AS 2870 Supplement 1—1996

Residential slabs and footings—Construction—Commentary

(Supplement to AS 2870—1996)

This Australian Standard was prepared by Committee BD/25, Residential Slabs and Footings. It was approved on behalf of the Council of Standards Australia on 12 April 1996 and published on 5 June 1996.

The following interests are represented on Committee BD/25:

The Association of Consulting Engineers, Australia

Australian Building Codes Board

Australian Chamber of Commerce and Industry

Australian Geomechanics Society

Australian Institute of Building Surveyors

Building Management Authority of W.A.

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This Standard was issued in draft form for comment as DR 94401.

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Residential slabs and footings—Construction—Commentary

(Supplement to AS 2870—1996)

PUBLISHED BY STANDARDS AUSTRALIA
(STANDARDS ASSOCIATION OF AUSTRALIA)
1 THE CRESCENT, HOMEBUSH, NSW 2140

ISBN 0 7337 0561 8

PREFACE

This Standard was prepared by the Standards Australia Committee BD/25 on Residential Slabs and Footings and contains explanations and additional background material that will assist in the interpretation of AS 2870, *Residential slabs and footings—Construction*.

The layout of this Commentary is identical to 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 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.

It is acknowledged that Koukourou and Partners (Adelaide) have made a significant contribution to the Waffle Raft (WR) technique.

@Seismicisolation

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Originated as AS 2870.1 Supplement 1—1989.
Revised and redesignated AS 2870 Supplement 1—1996.

STANDARDS AUSTRALIA

Australian Standard

Residential slabs and footings

Construction—Commentary

(Supplement to AS 2870—1996)

I N T R O D U C T I O N

PURPOSE

The Standard provides for simple standard methods for the design of residential footings based on sound structural and geotechnical principles. It applies to a variety of footing systems for most foundation conditions including reactive soils—a very common foundation in Australia. 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:

(a) **Design for swelling and shrinkage movements** The primary cause of foundation failure in 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 upon changes in moisture content. The reactivity of a soil depends upon the size of clay particles, their mineral composition and the proportion of clay in the soil. The reactivity of clay soils cannot be clearly evaluated by tests. In particular, the usual engineering index properties (i.e. liquid and plastic limits and linear shrinkage) on their own may not be reliable.

The 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, and include the following:

(i) Seasonal and long term climate changes, including dry summers, floods and droughts.

(ii) Influence of the house, garden and drainage; in particular trees which cause severe drying and covering the ground.

(iii) Long term effects of the whole urban infrastructure, including paving and drainage.

(iv) 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 will be quite complex. The form could 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.

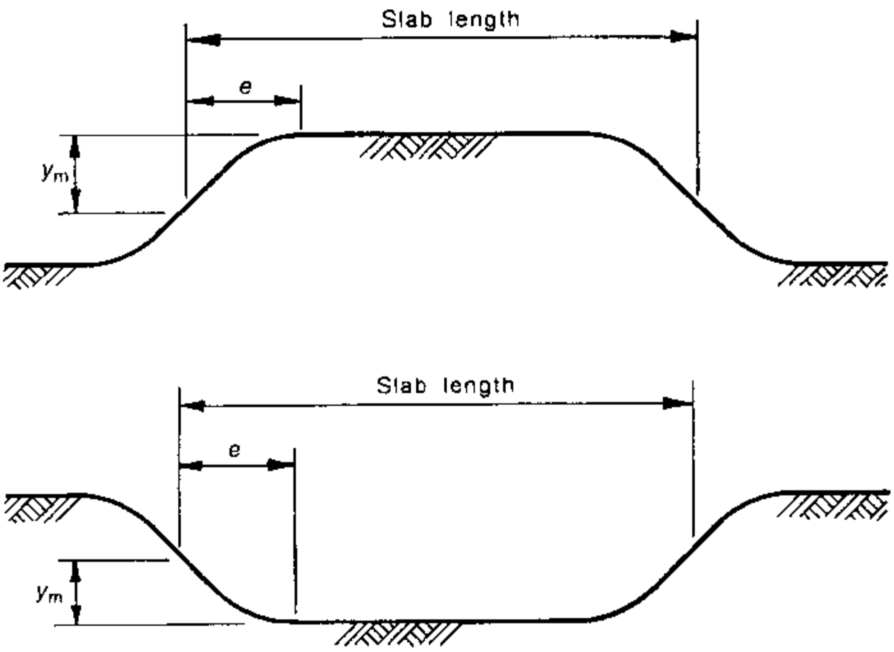

FIGURE C1 IDEALIZED GROUND MOVEMENT PATTERNS FOR FOOTINGS AND SLABS ON CLAYS (Walsh shapes)

The design of a slab to cope with ground movements relies on 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 controls the differential movement as a result of interaction of the foundation and structure. This interaction utilizes the weight 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, further controlling the movement.

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.

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 can be founded at depths where moisture changes should be less, in some cases (particularly where failures can occur by soil swelling) deep strip footings can 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 is preferred.

An alternative design philosophy is the removal and replacement of the clay or the covering of it with a suitable non-reactive material.

(b) **Design for settlement of compressible soils or fill** Uneven settlement can occur on filled or soft alluvial sites. The 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 to firm material in some circumstances. A stable foundation can be provided by properly compacted fill 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 is used to support the usually more heavily loaded external walls. These 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 can be determined by the assumption of uniform support. Alternatively, a more refined analysis may be carried out by 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.

(c) **Design for sensitivity of superstructure** Whether a house can tolerate movement without damage depends upon the type of construction and the various design details such as articulated walling.

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, the Building Authority, 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.

SECTION C1 SCOPE AND GENERAL

C1.1 SCOPE The Standard applies to site classification and footing system design for houses and the like, 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 contraction 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 that 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 be 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 merely outlines the procedures to be followed in using the Standard, but 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 failure are modest compared with the costs of conservative design. Moreover, if the designs in the Standard are followed, failures should be very rare. Expectations of performance of footing systems on reactive sites depend upon the adopted standard 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 normal 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. bricks expand, timber and plasterboard shrink. Consequently some cracking in houses 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 houses in Australia are constructed on clays that move with changes of soil moisture conditions arising from the imposition of the house on the ground. 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 Clause 1.4 and a CSIRO Information Sheet is available for distribution to homeowner's, entitled 'Guide to Home Owners on Foundation Maintenance and Footing Performance'. This pamphlet can be obtained from the Publications Officer of the Division of Building Construction and Engineering, P.O. Box 56, Highett, Victoria 3190. It is suggested that a copy be given to the new home owner 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 Authority will be interested in ensuring that the Information Sheet is disseminated. Eventually, it is anticipated that site maintenance will become part of an owner’s accepted responsibilities.

C1.4 DESIGN CONDITIONS

C1.4.1 General Design of footings to AS 2870 takes into account those environmental conditions arising from a normal site, maintained in accordance with Appendix B and CSIRO Note 10-91. These conditions are expected to cover most situations encountered in a normally maintained house site and the designs do include some provision for conditions slightly divergent from ideal. However, abnormal sites may arise as a consequence of either unusual previous land use or inadequate site maintenance. Special engineering consideration is needed for such sites, and so these sites are classified usually as Class P.

Principles for footing designs on sites with trees have not been adopted in this Standard for the following reasons:

(a) Soil drying patterns around trees of different species in a given climate and in the urban environment is not fully understood, leading to an unacceptable level of risk in the design.

(b) Assumptions on the empirical formulation of conventional soil mound shapes are not valid when trees affect ground movement.

(c) Adoption of tree design rules could be likely to lead to all houses being designed for severe tree effects, thereby wasting national resources. As the Standard is written, homeowner’s should be made aware of the limitations of the footing design. If they object to these limitations, they may then request from the builder or engineer a design more suited to their needs. Communication is essential to this strategy and CSIRO Note 10-91 is part of this approach. In short, only those clients with specific needs will require stiffer footing systems.

 If tree design is required for a raft a single side mound is appropriate and a specific estimate of the mound parameters should take into account the suction profile resulting from tree drying. One approach has been give by Walsh (1995) (Ref. 2).

 An alternative strategy is to isolate the footing system from the effects of trees (see Walsh, 1995) (Ref. 2).

 Similar approaches could be taken when it is expected that other major deviations from normal conditions may occur.

(d) Zero lot line developments make observance of some of the provisions of the maintenance more problematic but the technical requirements still apply. No simple guidance is currently available.

C1.4.2 Load effects The design (factored) load for both strength (safety against yield) and serviceability (deflection and crack control) is the same. Moreover the strength design load 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 (1985) (Ref. 1). These load factors are also consistent with the performance requirements given in Clause 1.3.1.

C1.7 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 load-bearing 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, viz. 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.

