BRANZ

ADDENDUM STUDY REPORT

No. 120A (2008)

SOIL EXPANSIVITY IN THE AUCKLAND REGION

Fraser Thomas Ltd
(B.J. Brown, J.P.M. Shorten,
D.N.R. Dravitzki, P.R. Goldsmith)

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Reference

Fraser Thomas Ltd, Brown BJ, Shorten JPM, Dravitzki DNR and Goldsmith PR. 2008. 'Soil Expansivity in the Auckland Region'. BRANZ *Study Report 120A.* BRANZ Ltd, Judgeford, New Zealand.

Preface

This addendum report provides further assessment on the expansivity of soils at six sites within the Auckland region and considers the applicability of the design methodology set out in AS 2870:1996 *Residential Slabs and Footings – Construction* for buildings constructed in accordance with NZS 3604:1999 *Timber Framed Buildings*.

Acknowledgments

This work was jointly funded by the Building Research Levy and and Manukau City Council, Rodney District Council, North Shore City Council, Franklin District Council and Auckland City Council.

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

This document draws on and incorporates work undertaken within the preceding Stage I project, whose authors were Mr Barry J Brown, Dr Peter R Goldsmith, Mr J Patrick M Shorten and Mrs Leanne Henderson.

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.

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APPENDICES

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 of 450 mm below cleared ground levels was specified in NZS 3604:1990 *Code of Practice for Light Timber Framed Buildings Not Requiring Specific Design*, 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 specifically 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 it is suggested that the designer refer to the Australian Standard AS 2870:1996 *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.

Traditionally, the founding depth of 450 mm below cleared ground level has been the benchmark for residential building construction in New Zealand for buildings supported on conventional shallow foundations. There is, however, a move in recent years towards waffle or rib-raft slab construction for residential buildings, which are founded at the ground surface and are "stiffened" to AS 2870 standards according to the site expansive soil classification.

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 characteristic 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.

Recognising the issues surrounding the conditions imposed by the 1999 edition of NZS 3604, the Building Research Association of New Zealand (BRANZ) and the Manukau City Council jointly funded an investigation of the expansive characteristics of soils in the Auckland region. A report was produced by Fraser Thomas Ltd entitled BRANZ *Study Report 120* (2003) 'Soil Expansivity in the Auckland Region'.

This current report has been prepared as an addendum report to the 2003 Study Report and follows the same methodology presented in this, and should therefore be read in conjunction with it. Some sections of the 2003 Study Report have been incorporated in the current report for ease of reference.

The research reported herein has involved field investigations and laboratory testing at six locations within the Auckland region, referred to as Sites 2A to 2F, between July 2004 and April 2006, and included two summer and two winter seasons. A pattern of extensometers at six different depths and two surface monuments have been installed at each of the six sites to measure the range of ground movements occurring between summer and winter periods within the Auckland region.

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

As an additional part of the investigation into the expansive characteristics of soils in the Auckland region, a review was undertaken of the geotechnical reports held by the local territorial authorities (TAs) with the largest areas of recent and immediate future development within the Auckland region, these being the Manukau, Waitakere and North Shore City Councils.

The information within the various geotechnical investigation and completion reports held by these councils has been reviewed and copies have been obtained of all the shrink-swell laboratory test information contained within those files. An analysis of the individual swell-strain and shrink-strain components making up the shrink-swell index has been undertaken to determine the relative influence that each of the strain components has on the overall index, and to determine if the relative influence changes significantly with soil type, time of season or sample depth.

2.0 RESEARCH APPROACH

2.1 Introduction

For the results of a research project of this type to be meaningful the following criteria should be met:

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

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

2.2 Staged investigation

The research project was conceived in two parts:

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

(b) Stage II – involving the correlation of the predicted soil shrink-swell movements with the measured ground surface movements and building performance observations.

Stage I, which is the subject of the 2003 Study Report, was jointly funded by the Building Research Levy and Manukau City Council.

Funding was obtained for Stage II from the Building Research Levy, Manukau City Council, Auckland City Council, Rodney District Council, North Shore City Council and Franklin District Council. The level of funding that was put in place was sufficient to provide for the installation of extensometers and further laboratory testing over a two-year period, but was not sufficient to extend the study to the observation and evaluation of building foundation performance.

Stage II has therefore been separated into two components, Stage IIA and Stage IIB, corresponding to the extensometer and laboratory investigation reported herein and to the observation of building performance respectively. The interaction of the investigations and their related stages are shown in Figure 1.

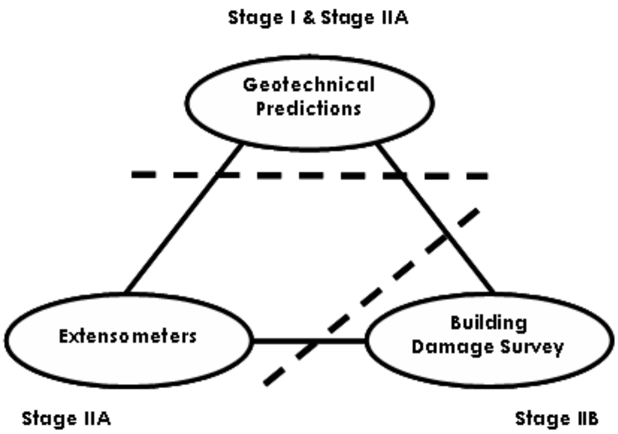


Figure 1 **Staged investigation showing overlap between Stage I, IIA and IIB areas of the expansive soils research.**

2.3 Overall investigation programme

2.3.1 Introduction

The Stage I investigation comprised eight sites selected from representative soil types within the Auckland region. It had been envisaged that the Stage IIA investigation would follow on from the completion of Stage I in October 2003 to provide continuous record of soil suction data. However, funding was not in place sufficiently early to enable the extensometers to be set up in time to monitor the 2003/04 summer period.

Stage IIA was commenced in April 2004 and ran until April 2006. The Stage IIA sites were selected to enable Stage IIB to be carried out should funding for Stage IIB be made available at a future time. The extensometer installation for Stage IIA therefore involved a review of the performance of the Stage I sites to determine if they would be appropriate to be used in Stages IIA and IIB. The level of funding received from the various local TAs was also a factor in selecting sites for Stage IIA.

2.3.2 Monthly testing

It was proposed that monthly soil suction testing should be carried out at all six sites selected for the Stage IIA investigation, to provide an indication of the trend of soil suction change over an 18 month period from winter 2004 to the end of summer 2006.

The monthly testing aimed to provide a guide for extrapolation of the measured results to provide data for the climatic extremes and correlation with the extensometer readings. Based on the level of funding for the project it was considered that eight discrete “months” could be tested, comprising two winter months (one in 2004 and one in 2005) and three months within each of the 2004/05 and 2005/06 summers.

The sampling times were selected on the basis of monitoring the soil moisture deficit

(SMD) data for the Auckland International Airport on the National Climate Centre 'Climate Now' website operated by NIWA in order to specifically target the driest periods during the summer months.

2.3.3 Seasonal testing

Soil classification tests, comprising Atterberg Limits and linear shrinkage tests, were obtained during the winter 2005 sampling round and submitted for testing. Core shrinkage (I_{cs}) tests were carried out on samples obtained during the 2005/06 summer.

The foregoing tests enable comparisons to be made for samples collected at the same time between the core shrinkage (I_{cs}) test (used in the Stage I report) and the shrink-swell (I_{ss}) test (the test most regularly carried out by consulting engineers in the Auckland region), in order to establish the validity of the assumptions adopted in the projections made within the Stage I report.

2.4 Extensometers

Extensometers were installed up to depths of approximately 4 m at the six Stage IIA sites and used to determine the depth at which no seasonal soil movement occurred, with the maximum depth confidently assumed to be a "zero" for measurements. The soil movements were generally measured at the same time as the laboratory test samples were obtained.

The extensometer data enabled a correlation to be obtained between the calculated soil movement from the soil suction readings and the actual movements measured on site for the soil moisture conditions existing at the time of measurement, and provides greater accuracy in the projection of the depth and amount of soil movement that may occur in drought conditions within the Auckland region.

A set of six extensometers were installed at each site, comprising steel rods cemented at depths of approximately 0.5 m, 1 m, 1.5 m, 2 m, 2.5 m and 4 m below the existing ground surface at the sites. The extensometers were buried below the ground surface to avoid accidental damage or vandalism to the equipment. Two surface monuments were also installed at each site at the top of the soil profile, one set below the topsoil layer and one set through the topsoil layer to the ground surface.

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 local TA.

(b) An assessment of damage to the building structure and fabric.

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

Although funding has not been made available for the building damage survey to be carried out, part of the Stage IIA investigation reported herein was to establish test sites in proximity to buildings that could be used within the building survey should funding lines be established for that work at a future time.

The site selection process required that buildings be identified that were publicly owned, by either the local TA or Housing New Zealand Corporation, and that the buildings had been constructed to approximate NZS 3604:1999 standards and be of a construction type that would show structural damage should any distortion to the building have occurred. Concrete slab-on-ground floor with conventional shallow pad or strip footings with full masonry or brick veneer cladding was a typical construction considered suitable for the purposes of the study. A minimum age of approximately 10 years was adopted in order to ensure that the buildings had experienced a reasonable history of summer and winter conditions.

The Stage II sites that comprise this part of the study have been identified as having a minimum of five buildings that meet the foregoing criteria, with the buildings identified being located at distances ranging from approximately 10 m to 500 m from the test site. Details of the locations of the identified buildings are held on Fraser Thomas Ltd file records.

3.0 LITERATURE REVIEW

A literature review was undertaken as part of the Stage I investigation and report. The published papers and other references that were reviewed for Stage I and which may be referred to in this report are presented in the bibliography in Appendix A of this report. Readers are referred to the 2003 Study Report for the literature review and associated discussion.

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 i.e. there was a one-year 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. In contrast, a similar database has yet to be developed for New Zealand soils.

While not referred to in the New Zealand or the Australian Standards, the American Association of State Highway and Transportation Officials (AASHTO) has also developed a soil expansivity test method, which is discussed further in later sections.

4.2 NZS 3604:1990 Code of Practice for Light Timber Framed Buildings Not Requiring Specific Design

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

NZS 3604:1990 referred to buildings on expansive soils sites. Section 3.2.2 *Expansive Clay* 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 450 mm below the cleared ground level and all other foundations (not into rock) be founded at a minimum depth of 300 mm. 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 – including the definition of expansive and plastic clays – from NZS 3604:1990. It also 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 450 mm 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 some eight years later.

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 (i.e. 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 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; and

(c) Any ground which could foreseeably experience movement of 25 mm 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 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 has a particular degree of expansivity.

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

AASHTO prescribe 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 viz:

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

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 condition of the soil profile changes from wet to dry design conditions. The estimation of y_s requires knowledge 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 of AS 2870:1996)**

Characteristic surface movement	Classification of site
0 mm $<y_s \leq 20$ mm	S – Slightly reactive
20 mm $<y_s \leq 40$ mm	M – Moderately reactive
40 mm $<y_s \leq 70$ mm	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 Sections 5.2 to 5.4.

Note that a classification above the lower end of the "moderately reactive" range will fall outside the definition of "good ground" from Section 1.3 of NZS 3604:1999 as set out in Section 4.3, which has a limiting maximum movement of 25 mm.

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 five 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 presented as Table 3 and Figure 2 of this report respectively.

$$y_s = \frac{1}{100} \int_0^{Hs} I_{pt}\, \Delta u dh \qquad \textbf{Equation 2}$$

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 (mm)
H_s = depth below which no moisture or soil suction change occurs (mm)

A worked example of Equation 2 is provided in the Australian Handbook HB 28:1997 *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. Sunflowers in pots are commonly used as the test plant, as discussed in Section 2.16.4 of HB 28:1997, with the permanent wilting point being defined as the moisture in a soil when plants in pots start to wilt and not recover.

Recommended values of soil suction change at the soil surface, Δu_s, 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 [(1)] the soil surface (Δu_s) 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

Note 1. The symbol Δu_s has been adopted to denote the Δu value at the soil surface.

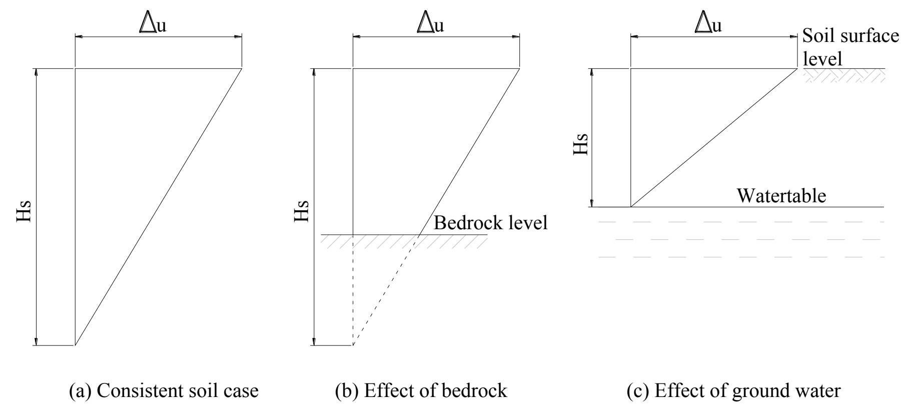


Figure 2 Effect of bedrock or watertable on design soils suction change profiles (from AS 2870:1996 – Figure 2.1)

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 pF to 5 pF or 100 kPa to 10000 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, 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 (1993) 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. In New Zealand, 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 100 mm, 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 1200 mm 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 evapo-transpiration 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 above is the same as that proposed in Tables Dl 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, i.e. 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 1.5 m 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 5 mm.

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 HB 28:1997). AS 2870 provides values of cracked depths as shown in Table 5.

Table 5 Examples of cracked zone depths (from HB 28:1997 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".

For the purposes of this study, a cracked zone depth of 0.5 H_s has been adopted, which is the same as recommended for Sydney, Newcastle and Brisbane in HB 28:1997, as shown in Table 5 of this report, and corresponds to a depth of 0.75 m if H_s is taken as 1.5 m.

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 28:1997.

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 previously (refer Table 2).

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; lack of maintenance of site drainage;

(f) Failure to repair plumbing leaks.

Guidance and advice is given in Appendix B of AS 2870 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; and

(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 and are reproduced in Appendix E.

In this regard 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.

Given the foregoing, for New Zealand use we propose a modification for the Acceptable Solution B2/AS1 which defines the “maintenance” required for foundations on expansive soils as follows:

Normal maintenance (Building Code B2/AS1 format)

Normal maintenance is that work generally recognised as necessary to achieve the expected performance of the foundation located on expansive soils over time.

Unless otherwise specified by the designer, and noted on the drawings, basic normal maintenance tasks shall ensure that:

(a) The drainage and wetting of the site is controlled so that extremes of wetting or drying of the soils is prevented;

(b) The positions and operation of gardens adjacent to the dwelling are controlled, and the planting of trees near to foundations of houses is suitably restricted;

(c) Any leaks which develop in plumbing, stormwater and sanitary sewage systems are reported promptly;

with the level of implementation matched to the expansivity of the underlying soils and the distortion tolerance of the cladding.

5.5.4 Site classification

AS 2870 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, AS 2870 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 ground surface movement (y_s).

Notwithstanding the foregoing, AS 2870 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 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 the accuracy of 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 acknowledging the basis of development of AS 2870 recommendations 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. The rectification of that situation is in part the purpose of the current study.

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" i.e. engineered fill or "uncontrolled" i.e. 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) up to 0.8 m for sand
  - (b) up to 0.4 m for other materials.
- (ii) Deep fill
  - (a) >0.8 m depth of sand may lead to a less severe reactive classification
  - (b) >0.4 m depth of other material requires the site to be considered as Class P (i.e. 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) up to 0.8 m depth of sand
  - (b) up to 0.4 m depth of other material.
- (ii) Deep fill – for fill depths greater than 0.8 m for sand and 0.4 m 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 long-term movement in the fill and the underlying soils; and

(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 O.5 H_s, whichever is greater [H_s being 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 five years prior to building construction.

5.5.7 Classification parameters – discussion

(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 28:1997 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 28:1997 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 assumes 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, e.g. data from an open field site subjected to seasonal moisture changes will not be applicable [without consideration of the effects of site development].

AS 2870 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 0.5 m; or
(b) the depth of fill exceeds the limits [described in 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 28:1997:

> 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 TMIs for the Auckland Airport meteorological station for the 34-year period between 1 January 1972 and 30 June 2006.

6.2.1 Thornthwaite Moisture Indices (TMIs)

Internationally TMIs are generally used for the following:

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

(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.

As noted in Section 4.5, the American method also uses the Atterberg Limits to define the clay mineralogy. However HB 28:1997 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 TM1 varies greatly between the various 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 34 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 1,240 mm. The average annual rainfall for the eight Stage I 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 evapo-transpiration less deep percolation" which, along with other meteorological data, yields a SMD for the particular location.

The SMD 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 2002/06 years; and

(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 (i.e. a surplus) and negative values indicate a deficit.

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

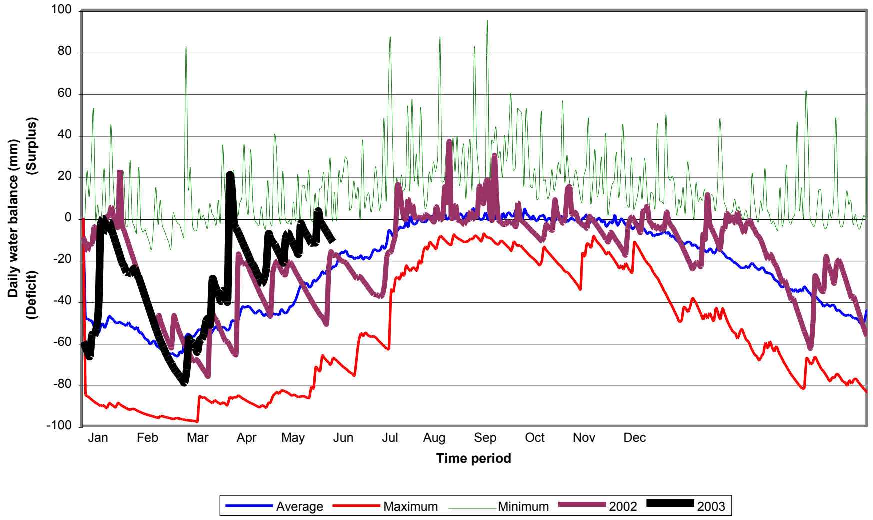


Figure 3 Daily water balance for Auckland Airport during 1971–2003

6.3 Climate factors for Australia and Auckland

As noted previously, 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.

Table 6 Climate factors (sourced from www.worldclimate.com, Fityus (1998), HB 28:1997 and Appendix D of AS 2870)

	Average Annual Readings			
City	**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

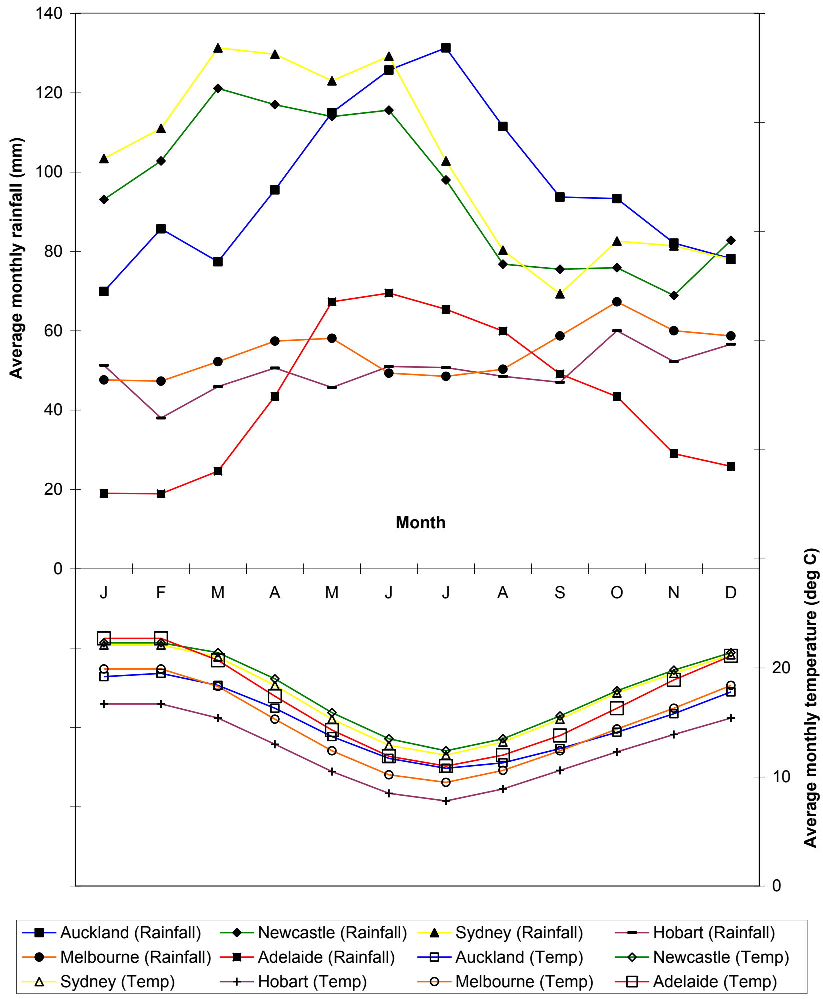


Figure 4 **Average monthly rainfall and temperature measurements for Auckland Airport and selected Australian cities (sourced from www.worldclimate.com)**

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 evapo-transpiration 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 evapo-transpiration 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 while clays with a high kaolinite content are known to be only slightly reactive. AS 2870 and HB 28:1997 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 has been taken from Australian Handbook HB 28:1997, 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 HB 28:1997, p29)

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

The data in Table 7 indicates that any sites in Sydney with clay depths greater than 2.5 m 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 this is ongoing. 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 (e.g. 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 (i.e. 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 to particular sites.

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 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	<0.6 m	S – Slight
Sydney	Non-alluvial	0.6 – 2.5 m	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	>2.5 m	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	>2.0 m	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 R 11, scale 1:50000.

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 the 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 with 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 age.

(a) Puketoka Formation (tp) – this 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 inter-bedded with carbonaceous deposits.

7.3.6 Auckland Volcanic Field – basaltic ash

The Auckland Volcanic Field comprises basaltic deposits of the 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, 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 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 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 STAGE II

8.1 Site selection

The test sites used for the Stage I investigation (refer Table 9) have in general been carried through into Stage II with the following modifications:

(a) The original Sites A (Newmarket), C (Brookby) and E (Swanson) have been discontinued.

(b) A new Site 2F, located at Princess Street Reserve, Pukekohe has been added to gain geographic representation from the South Auckland Volcanic Group soils located within the Papakura District Council and Franklin District Council areas.

(c) Site B (East Tamaki) and Site G (Hillcrest) have been replaced by sites at Otara and Mairangi Bay, respectively, in order to provide greater site control in the event that funding for building damage surveys becomes available.

