STUDY REPORT

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Soil Expansivity in the Auckland Region

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the Building Research Levy and Manukau City Council

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Preface

This report provides a preliminary assessment on the expansivity of soils at eight sites within the Auckland Region and considers the applicability of the design methodology set out in AS2870 for buildings constructed in accordance with NZS3604:1999.

Acknowledgments

The work was jointly funded by the Building Research Levy and Manukau City Council.

On behalf of Fraser Thomas Ltd, this report was written by Mr Barry B.J. Brown, Dr Peter R. Goldsmith, Mr J. Patrick M. Shorten and Mrs Leanne Henderson and was peer-reviewed by Dr Peter Mitchell, Adelaide and Professor Michael Pender, University of Auckland.

Note

This report is intended for researchers, geotechnical and structural engineers, property developers, and other workers in the field of building construction to NZS 3604:1999.

SOIL EXPANSIVITY IN THE AUCKLAND REGION

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1.0 INTRODUCTION

Expansive soils are those that experience appreciable volume change when the soil moisture is altered. Soil moisture may be altered by a number of factors which may act in combination including seasonal influence, the effects of trees, drains, roads etc. The swelling and shrinking of soils can adversely affect buildings.

A significant proportion of new residential construction in New Zealand has concrete slab on ground floors. Before the introduction of New Zealand Standard NZS 3604: 1999 "Timber Framed Buildings", a minimum founding depth was specified in NZS 3604:1990, its predecessor standard, as a means of mitigating the effects of expansive soils on light timber framed buildings in New Zealand. However, NZS 3604:1999 excludes foundations on expansive soils from its scope and refers the designer to Section 17 of the Standard, for additional information on expansive soils. In Section 17 of this Standard, it is suggested that the designer refer to the Australian Standard AS 2870 "Residential Slabs and Footings – Construction", as a means of classification of expansive soil sites and providing a standard footing design, or that a specific engineering design be provided.

There is uncertainty as to the relevance or applicability of AS 2870 to the design and construction of foundations for residential buildings in New Zealand. In particular, AS 2870 does not provide any New Zealand-specific design parameters to support its application to New Zealand climatic and soil conditions. AS 2870 specifically relies on knowledge of the seasonal change in soil suction profile for any particular region or soil profile, as well as the shrink-swell properties of the soil and the depth of seasonal shrinkage cracking. It does not address other factors that may affect soil moisture change, such as the effects of trees, drains, roads etc.

As the greater Auckland area is both a centre of high residential growth and is recognised as having zones with expansive clay soils, it provides a logical starting point for addressing expansive soil design issues in New Zealand.

The primary aim of the research project is to:

(a) Determine whether or not AS 2870 provides an appropriate means of site assessment with respect to expansive soils and the design of footing systems for domestic construction within the context of NZS 3604:1999, and

(b) If it is concluded that AS 2870 is appropriate, to provide recommended soil suction change profiles and site classification guidelines for the common soil types in the Auckland region for use within the AS 2870 design methodology.

To address these aims, the expansive characteristics of representative soils within the Auckland region have been investigated. This research has involved field investigations and laboratory testing at eight locations within the Auckland region, referred to as Sites A to H, between February 2002 and March 2003, which included two summers and one winter season.

The laboratory testing reported herein was undertaken by the Geomechanics Laboratory of the University of Auckland, School of Engineering. Climatic data was provided by the Climate Research and Information Services, National Institute of Water & Atmospheric Research (NIWA).

2.0 RESEARCH APPROACH

2.1 Introduction

For the results of the research to be meaningful, the following criteria should be met:

(a) That there be sufficient test results on which to meaningfully determine geotechnical properties

(b) That the conclusions arising from the research are able to be supported through the correlation of theoretical analyses with physical measures and observations of building performance.

2.2 Staged investigation

The research project was conceived in two parts, as shown in Figure 1:

(a) Stage I - involving the measurement and determination of geotechnical field and laboratory parameters

(b) Stage II – involving the correlation of the physical measurement of soil shrink-swell effects with observations of building performance.

Stage I, which is the subject of this report, was jointly funded by the Building Research Levy and Manukau City Council.

Stage II, originally intended to overlap Stage I, was unable to proceed as funding was not available.

2.3 Overall investigation programme

2.3.1 Introduction

As shown in Figure 1, it was proposed to carry out the geotechnical testing across eight initial sites, which would be extended to eleven as funding was obtained. To ensure continuity in the evaluation of results, it was proposed that there would be a minimum overlap between Stages I and II of at least one season.

2.3.2 Seasonal testing

As indicated in Figure 1, the seasonal testing proposed was in two levels:

(a) Basic – one borehole at each site (per season) to determine water content, soil suction and soil properties

(b) Advanced – two boreholes at each site (per season) to determine water content, soil suction and soil properties to provide more robust results for each site.

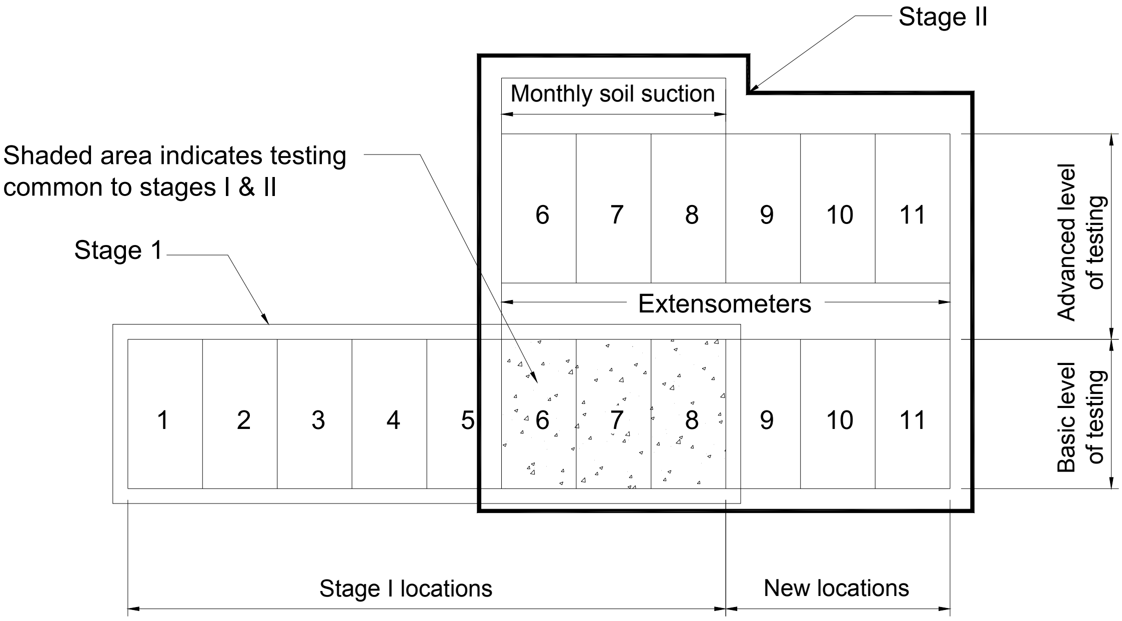


Figure 1: Correlation between Stage I and II of Expansive soils research

2.3.3 Monthly testing

It was proposed that monthly soil suction testing should be carried out, at three locations, to provide an indication of the trend of soil suction change over a 14 month period.

The monthly testing aimed to provide a guide for extrapolation of the measured results to provide data for the climatic extremes.

2.4 Extensometers

Extensometers were to be installed to a depth of around 3m and used to determine the depth at which no soil movement was experienced. The soil movements were to be measured monthly, to provide a soil movement profile at each site.

The extensometer data was intended to provide a correlation between the calculated soil movement from the soil suction readings and the actual movements measured on site. It was proposed that a minimum of two extensometers be installed at each site to provide sufficient data to carry out the correlation.

2.5 Building damage survey

A building damage survey was proposed to assess the performance, over time, of existing buildings built in accordance with NZS 3604. This survey was aimed at assessing the condition of five buildings in the vicinity of each test site, based on:

(a) A review of building records and drawings held by the property owner or territorial local authority

(b) An assessment of damage to the building structure

(c) A survey to assess uniformity of level of concrete floors.

The following report relates to the Stage I investigation.

3.0 LITERATURE REVIEW

A literature review has been undertaken to ensure that previous research projects and experiences from both New Zealand and overseas were taken into account when formulating the aims and methods of this research project.

The published papers and other references that have been reviewed are presented in the bibliography in Appendix A.

4.0 NEW ZEALAND AND RELATED STANDARDS

4.1 Introduction

NZS 3604:1999 "Timber Framed Buildings" was introduced in June 1999. From June 1999 to May 2000 both NZS 3604:1999 and its predecessor, NZS 3604:1990 "Code of Practice for Light Timber Framed Buildings not requiring specific design", were both accepted as design and construction standards, ie there was an overlap period.

The 1990 Standard specified minimum foundation embedment depths to mitigate soil expansivity effects. The 1999 Standard removed the specified minimum foundation embedment depths and introduced dependency on the processes and requirements of the Australian Standard AS 2870:1996 "Residential Slabs and Footings – Construction".

The requirements and processes of the Australian Standard have evolved over time and have included the development of a considerable information base of soil properties and performance, and methodologies that reflect this data. A similar data base has yet to be developed for New Zealand soils.

Whilst not referred to in either the New Zealand or the Australian Standards, the American Association of State Highway and Transportation Officials (AASHTO) have also developed a soil expansivity test method.

4.2 NZS 3604:1990 – Code of Practice for Light Timber Framed Buildings not requiring specific design

NZS 3604:1990 is a prescriptive standard for light timber framed buildings and is used by designers and builders for the types of construction defined within the standard where generic solutions can be applied. The standard provides for specific design beyond the limitations of the generic solutions.

NZS 3604:1990 refers to buildings on expansive soils in Section 3.2.2 "Expansive Clay" which provided for the following criteria for assessment:

"3.2.2.1
For the purpose of 3.3.2(b) expansive clay shall be assumed to be present in the soil supporting the foundations unless:

(a) Reasonable enquiry does not reveal any incidence of major cracks in dry weather on the building site itself or in the surrounding locality;

(b) The locality has not been identified as an area where expansive clay is likely to be found;

(c) Excavation for foundations does not reveal plastic clay."

Section 3.3.2 of the standard then required that foundations in expansive clays be founded at a minimum depth of 450mm below the cleared ground level and all other foundations (not into rock) be founded at a minimum depth of 300mm. It included a comment: "The cleared ground level is used as the depth datum because this level is not usually altered by future landscaping, thus retaining the lateral support of the building".

In July 1992, Amendment 1 was issued which, among other things, removed Sections 3.2.2 and 3.2.3 – the definition of expansive and plastic clays – from NZS 3604:1990, and revised Section 3.1.1 so that the foundation provisions of the Standard only applied to foundations supported on "good ground", which, with respect to expansive soils, excluded:

"(b) Expansive soils being those that have a liquid limit of more than 50% when tested in accordance with NZS 4402 Test 2.2, and a linear shrinkage of more than 15% when tested in accordance with NZS 4402 Test 2.6."

However, Section 3.3.2 of the Standard was retained, which provided for a minimum founding depth of 450mm in expansive clay.

Notwithstanding the 1992 amendment, it appears that geotechnical practitioners generally continued to rely on the original provisions of Sections 3.2.2 and 3.2.3 of NZS 3604:1990 until the introduction of NZS 3604:1999.

4.3 NZS 3604:1999 – Timber Framed Buildings

The 1999 revision of NZS 3604 introduced a provision in Section 1.1.2 that buildings designed to the standard were required to be founded on "good ground", which is defined in Section 1.3 as:

"Any soil or rock capable of permanently withstanding an ultimate bearing capacity of 300 kPa (ie an allowable bearing of 100 kPa using a safety factor of 3.0), but excludes:

(a) Potentially compressible ground such as top soil, soft soils such as clay which can be moulded easily in the fingers, and uncompacted loose gravel which contains obvious voids;

(b) Expansive soils being those that have a liquid limit of more than 50% when tested from the liquid limit in accordance with NZS 4402 Test 2.2, and a linear shrinkage of more than 15% when tested from the liquid limit in accordance with NZS 4402 Test 2.6, and

(c) Any ground which could foreseeably experience movement of 25mm or greater for any reason including one or a combination of:
land instability, ground creep, subsidence, seasonal swelling and shrinking, frost heave, changing ground water level, erosion, dissolution of soil in water, and effects of tree roots."

In circumstances where expansive soils are encountered the designer is referred to Section 17 "Expansive Soils" which in turn refers the designer to AS 2870 for classification of the soil into expansivity Classes S, M, H or E and for the methods to be used in the design of the footings.

4.4 AS 2870:1996 – Residential Slabs and Footings - Construction

The preface to AS 2870 states:

"…the purpose of this Standard is to establish performance requirements and specific designs for footing systems for foundation conditions commonly found in Australia and to provide guidance on the design of footing systems by engineering principles".

AS 2870 leads the designer through a process of site classification, standard designs, design by engineering principles, detailing and construction requirements. These are discussed in subsequent sections of this report.

It is relevant to note that AS 2870 does not contain prescriptive references to soil parameters, such as Atterberg limits or linear shrinkage, as a means of determining whether a soil is expansive, or the degree of expansivity.

4.5 AASHTO Designation: T – 258-81 – Standard Method of Test for Determining Expansive Soils

The American Association of State Highway and Transportation Officials (AASHTO) prescribes a method to detect whether a soil is expansive and to predict the amount of swell. This is done by relating the Atterberg limits of the soil to the natural soil suction, at the time of construction, as shown in Table 1.

Table 1 : AASHTO guidelines on assessing expansive soils (based on AASHTO T 258-81 Table 1)

Degree of Expansivity	**Liquid limit %**	**Plasticity index %**	**Soil suction τ_{nat} (tsf)**	
			kPa	**pF [1]**
Low	< 50	< 25	< 144	< 3.17
Marginal	50 - 60	25 – 35	144 – 383	3.17 – 3.59
High	> 60	> 35	> 383	> 3.59

Note 1: The authors have included the translation of soil suction units from kPa to pF to provide data that is comparable with the findings of this report. The conversion is based on Equation 1, taken from Clause C2.2.3(a) of the Commentary to AS 2870.

$$u(pF) = 1.01 + \log_{10} [u\ (kPa)] \qquad \textbf{Equation 1}$$

5.0 SITE CLASSIFICATION UNDER AS 2870

5.1 Introduction

AS 2870:1996 provides for the classification of sites in terms of soil expansivity based on:

(a) Visual inspection of the soil profile and the use of existing knowledge of the performance of existing residential footing systems within the surrounding region which are not less than 10 years old on similar soil profiles, or

(b) Estimation of the characteristic surface movement (y_s). The dimension y_s relates to the ground surface movement that occurs as the moisture conditions of the soil profile changes from wet to dry design conditions. The estimation of y_s requires determination of the design soil moisture conditions and the soil shrinkage index.

Very little historic data is held for Auckland soils in terms of their expansivity. The application of the procedures of AS 2870 to Auckland conditions therefore requires that the designer estimate the characteristic surface movement in order to classify the expansivity of a site for foundation design purposes, as shown in Table 2.

Table 2 : Classification by characteristic surface movement (from Table 2.3, AS 2870:1996)

Characteristic surface movement	Classification of site
0 mm < $y_s \leq$ 20mm	S – Slightly reactive
20 mm < $y_s \leq$ 40mm	M – Moderately reactive
40 mm < $y_s \leq$ 70mm	H – Highly reactive
70 mm < y_s	E – Extremely reactive

The design parameters that are required for determination of the characteristic surface movement (y_s) are addressed in the following sections 5.2 to 5.4.

5.2 Calculation of characteristic surface movement

The commentary to AS 2870 defines the characteristic surface movement (y_s) as "the characteristic value that has a 5 percent chance of being exceeded in the life of the house which may be taken as 50 years". The parameter y_s is calculated using Equation 2, with Δu and H_s being determined from the soil suction change profiles shown on Table 2.4 and Figure 2.1 of AS 2870. Table 2.4 and Figure 2.1 are also presented as Figure 2 and Table 3 of this report.

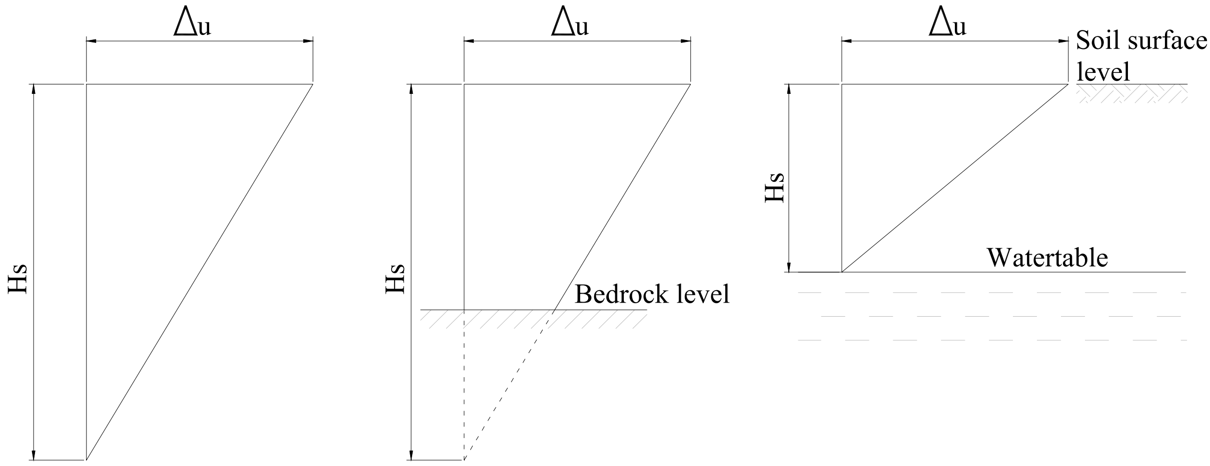
$$y_s = \frac{1}{100}\int_0^{H_s} I_{pt}\Delta u \Delta h \qquad \textbf{Equation 2}$$

where

y_s	=	design characteristic surface movement (mm)
I_{pt}	=	instability index (%)
Δu	=	suction change at depth (h) from the surface (pF)
Δh	=	thickness of soil layer under consideration (m)
H_s	=	depth below which no moisture change occurs (m)

A worked example of Equation 2 is provided in the Australian Handbook HB28:1997 "The Design of Residential Slabs and Footings".

In AS 2870, the design soil moisture conditions are expressed in terms of soil suction, u, which has units of pF. When a soil is saturated, it has a relatively low suction value of 3.2 pF or less, which increases to approximately 4.2 pF when the soil dries to the wilting point of vegetation, where sunflowers in pots are commonly used as the test plant, as discussed in Section 2.16.4 of HB28, with the permanent wilting point being defined as the moisture in a soil when plants in pots start to wilt and not recover.



(a) Consistent soil case (b) Effect of bedrock (c) Effect of ground water

Figure 2: Effect of bedrock or watertable on design soils suction change profiles
(from AS 2870:1996 – Figure 2.1)

Recommended soil suction change values, Δu and depth of design soil suction change, H_s for various locations in Australia are given in Table 3.

Table 3 : Recommended soil suction change profiles for various locations (from Table 2.4 of AS 2870)

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

5.3 Soil suction change profile

Soil suction is made up of two components:

(a) Matrix suction, due to the capillary action between soil particles

(b) Osmotic suction, due to the water-attracting action of salts.

The method for determining soil suction referenced in AS 2870, is detailed in AS 1289 Test Method 2.2.1-1998: Soil Moisture Content Test – Determination of the Total Suction of a Soil – Standard Method. This method involves the laboratory determination of the relative humidity of a small air space in equilibrium with a sealed soil sample, by measurement of the dewpoint temperature of a thermocouple. This method was followed for the study reported herein except that the specified Wescor HR33T microvoltmeter has been substituted with a Soil Mechanics Instrumentation (SMI) transistor psychrometer which, in Australia, is considered a suitable equivalent. In principle, the SMI transistor psychrometer is an electronic wet and dry bulb thermometer, in which a "wet" and "dry" transistor probe is used instead of "wet" and "dry" thermometer bulbs to measure the relative humidity of the air space in equilibrium with a soil sample. The temperature depression of the "wet" transistor, which holds a standard sized water drop, is measured and amplified within the probe. The relationship between relative humidity and soil suction is used to determine the soil suction. The transistor psychrometer improves on the thermistor or thermocouple psychrometer and other forms of suction measuring equipment in that it has a larger range (3.0 pF to 5.0 pF or 100 kPa to 10,000 kPa), faster response and is compatible with modem data logging facilities.

Soil suction readings from soil samples generally fall between 3.2 pF (wet) and 5 pF (dry), with higher readings in the order of 6.5 to 6.9 pF applying to oven dried soil samples.

Table 2.4 of AS 2870, presented as Table 3 of this report, provides recommended soil suction change profiles for various Australian locations, in terms of values of change in soil suction (Δu) at the ground surface, and depth of design suction change (H_s). Figure 2.1 of AS 2870, presented as Figure 2 of this report, shows triangular design suction change profiles, in terms of Δu and H_s. The triangular profiles are based on the assumption that Δu decreases linearly with increasing depth below the ground surface, becoming zero at a depth of H_s.

In the case of bedrock being encountered within the depth of H_s, the design profile is truncated to become trapezoidal.

Note 3 of Clause 2.2.3 of AS 2870 states that "The designer may extrapolate to other areas if due consideration is given to the climate and soil fabric. Alternatively, published values of H_s based on consideration of regional Thornthwaite moisture indices using the general principles in Appendix D [of AS 2870] and based on at least 20 years of climate data, may be used".

Fityus et al (1998), Fox (2000) and Smith (1992) have proposed that the Thornthwaite Moisture Index (TMI) be used to determine the depth of soil moisture change, H_s, for the purpose of site classification in terms of AS 2870.

The TMI is an aridity climate parameter. Fityus et al (1998) analysed the developments that the formulae for the TMI have undergone since first published by Thornthwaite in 1948, resulting in Equation 3. This equation has been used by NIWA to calculate the TMI for Auckland Airport for a hypothetical soil suction profile with a water storage of 100mm, where "water storage" is the depth of water available for plant use and ranges, for the purposes of the model, from field capacity to permanent wilting point and is nominally in the top 1000 to 1200mm of soil depth.

$$TMI = 100\left(\frac{P}{PE} - 1\right) \qquad \textbf{Equation 3}$$

where

P = annual precipitation at a site (mm)

PE = net potential for evapotranspiration at a site (mm)

The proposed correlation between TMI and H_s from Fityus et al (1998) is shown in Table 4.

Table 4 : Depth of moisture change based on TMI values

Climatic classification	Thornthwaite moisture index (TMI)	Depth of moisture change (H_s) m
Wet coastal/Alpine	>40	1.5
Wet temperate	10 to 40	1.8 to 1.5
Temperate	-5 to 10	2.3 to 1.8
Dry temperate	-25 to –5	3.0 to 2.3
Semi-arid	<-25	3.0

The correlation shown in Table 4 of this report is the same as that proposed in Tables D1 and D2 in Appendix D of AS 2870 except that a range of H_s values is proposed by Fityus et al (1998) whereas specific values are proposed by AS 2870.

Based on the TMI value of +50 for Auckland Airport, ie the value provided by NIWA, it is apparent that Auckland falls into the wet coastal/alpine climatic category of Table 4 (TMI >40), for which an H_s value of 1.5m is given.

5.4 Instability index

5.4.1 General

The instability index (I_{pt}) is defined in Appendix F of AS 2870 *"as the percent vertical strain per unit change in suction [in terms of pF], taking into account the expected design values of:*

(a) applied stress,

(b) degree of lateral restraint, and

(c) suction range".

The instability index (I_{pt}) is derived from the shrinkage index (I_{ps}) which may be determined from shrink-swell, loaded shrinkage or core shrinkage tests, in accordance with AS 1289 Test Methods 7.1.1, 7.1.2 or 7.1.3 respectively. In the case of this study, I_{ps} is a generic notation and has been determined using the core shrinkage method detailed in AS 1289 Test Method 7.1.3 – 1998: Soil Reactivity tests – Determination of the shrinkage index of a soil – Core shrinkage index, where the shrinkage index is referred to as I_{cs}.