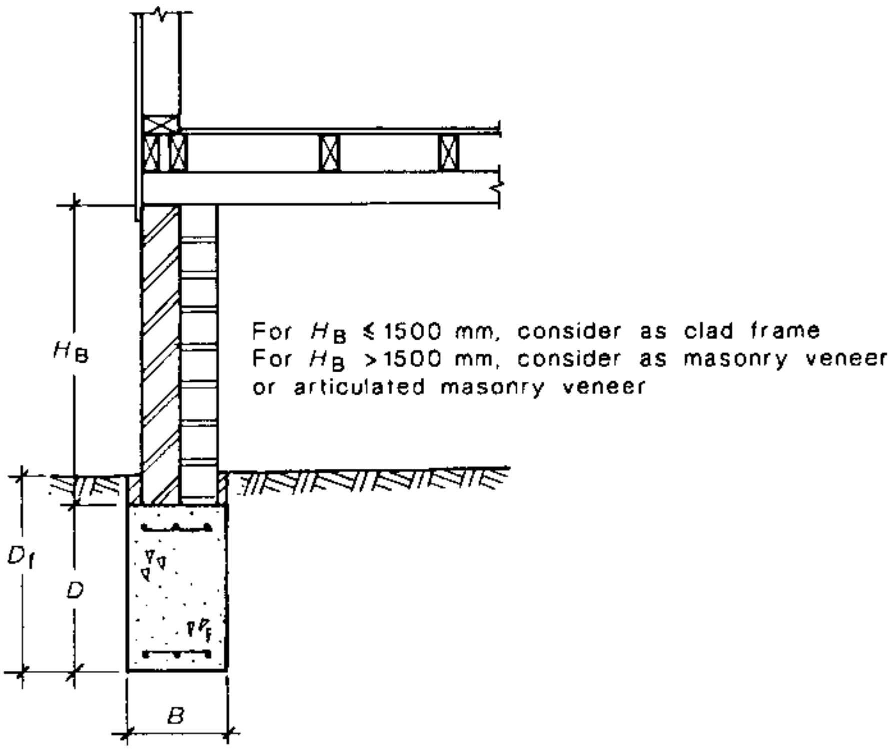

FIGURE C1.1 STRIP FOOTING SYSTEMS, CLAD FRAME

C1.9 REINFORCEMENT DESIGNATION

C1.9.4 High strength steel This Clause has been introduced to account for 500 MPa yield strength deformed wire fabrics. These fabrics were introduced with a reduction in steel area to the 450 MPa plain wire fabrics. They have been available in South East Queensland for over 10 years and were recently introduced nationally. Research was undertaken by a number of groups to verify that performance of standard designs was not diminished by the higher strength steels.

The Australian Standards for reinforcement are currently being revised with a proposal to adopt an international standard of 500 MPa characteristic yield strength for both reinforcing bar and deformed wire fabric.

Until the reinforcement standards are finalised either the standard plain wire fabrics or the proportionally reduced area 500 MPa fabrics can be used.

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 All sites have to be classified. The footing system must be suitable for the site and the only method of achieving this is to assess the site and to classify it.

The main types of soil are sands and clays, with silts as an intermediate type.

The various types of soil are distinguished in an engineering assessment by the size of the particles which constitute the soil such as—

(a) *sands*—which comprise material down to 0.075 mm;

(b) *silts*—which include the range 0.075 mm to 0.002 mm; and

(c) *clays*—which consist 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 that these simplified classifications are different from normal engineering classifications.

A general summary of the site classification system is given in Table C2.1 below. More detailed methods are given in the Standard. Class P sites have generally 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.

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 the soil can be estimated from penetrometers 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 are adequate guides. For these reasons engineering tests to assess allowable bearing capacity should not be required. The reactivity of the site is usually the most important aspect of the classification and is discussed below. More specific discussion for each State is to be found in the subsequent Sections.

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. (Part rock foundation should receive special attention.)
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 is material into which a 50 mm survey peg can be driven easily. 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 moulded in its natural moist state by strong pressure in the fingers and can be penetrated 50 mm by the thumb with moderate effort.

Generally the first source of information about the site conditions should be the Building Authority. In some areas, where there has been no history of troubles 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 Authority 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 will be needed. 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. If movements are quantified, slight could be taken to mean an upper limit of 20 mm of total settlement or reactive movement.

For a reactive clay site the classification is M, H or E. Although numbers for movement are attached to these classes in Clause 2.2, these values should not be over-emphasized. Of equal importance, although less definite, is classification by existing house 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 which 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 simply requires a '-D' to be attached to the primary classification to indicate that H_s is greater than three metres. The absence of '-D' would indicate that movements are relatively shallow. So 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.

As indicated in Figure 2.1, 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.

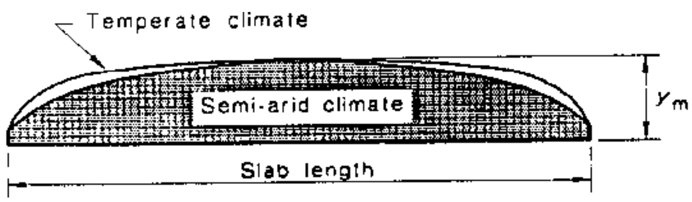

FIGURE C2.1 THE EFFECT OF CLIMATE ON MOUND SHAPE

The classification of a site on which controlled fill has been placed depends on —

(a) the nature of the fill (e.g. clay or sand);

(b) the depth of fill; and

(c) the nature of the underlying natural ground.

Thus controlled fill sites can be of any classification ranging from Class A for sand-fill on a sand site to Class P for fill over very soft compressible clay. Clay fills on clay are usually reactive and may be Class S, M, H or 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.

A number of underslab termiticide irrigation systems are now available. These are intended to allow the periodic re-treatment of the underslab soil with liquid (currently water based) termiticide.

The means of distributing the termiticide vary between the systems. Currently proposed application rates are 5 to 10 litres/m^2 of liquid averaged over the treated area. (This is equivalent to 5 mm to 10 mm height of water over the treated area.)

The uniformity of distribution of the termiticide will vary depending on the efficacy of the irrigation system and the nature and configuration of the underslab soils. The arrangement and nature of underslab bedding sand may also have a significant influence.

Much of the water component of the termiticide is likely to be eventually absorbed into the underlying soil profile. This can be expected to cause swelling and heave of reactive clay foundation soils.

The excess moisture and the soil movements may then dissipate over a period of several years.

The currently proposed re-treatment intervals are 5 to 10 years. The effects of treatment may entirely or largely dissipate before succeeding re-treatments.

The amount of heave caused by a given volume of water will depend on the characteristics of the soil. When absorbed by an uncracked reactive clay soil profile, 10 mm of water could be expected to cause not more than 10 mm of surface heave. The heave in a cracked reactive clay profile should be less. However, the normal wetting up that commonly occurs under a slab after construction may cause closing of the cracks. Consequently, at succeeding termiticide re-treatments, the soil profile may behave as an uncracked profile.

Non-uniformities in distribution of the termiticide can be expected to result in differential heave. The differential movement may need to be considered in relation to y_m rather than y_s because the differential movement is generally within the confines of the building.

Allowance for additional differential movement caused by these irrigation systems, may in some cases, require the adoption of higher site classification or other allowance in the footing design.

It is desirable that house owners and operators of the systems should be aware of the potential for the 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 considered that the data and experience with the systems are not sufficient to allow specific guidance to be incorporated in the Standard.

C2.2 METHODS FOR SITE CLASSIFICATION To identify accurately the reactivity of a clay site by means of tests on samples throughout the soil profile is too complex and expensive to be used routinely on individual house sites. Therefore other methods need to be used and in the Standard three procedures are offered as follows:

(a) Prior performance.

(b) Profile identification.

(c) Movement estimates.

The simplest method of 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 houses on light strip footings. An inspection of the neighbourhood should indicate whether such a category is likely for the area. Such an inspection would only be meaningful if some knowledge of the soil conditions were available. Thus at least, either a soil or geological map should be consulted to ensure that the neighbourhood has a similar soil profile to that of the proposed house 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 houses, this method is not applicable. Strengthened footings could be expected for the past 30 years or more in Adelaide but only for 10 years in Melbourne. In N.S.W., the Department of Housing has been using strengthened footings since the 1970s.

The method relies on assessment of damage (cracking) of houses 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 houses 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 that 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 free surface movement, y_s. This is the vertical movement range expected during the life of the house 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 house and ‘reasonable’ as the level that could be expected for 19 houses out of every 20. This does not mean that the house is not expected to last more than 50 years nor that 1 in 20 houses 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
H	= Highly reactive	40 mm $< y_s <$ 70 mm
E	= Extremely reactive	$y_s > 70$ mm

In this assessment, y_s should be interpreted as the characteristic value that has a 5 percent chance of being exceeded in the life of the house which may be taken as 50 years. Calculation of y_s shall assume that 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 movement.

Linear shrinkage, plastic 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 the surface movement may be estimated.

