FOUNDATIONS AND FOOTINGS ---
SOME OF THE PROBLEMS
AND SOME OF THE SOLUTIONS.

Dominic Lopes
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Part 1. The do's and don'ts of a site investigation.

Part 2. What are the new requirements of the new A.S.. 2870 (soon to be published)

Part 3. Do you really understand what "ys" is ?

Part 4 Foundation problems to expect. What and where ?

Part 5 How to solve these foundation problems.

Compacting Vs Removal and replacement of fill.
Concrete slabs Vs Timber floors.
Drilled Vs Excavated Piers.
Driven Treated Timber Vs Screw Piles.
Adjustable stumps and composite floors.

PART 1.

THE DO'S AND DON'TS

OF A SITE INVESTIGATION.

Perhaps you are thinking of doing your own site investigations. ---

Where do you start ?

Assuming you have a properly set up office and laboratory space your next outlay is as follows :

* A utility with canopy and various tools of torture.	$ 30,000
* Two augers and 3 metre extensions (different diameters)	$ 700
* 15 Geological maps of Melbourne and environs	$ 300
* A vane shear tester to save your thumb from severe bruising.	$ 1,200
* A 9kg dynamic penetrometer to check the compaction of Sand and fill. (instead of driving pegs in the ground)	$ 800
* Oven, sieves, balances, Atterberg and Triaxial Compression machines, Nuclear Density meter, compacting moulds and hammers, mould ejectors and various small laboratory items.	$ 37,000
* Pay for N.A.T.A. registration, calibrate all equipment.	$ 6,000
* Employ soil technician/N.A.T.A. signatory	$ 50,000/y
* Wait 6 months for N.A.T.A. registration and carry out all the regular calibrations and inter-laboratory efficiency testing.	$ 5,000/y

Take a crash course in Geology, navigation, augering, field testing, soil classification and law then put all your assets in the name of someone you can trust, so that the lawyers can't take it from you, when you make your first mistake **then wait for the money to roll in.**

Seriously though, if you feel you have sufficient background knowledge and you only want to test house sites and you have no need for back seats in your family car and your wife/friend does'nt mind wallowing in mud or dust on the way to the opera because you didn't have time to clean out the back,; you could probably equip yourself with a 2nd hand station wagon and the bare testing and excavation tools at a cost of about. $ 40,000

If you want to do it all yourself you will probably need a small mechanical drill and a cheap friendly motor mechanic. $ 18,000

So now you have the visible tools but do you have the mental tools ?

Do you know what "ys" is? (We will come back to this later)

Did you know that Devonian Clays sometimes have a Liquid Limit of 120% or more and what does this mean?

Did you know that the geology maps are often wrong and only tell you what to expect on the surface?

Did you know that contrary to some so-called expert opinion Berwick has highly expansive Pyroclastic Clays ?

Did you think (like this same expert) that all basaltic clays are the same ?

Did you know that Greenvale has 3 different Geological profiles within the space of one or two blocks and sometimes on top of each other?

Can you distinguish fill from natural ground?

Did you know that some Limestone clays in Geelong can create surface movements up 150 mm? And, that similar clays in Horsham can cause 200 mm of movement ?

The following are the minimum factors that the Foundation and Footings Society requires its members to address in a simple house investigation.

There are also a number of other problems that require several years of expert knowledge and familiarity with many sources of soil information.e.g.:

Slope Instability and Soil Creep.

Highly expansive clay sites.

Buried channels and dams.

Mining works, solution cavities and buried limestone caves.

Subdivisional clay fill, easements, etc., etc.

The inability to recognize even one such site can destroy your career or at least set it back very seriously.

FOUNDATIONS AND FOOTINGS SOCIETY (Vic)

Extract from --- "Special provisions for site classifications and recommendations for the design of residential slabs and footings for Victorian conditions" -- February 1992.

2.1. OBJECTIVES OF THE INVESTIGATION

2.1.1. The investigation should determine by consultation with the client the details of the proposed building and siteworks including:

a) size, shape and location of building;
b) building use (e.g. residential, industrial);
c) type of construction (e.g. solid brick, brick veneer, tilt-up concrete wall panels);
d) cut/fill proposals;
e) preferred footing systems.

2.1.2. The objectives of the investigation should include the identification, determination and assessment of the following within the area of influence for the proposed building and to the depths indicated in Section 2.3.2:

a) subsurface conditions
b) strength of foundation strata
c) groundwater levels
d) recommendations for suitable footing systems
e) recommendations for founding depths and allowable bearing pressures
f) the effect of works and vegetation on adjoining land, for example the effect of excavations adjacent to footings
g) surface features including terrain, vegetation and drainage
h) development effects including tree retention and/or tree removal, cut and fill operations.

SECTION 2. SITE INVESTIGATION

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2.2. METHODS OF INVESTIGATION

2.2.1. Use of existing information. Reasonably accessible and documented data should be taken into account during the site investigation. This data may include:

a) geological maps
b) regional terrain classification maps
c) topographical maps
d) air photographs
e) local knowledge and performance of existing footings.

2.2.2. Drilling and Sampling Equipment. Soil sampling may be carried out by any of the following methods :

a) hand auger
b) solid or hollow spiral auger
c) dynamic or hydraulic push tube
d) hydraulic hammer
e) back-hoe pits (not within 3m of proposed building - pits must be adequately reinstated)
f) coring, including diamond drilling
g) wash boring.

2.2.3. Field Testing Equipment. Drilling and sampling may be supplemented by one of the following:

a) standard penetrometer tests
b) dynamic cone penetrometer
c) Perth Sand (Cleg) penetrometer
d) vane shear
e) static cone penetrometer
f) geophysical methods.

2.3. PHYSICAL EXTENT OF INVESTIGATION

2.3.1. Number of test locations.

2.3.1.1. The minimum number of test locations for a single detached building should be two (2).

2.3.1.2. Where there are multiple detached buildings, there should be at least one test location in the area of each building (except outbuildings of less than $50m^2$).

2.3.1.3. For residential, commercial or industrial buildings the minimum number of test locations should be related to the approximate area covered by the proposed buildings, in accordance with the following:

Approx. Building Area (m^2)	Minimum No. of Test Locations
0 - 50	1
50 - 100	2
100 - 300	3
300 - 600	4
600 - 1000	5

2.3.1.4. In areas for which geological or soil maps are not available, where significant soil variation has been found on the building area or where there is insufficient local geological knowledge the minimum number of bores may need to be increased.

2.3.2. Depth of Investigation

a) The minimum depth of investigation should be 1.5m. However in all cases at least 50% of the test investigations should be taken to a depth at least twice the footing width below the expected foundation depth and deeper if soft layers are common in the area.

b) The depth of investigations may be reduced where natural rock is encountered.

c) If fill is encountered the depth of investigation should be increased to sufficient depth to confirm the underlying soil profile.

d) Where doubt exists the founding depths should be inspected during construction of the footings and suitable investigations should be carried out to confirm the supporting strata.

e) Where there are deep, highly "reactive" clay profiles at least one test hole should be dug to a depth equal to approximately 2/3 of the "H" depth.

BUILDING SURVEYOR'S CHECK LIST

RESIDENTIAL REPORTS

(Draft only 23/10/1991)

The following information should be supplied :

COMMISSION:

(1) The aims of the investigation and any limitations imposed by the client's commission.

BUILDING AND SITE DESCRIPTION:

(1) The address and position of the block.

(2) The type of superstructure, e.g. Brick veneer, single storey, garage under etc.

(3) Comments on drainage and slope.

(4) Geological formation identified.

(5) Any proposed excavations and/or filling.

(6) Date of investigation and recent climatic conditions.

TESTING PROGRAM:

(1) Drilling and testing equipment used.

(2) No of test site(s) and location.
(Location either by description or sketch to be located within +/- 1 metre accuracy)

(3) Type of tests and their purpose.
(Results to be summarized and stated in body of report or in bore hole logs.)

FINDINGS:

(1) A detailed description of the soil profile.

Natural soil to have the following information : Soil type, Unified Classification symbol, colour, consistency, moisture regime etc.
Filled ground to be described as above with special reference to voids, putrescible matter, building refuse, also degree of compaction. e.g. variable, moderate, well compacted, or "Rolled" or "Controlled".

(2) Location of water table or seepage if found.

(3) Location of excavations, existing buildings and trees, (even in adjoining properties) that may affect proposed building.

(4) Estimated or calculated Surface Ground Movement (Ys)

CONCLUSIONS AND RECOMMENDATIONS:

<u>For all sites other than "P" sites</u>

(1) Site classification and footing specifications.

(2) Recommended foundation depths.

(3) Allowable Bearing Pressure and description of the foundation material.

(4) Statement of possible building problems and recommended solutions.

(5) Additional special recommendations where necessary. e.g. brickwork articulation joints, soil drainage, soil compaction, retaining walls, batter slopes, deeper footings next to sewer excavations, removal of trees, etc.

(6) Reference to Appendix A of A.S. 2870.1, i.e. "Performance Requirements and Foundation Maintenance".

(7) Report to be signed by a suitably qualified Engineer or Engineering Geologist with relevant experience in Applied Geomechanics and Building Science.

<u>For "P" sites.</u>

(1) **In filled sites** an assessment of the total ground movements must be made considering:
(a) The homogeneity, degree and variability of compaction of the fill.
(b) The expected settlement of the fill and the underlying natural soil under its own load and all the imposed loads.
(c) The maximum footing pressure for which the degree of settlement has been assessed.
(d) The reactivity movements of the fill and the underlying natural soils considering the change in the moisture regime after construction.

(2) **On sites with slope instability or mining works** the investigation must be carried out by a suitably qualified and experienced Geologist. The report must assess degree of risk and suggest possible building solutions. If a "high risk" condition is found all footings must be designed by an Engineer familiar with such conditions.

<u>In all cases.</u>

All footing excavations and all design features must be inspected by the designing engineer.

To assess the behaviour of soils there are many other properties that need to be taken into consideration.

Our concerns are mainly "reactivity" and bearing capacity.

Reactivity depends on many factors but obviously clay percentage and clay type are very important.

Bearing capacity depends on many other factors, among these are Cohesion and Friction.

These two properties depend on such factors as moisture, density, particle distribution, particle surface roughness, particle size, soil mixtures, moisture content, clay types and soil structure.

Generally speaking soil bearing capacities at the low end of the scale are underestimated by most builders and engineers. However under high stresses even a stiff or dense soil can fail or at least create unacceptable degrees of settlement.

The definition of Allowable Bearing Capacity is often misunderstood. The following are my favourite definitions :

Ultimate Bearing Capacity :

The value of the loading intensity at which the foundation fails in shear.

Safe Bearing Capacity :

The net loading intensity considered appropriate to the particular foundation for preliminary design purposes.

Allowable Bearing Pressure :

The maximum allowable net loading intensity of the foundation taking into consideration the bearing capacity, the estimated amount and rate of settlement and the ability of the structure to accommodate the settlement. It is therefore a function both of the site and of the structural conditions.

The last statement is often forgotten and is one of the main reasons the geotechnical consultant should be well briefed.

PART 2.

NEW REQUIREMENTS OF THE NEW A.S. 2870.

As you may have already heard that A.S.2870.1 and 2870.2 have been combined and a number of significant changes have been made.

The most significant changes are :

1. Classification.

* We now have much more advice on the classification of different soil profiles for all parts of Melbourne and Victoria which considers climatic differences and the latest information.

* Trees, previous buildings and soil drainage have become important factors to consider.

* Climate considerations have been emphasized.

* Where Classification is carried out by the "soil profile identification" method the classifier shall be a Geologist or an engineer thoroughly familiar with the local geology.

* Some rudimentary investigation requirements are stated.

* Re-classification of filled sites by engineers has been greatly tightened.

* Slabs-on-ground have been stiffened in highly reactive areas and a "D" deeply reactive zone category has been introduced to cater for the different "design mound shape" thought to exist in dry climatic sites.

* Waffle slab solutions for "A" and "S" and "P" sites (on piers or piles) have been introduced.

* Strip and stump solutions include different house types and a more practical solution has been introduced for "H" sites but not for "H-D" sites.

* An easy to use calculation method has been introduced for engineers to "finess" their slab designs up to ys = 80mm.

* Under certain conditions footings or slabs may be lightened where reinforced masonry is used.

