AS 2870—2011

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Australian Standard®

Residential slabs and footings

STANDARDS Australia

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- Australian Building Codes Board
- Australian Chamber of Commerce and Industry
- Australian Geomechanics Society
- Australian Institute of Building Surveyors
- Cement Concrete and Aggregates Australia
- Concrete Masonry Association of Australia
- Construction Industry Advisory Council
- Engineers Australia
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- Housing Industry Association
- Master Builders Australia
- National Timber Development Council
- Plastics and Chemicals Industries Association
- Steel Reinforcement Institute of Australia
- Think Brick Australia
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Australian Standard®

Residential slabs and footings

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PREFACE

This Standard was prepared by the Standards Australia Committee BD-025, Residential Slabs and Footings, to supersede AS 2870—1996.

The objective of this Standard is to specify performance criteria and specific designs for footing systems for foundation conditions commonly found in Australia and to provide guidance on the design of footing systems by engineering principles.

This Standard places particular emphasis on design for reactive clay sites susceptible to significant ground movement due to moisture changes. The Standard takes account of the following:

(a) Swelling and shrinkage movements of reactive clay soils due to moisture changes.

(b) Settlement of compressible soils or fill.

(c) Distribution to the foundation of the applied loads.

(d) Tolerance of the superstructure to movement.

Notes are included for clarification and general advice only and are not part of the mandatory provisions of the Standard.

Changes to the previous edition are as follows:

(a) Revision of the overall Standard.

(b) Site Class H split into Classes H1 and H2.

(c) New Appendix H Guide to design of footings for trees.

The terms 'normative' and 'informative' have been used in this Standard to define the application of the appendix to which they apply. A 'normative' appendix is an integral part of a Standard, whereas an 'informative' appendix is only for information and guidance.

The Figures in this Standard are intended to show only the structural proportions of the footing system. All other details are purely illustrative.

Commentary to this Standard has been included at the back of this document. The Commentary is for information and advice only, and does not form part of the mandatory body of the Standard.

The layout of the Commentary follows that of the Standard. The numbering differs only in that its clauses, figures and tables are prefixed by the letter 'C', e.g. Clause C3.2.1 of this Commentary refers to Clause 3.2.1 of the Standard. Where there is no commentary to a Clause of the Standard it does not appear, therefore the Clause numbers in this Commentary are not consecutive. References to various publications and papers are listed as the last item of the Section or Appendix in which they occur. Section C7 provides recommendations not given in the Standard.

The Commentary is for information and advice only.

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STANDARDS AUSTRALIA

Australian Standard
Residential slabs and footings

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE

This Standard sets out the criteria for the classification of a site and the design and construction of a footing system for a single dwelling house, townhouse or similar structure which may be detached or separated by a party wall or common wall, but not situated vertically above or below another dwelling, including buildings classified as Class 1 and Class 10a in the Building Code of Australia.

The Standard may also be used for other forms of construction, including some light industrial, commercial and institutional buildings if they are similar to houses in size, loading and superstructure flexibility. The footing systems for which designs are given include slab on ground, stiffened rafts, waffle rafts, strip footings, pad footings and piled footings.

NOTE: This Standard gives no advice on detailing of the connection of superstructures to the footing systems for wind loads or earthquake loads.

For design purposes, the life of the structure is taken to be 50 years.

NOTES:

1 This Standard has been widely used for a number of years for the economical design of footings and slabs. Economical designs that avoid significant damage are practicable only if the soil moisture content of the foundation material under the footing or slab is stable or within reasonable limits of stability over the design life of the house or structure. For all sites (in particular sites with reactive soils) drainage and soil moisture conditions around the building need to be managed to avoid abnormal moisture conditions, as outlined in Clause 1.3.3, which may result in building damage.

2 Site management recommendations are given in Appendix B.

3 Where slab on ground construction is used for long slabs and large houses, particular consideration in design may be needed to avoid significant damage.

4 Information on earthquake actions is included in AS 1170.4. Information on wind actions is included in AS/NZS 1170.2 and AS 4055.

1.2 APPLICATION

To comply with this Standard—

(a) all sites shall be classified in accordance with Section 2; and

(b) footing system design shall be by either—

  (i) prescribing a standard design in accordance with Section 3; or

  (ii) applying the engineering principles described in Section 4; and

(c) all design and construction shall comply with Sections 5 and 6.

Residential footing system design, detailing and construction shall also comply with AS 3600 except that, where in conflict, this Standard (AS 2870) shall take precedence.

NOTE: The functions of the various parties included in the design and construction of residential slabs and footings are normally as described in Appendix A.

1.3 PERFORMANCE OF FOOTING SYSTEMS

1.3.1 General

Buildings supported by footing systems designed and constructed in accordance with this Standard on a normal site (see Clause 1.3.2) that is—

(a) not subject to abnormal moisture conditions; and

(b) maintained such that the original site classification remains valid and abnormal moisture conditions do not develop (see Note 1);

are expected to experience usually no damage, a low incidence of damage category 1 and an occasional incidence of damage category 2 (see Note 2).

Classification of damage shall be as defined in Appendix C.

NOTES:

1 Appendix B provides information and guidance on the maintenance of site foundation conditions.

2 Class A sites (as defined in Section 2) are not reactive to moisture and may have a lesser risk of damage to buildings constructed thereon.

1.3.2 Normal sites

Normal sites are those that are classified as one of Classes A, S, M, H1, H2 and E in accordance with Section 2 of this Standard and where foundation moisture variations are those caused by seasonal and regular climatic effects, effect of the building and subdivision, and normal garden conditions without abnormal moisture conditions (see Clause 1.3.3).

NOTES:

1 The application of the recommendations in Appendix B is expected to provide normal garden conditions.

2 Normal sites can be expected to be adversely impacted by irregular climatic effects—this could include prolonged droughts.

1.3.3 Abnormal moisture conditions

Abnormal moisture conditions are those that result in foundation moisture variations beyond those for normal sites (see Clause 1.3.2). Buildings constructed on sites subject to abnormal moisture conditions have a higher probability of damage than those described in Clause 1.3.1.

In the following examples, the identified factor may result in abnormal moisture conditions where the feature is sufficiently close to affect the ground moisture under the building and/or the event was sufficiently recent that the effect on ground moisture will be present at the time of construction.

Examples of abnormal moisture conditions existing prior to construction include the following:

(a) Removal of an existing building or structure likely to have significantly modified the soil moisture conditions under the footprint of the footing system of the building.

(b) Removal of trees prior to construction.

(c) Presence of trees on the building site or adjacent site.

(d) Unusual moisture conditions caused by drains, channels, ponds, dams, swimming pools, effluent disposal areas or tanks, which are to be maintained or removed from the site.

Examples of abnormal moisture conditions resulting from construction include the following:

(i) Failure to provide adequate site drainage.

(ii) Failure to detail or construct drainage in accordance with this Standard.

Examples of abnormal moisture conditions developing after construction include the following:

(A) The effect of trees too close to a footing.

(B) Excessive or irregular watering of gardens adjacent to the building.

(C) Failure to maintain site drainage.

(D) Failure to repair plumbing leaks.

(E) Loss of vegetation from near the building.

NOTE: Advice related to the effect of trees on footings is included in Appendix H.

1.4 DESIGN CONDITIONS

1.4.1 General

The design conditions specified in Clauses 1.4.2 and 1.4.3 for beams and slabs supported by the foundation on normal sites shall apply.

For other than normal sites, the design of the footing system shall be by engineering principles to ensure the footings perform in accordance with Clause 1.3. Design considerations that are particular to the site shall be considered.

1.4.2 Design action effects

Design for serviceability and safety against structural failure or bearing failure shall be based on design actions due to—

(a) permanent action plus 0.5 imposed action; and

(b) foundation movement.

The permanent and imposed actions to be resisted shall be in accordance with AS/NZS 1170.1.

Foundation movement shall be assessed as the movement that has less than 5% chance of being exceeded in the life of the building, which is taken to be 50 years.

Design soil suction profiles shall be based on this concept and the values of soil suction given in Section 2 are deemed to comply with this requirement.

Design for uplift shall be based on design action effects due to 0.9 permanent action plus wind action.

NOTE: For the wind actions to be resisted, see AS/NZS 1170.2 or AS 4055.

Reactive soil movements and soil settlements shall be determined from permanent action plus 0.5 imposed action.

Soil parameters shall be taken as mean values for each soil stratum.

Design bearing capacity, including uplift, shall be not more than 0.33 multiplied by the ultimate bearing pressure. Design bearing capacity shall take into consideration both the site conditions and the ability of the building system to accommodate load-related settlement.

1.4.3 Other design considerations

The design of footing systems shall consider the following:

(a) Effective drainage of the site.

(b) Past satisfactory performance of similar footings on similar sites.

(c) Control, but not prevention, of shrinkage cracking.

(d) Control, but not prevention, of cracking due to footing movement.

(e) Stiffness and ductility of the footing system.

(f) Strength of the wall system.

(g) Tolerance of the wall system to movement.

(h) Foundation bearing properties.

1.5 DEEMED-TO-COMPLY STANDARD DESIGNS

The standard designs given in Section 3 are deemed to comply with the performance criteria in Clause 1.3.

1.6 ARTICULATION REQUIREMENTS

Where the standard designs given in Section 3 are for articulated masonry veneer and articulated full masonry, articulation joints shall comply with the requirements of AS/NZS 4773.2 and TN 61.

1.7 NORMATIVE REFERENCES

The following are the normative documents referenced in this Standard.

NOTE: Documents referenced for informative purposes are listed in Appendix I.

AS	
1289	Methods of testing soils for engineering purposes
1289.6.3.3	Method 6.3.3: Soil strength and consolidation tests—Determination of the penetration resistance of a soil—Perth sand penetrometer test
1289.7.1.1	Method 7.1.1: Soil reactivity tests—Determination of the shrinkage index of a soil—Shrink swell index
1289.7.1.2	Method 7.1.2: Soil reactivity tests—Determination of the shrinkage index of a soil—Loaded shrinkage index
1289.7.1.3	Method 7.1.3: Soil reactivity tests—Determination of the shrinkage index of a soil—Core shrinkage index
1379	Specification and supply of concrete
1684	Residential timber-framed construction series
3600	Concrete structures
3700	Masonry structures
3798	Guidelines on earthworks for commercial and residential developments
3799	Liquid membrane forming curing compounds for concrete
AS/NZS	
1170	Structural design actions
1170.1	Part 1: Permanent, imposed and other actions
2904	Damp-proof courses and flashings
4347	Damp-proof courses and flashings—Methods of test
4347.6	Method 6: Determining impact resistance (falling dart impact test)
4347.9	Method 9: Determining thickness
4671	Steel reinforcing materials
4773	Masonry in small buildings
4773.1	Part 1: Design
4773.2	Part 2: Construction

Australian Building Codes Board
BCA Building Code of Australia

Cement Concrete and Aggregates Australia
TN 61 Articulated walling

CSIRO
Method for the determination of the penetration resistance of water vapour barriers to falling aggregate

1.8 DEFINITIONS

For the purposes of this Standard, the definitions below apply.

1.8.1 Articulated full masonry

Full masonry construction incorporating articulation of external and internal walls.

1.8.2 Articulated masonry veneer

Masonry veneer construction incorporating articulation of the masonry veneer.

1.8.3 Articulation

Provision for movement in walls through incorporation of permanent control joints.

1.8.4 Braced stump

Stump that forms part of a bracing element that resists lateral loads through diagonal members attached between two or more stumps.

1.8.5 Bracing stump

Stump that, in addition to vertical loads, resists horizontal loads applied more than 150 mm above ground level.

1.8.6 Bored pier

Cast in place concrete cylindrical load support element.

1.8.7 Bulk pier

Cast in place concrete load support element excavated by backhoe or similar machinery.

1.8.8 Characteristic surface movement (y_s)

Movement of the surface of a reactive site caused by moisture changes from characteristic dry to characteristic wet condition in the absence of a building and without consideration of load effects.

1.8.9 Clad frame

Timber or metal frame construction with the exterior wall clad with timber or sheet material not sensitive to minor movements. Includes substructure masonry walls up to 1.5 m high.

1.8.10 Clay

Fine-grained soil with plastic properties when wet. Includes gravelly, sandy or silty clays.

1.8.11 Collapsing soil

Weakly cemented soil subject to large settlements under load as a result of degradation by water on the cementing action.

1.8.12 Concrete wall panel

Precast (including tilt-up) or cast in place concrete wall designed to act as a unit and separated from adjacent panels or walls by a control joint.

1.8.13 Controlled fill

Fill that will be required to support structures or associated pavements, or for which engineering properties are to be controlled.

1.8.14 Design bearing capacity

The maximum bearing pressure that can be sustained by the foundation from the proposed footing system under service loads over the design range of soil moisture conditions.

1.8.15 Earth wall construction

Unfired earth bricks or blocks and unfired rammed earth wall construction.

1.8.16 Edge beam

Beam at the edge of a slab on ground or stiffened raft.

1.8.17 Edge footing

Footing at the edge of a footing slab.

1.8.18 Engineering principles

Principles of geotechnical and structural engineering as applicable for the purposes of this Standard.

NOTE: This means engineering principles that are commonly accepted by qualified engineers.

1.8.19 Extension

Additional construction abutting an existing building.

1.8.20 Fill depth

For a slab, depth measured from the underside of the footing to the natural surface level. For a strip or pad footing system, depth measured from the finished ground level to the natural surface level.

1.8.21 Finished ground level

Ground level adjacent to the footing system at the completion of construction and landscaping.

1.8.22 Fitment

Tie, ligature or stirrup reinforcement.

1.8.23 Footing

Construction that transfers the load from the building to the foundation.

1.8.24 Footing slab

Concrete floor supported on the ground with a separately poured edge strip footing.

1.8.25 Footing system

General term used to refer to slabs, footings, piers and pile systems that transfer load from the superstructure to the foundation.

1.8.26 Foundation

Ground that supports the footing system.

1.8.27 Framed double-leaf masonry

Construction with masonry double-leaf external wall and framed internal walls.

1.8.28 Full masonry

Construction with masonry double-leaf external walls and masonry single-leaf internal walls without full articulation.

1.8.29 Gilgai

Soil surface feature associated with reactive clay sites, characterized by regularly spaced and sized depressions on virgin land.

NOTE: Gilgais are formed by extreme, reactive soil movements. Soil profiles may vary markedly across sites with gilgais.

1.8.30 Infill slab

Slab cast on the ground between walls.

1.8.31 Landslip

Foundation condition on a sloping site where downhill foundation movement or failure is a design consideration.

1.8.32 Loadbearing wall

Wall imposing a load on the footing greater than 10 kN/m, when factored in accordance with Clause 1.4.2.

1.8.33 Masonry

Stone, brick, terracotta block, concrete block, or other similar building unit, single or in combination, assembled together unit by unit.

1.8.34 Masonry veneer

Construction consisting of a loadbearing frame clad with an outer leaf of masonry.

1.8.35 Maximum differential footing movement

Maximum movement of a footing relative to a straight line joining the ends of the footing system or, in the case of double curvature, joining the points of contraflexure.

1.8.36 Mine subsidence

Settlement, curvature, tilt and lateral strain, either individually or in combination, produced at the surface as a result of underground mining.

1.8.37 Mixed construction

Building consisting of more than one form of construction.

1.8.38 Natural site

Site that has not been subjected to cutting or filling.

1.8.39 Outbuilding

Detached building such as a carport, private garage, shed or similar structure.

1.8.40 Pad footing

Concrete footing used to support a pier or stump.

1.8.41 Pier-and-beam

Footing system incorporating bored piers, bulk piers or piles and reinforced concrete beams supporting a building where the floor is not integral with the beams.

1.8.42 Pier-and-slab

Footing system incorporating bored piers, bulk piers or piles supporting a suspended slab and including a slab partly supported on piers and partly supported on ground.

1.8.43 Pile

Structural member that is driven, screwed, jacked, vibrated, drilled or otherwise installed in the ground such as to transmit loads to the underlying soil or rock and provide a footing component for a structure.

1.8.44 Reactive site

Site consisting of a clay soil that swells on wetting and shrinks on drying by an amount that can damage buildings on light strip footings or unstiffened slabs. Includes sites classified as Class S, Class M, Class H1, Class H2 or Class E in accordance with Clause 2.1.

1.8.45 Reinforcement

Steel bars, wire or mesh.

1.8.46 Reinforced single-leaf masonry

Outer wall constructed of concrete blocks with some vertically reinforced cores at not greater than 2.0 m centres, such reinforcement being lapped with steel starter bars set in concrete beams or footings and a bond beam.

1.8.47 Rock

1 Strong material, including shaly material and strongly cemented sand or gravel, that does not soften in water or collapse under the combination of loading and wetting.

2 Material that cannot readily be excavated by a backhoe.

1.8.48 Sand

Granular soil that may contain a small proportion of fines including silt or clay. The amount of fines may be assessed as small by a visual inspection or if the amount that passes a 75 µm sieve is 15% or less.

NOTE: Material with a higher proportion of fines should be treated as silt or clay.

1.8.49 Silt

Fine-grained soil that is non-cohesive and non-plastic when wet and may include some sand and clay.

1.8.50 Single-leaf masonry

Outer walls constructed with a single thickness of masonry units.

1.8.51 Single-storey

Construction with wall height, excluding any gable, not exceeding 4.2 m and including only one trafficable floor.

1.8.52 Slab

General term used to refer to slab on ground, stiffened rafts, footing slabs, stiffened footing slabs and waffle rafts.

1.8.53 Slab on ground

Concrete floor supported on the ground and incorporating integral edge beams.

1.8.54 Slab panel

Part of a slab between beams.

1.8.55 Soil suction

Negative pore water pressure in soils, expressed in picofarads (pF).

NOTE: $pF = 1+\log(u)$, where u = total soil suction in kilopascals.

1.8.56 Stiffened raft

Concrete slab on ground stiffened by integral edge beams and, commonly, a grid of internal beams.

1.8.57 Strip footing

Footing of rectangular section.

1.8.58 Stump

Element supported on a footing used for the support of a frame construction.

1.8.59 Superstructure

Portion of a completed building that is supported by the selected footing system, including slab where applicable.

1.8.60 Two-storey

Construction with wall height, including any gable, not exceeding 8.5 m and including two trafficable floors.

1.8.61 Ultimate bearing pressure

Pressure, under normal moisture conditions, at which a footing sinks without increase of load.

1.8.62 Veneer

Construction of either masonry veneer or articulated masonry veneer.

1.8.63 Waffle raft

A stiffened raft with closely spaced ribs constructed on the ground and with slab panels suspended between ribs.

1.9 NOTATION

The symbols used in this Standard are as follows:

Symbol		Definition
A1, A2, B1, B2	=	exposure classification, reinforced or pre-stressed concrete members (per AS 3600)
B	=	footing diameter or width (Tables E1 to E4, Figure E2, Appendix E)
B_w	=	width of stem of edge, or internal beam (Clauses 4.4, 4.5, and 4.6)
D	=	overall depth of a footing or beam (Figures 3.1, 3.4, and 3.6, Clause 4.5.2, Figures 4.1, 5.4 and 5.6)
D_{cr}	=	critical depth (Paragraph F2, Appendix F)
D_e	=	depth of embedment of edge beam (Paragraph F2, Appendix F)
D_f	=	depth of strip footing from finished ground surface level (Figure 3.6)
D_i	=	ground movement influence distance of a tree or trees (Paragraph H2, Appendix H)
D_s	=	depth of pad footing from finished ground surface level (Figure 3.6 and Figures E1 and E2, Appendix E)
D_t	=	distance of tree to the building (Paragraphs H2, H3 and H4, Appendix H)
d	=	differential movement (Table 2.2)
e	=	edge distance (Paragraphs F1 and F2, Appendix F)

E = modulus of elasticity (Clause 4.4)

EC_e = saturated extract electrical conductivity, in deciSiemens per metre (Clause 5.5.3 and Table 5.1)

E_d = design action effect on a pile due to all imposed loads (Paragraph G4.3, Appendix G)

f'_c = design characteristic strength of concrete (Table 5.3)

H = geotechnical capacity of stump for horizontal load (Figure 2.1, Paragraph E3, Figure E2, Appendix E)

H^* = design horizontal load on stump (Paragraph E3, Figure E2, Appendix E)

H_s = depth of design soil suction change (Table 2.4, Figure 2.1 and Paragraph F2, Appendix F)

H_t = maximum design drying depth close to a tree or trees (Paragraphs H3, H4 and Figure H1, Appendix H)

HT = design height of single tree (Paragraphs H2.1, H2.6 and H4, Appendix H)

HT_g = design height of a group of trees (Paragraphs H2.2 and H2.6, Appendix H)

H_{ult} = ultimate strength in horizontal loading (Paragraph E3, Appendix E)

h = thickness of layer under consideration (Clause 2.3)

h_h = drop height of the hammer for driven piles (Paragraph G4.3, Appendix G)

h_w = maximum height of masonry wall retaining structure (Figure 6.3)

I = second moment of area (Clause 4.4)

I_{ps} = shrinkage index or instability index without lateral restraint or loading of soil (Clause 2.3.2)

I_{pt} = instability index, including allowance for lateral restraint and vertical load, (Clauses 2.3.1 and 2.3.2 and Table D1, Appendix D)

k = swelling soil stiffness (Paragraph F2, Appendix F, Paragraph G8, Appendix G)

L = footing or slab length in the design direction (Table 4.1, Clause 4.4)

L_s = minimum distance from internal stump or pier to perimeter stump, pier or wall (Figure 3.6)

M_{cr} = cracking moment capacity (Clause 4.4, and Paragraph H4, Appendix H)

M_u = ultimate bending moment strength (Clause 4.4 and Paragraph H4, Appendix H

M^* = design bending moment at a cross-section (Clause 4.4)

N = number of soil layers within the design depth of suction change (Clause 2.3.1)

p_s = swell pressure under footing (Paragraph G8, Appendix G)

R_{ug} = ultimate geotechnical strength of a pile (Paragraph G4.3, Appendix G)

s = centre to centre distance between group of trees (Paragraph H2.4, Appendix H)

S = pile set for driven piles in metres (Paragraph G4.3, Appendix G)

S_n = nominal beam spacing (Figure 5.5)

t = thickness of slab or pad footing (Figure E1, Appendix E)

u = total soil suction (Clauses 1.8.55 and 2.3.1, Table 2.4)

U = geotechnical capacity of stump in uplift (Paragraph E3 and Figure E2, Appendix E)

U^* = design uplift load on stump (Paragraph E3 and Figure E2, Appendix E)

U_{ult} = ultimate strength in uplift (Paragraph E3, Appendix E)

W = overall width of the slab normal to the direction of the beams being considered (Clause 4.5.2 and Figure 4.1)

W_{f} = shape factor for edge heave (Figures F1 and F2, Appendix F)

W_{h} = hammer weight for driven piles, in kilonewtons (Paragraph G4.3, Appendix G)

X = offset of re-entrant corner (Figure 5.4)

y_{m} = differential mound movement (Paragraph F2, Appendix F and Paragraph H4, Appendix H)

y_{s} = characteristic surface movement (Clauses 1.8.8, 2.2.3, 2.3.1, 4.5.1, 4.5.2, Figure 4.1, Clauses 5.6.4 and 6.6, Table D1, Appendix D, Paragraph G8, Appendix G, Paragraph H4, Appendix H)

y_{t} = potential surface movement due to the tree-induced suction change in addition to the normal design suction change, (Paragraph H4, Appendix H)

y_{tmax} = maximum potential surface movement due to the tree-induced suction change in addition to the normal design suction change (Paragraph H4, Appendix H)

α = lateral restraint factor (Clauses 2.3.1 and 2.3.2)

Δ = differential footing deflection (Table 4.1, Clauses 4.5.1 and 4.5.2)

φ_{g} = geotechnical strength reduction factor (Paragraph G4.3, Appendix G)

Δu = soil suction change (Clause 2.3.1)

Δu_{base} = maximum extra soil suction change caused by the vegetation at the maximum design drying depth (Paragraph H4, and Figure H1, Appendix H)

ϕ = strength reduction factor (Clause 4.4 and Paragraph E3, Appendix E)

1.10 REINFORCEMENT DESIGNATION

1.10.1 Trench mesh

For the purposes of this Standard, trench mesh is designated as x–L8TM, x–L11TM or x–L12TM, where x is the number of main longitudinal bars required of the appropriate trench mesh L8TM, L11TM or L12TM. Trench mesh shall comply with AS/NZS 4671.

1.10.2 Square and rectangular mesh

Square and rectangular meshes referred to in this Standard shall comply with AS/NZS 4671.

Wires designed in accordance with Australian and New Zealand Standards shall comply with the requirements for Grade 500L in accordance with AS/NZS 4671.

NOTE: For square and rectangular meshes referred to in this Standard, the mesh sizes are prefixed with SL and RL respectively.

1.10.3 Reinforcing bars

Reinforcing bars shall comply with AS/NZS 4671 Grade 500N.

NOTE: Reinforcing bars are specified as x–N12, x–N16, or x–N20, where x is the number of bars.

1.10.4 Substitution of reinforcement

The ductility and strength requirements of reinforcement specified in the standard designs of Section 3 shall not be reduced except as provided in Clause 3.7.

1.11 INFORMATION IN DOCUMENTS

1.11.1 Classification report

For class P sites, the classification report shall include the reason for the P classification and recommendations for further investigation, as required, to provide adequate data for footing system design.

1.11.2 Design documents

The site classification shall be stated on the drawings. The selected footing systems and any required site work and required site drainage shall be documented.

SECTION 2 SITE CLASSIFICATION

2.1 GENERAL

2.1.1 Classification

Site classification is based on the expected ground surface movement and the depth to which this movement extends. Sites shall be classified in accordance with Clauses 2.1.2 and 2.1.3 using the techniques and principles specified in Clauses 2.2, 2.3, 2.4 and 2.5.

NOTE: Site classification may require consideration of factors beyond the boundaries of the subject site.

2.1.2 Site classification based on soil reactivity

Classification of sites where ground movement is predominantly due to soil reactivity under normal moisture conditions shall be classified based on the expected level of ground movement as nominated in Table 2.1.

For Classes M, H1, H2 and E, further classification may be required, based on the depth of the expected moisture change. For sites with deep-seated moisture changes characteristic of dry climates and corresponding to a design depth of suction change (H_s) equal to or greater than 3 m, the classification shall be M-D, H1-D, H2-D or E-D as appropriate.

NOTE: For example, M represents a moderately reactive site with shallow moisture changes and M-D represents a moderately reactive site with deep moisture changes.

TABLE 2.1

CLASSIFICATION BASED ON SITE REACTIVITY

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites, which may experience only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which may experience moderate ground movement from moisture changes
H1	Highly reactive clay sites, which may experience high ground movement from moisture changes
H2	Highly reactive clay sites, which may experience very high ground movement from moisture changes
E	Extremely reactive sites, which may experience extreme ground movement from moisture changes

2.1.3 Classification of other sites

Sites with inadequate bearing strength or where ground movement may be significantly affected by factors other than reactive soil movements due to normal moisture conditions shall be classified as Class P. Class P sites include soft or unstable foundations such as soft clay or silt or loose sands, landslip, mine subsidence, collapsing soils and soils subject to erosion, reactive sites subject to abnormal moisture conditions and sites that cannot be classified in accordance with Clause 2.1.2.

A site shall be classified as Class P if—

(a) the bearing strength is less than that specified in Clause 2.4.5;

(b) excessive foundation settlement may occur due to loading on the foundation;

(c) the site contains uncontrolled or controlled fill as identified in Clause 2.5.3;

(d) the sites may be subject to mine subsidence, landslip, collapse activity or coastal erosion;

(e) the site may be subject to moisture changes due to site conditions more severe than the normal site conditions described in Clause 1.3.2; or

(f) the site may be subject to other factors resulting in foundation movement beyond the reactive soil movements resulting from moisture changes due to the normal site conditions described in Clause 1.3.2.

The basis for classification shall be recorded on the site classification report together with recommendations for further geotechnical investigation.

2.2 METHODS FOR SITE CLASSIFICATION

2.2.1 General

Classification of sites other than Class P sites shall adopt one or both of the following methods:

(a) Identification of the soil profile in accordance with Clause 2.2.2.

(b) Site classification based on characteristic surface movement in accordance with Clause 2.2.3.

2.2.2 Identification of the soil profile

Site classification based on identification of the soil profile shall include one or more of the following methods:

(a) Site classification based on typical soil profile data given in Appendix D. The soil profile shall be confirmed by investigation using a borehole(s) or other excavation or sampling method in accordance with Clauses 2.4.3 and 2.4.4. The classification report shall include details of the investigation method used and significant soil profile(s).

(b) Identification of the soil profile and interpretation of the current performance of existing buildings. The soil profile shall be confirmed by inspection of soil from a borehole(s) or other excavation or sampling method in accordance with Clauses 2.4.3 and 2.4.4. Interpretation of the performance of existing residential footing systems within the region that are not less than 10 years old and are founded on a similar soil profile shall be in accordance with Table 2.2. In areas of deep-seated moisture change the site classification shall be modified by the addition of '-D' as described in Clause 2.1.2.

TABLE 2.2

CLASSIFICATION OF NORMAL SITES BY INTERPRETATION OF FOOTING PERFORMANCE OF EXISTING BUILDINGS (see Note 1)

Wall construction	Performance of walls of existing buildings on lightly stiffened strip footing or slab on ground	Site classification in accordance with Table 2.1
Clad frame	Buildings with differential movements (d), in mm (lowest to highest points on perimeter of building)	
	$d \leq 15$	S
	$15 < d \leq 30$	M
	$30 < d \leq 45$	H1
	$45 < d \leq 55$	H2
	$d > 55$	E
Masonry (veneer or full)	Damage Category 0 to Category 1	S or M
	Damage often Category 1, but rarely Category 2	M or H1
	Damage often Category 1 and 2, but rarely Category 3	H1 or H2
	Damage often Category 3 or more severe and area usually well known for damage to buildings and structures	E

NOTES:

1 Damage categories shall be as given in Appendix C.

2 Where performance of existing buildings indicates highly or extremely reactive sites, a further investigation should be carried out for buildings other than small extensions, garages or outbuildings.

3 Timber subfloor structures in clad-frame buildings may have settled due to shrinkage or biodegradation of the timber structure and not only ground movement.

4 Lightly stiffened footings may be taken to be footings as detailed in this Standard for Class A and S sites.

2.2.3 Site classification based on characteristic surface movement

The characteristic surface movements (y_s) estimated in accordance with Clause 2.3 shall be used to determine the site class by applying the limits in Table 2.3. In areas of deep-seated moisture change, the site classification shall be modified by the addition of '-D' as specified in Clause 2.1.2.

TABLE 2.3

CLASSIFICATION BY CHARACTERISTIC SURFACE MOVEMENT (y_s)

Characteristic surface movement (y_s) mm	Site classification in accordance with Table 2.1
$0 < y_s \leq 20$	S
$20 < y_s \leq 40$	M
$40 < y_s \leq 60$	H1
$60 < y_s \leq 75$	H2
$y_s > 75$	E

2.3 ESTIMATION OF THE CHARACTERISTIC SURFACE MOVEMENT

2.3.1 Characteristic surface movement

For site classification purposes, the characteristic surface movement (y_s) shall be determined by estimating the movement of each soil layer 1 to N within the depth of design suction change and summing the movement for all layers, as follows:

$$y_s = \frac{1}{100}\sum_{n=1}^{N}\left(I_{pt}\ \overline{\Delta u}h\right)_n \qquad \text{. . . 2.3.1}$$

where

y_s = characteristic surface movement, in millimetres

α = lateral restraint factor (see Clause 2.3.2)

I_{pt} = instability index, in %/picofarads (pF) (see Clause 2.3.2)

$\overline{\Delta u}$ = soil suction change averaged over the thickness of the layer under consideration, in picofarads (pF)

h = thickness of layer under consideration, in millimetres

N = number of soil layers within the design depth of suction change

The estimation of surface movement shall be based on sufficient soil data to adequately describe the soil profile.

2.3.2 Instability index

The instability index (I_{pt}) is defined as the percent vertical strain per unit change in suction, taking into account the expected values of—

(a) applied stress;

(b) degree of lateral restraint; and

(c) soil suction range.

The instability index is not a constant for a particular clay, but it may be estimated from the soil shrinkage index (I_{ps}). The soil shrinkage index shall be derived using one or more of the following methods:

(i) Laboratory tests for soil reactivity, as set out in AS 1289.7.1.1, AS 1289.7.1.2 and AS 1289.7.1.3.

(ii) Correlations between shrinkage index (I_{ps}) and other clay index tests for the soil type.

(iii) Visual-tactile identification of the soil by a suitably qualified and experienced person.

NOTE: A suitably qualified and experienced person is an engineer or engineering geologist having appropriate expertise and local experience.

For method (iii) above, the suitably qualified and experienced person shall check the soil property identification against laboratory testing on reactive soils at a period not longer than six months and at least once in every 50 sites personally classified.

In the absence of more exact information, the instability index (I_{pt}) shall be estimated from the shrinkage index (I_{ps}) using the following correction:

$$I_{pt} = \alpha \times I_{ps} \qquad \text{. . . 2.3.2(1)}$$

α shall be taken as follows:

(A) In the cracked zone (unrestrained)

$\alpha = 1.0$

(B) In the uncracked zone (restrained laterally by soil and vertically by soil weight)

$$\alpha = 2.0 - z/5 \qquad \ldots 2.3.3(2)$$

where z = the depth from the finished ground level, in metres, to the centroid of the area defined by the suction change profile and the thickness of the soil layer under consideration in the uncracked zone.

In the absence of more exact information, the depth of the cracked zone shall be taken as—

(1) $0.5H_s$ to H_s where H_s is as given in Table 2.4.

(2) $0.75H_s$ in Adelaide and Melbourne; and

(3) $0.5H_s$ in other areas.

For reactive clay in controlled fill placed less than 5 years prior to building construction, the depth of the cracked zone shall be taken as zero. Where a site has been cut less than two years prior to building construction, the depth of the cracked zone shall be reduced by the depth of the cut.

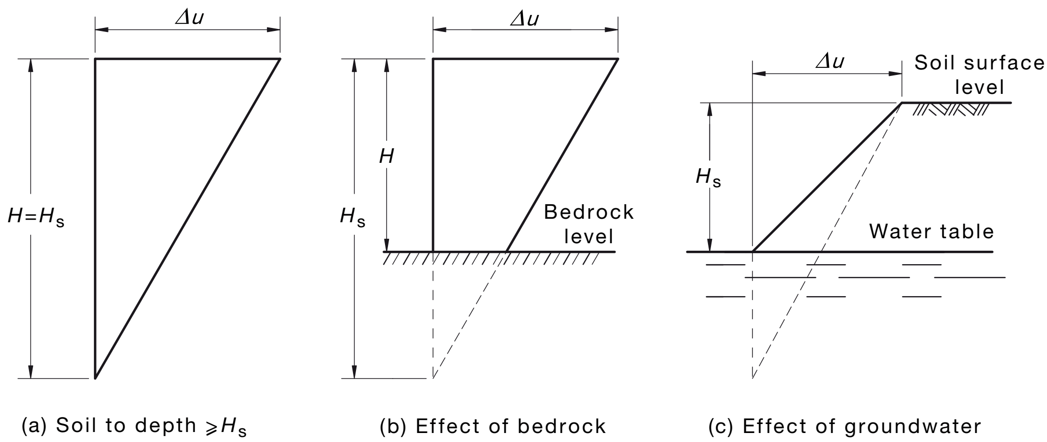
NOTE: The cracked zone relates to the zone in which predominantly vertical shrinkage cracks exist seasonally

2.3.3 Soil suction profile

Values of depth of design soil suction change (H_s) are given in Table 2.4 for various locations. Where a range is given, lower values correspond to wetter climate areas, typically near the coast or in the hills. The classifier may extrapolate to other areas based on climate. Alternatively, the value of H_s may be estimated from the Thornthwaite Moisture Index (TMI) for the region, based on at least 25 continuous years of climate data, using the relationship given in Table 2.5.

Where a permanent water table is encountered within the depth H_s from the surface, the suction change shall be modified in accordance with Figure 2.1. Shallow bedrock shall be treated as a non-reactive soil layer, having no effect on the design suction change as shown in Figure 2.1.

Where the soil profile indicates deep and open shrinkage cracking, the depth (H_s) given in Table 2.4 shall be increased to not less than the depth of cracking.



Δu and H_s are to be taken from Table 2.4 (except that H_s is taken as the depth to water table if it is less than the value in Table 2.4)

FIGURE 2.1 EFFECT OF BEDROCK OR WATER TABLE ON DESIGN SUCTION CHANGE PROFILES

TABLE 2.4

SOIL SUCTION CHANGE PROFILES FOR CERTAIN LOCATIONS

Location	Change in suction at the soil surface (Δu) pF	Depth of design soil suction change (H_s) m
Adelaide	1.2	4.0
Albury/Wodonga	1.2	3.0
Brisbane/Ipswich	1.2	1.5–2.3 (see Note)
Gosford	1.2	1.5–1.8 (see Note)
Hobart	1.2	2.3–3.0 (see Note)
Hunter Valley	1.2	1.8–3.0 (see Note)
Launceston	1.2	2.3–3.0 (see Note)
Melbourne	1.2	1.8–2.3 (see Note)
Newcastle	1.2	1.5–1.8 (see Note)
Perth	1.2	1.8
Sydney	1.2	1.5–1.8 (see Note)
Toowoomba	1.2	1.8–2.3 (see Note)

NOTE: The variation in H_s depends largely on climatic variation.

TABLE 2.5

RELATIONSHIP BETWEEN TMI, DEPTH OF DESIGN SOIL SUCTION CHANGE (H_s) AND CLIMATIC ZONE

TMI	Depth of design suction change (H_s), m	Climatic zone
>10	1.5 m	1
≥−5 to 10	1.8 m	2
≥−15 to ≤−5	2.3 m	3
≥−25 to ≤−15	3.0 m	4
≥−40 to ≤−25	4.0 m	5
≤40	>4.0 m	6

NOTE: Maps of regional climatic zones in Victoria are presented in Figures D1 and D2, Appendix D.

2.4 SITE INVESTIGATION REQUIREMENTS

2.4.1 General

Where a site investigation is required for the purpose stated in Clause 2.4.2, the requirements in Clauses 2.4.3 to 2.4.5 shall be met. An investigation in accordance with this Clause (2.4) may not provide sufficient information to allow design of footings on Class P sites. If, in the opinion of the classifier, the investigation does not include sufficient information for the design of a footing system, a recommendation shall be made regarding further investigation.

2.4.2 Purpose

The purpose of site investigation is to provide sufficient information to enable a site classification to be made, and to collect information on the presence and depth of fill material, natural soil profile, bearing strength and soil reactivity where required.

2.4.3 Depth of investigation

The soil profile shall be examined to a minimum depth below the surface or below the depth of cutting where known at the time of site classification equal to 0.75 times the depth of design suction change (H_s), for the locality, but not less than 1.5 m unless rock is encountered.

2.4.4 Minimum number of exploration positions

The following shall apply to building sites:

(a) A minimum of one borehole or pit per building site.

(b) A minimum of three boreholes per site in localities where $H_s \geq 3.0$ m, and areas where the soil profile is known to be highly variable.

The total number of boreholes across a housing subdivision may be reduced if soil profiling indicates uniform soil conditions.

The presence of gilgais in an area is evidence of highly variable soil profiles within a site.

For sites for extensions or outbuildings, essentially rectangular in plan, with walls articulated at the junction with any other building and not longer than 9 m in either direction, only one borehole is required if the original site classification for the building has proved satisfactory.

2.4.5 Bearing capacity

Determination of adequate bearing strength shall be considered as follows:

(a) The design bearing capacity at foundation level shall be not less than 100 kPa for strip and pad footings and under the edge footing of footing slabs used without tie bars between the edge footing and slab.

(b) The design bearing capacity at foundation level shall be not less than 50 kPa under all beams and slab panels and support thickenings for slab construction.

Determination of bearing capacity shall consider the weakest state of the foundation under normal site conditions. Local knowledge shall be used where available.

NOTE: Inadequate bearing capacity is not common, except for some sites with loose sand, collapsing soils or swampy deep silt deposits.

2.5 ADDITIONAL CONSIDERATIONS FOR SITE CLASSIFICATION

2.5.1 Sites consisting predominantly of sand or rock

Sites consisting predominantly of sand of adequate bearing capacity or rock, as defined in Clause 1.8, shall be classified as Class A.

NOTE: Loose sands may not have adequate bearing capacity for strip or pad footings.

2.5.2 Effect of site works on classification

The classification of a site shall take into account the effect of site works when these are known at the time of classification. Where the effect of site works has not been taken into account, the classification shall be reconsidered if—

(a) the depth of cut on an S, M, H1, H2 or E site exceeds the lesser of $0.25H_s$ or 0.5 m; or

(b) the depth of fill would result in a P classification in accordance with Clause 2.5.3.

2.5.3 Effect of fill on classification

For the purposes of this Standard, fill that is in accordance with the technical and control requirements specified in AS 3798 for structural fill for residential applications is controlled fill. Other fill is uncontrolled fill for the purposes of this Standard.

The classification of sites containing fill shall be in accordance with the following:

(a) *Controlled fill:*

   (i) *Shallow fill* The classification of a site with controlled fill not more than 0.8 m deep for sand and not more than 0.4 m deep for material other than sand shall be the same as the natural site, prior to filling.

   (ii) *Deep fill* The classification of a site with controlled sand fill deeper than 0.8 m shall be the same as the natural site prior to filling. However, the presence of the sand may be used to justify by engineering principles a less severe reactive site classification. The effect of the fill on the settlement of the underlying soil shall be taken into account. The classification of a site with controlled fill of material other than sand and deeper than 0.4 m shall be Class P.

(b) *Uncontrolled fill:*

   (i) *Shallow fill* The classification of a site with uncontrolled fill not more than 0.8 m deep for sand and not more than 0.4 m deep for material other than sand shall be Class P, unless all footings (edge beams, internal beams and load support thickenings) are founded on natural soil through the filling.

   (ii) *Deep fill* The classification of a site with uncontrolled fill deeper than 0.8 m for sand and 0.4 m for material other than sand shall be Class P.

(c) *Reclassification of filled sites* A site with controlled fill and classified P may be given an alternative site classification in accordance with Table 2.1 if assessed in accordance with engineering principles. The assessment shall consider the movement of the fill and the underlying soil from the condition at construction to the long-term equilibrium moisture conditions.

SECTION 3 STANDARD DESIGNS

3.1 SELECTION OF FOOTING SYSTEMS

3.1.1 Selection procedure

Standard deemed-to-comply designs shall be in accordance with Clauses 3.2 to 3.6. These designs shall not apply to—

(a) Class E or Class P sites;

(b) buildings longer than 30 m;

(c) slabs containing permanent joints (e.g. contraction or control joints);

(d) two-storey construction with a suspended concrete floor at the first floor level except in accordance with Clause 3.9;

(e) two-storey construction in excess of the height limitations (see Clause 1.8.60);

(f) support of columns or fireplaces not complying with Clause 3.10;

(g) buildings incorporating wing walls or masonry arches unless they are detailed for movement in accordance with TN 61;

(h) construction of three or more storeys; or

(i) single-leaf earth or stone masonry walls greater than 3 m in height.

On moderately and highly reactive sites, the entire footing system for a single building shall comprise only one standard design.

These designs shall not apply to construction using concrete strengths of 32 MPa and greater.

3.1.2 Design for single-leaf masonry, mixed construction and earth wall construction

The proportions for the selected footing system for single-leaf masonry, mixed construction and earth wall construction shall comply with Clause 3.1.1 using the equivalent construction set out in Table 3.1.

TABLE 3.1
EQUIVALENT TYPES OF CONSTRUCTIONS

Actual construction		Equivalent construction
External walls	**Internal walls**	
Single–leaf masonry		
Reinforced single-leaf masonry	Articulated masonry on Class A and Class S sites, or framed	Articulated masonry veneer
Reinforced single-leaf masonry	Articulated masonry or reinforced single–leaf masonry	Masonry veneer
Reinforced single-leaf masonry	Masonry	Articulated full masonry
Articulated single-leaf masonry	Articulated masonry	Articulated full masonry
Articulated single-leaf masonry	Masonry	Articulated full masonry
Other single-leaf masonry	Framed	Articulated full masonry
Other single-leaf masonry	Masonry	Full masonry
Mixed construction		
Full masonry	Framed	Articulated full masonry
Articulated full masonry	Framed	Masonry veneer
Articulated rendered or sheet clad frame	Framed	Articulated masonry veneer
Precast concrete panels		
Reinforced concrete panel	Framed	Articulated masonry veneer
Earth wall construction		
Infill panels of earth wall construction	Framed earth wall construction	Articulated masonry veneer
Loadbearing earth wall construction	Load bearing earth wall construction	Articulated full masonry

3.1.3 Construction with framed party walls

For the purpose of this Section, where construction involves framed party walls, the building shall be taken as equivalent to masonry veneer construction or design shall be based on engineering principles.

3.1.4 Design for masonry feature walls

Masonry feature walls may be used in basic masonry veneer construction on footings appropriate for masonry veneer, provided the wall is straight, one-storey, less than 4 m in length between joints and is supported by either—

(a) an internal beam in a stiffened raft; or

(b) an internal strip footing continuous from external strip footing to external strip footing.

3.1.5 Design for outbuildings and extensions to dwellings

The footing system design given in this Section shall be used for outbuildings and extensions.

Outbuildings of clad framed construction may use footing systems appropriate for one class of reactivity less severe than for a main building.

Walls of masonry extensions, or masonry veneer extensions, shall be articulated at the junction with the existing building.

Footings of similar proportions and details to those used in an existing building on the same allotment may be used, provided the performance of the existing building has been satisfactory over at least 10 years after construction and there are no unusual moisture conditions.

3.1.6 Design for rock outcrops

Where a footing or edge beam encounters a single local rock outcrop over a length less than 1 m, the depth of the footing or edge beam may be reduced by up to one-third, provided the amount of top and bottom reinforcement is doubled and extended 500 mm past the section with reduced depth.

Alternatively, the footing may be stepped or raised, provided the structural stiffness is preserved.

3.1.7 Design for partial rock foundation

Where part of the footing is on rock and part is on soil, provision for movement at the change between the two types of foundation shall be made by articulation of the superstructure or strengthening of the footing system.

On M, H1 and H2 sites where part of the footing is on rock and part is on soil, the design shall be in accordance with engineering principles.

3.1.8 Design for complete rock foundation

Where the edge beam or footing is to be founded entirely on rock, the footing or beam may be replaced by a levelling pad of concrete or mortar.

3.2 STIFFENED RAFT

3.2.1 General

The concrete section sizes, beam spacing and reinforcement requirements for stiffened rafts shall be as given in Figure 3.1. Stiffened rafts shall be detailed in accordance with Clauses 3.2.2 to 3.2.4 and 5.3.

3.2.2 Beam layout

Internal and external edge beams shall form an integral structural grid in accordance with Clauses 5.3.8 and 5.3.9.

