes complicates the fixing on site, ie do not specify 2N36 + 1N24 + 1N20 when 3N36 bars would be appropriate. Where bars are lapped it is permissible to change the size but again keep the same size of bars for that section of the beam, eg 2N20 not 1N20 + 1N24.
- Use the same size fitment and vary the spacing to suit shear requirements (but use only a limited number of different spacings). The spacing must not exceed the limits specified in AS 3600 Clause 8.2.12.2 but may need to be less for earthquake design.
- Always look at the beam/beam and beam/column junctions especially when the column and beam widths are the same, as the beam bars and column bars will usually clash, assuming the same cover to both. Figure 4.5 illustrates the situation.

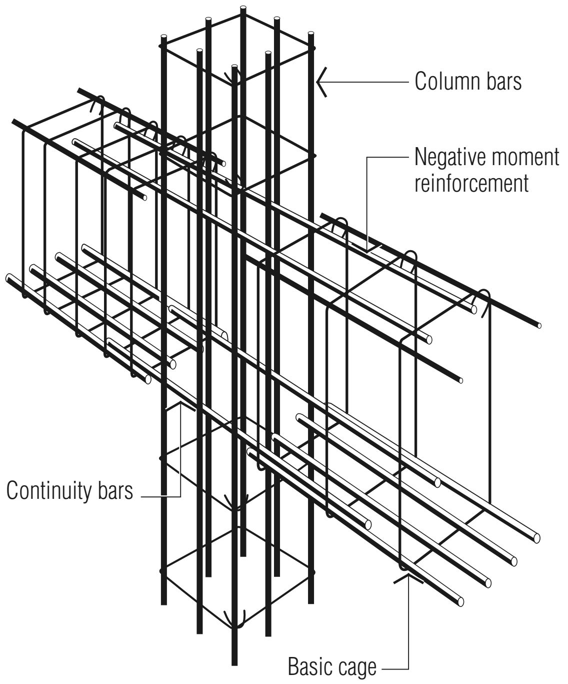


Figure 4.5 Intersection of a beam and column

- Avoid cogging bars into columns because of the congestion it will cause. Top bars can sometimes run into the slab. If cogged bars are required, consider drop-in bars.
- Provide a minimum of two bars top and bottom to support the fitments. Also provide continuity in longitudinal reinforcement at the supports with the bottom reinforcement and in the middle of the beam for the top reinforcement. 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.
- Always curtail the reinforcement where not required, eg excessive top bars in the middle of beams and excessive bottom bars at the ends of beams.
- Consider spreading top bars into the slabs at the junction of beams with columns to reduce congestion and facilitate concrete placing.
- Provide seismic detailing as required in AS 3600 Appendix C.
- Ensure that any compressive reinforcement is adequately restrained by the fitments.
- For cantilevers, ensure that the top bars are anchored well back in an area of low stress.
- If beams are shown in a schedule on the drawings then check the schedule to ensure that all detailing fits within the constructed shape.
- Always provide elevations, sections and details of complicated or unusual beams on the drawings. See Figure 4.6 for an example.

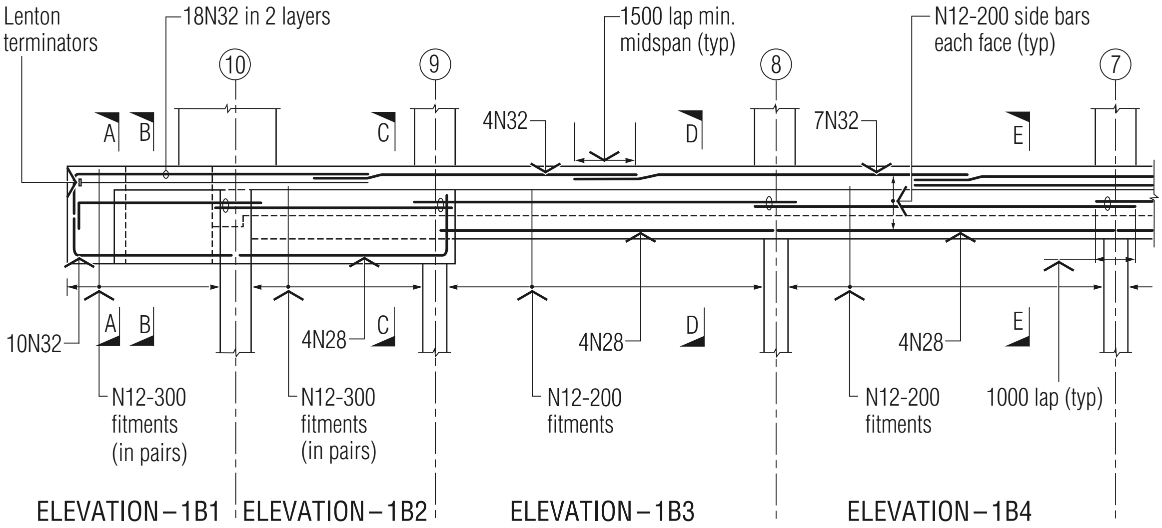


Figure 4.6 An example of a beam elevation

- Ensure that construction joints in beams are properly considered and specified.
- Provide side face reinforcement for beams deeper than 750 mm.

4.10 GENERAL GUIDANCE

The following will assist the designer in sizing and designing beams for a particular project.

- AS 3600 Clause 8.9 sets limits on the slenderness of beams
- The size of beams needs to be considered in the context of the overall building. The depth of beams generally needs to be minimised to reduce the storey height of the building and to allow building services to pass under them, generally above a ceiling.
- Where beams are exposed, eg around the perimeter of the building or opening, they should have the same depth irrespective of the span, subject to the architectural requirements.
- Generally, small horizontal penetrations in the middle third of a beam are possible. However, large horizontal penetrations through a beam will require careful design and detailing; in many cases they may not be possible. The reduced section needs to be analysed and the design checks carried out; any additional shear and crack control reinforcement required around the penetration may introduce congestion in the beam.
- Large services such as sewer, storm water or water pipes should not be built in since they will cause problems if they leak.
- Beams should be of a uniform depth in a span. Haunched beams (deepened at the ends) should be avoided if possible because of the formwork costs and extra detailing.
- If beams are notched in the middle to accommodate ductwork, shear often becomes a design issue.
- Beams which are deep or have dapped ends (reduced depth to sit on a corbel or accommodate ductwork) are usually designed using strut-and-tie methods (refer to Chapter 9).
- Linear elastic analysis is recommended for major projects and the simplified methods for small projects and simple elements, subject to the computer analyses available.

The following beam design data is divided into:

BENDING

Flowchart 4.1 pages 4.12–4.13

SHEAR

Flowchart 4.2 pages 4.14–4.15

TORSION

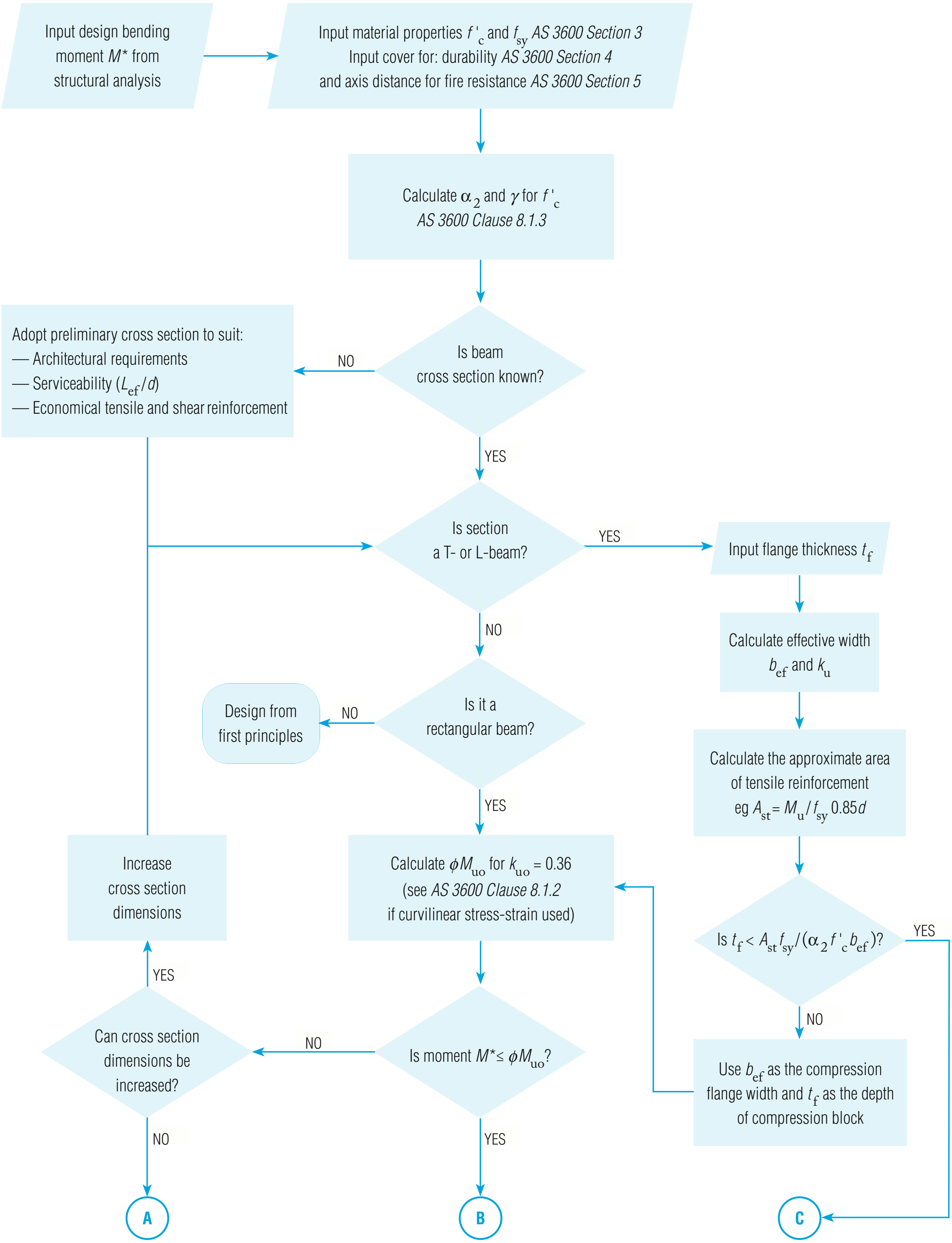
Flowchart 4.3 pages 4.16–4.17

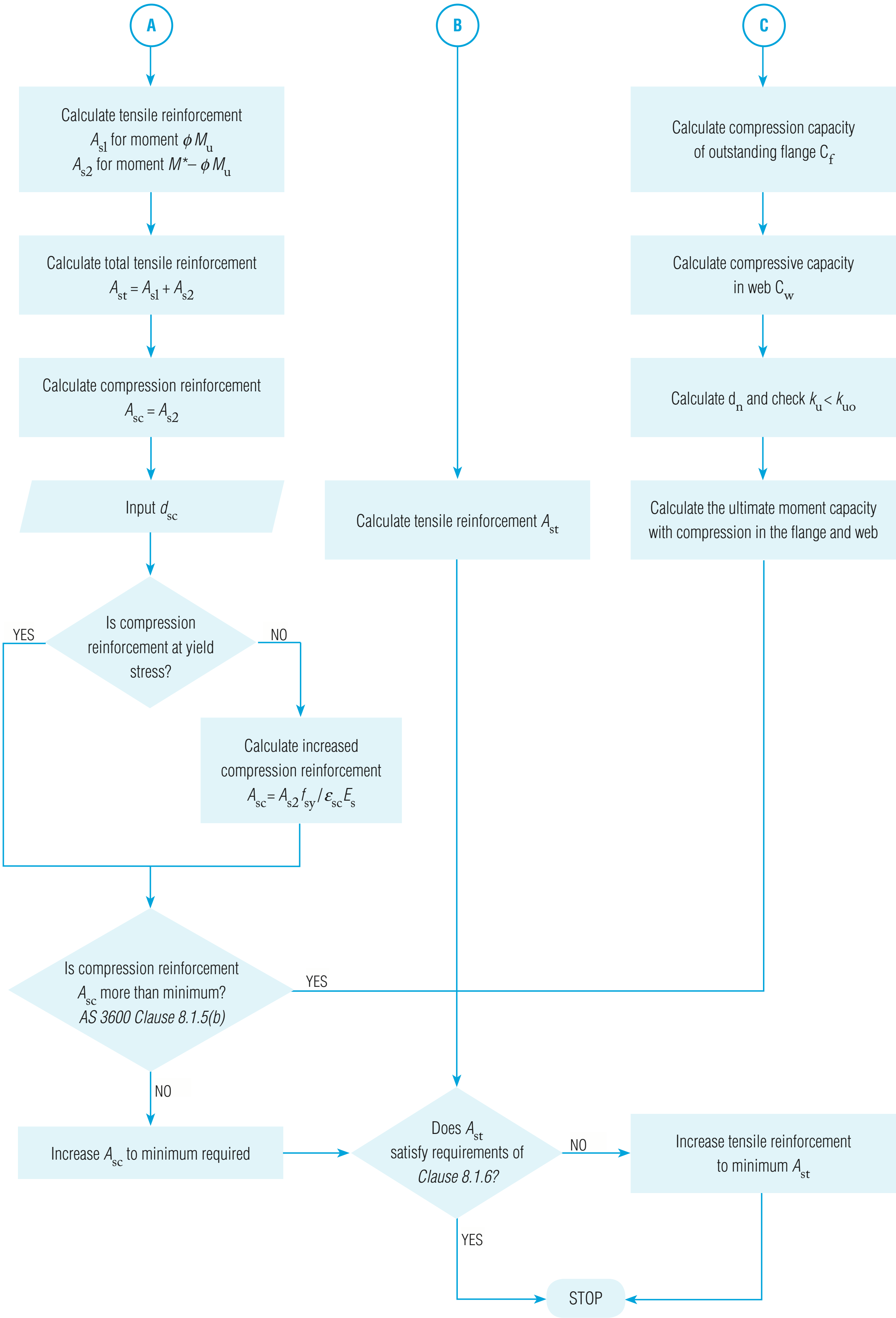
DEFLECTION

Flowchart 4.4 page 4.18

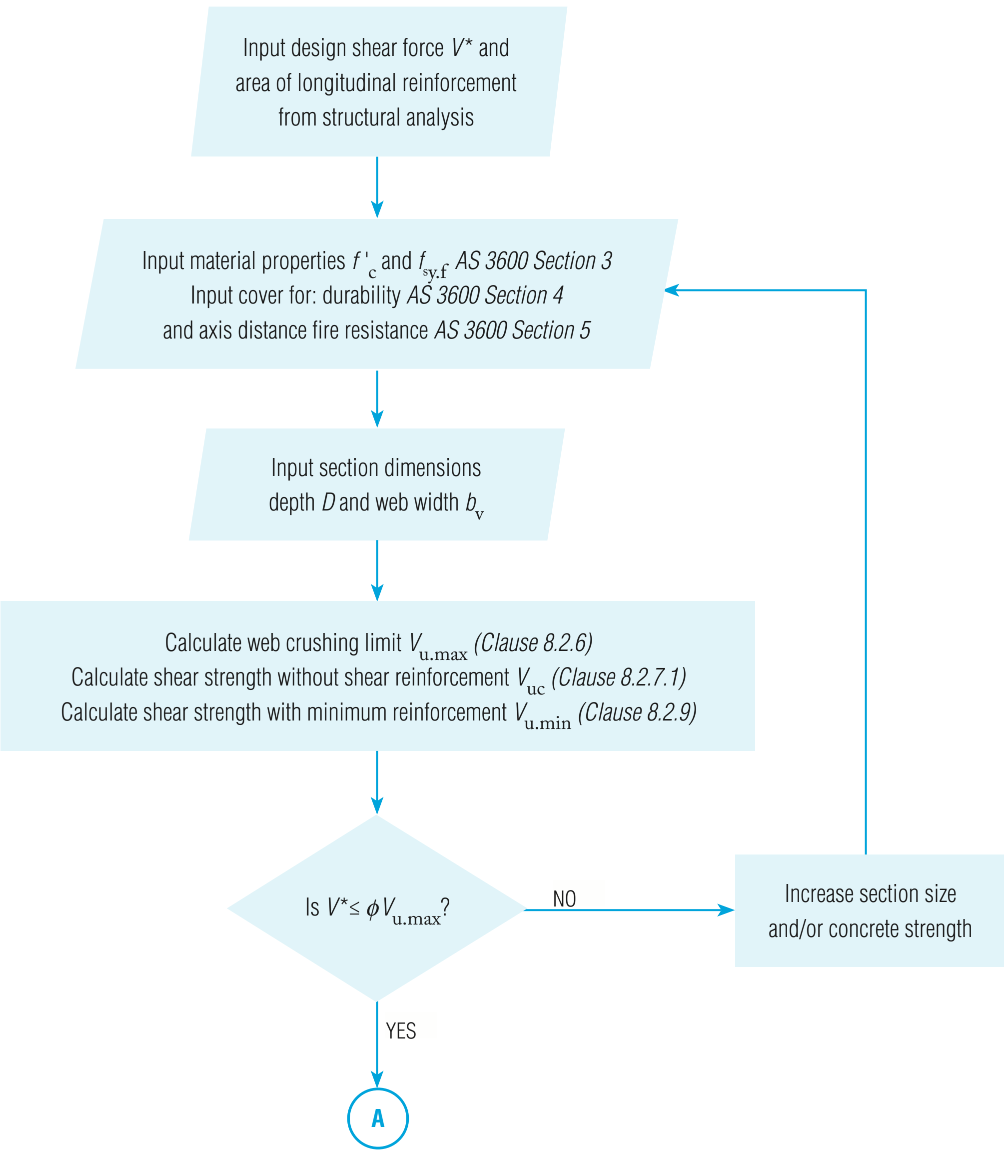
Spreadsheets 4.1, 4.2, 4.3, 4.4 and 4.5 *may be downloaded from the Cement Concrete & Aggregates Australia website* www.ccaa.com.au

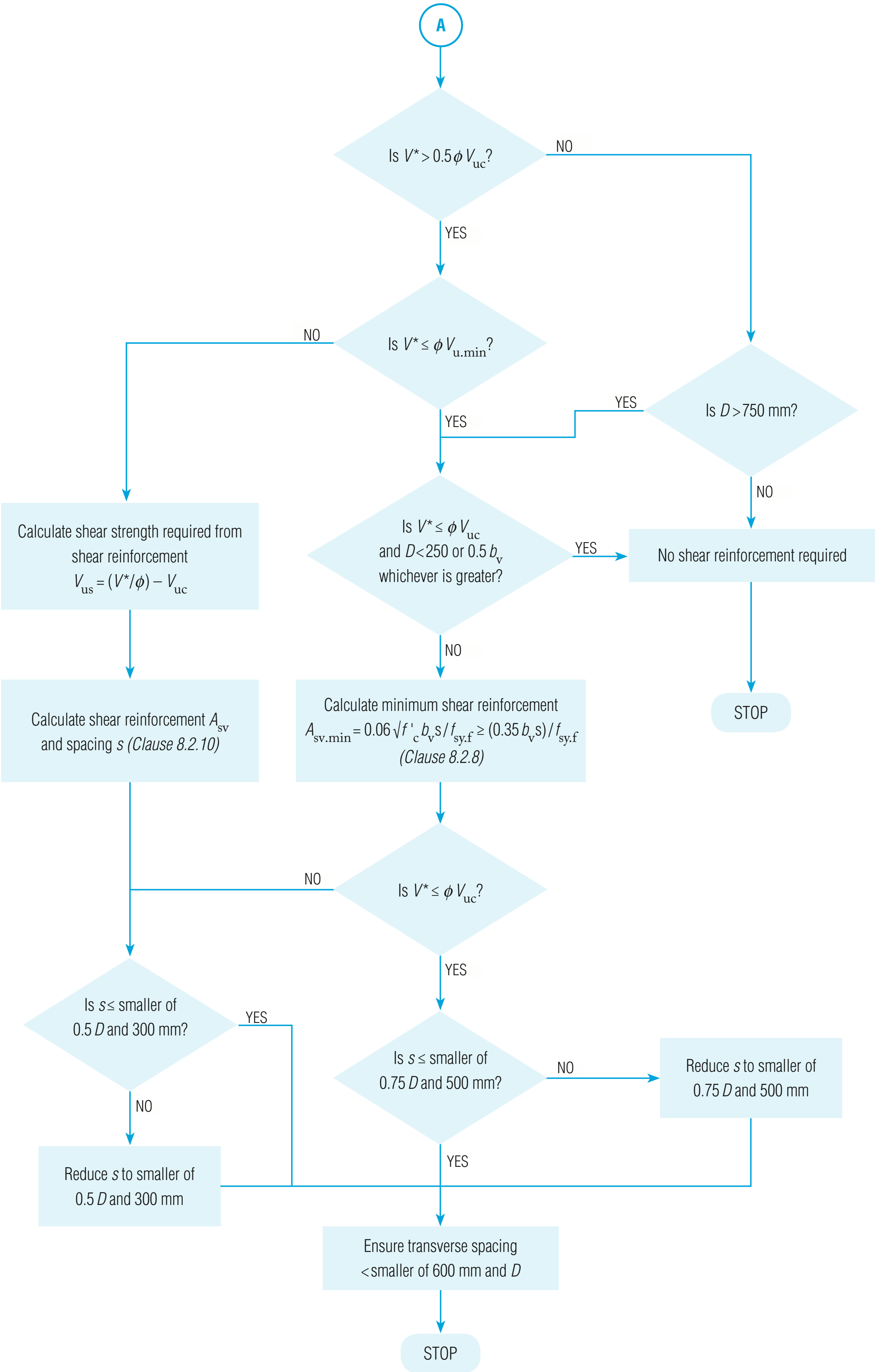
FLOWCHART 4.1 Design of reinforced concrete beams for bending *AS 3600 Clause 8.1*



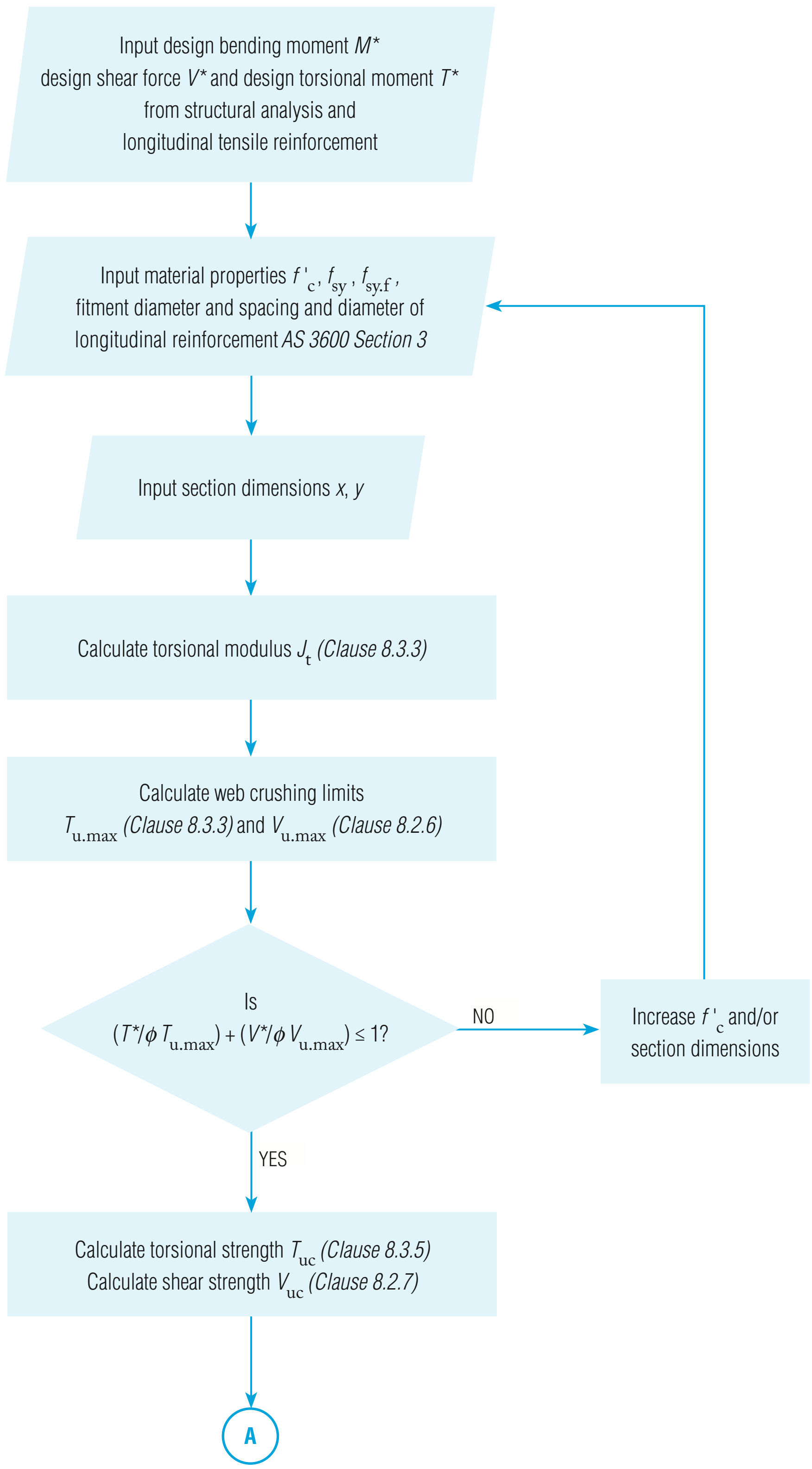

A
B
C
Calculate tensile reinforcement
A_{s1} for moment ϕM_u
A_{s2} for moment $M^* - \phi M_u$
Calculate total tensile reinforcement
$A_{st} = A_{s1} + A_{s2}$
Calculate compression reinforcement
$A_{sc} = A_{s2}$
Input d_{sc}
Is compression reinforcement at yield stress?
YES
NO
Calculate increased compression reinforcement
$A_{sc} = A_{s2} f_{sy} / \varepsilon_{sc} E_s$
Is compression reinforcement A_{sc} more than minimum?
AS 3600 Clause 8.1.5(b)
YES
NO
Increase A_{sc} to minimum required
Calculate tensile reinforcement A_{st}
Does A_{st} satisfy requirements of *Clause 8.1.6?*
NO
YES
Increase tensile reinforcement to minimum A_{st}
STOP
Calculate compression capacity of outstanding flange C_f
Calculate compressive capacity in web C_w
Calculate d_n and check $k_u < k_{uo}$
Calculate the ultimate moment capacity with compression in the flange and web

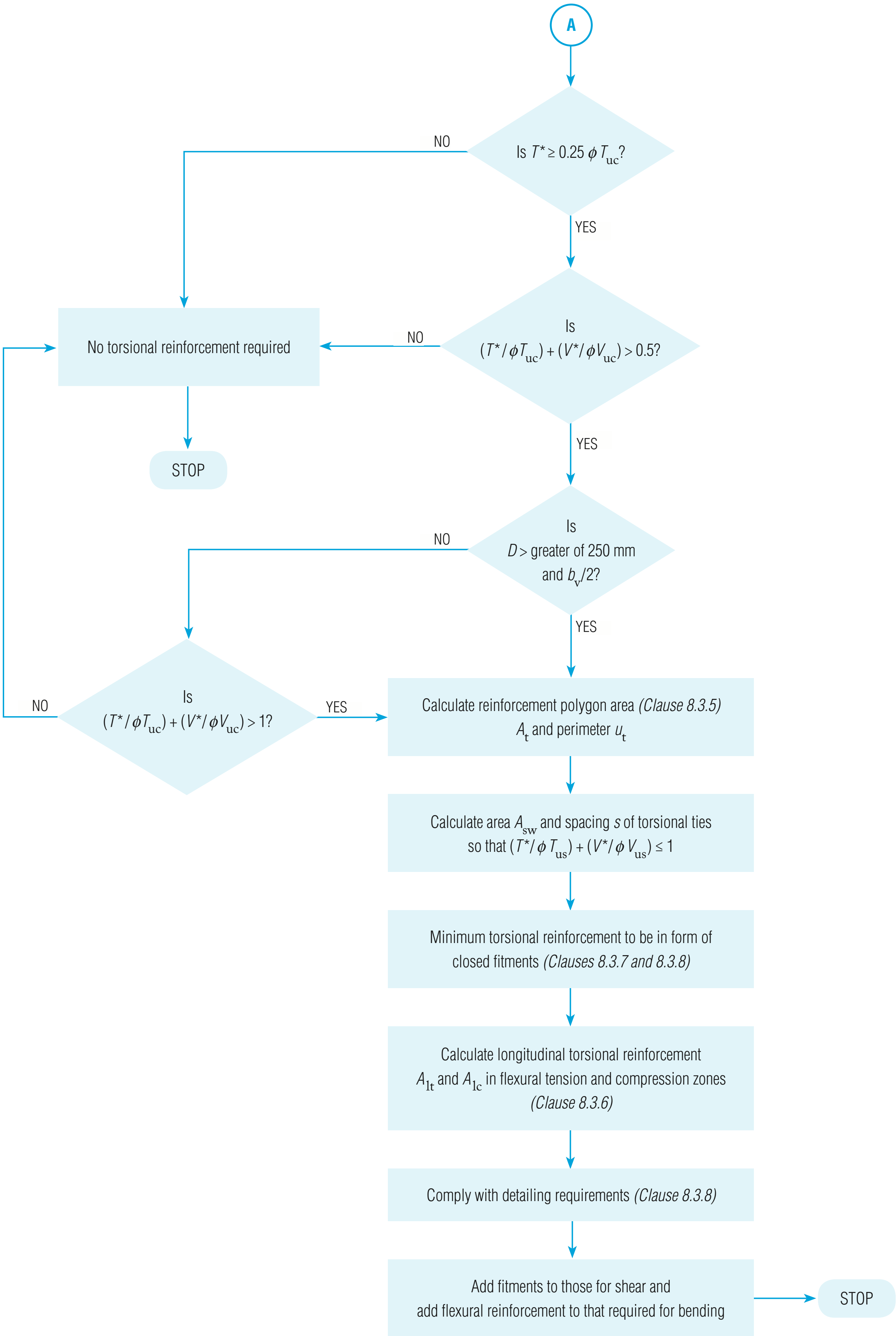
FLOWCHART 4.2 Design of reinforced concrete beams for shear *AS 3600 Clause 8.2*



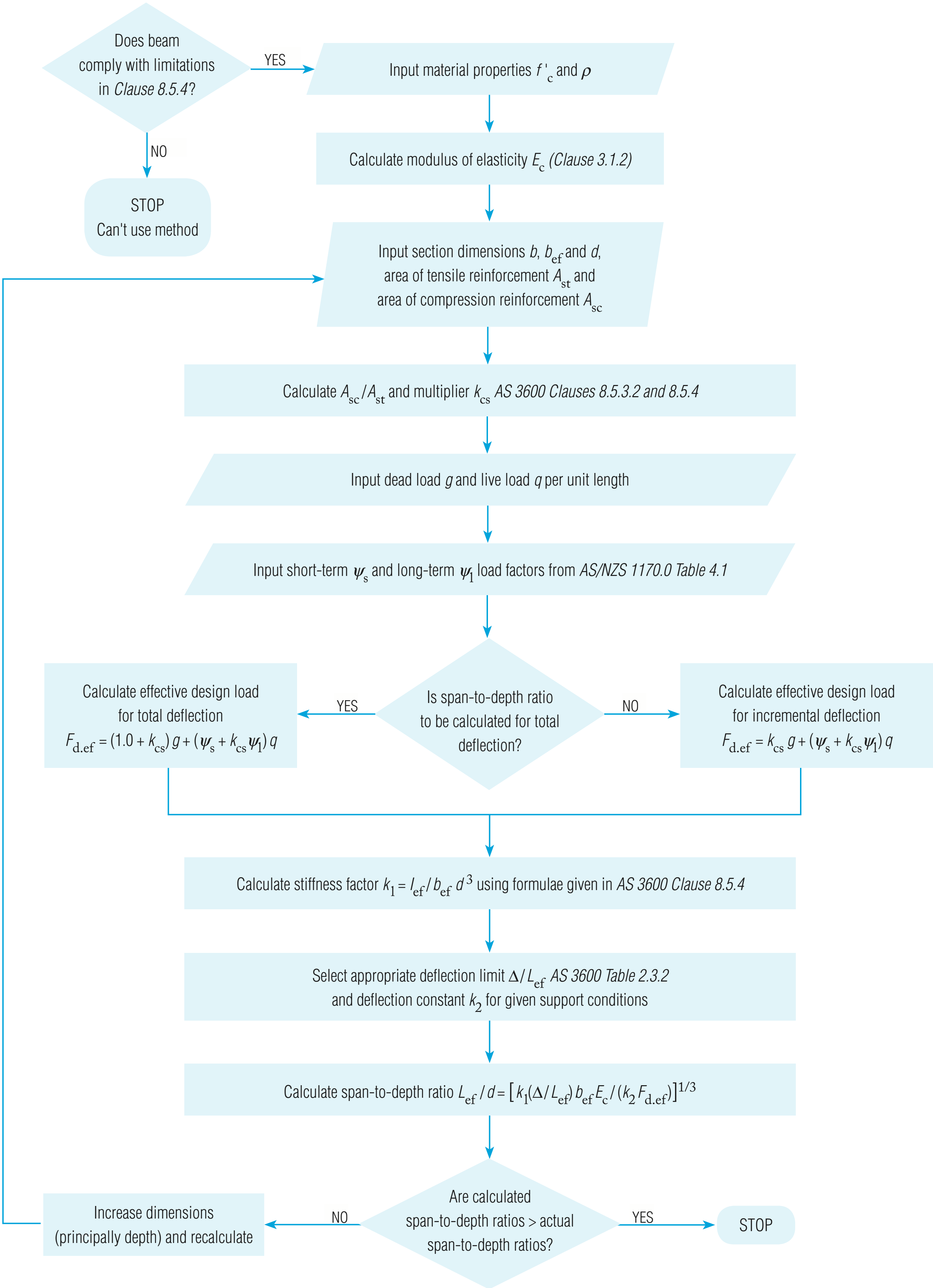

A
Is $V^* > 0.5\phi V_{uc}$?
NO
YES
Is $V^* \leq \phi V_{u.min}$?
NO
YES
Is $D > 750$ mm?
YES
NO
Calculate shear strength required from shear reinforcement $V_{us} = (V^*/\phi) - V_{uc}$
Is $V^* \leq \phi V_{uc}$ and $D < 250$ or $0.5\,b_v$ whichever is greater?
YES
No shear reinforcement required
NO
STOP
Calculate shear reinforcement A_{sv} and spacing s *(Clause 8.2.10)*
Calculate minimum shear reinforcement $A_{sv.min} = 0.06\sqrt{f'_c}\,b_v s / f_{sy.f} \geq (0.35\,b_v s)/f_{sy.f}$ *(Clause 8.2.8)*
NO
Is $V^* \leq \phi V_{uc}$?
YES
Is $s \leq$ smaller of 0.5 D and 300 mm?
YES
NO
Is $s \leq$ smaller of 0.75 D and 500 mm?
NO
Reduce s to smaller of 0.75 D and 500 mm
YES
Reduce s to smaller of 0.5 D and 300 mm
Ensure transverse spacing < smaller of 600 mm and D
STOP

FLOWCHART 4.3 Design of reinforced concrete beams for torsion *AS 3600 Clause 8.3*




A
Is $T^* \geq 0.25\,\phi\,T_{uc}$?
NO
YES
No torsional reinforcement required
NO
Is
$(T^*/\phi T_{uc}) + (V^*/\phi V_{uc}) > 0.5$?
STOP
YES
NO
Is
$D >$ greater of 250 mm
and $b_v/2$?
YES
NO
Is
$(T^*/\phi T_{uc}) + (V^*/\phi V_{uc}) > 1$?
YES
Calculate reinforcement polygon area *(Clause 8.3.5)*
A_t and perimeter u_t
Calculate area A_{sw} and spacing s of torsional ties
so that $(T^*/\phi\,T_{us}) + (V^*/\phi\,V_{us}) \leq 1$
Minimum torsional reinforcement to be in form of
closed fitments *(Clauses 8.3.7 and 8.3.8)*
Calculate longitudinal torsional reinforcement
A_{lt} and A_{lc} in flexural tension and compression zones
(Clause 8.3.6)
Comply with detailing requirements *(Clause 8.3.8)*
Add fitments to those for shear and
add flexural reinforcement to that required for bending
STOP

FLOWCHART 4.4 Deemed to comply span-to-depth ratios for reinforced beams *AS 3600 Clause 8.5.4*



REFERENCES

4.1 AS/NZS 4671 *Steel reinforcing materials* Standards Australia, 2001.

4.2 AS 3600 *Concrete structures* Standards Australia, 2009.

4.3 *Building Code of Australia* Australian Building Codes Board, 2010.

4.4 Warner RF, Rangan BV, Hall AS and Faulkes KA *Concrete Structures* Longman, 1998.

4.5 Foster SJ, Kilpatrick AE and Warner RF *Reinforced Concrete Basics* 2nd Ed, Pearson, 2010.

4.6 *Reinforcement Detailing Handbook* (Z06) 2nd Ed, Concrete Institute of Australia, 2010.

4.7 *Lecture 4, Design for Serviceability* National Seminar Series on AS 3600—2009, CIA, EA and CCAA, 2009.

4.8 *Design for Deflection and Crack Control* National Seminar Series on Serviceability, Concrete Institute of Australia, 2010.

BLANK PAGE

Chapter 5_Suspended slabs

5.1 GENERAL

Suspended slabs can be both floors and roofs. They are usually relatively thin members acting in flexure supporting their own mass and other vertical loads and transferring lateral loads into walls and columns. In particular, deflections, shear at supported edges, shear at columns and the design of free edges for slabs always require careful consideration.

The type of suspended slab chosen will depend on various architectural, structural and construction considerations, which are discussed in the *Guide to Long-Span Concrete Floors*[5.1]. *Reinforced Concrete Basics*[5.2] also has a chapter on the design of suspended slabs and background on flat slabs. (For slabs-on-ground, reference should be made to CCAA's *Guide to Industrial Floors and Pavements*[5.3].)

Types of suspended slabs include:

- Flat plate
- Flat slab (with drop panels at columns)
- Ribbed slab (waffle slab or similar where the ribs are at close centres)
- Slab and joist
- Beam and slab
- Band beam and slab
- Precast and composite.

However, this chapter does not cover precast and composite slabs, only insitu suspended reinforced concrete slabs.

'Flat plate', 'flat slab', 'beam and slab', and 'band beam and slab' are the simplest types of insitu slabs. They generally require only simple formwork and can be used economically for spans up to about 6–8 m, but can span further if required. Deflection will often govern the thickness of slabs (see *Guide to Long-Span Concrete Floors* for guidance on span ranges for the various slab types).

Generally, for spans over 8 m, insitu post-tensioned slabs or pretensioned precast systems should be considered. Because deflections usually govern the design of slabs, they are typically not heavily reinforced for flexure and the values of k_u tend to be low. Where edges of slabs are visible and deflection is obvious, consideration should be given to using an edge beam or similar stiffening member.

The span-to-depth ratios shown in Table 5.1 can be used for initial sizing of suspended slabs.

The bending moments in insitu concrete slabs are determined from the chosen method of structural analysis as discussed in Chapter 1. Concrete slabs can have the maximum bending moment at the middle of the span for a simply-supported slabs or varying moments (positive and negative) across a span based on the extent of continuity, spans and any cantilevers as analysed. It is the responsibility of the designer to establish the critical section(s) for bending and shear and the design ultimate moments and to design ultimate shears at these sections.

AS 3600 Clause 9.5 requires vibrations in slabs to be considered and appropriate action taken where necessary.

Designers should be aware that concrete slabs are not necessarily waterproof even with high levels of reinforcement. Because of the greater control of cracking, post-tensioned slabs perform better than reinforced slabs in this respect. Careful detailing, particularly of joints, is nevertheless always necessary if the risk of water penetration is to be minimised in exposed slabs unprotected by an applied waterproofing membrane or similar.

Section F1 in Volume 1 of the BCA covers the requirements for damp-proofing and waterproofing in Class 2–9 buildings. These minimum requirements may sometimes need to be exceeded.

TABLE 5.1 Span-to-depth ratios for reinforced concrete slabs

Type of slab	Simply-supported span	End span	Internal span
One-way slab	24	26	28
Flat plate	28	30	32
Flat slab	28	32	36

5.2 APPLICABILITY TO DUCTILITY CLASSES OF REINFORCING STEEL

Charts and Tables are provided for Ductility Class L and N reinforcing steel.

Designers need to remember that AS 3600[5.4] imposes limitations on the use of Ductility Class L reinforcing steel. Therefore, it is the designer's responsibility to ensure that the ductility class of the reinforcement specified is appropriate to the situation and member being designed. Other clauses in the Standard impose further restrictions on its use.

The capacity reduction factor, ϕ, for a strength check using a linear elastic analysis for Ductility Class L reinforcement (ie reinforcing mesh) is in the range of 0.60–0.64 compared to that for Ductility Class N reinforcement (ie bar) of 0.6–0.8 which reflects its lower ductility, see AS 3600 Table 2.2.2. As a result, using Ductility Class L mesh reinforcement may require up to 25% more area of reinforcement for a chosen design moment, than is required when using Ductility Class N bar reinforcement.

5.3 FLEXURAL REINFORCEMENT

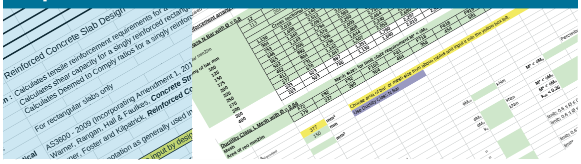
5.3.1 Spreadsheet 5.1 for flexure

Spreadsheet 5.1 can be used to calculate the reinforcement requirements for reinforced rectangular concrete slabs in flexure. The concrete strength and ϕ value are input. The spreadsheet checks the minimum reinforcement to AS 3600 Clause 9.1.1 and assumes that $k_{uo} \leq 0.36$, but it does not check cover, spacing requirements, crack control, shrinkage and temperature reinforcement or provide the detailing requirements to AS 3600.

As *Reinforced Concrete Basics* notes, for an under-reinforced section, the ultimate moment capacity, M_u is approximately equal to 0.85 $A_{st}\, f_{sy}\, d$ (within about 10% of a more accurate calculation), ie it is independent of the concrete strength and width of the slab. This approximation is used to estimate the area of tensile reinforcement required. Then the spreadsheet calculates the actual moment capacity and compares it with the design moment, which has been input.

If the calculated moment is less or significantly greater than the design moment then a further iteration with the input of a revised reinforcement area will be required.

Spreadsheet 5.1 is available at www.ccaa.com.au



5.3.2 Basis of Charts 5.1 to 5.8

These charts for the strength of slabs in bending are based on the principles set out in AS 3600 Clauses 8.1.2 and 8.1.3 with a maximum strain in the extreme compression fibre of the concrete of 0.003 and the appropriate capacity reduction factor, ϕ, depending on whether Ductility Class N bar reinforcement or Ductility Class L mesh reinforcement is used.

The Standard does not allow the use of Ductility Class L bar as main reinforcement. Charts 5.1 to 5.4 are for Ductility Class N bar reinforcement (capacity reduction factor ϕ = 0.8) while Charts 5.5 to 5.8 are for Ductility Class L mesh reinforcement (capacity reduction factor ϕ = 0.64).

The stress in the equivalent stress block is f'_c multiplied by a factor $\alpha_2 = 1.0 - 0.003\, f'_c$ (within the limits $0.67 \leq \alpha_2 \leq 0.85$). From 20 to 50 MPa, $\alpha_2 = 0.85$ but decreases after this for higher strength concrete to $\alpha_2 = 0.7$ for 100-MPa concrete. The charts are limited to 50 MPa with $\alpha_2 = 0.85$, which will cover most design situations.

The design bending moment for a given area of reinforcement in a one-metre width is derived from the equation:

$$M^* = \phi f'_c\, q\, (1 - q/1.7) d^2/1000 \text{ kN.m/m}$$

where

ϕ = 0.8 or 0.64 as appropriate

$q = p\, f_{sy}/f'_c$

$p = A_{st}/(1000d)$

and

f_{sy} = 500 MPa

At the ductile limit, ie $k_{uo} = 0.36$:

$$p_{max} = 0.85\, \gamma f'_c/f_{sy}\, k_{uo} = 0.306\, \gamma f'_c/f_{sy}$$

and

$\gamma = 1.05 - 0.007\, f'_c\, \gamma$ (within the limits $0.67 \leq \gamma \leq 0.85$) and γ varies from 0.85 for f'_c = 20 MPa to 0.7 for f'_c = 50 MPa and is 0.67 for $f'_c \geq 65$ MPa.

then

$$\phi M_{ud}/bd^2 = \phi\, 0.306\, \gamma f'_c (1 - 0.18\gamma)$$

As reinforcing mesh comes in standard sheet sizes, the moment capacity of slabs reinforced with mesh is limited by the various sizes of mesh commercially available. Slabs with bar reinforcement can have larger moment capacities because of the higher value of the capacity reduction factor, ϕ, and the fact that a greater area of reinforcement can be provided with larger bar sizes and/or closer spacings. In addition, where high levels of reinforcement are required for crack control for flexure, shrinkage and temperature effects, standard meshes may not be adequate in the secondary direction without additional bar reinforcement.

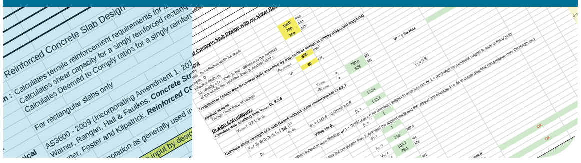
Note that for each chart the minimum reinforcement for a particular slab needs to be checked along with the minimum reinforcement for crack control for flexure, shrinkage and temperature effects. See AS 3600 Clauses 9.1.1 and 9.4 for when these requirements are applicable.

5.4 SLABS IN SHEAR

5.4.1 Spreadsheet 5.2 for slabs with shear

Spreadsheet 5.2 can be used to calculate the shear capacity at supported edges of slabs where shear failure can occur across the width of the slab, without shear reinforcement. Designers should note AS 3600 requirements for the reinforcement to be properly anchored; this may require a hook or cog at simply supported edges for bar reinforcement or the mesh anchored sufficiently beyond the shear plane.

Spreadsheet 5.2 is available at www.ccaa.com.au



5.4.2 Basis of Charts 5.9 to 5.12

These charts give punching shear strength for slabs at circular columns with no moment transfer and with no shear head, based on the equation in AS 3600 Clause 9.2.3(a):

$$V^* = \phi\, u\, d_{om}\, f_{cv}$$

where

$\phi = 0.7$

$d_{om} = d$ for uniform slabs

$f_{cv} = 0.17(1+2/\beta_h)\sqrt{f'_c} \leq 0.34\sqrt{f'_c}$

$\beta_h = 1.0$, for circular columns

$\therefore f_{cv} = 0.34\sqrt{f'_c}$; and the critical shear perimeter

$u = \pi$ (column diameter + d)

and

$M^*_v = 0$

5.4.3 Basis of Chart 5.13

This chart gives punching shear strength for slabs at rectangular columns with no moment transfer or shear head, based on the equation in AS 3600 Clause 9.2.3(a):

$$V^* = \phi\, u\, d_{om}\, f_{cv}$$

where

$\phi = 0.7$

$f_{cv} = 0.17(1+2/\beta_h)\sqrt{f'_c} \leq 0.34\sqrt{f'_c}$

β_h = (longest dimension of the effective loaded area, Y) / (shortest dimension of the effective loaded area, X)

and the critical shear perimeter

$u = 2(Y + X) + 4d_{om}$

and

$M^*_v = 0$

Figure 5.1 Long-term deflection of unsupported edge of insitu concrete slab

5.5 DEFLECTION OF SLABS

5.5.1 General

Deflection is probably the most important design criterion for slabs. Control of deflection is discussed in general terms in Section 1.4.3. Designers must review all slabs and satisfy themselves that the allowable deflections or span-to-depth ratios to be used are appropriate for the location. Figure 5.1 illustrates long-term deflection of unsupported edge of slab.

As noted earlier for beams, AS 3600 Clause 9.3 has a three-tier approach to deflection of slabs as follows:

- Refined calculation
- Simplified calculation
- Deemed-to-comply span-to-depth ratios for reinforced slabs.

Refined calculation This method is too complicated for most design. AS 3600 Clause 9.3.2 requires at least six items to be considered; specialist advice is usually required if this method is to be used.

Simplified calculation This involves the calculation of a short-term and long-term component using AS 3600 Clause 9.3.3 which in turn refers to AS 3600 Clause 8.5.3 for beams. This method will give thinner slabs than the deemed-to-comply solutions, so it should be used where possible, even though it is more tedious when calculated by hand. A number of the available commercial software programs will carry out these design checks.

Deemed-to-comply span-to-depth ratios for reinforced slabs This method is set out in AS 3600 Clause 9.3.4 and involves relatively straightforward calculations but will generally give more conservative results. It is limited to slabs of uniform section and that are:

- fully propped during construction;

- subject only to uniformly distributed loads and where the imposed action (live load), q, does not exceed the permanent action (dead load), g.

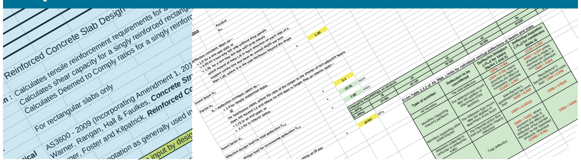
Slab deflections shall be deemed to comply with the requirements of AS 3600 Clause 2.3.2 if the ratio of effective span to effective depth satisfies the following equation:

$$L_{ef}/d \leq k_3\, k_4 \left[\frac{(\Delta/L_{ef})\,1000\,E_c}{F_{d.ef}}\right]^{1/3}$$

5.5.2 Spreadsheet 5.3 for deemed-to-comply span-to-depth ratios

Spreadsheet 5.3 can be used to calculate the deemed-to-comply span-to-depth ratios for reinforced concrete slabs based on the method set out in AS 3600 Clause 9.3.4.

Spreadsheet 5.3 is available at www.ccaa.com.au



5.5.3 Basis of Chart 5.14

This chart gives the proportion of load carried in the shorter span direction, L_x, for slabs supported on four sides as specified in AS 3600 Clause 9.3.3(b) for slab deflections by simplified calculations. The curves are derived from the equation:

$$\text{Proportion of load in } L_x \text{ direction} = 1/(\alpha L_x^{\,4}/L_y^{\,4} + 1)$$

where

α = values in AS 3600 Table 9.3.3 for various conditions of edge continuity.

5.5.4 Basis of Chart 5.15

The curves in this chart were derived for the slab-system multiplier k_4 (not the deflection coefficient) given in AS 3600 Table 9.3.4.2 , for each of the edge conditions listed. (Note that a value has been interpolated for $L_y/L_x = 0.75$.)

5.5.5 Basis of Table 5.2

The moments of inertia of cracked reinforced concrete slabs one metre wide provided in this table were derived from the formula:

$$I_{cr} = 1000\,d^3[k^3/3 + np(1-k)^2]$$

where

$k = [(np)^2 + 2np]^{0.5} - np$

n = modular ratio = E_s/E_c

$p = A_{st}/1000d$

I_{cr} is used in calculating I_{ef} which in turn is used in determining deflections by the simplified calculation method as set out in AS 3600 Clause 9.3.3.

5.6 CRACK CONTROL OF SLABS

AS 3600 Clause 9.4 has additional requirements for crack control in slabs both for flexure and for shrinkage and temperature effects irrespective of the ductility class of reinforcement. Generally, crack control reinforcement is provided in both faces of the slab except when the slab is very thin.

For flexure where the slab is internal in a building (except for a brief period during construction), AS 3600 requires the minimum area of reinforcement to be in accordance with Clause 9.1.1 and the centre-to-centre spacing of bars in each direction to not exceed the lesser of $2.0D_s$ or 300 mm. Bars with a diameter less than half the diameter of the largest bar in the cross-section shall be ignored.

For flexure with slabs exposed to the weather, in addition to the requirements above, AS 3600 requires compliance with Clause 9.4.1 (c) and (d); this involves limiting the stress in the reinforcing steel in flexure, depending on the size and spacing of the reinforcement.

For crack control in the primary direction, no additional reinforcement is required to control expansion or contraction cracking if the area of reinforcement in the direction of the span of a one-way slab, or in each direction of a two-way slab, is not less than:

- the area required by Clause 9.1.1; and
- 75% of the area required by one of Clauses 9.4.3.3 to 9.4.3.5, as appropriate.

For crack control and shrinkage where the slab is free to expand or contract in the secondary direction, the minimum area of reinforcement in that direction shall be $1.75\,D$ mm^2/m width.

The requirements for crack control and shrinkage with restrained slabs in the secondary direction inside a building are:

- Where a minor degree of control over cracking is required, A_{st} must be at least $1.75\,D$ mm^2/m width.
- Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view, A_{st} must be at least $3.5\,D$ mm^2/m width.
- Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes, A_{st} must be at least $6.0\,D$ mm^2/m width.

The requirements for crack control and shrinkage with restrained slabs in the secondary direction elsewhere and in exposure classification A1 and A2 are:

- Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view, A_{st} must be at least $3.5\,D$ mm^2/m width.

TABLE 5.2 Moment of inertia of cracked reinforced concrete sections I_{cr} per metre width (mm^4 x 10^6)

	np												
d (mm)	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.15	0.20	*d* (mm)
80	4	8	11	14	17	20	22	25	27	29	38	46	80
90	6	11	16	20	24	28	32	35	38	41	55	66	90
100	8	15	22	28	33	38	43	48	52	57	75	91	100
110	11	20	29	37	44	51	58	64	70	75	100	121	110
120	14	27	38	48	57	66	75	83	90	98	130	157	120
130	18	34	48	61	73	84	95	105	115	124	165	199	130
140	23	42	60	76	91	105	119	131	144	155	206	249	140
150	28	52	74	93	112	130	146	162	177	191	254	306	150
160	34	63	89	113	136	157	177	196	214	231	308	372	160
170	41	76	107	136	163	189	213	235	257	278	369	446	170
180	48	90	127	162	194	224	252	279	305	330	438	529	180
190	57	106	150	190	228	263	297	328	359	388	515	623	190
200	66	123	174	222	266	307	346	383	418	452	601	726	200
210	77	143	202	257	308	355	401	444	484	523	696	841	210
220	88	164	232	295	354	409	461	510	557	602	800	966	220
230	101	187	265	337	404	467	526	583	636	688	914	1104	230
240	115	213	301	383	459	530	598	662	723	781	1039	1255	240
250	130	240	341	433	519	600	676	748	817	883	1174	1418	250
260	146	271	383	487	584	674	760	842	919	993	1321	1595	260
270	163	303	429	545	654	755	851	943	1029	1112	1479	1786	270
280	182	338	479	608	729	842	950	1051	1148	1241	1650	1992	280
290	202	375	532	676	810	936	1055	1168	1276	1378	1833	2213	290
300	224	416	589	748	897	1036	1168	1293	1412	1526	2029	2450	300
310	247	459	649	825	989	1143	1289	1427	1558	1684	2239	2704	310
320	272	504	714	908	1088	1257	1417	1569	1714	1852	2463	2974	320
330	298	553	783	996	1193	1379	1555	1721	1879	2031	2701	3262	330
340	326	605	857	1089	1305	1508	1700	1882	2056	2221	2954	3567	340
350	356	660	935	1188	1424	1645	1855	2053	2242	2423	3222	3891	350
360	387	718	1017	1293	1549	1790	2018	2234	2440	2637	3506	4234	360
370	420	780	1104	1403	1682	1944	2191	2426	2649	2862	3807	4597	370
380	455	845	1196	1520	1822	2106	2374	2628	2870	3101	4124	4980	380
390	492	913	1293	1643	1970	2276	2566	2841	3102	3352	4458	5384	390
400	531	985	1395	1773	2125	2456	2768	3065	3347	3617	4810	5809	400

TABLE 5.3 Total area of reinforcement, A_{st}, for crack control and shrinkage (mm^2/m width)

	Depth of slab, *D* (mm)															
A_{st} (mm^2/m width)	100	110	120	130	140	150	160	170	180	190	200	210	220	230	240	250
1.75 D	175	193	210	228	245	263	280	298	315	333	350	368	385	403	420	438
3.5 D	375	413	450	488	525	563	600	638	675	713	750	788	825	863	900	938
6.0 D	600	660	720	780	840	900	960	1020	1080	1140	1200	1260	1320	1380	1440	1500

- Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes, A_{st} must be at least $6.0D$ mm^2/m width.

For restrained slabs in the secondary direction in exposure classification B1, B2, C1 and C2, A_{st} must be at least $6.0D$ mm^2/m width, which can be a significant amount of reinforcement.

Table 5.3 shows the various areas of reinforcement per metre width required for crack control and shrinkage in slabs up to 250 mm deep.

5.7 DETAILING OF SLABS

For all reinforced concrete slabs, appropriate detailing must be shown on the drawings.

AS 3600 Clause 9.1.3 sets out the detailing of flexural reinforcement for slabs. This requires the determination of the shape of the theoretical bending moment and the extending of the tensile reinforcement by the depth of the section D past the cut-off point. The clause also includes other requirements and a deemed-to-comply arrangement for continuous one-way and two-way slabs using the simplified method of analysis.

Detailing for shear reinforcement is covered in AS 3600 Clause 9.2.6. Wherever possible, shear reinforcement in slabs should be avoided, because of the difficulty in fixing it in thin members. Allowing for cover and fitment sizes including bends, a slab at least 200–300 mm thick would be required if shear reinforcement is required. It is usually more economical to increase the depth of slab and/or the concrete strength.

Designers should also refer to Chapter 14 of the *Reinforcement Detailing Handbook*[5.5] for further guidance.

Areas that designers need to consider in detailing of reinforcement for slabs include:

- Minimum reinforcement in accordance with AS 3600 Clause 9.1.1.
- One-way slabs with bar reinforcement in the direction of the span will also require transverse reinforcement for support of longitudinal reinforcement. AS 3600 Clause 9.4.1 requires the maximum spacing of reinforcement for crack control for slabs in both directions to be the lesser of $2.0D_s$ or 300-mm centres. Typically N10 (if available) or N12 bars are used in the transverse direction.
- For two-way slabs with bar reinforcement, it is important to nominate on the drawings which bar is to be placed first in the bottom layer and in which direction and which bar is to be placed last in the top layer and again in which direction.
- For both one-way and two-way slabs with square and rectangular mesh reinforcement, it is important to nominate on the drawings for each layer, where the top and bottom wires are placed.
- For rectangular meshes, the area of cross wires is limited to 227 mm^2/m so additional bar reinforcement may be required in the secondary direction for flexure, shrinkage and temperature control.
- For two-way slabs with diagonal edges, up to three layers of reinforcement both in the top and bottom of the slab may result. This may cause congestion and will need special consideration.
- 12-mm and 16-mm Ductility Class N bar can be either stock length bar or, more commonly, cut to length from coil.
- Stock length bars should be used where possible. Large size bars (20 mm and larger) are usually supplied in stock lengths of 12 m. For example, specifying a bar 4.00 m long instead of 3.75 m long will save cutting and wastage as three bars can be cut from one stock length bar. Staggering will often effectively allow the use of stock length bars.
- For flat slabs and flat plates with column and middle strips, the use of different size bars in the column and middle strips will make it easier to check the reinforcement layout on site, eg 20-mm in the column strip and 16-mm in the middle strip in the bottom and 24-mm in the column strip and 20-mm in the middle strip in the top.
- Slabs will often act as horizontal diaphragms carrying lateral actions such as wind and earthquake forces back to the vertical elements such as walls and columns. Designers may need to check the transfer of shear forces between the slab and vertical elements as well as the bending in the slab when acting as a deep beam. Section 5.6 of the *Precast Concrete Handbook*[5.6] covers this.
- Lapping of reinforcement in tension in areas of high moment should be avoided, and all laps and splices should be adequately detailed on the drawings. Typically, bottom reinforcement is lapped at the points of support of the slab and top reinforcement in the middle of the slab where laps are nominal.
- The reinforcement should always be curtailed where not required, eg excessive top reinforcement in the middle of slabs and excessive bottom reinforcement in the ends of slabs.
- For cantilevers, the top reinforcement should be anchored well back in an area of no or low stress if possible.
- Small diameter bars provide better crack control than large diameter bars (of the same area).

- Construction joints in the slabs (often at the quarter point of the span) should be properly detailed and shown on the drawings.
- Standard reinforcing meshes are manufactured in sheet sizes of 2.4 m by 6 m and again by proper detailing, cutting and wastage can be minimised.
- Care should be taken when lapping reinforcing mesh in thin slabs as up to eight layers of wires can occur in one location for one layer of reinforcement. There are techniques to reduce this by offsetting the laps and the sheets of mesh and by the use of tie bars.
- For slabs exposed to the weather, adequate slopes for drainage (taking into account long-term deflections) should be provided. Also, where there is no structural reinforcement, crack-control reinforce-ment should be provided in the top of the slab.
- Even with simply-supported slabs, concrete edge beams, walls and end supports will provide some restraint to the slab; nominal top reinforcement in the slab will therefore be required in these areas to control cracking.

CHARTS 5.1 TO 5.15

Charts 5.1–5.4 Flexural reinforcement in slabs Ductility Class N reinforcement pages 5.8–5.11

Charts 5.5–5.8 Flexural reinforcement in slabs Ductility Class L mesh reinforcement pages 5.12–5.15

Charts 5.9–5.12 Punching shear at circular columns where $M^*_v = 0$ pages 5.16–5.17

Chart 5.13 Punching shear at rectangular columns where $M^*_v = 0$ page 5.18

Chart 5.14 Slabs supported on four sides – proportion of load carried in shorter direction page 5.19

Chart 5.15 Slabs supported on four sides – deflection coefficient k_4 page 5.19

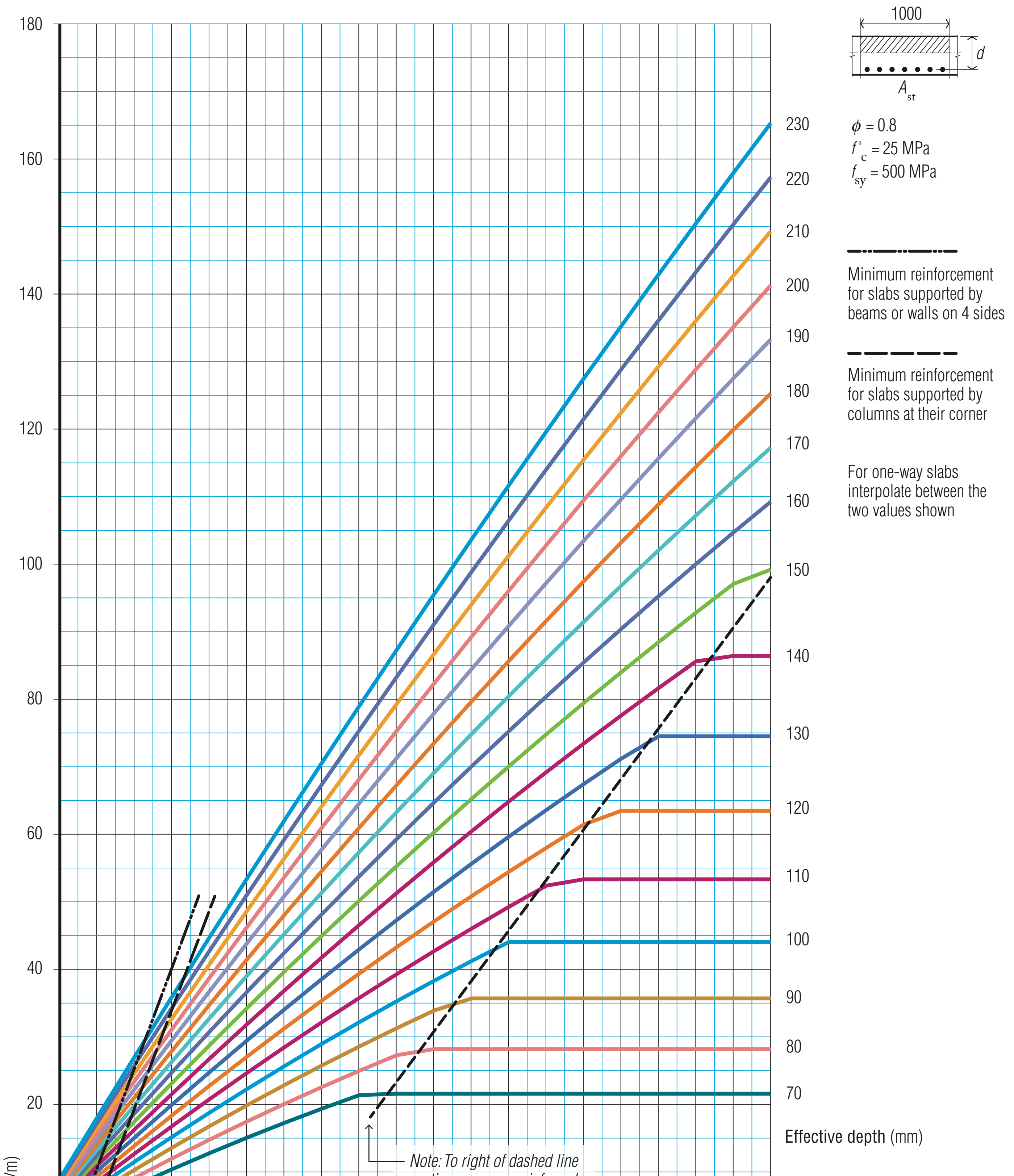
Spreadsheets 5.1, **5.2 and 5.3** may be downloaded from the Cement Concrete & Aggregates Australia website **www.ccaa.com.au**

5.8 GENERAL GUIDANCE

- The depth of a slab, unless it is a short span, is usually controlled by deflection considerations. The amount of reinforcement and its location are then determined to meet bending and shear requirements and constructability.
- The strength of concrete chosen must be appropriate for the exposure classification. For B2, C1 or C2 exposure classification, special class concrete needs to be specified as required by AS 3600. It is recommended that designers discuss these special requirements with the concrete suppliers and concrete technologist.
- In multi-storey construction a slab may be required to support the construction of other slabs over before it has attained its design strength (at 28 days). This may require higher strength concrete and consideration of the construction design loads as a design case[5.7].
- As with all concrete members, suspended insitu concrete slabs will shrink. If restrained by stiff elements such as supporting walls or core walls, unsightly cracking can result. Techniques to minimise this cracking include the use of pour strips or slip joints.
- Slabs are generally not suitable to support heavy line loads (such as masonry walls) or heavy point loads.
- Vertical penetrations through slabs for services or other openings, embedded items and unsupported edges always require careful consideration. Generally, vertical penetrations less than, say, D in size are not a problem unless there are many close together. Additional reinforcement may be required around larger penetrations.
- Sometimes, services such as electrical cables may need to be cast into the slab as shown in Figure 5.2. Large services such as sewer, storm water or water pipes should not be built in since they will cause problems if they leak.
- Set downs may be required to the top of the slab, eg for balconies, toilet areas and falls to drains.
- Consideration must be given to the finish and class of formwork to the formed surface of the slab (ie the underside of the slab) if exposed to view.
- Linear elastic analysis is recommended for major projects and the simplified methods for small projects and simple elements, subject to the computer analysis software available.