Table 9 Stage II test locations in the Auckland region

Stage I site code	Main soil type	Suburb	Stage II site code	Suburb
A	Basaltic ash	Newmarket	**–**	–
B	Tauranga Group (tpp)	East Tamaki	**2B**	Otara
C	Waipapa Group	Brookby	**–**	–
D	Tauranga Group (tp)	Manurewa	**2A**	Manurewa
E	Tauranga Group (tp)	Swanson	**–**	–
F	Waitemata Group	Howick	**2C**	Howick
G	Waitemata Group	Hillcrest	**2D**	Mairangi Bay
H	Onerahi Chaos Breccia	Red Beach	**2E**	Red Beach
–	South Auckland Volcanics	Pukekohe	**2F**	Pukekohe

The actual site selections made generally reflect the need to obtain a range of soil types and local climate conditions across the Auckland region on which to base the research findings. The selections also reflect the relevant TA's willingness to provide funding support for the current project work.

8.2 Extensometer field

From April to August 2004 the extensometers were installed at each of the Sites 2A to 2F inclusive. Six extensometers and two surface monuments were installed at each site. The extensometers were installed so that they would sit below the ground surface beneath a turf square in order to prevent damage occurring to the extensometers from mowing or vandalism.

Boreholes to six different depths were put down at each site for the extensometers. The extensometers comprised stainless steel rods with a welded base plate concreted in a 0.2 m plug in the base of the borehole. A profile of the extensometer installation is shown in Figure 5. The plug was set so that the centre of each plug would be at 0.5 m, 1 m, 1.5 m, 2 m, 2.5 m and 4 m depths. The plug comprised rapid-setting cement that was tremmied to the base of the extensometer via plastic hose pipe.

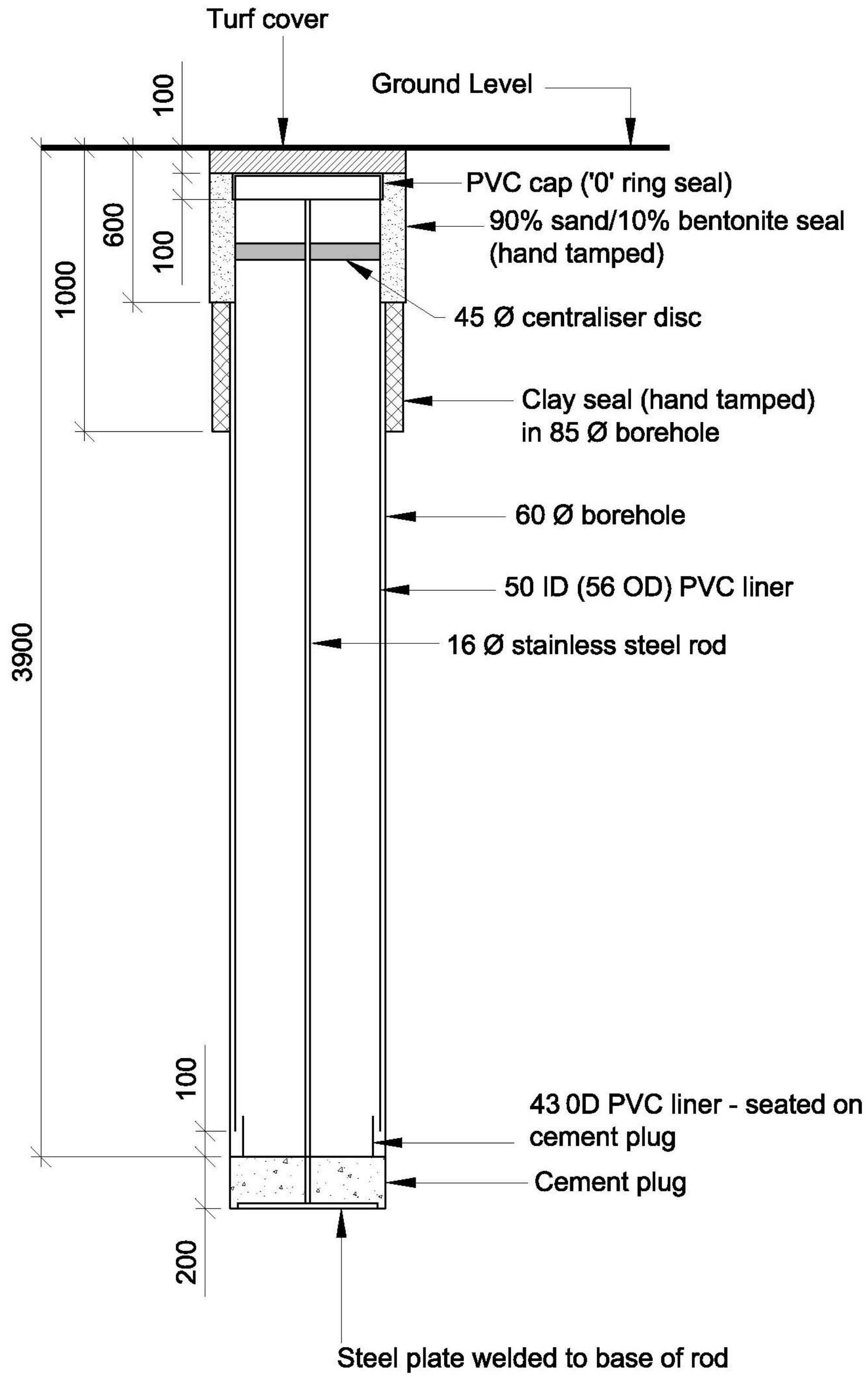


Figure 5 Sketch detail of extensometer construction (4 m deep rod used for example)

A 56 mm outside diameter (OD) PVC tube was installed to prevent the borehole from closing in on the extensometer. The PVC tube was installed approximately 100 mm above the top of the plug so that the extensometer was free to move without having to overcome any friction resistance between the PVC tube and the surrounding soil. A smaller 43 mm OD inner PVC tube of approximately 200 mm length was put down to obtain an overlap with the bottom of the main outer tube.

In order to mitigate against the risk of the PVC tube acting as a conduit for the ingress of stormwater and/or seepage from the topsoil layer migrating down the tubing and affecting the moisture conditions at the base of the extensometer, seals were installed in the upper metre of the extensometer. In order to install the seals an 85 mm diameter borehole was put down to approximately 1 m depth (less for the 0.5 m and 1 m extensometers). The seal comprised compacted (tamped) clay between approximately 1 m and 0.6 m depth below the ground surface and tamped sand/bentonite mix (90:10) between 0.6 m depth and the underside of the topsoil.

The two surface monuments constructed at each site have been designated as the 0.2 m and 0.0 m "extensometers". The 0.2 m surface monument was constructed by excavating a turf square to the base of the topsoil at site and a survey plate installed into a shallow concrete plug. The 0.0 m extensometer comprised a hand auger to the underside of the topsoil at the site with the borehole infilled with concrete and a survey nail set into the concrete. Profiles of the surface monuments are shown in Figure 6.

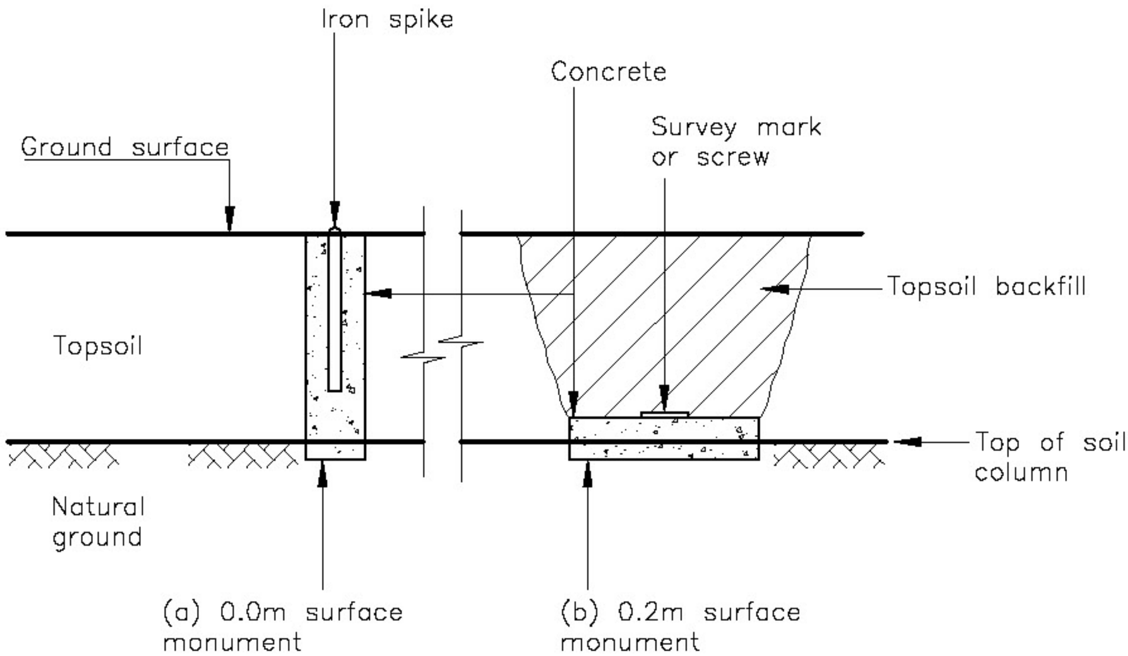


Figure 6 Sketch details of surface monuments

The extensometers and surface monuments were measured by level survey on each monitoring occasion and were measured in relation to the 4 m deep extensometer, which has been assumed to be located at a depth below which any ground movement could occur. It is noted that as the soil materials became too hard to auger at approximately 3.2 m below the ground surface at Site 2F, the "4 m deep" baseline extensometer at Site 2F is actually at 3.2 m depth.

In a parallel experiment the University of Auckland has installed two "spider magnet" extensometers at each of Sites 2A to 2F inclusive. The design, construction, monitoring, analysis and reporting of the data from the spider magnet extensometers is the entitlement of the University of Auckland and does not form a part of the Stage II research project. Any comparison between the two sets of extensometers is also outside the scope of this report.

8.3 Sampling and monitoring

8.3.1 General

Sampling and monitoring at the selected sites was programmed to coincide with the driest periods during the summers of 2004/05 and 2005/06 and typical wet periods during the winters of 2004 and 2005, with the aim of capturing soil suction, soil moisture and ground movement data that would be representative of wet and dry conditions.

Typically 17 hand auger boreholes were put down at each site over the duration of the project. The logs of the boreholes are presented in Appendix B. The borehole number relates to each of the six sites, viz Borehole 2A relates to Site 2A, Borehole 2B to Site 2B etc. The logs relate to the first borehole put down at each site during the installation of the 4 m deep extensometer. The subsequent boreholes put down at each site were located in an approximately 1 m grid from the original borehole and line of extensometers and were generally spaced out according to the plan shown in Figure 7. A cross-section through the extensometer field is shown in Figure 8.

While it is acknowledged that there is some variability in the shrink-swell characteristics of the soil profile across each site, it is our opinion that the variability is likely to be small, albeit that such variability could have affected the extensometer data.

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

In order to target the driest conditions within the 2004/05 and 2005/06 summers, the 'Climate Now' website operated by the National Climate Centre, NIWA, was monitored on a regular basis. The website provides SMD values for the weather stations within the region and is generally updated on a weekly basis. The SMD values are a daily water balance for a theoretical 150 mm thick topsoil layer which keeps track of the rainfall entering the pasture root zone and being lost from this zone by evapo-transpiration or plant use. The plots are intended as a guide for agricultural users to aid irrigation decisions.

Our monitoring of the website enabled more precise targeting of dry summer conditions for the Stage II extensometer measurements and obtaining of soil samples for the laboratory testing than was possible for the earlier Stage I investigation.

Following NIWA advice (refer Section 6.2), a SMD value of 90 mm for a theoretical 100 mm thick topsoil layer was taken to represent a drought condition for the Stage I 2003 report. The SMD over the duration of the project and the sampling times are shown in Figure 9. The 2004/05 summer period provided three measurements at SMD conditions approaching, but not reaching, the "theoretical drought condition" defined in Section 6.2.

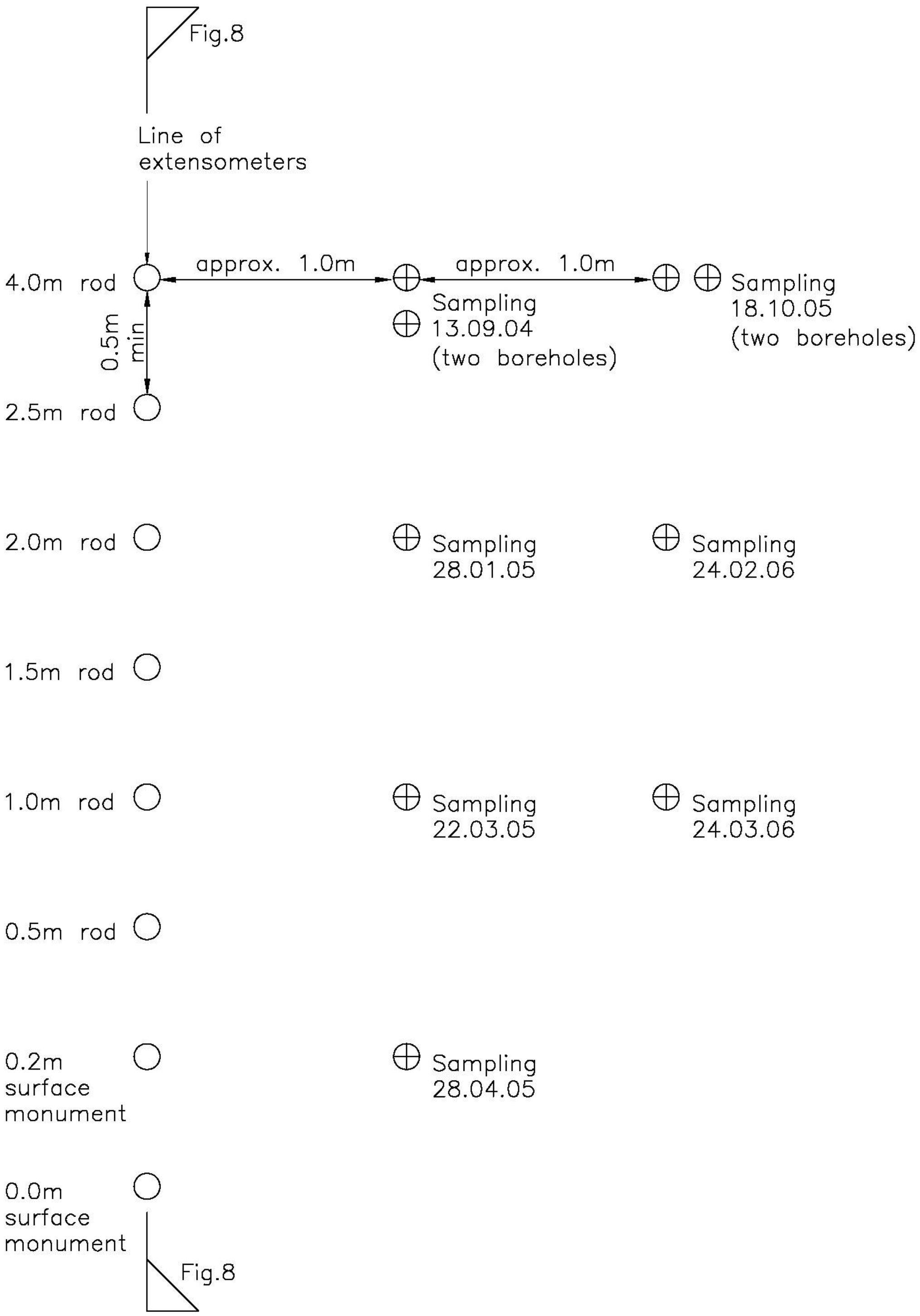


Figure 7 Sketch plan showing typical extensometer and borehole sampling layout

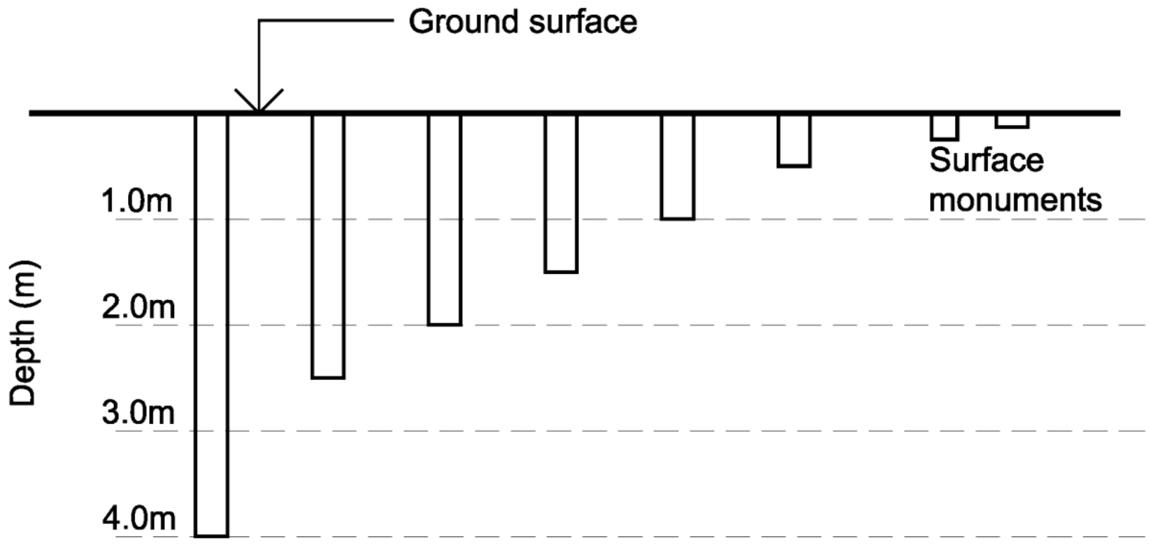


Figure 8 Cross-section of extensometer field

Note: Refer Figures 5 and 6 for detailed features of extensometer and surface monuments.

As discussed in Section 9.0 of this report, the extensometer readings and soil suction values generally recorded minor variations during the 2004/05 summer period. In order to prevent duplication of these results, the 2005/06 summer period sampling and monitoring was scaled back in order to hold funding in reserve until more severe dry conditions were reached.

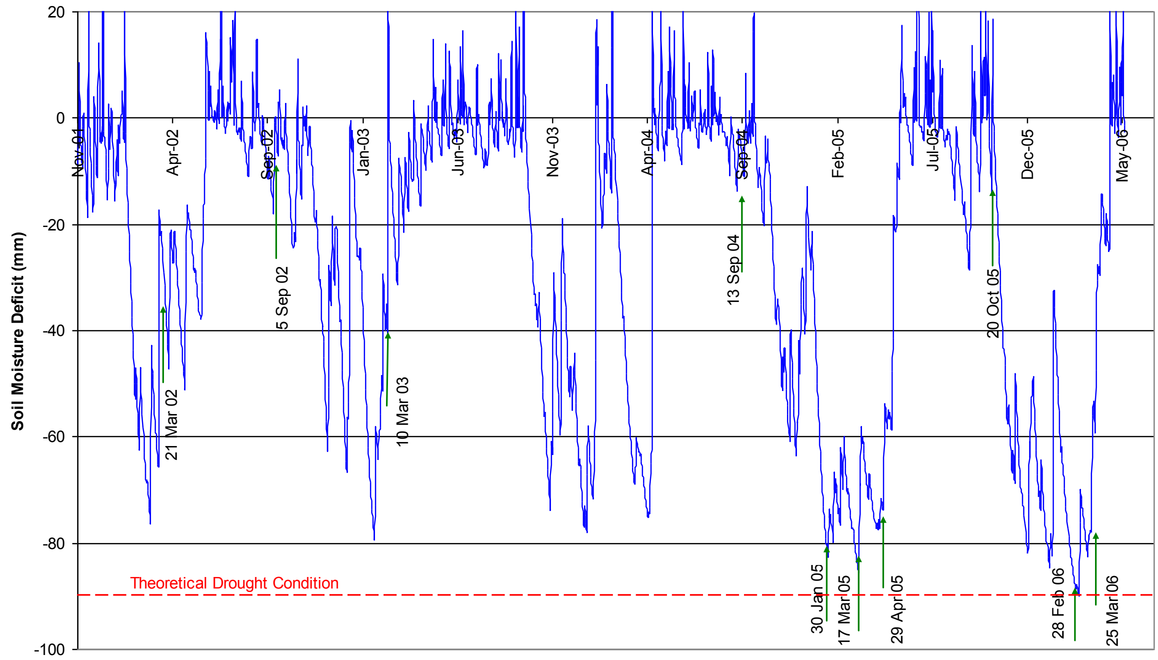


Figure 9 Soil moisture deficit variation over Stage I and Stage II monitoring periods. Stage II sampling dates shown in green. Stage I theoretical drought condition shown in red. Note the relatively low (dry) values obtained in the Stage II period compared to Stage I.

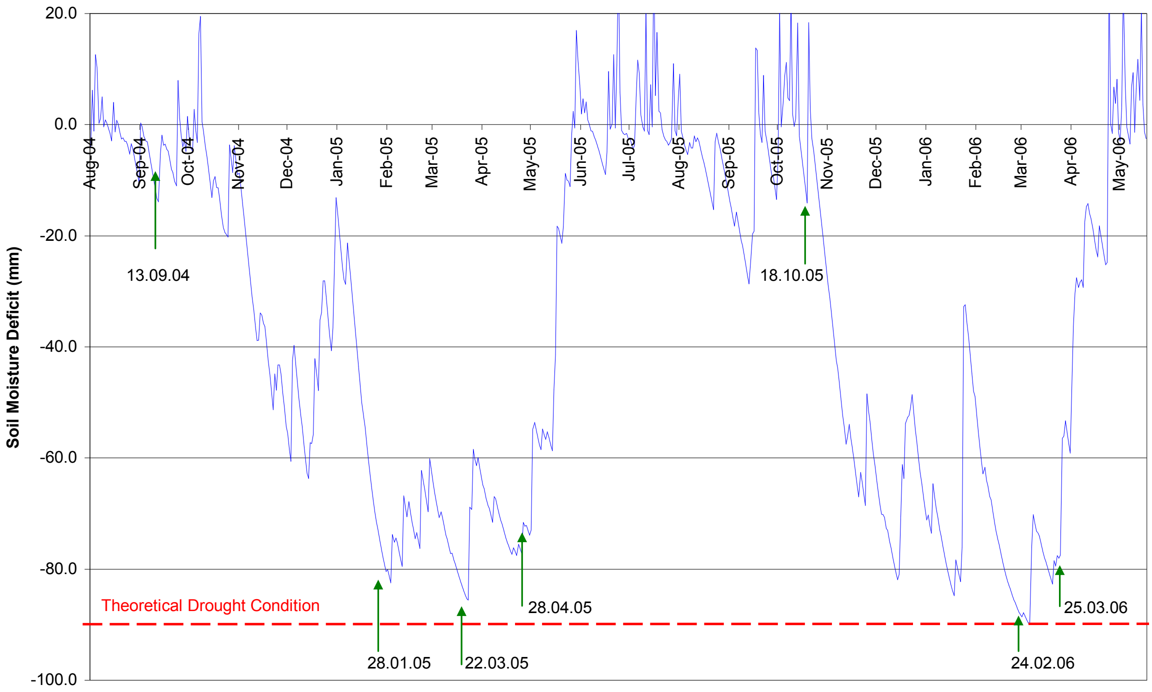


Figure 10 **Soil moisture deficit variation over Stage II monitoring period. Note the February 2006 sampling point approximates the theoretical drought condition.**

An issue identified in the Stage I Study Report was that the Stage I monitoring period was carried out over two relatively wet summer periods in 2002 and 2003, and that it was difficult to target the driest conditions within those summer periods. Figure 9 illustrates the SMD values and sampling points over the Stage I and Stage II monitoring periods.