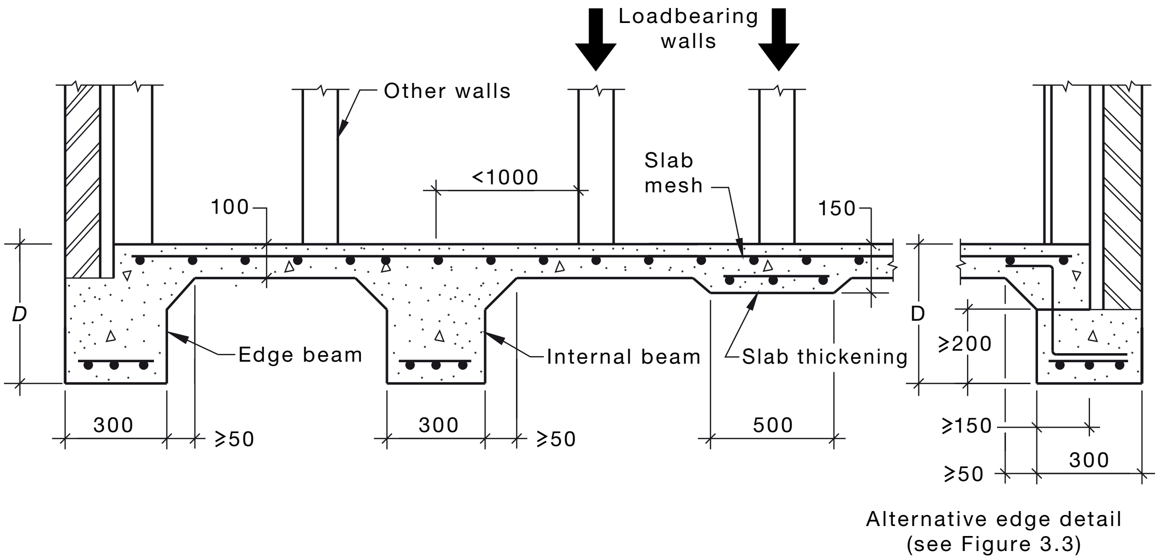
Where the number of beams in a particular direction satisfies the requirements of the maximum spacing given in Figure 3.1, the spacing between individual beams may be varied, provided the spacing between any two beams does not exceed the spacing given in Figure 3.1 by 25%. These allowances for increased beam spacings shall not override the maximum spacings between edge beams and first internal beams, as required by Clause 5.3.9.

3.2.3 Reinforcement

Where external beams wider than 300 mm are specified, an extra bottom bar or equivalent of the same bar size shall be used for each 100 mm additional width.

If a beam depth greater than that required in Figure 3.1 for the particular class of site and type of construction is to be used, the bottom reinforcement for the deeper beam given in Figure 3.1 shall be used.

The slab mesh specified in Figure 3.1 may be replaced with alternative reinforcement as given in Table 3.2.



DIMENSIONS IN MILLIMETRES

FIGURE 3.1 (in part) STIFFENED RAFT DESIGNS SITE CLASSES A, S, M, M-D, H1, H2, H1-D,H2-D

Site class	Type of construction	Edge and internal beams				
		Depth (*D*) mm	Bottom reinforcement		Top bar reinforce-ment	Max beam spacing cc m
			Mesh	Bar alternative		
Class A	Clad frame	300	3-L8TM	2N12	—	—
	Articulated masonry veneer	300	3-L8TM	2N12	—	—
	Masonry veneer	300	3-L8TM	2N12	—	—
	Articulated full masonry	400	3-L8TM	2N12	—	—
	Full masonry	500	3-L8TM	2N12	—	—
Class S	Clad frame	300	3-L8TM	2N12	—	—
	Articulated masonry veneer	300	3-L8TM	2N12	—	—
	Masonry veneer	300	3-L11TM	3N12	—	—
	Articulated full masonry	500	3-L11TM	3N12	2N12	—
	Full masonry	700	2x3-L11TM	3N16	2N16	5
Class M	Clad frame	300	3-L11TM	3N12	—	6
	Articulated masonry veneer	400	3-L11TM	3N12	—	6
	Masonry veneer	400	3-L11TM	3N12	—	5
	Articulated full masonry	625	3-L11TM	3N12	2N12	4
	Full masonry	950	2x3-L11TM	3N16	2N16	4
Class M-D	Clad frame	400	3-L11TM	3N12	—	5
	Articulated masonry veneer	400	3-L11TM	3N12	1N12	4
	Masonry veneer	500	3-L12TM	3N12	2N12	4
	Articulated full masonry	650	3-L12TM	2N16	2N16	4
	Full masonry	1050	2x3-L11TM	3N16	3N16	4
Class H1	Clad frame	400	3-L11TM	3N12	—	5
	Articulated masonry veneer	400	3-L11TM	3N12	1N12	4
	Masonry veneer	500	3-L11TM	3N12	3N12	4
	Articulated full masonry	750	2x3-L11TM	3N16	2N16	4
	Full masonry	1050	2x3-L12TM	3N16	3N16	4
Class H1-D	Clad frame	400	3-L11TM	3N12	1N12	4
	Articulated masonry veneer	500	3-L11TM	3N12	2N12	4
	Masonry veneer	650	2x3-L11TM	3N16	1N16	4
	Articulated full masonry	800	2x3-L11TM	3N16	2N16	4
	Full masonry	1100	2x3-L12TM	3N16	3N16	4
Class H2	Clad frame	550	3-L11TM	3N12	2N12	4
	Articulated masonry veneer	600	3-L12TM	3N12	2N12	4
	Masonry veneer	750	2x3-L11TM	3N16	2N16	4
	Articulated full masonry	1000	2x3-L11TM	3N16	2N16	4
	Full masonry	—	—	—	—	—
Class H2-D	Clad frame	550	2x3-L11TM	3N16	2N16	4
	Articulated masonry veneer	700	2x3-L11TM	3N16	2N16	4
	Masonry veneer	750	2x3-L11TM	3N16	2N16	4
	Articulated full masonry	1000	2x3-L11TM	3N16	2N16	4
	Full masonry	—	—	—	—	—

NOTE: Slab reinforcement for all site classes shall be as follows:

(a) SL72, where slab length <18 m

(b) SL82, where slab length 18 to 25 m

(c) SL92, where slab length >25 and <30 m

FIGURE 3.1 (in part) STIFFENED RAFT DESIGNS FOR SITE CLASSES A, S, M, M-D, H1, H2, H1-D, H2-D

TABLE 3.2

ALTERNATIVE SLAB REINFORCEMENT

Alternative slab mesh	Specified slab mesh		
	SL102	**SL92**	**SL82**
	Additional reinforcement at top of beams		
SL92	3-L11TM	—	—
SL82	3-N16	3-L11TM	—
SL72	4-N16	4-L12TM	2-L12TM

3.2.4 Construction

Except on site Classes M-D, H1-D and H2-D, a horizontal construction joint is permitted in the edge and internal beams, provided the concrete-to-concrete joint is at least 150 mm wide and traversed by R10 or N10 fitments at 600 mm centres or equivalent.

NOTE: See alternative edge beam detail in Figure 3.1.

Construction details shall be as given in Clauses 6.4 and 6.6.

Requirements for shrinkage crack control shall be as given in Clause 5.3.7.

3.2.5 Reinforced masonry

Where a reinforced single-leaf masonry wall with a continuous reinforced bond beam is constructed directly above and structurally connected to a concrete edge beam, and complies with the minimum requirements for reinforced masonry walls in AS 4773.1, the beam may be 300 mm wide by 300 mm deep with 3-L11TM reinforcement.

3.3 FOOTING SLAB

Footing slabs shall be selected in accordance with Figure 3.2 (Class A) or Figure 3.3 (Class A or Class S).

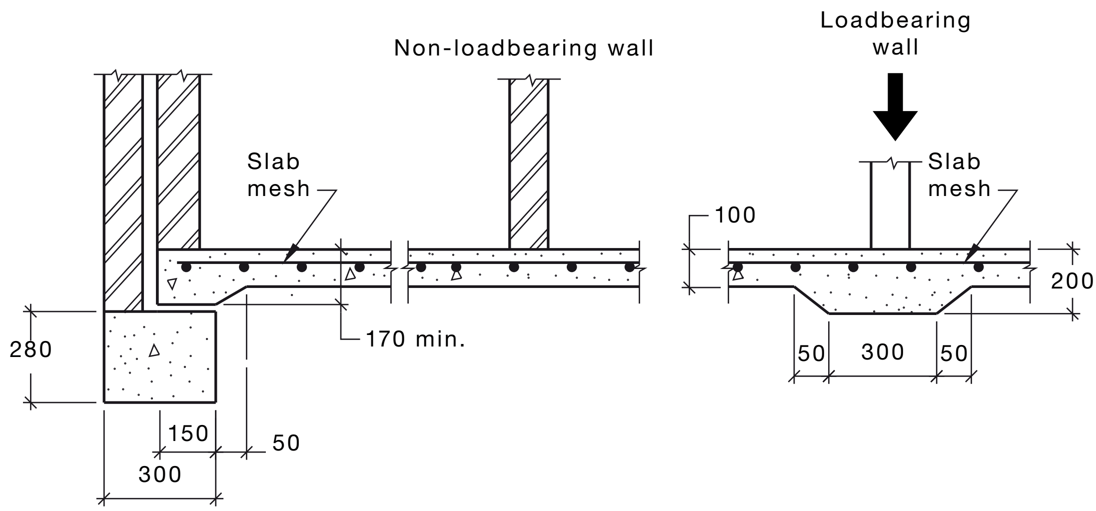
(a) *Unreinforced footing* Figure 3.2 sets out requirements for unreinforced footings.

(b) *Reinforced footing* The proportions for the tied edge beam apply only where there is a structural connection by concrete-to-concrete contact tied with fitments.

Where the edge beam supports but is not tied to the slab such as is shown for the alternative edge beam treatment in Figure 3.3, the footing proportions and footing reinforcement shall comply with Figure 3.6.

Construction shall be in accordance with Section 6. In particular, for the alternative edge treatment, the retaining wall details shall be in accordance with Clause 6.4.5.

Fitments shall be structurally anchored above and below the joint.



NOTES:

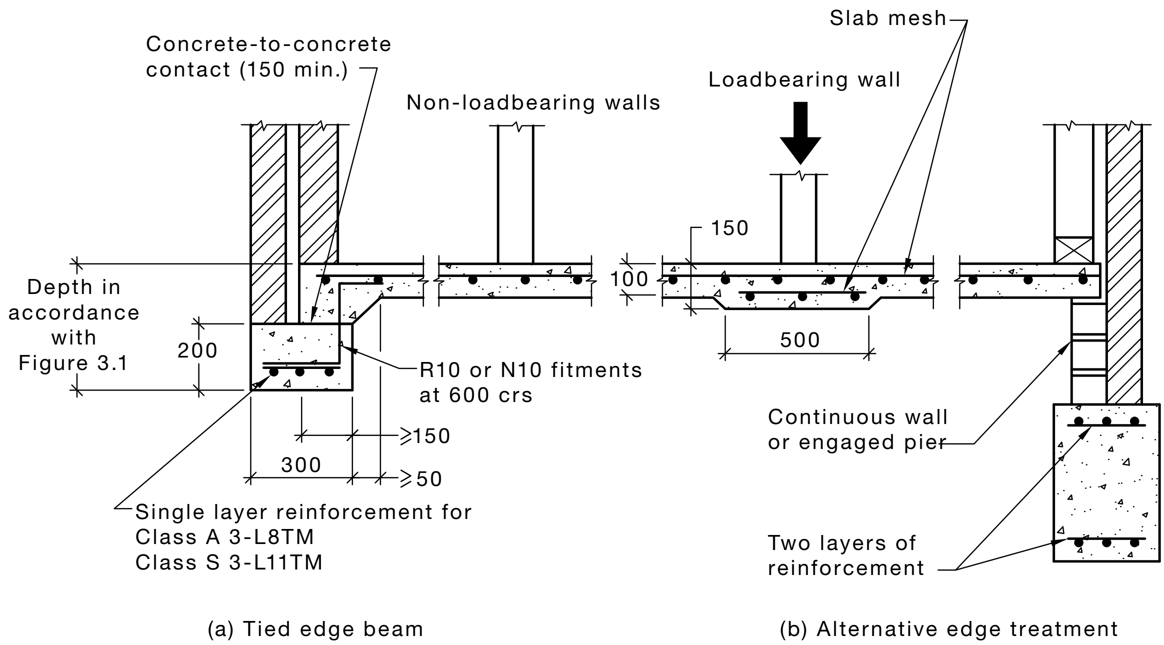
1 Slab reinforcement shall be as follows:
   (a) SL63 when maximum slab length ≤12 m.
   (b) SL62 when maximum slab length >12 m and <18 m.
   (c) SL72 when maximum slab length ≥18 m and <25 m.
   (d) SL82 when maximum slab length ≥25 m and <30 m.

2 In Western Australia, for slabs under 25 m in length, where specified by an appropriately qualified engineer, the slab thickness may be reduced to 85 mm with reinforcement as specified below. All other details remain the same as follows.
   (a) SL53 when maximum slab length ≤12 m.
   (b) SL63 when maximum slab length >12 m and <18 m.
   (c) SL62 when maximum slab length ≥18 m and <25 m.

3 Dune sands may require compaction.

DIMENSIONS IN MILLIMETRES

FIGURE 3.2 FOOTING SLAB FOR CLASS A SITES FOR CLAD FRAME, ARTICULATED MASONRY VENEER, MASONRY VENEER, ARTICULATED FULL MASONRY OR FULL MASONRY



DIMENSIONS IN MILLIMETRES

FIGURE 3.3 FOOTING SLAB FOR CLASS A AND FOR CLASS S SITES FOR CLAD FRAME, ARTICULATED MASONRY VENEER, MASONRY VENEER, ARTICULATED FULL MASONRY OR FULL MASONRY

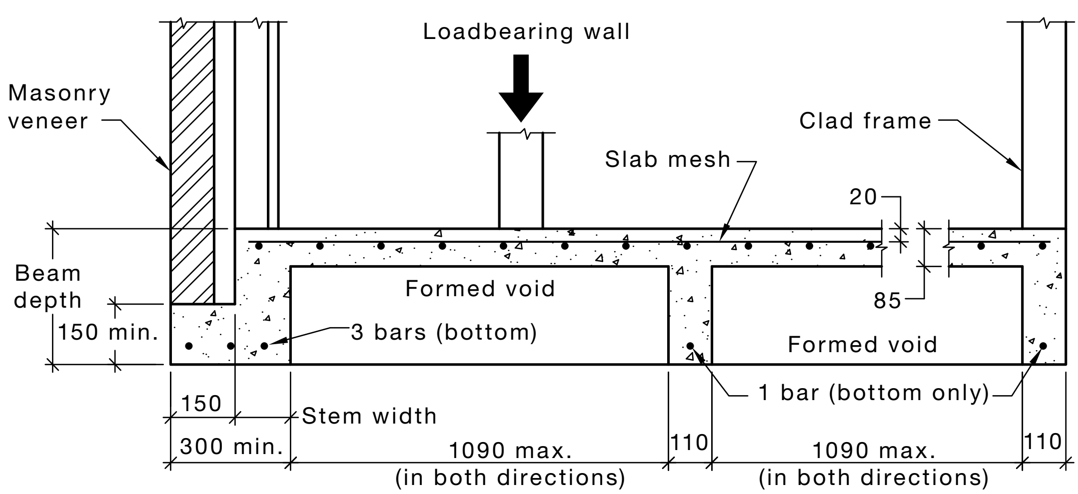
3.4 WAFFLE RAFTS

3.4.1 General

Waffle rafts shall be specified in accordance with Figure 3.4. Modifications to the details given in Figure 3.4 shall not be undertaken without engineering design in accordance with Section 4.

A slab thickness of 85 mm may be used in garage areas.

Internal and edge beams shall be arranged such as to maintain continuity at re-entrant corners, in accordance with Clause 5.3.8.



DIMENSIONS IN MILLILMETRES

FIGURE 3.4 (in part) WAFFLE RAFT

Site class	Type of construction	Beam reinforcement				Slab mesh	
		Depth (D) mm	Edge beam		Internal beam	Slab length, m	
			Mesh alternative	Bar alternative		<20	≥20 and <30
A	Clad frame	260	3-L8TM	3N12	1N12	SL72	SL82
	Articulated masonry veneer	310	3-L8TM	3N12	1N12	SL72	SL82
	Masonry veneer	310	3-L8TM	3N12	1N12	SL72	SL82
	Articulated full masonry (S/S)	310	3-L8TM	3N12	1N12	SL72	SL82
	Full masonry	—	—	—	—	—	—
S	Clad frame	260	3-L8TM	3N12	1N12	SL72	SL82
	Articulated masonry veneer	310	3-L8TM	3N12	1N12	SL72	SL82
	Masonry veneer	310	3-L8TM	3N12	1N12	SL72	SL82
	Articulated full masonry (S/S)	385	3-L8TM	3N12	1N16	SL72	SL82
	Full masonry	—	—	—	—	—	—
M	Clad frame	310	3-L11TM	3N12	1N12	SL72	SL82
	Articulated masonry veneer	310	3-L11TM	3N12	1N12	SL72	SL82
	Masonry veneer	310	3-L11TM	3N12	1N12	SL72	SL82
	Articulated full masonry (S/S)	610	2x3-L11TM	3N16	1N16	SL72	SL82
	Full masonry	—	—	—	—	—	—
M-D	Clad frame	310	3-L11TM	3N12	1N12	SL72	SL92
	Articulated masonry veneer	310	3-L11TM	3N12	1N12	SL72	SL92
	Masonry veneer	385	2x3-L11TM	3N16	1N16	SL72	SL92
	Articulated full masonry (S/S)	610	2x3-L11TM	3N16	1N16	SL72	SL92
	Full masonry	—	—	—	—	—	—
H1	Clad frame	310	3-L11TM	3N12	1N12	SL82	SL92
	Articulated masonry veneer	385	3-L11TM	3N12	1N12	SL82	SL92
	Masonry veneer	460	2x3-L11TM	3N16	1N16	SL82	SL92
	Articulated full masonry (S/S)	610	2x3-L11TM	3N16	1N16	SL82	SL92
	Full masonry	—	—	—	—	—	—
H1-D	Clad frame	310	3-L11TM	3N12	1N12	SL82	SL92
	Articulated masonry veneer	385	3-L11TM	3N12	1N12	SL82	SL92
	Masonry veneer (S/S)	460	2x3-L11TM	3N16	1N16	SL82	SL92
	Articulated full masonry	—	—	—	—	—	—
	Full masonry	—	—	—	—	—	—
H2	Clad frame	310	3-L11TM	3N12	1N12	SL82	SL92
	Articulated masonry veneer	385	2x3-L11TM	3N16	1N16	SL82	SL92
	Masonry veneer	—	—	—	—	—	—
	Articulated full masonry	—	—	—	—	—	—
	Full masonry	—	—	—	—	—	—
H2-D	Clad frame	385	2x3-L11TM	3N16	1N16	SL82	SL92
	Articulated masonry veneer (S/S)	460	2x3-L11TM	3N16	1N16	SL82	SL92
	Masonry veneer	—	—	—	—	—	—
	Articulated full masonry	—	—	—	—	—	—
	Full masonry	—	—	—	—	—	—

NOTE: S/S refers to single-storey construction.

FIGURE 3.4 (in part) WAFFLE RAFT

3.4.2 Stem width

The minimum stem width shall be 110 mm for clad frame and 150 mm for masonry construction. The minimum width of the base of an external beam shall be 110 mm for clad frame, single-storey articulated masonry veneer and single-storey masonry veneer and 300 mm for two-storey articulated masonry veneer, two-storey masonry veneer, single-storey articulated full masonry and single-storey full masonry.

3.4.3 Reinforcement

Additional reinforcement shall be provided for all beams where the stem width exceeds 150 mm. The size and specification of top bars shall be the same as bottom bars except as specified in Figure 3.4. The total number of reinforcement bars in beams shall be as follows:

Stem width mm	Top bars (additional to slab mesh)	Beam base width mm	Bottom steel
110 to 150	0	110 to 150	1
151 to 220	1	151 to 220	2
221 to 330	2	221 to 330	3
331 to 440	3	331 to 440	4

3.4.4 Construction

Construction details shall be in accordance with Clauses 6.4 and 6.6.

3.4.5 Piers

The waffle raft for a one-storey building for clad frame or masonry veneer on moderately or highly reactive sites may be supported on piers as follows without structural design of the waffle raft:

(a) Piers to be located on the intersection of every third internal beam.

(b) An additional N12 bar at the top to be provided in the internal beams intersecting the piers, but no shear fitments are required.

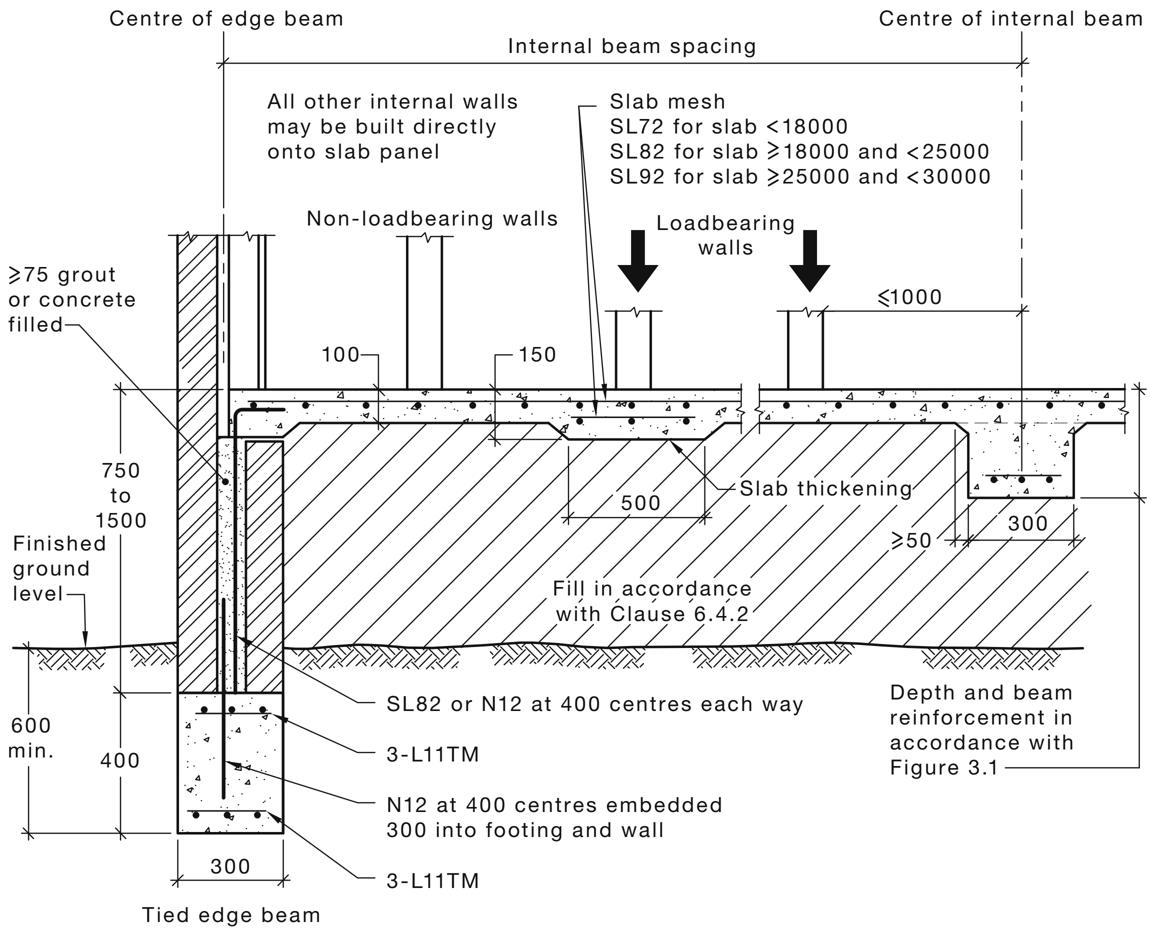
3.5 STIFFENED SLAB WITH DEEP EDGE BEAM

3.5.1 General

A stiffened slab with edge beam may be used on Class M sites for masonry veneer or articulated masonry veneer construction. Details shall be in accordance with Figure 3.5.

3.5.2 Beam spacing

Beam spacing shall not exceed 5.0 m for masonry veneer construction or 6.0 m for articulated masonry veneer construction.



DIMENSIONS IN MILLIMETRES

FIGURE 3.5 STIFFENED SLAB WITH DEEP EDGE BEAM FOR MASONRY VENEER AND ARTICULATED MASONRY VENEER CLASS M SITE

3.5.3 Edge wall construction details

The reinforcement of the cavity shall consist of N12 bars at 400 mm centres in each direction, with the vertical bars anchored into both the footing and the slab. The cavity shall be filled with well-compacted 20 MPa concrete, or grout in accordance with AS 3700. Single-leaf reinforced masonry may also be used.

3.5.4 Construction joints

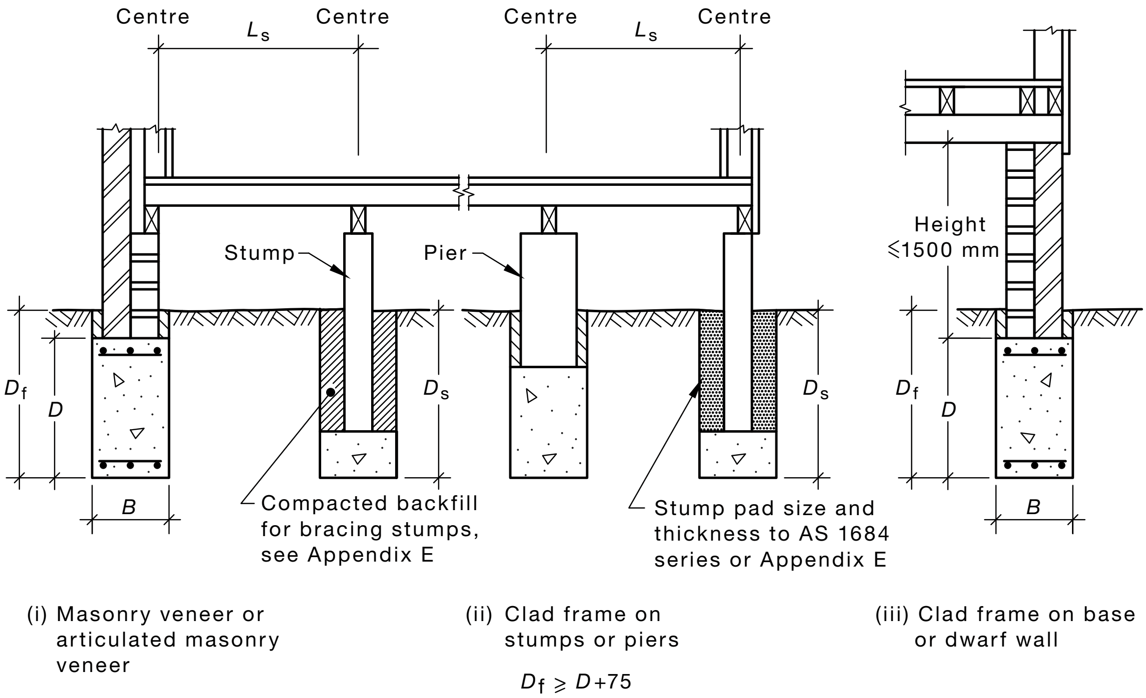
Horizontal construction joints may be used between the beams and the slab, provided the concrete-to-concrete joint is at least 150 mm wide and is traversed by R10 or N10 fitments at 600 mm centres or equivalent reinforcement.

3.6 STRIP FOOTINGS

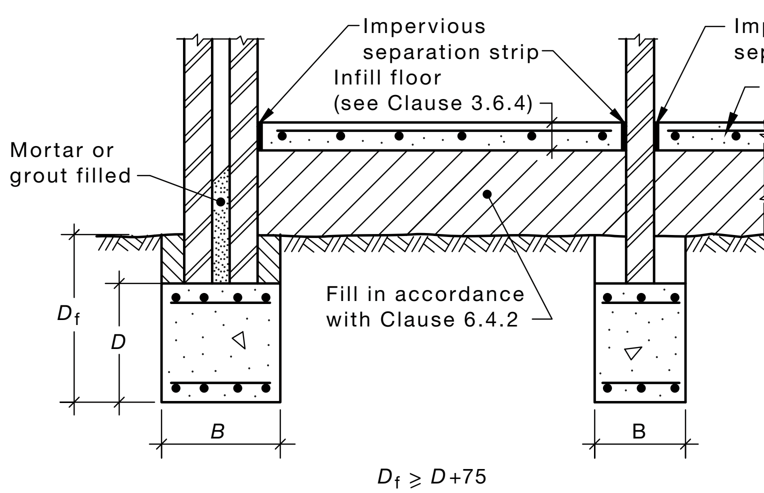
3.6.1 General

All masonry walls shall be supported on strip footings. Details of strip footings shall be in accordance with Figure 3.6.

NOTE: Figure 3.6 does not include designs for Classes H1-D, H2 or H2-D.



(a) Suspended floors (timber or concrete single-storey construction <4 kPa dead load)

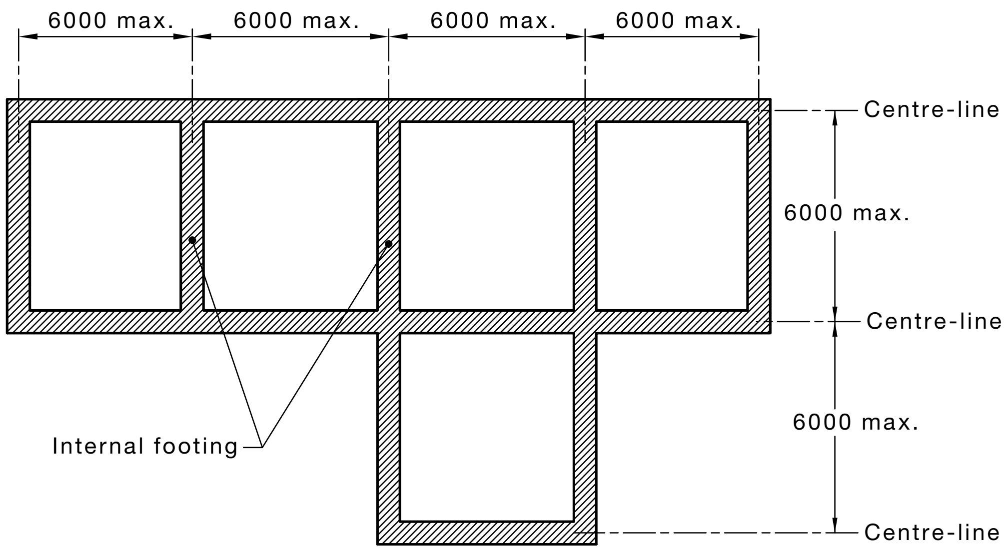


(b) Infill floor Class A and Class S sites

* Slab mesh;
SL62, when slab length <18000
SL72, when slab length ⩾18000 and <25000
SL82, when slab length ⩾25000 and <30000

DIMENSIONS IN MILLIMETRES

FIGURE 3.6 (in part) STRIP FOOTING SYSTEMS



(c) Example of footing system with re-entrant corners

Site class	Type of construction	Depth *(D)*	Width *(B)*	Reinforcement	D_s	L_s
Class A	Clad frame	300	300	3-L8TM	400	—
	Articulated masonry veneer	300	300	3-L8TM	400	—
	Masonry veneer	300	300	3-L8TM	400	—
	Articulated full masonry	300	400	4-L8TM	400	—
	Full masonry	300	400	4-L8TM	400	—
Class S	Clad frame	400	300	3-L8TM	400	—
	Articulated masonry veneer	400	300	3-L8TM	400	—
	Masonry veneer	400	300	3-L8TM	400	—
	Articulated full masonry	400	400	4-L11TM	400	—
	Full masonry	500	400	4-L11TM	400	—
Class M	Clad frame	400	300	3-L11TM	500	—
	Articulated masonry veneer	450	300	3-L11TM	500	—
	Masonry veneer	500	300	3-L12TM	500	—
	Articulated full masonry	600	400	4-L12TM	500	—
	Full masonry	900	400	4-L12TM	500	—
Class M-D	Clad frame	500	300	3-L11TM	800	—
	Articulated masonry veneer	550	300	3-L12TM	800	—
	Masonry veneer	700	300	3-N16	800	—
	Articulated full masonry	1 100	400	4-N16	800	—
Class H1	Clad frame	500	300	3-L11TM	1 000	≥2400
	Articulated masonry veneer	600	300	3-L12TM	1 000	≥2400
	Masonry veneer	850	300	3-N16	1 000	≥2400
	Articulated full masonry	1 100	400	4-N16	1 000	≥2400

DIMENSIONS IN MILLIMETRES

FIGURE 3.6 (in part) STRIP FOOTING SYSTEMS

3.6.2 Deep beams

All beams 700 mm or deeper, as specified in the table of Figure 3.6, shall be detailed as follows:

(a) Internal footings shall be provided at no more than 6 m centres, and at re-entrant corners to continue the footings to the opposite external footing (see Figure 3.6). Internal strip footings shall be of the same proportions as the external footing and run from external footing to external footing.

(b) 'Side slip joints' consisting of a double layer of polyethylene shall be provided at the sides of the footing only.

3.6.3 Pads and stumps

The size and thickness of pads for stumps or piers shall be selected using AS 1684. Sizes for larger loads shall be selected in accordance with Appendix E.

Bracing forces and uplift forces to stumps shall be detained in accordance with Appendix E.

3.6.4 Infill floors

Infill floors shown in Figure 3.6(b) shall only be used for Class A and Class S sites. Infill floors may be concrete slabs (100 mm thick), brick paving stone flags, or compacted or stabilized earth.

3.6.5 Reinforcement

If strip footings deeper than those required are used, the reinforcement shall be increased to match that specified for the deepened proportions.

Where footings are wider than the specified width, an extra bar of the same bar size is required top and bottom for each 100 mm additional width.

3.7 REINFORCEMENT EQUIVALENCES

The bar sizes given in Table 3.3 are deemed to comply with the requirements for trench mesh reinforcement in beams and footings. L11TM and L12TM may be replaced by RL1118 and RL1218 mesh respectively. Two layers of L8TM may be used as a replacement for L11TM. Where a single layer of trench mesh would be too wide for the footing or beam, multiple layers, bundled together, or equivalent reinforcement shall be used.

Alternative arrangements of beam or footing reinforcement may be used, provided the flexural strength of the section is unimpaired and the ductility requirement is met.

TABLE 3.3

REINFORCING BAR SIZES DEEMED TO BE EQUIVALENT TO TRENCH MESH REINFORCEMENT

Trench mesh	Area of steel, mm^2	Bar alternative	Trench mesh alternative
2-L8TM	91	1-N12	—
3-L8TM	136	2-N12	—
4-L8TM	182	2-N12	2-L11TM
5-L8TM	227	2-N12	3-L11TM
2-L11TM	178	1-N16 or 2-N12	2 × 2-L8TM
3-L11TM	267	3-N12	2 × 3-L8TM
4-L11TM	356	2-N16	2 × 4-L8TM
2-L12TM	222	2-N12	3-L11TM
3-L12TM	333	3-N12	4-L11TM
4-L12TM	444	4-N12	5-L11TM

3.8 SUSPENDED CONCRETE FLOORS IN ONE-STOREY CONSTRUCTION

Suspended concrete floors in one-storey construction shall be designed in accordance with engineering principles. For short span floors on Class A and Class S sites, the following criteria may be used for design:

(a) Fill used as temporary support need not be controlled or rolled fill.

(b) Internal concrete slabs that are suspended between at least two opposite walls and do not support walls or columns may be 125 mm thickness with SL72 top and SL72 bottom minimum reinforcement, with 20 mm cover top and bottom, for clear spans up to 2.4 m length.

(c) Such floors may be supported on dwarf masonry walls on strip footings.

Where clay is used as temporary fill, it shall be placed at a moisture content that will minimize subsequent swell

3.9 FOOTING SYSTEMS FOR TWO-STOREY CONSTRUCTION WITH SUSPENDED CONCRETE FLOOR

For a two-storey building with a suspended concrete floor at the first floor level, the footing system designs given in Figures 3.1 to 3.6 for Class A and Class S sites may be used, provided that, for the suspended floor—

(a) the thickness is not greater than 175 mm, or the dead load per unit area is not greater than 4 kPa;

(b) the span is less than 5 m; and

(c) the support is on masonry walls at each end with openings not greater than 2.5 m.

In addition, the width given in those figures for the edge beams and footings shall be increased by 100 mm and, where reinforcement is specified, the top and bottom reinforcement shall be increased by one bar of the same diameter.

Where the suspended floor is supported through an internal wall onto the slab panel on the ground, the thickness of the slab panel shall be increased to 200 mm for a width of 500 mm and, where reinforcement is specified, an extra strip containing three wires of the slab mesh placed in the bottom of the thickened section shall be provided.

3.10 FOOTINGS FOR CONCENTRATED LOADS

3.10.1 Footings for columns

Loads from columns shall be supported by—

(a) pad footings, which may be integral with a slab, of the proportions given in Figure 3.6;

(b) edge beams or strip footings; or

(c) slab panels for dead loads less than 15 kN.

On reactive clays, if the supported area is not greater than 20 m^2, concentrated loads from columns shall be supported directly on an edge or internal beam or strip footing.

NOTE: See Note 2 to Figure E1, Appendix E.

On moderately and highly reactive sites, separate footings shall not be used unless the supported construction is structurally isolated from the rest of the building.

3.10.2 Footings for fireplaces on Class A and Class S sites

Fireplaces shall be supported on a pad footing 150 mm thick for one-storey construction or 200 mm thick for two-storey construction. Pad footings shall be reinforced top and bottom with SL72 and extending 300 mm past the edges of the masonry except for any edge flush with the outer wall. This footing may be integral with a slab.

SECTION 4 DESIGN BY ENGINEERING PRINCIPLES

4.1 GENERAL

To comply with this Standard, slabs or footing systems designed in accordance with engineering principles shall also be designed in accordance with this Section and AS 3600. Where a specific provision given in this Section differs from a similar provision in AS 3600, the provision given in this Section shall apply.

This Section may be used to extend the range of validity of, or to modify, the deemed-to-comply designs contained in Section 3 of this Standard.

4.2 DESIGN CRITERIA

Slabs and footings and associated superstructures shall be designed to satisfy the performance criteria set out in Clause 1.3 when subjected to the loads noted therein.

The tolerable limits for differential movement depend on the form of construction, surface finish and the actual detailing of the superstructure and, in the absence of more specific information, shall be the lesser of the two values given in Table 4.1 for the applicable type of construction.

TABLE 4.1

MAXIMUM DESIGN DIFFERENTIAL FOOTING DEFLECTION (Δ) FOR DESIGN OF FOOTINGS AND RAFTS

Type of construction	Maximum differential deflection, as a function of span, mm	Maximum differential deflection, mm
Clad frame	L/300	40
Articulated masonry veneer	L/400	30
Masonry veneer	L/600	20
Articulated full masonry	L/800	15
Full masonry	L/2000	10

4.3 DESIGN OF FOOTING SYSTEMS

Footing systems shall be designed in accordance with one of the following:

(a) Stiffened raft footing systems supporting a superstructure that relies entirely on the raft to resist cracking (see Clause 4.4 or 4.5).

(b) Shallow footing systems other than stiffened rafts (see Clause 4.6).

(c) Footing systems supporting walls with sufficient strength to span without cracking (see Clause 4.7).

(d) Piered or piled footing systems (see Clause 4.8).

4.4 STIFFENED RAFT FOOTING SYSTEMS

A stiffened raft footing system that supports a superstructure that relies entirely on the raft stiffness to resist movement and cracking shall be proportioned as follows:

(a) The raft structure shall comprise a grid of approximately orthogonal beams structurally connected to a concrete slab.

(b) For rafts with beams embedded deeper than 1 m in depth, or with connected piers greater than 1 m in depth, the analysis shall consider the influence of skin friction on the sides of the beams or piers according to engineering principles. The uplift resistance of connected piers or anchors shall be taken into account.

(c) The tolerable limits for relative differential movement depend on the form of construction, surface finish and the actual detailing of the superstructure. In the absence of more specific information, the tolerable limit shall be the lesser of the two values given in Table 4.1 for the applicable type of construction.

(d) Where permanent slab joints are used, the design shall consider the variation in section stiffness at the joint location.

(e) The effective total width of the flange for slab beams shall be determined as follows:

   (i) In sagging mode (edge heave), the effective total width shall be taken as $B_w + 0.1L$ for an edge beam and $B_w + 0.2L$ for an internal beam.

   (ii) In hogging mode (centre heave), the effective total width shall be assessed by the designer as between $B_w + 0.1L$ and $B_w + 1$ m for an edge beam, and $B_w + 0.2L$ and $B_w + 2$ m for an internal beam.

   In no case shall the flange width be taken as greater than the distance halfway to the adjacent beams.

(f) From the soil-structure analysis using the action effects given in Clause 1.4, the design bending moment (M^*) at a cross-section and the required stiffness (E × I) may be determined. The stiffness, E × I, of the slab shall be not less than that required by the analysis. In the determination of E × I the value of E shall be taken as 15 000 MPa for N20 concrete and I shall be as defined in AS 3600. The structural design for strength of the cross-section shall satisfy the following:

$$M^* \leq \phi M_u \qquad \ldots 4.4$$

   where

   M_u = ultimate bending moment strength calculated in accordance with AS 3600

   ϕ = strength reduction factor given in AS 3600

   NOTE: Two acceptable methods of design (Walsh and Mitchell) using soil structure interaction for stiffened rafts are described in Appendix F.

(g) Internal and external beams shall be arranged in accordance with Clauses 5.3.8 and 5.3.9.

(h) Beam spacing shall be adequate to ensure the structural integrity and stiffness of the raft. The maximum beam spacing shall not exceed 1.25 times the maximum values given in Figure 3.1 for the applicable site classification. For Class E sites, the beam spacing shall not exceed 5 m.

(i) For ductility, the cross-section shall be reinforced so the ultimate strength calculated on the basis of a reinforced section (M_u) is 20% greater than the cracking moment capacity (M_{cr}), where M_{cr} may be determined for sagging moments for 20 MPa concrete using a tensile strength of 2.7 MPa and for hogging moments 1.8 MPa.

(j) Where the calculated shear force exceeds the design strength of the unreinforced section, shear reinforcement shall be required in raft beams. Side face reinforcement is not required in deep raft beams.

4.5 SIMPLIFIED METHOD FOR RAFT DESIGNS

4.5.1 Application

Stiffened rafts may be designed in accordance with this Clause, provided the design parameters are within the following range:

Design parameter	Range
y_s	10 mm to 70 mm if H_s > 3 m *or* 10 mm to 100 mm if H_s < 3 m
Δ	5 mm to 50 mm
Span	5 m to 30 m
Beam spacing	≤1.25 values in Figure 3.1 Clause 5.3.9 shall apply at external corners of the building. For Class E sites the beam spacing shall not exceed 5 m.
Beam depth	250 mm to 1200 mm
Minimum depth of any beam	≥0.8 max. beam depth
Beam width	110 mm to 400 mm
Design distributed load	≤10 kPa
Design edge line load	≤25 kN/m

The specified slab mesh shall be not less than SL72 for slab spans <18 m; SL82 for spans ≥18 m and <25 m; and SL92 for spans ≥25 m and the ductility requirements of Clause 4.4(i) shall be satisfied. It is not necessary to use a design stronger than the standard design for the site classification nor is it permitted to use a design weaker than the design given for the next lower site Class.

For types of construction outside the range given in Figure 3.1 and Table 3.1 for which no standard design is appropriate, this method may be used.

4.5.2 Modification procedure

The value of y_s/Δ shall be determined where Δ is the permissible maximum differential movement given in Column 3 of Table 4.1 for the appropriate construction. From Figure 4.1 and the value of y_s/Δ, the design shall provide in each direction the following stiffness parameter:

$$\log\left[\sum\left(\frac{B_w D^3}{12}\right)/W\right]$$

where the summation is determined over all the edge and internal beams, and

B_w = beam web width, in millimetres

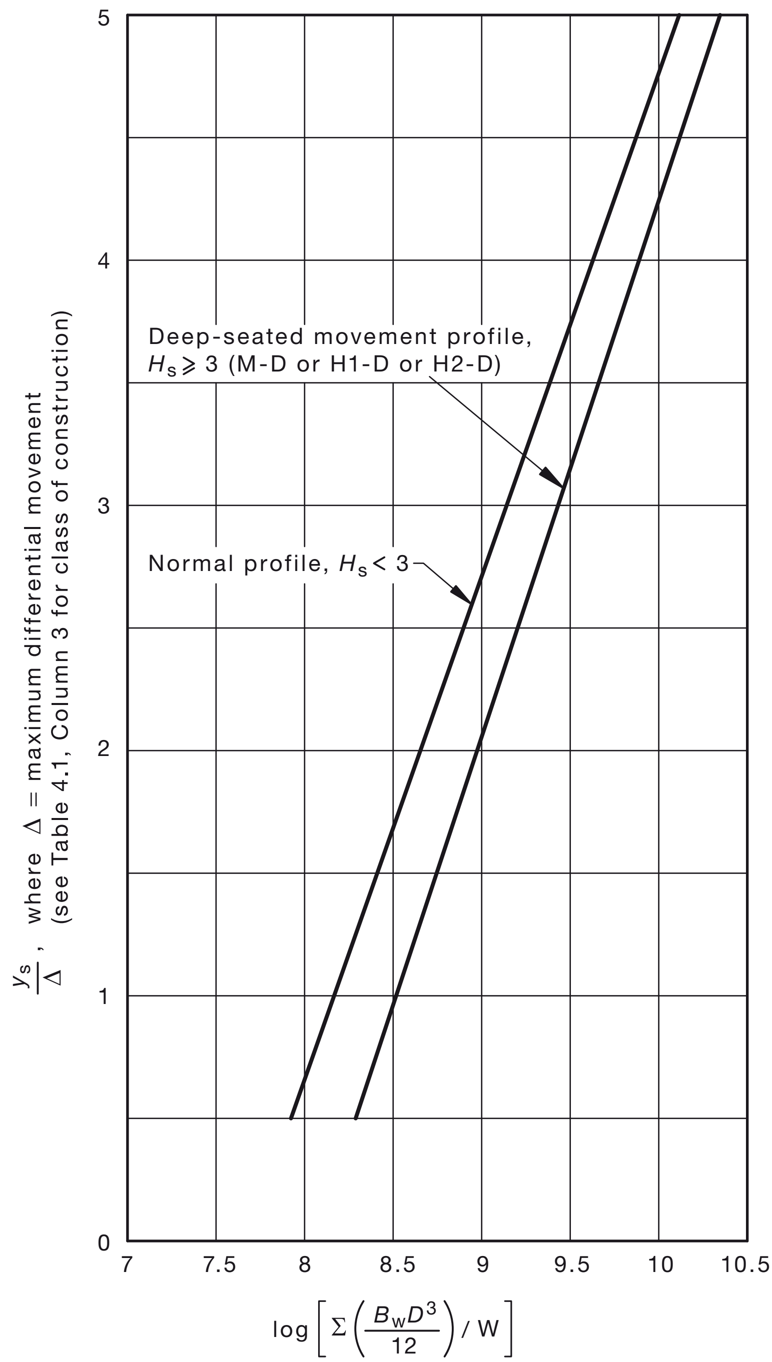
D = overall depth of the beam, in millimetres

W = overall width of the slab, in metres, normal to the direction of the beams being considered

The strength shall be provided by the satisfaction of the ductility requirements of Clause 4.4(i). For non-rectangular plans, the design shall be based on overlapping rectangles.

4.6 DESIGN OF FOOTING SYSTEMS OTHER THAN STIFFENED RAFTS

The design of shallow footing systems, other than stiffened rafts, shall be in accordance with the general principles outlined in Clause 4.4, modified to take into account the soil-structure interaction of the footing system. Lift-off shall not be considered in strip footings deeper than 0.6 m.



where B_w = width of beam web, in millimetres
D = overall depth of beam, in millimetres
W = overall width of slab, in metres

FIGURE 4.1 MOVEMENT RATIO VERSUS UNIT STIFFNESS

4.7 FOOTING SYSTEMS FOR REINFORCED SINGLE–LEAF MASONRY WALLS

For buildings whose walls have sufficient strength to span for significant distances over sagging or hogging footings (either as a cantilever over hogging footings or simply supported over sagging footings), it shall be permissible to proportion the wall-footing system to utilize the flexural strength of the wall.