Figure 5.2 Electrical ducts cast into a slab

CHART 5.1 Flexural reinforcement in slabs Ductility Class N reinforcement



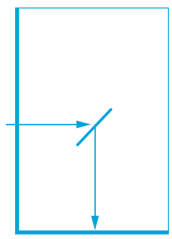


CHART 5.2 Flexural reinforcement in slabs Ductility Class N reinforcement

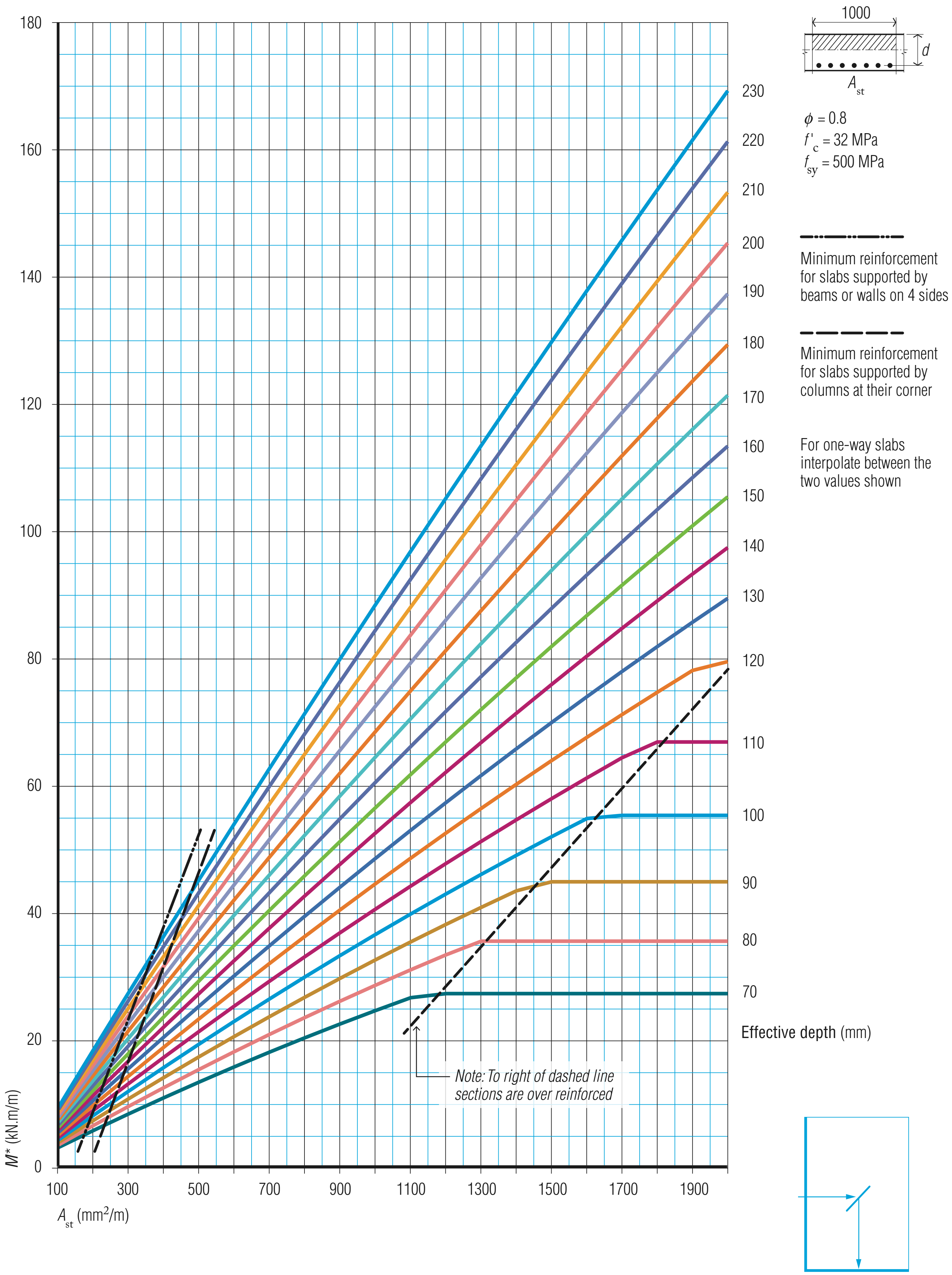
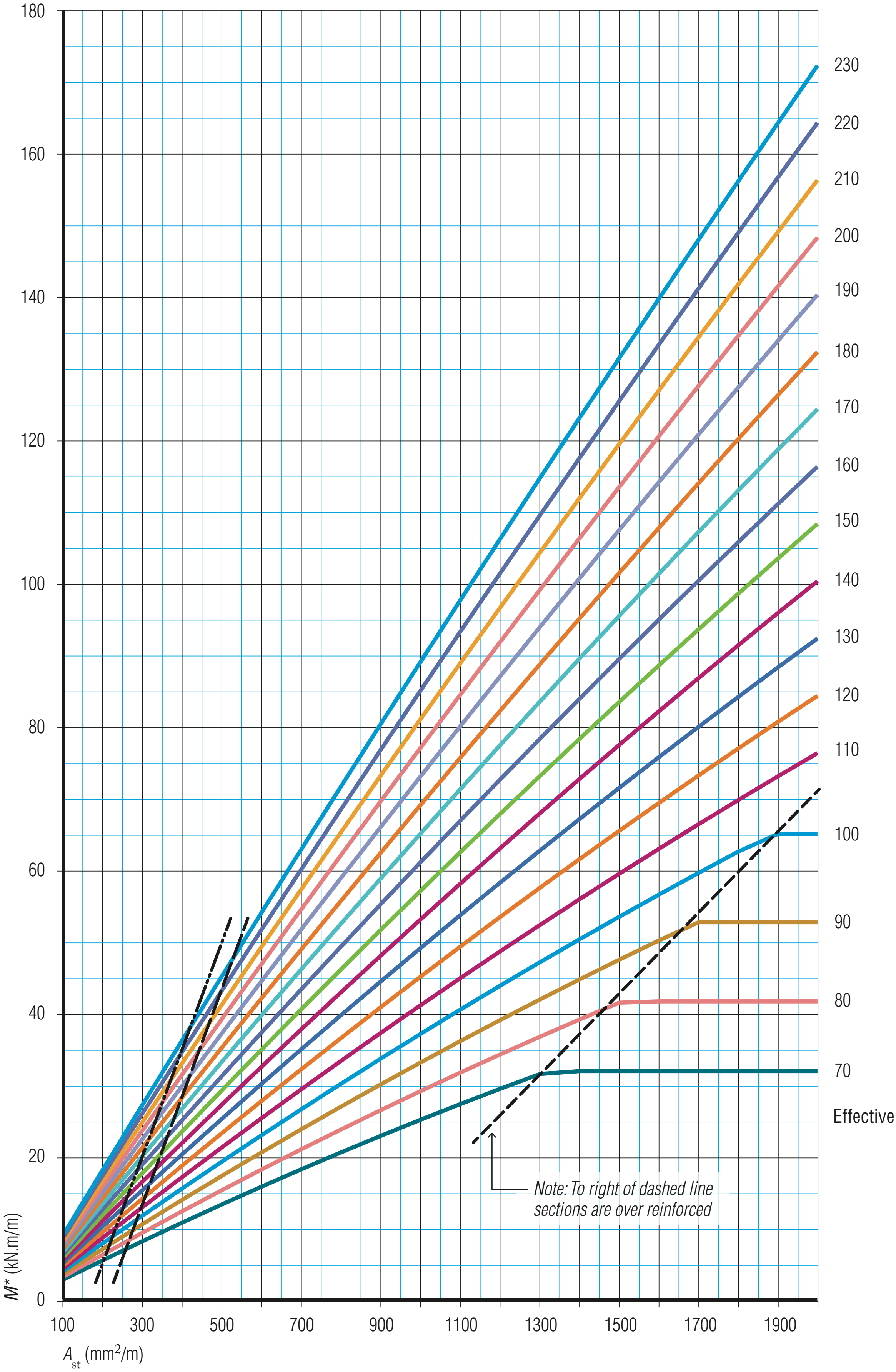
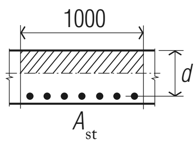


CHART 5.3 Flexural reinforcement in slabs Ductility Class N reinforcement





ϕ = 0.8
f'_c = 40 MPa
f_{sy} = 500 MPa

Minimum reinforcement for slabs supported by beams or walls on 4 sides

Minimum reinforcement for slabs supported by columns at their corner

For one-way slabs interpolate between the two values shown

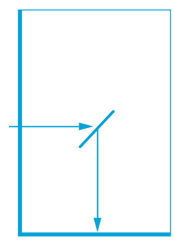


CHART 5.4 Flexural reinforcement in slabs Ductility Class N reinforcement

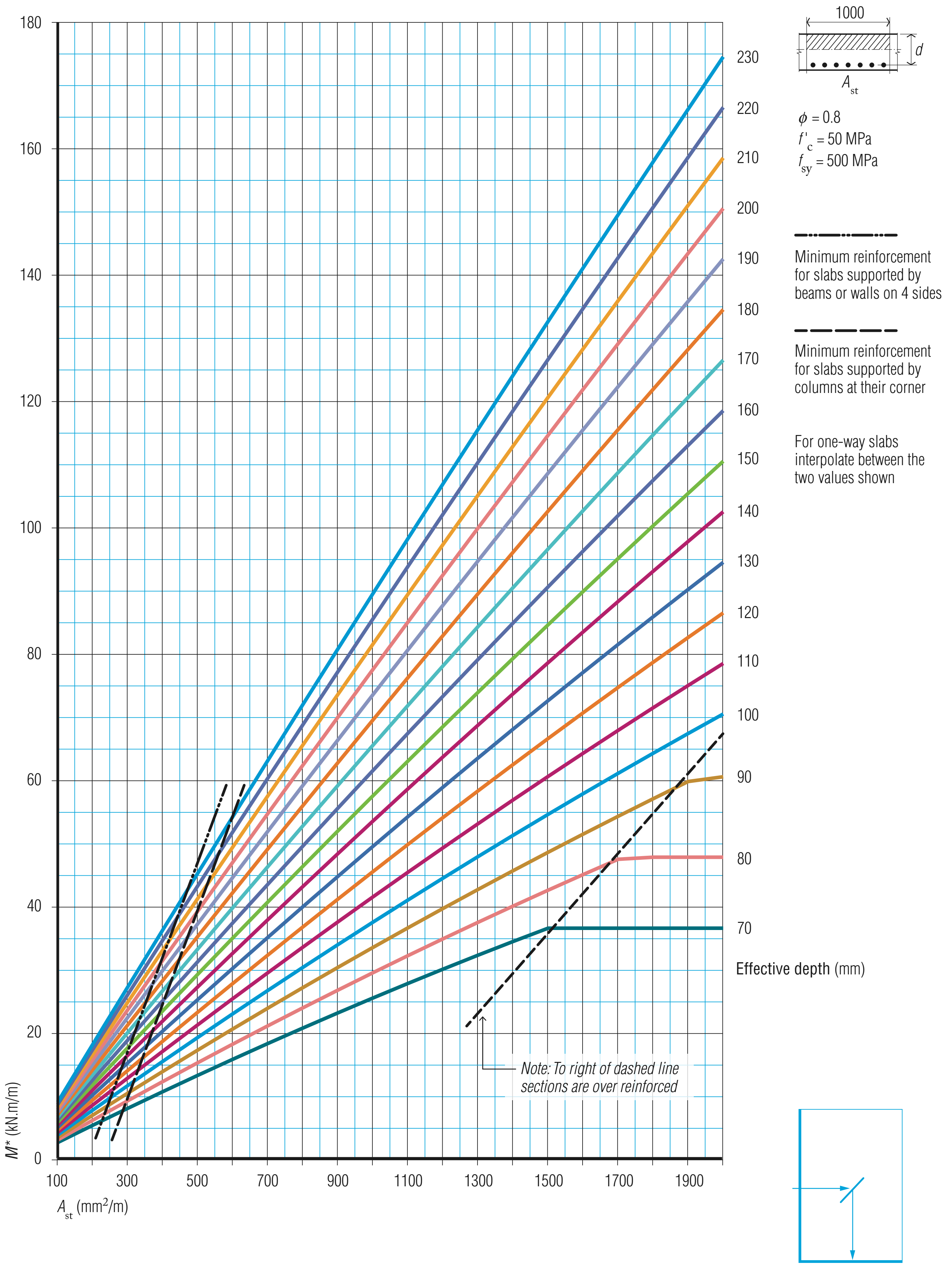


CHART 5.5 Flexural reinforcement in slabs Ductility Class L mesh reinforcement

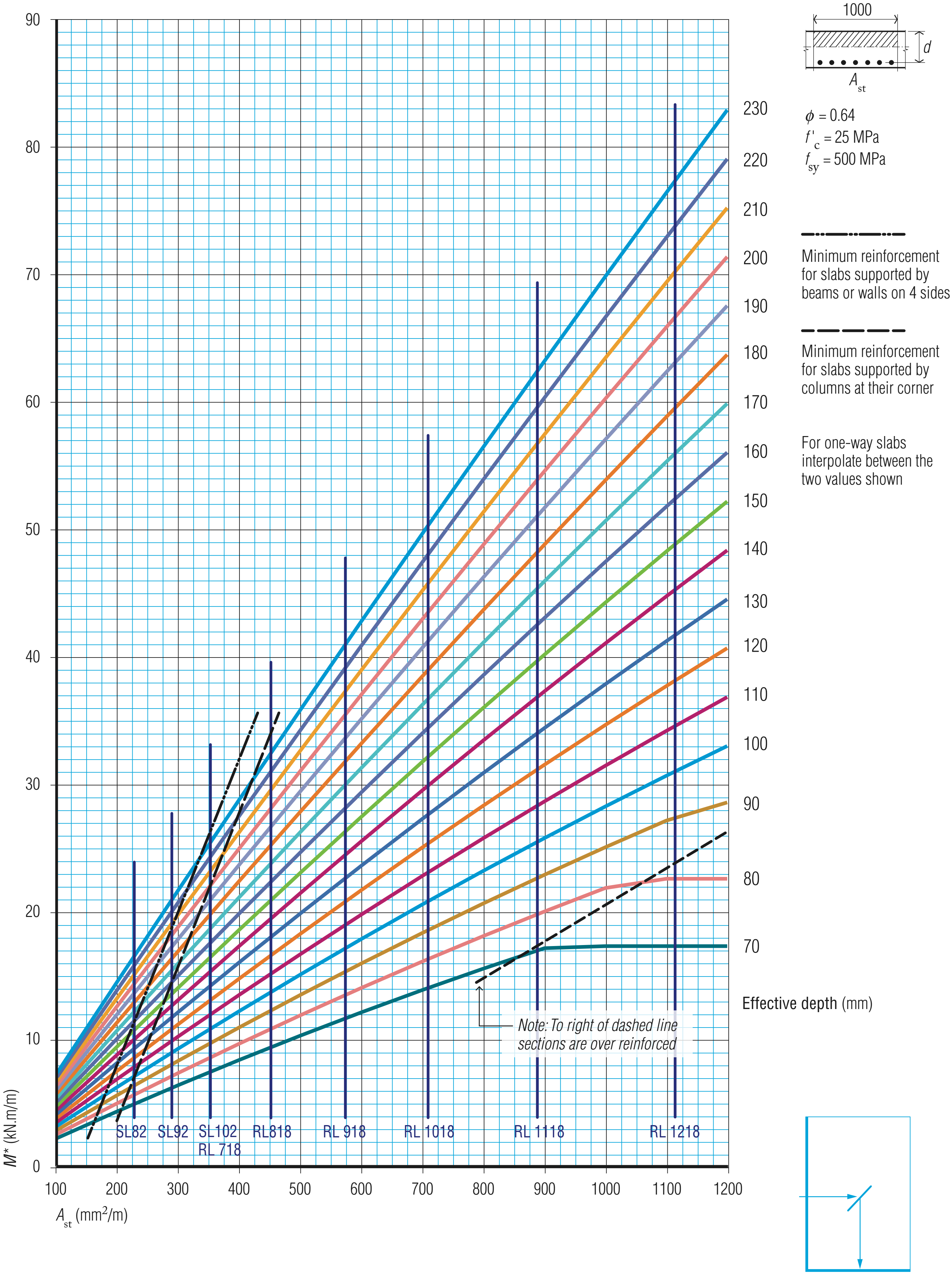


CHART 5.6 Flexural reinforcement in slabs Ductility Class L mesh reinforcement

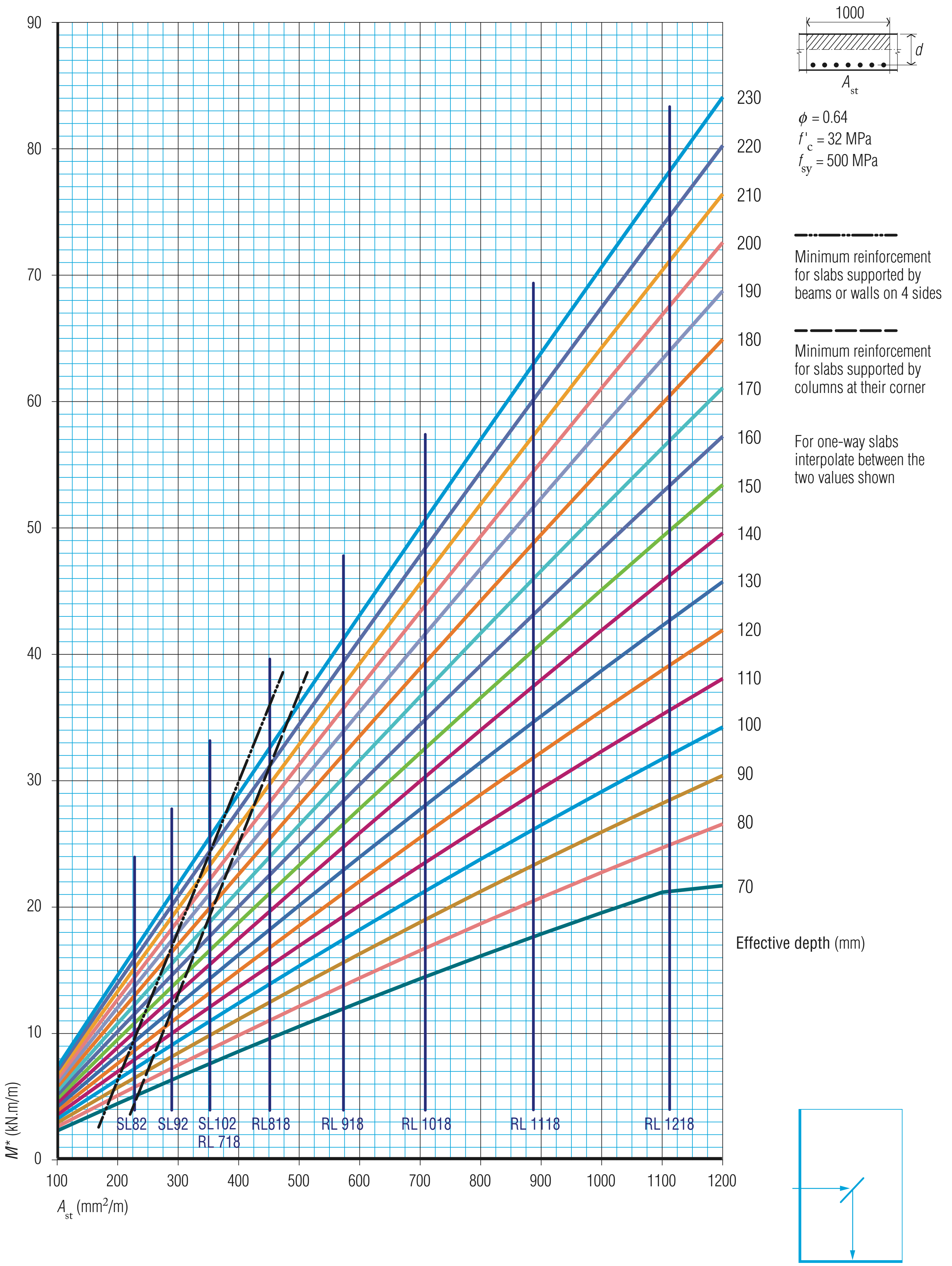


CHART 5.7 Flexural reinforcement in slabs Ductility Class L mesh reinforcement

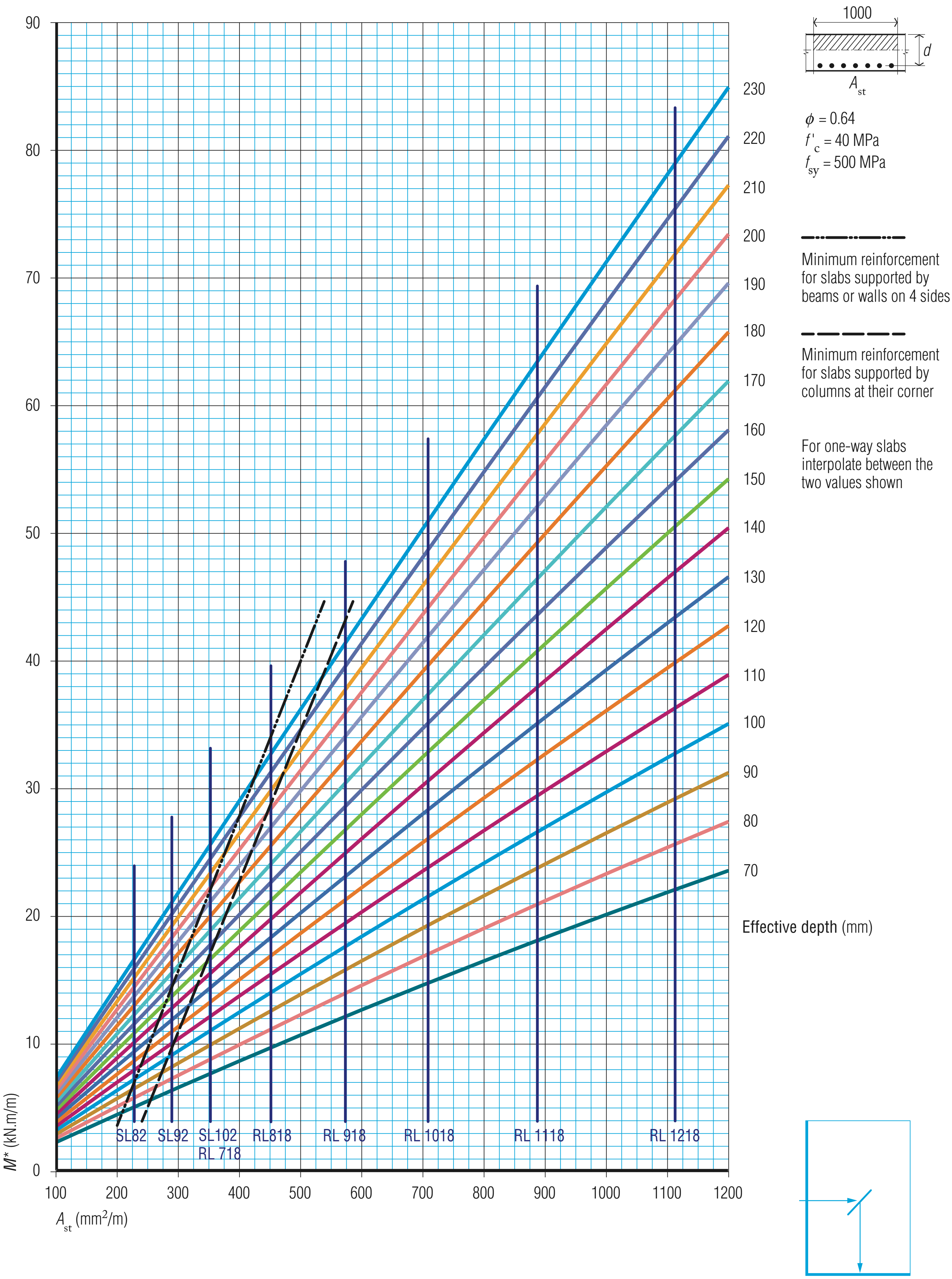
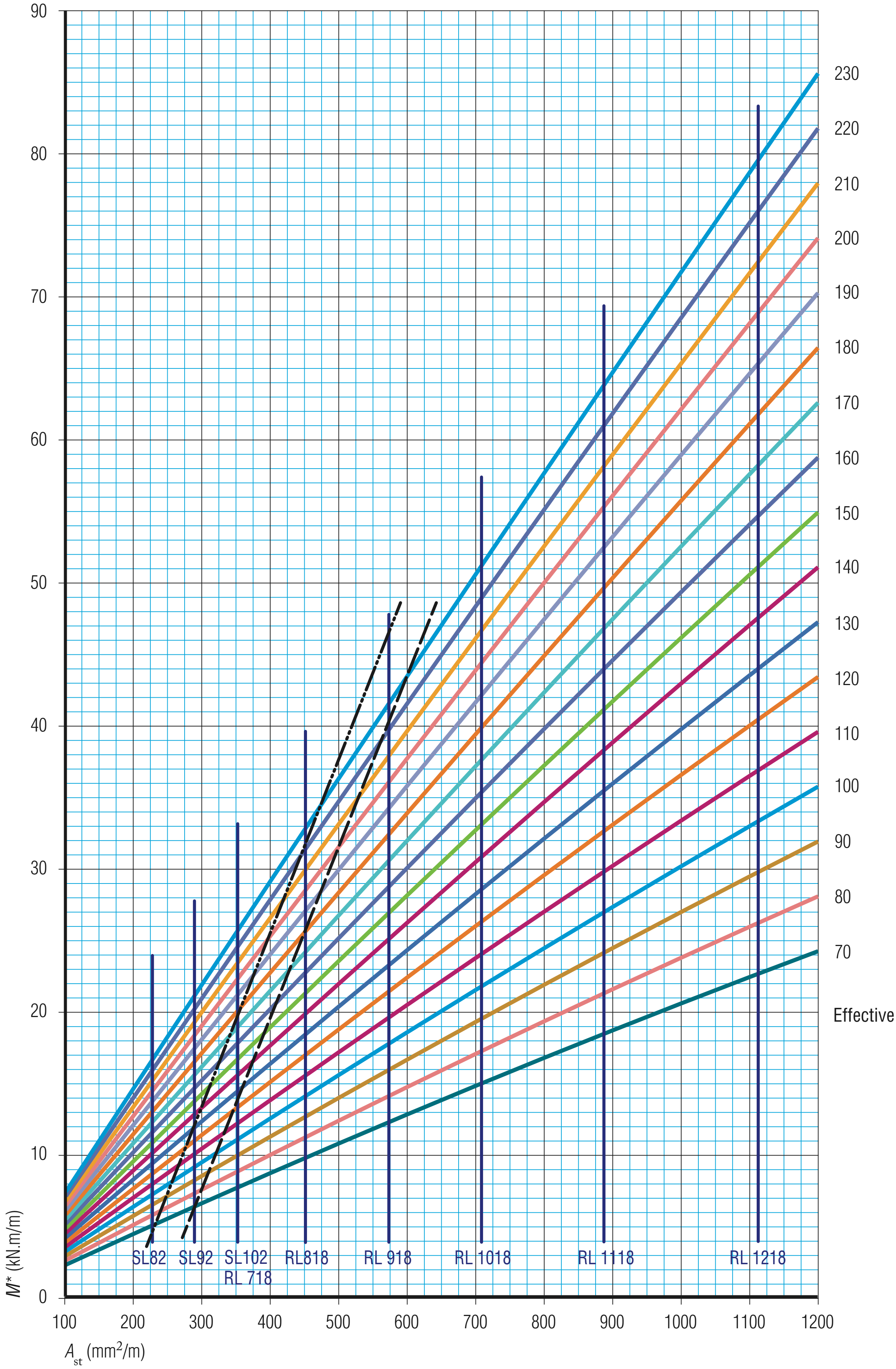
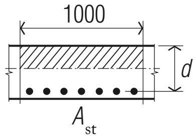


CHART 5.8 Flexural reinforcement in slabs Ductility Class L mesh reinforcement





ϕ = 0.64
f'_c = 50 MPa
f_{sy} = 500 MPa

Minimum reinforcement for slabs supported by beams or walls on 4 sides

Minimum reinforcement for slabs supported by columns at their corner

For one-way slabs interpolate between the two values shown

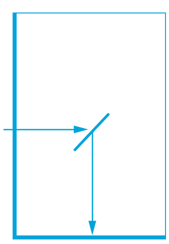


CHART 5.9 Punching shear at circular columns where $M^*_v = 0$

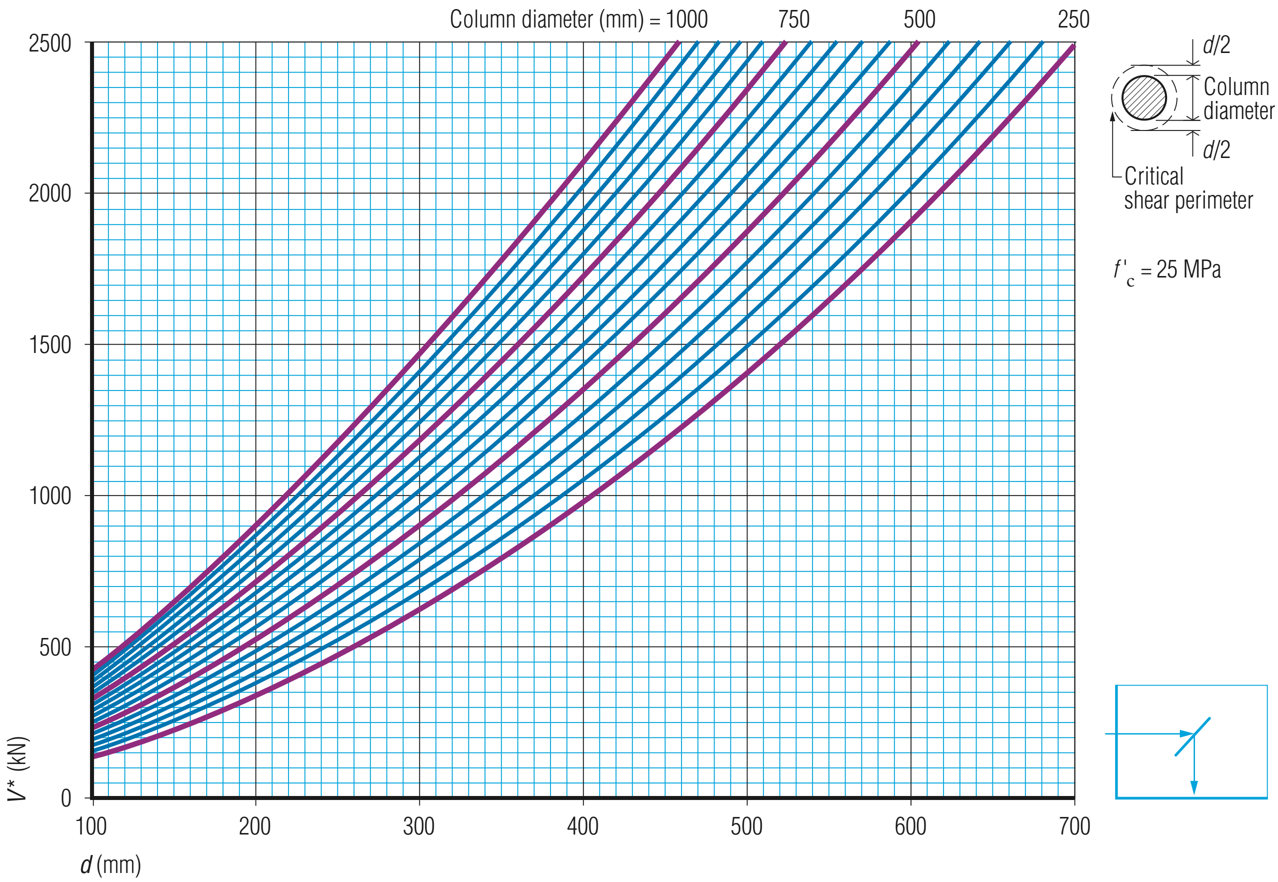


CHART 5.10 Punching shear at circular columns where $M^*_v = 0$

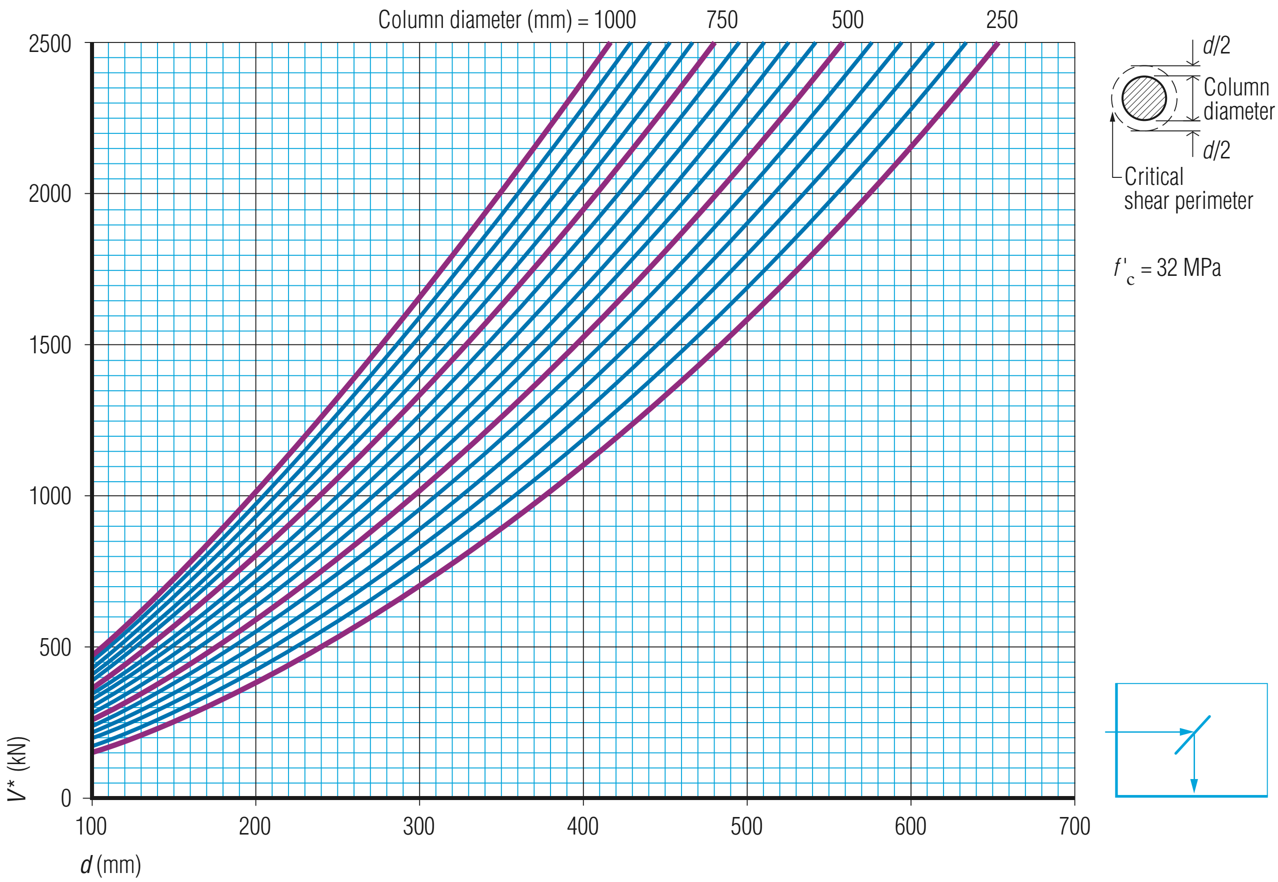


CHART 5.11 Punching shear at circular columns where $M^*_v = 0$

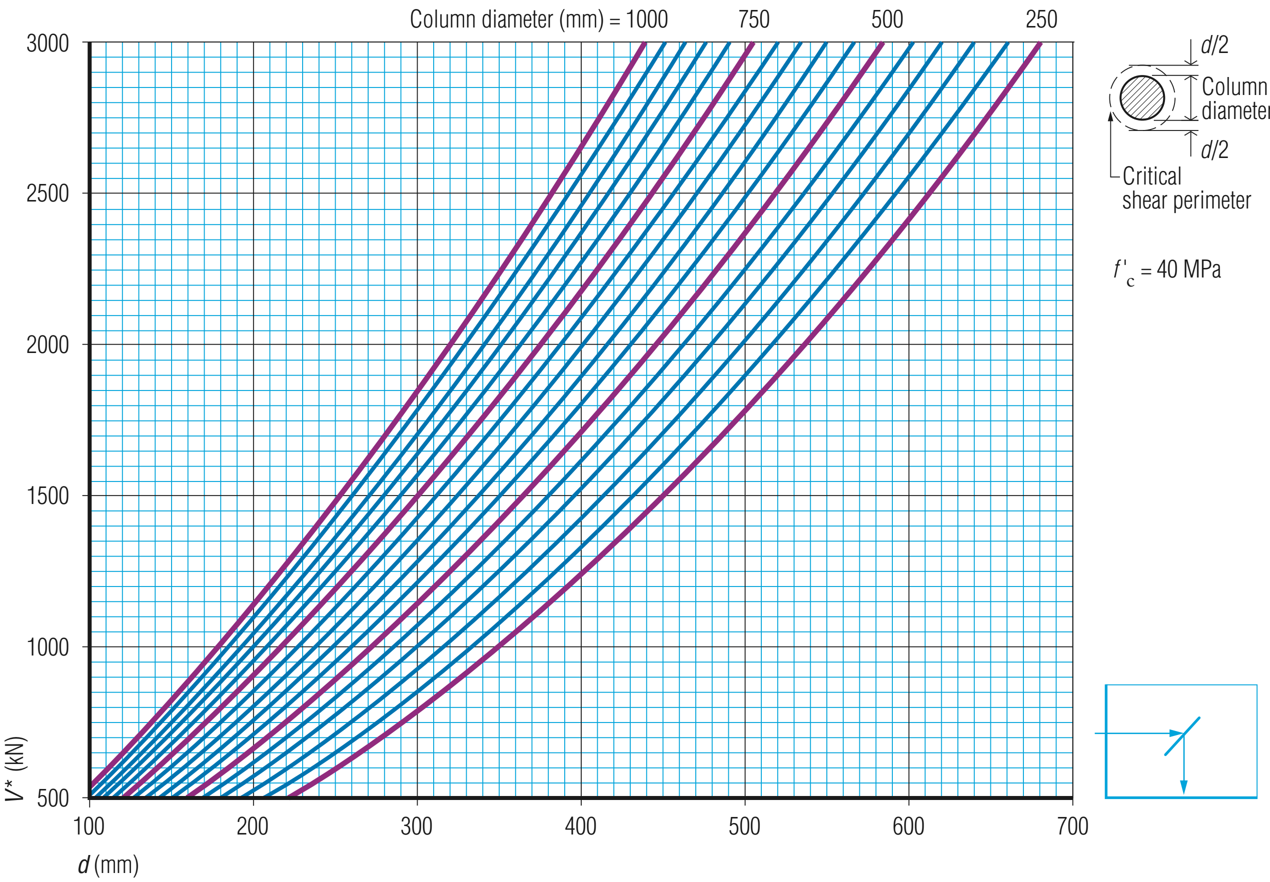


CHART 5.12 Punching shear at circular columns where $M^*_v = 0$

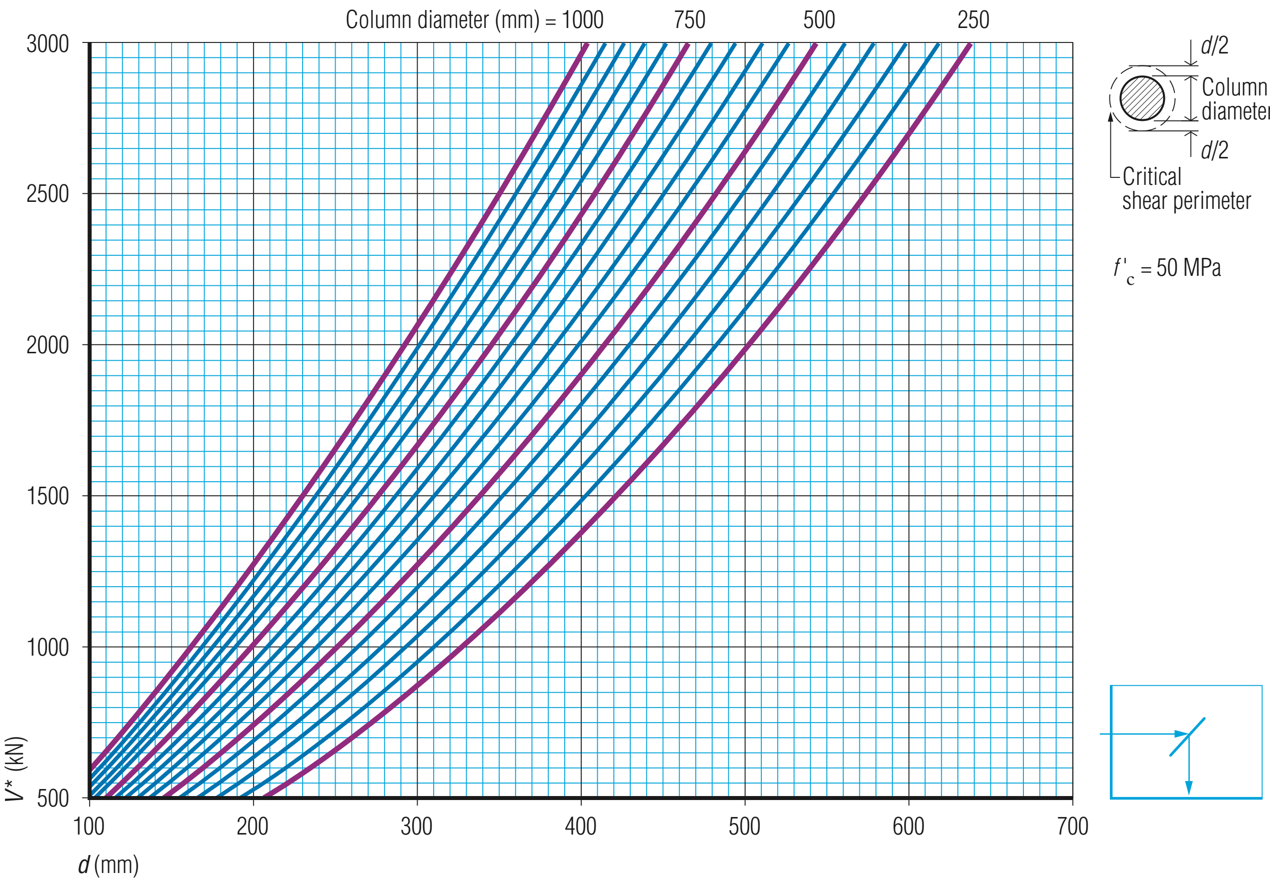


CHART 5.13 Punching shear at rectangular columns where $M^*_v = 0$

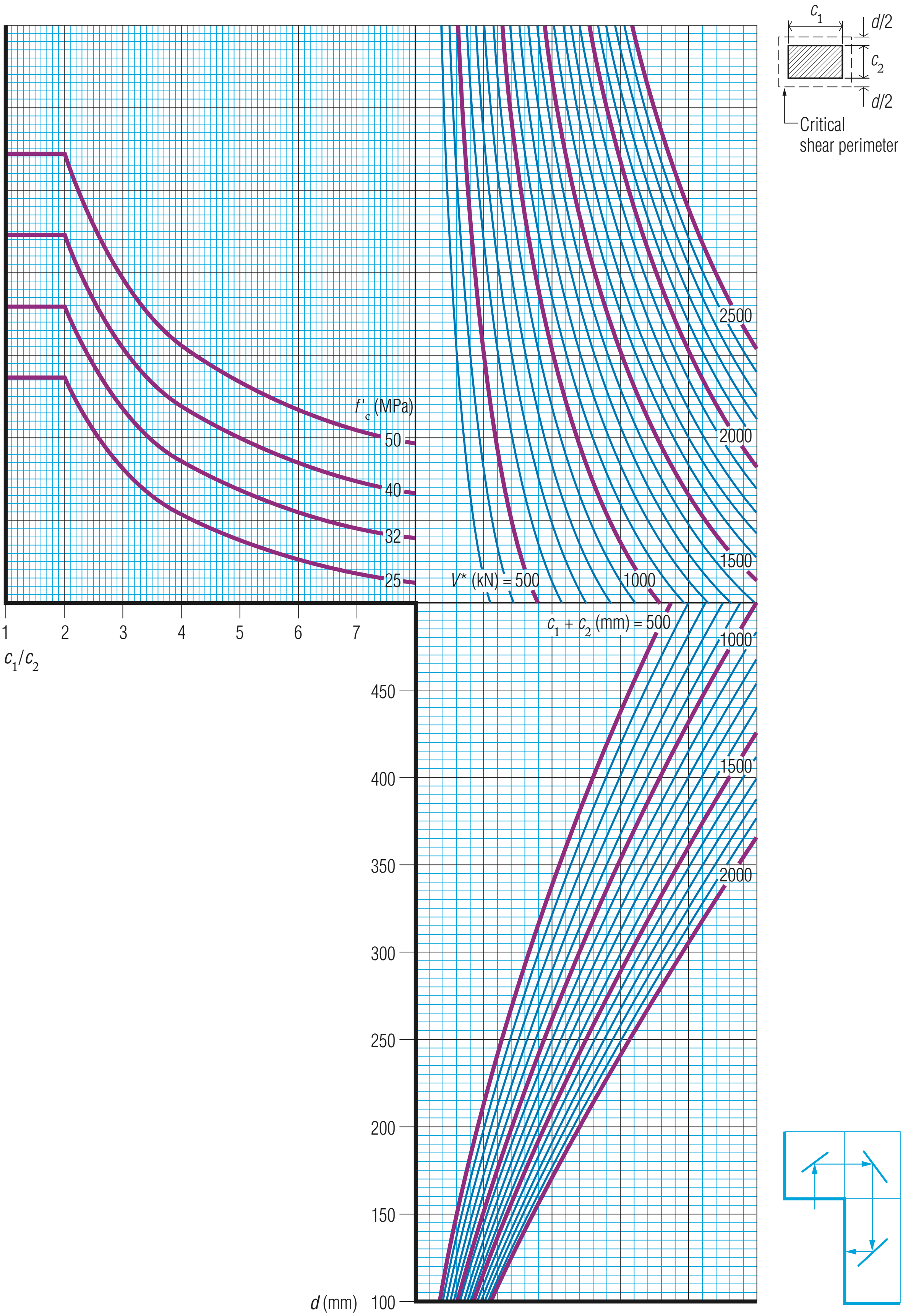
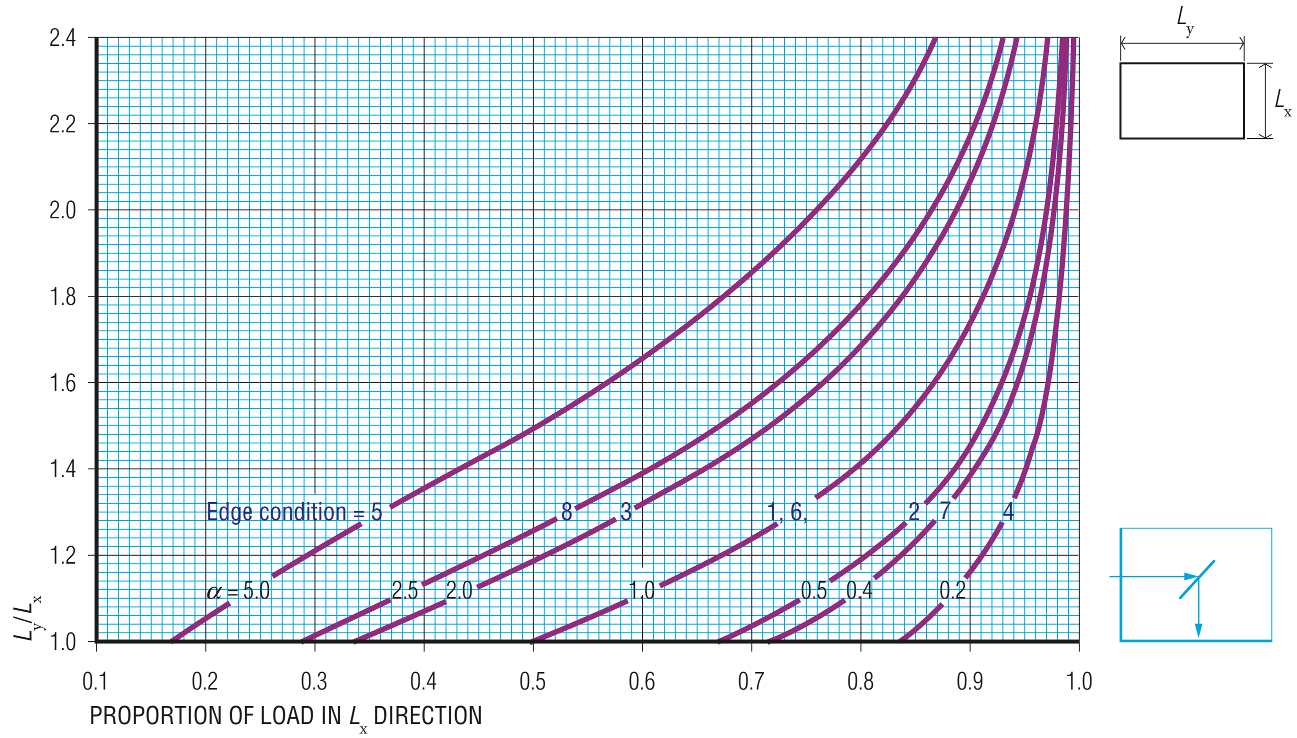
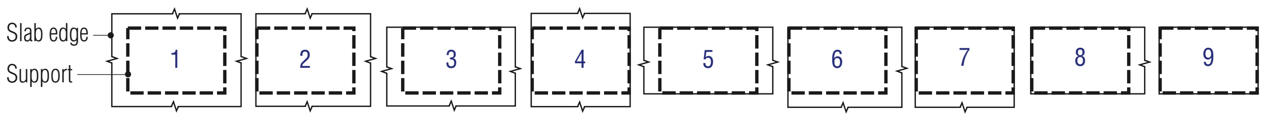


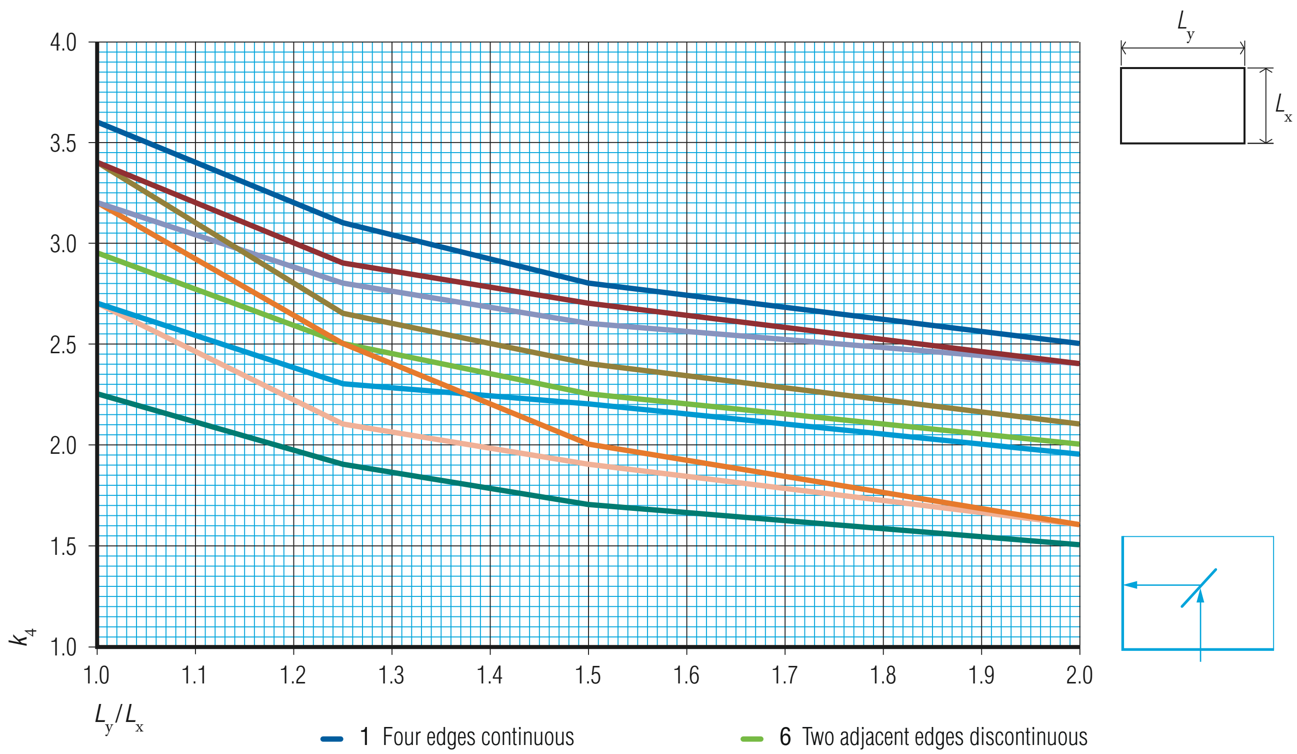
CHART 5.14 Slabs supported on four sides – proportion of load carried in shorter direction





EDGE CONDITION – Slabs supported on four sides

CHART 5.15 Slabs supported on four sides – k_4 for deflection calculations



1 Four edges continuous
2 One short edge discontinuous
3 One long edge discontinuous
4 Two short edges discontinuous
5 Two long edges discontinuous
6 Two adjacent edges discontinuous
7 Three edges discontinuous (one long edge continuous)
8 Three edges discontinuous (one short edge continuous)
9 Four edges discontinuous

REFERENCES

5.1 *Guide to Long-Span Concrete Floors* (T36) 2nd Ed, Cement Concrete & Aggregates Australia, 2003.

5.2 Foster SJ, Kilpatrick AE and Warner RF *Reinforced Concrete Basics* 2nd Ed, Pearson, 2010.

5.3 *Guide to Industrial Floors and Pavements* (T48) 3rd Ed, Cement Concrete & Aggregates Australia, 2009.

5.4 AS 3600 *Concrete structures* Standards Australia, 2009.

5.5 *Reinforcement Detailing Handbook* (Z06) 2nd Ed, Concrete Institute of Australia, 2010.

5.6 *Precast Concrete Handbook* 2nd Ed, National Precast Concrete Association Australia and Concrete Institute of Australia, 2009.

5.7 *Multi-Storey Formwork Loading* (Z35), Concrete Institute of Australia, 1990.

Chapter 6_Columns

6.1 GENERAL

Concrete columns are small but important structural elements in most buildings. Their design can be more complex than other concrete elements, while, along with walls, they are frequently the most obvious, and sometimes intrusive, parts of a structure. Architectural and engineering judgement is required to determine their position, size, shape, the spans of the horizontal elements they support and their location for the economy of structure. Prior to final design, it is important to ensure that the design team agrees the proposed column sizes and positions, especially where they are located in car parking areas and architecturally sensitive areas.

It is also important to consider the implications of the location of each of the columns on each floor and, if possible, to avoid offsetting columns from one storey to another. Such changes in position usually involve transfer beams, which can be expensive and time-consuming to build. This might occur for example in a building where there is a basement car park, ground floor retail and upper floor residential areas and optimum column locations will be different for each area.

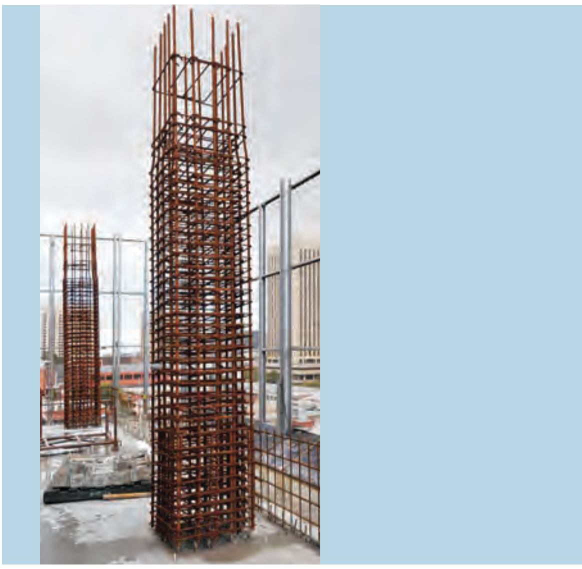
Generally, columns are designed for axial actions (loads), and bending moments about each axis as required, for the various load cases at each floor level. For single-storey buildings and higher buildings where the last lift of columns is supporting a lightweight roof, they may act as vertical cantilever beams carrying only small vertical actions and resisting lateral actions.

Insitu concrete columns can require expensive formwork and take time to build. In recent years, there has been a trend to precast concrete columns, particularly in low-rise buildings, for ease of construction and to reduce costs. To facilitate erection and the casting of the floor over, precast columns are usually most economical as single-storey-height elements. Two-storey-height columns are possible, especially if the bracing is kept below the floor and the connection to the floor in the middle of the column is appropriately considered. When precast columns are used, they need to be temporarily braced in two directions until the floor over is cast. The *Precast Concrete Handbook*[6.1] includes information on precast concrete columns. Their design is in accordance with AS 3600[6.2] and will depend on the end fixity adopted and whether or not they are braced.

Concrete columns can be subject to a combination of actions including:

- Vertical actions. These are calculated by apportioning the actions on each floor to the column by the selected frame-analysis software or on an area basis, sometimes known as column rundowns or similar.
- Bending moments from slabs and beams. The bending moments are usually assessed from the slab or beam design or by the selected frame-analysis software.
- Horizontal actions on the structure resulting in shear forces and bending moments in the column when it is used to resist lateral actions as part of the building frame and when there are no shear walls. These actions are usually assessed by the selected frame-analysis software.

Off-form columns to a country bank



Column bars offset at the top to facilitate joining above the floor level with fitments through the floor level and also closer spacings of the fitments

Polished round reinforced and post tensioned precast columns These are unusual non-vertical columns, but illustrate how complex columns can be.

6.2 INITIAL SIZING AND ACTIONS

For the design of a column, an initial size, concrete strength, number, size and location of longitudinal bars and fitments, cover for durability and axis distance (for fire) are assumed along with an assessment of whether the column is braced or not and its effective length. The chosen configuration is then checked for adequacy and the various parameters adjusted as required. The quickest way to size a concrete column is usually to design the chosen configuration using appropriate concrete column design software or spreadsheet. It is recommended that designers start with the design of the lower lifts of columns where the actions are highest and the maximum concrete strength will be needed.

Initial sizing, concrete strength and reinforcement for columns can also be based on experience, previous designs or chosen from the design Charts 6.1 to 6.6 in this Chapter. Further advice on designing columns is also given by Warner et al[6.3, 6.4].

The implications of Clause 10.8 for the transmission of axial forces through the floor systems must be assessed early in the design process. Where possible, the requirement of AS 3600 Clause 10.8 should be met by specifying a concrete strength for the floor system of not less than 0.75 of that specified for the columns (eg 40 MPa for columns with 32 MPa for floors, or 32 MPa and 25 MPa respectively). Where the floor strength is less than 0.75 times that of the column strength. The effective strength of the column, f'_{ce}, in the column/slab or column/beam area can be significantly less than the strength of the column, f'_c. This reduction is sometimes of the order of 20–30% for very-high-strength columns. Additional reinforcement can be used to compensate for this reduction at the floor level. However, these extra bars in the junction area, in what almost certainly will be heavily reinforced columns, may lead to congestion and difficulties in placing and compacting the concrete. Solutions to overcome these difficulties include: the use of mechanical end splices; and, where the reinforcement percentage is high, locating the lap splices outside this area; or, as AS 3600 suggests, confining fitments can be used to increase the effective strength of the concrete in the joint – presumably in accordance with Clause 10.7.3 but the specifics on how precisely this is to be achieved are not spelt out. Alternatives such as 'blobs' of high-strength concrete placed at the column joint when the slab is being cast can result in cold joints in the slab. Further, such processes are difficult to supervise on site, are usually impractical and therefore generally not recommended.

For columns there should be a minimum of four bars in a rectangular section and six bars in a circular section. The minimum area of reinforcement, A_{sc}, required by AS 3600 is 1% of the gross concrete area. Generally, the reinforcement ratio should be kept below 2.5% for economy and ease of splicing. AS 3600 permits a maximum area of reinforcement of 4% and this implies a maximum of 8% at laps. For over-sized columns (eg for aesthetic reasons), AS 3600 allows a lower value for A_{sc} to be used provided $A_{sc} f_{sy} > 0.15 N^*$. It is suggested that the minimum area of reinforcement should not be less than about 0.5 %.

In the lower levels of high-rise buildings, to keep the columns to a reasonable size, the longitudinal bars are sometimes spliced using end bearing splices, mechanical splices, welding or the bars are bundled and high-strength concrete is used. It should be noted that end bearing splices may not be readily available, mechanical splices will take up more space and welding is an expensive alternative.

Assessing the vertical actions carried by columns (and walls) requires a full understanding of the building structure and its behaviour and knowledge of all actions to be carried by the building. These vertical actions from permanent and applied floor actions are calculated by assessing the actions supported by each column (or wall) on a floor-by-floor basis based on the tributary areas to each column (or wall). This can be determined by using a spreadsheet to calculate the actions on the column or using appropriate structural analysis software (or by hand). Nevertheless, judgement is required in assessing the vertical actions regardless of the method used.

One of the problems when using a full 3D building model to assess the column actions is that it may not take into account all loading cases and usually does not take into account construction sequences and the redistribution of actions that may occur due to deflections or shortening in some supports, etc. As a result, the 3D building model may be non-conservative in some cases. On the other hand the use of tributary areas may be conservative in some cases and non-conservative in others. Column rundowns calculated by spreadsheet (or by hand) are usually based on a simple area or length basis, with the proportion of the actions to each vertical element calculated by taking half the distance in each direction to the adjacent vertical element supporting vertical actions. Further, the column rundown may not include all the loads to the column (or wall) because of continuity and frame action. These additional actions are sometimes known as *moment shears* and can be up to 15% of the floor actions. For example, an edge column will generally have less actions on it than is implied by a calculation on an area basis while the first column in from the edge of a building will have more. Actions at each level are usually calculated just above the floor below. (For example the heading 'On Level 4' as shown in Table 6.1 on the column rundown means the actions from the floors above, just above the fourth floor and these actions would be used to design the column from Level 4 to Level 5.)

In addition, for imposed actions reductions can be deducted from rundowns, where applicable, in accordance with AS 1170.1 Clause 3.4.2 (including a reduction from the *moment shears*). Structural designers should always check the architect's and other members of the design team's drawings to see that all loads have been included. Actions to be considered in the design of columns for a multi-storey building can include some or all of the following:

- Roof (including finishes)
- Floors (including finishes)
- Internal masonry walls
- Internal lightweight walls
- External walls including precast walls, curtain walls, etc
- Beams framing into the column
- Precast and tilt-up concrete walls
- Fascias and sun hoods
- Lift machinery
- Air conditioning and other mechanical plants
- Stairs
- Heavy load areas, eg storage areas
- Special equipment, eg water tank, generator, etc.

The bending moments and horizontal shear actions in a column will be determined by the frame-analysis software used or by other structural design methods.

TABLE 6.1 Sample column rundown

Load element	Unit area length	Permanent actions (DL)	Imposed actions (LL)	Permanent axial actions (DL)	Permanent bending moments E/W	Permanent bending moments N/S	Imposed axial actions (LL)	Imposed bending moments E/W	Imposed bending moments N/S
ON LEVEL 4									
1 Roof	12.96	0.6	0.25	7.8			3.2		
2 Precast edge beam	2.8	7.68		21.5			0.0		
3 Wall load	0	1		0.0			0.0		
4 Column	2.8	11.52		32.3			0.0		
5 Moment shears	0	0		0.0			0.0		
Total this level				61.5	0	0	3.2	0	0
Total on Level 4		PA (DL)		61.5					
		IA (LL)					3.2		
ON LEVEL 3									
1 Floor	12.96	8.4	4	108.9	0	35.2	51.8		10.3
2 Precast edge beam	2.8	7.68	0	21.5	12		0.0	3	
3 Wall load	4.5	3	0	13.5			0.0		
4 Column	3	11.52		34.6			0.0		
5 Moment shears	12.96	0.84	−0.4	10.9			−5.2		
Total this level				189.3	12.0	35.2	46.7	3.0	10.3
Total on Level 4		PA (DL)		189.3					
		IA (LL)					46.7		

Notes:

— Actions (loads) are in kN or kPa. All loads are unfactored.

— Moments are in kN.m.

The bending moments in the columns are the moments at the top or bottom face of the slab or beam at the beam/slab-column joint and not at the centre line of the slab or beam. In column design, it is important to correctly calculate the applied moments, particularly at the top of the building where columns can be relatively slender, vertical actions low but bending moments can still be large.

When there are a large number of columns, it is usual to group them into a series of typical columns, in order to rationalise the design and to avoid detailed analysis of every column. Normally, at least one corner column, one edge column, one internal column and all non-typical columns should be designed at each floor level, keeping the size and details as uniform as possible. For example, if the cover (and axis distance indirectly) and concrete strength are determined by durability considerations for the external columns, then the designer should consider using the same size, cover and concrete strength with less reinforcement for internal columns, even though they could be smaller, use a lower strength concrete and could have less cover to the reinforcement. Since the volume of concrete in columns is generally small, the benefit of helping to avoid errors on site will far outweigh the cost of additional concrete.

6.3 DESIGN PROCESS

The design process for columns will follow these general steps.

STEP 1

Assume an initial size of column, concrete strength and reinforcement

See Section 6.2.

STEP 2

Assess the actions on the column

These will be determined from the structural analysis of the building in accordance with one of the strength check/analysis procedures given in AS 3600 Section 2. These are usually carried out using a proprietary computer program or a column rundown for vertical actions. Note that the choice of method has a direct influence on the design procedure to be adopted in accordance with AS 3600 Clause 10.2. See Step 4.

Columns are usually designed only for strength. Stability and serviceability are considered only for slender and unbraced columns.

The strength action effects will be in accordance with Table 1.1 in Chapter 1 of this Handbook. Considering the entire possible axial actions and bending moment combinations for each loading case for a given cross section for each column at each floor, and manually checking the strength of the chosen column size and reinforcement configuration can be a tedious process. Frequently the load combination $1.2G + 1.5Q$ will be the critical design case. Table 6.2 shows an example of various load combinations that may have to be considered for the design of a particular column.

TABLE 6.2 Load combinations for a particular column

Load case	Load combination	Axial load (kN)	BM top (kN.m)	BM bot (kN.m)
1	$1.35G$	1776	10	14
2	$1.2G + 1.5Q$	1850	45	55
3	$1.2G + \psi_c Q + W_u$	1650	50	65
4	$0.9G + W_u$	1147	45	60
5	$G + \psi_c Q + E_u$	1550	45	60

STEP 3

Assess the durability requirements, cover and fire ratings to determine the axis distance and the cover of the longitudinal bars from the column face

This is done in accordance with the requirements of AS 3600 Sections 4 and 5. See also Chapter 3 of this Handbook.

For example, an external column in a coastal area within 1 km of the sea would have a durability classification of B2 and would require 40-MPa concrete minimum (Special Class Concrete) and 45-mm cover to the fitments, ie 71-mm axis distance assuming 12-mm fitments and 28-mm main reinforcing bars. Suggest adopting a 75-mm axis distance.

Next determine the required fire resistance period (FRP) from the BCA. Then determine the required axis distance (cover plus fitment plus half-bar diameter) for the FRP, eg from AS 3600 Table 5.6.3. Assuming the required FRP is 120 minutes, $N^*_f/N_u = 0.7$ and that the column is exposed on more than one side, a minimum column size of 350 mm and an axis distance of 57 mm is required.

Finally, check whether durability (cover) or fire-resistance (axis distance) will govern the axis distance to the main column bars from the outside face of the column. In this example, durability governs and the required axis distance is 75 mm. (However, if the column was inland in an arid area with an A1 Exposure Classification then the axis distance required for fire resistance would govern the distance to the column bars from the outside face of the column.)

STEP 4

Choose a design procedure based on AS 3600 Clause 10.2

Generally, a linear elastic analysis (Clause 2.2.2) will be used. Design using rigorous analysis, eg a non-linear analysis (Clause 2.2.5) will be appropriate only in special circumstances, eg for long, slender columns, tapered columns and columns of other special types or shapes. This approach should be undertaken only after careful consideration, because of the specific and detailed requirements for these methods of analysis. Where the axial forces and bending moments are determined by a rigorous analysis in accordance with AS 3600 Clauses 6.5 and 6.6, a column shall be designed in accordance with Clauses 10.6 and 10.7 without further consideration of additional moments due to slenderness.

Normally, column design will be carried out where the axial actions (forces) and bending moments are determined by a linear elastic analysis. The column is then designed as either:

- a short column, in accordance with AS 3600 Clauses 10.3, 10.6 and 10.7; or
- a slender column, in accordance with AS 3600 Clauses 10.4 to 10.7.

Where the axial actions (forces) and bending moments are determined by an elastic analysis incorporating secondary bending moments due to lateral joint displacements, as provided in AS 3600 Clause 6.3, a column shall be designed in accordance with Clauses 10.6 and 10.7.

If the bending moment in a column causes significant lateral deflection, the effective eccentricity of the axial load at mid-height is increased, increasing the moment, having an iterative effect. AS 3600 Clause 10.4 defines when a column is sufficiently slender for this to be taken into account. The design procedure applies an amplification factor to the moment acting on the column so that the short column moment-strength interaction design curves can be used.

Flowchart 6.1 covers the general design of columns in uniaxial or biaxial bending in accordance with AS 3600.

Assuming a linear analysis is to be used, the general design procedure will be:

- Determine the unsupported length of the column, L_u.
- Determine if the column is braced or unbraced.
- Determine the distance of the longitudinal reinforcement from the face of the column based on durability and fire requirements.
- Determine the ultimate axial actions and the design moments at each end of the column about each axis, as required, and whether the column is in single or double curvature.
- Determine the minimum moment, $0.05\,DN^*$.
- Calculate the effective length, L_e, in accordance with Clause 10.5.3, in both directions as required and calculate the slenderness ratio, L_e/r. (For braced columns L_e will be $\leq L_u$ and for unbraced columns L_e will be $> L_u$.)

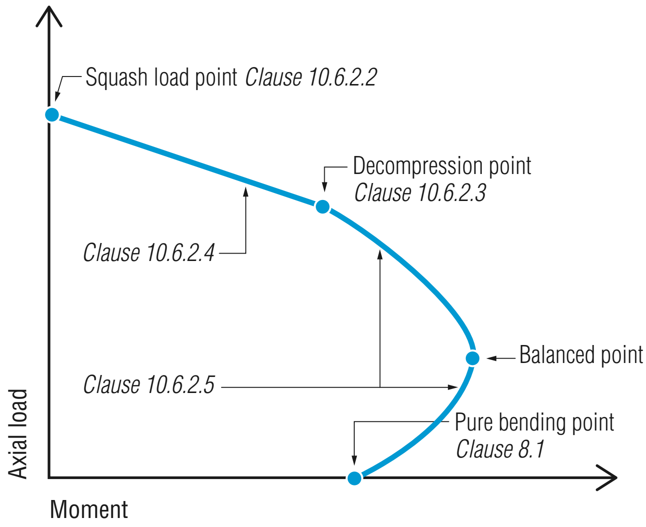


Figure 6.1 Axial load vs moment diagram (after AS 3600 Figure 10.6.2.1)

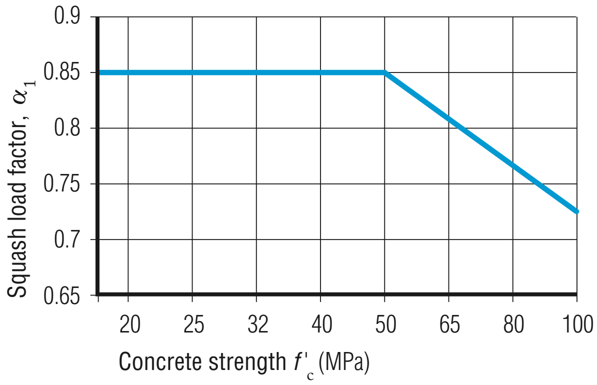
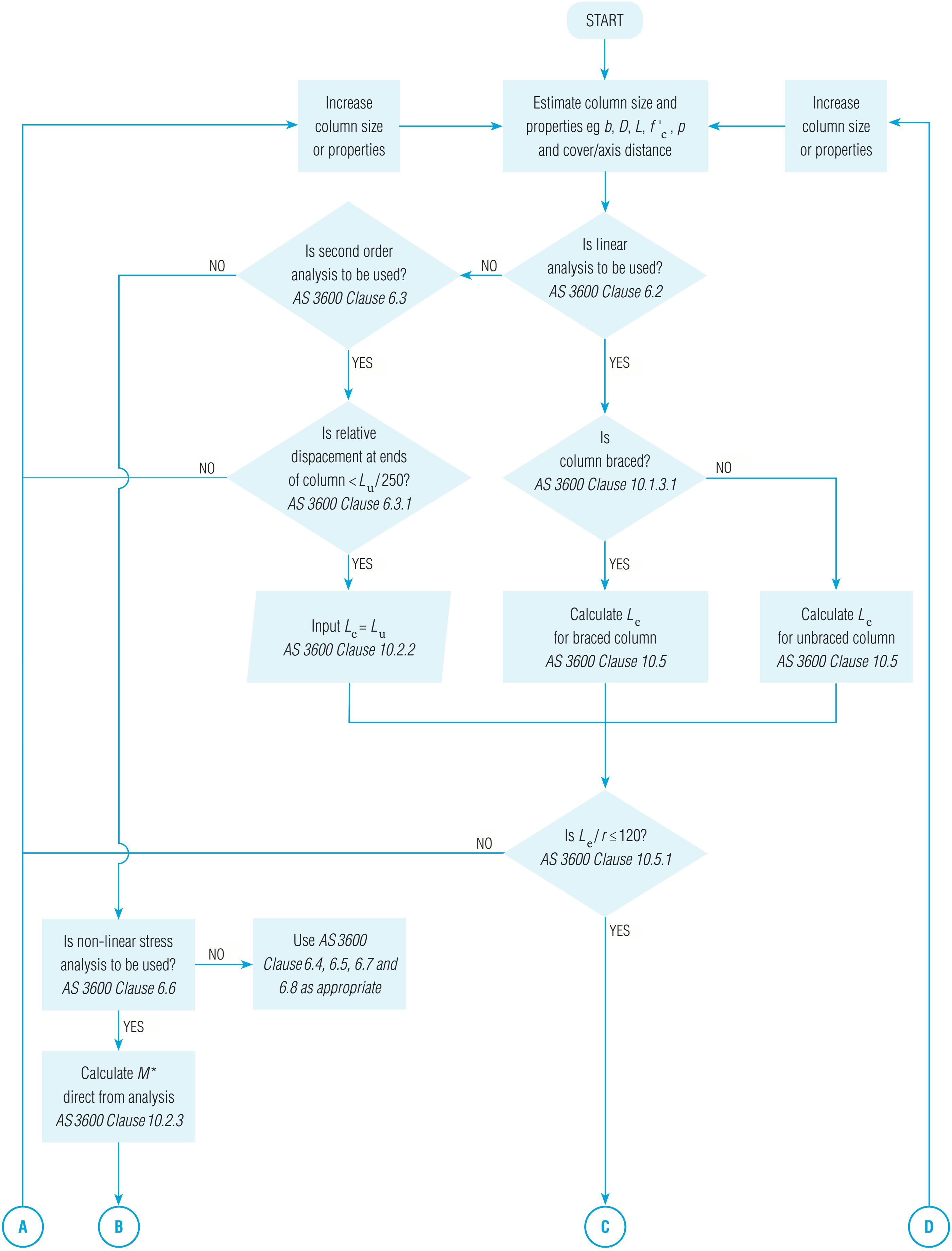
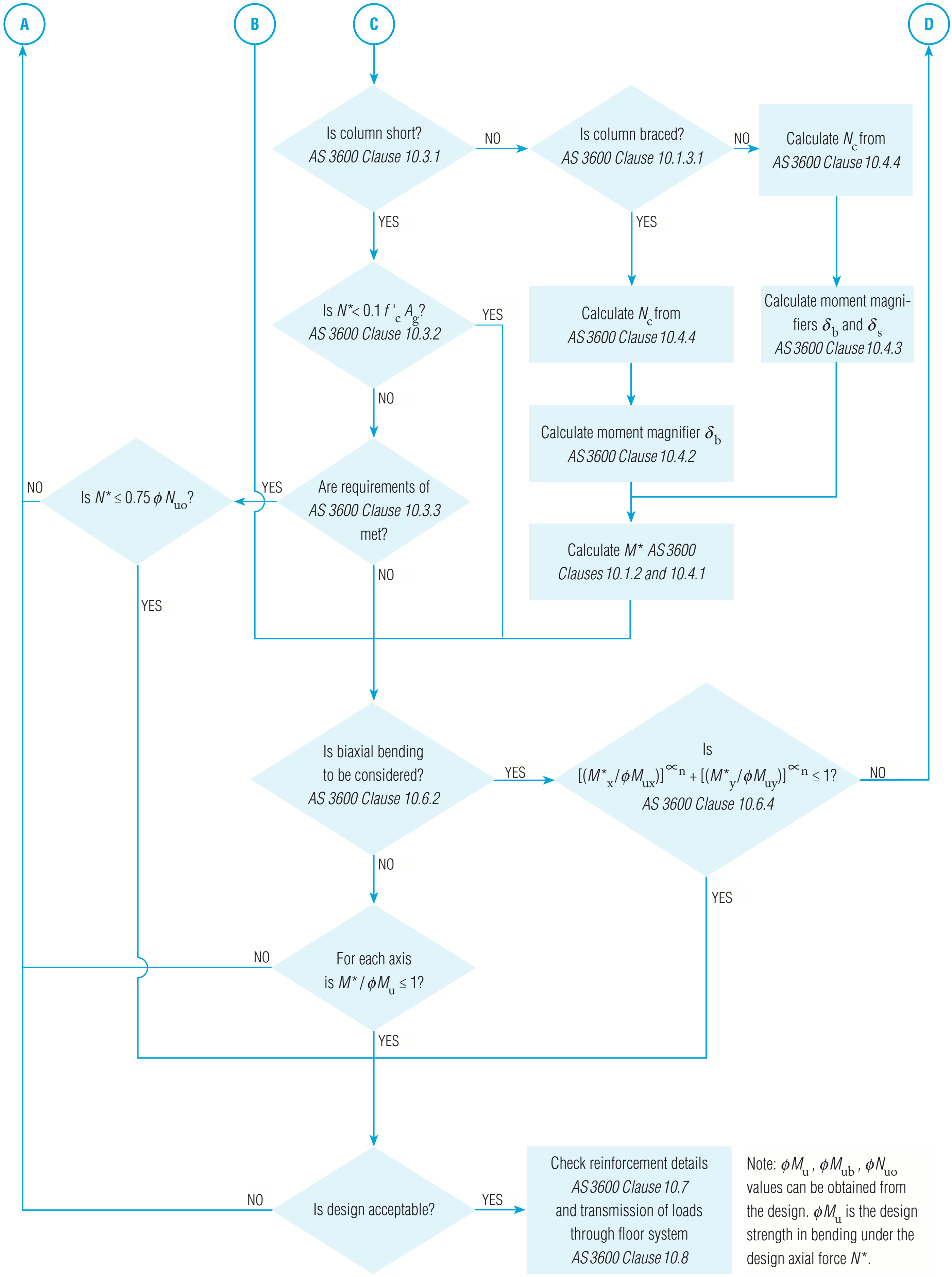


Figure 6.2 Variation of α_1 with f'_c for calculating squash load (Clause 10.6.2.2)

- Determine if the column is short, or slender.
- If the column is slender, determine the moment magnifiers including the buckling load depending on whether it is braced or unbraced.
- Determine the (magnified) moments and choose the larger moment at each end of the column.
- For each load case chosen, check that the applied axial actions and moment are less than the maximum allowed by the moment-strength interaction design curves calculated in accordance with AS 3600 for the chosen column dimensions, concrete strength and area and configuration of reinforcement.
- Iterate as required if the column is under-designed or significantly over-designed.
- Check the design about the other axis, if required, and comply with Clauses 10.6.3 and 10.6.4, if required, for bending about two principal axes.
- Check minimum and maximum reinforcement ratios along with the spacing of bars and fitments and detail the reinforcement as required in accordance with AS 3600 Clause 10.7.

FLOWCHART 6.1 Design of columns in uniaxial or biaxial bending





A
B
C
D
Is column short?
AS 3600 Clause 10.3.1
NO
YES
Is column braced?
AS 3600 Clause 10.1.3.1
NO
YES
Calculate N_c from
AS 3600 Clause 10.4.4
Is $N^* < 0.1 f'_c A_g$?
AS 3600 Clause 10.3.2
YES
NO
Calculate N_c from
AS 3600 Clause 10.4.4
Calculate moment magnifiers δ_b and δ_s
AS 3600 Clause 10.4.3
Calculate moment magnifier δ_b
AS 3600 Clause 10.4.2
NO
Is $N^* \leq 0.75 \phi N_{uo}$?
YES
Are requirements of
AS 3600 Clause 10.3.3
met?
NO
YES
Calculate M^* AS 3600
Clauses 10.1.2 and 10.4.1
Is biaxial bending
to be considered?
AS 3600 Clause 10.6.2
YES
Is
$[(M^*_x/\phi M_{ux})]^{\alpha_n} + [(M^*_y/\phi M_{uy})]^{\alpha_n} \leq 1$?
AS 3600 Clause 10.6.4
NO
NO
YES
NO
For each axis
is $M^* / \phi M_u \leq 1$?
YES
NO
Is design acceptable?
YES
Check reinforcement details
AS 3600 Clause 10.7
and transmission of loads
through floor system
AS 3600 Clause 10.8
Note: ϕM_u, ϕM_{ub}, ϕN_{uo}
values can be obtained from
the design. ϕM_u is the design
strength in bending under the
design axial force N^*.

Because of the many variables involved, design is usually best carried out using appropriate column design software or a suitable spreadsheet developed to AS 3600.

6.4 BASIS OF CHARTS 6.1 TO 6.6

Load-moment interaction curve

In its simplest form, the load-moment interaction curve for a column cross-section is constructed by calculating four points on the boundary of the curve and the connecting straight lines or curves between them. These points plot: the axial strength at zero moment on the vertical axis; the bending capacity at zero axial action on the horizontal axis; the point at which the neutral axis coincides with the outermost layer of tension reinforcement; and the point at which the tension reinforcement just begins to yield. Figure 6.1 shows a typical load-moment interaction curve for a column cross-section using the rectangular stress block.

AS 3600 Clause 10.6.1 allows for the strength of column cross-sections to be calculated using stress-strain relationships as given in Clauses 3.1.4 and 3.2.3. Note that a limit is placed on the maximum value of concrete stress. While the CEB[6.5] curves may be adequate for normal concrete, other stress-strain curves[6.6] may be more appropriate for high strength concrete and such curves can be used, provided they comply with AS 3600. The design booklet[6.7] contains a discussion on the design of columns and the principles of design.

The squash load, N_{uo}, is a theoretical design point because AS 3600 requires a minimum moment of $0.05\,DN^*$ for any concrete column. It is calculated in accordance with AS 3600 Clause 10.6.2.2 where

$$N_{uo} = \alpha_1 f'_c A_c + A_{sc} f_{sy}$$

and

A_c = the net area of concrete and

A_{sc} = the area of column reinforcement.

