DESIGN OF RESIDENTIAL FOOTINGS BUILT ON EXPANSIVE SOIL USING PROBABILISTIC METHODS

RANGANATH BARTHUR

B.E (CIVIL)

A THESIS SUBMITTED FOR THE DEGREE OF
MASTER OF ENGINEERING
SCIENCE

The University of Adelaide
Department of Civil and Environmental Engineering

January, 1997

To my parents Sreenivasa Murthy and Rajalakshmi

and my wife Suma

Contents

List of Figures

Chapter 2. Literature Review

Chapter 3. Compilation of Data

Chapter 4. Development of Probabilistic Design Charts

Chapter 5. Application of Probabilistic Design Methodology

List of Tables

Chapter 2. Literature Review

Chapter 3. Compilation of Data

Chapter 4. Development of Probabilistic Design Charts

Chapter 5. Application of Probabilistic Design Methodology

Abstract

Expansive soils are clays which undergo large shrinkage and swelling movements as a result of changes in subsoil moisture. Previous methods for the design of footing systems to minimise distortion and cracking in buildings constructed on expansive soils have been based on idealised mathematical models. These models inherently simplify the complex nature of expansive soil behaviour. An alternative *probabilistic design approach*, based on data derived from many built and tested footings is proposed, and has a number of significant advantages over traditional deterministic methods.

Relevant data required for this project were obtained from six local councils within the Adelaide metropolitan area. These data include: current owner's name and address; building type; raft footing details and engineer's soil classification and borehole details. The probabilistic approach enables the level of risk associated with each individual design to be quantified, whereas the current deterministic design methods give no indication of the associated risk. A probabilistic approach enables the design engineer and the client to make informed decisions regarding the desired level of risk and the economic cost, which is likely to reduce the possibility of future litigation. This thesis focuses on stiffened raft footings built on Slightly (S), Moderately (M) and Highly (H) expansive sites as defined by the Australian standard for the design of residential foundations, AS2870.1-1988 (Standard Associations of Australia, 1988a).

A series of design charts are developed which relate crack widths, and associated probabilities of exceedence to standard raft footing. It has been found that the probabilistic design approach, while not intended to replace existing deterministic methods, provides a valuable design tool. The technique enables the designer and client to make rational decisions based on probabilities of cracking and associated costs of footing construction.

Acknowledgments

I wish to thank my supervisor Dr. Mark Jaksa, of the Department of Civil and Environmental Engineering for his invaluable time, patience, guidance and his encouragement in the development of this research and thesis. In addition, I wish to thank Dr. Richard Jarrett, of the Department of Statistics, for his patience and assistance. Valuable assistance given by Dr. Peter Mitchell is also acknowledged.

I wish to express my gratitude to the Department of Civil and Environmental Engineering and in particular, Dr. Stephen Carr, Computing Officer, for all the help provided during the course of this project.

I wish to express my heartfelt thanks to Dr. Valli Rao for accepting to proof read this report. In addition, the help and support given by Dr.Seshadri and family, Dr. Rao and family, Mr. Ramesh and family, Mr. Prasad Krishna and family and Mr. Badrinath (Dhool) and family is sincerely acknowledged. The continuous help and support extended by Mr.Eugene White is also sincerely acknowledged.

I also wish to thank the assistance and friendship offered by my fellow postgraduates, Gautam Chari and Matthew Burnet, in particular, who provided valuable assistance.

I acknowledge the support of the Australian Research Council for providing partial funding for this project. In addition, I thank the support given by the Councils of Campbelltown, Enfield, Happy Valley, Marion, Prospect and Tea Tree Gully and all the house owners for their kind participation in this study.

Statement of Originality

This work contains no material which has been accepted for the award of any other degree or diploma in any university or other tertiary institution and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference has been made in the text.

I give consent to this copy of my thesis, when deposited in the University Library, being available for loan and photocopying.

Signed:

Date: ~~8-8~~ 8-1-97

Notation

Throughout the thesis, the following terms refer to the definitions presented below.

A = Soil Activity;

ABV = Articulated Brick Veneer;

ASB = Articulated Soild Brick;

a, b = Curve Fitting Parameters;

a_s = Depth of the Suction Change;

b_b = Breadth;

C = Clay Content;

C_1; C_2 = Walsh's Design Factors;

C_w = Climatic Factor;

CF = Cumulative Frequency;

c_m = Moisture Characteristic;

c_r = Ratio of Support;

E = Young's Modulus;

ECW = Equivalent Crack Width;

e = Edge Distance;

e_f = Final Void Ratio;

e_o = Initial Void Ratio;

G_s = Soil Specific Gravity;

H = Highly Expansive;

H_r = Relative humidity;

h = Total Suction;

h_m = Matrix Suction;

h_s = Solute Suction;

h_t = Free Energy;

I = Second Moment of Area;

I_{pt} = Instability Index;

K = Constant;

k = Swell Stiffness;

k_o = Subgrade Modulus;

L = Thickness of Soil layer, Length of a Structure;

M = Moderately Expansive;

M_o = Moment at the Centre of the Footing;

m = Mound Exponent;

m_c = Zero Cracked Houses;

m_s = Mean of the Sample;

m_w = Molecular Weight of Water;

n_c = Non zero Cracked Houses;

PC = Probability of Occurences;

PI = Plasticity Index;

POE = Probability of Exceedence;

p_o = Overburden Pressure;

p_s = Swell Pressure;

p_f'' = Negative Pressure;

R = Universal Gas Constant;

r = Coefficient of correlation;

S = Slightly Expansive;

S_p = Swell Potential;

s = Sample Size;

t = Constant;

V = Shear Force;

W = Width;

W_c = Maximum Crack Width;

W_d = Driest Moisture Content;

W_w = Wettest Moisture Content;

x = Horizontal Distance;

Y = Maximum Differential Heave;

y = Difference in Elevation;

y_m = Initial Differential Movement, Design Value of Differential Movement;

y_s = Vertical Surface Displacement;

Z_o = Depth of Seasonal Ground Movement;

z_b = Constant;

Δh = Seasonal Heave;

ΔL = Displacement of Individual Soil Layer;

$\Delta \mathrm{p}$ = Applied Load;

Δu = Suction Change;

ΔV_o = Volumetric Heave;

Δw = Change in Moisture Content;

δ = Footing Displacement;

ε_v = Vertical Strain;

ω = Load per Unit Area;

σ'_f = Effective Pressure;

Chapter One

Introduction

1.1 INTRODUCTION

Damage to structures, which are built on soils which shrink and swell with change in the sub-soil moisture, is a problem that is not uncommon throughout the world. These soils are known as expansive soils. An increase in soil moisture causes swelling of the clay, whereas a decrease in moisture, causes shrinkage, both of which result in vertical movements of the soil layers. Furthermore, the movements are accompanied by strong uplift forces which can cause damage to structures. If the foundation movements are uniform throughout the building, distortion of the building is not affected. However, due to variation of the soil profile or soil moisture over small lateral distances, differential movements occur, which can compound footing performance.

This expansive soil problem is not only a concern for engineers and home owners of Australia, but also for people throughout the world. According to relatively recent studies in the United States, it is estimated that damage losses to man-made structures, due to expansive soil related movements, is about $7000 million annually (Krohn and Slosson, 1980). These damage losses are greater than those from the sum of all natural catastrophes such as earthquakes, cyclones and landslides.

This problem is also very common throughout various parts of Australia, particularly Adelaide. In South Australia, considerable sums of money are spent on the repair of structures damaged by expansive soil movements. Even though stringent local government regulations are enforced in South Australia, some of the structures built on expansive soil still undergo distortion and cracking within the first few years of construction. This distress can be tolerated if it falls within the acceptable limits as specified by the relevant Australian standards for residential footing construction, AS2870.1-1988 and AS2870.2-1990 (Standards Association of Australia, 1988a and 1990).

The majority of houses which are built on expansive soils in Australia, are either based on the Mitchell (1979) or Walsh (1984) methods. These techniques utilise highly idealised two-dimensional and deterministic models to describe soil-footing interaction. Even though these procedures are very useful for the design of residential footings, inherent uncertainties in the methods may sometimes lead to unsatisfactory footing designs. An alternative and, perhaps more logical, course of action, under these circumstances, is to revert to an empirically-based, probabilistic design technique.

1.2 AIMS AND SCOPE OF THE STUDY

This research aims to investigate the effects of expansive soils on residential footings and to produce probabilistic design charts based on a survey conducted in different regions of metropolitan Adelaide. The probabilistic method provides the design engineer with some guidance to the level of risk associated with the design process. Furthermore, this technique aims to provide a more realistic approach to the footing design by incorporating the behaviour of actual footings tested under a variety of site conditions. This study extends the work originally proposed by Kay and Mitchell (1990) and, in the process, seeks to eliminate the following limitations that existed in their research:

- The data required for the study was confined to only one local council area, thus limiting the number soil types;
- Due to time constraints and other reasons, the survey was restricted only to the exterior of the dwellings.

In this study, a survey was carried out in six different local council areas of metropolitan Adelaide, thereby incorporating a number of different soil types. The six different councils selected were:

- Campbelltown;
- Enfield;
- Happy Valley;
- Marion;
- Prospect; and
- Tea Tree Gully.

This study focuses on the design of raft foundations built on slightly, moderately and highly expansive sites as given by AS2870.1-1988 (Standards Association of Australia, 1988a). Visual inspections were carried out for the exterior as well as the interior of the dwellings.

1.3 THESIS LAYOUT

This thesis has been undertaken with the aim of producing probabilistic design models for residential footings built on expansive soil. Chapter 2 deals with the literature review dealing with recent developments in the field of expansive soil engineering. In addition, footing types, superstructure and current design practices are discussed as background, which is extended in later chapters.

Chapter 3 describes the compilation of the data which was carried out in two stages over a period of 18 months. The first stage involved the compilation of data from the six different local councils. The second stage involved the visual inspection of those houses who responded to a questionnaire inviting the owners to participate. The resulting information was then stored in a database specifically developed using *MS Access* 1.1® software. In addition, this chapter discusses and compares various databases developed by a number of other researchers in geotechnical engineering.

In Chapter 4, analyses are discussed which are based on the methodology proposed by Kay and Mitchell (1990). These analyses have been performed using the data detailed in Chapter 3 and normalisation and data smoothing operations. In addition, this chapter contains various probabilistic design charts developed from the results of these analyses.

In Chapter 5 the probabilistic design charts are compared with footing designs proposed by the Mitchell Method, as implemented by the program *SLOG*, and deemed-to comply designs recommended by AS2870.1-1988 (Standards Association of Australia, 1988a). A number of different case studies are presented and compared.

Finally, summary and conclusions are presented in Chapter 6, along with areas for future research.

It should be noted that in mid-1996, the Australian Standard AS2870 was updated (Standards Association of Australia, 1996). While the previous version of this code was used in this study, the 1996 edition does not contain any modifications that would significantly affect the results of this research.

Chapter Two

Literature Review

2.1 INTRODUCTION

Expansive soils, that is, those which shrink and swell with changes in moisture content, are a problem encountered throughout the world. Structures built on such soils may undergo considerable damage. This chapter reviews the progress that has been made in the design of residential footings built on expansive soil. In addition, this chapter contains essential background knowledge which will be developed throughout the thesis.

The chapter begins with a discussion of the nature and formation of expansive soils, and the occurrence of these soils in the metropolitan area of Adelaide, South Australia. Following this, the chronological development in footing design and the type of construction used to limit structural distortion is discussed. A critical review of the current residential footing design practice is presented, in addition to treatment of probabilistic methods which are gaining wider popularity in the field of geotechnical engineering. The summary at the end of the chapter provides a justification for the proposed research.

2.2 EXPANSIVE SOILS

A soil which undergoes some degree of volume change as a result of moisture changes is termed expansive. Expansive soils are mostly well defined clay layers containing minerals which react to changes in moisture. The clay particles are plate-like and are extremely fine grained, having a particle size of generally less than 0.002 mm. For illustrative purposes, if one imagines a coarse grained sand that is the size of a cricket ball, a clay particle would then be the size of a pinprick. This section examines the nature and formation of expansive soils, the different soils of Adelaide, different methods of quantifying the reactivity of soil, factors which influence the heave, fundamental concepts in expansive soil mechanics and the Australian standard used to design residential footings.

2.2.1 Nature and Formation of Expansive Soils

Soils are surface deposits formed by the weathering of rocks and sediments. Weathered material is transported by wind, water, or downslope movements and redeposited in a different form, often over great distances. These are known as transported soils and most of the soils which cause problems for domestic footings in South Australia are in this group (Selby, 1979).

The three predominant minerals which influence the expansiveness of soils, in order of increasing reactivity, are:

- Kaolinite;
- Illite; and
- Montmorillonite.

Kaolinite consists of alumina sheets joined to silica sheets by a hydrogen bond. This bond is relatively strong, and as a result, water is unable to penetrate between the layers giving kaolinite a relatively stable structure. Hence kaolinite exhibits low expansive behaviour.

In contrast, illite consists of alumina sheets between two silicon sheets resulting in a weak bond between layers. Here the layers are held together in a strong bond and, as a consequence, illite exhibits low expansive behaviour, although more expansive than kaolinite.

Montmorillite has a similar structure to illite, with the difference being that the basic units are held together by a weaker bond. As a result, water is able to penetrate easily between the sheets and causes a separation between the layers resulting in the soil swelling several times its volume. Thus montmorillonite exhibits more expansive behaviour than kaolinite and illite.

It follows that the expansiveness of a given soil depends upon the proportion of clay minerals present in the soil mass. An increase in soil moisture causes swelling of the clay which results in vertical movements of the soil layers. This vertical movement, which occurs within the clay mass, is transmitted to the soil surface. Furthermore, the movements are accompanied by strong uplift forces which can cause damage to structures founded on them. Foundation movement, if uniform over a building site, does not cause structural distortion. Differential movements, due to lateral variations of the soil profile or soil moisture can, on the other hand lead to distress in structures built on such sites.

A common cause of differential movement, often found in Adelaide, is the presence of gilgais. These gilgais are formed as the result of extreme wet and dry seasons and frequently show an undulating surface. These undulations are apparently caused by swelling (heave) caused by moisture changes and have given rise to the local term "Bay of Biscay Soils". Gilgai undulations commonly have amplitudes from one to two metres and consist of two types: surface and sub-surface gilgais.

An example of a surface gilgai is shown in Figure 2.1 and are developed within the uniform clay profile of highly reactive soils. Shrinkage of the soil during the dry season forms fissures, which allow non-reactive soil particles to infill these cavities. During the wet season moisture penetrates through the fissures causing expansion of the reactive material, which forces the soil vertically upwards to form a ridge or dome between the fissures.

Sub-surface gilgai, as shown in Figure 2.2, is a type of formation which is very common in the Adelaide region. They are formed because of the presence of deeper reactive clays in the soil profile. The mechanism of sub-surface gilgai formation is similar to that of a surface gilgai formation. Undulations, which are formed at the base of the profile are transmitted to the surface. Sub-surface gilgais cause particular difficulties for domestic foundations as the distortion of the soil profile can make superficially conducted site investigations unreliable.

Figure 2.1 Surface gilgai. (After *Selby*, 1979)

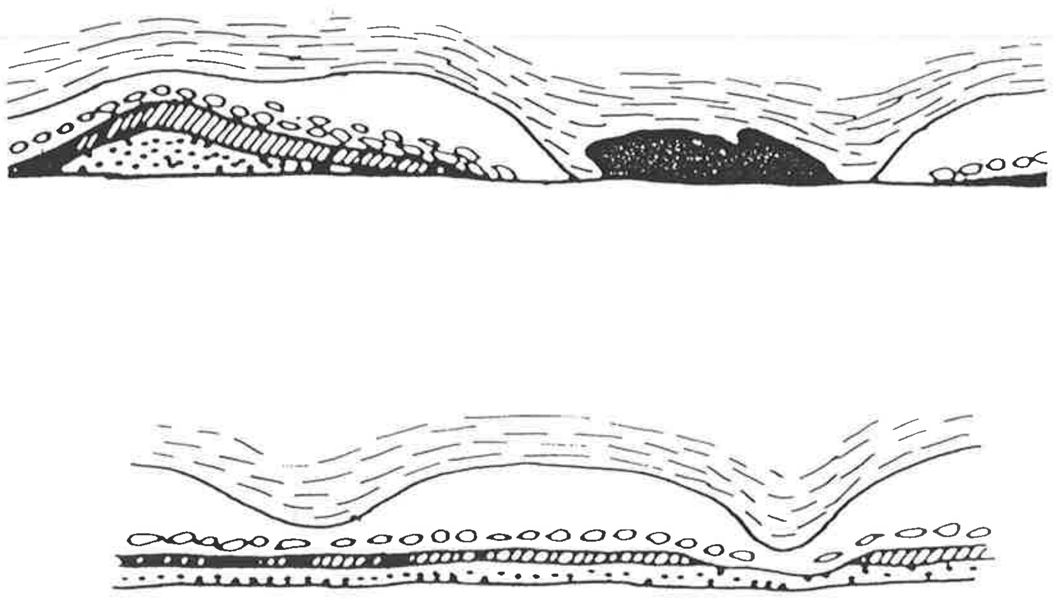


Figure 2.2 Sub-surface gilgai. (After *Selby*, 1979)

2.2.2 Soils of the Adelaide City

The soils of the Adelaide region are divided into 12 groups depending upon their genetic history (Taylor et al., 1974). Each of these groups exhibits a specific soil profile with recognisable horizons according to its mode of formation. These soil groups are tabulated in Table 2.1. Some of the expansive soils which are of concern for footing design in South Australia, are associated with the soil types of *black earth* (BE); *red brown earth, type 3* (RB3); *brown solonised soil* (BS); and *terra rossa* (TR). It is necessary to discuss each of these soil types, as they differ significantly in the magnitudes of their respective shrink/swell potentials. In addition, these soil types will form part of the database of footing performance detailed in Chapter 3.

2.2.2.1 Black Earth

The Black Earth profiles are found in large areas towards the north of the River Torrens at Gilles Plains, Modbury and Hope Valley, as well as in small pockets to the east and south of the city. In Adelaide the Black Earth group consists of an assemblage of dark coloured expansive clay soils which show a wide variety of profile characteristics, related to the origin of the parent material and has been formed on fine grained alluvium subject to slow drainage and periodic wetness (Taylor et al., 1974). Most of the black earths have high shrinkage and expansion properties.

A strongly variable feature of black earth profiles is the content of lime and its depth of occurrence. Due to this irregular and unpredictable occurrence of lime, severe structural distortion can occur. A black earth profile is normally dark grey to black, in its dry state, and extremely sticky, very firm and highly plastic when wet. On drying, black earths develop wide, fissure-like shrinkage cracks, which lead to gilgai, as described previously. Absorption of water is, at first, rapid through the granular surface and passing down shrinkage cracks until swelling of the wetted clay closes them. In all other regions, expect in the calcareous horizon the permeability is low. This condition is intensified if surface drainage around the building allows undue wetness to develop in the calcareous zone. In summary, the Black Earth profile allows large shrinkage and swelling movements in all clay horizons and the soil materials generally have high bearing capacity except where calcareous material is abundant.

Table 2.1 Soil types of the Adelaide city. (After *Taylor et al.,* 1974).

Soil Group	Group Symbol	Soil Type Symbol
Red Brown Earths	RB	RB1
		RB2
		RB3,3a,3b
		RB4
		RB5,5a
		RB6
		RB7
		RB8
		RB9
Black Earths	BE	BE
Rendzina	RZ	RZ
Terra Rossa	TR	TR
Podzolic Soils	P	P1
		P2
		P3
		P4
Solodic Soils	S	S1
		S2
Brown Solonized Soils	BS	BS
Alluvial Soils	AL	AL
Estuarine Muds and Sands	EMS	EMS
Dune Sands	DS	DS1
		DS2
Slope Wash	SW	SW
Skeletal Soils	SK	SK

2.2.2.2 Red Brown Earth Type 3

Red Brown Earths usually occur on smooth, moderate to gentle slopes associated with the drainage systems of the Torrens and Sturt Rivers, and with the very numerous creek lines with undefined channels issuing briefly from the escarpment (Taylor et al., 1974). Aitchison (1956), stated that the most widespread of the shrinkable clay soils occurring in Adelaide is the Red Brown Earth Type 3 (RB3).

The common profile characteristic of RB3, as described by Taylor et al., (1974), are a brown or grey-brown, fine sandy or silty surface soil (A horizons) followed by a well developed, prismatic to angular blocky structure, red-brown clay subsoil (B horizon). The BCa horizon is usually less than one metre and is browner in colour and variably calcareous. This horizon usually merges with the parent material (C Horizon). Large shrinkage and swelling movements occur in the subsoil. The movement in the BCa horizon is smaller than in the B horizon. Subsequently when there is sufficient moisture change, large soil movements can also occur at greater depths.

2.2.2.3 Brown Solonised Soil

This type of soil occurs principally on the western side of the Para Fault Block and its escarpment, from Ingle Farm to South Adelaide. The main spread of the brown solonised soil is the broad band running down from Para Hills to cover the northern city area and continuing past the Torrens Valley to the southern margin of the central city block. This soil varies from a thin layer up to three metres thick, overlying the Keswick and Hindmarsh Clay in most of the city area (Taylor et al., 1974).

Brown solonised soil is highly calcareous and is primarily derived from windblown material. The combination of fine and coarse mineral materials, give rise to a wide variety of soil profiles (Taylor et al., 1974). The common physical characteristic of these different varieties is the zone of lime rich silt and limestone nodules at shallow depth. The soils may be grey, brown or reddish-brown, and may be loose and powdery or firm. The soft lime may be dispersed or may occur in pockets. Hard lime occurs as nodules and lumps in a sandy to clayey matrix, and with the absorption of moisture, this matrix loses its bearing capacity and has the potential for collapse. Physical properties of the brown solonised soil are soil horizons which are subject to small shrinkage and swelling movements. Brown solonised soil frequently overlies clay of highly expansive nature at depths of less than three metres, and is capable of creating a water table in the overlying highly permeable soil.

2.2.2.4 Terra Rossa

In this group, the soils are red or red-brown in colour, and are derived from limestone or other calcareous rocks. Taylor et al. (1974) stated that the depth of these soil is generally shallow, about 450 mm, and they show little development of profile horizons. Deeper profiles may grade to Red Brown Earths which are liable to some shrinkage and swelling movements, as discussed previously. The bearing capacity of those soils and the underlying weathered rock materials, is usually high. Again, powdery forms of lime, if present as a layer, tends to reduce the strength of the soil upon wetting.

2.2.3 Fundamental Concepts in Expansive Soil Mechanics

The fundamental concepts in expansive soil mechanics are classified into three main components, which are: soil suction; Instability Index and moisture content. A brief treatment of each of these is given in the following sections.

2.2.3.1 Soil Suction

Soil suction is a measure of a soil's affinity for water. In general, the drier the soil, the greater is the soil suction. Mitchell, (1979), described that every soil has a property called its state of total suction or free energy h_t, which is a measure of the tendency of the soil to undergo a change in moisture content. Thus, total suction, h_t, is comprised of two components as given by Equation (2.1):

$$h_t = h_m + h_s \tag{2.1}$$

where: h_m = matrix suction;
h_s = solute suction.

Matrix suction is a result of the surface tension of the water "drawing" moisture into the void channels of the soil matrix. Solute suction is due to the osmotic potential of the soil which is dependent on the presence of dissolved salts within the soil.

Mitchell (1979) stated that, when the soil is drier, the matrix suction is higher since the capillary tension is greater, and solute suction is higher since the concentration of dissolved salts is greater. Conversely, when the soil is wetter, the state of soil suction is lower. However, due to the solute suction component, a soil which is very wet may still have a high total suction.

Soil suction are often measured in logarithmic units, pF, because of the high values encountered, and are given by Equation (2.2):

$$u = \log_{10}(h) \tag{2.2}$$

where: u = suction (pF);
h = suction (cm).

Thus, the flow of moisture through a soil mass is governed by the suction gradient within the soil profile, with moisture travelling from regions of low suction to regions of high suction. The total suction, h_t, can be determined by measuring the magnitude of relative humidity, RH, of the enclosed air surrounding a soil sample from Equation (2.3).

$$h_t = \frac{RT}{M}\ln(RH) \tag{2.3}$$

where: h_t = total soil suction;
R = universal gas constant ($0.08207\frac{1}{^{\circ}C}$)
T = temperature ($^{\circ}K$);
M = molecular weight water (18.02 g/mol);
RH = relative humidity.

When the state of suction is measured at intervals of depth down the soil profile, the resulting relation between suction and depth is termed the suction profile. Since the state of suction is dependent on the state of moisture, the suction profile will tend towards equilibrium with a moisture source either at the boundaries or within the soil mass. An example of a suction profile, typical of a semi-arid climate, is shown in Figure 2.3.

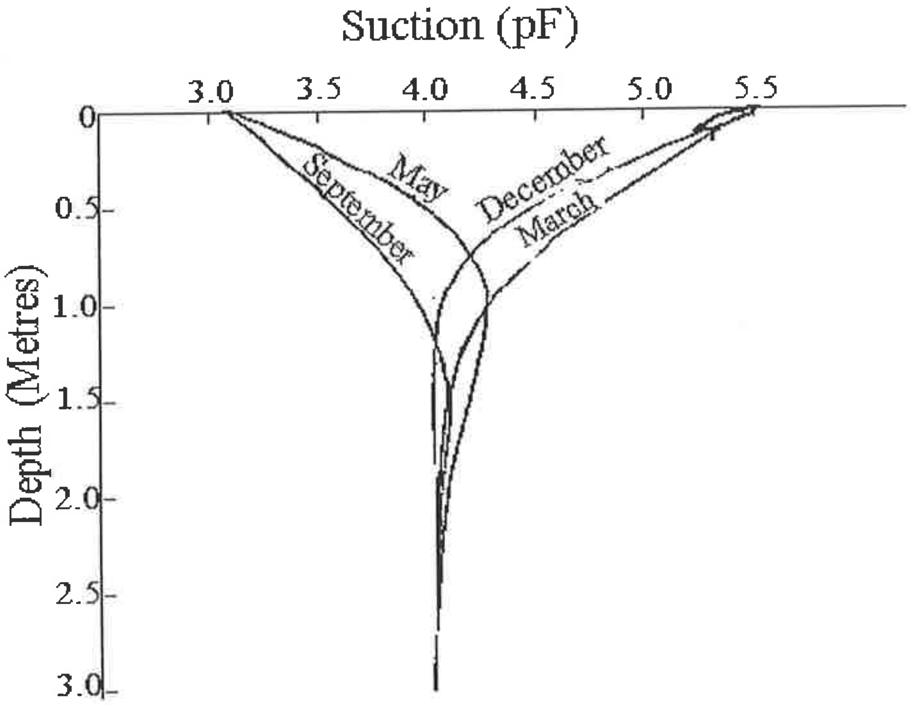


Figure 2.3 Typical soil suction profile for semi-arid climate.

Generally, it will be convenient to express the suction change as decreasing linearly with depth. As a consequence, the suction profile is generally modified as shown in Figure 2.4.

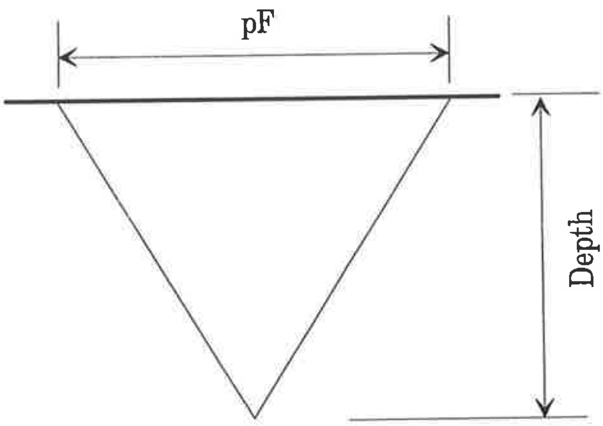


Figure 2.4 Modified suction profile.

Soil suction can be easily measured by pressure and suction plates, vacuum desiccators, psychrometers or calibrated filter papers (Peter, 1979; McKeen, 1980; Wray, 1984).

2.2.3.2 Moisture Characteristic

The relationship between moisture content and suction is termed the *moisture characteristic*, c, and is defined as the ratio of change in moisture content, Δw, to the change in soil suction, Δu, and is shown in Equation (2.4).

$$c = \frac{\Delta w}{\Delta u} \tag{2.4}$$

The moisture characteristic is often expressed in the units of pF.

It has been found that the same value of soil suction leads to different values of moisture content in soils of different textures (Mitchell, 1984). According to Morris and Gray (1976), the more clayey the soil, the higher the value of its moisture content at a given suction. When soils are wetted and dried, considerable hysteresis occurs in the moisture content - suction relationship. Croney (1952) subsequently observed that this hysteresis is smaller for clay than it is for sand. For all practical purposes, the value of c can be taken as a constant defining both the wetting and drying behaviour of the soil.

2.2.3.3 Instability Index

The Instability Index, I_{pt}, is the ratio of vertical strain to suction change. This I_{pt} value has been experimentally observed by Aitchison and Woodburn (1969), Aitchison (1970), and Lytton and Woodburn (1973). The I_{pt} is equivalent to the *suction index* used by Snethen (1980) and Johnson (1979), or the *suction compression index* used by McKeen and Hamberg (1981).

The limitations of this simplified procedure have been summarised by Richards et al, (1984) as follows:

- it assumes that I_{pt} is linear over the possible range of soil suction and soil stress;
- it ignores stress and moisture paths, including hysteresis;
- it ignores solute effects particularly over long periods of time;
- it ignores in situ soil conditions, such as fissures, lateral stresses and long term and large scale effects.

Despite these limitations reasonable results can be achieved provided that some care and judgement are exercised. However, Cameron (1989) described that like other soil mechanics properties, an approximate constant value for I_{pt} can be determined.

Commonly, three soil tests are used to measure I_{pt}:

- Core shrinkage test;
- Loaded shrinkage test; and
- Shrink Swell test.

Details about the tests could be found in various sources (e.g. Cameron, 1989; AS2870.2-1990). Of the above three tests the most reliable test is the shrink swell test. According to Cameron (1989) this is true since this test does not require suction measurement. Cameron (1989) also observed that core shrinkage test is the least reliable test, since this test requires determination of the moisture characteristic.

In addition to the three tests listed above an alternative method for the determination of I_{pt} has been suggested by Mitchell and Avalle (1984), which uses the correlation between Plasticity Index (*PI*) and Instability Index. Figure 2.5 shows the results of 80 samples from 18 sites in South Australia, and three samples from Victoria. In each case the Atterberg limits and pedological classifications of each of the soil profiles have been obtained in an attempt to develop a relationship between *PI* and I_{pt}.

It can be seen from the Figure 2.5 that considerable scatter exists in the relationship between I_{pt} and *PI*. However in general, the higher the plasticity index, the higher the I_{pt}. Furthermore, observed scatter was reduced, when the samples were grouped according to pedological classification. This implies, that once the soil has been pedologically classified, it is possible to estimate the I_{pt} by means of the *PI*. Since the *PI* can be determined reasonably well using the standard field identification toughness test, it is also possible to estimate I_{pt} via the relationship suggested by Mitchell and Avalle (1984). This estimation procedure is known as the *visual-tactile method.* It involves visual and manual inspection of the soil and is very much dependent on the experience of the assessor. The method is widely used in South Australia for the estimation of I_{pt}.

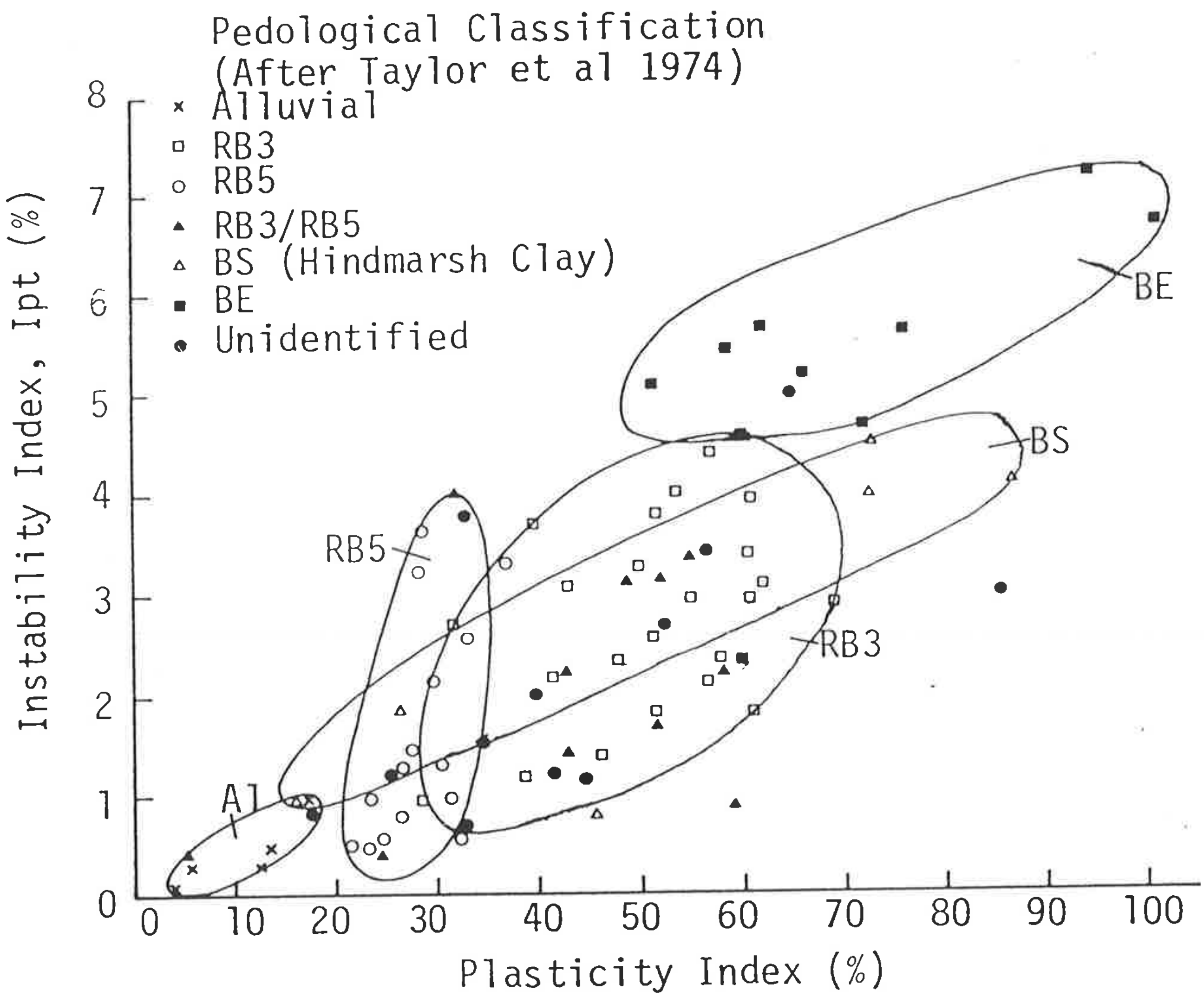


Figure 2.5 Correlation of Instability Index and Plasticity Index. (After *Mitchell*, 1979).

2.2.3.4 Modes of Distress Due to Soil Movement in Super Structure

As soil under a building swells and shrinks relative to surrounding soil, doming and dishing develops in the footing. This doming and dishing of the foundation soil and footing, although it may be only few millimetres, causes the distortion in super structure. When the moisture state in the subsoil under structure is variable across the structure, a differential soil movement occurs. Due to these differential soil movements, two modes of distortion which are relevant to this project have been identified.

When the soil moisture content at the perimeter of the structure is greater than that in the interior, footing will be deformed into a sagged shape. This mode of distortion is subjected to a condition of edge heave. On the contrary when the soil moisture content at the perimeter of the structure is less than that at the interior, footing will be deformed into a hogged shape. This mode of distortion is subjected to a condition of centre heave. Two modes of distortion are shown in Figures 2.6 and 2.7.

Two modes of distortion are easily differentiated by their crack pattern. The crack pattern of the edge heave will be always wider at the bottom and narrower at the top. Alternatively, crack pattern of the centre heave will be always narrower at the bottom and wider at the top. If the structure is of insufficient flexibility making it unable to accommodate the footing distortion arising from these differential soil movements, the structure will be affected. This is usually evident by the development of cracks in the walls of the structure, and the distortion of internal fittings, which can be of a magnitude large enough to make the occupancy of the structure aesthetically unsatisfactory and physically inconvenient. The potential causes for these distortions are:

- Effects of vegetation;
- Poor drainage;
- Extreme dry or wet conditions; and
- Poor irrigation practices.

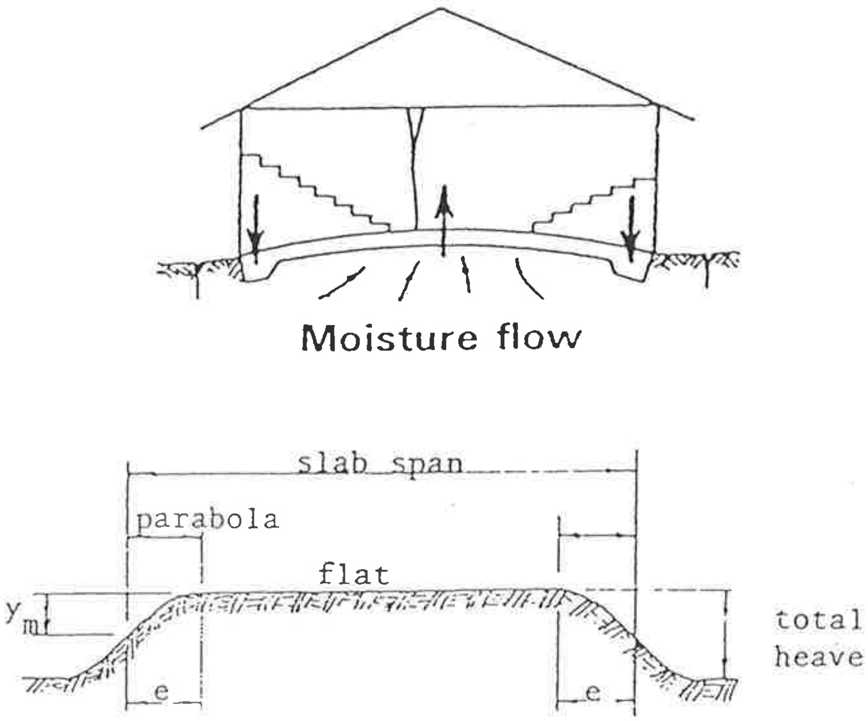


Figure 2.6 Centre heave.

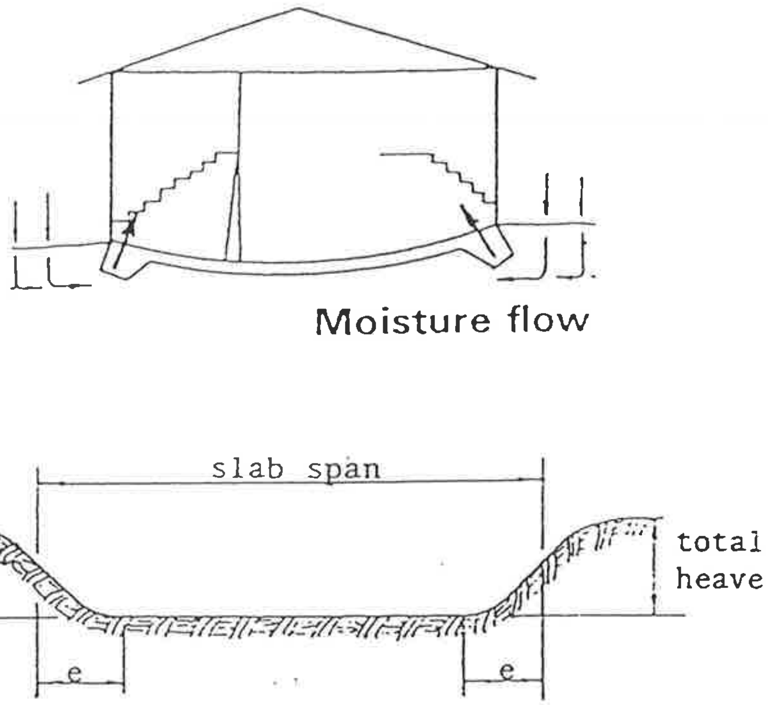


Figure 2.7 Edge heave.

2.2.4 Methods Used to Quantify the Reactivity of Soils

As mentioned before, volume changes or *heave,* in soils are caused by changes in the soil moisture content. This is generally the case in expansive soils. This volume change may be the result of either natural or man-made influences. Experience has clearly demonstrated that more substantial foundations are required at sites where high seasonal heaves occur. The majority of techniques used to design foundations for lightly loaded structures require an estimate of the seasonal heave at the site.

Since the 1950s researchers have endeavoured to develop reliable seasonal heave prediction methods which are similar to those used for clay settlement calculations. These methods can be classified into following four types:

- Empirical;
- Semi-empirical;
- Consolidometer technique; and
- Mathematical moisture flow model.

A brief treatment of each of these is given in the following sections.

2.2.4.1 Empirical Mehods

The main parameter involved in the prediction of heave from an empirical approach is the assessment of swell potential; usually given the qualitative states of low, medium or high. The prediction of heave is made by multiplying the appropriate potential heave strain by the layer thickness and by the depth reduction factor, and summing each of the layer components to determine the total seasonal heave (Holland and Cameron, 1981). Seed et al. (1962) assessed the swell potential as the percentage swell of a laterally confined sample which had been soaked under a surcharge of 7 kPa after compaction to its maximum standard density at its optimum moisture content. The authors then proposed an empirical relation between swell potential, soil activity and clay content. This relationship is shown in Equation (2.5).

$$S_p = 0.000036\, A^{2.44}\, C^{3.44} \tag{2.5}$$

where: S_p = swell potential;
A = soil activity;
C = clay content.

Van der Merwe (1964) used a similar approach to Seed et al. (1962) but instead of swell potential he has used plastic index, clay content and activity to predict the heave potential.

Ranganathan and Satyanarayana (1965) argued that the logical parameter for assessing swell potential is the shrinkage index of the soil. The authors then proposed a relationship between swell potential and clay content and it is shown in Equation (2.6).

$$S_p = KC^t \tag{2.6}$$

where: S_p = swell potential;
K = constant;
C = clay content;
t = constant.

Holland and Cameron (1981) found that swell potential is a poor predictor of heave and should only be used to indicate the likely degree of heave potential. In addition, Holland and Cameron (1981) found that the site climate, environmental influences and unsaturated effective stresses are ignored while assessing the swell potential.

2.2.4.2 Semi-Empirical Methods

Semi-empirical methods are based on generalised heave/swell pressure curves derived from a series of laboratory tests. Such an approach was first presented by McDowell (1956) and were based on tests performed on clay samples remoulded at various combinations of moisture and density. He then derived the generalised form of heave/swell pressure curves, shown in Figure 2.8. These pressure curves were characterised by the magnitude of volumetric heave, ΔV_0, experienced by a representative clay sample under free swell conditions. To enable evaluation of ΔV_0, he then suggested empirical correlations between it and shrinkage limit, shrinkage ratio and moisture content.

The basic assumption of this method is that it is universally applicable to all soils under all conditions. With the help of the heave curve, direct readings are made of volumetric clay expansion to saturation under a vertical restraining pressure. He also assumed that linear strain is approximately equal to 33 percent of the volumetric strain. Subsequently, seasonal heave is calculated by summing the incremental movements throughout the soil profile.

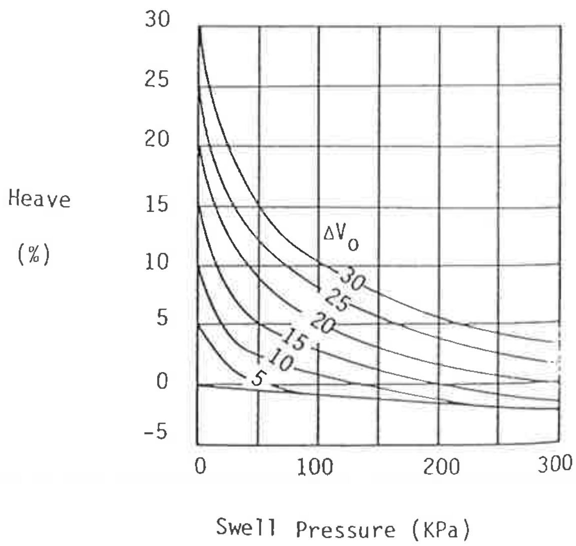


Figure 2.8 Heave/swell pressure curves. (After *McDowell*, 1956).

2.2.4.3 Consolidometer Technique

The consolidometer technique can be achieved by using either a direct or an indirect approach. The US Army Corps of Engineers (1961) used the direct method to predict the seasonal heave. Their approach requires undisturbed samples to be taken, when the soil profile is in its driest state. The samples are then placed in the consolidometer and allowed to swell to saturation under a vertical applied pressure which is equivalent to in situ overburden pressures.

Jennings and Knight (1957) introduced an effective stress approach which accounted for the influence of soil suction on the vertical stress. The method proposed by Jennings and Knight (1957) involves the following steps:

1. The in situ soil conditions are represented on the "adjusted natural moisture content (NMC) curve", as shown in Figure 2.9.

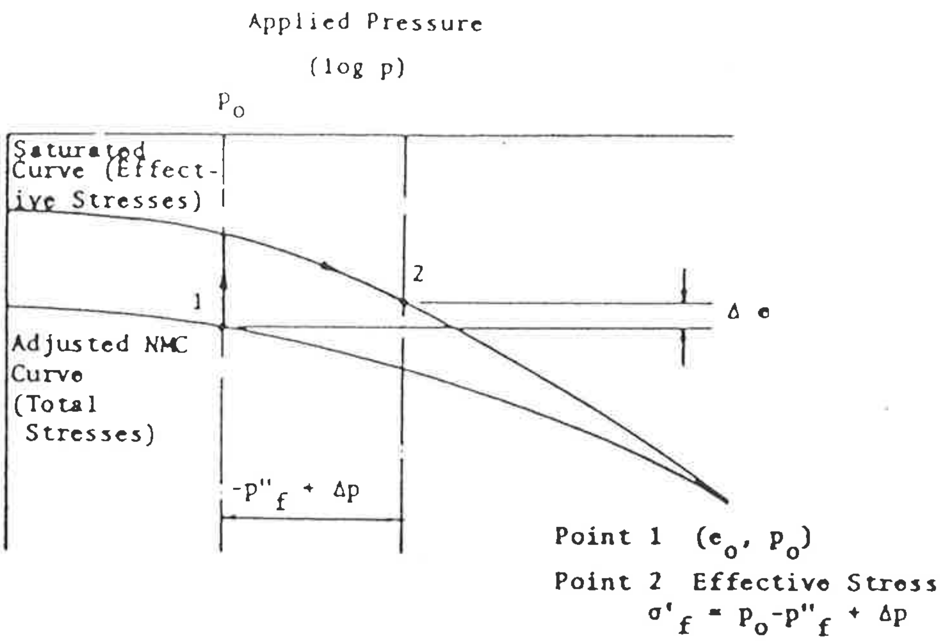


Figure 2.9 Adjusted natural moisture content curve.
(After *Jennings* and *Knight*, 1957).

$$\sigma'_f = p_0 - p''_f + \Delta p \tag{2.7}$$

where: σ'_f = effective pressure;

p_o = overburden pressure;

p_f'' = negative pressure, which results from the sample being wetted to near saturation;

Δp = applied load.

2. The effective pressure may be approximated using the Equation (2.7).
3. The final void ratio e_f is read from the saturated consolidation curve at the pressure, σ'_f.

4. The difference, Δe, between e_o (in situ void ratio) and e_f is then determined.
5. Finally, the seasonal heave, Δh, is determined by using Equation (2.8):

$$\Delta h = \sum_{i=1}^{N} \left(\frac{\Delta e}{1+e_o}\right)_i H_i \tag{2.8}$$

where:

Δh	= seasonal heave;
Δe	= change in void ratio between the pressures;
e_o	= in situ void ratio;
H_i	= thickness of the ith soil layer;
N	= total number of soil layers contributing to the vertical heave.