Some areas such as Sydney and Melbourne can be classified without tests by identifying soil profiles, the behaviour 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. In any case the Tables in Appendix D of the Standard provides a ready guide to the expected level of site classification. These methods of soil profile identification are as follows:

(a) **Classification of Victorian clay sites** In Melbourne and the surrounding district, it is recommended that the classification be based on both the prevailing climate (refer Figures D1 and D2 in Appendix D) and the soil profile, i.e. the texture, feel and colour of the layers of soil if a hole is dug with a spade, backhoe or simple auger. Some guidance can be found in geological maps, which can be purchased from the Department of Mines. The 1:63 360 series is recommended. The accuracy of these maps near geological boundaries may not be sufficient to forecast soil types in areas on the scale of residential allotments. In such cases, the most reactive soil profile in the vicinity of the allotment should be assumed, unless proven otherwise.

   Classification is generally simple. For all clays in the Melbourne area, the theoretical depth of movement varies between 1.5 and 2.3 metres, depending upon the climatic zone. Therefore the ground movements are relatively shallow-seated and the subclassification, -D, does not apply. Generally speaking, the highest classification level will be Class H, which may be applied to the Tertiary basaltic clays near Berwick and Flinders, together with the extensive area of Quaternary basaltic clays to the north and west, depending upon the thickness of the clay layer at the site. The Class E category is unusual for a Melbourne basaltic clay as is Class H for other residual clays.

   Outside the Melbourne area, deep-seated movements (>3 m) are possible in regions where more semi-arid conditions apply, for example, Horsham, Mildura and Wodonga. It is essential that the soil profile is identified (or tested) throughout the projected depth of movement. Some guidance is given for a restricted number of areas but in most locations, unless there is well established local experience, a consultant will need to be engaged for the classification.

   In summary, site classification in Melbourne is normally a simple process and no testing is needed. Typically—

   (i) basaltic clays are Class H;

   (ii) other clays are Class M;

   (iii) deep sands (>1 m) are Class A; and

   (iv) shallow (<0.6 m) clay sites can drop down to Class M for basaltic clays and Class S for other clays.

(b) **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, references 1, 2 and 3. 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 it 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 design depth of movement four metres, the sub-classification '-D' applies to all site classifications.

(c) **Classification of N.S.W. clay sites** It is expected that some Building Authorities in the Sydney area will give the classification or a range of classifications within their municipality. This information may be obtained from a specially prepared consultant's report or the Council may request the information with subdivision documents. This type of zoning greatly increases the efficiency of the building process and assists potential home owners to obtain realistic quotes for the footing system needed on their site.

Even if the Building Authority has no specific information, the classification of a clay site in and around Sydney can be a simple process and may be carried out by an informed builder.

An extensive testing program has been carried out for the N.S.W. Builders Licensing Board in the major development regions of Sydney and, arising from this study, a classification system for the County of Cumberland was developed. This is presented in the publication 'Classification of N.S.W. Soils for Housing' (Ref 4). The site classifications discussed below are drawn from this document.

Many sites in Sydney are in sandstone or sand areas and would be classified as Class A or S.

Most of the soils in the western part 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 is reduced to Class S. Some sites in this group should be classified as Class H where unusually severe moisture changes can be expected, e.g. where the natural drainage is altered in a major way or in fringe areas of landslides where major water content changes can occur. Many of these sites are also prone to landslip and therefore must 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 can have clay deposits of various thicknesses and the classification varies accordingly. Deep clay layers could potentially be affected by changing groundwater regimes arising from either extreme drought or by urbanization. Therefore where depths exceed 2.5 m, a Class H classification is conservatively given, otherwise the classification is Class M. If test data are used to check the classification, the theoretical depth of suction change must be increased to two metres 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.

Inland of the Dividing Range a more severe climate can be expected leading to greater depths of ground movement, and the sites should be classified on the basis of well-established past performance as described in the Standard or by a specific engineering investigation. In Albury, mainly because of the depth of influence, Class H-D sites are relatively common.

(d) **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 ground water 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 which 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.

(e) **Classification of Queensland clay sites** In Queensland a wide variety of soils and climates can be encountered. Unfortunately, the reactivity of clay profiles in Queensland has not been the subject of extensive research and little specific guidance on site classification can be offered. Generally on clay sites, it will be necessary to engage a qualified engineer to classify the site. In some cases this classification need not be undertaken on a site-by-site basis but for an entire subdivision.

The classification should be based on engineering principles if 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.

There are areas of ‘black earths’ which consist of very reactive clays, such as Toowoomba and the Darling Downs in the south-east of Queensland. Such clays are well known for their large movement and would warrant Class H or E classification. Since climate changes are pronounced with distance from the coastline, careful consideration must be given to the potential depth of movement. For example, this depth increases from 1.5 to 2.3 metres between Brisbane and Ipswich.

In summary, unless there is well-established local knowledge about the behaviour of the clay sites, the site classification will require some engineering input.

(f) **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.

C2.2.3 Estimation of the characteristic surface movement Estimation of the characteristic surface movement, y_s, for classification requires the soil suction model proposed by Aitchison (1973) (Ref. 5). 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:

(a) *Soil suction* Soil suction is a measure of the internal stress caused by moisture in unsaturated clays. It is useful in the analysis of reactive clays because it is more strongly a function of the climate and vegetation than it is of soil type.

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 can 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 = suction, in pF units = $\log_{10}$ (suction in kPa) +1.01

Suctions can 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 (AS 1289.2.2.1).

The design suction profiles for the estimation of y_s should be related to local experimental data to give the characteristic wet and dry profiles. It needs to be emphasized that such data must be relevant to the definition of y_s, e.g. data from an open field site subjected to seasonal moisture changes will not be applicable, nor will a suction profile taken adjacent to either a tree or a leaking pipe.

The characteristic value is defined as the value that has a 95 percent chance of occurring in the life of the structure. Thus it is not necessary to consider 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 distribution can lead to overestimates of movement if the depth is estimated as the extreme value for suction change.

(b) **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, 7.1.2 and 7.1.3). A description of these tests is given in Cameron (1989) (Ref. 6).

Guidance on estimation of instability index from shrinkage index is given in Appendix F of the Standard.

(c) **Design movement** The design surface movement (y_s), for site classification is obtained by summing the movement for each layer as follows:

$$y_s = \frac{1}{100} \int_0^{H_s} I_{pt} \, \Delta u \Delta dh \qquad \ldots \text{C2}$$

where

y_s = design surface movement

I_{pt} = instability index

Δu = suction change at depth (h) from the surface, expressed in pF units

Δh = thickness of soil layer under consideration, in metres.

Having determined the surface characteristic movement, 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 (1989) (Ref. 6).

C2.3 SITE INVESTIGATION REQUIREMENTS The physical requirements of site investigation relating to the frequency and depth of subsoil exploration are given in this clause. The number of boreholes per site is dependent upon 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 1.7.26), 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 adequately determine the site classification. Gilgai structures are well known in suburban Adelaide.

C2.3.5 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 pF 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 can 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:

(a) 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 shall 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.

(b) Soft silt or clay means a fine-grained soil which can easily be penetrated 25 mm in its natural condition by the thumb. Soft silt or clay shall 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.

C2.4 ADDITIONAL CONSIDERATIONS FOR SITE CLASSIFICATION

C2.4.3 Natural sand sites underlain by clay An upper profile of sand (or non-reactive silt) can greatly reduce the amount of movement of the profile. Firstly, it is the upper layer that is subject to the greater moisture change and if non-reactive, the potential movement will be less. Secondly, sands and some silts in the upper layer protect the underlying soil from significant moisture changes thereby further reducing potential movements.

If the clay occurs at a shallower depth than specified, an assessment of the clay is necessary. The depths specified **do not** allow for the possibility of some cutting of the site reducing the depth of the sand layer.

C2.4.4 Class P Sites Certain sites may become Class P sites for the reasons listed in the Clause.

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 norma. 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 old 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 full. 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.

In Item (e) 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.

C2.4.5 Effect of site works on classification In Clause 2.4.5, the Standard sets limits to the amount of cut or fill which can be made to a site before reclassification is necessary. Some examples of the effect of cut or fill on classification are —

(a) 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);

(b) reactive movements worsened by the addition of clay fill; and

(c) settlements caused by the weight of fill where weaker material underlies the site.

C2.4.6 *Effect of fill on classification* Specification of compacted fill which is deemed to be controlled, without formal testing, is given in Section 6. Maximum thickness are also given. Where mechanical compaction is used, the amount of compacted fill which is allowed is up to 0.8 m of sand (using a vibrating roller or plate) or 400 mm of material other than sand (using a mechanical roller).

Consideration has 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, S, M, H or E to the site. In assigning such a classification, the classifier must assume all shrinkage cracks are effectively closed, so that lateral strains are limited and swelling and shrinking movements will take place vertically.

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 will 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.

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6 CAMERON, D.A. *Tests for Reactivity and Prediction of Ground Movement*. Civ. Engg. Trans., I.E.Aust., Vol. CE31, No.3, December 1989, pp 121–132.

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SECTION 3 STANDARD DESIGNS

C3.1 SELECTION OF FOOTING SYSTEMS The choice in footing systems is most commonly 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 heavy structural columns, suspended concrete floors or highly brittle features would not be included.