The factor α_1 is to allow for shrinkage and the fact that the concrete will carry less load in the long term than it would when initially loaded. AS 3600 defines α_1 as $\alpha_1 = 1.0 - 0.003 f'_c$ with lower and upper limits of 0.72 and 0.85 respectively. The modification of $0.9 f'_c$ is included in the calculation of α_1.

Figure 6.3 shows graphs of the squash load for a square column with various percentages of reinforcement and various concrete strengths. As can be seen from the graph, the addition of reinforcement has the greatest effect for low concrete strengths.

The decompression point is calculated by taking the strain in the extreme compressive fibre as 0.003, the strain in the extreme tensile fibre as zero and using the rectangular stress block given in AS 3600 Clause 10.6.2.5.

For the transition from squash load to the decompression point, AS 3600 allows the section strength to be calculated using a linear relationship between the decompression point and the squash load.

The transition from decompression point to the balanced point to the bending strength is set out in AS 3600 Clause 10.6.2.5.

The design strength in bending, M_{ub}, and the corresponding design strength in compression, N_{ub}, are referred to as the balance point and can be determined as the locus of values for which $k_{uo} = 0.545$ (with $\phi = 0.6$). The value of k_{uo} is determined assuming the balance point is the point at which the steel yields at a strain of 0.0025 and the strain at the extreme compressive fibre of the concrete is 0.003.

Because each load-moment strength interaction diagram depends on so many variables, hand calculations are generally not feasible. Either spreadsheets or column design software are used to develop a load-moment strength interaction diagram for each chosen column size and configuration.

The calculated load-moment strength interaction diagram as shown in Figure 6.1 is then modified by applying the capacity reduction factor, ϕ, to give a design load-moment strength interaction diagram for design from which the chosen column configuration can then be checked.

The capacity reduction factor, ϕ, varies in accordance with AS 3600 Table 2.2.2.

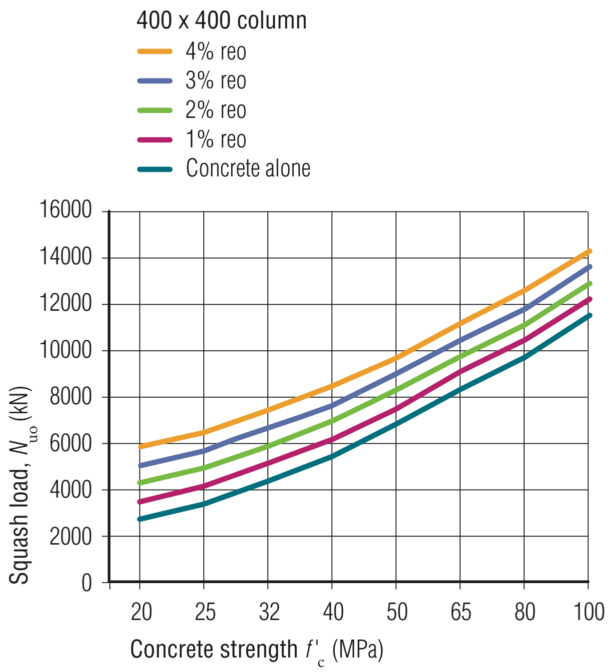


Figure 6.3 Squash load vs concrete strength

Design charts

The design charts provided in this Chapter give the design strength of square reinforced concrete cross-sections in combined uniaxial bending and compression in one direction only and are for specific design parameters only. The strength reduction factor, ϕ, is included. The square columns have equal reinforcement only in two faces and have an axis distance of 60 mm.

Preliminary design of circular columns can be made using the square colmn charts with some interpolation.

The strengths were calculated based on equilibrium and strain-compatibility considerations consistent with the assumptions of AS 3600 Clause 10.6.1.

6.5 AXIAL SHORTENING

All concrete compression members shorten under axial load because of shrinkage and creep. Shortening for columns is typically about 1 mm per m in height but it can be calculated by various methods in accordance with AS 3600. This shortening must be considered in the design of taller buildings including for example: the support of the facades of multi-storey buildings to avoid non-loadbearing walls carrying load, in the detailing of lift guide rails in the services core, and the junction of the high-rise core, of a building with a low-level podium.

Further investigation is also required where a highly stressed column is adjacent to a much-lower-stressed column or wall. For example, an internal column located, say, 2 m from concrete core walls to reduce the floor span may result in significant differential deflections of the slab across the short span.

There can also be problems if there is a change in the plan dimensions of floors in a tall building resulting in adjacent columns on the floor below the change carrying significantly different loads, eg one column might be supporting 20 storeys and the next, just a roof. In such a case the designer should try to even out the loads, eg by prestressing the lightly loaded column. Another solution is to provide separation joints between the low-rise section and a high-rise section.

6.6 HIGH-STRENGTH CONCRETE (HSC)

For the purposes of this Handbook, high-strength concrete has been defined as a concrete where the concrete strength, f'_c, is greater than 50 MPa. This is consistent with AS 1379, where such concrete is required to be Special Class Concrete.

High-strength concrete is becoming increasingly available and is used to increase the load capacity of columns or to reduce their dimensions. HSC is less ductile than normal concrete, while its use results in an increase in axial capacity but has limited effect on moment capacity. Research in recent years has shown that an increase in strength and ductility of HSC columns can be achieved by well-detailed lateral confinement reinforcement. Robustness and ductility are important for columns and extra attention is required in the detailing of the fitments in HSC columns.

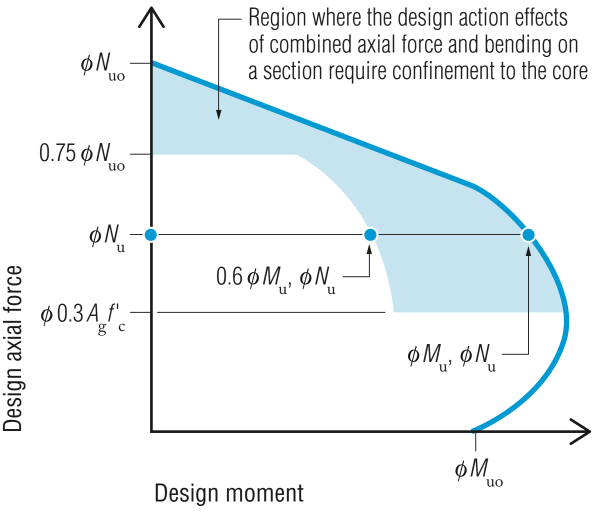


Figure 6.4 Confinement of the core *(after AS 3600)*

AS 3600 Clause 10.7.3 sets out the requirements for confinement reinforcement both in the special confinement areas and outside these areas as shown in Figure 6.4.

Where the fitment spacing does not exceed the values set out in the Standard, AS 3600 Clause 10.7.3.4 provides a deemed-to-comply approach for the design of the core confinement for rectangular and circular sections in lieu of detailed calculations of the core confinement by rational methods.

For further information on HSC, designers can refer to Foster[6.8], Mendis[6.9], and other papers[6.10].

6.7 DETAILING

Designers should refer to Chapter 12 of the *Reinforcement Detailing Handbook*[6.11] for further information on detailing of columns.

Designers should:

- Provide sufficient information on the drawings to build the columns including elevations, schedules, sections and details along with the column orientations and the required concrete strengths.
- Provide the north point on the schedules to match the plans and orientation of the column if the column has different reinforcement in different faces or is rectangular.
- Provide offset dimensions if columns are not concentric on grid lines.

- Show any special detail such as at the footings and floors, column offsets, columns supported by transfer beams, columns terminating or changing size, column heads, cast-in bolts, etc.
- Always use the same bar size in one lift of a column to avoid errors in fixing on site.
- Generally provide the longitudinal reinforcement in one lift, ie one storey height as double-storey column bars in larger sizes may be difficult to restrain in the correct position above the formwork.
- Try to use larger and fewer bars rather than many small bars to reduce the number of fitments.
- Consider the beam/column intersection as discussed in Chapter 4 of this Handbook and the *Reinforcement Detailing Handbook*.
- Consider using the cranked section of the column bars at the top of the column when lapping column bars so the straight bar can go through a beam. An alternative is to crank the bars below the beam so that the straight section goes through the beam. This is not so critical for band beams or flat slabs. Figure 6.5 illustrates this issue.
- Check that the longitudinal reinforcement provided satisfies the requirements for minimum and maximum reinforcement given in AS 3600 Clause 10.7.1.
- Provide minimum bar diameter for fitments for rectangular columns and helices for circular columns in accordance with AS 3600 Table 10.7.4.3.
- Provide appropriate lapped splice lengths for the longitudinal column bars depending on whether the bars are in tension or compression.

Designers should note that:

- While end-bearing splices are allowed by AS 3600, the thin sleeve splices to the bars are no longer generally available in Australia. Several types of thicker sleeve systems and a threaded coupler system are available for splicing bars in line.
- Column cages can often be fabricated away from the forms and lifted into place by a crane. Bundled bars cannot usually be prefabricated.
- It is usual to terminate the column reinforcement with cogs into the slab or beam at the top of the building for fixity. Consider drop-in bars prior to the column being cast, to allow easier alignment with beam or slab reinforcement.
- Fitments are provided to prevent the vertical column bars from buckling under compression loads, to resist shear and torsion, for ductility for earthquake actions if required and for HSC to confine the core. A number of design conditions therefore need to be satisfied in choosing the size and spacing of fitments.

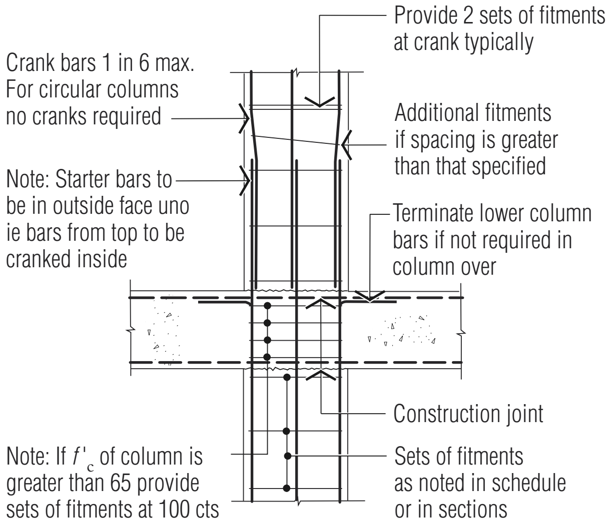


Figure 6.5 Cranking of longitudinal bars

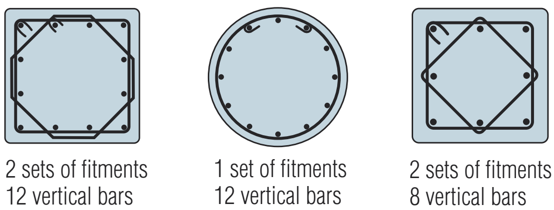


Figure 6.6 Restraint of longitudinal bars

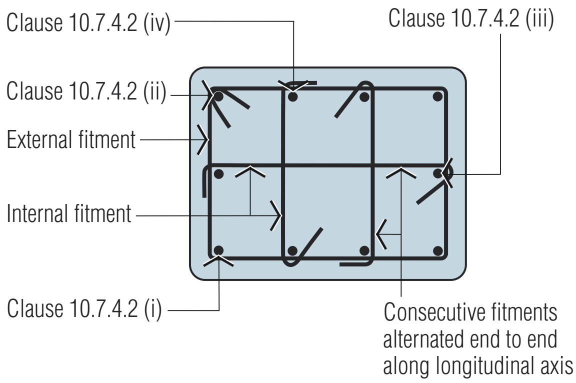


Figure 6.7 Lateral restraint of longitudinal bars *(after AS 3600)*

- For longitudinal bars in square or rectangular columns, all corner bars, all bars spaced further apart than 150 mm, and every alternate bar where the spacing is less than 150 mm have to be restrained in two directions (see AS 3600 Clause 10.7.4.2). Circular columns do not need this restraint provided a circular fitment or helical reinforcement (anchored in accordance with Clause 10.7.4.4) is used and the longitudinal bars are equally spaced around the circumference. See Figure 6.6 for some examples of fitments and restraint of vertical bars.

- To facilitate column construction an alternative method of assembly of internal column fitments is permitted by AS 3600 Clause 10.7.4.2. It allows internal fitments to include a 135° hook at one end with a 90° hook at the other, provided consecutive internal fitments are alternated end to end along the longitudinal reinforcement. This allows the internal fitments to be fixed on site. See Figure 6.7.
- Where bending moments from a floor system have to be transferred to a column and the column is not fully surrounded by a slab or beams of approximately equal depth, then lateral shear reinforcement of area $A_{sv} \geq 0.35\, b\, s/f_{sy.f}$ in the form of fitments must be provided through the joint (Clause 10.7.4.5). This Clause will usually apply to all external columns. This requirement can sometimes cause difficulty with the fixing of the beam reinforcement. See Figures 6.8 and 6.9.
- Where there are large changes in column dimensions, column bars cannot usually be offset and separate starter bars will be required to be cast into the column below. See Figure 6.10.
- Seismic detailing, if required, in accordance with AS 3600 Appendix C.

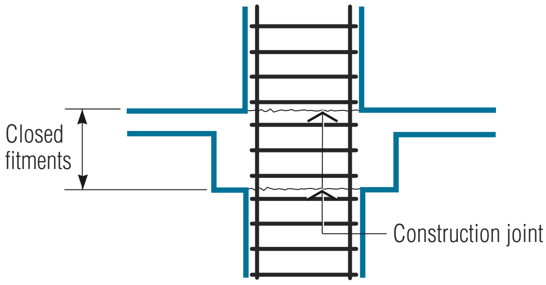


Figure 6.8 Lateral shear reinforcing

Figure 6.9 Columns requiring lateral shear reinforcement

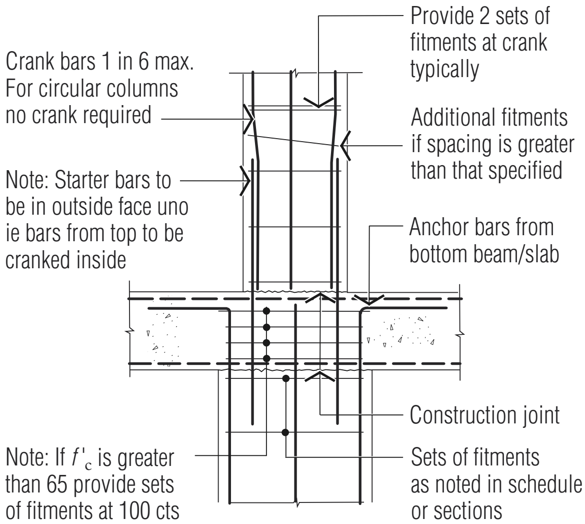


Figure 6.10 Change in column size

6.8 GENERAL GUIDANCE

The following points will assist in the design and inspection of columns for a particular project.

- While AS 3600 permits the use of 200-mm x 200-mm columns, a minimum dimension (for square, rectangular or circular columns) of 250 mm is recommended. Column dimensions are usually a multiple of 50 mm.
- The position and shape of columns is often dictated by architectural and spatial requirements, eg circular columns for appearance or rectangular columns in carparks and blade columns in apartment buildings.
- For circular columns, thin spiral galvanised metal or plastic is sometimes used as permanent formwork. It is important that cleaning out of the bottom of the column is achieved before placing of the formwork.
- Consider using HSC rather than increasing the reinforcement ratio or changing column sizes, subject to the transmission of axial forces through the floor. AS 3600 allows the use of HSC up to 100 MPa but concrete strengths higher than 50 MPa involve Special Class Concrete with the requirements of project control testing and extra detailing for confinement. Also check if the local concrete suppliers can supply the HSC concrete specified.
- Before specifying above 65-MPa concrete, an alternative verification path will be required to satisfy the requirements of the BCA until such time as AS 3600—2009 is called up in the BCA.
- On successively higher floors, reduce the concrete strength and or reinforcement rather than changing the column sizes, as column formwork is expensive. A uniform column size will also often be desirable for visual reasons, particularly with facade columns.

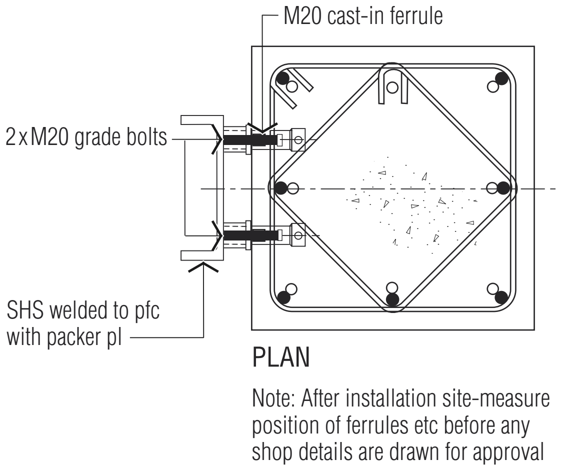


Figure 6.11 Cast-in ferrules to columns

- Check that the layout of fitments will allow placing and compaction of the concrete and the forms to be cleaned out.
- Remember to check the transmission of axial forces through the floor (see AS 3600 Clause 10.8).
- Edge columns and corner column are often more critical for design than internal columns because of high bending moments.
- Avoid, wherever possible, casting in large services such as downpipes, sewer stacks, etc in columns because of future maintenance and durability problems. There is also the complication of getting the services in and out of the column and clashes with reinforcement, especially at the bottom of the column. This area of a column is usually highly stressed and there is the problem of assessing what the reduced strength of the column will be with a section of the column removed for a large service duct.
- The column below a slab is sometimes cast with the floor slab to minimise scaffolding and allow access from the floor formwork to cast the column. In this situation, columns should be poured at least 1–2 hours before the slab over, to allow for settlement of the concrete in the column to take place, before the concrete in the slab is placed.
- Avoid cast-in bolts and cleat plates projecting from the side of columns because of the complication with formwork (especially in stripping) and the difficulty of placing and compacting the concrete. Consider using cast-in ferrules or a cast-in plate, which allows some tolerance and onto which cleat plates and the like can be welded after the formwork is stripped. See Figure 6.11.
- Where bolts are cast into the top of the column for the roof structure and the like, use a template to align the bolts correctly. Avoid the use of threaded reinforcement for this purpose because of the difficulty in accurately aligning the reinforcement and achieving the tolerances usually required for holding-down bolts.
- Consider using concrete spacer blocks when durability is an issue. Care is needed to ensure that voids do not occur under the blocks, so consideration may need to be given to the use of super plasticisers and smaller aggregates.
- For precast columns, be careful with choice of dowel ducts, which are commonly used to provide sleeves into which the projecting dowel bars from the concrete below are grouted. Generally, dowel ducts should be of thin galvanised steel 2–3 times the size of the dowel bar. Smooth-faced plastic dowel ducts should be used only where the joint is pinned and there is no tension.

CHARTS 6.1 to 6.7

These charts have been developed on the following assumptions:

- Equal reinforcement is in only two opposite faces, resisting bending.
- An axis distance of 60 mm from the face of concrete to the centre of the longitudinal bar, ie a cover of about 30 to 40 mm depending on the fitment and column bar size.
- Bending moments about the one axis only.
- Concrete strengths in the range of 25 to 50 MPa.

Chart 6.1 300 x 300 Columns
pages 6.14–6.15

Chart 6.2 400 x 400 Columns
pages 6.16–6.17

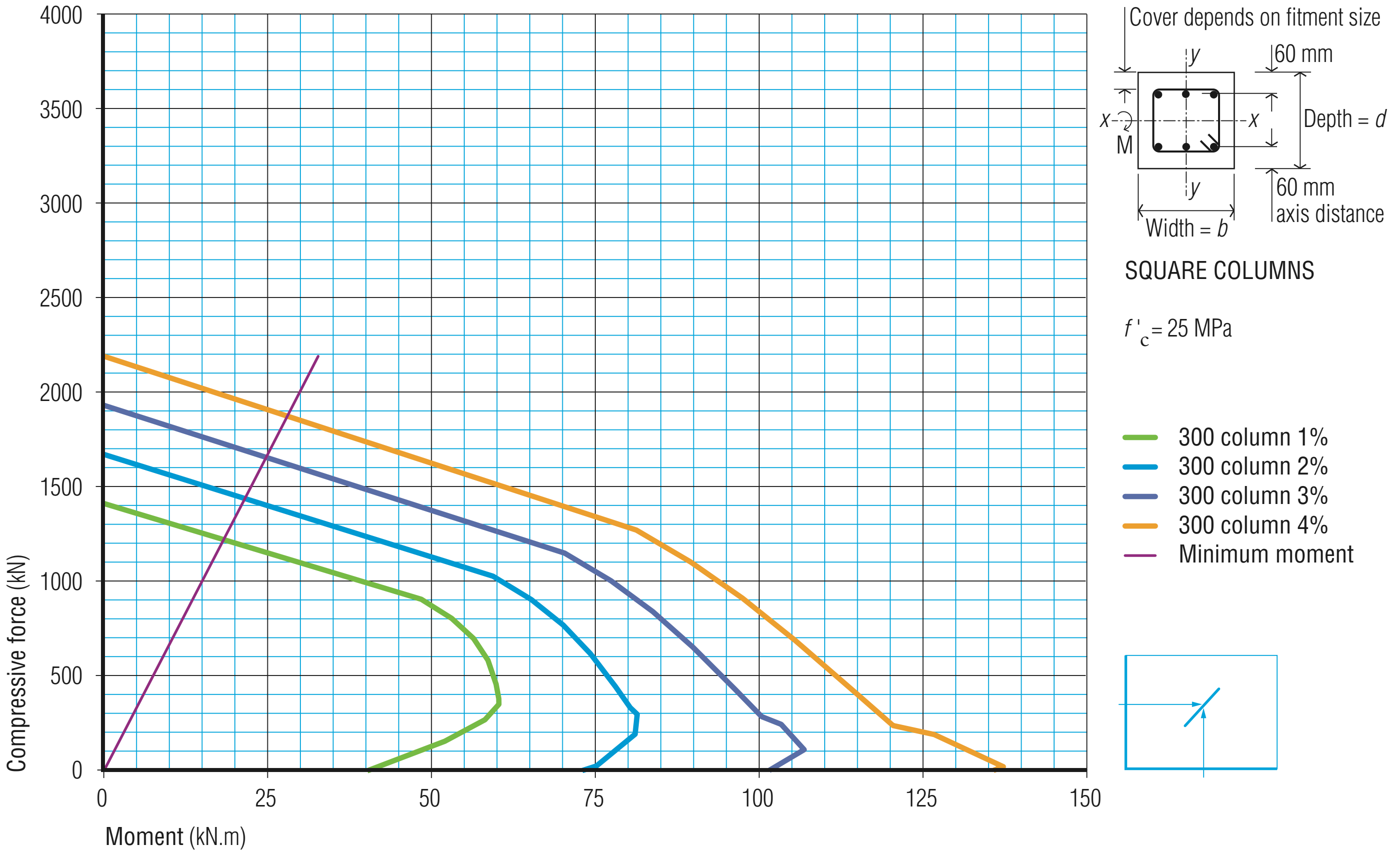
Chart 6.3 500 x 500 Columns
pages 6.18–6.19

Chart 6.4 600 x 600 Columns
pages 6.20–6.21

Chart 6.5 700 x 700 Columns
pages 6.22–6.23

Chart 6.6 800 x 800 Columns
pages 6.24–6.25

CHART 6.1 300 x 300 Columns



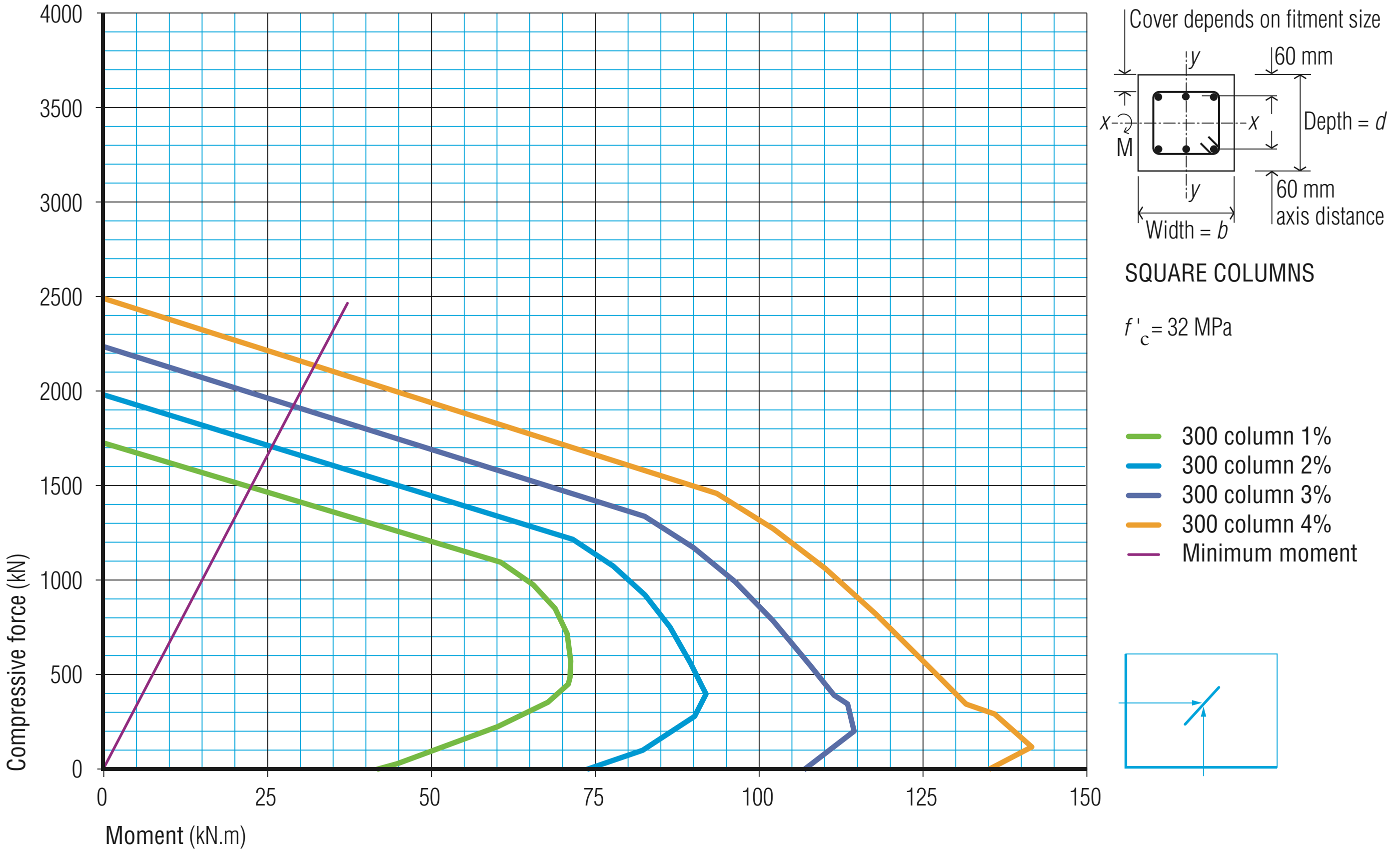
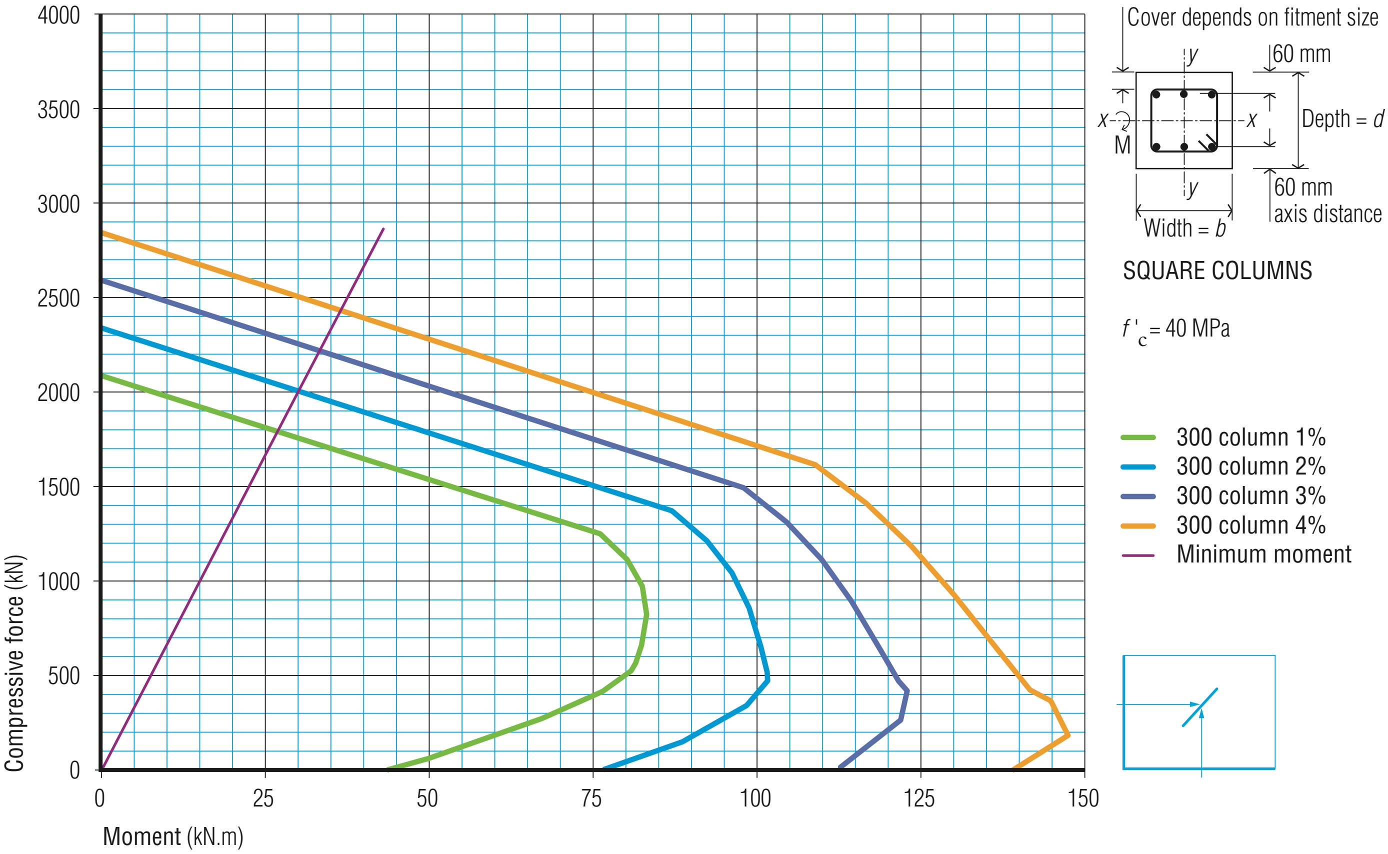


CHART 6.1 *(continued)* 300 x 300 Columns



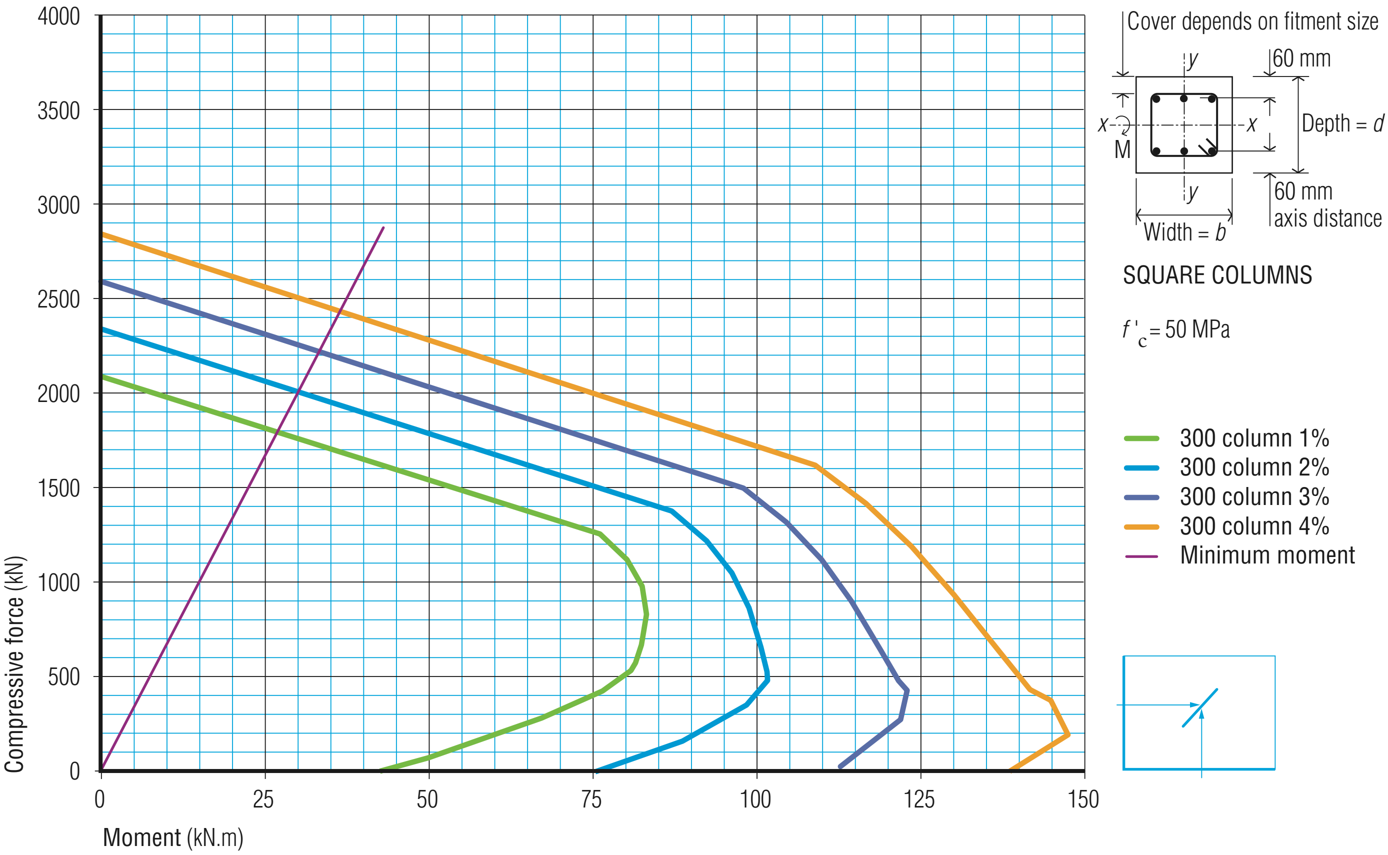
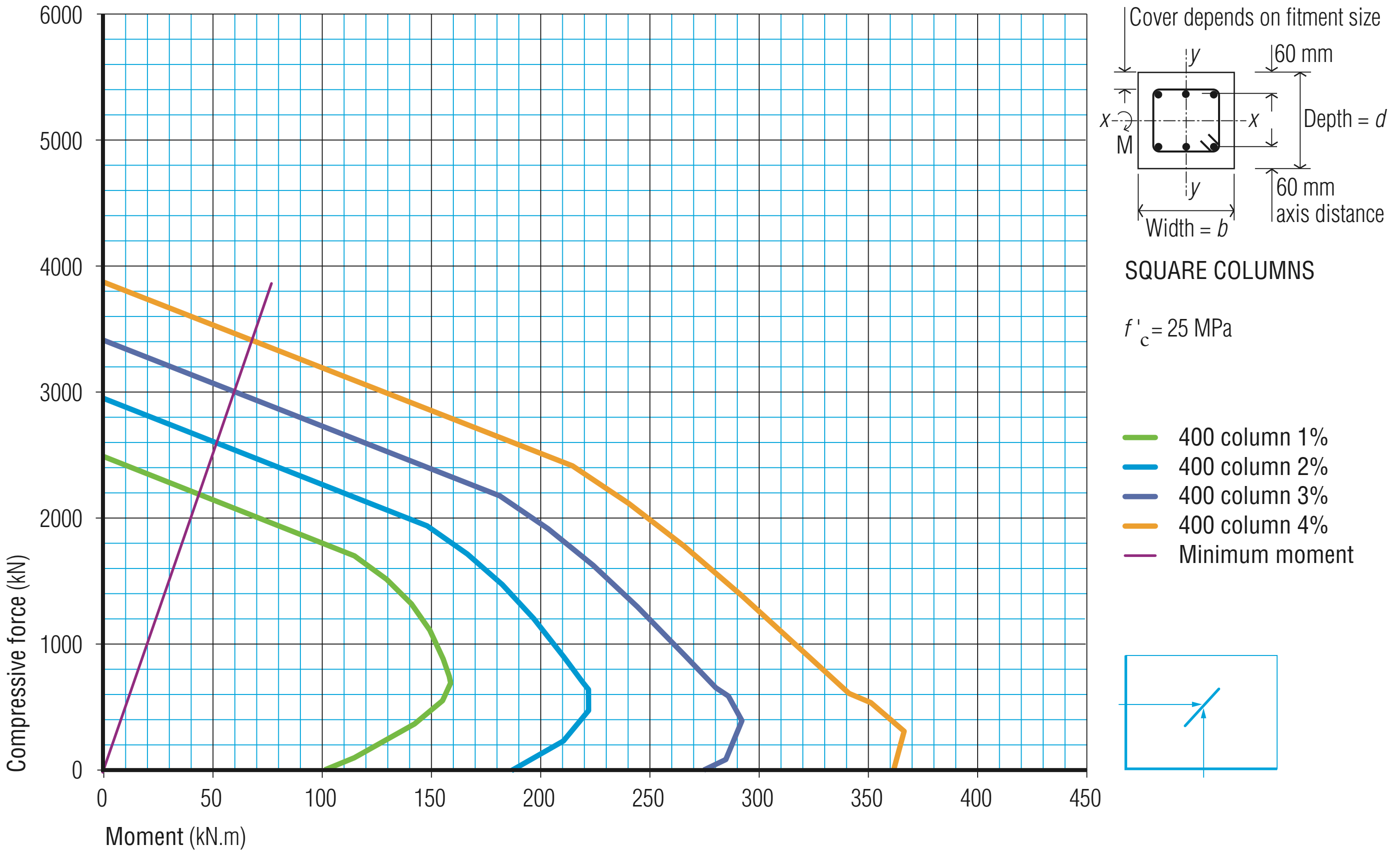


CHART 6.2 400 x 400 Columns



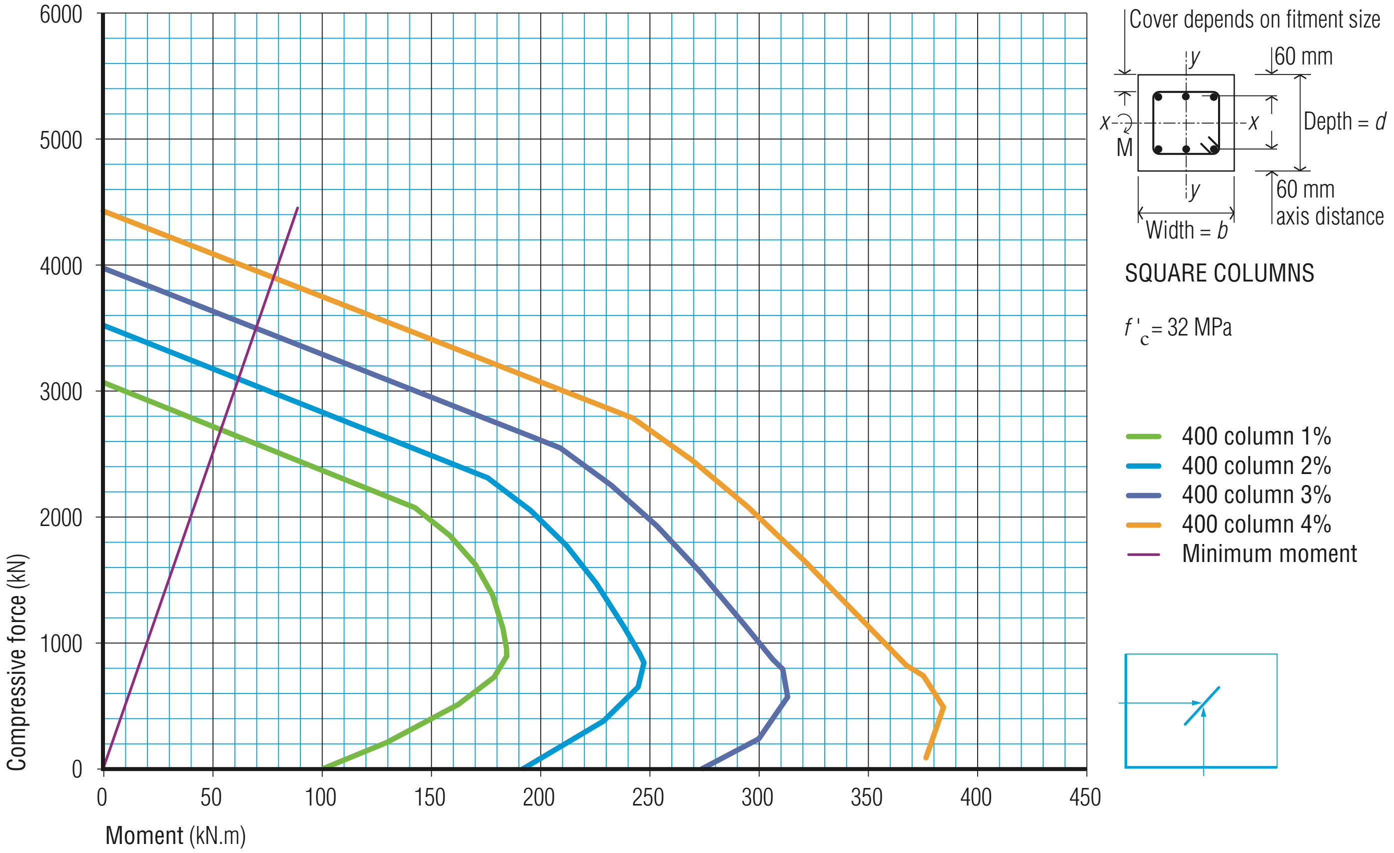
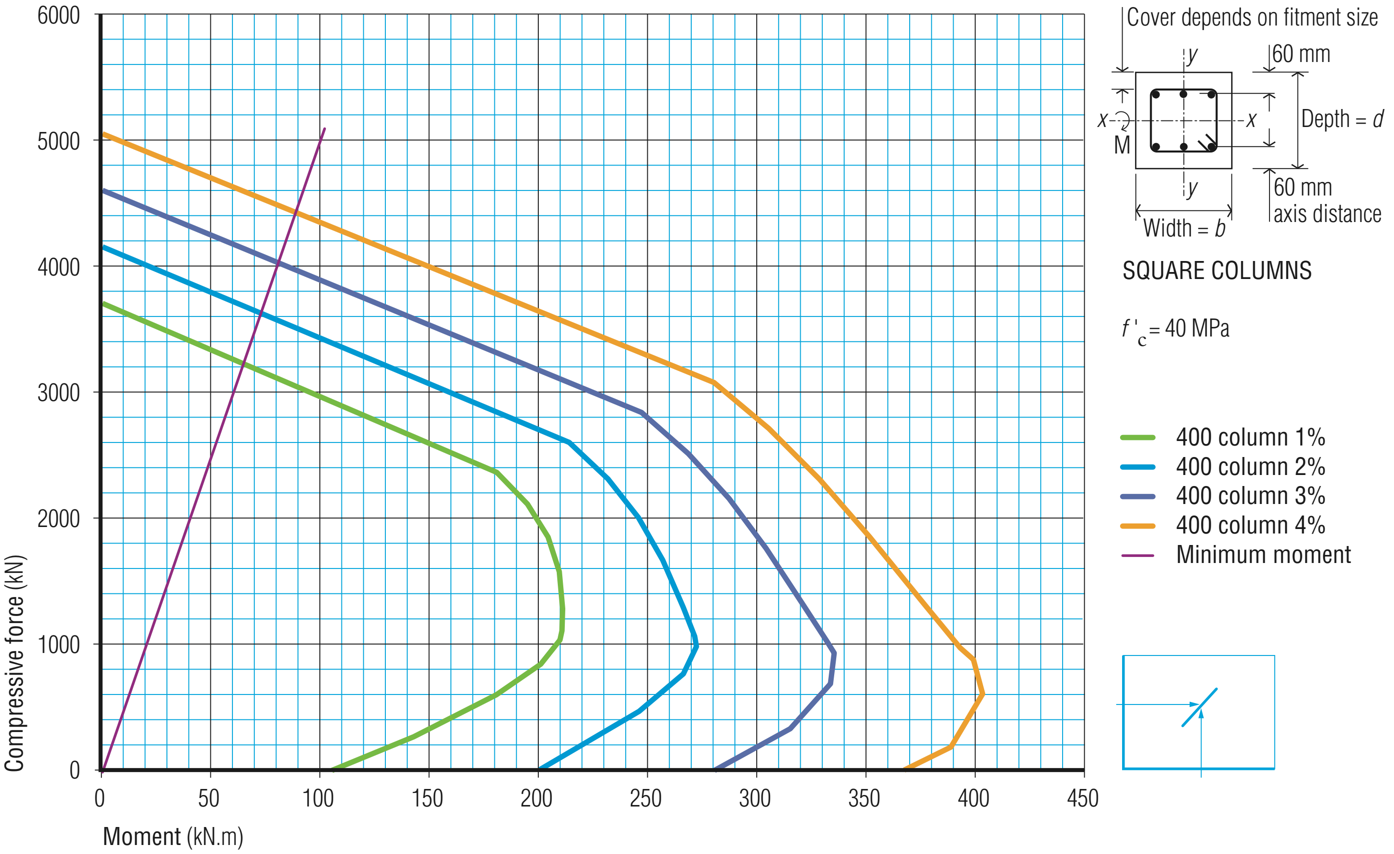


CHART 6.2 *(continued)* 400 x 400 Columns



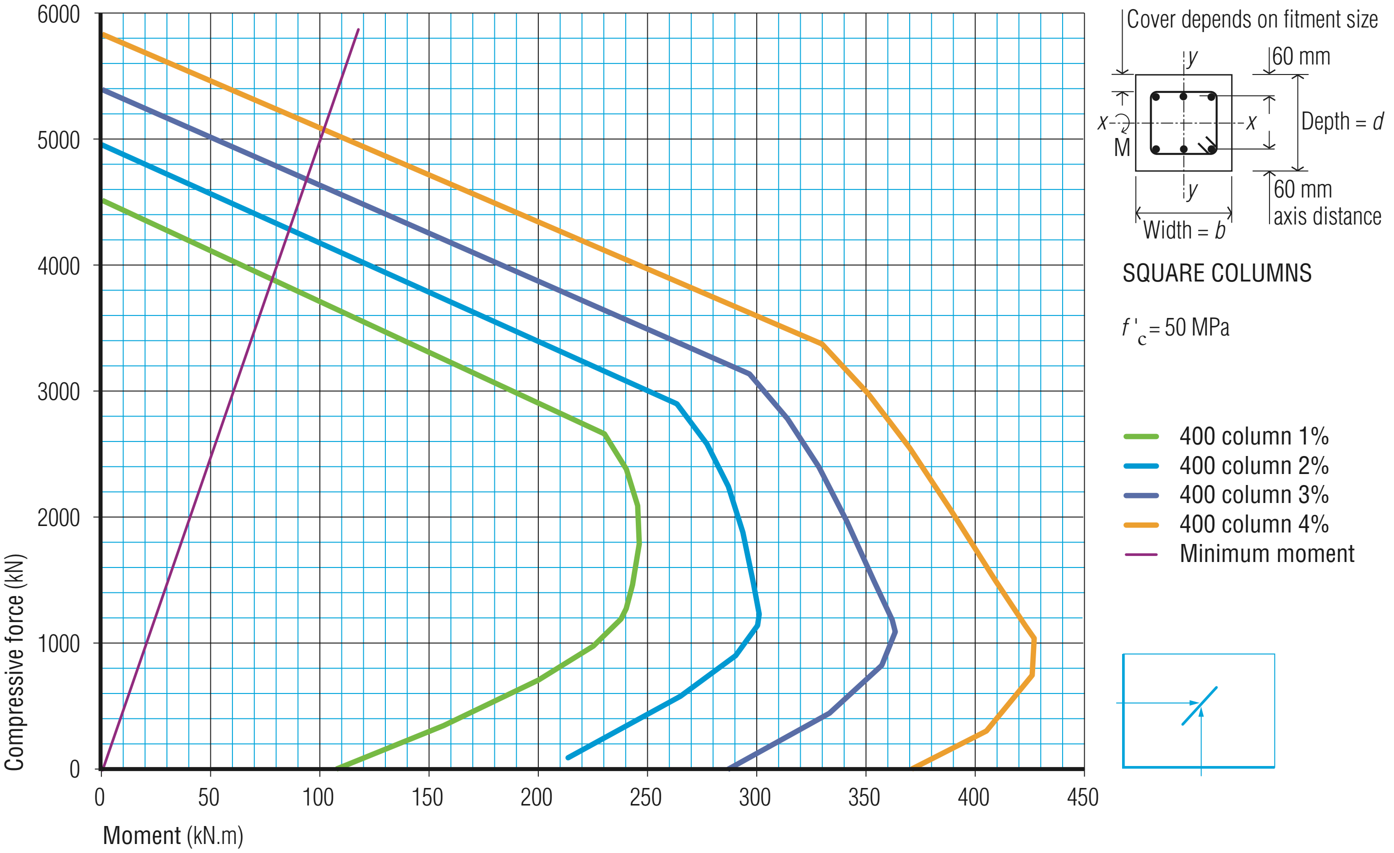
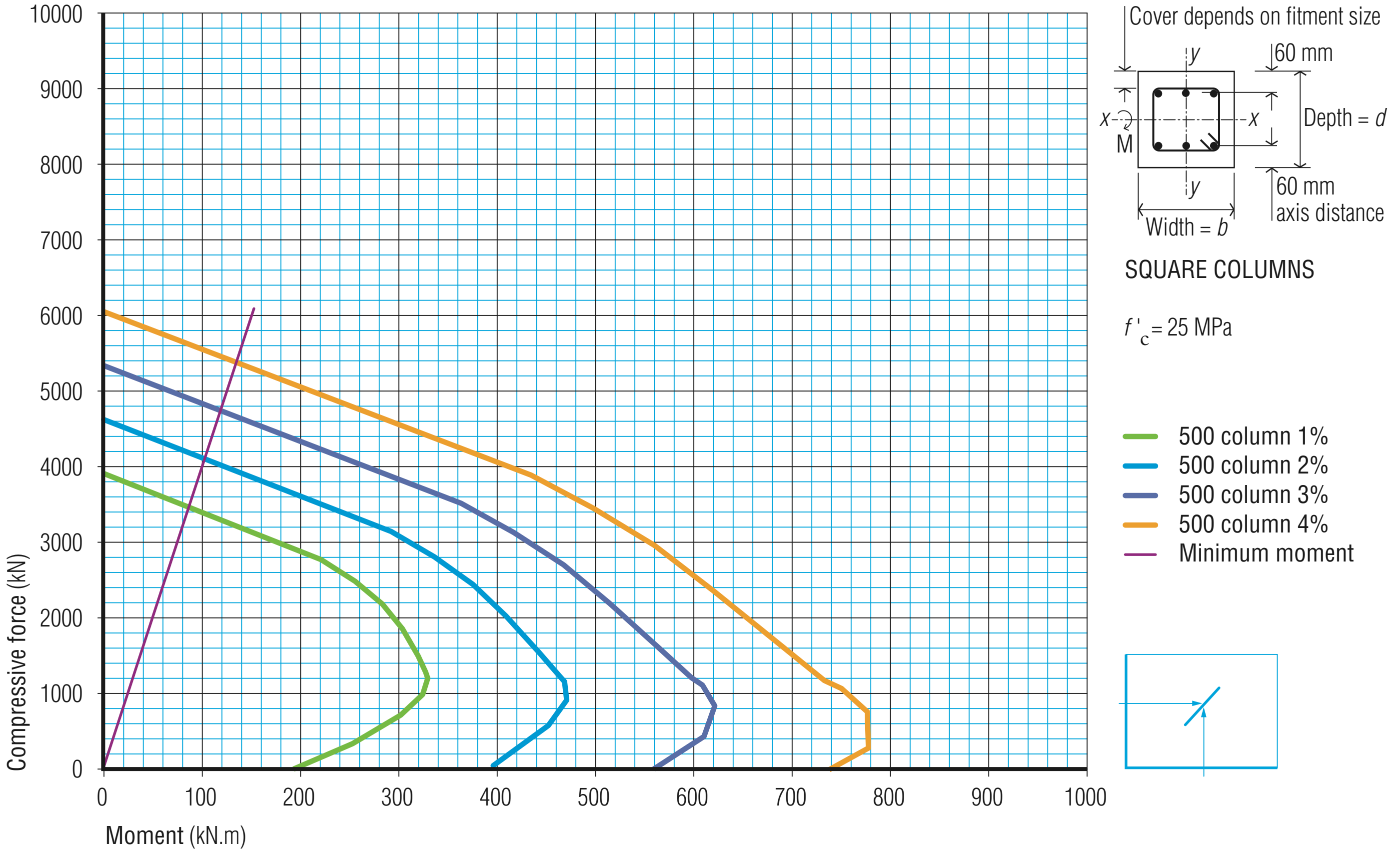


CHART 6.3 500 x 500 Columns



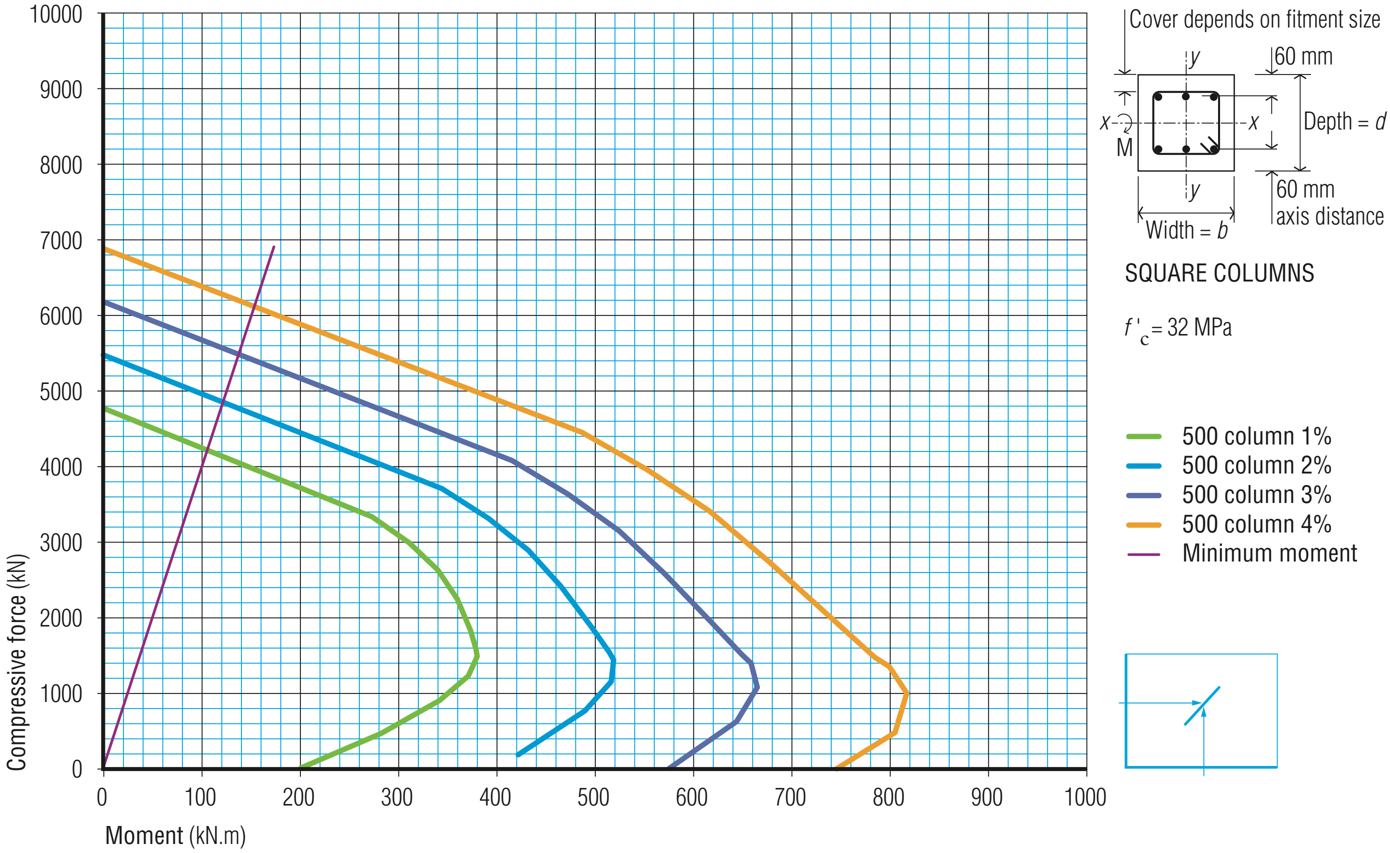
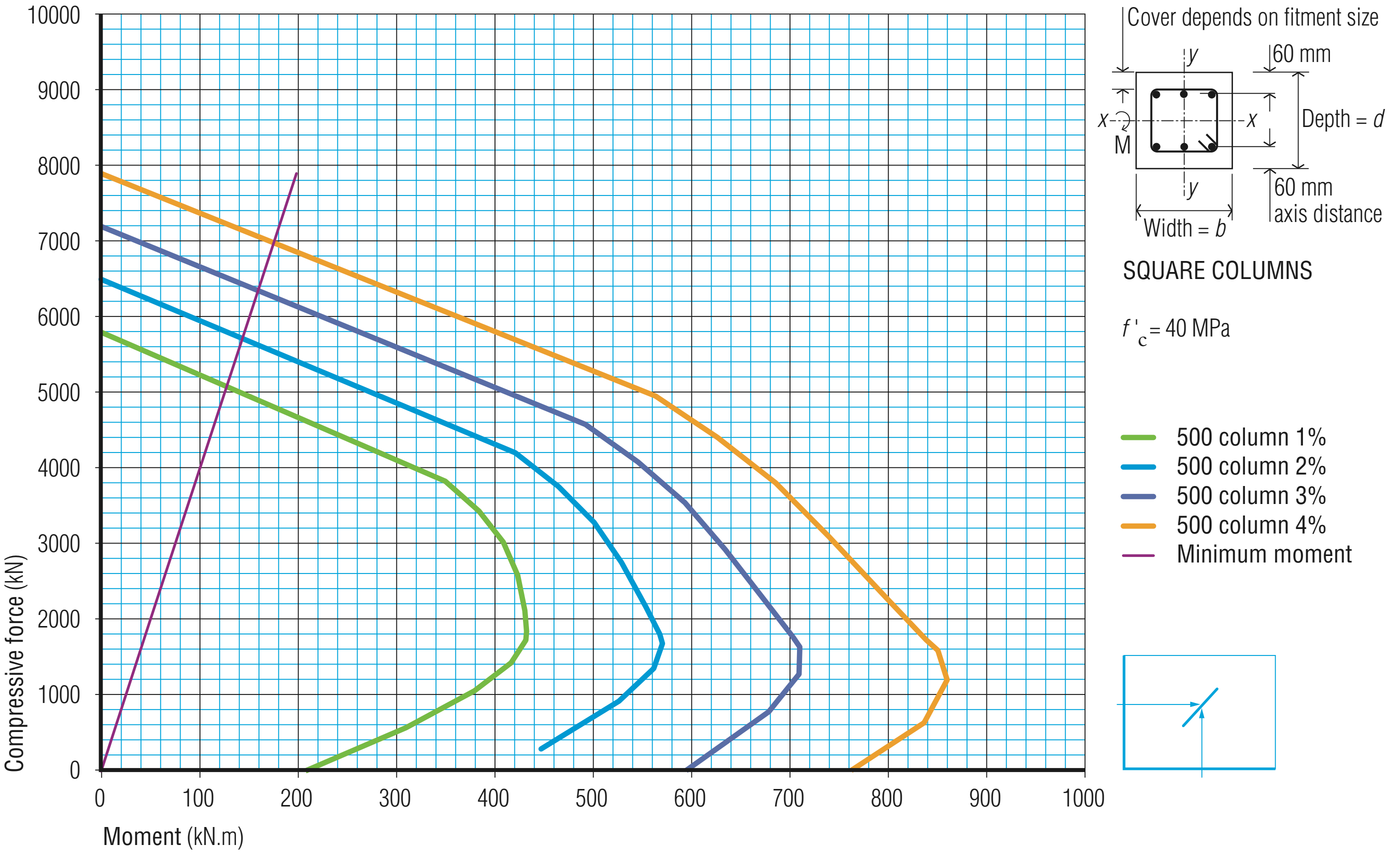


CHART 6.3 *(continued)* 500 x 500 Columns



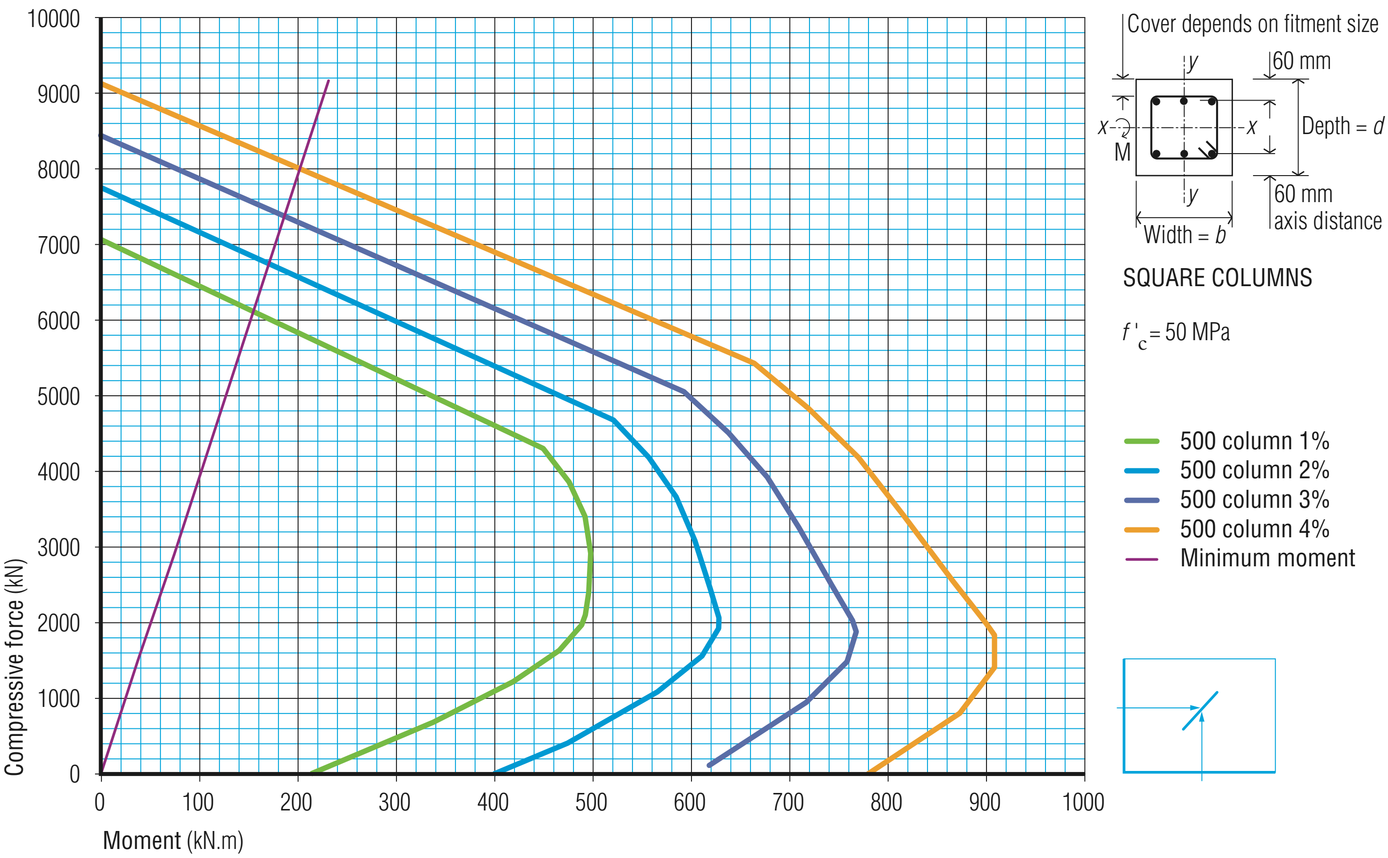
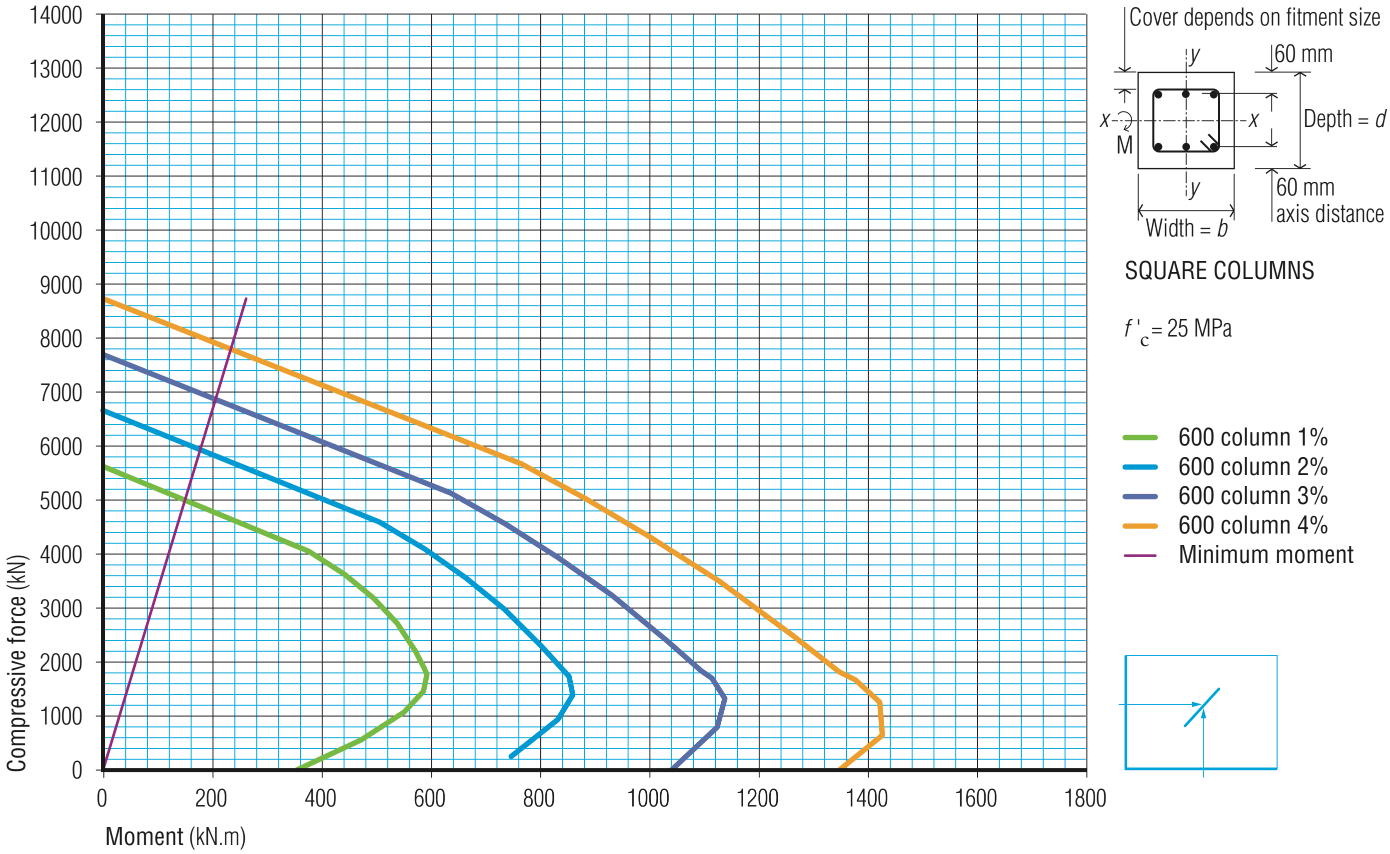
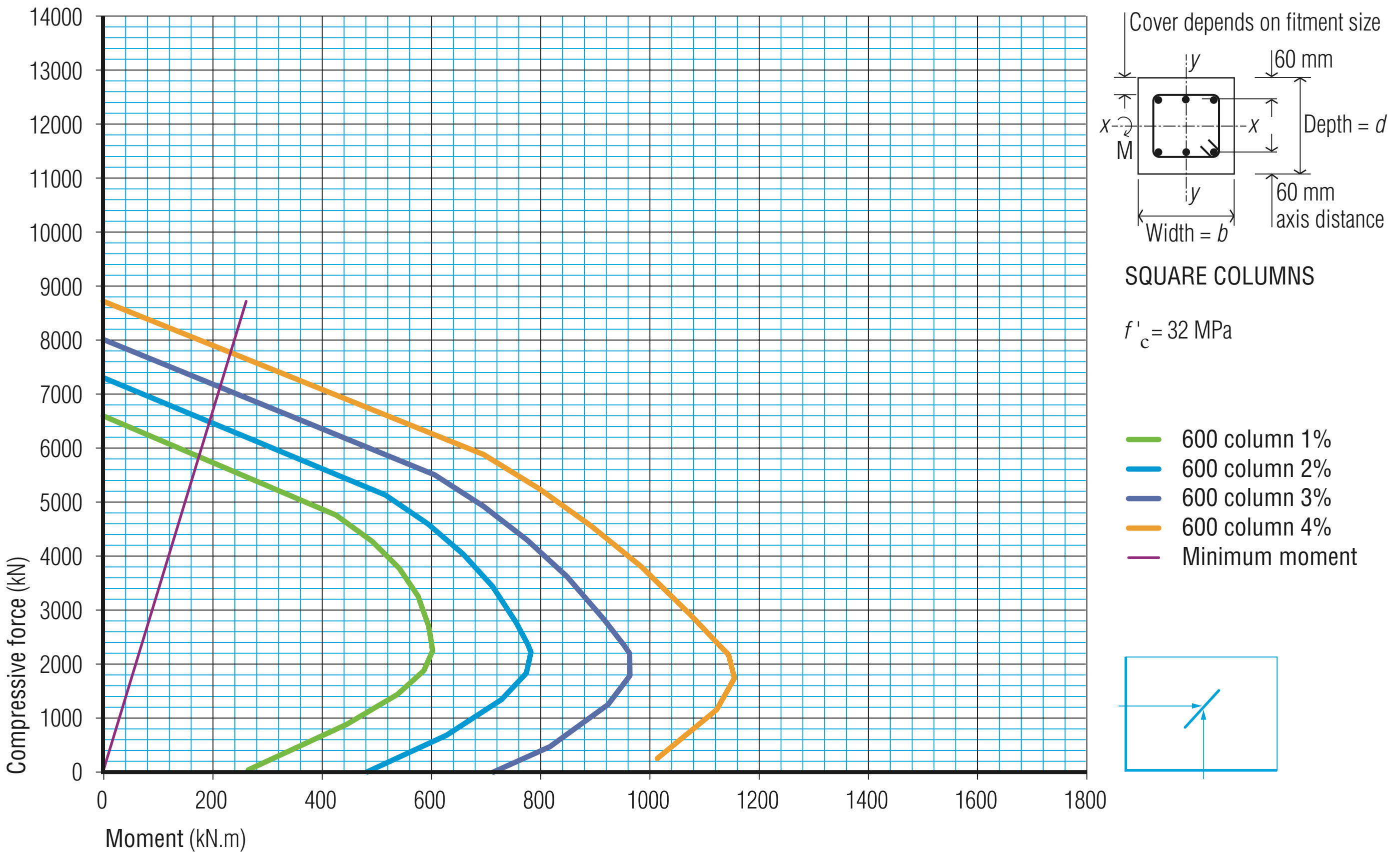
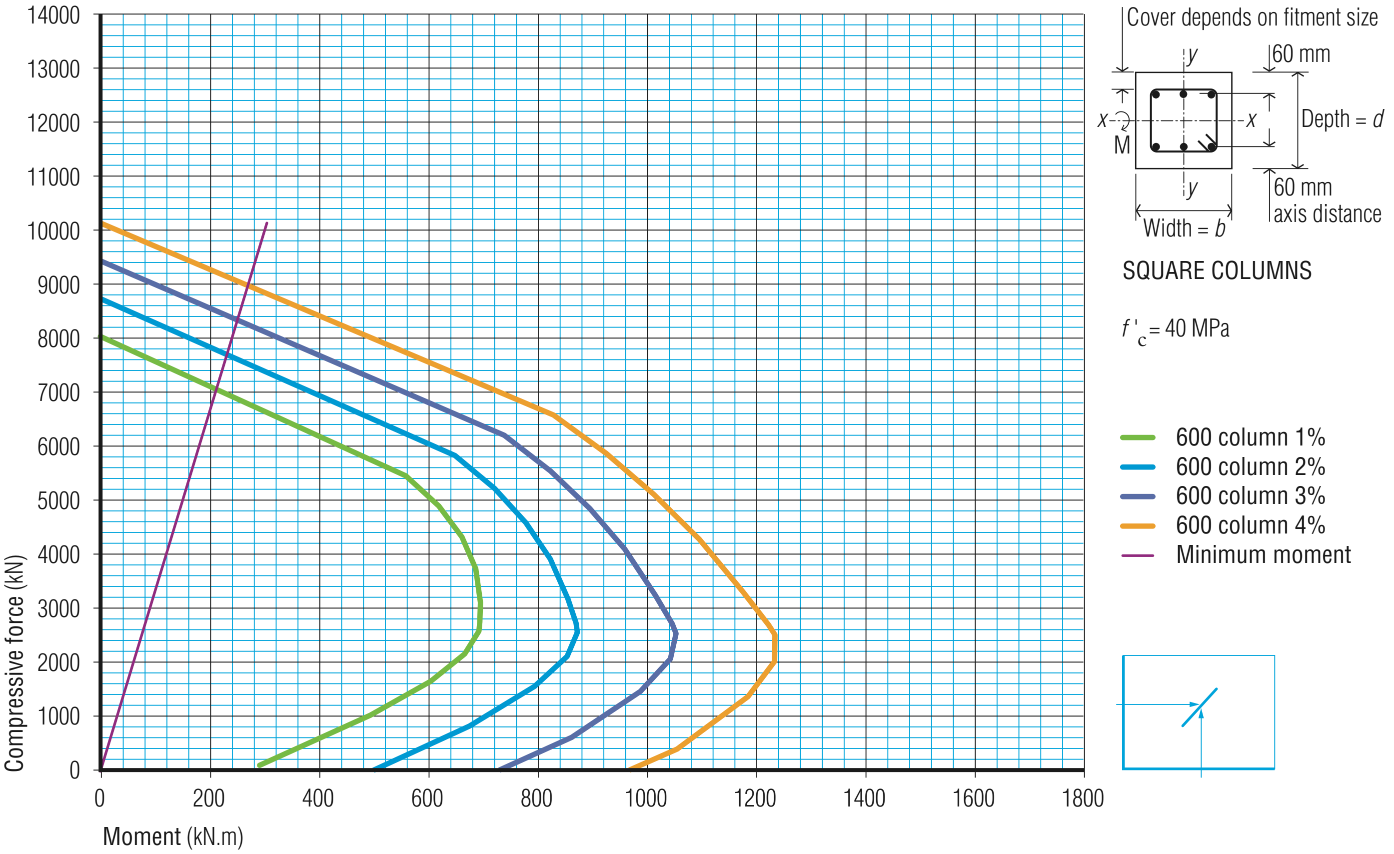