As can be observed from Figure 9, the Stage II summer sampling points have measured significantly drier SMD conditions than the Stage I summer sampling points. Further, as can be seen in Figure 10, the 2005/06 summer sampling generated SMD values approximating the 2004/05 summer period values i.e. similar summer conditions were obtained over both the Stage II summers monitored.

In particular, the 24 February 2006 sampling day essentially reached the theoretical drought condition, thereby providing an opportunity of evaluating the measured conditions against the predictions of the Stage I report.

The -90 mm condition has been reached 12 times over the 44-year record available for the airport weather recording station, simplistically approximating a one-in-four-year return period drought condition.

The Stage II results, encompassing periods having significantly drier soil conditions than the Stage I study, should be able to provide correspondingly more meaningful data, giving greater accuracy to the analyses, interpretations, extrapolations and conclusions.

For completeness, the *Sampling and Monitoring* record and *Results of Field Investigation and Testing* from Sections 8 and 9 of the Stage I report are reproduced in Appendix C.

8.3.2 Winter 2004

The "winter 2004 sampling" was carried out on 13 and 14 September 2004 after the extensometers had been installed for a minimum period of one month prior to sampling.

The sampling comprised:

(a) Five "undisturbed" soil samples at 0.5 m depth intervals, between 0.5 m and 2.5 m depth, taken with a 45 mm diameter thin-walled stainless steel tube driven into the base of each borehole at the required depth using a Scala Penetrometer hammer and rods; and

(b) Level survey measurement of extensometers and surface monuments.

The ends of the tube samples were sealed 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.

8.3.3 Summer 2004/05

Three sampling rounds were carried out over the 2004/05 summer period. The first sampling round was carried out between 28 January and 1 February 2005 and comprised:

(a) One hand auger borehole at each site to provide five "undisturbed" soil samples at 0.5 m depth intervals for laboratory testing as for 8.3.2(a) above; and

(b) Level survey measurement of extensometers and surface monuments.

After the evaluation of the extensometer and surface monument data from the above sampling round it was apparent that no significant movement was being measured within the extensometers, but that some movement was being measured in the surface monuments. It was therefore inferred that taking soil suction samples to a depth of 2.5 m would measure the soil suction below the zone of seasonal changes and that more information would be obtained by targeting the upper soil profile. Therefore, for the balance of the Stage II study, the depth of the undisturbed sampling was reduced to 2 m with an undisturbed soil sample obtained from immediately below the surficial topsoil at each site i.e. at 0.2 m or 0.3 m depth below the ground surface.

The second and third summer sampling rounds were carried out between 17 and 23 March 2005 and 28 and 29 April 2005 respectively and comprised:

(c) One hand auger borehole, to 2 m depth at each site to provide five "undisturbed" soil samples at approximately 0.5 m depth intervals for laboratory testing as for 8.3.2(a) above; and

(d) Level survey measurement of extensometers and surface monuments.

In addition to the foregoing, samples were obtained during the second sampling round for shrink-swell index testing, which comprised:

(e) Taking two “undisturbed” soil samples at between 0.5 m and 1 m depth, with a 63 mm diameter thin-walled stainless steel tube driven into the base of each borehole at the required depth using a Scala Penetrometer hammer and rods.

8.3.4 Winter 2005

The “winter 2005 sampling” was carried out on 18–20 October 2005 and comprised:

(a) One hand auger borehole to 2 m depth at each site to provide five “undisturbed” soil samples at approximately 0.5 m depth intervals for laboratory testing as for 8.3.2(a) above;

(b) Taking three disturbed samples at approximately 0.5 m, 1 m and 1.5 m depths, immediately above the level of the foregoing undisturbed samples;

(c) Taking two “undisturbed” soil samples at 0.5 m and 1 m depth in an additional hand augered borehole located adjacent to the first borehole for shrink-swell index testing as for 8.3.3(e) above; and

(d) Level survey measurement of extensometers and surface monuments.

The samples obtained in (a) and (b) above were submitted to the Auckland University Geomechanics Laboratory, while the samples in (c) above were submitted to Geotechnics Ltd, an IANZ accredited laboratory, for laboratory testing.

8.3.5 Summer 2005/06

Two sampling rounds were carried out over the 2005/06 summer period. The first sampling round was carried out between 24 and 28 February 2006 and comprised:

(a) One hand auger borehole to 2 m depth at Sites 2A, 2B and 2C to provide five “undisturbed” soil samples at approximately 0.5 m depth intervals for laboratory testing as for 8.3.2(a) above; and

(b) Level survey measurement of extensometers and surface monuments.

The second sampling round was carried out on 24 and 25 March 2006 and comprised:

(c) One hand auger borehole to 2 m depth at each site to provide five “undisturbed” soil samples at approximately 0.5 m depth intervals for laboratory testing as for 8.3.2(a) above;

(d) Taking two “undisturbed” soil samples at 0.5 m and 1 m depth in an additional hand augered borehole adjacent to the first borehole for shrink-swell index testing as for 8.3.3(e) above; and

(e) Level survey measurement of extensometers and surface monuments.

8.4 Laboratory testing

The soil laboratory testing, shown in Table 10 of this report, was generally undertaken by the Geomechanics Laboratory of the University of Auckland, School of Engineering during the periods discussed in Section 8.3, with the exception of the shrink-swell testing which was carried out by Geotechnics Ltd.

The Atterberg Limits and linear shrinkage tests were undertaken to NZS 4402:1987 *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).

The shrink-swell tests were undertaken to AS 1289 Test Method 7.1.1:1998 *Soil Reactivity Tests – Determination of the Shrinkage Index of a Soil – Shrink-Swell Index*.

Table 10 Laboratory test programme

Laboratory test	Winter 2004	Summer 04/05			Winter 2005	Summer 05/06	
		Month 1	Month 2	Month 3		Month 1	Month 2
Water content	✓	✓	✓	✓	✓	✓	✓
(5 depths)							
Soil suction	✓	✓	✓	✓	✓	✓	✓
(5 depths)							
Atterberg Limits					✓		
(3 depths)							
Linear shrinkage					✓		
(3 depths)							
Core shrinkage					✓[2]		✓
(3 depths)							
Shrink-swell Test			✓[1]		✓		✓
(2 depths)							

Note 1. Core shrinkage test carried out on samples obtained instead of shrink-swell test.
2. No results available for tests as a component of the core shrinkage test was not obtained.

9.0 RESULTS OF FIELD EXTENSOMETERS AND LABORATORY TESTING

9.1 General

The soil profiles encountered in the boreholes at Sites 2A to 2F are shown on the borehole logs presented in Appendix B of this report. The soil profiles at each site are summarised in Table 11. The groundwater levels measured during the Stage II period are shown on the individual site reports in Appendix C of this report.

The extensometer and laboratory test results will be discussed in general in Sections 9.2 and 9.3 before more detailed treatment of the "as measured" dry summer conditions and analysis of the soil suction results in Sections 10.0 and 11.0.

Table 11 **Summary of soil types at each test site**

Site code	Suburb	Depth (m)	Soil unit	Soil description
2A	Manurewa	0.2-4.0	Tauranga Group (tp)	silty CLAY
2B	Otara	0.2-4.0	Tauranga Group (tp)	sandy and silty CLAY
2C	Howick	0.2-1.3 1.3-4.0	Waitemata Group Waitemata Group	silty CLAY sandy and clayey SILT
2D	Mairangi Bay (North Shore)	0.3-4.0	Waitemata Group	silty CLAY
2E	Red Beach	0.3-3.0 3.0-4.0	Onerahi Chaos Breccia Onerahi Chaos Breccia	silty CLAY clayey SILT
2F	Pukekohe	0.2-1.5 1.5-2.4 2.4-3.2	Lithic Tuff / Basaltic Ash Ash / Alluvials Lithic Tuff	clayey SILT silty CLAY gravelly SILT

9.2 Extensometer results

9.2.1 General

The results of the measured survey levels of the extensometers installed at each of the six sites are presented in item 4.0 of the site summary sheets presented in Appendix C of this report. All values presented for the extensometers and surface monuments are in millimetres (mm).

The extensometer values presented in Appendix C for each site have been compared to the winter 2005 readings, which are considered to best represent "zeroed" conditions. These are the most complete set of winter readings available as some of the surface monuments were damaged during the 2004/05 summer period and had to be reinstalled.

The negative values therefore indicate the amount of shrink that has occurred between winter 2005 and the individual measured summer point relative to the 4 m deep extensometer. As the level survey has an accuracy of ±1 mm per measurement, the extensometer readings presented in Appendix C, being the difference between two readings, should all be considered to have a combined accuracy of ±1.4 mm.

9.2.2 Observations

In the first instance it is noted that, in general, the order of magnitude of the measured readings is relatively low in comparison to the soil expansivity classifications presented in AS 2870. The range of maximum measured extensometer movements over the Stage II monitoring presented in Appendix C is in the order of 15 mm to 25 mm which, if it took place over a full range of climate extremes, would correspond to the slightly reactive and the lower end of moderately reactive soil classification classes according to AS 2870 as indicated in Table 12.

Table 12 Classification by characteristic surface movement (y_s) (from Table 2.3 of AS 2870:1996)

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

Detectable movements were generally consistently obtained at the Manurewa, Otara, Howick and Red Beach sites over the Stage II monitoring period. No significant movement was measured in the extensometers and surface monuments at the North Shore site. Detectable movement at the Pukekohe site was only identified in one of the surface monuments installed.

Examining the depth of recorded movement it is noted that, as expected, the largest movements recorded occur in the surface monuments and that the size of the movements decreases with depth. What is of particular interest to the study is the depth at which no measurable movement occurs. Examination of the extensometer data indicates that, for Sites 2B to 2F inclusive, measurements greater than the survey tolerance were obtained down to and including the 0.5 m deep extensometer, but not in the 1 m deep extensometer. The exception to this observation is Site 2A (Manurewa), which generally recorded measurable movements within the 1 m deep extensometer and, in one instance, recorded measurable movement in the 1.5 m deep extensometer.

9.3 Laboratory test results

9.3.1 General

The results of the soil classification tests (Atterburg Limits and linear shrinkage tests) and the shrinkage index tests (shrink-swell index and core shrinkage tests) for Sites 2A to 2F are shown on the individual site reports presented in Appendix C.

R-squared values are shown on the various following plots to indicate the correlation between the parameters. The R-squared value provides a linear regression between the actual test data points and the theoretical line and is an indication of the amount of variation that is inherent in the linear model. An R-squared value of zero indicates that there is no correlation between the parameters and a value of one shows a perfect correlation. A rule of thumb is that R-squared values of below 0.3 are not considered to be statistically significant in demonstrating the existence of a correlation at about the 90% confidence interval.

9.3.2 Atterberg Limits and linear shrinkage

A Casagrande plot of the Atterberg Limits test data for Stage II is shown on Figure 11, which indicates that the soils plot slightly above or below the A line and are of high to extremely high plasticity with liquid limits ranging from 50% to 115%. As shown on Figure 12, the linear shrinkage values of the soils range from 11% to 25%.

Figure 12 indicates that approximately 70% of the test locations for Stage II fall outside the definition of “good ground” as defined by NZS 3604:1999. The plot shown in Figure 12 also illustrates that a good relationship exists between linear shrinkage and liquid limit, which is not unexpected but was not, however, borne out by the original Stage I data.

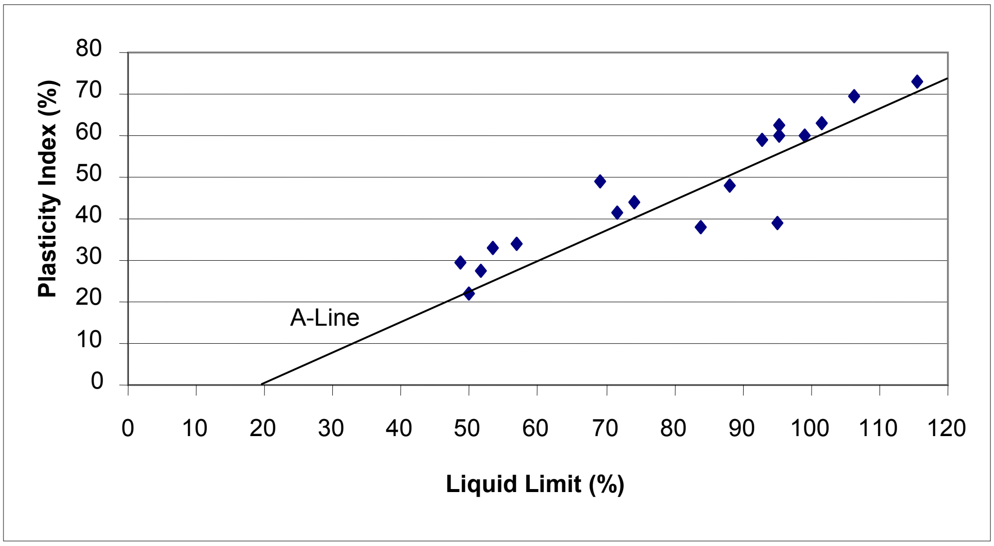


Figure 11 Casagrande plot of plasticity index against liquid limit for Stage II test data

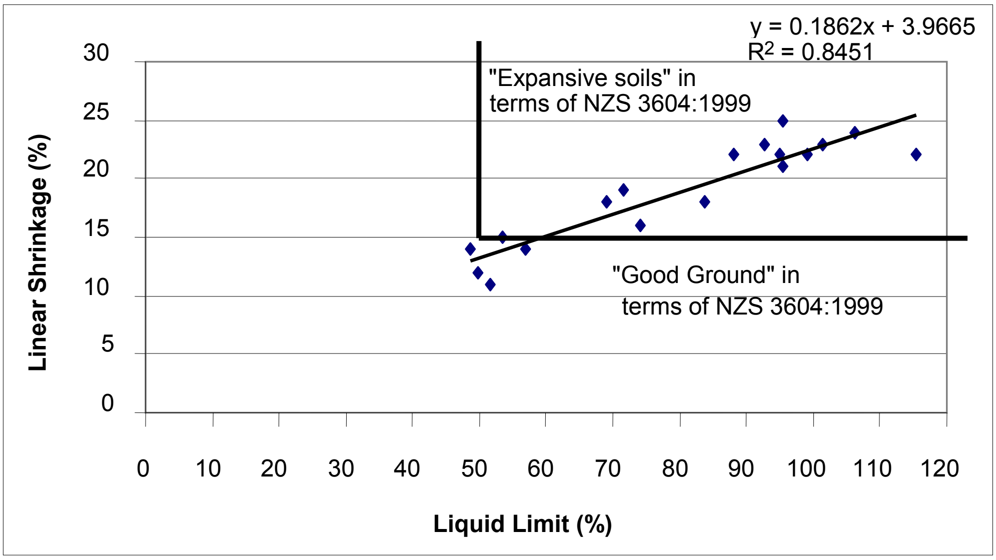


Figure 12 Plot of linear shrinkage against liquid limit for Stage II test data

9.3.3 Shrinkage index, I_{ps}

The shrinkage index (I_{ps}) 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 Section 8.4, the shrinkage index testing for Stage II comprised two sets of shrink-swell tests (I_{ss}) and three sets of core shrinkage tests (I_{cs}), with one of the core shrinkage test sets being incomplete.

The purpose of carrying out two different test methods on the samples was to:

(a) Compare and evaluate the test method carried out for the Stage I investigation (core shrinkage) to the test method generally being carried out by geotechnical practitioners in the Auckland region (shrink-swell); and

(b) Compare and evaluate any differences in the test results from samples taken in summer to those taken in winter.

As discussed in Sections 8.3.3 and 8.4 the samples submitted for shrink-swell testing in the 2004/05 summer had the core shrinkage test carried out.

The results of the various shrinkage tests that were carried out during the Stage II investigation are shown in Table 13.

Table 13 Stage II shrinkage index test results

		A	B	C	D	
Site	**Depth (m)**	**Oct-05**	**Mar-06**	**Mar-05**	**Mar-06**	**Average shrinkage index**
		I_{ss}	I_{ss}	I_{cs}	I_{cs}	**(A, B, C)**
Manurewa	0.5	2.4	2.9	1.30	6.08	2.2
	1.0	4.2	2.4	4.39	6.10	3.7
	1.5	–	–	–	5.96	
Otara	0.5	–	1.5	2.22	4.83	1.9
	1.0	3.3	3.9	3.74	5.87	3.6
	1.5	–	–	–	0.88	
Howick	0.5	2.9	2.2	2.71	4.99	2.6
	1.0	2.6	3.0	2.14	4.86	2.6
	1.5	–	–	–	3.86	
Mairangi Bay (North Shore)	0.5	1.1	0.5	1.92	0.12	1.2
	1.0	–	0.9	2.4	3.9	1.7
	1.5	–	–	–	4.2	
Red Beach	0.5	3.6	2.1	3.19	4.94	3.0
	1.0	6.7	2.8	4.77	4.74	4.8
	1.5	–	–	–	7.96	
Pukekohe	0.5	2.8	2.5	2.75	2.30	2.7
	1.0	3.7	2.7	2.30	–	2.9
	1.5	–	–	–	5.49	
Average		3.3	2.3	2.8	4.5	2.8
Approx sample diameter (mm)		60	60	60	41	
Approx L:D		2.0	2.0	1.7-1.8	1.7-1.8	

Note: I_{ss} = shrink-swell index I_{cs} = core shrinkage index

Although there is some variance between the October 2005 and March 2006 shrink-swell test results, they are generally of the same order, indicating that with respect to (b) in Section 9.3.3, there is no significant difference between samples obtained and tested in winter and summer on the basis of the test results reported herein.

9.4 Correlations between shrinkage index and other soil classification parameters

The relationships between the shrinkage index and the soil classification parameters of linear shrinkage, liquid limit and plasticity index have 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 13, 14 and 15 respectively.

It is apparent from Figures 13, 14 and 15 that there are weak correlations between the shrinkage index and all three of the soil classification parameters. The R-squared values increase to greater than 0.5 if the upper left value in the plots is considered to be an outlier and removed from the data set.

Obtaining these correlations is in contrast to the analyses presented in the Stage I Study Report which did not identify any "readily discernable correlations" for the same classification parameters, but is in agreement with Cameron (1989) which 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.

NZS 3604:1999 defines expansive soils as "being those that have a liquid limit of more than 50% ... *and* a linear shrinkage of more than 15% ...". If the linear regression equations shown in Figures 14 and 15 are applied to these threshold values, they yield corresponding shrinkage index values of 2% and 2.6% respectively.

It is therefore inferred that, on the basis of the Stage II data reported herein, that a shrinkage index threshold value of around 2.5% could be adopted as a corresponding value for classifying a soil as being within the definition of "good ground" as defined by NZS 3604:1999.

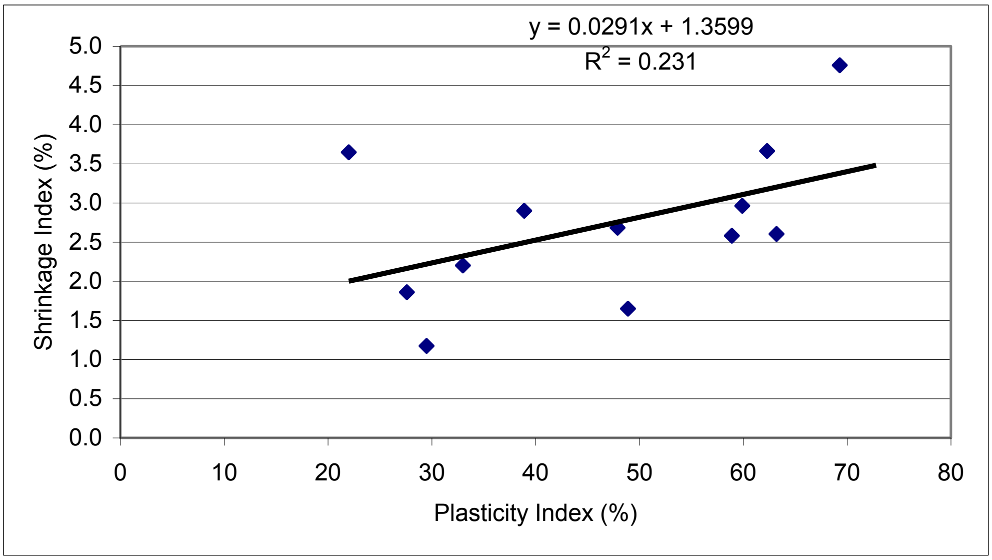


Figure 13 Shrinkage index vs plasticity index for Stage II test data

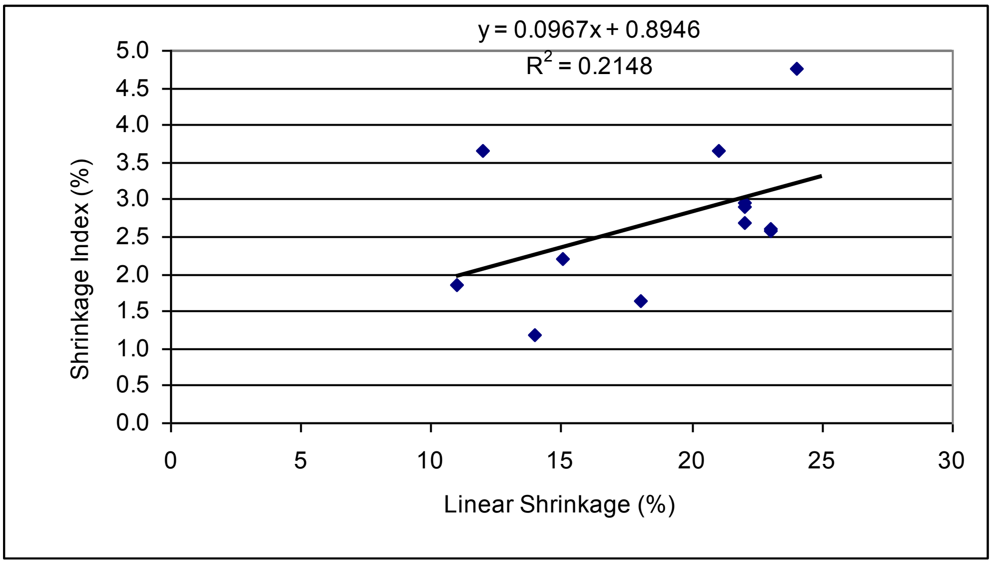


Figure 14 Shrinkage index vs linear shrinkage for Stage II test data

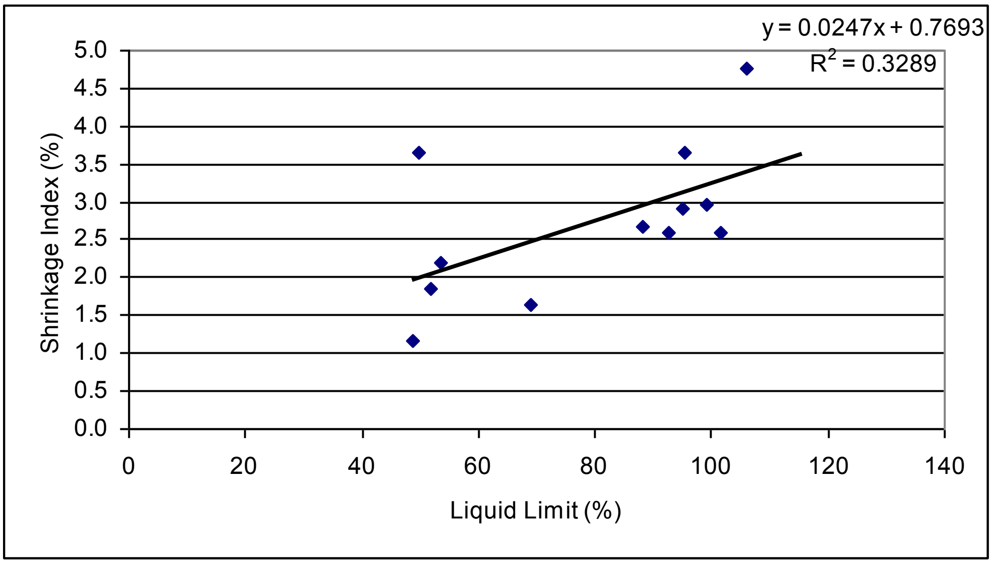


Figure 15 Shrinkage index vs liquid limit for Stage II test data

10.0 CLASSIFICATION OF SUMMER CONDITIONS

10.1 Extrapolation for design drought conditions

When the Stage II expansive soils investigation was set up it was anticipated that some correlation between the extensometer readings and the SMD values on the day of the reading would be obtained, and that this correlation could be extrapolated to the designated drought condition. During the first order appraisal of the movements recorded in the extensometers over the Stage II monitoring period and comparison with the SMD values occurring at the time of measurement, it became apparent that the expected correlation was not occurring, with the larger recorded movements generally not matching the most negative SMD values. Examples from the Manurewa and Otara test sites illustrating the lack of correlation (and even a negative correlation) are presented in Figure 16.