The core shrinkage index method was chosen over the other two permissible options (shrink-swell index and loaded shrinkage index), following "personal communications" with Australian consultants, which indicated that the core shrinkage index was the more commercially viable test and therefore the more likely one to be adopted within the geotechnical testing industry of New Zealand.

However, as discussed in Cameron (1989), all methods of estimating the instability index are known to have a degree of inaccuracy. For this reason, the commentary to AS 2870 recommends that the calculated y_s value is rounded up to the nearest 5mm.

Grayson (2000) states that "normally, there is some clay in the topsoil on a site. The reactivity of the topsoil is rarely tested, but is typically assumed as approximately 50% of the reactivity of the underlying clays." This assumption has been adopted in this report.

5.4.2 Calculation of instability index

The relationship between I_{pt} and I_{ps} given in AS 2870 is as follows:

$$I_{pt} = \alpha \, I_{ps} \qquad \textbf{Equation 4}$$

where

α is a constraint effect coefficient and is taken as follows:

= 1.0 in the cracked zone (unrestrained), and

= 2.0 – z/5 in the uncracked zone (restrained), where

z is the depth below the finished ground surface, m

5.4.3 Determining cracked zone depth

The depth of the cracked zone "refers to the depth in which predominantly vertical shrinkage cracks exist seasonally" (AS 2870: 1996 and HB28:1997). AS 2870 provides values of cracked depths as shown in Table 5.

Table 5 : Examples of cracked zone depths (from HB28-1997 p20)

Region	Depth of cracked zone
Adelaide and Melbourne	$0.75\ H_s$
Sydney and Newcastle/Gosford	$0.5\ H_s$
Brisbane/Ipswich	$0.5\ H_s$

Cracked zones are incorporated into Equation 4, for I_{pt}, through the α value, which allows the designer to consider the cracked depth of a soil to be laterally unrestrained.

At the time of this report, insufficient data exists to allow crack depths to be determined for Auckland's soils so as to allow the determination of parameters corresponding to those shown in Table 5. The investigation of crack depths for soils within the Auckland region is beyond the scope of this study, but comments are given in the following sections on how appropriate allowances might be made.

Pender (2001) notes that "excavations in Auckland clays reveal that the upper part of the soil profile, up to depths of a metre or so but usually less, is fissured … One possible explanation for the fissures is the cracking of the ground surface that occurs in the summer". A crack depth of 1.0m would therefore correspond to $0.5H_s$, if H_s is taken as 2.0m as proposed in the following Section 10.0.

In the absence of any other information, it is considered that a crack depth of up to 1.0m is reasonable for the purposes of this study.

For the purposes of this study a cracked zone of $0.5H_s$ has therefore been adopted, which is the same as recommended for Sydney and Newcastle in HB-28.

5.5 Other relevant considerations relating to AS 2870

5.5.1 General

The foregoing sections of this report address the composition and framework of the process outlined in AS 2870 for the classification of a site.

The following sections of this report collate various other considerations that define the applications of AS 2870 to the site classification process and comments on the interpretation of some of the requirements, as determined from AS 2870, AS 2870 Supplement 1 and HB 28.

5.5.2 Application

AS 2870 requires that all sites on which slabs and footings are to be constructed for residential dwellings, be classified in accordance with the process set out in the Standard.

The sites are required to be classified as Class A, S, M, H, E or P.

Class A sites are defined as most sand and rock sites with little or no ground movements from moisture change. Sites determined to be reactive are classified as slightly (S), moderately (M), highly (H) or extremely (E) as discussed in the foregoing.

Those sites that incorporate ground conditions that cannot be classified within the definitions for Classes A to E, are classified as Class P sites. A classification of Class P does not signify any particular severity of problem, but rather that the site is disqualified from the criteria for the other classes and therefore requires special considerations using engineering principles. Class P sites would include, for example, soft soils, landslips, subsidence areas etc.

Filled sites may be classified as any of Classes A to P.

5.5.3 Foundation performance considerations

The underlying philosophy of AS 2870 is that footings designed and constructed on a "normal" site in accordance with the requirements of the Standard, are expected to have a low risk of damage.

A "normal" site is one which is:

"*(a)* *not subject to abnormal moisture conditions, and*

(b) *maintained such that the original site classification remains valid and abnormal moisture conditions do not develop."*

A "normal" site is further described as one "where foundation moisture variations are caused by seasonal and climatic changes, effect of the building and subdivision, and normal garden conditions without abnormal moisture conditions."

On sites where "abnormal moisture conditions" apply, footings are expected to have a higher probability of damage. Examples of "abnormal moisture conditions" are given as:

"*(a)* *recent removal of an existing building or structure likely to have significantly modified the soil moisture conditions under the proposed plan of the building;*

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

(c) *recent removal of large trees prior to construction;*

(d) *growth of trees too close to a footing;*

(e) *excessive or irregular watering of gardens adjacent to the house;*

(f) *lack of maintenance of site drainage;*

(g) *failure to repair plumbing leaks."*

Guidance and advice is given in Appendix B of the Standard for the requirements for the maintenance of a "normal" site, and which relates to:

(a) Drainage or wetting of the site

(b) Positioning and operation of gardens adjacent to a house

(c) Restrictions on the planting of trees near the foundations of a house or a neighbouring house

(d) Repair of leaks in plumbing, stormwater and sewerage systems.

The recommendations of the Standard were developed from research and experience in the design and performance of house footings and slabs.

The commentary to AS 2870 notes that:

(a) "*The current costs of failure are modest compared with the cost of conservative design ... Expectations of performance of footing systems on reactive sites depends upon the adopted standard of post-construction maintenance"*

(b) "*To avoid extreme moisture conditions it is essential that owners become aware of their responsibility to care for and adequately maintain a reactive clay site."*

5.5.4 Site Classification

AS2870 requires that "natural sites" be classified as to the expected extent of soil movement and the depth to which the movement extends. It defines a "natural site" as a "site which has not been subject to cut or fill".

For other than sites classified as Class P sites, the Standard requires that site classification *"shall include one or more of the following methods:*

(a) *Identification of the soil profile and either:*

   (i) *Established data on the performance of houses on the soil profile; or*
   (ii) *Interpretation of the current performance of existing buildings on the soil profile.*

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

Notwithstanding the foregoing, the Standard describes the properties of the foundation by one parameter, y_s, also described as "the expected free surface movement". This is the vertical movement range expected during the life of a house from a reasonable estimate of dry conditions to a similar estimate of wet conditions, and does not take into account the moderating effect of the footing system.

The effects of trees, poor site drainage, leaking plumbing and exceptional moisture induced movements are not taken into account in the calculation of y_s.

As discussed in the foregoing Section 5.5.2, abnormal site environment factors lead to a classification of Class P. For a reactive clay site the classification is S, M, H or E, based on comparison of numerical values calculated for y_s. The commentary to AS 2870 advises that these numerical values "should not be over emphasised. Of equal importance, although less definite, is classification by existing house performance or by soil profile identification."

The commentary also notes that to identify accurately the reactivity of a clay site by means of tests on samples through the soil profile is too complex and expensive to be used routinely on individual house sites.

The observation is made that:

"Overall estimates of the range of potential for movements in a whole area based on many tests are a more reliable guide to design at a site than limited testing on the individual site"

It is in this context and from the basis of development of the recommendations of AS 2870 from research and experience in the design and performance of house footings and slabs, that Appendix D of the Standard provides a ready guide to the expected level of site classification for the principal areas of Melbourne and environs, Victoria, Sydney and Adelaide.

The data to provide such a generic appreciation for the principal areas of New Zealand, and more specifically Auckland, either does not exist or has not been collected.

5.5.5 Classification of Filled Sites

A filled site is required by AS 2870 to be classified as Class P, except where the provisions of the Standard allow another classification. Differentiation is made between whether the fill is "controlled", ie engineered fill or "uncontrolled", ie non-engineered fill, as follows:

(a) Controlled fill (engineered fill)

   (i) Shallow fill – the classification is required to be the same as the natural site prior to filling where the depth of fill is:
      (a) not greater than 0.8m for sand
      (b) not greater than 0.4m for other materials

(ii) Deep fill

(a) >0.8m depth of sand may lead to a less severe reactive classification

(b) >0.4m depth of other material requires the site to be considered as Class P (ie the classification is subject to specific engineering considerations).

(b) Uncontrolled fill (non-engineered fill)

(i) Shallow fill – unless building foundations are founded on natural soil under the fill, the site is required to be classified as Class P for:

(a) not greater than 0.8m depth of sand

(b) not greater than 0.4m depth of other material

(ii) Deep fill – for fill depths greater than 0.8m for sand and 0.4m for other materials, the site is required to be classified as Class P.

5.5.6 Reclassification of Filled Sites

Subject to the following proviso, AS 2870 provides that Class P controlled fill sites may be reclassified in accordance with engineering principles including consideration of:

(a) Expected moisture movement in the fill and the underlying soils

(b) The depth of the cracked zone.

The proviso is:

"The reclassification shall not be less severe than the natural site classification unless the controlled fill consists of non reactive material and is deeper than one metre or 0.5H_s, whichever is greater" [H_s is the depth of the design soil suction profile].

In addition, the Standard requires that:

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

5.5.7 Classification Parameters

(a) General

The commentary to AS 2870 makes it clear that the reactivity of clay soils cannot be clearly evaluated by tests. Reactive clays are clay soils that shrink as they dry and swell as they wet up. If the movement is significant such clay soils are termed "reactive".

The amount of movement depends on:

(i) The clay minerals present
(ii) The proportion of clay in the soil and in the profile
(iii) The moisture changes and their extent
(iv) Loading
(v) Lateral restraint.

Individual tests for clay reactivity are subject to wide scatter. Thus individual high or low results may often represent testing variations rather than real variations in the overall properties of the site.

It follows that there is no single test that can confidently assess a particular site.

(b) Soil suction

Soil suction is not simple to determine. It is useful in the analyses of reactive clays because it is more strongly a function of the climate and vegetation than it is of soil type.

The distribution of soil suction is approximated in the Standard to be triangular and to generally be conservative, but it is recognised that near surface soil suctions may be underestimated slightly.

The design profile includes the expected influence of the building and the garden and to some extent, droughts. The design soil suction and hence movement are not merely cyclic seasonal values. The effects of very large trees, poor site drainage and long-term plumbing leaks are not included.

(c) Cracked zone

This zone refers to the depth in which predominantly vertical shrinkage cracks exist seasonally.

(d) Characteristic design surface movement

The characteristic design surface movement, y_s, is a hypothetical parameter.

This surface movement is described in HB 28 as *"a relative movement within the site between a low point during a dry time and a high point during a wet period including the effects of site development on the moisture regime. Thus the two extremes occur at different times. It is not simply the extreme range of seasonal movement experience in the field before development"*.

HB 28 further advises that:

"It is important that too much emphasis should not be placed on this method in comparison with other techniques, as the results are rather imprecise"

and that:

"... the use of suction profile and instability index values is the most accurate method of calculation available (but not necessarily the most accurate method of classification)".

As also noted in the foregoing, it is a requirement of AS 2870 that the calculation of y_s assume that the maintenance of the site complies with Appendix B of the Standard.

(e) Practical and implementation considerations

Fundamental to the application of the requirements of AS 2870 is the need to evaluate the classification of a site with regard to the effect of the reactivity of the site soils on a structure.

The commentary to AS 2870 states that:

"It needs to be emphasised that such data [local experimental data] *must be relevant to the definition of y_s, eg data from an open field site subjected to seasonal moisture changes will not be applicable* [without consideration of the effects of site development]*".*

The Standard requires the classification of a site to take into account the effect of site works when these are known at the time of classification. When the effect of site works is not taken into account the Standard requires that the classification be reconsidered if:

"(a) the depth of cut on an S, M, H or E site exceeds 0.5m, or
(b) the depth of fill exceeds the limits [described in the foregoing Section 5.5.5]*".*

AS 2870 further requires that the soil type and site conditions at a building site be inspected at footing excavation stage by the classifier to confirm the soil profile.

Examples of the effect of cut or fill on the classification of a site, arising from either sub divisional or site development earthworks include:

(i) Increase in reactive movement by removal of part or all of a protective non reactive soil layer
(ii) Reactive movements worsened by the addition of clay fill
(iii) Reduction in reactive movement by the addition of an upper profile of sand or non reactive silt.

As noted in HB 28:

"It is difficult to see how a classifier can accurately assess the implication of future fill except by warnings in the fine print attached to the classification that reconsideration of the classification is needed if the fill is not shallow".

6.0 CLIMATE

6.1 Introduction

The expansivity of a soil is determined by the soil mineralogy and its response to the change in soil moisture levels, which are a consequence of climatic changes. Soils that experience little change in soil moisture are, in general, those that experience little seasonal climate change and those with a low shrinkage index.

6.2 Auckland climate information

For the purposes of this study, the National Climate Centre for Monitoring and Prediction (a division of NIWA) was contracted to provide daily rainfall, temperature and Thornthwaite Moisture Indices (TMIs) for the Auckland Airport meteorological station for the 31-year period between 1 January 1972 and 31 March 2003.

6.2.1 Thornthwaite moisture indices (TMIs)

Internationally, TMIs are generally used for the following:

(a) In the United States, using the Post Tension Institute (PTI) method, the TMI is correlated to "design edge distance" which determines the width of a floor slab that is subject to surface movement

(b) In Australia, the TMI is used to determine the depth, H_s, below which soils do not experience volume changes, which is then used to determine the site expansivity classification in accordance with AS 2870.

The American method also uses the Atterberg limits to define the clay mineralogy. However HB28:1997 "The Design of Residential Slabs and Footings" states that the American method has poor correlation with movements measured in Australia.

In this investigation, the TMIs provided by NIWA have been used to compare the Auckland climate with that at Australian locations for which the climate related soil expansivity factors are known. It has been found that the TMI varies greatly between the regions of Australia.

The correlation between climatic zone and TMI, as proposed in Appendix D of AS 2870, is reproduced in Table 4.

As noted in Section 5.3, NIWA calculated an annual TMI (over 31 years of record) for Auckland Airport of approximately +50, corresponding to the Wet Coastal/Alpine classification in Table 4.

As shown in Table 6, Auckland Airport has an annual average rainfall of 1240mm. The average annual rainfall for the eight test sites (Sites A to H) falls within the range of 1200 mm to 1400 mm, except for Site E which falls within the range of 1400 mm to 1600 mm.

It is therefore apparent that Auckland Airport rainfall corresponds to the lower end of the range of annual rainfall for the Auckland region. The TMI for Auckland Airport is therefore likely to be conservatively low with respect to the eight test sites.

6.2.2 Water balance

NIWA has identified that daily TMI values for the Auckland region are likely to have a high degree of variability and has recommended that a better indication of daily soil moisture is the running water balance. The "water balance" is represented by "precipitation less evapotranspiration less deep percolation" which, along with other meteorological data, yields a "soil moisture deficit" for the particular location.

The soil moisture deficit data (shown in Figure 3) has been used to:

(a) Estimate the time of year that testing should take place to obtain the soil suction profiles corresponding to the wettest and driest periods during the 2002-3 year

(b) Relate the soil moisture conditions at the time and location of sampling to "extreme wet winter" and "drought" conditions to allow the measured soil suction values to be extrapolated to provide estimated values corresponding to "wet winter" and "drought" conditions at the ground surface, as required by AS 2870. The winter 2002 samples corresponded to saturated conditions, and have therefore been assumed to be representative of wet winter conditions.

AS 2870 refers to design wet and dry conditions. For the purposes of this report, design dry conditions have been assumed to be "drought" conditions.

The "water balance" is expressed as a soil moisture surplus or deficit. A zero soil water balance indicates that the soil is saturated. Positive values indicate runoff (ie a surplus) and negative values indicate a deficit.

NIWA advised that soil moisture deficits (ie negative values) greater than 90 indicate drought conditions and deficits less than about 10 mm are likely to be saturated with some runoff and drainage occurring.

6.3 Climate factors for Australia and Auckland

The expansivity of soils is dependent on soil mineralogy and climate. Relevant climate factors for Auckland and selected Australian cities are summarised in Table 6. Average annual rainfall and temperature data for Auckland and four selected Australian centres are shown on Figure 4.

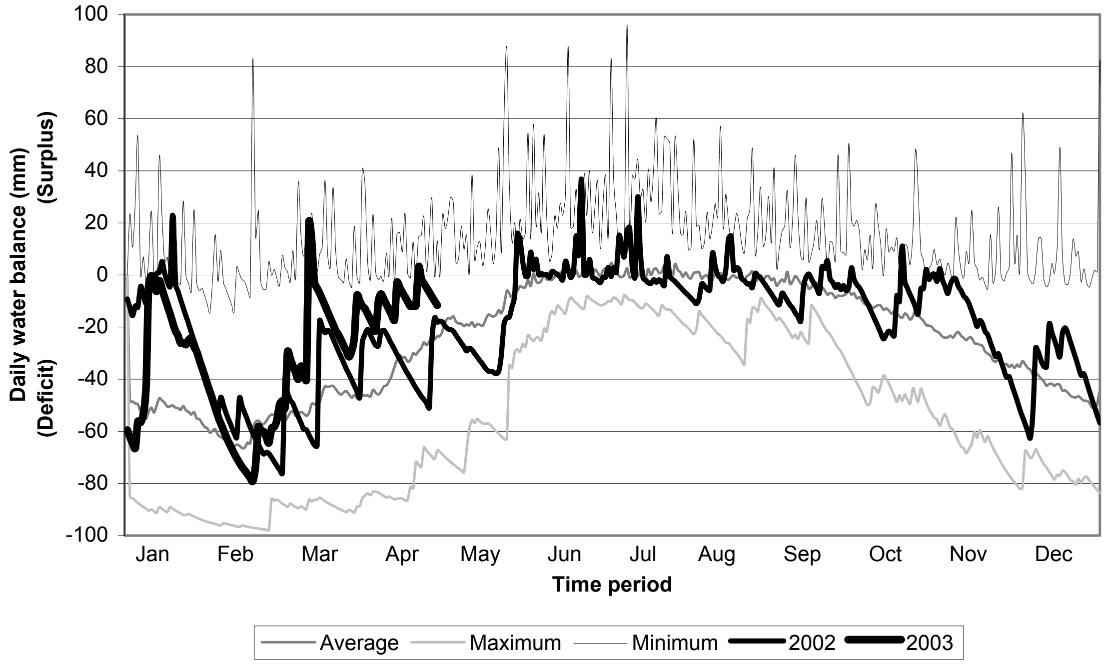


Figure 3: Daily water balance (mm) for Auckland Airport during 1971-2003

Table 6 : Climate Factors
(sourced from www.worldclimate.com, Fityus (1998), HB28 and Appendix D of AS 2870)

City	Average Annual Readings			
	Rainfall (mm)	**Minimum temperature (˚C)**	**Maximum temperature (˚C)**	**Thornthwaite Moisture Index**
Auckland	1240	11.3	18.9	+50
Adelaide	516	11.0	22.3	-40
Brisbane	1150	15.6	25.4	+20
Hobart	598	7.8	17.2	+10
Launceston	694	6.1	16.8	+80
Melbourne	656	9.3	19.4	+10
Cape Otway, Victoria	892	10.4	17.2	+40
Newcastle	1143	14.1	21.7	+30 to +40
Sydney	1222	12.9	21.0	+20

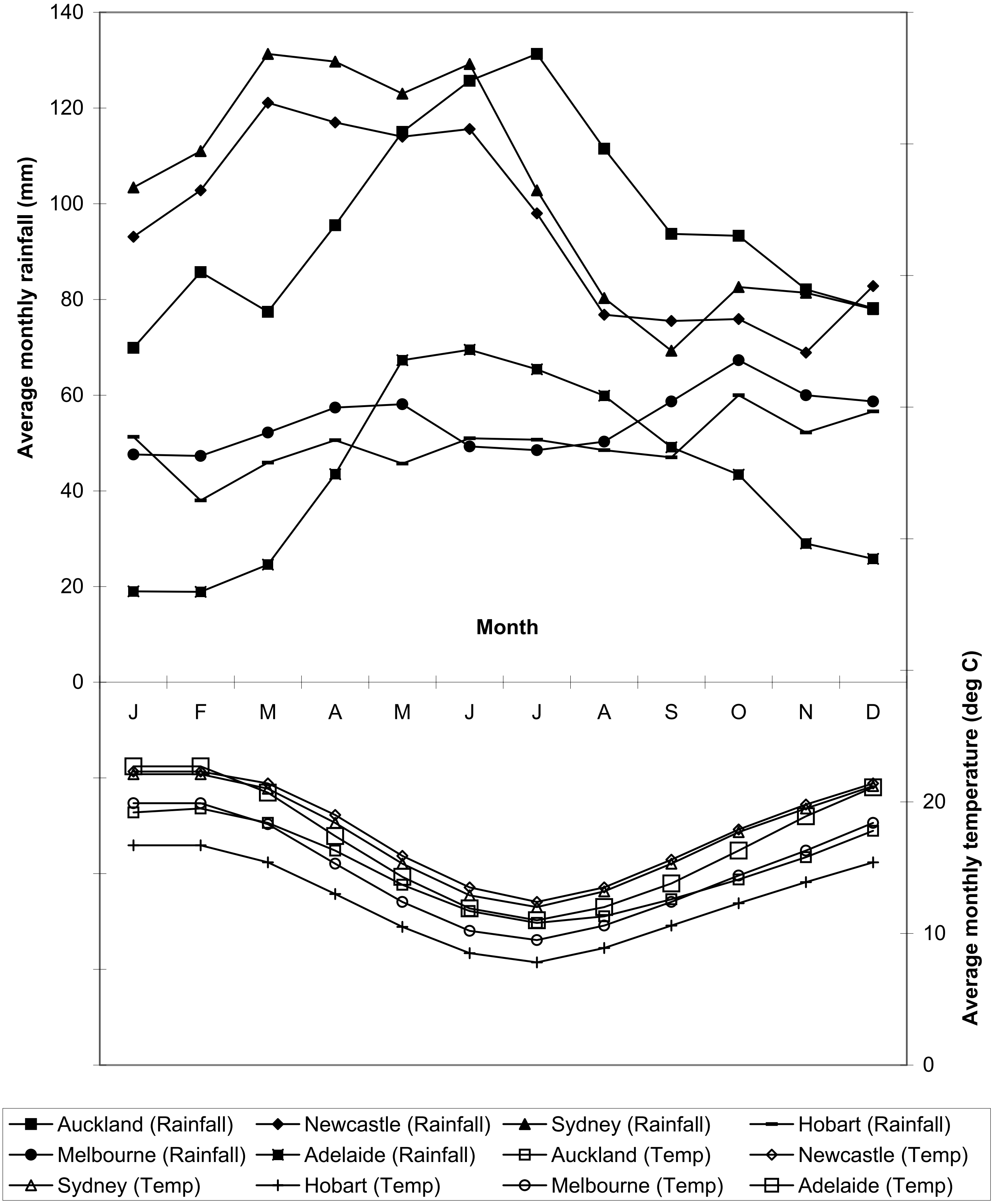


Figure 4: Average monthly rainfall and temperature measurements for Auckland Airport and selected Australian cities (sourced from www.worldclimate.com)

6.4 Comparison of Australian and Auckland climates

From the climate data summarised in Table 6 and Figure 4, it is apparent that:

(a) The TMI for Auckland (+50) is similar to that of Newcastle (+30 to +40) and the Cape Otway area in eastern Victoria (+40), indicating that the respective climates are similar.

(b) Although the Newcastle average annual rainfall is similar to that of Auckland, it is important to note that some of the higher rainfall months in Newcastle are in summer, when potential evapotranspiration is at its highest. This would tend to give lower annual percolation of water into the soil than in Auckland, where the higher rainfall months are in winter. Hence the TMI would be expected to be higher in Auckland. Thus the TMI value of around +50 for Auckland, when compared to the TMI value of +30 to +40 for Newcastle, would seem to be reasonable,

(c) The TMI for Auckland (+50) is significantly higher than that for Sydney and Brisbane (+20), and Melbourne and Hobart (+10), indicating that the Auckland climate is significantly wetter than these Australian centres.