The background to the design of slabs and footings involves the following:

(a) **Design details and construction** All footing systems designed under Clause 3.1 must comply with Sections 5 and 6, which include design details and construction requirements.

(b) **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 takes two pours to construct but it has a number of advantages; it adapts to sloping sites better than a slab-on-ground, 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, e.g. footing slabs are dominant in Queensland and Western Australia, slab-on-ground in South Australia, Victoria and Tasmania, while both slab types are common in New South Wales.

(c) **Structural proportions of slabs** A considerable part of the design process for slabs in Class A and Class S sites relies on past satisfactory performance.

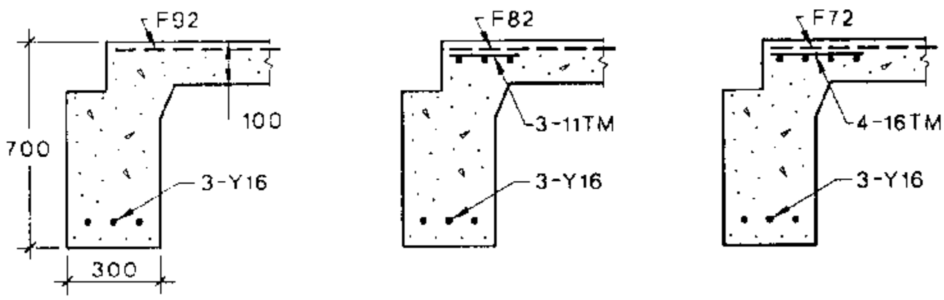
Most of the structural strength of slabs for Class A and Class S sites is provided by the concrete. The reinforcement is included to control shrinkage and to provide some flexural strength in the case of cracking due to foundation movement. For the 300 mm deep edge beams, the flexural reinforcement F8TM 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 is 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 of the adjoining slab panels is only available for the tied form of construction. Under footing slabs with separate strip footings, it is required that 100 kPa be the allowable bearing pressure and that the width be as specified for individual strip footings.

(d) **Slab thickness** For slab on ground construction 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 fabric 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 load-bearing framed walls.

(e) **Slab reinforcement** The reinforcement for slabs-on-ground is specified as F72 but for footing slabs without tied beams only F62 is required. For this form of footing slab construction the slab is subject to less restraint at the edges, so there is a lower shrinkage fabric requirement. Nonetheless, these levels of reinforcement are very low—F72 represents 0.2% and F62 represents 0.16%. For slabs longer than 18 m the cracking problems are potentially greater and fabric sizes F82 and F72 are required. Extra reinforcement or other measures may also be required where brittle floor coverings are used (see Clause 5.3.7). The fabric 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

(f) **Stiffened rafts** A variety of systems is available for placing a concrete floor on a reactive clay. Of the systems, the stiffened raft, shown in Figure C3.2, is recommended.

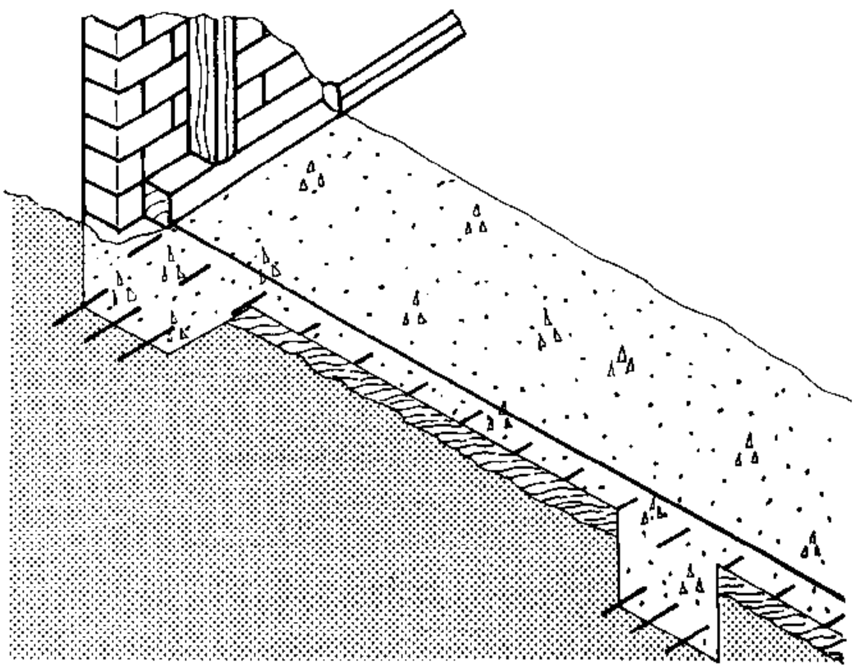


FIGURE C3.2 STIFFENED RAFT CONSTRUCTION

Stiffened rafts are constructed in Queensland and northern N.S.W. 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.

The designs prescribed in the Standard were developed in part from an assessment of the performance of 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 houses on moderately and highly reactive sites in Melbourne. This information was used to check the accuracy of the model based on engineering principles (Ref. 3). 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 since there is a grid of internal beams. If a beam is within 1000 mm centre-to-centre of a wall, 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 other direction is reduced the spacing 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 Adelaide experience on the vulnerability of such wall construction to cracking, and the sizes needed to achieve satisfactory performance.

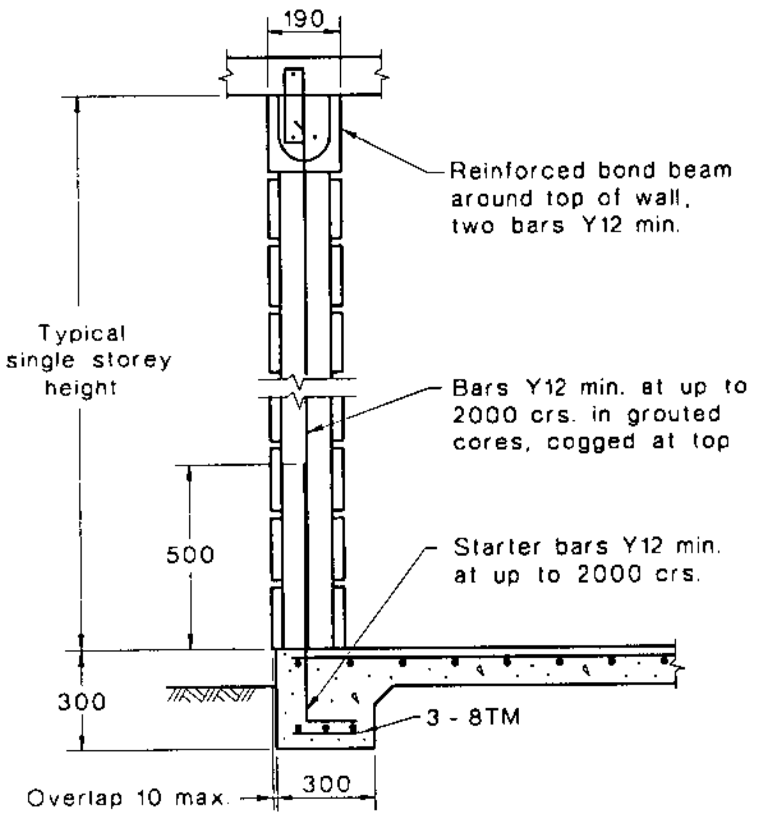
For the highly reactive sites, much stronger designs are given. The ‘standard case’ of masonry veneer has edge beams 300 mm × 500 mm with 3 wires of F12TM. This is stronger than the intermediate slab which has often been successfully used in Melbourne on such sites, but has been adopted to allow for the wider variety of sites that may be covered by this category of highly reactive sites.

No standard designs are given for full masonry on Class H sites as it is recommended that such construction should be either avoided or subject to an engineering design.

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, Note 11 permits a reduction in these beam sizes to 300 mm × 300 mm with 3-8TM reinforcement, if reinforced hollow concrete blockwork walls are structurally connected to the beams and act with them to resist movement.