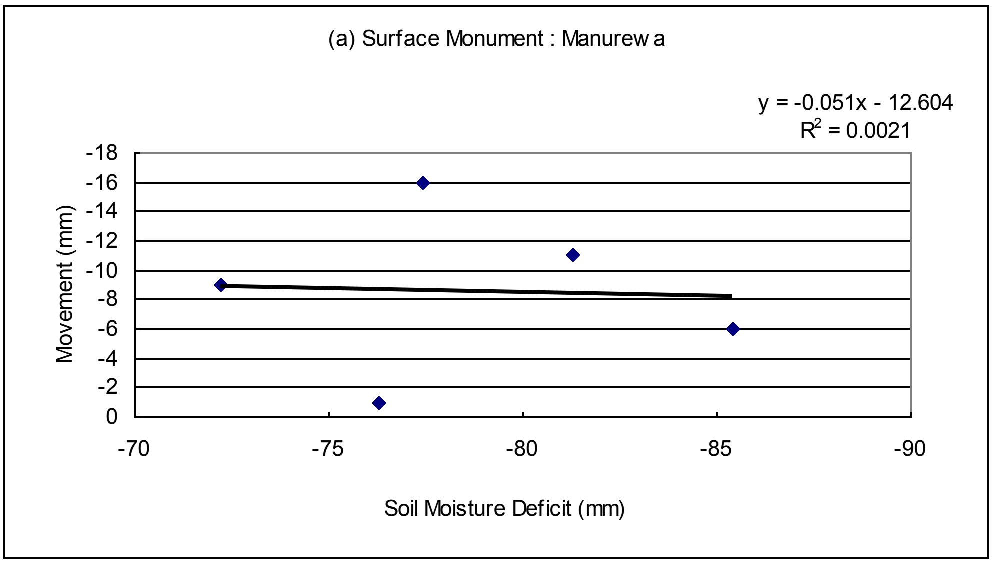
Furthermore, it was observed that the largest recorded movements generally occurred towards the end of the summer period, indicating that there is a lag between "dry" soil moisture conditions and response within the soil zone. On reflection, this observation is not unexpected in that the intensity and duration of preceding moisture conditions would be expected to have a bearing on the soil response. For example, a spell of prolonged dry weather without significant rainfall would be expected to have less effect on subsurface conditions if it occurs at the start of a summer period than in the middle of the summer after the subsoils have already experienced a period of relatively dry weather.

A method of quantifying the intensity of summer conditions was therefore devised that would be better able to reflect the climate conditions preceding the sampling dates. Discussions were undertaken with Mr Alan Porteous of NIWA, which confirmed that the proposed method was valid.

The method applies an "area under the curve" approach to the plot of SMD (mm) against time (days). The deficit below a "nominated threshold" value leading up to the sampling date is accumulated on a daily basis. The "nominated threshold" value is the SMD (SMD) value corresponding to the onset of relatively dry conditions and has been set at a value of -70 mm for the purposes of this study. No negative influence for occasional significant rainfall events, which might otherwise reduce the cumulative deficit value, has been allowed for. The "area under the curve" output so computed is referred to as "cumulative deficit days" (CDD).

The CDD values are provided in the site summary reports presented in Appendix C. The effectiveness of this approach is indicated in Figure 17 by the markedly increased R^2 values for the same Manurewa and Otara test sites from Figure 16.

In adopting this approach, it is acknowledged that it is a simplified method involving judgement and that there are a number of areas where errors can arise as a consequence. For example, a difficulty arises in evaluating and quantifying the differences of dry periods in terms of the SMD values over shorter periods of more intense drying, to values greater than say -90 mm, compared to a longer period with less intense values varying between say -85 mm and -90 mm. In a specific example from the Stage II data, it is noted that during the 2005 summer period the SMD reading did not reach the theoretical drought condition of -90 mm. However, the total rainfall that fell between January and April was the lowest on record for that four month period.



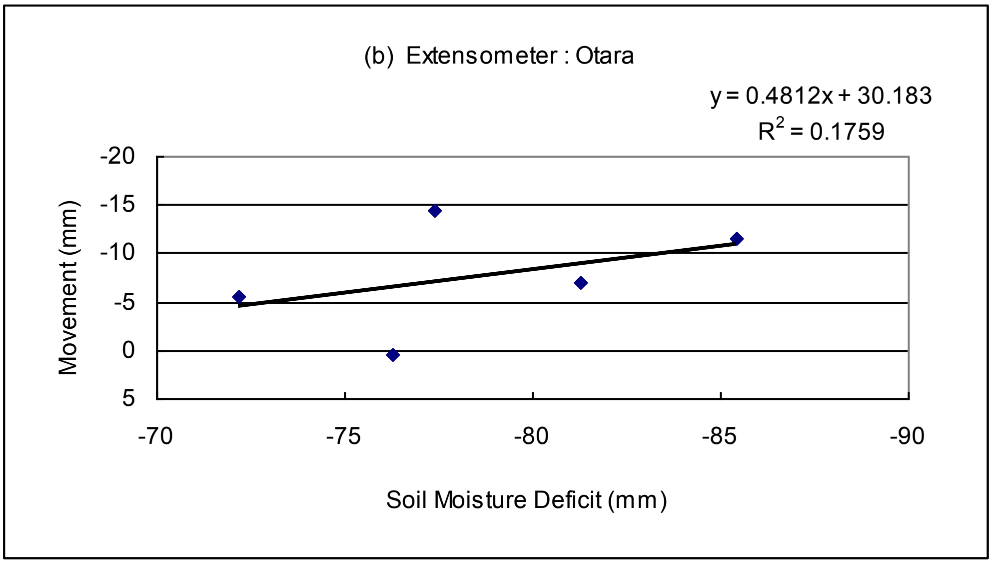


Figure 16 **Extensometer results plotted against soil moisture deficit (SMD) values (mm) for:**
(a) Manurewa surface monument; and
(b) Otara 0.5 m extensometer.

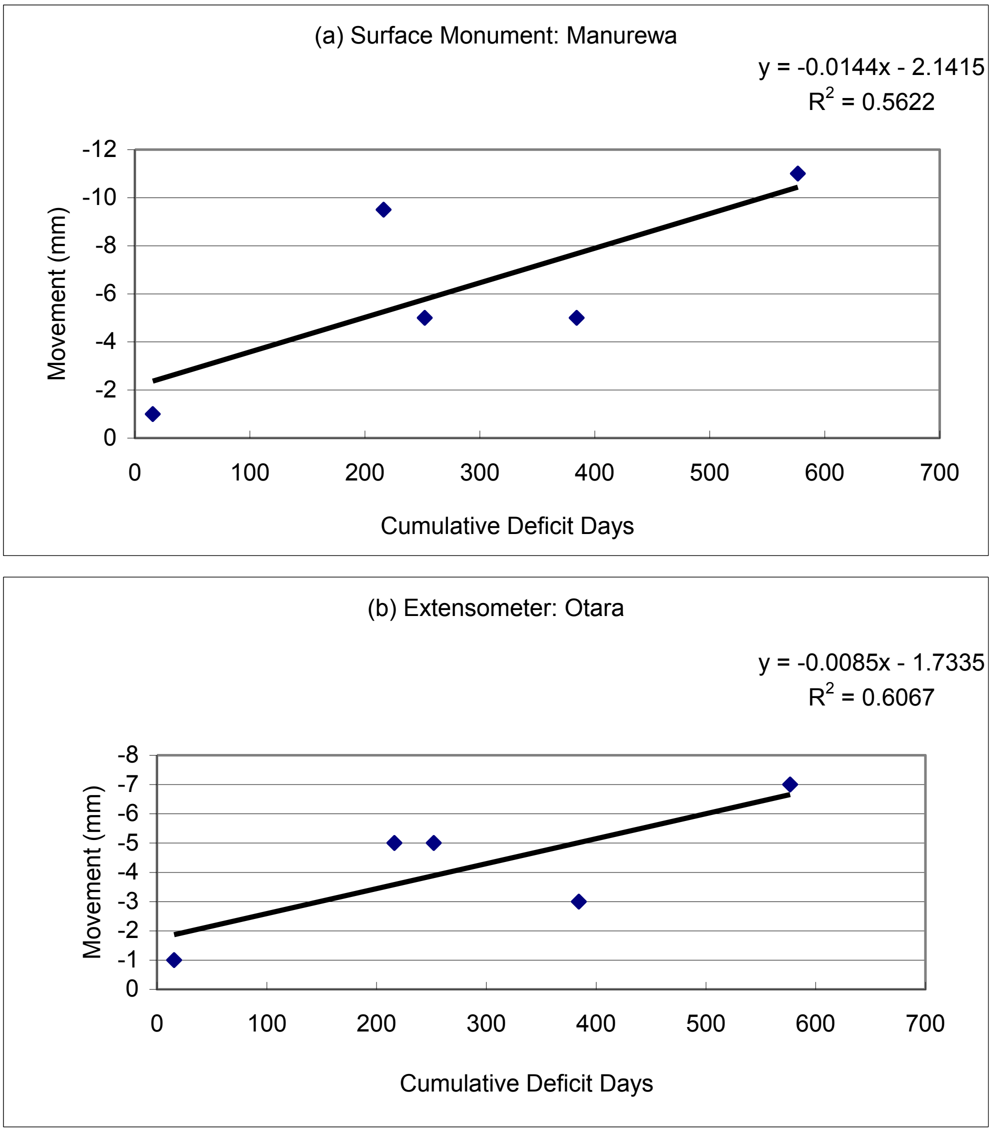


Figure 17 **Extensometer results plotted against cumulative deficit days (CDD) for:**
(a) Manurewa surface monument; and
(b) Otara 0.5 m extensometer

The CDD approach does, however, provide a means of estimating the dryness of the ground conditions preceding the recording date and a "best-guess" estimation of the movement that is likely to occur for extrapolating to future drought conditions.

Notwithstanding its acknowledged limitations, analyses have been carried out to determine the robustness of the CDD methodology. SMD data has been examined for the Auckland Airport and Auckland Owairaka (Mt Albert) weather stations extending back to 1962 and 1954 respectively.

In order to test the robustness of the nominal -70 mm threshold that has been selected, the CDD values for each summer were computed and ranked from highest to lowest for the Auckland Airport data for threshold values of -67.5 mm, -70 mm and -72.5 mm. All three threshold values yielded similar rankings of the summer periods, indicating that the nominal value of -70 mm is appropriate.

The CDD values for each summer were computed and ranked from highest to lowest, and then compared between the Auckland Airport and Auckland Owairaka stations. It was observed that although some variation of the ranking of the years occurred, the approach generally yielded similar results, again indicating that some degree of robustness around the approach that has been used for this study.

It is the opinion of the authors that while acknowledging the simplifications and assumptions within the CDD approach, the method provides a suitable means of analysing and extrapolating the Stage II data so that:

(a) Inferences can be drawn as to the expansive characteristics and behaviour of soils in the Auckland region; and

(b) The response of the soils under potential drought conditions can be predicted.

10.2 Soil suction results

As part of the Stage II investigation the measurement and testing of the soil suction profile has been carried out at the six sites according to the methodology and practices developed for the Stage I Study Report. The soil suction test results are shown in Section 1.0 of the site summary sheets presented in Appendix C of this report.

The winter 2004 suction tests and the first set of the summer 2004/05 period suction tests were taken at sample depths of 0.5 m, 1 m, 1.5 m, 2 m and 2.5 m. As discussed in Section 9.2, the extensometer data obtained during the course of the Stage II investigation indicated that the ground movements were occurring only within the upper metre of the soil profile and did not extend to greater than approximately 1.5 m depth. In order to better target the soils over the depth of soil suction change, the remaining sets of suction samples were obtained at depths of 0.2 m or 0.3 m (immediately below the topsoil layer), 0.5 m, 1 m, 1.5 m and 2 m.

In accordance with the Stage I methodology, soil suction test results of less than 3.2 pF for Sites 2A to 2F inclusive shown in Appendix C have been recorded as 3.2 pF. Graphic representations of the water content results and change in soil suction values are also shown in Appendix C. In calculating the change in soil suction values the summer test values have been generally compared to the preceding winter soil suction values, with the exception of the 0.2 m/0.3 m suction test results, which are all compared to the winter 2005 test results.

With respect to the soil suction test results presented on the site summary sheets in Appendix C of this report, the following observations are made:

(a) The maximum soil suction measurements recorded ranged from 3.70 pF at Sites 2A and 2F (Manurewa and Pukekohe) to 4.05 at Site 2E (Red Beach).

(b) In general, the maximum change in soil suction occurred in the samples obtained from immediately below the ground surface for each of the summer periods monitored, although there are some examples where the maximum suction change was located in the deeper parts of the soil profile.

(c) The maximum amount of soil suction change at the six sites was in the range of 0.4 pF to 0.85 pF.

(d) In some cases, particularly in the deeper parts of the profiles, negative changes were identified i.e. the soil suction measured in summer was less than that measured in winter.

(e) The maximum change in soil suction values at the Manurewa, Howick and Red Beach sites during Stage II was 0.5 pF, 0.4 pF and 0.6 pF. These are not particularly greater than the maximums of 0.4 pF, 0.3 pF and 0.3 pF respectively recorded for the significantly wetter summer periods during the Stage I investigation.

(f) The change in soil suction values for the 0.2/0.3 m deep test (and using the 0.5 m deep sample for S1) were plotted against the CDD values discussed in Section 10.0. No discernable relationship was determined between these parameters.

Table 14 shows the measured extensometer movements plotted against the change in soil suction values over the various depths for all six sites.

As can be observed from the comparison of the data, examples can be shown where high recorded suction changes match high extensometer movements, such as those cases highlighted in yellow.

However, many more cases have been observed such as those highlighted in green, where high extensometer movements were recorded with little or no soil suction change and where high soil suction changes were determined with little or no extensometer movement.

Cross-correlation of the recorded changes in soil suction with the measured extensometer movements and calculated CDD values suggest that:

(a) The recorded changes in soil suction are not readily able to be correlated with the measured extensometer movements; and

(b) There is only a weak correlation between soil suction change at the soil surface and CDD.

Table 14 Comparison table between extensometer movement and soil suction change

SITE	Depth(m)	28-Jan-05		17-Mar-05		28-Apr-05		24-Feb-06		25-Mar-06	
		Movement	delta U	Movement	delta U	Movement	delta U	Movement	delta U	Movement	delta U
Manurewa	0.2	-	-	-10	0.4	-4	0.0	-4	0.4	-11	0.5
	0.5	-1	0.1	-11	0.0	-9	0.0	-6	0.3	-16	0.1
	1	-1	0.2	-6	0.2	-6	0.0	-3	0.2	-11	0.3
	1.5	0	0.0	-1	0.0	-2	0.0	0	0.4	-5	0.3
	2	0	0.3	0	0.1	-1	0.1	0	0.0	-1	0.0
Otara	0.2	1	-	-7	0.4	-6	0.0	-12	0.4	-15	0.8
	0.5	-1	0.2	-5	0.3	-3	0.2	-5	0.6	-7	0.7
	1	-2	0.3	-2	0.3	-1	0.4	-2	0.1	-2	0.2
	1.5	-1	-	-1	-	0	0.0	-1	-0.1	-1	0.0
	2	-1	0.2	-1	-0.1	0	-0.1	-1	0.0	-1	0.0
Howick	0.2	-8	-	-12	0.4	-2	0.0	-16	0.2	-21	0.2
	0.5	-2	0.2	-3	0.1	-1	0.0	-2	0.1	-4	0.2
	1	-1	0.1	0	0.0	-1	-0.1	-2	0.2	-1	0.0
	1.5	0	0.0	1	-0.2	-1	-0.2	0	0.1	0	0.2
	2	0	0.0	1	0.0	0	0.0	0	0.0	0	0.0
North Shore	0.3	-1	-	0	0.4	-1	0.0	-2	-	1	0.8
	0.5	-1	-	0	0.3	0	0.0	-1	-	-1	0.85
	1	0	-	0	0.0	0	0.0	0	-	1	0.7
	1.5	-3	-	-1	-0.2	-3	-0.1	-3	-	-2	0.3
	2	-1	-	1	-0.2	-1	-0.3	-1	-	0	0.1
Red Beach	0.2	-1	-	-12	0.4	-17	0.0	-19	-	-19	0.6
	0.5	1	-0.3	-1	0.0	-5	-0.1	-7	-	-8	0.6
	1	0	0.1	-1	0.0	0	0.0	-1	-	-1	0.5
	1.5	0	0.0	-1	0.0	0	0.0	0	-	1	0.3
	2	0	-0.1	-1	-0.1	0	0.0	-1	-	0	0.2
Pukekohe	0.2	1	-	-2	0.4	-1	0.0	-4	-	-5	0.3
	0.5	1	0.5	0	0.2	0	0.1	0	-	-1	0.2
	1	2	0.2	0	0.0	1	0.0	0	-	-1	0.0
	1.5	1	0.2	0	0.1	0	0.0	-1	-	-1	0.0
	2	1	0.1	0	0.0	0	0.0	0	-	0	0.0

NOTE: (1) 0.2m or 0.3m depth of movement has been calculated as the average of the two surface monument measurements.
(2) Movement measurements and soil suction changes are in respect to a reference measurement taken on 18-20 October 2005

10.3 Measured ground surface movements and correlation with inferred soil suction change

The theoretical ground surface movement (y_s) for Sites 2A to 2F have been calculated for the measured soil suction change values obtained during the Stage II investigation reported herein. Theoretical surface movements (y_s) have been calculated according to the relationship given by Equation 2 below (from Section 5.2) and using the shrinkage index test results discussed in Section 9.3.3, a depth of design suction change (H_s) of 1.5 m, and the soil suction change values (Δu) obtained for each soil layer.

$$y_s = \frac{1}{100}\int_0^{Hs} I_{pt}\,\Delta u \Delta \mathrm{d}h$$

Equation 5
(refer Section 5.2)

where

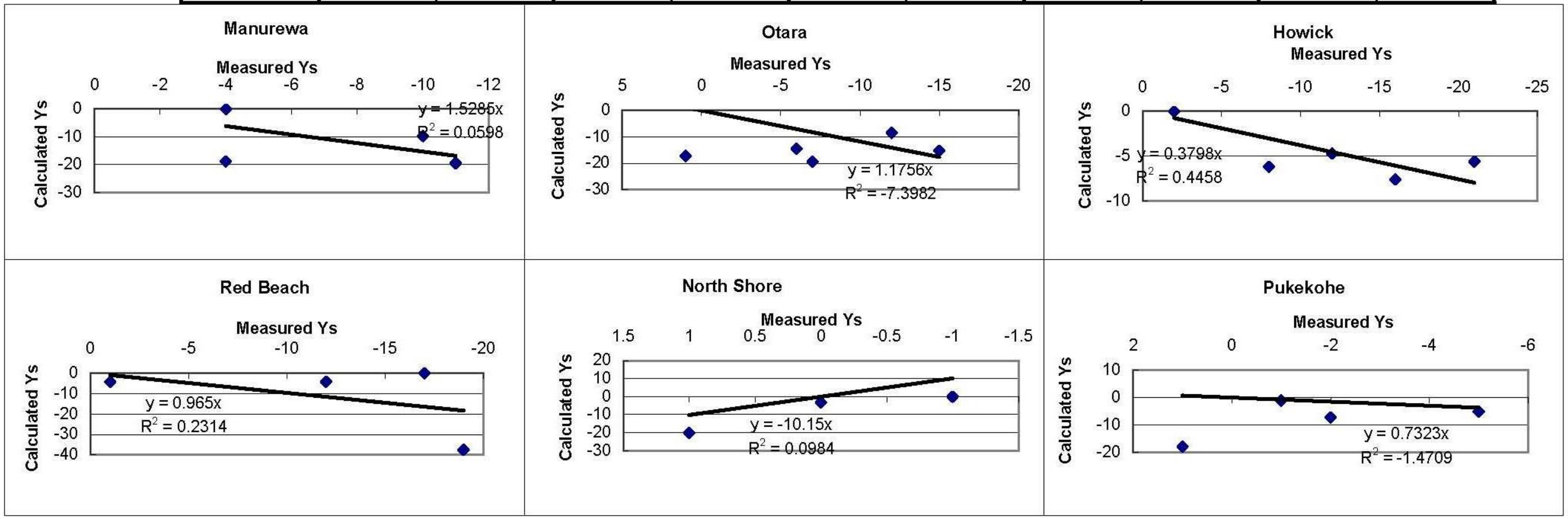
y_s	=	design characteristic surface movement (mm)
I_{pt}	=	instability index (%)
Δu	=	suction change at depth (h) from the surface (pF)
Δh	=	thickness of soil layer under consideration (m)
H_s	=	depth below which no moisture or soil suction change occurs (m)

The calculated theoretical ground surface movements for the five summer monitoring periods are shown alongside the corresponding measured extensometer movements at the top of the soil column (being the average of the two surface monuments) in Table 15. The theoretical and measured movements for each site have been plotted against each other on the graphics underlying Table 15.