(d) Although the Auckland annual rainfall is similar to that of Sydney, Brisbane and Newcastle, the minimum and maximum temperatures in Auckland are lower. Auckland would therefore be expected to have a lower evapotranspiration rate and consequently a higher TMI value than these Australian cities.

(e) The Auckland annual rainfall is approximately double that for Adelaide, Melbourne, Hobart and Launceston.

(f) Whilst the average annual rainfall for Launceston is approximately half that of Auckland, the average annual minimum and maximum temperatures are significantly lower than Auckland. The TMI for Launceston (+80) is significantly higher than that of Auckland (+50). This data suggests that the Launceston climate is "wetter" than Auckland.

In summary, therefore, the TMI values indicate that within Australia the climates in Newcastle and eastern Victoria are the closest comparisons to that of Auckland, although the Auckland climate is slightly wetter than these two areas.

7.0 MINERALOGY OF SOILS

7.1 General

Soils swell on wetting and shrink on drying, resulting in ground movement. If the ground movement is sufficiently large to affect any structures, the soil is said to be reactive.

One of the factors governing the reaction of a soil to moisture change is the mineralogy of the individual soil types. Some clays, such as smectite, are extremely reactive to moisture change whilst clays with a high kaolinite content are known to be only slightly reactive. AS 2870 and HB-28 both discuss the range of soils found within the main geographical regions of Australia and then incorporate the effects of regional climatic changes to provide the designer with foundation solutions.

This section provides a brief overview of the soils found within both Australia and Auckland, with the aim of ascertaining whether there are direct comparisons between the design factors specified within AS 2870 for the varying Australian soil types and those soils found in Auckland.

7.2 Australian soils

7.2.1 Introduction

The following information in Section 7.2 has been taken from Australian Handbook HB28, which provides a brief summary of the main soil types within each geographical area:

7.2.2 Sydney clays

Sydney clays have been found, in general, to derive from sandstones or shales, although there are a few well-defined areas that are founded on alluvial clays which form deeper deposits than the rest of Sydney and which are highly reactive. As shown in Table 7, the reactivity of all clays, other than alluvial, tends to be related to depth as opposed to mineralogy.

Table 7 : Classification of Sydney clays (from HB28-1997 p29)

Type of clay	Depth of clay	Expansivity class
Alluvial	All depths	H - Highly
Other	< 0.6m	S - Slightly
Other	0.6 - 2.5m	M - Moderately
Other	> 2.5m	H - Highly

The data in Table 7 indicates that any sites in Sydney with clay depths greater than 2.5m are classed as highly expansive (Class H).

7.2.3 Newcastle/Hunter Valley clays

A significant amount of research into soil expansivity in the Newcastle region has recently been undertaken and is on going. There is still insufficient information to provide blanket recommendations of variables to be used in AS 2870 and testing is still recommended in most parts of Newcastle.

The Newcastle area has a more variable geology than that of other Australian cities (eg Sydney). The sedimentary rocks include mudstone, shale, sandstone and conglomerate as well as coal seams. Some of the sedimentary rocks contain thin layers of volcanic ash. The volcanic ash has been said to contain up to 10% smectite, which can have a marked effect on the reactivity of the soil. Most clay sites derive from sandstones and conglomerates, producing mainly Class M sites.

7.2.4 Melbourne clays

Melbourne is generally founded on residual soils weathered in place from the underlying rock. Although most residual soils are classed M, it has been observed that the clays derived from basic igneous lava flows (ie basalts) are more reactive, and are generally classed as H. Further investigation is required for possibly highly reactive limestone and alluvial clays. The climate of Melbourne varies, with the west being significantly drier and the east being more moderate (wet temperate). Testing has shown that the clay sites are generally less reactive in the wetter areas of Melbourne.

7.2.5 Brisbane clays

The founding soils within the Brisbane region vary considerably between sites due to topography and relatively complex geology. The following soils are found within the Brisbane area:

(a) Residual and alluvial soils weathered from basalt are considered highly or extremely reactive

(b) Rhyolitic tuff can be highly reactive but generally not as much as (a)

(c) Black or brown clays found around Ipswich are of high/extreme reactivity

(d) Clays derived from volcanic ash are extremely reactive.

The climate varies greatly in the east-west direction and classification can range from Class A to H and sometimes E. Due to the variability of the clays, tests are still generally carried out to confirm which expansivity classification applies.

7.2.6 Adelaide clays

Adelaide is generally founded on sediments which vary from red-brown earths of moderate to high reactivity to highly reactive black earths and fissured Pleistocene clays (50% illite, >20% kaolinite and <20% smectite) which also display evidence of shrinkage cracking at the surface of the more reactive soils during summer and autumn.

Ground water is generally deep. Leaching of lime layers occurs, which assists in reducing the reactivity of the soil. The Pleistocene clays are considered to be less reactive than the smectite-rich basaltic clays in Melbourne.

In general, the classification of soils within the Adelaide region is based on the opinions of experienced soil classifiers who log bores for each site. The red-brown earth group has been assigned nine typical profiles which correspond to either an M, H or E classification, while the "sites underlain by Pleistocene clays or black earth are generally E [classification] for clay layers greater than two metres [depth]."

7.2.7 Generalisation of Australian soils

The soil types and their expansivity as detailed in the foregoing Sections 7.2.2 to 7.2.6, are summarised in Table 8.

Table 8 : Generalisation of Australian soil classifications

Location	Clay Type	Depth of Clay	Expansivity Class
Sydney	Non-alluvial	< 0.6m	S – Slight
Sydney	Non-alluvial	0.6 - 2.5m	M - Moderate
Melbourne	Non basaltic residual	All depths	M - Moderate
Newcastle	Sandstone/Conglomerate derived	All depths	M - Moderate
Adelaide	Red/brown	All depths	M – Moderate/H – High
Melbourne	Basaltic	All depths	H - High
Sydney	Non-alluvial	> 2.5m	H - High
Sydney	Alluvial	All depths	H - High
Brisbane	Rhyolitic tuff	All depths	H - High
Newcastle	Volcanic ash derived	All depths	H - High
Adelaide	Red/brown	All depths	H – High/E - Extreme
Brisbane	Residual/Alluvial	All depths	H – High/E - Extreme
Brisbane/Ipswich	Black/brown	All depths	H – High/E - Extreme
Brisbane	Volcanic ash derived	All depths	E - Extreme
Adelaide	Pleistocene/Black earth	> 2.0m	E - Extreme
Melbourne	Limestone/Alluvials	All depths	Further investigation necessary

7.3 Auckland soils

7.3.1 Introduction

The following information has been adopted from the handbook accompanying the New Zealand Geological Map, Auckland Urban Area, Sheet R11, scale 1:50,000.

7.3.2 Waipapa Group

The oldest known rocks in the Auckland region are indurated marine sedimentary strata constituting the "greywacke basement" of Late Triassic to Late Jurassic age. The Waipapa Group forms the rolling to steep hills in the Whitford and Brookby districts, in the Hunua Ranges and on Waiheke Island, and comprises indurated sandstone and mudstone.

The Waipapa Group commonly comprises deep weathering profiles, with the surficial soils comprising yellow-brown, sandy and silty clays.

7.3.3 Waitemata Group – East Coast Bays Formation

The Waitemata Group comprises alternating mudstone and lithic sandstone of Miocene age and underlies most of urban Auckland. The East Coast Bays Formation is the dominant member of the Waitemata Group within the Auckland region and forms the conspicuous alternating beds exposed in cliffs and on intertidal platforms around the Waitemata Harbour.

The greater part of the East Coast Bays Formation consists of graded turbidite sandstones alternating with poorly sorted interturbidite mudstones. The residual soils formed on this formation produce greyish white to orange-brown clays. The clay mineralogy of the Waitemata Group residual soils, as indicated by X-ray diffraction, comprises a mixture of kaolinite, illite, and montmorillonite, with kaolinite being more dominant at the ground surface and montmorillonite being more dominant at depth (Harvey *et al.* 1982).

7.3.4 Onerahi Chaos Breccia

The Onerahi Chaos Breccia forms part of the Northland Allochthon, where oceanic crust was thrust above continental crust and tilted to allow the lower Miocene deposits to slide and shear off, followed by sliding and shearing of the upper Cretaceous deposits, resulting in inversion of the normal stratigraphy. The deposits occur both above and below the Waitemata Group sandstones and siltstones of lower Miocene age (Beca Carter, 1980).

The Onerahi Chaos Breccia comprises chaotic, irregularly-bedded rocks that are present near the ground surface over wide areas of North Auckland. The deposit has been associated with several large ground creep movements.

Residual soils formed on the Onerahi Formation mudstone or siltstone are very smooth impervious clays. High montmorillonite contents are associated in areas where ground movement has been encountered.

7.3.5 Tauranga Group

Tauranga Group sediments occur throughout the extensive lowlands mainly south and west of Auckland City, and were deposited in fluvial, lacustrine, estuarine and shallow marine settings from the late Pliocene to late Pleistocene.

(a) Puketoka Formation (tp) - The Puketoka Formation forms the lowlands to the west and south of Auckland City, and comprises undifferentiated, mainly pumiceous, light-grey to orange-brown mud, sand, and gravel formed in terrestrial to estuarine environments.

The deposits typically comprise clay with occasional lenses of sand and peat. The formation is characterised by a high variability in the nature and type of the sediments resulting from the nature of the deposition of the formation.

(b) Rhyolitic Pumice (tpp) - This member of the Puketoka Formation comprises rhyolitic pumice deposits derived from non-welded distal ignimbrites originating in the Taupo volcanic zone and deposited into terrestrial, fluviatile, or shallow marine environments.

Weathering of the rhyolitic pumice deposits results in white clay. Derived from one or more non-welded distal ignimbrites, the deposits are often interbedded with carbonaceous deposits.

7.3.6 Auckland volcanic field - basaltic ash

The Auckland volcanic field comprises basaltic deposits of Pleistocene to Holocene age erupted from numerous small volcanoes within a 360 km^2 area centred on One Tree Hill. The erupted material comprises basaltic lava, scoria, lithic tuff, and ash and lapilli.

The basaltic ash deposits can mantle the terrain up to several kilometres downwind from some of the volcanoes. Owing to the distribution of the multiple volcanoes in the Auckland region, ash deposits can be found over much of the area, particularly in the overlapping volcanoes in Auckland City, and less so in the more isolated volcanoes in Manukau.

The basaltic ash deposits weather to form red-brown sandy clays.

7.4 Comparison of soils from Auckland and selected Australian centres

As discussed in the previous Section 6.0, it is considered that only the Newcastle, and eastern Victoria regions of Australia have similar climatic conditions to Auckland. Comparison of Auckland soils to Australian soils has therefore been limited to the foregoing regions of Australia, that have similar climatic conditions to those in Auckland. It was noted in the foregoing Section 7.2.4 that repeated testing has shown that the clay sites are less reactive in the wetter areas than the drier areas of Melbourne. As Auckland is comparatively wetter than the comparable regions in Australia, Auckland soils may also be less reactive.

It is considered likely that the soils, formed on the sedimentary coal measures (Class M) of Newcastle would be similar to the Waitemata Group residual soils in Auckland.

The clays formed on the sandstones and shales in Sydney (Class S to H) could be comparable to the Waitemata Group residual soils and possibly even the Waipapa Group residual soils in Auckland. It is, however, recognised that the clays formed on the Sydney sandstones are of a lesser thickness and of a more uniform profile than those formed on the Waitemata Group sandstone and mudstone. The alluvial soils in Sydney (Class H) are likely to be similar to the Tauranga Group/Puketoka Formation (tp) alluvial soils of Auckland.

It is possible that the clays formed on the basaltic deposits in eastern Victoria (Class S to H) may be comparable to the basaltic ash deposits in Auckland.

Soils similar to the Onerahi Chaos Breccia and Tauranga Group/Rhyolitic Pumice (tpp) are not found in the Newcastle/Sydney or eastern Victoria regions of Australia.

8.0 FIELD AND LABORATORY TESTING FOR AUCKLAND STUDY REPORT

8.1 Site selection

Fraser Thomas has liased with representatives of five local authorities and two private property owners within the Auckland region to gain access to thirty-three sites. The aim of the site selection process was to provide a geographical spread of test locations throughout Auckland and to identify sites that provide representative soil profiles that commonly underlie sites for residential building development.

An inspection of all thirty-three sites was carried out to assess the suitability with respect to cut/fill locations, access and general stability. From this visual inspection, soil profiles were established by hand auger for some twenty sites. From these, eight sites (coded A through H, as shown in Table 9) were selected as representative test locations for more detailed investigation within this research project.

Table 9 : Test locations in the Auckland region

Site code	Main soil type	Suburb
A	Basaltic ash	Newmarket
B	Basaltic ash/Tauranga Group (tpp)	East Tamaki
C	Waipapa Group	Brookby
D	Tauranga Group (tp)	Manurewa
E	Tauranga Group (tp)	Swanson
F	Waitemata Group	Howick
G	Waitemata Group	Birkenhead
H	Onerahi Chaos Breccia	Red Beach

Note 1: Detailed soil profiles are shown on the borehole logs presented in Appendix B.
Note 2: Topsoil depths vary as shown on the borehole logs.

It is intended that all future reporting will be referenced to the site codes shown in Table 9 (A through H), with only a suburban reference given to provide anonymity to the specific test locations given the potentially sensitive nature of the results.

Whilst no sites have been selected within the Papakura or Franklin District Council boundaries, it has been assumed that Site D would be representative of a large proportion of Papakura and Franklin’s Patumahoe Ward and Site C would be representative of Franklin’s Hunua Ward in the close vicinity to State Highway One.

8.2 Sampling and monitoring

8.2.1 General

Sampling and monitoring at the selected sites was programmed to coincide with the driest periods during the summer of 2002 and 2003 and the wettest period during the winter of 2002, with the aim of capturing soil suction and soil moisture data that would be representative of wet and dry conditions.

It was anticipated that there was only a slim chance that an extreme dry or drought period would occur during the two summer seasons but that it was likely that representative wet conditions would occur during the winter season.

Due to the unpredictable nature of the Auckland weather, it was difficult to programme the summer 2002 and 2003 sampling to coincide with the driest periods. Conversely, it was easier to programme the winter 2002 sampling to coincide with wet winter conditions.

The logs of the hand auger boreholes are presented in Appendix B. The borehole number relates to each of the eight sites, viz Borehole A relates to Site A, Borehole B to Site B etc. The logs relate to the first borehole put down at each site, during summer 2002. The subsequent boreholes put down at each site, in winter 2002 and summer 2003, were located within approximately one metre from the original borehole.

8.2.2 Summer 2002

The "summer 2002 sampling" was carried out between March 19 and 21, 2002. Laboratory testing was carried out between March 27 and May 2, 2002. The sampling comprised:

(a) One hand auger borehole to 3m depth at each site to determine the subsoil profile

(b) Taking five "undisturbed" soil samples at 0.5m depth intervals, between 0.5m and 2.5m depth, with a 60mm diameter thin-walled stainless steel tube driven into the base of each borehole at the required depth using a Scala Penetrometer hammer and rods

(c) Installation of a 32mm diameter PVC standpipe piezometer in each borehole to monitor the groundwater level.

The ends of the tube samples were sealed with molten wax and carefully stored until the samples were extruded and prepared for laboratory testing at the Geomechanics Laboratory at the University of Auckland School of Engineering.

8.2.3 Winter 2002

The "winter 2002 sampling" was carried out between August 15 and September 17, 2002. Laboratory testing was carried out between August 19 and October 2, 2002. The sampling comprised:

(a) One hand auger borehole, at each site to provide five "undisturbed" soil samples at 0.5m depth intervals for laboratory testing as for the foregoing item 8.2.2(b).

(b) Monitoring of groundwater levels in the standpipe piezometers installed during the initial summer 2002 testing.

8.2.4 Summer 2003

The "summer 2003 sampling" was carried out between March 10 and 20, 2003. Laboratory testing was carried out between March 14 and May 1, 2003. The sampling comprised:

(a) One hand auger borehole, at each site to provide five "undisturbed" soil samples at 0.5m depth intervals for laboratory testing as for the foregoing item 8.2.2(b).

(b) Monitoring of groundwater levels in the standpipe piezometers installed during the initial summer 2001/2002 testing.

Climate data obtained from NIWA was then used to confirm the relative "dryness" of the test period relative to the absolute driest period measured at various reference sites in Auckland during the summer of 2001-2002.

8.3 Laboratory testing

The soil laboratory testing listed in Table 10, was undertaken by the Geomechanics Laboratory of the University of Auckland, School of Engineering during the periods discussed in the foregoing Section 8.2.

Table 10 : Laboratory testing

Sample depth (m)	Soil suction [1]	Water content [1]	Core shrinkage [2]	Atterberg limits [2]	Linear shrinkage [3]
0.5	✓	✓	✓	✓	✓
1.0	✓	✓	✓	✓	✓
1.5	✓	✓	✓	✓	✓
2.0	✓	✓	-	-	-
2.5	✓	✓	-	-	-

Note 1: Samples taken in Summer 2002, Winter 2002 and Summer 2003
Note 2: Samples taken in Winter 2002
Note 3: Samples taken in Summer 2003

The Atterberg limits and linear shrinkage tests were undertaken to NZS 4402:1987 New Zealand Standard, Methods of testing soils for civil engineering purposes.

The core shrinkage tests were undertaken to AS 1289 Test Method 7.1.3-1998: Soil reactivity tests – Determination of the shrinkage index of a soil – Core shrinkage index.

The soil suction tests were undertaken to AS 1289 Test Method 2.2.1-1998: Soil moisture content tests – Determination of the total suction of a soil – Standard method, except that the thermocouple psychrometer referred to in the Standard was replaced with a transistor psychrometer. The method is stated in the Standard as being applicable for suctions ranging from 3.2pF to approximately 5pF. The transistor psychrometer is discussed by Woodburn et al (1993) and Woodburn and Lucas (1995).

9.0 RESULTS OF FIELD INVESTIGATION AND LABORATORY TESTING

9.1 Results of field investigation

The soil profiles encountered in the boreholes at Sites A to H are shown on the borehole logs in Appendix B. The soil profiles at each site are summarised in Table 11. The measured groundwater levels are shown in the individual site reports in Appendix C.

Table 11 : Summary of soil types at each test site

Site Code	Suburb	Depth (m)	Soil Unit	Soil Description
A	Newmarket	0.4-1.5 1.5-3.0	Basaltic Ash Basaltic Ash	clayey SILT sandy SILT
B	East Tamaki	0.3-1.1 1.1-2.7	Basaltic Ash Tauranga Group (tpp)	SILT sandy CLAY
C	Brookby	0.3-2.8	Waipapa Group	silty CLAY
D	Manurewa	0.2-3.0	Tauranga Group (tp)	sandy and silty CLAY
E	Swanson	0.2-0.8 0.8-2.6	Tauranga Group (tp) Tauranga Group (tp)	sandy and clayey SILT silty CLAY
F	Howick	0.4-0.7 0.7-2.6	Waitemata Group Waitemata Group	silty CLAY slightly clayey SILT
G	Hillcrest	0.3-2.5	Waitemata Group	CLAY
H	Red Beach	0.1-2.1 2.1-3.0	Onerahi Chaos Breccia Onerahi Chaos Breccia	silty CLAY clayey SILT

9.2 Laboratory test results

9.2.1 General

The results of the soil classification tests (Atterberg limits and linear shrinkage tests) and the core shrinkage tests for Sites A to H are shown in the individual site reports in Appendix C.

9.2.2 Atterberg limits and linear shrinkage

A Casagrande plot of the Atterberg limits test data is shown on Figure 5, which indicates that the soils generally plot below or slightly above the A line and are of high to extremely high plasticity, with liquid limits ranging from 50 to 104. As shown on Figures 7 and 9, the linear shrinkage values of the soils range from 12% to 23%.

9.2.3 Shrinkage index, I_{ps}

The shrinkage index is defined as the percent vertical strain per unit change in soil suction (pF), determined from shrink-swell, loaded shrinkage or core shrinkage tests, in accordance with AS 1289 Test Methods 7.1.1, 7.1.2 or 7.1.3 respectively. As stated in the foregoing Section 5.4.1, the core shrinkage test has been used for this study.

As shown on Figure 6, the shrinkage index for soil samples from Sites A to H ranged from 0.75% to 6.38%, with a mean value of 3.29%. The following observations are noted:

(a) The highest values of shrinkage index were obtained for the Tauranga Group soil samples from Site D, with a mean value of 5.72% for the three samples.

(b) The lowest values of shrinkage index were obtained for the Waitemata Group soil samples from Site F, with a mean value of 1.65% for the three samples.

(c) The range of shrinkage index values for Sites A to H, of 0.75% to 6.38%, is similar to the range for the Newcastle region of less than 1% to 7%, reported by Fityus and Welbourne (1996). Although the shrinkage index values reported by Fityus and Welbourne (1996) relate to shrink-swell tests it is assumed that the values would be equivalent to shrinkage index values determined from core shrinkage tests.

(d) The average shrinkage index value for Sites A to H of 3.29% is similar to the average value of 3.17% for the Newcastle region, reported by Fityus and Welbourne (1996).

9.3 Correlations between shrinkage index and other soil classification parameters

The relationship between the shrinkage index and the soil classification parameters of linear shrinkage, liquid limit and plasticity index has been investigated. The values of shrinkage index for each test site are plotted against the corresponding values of plasticity index, linear shrinkage and liquid limit on Figures 6, 8 and 9 respectively.

It is apparent from Figures 6, 8 and 9 that there is no readily discernable correlation between the shrinkage index and any of the three soil classification parameters. In particular, no correlation is apparent between shrinkage index and linear shrinkage or liquid limit, which indicates that the linear shrinkage and liquid limit values do not provide a reliable measure of soil expansivity.

The lack of correlation is perhaps not unexpected, given that the shrinkage index relates to an "undisturbed" sample while the other classification parameters relate to fully remoulded samples.

It is noted, however, that Cameron (1989) found a "reasonably satisfactory" linear correlation between the shrink-swell index (the shrinkage index determined from a shrink-swell test) and linear shrinkage for several soils from three Australian states, although the scatter of data points reported by Cameron (1987) would preclude confident implementation of the correlation.

As stated in Section 4.3, NZS 3604 defines expansive soils as "being those that have a liquid limit of more than 50%…. *and* a linear shrinkage of more than 15%…..".

As shown on Figure 6, all the soils with a shrinkage index of greater than 4% had a liquid limit of greater than 50% and are therefore considered to be expansive in terms of the definitions in NZS 3604. However, several soils with relatively low shrinkage index values of less than 3% also had liquid limit values greater than 50%.

If, as indicated on Figure 7, the limiting linear shrinkage value of 15% and liquid limit value of 50%, stated in NZS 3604, are considered together as required by NZS 3604, it is apparent that only one of the soil samples, from Site G (Waitemata Group), would be classified as not being expansive under NZS 3604.

As shown on Figure 9, some soils with relatively high shrinkage index values of greater than 4% have a linear shrinkage value of less than 15%. The limiting linear shrinkage value of 15% given in NZS 3604, taken on its own, is not therefore considered to be a reliable indicator for expansive soil.

Based on the foregoing, it is concluded that almost all Auckland silt and clay soils are expansive in terms of NZS 3604 and that the linear shrinkage, liquid limit and plasticity index do not provide a reliable indication of the degree of soil expansivity expected in such soils.

The findings of this study, for the range of soils investigated, are consistent with the conclusion reached in Cameron (1989), that no simplistic and reliable relationship exists between the shrinkage index and other soil properties.