In contrast, Clisby (1963) preferred to use an indirect consolidometer technique and he considered only total stresses in estimating the seasonal heave. From a series of tests he found that the swell curve of a soil can be defined by the e-log(p) curve obtained from the progressive unloading of the swell pressure, p_s. An undisturbed sample is placed in a consolidometer and preloaded to restore the original lowest in situ void ratio, e_o. The sample is then flooded and allowed to swell for 24 hours, after which time a consolidation-rebound test is performed. The graphical representation of the consolidation test is shown in Figure 2.10.

It is evident from the figure that the consolidation curve is commenced at an overburden pressure p_o, at a point A. A straight line through A, parallel to the rebound curve is drawn, which represents the swell curve of the sample. Where this line intersects the in situ void ratio, e_o, at point B, the corresponding pressure represents the "no volume change pressure", p_s. The seasonal heave is then estimated using the Equation (2.8).

The major disadvantage of this technique is its extremely long testing period, which makes it both tedious and costly to use.

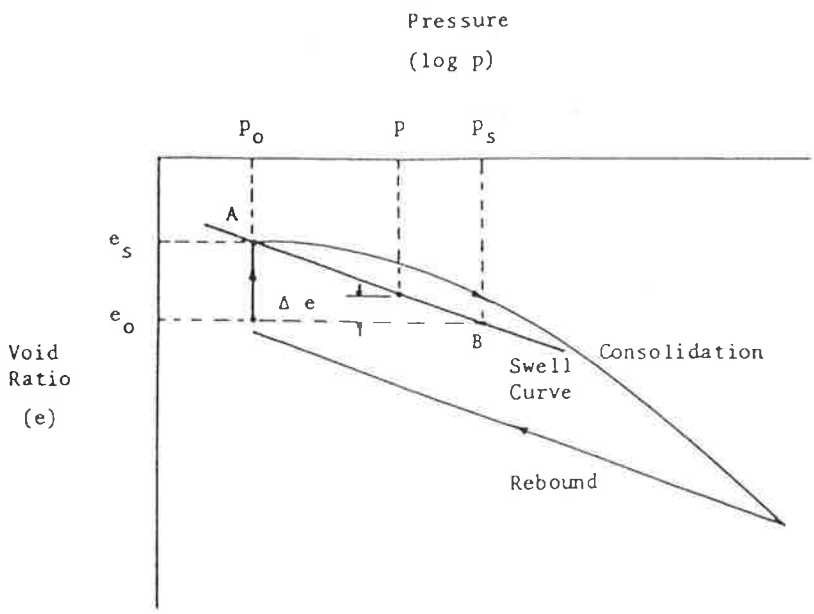


Figure 2.10 Graphical representation of consolidation test. (After *Clisby*, 1963).

2.2.4.4 Mathematical Moisture Flow Models

In this approach the prediction is based on solving an unsaturated diffusion equation using mathematical principles. Richards and Chan (1971) solved the diffusion equation by a finite difference technique. By entering the initial suction, end boundary conditions and the soil permeability constants, they determined the soil suction profile. Heave values were then estimated by converting suction profiles into moisture contents using experimentally determined relationships for each clay type encountered. Finally the vertical heave, Δh, was estimated by means of Equation (2.9):

$$\Delta h = \frac{1}{3}\sum_{i=1}^{N}\left[\frac{(w_w - w_d)G_s}{100 + w_d G_s}\right]_i \Delta H_i \tag{2.9}$$

where:

	Δh	= vertical seasonal heave;
	w_w	= seasonally wettest moisture content (%);
	w_d	= seasonally driest moisture content (%);
	G_s	= specific gravity of solids;
	ΔH_i	= thickness of the ith soil layer.

Equation (2.9) assumes that the linear strain is one third of the volumetric strain and the soil voids are filled with water, and is valid for only saturated and quasi-saturated soil moisture conditions. While using Equation (2.9) for unsaturated soil conditions such as high suctions or low moisture content, significant error may be introduced. This is the major limitation of this method.

Later Richards (1973a, b, c; 1974) developed a finite element approach to solve unsaturated soil conditions. A similar approach was adopted by Lytton and Watt (1970) to solve the diffusion equation by using suction as a moisture variable and again assuming linear heave to be one third of the volumetric heave. The major limitation of their approach is that the critical correlation between specific water volume and total specific volume has been formulated on the basis of the behaviour of a very limited number of clays (Holland and Camron, 1981).

Johnson and Desai (1975) developed a similar technique which involved solving the unsaturated diffusion equation using suction as the moisture variable, but employed a finite difference technique. Suctions were converted into moisture contents using an empirical relationship chart developed by Black (1962), and shown in Figure 2.11. Then the surface heave was estimated by a technique which involved the approximations of both McDowell's and Lytton and Watt's approaches.

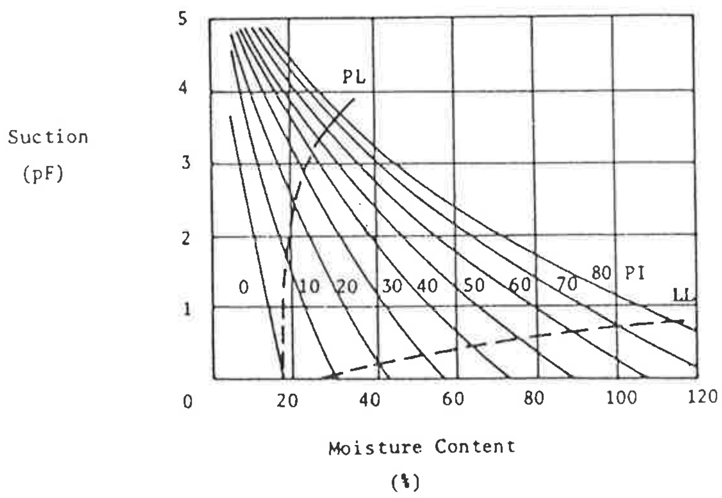


Figure 2.11 Relationship between soil suction and moisture content. (After *Black*, 1962).

2.2.4.5 Summary

The major conclusions given by Holland and Cameron (1981) relating to these general heave prediction methods are as follows:

- The empirical and semi-empirical methods of heave prediction, which are generally based on clay soils, are of little value;
- Consolidometer methods, employing full sample saturation, do not accurately model the real seasonal moisture conditions and tend to over estimate heave, unless the necessary undisturbed clay samples are taken during the seasonally driest time. More accurate values can be determined using more complex methods based on the Modified Oedometer technique (Peter, 1979). The major disadvantage of this method is its high relative cost; and
- The simple mathematical moisture flow model of Richards (1967) and expressed in Equation (2.10) considers both soil profile and annual extremes of soil moisture. It is presently the most satisfactory method of heave prediction for design purposes.

$$SH = \tfrac{1}{3}\sum layers\left[\frac{(w_w - w_d)G_s}{100 + w_d G_s}\right]\Delta H \tag{2.10}$$

where:

	SH	= seasonal soil heave of the soil layers with individual thickness of ΔH;
	w_w	= seasonally wettest moisture content;
	w_d	= seasonally driest moisture content;
	G_s	= soil specific gravity.

However, Equation (2.10) does not take into account the effects of the applied loads on the soil thus this method may be more realistic, if it is used for relative shallow soil profiles.

Another method which is gaining greater popularity in predicting soil movement is based on the concept of soil suction (Mitchell, 1980). Predictive techniques using soil suction appear to be more reliable because they account for many of the fundamental concepts associated with the behaviour of expansive soil. Central to this method is the Instability

Index, I_{pt}, discussed previously in §2.2.3.3, is defined as the ratio of vertical strain ε_v to the suction change, Δu. Thus:

$$I_{pt} = \frac{\varepsilon_v}{\Delta u} \tag{2.11}$$

Since

$$\varepsilon_v = \frac{\Delta L}{L} \tag{2.12}$$

combining and rearranging Equations (2.11) and (2.12), yields:

$$\Delta L = I_{pt} . \Delta u . L \tag{2.13}$$

Thus the total vertical surface displacement, y_s, of the soil profile is the sum of the displacements of the individual soil layers undergoing a suction change Δu.

Hence

$$y_s = \sum_{i=1}^{N} I_{pt_i} \Delta u_i L_i \tag{2.14}$$

where: I_{pt_i} = instability Index of layer, i;

Δu_i = total suction change over layer, i;

L_i = thickness of soil layer, i.

2.2.5 Factors Which Influence Heave

The main factors which influence the magnitude of seasonal heave can be divided into two broad categories:

- Geotechnical characteristics and site conditions; and
- Environmental influences.

The environmental influences are further classified into natural effects such as tree root activity and normal seasonal effects, and human influences such as garden watering, leaking underground water services and deficient stormwater drainage system. This section briefly discusses each of the factors which affect heave.

- **Type and Amount of Clay**

The major factors which govern the potential for volume change are: the composition of the clay minerals present within the soil mass; its surface chemistry; the fabric of a particular clay; and the salt concentration. Holland and Cameron (1981) stated that, for engineering purposes, the type of clay mineral present in a soil mass is indirectly determined by the use of standard soil index tests, such as linear shrinkage or plasticity index tests and are often empirically related to heave.

- **Soil Profile**

The thickness and location of potentially expansive clay layers in the soil profile considerably influences the seasonal heave. If the expansive clay is overlain by a layer of non-expansive topsoil, or overlies bedrock at a shallow depth, the swelling of the clay will be greatly reduced (Holland and Cameron, 1981).

- **Climate**

Generally, expansive soils will only shrink swell and if the prevailing climatic conditions lead to significant seasonal wetting and drying. As a result, seasonal heave will be greater in semi-arid climates where short wet and long dry periods lead to substantial moisture changes in the soil.

- **Moisture Variations**

Since heave is directly related to suction changes within the subsurface profile, any mechanisms which affect the subsoil moisture regime will consequently influence the heave. Such mechanisms include:

- Site drainage;
- Leaking services, such as water mains, sewer and storm water drainage pipes; and
- Irrigation practices which cause an excess, or a deficiency, in the subsoil regime.

- **Tree Root Activities**

Trees have the ability to dry out soils within the zone of influence of their lateral root system (Ward, 1948; Hammer and Thompson, 1966; Burn and Penner, 1975). It is commonly accepted that trees may dry clay soils via their root system at lateral distances of up to one and a half times the tree height if planted in line or in a group (Holland, 1981). Thus, drying out of the soil in the vicinity of vegetation alters the pattern of seasonal movements by extending the drying cycle. The extent of desiccation is controlled by the type of vegetation, the location and distribution of the vegetation, and finally, the stage of vegetative growth and its possible concurrence with drought conditions. Even grass, by itself, has been shown to extend the drying cycle of some soils (Russam and Dagg, 1965).

Apart from the above factors other in situ phenomena which influence the seasonal heave to a lesser extent are dry density, stress history, particle orientation, magnitude of horizontal soil stresses and permeability.

2.2.6 Standard for the Design of Residential Foundations on Expansive Soils

In response to requirements by the geotechnical and structural engineering profession, Standard Australia published the "Residential Slabs and Footings" code of practice: AS2870.1-1988 and AS2870.2-1990 (Standard Association of Australia, 1988a and 1990).

The standard bases footing design on a *site classification*, which is, in turn, based on the free surface movement, y_s. The value of y_s is a site characteristic and is the amount of total movement at the surface due to moisture variations from the design dry state to the design wet state. These design conditions may include considerations of the influence of the house and a reasonable garden on the site, as well as seasonal and climatic influences.

Due to the variation of climatic and other environmental influences the standard uses a statistical definition of y_s; that is, the value that has a 5% chance of being exceeded in the life of the house (taken as 50 years). While determining free surface movement, the effects of trees, man made effects such as garden watering, leaking underground water services and deficient storm water drainage system are not considered.

AS2870.1-1988 (Standard Association of Australia, 1988a) recommends one or more of the following methods while classifying a site:

1. A site can be assessed in accordance with Table 2.1 of AS2870.1-1988 (Standard Association of Australia, 1988a) and are reproduced in Table 2.2. Visual assessment and interpretation of existing masonry building walls on light strip footings which have existed for no less than 15 years in a similar soil is the main criteria for this method.

Table 2.2 Simple classification of sites. (After AS2870-1988).

Characteristic Performance of Masonry Buildings	Site Classification
Rare Category 0 or 1 damage	S
Often category 1 damage but rarely category 2 damage	M
Often Category 1 and 2 with occasional examples of Category 3 damage or more severe	H
Often Category 3 or more severe damage and area is usually well known for damage to houses and structures	E

2. Sites may be classified on the basis of the strength of the soil. The strength of the soil can be estimated using either penetrometers or from the simple field rules given in Table C2.1 of the AS2870-1988a (Standard Association of Australia, 1988) and is reproduced in Table 2.3. However, this method may not be suitable for sites having highly variable soil types.

Table 2.3 General summary of the site classification. (After AS2870-1988).

Soil Type	Physical Characteristics	Classification
Rock	Strongly cemented sand or gravel	A
Sand or Gravel	Medium dense sand or gravel	A
	Loose sand	A, S or P
Silts and Clays	Very soft clay or silt	P
	Soft clay or silt	S, M, H, E or P
Slightly Reactive Clays	Shallow and less plastic	S
Moderate to Highly Reactive Clays	Deeper and more plastic	M or H
Extremely Reactive Clays	Deep cracks occur in ground	E

3. A site can also be classified on the basis of the predicted surface movement, y_s. AS2870.2-1990 (Standard Association of Australia, 1990) recommends that the method of calculation of y_s be based on the Instability Index and soil suction. The Instability Index can be estimated using any of the methods described previously in §2.2.3.3, whereas the soil suction is estimated using the appropriate values given Table 2.4. For the purpose of designs, the site classifications are related to the calculation of y_s is shown in Table 2.5.

Table 2.4 Wet and dry extremes for soil suction at different locations. (After AS2870.2-1990).

Location	Change in suction Δu (pF)	Depth H (m)
Adelaide	1.2	4.0
Melbourne	1.2	2.0
Sydney	1.5	1.5
Hunter Valley	1.5	2.0
Brisbane	1.2	1.5
Albury	1.5	3.0

Table 2.5 Site classification. (After AS2870.1-1988).

Surface Movement (mm)	Class	Foundation
$y_s \leq 20$	S	Slightly reactive
$20 \leq y_s \leq 40$	M	Moderately reactive
$40 \leq y_s \leq 70$	H	Highly reactive
$y_s \geq 70$	E	Extremely reactive

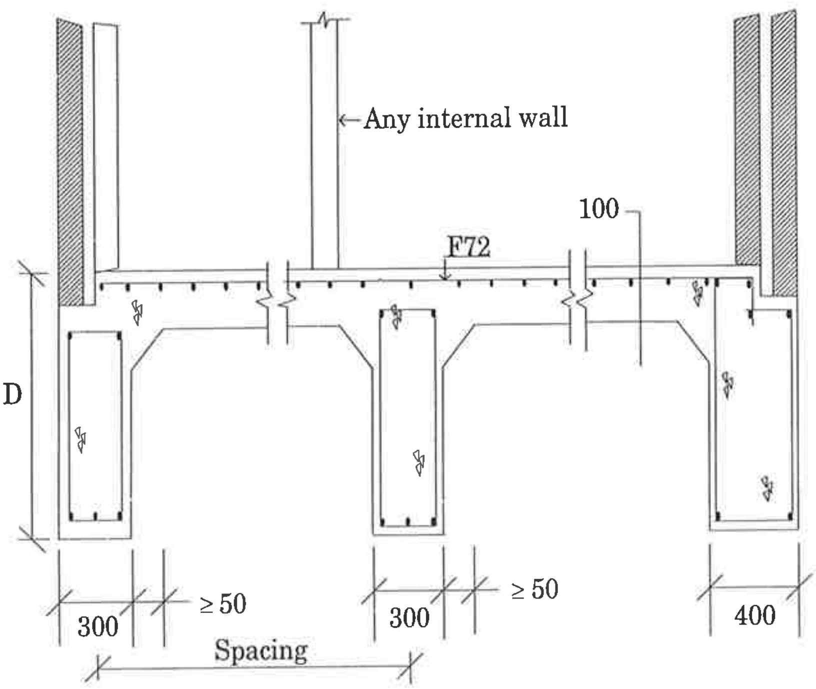
For South Australia extremely reactive sites are further sub-divided into:

- E1 = Extremely reactive 1 where 70 mm $\leq y_s \leq$ 100 mm
- E2 = Extremely reactive 2 where $y_s >$ 100 mm

2.2.7 Standard for Footing Design

The footing systems shall be designed in accordance with either of Sections 3, 4 and 5 of AS2870.1-1988 (Standard Association of Australia 1988a) for expansive soil sites, classified as described in the preceding sections. Design of footing systems on slightly reactive sites shall comply with Section 3 of the standard, while the Section 4 concentrates on moderately and highly reactive sites, and Section 5 relates to extremely reactive sites. AS 2870.1-1988 (Standard Association of Australia 1988a) points out that the Section 5 applies to the region where there is well established local knowledge of Class E sites, deep clays and semi-arid climate and have significant effect on building. These regions include Adelaide and its environs.

As a guide a stiffened raft footing system for a reactive site can be designed either of the two ways. These approaches are referred to as, *deemed-to-comply standard footing system design* and *design by engineering principles*. The former method is based on an empirical approach that consists of selecting the depths of stiffening beams for the raft directly from the Figure 5.1 of the AS2870.1-1988 (Standards Association of Australia 1988a). These are reproduced in Figure 2.12 and showing depth and reinforcement details. Selection of depths of stiffening beams are made on the basis of house flexibility and site reactivity.



(a) Articulated brick veneer Edge beam (b) All internal beams (c) Articulated solid brick

All dimensions in mm

Site Class and Type of Construction	Edge and Internal Beams			Beam Spacing (m)
	Beam Depth (mm)	Reinforcement		
		Top	Bottom	
Class M Site				
Clad frame	400	2-Y12	3-Y12	6.0
ABV	400	2-Y12	3-Y12	4.0
ASB	625	2-Y12	2-Y16	4.0 (internal beam)
ASB	625	3-Y16	3-Y16	4.0 (edge beam)
Class H site				
Clad frame	500	2-Y12	3-Y12	4.5
ABV	500	3-Y12	3-Y12	4.0
ASB	800	3-Y16	3-Y16	4.0 (internal beam)
ASB	800	4-Y16	4-Y16	4.0 (edge beam)
Class E1 site				
Clad frame	625	2-Y16	2-Y16	4.0
ABV	800	3-Y16	3-Y16	4.0
Class E2 site				
Clad frame	800	3-Y16	3-Y16	4.0
ABV	1000	3-Y16	4-Y16	4.0

Figure 2.12 Standard for stiffened raft design. (After AS2870.1-1988).

The raft beam sizes shown in Figure 2.12 were determined using Mitchell Method of analysis. Thus this method indirectly adopts the same ideology used by the Mitchell Method.

The second approach involves applying one of the various mathematical techniques developed by a number of researchers (e.g. BRAB, 1968; Lytton, 1970 and 1971; Walsh 1974, 1978 and 1984; Mitchell, 1979). The general procedures for the design of stiffened raft using engineering principles should take the following steps into account (AS2870.2 1990).

- The characteristic surface ground movement, y_s, is estimated from the site classification or in accordance with Appendix D of AS2870.2 1990.
- The design value of differential movement, y_m, is then estimated by taking into account the moisture conditions at the time of construction and the influence of the footing system and edge paths on the design moisture conditions. In the absence of more accurate calculations y_m can be taken as:
  - (i) 0.7 y_s for centre heave;
  - (ii) 0.5 y_s for edge heave on an initially dry site; and
  - (iii) 0.3 y_s for edge heave on an initially wet site.
- The live loads and the dead loads are assessed in accordance with AS1170.1 and AS1170.2.
- The following load and foundation movement combinations should be used:

  Dead load + 0.5 (live load) + foundation movement.
- Relative differential movement limits for houses are taken from the Table 2.6 in the absence of specific information.

Table 2.6 Limits of relative differential movements.

Type of Construction	Deflection Ratio	Max Differential Ratio
Full Masonry	1/2000	10
Articulated Full Masonry	1/800	15
Masonry Veneer	1/600	20
Articulated Masonry Veneer	1/400	30
Timber Frame	1/300	40

- The structural moments can then be determined by computer analysis of the soil footing interaction. The recommended methods of computer analysis are the Walsh or Mitchell methods. The relevant structural elements can then be determined using conventional reinforced concrete design methods.

2.3 FOOTING TYPES

The footing is that part of the house which is in direct contact with the soil. The footing type required for a house will depend on the type of construction, floor system and drainage. Since the 1940s, the following footing types have been developed in order minimise structural distress resulting from expansive soil movements:

- Strip Footing;
- Pier and Beam Footing;
- Deep Tee Beam Footing; and
- Stiffened Raft Footing.

Each one of these footing types is discussed in the following sections.

2.3.1 Strip Footing

A strip footing consists of reinforced-concrete beams, rectangular in cross section, which support the external and internal walls of a house. This type of footing is used in relatively stable soil and is normally used in conjunction with timber floor and has been in use in Australia since the 1920s. Principal features of this type are: beams are usually seated on stable layers and which loosely form a structural grid; trenches are excavated to a designed level and the sides are formed above the ground level. A typical cross section of a strip footing is shown in Figure 2.13.

The classical approach used to design strip footings is based on limiting the bearing pressure to an allowable value by selecting an appropriate width of the beam (Woodburn, 1979; AS2870.1-1988a). Selby (1984) stated that the combination of timber floor and strip footing proved unsuitable in areas of expansive soil as ventilation beneath the floor can cause long term drying out and shrinkage of the foundation. A disadvantage of this type of footing is that when the foundation is unstable to a substantial depth, it is unlikely to resist movements and is uneconomical on deeply unstable soils (Woodburn, 1979).

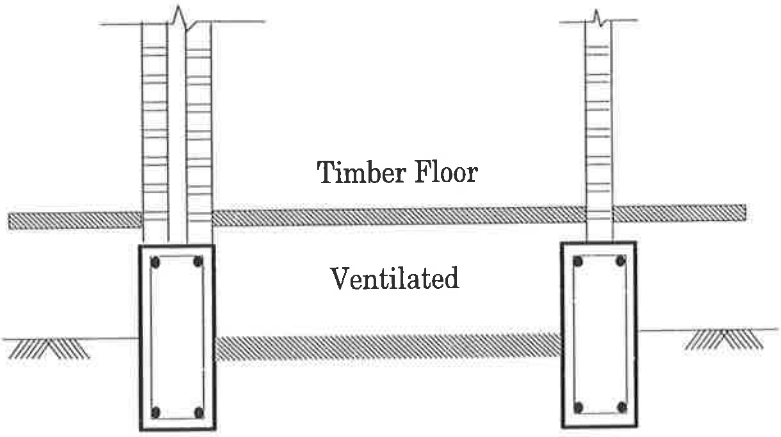


Figure 2.13 Cross section of a strip footing.

2.3.2 Pier and Beam Footing

In the 1950s this type of footing was introduced to overcome the problem of cracking due to the light strip footings used for the interior walls and when it is necessary to isolate the structure from foundation movements. This type of footing normally used in conjunction with timber floors and is suitable for use in the following situations:

- On cut and fill sites;
- In areas of deep reactive soils;
- In areas where surface layers are soft; and
- Loose or reactive soils are underlain by more stable soil.

The principal features are: beams are set clear of the soil on deep seated pier and piers are seated on relatively stable soil. A typical cross section of a pier and beam footing is shown in Figure 2.14.

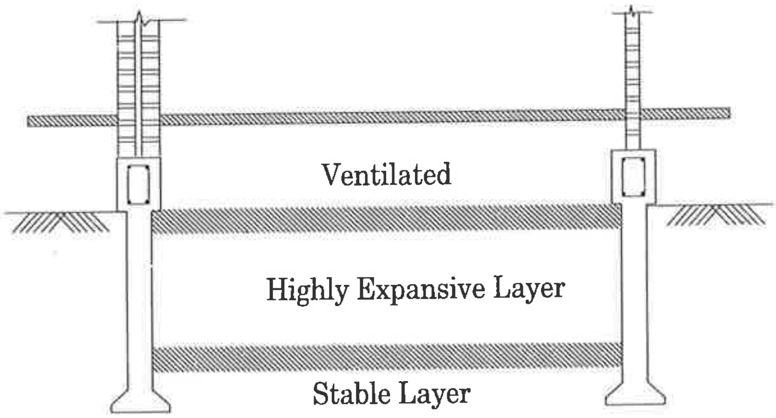


Figure 2.14 Pier and beam footing.

Details regarding the design procedure for pier and beam footings may be found in Woodburn (1979). Disadvantages of this type of footing are:

- When the level the of stable soil is very deep, the cost of construction is high;
- Dwarf walls, which supports the floor are likely to distort due to the movements in surface soil;
- If the base of the pier is seated on unstable soil any resulting movements may cause distress; and
- The method of construction is comparatively difficult.

2.3.3 Deep Tee Beam Footing

This type of footing consists of narrow, deep reinforced-concrete tee beams. With the construction of deep beams, very large stiffnesses can be achieved (Mitchell, 1984). This type of footing can be used in reactive soils in conjunction with timber floors and the beams should form a structural grid. The site is usually levelled and then trenches are excavated to the required dimensions by a trenching machine. Concrete is poured separately from the excavated section to the tee head, but are connected by reinforcement. This type of footing can be used in either cut and fill sites or filled sites. It is desirable to ventilate the space between the floor and ground level. The main limitations of this type are: the subsoils must be capable of excavation by trenching machine and, in highly reactive sites, problems still arise due to side thrust and adhesion on the beams. A typical cross section of this type is shown in Figure 2.15.

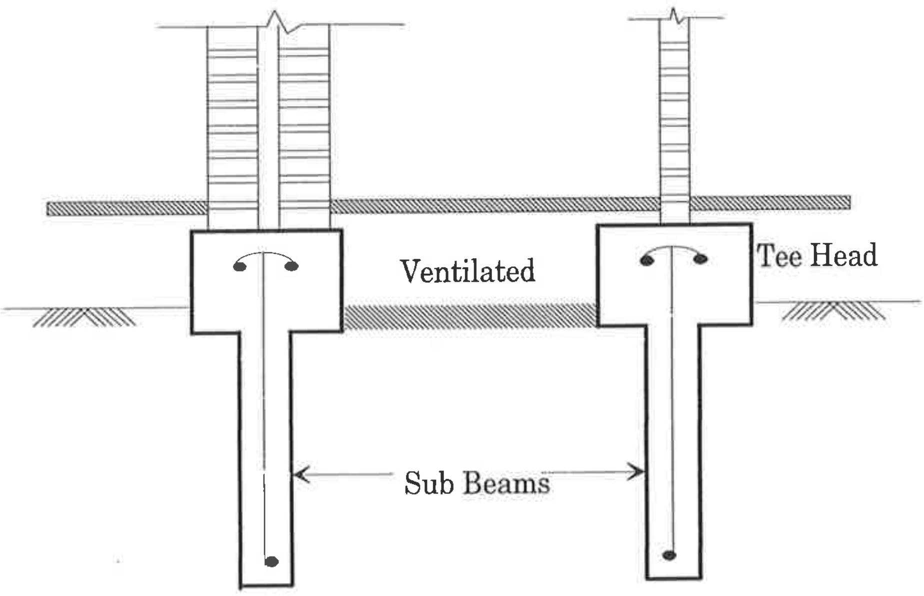


Figure 2.15 Deep tee beam footing.

2.3.4 Stiffened Raft

In the 1960s it had become apparent that in many situations, particularly in relation to expansive soils, the footing types in current use provided poor resistance to structural distress. In response to this the stiffened raft footing was developed, and since then, has become the most popular footing for residential construction in south Australia.

The stiffened raft footing system consists of reinforced concrete sub-beams arranged to form a grid or grillage. These sub-beams are cast integrally with a floor slab to form the footing system of high strength and stiffness. As described previously, it is this type of footing which will form the basis of this research. The main types of stiffened are:

- Standard Raft; and
- Grillage Raft.

2.3.4.1 Standard Raft

The standard raft footing consists of a grid of reinforced concrete sub-beams cast integrally with the reinforced concrete slab. This type of footing is used on compacted soil in conjunction with a reinforced concrete floor. The site is generally be levelled prior to excavation of the beams and a vapour proof barrier is laid prior to placing of reinforcement. The footing beams and floor slab are usually poured in one operation. A typical cross section of this type is shown in Figure 2.16.

There are many techniques available for the design of this type of footing and these techniques are discussed in following sections. Walsh (1984) stated that the advantage of stiffened rafts in poor foundation areas is the utilisation of its strength and stiffness in reducing differential movements. The disadvantages of this type of footing are:

- It does not prevent movement on unstable soils;
- Floors are likely to heave if the reactive soil is dry at the time of construction; and
- Inadvisable on soil with a high water table.

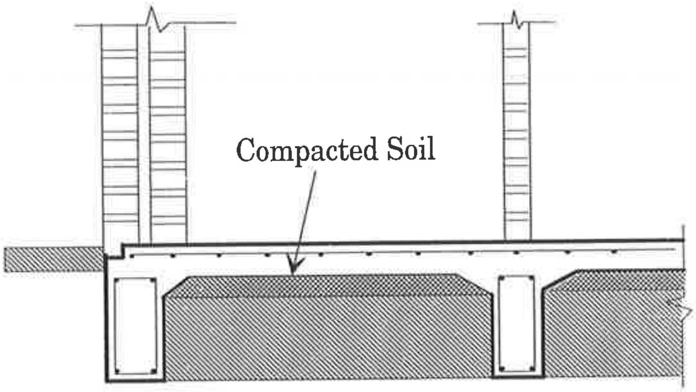


Figure 2.16 Typical cross section of a standard raft.

2.3.4.2 Grillage Raft

The grillage raft footing consists of narrow and deep reinforced concrete sub-beams integral with a reinforced concrete slab. A typical cross section of this type is shown in

Figure 2.17. As for the standard raft, the site is generally levelled prior to beam excavation and a vapour barrier is laid prior to placement of reinforcement. Excavation of the beams is performed by a trenching machine. The slab and the sub-beams are poured separately usually on subsequent days. The advantages of using this type of footing are:

- The moisture is maintained at a uniform level;
- Deep narrow beams provide both structural and moisture barrier benefits; and
- The structural stiffness is greater than that of a standard raft due to increased depth of the beams.

The main disadvantages of this type are:

- It is only possible if the site can be trenched;
- If the soil is dry at the time construction some degree of underfloor heave may occur; and
- It is not advisable for high water table.

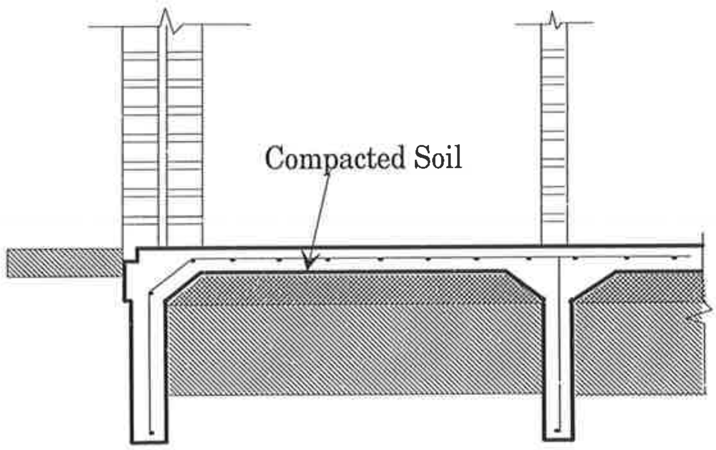


Figure 2.17 Typical cross section of a grillage raft.

2.4 BUILDING CONSTRUCTION

The type of wall construction and interior finish can be important in masking the visual evidence of cracking caused by foundation movement in problem soils. In order to rationalise the footing design procedures for the types of structures normally used in domestic and light commercial construction, it has been necessary to classify the various construction types in terms of their response to deformation. The type of wall construction generally determines the flexibility of the super structure which, in turn, determines the ability of the building to withstand footing movement without cracking. The following section briefly discusses the various types of construction in common use for residential structures.

2.4.1 Super Structure Classification

In domestic construction there are at least five recognised types of construction, as shown in Table 2.7, in order of increasing flexibility. Deflection ratio is the ratio of the maximum displacement between two points divided by distance between them. In addition, it is the ratio that defines the deformation caused within the structure as a consequence of footing movements.

Table 2.7 Different types of super-structure. (After *Woodburn*, 1979).

Wall Construction	Allowable Deflection Ratio	Flexibility
Solid masonry	1/2000 min	Brittle
Articulated masonry	1/800 to 1/1000	
Brick veneer	1/500 to 1/800	
Articulated Brick veneer	1/300 to 1/600	
Timber clad or Pre fabricated	1/200 to 1/300	Very flexible

2.4.1.1 Solid Masonry

This structural type has masonry walls throughout, often internally plastered and with continuous masonry over doorways and windows. A solid masonry house, when compared with a timber framed or masonry veneer house, has three advantages which distinguish it from the others:

- Superior fire resistance;
- Higher thermal capacity; and
- Better acoustic properties.

However, a distinct disadvantage of this type of construction is that it readily displays signs of distress or cracking as a result of relatively small movements. Footing movements of the order of 5 mm will generally cause some cracking over doorways and windows (Woodburn, 1979). However these problems can be overcome, to a certain extent by providing articulation, or control joints.

2.4.1.2 Articulated Solid Brick

In this type of construction, the walls are separated into distinct sections which are free to move relative to one another. This can be achieved by the use of full height door and window frames. Vertical control joints may be formed in the brickwork in both the interior and exterior of the building, which may be visible. Such door openings break the structure internally into a number of separate masonry panels, each of which would rarely exceed 5 m in length (Woodburn, 1979). This type of construction has evolved as a means of controlling super-structure movements to parts of the structure where they will be mainly unnoticed, such as over window and door frames.

2.4.1.3 Brick Veneer

In this type of construction a brittle external masonry skin and flexible timber framed interior are combined. The allowable deformation of this type of structure is considerably higher than that of the solid masonry types because movements which occur within the structure rarely show. Due to the ever increasing demand for low cost housing the use of flexible super-structures on lighter footings is becoming more wide spread.

2.4.1.4 Articulated Brick Veneer

This type of structure has a fully articulated masonry skin and plasterboard-lined interior. The method of articulation is the same as discussed in §2.4.1.2. Articulation of the internal plasterboard skin is also carried out by leaving joints in the plasterboard over some windows and doorways and covering these with battens and beads.

2.4.1.5 Timber Framed or Prefabricated Construction

These structures are the most flexible and can accommodate large foundation movements with minimal distress. Footings often consist only of pads or supports to raise the structure off the ground.

2.5 CURRENT PRACTICE OF FOOTING DESIGN ON EXPANSIVE SOILS

There are a number of methods presently being used for the design of residential rafts on expansive clays. The following sections summarise the more common methods employed.

2.5.1 BRAB (Building Research Advisory Board) Method

The BRAB (1957) method of raft footing design was developed in the USA, in the late 1950s and was based on local experience.

This method is based on the following assumptions:

- The footing slab is supported by the soil at the centre of the slab for hogging, and at the edges for the sagging mode of deformation as shown in the Figure 2.18;
- The ratio of support, c is constant for all sizes of slab, and is dependent upon the Plasticity Index, PI and a climatic factor C_w; and
- The weight of the structure is distributed uniformly over the entire area of the slab, giving an equivalent uniformly distributed load.

For these conditions, the following equations were developed for a structure of length, L, width, B, and footing stiffness, EI.

$$M = \frac{WL^2\,B(1-c)}{8} \tag{2.15}$$

$$V = \frac{4M}{L} \tag{2.16}$$

$$\Delta = \frac{ML^2}{6EI} \tag{2.17}$$

Further the rafts are broken down into series of overlapping rectangles. The bending moment and shear force are then calculated for each rectangle using the above equations. Once the bending moment and shear force have been determined the slab can be proportioned using conventional concrete design practice. The BRAB (1962) report recommended that the calculated deflection ratio should be less than values given in Table 2.8.

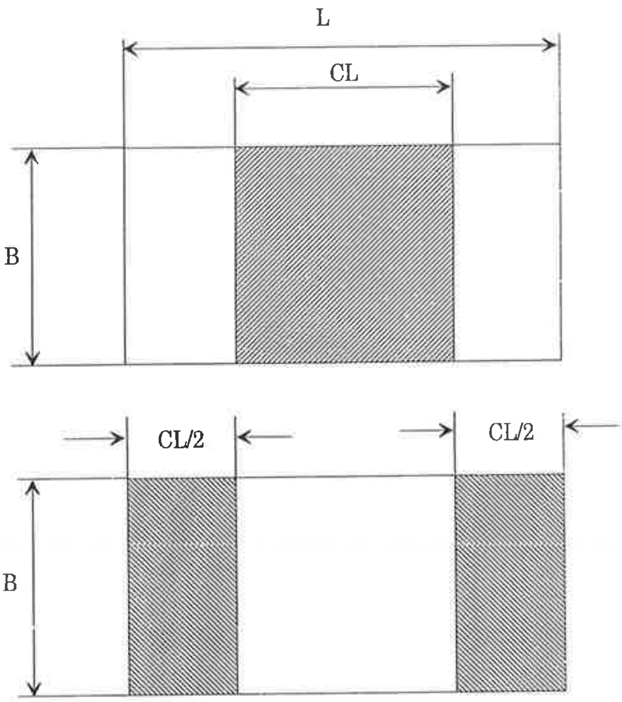


Figure 2.18 BRAB mode of deformation. (After *BRAB*, 1962).

The BRAB (1968) has set an example for other researchers to develop more realistic models. Lytton (1970), Mitchell (1979) and Walsh (1984) all developed their models broadly dependent on the BRAB (1968) approach. All these researchers have tried to reduce the inefficiencies of the BRAB (1968) method. These limitations are:

- The method assumes the support index and design equations are the same in both deformation modes, however, many designers believe the centre heave moment is more than the edge heave moment (Snowden and Meyer, 1976);

- The method gives very conservative beam depths especially for larger rafts;
- The critical relationships between c, PI and C_w are determined empirically, but no explanation for these relationships are available (Holland et al., 1975); and

Since the method was developed in the USA, and because the support index is dependent on a climatic rating, the BRAB method is applicable only within the USA (Wray, 1980; Pidgeon, 1980).

Table 2.8 Permissible differential settlements for stiffened slabs. (After *BRAB*, 1962).

Superstructure Type	Maximum Permissible Deflection Ratio (Δ/L)
Timber Frame	1/240
BV or ABV	1/300
ASB	1/360

2.5.2 Lytton Method

This method (Lytton, 1970) is based on the analysis of a raft on an already formed mound of soil. The critical mound shapes are shown in Figure 2.19, and is given by Equation (2.18).

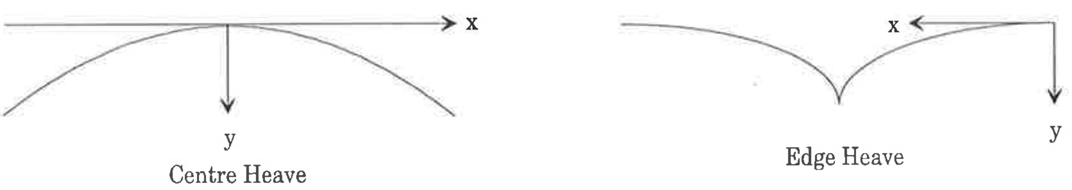


Figure 2.19 Lytton's modes of deformation. (After *Lytton*, 1970).

$$y = kx^m \tag{2.18}$$

where: m = mound exponent;

k = constant;

y = difference in elevation of the soil between the centre and the edge;

x = horizontal distance from the high point.

If the raft is non rectangular it is divided into overlapping rectangles similar to the BRAB (1968) method. Each rectangle is then designed separately. Lytton (1970) originally developed chart solutions for different modes of raft behaviour. Later, Lytton (1971) modified his approach to include a climatic risk factor. He also determined the support coefficient equivalent to BRAB (1968) by assuming the superstructure applies a uniform load over the rigid slab. The following equation for the support coefficient was also derived.

$$c = \frac{m}{m+1}\left[\frac{m+1}{m}\right]^{\frac{m+2}{m+1}}\left[\frac{\omega}{k_o y_m}\right]^{\frac{1}{m+1}} \tag{2.19}$$

where c = support coefficient;

m = mound exponent;

k_o = subgrade modulus(assumed constant);

y_m = initial differential movement of the mound before the slab is applied.

It is evident from the Equation (2.19) that the support coefficient is dependent on the selection of a mound exponent. Subsequently, Lytton (1971) has suggested that the mound exponent could be possibly obtained from:

$$m = \frac{L}{Z_o} \tag{2.20}$$

where: L = slab length;

Z_o = depth of seasonal ground movement.

Woodburn (1979) suggested that, for the Adelaide region, *m* could be taken to equal 4.

With the support and loading conditions determined Lytton (1971) then considered a rigid beam on a Winkler foundation to obtain formulae for the convenient determination of the one-dimensional bending moment, the shear force and the required stiffness of a raft. The one-dimensional bending moments are corrected for the two-way action in a raft design by using empirical factors developed from computer analyses. Once the design moments are calculated using the concrete theory the beam and the slab depths and the reinforcement can be selected. These selections are then compared for their allowable values.

The main limitations of this method are :

- The magnitude of the mound exponent *m*, has a significant influence on the magnitude of the bending moment. Hence, the lower the value of *m* the higher the bending moment; and
- The adoption of a rigid raft on the mound can lead to extremely conservative designs for very large rafts (Holland et al., 1975).

2.5.3 Walsh Method

The Walsh design procedure follows the BRAB (1968) method and is a development of the Lytton (1971) method. The support index, *c*, is derived using the same differential equations as Lytton (1971) method. However, the Walsh method differs from the Lytton method in that it assumes that the raft is placed on the soil before the formation of a mound shape. Also, it assumes that the modulus of subgrade reaction, k_o is under a swelling, rather than a shrinkage.

The initial soil shape for centre and edge heaves were represented by a central flat top and base respectively, with parabolic convex edges over a distance *e*, as shown in the Figure 2.20. Finite element theory was used to solve the differential equations describing the beam on a coupled Winkler foundation. Walsh then developed design factors C_1 and C_2 by carrying out a series of computer analyses for a range of the non-dimensional parameters:

$$\frac{\Delta}{Y}, \ \frac{e}{L} \text{ and } \frac{\omega}{kY}$$

where:

	Δ	= allowable differential deflection;
	Y	= maximum differential heave;
	e	= edge distance;
	L	= footing length;
	ω	= load per unit area; and
	k	= swell stiffness.

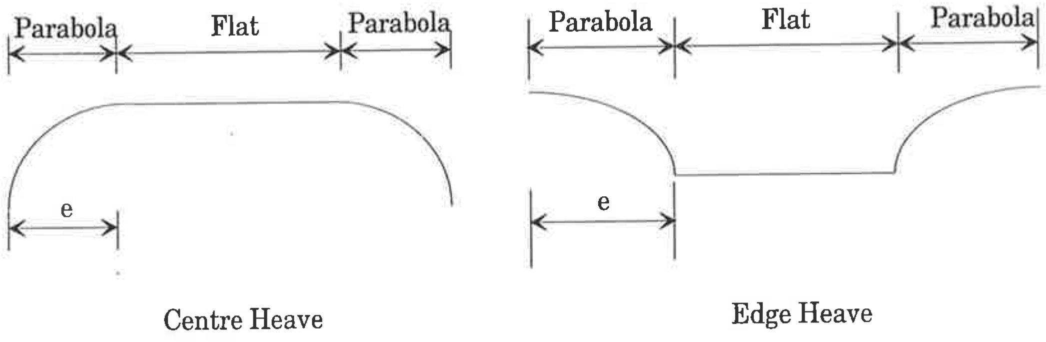


Figure 2.20 Walsh's modes of deformation. (After *Walsh*, 1984).

Walsh (1978) found that shear strength was insignificant and therefore considered only bending moment and stiffness. The results for bending moment and stiffness were defined by:

$$M = (1 - C_1)\ \omega \frac{L^2}{8} \tag{2.21}$$

$$EI = (1 - C_2)\ \omega \frac{L^4}{96\Delta} \tag{2.22}$$

The design factors C_1 and C_2 determined in terms of the non dimensional parameters $\frac{\Delta}{Y}$, $\frac{e}{L}$ and $\frac{\omega}{kY}$.

Walsh (1984) published a revised design procedure for raft footings, a brief summary of which, follows:

Step 1: Determine the expected differential heave y_m, the edge distance, e, and the swell stiffness, k.

Step 2: Calculate the average total long term load, ω, on the foundation over the entire slab.

Step 3: Subdivide the slab into a number of overlapping rectangles and use the larger section in the overlapping regions.

Step 4: Determine the allowable differential movement, Δ, using Table 2.9.

Table 2.9 Allowable deflection.

Type of Superstructure	Allowable Deflection (Δ/L)
For Masonry Structures without Articulation	1/1000
For Brick Veneer and Articulated Masonry	1/500
For Timber and Clad Houses	1/250

Step 5: Determine the support indices C_1 and C_2 using the values for $\frac{\Delta}{Y}$, $\frac{e}{L}$ and $\frac{\omega}{kY}$.

Step 6: Using Equations (2.21) and (2.22) calculate the moment, M, and stiffness, EI. The section properties are then determined in accordance with conventional reinforced concrete design.

Step 7: Arrange the design for the entire slab so that where the sections overlap, the larger section properties are used. All beams must be continuous from edge to edge of the slab.

The limitations of this method are:

- In an uniformly loaded footing, bending moment in centre heave will lead to a less conservative value (Mitchell, 1984).
- Uncertainty exists in the appropriate selection of edge distance e, of the initial shape of the distorted soil surface even with the use of computer program (Mitchell, 1984).

2.5.4 Swinburne Method

This method was developed by considering the two-way action of a plate with stiffening beams along discrete grid-lines. The method was examined using the finite element computer program FOCALS (Fraser and Wardle, 1975). Holland (1981), Holland et al. (1980) and Pitt (1982) of the Swinburne Institute of Technology, modified FOCALS to help develop design charts for the routine design of housing footing on expansive soil. These form the basis for the development of the Swinburne design method.

This method uses a distorted soil surface having a flat top with parabolic convex edges over a distance, *e*, for both centre heave and edge heave, it has flat edges over distances *e* that transform to parabolic, convex central surfaces meeting at the centre as shown in the Figure 2.21. Because of the considerable support given in the edge heave, the edge heave moments are much less than the centre heave moments (Mitchell, 1984). Centre heave moments are considered critical for the design purpose thus neglecting the edge heave moments.

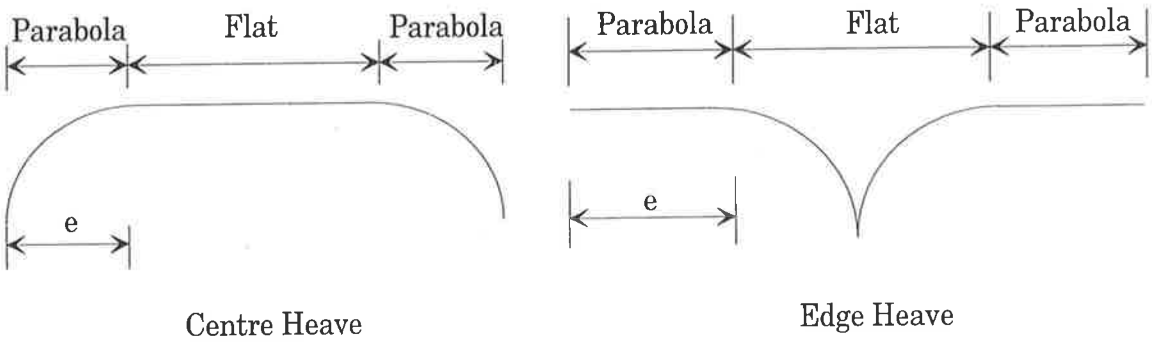


Figure 2.21 Swinburne's modes deformation. (After *Holland*, 1981).

Mitchell (1984) stated that, only single-storey brick veneer loading was considered in the computer analysis. Which is the major limitation of this method. Holland and Richards (1984), justified this by claiming that the heavier edge loads are compensated by a greater soil support while using for more than one storey. Wray (1980) and Pidgeon (1983) argued and showed that the magnitude of the perimeter wall loading have considerable impact on the design. And also the program FOCALS requires a prior knowledge of the sub-beam depth for analysis and it can be expected that the bending moment will be dependent on the depth adopted (Mitchell, 1984).

2.5.5 Mitchell Method

This method was developed by Mitchell (1979), by considering a non-uniformly loaded beam on a Winkler spring foundation. This allows beam to deflect to the permissible deflection ratio of the structure. Inorder to obtain the analytical expressions for the bending moment, beam equation was integrated at the footing centre for both centre and edge heave conditions. The shape of the initial distorted soil surface was taken as concave for edge heave and convex for centre heave, which is shown in the Figure 2.22.