* Class "H" and "E" sites have some special requirements and NO SOLUTIONS are offered as "Deemed to Comply" for E sites.

* Site Classification.

2.1 GENERAL Site classification is performed to allow the selection of standard footing designs presented in Section 3 or for the design of footing systems by engineering principles as described in Section 4.

2.1.1 Classification Natural sites shall be classified in accordance with Clauses 2.2, 2.3 and 2.4 for both the expected extent of soil movement and the depth to which this movement extends, into one of the classes given below. In the classification account shall be given to the possibility of a Class P site caused by conditions described in Clause 1.3.3(a), (b) or (c). For the effects of fill on classification see Clause 2.4.4.

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

For Classes M, H and E further division, based on the depth of the expected movement, is required. For deep-seated movements, characteristic of dry climates and corresponding to a design depth of suction change, H_s, equal to or greater than 3 m, the classification shall be M-D, H-D or E-D as appropriate.

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

2.2 METHODS FOR SITE CLASSIFICATION

2.2.1 General For sites that do not qualify as Class P sites, site classification shall include one or more of the following methods:

(a) Identification of the soil profile and either—

  (i) established data on the performance of houses on the soil profile; or
  (ii) interpretation of performance of existing buildings on the soil profile.

(b) Estimation of the characteristic surface movement (y_s).

2.2.2 Identification of the soil profile

(a) Identification of the soil profile and a classification from established data on the performance of houses on the soil profile. The typical soil profile data given in Appendix E shall be used. Where no data are provided in Appendix E, local knowledge where available shall be applied.

(b) Identification of the soil profile and interpretation of the performance of existing buildings.

Identification of the soil profile and interpretation of the performance of existing residential footing systems within the region which are not less than 10 years old and are founded on a similar soil profile shall be assessed in accordance with Table 2.2.

NOTE: The soil type and site conditions should be inspected at footing excavation stage by the classifier to confirm the soil profile.

2.2.3 Estimation of the characteristic surface movement Estimation of the characteristic surface movements (y_s) shall be used to prescribe site class by applying the limits in Table 2.3.

TABLE 2.3

CLASSIFICATION BY CHARACTERISTIC SURFACE MOVEMENT

Surface movement	Primary classification of site
0 mm < y_s ≤ 20 mm	S
20 mm < y_s ≤ 40 mm	M
40 mm < y_s ≤ 70 mm	H
y_s > 70 mm	E

The value of y_s shall be determined using soil shrinkage indices (I_{ps}) appropriate to the soil profile of the site and suction change profiles which represent the design moisture changes. (See Note 1.)

Soil shrinkage indices shall be derived using one of the following methods:

(a) Laboratory tests for soil reactivity (AS 1289.7.1.1, AS 1289.7.1.2, AS 1289.7.1.3).

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

(c) Visual-tactile identification of the soil by an engineer or engineering geologist having appropriate expertise and local experience.

NOTES:

1 The general principles of calculating y_s are given in AS 2870 Supp 1 (Commentary).

2 The estimation of surface movement shall be based on sufficient soil data to adequately describe the soil profile. Estimations based on a single test result to describe a full soil profile may be misleading.

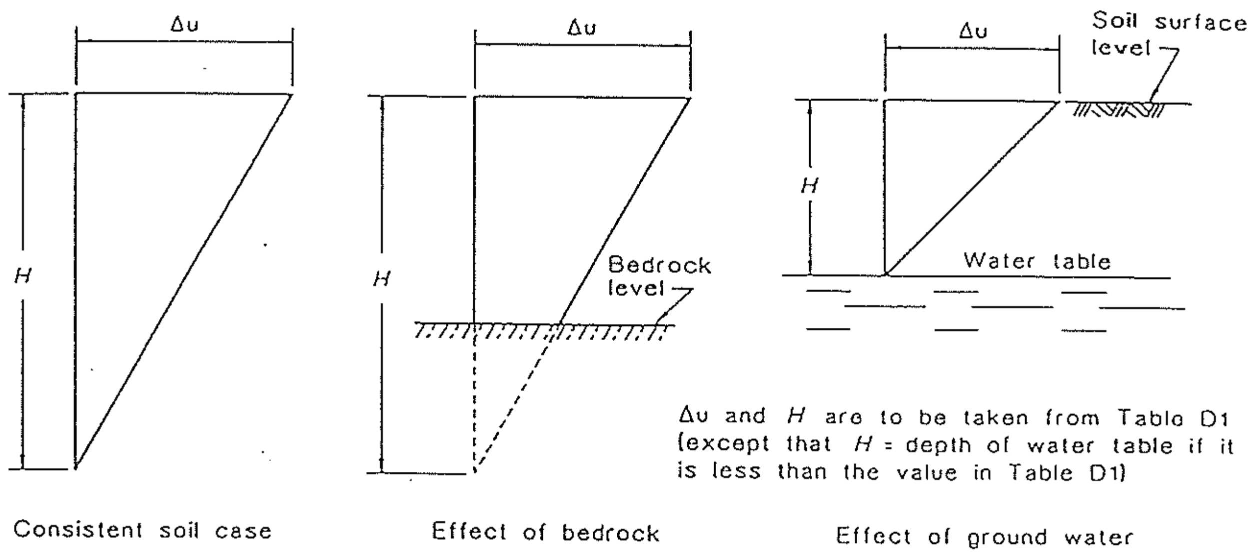
3 As a guide to designers the values of design suction changes are given in Table 2.4 for those locations where data are available. The designer may extrapolate to other areas if due consideration is given to the climate and soil fabric. Where bedrock or a watertable is encountered within the depth H_s from the surface, the suction change shall be modified in accordance with Figure 2.1. Where the soil profile indicates deep and open shrinkage cracking, the depth (H_s) given in Table 2.4 shall be increased to not less than the depth of cracking.

TABLE 2.4

RECOMMENDED SOIL SUCTION CHANGE PROFILES FOR CERTAIN LOCATIONS

Location	Change in suction at the soil surface (Δu) pF	Depth of design suction change (H_s) m
Adelaide	1.2	4.0
Albury/Wodonga	1.2	3.0
Brisbane/Ipswich	1.2	1.5–2.3 (See Note)
Hunter Valley	1.5	2.0
Melbourne	1.2	1.5 to 2.3 (See Note)
Newcastle/Gosford	1.5	1.5
Sydney	1.5	1.5
Toowoomba	1.2	1.8 to 2.3 (See Note)

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



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

Designer The designer is the person or organization responsible for the design of the footing system. Where the design consists of the selection of a design given in the Standard for a Class A, S or M site, the designer may be the builder (see below) or other person experienced in residential building construction. For Class P, H or E sites, however, the designer should be a qualified engineer experienced in the design of footing systems for houses.

2.4.3 Class P sites Sites shall be classified as Class P if—

(a) the allowable bearing pressure at foundation level is less than 100 kPa for strip and pad footings, or under the edge footing of footing slabs used without ties;

(b) the allowable bearing pressure at foundation level is less than 50 kPa under beams and slab panels for slabs;

(c) excessive foundation settlement may occur due to the effects of fill loading on the foundation;

(d) the sites contain uncontrolled fill or certain controlled clay fill as stipulated in Clause 2.4.5;

(e) the sites are subject to mine subsidence, landslip, collapse activity or coastal erosion; or

(f) the sites are subject to moisture changes due to extreme site conditions significantly more severe than the reasonable site conditions described in Clause 1.3.2.

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

(a) the depth of cut on an S, M, H or E site exceeds 0.5 m; or

(b) the depth of fill exceeds the limits provided in Clause 2.4.5.

2.4.5 Effect of fill on classification The following shall be observed.

(a) *Controlled fill.*

(i) *Shallow fill* The classification of a site with controlled fill up to 0.8 m deep for sand and 0.4 m deep for material other than sand shall be the same as the natural site, prior to filling.

(ii) *Deep fill* The classification of a site with controlled sand fill deeper than 0.8 m shall not require a more severe Class than the natural site classification, but may be used to justify by engineering principles a less severe reactive site classification. The effect of the fill on the settlement of the underlying soil shall be taken into account. The classification of a site with controlled fill of material other than sand and deeper than 0.4 m shall be Class P (refer Item (c) regarding classification).

(b) *Uncontrolled fill.*

(i) *Shallow fill* The classification of a site with uncontrolled fill up to 0.8 m deep for sand and 0.4 m deep for material other than sand shall be Class P, unless the footing system is founded on natural soil through the filling.

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

(c) *Reclassification of filled sites* A site filled with controlled fill and classified as Class P can be given an alternative site classification if assessed in accordance with the following engineering principles:

(i) The assessment shall consider the movement of the fill and the underlying soil from the as-constructed condition to the long-term equilibrium moisture conditions. Allowance shall be made for construction variations of moisture conditions. Alternatively the movement may be estimated by reference to established knowledge of the behaviour of similar fills in a similar area.

(ii) The depth of the cracked zone should be taken as zero for reactive clay in controlled fill placed less than 5 years prior to building construction.

NOTES:

1 Long term equilibrium moisture conditions may be taken as—

(a) marginally wet of optimum moisture condition (standard compactive effort) in the eastern coastal area; or

(b) marginally dry of optimum moisture condition (standard compactive effort) in arid areas.

2 Equilibrium moisture conditions may be estimated by reference to similar clays that have stabilized under sealed surfaces.

3 The reclassification should not be less severe than the natural site classification, unless the controlled fill consists of non-reactive material and is deeper than one metre or $0.5H_S$, whichever is greater.

TABLE E2
CLASSIFICATION BASED ON TYPICAL PROFILES - VICTORIA

Soil Profiles	Climatic Zones 1	2	3	4/5
BASALTIC CLAYS (Including "black cotton" soils, pyroclastics, and alluvial clays mainly derived from basaltic and similar volcanic rocks)				
<0.6m depth of Clay.	S/M	S/M	M	M-D
>0.6m - 1.8m depth of Clay.	M	H	H	H-D/E
>1.8m depth of Clay.	M	H	H/E	E
Predominantly friable red brown gravelly Clay (Lateritic)	M	M	M/H	M-D/H-D
NON BASALTIC RESIDUAL CLAYS (Including residual clays derived from sedimentary, metamorphic and granitic rocks)				
<0.6m depth of Clay.	S	S	S	S
>0.6m depth of Clay.	M	M	M/H	M-D/H-D
LIMESTONE CLAYS (Including clays derived from Marls, Limestones and other highly calcareous sediments)				
<0.6m depth of Clay.	M	M	M	M-D
>0.6m - 1m depth of Clay.	M	H	H	H-D
>1.0m depth of Clay.	H	H	H/E	H-D/E
QUATERNARY AGE ALLUVIALS & TERTIARY AGE SEDIMENTS (Including delta, dune, lake, stream, colluvial and wind laid deposits)				
Where predominantly Silts or Sands overlie Clays.				
<0.6m Silts or Sands overlying Clays.	S/M	M	M/H	M-D/H-D/E
0.6m - 1m Silts or Sands overlying Clays.	A/S	S/M	M	M-D/H-D
>1 m Silts or Sands overlying Clays *.	A	A/S	S	S/M-D
Interbedded Silts, Sands and Clay mixtures. (Assess on the basis of total depth of clay in Hs depth)				
< 0.6m total depth of Clay.	A/S	S	S/M	M-D
0.6m-1m total depth of Clay.	S	M	M	M-D/H-D
> 1m total depth of Clay.	S	M	M/H	M-D/H-D/E

Notes :

1 Determination of Hs. The depth categories of design suction change Hs, are based on climatic zones as adopted in this standard. The maps in Figure E1 and E2 are offered as a guide only. Local knowledge may be used to modify the Hs depth given.

2 Soil layers shall be considered within the depth of Hs.

3 In the deeper Basaltic and Limestone Clay profiles in the drier areas a ys greater than 70mm is possible. In such cases local knowledge and expert professional advice should be sought.

4 The "reactive depth" Hs shall be taken as follows :

Climatic Zone 1	*(Alpine or Wet Coastal)*	=	*1.5 m.*
Climatic Zone 2	*(Wet-Temperate)*	=	*1.8 m.*
Climatic Zone 3	*(Temperate)*	=	*2.3 m.*
Climatic Zone 4	*(Dry-Temperate)*	=	*3.0 m.*
Climatic Zone 5	*(Semi Arid)*	=	*4.0 m.*

5 Table E 1 and E 2 may only be used where the soil profiles have been established from published Geological information and by field confirmation or identification by a Geologist or an Engineer familiar with the local Geology.