CHART 6.4 600 x 600 Columns






14000
13000
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Compressive force (kN)
0
200
400
600
800
1000
1200
1400
1600
1800
Moment (kN.m)
Cover depends on fitment size
60 mm
y
x
x
M
Depth = d
60 mm
axis distance
Width = b
SQUARE COLUMNS
f 'c = 40 MPa
600 column 1%
600 column 2%
600 column 3%
600 column 4%
Minimum moment

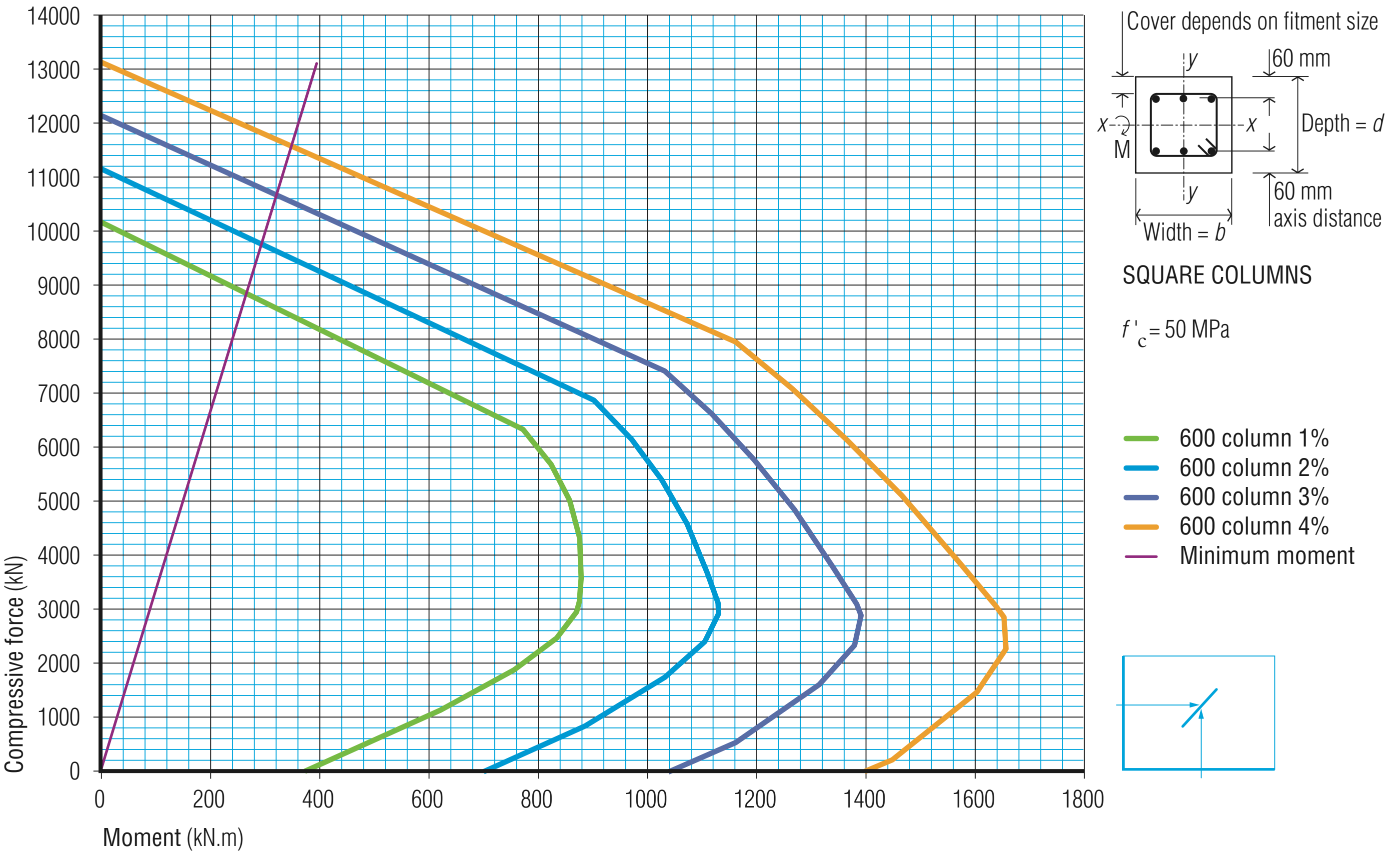
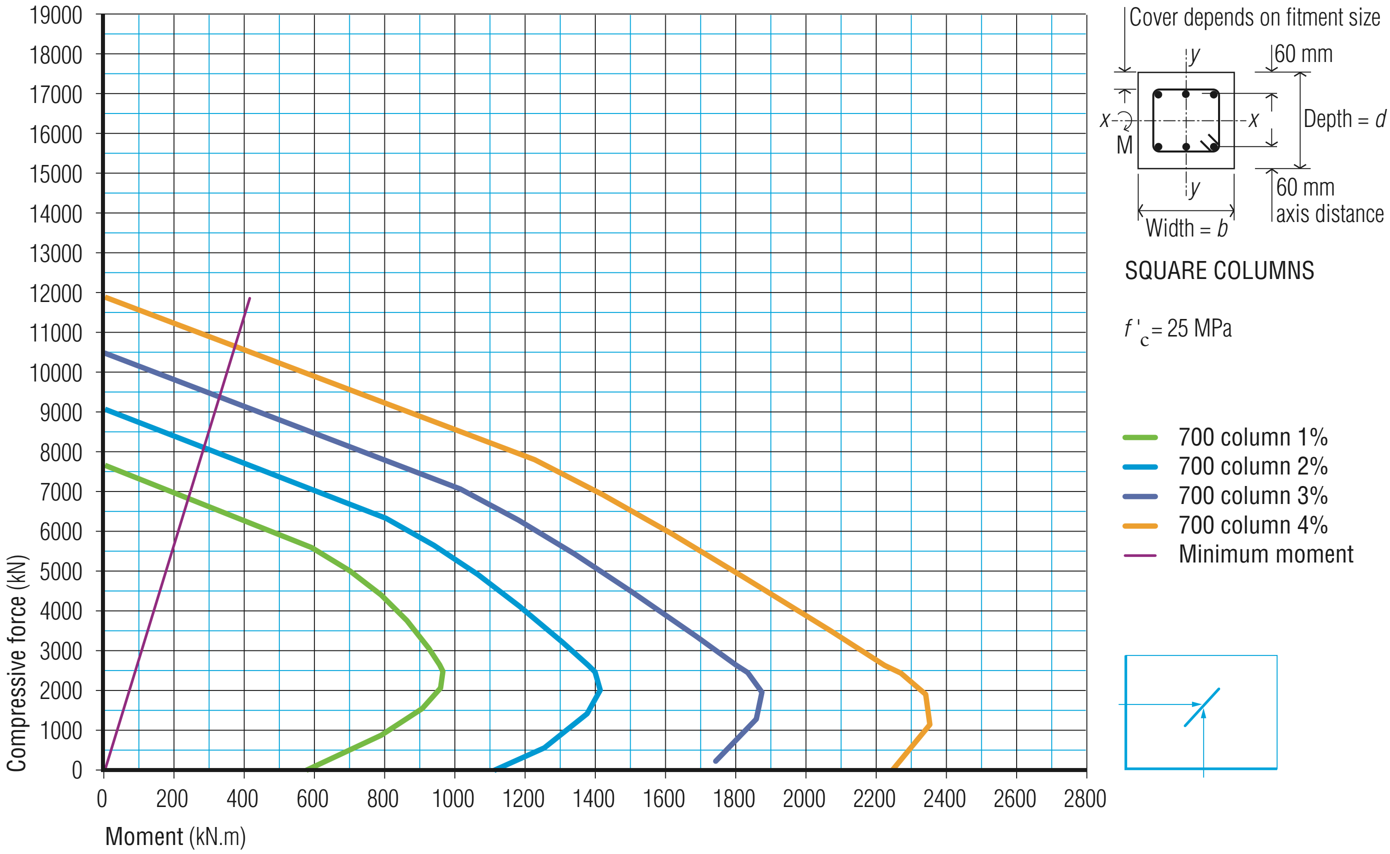

14000
13000
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Compressive force (kN)
0
200
400
600
800
1000
1200
1400
1600
1800
Moment (kN.m)
Cover depends on fitment size
60 mm
y
x
x
M
Depth = d
60 mm
axis distance
Width = b
SQUARE COLUMNS
f 'c = 50 MPa
600 column 1%
600 column 2%
600 column 3%
600 column 4%
Minimum moment

CHART 6.5 700 x 700 Columns



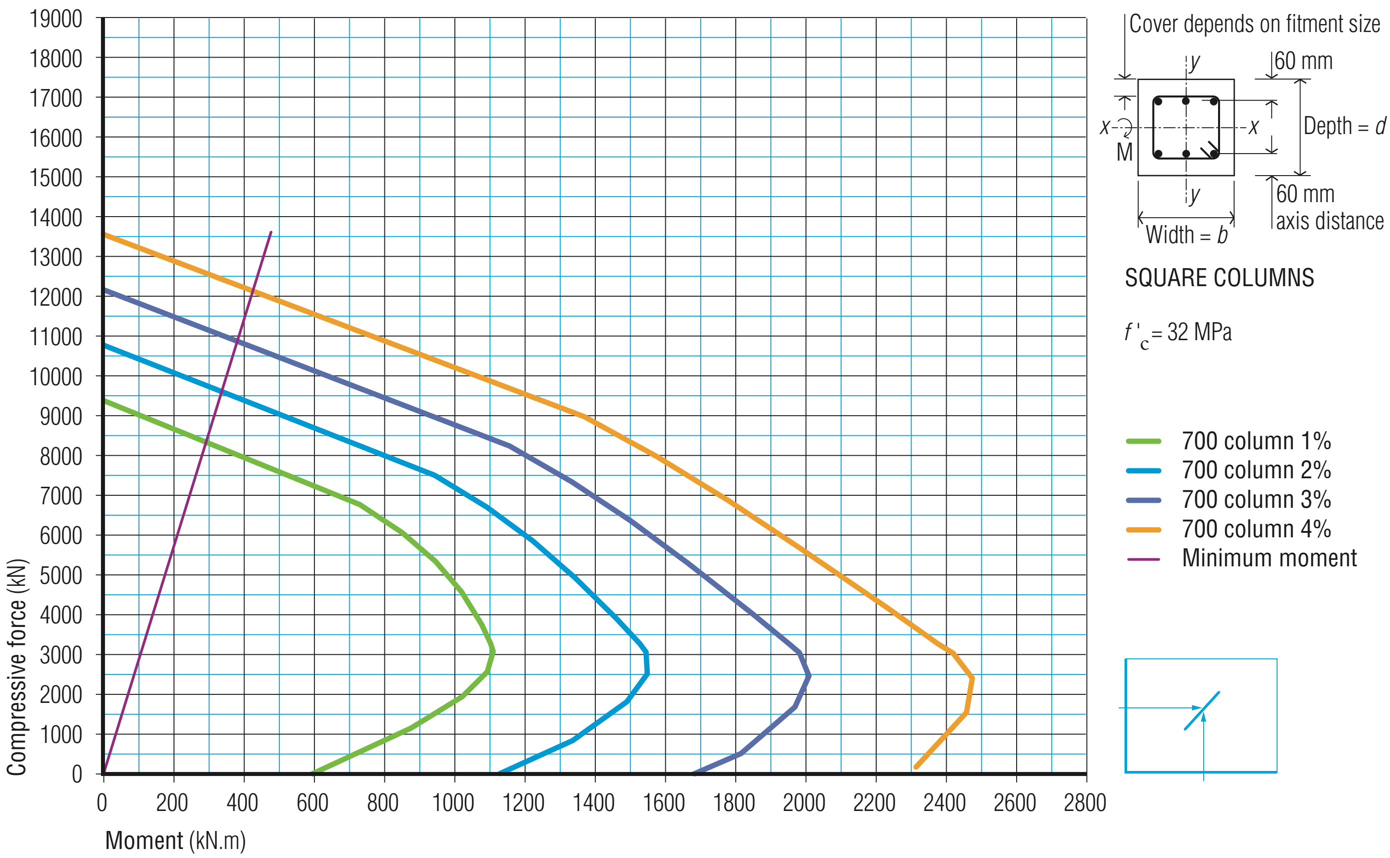
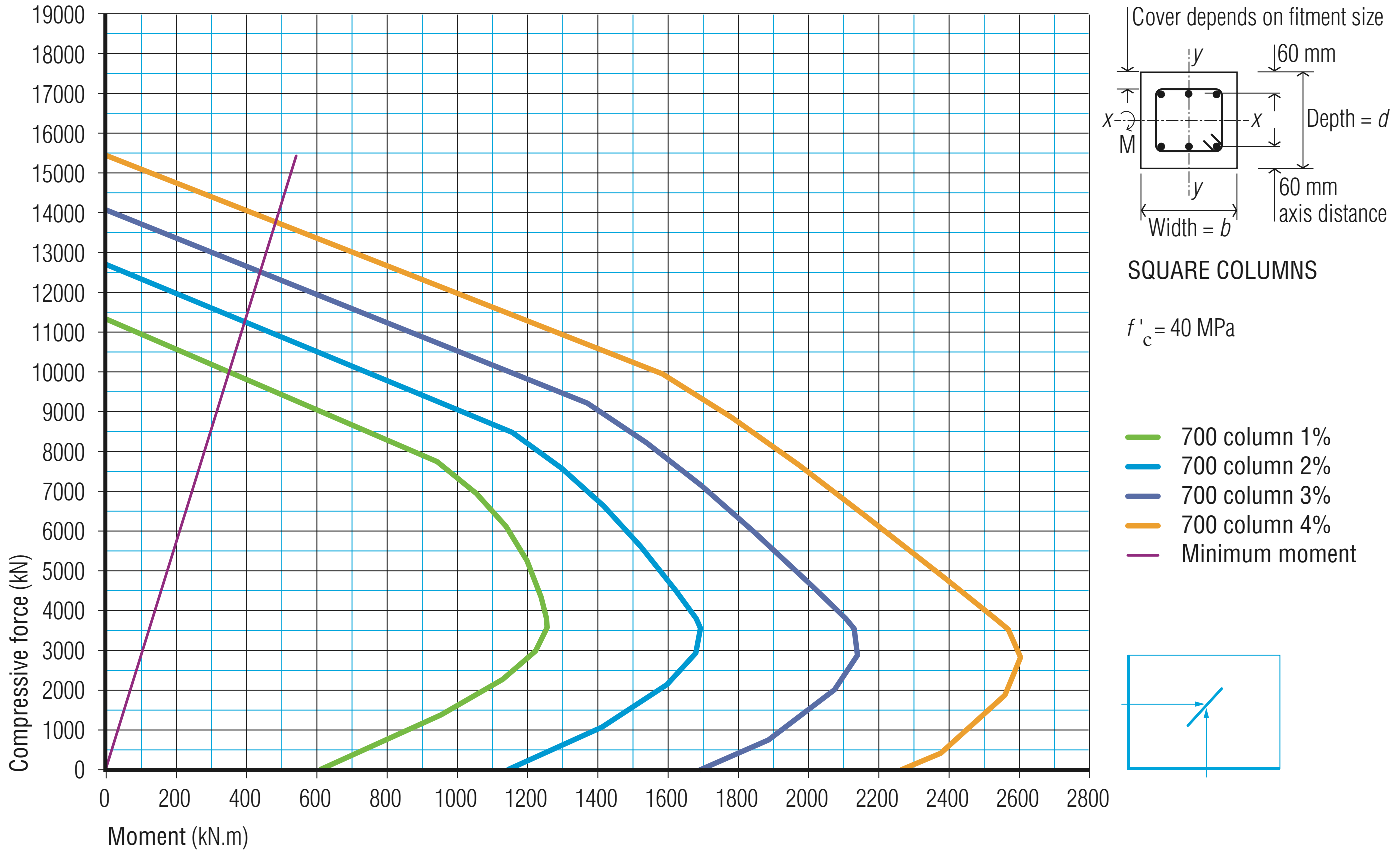


CHART 6.5 *(continued)* 700 x 700 Columns



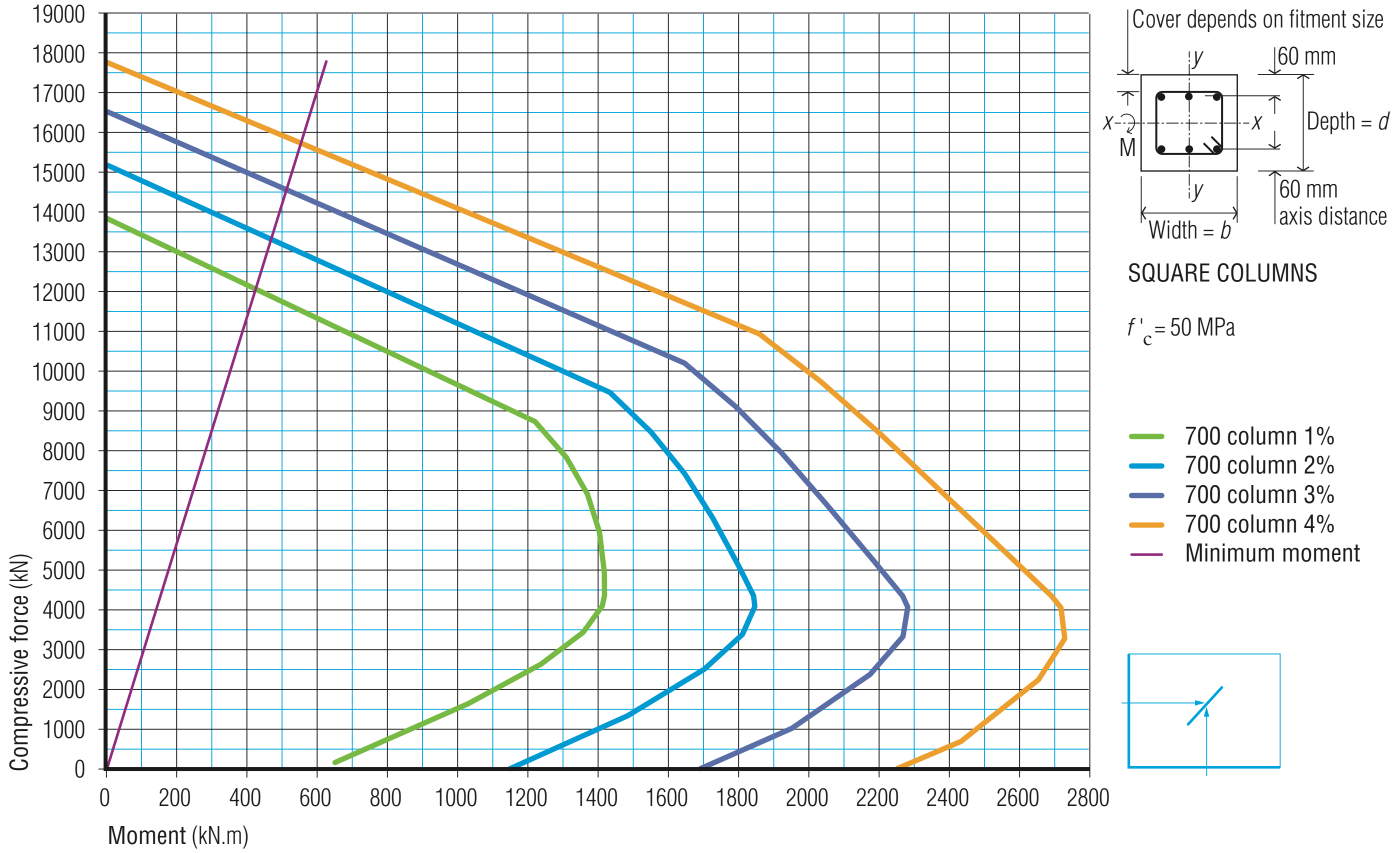
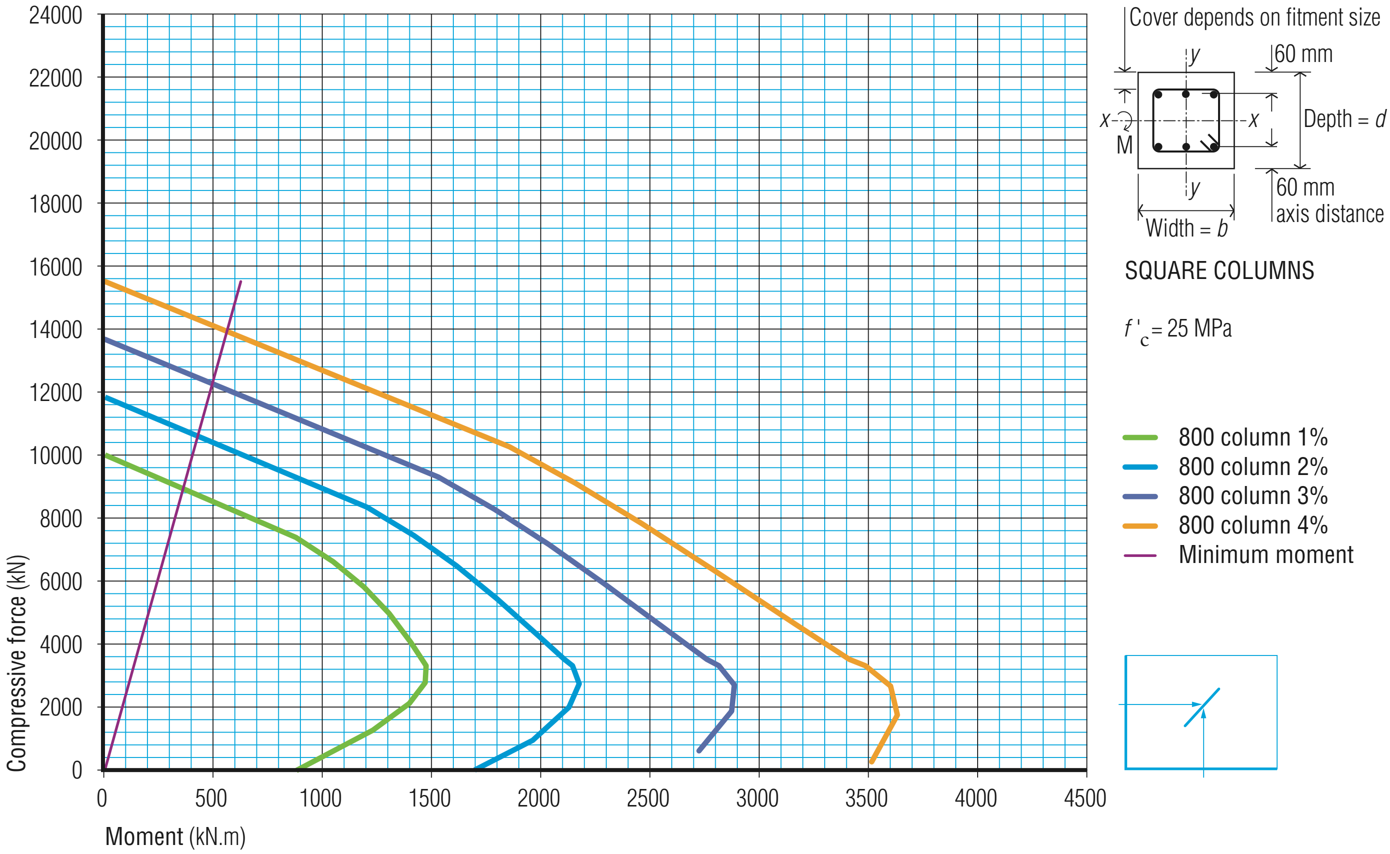


CHART 6.6 800 x 800 Columns



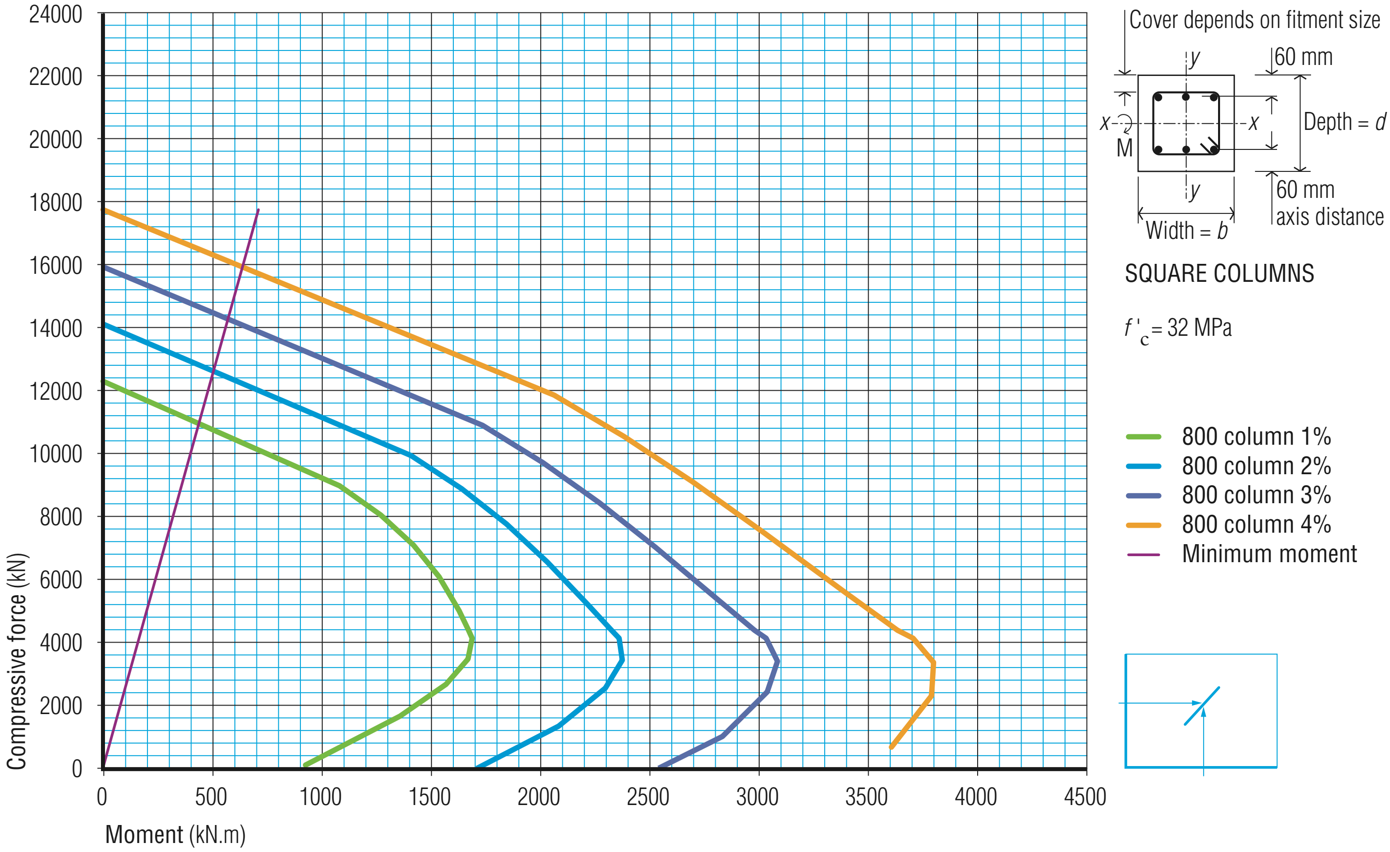
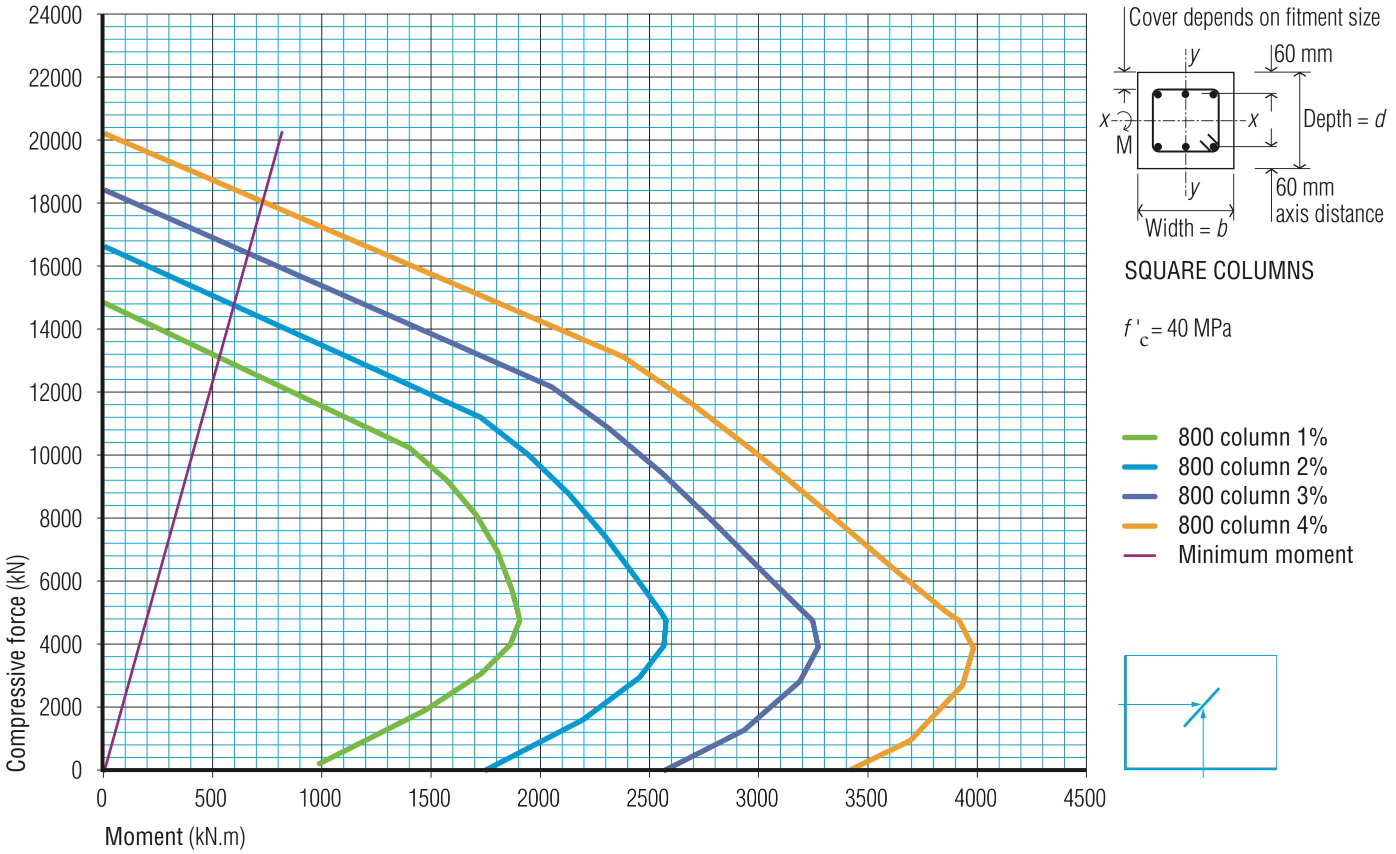
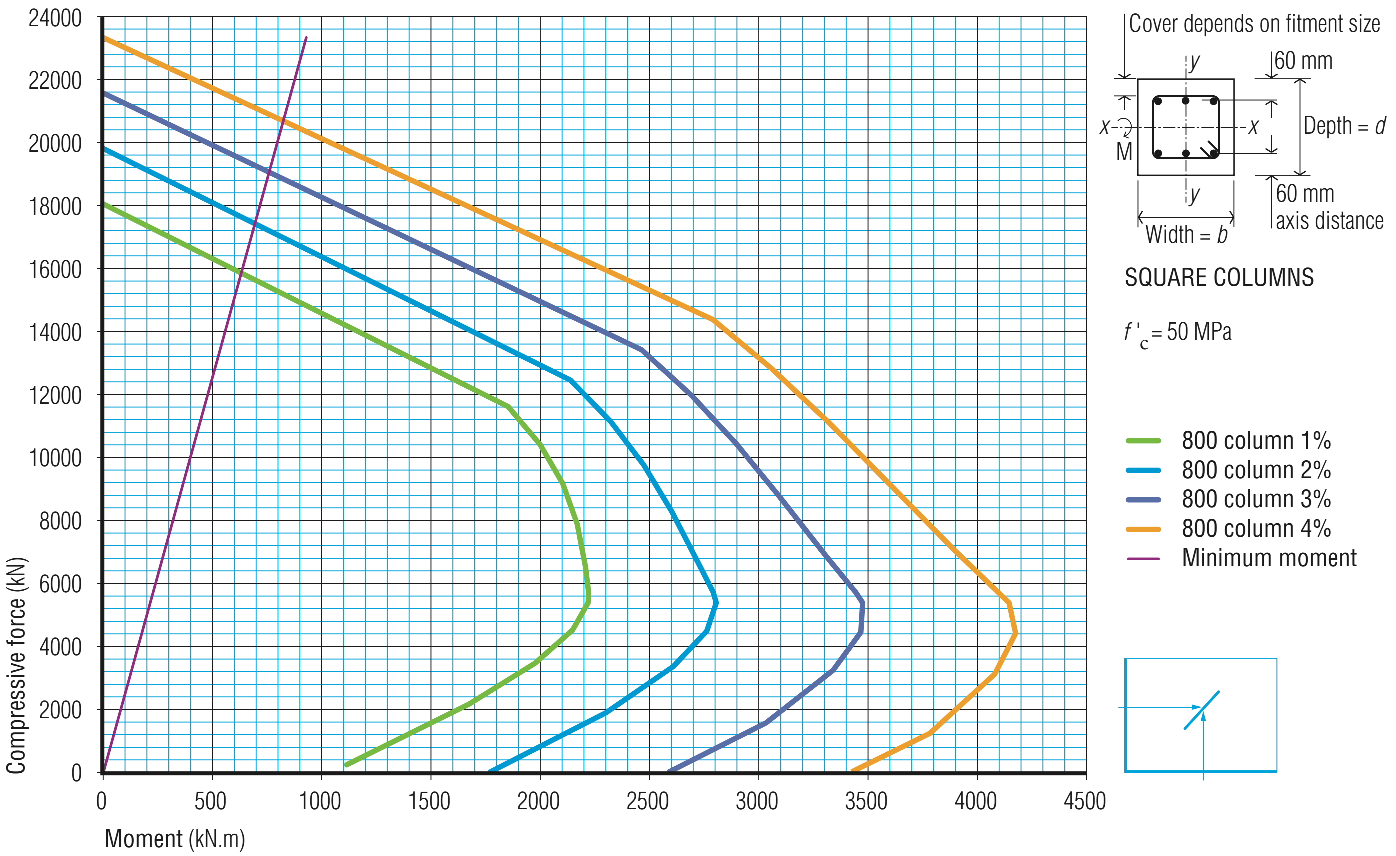


CHART 6.6 *(continued)* 800 x 800 Columns





REFERENCES

6.1 *Precast Concrete Handbook* 2nd Ed, National Precast Concrete Association Australia and Concrete Institute of Australia, 2009.

6.2 AS 3600 *Concrete structures* Standards Australia, 2009.

6.3 Warner RF, Rangan BV, Hall AS and Faulkes KA *Concrete structures*, Longman, 1998.

6.4 Warner RF, Foster SJ and Kilpatrick AE *Reinforced Concrete Basics* 2nd Ed, Pearson, 2010.

6.5 CEB, Commission IVc *Deformability of Concrete Structures – Basic Assumptions* Bulletin D'Information No. 90, 1973.

6.6 Bridge R and Wheeler A 'Advanced Design of Columns' *One Steel* September 2001.

6.7 *Guide to Reinforced Concrete Design* One Steel Design Booklet RCB-3.1(1) *Cross-section Strength of Columns – Part 1* AS 3600 Design, August 2000.

6.8 Foster SJ *Design and Detailing of High Strength Concrete Columns,* University of NSW, 1999.

6.9 Mendis P *Design of High-Strength Concrete Members,* Engineers Australia, 2001.

6.10 National Seminar Series on AS 3600—2009, Engineers Australia, Lecture 3, 2009.

6.11 *Reinforcement Detailing Handbook* (Z06) 2nd Ed, Concrete Institute of Australia, 2010.

Chapter 7_Walls

7.1 WALL TYPES

Concrete walls can be categorised in terms of their method of construction, ie:

- Insitu
- Tilt-up
- Precast.

Insitu concrete walls require extensive formwork and take longer to build than tilt-up or precast. In recent years there has therefore been a considerable shift to the use of tilt-up or precast concrete walls where possible. For further information, designers should refer to *Guide to Tilt-up Design and Construction*[7.1] for tilt-up walls and to Chapter 2 of the *Precast Concrete Handbook*[7.2] for precast walls.

Tilt-up or precast walls may need to be temporally braced in accordance with AS 3850 *Tilt-up concrete construction*[7.3], unless they can be fixed in place top and bottom at the time of erection. There are also statutory requirements in some States and Territories and there is a *National Code of Practice – for Precast, Tilt-up and Concrete Elements in Building Construction*[7.4] for the design and erection of tilt-up and precast concrete walls. This document requires the design for erection of tilt-up and precast concrete walls to be carried out by an Erection Design Engineer.

Walls can also be categorised in terms of their form and function within a building, eg:

- Basement retaining walls including cantilever retaining walls
- Core walls (to lift shafts, service ducts and stairs, etc)
- External and internal walls to single-storey and low-rise buildings
- Cladding wall panels to the facades of multi-storey buildings
- Internal walls in multi-storey buildings
- Walls for housing
- Other, eg curved walls.

Comments on these various types are given below.

7.2 GENERAL

Walls are thin vertical elements, commonly defined as where the length is four times or greater than their thickness. This definition is implied in AS 3600[7.5] Clause 5.6.2(b) relating to fire resistance but is not mentioned in AS 3600 Section 11. Depending on their function, walls can act like a column supporting vertical loads, a slab resisting horizontal loads, a cantilever beam in flexure (ie a shear wall), a deep beam, or sometimes a combination of these.

The design of walls is set out in AS 3600 Section 11 and is in line with the rules adopted for columns in Section 10. Generally, their design, when carrying large vertical loads does not differ significantly from the design of columns, in that axial loads and moments about each axis need to be assessed and allowed for. Walls resisting lateral loads perpendicular to their face are usually designed as slabs.

However, AS 3600 Section 11 gives design rules for only a limited range of walls. Where walls are outside this range, the designer can adopt any appropriate rational design method as permitted under the Building Code of Australia (BCA)[7.6]. Flowchart 7.1 can be used to determine whether or not Section 11 is applicable.

AS 3600 Section 11 applies to the following walls:

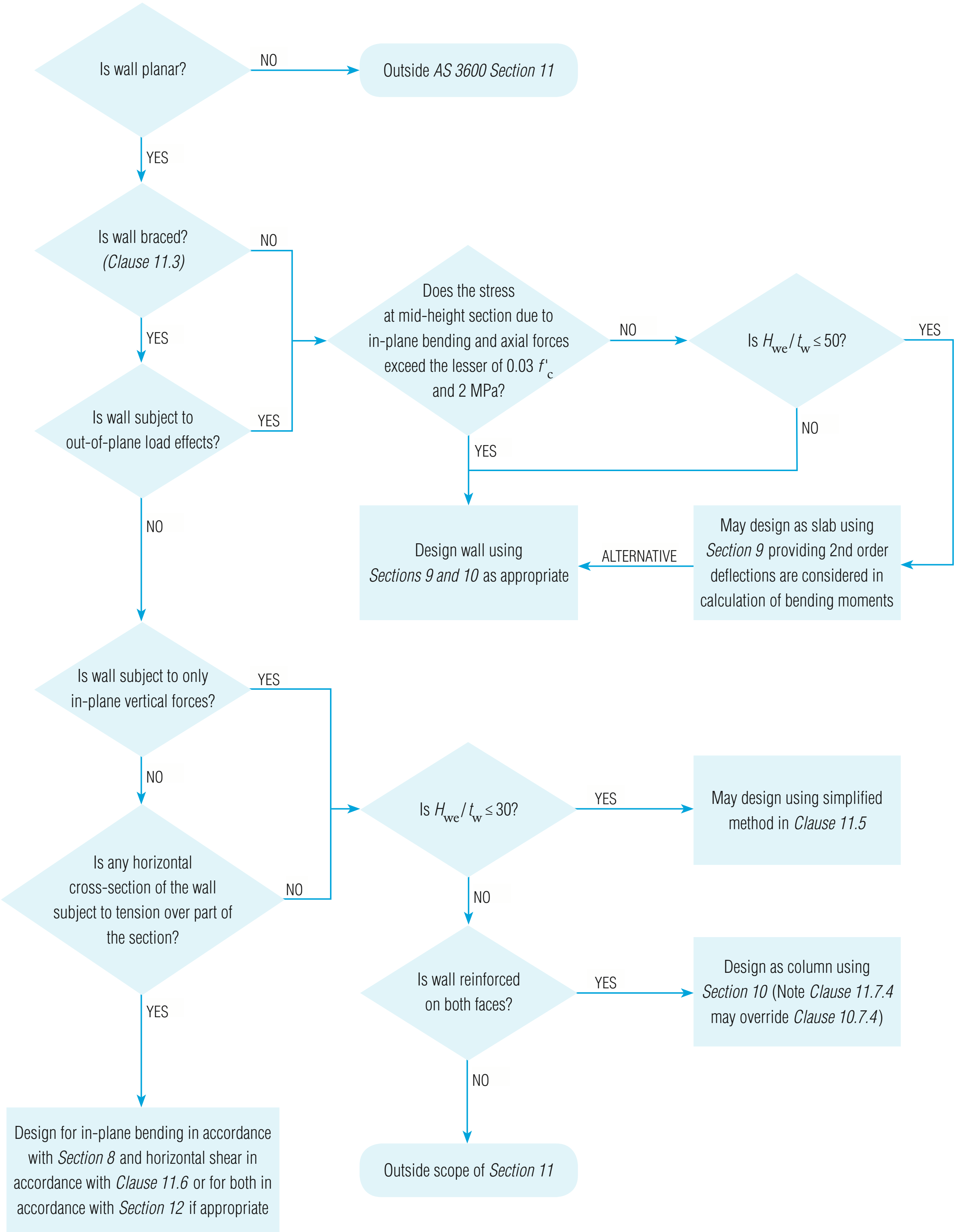
(a) Braced walls (as defined in AS 3600 Clause 11.3) that are subject to in-plane load effects which have to be designed in accordance with AS 3600 Clauses 11.2 to 11.7.

(b) Braced walls that are subject to simultaneous in-plane and out-of-plane load effects and unbraced walls which have to be designed in accordance with Section 9 for slabs and Section 10 for columns as appropriate.

Except that where the stress at the mid-height section of a wall due to factored in-plane bending

Insitu and precast concrete walls

FLOWCHART 7.1 Design of walls *AS 3600 Section 11*



and axial forces does not exceed the lesser of $0.03 f'_c$ and 2 MPa, the wall may be designed as a slab in accordance with Section 9, provided:

(i) second-order deflections due to in-plane loads and long-term effects are considered in the calculation of bending moments; and

(ii) the ratio of effective height to thickness does not exceed 50.

Designers should be aware that the ratio of effective height to thickness may be restricted by design for fire as set out in AS 3600 Section 5.

For off-form finishes to walls, designers should refer to the *Guide to Off-form Concrete Finishes*[7.7].

Designers are reminded that AS 1170.4[7.8] Clause 5.2.3 requires that stiff components of a building such as concrete walls need to be considered as part of the seismic-force-resisting system and designed accordingly, or separated from all structural elements such that no interaction takes place as the structure undergoes deflections due to the earthquake effects determined in accordance with AS 1170.4.

The use of high-strength concrete in walls is not mentioned in Section 11 except indirectly in Clause 11.7.4 relating to the restraint of vertical reinforcement. Designers should refer to AS 3600 Section 10 and texts such as *Design of High-Strength Concrete Members*[7.9] for further information where such concrete is used.

Concrete walls behave in different ways depending on whether they are subject to in-plane or out-of-plane bending. They can be subject to a combination of actions including:

- vertical loads when the walls are commonly known as loadbearing walls (walls supporting their own weight only such as facade cladding walls are known as non-loadbearing walls);
- loads perpendicular to the face of the wall (out-of-plane bending), principally wind and earthquake loads and sometimes other lateral loads such as soil, surcharge and hydrostatic loads;
- horizontal loads in the plane of the wall (in-plane bending and shear) when the walls are commonly known as shear walls resisting the lateral loads, although seldom is shear the critical design case.

Shear walls that act as vertical cantilever elements, are able to resist large lateral loads such as wind and seismic forces in buildings and usually also support vertical loads. They are an efficient way to resist horizontal loads and generally provide lateral strength much more economically than a framed structure using flexure in columns and beams or slabs. Shear walls have three basic failure modes:

- Sliding (shear)
- Overturning
- Bending.

The possibility of any of these occurring can be minimised by increasing the vertical load on the wall. For low-rise walls this may be done by increasing the dead load, eg by increasing the thickness of the wall.

AS 3600 Appendix C covers specific detailing requirements for earthquake loads for ductile shear walls, including boundary elements.

AS 3600 Clause 11.2.1(b) requires that when the horizontal forces in the plane of the wall as shown in Figure 7.1 cause tension over part of the section, then such walls be designed as beams in flexure in accordance AS 3600 Section 8 and that for in-plane shear they be designed in accordance with Clause 11.6. Alternatively, they can be designed for in-plane bending and shear in accordance with AS 3600 Section 12, if appropriate.

Concrete walls have good strength, fire resistance, acoustic and thermal properties. Typically, two- to four-hour fire resistance levels can be met by walls from 150–220 and 230–350 mm thick with 25- and 55-mm axis distances respectively. The varying thickness of wall depends on the minimum thickness for insulation. More particularly it depends on the 'structural adequacy', whether the wall is exposed on one or two sides and the ratio of N_f^*/N_u as set out in AS 3600 Table 5.7.2.

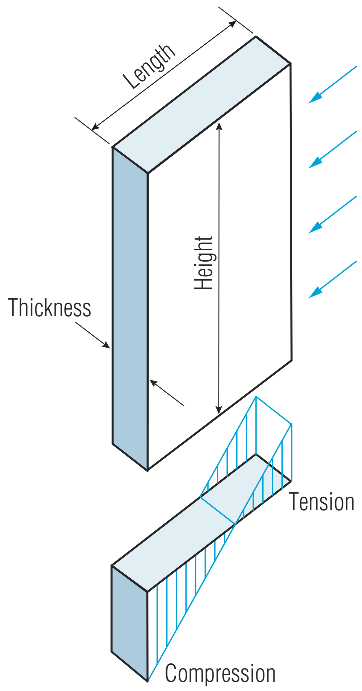


Figure 7.1 Loads on a shear wall

It should be understood that two-sided exposure refers to walls within a single fire compartment which can be surrounded by fire. A wall forming part of the boundary of a fire compartment is deemed to be exposed only from one direction at a time. Exposure from the other direction implies that the fire has breached the compartment boundary and the wall has failed. However, if a wall projects beyond the facade or compartment boundary and is exposed on three or more faces then it is logical to assume that the wall is exposed on two opposite sides.

However, all concrete walls have to comply with the requirements for durability and fire resistance as set out in AS 3600. Nevertheless, there are a large number of walls for which the BCA does not specify a Fire Resistance Level (FRL), eg internal walls in Type C construction.

7.3 BASEMENT AND RETAINING WALLS

Concrete basement retaining walls (generally used to retain soil at the base of a building) can be either cantilever walls or, more often, walls spanning vertically between supports such as the basement floor and lower or ground floor. They sometimes span horizontally between columns, walls, abutments or similar. They can be constructed either of insitu or precast concrete.

For embedded retaining walls, piling systems or diaphragm walls constructed from the top of the ground can be used to form a retaining wall and then the soil from the basement excavated from within an enclosed area. Designers can refer to the ICE, *Specification for Piling and Embedded Retaining Walls*[7.10] for further information.

Retaining walls resist lateral actions due to soil, water, surcharge loads, etc. When they resist soil loads, their design should be in accordance with AS 4678[7.11] and should be based on design lateral pressures provided by a geotechnical investigation with appropriate geotechnical advice They may also have to prevent the ingress of water, which will require consideration of hydrostatic pressures, tanking, drainage, sealing of joints, the risk of damage from salts in the ground, etc.

The main reinforcement in cantilever walls retaining soil will be located in the tension face. For walls thicker than 175 mm, a layer of mesh or grid of small bars is normally provided in the other face for crack control.

Retaining walls spanning horizontally are usually designed as pinned each end. Retaining walls spanning vertically can be designed as 'pinned' or 'fixed' at both the top and bottom, giving a number of design options. Where spans are not too great, it is common to design them as pinned top and bottom. These walls can also be loadbearing, so there may be a number of different design criteria.

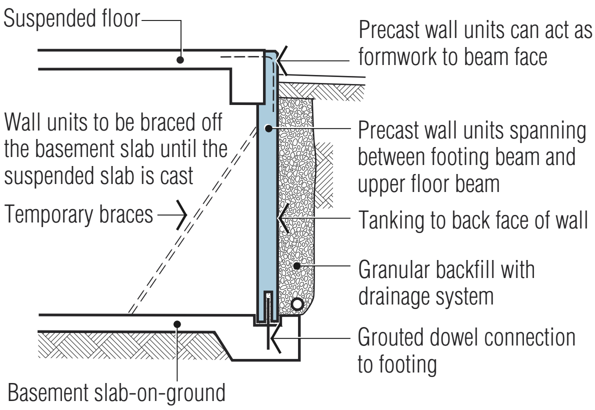


Figure 7.2 Precast retaining walls (after the *Precast Concrete Handbook*)

Insitu concrete walls should be constructed with vertical construction joints at 4- to 6-m spacing along the wall. Constructing longer sections of walls may result in thermal cracking/shrinkage cracking at the bottom of the wall with cracks tapering up the wall, due to restraint by the footing or slab under.

Retaining walls that span vertically between restraints are typically supported on a strip footing, slab edge, footing beam or similar. Restraint fixing to the top of the wall is usually by cast-in bars to the slab, or similar, as shown in Figure 7.2 for a precast wall. In cantilever retaining walls and those retaining walls spanning vertically, large horizontal forces (reactions) will usually occur at the bottom of the walls, which the horizontal joint may be unable to resist. A shear key or rebate should be provided to give adequate restraint at the base of retaining walls subject to large lateral forces.

7.4 CORE WALLS (to lift shafts, service ducts and stairs)

In multi-storey buildings the core walls are usually loadbearing walls and shear walls and are usually insitu concrete because of the need to transmit tension and compression forces down the building and then to the footings. For low-rise buildings, precast walls are sometimes used. Precast walls, however, usually do not have the same structural capacity as insitu walls even when expensive and time-consuming site connections are used.

Core walls generally are loadbearing and usually function as shear walls and can be coupled to form a group of walls in accordance with AS 3600 Clause 11.2.2. They are usually designed in accordance with AS 3600 Clause 11.2. When there are number of shear walls, it is common to assume that the concrete floor or roof acts as a rigid element or diaphragm for loads in the plane of the floor, and that the lateral loads on the building are distributed to each shear wall in proportion to its rigidity, based on gross section properties.

Insitu and precast concrete walls *A series of insitu concrete core walls including both stair and lift shafts, constructed using climbing formwork before the floors below are cast.*

Precast wall panels *The erection of precast concrete core walls for a lift shaft and stair core. Note the bracing of the panels with a minimum of two braces per panel and the precast panels would sit over projecting starter bars which fit into grout ducts in the precast walls, and which are later grouted.*

When resisting the lateral loads on the building as individual walls, they are often considered as a cantilever beam from the base of the building, which adds flexural stresses to the calculated vertical stresses. This may result in tension in the wall, in which case the wall is designed as a beam in accordance with AS 3600 Section 8. In addition, in-plane shear forces will need to be checked in accordance with Clause 11.6.

When two or more walls, or a wall with a number of large openings, are joined together by a coupling or header beam or similar, the effect of combining walls or sections of a wall is to increase the stiffness of the combined wall system – as a result of transfer of shear and flexure through the beam (commonly known as a tie or header beam). This tie beam must be designed for these actions and the shear at the interconnected vertical edges of the walls must be checked. These coupling beams are often heavily reinforced, as their depth is usually limited for architectural reasons. The wall curvatures are also altered from that of a cantilever wall because of the frame action developed, and the combined walls and coupling beams will require detailed analysis. These matters are discussed in more detail in Chapter 5 of the *Precast Concrete Handbook*[7.2].

7.5 EXTERNAL WALLS TO SINGLE-STOREY AND OTHER LOW-RISE BUILDINGS

Precast or tilt-up concrete panels are commonly used for external walls of single-storey and low-rise industrial and commercial buildings. They can be either cladding panels or, more commonly, loadbearing panels combined to form a braced box where the roof or floor acts as a diaphragm as shown in Figures 7.3 and 7.4 respectively. They are generally designed as braced walls carrying the vertical and in-plane horizontal loads in accordance with Clause 11.1(b) and as slabs carrying the out-of-plane forces in accordance with AS 3600 Section 9.

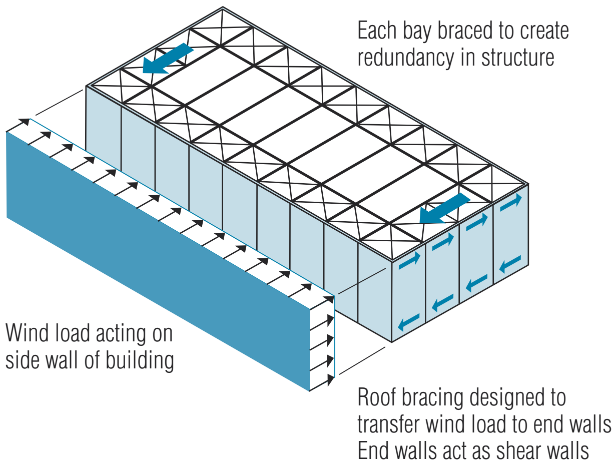


Figure 7.3 Single-storey concrete panel building

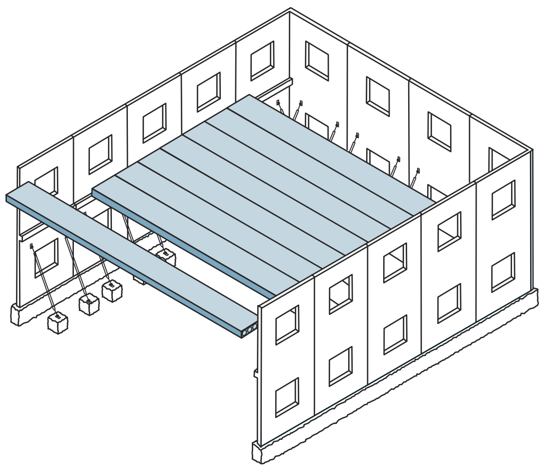


Figure 7.4 A two-storey precast concrete panel building (after Hughes[7.12])

A discussion on the design considerations for a single-storey building can be found in Briefing 08[7.13]. It notes that the roof sheeting and purlins should not be used as part of the bracing system and redundancy in the bracing system is strongly recommended.

In general, the effective-height-to-thickness ratio for walls is limited to ≤ 40 (see AS 3600 Clause 5.7.3). However, this restriction is waived where the top of the wall is supported by a member not required to have a FRL, eg concrete panel walls used as cladding for steel-portal-framed buildings or walls for a concrete panel building with a steel-framed roof. For preliminary sizing, an effective-height-to-thickness ratio of up to 50 may be used.

A minimum practical thickness of external walls is about 125 mm but, more typically, 150 mm or greater thicknesses are used. For walls up to 170–200 mm one layer of reinforcement placed centrally is generally used; above that thickness, two layers are used, one layer in each face. Designers need to exercise caution when considering cantilevering heavy items such as large awnings off concrete walls that have only a central layer of reinforcement because of their limited bending capacity.

Single-storey loadbearing and cladding walls are usually supported on a strip footing, a slab edge, beam or similar. Tilt-up or precast walls have to be restrained at the base at the time of erection and usually temporally braced until connected to the structure. It is also important not to provide a moment connection between the steel roof beams and concrete walls for buildings because of the wall's limited bending capacity about its weak axis.

Non-loadbearing walls are subject to lateral loads usually perpendicular to the face of the wall, principally wind and earthquake loads and designers should refer to the *Precast Concrete Handbook* (Chapters 2 and 7) for further information on the design of such precast panels.

In addition to lateral loads perpendicular to the face, loadbearing panels need to be designed for vertical and lateral loads in the plane of the wall as applicable.

The key design issues for precast concrete facades are appearance, the correct and adequate design and inspection, durability, waterproofing, building movements, and buildability. Coverage of this topic can be found in *Woodside*[7.14].

Precast cladding wall panels

7.6 PRECAST EXTERNAL WALL PANELS IN MULTI-STOREY BUILDINGS

Precast external wall panels in multi-storey buildings may be either non-loadbearing, ie serve only as claddings or, more commonly, loadbearing, ie serve as cladding and have a structural role or roles. The design, detailing and layout of panels will involve considerable interaction between the structural engineer, architect and precaster.

Non-loadbearing panels are typically supported on a floor-by-floor basis using corbels and restraint brackets. A suitable horizontal joint (15–20 mm) must be provided at each floor level to accommodate both manufacturing and construction tolerances and for axial shortening of the building if it is a concrete frame. The detailing of this joint is very important to allow for these tolerances and movement.

7.7 INTERNAL WALLS IN MULTI-STOREY BUILDINGS

In multi-storey buildings, internal concrete walls are typically either precast or insitu. They often have a fire-separation role, carry vertical loads and usually act as shear walls. They are typically supported on a floor-by-floor basis.

These walls are usually designed for vertical loads in accordance with AS 3600 Clause 11.2.1, using either Clause 11.5, the simplified design method, or Section 10 *Column Design*. However, if they are acting as shear walls they may need to be checked for bending in accordance with AS 3600 Section 8 if the wall is in tension and designed for in-plane shear forces in accordance with AS 3600 Clause 11.6.

Internal precast walls The internal precast walls shown above include a column at one end of each wall panel. They are loadbearing walls supporting precast flooring. They also act as shear walls carrying in-plane horizontal loads from the floors over into the floor below and have vertical starter bars from the floor into dowel ducts in the wall which are subsequently grouted. The columns sections are used to support the precast concrete beams on either side of the walls which in turn support precast floor units. Note how the precast walls are braced with two diagonal braces per panel until the floor over is cast.

Precast walls for a garage and house extension

7.8 WALLS FOR HOUSES

Concrete is increasingly being used for walls in both individual and modular housing. Designers should refer to *The Concrete Panel Homes Handbook*[7.15] for further information.

These walls are usually tilt-up or precast walls and one- or two-storey in height. In traditional houses, the structural engineer would typically design only the concrete footings, concrete slab-on-ground, suspended concrete floors, some wall bracing for lateral loads and long-span beams as required. However, when tilt-up or precast concrete walls are used, full design for all the structure by the structural engineer will be required to ensure adequate restraint of the concrete walls both in the temporary and final conditions.

Tilt-up or precast walls are typically supported on a concrete floor and/or edge beams at ground level, while the concrete slab-on-ground will often provide a suitable element to temporarily brace the tilt-up or precast panels during erection. Designers should note that most temporary brace fixings used to fix the diagonal braces to concrete slab-on-ground are heavy-duty drilled load controlled, torque-setting expansion anchors which commonly require a minimum slab thickness of 125 mm whereas slabs-on-ground are commonly only 100 mm thick when designed in accordance with AS 2870[7.16].

The main structural design issue is how the concrete walls are laterally restrained by either concrete walls perpendicular to them or by the floor or roof, acting as a diaphragm. Light timber-framed floors and roofs are unlikely to be adequate to act as a diaphragm for the restraint of heavy concrete wall panels under lateral loads such as wind or to provide restraint in the event of fire and/or earthquake.

7.9 WALLS DESIGNED AS COLUMNS

In Section 11 of AS 3600, the floor-to-floor height of the wall is defined as H_w. However, in Section 10 of AS 3600 (used when designing walls as columns), the effective height of a column (wall) is referred to as an effective length, L_e.

In Table 7.1 and in Charts 7.1–7.5, the effective length, L_e, is in fact the effective height of the wall and not the length of the wall in plan.

The charts in this chapter are based on the same assumptions as the charts in Chapter 6. They assume the wall is braced and short, ie $L/r \leq 25$. For a slender wall, a moment magnifier will need to be applied in accordance with Clause 10.4.2 for slender (columns) walls. As can be seen from Table 7.1, walls 200–300 mm thick are likely to be slender.

The charts are for walls reinforced on two faces for f'_c = 25, 32, 40 and 50 MPa with 50-mm axis distance which should provide a FRR of 120 minutes. Because they are relatively thin, such walls cannot resist high bending moments about their smaller dimension.

The design procedure for walls when acting as columns will be:

- Assess the actions on the wall.
- Assume an initial size of wall and concrete strength.
- Assess the durability requirements, cover and FRR to determine the axis distance and the distance of the vertical bars from the wall face.
- Choose a design procedure based on Clause 11.1 of AS 3600 and the Flowchart 7.1.

TABLE 7.1 Slenderness ratios for braced walls when designed as columns

Thickness of braced wall (mm)	Radius of gyration	Maximum height H_w (mm)	Effective length* L_e Factor k = 0.7	Factor k = 0.85	Factor k = 1.0
SHORT WALL $L_e/r \leq 25$					
200	60	1500	2143	1765	1500
300	90	2250	3214	2647	2250
400	120	3000	4286	3529	3000
500	150	3750	5357	4412	3750
SLENDER WALL $L_e/r \leq 120$					
200	60	7200	10 286	8471	7200
300	90	10 800	15 429	12 706	10 800
400	120	14 400	20 571	16 941	14 400
500	150	18 000	25 714	21 176	18 000

* Effective length factor, *k*, from AS 3600 Figure 10.5.3(A)

TABLE 7.2 Reinforcement area, A_{st}, for crack control (mm²/m)

A_{st}	Thickness of wall (mm) 100	125	150	175	200	225	250	275	300	325	350	375	400	425	450	475	500
$1.5\ t_w$	150	188	225	263	300	338	375	413	450	488	525	563	600	638	675	713	750
$2.5\ t_w$	250	313	375	438	500	563	625	688	750	813	875	938	1000	1063	1125	1188	1250
$3.5\ t_w$	350	438	525	613	700	788	875	963	1050	1138	1225	1313	1400	1488	1575	1663	1750
$6.0\ t_w$	600	750	900	1050	1200	1350	1500	1650	1800	1950	2100	2250	2400	2550	2700	2850	3000

- Determine the effective height, the slenderness ratio and determine the design moments.
- For the chosen axial load and moments determine the area of reinforcement required.

(See Chapter 6 for a more-detailed discussion of these design steps.)

Generally, walls are designed for strength only; stability and serviceability are considered only for slender and/or unbraced walls. The action effects in accordance with Table 1.1 of Chapter 1 of this Handbook that should be considered include:

- $1.35G$
- $1.2G + 1.5Q$
- $1.2G + \psi_c Q + W_u$
- $0.9G + W_u$ (not generally a design case – may be applicable for a lightweight roof supported by the wall in a cyclonic area and where large uplift forces may have to be resisted)
- $G + \psi_c Q + E_u$

Considering all the possible axial load and bending moment combinations for each load case for a given cross-section for each wall at each floor using manual determination of the strength of a chosen wall size can be a tedious process, although often the load case $1.2G + 1.5Q$ will be the critical design case. The design of a particular section is a trial-and-error process and is much more easily accomplished with a load-moment interaction diagram calculated for the chosen wall section and reinforcement, using appropriate design software or a spreadsheet that complies with AS 3600.

When using Charts 7.1 to 7.5, note that the wall is assumed to have reinforcement in each face.

Note that for walls designed as columns the vertical reinforcement needs lateral restraint using fitments when:

- $N^* > 0.5\,\phi N_u$, or
- the concrete strength is greater than 50 MPa and:
  - — the vertical reinforcement ratio is used as compression reinforcement, or
  - — the vertical reinforcement ratio is greater than 0.02, and a minimum horizontal reinforcement ratio of 0.0025 is not provided.

For 200-mm-thick walls, it is difficult to place and fix fitments unless the covers are small. 250 mm is the practical minimum thickness for walls with fitments.

7.10 SIMPLIFIED DESIGN METHOD FOR WALLS SUBJECT TO VERTICAL COMPRESSIVE FORCES

The axial load capacity for a unit length of wall is given in AS 3600 Clause 11.5.1, ie:

$$N^* \leq \phi N_u = 0.6\,(t_w - 1.2e - 2e_a)\,0.6\,f'_c$$

where e_a is taken as $= (H_{we})^2/2500\ t_w$.

This requires the effective height to be calculated, a factor k is determined (see AS 3600 Clause 11.4), and the unsupported height H_w is multiplied by k to give the effective height H_{we}.

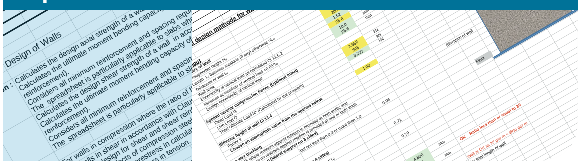
The simplified design approach given in AS 3600 Clause 11.5 is based on BS 8110[7.17]. The equation gives a conservative estimate of the load capacity of the wall and ignores any load capacity due to the vertical reinforcement.

There are also a number of limitations/restrictions on the use of the simplified method. These include the requirements that:

- the wall is assumed to be braced in accordance with AS 3600 Clause 11.3; and
- $H_{we}/t_w \leq 30$.

Spreadsheet 7.1 can be used to calculate the axial load capacity for a wall using the simplified method.

Spreadsheet 7.1 is available at www.ccaa.com.au



7.11 DESIGN OF WALLS FOR IN-PLANE SHEAR FORCES

The design shear strength of a wall subject to in-plane horizontal shear is given by the sum of the concrete shear strength and the strength due to the reinforcement, ie:

$$\phi V_u = \phi V_{uc} + \phi V_{us}$$

with an upper limit of $\phi 0.2\,f'_c\,(0.8L_w\,t_w)$.

V_{uc} and V_{us} are determined from AS 3600 Clause 11.6.3 and Clause 11.6.4 respectively.

The two equations given in Clause 11.6.3 to determine V_{uc} together with the lower limit, which is the third equation, are:

$$\phi V_{uc} = \phi[0.66\sqrt{f'_c} - 0.21\sqrt{f'_c}\,(H_w/L_w)]\ 0.8L_w t_w$$

for $H_w/L_w \leq 1$

or

$$\phi V_{uc} = \phi[0.05\sqrt{f'_c} + 0.1\sqrt{f'_c}/(H_w/L_w - 1)]\ 0.8L_w t_w$$

for $H_w/L_w > 1$ with the lower limit of

$$\phi V_{uc} \geq \phi 0.17\sqrt{f'_c}\,(0.8L_w t_w).$$

The contribution to in-plane shear strength due to reinforcement, ϕV_{us}, is given by the equation in AS 3600 Clause 11.6.4, ie:

$$\phi V_{us} = \phi p_w\,f_{sy}\ 0.8L_w t_w$$

where

p_w is the reinforcement ratio and $= p_h$ (when $H_w/L_w > 1$)

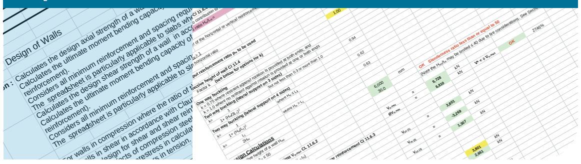
or

p_w = lesser of p_h and p_v (when $H_w/L_w \leq 1$).

The reinforcement ratios p_h and p_v are for horizontal and vertical directions respectively.

Spreadsheet 7.2 based on these equations can be used to calculate the shear capacity for a wall.

Spreadsheet 7.2 is available at www.ccaa.com.au



7.12 REINFORCEMENT REQUIREMENTS FOR WALLS

AS 3600 Clause 11.7 gives requirements for minimum reinforcement in walls and for crack control. These requirements are independent of the ϕ value used.

For minimum vertical reinforcement, the reinforcement ratio p_w shall be the larger of 0.0015 (1.5 t_w mm^2/m) or the value required for strength.

For minimum horizontal reinforcement, the reinforcement ratio p_w shall be not less than 0.0015 (2.5 t_w mm^2/m), except that for a wall designed assuming one-way buckling using AS 3600 Clause 11.4(a) and where there is no restraint against horizontal shrinkage or thermal movements, this may be reduced to zero if the wall is less than 2.5 m wide, or to 0.0015 (1.5 t_w mm^2/m length) otherwise.

For crack control and shrinkage where the wall is restrained from expanding or contracting in the horizontal direction, the minimum area of horizontal reinforcement in that direction shall be:

For exposure classifications A1 and A2

- where a minor degree of control over cracking is required A_{st} must be at least 2.5t_w mm^2/m
- where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view A_{st} must be at least 3.5t_w mm^2/m
- where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes A_{st} must be at least 6.0 t_w mm^2/m.

For exposure classifications B1, B2, C1 and C2, A_{st} in the horizontal direction must be at least 6.0 t_w mm^2/m , which can be a significant amount of reinforcement.

Table 7.2 shows the various areas of horizontal reinforcement per metre of length required for walls up to 500 mm thick for crack control. Note that this reinforcement is the total of all layers.

AS 3600 requires the spacing of the reinforcement to be adequate to place the concrete but not less than $3d_b$.

Reinforcement must be in two layers, one near each face when:

- the wall is greater than 200 mm thick;
- where in any part of the wall structure the tension exceeds the tensile capacity of the concrete under the design ultimate load (this means shear walls may require two layers of reinforcement);
- when walls are designed for 2-way buckling.

The maximum spacing of the reinforcement is the lesser of $2.5t_w$ and 350 mm. For walls greater than 200 mm thick, or where the wall is in tension greater than the tensile capacity of the wall, or where the wall is designed for 2-way buckling in accordance with AS 3600 Clause 11.4, the reinforcement has to be in two layers, one in each face.

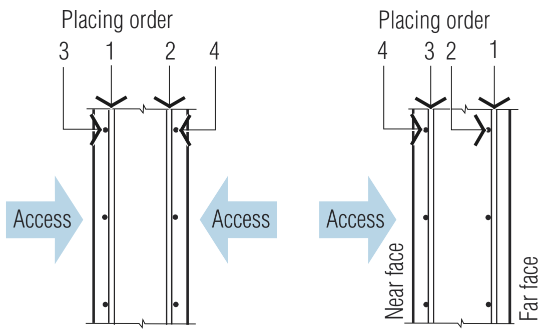


Figure 7.5 Placing order of bars

7.13 DETAILING

All wall elevations and details should be shown on the drawings.

Designers should refer to Chapter 15 of the *Reinforcement Detailing Handbook*[7.18] for further advice on the detailing of walls.

Designers should note that:

- Sufficient information should be provided on the drawings (including plans, elevations, sections and details) to enable the walls to be built.
- Standard details, if used, should be appropriate to the walls being designed.
- The order in which the various layers, ie the vertical and horizontal bars, are to be placed should be specified.
- For ease of construction for insitu concrete walls, it is often desirable to tie the horizontal bars on the outside of the vertical bars or to place the horizontal bars from one side only such as in a core wall where the inside formwork is placed first and the reinforcement then fixed. This should be taken into consideration when designing the wall, especially considering cover and axis distance, see Figure 7.5. It is important to note that when a wall is designed and reinforced as a column with fitments to restrain the vertical bars, the vertical bars cannot be on the outside.
- When header or coupling beams are within an insitu wall, the thickness of the wall must allow for cover/axis distances, heavy reinforcement and allow the concrete to be properly placed.
- The effect of bend radii at corner and T junctions in plan where walls intersect, especially if they have to resist bending moments around the junctions.
- Trimmer bars are needed at openings in walls such as doors and windows to minimise cracking at re-entrant corners. Note that trimmer bars at 45° will form a third layer which may make placing of concrete difficult.
- Minimum reinforcement is to be provided in accordance with AS 3600 Clause 11.7.
- Trimmer bars usually are one N12 per layer of reinforcement, around the perimeter of precast and tilt-up wall panels. Larger diameter trimmer bars may not fit within thin walls depending in which layer they are in.
- Appropriate splice lengths for bars and mesh are specified depending on whether the bars and mesh are in tension or compression.
- For walls the strength of the concrete in the floor slab also needs consideration, as the strength of the floor cannot be less than 0.75 the strength of the wall without specific design as set out in AS 3600 Clause 10.8. This clause allows for the concrete in the walls to be one strength grade higher than that of the slab. For greater differences in strength, additional calculations are required to determine the effective strength of the concrete in the wall for transmission of axial forces through the floor systems
- Thin insitu concrete walls can be difficult to cast. Designers should consider the use of 10-mm aggregate and super plasticisers to allow adequate compaction when thin walls are proposed.
- Avoid the use of fitments if possible as they are difficult to fix.

7.14 GENERAL GUIDANCE

The following will assist the design and inspection of walls for a particular project.

- Stiff walls such as retaining walls, core walls or other walls in a building, can significantly restrain concrete floors and roofs when they are rigidly connected to them. This can result in unsightly diagonal cracking in the concrete floors and roofs (or in the wall) due to shrinkage. Construction techniques to minimise such problems include locating the connecting bars in flat prestressing ducts, providing slip connections and then grouting the ducts after the floor over is cast and some of the shrinkage allowed to occur.
- If the wall is to have an off-form finish and is exposed to view, are there any specific requirements for class of finish, type of formwork, arrangement of form ties for insitu walls, etc.
- For precast walls, are any special finishes required including applied finishes such as painted, acid washed, sandblasted, honed or polished? Designers should refer to Chapter 10 of the *Precast Concrete Handbook* for information on finishes, remembering that not all finishes may be available in the given location.
- If the construction joints will be exposed to view, consider providing a small vee or rebate to the joints to give a neat appearance, and ensure that this is allowed for in the cover specified.
- For precast and tilt-up walls, a positive connection must be provided at the bottom of the wall at the time of erection to prevent kick-out prior to unhooking the panel from the crane.
- When mixing loadbearing and non-loadbearing precast wall panels, differential movements can be a problem.
- Casting insitu columns and walls as a combined unit should be avoided because of the complications of the formwork.
- The rules given in AS 3600 Clause 5.7.4 under *Fire resistance for recesses and chases* are empirical rules adopted to provide consistency with the rules for masonry. They should therefore be viewed with caution.

Insitu off form concrete walls *Note the attention to the joint layout, construction joint locations (which will occur at the expressed joints) and the locations of the bolt holes or the form ties in this insitu wall to form a uniform pattern.*

CHARTS 7.1 TO 7.5

These charts have been developed assuming the wall is braced in both directions and is short. If the wall is a slender braced wall then the appropriate moment magnifiers in accordance with AS 3600 Clause 10.4.2 must be used.

They are based on moments about the weak axis only.

The design of walls as columns should be in accordance with AS 3600 and appropriate design aids or software should be used.