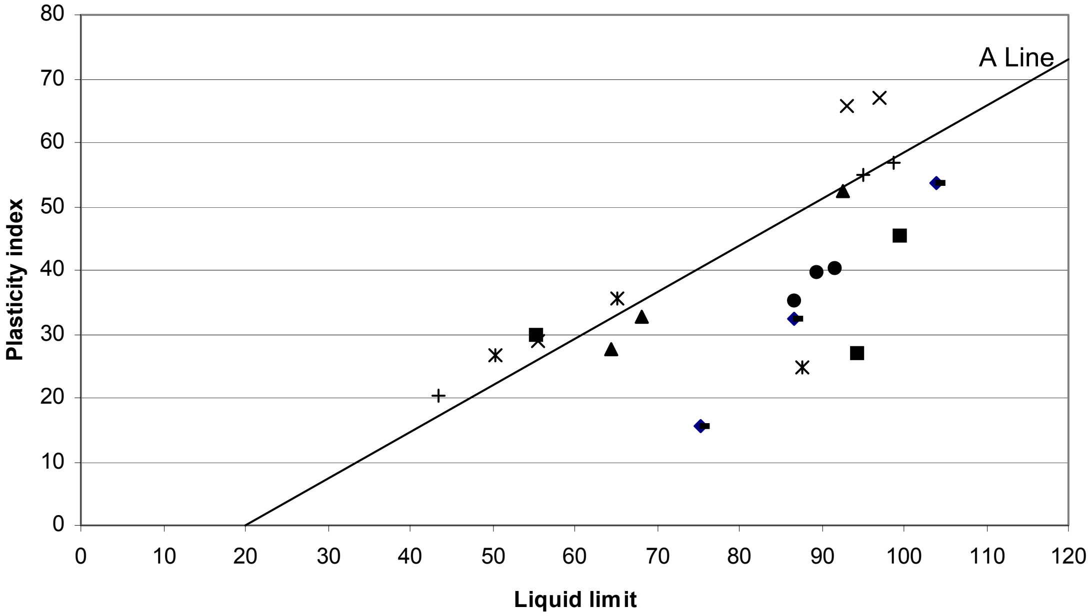


Figure 5: Casagrande plot of plasticity index against liquid limit

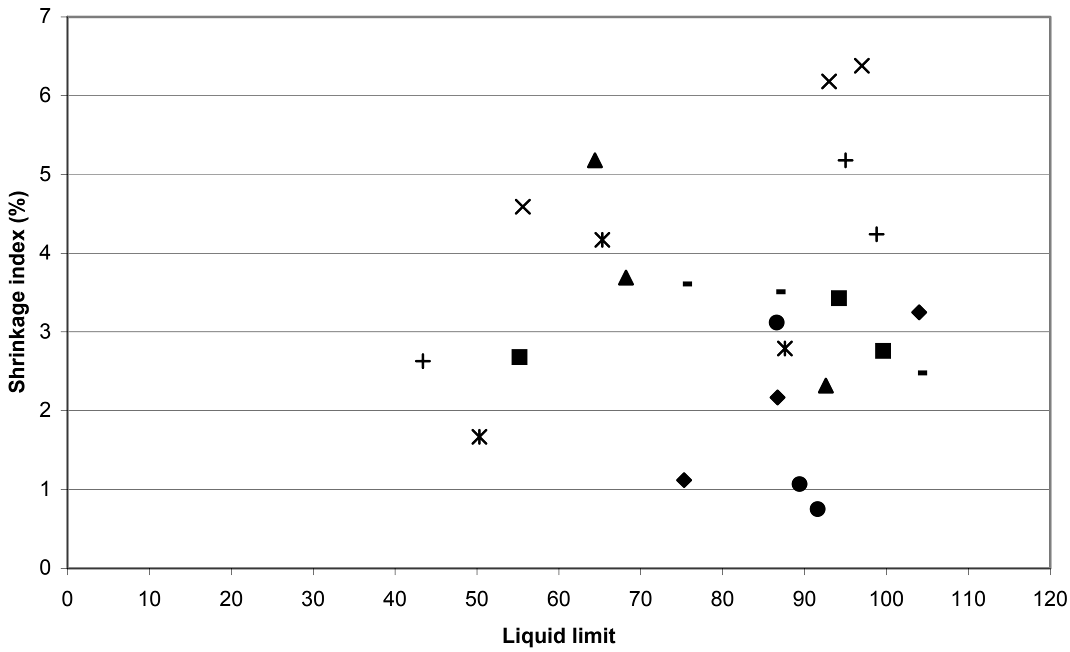


Figure 6: Shrinkage index against liquid limit

◆ A - Basaltic Ash ▲ C - Waipapa ✳ E - Tauranga Group (tp) + G - Waitemata Group

■ B - Basaltic Ash/Tauranga Group (tpp) × D - Tauranga Group (tp) ● F - Waitemata Group - H - Onerahi Chaos Breccia

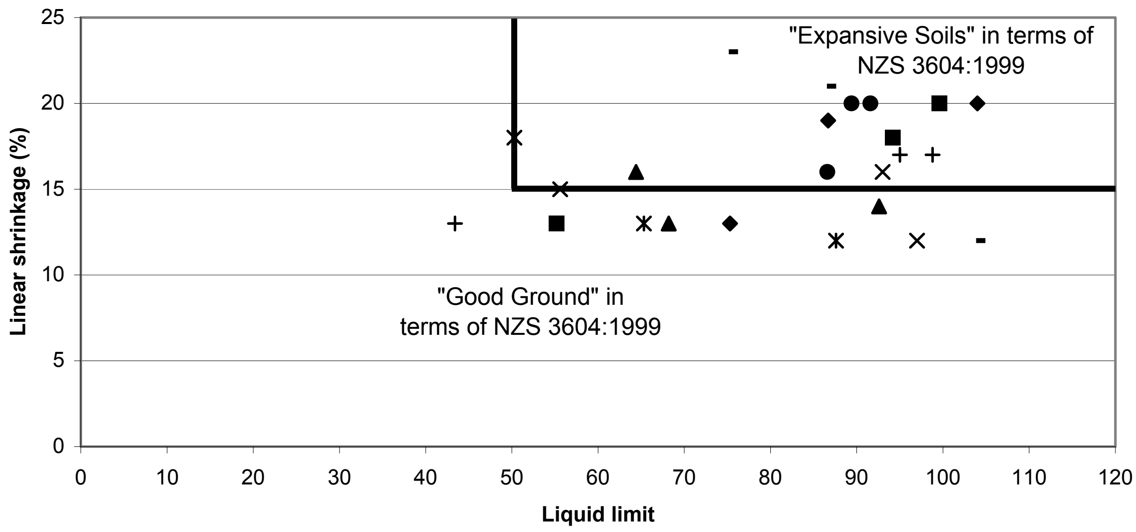


Figure 7: Linear shrinkage against liquid limit

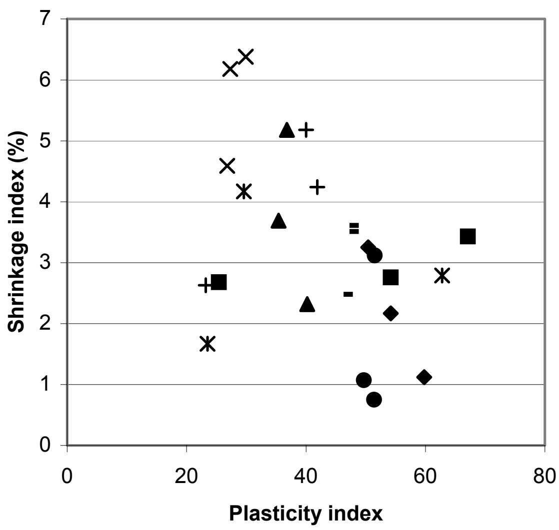


Figure 8: Shrinkage index against plasticity index

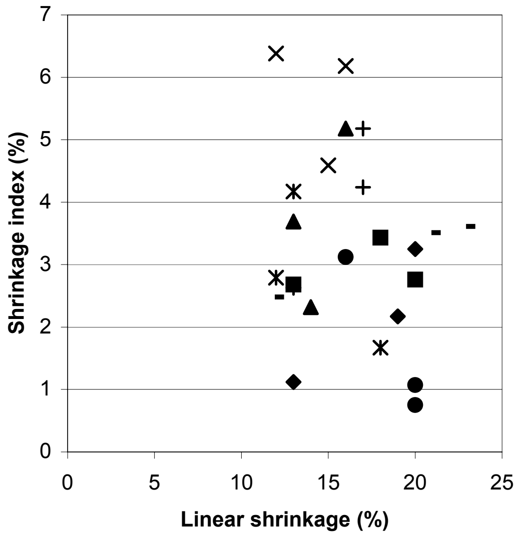


Figure 9: Shrinkage index against linear shrinkage

◆ A - Basaltic Ash ▲ C - Waipapa ✳ E - Tauranga Group (tp) + G - Waitemata Group
■ B - Basaltic Ash/Tauranga Group (tpp) × D - Tauranga Group (tp) ● F - Waitemata Group - H - Onerahi Chaos Breccia

10.0 SOIL SUCTION PROFILES

10.1 Introduction

Appendix C of this report presents the results of the site investigation and monitoring, laboratory testing, and surface movement calculations for each of the study sites. The following data is presented on the "Results for Expansive Soil Testing" for each site in Appendix C:

(a) Section 1.0 – General testing information and soil suction results

This part of the result sheet details:

(i) Dates of site sampling and laboratory testing
(ii) Soil moisture deficit reading at time of sampling
(iii) Depth to groundwater level
(iv) Water content and soil suction results for each borehole at depths of 0.5m, 1.0m, 1.5m, 2.0m and 2.5m. The soil suction results have been normalised to 3.2 pF where readings fell below this lower bound, as discussed in this Section
(v) The "graphs of soil suction and water content data" present the data in (iv) above for the three samples taken and also a plot of the "change in soil suction" between the two summers and one winter sample taken.

(b) Section 2.0 – Results of soil classification testing

This part of the result sheet presents the results of the laboratory testing for the samples taken at depths of 0.5m. 1.0m and 1.5m to provide Atterberg limits, linear shrinkage and shrinkage index data for each site.

(c) Section 3.0 – Calculation of surface movement using assumed design Soil Suction Change Profile Alpha.

This part of the result sheet presents the results of theoretical calculations based on the assumptions, which are discussed in the following Section 11.0 of this report.

The change in soil suction profiles for Sites A to H are shown in Appendix C.

Woodburn (2003) noted that low suction values of less than 3.2 pF are generally recorded as being <3.2 pF, given that for most soils, very little volume change occurs below 3.2 pF as any swelling is resisted by the overburden pressure.

The manual for the Soil Mechanics Instrumentation Transistor Psychrometer also notes that "accurate measurement of soil suctions below 200 kPa [3.3 pF] is very difficult" and that "often for engineering purposes it is of little consequence because soils are too wet to cause further heave type problems (the soils lack sufficient strength to exert significant swelling pressures). Reporting of results at this low end of the suction range is often limited to "below 200 kPa"."

The change in soil suction profiles shown in Section 2.0 of the individual site data sheets in Appendix C are therefore based on all low suction values, less than 3.2 pF, being recorded as 3.2 pF.

On the basis of the pF results normalised to a base of 3.2 pF, it is evident from the change in soil suction profiles in Appendix C that the maximum soil suction change between the winter 2002 measurements, and either the summer 2002 or summer 2003 measurements, was 0.3 pF or less for all sites except Sites D and H, for which a maximum value of 0.4 pF was recorded.

10.2 Depth of soil suction change

The results recorded in Section 2.0 of the individual site data sheets in Appendix C generally indicate that, on the basis of the normalised pF results, no significant change in suction occurred below approximately 2.0m depth, except for Sites B and H. It should, however, be noted that due to the confinement of the overburden pressure, the change in suction values at depths below 1.5m to 2.0m depth may not necessarily result in any significant ground movement.

As discussed in Section 14.0 of this report, it is suggested that in the recommended future extension of the initial Stage I study reported herein, that the ground movement be monitored to confirm the depth below which no significant ground movement occurs. In the absence of definitive measured values, it is concluded that a depth of soil suction (H_s) value of 2.0m is an appropriate value for all sites except Site H, which is underlain by Onerahi Chaos deposits. Further monitoring would be required in order to determine an appropriate H_s value for Site H.

It is noted that the H_s value of 2.0m indicated from appraisal of the measured soil suction profiles reported herein is greater than the value of 1.5m determined from the correlation proposed by Fityus (1998) and shown in Table 4 of this report. Until further monitoring of seasonal soil suction changes and ground movement is undertaken, as was proposed for the Stage II investigations as discussed in the foregoing Section 2.2(b), to provide a more reliable value of H_s, it is proposed that a value of 2.0m be adopted. A sensitivity analysis indicates that the computed "design characteristic surface movement" (y_s) increases by between approximately 35% and 67% if H_s is increased from 1.5m to 2.0m.

On the basis of the limited data base developed by the study reported herein, a design H_s value of 2.0m is therefore considered to be potentially conservative.

It is noted that Table 2.4 of AS 2870 provides a recommended H_s value of 1.5m for Newcastle and Sydney and 1.5m or 1.8m for eastern and coastal Victoria. The values adopted in AS 2870 indicate that the assumed Hs value for Auckland of 2.0m indicated by the soil suction change profiles shown on the site data sheets in Appendix C, approximates the upper end of the range of 1.5 to 1.8m.

10.3 Change in soil suction at ground surface

The processes provided in AS 2870 require that the change in soil suction at the ground surface be defined for drought conditions.

To determine an appropriate value of change in suction at the ground surface (Δu) for drought conditions in Auckland, a correlation between the measured soil suction values at 0.5m depth for each site and the corresponding values of daily soil moisture deficit has been used. It has been assumed that a linear correlation exists between the daily soil moisture deficit and soil suction at 0.5m depth to allow the Δu value corresponding to drought conditions at the ground surface to be inferred.

As discussed in Section 6.2.2, NIWA has advised that a daily soil moisture deficit of 90mm is representative of drought conditions in Auckland.

An inferred triangular change in soil suction profile has then been constructed by projecting a line from a Δu value of zero at 2.0m depth, corresponding to the H_s value suggested in the foregoing Section 10.2, through the inferred drought Δu value at 0.5m depth, to the ground surface so as to determine an inferred change in soil suction at the ground surface for drought conditions.

The inferred, triangular soil suction change profiles for Sites A to H are shown on Figure 10.

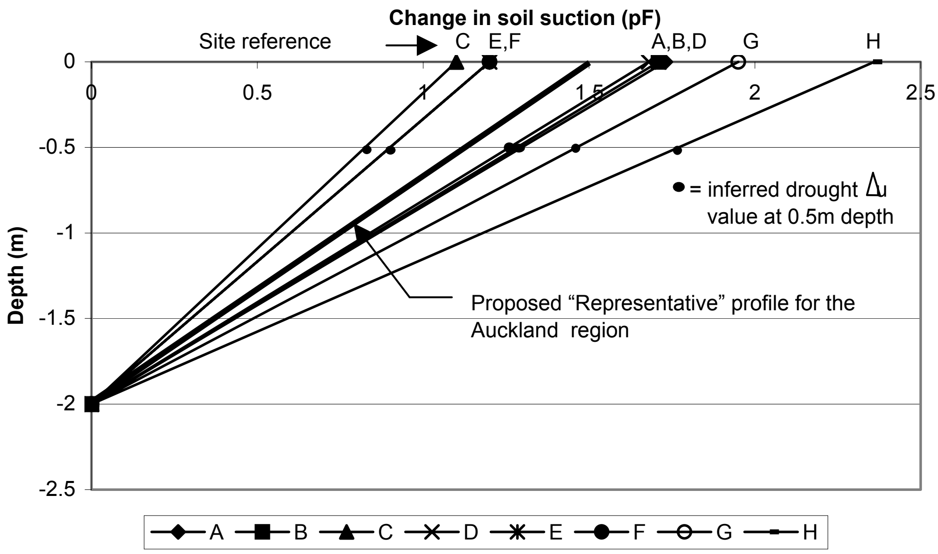


Figure 10: Assumed soil suction profiles for the study sites in the Auckland region

It is acknowledged that the assumption of a linear correlation between Δu and soil moisture deficit is not able to be substantiated. The extrapolation has, however, been adopted due to the absence of any data upon which to make a reliable assessment of the change in soil suction at the ground surface for drought conditions.

The soil suction change profiles on Figure 10 indicate that, for the suggested H_s value of 2.0m, a Δu value at the ground surface of 1.5 pF would be an appropriate "median" value. It is noted that Table 2.4 of AS 2870 provides recommended values of soil suction change at the ground surface (Δu) for various Australian centres of either 1.2 pF or 1.5 pF. It is therefore apparent that the extrapolated Δu value of 1.5 pF, derived by correlation with the soil moisture deficit at 0.5m depth, corresponds to the upper value given in Table 2.4 of AS 2870.

The validity of the foregoing extrapolation of the Δu profile to the ground surface for assumed drought conditions requires to be verified by further monitoring of soil suction at the test sites, as was proposed for the Stage II investigations discussed in the foregoing Section 2.2(b).

11.0 CALCULATED GROUND SURFACE MOVEMENTS

11.1 Introduction

The ground surface movement (y_s) for Sites A to H have been calculated for four suction change profiles, referred to as Profiles Alpha, Beta, Gamma and Delta in the following Sections 11.2 to 11.5.

(a) Profile Alpha is the recommended profile, for drought conditions, derived from a rationalisation of all discussed data. It represents the suggested suction change profile derived by appraisal of the H_s values measured at Sites A to H, as discussed in the foregoing Section 10.2, and extrapolating the measured Δu values measured at 0.5m depth to provide an inferred Δu value at the ground surface for drought conditions, as discussed in the foregoing Section 10.3.

Soil Suction Change Profile Alpha is triangular, with an H_s value of 2.0m and Δu value of 1.5 pF at the ground surface.

(b) Profile Beta is the profile derived for drought conditions for Sydney and Newcastle using the criteria of AS 2870.

Soil Suction Change Profile Beta is triangular, with an H_s value of 1.5m and Δu value of 1.5 pF at the ground surface.

(c) Profile Gamma is the profile derived for drought conditions by using the Auckland data to obtain the lowest values from the corresponding Australian region using the criteria of AS 2870.

Soil Suction Change Profile Gamma is triangular, with an H_s value of 1.5m and Δu value of 1.2 pF at the ground surface.

(d) Profile Delta is the profile determined from the site data for each specific study site and corresponds to the measured suction change profiles measured at each site, relating to the 2002 winter and the driest of the 2002 and 2003 summers and an H_s value of 2.0m.

The y_s values corresponding to Soil Suction Change Profiles Alpha, Beta, Gamma and Delta are summarised in Table 12.

Table 12 : Calculated ground surface movements and expansivity classification for study sites

Site Code	Soil Type	Suction Change Profile Alpha Δu = 1.5 pF H_s = 2.0 m		Suction Change Profile Beta Δu = 1.5 pF H_s = 1.5 m		Suction Change Profile Gamma Δu = 1.2 pF H_s = 1.5 m		Suction Change Profile Delta Δu = Normalised measured values H_s = 2.0 m	
		y_s [1] (mm)	Class-ification	y_s [1] (mm)	Class-ification	y_s [1] (mm)	Class-ification	y_s [1] (mm)	Class-ification
A	Basaltic Ash	39	M	31	M	25	M	10	S
B	Basaltic Ash/Tauranga Group (tpp)	43	H	34	M	27	M	8	S
C	Waipapa Group	56	H	37	M	30	M	3	S
D	Tauranga Group (tp)	89	E	64	H	51	H	49	H
E	Tauranga Group (tp)	45	H	29	M	24	M	15	S
F	Waitemata Group	27	M	21	M	17	S	6	S
G	Waitemata Group	58	H	39	M	31	M	15	S
H	Onerahi Chaos Breccia	53	H	38	M	31	M	26	M

Note 1: The cracked zone depth has been assumed to be 0.5 H_s in Soil Suction Change Profiles Alpha, Beta, Gamma and Delta

11.2 Profile Alpha - Triangular suction change profile (H_s = 2.0m and Δu = 1.5 pF)

The calculated values of ground surface movement for each site based on the suggested triangular design suction profile with an H_s value of 2.0m, Δu value at the ground surface of 1.5 pF and crack depth value of 1.0m (0.5H_s) are shown in Table 12. As shown in Table 12, the calculated y_s values range from 27mm for the Waitemata group soils at Site F, to 89mm, for the Tauranga group soils at Site D, corresponding to slightly to extremely expansive site classifications respectively.

From Table 12, it is apparent that:

(a) The two sites underlain by basaltic ash (Sites A and B) fall into the M and H classifications respectively

(b) The two sites underlain by Waitemata group soils (Sites F and G) fall into the M and H classification respectively, indicating that the shrinkage index and expansivity of Waitemata group soils is variable

(c) The two sites underlain by Tauranga group soils (Sites D and E) fall into the E and H site classification respectively

(d) The sites underlain by Waipapa group (Site C) and Onerahi Chaos (Site H) fall into the H site classification.

11.3 Profile Beta - Triangular suction change profile (H_s = 1.5m and Δu = 1.5 pF)

This suction change profile corresponds to the H_s value of 1.5m for a wet coastal/alpine classification and the Δu value of 1.5pF at the ground surface given in Table 2.4 of AS 2870 for various Australian locations, including Newcastle and Sydney.

The calculated y_s values for Suction Profile Beta are shown in Table 12.

As shown in Table 12, the calculated y_s values range from 21mm to 64mm, corresponding to a slightly to highly expansive site classification. It is apparent from Table 12 that the y_s values for Profile Beta are 64% to 79% of the corresponding values for Profile Alpha and, for most cases, the site classifications for Profile Beta are one grade lower than those for Profile Beta. The exceptions are Sites A and F, which fall into the M classification in both cases.

11.4 Profile Gamma - Triangular suction change profile (H_s = 1.5m and Δu = 1.2 pF)

This suction change profile corresponds to the H_s value of 1.5m derived on the basis of the TMI of +50 for Auckland and the lowest Δu value of 1.2pF at the ground surface given in Table 2.4 of AS 2870 for various Australian locations, including Melbourne, Brisbane and Launceston. This design soil suction change profile therefore represents the “best case” scenario derived from AS 2870 and Fityus et al (1998).

The calculated y_s values for Suction Profile γ are shown in Table 12.

As shown in Table 12, the calculated y_s values range from 17mm to 51mm, corresponding to a slightly to highly expansive site classification. It is apparent from Table 12 that the y_s values for Profile Gamma are 53% to 64% of the corresponding values for Profile Alpha and, for most cases, the site classifications for Profile Gamma are one grade lower than those for Profile Alpha. The exception is Site A, which changes from the “high end” to the “low end” of the M classification.

11.5 Profile Delta – Normalised soil suction change profile

The calculated y_s values for each site, based on the measured, and normalised, soil suction change profiles for each site, normalised to a base of 3.2 pF with an H_s value of 2.0m and a crack depth of 1.0m (0.5H_s) are shown in Table 12.

As shown in Table 12, the calculated y_s values range from 3mm to 49mm and are substantially less than the corresponding values obtained using suction change Profile Alpha, being between 5% and 55% of the corresponding values for Profile Alpha.

The most notable change from the values calculated from Profile Alpha occurs for the Waipapa Group soils at Site C, where the calculated y_s value reduces from 56mm to 3mm, and the classification from H to S.

11.6 Summary

From the foregoing, it is therefore apparent that;

(a) If Soil Suction Change Profile Alpha is adopted, being a triangular profile with an H_s value of 2.0m and Δu value at the ground surface of 1.5 pF, the sites generally fall into the M and H classifications whereas, if Profile Gamma is adopted, being a triangular profile with an H_s value of 1.5m and Δu value at the ground surface of 1.2pF, the sites generally fall into, or close to the M classification

(b) The calculated y_s values for Soil Suction Change Profile Gamma are between 53% to 64% of the corresponding values for Profile Alpha

(c) The calculated y_s values for Profile Delta are between 5% to 55% of the corresponding values for Profile Alpha. However, it should be noted that the summers of 2002 and 2003 were not representative of drought conditions, ie it is expected that higher suction change values would be measured at the end of a drought period than were measured during 2002 and 2003 summers, which would result in higher calculated y_s values.

It is therefore concluded that the suggested design Soil Suction Change Profile Alpha results in a potentially conservative calculated value of y_s and site classification in terms of AS 2870 for sites in the Auckland Region. It is possible that the results of the recommended further research will show that the suggested design Soil Suction Change Profile Alpha should be modified, resulting in lower design values of H_s and Δu at the ground surface and, consequently, lower calculated y_s values and site classification for any particular site.

12.0 FOUNDATION ANALYSIS

12.1 Introduction

The analyses provided in the following summarises the requirements of the Australian Standard, AS 2870 and comments on the applicability of these requirements for buildings within the Auckland Region.

12.2 Buildings included in NZS 3604

Clause 1.1.2 of NZS 3604 describes the buildings (and sites), which are covered by the Standard and to which the requirements of NZS 3604 apply.