As can be observed by the theoretical and measured ground surface movements and the associated plots, there is generally poor or negative correlation between the calculated theoretical and actual measured ground surface movements.

Table 15 Comparison between measured soil surface movements and calculated soil surface movements based on soil suction results

SITE	28-Jan-05		17-Mar-05		28-Apr-05		24-Feb-06		25-Mar-06	
	Measured	Calculated	Measured	Calculated	Measured	Calculated	Measured	Calculated	Measured	Calculated
Manurewa	-	-8.3	-10	-9.7	-4	0	-4	-18.8	-11	-19.5
Otara	1	-17.2	-7	-19.3	-6	-14.5	-12	-8.5	-15	-15.2
Howick	-8	-6.2	-12	-4.7	-2	0	-16	-7.6	-21	-5.6
North Shore	-1	-	0	-3.1	-1	0	-2	-	1	-20.3
Red Beach	-1	-4.3	-12	-4.2	-17	0	-19	-	-19	-37.5
Pukekohe	1	-17.8	-2	-7.2	-1	-1.1	-4	-	-5	-5



As the shrinkage index and the depth of moisture change are relatively simple parameters to measure, it is considered that the more variable soil suction change values are resulting in the lack of correlation between the measured and theoretical y_s values, as further discussed in Section 11.0.

Because of the higher confidence in the measured y_s values obtained from the extensometers, as compared with the theoretical y_s values obtained from the soil suction test data, it is considered more appropriate on the basis of the data obtained during the Stage II investigation to adopt the measured y_s values for back analysis to obtain the change in soil suction value at the soil surface (Δu_s) for use in the extrapolation to the drought conditions for the Auckland region discussed in Section 13.0.

As a check on this method an analysis was undertaken for the Stage II data to determine if any correlation existed between the maximum recorded values rather than the individual profiles. The maximum soil suction change value recorded at each site over the Stage II period was entered into the theoretical movement calculations for depths of moisture change of 1 m and 1.5 m and compared to the maximum extensometer reading obtained at each site. These calculations and comparisons are shown in Table 16.

Table 16 **Comparison of theoretical movements using maximum soil suction change to maximum measured extensometer movement for each site**

SITE	Maximum delta U (pF)	Theoretical y_s		Maximum measured y_s (mm)
		H_s = 1.0m	H_s = 1.5m	
Manurewa	0.5	8.4	12.4	16
Otara	0.8	12.4	18.2	17
Howick	0.4	6.3	9.3	21
North Shore	0.85	7.2	10.6	2
Red Beach	0.6	13.5	19.8	22
Pukekohe	0.4	6.8	10.0	7

The calculated theoretical y_s values shown in Table 16 give a reasonably good approximation to the maximum y_s values measured by the extensometers for a depth of moisture change (H_s) of 1.5 m used with the maximum soil suction change over the Stage II period. The main anomalies appear to be the Howick site where high measured movement occurred with only small soil suction changes, and the North Shore site where high soil suction changes were recorded but almost imperceptible extensometer movement was measured over the Stage II period.

In general, however, it would appear that the "predicted maximum" assessment process is sufficiently robust to give reasonable correlations between the extensometer movements and the theoretical calculated movements. It is therefore considered that it is appropriate to extrapolate the extensometer data for use in the back analyses to determine the design soil suction profile for the Auckland region.

11.0 DRY SUMMER PREDICTIONS FOR AUCKLAND REGION

11.1 General

As discussed in Section 10.0, the CDD approach is considered to give the most reliable method of evaluating dry summer soil conditions for the purposes of this report.

Table 17 lists the top 10 ranked summer periods for the Auckland Airport data over the period 1962 to 2006 on the basis of the CDD calculations.

Table 17 Cumulative deficit days (CDD) values for top 10 summer periods at Auckland Airport

Rank	Year	CDD
1	1974	1376
2	1978	1263
3	1973	1181
4	1998	1164
5	1970	827

Rank	Year	CDD
6	1983	716
7	1981	694
8	1994	661
9	2006	654
10	1993	634

It is noted that the 2006 CDD value is ranked 9th out of the 44-year data set. The 2005 summer period was ranked 17th. These rankings are indicative of the general opinion that the Stage II summer periods were "reasonably dry", particularly when compared to the Stage I summer periods of 2002 and 2003, which had rankings of 37th and 34th respectively.

Of the years on record, the 1974 and 1998 summer periods encompassed the lowest consecutive sets of daily SMD readings below the -90 mm threshold, being 34 days and 23 days respectively. The CDD method bares this out, albeit imperfectly, ranking these summers as 1st and 4th respectively. Our recollections of the 1998 summer as being particularly dry, with some issues relating to expansive soils and subgrade preparation occurring is, in our opinion, consistent with its ranking of 4th shown in Table 17.

The commentary to AS 2870:1996 – Supplement 1 states that:

> The Standard describes the properties of the foundation by one parameter, the expected free surface movement, y_s. This is the vertical movement range expected during the life of the house from a reasonable estimate of dry conditions to a reasonable estimate of wet conditions and does not take into account the moderating effect of the footing system. The Standard nominates 50 years as the "life" of the house and "reasonable" as the level that could be expected for 19 houses out of every 20. This does not mean that the house is not expected to last more than 50 years nor that 1 in 20 houses could fail. It is, however, more reliable than using average conditions or an undefinable "extreme" concept.
>
> In this assessment, y_s should be interpreted as the characteristic value that has a 5 percent chance of being exceeded in the life of the house which may be taken as 50 years.
>
> The characteristic value is defined as the value that has a 95 percent chance of occurring in the life of the structure. Thus it is not necessary to consider extremes of drying or wetting of the profile.

A strict interpretation of the AS 2870 commentary would lead to the consideration that in the 50-year design life of a residential building only one-in-20 buildings will develop a problem relating to expansive soils i.e. that the probability of any one house developing a problem due to expansive soils within any given year is:

= 1/20 x 1/50
= 0.001 or one-in-1000
= 0.1% annual exceedance probability (AEP).

This interpretation of the design probabilities is supported by HB 28:1997 which states:

> The definition also includes the concept that y_s is a value that has a 5% chance of being exceeded in the 50-year nominal life of a house. Thus it is not the most extreme value possible at a particular site, although with only one chance in a thousand of occurring at a site in any one year, it should not be a common event. The definition of the design surface movement is similar to that used for wind and floor loads.

It is noted that a strict mathematical interpretation of a "5% chance in 50 years" gives a return period of 975 years i.e. 1000 years is an approximation of sufficient accuracy for the purposes of this report.

To show:

A 5% probability in 50 years corresponds to a return period of 975 years (so that 1000 years is only an approximation).

Explanation:

Let P = Probability in one year
$1-P$ = Probability not occurring in one year

$(1-P)^n$ = Probability not occurring in n years
$1-(1-P)^n$ = Probability occurring in n years

i.e. for 5% probability in 50 years

$0.05= 1-(1-P)^{50}$

Rearranging to solve for P gives
$P =1-0.95^{1/50}$
$=0.001025$
$=0.1025\%$

Let T be the period, then
$T=1/P=975$ *years*

Hence annual probability is 0.1%, corresponding to a return period of 975 years.

11.2 Statistics – extreme value analysis

11.2.1 Theory

The analysis and statistical treatment of extreme events such as the return periods of floods comes within Type 1 Extreme Value Theory, commonly known as Gumbel Analysis. This provides a statistical treatment of data sets that follow a Gumbel Distribution for "many small events and few large events".

Although periods of dry weather and droughts follow a different spatial time span and intensity than one-off events such as rainfall storms, Gumbel Analysis nevertheless provides a means of estimating and modelling the true statistical distribution and behaviour of dry periods represented by the CDD data adopted in this study.

For the Gumbel Analyses carried out for this report, the CDD values have been calculated and ranked from lowest to highest from $i = 1$ for the lowest CDD values and $i = N$ for the highest ranked CDD value. Only positive CDD values were included in the analyses so that CDD values of zero did not impose a skew to the tail of the analysis.

For each CDD the estimated probability of a smaller CDD is given by:

$$P_i = i / (N+1)$$

and

$$X_i = -\ln [-\ln (i/N+1)],$$ where

i = rank of event (CDD value) for $i = 1, 2, 3 .... N$
where N = total number of events (i.e. CDD sample size)

P_i = estimated probability of the occurrence of a smaller CDD event than i

X_i = dimensionless expression of rank of i to enable plotting with CDD value of i

Plotting CDD against X_i gives a linear relationship that can be modelled by a simple regression analysis from which the CDD value corresponding to a return period T is given by:

$$X_T = -\ln [-\ln (1-1/T)]$$

11.2.2 Data sets adopted in Stage II

The CDD data sets adopted for this report are provided in Appendix D. NIWA have provided data extending from 1963 to 2006 for the Auckland Airport monitoring station (used for the Manurewa, Otara, and Howick sites), from 1967 to 2006 for the Albany station (used for Mairangi Bay and Red Beach sites), and from 1996 to 2006 for the Pukekohe station (Pukekohe site).

The return periods of interest to this study are the 20, 50, 100, 300, 500 and 1000-year return period CDD values. The calculated CDD values associated with each of the foregoing return periods for the Auckland Airport, Albany and Pukekohe weather stations are presented in Table 18. The CDD values for the Stage II monitoring period are also presented for comparison.

From Table 18 it can be observed that the 2005 summer event at Albany and the 2006 summer event at Pukekohe were greater than one-in-20-year return period events. In particular, the 2005 Albany CDD value was the highest value within the 39-year record available. In comparison, the 2006 Airport CDD value was not even a one-in-five-year event (CDD value of 721) for that data set.

Table 18 **Cumulative deficit days (CDD) calculated by Gumbel Analyses for various return periods**

	Cumulative deficit days		
Return period (years)	**Auckland Airport**	**Albany**	**Pukekohe**
20	1178	980	631
50	1468	1220	800
100	1686	1402	926
300	2028	1687	1126
500	2186	1820	1218
1000	2404	2001	1345
2005 summer	394	1053	308
2006 summer	654	482	672

It is considered that this variability of the CDD values highlights two important factors:

(a) The high variability of weather conditions in the Greater Auckland Region and the difficulty in extrapolating weather phenomena. The corollary to this is that there is uncertainty in the assumption that the data recorded at the weather stations is representative of the weather conditions and soil responses at the test sites. If further research is undertaken into the expansivity of soils in the Auckland region, it is recommended that consideration be given to installing and monitoring a set of extensometers adjacent to a weather recording station to help remove some of this uncertainty.

(b) A potential lack of robustness in the CDD approach. While considerable effort has been applied to come up with an accurate method as possible to assess drought/dry period conditions which is based on a reasonably scientific approach, the method is unsubstantiated. It is a broad-brush approach that is the best that the authors could devise within the confines and limitations of this study.

11.3 Results and extrapolations

The correlations between the measured ground surface movements (measured by the extensometers) and the CDD values for each of the five relevant sites (i.e. excluding the North Shore site) are shown on Figure 18 of this report. The projection equations from these correlations have been used to determine the predicted ground surface movement (y_s) for the CDD values that correspond to the 20, 50, 100, 300, 500 and 1000-year return periods shown in Table 18. The predicted y_s values for the foregoing return periods are shown in Table 20.

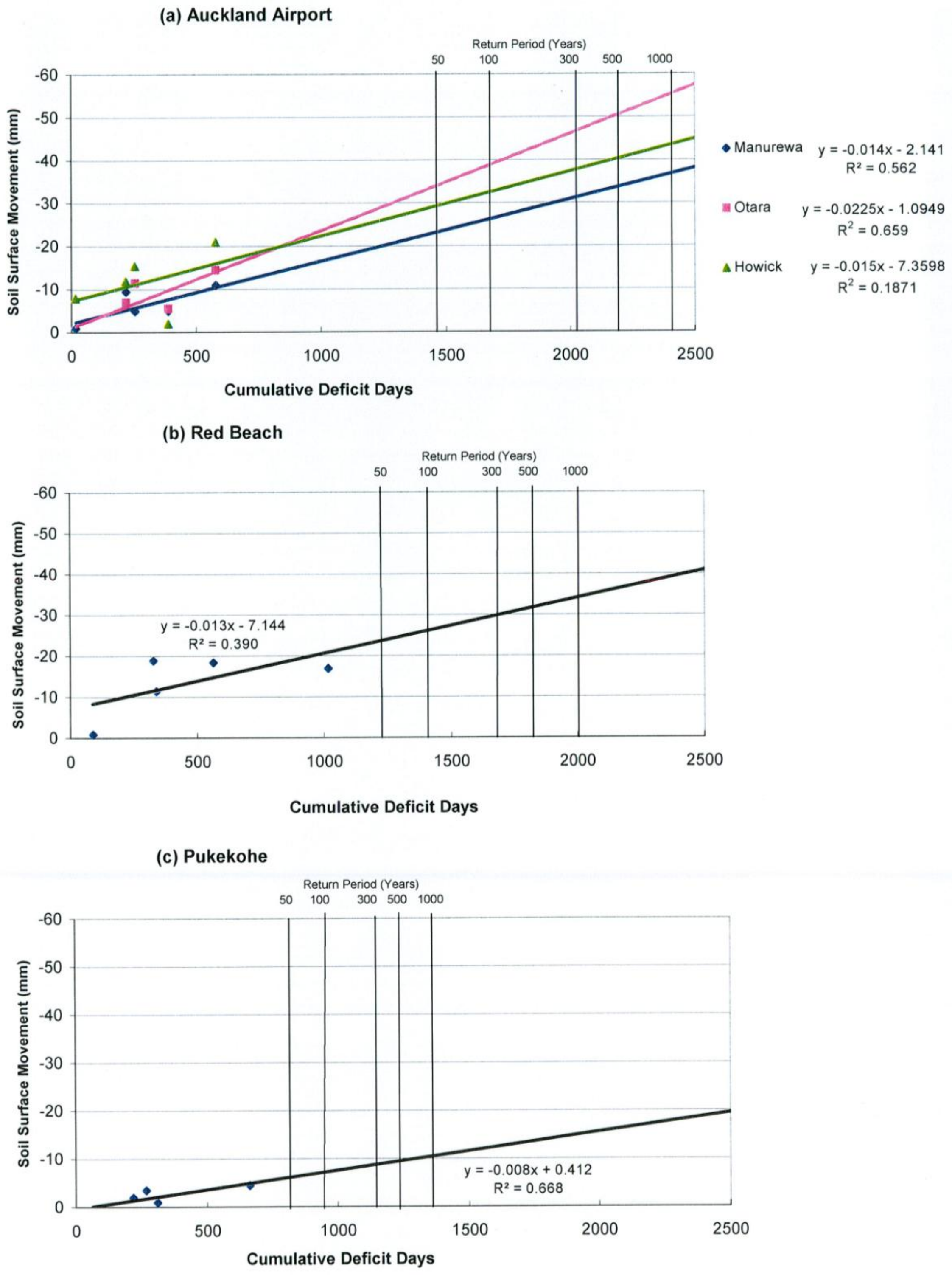


Figure 18 Measured soil surface movements plotted against CDD and return period

Having obtained the predicted ground surface movement (y_s) for a particular return period, knowing the tested shrinkage index values (I_{ps}, Section 9.3.3), and adopting an H_s value of 1.5 m, analyses have been undertaken using Equation 2 to determine the theoretical soil suction at the soil surface (Δu_s) that is required to match the predicted ground surface movement.

The equivalent I_{ps} values for each site, for an H_s value of 1.5 m, are shown in Table 19.

Table 19 Equivalent I_{ps} values

Site	0 to 0.5 H_s	0.5 H_s to 1.0 H_s
Manurewa	2.2	3.7
Otara	1.9	3.6
Howick	2.6	2.6
Red Beach	3.0	4.8
Pukekohe	2.7	2.9

Note: H_s = 1.5 m

The predicted y_s and the corresponding Δu_s values for the 20, 50, 100, 300, 500 and 1000-year return periods are presented in Table 20. As no reliable correlation or measured extensometer movement was obtained from the North Shore site, that site has been omitted from the extrapolations.

Table 20 Analysis results showing Δu_s values required to match y_s movements predicted by Gumbel Analyses for 20, 50, 100, 300, 500 and 1000-year return period events.

SITE	Prediction Equation	1 in 20 yr event		1 in 50 yr event		1 in 100 yr event	
		Y_s Predicted	Δu_s required	Y_s Predicted	Δu_s required	Y_s Predicted	Δu_s required
Manurewa	Y_s = -0.0144x - 2.1415	-19.1	0.77	-23.3	0.94	-26.4	1.06
Otara	Y_s = -0.0225x - 1.0949	-27.6	1.21	-34.2	1.50	-39.0	1.71
Howick	Y_s = -0.015x - 7.3598	-25.1	1.08	-29.4	1.26	-32.6	1.40
Red Beach	Y_s = -0.0135x - 7.1449	-20.4	0.62	-23.6	0.72	-26.1	0.79
Pukekohe	Y_s = -0.008x + 0.4126	-4.6	0.19	-6.0	0.24	-7.0	0.28
Average		-19.4	0.77	-23.3	0.93	-26.2	1.05

SITE	Prediction Equation	1 in 300 yr event		1 in 500 yr event		1 in 1000 yr event	
		Y_s Predicted	Δu_s required	Y_s Predicted	Δu_s required	Y_s Predicted	Δu_s required
Manurewa	Y_s = -0.0144x - 2.1415	-31.3	1.26	-33.6	1.35	-36.8	1.48
Otara	Y_s = -0.0225x - 1.0949	-46.7	2.05	-50.3	2.21	-55.2	2.42
Howick	Y_s = -0.015x - 7.3598	-37.8	1.62	-40.1	1.72	-43.4	1.86
Red Beach	Y_s = -0.0135x - 7.1449	-29.9	0.91	-31.7	0.96	-34.2	1.04
Pukekohe	Y_s = -0.008x + 0.4126	-8.6	0.35	-9.3	0.37	-10.3	0.41
Average		-30.9	1.24	-33.0	1.32	-36.0	1.44

Note: 1. The Δu values are based on an H_s value of 1.5 m.
2. The prediction equations correspond to the equations for the surface monuments presented in Appendix C.

Using the 100 and 500-year return period drought events with a mid-range return period of 300 years to define a benchmark range for foundation performance, the following conclusions can be drawn from the data presented in Table 20:

(a) For an assumed H_s of 1.5 m, the average Δu_s ranges between 1.05 pF and 1.32 pF, with a value of 1.24 pF for the 300-year return period drought event, which approximates the lower Δu value of 1.2 pF in AS 2870 Table 2.4.

(b) The predicted ground surface movements (y_s) for four of the five sites are less than or equal to 40 mm and therefore fall into the slight to moderate soil classification category of AS 2870 (i.e. $y_s \leq 40$ mm).

(c) Using the 1000-year return period event as the basis for the foundation distortion limit for an extreme drought condition, for an assumed H_s of 1.5 m, the average Δu_s is 1.44.

The Stage I report presented three possible soil suction profiles to be used in the calculation of the design surface movement y_s, viz Profiles Alpha, Beta and Gamma, as shown in Table 21.

Table 21 Stage I soil suction profiles

Profile	H_s	Δu_s
Alpha	2.0	1.5
Beta	1.5	1.5
Gamma	1.5	1.2

In the Stage I study it was recommended that soil suction Profile Alpha be adopted as a conservative measure until further evidence had been obtained. The projected Δu_s values from the CDD values presented in Table 18 and (b) and (c) above, suggest that soil suction Profile Gamma is appropriate for use in calculating the theoretical design surface movement in assessing the expansivity of soils in the Auckland region.

Performance and correlation of extensometer movements and soil suction measurements under severe drought conditions would, however, be required in order to confirm the design conditions should such a severe drought occur.

11.4 Scaling factors

It is recommended in Section 14.0 that foundations be designed to perform under drought conditions with a return period in the range 100 to 500 years, with a mid-range value of 300 years, with the design verified for an extreme drought with a return period of 1000 years.

If a scaling factor of 1.00 is adopted for the calculated y_s corresponding to the 300-year return period design drought, the scaling factors shown in Table 22 have been determined for various other return period events, based on the predicted y_s values shown in Table 20.

For any selected return period drought event, the y_s value calculated using soil suction Profile Gamma (H_s = 1.5 m, Δu_s = 1.2 pF) may be scaled using the appropriate scaling factor from Table 22.

Table 22 Scaling factors

Return period	Scaling factor
20 years	0.65
50 years	0.78
100 years	0.88
300 years	1.00
500 years	1.11
1000 years	1.21

It is suggested the scaling factors in Table 22 be used to assess foundation performance as follows:

(a) The serviceability performance of any particular foundation system be determined for the design drought, with a 300-year return period, using the y_s value calculated for soil suction Profile Gamma and a scaling factor of 1.0.

(b) The foundation performance for the extreme drought event, with a 1000-year return period, be determined either:

(i) By applying a scaling factor of 1.2 to the y_s value calculated for soil suction Profile Gamma for the purposes of the soil foundation interaction analysis prescribed by AS 2870 Part 4 (e.g. Mitchell Method); or

(ii) By calculating the flexural demand ($øM_u$) for foundation elements calculated from Profile Gamma (H_s = 1.5, Δu = 1.2) to incorporate a universal load factor of 1.2 rather than 1.0.

The result of this will be that foundation strength and stiffness will cover the design drought (i.e. the mid-range one-in-300-year drought event) while the extreme drought event is covered for strength limit state only. Refer also Section 14.2.

The basis for this assessment of foundation performance, based on AS 2870/HB 28:1997 concepts, is discussed further in Appendix E of this report.

11.5 Foundation performance assessment

The shrink-swell index values for tests carried out in the Auckland region are generally in the range of 1% to 7%. Using soil suction Profile Gamma (H_s = 1.5, Δ_u = 1.2), the incremental I_{ss} values will correspond to the theoretical y_s movements and site classifications shown in Table 23.

Table 23 Correlation between shrink-swell index and y_s and site classification

I_{ss} (%)	y_s (mm)	Site classification
1%	10.8	S
2%	21.6	S to M
3%	32.4	M
4%	43.1	M to H
5%	53.9	H
6%	64.7	H
7%	75.5	H to E

From Table 2, the y_s values for the various site classifications are:

(a) Class S = 0–20 mm;

(b) Class M = 20–40 mm;

(c) Class H = 40–70 mm; and

(d) Class E = >70 mm.

It is therefore apparent that the greater part of the measured I_{SS} range (<4%) corresponds to Class S and M, which for normally clad frames will generally be considered to be adequately mitigated by specifying a foundation embedment depth of 450 mm below cleared ground level.

For I_{SS} values in the upper region of the natural range, i.e. $>4\% \leq I_{SS} < 6\%$, a minimum foundation embedment depth of 600 mm below finished external ground levels would generally be sufficient to mitigate against potential shrink-swell issues associated with expansive soils for clad frame, masonry veneer and articulated veneer cladding.

These indicative foundation depth requirements will satisfy angular distortion requirements for external wall cladding, but will not mitigate heave or sag which might arise from insufficient site pre-treatment (refer Section 12.0).

Specific design of slab and foundation systems to limit angular distortion will generally be required for foundations on Class H and E sites.

11.6 Effects of climate change

The methodology and field data correlation presented in this report is based on investigations carried out in the period 2002 to 2006.