Chart 7.1 200 Thick walls
pages 7.12 and 7.13

Chart 7.2 250 Thick walls
pages 7.14 and 7.15

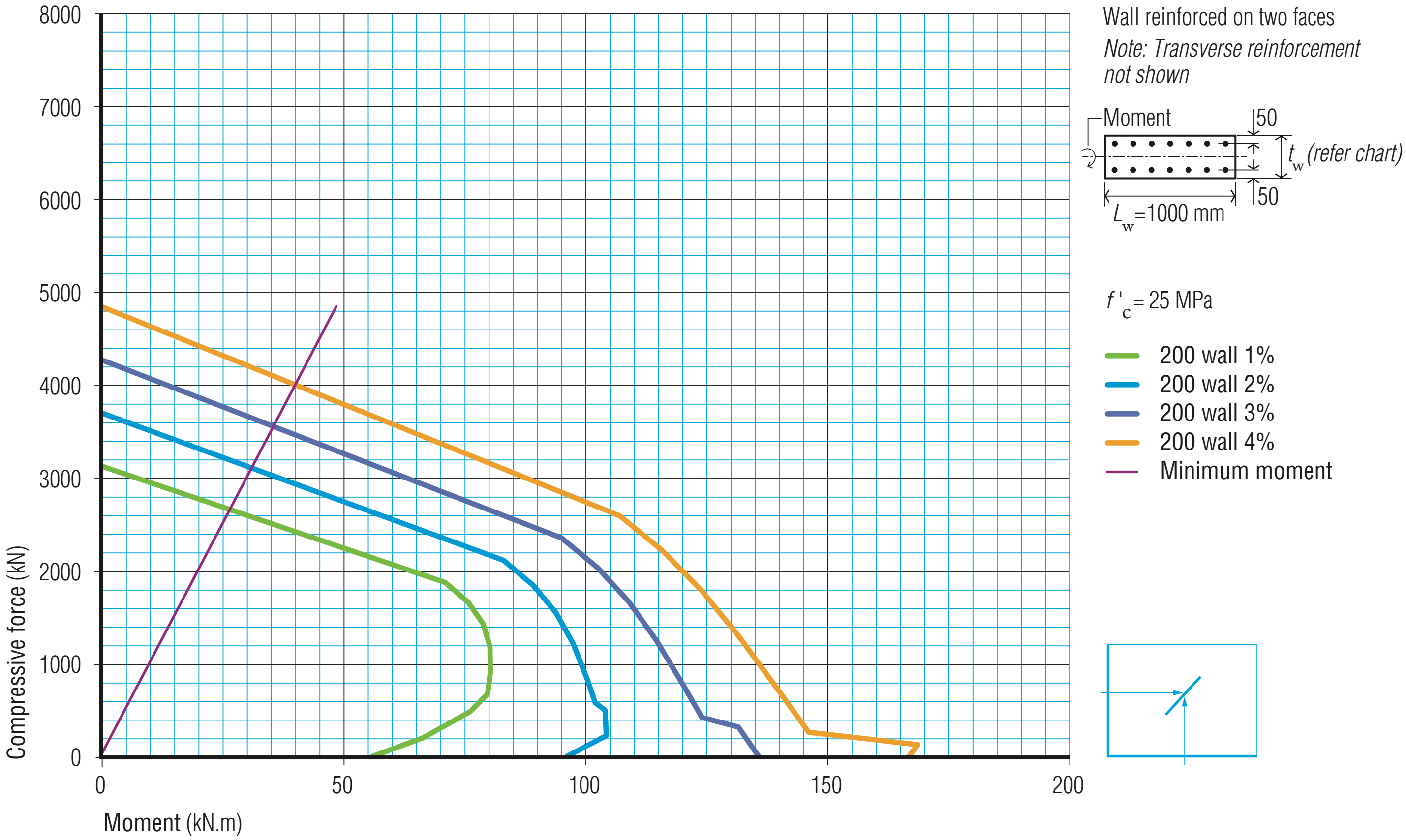
Chart 7.3 300 Thick walls
pages 7.16 and 7.17

Chart 7.4 350 Thick walls
pages 7.18 and 7.19

Chart 7.5 400 Thick walls
pages 7.20 and 7.21

Spreadsheets 7.1, **7.2 and 7.3** may be downloaded from the Cement Concrete & Aggregates Australia website www.ccaa.com.au

CHART 7.1 200 Thick walls *axis distance = 50 mm*



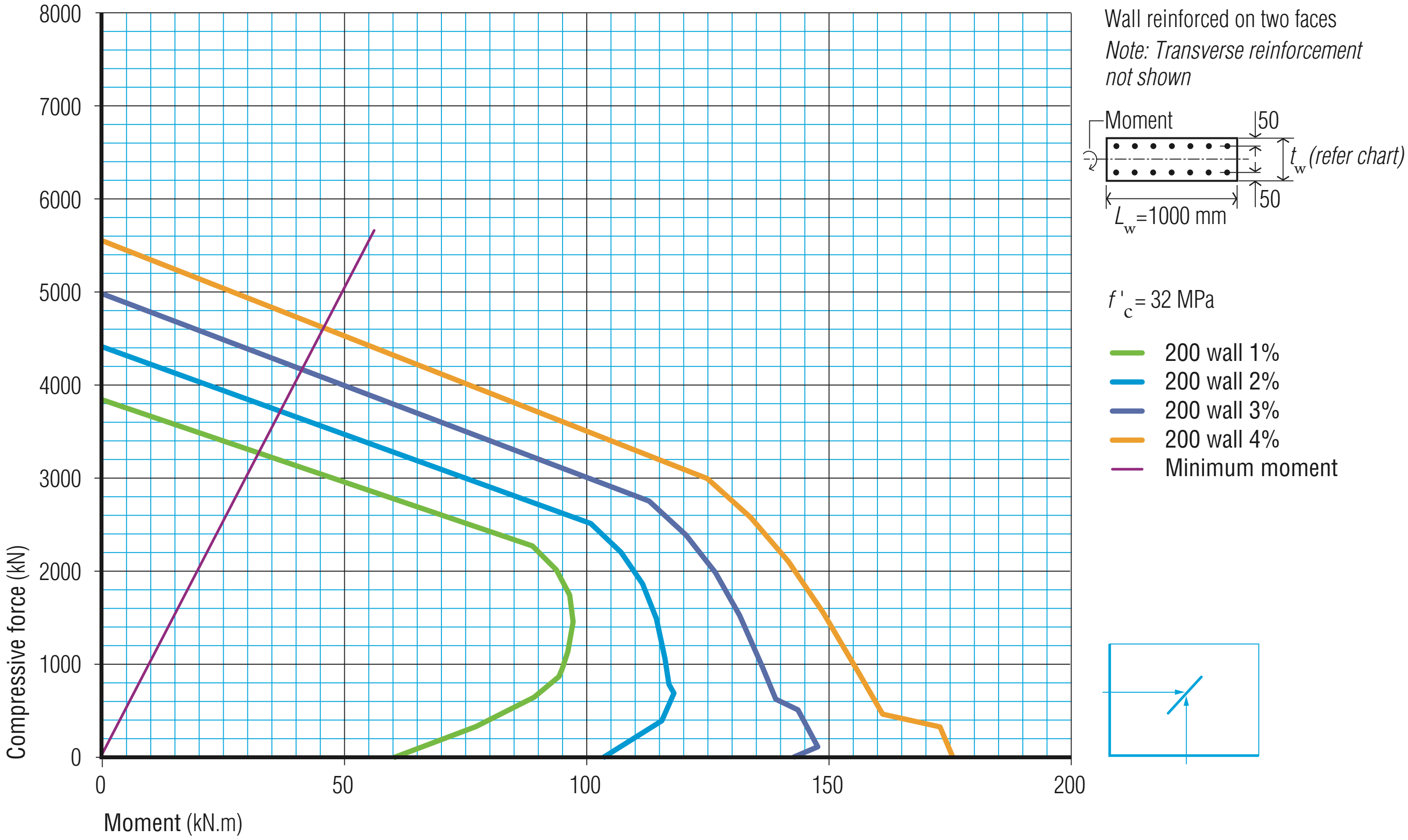
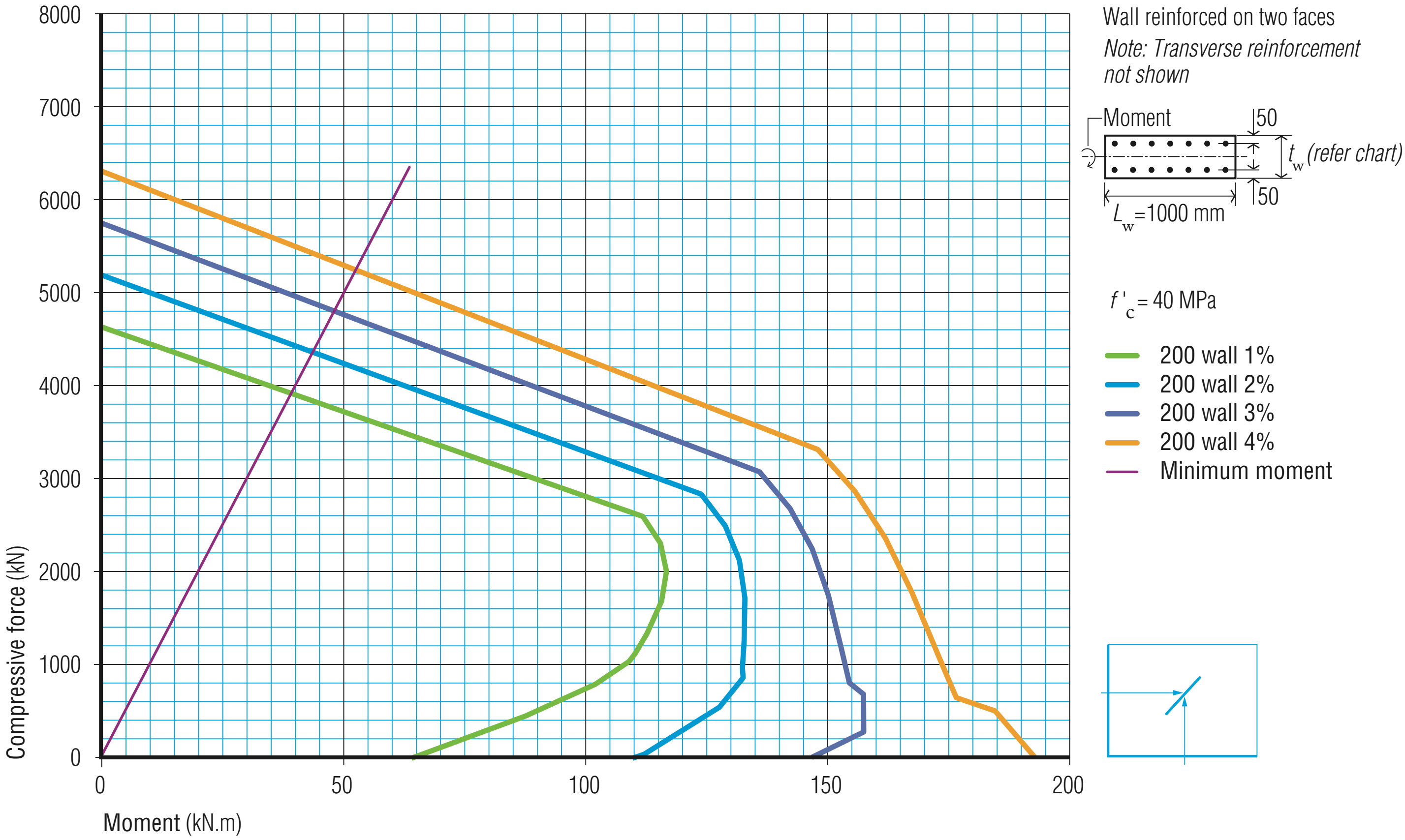


CHART 7.1 *(continued)* 200 Thick walls *axis distance = 50 mm*



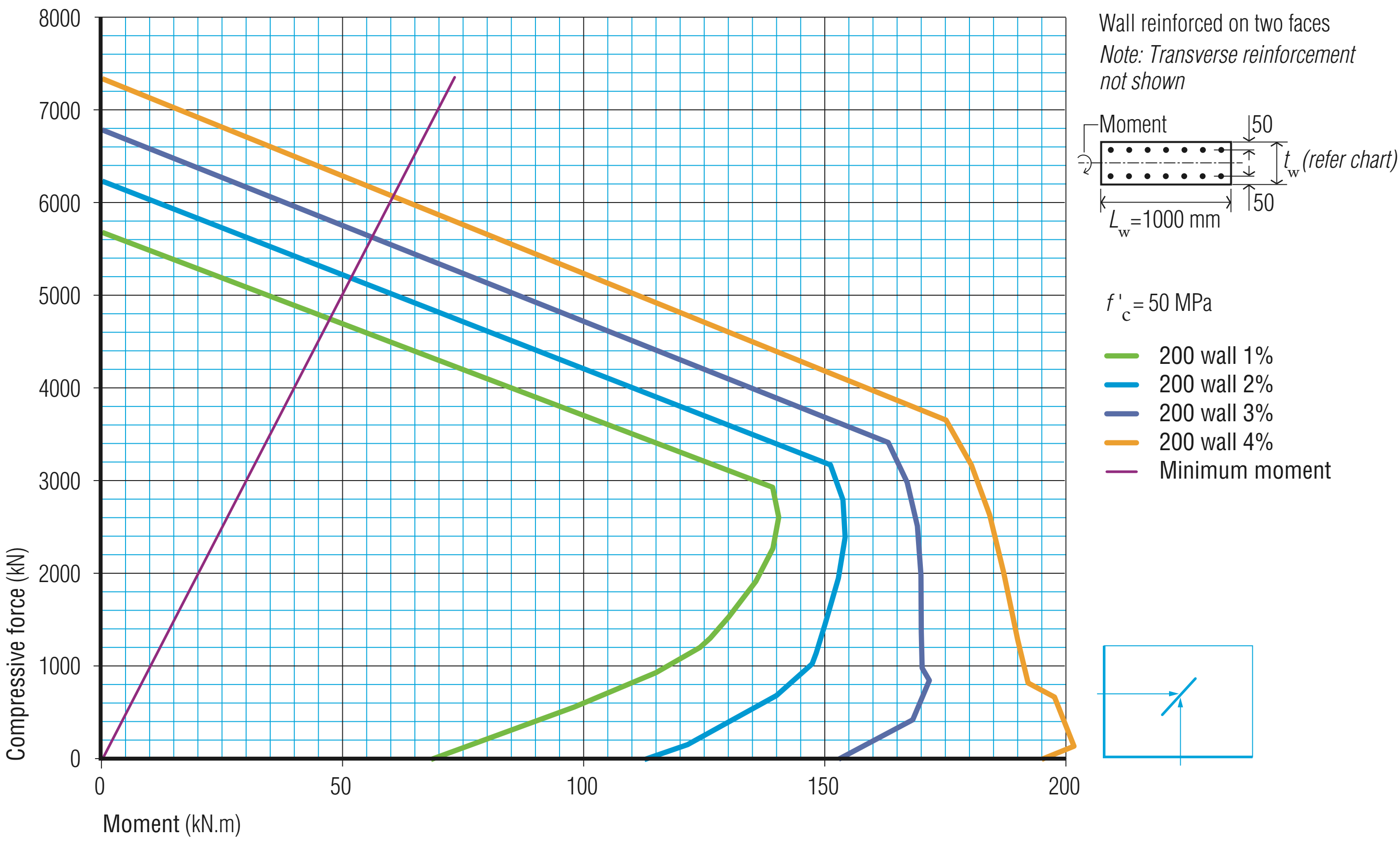
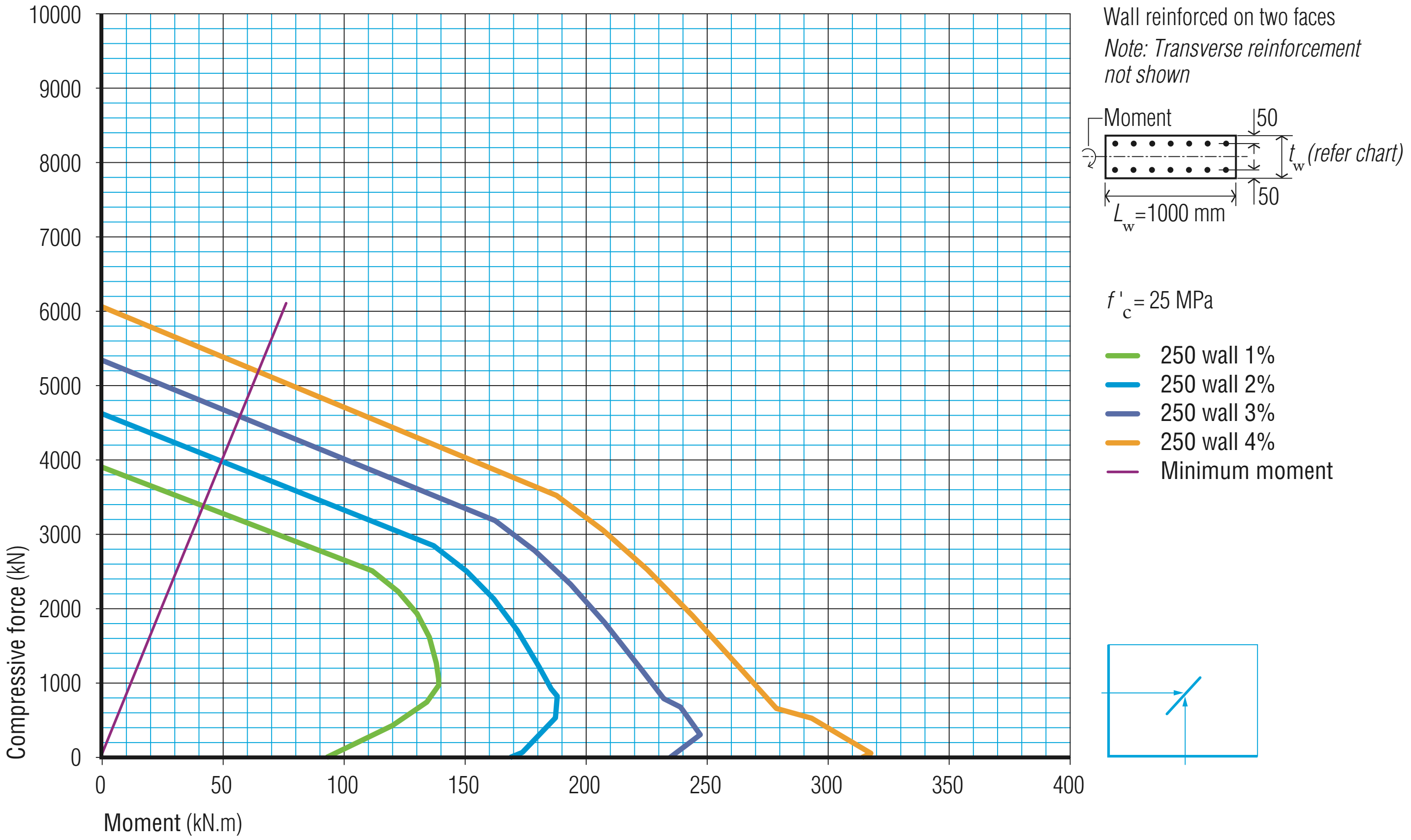


CHART 7.2 250 Thick walls *axis distance = 50 mm*



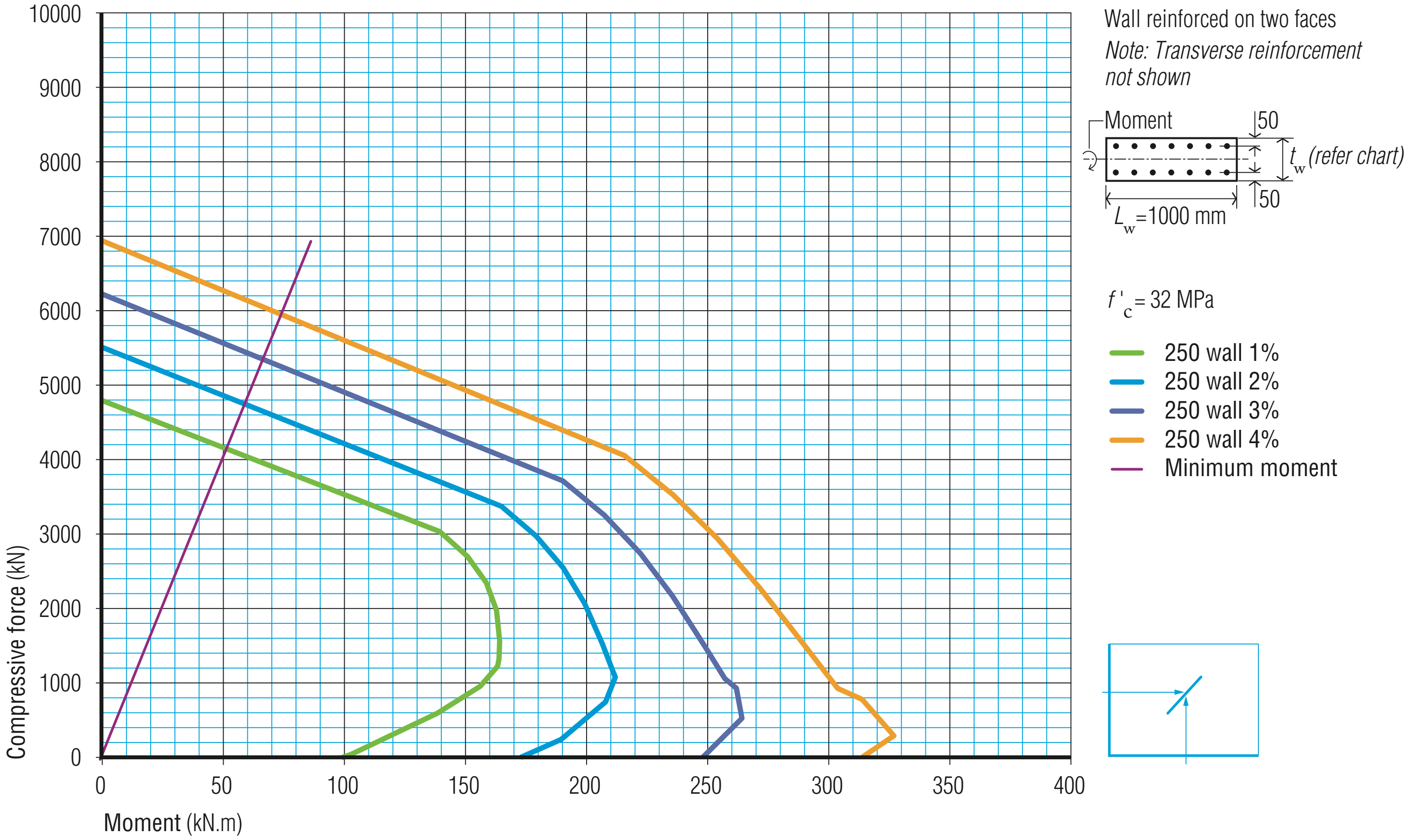
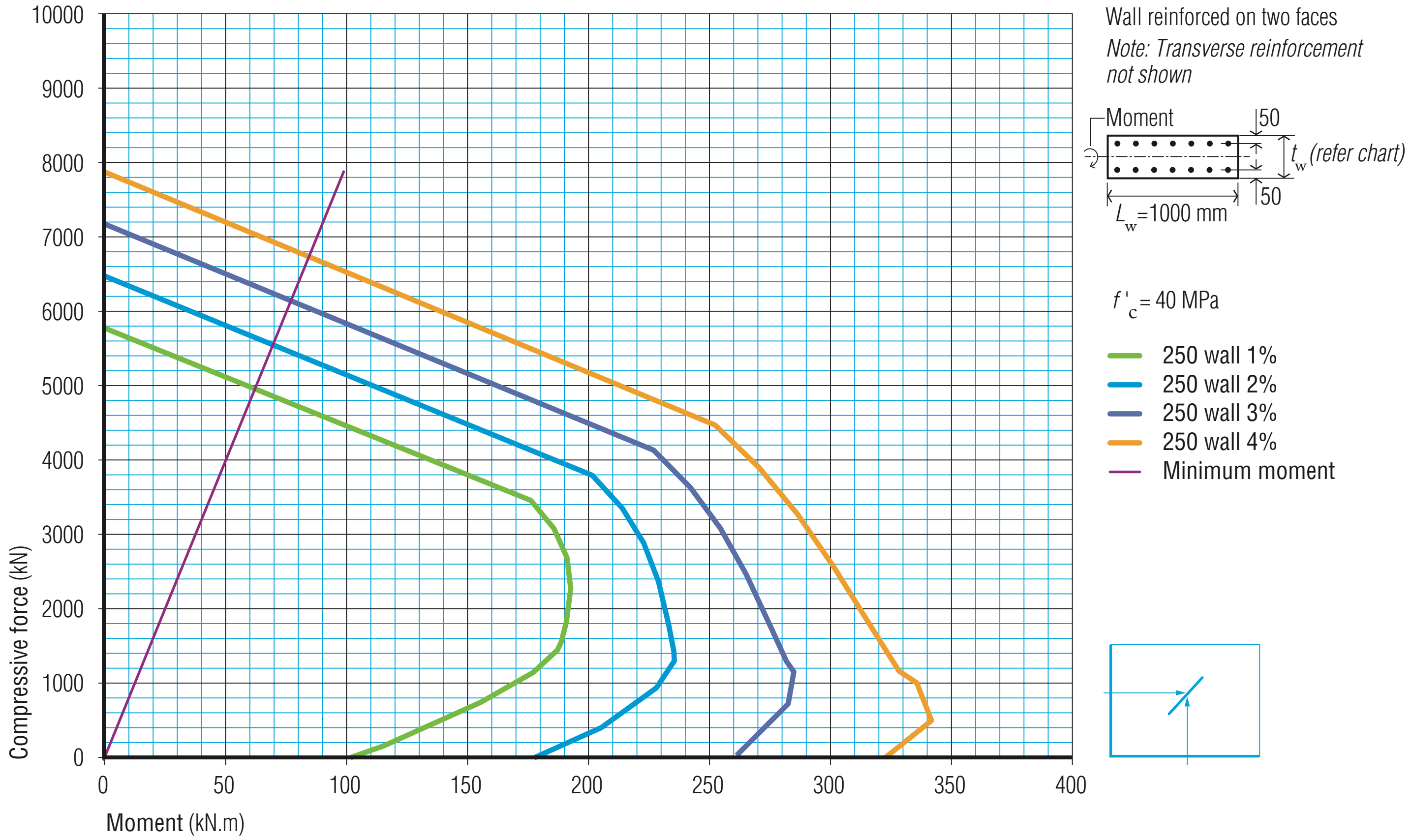


CHART 7.2 *(continued)* 250 Thick walls *axis distance = 50 mm*



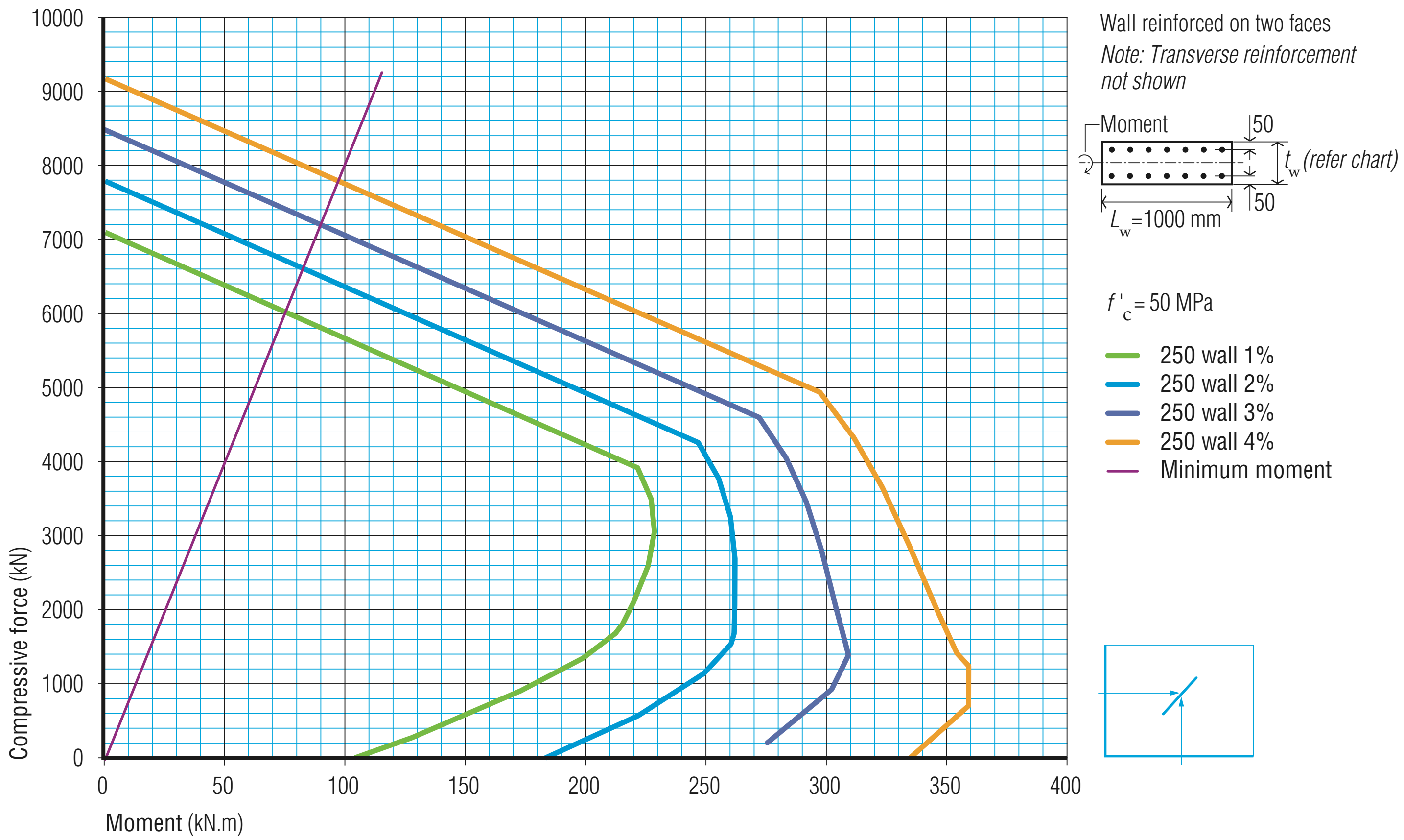
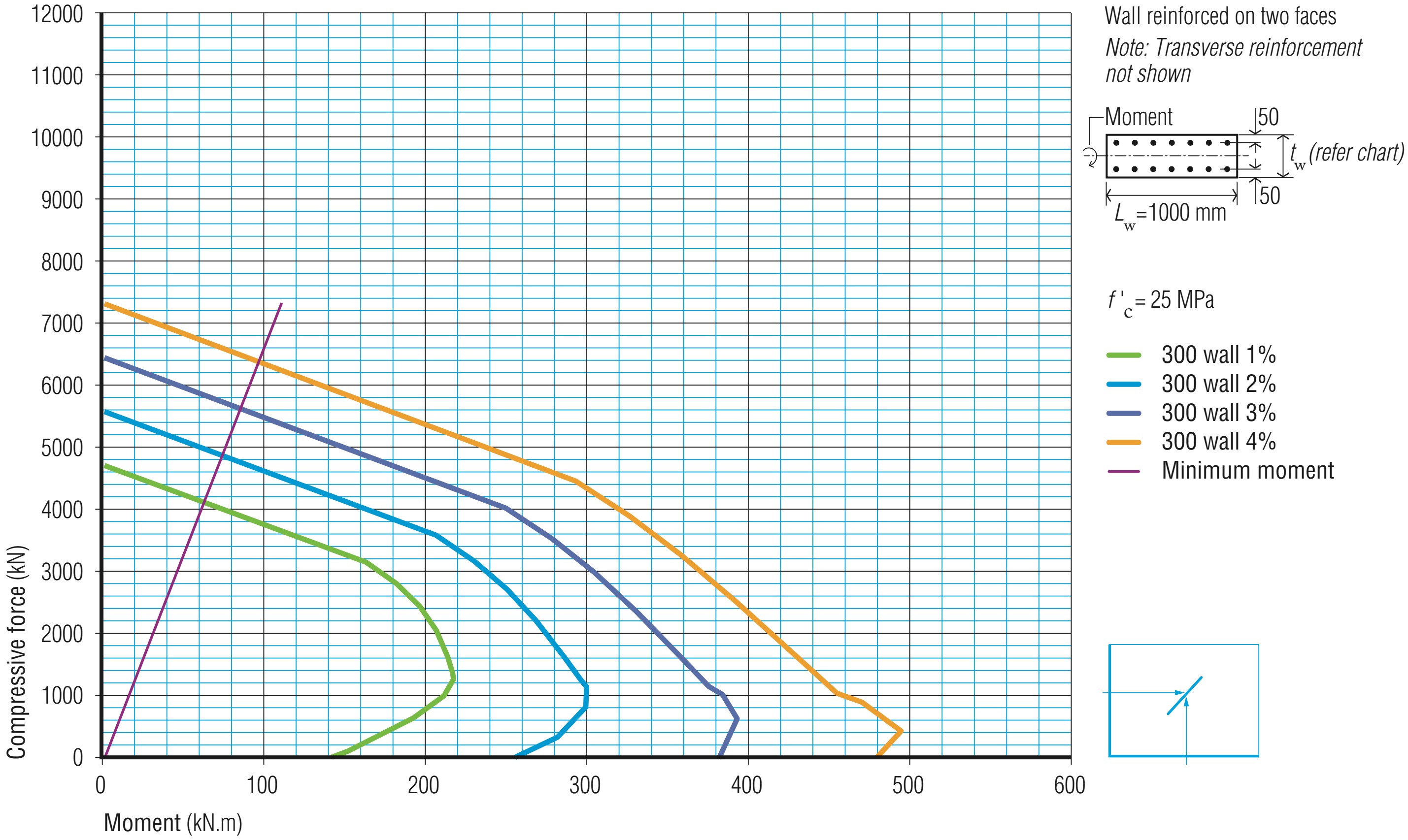


CHART 7.3 300 Thick walls *axis distance = 50 mm*



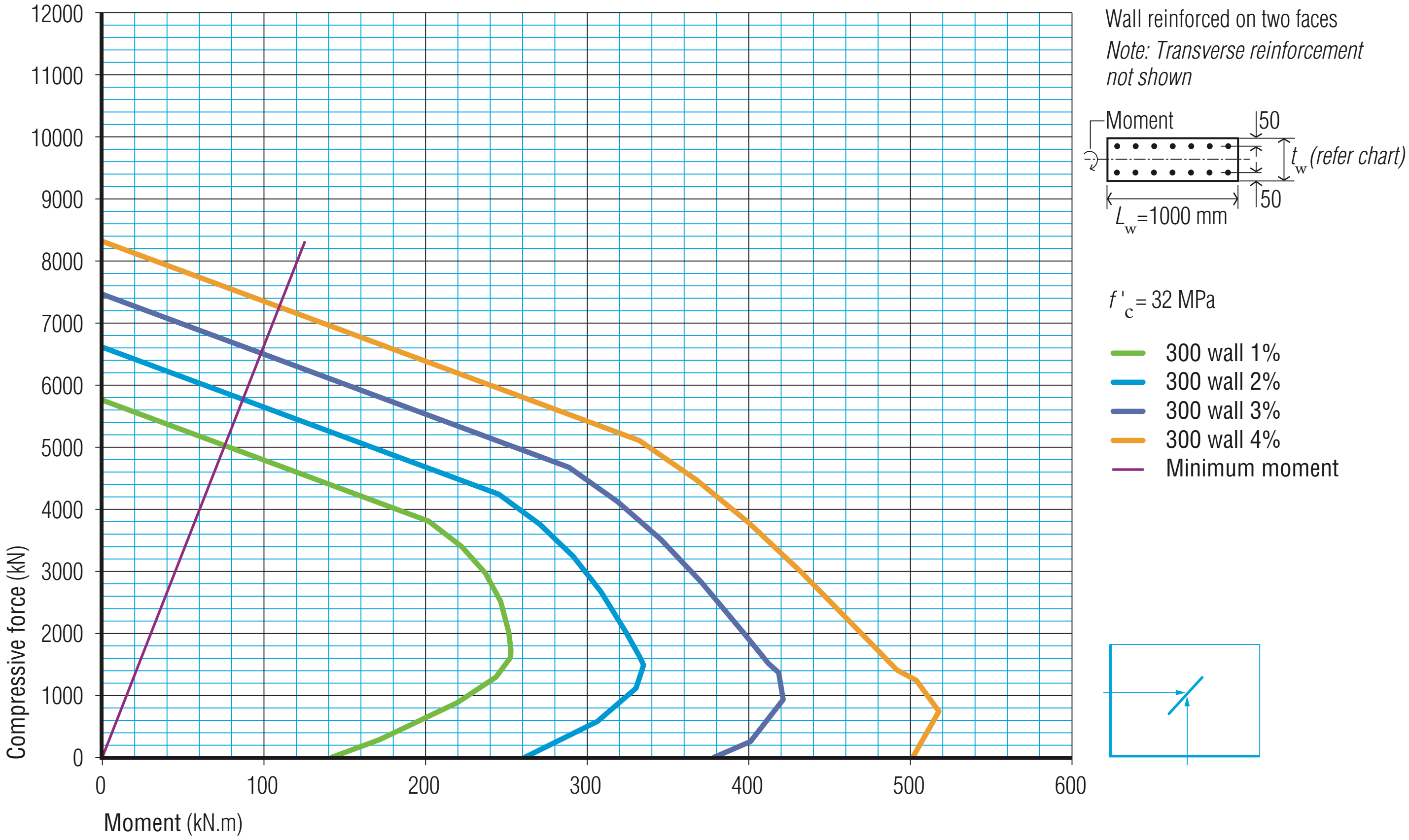
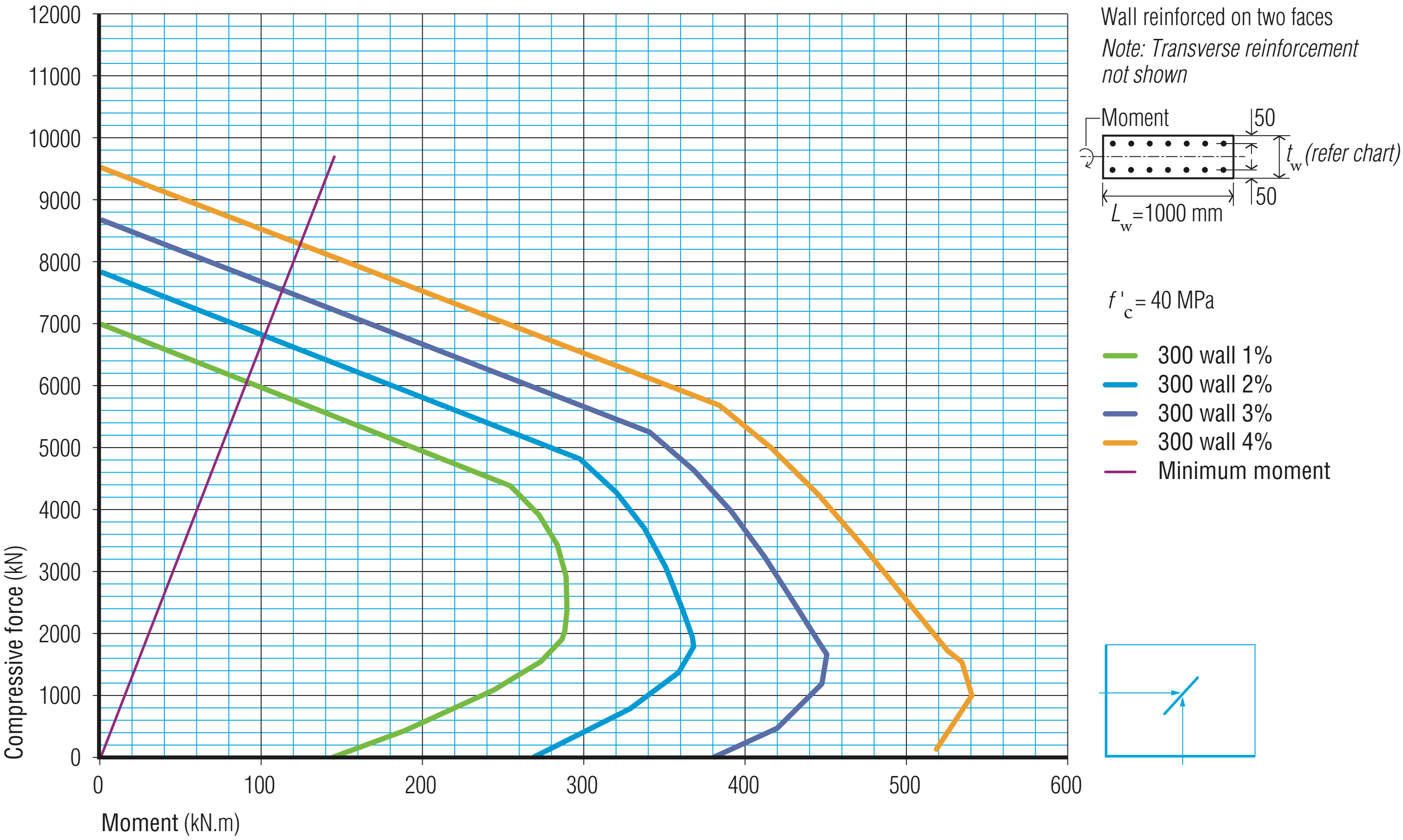


CHART 7.3 *(continued)* 300 Thick walls *axis distance = 50 mm*



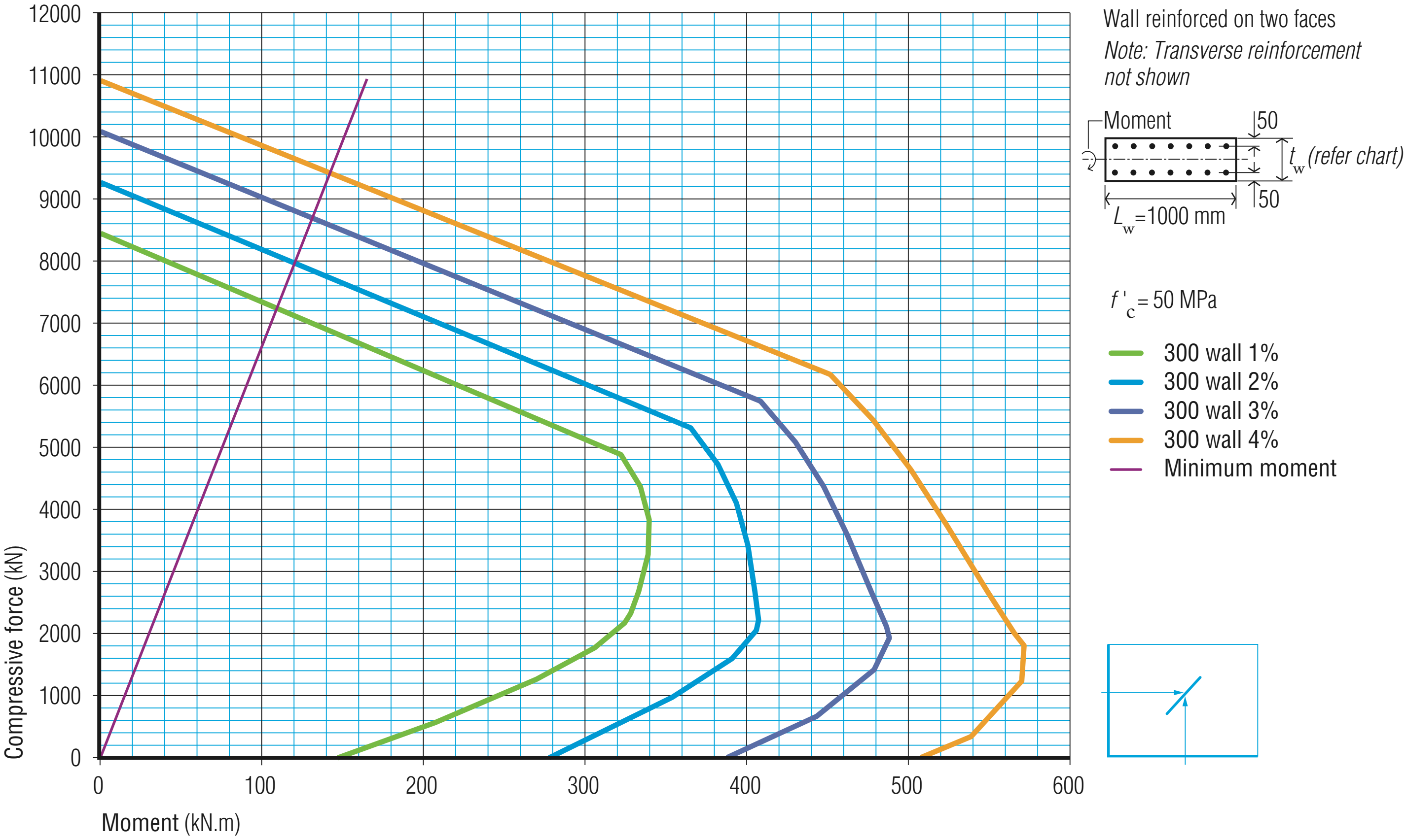
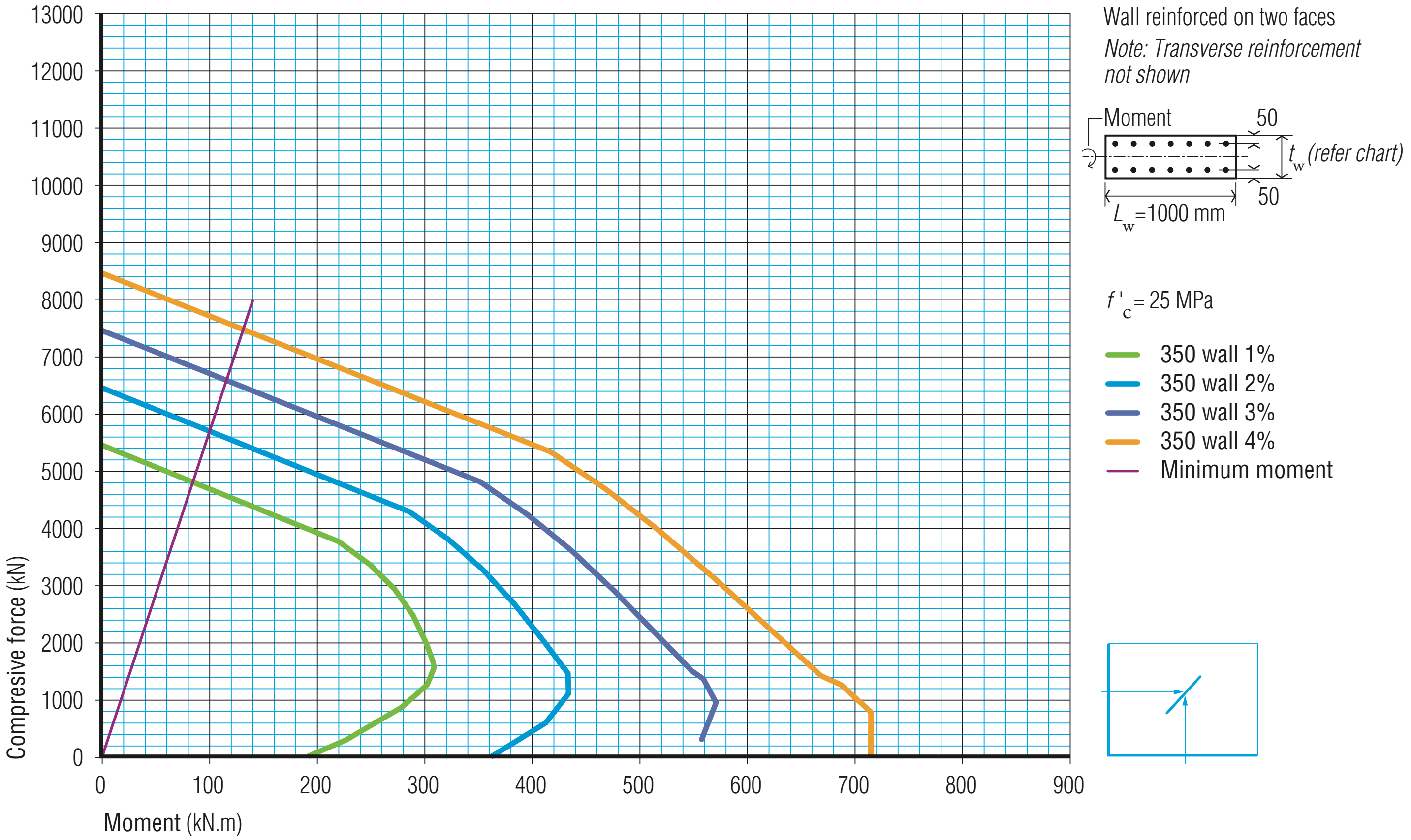


CHART 7.4 350 Thick walls *axis distance = 50 mm*



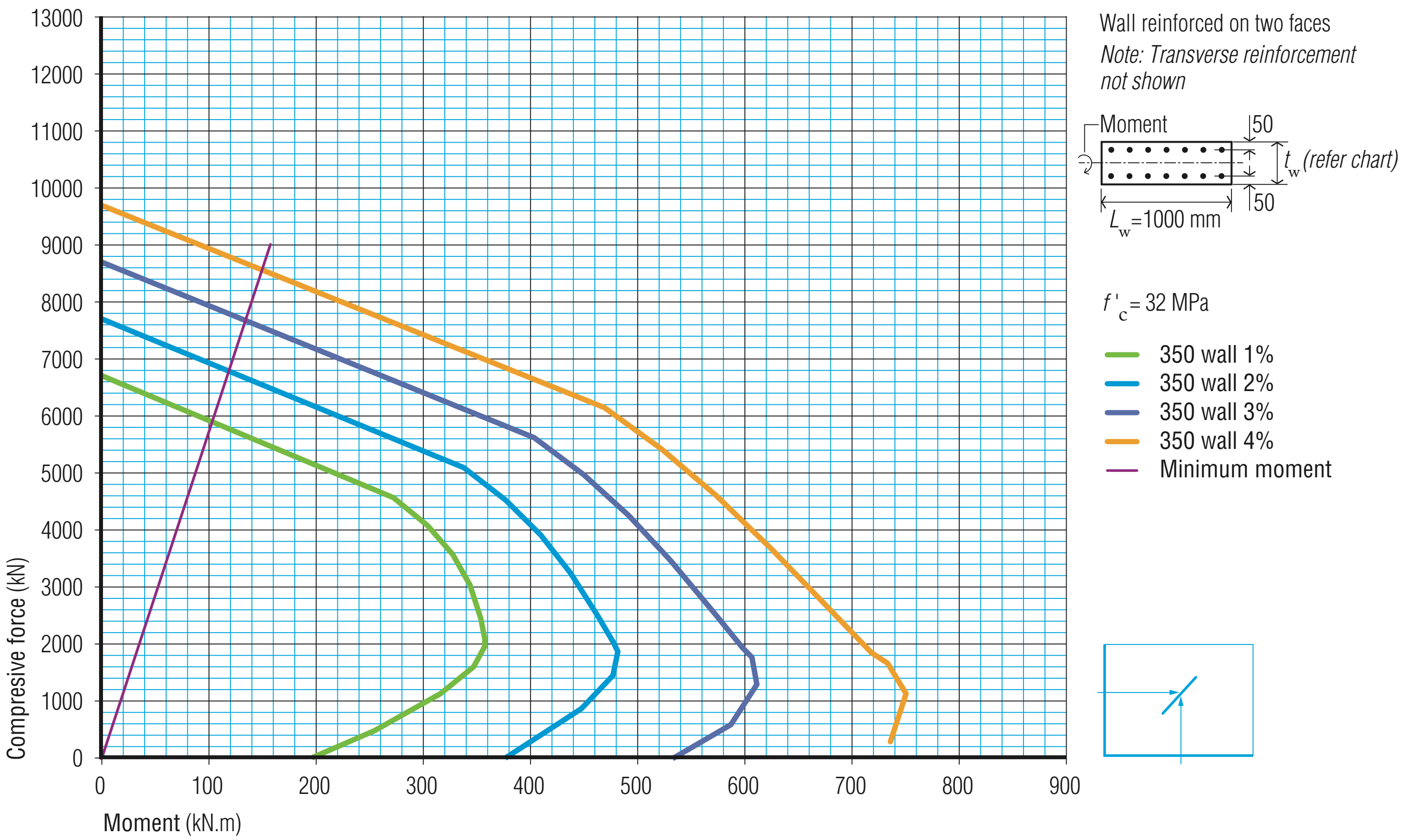
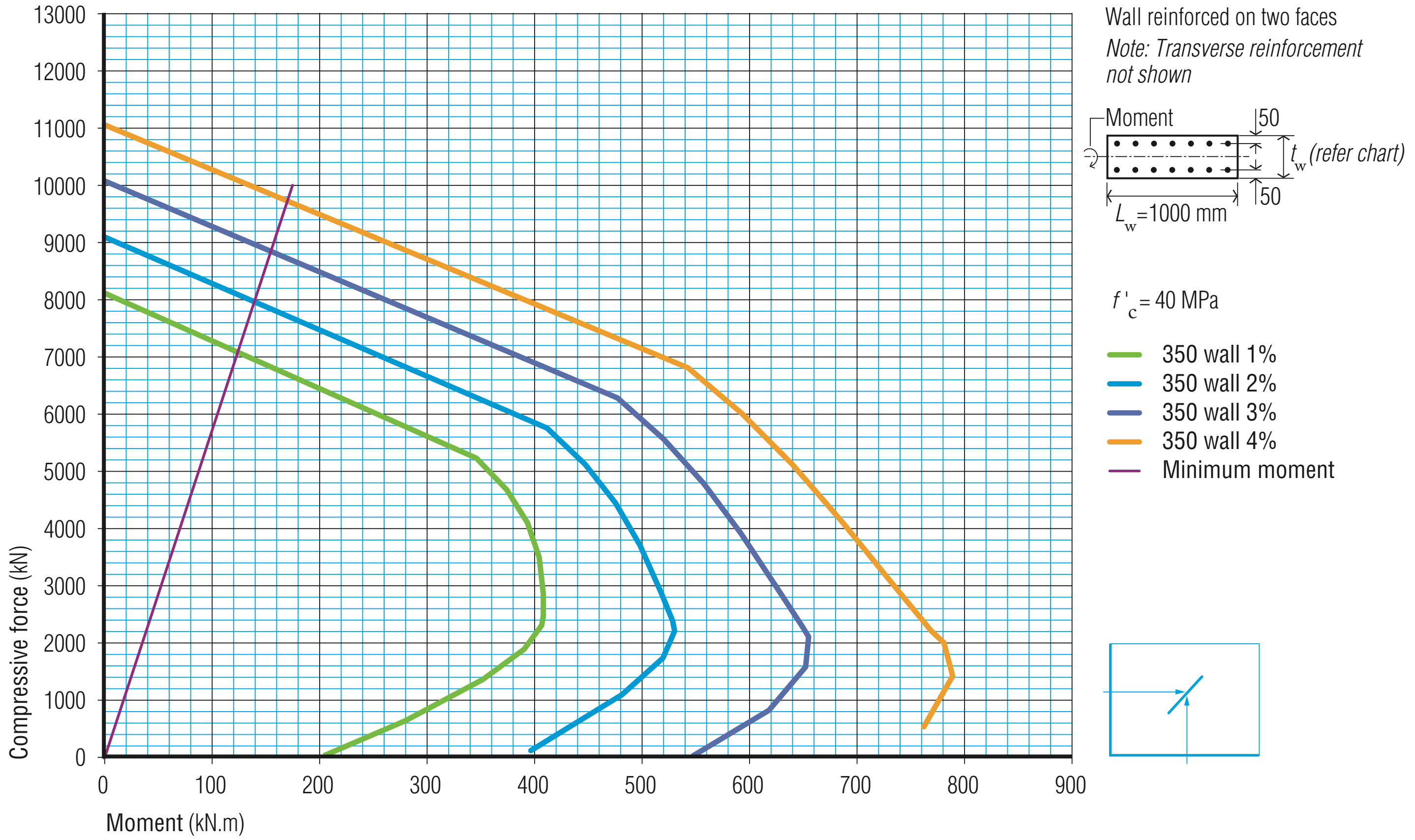


CHART 7.4 *(continued)* 350 Thick walls *axis distance = 50 mm*



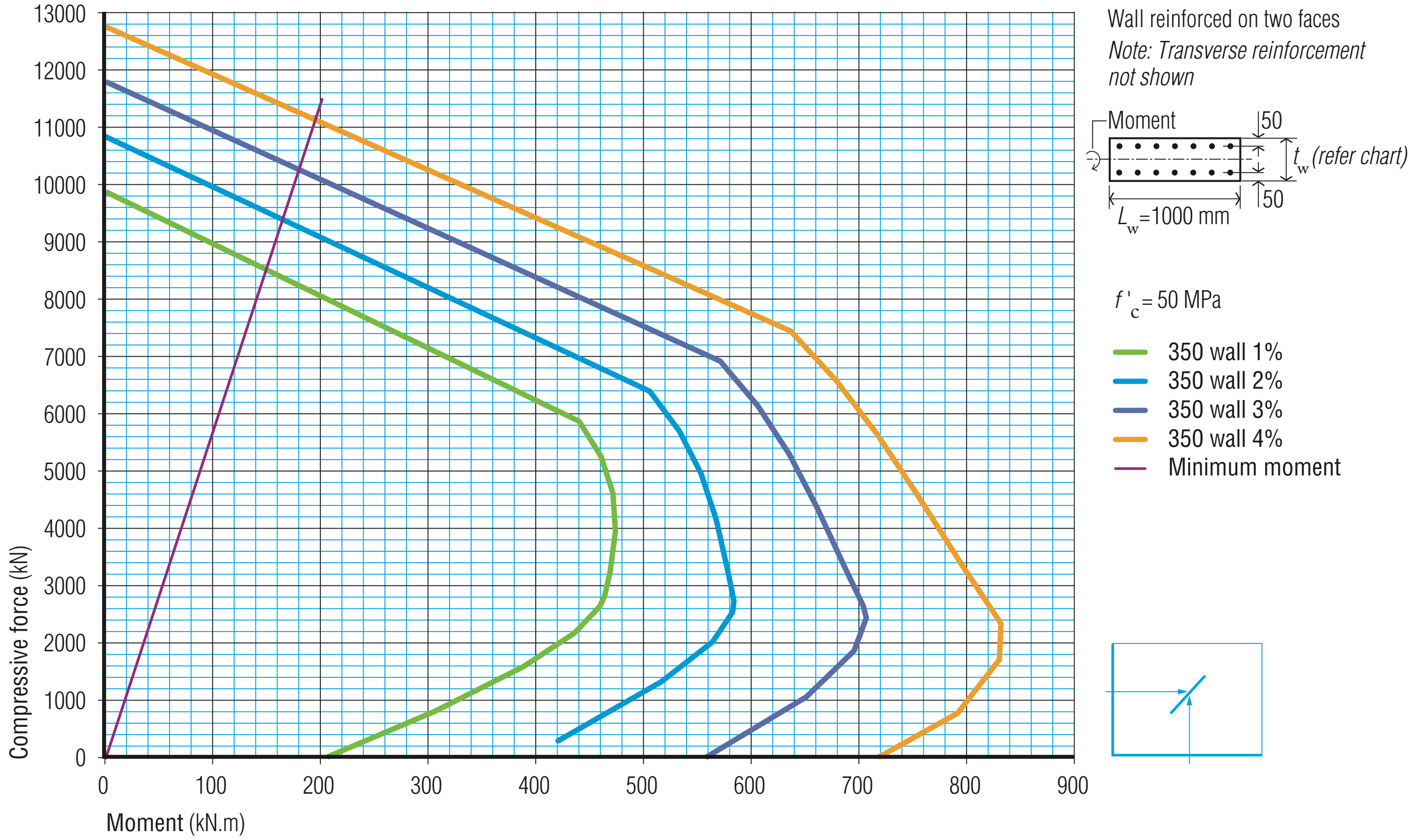
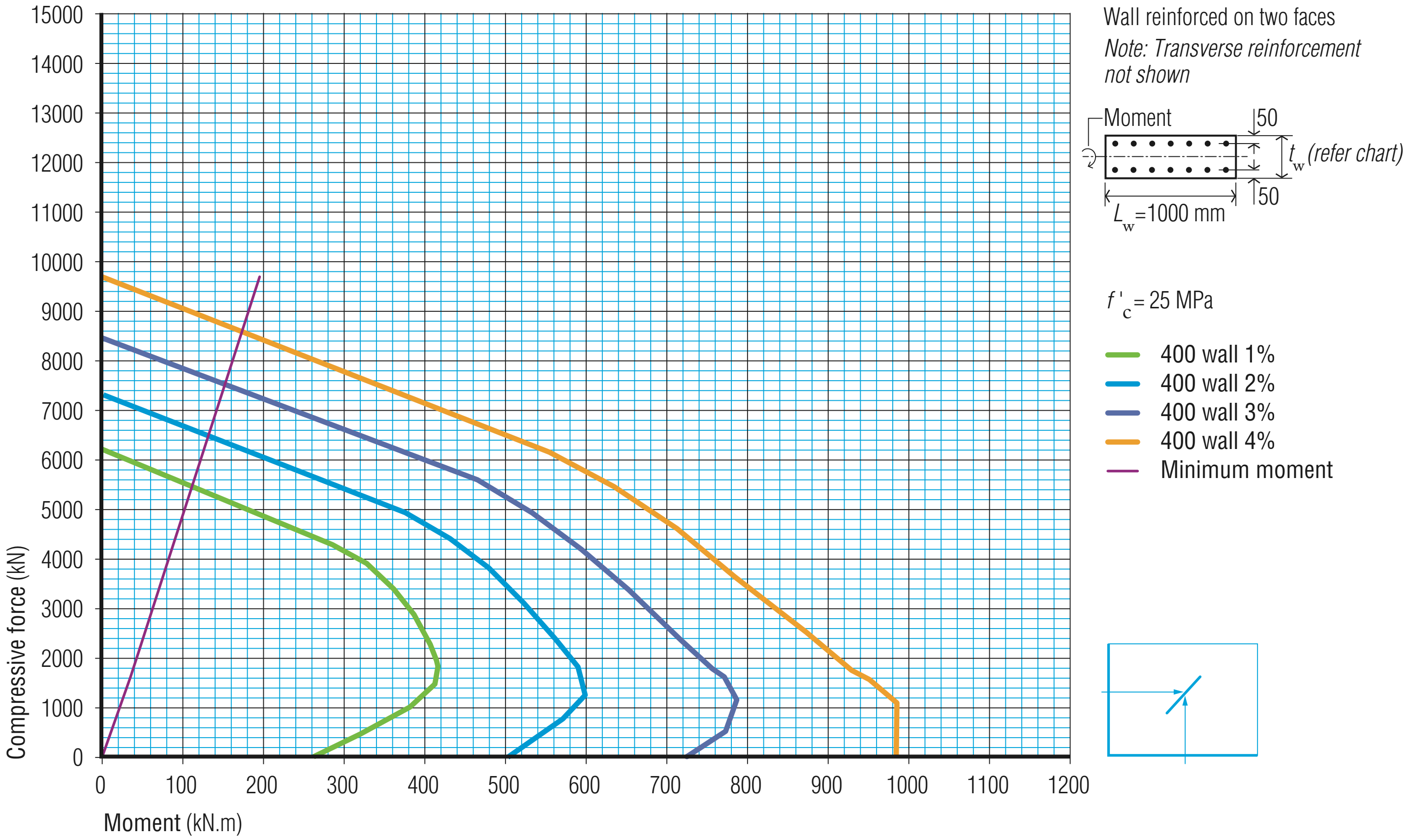


CHART 7.5 400 Thick walls *axis distance = 50 mm*



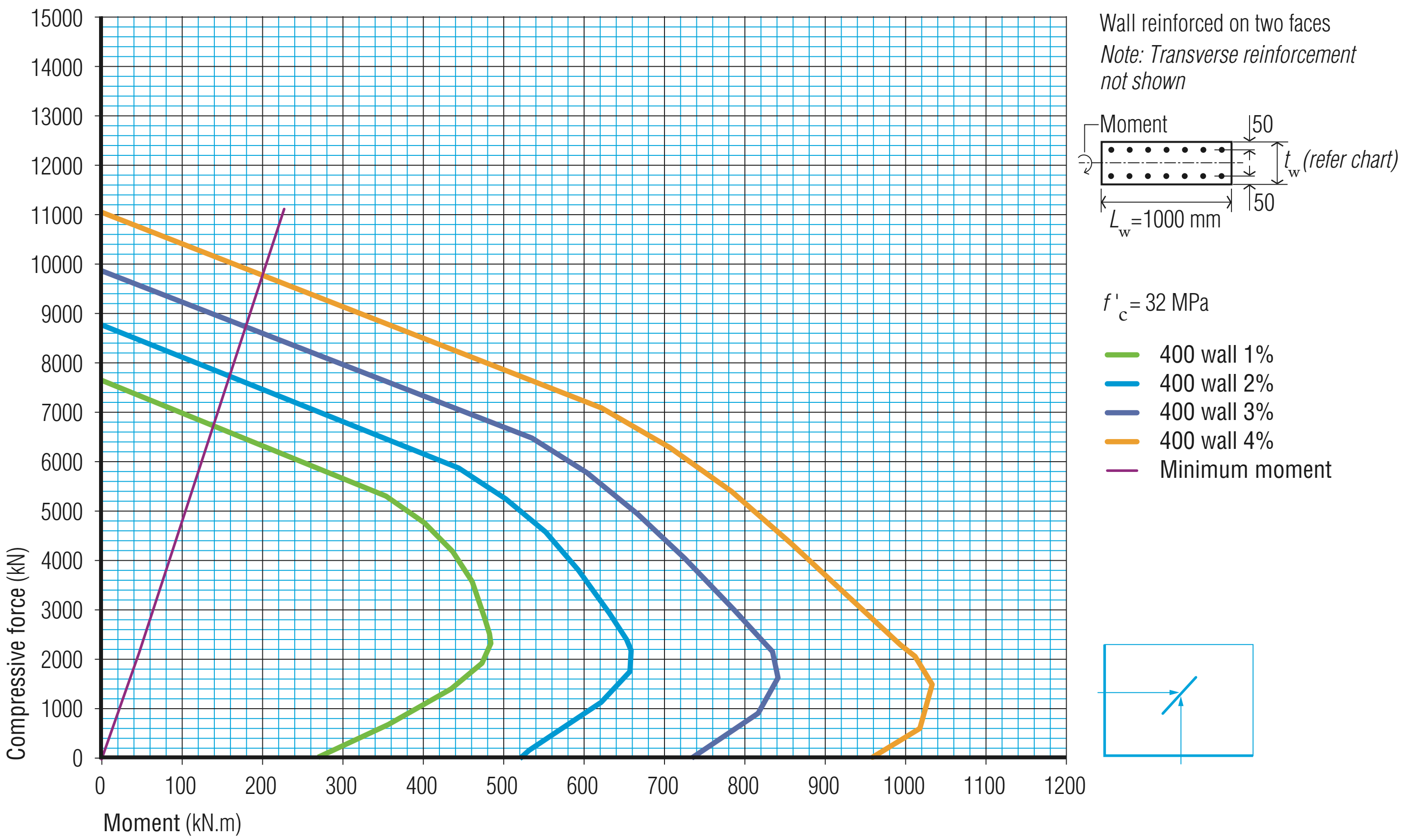
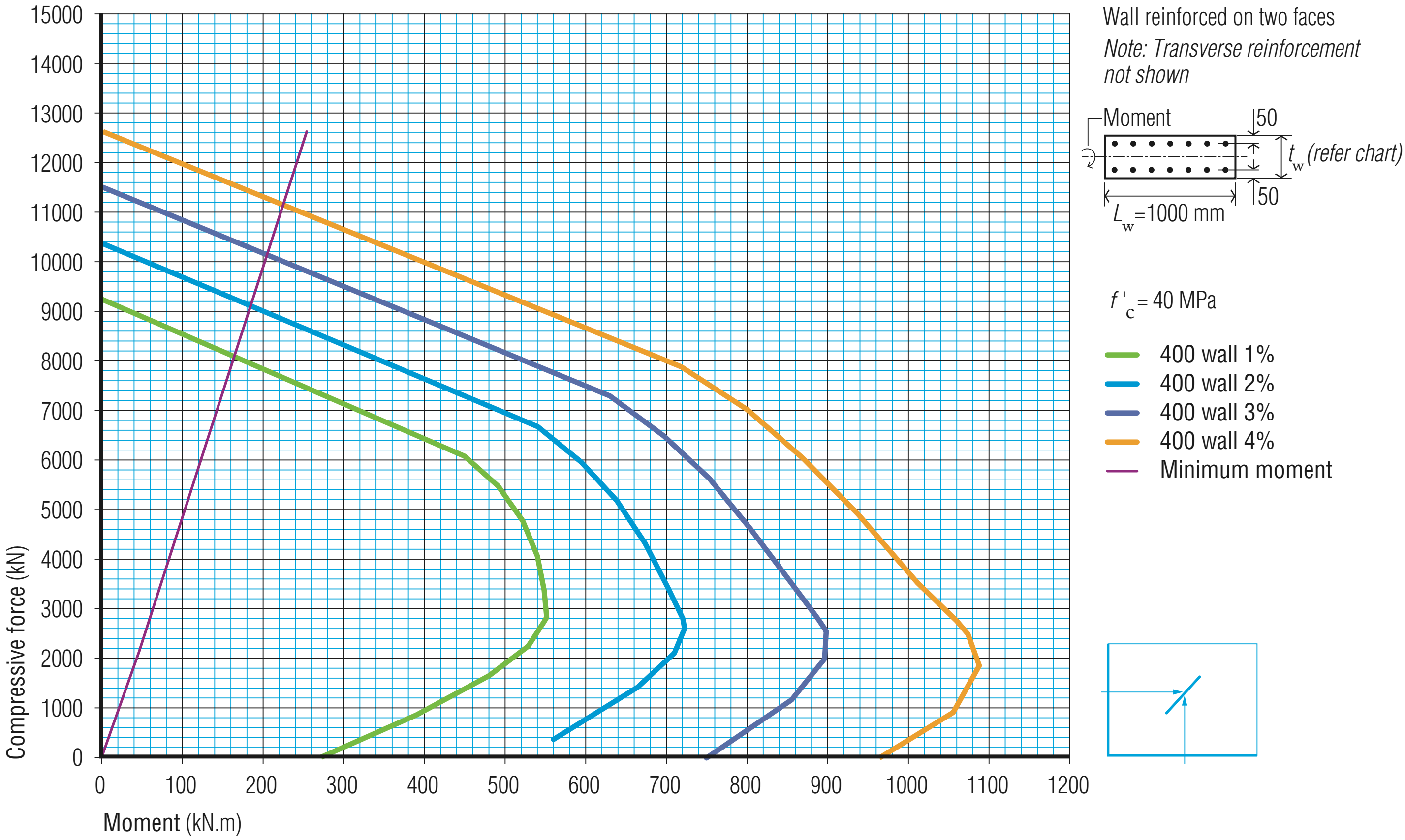
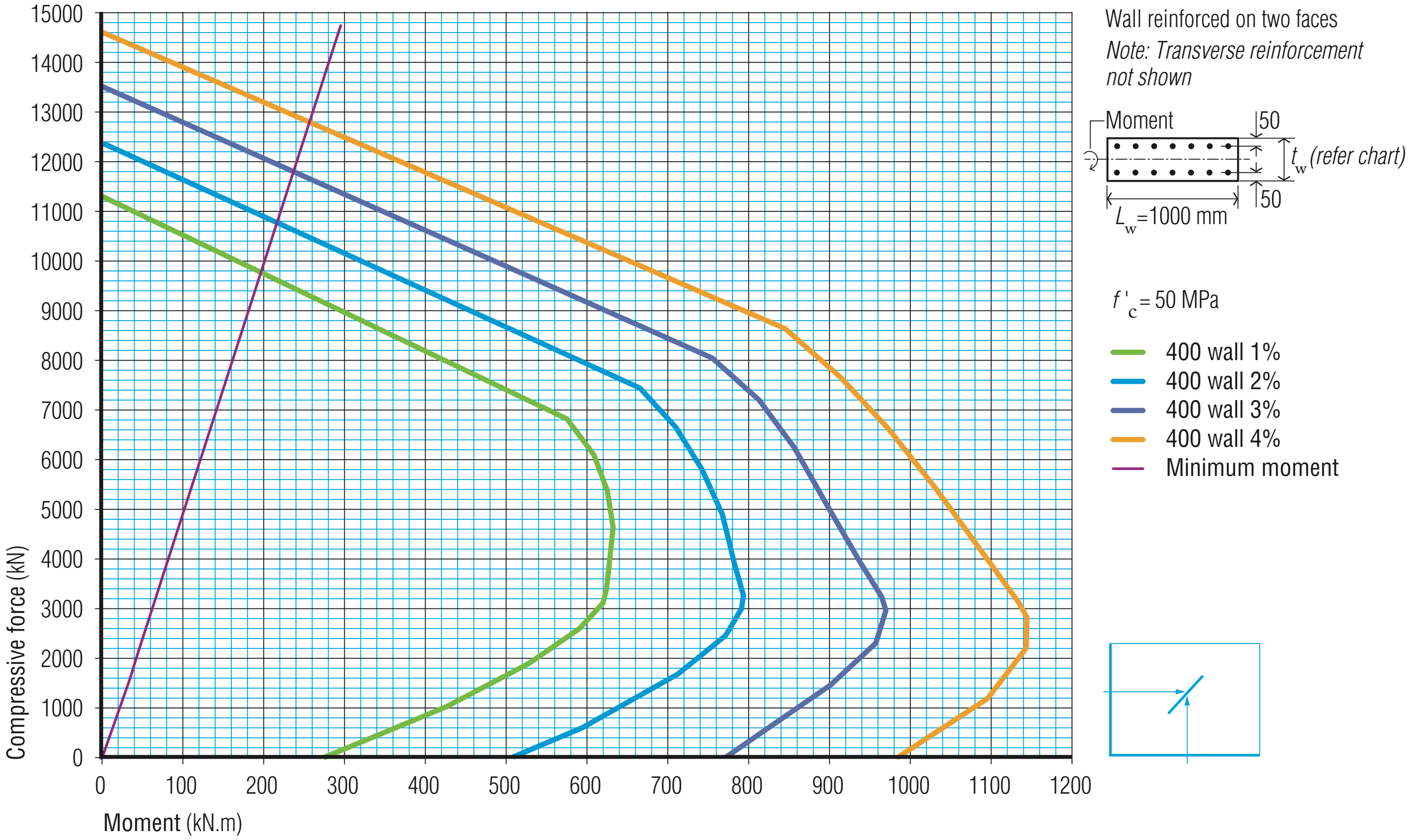


CHART 7.5 400 Thick walls *axis distance = 50 mm (continued)*





7.15 SOME TYPICAL DETAILS

The following figures illustrate some detailing of larger walls. They are for general information and show the sorts of details that may need to be included on the drawings.