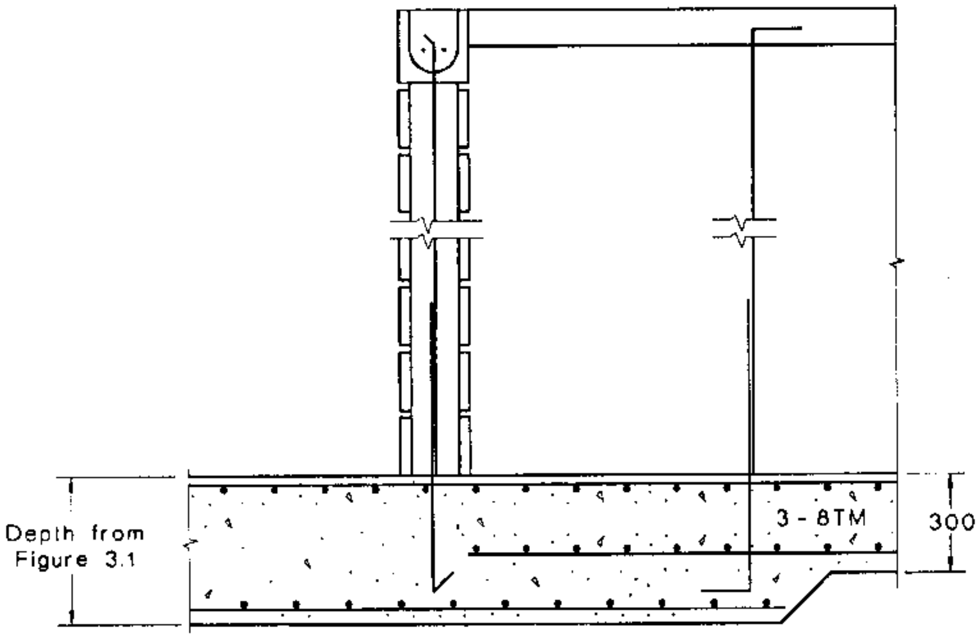
In this case the walls must be 190 mm single leaf hollow concrete blockwork, reinforced with at least Y12 bars at not more than 2.0 m centres, tied into the footings with starter bars and incorporate a continuous bond beam with at least two Y12 bars around the top of the wall. (See Figure C3.3.) The 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.

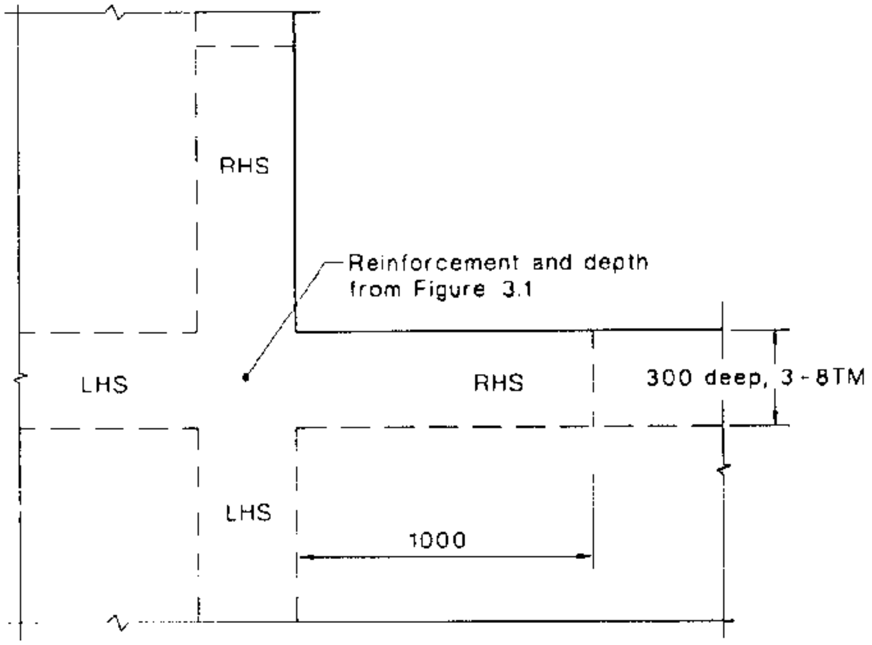


NOTE: Waterproofing is required to exterior face walls constructed and reinforced to AS 3700. Footings are suitable for openings up to 1800 mm. For wider openings use established concrete and reinforced concrete masonry analysis methods to determine the required footing sizes.

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

(g) **Waffle rafts** Waffle rafts are a particular form of raft slab where the raft, including ribs, is constructed on a flat ground surface. The ribs are formed with void formers. The structural design method in Appendix F is also suitable for waffle rafts. Although the bearing area is less than that of a raft, an allowable bearing pressure of 50 kPa under the ribs is adequate.

The designs given in this Section are based on an engineering analysis using the same principles as for stiffened rafts. The construction is completely on ground and this has several features:

(i) The shrinkage behaviour is improved due to the lower restraint compared with a raft with embedded beams.

(ii) The structural performance is enhanced as there is no concern about down drag of embedded beams due to clay shrinkage.

(iii) The proportions of the cross-section can be achieved reliably without excess concrete being needed for over excavation.

(iv) Reduced protection against surface moisture ingress under slab.

Bond breakers should be provided between the tops of the piers and the waffle raft in cases where the waffle raft is supported by piers on a site subject to reactive soil movement. Bond breakers allow the raft to lift off the piers if for example there is swelling of reactive soil under the raft. This prevents the piers from imposing any unnecessary additional loads or restraints on the raft.

(h) **Stiffened slab with deep edge beams, SSD (Class M)** This form of slab, illustrated in Figure 3.5 in 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- and L-shaped houses would have a significant effect but they are not always required (see Figure 3.5, Note 4). The edge beams may include reinforced masonry.

(i) **Strip footings proportions** A strip footing is a footing of rectangular cross-section used to support the external or internal walls of a house. 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 allowable bearing pressure is set at 100 kPa.

TABLE C3.1

TYPICAL APPLIED 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 this Table 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 could occur with very narrow footings. For simplicity, the Standard has adopted the same sizes for both one and two-storey construction.

The footings can withstand some movement. For example, the masonry veneer footing could withstand loss of support up to 3 m within its length, though only about 1.2 m at the corner.

(j) **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 can be determined from Appendix E and AS 1684, 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 use footings at completely stable depths. Shallower footings with the consequent possible need for minor maintenance, such as repacking, may be more economical.

(k) **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 can be on deep stumps or grided internal strip footings. A combination is possible, but is not recommended.

In some cases, to minimize the damage caused by differential movements, a minimum spacing is specified from the outside footing to the first row of internal stumps. This is intended to increase the tolerance to any differential movement compared with conventional spacings.

The cross-section proportions are chosen to give high contact pressure, strength and stiffness to cope with ground movement. Internal settlement under stumped or piered footings is common but simple to repair. Such settlement occurs because the clay under the house dries to equilibrium with 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 depth to counter these movements may not always be adequate but was 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 differential movement between the floor and the wall.

(l) **Reactive designs on stable sites** In some circumstances it may be economical to use a reactive clay design (eg. a waffle raft) on a Class A or S site.

C3.1.1 Selection procedure 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 noted below.

Slab size Factors that normally are not critical may become so when slabs are longer than 25 m in their longest dimension. Examples are concrete shrinkage and raft action.

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 may be critically reduced.

Concentrated loads The restrictions relating to wall heights, columns and the like 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.

C3.1.2 Design for single-leaf masonry, mixed construction and earth masonry 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.

C3.1.4 Design for masonry feature walls Masonry veneer or strip footing houses 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 achieved by a full height door or window.

A minor concession is offered in this Clause for clad framed outbuildings and extensions. Design can be used for one class less severe than that required by that site classification. Thus Class M design may be used on a Class H site. For the purpose of this Clause an outbuilding should be limited to 9 m length.

This Clause also permits the use of footings of the same proportions as the main house.

C3.1.6 Design for rock outcrops Additional reinforcement can 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 house 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 house 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.

C3.6 FOOTINGS FOR CONCENTRATED LOADS 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.

REFERENCES

1 WALSH P.F. ‘To Beam or not to Beam’. *Building Surveyor, 1978.* Vol. 2 p.24/26

2 CAMERON D.A. AND WALSH P.F. *Footing Systems and Floors for Houses; Opinions, Experiences and Trends.* CSIRO Division of Building Research Report, 1984.

3 WALSH P.F. *The Analysis of Stiffened Rafts on Expansive Clays.* CSIRO—Division of Building Research, Technical Paper No 3, 1978.

4 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.

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 the designs given in Section 3 of the Standard. With adequate justification a slightly or completely different footing could be designed, but this option should not be used unless there are good grounds for changing the standard designs.

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 that 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 can 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 Clauses C4.1(b)(ii) and (iii).

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 else 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 which contributes to soil swelling.

Experiments in Melbourne (Ref. 1) have shown that piles can be very effective in resisting reactive clay movements. Piles can 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.

C4.4 RAFT FOOTING SYSTEMS

Table 4.1 provides guidelines for acceptable design differential movements for different types of construction. The values from this Table should not be used out of context, or inordinate importance placed on them. Various limitations on these values are:

(a) 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.

(b) 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.

(c) 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 should be considered.

C4.4(c) *Flange width* Footing beams in rafts are effectively continuously supported along their length, and the application of the AS 3600 formula is not appropriate to the deformation pattern. The effective total flange width has now been simplified to 2 m and 4 m for edge and internal beams respectively.

C4.5 MODIFICATION OF STANDARD 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.

REFERENCES

1 CAMERON D.A. AND WALSH P.F. *The Pile Experiment*. CSIRO—Division of Building Research Report, 1983.

SECTION C5 DETAILING REQUIREMENTS

C5.2 DRAINAGE REQUIREMENTS

C5.2.1 General requirements Defective surface drainage is a common causal factor in reactive clay foundation movement problems. The selection of 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 house 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 requirement 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 recognised 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.