Because the primary determinant of the recurrence of the design drought condition is the SMD record supplied by NIWA for the period up to 2006, any impact of future climate change should be reflected through the ongoing monitoring of that record and the CDD parameters derived from it.

Whilst domestic dwellings currently being constructed will have a design life extending some 50 years into the future, the nature of the potential change to the SMD record for Auckland projected over that period is not clear.

It is therefore recommended that the SMD profile for Auckland be re-evaluated at not less than 20-year intervals using the methodology adopted in this report.

Any changes recorded would be reflected in the values of H_s and Δu recommended for foundation design.

11.7 Soil suction change profile for regions other than Auckland

The soil suction change profile recommended in this report is specific to the Auckland region.

The soil suction change profile for other regions in New Zealand depends on the SMD profile for the particular region, which in turn depends on the local climate (plus any change to that local climate in the future).

12.0 RESIDUAL SWELLING POTENTIAL IN AUCKLAND SOILS

12.1 Objective

Since the introduction of NZS 3604:1999, it has become progressively more common for geotechnical practitioners to undertake laboratory testing of subsoils within a subdivisional development in order to classify the expansive characteristics of the subsoils.

The most common method for calculating the design characteristic surface movement by geotechnical practitioners is by determining the shrink-swell index (I_{ss}) in order to estimate the instability index (I_{pt}) for use in calculating the design characteristic surface movement (y_s) value.

The investigation reported herein involved a review of the records on file at the various local TAs in the Auckland region over the previous five years or so in order to obtain copies of the shrink-swell index test data held in those files, and to investigate the extent of swell contribution to the I_{ss} index, and to determine if any significant variation or trends could be established to show if relationships could be inferred for Auckland soils by:

(a) Depth; or

(b) Soil type; or

(c) Sampling condition (summer vs winter).

The investigation methodology and results obtained are discussed separately.

12.2 Option for limiting soil expansivity

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 pre-treated to maintain a high water content for the full duration of the pre-construction period.

The adoption of a scaling factor of around 60% gives rise to the possibility of two considerations that form the basis of the investigation and analyses reported herein:

(a) To determine, on the basis of the available test data for the Auckland region, if the amount of residual swelling can be limited or reduced further than the factor of 0.7 recommended in AS 2870.

(b) To investigate the possible effects of pre-treatment of building sites in the Auckland region.

This report provides the results of the investigation of (a) and (b) above as an interim report to Manukau City Council as part of the Stage II BRANZ investigation into the expansivity characteristics of soils in the Auckland region.

12.3 Conclusion

Based on the analysis of the collected data, it can be inferred that the proportion of swell component within the shrink-swell index test does not exhibit significant variation by season or SMD value, soil type or sample depth.

It therefore follows that, for foundation design purposes, the swell component can be treated equally across soils in the Auckland region. On the basis of the data collected to date, a case can be argued for the soils in the Auckland region being generally at or near their potential maximum swell, even for typical summer conditions.

This means that for a typical building platform, the foundations and grade slab will be unlikely to be subject to significant heave conditions, provided that the subgrade within the footprint of a dwelling is maintained at or close to its natural water content during construction.

12.4 Limited swelling potential

On the basis of the foregoing, it is considered that soil expansivity at a particular site can be limited by specific design (incorporating site pre-treatment), as suggested in Section 12.4.1(b). As a consequence, the following reductions in ground deformation due to shrink-swell can be assumed:

(a) The free unloaded heave (y_m) value can be reduced from a value of 0.7 y_s to 0.4 y_s.

(b) For a site that is subject to pre-treatment or maintenance protection, by either pre-saturation or by the placement of a barrier to subgrade moisture loss respectively, then a design allowance of y_m for edge heave not exceeding 0.2 y_s may be made.

The impacts of these recommendations on sites with highly expansive soils are described in Sections 12.4.1(a) and 12.4.1(b).

12.4.1 AS 2870Appendix F: Soil Parameters and Footing Design Methods

(a) Design recommendation for stiffened raft/waffle slab foundations incorporating perimeter piling on Class H and E sites

Appendix F4 (a) (on p67) 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% can be made.

For stiffened rafts/waffle slab foundations incorporating “unconnected” perimeter piles, where the subgrade has been pre-treated to achieve “near-saturation” conditions prior to casting slab – this leads to the design cases shown in Table 24.

Table 24 Design y_m recommendations for slab foundations incorporating perimeter piles cast on "near saturated" sites

Design case		Design y_m value[1]	Remarks [2], [3]
1(a)	centre heave	+0.2 y_s	Centre heave suppressed by subgrade pre-treatment – to near saturation conditions – residual design allowance say 0.2 y_s max
1(b)	edge subsidence	-0.7 y_s	Edge beam system to be supported by perimeter piling to suppress this case
2(a)	centre subsidence	-0.4 y_s	Refer AS 2870 Appendix F4(a), which allows a 40% reduction in y_m for this case
2(b)	edge heave	+0.2 y_s	Edge beam system either lifts off perimeter piles or piles/edge beam system must resist swelling pressures (say 20 kPa based on residual 10% swelling) if structurally connected

Note:
1. Mound movements are (+) upwards, (-) downwards referenced against mound surface level at time of construction.
2. Where perimeter edge beam supported on piles (as "hard supports" without structural connection to foundation).
3. Piled foundations supporting perimeter beams to be designed for negative skin friction effects, as well as factored design loads imposed onto the pile head.

Immediately prior to casting foundation slab, the residual swell strain must be less than 15% of the confirmed shrink-swell index value when measured in accordance with AS 1289:1998 Method 7.1.1 for the above design basis to apply.

(b) Site pre-treatment (saturation) recommendations in accordance with AS 2870 Appendix F4(a) for Class H and E sites

(i) Record shrink-swell tests (2No) for particular site to verify recommendation of subdivision completion report.

NB: Output = I_{ss} and y_s

(ii) Recover samples for swell tests (0.25 m and 0.75 m depth) at each of 2No locations on-site (1No under edge beam, 1No in central area of the building envelope) and a single sample from 1.5 m depth within the central area of the building envelope.

NB: Samples to be taken after saturation pre-treatment of subgrade and immediately prior to casting of foundation slab, and to be tested in accordance with AS 1289:1998 Method 7.1.1.

(iii) If the swelling strain is less than 15% of the shrink-swell index I_{ss} (%) value, then the reduced centre subsidence (edge heave) provisions of AS 2870 Appendix F4(a) can be used, following the (4No) load cases specified by the design cases in Section 12.4.1(a).

12.5 Application of AS 2870 standard methods

(a) The design values for y_m derived for pre-treated sites described above are directly applicable to standard designs formulated for use with AS 2870, subject to:

(i) Sites being "re-classified" consistent with the foundation system adopted, by direct scaling of the derived y_s values.

(ii) Edge beams associated with standard designs being modified as necessary to accommodate e.g. piled supports.

(b) In general, specific design will be necessary to confirm performance adequacy of foundation elements e.g. edge beams, piles.

13.0 FOUNDATION ANALYSIS: PERFORMANCE FRAMEWORK

13.1 In general, the performance requirement for buildings can be related to

(a) The "**factors** and **events**" that buildings might be subject to;

(b) The **impacts** on buildings, and building users, that society is prepared to tolerate generally; and

(c) How society might **tolerate** different impacts for different types of buildings.

13.2 The factors and events of Section 13.1(a) can be categorised into three groups:

(a) Factors that affect buildings **all the time**, such as effects of gravity or human activity;

(b) **Specific events**, such as earthquake, storms that bring strong winds, heavy rain or snow, and noise nuisance; and

(c) Factors that affect the **ability** of a building to **respond** to demands over a long period of use, such as corrosion, rot and decay, or exposure to UV radiation.

13.3 For the performance requirements of Section 13.1(b) and (c), the Building Code seeks to provide reasonable protection from the effects of demands on buildings, acknowledging that it would be uneconomic and too restrictive to aim to eliminate all risk.

Society, therefore, tolerates some impacts on buildings in certain circumstances. The impacts that are tolerated depend partly on the size of the event that caused them and the likelihood that such an event will happen.

13.4 Using this approach, Code writers have developed descriptions for "tolerable impact levels" (TIL) ranging from "insignificant" to "extreme", setting out in each case what society might tolerate in terms of:

(a) Impacts for occupants;

(b) Economic impacts;

(c) Social impacts;

(d) Environmental impacts;

and then classifying buildings by type and/or intended use against their "tolerable impact".

13.5 Society's tolerance of impacts also depends on how vulnerable the people in the buildings are, and how important the building is to society. Regulators might therefore classify different types of buildings into four main **"performance groups"**, depending on e.g. "life risk" factors such as:

(a) **Function** of the building;

(b) Proportion of time the building is **occupied by people**;

(c) **Familiarity** of occupants within the building; and

(d) Whether vulnerable or **special populations** use the building.

13.6 Under this classification, Performance Group (PG) 4 at the high end of the scale might cover "buildings that are essential to post-disaster recovery, or are associated with hazardous facilities (e.g. hospitals), whereas at the low end of the scale Performance Group (PG) 1 might describe buildings posing low risk to human life, or a low economic cost should the building fail (e.g. ancillary buildings)".

13.7 In addressing the TIL for domestic/residential buildings, it is noted that there has recently been an increased focus on prevention of economic loss (by adequate design, construction, maintenance or otherwise) for homeowners. This is reflected in Building Act 2004 (BA04) Section 4 'Principles to be applied in performing functions or duties, or exercising powers, under this Act'.

BA(4)(2)(a)(i) states in part:

> (a) when dealing with any matter relating to 1 or more household units—
> (i) the role that household units play in the lives of the people who use them, and the importance of—
> (A) the building code as it relates to household units; and
> (B) the need to ensure that household units comply with the building code:
> (ii) the need to ensure that maintenance requirements of household units are reasonable:
> (iii) the desirability of ensuring that owners of household units are aware of the maintenance requirements of their household units.

13.8 It is appropriate that these principles also be acknowledged in the level of conservatism brought to setting up of TILs for dwellings founded on expansive soils.

Range of specific events considered	Tolerable impacts for buildings in Performance Group 2		
	TIL5	TIL4	TIL3
Flooding	1/200	1/100	1/50
Volcanic activity	1/2000	1/1000	1/500
Snow/ice	1/250	1/150	1/50
Wind	1/500	1/500	1/100
Earthquake	1/1000	1/500	1/100
Soil expansivity	1/1000	1/500	1/100

13.9 The Department of Building and Housing (DBH) has recently published a consultation document *Building for the 21st Century: Review of the Building Code* (July 2007) signalling the intended basis for the review of New Zealand's Building Code. From Table 10 of that document, the following "comparable events" are noted that might impact on homeowners with the same TILs.

TABLE 10: PERFORMANCE FRAMEWORK										
Annual Probability of Event					Chances of Event	Tolerable impacts				
						Performance Group				
Flooding	Volcanic Activity	Snow/Ice	Wind	Earthquake		1	2	3	4
1/500	1/5000	1/500	1/2500	1/2500	Extremely Low		Tolerable Impact Level 6 (Extreme)	Tolerable Impact Level 5 (Very Severe)	Tolerable Impact Level 5 (Very Severe)
1/200	1/2000	1/250	1/500	1/1000	Very Low	Tolerable Impact Level 6 (Extreme)	Tolerable Impact Level 5 (Very Severe)	Tolerable Impact Level 4 (Severe)	Tolerable Impact Level 4 (Severe)
1/100	1/1000	1/150	1/500	1/500	Low	Tolerable Impact Level 5 (Very Severe)	Tolerable Impact Level 4 (Severe)	Tolerable Impact Level 3 (High)	Tolerable Impact Level 2 (Moderate)
1/50	1/500	1/50	1/100	1/100	Medium	Tolerable Impact Level 4 (Severe)	Tolerable Impact Level 3 (High)	Tolerable Impact Level 2 (Moderate)	Tolerable Impact Level 1 (Mild)
1/20	1/25	1/25	1/25	1/25	High	Tolerable Impact Level 2 (Moderate)	Tolerable Impact Level 1 (Mild)	Tolerable Impact Level 1 (Mild)	Tolerable Impact Level 1 (Mild)
"Everyday"						Tolerable Impact Level 0 (Insignificant)	Tolerable Impact Level 0 (Insignificant)	Tolerable Impact Level 0 (Insignificant)	Tolerable Impact Level 0 (Insignificant)

Notes:
1. This table shows the tolerable impacts for a range of events and performance groups. The annual probabilities of events are intended to give an indication of the scale of event being considered in setting the tolerable impacts. They have been chosen to align with values given in AS/NZS 1170.
2. The table has been derived for structural performance requirements, viz B1. The DBH intend to develop the concept to cover all relevant disciplines.

13.10 For groups of domestic dwellings (Performance Group (PG) 2) within a community which are, for example, located on expansive soils, and potentially subject to economic damage through extreme drought conditions, the following is able to be derived from the attached Table 10 *Performance Framework* which suits structural design actions.

13.11 Given that "domestic" dwellings and/or "residential" buildings (as a general "ownership class") will be the main focus of the economic analysis, the following is derived:

(a) Domestic/residential buildings – PG2

(b) Design event for evaluation with corresponding TIL

(i) Extreme drought event (TIL5/very severe) with event frequency rated "very low"
(ii) Design drought event (TIL4/severe) with event frequency rated "low"
(iii) Serviceability drought event (TIL3/mild) with event frequency rated "mild".

(c) Evaluation threshold – 90% of buildings will perform better.

14.0 NEW ZEALAND BUILDING CODE REQUIREMENT

14.1 Performance statements (B1.3.1 and B1.3.2)

Structural performance requirements for foundations in terms of the New Zealand Building Code Regulations are substantively:

> **B1.3.1** *Buildings, building elements* and *sitework* shall have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during *construction* or *alteration* and throughout their lives.
>
> **B1.3.2** *Buildings, building elements* and *sitework* shall have a low probability of causing loss of *amenity* through undue deformation, vibratory response, degradation, or other physical characteristics throughout their lives, or during *construction* or *alteration* when the *building* is in use.
>
> **B1.3.3** Account shall be taken of all physical conditions likely to affect the stability of *buildings, building elements* and *sitework,* including:
>
> (a) Self-weight,
> (b) Imposed gravity loads arising from use
> …
> (m) Differential movement
> (n) Vegetation
> …
> (q) Time dependent effects including creep and shrinkage, and
> (r) Removal of support.
>
> **B1.3.4** Due allowance shall be made for:
> …
> (c) Effects of uncertainties resulting from *construction* activities, or the sequence in which *construction* activities occur,
> (d) Variation in the properties of materials and the characteristics of the site, and
> (e) Accuracy limitations inherent in the methods used to predict the stability of *buildings*.
>
> **B1.3.7** Any *sitework* and associated supports shall take account of the effects of:
>
> (a) Changes in ground water level,
> (b) Water, weather and vegetation, and
> (c) Ground loss and slumping.

Under Clause A2 *Interpretation* the term "siteworks" (referred to in B1.3.7 etc) means:

> **Sitework** means work on a *building* site, including earthworks, preparatory to or associated with the *construction, alteration,* demolition, or removal of a *building*.

Subsequent AS 2870 performance requirements should be read in the context of the above-mentioned New Zealand Building Code requirements.

14.2 Basis of design

Given the stiffness-based design approach for the "5% probability of exceedance in 50 years" – i.e. 1000-year return period – drought event advocated by the AS 2870

Standard, and the performance framework for building design set out in Section 11.0, the impacts have been considered of using a purely strength-based analysis for the representative floor slab analysed in Appendix F, Section F5.5.2 (Figure F3).

From the structural engineering perspective, one question that could be asked is:

> Can we not proportion the foundation for a lesser "baseline" event, e.g. 300-year drought, and cover the extreme event, viz 1000-year drought, by introduction of appropriate load factors?

At face value, this ignores the fact that "foundation stiffness" is the primary design driver for foundations on expansive soils, and merely providing a foundation system with adequate flexural strength will generally not achieve a complying design i.e. a design which limits superstructure damage to levels prescribed within AS 2870 Appendix C.

However, in order to check out the impacts of the normalised T=300 return period decision, some parameter studies were undertaken to investigate the variation of various design parameters, viz:

(a) Foundation unsupported edge distance, viz [e(T)];

(b) Foundation flexure under the centre heave, viz [M(T)];

(c) Foundation deflection or stiffness, viz [Δ(T)]; and

(d) Angular distortion derived from (a) and (c), viz [α(T)=Δ(T)/e(T)];

on a stiffened foundation slab system subject to a drought-generated "centre heave" ground profile, where the drought return period "T" varies over the range 20, 50, 100, 300, 500 and 1000 years.

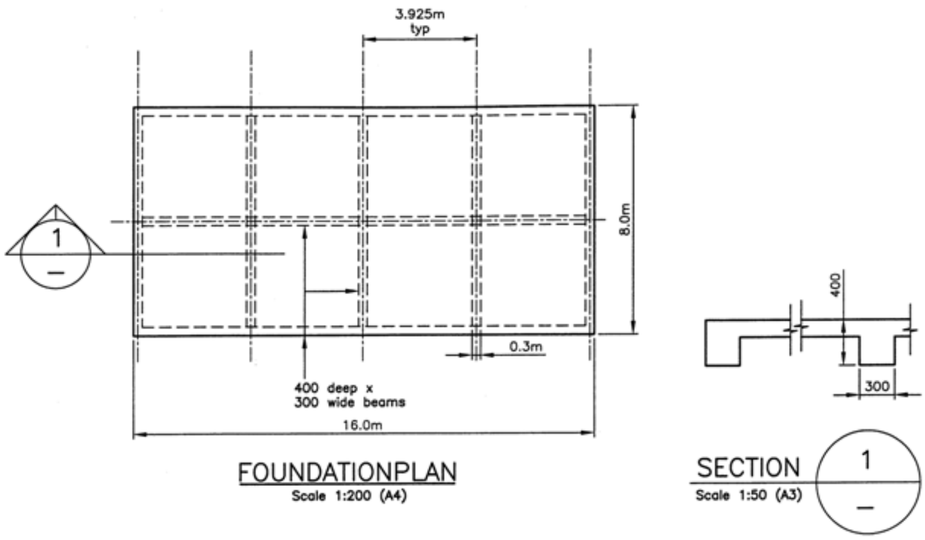


Figure 19 Foundation plan and section

A representative rectangular stiffened slab system (8 x 16 m), as shown in Figure 19, was analysed by the Mitchell Method using the soil structure interactive "SLOG" software as described in Appendix F.

The raft comprised 400 deep x 300 wide foundation beams in a nominal 4 x 4 m grid cast integrally into the slab, these being subject to a uniformly distributed load of 4 kPa and a line load of 8 kN/m to represent the weight of the external perimeter walls.

The results, normalised to the T=300-year design drought, are summarised for the "centre heave" design condition for the short and long span directions in the following table.

Long span: centre heave							
Return period T (yrs)	**Edge distance**		**Foundation flexure**		**Foundation deflection/stiffness**		**Angular distortion**
	e(m)	**Normalisation factor**	**M (kNm)**	**Normalisation factor**	**Δ (mm)**	**Normalisation factor**	**Normalisation factor = Δ/e**
20	0.65	0.74	6.80	0.52	7.5	0.48	0.66
50	0.70	0.83	9.00	0.69	10.3	0.66	0.79
100	0.76	0.89	10.60	0.82	12.3	0.79	0.88
300	0.85	1.00	13.00	1.00	15.6	1.00	1.00
500	0.89	1.05	14.10	1.08	17.1	1.09	1.04
1000	0.95	1.12	15.50	1.19	19.1	1.23	1.10

Short span: centre heave							
Return period T (yrs)	**Edge distance**		**Foundation flexure**		**Foundation deflection/stiffness**		**Angular distortion**
	e(m)	**Normalisation factor**	**M (kNm)**	**Normalisation factor**	**Δ (mm)**	**Normalisation factor**	**Normalisation factor = Δ/e**
20	0.63	0.74	9.90	0.58	9.4	0.53	0.72
50	0.70	0.83	12.50	0.73	12.6	0.71	0.85
100	0.76	0.89	14.40	0.84	14.4	0.81	0.91
300	0.85	1.00	17.20	1.00	17.8	1.00	1.00
500	0.89	1.05	18.50	1.08	19.2	1.08	1.03
1000	0.95	1.12	20.30	1.18	21.3	1.20	1.07

A plot of the "normalisation factors" for flexure, stiffness and angular distortion for the long span case under different return period droughts is reproduced in Figure 20.

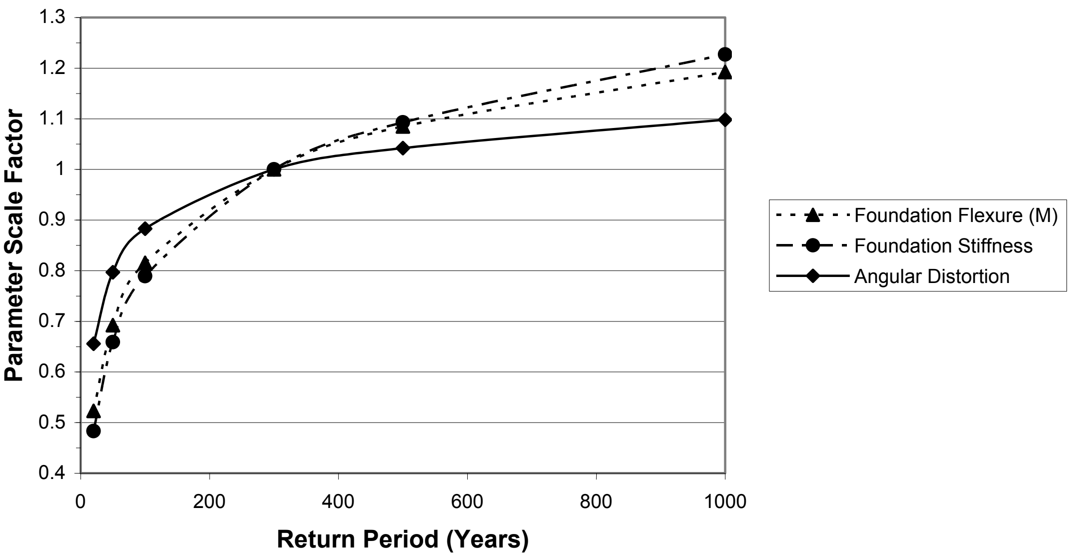


Figure 20 SLOG parameters normalised to T=300 long span, centre heave

These indicate that:

(a) Scaling the foundation ULS flexural moment calculated for the T=300-year design drought condition by a factor of 1.2 is conservative in providing reserve strength for the 1000-year extreme drought condition i.e. actually 1.19; and

(b) The corresponding angular distortion occurring in the extreme drought event will exceed that specified for the normalised T=300-year design threshold by around 10%. This is not considered critical given the limited potential for this to occur.

Similar results are obtained when the normalisation is applied to the "reverse flexure", which occurs to foundations subject to the edge heave condition.

It is therefore recommended that conforming designs for Auckland be developed using conditions for the T=300 design drought, scaled for strength only to accommodate the extreme drought event, viz 1000-year return period case.