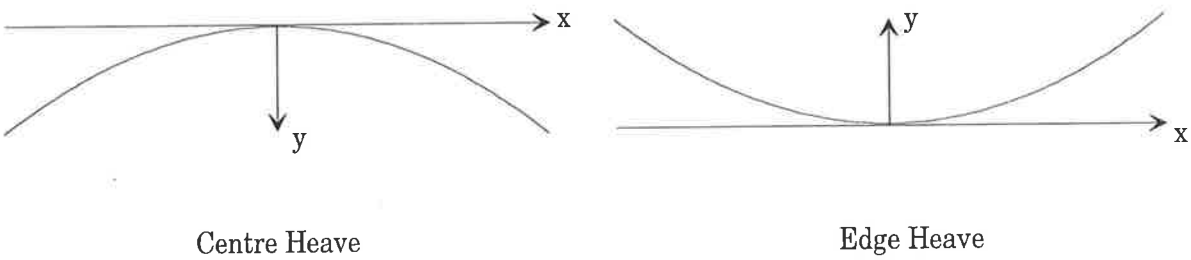


Figure 2.22 Mitchell's modes of deformation. (After *Mitchell*, 1979).

An approximation to the shape of this initial distorted soil surface is reasonably defined by polynomial known as the Lytton equation and can be represented as:

$$y = \left[\frac{2x}{L}\right]^m Y \tag{2.23}$$

where: y = free swell movement

Y = maximum differential heave

m = a shape factor, represented by the expression:

$$m = \frac{0.75L}{a} \tag{2.24}$$

where: $L =$ length of the cover;

$a =$ depth of the suction change.

A number of experimental studies (eg. Ward, 1953; DeBruyn, 1965 and Washusen, 1977) have been conducted to test the accuracy of Equation (2.23) in defining the predicted shape. Mitchell (1979) verified the Equation (2.23) using the results of these studies. In response to the soil movements, the footing will be subjected to soil pressures because of its own rigidity. The soil pressure can be large enough to cause excessive displacements of the structure supported on the footing. Thus it is necessary to determine the soil pressures on the footing. In order to determine the soil pressures on the footing, the equilibrium position of the deformed footing in both modes must be found. Mitchell (1979) arbitrarily used Equation (2.25) to define approximate shape of the footing.

$$\delta = \left(\frac{2x}{L}\right)^{t} \Delta \tag{2.25}$$

where: Δ = the maximum allowable differential displacement.

Furthermore, Mitchell (1979) established following three conditions if the footing is in equilibrium:

- The superstructural loads must equal the soil forces acting over the length of the footing in contract with the soil;
- The bending moment due the external loads and soil forces must be equal to the bending moment corresponding to the curvature of the particular footing stiffness; and
- At any point of intersection between the soil profile and the footing profile, the soil displacement y, is equal to footing displacement δ. Thus Equations (2.23) and (2.24) are equal.

The three conditions were solved simultaneously by Mitchell (1979) to determine the distribution of soil forces along the footing length. Thus enabling the determination of the moment and shear forces acting on the footing to permit design.

The above analysis enabled Mitchell (1979) to accurately predict both the expansive movements and the determination of the deflection of a footing, and developed a design procedure. Brief summary of that design procedure is as follows:

- Determine the geometry of the structure and footing, particularly the length, L, the breadth, b, various types of loads on the footing, moment of the structural loads about the centre of the footing, M_0 and the allowable deflection, Δ, from the deflection ratio. Allowable deflection has been determined empirically and is shown in Table 2.10.

Table 2.10 Allowable deflection ratio. (After *Mitchell*, 1979).

Super Structure	Deflection Ratio
Solid Brick	1/2000 min
Articulated Brick	1/800 to 1/1000
Brick Veneer	1/500 to 1/800
Articulated Brick Veneer	1/300 to 1/600
Timber Frame	1/200 to 1/300

- Conduct a site investigation to determine the parameters like in situ soil suction, Instability Index, diffusion coefficient and swell stiffness.
- Define the expected environmental conditions and determine the differential soil movements for both edge and centre heave.
- Determine the required stiffness, *EI*, moment and deflection for the footing for both edge heave and centre heave.
- Structural elements can then be determined using conventional reinforced concrete design methods.

It is now evident from the preceding sections that these techniques utilise highly idealised two-dimensional and deterministic models to describe soil-footing interaction. Even though these procedures are very useful for the design of residential footings, inherent uncertainities as described previously in the methods may sometimes lead to unsatisfactory footing design. An alternative course of action is to adopt empirically based probabilistic method. The following section summarises the probabilistic design technique.

2.6 PROBABILISTIC METHODS

There is some degree of uncertainty associated with all phenomena with which civil engineers must work. For example, peak traffic demands, total annual rainfalls, steel yield strengths and performance of buildings will never have exactly the same observed values, even under seemingly identical conditions. Subsequently, the task of the design engineer now is to deal with this uncertainty in a realistic and economical manner. How the engineer chooses to treat the uncertainty in a given phenomenon depends upon the situation. The probabilistic approach appears to be realistic in this case. This section strengths and performance of buildings will never have exactly the same observed values, even under seemingly identical conditions. Subsequently, the task of the design engineer now is to deal with this uncertainty in a realistic and economical manner. How the engineer chooses to treat the uncertainty in a given phenomenon depends upon the situation. The probabilistic approach appears to be realistic in this case. This section summarises about the significant benefits of the probabilistic method over deterministic methods and the ideology of Kay and Mitchell (1990).

2.6.1 Significant Benefits of Probabilistic Methods

The mathematical models discussed previously, inherently simplify the complex nature of expansive soil behaviour and the soil-footing interaction. Subsequently, an alternative probabilistic design method has a number of benefits over deterministic methods and which include:

- *The degree of risk associated with a footing design can be quantified*: Litigation, as a result of residential footing failures, has cost the community several millions of dollars. Since the probabilistic method enables the degree of risk to be quantified, a probabilistic footing design will allow the client and engineer to make informed decisions regarding the desired level of risk. As a consequence, such an approach is likely to reduce litigation.
- *More reliable and rational design technique*: Since the probabilistic design approach is based on the performance of actual footings which have been built and tested in a wide variety of design situations. Such an approach provides a more realistic and reliable design methodology than that given by deterministic techniques.
- *Compliments existing deterministic design techniques.*

2.6.2 Probabilistic Design Format of Kay and Mitchell (1990)

This method was aimed to provide the design engineer with some guidance to the level of uncertainty that exists in the design process. Most of the methods currently used have camouflaged this uncertainty. Frequently, an entirely unwarranted level of faith is placed on results that are obtained from sophisticated computer analyses. To convey such a level developed permits selection from a range of beam depths subject to a maximum crack width and the desired level of risk. The following section outlines the analysis of this method.

2.6.2.1 Kay and Mitchell (1990) Method of Analysis

Kay and Mitchell (1990) argued that the outer fibre strain for a simply supported elastic beam is simply supported elastic beam is inversely proportional to the beam stiffness parameter, *EI,* where *E* is the Young's modulus of the concrete and *I* is the second moment of area about the axis of bending. The same applies to a cantilevered beam and a similar assumption would not be unreasonable for an elastic beam raft interacting with an elastic foundation. The width of a crack in brickwork placed on such a beam should be approximately proportional to the beam stiffness. To support this Walsh (1985) indicated that, on the basis of small number of trials using analytical model that he had developed, crack width varied inversely with the second moment of inertia raised to the power of 0.8. Subsequently Kay and Mitchell (1990) considered using this inverse proportionality in their study to normalise the crack width for some arbitrary stiffness. They have used 100 MN/m^2 as an arbitrary stiffness in order to normalise the data. It should also noted that this arbitrary stiffnesses were later removed in the analyses. Normalised crack widths were then divided into ranges according to a square root scale bounded by the values 0-0.25, 0.25-1.0, 1.0-2.25, 2.25-4.0,4.0-6.25 and so on. The results were subsequently plotted at the end of the range.

A curve fitting operation was carried out in order to produce a continuous representation of the data. They have used exponential type of curves for the representation of data. For the measured results an equation was used of the form and is shown in Equation (2.26).

$$CF = 100(1-ae^{-bECW}) \tag{2.26}$$

where: *CF* = the percent cumulative frequency of the equivalent crack width (*ECW*);

a, b = curve fitting parameters.

Kay and Mitchell (1990) then argued that a procedure based on measurements taken within the first ten years of construction is likely to be unconservative for engineering design purposes. In addition, Domaschuk et al. (1984) and Osman and Hamadto (1984) has demonstrated that there is an increase in damage level with the age. In light of these practical and theoretical evidence Kay and Mitchell (1990) then considered reasonable for design to incorporate long term effects. As a result, they doubled the crack widths actually measured in the study. This is included in the Equation (2.26) by halving the exponent and shown in Equation (2.27)

$$CF = 100(1 - a e^{-bECW/2}) \tag{2.27}$$

The probability of occurrence, PC, for a particular crack width, W_c, was taken as $100 - CF$ and also using the inverse proportionality between the crack width and the raft stiffness, PC was then written as and it is shown in Equation (2.28).

$$PC = 100 a e^{-bEIW_c/200} \tag{2.28}$$

They then used the standard arrangement of stiffening beams and obtained a direct relationship between beam depth, D, and raft stiffness, EI. The EI value was determined for a wide variety of cases and an empirical relationship was obtained using the Equation (2.29).

$$EI = e^{3.03\ln(D) - 15.9} \tag{2.29}$$

Using Equations (2.28) and (2.29) a design table was established for various range of crack widths to suit a particular circumstances. Three different crack widths such as 1 mm, 2 mm and 5 mm were selected for a range of possible beam depths, the corresponding probabilities were evaluated.

2.7 SUMMARY

The emphasis of this chapter has been to review the development of relevant research arisen since engineers have realised that special precautions have to be followed when constructing dwellings on expansive soil. Although considerable progress has been made in the development of reliable and accurate heave prediction methods for expansive soil, a number of research over the past three decades has led to many theories for the rational design of residential footings on expansive soil.

Techniques which are discussed in §2.5, for the design of footings on expansive soils, are very useful for the design of residential footings but inherent uncertainties in the methods may sometimes lead to unsatisfactory footing designs which include:

- Provision is made in the design models for two models, pre-formed, two-dimensional mound shapes, described as centre and edge heaves. The use of the simple soil distortion mounds has a number of inaccuracies such as the assumption of a raft interacting on a pre-formed soil surface differs from the real situation where mounds develop after the raft has been loaded and constructed; the assumption of simplified deformation modes used in the design ignore the complex distorted shapes which can occur in the field, due to varying site environments, soil variability and man-made influences; all the methods are based on the two-dimensional analysis of a beam on a deformed mound, which contradicts the real situation where the problems are invariably three dimensional.

- The contribution of the super-structure stiffness is neglected in evaluating the raft stiffness.

- The allowable deflection ratios used in the design of residential footings were developed empirically using only a limited number of cases, and only domestic construction was considered. Furthermore, deflection ratio will also vary with changes in the environment subsequent to the construction.

The assumptions and uncertainties impose caustic limitations on the accuracy of the methods described in §2.5.

The methods adopted by AS2870.1 and 2 (Standard Association of Australia, 1988a and 1990) as the recommended procedures for the design of residential footings are those proposed by Mitchell (1979) and Walsh (1984). In addition this method uses a value of free surface heave, y_s , to determine the site classification. The prediction of free surface heave, in itself, is subject to the following limitations:

- A parameter referred to as Instability Index, I_{pt}, is used to predict the free surface heave of the soil. However, the Instability Index is non-linear and its magnitude changes according to the direction of the suction change. In addition, common laboratory methods used to determine the Instability Index have poor correlation between the predicted and the measured values. Furthermore, these laboratory methods are time consuming and very expensive for general footing design. The visual-tactile method which is discussed in §2.2.3.3 involves the manual and visual inspection of the soil to determine the Instability Index using Plasticity Index. However, Eden and Hill (1994) have shown that estimating Instability Index using this method, results in extreme variability and depends greatly on the classifier.

- In the prediction of y_s, AS2870.2-1990 (Standards Association of Australia, 1990) recommends the use of linearly-increasing, triangular suction change profiles. This profile is a simplification of a limited number of case studies where no allowances have been made for: extreme dry or wet conditions; effects of vegetation; leaking services; poor irrigation practices and poor drainage.

The predicted value of y_s, is usually based on the results of two or three boreholes. As a consequence, there is little allowance for the variability of soil properties across the site.

As can be seen, none of the methods account for the complex site conditions that the footing and the soil will experience in practice. An alternative course of action, in order to minimise the effects of expansive soil, is to revert to an empirically based design procedure. With the introduction of the empirically based probabilistic method, the associated risk may be quantified prior to the construction of the structure.

The following chapters present the research carried out on the performance of footings constructed on expansive soil, the risks associated with this performance and the development of probabilistic methods for the design of footings built on expansive soils.

Chapter Three

Compilation of Database

3.1 INTRODUCTION

A *database* is a collection of data that are relevant to a particular topic or purpose. Further, the system of storing and retrieving information in a database is called a *database management system* (DBMS). A computerised DBMS is a program, which is used to store and retrieve data on any type of output. This chapter reviews the progress that has been made in the geotechnical database management systems. In addition, this chapter contains the methodology of the data collection and examines the relationship between a number of parameters stored in the database.

3.2 EXISTING DATABASE MODELS

Collecting information about the geotechnical engineering and documenting them in a sensible format in order to provide relevant information for the future work is a challenge to the engineering community. In addition, when the size of the collected data is too large, electronic data retrieval and sorting becomes inevitable. Consequently, a number of electronic databases have been developed over the last decade or so. A number of the relevant geotechnical engineering databases will be examined below. To ease the operation, most of the databases are formulated with the following objectives:

- The system must be operable by persons with, at most, only limited knowledge of computing;
- Data must be input in a manner that is both efficient and simple;
- Only factual information should be stored and it must be as complete and accurate as possible;
- Flexibility of terms must be allowed on input; and
- Retrieval of the information should be designed to the user's needs.

Over the years, several authors have developed databases to suit their requirements and these include: *database system for soil classification using CPT data* (Chan and Tumay, 1991), *reliability analyses in geotechnics* (Favre et al., 1991). Each of these authors have emphasised the requirement of the databases for storing data in geotechnical engineering. Some of the advantages have been listed and these include:

- Free of redundancies;
- Inconsistencies can be avoided;
- Unified integrity control as well as security and safety can be obtained;
- The data can be shared;
- Standards can be enforced;
- Conflicting requirements can be balanced;
- Provide the community with an improved planning and decision making tool (Wood et al., 1982; Touran and Martinez, 1991); and
- Supports query answering facility.

GEOSHARE

GEOSHARE (Wood, 1980; Wood et al., 1982; Wood et al., 1983; Day et al., 1983) is a database developed for the Construction Industry Research and Information Association (CIRIA) in Britain, to compile site investigation information. In addition, it was compiled to investigate the feasibility of the storage of geotechnical records in a computerised databank and avoid the use of external codes (Wood et al., 1982).

In the GEOSHARE database the information was stored in two fields, which were:

- Numerical data; and
- Descriptive data.

The former includes numerical borehole reference data, sample tests and water level tests. The latter incorporates soil description and non-numeric borehole reference. These are summarised in Table 3.1.

Table 3.1 Information stored in GEOSHARE.
(After *Wood et al., 1983* and *Day et al., 1983*).

Types of Data	Examples
Numeric	
Borehole reference data	Grid reference
	Ground surface height
	Date of drilling
	Borehole diameter
	Casing limits
Sample and test data	Depths of strata
	In situ tests
	Sample recovery
	Laboratory tests
	Water level readings
Descriptive	
Soil description	Consistency/compaction
	Colour
	Structure and organic content
	Primary soil type
	Secondary soil type
	Formation name/geological origin
General comments	50 character per borehole

By formulating the data in this way the computer will check automatically for logical errors. However, the non-logical errors can only be found by manual checks. The descriptive data involve terms and phrases of a particular type and coding of this information is necessary for efficient storage. Consequently, a scheme has been devised to perform transformation of the normal English into coding language which is performed internally by the computer.

The input operation is carried out by providing information relating to each category and responding to programmed queries using normal English. Either the complete term or a truncated form comprising the first four letters are entered. In addition, it checks for the appropriate vocabulary, if present, it is coded and stored. Otherwise, two closest alphabetical terms are displayed to the user as possible alternatives. The user can either accept one of the alternative terms, re-input the data or up-data the vocabulary file.

ASCE Database

ASCE (1991) compiled a database to collect, organise, and disseminate case histories of shallow foundation behaviour. The database program was developed using DBASEIV. It is a menu driven interactive database and has several options such as input, viewing and analysis. The input allows the user to enter new data, correct existing data or cancel data. The viewing option allows the user to look at any case history in the database, and to create a subset of case histories satisfying a number of conditions. The analysis option allows the operator to use a number of design methods to predict the response of a shallow foundation and to perform correlation studies.

Kay and Mitchell Database

The Kay and Mitchell (1990) database was compiled to collect, analyse and develop probability design charts for houses built on expansive soils. In addition, this database includes only houses constructed on raft footings from the City Council of Campbelltown in Adelaide, South Australia. The database consisted of relevant data from houses built between 1972 to 1985. The information pertaining to each house was then assembled on a simple coding form. The information included in the database was:

- The borelogs from the site investigation;
- Dimensions and reinforcement details of the footing beams and slabs;
- The footing layout from the engineer's construction report;

- Details of the super-structure;
- The name of the current owner of each house;
- The width of cracks on the exterior of each house; and
- Various site conditions, such as positions of downpipes, trees, taps, concrete pathways and drainage.

Current Study

The current study utilises the same database formulation methodology used by Kay and Mitchell (1990). However, for ease of data storage, manipulation and presentation, the database was developed using *MS Access* 1.1® software and is known as CRACK_SMH. The following section details the formulation of the database used in this study.

3.3 FORMULATION OF THE CRACK_SMH DATABASE

The CRACK_SMH database has been formulated to reduce the effort required to enter, store, manipulate and output data. This database has numeric as well as text parameters. The information stored in the database include:

- Name of the council;
- The address of the site;
- Building application number;
- Date of construction;
- Type of super-structure;
- Soil borelog;
- Footing details and layout of existing houses built on expansive soil;
- Internal crack width;
- Position of the crack; and
- External crack width.

3.4 DESCRIPTION OF THE DATABASE

Databases formulated using the *MS Access* 1.1® software utilises a collection of objects, such as tables, queries, forms, reports and macros. It is a relational database management system (RDBMS) which allows data to be organised according to any subject. The advantage in a RDBMS, such as *MS Access* 1.1®, is its ability to quickly search for, find and bring together information stored in separate tables. In order for it to work most efficiently each table should include a field or set of fields that uniquely identifies each individual row or record stored in the table. In a database terminology it is named as *primary key*. In addition, it can store information about how subjects are related to each other.

Data were compiled over the period from September 1993 to December 1994, and were arranged in the database in the form of tables. The tables used were labelled as:

- Main;
- Address;
- Technical; and
- Inspection.

In each table various fields were included for the effective use of the database, which included:

- **Main:**

  Building application number - Each council describes a unique number to each building application. This number is used in each table of the database to uniquely define each dwelling.

  Year of construction - The date of construction of the stiffened raft footing.

  Estimated heave - Estimated maximum surface heave of the site.

Site classification - The site classifications with reference to AS2870.1-1988 (Stanadard Association of Australia, 1988a).

- **Address**

Building application number - As mentioned in the table Main this information uniquely identifies each dwelling.

Address - Information regarding addresses of all the houses for correspondence with a home owners or residents.

Postcode - The postcodes of different suburbs and helps in creating the mailing lists to a required format.

- **Technical:**

Building application number - Primary key of this table to define each dwelling as described in the previously.

Type of stiffened raft - Identifies the rafts as either grillage raft or standard raft.

Raft beam details - Dimensions of external and internal beams are stored in this column.

Shape of the raft - Shape of the raft are sub-divided into L, rectangle and other complex shapes.

Slab thickness - Thickness of the concrete slab which contributes towards the stiffness of the raft.

Reinforcement details - Beam and concrete slab reinforcement details.

- **Inspection:**

Building application number - Same number as in the other tables.

Names of the residents - Names residents/owners who had agreed to participate in the research.

Date of inspection - Date of inspection of the house.

Maximum width of crack - The maximum crack width noticed with the walls, cornices, floors and the ceilings of the structure.

Type of crack - To differentiate between center heave and edge heave.

Damage category - Damaged houses are classified as per Appendix A of AS2807.1-1988

Calculated critical raft stiffness - Raft stiffnesses at the critical section of the stiffened raft.

3.4.1 Methodology of Data Collection

To provide necessary information for the development of probabilistic design charts for residential footings built on expansive soil, a survey was carried out in two stages. In the first stage footing details were compiled from six local councils. In order to include a wide range of soil types within the database, the following council areas were targeted.

- Campbelltown;

- Enfield;
- Happy Valley;
- Marion;
- Prospect; and
- Tea Tree Gully.

These council areas are located in the metropolitan area of Adelaide and are shown in Figure 3.1. The soil profiles encountered within these council areas contain most of the major soil groups of Australia (Soil map of Adelaide). In addition, these council areas encounter each of the site classifications included in this study, that is slightly (S), moderately (M) and highly (H) expansive soils, as specified by AS2870.1-1988a. In order to compile the necessary data, a letter was sent to each of the councils seeking permission to use their databases. A sample letter is included in Appendix A. The main objective of this survey was to identify possible houses that could be used in this research. Subsequently, houses built on S, M and H site classifications were targeted. Extremely expansive soils, E1 and E2 were investigated by Leach et al. (1995). The houses were selected from these councils randomly but were included they met a number of criteria such as houses built on S, M and H site classifications and having stiffened raft footing. Description of the site classification has been discussed previously in §2.2.7. In addition, only houses built between 2 to 10 years were targeted. The present study focused only on stiffened raft footings as these are the most common type in current use. Some of the councils included in the study are relatively old, consequently, construction of new houses in those councils are rare and as a result, fewer new of houses were encountered.

Data collected from all councils consisted of 759 footings and are separated into six groups according to site reactivity and super-structure. These are tabulated in Table 3.2.

Footing details, soil borelogs and the engineer's construction report sheets were photocopied and other pertinent information were noted down in a survey sheet specially prepared for this research. A sample survey sheet is included in Appendix B. The distribution of S, M and H sites for each council, as well as the relative proportion, is shown in Figure 3.2. It is evident from this figure that the Campbelltown city council contributes greatest number of houses in the database, whereas the Prospect city council contributes the fewest.

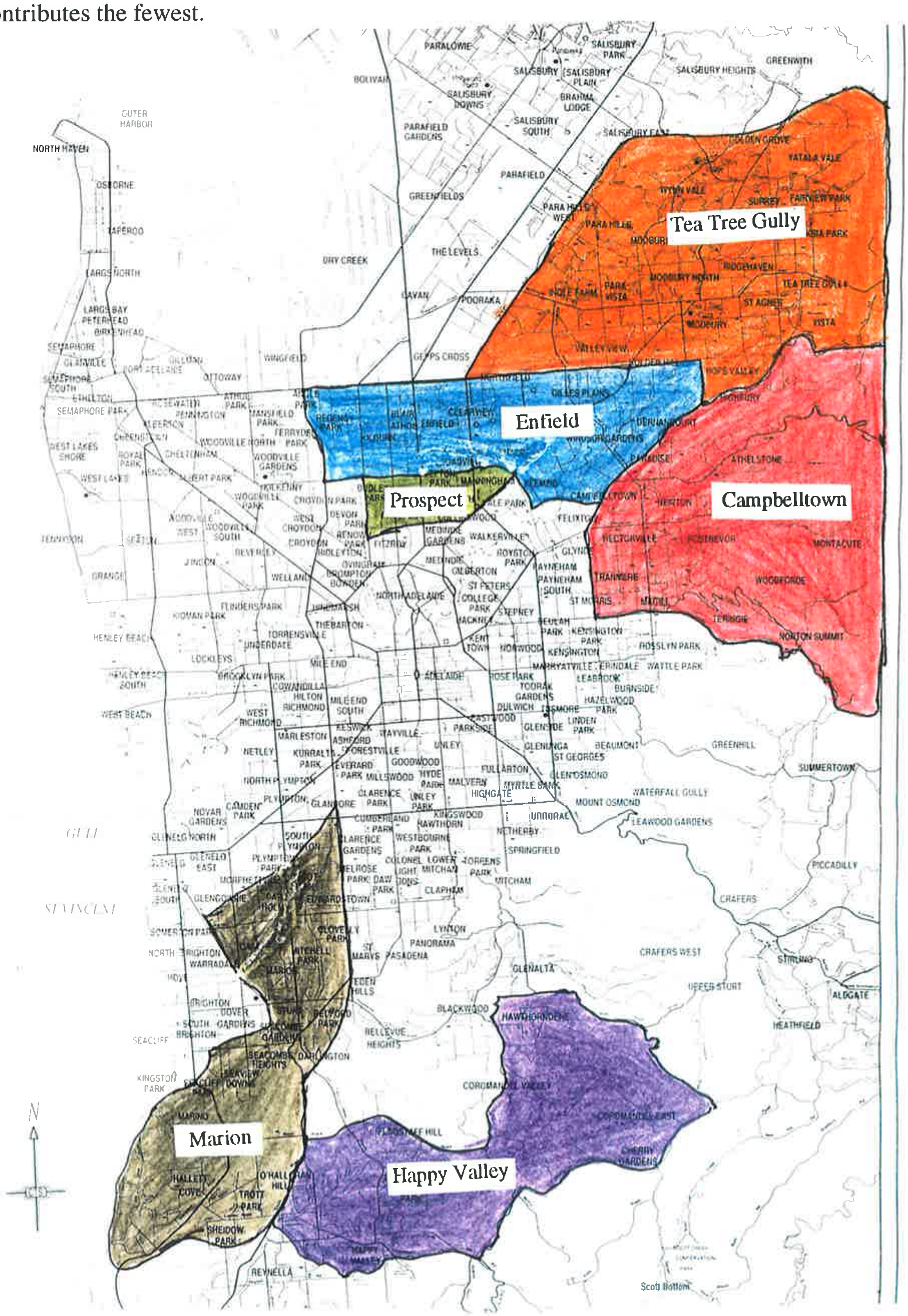


Figure 3.1 Councils of metropolitan Adelaide.

Table 3.2 Number of houses for different site classifications.

Council	Total	Site Classification					
		Slight (S)		Medium (M)		High (H)	
		ABV	ASB	ABV	ASB	ABV	ASB
Campbelltown	177	16	7	46	8	83	17
Enfield	159	37	5	52	6	50	9
Happy Valley	139	51	1	57	1	29	0
Marion	84	33	0	22	2	26	1
Prospect	54	8	4	13	14	6	9
Tea Tree Gully	146	48	4	46	3	41	4

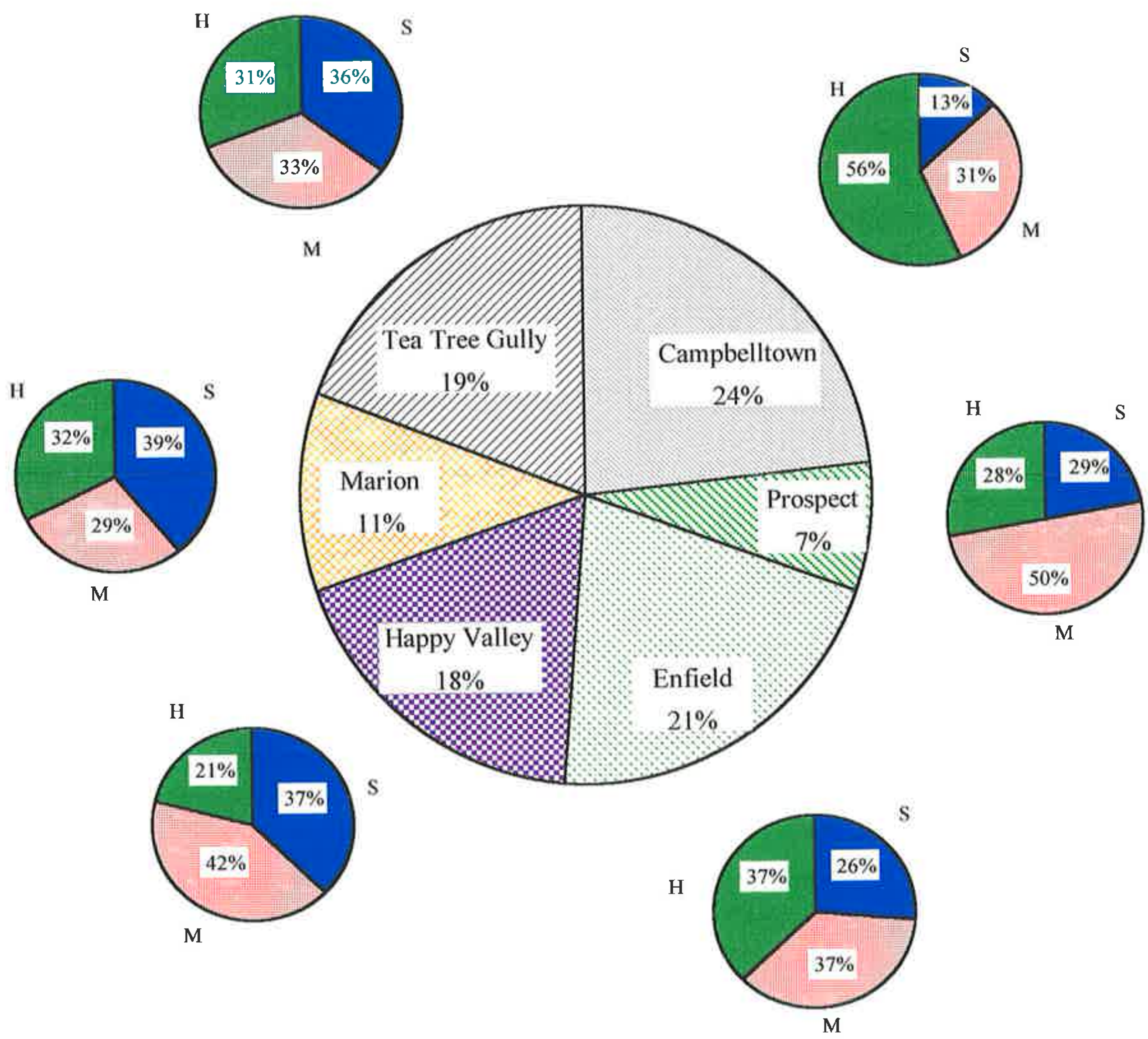


Figure 3.2 Pie chart showing the distributions among different councils.

Figure 3.3 describes the total distribution of different soil classifications. The number of houses in each group is almost distributed evenly in the database.

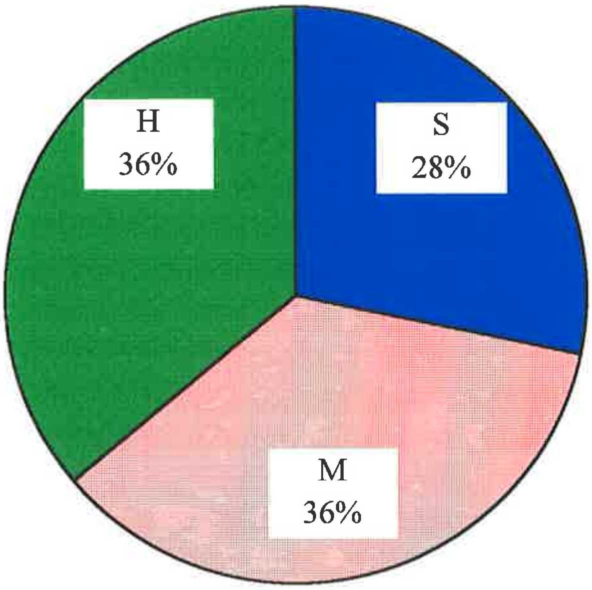


Figure 3.3 Different types of site classifications.

3.4.2 Visual Inspection

In order to carry out a visual inspection of each house, letters were sent to each resident seeking permission to survey their houses. The letter outlines, in some detail, the purpose of the inspection. A sample letter is included in the Appendix C. Of 759 letters sent, relatively few, only 258 (34%) questionnaires were returned. Of those returned, 42 owners declined to take part in the research and the rest did not respond to the questionnaire. A visual inspection of each of these houses was carried out in order to quantify the amount of damage. Table 3.3 shows the summary of the number of houses inspected.

Each of the residents who had agreed to participate in the survey received a phone call to arrange an appointment to inspect their house. Each inspection took approximately 20-30 minutes, during which time the following procedure was adopted:

- An internal inspection was carried out to record the locations and widths of cracks within the walls, cornices, floors and the ceilings of the structure;

- An external inspection, similar in nature to the internal inspection, was carried out. In addition, expansion joints were assessed for movement in the external leaf;
- To assist with damage assessment, various site conditions, such as positions of downpipes, trees, taps, concrete pathways and drainage were recorded; and
- The residents/owners were asked if they were aware of any cracking, or any other previous damage, that had been repaired earlier in the life of the structure.

Crack widths, greater than 1 mm, were measured using a ruler, whereas smaller cracks were measured using a hand lens which incorporated an inscribed grid. As a result, it is expected that the accuracy of crack measurement is ± 0.25 mm for cracks exceeding 1 mm in width, and ± 0.01 mm for cracks less than 1 mm in width.

In order to maintain a visual record of the visual inspections a large number of cracks were photographed. In order to achieve as many positive responses to the questionnaire as possible, a survey report was prepared and sent to each of the occupiers of inspected houses, outlining the probable cause, maximum width of crack and the associated damage category. An example of such footing assessment sheet is included in Appendix D.

Table 3.3 Summary of number of houses.

Name of the Council	Total Letters Sent	Total Interested	Total Declined	Total Inspected
Campbelltown	177	53	10	43
Enfield	159	48	5	43
Happy Valley	139	52	5	47
Marion	84	26	8	18
Prospect	54	30	7	23
Tea Tree Gully	146	49	7	42

It should be noted that, although various site conditions, such as positions of down pipes, trees, taps, concrete pathways and drainage were recorded, the effects of each of these were not separately considered in the analyses which follow. A study investigating the effects of each of these site conditions may be a worthwhile exercise for future work.

3.4.3 Summary of Visual Inspections

A summary of the visual inspection is shown in the Table 3.4.

Table 3.4 Summary of visual inspection.

		Super -Structure	
		Articulated Brick Veneer	**Articulated Solid Brick**
Total No. of Houses	S	67	10
	M	58	9
	H	56	16
Cracked	S	48	10
	M	42	9
	H	42	14
Damage Category as per AS2870.1 1988	0	1	0
	1	82	9
	2	49	23
	3	0	1
Crack Widths greater than 2 mm	S	4	2
	M	8	5
	H	11	4

The following conclusions can be made from the visual inspection:

- Approximately 75% articulated brick veneer houses built on S, M and H site classifications have suffered relatively minor (hairline) cracking;
- Of all the articulated solid brick houses inspected on S and M site classifications, none of them performed well, that is, none had a damage category of 0; and
- 88% of articulated solid brick houses built on H sites suffered from relatively minor cracking.

Table 3.5 summaries the distribution of houses according to the damage category as specified by AS2870.1-1988. The damage category as specified by AS2870.1-1988 (Standard Association of Australia, 1988a) is reproduced in Appendix E of this thesis.

Table 3.5 Distribution of houses according to damage category.

Damage Category	ABV	ASB
0	0.5%	0%
1	38%	4%
2	23%	10%
3	0%	0.5%

3.5 RAFT STIFFNESS

3.5.1 Introduction

As discussed in §2.6.3, Kay and Mitchell (1990) argued that the outer fibre strain for a simply supported beam is inversely proportional to the beam stiffness parameter, *EI*, where *E* is the Young's modulus of the concrete and *I* is the second moment of area about the axis of bending. They also argued that the same relationship, which also applies to a cantilevered beam, would not be unreasonable for an elastic raft interacting with an elastic soil. Furthermore, the width of a crack in brickwork, which is placed on such a beam, should be approximately proportional to the outer fibre strain and, hence, inversely proportional to beam stiffness. To support this notion, Walsh (1985) indicated that the crack width varied inversely with the second moment of area raised to the power of 0.8. In light of this, Kay and Mitchell, (1990) concluded that the assumption of inverse proportionality seems to be a reasonable approximation. This same logic was adopted in the present study. Consequently, the raft stiffnesses of each of the houses that were inspected visually, were calculated. The following sections discuss the procedure involved in the calculation of the raft stiffness.

3.5.2 Critical Raft Stiffnesses of S, M and H Site Classifications

Using the visual inspection results which included the location of cracking within the super-structure, and the footing details obtained from the engineer's reports, the primary direction of bending was determined. The critical section of the raft is the section at which cracks are developed in the brickwork, an example of which is shown in the Figure 3.4. Consequently, raft stiffnesses were determined for both the long and short directions at the position where the maximum level of cracking was evident. However, for analysis purposes, only the minimum raft stiffness of the long and short raft directions was considered. By using the minimum raft stiffness as stated here, the structure was aimed to design for the worst situation and also increases its factor of safety. It was this stiffness that is herein referred to as the *critical raft stiffness. A* value of 11.3 GPa for the long term Young's modulus was used in accordance with AS3600-1988 (Standards Association of Australia, 1988b), for 20 MPa compressive strength concrete, which is typical for domestic construction. The second moment of area was determined using a flange width of 1000 mm for edge beams and 2000 mm for internal beams, in accordance with the AS3600-1988 (Standards Association of Australia, 1988b), as is shown in Figure 3.5.

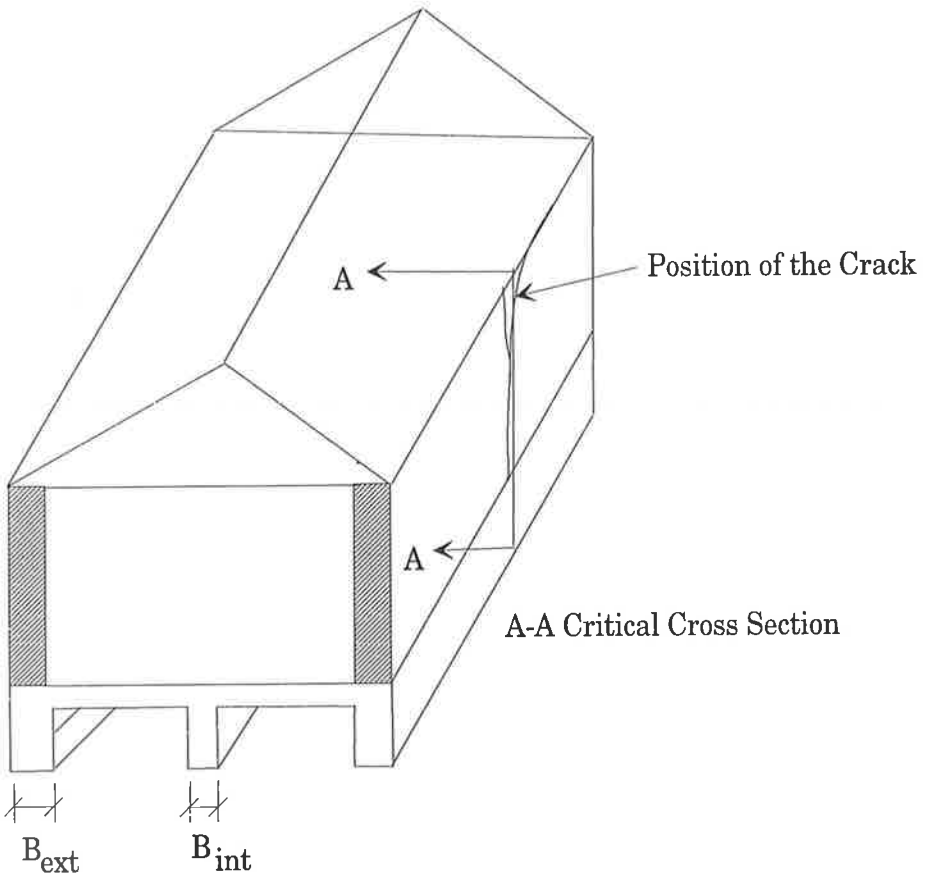


Figure 3.4 The critical section of a raft.

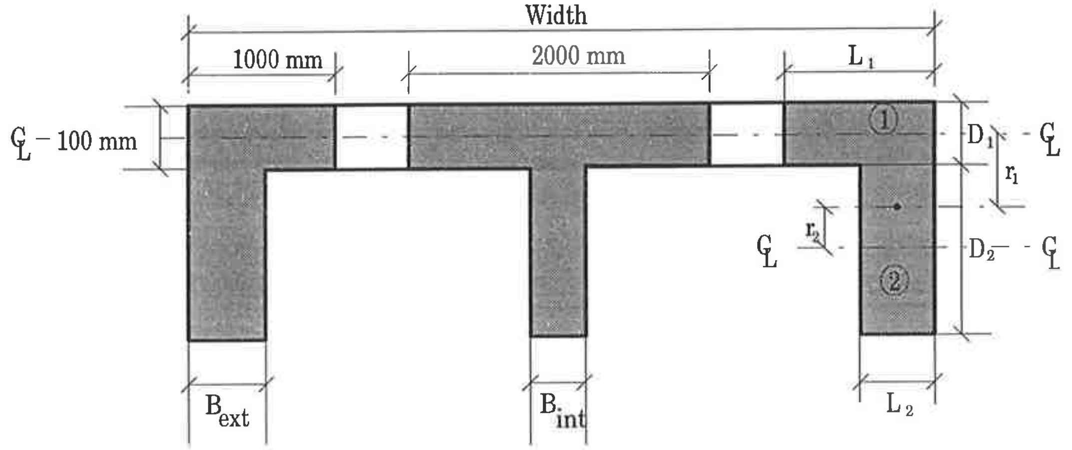


Figure 3.5 Cross section of a raft footing.

The procedure used in the calculation of the second moment of area of the raft is as follows. The edge beams were considered as L sections of flange width 1000 mm and the internal beams were considered as T sections of flange width 2000 mm. The second moment of area for each section was calculated separately using the standard relationship as shown in Equation (3.1). These individually calculated second moments of area were then summed to determine the total second moment of area, as is shown in Equation (3.2).

$$I = \frac{LD^3}{12} + Ar^2 \tag{3.1}$$

where: I = second moment of area;
L = length of the flange;
D = depth of the flange;
A = area of the section;
r = distance from the center of gravity.

$$EI = E(2I_e + xI_i) \tag{3.2}$$

where: I_e = second moment of area of edge beams;
I_i = second moment of area of internal beams;
x = number of internal beams.

To reduce the number of variables involved in the analysis, so that direct correlations between the critical raft stiffness and the crack width can be obtained by determining the unit stiffness. Subsequently, unit raft stiffness was determined for each raft by dividing the total stiffness by the total width of the raft. It was decided that, to enable efficient calculations of all types of rafts, a macro was written using *MS Excel*, 4.0®. A sample sheet is attached in Appendix F. Critical raft stiffnesses were then separated into three classes of site reactivity (S, M, H) and into two classes of house flexibility (ABV, ASB).

3.5.3 Relationship between the Critical Raft Stiffness and the Maximum Crack Width.

Figures 3.6 to 3.11 show the relationship between the critical raft stiffness and the observed maximum crack width. In these charts the abscissa shows critical raft stiffness as well as the external depth of beam of the raft corresponding to various raft stiffnesses. The ordinate, on the other hand, shows the maximum observed crack width.

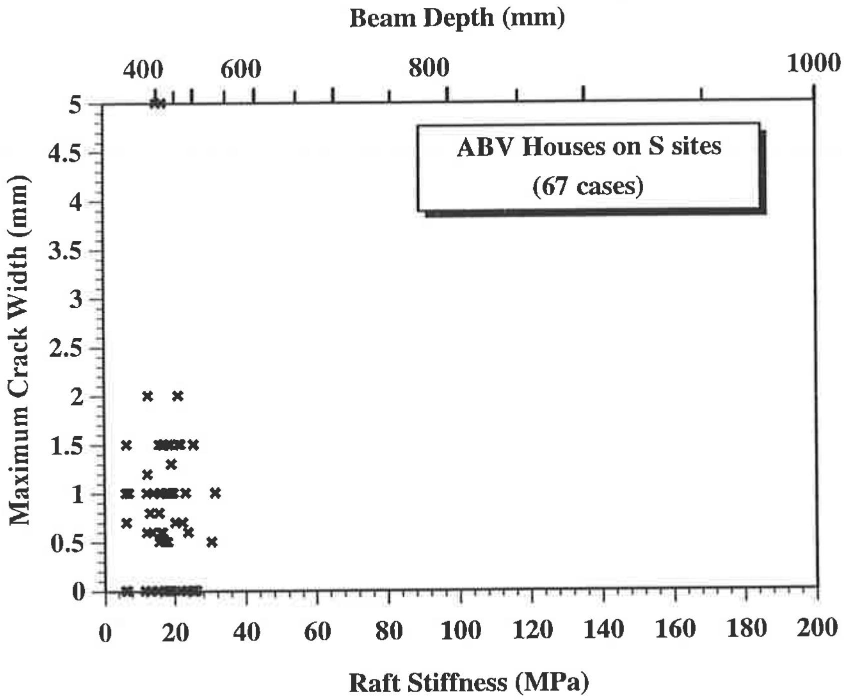


Figure 3.6 Relationship between the critical raft stiffness and the maximum crack width for ABV houses on S sites.

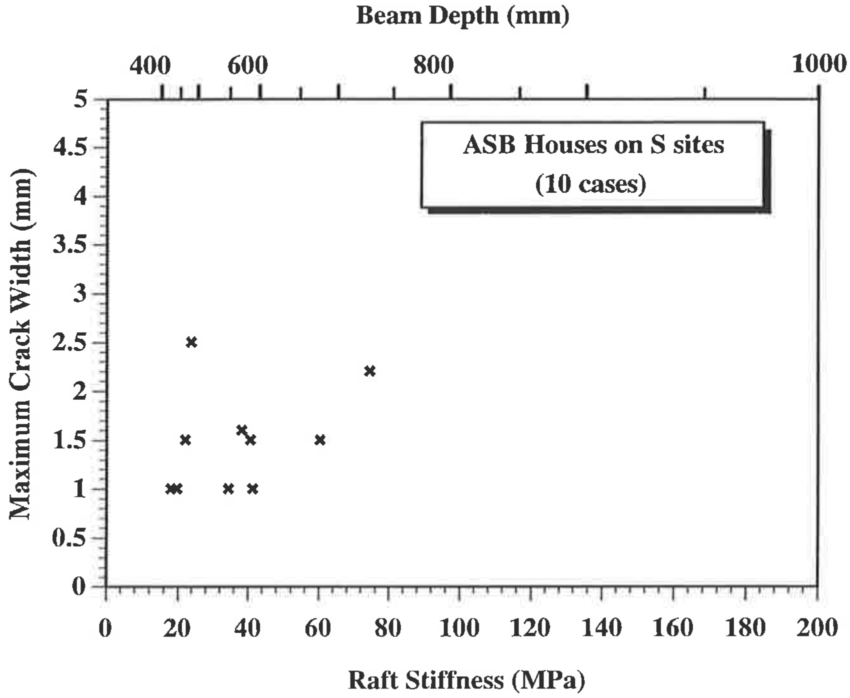


Figure 3.7 Relationship between the critical raft stiffness and the maximum crack width for ASB houses on S sites.