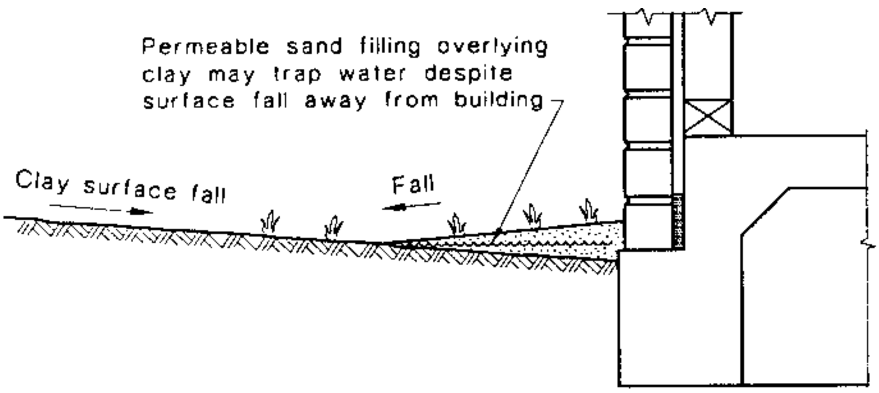


FIGURE C5.1 UNSATISFACTORY METHOD OF ACHIEVING SURFACE FALL AWAY FROM BUILDING

C5.2.2 Specific requirements for slabs 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 areas.

The relative heights of the overflow relief gully, slab and finished ground are intended to stop sewage flooding the house if a blockage occurs, and to prevent rainfall runoff flowing into the sewer. However, this requirement only restricts the freeboard locally. The actual dimensions depend on local plumbing regulations and AS 3500.2, *National Plumbing and Drainage Code*, 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.

C5.3.2 Reinforcement In the Standard, fabric for slab and beam reinforcement is specified, mainly as a result of experience in most States. Generally, trench mesh fabric 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.

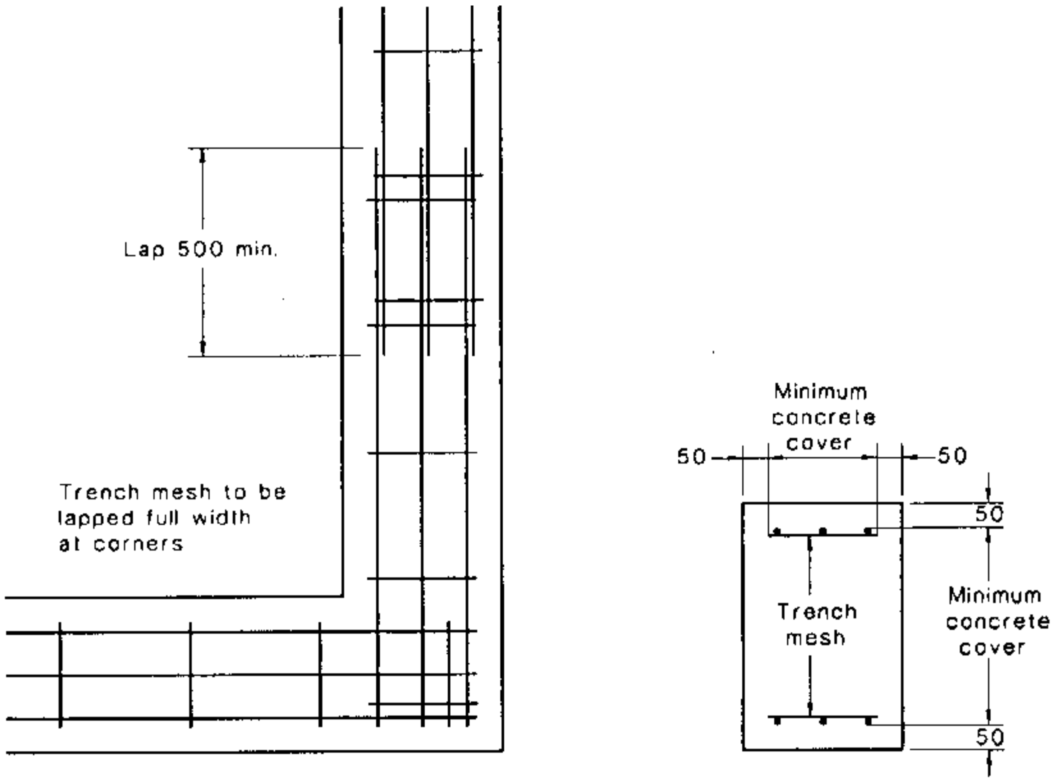
The builder and Building Authority should be cautious about substituting new forms of reinforcement for conventional steel reinforcement. It should be appreciated that the slab fabric acts not only as shrinkage control but also as structural reinforcement and cannot be replaced by alternative methods 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 mesh in beams at T- 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 or 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.

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 C5.3 TRENCH MESH DETAILS

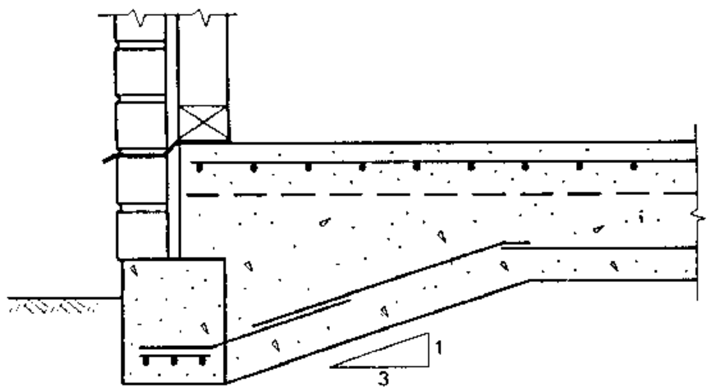


FIGURE C5.4 JUNCTION WHERE EDGE AND INTERNAL BEAMS ARE AT DIFFERENT LEVELS

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.

In areas such as South Australia which 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 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. Deeper rebates should be formed, shallow rebates may be trowelled.

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 the Standard (Figure 5.3) apply to slabs with recesses located away from beams.

Recesses can 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.

C5.3.7 Shrinkage control The prevention of shrinkage cracks is a difficult problem in slabs if brittle floor coverings are to be used or if the slab is to be a termite barrier. In most other cases, shrinkage cracking is not of concern.

The minimum requirements of AS 3600 are just satisfied by the standard fabrics specified in Sections 3 and 4, but this level of reinforcement only provides nominal control of cracking. Such control is offered by the force exerted across the crack which drags the sections of the slab together and stretches the uncracked section due to elastic and creep extension. Concrete can be expected to shrink by 600 to 1000 microstrain for laboratory specimens. After allowing for time and shape effects, for a rectangular house 20 m long this means a shortening of 12 mm to 20 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 10 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 would be expected. To reduce the crack widths to negligible proportions would require around 0.6% reinforcement (or more than F81) and would add several hundred dollars or so to the 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. Some evidence exists that increasing the mesh in critical areas is beneficial and this practice is encouraged in the Standard. The shrinkage fabric operates more efficiently near the top surface and this location is also preferred in the Standard.

The Clause sets out some rules 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:

(a) 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.

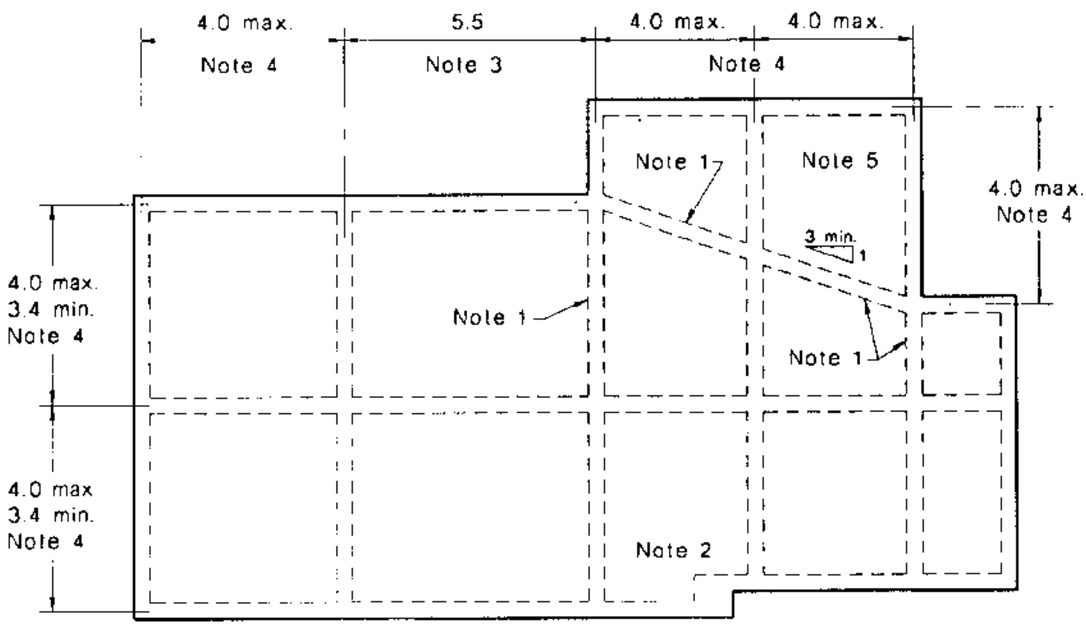
(b) For L- and T-shaped houses the beams should be located to continue the edge beams at the internal corners.