Design of the masonry shall be in accordance with AS 3700 to resist the action effects derived in this Standard. The required dimensions, reinforcement and disposition of footings shall be designed to satisfy the principles of Clause 4.4 and in accordance with the following:

(a) If the walls are not structurally connected to the footing or slab system, the length over which a particular type of wall can span before it cracks or experiences excessive deflection shall be determined and the footings proportioned to ensure that this length is not exceeded and the potential for the footing to separate from the wall is limited to 5 mm, unless specific provision is made for movement.

(b) If the walls of a building are structurally connected to the footing or slab system by means of steel starter bars, anchors or equivalent, the combined strength and stiffness of the wall and footing/slab system shall be considered to determine the length over which the particular combination can span before it cracks or experiences excessive deflection. The footings, floor and the wall shall be proportioned and reinforced to ensure this length is not exceeded.

The joints between adjacent wall panels shall be designed to accommodate any movement resulting from footing movement. The wall configuration shall be such that each wall is prevented from tilting, twisting or distorting to an extent that limits the serviceability of the building.

In determining the spanning ability, the following points shall be considered:

(i) The effect of increased load from upper floors or roof structure in diminishing this ability to span.

(ii) The strengthening effect (if any) of joint reinforcement in masonry walls.

(iii) The strengthening effect (if any) of steel reinforcement in the cores and bond beams of reinforced or partially reinforced hollow masonry.

(iv) The strengthening effect (if any) of render, plaster, plasterboard or other veneers fixed to the wall.

4.8 DESIGN FOR PILED OR PIERED FOOTING SYSTEMS

A pier-and-beam, pier-and-slab or piled footing system shall be designed in accordance with engineering principles.

NOTE: Design of deep footings should be carried out in accordance with Appendix G.

SECTION 5 DETAILING REQUIREMENTS

5.1 GENERAL

The detailing of all footing systems designed or selected in accordance with Section 3 or Section 4 shall comply with Clauses 5.2 to 5.5. For highly reactive and extremely reactive sites, detailing shall comply with Clause 5.6.

5.2 DRAINAGE DESIGN REQUIREMENTS

5.2.1 General requirements

The building and site drainage design and height of the floor level above finished ground level may be affected by factors other than structural design requirements. Such factors include the following:

(a) The run-off of water and the influence of local topography.

(b) The effect of excavation or filling.

(c) The possibility of flooding.

(d) The effects of existing and post-construction landscaping.

(e) The level of legal point of stormwater discharge.

(f) Plumbing and drainage requirements.

NOTE: For example, the height of the overflow relief gully relative to the top of the lowest plumbing fixture, and the surrounding ground level (see AS/NZS 3500).

(g) Minimum height from finished ground level to the damp-proof course level.

(h) Termite management.

NOTE: For guidance on termite management, see AS 3660.1.

Surface drainage shall be designed and constructed to avoid water ponding against or near the footing. The ground in the immediate vicinity of the perimeter footing, including the ground uphill from the slab on cut-and-fill sites, shall be graded to fall 50 mm minimum away from the footing over a distance of 1 m and shaped to prevent ponding of water. Where filling is placed adjacent to the building, the filling shall be compacted and graded to ensure drainage of water away from the building.

The requirements of Clause 5.2.2 shall be applied to reduce the possibility of surface water entering living areas.

Alternative drainage systems will be required on zero lot line construction. Any paving shall also be suitably sloped.

5.2.2 Specific requirements for slabs for Class 1 buildings

For Class 1 buildings the minimum height of the slab above finished ground, landscaping or paving level shall be 150 mm, except in the following cases:

(a) In sandy, well-drained areas, the minimum height shall be 100 mm.

(b) Where adjoining paved areas slope away from the building, these heights may be reduced to 50 mm.

(c) These heights may be further reduced locally at entrances that are shielded from the weather.

5.3 REQUIREMENTS FOR RAFTS AND SLABS

5.3.1 Concrete

The grade of concrete shall be N20 with slump of 100 mm in accordance with AS 1379, with 20 mm maximum nominal aggregate size, or as specified in Clauses 5.5, or as specified by the designer.

5.3.2 Reinforcement

Reinforcement in rafts and slabs shall be placed in accordance with the following:

(a) Minimum concrete cover for the reinforcement shall be 40 mm to unprotected ground, 40 mm to external exposure, 30 mm to a membrane in contact with the ground, and 20 mm to an internal surface.

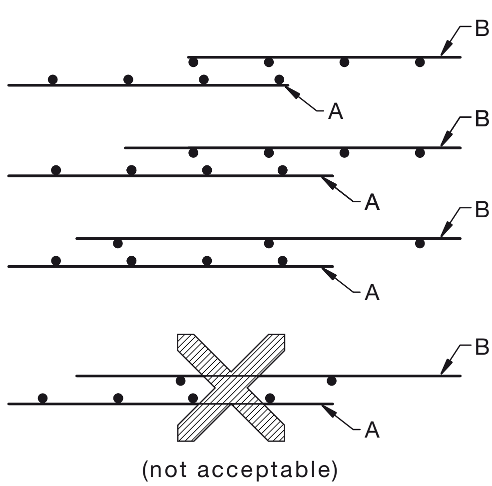
(b) The slab mesh shall be placed towards the top of the raft or slab (see also Clause 5.5).

(c) Raft or slab mesh shall be lapped as shown in Figure 5.1.

(d) Trench mesh shall have all cross-wires cut flush with the outer main wires. Trench mesh in beams shall be overlapped by the width of the mesh at T- and L-intersections. Trench mesh shall be spliced, where necessary, by a lap of 500 mm.

(e) Reinforcing bars shall have a lap length at splices not less than 500 mm up to a bar diameter of 12 mm, and not less than 700 mm up to a bar diameter of 16 mm. At T- and L-intersections, the bars shall be continued across the full width of the intersection. At L-intersections, one outer bar shall be bent and continued 500 mm, or a bent lap bar 500 mm long shall be provided on each leg.

(f) Service penetrations are permitted through the middle third of the depth of edge and stiffening beams. The effect of other service penetrations shall be taken into account by the provision of extra concrete depth or reinforcement.



NOTE: The wire orientation is illustrative only.

FIGURE 5.1 ALTERNATIVE METHODS OF LAPPING OF MESH

5.3.3 Vapour barriers and damp-proofing membranes

5.3.3.1 *General*

Where required, the raft or slab shall be provided with a vapour barrier, or a damp-proofing membrane.

NOTE: In South Australia and New South Wales damp-proofing membranes are required. Their use is also recommended in areas prone to rising damp and salt attack.

5.3.3 Vapour barriers and damp-proofing membranes

5.3.3.1 *General*

Where required, the raft or slab shall be provided with a vapour barrier, or a damp-proofing membrane.

NOTE: In South Australia and New South Wales damp-proofing membranes are required. Their use is also recommended in areas prone to rising damp and salt attack.

5.3.3.2 *Materials*

The materials required for vapour barriers and damp-proofing membranes are as follows:

(a) 200 μm (0.2 mm) thick polyethylene film in accordance with Clause 5.3.3.3, Item (a), as follows:

  (i) *Vapour barrier*—medium impact resistance in accordance with Item 5.3.3.3 (b).

  (ii) *Damp-proofing membrane*—high impact resistance in accordance with Clause 5.3.3.3, Item (b), and resistant to puncture and penetration in accordance with Clause 5.3.3.3, Item (c).

(b) Film branded continuously 'AS 2870 Concrete underlay, 0.2 mm—Medium (or high as appropriate) impact resistance', together with manufacturer or distributors name, trademark or code.

5.3.3.3 *Properties*

Properties specified for vapour barriers and damp-proofing membranes shall be determined by the following methods:

(a) *Film thickness 0.2 mm*—shall be determined using the method of test outlined in AS/NZS 4347.9, except that three tests per metre width of film shall be carried out across the full width of the film, with the resulting mean average thickness to be between 180 μm and 220 μm and a maximum of only one measurement to be below 170 μm for a material pass to be recorded.

(b) *Impact resistance*—shall be determined using the falling dart impact test outlined in AS/NZS 4347.6 and the following:

  (i) Using a load of 180 g for medium impact film and 310 g for high impact film and a drop height of 660 mm, one test shall be carried out on the fold of the film and the film shall not fail.

  (ii) Using a load of 200 g for medium impact film and 340 g for high impact film and a drop height of 660 mm, two tests per metre width of film shall be carried out across the full width of the body of the film and 75% of these tests shall pass for a material pass to be recorded.

(c) *Resistance to puncture and moisture penetration*—shall be determined using the CSIRO 'Method for determination of the penetration resistance of water vapour barriers to falling aggregate'. Vapour permeance following this test shall not exceed 0.02 mg/N.s with no punctures or rips in the film.

5.3.3.4 *Installation*

Both vapour barriers and damp-proofing membranes shall be installed as follows:

(a) The sheet shall be placed beneath the slab so that the bottom surface of the slab and beams, including internal beams, is entirely underlaid. The membrane shall extend under the edge beam to ground level; however, where justified by satisfactory local experience, a vapour barrier may be terminated at the internal face of external beams as shown on Figure 5.2(a).

(b) Lapping for continuity at joints shall be not less than 200 mm.

(c) Penetrations by pipes or plumbing fittings shall be taped or sealed with a close-fitting sleeve or made continuous with the vapour barrier or damp-proof membrane by taping or by lapping in accordance with Item (b).

5.3.4 Edge rebates

Edge rebates for slab on ground, stiffened raft or waffle raft with masonry cavity or veneer construction shall comply with the following:

(a) The rebate depth shall be not less than 20 mm. The edge rebate may be stepped along its length.

(b) Where the edge rebate exceeds 150 mm in depth, the minimum horizontal width of the edge beam at the base of the rebate shall be not less than 200 mm, except that if R10 or N10 ties at 900 mm spacings are provided to resist vertical forces, this minimum width may be reduced to 150 mm. This requirement shall not apply to waffle rafts.

(c) The depth of concrete below the edge rebate shall be not less than 150 mm.

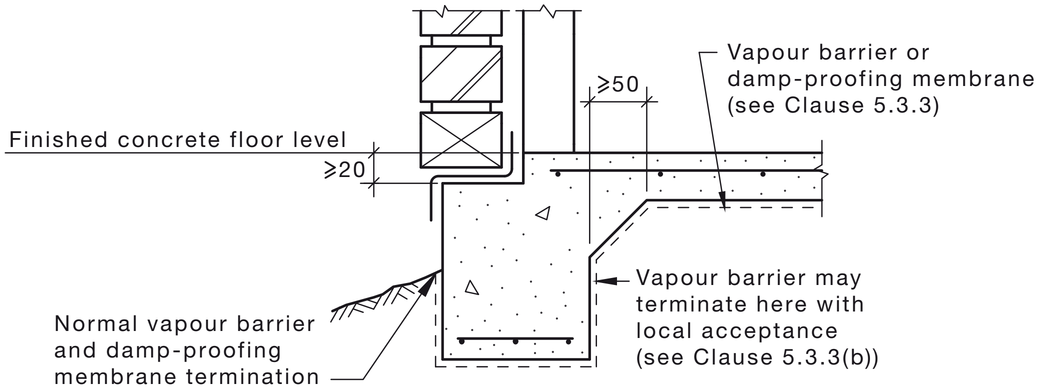
(d) Edge rebates are not required for construction with single-leaf masonry.

(e) Where the edge beams are retaining more than 450 mm of fill, the requirements of Clause 6.5.4 shall apply. Alternatively, the design shall be in accordance with engineering principles.

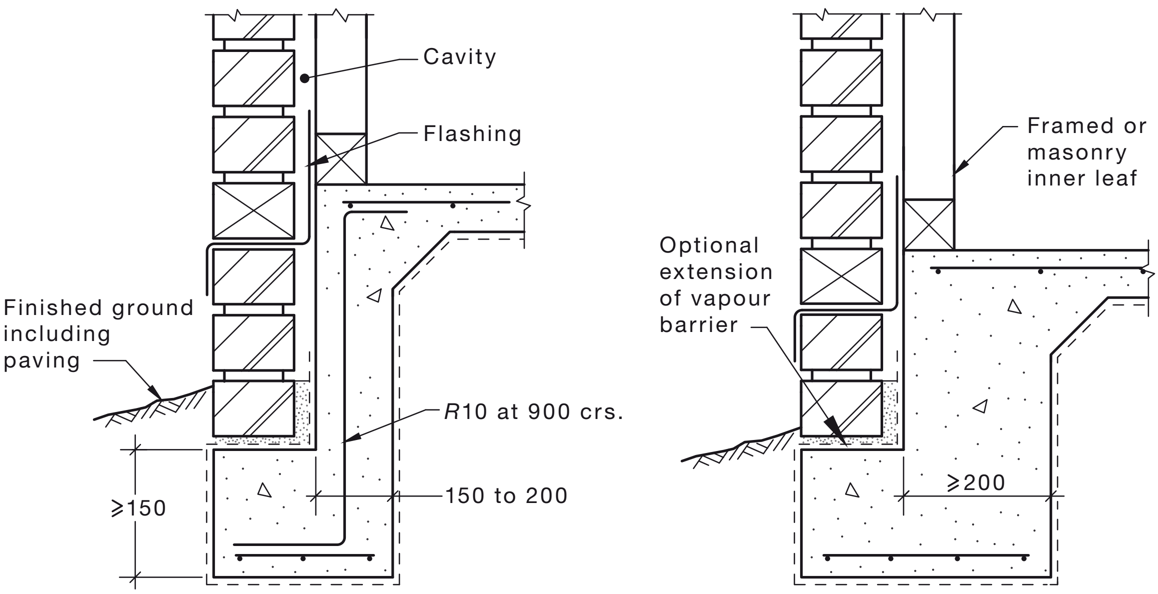
(f) Where the edge rebate depth is greater than 400 mm, the minimum stem width shall be 200 mm. The effect of the rebate shall be assessed in accordance with engineering principles.

Arrangements of the edge rebate are shown in Figure 5.2.

NOTE: Typical detailing for footings supporting single-leaf masonry walls is shown in the Commentary.



(a) Minimum rebate for cavity masonry or veneer wall



(b) Deep edge rebate alternative

NOTE: The cavity and flashing details shown are diagrammatic only (see AS 3700 and AS 3660.1).

DIMENSIONS IN MILLIMETRES

FIGURE 5.2 EDGE REBATE DETAILS

5.3.5 Recesses in slab panels

Where the raft or slab surface is recessed to provide for services, the soffit of the slab shall be deepened to maintain the required thickness and the reinforcement shall be continuous or lapped as shown in Figure 5.3.

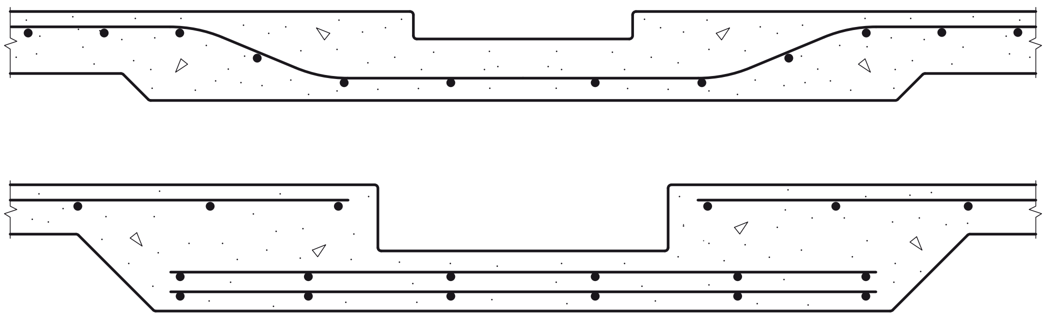


FIGURE 5.3 SLAB DETAIL AT A RECESS

5.3.6 Heating cables and pipes

Electric heating cables may be embedded in the slab without any increase in thickness or reinforcement.

Where hot water heating pipes are to be embedded in a slab, the slab thickness shall be increased by 25 mm and an increase made in the mesh of one level. (For example, SL72 for SL62, SL82 for SL72 and SL92 for SL82.) The mesh shall be placed at a suitable level to accommodate the pipes, subject to the requirements of Clause 5.3.2 (a).

5.3.7 Shrinkage cracking control

At re-entrant corners, two strips of 3-L8TM, or one strip of 3-L11TM or 3-N12 bar, shall be placed across the direction of potential cracking. All such reinforcement shall have a minimum length of 2 m.

Where brittle floor coverings are to be used over an area greater than 16 m^2, extra measures shall be taken to control shrinkage cracking. Such measures shall include one or more of the following:

(a) The amount of slab reinforcement in that part of the slab on which brittle finishes are to be applied shall be not less than SL92 or equivalent. Alternatively, an additional sheet of slab mesh shall be placed in those areas.

(b) The bedding system for brittle coverings shall be selected on the basis of the expected slab movement and the characteristics of the floor covering.

(c) The placement of floor coverings shall be delayed.

NOTE: A minimum period of three-months drying of the concrete is usually required before the placement of brittle floor coverings. Appendix B discusses performance criteria and foundation maintenance.

5.3.8 Beam continuity in rafts

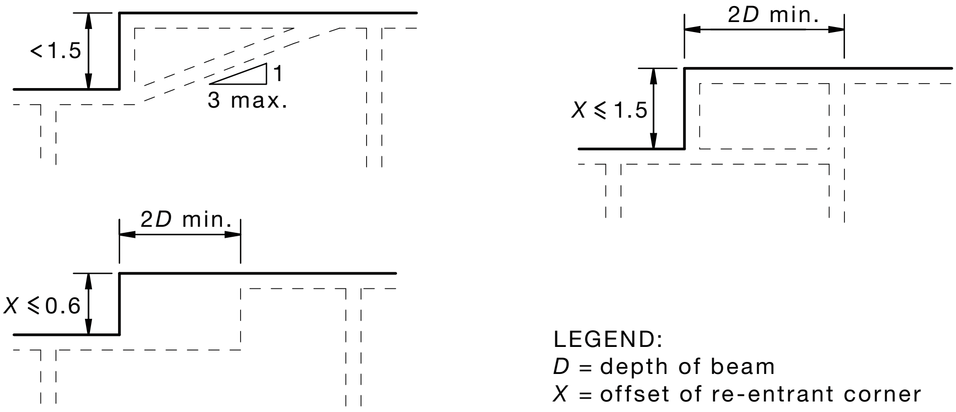
Where the raft design includes internal beams, the structural continuity of internal and external beams in stiffened rafts, including waffle rafts, shall be maintained in accordance with the following criteria.

Internal beams shall be continuous from edge to edge of the slab. Where beams are at different levels, as may occur in two-pour systems, special detailing is required to provide continuity. The requirements apply to stiffened rafts, including waffle rafts. Internal beams shall be located to provide continuity with the edge beams at re-entrant corners. Where one side of the re-entrant corner is less than 1.5 m, any one of the details specified in Figure 5.4 is deemed to provide continuity of beams.

At a re-entrant corner where an external beam continues as an internal beam, the external beam details shall be continued for a length of 1 m into the internal beam.

Where the footprint of the building includes courtyards or indentations resulting in discontinuity of internal or external stiffening beams, the overall strength and stiffness of the raft shall be maintained by the addition of either strip footings or additional stiffening beams.

NOTE: Examples are included in the Commentary.

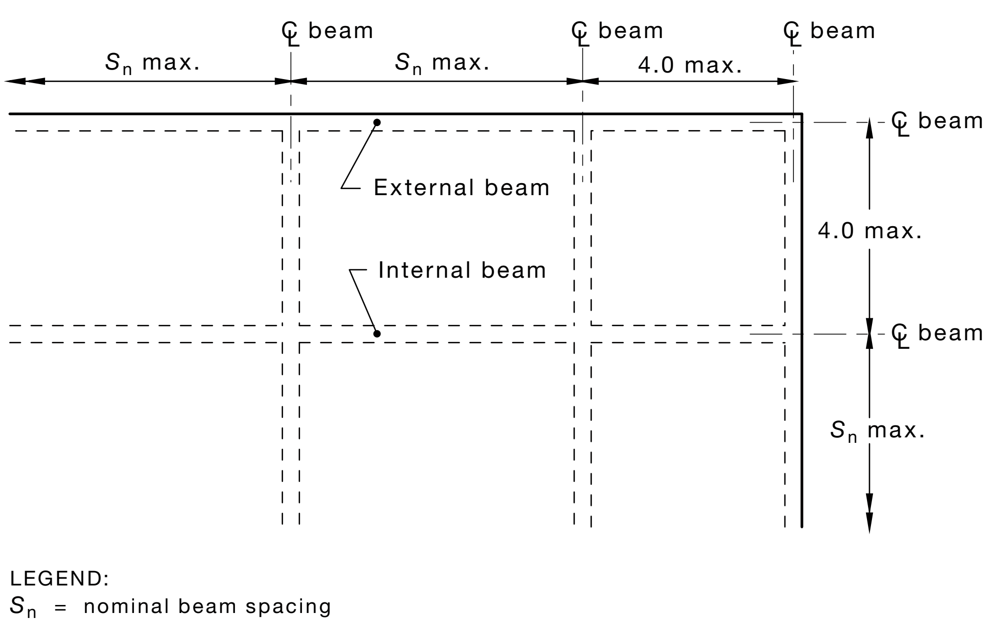


DIMENSIONS IN METRES

FIGURE 5.4 CONTINUITY OF FOOTING BEAMS

5.3.9 Beam layout restrictions

Where the raft design includes internal beams, at all external corners the maximum distance between the corner and the intersection of the first internal beam with the edge beam shall be 4.0 m, as shown in Figure 5.5.



DIMENSIONS IN METRES

FIGURE 5.5 BEAM SPACING AT EXTERNAL CORNERS

5.4 REQUIREMENTS FOR PAD AND STRIP FOOTINGS

5.4.1 Concrete

The grade of concrete shall be N20 with slump of 100 mm in accordance with AS 1379, with 20 mm maximum nominal aggregate size, or as provided in Clause 5.5, or as specified by the designer.

5.4.2 Reinforcement

Reinforcement in pad and strip footings shall comply with the following:

(a) Trench mesh reinforcement may be replaced by the equivalent reinforcing bars.

(b) Design cover to the reinforcement shall be 40 mm.

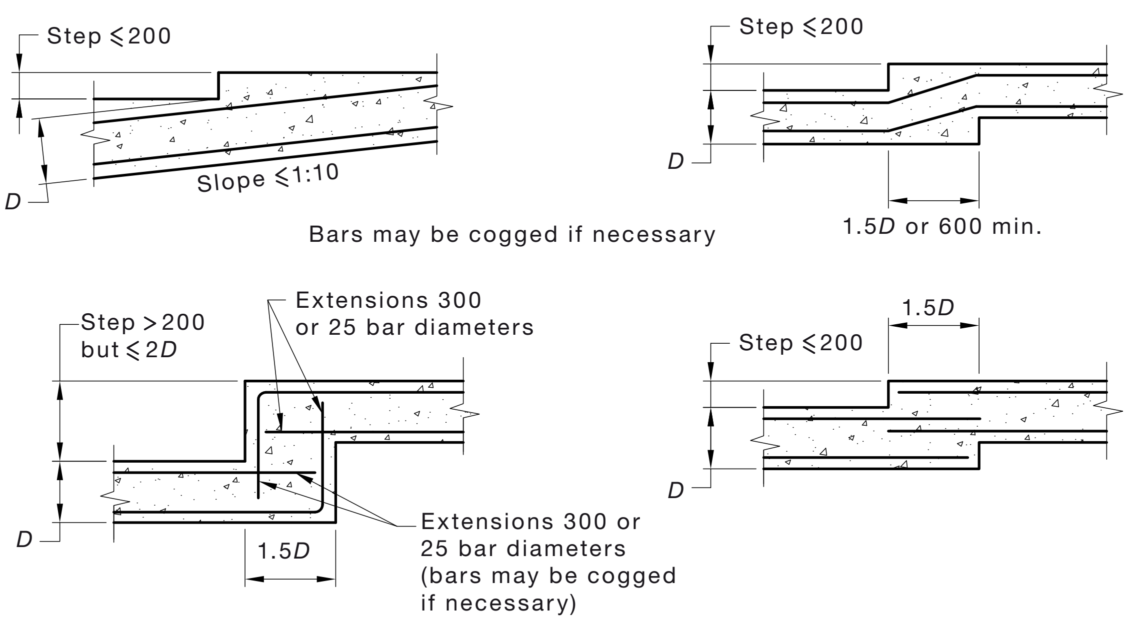
(c) Trench mesh in footings shall be anchored by the width of the mesh at T-intersections and L-intersections and shall be lapped by 500 mm at splices.

(d) The lap length of bar splices shall be not less than 500 mm. At T-intersections and L-intersections, the bars shall continue across the full width of the intersection. At L-intersections, one outer bar shall be bent and continued for 500 mm, or a bent lap bar 500 mm long shall be provided on each leg.

(e) Service penetrations are permitted through the middle third of the depth of the footing. The effect of other footing penetrations shall be taken into account by the provision of extra concrete depth or reinforcement.

5.4.3 Stepping of strip footings

The base of a strip footing shall be horizontal or at a slope of not more than 1:10, or the footing shall be stepped in accordance with one of the methods given in Figure 5.6.



DIMENSIONS IN MILLIMETRES

FIGURE 5.6 ACCEPTABLE METHODS OF STEPPING STRIP FOOTINGS

5.5 REQUIREMENTS IN AGGRESSIVE SOILS

5.5.1 General

In buildings with masonry and/or concrete surfaces exposed to saline soils or to acid sulfate soils with a magnesium content of 1000 ppm or more, the concrete raft, slab, strip or pad footing shall be protected from the aggressive soil or groundwater by—

(a) the isolation of the concrete or masonry member from the aggressive soil in accordance with Clause 5.5.2, or

(b) the application of the concrete strength and detailing requirements of Clause 5.5.3.

NOTE: In highly saline areas the likelihood of damage will be reduced if the requirements of both Clauses 5.5.2 and 5.5.3 are implemented. For acid sulfate and sulfate soils, additional recommendations are given in the Commentary.

5.5.2 Isolation of concrete from the ground

The concrete member shall be isolated from the aggressive soil or groundwater by a damp-proofing membrane in accordance with Clause 5.3.3, installed and terminated in accordance with one of the following:

(a) Terminated at or below finished ground or paving level as shown in Figure 5.7.

(b) Installed and terminated at finished ground or paving level and lapped with a suitable 0.5 mm thick damp-proofing material complying with AS/NZS 2904 and lapped a minimum of 75 mm vertically or horizontally in accordance with Figure 5.7. The damp-proofing material shall extend up to the finished ground or paving level, and be sealed around all penetrations by pipes or plumbing fittings.

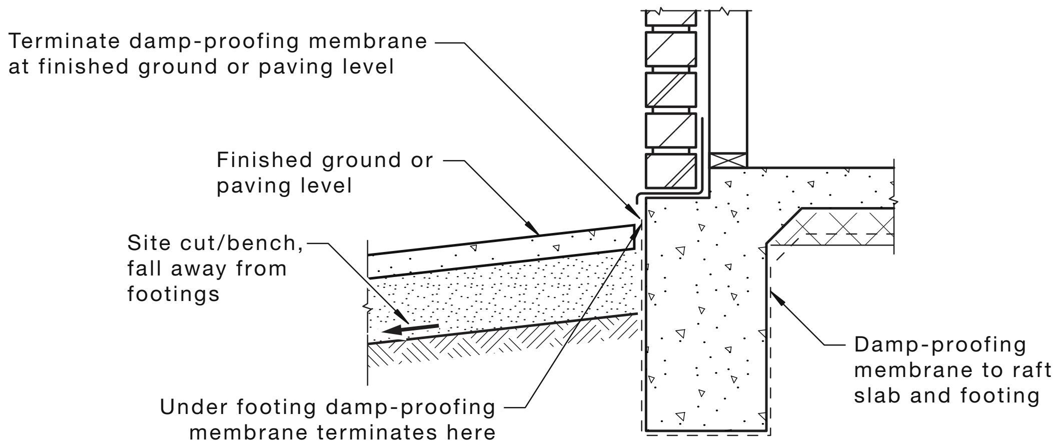
NOTE: A suitable 0.5 mm thick damp-proofing material may be embossed black polythene film of high impact resistance of 0.5 mm thickness prior to embossing and meeting the requirements of AS/NZS 2904.

(c) Installed and terminated at below finished ground or paving level and lapped a minimum 75 mm with a suitable liquid-applied waterproofing compound applied to the face of the concrete. The liquid-applied waterproofing compound shall extend up to the finished ground or paving level and be sealed around all penetrations by pipes or plumbing fittings (see Figure 5.7).

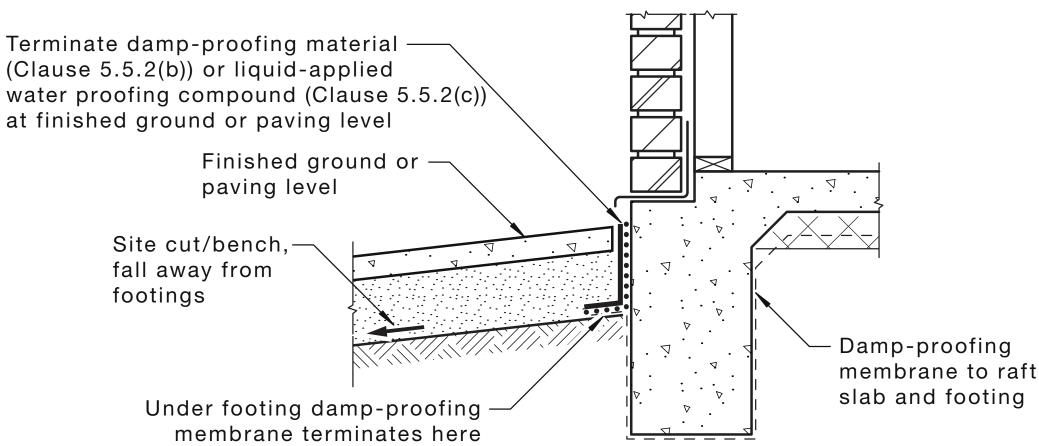
NOTE: A damp-proofing material may be used in addition to the liquid-applied coating during construction to avoid damage to the coating.

Where the damp-proofing membrane is damaged during installation, or the finished ground or paving level is altered, the provisions of either Item (b) or (c) shall be complied with.

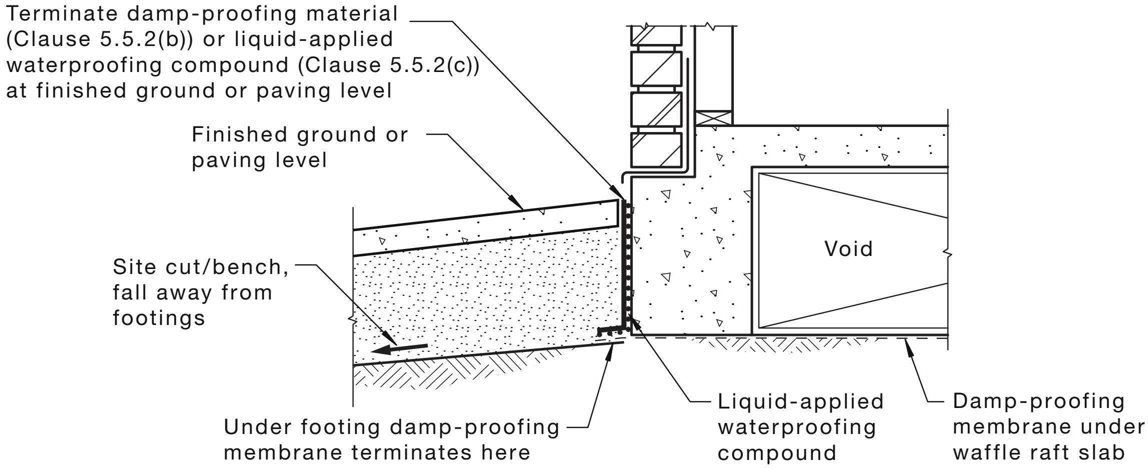
Where a damp-proofing membrane is installed, a layer of bedding sand shall be provided under the slab panels. Where this layer is deeper than 100 mm, it shall comply with Clause 6.4.2.



(a) Stiffened raft in accordance with Clause 5.5.2(a)



(b) Stiffened raft in accordance with Clause 5.5.2(b) or 5.5.2(c)



(c) Waffle raft in accordance with Clause 5.5.2(b) or 5.5.2(c)

FIGURE 5.7 USE OF DAMP-PROOFING MEMBRANE FOR SLAB PROTECTION

5.5.3 Concrete strength and detailing requirements

Concrete strength and detailing requirements shall be as follows:

(a) The exposure classification of the concrete in saline soils shall be in accordance with Table 5.1, where EC_e is the saturated electrical conductivity of a soil water extract.

(b) The exposure classification of the concrete in acid sulfate soils and sulfate soils shall be in accordance with Table 5.2.

(c) The concrete strength and curing requirements shall be as specified in Table 5.3. All concrete, including exposed edges of slabs or edge beams, shall be cured for the minimum period specified.

(d) Curing shall be achieved by the application of water to, or the retention of water in, the freshly cast concrete and shall commence as soon as practicable after finishing of any unformed surfaces has been completed. Where the retention of water in the fresh concrete relies on the application to exposed surfaces of sprayed membrane-forming curing compounds, the compounds used shall comply with AS 3799.

(e) A raft, waffle raft or slab shall be provided with a vapour barrier or damp-proofing membrane complying with the requirements of Clause 5.3.3 and installed in accordance with the details shown in Figure 5.9.

(f) All concrete shall be adequately compacted.

(g) The minimum reinforcement cover for concrete members in contact with the ground or protected by a vapour barrier shall be as specified in Table 5.4.

TABLE 5.1

EXPOSURE CLASSIFICATION FOR CONCRETE IN SALINE SOILS

Saturated extract electrical conductivity (EC_e), dS/m	Exposure classification
<4	A1
4–8	A2
8–16	B1
>16	B2

NOTES:

1 Guidance on concrete in saline environments can be found in CCAA T56.

2 Exposure classifications are from AS 3600.

3 The currently accepted method of determining the salinity level of the soil is by measuring the extract electrical conductivity (*EC*) of a soil and water mixture in deciSiemens per metre (dS/m) and using conversion factors that allow for the soil texture, to determine the saturated extract electrical conductivity (EC_e).

4 The division between a non-saline and saline soil is generally regarded as an EC_e value of 4dS/m, therefore no increase in the minimum concrete strength is required below this value.

TABLE 5.2

EXPOSURE CLASSIFICATION FOR CONCRETE IN SULFATE SOILS

Exposure conditions			Exposure classification	
Sulfates (expressed as SO_4)*		pH	Soil conditions A†	Soil conditions B‡
In soil ppm	In groundwater ppm			
<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 (e.g., sands and gravels) that are in groundwater

‡ Soil conditions B—low permeability soils (e.g., silts and clays) or all soils above groundwater

TABLE 5.3

MINIMUM DESIGN CHARACTERISTIC STRENGTH (f'_c) AND CURING REQUIREMENTS FOR CONCRETE

Exposure classification	Minimum f'_c MPa	Minimum initial curing requirement
A1	20	Cure continuously for at least 3 days
A2	25	
B1	32	Cure continuously for at least 7 days
B2	40	
C1	≥50	
C2	≥50	

TABLE 5.4

MINIMUM REINFORCEMENT COVER FOR CONCRETE

Exposure classification	Minimum cover in saline soils* mm	Minimum cover in sulfate soils† (mm)
A1	See Clause 5.3.2	40
A2	45	50
B1	50	60
B2	55	65
C1	‡	70
C2	‡	85

* Where a damp-proofing membrane is installed, the minimum reinforcement cover in saline soils may be reduced to 30 mm.

† Where a damp-proofing membrane is installed, the minimum reinforcement cover in sulfate soils may be reduced by 10 mm.

‡ Saline soils have a maximum exposure classification of B2 as per Table 5.1.

5.6 ADDITIONAL REQUIREMENTS FOR CLASSES M, H1, H2 AND E SITES

5.6.1 Masonry detailing

The following aspects of masonry detailing shall be used to reduce the effects of movement:

(a) Extensions shall be isolated from the original structure by means of control joints to allow for differential movement.

(b) In masonry construction, control joints shall be introduced at abrupt changes in profile such as at large openings or near corners except where the wall is designed to be reinforced masonry.

5.6.2 Variations in foundation material

If the footing or slab is partly on rock and partly on reactive clay, structural continuity of the entire footing shall be maintained and allowance shall be made for potential movement in the superstructure near the junction of foundation types (see also Clauses 3.1.6, 3.1.7 and 3.1.8).

5.6.3 Drainage requirements

Buildings on moderately, highly or extremely reactive sites shall be provided with drainage systems designed in accordance with the following:

(a) Surface drainage shall be considered in the design of the footing system and necessary modification shall be included in the design documentation. Surface drainage of the site shall be controlled from the start of site preparation and construction. The drainage system shall be completed by the finish of construction of the building.

(b) The base of trenches shall be sloped away from the building. Trenches shall be backfilled with clay in the top 300 mm within 1.5 m of the building. The clay used for backfilling shall be compacted. Where pipes pass under the footing system, the trench shall be backfilled full depth with clay or concrete to restrict the ingress of water beneath the footing system.

(c) Where pipes pass under the footing system, the trench shall be backfilled full depth with clay to act as a barrier to the ingress of water beneath the footing system. Alternatively, a plastic membrane across the cross-section of the trench, taped to the pipe and keyed into the sides and base of the trench may be used.

(d) Subsurface drains to remove groundwater shall not be used within 1.5 m of the building unless designed in accordance with engineering principles.

5.6.4 Plumbing requirements

Buildings on highly or extremely reactive sites shall be provided with a system of plumbing detailed in accordance with the following:

(a) Penetrations of the edge beams of a raft and perimeter strip footings shall be avoided where practicable, but where necessary shall be detailed to allow for movement.

Closed-cell polyethylene lagging shall be used around all stormwater and sanitary plumbing drain pipe penetrations through footings. The lagging shall be a minimum of 20 mm thick on Class H1 sites and 40 mm thick on Class H2 and Class E sites. Vertical penetrations do not require lagging.

NOTE: Sleeves allowing equivalent movements may be used as an alternative to the lagging.

(b) Drains attached to or emerging from underneath the building shall incorporate flexible joints immediately outside the footing and commencing within 1 m of the building perimeter to accommodate a total range of differential movement in any direction equal to the estimated characteristic surface movement of the site (y_s). In the absence of specific design guidance, the fittings or other devices that are provided to allow for the movement shall be set at the mid-position of their range of possible movement at the time of installation, so as to allow for movement equal to $0.5y_s$ in any direction from the initial setting. This requirement applies to all stormwater and sanitary plumbing drains and discharge pipes.

(c) On-site wastewater treatment units and associated land application areas shall be located to minimize soil moisture increase within the foundation.

(d) Drainage under a slab shall be avoided where practicable.

NOTES:

1 Pipes may be encased in concrete or in recesses in the slab when provided with flexible joints at the exterior of the slab.

2 Methods used should comply with the AS/NZS 3500 series.

(e) Cold water pipes and heated or hot water pipes shall not be installed under a slab, unless the pipes are installed within a conduit so that if the pipe leaks water it will be noticed above the slab or outside the slab and will not leak unnoticed under the slab.

NOTE: Water service pipes installed under concrete slabs should comply with the relevant requirements of AS/NZS 3500.1. Heated water service pipes installed under concrete slabs should comply with the relevant requirements of AS/NZS 3500.4.

SECTION 6 CONSTRUCTION REQUIREMENTS

6.1 GENERAL

The construction of footing systems designed in accordance with Sections 3, 4 and 5 shall comply with Clauses 6.2 to 6.5. For moderately, highly and extremely reactive sites, additional requirements are given in Clause 6.6.

6.2 PERMANENT EXCAVATIONS

Any vertical or near-vertical permanent excavation within 2 m of the building and deeper than 0.6 m in material other than rock shall be adequately retained or battered. The effects of excavations on drainage or foundation drying shall be considered.

6.3 TEMPORARY EXCAVATIONS

Temporary excavations in the area of the footing shall be carried out only where adequate support for the footing system is maintained. Examples of such temporary excavation include levelling of the building platform and trenching for services.

Where it is expected that future excavation in the area of the footing system may be required for maintenance of underground services, provision shall be made for continued support of the footings, for example by provision of piers to beneath the expected excavation level.

NOTE: Excavations should not extend below a line drawn 30° to the horizontal for sand, or 45° to the horizontal for clay, from the bottom edge of the edge beam, strip footing or pier without prior assessment in accordance with engineering principles.

6.4 CONSTRUCTION OF SLABS

6.4.1 General

The construction of slab footing systems including slab on ground, footing slab, stiffened slab with deep edge beam, stiffened raft and waffle raft shall comply with the requirements of Section 5 and of this Clause. For Classes H1, H2 and E sites additional requirements are given in Clause 6.6.

The methods given for construction on sloping sites assume that the site is not subject to landslip.

6.4.2 Filling

Filling used for the support of a slab shall be controlled fill or rolled fill as follows:

(a) *Controlled fill* Sand fill up to 0.8 m deep that is well compacted by a vibrating plate or vibrating roller in layers not more than 0.3 m thick is deemed to be controlled fill. For sand fill not containing gravel-sized material a blow count of 7 or more per 0.3 m using the penetrometer test described in AS 1289.6.3.3 is deemed to satisfy this requirement.

Non-sand fill up to 0.4 m deep that is well compacted by a mechanical roller in layers not more than 0.15 m thick is deemed to be controlled fill. Clay fill shall be moist during compaction.

(b) *Rolled fill* Rolled fill consists of material compacted in layers by repeated rolling with an excavator or similar equipment. The depth of rolled fill shall not exceed 0.6 m compacted in layers not more than 0.3 m thick for sand material or 0.3 m compacted in layers not more than 0.15 m thick for other material.

NOTE: The depths of fill given in this Clause are the depths measured after compaction.

6.4.3 Foundation for slabs

The foundation shall satisfy the following:

(a) Top soil containing grass roots or other organic material shall be removed from the area on which the slab is to rest.

(b) On sites subject to wind or water erosion, the foundation of the edge beam or footing shall be protected.

(c) The following apply to the foundation for slabs, including edge and internal beams:

(i) Where slab panels, edge beams, internal beams and load-support thickenings are to be supported on natural soil, the bearing capacity of the soil shall be not less than 50 kPa.

NOTE: Slab panels, internal beams and load support thickenings may be founded on controlled or rolled fill.

(ii) Edge beams may be founded on controlled fill. This fill shall continue past the edge of the building by at least 1 m and shall be retained or battered beyond this point by a slope not steeper than 1:2. Edge beams shall not be founded on rolled fill.

(iii) Edge footings not tied to a footing slab [see Figures 3.2 and 3.3 (b)] shall be founded in natural soil with a bearing capacity of 100 kPa or may be founded on controlled sand fill on a Class A or Class S site.

(d) The bases of edge beams and footings may be stepped or sloped not more than 1:10.

(e) Except for sites with aggressive soils as detailed in Clause 5.5.2, a blinding layer of sand is not required. Where used, the blinding layer of sand shall comply with Clause 6.4.2 if deeper than 100 mm.

6.4.4 Treatment of sloping sites

The treatment of slabs on cut-and-fill sloping sites shall comply with one of the following methods:

(a) The site shall be cut and filled and the fill [see Figures 6.1(a) and 6.1(b)] shall continue past the edge of the building by at least 1 m and shall be retained or battered beyond this point by a slope protected from erosion and not steeper than 1:2. The interior of the slab shall be founded on compacted material satisfying the requirements of Clause 6.4.3(c). The edge beams shall be founded on natural soil or on controlled fill or may be supported by piers designed in accordance with engineering principles.

(b) The site shall be cut and filled with fill material that satisfies the requirements of Clause 6.4.3(c) and the fill shall be retained at the edge in accordance with Clause 6.4.5 as shown in Figure 6.1(c).

(c) The slab and beams may be stepped in combination with methods in Item (a) or (b) above and with Figure 6.1(c) to reduce the extent of excavation or fill. At a change in elevation, the step shall comply with the following:

   (i) The ground behind the step shall be drained to prevent moisture build-up and the face of the slab step against the soil shall be waterproofed.

   (ii) The edge rebate requirements of Clause 5.3.4 shall be incorporated in the construction.

   (iii) Steps in stiffened rafts, including waffle rafts, shall be designed to preserve the structural continuity of the footing system.

   (iv) Steps in slabs for Class A and Class S sites shall comply with Figure 6.2 where the height of the step is less than 1.2 m. The masonry retaining wall shown in Figure 6.2 shall comply with Clause 6.4.5(b). Steps in beams shall comply with the principles of Clause 5.4.3.

(d) The site shall be cut and filled and, where the fill does not satisfy Clause 6.4.2, the slab shall be designed as pier-and-slab in accordance with the following:

   (i) The suspended slab shall be designed in accordance with AS 3600.

   (ii) On Class M, Class H1 or Class H2 sites, the strength and stiffness of the suspended slab shall be not less than required by Section 3.

   Where the fill consists of reactive clay, the fill shall be placed in a moist condition to minimize subsequent reactive soil movements.

   NOTE: On natural slopes greater than 1:8, benching and consideration of slope stability may be required.