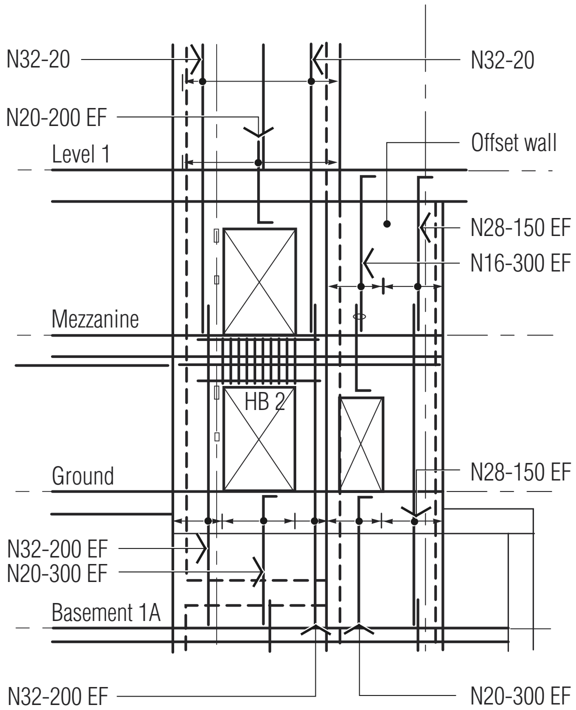


Figure 7.6 This figure shows a heavily reinforced core wall with a header beam under the door openings and heavy vertical reinforcement in each face.

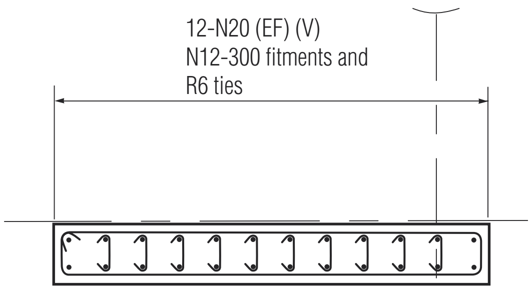


Figure 7.7 This figure shows a reinforced wall with straight bars to the vertical reinforcement, together with fitments to all the vertical bars, so the wall is acting as a column.

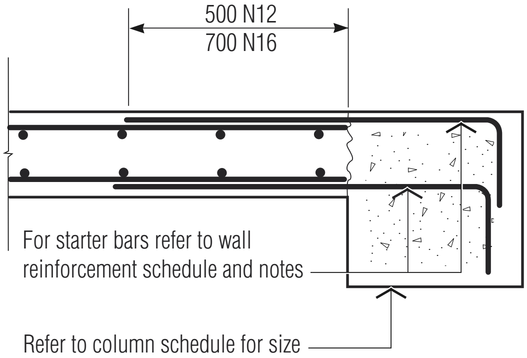


Figure 7.8 This figure shows the connection between a column and a wall in plan. Usually the column is poured first and either starter bars or cast-in ferrules with screw-in bars which are then used to connect the column to the wall which is poured later.

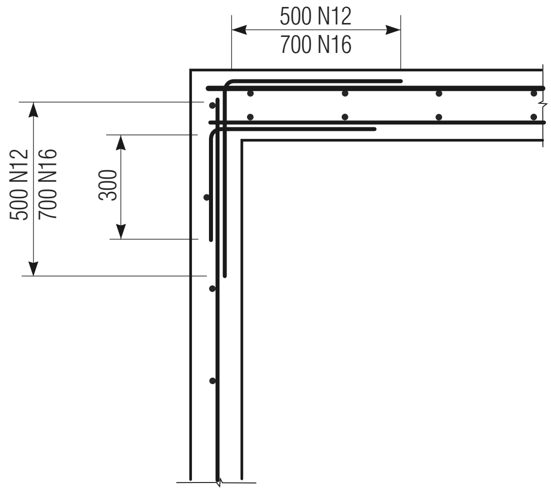


Figure 7.9 This figure shows the corner detail between a wall with a single layer of reinforcement and a wall with two layers of reinforcement to achieve a moderate degree of moment capacity at the corner.

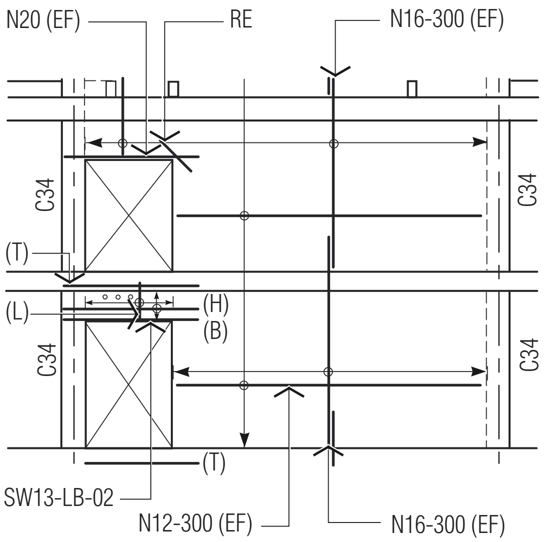


Figure 7.10 This figure shows the elevation of an internal wall with two layers of reinforcement in each face and a header beam over the door opening.

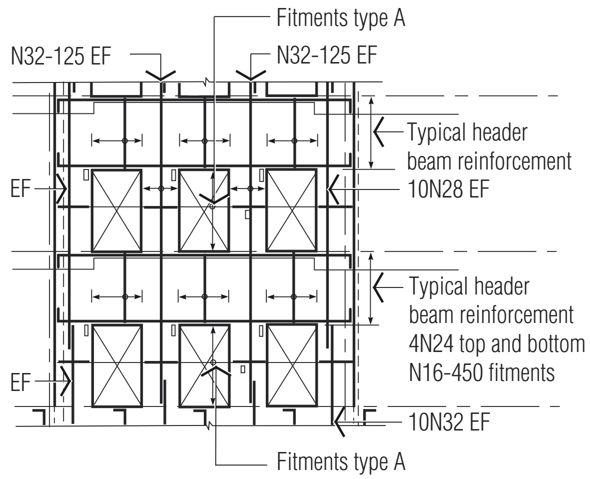


Figure 7.12 This figure shows an elevation of a lift shaft with reinforcement on both faces including the header beam details, etc.

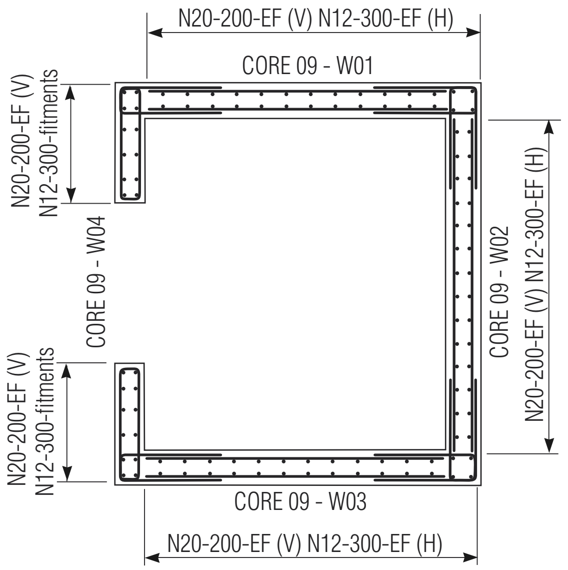


Figure 7.11 This figure shows a section of a lift shaft with reinforcement on both faces including the corner details, etc.

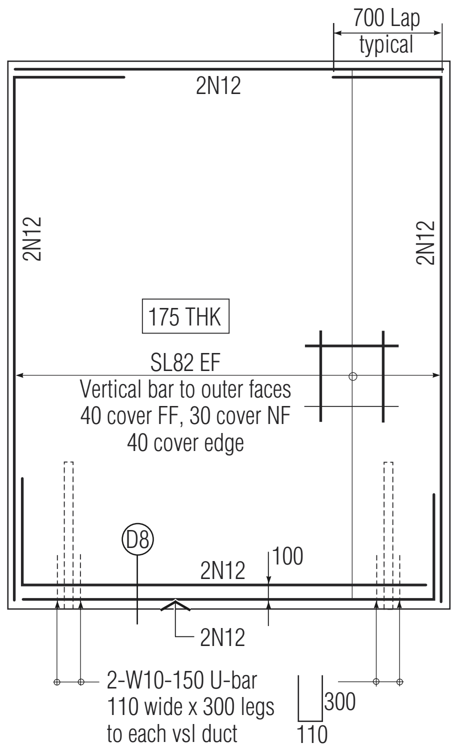


Figure 7.13 This figure shows an elevation of a precast wall panel with two layers of mesh reinforcement, trimmer bars, grout tubes, etc.

REFERENCES

7.1 *Guide to Tilt-up Design and construction,* Cement Concrete and Aggregates Australia and Concrete Institute of Australia, 2005.

7.2 *Precast Concrete Handbook* 2nd Ed, National Precast Concrete Association Australia and Concrete Institute of Australia, 2009.

7.3 AS 3850 *Tilt-up concrete construction*, Standards Australia, 2003.

7.4 *The National Code of Practice for Precast, Tilt-up and Concrete Elements in Building* Australian Safety and Compensation Council, 2008. http://www.ascc.gov.au/ascc/AboutUs/Publications/NationalStandards/National

7.5 AS 3600 *Concrete structures,* Standards Australia, 2009.

7.6 *Building Code of Australia* Australian Building Codes Board, 2010.

7.7 *Guide to Off-form Concrete Finishes,* Cement Concrete and Aggregates Australia, 2006.

7.8 AS 1170.4 *Structural Design Actions, Part 4 Earthquake actions in Australia,* Standards Australia, 2007.

7.9 Mendis P *Design of High-Strength Concrete Members: State-of-the-Art*, Engineers Australia, 2001.

7.10 *Specification for Piling and Embedded Retaining Walls,* 2nd Ed, The Federation of Piling Specialists in Association with BGA and ICE, 2007.

7.11 AS 4678 *Earth-retaining structures,* Standards Australia, 2002.

7.12 Hughes S R and Crisp B C *Structural Precast Concrete in Melbourne Australia* Concrete Institute of Australia Biennial Conference 2007.

7.13 *Concrete Panel Buildings*, Briefing 08, Cement Concrete and Aggregates Australia, 2003.

7.14 Woodside J *The Evolution of Architectural Precast Concrete Facades in Australia over the last 50 years*, Concrete Institute of Australia Biennial Conference, 2009.

7.15 *The Concrete Panel Homes Handbook* Cement Concrete and Aggregates Australia, 2001.

7.16 AS 2870 *Residential slabs and footings – Construction*, Standards Australia, 2011.

7.17 BS 8110 *Structural use of concrete* Part 1: *Code of practice for design and construction* British Standard Institution, 1997.

7.18 *Reinforcement Detailing Handbook* (Z06), 2nd Ed, Concrete Institute of Australia, 2010.

Chapter 8_Footings

8.1 GENERAL

Reinforced concrete footings support the columns and walls at the base of a building. As they are usually concealed, typically cannot be inspected, or maintained, and are constructed in variable and sometimes unstable ground, a particular level of care needs to be given to many aspects of their design. These include such matters as the allowable bearing pressures, settlements, durability and cover to reinforcement. In some ground conditions, a blinding layer of 50–75 mm of about 10 MPa unreinforced concrete should be used to seal the bottom of the footing and provide a clean and stable surface for fixing of reinforcement, especially when the excavation is likely to be unstable, wet or muddy.

Footings transfer the loads from the structure to its foundation—the natural soil or rock on which it rests. ('Foundation' is sometimes used to describe footings defined above; in this Handbook, it is used to describe the material on which the footing is supported.)

Founding conditions vary widely in Australia and more often than not vary across a site. Before any footing design is undertaken, the properties of the foundation material must be determined. This involves an appropriate geotechnical investigation by a suitably qualified person, usually a geotechnical engineer, and testing as required. In addition to assessing the allowable bearing pressure for pad and strip footings, the investigation should include advice on such matters as the expected soil profile across the whole site, any water tables and dewatering requirements, likely short-term and long-term settlements and differential settlements under load between adjacent footings, whether the footing will be cast in aggressive soils, etc. Also, with expansive clay soils, shrink-swell movements may also need to be considered. Local knowledge and contractor's experience will often dictate appropriate footing types and construction techniques.

The selection of the appropriate footing system is often a key structural design decision. The design of footings requires a disciplined approach and consideration of many factors such as the site history, geotechnical conditions including the various layers of soils, the site levels and future levels, the site conditions and constraints, environmental conditions, the building and its constraints. Designers should refer to texts such as *Craig's Soil Mechanics* [8.1] and *Structural Foundation Designer's Manual* [8.2] for further information on such matters.

Using the geotechnical investigation, a decision has to be made on what stratum of soil will support the footing, the allowable bearing pressure, skin friction if it is a pile, the founding level of the footing, possible types of footings and any potential problems with excavation, aggressive soils and durability. This founding level must take account of both the proposed excavated and any future excavated levels, as appropriate.

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

- pad (spread) footings or combined footings
- strip footings
- piled (or pier) footings
- raft footings
- balanced or coupled footings.

It should be noted that there are a number of variations within each type, including, in some cases, unreinforced footings. Designers should refer to appropriate texts for further information on these various alternatives.

This chapter deals only with the detailed design of reinforced concrete pad or spread footings with concentric vertical loads, although strip footings and piled footings are also mentioned. Raft footings, combined and balanced footings and coupled footings are mentioned only briefly. Generally, except for small or light structures, footings should be founded 300–1000 mm into the ground, and 200–600 mm into undisturbed material or engineered fill. The founding layer and allowable bearing pressure should be confirmed on site, by a suitably qualified person or the geotechnical engineer, following the excavation of the footing. Where ground conditions are difficult or variable, changes may be necessary to suit site conditions, eg deeper or larger footings.

There are a number of different types of raft footings. Stiffened-raft footings are commonly used in Australia for houses in areas with expansive clay soils and sometimes for slabs-on-ground for houses that have masonry walls. For domestic construction, the stiffened raft footing consists of concrete slab, edge beams and internal beams at close centres, usually all poured at one time. The purpose of the stiffened raft footing in domestic construction is generally to provide a stiff element, which will cope with soil moisture movements. Alternatively, strip footings can be used where the ground floor is above the ground level and where lightweight floors are used. Bearing pressures are usually not that critical for such domestic footings. Designers should refer to AS 2870[8.3] for the design of residential footings.

For high-rise buildings, raft footings can be thick reinforced concrete plates, sometimes with thickening of 900 to 2000 mm under the columns and loadbearing walls to spread the column and wall loads over as large an area as possible – ie in effect, a very large pad footing.

For industrial and other single-storey buildings, strip footings are sometimes combined with the slab-on-ground as an edge thickening to support external precast concrete walls or the like.

8.2 SPREAD FOOTINGS

8.2.1 General

For pad or spread footings, the soil contact pressure under axial load produces well-defined conditions of moment, beam or slab shear, and punching shear in the footing. Although it is recognised that the soil pressure distribution is non-linear, for the simplification of the design of concentrically loaded footings linear distribution is usually assumed. The designer may choose a more accurate soil pressure distribution to suit actual conditions, based on geotechnical advice.

For a pad footing on deformable soils, it is assumed that the loads are resisted by flexural slab action in which adequate thickness is usually assured by a strength assessment in accordance with AS 3600. However, a footing on rock or other stiff medium may require an assessment as a stiff shear element, in which the ability to assume plate deformation is limited by the stiffness of the supporting medium. In these cases, the designer may choose to assess the structural action from strut-and-tie theory, deep-beam theory or similar.

The size of a pad footing is determined from the design actions resulting from the appropriate combination of loads or other actions. The footing may take vertical loads, horizontal loads and moments depending on the assumptions made in the structural analysis and the ability of the footing and foundation to resist such actions.

For a particular building it is preferable to maintain similar soil pressures under all pad footings to avoid significant differential settlements. However, large footings will settle more than smaller footings with similar soil pressures, due to a deeper zone of influence.

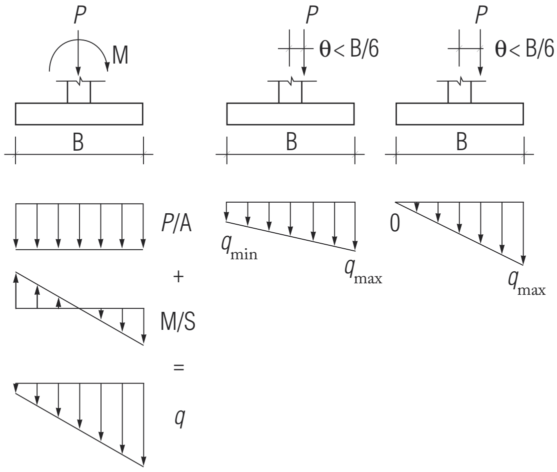


Figure 8.1 Eccentric loads

For eccentric columns and walls or footings with moments, the bearing pressure will vary under the pad footing which should generally be proportioned so that zero bearing pressure occurs at one edge in the worst case—so that tension does not occur under the footing and q_{max} does not exceed the allowable bearing pressure as shown in Figure 8.1.

Square pad footings are used where possible as this simplifies the design, although rectangular ones can be used. Pad footings generally are not reinforced for one-way shear, consequently the thickness of the pad footing may be a function of one-way shear stress in the section. The initial thickness of the footing will often be determined by the development length of the column or wall starter bars, assuming full compression development length is required for the starter bars.

The allowable bearing pressure provided by the geotechnical engineer is the maximum bearing pressure that can be applied to the foundation such that there is an adequate factor of safety against instability due to shear failure of the founding material and the maximum tolerable settlement is not exceeded.

Settlements can be immediate and/or long term and calculations of settlements if required are generally carried out by the geotechnical engineer based on the loads provided by the structural engineer. Exact estimates of settlements are difficult and often of the order of accuracy of 5–25 mm. Settlements can affect services, finishes, the concrete structure and the building as a whole. Generally, it is differential settlement that is of most concern and whether the superstructure can tolerate such settlements. Examples of where differential settlements may occur are at the junction between a tall section of a building and a low-rise section, or a change in ground conditions under a building, which might require movement joints at appropriate locations. Experience and engineering judgment is needed in assessing such settlements and how to design for them.

The total allowable bearing pressure provided by the geotechnical engineer is normally calculated from the ultimate bearing capacity using a factor of safety (usually of the order of 3). The footing size is then based on the total load divided by the total allowable bearing pressure. While it is possible to deduct the weight of any surcharge loads to get a net allowable bearing capacity for the design of the concrete, designers need to check with the geotechnical engineer that the allowable bearing capacity allows this deduction, for the footing being designed.

For the design of both plain concrete pedestals and plain concrete pad footings (unreinforced), designers should refer to AS 3600 Section 15, and for reinforced footings to AS 3600 Section 16, which in turn refers designers to Section 9.

8.2.2 Development of column starter bars

The transfer of load from the column to the footing is by a combination of end bearing of the column and the starter bars in the footing.

AS 3600 Clause 12.6 requires that, unless special confinement reinforcement is provided, the design bearing stress at a concrete surface shall not exceed $0.9\,\phi f'_c \sqrt{(A_2/A_1)}$ or $\phi\,1.8\,f'_c$ whichever is the less. This usually means that the area of the starter bars does not have to match that of the column reinforcement. However, for both columns and walls, it is common to use the same size and number of column or wall starter bars as used for reinforcement in the column or wall above as shown in Figure 8.2.

AS 3600 includes no specific requirements for the minimum area of starter bars but ACI318[8.4] requires a minimum area of 0.5% of A_g, which seems to be prudent. This issue is discussed in more detail in the *Design Handbook for Reinforced Concrete Elements*[8.5] and with design examples.

The initial estimate for the footing depth is often based on the ability of the column or wall starter bars to transfer the applied forces into the footing. The usual condition is compression in the column or wall bars, and sufficient depth must be available to develop the expected compressive force (see AS 3600 Clause 13.1.5.1 and Chapter 2).

However, if the footing is fixed, ie resisting moment, the development length of the bars on one or more faces of the column or wall may be in tension under certain loads. This may require the development length in tension to be considered, frequently with cogged starter bars being needed.

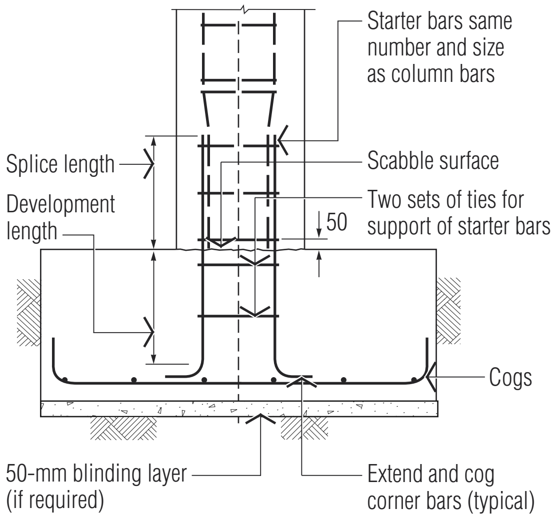


Figure 8.2 Typical pad footing

TABLE 8.1 Footing depths (rounded to the nearest 50 mm) assuming full development length of starter bars in compression

Starter bar size						
N12	N16	N20	N24	N28	N32	N36
400	500	600	700	800	900	1000

Table 8.1 can be used for initial sizing of footings. It assumes: a concrete of $f'_c \geq 25$ MPa; a bottom reinforcing mat consisting of two layers of N20 bars each way; 50 mm cover to the bottom bar of the mat; the column or wall starter bars have to develop their full development length in compression. Note that in a column the corner bars are usually cogged to support the reinforcement cage off the bottom reinforcement mat and fitments are used at about 300-mm centres to allow the cage to be held in place and fixed.

If the column or wall bars do not need to develop their full development length in compression then the depths shown can be reduced. All of these assumptions will need to be checked against the final design requirements and adjusted appropriately.

With a minimum embedment length of 200 mm for the column or wall starter bars required by AS 3600, and using the design parameters above, the minimum depth of any reinforced pad or strip footing will be about 300 to 400 mm. The minimum depth of any unreinforced footing allowed by AS 3600 Clause 15.4.1 is 200 mm.

Consideration needs to be given to the minimum concrete strength of the footing required to meet the durability requirements of AS 3600 Section 4.

For large differences in concrete strength between the column and footing, additional calculations may be required to determine the effective strength of the concrete at the column/footing interface; this may require additional starter bars and possible confinement. This is particularly important for high-strength columns and walls with combined or coupled footings where the column or wall may be at the edge of a footing, eg at a boundary. With this type of footing the moments due to the offset column or wall are resisted by a combined footing or by a tie or coupling beam back to an adjacent footing. Designers can refer to texts such as *Reynolds Reinforced Concrete Designer's Handbook*[8.6] for the design of combined and coupled footings.

Having established an initial pad footing depth, then the shear and bending at the following critical sections are checked as required and the footing depth, concrete strength or both are adjusted as necessary.

8.2.3 One-way (beam) shear action

The nominal shear stress appropriate to this condition is calculated as for a beam across the critical shear plane located a distance d_o (the footing depth) from the face of the column. The beam shear perimeter extends across the whole width of the footing, and the load carried is that portion of the total load located between this perimeter and the outer edge of the footing Figure 8.3.

Shear is calculated along each axis of the footing using the appropriate value of d_o for each direction although it is conservative to use the lesser value of d_o. The value of β_1 used in calculating V_{uc} in AS 3600 Clause 8.2.7.1 will generally be about 0.8, assuming no shear reinforcement is provided. This means that the shear strength, ϕV_{uc}, will be up to about 30% less than when using the previous edition of AS 3600 and it may be a critical design case especially for higher bearing pressures.

8.2.4 Two-way or punching shear action

Nominal punching shear stress is determined around the critical perimeter at a distance of $d_{om}/2$ from the column face, where d_{om} is the mean value of d_o around the column Figure 8.4. The total shear force to be resisted by the punching-shear perimeter is the total footing reaction minus the load on the zone inside the punching-shear perimeter. At higher bearing pressures, it also may be a critical design case.

8.2.5 Flexural action

The design of a footing is similar to that of a cantilevered member. The critical perimeters for bending moment are located, along each axis, at the column face for concrete columns or wall face

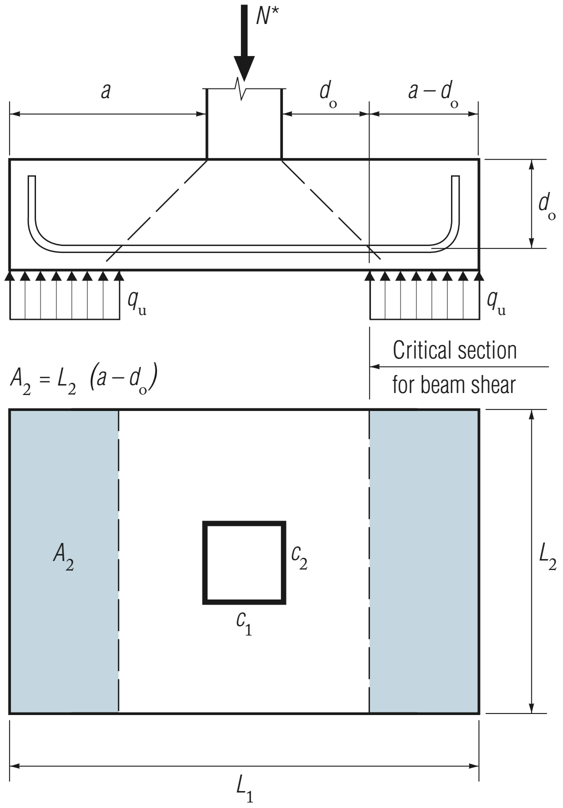


Figure 8.3 One-way (beam) shear action of spread footing

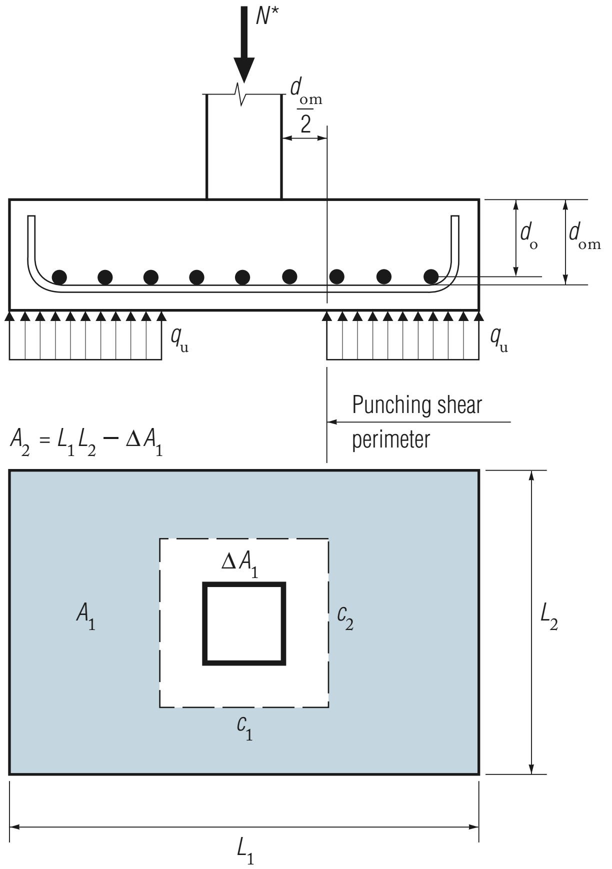


Figure 8.4 Two-way or punching shear action of spread footing

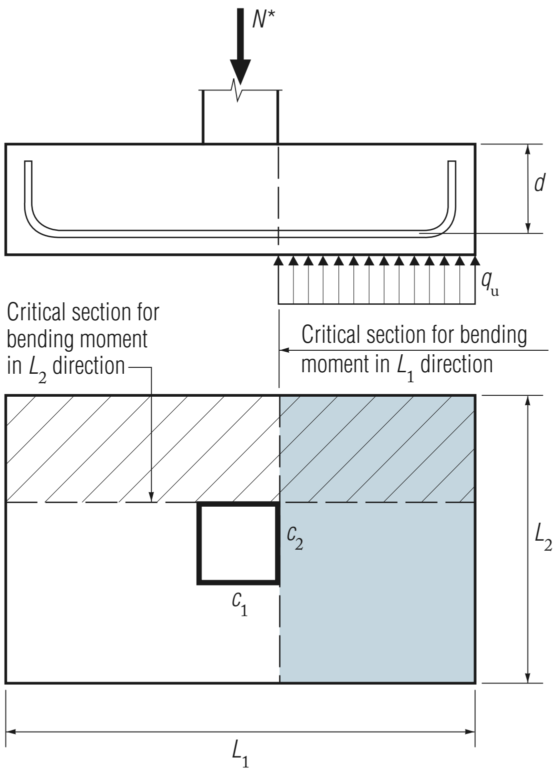


Figure 8.5 Flexural action of spread footing for concrete members

for walls. The bending moment is calculated as the cantilever moment carrying the design pressure over the full width of the footing Figure 8.5. For simplicity, the smaller value of d_o is sometimes used, as minimum reinforcement can often control. Generally, pad footings are only lightly reinforced.

An appropriate reference should be consulted for the position of critical sections for steel columns. It can, however, conservatively be assumed to be at the edge of the column or, sometimes, halfway between the column face and the edge of the base plate if it is unstiffened.

Designers should note that if straight bars are used in flexural action in footings, the development length of the bar must be checked at the critical section for the bending moment at the face of the column (see Table 2.7). In addition, the minimum anchorage may be needed to develop the stress in the bar at the outer edge of the footing, as the moment increases. A cog will usually meet that requirement. When using larger diameter bars in the footing, designers should also check the stress at the location where the cog starts; a hook may be required.

8.2.6 Basis of Spreadsheet 8.1

Spreadsheet 8.1 can be used to calculate the reinforcement requirements for a concentrically loaded reinforced rectangular pad footing in flexure and shear in accordance with the Flowchart 8.1. It uses a reduced bearing pressure (which is less than the total allowable bearing pressure) for the design of the concrete based on the actual bearing pressure at the foundation less the weight of the footing. While it checks the minimum reinforcement required, it does not check cover, spacing requirements or detailing requirements. It is limited to footings with a concrete strength of up to 50 MPa.

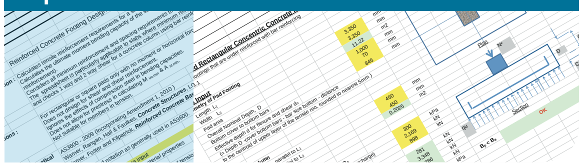
The spreadsheet assumes the footing is not over reinforced and that $k_u \leq k_{uo}$.

For an under reinforced section, the ultimate moment capacity, M_u,is approximately equal to 0.85 $A_{st}\, f_{sy}\, d$ within about 10% of a more accurate calculation, ie it is independent of the concrete strength and the width of the footing. This approximation is used to estimate the reinforcing steel required.

The spreadsheet checks the actual bearing capacity against the allowable bearing capacity. If it exceeds the allowable bearing capacity, the plan dimensions of the footing will need to be increased. It also makes an initial approximation of the reinforcement required as well as the minimum amount required. The designer then selects and inputs an actual area of reinforcement (provided by a number of bars, usually of one size in both directions) to satisfy the larger of these two amounts.

Then the spreadsheet calculates the actual moment capacity and compares it with the design moment, which has been input. It also checks the one-way and two-way shear. If any of these is less than the required or allowable values, then a further iteration will be required. This iteration can involve larger areas of flexural reinforcement, higher concrete strengths or a deeper footing, or a combination of these.

Spreadsheet 8.1 is available at www.ccaa.com.au



8.2.7 Basis of Charts 8.1 and 8.2

After calculating the flexural reinforcement, designers must check the minimum tensile reinforcement required in accordance with AS 3600 Clause 8.1.6.1. Charts 8.1and 8.2 show these minimum reinforcement requirements assuming 50 mm and 75 mm cover respectively with two layers of N20 bars and various concrete grades. Pad footings are not usually heavily reinforced, so minimum reinforcement may be the critical tensile reinforcement, especially at low bearing pressures.

It is common practice to use N12 bars in footings up to 1000 mm wide, N16 bars for those up to 2000 mm wide, N20 bars for those up to, say, 4 m wide and N24 bars for those more than 4 m wide. It has been shown in practice that these sizes provide bars at reasonable spacing. Spacing should not exceed about 300 mm. Mesh reinforcement is not normally used for pad footings, as the minimum reinforcement would usually require at least two layers of mesh, unless the footing is very small or has been designed as unreinforced concrete.

Refer to AS 3600 Section 4 and Chapter 3 of this Handbook for minimum concrete cover requirements.

8.3 STRIP FOOTINGS

The determination of force resultants in strip footings generally falls into two cases:

- Rigid footings
- Flexible footings.

Strip footings are used to support line loads, which are applied to the footings from a continuous wall or from closely spaced columns. Stiff footings, or strips supporting stiff superstructure elements that will not allow significant differential settlement to occur along the line of the footing, may be designed using rigid body theory with linear distribution of soil pressure. An assessment must be made of the relative stiffness of the combined foundation and superstructure system (including the soil). This will determine the type of design method to be used. Designers should consult relevant texts, or the method proposed by Meyerhof[8.7] may be appropriate.

The strength design of a strip footing will follow the same principles as for the spread footing, after the determination of the design forces. Strip footings carrying a stiff longitudinal load, eg a concrete wall, may require only a design check for transverse bending and shear forces.

The designer must ensure that the design of the elements containing stiff and strong superstructure components does not result in specific forces that cannot be tolerated by the supporting strip footing. An example of this would be a shear wall with moments parallel to the wall resulting in tension or high bearing pressures under the footing.

8.4 PILE CAPS

Soils that provide insufficient strength to economically support spread or strip footings often require the installation of piles (or piers) to transfer the applied loads to a stiffer stronger stratum, or by skin friction over the length of the pile, or a combination of both. There are many types of piles and the selection of the type of pile will depend on experience, availability of piling contractors, ground conditions and other factors. When two or more piles support a column, then a pile cap is normally required to transfer the load from the column to the piles. As for all footings, specialist geotechnical advice should be sought on choice of pile, pile design and pile testing.

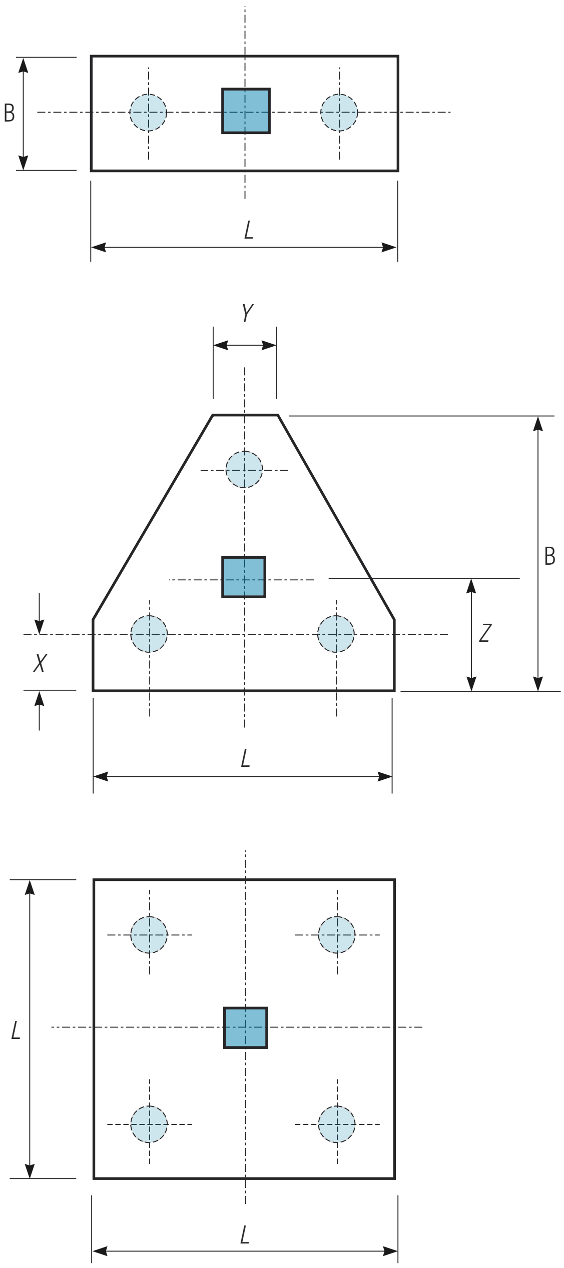


Figure 8.6 Some simple pile cap layouts

The action of simple small pile caps can be treated in a similar manner to a beam or slab as described in AS 3600 Clause 8.1. A pile cap transfers the column loads to discrete points on the underside of the footing, instead of the continuous contact area of the footing supported directly by the soil. Figure 8.6.

The size of the pile cap (footing) is determined by the number of piles required to carry the vertical and horizontal load and moments with the minimum

FLOWCHART 8.1 Design of spread footings

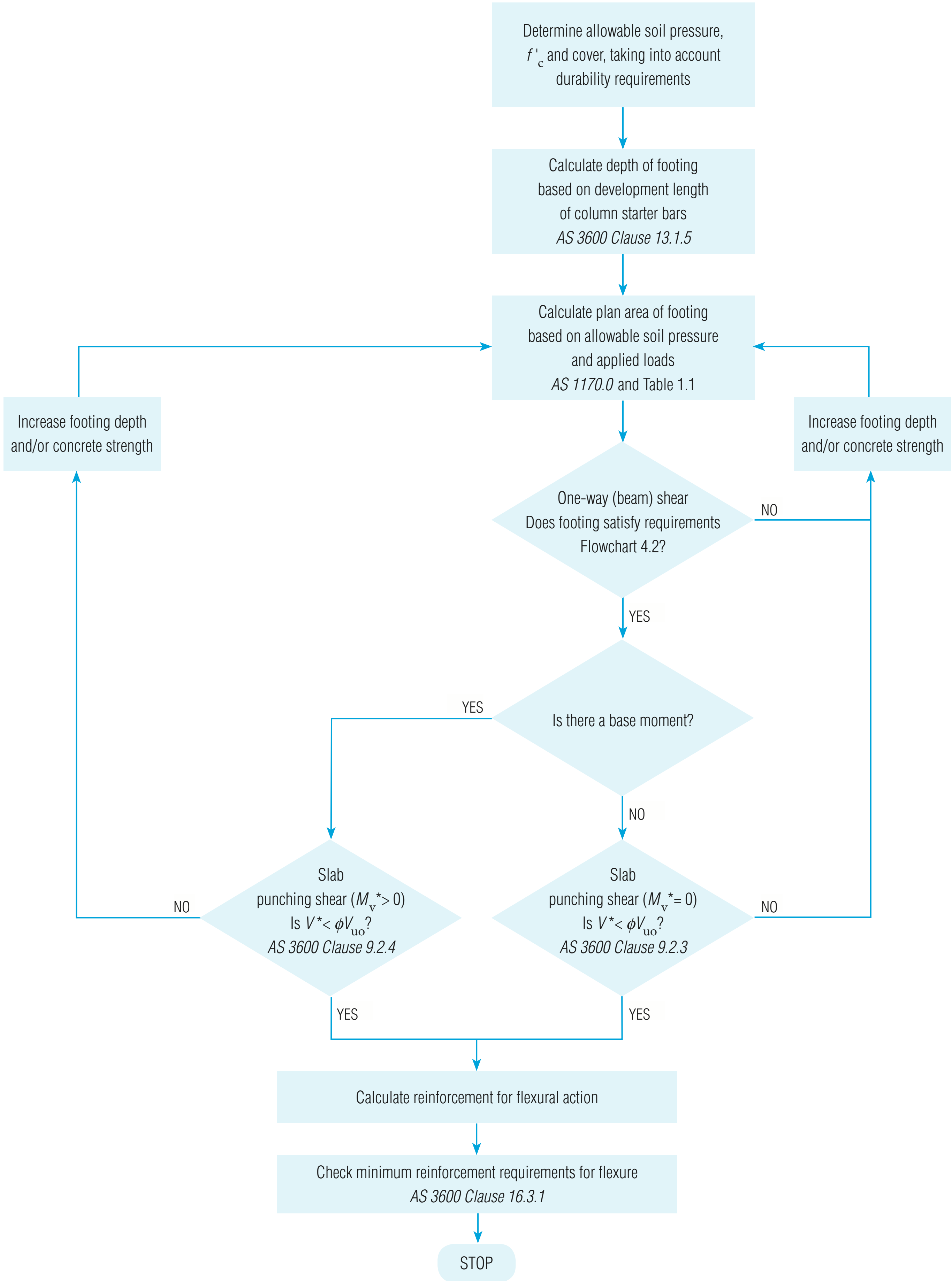


CHART 8.1 Minimum reinforcement in pad footings with 50 mm cover

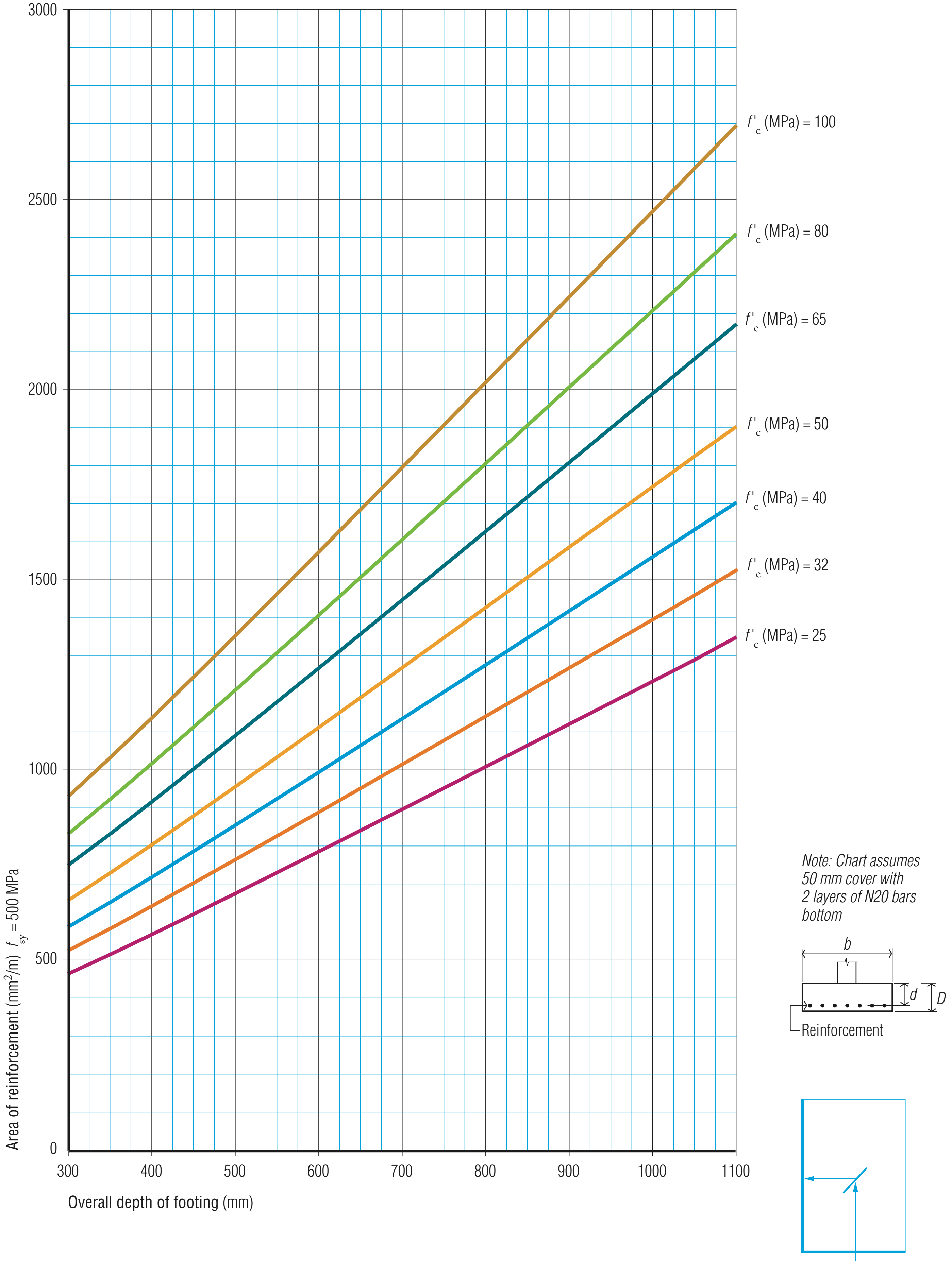
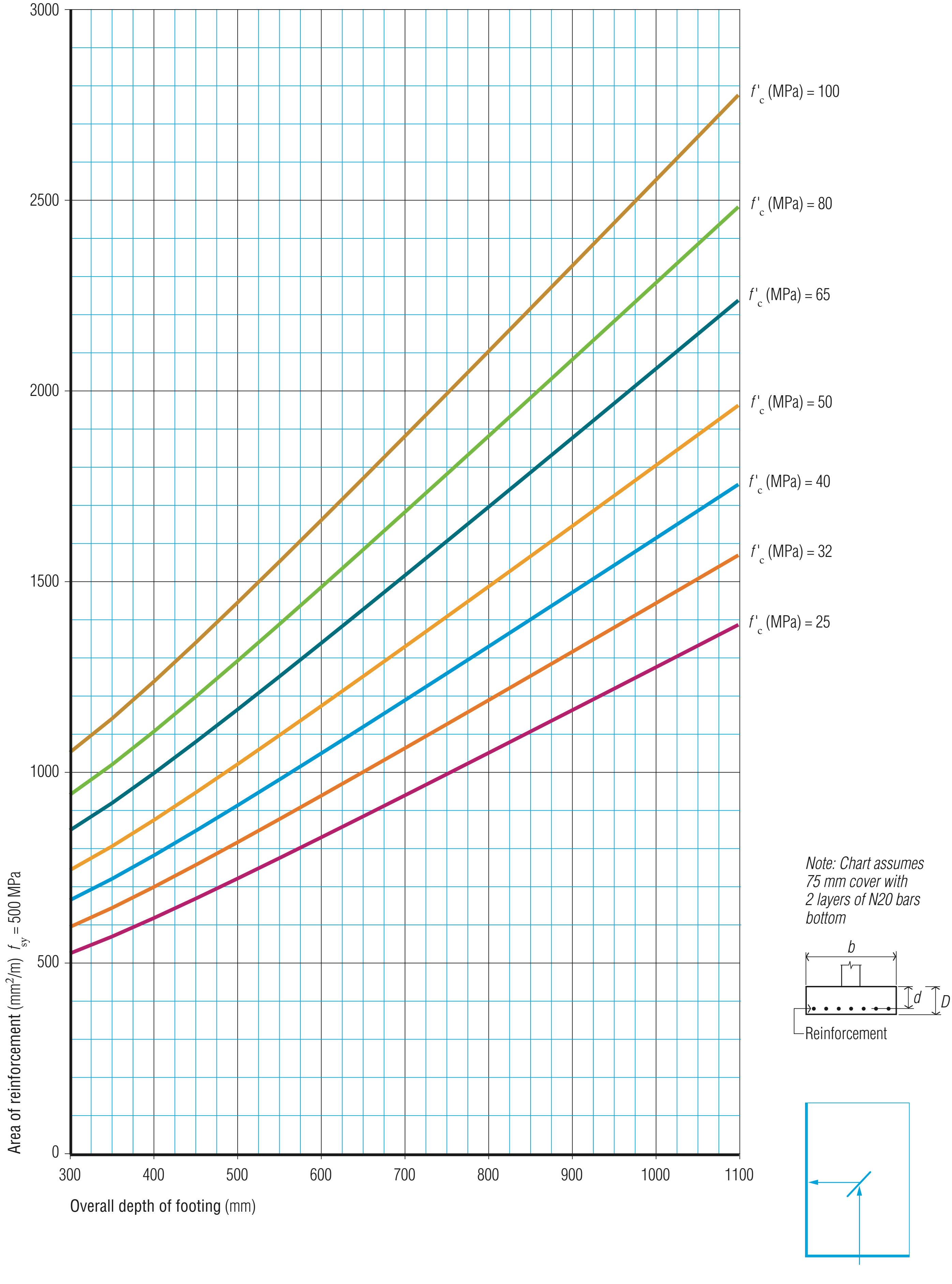


CHART 8.2 Minimum reinforcement in pad footings with 75 mm cover



practical spacing between the piles. This depends on driving conditions, the driving tolerances and load performance of the pile as specified by the designer or supplier. Typically, the pile spacing is 2½–3 times the pile diameter, but the spacing must be confirmed prior to design. It should be noted that the settlement of a group of piles is greater than a single pile for a similar axial load. Settlement generally increases as the pile spacing decreases. When a single large diameter pile is used, a pile cap may not be required.

Design of piles for buildings, including load testing is covered by AS 2159[8.8]. Designers can also refer to texts such as *Pile Design and Construction Practice*[8.9] for the design of piles and pile caps.

Piles are typically cast to about 75–150 mm above the bottom of the pile cap and then cut back to about 25 mm above the bottom of the pile cap with the reinforcement projecting into the pile cap as required.

The initial pile cap depth, subject to final design, can be taken as the horizontal distance from the centre of the column to the centre of the outermost pile. Pile caps should extend at least 150 mm beyond the face of any pile. The pile cap depth will need to be checked against any required development lengths for wall or column starter bars and for shear.

Bending moments and shear (punching and beam) forces are determined as above except for the following:

- Any pile located $d_o/2$ or less from the face of the column is not considered to be adding external shear forces to the cap.
- The pile cap must be checked for the punching shear applied by the piles from the underside of the pile cap as well as the punching shear applied from the top of the pile cap by the column.

Pile caps can also be designed by the strut-and-tie method as set out in Chapter 9 *Strut-and-tie modelling*.

A minimum cover of 75–100 mm should generally be used for piles and the bottom and sides of pile caps for practical reasons, unless larger covers are required, eg by AS 3600 for durability reasons, as discussed in Chapter 3 *Durability and fire resistance*.

Flexural reinforcement is provided in the top and bottom of the pile cap and the bars usually have cogged ends. Generally, the bars are in two orthogonal directions for rectangular pile caps, although for a pile cap for two piles, fitments are usually used in the short direction. For non-rectangular pile caps there can be three or more layers of flexural reinforcement, top and bottom. Horizontal reinforcement should also be provided to the vertical faces of the pile cap at about 200-mm centres vertically and lapped as required.

For very deep and/or large pile caps, consideration may need to be given to heat of hydration of the concrete and the control of thermal cracking, especially if the pile cap is over, say, 1200 mm deep.

8.5 DETAILING

For all reinforced concrete footings, details of the footings and any special detailing must be shown on the drawings.

AS 3600 Clause 9.1.3.1 sets out the detailing of flexural reinforcement for slabs which is used for footings. This requires that where the bending moment envelope has been calculated, the termination and anchorage of flexural reinforcement is based on a hypothetical bending-moment diagram formed by displacing the calculated positive and negative bending-moment envelopes a distance D along the footing from each side of the relevant sections of maximum moment and to which the development length must be added. This development length in pad footings is usually achieved with cogs (or hooks) to the footing bars as shown in Figure 8.2 and discussed above. Designers should also refer to Chapter 11 of the *Reinforcement Detailing Handbook*[8.10] for further guidance.

Areas needing consideration in the documentation of footings include:

- A plan of the footing (including pile caps) should be shown on the footing drawings, which may be combined with other schedules.
- The footing sizes (including pile caps) should be shown on the drawings, which are sometimes combined with the column schedule. It is usual to size pad footings and pile caps in plan dimensions in 100-mm increments and in depth in 50-mm increments, eg 2.0 m x 600 mm deep, or 2.1 m x 3.4 m x 850 mm deep, etc.
- Footing beams should be shown on a plan or a beam schedule or similar.
- Details of footings and pile caps should include (as a minimum) the founding level or stratum, the size of the footing or pile cap and the RL to the bottom or top of the footing/pile cap, the reinforcement including starter bars, the concrete strength, the cover and any construction joints. The details should also show if the columns are concentric to the footing or pile cap and if not, offsets should be dimensioned.
- It is suggested that the minimum concrete strength for reinforced pad and strip footings be 25 MPa unless higher strengths are required by AS 3600, eg for durability reasons as discussed in Chapter 3.
- The allowable covers in AS 3600 do not often reflect the actual conditions on site when

excavating in ground, which can be variable. It is recommended that the minimum cover to reinforcement to the bottom of pad and strip footings be 50 mm when a layer of blinding concrete is used or where the ground is firm, unless larger covers are required by AS 3600, eg for durability reasons as discussed in Chapter 3. Where the ground conditions are difficult, 75 mm or greater cover may be appropriate.

- AS 3600 Clause 4.10.3.5 specifies that where footings are cast against ground the cover is to be increased by 10 mm when a damp proof membrane (DPM) is provided, otherwise by 20 mm. It is not usual to provide a DPM to pad footings and generally not possible to pile caps.
- Top reinforcement is generally not needed in pad footings unless the footing has to carry uplift forces (such as in cyclonic conditions), or if it is part of a combined or tied footing system, or is a pile cap.
- Footing beams that intersect with footings may have different effective depths because of the bars being in layers.
- The same size bar in flexure should be used in both directions in square pad footings and each direction in rectangular footings, to avoid fixing errors on site.
- Side face reinforcement is not normally provided for footings but is usually provided in pile caps or deep strip footings.
- Footing beams and strip footings will usually have top and bottom reinforcement with fitments.
- Fitments in footing beams and strip footings may have to be designed for shear along the member. For footings, the fitments may also act as the flexural reinforcement perpendicular to the span of the footing for the outstand of the footing beyond the wall.
- Reinforcement at the junction of strip footings, footing beams and spread footings with columns should be reviewed to ensure that it all fits.
- If footing beams and strip beams are shown on a schedule on the drawings, the schedule should be checked to ensure that all details fit within the constructed shape.
- Complicated or unusual footings, footing beams and pile caps should be shown in plan and elevation.
- Construction joints in strip footings and footing beams should be properly considered and detailed.
- If earthquake design is a consideration, compliance with AS 1170.4[8.11] and AS 3600 Appendix C should be checked.

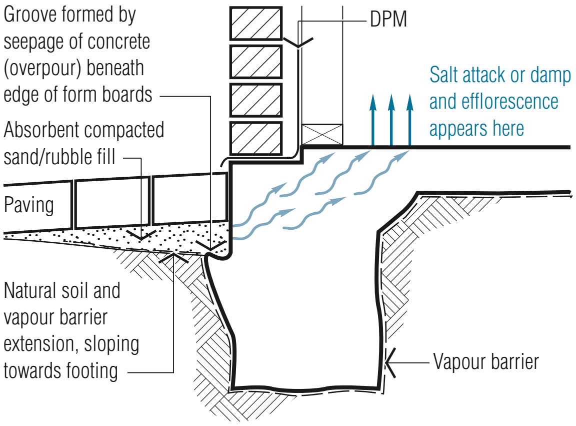


Figure 8.7 Slab edge dampness

8.6 GENERAL GUIDANCE

On any particular project, other matters that may need consideration include:

- Advice in the geotechnical investigation whether the footing will be likely to be cast in aggressive soils as defined in AS 3600 Clause 4.8.
- Tie beams or similar in both horizontal directions may be needed for earthquake design for pad footings or pile caps that are located in or on soils with a maximum vertical ultimate bearing capacity of less than 250 kPa. This is to limit differential horizontal movement during an earthquake as required by AS 1170.4 Clause 5.2.
- Will blinding concrete be required to the base of the footing or pile cap?
- Specify who is to confirm the bearing stratum and bearing pressure (assumed for design) in the excavated ground, before placing the blinding concrete or fixing of the reinforcement.
- Will the ground conditions allow vertical sides of excavations to stand for sufficient time to permit placement of reinforcement and concrete without danger to those carrying out the work? If not, batters may have to be provided and the vertical faces of the footings formed and the excavation later backfilled. This may require advice from the geotechnical engineer.
- Generally, excavations deeper than 1.5 m will require shoring or battering of the excavated sides, in accordance with the various state and territory Worksafe safety regulations.
- For strip footings and footings associated with a slab-on-ground with a habitable area above, a 200-µm DPM must be provided under the entire slab and footing and be returned up the edges of the slab/footing beam.

- For strip footings in non-habitable areas consideration should be given to providing a 200-µm DPM under the footing to assist in preventing any rising damp.
- For footings and footing beams, projecting concrete outside the theoretical formed face due to poor construction procedures can allow slab edge dampness to occur as shown in Figure 8.7. Slab edge dampness can be a problem in arid and saline type soils and designers should seek specific advice from the geotechnical engineer on this matter as required. Refer to CCAA Data Sheet[8.12] and CPN 30[8.13] for more information.
- Footing faces and footing beams on property boundaries should be formed or constructed so that the concrete does not encroach on adjoining properties.
- Excavations of footings and pile caps must not undermine adjacent footings or structure, especially on adjacent sites.
- Provide any rebates or set downs for walls over as required.

REFERENCES

8.1 Craig R F *Craig's Soil Mechanics* 7th Ed, Spon Press, 2004.

8.2 Curtin WG, Shaw G, Parkinson G and Golding J (revised by Seward NJ) *Structural Foundation Designers' Manual*, 2nd Ed, Blackwell Publishing, 2006.

8.3 AS 2870 *Residential slabs and footings – Construction* Standards Australia, 2011.

8.4 *Building Code Requirements for Structural Concrete* (ACI 318-08) ACI Manual of Concrete Practice, 2010.

8.5 Beletich AS and Uno PJ *Design Handbook for Reinforced Concrete Elements*, 2nd Ed, UNSW Press, 2003.

8.6 Reynolds CE, Steedman JC and Threlfall AJ *Reynolds Reinforced Concrete Designer's Handbook*, 11th Ed, 2008.

8.7 Meyerhof GG 'Some recent foundation research and its application to design' *The Structural Engineer* (London) Vol. 31, No. 6, June 1953, pp 151–167.

8.8 AS 2159 *Piling – Design and installation* Standards Australia, 2009.

8.9 Tomlinson M and Woodward J *Pile Design and Construction Practice*, 5th Ed, Taylor and Francis, 2008.

8.10 *Reinforcement Detailing Handbook* (Z06), 2nd Ed, Concrete Institute of Australia (CIA), 2010.

8.11 AS 1170.4 *Minimum design loads on structures* Part 4: *Earthquake loads* Standards Australia, 2007.

8.12 *Slab Edge Dampness and Moisture Ingress* Data Sheet, Cement Concrete & Aggregates Australia, 2005.

8.13 *Slab Edge Dampness* (CPN 30) Concrete Institute of Australia, 1998.

Chapter 9_Strut-and-tie modelling

9.1 INTRODUCTION

Strut-and-tie modelling is covered in Section 7 of AS 3600[9.1], with formal design rules for three types of models. This section should be read in conjunction with Section 12 which also covers the design of non-flexural members, end zones, concrete nibs, corbels, stepped joints and bearing surfaces. AS 3600 Clause 2.2.4 sets out the strength check to be used when using strut-and-tie analysis.

Strut-and-tie modelling has been around for many years, has attracted quite a deal of research and is now an established design method allowed by most concrete codes, including the ACI[9.2], since 2002. Chapter 7 of the *Precast Concrete Handbook*[9.3] has a section on strut-and-tie design based on the ACI which is similar to AS 3600, with two worked examples.

This design method is used for non-flexural members such as deep beams, pile caps, corbels and nibs and the like. It can also be used for non-flexural portions of other structural members such as dapped ends of beams, holes in beams, etc. The strut-and-tie approach is based on the designer selecting the load paths and designing according to the chosen design model. This requires the designer to choose realistic load paths within a member in the form of an idealised or notional truss. See Figure 9.1 for a typical example of a simple strut-and-tie model (STM).

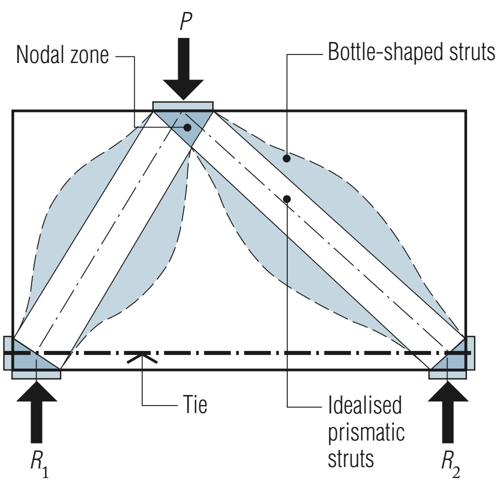


Figure 9.1 A simple strut-and-tie model

Non-flexural members are defined as members where the ratio of the clear span or projection to the overall depth is small. The ratios must not exceed those set out in AS 3600 Clause 12.1.1:

- Cantilevers 1.5
- Simply-supported members 3
- Continuous members 4.

The geometry of the notional truss is determined by following the flow of forces from the support reaction into the body of the supported element to the points of the applied load(s). The intersection of compressive struts with tension ties or support reactions defines the nodal zones. The axes of the struts and ties are chosen to coincide approximately with the axes of the compression and tension fields in the real element. Struts should be generally parallel to the axes of cracking. The struts, ties, and nodal zones making up the STM all have finite dimensions that must be calculated and must be taken into account in selecting the dimensions of the truss and the member.

Some of the key assumptions for strut-and-tie analysis are:

- Ties must yield before failure of the struts for ductility.
- Forces in the ties and struts are axial only.
- Tension in the concrete is ignored.
- External forces are applied at nodes.
- The ties are fully developed before the node, which requires anchorage outside the node.

Traditionally, STMs are developed using the elastic stress distribution and load path methods. These methods involve a trial-and-error process based on the designer's intuition and experience. The STM obtained by such methods is not unique and can vary with the designer's understanding of this method of analysis.

While the method is a useful design tool, to achieve the assumed load path, significant redistribution of the internal forces may be required which the structure may not be capable of accommodating due to limitations of the concrete and steel. Care is therefore needed to ensure that the member is capable of accommodating the redistributions required without undue distress to itself or adjacent components. As a general rule, designers should choose an STM that allows the structure to behave as they would expect it to behave and then to design as close to possible to that model, to minimise the demands on the structure. The strut-and-tie method is therefore in many cases a design tool for the experienced engineer and is not a simple standard analysis procedure. Even where the design of an STM is simple, such as for a corbel, the design by inexperienced engineers must be checked by others to ensure they have fully understood the design issues involved because of the usual critical nature of the element. STM design involves a trial-and-error approach with hand calculations and hand-drawn sketches to scale, etc and usually cannot be undertaken totally on the computer.

For further information on the design of STMs refer to Foster et al[9.4], PCA[9.5,9.6] and others[9.7].

9.2 TRUSS GEOMETRY

9.2.1 General

AS 3600 Section 7 requires the following:

(a) *Loads shall be applied at nodes, and the struts and ties shall be subjected only to axial force.*

(b) *The model shall provide load paths to carry the loads and other actions to the supports or into adjacent regions.*

(c) *The model shall be in equilibrium with the applied loads and the reactions.*

(d) *In determining the geometry of the model, the dimensions of the struts, ties, and nodal zones shall be taken into account.*

(e) *Ties shall be permitted to cross struts.*

(f) *Struts shall cross or intersect only at nodes.*

(g) *For reinforced concrete members, at a node point the angle between the axes of any strut and any tie shall be not less than 30°.*

Once the geometry of the truss is chosen, the forces in the struts and ties are determined by statics, with the applied loads and the reactions being in equilibrium. For equilibrium, at least three forces should act on a node. Nodes are classified according to the signs of these forces as C-C-C (all compression, ie only struts entering the node), C-C-T (when there are two or more compression struts and a tension tie) and C-T-T (when there are two or more tension ties entering the node with a compression strut). See Figure 9.2.

9.2.2 B- and D-regions

The B-regions (B for Bernoulli or Beam) are flexural areas within a member where the assumption that plane sections remain plane can be applied.

The D-regions (D for Disturbance) are the areas of discontinuity. For design purposes, D-regions can be idealised as an STM. See Figure 9.3.

9.2.3 Nodes and nodal zones

A node is a point in an STM where the axes of the struts, ties, and any applied concentrated forces, if applicable, acting on the joint, all intersect. Around this area is the nodal zone, which is the volume of concrete surrounding the node, and which is the element that transfers the compression and tension through the node.

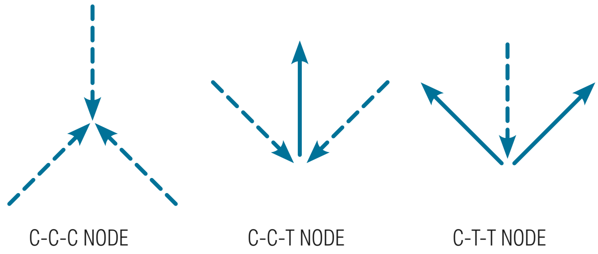


Figure 9.2 Various nodes (dashed line indicates compression struts and full lines indicate ties)

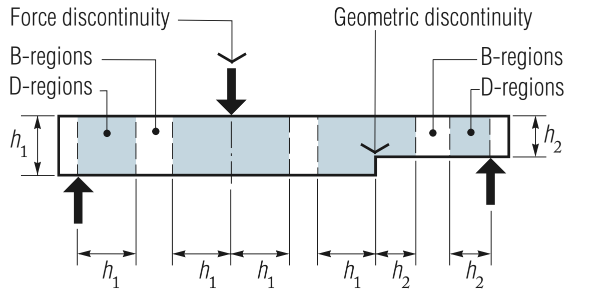


Figure 9.3 Examples of B- and D-regions in a horizontal member

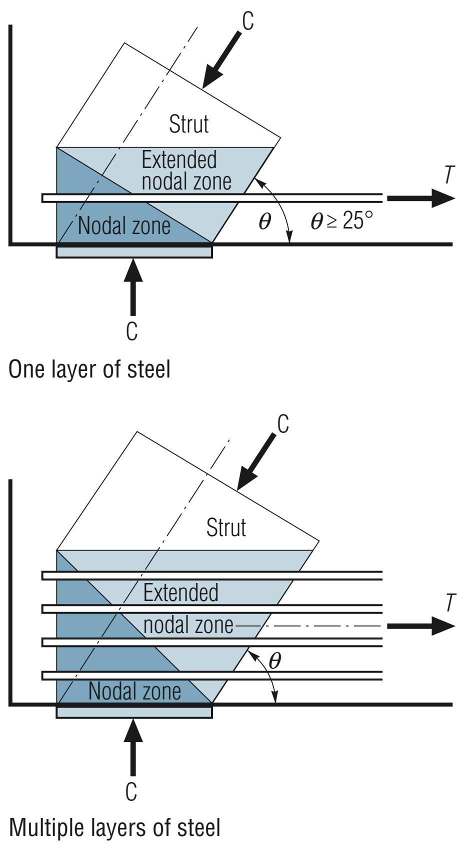


Figure 9.4 Extended nodal zone

The greater the number of truss members meeting at a node, the less efficient the nodal zone is and this is recognised by the β_n factor which varies from 1.0 to 0.6 as set out in AS 3600 Clause 7.4.2. The principal compressive force on any nodal face must not be greater than $\phi_{st}\beta_n 0.9 f'_c$.

A nodal zone is called a 'hydrostatic' node when its loaded faces are perpendicular to the axis of the struts and ties acting on it, and the loaded faces have equal stresses. The nodal zone often needs to be extended because of the number of ties required. It is then sometimes referred to as an extended nodal zone Figure 9.4. In a hydrostatic C-C-C nodal zone, the ratios of the lengths of the sides of the node are in the same proportion as the forces acting on it.

A C-C-T nodal zone at the end of a member can be represented as a hydrostatic node if the tie is assumed to extend through the node and is anchored by a 'notional plate' on the far side of the node as shown in Figure 9.4. The notional plate has bearing stresses that are equal to the stresses in the incoming struts. If the tie is to be anchored beyond the nodal zone, then at least 50% of the anchorage must be made beyond the zone. The use of threaded bar may be appropriate for such ties.

Nodes can also be non-hydrostatic as shown in Figure 9.5 but the modelling becomes more complex. Foster et al[9.4] provide further information on this type of node.

9.2.4 Struts

Struts are normally concrete members idealised as either prismatic or uniformly tapered although they can be fan-shaped as shown in Figure 9.6. They can also be thicker at mid-length where the compressed concrete can spread laterally into the adjacent concrete to form a bottle-shaped strut. Prismatic struts can be used only when the stress field cannot diverge.

Without proper transverse reinforcement, a bottle-shaped strut cannot maintain equilibrium after significant cracking. Reinforcement is therefore needed, usually in two orthogonal directions, to maintain the strength of a bottle-shaped strut as well as reduce crack widths under service load.

Longitudinal reinforcement may be used, located within the strut to increase its strength. Such reinforcement should be parallel to the axis of the strut and enclosed by ties satisfying AS 3600 Clause 10.7. The longitudinal reinforcement should be properly anchored beyond the nodal zone. The strength of a longitudinally reinforced strut may be calculated as for a prismatic, pin-ended short column of similar geometry.

9.2.5 Ties

A tie usually consists of either bar reinforcement (sometimes threaded), a prestressing bar or strand. It is usual to include a small portion of the surrounding concrete that is concentric with the axis of the tie to define the zone in which the forces in the struts and ties are to be anchored. However, the concrete in the tie is not used to resist the axial tension in the tie.

The effective thickness in elevation of a tie for design can vary with the distribution of reinforcement in it. If the bars are in one layer, the effective thickness can be taken as the diameter of the bars in the tie plus twice the cover to the surface of the bars. Multiple-layers of reinforcement should be distributed approximately uniformly over the thickness and width of the tie.

The reinforcement in ties can be anchored by hooks or cogs, or straight bar development beyond the node, or by anchorage plates. Statics must be satisfied at each node. At least 50% of the development length must extend beyond the nodal zone as set out in AS 3600 Clause 7.3.3. Mechanical anchorage, including welding, can also be used outside the node.

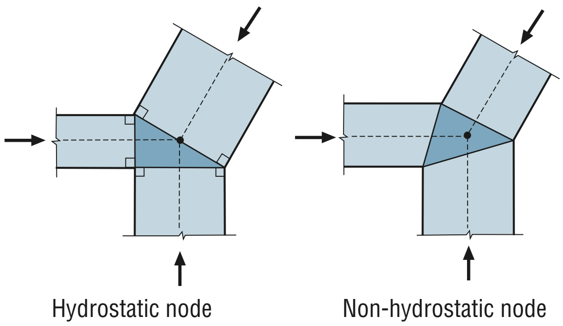


Figure 9.5 Hydrostatic and non-hydrostatic nodes *(after Foster et al)*

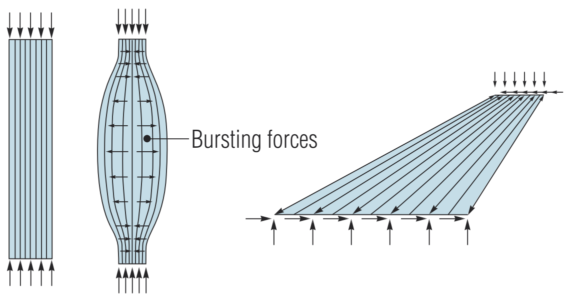


Figure 9.6 Prismatic, bottle and fan-shaped struts *(after AS 3600)*

9.3 ANALYSIS OF STRUT-AND-TIE MODELS

AS 3600 requires that the analysis of an STM to determine the internal forces in the struts and ties, has to meet the requirements of Clauses 6.1.1 and 6.1.2 and also compliance with Clause 6.8. This may require several STMs to be considered and analysed to compare results and from which a chosen model is then adopted and fully analysed and detailed.

AS 3600 Clause 2.2.4 stipulates the following procedure for the strength check for strut-and-tie analysis:

(a) The strut-and-tie model shall satisfy the requirements of Section 7 of AS 3600.

(b) The forces acting on all struts and ties and nodes shall be determined for the critical combination of factored actions as specified in AS/NZS 1170.0 and Clause 2.4 by an analysis of the strut-and-tie model in accordance with Section 7.

(c) The compressive force in any concrete strut shall not exceed the design strength of that strut determined in accordance with Clause 7.2.3. The strength reduction factor (ϕ_{st}) to be used in determining the design strength shall be in accordance with Table 2.2.4.

(d) The tensile force in any tie shall not exceed the design strength of the tie determined in accordance with Clause 7.3.2 where the strength reduction factor (ϕ_{st}) is given in Table 2.2.4.

(e) The reinforcement and/or tendons in the ties shall be anchored in accordance with Clause 7.3.3.

(f) The design strength of nodes shall be calculated in accordance with Clause 7.4.2 and shall not be exceeded. The strength reduction factor (ϕ_{st}) shall be in accordance with Table 2.2.4.

The strength reduction factor, ϕ_{st}, for concrete in compression is taken as 0.6 and for steel in tension is taken as 0.8, reflecting that the tie should yield first.

AS 3600 Section 12 defines three types of design models for non-flexural members. Type I is where the load is carried to the supports directly by major struts and ties. It will cover many simple STMs. Type II is when the load is taken to the supports by a combination of primary (major) and secondary (minor) struts. Hanger reinforcement is required to return the vertical components of forces developed in the secondary struts to the top of the member. Type III is similar to a conventional truss, where the load is carried to the supports via a series of minor struts with hanger reinforcement used to return the vertical components of the strut forces to the top of the member. The use of Type II and III models will require careful consideration by the designer. These are discussed in more detail in the paper by Foster and Gilbert[9.8].

For the best results, it is recommended that a preliminary design be carried out before finalising the design of the chosen strut-and-tie model. The design of a strut-and-tie member will therefore involve the following steps:

1. Check that the member or section of member to be designed complies with the requirements of AS 3600 and define which areas of the member are areas of discontinuity, ie D regions, and which areas are flexural areas, ie B regions.
2. Determine the external actions and where the member is a mixture of B and D regions, determine the boundary actions, including any concentrated and distributed actions such as moments, shear and axial actions that will act on the STM being designed.
3. Sketch to scale one or more suitable strut-and-tie models to suit the member being designed, including any boundary forces and choose the STM that will best reflect the internal actions and will not cause significant redistribution of internal actions.
4. Analyse the chosen STM to obtain the actions in each of the individual strut-and-tie members of the model.
5. Check the dimensions of the struts, ties and nodes required, including the necessary concrete strength and amend the STM as required.
6. Choose the material for the tie member which is often reinforcement and ensure that the tie capacity and the anchorages beyond the nodes are adequate. If insufficient, then amend the STM as required.
7. Design the struts, including any compression reinforcement, fitments and bursting reinforcement.
8. Complete the design and drafting, including the detailing as set out below.

Figures 9.7 to 9.10 illustrate some examples of STMs (including Type I and III models).

9.4 DETAILING

All strut-and-tie element details must be shown on the drawings. This is a very important aspect for this design procedure. Designers can refer to Chapter 16 of the *Reinforcement Detailing Handbook*[9.9] for typical detailing of nibs and corbels.

Designers should note that:

- Sufficient details including plans, elevations with large scale sections and details of the strut-and-tie elements should be provided.
- Struts must be properly detailed and with transverse reinforcement to the member in two orthogonal directions as required, so that splitting cannot occur.