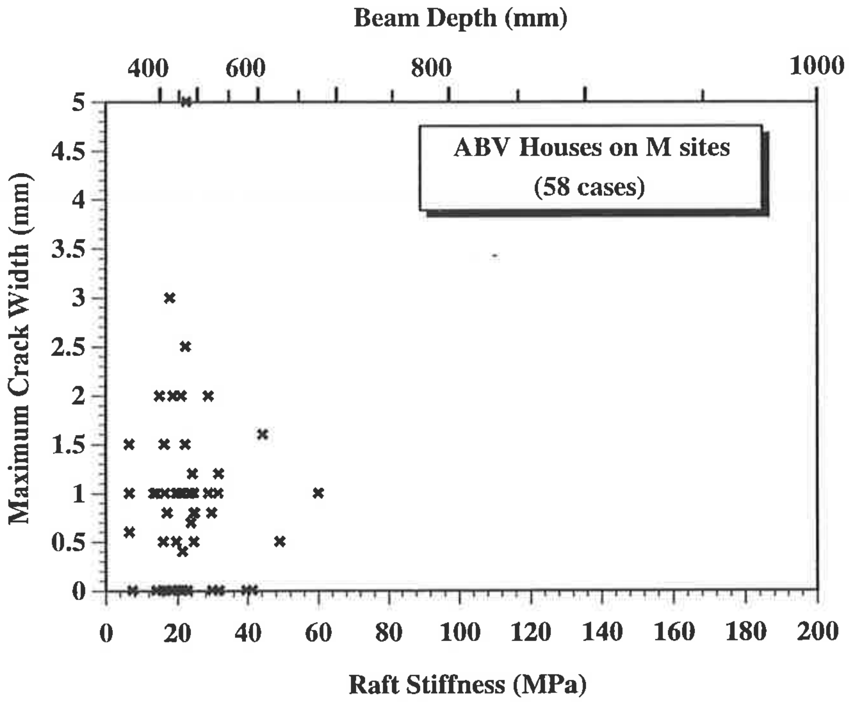


Figure 3.8 Relationship between the critical raft stiffness and the maximum crack width for ABV houses on M sites.

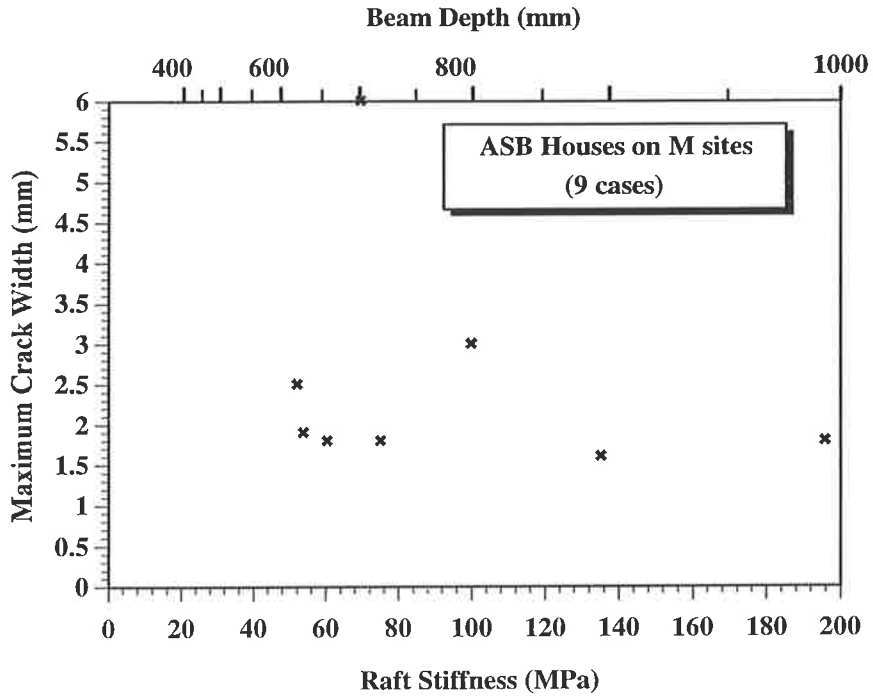


Figure 3.9 Relationship between the critical raft stiffness and the maximum crack width for ASB houses on M sites.

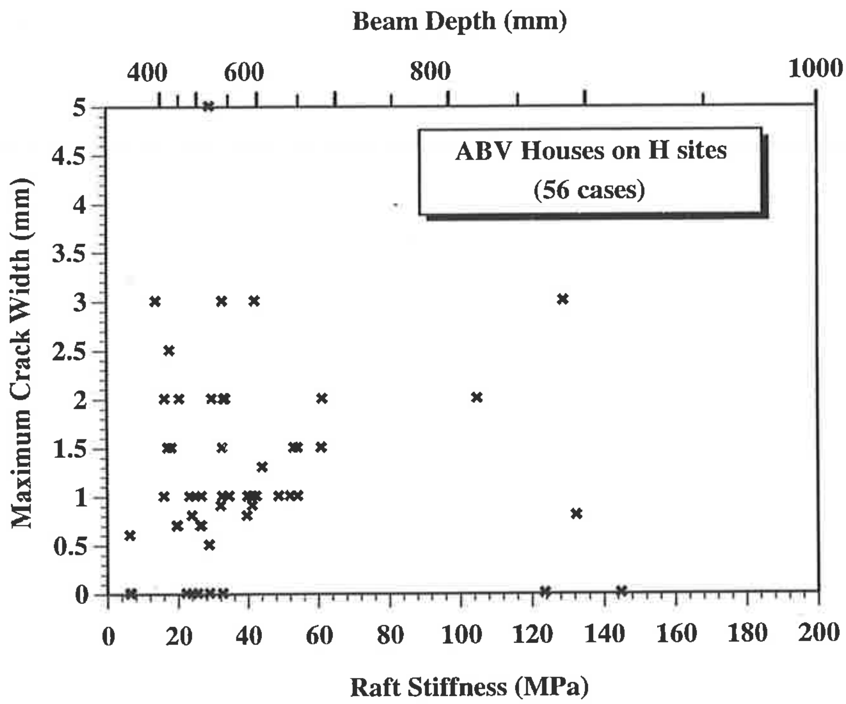


Figure 3.10 Relationship between the critical raft stiffness and the maximum crack width for ABV houses on H sites.

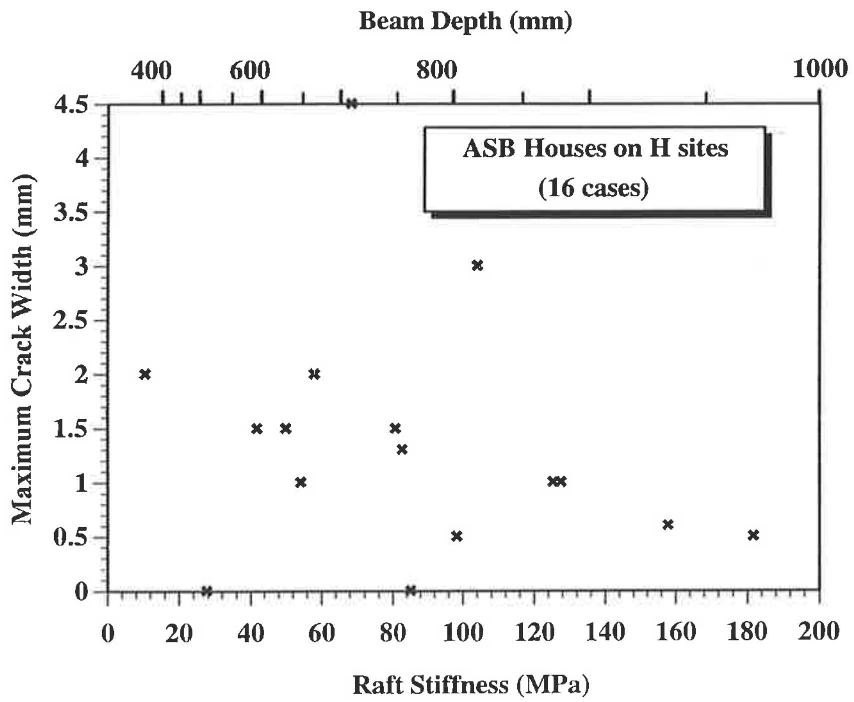


Figure 3.11 Relationship between the critical raft stiffness and the maximum crack width for ASB houses on H sites.

It is evident from these figures that there appears to be little or no relationship between the data. This result is not unexpected. Leach et al. (1995) found considerable variations in the size and density of cracks from one site to another. It is difficult to quantify the scatter, however, reasons for the scatter may include any one or more of the following:

- The reactivity of the soil is expressed in terms of the site classifications specified in AS2870.1-1988. As detailed previously, these classifications include ranges of surface heave, y_s. Inaccuracies occur when the estimates of y_s fall close to the boundary between two site classifications.
- The estimation of y_s , is itself, subject to uncertainty. As discussed previously, y_s is based on estimations of the instability index, I_{pt}; a soil suction profile and layer thicknesses. Each of these contain uncertainty which contributes to the variability in Figures 3.6 to 3.11.
- Variability exists in the site environments. For example, as discussed previously, tree effects, improper drainage, leaking service pipes, cut and fill sites, pits, barren areas and inadequate paving influence the behaviour and distortion of a footing built on

expansive soil. Since the database contains many different site environments this contributes to the variability in Figures 3.6 to 3.11.

- It is assumed that all the footings involved in this study have used concrete with a 28 day compressive strength of 20 MPa and the raft stiffnesses were calculated accordingly. The majority of residential footings are constructed using 20 MPa concrete, however, it is likely that some of the footings were constructed from higher strength concrete. In addition, it is well recognised that concrete strength increases with age (Warner et al., 1989).
- The raft stiffness calculations do not include the influence due to reinforcement.

3.6 SUMMARY

A database was formulated using *MS Access* 1.1® to store large quantities of relevant data collected from various sources. It is evident from the previous sections that databases are extremely useful in obtaining relationships among different parameters. In addition it is an efficient and effective means of data storage and retrieval, particularly when large quantities of data are manipulated.

Relevant information was collected through a survey conducted in two stages. The first being a compilation of data from various councils. Six different councils in metropolitan Adelaide were selected in order to encompass a wide range of soil classifications. In the second stage, visual inspections were carried out to measure crack widths as well as the locations of each cracks. A database, called CRACK_SMH was formulated for the efficient storage and retrieval of data. Relationships between raft stiffnesses and the measured crack widths exhibited considerable scatter with no trend evident. The following chapter presents analyses and the probabilistic design charts.

Chapter Four

Development of Probabilistic Design Charts

4.1 INTRODUCTION

This chapter deals with the analysis of the data collected from an extensive survey, as described in the Chapter 3. Using the results of this analysis, various probabilistic design charts are generated.

The chapter begins with the normalisation of raft stiffnesses, in terms of a single specified raft stiffness. Frequency distribution and probabilistic design charts are then produced using the results of these analyses. In addition, this chapter details the error associated with the design charts in terms of standard variance. Finally these probabilistic design charts are compared with probabilistic design charts presented by Kay and Mitchell (1990).

4.2 NORMALISATION OF DATA

Direct statistical analysis is only possible if the associated data are not variable. Although, a number of houses are included in the survey, the large number of variables involved, prevents a direct statistical analysis. It is evident from §3.5.3 that, for a particular measured crack width the raft stiffness varies, and in a relatively wide range. In order to overcome this problem, normalisation has been carried out, as originally proposed by Kay and Mitchell (1990). Normalisation is the process of establishing a standard variable, and the normalisation of the data is discussed in the following section.

4.2.1 Normalisation of Data

By using the inverse proportionality described previously in §3.5.1, equivalent crack widths were determined by standardising the raft stiffness to some arbitrarily selected number. These arbitrary selected numbers varied for each site classification and are tabulated in Table 4.1. The equivalent crack widths thus determined, are those expected to occur if the footing had a stiffness equal to these respective arbitrary numbers. These arbitrary stiffness are later removed from the final analysis and will be demonstrated that these numbers have no effect on the final results.

Table 4.1 Arbitrary raft stiffnesses (MN/m²) used in the normalisation of data.

Site Classification	ABV	ASB
S	17	25
M	25	150
H	25	60

Using these numbers equivalent crack widths were obtained for all footings incorporated in the data base. An example of one such calculations is shown below:

$$0.5\ \text{mm} \longrightarrow \frac{1}{40.34}$$

$$W_c \propto 1/EI$$

$$\text{ECW} \longrightarrow \frac{1}{25}$$

where: W_c = crack width;

EI = raft stiffness.

$$\text{ECW} = \frac{0.5 \times \frac{1}{25}}{\frac{1}{40.34}} = \frac{0.5 \times 40.34}{25}$$

If the raft stiffness = 40.34 MN/m² and the maximum crack width = 0.5 mm, on M site classification and for ABV superstructure,

then the equivalent crack width = 0.5 × 40.34/25 = 0.8068 mm

4.3 PRESENTATION OF DATA

In this study the data were grouped according to their site reactivity and super-structure. For each group frequency table was created accordingly. To aid in the interpretation of a frequency distribution, it is often helpful to express each frequency as either a proportion or a percentage of the total number of cases. Subsequently, frequencies were expressed in percentage frequencies or *relative frequency* because a comparison of relative frequencies is more meaningful than a comparison of frequencies. The observed data consists of observations over a large range of values. As a result, it was necessary to divide the crack widths into a series of cells. (eg. 0-0.25, 0.25-1.0, 1.0-2.25, 2.25-4.0, 4.0-6.25).

4.3.1 Histograms

Figures 4.1 to 4.6 show the histograms of all the six groups described previously in §3.5.2. As can be seen from these figures, it is possible to quantify the percentage of houses that have suffered damage in a particular range of normalised crack width. In addition, it can be seen from Figures 4.1 to 4.3, that the articulated brick veneer houses built on S and M site classifications show relatively similar performance, in that the shape of the frequency distributions is approximately the same.

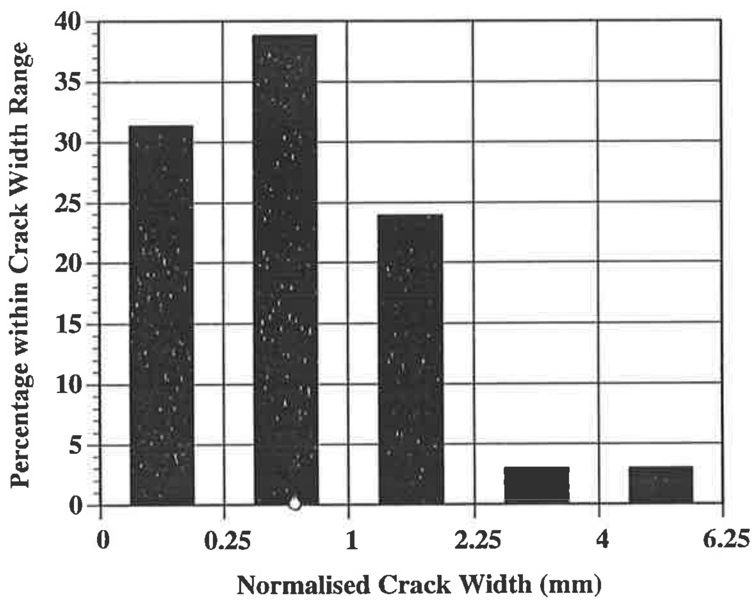


Figure 4.1 Frequency of normalised crack widths for ABV houses on S sites.

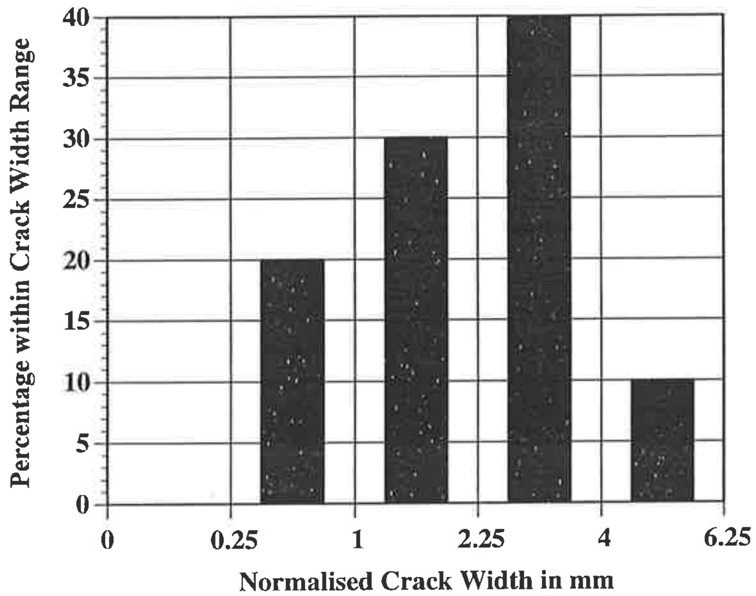


Figure 4.2 Frequency of normalised crack widths for ASB houses on S sites.

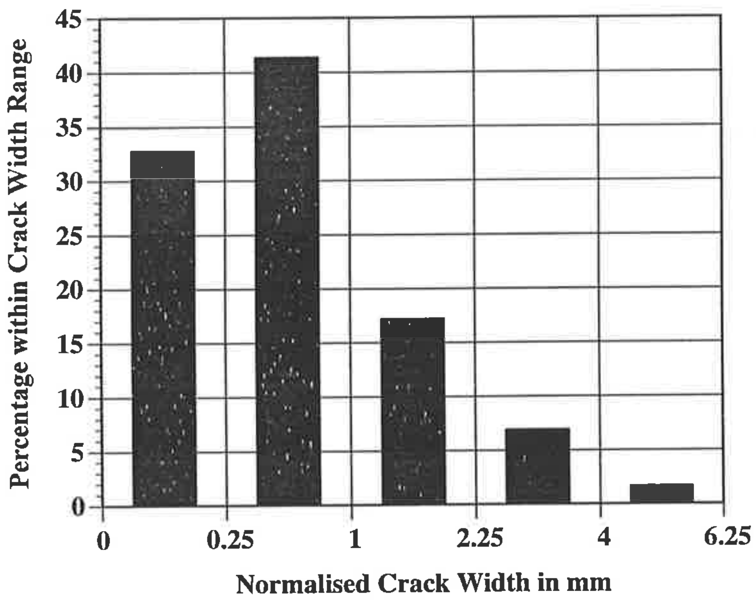


Figure 4.3 Frequency of normalised crack widths for ABV houses on M sites.

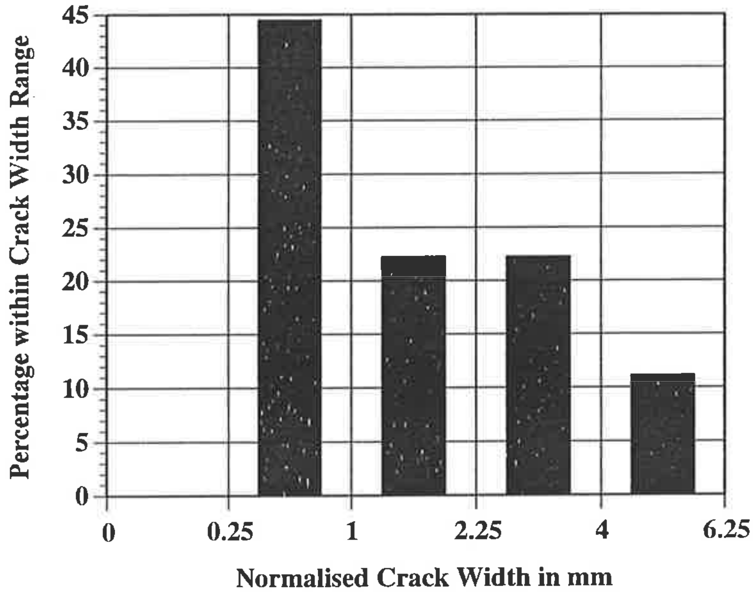


Figure 4.4 Frequency of normalised crack widths for ASB houses on M sites.

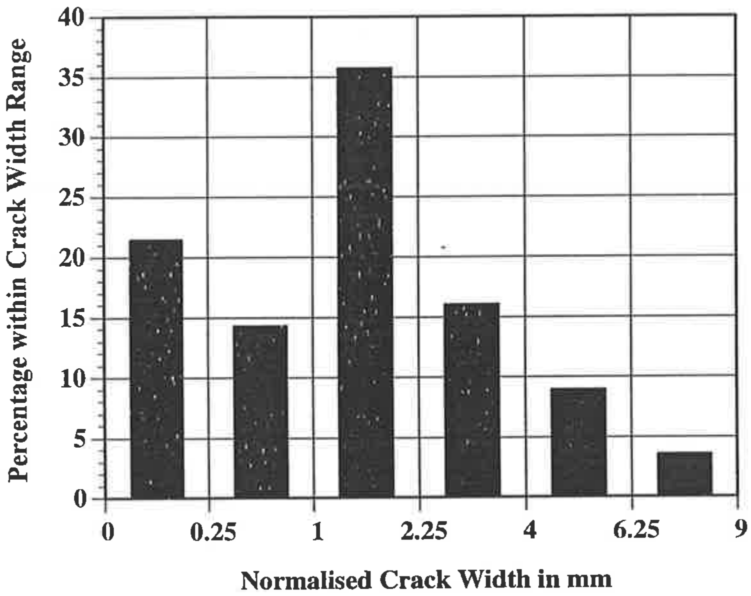


Figure 4.5 Frequency of normalised crack widths for ABV houses on H sites.

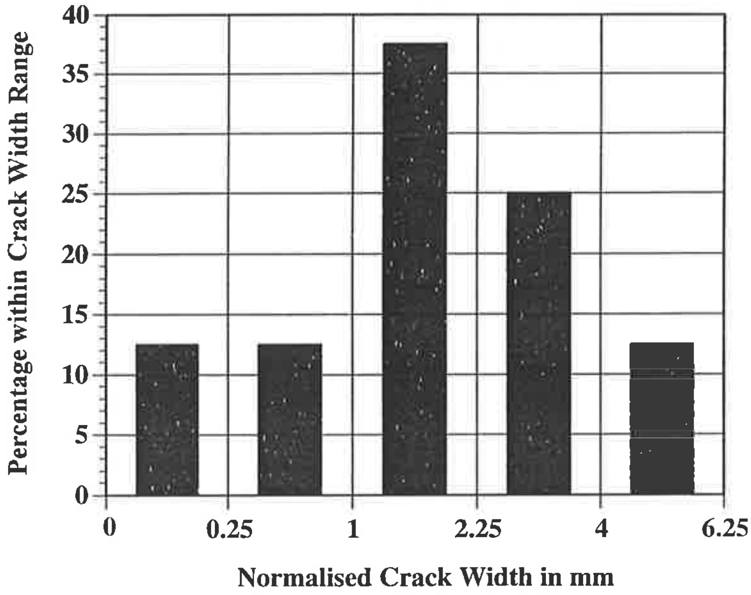


Figure 4.6 Frequency of normalised crack widths for ASB houses on H sites.

4.3.2 Cumulative Frequency

In this study, the cumulative frequency distribution is adopted to estimate the probability of occurrence of a particular crack width. This type of distribution is obtained by plotting the cumulative frequency less than any upper class boundary against the upper class range and joining all the consecutive points by a straight line. The results are tabulated, along with cumulative frequency for critical raft stiffnesses, in Table 4.2 to Table 4.7. Figures 4.10 to 4.12 show the cumulative frequency distributions for houses built on S, M and H sites, respectively.

Table 4.2 Frequency of normalised crack widths for ABV houses on S sites.

Normalised crack width range	Frequency	% Relative Frequency	% Cumulative Frequency
0-0.25	21	31.34	31.34
0.25-1.0	26	38.80	70.14
1.0-2.25	16	23.88	94.02
2.25-4.0	2	2.98	97.01
4.0-6.25	2	2.98	100.00

Table 4.3 Frequency of normalised crack widths for ASB houses on S sites.

Normalised crack width range	Frequency	% Relative Frequency	% Cumulative Frequency
0-0.25	0	0.00	0.00
0.25-1.0	2	20.00	20.00
1.0-2.25	3	30.00	50.00
2.25-4.0	4	40.00	90.00
4.0-6.25	1	10.00	100.00

Table 4.4 Frequency of normalised crack widths for ABV houses on M sites.

Normalised crack width range	Frequency	% Relative Frequency	% Cumulative Frequency
0-0.25	19	32.75	32.75
0.25-1.0	24	41.37	74.13
1.0-2.25	10	17.24	91.37
2.25-4.0	4	6.89	98.26
4.0-6.25	1	1.74	100.00

Table 4.5 Frequency of normalised crack widths for ASB houses on M sites.

Normalised crack width range	Frequency	% Relative Frequency	% Cumulative Frequency
0-0.25	0	0.00	0.00
0.25-1.0	4	44.44	44.44
1.0-2.25	2	22.22	66.66
2.25-4.0	2	22.22	88.88
4.0-6.25	1	11.11	100.00

Table 4.6 Frequency of normalised crack widths for ABV houses on H sites.

Normalised crack width range	Frequency	% Relative Frequency	% Cumulative Frequency
0-0.25	12	21.42	21.42
0.25-1.0	8	14.28	35.71
1.0-2.25	20	35.71	71.42
2.25-4.0	9	16.07	87.50
4.0-6.25	5	8.92	96.42
6.25-9.0	2	3.57	100.00

Table 4.7 Frequency of normalised crack widths for ASB houses on H sites.

Normalised crack width range	Frequency	% Relative Frequency	% Cumulative Frequency
0-0.25	2	12.50	12.50
0.25-1.0	2	12.50	25.00
1.0-2.25	6	37.50	62.50
2.25-4.0	4	25.00	87.50
4.0-6.25	2	12.50	100.00

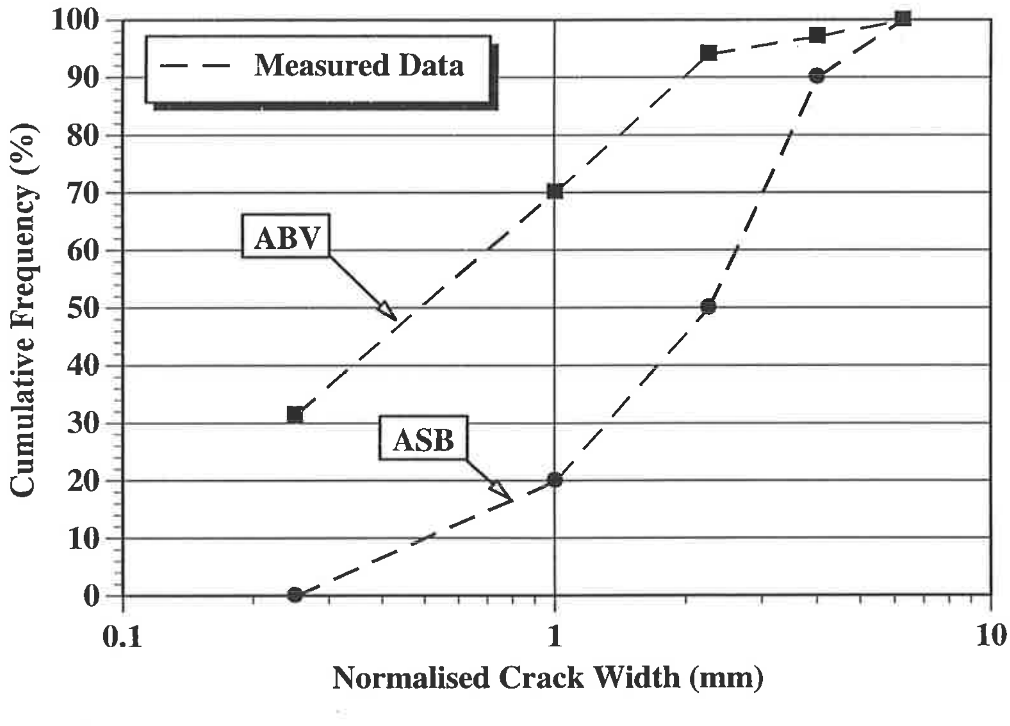


Figure 4.7 Cumulative frequencies of normalised crack widths for S sites.

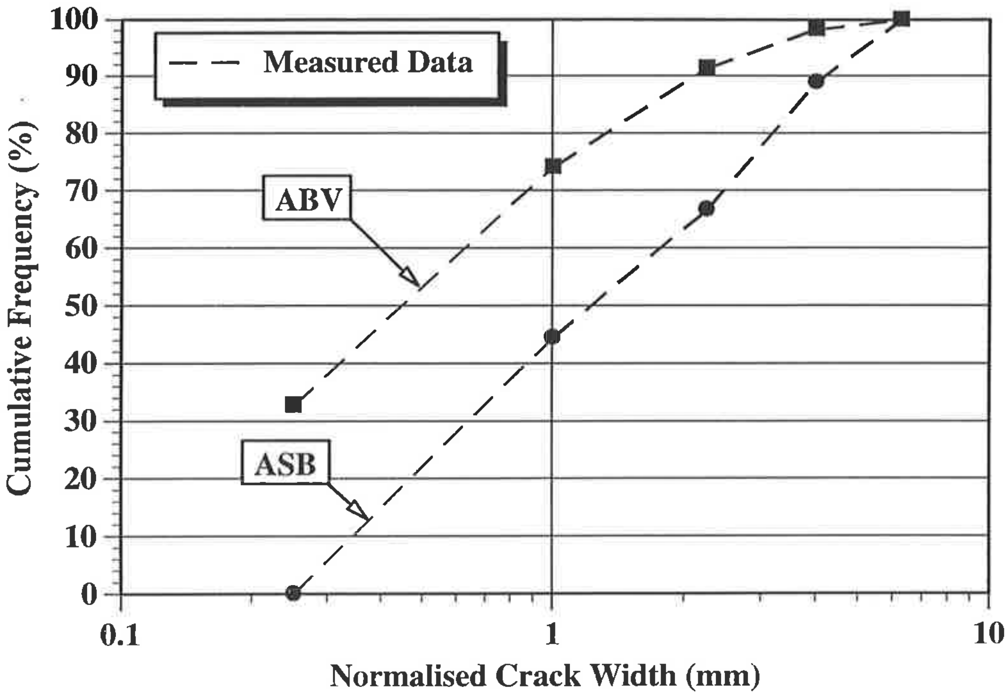


Figure 4.8 Cumulative frequencies of normalised crack widths for M sites.

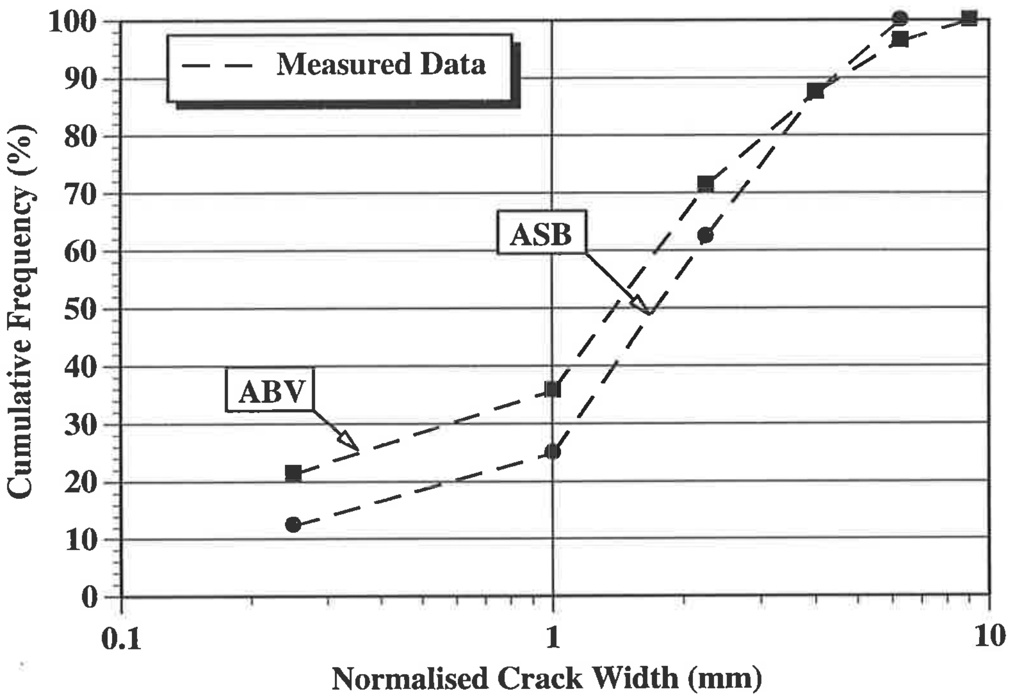


Figure 4.9 Cumulative frequencies of normalised crack widths for H sites.

4.3.3 Regression Analysis

In order to generate a continuous function from the discrete data, it is necessary to use curve fitting or regression analysis techniques.

Regression analysis is a very useful statistical technique for developing a quantitative relationship between a dependent variable and one or more independent variables. It utilises experimental data on the pertinent variable to develop a numerical relationship showing the influence of the independent variable on a dependent variable of the system. If nothing is known from the theory about the relationship among the pertinent variables, a function may be assumed and fitted to experimental data on the system. Frequently, a linear function is assumed. However if the linear function does not fit the experimental data well, the engineer might use some other function such as a polynomial or an exponential model. Kay and Mitchell (1990) fitted an exponential model to their data, as described previously in §2.6.2. In this study, Kay and Mitchell's exponential model will be applied to the present experimental data. How well a particular function fits the experimental data is known as the *goodness of fit* and is quantified by means of the *correlation coefficient*, r, or the *coefficient of determination*, r^2. Regression analyses can be applied to many situations in geotechnical engineering and earth sciences which involve the analysis of bivariate and mutlivariate data. The coefficient of correlation is achieved by minimising the sum of the standard deviations and the technique is known as the method of least squares.

When:

- $r = +1$ the data fit a model of positive slope, perfectly;
- $r = 0$ the data exhibit no correlation;
- $r = -1$ the data fit a model of negative slope, perfectly.

Various computer programs such as *Excel®*, *DeltaGraph®* use the method of least squares to determine the coefficient of determination. The *DeltaGraph®* software was used in this study mainly because of its ease of use and quality of presentation.

4.3.3.1 Exponential Model

In order to smooth the cumulative frequency distribution curves developed in §4.4.2, an exponential model, as used previously by Kay and Mitchell (1990), was adopted in this study. As mentioned in the preceding section, *DeltaGraph®* was used to develop curves for the cumulative frequency charts shown in Figures 4.7 to 4.9. Equation (4.1) was used in *DeltaGraph®* to define the exponential model.

$$CF = 100(1-ae^{-bECW}) \tag{4.1}$$

where: CF = the percent cumulative frequency;
ECW = the equivalent crack width;
a, b = curve fitting parameters.

The same model was used on all six groups dealt in this study, that is ABV houses on S, M and H sites and ASB houses on S, M and H sites. These curves are superimposed on Figures 4.7 to 4.9 and are reproduced in Figures 4.10 to 4.12. The curve fitting parameters and the coefficients of determination, r^2, are shown in Table 4.8.

Table 4.8 Curve fitting parameters.

Site Reactivity	ABV			ASB		
	a	b	r^2	a	b	r^2
S	0.912	1.131	0.998	1.173	0.452	0.970
M	0.898	1.186	0.997	1.137	0.613	0.988
H	0.923	0.476	0.986	1.042	0.458	0.974

It is evident from Table 4.8, that the values of r^2 for all the six groups are very close to one. Consequently, with reference to previous sections, it can be concluded that the exponential model correlates well with the measured data points. It is evident from Figures 4.10 and 4.11, that two of the curves are showing negative cumulative frequencies, realistically which is incorrect. Consequently, this research has further used the method of maximum likelihood, an alternative technique to overcome this limitation, is described in the later sections.

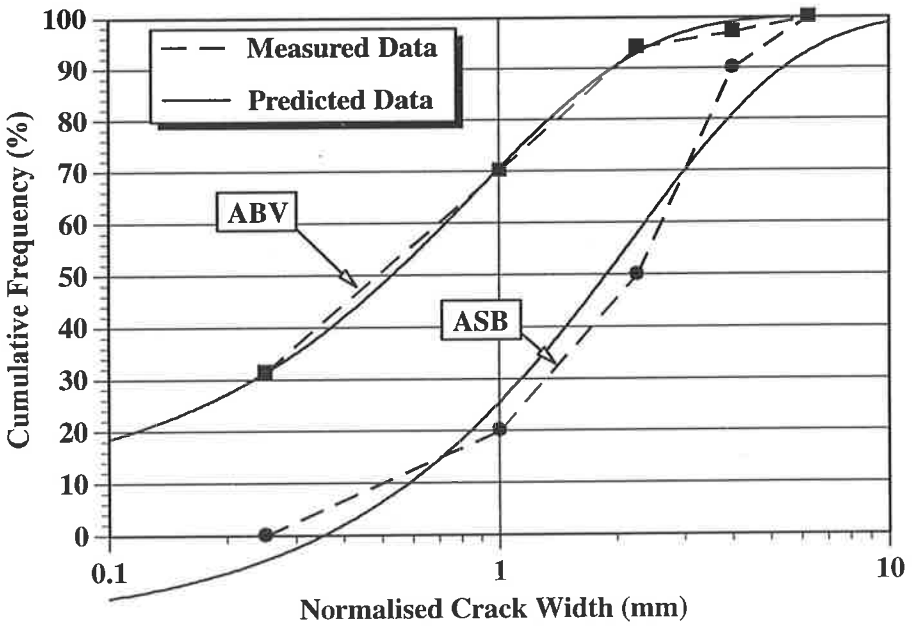


Figure 4.10 Predicted cumulative frequencies for S sites.

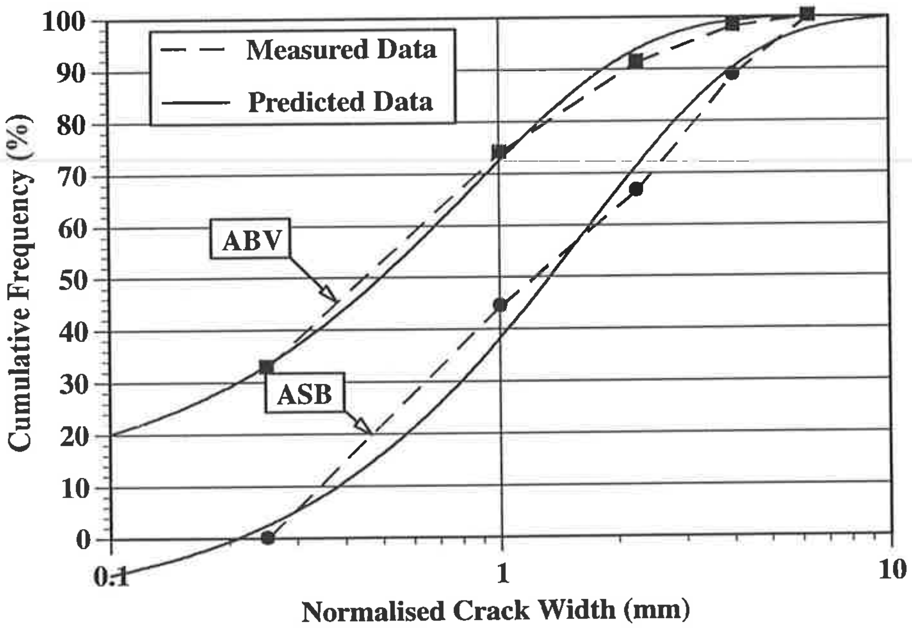


Figure 4.11 Predicted cumulative frequencies for M sites.

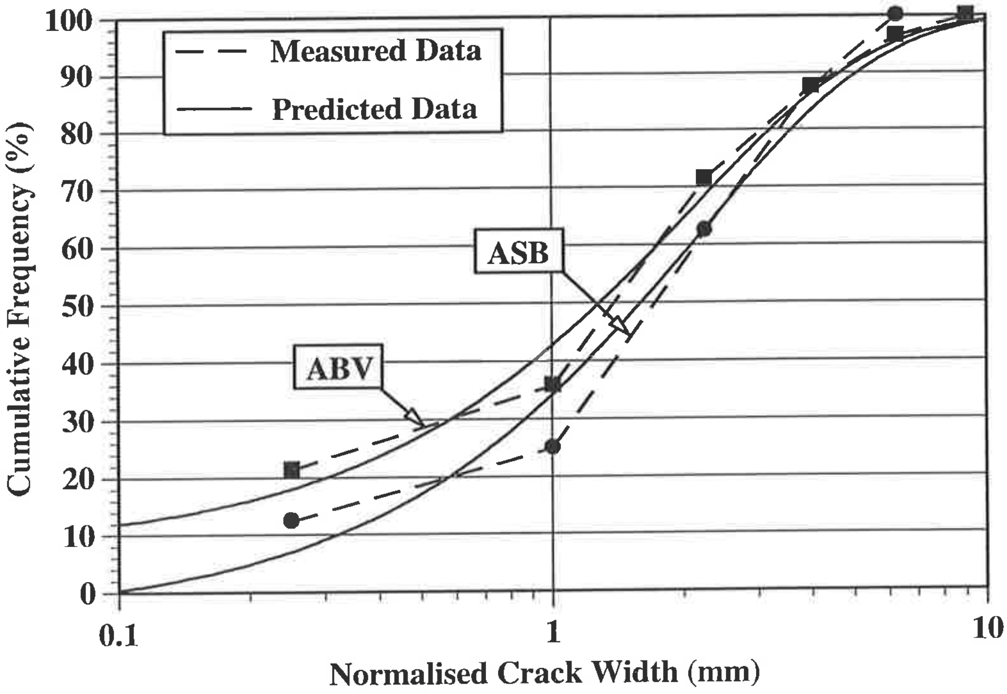


Figure 4.12 Predicted cumulative frequencies for H sites.

4.3.3.2 Long-Term Effects

Kay and Mitchell (1990) argued that a procedure based on measurements taken within the first ten years of construction are likely to be unconservative for engineering design purposes. In addition, both theoretical and practical evidence indicate that cracks measured over longer periods may be more severe. Mitchell (1980) demonstrated using diffusion theory, that centre heave effects under constant boundary conditions, may not reach equilibrium for several years, perhaps as many as ten. In addition, extreme environmental conditions such as periods of drought, are likely to result in extreme damage.

A statistical survey, conducted by Domaschuk et al. (1984) to ascertain the extent and severity of damage to houses built on expansive soils Canada, clearly demonstrated that there is an increase in damage levels with the age. A similar survey conducted by Osman and Hamadto (1984) in Sudan also concluded that the crack widths increase with the age of the building.

As a result, Kay and Mitchell (1990) doubled the crack widths measured in their study. Leach et al. (1995) also incorporated a long term factor of two in their study of extremely expansive sites. In light of this previous work it is not unreasonable for this study to also incorporate a factor of two to account for long-term effects. This is included in the analysis by halving the exponent in Equation 4.1 as shown in Equation 4.2.

$$CF = 100(1 - ae^{-bECW/2}) \tag{4.2}$$

However, whether the measured crack widths are doubled or multiplied by some other factor greater than unity, remains a matter of conjective because of the limited number of studies carried out in this regard.

4.4 PROBABILITY MODEL

The probability of exceedence of a crack width for a particular beam depth has been taken as 100 – *CF*. Allowing for long-term performance, this probability of exceedence, *POE*, is given by:

$$POE = ae^{-bECW/2} \tag{4.3}$$

From Equation 4.3, it is possible to obtain, for a given site reactivity, the probability of exceedence of a equivalent crack width. In order to determine the associated crack widths, Equation 4.3 was modified by implementing the previously assumed inverse proportionality between crack width and raft stiffness as discussed in §2.6.2. The modified form is given by the Equation 4.4.

$$POE = ae^{-bEIw_c/2k} \tag{4.4}$$

where: k = arbitrary raft stiffness from Table 4.1.

This relationship was then used to calculate the crack widths for various probability of exceedence, that is, 1%, 5%, 10%, and 50%. The probability of exceedence is the likelihood that, throughout the life of the structure, that a particular crack width will be exceeded. Rearranging Equation (4.4) in terms of crack widths for various probabilities of exceedence, *POE*, yields:

$$W_c = \frac{2k}{bEI} \ln\left(\frac{a}{POE}\right) \tag{4.5}$$

where: EI = unit raft stiffness.

Raft stiffnesses for various external beam depths were calculated using the standard arrangement of stiffening beams as indicated by Figure 5.1 of the AS2870.1-1988 (Standard Association of Australia 1988a), which is reproduced in Figure 4.13. This figure specifies that the slab thickness is 100 mm; external footing beams for solid masonry walls are 400 mm wide and all other footing beams are 300 mm wide. In addition, the maximum footing beam spacing is 4 metres. In this study an 8 metre wide dwelling and footing has been used, any such dimension of the dwelling can be argued upon still having very less effects on the unit stiffness. As a result, two external beams and one internal beam contribute to the stiffness, *EI*, of the footing. In order to calculate *EI*, a value of *E* is required. As the majority of footings constructed in Adelaide use concrete with 28 day compressive strength f'_c of 20 MPa, the Australian concrete structures code (AS3600-1988) specifies that, $\mathrm{E} = \frac{5050 f'_c}{2}$ = 11.3 GPa. By calculating the footing stiffness, and substituting the result into Equation (4.5), a value of crack width, W_c, was evaluated.

An example of such a calculation is shown below.

Calculate the associated crack width, W_c, for a 10% probability of exceedence (*POE*); 400 mm footing beam depth; articulated brick veneer (ABV) dwelling built on an H site.

E = 11.3 GPa

I = $9.1\mathrm{E}+10^9$ mm^4 (from *MS Excel* 4.0 spreadsheet)

a = 0.923 (from Table 4.8)

b = 0.476 (from Table 4.8)

k $= 25$ MN/m^2 (from Table 4.1)

Substituting these values into Equation (4.5), yields $W_c = 18$ mm.

Similar calculations were performed for several footing depths, and probabilities of exceedence (*POE*) of 1%, 5%, 10% and 50%, in order to generate probabilistic design charts. These are shown in Figures 4.14 to 4.19. The application of these design charts will be discussed in Chapter 5.

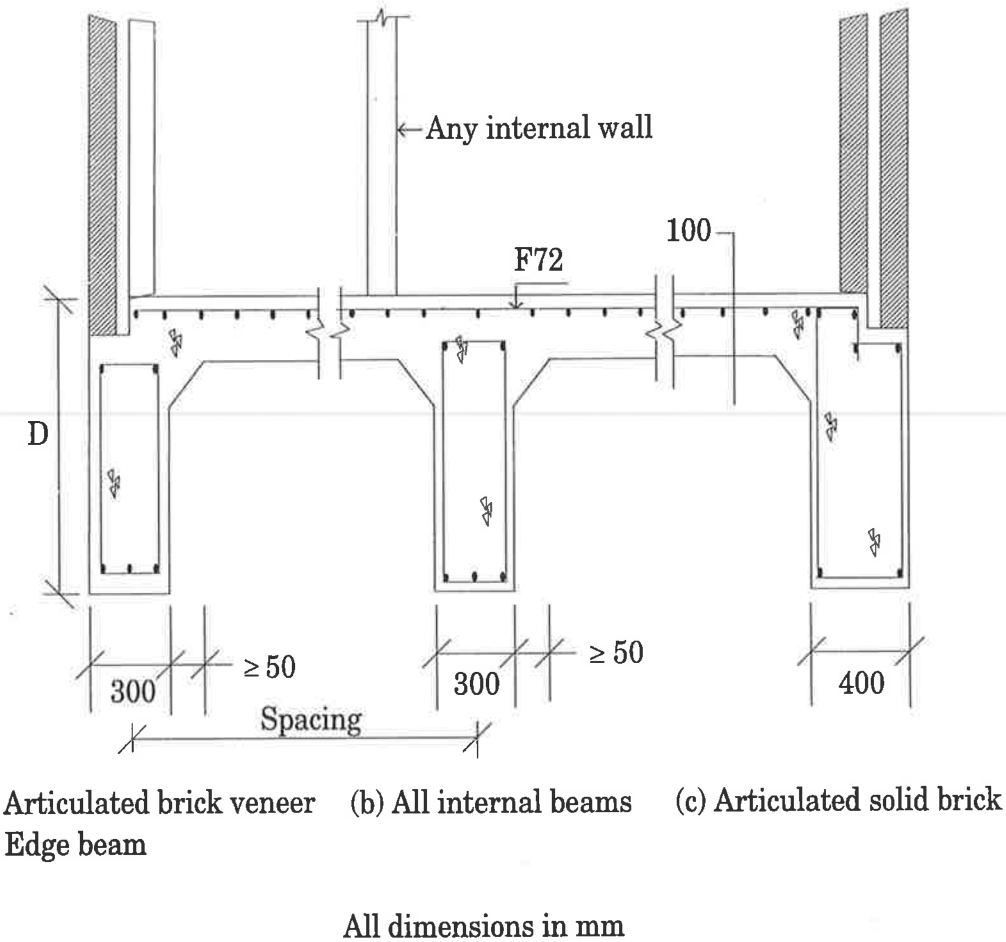


Figure 4.13 Standard stiffened raft design.