The following types of construction covered by the Standard are summarised from NZS 3604 Figure 1.2:

(a) One and two-storey buildings – slab on ground with clad framing or masonry veneer

(b) One and two-storey buildings with a foundation wall no higher than 2m and with clad framing or masonry veneer

(c) Three-storey buildings with a foundation wall no higher than 2m, and with the lower storey in concrete masonry.

12.3 Foundation design to AS 2870

12.3.1 General

Clause 1.4.2 of AS 2870 requires that foundations are to be designed for both serviceability and strength for foundation movement and the effects of gravity loads.

AS 2870 provides a building foundation designer with two options with respect to designing the foundations of a structure, they are:

(a) Standard Designs – these are prescriptive designs for a range of site classifications and construction types. The standard provides solutions for stiffened raft, waffle raft, and strip footings.

(b) Specific Designs – these are engineering design principles, detail in Section 4 of AS 2870, which allow the designer to alter the "standard designs" for footings listed in Clause 4.3 of AS 2870, viz:

"(i) Raft footing systems supporting a superstructure that relies entirely on the raft to resist cracking,
(ii) Footing systems for walls which are able to cantilever without cracking,
(iii) Other footing systems".

This section also allows a "designer", who is designated in AS 2870 as a "qualified engineer", to utilise the engineering design principles of AS 2870 for buildings that are generally excluded from the standard designs of AS 2870, ie buildings beyond the limits prescribed in the foregoing Section 12.3.1(a). However, for standard raft designs, AS 2870 specifies acceptable ranges for design parameters in Clause 4.5.1 of the Standard.

Although Clause 1.0 of AS 2870 indicates that AS 2870 will generally be applied to "Class 1 and 10A Buildings", ie residential dwellings and non-habitable auxiliary buildings, it is understood that AS 2870 is often applied also to commercial, industrial and educational buildings where the construction types fall within the clad frame and masonry construction types.

12.3.2 Buildings excluded from standard design under AS 2870

Clause 3.1.1 of AS 2870 identifies the situations where the standard designs provided by the Standard cannot be applied:

"(a) Class E or P sites [Expansivity classes – (E)xtreme and (P)roblem sites]*;*

(b) *Buildings longer than 30m;*

(c) *Slabs containing permanent joints eg contraction or control joints;*

(d) *Two-storey construction with a suspended concrete floor at the first floor level except in accordance with Clause 3.5* [which specifies geometric limitations of concrete floors for buildings on Class A and S sites]*;*

(e) *Two-storey construction in excess of the height limitations* [of 8.0m]*;*

(f) *Support of columns or fireplaces not complying with Clause 3.6* [which specifies footing construction for columns and fireplaces]*;*

(g) *Buildings including wing-walls or masonry arches unless they are detailed for movement in accordance with TN 61* [which is an industry guideline published by Cement and Concrete Association of Australia for Articulated Walling]*;*

(h) *Construction of three or more storeys; or*

(i) *Single-leaf earth or stone masonry walls greater than 3m height".*

Where the standard designs are precluded from use, AS 2870 provides for a "qualified engineer" to design the footings in accordance with Section 4 of AS 2870.

12.3.3 Construction types included in AS 2870

AS 2870 provides for the design of foundations for buildings of brick (or earth) masonry and clad framing construction. The three main construction types in Auckland are defined as follows:

(a) **Clad frame** - is defined in Clause 1.7.9 of AS 2870 as "timber or metal frame construction with the exterior wall clad with timber or sheet material not sensitive to minor movements. Includes substructure masonry walls up to 1.5m high"

(b) **Articulated masonry veneer** – is defined in Clause 1.7.3 of AS 2870 as "masonry veneer construction in which the provisions for articulated masonry have been applied to the masonry veneer" ie construction joints etc

(c) **Masonry veneer** – is defined in Clause 1.7.32 of AS 2870 as "house construction consisting of a load-bearing frame clad with an outer leaf of masonry".

Where mixed construction types are used within a building, the "equivalent construction types" of buildings including masonry for some or all the walls are detailed in the Standard. Those requirements are summarised in the following Table 13.

Table 13: Equivalent constructions for application in AS 2870
(taken from AS 2870 Table 3.1)

External walls	Internal walls	Equivalent construction
Single leaf masonry		
Reinforced single leaf masonry	Articulated masonry on Class A and S sites, or framed	Articulated masonry veneer
Reinforced single leaf masonry	Articulated masonry or reinforced single leaf masonry	Masonry veneer
Reinforced single leaf masonry	Masonry	Articulated full masonry
Articulated single-leaf masonry	Articulated masonry	Articulated full masonry
Articulated single-leaf masonry	Masonry	Articulated full masonry
Other single-leaf masonry	Framed	Articulated full masonry
Other single-leaf masonry	Masonry	Full masonry
Mixed construction		
Full masonry	Framed	Articulated full masonry
Articulated full masonry	Framed	Masonry veneer
Earth masonry		
Infill panels of earth masonry	Framed earth masonry	Articulated masonry veneer
Load bearing earth masonry	Load bearing earth masonry	Articulated full masonry

12.3.4 AS 2870 Design Philosophy for Foundation Movement

Although building foundations are often unsymmetrical and irregular in layout it is a common design approach that the footing layout, for design calculations, is approximated to overlapping rectangular areas which can be defined to have a "mound shape", ie a profile that the soil changes to when it experiences a moisture change.

For design purposes it is assumed that the mound is symmetrical and experiences centre heave and/or edge heave as shown in Figure 11. The mound shape is defined by two parameters – the edge distance (e) and the free unloaded mound heave within the confines of the plan area of the structure (y_m).

12.3.5 Design Parameters in the consideration of foundation movement

The following soil parameters are taken from AS 2870:

(a) Mound stiffness – is defined in Appendix F4 (c) of AS 2870 and is in the range of 400 – 1500 kPa/m for beams in contact with swelling soil, with the further limitation of 100q (but not less than 1000 kPa/m) where q is the total building load divided by the slab area. It is also noted that "computed forces and displacements are generally not particularly sensitive to the value of [mound stiffness] used except for certain heave situations".

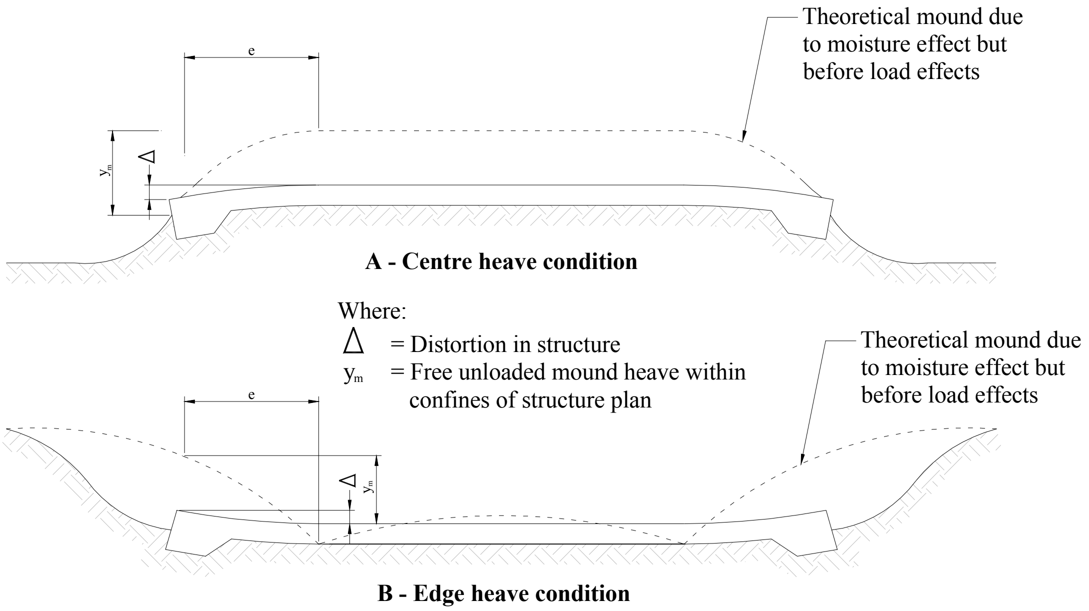


Figure 11: Soil structure interaction (from HB-28 Figures 1.6 and 5.4)

(b) Soil heave – the free unloaded heave (y_m) calculated using Equation 5.

$$y_m = 0.7\ y_s \qquad \textbf{Equation 5}$$

where

y_s = design characteristic surface movement (mm)

y_m is always less than y_s due to the slab stiffness and weight of the structure above. Due to the inaccuracy of field assessment of these values (as discussed in the foregoing Section 5.4.2) the maximum in the range is always used for analysis to ensure conservatism.

AS 2870 Section F4 states that "on a site that is wet throughout the profile at the time of construction, a reduction of y_m for edge heave not exceeding 40% may be made". This reduction factor is directly applicable to sites that have been pretreated to maintain a high water content.

(c) Edge distance –

$$e = (H_s/8 + y_m/36) \quad \text{for centre heave} \qquad \textbf{Equation 6}$$

$$e = 0.2\ \text{slab length, or} \quad \text{for edge heave} \qquad \textbf{Equation 7}$$

$$(0.6 + y_m/25)$$

where

H_s = depth below ground level at which no moisture change occurs (m)

(d) Mound exponent (m) – is a shape factor utilised in the Mitchell Method (1984) and is defined in Equation 8.

$$m = \frac{1.5L}{a} \qquad \textbf{Equation 8}$$

where

L = span of footing (m)

a = $D_{cr} - D_e$, where **Equation 9**

$D_{cr} = \frac{H_s}{7} + \frac{y_m}{25}$ **Equation 10**

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

(e) Permissible deflection - the level of differential movement that a foundation can undergo before the building will show unacceptable levels of damage is specified in AS 2870 and summarised in Table 14 and Figure 12.

Table 14 : Maximum design differential movement, Δ, for design of footings and rafts (from AS 2870 Table 4.1)

Type of construction	Maximum differential footing movement, Δ	
	As a function of span, mm	Absolute, mm
Clad frame[1]	*≤ L/300*	*40*
Articulated masonry veneer[1]	*≤ L/400*	*30*
Masonry veneer[1]	*≤ L/600*	*20*
Articulated full masonry	≤ L/800	15
Full masonry	≤ L/2000	10

Note 1: These construction types are the three most common in Auckland

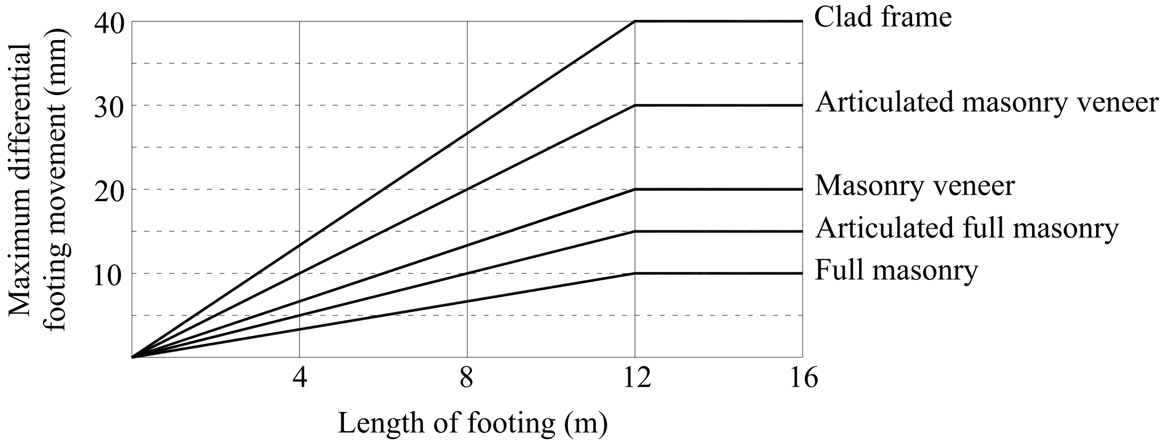


Figure 12: Acceptable footing movement as a function of length of footing

12.3.6 AS 2870 Design philosophy for gravity loads

AS 2870 relies on the Australian Loading Standard – AS 1170 Part 1, in terms of:

(a) Limit State Design philosophy being applied,

(b) 95% characteristic values of design actions and member resistances being used,

(c) Load combinations and load factors being consistently applied.

The requirements of AS 1170 Part 1 are generally similar to those prescribed in the New Zealand limit state Standard NZS 4203:1992 "Code of Practice for General Structural Design and Design Loadings for Buildings" and the related materials standards, eg NZS 3101 "Code of Practice for the Design of Concrete Structures".

Clause 1.4.2 of AS 2870 specifies the foregoing Equation 5 as a means of determining the "design load" for the calculation of settlement, whilst a reduction factor of 0.3 is applied to the "Design Load" for assessing bearing and uplift failures of the foundations.

Design load = Dead load + Load combination factor x Live load **Equation 11**

where

Dead load	=	the weight of the slab, foundations and superstructure of a building.
Load combination factor	=	0.5, as shown in AS 2870 Clause 1.4.2
Live load	=	as specified in AS 1170.1 "Structural Design Actions – Part 1: Permanent, imposed and other actions", eg domestic buildings have a live load of 1.5 kPa

The load combination factor was included in AS 2870 "to take into account the long term development of soil pressures induced by the soil heave where an unfactored dead load and factored live loads are appropriate" (Mitchell peer review comments).

Table 15 is reproduced from Clause 6.2.2 of HB28, and is based on a single storey building with tiled roof supported by trusses and is suggested as a guide for preliminary designs.

The distributed loads listed in Table 15 include consideration of the self weight of the foundation as full contact between the foundation and the underlying ground is not always achieved. The distributed loads vary depending on the depth and spacing of the foundation beams.

From analyses carried out for this report, it is noted that the edge beam depth is sensitive to the level of edge loading imposed on the building perimeter foundations for the "centre heave" case.

Table 15: Typical stiffened slab loads for a single storey building with tiled roof and trusses
(taken from HB 28 Clause 6.2.2)

Site class	Type of construction	Distributed loads		Edge load (kN/m) [1]
		Short direction (kPa) [1]	Long direction (kPa) [1]	
M	Clad frame	4.8	5.3	3.3
	Articulated masonry veneer	5.7	6.9	8.0
	Masonry veneer	5.8	7.0	8.0
	Articulated full masonry	9.2	11.2	13.1
	Full masonry	10.7	12.6	13.1
H	Clad frame	5.3	5.8	3.3
	Articulated masonry veneer	6.8	8.0	8.0
	Masonry veneer	7.2	8.5	8.0
	Articulated full masonry	11.6	13.6	13.1

Note 1: HB 28 Clause 6.2.2 explains "there is a difference between the short direction … and the long direction … because of the contribution of edge beams to uniform loading. The edge load is superimposed."

12.4 Comparison of NZS 3604 to AS 2870

12.4.1 Introduction

The following is a comparison of geometrical limitations and design parameters set out in NZS 3604 and AS 2870.

12.4.2 Comparison of the geometrical limitations in NZS 3604 and AS 2870

Table 16 summarises the building variables for which both AS 2870 and NZS 3604 provide geometrical limitations for the building structure above foundation level.

Table 16: Comparison of NZS 3604 and AS 2870 building geometry limitations

Factor		AS 2870	NZS 3604
Maximum height		8 m	10 m
Maximum number of storeys		2	3
Maximum height of foundation wall		1.5 m	2 m
Maximum length of building		30 m	- [1]
Plan floor area	1 and 2 storeys – timber framed	- [1]	Unlimited
	2 storey – other forms of construction	- [1]	300 m^2
	3 storey – other forms of construction	No 3 storey buildings	250 m^2

Note 1: A (-) indicates that the applicable standard makes no reference to a limitation on the factor within Table 16.

12.4.3 Design parameters for Australian and Auckland soils

Based upon the soil testing undertaken for this study and a review of AS 2870, the foundation design parameters for Australia and those proposed for the Auckland region are summarised in Table 17.

Table 17: Foundation design parameters for Australia and Auckland

Design parameter	Australian parameter range (from AS 2870 Clause 4.5.1)		Auckland parameter range
H_s	>3m	<3m	2m [1]
y_s	100mm	10-70 mm	15-85mm [1]
y_m	7-49mm	70mm	10-60mm [1]
Δ	5 to 50mm	5 to 50mm	5 to 50mm [2]
Span	5 to 30m	5 to 30m	5 to 30m [2]
Average load	to 15 kPa	to 15 kPa	to 15 kPa [2]
Edge line load	to 15 kN/m	to 15 kN/m	to 15 kN/m [2]

Note 1: These proposed parameters have been taken from Soil Suction Change Profile Alpha as discussed in the foregoing Section 11.2.

Note 2: These proposed parameters have been adopted from AS 2870 as they have not been a focus of this particular study.

12.5 Structural review of foundation designs

12.5.1 Introduction

A desk-top review was carried out utilising the DOS based software programme, called "Slab on Ground (SLOG)", provided by Dr Peter Mitchell (Adelaide) as discussed in Mitchell (1984), to determine the minimum founding depth of footings for a defined building geometry, loading and proposed foundation reinforcement.

This review is set out in the following sections.

12.5.2 Structural review methodology

The aim of the review was to determine the area of reinforcement steel required in each foundation type to achieve the target depths indicated in Table 19.

The "target depths" are the minimum beam depth that meets the requirements of NZS 3406 or AS 2870, and the analyses undertaken alters the reinforcing steel to achieve these beam depths.

Using the "design" module of SLOG, the depth of edge beam was found using the Foundation Design Variables discussed in the following Section 12.5.3.

The results of the analyses are reported in the following Section 12.5.5.

12.5.3 Foundation design variables

(a) Geometry - A typical rectangular single storey house of masonry veneer construction and of external dimensions, 16m x 8m, was modelled with the following foundation layouts, as shown in Table 19:

(i) Foundation Layouts 1A and 1B – are a standard foundation design from Figure 7.15A of NZS 3604 where the footing is nominally tied to the slab for placement, but does not have adequate connection for the footing to act integrally with the slab

(ii) Foundation Layouts 2A, 2B and 2C – are a standard foundation design from Figure 7.14B (Right hand side diagram) of NZS 3604 where the footing and slab are an integral structural element

(iii) Foundation Layout 3 – is a standard stiffened slab design from Figure 3.1 of AS 2870 and is deemed suitable for use on Class M sites for masonry veneer construction

(iv) Foundation Layout 4 – is a standard waffle slab design from Figure 3.4 of AS 2870 and is deemed suitable for use on Class M sites for masonry veneer construction.

(b) Loadings - the SLOG program allows for seven different loading inputs. The load parameters and values used for the analyses reported herein are summarised in Table 18

(c) Number of beams – this input allows the user to specify the number of beams parallel to the long and short spans to model both strip footings and raft foundations

(d) Permissible deflection – the AS 2870 recommended limits of span/600 and an absolute value of 20mm, as indicated in Table 14 and Figure 12 for masonry veneer construction, have been used

(e) Design characteristic surface movement, y_s, of 40mm has been used, which corresponds with a Site Classification of M and a free unloaded mound heave, y_m, of 28mm

Table 18 : Design loadings for structural analysis

Load parameter		Design loads used for study
Edge load on "west" end		8 kN/m
Edge load on "east" end		8 kN/m
Edge load on "north" side		8 kN/m
Edge load on "south" side		8 kN/m
"North-south" centre load		0 kN/m
"East-west" centre load		0 kN/m
Uniform distributed floor load	– Foundations 1 and 2	4.0 kPa
	- Foundations 3 and 4	5.8 kPa

(f) Youngs modulus of concrete – for the analyses reported herein, an E_c value of 15,000 MPa has been adopted as a default value and "is commonly used in footing designs in Australia to account for the development of shrinkage cracks, and is specified in Clause 4.4(e) of AS 2870

(g) Compressive strength of concrete – for the analyses reported herein, an f'_c value of 8 MPa has been adopted for Foundation 1A, 1B and 1C, and an f'_c value of 20 MPa has been adopted for all other foundations

(h) Tensile strength of concrete – there are two inputs for tensile strength for calculating the cracking moment capacity for 20 MPa concrete – 2.7 MPa for sagging moments and 1.8 MPa for hogging moments. These values of tensile strength have also been adopted for foundations constructed using concrete masonry blockwork.

12.5.4 Theory that SLOG is based upon

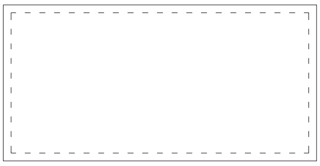
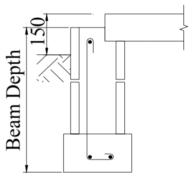
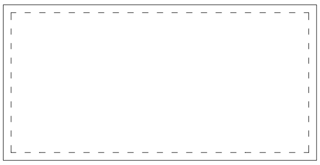
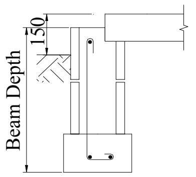
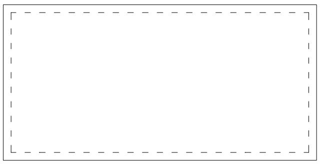
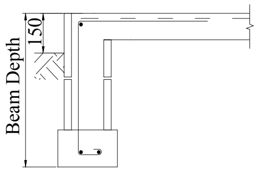
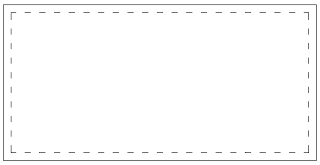
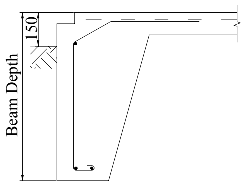
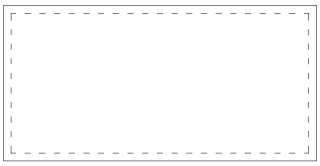
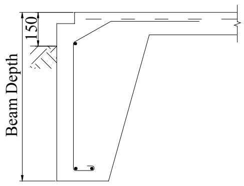
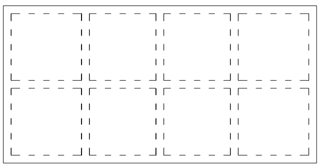
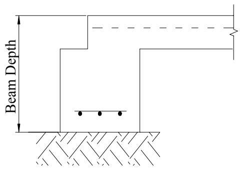
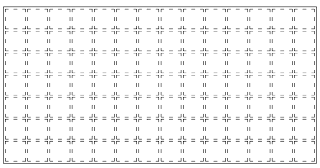
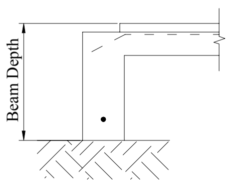
Mitchell (1984) suggests that the simplest structural analysis for the design of foundations on expansive soils is the beam-on-mound method, for which the primary design conditions discussed in Section 12.0 and shown on Figure 11 are:

(a) The centre heave condition

(b) The edge heave condition.

SLOG analyses both conditions for each foundation span by calculating the mound shape factor and utilising a predetermined set of boundary conditions to calculate either the required bending moment capacity or the minimum founding depth required for the foundation system to resist both the imposed loads from the building superstructure and those loads created by the soil-structure interaction.

Table 19: Foundation layouts for SLOG analysis (Site expansivity class – M)

Foundation reference	Foundation layout	Foundation detail	Reference in standard	Minimum specified reinforcement in footing	Target depth [1] (mm)
1A	 Foundation beams around perimeter	 Slab not tied into beam	NZS 3604 Fig 7.15A	Top 1-D12 Bottom 2-D12	- [2] / - [2]
1B	 Foundation beams around perimeter	 Slab not tied into beam	NZS 3604 Fig 7.15A	Top 1-D12 Bottom 2-D12	550 / 600
2A	 Foundation beams around perimeter	 Slab tied into beam	NZS 3604 Fig 7.13B (without DPM slip layer)	Slab 665 M Top 1-D12 Bottom 2-D12	600 / 600
2B	 Foundation beams around perimeter	 Slab tied into beam	NZS 3604 Fig 7.14B (Right hand side diagram)	Slab 665 M Top 1-D12 Bottom 2-D12	600 / 600
2C	 Foundation beams around perimeter	 Slab tied into beam	NZS 3604 Fig 7.14B (Right hand side diagram)	Slab 665 M Top 1-D12 Bottom 2-D12	750 / 750
3	 5 x 3 Foundation beam grid	 Slab tied into beam	AS 2870 Fig 3.1	Slab SL72 Bottom 3-L11TM *NZ Equivalent Slab 664M Bottom 3-H12*	400 / 400
4	 15 x 8 Foundation beam grid	 Slab tied into beam	AS 2870 Fig 3.4	Slab SL72 Bottom 1-N12 *NZ Equivalent Slab 664M Bottom 1-H12*	310 / 310

Note 1: Two definitions of "target depth" apply – the upper value is the "structural beam depth" and the lower value is the "top of slab to bottom of footing dimension"

Note 2: The analysis of Foundation 1A was undertaken to determine the minimum beam depth required if the foundation specified in NZS 3604 was constructed on a Class M site. The "-" signifies that the authors had no predefined target depth for this portion of the analyses.