14.3 For foundation "performance" evaluation, the design basis shown in Figure 21 is proposed for expansive soil sites.

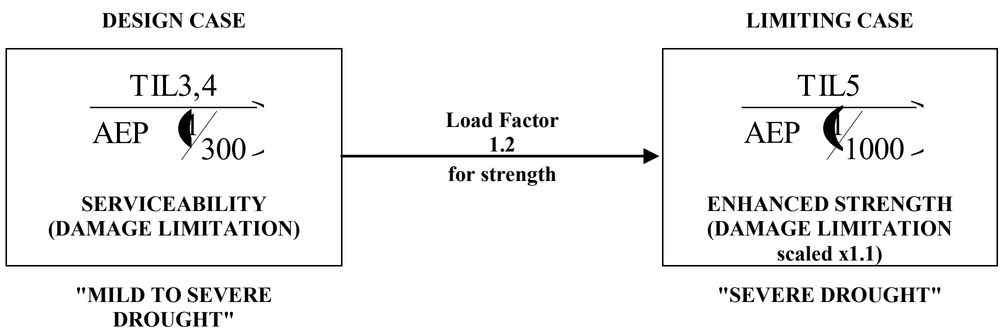


Figure 21 **Design basis for foundations on expansive soils**

This reflects, in general terms, the performance framework for wind and/or earthquake under the New Zealand Building Code referred to in Section 14.1.

15.0 FOUNDATION DESIGN – DECISION TREE AND FLOW DIAGRAM

15.1 The background data presented in this report provides a basis for a design approach that is matched to the characteristics of Auckland soils.

15.2 The design process essentially requires designers to address the following two questions:

(a) Is the site classified as having "expansive soils"? (Yes/No) as shown in Figure 22?

(b) If the answer to (a) is "No", then the foundation design may proceed following NZS 3604:1999 Sections 3 and 7 rules.

If the answer to (a) is "Yes", then the foundation design needs to make provision for the expansivity of the site for the specified design drought conditions using AS 2870 rules, with two alternative solution paths possible, as shown in Figure 23.

Option 1

A specific design is prepared to cover both "centre heave" and "edge heave" ground profile conditions for the 300-year design drought event, with a ULS load factor of 1.2 included to cover the extreme drought (1000-year return period) case.

Option 2

A specific design is prepared which incorporates, for example, perimeter piling to critical depths to cover the "edge heave" ground profile condition, plus pre-treatment of the foundations soils to minimise the residual swelling potential, thereby limiting the potential for the design to be dominated by the "centre heave" ground profile condition covering the same drought events as for Option 1.

15.3 The following flowcharts indicate the process involved:

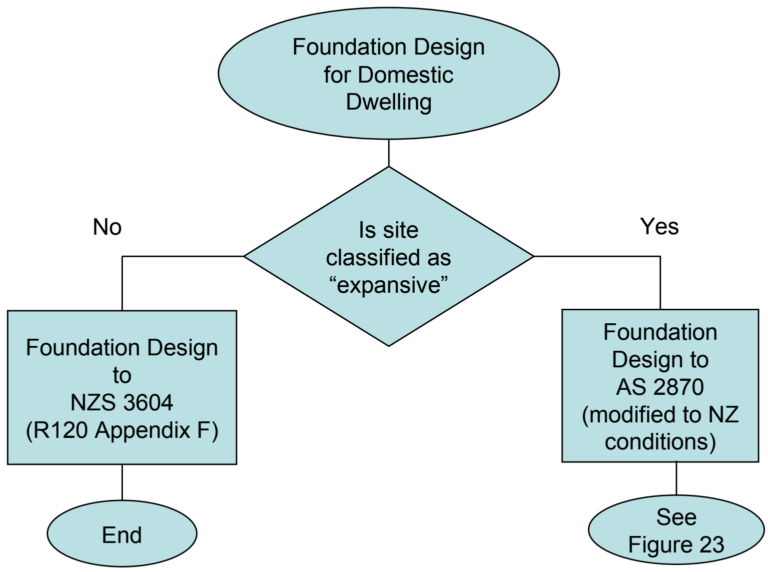


Figure 22 General foundation design

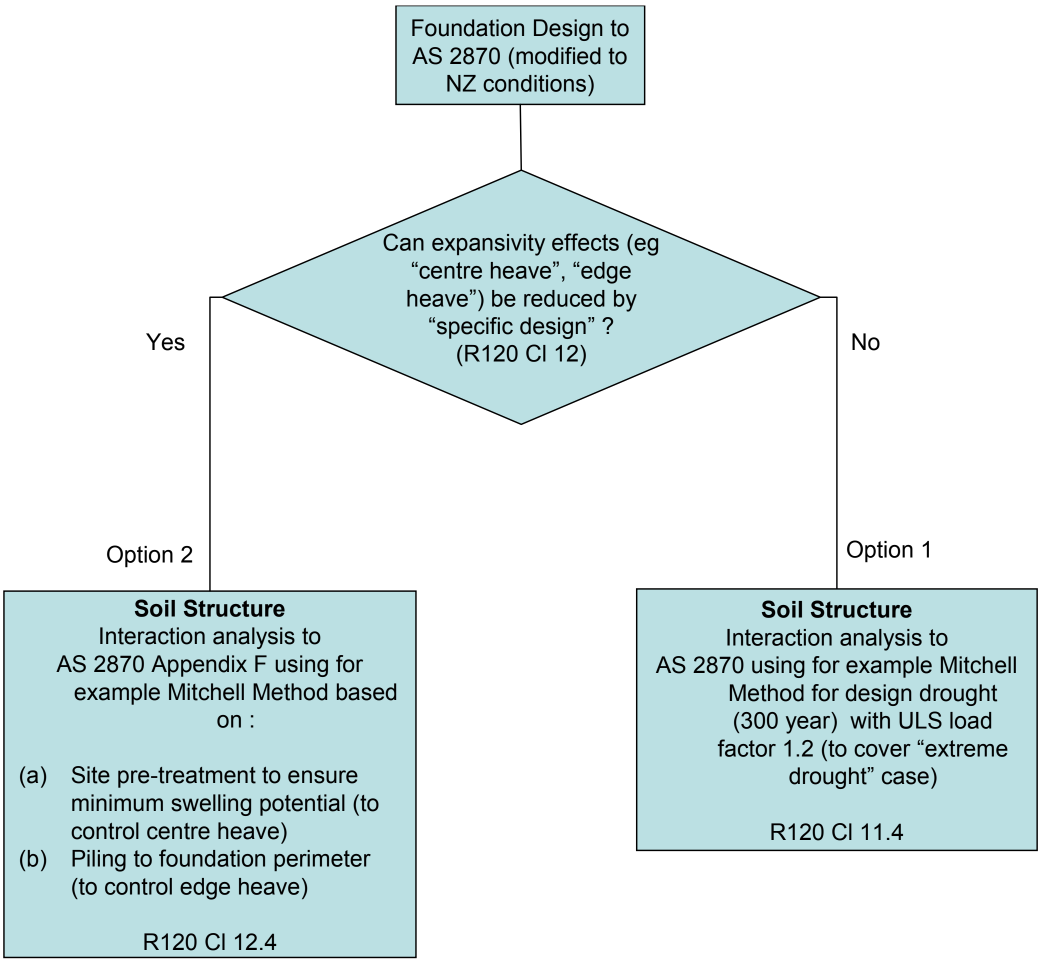


Figure 23 **Option for expansive soil solutions**

16.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on the results of the Stage I and Stage II studies and supersede those presented in the Stage I BRANZ *Study Report 120*.

16.1 Conclusions

(a) AS 2870 procedures:

(i) In the absence of established data for houses on the soil profile, AS 2870 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 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 Tables D1 and D2 of Appendix D of AS 2870 for Melbourne and Victoria and by Fityus et al (1998).

(iii) Table 2.4 of AS 2870 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 1.5 m for Auckland recommended in Section 11.3 of this report is equal to the lower end of the range of 1.5 m to 1.8 m.

(iv) The AS 2870 commentary states that y_s should be interpreted as the characteristic value that has a 5% chance of being exceeded in the life of the dwelling, which may be taken as 50 years. This corresponds to a return period of one-in-1000 years or an annual exceedance probability (AEP) of 0.1%.

It is considered that one-in-300-year return period drought conditions are applicable to the assessment of expansive soil movements in residential buildings subject to:

(a) Foundations being proportioned with sufficient strength and stiffness to restrict differential movements within the dwelling superstructure to limiting values prescribed in AS 2870 Clause 4.4 in the design drought (one-in-300-year return period) condition; and

(b) Foundations being provided with sufficient strength to resist imposed ground deformations i.e. edge heave and/or centre heave in the more severe extreme drought (one-in-1000-year return period) condition.

(v) AS 2870 relies on the loading Standard AS/NZS 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;

and assumes that a structural analysis incorporating appropriate soil structure interaction modelling, e.g. Mitchell method, will be used in proportioning the foundation for the required stiffness.

The requirements of AS/NZS 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 e.g. NZS 3101 *Code of Practice for the Design of Concrete Structures.*

(vi) For the purposes of satisfying the foundation structural performance requirements of (a)(iv) above, the following load factors may be applied to design **strengths** derived from the one-in-300-year return period design drought condition:

(a) Design drought (one-in-300-year) = 1.0;

(b) Extreme drought (one-in-1000-year) = 1.2.

(b) Classification of summer soil moisture conditions

(i) The CDD value, defined as the SMD below a threshold value of -70 mm (representative of “dry” soil conditions) accumulated on a daily basis over the period leading up to the sampling date, provides a method of quantifying the intensity of summer conditions for any particular sampling date.

(ii) The CDD approach for determination of the intensity of summer conditions provides a means for extrapolating the extensometer data to predict the ground surface movement under the design drought conditions.

(iii) Gumbel Analysis (Type 1 Extreme Value Theory) has been adopted as a means of estimating and modelling the statistical distribution of dry periods, represented by the CDD data adopted for this study.

(iv) Based on the Gumbel Analyses, the summer periods of 2005 and 2006 during the Stage II study period are found to correspond to less than a one-in-five-year return period event for Auckland Airport. The summer periods of 2005 and 2006 approximately correspond to the one-in-20-year return period event for the Albany and Pukekohe weather stations.

(c) Results of field and laboratory investigations for this study are:

(i) The Atterberg Limits for the soils from Sites 2A to 2F plot slightly above or below the “A line” and are of high plasticity, with liquid limit values ranging from 50 to 115. The linear shrinkage values of the soils range from 11% to 25%.

(ii) The shrinkage index for the soil samples from Sites 2A to 2F range from 1.2% to 4.8% with a mean value of 2.74%. 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 a weak correlation between the shrinkage index and Atterberg Limits or linear shrinkage for the soils from Sites 2A to 2F. In particular, a weak correlation is apparent between the shrinkage index and linear shrinkage or liquid limit, which indicates that the linear shrinkage and liquid limit values are only indicative of soil expansivity.

(iv) The weak 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) Because of the higher confidence in the measured y_s values obtained from the extensometers as compared with the theoretical y_s values obtained from the soil suction test data, it is considered more appropriate, on the basis of the data obtained during the Stage II investigation, to adopt the measured y_s values for back analysis to obtain the change in soil suction value at the soil surface (Δu_s) for use in the extrapolation to the design drought conditions for the Auckland region.

(vi) On the basis of the data collection and analyses carried out for the Stage II Study Report discussed herein, it is considered that there is sufficient evidence to support a recommendation that soil suction Profile Alpha derived in the Stage I report be abandoned and soil suction Profile Gamma be adopted for determining the expansive soil characteristics of soils in the Auckland region, based on a notional 300-year return period drought event i.e. that a triangular suction change profile, having an H_s of 1.5 m and a Δu_s (suction change at the ground surface) of 1.2 pF, can be adopted.

(vii) If the recommended design suction change Profile Gamma is adopted, Sites 2A to 2F generally fall into or close to the M classification of AS 2870.

(d) In adopting the above-mentioned soil suction Profile Gamma (as Section 17(c)(vi)), it is noted that the Stage II data extrapolation to an extreme drought (1000-year return period) event justifies the use of an average Δu = 1.44 pF in lieu of the $\Delta u = 1.2$ pF proposed for the corresponding y_s calculation. To address this, the following approach to determining the foundation performance for the extreme drought (1000-year return period) event prescribed by AS 2870 is proposed:

(i) By applying a scaling factor of 1.2 to the y_s value calculated from soil Profile Gamma for the purposes of soil foundation interaction analysis e.g. Mitchell Method; or

(ii) By calculating the flexural demand ($øM_u$) for foundation elements calculated from Profile Gamma (H_s = 1.5, Δu = 1.2) to incorporate a universal load factor of 1.2 rather than 1.0.

(e) The maximum soil suction change between the winter 2004 measurements and either the summer 2005 or 2006 measurements, normalised to a base of 3.2 pF, ranged from 0.4 pF to 0.85 pF. The following became apparent:

(i) A weak relationship was apparent between the soil suction change at the soil surface (Δu_s) and the CDD value.

(ii) No discernible relationship was apparent between the soil suction change at the soil surface (Δu_s) and the vertical ground movement measured by the extensometers.

(iii) The sampling and measurement of soil suction using the transistor psychrometer requires careful execution by a skilled and experienced technician, and has a high potential for the introduction of variability to the calculation of y_s values for any particular site.

(f) 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) Weak 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 be calculated, which in turn requires the depth of the 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 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) As set out in Appendix D to this report, the following can be concluded with respect to the applicability of foundation details in NZS 3604, for use on Auckland sites:

(1) Type 1 foundations are unsuitable for construction on

expansive soil sites in the Auckland region without modification.

(2) 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.

(3) NZS 3604 appears to be suitable for construction on Class M expansive soils sites in the Auckland region.

(4) 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 750 mm. For reduced beam depth additional steel reinforcement will be required, as indicated in Table 26 for Foundation Type 2B.

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

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

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

(8) Back analyses undertaken using the Type 2A foundations, from Table 28 and Figure 25, have confirmed that Type 2A foundations with an embedment depth of 450 mm supporting clad frame, articulated masonry veneer and masonry veneer buildings, perform satisfactorily in Auckland.

(g) 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, 0.5 m to 0.8 m depth and 1 m 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 Section 5.4.2, which are taken from Appendix F of AS 2870, and assuming a cracked zone depth equivalent to 0.5 H_s.

(iii) That the characteristic surface movement (y_s) be calculated using the equation given in the Section 5.2, which is taken from AS 2870, and on the basis of a triangular suction profile, having an H_s value of 1.5 m and Δu_s at the ground surface of 1.2 pF, and an I_{pt} value calculated as discussed in (e)(ii) above.

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

(h) Limitations on research

(i) As the study period did not include a severe drought period, it was not possible to measure the ground movement and soil suction profiles at Sites 2A to 2F 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 2A to 2F corresponding to the driest of the 2005 and 2006 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 correlation between the CDD values and ground surface movement (y_s), as discussed in Section 10.0.

(i) Soil suction change profile for regions other than Auckland

The soil suction change profile recommended in this report is specific to the Auckland region.

The soil suction change profile for any other region in New Zealand depends on the SMD profile for the particular region, which in turn depends on the local climate (plus any change to that local climate in the future).

16.2 Recommendations

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

(i) Building damage survey, as discussed in 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 25 of this report for a range of building sizes to be used by designers and local TAs 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.

(b) Additional work would be required at Sites 2A to 2F, over a number of years, to resolve limitations in the work carried out for this study:

(i) Continue to monitor the SMD on the NIWA 'Climate Now' website during summer periods in order to determine whether a severe drought event occurs i.e. conditions similar to, or approaching, the 1998 event. If such a drought event occurs, measure the ground movement at Sites 2A to 2F by means of the extensometers and take soil suction samples to determine the soil suction profile for each site.

(ii) If such drought conditions occur, review the correlation between the ground movement and CDD values, including the values relating to the severe drought, and verify the design Δu_s value.

(iii) If Recommendations 16.2(b)(i) and (ii) are adopted, undertake monitoring of the extensometers at Sites 2A to 2F on a regular basis (say twice per year) to confirm that the extensometers are continuing to function satisfactorily.

(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, is required to determine the instability index (I_{pt}).

(v) It is recommended, during the course of an identified future dry period, that Fraser Thomas Ltd be engaged to undertake monitoring of the extensometers to identify the lag between dry conditions and the response in the soil column.

(c) If further research is undertaken into the expansivity of soils in the Auckland region, it is recommended that consideration be given to installing and monitoring a set of extensometers adjacent to a weather recording station to help remove some of the foregoing uncertainties.

(d) Length of time to develop database

Given that AS 2870 refers to the validity of a 10-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 10 years.

(e) Climate change

It is recommended that the SMD profile for Auckland be re-evaluated at not less than 20-year intervals, using the methodology adopted in this report, in order to take account of any future climate change.

(f) Proposed form of reporting by geotechnical practitioners

The authors recommend that every residential land subdivision site on soils which are potentially expansive should be subject to a geotechnical investigation report in support of a sub-division 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 Gamma, as discussed in Section 16.1(e).

(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 local TAs with the information upon which to build a database for their region.

(g) Applicants for building consents on such sites be advised of the importance of ongoing foundation maintenance as set out in Appendix B of this report in order to ensure owner obligations become more widely known.

Appendix A

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Appendix B

Borehole Logs

HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. 2A

PROJECT. BRANZ
ALFRISTON ROAD
MANUREWA

PROJECT NO. 47141

CO-ORDINATES E N

GROUND LEVEL DATUM

Date Drilled 29.03.04 Logged by D. Dravitzki 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	SILT, brown, friable [TOPSOIL]						
	CLAY, silty, orange, moderately plastic, very stiff, moist [TAURANGA GROUP-tp]						
0.5				>176			
	becomes light grey streaked orange						
1.0				>176			
1.5				o x			
2.0				o x			
2.5				>176			
3.0				>176			
3.5				>176			
	becomes sandy (fine grained)						
4.0				o x			
	EOB @ 4.0 m TARGET DEPTH						
4.5							
5.0							
5.5							
6.0							
6.5							
7.0							

REMARKS: 1. GWL @ 4.0 m on 29.03.04

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HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. 2B

PROJECT. BRANZ
BELINDA AVENUE
OTARA

PROJECT NO. 47141

CO-ORDINATES E N

GROUND LEVEL DATUM

Date Drilled 19.04.04 Logged by D. Dravitzki Checked

DEPTH (m)	DESCRIPTION OF STRATA
0.0	SILT, brown, friable [TOPSOIL]
	SILT, clayey, orange-brown streaked grey, slightly-moderately plastic, very stiff, moist [TAURANGA GROUP - tpp]
0.5	becomes grey
1.0	SILT, sandy (fine grained), blue-grey, slightly plastic, very stiff, moist-wet
	CLAY, silty, blue-grey, slightly plastic, very stiff, moist
1.5	becomes SILT/SAND (fine grained)
2.0	CLAY, silty, green-grey streaked orange, very plastic, stiff
	becomes CLAY, blue-grey, very plastic, soft-firm, wet
2.5	
3.0	CLAY, sandy (fine grained), dark grey, moderately plastic, very stiff
	SILT, green-grey, slightly plastic, very stiff
3.5	CLAY, silty, brown, moderately plastic, very stiff
4.0	EOB @ 4.0 m TARGET DEPTH
4.5	
5.0	
5.5	
6.0	
6.5	
7.0	

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

REMARKS: 1. GWL @ 2.0 m on 19.04.04

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HAND AUGER LOG	SHEET 1 OF 1	BOREHOLE NO. 2C

PROJECT. BRANZ HOWICK COMMUNITY CENTRE HOWICK	CO-ORDINATES	E	N
	GROUND LEVEL	DATUM	
PROJECT NO. 47141	Date Drilled 22.07.04	Logged by D. Dravitzki	Checked

DEPTH (m)	DESCRIPTION OF STRATA	UNDRAINED SHEAR STRENGTH (kPa) Vane readings corrected as per BS 1377; X Shear Vane; ○ Residual Shear Vane	WATER CONTENT (%)
0.0	SILT, brown, friable [TOPSOIL]		
0.5	CLAY, silty, cream-grey mottled orange, slightly-moderately plastic, very stiff, moist [WAITEMATA GROUP]	o x	
1.0		>240	
1.5	SILT, clayey, sandy (fine grained), light grey mottled orange streaked pink, slightly plastic, very stiff-hard, moist	o x	
2.0		>240	
2.5	becomes very sandy, pink streaked dark orange and light grey	>240	
3.0		>240	
3.5		>240	
4.0	numerous fine gravels (siltstone fragments)	>240	
	EOB @ 4.0 m TARGET DEPTH		

Shear strength scale: 50, 100, 150, 200. Water content scale: 20, 40, 60, 80 (W_p, W_f, W_l).

REMARKS: 1. Borehole dry on 22.07.04

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HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. 2D

PROJECT.	BRANZ WINDSOR PARK MAIRANGI BAY
PROJECT NO.	47141
CO-ORDINATES	E N
GROUND LEVEL	DATUM
Date Drilled	26.07.04
Logged by	D. Dravitzki
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	SILT, brown, friable [TOPSOIL]	~					
0.5	CLAY, silty, orange-brown, slightly plastic, very stiff, moist [WAITEMATA GROUP]			o x			
	becomes light grey streaked orange and pink						
1.0				>225			
1.5				>225			
2.0				>225			
2.5				>225			
	becomes sandy (fine grained)						
3.0	CLAY, silty, light grey streaked orange, moderately plastic, very stiff, moist			o x			
3.5				o x			
4.0				o x			
	EOB @ 4.0 m TARGET DEPTH						
4.5							
5.0							
5.5							
6.0							
6.5							
7.0							

REMARKS: 1. Borehole dry on 26.07.04

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HAND AUGER LOG

SHEET 1 OF 1

BOREHOLE NO. 2E

PROJECT. BRANZ
HIBISCUS VILLAGE
WHANGAPARAOA

PROJECT NO. 47141

CO-ORDINATES E N

GROUND LEVEL DATUM

Date Drilled 27.04.04 Logged by D. Dravitzki Checked

DEPTH (m)	DESCRIPTION OF STRATA	UNDRAINED SHEAR STRENGTH (kPa) (Vane readings corrected as per BS 1377; X Shear Vane, O Residual Shear Vane)
0.0	SILT, brown, friable [TOPSOIL]	
	[FILL] SILT, clayey, occasional fine gravels, orange mottled cream, slightly plastic, very stiff	
0.5	CLAY, silty, orange mottled cream, moderately plastic, very stiff, moist [ONERAHI CHAOS BRECCIA]	o x
1.0		o x
1.5		o x
	becomes cream streaked orange, stiff	
2.0		o x
2.5		o x
3.0	SILT, slightly clayey, slightly sandy (fine grained), grey streaked light green, slightly plastic, very stiff, moist	o x
3.5		>176
	becomes dark grey mottled brown and green	
4.0	EOB @ 4.0 m TARGET DEPTH	>176

Columns also printed: GRAPHIC LOG; SAMPLE TYPE; WATER CONTENT (%) (W_p, W_f, W_l; 20, 40, 60, 80); WATER CONTENT (%); TESTING AND COMMENTS.