6 Where classification choices are given the Classifier shall consider the type, position and depth of Clay in the profile, the depth of water table and or rock and the local Climate and environmental factors.

7 * This profile in climatic Zone 4 or 5 should be classified as "M-D" unless the depth of Sand over the Clay exceeds, 2/3 of the depth Hs for that Climatic Zone, in which case an "S" Classification may be used.

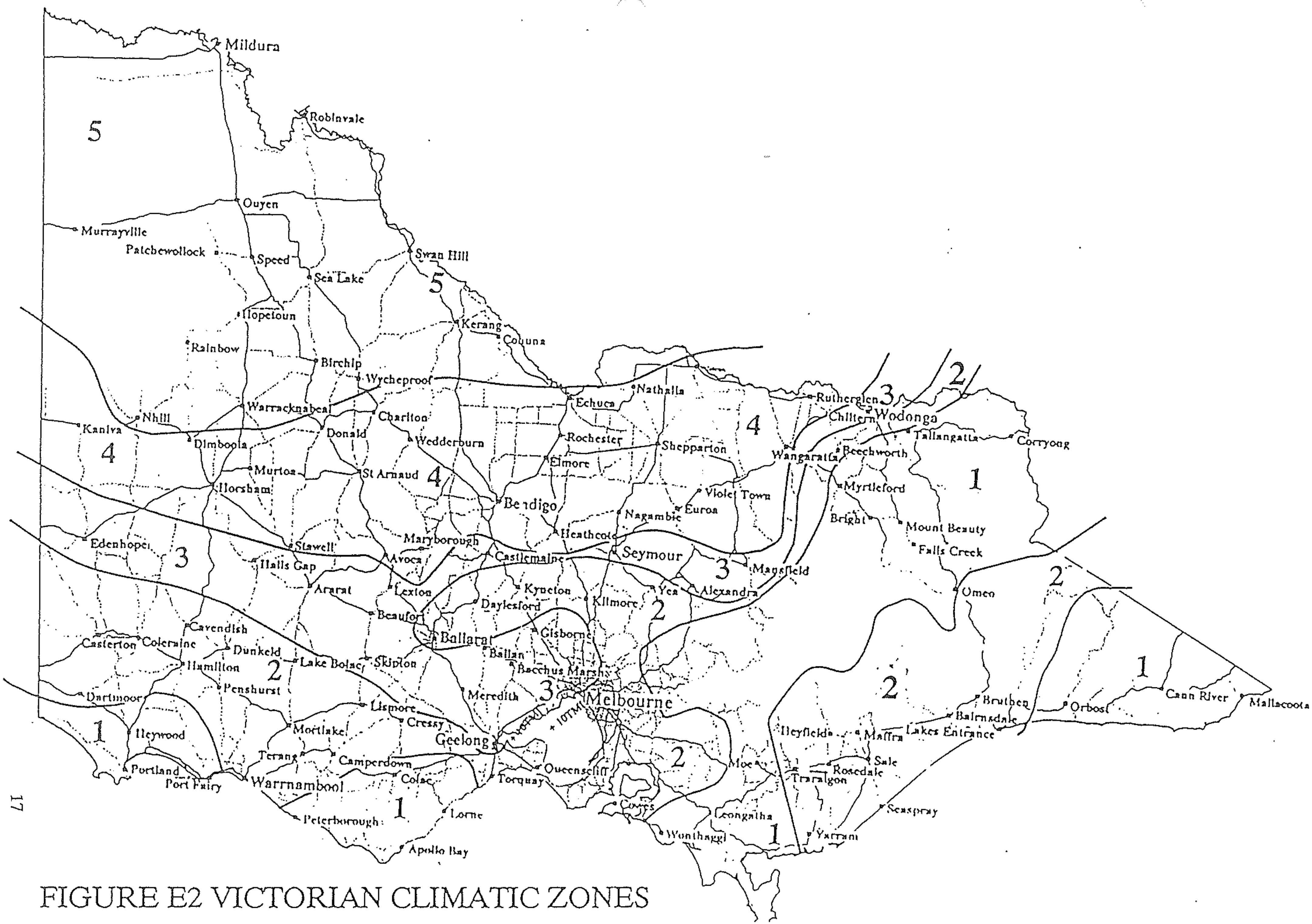
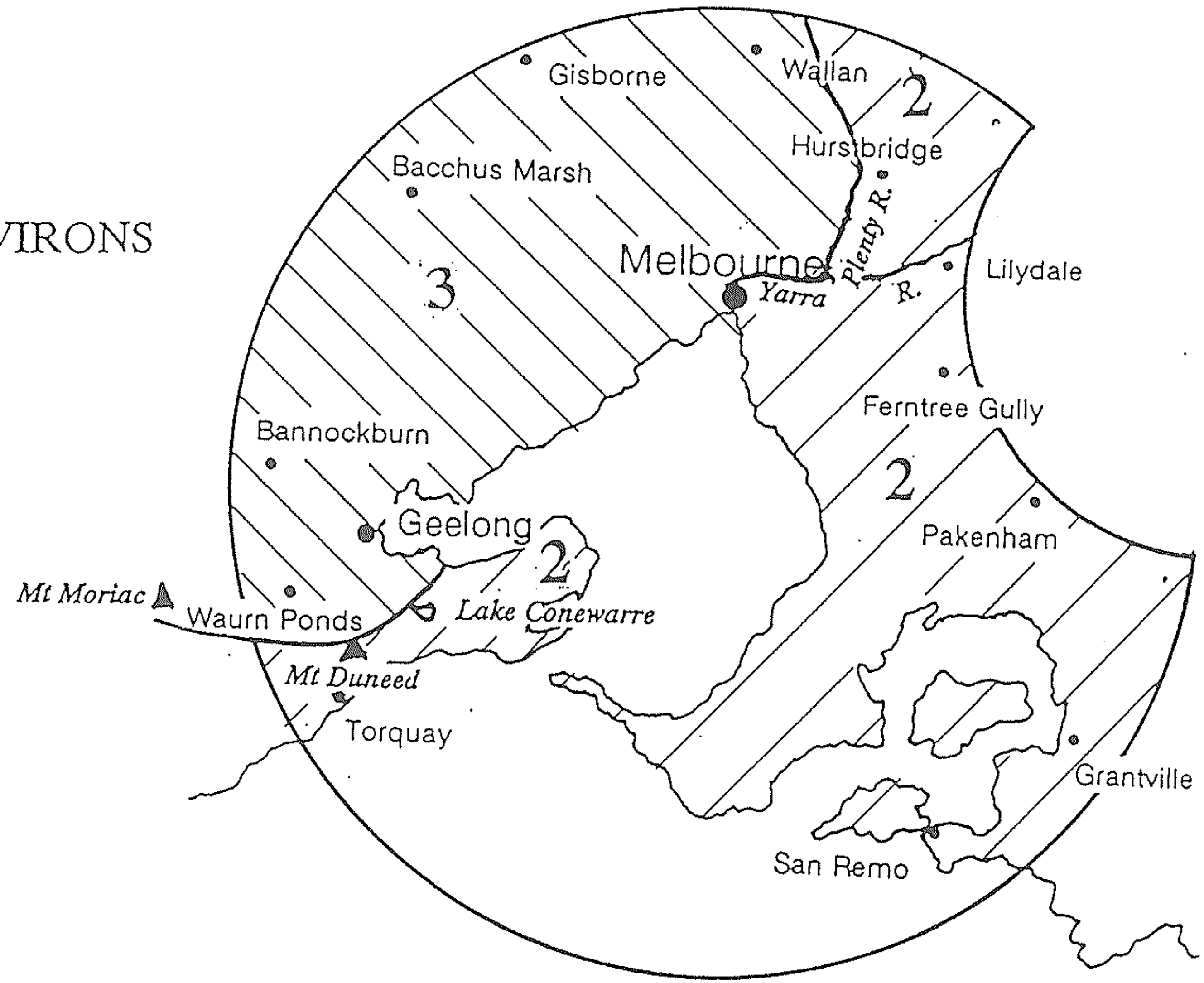


FIGURE E2 VICTORIAN CLIMATIC ZONES

FIGURE E1

MELBOURNE AND ENVIRONS

CLIMATIC ZONES



Abnormal moisture conditions and other design considerations.

1.3.3 **Abnormal moisture conditions** Where the following factors are present, footings will have a higher probability of damage than that given in Clause 1.3.1:

(a) Recent removal of an existing building or structure likely to have significantly modified the soil moisture conditions under the proposed plan of the building.

(b) Unusual moisture conditions caused by drains, channels, ponds, dams or tanks which are to be maintained or removed from the site.

(c) Recent removal of large trees prior to construction.

(d) Growth of trees to close to a footing.

(e) Excessive or irregular watering of gardens adjacent to the house.

(f) Lack of maintenance of site drainage.

(g) Failure to repair plumbing leaks.

5.2 DRAINAGE REQUIREMENTS

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

(a) The local plumbing requirements, in particular the height of the overflow relief gully relative to drainage fittings and ground level.

(b) The run-off from storms and local topography.

(c) The effect of excavation or filling.

(d) The possibility of flooding.

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

(f) The level of existing legal point of stormwater discharge.

(g) Termite management (see AS 3660.1).

5.2.2 **Specific requirements for slabs** The minimum height of the slab above finished ground level shall be 150 mm, except in sandy, well-drained areas where the minimum height shall be 100 mm. These heights can be reduced locally to 50 mm near adjoining paved areas that slope away from the building.

NOTE: These minimum heights are to the top of the finished ground level after completion of paving, etc.

Drainage shall be designed and constructed to avoid water ponding against or near the footing. The ground in the immediate vicinity of the perimeter footing, including the ground uphill from the slab on cut-and-fill sites, shall be graded to fall 50 mm minimum away from the footing over a distance of 1 m from the footing.

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

1.4.3 **Other design considerations** The design of footing systems shall consider:

(a) Effective drainage of the site.

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

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

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

(e) Stiffness and ductility of the footing system.

(f) Strength of the wall system.

(g) Tolerance of the wall system to movement.

(h) Allowable bearing pressure.

2. Engineering Design.

* There is now a new section for engineers which provides a simple method of slab design by "engineering principles". This method will allow modifications of standard slabs up to a "ys" of 80 mm.