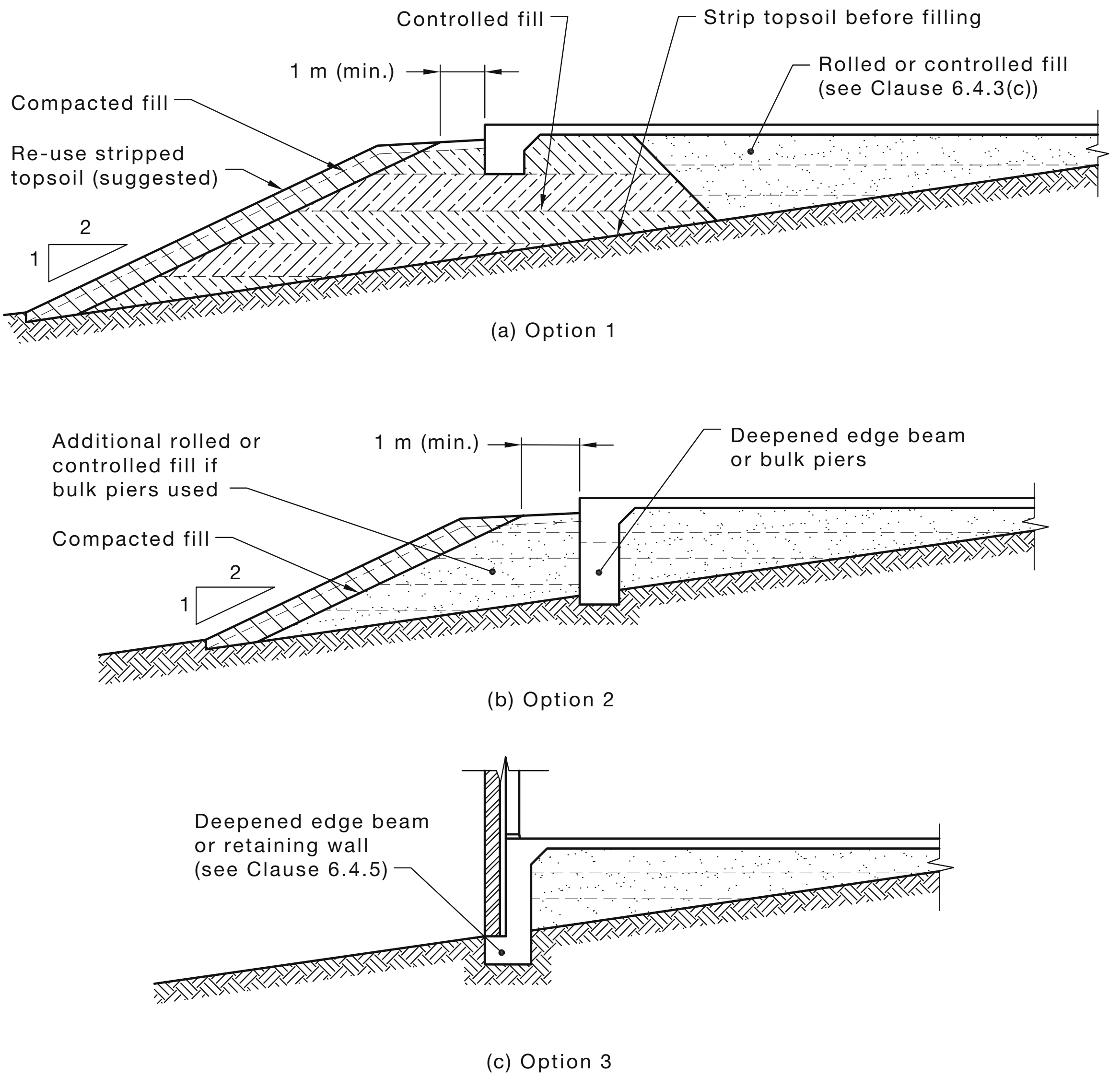
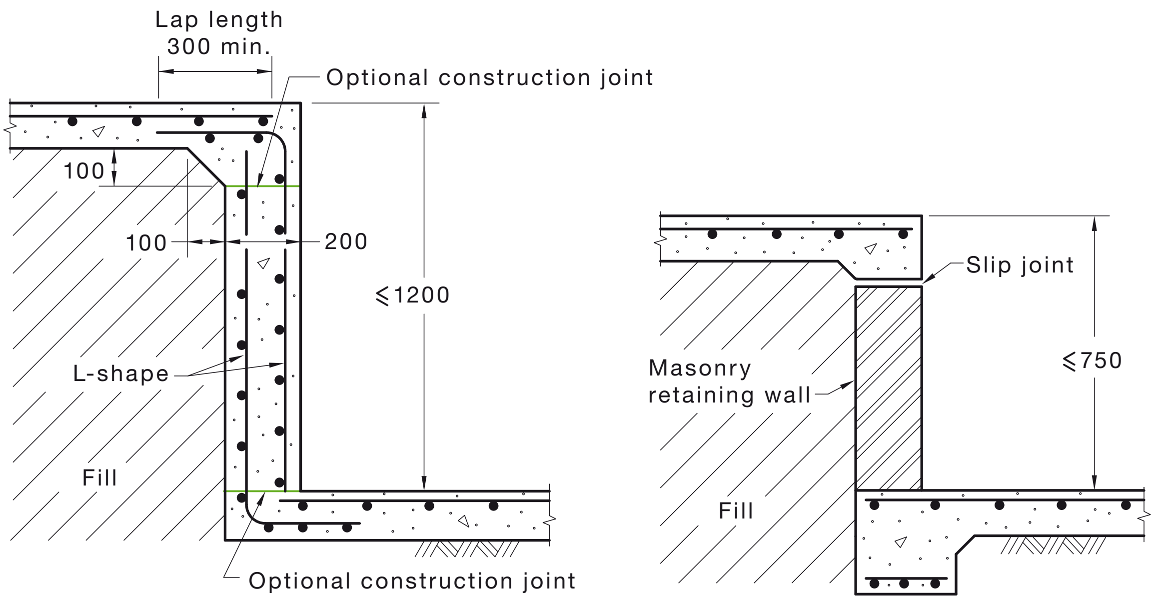


FIGURE 6.1 FILL AND SLAB EDGE OPTIONS FOR LOW SIDE OF SLOPED SITES



NOTE: Drainage provision should be made.

DIMENSIONS IN MILLIMETRES

FIGURE 6.2 SLAB STEP OPTIONS ON CLASS A OR CLASS S SITES

6.4.5 Retention of fill under slabs for Classes A, S and M sites

At the edge of a slab (or at a step) where more than 0.45 m of fill is retained, one of the following edge treatments shall be used:

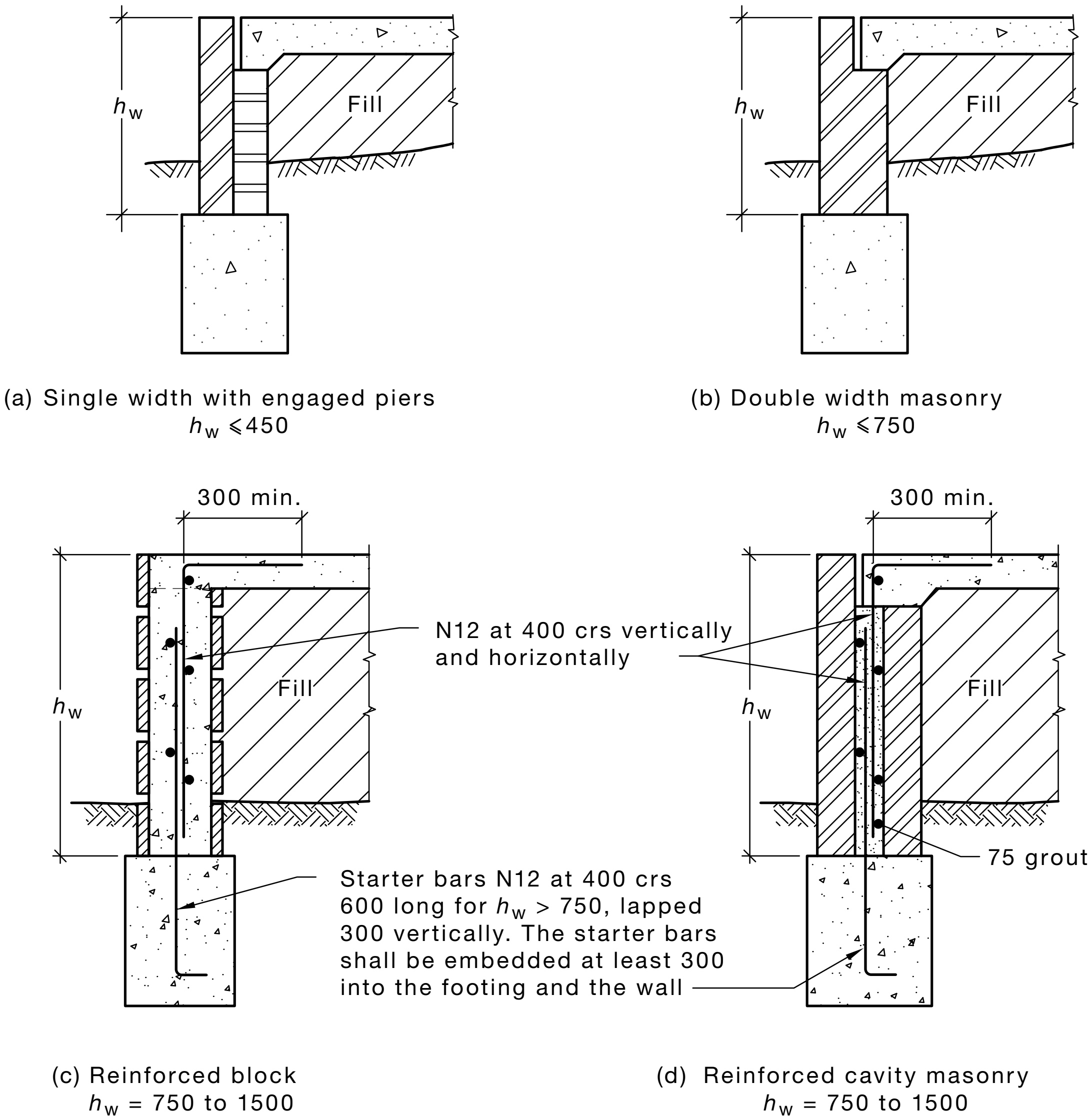
(a) The fill up to a height of 750 mm shall be retained by a deepened edge beam structurally continuous with the slab and of not less than 200 mm width. If the fill is greater than 0.75 m but not more than 1.2 m in depth, vertical reinforcement of centrally placed SL82 mesh shall be provided. Where the height exceeds 1.2 m, the edge beam shall be designed by engineering principles.

(b) Where the fill is retained by a masonry wall, the following shall be satisfied:

   (i) The methods of construction shall be as shown in Figure 6.3. Compaction of the fill shall be undertaken in a manner that does not cause damage to the wall.

   It is recommended that clay fill be avoided; however, where used, it shall be placed in a moist condition.

   (ii) For footing slabs on Class M sites, the slab and the footing shall be tied by N12 bars at 400 mm centres.



Wall height (*h*)	Wall construction
h_w < 450	Single width masonry with engaged piers at 1200 centres
$h_w \leq 750$	Double width masonry wall 230 thick. Solid or filled concrete block wall 200 nominal thickness
$h_w \leq 1\,500$	Double width masonry with a 75 filled cavity or a 200 filled block wall reinforced with tied N12 bars at 400 spacing horizontally and vertically. For h_w > 750, the wall and footing shall be tied to the slab. Cavity filling shall be well compacted 20 MPa concrete or grout in accordance with AS 3700.
h_w > 1 500	Designed in accordance with engineering principles

LEGEND:

h_w = maximum height of masonry wall retaining structure

NOTE: Drainage provisions should be made.

DIMENSIONS IN MILLIMETRES

FIGURE 6.3 STRUCTURAL DETAILS FOR WALLS RETAINING NON-REACTIVE FILL UNDER SLAB

6.4.6 Fixing of reinforcement and void formers

Reinforcement and void formers shall be fixed in position prior to concreting by means of proprietary spacers, bar chairs with bases, ligatures or other appropriate fixings so as to achieve the required reinforcement position and concrete covers. Reinforcement shall not be placed or located after concreting.

6.4.7 Placing, compaction and curing of concrete

The concrete shall be transported, placed, compacted and cured in accordance with good building practice.

6.5 CONSTRUCTION OF STRIP AND PAD FOOTINGS

6.5.1 General

The construction of strip and pad footings shall comply with Clause 6.5.2. For Class H1, Class H2 or Class E sites, additional requirements are given in Clause 6.6.

6.5.2 Foundation

For the strip and pad footing designs in Section 3, the foundation shall satisfy the following:

(a) The foundation shall have minimum bearing capacity of 100 kPa or the footing shall be founded on controlled sand fill on a Class A or Class S site.

(b) Topsoil containing grass roots shall be removed from the area on which the footing is to rest.

(c) On sandy sites or sites subject to wind or water erosion, the minimum depth below finished ground level of the underside of the footing shall be 300 mm.

(d) Trenches shall be dewatered and cleaned prior to concrete placement such that no significant softened or loosened material remains.

6.6 ADDITIONAL REQUIREMENTS FOR MODERATELY, HIGHLY AND EXTREMELY REACTIVE SITES

For stiffened rafts, waffle rafts, or strip footings on moderately, highly and extremely reactive sites, the following requirements apply to the building services and footing system in addition to the requirements of Clauses 6.4 and 6.5:

(a) Where the design of the footing system relies on particular detailing of masonry construction to minimize any damage caused by foundation movement, that detailing shall be included on the drawings.

(b) Penetrations of the edge beam and footing by drain pipes shall be sleeved using closed-cell polyethylene lagging or similar.

(c) During construction, water run-off shall be collected and channelled away from the building.

(d) Excavations near the edge of the footing system shall be backfilled in such a way as to prevent access of water to the foundation as described in Clause 5.6.3(b).

    NOTES:

    1 For example, excavations should be backfilled above or adjacent to the footing with moist clay compacted by hand-rodding or tamping.

    2 Porous material such as sand, gravel or building rubble should not be used.

(e) Water shall not be allowed to pond in the trenches.

For slab or strip footings on highly and extremely reactive sites, the following requirements apply:

(i) Drains attached to or emerging from underneath the building shall incorporate flexible joints immediately outside the footing and commencing within 1 m of the building perimeter to accommodate a total range of differential movement in any direction equal to the estimated characteristic surface movement of the site (y_s). In the absence of specific design requirements, the fittings or other devices that are provided to allow for the movement shall be set at the mid position of their range of possible movement at the time of installation, so as to allow for movement equal to $0.5y_s$ in any direction from the initial setting. This requirement applies to all stormwater and sanitary plumbing drains and discharge pipes.

(ii) Concrete in beams shall be mechanically vibrated.

APPENDIX A

FUNCTIONS OF VARIOUS PARTIES

(Informative)

This Standard is based on the general assumption that one or more of the parties listed below are involved in the design, construction and maintenance of residential slabs and footings, and their functions and responsibilities are as follows:

(a) *Classifier* The classifier is the person or organization responsible for classifying the site. Classification of a site should be carried out by a qualified engineer or engineering geologist experienced in the field of geomechanics; however, where there is established local knowledge, classification may be carried out by the builder, except where otherwise stated.

(b) *Designer* The designer is the person or organization responsible for the design of the footing system. When the design consists of the selection of a design given in Section 3 for a Class A, Class S or Class M site, the designer should be experienced in residential building design or construction. For Class P, Class H1, Class H2 or Class E sites, the designer should be a qualified engineer experienced in the design of footing systems for buildings.

(c) *Builder* The builder is the person or organization responsible for the construction of the entire building in accordance with the plans and specifications. The builder should be experienced in footing construction and, where required by State legislation, should be licensed.

(d) *Owner* The owner is the person or organization responsible for the maintenance of the building and the site. The owner should be familiar with the performance and maintenance recommendations set out in Appendix B.

(e) *Qualified engineer* A professional civil engineer specializing in either geotechnical or structural engineering and experienced in the design of footing systems for buildings or similar structures.

APPENDIX B

FOUNDATION PERFORMANCE AND MAINTENANCE

(Informative)

B1 GENERAL

The designs and design methods given in this Standard are based on the performance criteria of Clause 1.3. Importantly, significant damage may be avoided provided the foundation site conditions are properly maintained. This is expressed in Section 1 by the statement that the probability of failure for reasonable site conditions is low, but is higher if extreme conditions are encountered. It is neither practicable nor economical to design for the extreme conditions that could occur in the foundation if a site is not properly maintained. The expected standard of foundation maintenance is described in Paragraph B2.

Some minor cracking and movement will occur in a significant proportion of buildings, particularly those on reactive clays, and the various levels of damage are discussed in Paragraph B3.

The performance requirements of a concrete floor in respect to shrinkage cracking and moisture reaction with adhesives are discussed in Paragraph B4.

A more extensive discussion of the material in Paragraphs B2 to B4 is contained in the CSIRO pamphlet, Building Technology File 18, *Foundation maintenance and footing performance: A homeowner's guide*, and its recommendations should be followed.

B2 FOUNDATION MAINTENANCE

B2.1 Foundation soils

All soils are affected by water. Silts are weakened by water and some sands can settle if heavily watered, but most problems arise on clay foundations. Clays swell and shrink due to changes in moisture content and the potential amount of the movement is implied in the site classification in this Standard, which is designated as follows:

(a) A — stable (non-reactive).

(b) S — slightly reactive.

(c) M — moderately reactive.

(d) H1 and H2 — highly reactive.

(e) E — extremely reactive.

Sites classified Class A and Class S may be treated as non-reactive sites in accordance with Paragraph B2.2. Sites classified as Class M, Class H1, Class H2 and Class E should comply with the recommendations given in Paragraph B2.3.

B2.2 Class A and Class S sites

Sands, silts and clays should be protected from becoming extremely wet by adequate attention to site drainage and prompt repair of plumbing leaks.

B2.3 Classes M, H1, H2 and E sites

Sites classified as M, H1, H2, or E should be maintained at essentially stable moisture conditions and extremes of wetting and drying prevented. This will require attention to the following:

(a) *Drainage of the site* The site should be graded or drained so that water cannot pond against or near the building. The ground immediately adjacent to the building should be graded to a uniform fall of 50 mm minimum away from the building over the first metre. The subfloor space for buildings with suspended floors should be graded or drained to prevent ponding where this may affect the performance of the footing system.

The site drainage recommendations should be maintained for the economic life of the building.

(b) *Limitations on gardens* The development of the gardens should not interfere with the drainage requirements or the subfloor ventilation and weephole drainage systems. Garden beds adjacent to the building should be avoided. Care should be taken to avoid overwatering of gardens close to the building footings.

(c) *Restrictions on trees and shrubs* Planting of trees should be avoided near the foundation of a building or neighbouring building on reactive sites as they can cause damage due to drying of the clay at substantial distances. To reduce, but not eliminate, the possibility of damage, tree planting should be restricted to a distance from the house as follows:

(i) $1\frac{1}{2} \times$ mature height for Class E sites.

(ii) $1 \times$ mature height for Class H1 and Class H2 sites.

(iii) $\frac{3}{4} \times$ mature height for Class M sites.

Where rows or groups of trees are involved, the distance from the building should be increased. Removal of trees from the site can also cause similar problems.

Alternatively, the footing system may be designed for the effect of trees, for example as given in Appendix H.

(d) *Repair of leaks* Leaks in plumbing, including stormwater and sewerage drainage, should be repaired promptly.

The level to which these measures are implemented depends on the reactivity of the site. The measures apply mainly to masonry buildings and masonry veneer buildings. For frame buildings clad with timber or sheeting, lesser precautions may be appropriate.

B3 PERFORMANCE OF WALLS

It is acknowledged that minor foundation movements occur on nearly all sites and that it is impracticable to design a footing system that will protect the building from movement under all circumstances. The expected performance of footing systems designed in accordance with the Standard is defined in terms of the damage classifications in Table C1, Appendix C.

Crack width is used as the major criterion for damage assessment, although tilting and twisting distortions can also influence the assessment. Local deviations of slope of walls exceeding 1:150 are undesirable. The assessment of damage may also be affected by where it occurs and the function of the building, although these effects are not likely to be significant in conventional buildings. In the classification of damage, account should also be taken of the history of cracking. For most situations Category 0 or 1 should be the limit; however, under adverse conditions, Category 2 should be expected although such damage should be rare. Significant damage is defined as Category 3 or worse.

For Category 1 or 2 damage, remedial action should consist of stabilizing the moisture conditions of the clay and paying attention to repairing or disguising the visual damage. This should be regarded as part of the normal maintenance of buildings on reactive clays.

Even significant masonry cracking with crack widths over 5 mm often has no influence on the function of the wall and only presents an aesthetic problem. Generally, the remedial action for such damage should start with an investigation to establish the cause of the damage. In many cases the treatment should consist of stabilizing moisture conditions by physical barriers or paths, or replenishing moisture in dry foundations. This may be followed by repair of the masonry and, wherever possible, added articulation should be included while repairs are being effected. Structural repairs to the footing system, such as deep underpinning, should only be considered as the last resort.

Underpinning should generally be avoided where the problem is related to reactive clays, although it is recognized there may be occasional situations where underpinning or other structural augmentation work is appropriate. None of this structural augmentation work should be undertaken without proper engineering appraisal.

In some cases, walls may be designed to span sagging footings and cantilever beyond hogging footings. In such cases, satisfactory performance will involve the wall remaining free of cracks and articulation joint movements, and remaining within the limits for the particular jointing system.

B4 PERFORMANCE OF CONCRETE FLOORS

Shrinkage cracking can be expected in concrete floors. Concrete floors can also be damaged by shrinkage or swelling of reactive clays or settlement of fill. The categories of movement causing the damage are given in Table C2, Appendix C. In the classification, account should be taken of whether the damage is stable or likely to increase, and an allowance should be made for any deviations in level which resulted from, or occurred during, construction.

The time of attachment of floor coverings and the selection of the adhesive for them should take into account the moisture in the concrete floor and its possible effect on adhesion. Concrete floors can take a considerable time to dry (three to nine months).

Floor coverings and their adhesives can be damaged by moisture in the concrete and by the shrinkage that occurs as the concrete dries. The time of fixing of floor coverings and the selection of the adhesive should take these factors into account.

APPENDIX C

CLASSIFICATION OF DAMAGE DUE TO FOUNDATION MOVEMENTS

(Normative)

Classification of damage with reference to wall is given in Table C1. Classification of damage with reference to concrete floors is given in Table C2.

TABLE C1

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	**Approximate crack width limit (see Note 1)**	**Damage category**
Hairline cracks	<0.1 mm	0 Negligible
Fine cracks that do not need repair	<1 mm	1 Very slight
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2 Slight
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weather tightness often impaired	5 mm to 15 mm (or a number of cracks 3 mm or more in one group)	3 Moderate
Extensive repair work involving breaking out and replacing sections of walls, especially over doors and windows. Window frames and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15 mm to 25 mm but also depends on number of cracks	4 Severe

NOTES:

1 Where the cracking occurs in easily repaired plasterboard or similar clad-framed partitions, the crack width limits may be increased by 50% for each damage category.

2 Crack width is the main factor by which damage to walls is categorized. The width may be supplemented by other factors, including serviceability, in assessing category of damage.

3 In assessing the degree of damage, account shall be taken of the location in the building or structure where it occurs, and also of the function of the building or structure.

TABLE C2

CLASSIFICATION OF DAMAGE WITH REFERENCE TO CONCRETE FLOORS

Description of typical damage	Approx. crack width limit in floor	Change in offset from a 3 m straightedge centred over defect (see Note 1)	Damage category
Hairline cracks, insignificant movement of slab from level	<0.3 mm	<8 mm	0 Negligible
Fine but noticeable cracks. Slab reasonably level	<1.0 mm	<10 mm	1 Very slight
Distinct cracks. Slab noticeably curved or changed in level	<2.0 mm	<15 mm	2 Slight
Wide cracks. Obvious curvature or change in level	2 mm to 4 mm	15 mm to 25 mm	3 Moderate
Gaps in slab. Disturbing curvature or change in level	4 mm to 10 mm	>25 mm	4 Severe

NOTES:

1 The straightedge is centred over the defect, usually, and supported at its ends by equal height spacers. The change in offset is then measured relative to this straightedge, which is not necessarily horizontal.

2 Local deviation of slope, from the horizontal or vertical, of more than 1:100 will normally be clearly visible. Overall deviations in excess of 1:150 is undesirable.

3 Account should be taken of the past history of damage in order to assess whether it is stable or likely to increase.

APPENDIX D

SITE CLASSIFICATION BY SOIL PROFILE IDENTIFICATION

(Normative)

In some areas, where sufficient data have been established, site classification of a reactive clay soil profile may be associated with the typical soil profiles given for sites in Tables D1, D2, D3 and D4 for the regions associated with each table. Where variable soil conditions are expected across a site, the Tables shall be used only as an aid to a site investigation. Where soil profiles are relatively consistent, geological or pedological maps may be used to assist in classifying a site. The soil profile shall be checked by a site visit before construction proceeds and the site classification updated if necessary.

The classification of sites for regions other than those in the Tables may be based on an appropriate Table, provided the climates and soil types and soil profiles are similar between the regions.

The levels of classification expressed in the Tables relate to 'normal' site conditions as defined in Clause 1.3.2 of this Standard.

NOTES:

1 'Depth of clay' refers to the thickness of the clay in the soil profile within the depth of H_s (see Table 2.4).

2 Where a range of site Classes is given, the classification may be based on the depth of clay, the depth of a permanent water table, if present, and a visual assessment of the soil reactivity.

TABLE D1

CLASSIFICATION BASED ON TYPICAL PROFILES—VICTORIA

Soil profiles	Climatic zone			
	1	2	3	4–5
Group (1) soils				
Clays derived from limestones, marls, and other calcium-rich sediments. Including alluvial clays and calcareous earths derived from these deposits.				
≤0.6 m depth of clay over massive rock	M	M	M	M-D
>0.6 m to ≤1 m depth of clay over massive rock	M to H1	H1 to H2	H1 to H2	H1-D to H2-D
>1.0 m depth of clay over massive rock	H2	H2	H2 to E	E-D
Group (2) soils				
Clays derived from alkaline volcanics (e.g. basalts, dolerites, greenstones) or sedimentary rocks with interbedded alkaline volcanics or pyroclastics.				
Including alluvial clays derived from any of these deposits.				
≤0.6 m depth of clay over massive rock	M	M	M	M-D
>0.6 m to ≤1.5 m depth of clay over massive rock	M	M to H1	M to H1	M-D to H1-D
>1.5 m depth of clay over massive rock	H1	H1 to H2	H2 to E	H2-D to E-D
Deep lateritic, gravelly or coarse sandy clay profiles (see Note 1)	M	M	M to H1	H1-D to H2-D
Group (3) soils				
Non-basaltic and non-calcareous residual clays derived from sedimentary, metamorphic, granitic or other acid volcanic rocks.				
≤1.0 m depth of clay over massive rock	M	M	M	M-D
>1.0 to ≤1.8 m depth of clay over massive rock	M	M	M to H1	M-D to H1-D
>1.8 m depth of clay* over massive rock	M	M to H1	M to H1	H1-D to H2-D
Group (4) soils				
Alluvial, glacial and estuarine soils silts, sands or gravels which overlie deep clays. The sand cover may be aeolian (wind-blown). The classification is highly dependent on clay type, total thickness and its proximity to the surface.				
≤0.6 m silts, sands or gravels overlying deep clays	M	M to H1	M to H2	H1-D to E-D
>0.6 to ≤1 m silts, sands or gravels overlying deep clays	S	S to M	M to H1	M-D to H2-D
>1 m silts, sands or gravels overlying deep clays	A	S	S to M	M-D
Group (5) soils				
Interbedded silts, sands, and clay mixtures. The classification is highly dependent on clay type, depth, thickness and its proximity to the surface				
Total clay depth <50% of H_s depth	S	S to M	S to M	M-D to H2-D

NOTES:

1 Where the sites are to be excavated for levelling purposes, the worst case soil profile shall be used for classification.

2 Maps of climate zones are presented in Figures D1 and D2. These Tables should only be used by practitioners with local geological knowledge and experience derived from many years of successful investigations.

3 The above classifications may not apply to sites that have organic or peaty soils, collapsing soils, unstable or creeping slopes, mining works and conditions that have or may cause abnormal foundation soil moisture.

4 Where a range of classifications is provided, the classifier should make a judgement within this range based on site conditions and local experience or y_s calculated by using the I_{pt} for each soil stratum.

TABLE D2

CLASSIFICATION OF ALL SYDNEY CLAY SOILS

Depth of clay in profile m	Classification
<0.6	S
≤0.6 to ≤1.8	M
>1.8	H1 to H2

NOTE: The H1 to H2 classification arises from the possibility of moisture changes at depths in excess of 1.8 m because of changing groundwater regimes, and hence the depth of design suction change of Section 2 is inappropriate. Some less reactive soils do occur and, if a check is desired, the methods of Section 2 may be used, but with a depth of design suction change equal either to a maximum depth of 2 m or to the depth from the surface to extremely to highly weathered rock. In addition, the crack depth should be taken as 0.5 m.

TABLE D3

SITE CLASSIFICATION BASED ON LOCATION AND TYPICAL PROFILE—PERTH

Examples	Range
Clays derived from weathered dolerite in Darling Range or along foothills	M to H2
Clay material of Guildford formation	S to H2

NOTES:

1 Actual classification of clay material of Guildford formation depends on depth of sand cover and clay type.

2 Refer to 1:50 000 Scale Environmental Geology Map Series, published by the Department of Minerals and Energy, Western Australia.

TABLE D4

SITE CLASSIFICATION BASED ON LOCATION AND TYPICAL PROFILE—ADELAIDE

Soil group	Typical soil types	Classification
Silts sands and gravels	Sand A1, DS EMS	A to S
Shallow clays (over rock)	SR	S
Silty and sandy clays (less reactive)	Clayey A1, RZ, TR, P4, SW	M-D
Podsolic and solodic soil	P1, P2, P3 and S	S to H2-D
Red brown soils		
Profiles with shallow layers of less reactive clay	RB2, RB4, RB6, RB7, RB9	M-D to H2-D
Profiles with deeper layers of more reactive clay	RB1, RB3, RB5, RB8	H2-D to E-D
Hindmarsh or Keswick clay underlying any soil (except black earth)		
Depth to clay > 2 m		H2-D to E-D
Depth to clay from 1 m to 2 m		E-D
Black earth	BE	E-D

NOTE: Typical soil type profiles are illustrated in Sheard, M. J. and Bowman, G. M. (1996). *Soils, Stratigraphy and Engineering Geology of Near Surface Materials of the Adelaide Plains*. Dept. Mines and Energy (PIRSA), Report Book 94/9, Volumes 1, 2 and 3, Adelaide.

FIGURE D1 MELBOURNE AND ENVIRONS CLIMATIC ZONES
(Outside Melbourne inset area—refer Figure D2)

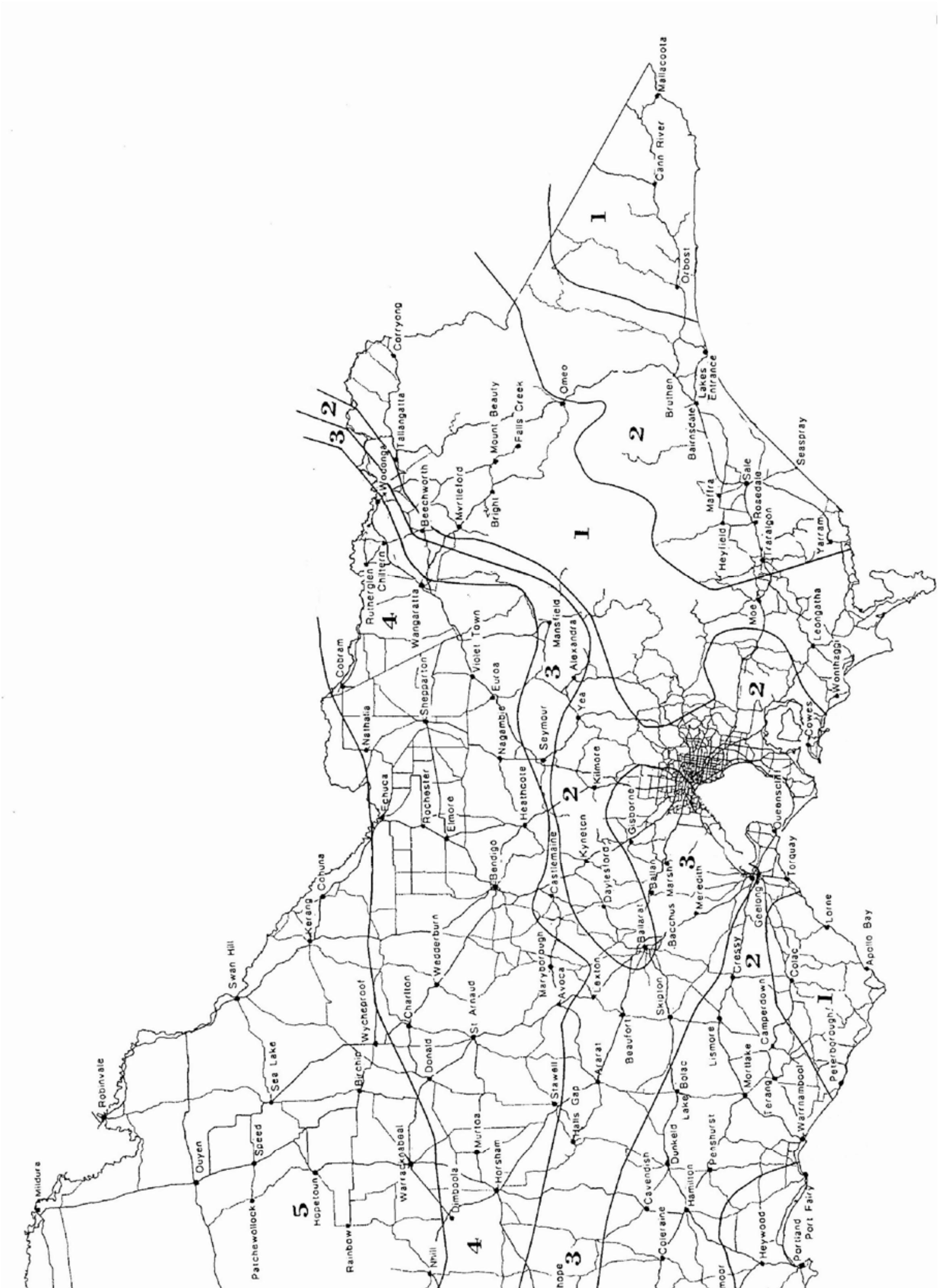


FIGURE D2 VICTORIAN CLIMATIC ZONES

APPENDIX E

STUMP PAD SIZES, BRACED STUMP UPLIFT HORIZONTAL LOAD CAPACITY

(Normative)

E1 GENERAL

Stumps positioned beneath the floor shall be designed for vertical gravity loads, vertical uplift loads and horizontal forces (where applicable). This Appendix is applicable to braced stumps only and is not applicable to bracing stumps.

E2 VERTICAL GRAVITY LOAD CAPACITY

The vertical gravity load capacity shall be calculated by the area of the footing and the assessed bearing capacity. Pad footing systems shall comply with Figure E1. Braced stumps with combined gravity loads (no net uplift) and horizontal loads shall comply with Figure E1 for gravity loads and Table E2 or Table E4 for horizontal design strength. No allowance needs to be made for combined effects.

E3 UPLIFT AND HORIZONTAL CAPACITY

The uplift and horizontal design strength of braced stumps shown in Figure E2 shall be determined from Tables E1 to E4. The design action effects U^* and H^* due to design load for strength shall not exceed the following limits:

$\frac{U^*}{U}$ <1.0 and $\frac{H^*}{H}$ <1.0 for Class A and Class S sites

<0.9 <1.0 for Class M site

<0.7 <0.8 for Classes M-D, H1, H1-D, H2 and H2-D sites

and for combined uplift and horizontal load,

$\frac{U^*}{U}+\frac{H^*}{H}$ <1.0 for Class A and Class S sites

<0.9 for Class M sites

<0.7 for Classes M-D, H1, H1-D, H2 and H2-D sites

where

U^* = design uplift load on stump, see Figure E2

H^* = design horizontal load on stump, see Figure E2

U = $\phi\, U_{\mathrm{ULT}}$, geotechnical design capacity of stump in uplift

H = $\phi\, H_{\mathrm{ULT}}$, geotechnical design capacity of stump for horizontal load

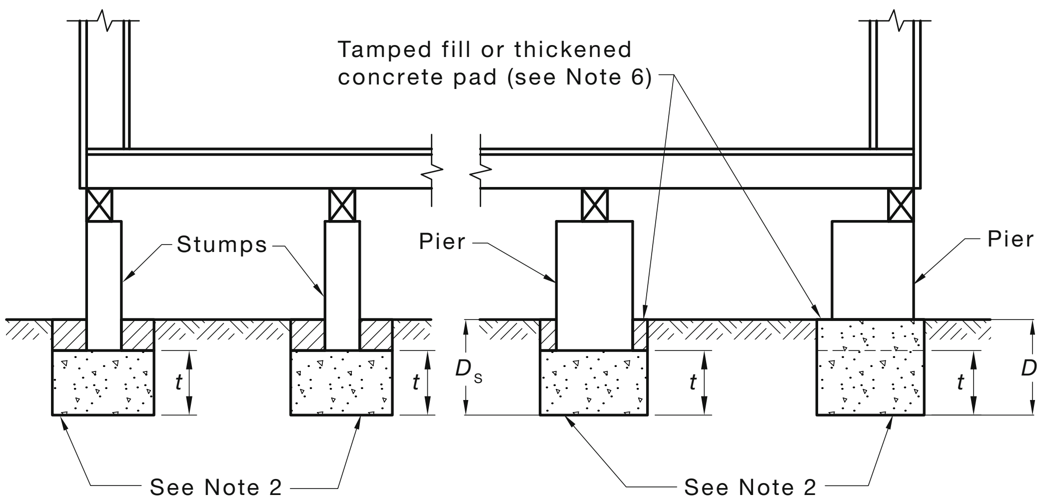
U_{ULT}, H_{ULT}= ultimate strength in uplift and horizontal load respectively

The lower ends of diagonal members shall be attached to stumps not more than 150 mm above ground level.

For horizontal bracing loads applied higher than shown in Figure E2, capacity shall be determined by engineering principles.

Stump horizontal capacity (see Table E2) is for compacted soil backfill suitable for 100 kPa bearing. For soil with less than 100 kPa lateral bearing strength, the horizontal capacity from Table E2 shall be reduced by multiplying by $\left(\frac{\text{allowable bearing pressure}}{100}\right)$.

The structural strength of the stump and connection to pad for backfilled stumps shall not be less than defined in the design Standard appropriate for the stump material.



NOTES:

1 Footing sizes that comply with AS 1684 shall be used.

2 Footing sizes for larger loads shall be selected from the following:

Effective supported area m^2	Width of square pad mm	Diameter of circular pad mm	Thickness (t) mm
10	400	500	200
20	500	600	200
40	600	750	250

The effective area supported by a pad footing is the sum of—

(a) the supported floor area;

(b) the supported roof area (if applicable); and

(c) half the supported wall area in elevation (if applicable).

3 For footings on rock, the width or diameter may be reduced to one-half the above.

4 The pad footing may be constructed in concrete except that masonry footings may be used under masonry piers.

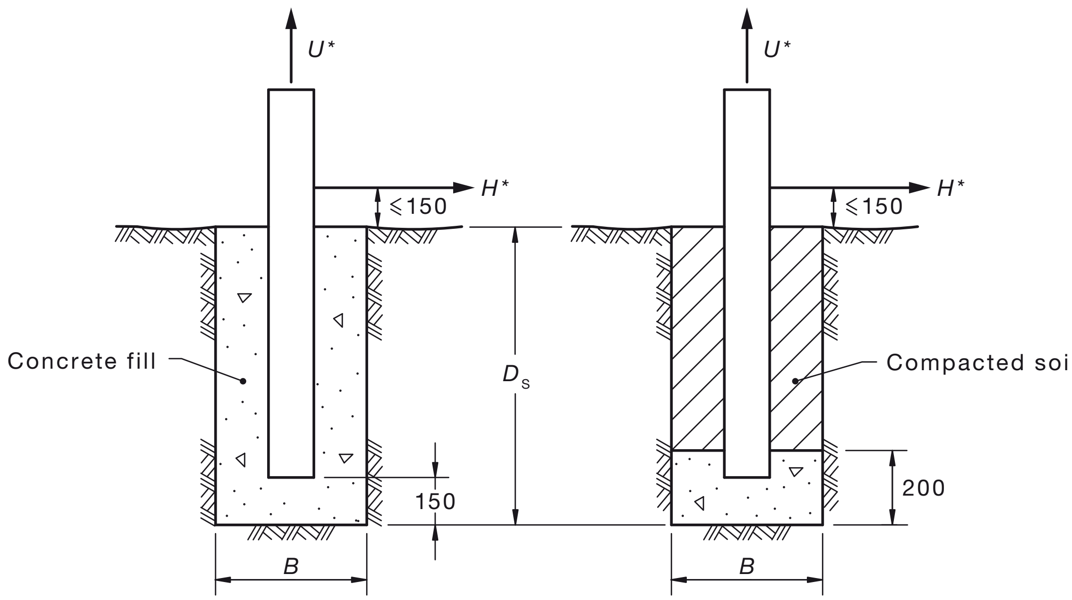
5 Pad footing sizes shall also apply to footings supporting roof or floor loads only.

6 The excavation shall be backfilled with manually rodded or tamped soil, or the footing thickness shall be increased.

7 Construction details are given in Clause 6.5.

8 The capacity of braced stumps may be detailed to resist subfloor bracing where no shear walls exist.

FIGURE E1 PAD FOOTING SYSTEM FOR CLAD FRAME, CLASS A, CLASS S, CLASS M, CLASS H1 AND CLASS H2 SITES



DIMENSIONS IN MILLIMETRES

FIGURE E2 BRACED STUMPS

TABLE E1

SOIL BACKFILL BRACED STUMPS— UPLIFT CAPACITY, kN

Stump depth (D_s) mm	Footing diameter (B)				
	250	300	350	400	450
400	0.8	1.1	1.3	1.5	1.6
600	2.0	2.4	2.7	3.1	3.6
800	3.8	4.4	5.0	5.6	6.2
1000	6.5	7.3	8.1	9.0	9.9

TABLE E2

SOIL BACKFILL BRACED STUMPS— HORIZONTAL LOAD CAPACITY, kN

Stump depth (D_s) mm	Footing diameter (B), mm			
	100	125	150	200
400	2.2	2.7	3.3	4.4
600	3.6	4.5	5.4	7.2
800	5.1	6.3	7.6	10.1
1000	6.6	8.2	9.8	13.0

NOTE: Loose sand is not suitable for soil-backfilled braced stumps; concrete backfill may be used for braced stumps.

TABLE E3

CONCRETE BACKFILL BRACED STUMPS— UPLIFT CAPACITY, kN

Stump depth (D_s) mm	Footing diameter (B), mm				
	250	300	350	400	450
400	1.0	1.2	1.5	1.8	2.2
600	2.2	2.6	3.1	3.6	4.2
800	4.1	4.7	5.4	6.2	7.1
1000	6.8	7.7	8.7	9.8	10.9

TABLE E4

CONCRETE BACKFILL BRACED STUMPS— HORIZONTAL LOAD CAPACITY, kN

Stump depth (D_s) mm	Footing diameter (B), mm				
	250	300	350	400	450
400	4.0	4.8	5.6	6.4	7.2
600	6.8	8.2	9.5	10.9	12.3
800	9.8	11.7	13.7	15.6	17.6
1000	12.8	15.3	17.9	20.4	23.0

APPENDIX F

SOIL STRUCTURE INTERACTION ANALYSIS FOR STIFFENED RAFTS

(Informative)

F1 ANALYSIS PROCEDURE

Design parameters may be determined by an analysis that allows for interaction of the structure with the foundation over a design range of soil moisture conditions. Generally, the raft should be proportioned to resist positive and negative moments of approximately the same magnitude. The recommended procedure is a computer analysis for the actual loading pattern in accordance with the Walsh and Walsh or Mitchell methods (Refs 1 and 2, Appendix I).

The analysis of non-rectangular buildings is commonly on the basis of overlapping rectangles.

The analysis and design may be based on the total slab cross-section, modified if applicable to incorporate the effective flange widths as defined in Clause 4.4(e).

For the Walsh method, the mound shape should be taken as a flat section with movement occurring over an edge distance (e), as shown in Figure F1. The shape factor for edge heave (W_f) used to define the compound parabola in edge heave is given in Figure F2.

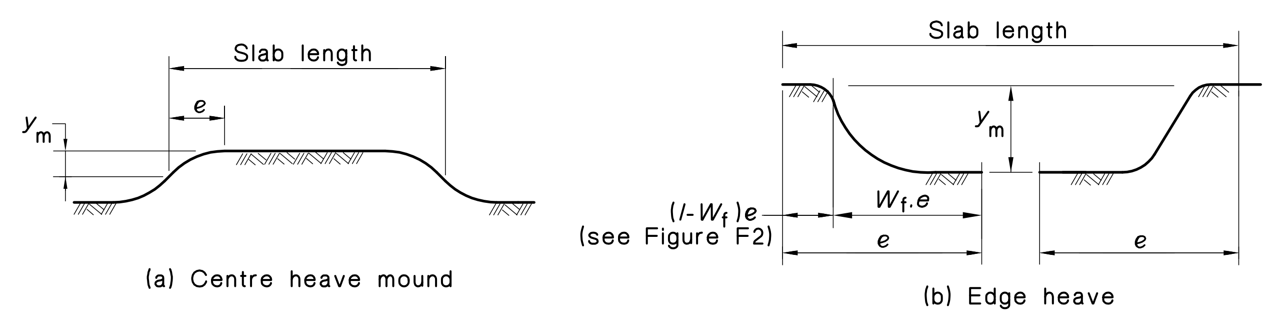


FIGURE F1 IDEALIZED MOUND SHAPES TO REPRESENT DESIGN GROUND MOVEMENT (WALSH METHOD)

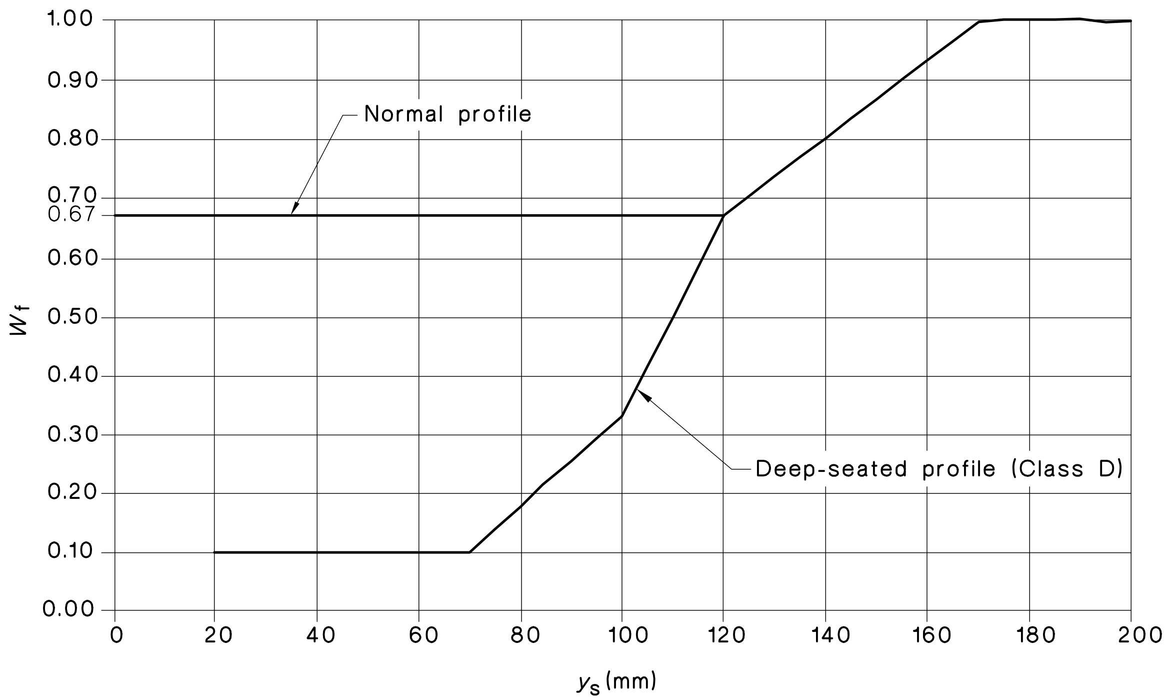


FIGURE F2 W_f FACTOR FOR WALSH MOUND SHAPE

F2 ANALYSIS PARAMETERS FOR STIFFENED RAFTS

The general procedures for the analysis of a stiffened raft incorporated in an engineering design method should take into account the following:

(a) *Differential mound movement* The design value of differential mound movement (y_m), across the foundation may be estimated taking into account the moisture conditions at the time of construction and the influence of the footing system and edge paths on the design moisture conditions. In the absence of more accurate calculations, y_m may be taken as:

	Walsh method	Mitchell method
Centre heave	$0.7y_s$	$0.7y_s$
Edge heave on initially dry site	$0.5y_s$	$0.7y_s$

On a site that is wet throughout the profile at the time of construction, a reduction of y_m for edge heave not exceeding 40% may be made.

Where the slab length is less than $2e$, the value of y_m may be reduced linearly with span/$2e$.

This movement is represented as an idealized mound, and incorporates some estimate of the edge distance (from the edge to uniform condition) as shown in Figure F1. Where the movement y_m is selected to represent an extreme moisture condition (rather than the design value described in the Standard) then the mound shape should be taken as single-sided (i.e. heave or shrinkage at one end only).

Where highly variable site conditions such as gilgais or residual soils on steeply dipping strata have been found, account should be taken of such variability in the idealization of the mound behaviour.