(c) 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 C5.5.

(d) 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. Examples of appropriate details are provided in Figure 5.4.

(e) 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 C5.5.

It may also be necessary to take the location of plumbing into account when selecting the beam positions.



DIMENSIONS IN MILLIMETRES

FIGURE C5.5 ARRANGEMENT OF STIFFENING BEAMS

This example is based on a nominal 5.0 m beam spacing criterion.

NOTES:

1 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.

2 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.

3 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.

4 The spacing between the beam and the edge beams and the first internal beam should not exceed 4.0 m.

5 The internal beam has been deflected to provide continuity at the re-entrant corners. The deflection is not greater than 1 in 3.

Two-pour raft construction techniques may require special reinforcement details at construction joints. A variant of the two-pour raft construction technique remains 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' and 'L' shaped house 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 C5.6 for an example of this situation.

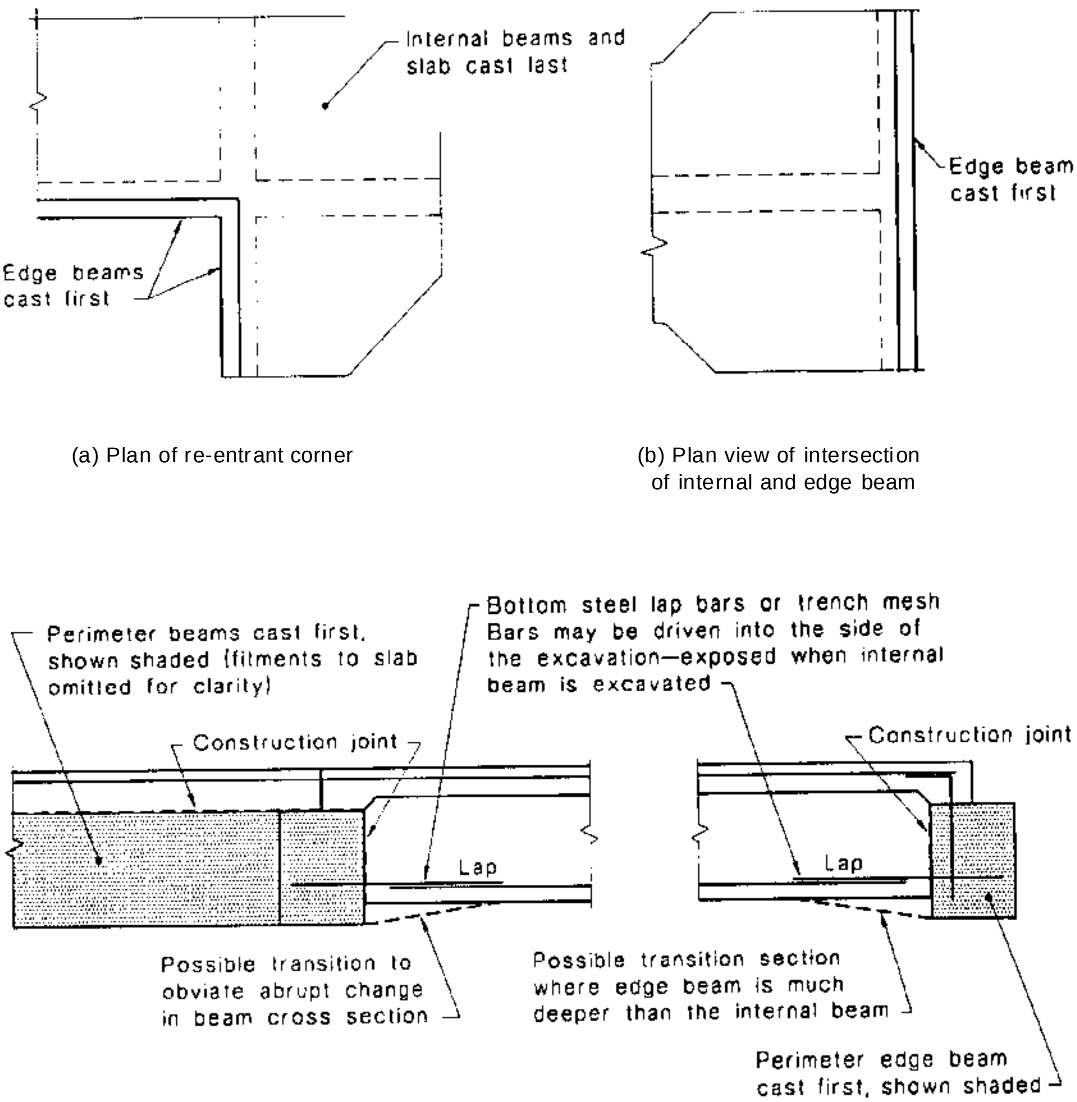
C5.5 ADDITIONAL REQUIREMENTS FOR CLASS H AND CLASS E SITES

Considerable care is required on the more reactive sites to minimize the risk of damage, through both careful detailing of the design of the house 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.

C5.5.2 Variations in foundation material Isolated outcrops of rock may be simply removed. Alternatively, the footing depth may be reduced and the footing stiffness maintained, despite a reduction in section, by the use of substantially more reinforcement.

C5.5.3 Drainage requirements Trenches for service piping may introduce water into the subsoil beneath a house. Backfills are usually highly permeable relative to the surrounding clay soils. Therefore the surface of the backfill within the vicinity of the house should be 'sealed' to reduce moisture ingress. Additionally, the base of the trench should be sloped away from the house, to drain any water away.

Subsurface drains should be avoided near the footings, where practical, as they can introduce water to the foundation if the drains become blocked. However it is recognised 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.



(a) Plan of re-entrant corner

(b) Plan view of intersection of internal and edge beam

(c) Section at re-entrant corner

(d) Section at intersection of internal and edge beam

FIGURE C5.6 EXAMPLE OF INTERSECTION OF INTERNAL AND EDGE BEAM

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 Class H and E sites stated in Clause 6.6.

For durability AS 3600, Concrete Structures Code, 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 6.4.8. Otherwise normal building practice is expected.

Compaction by a vibrator is recommended for Class H and E sites and for areas with edge moisture penetration discussed above.

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 can 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 can be used.

Propping may be required to the sides of excavations to ensure safe working conditions and to maintain the integrity of the foundations.

C6.4 CONSTRUCTION OF SLABS

C6.4.1 General For Class H reactive sites the recommendations in Clause 6.6 are essential to the satisfactory performance of the house and footing system and should not be overlooked.

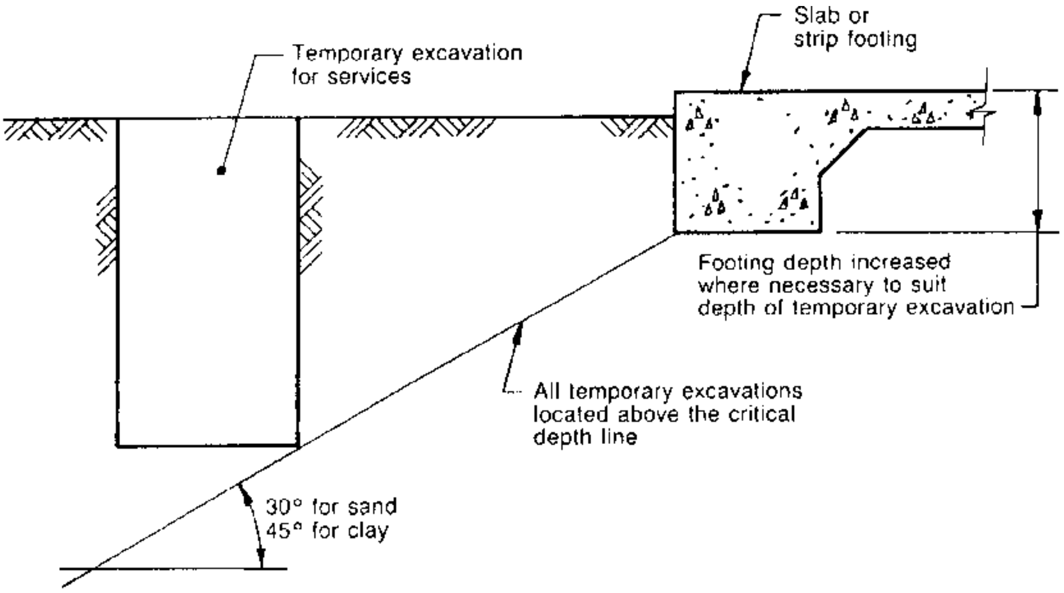
No specific provisions are made about the placement of services beneath the slab, but experience and good practice have shown that:

(a) Services should be bedded on, and backfilled with, properly compacted material which is compatible with the natural material on site.