4.6 MODIFICATION OF STANDARD RAFT DESIGNS

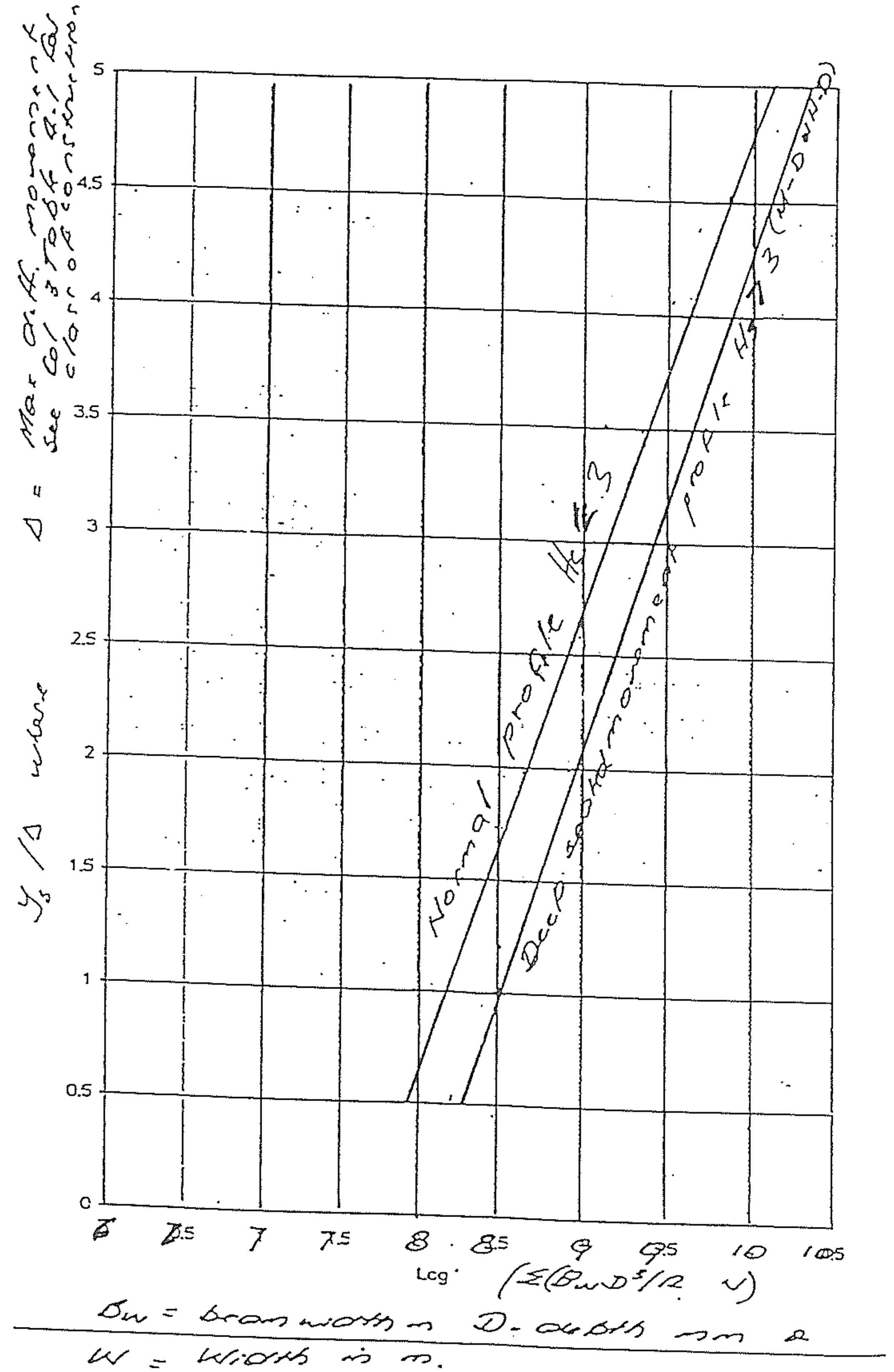
4.6.1 Application This clause may be used provided that the design parameters are within the following range:

y_s	10 to 70 mm if $Hs > 3$ m or 80 mm if $Hs < 3$ m
Δ	5 to 50 mm
Span	5 to 25 m
Beam spacing	$\leq$ 1.25 values in Figure 3.1
Beam depth	250 to 1000 mm
Minimum depth of any beam	$\geq$ 0.7 max. beam depth
Beam width	110 to 400 mm
Average load	to 15 kPa
Edge line load	to 15 kN/m

This clause may be used to modify the standard raft designs in Section 3 where the value of y_s is lower than the upper limit in Section 3, or where the beam layout provides a spacing less than the minimum required. It may be used to override the beam spacing and reinforcement provisions provided that the slab fabric is not less than F72 for slab spans $\leq$ 18 m and F82 for spans 18 to 25 m and provided that the ductility requirements of Clause 4.4(f) are satisfied. It shall not be necessary to use a design stronger than the standard design for the site classification nor is it permitted to use a design weaker than the design given for the next lower site Class.

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

4.6.2 Modification procedure The value of y_s/Δ shall be determined where Δ is the permissible maximum differential movement given in column 3 of Table 4.1 for the appropriate construction. From Figure 4.1 and the value of y_s/Δ, the design shall provide in each direction the stiffness parameter log $(\sum B_w D^3/(12W))$ where the summation is determined over all the edge and internal beams and B_w is the beam web width in mm, D is the overall depth of the beam in mm and W is the overall width of the slab in metres normal to the direction of the beams being considered. The strength shall be provided by the satisfaction of the ductility requirements of Clause 4.4(f). For non-rectangular plans the design shall be based on overlapping rectangles.



NOTE: This chart is only valid for values of y_s up to a maximum of ~~70~~ 80 mm. The graph should not be extrapolated.

FIGURE 4.1 MOVEMENT RATIO VERSUS UNIT STIFFNESS

3. Concrete Floors.

* In recognition of the poor theory in their design and their poorer-than-promised performance in reactive clay sites the Deemed to Comply slabs have been stiffened.

* A solution for Waffle slabs on piers or piles on "P" sites has been included.

* Also a Waffle slab solution for "A" and "S" sites has been added.

Site class	Type of construction	Edge and internal beams (see Note 1) Depth (D) mm	Bottom reinforcement	Spacing centre to centre (m)	Slab fabric
Class A	Clad Frame	300	3-8TM	—	F72
	Articulated masonry veneer	300	3-8TM	—	F72
	Masonry veneer	300	3-8TM	—	F72
	Articulated full masonry	400	3-8TM	—	F82
	Full masonry	400	3-8TM	—	F82
Class S	Clad frame	300	3-8TM	—	F72
	Articulated masonry veneer	300	3-8TM	—	F82
	Masonry veneer	300	3-11TM	—	F82
	Articulated full masonry	400	3-11TM	—	F82
	Full masonry	450	3-11TM	5.0*	F82
Class M	Clad frame	300	3-11TM	6.0*	F72
	Articulated masonry veneer	400	3-11TM	6.0*	F72
	Masonry veneer	400	3-11TM	5.0*	F72
	Articulated full masonry	500	3-12TM	4.0	F82
	Full masonry	800	3-Y16	4.0	F92
Class M-D	Clad frame	400	3-11TM	5.0*	F72
	Articulated masonry veneer	400	3-11TM	4.0	F72
	Masonry veneer	500	3-12TM	4.0	F82
	Articulated full masonry	625	3-12TM	4.0	F92
	Full masonry	—	—	—	—
Class H	Clad frame	400	3-11TM	5.0*	F72
	Articulated masonry veneer	500	3-12TM	4.0	F82
	Masonry veneer	700	3-Y16	4.0	F92
	Articulated full masonry	1000	4-Y16	4.0	F102
	Full masonry	—	—	—	—
Class H-D	Clad frame	500	3-11TM	4.0	F82
	Articulated masonry veneer	600	3-12TM	4.0	F92
	Masonry veneer	—	—	—	—
	Articulated full masonry (See Note 8)	1200	4-Y16	4.0	F102
	Full masonry	—	—	—	—

* See Notes 2 and 3.

DIMENSIONS IN MILLIMETRES

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

NOTES:

1 Internal beams shall be continuous from edge to edge of the slab. Internal beams shall be located to provide continuity with the edge beams at re-entrant corners. Where one side of the re-entrant corner is less than 1.5 m the details in Figure 5.4 shall be considered to provide continuity.

2 A 10% increase in these spacings is permitted where the spacing in the other direction is 20% less than specified.

Where the number of beams in a particular direction satisfies the requirements for the maximum spacing given above, the spacing between individual beams can be varied provided that the spacing between any two beams does not exceed the spacing given in the above Figure by 25%.

3 Beam layouts shall satisfy the requirements of Clause 5.3.9.

4 Where external beams are wider than 300 mm, an extra bottom bar or equivalent of the same bar size is required for each 100 mm additional width.

5 Except on site Classes M-D or H-D, a horizontal construction joint is permitted in the edge and internal beams provided the concrete-to-concrete joint is at least 150 mm wide and traversed by R10 fitments at 600 mm centres or equivalent. (See alternative edge beam detail.)

6 Construction details are given in Clauses 6.4 and 6.6.

7 For Class A and Class S sites, where the slab length is less than or equal to 18 m, wherever F82 fabric is specified, the fabric can be reduced to F72.

8 Where reinforcement is required to be accurately located ligatures shall be provided.

9 If crack control is a design consideration, then refer to Clause 5.3.7.

10 Alternative reinforcement shown in the chart below may be selected in lieu of the slab fabric specified in the chart above.

11 Where a reinforced single leaf masonry wall is constructed directly above and structurally connected to a concrete edge beam, the beam may be reduced to 300 mm wide by 300 mm deep and reinforced with 3-8TM reinforcement.

Internal beam details and spacings shall comply with Figure 3.1. At a re-entrant corner where an external beam continues as an internal beam, the internal beam details shall be continued for a length of 1 meter into the external beam.

Alternative slab fabric	Specified slab fabric		
	F102	F92	F82
	Additional reinforcement at top of beams		
F92	3-11TM	—	—
F82	3-Y16	3-11TM	—
F72	4-Y16	4-12TM	2-12TM

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

Site class and type of construction	Beam depth mm	Bar size beam reinforcement	Slab fabric
Class A			
Clad frame, articulated masonry veneer and masonry veneer	260	Y12	F72
Single storey articulated full masonry and single storey full masonry	310	Y12	F72
Class S			
Clad frame, articulated masonry veneer and masonry veneer	260	Y12	F72
Single storey articulated full masonry	310	Y12	F72
Class M			
Clad frame, articulated masonry veneer and masonry veneer	310	Y12	F72
Class M-D			
Clad frame, articulated masonry veneer and masonry veneer	310	Y12	F72
Class H			
Clad frame	310	Y12	F72
Articulated masonry veneer	385	Y12	F82
Class H-D			
Clad frame	385	Y16	F72
Articulated masonry veneer	385	Y16	F82

DIMENSIONS IN MILLIMETRES

FIGURE 3.4 (in part) WAFFLE RAFT

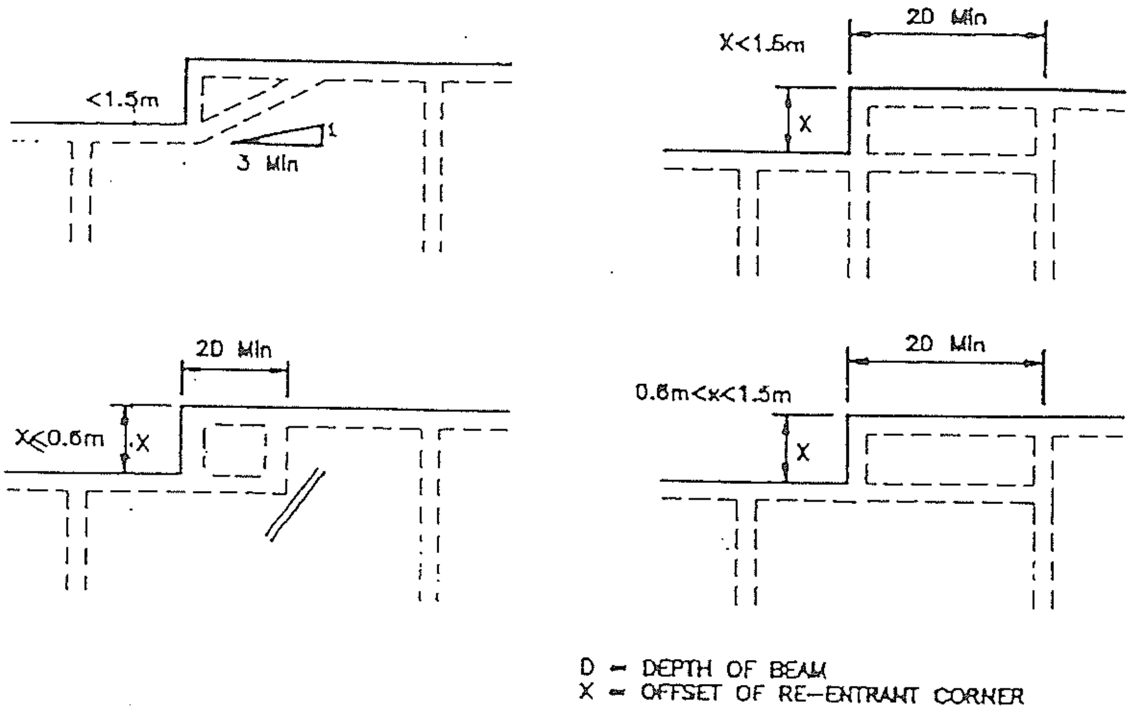
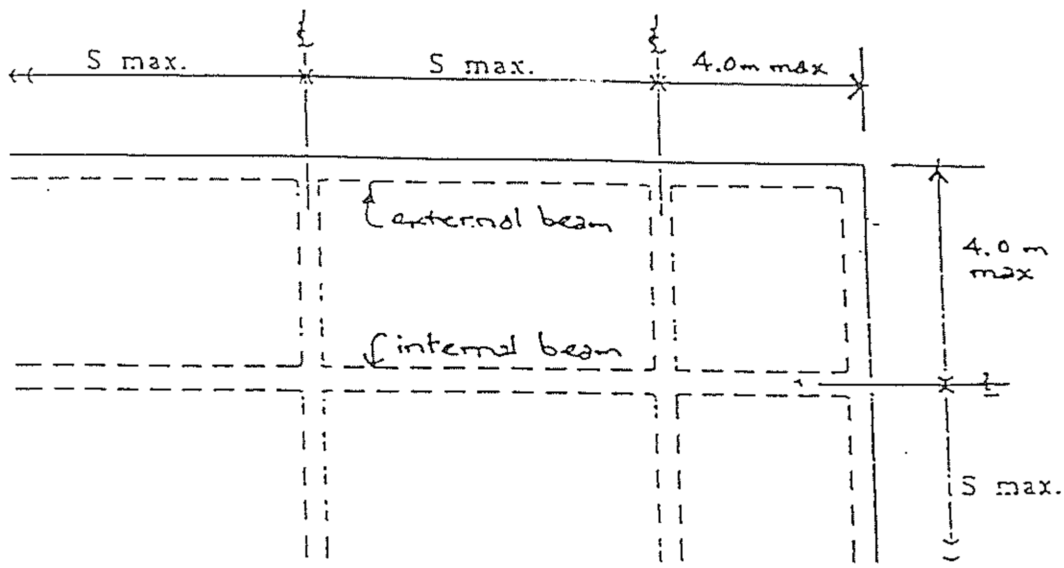