(b) *Edge distance* The edge distance (e), is taken as:

(i) For centre heave, in metres:

$$e = \frac{H_s}{8} + \frac{y_m}{36}\text{, where } y_m \text{ is in millimetres and } H_s \text{ is in metres} \qquad \ldots\text{F4(1)}$$

(ii) For edge heave, in metres:

$$e = 0.2\text{L} \leq 0.6 + \frac{y_m}{25}\text{, where } y_m \text{ is in millimetres} \qquad \ldots\text{F4(2)}$$

For the Mitchell method:

$$\text{Mound exponent } (m) = \frac{1.5L}{D_{cr} - D_e} \qquad \ldots\text{F4(3)}$$

where

D_{cr} = critical depth

$= \dfrac{H_s}{7} + \dfrac{y_m}{25}$, where y_m is in millimetres and H_s is in metres

D_e = depth of embedment of edge beam from the finished ground level

(c) *Mound stiffness* For beams in contact with swelling soil, the soil stiffness will range from k = 400 kPa/m to k = 1500 kPa/m. The computed forces and displacements are generally not particularly sensitive to the value of k used except for certain edge heave situations.

A soil stiffness of 100q but not less than 1000 kPa/m may be used, where q is the total building load divided by the plan area of the slab. Other values may be adopted if supported by local experience or experimental data.

For Melbourne basaltic clays, a soil stiffness of 400 kPa/m minimum or 50q may be used.

For beams in contact with shrinking or stable soil, the soil stiffness should be taken as at least 5000 kPa/m.

APPENDIX G

DEEP FOOTINGS

(Informative)

G1 DESIGN PROCEDURES FOR DEEP FOOTINGS

Deep footings may be driven or jacked timber, concrete or steel piles, excavated or bored piers and steel screw piles. The design of deep footings should be by engineering principles using this Appendix and AS 2159 when appropriate. The structural strength of deep footings should be designed using AS 1720 series, AS 4100 or AS 3600 as appropriate.

The recommendations given in Paragraphs G2 to G8 do not include the possible effect of trees.

G2 LOADS

The loads from the residential building to be supported by deep footings should be determined using engineering principles and the dead, live, earthquake and wind loads defined in the AS/NZS 1170 series.

The permanent and imposed actions in Clause 1.4.2 of this Standard should not be used in the design of deep footings.

G3 GEOTECHNICAL SITE INVESTIGATIONS

A geotechnical site investigation for the design of deep footings should be taken to a depth not less than 1.5 m beyond the founding depth of the footings, and not less than 1.5 times H_s for the site.

The geotechnical strength of the foundation should be determined by appropriate field and laboratory testing of the ground at depths relevant to the design. The information required from the site investigation, as defined in AS 2159, should also apply to this Standard for deep footings.

G4 DRIVEN PILE FOOTINGS

G4.1 General

Driven piles to this Standard generally support imposed live and dead loads that are of a similar magnitude to loads caused by shrinking or swelling of the soil foundations.

Driven piles generally used in residential construction have a mass of less than the pile driving hammer.

The criteria given in Paragraphs G4.2 and G4.3 give consideration to the conditions normal in residential scale construction.

G4.2 Design actions

The design action on the piles should include imposed loads from the residential structure plus actions from swelling or shrinking foundations.

For natural foundations, the uplift action due to soil swelling on a driven pile may be assumed to act on the pile for a depth from 0.25 H_s to 1.0 H_s. A load factor of 1.5 should be used to calculate the uplift design action on the pile.

For a shrinking natural foundation no action effect needs to be considered.

G4.3 Design geotechnical strength of driven piles

Driven piles in natural ground should satisfy the following requirements:

(a) For unspliced piles driven by drop hammer, the design geotechnical strength may be calculated as follows:

$$R_{ug} = (R_{ug})_2 - (R_{ug})_1 \qquad \text{. . . G4.3(1)}$$

where

R_{ug} = ultimate geotechnical strength of the pile, in kilonewtons

$(R_{ug})_1$= ultimate geotechnical strength of the pile, in kilonewtons determined at a depth equal to $0.75H_s$

$(R_{ug})_2$= ultimate geotechnical strength of the pile, in kilonewtons determined at the final depth of installation of the pile

$$(R_{ug})_1 \text{ and } (R_{ug})_2 = 0.4\frac{W_h \times h_h}{S} \qquad \text{. . . G4.3(2)}$$

where

W_h = hammer weight, for driven piles, in kilonewtons

h_h = drop height of the hammer, for driven piles, in metres

S = pile set, average for five blows, in metres

The design geotechnical strength of the pile should be taken as—

$\varphi_g R_{ug}$

where

R_{ug} is from Equation G4.3(1)

and

φ_g = 0.45 for compressive load and 0.35 for tension load

The design geotechnical strength of the pile should be equal to or greater than the design action effect (E_d) due to all imposed loads.

$$E_d \leq \varphi_g R_{ug} \qquad \text{. . . G4.3(3)}$$

For deep filled sites pile design should use AS 2159.

(b) For piles driven by methods other than drop hammer or where the pile hammer weight is less than the pile weight, the design strength of the pile should be determined by engineering principles using AS 2159.

(c) The structural strength of piles should be determined using engineering principles in accordance with AS 2159 and AS 1720 series, AS 4100 or AS 3600 as appropriate.

G5 DESIGN OF BORED AND EXCAVATED PIERS

G5.1 Pier system

Bored and excavated piers should be designed in accordance with this Appendix and the appropriate section of AS 2159. Bored and excavated piers may be used in piled footing systems to support all types of residential construction.

G5.2 Design actions

The design of bored and excavated piers should consider all imposed loads from the residential construction plus loads caused by swelling or shrinking foundations determined by engineering principles.

G5.3 Design geotechnical strength of bored and excavated piles

For piers, the geotechnical design strength should be based on base resistance plus side friction or adhesion where effective. No side adhesion or friction should be assumed to exist to a depth of $0.75H_s$ for down loads. For uplift load due to soil swelling, side friction or adhesion should be assumed to be effective.

G6 DESIGN OF STEEL SCREW PILES

G6.1 Pile system

Screw piles should be designed in accordance with this Appendix and the appropriate Section of AS 2159. Screw piles may be used in piled footing systems to support all types of residential construction.

G6.2 Design actions

Screw piles should be designed for all imposed vertical and lateral loads from the supported residential construction plus load due to swelling or shrinking of the foundation, and including loads imposed during installation. For sites with deep fill, the effect of negative skin friction should be considered.

G6.3 Minimum depth

The installed depth of screw piles in reactive foundations should not be less than $1.25H_s$, where H_s is given in Table 2.4. Where screw piles are used to support footing systems adjacent to deep service trenches, the depth of pile should be not less than the depth of the trench.

G6.4 Design strength of screw piles

Screw piles should satisfy the following requirements:

(a) The design geotechnical strength of a screw pile should be in accordance with AS 2159.

(b) The design structural and geotechnical strength of the pile in compression and bending should consider the effective supported length of the shaft. The freestanding portion of the pile above ground is unsupported. The pile may effectively be unsupported in a soft or loose soil layer, or dry clay soil with shrinkage cracks.

(c) Where vertical screw piles are used to resist horizontal actions, the piles should have adequate strength and stiffness. When determining the structural and geotechnical strength, a loss of ground support due to soil shrinkage in reactive soils should be considered. Loss of ground support over a depth of 0.3, 0.4, 0.5 and 0.75 times H_s for Class S, Class M, Class H1, Class H2 and Class E, respectively, may be used.

(d) If a screw pile is to resist bending actions, the shaft should be embedded into the pile cap or footing sufficient to generate the required resistance.

G7 PRE-BORING FOR PILES IN REACTIVE SITES

The use of pre-boring to allow pile installation in hard or dense ground conditions may be used. It is critical for future performance of the piles that the pre-boring does not create an oversized hole that allows surface water ingress into the foundation. The maximum pre-bore diameter that should be used is 90% of the minimum pile diameter.

The installation of piles should not create voids or permeable paths that could allow water ingress to reactive clay foundations.

G8 SWELL PRESSURE ON PILE-SUPPORTED FOOTING OR BEAM

The swell pressure that may be generated against pile-supported beams or slabs may be high and difficult to resist in residential scale construction. Detailing to avoid uplift on pile-supported structures is recommended.

Where the foundation may swell against piled beams the pressure may be estimated using the following equation:

$$p_s = \frac{y_s k}{1000} \qquad \text{. . . G8(1)}$$

where

p_s = swell pressure under footing, in kilopascals, unfactored

y_s = characteristic surface movement, in millimetres

k = swelling soil stiffness, in kilopascals per metre

For strength design, use a load factor of at least 1.5 on swell forces.

For beams or strip footings, the swell pressure should be assumed to act over the footing or beam width plus 300 mm. The swell stiffness (k) should be assumed to be 1500 kPa/m as a minimum value.

APPENDIX H

GUIDE TO DESIGN OF FOOTINGS FOR TREES

(Informative)

H1 LIMITATIONS

The method given in this Appendix is essentially the one that has been used with apparent success since July 1990 in South Australia. This method does not separately assess all characteristics that affect a tree's ability to draw moisture. However, when combined with engineering judgement, this method has been found to be sufficient to encompass the tree impact on foundation performance in the South Australian context.

This approach to the design of footing systems in the presence of tree effects will not necessarily result in a footing system that achieves the performance requirements of this Standard. The risks of underperformance arise from factors that include the inherent variability and unpredictability of living, growing trees and their interaction with the environment, as well as imperfections in the method of modelling the effect of trees. A reason for the success of this approach to design for trees in South Australia is that the increased risk of underperformance is understood by designers and the existence of the increased risk and the potential effects of the underperformance are effectively communicated to owners.

It is recognized that in different climate zones the recommended depth of tree drying effect and the design suction changes attributed to trees may vary from that which has been adopted in South Australia. The modified parameters for more temperate climates are given in Table H1. The modified values have been based on consideration of limited data from South-east Queensland juxtaposed with the South Australian experience.

H2 DEFINITIONS

H2.1 Design height of single tree (HT)

The overall height of tree from ground level to the top of the crown. Depending on circumstances, the height of tree to be taken may be the existing height, an estimate of the mature height of the tree or an estimate of the height that it will attain within the design life of the building.

H2.2 Design height of a group of trees (HT_g)

Where the heights within a group of trees are variable, HT_g is taken to be 0.9 times the design height of the tallest tree in the group.

H2.3 Distance of tree to the building (D_t)

The shortest horizontal distance between a tree trunk and the nearest edge of the proposed footing.

H2.4 Group of trees

Either a group or row of trees in which three or more adjacent trees are spaced on a centre to centre distance (s), such that s is less than 1.0 times the average height of the trees under consideration and the minimum distance from the building to any tree under consideration, (D_t), is respectively less than $1.5HT$, or $2.0HT$ for a row of 4 or more trees.

H2.5 Maximum design drying depth (H_t)

The design depth below ground level of soil drying attributable to the effect of a single tree or group of trees.

H2.6 Influence distance (D_i)

The maximum lateral reach of the drying influence of the tree under consideration. For a single tree D_i should be taken as 1.0 times HT and for a group of trees, D_i should be 1.5 times HT_g. For a group of 4 or more trees in a row, D_i to be 2.0 times HT_g.

H3 MAXIMUM DESIGN DRYING DEPTH (H_t)

For the greater South Australian climate, the design depth of tree drying for a single tree or tree group may be taken as 4 m and 4.5 m, respectively. For other climate zones and associated design depths of suction change (H_s), recommendations on depth of drying by trees are provided in Table H1.

H4 DESIGN PROCEDURE

The design procedure should include the following steps:

(a) Determine characteristic surface movement (y_s), in accordance with this Standard (neglecting tree influence).

(b) In the absence of advice on mature tree heights, the ratio D_t/HT may be taken to be 0.5. In the absence of advice on mature heights, single trees with D_t greater than 25 m and groups of trees with D_t greater than 50 m may be ignored.

(c) Find the maximum extra suction change caused by the vegetation (Δu_{base}) at the maximum design drying depth (H_t), (see Figure H1 and Table H1).

(d) Determine the maximum potential surface movement due to the tree-induced suction change that is in addition to the normal design suction profile (y_{tmax}). The ground movement caused by the added suction change may be calculated in accordance with the principles outlined in estimating y_s as given in Clause 2.3.1. In the calculation of y_{tmax}, the depth of soil cracking should be taken to be equal to H_t. The depth of soil cracking adopted in the calculation of y_s should be as would be adopted in the absence of the tree or trees.

(e) Calculate the design tree effect as a surface movement y_t as follows:

  (i) For single trees, or groups of trees, with D_t/HT less than $0.5 y_t$ should be taken as y_{tmax}. For D_t/HT greater than 0.5, y_t should be determined from Equation H4 and should not be less than zero.

$$y_t = \left\{ 1 - \left[\frac{\dfrac{D_t}{HT} - 0.5}{\dfrac{D_i}{HT} - 0.5} \right] \right\} y_{t\max} \qquad \text{... H4}$$

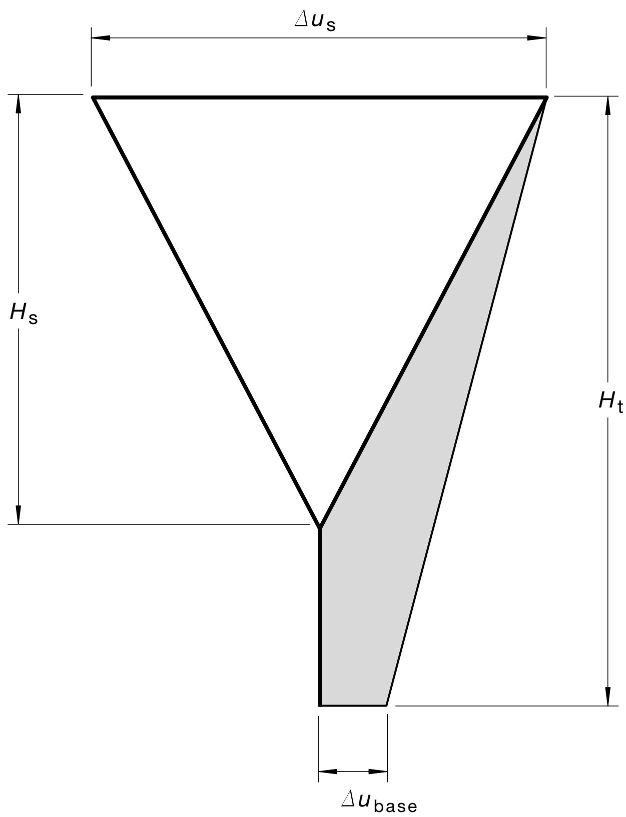
  (ii) For design of the footing system, adopt a double-sided mound design together with the same mound shape parameters as used in design without a tree effect.

  (iii) Design the footing system for a tree-induced differential, centre heave, mound height (y_m) trees, equal to $(0.7y_s + y_t)$.

  (iv) Where footings are designed for tree drying effects, M_u for the centre heave case should be not less than $1.5M_{cr}$, as calculated for centre heave bending, and M_u, for edge heave should be not less than $1.5M_{cr}$ as calculated for edge heave bending.

  (v) Where footings are designed for the effects of tree removal, or for anticipated tree removal or death, M_u for centre heave should be not less than $1.5M_{cr}$ as calculated for centre heave bending, and M_u for edge heave bending should be not less than the moment resistance M_u for centre heaves.

(vi) The height of tree and the number and location of trees assumed in the design should be stated and communicated to the owner as important parameters and limitations of the design.



Depth of design suction change, (H_s)	Single tree		Tree group	
	Maximum extra suction change (Δu_{base})	Maximum design drying depth (H_t)	Maximum extra suction change (Δu_{base})	Maximum design drying depth (H_t)
m	pF	m	pF	m
1.5	0.30	2.5	0.38	3.0
1.8	0.33	2.7	0.40	3.3
2.3	0.35	3.0	0.43	3.6
3	0.38	3.4	0.46	4.1
4	0.43	4.0	0.55	4.5

NOTE: Further information on the drying effects of trees can be found in Cameron, D.A. (2001), *The extent of soil desiccation near trees in a semi-arid environment* (Int. J. Geotechnical and Geological Engineering, Kluwer Academic Publishers, v19, no. 3 and 4, pp 357-370) and Cameron, D.A and Beal, N (2007), *A method for evaluating the influence of trees on expansive soil movement in light of case studies from SE Queensland* (Proc. 10th ANZ Conference on Geomechanics, Common Ground, Brisbane, Oct 2007, V2, pp 200-205).

FIGURE H1 DESIGN SUCTION CHANGE DISTRIBUTION WITH DEPTH FOR TREE DRYING EFFECTS FOR DIFFERENT CLIMATE ZONES

H5 ALTERNATIVE DESIGN METHODS

Alternative design methods for the impact of trees on foundations are available.

APPENDIX I
BIBLIOGRAPHY
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1684 Residential timber-framed construction series

1720 Timber structures (series)

2159 Piling—Design and installation

4100 Steel structures

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3660.1 Part 1: New building work

4055 Wind loads for housing

AS/NZS
1170.2 Part 2: Wind actions

3500 Plumbing and drainage (series)

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NOTES

COMMENTARY TO AS 2870—2011

Residential slabs and footings
Informative

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INTRODUCTION

PURPOSE

This Standard provides for simple standard methods for the design of residential footings based on current structural and geotechnical principles. It applies to a variety of footing systems for most foundation conditions, including reactive soils. Reactive soils are common in many parts of Australia and the Standard is strongly focussed on providing appropriate design solutions for footings and slabs on such soils. The Standard is in mandatory form for use in building control.

DESIGN REQUIREMENTS

In order to provide more background to footing design, a brief discussion follows on the aspects that are taken into account in the Standard:

- *Design for swelling and shrinkage movements* The primary cause of footing failure of domestic structures is associated with the movement of reactive clay soils. A soil is said to be reactive or expansive when it undergoes appreciable volume change with changes in moisture content. The reactivity of a soil depends on the size of clay particles, their mineral composition and the proportion of clay in the soil. Laboratory tests of soil reactivity on a single or a small number of samples may not accurately characterize the overall reactivity of a clay profile. In particular, the usual engineering index properties (e.g. liquid and plastic limits and linear shrinkage), when assessed on their own may not be reliable.

  Soil movement that might occur on a site depends not only on the reactivity of the clay but also on the depth and distribution of the clay in the soil profile and on changes in moisture content. Moisture changes usually occur slowly in clays and produce swelling upon wetting and shrinkage upon drying. These moisture changes often result from a combination of causes, which include the following:

  - Seasonal and long-term climate changes, including dry summers, floods and droughts.
  - Influence of the building, covering the garden and drainage; particularly trees, which may cause severe drying.
  - Long-term effects of the whole urban infrastructure, including paving and drainage.
  - Initial moisture conditions at the site relative to the long-term design conditions, including special conditions such as demolition of an existing house, removal of large trees and similar.

  The actual pattern of the movement of a reactive clay foundation depends on the moisture and clay variation and may be quite complex. Building distortions may often include asymmetric and warping components. Nonetheless, for the purpose of design, the pattern of differential movement can generally be represented by one of the forms given in Figure C1.

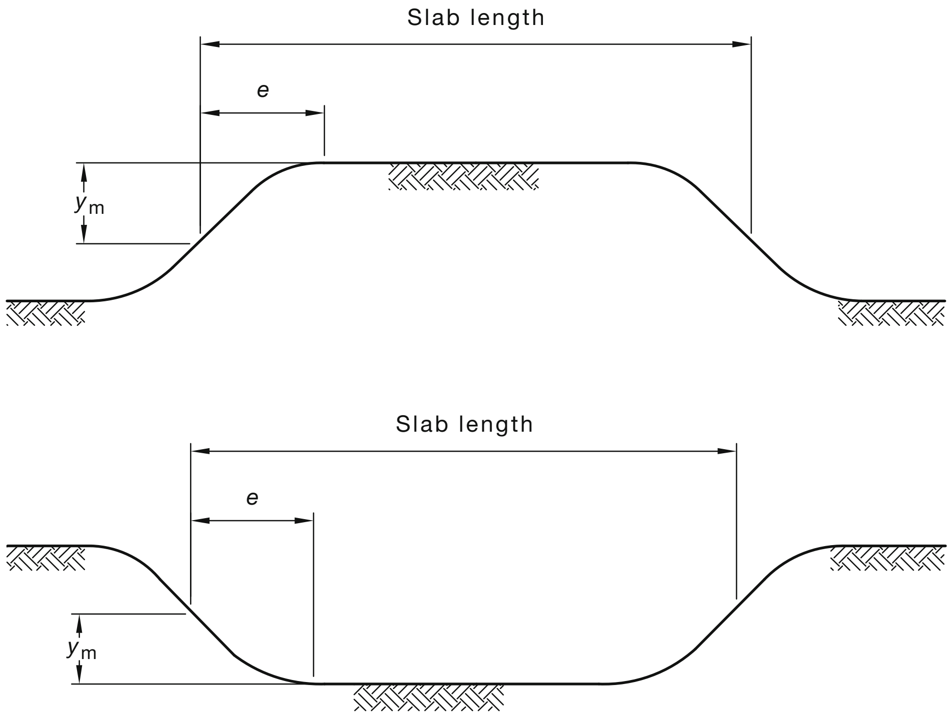

FIGURE C1 IDEALIZED GROUND MOVEMENT PATTERNS FOR FOOTINGS AND SLABS ON CLAYS (Walsh shapes)

The design of a slab to accommodate ground movements requires the provision of sufficient overall strength and stiffness. Whereas a very flexible slab could deform in the same way as the foundation, the stiffness of a properly designed slab limits the differential movement as a result of interaction of the foundation and structure. This interaction utilizes the mass of the slab and structure and its flexural stiffness and strength. Some contribution may be made by tensile membrane action of the slab. The stiffness of the slab not only reduces the deformations, but also transfers load to the relatively high areas of the foundation, and thereby tends to suppress heave at those locations.

Protection of the clay from extreme moisture changes is also important. Although some measures such as perimeter paths can be incorporated in the design, generally the owner has the immediate responsibility for protection of the foundation from severe moisture changes after completion of a building.

Strip footings undergo similar ground movement patterns and are designed on the same general basis as raft slabs, that is, strength and stiffness. However, although strip footings may be founded at depths where moisture changes should be less, in some cases (particularly where failures can occur by soil swelling) deep strip footings may trap moisture, increasing the soil swell. Generally, strip footings are more vulnerable to sideways and twisting movements and such movements can cause damage. Therefore, for highly reactive sites, the alternative of an integral stiffened raft footing system is preferred.

An alternative design philosophy requires the removal and replacement of reactive clay or the covering of it with a suitable non-reactive material. In such a case, consideration should be given to the effects that such replacement will have on the soil moisture regime particularly when, as is likely, the replacement soil is more porous and permeable than the natural material. The resulting infiltration and impoundment of water in the reservoir formed by the excavation in the natural clay may lead to deep and severe moisture changes in the underlying natural clay.

- *Design for settlement of compressible soils or fill* Uneven settlement may occur on filled or soft alluvial sites. A solution for filled or soft sites could involve compaction of the soft or loose soil and fill, stiffening of the footing or slab to resist the differential movements or the provision of piers or deep beams taken down to found on firmer strata in some circumstances. A stable foundation may be provided by properly compacted fill material not containing deleterious material.

  For slabs on non-reactive soils, distribution of imposed loads to the foundation is generally not a significant problem. Around the edge of the slab, either a thickened beam or a separate strip footing may be used to support the usually more heavily loaded external walls. Thickened beams and external footings distribute the load along the beam as well as laterally to reduce foundation pressures. Under internal walls, in most cases the slab panel itself is sufficient to support the load from the wall and roof. Nonetheless, to allow for some unevenness in the loading and the foundation, additional support is appropriate for some of the loads that occur in two-storey construction.

  For strip footings, distribution of imposed loads to the foundation requires that the footing possesses adequate strength to transfer the load laterally and longitudinally. The required flexural strength is usually moderate and may be determined by the assumption of uniform support. Alternatively, a more refined analysis may be carried out using an elastic representation of the foundation.

  For reactive soils, the distribution of loads to the foundation requires consideration of soil behaviour and soil-structure interaction as discussed previously.

- *Design for sensitivity of superstructure* Whether a building can tolerate movement without damage depends on the type of construction and the various design details such as whether or not the walling is articulated.

RESPONSIBILITIES

Footing design and construction involves a number of steps: site classification, selection of the footing system, structural design, construction in accordance with the required design details and construction methods, and proper maintenance. In addition to the builder, this process may involve an engineer, building certifier, the owner, and all parties who share responsibility for any failure. In particular, the owner has a responsibility to ensure the site is properly maintained and the Standard attempts to guide owners in this area.

The functions of the various parties likely to be involved in the overall building process are set out in Appendix A of the Standard.

SECTION C1 SCOPE AND GENERAL

C1.1 SCOPE

The Standard applies to site classification and footing system design for houses and similar structures, extensions and outbuildings.

The recommendations in the Standard were developed from research and experience in the design and performance of house footings and slabs, but there is no reason why they cannot be applied to other similar structures. The similarities should be in the size, the loading and the type of construction.

Different building practices, such as the use of control joints in concrete slabs, are used in large non-residential structures but the Standard makes no design provision for these. As well, it is unlikely the Standard will be appropriate for industrial floors, except for the lightest applications.

The Standard has been based on methods of construction that are generally well accepted throughout Australia. Nonetheless, footing design is a developing field and it is possible that new or locally effective footing systems may not have been included. The Standard should not be used to inhibit the development of such systems provided they comply with the design and performance considerations set out in Clauses 1.3 and 1.4.

C1.2 APPLICATION

The application Clause outlines the procedures to be followed in using the Standard. Where conflict with AS 3600 arises with regard to footing system design and construction, the provisions of AS 2870 prevail.

C1.3 PERFORMANCE OF FOOTING SYSTEMS

The current costs of building failure are modest compared with the costs of overly conservative design. Moreover, if the designs in the Standard are followed, failures will be very rare. The performance of footing systems on reactive sites depends in part on the adopted routine of post-construction maintenance. If the homeowner’s maintenance role is to be diminished, or higher expectations of performance are demanded, then the footing system should be designed according to engineering principles. Furthermore, the performance criteria adopted in the Standard may not be adequate for party walls, which have special architectural performance requirements.

Performance is based largely on the size and frequency of cracking in walls and concrete floors. All building materials move (e.g. clay bricks expand, timber and plasterboard shrink). Consequently, some cracking in buildings is inevitable and is independent of foundation movement. On reactive and soft or non-uniform soils, foundation movement adds to this tendency to crack. A large number of buildings in Australia are constructed on clays that move with changes of soil moisture conditions that arise in part from effects of covering the ground with the building. Generally, the movements will be moderate and the prescribed designs in the Standard will cope with the movement. If extreme moisture conditions occur (which may have been avoided had a reasonable level of site maintenance been achieved), then significant damage will be more likely and probably more severe. To attempt to design for such conditions on every clay site would add significantly to the cost of housing throughout Australia.

To avoid extreme moisture conditions, it is essential that owners become aware of their responsibility to care for and adequately maintain a reactive clay site. Guidance to the owner is given in Clauses 1.3.2 and 1.3.3 and a CSIRO Building Technology File 18 (formerly known as Information Sheet 10-91), entitled: *Foundation maintenance and footing performance*: *A Homeowner's Guide*, is available for distribution to homeowners. This pamphlet may be obtained from CSIRO Publishing. It is suggested that a copy be given to the new homeowner by the builder. The problem of subsequent owners is not simple and it is suggested that the owner should pass on the information sheet. In reactive clay areas, it is expected that the building certifier will be interested in ensuring that the Information Sheet is disseminated. Site maintenance after occupation becomes part of an owner's accepted responsibilities.

C1.4 DESIGN CONDITIONS

C1.4.1 General

Design of footings in accordance with this Standard (AS 2870) takes into account those environmental conditions arising from a normal site, maintained in accordance with Appendix B and CSIRO Building Technology File 18. These conditions are expected to cover most situations encountered in a normally maintained building site and the designs do include some provision for conditions slightly divergent from ideal; however, abnormal sites may arise as a consequence of previous land use, inadequate site maintenance or the presence, growth or removal of trees. Special engineering consideration is needed for such sites, and these sites are usually classified as Class P.

The effect of trees on a reactive clay site will depend on matters such as climate, tree species, tree size, soil type and profile, watering and the interaction between the tree and the site development. These matters are not fully understood.

As a first step, guidance is provided for the design for tree effects in Appendix H of the Standard. Further discussion is provided in Ref 2.

Zero lot line developments make observance of some of the provisions of the maintenance more problematic but the technical requirements still apply.

C1.4.2 Design action effects

The design (factored) action for both strength (safety against yield) and serviceability (deflection and crack control) is the same. Moreover, the strength design action is significantly less than the value given in AS 3600. Mostly, the low value arises from the relatively low cost of failure as explained in Walsh (Ref. 1). These design actions are also consistent with the performance requirements given in Clause 1.3.1.

C1.4.3 Other design considerations

(No Commentary)

C1.5 DEEMED-TO-COMPLY STANDARD DESIGNS

(No commentary)

C1.6 ARTICULATION REQUIREMENTS

(No commentary)

C1.7 NORMATIVE REFERENCES

(No commentary)

C1.8 DEFINITIONS

There are no universally accepted sets of definitions for footing and building types so some definitions may differ from local custom. In the interests of a national Standard, certain terms have been chosen and defined for use in this Standard. Where possible, the definitions are consistent with building regulations and other Standards. Attention is drawn to the definitions of silt, sand and loadbearing walls, which are different from the usual engineering definitions. The definition of reinforced single-leaf masonry is different to the definition used in AS 3700.

A distinction has been made between the various forms of slabs, for example a slab on ground, a stiffened raft or a footing slab. In addition, various specific terms for masonry construction have been defined. Clad frame construction is defined, the definition being illustrated in Figure C1.1 of this Commentary.

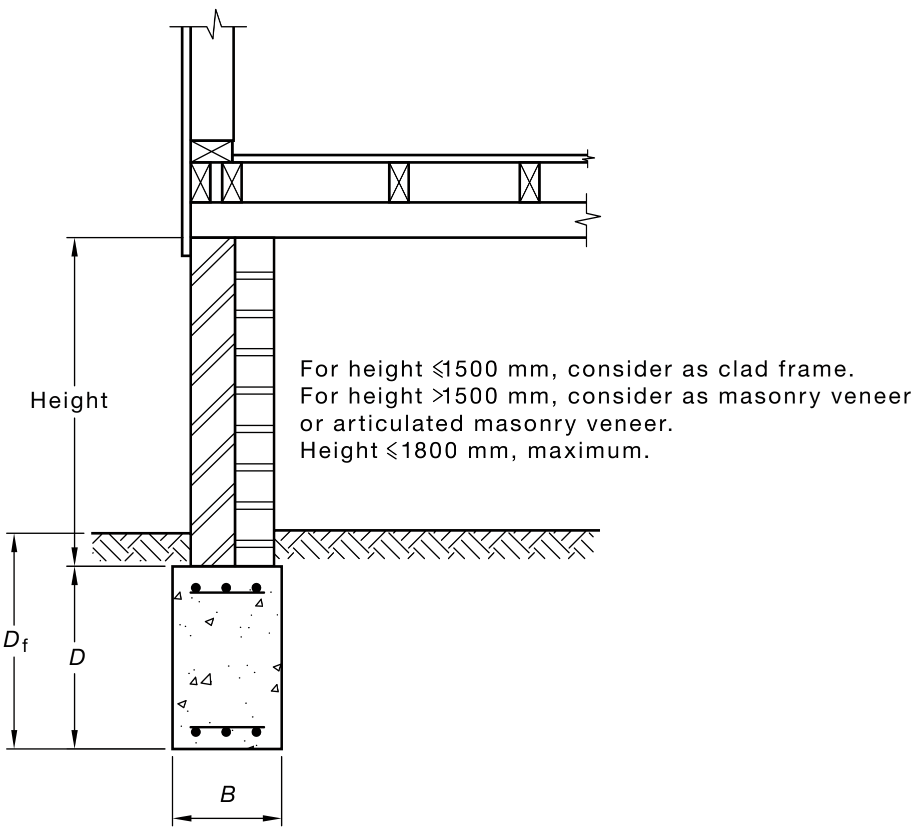

FIGURE C1.1 STRIP FOOTING SYSTEMS—CLAD FRAME

C1.9 NOTATION

(No commentary)

C1.10 REINFORCEMENT DESIGNATION

There has been some debate about ductility levels for reinforced concrete building components as a consequence of the availability of reinforcing steel products with higher characteristic yield strengths.

AS/NZS 4671, *Steel reinforcing materials*, introduced various ductility grades and measures for reinforcing steel. For instance, these distinctions have particular relevance in the design of suspended concrete beams and slabs and lesser relevance for footing systems.

Nonetheless, appropriate ductility (the ability of a structure to undergo large deformations without rupture) is a common and desirable feature in structural systems and has been taken into account in the process of selection of the Standard Designs.

In general, both deformed bars (D500N) and round and deformed mesh (D500L) are now distributed nationally and form the basis of design for the Standard.

C1.11 INFORMATION IN DOCUMENTS

(No Commentary)

REFERENCES

1 WALSH (1985) *Load Factors and Design Criteria for Stiffened Rafts on Expansive Clays*. Civ. Eng. Trans. Vol. CE 27 No. 1 February, Inst of Eng. Australia.

2 WALSH (1995) *Buildings Foundations and Movements with Particular Reference to the Effect of Trees.* ACSE Seminar—Building movements, Sydney, August.

SECTION C2 SITE CLASSIFICATION

C2.1 GENERAL

C2.1.1 Classification

(No commentary)

C2.1.2 Site classification based on soil reactivity

All sites are required to be classified. The footing system has to be suitable for the site and the only method of achieving this is to assess the site and to classify it.

The main soil types are sand and clay, with silt as an intermediate type.

The various types of soil are distinguished in an engineering assessment by the size of the particles that constitute the soil such as—

- *sand*—which comprises material down to 0.075 mm;
- *silt*—which includes the range 0.075 mm to 0.002 mm; and
- *clay*—which consists of very fine particles smaller than 0.002 mm.

For the purposes of this Standard, the terms 'sand', 'silt' and 'clay' have been broadened. When soils contain mixed types, the finer particles usually control behaviour. For example, clayey sand behaves more like a clay than a sand. For the purposes of the Standard, sand is defined as soil with less than 15% clay and silt fines, and silt is redefined as a fine-grained but non-plastic and non-cohesive soil. It is important to realize these simplified classifications are different from conventional geotechnical engineering classifications.

A general summary of the properties of these common soil types is given in Table C2.1 below. Detailed methods of site classification are given in the Standard. A guide to site classification based on site reactivity is given in Table 2.1 of the Standard. Class P sites have been excluded from this Table. Abnormal site environment factors lead to a classification of Class P. Class P also includes sites subject to landslip and mine subsidence. Common types of sites that are deemed to be Class P are described in Clause 2.1.3 of the Standard.

The Standard does not provide specific designs for Class P sites. A classification of P, by itself, will not usually provide sufficient information to enable an appropriate footing system design to be prepared. Additional information will usually be required, according to the nature of the factors leading to the P classification.

Allowable bearing capacity or soil strength and stiffness may affect the classification of soft clay or silt, or loose sand. In most cases, the strength of a soil may be estimated from penetrometer tests or from the simple field rules given in Table C2.1. Foundation strength is rarely a cause of failure and simple rules or past experience provide adequate guidance. For these reasons, engineering tests to assess allowable bearing capacity should not be required. On natural sites, the reactivity of a site is usually the most important aspect of the classification and is discussed below. Specific discussion for each State is to be found in the subsequent sections of the Commentary.

TABLE C2.1

SIMPLE FIELD RULES FOR IDENTIFICATION OF MATERIALS

Foundation soil type	Physical characteristics
Rock	Rock is a strong material and includes shaley material and strongly cemented sand or gravel that does not soften in water. Material that cannot be excavated by a backhoe may be taken to be rock (see Note).
Sand and gravel	Medium dense sand or gravel is granular material into which a 50 mm survey peg can be driven with difficulty. Loose sand (including silty sand) should be checked to determine if the soil is subject to collapse. Collapsing soils experience a sudden settlement or show a decrease in volume on watering and loading, or excavation and backfilling. Collapsing soil is Class P.
Silts and clays	Very soft clay or silt soil can be penetrated by the fist and is unsuitable as a foundation. Soft clay or silt is stronger than 'very soft' but not as strong as the firm material described below. The classification can be based on local experience or on an engineering assessment. Firm clay can only be mounded in its natural moist state by strong pressure in the fingers and can be penetrated 50 mm by the thumb with moderate effort.

NOTE: Foundation partly on rock and partly on soil should receive special attention.

Generally, the first source of information about site conditions should be the building certifier. In some areas, where there has been no history of trouble with reactive clay, advice might be given that the area is not reactive and a special investigation is not needed. The selection of sand or clay classification should be fairly obvious from local knowledge or from a simple site investigation. On the other hand, the building certifier may suggest that reactive clays could be expected and care will be needed with the classification. Unless local knowledge is available, a qualified engineer or engineering geologist should be consulted. In areas subjected to deep climate-induced moisture changes, classification by a qualified engineer or engineering geologist is recommended.

Class A sites include sands and rock for which moisture-induced movement is not expected. Class S sites include silts and some clays for which only slight movements are expected. For a reactive clay site, the classification is M, H1, H2 or E. Although numerical measures for surface movement are attached to these classes in Clause 2.2, the significance of these values should not be over-emphasized. Of equal importance, although less definite, is classification by existing building performance or by soil profile identification.

The site classification process requires a secondary classification based on the regional climate and, accordingly, the expected depth of soil moisture change or depth of movement, (H_s). Experience has shown that slightly stiffer footing systems are required in semi-arid areas than in more temperate regions for sites of the same level of classification. This experience suggests that it is not only the magnitude of the movement that dictates the design of the footing; the shape of the distorted ground, as represented by the design parameters of edge distance or mound exponent, also plays an important part in the design. It is proposed that the shape is dependent on the depth of movement, with the most severe distortions occurring in semi-arid areas. This dependency has been expounded in Appendix F of the Standard. Figure C2.1 illustrates the effect of depth of movement on mound shape.

Secondary classification requires a '-D' to be attached to the primary classification to indicate that H_s is greater than 3 m. The absence of '-D' would indicate that movements are relatively shallow. In Melbourne or Sydney, a site having a y_s of 35 mm would be classified as M, but a site with the same movement in either Adelaide or Mildura would be classified as Class M-D.

The local presence of shallow bedrock does not alter H_s; however, a proven local permanent water table level may change the secondary classification, since H_s is reduced.

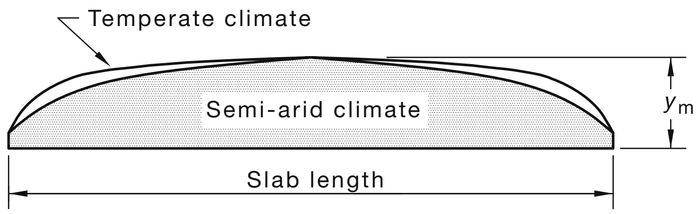

FIGURE C2.1 THE EFFECT OF CLIMATE ON MOUND SHAPE

The classification of a site on which controlled fill has been placed depends on—

- the nature of the fill (e.g. clay or sand);
- the depth of fill; and
- the nature of the underlying natural ground.

Controlled fill sites can be of any classification ranging from Class A for a sand-fill on a sand site to Class P for fill over very soft compressible clay. Clay fill on clay site is usually reactive and may be Class S, Class M, Class H1, Class H2 or Class E.

Where slab on ground construction incorporates underslab termiticide irrigation systems, the potential for these systems to cause extra foundation movement may need to be considered.

It is desirable that building owners and operators of the systems be aware of the potential for such systems to cause foundation movement on reactive clay sites. Improper installation, operation or damage to the irrigation system may increase the potential for it to cause differential movement.

The Committee consider that the data and experience with the systems are not sufficient to allow specific guidance to be incorporated in the Standard.

Physical termite barrier systems have now, in large part, supplanted chemical termite barriers throughout mainland Australia. Consequently, the likelihood of damage previously associated with the impact of water-based termiticides on reactive clay soils has been reduced (see AS 3660.1, *Termite management*, Part 1: *New building work*, AS 3660.2, *Termite management*, Part 2: *In and around existing building structures*, and AS 3660.3, *Termite management*, Part 3: *Assessment criteria for termite management systems*.

C2.1.3 Classification of other sites

Sites with unusual foundation problems such as mine subsidence, uncontrolled fill, landslip conditions or soft soil are classified as problem sites and will require a footing design by a qualified engineer. It is important for the problem sites to be correctly identified as in some cases they can appear to be similar to stable sites. For example, collapsing soils have a high bearing capacity when dry, but a much lower bearing pressure when wet, and hence need to be classified as a soft foundation.

Uncontrolled fill is a common site problem. Where the building site is an infill site in an older area, uncontrolled fill should, in particular, be considered more likely than normal. Fill is often difficult and sometimes impossible to recognize. Often the layout of the subdivision will indicate areas likely to have been filled, such as previous gullies and similar. Rubbish buried in the soil profile is a clear indication of fill. Another indicator is the appearance of a top soil layer or a normal soil profile typical of the immediate vicinity, in the area under the fill. A useful method is to test the soil for consistent resistance to a penetrometer. Loose or soft fill can be located by probing the site with a length of reinforcing rod.

Classification of mine subsidence sites is usually provided by mine subsidence authorities. Their requirements apply not only to mined areas but also to future leases. Where underground mining does occur in the area and there is no statutory control of mine subsidence, the classifier should take necessary precautions.

For Item (e) of the Clause, severe moisture changes need only be considered when such conditions are known at the time of classification, for example, an existing large tree is to be removed, or a request is made to design footings for close tree plantings on a reactive site.

A classification of P, of itself, will not usually provide sufficient information for the preparation of a footing design. Depending on the cause for the P classification, supplementary geotechnical information will normally be required to allow a design to proceed.

C2.2 METHODS FOR SITE CLASSIFICATION

Accurate identification of the reactivity of a clay site by means of tests on samples throughout the soil profile is complex and expensive and may not be justified routinely on individual building sites.

Other methods are available and the Standard provides three procedures as follows:

- Prior performance.
- Profile identification.
- Movement estimates.

The simplest method, based on the history of performance, should not be underrated. Classification by history of performance is based on the fact that highly and extremely reactive clay sites cause clearly visible cracking of older masonry buildings on light strip footings. An inspection of the neighbourhood should indicate whether such a category is appropriate for the area. Such an inspection would only be meaningful if some knowledge of the soil conditions is available. Thus, either a soil or geological map should be consulted to ensure the neighbourhood has a similar soil profile to that of the proposed building site. It is also necessary to have some idea of the type of footings common in the area. If strengthened footings for reactive clays have been used for most of the buildings, this method is not applicable. Strengthened footings could be expected for the past 40 years or more in Adelaide but only for 20 years in Melbourne. In New South Wales, stiffened footings were generally introduced in the mid-1980s.

The method relies on an assessment of damage (cracking) of buildings of masonry (either veneer or full) construction, or the level of maximum differential movement of clad frame houses. Preferably, the appraisal should be based on buildings with similar wall construction to that which is intended to be built and which are at least 10 years old. If light footings have been used satisfactorily in the past, the classification of a site in that area should be Class S or at the worst Class M.

The degree of clay movement depends on the nature of the clay, depth of the clay, change in moisture content, and the ease with which water can soak into the clay. The extent of the moisture changes the clay will undergo is largely a function of the prevailing climate.

No single test can identify all these parameters. The Standard describes the properties of the foundation by one parameter, the expected characteristic surface movement (y_s). This is the vertical movement range expected during the life of the building from a reasonable estimate of dry conditions to a similar estimate of wet conditions and does not take into account the moderating effect of the footing system. The Standard nominates 50 years as the 'life' of the building and 'reasonable' as the level that could be expected for 19 buildings out of every 20. This does not mean that the building is not expected to last more than 50 years nor that 1 in 20 buildings could fail. It is, however, more reliable than using average conditions or an undefinable 'extreme' concept.

The effects of trees, poor site drainage, leaking plumbing and exceptional moisture-induced movements as outlined in Clause 1.3.3 are not taken into account in the calculation of y_s.

With the following definitions certain classifications are made:

S	=	Slightly reactive		y_s	≤20 mm
M	=	Moderately reactive	20 mm	$<y_s$	≤40 mm
H1	=	Highly reactive	40 mm	$<y_s$	≤60 mm
H2	=	Highly reactive	60 mm	$<y_s$	≤75 mm
E	=	Extremely reactive		y_s	>75 mm

In this assessment, y_s should be interpreted as the characteristic value that has a 5% chance of being exceeded in the life of the building, which may be taken as 50 years. Calculation of y_s have to assume the site maintenance complies with Appendix B.

To classify the site from estimates of soil movement requires geotechnical testing for, or assessment of, instability indices of the clay soils throughout the depth of soil affected by moisture change. In combination with a design moisture change profile (expressed as suction), this gives a good estimate of the likely movement.

Linear shrinkage, plasticity index and similar tests are not recommended for guesstimating movement unless sufficient data has been accumulated for soils of particular geological origin and type to correlate these simple tests with instability index values and hence allow the surface movement to be estimated.

Some areas such as Sydney and Melbourne may be classified without tests by identifying soil profiles, the behaviours of which are well known in the region. Such methods may be more accurate than movement estimates based on soil tests and recommended suction changes. The Tables in Appendix D of the Standard provide a ready guide to the expected level of site classification. These methods of soil profile identification are as follows:

- *Classification of Victorian clay sites* The Victorian clay profiles are generally derived from alkaline and acid volcanic rocks, Palaeozoic and Mesozoic age sedimentary and metamorphic rocks, limestones, alluvial, aeolian (wind-blown) and estuarine sediments. The most reactive clays are derived from limestones and alkaline volcanics.

  Climatic zones and local Thornthwaite Moisture Indices (TMI) are most important; therefore, it is recommended that Table 2.5, and Table D1 and notes, and Figures D1 and D2 of Appendix D are consulted. There are a number of published Australian TMI papers which can also be consulted; however, it should be noted that, the TMI calculation method has not been standardized and it is therefore difficult to compare from State to State.

Where the classification in Table D1 is to be refined, appropriate field and laboratory tests should be used. The geological terms used are explained by J.G. Douglas and J.A. Ferguson (Ref. 2). For further information refer to Atlas of Australian Resources, Department of National Development and Australian Resource and local Geological maps.

The accuracy of geological maps and geological boundaries varies and hence may not be sufficient to forecast soil profiles in areas on the scale of residential allotments. The behaviour of alluvial clays can be difficult to predict and knowledge of their source material is important to assess. This is particularly important where the sources are limestones or alkaline volcanic rocks that are exposed or have been fully eroded from nearby, or upstream, rocks or sediments. The use of geological maps and other information can assist in the classification but requires expert interpretations and is best used by a practitioner with many years of local geological knowledge and experience.

- *Classification of South Australian clay sites* For the Adelaide region, reference may be made to publications and maps published by the Department of Mines and Energy, South Australia and, in particular, Refs. 1, 3 and 4. More recent information is available from Ref. 5, which provides details of much deeper exploration and engineering classifications and tests, than the previous agriculture-driven explorations; however the area covered is the same as that covered by Ref. 1. Soil profile identification is only one part of the classification process in Adelaide, due to both soil variability and high levels of site reactivity. Soil cores are retrieved and a visual-tactile examination for reactivity is conducted by an experienced geotechnical engineer. From this assessment, the maximum site movement is estimated and the site is accordingly classified. Soil testing is not normally required, but is recommended for unusual soils or as a periodic check on the competence of the classifier. As Adelaide experiences a distinctly semi-arid climate, with a depth of design suction change of 4 m, the sub-classification '-D' applies to all site classifications.

- *Classification of New South Wales clay sites* Reactive soils are common throughout New South Wales.

  In rural areas (particularly the semi-arid and sub-humid interior) well-known examples of reactive soils include black, grey and brown clays. Many of these are found in north-western NSW and, to a lesser extent, mid- and south-western NSW.

  Due to wide climatic variations, these soils have high movement potential in both the horizontal and vertical planes.