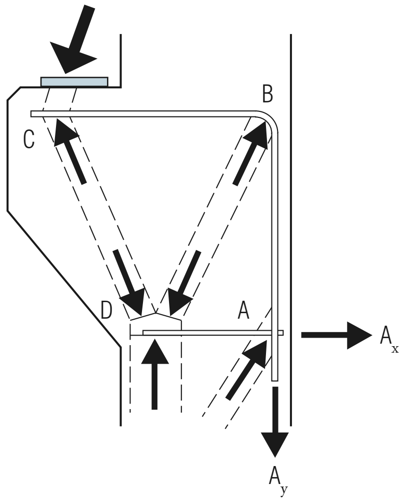


Figure 9.7 An STM for a corbel on a column (Type I)

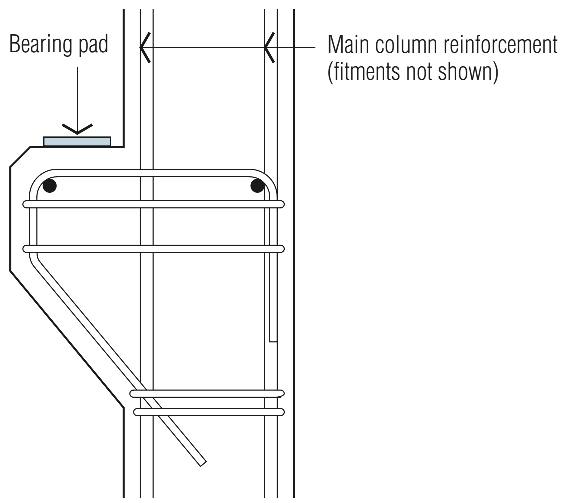


Figure 9.8 An STM for a corbel on a column showing the proposed reinforcement

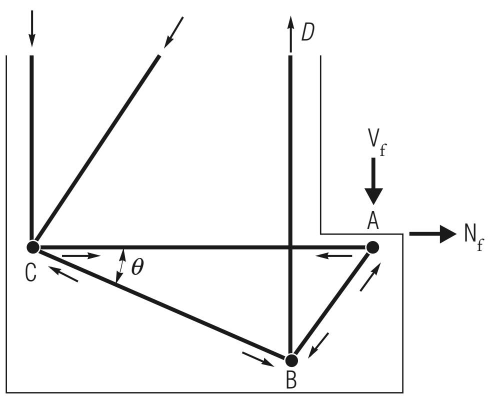


Figure 9.9 STM for corbel on side of beam

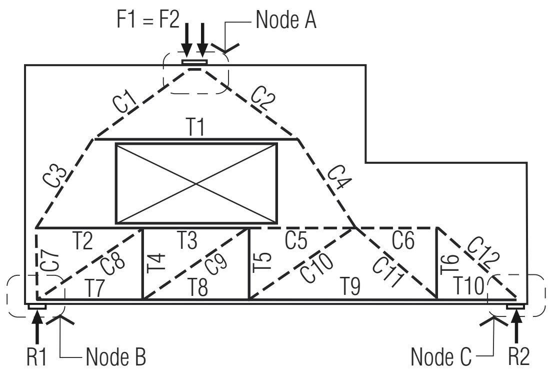


Figure 9.10 A truss model for a deep beam with an opening (Type I and III)

Figure 9.11 Double corbels on precast concrete columns

Figure 9.12 A reinforcing cage for a precast concrete column showing the reinforcement for a corbel

- Ties need to be properly anchored beyond the node as set out in AS 3600 Clause 7.3.3. This often requires development with hooks, cogs or anchorage by mechanical anchors such as welding, plates, etc.
- Hooks and cogs have real dimensions and can take up a considerable space in strut-and-tie members. Smaller bars allow tighter cogs and bends and require smaller development lengths.

9.5 GENERAL GUIDANCE

The following will assist in both the design and the inspection of a strut-and-tie member for a particular project.

- Draw the truss model to scale to see it all fits within the overall member dimensions and is logical. The dimensions of STM elements should allow the strut reinforcement (if required), the bursting reinforcement (as required), the tie reinforcement and any other associated reinforcement all to fit together and within the overall dimensions of the member and to develop the tie anchorages. Strut-and-tie elements will generally not fit within thin sections.
- The actual design of the nodes and nodal zones needs to be considered carefully and can involve a considerable amount of hand calculations.
- Always inspect the reinforcement in place prior to concreting to ensure that the design model used and what was detailed on the drawings is what is being constructed, as reinforcement fixing may not be simple and the reinforcement fixers may have changed the detail to facilitate their task.
- With stepped joints, corbels, dapped ends and the like, do not ignore horizontal forces at the joints even if they are not evident, as shrinkage, thermal movement, etc may induce horizontal forces due to friction. AS 3600 Clause 12.4 requires a minimum of 20% of the vertical force to be used as a nominal horizontal design force. The actual horizontal design force may be greater than this minimum and this needs to be assessed for each design case.
- Because of the dimensions of bends to ties and the cover, do not load corbels, nibs, dapped ends and the like, too close to the vertical edge of the concrete as the reinforcement may be inadequate and the edges will spall and may fail. AS 3600 Clause 12.3(a) requires the bearing to be located over the straight portion of the bar where looped (or cogged) bars are used. Always ensure such elements are loaded over the reinforcement and the actual bearing is back from the edge as shown in Figure 9.13.

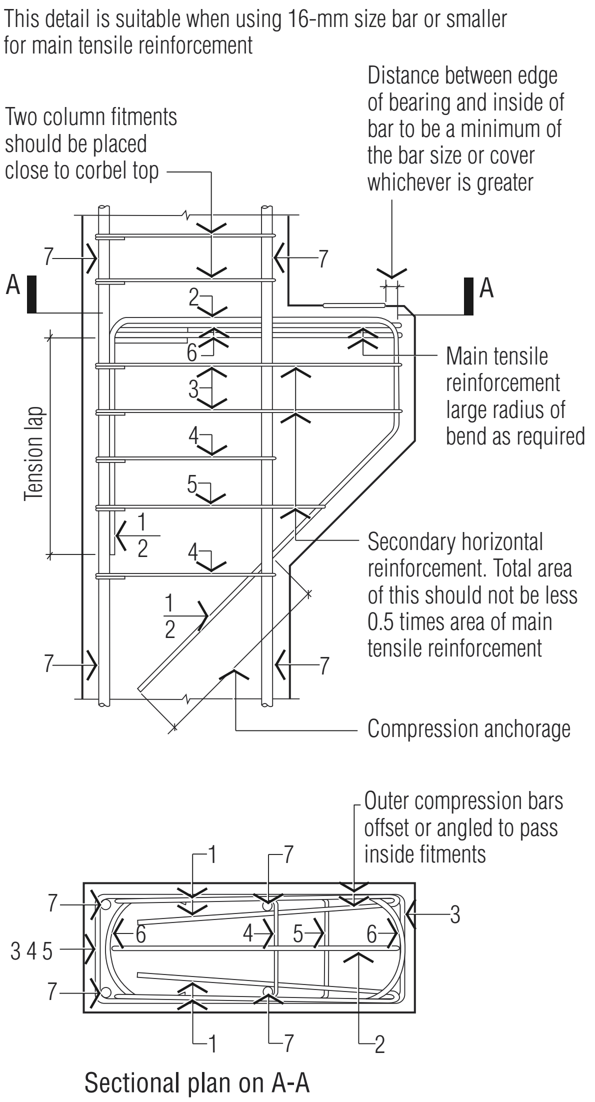


Figure 9.13 Detailing of a corbel to a column (after IStructE *Standard method of detailing structural concrete*)

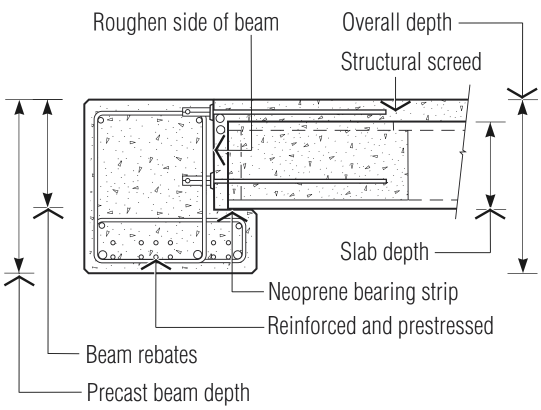


Figure 9.14 Concrete corbel or nib on a precast beam supporting a precast concrete floor

- Always be careful with concrete (and steel) members bearing on the concrete surface in corbels, dapped ends, slip joints and the like, as invariably spalling and cracking will occur if they are not correctly detailed. This can result in both unsightly and sometimes dangerous conditions if the concrete can fall. It is preferable to use a neoprene bearing strip or a slip-type bearing as appropriate, between the two surfaces. As a minimum, provide a chamfer on the corners and set the bearing area back from the edge as discussed above. See Figure 9.14.

REFERENCES

9.1 AS 3600 *Concrete structures,* Standards Australia, 2009.

9.2 *Building Code Requirements for Reinforced Concrete* ACI 318–08, American Concrete Institute, 2010.

9.3 *Precast Concrete Handbook* 2nd Ed, National Precast Concrete Association Australia and Concrete Institute of Australia, 2009.

9.4 Foster SJ, Kilpatrick AE and Warner RF *Reinforced Concrete Basics 2E* 2nd Ed, Pearson, 2010.

9.5 *Strut-and-Tie Model for Structural Concrete Design* Professional Development Series, PCA, October 2007.

9.6 *Notes on ACI318-08 Building Code – Requirements for Concrete* PCA, 2008.

9.7 *National Seminar Series on AS 3600—2009*, *Lecture 2*, Engineers Australia, 2009.

9.8 Foster, SJ and Gilbert, RI 'Strut and Tie Modelling of Non-flexural Members', *Australian Civil Engineering Transactions* Vol. 39, No. 2/3, 1997.

9.9 *Reinforcement Detailing Handbook* (Z06), 2nd Ed, Concrete Institute of Australia, 2010.

BLANK PAGE

Chapter 10 _ Design examples

The following pages provide examples of the design of the various types of reinforced concrete members in a hypothetical structure, demonstrating the use of the relevant preceding chapters using the design charts, Excel spreadsheets and tables presented in this Handbook and in accordance with AS 3600—2009.

DESIGN NOTES ON STRUCTURAL DESIGN AND COMPUTATIONS

The structural solutions presented in this Chapter have been chosen for the purposes of illustrating the analysis, design and reinforcement detailing of concrete members to AS 3600 and in the use of this Handbook. No lateral analysis has been undertaken and only selected members of the structure of the building have been chosen for design. Considerably more computations would be required for the complete design of this project. Where possible these computations are based on AS 3600—2009 with limited use of design aids such as spreadsheets, to illustrate concrete design from first principles. While some basic analysis has been carried out using a 2D analysis program and some of these results used for the design computations following, some input values have been chosen to illustrate specific design issues and cannot be validated by an analysis using the load data provided.

A typical concrete structure for a building may account for only 25% of the project cost, but the design affects the whole building, including the architecture and services. The choice and details of a building's structure should reflect both buildability and overall building cost. The aim in any design is to provide a functional and economical structure, on time and on budget. A functional structure is one that has sufficient strength to carry the applied actions (loads) and has adequate stiffness to limit deflection and vibrations. It must also be durable against corrosion and deterioration and have adequate resistance against fire, must be aesthetic and sustainable and meet the expectation of those involved in the project. A structure must not be over-designed nor must it be under-designed. Unfortunately, when professional fees are limited, design and detailing is often minimised, resulting in an under designed structure with excess material. For a sustainable structure, a comprehensive design of all structural elements can be one of the most cost-effective of all of the sustainable measures adopted for any building.

An economical structure is one that has an optimisation of material and labour costs and contributes to the overall economics of the project. Minimum weight does not always result in minimum cost. A structure that requires a longer time to construct, even though it may result in a lower construction cost, may nevertheless cost more money. Similarly, rationalisation and simplification of reinforcement will normally speed construction, reduce overall construction costs and construction time. However, excessive curtailment and tailoring of reinforcement to save material at the expense of rationalisation can be counter-productive.

Structures, particularly those exposed to view, should be designed and detailed to be attractive and suitably proportioned and, although the appearance of the structure is often determined by the architect, the structural engineer must be aware of aesthetics and can significantly contribute to this area.

The amount of time spent in analysing a structure or member should be related to the value to be gained from the analysis. Rigorous analysis is justified and necessary for a member when the member occurs many times in the structure, as refinement can result in a significant cost saving, whereas the use of simplified methods of design in AS 3600 can be used where the problem is a simple and basic one.

The accuracy of structural behaviour theories and analyses, applied loads and material properties is such that determining actions to a high degree of accuracy is meaningless. For example, action assessment to 1% accuracy is all that is justified, eg 260 kN not 259.67 kN.

When designing structures, avoid being bogged down in numbers, computer output and paper. Mistakes can be avoided by applying simple checks such as replacing a complicated load pattern by equivalent distributed or concentrated loads. Frequently check your computations and design assumptions to avoid duplication of errors. A few minutes spent checking your work on a regular basis can save hours of corrective work on design and drafting later on.

Do not design members in isolation. Recognise that structures and especially reinforced concrete, will want to act structurally how it has been built and reinforced, not necessarily in the manner assumed in the design, including the computer analysis. The structure as a whole may behave differently from individual members. Remember also that when concrete slabs sit on beams or run into beams and beams frame into concrete columns and walls, the reinforcement must pass through these areas and it must all fit together. Aim for a general uniformity of members and details, and do not have a multitude of slab, beam and column sizes, bar sizes, fitment spacing, etc.

The drawings and specifications for a building are the means of communicating the structural design to the builder/contractor, for construction. The drawings and specification usually form part of the Building Contract and are therefore important legal documents.

Computations are a means of verification of structural behaviour. While they have no legal status in the formal building contract, they still are an important part of the design of the structure of a building. They are the main method of verification used for the design of buildings by structural engineers, to demonstrate compliance with relevant requirements of the BCA. Computations are also the basis of the structural drawings. Therefore, they should be treated with care, respect, and attention to detail. They should be neat, legible, arranged in a suitable order and properly referenced. Always put your conclusions at the end of each section, usually on the right hand side to allow easy identification and checking.

Standardisation in approaching the documentation of a project, neat setting out and the order of computations is essential to allow for effective checking and avoiding costly mistakes.

It is recommended you use a pencil (HB or softer) for hand computations as that allows easy alterations when changes need to be made (and they usually will be) and use squared paper.

Make use of sketches in your computations and prepare free-hand drawings of sections and details to illustrate your design, as a picture is much easier to visualise than many lines of computations. A4 photocopies of architectural or structural details can also be used. At all times, sketch neatly and to scale to assist your feel for the structure.

Always set out at the beginning of any computations, your design philosophy and assumptions to allow the checkers and others who may have to access your computations, to understand how you have designed the structure. After establishing the design philosophy, actions, lateral analysis etc, consider designing the structure from the top down, even if the final computation may show a different order.

All significant design computations should be sub-divided into a number of design sections. These sub-divisions will form the basis for page numbering, list of contents, referencing, etc. The particular letter(s) allocated in the list below for a multi-storey building is an example of a lettering system that can be used, but other logical systems can be used. The numbering or lettering system to be used should be set up at the beginning of the computations and the example below is the sort of order in which computations might be made. For small projects, numerical numbering with a suitable index may be sufficient.

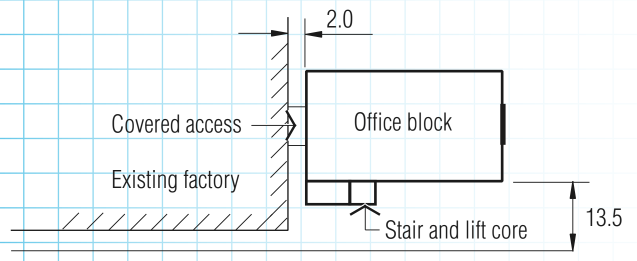
DP	Design Philosophy
AC	Actions (loads)
SK	Sketches
DFM	Durability, Fire and Materials
UT	Underpinning and Temporary walls
WA	Wind Analysis
EQ	Earthquake analysis
FT	Footings
CW	Column and Wall actions (rundowns)
CD	Columns
RT	Retaining walls
W	Loadbearing Walls including core walls
BF	Basement Floors
MF	Mezzanine Floors
1F	First Floor
TF	Typical Floors
PF	Plant room Floor
LM	Lift Motor room
PR	Plant room Roof
PC	Precast Cladding
R	Roof (including lift motor room floor and roof where applicable)
ST	Stairs
CP	Construction Procedures
M	Miscellaneous

The design notes scattered through this Chapter discuss various design matters and would not be normally included in any computations. They are provided for background and information for the users of this Handbook.

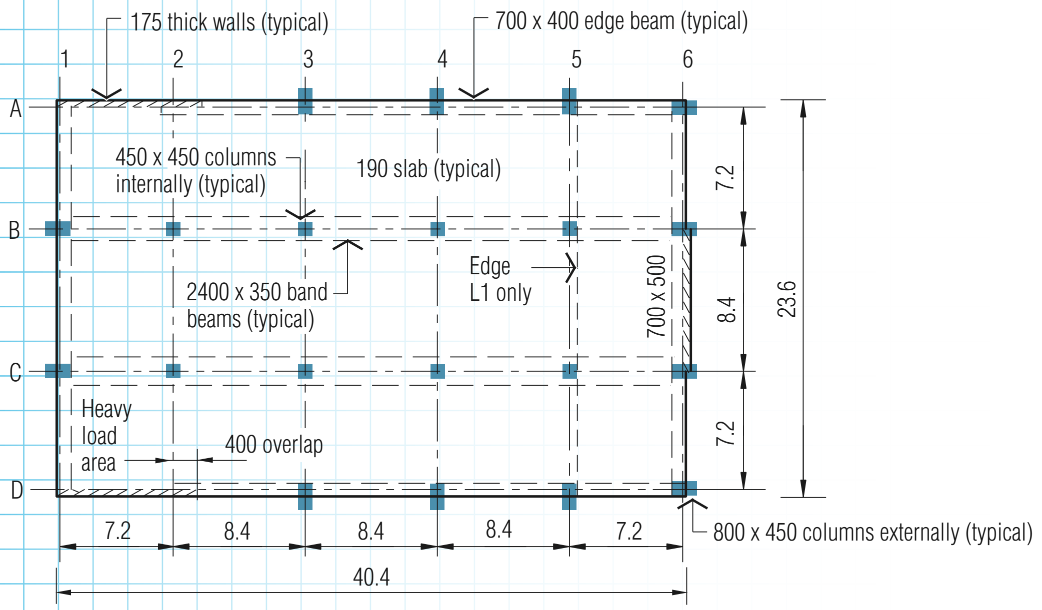
DESIGN PHILOSOPHY

CLIENT	A B Consolidated Pty Ltd
ADDRESS	25 Print Street, Coogee, NSW 2034
USE	Office building
LOCALITY	Light industrial area No polluting industry nearby Approximately 1 km to coast.
FOUNDATION	Dense sands with underlying clays in some local areas. Allowable bearing pressure 300 kPa in dense cemented sands at 1.2 m below NGL with sulfate and saline soil as advised by the geotechnical investigation.
GENERAL	The project consists of a new office block to an existing factory complex based on an overall master plan for the site. The new office block has been submitted for Development Approval (DA) and has received Council approval. Final structural drawings are being prepared based on the current architectural drawings. The computations along with the structural drawings will be submitted for structural approval by the private certifier or certifying authority. The following computations are part of what would be expected be submitted for review and approval.
STRUCTURAL FORM OF THE NEW BUILDING	The building will have a reinforced concrete structure with a suspended concrete roof at level 4 for a future floor extension, suspended concrete floors at first, second and third floors and a slab-on-ground at the ground floor. The concrete roof slab will have metal deck roofing over it initially for waterproofing, which will be removed, when the fourth floor is constructed in the future. The future roof and structure over at level 4 will be of lightweight construction with a metal deck roof with a braced steel frame supported on steel columns onto the concrete columns and concrete walls at level 4 and with lightweight walls externally. The floors and roof will be supported on square concrete columns internally, rectangular columns externally or loadbearing concrete walls with all the vertical elements supported on pad footings or strip footings founded on dense cemented sands. The suspended floors will consist of band beams in the east-west direction with one-way slabs in the other direction. A 700 deep x 400 wide edge beam is provided externally around each floor level where there are no concrete walls to support the glass facade and lightweight spandrel walls at each level. Concrete shear walls are provided on the south, west and north elevations. These will carry all lateral loads. In the southwest corner will be a stair and lift core, which will be part of the lateral-load-resisting system. Part of the external columns, footings, the bottom of the slab-on-ground and external faces of the external concrete shear walls are exposed to the environment but the remaining structure is internal except for a brief period during construction. Concrete stairs will be provided to both the core area and internally in a lightweight, fire-rated shaft to access and egress the building and to meet the egress requirements of the BCA as advised by the architect.
BUILDING REGULATION	Building Code of Australia (BCA). Assume an Importance Level 2 for the building in accordance with Table B.2a, Volume 1 of the BCA. This results in a Deemed-to-Satisfy Provision with an annual probability of exceedence of 500 years for wind and earthquake design.

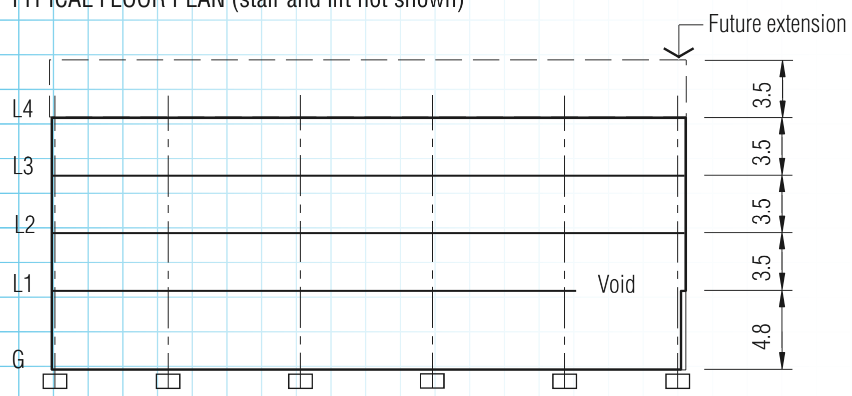
DESIGN ACTIONS (loads)	Design actions (loads) will be generally in accordance with the following Standards: AS/NZS 1170.1 *Structural design actions: Permanent, imposed and other actions* AS/NZS 1170.2 *Structural design actions: Wind actions* AS 1170.4 *Structural design actions: Earthquake actions in Australia* Permanent actions (dead loads) will include the structure self weight, permanent loads, ceilings and services, partitions and the like. Imposed actions (live loads) of 3.0 kPa for the office areas generally, 5.0 kPa to public areas and 7.5 kPa for storage areas will be adopted at the south-west corner between grids C and D and 1 and 2. All combinations of actions for design will be factored to provide limit state actions as required by AS/NZS 1170.0 and respective design standards.
WIND ACTIONS	The wind actions applied to the building are determined using a basic regional wind velocity of V_{500} = 45 m/sec for an A2 region. The site is assumed to be in Terrain Category 3. The lateral actions will be determined from a wind analysis. Although wind actions are likely to be the governing design case for lateral actions on the building as a whole, consideration will be given to earthquake actions on the building and individual elements such as walls, etc. Typically lateral actions are designed using some form of frame program, sometimes three-dimensional.
EARTHQUAKE ACTIONS	From a consideration of the acceleration coefficient and site factors a lateral analysis for EQ Forces using a static analysis will be required. Also in accordance with the Standard, structural design of parts and detailing is required.
FOOTINGS	The columns and walls will be supported on pad or strip footings based on the previous satisfactory performance of this type of footing for similar buildings at this industrial complex. The geotechnical investigation has provided the necessary background to decide on the appropriate footing system founded at about 1200 mm below NGL as about 100 mm of top soil is to be removed. The site is known to be variable with moderately reactive soils underlying the dense sands. The site investigation reports prepared by the geotechnical consultant set out the necessary design parameters for footings, slabs-on-ground, any retaining walls, pavements, etc for the project. The slab-on-ground is to be constructed directly on natural ground or engineered fill with appropriate allowance for movement. This approach is based on previous experience on the site. However, detailing will need further consideration.
CURTAIN WALLS, WINDOWS & CLADDING	To be designed by others to a performance specification prepared by the architect with input from the structural engineer. Separate computations will need to be supplied by the curtain wall designer to the Checking Authority.
STANDARDS	AS/NZS 1170 Series, *Structural design actions* AS 3600—2009 *Concrete structures*



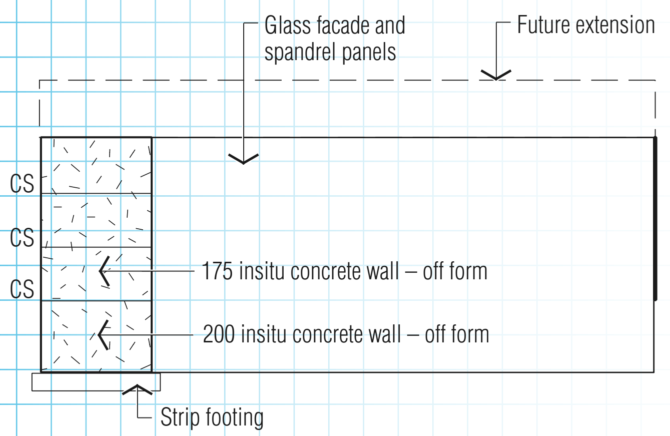
SITE PLAN



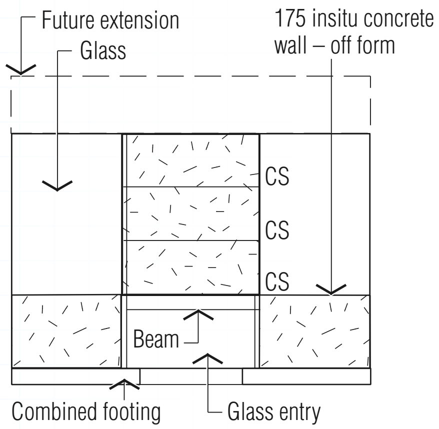
TYPICAL FLOOR PLAN (stair and lift not shown)



CROSS SECTION



SOUTH ELEVATION (North elevation similar)



EAST ELEVATION

DURABILITY, FIRE AND MATERIALS CONSIDERATIONS

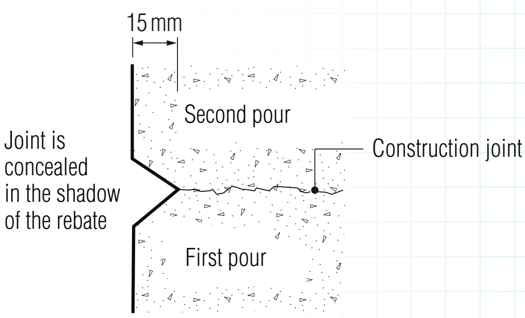
DURABILITY CONSIDERATIONS

Reinforced Concrete Design Handbook (RCDH)	Determine f'_c and cover options for evaluation with durability before starting structural design. (Follow Flowchart 3.1)
AS 3600	
Table 4.6	Abrasion resistance – not applicable
Clause 4.7	Freezing and thawing – not applicable
Clause 4.8	Aggressive soils (as advised by Geotechnical Report) Sulfate soils with 8,000 ppm in soil (Table 4.8.1) Saline soils with a soil electrical conductivity (EC_e) = 10
Table 4.3	External exposure classification B2, internal exposure classification A2 and surface in contact with the ground as per Table 4.8.1 and Table 4.8.2 for saline soils both with a B1 exposure classification
Table 4.10.3.2	Corrosion protection Exposure classification, f'_c and cover
Table 4.10.3.2	Cover (assume standard formwork and compaction)

	Member	Exposure class	f'_c (MPa)	Cover (mm)
Table 4.8.2 and Clause 4.10.3.5	Footings against ground (no dpm)	B1 (min. cover from Table 4.8.2 is 50 mm ∴OK)	32	40 + 20 = 60 [1]
Table 4.8.2 and Clause 4.10.3.5	Ground slab-on-ground Bottom of slab cast on dpm	B1	32	50 + 10 = 60
Table 4.10.3.2	Top of floor surface	A2	25	30
Table 4.10.3.2	Suspended floors	A2	25	30
	Roof (protected by roof until extension)	A	25	30
Table 4.10.3.2	Beams generally	A2	25	30
Table 4.10.3.2	Columns internally	A2	25	30
	Columns externally	B2	40	45 + 15 = 60 [2]
	Walls internal	A2	25	30 mm
	Walls exterior	B2	40	45 + 15 = 60 [2]

NOTE 1 Footing cast in ground without DPM. Add 20 mm to cover (Cl 4.10.3.5). Note that Table 4.8.2 requires a minimum cover of 50 mm also ∴OK

NOTE 2 Both the insitu external columns and walls are to have a 15-mm rebate at the construction joints externally as shown following, so increase the external cover to 60 mm to allow for rebate at a construction joint.



CONSTRUCTION JOINT

Details of wall joint showing rebate in external face

DESIGN NOTE: For the structure and each type of member, normally the architect or designer will determine the required FRL from the BCA and then the structural engineer from this information will determine the design requirements (minimum dimensions, axis distance to main reinforcement, etc) to achieve the specified FRPs from AS 3600 Section 5, assuming the deemed-to-comply approach is used. In order to compare axis distance with covers, a notional cover will need to be determined using an estimated fitment size for the column or beam plus an estimate of bar diameter. Generally, covers for durability will be greater than the notional cover determined for fire.

BCA reference	Determine FRLs	
Clause A 3.2	Building Classification (office)	= Class 5
Clause C1.2	Number of storeys	= 4
Clause C1.1 Table C1.1	Type of construction	= A
Spec. C1.1	Required FRL	
Table 3	Member	FRL

Member	FRL
Floors ignoring concession Spec. C1.1 CC3.3	120/120/120
Roof ignoring concession Spec. C1.1 CC3.3	120/60/30
Columns	120/ – / –
External walls (loadbearing)	
North and South 1–2	120/120/120
North and South 2–6	120/60/30
West	120/120/120
East	120/60/30
Internal walls (lift and stair)	120/120/120

DESIGN NOTE: The BCA also contains a number of other requirements regarding compartmentation, distance to egress, separation, etc, which will all have to be complied with. Normally the architect or designer for the project will determine all of these requirements prior to final design and confirm these to the design team.

Reinforced Concrete Design Handbook	Determine requirements to achieve specified FRPs in accordance with Section 5 AS 3600	
AS 3600	FLOORS AND ROOF	FRL 120/120/120
	SLABS (assume continuous, one way)	
Table 5.5.1	Insulation min. effective thickness	= 120 mm
Table 5.5.2 (A)	Structural adequacy axis distance	= 20 mm
	ie approximate notional cover	= 20–6 = 15 mm
	Integrity deemed OK if complies with structural adequacy and insulation (AS 3600 Cl 5.3.1)	
	BEAMS (assume continuous)	FRL 120/120/120
Figure 5.4.1(B)	Structural adequacy	

b_w (mm)	Axis distance (mm)	(notional cover)
400	35	(15)

(Note the notional cover 15 mm assuming with L10 fitment and 20-mm bar)

	COLUMNS		FRL 120/ – / –

Insulation and integrity for columns only required if part of wall

Structural adequacy columns for fire resistance (AS 3600 Cl 5.6.3)

Table 5.6.3	*b* (mm)	Axis distance (mm)	(notional cover)
	450	40	(16)

(Note the value of axis distance is based N^*_f / N_u = 0.5 and the notional cover of 16 mm has been calculated assuming N12 fitment and 24-mm column bars. These assumptions will need to be checked, but it is likely that cover for durability will control the design.)

WALLS — FRL 120/120/120 or FRL 120/ 60/ 30

AS 3600 Clause 5.7.2 — Check the structural adequacy of the wall using AS 3600 Cl 5.7.2

Minimum effective thickness for insulation for N^*_f / N_u = 0.7 is 160 mm and an axis distance 35 mm, assuming wall is exposed to fire on one side only.

DESIGN NOTE: In the design example, the wall is on the boundary of a fire compartment and therefore is subject to fire on one face only. If the wall was not part of the boundary of a fire compartment and could be subjected to fire on both sides, then it would need to be 220 mm thick and have a minimum axis distance of 35 mm.

AS 3600 Clause 5.7.3 — Limited $H_{we} / t_w \leq 30$ Check if more critical

Calculate H_{we} from AS 3600 Cl 11.4 using k = 1.0 or 0.75 as noted below

DESIGN NOTE: Clause 5.7.3 of AS 3600 limits the ratio of effective height to thickness to 40. However, as the wall is to be designed in accordance with Clause 11.5 of AS 3600 using the Simplified Design Method, this limits the ratio of effective height to thickness to 30 and that is what has been used in the table below.

Wall	$H_{we} = k \times H_w$		H_{we}	$t_w = H_{we}/30$
Grnd – L1	1.0 x 4800	=	4800	160
L1 – L2	0.75 x 3500	=	2625	88
L2 – L3	0.75 x 3500	=	2625	88
L3 – Roof	1.0 x 3500	=	3500	88

DESIGN NOTE: Designers can either use one or two layers of reinforcement in walls of say 10- or 12-mm bars (normal ductility) or reinforcing mesh say RF 82 or RF 92. For two layers of reinforcement both ways in each face, this will requires about 80–100 mm between the bars or mesh to place and compact the concrete when placed in vertical formwork.

Therefore, the minimum thickness of an insitu reinforced concrete wall using bar reinforcement in two layers, assuming the covers above = 60 + 12 + 12 + 80 + 12 +12 + 30 = 218 mm, say 225 mm minimum thickness. However, if the wall is precast or tilt- up and is poured on flat, then a minimum thickness of only 150 to 180 mm is usually required with two layers of reinforcement, as it is much easier to cast a wall on flat than in a vertical position. AS 3600 requires that walls over 200 thick have two layers of reinforcement.

∴ Adopt one layer of reinforcement of Ductility Class N bars placed centrally as ductility is important for this wall.

∴ adopt	Grnd – L1	t_w = 200 mm > 160 mm ∴OK
	L1 – Roof	t_w = 175 mm > 160 mm ∴OK

Table 5.7.2

Required axis distance to vertical reinforcement = 35 mm
By inspection, these thicknesses with the reinforcement in the middle of a wall, the axis distance will be adequate for both insulation and integrity and the minimum covers for durability will also be achieved.

DESIGN NOTE: Actual axis distance = 95 mm or 80 mm and covers = 85 mm or 75 mm and therefore satisfactory.

CHOICE OF CONCRETE STRENGTH
From a consideration of durability and fire-resistance requirements adopt the following concrete strengths:

— for footings	f'_c = 32 MPa
— for slab-on-ground	f'_c = 32 MPa
— for suspended slabs including roof	f'_c = 25 MPa
— for columns	f'_c = 40 MPa (SB)
— for walls	f'_c = 40 MPa (SB)
— rest of the concrete structure	f'_c = 25 MPa

(SB denotes special class concrete for exposure classification B2)

COVER
Note the requirements on cover for concrete placement in AS 3600 Clause 4.10.2

— for footings	cover = adopt 75 mm [1]
— for slab-on-ground *(assuming reinforcement mesh in top only)*	cover = 30 mm (top)
— for suspended slabs including roof *(cover > axis distance top and bottom)*	cover = 30 mm
— for columns *(cover > axis distance and use the same covers both internally and externally to avoid any errors on site)*	cover = 60 mm
— for walls externally *(cover > axis distance)*	cover = 60 mm
— for walls internally *(cover > axis distance)*	cover = 30 mm

NOTE 1 Although only 60 mm cover is required by the Standard, a decision was made to adopt 75 mm because of the fact that the footings are likely to be poured during the winter time. If this was also difficult ground, the cover might be increased to say 90 mm. This increased cover of 75 mm is also consistent with the discussion in Chapter 8 of this Handbook.

DESIGN NOTE: A good rule of thumb:

- Internally, cover should be the greater of 30 mm (minimum A2 Exposure classification with 25-MPa concrete is required for all non-residential construction), d_b and the maximum nominal size of aggregate
- Externally, cover should be the greater of 40 mm, 1.5 d_b and twice the maximum nominal size of aggregate.)

DESIGN NOTE: To finalise the design of band beams and beams, the effective depths for design for flexure must be determined. These effective depths in turn will depend on which direction the primary reinforcement in slabs will be placed, ie which way the top layer and bottom layers of the slab reinforcement are constructed. As the slabs essentially span one-way, then adopt the primary direction of slab reinforcement as running north/south, ie upper layer of top bars to run N/S and bottom layer of bottom bars to run N/S.

∴ Adopt the following minimum covers:

	Top	Bottom	External faces	Internal faces
Footings	75	75	75	–
Slab-on-ground	30	60	60	–
Suspended slabs incl roof	30	30	–	–
Band beams east/west	40 [1]	30	–	30
External beams east/west	40 [1]	30	–	30
External beams north/south	50 [2]	30	–	30
Columns	–	–	60	60
Walls	–	–	60	30

NOTE 1 Top cover to band beams and beams running east west must allow for the cover to slab plus an assumed 16-mm slab top bar, rounded up to nearest 5 mm less 12-mm fitment as slab bars will sit between fitments ie 30 + 16 – 12 = 34 round up to nearest 5 mm. Therefore, adopt 40 mm cover to the fitment of the band beams and beams running east west.

NOTE 2 Top cover to external beams running north south must allow for cover for slab plus 16-mm bar plus 12-mm bar rounded up minus 12-mm fitment = 30 + 16 + 12 – 12 = 46 mm round up to nearest 5 mm say 50 mm to fitment of the beam.

LOADS

GENERAL

Reinforced concrete 25.0 kN/m^3

DESIGN NOTE: Some designers use 24.5 kN/m^3 for the density of reinforced concrete, but 25.0 kN/m^3 is within the required order of accuracy and is slightly conservative.

175 thick walls = 4.375 kN/m^2
200 thick walls = 5.0 kN/m^2
350 deep band beams = 2.4 x 0.35 x 25 = 21.0 kN/m

Weight of concrete in band beams below soffit of slab = 2.4 x (0.35 – 0.19) x 25 = 9.6 kN/m

700 x 400 deep beams = 0.7 x 0.4 x 25 = 7.0 kN/m
190 slabs = 0.19 x 25.0 = 4.75 kN/m
Lightweight walls 3.5 m high allow = 1.00 kN/m
450 x 450 cols = 0.45 x 0.45 x 25 = 5.1 kN/m
800 x 450 cols = 0.8 x 0.45 x 25 = 9.0 kN/m

FUTURE ROOF (L5)

Permanent actions (dead loads)

Sheeting	0.05 kPa
Purlins	0.05 kPa
Steel beams	0.15 kPa
Ceilings and services	0.25 kPa
TOTAL	0.50 kPa

Imposed actions (live loads)

AS/NZS 1170.1 — (1.8/A + 0.12) but not less than 0.25 kPa

ROOF SLAB L4 (Future office)
Permanent actions (dead loads)

190 slab	4.75 kPa
Fixed partitions, finishes	1.00 kPa
Ceilings and services	0.25 kPa
TOTAL	6.00 kPa
Imposed floor actions (live loads) Generally	
Office including allowance for moveable partitions, etc	3.0 kPa
Imposed floor actions (live loads) Heavy load area	
Heavy load area including compactus etc	7.5 kPa

FLOOR SLAB L1–3
Permanent actions (dead loads)

190 slab	4.75 kPa
Fixed partitions, finishes	1.00 kPa
Ceilings and services	0.25 kPa
TOTAL	6.00 kPa
Imposed floor actions (live loads) Generally	
Office including allowance for moveable partitions, etc	3.0 kPa
Imposed floor actions (live loads) Heavy load areas	
Heavy load area including compactus etc	7.5 kPa

WIND ACTIONS TO WALLS AND FACADES

The wind actions applied to the building have been determined using AS/NZS 1170, *Structural design actions* Part 2: *Wind Actions.*

Design wind speeds to AS/NZS 1170 Part 2: *Wind Actions*

Region	A2
Regional wind speed V_{500}	= 45 m/s
Wind direction multiplier M_d	= 1.0
Terrain category	3
Height	= 18.8 m

$M_{z.cat} = 0.94$ $M_s = 1.0$ $M_t = 1.0$

$V_{sit\ Beta} = V_r\ M_d\ (M_{zcat}\ M_s\ M_t) = 42.3$ m/sec

$C_{fig} = C_{p.e} + C_{p.i} = 0.7 + 0.2 = 0.9$

$p = Q_z = 0.5 \times 1.2 \times V_r^2 /1000 \times C_{fig} = 0.6 \times 42.3 \times 42.3 \times 0.9/1000 = 0.97$ kPa (ultimate)

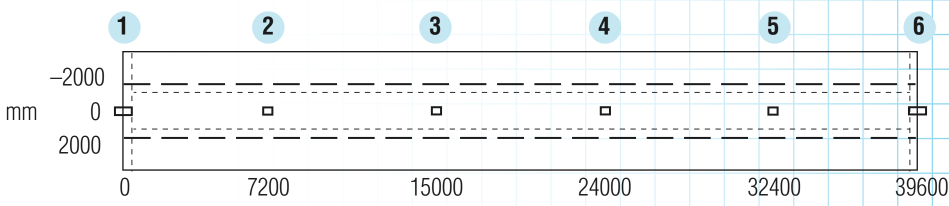
DESIGN OF TYPICAL FLOOR

ANALYSIS OF STRUCTURE

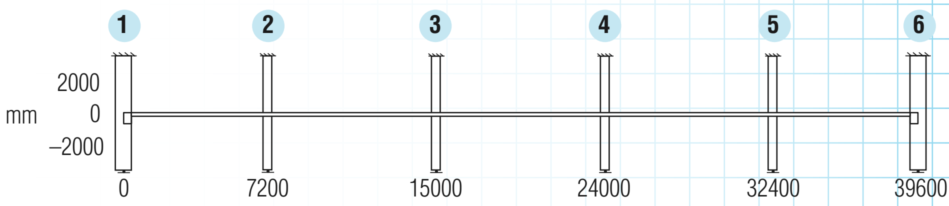
Use Linear Elastic Analysis (AS 3600 Cl 6.2)
Use a computer program to analyse and to determine bending moments and shears as shown on the following calculations.

ASSUMPTIONS

1 MODEL OF IDEALISED FRAME – bent on Grid B Level 2 (Level 3 similar)
Clause 6.9 of AS 3600

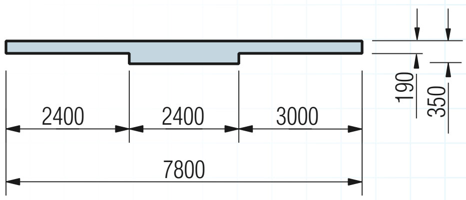


PLAN VIEW



FULL ELEVATION VIEW

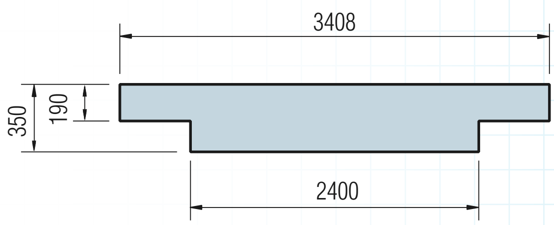
2 SECTION PROPERTIES



General details of bent

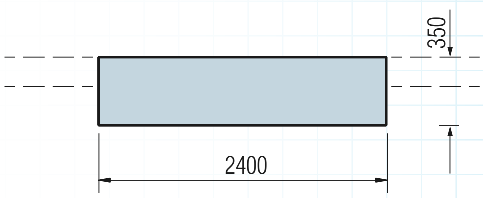
For band beam, assume T section in middle of span.

b_{ef} $= b_w + 0.2a$ where $a = 0.7\,L$ (Clause 8.8.2 of AS 3600)
$= 2,400 + 0.2 \times 0.7 \times 7,200$
$= 3,408$ mm



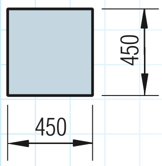
$I = 10.15 \times 10^{-3}\ m^4$
$A = 1.032\ m^2$

Band beam assume section following at a support



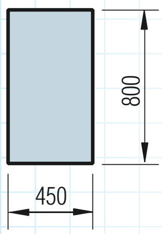
$I = 8.575 \times 10^{-3}$ m
$A = 0.84\ m^2$

Internal column assume



$I = 3.42 \times 10^{-3}\ m^4$
$A = 0.203\ m^2$

External column assume



$I = 19.2 \times 10^{-3}\ m^4$
$A = 0.36\ m^2$

3 MATERIAL PROPERTIES (RCDH Table 2.1)

	f'_c (MPa)	E_c (MPa)
Band beam	25	26,700
Column and walls	40	32,800

4 LOAD ON BAND BEAM

Permanent action (dead load)

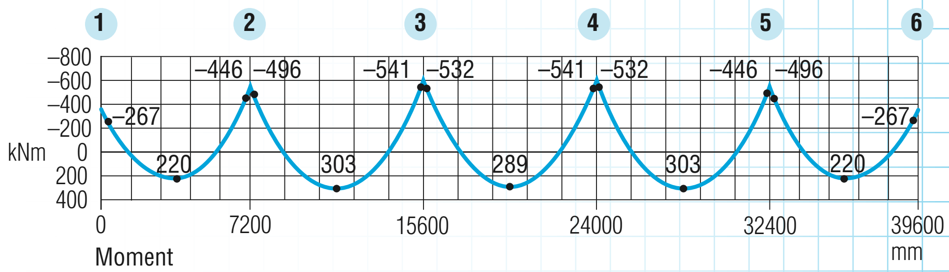
Band beam plus slab	= 46.65 kN/m
Fixed partitions	= 7.80 kN/m
Ceilings and services etc	= 1.95 kN/m
TOTAL	= 56.4 kN/m
Imposed actions (live load)	= 23.4 kN/m

Note: Live load ≤ 0.75 x dead load

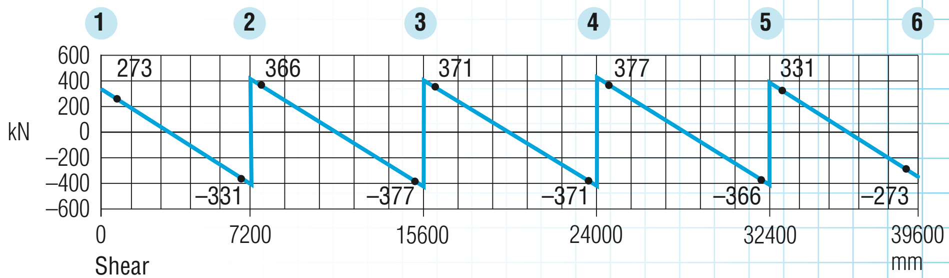
∴ Consider live load on all spans (AS 3600 Cl 2.4.4 (c) iii))

TYPICAL COMPUTER OUTPUT FOR BAND BEAM GRID C

DESIGN NOTE: The bent below has been analysed using a computer program; the output from such a program would look something like what is shown following. The output will depend on the model as analysed, which in this case has used the idealised frame method of analysis as set out in Clause 6.9 of AS 3600. Note that this program has calculated the moments and shears at the critical sections, rather than the peak moments and shears at the centre line of supports. Designers are responsible for ensuring that any software used for analysis is appropriate for the analysis being undertaken, as set out Clause 6.1.2 of AS 3600.



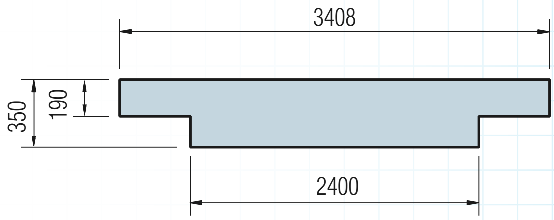
Moment



Shear

ULTIMATE FLEXURE

Design band beam at column C4 between spans 4–5
Band beam properties AS 3600 Cl 8.8.2



d_{top} = 350 – 30 – 16 – 14 = 290 mm (depth – cover – dia slab bar – ½ dia of band beam reinforcement assuming N24 top bars)

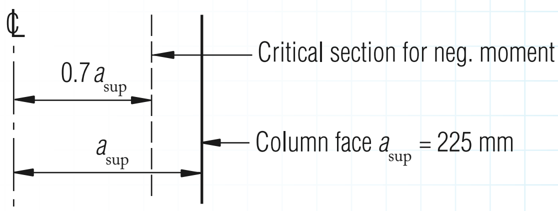
Adopt d_{top} = 285 mm as actual bar sizes are nominally larger than actual sizes

d_{bot} = 350 – 30– 14 = 306 mm (depth – cover – ½ dia of band beam reinforcement assuming N24 as bottom reinforcement)

Adopt d_{bot} = 300 mm as actual bar sizes are nominally larger than actual sizes.

FLEXURE

Negative BM @ critical section at column C4
AS 3600 Cl 6.9.2



DESIGN NOTE: The calculations below are for the critical section for negative moment. The bending moment at the centre line of the support is 564.6 kN.m where the shear is equal to 429 kN. These values are not shown above in the results of the computer analysis, but are generally are available in the output data. Normally the analysis program will calculate this figure for the designer at the critical sections, but calculations below by simple statics illustrate the process from first principles.

$$M^* = 564.6 - \frac{429}{a_{sup}} \times \left[\frac{0.7\,a_{sup}}{2}\right]^2$$

$= 564.6 - 23.6$

$= 541$ kN.m

$D = 350$ mm

$d = 285$ mm

$b_{ef} = 2{,}400$ and assumes no T beam action over the supports

$f'_c = 25$ MPa

$f_{sy} = 500$ MPa

Calculation of reinforcement required for M^*

Use RCDH Excel Spreadsheet 4.1 for rectangular beams to calculate reinforcement

A_{st} nominal = 5,583 mm^2 (initial estimate)

Try 12 – N24 = 5,424 mm^2 (spacing = 209 mm nom)

Minimum A_{st} = 5,234 mm^2 < 5,424 mm^2 ∴ OK

$k_u = 0.24 < 0.36$ ∴ Well OK

ϕM_u = 561 kN.m > 541 kN.m ∴ OK

ϕM_{uo} = 859 kN.m > 541 kN.m ∴ Well OK

$A_{st\,min}$ = 1,238 mm^2 < 5,424 ∴ Well OK

∴ Adopt 12 N24 as top reinforcement

Positive BM @ mid-span

M^* = 303 kN.m

D = 350 mm

d = 300 mm

t = 190 mm

b_{ef} = 3,408 and assumes T beam action at the midspan

f'_c = 25 MPa

f_{sy} = 500 MPa

Use RCDH Excel Spreadsheet 4.2 for T beams with the stress block within flange

A_{st} nominal = 2,971 mm^2 (initial estimate)

Provide A_{st} = 3,140 mm^2 = 10 N20 or 7 N24 (spacing = 240 or 342 mm nom)

Note: Clause 8.6.1(b) requires a maximum spacing between bars in tension to be 300 mm.

∴ Adopt 10 N20 bottom reinforcement

Minimum A_{st} for flexure = 2,620 mm^2 < 3,100 mm^2 ∴ OK

$k_u = 0.17 < 0.36$ ∴ Well OK

ϕM_{uo} = 952 kN.m > 303 kN.m ∴ Well OK

ϕM_u = 363 kN.m > 303 kN.m ∴ OK

$A_{st.min}$ = 1,176 mm^2 < 3,140 ∴ Well OK

CRACK CONTROL

AS 3600 Clause 8.6.1

Band Beam is fully enclosed within building except for brief period during construction ∴ only need to satisfy Item (a) and (b) of Cl 8.6.1 ie minimum reinforcement must comply with Cl 8.1.6.1 of AS 3600

See above reinforcement > minimum ie cover to centre of bars ≤ 100 mm and bar spacing ≤ 300 mm. See above. ∴ OK

Bottom reinforcement

cover = 30 mm ≤ 100 mm ∴ OK
spacing use 10 N20
9 x 300 = 2,700 mm

∴ OK — Bottom reinforcement 10 N20

Top reinforcement

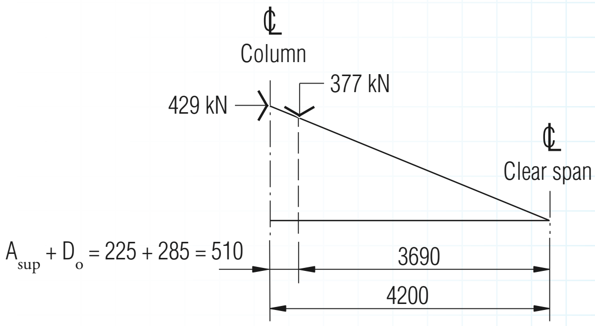
cover = 40 mm ∴ OK
spacing use 12 N24
11 x 300 = 3,300 > 2,400

∴ OK — Top reinforcement 12 N24

SHEAR

AS 3600 Clause 9.2.2 (a) Shear Design (treat as shallow beam Clause 8.2)

(a) Beam shear (follow **Flowchart 4.2**)



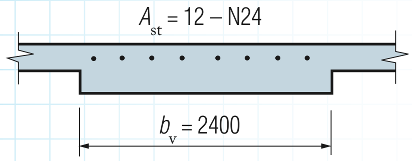
Clause 8.2.4 (b)

At the critical section for shear

V* = 429 x 3,690/4,200 = 377 kN

DESIGN NOTE: The calculations above are for the critical section for shear. The bending moment at the centre line of the support is 564.6 kN.m and the shear is equal to 429 kN. These values are not shown above in the results of the computer analysis, but are generally are available in the output data. Normally the analysis program will calculate this figure for the designer at the critical sections, but calculations above by simple statics illustrates the process from first principles.

BEAM PROPERTIES



A_{st} = 12 N24 = 5,424 mm^2

b_v = 2,400 mm

f'_c = 25 MPa

d_o = 285 mm

D = 350 mm

V^* = 377 kN

Clause 8.2.6

Determine $\phi V_{u.max} = \phi 0.2\, f'_c\, b_v d_o$

= 0.7 x 0.2 x 25 x 2,400 x 285 x 10^{-3}

= 2,394 kN > 377 kN $\therefore$ well OK

RCDH Excel Spreadsheet 4.3

Refer Excel Spreadsheet for beams, which will determine all the following values

β_1 = 1.447

For members where the cross-sectional area of shear reinforcement provided (A_{sv}) is not equal to or greater than the minimum area specified in Clause 8.2.8

β_2 = 1.0

β_3 = 1.0

Where $f_{cv} = f'^{\,1/3}_c \leq 4$ MPa = 2.92 MPa

V_{uc} = 577 kN

Clause 8.2.7 Shear strength of a beam excluding shear reinforcement

Determine ϕV_{uc}

$\therefore \phi V_{uc}$ = 0.7 x 577 = 404 kN

$\therefore 0.5\,\phi V_{uc}$ = 202 kN

Clause 8.2.9 Shear strength of a beam with minimum shear reinforcement

Determine $\phi V_{u.min} = \phi(V_{uc} + 0.10\sqrt{f'_c}\, b_v\, d_o) \geq \phi V_{uc} + \phi 0.6\, b_v\, d_o$

$\phi V_{uc} + \phi 0.6\, b_v\, d_o$ = 692 kN

$\phi(V_{uc} + 0.10\sqrt{f'_c}\, b_v\, d)$ = 644 kN

$\phi V_{u.min}$ = 0.7 x 853 = 692 kN

$0.5\,\phi V_{uc}$ = 346 kN

$\therefore 0.5\,\phi V_{uc} < V^* \leq \phi V_{u.min}$

Normally $A_{sv.min}$ is required in accordance with Cl 8.2.5(b) but as $V^* \leq \phi V_{uc}$ and $D < b_v/2$, no shear reinforcement is required in accordance with Cl 8.2.5(1).

DESIGN NOTE: Generally, designers should try to avoid shear reinforcement in band beams as it results in fitments at close centres and multiple fitments which are difficult to fix and are expensive. This is because the transverse spacing of fitments is limited to the lesser of 600 mm or D (Clause 8.2.12.2) and this spacing has to be reconciled with the maximum spacing of bars in tension of 300 mm. It is recommended that the depth or concrete strength of the band beam be increased or width decreased as required, to avoid the need for shear reinforcement where possible. However, designers should always provide nominal fitments to hold the bottom bars in place for fixing the reinforcement on site, for crack control to the soffit and sides and to lap with the bottom bars of the adjacent slabs.

The sections on page 10.21 clearly illustrate the differences between a band beam with nominal fitments to support the reinforcement and one with the required shear reinforcement (which is significant).

∴ Provide N12 fitments at 200 cts through the length of the band beam.

N12 fitments @ 200

PUNCHING SHEAR (AS A SLAB)

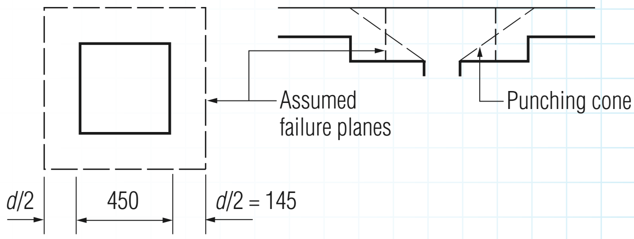
AS 3600 Cl 9.2.2 (b) — Case 1 Punching shear at column C3

AS 3600 Cl 9.2.3 (a) — Refer to bending moments and shear forces above.

$V^* = 377 + 371 = 748$ kN

$M^*_v = 541 - 532 = 9$ kN.m

treat as $M^*_v = 0$



Beam width = 2,400 > 450 + 285

∴ OK to use Chart 5.13

RCDH Chart 5.13

$c_1 = 450$ mm

$c_2 = 450$ mm

$d = 285$ mm

$f'_c = 25$ MPa

Calculate $c_1/c_2 = 1$

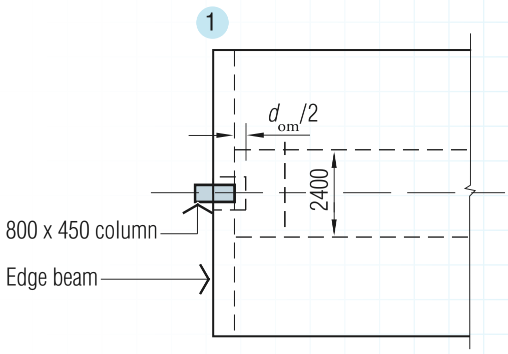
$c_1 + c_2 = 900$ mm

Read for $V^* = 748$ kN

Minimum depth = 240 mm < 350 mm ∴ OK

AS 3600 Cl 9.2.2 (b) — Case 2 Punching shear at column C1

AS 3600 Cl 9.2.4



$V^* = 273$ kN

$M^*_v = 267$ kN.m

$d_{om} = 285$ mm

AS 3600 Cl 9.2.4 (a) ie no closed fitments

$\phi V_u = \phi V_{uo} / [1.0 + (u\,M^*_v / 8\,V^* a\,d_{om})]$

$u = [450 + 285 + 2 \times (400 + 285/2)] = 1{,}820$ mm

DESIGN NOTE: The calculation of u is conservative as d_{om} is assumed the same for band beam and edge beam.

$f'_c = 25$ MPa

$a = 450 + 285 = 735$ mm

$\beta_h = X/Y = 450/400 = 1.125$

$f_{cv} = 0.17\,(1 + 2/\beta_h)\,\sqrt{f'_c} \leq 0.34\,\sqrt{f'_c}$

$= 0.472\,\sqrt{f'_c} > 0.34\,\sqrt{f'_c}$

$\therefore\ f_{cv} = 0.34\,\sqrt{f'_c} = 1.7$ MPa

$V_{uo} = u\,d_{om}\,f_{cv}$

$= 1{,}820 \times 285 \times 1.7/1{,}000 = 881.8$ kN

$\phi V_u = \phi V_{uo} / [1.0 + (u\,M^*_v / 8\,V^* a\,d_{om})]$

$$\therefore\ \phi V_u = \phi 882 / [1.0 + \frac{(1{,}820 \times 267 \times 10^6)}{(8 \times 273 \times 1{,}000 \times 735 \times 285)}]$$

$= 0.7 \times 882\,(1.0 + 1.06)$

$= 1{,}273$ kN > 273 kN $\therefore$ Well OK

CHECK DEPTH USING DEEMED TO COMPLY SPAN-TO-DEPTH RATIOS

AS3600 Clause 8.5.4 — Determine maximum value of L_{ef}/d

Clause 2.3.2 — Total deflection limit for

$\Delta/L_{ef} = 1/250$ (ie no masonry)

$f'_c = 25$ MPa

$\therefore$ Use RCDH Excel Spreadsheet 4.5

RCDH Spreadsheet 4.5 — Determine input values

$b = 2{,}400$ mm

$b_{ef} = 3{,}408$ mm

$D = 350$ mm

$c = 30$ mm (cover)

$d_o = 300$ mm

$A_{st} = 10$ N20 $= 3{,}140$ mm^2

$A_{sc} = 12$ N16 $= 3{,}140$ mm^2 (to lap with 12 N24)

$g = 56.4$ kN/m

$q = 23.4$ kN/m

$\psi_s = 0.7$

$\psi_1 = 0.4$

$L_{ef} = 8{,}400 - 400 + 350 = 8{,}350$ mm (interior span)

$k_1 = 0.0792$ (calculated by the program)

$k_2 = 0.00625$

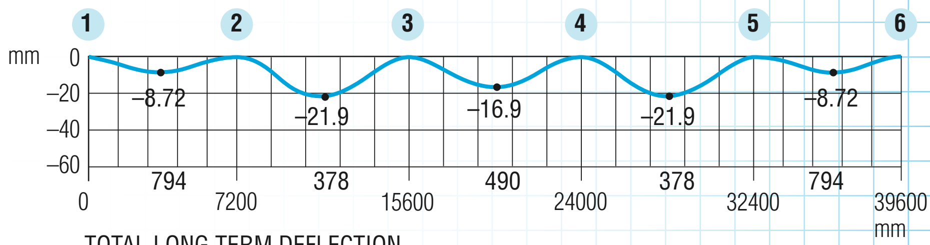
$E_c = 26{,}700$ MPa

Actual $L_{ef}/d = 8{,}350/300 = 27.8$

Calculated allowable $L_{ef}/d = 31.8$ for total deflection and is > 27.8 so the band beam as designed complies.

However, if this band beam was supporting a masonry partition it would not comply.

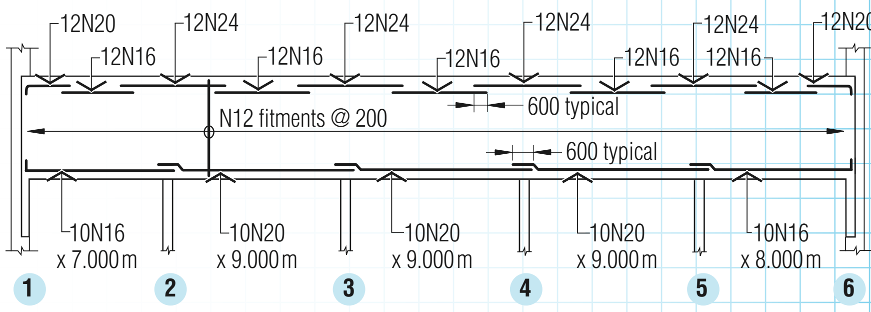
DESIGN NOTE: As discussed in Chapter 4, the depth of the beam determined by the deemed-to-comply method can be conservative, especially for shallow beams. Deflection calculations by the simplified calculations may result in thinner and more economical sections. The deemed-to-comply method suggests a deflection of about 25 mm by interpolation. Carrying out a more refined calculation, using a computer analysis program, results in the deflections as shown below. For span/250 long-term deflection 8,350/250 = 33.4 mm and the results below show a long-term deflection of 21.9 mm < 33.4 mm so again a 350 band beam is OK using this method of analysis.



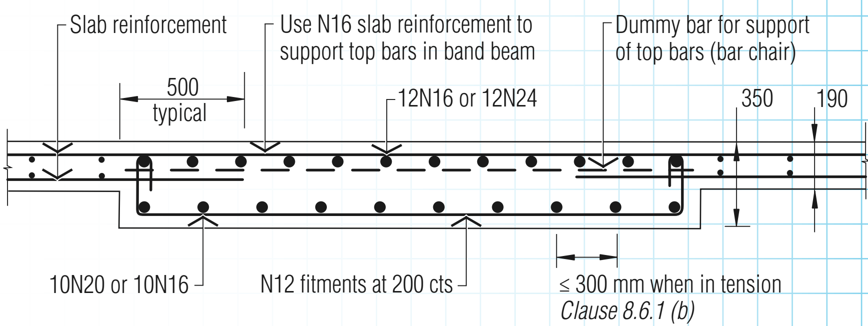
TOTAL LONG TERM DEFLECTION

Long term deflection as determined by computer analysis for a 350-deep band beam.

The following is the proposed reinforcing layout to be shown on the drawings using a beam elevation.

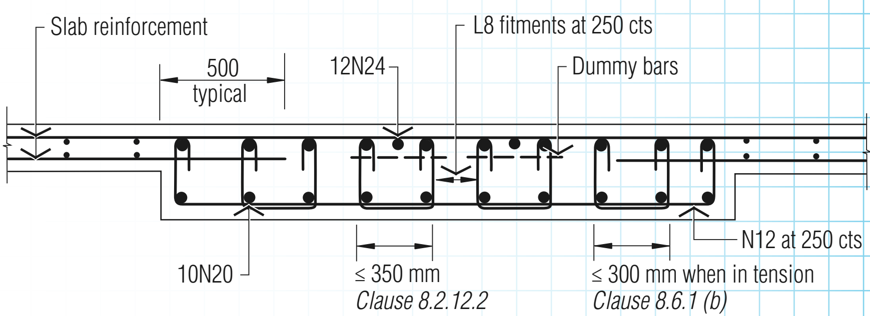


ELEVATION



SECTION Where only norminal shear reinforcement is required

Note: For fixing on site N16 dummy bars at about 1 m centres are used under to support the top reinforcement until the slab reinforcement is placed

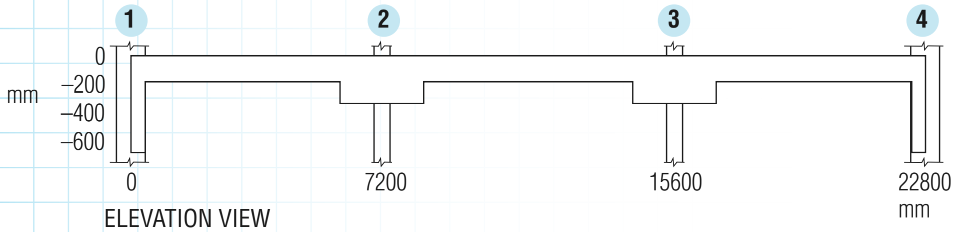


SECTION This section illustrates the extent of detailing if shear reinforcement is required

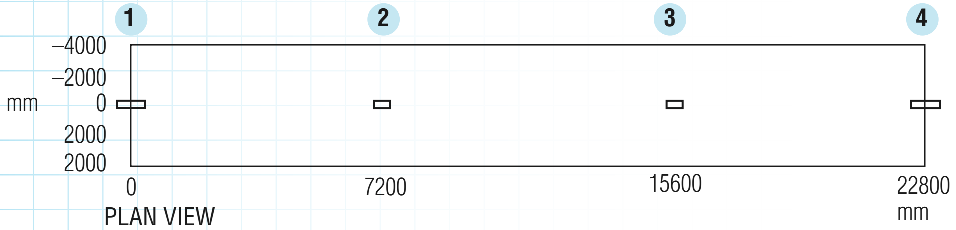
REINFORCEMENT LAYOUT

FLEXURAL DESIGN OF SLAB IN TRANSVERSE DIRECTION, eg GRID 4

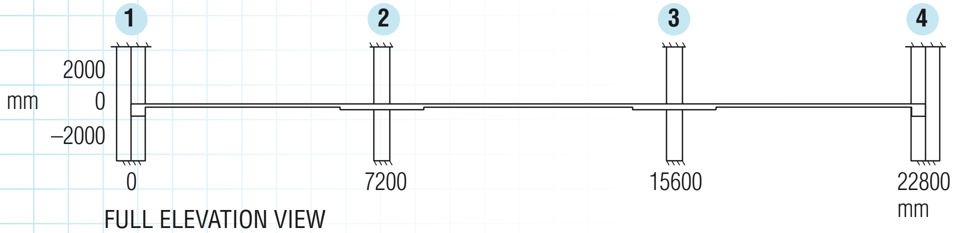
The following is the design model that was input into the computer using the loads previously calculated.



ELEVATION VIEW

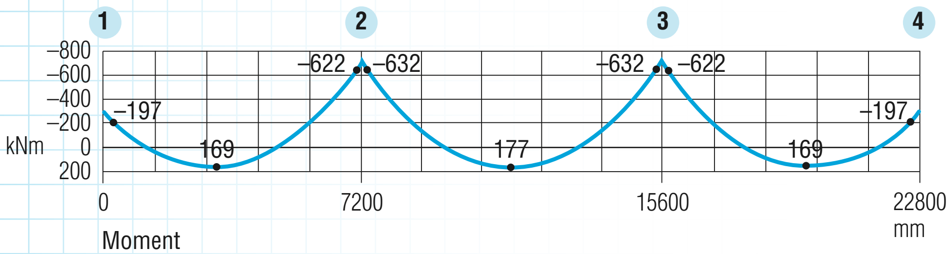


PLAN VIEW

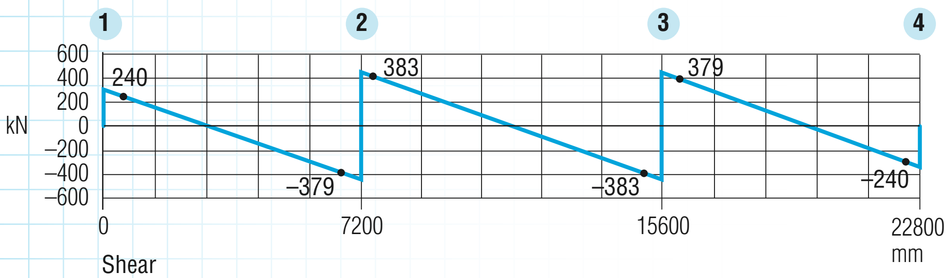


FULL ELEVATION VIEW

BMs and shears from computer analysis



Moment



Shear

ULTIMATE FLEXURE

DESIGN FOR FLEXURE

	Interior span
AS 3600 Clause 6.9.2	(a) Negative BM @ critical section near centre line of the column
	M^* = 632 kN.m
	M^* per metre strip = 632/8.4
	M^* = 75.2 kN.m/m
	b = 1,000 mm
	d = 350 – 30 – 10 = 310 mm
	f'_c = 25 MPa
	f_{sy} = 500 MPa
RCDH Chart 5.1	Try Chart 5.1
	M^* = 75 kN.m/m
	Outside the range $\therefore$ use RCDH Excel Spreadsheet
Spreadsheet Clause 9.4 AS 3600	$\therefore$ A_{st} nominal required = 713 mm^2/m
	Maximum spacing of bars = 300 mm or 2.0 D = 380 mm
	Provide A_{st} = 670 mm^2/m (N16 @ 300)
	Minimum A_{st} for flexure = 622 mm^2
	$\therefore$ Use = N16 @ 300 (670 mm^2/m > 622)
	or = N12 @ 175 (646 mm^2/m > 622)
	k_u = 0.06 < 0.36 $\therefore$ Well OK
	ϕM_{uo} = 423 kN.m > 73 kN.m $\therefore$ Well OK
	ϕM_u = 81 kN.m > 73 kN.m $\therefore$ OK
	$A_{st\ min}$ = 569 mm^2 < 670 mm^2 $\therefore$ OK
	(b) Negative BM @ edge of band beam (from computer analysis)
	M^* = 283/8.4 = 33.7 kN.m/m
	b = 1,000 mm
	d = 190 – 30 – 10= 150 mm
	f'_c = 25 MPa
	f_{sy} = 500 MPa
RCDH Chart 5.1	$\therefore$ A_{st} nominal required = 661 mm^2/m
	Try A_{st} = 670 mm^2/m
	= N16 @ 300 (670 mm^2/m > 593)
	or = N12 @ 175 (646 mm^2/m > 593)
	Minimum A_{st} for flexure = 593 mm^2 < 670 mm^2 $\therefore$ OK
	k_u = 0.12 < 0.36 $\therefore$ Well OK
	ϕM_{uo} = 99.1 kN.m > 33.7 kN.m $\therefore$ Well OK
	ϕM_u = 38.1 kN.m > 33.7 kN.m $\therefore$ OK
	$A_{st\ min}$ = 347 mm^2 < 670 mm^2 $\therefore$ OK
	Adopt N12 @175 top reinforcement

(c) Positive BM @ midspan (from computer analysis)

M^* = 177/8.4 = 21.1 kN.m/m

b = 1,000 mm

d = 190 – 30 – 10 = 150 mm

f'_c = 25 MPa

f_{sy} = 500 MPa

RCDH Chart 5.1 — A_{st} = 370 mm^2/m

∴ use = N12 @ 250 (411 mm^2/m > 370)

Adopt N12 @ 250 bottom reinforcement

CRACK CONTROL

AS 3600 Clause 9.4.1 — Slab is fully enclosed within building except for brief period during construction

∴ need only to satisfy Items (a) and (b)

Item (a) $A_{st.min} = 0.20\,(D/d)^2\,f'_{ct.f}/f_{sy}\,b_w\,d$ = 150

$A_{st.min}$ = 289 mm^2/m < 411 mm^2/m ∴ OK

Item (b) Centre to centre spacing

≤ the smaller of

2.0 x 190 = 380 or = 300

∴ max spacing 300 mm

∴ N12 @ 250 mm from above for flexure meets both these requirements. How provide N12 at 500 as top reinforcement in centres of spans for robustness and crack control in the top of the slab.

Adopt N12 @ 500 top reinforcement in centre of spans

CRACK CONTROL FOR SHRINKAGE AND TEMPERATURE IN THE SECONDARY DIRECTION

AS 3600 Clause 9.4.3 — Slab fully enclosed within building

Check area for crack control for shrinkage and temperature effects

Where a moderate degree of control over cracking is required in exposure classification in A1 and A2 then

$A_s \leq 3.5 \times 1{,}000 \times D \times 10^{-3}$ mm^2/m

3.5 x 190 = 665 mm^2/m

Where a strong degree of control over cracking is required is required for appearance and for exposure classifications A1, A2, B1, B2, C1 and C2, then

$A_s \leq 6.0 \times 100 \times D \times 10^{-3}$ mm^2/m = 1,140 mm^2/m

∴ Adopt moderate degree of control over cracking, as concrete is within an enclosed building and carpet will be used as floor coverings, which will hide any cracking.

∴ use N16 @ 250 mm (801 mm^2/m > 665)

or use N12 @ 150 mm (673 mm^2/m > 665)

Adopt N12 @ 300 as top and bottom reinforcement in the transverse direction ie equivalent to one layer of N12 @150.

DESIGN NOTE: This transverse reinforcement will serve two purposes as it will be used to support the main reinforcement in the direction of the span of the slab as well as providing crack control.

CHECK DEPTH USING SPAN-TO-DEPTH RATIOS

RCDH Use RCDH Excel Spreadsheet 5.1

A_{st} = 411 mm^2/m

A_{sc} = 205 mm^2/m

g = 5.75 kN/m

q = 3.0 kN/m

ψ_s = 0.7

ψ_1 = 0.4

L_{ef} = 6,000 mm

E_c = 26,700 MPa

Calculate A_{sc}/A_{st} = 0.5 (at mid-span)

Calculate k_{cs} = 1.4

Calculate $F_{d.ef} = (1 + k_{cs})\, g + (\psi_s + k_{cs}\, \psi_l)q$

$= 2.4 \times 5.75 + 1.26 \times 3.0 = 17.58$ kPa

AS 3600 Clause 9.3.4.1	(b) k_3	k_3 = 1.0 (one-way slab)
Clause 9.3.4.1	(c) k_4	k_4 = 2.1 (interior span)
Clause 2.4.2	(d) Δ/L_{ef}	Δ/L_{ef} = 1/250 (total deflection)
	(e) f'_c	f'_c = 25 MPa

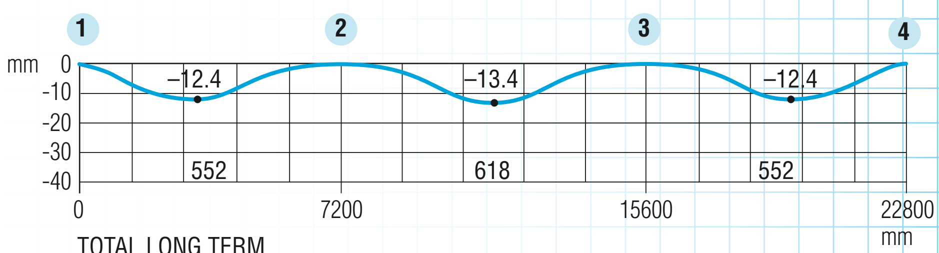
RCDH

Calculated from spreadsheet L_{ef}/d for total deflection = 38.31

Calculate actual L_{ef}/d = 6,000/150 = 40 > 38.31 ∴ Not OK

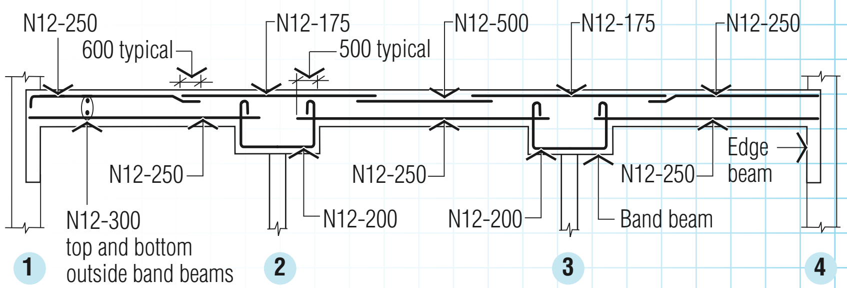
DESIGN NOTE: As discussed in Chapter 5, the depths of members determined using the deemed-to-comply method can be conservative, especially for shallow slabs and the deflection calculations by the simplified method will result in thinner and more economical sections. Carrying out a more refined calculation using the chosen analysis program gives the deflections shown below.