FIGURE 5.4 LAYOUT OF FOOTING BEAMS

5.3.9 Beam layout restrictions For Class M and H sites, in addition to the beam spacing criterion given in Figure 3.1, the maximum spacing between intersections of an internal beam with an external beam shall be 4.0 m. See Figure 5.5.



NOTE: S = beam spacing from Figure 3.1.

FIGURE 5.5 BEAM SPACING MEASUREMENT FOR CLASS M & H SITES

4. Timber Floors.

* These have made a come back in "H" sites. We now have a a much more practical solution for "H" sites. There is also a similar recognition of the flexibility of the structure as for slabs-on-ground.

Site class and type of construction	D	B	Reinforcement	D_s	L_s
Class A					
Clad frame	300	300	3-8TM	400	—
Articulated masonry veneer	300	300	3-8TM	400	—
Masonry veneer	300	300	3-8TM	400	—
Articulated full masonry	300	400	4-8TM	400	—
Full masonry	300	400	4-8TM	400	—
Class S					
Clad frame	400	300	3-8TM	400	—
Articulated masonry veneer	400	300	3-8TM	400	—
Masonry veneer	400	300	3-8TM	400	—
Articulated full masonry	400	400	4-11TM	400	—
Full masonry	500	400	4-11TM	400	—
Class M					
Clad frame	400	300	3-11TM	500	—
Articulated masonry veneer	450	300	3-11TM	500	—
Masonry veneer	500	300	3-12TM	500	—
Articulated full masonry	600*	400	4-12TM	500	—
Full masonry	900*	400	4-12TM	500	—
Class M-D					
Clad frame	500	300	3-11TM	800	—
Articulated masonry veneer	550	300	3-12TM	800	—
Masonry veneer	700*	300	3-Y16	800	—
Articulated full masonry	1100*	400	4-Y16	800	—
Class H					
Clad frame	500	300	3-11TM	1000	≥2.4 m
Articulated masonry veneer	600*	300	3-12TM	1000	≥2.4 m
Masonry veneer	850*	300	3-Y16	1000	≥2.4 m
Articulated full masonry	1100*	400	4-Y16	1000	≥2.4 m
	*See Note 2				

NOTES:

1 All masonry walls shall be supported on strip footings.

2 For Class H sites internal footings shall be provided at re-entrant corners to continue the footings to the opposite external footing as shown.

Internal strip footings shall be of the same proportions as the external footing and run from external footing to external footing.

3 In Class H sites footings deeper than 700 mm shall be provided with 'side slip joints' consisting of a double layer of polyethylene at the sides of the footing only.

4 The size and thickness of pads for stumps or piers shall be selected using AS 1684. Sizes for larger loads may be selected using Appendix D.

5 Bracing forces and uplift forces to stumps may be provided for, using the details in Appendix D.

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

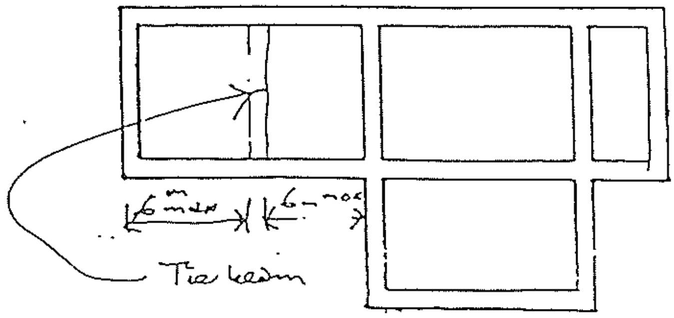
7 Infill floors in Figure (b) may be concrete slabs, brick paving, stone flags or compacted or stabilized earth (which shall only be used for A, S or M sites). For concrete slab infill panels, mesh may be required to control shrinkage in slab panels and around openings or restrained regions. F62 should normally be provided. See also Clause 5.3.7.

8 Where footings are wider than 300 mm, an extra bottom bar or equivalent of the same bar size is required for each 100 mm additional width.

9 $D_f \geq D + 75$ mm.

10 For site Classes M-D and H a provision shall be made by methods such as an adequate crawl space to allow for future re-levelling due to drying effects.

FIGURE 3.6 (in part) STRIP FOOTING SYSTEMS



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

DIMENSIONS IN MILLIMETRES

FIGURE 3.6 (in part) STRIP FOOTING SYSTEMS

6.4.6 Fixing of reinforcement Prior to the placement of concrete the cover to and location of reinforcement shall be maintained by the use of proprietary spacers and bar chairs with bases. Reinforcement shall not be placed or located after concreting.

6.4.7 Placing, compaction and curing of concrete The concrete shall be placed and compacted in accordance with good building practice. Where required for termite protection and for durability requirements for moisture penetrations from edges the concrete shall be vibrated and cured for at least 3 days.

6.5 CONSTRUCTION OF STRIP AND PAD FOOTINGS

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

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

(a) The foundation shall provide an allowable bearing pressure of 100 kPa or the footing shall be founded on controlled sand fill on a Class A or Class S site.

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

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

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

5. Miscellaneous Changes

Site Investigation requirements.

2.3 SITE INVESTIGATION REQUIREMENTS Where a site investigation is required for the purpose stated in Clause 2.3.1, the requirements in Clauses 2.3.2 to 2.3.4 shall be met. (See also AS 3798.)

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

2.3.2 **Depth of investigation** The soil profile shall be examined to a minimum depth equal to 0.75 times the depth of the suction change, H_s, as given in Table 2.4, but not less than 1.5 m, unless rock is encountered or in the opinion of the classifier, further drilling is unnecessary for the purpose of identifying the soil profile in accordance with Clause 2.2.1(a).

2.3.3 **Minimum number of exploration positions** For house sites, either of the following shall apply:

(a) A minimum of one borehole or pit per house site, (see Note 1).

(b) A minimum of three boreholes per site in areas with deep-seated movements (H_s >3.0), or areas where the soil profile is known to be highly variable. (See Notes 1 and 2.)

NOTES:

1 The total number of boreholes across a housing subdivision may be reduced if soil profiling indicates uniform soil conditions which are predictable from soil maps. The presence of gilgais in an area is evidence of highly variable soil profiles within a site.

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

Extra drainage requirements.

5.5.3 **Drainage requirements** Allotments containing reactive sites classified as H or E shall be provided with an adequate system of drainage designed in accordance with the following:

(a) Surface drainage of allotments for reactive sites for both footings and slabs shall be considered in the design of the footing system, and care shall be taken with surface drainage of the allotment from the start of construction. The drainage system shall be completed by the finish of construction of the house.

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

(c) Subsurface drains shall be free draining and shall be able to be inspected and maintained. Subsurface drains shall be protected by filters and geotextiles.

NOTE: Subsurface drains should be avoided near footings wherever practical.

Foundation Maintenance.

C2 FOUNDATION MAINTENANCE

C2.1 Foundation soils All soils are affected by water. Silts are weakened by water and some sands can settle if heavily watered, but most problems arise on clay foundations. Clays swell and shrink due to changes in moisture content and the potential amount of the movement is implied in the site classification in this Standard, which is designated as follows:

(a) A means Stable (Non-reactive).

(b) S means Slightly Reactive.

(c) M means Moderately Reactive.

(d) H means Highly Reactive.

(e) E means Extremely Reactive.

Sites classified Class A and S may be treated as non-reactive sites in accordance with Paragraph C2.2. Sites classified as M, H and E should comply with the recommendations given in Paragraph C2.3.

C2.2 Class A and S sites Sands, silts and clays shall be protected from becoming extremely wet by adequate attention to site drainage and prompt repair of plumbing leaks.

C2.3 Class M, H, and E Sites classified as M, H, or E shall be maintained at stable moisture conditions and extremes of wetting and drying prevented. This will require attention to the following:

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

The site drainage requirements shall be maintained for the economic life of the building.

(b) *Limitations on gardens* The development of the gardens shall not interfere with the drainage requirements nor the subfloor ventilation and weephole drainage systems. Garden beds adjacent to the house should be avoided. Care should be taken to avoid overwatering of gardens close to the house footings.

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

(i) 1½ × mature height for Class E sites.

(ii) 1 × mature height for Class H sites.

(iii) ¾ × mature height for Class M sites.

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

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

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

Braced Stumps

D3 UPLIFT AND HORIZONTAL CAPACITY The basic uplift and horizontal capacity of braced stumps shall be determined from Figure D2. The applied design load to the stump shall not exceed the following limits—

$\frac{P_u}{U}$ <1.0 and	$\frac{H^*}{H_H}$	<1.0 for Class A and S sites
<0.9		<1.0 for Class M site
<0.7		<0.8 for Class H and M-D sites

and for combined uplift and horizontal load,

$\frac{P_u}{U} + \frac{H^*}{H_H}$	<1.0 for Class A & S sites
	<0.9 for Class M sites
	<0.7 for Class M-D and H sites

where

P_u = uplift load on stump

U = design uplift capacity on stump

For horizontal bracing loads applied higher than shown in Figure D2, capacity shall be determined by engineering principles.

Stump horizontal capacity (see Table 2) is for compacted soil backfill suitable for 100 kPa bearing as described in Appendix F. For soil with less than 100 kPa lateral bearing the above horizontal capacities shall be reduced by multiplying by $\left(\frac{\text{allowable bearing}}{100}\right)$.