12.5.5 Results of analyses

Table 20 summarises the results of the analyses undertaken for the foundation systems discussed in the foregoing Section 12.5.2 and Table 19.

Table 20 : Results of analyses of foundations using SLOG program
(Site expansivity class M)

Foundation reference	Beam width (mm)	Slab thickness (mm)	Reinforcing steel in footings			Minimum target depth required [3]
			Bottom	Slab	Top	
1A	240	N/A	2-D12	NS [1]	D12	1725
1B	240	N/A	2-D12	NS [1]	2-H30 [2]	600
2A	240	N/A	2-D12	665M	D16 + D10 [2]	600
2B	240	100	2-D12	665M	D16 [2]	600
2C	240	100	2-D12	665M	D12	750
3	300	100	3-L11TM	SL72	NS [1]	400
4	110	85	1-N12	SL72	N12 [4]	350[4]

Note 1: NS indicates where there is no reinforcing steel specified, either in the slab, due to lack of connection between slab and beam, or in the top of the footing, because AS 2870 specifies only mesh in the slab for these specific footings.

Note 2: Reinforcing steel specified differs from that required by NZS 3604 to assess the sensitivity of the "minimum beam depth" derived by the programme to the amount of top steel in the footing.

Note 3: In this table the "target depth" is the "top of slab to bottom of footing dimension". A (-) entry indicates that this foundation type is not permitted by AS 2870 for Class M sites.

Note 4: There appears to have to be an "experience factor" used in the design of standard footings based on the historical knowledge and judgement of Australian engineers in the preparation of some of the standard designs specified in AS 2870. In this case the AS 2870 specified reinforcing is adequate for a Target Depth of 400mm. An additional N12 bar at the top of the footing reduces this to 350mm however increasing the bar size beyond 12mm diameter has no advantage in decreasing beam depth.

12.6 Back analyses of foundations used in the Auckland region

12.6.1 Introduction

A foundation type similar to the Type 2A foundation from the foregoing Table 19 and Figure 13 is commonly used in Auckland. As discussed in the foregoing Section 4.2, minimum "embedment depths" of 300 mm and 450 mm, specified in NZS 3604:1999, and 600mm, have the corresponding "beam depths" of 450 mm, 600 mm and 750 mm respectively, once the 150 mm freeboard to the floor of the "habitable space" has been incorporated.

Section 2.2 of AS 2870 permits identification of the soil expansivity class of a site by consideration of "established data on the performance of houses on the soil profile". The commentary of AS 2870 states that "the method relies on assessment of damage (cracking) of houses of masonry (either veneer or full) construction, or the level of maximum differential movement of clad frame houses. Preferably, the appraisal should be based on houses with similar wall construction to that which is intended to be built and which are at least 10 years old. If light footings have been used satisfactorily in the past, the classification of a site in that area should be Class S or at worst Class M".

On the basis of the foregoing, the following section examines the likely design characteristic surface movements, y_s, in the Auckland region arising from the common forms of footing construction, given that there is little anecdotal evidence to indicate the foundations designed and constructed to a particular detail in the past 10 years have failed.

12.6.2 Methodology for back analyses of foundations

The foundations were analysed using the foundation design variables set out in the foregoing Section 12.5.3, and the NZS 3604 specified reinforcement, as shown in Figure 13, of 665 mesh in the slab, 1-D12 bar at the top of the footing and 2-D12 bars in the base of the footing.

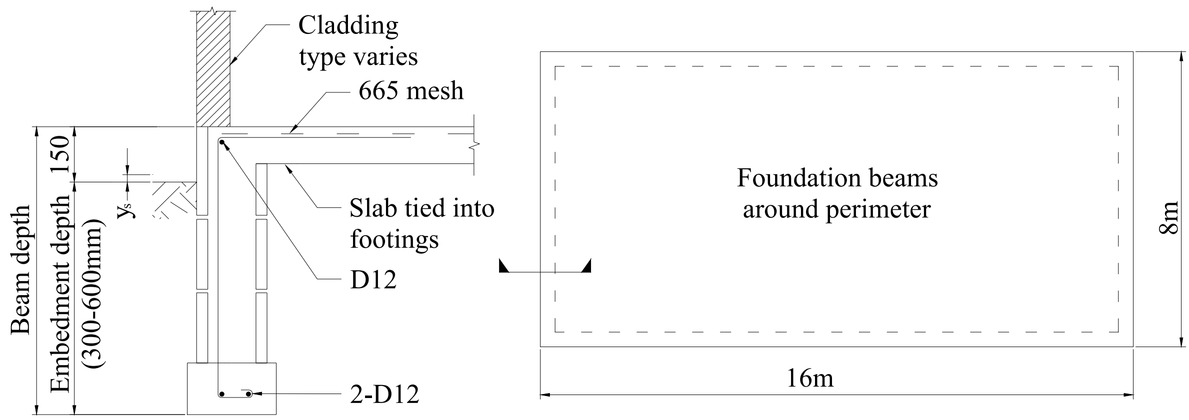


Figure 13: Typical Type 2A foundation

The design variables discussed in the foregoing Sections 12.4.3 and 12.5.3 were used in these analyses however the edge loads specified in Section 12.5.3(b) were reduced from 8.0 kN/m to 3.3 kN/m for "clad frame" construction.

The aim of the back analyses was to ascertain the value of free unloaded mound heave, y_m, and subsequently the design characteristic surface movement, y_s, that the foundation beam could withstand, if the permissible deflection was based on the AS 2870 recommended limits indicated in Table 14 and Figure 12, for clad frames, articulated masonry veneer and masonry veneer construction respectively.

12.6.3 Results of back analyses of foundations

Back analyses of the Type 2A foundations, on the basis of the embedment and beam depths set out in Section 12.6.1, suggest that the following limits might apply for the design characteristic surface movement (y_s) if cladding damage is to be avoided, for a building with a footprint of 16m by 8m.

Table 21 : Results of back analyses of Type 2A foundations for a 16m x 8m building using SLOG program

Construction type	Beam depth (mm)	Embedment depth (mm)	Calculated maximum surface movement, y_s, to avoid cladding damage (mm)	Expansivity class
Clad frame	450	300	48	H
	600	450	58	H
	750	600	68	H
Articulated masonry veneer	450	300	36	M
	600	450	46	H
	750	600	56	H
Masonry veneer	450	300	24	M
	600	450	34	M
	750	600	44	H

12.6.4 Interpretation of back analyses

Based on the back analyses results in the foregoing Section 12.6.3, it can be concluded that the embedment depth of Type 2A foundations for a building with a footprint of 16m by 8m could be determined for a specified site design characteristic surface movement, y_s, using a monograph as in Figure 14, which reflects the cladding construction type adopted.

The historical performance of residential building foundations in Auckland indicates that there have been few masonry veneer buildings, built in accordance with NZS 3604:1990 with an embedment depth of 450 mm, and in some cases 600mm, which have displayed significant levels of cladding damage over reasonably long periods of time. One conclusion could be drawn from this is that the footing depths historically provided by NZS 3604:1990, discussed in the foregoing Section 4.2, are reasonable for masonry veneer, articulated masonry veneer and clad frame buildings respectively.

It is the authors' recommendation that a building damage survey, as discussed in the foregoing Section 2.5, is carried out to validate the performance of residential building foundations in Auckland, and to provide designers and local authorities with design nomographs similar to that shown in Figure 14 for varying sizes of buildings.

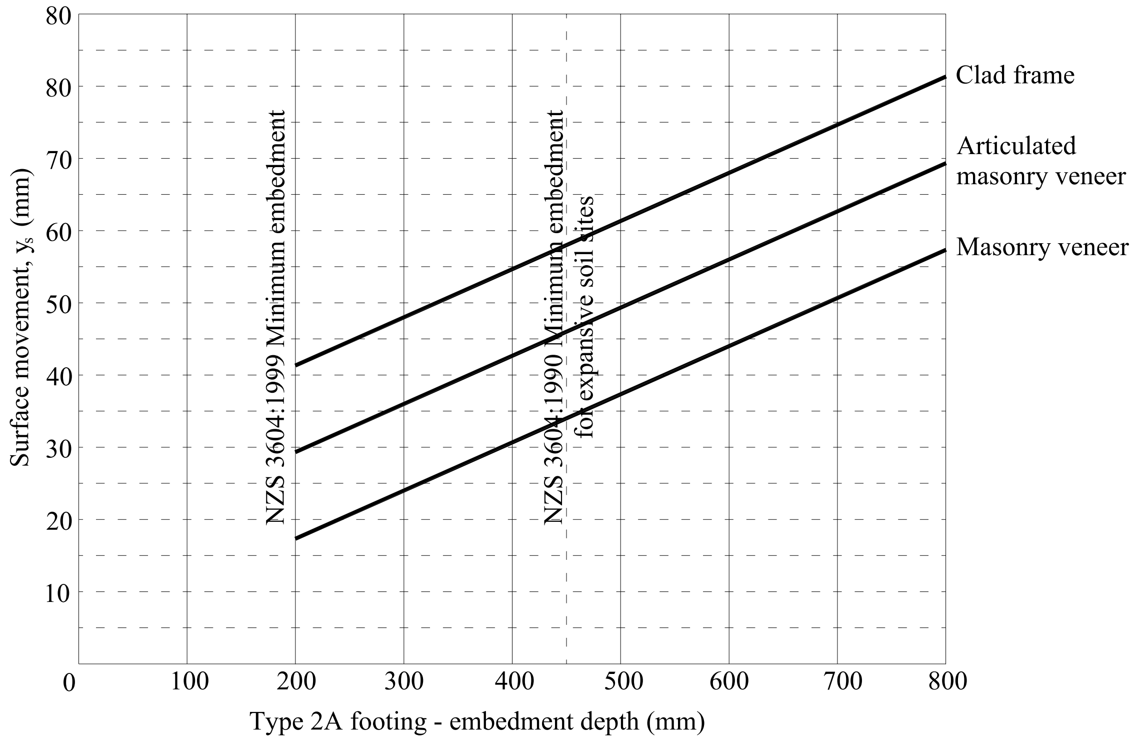


Figure 14: Surface movement against embedment depth

12.7 Application of AS 2870 in Auckland

12.7.1 Introduction

Several factors contribute to whether or not AS 2870 is applicable to the Auckland situation; they include:

(a) Do the geometrical limitations within NZS 3604 align with those described in AS 2870?

(b) Do the construction types within NZS 3604 align with those described in AS 2870?

(c) Are the design assumptions and parameters from AS 2870 directly transferable to the New Zealand situation?

12.7.2 Conclusions

As discussed in Section 12.3.1, AS 2870 provides for two areas of design - standard designs and specific designs. The following summarises the findings of the structural analysis of foundations on expansivity class M sites using the SLOG programme:

(a) Type 1 foundations are unsuitable for construction on expansive soil sites in the Auckland region without modification.

(b) Foundations where the floor slab is fully integrated with the block base wall to provide structural performance similar to that of Type 2A foundations, also require modification from the details provided in NZS 3604 to be suitable for construction on expansive soils sites in the Auckland region.

(c) Foundation Type 2C requires no modification from the detailing specified in NZS 3604 to be suitable for construction on Class M expansive soils sites in the Auckland region where the beam depth is at least 750mm. For reduced beam depth additional steel reinforcement will be required, as indicated in Table 20 for Foundation Type 2B.

(d) Based upon the geometry limitations for AS 2870 summarised in Table 16, it is concluded that the standard designs from AS 2870 for stiffened slab construction or waffle slab (similar to Foundations 3 and 4 in Table 19) are applicable to the following buildings:

   (i) Clad frame construction of up to two storeys with concrete masonry blockwork foundation walls less than 1.5m high
   (ii) Masonry veneer construction of up to one storey with concrete masonry blockwork foundation walls less than 1.5m high.

All other types of construction need to be subject to specific design in accordance with Section 4 of AS 2870.

12.8 Research into the performance of existing footings

As discussed in the foregoing Sections 2.0 and 12.6.4, Stage II of the research programme was proposed to include an investigation into the actual performance of existing footings in regions where the founding soils are considered to be expansive.

To effectively monitor the movement of the foundations of a building, the "at-construction flatness" of the floor slab needs to be established.

The aim of such an investigation would be to establish the performance of previously accepted footing details in specific regions throughout Auckland thus allowing performance to be determined for buildings designed and constructed in accordance with the various requirements of NZS 3604.

Walsh et al (2001) reported on a testing programme to measure the deviation from level of 89 residential concrete slab-on-grade floors in Phoenix, Arizona within a few days of concrete placement. Based on the assumption that placement techniques and quality of workmanship was of a similar level for the 89 test sites, Walsh et al (2001) concluded that it could be assumed that the probability of a 10mm maximum elevation difference (ie range of largest deviations from flat) being as-built at time of concrete placement is 80%, whilst a probability of 20mm difference is only 6% ie if field measurements indicated that a slab has an elevation difference of up to 10mm then it would be more likely to be as a result of placement techniques, however, if a slab has a difference of 20mm then it is more likely to be experiencing some heave.

13.0 CONCLUSIONS

On the basis of the limitations and analyses reported here, the following is concluded:

(a) AS 2870 procedures:

(i) AS2870 requires that the designer estimate the characteristic surface movement, y_s, in order to determine the level of expansivity to be used in foundation design.

(ii) Based on the Thornthwaite Moisture Index (TMI) value of +50 for Auckland Airport, it is apparent that Auckland falls into the wet coastal/alpine climatic category (TMI >40) defined by AS 2870, for which a depth of moisture change (H_s) value of 1.5 m is proposed in Appendix D of AS2870 for Melbourne and Victoria and by Fityus et al (1998).

(iii) AS 2870 relies on the Australian Loading Standard – AS 1170 Part 1, in terms of:

(a) Limit state design philosophy being applied
(b) 95% characteristic values of design actions and member resistances being used
(c) Loads, combinations and load factors being consistently applied.

The requirements of AS 1170 Part 1 are generally similar to those prescribed in the New Zealand limit state Standard NZS 4203:1992 "Code of Practice for General Structural Design and Design Loadings for Buildings" and the related materials standards, eg NZS 3101 "Code of Practice for the Design of Concrete Structures".

(iv) Table 2.4 of AS2870 provides a recommended H_s value of 1.5 m for Newcastle and Sydney and 1.5 m or 1.8 m for eastern and coastal Victoria. The value of H_s of 2.0 m for Auckland recommended in Section 10.2 in this report, approximates the upper end of the range of 1.5 m to 1.8 m.

(b) Results of field and laboratory investigations for this study

(i) The Atterberg limits for the soils from Sites A to H generally plot below, or slightly above, the "A line" and are of high plasticity with liquid limit values ranging from 50 to 104. The linear shrinkage values of the soils range from 12% to 23%.

(ii) The shrinkage index for the soil samples from Sites A to H range from 1.12% to 6.38% with a mean value of 3.29%. These shrinkage index values are comparable to those reported by Fityus et al (1998) for soils in the Newcastle/Hunter Valley region and by Coffey and Partners (1985) for the Sydney region.

(iii) There is no discernable correlation between the shrinkage index and Atterberg limits or linear shrinkage for the soils from Sites A to H. In particular, no correlation is apparent between the shrinkage index and linear shrinkage or liquid limit, which indicates that the linear shrinkage and liquid limit values do not provide a reliable measure of soil expansivity.

(iv) The lack of correlation between the shrinkage index and the linear shrinkage or liquid limit is not unexpected, given that the shrinkage index relates to "undisturbed" samples while the other two classification parameters relate to fully remoulded samples.

(v) Until further research has been undertaken to establish the change in soil suction profiles and ground surface movement that occurs between design wet winter and dry summer conditions, it is suggested that y_s for Auckland sites be calculated on the basis of a triangular suction change profile, having an H_s value of 2.0 m and Δu at the ground surface of 1.5 pF and a crack zone depth of 0.5 H_s. This value of 2.0m approximates the upper end of the range discussed in the foregoing item (a)(iii).

(vi) If the suggested design Suction Change Profile Alpha is adopted, being a triangular profile with an H_s value of 2.0 m and Δu value at the ground surface of 1.5 pF, Sites A to H generally fall into the M and H classifications, whereas if the Soil Suction Change Profile Gamma derived by using the Auckland data to obtain the lowest values from the corresponding Australian regions using the criteria of AS2870 is adopted, being a triangular profile with an H_s value of 1.5 m and Δu value at the ground surface of 1.2 pF, Sites A to H generally fall into or close to the M classification.

(vii) The validity of the assumption of a linear correlation between the change in soil suction at 0.5 m depth for each site and the corresponding values of daily soil moisture deficit, in order to allow the measured Δu profile to be extrapolated to derive a Δu value at the ground surface for assumed drought conditions, requires to be verified.

(viii) The calculated y_s values corresponding to Soil Suction Change Profile Beta are between 64% to 79% of the corresponding values for Soil Suction Change Profile Alpha.

(ix) The calculated y_s values corresponding to Soil Suction Change Profile Gamma are between 53% to 64% of the corresponding values for Soil Suction Change Profile Alpha.

(x) The calculated y_s values for the measured Soil Suction Change Profile Delta, with an H_s value of 2.0 m, are between 5% and 55% of the corresponding values for Soil Suction Change Profile Alpha. However, it should be noted that the summers of 2002 and 2003 were not representative of drought conditions and higher soil suction change values would be expected at the end of a drought period, which would result in higher calculated y_s values.

(xi) The maximum soil suction change between the winter 2002 measurements and either the summer 2002 or 2003 measurements, normalised to a base of 3.2pF, was 0.3 pF or less for all sites except Sites D and H, for which a maximum value of 0.4 pF was recorded.

(xii) No significant change in suction occurred below approximately 2.0 m depth except for Sites B and H. However, it should be noted that, due to soil confining pressures, the change in suction values at depths below 1.5 m to 2.0 m depth may not necessarily result in any significant ground movement.

(xiii) Further monitoring would be required in order to determine an appropriate H_s value for Site H, which is underlain by Onerahi Chaos deposits.

(c) Applicability of AS 2870 to Auckland

(i) The method of site classification in terms of soil expansivity, involving estimation of the characteristic surface movement (y_s), in accordance with AS 2870, appears to be applicable to Auckland sites.

(ii) The soil shrinkage index (I_{ps}) is able to be determined by means of the core shrinkage test method given in AS 1289 Test Method 7.1.3 and referred to in AS 2870.

(iii) No correlation was apparent between I_{ps} and other soil classification tests, such as liquid limit, plasticity index and linear shrinkage.

(iv) To calculate the characteristic surface movement (y_s) for a site, the instability index (I_{pt}) requires to the calculated, which in turn requires the depth of cracked zone (the zone in which predominantly vertical shrinkage cracks exist seasonally) to be estimated. Investigation of the depth of the cracked zone was beyond the scope of the study reported herein. However, since the value of I_{pt} is sensitive to the estimated depth of the cracked zone, as indicated by the equations given in the foregoing Section 5.4.2, it follows that y_s is also sensitive to the estimated depth of the cracked zone.

(v) Investigation into the depth of the seasonal cracked zone in Auckland soils is required to provide design values for determination of the instability index (I_{pt}) and to enable reliable estimates of y_s to be made.

(vi) The following can be concluded with respect to the applicability of foundation details in NZS 3604, for use on Auckland sites:

(a) Type 1 foundations are unsuitable for construction on expansive soil sites in the Auckland region without modification

(b) Foundations where the floor slab is fully integrated with the concrete masonry blockwork foundation wall to provide structural performance similar to that of Type 2A foundations, also require modification from the details provided in NZS 3604 to be suitable for construction on Class M expansive soils sites in the Auckland region

(c) Foundation Type 2C requires no modification from the detailing specified in NZS 3604 to be suitable for construction on Class M expansive soils sites in the Auckland region where the beam depth is at least 750mm. For reduced beam depth additional steel reinforcement will be required, as indicated in Table 20 for Foundation Type 2B.

(vii) The analyses of the AS 2870's "standard designs" of stiffened raft and waffle slab construction types using specialist (SLOG) software indicates that the AS 2870 "standard designs" are generally suitable for single-storey construction.

There appears to have to be an "experience factor" used in the design of standard footings based on the historical knowledge and judgement of Australian engineers in the preparation of some of the standard designs specified in AS 2870. In this case the AS 2870 specified reinforcing is adequate for a target depth of 400mm

(viii) Section 2.2 of AS 2870 permits consideration of 10 years (or more) of satisfactory field performance as providing guidance for the deign of foundations on expansive soils.

The angular distortions which occur in essentially brittle cladding, eg masonry veneer can be used to derive "permissible y_s values" at particular sites where particular damage levels have been observed in that cladding.

Back analyses undertaken using the Type 2A foundations, from the foregoing Table 19 and Figure 13, have confirmed that Type 2A foundations with an embedment depth of 450mm supporting clad frame, articulated masonry veneer and masonry veneer buildings, perform satisfactorily in Auckland.

(d) Design/interpretation procedure recommended for Auckland until such time as the results of further studies are available:

(i) That the soil expansivity of any particular site be determined by taking core samples at, say, 0.5m to 0.8m depth and 1.0m to 1.3m depth below the ground surface and measuring the soil shrinkage index (I_{ps}) by means of either the shrink-swell index or core shrinkage index methods given in AS 1289 Test Methods 7.1.1 or 7.1.3.

(ii) That the instability index (I_{pt}) be calculated using the equations given in the foregoing Section 5.4.2, which are taken from Appendix F of AS 2870, and assuming a cracked zone depth equivalent to $0.5H_s$.

(iii) That the characteristic surface movement (y_s) be calculated using the equation given in the foregoing Section 5.2, which is taken from AS 2870, and on the basis of a triangular suction profile, having an H_s value of 2.0m and Δu at the ground surface of 1.5 pF, and an I_{pt} value calculated as discussed in item (d)(ii) above.

(iv) That the site classification be determined on the basis of y_s, using Table 2.3 of AS 2870.

(e) Limitations on research

(i) As the study period did not include a dry summer or drought period, it was not possible to measure the soil suction profiles at Sites A to H that relate to such dry weather conditions, in order to determine the corresponding soil suction change profile that is required for calculation of the characteristics surface movement (y_s).

(ii) It was therefore necessary to extrapolate the actual soil suction profile for Sites A to H corresponding to the driest of the 2002 and 2003 summers in order to derive a suggested design soil suction change profile for the Auckland region. The extrapolation was based on the assumption that there is a linear correlation between the daily soil moisture deficit and soil suction at 0.5m depth, as discussed in Section 10.3.

14.0 RECOMMENDATIONS FOR FURTHER RESEARCH

To develop the understanding of expansive soils within the Auckland region the following further work is recommended:

(a) The work previously recommended as a Stage II study comprising the following:

(i) Building damage survey, as discussed in the foregoing Section 2.5, to ascertain the actual foundation performance of a prescribed number of dwellings within Manukau City, which were constructed with concrete ground bearing slabs and perimeter footings complying with Appendix E of NZS 3604:1990 "Code of Practice for Light Timber Framed Buildings not requiring specific design".

The number of dwellings would be dependent on the availability of suitable building developments within representative soil type zones. The recommended building damage survey would allow site classification on the basis of the method outlined in Section 2.2.2 of AS 2870, for comparison with site classification based on the alternative method, involving estimation of the characteristic surface movement (y_s) as outlined in Section 2.2.3 of AS 2870.

The output of the building damage survey would be a series of figures similar to Figure 14 for a range of building sizes to be used by designers and local authorities to determine the suitability of NZS 3604 specified footings or the need to use AS 2870 specified standard or specific designs.