(After AS2870.1-1988)

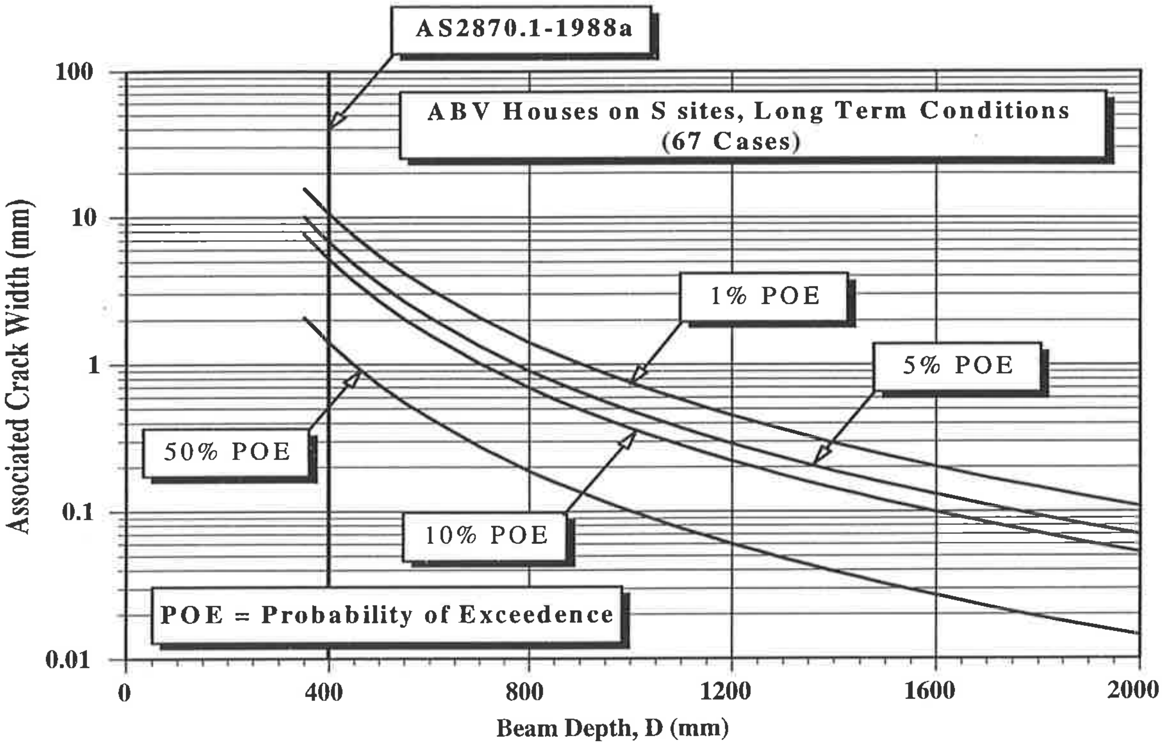


Figure 4.14 Probabilistic design charts for ABV houses on S sites.

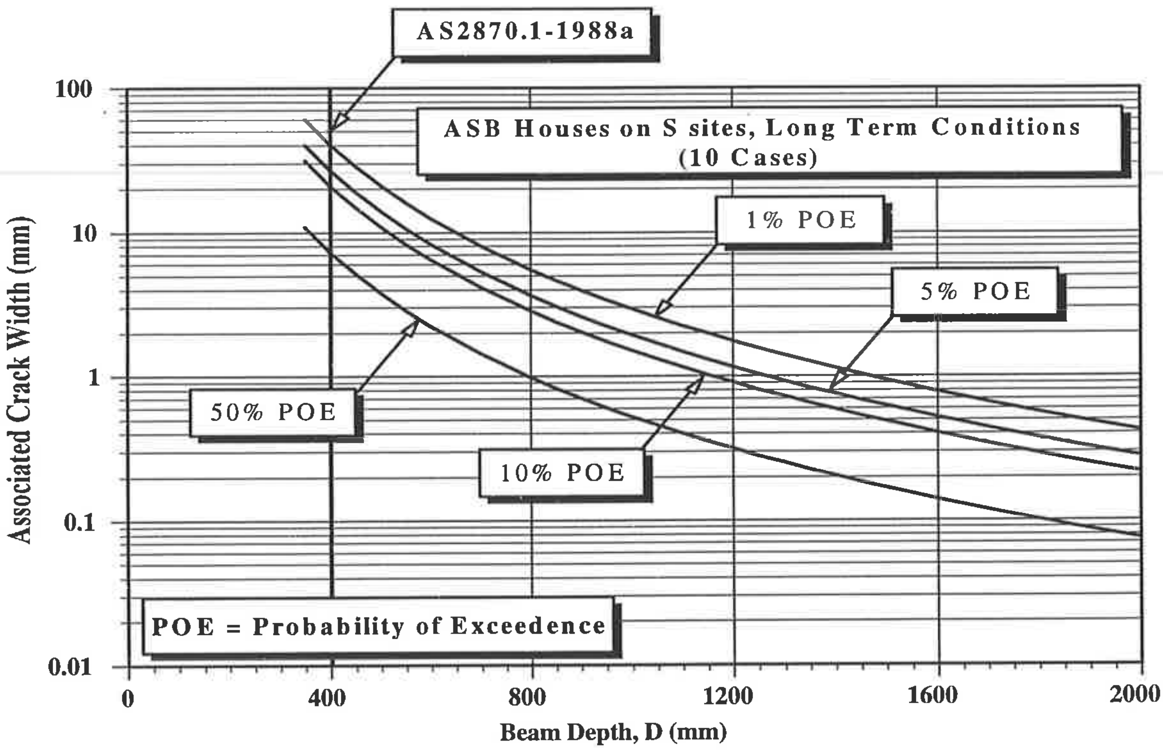


Figure 4.15 Probabilistic design charts for ASB houses on S sites.

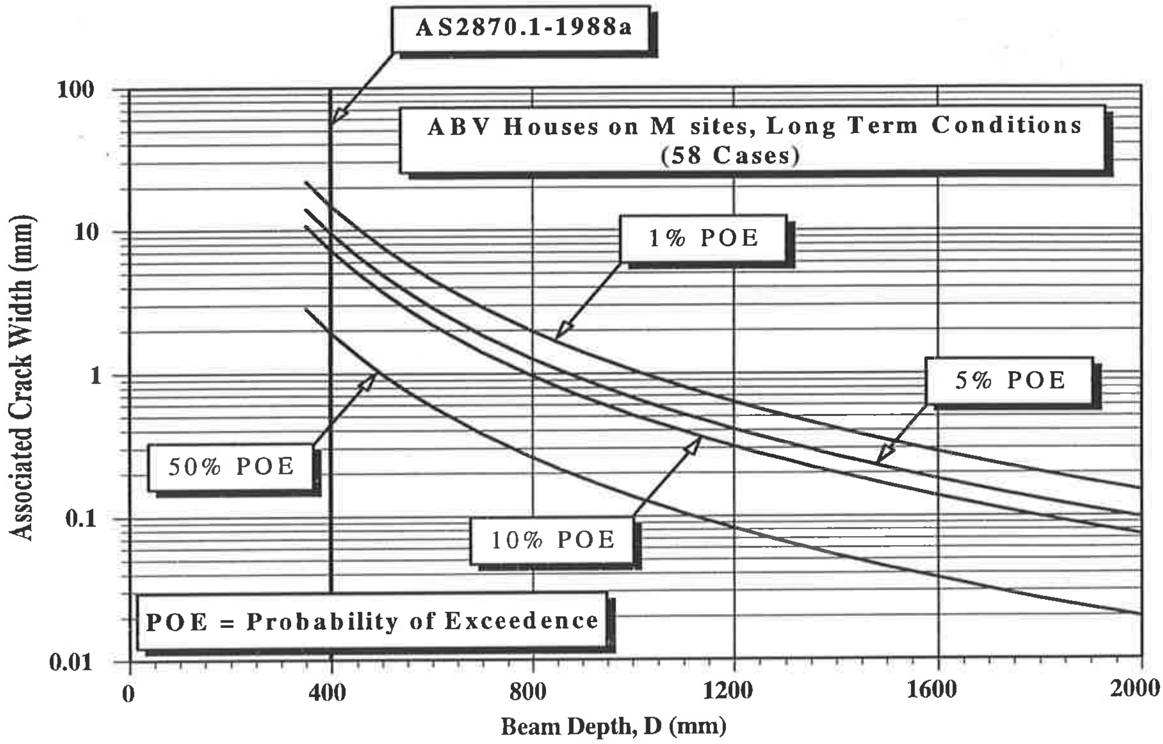


Figure 4.16 Probabilistic design charts for ABV houses on M sites.

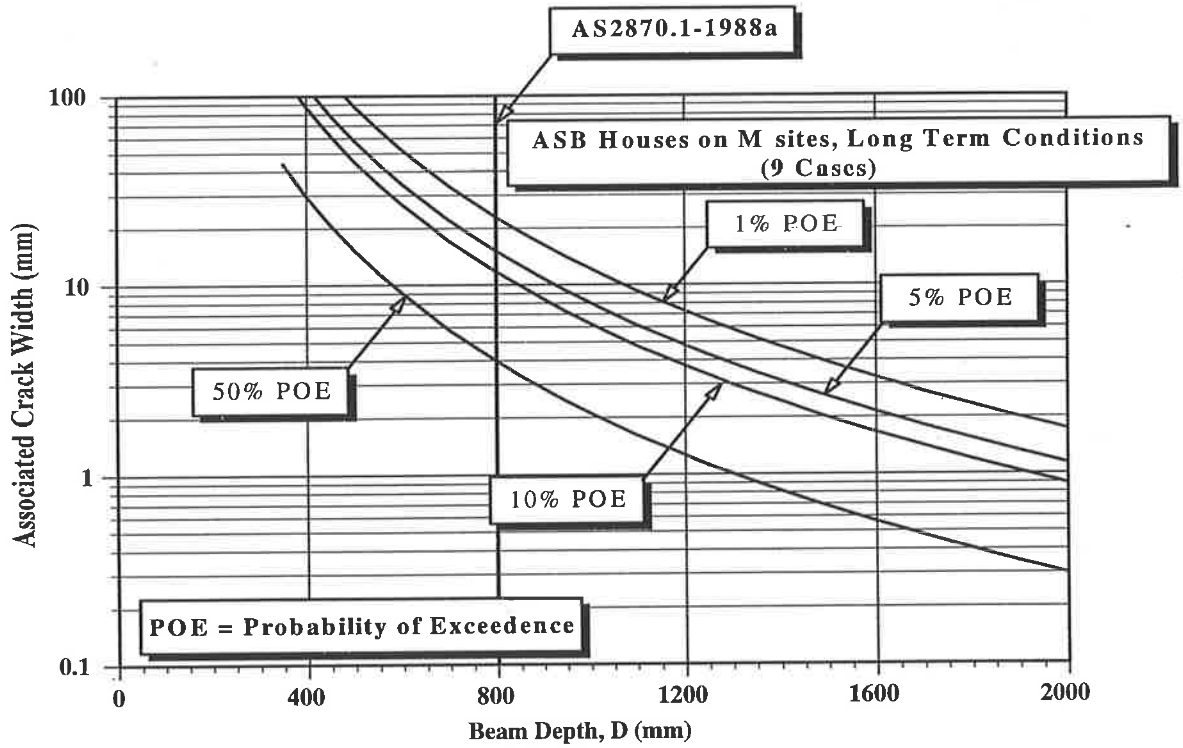


Figure 4.17 Probabilistic design charts for ASB houses on M sites.

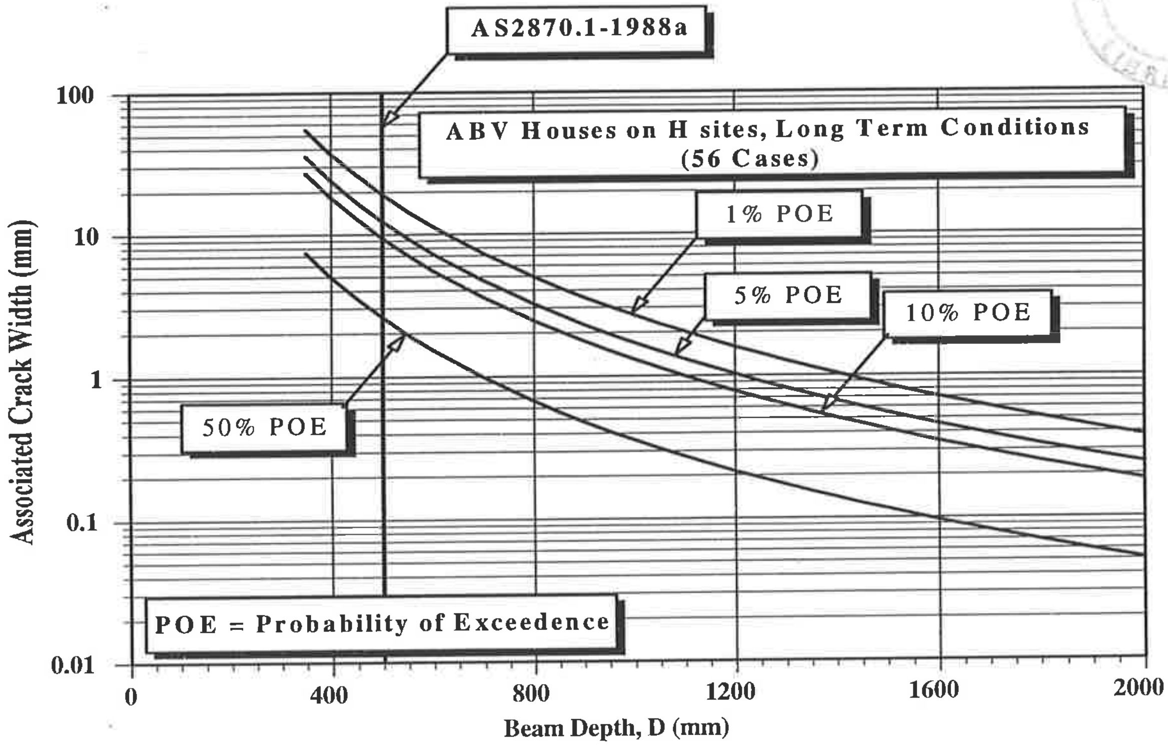


Figure 4.18 Probabilistic design charts for ABV houses on H sites.

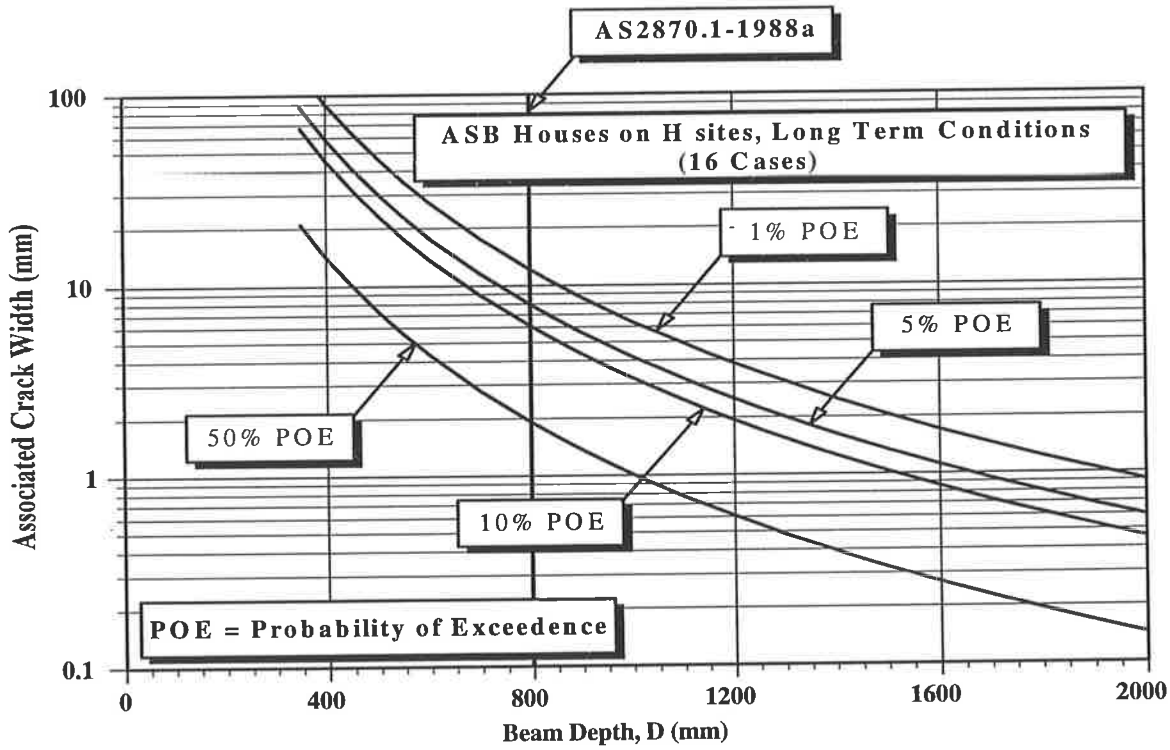


Figure 4.19 Probabilistic design charts for ASB houses on H sites.

4.5 ALTERNATIVE ANALYSIS OF THE DATA USING THE METHOD OF MAXIMUM LIKELIHOOD

The charts presented in Figures 4.14 to 4.19, which were based on the approach adopted by Kay and Mitchell (1990), suffer from two significant errors.

Firstly, as mentioned previously, two of the cumulative frequency distribution functions (Figures 4.10 and 4.11) indicate negative frequencies, which are an impossible outcome.

Secondly, effects of zero thickness cracks were not considered and were grouped into the 0-0.25 mm range.

An alternative approach, based on the method of maximum likelihood was developed in association with Jarrett (1996). This approach is detailed in the following sections.

4.5.1 Estimation of Parameters

The method of maximum likelihood can be applied in situations where the true distribution of the sample is known. Furthermore, the method has a strong intuitive appeal by which true parameter can be estimated which maximises the likelihood function. The equivalent crack widths so obtained in §4.2 were again sub-divided into zero cracked houses and non-zero cracked houses. The total sum, Σy_i, for non-zero cracked houses were determined and number of zero cracked houses, m_c, and non-zero cracked houses, n_c, were also determined. Then the parameters, a and b were estimated using Equations (4.6) and (4.7) and are given in Table 4.9. These equations were derived using the first principles of binomial and exponential distributions.

$$a = \frac{n_c}{m_c + n_c} \tag{4.6}$$

$$b = \frac{n_c}{\Sigma ECW} \tag{4.7}$$

where: m_c = total number of zero cracked houses;

n_c = total number of non-zero cracked houses; and

ECW = equivalent crack widths of non zero cracked houses.

Table 4.9 Curve fitting parameters.

Site Reactivity	ABV			ASB		
	a	b	r^2	a	b	r^2
S	0.686	0.823	1.000	1.000	0.380	1.000
M	0.689	0.868	1.000	1.000	0.550	1.000
H	0.800	0.390	1.000	0.880	0.500	1.000

It is evident from Table 4.9, that the values r^2 for all the six groups are one. Consequently with reference to §4.3.3, it can be concluded that the exponential model correlates perfectly with the measured data points.

4.5.2 Cumulative Frequency Distributions of the Predicted Data

The cumulative frequency distributions of the measured data were calculated in the same manner as described in §4.3.3.1. However, in this case the curve fitting parameters a and b given in Table 4.9 were substituted into Equation (4.1). The resulting distributions are shown in Figures 4.20 to 4.25. It is evident from these graphs, that none of the curves show negative cumulative frequencies.

The following section discusses the technique used to estimate crack widths and associated probabilities of exceedence.

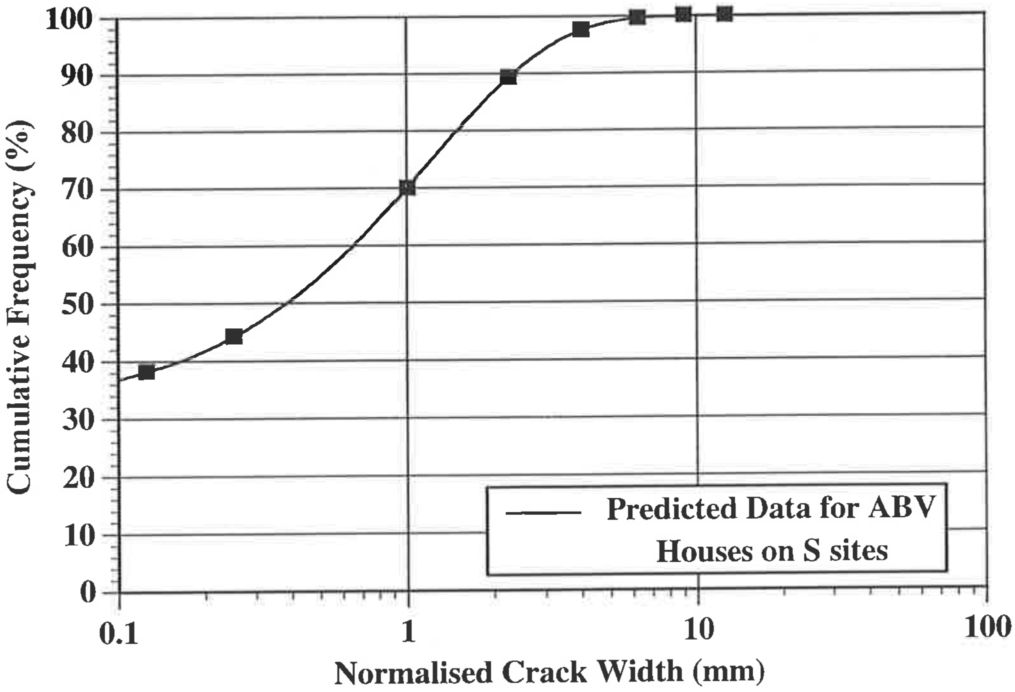


Figure 4.20 Predicted cumulative frequency for ABV houses on S sites.

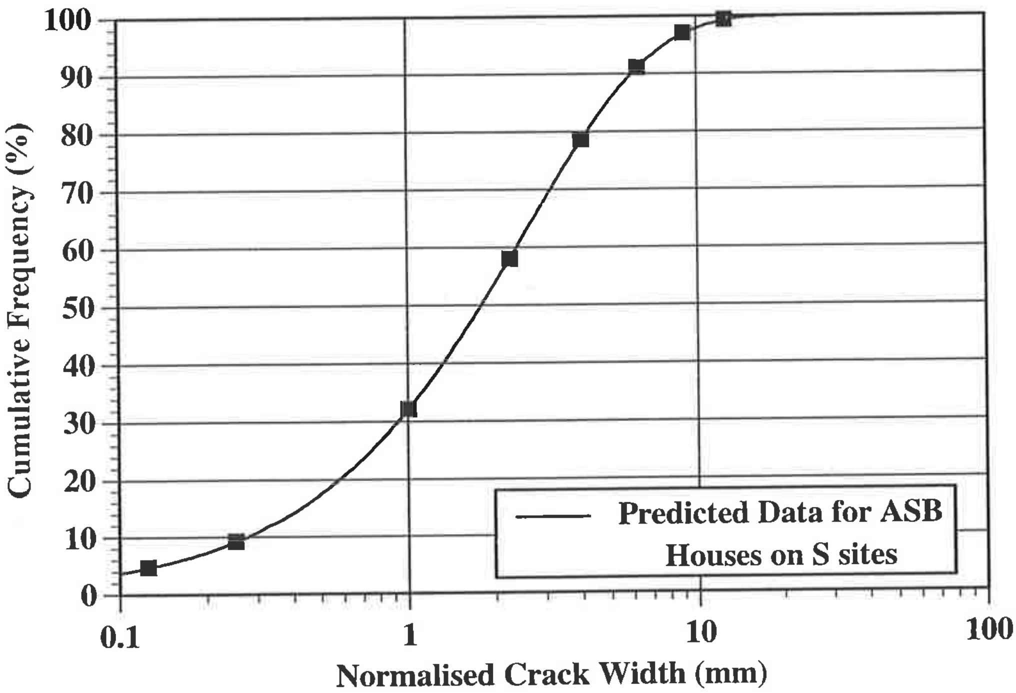


Figure 4.21 Predicted cumulative frequency for ASB houses on S sites.

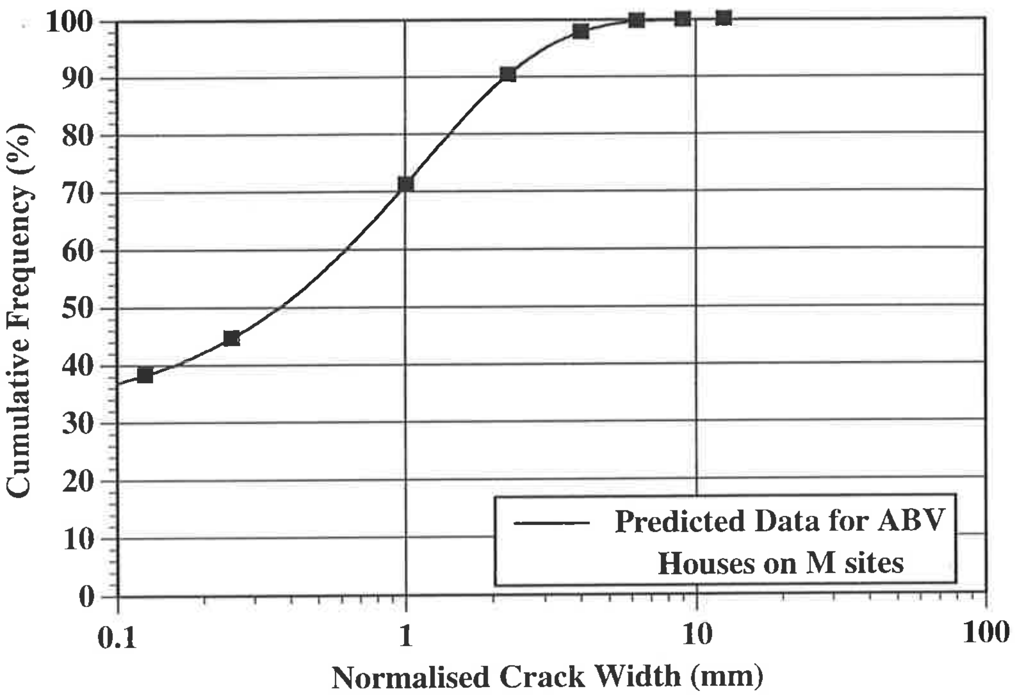


Figure 4.22 Predicted cumulative frequency for ABV houses on M sites.

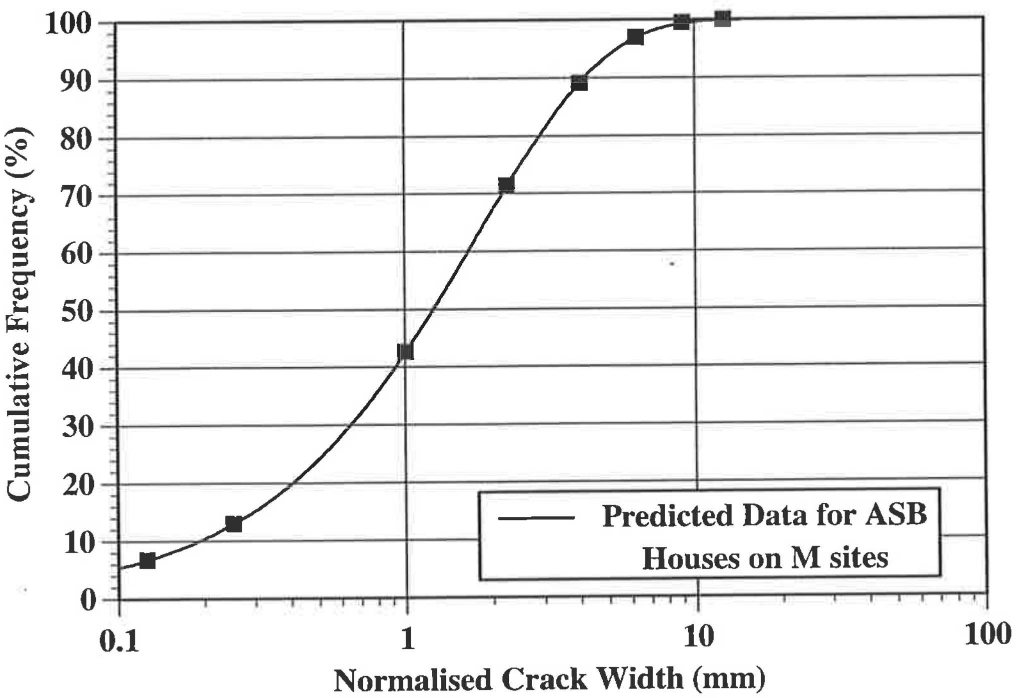


Figure 4.23 Predicted cumulative frequency for ASB houses on M sites.

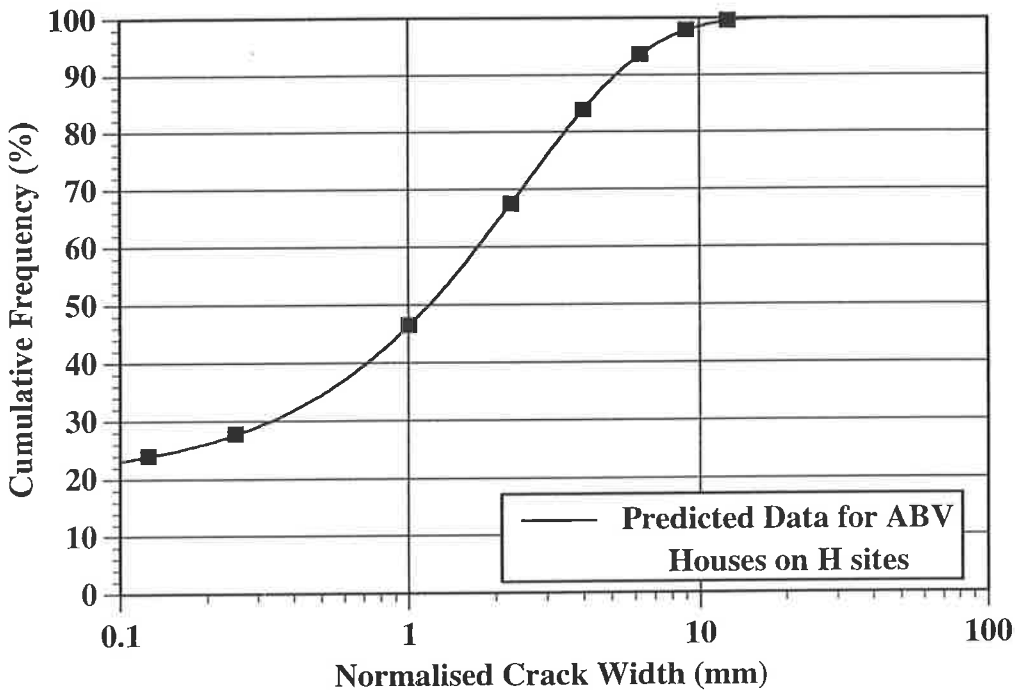


Figure 4.24 Predicted cumulative frequency for ABV houses on H sites.

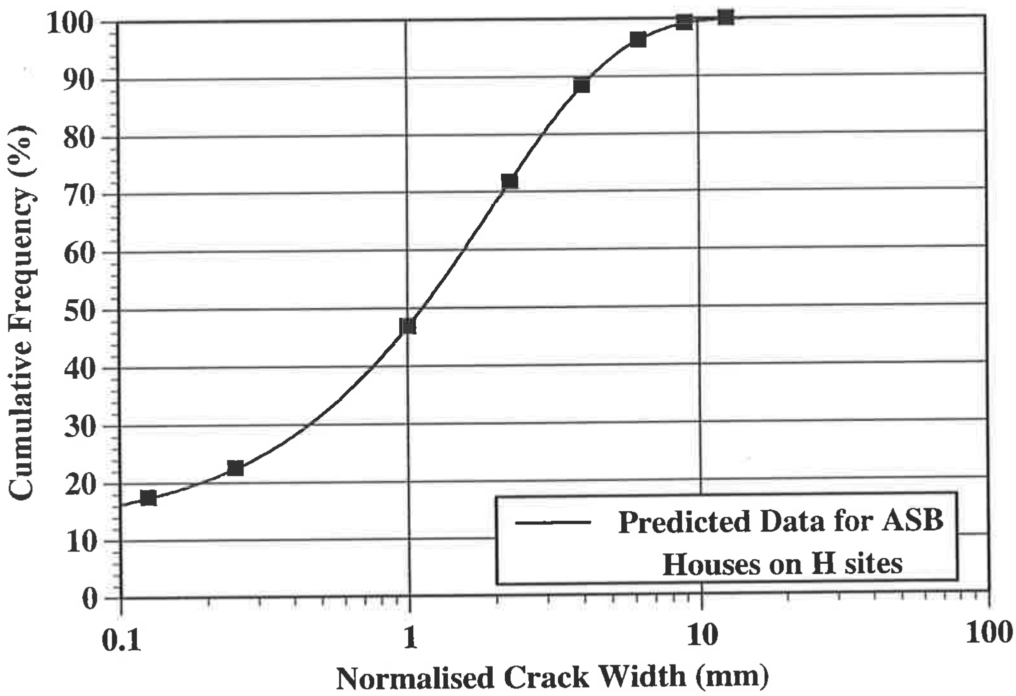


Figure 4.25 Predicted cumulative frequency for ASB houses on H sites.

4.5.2.1 Modified Probabilistic Design Technique

The maximum crack widths were determined for various beam depths under different probabilities of exceedence using the relationship shown in Equation (4.8).

$$POE = ae^{-bECW} \tag{4.8}$$

where: POE = probability of exceedence;
a, b = curve fitting parameters;
ECW = equivalent crack width.

The arbitrary stiffnesses, which were used to normalise the data earlier implemented here to derive a relationship in terms of equivalent crack width and it is shown in Equation (4.9).

$$ECW = \frac{EIW_c}{k} \tag{4.9}$$

where: EI = unit raft stiffness;
W_c = associated crack width;
k = arbitrary raft stiffness from Table 4.1.

Now substituting Equation (4.10) in Equation (4.9) and rearranging in terms of associated crack width, W_c yields:

$$W_c = \frac{k}{bEI} \ln\left(\frac{a}{POE}\right) \tag{4.10}$$

where: POE = probability of exceedence.

This equation is similar to that used previously (Equation 4.3), however, the long-term factor of two has not been used. The long-term factor has not been used because it is believed that the sample population is biased towards the more damaged houses. This is because the occupiers who agreed to participate in the research were more likely to do so if their dwelling suffered a higher level of cracking. It is likely that the occupiers, whose dwellings exhibited no distress, were reluctant to participate in the study because they felt there was little to gain, personally, by being involved. As a consequence, the sample population is likely to be skewed more heavily towards the dwellings with greater cracking. Hence, by including a long-term factor of two, or some other value, would distort the distribution further.

Using Equation (4.10), various probabilistic design curves were generated for each group of site reactivity and super-structure. Six such charts are shown in Figures 4.26 to 4.31. Superimposed on these charts are the footing depths recommended by AS2870.1-1988a (Standards Association of Australia, 1988a). AS2870.1-1988a states that the footing sizes recommended by the deemed-to-comply specifications are 5% likely to yield crack widths in excess of 5 mm. It does not state, however, how these figures have been derived. The crack widths associated with the footing sizes recommended by AS2870.1-1988a and a 5% *POE*, yield values of 5 mm or greater, in accordance with the standard and, as a consequence, seem appropriate.

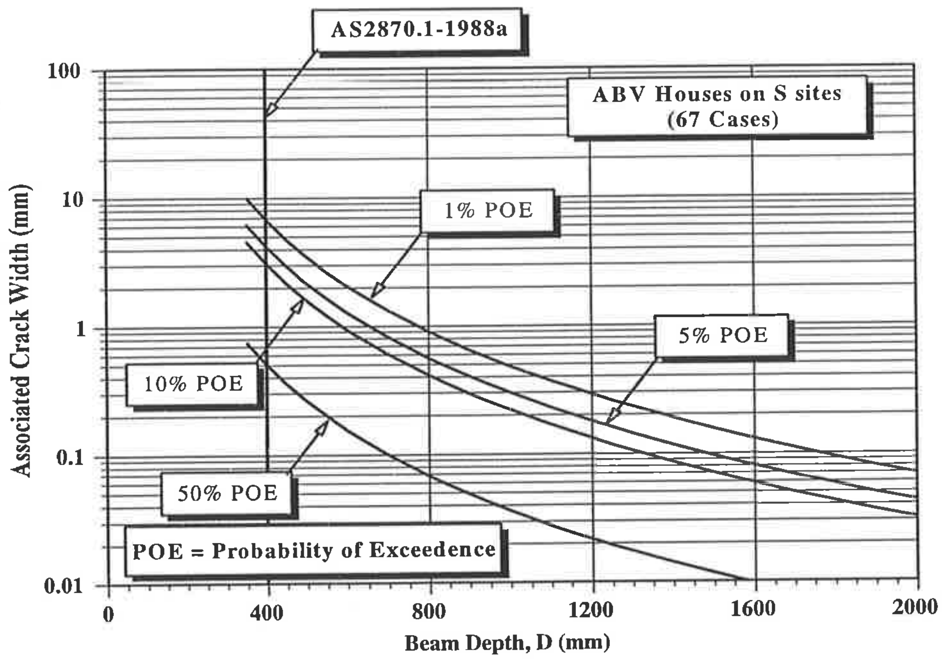


Figure 4.26 Probabilistic design charts for ABV houses on S sites based on the method of maximum likelihood.

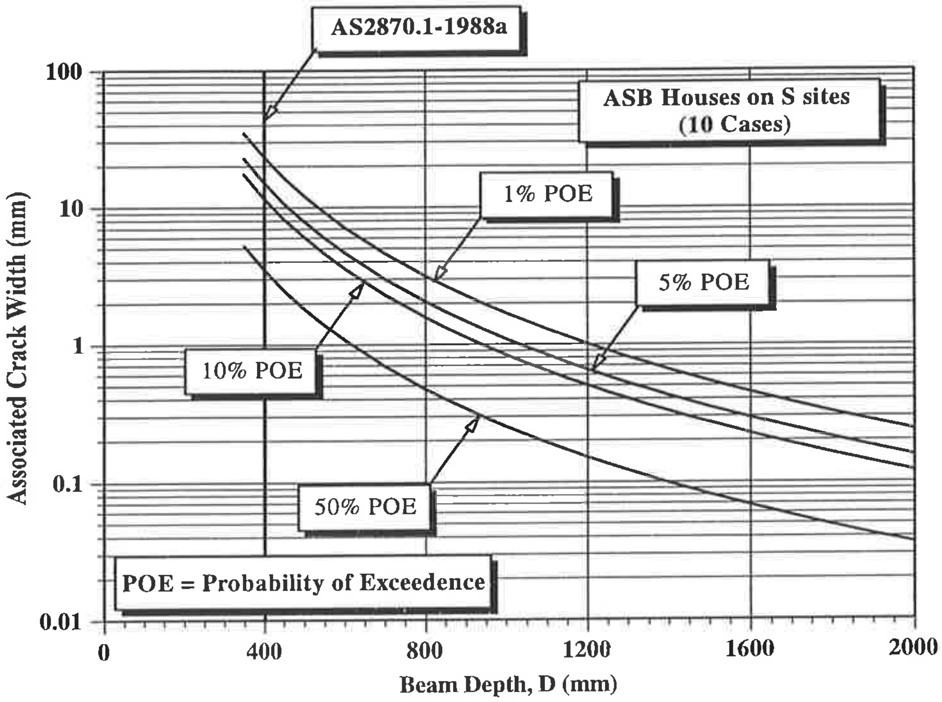


Figure 4.27 Probabilistic design charts for ASB houses on S sites based on the method of maximum likelihood.

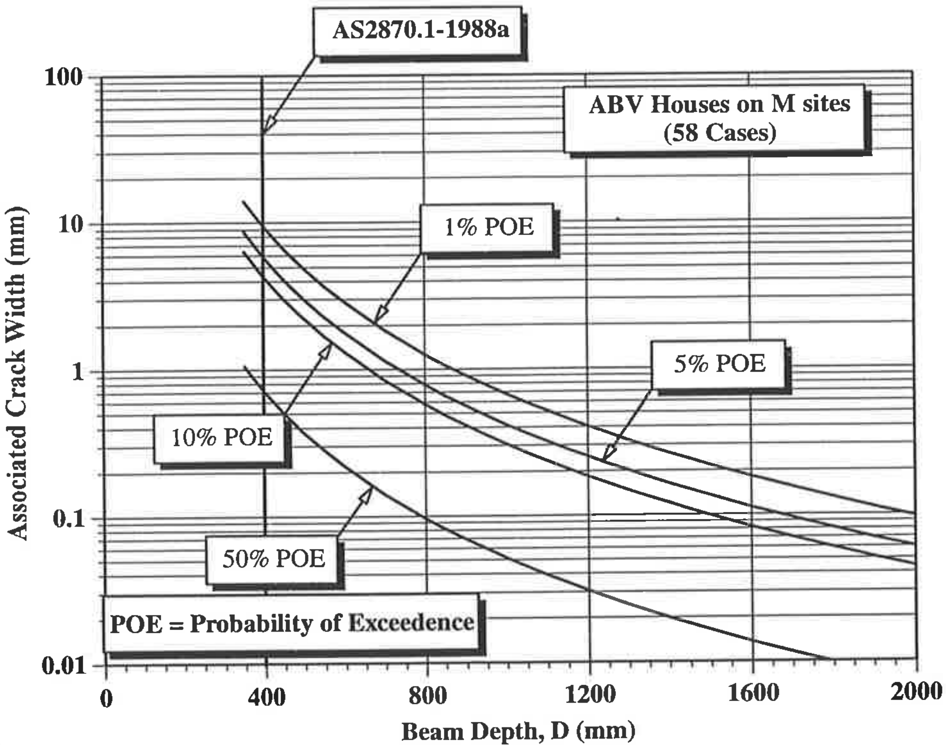


Figure 4.28 Probabilistic design charts for ABV houses on M sites based on the method of maximum likelihood.

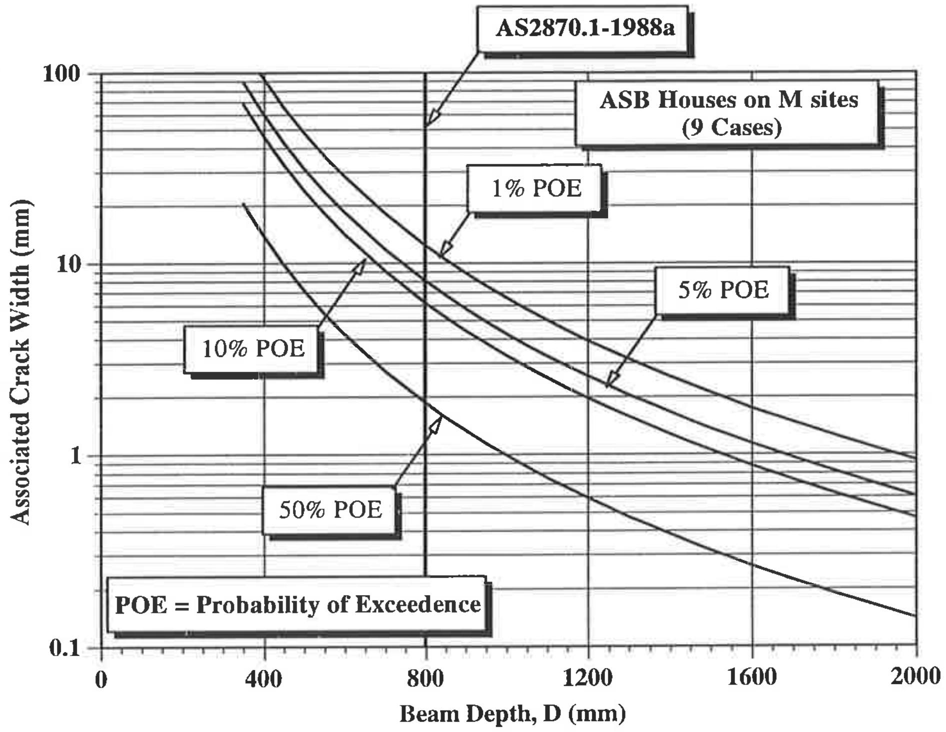


Figure 4.29 Probabilistic design charts for ASB houses on M sites based on the method of maximum likelihood.

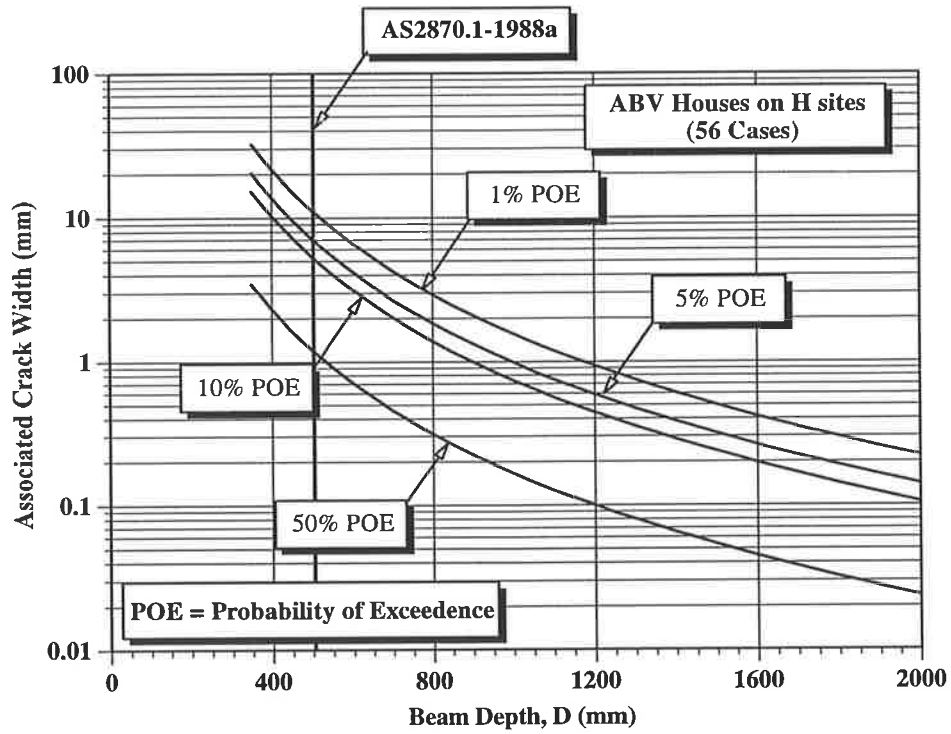


Figure 4.30 Probabilistic design charts for ABV houses on H sites based on the method of maximum likelihood.

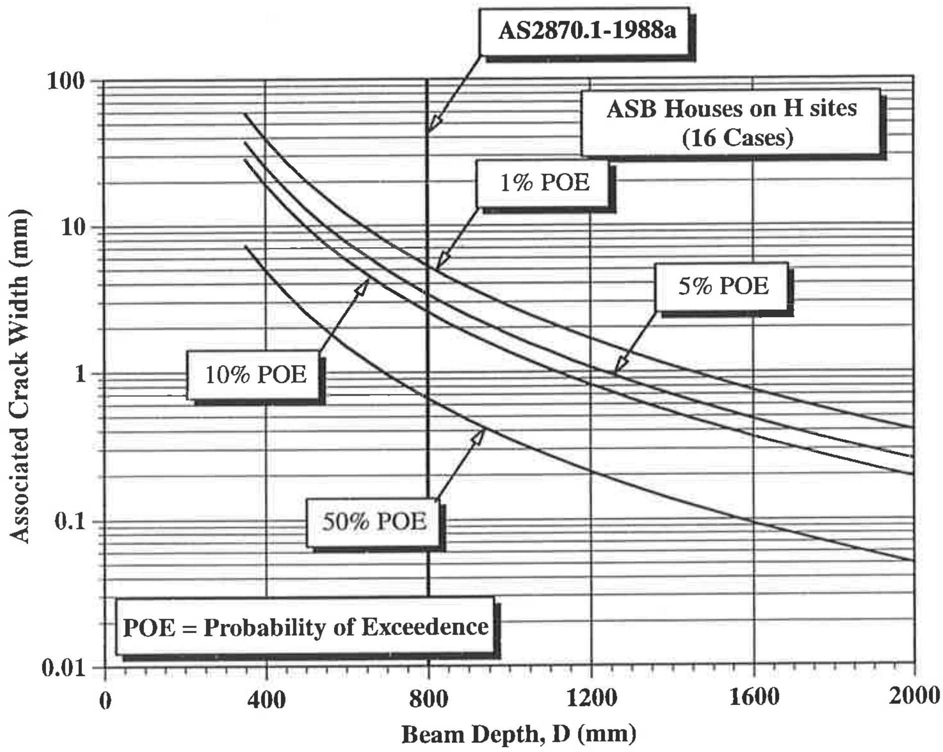


Figure 4.31 Probabilistic design charts for ASB houses on H sites based on the method of maximum likelihood.