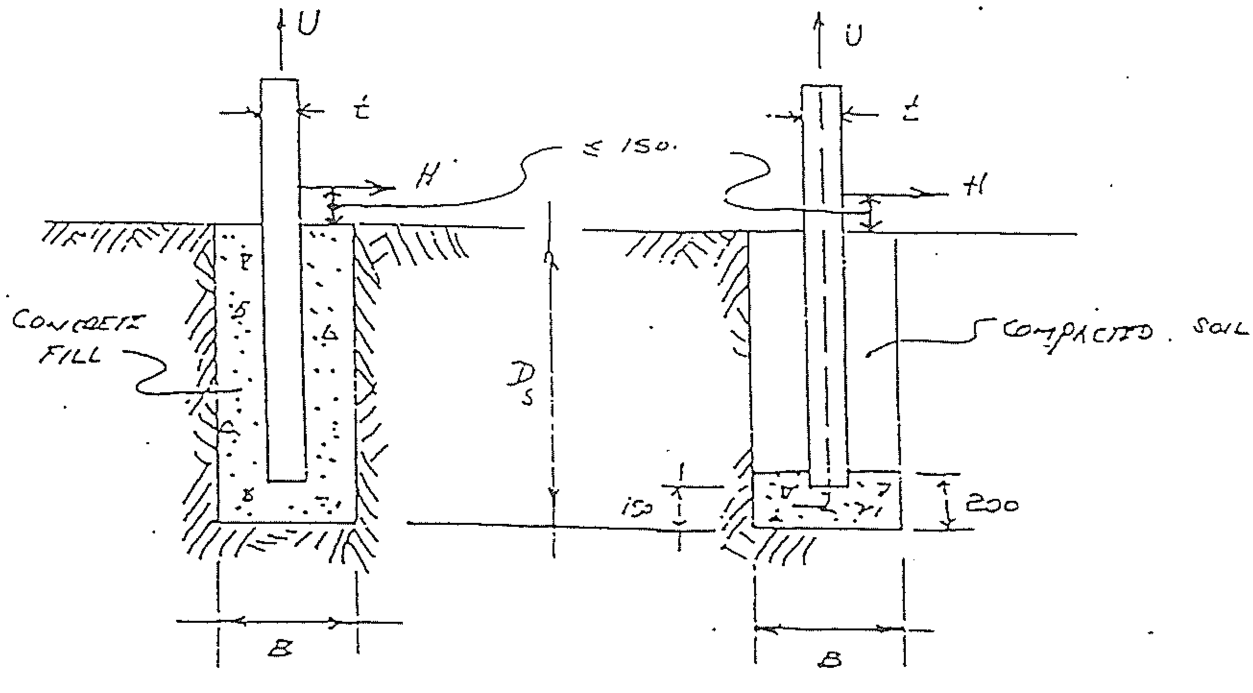


FIGURE D2 BRACED STUMPS

APPENDIX F

DESIGN OF DRIVEN TIMBER PILED FOOTING SYSTEMS

(Informative)

F1 GENERAL Timber piles should be designed in accordance with Paragraphs F1 to F5 below and the appropriate Sections of AS 2159.

F1.1 Pile systems Timber piles may be used in pile systems to support all types of framed and masonry construction.

F1.2 Proportions and load capacity of piles Piles should satisfy the following requirements:

(a) For unspliced piles driven by a drop hammer, the safe working load may be calculated in accordance with:

$$a = 0.001\ bc/s$$

where

a = safe working load of pile, in kilonewtons

b = drop hammer mass, in kilograms

c = hammer drop height, in millimetres

s = pile set, in millimetres.

NOTE: Set is the average pile penetration per hammer impact, for five hammer impacts.

(b) Where piles are used to replace stumps and pad footings, the required safe working load (kN) of the pile may be back-calculated by multiplying the replaced pad bearing area (m^2) by 100 kPa. For stump pad sizes, reference should be made to AS 1684. Otherwise the required load should be estimated by engineering principles.

(c) For piles driven by methods other than by drop hammer, the allowable load capacity should be determined by engineering principles using AS 2159.

(d) Stresses in timber piles should not exceed the allowable stresses in AS 1720.

NOTE: In some areas, there are large differences in soil strengths between wet and dry periods. In such cases, possible effects on pile capacity should be considered.

F2 PILE SYSTEMS FOR CLASS A AND S SITES Piles may be used to support framed or masonry construction on Class A and S sites. The minimum driven depth of piles shall be not less than 1 m for these sites and 1 m below the fill for filled sites or to rock.

For the support of external walls, the standard details for the piles and footing beam given in Figure F1 may be adopted. Piles should be located in all corners and steps in beams.

F3 PILE SYSTEMS FOR CLASS M AND H SITES

F3.1 Footing systems Driven timber piles may be used to support framed or masonry construction on Class M and H sites. Piles should be located in all corners and steps in beams.

F3.2 Load capacity Piles should comply with the requirements of Paragraphs F1.2 for allowable load.

NOTE: Load capacity should be reduced to allow for loss of pile-soil contact due to shrinkage of reactive soils.

PART 3.

WHAT IS "ys" AND HOW IS IT MEASURED ?

There has been a serious misconception of the nature of this value which in many ways can be attributed to poor wording in previous codes and a lack of understanding by many engineers.

This value is now referred-to as the "***Characteristic Ground Movement*** ".

Ys was never meant to represent the actual ground movement expected after the construction of a building; it is merely an approximate "***characteristic***" or range of movement which is calculated by approved methods or assigned to "**grade**" various soil profiles.

The calculation methods given in the Standard have a poor accuracy and are based on certain assumptions and formulas.

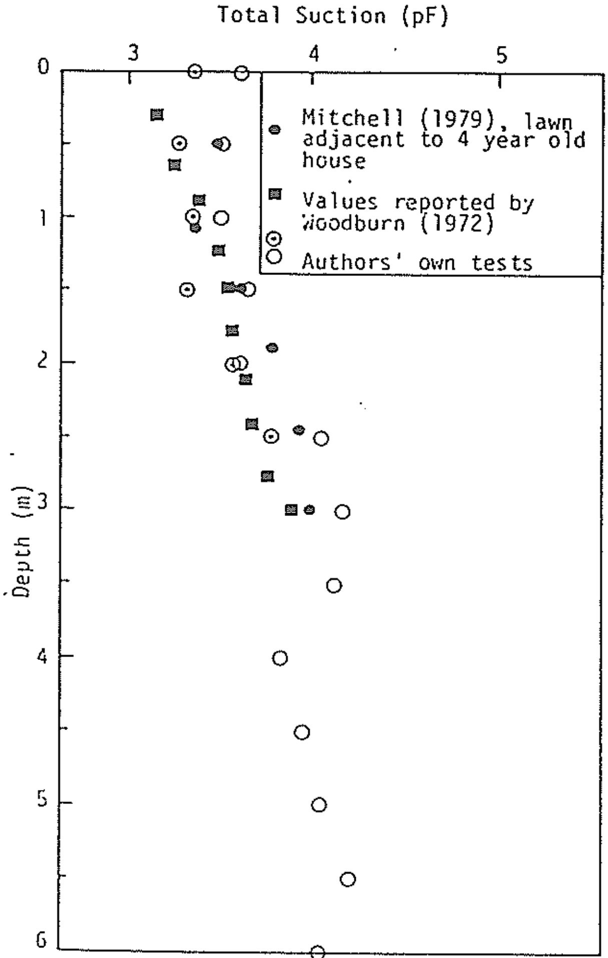
One of the most important assumptions is the depth of soil movement and the changes in soil suction from drought to wet periods. (this depth is called "hs")

The new Standard states the following "hs" depths:

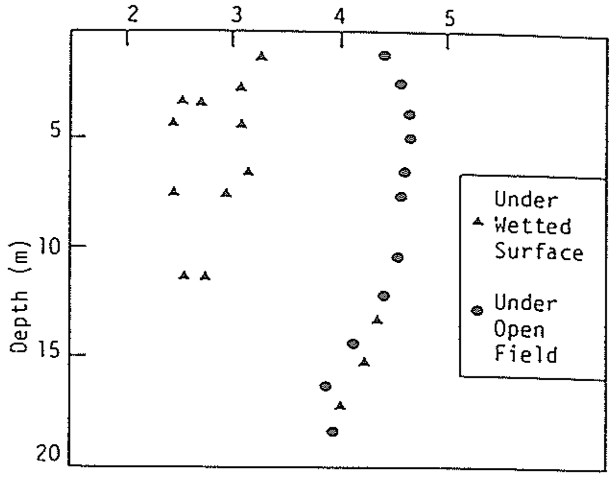
Climatic Zone	*1*	*(Alpine & Wet Coastal)*	=	*1.5 m.*
Climatic Zone	*2*	*(Wet-Temperate)*	=	*1.8 m.*
Climatic Zone	*3*	*(Temperate)*	=	*2.3 m.*
Climatic Zone	*4*	*(Dry-Temperate)*	=	*3.0 m.*
Climatic Zone	*5*	*(Semi Arid)*	=	*4.0 m.*

The changes in soil suction diminish with depth and the Standard assumes a triangular representation.

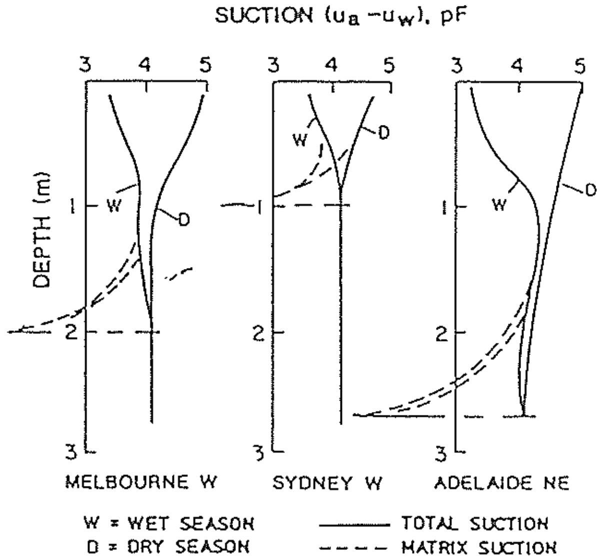
To demonstrate the difficulty in these assumptions I have provided the following actual soil suction readings taken by numerous researchers.

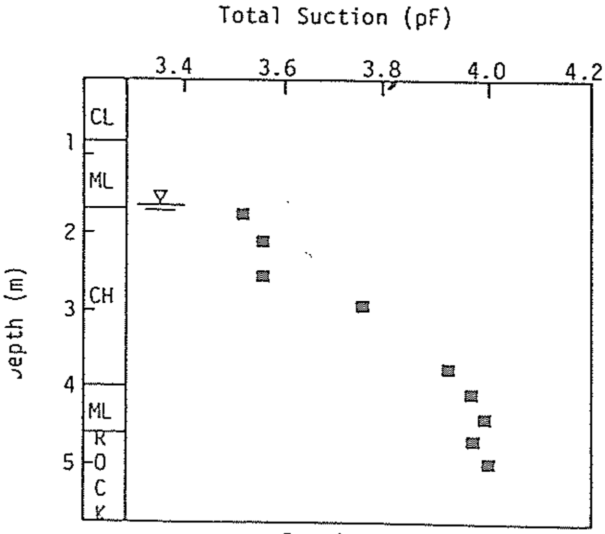


Well Watered Lawns

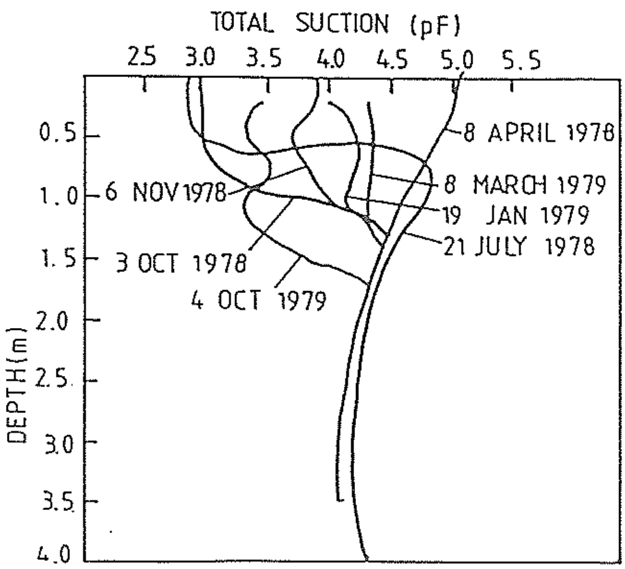


Moisture Penetration into Fissured Soil (Williams, 1980)



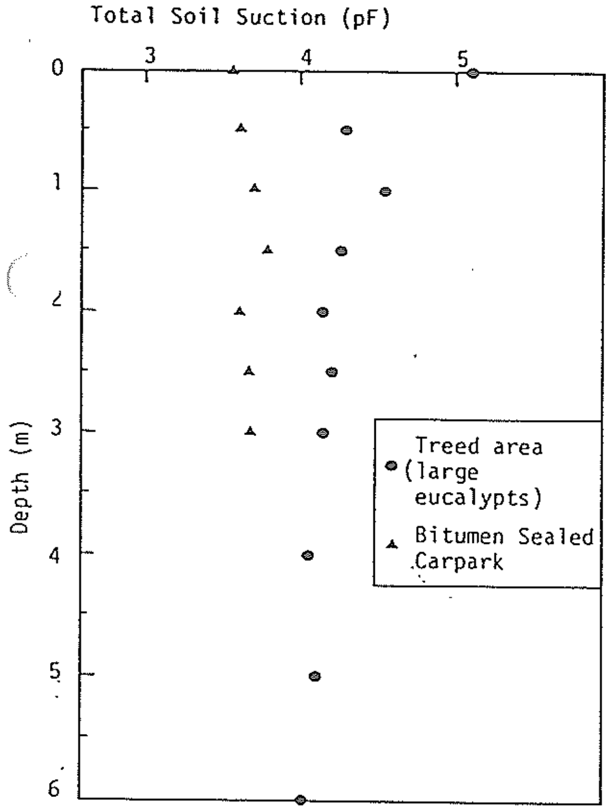


Suction within a Profile containing a Perched Water Table (Aitchison et al, 1973)