  Gilgai formation, evident as surface humps and hollows formed by massive shrinking and swelling of the soils, are not uncommon.

  Most of the soils in the western suburbs of Sydney and along most of the north shore ridge are clay soils weathered from shales (termed the Wianamatta Group). Some soils still cover the rock from which they were formed (residual soils) and others have been washed downhill (slopewash soils). For this group, the classification is generally Class M. For those areas where it can be shown by simple site excavation that the depth of clay on the site is less than 0.6 m, the classification may reduce to Class S. Some sites in this group should be classified as Class H, where unusually severe moisture changes may be expected, for example, where the natural drainage is altered in a major way or in fringe areas of landslides where major water content changes may occur. Many of these sites are also prone to landslip and therefore have to be classified as Class P.

  In other areas of Sydney, the clays have not merely been washed downhill, but have been transported by rivers and streams to form deep alluvial deposits. These forms of alluvium are clearly identified on the soil maps as Mulgoa, Elderslie and Nepean.

These may have clay deposits of various thicknesses and the classification varies accordingly. Deep clay layers may potentially be affected by changing groundwater regimes arising from either extreme drought or by urbanization. Therefore, where clay depths exceed 2.5 m, a Class H classification may be more appropriate. If test data are used to check the classification, the theoretical depth of design suction change has to be increased to 2 m or the depth to weathered rock, whichever is less.

Generally, the site classification for most of the clay soils in Sydney will be Class M.

Some sites around Sydney are in sandstone or sand areas and may therefore be classed as S, or even A, if a site-specific geotechnical assessment has been carried out.

- *Classification of Western Australian clay sites* Although many sites in the Perth area comprise stable sand soils, care must be exercised as reactive clay, loose sand and peat may give problems. Low-density deposits of sand, particularly of calcareous composition, are prone to long-term settlement. Drainage of surface water or groundwater at shallow depth overlying or within clay may also be required. Guidance in terms of soil conditions in particular areas may be obtained by reference to the 1:50 000 scale Environmental Geology Series maps of the Perth Metropolitan area published by the Department of Minerals and Energy, Geological Survey of W.A.

  Soil conditions in many coastal towns vary with proximity to the foreshore and river courses. For example, in some coastal towns, sites may be located on reactive clay, or problem categories, whereas in other coastal towns stable sand sites predominate.

  Many inland country towns for example, Kalgoorlie, Northam, York, Dalwallinu, Ravensthorpe, Manjimup and Kununurra have reactive clay soils. In other areas (low density), sandy soils may show collapse on wetting. Examples include clayey sand in the Pilbara Region and sand that is in a loose condition, associated with limestone pinnacles in coastal areas. Local building knowledge, behaviour of surrounding buildings and site inspection are required to ascertain if testing for unstable soils is necessary.

- *Classification of Queensland clay sites* In Queensland a wide variety of soils and climates can be encountered. Soil mapping for site classification purposes has not been developed extensively for Queensland areas. Such mapping is problematic in areas of Brisbane due to hilly terrain and consequent complexity of distribution and variation in soil and soil types. Localized basalt flows and the intrinsically highly reactive soils that are weathered from them can result in sharp local variation in profile reactivity and hence site classification. Nevertheless, mapping could well be of assistance in other areas of the State where there is more consistency of soil over broad areas.

  Generally on clay sites, it will be necessary to engage a qualified engineer to classify the site. There are areas of 'black earths', which consist of intrinsically highly reactive clays. These occur in broad areas of the Darling Downs, which includes Toowoomba but also in smaller areas surrounding Brisbane and Ipswich and more generally in the Lockyer Valley. Such clays are well known for their large movement and would warrant Class H or Class E classification depending on location and climate zone. Since climate changes with distance from the coastline, careful consideration has to be given to the selection of the depth of design soil suction change (H_s). For example, this depth increases from 1.5 to 2.3 m across Brisbane and Ipswich. Fox (2000 and 2002) (Refs. 6 and 7) provides guidance on climate zones and depth of design suction change throughout Queensland.

The classification should be based on engineering principles if, as is usual, soil testing is to be used. Shrinkage index tests on an 'undisturbed' core sample are recommended rather than the conventional plastic index and linear shrinkage tests unless a reliable correlation of these tests with ground movements in the region is available.

In summary, unless there is well-established local knowledge about the behaviour of the clay sites, the site classification will require some engineering input.

- *Classification of Tasmanian and Northern Territory clay sites* No general information is available about the reactivity of Tasmanian and Northern Territory clays, but considerable local expertise is available from the building and consulting engineering profession. Information in Table D1, Appendix D, may be considered to supplement local expertise.

C2.3 ESTIMATION OF THE CHARACTERISTIC SURFACE MOVEMENT

C2.3.1 Characteristic surface movement

Estimation of the characteristic surface movement (y_s), for classification requires the soil suction model proposed by Aitchison (Ref. 8). In this model, soil suctions and instability indices are required to predict movement. These two parameters and the suction model are discussed in the following:

- *Soil suction* Soil suction is a measure of the internal stress caused by moisture in unsaturated clays. It is also a measure of the affinity that unsaturated soil has for water and can be expressed in terms of the energy required to extract a unit of water from the soil. The gradient of suction determines the direction of moisture diffusion in the soil. Suction is useful in the assessment of reactive site movement because it is a measure that is independent of soil type. It is further useful because it is more readily related to the effect of climate on soil moisture state than is the case for other measures of soil moisture. (For example, soil moisture content varies depending on soil type as well as climate.)

  Total soil suction consists of two components, namely matrix and solute suction. The matrix suction refers to the soil's affinity for water at the same salinity level. Solute suction is related to the salinity of the pore water. Changes in either form of suction may cause soil volume changes. Large changes in the solute suction and consequent movements are usually associated with leaking plumbing services.

  For convenience, suction is expressed as follows:

  $$u = \text{suction, in pF units} = \log_{10} (\text{suction in kPa}) + 1.01$$

  Suctions may be measured by a commercially available psychrometer or thermistors (total suction) or by filter paper techniques (matrix and total suction). Both techniques require considerable care. An Australian Standard test method is available for soil suction determination using a psychrometer to measure the dew point temperature (see AS 1289.2.2.1 *Methods of testing soils for engineering purposes*, Part 2.2.1: *Soil moisture content test—Determination of the total suction of a soil—Standard method)*. Whilst the methods of ground movement estimation set out in the Standard are based around soil suction, they do not necessarily require the measurement of the suction of soil samples in their implementation.

  The design suction profiles for the estimation of y_s should be related to local experimental data for the characteristic wet and dry profiles. It needs to be emphasized that such data must be relevant to the definition of y_s. The suction data should reflect the characteristic wet and dry condition in the soil profile. The wet condition should reflect the effect of development which, particularly in arid areas, may be wetter than the ordinary open field seasonal wet condition. Data from an open field site subjected to seasonal moisture changes may underestimate the suction change. Experimental suction profile data influenced by anomalies such as trees or other leaking pipes will also be invalid.

The characteristic value is defined as the value that has a 95% chance of occurring in the life of the building. Thus it is not necessary to consider the extremes of drying or wetting of the profile. The Standard provides recommended design suction change profiles and conveniently expresses the suction change as decreasing linearly with depth; however, it should be recognized that this simplification of the profile may lead to overestimates of movement if the depth of design suction change is estimated as the extreme value for suction change.

- *Instability index* The instability index (I_{pt}) may be estimated from the shrinkage index (I_{ps}), which is determined from shrink-swell, loaded shrinkage or core shrinkage tests (AS 1289.7.1.1, AS 1289.7.1.2 and AS 1289.7.1.3). A description of these tests is given in Cameron (Ref. 9).

  Guidance on estimation of the instability index from the shrinkage index is given in Clause 2.3.1 of the Standard.

- *Characteristic surface movement* The characteristic surface movement (y_s), for site classification is the integral of movement over the depth of suction change as follows:

$$y_s = \frac{1}{100}\int_0^{H_s} I_{pt}\ \Delta u dh \qquad \text{. . . C2.3.1}$$

where

y_s = characteristic surface movement, in millimetres

H_s = depth of design soil suction change

I_{pt} = instability index, see Clause 2.3.2

Δu = soil suction change at depth (h) from the surface, expressed in pF units

h = thickness of layer under consideration, in millimetres

Having determined the characteristic surface movement (y_s), the reported value for site classification should be to the nearest 5 mm. Information on the accuracy of estimates using the suction model is given in the study conducted by Cameron (Ref. 10).

C2.3.2 Instability index

The instability index (I_{pt}) is affected by the depth of the cracked zone in the soil profile. The relevant cracking to be considered is the predominantly vertically oriented cracking that extends downwards from the natural surface. Subject to the extent that these cracks are open, they freely allow horizontal swelling of the soil and thereby reduce the amount of vertical heave that would otherwise occur.

The predominantly vertical cracking under consideration here should not be confused with the slickensided fissuring that is very often present in reactive clay soils commencing at about the bottom of the vertically cracked zone and extending below it. A major cause of the slickensided fissuring is the shearing displacements that occur in the clay soil when it undergoes swelling whilst laterally restrained, that is to say, when swelling occurs in the deeper parts of the profile where vertical cracking is not present. The absence of vertical cracking forces all swelling to occur in the vertical direction, which is necessarily accompanied by shearing distortion of the soil. Over many cycles of seasonal swelling and shrinkage, the cyclic shearing displacements produce the polished fissure surfaces. The preferred inclination of the fissure surfaces is generally oblique rather than vertical. The presence of such polished or slickensided fissures can generally be taken as a positive indication that the soil is reactive. The fissures are a concomitant of the relief of lateral swell pressure through the occurrence of extra vertical swelling, which is the phenomenon that is addressed by the factor α in the calculation of y_s

In the calculation of y_s, the lateral resistant factor (α) is included to make allowance for the effect of both lateral restraint and vertical stress on the amount of vertical movement generated by swelling or shrinkage of the soil. In the cracked zone, α is set equal to 1.0, which is equivalent to assuming the swell and shrinkage of the soil laterally is unconfined and unaffected by vertical stress, except as already accounted for in the test procedure used to measure the shrinkage index (I_{ps}) of the soil. At the bottom of the cracked zone, there is a step increase in the value of α to around 1.6 to 1.8 (depending on the depth of cracking) and below that, α linearly reduces towards 1.0 at a depth of 5 m, all according to the following equation:

$$\alpha = 2 - \frac{z}{5} \qquad \text{. . . C2.3.2}$$

The adoption of a value of α greater than unity is intended to allow for the fact that, below the cracked zone, the tendency for lateral swelling of the soil is restrained and thereby partially converted into additional vertical swelling.

In fills that have been in place for less than 5 years before construction, it is deemed that the relieving effect of soil shrinkage cracking is absent. Similarly, when a site has been cut less than 2 years prior to construction, the effect of the depth of the cut in reducing the depth of the remaining cracked zone is to be taken into account. These requirements substantially increase the calculated y_s above that for a natural site comprised of the same soil. The effect of these requirements is illustrated by way of a specific example in Table C2.3 below.

TABLE C2.3

EXAMPLE CALCULATION OF y_S FOR EFFECT OF CUTTING AND FILLING

H_s	2.0 m		
I_{ps} throughout	4.2 %/pF		
Depth of cracking	1.0 m		
Depth of cutting or filling	1.0 m		
	Natural site	**Filled site**	**Cut site**
y_s (before rounding)	59.6 mm	94.1 mm	94.1 mm
Increase in y_s over that of natural site	—	58%	58%

The Standard does not indicate any requirement for modification of the suction change profile that is to be used in the calculation of y_s as a result of either filling or cutting of the site. The committee considered the question but concluded there was insufficient basis for a specific modification of the ordinary suction change profile that would otherwise be adopted in the absence of cutting or filling.

Generally, after cutting a site, the moisture profile of the remaining soil is likely to be closer to moisture equilibrium and therefore less predisposed to subsequent change and consequent movement than that of the natural profile. If taken into account, the more moderate moisture profile would lessen the calculated increase in y_s due to the removal of the cracked zone. This contrasts with the situation where fill is placed over a natural profile, in which case, the extent of any disequilibrium in the now buried natural profile forms part of the initial condition, and if allowed for, would increase the calculated y_s over and above that is due to the simple disallowance of cracking in the fill layer.

The question of whether the moderation of the extent of the initial moisture profile caused by cutting can be taken into account depends on whether it is considered that a cracked zone can fully redevelop before seasonal extremes affect the soil moisture profile. The effect of the present requirements, in the absence of an allowance for modification of the normal suction change profile, is consistent with the assumption that a cracked zone does not necessarily fully redevelop before moisture extremes may affect the finished cut soil profile.

C2.3.3 Soil suction profile

(No commentary)

C2.4 SITE INVESTIGATION REQUIREMENTS

C2.4.1 General

The physical requirements for site investigation relating to the frequency and depth of subsoil exploration locations are given in this Clause of the Standard.

C2.4.2 Purpose

(No commentary)

C2.4.3 Depth of investigation

(No commentary)

C2.4.4 Minimum number of exploration positions

The number of boreholes per site is dependent on the variability of the soil deposit, as well as the size of the planned building. Less boreholes are required for small extensions and outbuildings. Soil variability over a site is likely if either uncontrolled filling or gilgais are present. Gilgais are undulating surface structures (see Clause 1.8.29), which are indicative of highly expansive soils, and which give rise to variability of soil layering over relatively short distances. Where gilgais are recognized, more boreholes will be required per site to determine adequately the site classification. Gilgai structures are well known in suburban Adelaide.

C2.4.5 Bearing capacity

Assessment of allowable bearing pressure

For natural soils, the most convenient expression of the load-carrying capacity of the soil without the risk of failure or excessive settlement is the allowable bearing pressure. Most natural soils should be able to sustain the required pressures of 50 kPa or 100 kPa. Simple methods for assessing allowable bearing pressure are given below, but these should never be used to override an engineering assessment. Even the use of a pocket penetrometer for clays is preferred to 'rules of thumb'.

Conventional engineering techniques may be used to assess the allowable bearing pressure of soils. For example, in sand the Perth penetrometer may be used as a field test for safe bearing capacity. In general, the allowable bearing pressure takes into consideration both the strength and settlement characteristics of the soil. In particular, where the soil includes deposits of soft silt or clay, or loose sand, then settlement may govern and further investigation would be required. The following simple rules for safe bearing capacity may be used in conjunction with local knowledge:

- Loose sand means deposits into which a sharp pointed wooden post 50 mm square can easily be driven by a 5 kg hammer. (Loose sand must not be used as foundation without an engineering investigation, except that a stiffened slab may be used on loose sand where there is well-established local knowledge of satisfactory performance.)

Sand deposits into which a sharp pointed wooden post 50 mm square can be driven with difficulty by a 5 kg hammer may be taken as having acceptable bearing pressure.

- Soft silt or clay means a fine-grained soil that can easily be penetrated 25 mm in its natural condition by the thumb. Soft silt or clay must not be used as a foundation without an engineering investigation.

  Silt or clay that can be penetrated to a depth of 25 mm or less by the thumb with a moderate effort may be taken as having adequate pressure.

  NOTES:

  1 Tests on silts and clays should be made at moisture contents typical of wet conditions by testing fresh samples at suitable depths or by avoiding tests during dry periods.

  2 Tests for allowable bearing pressure should be made at a depth immediately beneath the foundation level. The soil at deeper levels should be checked to confirm that no weaker strata exist.

  3 If collapsing soils are suspected, then their presence may be further confirmed by the response of the soil to heavy watering or by excavation and backfilling (a lower volume after backfilling indicates a collapsing soil).

  4 The above guidelines are approximate and should not be used to limit allowable bearing pressures assessed by more accurate methods.

  5 The above methods do not apply to fill material.

C2.5 ADDITIONAL CONSIDERATIONS FOR SITE CLASSIFICATION

C2.5.1 Sites consisting predominantly of sand or rock

(No commentary)

C2.5.2 Effect of site works on classification

In Clause 2.4.5, the Standard sets limits to the amount of cut or fill that can be made to a site before reclassification is necessary. Some examples of the effect of cut or fill on classification are—

- increase in reactive movements by removal of part or all of a protective non-reactive soil layer (cuts up to half a metre deep are assumed not to affect the classification);
- reactive movements worsened by the addition of clay fill; and
- settlements caused by the weight of fill where weaker material underlies the site.

C2.5.3 Effect of fill on classification

The specification of compacted fill that is deemed to be controlled, without formal testing, is given in Section 6. Maximum thicknesses are also given. Where mechanical compaction is used, the depth of compacted fill that is allowed is up to 800 mm of sand (using a vibrating roller or plate) or 400 mm of material other than sand (using a mechanical roller).

Consideration needs to be given to the combined effects of the existing soil profile and the superimposed compacted fill. Where controlled fill has been used and has been subject to an engineering investigation of the fill and the underlying soil, it may be reasonable to assign a classification of Class A, Class S, Class M, Class H1, Class H2 or Class E to the site. In assigning such a classification, the classifier must assume all shrinkage cracks are effectively closed, which means lateral swelling of the soil is constrained and thus all soil volume change is expressed as vertical movement. The placement and compaction of fill is likely to diminish any existing cracking in the underlying natural ground due to closure of the cracks by the compactive effort and also by infilling of the cracks by the fill material.

A site classification can be improved by removal of clay and replacement with fill or by covering the clay with permanent fill. The depth of the fill should be based on an assessment of the effect of the fill on the movement in accordance with engineering principles but normally a depth of at least 1 m would be needed. Where the clay is replaced, the fill should be carefully chosen and compacted to protect the underlying clay from moisture changes. Fill materials should be selected to limit moisture changes in underlying reactive clay. Sealing and drainage of both the fill and the underlying clay may be required, particularly where excavation of the site may lead to entrapment of water. Where the site is covered with selected fill, the main effect of such fill is the establishment of uniform stable moisture conditions. To achieve this, the fill should be placed well before construction begins (e.g. two to five years).

Non-sand fill should be placed at a moisture content close to the optimum moisture content (OMC) for standard compactive effort. Compaction with heavy equipment at a lower moisture content may provide an initially strong and dense soil fill; however, in the long term, moisture will be re-distributed throughout the covered fill, leading to a wetting up of the soil towards the value of the OMC. Density and strength will be lost as the soil subsequently swells.

Reclassification of filled sites

Long-term equilibrium moisture conditions may be taken as marginally wet of optimum moisture condition (standard compactive effort) in the eastern coastal area or marginally dry of optimum moisture condition (standard compactive effort) in arid areas. Equilibrium moisture conditions may be estimated by reference to similar clays that have stabilized near the centre of large sealed surfaces. Allowance should be made for variation of moisture conditions in the foundation due to construction of the fill.

Alternatively, the movement may be estimated by reference to established knowledge of the behaviour of similar fills in a similar area. The alternative site classification must not be less severe than the site classification of the natural ground unless the controlled fill consists of non-reactive material and is deeper than one metre or 0.5 H_s, whichever is greater.

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SECTION C3 STANDARD DESIGNS

C3.1 SELECTION OF FOOTING SYSTEMS

C3.1.1 Selection procedure

The choice of an appropriate footing systems is most commonly made between a concrete slab and a suspended timber floor. The selection is made to suit site conditions, and the preferences of the builder and owner.

The Standard designs assume generally adopted building practices, and unusual or extreme forms of construction involving heavily loaded structural columns, suspended concrete floors or highly brittle features are not included.

The background to the design of slabs and footings involves the following:

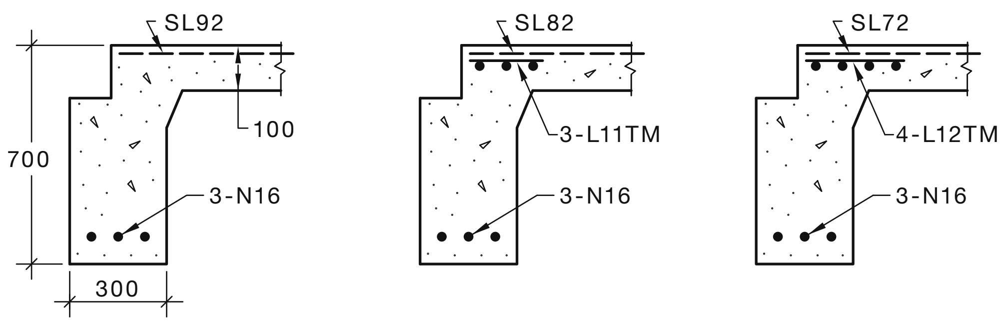
- *Design details and construction* All footing systems designed in accordance with Clause 3.1 have to comply with Sections 5 and 6, which provide design details and construction requirements.
- *Slab systems* For a concrete slab, a choice is needed between two main types of slab, namely the slab on ground with integral edge beams and the footing slab with separately poured edge footings. The footing slab requires more material and requires two pours to construct but it has a number of advantages; for instance, it adapts to sloping sites better than a slab on ground, and it does not require complex formwork and the trenches are open for less time. The integral slab on ground is stronger and is often more economical with materials. The choice will often be related to local experience; for example, footing slabs are dominant in Western Australia, slab on ground in South Australia, Victoria and Tasmania and New South Wales. Two-pour footing slabs were previously dominant in Queensland but now practice has moved towards slab on ground.
- *Structural proportions of slabs* A considerable part of the design process for slabs on Class A and Class S sites is largely based on past satisfactory performance of similar slabs, rather than any theoretical justification.

  Most of the structural strength of slabs for Class A and Class S sites is provided by the concrete. The reinforcement is included to control shrinkage and to provide some flexural strength in the case of cracking due to foundation movement. For the 300 mm deep edge beams, the flexural reinforcement L8TM is just above the minimum required to ensure that the reinforced flexural strength is greater than the cracking strength by 20%.

  The width of the edge beam and footings reflects the load applied to the footing. For the slab on ground it is assumed that a significant part of the load will be supported by the adjacent slab panels, consequently the minimum width of 300 mm is adopted, assuming an allowable bearing pressure of only 50 kPa is required. For the edge footing of a footing slab, the contribution by the adjoining slab panels is only available for the tied form of construction. Under footing slabs with separate strip footings, 100 kPa is the allowable bearing pressure and that the width is to be as specified for individual strip footings.
- *Slab thickness* For slab on ground construction (other than waffle rafts), slabs are generally required to be 100 mm thick. This is regarded as the practical minimum thickness for normal building construction, unless the construction is supervised by a qualified engineer, in which case, the minimum slab thickness may be 85 mm.

For two-storey construction under walls supporting the upper floor or under any masonry walls, the slab should be thickened to 150 mm over a width of 500 mm and provided with an extra strip of mesh as shown in the Standard. Otherwise, the loads are within the capacity of the slab and there is no need to thicken the slab. Thus, in single-storey construction thickening is not needed under either masonry or loadbearing framed walls.

- *Slab reinforcement* The reinforcement for slabs on ground is specified as SL72. For footing slabs without tied beams, the required reinforcement is SL82 where the slab length is between 18 m and 25 m and SL92 where the slab length is between 25 m and 30 m, due to the slab being freer to shrink without restraint. Nonetheless, these levels of reinforcement are very low—SL72 represents 0.2% and SL62 represents 0.16%. For slabs longer than 18 m, the cracking problems are potentially greater and mesh sizes SL82 and SL72 are required. Additional or heavier reinforcement or other measures may also be required where brittle floor coverings are used (see Clause 5.3.7). The slab mesh also acts as negative reinforcement for the edge beams. This avoids the need to locate bar reinforcement near the edge rebate (see Figure C3.1).



DIMENSIONS IN MILLIMETRES

FIGURE C3.1 REINFORCEMENT OPTIONS

- *Stiffened rafts* A variety of systems are available for placing a concrete floor on a reactive clay. The general form of construction of a stiffened raft is shown in Figure C3.2.

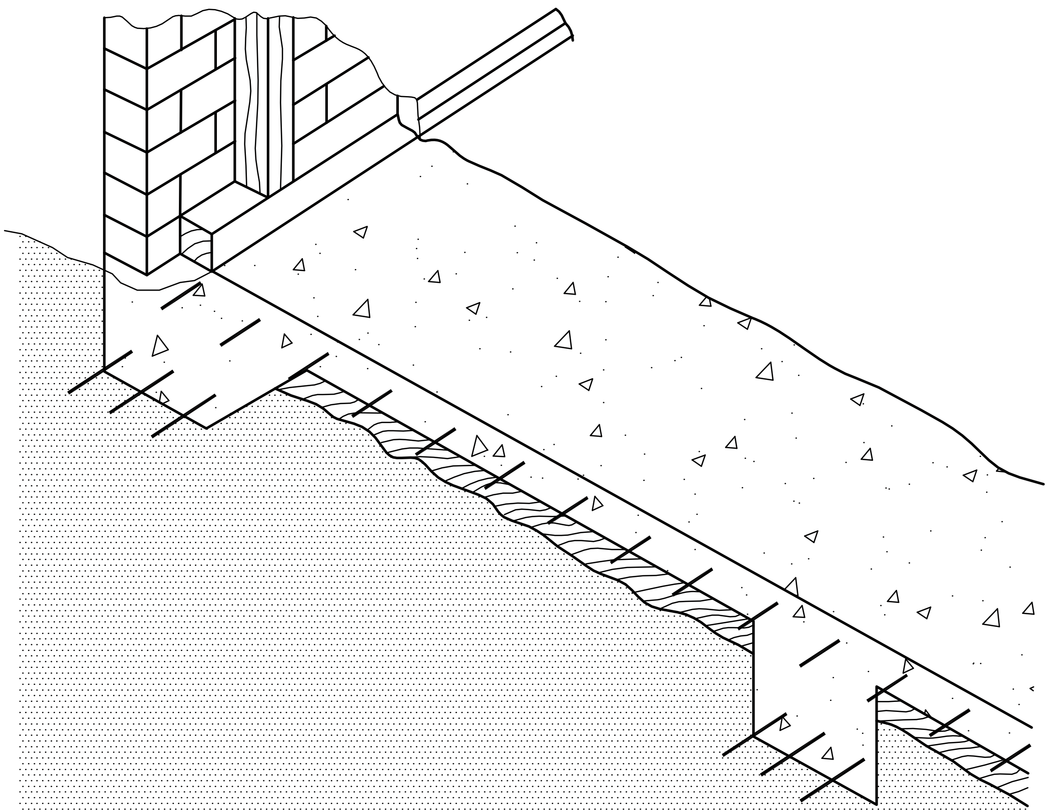


FIGURE C3.2 STIFFENED RAFT CONSTRUCTION

Stiffened rafts are constructed in Queensland and northern New South Wales with construction joints in the beams. The system relies on a structural connection between the edge footing and the stiffened slab by a reasonable concrete bond and by steel ties. This is not always easy to achieve and construction methods need to be carefully planned and controlled. This form of construction is now less common in Queensland, having been replaced by single pour raft construction.

The designs prescribed in the Standard were developed in part from an assessment of the performance of actual footing systems. This evaluation relied heavily on experience in Melbourne, with Sydney and Brisbane conditions also taken into account. Adelaide soil and climate were found to be quite different and separate designs were developed.

The most reliable data were for one-storey brick veneer buildings on moderately and highly reactive sites in Melbourne. This information was used to check the accuracy of the model based on engineering principles (Ref. 1). This model was then used to obtain designs for other conditions by extrapolation.

In a stiffened raft, the beams provide the double function of load support and stiffness against foundation movement. In order to provide the latter function, the beams have to be arranged in a grillage. Particular care needs to be taken to ensure that irregularities in the perimeter plan shape of the building are supported by the stiffening beam grillage. When designing the stiffening beam layout, the designer needs to keep in mind that under centre heave or edge settlement conditions, the perimeter-stiffening beam may lose ground support and depend largely on internal stiffening beams for support. Salient parts of the footing system plan, such as may be required for a brittle portico structure, need careful consideration if they are to be effectively integrated with the stiffened footing system for the building.

The layout of the beams should not be dictated by the layout of walls over the slab but rather should be arranged to provide a rational stiffening grillage for the whole slab.

If a beam is within 1000 mm centre to centre of a wall over the slab, the slab is considered strong enough to transfer the load to the beam. This may not be adequate in two-storey solid masonry construction with a concrete floor at the first floor level. In such cases, the slab should be specially designed.

For clad frame and masonry veneer, only light beams are required and this reflects the movement tolerance of the framed construction relative to the differential movement expected. The beam spacing of 6 m to 7 m is permitted and this should only require relatively few beams. If the spacing in the one orthogonal direction is reduced, the spacing in the other direction may be increased. The top reinforcement to the beams is provided by the mesh used in the slabs.

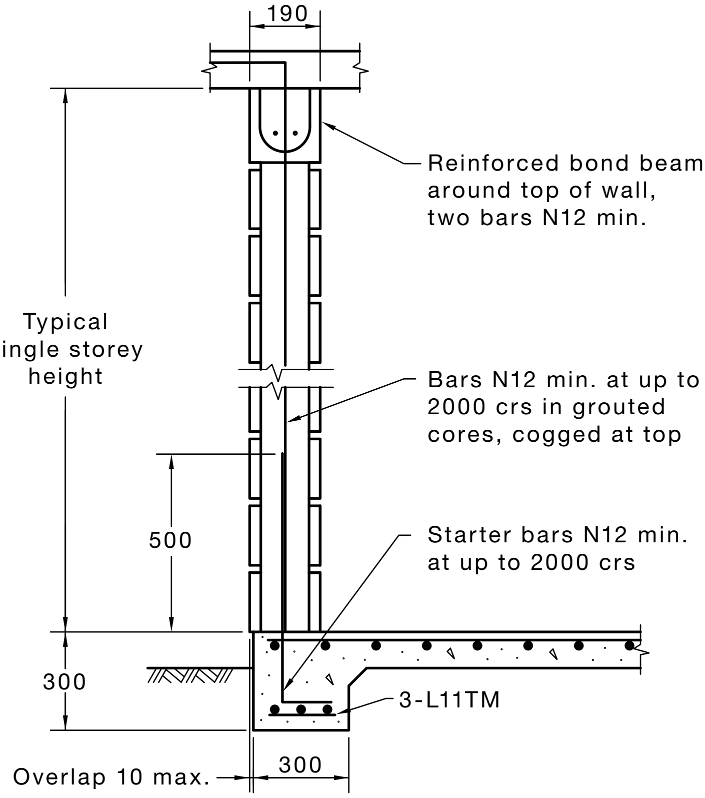
For full and articulated masonry, the designs are much stiffer and stronger. This reflects experience in Adelaide regarding vulnerability of such wall construction to cracking, and the sizes needed to achieve satisfactory performance.

For highly reactive sites, much stronger designs are given. The 'standard case' of masonry veneer has edge beams 300 mm × 500 mm with three wires of L12TM. This is stronger than the intermediate slab, which, prior to the advent of the 1986 version of this Standard, has often been successfully used in Melbourne on such sites. The stronger design was adopted to allow for the wider variety of sites that may be covered by this category of highly reactive sites (H1 and H2).

The beam sizes of Figure 3.1 provide adequate stiffness to ensure that non-structural wall systems placed on the slab are not subjected to excessive deflection; however, Clause 3.2.5 permits a reduction in these beam sizes to 300 mm × 300 mm with 3-L11TM reinforcement, if reinforced hollow concrete blockwork walls are structurally connected to the beams and act with them to resist movement.

In this case the walls have to be 190 mm single-leaf hollow concrete blockwork, reinforced with at least N12 bars at not more than 2.0 m centres, tied into the footings with starter bars and incorporate a continuous bond beam with at least two N12 bars around the top of the wall (see Figure C3.3). The walls should be adequately waterproofed.

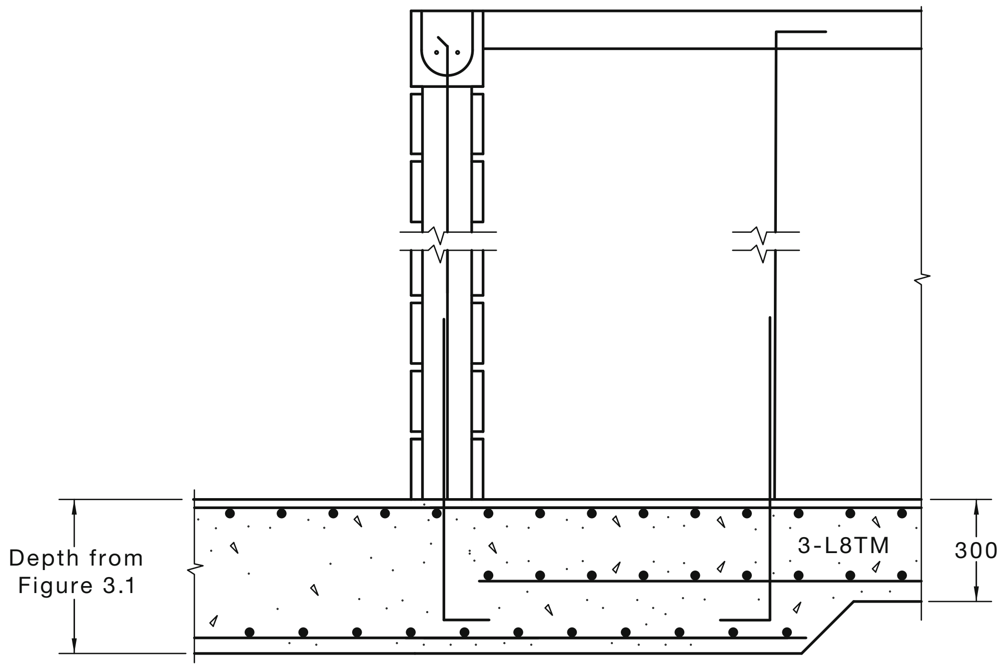
This construction behaves as a 'stiff box'. Articulation of the bond beams should not be included since it destroys the continuity. When using this detail, care must be taken to ensure the adequacy and continuity of internal beams, particularly at re-entrant corners where internal beams are deeper than the external beams. Figure C3.4 shows a typical section and detail at re-entrant corners. This system is further considered in Clause 4.7



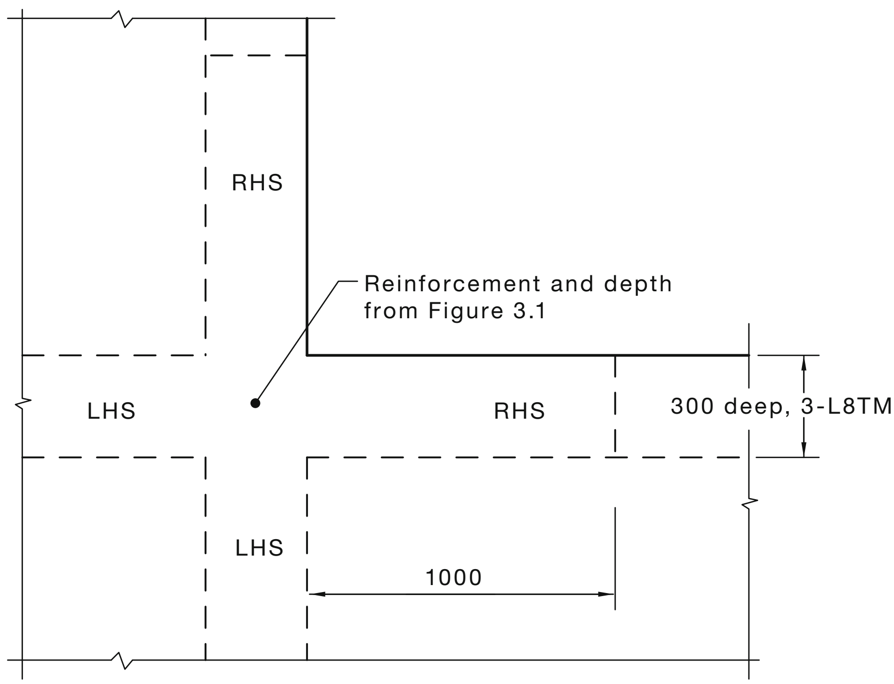
NOTE: Waterproofing is required to exterior face walls constructed and reinforced in accordance with AS 3700. Footings are suitable for openings up to 1800 mm. For wider openings, use established concrete and reinforced concrete masonry analysis methods to determine the required footing sizes.

DIMENSIONS IN MILLIMETRES

FIGURE C3.3 TYPICAL DETAILING FOR FOOTING AND SINGLE-LEAF REINFORCED MASONRY WALL COMBINATIONS



(a) Section at re-entrant corner



(b) Plan at re-entrant corner

DIMENSIONS IN MILLIMETRES

FIGURE C3.4 TYPICAL RE-ENTRANT CORNER DETAILS

- *Waffle rafts* Waffle rafts are a particular form of raft slab where the raft, including ribs, is constructed on a prepared flat ground surface. The regular grid of ribs is formed using void formers. The structural design method in Appendix F is also suitable for waffle rafts. Although the bearing area provided by the narrow ribs is less than that of a conventional raft, an allowable bearing pressure of 50 kPa under the ribs is adequate.

  The designs provided in the Section on waffle rafts are based on engineering analysis using the same principles as for stiffened rafts. The construction is completely on-ground rather than in-ground and this has several features:

  - The shrinkage behaviour is improved due to the lower restraint compared with a raft with embedded beams.
  - The structural performance is enhanced, as there is no concern about down-drag of embedded beams due to clay shrinkage.
  - The proportions of the cross-section may be achieved reliably without excess concrete being needed due to over-excavation.
  - There is a greater propensity for ingress of moisture under the slab.

  Bored piers (if required) should be designed in accordance with engineering principles and Paragraph G5, Appendix G, of the Standard.

  Both down-load and uplift conditions should be considered and the design of bored and excavated piers should be determined by engineering principles.

- *Stiffened slab with deep edge beams SSD (Class M)* This form of slab, illustrated in Figure 3.5 of the Standard, is restricted to Class M sites and the more flexible internal construction of clad frame or masonry veneer. This form of construction relies on deep strong edge beams to provide stiffness. Internal beams at re-entrant corners in T-shaped and L-shaped buildings would have a significant effect but they are not always required (see Figure 3.5 of the Standard). The edge beams may include reinforced masonry.

- *Strip footing proportions* A strip footing is a footing of rectangular cross-section used to support the external or internal walls of a building. For clad-frame construction, a strip footing is only required if a masonry dwarf wall is used. For masonry veneer, the external wall is supported by a strip footing and the internal frame is usually supported on pad footings with stumps or piers, although internal strip footings are possible. For solid masonry, strip footings will be required under both internal and external walls.

  The proportions of a strip footing are controlled by several factors including the allowable bearing pressure, the need for strength and stiffness to cope with some minor movements, practical limits on size and a suitable foundation depth.

  The typical applied bearing pressures are given in Table C3.1 for the footing widths in the Standard and, except for two-storey masonry construction, are usually quite moderate. To allow for uneven loading and eccentricity, the minimum required allowable bearing pressure is set at 100 kPa.

TABLE C3.1

TYPICAL APPLIED BEARING PRESSURE UNDER STRIP FOOTINGS, kPa

Type of construction	Single storey	Double storey
Clad frame	33	50
Masonry veneer	50	65
Solid masonry	65	80

Although theoretically Table C3.1 suggests the sizes could be even smaller for the lightly loaded footings, there are practical difficulties in constructing footings narrower than 300 mm and problems with possible eccentricity of the load may occur with very narrow footings. For simplicity, the Standard has adopted the same sizes for both one-storey and two-storey constructions.

Strip footings are able to withstand some foundation movement. For example, the masonry veneer footing could withstand loss of support over a length of up to 3 m within its length, reducing to approximately 1.2 m at the corner.

- *Pad footings* Pad footings are used to support stumps or piers which in turn support a framed structure. Considerable differences exist between State practices in this area.

  The loading on a stump may be determined from Appendix E and AS 1684 series, depending on the bearer spacing and similar. For larger loads, an appropriate pad size is specified depending on the area supported. The design of pad footings on reactive clays should take into account the expected depth of moisture change; however, it will frequently be uneconomical to found footings at completely stable depths. Shallower footings with the consequent possible need for minor maintenance, such as repacking, may be more economical.

- *Strip and pad footings on reactive sites* Standard designs for strip and pad footing systems on reactive sites have been extended to cover class M-D sites and some full masonry applications. This reflects the findings of recent surveys in the Melbourne area which indicate good performance from such systems when correctly specified.

  The plastic membrane around the strip footings, where specified, is intended to limit down-drag where clay soil shrinks. The internal support may be on deep stumps or gridded internal strip footings. A combination of the two types of internal support is possible, but is not recommended due to the potential for differential movement.

  In some cases, to minimize the damage likely to be caused by differential movements, a minimum spacing is specified from the outside footing to the first row of internal stumps. This is intended to ensure that any differential movement between the internal and external footings is only expressed over a reasonable length of structure thus limiting the rotations and deflection ratio.

  The cross-section proportions are chosen to give high contact pressure, strength and stiffness to both suppress and resist ground movement. Settlement of stumped or piered footings under the internal area of buildings is common but fairly simple to repair by packing. Such settlement occurs because the clay under the building dries to equilibrium within the ventilated subfloor space. This subfloor space is very dry in comparison with the clay and can cause clay drying to significant depths with associated 'settlement' movements. The founding depths specified in the Standard may not always be sufficient to avoid settlement of internal footings but have been chosen for economic reasons.

A variety of materials may be used as infill floors to strip footing systems. When selecting floor and wall finishing systems, designers should take account of the potential for differential movement between the floor and the wall.

- *Reactive designs on stable sites* In some circumstances it may be economical to use a reactive clay design (e.g. a waffle raft) on a Class A or Class S site.
- *Selection procedure—Limitations on application* The Standard provides standard designs for a number of different styles of footing. In selecting from the Standard designs, it is important to remember that the solutions are intended to cover a range of the systems most commonly used Australia-wide.

  There are distinct limitations on the 'deemed-to-comply' application of these standard solutions and these are listed in the Standard. Some particular points are as follows:

  - *Slab size* Factors that normally are not critical may become so when slabs are longer than 30 m in their longest dimension. Examples are concrete shrinkage and raft action. A more subtle limitation relates to the interaction of slab distortion and stiff roof structures that span the full width of the building, such as a truss roof. The deemed-to-comply designs tacitly assume that trusses will be arranged to span across the lesser width of the building, which dimension will normally be much smaller than the overall length of the building. The truss span will typically be less than 10 m. If for any reason the trusses are arranged to span the length of the building or if the building is more nearly square in plan, then the Standard footing system designs for a given maximum length may not be adequate to limit damaging interactions between the trussed roof and other parts of the building. This circumstance is more likely to arise in buildings of a character other than detached domestic residences.
  - *Joints* The Standard slabs systems rely on composite action between the footing and slab components. If the structural integrity of the slab is interrupted by a permanent joint, the strength and stiffness of the full section will be less than adequate in most cases.
  - *Concentrated loads* The restrictions relating to wall heights, columns and similar structures are to ensure that the load limits implicit in the Standard designs are not exceeded.
  - *Unreinforced masonry arches* Unreinforced masonry arches are specifically excluded as they are not only physically crack-sensitive but they also usually represent an architectural feature and owners have a lower than usual tolerance for cracks in such features.

Further information on footing system design is available in References 2, 3, 4, 5 and 6.

C3.1.2 Design for single-leaf masonry, mixed construction and earth wall construction

Some concessions are permitted in this Clause to allow for the greater strength and improved crack control of reinforced masonry. This Clause is generally expected to apply to the masonry construction typical of the cyclonic areas of Queensland.

C3.1.3 Construction with framed party walls

(No commentary)

C3.1.4 Design for masonry feature walls

Masonry veneer or strip footing buildings may include isolated masonry walls such as feature walls and walls for garages. This Clause permits the use of the masonry veneer slab or footing system with minor local modifications under the wall concerned. If an additional tie beam is required, it should be integral with the main footing system to reduce the risk of differential movement.

C3.1.5 Design for outbuildings and extensions to dwellings

The Clause requires articulation at the junction of the extension and the existing building. Articulation may be provided by a full height door or window.

A minor concession is offered in this Clause for clad-frame outbuildings and extensions. Design may be used for one class less severe than that required by the site classification. Thus Class M design may be used on a Class H1 site. For the purpose of this Clause, an outbuilding or an extension should be limited to 9 m length.

This Clause also permits the use of footings of the same proportions as the main building.

C3.1.6 Design for rock outcrops

Additional reinforcement may be used instead of expensive excavation of isolated rock outcrops. This provision is also relevant to floaters or isolated detached rocks within the soil profile.

C3.1.7 Design for partial rock foundation

Where a cut and fill or similar condition results in part of a building being on rock and part on natural soil (or controlled fill), then the possibility of minor differential movement exists. To accommodate such movement articulation or strengthening is required.

Where the building is supported over a large area by rock with the balance on reactive clay, there is a potential for severe differential movement and appropriate design changes by a professional engineer will be needed.

C3.1.8 Design for complete rock foundation

(No commentary)

C3.2 STIFFENED RAFT

(No commentary)

C3.3 FOOTING SLAB

(No commentary)

C3.4 WAFFLE RAFTS

(No commentary)

C3.5 STIFFENED SLAB WITH DEEP EDGE BEAM

(No commentary)

C3.6 STRIP FOOTINGS

On a reactive clay site it is important that separate footings are not used for concentrated loads because of the possibility of differential movements. Such movements can be tolerated if the structure giving rise to the loads is fully isolated from the rest of the house.

C3.7 REINFORCEMENT EQUIVALENCES

(No commentary)

C3.8 SUSPENDED CONCRETE FLOORS IN ONE-STOREY CONSTRUCTION

(No commentary)

C3.9 FOOTING SYSTEMS FOR TWO-STOREY CONSTRUCTION WITH SUSPENDED CONCRETE FLOOR

(No commentary)

C3.10 FOOTINGS FOR CONCENTRATED LOADS

(No commentary)

C3.10.1 Footings for columns

(No commentary)

C3.10.2 Footings for fireplaces on Class A and S sites

(No commentary)

REFERENCES

1 WALSH, P.F. *The Analysis of Stiffened Rafts on Expansive Clays*. CSIRO—Division of Building Research, Technical Paper No 3, 1978.

2 WALSH, P.F. *To Beam or not to Beam*. Building Surveyor, 1978. Vol. 2 p.24/26

3 CAMERON, D.A. AND WALSH P.F. *Footing Systems and Floors for Houses; Opinions, Experiences and Trends*. CSIRO Division of Building Research Report, 1984.

4 WALSH, P.F. *The Analysis of Stiffened Rafts on Expansive Clays*. CSIRO—Division of Building Research, Technical Paper No 3, 1978.

5 JUNIPER, P.M. AND YTTRUP, P.J. *Footing systems for small scale buildings—A new era*. Queensland Master Builders Association, Housing and Construction in the Age of Technology International Conference 1988, Gold Coast, Queensland.

6 MITCHELL, P. J. *Footing Design for Residential Type Structures in Arid Climates*. Australian geomechanics 33(4) Dec, 2008, pp 51-68.

SECTION C4 DESIGN BY ENGINEERING PRINCIPLES

C4.1 GENERAL

The Standard allows the modification by engineering principles of the standard designs given in Section 3. Footing system design by engineering principles is permitted as a complete alternative to adoption of the designs given in Section 3 of the Standard. With adequate justification, a slightly or completely different footing system could be designed, but this option should not be used unless there are good grounds for changing the standard designs.

The Standard also allows for the design of slabs or footings in accordance with AS 3600. This alternative should only be adopted by engineers who have considerable knowledge of the concepts of soil/structure interaction and of the related structural design procedures.

It is also possible for an engineer to select a standard design without modification. In such case, the engineer may judge that some of the limitations in Clause 3.1.1 may not apply. If a footing system is designed by a qualified engineer, that design need not follow any of the structural proportions set out for the Standard designs. It is also possible to use engineering principles to extend the applicability of the Standard designs or to modify them for special purposes.