In order to generate design charts which account for long-term effects, in a manner similar to that adopted by Kay and Mitchell (1990), that is, using a factor of two, Equation (4.10) can be amended as shown in Equation (4.11).

$$W_c = \frac{2k}{bEI}\ln\left(\frac{a}{POE}\right) \tag{4.11}$$

By substituting into Equation (4.11), probabilistic design charts which account for long-term effects have been generated for each site reactivity group and super-structure type. These are shown in Figures 4.32 to 4.37. It should be re-emphasised, however, that due to the likely biasedness of the measured data, it is probable that these long-term design charts are over conservative, that is, the crack widths are likely to be over-estimated.

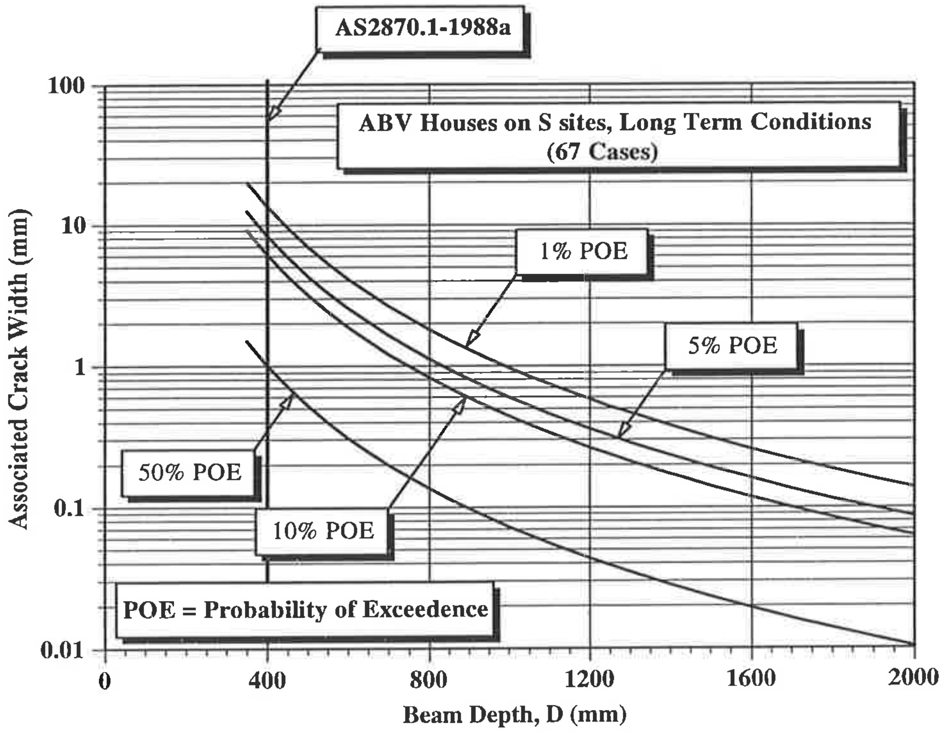


Figure 4.32 Probabilistic design charts for ABV houses on S sites with long term effects based on the method of maximum likelihood.

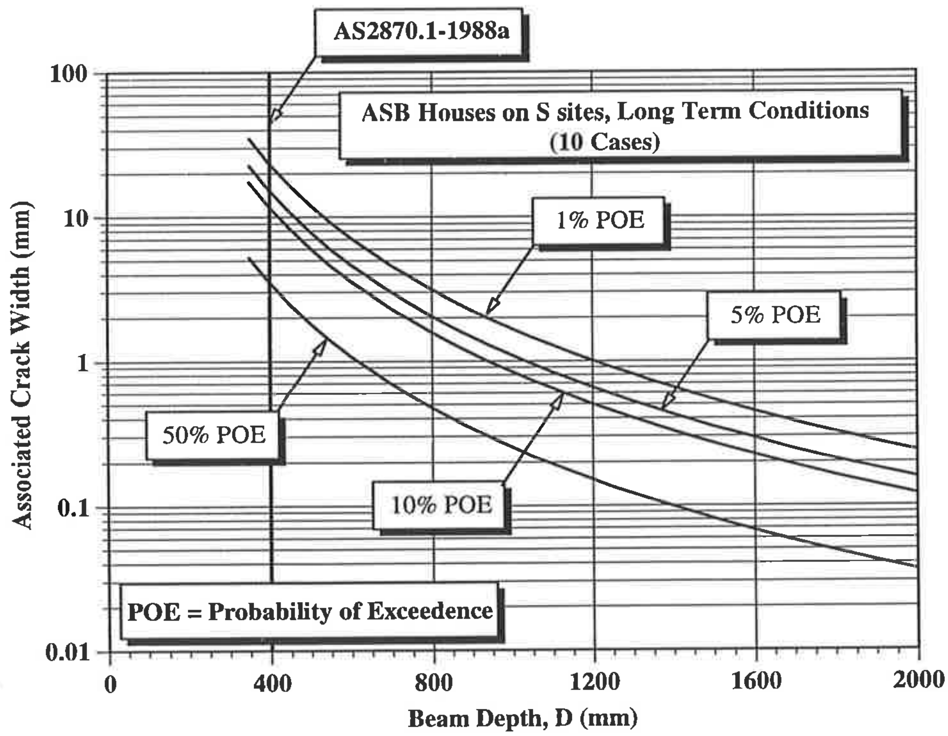


Figure 4.33 Probabilistic design charts for ASB houses on S sites with long term effects based on the method of maximum likelihood.

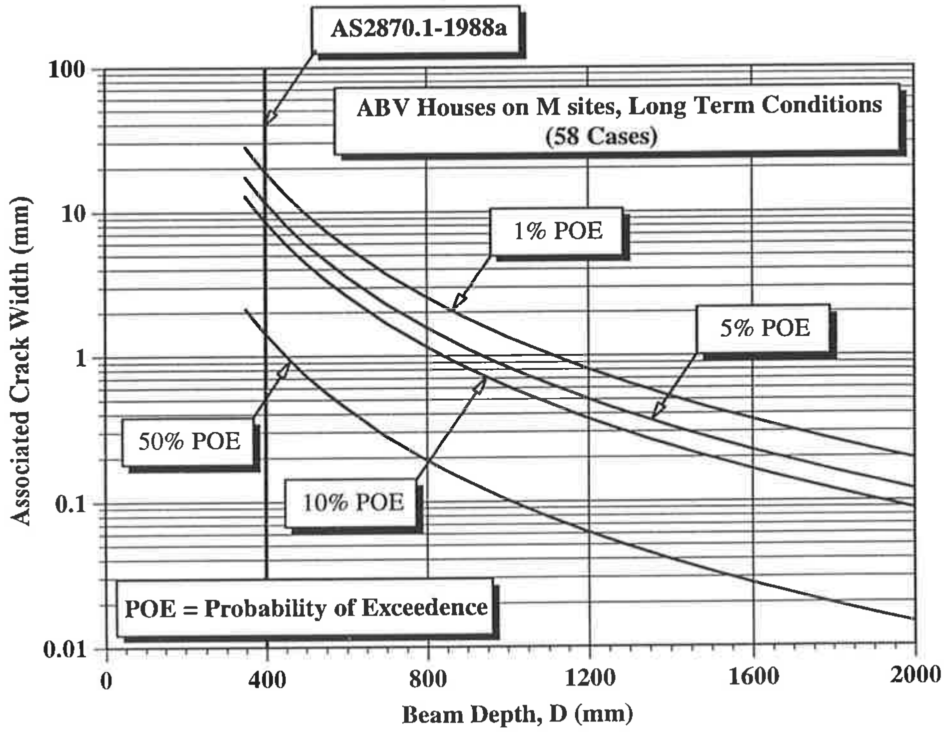


Figure 4.34 Probabilistic design charts for ABV houses on M sites with long term effects based on the method of maximum likelihood.

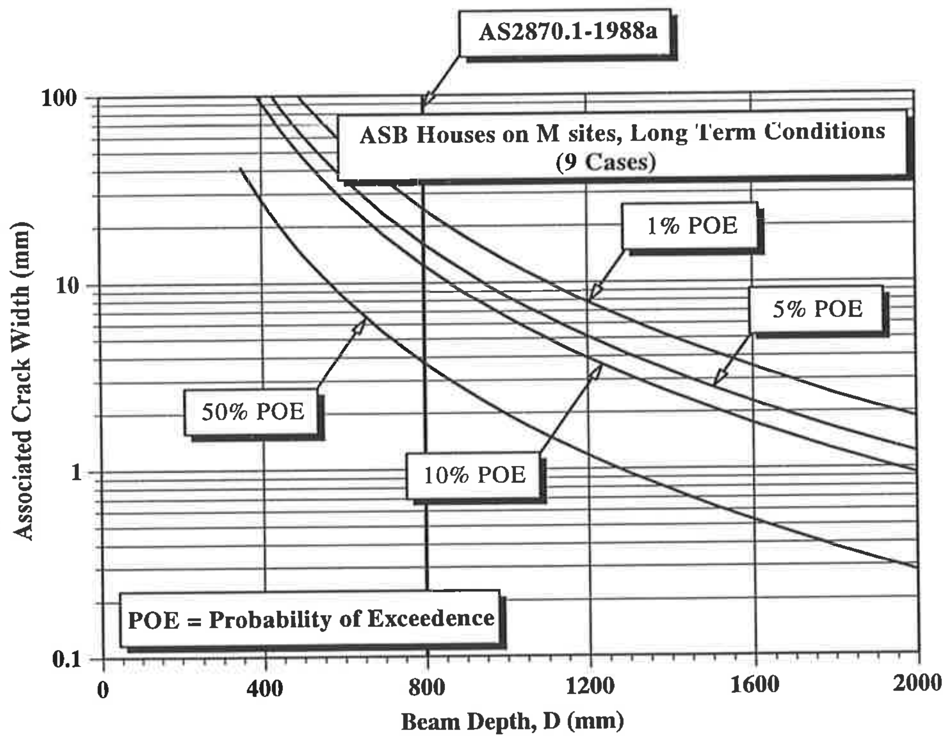


Figure 4.35 Probabilistic design charts for ASB houses on M sites with long term effects based on the method of maximum likelihood.

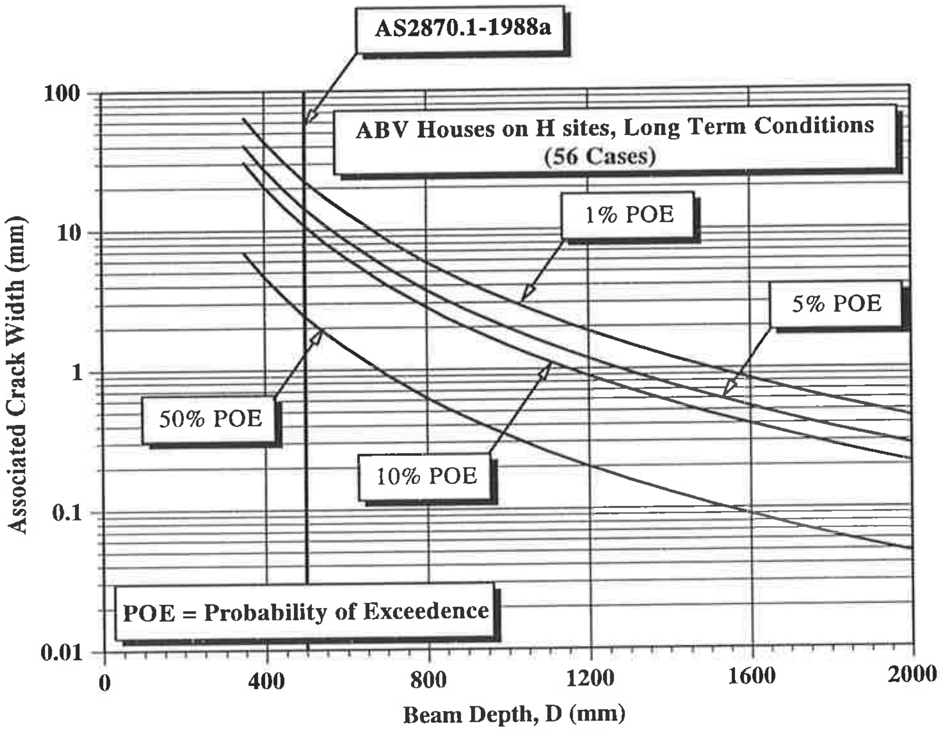


Figure 4.36 Probabilistic design charts for ABV houses on H site with long term effects.

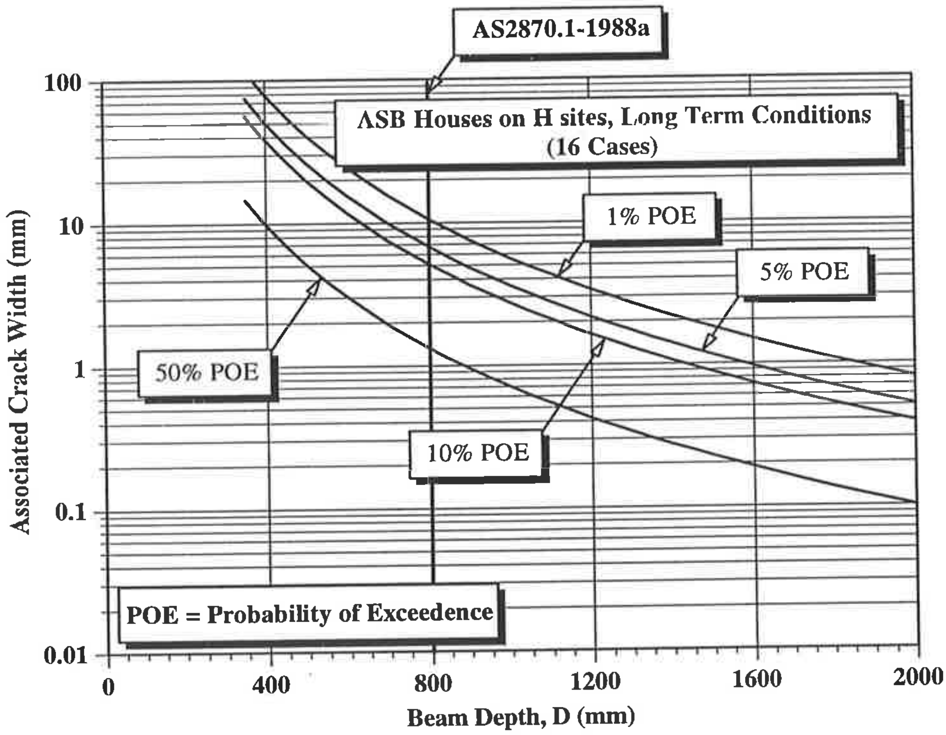


Figure 4.37 Probabilistic design charts for ASB houses on H sites with long term effects based on the method of maximum likelihood.

4.5.3 Summary

The preceding sections have discussed the development of a series of probabilistic charts for raft footings built on expansive soils. These charts will be applied to a series of case studies and compared with traditional forms of footing design in Chapter 5. The following section examines the limitations of the probabilistic model.

4.6 LIMITATIONS OF PROBABILISTIC MODEL

It is evident from the preceding sections that the probabilistic method has a number of advantages and at the same time it also has a number of limitations such as:

- The final model is dependent on the quality and quantity of information within the database; and
- The probabilistic method is only able to account for design situations which are included in this database.

The following section examines the confidence intervals of the model.

4.6.1 Confidence Intervals

In geotechnical engineering and the earth sciences, it is common to calculate 95% confidence intervals. These intervals show the limits within which 95% of the data are likely to lie, or where the sample mean is likely to lie 95% of the time.

The general form of the confidence intervals, *CI*, is expressed by:

$$CI = m_s \pm z_b\left(\frac{\sigma}{\sqrt{e}}\right) \tag{4.12}$$

where: m_s = mean of the sample;

z_b = constant;

σ = standard deviation; and

s = sample size.

Equation (4.12) is modified to include the exponential distribution of non-zero values of crack width in Equation (4.13). The standard deviation of this relationship was derived using a Taylor's series expansion. A factor of 1.96 was used in Equation (4.13) to determine 95% confidence intervals from the normal distribution.

$$CI = \frac{k}{EI}\left\{\ln\left(\frac{a}{POE}\right) \pm 1.96\left[\frac{1-a}{a(n_c+m_c)} + \frac{1}{m_c}\ln^2\left(\frac{a}{POE}\right) + \frac{1-a}{a(n_c+m_c)m_c}\right]\right\}^{\frac{1}{2}} \qquad (4.13)$$

where: CI = 95% confidence intervals

EI = unit raft stiffness;

a, b = curve fitting parameters;

POE = probability of exceedence;

m_c = total number of zero cracked houses; and

n_c = total number of non-zero cracked houses.

By using Equation (4.13), 95% confidence intervals were calculated for each of the site classifications with the probabilities of exceedence given in Figures 4.26 to 4.31. By superimposing the 95% confidence intervals ($\pm 1.96\sigma$) on the design curves, it is possible to assess the deviation from the estimated mean. Figures 4.38 to 4.49 show each of the probabilistic design curves given in Figures 4.26 to 4.31, with the superimposed 95% confidence intervals.

It is evident from Figures 4.38 to 4.49 that the 95% envelopes are somewhat larger than one would desire. However, the size of the envelope can only be reduced by increasing the number of measured data.

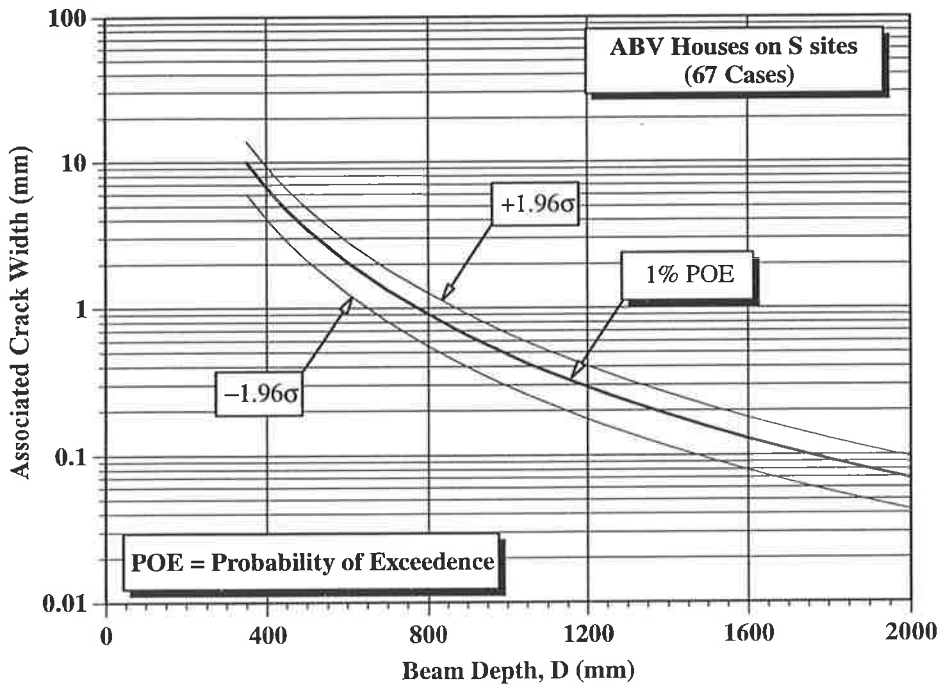


Figure 4.38 1% POE for ABV houses on S site and the 95% confidence limits ($\pm 1.96\sigma$).

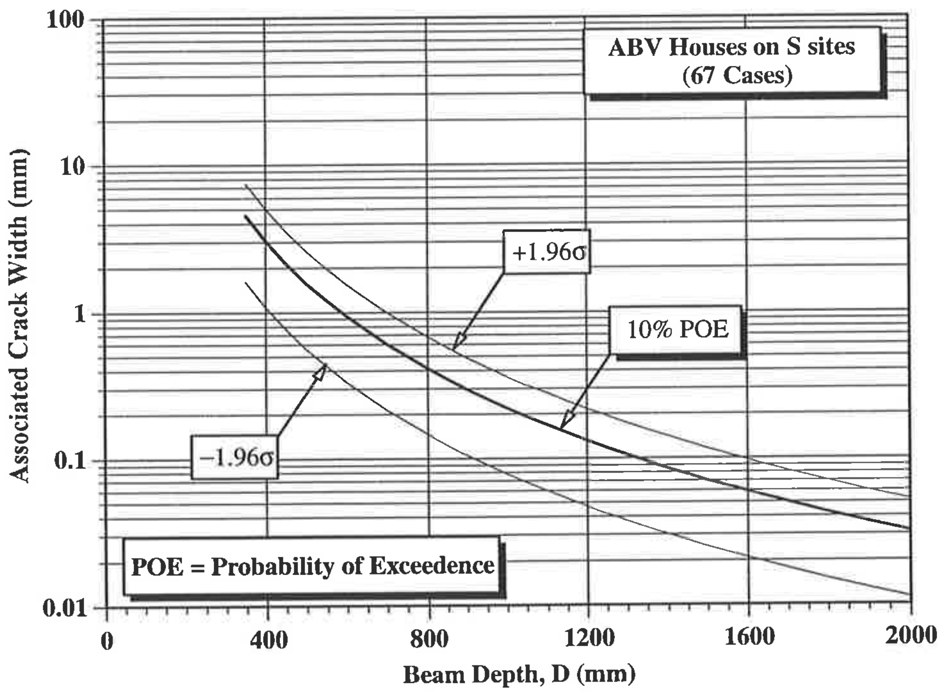


Figure 4.39 10% POE for ABV houses on S site and the 95% confidence limits ($\pm 1.96\sigma$).

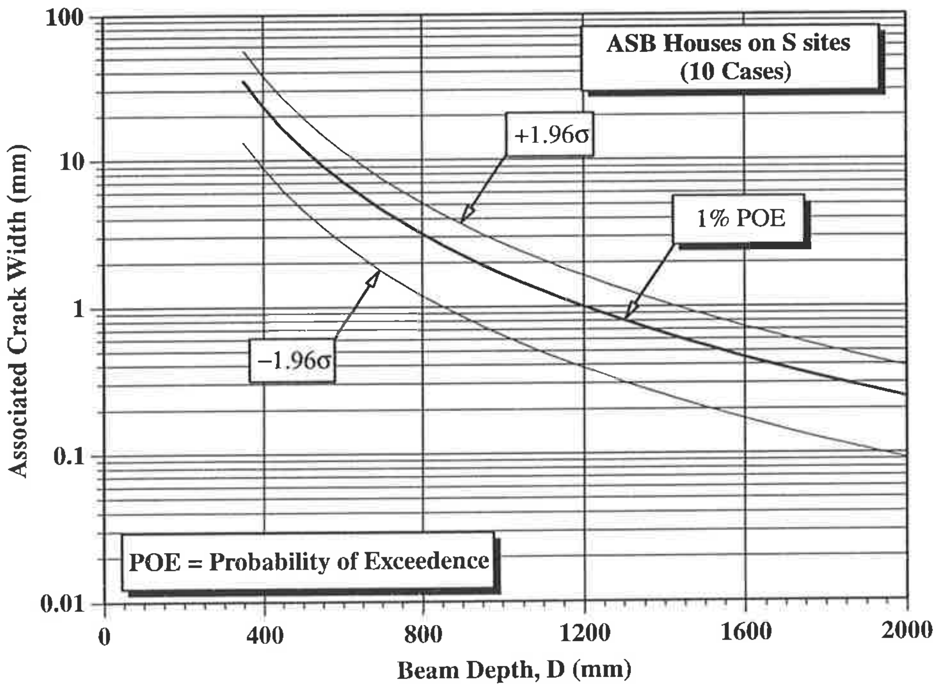


Figure 4.40 1% POE for ASB houses on S site and the 95% confidence limits ($\pm 1.96\sigma$).

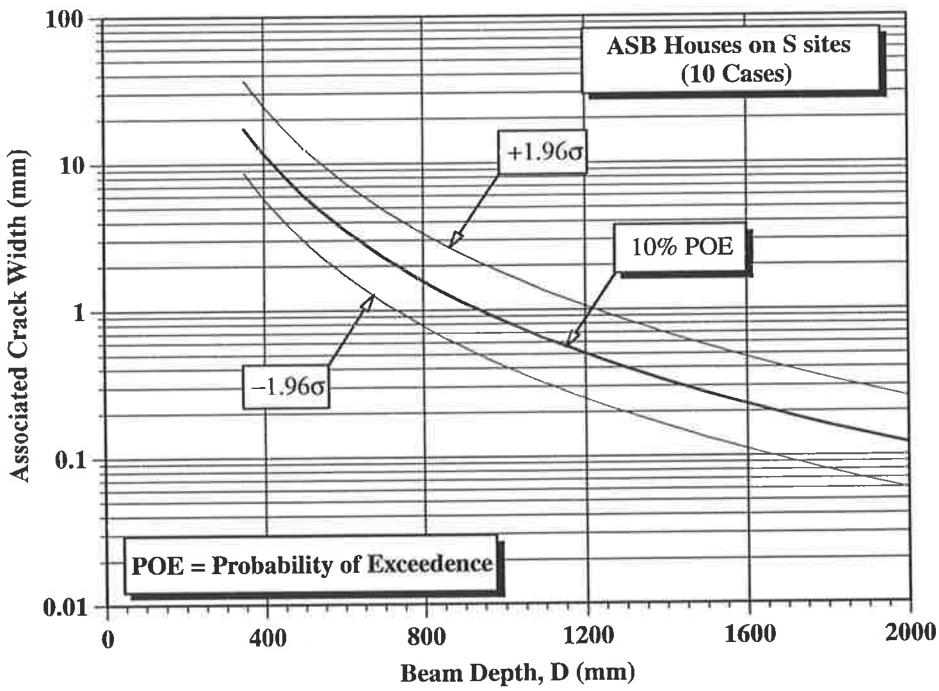


Figure 4.41 10% POE for ASB houses on S site and the 95% confidence limits ($\pm 1.96\sigma$).

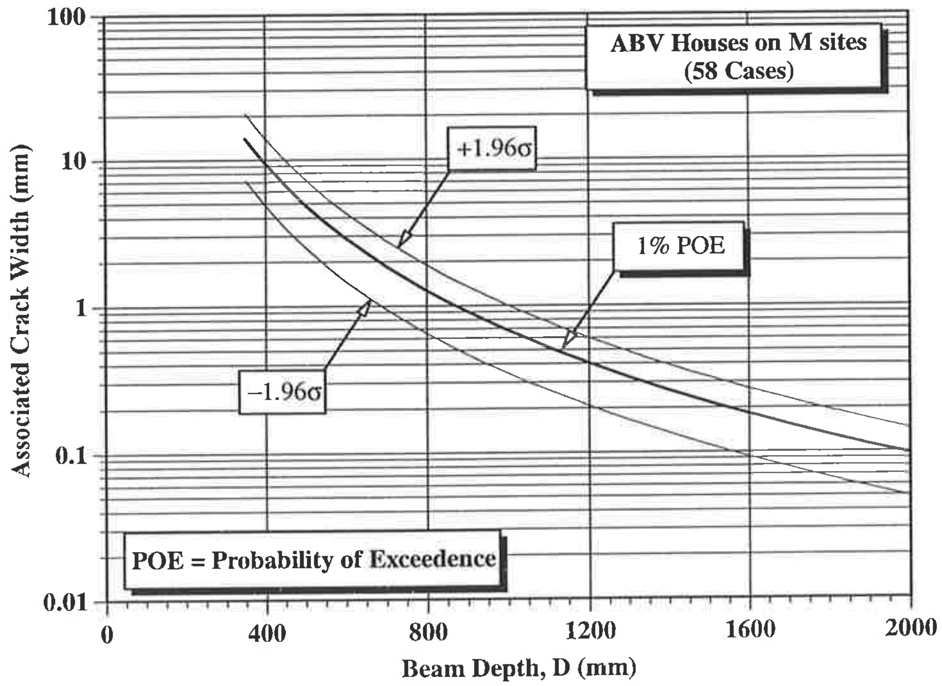


Figure 4.42 1% POE for ABV houses on M site and the 95% confidence limits ($\pm 1.96\sigma$).

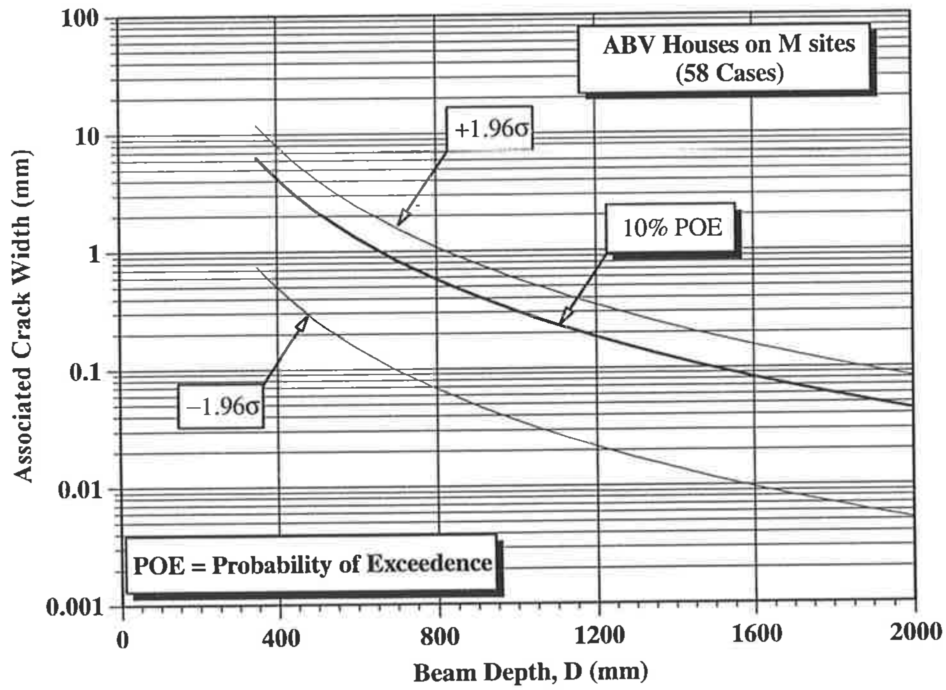


Figure 4.43 10% POE for ABV houses on M site and the 95% confidence limits ($\pm 1.96\sigma$).

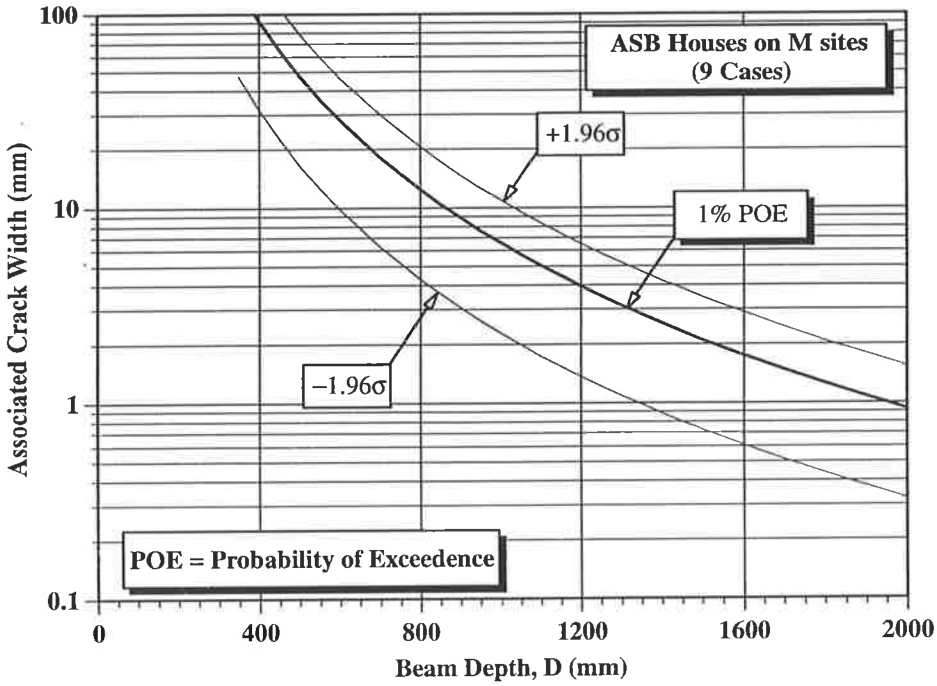


Figure 4.44 1% POE for ASB houses on M site and the 95% confidence limits ($\pm 1.96\sigma$).

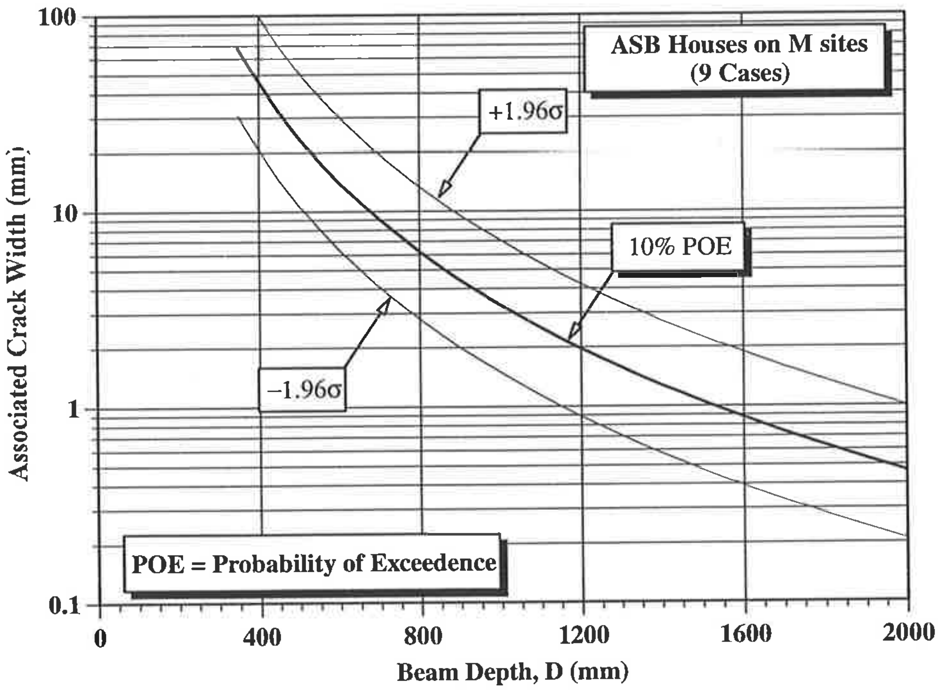


Figure 4.45 10% POE for ASB houses on M site and the 95% confidence limits ($\pm 1.96\sigma$).

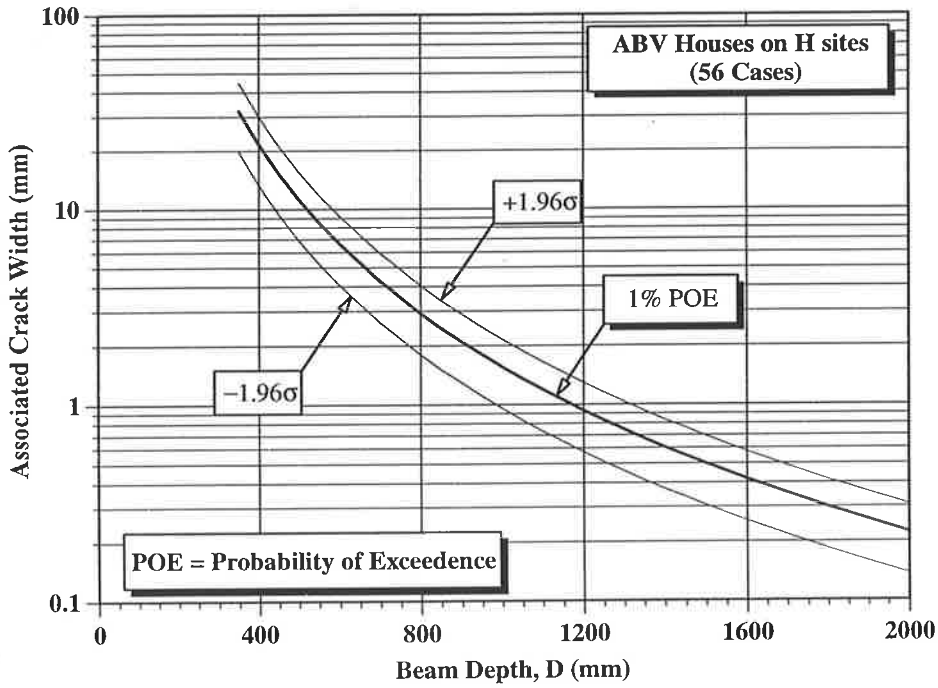


Figure 4.46 **1% POE for ABV houses on H site and the 95% confidence limits (±1.96σ).**

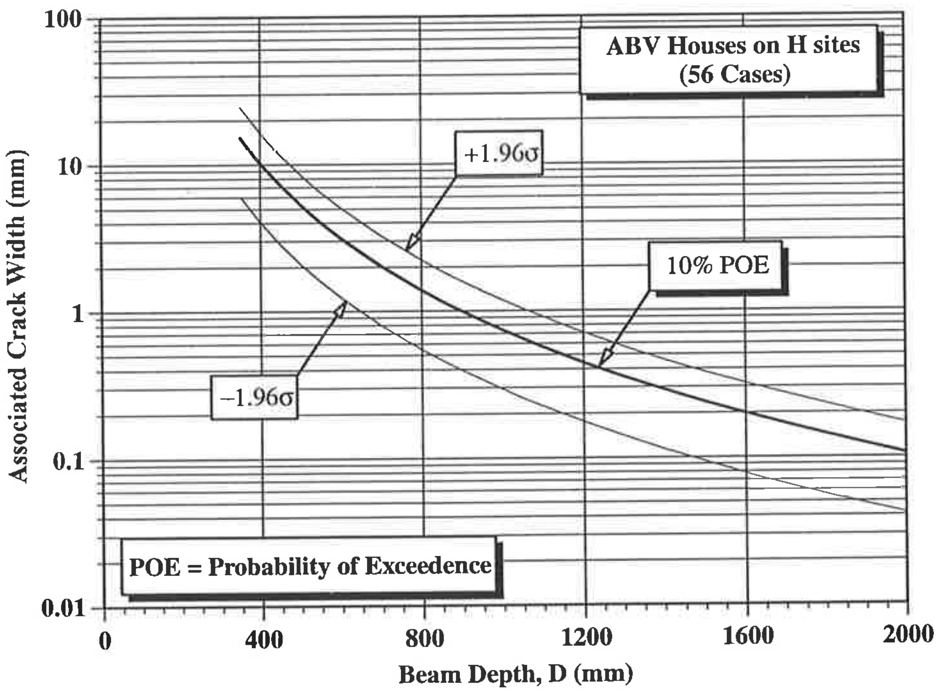


Figure 4.47 **10% POE for ABV houses on H site and the 95% confidence limits (±1.96σ).**

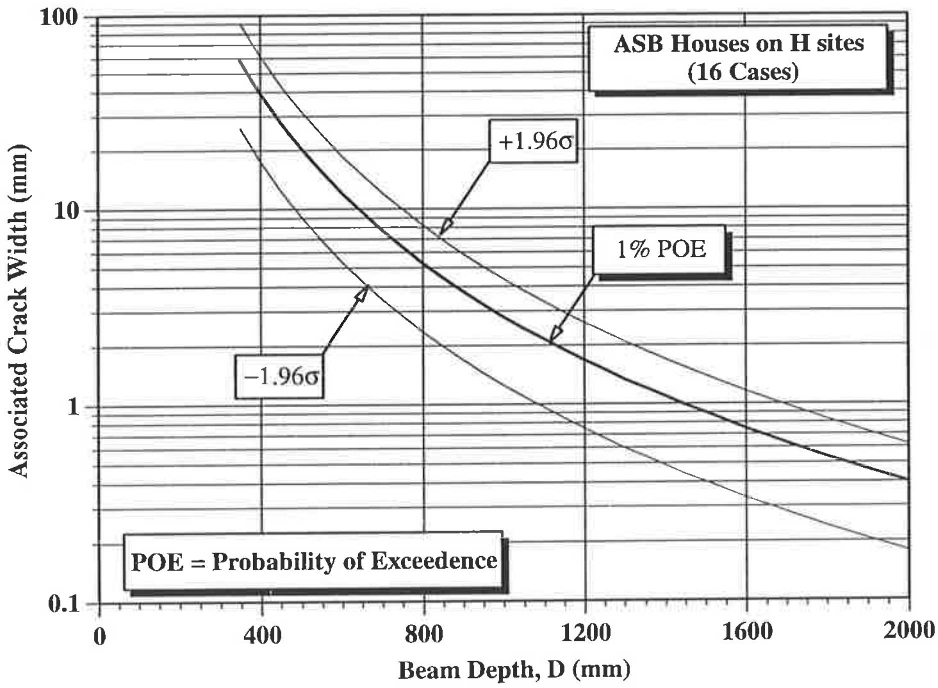


Figure 4.48 1% POE for ASB houses on H site and the 95% confidence limits ($\pm 1.96\sigma$).

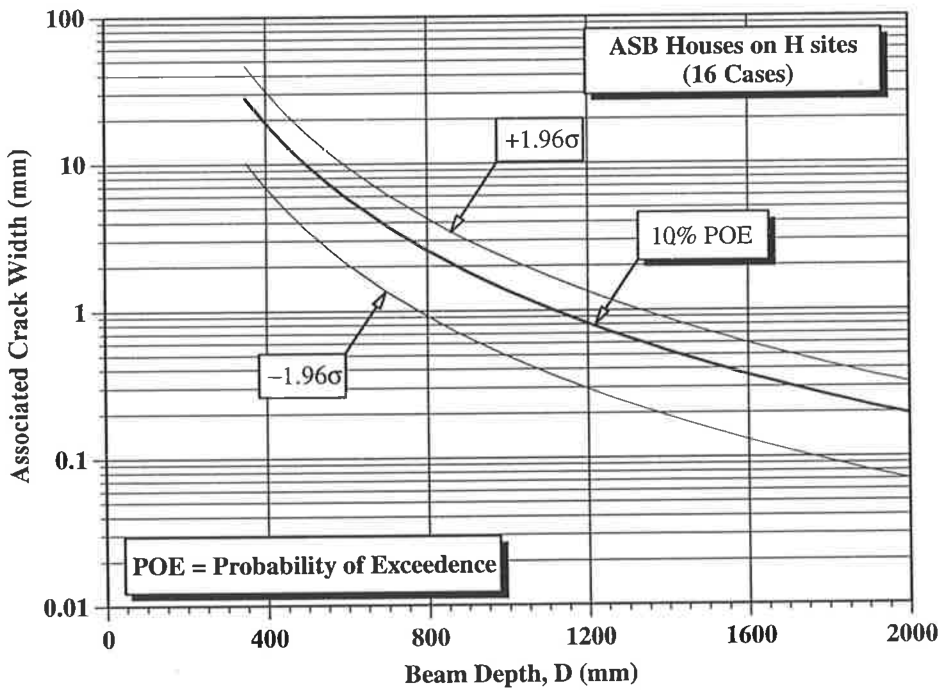


Figure 4.49 10% POE for ASB houses on H site and the 95% confidence limits ($\pm 1.96\sigma$).

4.6.2 Uncertainties Involved in the Model

Even though the probabilistic method has a number of significant advantages over traditional deterministic methods, the model includes inherent uncertainties. Some of these are listed below.

- The visual-tactile method, the commonly used technique to estimate I_{pt} due its dependency on the classifier results in extreme variabilities which in turn affects the prediction of free surface heave, y_s, and as a consequence, inaccuracies occur when the estimate of y_s fall close to the boundary between two site classifications;
- The borelogs may not necessarily represents the maximum value of y_s, because the positions of the borelogs are selected arbitrarily across the site;
- Most of dwellings contained in the database are biased towards the most damaged end of the distribution;
- Cracks measured over a longer period may be more severe and as a consequence, long-term factor included in the model may predict inaccurate results;
- It is difficult to assess the cracks which might have been repaired by previous owners;
- Assumed concrete properties and dimensions may be different; and
- The raft stiffness calculations do not include reinforcement details.

The following section compares the results of this study with those obtained by Kay and Mitchell (1990).

4.7 COMPARISON OF DESIGN METHODS

A comparative study has been carried out between the results given by Kay and Mitchell (1990) with those presented earlier in this chapter; both the Kay and Mitchell method and the maximum likelihood method. The results of this comparative study are given Figures 4.50 to 4.54. Superimposed on these curves are the footing depths specified by AS2870.1-1988 (Standard Association of Australia 1988a). As stated by AS2870.1-1988 (Standard Association of Australia 1988a), the footing sizes recommended by the standard have a 5% probability of significant damage (> 5 mm crack width) occurring within the design life of

50 years. Hence, it is appropriate to compare these dimensions with those given by the 5% POE curves.

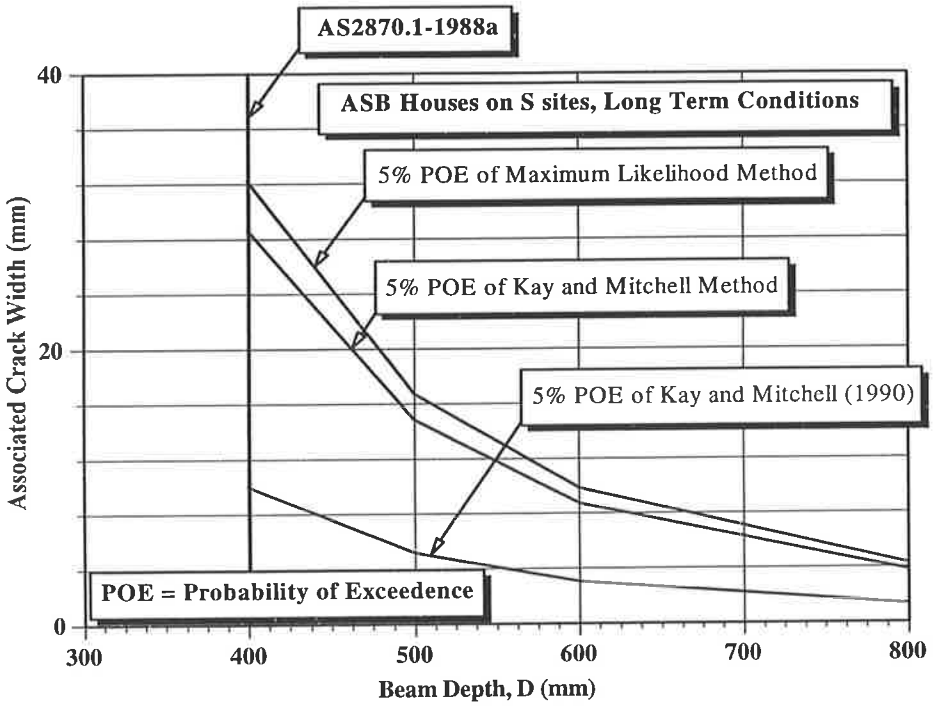


Figure 4.50 Results of comparative study for ASB houses on S sites.

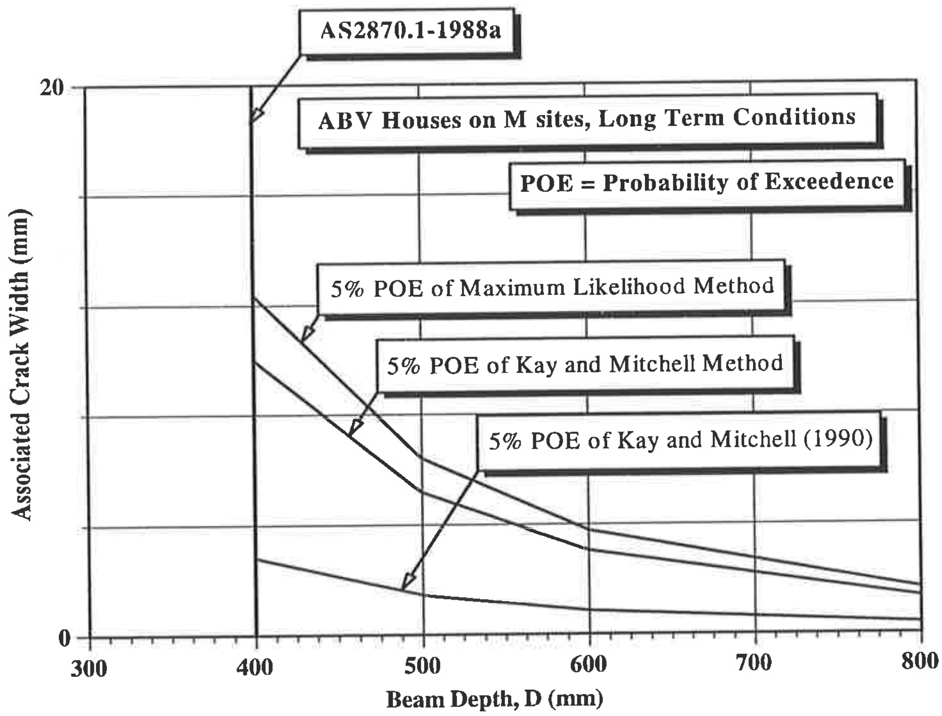


Figure 4.51 Results of comparative study for ABV houses on M sites.