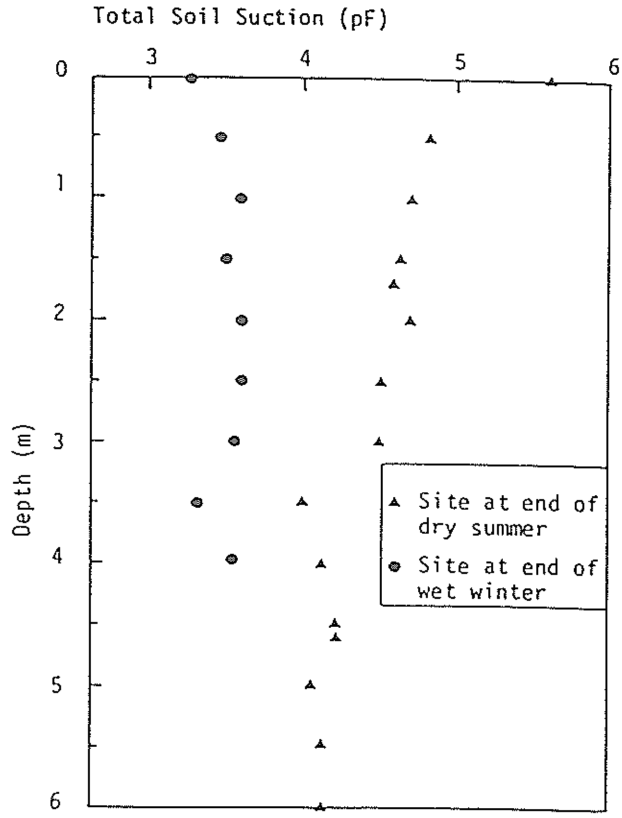


MITCHELL

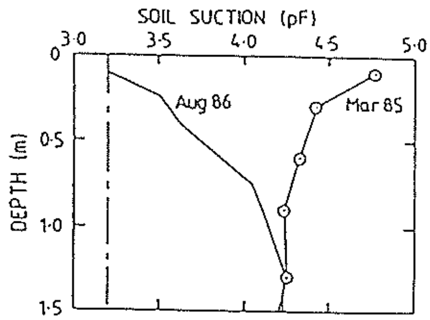
Soil Suction Profiles at O'Halloran Hill



Measured Suction Profiles in an Area of Trees and a nearby Sealed Carpark

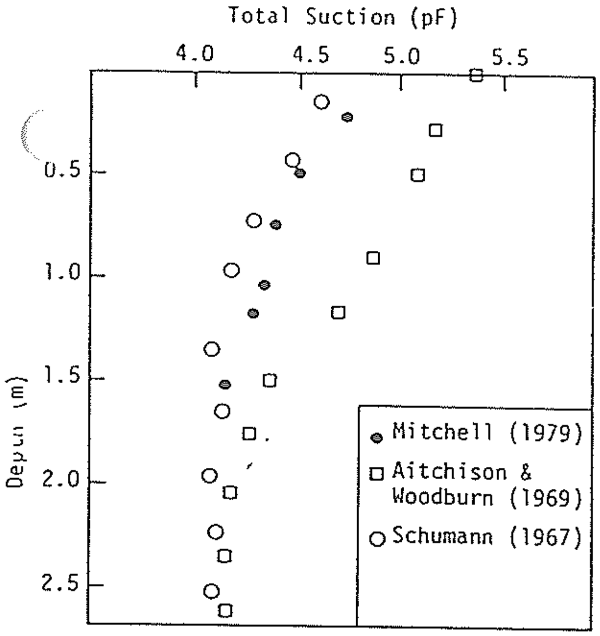


Open Sites, Extreme Seasonal Conditions

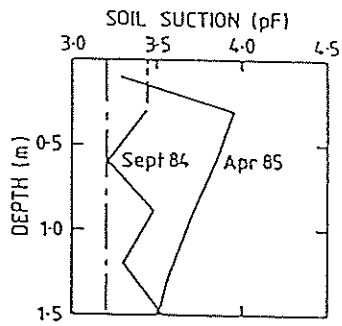


SITE: FLINDERS

Sept 84 to Apr 85

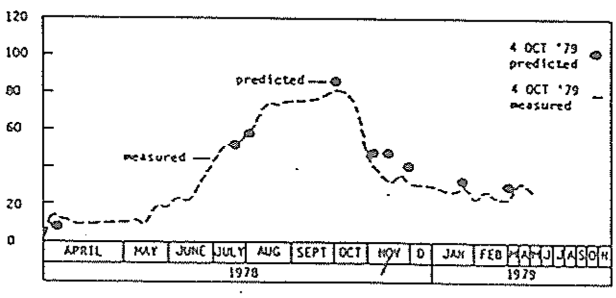


Measured Suction Profiles under well Ventilated Floors

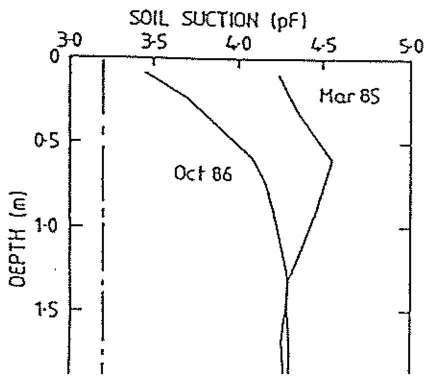


SITE: KEILOR

Mar 85 to Oct 86



Comparison Between Predicted and Observed Seasonal Heave at O'Halloran Hill Test Site



Clearly it is difficult to make a triangle from these values. But one should remember that these readings were taken over relatively short periods e.g. no longer than 18 months- 2 years and have not considered even one full drought to wet period cycle.

Perhaps the best representation is that taken by Mitchell in O'Hallaron Hill in Adelaide. The outer limits of these readings show what you would expect. i.e. a large suction variation in the cracked near-surface soils and then a sharp decline in variation in the uncracked zone.

I believe that this is a more realistic model than the triangle but since it is difficult to predict we are stuck with the triangle until much more research is done.

It is also important to note that the shape of the suction model may differ with different soils in the same climatic zone and in the same soils in different climatic zone. For example some soils such as those derived from limestones have a deeper "hs" than other soils nearby.

In the Melbourne/Geelong area it is suspected that the moisture variation depth during droughts can be as deep as 1.2 metre in the Dandenongs, 1.8 metre in the East and South Eastern Suburbs and the Bellarine Peninsula, up to 2.5 - 3 metre in parts of the Western suburbs and possibly deeper in some parts of the Waurn Ponds formation in Geelong and occasionally in the deep basaltic clays in the drier Melbourne suburbs.

PART 4

FOUNDATION PROBLEMS -- WHAT & WHERE ?

Here are some possible foundation problems that you may find in different parts of Melbourne.

PART 5

HOW TO SOLVE SOME OF THESE PROBLEMS.

* FIND THE CAUSE AND TREAT THE SYMPTOMS.

There are a number of footing systems which have their application provided you have correctly identified the foundation problems and how the superstructure will react.

The rules are :
* Find the cause and treat the symptom.
* Use extra concrete and steel as a last and not first resort.

The best approach in order of importance is as follows:

1. **Do whatever is economical to reduce the ground movements.**

2. **Design a footing which best "marries" the soil behaviour with the capacity of the superstructure to react to the movements.**

3. **Design the superstructure to accommodate the expected footing movements**

* COMPACTION Vs REPLACEMENT OF FILL.

Whether you decide to remove or compact the fill depends on:
-- its depth.
-- its contents
-- the cost of good fill material.

The following are some "rules of thumb" which applies to small developments:

Don't attempt to remove and replace the fill if it is deeper than one metre this will cost a minimum of $\$60/m^3$ and will still require an expensive slab.

Don't attempt to compact the existing fill unless you have enough testing information to be sure that it does not contain significant amounts of rotting matter and that the underlying natural soils are not too soft or too highly organic.

Don't attempt to compact only from the surface in any sand fill greater than 1000 mm deep or expansive clay fill deeper than 600 mm. If you are compacting any existing fill shallower than these depths you must use a heavy smooth drum or impact roller. (Although sheep foot rollers have a high compactive effort its penetration is very shallow)

Where the fill is deeper and needs to be pulled out and re-compacted in shallow layers make sure it is supervised by an independent expert. Constant expert supervision is better than the occasional test and sometimes cheaper. A N.A.T.A registered laboratory is best to do this work.

Remember that the requirements of A.S.. 3798 are only advisory minimums and you need to be satisfied that the compacted fill will behave as you have assumed in your footing and superstructure design.

Take care of consultants that classify fills that they have not supervised or tested as "Controlled" fill.

Make sure that you are given enough information for you to predict how the fill will behave. Even if you accept the value of these parameters from your consultant make an effort to confirm that these values are reasonable in your own experience. If necessary, speak to some knowledgeable colleagues.

Be very careful in considering the effect of greatly differential depth of fill under a building. The concern is differential settlement and differential "reactivity" in clays and differential settlements in sandy soils.

* CONCRETE SLABS Vs TIMBER FLOORS.

Much has been written about this subject; unfortunately quite a deal of it from a biased view point.

The evidence is that the A.S. 2870 slabs have not performed as well as we were promised but they generally perform a little better than timber floors in "H" sites.

On the other hand once they are damaged they are extremely difficult to fix or reduce the ground movement.

Generally speaking slabs work better when they are built on very moist sites. This is because they are more sensitive to soil heave than soil shrinkage since they can better span shrinkage "dishes".

In fact slabs promote soil heave both because of they trap rising water and because the required site levelling work often causes water to pond against parts of the slab.

Strip footing and timber floor systems don't trap water (unless they are deep) but they suffer from under-floor soil shrinkage as it dries out. Deep strip footings in highly expansive clays can be heaved by the clay by the side adhesion.

These systems are more sensitive to shrinkage than slabs because of their much lower stiffness. The consolation is that timber floor settlement due only to drying can be fairly easily repaired. Strip footings also apply a greater pressure on the ground and thus reduce the heave movements in clays.

Another problem which strip footings suffer more than slabs is the horizontal shrinking and swelling of the surface clay. I have noticed this many times causing horizontal brickwork separation. Waffle slabs or other surface footings are less affected by this movement.

In filled sites properly designed slabs work better, but again, if the movement is greater than predicted or designed-for, they are very difficult to fix. In most cases pumping grout under pressure under the slab only increases the load on the fill and accelerates and increases its settlement.

* DRILLED Vs EXCAVATED PIERS.

These footing systems are often comparable systems in filled sites but they do have significant differences and advantages.

Obviously excavated piers use more concrete than drilled piers so why are they popular?

Where the fill is less than 3 metre deep and particularly where it does not occupy the whole building site it is sometimes more economical to excavate the Trapezium shaped piers with the back-hoe rather than causing delays and the costs of bringing another machine (drill rig) on site.

Excavated piers are popular where there are deep easements near the building or where they are to replace deep continuous footings.

However these piers have some disadvantages in that they promote soil collapses, particularly in the corners and are prone to heave in expansive clay sites. This is a particular problem in slab-on-pier footings.

Some typical design considerations for excavated piers.

* Consider the soil that you are expecting the concretor to excavate.

* Do not design steep sided piers in poorly filled or loose or very moist ground.

* In fact do not design excavated piers if the soil is not capable of standing at a minimum of 45% at least for a day or two.

* Assume that piers deeper than 1.5 metre are not going to be cleaned out therefore be conservative with your bearing pressure calculations.

* The backhoe operator should clean the trenches as best he can then compact any loose lumps at the bottom.