The following points relate to the engineering principles involved in the development and modification of the standard designs:

(a) *Stiffened rafts* The design for stiffened rafts requires the provision of strength and stiffness to control the effects of ground movements so the relative deformations of the raft are within the tolerance limits for the building. A suitable method of design is given in Section 4, although design may also be based on a history of past satisfactory performance. As with strip footings, very deep beams should be avoided, otherwise account should be taken in the design of the factors referred to in Clause 3.5.

The stiffness of the raft relies on the full depth of the beam and slab. Permanent joints in the slab (e.g. control joints for contraction and expansion) reduce the concrete section at the joint location and should not be used unless the raft design makes special provision for the reduction in the effective section.

(b) *Strip footing design* In addition to the normal requirements of load distribution and limitation of the bearing pressure, the design of a strip footing on reactive clay should take into account—

(i) expected ground movements;

(ii) adhesion on the sides of the footing; and

(iii) differential moisture conditions created by the intrusion of the footing into the soil.

In the absence of more accurate information, the effect of adhesion may be taken into account by considering the loading on the uplifting member calculated according to soil mechanics principles. It is also possible to reduce adhesive soil loads using plastic sheeting.

It has been found that deeper strip footings are not always effective due to Items (ii) and (iii) above and that shallow more heavily reinforced footings may be more appropriate.

The design methods in Section 4 may be useful for strip footings although, more often, past satisfactory experience is appropriate.

(c) *Concentrated loads* Normally the method of support for concentrated loads will be directly on a beam or footing for moderate loads or on a strengthened beam or footing for heavier loads. The amount of strengthening required for the beam can only be clearly assessed using the beam-on-mound analysis in a design by engineering principles. As a rough guide, loads near the corners should be provided with sufficient support assuming a cantilevered corner over say 2 m. Internal beams may need to be strengthened to provide load transfer to intersecting beams at each end.

(d) *Pier-and-beam, pier-and-slab or pile footing systems* In addition to the structural design requirements of AS 3600, the design of a pier-and-beam or pier-and-slab footing system should take into account—

   (i) depth of piers to natural or stable soil including allowance for anchorage; and

   (ii) provision against uplift in design or isolation of the footing system or superstructure.

   Pier-and-beam and pier-and-slab design generally require attention to pier anchorage and isolation of the slab and beams from swelling soil. Piers should be founded in soil at a level unaffected by moisture variation. Piers can have under-reamed bases within stable soil or they may be lengthened in the stable soil to ensure adequate anchoring by skin friction. In either case, the tensile capacity of the pier section has to be carefully considered. The design anchorage requirements may be reduced by requiring piers to be sleeved within part or all of the swelling soil zone.

   Isolation of the beams and slabs is difficult to achieve economically. Void formers that rely on degradation of organic products should not be used unless it can be ensured that the material will rot away rapidly. Alternatively, it may be feasible to tie down beams. Where void formers are used, it is essential that the space created does not become a water trap that contributes to soil swelling.

   Experiments in Melbourne (Ref. 1) have shown that piles can be very effective in resisting reactive clay movements. Piles may be used as point supports or in combination with beams. The comments on pier-and-beam and pier-and-slab are also relevant to piled systems.

C4.2 DESIGN CRITERIA

The deflection of a footing system should be measured from a straight line joining the ends. This removes the rotational deformation from the determination of differential deflections (see Figure C4.4).

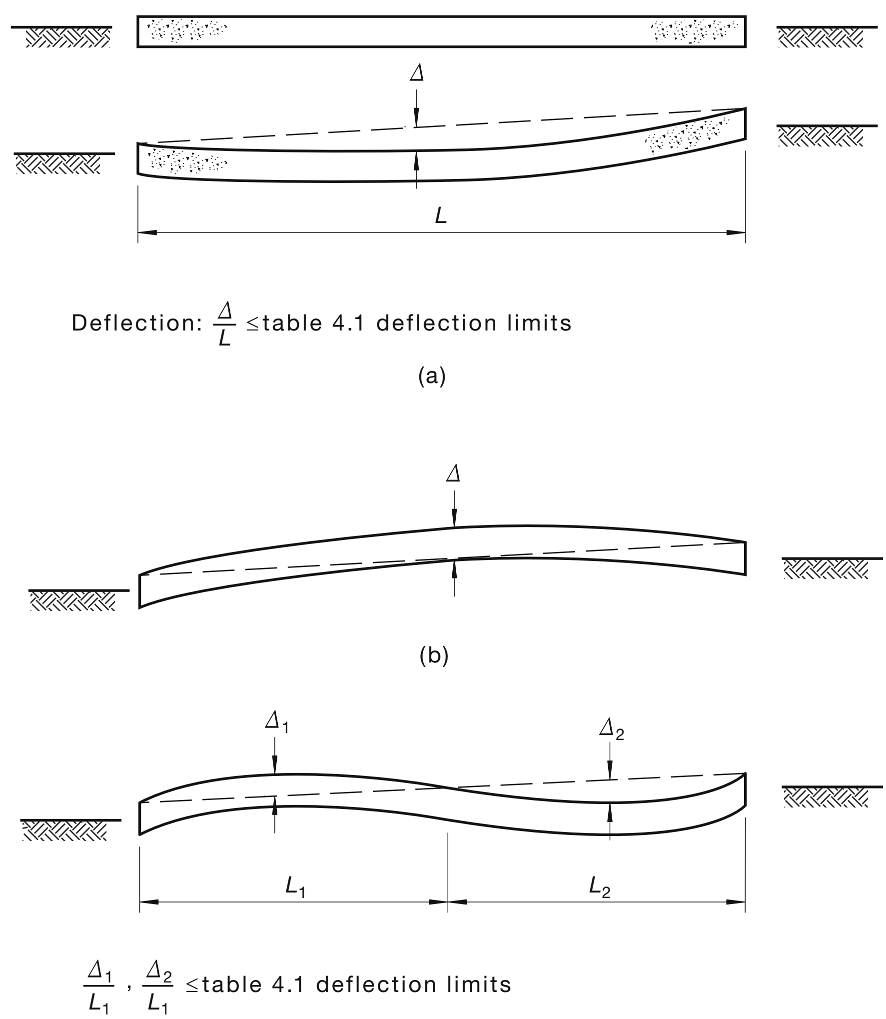


FIGURE C4.1 MEASUREMENT OF DEFLECTION

C4.3 DESIGN OF FOOTING SYSTEMS

(No commentary)

C4.4 STIFFENED RAFT FOOTING SYSTEMS

Table 4.1 provides guidelines for maximum design differential movements for different types of construction. The values selected from this Table should not be used out of context, or have inordinate importance placed on them. Various limitations on these values are as follows:

- The values are for use in footing system design. They do not necessarily refer to a measurement that can be applied to an existing structure.
- The differential movements referred to in this Table are between elements contained within the structure, as measurements of distortion of the frame. A complete footing system may move as a unit, without causing structural distress or serviceability problems.
- While movements stated as a function of span are useful design parameters, their applicability in assessing existing structures is limited. For example, it is very difficult to determine the actual span that is applicable for the formula.

C4.5 SIMPLIFIED METHOD FOR RAFT DESIGNS

C4.5.1 Application

The lines given in Figure 4.1 are derived from the Standard stiffened raft footings of Section 3. They can be considered to represent the 'families' of footings for shallow and deep clay profiles. It follows that alternative footing systems that fall on the respective line will perform in a similar manner to the Section 3 footings.

The inclusion of Figure 4.1 in the Standard is to provide an intermediate tier of analysis, allowing the engineer to interpolate between the Section 3 systems. This procedure is expected to be of value in allowing a rational determination of different beam depth and spacing, of footing needs for different types of superstructure or of sites of particular y_s, without the need for full engineering analysis.

C4.5.2 Modification procedure

Note that the calculation of the stiffness parameter is based on the rectangular beam section, that is neglecting the flange. This does not cause a loss in accuracy, as the lines are derived from the rectangular sections only of the Standard designs.

C4.6 DESIGN OF FOOTING SYSTEMS OTHER THAN STIFFENED RAFTS

(No commentary)

C4.7 FOOTING SYSTEMS FOR REINFORCED SINGLE-LEAF MASONRY WALLS

(No commentary)

C4.8 DESIGN FOR PILED OR PIERED FOOTING SYSTEMS

(No commentary)

REFERENCE

1 CAMERON, D.A. AND WALSH, P.F. *The Pile Experiment*. CSIRO—Division of Building Research Report, 1983.

SECTION C5 DETAILING REQUIREMENTS

C5.1 GENERAL

(No commentary)

C5.2 DRAINAGE DESIGN REQUIREMENTS

C5.2.1 General requirements

Defective surface drainage is a common causal factor in reactive clay foundation movement problems. The selection of an appropriate site falls and floor levels should be part of the planning and setting out process.

The effective drainage of the site is a prerequisite for satisfactory performance of the footing system, particularly on reactive clay sites. Problems can arise where the landscaping and other finishing earthworks are not part of the builder's contract, even though drainage requirements have been stipulated as part of the footing design.

In such cases, the owner may be directly or indirectly responsible for the completion of the site works. This highlights the need for the owner to be advised of the general requirement for drainage and any particular requirements attached to the footing design.

The selection of slab levels and slab edge details should take account of the subsequent earthworks that may be required to achieve satisfactory drainage of the site. Intractable foundation problems can be created where the floor level is set too low on flat reactive clay terrain.

The finished ground surface must fall away from the perimeter footing. Where this is achieved by filling, the nature and permeability of the filling should be considered in relation to the underlying soil. Figure C5.1 illustrates an unsatisfactory situation that can result where surface falls are achieved by placing sand over less permeable clay. The permeable filling in combination with the back-fall in the underlying clay can trap water and allow it to infiltrate into the foundation soils.

The drainage of zero lot line sites may pose special problems. The Committee considered that there is not sufficient experience with zero lot line construction to enable specific requirements to be included in the Standard. It is also recognized that zero lot line construction on reactive clay sites has the potential to create problems that involve a complex mix of technical and legal aspects.

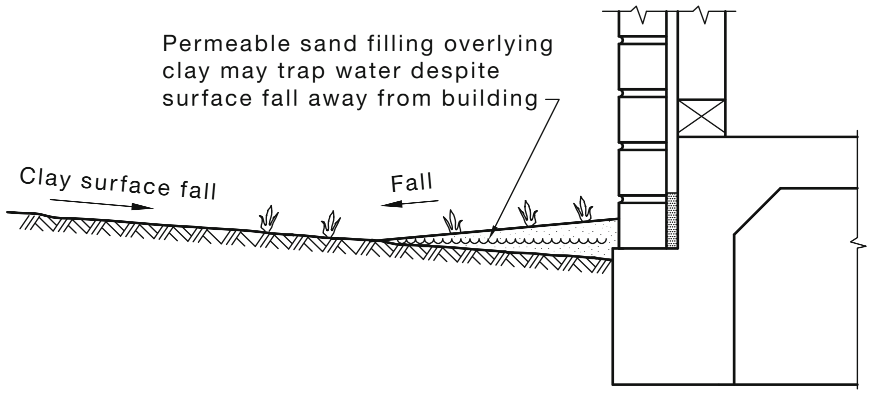


FIGURE C5.1 UNSATISFACTORY METHOD OF ACHIEVING SURFACE FALL AWAY FROM BUILDING

C5.2.2 Specific requirements for slabs for Class 1 buildings

The freeboard of the slab or height of the slab surface above finished ground is often over-emphasized because drainage of the ground around the slab is far more important, particularly on sloping sites. On low-lying level sites, freeboard may be of concern and will certainly be of concern in flood-prone areas.

The relative heights of the overflow relief gully, slab and finished ground are intended to stop sewage flooding the building if a blockage occurs, and to prevent rainfall run-off flowing into the sewer. However, this requirement only restricts the freeboard locally. The actual dimensions depend on local plumbing regulations and AS 3500.2, *Plumbing and drainage*, Part 2: *Sanitary plumbing and sanitary drainage*. There may also be building regulations controlling this aspect.

C5.3 REQUIREMENTS FOR RAFTS AND SLABS

C5.3.1 Concrete

Concrete quality is specified generally in accordance with AS 3600. A concession based on past satisfactory performance is offered for exposed concrete at the edge of the slab and for concrete in external patios in that only N20 grade concrete is required.

The recent recognition of the presence of saline areas, and the need to design for more aggressive soil environments in some locations, impacts on the selection of appropriate concrete grades (see Commentary C5.5).

C5.3.2 Reinforcement

In the Standard, mesh for slab and beam reinforcement is specified mainly as a result of experience in most States. Generally, trench mesh is simple to use, can be placed fairly reliably with adequate laps and is easily supported in place. Fitments may be used to hold reinforcement in place but are not required by the Standard.

The builder and building certifier should be cautious about substituting new forms of reinforcement for conventional steel reinforcement. It should be appreciated that the slab mesh acts not only as shrinkage control but also as structural reinforcement and cannot be replaced by alternative methods on the grounds of equivalent shrinkage control alone. In particular, the claims for polypropylene at the low rates used in Australia were treated with caution by the Committee, and substitution should only be made when adequate provision is made to ensure shrinkage control and structural performance.

Trench mesh overlapping, splicing and minimum cover requirements described in Clause 5.3.2(c) are shown in Figure C5.3.

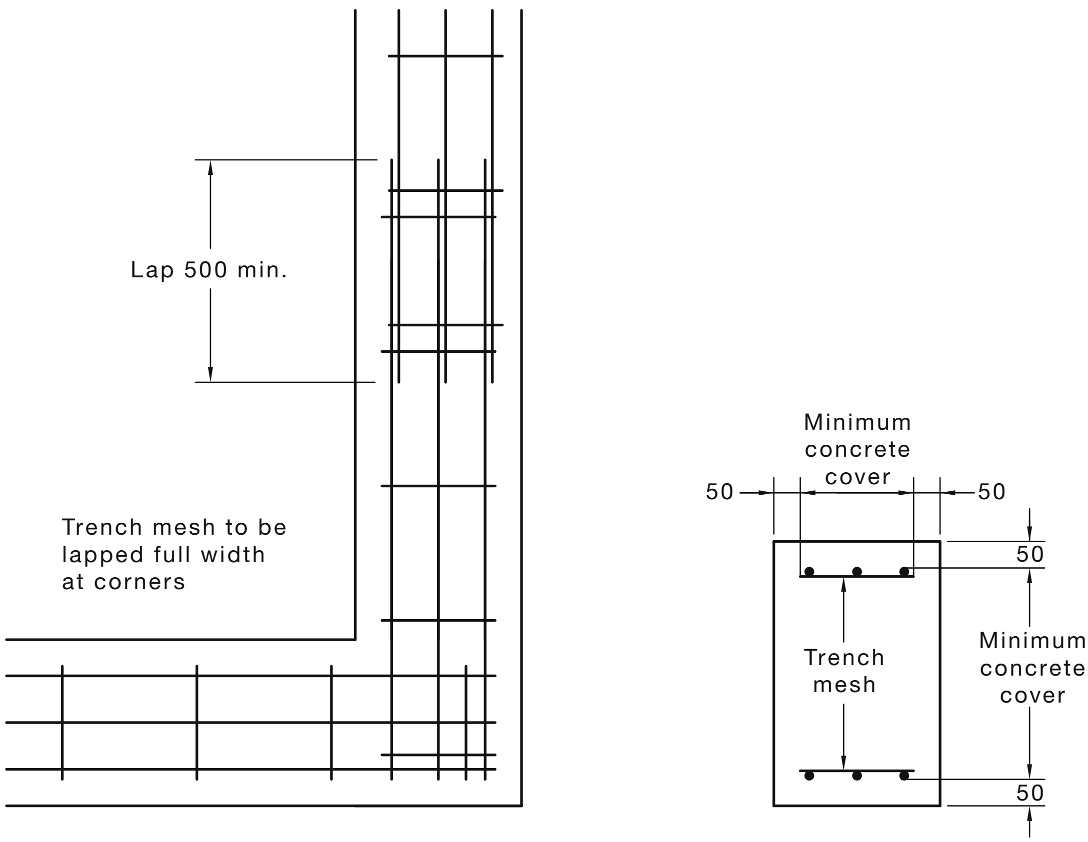
Clause 5.3.2(d) refers to the lapping of bars in beams at T-intersections and L-intersections. Where the edge and internal beams are not at the same level it is necessary to provide sufficient load transfer across the construction joint. A detail such as that shown in Figure C5.4 would be helpful in avoiding possible weakness of the joint.

C5.3.3 Vapour barrier and damp-proofing membranes

The vapour barrier is a barrier against vapour rising through the air in the soil and otherwise condensing in the slab or being trapped under impermeable floor coverings. It is an important part of construction but the need for it should not be exaggerated. Direct water transmission is best dealt with by effective site drainage, adequate freeboards and good quality concrete, well compacted as it is placed. As the barrier is against vapour, not water, minor punctures are not important. Similarly, joints need only to be lapped, not taped; however, intermittent taping is recommended to help to keep the vapour barrier in place.

The BCA now incorporates specific requirements for vapour barriers and damp-proofing membranes. In particular, reference should be made to the NSW and SA variations.

Experience in Australia has indicated that the vapour barrier may be terminated on the inside of edge beams and at the faces of internal beams. This is permitted by the Standard, where supported by local practice. This concession was intended mainly for two-stage stiffened footing slab construction with very deep (600 mm) beams. For normal slabs and lightly stiffened rafts, the vapour barrier should completely underlay the slab, including all beams. It may be terminated at the bottom outside face of the edge beam. Indeed, unless a multiple brick rebate is used and the membrane is not exposed, it seems more satisfactory to terminate the vapour barrier below ground.



(a) Plan of strip footing at corner

(b) Section through strip footing

DIMENSIONS IN MILLIMETRES

FIGURE C5.2 TRENCH MESH DETAILS

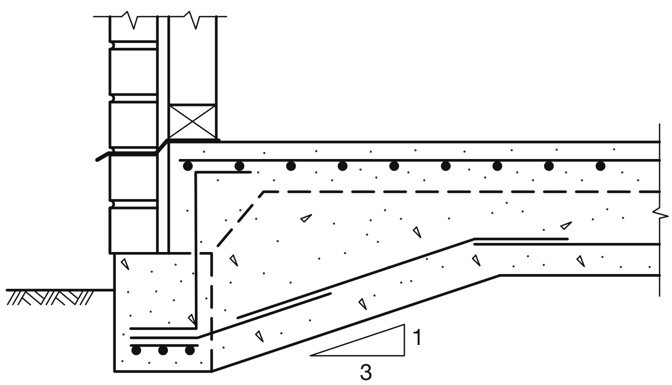


FIGURE C5.3 JUNCTION WHERE EDGE AND INTERNAL BEAMS ARE AT DIFFERENT LEVELS

Vapour barriers should consist of polyethylene sheet of 0.2 mm minimum thickness. An acceptable level of impact resistance has also been specified for practical construction purposes. Barriers with less impact resistance are likely to be excessively damaged during placement of the reinforcement and concrete.

The answer to most concerns about dampness is proper site drainage and appropriately selected slab levels and good quality concreting practices. In areas such as South Australia that are subject to extreme wetting and drying cycles and where high levels of salts are present (in the soil and building materials) moisture can also migrate by capillary action into the concrete slab and deposit salts at the surface during evaporation.

Continual salt deposits may cause ‘salt damp’ damage such as powdering and fretting of the concrete or masonry, together with deterioration of floor coverings due to damp and mould. The damp may also cause rotting of timber framing and corrosion of steel framing. The installation of a damp-proof membrane rather than a moisture vapour barrier under concrete slabs will provide a more effective barrier to moisture in these situations.

Where a damp-proof membrane is required it must form a continuous barrier under and around the whole slab and any damage occurring to the membrane during installation must be repaired by taping.

C5.3.4 Edge rebates

The purpose of the rebate is to allow drainage of the cavity and prevent water ingress into the building. Shallow rebates may be trowelled. A deeper rebate has to be formed.

C5.3.5 Recesses in slab panels

A deepening of the slab soffit is required at recesses to maintain the strength of the slab. The details shown in Figure 5.3 of the Standard apply to slabs with recesses located away from beams.

Recesses may be provided across beams, as long as the beam depth is increased and the beam reinforcement details are amended to maintain equivalent strength and stiffness.

C5.3.6 Heating cable and pipes

(No commentary)

C5.3.7 Shrinkage cracking control

The limitation of shrinkage cracks is a difficult problem in slabs, particularly when brittle floor coverings are to be used or if the slab is to act as a termite barrier. In most other cases, shrinkage cracking is not of concern.

The minimum requirements of AS 3600 are just satisfied by the meshes specified in Sections 3 and 4, but this level of reinforcement only provides nominal control of cracking. Such control is offered by the tying force exerted across the crack by the reinforcement, which keeps the sections of the slab on either side of the crack together and thereby stretches the uncracked section due to elastic and creep extension. Concrete can be expected to shrink by 600 to 800 microstrain for laboratory specimens. After allowing for time and shape effects, for a rectangular house 20 m long this implies a shortening of 12 mm to 16 mm. If the slab was fully restrained and unreinforced, the most likely result would be two cracks at 6 m to 7 m spacing, and of 6 mm to 8 mm width, which would be unacceptable. This situation does not occur because the actual slab shrinkage will be less than laboratory values and shrinkage forces actually shorten the overall slab length and stretch the slab panels between cracks.

Even so, cracks up to 1 mm wide may be expected. To reduce the crack widths to negligible proportions would require around 0.6% reinforcement (or more than SL81) and would add several hundred dollars or so to the construction cost, hence some cracking is accepted as part of normal slab performance.

Where crack-sensitive floor coverings are planned, two options are available: use heavier and more expensive reinforcement or delay the installation of floor covering for three to six months until most of the shrinkage has occurred. The use of flexible tile adhesives is very beneficial in increasing the tolerance of tiles to cracking in the underlying slab. Some evidence exists that increasing the mesh in critical areas is beneficial and this practice is encouraged in the Standard. The mesh operates more efficiently to control cracking if it is placed near the top surface and this location is also preferred in the Standard [see Clause 5.3.2(a)].

The Clause sets out some requirements that will moderate, but not eliminate shrinkage cracks.

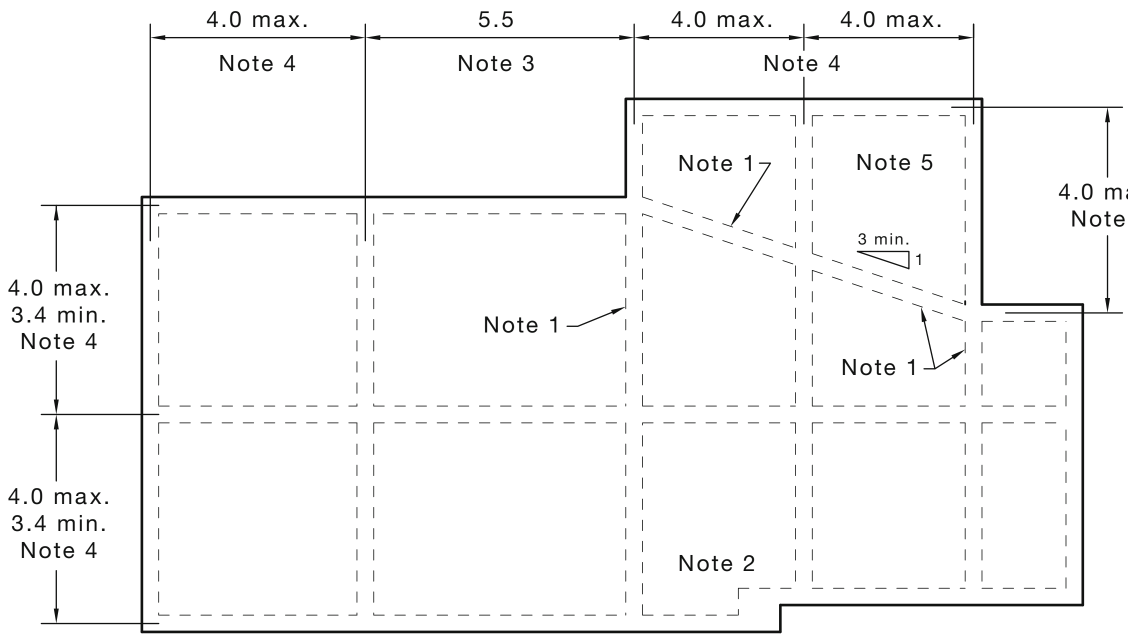
C5.3.8 Beam continuity in rafts

An important aspect of the structural design of rafts is the arrangement of the internal beams. The following points should be observed:

- Beams should generally be arranged in an orthogonal grid.
- The beams should be continuous in a straight line from edge to edge of the slab. This is more important than putting beams under walls. When beams cannot be placed in a straight line, a maximum deviation of 1 in 3 is allowed.
- For L-shaped and T-shaped buildings, the beams should be located to continue the edge beams at the internal corners. Considerable flexibility is allowed in the spacing of the beams. The Standard specifies the maximum spacing for beams and allows an increase in spacing when there are extra beams in the transverse direction (see Figure C5.4).
- The beam layout provisions for re-entrant corners do not apply to minor changes in plan such as doorways and protrusions of less than 1.5 m. Special details may be required to maintain beam continuity in these cases. Appropriate details are provided in Figure 5.4 of the Standard.
- When a raft is subjected to foundation movement, the ends and corners are particularly vulnerable areas of the raft structure. This vulnerability should be taken into account when laying out the stiffening beams of a raft. In the absence of engineering design, the spacing between the edge beam and first internal beam should not exceed 4.0 m (see Figure C5.4).

- Projecting elements of the house plan such as may occur with portico structures, require careful consideration to ensure that they are adequately supported by the raft structure.
- Where stiffening beams are at different levels, such as may occur in the provision of beams to support portico columns, or at a step in a raft slab, appropriate provision of reinforcement has to be made to provide for full continuity of strength and stiffness through the change in level.

When selecting the beam positions, it may also be necessary to take the location of plumbing into account.



NOTES:

1 The example in this Figure is based on a nominal 5.0 m beam spacing.

2 The re-entrant corner has both sides greater than 1.5 m long, and the internal beams are arranged to provide full continuity at the intersection.

3 The re-entrant corner is less than 1.5 m on one side, and continuity has been provided using one of the techniques from Clause 5.3.8.

4 The nominal beam spacing of 5.0 m has been increased by 10%, as the beam spacing in the opposite direction is more than 20% below 5.0 m.

5 The spacing between the beam and the edge beams and the first internal beam should not exceed 4.0 m.

6 The internal beam has been deflected to provide continuity at the re-entrant corners. The deflection is not greater than 1 in 3.

DIMENSIONS IN METRES

FIGURE C5.4 ARRANGEMENT OF STIFFENING BEAMS

Two-pour raft construction techniques may require special reinforcement details at construction joints. A variant of the two-pour raft construction technique was in common use in Queensland. In this variant, the perimeter edge beams are cast first and then the slab and internal beams are cast in a second pour.

This construction method creates particular difficulties in relation to beam continuity at re-entrant corners such as occurs in T-shaped and L-shaped building plans. Special reinforcement details and attention during construction are required at these intersections to ensure that full structural continuity of the beams is achieved. Attention may also be required at ordinary intersections between internal beams and edge beams, particularly where beams of deeper cross-section are used. Figure C5.3 illustrates these issues and shows one of many possible reinforcement-detailing solutions.

A related issue in the Queensland two-pour method is that, for various reasons, edge beams may be deeper than the internal stiffening beams. In re-entrant corners, this results in the situation where the beam depth changes at the continuation from edge beam to internal beam. Large and abrupt changes in stiffening beam strength and stiffness along the length of a beam are undesirable. In such cases, design modifications to lessen or transition the change in beam cross-section should be considered. See Figure C5.8 for an example of this situation.

In the cases of an irregular (i.e. non rectangular) building footprint, additional stiffening beams may be provided.

As a consequence of the 'cut out' of some of the mesh reinforcement, consideration may need to be given to adding top bars to the additional or supplementary beams shown below and/or crack control bars in accordance with Clause 5.3.7(b).

Some examples of how these departures from a simple rectangular footprint could be dealt with are given in Figure C5.5 (more than one strengthening option may be available).

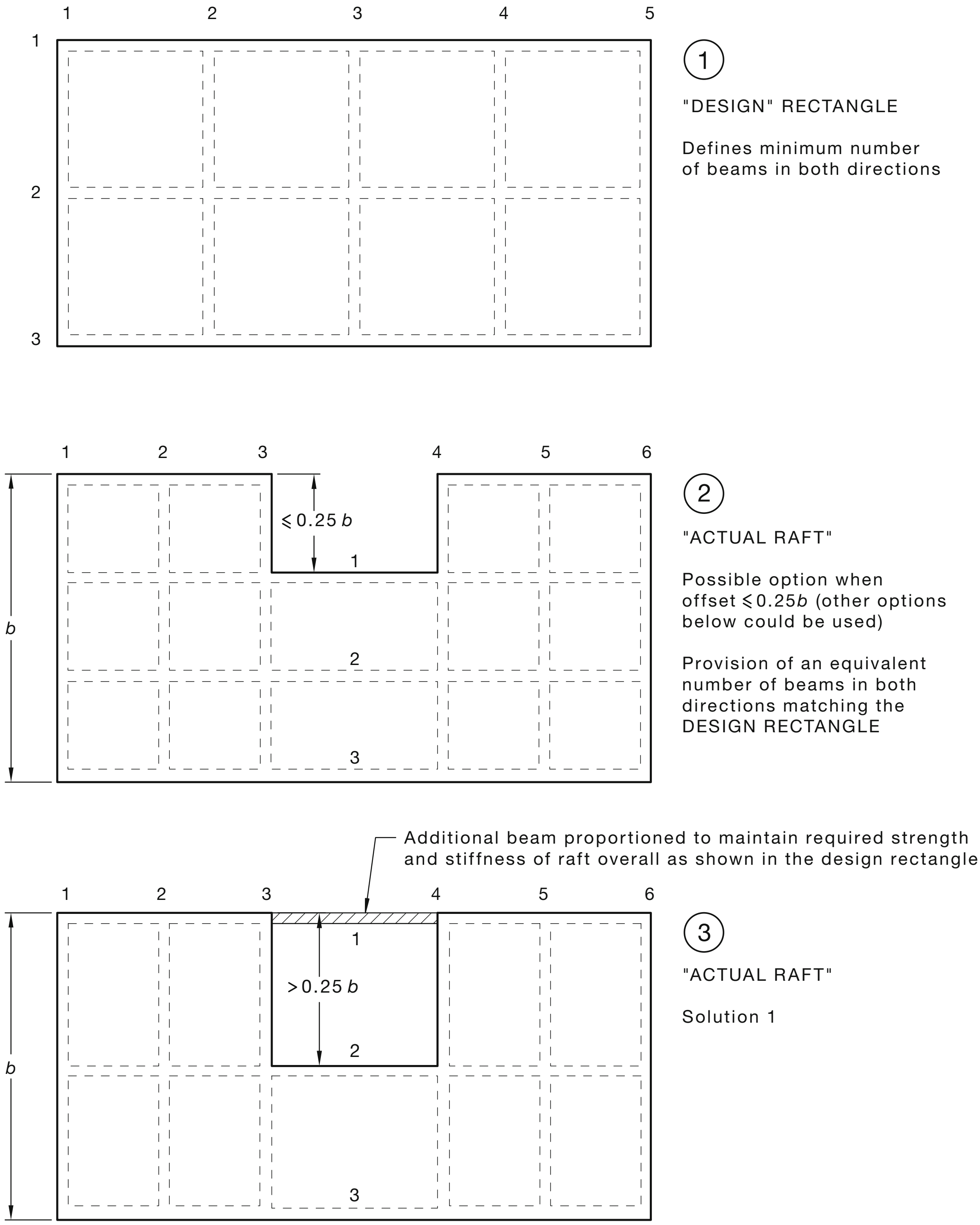


FIGURE C5.5 (in part) INTERNAL 'INSET' DESIGN

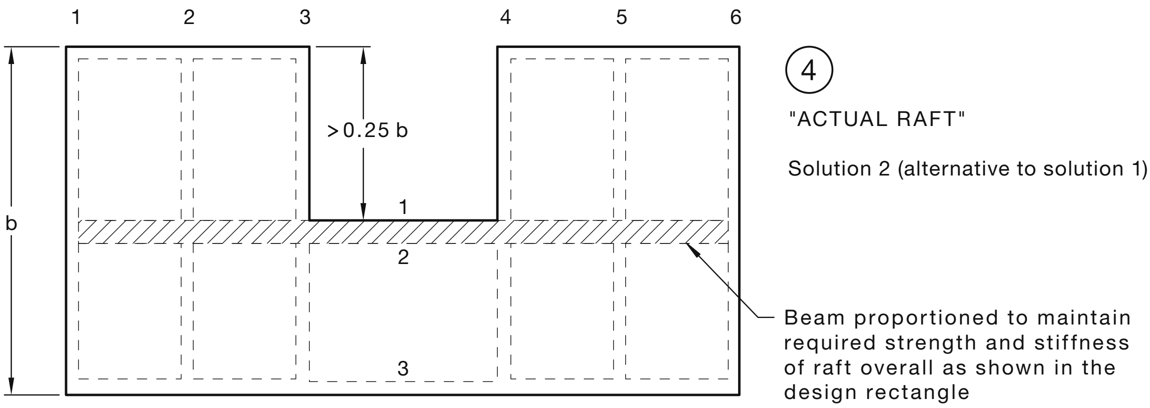
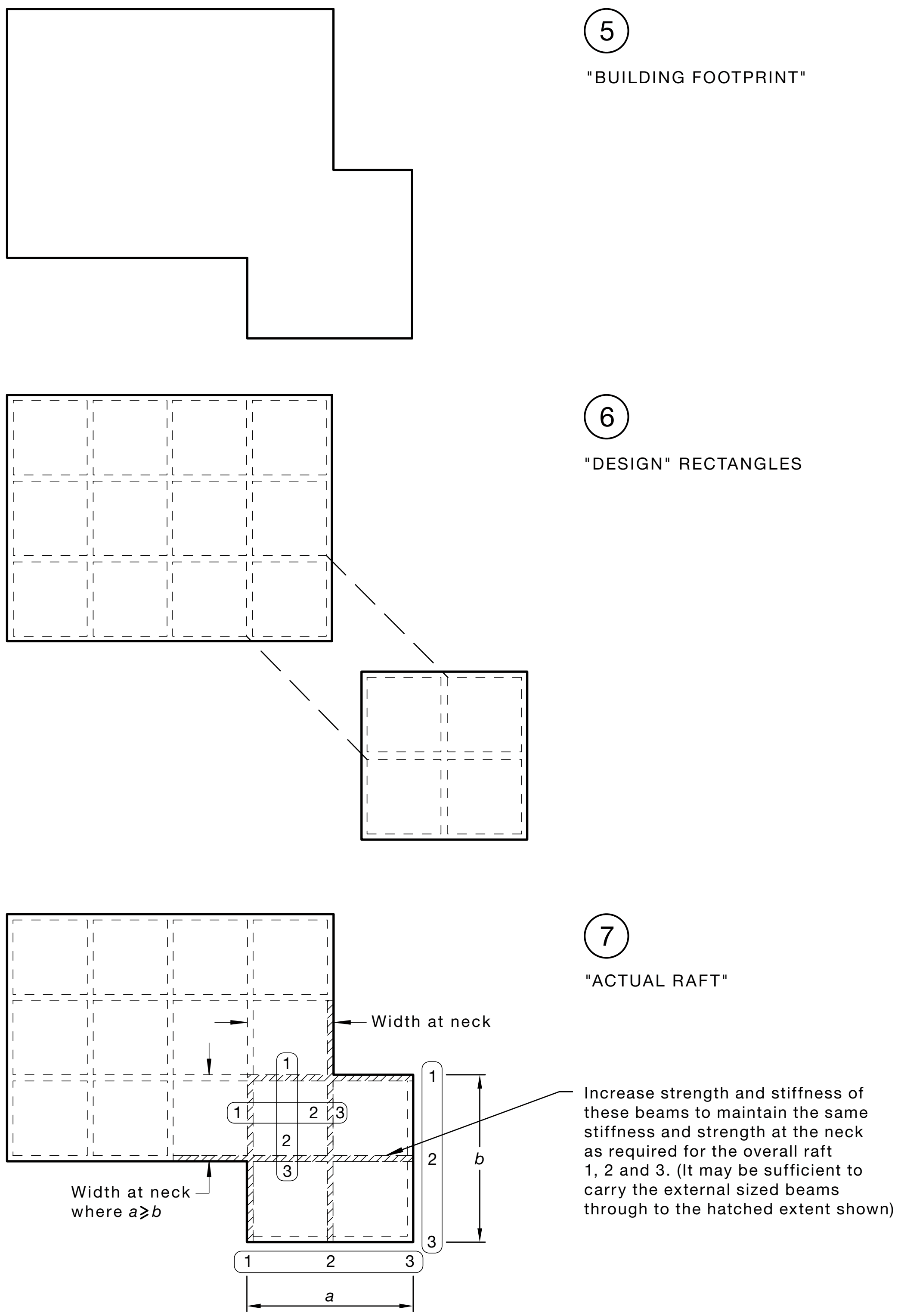


FIGURE C5.5 (in part) INTERNAL 'INSET' DESIGN



NOTE: Neck widths of less than 0.5*a* are undesirable due to the difficulty of achieving structural continuity. Instead of attempting to provide continuity through a narrow neck, consideration should be given to providing thorough and complete articulation of the footing systems and superstructure at the point of conjunction of the building subsections, or alternatively, consideration should be given to modifying the house plan so as to physically separate the building sub sections.

FIGURE C5.5 (in part) INTERNAL 'INSET' DESIGN

C5.3.9 Beam layout restrictions

(No commentary)

C5.4 REQUIREMENTS FOR PAD AND STRIP FOOTINGS

(No commentary)

C5.5 REQUIREMENTS IN AGGRESSIVE SOILS

C5.5.1 General

This Clause is new to the Standard.

C5.5.2 Isolation of concrete from the ground

The additional damp-proofing details focus on the edge and side of the slabs where discontinuities in the construction of the damp-proofing membrane can often occur. This occurs when some of the building work (e.g. paving and landscaping) is carried out by the owner rather than the original builder.

The additional details take account of the type of imperfections that may occur in the concreting and paving works and provides methods of achieving the BCA's performance requirements under these real world circumstances.

These additional details have been used generally in South Australia for many years in response to slab edge dampness and salt-damp risks associated with saline soils.

The mechanism by which dampness problems can arise is set out below.

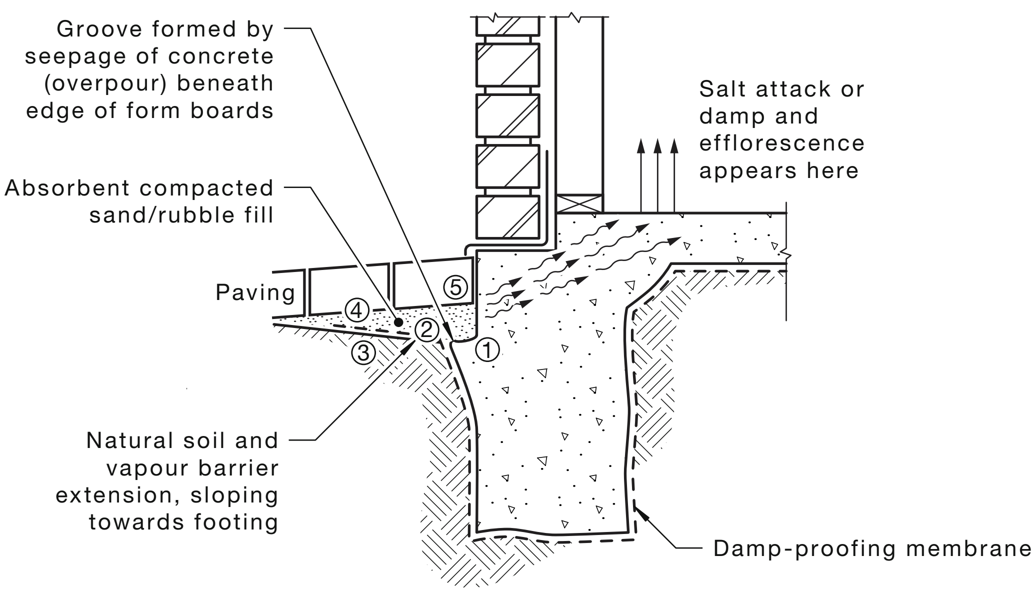


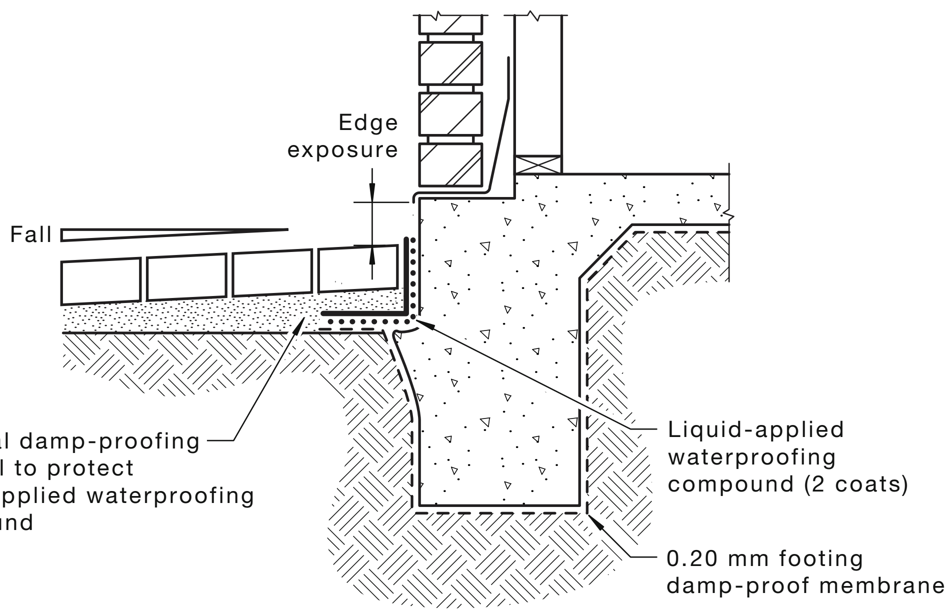
FIGURE C5.6 SLAB EDGE DAMPNESS MECHANISM

From Figure C5.6 it can be seen how the construction process can exacerbate the potential for future slab edge dampness:

- Irregular excavation edges or trench collapsing contribution to formation of concrete overpour under formwork at the edge.
- Damage to remaining plastic membrane during house building.
- Site-cut ground surface may slope towards house.
- Granular fill used for paving may allow subsurface ponding of water
- Damp-proof membrane may not be placed against side of footing or properly lapped with under-footing damp-proof membrane prior to placing paving fill.

The method of implementing the requirements of Clause 5.5.2 is as follows:

- The floor level bench level should be chosen to be high enough that the ground surface adjacent the house can be graded away.
- During concrete pour the edge of the concrete footing should be managed to minimize over-spill and groove formation under edge formwork.
- After concrete pour and during construction in accordance with Clause 5.5.2(b) and details shown in Figure C5.7:
    - Remove and 'tidy up' overspill (as early as possible).
    - Tidy up and repair plastic membrane after boards stripped and ensure following trades protect plastic membrane.
    - Tidy up and repair plastic membrane as part of final clean up to leave plastic ready for paving.
    - Construct paving with protective 0.5 mm thick damp-proofing material layer placed vertically against edge of slab and lapped over slab damp-proofing membrane as shown in Figure C5.7; or in accordance with Clause 5.5.2(c)).
- Alternatively after concrete pour and during construction in accordance with Clause 5.5.2(c) and details shown in Figure C5.7:
    - When formwork is stripped, apply a two coat (minimum dry film thickness of 0.35 mm) spray of a liquid applied waterproofing compound to the footing edge from top of slab level continuously down to the point of emergence of the under footing damp-proofing membrane. Ensure 75 mm overlap with the footing damp-proofing membrane. Ensure full coverage over all vertical edge concrete and any horizontal overpoured concrete.
    - Before spraying ensure—
        - if gutter depression longer than 2 m is left in concrete after boards are stripped, chip a drain outlet; and
        - thoroughly clean and blow away any loose material from around footing before spraying.
    - Protect the waterproof coating during construction.



Raft footing option

FIGURE C5.7 USE OF DAMP-PROOFING MEMBRANE FOR SLAB PROTECTION

C5.5.3 Concrete strength and detailing requirements

Classification of saline sites

The Standard does not envisage that the salinity requirements for site would be determined on a site by site or allotment by allotment basis.

The need for the implementation of measures to protect concrete footings from saline or aggressive soils is expected to be determined on a regional basis.

Salt attack on concrete and other porous building materials is a function of the salinity of the soils (in contact with the concrete footing) and the soil moisture state in combination with a particular range and combination of temperature and humidity. The adverse temperature and humidity conditions are most often associated with semi-arid and arid climates.

It is on this basis that it is expected that salinity protection requirements would be determined on a regional basis potentially by the Building Code of Australia or as part of the development approval process for new land divisions.

Exposure to saline soils

Salts exist not only in coastal regions, but are present throughout the landscape, including the drier inland areas of Australia. Sources of salinity include naturally occurring salts from marine sediments, salts released from the process of soil/rock weathering, salts transported from the ocean and deposited by rainfall, or use of recycled ‘grey’ water containing salts.

Problems with salinity are generally linked to the groundwater system, as water both dissolves and transports the salts through the soil. Saline groundwater can reach the footing system through rising groundwater table levels or by capillary suction of the soil, which may raise water by up to 2 m depending on the soil type (mainly clays).

The criterion for assessing soil salinity that is adopted in the standard is the electrical conductivity determined in the saturation extract test. In this test the soil is mixed with sufficient water to just bring it to a saturated state (described as glistening, verging on liquid consistency). Water is then drawn off the soil by vacuum assisted filtration or centrifugal filtration of the soil water mixture. The conductivity of the extracted soil water is then measured to give the saturation extract conductivity, (EC_e). A simpler and quicker test can be performed wherein a 1:5 soil:water dilution is prepared and then the conductivity of the supernatant solution is measured giving the EC(1:5). The EC(1:5) can be related to the EC_e by correlation; however, due to a variety of factors there is no simple or constant relationship between the two conductivity measures. The confounding factors include the differences in solubility of soil salts at the different dilutions rates involved in the different tests and the fact that in the EC(1:5) test some of the soil remains in suspension in the supernatant solution (Ref. 1).

While salinity levels are generally low enough not to have any effect on the concrete, some increase in strength and cover are required for more aggressive soils if the concrete is not protected by a damp-proofing membrane. As noted in Clause 5.5.1, for highly saline soils, it is recommended that both isolation of the concrete from the soil and increased strength and cover requirements be adopted to reduce the risk of damage.

Further information on concrete exposed to saline soils can be found in *Guide to Residential Slabs and Footings in Saline Environments* published by Cement Concrete and Aggregates Australia.

Exposure to sulfate soils

Naturally occurring sulfates of sodium, potassium, calcium or magnesium may also be found in the soil or dissolved in the groundwater.

The measure of sulfates in Table 5.2 (expressed as SO_4) is a simplistic and sometimes conservative approach to the definition of aggressivity. It is common to find more than one chemical in the service environment and the effect of these chemicals may be modified in the presence of others. For example, sulfate ions become aggressive at levels of 600 to 1000 ppm when combined with magnesium or ammonium ions. In the presence of chloride ions, however, attack by sulfate ions generally exhibits little disruptive expansion with the exception of conditions of wetting and extreme drying where crystallization can cause surface fretting of concrete.

The chemical concentrations (in ppm) relate only to the proportion of chemical present that is water soluble.

Where exposure classifications B1, B2, C1 or C2 are indicated in Table 5.2, it is recommended that the cement be Type SR.

Where exposure classifications B2, C1 or C2 are indicated in acid sulfate soil conditions, it is recommended that a protective coating be used on the concrete surface. If the pH level is below 3.5, specialist advice regarding suitable coatings and other protective measures should be obtained. If a protective coating is used for these exposure classifications, it may be possible to reduce the minimum required reinforcement cover to 50 mm.