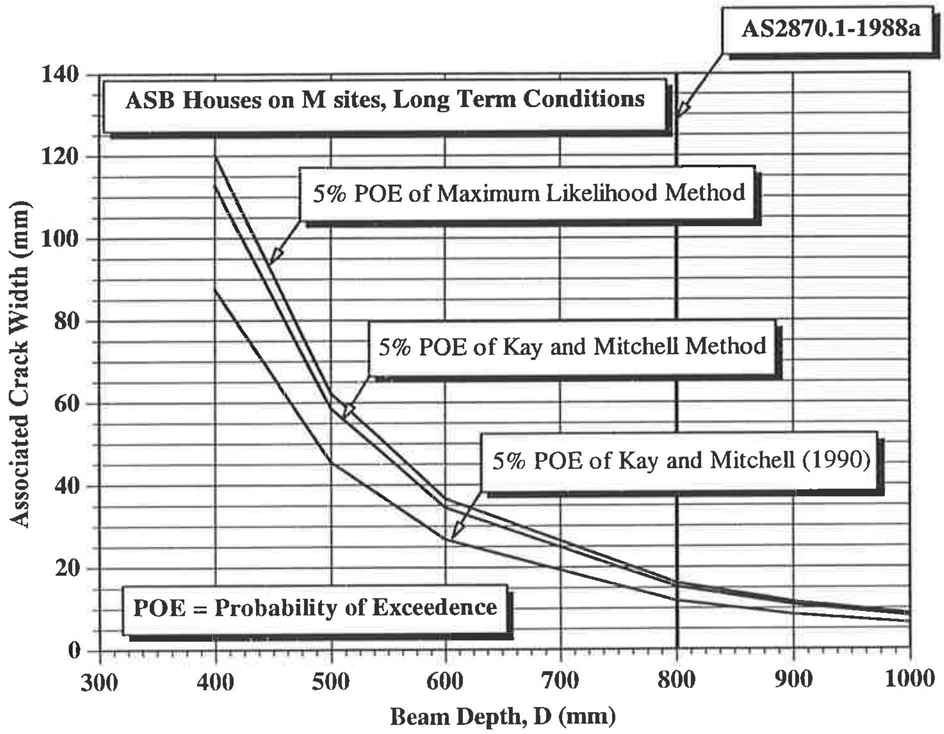


Figure 4.52 Results of comparative study for ASB houses on M sites.

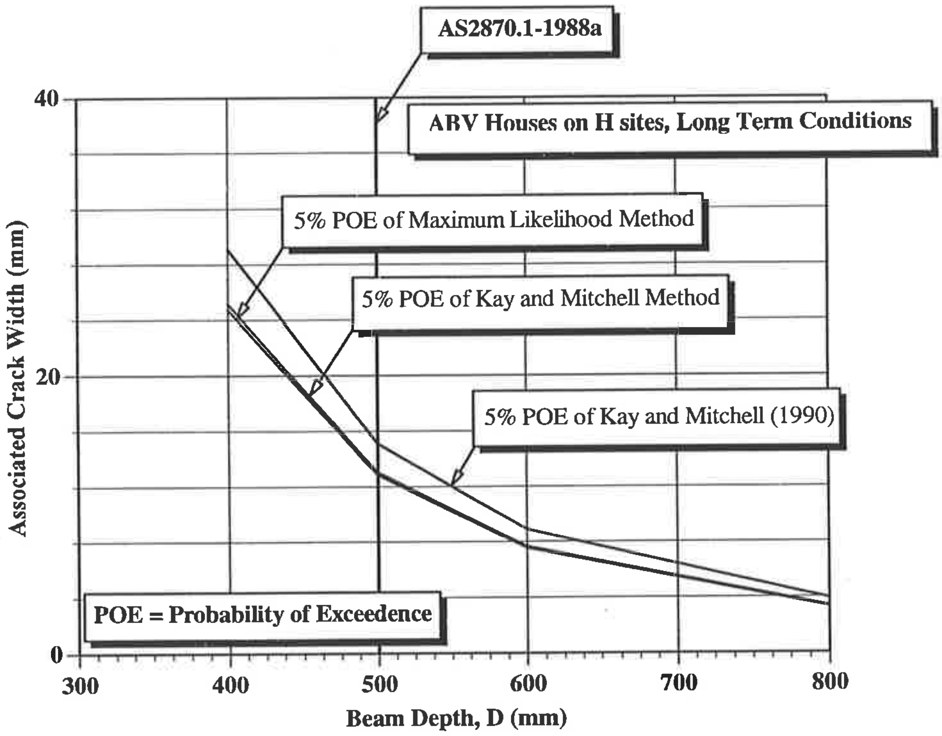


Figure 4.53 Results of comparative study for ABV houses on H sites.

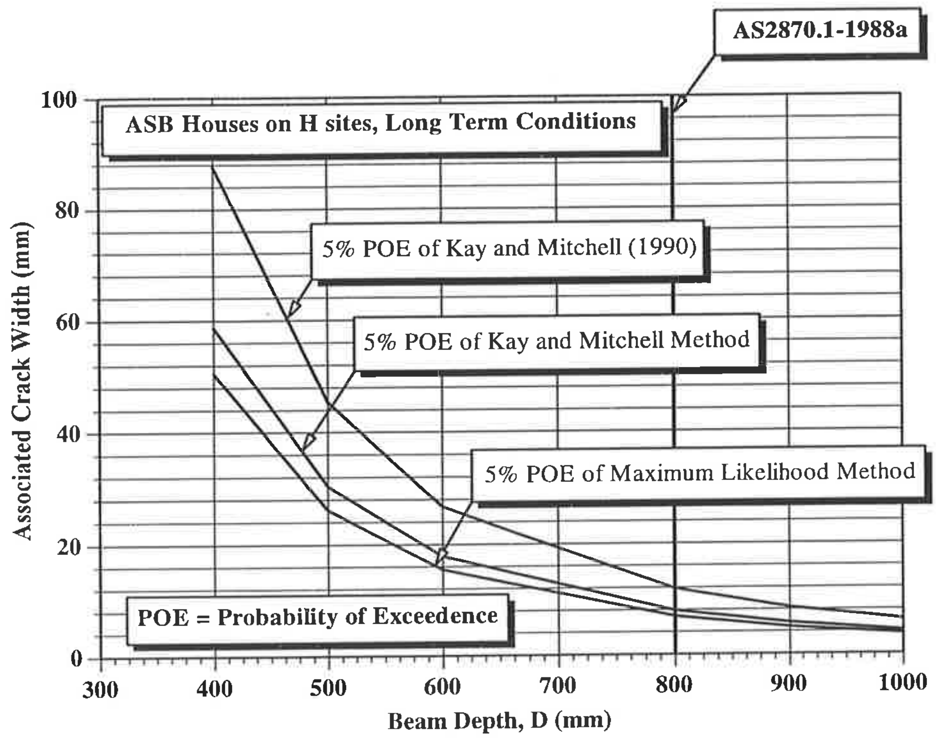


Figure 4.54 Results of comparative study for ASB houses on H sites.

Figure 4.50 shows the results of the comparative study for articulated solid brick houses built on S site. Due to insufficient data available for Kay and Mitchell (1990) method in articulated brick veneer houses built on S site comparative study was not carried out. Whereas Figures 4.51 and 4.52 show the results of the comparative study for articulated brick veneer and articulated solid brick houses respectively on M sites. Figures Figure 4.53 to Figure 4.54 show results of the comparative study for articulated brick veneer and articulated solid brick, respectively, on H sites.

It is evident from these figures that, in the majority of cases, the results presented in this study indicate a more severe level of cracking than that resulting from Kay and Mitchell (1990) study. This is not unexpected since Kay and Mitchell's work was confined to the exterior of the dwellings and, as a consequence, is likely to have resulted in lower levels of cracking.

In addition, it is evident from Figures 4.50 to 4.54 that the difference between the results presented in this study and those from Kay and Mitchell's work decreases as the footing depth increases.

4.8 SUMMARY

Data obtained and described in Chapter 3 was analysed in this chapter in order to develop probabilistic design charts. Due to the large number of variables involved, normalisation of the data was carried out in accordance with the method suggested by Kay and Mitchell (1990). Equivalent crack widths were then calculated and cumulative frequency distributions generated. These were then used to develop probabilistic design charts which express, for various probabilities of exceedence, crack widths associated with standard raft footing depths.

However, because the method suggested by Kay and Mitchell (1990) yielded unrealistic cumulative frequency distributions, a new method, based on maximum likelihood, was developed. As indicated by a comparative study, the results presented in this study suggest a higher level of cracking than that proposed by Kay and Mitchell's work.

The following chapter presents a series of case studies which compares the footing sizes and costs associated with the probability method with those from designs based on traditional deterministic methods.

Chapter Five

Application of Probabilistic Design Methodology

5.1 INTRODUCTION

As described in the previous chapter, a probabilistic method has been developed for the design of residential footings built on expansive soil. This chapter presents the application of this methodology to a series design case studies.

The chapter begins with the design of residential footing built on various site classifications and super-structure using AS2870.1-1988 (Standard Association of Australia, 1988a) and the Mitchell Method (*SLOG*). Subsequently, the residential footings, so designed were assessed with the probabilistic method developed in this study.

5.2 DESIGN OF ABV AND ASB HOUSES ON S, M AND H SITES

Figures 5.1 to 5.3 show the footing layouts for articulated brick veneer and articulated solid brick houses on S, M and H sites. The value of free surface heave, y_s, selected are approximately at the mid-range of the site classification in order to have equivalent effects on the design. In order to have a variety in the design, three different footing layouts were selected. These houses were designed using AS2870.1-1988 and the Mitchell Method (Mitchell, 1979) as implemented by the program *SLOG* (Rust PPK, 1990). Later, these designs were compared with the charts developed in the previous chapter. The following sections discuss the procedure involved in the design of these footings.

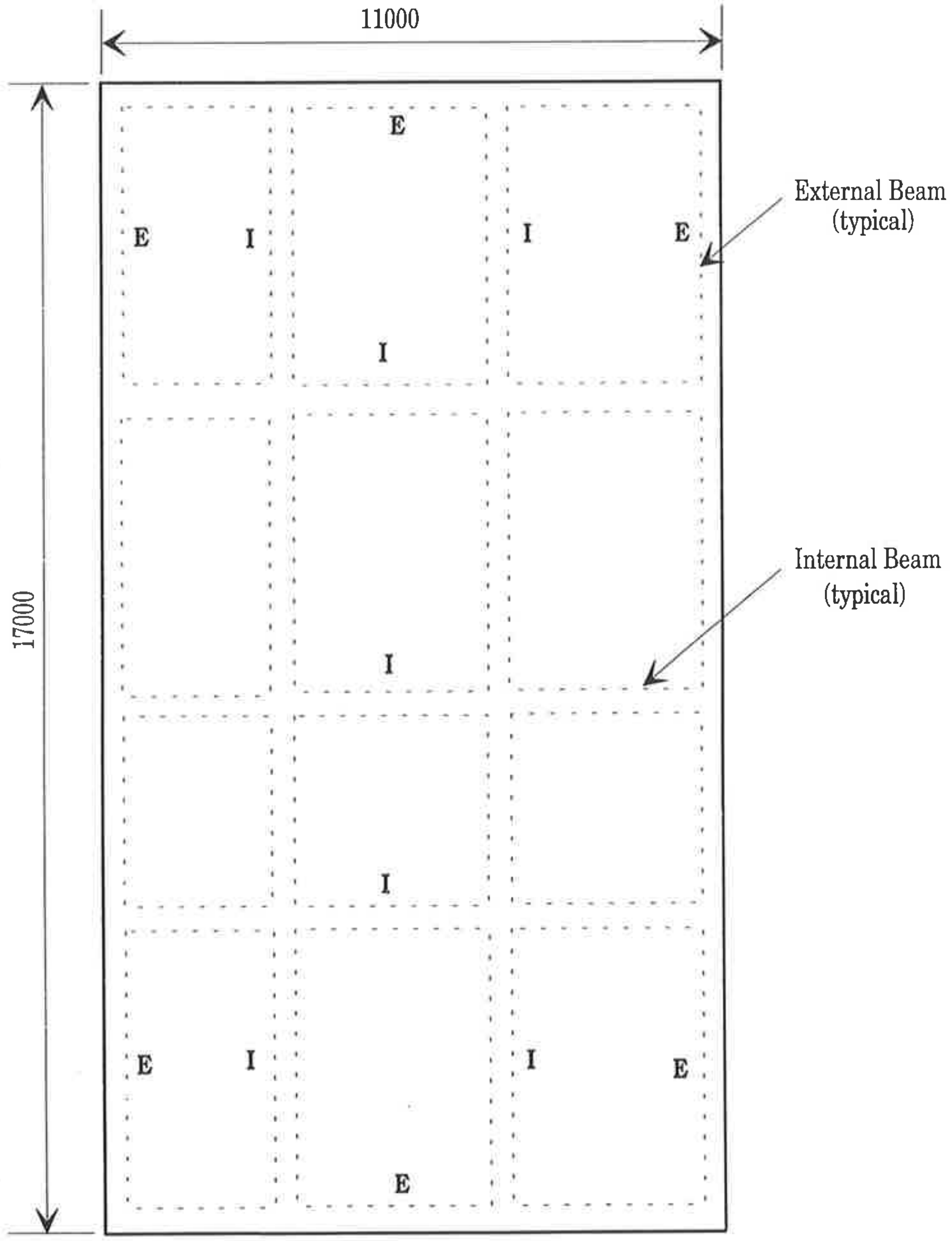


Figure 5.1 Footing layout plan for ABV and ASB houses on S sites.

(All dimensions in mm)

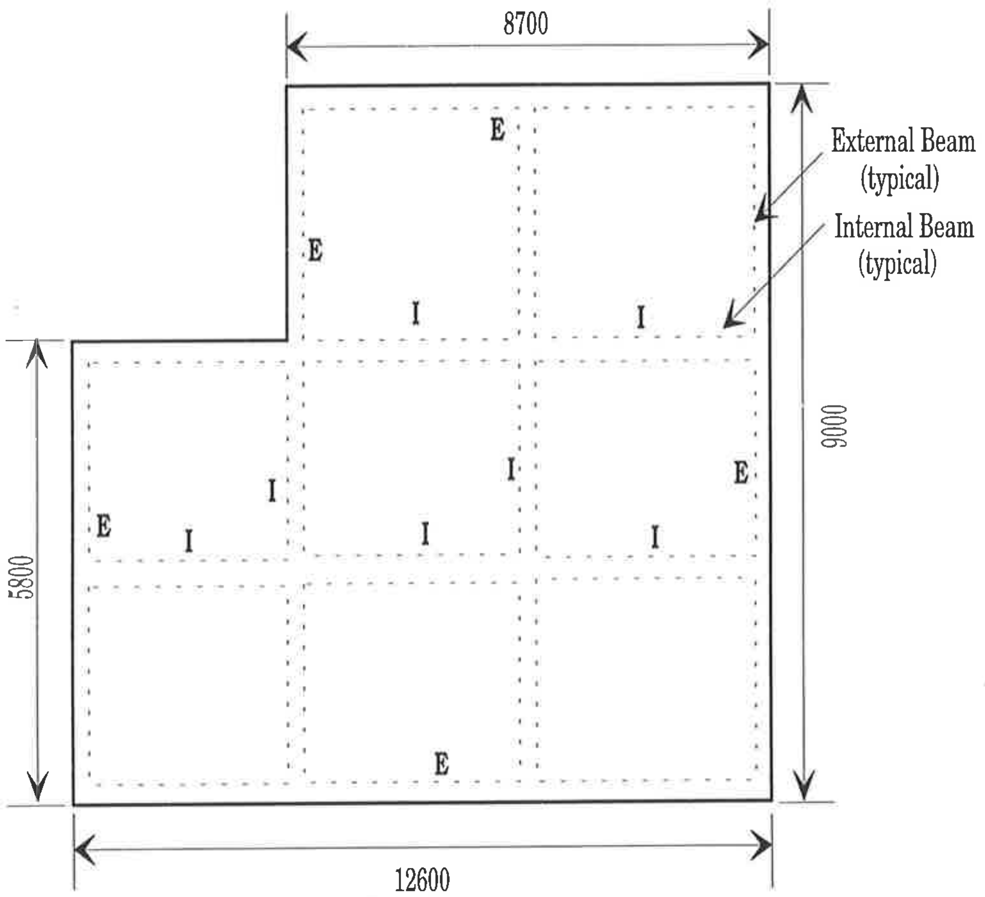


Figure 5.2 Footing layout plan for ABV and ASB houses on M sites.

(All dimensions in mm)

5.2.1 Design of Footings Using AS2870.1-1988

Using the concepts specified in Figures 3.1 and 5.1 of the AS2870.1-1988 (Standard Association of Australia, 1988a), the rafts shown in Figures 5.1 to 5.3 were designed for three classes of site reactivity and two classes of house flexibility for each group, a total of six groups. The design details of the raft footings for articulated brick veneer and articulated solid brick built on S sites is shown in Figure 5.4, whereas the design details for M and H sites are shown in Figure 5.5.

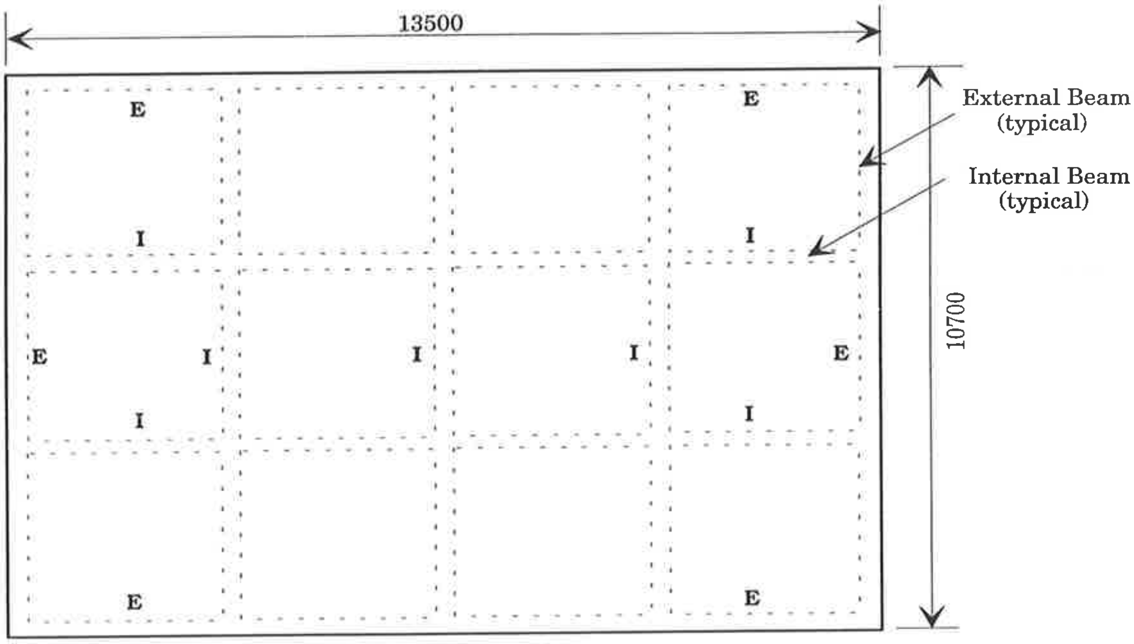
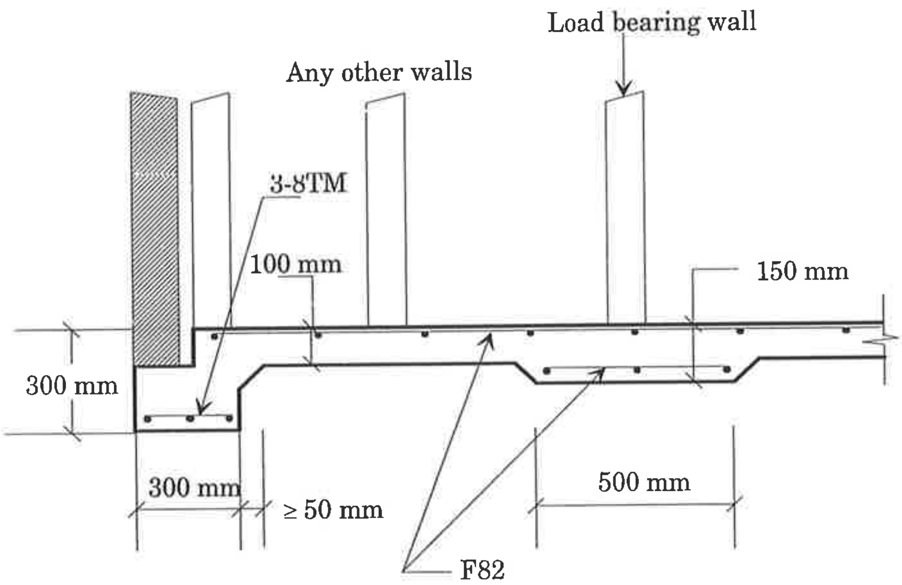


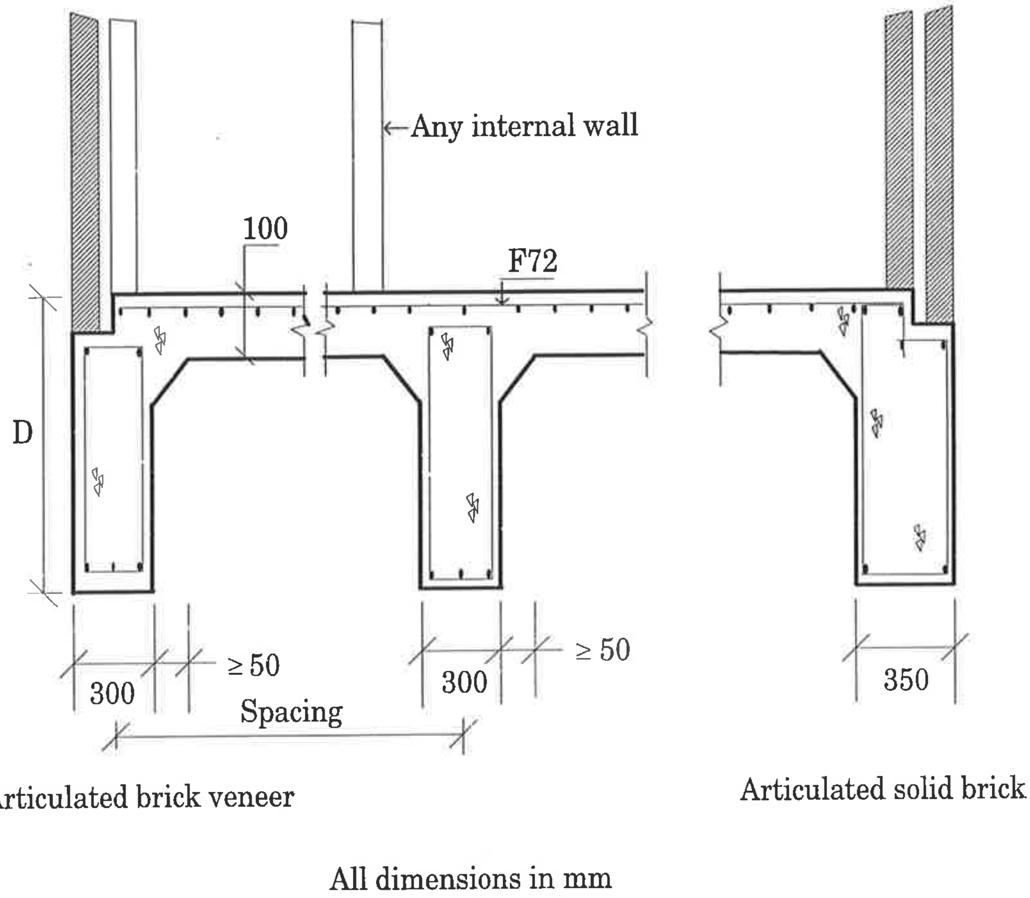
Figure 5.3 Footing layout plan for ABV and ASB houses on H sites.

(All dimensions in mm)



Surface Heave (mm)	Site Classification	Super-Structure	Depth D (mm)	Reinforcement		
				Top	Bottom	Slab
19	S	ABV	300	-	3-8TM	F82
19	S	ASB	400	-	3-11TM	F82

Figure 5.4 Raft details of ABV and ASB houses on S site.



Surface Heave (mm)	Site Class'n	Super Structure	Depth *D* (mm)	Reinforcement				Spacing *S* (mm)
				External		Internal		
				Top	Bottom	Top	Bottom	
25	M	ABV	400	2-Y12	3-Y12	-	-	4100
25	M	ASB	625	3-Y16	3-Y16	2-Y16	2-Y16	4100
53	H	ABV	500	3-Y16	3-Y16	-	-	3375
53	H	ASB	800	4-Y16	4-Y16	3-Y16	3-Y16	3375

Figure 5.5 Raft details of ABV and ASB houses on M and H sites.

5.2.2 Design of Footings Using the Mitchell Method

As discussed in Chapter 2, the Mitchell Method (Mitchell, 1979) is a numerical deterministic method for the design of footings built on expansive soils. By its very nature, the Mitchell Method is numerically intensive and commercially available program, *SLOG* (Rust PPK, 1990), has been developed to implement the method.

As input, *SLOG* requires details regarding the geometry, deflection ratios and loads associated with the design rectangle, as well as the design heave, y_m, and the subgrade modulus, k, of the underlying soil. A standard value of 1000 kPa/m was adopted for the subgrade modulus parameter, k within the program. The values of depth of suction change, a, shape factor, m, and deflection ratios, Δ/L, were determined as discussed previously in §2.5.6. Subsequently, all these parameters are incorporated in the program to design the footings as shown in Figures 5.1 to 5.3. The results of one such analysis is included in Appendix G. The design details as suggested by this program for the construction of rafts are shown in Table 5.1.

Table 5.1 Raft details of ABV and ASB houses on S, M and H sites.

(Mitchell Method)

Surface Heave (mm)	Site Class'n	Super-Structure	Depth D (mm)	Reinforcement				Spacing S (mm)
				External		Internal		
				Top	Bottom	Top	Bottom	
19	S	ABV	300	2-Y12	3-Y12	2-Y12	2-Y12	3700
19	S	ASB	300	2-Y12	3-Y12	2-Y12	3-Y12	3700
25	M	ABV	300	2-Y12	3-Y12	2-Y12	2-Y12	4100
25	M	ASB	625	3-Y12	3-Y12	2-Y12	2-Y12	4100
53	H	ABV	500	3-Y12	3-Y12	2-Y12	3-Y12	3375
53	H	ASB	800	4-Y12	4-Y12	3-Y12	3-Y12	3375

5.3 APPLICATION OF THE CURRENT PROBABILISTIC METHOD

Superimposing the design details developed previously in §5.2, on the appropriate charts of the current probabilistic method developed in the previous chapter are shown in Figures 5.6 to 5.11. It is evident from these charts that the sizes recommended by AS2870.1-1988 and the Mitchell Method (*SLOG*) are associated with crack widths, of the order of 1 mm and greater at 50% probability of exceedence. It is obvious that to reduce this level of distress one should use deeper footings resulting in an increase in the cost of construction of the raft.

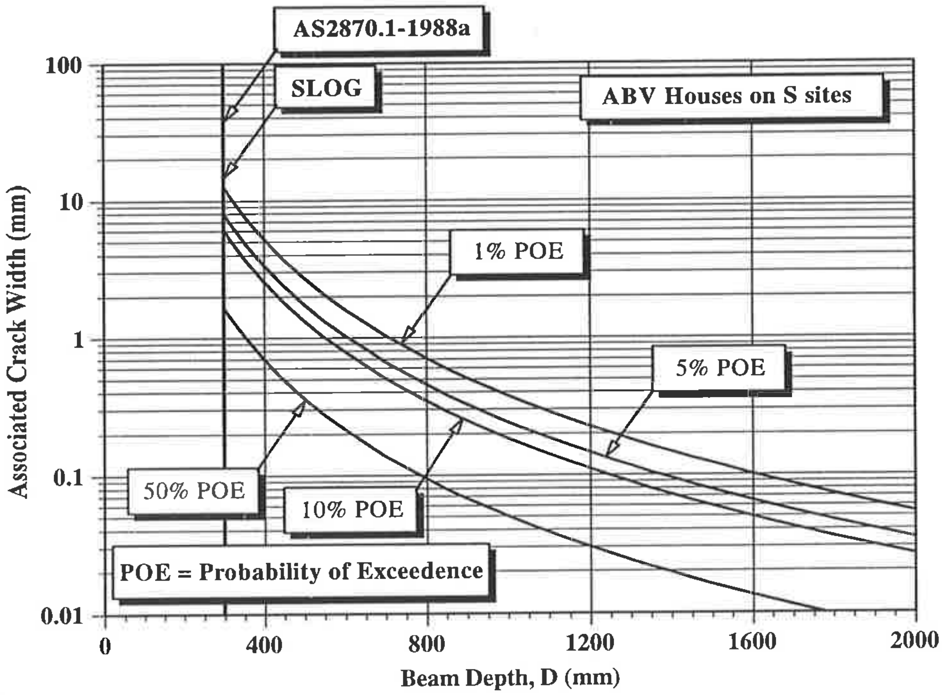


Figure 5.6 Crack widths associated with designs of AS2870 and *SLOG* for ABV houses on S sites.

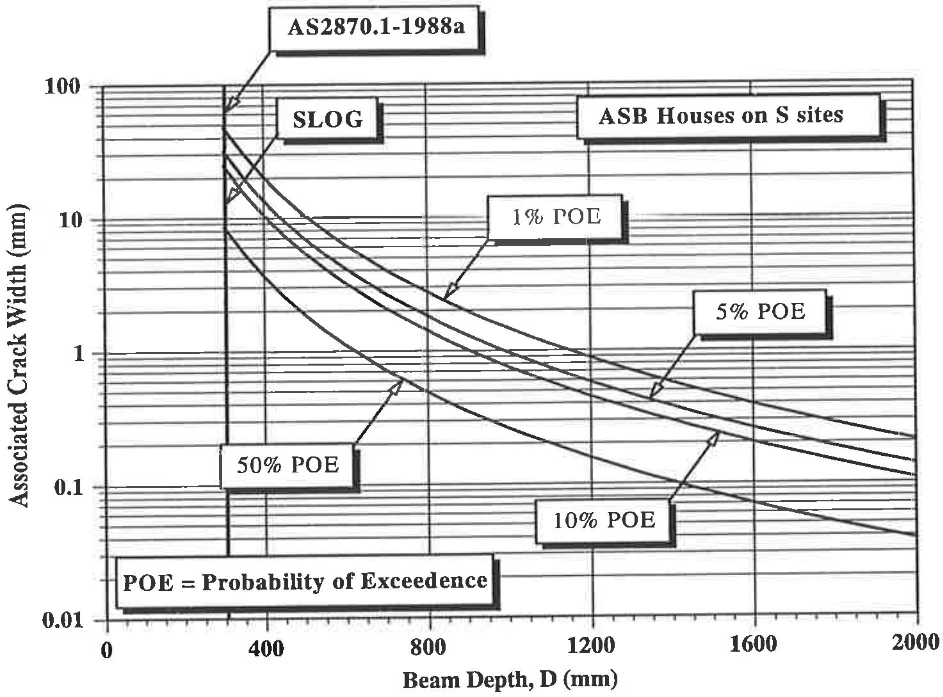


Figure 5.7 Crack widths associated with designs of AS2870 and *SLOG* for ASB houses on S sites.

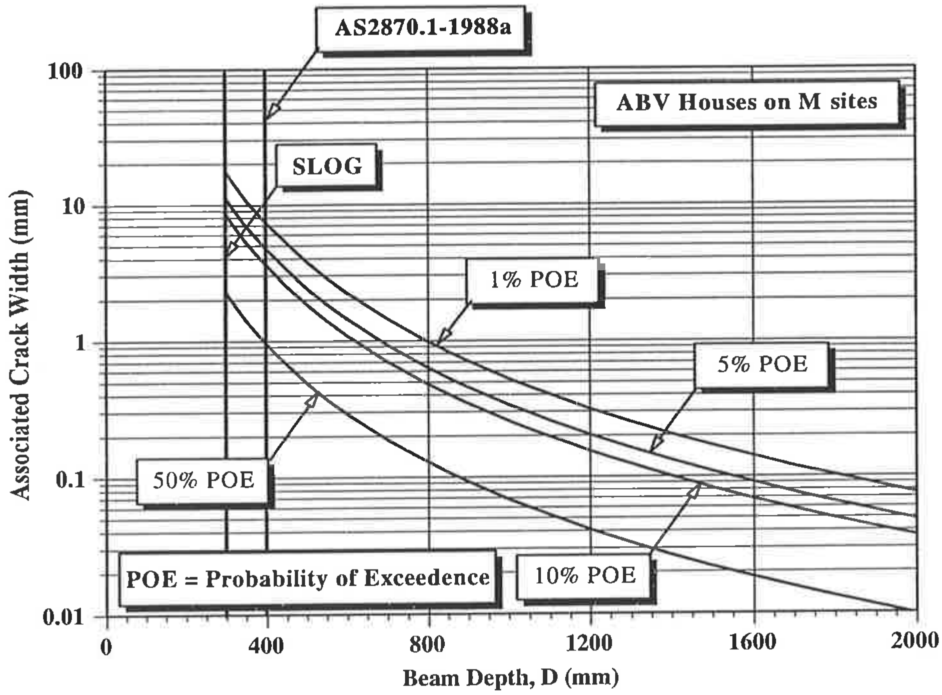


Figure 5.8 Crack widths associated with designs of AS2870 and *SLOG* for ABV houses on M sites.

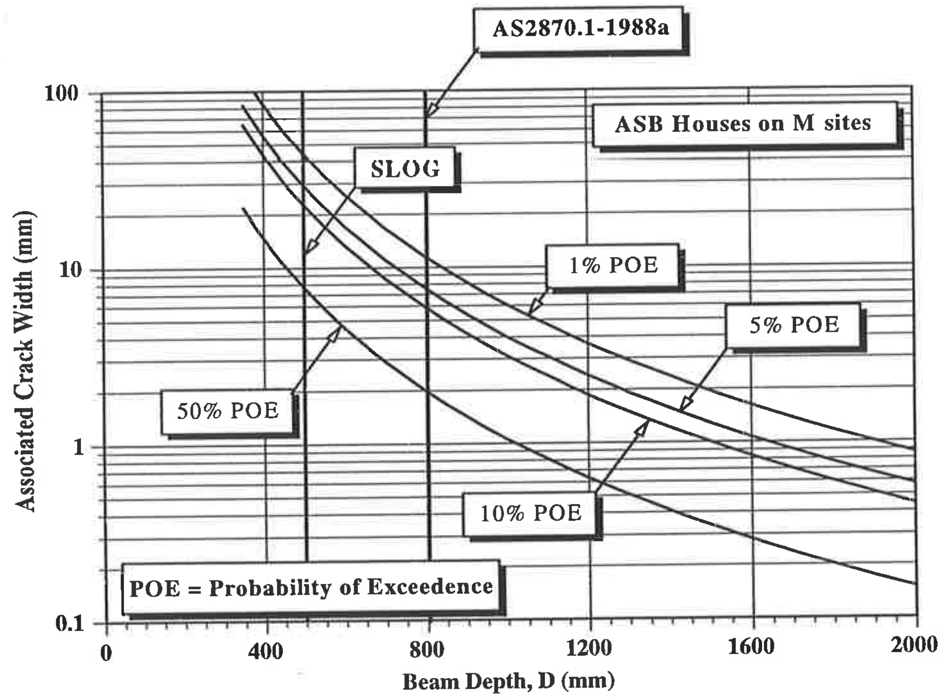


Figure 5.9 Crack widths associated with designs of AS2870 and *SLOG* for ASB houses on M sites.

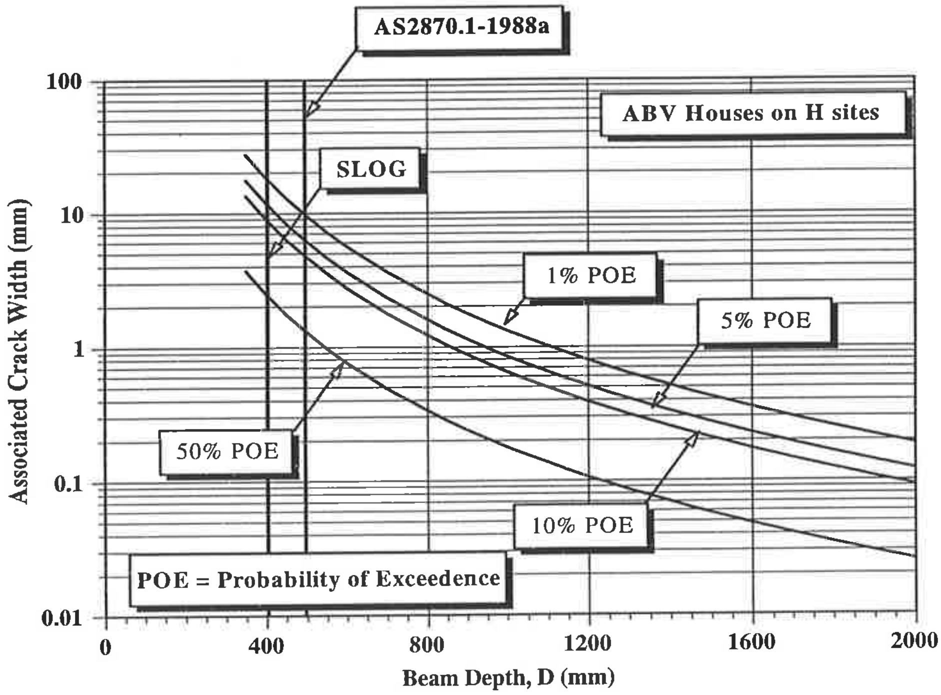


Figure 5.10 Crack widths associated with designs of AS2870 and *SLOG* for ABV houses on H sites.

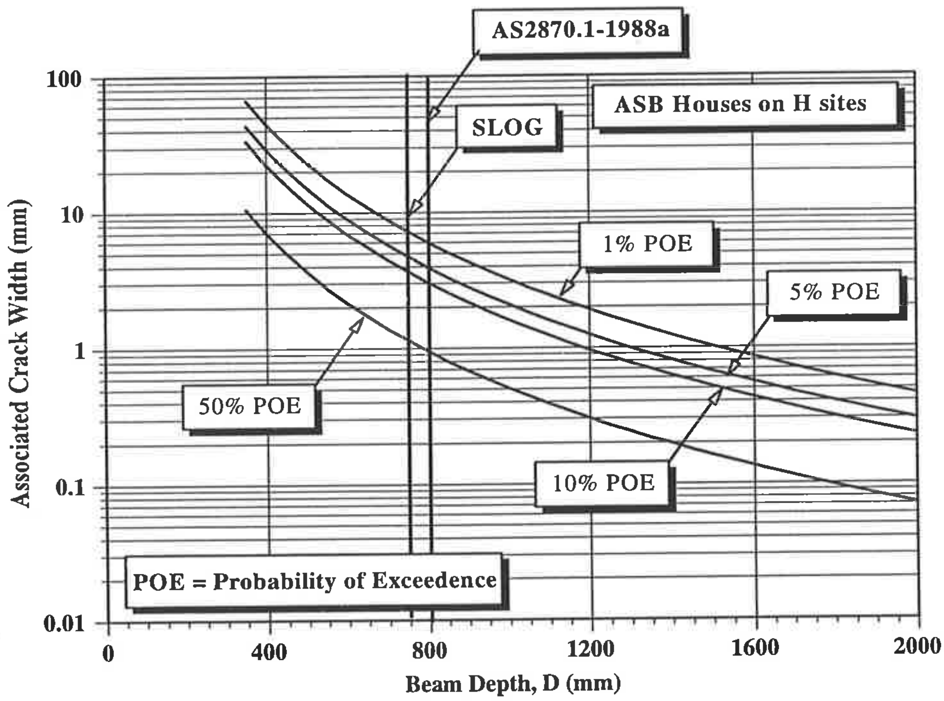


Figure 5.11 Crack widths associated with designs of AS2870 and *SLOG* for ASB houses on H sites.

5.4 COST ESTIMATION OF RAFT CONSTRUCTION

Since one of the main objectives for developing a probabilistic design approach, is to provide the engineer and client with quantitative estimates of the risk of damage in relation to the cost of footing construction, it is appropriate to compare the various footings in terms of cost of construction. In order to achieve this, the Australian Construction Handbook (Rawlinson's, 1995) was used. A value of $295/m³ was used which includes cost of reinforcement and the form work. The various cost of construction for AS2870 and the Mitchell Method are included in Table 5.2, whereas, for different crack widths at various level of probabilities are included in Table 5.3.

Table 5.2 Estimated cost of construction for various rafts designed using AS2870 and Mitchell method.

Super-Structure	Site Classification	Cost of Construction	
		AS2870.1-1988	SLOG
ABV	S	$7530	$8250
ASB	S	$8150	$8250
ABV	M	$5230	$4500
ASB	M	$7300	$6120
ABV	H	$8000	$7070
ASB	H	$11350	$10850

Table 5.3 Cost of construction of footings at different level of probabilities.

Surface Heave (mm)	Super-Structure	1% POE		10% POE		50% POE	
		0.1 mm	1 mm	0.1 mm	1 mm	0.1 mm	1 mm
19	ABV	$23840	$14540	$19640	$11240	$16580	$8840
19	ASB	*	$18440	*	$15440	$21740	$12440
25	ABV	$16070	$9330	$13080	$7270	$9150	$5590
25	ASB	*	$18110	*	$16060	*	$10930
53	ABV	*	$13620	*	$11270	*	$8470
53	ASB	*	$18440	*	$14900	*	$12100

* Costs associated with proposed footing sizes are prohibitive.

It is evident from Table 5.3 that an extra few hundred dollars spent on the footing construction may reduce the level of risk associated with the design. With reference to Tables 5.2 and 5.3, for example, the cost of construction of a raft footing for ABV house on an M site designed using AS2870.1-1988 is \$5230 whereas using Mitchell's Method, it is \$4500. Alternatively, for the same raft footing, with a 1% POE for an associated crack width of 0.1 mm the cost is \$16070, and for the same level of cracking with 10% POE one could reduce the cost of construction to \$13080. If the client is prepared to accept more risk by adopting a 1 mm associated crack width with 50% POE, the cost of construction can be reduced further, to \$5590. It thus enables the client and engineer to make informed decisions with regard to the level of risk and the economic cost.

5.5 SUMMARY

In this chapter, six raft footings, which include articulated brick veneer and articulated solid brick houses on S, M and H sites, were designed using AS2870 and the Mitchell Method as implemented by the program *SLOG*. Subsequently, these designs were superimposed on the probabilistic design charts developed in Chapter 4 and the level of distress was assessed at various probabilities of exceedence.

It has been observed that the estimated crack widths associated with the footing sizes recommended by AS2870 and Mitchell Method are likely to exceed 1 mm with 50% probability of exceedence. It was also shown that a more effective footing design can be obtained by slightly increasing the cost associated with its construction.

Chapter Six

Summary and Conclusions

6.1 SUMMARY

This study has examined the effects of expansive soils on residential buildings within several Adelaide metropolitan councils. A method for the design of residential footings built on S, M and H soils has been presented based on a probabilistic approach.

In Chapter 2 it was shown that expansive soils undergo large shrinkage and swelling movements as a result of changes in subsoil moisture. It was also shown that a considerable amount of uncertainty exists in the estimation of free surface heave. Furthermore, previous methods for the design of raft footing systems built on expansive soils have been based on idealised mathematical models. In addition, it was observed that the probabilistic approach, suggested by Kay and Mitchell (1990), provides the design engineer and the client with some guidance to the level of risk associated with any footing size. Finally, it was shown that a number of limitations existed in their study including: (i) the survey was restricted only to the exterior of the dwellings, and (ii) the study was confined to one local council area.

Chapter 3 described the compilation of the database which was carried out in two stages. In the first stage, the raw information was collected from six different local council areas in the Adelaide metropolitan area. The relevant information recorded for each footing included:

- Soil borelog;
- Footing details and layout;
- Type of super-structure;
- Address of the site; and
- Date of construction of the footing.

The second stage of the research involved a visual inspection of the houses whose occupiers had agreed to participate in the study. The following information was recorded in the visual inspection:

- Measurement of crack widths within internal and external walls, floors and ceilings;
- Recording of various site conditions; and
- The residents/owners were asked if they were aware of any cracking or any other previous damage that had needed repair.

Chapter 3 also detailed the procedure involved in the calculation of critical raft stiffness. It was observed that considerable scatter exists in the relationship between the measured crack width and the critical raft stiffness.

Chapter 4 described the analysis of the data and the development of probability design charts. It was found that, even though a large number of houses were examined in the study, the large number of variables involved prevented a direct statistical analysis. In order to apply direct statistical methods of analysis, it was necessary to adopt some sort of normalisation of the results in terms of a single specified raft stiffness. Subsequently, the process of normalising the raft stiffness was carried out using the method suggested by Kay and Mitchell (1990), from which equivalent crack widths were determined. Furthermore, it was shown that all of the 216 cases were separated into three classes of site reactivity (S, M, H) and into two classes of house flexibility (ABV, ASB). For each group, the normalised crack widths were then divided into ranges according to a square root scale bounded by values: 0-0.25, 0.25-1.00, 1.00-2.25, 2.25-4.00, 4.00-6.25, 6.25-9.00 mm, in order to obtain a cumulative frequency distribution. By means of regression analysis, the following relationship between the equivalent crack width and the cumulative frequency was obtained for all the six groups:

$$CF = 100\left(1 - ae^{-bECW}\right)$$

where: CF = cumulative frequency;

a, b = curve fitting parameters;

ECW = equivalent crack width.

This relationship correlated well with the measured data, due to the fact that the coefficient of correlation for all of the groups was very close to one. Probabilistic design charts were developed, based on these cumulative frequency distributions and Kay and Mitchell's approach, with 1%, 5%, 10% and 50% probability of exceedence contours. The design charts expressed crack widths associated with various standard raft footing beam depths.

Because of limitations associated with this approach, namely negative cumulative frequencies, an alternative method was proposed based on the method of maximum likelihood. Probabilistic design curves developed using this approach indicated a higher level of cracking than that proposed by Kay and Mitchell (1990). In addition, it was shown that the footing sizes recommended by AS2870.1-1988 result in significant damage and should be increased. However, further research is needed to verify and quantify this.

Chapter 5 presented a series of case studies which compared the footing sizes and cost associated with the probability method with those from designs based on traditional deterministic methods. It was shown that a more effective footing design can be obtained by slightly increasing the cost associated with its construction.

6.2 RECOMMENDATIONS FOR FUTURE RESEARCH

To assess the level of deformation associated with each building, visual inspections were carried out. However, in this study, only crack widths and the density of cracks were noted for the purpose of the analysis. A level survey of the dwellings will yield more useful information than the assessment of crack widths, alone. However, an investigation of this type would be extremely time consuming and labour intensive. Nevertheless, level surveys would be extremely valuable.

As indicated in Chapter 3, it may be worthwhile to carry out a detailed study investigating the relative importance of each of the various site conditions such as positions of down pipes, trees, taps, concrete pathways and drainage on the distortion and cracking of residential structures.