* You will need a minimum of 200 kPa Safe Bearing Capacity at the base otherwise the piers will be uneconomical.

* Do not use any adhesion or friction values in your calculation. The soil/concrete contact in these piers is rough and allows water to flow down the sides and collect at the base. This is also another reason for being conservative in your Allowable Bearing Pressure calculations.

* In highly expansive ground you may have to reinforce them and connect them to the strip footings above and perhaps even bell them at the base. This can be done with the backhoe (with lots of persuasion) by curling the bucket at each corner while travelling clockwise around the job then anticlockwise. (this can only be done in stiff clay and not in sand or fill)

* Unless you can ensure that the site is well drained, forget about using void formers. They do not solve the clay swelling problem because they allow water to collect under the footings. This water will exacerbate the swelling both under the "beams" and at around the piers.

* Do not ask piers to be excavated around corners; they will collapse.

* Excavated piers are difficult to excavate beyond 2.5 metre depth and the percentage of concrete saved reduces with depth.

* Ensure that your design shows the minimum base length you require.

* **Most of all, do not assume that any one on site knows what they are doing!**

Another very important consideration is the danger in inspecting their base.

Collapses of big slabs of clay are common. If there is any disturbed soil or lumps of clay at the base it is best to compact this soil with the back-hoe bucket.

In three cases that I recently inspected the 100 mm thick cardboard had been totally squashed and was flat and saturated.

Tests carried out by C.S.I.R.O. in the early 80's showed that even in a timber pile and beam system (built in basaltic clay in Broadmeadows) the 100 mm cardboard spacers did not achieve their objective. It was suggested that the 150 mm thick cardboard should be used. This was never tested in the long-term and my personal view is that if there is enough water around e.g. garden watering then it would be only a matter of time before there is sufficient clay swell.

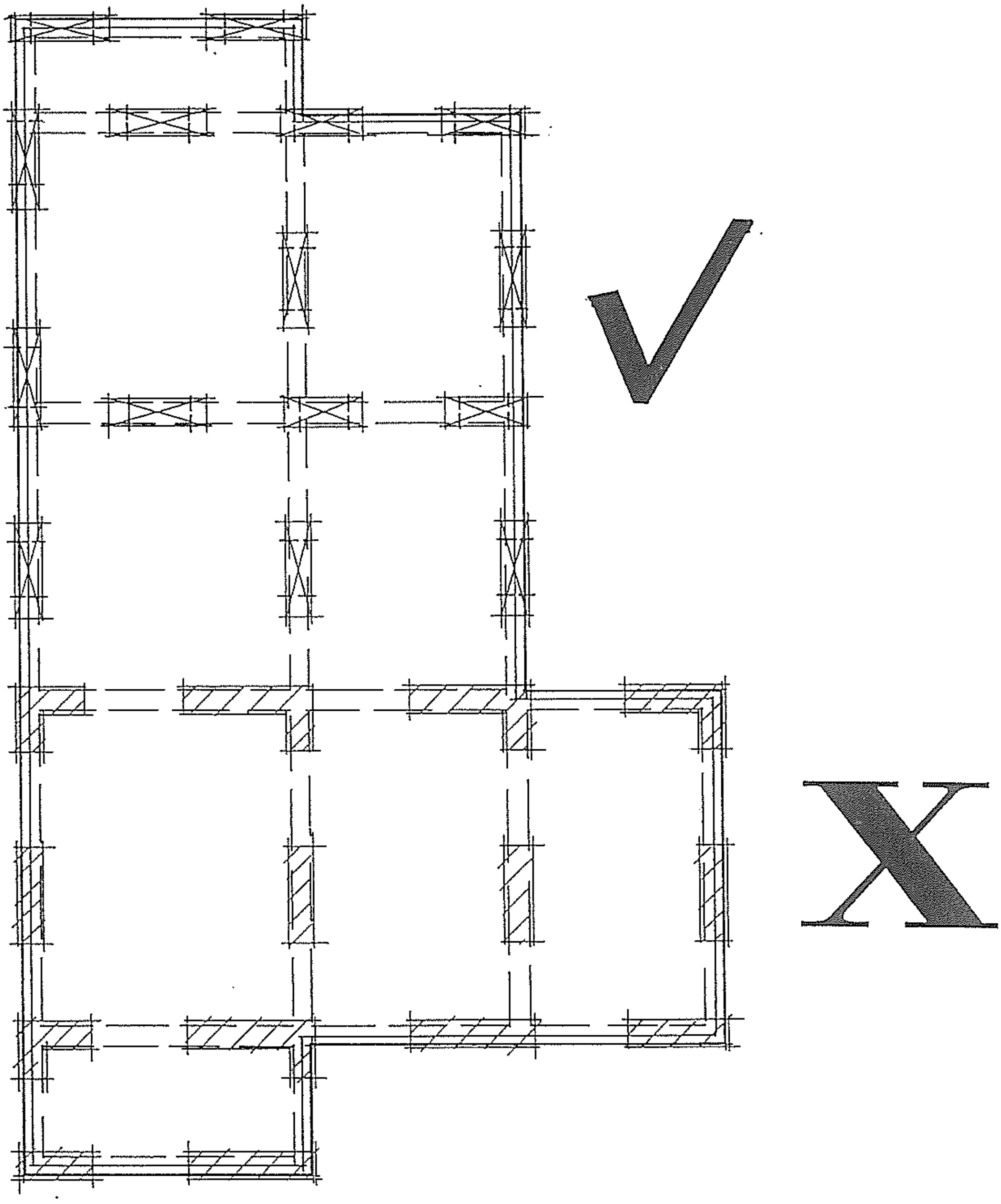
Whether excavated or drilled piers are used in expansive clay sites they should be designed to resist and reduce heave. This can be more easily achieved with drilled piers by lining them with a double layer of plastic and most importantly by installing an effective drainage system around the perimeter of the building.

Generally speaking excavated piers use about 40% less concrete than continuous strips but twice as much as drilled piers.

Low strength concrete may be used where soil heave is not a problem.

Therefore, in summary, excavated piers are viable in "M" or less reactive sites and particularly in supporting strip and stump footings. They may be used in reactive clay sites but only with great care and good drainage.

EXCAVATED PIERS --- Typical Designs.



* TREATED TIMBER PILES Vs STEEL SCREW PILES.

Here we have more footing systems which are ideal in filled sites but are to be used with care in basaltic and other highly expansive clay sites.

In filled sites these systems are comparable except that in some cases where there is a strong founding material (unless this is rock) directly under the fill the screw piles achieve the required Working Load at a shallower depth than the driven piles. Driven timber piles are, mor often than not, more economical than steel screw piles and reinforced concrete piers.

Driven treated pine piles are about $25-$30/metre.
Unreinforced 600 mm diameter piers are about $35-$40/metre
Steel screw piles are about $40-$50/metre.
Reinforced 600 mm diameter concrete piers are about $60-$70/metre

If the fill is very poor and/or there is water seepage the drilled piers will require some type of casing. The cheapest is the ribbed plastic ("pier former") at about $35/metre for the 600 mm diameter tubes.

Both the screw and driven timber piles are all tested as they are driven and do not rely on calculations from a few strength tests. A foundation investigation for both driven piles and screw piles should be deeper and more comprensive than for normal footings. **If screw piles are to be used the soil acidity should be also be tested.**

Thousands of houses and factories have been built with treated pine piles in Victoria since 1975. I have inspected only two such footings that developed problems. The pile life treated to Hazard 5 (20kg/m^3) is expected to be over 100 years. Galvanized steel screw piles have been around for a few years and have their application. Their longevity greatly depends on the steel thickness, the quality of galvanizing and the soil pH.

They are made of tubular steel with usually one or two helixes and in comparison with driven treated pine piles they are about 50% dearer to a depth of about 6 metre then come closer in cost to the timber piles beyond this depth.

The screw piles are easy to join whereas the joining of timber piles is difficult and unreliable unless considerable time and money is spent on the connection method.

Both piles have problems in our basaltic clay plains west and north of Melbourne because of the presence of impenetrable, shallower rock "floaters".

* ADJUSTABLE STUMPS AND COMPOSITE FLOORS.

These are two moderately new innovations which make timber floors more appropriate in "H" sites and sloping sites.

Economical adjustable stumps have been around for 5 or 6 years. Some are made out of concrete and some from very lightweight galvanized steel. New and more efficient adjustable stumps are on the horizon.

These stumps are being marketed to meet the demand for an easy method to level timber floors. The new A.S. 2870 will state that provision should be made to allow for the adjustment of timber floors in expansive clay sites.

Of course floors with Non-adjustable stumps are still fairly easily levelled provided there is sufficient sub-floor space.

Adjustable stumps will also solve the problem in sinking floors where there are some other causes for the settlement.

Composite floors have been tested and marketed in the past 4 years in Australia. The L.P.41 floor marketed by Ply Timber Association of Queensland and the SUPA floor marketed by the U.S.L. Group.

Although different, these floor systems aim at the same outcome i.e. an economical long-span timber floor with a shallower profile than a conventional long-span floor system.

They achieve this by using "T" beams and "L" beams in combination with elostomeric glues and sheet ply flooring.

The SUPA floor is under patent and has been used in a number of houses in Melbourne. it is ideal for "H" or "P" sites ,where it is an advantage to reduce the number of floor supports.

All long-span timber floors have an advantage in "H" and "P" sites since they reduce the number of supports and reduce the floor curvature should there be any foundation movement.

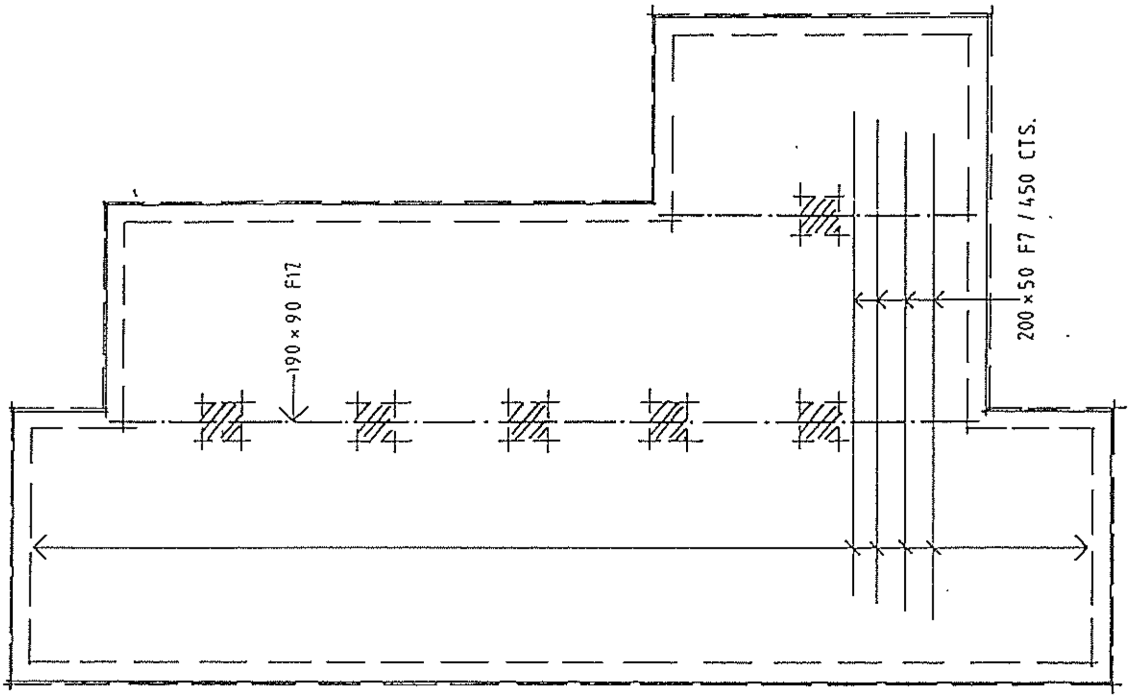
They are also ideal as a combination with adjustable stumps, driven timber or screw piles or piers.

This reduction in floor supports makes the use of adjustable stumps/piles economically viable.

It may even lead to the development and use of the "**Self Levelling Floor**"

Typical comparison of the number of supports required in conventional long-span timber floors and composite floors.

Conventional LONG-SPAN floor.



SUPA FLOOR.