(ii) Geotechnical investigation to determine the soil profiles, soil properties and seasonal soil suction profiles at each of the sites selected for the foregoing building damage survey.

(iii) Installation of extensometers, as discussed in the foregoing Section 2.4, to monitor the monthly soil movements at sites selected for the foregoing building damage survey. This would provide a correlation between the ground movement calculated from the measured soil suction and the actual ground movements measured on site.

(b) Additional work at Sites A to H, over a number of years, to resolve limitations in the work carried out for this study:

(i) Taking additional soil suction samples at Sites A to H at the end of a representative dry summer period to confirm the design soil suction change profile for each site.

(ii) Installation of extensometers at Sites A to H to allow correlation between the ground movements calculated from the soil suction profiles and shrinkage index data and the actual measured movements.

(iii) Measurement of the soil suction profiles at Sites A to H at the same time as the ground movements are measured in the extensometers.

(iv) Excavation of test pits at selected sites at the end of a representative dry summer period, in order to determine the depth of seasonal shrinkage cracks, required to determine the Instability Index (I_{pt}).

(v) Verification of an appropriate means of determining the design dry moisture condition.

(c) Length of time to develop database

Given that AS 2870 refers to the validity of a ten-year experience record to evaluate the performance of existing structures, it is recommended that the length of time to develop the database should record results for at least five years before the preliminary recommendations of this report are modified, but otherwise should not be less than ten years.

(d) Proposed form of reporting by geotechnical practitioners.

The authors recommend that every residential land subdivision site should be subject to a Geotechnical Investigation Report in support of a Subdivision Consent, which provides for the reactivity classification of the development site as follows:

(i) Site classification in accordance with the processes set out in AS 2870, and reporting of the parameters on which this classification is made.

(ii) The design characteristic surface movement, y_s, value calculated using Soil Suction Profile Alpha, as discussed in the foregoing Section 11.2 of this report.

(iii) Recommendations for reassessments of the Site Classification in the Geotechnical Completion report on the completion of land development and at the Building Consent stage to reflect any variation in classification arising from land development earthworks or building site works.

(iv) Recommendations on foundation types and embedment depth for particular building construction types.

These requirements will provide the building designer with the information required to design the building's foundations and provide the territorial local authorities with the information upon which to build a database for their region.

Appendix A

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Appendix B

Borehole Logs

HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. A

PROJECT.	BRANZ - EXPANSIVE SOILS SITE A - AUCKLAND DOMAIN PARNELL	CO-ORDINATES	E N
		GROUND LEVEL	DATUM
PROJECT NO.	46635	Date Drilled 05.03.02	Logged by V. Toy Checked

DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC LOG	SAMPLE TYPE	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377 X Shear Vane ◯ Residual Shear Vane 50 100 150 200	WATER CONTENT (%) W_p W_f W_l X —●— l 20 40 60 80	WATER CONTENT (%)	TESTING AND COMMENTS
0.0	SILT, brown, friable, dry [TOPSOIL]						
	SILT, slightly clayey, darkish orange, slightly plastic, very stiff, moist [BASALTIC ASH]						
0.5	becomes mottled dark orange						
	occasional fragments of charred wood						
	becomes streaked cream, slightly sandy (coarse-grained, pumiceous)						
1.0	becomes mottled cream						
	becomes moist, dark orange-brown streaked cream and black, occasional gravels (scoria)						
	SILT, slightly sandy (fine-grained), light orange mottled pink, cream and brown, streaked black, non-plastic, stiff, moist						
1.5	becomes predominantly pinkish-brown, non-sandy						
	becomes predominantly light brown mottled cream, streaked black						
2.0	becomes mottled black, sandy (coarse-grained, pumiceous)						
	becomes light pinkish-brown streaked medium brown						
	becomes predominantly medium brown. Occasional fine black gravels (scoria)						
	becomes lighter brown streaked pinkish orange, red and black						
2.5	becomes cream-brown mottled brown, streaked with black scoriaceous gravels and cream pumiceous gravels						
	becomes sandy (coarse-grained, pumiceous)						
	becomes predominantly grey-brown mottled black, very moist						
3.0	EOB @ 3.0 m TARGET DEPTH						
3.5							

REMARKS: 1. Borehole dry on 05.03.02

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- RESOURCE MANAGERS
- ENVIRONMENTAL CONSULTANTS
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HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. B

PROJECT: BRANZ - EXPANSIVE SOILS SITE B - EAST TAMAKI DOMAIN OTARA	CO-ORDINATES E N
	GROUND LEVEL DATUM
PROJECT NO. 46635	Date Drilled 12.03.02 Logged by V. Toy Checked

DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC LOG	SAMPLE TYPE	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377 X Shear Vane ◯ Residual Shear Vane 50 100 150 200	WATER CONTENT (%) W_p W_f W_l 20 40 60 80	WATER CONTENT (%)	TESTING AND COMMENTS
0.0	[TOPSOIL] SILT, brown, friable, slightly moist						
	SILT, dark orange-brown, slightly plastic, very stiff, slightly moist [BASALTIC ASH]						
0.5	becomes light orange-brown						
	becomes light creamy yellow						
1.0	CLAY, slightly sandy (fine-grained, pumiceous), cream streaked yellow, moderately plastic, very stiff, moist [TAURANGA GROUP - tpp]						
1.5	becomes less sandy grey mottled cream-brown streaked pink						
2.0	occasional fine gravels (pumice fragments)						
	SILT, slightly sandy (fine grained, pumice), cream streaked pink, moderately plastic, stiff, moist						
2.5							
	EOB @ 2.7 m TARGET DEPTH						
3.0							
3.5							

REMARKS: 1. Borehole dry on 12.03.02

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- ENVIRONMENTAL CONSULTANTS
- SURVEYORS & PLANNERS

HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. C

PROJECT. BRANZ - EXPANSIVE SOILS
SITE C - WEST ROAD
BROOKBY

PROJECT NO 46635

CO-ORDINATES E N

GROUND LEVEL DATUM

Date Drilled 21.03.02 Logged by V. Toy Checked

DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC LOG	SAMPLE TYPE	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377 X Shear Vane ◯ Residual Shear Vane 50 100 150 200	WATER CONTENT (%) W_p W_f W_l 20 40 60 80	WATER CONTENT (%)	TESTING AND COMMENTS
0.0	[TOPSOIL] SILT, brown, friable, dry						
	CLAY, silty, light grey yellow, slightly plastic, very stiff, slightly moist [WAIPAPA GROUP]						
0.5	becomes less silty, light brownish yellow						
1.0	becomes mottled orange, more silty						
	becomes cream mottled orange						
1.5							
2.0							
	becomes slightly gravelly (siltstone fragments)						
2.5							
	becomes streaked pinkish brown						
	EOB @ 2.8 m TARGET DEPTH						
3.0							
3.5							

REMARKS: 1. Borehole dry on 21.03.02

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HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. D

PROJECT. BRANZ - EXPANSIVE SOILS SITE D - ALFRISTON ROAD MANUREWA	CO-ORDINATES E N
	GROUND LEVEL DATUM
PROJECT NO. 46635	Date Drilled 14.02.02 Logged by V. Toy Checked

DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC LOG	SAMPLE TYPE	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377 X Shear Vane ◯ Residual Shear Vane (50, 100, 150, 200)	WATER CONTENT (%) W_p W_f W_l X —●— I (20, 40, 60, 80)	WATER CONTENT (%)	TESTING AND COMMENTS
0.0	[TOPSOIL] SILT, brown, friable, moist						
	CLAY, silty, yellow mottled brown, slightly plastic, very stiff, moist [TAURANGA GROUP - tp]						
	becomes slightly sandy (fine grained)						
0.5	no longer sandy						
	becomes grey mottled yellow, moderately plastic						
1.0							
	becomes predominantly grey						
1.5	becomes slightly sandy (fine grained)						
2.0	becomes mottled yellow and pink						
	becomes more sandy						
2.5							
3.0	EOB @ 3.0 m TARGET DEPTH						
3.5							

REMARKS: 1. Borehole dry on 14.02.02

Fraser Thomas

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- SURVEYORS & PLANNERS

HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. E

PROJECT: BRANZ - EXPANSIVE SOILS
SITE E - STARLING PARK
SWANSON

PROJECT NO. 46635

CO-ORDINATES E N

GROUND LEVEL DATUM

Date Drilled 14.03.02 Logged by V. Toy Checked

DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC LOG	SAMPLE TYPE	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377 X Shear Vane ◯ Residual Shear Vane 50 100 150 200	WATER CONTENT (%) W_p W_f W_l 20 40 60 80	WATER CONTENT (%)	TESTING AND COMMENTS
0.0	[TOPSOIL] SILT, brown, friable, moist						
	SILT, very clayey, orange-brown, slightly plastic, stiff, moist [TAURANGA GROUP - tp]						
0.5	SILT, slightly sandy (fine grained), light cream-brown, slightly plastic, very stiff, moist becomes mottled orange						
1.0	CLAY, slightly silty, light yellow-orange streaked cream, moderately plastic, very stiff, moist. Occasional fine grained pumiceous sands						
1.5							
2.0	CLAY, silty, light yellow mottled cream, streaked red, moderately plastic, very stiff, moist. becomes very plastic, slightly sandy (fine grained, pumiceous)						
2.5	SILT, clayey, yellow mottled brown, moderately plastic, very stiff, moist						
	EOB @ 2.6 m TARGET DEPTH						
3.0							
3.5							

REMARKS: 1. G.W.L @ 2.5 m on 14.03.02

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HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. F

PROJECT. BRANZ - EXPANSIVE SOILS SITE F - HOWICK COMM. CENTRE PAKURANGA	CO-ORDINATES E N
	GROUND LEVEL DATUM
PROJECT NO. 46635	Date Drilled 21.03.02 Logged by V. Toy Checked

DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC LOG	SAMPLE TYPE	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377; X Shear Vane; ○ Residual Shear Vane; 50 100 150 200	WATER CONTENT (%) W_p W_f W_l; 20 40 60 80	WATER CONTENT (%)	TESTING AND COMMENTS
0.0	[TOPSOIL/FILL] SILT, brown, friable, slightly moist. Occasional gravels.						
0.5	CLAY, silty, yellow mottled light brown streaked cream, moderately plastic, very stiff slightly moist [WAITEMATA GROUP]						
1.0	SILT, cream mottled yellow and pink, slightly plastic, very stiff, slightly moist						
	becomes slightly clayey, yellow						
1.5	no longer clayey, becomes pink mottled cream. Occasional fine gravels (siltstone fragments).						
2.0	becomes clayey, yellow						
	no longer clayey, pink mottled cream. Occasional fine gravels (siltstone fragments)						
2.5	becomes clayey, mottled yellow						
	EOB @ 2.6 m TARGET DEPTH						
3.0							
3.5							

REMARKS: 1. Borehole dry on 21.03.02

Fraser Thomas

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- SURVEYORS & PLANNERS

HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. G

PROJECT. BRANZ - EXPANSIVE SOILS
SITE G - BIRKENHEAD DOMAIN
BIRKDALE

PROJECT NO. 46635

CO-ORDINATES E N

GROUND LEVEL DATUM

Date Drilled 08.03.02 Logged by V. Toy Checked

DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC LOG	SAMPLE TYPE	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377 X Shear Vane ◯ Residual Shear Vane (50, 100, 150, 200)	WATER CONTENT (%) W_p W_f W_l X —●— I (20, 40, 60, 80)	WATER CONTENT (%)	TESTING AND COMMENTS
0.0	[TOPSOIL] SILT, sandy, brown, friable, moist						
	CLAY, yellow, very plastic, stiff, moist. [WAITEMATA GROUP] Contains frequent rootlets.						
0.5							
1.0							
	becomes mottled cream						
1.5							
2.0							
2.5	EOB @ 2.5 m TARGET DEPTH						
3.0							
3.5							

REMARKS: 1. Borehole dry on 08.03.02

Fraser Thomas

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- SURVEYORS & PLANNERS

HAND AUGER LOG | SHEET 1 OF 1 | BOREHOLE NO. H

PROJECT. BRANZ - EXPANSIVE SOILS SITE H - HIBISCUS VILLAGE WHANGAPAROA	CO-ORDINATES E N GROUND LEVEL DATUM
PROJECT NO. 46635	Date Drilled 09.03.02 Logged by V. Toy Checked

DEPTH (m)	DESCRIPTION OF STRATA	GRAPHIC LOG	SAMPLE TYPE	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377 X Shear Vane ◯ Residual Shear Vane 50 100 150 200	WATER CONTENT (%) W_p W_f W_l X —●— I 20 40 60 80	WATER CONTENT (%)	TESTING AND COMMENTS
0.0	[TOPSOIL] SILT, brown, friable, moist						
	SILT, clayey, blue-grey mottled brown, streaked yellow, moderately plastic, stiff, moist [ONERAHI CHAOS BRECCIA]						
0.5							
	CLAY, silty, yellow mottled cream and orange, streaked dark green, moderately plastic, stiff, moist						
	becomes mottled yellow-orange, slightly plastic, very stiff						
1.0	patches of yellow-brown silt and occasional sand (fine grained)						
	becomes cream mottled yellow						
1.5							
2.0							
	SILT, clayey, sandy (fine grained) light grey mottled yellow, slightly plastic very stiff moist						
2.5	SILT/CLAY, dark green mottled pink, brown and yellow, moderately plastic, very stiff, moist						
	becomes dark green mottled cream and light grey						
3.0	EOB @ 3.0 m TARGET DEPTH						
3.5							

REMARKS: 1. Borehole dry on 09.03.02

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- RESOURCE MANAGERS
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- SURVEYORS & PLANNERS

Appendix C

Site Summary Sheets

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference A
Location Newmarket
Main Soil Type Basaltic Ash

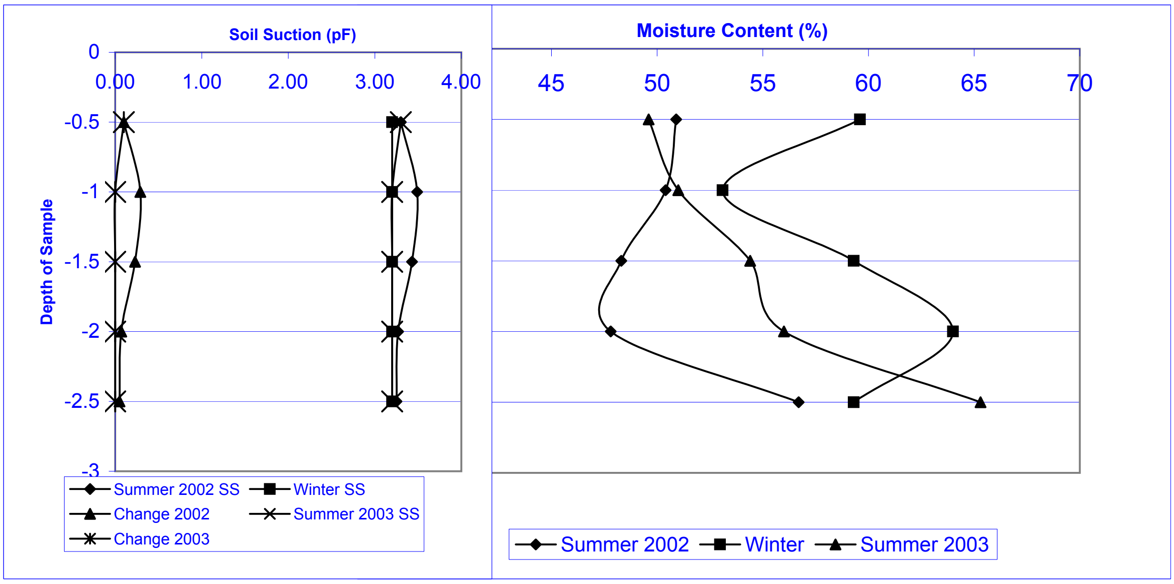
1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

	Summer 2002		Winter 2002		Summer 2003	
Date of Sampling	20-Mar-02		4-Sep-02		19-Mar-03	
Deficit Reading at Sampling (mm)	-24.4		-7.0		-16.5	
Date of Testing	27&28-Mar-02, 5,9&15-May-02		20 & 27-Sep-02		1-3&15-Apr-03	
Borehole Log No.	A2		A3		A4	
Depth to Groundwater Level (m)	Not Encountered		Not Encountered		Not Encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.5m below GL	50.9	3.30	59.6	3.2	49.6	3.30
Sample at 1.0m below GL	50.4	3.49	53.1	3.2	51	3.2
Sample at 1.5m below GL	48.3	3.43	59.3	3.2	54.4	3.2
Sample at 2.0m below GL	47.8	3.27	64	3.2	56	3.2
Sample at 2.5m below GL	56.7	3.25	59.3	3.2	65.3	3.2

2.0 SOIL CLASSIFICATION TESTING UNDERTAKEN DURING WINTER 2002

	Atterburg Limits			Linear Shrinkage (%)	Shrinkage Index (%)
	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index		
Sample at 0.5m below GL	50	104	54	20	3.25
Sample at 1.0m below GL	54	87	33	19	2.17
Sample at 1.5m below GL	60	75	16	13	1.12

Graphing of Field Data



3.0 CALCULATION OF SURFACE MOVEMENT USING ASSUMED SOIL SUCTION PROFILE ALPHA

Depth (m)	Ips (%)	alpha	Ipt (%)	delta u (pF)	avg delta u (pF)	delta z (m)	surface movement (mm)
0		1.00		1.5			
0.35	1.63	1.00	1.63	1.2375	1.36875	0.35	7.78
0.35		1.00		1.2375			
0.75	3.25	1.00	3.25	0.9375	1.0875	0.4	14.14
0.75		1.00		0.9375			
1	2.17	1.00	2.17	0.75	0.84375	0.25	4.58
1		1.80		0.75			
1.5	2.17	1.70	3.80	0.375	0.5625	0.5	10.68
1.5		1.70		0.375			
2	1.12	1.60	1.85	0	0.1875	0.5	1.73

Total Surface Movement (mm) 38.91 mm
say 40 mm

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference B
Location East Tamaki
Main Soil Type Basaltic Ash/Tauranga Group (tpp)

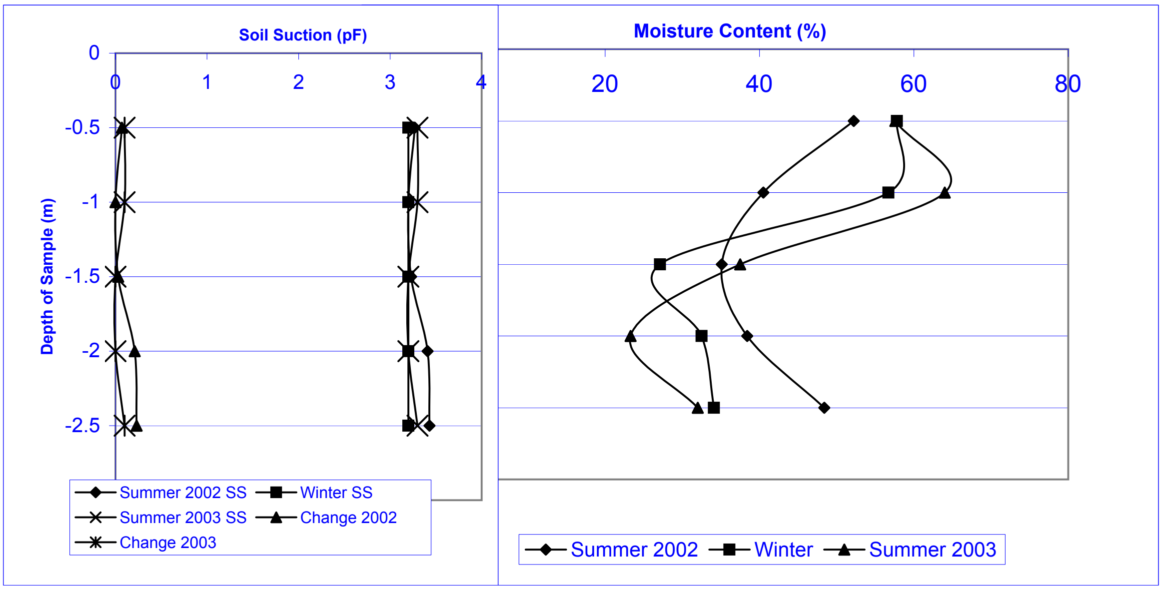
1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

	Summer 2002		Winter 2002		Summer 2003	
Date of Sampling	19-Mar-02		4-Sep-02		20-Mar-03	
Deficit Reading at Sampling (mm)	-22.0		-7.0		-18.9	
Date of Testing	27-Mar-02 & 15-May-02		28-Aug-02 & 23-Sep-02		3,4,8&15-Apr-03	
Borehole Log No.	B2		B3		B4	
Depth to Groundwater Level (m)	Not Encountered		1.5m		Not Encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.5m below GL	52.2	3.27	57.8	3.2	57.6	3.3
Sample at 1.0m below GL	40.5	3.2	56.7	3.2	64	3.3
Sample at 1.5m below GL	35.1	3.23	27.1	3.2	37.5	3.2
Sample at 2.0m below GL	38.4	3.41	32.5	3.2	23.3	3.2
Sample at 2.5m below GL	48.4	3.43	34.1	3.2	32	3.3

2.0 SOIL CLASSIFICATION TESTING UNDERTAKEN DURING WINTER 2002

	Atterburg Limits			Linear Shrinkage (%)	Shrinkage Index (%)
	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index		
Sample at 0.5m below GL	54	100	45	20	2.76
Sample at 1.0m below GL	67	94	27	18	3.43
Sample at 1.5m below GL	25	55	30	13	2.68

Graphing of Field Data



3.0 CALCULATION OF SURFACE MOVEMENT USING ASSUMED SOIL SUCTION PROFILE ALPHA

Depth (m)	Ips (%)	alpha	Ipt (%)	delta u (pF)	avg delta u (pF)	delta z (m)	surface movement (mm)
0		1.00		1.5			
0.3	1.38	1.00	1.38	1.275	1.3875	0.3	5.74
0.3		1.00		1.275			
1	2.76	1.00	2.76	0.75	1.0125	0.7	19.56
1		1.80		0.75			
1.1	3.43	1.78	6.14	0.675	0.7125	0.1	4.37
1.1		1.78		0.675			
2	2.68	1.60	4.53	0	0.3375	0.9	13.76

Total Surface Movement (mm) 43.44 mm
say 45 mm

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference C
Location Brookby
Main Soil Type Waipapa Group

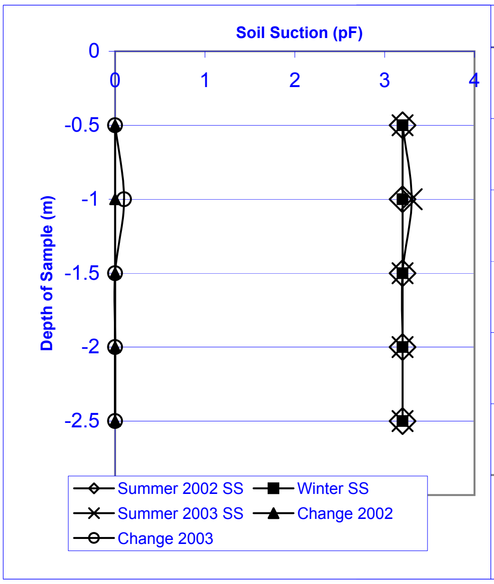
1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