Acidic ground conditions may be caused by dissolved 'aggressive' carbon dioxide, pure and very soft waters, organic and mineral acids and bacterial activity. Care is required in assessment of pH under ground structure and lifetime conditions since pH can change over the lifetime of the member. Therefore, the pH should not be assessed only on the basis of a present-day test result, rather the ground chemistry should be considered over the design life of the ground structure. Testing for pH should be carried out either in situ or immediately after sampling as there is otherwise a risk of oxidation with time, leading to apparent acidity, which does not correctly represent in situ conditions.

pH alone may be a misleading measure of aggressivity without a full analysis of causes (e.g. still versus running water).

Contamination by the tipping of mineral and domestic wastes or by spillage from mining, processing or manufacturing industries presents special durability risks due to the presence of certain aggressive acids, salts and solvents, which can either chemically attack concrete or lead to a corrosion risk. Certain ground conditions cannot be properly addressed by reference only to Tables 5.1 and 5.2. These conditions include, for example, areas where acid-sulfate soils exist, contamination by industrial and domestic waste, or spillage from mining, processing or manufacturing industries. In the absence of site-specific chemical information, the exposure condition should be assessed as ‘exposure classification B2’ for domestic refuse and ‘exposure classification C2’ for industrial/mining waste tips. Chemical analysis of the latter may, however, allow a lower risk classification.

Further information on concrete exposed to sulfate soils can be found in Commentary to AS 3600 Supp 1, *Concrete structures—Commentary (supplement to AS 3600—1994*).

C5.6 ADDITIONAL REQUIREMENTS FOR CLASSES M, H1, H2 AND E SITES

C5.6.1 Masonry detailing

Considerable care is required on the more reactive sites to minimize the risk of damage, through both careful detailing of the design of the building and thoughtful construction procedures. In particular, masonry should be articulated and susceptible masonry structures avoided. For example, masonry over doors and windows and in wing-walls and arches should either be avoided or detailed in accordance with TN61. Alternatively, masonry may be reinforced to control cracking.

C5.6.2 Variations in foundation material

Isolated outcrops of rock may simply be removed. Alternatively, the footing depth may be reduced and the footing stiffness maintained, despite a resulting reduction in section, by the use of substantially more reinforcement.

C5.6.3 Drainage requirements

Trenches for service piping may introduce water into the subsoil beneath a building. Backfills are usually highly permeable relative to the surrounding clay soils. Therefore, the surface of the backfill within the vicinity of the building should be ‘sealed’ to reduce moisture ingress. Additionally, the base of the trench should be sloped away from the building, to drain any water away.

Subsurface drains should be avoided near the footings, where practicable, as they can introduce water to the foundation if the drains become blocked; however it is recognized that such drains may be essential behind steps in slabs or for the relief of subsurface water flow. The base of the subsurface trench should be capable of providing some drainage away from the footings in the event of the main drain becoming blocked.

C5.6.4 Plumbing requirements

Drains that pass through footings are required to be wrapped with closed cell foam so as to allow movement between the pipe and the footing. Particular care and vigilance is required to ensure that the lagging is arranged to ensure that concrete, as it is poured, cannot creep around the ends of the lagged section and thereby form a close fitting collar around the pipe, which defeats the purpose of the lagging.

Plumbing and drainage under the slab should be avoided where possible.

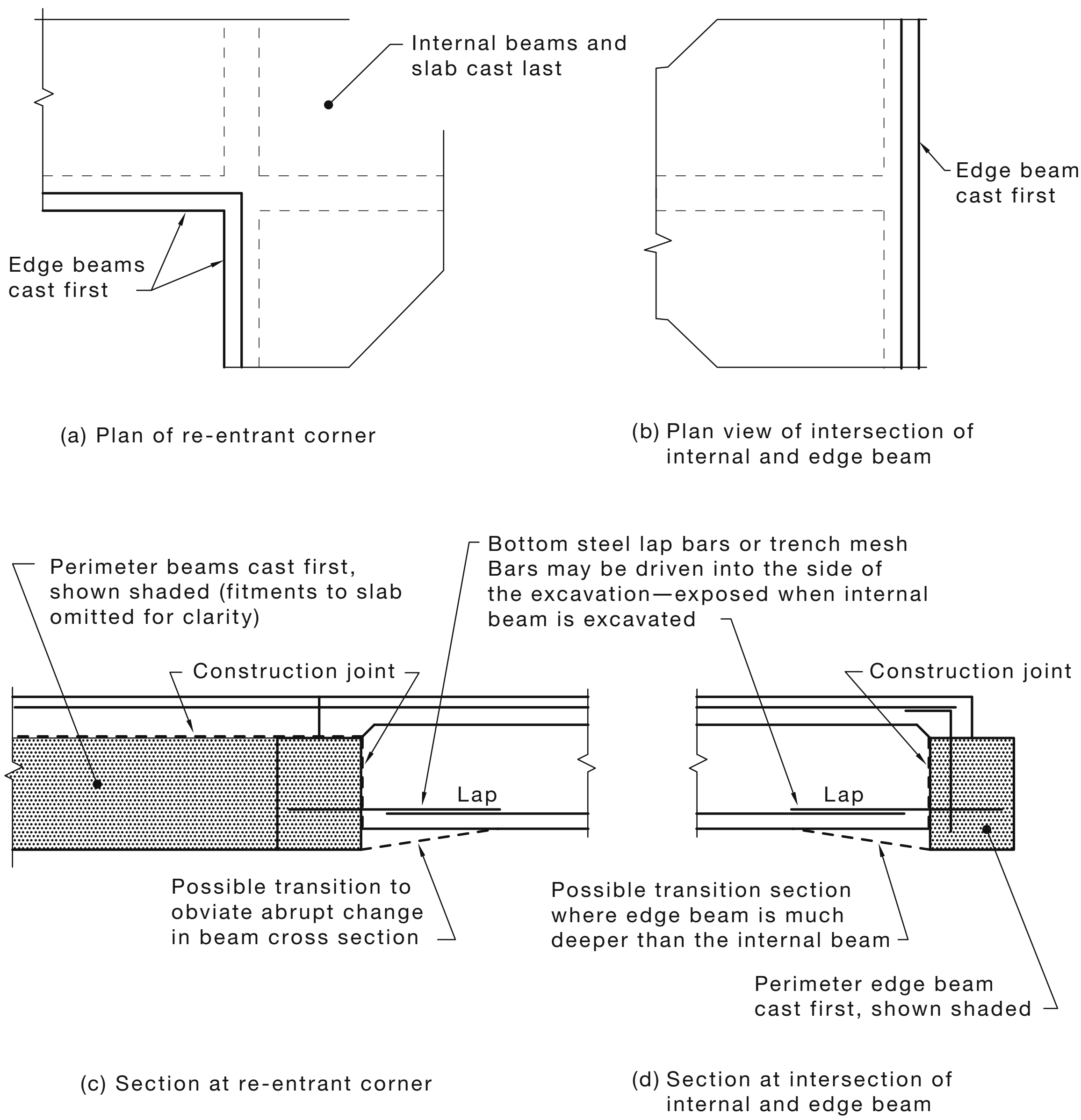


FIGURE C5.8 EXAMPLE OF INTERSECTION OF INTERNAL AND EDGE BEAM

REFERENCE

1 SHAW. Estimation of the electrical conductivity of saturation extracts from the electrical conductivity of 1:5 soil:water suspensions and various soil properties. Project report no. QO 94025, Dept of Primary Industries QLD.

SECTION C6 CONSTRUCTION REQUIREMENTS

C6.1 GENERAL

The construction requirements are set out in Clauses 6.2 to 6.5 with some extra requirements for Classes M, H1, H2 and E sites stated in Clause 6.6.

For durability AS 3600 requires 3 days initial moist curing. The Committee for slabs and footings carefully considered curing and decided that the only durability condition that would definitely require curing was moisture penetration from the edge of the slab. This seems to be a problem mainly encountered in Adelaide where some of the soils are high in salt. Under such circumstances, curing is required by Clause 6.4.7. Otherwise, normal building practice is expected.

Table 5.3 sets out the minimum strength and curing requirements for concrete for a range of exposure classifications.

Compaction by a vibrator is required for Classes H1, H2 and E sites.

Temporary service excavations can remove support from footings. This lack of support can result in settlement or rotation of the footing.

Excavation location and depth should be such that the excavation is not deeper than the critical depth line shown in Figure C6.1. The critical depth line may be lowered, if required, by lowering the founding level of the footing.

When trenches are to be excavated below the critical depth line, the critical backfill area should be backfilled with material of adequate strength and low permeability to minimize water migration and settlement. Concrete, mortar or (preferably) cement-stabilized soil may be used.

Propping may be required to the sides of excavations to ensure safe working conditions and to maintain the integrity of the foundations.

C6.2 PERMANENT EXCAVATIONS

(No commentary)

C6.3 TEMPORARY EXCAVATIONS

(No commentary)

C6.4 CONSTRUCTION OF SLABS

C6.4.1 General

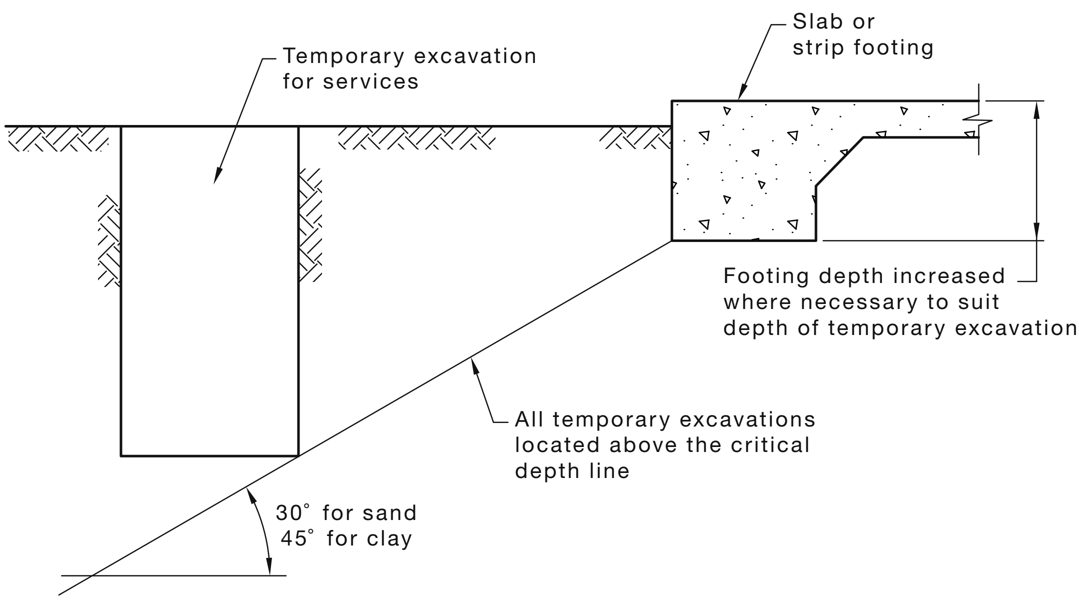
For Classes H1, H2 and E sites, the recommendations in Clause 6.6 are essential to ensure the satisfactory performance of the building and footing system and should not be overlooked.

No specific provisions are made about the placement of services beneath the slab, but experience and good practice have shown that—

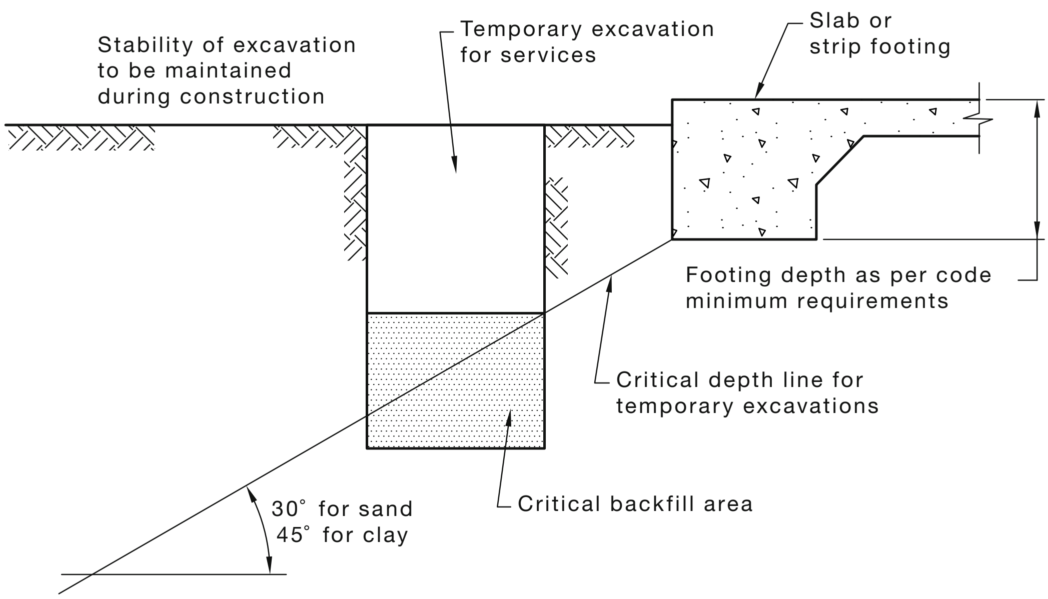
- services should be bedded on, and backfilled with, properly compacted material that is compatible with the natural material on site;
- services running parallel with edge or internal beams should not be positioned beneath these beams;

- beams through which services pass may need to be locally deepened and may require additional reinforcement [see also Clauses 5.3.2(e) and 5.4.2(e)]. The pipe or conduit should be wrapped with void-forming material; and
- services should not rise vertically through beams.

If risers are unavoidable, beams may need to be locally widened and may require additional reinforcement. The riser should be wrapped with void-forming material.



(a) Excavation above critical depth line



(b) Excavation below critical depth line

FIGURE C6.1 TEMPORARY EXCAVATIONS

C6.4.2 Filling

The method of compacting fill depends on the depth and type of fill.

Sandfill, up to a compacted depth of 600 mm, may be compacted by repeated rolling with the wheeled or tracked excavator being used on site. Such fill is termed rolled fill. This depth may be increased to 800 mm if the compaction is achieved by means of a vibrating plate or vibrating roller, and provided that the material is placed in layers having a depth of not more than 300 mm. The fill is then designated as controlled fill. For compacted depths greater than 800 mm, the sandfill should be subject to control and testing. Large depths of controlled sand, gravel and rocky fill have been used beneath buildings without problems.

Clean sands may be compacted by flooding but this method is rarely reliable and the final result should be checked by compaction tests. On reactive clay sites, swelling problems could be caused by the introduction of water to the foundation, therefore this method of compaction is not recommended for clay sites.

Generally clay fill should be avoided unless great care is taken. The permitted depths for clay fill are less, and the moisture content should be checked to ensure the clay fill is placed and compacted in a moist condition.

C6.4.3 Foundation for slabs

The preparation of a foundation for a slab involves attention to a variety of matters including the following:

- *Top soil* The usual statement for top soil 'containing significant organic matter' is made more specific by reference to grass roots. If the site includes shrubs and small trees, the soil containing their surface roots should be removed. On the other hand, it is not necessary to remove soil containing small amounts of root material.

- *Erosion* Erosion is generally only a problem for sandy soils and can be a serious design consideration near beachfronts or on filled sloping sites. On sites subject to erosion by wind or surface water, edge beams should be protected by one or more of the following methods:

  - Grading the ground surface to limit the catchment area adjacent to a building to less than 100 m^2.
  - Providing a drainage system that prevents run-off adjacent to the building.
  - Providing a 600 mm wide concrete path around the building.
  - Founding the edge beams at least 300 mm below the finished ground level.

- *Allowable bearing pressure* An allowable bearing pressure of only 50 kPa is required under slab panels and beams, with the exception of separate footings of footing slabs where the requirement is 100 kPa. Virtually, all natural soils should be able to provide 50 kPa.

  It is also permissible to found the internal panels and beams of slabs on fill in accordance with Clause 6.4.3(c)(ii). It is not necessary to excavate through the fill to support the internal beams. Where controlled fill is used, even the edge beams may be founded on fill in accordance with Clause 6.4.3(c)(iii); however, with shallow depths of fill it will be more often convenient to found the beams in natural soil.

- *Base slope of beams* The base of edge beams and footings may be sloped or stepped. The slope is restricted to 1 in 10 although lateral stability will often be provided by slab membrane action and beams across the slope.

- *Bedding sand* The layer of bedding sand is not a requirement but a construction convenience. On rough ground it does help to protect the membrane and on most sites it reduces wastage of concrete due to over-excavation. Bedding sand is required for aggressive soils to protect the membrane from puncturing [see Clauses 5.5.2. and 6.4.3(e)]. It is not recommended under edge beams on reactive sites.

C6.4.4 Treatment of sloping sites

Most sites include some slope and although it is convenient to illustrate the prescribed designs for flat sites, often modifications for sloping sites will be needed.

For moderate slopes, the edge beam may generally be deepened and a very deep edge rebate may be used. For steeper slopes, controlled fill past the edge of the slab may be useful. Footing slabs are particularly relevant for sloping sites, and with an appropriate retaining wall can accommodate significant differences in level. The compaction of the fill behind the wall needs to be carefully carried out or the wall may be damaged. Since a 100 mm thick slab can span up to a distance of 1 m, moderate compaction may be accepted for only the first metre inside the perimeter wall for a depth of fill up to 1 m. For depths of fill over 1 m, complete compaction is required and temporary propping of the wall during compaction may be necessary unless proved otherwise by engineering design.

For very steep sites, the slab may need steps to accommodate changes in level.

Many of the details such as steps and edge retaining walls have an influence on stiffened raft performance, and care should be taken on reactive sites. For example, the beams have to be structurally continuous through the step and, where retaining walls are introduced, the slab and footing should be tied together.

For steep slopes, the effect of cut and fill on the possibility of landslip should be considered.

C6.4.5 Retention of fill under slabs for Classes A, S and M sites

Some simple prescribed systems are given, but other engineered designs are feasible.

C6.4.6 Fixing of reinforcement and void formers

(No commentary)

C6.4.7 Placing, compaction and curing of concrete

(No commentary)

C6.5 CONSTRUCTION OF STRIP AND PAD FOOTINGS

Many of the details given in Commentary Clause C6.4.1 construction of rafts and slabs are also applicable to the construction of strip and pad footings.

C6.6 ADDITIONAL REQUIREMENTS FOR MODERATELY, HIGHLY AND EXTREMELY REACTIVE SITES

(No commentary)

APPENDIX CA
FUNCTIONS OF VARIOUS PARTIES
(No commentary)

APPENDIX CB

FOUNDATION PERFORMANCE AND MAINTENANCE

(No commentary)

APPENDIX CC

CLASSIFICATION OF DAMAGE DUE TO FOUNDATION MOVEMENTS

Appendix C describes a system of damage classification that is used in Clauses 1.3 and 2.2. The Appendix is also intended for use in the description of damaged buildings.

APPENDIX CD

SITE CLASSIFICATION BY SOIL PROFILE IDENTIFICATION—VICTORIA

The maps in Figures D1 and D2, Melbourne and environs and Victoria respectively, show the depth categories of design suction change (H_s) based on general climatic zones.

Where Table D1 gives a classification choice and the site to be classified is within 1 km (in the Melbourne map) of a more severe depth category, consideration should be given to using the higher classification choice. The higher classifications are generally associated with the more arid regions to the west of Melbourne.

Clay profiles derived from limestones, marls or highly calcareous sediments cause greater ground movements than indicated by the plasticity values. There are indications that in these profiles the depth of design suction change (H_s) is deeper than that stated in Table D1. This may be due to their open fabric causing a deeper water penetration and evaporation. Although the classifications in Table D1 are an attempt to consider this effect, it is advised that local knowledge and expert professional advice be sought.

The classification of the quaternary alluvials and tertiary sediments profile (Table D2) depends on the depth of silts or sands covering the clay and the type of clay. Where the covering depth exceeds of the depth (H_s), for that climatic zone, an ‘S’ classification may be used.

APPENDIX CE

STUMP PAD SIZES, BRACED STUMP UPLIFT HORIZONTAL LOAD CAPACITY

(No commentary)

APPENDIX CF

SOIL STRUCTURE INTERACTION ANALYSIS FOR STIFFENED RAFTS

Edge heave is usually a transitory phase that may occur before centre heave becomes established. The depth of moisture change leading to edge heave is likely to be similar to the depth of seasonal movement rather than the design depth of suction change H_s. The latter depth is usually greater than the depth of seasonal movement, particularly in semiarid regions.

In recognition of these differences, the formulae for edge distance (e) and mound exponent (m) depend on both y_m and H_s for the case of centre heave, but only on y_m in the case of edge heave.

Thus, in the case of centre heave, the form of the mound shape depends on climate, whereas in edge heave, the mound shape is dependent on y_m.

While the shape (given by either e or m) of the centre heave mound has been related to H_s and y_m, the shape of the edge heave mound has been assumed to be independent of H_s and, therefore, the prevailing climate.

APPENDIX CG
DEEP FOOTINGS
(No commentary)

APPENDIX CH

GUIDE TO DESIGN OF FOOTINGS FOR TREES

CH1 LIMITATIONS

(No commentary)

CH2 DEFINITIONS

(No commentary)

CH3 MAXIMUM DESIGN DRYING DEPTH (H_t)

(No commentary)

CH4 DESIGN PROCEDURE

Introduction

This Appendix is concerned with the design of new buildings for the potential drying effects of existing or proposed trees planted in the vicinity of the dwelling, concurrent with, or after, the construction of the dwelling.

As such, trees are expected to exacerbate centre heave deformations by contributing to soil drying (and hence shrinkage settlement) below the edges of buildings, but are not expected to impact adversely on edge heave deformations. Therefore only centre heave design needs to be modified to take account of 'new trees'.

Nevertheless, designers must be aware that trees removed prior to construction will provide an initially extreme soil moisture condition, outside the scope of AS 2870. As moisture is slowly regained beneath the new construction, swelling movements may be exacerbated in the vicinity of the removed trees, which, depending on the locations of the trees, could impact adversely on either or both centre and edge heave. To counteract this extra swelling, deep soaking may be conducted in boreholes across the site, prior to construction. Surface soaking is unlikely to provide significant benefit, except on very shallow soil profiles. Soaking may be required over a period of six to 12 months and so may not be practical on many sites. Ground movements and soil moisture (or suction) profiles have to be monitored to verify effectiveness of soaking for any benefit to apply in the footing design.

When a building is sited near large mature trees, the possibility of death of any of the trees should be considered and the subsequent rebound of the soil in the vicinity of the trees be taken into account in the design of the footing.

If trees exist on a site, which cannot be regarded as forming a group, then the influence of each tree should be designed for and due regard be given to the locations of the trees with respect to the plan of the building and the potential movement patterns across the whole site.

Design of buildings for tree drying of the soil

The Standard provides one approach to design of footings, which has proved to be effective in South Australia. The method is a botanically naive method, relating the potential for movement simply to the distance away from the trees, relative to the mature height of the tree. As a worst-case default value, the minimum distance of half the tree design height may be used as a conservative estimate.

The species of tree, leaf area, and site environment are all important factors that may impact on the potential for soil shrinkage settlement due to an active root system. Although these complexities have not been considered in the design method, the method has been successfully applied over the last 15 years in South Australia.

The assumption that the extent of roots is only related to tree height is a simplification. The extent of the root zone may be influenced by climate, because greater water availability in wetter climates should better satisfy tree water demands and reduce the extent of root distribution.

The wilting point suction concept is useful in realizing where tree roots must extend to in order to find water in the soil profile. Seasonably dry soil will often be too dry in the top metre or so to release any water to the vegetation throughout a year. Therefore, the roots must feed more deeply to survive, especially in a semi-arid or arid climate. Although the restriction of drying imposed by the concept of a wilting point is followed, a simplified approach has been adopted, which does not extend the depth of movement past 4 m for single trees and assumes that trees can affect design suction changes from the surface downwards. A more accurate reflection of tree-drying effects would be afforded by the 'observed' suction change distributions close to trees illustrated in Figure CH4.1. Further information on the method of assessment of tree-induced movement can be found in Cameron (Ref. 1) and Cameron and Beal (Ref. 2).

A further simplifying assumption is that mound shapes are only affected by an increase of the differential mound height (y_m). Accordingly, the edge distance is increased but not to the degree that might be expected. Offsetting this simple approach, tree influence is considered to occur on both sides of the building, whether or not trees exist on both sides.

Footing systems

Stiffened raft systems have been used successfully with this design approach. However, well-designed deep piling floor systems, with piling founded well below the depth of drying, can be extremely effective against soil shrinkage, although generally, these systems are a costly alternative when based on construction costs alone.

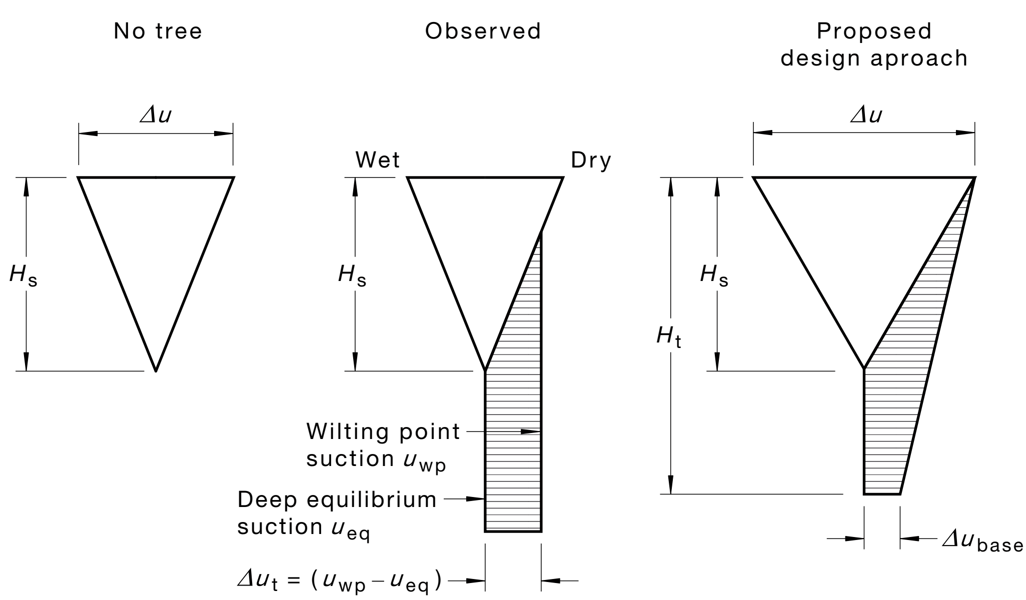


FIGURE CH1 OBSERVED SUCTION CHANGE DISTRIBUTION WITH DEPTH FOR TREE DRYING EFFECTS INCORPORATING THE WILTING POINT CONCEPT

Design examples

The following examples assume a deep soil profile ($H_s = 4$ m with uniform reactivity to moisture change throughout the soil profile and the only concern is for the design for trees for centre heave deformation (or edge drying). The three cases are illustrated in Figure CH4.2.

Example 1:

Site classification H-D, y_s	=	70 mm
$\therefore y_m$	=	49 mm (centre heave) and 35 mm (edge heave)
Single tree, D_t/HT	=	0.6
$y_{t\ max}$	=	25.1 mm
D_i/HT	=	1.0
$\therefore$ from Equation H4, y_t	=	$0.8\, y_{t\ max}$
$\therefore y_t$	=	20.05 mm
$y_{m\ tree}$	=	$(0.7\, y + y_t)$
	=	49.0 + 20.5
	=	69.1 mm

Design footing for:

$y_{m\ tree}$	=	69.1 mm (centre heave) and 35 mm (edge heave)

Example 2:

Using the same site classification and y_s value, but with a group of trees, having the same value of D_t/HT.

Group of trees, D_t/HT	=	0.6
$y_{t\ max}$	=	40.5 mm
D_i/HT	=	1.5
$\therefore$ from Equation H4, y_t	=	$0.9 y_{t\ max}$.
$\therefore y_t$	=	36.4 mm
$y_{m\ tree}$	=	$(0.7 y_s + y_t)$
	=	49 + 36.4
	=	85.4 mm

Design footing for:

$y_{m\ tree}$	=	85.4 mm (centre heave) and 35 mm (edge heave)

Example 3:

Using the same site classification and y_s value, but with a row of 4 trees or more, having the same value of D_t/HT.

Site classification H-D, y_s	=	70 mm
Single tree, D_t/HT	=	0.6

$y_{t\ max}$.	=	40.5 mm
D_i/HT	=	2.0
∴ from Equation H4, y_t	=	$0.933 y_{t\ max}$
∴ y_t	=	37.8 mm
$y_{m\ tree}$	=	$(0.7\ y + y_t)$
	=	49.0 + 37.8
	=	86.8 mm

Design footing for:

$y_{m\ tree}$	=	86.8 mm (centre heave) and 35 mm (edge heave)

The examples given above do not consider the case of tree removal. With removal of a tree, depending on the location and circumstance, the effect of the tree y_t may be additive to either the centre heave or edge heave design cases.

The impacts on design of the raft slab footings in these two selected cases using the Walsh method of analysis and assuming typical articulated masonry veneer construction are compared in Table CH4.1. Both internal and external beams are assumed to have the same depth (D) and the beam spacing is kept consistent between the analyses.

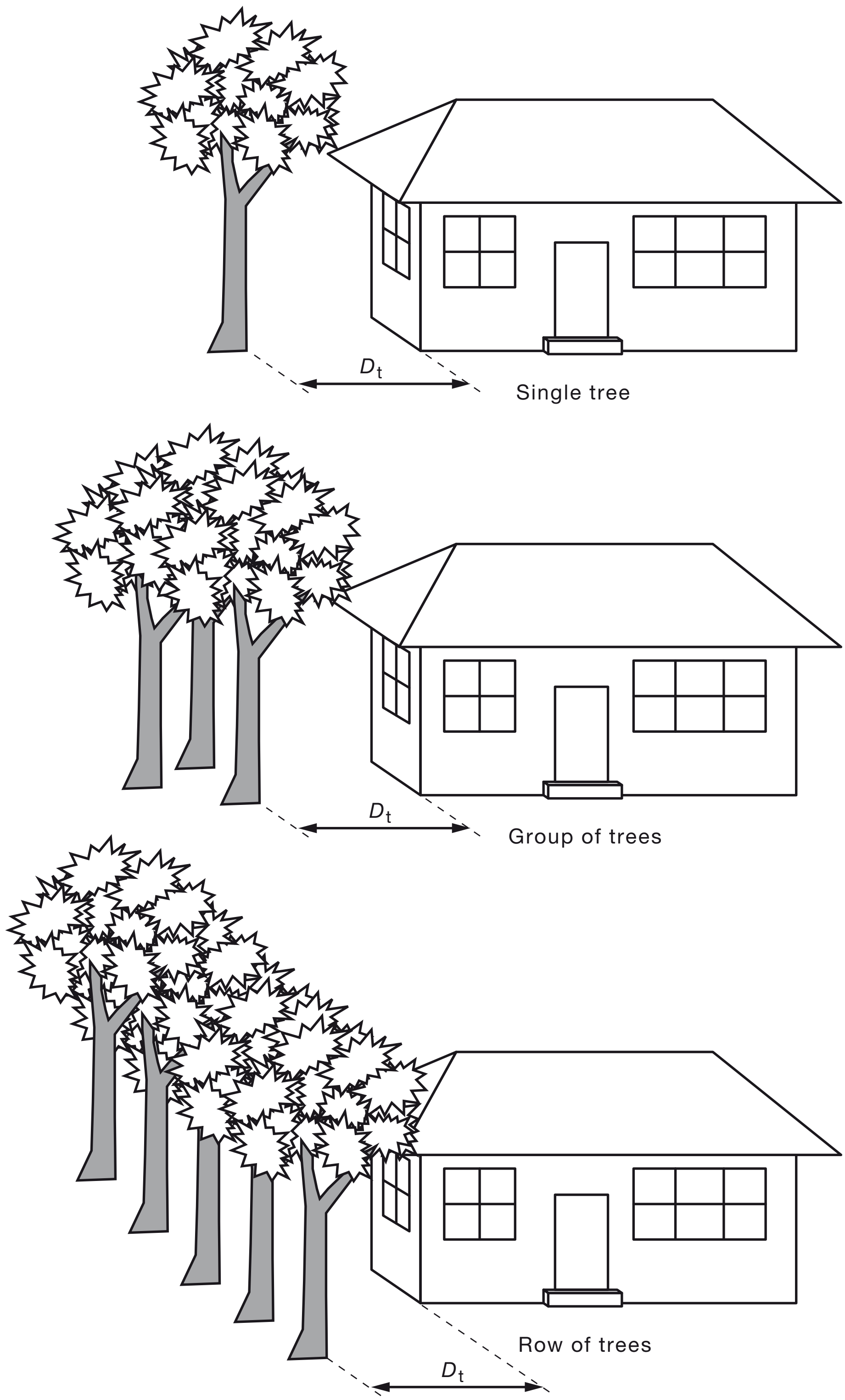


FIGURE CH2 ILLUSTRATION OF THE THREE DESIGN CASES FOR TREE DRYING

TABLE CH4.1
COMPARISON OF DESIGN REQUIREMENTS

Example	Beam depth (*D*) mm	Slab reinforcement	Top Reinforcement	Bottom reinforcement
CH1, CH2 and CH3 No tree	460	SL72	2N12	3N12
CH1 With tree	550	SL72	2N16	3N16
CH2 With group of trees	650	SL72	2N16	3N16
CH2 With row of 4 or more trees	650	SL72	N16	3N16

CH5 ALTERNATIVE DESIGN METHODS

Foundation and Footings Society of Victoria method

Introduction

The method presented has been developed by The Foundations and Footing Society (Vic) based on the input of arboriculturists, engineers and geologist in the Victorian residential building industry.

The method proposes a grading of trees with respect to the effect of their roots on nearby structures and suggests how their influence may be reduced. It is a simple method designed to approximate and plot the area that may be affected by tree(s) by investigators of building sites. Since the method is relatively new and the growth of tree roots and their effects on infrastructure are difficult to quantify, it is advisable that the parameters for this design method be used with care and supported by an inspection from an arboriculturist with particular expertise of tree roots growth and their interaction with infrastructure.

Practitioners are advised that the effect of trees be considered by all trees that could affect the structures, including those beyond the property boundaries. The mature height of trees is to be conservatively assessed by taking advice from expert arboriculturists and reliable references (see Refs. 3, 4, 5, 6, 7, 8 and 9). No advice is given within the canopy area of trees since the tree effects to this area are considered to be the most difficult to assess.

Wherever possible, common terms have been used.

Tree water use and flow

Arboriculturists have found that the water uptake by trees is mainly related to—

- tree species;
- tree health;
- stage of growth;
- total leaf area;
- height;
- root, trunk and branch mass;
- soil type;
- climate; and
- tree water suction capacity.

In their advanced growing stage, trees require more water and nutrients to construct their frame than to maintain their frame after maturity.

Structural roots are concentrated nearer the trunk of trees, whereas the fine, fibrous feeder roots extend well beyond the drip line (edge of canopy) and preferentially grow their roots along favourable soil moisture gradients or water paths such as leaky drains or soil aquifers. Water is also stored in the wood of the tree (roots and tree frame)

Climate effect

The availability of water has a major effect on the extent of tree root growth. Clayey soils have a 'reservoir' of water that can help trees to survive during dry period; however, in very dry arid climates some of this soil moisture is locked in the clay by the action of soil suction. At suction values of 4.2–4.5 pF (i.e. 'tree wilting point') clays can have up to 10%–15% water content that is not available to trees.

In wet climates, more soil water is available and therefore trees rarely need to extend their roots to any great distance from the trunk other than for their stability.

In temperate climates, there is sufficient water available other than in droughts. During drought times, they can cause considerable damage by extending their roots into new territory and particularly, if closer to structure, services and watered garden beds, close to buildings.

In dry climates, trees struggle unless close to a permanent water source. Those that survive usually extend their roots to greater depths, rather that greater distances. Clayey soils in dry climates are often highly fractured allowing tree roots to grow preferentially downwards along the fractures to soil that is more moist and has a lower suction value.

In arid climates, trees are scrubby and survive on very little water, either from scarce storms, mist or from water that condenses on the ground and around surface roots overnight.

Climate Zones 4 and 5 are considered similar to Zone 3 since the root growth is more likely to be downwards rather that outwards hence the engineer should consider a higher 'y_s' nearer the tree than stated in Figure CH3.

Pipe leaks and garden maintenance

Garden watering patterns or development of leaks will have a considerable effect on the behaviour of trees. The designing engineer has to consider these effects and discuss them with his client.

The establishment of a garden in dry, 'virgin', clayey allotments will change the soil moisture patterns and attract the root growth of trees. If building in clayey sites is carried out in drought conditions, subsequent foundation heave is more likely. The soil around leaky pipes will encourage root growth because of the availability of water and oxygen in the disturbed soil in the pipe excavations.

Engineering considerations

No design guidance is given for footings under the tree canopy (Area 1). The engineer should use engineering principles for designs in this area and in some cases may consider cantilevered floors.

The footings suggested for soil Area 2 are suspended systems supported by drilled concrete piers or timber or steel screw piles founded at minimum depth of $1.5H_s$ or rock at P1 and H_s or rock at P2. Excavated piers or deep continuous footings are not recommended in these areas. The engineer may also design a concrete slab footing in Area 2 by calculating a design 'y_t'. For simplicity, TD1and TD2 distances are considered equal.

In soil Area 3, the range of 'y_t' for design purpose is 1.5–1.0 y_t.

The design engineer should determine any suitable treatments at the change of footing systems. Wall articulation joints are recommended at these points.

Calculation method for tree effect distance (TED)

The method outlined in Figure CH3 and Tables CH5.1 and CH5.2 is used to calculate the extent of the land that is affected due to the presence of tree(s) or by possible soil rebound within a certain period after their removal. Suggestions are also made for design solutions within the (TED) other than for the area within the canopy of the tree(s).

Tree characteristics

Canopy density is assessed as—

- DENSE for canopy that shows little background light;
- MED. DENSE for canopy that shows approximately 50% background light; and
- SPARSE for canopy that shows a high degree of background light.

Since the total leaf area is most important, the shape as well as the size of the leaves also needs to be considered. Pinus radiata have numerous needle-like leaves, cypress pines have dense scaly leaves and Melaleucas have varying leaf shapes. In total, all of these trees have large leaf areas and canopy density.

Some trees have a small canopy area relative to their height and vice versa, hence their water uptake is not necessarily proportional to their height; however the taller trees need a greater soil suction capacity to draw water up to their canopy and therefore during dry periods they have the potential for greater water uptake than trees with lower suction potential. Other trees have seasonal dormant periods (e.g. deciduous) during which time they require less water, others (e.g. eucalypts) are capable of reducing their water needs during long dry periods, and yet others are capable of storing water during in their roots ball and trunk during wet seasons (e.g. Boab family). Melaleucas have the capacity to draw moisture in all seasons.

There are some exceptions to these simple rules. Some trees have a very large leaf area but not a dense canopy (e.g. Norfolk Island pines, Moreton Bay figs and London plane). These and similar trees should be given the highest canopy score (+3) since their total leaf area and water demand is large. Trees such as ‘pencil pines’ and Chilean Willows have dense but narrow canopies therefore the canopy score is (+2). Plants such as Palms (which is strictly a grass) can be tall with a relatively small canopy compared to their height but the larger palms have a very dense root ball and have a high water usage. Although they usually grow in sandy soils, they can survive in the more porous clays and may cause foundation problems.

Tree height

Tree height alone is not a good guide to the drying effect of trees in clay profiles. Taller and more massive trees require more water to maintain their trunk and branches, hence some trees with a large structure but sparse canopy may require as much water as trees with a smaller structure but denser and larger canopy. In the tree score, the height of tree(s) is considered but does not dominate the calculation of the TED. It is difficult to predict the mature height of trees, hence a conservative approach and/or an expert arboriculturist is required.

Stage of growth

This category scores trees with respect to their root growth. The rate of root growth is dependent on the life stage of the tree, its health, water need and availability of oxygen and nutrients. A healthy young tree will develop its roots much faster during its early growth than after maturity. Any large species at an immature stage of growth should be given a score of 2 for stage of growth.

Any large species at a high-growth stage should be given a score of 2.

Drought tolerant trees

Trees that have evolved in dry continents developed higher suction potentials, which allows them to continue to draw moisture even from relatively dry soils. Some trees have a type of dormancy during droughts allowing them to survive when there is very little water available. These trees are more likely to cause clay shrinkage than other trees even during droughts and their tree effect score for drought resistance is 2. Many of these trees are well known for their drought resistance; however, when in doubt, an expert arboriculturist should be consulted.

Ground and site conditions

Deep fill

Trees growing in filled ground often grow their roots preferentially along the soil layer interfaces and therefore spread their roots at a greater lateral distance than in most natural profiles. The soil interfaces have more oxygen and water and allow the roots to grow more easily. In these conditions the score is 2.

Adverse conditions

Adverse conditions, such as the following, encourage root growth preferentially towards buildings or greater lateral root growth than normal:

- Pavements or other covers between the tree and the building.
- A wet garden or leaky services near the building footings.
- Highly expansive clay over shallow rock.
- Disturbed ground or excavations near building footings.
- Vertical and lateral clay shrinking due to a moisture gradient from the foundation soil to the tree.

Other 'adverse conditions' may also be identified and advice sought.

Soil profile effect

Clayey soils have more constant water content than sandy soils. In dry conditions, tree roots will grow through the sand in search of a more constant water reservoir. In dense and highly reactive soils, roots grow in the fractures where there is more oxygen and water vapour. The high suction in the dry clay nearer the tree draws moisture well beyond the limit of the roots and creates moisture transference towards the tree. This often causes lateral and rotational movements of paths and non-integrated footings as well as settlement. In clay soils with a higher reactivity, a larger area of moisture gradient is created. Soils with a more open structure, such as top soils, layered clay fill or calcareous clays allow easier root growth than dense uncracked clays. In multi-layered soils, the roots will grow preferentially along the top of the most impervious layers where water flows or collects; for example, where fill overlies a more impervious layer, where sand overlies clay or where clay overlies rock. Soft and highly fractured rock also encourages deep root growth but with very little foundation effect.

Table CH5.3 shows mature heights and canopy diameters for some common Australian trees that can damage drains, structures and roads. Where the trees are not identified, an arboriculturist's report is recommended or the worst case values used.

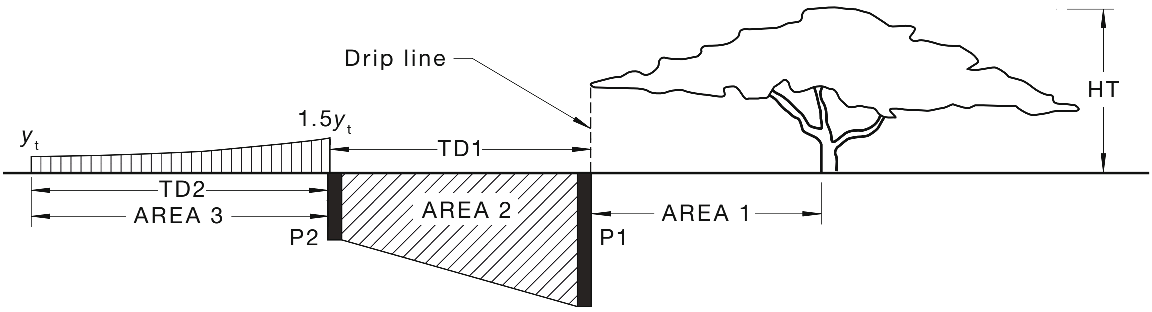


FIGURE CH3 TREE HEIGHTS AND SOIL SUCTION CHANGES

TABLE CH5.1
TREE EFFECT SCORE—SINGLE TREE

	Characteristic	**Option**	**Option score**	**Characteristic score**
Tree characteristics	Canopy	Dense	3	
		Med Dense	2	
		Sparse	1	
	Height	Tall = >15 m	3	
		Med = 8 – 15 m	2	
		Small = <8 m	1	
	Stage of growth	Growing	2	
		Mature	1	
	Drought resistance	Resistant	2	
		Not resistant	0	
Ground and site conditions	Depth of fill	≥1 m	2	
		<1 m	0	
	Adverse conditions	Yes	1–2	
		No	0	
	Soil profile reactivity	High/Extreme reactivity	2	
		Moderate reactivity	1	
Total tree effect score (sum characteristic scores above)				

TABLE CH5.2

TD1 AND TD2 VALUES FOR TREE EFFECT SCORES BY CLIMATE ZONE

Tree effect score	Tree effect	Climate zones		
		1	2	3/4/5
		Distances (TD1 or TD2), m		
<6	Low	1	2	3
6-9	Moderate	3	4	5
9-12	High	5	6	7
12-15	Very High	7	8	9
≥15	Extreme	9	10	11

NOTES:

1 P1 = minimum pile/pier depth = 1.5 H_s or bedrock, whichever is shallower.

P2 = minimum pile/pier depth = H_s or bedrock, whichever is shallower.

2 TD1 = TD2.

3 Table 2 only applies to predominantly clay sites.

4 For tree groups distances TD1 and TD2 should be increased by up to 50%.

5 Pier/pile footings in soil Area 2 may be extended to soil Area 3 at P2 depths. Alternatively, this part of the footing should be designed to an increased y_t.

TABLE CH5.3

COMMON TREE NAMES, MATURE HEIGHTS AND CANOPY SIZE

Common name	Mature height m	Mature canopy diameter, m
Norfolk Island Pine	30–60	15–30
Moreton Bay Fig	20–30	25–40
River/Swamp Sheoak	10–30	10–12
Drooping Sheoak	5–20	5–10
Silky Oak (Grevillea Robusta)	15–30	6–10
False Acacia, Black Locust	9–15	—
Willow/Ovens Wattle	6–10	3–5
Silver/Black Wattle	12–20	4–7
Cedars	Variable	—
Cypress/Radiata Pines	Variable	—
Red Ironbark	10–20	5–10
Lemon-scented Gum	Up to 15	Up to 8
Sugar Gum	15–30	8–15
Tasmanian Blue Gum	30–60	8–20

(*continued*)

TABLE CH5.3 (*continued*)

Common name	Mature height m	Mature canopy diameter, m
Spotted Gum	15–30	8–15
Manna Gum	9–60	6–20
Red-flowered Yellow Gum	5–8	5–8
River Red Gum	20–40	10–25
Yates	5–18	4–12
Karri	Up to 60	—
Lilly Pilly	10–20	5–15
Willow-Myrtle	8–15	5–15
Smooth Barked Apple Myrtle	10–30	6–15
Paperbark (including varieties)	4–8	3–8
Palm (including varieties)	Variable	Variable
Poplar (including varieties)	9–24	3–8
English Oak	Up to 20	10–20
Weeping Willow	9–15	6–12
Chilean Willow	9–15	2–6
Pepper Tree	6–15	—
English Elm	Up to 30	10–15
Strawberry Tree	≈10	—
Flame Tree	10–40	10–15
White Kurrajong	10–30	5–15
Kurrajong	6–20	3–6
Golden/Manna Ash	≈12	6–8
Hakea	3–8	3–6
Fruiting Figs	10–20	10–20
Prunus (inc. fruiting varieties)	3–9	2–6
Liquidambar(inc. varieties)	7–10	7–10
Pittosporums (inc. varieties)	4–14	2–8
Brush Box	≈18	—
Desert Ash	≈15	—
Claret Ash	15–20	8–10
Pyramid Tree	8–12	3–6
Plane Tree (inc. London variety)	12–24	8–20
English Elm (inc. golden variety)	Up to 30	12–16

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APPENDIX CI
BIBLIOGRAPHY
(No commentary)

NOTES

NOTES

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