As mentioned in Chapter 4, the probabilistic design approach was based on the exponential model. The trend may be modelled using a number of different mathematical functions, for example, Weibull distributions. Future research may investigate the consequences of using different distribution functions. Furthermore, the coefficient of correlation was determined using the method of least squares for the evaluation of the model. Even though, the results this method are acceptable, there are number of other methods that may correlate better than the method of least squares.

6.3 CONCLUSIONS

The probabilistic method proposed in this study allows the community to make informed decisions regarding the probability of failure and the economic cost of various footing designs. The proposed design method also improves and compliments current deterministic design techniques. In addition, this approach provides the community with a more reliable and rational technique, and may also reduce the likelihood of future litigation. This approach can be applied elsewhere in the world, where similar situations exist.

References

Aitchison, G. D. (1956). Design Criteria for the Determination of Foundation Practices for Domestic Style Buildings with Particular Reference to the Soils of the Adelaide Plains. *Proc. of a Conf. on Foundation Problems of Domestic and Industrial Buildings,* C.S.I.R.O., Adelaide.

Aitchison, G. D. (1970). The Stability of Lightly Loaded Building Foundations on Clay Soils. *Technical Report No. 26, CSIRO, Division of Applied Geomechanics.*

Aitchison, G. D. and Woodburn, J. (1969). Soil Suction in Foundation Design. *Proc. of the 7th Int. Conf. on Soil Mechanics*, Mexico City, pp. 1-8.

Black, W. P. M. (1962). A Method of Estimating the California Bearing Ratio of Cohesive Soils from Plasticity Data. *Geotechnique,* Vol. 12, pp. 271-282.

BRAB (1968). Criteria for Selection and Design of Residential Slab-on-Ground. Publication No. 1571, *Building Research Advisory Board, National Academy of Sciences, National Research Council*, Washington, DC.

Building Act 1923-1964 for the State of South Australia.

Burn, K.N. and Penner, E. (1975). Fast-Growing Trees can Cause House Damage. *Building Research Note 100, National Research Council,* Ottawa, 10p.

Cameron, D. A. (1989). Tests and Reactivity and Prediction of Ground Movements. *Aust. Civil Engrg. Transactions,* Inst. Eng. Aust., Vol. CE31, No. 3, pp. 121-132.

Chan, A. S. and Tumay, M. T. (1991). Architecture of an Expert Database System for Soil Classification Using CPT Data. *Proc. Geotechnical Engrg. Congress,* ASCE, Geotech. Div., Boulder, Colorado, pp. 723-732.

Clisby, M. B. (1963). Predicting the Movement of Clays. *Annual Meeting of the Highway Research Board,* Mississippi State University, State College.

Croney, D. (1952). The Movement and Distribution of Water in Soils. *Geotechnique,* Vol. 3, No. 1, pp. 1-16.

Day, R., Tucker, E. V. and Wood, L. A. (1983). The Computer as an Interactive Geotechnical Data Bank and Analytical Tool. *Proc. Geol. Assoc.,* Vol. 94, No. 2, pp. 123-132.

DeBruyn, C. M. A. (1965). Annual Redistribution of Soil Moisture Suction. *Moisture Equilibria and Moisture Changes in Soils,* Butterworths, London, pp. 122-134.

Domaschuk, L., Flatt, D. G., Kostanski, J. and Kwok, R. (1984). Performance of House Foundations on Expansive Soil. *Proc. of the 5th Int. Conf. on Expansive Soils*, Adelaide, pp. 207-211.

Eden, C. and Hill, C. (1994). The Variability of the Instability Index in Expansive Soils. *Student Project Report,* Uni of Adelaide.

Favre, J. L., Hicher, P. Y. and Kerilis, J. M. (1991). MODELISOL: A Database for Reliability Analyses in Geotechnics. *Proc. of 6th Int. Conf. on Applications of Statistics and Probability in Soil and Struct. Engrg.,* Mexico City, pp. 746-752.

Fraser, R. A. and Wardle, L. L. (1975). The Analysis of Stiffened Raft Foundations on Expansive Soil. *Analysis of Soil Behaviour and its Appl. to Geotech. Structures,* Uni. of NSW.

Hammer, M. J. and Thompson, O. B. (1966). Foundation Clay Shrinkage Caused by Large Trees. ASCE, Vol. 92, SM6.

Holland, J. E. and Cameron, D. A. (1981). Seasonal Heave of Clay Soils. *Transactions College of Civil Engineering,* Inst. of Eng. Aust, pp. 55-67.

Holland, J. E. and Richards, J. (1984). The Repair of Light Structures Damaged by Expansive Clay Movements. *Proc. of the 5th Int. Conf. on Expansive Soils*, Adelaide, pp. 258-262.

Holland, J. E. (1981). The Design, Performance and Repair of Housing Foundations. *Swinburne College Press,* Melbourne, 74 p.

Holland, J. E., Washusen, J. and Cameron, D. A. (1975). Seminar Residential Raft Slabs. *Swinburne College of Technology.,* Melbourne.

Jarrett, R. (1996). Personal Communication.

Jennings, J. E. B. and Knight, K. (1957). The Prediction of Total Heave from the Double Oedometer Test. *Trans. South African Inst. of Civil Eng.*

Johnson, L. D. (1979). Overview for Design of Foundations on Expansive Soils. *US Army Eng. Waterways Expt. Station.,* Misc. Paper GL-79-21, Vicksburg.

Johnson, L. D. and Desai, C. S. (1975). A Numerical Procedure for Predicting Heave. *Second Australian and New Zealand Conf. on Geomechanics.,* Brisbane, pp. 269-273.

Pidgeon, J. T. (1980). The Rational Design of Raft Foundations for Houses on Heaving Soil. *Proc. 7th Reg. Conf. for Afr. On Soil Mech. and Found. Eng.,* Accra.

Pidgeon, J. T. (1983). The Design of Stiffened Raft Foundations on Expansive Soils. *Ground Profile,* No. 33.

Pitt, W. G. (1982). Correlation Between the Real Behaviour and the Theoretical Design of Residential Raft Slabs. *Master of Engineering Thesis,* Swinburne Institute of Technology.

Ranganathan, B. V. and Sathyanarayana, B. (1965). A Rational Method of Predicting Swelling Potential for Compacted Expansive Clays. *Proc. 5th. Int. Conf. on Soil Mechanics and Foundation Engrg.,* Vol. 1, pp. 92-96.

Rawlinsons (1995). *Australian Construction Handbook,* Rawlhouse Publishing Pty. Ltd, NSW, Australia, 904p.

Richards, B. G. (1967). Moisture Flow and Equilibria in Unsaturated Soils for Shallow Foundations. *Permeability and Capillarity of Soils,* ASTM Spec. Tech. Public. No. 417.

Richards, B. G. (1973a). Theoretical Transient Behaviour of Saturated and Unsaturated Soils under Load and Changing Moisture Conditions. *C.S.I.R.O Div. of Applied Geomechanics.,* Tech. Paper 17.

Richards, B. G. (1973b). Model for Slab Foundations on Expansive Clays. *Proc. 8th. Int. Conf. on Soil Mechanics and Foundation Engrg.,* Vol. 2.2, Moscow, 185 p.

Richards, B. G. (1973c). The Analysis of Flexible Road Pavements in the Australian Environment-Changes of Pore Pressure or Soil Suction. *C.S.I.R.O. Div. of Applied Geomechanics,* Tech. Paper 17.

Richards, B. G. (1974). The Analysis of Flexible Road Pavements in the Australian Environment-Stresses, Strains and Displacements Under Traffic Loading. *C.S.I.R.O. Div. of Applied Geomechanics,* Tech. Paper 20.

Richards, B. G. and Chan, C. V. (1971). Theoretical Analyses of Subgrade Moisture Under Australian Environmental Conditions and Their Practical Implications. *Australian Road Research Board,* Vol. 4, No. 6, pp. 32-49.

Richards, B. G., Peter, P. and Martin, R. (1984). The Determination of Volume Change Properties in Expansive Soils. *Proc. of the 5th Int. Conf. on Expansive Soils*, Adelaide, pp. 179-186.

Russam, K. and Dagg, M. (1965). The Effect of Verge Slope and Cover on Soil Moisture Distribution Under a Road in Kenya. *Moisture Equilibrium and Moisture Changes in Soil Beneath Covered Areas,* Butterworth, Sydney, pp. 100-121.

Seed, H. B., Woodward, R. J. Jr. and Lundgren, R. (1962). Prediction of Swelling Potential for Compacted Clays. *Jour. Soil Mechanics and Foundations Div.,* ASCE, Vol. 88, SM3, pp. 53-87.

Selby, J. (1979). Soil Classification and Mapping. *Footings and Foundations for Small Buildings in Arid Climates,* Inst. of Eng., Aust. pp. 20-27.

Selby, J. (1984). Geology and the Adelaide Environment. *South Australian Dept. of Mines and Energy,* Adelaide, 168 p.

Snethen, D. R. (1980). Characterisation of Expansive Soils Using Soil Suction Data. *Proc. of the 4th Int. Conf. on Expansive Soils*, Vol. 1, Denver, pp. 54-75.

Snowden and Meyer Inc. (1976). Design of Light Foundations on Expansive Clay Soils. *Continuing Educ. Seminar,* Texas Section ASCE, Fort Worth, Texas.

Standards Associations of Australia (1988a). *Residential Slabs and Footings: Part 1-Construction*, AS2870.1, 36 p.

Standards Associations of Australia (1988b). *Concrete Structures*, AS3600, 36 p.

Standards Associations of Australia (1990). *Residential Slabs and Footings: Part 2- Guide to Design by Engineering Principles*, AS2870.2, 28 p.

Taylor, J. K., Thomson, B. P. and Shepherd, R. G. (1974). The Soils and Geology of the Adelaide Area. *Bulletin 46, Geol. Survey of SA.*, Dept. of Mines, 84 p.

Touran, A. and Martinez, J. (1991). A Database for Tunnel Planning and Estimating. *Proc. 7th Conf. on Computing in Civil Engrg.*, ASCE, Washington DC, pp. 920-929.

U.S. Army Corps of Engineers (1961). Engineering and Design: Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures). *Eng. Manual EM 1110-345-147,* Washington DC.

Van der Merwe, D. H. (1964). The Prediction of Heave from the Plasticity Index and Percentage Clay Fraction of Soils. *Trans. South African Inst. of Civil Eng.*, Vol. 6 pp. 103-107.

Walsh, P. F. (1974). The Design of Residential Slabs on Ground. *C.S.I.R.O.* Div. of Building Research, Tech. Paper (2nd. Series) No. 5, Australia, 25 p.

Walsh, P. F. (1978). The Analysis of Stiffened Rafts on Expansive Clay. *C.S.I.R.O.* Div. of Building Research, Tech. Paper (2nd. Series) No. 23, Australia, 14 p.

Walsh, P. F. (1984). Concrete Slabs for Houses. *C.S.I.R.O.* Div. of Building Research, Tech. Paper (2nd. Series) No. 48, Australia, 23 p.

Walsh, P. F. (1985). Load Factors and Design Criteria for Stiffened Rafts on Expansive Clays. *Aust. Civil Engineering Transactions,* Inst. Eng., Aust, Vol. CE27, No. 1, pp. 119-123.

Ward, W. H. (1948). The Effects of Vegetation on the Settlement of Structures. *Proc. Conf. of Biology and Civil Engrg.*, Inst. of Civil Eng, pp. 181-194.

Ward, W. H. (1953). Soil Movement and Weather. *Proc. 3rd Int. Conf. on Soil Mech. and Foundation Engrg.*, Switzerland, pp. 477-483.

Warner, R. F., Rangan, B. V. and Hall, A. S. (1989). *Reinforced Concrete.* Longman Cheshire, Melbourne, Australia, 553p.

Washusen, J. A. (1977). The Behaviour of Experimental Raft Slabs on Expansive Clay Soils in the Melbourne Area. *Res. Dept. EC2, Swinburne College of Technology.*

Wood, L. A. (1980). A Database of Site Investigation Reports. *Ground Engineering,* Vol. 13, No. 6, pp. 2-6.

Wood, L. A., Tucker, E. V. and Day, R. B. (1982). Geoshare: The Development of a Databank of Geological Records. *Adv. Eng. Software,* Vol. 4, No. 4, pp. 136-142.

Wood, L. A., Tucker, E. V. and Day, R. B. (1983). Further Development of a Geotechnical/Geological Database. *Adv. Eng. Software.*, Vol. 5, No. 2, pp. 81-85.

Woodburn, J. A. (1979). Interaction of Soil, Footings and Superstructures. *Footings and Foundations for Small Buildings in Arid Climates,* Fargher, P. J., Woodburn, J.A. and Selby, J., (eds) Inst. of Eng. Aust., S.A. Div., pp. 79-89.

Wray, W. K. (1980). Discussion of The Behaviour and Design of Heaving Slabs on Expansive Clays. *Proc. of the 4th Int. Conf. on Expansive Soils.*, Vol. 2, Denver, pp. 757-763p.

Wray, W. K. (1984). The Principle of Soil Suction and its Geotechnical Engineering Applications. *Proc. of the 5th Int. Conf. on Expansive Soils*, Adelaide, pp. 114-118.

APPENDIX A

SAMPLE COUNCIL SURVEY SHEET

RESIDENTIAL FOUNDATIONS ON EXPANSIVE SOILS
COUNCIL SURVEY SHEET

Information from Construction Report:

Building Application No.	

Name of the Council.	

House Holder:
Address:

Date of Construction	

Super-Structure Construction:

Solid Brick	
Articulated Solid Brick	
Brick Veneer	
Articulated Brick Veneer	
Timber Framed	

Type of Stiffened Raft:

Standard	
Grillage	

Regular Grid	
Irregular Grid	

Shape of Footing:

Rectangular	
L-Shaped	
Other	

Site Preparation:

Level	
Cut	
Fill	
Split Level	
Cut and Fill	

Soil Classification:

S	
M	
H	
E1	
E2	

RB1	
RB3	
RB4	
RB5	
BE	
P	
Other	

Heave Ys (mm)	

APPENDIX B

A SAMPLE LETTER TO OBTAIN PERMISSION FOR THE USE OF COUNCIL'S DATABASE

The Chief Executive Officer
Campbelltown City Council
172 Montacute Road
ROSTREVOR
S.A. 5073

Dear Sir/Madam,

The Department of Civil and Environmental Engineering of this University has recently commenced a research project that focuses on improving the performance of residential reinforced concrete raft footings founded on expansive clay soils.

Unlike existing design techniques that rely on highly idealised mathematical models, this research will develop a probabilistic approach to the design of residential foundations. The benefits of such a project will include a quantification of the probability of footing failure, a more realistic approach to the design of residential foundations and a reduction of the likelihood of future litigation.

To establish an accurate and reliable design technique, a large amount of data is required. In this regard we would greatly appreciate your assistance in obtaining copies of records including applications for approval of building work and construction reports. Data that we will require includes:

- Current owner's name and address;
- Building type;
- Raft footing dimensions and details;
- Engineer's soil classification and borehole details.

At a later stage, we will be contacting owners so that a site inspection can be arranged.

As result, we are writing to you to seek your cooperation in permitting Ranganath Barthur, a Research Assistant directly related to this project, to visit your Building Applications Branch and obtain relevant data in approximately two weeks time, if convenient.

It is estimated that data collection will take approximately 1 week to complete.

The results of this project will be made available to the Council and the engineering profession. Additionally, your assistance will be formally recognised in the publications resulting from this research.

We believe that a project of this nature will be of extreme benefit to the engineering profession and we look forward to your reply. If you have any queries related to this work, do not hesitate to contact the undersigned.

Yours sincerely,

Mark Jaksa B.E.(Hons), M.I.E. Aust. CPEng.
Lecturer in Geotechnical Engineering

Ranganath Barthur B.E.
Research Assistant, Postgraduate Student.

APPENDIX C

A SAMPLE LETTER TO OBTAIN PERMISSION FOR THE INSPECTION OF HOUSES

Mark Jaksa
Lecturer
Phone : (08)3034317

18 October 1994

The Resident / Householder

Dear Sir / Madam,

The Civil and Environmental Engineering Department of this University has commenced a research project titled "A Probabilistic Approach to the Design of Residential Foundations", that focuses on improving the performance of residential reinforced concrete raft footings, or foundations, founded on expansive soils.

The current stage of the project requires that a survey of approximately 500 houses, built between 1983 and 1992, be carried out within the next 2 - 3 months to provide information on how the footing has performed since its construction. The outcome of this survey and the consequent research will help provide a more reliable technique for the design of these footings.

There is evidence that in a few instances, even with current design methods, some cracking has occurred and we wish to determine the prevalence of these cases. We will be interested to take note of even the slightest evidence of cracking, cracking of a type that would frequently not be observed by the home owner and certainly be of little concern in terms of building stability.

Your local council is supporting the project and has made available its records of building applications. These records show that your house was constructed with a reinforced concrete raft footing on either an expansive or stable soil, making it an ideal candidate for a survey of this nature.

Accordingly, we are writing to you seeking your cooperation in permitting Ranganath Barthur, a Postgraduate Student & Research Assistant directly related to this project, to visit your house to inspect and measure for possible cracking both internally and externally. It is anticipated that the inspection should take between 10 to 30 minutes. We would be grateful if you would return the enclosed questionnaire in the stamped return addressed envelope provided, and indicate your willingness or otherwise to participate in the inspection survey.

Please be assured that the results of any survey will be treated with strict confidentiality in that no names and addresses will be published. In addition, a short report summarising the visit and providing advice on the cause of cracking will be provided to all participants of the survey.

Your's faithfully,

Mark Jaksa B.E.(Hons.), M.I.E Aust. CPEng
Lecturer in Geotechnical Engineering

Ranganath Barthur B.E
Research Assistant, Postgraduate Student

A PROBABILISTIC APPROACH TO THE DESIGN OF RESIDENTIAL FOUNDATIONS
QUESTIONNAIRE TO RESIDENT/HOUSEHOLDER

Resident's Name: --

Address:--

(1) Will you allow your house to be included in this research survey? **YES** **NO**

(2) If yes, which day would you prefer for the inspection?

MON

TUE

WED

THU

FRI

SAT

SUN

(3) What time of day best suits you?

8 - 11

11 - 2

2 - 5

(4) Alternatively, is there a specific time and date that you might want the survey done? --------------------------------

(5) How may we contact you to confirm a suitable time for the inspection? Phone: -------------------------(H)

-------------------------(W)

Signed: --

Additional Comments:

Thank you for taking the time to answer this questionnaire.

APPENDIX D

A PRELIMINARY RESIDENTIAL FOOTING ASSESSMENT SHEET

The University of Adelaide
Preliminary Residential Footing Assessment

Building Application No.	

Resident's Name	

Footing Construction Date	
Date of Inspection	
Age of Footing	

Super-structure	
Footing Construction Type	
Soil Type*	
Est. Maximum Heave*	

Max. Crack Width Encountered	
Interior	Exterior
Damage Category (AS2870.1-1988)	

Comments:

The comments expressed in this report are of a preliminary nature based on visual inspections.
Should you require a more detailed assessment and repair strategy it is suggested that you
consult a consulting civil engineer, many of which may be found in the yellow pages.
Your assistance in this research is greatly appreciated and it is expected that the results of this
study will lead to a more reliable design technique.

Mark Jaksa B.E (Hons), M.I.E. Aust. CPEng	Ranganath Barthur B.E
Lecturer in Geotechnical Engineering	Research Assistant, Postgraduate Student

APPENDIX E

CLASSIFICATION OF DAMAGE AS SPECIFIED BY AS2870.1-1988

Table F1 Classification of damage with reference to walls.

Description of Typical Damage	Crack Width Limit	Damage Category	Degree of Damage
Hairline	0.0 - 0.1 mm	0	Negligible
Fine Cracks	0.1 - 1.0 mm	1	Very Slight
Distinct Cracks	1.0 - 5.0 mm	2	Slight
Wide Cracks	5.0 - 15 mm	3	Moderate
Extensive Cracks	15 - 25 mm	4	Severe

Table F2 Classification of damage with reference to concrete floors.

Description of Typical Damage	Crack Width Limit	Damage Category	Degree of Damage
Hairline	0.0 - 0.3 mm	0	Negligible
Fine Cracks	0.1 - 1.0 mm	1	Very Slight
Distinct Cracks	1.0 - 2.0 mm	2	Slight
Wide Cracks	2.0 - 4.0 mm	3	Moderate
Extensive Cracks	4.0 - 10 mm	4	Severe

APPENDIX F

A SAMPLE MACRO SHEET FOR RAFT STIFFNESS CALCULATIONS

Program for calculating : Raft Stiffness	Appn. No.

Address :	

Bending Direction :	Short	
Young's Modulas E=	**1.13E+04**	*Mega Pascals*

Over Lapping Rectangle	1

Beam Type		**All Dim are in *MM***		**Moment of Inertia** *(mm^4)*
External		**ybar**	**Ybar**	
Slab W	1000			
D	100	50		
Beam W				
D				
Internal 1				
Slab W	2000			
D	100	50		
Beam W	250			
D	500	250	133.33	
Internal 2				
Slab W	2000			
D	100	50		
Beam W	300			
D	650	325	196.91	
Internal 3				
Slab W	2000			
D	100	50		
Beam W	300			
D	500	250	143.75	

Input no. external beams	w=	
Input no. internal beams	x=	
Input no. internal beams	y=	
Input no. internal beams	z=	

Raft width	**L=**

Raft stifness K= **E*(w*M11+x*M12+y*M13+z*M14)/raft width**	*MN/m^2*

APPENDIX G

A SAMPLE ANALYSIS OF MITCHELL METHOD

SITE : Articulated brick veneer
REF : 5.2.2
DATE : 28.11.96

Mitchell Method Design

Length of Structure	- L	18.00	(m)
Breadth of Structure	- B	11.00	(m)
Edge Load on West End	- Pl	12.30	(kN/m)
Edge Load on East End	- Pr	12.30	(kN/m)
Edge Load on North Side	- Pn	12.30	(kN/m)
Edge Load on South Side	- Ps	12.30	(kN/m)
North-South Centre Load	- Tns	.00	(kN/m)
East-West Centre Load	- Tew	.00	(kN/m)
Uniform Distributed Load	- w`	5.55	(kPa)
Subgrade Modulus	- k	1000.00	(kPa/m)
No. beams parallel to long span		4	
No. beams parallel to short span		6	
Youngs Modulus of concrete	- Ec	15000.00	(MPa)

LONG SPAN - CENTRE HEAVE CONDITION

Soil Heave	Ym	15.00	mm
Depth Soil Suction Change	a	.67	m
Mound shape factor	Ml	40.30	
Total Stiffness required	EI	1.00	MNm^2/m
Giving Deflection of		9.58	mm (NON-CRITICAL)

DISTANCE FROM EDGE (m)	BENDING MOMENT (kNm/m)	SHEAR FORCE (kN/m)	SOIL HEAVE (mm)	FOOTING MOVEMENT (mm)	SOIL REACTION (kN/m)
.000	.000	1.301	.000	-1.833	10.999
.600	-2.182	2.275	14.070	1.743	7.396
1.200	-2.730	.122	14.953	4.533	6.252
1.800	-2.329	-.929	14.998	6.340	5.195
2.400	-1.615	-1.162	15.000	7.309	4.615
3.000	-.935	-.989	15.000	7.696	4.382
3.600	-.428	-.684	15.000	7.747	4.352
4.200	-.114	-.395	15.000	7.644	4.414
4.800	.046	-.180	15.000	7.500	4.500
5.400	.102	-.046	15.000	7.372	4.577
6.000	.101	.022	15.000	7.281	4.631
6.600	.076	.046	15.000	7.226	4.664
7.200	.046	.045	15.000	7.199	4.681
7.800	.022	.033	15.000	7.188	4.687
8.400	.007	.017	15.000	7.186	4.689
9.000	.002	.000	15.000	7.185	4.689
9.600	.007	-.017	15.000	7.186	4.689
10.200	.022	-.033	15.000	7.188	4.687
10.800	.046	-.045	15.000	7.199	4.681
11.400	.076	-.046	15.000	7.226	4.664
12.000	.101	-.022	15.000	7.281	4.631
12.600	.102	.046	15.000	7.372	4.577
13.200	.046	.180	15.000	7.500	4.500
13.800	-.114	.395	15.000	7.644	4.414
14.400	-.428	.684	15.000	7.747	4.352
15.000	-.935	.989	15.000	7.696	4.382
15.600	-1.615	1.162	15.000	7.309	4.615
16.200	-2.329	.929	14.998	6.340	5.195
16.800	-2.730	-.122	14.953	4.533	6.252
17.400	-2.182	-2.275	14.070	1.743	7.396
18.000	0.000	-1.301	0.000	-1.833	0.999

LONG SPAN - EDGE HEAVE CONDITION

Soil Heave	Ym	15.00 mm
Depth Soil Suction Change	a	.67 m
Mound shape factor	Ml	40.30
Total Stiffness required	EI	1.00 MNm^2/m
Giving Deflection of		.99 mm (NON-CRITICAL)

DISTANCE FROM EDGE (m)	BENDING MOMENT (kNm/m)	SHEAR FORCE (kN/m)	SOIL HEAVE (mm)	FOOTING MOVEMENT (mm)	SOIL REACTION (kN/m)
.000	.000	-.180	15.000	-1.330	12.480
.600	-1.294	.527	.930	-.614	7.930
1.200	-.633	-.966	.047	-.364	4.401
1.800	-.134	-.548	.002	-.342	4.107
2.400	.025	-.140	.000	-.368	4.420
3.000	.034	.009	.000	-.385	4.625
3.600	.014	.026	.000	-.390	4.685
4.200	.002	.012	.000	-.390	4.686
4.800	-.001	.003	.000	-.390	4.677
5.400	-.001	-.001	.000	-.389	4.673
6.000	.000	-.001	.000	-.389	4.671
6.600	.000	.000	.000	-.389	4.671
7.200	.000	.000	.000	-.389	4.672
7.800	.000	.000	.000	-.389	4.672
8.400	.000	.000	.000	-.389	4.672
9.000	.000	.000	.000	-.389	4.672
9.600	.000	.000	.000	-.389	4.672
10.200	.000	.000	.000	-.389	4.672
10.800	.000	.000	.000	-.389	4.672
11.400	.000	.000	.000	-.389	4.671
12.000	.000	.001	.000	-.389	4.671
12.600	-.001	.001	.000	-.389	4.673
13.200	-.001	-.003	.000	-.390	4.677
13.800	.002	-.012	.000	-.390	4.686
14.400	.014	-.026	.000	-.390	4.685
15.000	.034	-.009	.000	-.385	4.625
15.600	.025	.140	.000	-.368	4.420
16.200	-.134	.548	.002	-.342	4.107
16.800	-.633	.966	.047	-.364	4.401
17.400	-1.294	-.527	.930	-.614	7.930
18.000	.000	.180	15.000	-1.330	12.480

SHORT SPAN - CENTRE HEAVE CONDITION

Soil Heave	Ym	15.00 mm
Depth Soil Suction Change	a	.67 m
Mound shape factor	Ml	24.63
Total Stiffness required	EI	1.00 MNm^2/m
Giving Deflection of		11.18 mm (NON-CRITICAL)

DISTANCE FROM EDGE (m)	BENDING MOMENT (kNm/m)	SHEAR FORCE (kN/m)	SOIL HEAVE (mm)	FOOTING MOVEMENT (mm)	SOIL REACTION (kN/m)
.000	.000	3.175	.000	-2.489	9.125
.367	-1.629	3.736	12.257	.074	4.494
.733	-2.740	2.071	14.558	2.418	4.451
1.100	-3.148	.448	14.938	4.393	3.867
1.467	-3.068	-.608	14.993	5.945	3.317
1.833	-2.702	-1.181	14.999	7.085	2.902
2.200	-2.202	-1.405	15.000	7.862	2.617
2.567	-1.672	-1.398	15.000	8.342	2.441
2.933	-1.176	-1.256	15.000	8.598	2.347
3.300	-.750	-1.050	15.000	8.696	2.312
3.667	-.407	-.826	15.000	8.692	2.313
4.033	-.145	-.613	15.000	8.634	2.334
4.400	.043	-.425	15.000	8.557	2.362
4.767	.167	-.265	15.000	8.485	2.389
5.133	.237	-.126	15.000	8.436	2.407
5.500	.259	.000	15.000	8.419	2.413
5.867	.237	.126	15.000	8.436	2.407
6.233	.167	.265	15.000	8.485	2.389
6.600	.043	.425	15.000	8.557	2.362
6.967	-.145	.613	15.000	8.634	2.334
7.333	-.407	.826	15.000	8.692	2.313
7.700	-.750	1.050	15.000	8.696	2.312
8.067	-1.176	1.256	15.000	8.598	2.347
8.433	-1.672	1.398	15.000	8.342	2.441
8.800	-2.202	1.405	15.000	7.862	2.617
9.167	-2.702	1.181	14.999	7.085	2.902
9.533	-3.068	.608	14.993	5.945	3.317
9.900	-3.148	-.448	14.938	4.393	3.867
10.267	-2.740	-2.071	14.558	2.418	4.451
10.633	-1.629	-3.736	12.257	.074	4.494
11.000	.000	-3.175	.000	-2.489	9.125

SHORT SPAN - EDGE HEAVE CONDITION

Soil Heave	Ym	15.00	mm
Depth Soil Suction Change	a	.67	m
Mound shape factor	Ml	24.63	
Total Stiffness required	EI	1.00	MNm^2/m
Giving Deflection of		1.34	mm (NON-CRITICAL)

DISTANCE FROM EDGE (m)	BENDING MOMENT (kNm/m)	SHEAR FORCE (kN/m)	SOIL HEAVE (mm)	FOOTING MOVEMENT (mm)	SOIL REACTION (kN/m)
.000	.000	3.688	15.000	-1.599	8.612
.367	-1.817	2.299	2.743	-.933	7.850
.733	-1.686	-1.049	.442	-.512	3.917
1.100	-1.048	-1.646	.062	-.317	2.351
1.467	-.479	-1.254	.007	-.264	1.937
1.833	-.129	-.693	.001	-.274	2.013
2.200	.029	-.272	.000	-.302	2.218
2.567	.071	-.042	.000	-.326	2.394
2.933	.060	.048	.000	-.341	2.500
3.300	.036	.060	.000	-.347	2.547
3.667	.016	.044	.000	-.349	2.558
4.033	.004	.023	.000	-.348	2.554
4.400	-.001	.009	.000	-.347	2.547
4.767	-.003	.002	.000	-.346	2.540
5.133	-.003	.000	.000	-.346	2.536
5.500	-.002	.000	.000	-.346	2.535
5.867	-.003	.000	.000	-.346	2.536
6.233	-.003	-.002	.000	-.346	2.540
6.600	-.001	-.009	.000	-.347	2.547
6.967	.004	-.023	.000	-.348	2.554
7.333	.016	-.044	.000	-.349	2.558
7.700	.036	-.060	.000	-.347	2.547
8.067	.060	-.048	.000	-.341	2.500
8.433	.071	.042	.000	-.326	2.394
8.800	.029	.272	.000	-.302	2.218
9.167	-.129	.693	.001	-.274	2.013
9.533	-.479	1.254	.007	-.264	1.937
9.900	-1.048	1.646	.062	-.317	2.351
10.267	-1.686	1.049	.442	-.512	3.917
10.633	-1.817	-2.299	2.743	-.933	7.850
11.000	.000	-3.688	15.000	-1.599	8.612

Dept.Civil Eng.Univ.of Adelaide

SITE:Articulated brick veneer DATE:28.11.96

REF :5.2.2

MITCHELL METHOD ANALYSIS

INPUT DATA

LENGTH	L	:	18.00	m
BREADTH	B	:	11.00	m
LOAD ON SCREEN WEST	Pl	:	12.30	kNm
LOAD ON SCREEN EAST	Pr	:	12.30	kNm
LOAD ON SCREEN NORTH	Pn	:	12.30	kNm
LOAD ON SCREEN SOUTH	Ps	:	12.30	kNm
CENTRELINE LOAD N/S	Tns	:	.00	kNm
CENTRELINE LOAD E/W	Tew	:	.00	kNm
UNIFORMLY DISTRIBUTED LOAD	w`	:	5.55	kPa
LONG SPAN - DEFLECTION RATIO		:	600.00	
- PERMISSIBLE DEFLECTION		:	30.00	mm
SHORT SPAN - DEFLECTION RATIO		:	600.00	
- PERMISSIBLE DEFLECTION		:	18.33	mm
SUBGRADE MODULUS	k	:	1000.00	kPa/m
SOIL HEAVE - CENTRE HEAVE	Ym	:	15.00	mm
- EDGE HEAVE	Ym	:	15.00	mm
DEPTH OF SUCTION CHANGE - CENTRE HEAVE	a	:	.67	m
- EDGE HEAVE	a	:	.67	m

SUMMARY OF ANALYSIS

	LONG SPAN	SHORT SPAN
CENTRE HEAVE		
Moment kNm/m	-2.7	-3.1
Shear kN/m	-2.3	-3.7
Stiffness MNm2/m	1.000	1.000
EDGE HEAVE		
Moment kNm/m	-1.3	-1.8
Shear kN/m	1.0	-3.7
Stiffness MNm2/m	1.000	1.000

REQUIRED CAPACITIES PER BEAM

	LONG SPAN	SHORT SPAN
CENTRE HEAVE		
Ultimate strength kNm	-9.4	-11.8
Irequired mm4	.1833E+09	.2000E+09
EDGE HEAVE		
Ultimate strength kNm	-4.4	-6.8
Irequired mm4	.1833E+09	.2000E+09
Number of beams	4	6
Youngs modulus MPa	15000.0	15000.0

SITE: Articulated brick veneer DATE: 28.11.96
REF : 5.2.2

Dept.Civil Eng.Univ.of Adelaide

RAFT FOOTING BEAM DESIGN

INPUT DATA

TOP CONCRETE COVER	50.0mm	BOTTOM CONCRETE COVER	65.0mm
SLAB WIDTH	1000.0 mm	BEAM WIDTH	300.0mm
AREA OF TOP STEEL	418.0 mm2	AREA OF BOTTOM STEEL	330.0mm2
SLAB THICKNESS	100.0mm	fsy	400.0MPa
F\`c	20.0MPa	Ec	15000.0MPa
F\`t (centre heave)	1.8MPa	F\`t (edge heave)	2.7MPa
Mw (centre heave)	9.4 kNm	Mw (edge heave)	5.5kNm
Ireq (centre heave)	.20000E9 mm4	Ireq (edge heave)	20000E9 mm4

RESULTS

	CENTRE HEAVE	EDGE HEAVE
REQUIRED BEAM DEPTH (mm)	300.0	300.0
ULTIMATE MOMENT (kNm)	39.0	30.5
ULTIMATE/WORKING MOMENT	4.13	5.59
ULTIMATE/CRACKING MOMENT	2.04	1.94
Ieff (E9 mm4)	1.127083	1.127083
AREA OF STEEL INPUT OR VALUE FOR Mu/Mcr>1.20 IF GREATER (mm2)	418.0	330.0

NOTE: AS2870 App. E2.f requires raft to resist centre and edge heave moments of approximately the same magnitude.

SUMMARY OF SECTION TO BE USED

DEPTH	:	300.00 mm
WIDTH	:	300.00 mm
TOP STEEL	:	2 Y12
BOTTOM STEEL	:	3 Y12
LIGATURES	:	w6 1000cts
SLAB	:	F72

CORRIGENDA

"Design of Residential Footings Built on Expansive Soil Using Probabilistic Methods"

by R. Barthur

March 1997

---

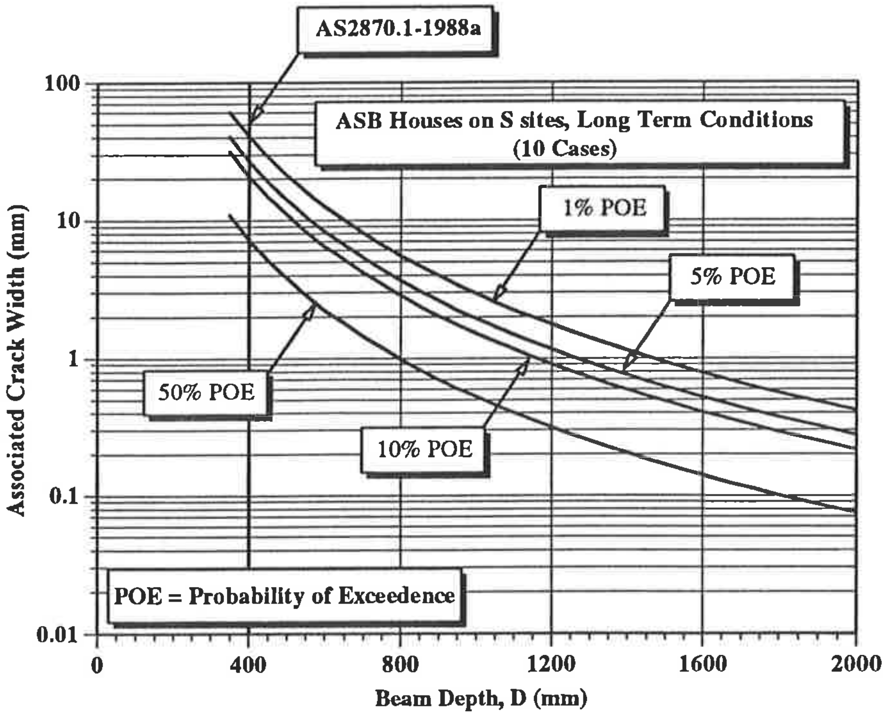
pg. xii	In the 6th line of the 1st paragraph, *replace* "many" *with* "216"
pg. xviii, **Notation**	Adjacent to y_m, *replace* "Initial Differential Movement, Design value of differential Movement" *with* "Differential mound movement"
pg. 14, Figure 2.3	*Include* after the caption "(Mitchell, 1979)"
pg. 15	In the sentence after Equation (2.4), *replace* "pF" *with* "%/pF"
pg. 16	*Delete* 3rd sentence in the paragraph 4
pg. 18	*Include* the following *after* the last dot point: "• Erroneous site classification; • Inadequate design detailing; and • Poor construction."
pg. 27	*Include* after the 3rd sentence in the 3rd dot point "In addition, it assumes soil volume changes are directly related to water volume changes, so is only applicable if the soil is close to saturation."
pg. 31, point 2	*Replace* "strength" in the first and second sentences *with* "allowable bearing capacity"
pg. 32, point 3	In the 5th line, *include* "in" *after* "given"
pg. 32, point 3	*Include* after the second sentence "Δu representing the change in suction, pF, and H representing the depth, as shown in Figure 2.4."
pg. 35	*Delete* first paragraph
pg. 36	In the 4th line of Section 2.3, *include* "to" *before* "minimise"
pg. 39, Section 2.3.4	In the 4th line of the 2nd Paragraph, *include* "raft" *after* "stiffened"
pg. 40	In the 3rd sentence of the 1st paragraph, *delete* "be"
pg. 40	*Delete* 1st dot point
pg. 48, Section 2.5.3	In the 3rd line *include* "the" *before* "Lytton (1971)"
pg. 48, Section 2.5.3	In the 5th and 6th lines *remove* "a"
pg. 50	In the 2nd line of the 2nd dot point, *delete* "even with the use of computer program"
pg. 51	In the 5th line of 2nd paragraph, *replace* "have" *with* "has"

pg. 51	In the right hand side of Figure 2.21, *interchange* "flat + parabola"
pg. 51	In the last sentence of the 2nd paragraph, *delete* "and it can be expected that the bending moment will be dependent on the depth adopted"
pg. 51	Second to last line *include* "the" *before* "beam"
pg. 51	Last line *replace* "Inorder *with* "In order"
pg. 52	In the 1st sentence of the 2nd paragraph, *include* "the" *after* "by"
pg. 53	In the 4th line of the 1st paragraph, *include* "the" *after* "Equation (2.25)"
pg. 53	In the 1st line of the 2nd paragraph, *include* "the" *after* "established"
pg. 55	In the 6th sentence of Section 2.6, *delete* "strengths and performance of buildings will never have exactly the same observed values, even under seemingly identical conditions. Subsequently, the task of the design engineer now is to deal with this uncertainty in a realistic and economical manner. How the engineer chooses to treat the uncertainty in a given phenomenon depends upon the situation. The probabilistic approach appears to be realistic in this case. This section"
pg. 55	*Include* after the last line of Section 2.6.2, "of faith to the client is not in the design engineer's interests. It is preferable to make the level of uncertainty as clear as possible to the client. In addition, it is worthwhile to discuss the additional cost required to reduce the level of risk associated with each design. Subsequently, an extensive programme of observation of houses in South Australia has been carried out as a basis to develop such a probabilistic design procedure. A design table so"
pg. 55	In the 2nd line of the 1st dot point *replace* "millions" *with* "hundreds of thousands"
pg. 55	In the 3rd dot point, *replace* "compliments" *with* "complements"
pg. 56	In the 2nd line of the 2nd paragraph, *delete* "is simply supported elastic beam"
pg. 56	In the 12th line of the 2nd paragraph, *include* "be" *after* "also"
pg. 56	In the 13th line of the 2nd paragraph, *replace* "stiffenesses were" *with* "stiffness was"
pg. 57	In the 3rd line of the 1st paragraph, *replace* "has" *with* "have"
pg. 57	In the 3rd sentence of the 1st paragraph, *replace* "these" *with* "this"

pg. 58	In the 4th and 5th lines *replace* "a number of" *with* "much"
pg. 58	*Delete* the 2nd sentence of the 3rd dot point
pg. 61	in the last sentence *replace* "ease" *with* "simplify"
pg. 66	In the 1st line of the 1st paragraph, *replace* "utilises" with "utilise"
pg. 67	In the 1st point under "Technical", *delete* "in the"
pg. 69	In the 11th line of the 1st paragraph, *include* "because" *after* "included"
pg. 69	In the last paragraph after the 1st sentence, *include* "The engineers' estimate of heave was adopted as the site heave in the data base. Where the engineer modified the surface heave for the effects of trees, in accordance with local practice, these modifications were ignored."
pg. 73	In the first dot point, at the end of the sentence *include* "Movement in interior expansion joints was not measured"
pg. 74	In the 1st line of the last paragraph *replace* "summaries" *with* "summarises"
pg. 75, Table 3.5	*Include* after the caption "(percentages are relative to the total population)"
pg. 76	In the 2nd line, *include* "of the critical section" *after* "directions"
pg. 77	In the last sentence of the 1st paragraph, *replace* "total" *with* "total uncracked"
pp 78-81	On the horizontal axis of Figures 3.6 to 3.11, *replace* "MPa" *with* "MPa/m"
pg. 84	In the 2nd line of the 3rd paragraph, *replace* "calculations" *with* "calculation"
pg. 93	In the 2nd to last line of the last paragraph, *include* "which" *after* "limitation,"
pg 96	In the 2nd line of the 2nd paragraph, *replace* "conjective" *with* "conjecture"
pg. 97	*Replace* the 4th sentence of the 2nd paragraph," *with* "In this study an 8 metre wide dwelling was used to determine the second moment of area."
pg. 100	In Figure 4.17, *replace* the bold vertical line at 800 mm (representing the size indicated by AS2870.1-1988a) *with* a bold vertical line at 625 mm
pg. 110	In Figure 4.30, *replace* the bold vertical line at 800 mm (representing the size indicated by AS2870.1-1988a) *with* a bold vertical line at 625 mm
pg. 111	At the beginning of the first paragraph, *include* "It is evident from Equations 4.6 and 4.7, the curve fitting parameters from the maximum likelihood method to a large extent, depend on the number of cracked houses and the severity of their damage. It is also evident from Table 3.4, that the number of cracked articulated solid brick houses built on M

sites is more than those built on H sites. Due to these reasons, the curves tend to predict more conservative values for articulated soild brick houses on M sites than houses built on H sites.

pg. 112 — *Replace* Figure 4.33 *with* the following figure:



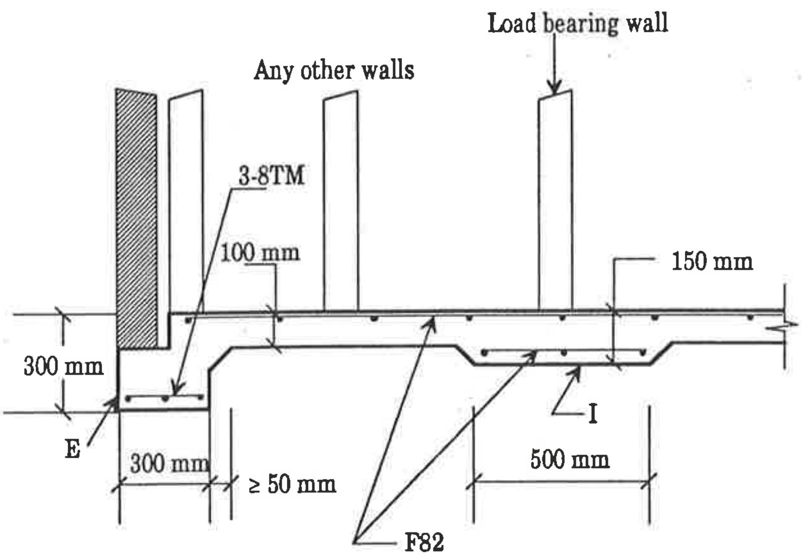
pg. 113 — In Figure 4.35, *replace* the bold vertical line at 800 mm (representing the size indicated by AS2870.1-1988a) *with* a bold vertical line at 625 mm

pg. 115 — Equation 4.12 should read $CI = m_s \pm z_b\left(\frac{\sigma}{\sqrt{s}}\right)$

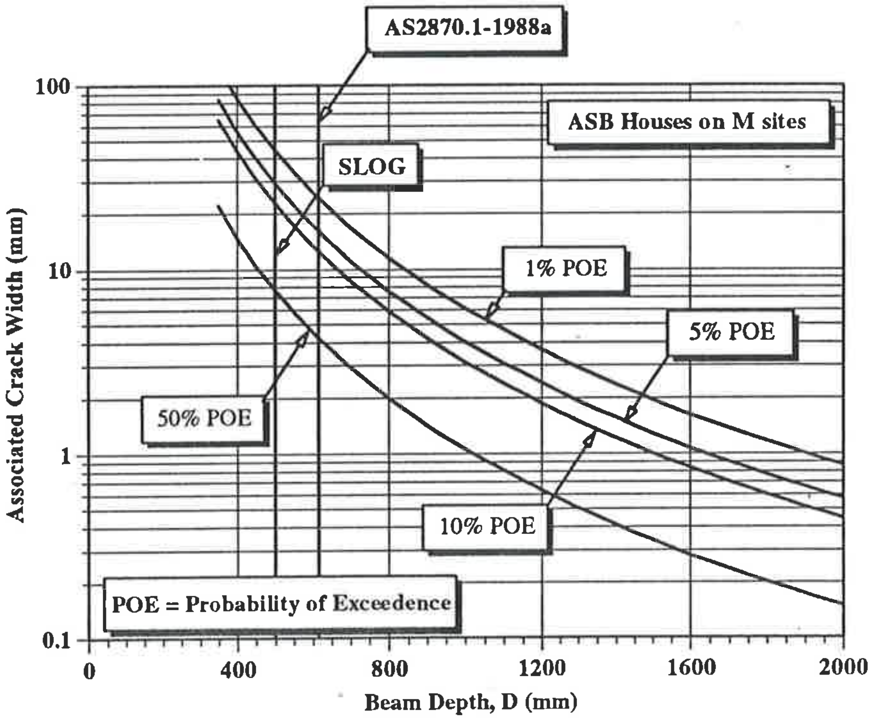
pg. 125 — In Figure 4.52, *replace* the bold vertical line at 800 mm (representing the size indicated by AS2870.1-1988a) *with* a bold vertical line at 625 mm

pg. 128 — In the 3rd line of the 1st paragraph, *include* "of" *after* "series"

pg. 131, Figure 5.4 — *Replace* Figure 5.4 *with* the following figure:



pg. 132	In Figure 5.5, *replace* "350" *with* "400"
pg. 132	In Figure 5.5, *replace* "Spacing, *S*" *with* "Maximum spacing"
pg. 133, Section 5.3	In the 1st line of Section 5.3, *include* "(Figures 4.14 to 4.19)"*after* "chapter"
pg. 135	*Replace* Figure 5.9 *with* the following figure:



pg. 138	*Delete* 1st sentence of the 1st paragraph
pg. 141	In the last line of the 4th paragraph *replace* "slightly" *with* "significantly"
pg. 142	In the 3rd line of the last paragraph, *replace* "compliments" *with* "complements"