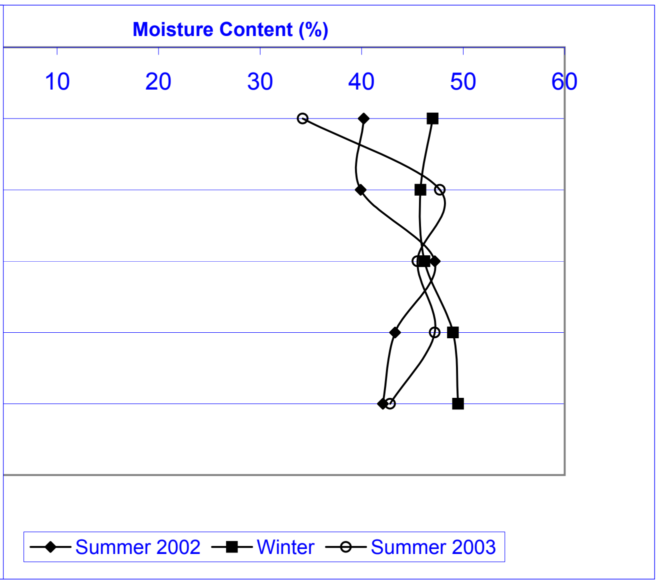
	Summer 2002		Winter 2002		Summer 2003	
Date of Sampling	21-Mar-02		15-Aug-02		10-Mar-03	
Deficit Reading at Sampling (mm)	-26.8		13.9		-40.2	
Date of Testing	21 & 22-May-02		19 & 26-Aug-02		21,26&27-Mar-03	
Borehole Log No.	C1		C2		C3	
Depth to Groundwater Level (m)	No Encountered		1.63		No Encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.5m below GL	40.2	3.2	47	3.2	34.2	3.2
Sample at 1.0m below GL	39.9	3.2	45.8	3.2	47.7	3.3
Sample at 1.5m below GL	47.2	3.2	46.2	3.2	45.5	3.2
Sample at 2.0m below GL	43.3	3.2	49	3.2	47.2	3.2
Sample at 2.5m below GL	42.1	3.2	49.5	3.2	42.8	3.2

2.0 SOIL CLASSIFICATION TESTING UNDERTAKEN DURING WINTER 2002

	Atterburg Limits			Linear Shrinkage (%)	Shrinkage Index (%)
	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index		
Sample at 0.5m below GL	35	68	33	13	3.69
Sample at 1.0m below GL	40	93	52	14	2.32
Sample at 1.5m below GL	37	64	28	16	5.18

Graphing of Field Data





3.0 CALCULATION OF SURFACE MOVEMENT USING ASSUMED SOIL SUCTION PROFILE ALPHA

Depth (m)	Ips (%)	alpha	Ipt (%)	delta u (pF)	avg delta u (pF)	delta z (m)	surface movement (mm)
0		1.00		1.5			
0.3	1.85	1.00	1.85	1.275	1.3875	0.3	7.68
0.3		1.00		1.275			
0.75	3.69	1.00	3.69	0.9375	1.10625	0.45	18.37
0.75		1.00		0.9375			
1	2.32	1.00	2.32	0.75	0.84375	0.25	4.89
1		1.80		0.75			
1.25	2.32	1.75	4.12	0.5625	0.65625	0.25	6.76
1.25		1.75		0.5625			
2	5.13	1.60	8.59	0	0.28125	0.75	18.13

Total Surface Movement (mm) 55.82 mm
say 60 mm

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference D
Location Manurewa
Main Soil Type Tauranga Group (tp)

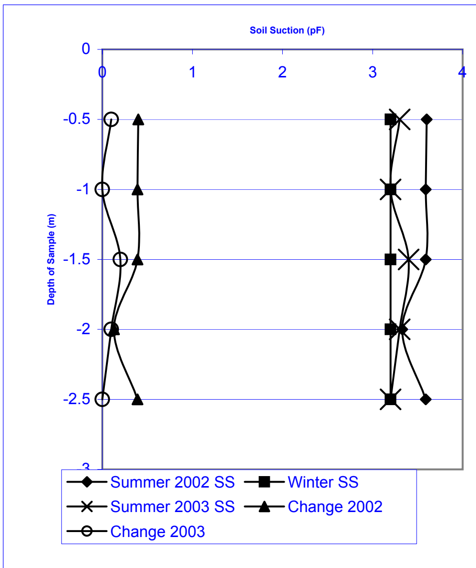
1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

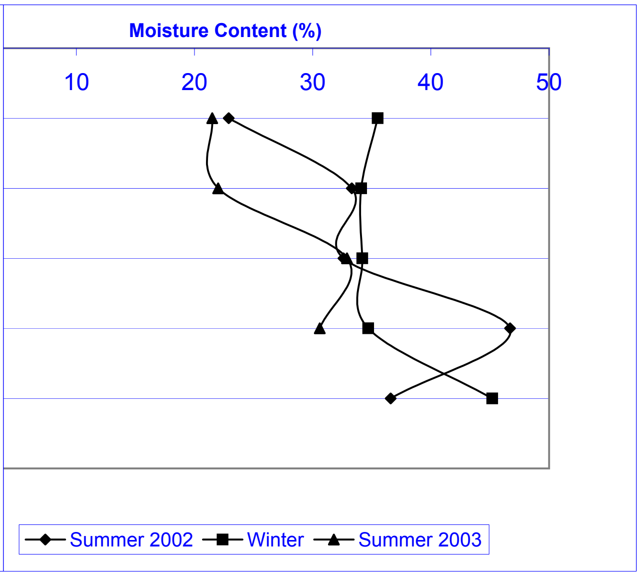
	Summer 2002		Winter 2002		Summer 2003	
Date of Sampling	20-Mar-02		30-Aug-02		10-Mar-03	
Deficit Reading at Sampling (mm)	-24.4		-5.7		-37.6	
Date of Testing	4-Apr-02 & 4-May-02		18-Sep-02 & 2-Oct-02		19&20-Mar-03 & 15-Apr-03	
Borehole Log No.	D2		D3		D4	
Depth to Groundwater Level (m)	Not Encountered		Not Encountered		Not Encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.5m below GL	22.9	3.6	35.5	3.2	21.5	3.3
Sample at 1.0m below GL	33.3	3.59	34.1	3.2	22	3.2
Sample at 1.5m below GL	32.6	3.59	34.2	3.2	32.9	3.4
Sample at 2.0m below GL	46.7	3.33	34.7	3.2	30.6	3.3
Sample at 2.5m below GL	36.6	3.59	45.2	3.2		3.2

2.0 SOIL CLASSIFICATION TESTING UNDERTAKEN DURING WINTER 2002

	Atterburg Limits			Linear Shrinkage (%)	Shrinkage Index (%)
	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index		
Sample at 0.5m below GL	27	56	29	15	4.59
Sample at 1.0m below GL	30	97	67	12	6.38
Sample at 1.5m below GL	27	93	66	16	6.18

Graphing of Field Data





3.0 CALCULATION OF SURFACE MOVEMENT USING ASSUMED SOIL SUCTION PROFILE ALPHA

Depth (m)	Ips (%)	alpha	Ipt (%)	delta u (pF)	avg delta u (pF)	delta z (m)	surface movement (mm)
0 0.2	2.30	1.00 1.00	2.30	1.5 1.35	1.425	0.2	6.54
0.2 0.75	4.59	1.00 1.00	4.59	1.35 0.9375	1.14375	0.55	28.87
0.75 1	6.38	1.00 1.00	6.38	0.9375 0.75	0.84375	0.25	13.46
1 1.25	6.38	1.80 1.75	11.32	0.75 0.5625	0.65625	0.25	18.58
1.25 2	6.18	1.75 1.60	10.35	0.5625 0	0.28125	0.75	21.84

Total Surface Movement (mm) 89.29 mm
say 90 mm

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference E
Location Swanson
Main Soil Type Tauranga Group - tp

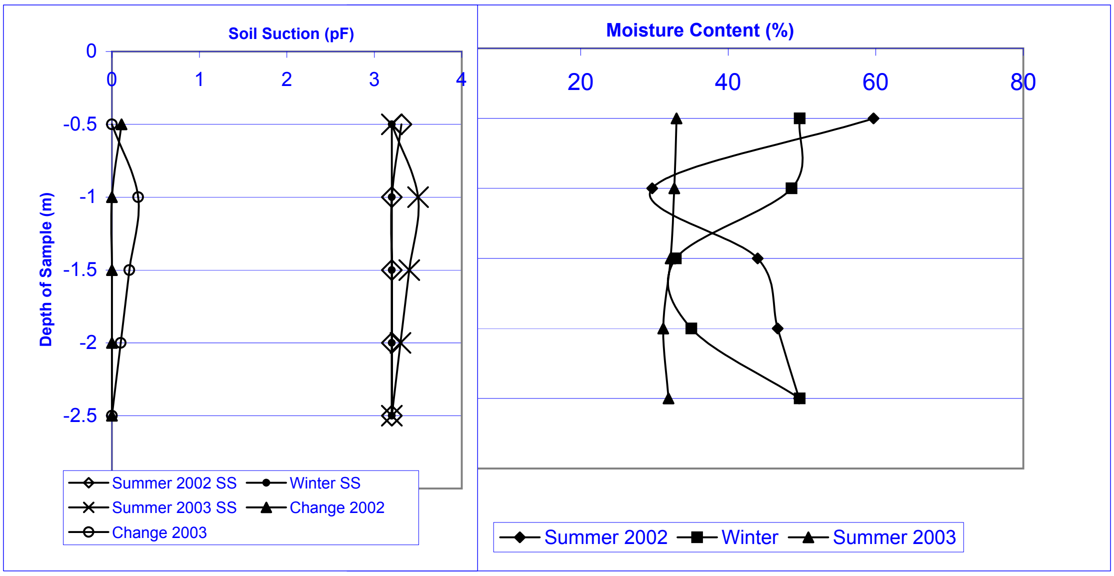
1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

	Summer 2002		Winter 2002		Summer 2003	
Date of Sampling	21-Mar-02		5-Sep-02		10-Mar-03	
Deficit Reading at Sampling (mm)	-31.4		-8.2		-40.2	
Date of Testing	23 & 24-May-02		19-Sep-02 & 1&2-Oct-02		14,18&27-Mar-03 & 17-Apr-03	
Borehole Log No.	E2		E3		E4	
Depth to Groundwater Level (m)	2.4		Not Encountered		Not Encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.5m below GL	59.7	3.31	49.7	3.2	33	3.2
Sample at 1.0m below GL	29.7	3.2	48.6	3.2	32.7	3.5
Sample at 1.5m below GL	44	3.2	32.9	3.2	32.2	3.4
Sample at 2.0m below GL	46.7	3.2	35	3.2	31.2	3.3
Sample at 2.5m below GL	49.6	3.2	49.7	3.2	31.9	3.2

2.0 SOIL CLASSIFICATION TESTING UNDERTAKEN DURING WINTER 2002

	Atterburg Limits			Linear Shrinkage	Shrinkage Index
	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index	(%)	(%)
Sample at 0.5m below GL	63	88	25	12	2.79
Sample at 1.0m below GL	24	50	27	18	1.67
Sample at 1.5m below GL	30	65	36	13	4.17

Graphing of Field Data



3.0 CALCULATION OF SURFACE MOVEMENT USING ASSUMED SOIL SUCTION PROFILE ALPHA

Depth (m)	Ips (%)	alpha	Ipt (%)	delta u (pF)	avg delta u (pF)	delta z (m)	surface movement (mm)
0 0.2	1.40	1.00 1.00	1.40	1.5 1.35	1.425	0.2	3.98
0.2 0.75	2.79	1.00 1.00	2.79	1.35 0.9375	1.14375	0.55	17.55
0.75 1	1.67	1.00 1.00	1.67	0.9375 0.75	0.84375	0.25	3.52
1 1.25	1.67	1.80 1.75	2.96	0.75 0.5625	0.65625	0.25	4.86
1.25 2	4.17	1.75 1.60	6.98	0.5625 0	0.28125	0.75	14.73

Total Surface Movement (mm) 44.65 mm
say 45 mm

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference F
Location Pakuranga
Main Soil Type Waitemata Group

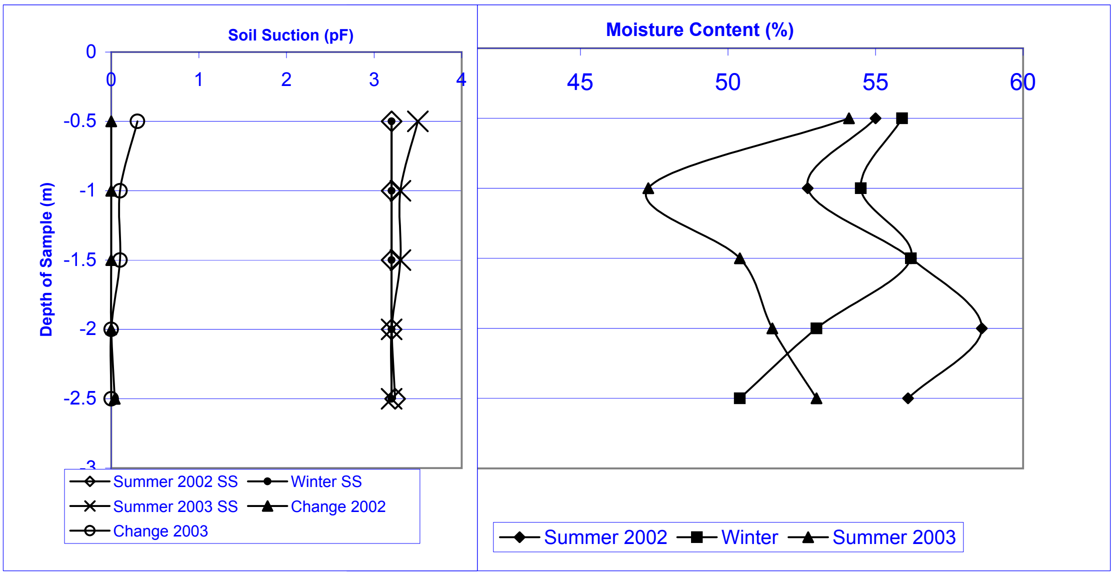
1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

	Summer 2002		Winter 2002		Summer 2003	
Date of Sampling	21-Mar-02		27-Aug-02		19-Mar-03	
Deficit Reading at Sampling (mm)	-31.4		-1.2		-16.5	
Date of Testing	22 & 24-Mar-2002		4 & 24-Sep-02		9-11 &17-Apr-03	
Borehole Log No.	F1		F2		F3	
Depth to Groundwater Level (m)	Not Encountered		2.6		Not Encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.5m below GL	55	3.2	55.9	3.2	54.1	3.5
Sample at 1.0m below GL	52.7	3.2	54.5	3.2	47.3	3.3
Sample at 1.5m below GL	56.2	3.2	56.2	3.2	50.4	3.3
Sample at 2.0m below GL	58.6	3.2	53	3.2	51.5	3.2
Sample at 2.5m below GL	56.1	3.24	50.4	3.2	53	3.2

2.0 SOIL CLASSIFICATION TESTING UNDERTAKEN DURING WINTER 2002

	Atterburg Limits			Linear Shrinkage (%)	Shrinkage Index (%)
	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index		
Sample at 0.5m below GL	52	87	35	16	3.12
Sample at 1.0m below GL	51	92	40	20	0.75
Sample at 1.5m below GL	50	89	40	20	1.07

Graphing of Field Data



3.0 CALCULATION OF SURFACE MOVEMENT USING ASSUMED SOIL SUCTION PROFILE ALPHA

Depth (m)	Ips (%)	alpha	Ipt (%)	delta u (pF)	avg delta u (pF)	delta z (m)	surface movement (mm)
0		1.00		1.5			
0.4	1.56	1.00	1.56	1.2	1.35	0.4	8.42
0.4		1.00		1.2			
0.7	3.12	1.00	3.12	0.975	1.0875	0.3	10.18
0.7		1.00		0.975			
1	0.75	1.00	0.75	0.75	0.8625	0.3	1.94
1		1.80		0.75			
1.25	0.75	1.75	1.33	0.5625	0.65625	0.25	2.18
1.25		1.75		0.5625			
2	1.07	1.60	1.79	0	0.28125	0.75	3.78

Total Surface Movement (mm) 26.51 mm
say 30 mm

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference G
Location Hillcrest
Main Soil Type Waitemata Group

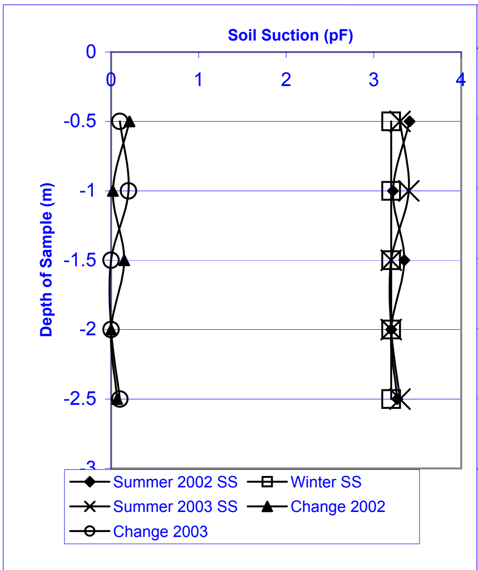
1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

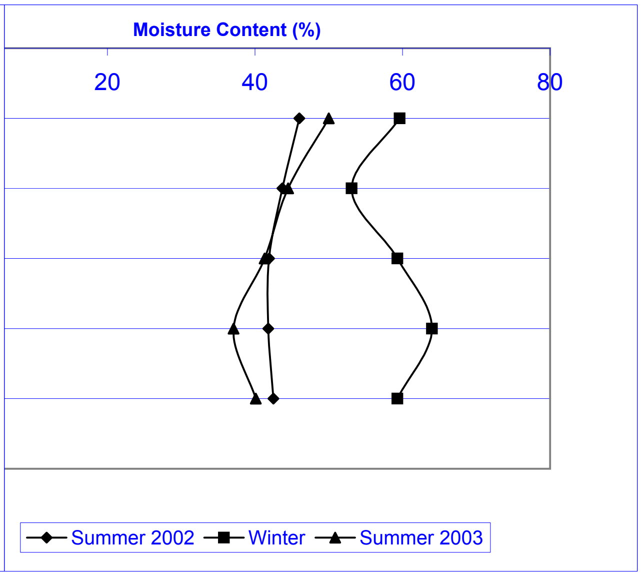
	Summer 2002		Winter 2002		Summer 2003	
Date of Sampling	21-Mar-02		17-Sep-02		20-Mar-03	
Deficit Reading at Sampling (mm)	-31.4		-5.0		-18.9	
Date of Testing	4-Apr-02 & 4-May-02		25 & 27-Sep-02		11, 26 & 29 Apr 03	
Borehole Log No.	G2		G3		G4	
Depth to Groundwater Level (m)	2		0.4		2.4	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.5m below GL	46	3.41	59.6	3.2	50	3.3
Sample at 1.0m below GL	43.7	3.22	53.1	3.2	44.5	3.4
Sample at 1.5m below GL	41.9	3.35	59.3	3.2	41.3	3.2
Sample at 2.0m below GL	41.8	3.2	64	3.2	37.1	3.2
Sample at 2.5m below GL	42.5	3.27	59.3	3.2	40.1	3.3

2.0 SOIL CLASSIFICATION TESTING UNDERTAKEN DURING WINTER 2002

	Atterburg Limits			Linear Shrinkage (%)	Shrinkage Index (%)
	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index		
Sample at 0.5m below GL	23	43	20	13	2.63
Sample at 1.0m below GL	42	99	57	17	4.24
Sample at 1.5m below GL	40	95	55	17	5.18

Graphing of Field Data





3.0 CALCULATION OF SURFACE MOVEMENT USING ASSUMED SOIL SUCTION PROFILE ALPHA

Depth (m)	Ips (%)	alpha	Ipt (%)	delta u (pF)	avg delta u (pF)	delta z (m)	surface movement (mm)
0		1.00		1.5			
0.3	1.32	1.00	1.32	1.275	1.3875	0.3	5.47
0.3		1.00		1.275			
0.75	2.63	1.00	2.63	0.9375	1.10625	0.45	13.09
0.75		1.00		0.9375			
1	4.24	1.00	4.24	0.75	0.84375	0.25	8.94
1		1.80		0.75			
1.25	4.24	1.75	7.53	0.5625	0.65625	0.25	12.35
1.25		1.75		0.5625			
2	5.18	1.60	8.68	0	0.28125	0.75	18.30

Total Surface Movement (mm) 58.16 mm
say 60 mm

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference H
Location Red Beach
Main Soil Type Onehari Chaos Breccia

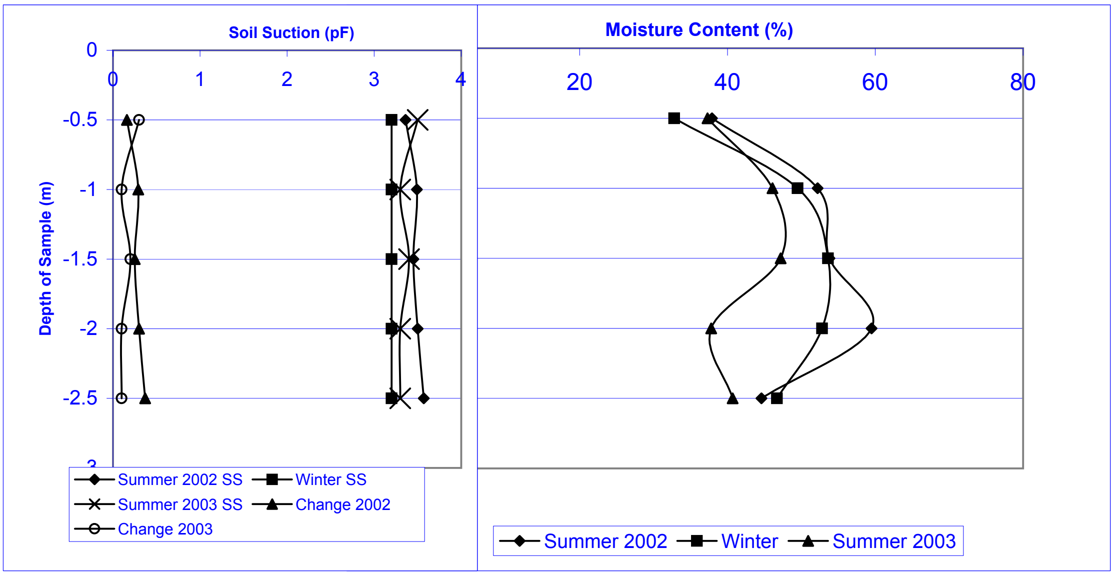
1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

	Summer 2002		Winter 2002		Summer 2003	
Date of Sampling	19-Mar-02		12-Sep-02		20-Mar-03	
Deficit Reading at Sampling (mm)	-22.0		-0.8		-18.9	
Date of Testing	28-Mar-02 & 4-May-02		26-Sep-02 & 1-Oct-02		30-Apr-03 & 1-May-03	
Borehole Log No.	H2		H3		H4	
Depth to Groundwater Level (m)	Not Encountered		Not Encountered		Not Encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.5m below GL	37.9	3.36	32.8	3.2	37.3	3.5
Sample at 1.0m below GL	52.2	3.49	49.5	3.2	46.1	3.3
Sample at 1.5m below GL	53.8	3.45	53.6	3.2	47.2	3.4
Sample at 2.0m below GL	59.5	3.5	52.8	3.2	37.8	3.3
Sample at 2.5m below GL	44.6	3.57	46.7	3.2	40.7	3.3

2.0 SOIL CLASSIFICATION TESTING UNDERTAKEN DURING WINTER 2002

	Atterburg Limits			Linear Shrinkage (%)	Shrinkage Index (%)
	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index		
Sample at 0.5m below GL	47	104	54	12	2.48
Sample at 1.0m below GL	48	87	33	21	3.51
Sample at 1.5m below GL	48	75	16	23	3.61

Graphing of Field Data



3.0 CALCULATION OF SURFACE MOVEMENT USING ASSUMED SOIL SUCTION PROFILE ALPHA

Depth (m)	Ips (%)	alpha	Ipt (%)	delta u (pF)	avg delta u (pF)	delta z (m)	surface movement (mm)
0		1.00		1.5			
0.1	1.24	1.00	1.24	1.425	1.4625	0.1	1.81
0.1		1.00		1.425			
0.6	2.48	1.00	2.48	1.05	1.2375	0.5	15.35
0.6		1.00		1.05			
1	3.51	1.00	3.51	0.75	0.9	0.4	12.64
1		1.80		0.75			
1.25	3.51	1.75	6.23	0.5625	0.65625	0.25	10.22
1.25		1.75		0.5625			
2	3.61	1.60	6.05	0	0.28125	0.75	12.75

Total Surface Movement (mm) 52.77 mm
say 55 mm