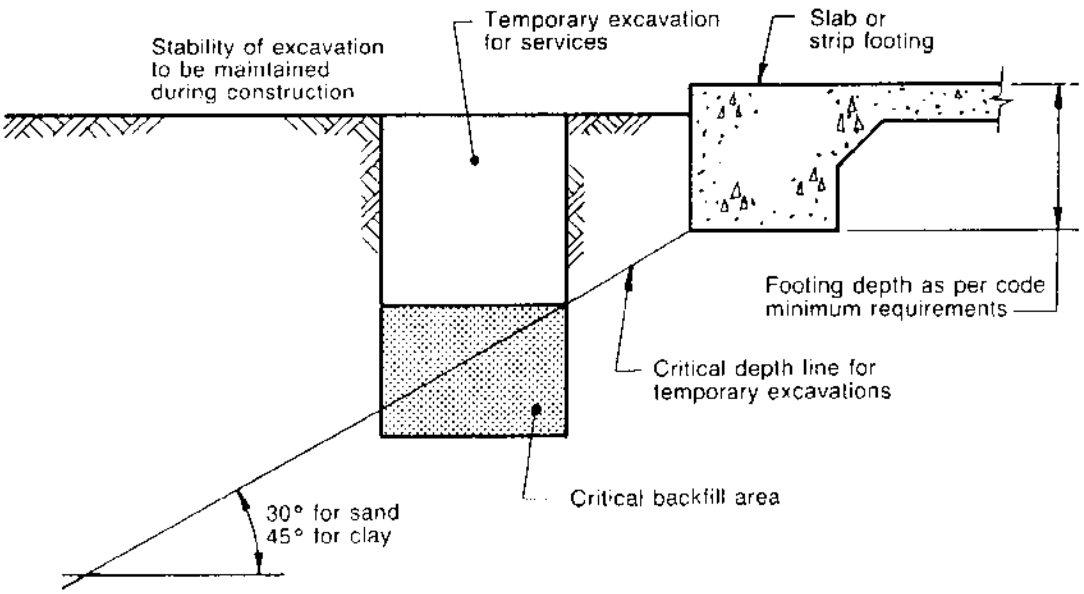
(b) Services running parallel with edge or internal beams should not be positioned beneath these beams.

(c) Beams through which services pass may need to be locally deepened and may require additional reinforcement (see also Clauses 5.3.2(e), 5.4.2(e) and 5.5.4(a)). The pipe or conduit should be wrapped with void-forming material.

(d) Services should not rise vertically through beams. If such 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 houses 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. Moreover, swelling problems could be caused on reactive clay sites by the introduction of water to the foundation. On clay sites this method of compaction is not recommended.

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 that 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:

(a) *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.

(b) *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:

(i) Grading the ground surface to limit the catchment area adjacent to a building to less than 100 square metres.

(ii) Providing a drainage system which prevents run-off adjacent to the building.

(iii) Providing a 600 mm wide concrete path around the building.

(iv) Founding the edge beams at least 300 mm below the finished ground level.

(c) *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.

(d) *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.

(e) *Blinding layer* The blinding layer of 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. 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 can generally be deepened and a very deep edge rebate can 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 the change 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 must 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 Class A, S and M sites Some simple prescribed systems are given, but other engineered designs are feasible.

C6.5 CONSTRUCTION OF STRIP AND PAD FOOTINGS

Many of the details given above for construction of rafts and slabs are also applicable to the construction of strip and pad footings.

SECTION C7 DESIGN OF FOOTING SYSTEMS—CLASS P SITES

C7.1 GENERAL The design of footing systems for a Class P site should be carried out generally by a qualified engineer who should take into consideration the recommendations in this Section.

C7.2 DESIGN FOR SOFT FOUNDATIONS For soft foundations the footing system may consist of one of the following:

(a) Pier-and-beam or pier-and-slab.

(b) Slab-on-ground or stiffened raft, designed to cope with the low allowable bearing pressure of the soil.

Where the site conditions include soils of adequate bearing capacity but which are liable to excessive settlement under a 50 kPa foundation pressure, a stiffened raft may be designed on the basis of the movement limits in Clause 2.1, permitting the use of a stiffened raft for Class M and Class H sites. Account should be taken of the expected movement pattern since the edge distance is commonly much greater for load induced movements than for reactive movements.

(c) Widened strip footing suitably designed to cope with the low allowable bearing pressure of the soil.

(d) Deep strip footing founded below the level of the soft soil.

C7.3 DESIGN FOR FILLED SITES The design of a footing system on a filled site may be based on the following:

(a) Where the footing system is to be founded in the fill, an assessment should be carried out of the potential settlement and reactive movements of both the fill and the underlying soil.

A slab-on-ground or stiffened raft may be adopted if this assessment indicates the movements are within the limits in Clause 2.1.

Allowance may be required for possible differential movement in accordance with Appendix B.

(b) Where the footing system can be founded on the natural soil, a pier-and-beam or pier-and-slab design may be suitable.

(c) Where the fill is shallow, deep strip footings founded on the underlying soil may be suitable.

C7.4 DESIGN FOR MINE SUBSIDENCE SITES

C7.4.1 General Footings and slabs in sites subject to mine subsidence should comply with the requirements of the appropriate authority.

It is recommended that this Clause be used where the design is carried out to specified subsidence parameters or for subsidence prone areas not covered by a mine subsidence authority.

Mine subsidence can subject footing systems and the houses they support to severe movements including lateral strain, settlement, slope and curvature. Lateral strains are of particular concern since they are not encountered so severely in other forms of foundation movement. Frequently, the subsidence is associated with the removal of a specific coal seam and after this event, conditions become stable and repairs are possible. The performance expectations in Appendix B are particularly relevant to this form of foundation movement. The recommendations given in this Section are limited and the designer may need to seek additional information.

C7.4.2 Design to reduce the effects of lateral strain In order to minimize the effect of lateral strain (tension and compression) the design should comply with the following recommendations:

(a) The footing system should be selected to suit lateral strain conditions. Slabs-on-ground or stiffened rafts are recommended. Deep beams or piers should be avoided unless slip joints are provided between the beams and the supporting piers.

(b) Except on reactive clays, friction forces should be reduced by over-excavating the trenches and introducing a layer of compacted sand beneath the footing and compressible material at the sides of the footing or edge beams.

(c) The amount of reinforcement in both footings and slab panels should be increased above that required for other structural actions to resist tensile and compressive forces.

(d) The lateral strength should be increased by designing the footing to withstand lateral earth pressure, and by decreasing grid beam spacing.

C7.4.3 Design to reduce the effect of curvature In order to reduce the effect of curvature, the strength and stiffness of the footing system should be designed to accommodate the expected curvature in accordance with the general principles in Appendix F, but allowing for the shape of the deformed ground surface.

C7.4.4 Superstructure design The superstructure design should comply with the general recommendations for reactive sites and the following:

(a) Sewerage and stormwater lines should not be connected to the footing system in order to allow regrading after total settlement has taken place. All plumbing connections to the building should allow for the movement.

(b) Provision should be made, where possible, for relevelling and simple restoration.

(c) Articulation of both the superstructure and footing system should be considered where large curvatures are expected.

(d) Where possible, the walls should be of uniform structural stiffness. Otherwise, control joints should be introduced between components of different stiffness to isolate brittle elements.

C7.5 DESIGN FOR COLLAPSING SOILS Collapsing soils should be treated by one of the following methods:

(a) Compacting the soil.

(b) Founding the footing systems below the level of the collapsing soil using piers, piles or deep strip footings.

(c) Using slab on ground or stiffened raft, designed to cope with the lower allowable bearing pressure of the soil when wet.

C7.6 DESIGN FOR LANDSLIP Sloping sites where landslip is suspected should be assessed, a footing system designed, and appropriate land management procedures applied (see Ref. 1).

REFERENCE

1 WALKER B., DALE E., FELL R., JEFFERY R., LEVENTHAL A., McMAHON M., MOSTYN G., AND PHILLIPS A. 'Geotechnical Risks Associated with Hillside Development', *Australian Geomechanics News*, No. 10, December, 1985, pp 29–35.

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 'reactive depth' (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 ⅔ of the depth H_s for that climatic zone an 'S' classification may be used. Where the covering depth is shallower and the clay highly expansive an 'M' or M-D' classification should be considered.

APPENDIX CF

SOIL PARAMETERS AND FOOTING DESIGN METHODS

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 semi-arid 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.

This, in the case of centre heave, the form of the mound shape depends on climate, whereas in edge heave, the mound shape is only dependent on Y_m.

Therefore, 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.