For span/250 long term > 6,000/250 = 24 mm and the calculation below show a long-term deflection of 13.4 mm < 24 mm so a 190 slab is well OK. Note: To get the maximum deflection one has to add the deflection of the slab to that of the bonded beam, ie 13.4 + 21.9 = 35.3 mm which is about the maximum visual limit for deflection.



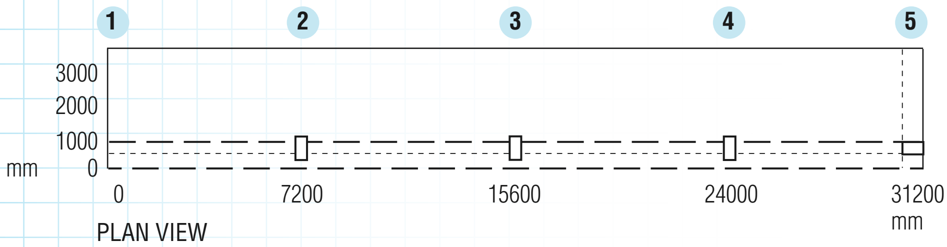
TOTAL LONG TERM

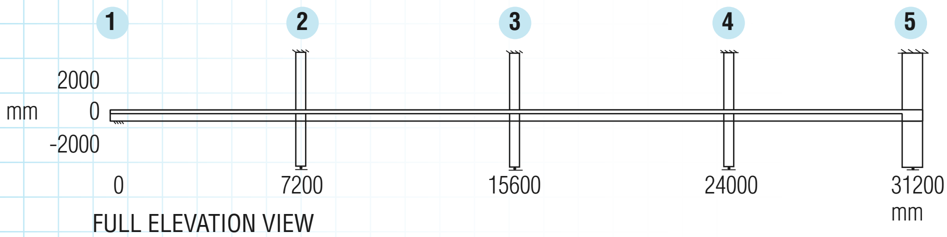
The following shows the details of the reinforcement for slabs.

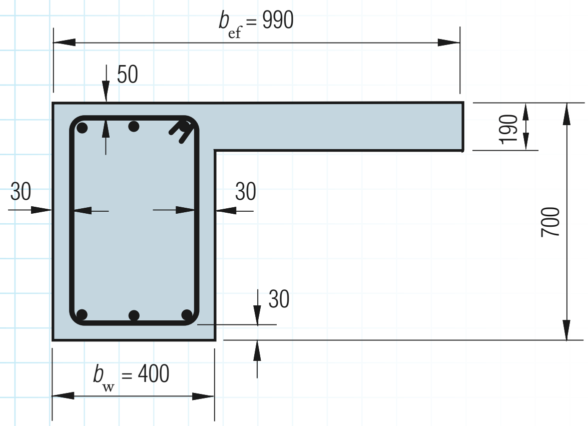


REINFORCEMENT LAYOUT

Details of bent to be analysed







Beam properties

f'_c = 25 MPa

f_{sy} = 500 MPa

cover c = 30 mm to fitments side and bottom

cover c = 50 mm top *(see page 10.11)*

d = 700 – 50 – 12 – 12

= 626 say 620 mm top and

= 700 – 30 – 12 – 12

= 646 say 640 mm bottom

AS 3600 Clause 8.8.2

$b_{ef} = b_w + 0.1 \times 0.7L$

= 400 + 0.1 x 0.7 x 8,400

= 988 say 990 mm in the middle of the span

LOADS

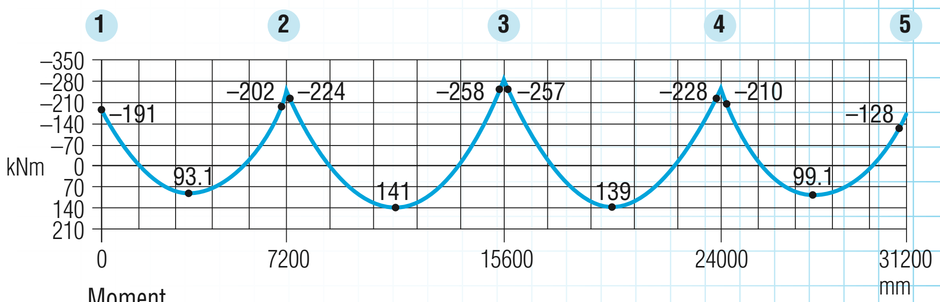
Permanent actions. *See pages 10.11 and 10.12 (dead loads)*

Beam	= 7.0 kN/m
Lightweight walls	= 1.0 kN/m
190 slab	= 2.9 x 4.75 = 13.8 kN/m
Partitions, ceilings and services	= 2.9 x 1.25 = 5.1 kN/m
TOTAL	= 26.9 kN/m

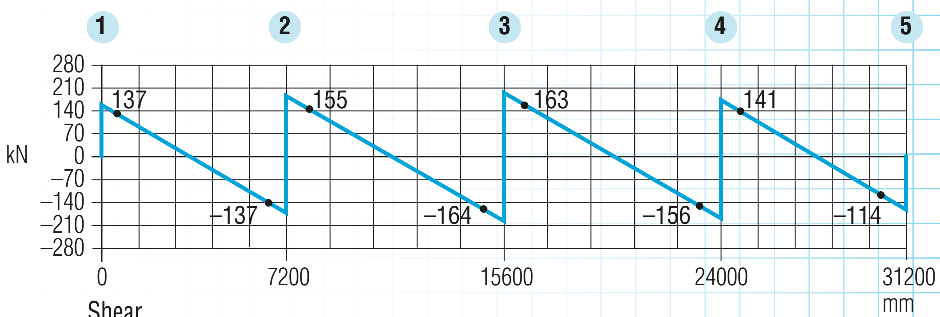
Imposed actions. *See page 10.12 (live loads)*

Floor	= 3.8 x 3.0 = 11.4 kN/m
TOTAL	= 11.4 kN/m

TYPICAL COMPUTER OUTPUT FOR BEAMS ALONG GRID D



Moment



Shear

ULTIMATE FLEXURE

Design moments — Max –ve $M^* = 258$ kN.m
Max +ve $M^* = 141$ kN.m

At Column 3 — $M^* = 258$ kN.m

$D = 700$ mm

$d = 620$ mm

$b_{ef} = 400$ and assume no L beam action over support

$f'_c = 25$ MPa

$f_{sy} = 500$ MPa

From RCDH Excel Spreadsheet $\therefore A_{st}$ nominal required = 1,244 mm^2

Provide A_{st} = 1,232 mm^2

$\therefore$ use = 2 N28 (1,232 mm^2 > 1105)
use = 3 N24 (1,356 mm^2 > 1105)

Minimum A_{st} for flexure = 1,105 mm^2 < 1,356 mm^2 $\therefore$ Well OK

$k_u = 0.14 < 0.36$ $\therefore$ Well OK

ϕM_{uo} = 677 kN.m > 258 kN.m $\therefore$ Well OK

ϕM_u = 288 kN.m > 258 kN.m $\therefore$ OK

A_{st} min = 379 mm^2 < 1,356 mm^2 $\therefore$ OK

$\therefore$ Top reinforcement 3N24

At midspan | $M^* = 141$ kN.m

$D = 700$ mm

$d = 640$ mm

$t = 190$ mm

$b_{ef} = 990$ and assume T-beam action in middle of span

$f'_c = 25$ MPa

$f_{sy} = 500$ MPa

From RCDH Excel Spreadsheet 4.2
As a L-beam with the stress block within the flange $\therefore$ A_{st} nominal required = 648 mm^2

Provide $A_{st} = 628$ mm^2 (2 N20)

Minimum A_{st} for flexure = 557 mm^2 < 628 mm^2 $\therefore$ OK

$\therefore$ use = 2N20 (628 mm^2 > 557 mm^2)

$k_u = 0.03 < 0.36$ $\therefore$ Well OK

$\phi M_{uo} = 722$ kN.m > 141 kN.m $\therefore$ Well OK

$\phi M_u = 159$ kN.m > 141 kN.m $\therefore$ OK

A_{st} min = 368 mm^2 < 628 mm^2 $\therefore$ OK

$\therefore$ Bottom reinforcement 2 N20

Check crack control | Beam not exposed to the weather on external surface
Therefore need to satisfy Items (a) and (b) only as appropriate of AS 3600 Cl 8.6.1

AS 3600 Clause 8.6.1 | (a) As noted above meets the minimum requirements of Cl 8.1.6.1 $\therefore$ OK

(c) Bars less than 100 mm from side and soffit of beam and less than 300 mm spacing $\therefore$ OK.

AS 3600 Clause 8.6.3 | Check crack control in side faces of beam. As overall depth < 750 mm it is not required. However, provide 1 N12 each face (EF) at the centre of the beam for crack control, as side face reinforcement (SFR).

SFR 1 N12 EF

SHEAR DESIGN

AS 3600 Clause 8.2.4 | At critical section

$V^* = 164$ kN

$D = 700$ mm

$d = 620$ mm

$b_v = 400$

$f'_c = 25$ MPa

$f_{sy} = 500$ MPa

$A_{st} = 1{,}232$ mm^2 (3 N24)

Using Spreadsheet 4.3
AS 3600 Clause 8.2.6 | Determine $\phi V_{u.max} = \phi 0.2\, f'_c\, b_v d_o$

$= 0.7 \times 0.2 \times 25 \times 400 \times 620 \times 10^{-3}$

$= 868$ kN

$V^* < \phi V_{u.max}$ $\therefore$ OK

Refer RCDH Excel Spreadsheet 4.3 for beams, which will determine all the following values

$\beta_1 = 1.1$

For members where the cross-sectional area of shear reinforcement provided (A_{sv}) is equal to or greater than the minimum area specified in Clause 8.2.8

$\beta_2 = 1.0$

$\beta_3 = 1.0$

Where $f_{cv} = f'_c{}^{1/3} \leq 4$ MPa = 2.92 MPa

Clause 8.2.7 — Shear strength of a beam excluding shear reinforcement

Determine ϕV_{uc}

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o f_{cv} \left[\frac{A_{st}}{b_v d_o} \right]^{1/3}$$

$\therefore \phi V_{uc} = 0.7 \times 136.1 = 98.4$ kN

$\therefore 0.5 \phi V_{uc} = 49.2$ kN

Clause 8.2.9 — Shear strength of a beam with minimum shear reinforcement

Determine $\phi V_{u.min} = \phi(V_{uc} + 0.10 \sqrt{f'_c}\, b_v d_o) \geq \phi V_{uc} + \phi 0.6 b_v d_o$

$\phi V_{uc} + \phi 0.6 b_v d_o = 202.5$ kN

$\phi(V_{uc} + 0.10 \sqrt{f'_c}\, b_v d) = 185.2$ kN

$\phi V_{u.min} = 202.5$ kN

$\therefore 0.5 \phi V_{uc} < V^* < \phi V_{u.min}$

$\therefore$ shear reinforcement is required in accordance with Clause 8.2.5.

Determine required shear reinforcement

RCDH Excel Spreadsheet 4.3

$A_{sv.min} = 140$ mm^2 at 500 spacing

$\therefore$ Adopt fitments L10 @ 300 cts throughout

CHECK DEPTH USING DEEMED-TO-COMPLY SPAN-TO-DEPTH RATIOS

AS 3600 Clause 8.5.4 — Determine maximum value of L_{ef}/d

Clause 2.3.2 — Total deflection limit for

$\Delta/L_{ef} = 1/250$

$f'_c = 25$ MPa

$\therefore$ use RCDH Excel Spreadsheet 4.5

RCDH Spreadsheet 4.5 — Determine input values

$b = 400$ mm

$b_{ef} = 990$ mm

$D = 700$ mm

c = 30 mm

$d_o = 640$

$A_{st} = 2$ N20 = 628 mm^2

$A_{sc} = 2$ N20 = 628 mm^2

$g = 26.9$ kN/m

$q = 11.4$ kN/m

$\psi_s = 0.7$

$\psi_1 = 0.4$

$L_{ef} = 8{,}400$ mm

k_1 = 0.0383 (calculated by the program)

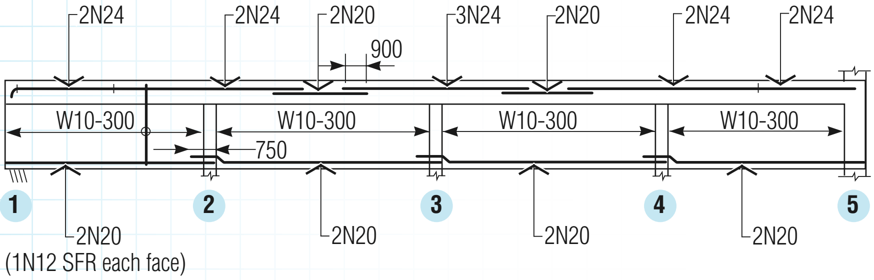
k_2 = 0.00391 (internal span)

E_c = 26,700 MPa

Actual L_{ef}/d = 8,400/640 = 13.1

Calculated allowable L_{ef}/d = 25.8 for total deflection > 13.1 so the beam as designed complies.

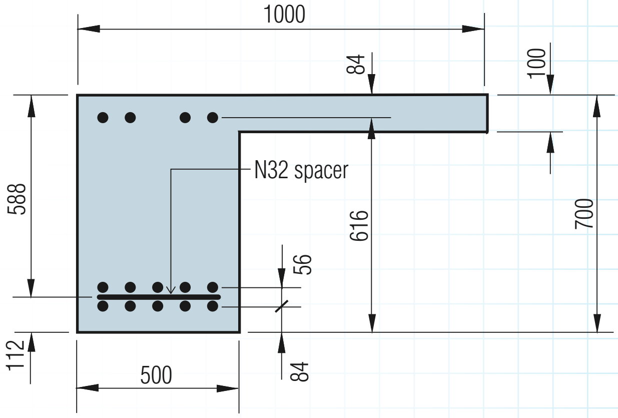
DESIGN NOTE: More refined computations suggest the long-term deflection is about 5 mm compared to about 17 mm suggested by the deemed-to-comply approach, again indicating the deemed-to-comply solution can be conservative.



REINFORCEMENT LAYOUT

BEAM LEVEL 1 SUPPORTING ENTRY FACADE

Beam properties



f'_c = 25 MPa

f_{sy} = 500 MPa

cover to main reo = 50 + 10 = 60 mm top and bottom

AS 3600 Clause 8.8.2

$b_{ef} = b_w + 0.1 \times 0.7L$

= 500 + 0.1 x 0.7 x 8,400 = 1,088 mm say 1,000 mm

$\therefore$ assume b_{ef} = 1,000 mm

Design actions at critical section

Max –ve M^* = 380 kN.m

Max +ve M^* = 960 kN.m

V^* = 360 kN

T^* = 50 kN.m

At column at the critical section for flexure

M^* = 380 kN.m

D = 700 mm

d = 615 mm (rationalised from 616 mm above)

b_{ef} = 500 mm and assume no L beam action over support

f'_c = 25 MPa

f_{sy} = 500 MPa

From RCDH Excel Spreadsheet 4.2

$\therefore$ A_{st} nominal required = 1,817 mm^2

Provide A_{st} = 1,808 mm^2 (4N24)

Minimum A_{st} for flexure = 1,660 mm^2 < 1,808 mm^2 $\therefore$ OK

k_u = 0.16 < 0.36 $\therefore$ Well OK

ϕM_{uo} = 833 kN.m > 380 kN.m $\therefore$ Well OK

ϕM_u = 414 kN.m > 380 kN.m $\therefore$ OK

$\therefore$ use = 4 N24 (1,808 mm^2)

4 N24 top reinforcement

At centre of span
Check if stress block extends into web of beam

$M^* = 960$ kN.m

$D = 700$ mm

$d = 585$ mm (rationalised from 588 mm above)

$t = 100$ mm

$b_w = 500$ mm

$b_{ef} = 1{,}000$ mm and assume L beam action in middle

$f'_c = 25$ MPa

$f_{sy} = 500$ MPa

From RCDH Excel Spreadsheet $\therefore A_{st}$ nominal required = 4,827 mm^2

$t = 116$ mm > 100 mm $\therefore$ stress block extends into web and the compression block is in both the flange and web

Provide $A_{st} = 4{,}928$ mm^2 (8 N28)

Minimum A_{st} for flexure = 4,563 mm^2 < 4,928 mm^2 $\therefore$ OK

$k_u = 0.27 < 0.36$ $\therefore$ OK

$\phi M_u = 1{,}036$ kN.m > 960 kN.m $\therefore$ OK

$\therefore$ use = 8N28 (4,928 mm^2)

8 N28 bottom reinforcement

DESIGN FOR SHEAR AND TORSION AT CRITICAL SECTION NEAR COLUMN (Follow Flowchart 4.3)

Note the results of the calculations for torsion is combined with the shear and flexural design. The calculations below are by hand and validated by the spreadsheets.

$M^* = 380$ kN.m

$V^* = 360$ kN

$T^* = 50$ kN.m

SHEAR DESIGN

AS 3600 Cl 8.2.4 — At critical section

$V^* = 360$ kN

$D = 700$ mm

$d_o = 615$ mm

$b_v = 500$

$f'_c = 25$ MPa

$f_{sy.t} = 500$ MPa

$A_{st} = 1{,}808$ mm^2 (4 – N24)

Using the RCDH Excel Spreadsheet 4.3

AS 3600 Clause 8.2.6 — Determine $\phi V_{u.max} = \phi 0.2 f'_c b_v d_o$

$= 0.7 \times 0.2 \times 25 \times 500 \times 615 \times 10^{-3} = 1{,}076$ kN

Refer RCDH Excel Spreadsheet 4.3 for beams, which will determine all the following values

$\beta_1 = 1.11$

For members where the cross-sectional area of shear reinforcement provided (A_{sv}) is equal to or greater than the minimum area specified in Clause 8.2.8

$\beta_2 = 1.0$
$\beta_3 = 1.0$

Where $f_{cv} = f'_c{}^{1/3} \leq 4$ MPa = 2.92 MPa

Clause 8.2.7

Shear strength of a beam excluding shear reinforcement

Determine ϕV_{uc}

$$\phi V_{uc} = \phi \beta_1 \beta_2 \beta_3 b_v d_o f_{cv} \left[\frac{A_{st}}{b_v d_o}\right]^{1/3}$$

= 0.7 x 1.11 x 1 x 1 x 500 x 615 x 2.92 (1,808/500/615)$^{1/3}$/1,000

$\therefore \phi V_{uc}$ = 126 kN

$\therefore 0.5 \phi V_{uc}$ = 63 kN

Clause 8.2.9

Shear strength of a beam with minimum shear reinforcement

Determine $\phi V_{u.min} = \phi(V_{uc} + 0.10\sqrt{f'_c}\, b_v d_o) \geq \phi V_{uc} + \phi 0.6\, b_v d_o$

$\phi V_{uc} + \phi 0.6\, b_v d_o$ = 254 kN

$\phi(V_{uc} + 0.10\sqrt{f'_c}\, b_v d)$ = 233 kN

$\therefore \phi V_{u.min}$ = 254 kN

$\therefore V^* < \phi V_{u.max}$ $\therefore$ OK

$> \phi V_{uc}$

$\therefore$ shear reinforcement is required.

RCDH Excel Spreadsheet 4.3

Determine required shear reinforcement

Calculate V_{us} = 335.8 kN

If $V_{us} < V_{us.min}$ then $V_{us} = V_{us.min}$ ie V_{us} = 336 kN

$A_{sv} = V_{us} / [(f_{sy.f}\, d_o/s) \cot \theta_v]$

= 229 mm^2 and a maximum spacing of 300 mm

and where θ_v = 45 deg for vertical fitments

Consider N12 fitments where area of fitment (two legs) = 226 mm^2

Maximum spacing of N 12 fitment by interpolation = 300 x 226/328 = 296 mm

$\therefore$ Fitments N12 @ 225 cts throughout

DESIGN NOTE: It is normal for fitment spacing to be a multiple of 25 mm. Do not use spacings such as 296 mm calculated above as it is just not realistic on site.

TORSION DESIGN

Design using RCDH Excel Spreadsheet 4.4.

T^* = 50 kN.m

Calculate torsion modulus J_t and ignore the flange as it only contributes a very small part to the overall torsional strength.

A_{st} = 1,808 mm^2

$J_t = J_{web}$

Table 4.1

for x = 500 mm

y = 700 mm

J_{web} = 57.7 x 10^6 mm^3

AS 3600 Clause 8.3.3

Determine $\phi T_{u.max} = \phi 0.2 f'_c J_t$

= 0.7 x 0.2 x 25 x 57.7 x 10^6 x 10^{-6} = 202 kN.m

Clause 8.2.6

Determine $\phi V_{u.max} = \phi 0.2 f'_c b_v d_o$

= 0.7 x 0.2 x 25 x 500 x 615 x 10^{-3} = 1,076 kN

Clause 8.3.3	Check torsional strength not limited by web crushing $T^*/\phi T_{u.max} + V^*/\phi V_{u.max} = 50/202 + 360/1{,}024$ $= 0.25 + 0.33 = 0.58$ $< 1 \therefore$ OK
Clause 8.3.5	Check if torsional reinforcement is required. Calculate torsional strength of a beam
Clause 8.3.5 (a)	Calculate $\phi T_{uc} = \phi 0.3 J_t \sqrt{f'_c}$ $= 0.7 \times 0.3 \times 57.7 \times 10^6 \text{ mm}^3 \times \sqrt{25}/1{,}000 = 60.6$ kN.m
Clause 8.3.5 (b)	Calculate $\phi T_{us} = \phi f_{sy.f} (A_{sw}/s)\, 2A_t \cot \theta_v$ Where A_t = area of a polygon with vertices at the centre of longitudinal bars at the corners of the cross-section $= (500 - 40 - 40 - 12 - 12 - 24/2) \times (700 - 40 - 40 - 12 - 12 - 24/2)$ $= 224{,}256 \text{ mm}^2$ u_t = perimeter of the polygon defined for A_t $= 2\,[(500 - 40 - 40 - 12 - 12 - 24/2) + (700 - 40 - 40 - 12 - 12 - 24/2)]$ $= 1{,}936$ mm
Clause 8.2.10 (b) (1)	Where θ_v = angle between the axis of the concrete compression strut and the longitudinal axis of the member and shall be taken as 45° $\phi = 0.7$ $A_{sw} = 113 \text{ mm}^2$ (assumes N12 fitment) $f_{sy.f} = 500$ MPa (yield strength of fitment) $s = 200$ mm (spacing of fitments) $\phi T_{us} = \phi A_{sw} f_{sy.f}\, 2A_t \cot \theta_v / s$ $= 0.7 \times 113 \times 500 \times 2 \times 224{,}256 \times 1/200 \times 10^6$ $= 88.7$ kN.m
Clause 8.2.7	Calculate shear strength of a beam without shear reinforcement $V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o f_{cv} \left[\dfrac{A_{st}}{b_v d_o}\right]^{1/3}$

Refer RCDH Excel Spreadsheet 4.3 for beams, which will determine all the following values

	$\beta_1 = 1.11$ For members where the cross-sectional area of shear reinforcement provided (A_{sv}) is equal to or greater than the minimum area specified in Clause 8.2.8 $\beta_2 = 1.0$ $\beta_3 = 1.0$ Where $f_{cv} = f'^{1/3}_c \leq 4 \text{ MPa} = 2.92$ MPa $\therefore \phi V_{uc} = 125.0$ kN
Clause 8.3.4 (a) (i)	Check if $T^* \leq 0.25 \phi T_{uc} = 15.2$ kN.m and as $T^* = 50$ kN.m > 15.2 kN.m $\therefore$ torsional reo is required
Clause 8.3.4 (a) (ii)	Check if $T^*/\phi T_{uc} + V^*/\phi V_{uc} \leq 0.5 = 50/60.6 + 360/125.0 = 3.7 >> 0.5$ $\therefore$ torsional reo is required
Clause 8.3.4 (a) (iii)	$T^*/\phi T_{uc} + V^*/\phi V_{uc} > 1.0$ $\therefore$ torsional reo is required

Clause 8.3.4 (b) — Check requirements for torsional reinforcing ie $T^*/\phi T_{us} \le 1.0$

$= 50/88.7 = 0.56 < 1.0 \therefore$ OK

Clause 8.3.6(a) — Requirements for additional longitudinal tensile reinforcement

$A_{lt} = (0.5\ f_{sy.f}/f_{sy})\ (A_{sw}/s)(u_t \cot^2\theta_v)$

$= 0.5 \times 500/500 \times 113/200 \times 1{,}936 \times 1$

$= 547\ mm^2$

$\therefore$ provide 2 additional N24 top bars ie 6 total

Clause 8.3.6(b) — Requirements for additional longitudinal compressive reinforcing

$A_{lt} = (0.5\ f_{sy.f}/f_{sy})\ (A_{sw}/s)(u_t \cot^2\theta_v)$

$= 0.5 \times 500 / 500 \times 113/200 \times 1{,}936 \times 1$

$= 547\ mm^2$

$\therefore$ provide 2 additional N28 bottom bars ie 10 total

Requirements for additional torsional fitments

= N12 at 225 mm max centres

$\therefore$ reinforcement to spandrel beam is
4 + 2 = 6 N24 top, 8 + 2 = 10 N28 bottom

Fitments N12@175 (shear) + N12@175 (torsion)
$\therefore$ say = N12@175 in pairs for extent of torsion and then N12@250 as single fitments for the remainder of the beam.

DESIGN NOTE: In the above design example, the extent of shear and torsion along the length of beam has not been fully defined, which would usually be provided by the analysis of the beam. This then would allow the extent of design for shear and torsion to be calculated along the length of the beam and define the extent of fitments for both shear and torsion. In the above example, the N12 @ 175 could possibly be reduced to N12 @ 300 towards the centre of the beam, if the torsion is not along the full length of the beam.

AS 3600 Clause 8.3.8 (a) — Note all fitments are to be closed.

VERTICAL ACTIONS (LOADS) AND BENDING MOMENTS (COLUMN RUNDOWNS)

DESIGN NOTE: In order to design the columns, both the vertical actions (loads) at each floor and the bending moments need to be determined.

As discussed in Chapter 6, assessing the vertical actions (loads) carried by columns and walls requires a full understanding of the building, the behaviour of the structure and all actions (loads) carried by the structure. These vertical actions (loads) are usually calculated by assessing the actions (loads) supported by each column or walls on a floor-by-floor basis based on the tributary areas to each column or wall. This can be calculated by using a spreadsheet or appropriate structural analysis software.

Column rundowns calculated by spreadsheet are typically based on a simple area or length basis, with proportioning of the actions (loads) to each vertical element by taking half the distance in each direction to the adjacent vertical element. These rundowns may not include all the actions (loads) to the columns (and walls) because of continuity and frame action. These additional actions (loads) are sometimes known as 'moment shears' or similar and can be up to 15% of the floor actions (loads). For example, an edge column will generally have fewer actions (loads) on it when using an area basis, while the first column in from the edge of a building will have more actions (loads) on it. It is usual not to deduct moment shears from external columns.

Usually, actions (loads) at each level are calculated just above the floor below. The heading 'on Level 3' as shown in the spreadsheet on the column rundown means the actions (loads) from the floors above, just above the third floor and would be used to design the column from Level 3 to Level 4.

Clause 3.4.2 of AS/NZS 1170.1 also allows a reduction of uniformly distributed imposed actions. However, as slabs are essentially one-way, no reduction is allowed.

COLUMN RUNDOWN C4

DESIGN NOTE: The following rundown is for column C4. The column moments have been determined from the 2D analysis of the bents in both directions.

Load element	Unit area length	Permanent actions (DL)	Imposed actions (LL)	Permanent axial actions (DL)	Permanent bending moments E/W	Permanent bending moments N/S	Imposed axial actions (LL)	Imposed bending moments E/W	Imposed bending moments N/S
ON LEVEL 4									
1 Roof	65.5	0.5	0.25	32.8			16.4		
2 Column	3.5	1		3.5			0.0		
3 Moment shears	7.8	0.5	0.25	3.9			2.0		
Total this level				40.2	0	0	18.3	0	0
Total on Level 4		PA (DL)	40.2						
		IA (LL)	18.3						

(continues)

COLUMN RUNDOWN C4 *(continued)*

Load element	Unit area length	Permanent actions (DL)	Imposed actions (LL)	Permanent axial actions (DL)	Permanent bending moments E/W	Permanent bending moments N/S	Imposed axial actions (LL)	Imposed bending moments E/W	Imposed bending moments N/S
ON LEVEL 3									
1 Floor	65.5	4.75	3	311.1	3	42	196.5	1	21
2 Band beam	8.4	9.6	0	80.6			0.0		
3 Other permanent loads	65.5	1.25	0	81.9			0.0		
4 Column	3	3.15		9.5			0.0		
5 Moment shears	7.8	6.15	3	48.0			23.4		
6 Live load reduction	65.5	0	0	0.0			0.0		
Total this level				531.1	3.0	42.0	219.9	1.0	21.0
Total on Level 3		PA (DL)	571.2						
		IA (LL)	238.2						
ON LEVEL 2									
1 Floor	65.5	4.75	3	311.1	3	42	196.5	1	21
2 Band beam	8.4	9.6	0	80.6			0.0		
3 Other permanent loads	65.5	1.25	0	81.9			0.0		
4 Column	3	3.15		9.5			0.0		
5 Moment shears	7.8	6.15	3	48.0			23.4		
6 Live load reduction	65.5	0	0	0.0			0.0		
Total this level				531.1	3.0	42.0	219.9	1.0	21.0
Total on Level 2		PA (DL)	1,102.3						
		IA (LL)	458.1						
ON LEVEL 1									
1 Floor	65.5	4.75	3	311.1	3	42	196.5	1	21
2 Band beam	8.4	9.6	0	80.6			0.0		
3 Other permanent loads	65.5	1.25	0	81.9			0.0		
4 Column	3	3.15		9.5			0.0		
5 Moment shears	7.8	6.15	3	48.0			23.4		
6 Live load reduction	65.5	0	0	0.0			0.0		
Total this level				531.1	3.0	42.0	219.9	1.0	21.0
Total on Level 1		PA (DL)	1,633.3						
		IA (LL)	678.0						
ON FOOTING									
1 Floor	65.5	4.75	3	311.1	3	42	196.5	1	21
2 Band beam	8.4	9.6	0	80.6			0.0		
3 Other permanent loads	65.5	1.25	0	81.9			0.0		
4 Column	3	4.8		14.4			0.0		
5 Moment shears	7.8	6.15	3	48.0			23.4		
6 Live load reduction	65.5	0	0	0.0			0.0		
Total this level				536.0	3.0	42.0	219.9	1.0	21.0
Total on Footing		PA (DL)	2,169.3						
		IA (LL)	897.9						
	Total PA	(DL)	**2,169.3 kN**						
	Total IA	(LL)	**897.9 kN**						

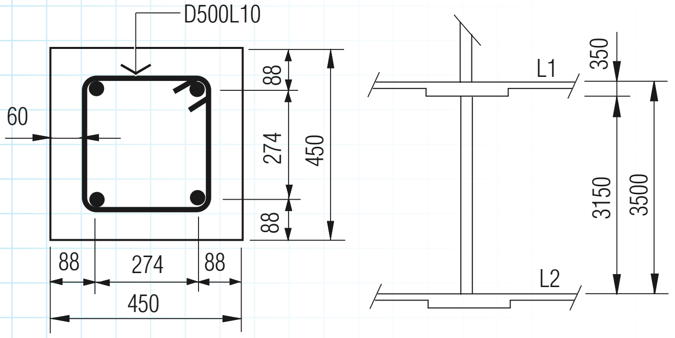
Notes:

— Actions (loads) are in kN or kPa. All loads are unfactored.

— Moments are in kN.m.

INTERNAL COLUMN C4, LEVEL L1 TO L2

Column properties



f'_c = 40 MPa

f_{sy} = 500 MPa

Cover = 60 mm to fitment

AS 3600 Clause 10.1.3 — Braced column, ie lateral loads resisted by shear walls

Design actions (from computer analysis)

N^* = 1,633 x 1.2 + 678 x 1.5 = 2,977 kN

M^*ns_{max} = 42 x 1.2 + 21 x 1.5 = 82 kN.m

M^*ew_{max} = 3 x 1.2 + 1 x 1.5 = 5 kN.m

Check if column short

Calculate L_e/r

Clause 10.3.1 — Assume $L_e = L_u$ = 3,150 mm

$r = 0.3D$ = 0.3 x 450 = 135 mm

$\therefore L_e/r$ = 3,150/135 = 23.5

Clause 10.3.1(a) — < 25 $\therefore$ column short

$\therefore$ moment magnifiers are not required.

Assume column reinforced on four faces

f'_c = 40 MPa

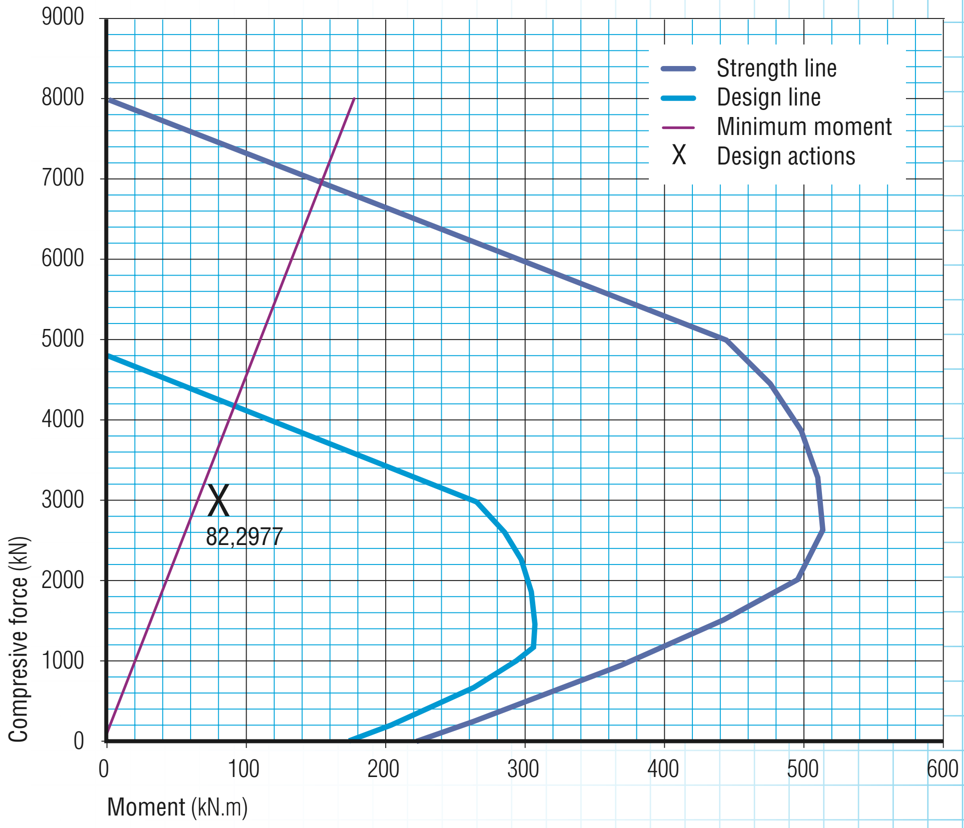
DESIGN NOTE: Because of the design decision to use 40 MPa minimum for all columns with 60 cover to the fitment say 70 mm to the main bar or an axis distance of about 85 mm, then the columns in the upper levels are likely to be not heavily reinforced. Therefore try 1% reo as a minimum, ie 4 N28 at 1.2% and see how the column details fit the interaction diagram following. From various design checks, the concrete strength could be reduced to 32 MPa. However, the volume of concrete for all the internal columns is about 5 m^3 total per floor (ie a truck load of concrete and all columns on the floor are likely to be cast at the one time) so the saving in changing to a lower strength concrete is minimal and will only result in confusion on site and possibly add cost because of smaller batches of concrete.

However, the designer will need to check the transmission of axial forces through the slab because of the column's concrete strength. Refer to Clause 10.8. An alternative is to design the column with 32-MPa concrete but use 40-MPa concrete for practical reasons.

INPUT DATA INTO COLUMNS DESIGN SOFTWARE

The following interaction diagram has been calculated using an excel spreadsheet for a short column in accordance with AS 3600. As can be seen below the design actions are under the design line for 4 N28 and 40 MPa so well OK.

Area of reo = 1.22% < 4% $\therefore$ OK



$\therefore$ Adopt 4 N28

Determine size and spacing of fitments

AS 3600 Clause 10.7.4.3 — Size use L10 Table 10.7.4.3

Spacing smaller of 15 d_b or D

15 d_b = 15 x 28 = 420 mm

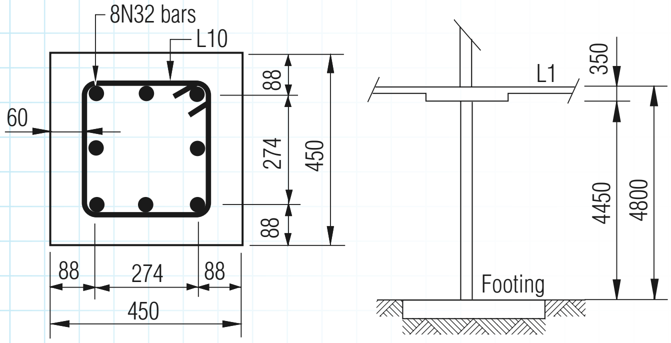
D = 450 mm

However, 420 mm too large $\therefore$ adopt 300 mm

Fitments L10 @ 300

INTERNAL COLUMN C4, FOOTING LEVEL TO LEVEL L1

Column properties



f'_c = 40 MPa

f_{sy} = 500 MPa

AS 3600 Clause 10.1.3 — Braced column, ie lateral loads resisted by shear walls

Design actions (from computer analysis)

N^* = 2,169 x 1.2 + 898 x 1.5 = 3,950 kN

M^*ns_{max} = 42 x 1.2 + 21 x 1.5 = 82 kN.m

M^*ew_{max} = 3 x 1.2 + 1 x 1.5 = 5 kN.m

Check if column short

Calculate L_e/r

Clause 10.5.3 — Assume $L_e = 0.85\,L_u$ = 0.85 x 4,450 = 3,783 mm

DESIGN NOTE: Calculate L_e from Clause 10.5.3 rather than Clause 10.5.4 as restraint at footing uncertain.

Clause 10.5.2 — $r = 0.3\,D$ = 0.3 x 450 = 135 mm

$\therefore L_e/r$ = 3,783/135 = 28

Determine limit for L_e/r *for the column to be classed as short*

AS 3600 Clause 10.3.1(a) — = 25; or

$\leq \alpha_c\,(38 - f'_c/15)\,(1 + M^*_1/M^*_2)$

whichever is greater

$N_{uo} = \gamma_1\, f'_c\, A_c\, f_{sy}\, A_s$ = 10,257 kN

$N^*/0.60\,N_{uo}$ = 3,950 / 0.6 x 10,257

= 0.64 > 0.15

$\therefore \alpha_c = \sqrt{}\,(2.25 - 2.5\,N^*/0.6\,N_{uo})$

= $\sqrt{}$ (2.25 -2.5 x 0.64) = 0.81

Check if $M^*_2 > 0.05\,DN^*$

$0.05\,DN^*$ = 0.05 x 0.45 x 3,950

= 88.9 kN.m > M^*_2 (82 kN.m)

As less than the minimum value and column in single curvature

take $M^*_1/M^*_2 = -1$

$\therefore$ limit = 25

L_e/r = 28

> 25 $\therefore$ column is slender

Determine moment magnifier

Clause 10.4.2 — $\delta_b = k_m/(1.0 - N^*/N_c) \geq 1.0$

Calculate $k_m = (0.6 - 0.4\, M^*_1/M^*_2)$

as above $M^*_1/M^*_2 = -1$

$\therefore\ k_m = 1.0$

$N^* = 3{,}950$ kN

Clause 10.4.4 — Determine $N_c = (\pi^2/L_e^{\,2})\ [182\, d_o(\phi M_{ub})/(1 + \beta_d)]$

Determine $\beta_d = G/(G + Q)$

$= 2{,}169/\ (2{,}169 + 898) = 0.707$

Assume column reinforced on 4 faces with 9 N32, ie 3 bars in each face

$= 3.57\% < 4\%\ \ \therefore$ OK

$f'_c = 40$ MPa

From a program for the design of columns $M_{ub} = 614$ kN.m

$\therefore\ \phi M_{ub} = 0.6 \times 614 = 368.4$ kN.m

$L_e = 3{,}783$ mm

$d_o = 362$ mm

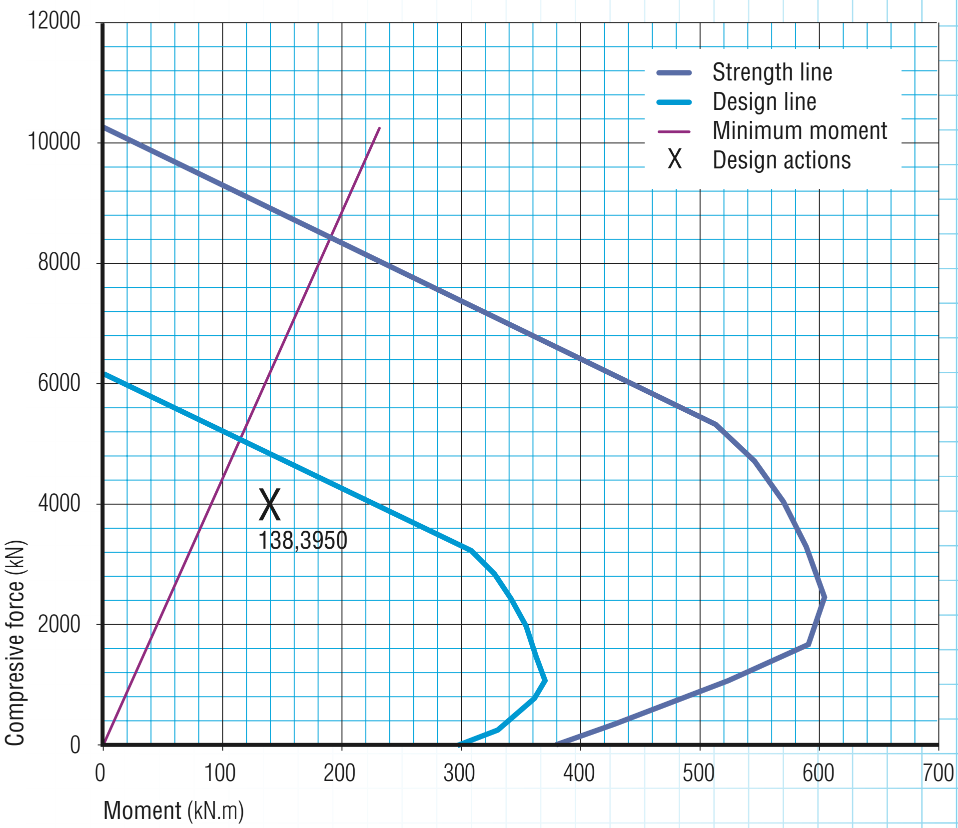
$\therefore\ N_c = (\pi^2/3.7832) \times [182 \times 0.362 \times 368/(1 + 0.71)] = 9{,}778$ kN

$\therefore \delta_b = 1.0/(1.0 - 3{,}950/9{,}778) = 1.68$

$\therefore M^* = 1.68 \times 82 = 138.1$ kN.m

See interaction diagram below for bending in one direction so minimum moments in the other direction will need to be considered. By inspection should be OK for bending in the other direction.

DESIGN NOTE: Column software will probably give marginally more accurate results and will probably cover biaxial bending)



$\therefore$ Adopt 9 – N32, 3 in each face

Clause 10.7.4.3

Determine size and spacing of fitments

Use N12

Spacing = 15 d_b

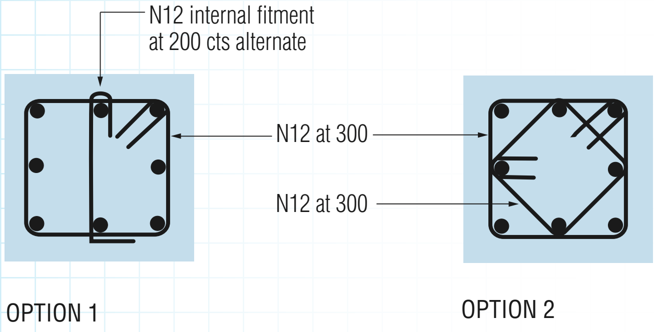
$= 15 \times 32 = 480 > D$

D = 450 mm

But adopt closer spacing for fitments better confinement, eg 300 mm.

Provide fitments N12 @ 300 restraint to middle bars as per Cl 10.7.4.1 of AS 3600

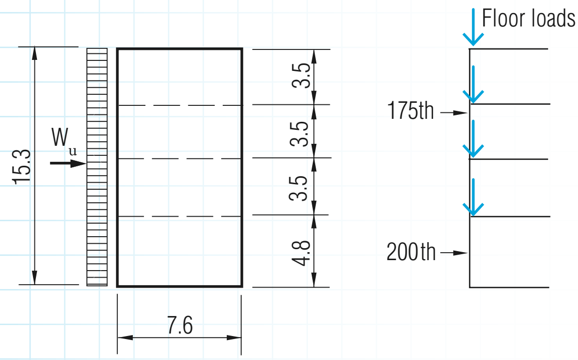
Note: adopt Option 2 as better arrangement to place concrete into the column.



DESIGN NOTE: For external columns, provide lateral shear reinforcement through the joint as required by Clause 10.7.4.5.

WALL D1 – D2

Wall properties



f'_c = 40 MPa

f_{sy} = 500 MPa

Clause 11.3

Wall is braced

Design loads. Refer to following rundown of actions for this wall

$G = 1.20 \times 1{,}958 = 2{,}350$ kN

$Q = 1.5 \times 935 = 1{,}403$ kN

Ultimate load = 3,752 kN = 493.7 kN/m

Check compression stress in wall at bottom of the wall

$= 3{,}752/0.2 \times 7.6/1{,}000 = 2.47$ MPa $\therefore$ wall is not heavily loaded

Also Clause C5.3 compression stress $< 0.15 f'_c = 6$ MPa

$\therefore$ no boundary elements are needed

Wind load W_u (from other computations) = 248 kN

Table 1.1 RCDH

Check stability for overturning parallel to the wall

$W_u \times 15.3/2 = 248 \times 15.3/2 = 1{,}897$ kN.m

$0.9\ G \times 7.6/2 = 0.9 \times 1{,}958 \times 7.6/2 = 6{,}696$ kN.m

$>1{,}897$ kN.m $\therefore$ no overturning, ie stable

Check if any tension in the wall. Tensile stress = M / Z

$M^* = 1{,}897$ kN.m

$Z = bd^2/6$

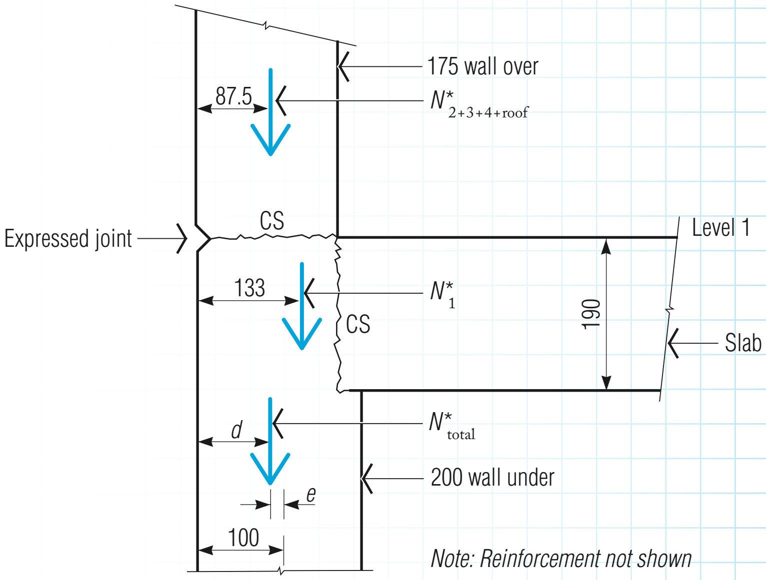
$\therefore f_t = 1{,}897 \times 10^6/200 \times 7{,}600^2/6 = 0.98$ MPa < 2.47 MPa $\therefore$ wall is not in tension

Clause 11.2.1(a)

Wall subject to in plane vertical and horizontal forces. Design for vertical and horizontal forces independently. Use Cl 11.5 for axial forces and Cl 11.6 for shear.

Design for vertical forces at change of section from 175 mm to 200 mm thickness at Level 1. Refer to the wall rundown following.

Determine load eccentricity, e at level 1



$e = (d - 100)$ mm

$d = (N^*_{2+3+4+\text{roof}} \times 87.5 + N^*_1 \times 133.3) / N^*_{\text{total}}$

$= [(14.27 \times 1.2 + 790 \times 1.5) \times 87.5 + (532 \times 1.2 + 145 \times 1.5) \times 133.5] / 3{,}228 = 98.0$ mm

$\therefore e = 98.0 - 100 = -2$ mm

Clause 11.5.2

$< 0.05\, t_w$

$= 0.05 \times 200 = 10$ mm

$\therefore$ use $e = 10$ mm

Clause 11.5.1

Calculate the design axial strength of the wall using the simplified design method for walls subject to vertical compression forces

Calculate $H_{we} = 1.0 \times 4.8 = 4.8$ m

$e_a = H_{we}^2/2{,}500\, t_w$

$= 4.8 \times 4.8 \times 10^6 / (2{,}500 \times 200) = 46.08$ mm

$f'_c = 40$ MPa

$t_w = 200$ mm

$\phi N_u = \phi(t_w - 1.2e - 2e_a)\, 0.6 f'_c$

$= 0.6 \times (200 - 1.2 \times 10 - 2 \times 46.08) \times 0.6 \times 40 = 1{,}381$ kN/m

> 494 kN/m $\therefore$ well OK

DESIGN NOTE: The RCDH Excel Spreadsheet 7.2 for the design of walls for axial loads using the simplified method can be used for the above calculations.

WALL ACTIONS AND BENDING MOMENTS (wall rundown) D1 – D2

Load element	Unit area length	Permanent actions (DL)	Imposed actions (LL)	Permanent axial actions (DL)	Permanent bending moments E/W	Permanent bending moments N/S	Imposed axial actions (LL)	Imposed bending moments E/W	Imposed bending moments N/S
ON LEVEL 4									
1 Roof	25.9	0.5	0.25	13.0			6.5		
2 Wall LW	7.6	1.0		7.6			0.0		
3 Moment shears	0	0	0	0.0			0.0		
Total this level				20.6	0	0	6.5	0	0
Total on Level 4		DL	20.6						
		LL	6.5						
ON LEVEL 3									
1 Floor	25.9	6.0	7.5	155.4	3	42	194.3	1	21
2 Wall 175 thick	27.36	4.4	0.0	119.7			0.0		
3 Edge beam	7.2	26.9	9.3	193.7			67.0		
4 Moment shears	0	0.0	0.0	0.0			0.0		
5 Live load reduction	25.9	0.0	0.0	0.0			0.0		
Total this level				468.8	3.0	42.0	261.2	1.0	21.0
Total on Level 3		DL	489.3						
		LL	267.7						
ON LEVEL 2									
1 Floor	t25.9	6.0	7.5	155.4	3	42	194.3	1	21
2 Wall 175 thick	27.36	4.4	0.0	119.7			0.0		
3 Edge beam	7.2	26.9	9.3	193.7			67.0		
4 Moment shears	0	0.0	0.0	0.0			0.0		
5 Live load reduction	25.9	0.0	0.0	0.0			0.0		
Total this level				468.8	3.0	42.0	261.2	1.0	21.0
Total on Level 2		DL	958.1						
		LL	528.9						
ON LEVEL 1									
1 Floor	25.9	6.0	7.5	155.4	3	42	194.3	1	21
2 Wall 175 thick	27.36	4.4	0.0	119.7			0.0		
3 Edge beam	7.2	26.9	9.3	193.7			67.0		
4 Moment shears	0	0.0	0.0	0.0			0.0		
5 Live load reduction	25.9	0.0	0.0	0.0			0.0		
Total this level				468.8	3.0	42.0	261.2	1.0	21.0
Total on Level 1		DL	1,426.9						
		LL	790.1						
ON FOOTING									
1 Floor	25.9	6.0	3.0	155.4	3	42	77.7	1	21
2 Wall 200 thick	36.48	5.0	0.0	182.4			0.0		
3 Edge beam	7.2	26.9	9.3	193.7			67.0		
4 Moment shears	0	0.0	0.0	0.0			0.0		
5 Live load reduction	25.9	0.0	0.0	0.0			0.0		
Total this level				531.5	3.0	42.0	144.7	1.0	21.0
Total on Footing		DL	1,958.4						
		LL	934.8						
		Total DL	**1,958 kN**						
		Total LL	**935 kN**						

Notes: — Actions (loads) are in kN or kPa. All loads are unfactored.
— Moments are in kN.m.

DESIGN WALL FOR IN-PLANE HORIZONTAL SHEAR FORCES

Clause 11.6

$V^* = 248$ kN

$f'_c = 40$ MPa

Calculate $H_w / L_w = 4.8/7.6 = 0.63$

Calculate $H_{we} / t_w = 4.8/0.2 = 24 \leq 50$ ∴ OK

Calculate $\phi V_{u.max} = 0.2 f'_c (0.8 L_w t_w)$

$= \phi \times 0.2 \times 40 \times 0.8 \times 7{,}600 \times 200/1{,}000 = 6{,}810$ kN >> 248 kN

Clause 11.6.3

Calculate shear strength of a wall without shear reinforcement

$H_w / L_w = 0.63 \leq 1$ therefore use Clause 11.6.3 (a)

Clause 11.6.3 (a)

$\phi V_{uc} = \phi (0.66 \sqrt{f'_c} - 0.21 H_w / L_w \sqrt{f'_c})\, 0.8 L_w t_w$

$= 0.7 \times (0.66 \times \sqrt{40} - 0.21 \times 0.63 \times \sqrt{40}) \times 0.8 \times 7{,}600 \times 200/1{,}000$

$= 2{,}840$ kN >> 248 kN without the contribution of the shear reinforcement which is $\phi V_{us} = 1192$ kN ∴ shear capacity of wall is well OK

DESIGN NOTE: The RCDH Excel Spreadsheet 7.3 can be used for the design of walls for shear for the above calculations.

∴ As no reinforcement required for shear use minimum reinforcement from Cl 11.7.1. As wall thickness is 200 mm only need to have only 1 layer of reinforcement.

CHECK BENDING TO WALL UNDER LATERAL WIND LOADS

Clause 11.1 (b)

Note the stress in the wall in the middle is ≈ 2.06 MPa which is about 3% over the 2 MPa limit and as no reduction for moment shears taken, so OK and design as a slab.

$M^* \approx w l^2/10 = 0.97 \times 4.8^2/10 = 2.23$ kN.m/m

For N12@300 $A_{st} = 377$ mm^2 and $\phi M_u = 14.7$ kN.m > 2.23 kN.m ∴ well OK

DESIGN NOTE: Designers do not need to do sophisticated analysis for such simple load cases. They should try to use simple design methods, where possible, as they are in many cases quicker than running a computer program. Clause 6.10.2.3 using the simplified method of analysis was used for the above calculation which would have given a moment of $w l^2/11$ but was rounded down to $w l^2/10$ which is slightly conservative. As shown above it does not make a significant difference.

AS 3600 Clause 11.7.1

(a) Vertical

Reinforcement

$A_{s.v} = 0.0015 \times 200 \times 1{,}000 = 300$ mm^2/m

Adopt N12 @ 200 mm = 565 mm^2/m > 300 mm^2/m

Vertically N12@200

DESIGN NOTE: While the minimum vertical reinforcement is nominally required only for cracking with N12 @ 300, both for crack control and robustness N12 @ 200 has been adopted as vertical reinforcement.

(b) Horizontal

$A_{s.h} = 0.0025 \times 200 \times 1{,}000 = 500$ mm^2/m

Table 2.3 RCDH

N12@200 mm = 565 mm^2/m

AS 3600 Clause 11.7.2

Check horizontal reinforcement for crack control

Exposure classification B2

∴ $p = 0.006$

∴ $A_{s.h} = 0.006 \times 200 \times 1{,}000 = 1{,}200$ mm^2/m

Table 2.3 RCDH

N12 @ 90 mm or
N16 @ 160 mm

Horizontally N16@150

FOOTING DESIGN

FOOTING COLUMN C4

DESIGN DATA

Allow bearing pressure (from geotechnical report) q_u = 300 kPa

Design loads

g = 2,169 kN

q = 898 kN

N^* = 3,950 kN

M^* = 0 kN.m

Column dimensions and details

c_1 = 450 mm

c_2 = 450 mm

longitudinal reo to column = 9 N32

Concrete strength footing f'_c = 32 MPa

Concrete cover = 75 mm

Estimate footing size. Assume a square footing side length = $b_{initial}$

∴ $b_{initial} = \sqrt{[(2{,}169 + 898)/300]}$ = 3.2 m

Check footing depth for column bar development length in compression
Cl 13.1.5 and Table 2.11 of RCDH, the development length for 32 mm bar = 700 mm

Initial depth of footing = cover + two layers of 32 mm bar + basic development length.

∴ Initial depth of footing = 75 + 64 + 700 = 839 say 900 mm

∴ Effective depth d_o = 900 – 75 – 32 – 16 = 775 mm

Calculate footing weight and confirm initial estimates of sizes

$w_{f\,initial}$ = 3.2 x 3.2 x 0.9 x 25 = 230 kN

∴ Approx total working load = 2,169 + 898 + 230

= 3,067 + 230

= 3,297 kN

Check $b_{final} = \sqrt{3{,}297/300}$ = 3.32 m OK

∴ Adopt footing 3.35 x 3.35 x 0.9m deep

$w_{f\,final}$ = 3.35 x 3.35 x 0.9 x 25 = 252 kN

∴ Total working load = 2,169 + 898 + 252 = 3,319 kN

Actual bearing pressure = 3,319/3.35/3.35

= 296 kPa < 300 kPa ∴ OK

Actual design bearing pressure on concrete = (3,319 – 252)/3.35/3.35 = 273 kPa

Calculate total load

∴ Total actions N_f^* = (2,169 + 252) x 1.2 + 898 x 1.5 =4,252 kN

∴ Load factor = 4,252/(2,169 + 252 + 898) = 1.28

Check BM capacity as cantilever about face of column

Calculate BM total

$M^* = q_u L^2/2$

= 1.28 x 273 x 3.35 x $[(3.35 - 0.45)/2]^2/2$ = 1,230 kN.m

Use RCDH Excel Spreadsheet 4.1

Calculate A_{st} = 4,710 mm^2 (15 N20)

and ϕM_u = 1,435 kN.m > 1,230 kN.m $\therefore$ OK

AS 3600 Clause 16.3.1 and Table 8.2 of RCDH Min. reo ≈ 1,400 x 3.35 mm^2 = 4,690 < 4,710 $\therefore$ OK

Clause 8.2.4

Check one-way beam shear

$x = [(L_1 - c_1)/2] - d_o$

$= [(3{,}350 - 450)/2] - 775 = 675$ mm

$\therefore V^* = 0.675 \times 3.35 \times 273 \times 1.28 = 790$ kN

Use RCDH Excel Spreadsheet 4.1

$\beta_1 = 0.91$ (no shear reinforcement)

$\beta_2 = \beta_3 = 1.0$

From AS 3600 Clause 8.2,5 (ii)

ϕV_{uc} = 639 kN < 790 kN, NG

$\therefore$ Increase depth to 1,050 mm and d_{om} to 925 mm

ϕV_{uc} = 719 kN > 615 kN $\therefore$ OK

$\therefore$ by inspection one-way shear OK both ways

Clause 9.2.3

Check punching shear with $M^* = 0$ with $d_{om} = 935$

$d_{om} = 785$ mm

Use RCDH Excel Spreadsheet 8.1

V^* punching = 3,255 kN

ϕV_u = 6,974 kN > 3,255 kN $\therefore$OK

As depth has increased, so has the minimum reinforcement to = 5,150 mm^2, ie 17 N20

Check development length of column bars = 1,050 – 75 – 20 – 20 = 935 > 700 $\therefore$OK

Therefore footing C4 3.35 x 3.35 x 1.05 m deep
with 17 N20 both ways bottom with 75 mm cover
to bottom and sides

FOOTING WALL D1 – D2

DESIGN DATA

Allow bearing pressure (from geotechnical report) q_u = 300 kPa

Design loads (see previous wall rundown for actions)

g = 1,958 kN

q = 935 kN

N^* = 3,752 kN

M^* = 0 kN.m

Wall dimensions and details

c_1 = 7,600 mm

c_2 = 200 mm

Longitudinal reo to wall = N12 @ 300 cts

Concrete strength footing f'_c = 32 MPa

Concrete cover = 75 mm

Estimate footing size. Area of footing = (1,958 + 935)/300 = 9.6 m^2

Adopt a footing say 8,000 x 1,300 = 10.4 m^2 > 9.6 m^2

Check footing depth for column bar development length in compression
Cl 13.1.5 and Table 2.11 of the RCDH development length for 12 mm bar = 300 mm

Depth of footing = 75 + 40 + 300 = 415 say 500 mm

∴ Effective depth d_o = 500 – 75 – 20 – 10 = 395 mm

Calculate footing weight and confirm initial estimates

$w_{f\ initial}$ = 8 x 1.3 x 0.5 x 25 = 130 kN

∴ Approx total working load = 1,958 + 935 + 130 = 3,023 kN

Check area = 3,023/300 = 10.07 m^2 > 9.6 m^2

∴ Adopt footing 8.0 x 1.3 x 0.5 m deep

∴ Total working load = 1,958 + 935 + 130 = 3,023 kN

Actual bearing pressure = 3,023/8/1.3 = 291 kPa < 300 kPa ∴ OK

Actual design bearing pressure on concrete = (3,023 – 130)/8/1.3 = 278 kPa

Calculate total load

∴ Total load N^*_f = (1,958 + 130) x 1.2 + 935 x 1.5 = 3,908 kN

∴ Load factor = 3,908/(1,958 + 935 + 130) = 1.29

Check BM capacity as cantilever about face of wall

Calculate BM total

$M^* = q_u L^2/2$

= 1.29 x 278 x 8 x [(1.3 – 0.2)/2]^2/2 = 434 kN.m

Use RCDH Excel Spreadsheet 4.1

Calculate A_{st} = 3,232 mm^2

Use N12 at 200 cts nominal = 4,520 mm^2

DESIGN NOTE: To calculate the number of fitments take the total length of the footing minus the cover each end and divide by 200 and rationalise up ie (8,000 – 140) divided by 200 = 40 bars.

and ϕM_{uo} = 6,876 kN.m > 434 kN.m ∴ well OK

and ϕM_u = 705 kN.m > 434 kN.m ∴ well OK

AS 3600 Clause 16.3.1 and Table 8.2

Min. reo ≈ 700 x 8 = 5,600 mm^2 > 3,232 mm^2

∴ Number of bars = 5,600/113 = 50

∴ adopt = say 53 N12 @150 cts = 5,989 mm^2 > 5,600 mm^2

∴ 53 N12 @ 150 mm nom cts as fitments

Minimum reo along footing = 1,061 x 1.3 = 1,379 mm^2

Provide 7 N16 top and bottom, ie 2,800 mm^2 > 1,379 mm^2 ∴ OK.

Clause 8.2.4 Check one-way beam shear

$x = (L_1 - c_1 - 2d_o)/2$

$= (1{,}300 - 200 - 2 \times 395)/2 = 155$ mm

$\therefore$ $V^* = 0.155 \times 8 \times 278 \times 1.29$ (area x bearing pressure x load factor) = 445 kN

Use RCDH Excel Spreadsheet 4.1

$\beta_1 = 1.32$

$\beta_2 = \beta_3 = 1.0$

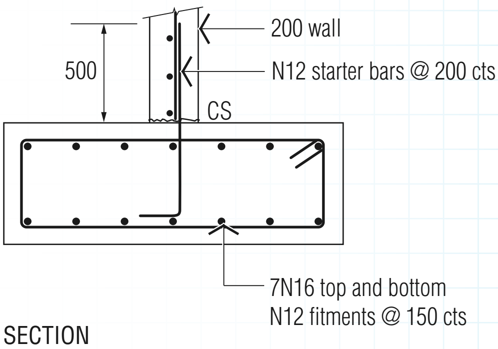
From AS 3600 Clause 8.2,5 (ii)

$\phi V_{uc} = 1{,}095$ kN > 445 kN $\therefore$ no shear reinforcing required.

$\therefore$ by inspection one-way shear OK both ways

Clause 9.2.3 Check punching shear with $M^* = 0$. By inspection well OK.

Footing D1 4 8.9 x 1.3 x 0.5 deep with 7 N16 long way top and bottom with N12 fitments as closed fitments @ 150 cts and 75 mm cover to top, bottom and sides. See Section below.



SECTION

BLANK PAGE

Appendix A_The design process

PURPOSE

This Appendix is to assist the designer in appreciating and understanding some of the issues involved with designing a concrete structure. It should be read in conjunction with Section 1.2 of this Handbook.

CONCEPTUAL DESIGN

The conceptual design phase will involve considering some or more of the following general issues:

- The broad principles of the structure (which may include structural sketch plans) to suit the sketches and layout the architect or designer has proposed and to meet the client's needs
- Basic design information such as site plan, geotechnical information, site constraints, survey, etc
- The likely structural form(s)
- How the services and structure may be integrated
- How it is likely to be constructed
- The likely time frames for design and construction
- Environmental considerations
- Preliminary budget estimate to confirm that the project appears economically viable
- The client's approval for the project to proceed to the next phase.

Designers need to carry out sufficient structural design to ensure that concepts are feasible and to avoid subsequently finding that final design does not work.

PRELIMINARY DESIGN

Following the acceptance of the conceptual design, further development of it, in more detail, in this phase will include:

- Evaluation of different structural options as required, taking into consideration:
  - — Lateral load-resisting systems in two orthogonal directions including shear and core walls
  - — Vertical load-resisting systems, ie walls and columns
  - — Robustness
- Design information such as site plan, site constraints, survey, etc
- Detailed geotechnical and environmental information if possible
- Structural framing and, for floors, indicative sizes based on span-to-depth ratios, some simple design, experience, etc. Designers should refer to *Guide to Long-Span Concrete Floors*[A.1] for initial sizing of concrete floors)
- Likely sizes of structural members including footings, columns and walls including sizing of any precast elements to be used, etc (designers should refer to this Handbook for preliminary sizes for footings, columns, walls and *Precast Concrete Handbook* for preliminary sizes for precast members)
- Coordination with building services
- Movement and construction joints
- Construction sequence and temporary stability if it is an unusual or complicated structure
- Approximate member sizes for alternative designs so that options can be costed to get the optimum solutions.

If necessary, a further budget costing is carried out to confirm the project is on budget.

FINAL DESIGN

The final design stage is where the chosen optimum preliminary design is designed and detailed. This will include the preparation of project documentation and specifications. It is important that the designer remember that the documentation is the means of communicating the design intentions to the contractor/builder and subcontractors, the documentation should be reviewed from this viewpoint before being issued. This stage will include:

- A review of all design data to ensure its validity
- Full analysis of the chosen design for all combinations of lateral and vertical actions. The effect of loads, forces and deformations on the structure and the behaviour of the total structure under the various design load cases are evaluated. Design for durability, fire resistance, deflection and other relevant design loadings should also be carefully considered
- Coordination of the structural design with the design of other aspects of the building, eg hydraulic services and external cladding, including liaising with other members of the project team (the architect, services engineers, etc)
- Full design and detailing of the project. The project must be adequately documented including drawings, conditions of contract and specifications as incomplete documents may delay the project and result in extra costs

- The provision of guidance on how the structure is stabilised during erection of complex or unusual elements such as precast elements. Until lateral stability is achieved by the completed structure, it may be necessary to nominate the sequence for construction to ensure the design concept is not compromised and the structure remains stable during erection
- Independent design checks and Quality Assurance (QA) procedures.

DOCUMENTATION

It is important to detail and document the project sufficiently so that it can be built without undue reference to the designer and to avoid problems with construction. It is well known within the building and construction industry in Australia that poor documentation has led to an inefficient, non-competitive industry, cost overruns, rework, extensions of time, high stress levels, loss of morale, reduced personal output and adversarial behaviour, and with diminished reputations (see *Getting it Right the First Time*[A.2]).

REFERENCES

A.1 *Guide to Long-Span Concrete Floors* (T36) 2nd Ed, Cement Concrete & Aggregates Australia, 2003.

A.2 *Getting it Right the First Time*, Engineers Australia Queensland Division Industry-wide Taskforce on Documentation within the building and construction industry, 2005 (www.qld.engineersaustralia.org.au).

Appendix B_Development and use of the spreadsheets

GENERAL

The spreadsheets in this Handbook have been developed to illustrate design to AS 3600—2009 and the formulae used in the Handbook. They are not intended to be all embracing or to replace commercially available or purpose-written software.

Some of the following discussion on spreadsheets has been based on the Reinforced Concrete Council's project *Spreadsheets for concrete design to BS 8110 and EC2* [B.1] and that source is acknowledged.

The design of concrete structures has been described as time-consuming and costly. Computer programs are now used extensively but experienced designers are often reluctant to rely on 'black box' technology over which they have little control. Computer spreadsheets, on the other hand, are usually user-friendly, generally transparent, powerful, and are popular in structural engineering. They have good graphical presentation facilities and established links with other software, notably word processing. They are an ideal medium to deal with the intricacies of concrete design in that they can carry out a series of mathematical calculations and, as in manual design, can check whether certain conditions are met.

The spreadsheets presented in this Handbook will help students and inexperienced engineers gain an understanding of reinforced concrete design. For the experienced engineer, the spreadsheets will also help in the production of clear and accurate design calculations.

In producing the spreadsheets, Microsoft Excel 2003 was adopted as being the *de facto* standard and the most widely available spreadsheet used.

Designers can also refer to texts such as *The Engineers Tables* [B.2] and *Engineering with the Spreadsheet* [B.3] for guidance when preparing such spreadsheets for engineering calculation.

ADVANTAGES

For the design engineer, these spreadsheets will assist in the preparation of clear and accurate design calculations for individual reinforced concrete elements.

Spreadsheets allow users to gain experience by studying their own 'what if' scenarios. Should they have queries, users should be able to answer their own questions by chasing through the cells to give them an understanding of the logic used. Cells within each spreadsheet can be interrogated, formulae checked and values traced. Engineers are sometimes criticised for not thoroughly costing their designs. With spreadsheets, it is a very simple matter to multiply the quantities of reinforcement, concrete and formwork required by current rates to give an idea of material costs.

USE

Spreadsheets are a very powerful tool. Their use is increasingly common in the preparation of design calculations. They can save time, money and effort. They provide the facility to optimise designs and can help gain experience. However, these benefits have to be weighed against the risks involved which must be recognised and managed. In other words, appropriate levels of supervision and checking, including self-checking must, as always, be exercised when using spreadsheets.

In its deliberations, the Standing Committee on Structural Safety (SCOSS) [B.4] noted the increasingly widespread availability of computer programs and circumstances in which their misuse could lead to unsafe structures. These circumstances include:

- People without adequate structural engineering knowledge or training may carry out the structural analysis.
- There may be communication gaps between the design initiator, the computer program developer and the user.
- A program may be used out of context.
- The checking process may not be sufficiently fundamental.
- The limitations of the program may not be sufficiently apparent to the user.
- For unusual structures, even experienced engineers may not have the ability to spot weaknesses in programs for analysis and detailing.

The committee's report continued: *Spreadsheets are, in principle, no different from other software …*

LIABILITY

A fundamental condition of the use of the spreadsheets in this publication is that the user accepts responsibility for the input data and output of the program/spreadsheet, its interpretation and how they are used.

As with all software, users must be satisfied with the answers these spreadsheets give and be confident in their use. These spreadsheets can never be fully validated for every situation but have been through some testing, both formally and informally. However, users must satisfy themselves that the uses to which the spreadsheets are put are appropriate.

CONTROL

Users and managers should be aware that spreadsheets can be changed and must address change control and versions for use. The flexibility and ease of use of spreadsheets, which account for their widespread popularity, also facilitate ad-hoc and unstructured approaches to their subsequent development.

Quality Assurance procedures may dictate that spreadsheets are treated as controlled documents and subject to comparison and checks with previous methods prior to adoption. Users' Quality Assurance schemes should address the issue of changes. The possibilities of introducing a company's own password to the spreadsheets and/or extending the revision history contained within the sheet entitled *Notes* might be considered.

APPLICATION

The spreadsheets have been developed with the goal of producing calculations to show compliance with AS 3600—2009. Whilst this is the primary goal, there is a school of thought that designers are primarily responsible for producing specifications and drawings which work on site and are approved by clients and/or checking authorities. Producing calculations happens to be a secondary exercise, regarded by many experienced engineers as a hurdle on the way to getting the project approved and completed. From a business process point of view, the emphasis of the spreadsheets might, in future, change to establishing compliance once members, loads and details are known. Certainly, this may be the preferred method of use by experienced engineers.

The spreadsheets have been developed with the ability for users to input and use their own preferred material properties, bar sizes, etc. However, user preferences should recognise efficiency through standardisation.

It is up to the user what use is made of the output. The spreadsheets have been produced to cater for both first-time designers and the more experienced designers without putting off first-time designers.

SUMMARY

With spreadsheets, long-term advantages and savings come from repeated use but there are risks that need to be managed. Spreadsheets demand an initial investment in time and effort – but the rewards are there for those who make this investment. Good design requires sound judgement based on competence derived from adequate training and experience – not just computer programs.

REFERENCES

B.1 *Spreadsheets for concrete design to BS 8110 and EC2,* Reinforced Concrete Council, Department of Trade and Industry, UK, 2000.

B.2 Mote Dr R *The Engineer's Tables,* Trafford Publishing, 2009.

B.3 Christy CT *Engineering with the Spreadsheet,* ASCE Press, 2006.

B.4 SCOSS – the Standing Committee on Structural Safety http://www.scoss.org.uk/.

BLANK PAGE

BLANK PAGE