Depth scale: 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0

REMARKS: 1. Borehole dry on 27.04.04

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HAND AUGER LOG	SHEET 1 OF 1	BOREHOLE NO. 2F

PROJECT. BRANZ PRINCES STREET RESERVE PUKEKOHE	CO-ORDINATES E N	
	GROUND LEVEL	DATUM
PROJECT NO. 47141	Date Drilled 17.08.04	Logged by D. Dravitzki Checked

UNDRAINED SHEAR STRENGTH (kPa) — Vane readings corrected as per BS 1377; X Shear Vane; ○ Residual Shear Vane; scale 50, 100, 150, 200

WATER CONTENT (%) — W_p, W_f, W_l; scale 20, 40, 60, 80

DEPTH (m)	DESCRIPTION OF STRATA	Residual Shear Vane (○)	Shear Vane (X)
0.0	SILT, brown, friable [TOPSOIL]		
0.2–1.5	SILT, very clayey, occasional fine gravels, orange-brown streaked grey, slightly plastic, very stiff, moist [LITHIC TUFF / BASALTIC ASH]		
0.5		o	x
0.7	becomes creamish orange		
1.0		o	x
1.3	becomes dark grey		
1.5	CLAY, very silty, light grey streaked orange-brown, slightly-moderately plastic, very stiff [ASH / ALLUVIAL]	o	x
2.0		o	x
2.4	SILT, gravelly (fine grained, scoria), orange-brown streaked light grey, non plastic, very stiff-hard [LITHIC TUFF]		
2.5			UTP
3.0			UTP
3.2			UTP
3.2	EOB @ 3.2 m TOO HARD TO AUGER		

REMARKS: 1. GWL @ 0.8 m on 17.08.04

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Appendix C

Site Summary Sheets

SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference 2A
Location Manurewa
Main Soil Type Tauranga Group - tp

1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

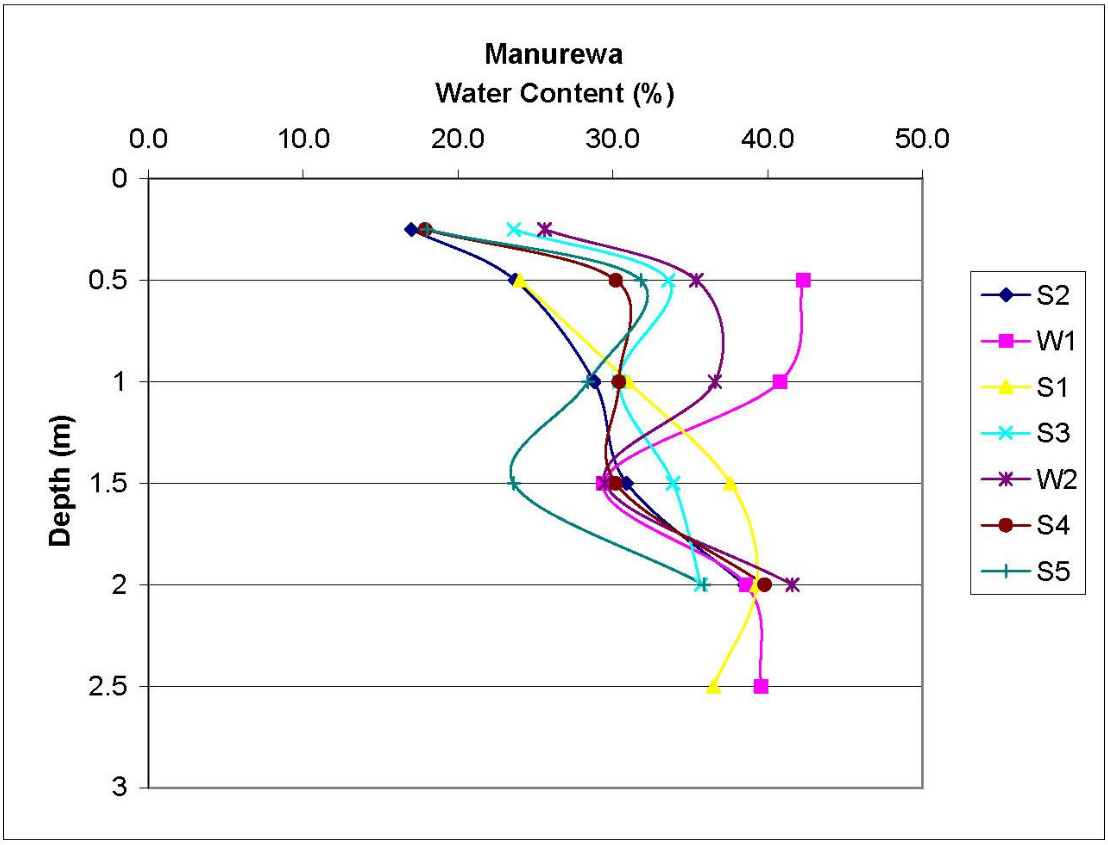
	Winter 04		Summer 04/05						Winter 05		Summer 05/06			
Date of Sampling	13.09.04		28.01.05		22.03.05		28.04.05		18.10.05		24.02.06		25.03.06	
Deficit Reading at Sampling (mm)	-5.3		-76.3		-82.3		-72.2		-11.6		-85.4		-77.4	
Date of Testing	19.10.04		20.04.05		26.04.05		05.05.05		04.11.05		30.05.06		25.05.06	
Depth to Groundwater Level (m)	Not encountered		Not encountered		Not encountered		Not encountered		Not encountered		Not encountered		Not encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.2/0.3m below GL	-	-	-	-	17.0	3.6	23.6	3.2	25.6	3.2	17.9	3.6	18.0	3.70
Sample at 0.5m below GL	42.3	3.5	24.0	3.6	23.7	3.5	33.6	3.5	35.4	3.4	30.2	3.70	31.8	3.5
Sample at 1.0m below GL	40.8	3.4	31.0	3.6	28.8	3.6	30.4	3.4	36.6	3.4	30.4	3.6	28.4	3.70
Sample at 1.5m below GL	29.4	3.4	37.6	3.4	30.9	3.4	33.9	3.4	29.5	3.2	30.2	3.6	23.6	3.5
Sample at 2.0m below GL	38.6	3.3	39.3	3.6	38.5	3.4	35.7	3.4	41.6	3.5	39.8	3.5	35.9	3.5
Sample at 2.5m below GL	39.6	3.5	36.5	3.6	-	-	-	-	-	-	-	-	-	-

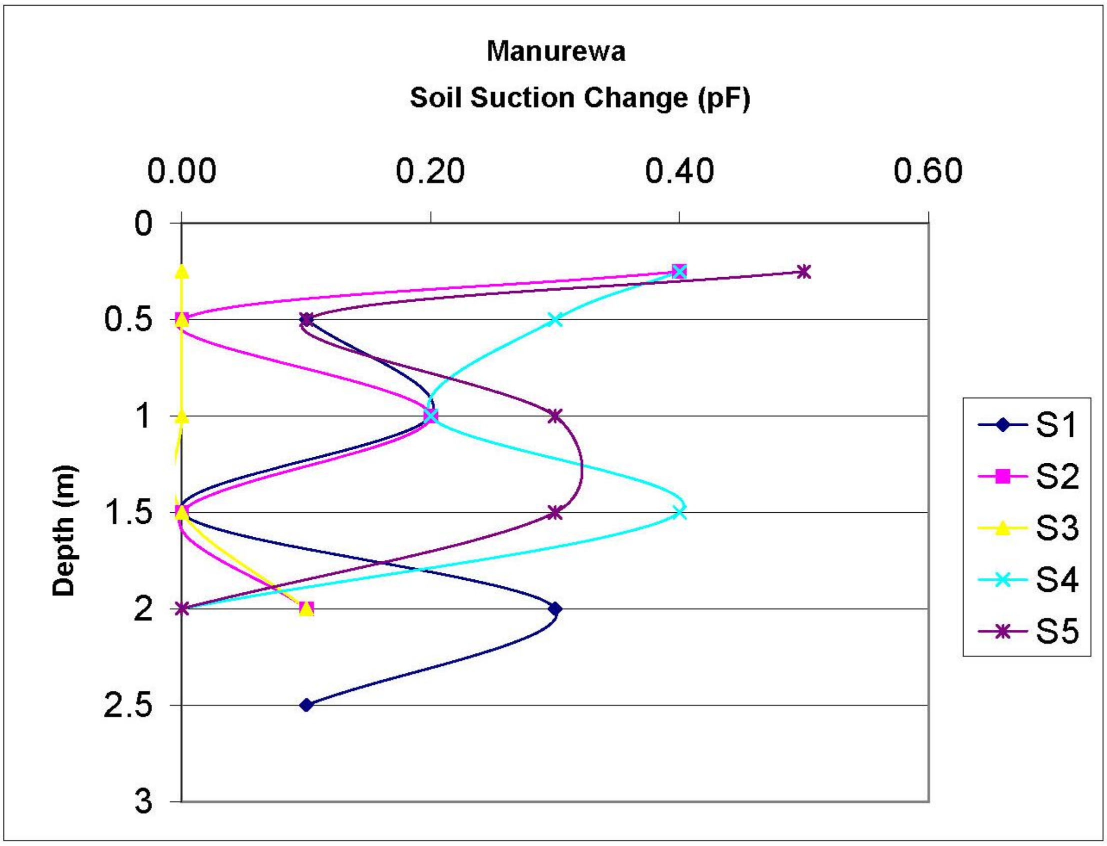
2.0 SOIL CLASSIFICATION TESTS - WINTER 2005 SAMPLES

	Atterburg Limits			Linear Shrinkage (%)
	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	
Sample at 0.5m below GL	53	20	33	15
Sample at 1.0m below GL	95	33	62	21
Sample at 1.5m below GL	95	35	60	25

3.0 SOIL INDEX TESTS

	Swell-Shrink Index (%)		Core Shrinkage Index (%)	
	Oct-05	Mar-06	Mar-05	Mar-06
Sample at 0.5m below GL	2.4	2.9	1.30	6.08
Sample at 1.0m below GL	4.2	2.4	4.39	6.10
Sample at 1.5m below GL	-	-	-	5.96

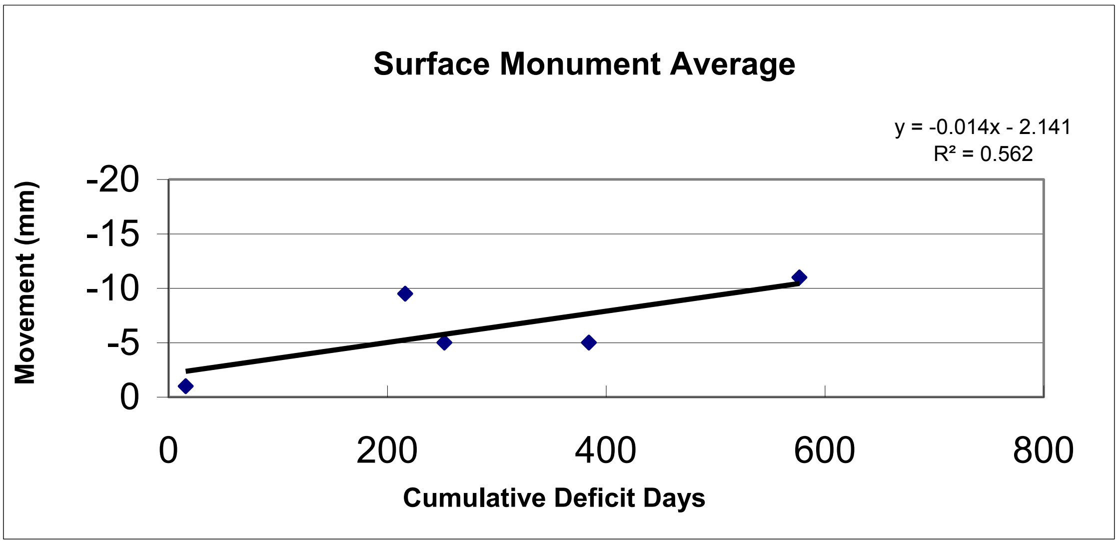

Manurewa
Water Content (%)
0.0
10.0
20.0
30.0
40.0
50.0
0
0.5
1
1.5
2
2.5
3
Depth (m)
S2
W1
S1
S3
W2
S4
S5

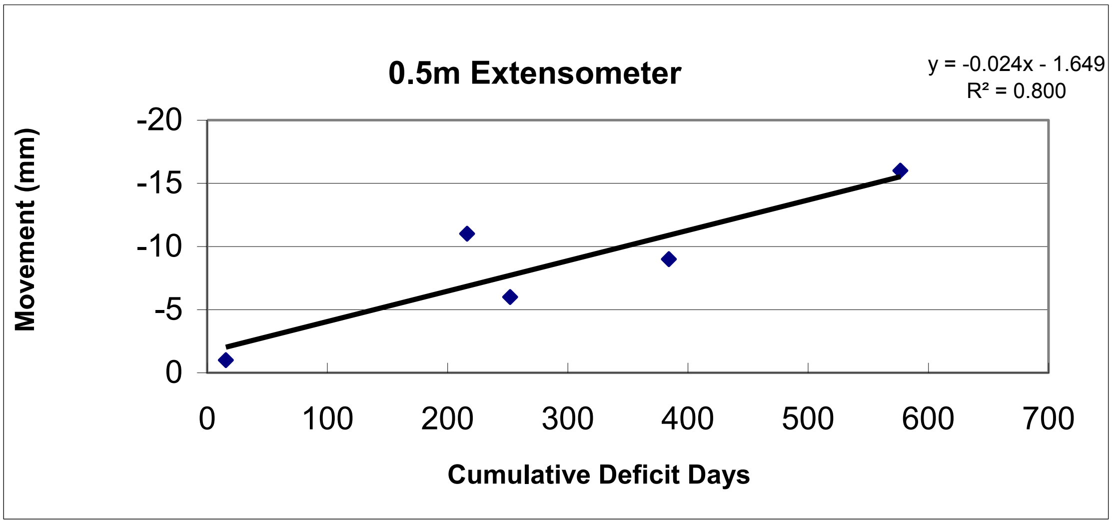

Manurewa
Soil Suction Change (pF)
0.00
0.20
0.40
0.60
0
0.5
1
1.5
2
2.5
3
Depth (m)
S1
S2
S3
S4
S5

4.0 EXTENSOMETER READINGS - SITE 2A: MANUREWA

	Summer 04/05			Summer 05/06	
Date of Sampling	28.01.05	17.03.05	28.04.05	24.02.06	25.03.06
Deficit Value at Reading (mm)	-76.3	-81.3	-72.2	-85.4	-77.4
Cumulative Deficit Days	15.6	216.3	384.2	252.1	576.7
Surface Monument at 0.0 mbgl	-2	-7	-3	-3	-6
Surface Monument at 0.2 mbgl	0	-12	-7	-7	-16
Rod at 0.5 mbgl	-1	-11	-9	-6	-16
Rod at 1.0 mbgl	-1	-6	-6	-3	-11
Rod at 1.5 mbgl	0	-1	-2	0	-5
Rod at 2.0 mbgl	0	0	-1	0	-1
Rod at 2.5 mbgl	0	0	-1	1	0

NOTES: 1. Extensometer readings in mm compared to Winter 05 readings
2. NR = no reading due to damage of surface monument
3. Cumulative Deficit Days = sum of values below -70 mm threshold





SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference 2B
Location Otara
Main Soil Type Tauranga Group - tpp

1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

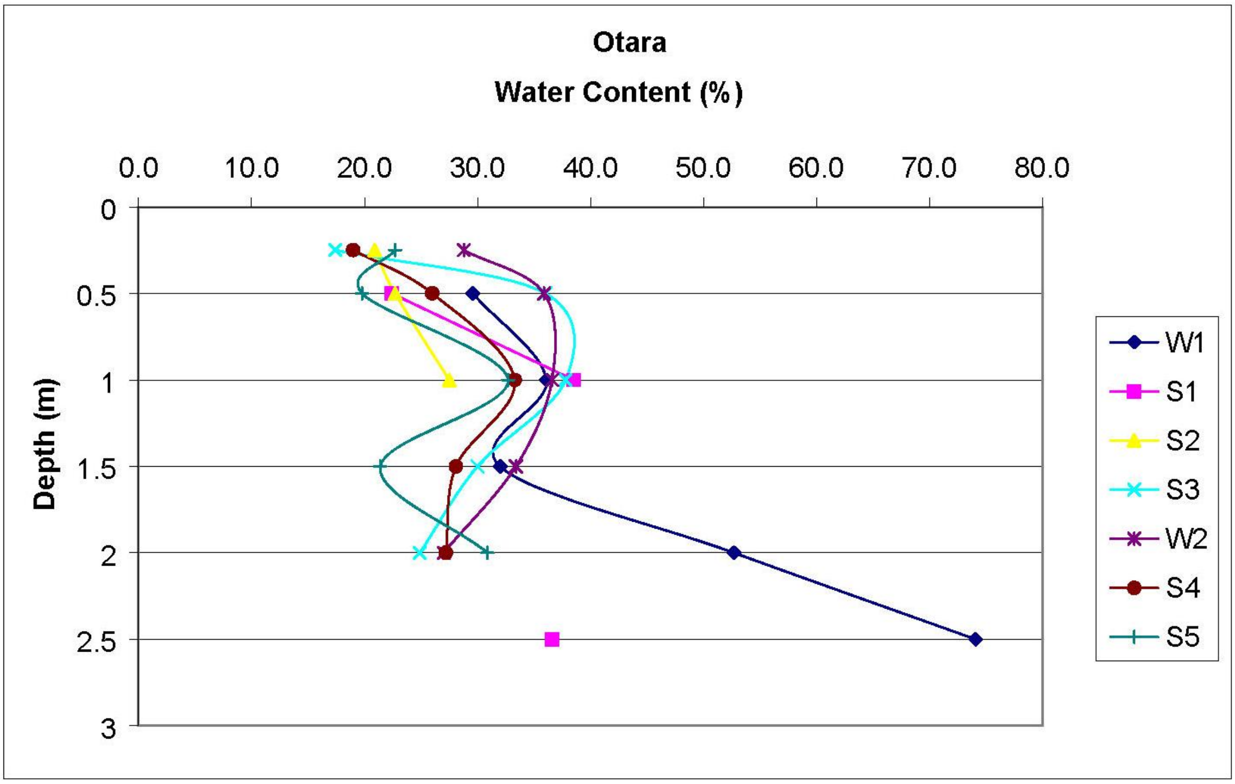
	Winter 04		Summer 04/05						Winter 05		Summer 05/06			
Date of Sampling	13.09.04		28.01.05		22.03.05		28.04.05		18.10.05		16.02.06		25.03.06	
Deficit Reading at Sampling (mm)	-5.3		-76.3		-85.3		-72.2		-11.6		-85.4		-77.4	
Date of Testing	04.10.04		21.04.05		27.04.06		06.05.05		18.11.05		30.05.06		25.05.06	
Depth to Groundwater Level (m)	2.0		Not encountered		Not encountered		Not encountered		1.5		1.9		Not encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.2/0.3m below GL	-	-	-	-	20.9	4.00	17.4	3.80	28.8	3.2	19.0	3.6	22.7	3.95
Sample at 0.5m below GL	29.6	3.4	22.4	3.6	22.7	3.70	36.0	3.6	35.9	3.2	26.0	3.75	19.8	3.90
Sample at 1.0m below GL	36.1	3.2	38.5	3.5	27.5	3.5	37.8	3.6	36.6	3.5	33.3	3.5	32.7	3.6
Sample at 1.5m below GL	32.0	3.2		-		-	30.0	3.2	33.4	3.3	28.1	3.2	21.4	3.3
Sample at 2.0m below GL	52.7	3.3	36.6	3.5	40.6	3.2	24.9	3.2	27.0	3.2	27.2	3.2	30.9	3.2
Sample at 2.5m below GL	74.1	3.2	30.3	3.4	-	-	-	-	-	-	-	-	-	-

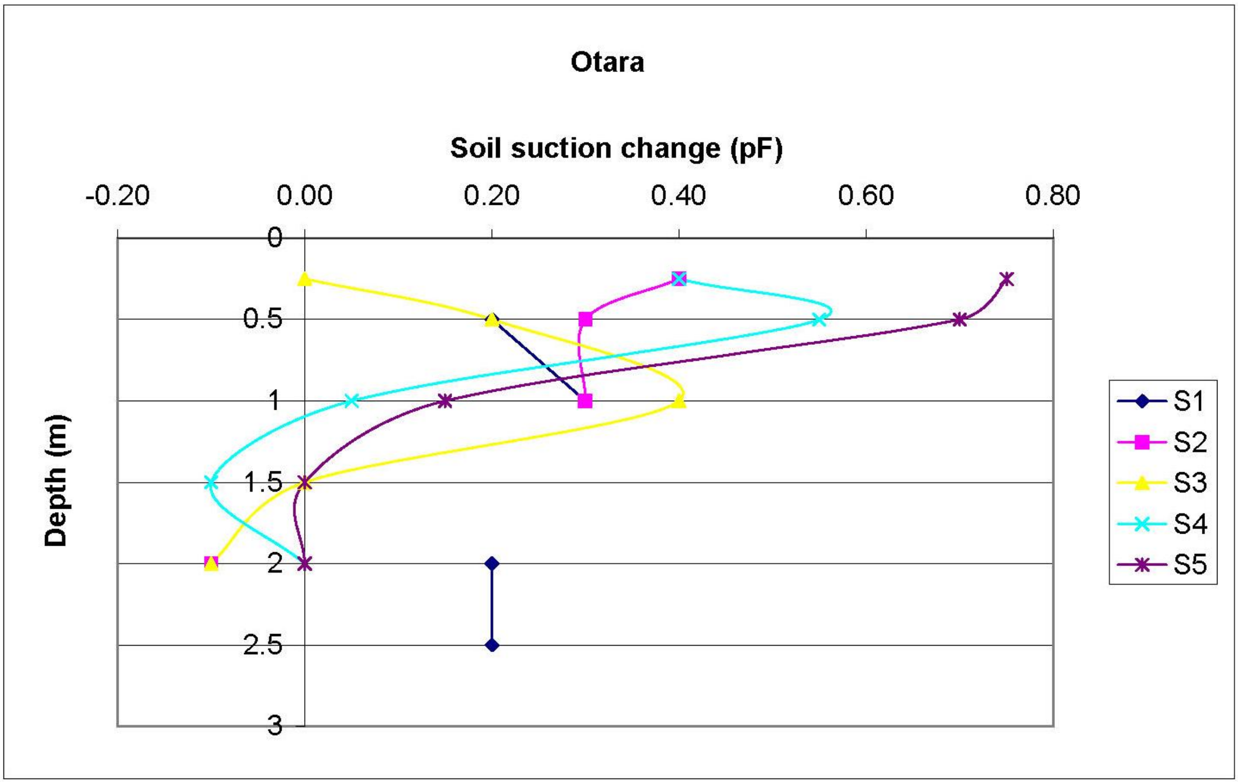
2.0 SOIL CLASSIFICATION TESTS - WINTER 2005 SAMPLES

	Atterburg Limits			
	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
Sample at 0.5m below GL	52	24	28	11
Sample at 1.0m below GL	50	28	22	12
Sample at 1.5m below GL	74	30	44	16

3.0 SOIL INDEX TESTS

	Swell-Shrink Index (%)		Core Shrinkage Index (%)	
	Oct-05	Mar-06	Mar-05	Mar-06
Sample at 0.5m below GL	-	1.5	2.22	4.83
Sample at 1.0m below GL	3.3	3.9	3.74	5.87
Sample at 1.5m below GL	-	-	-	0.88

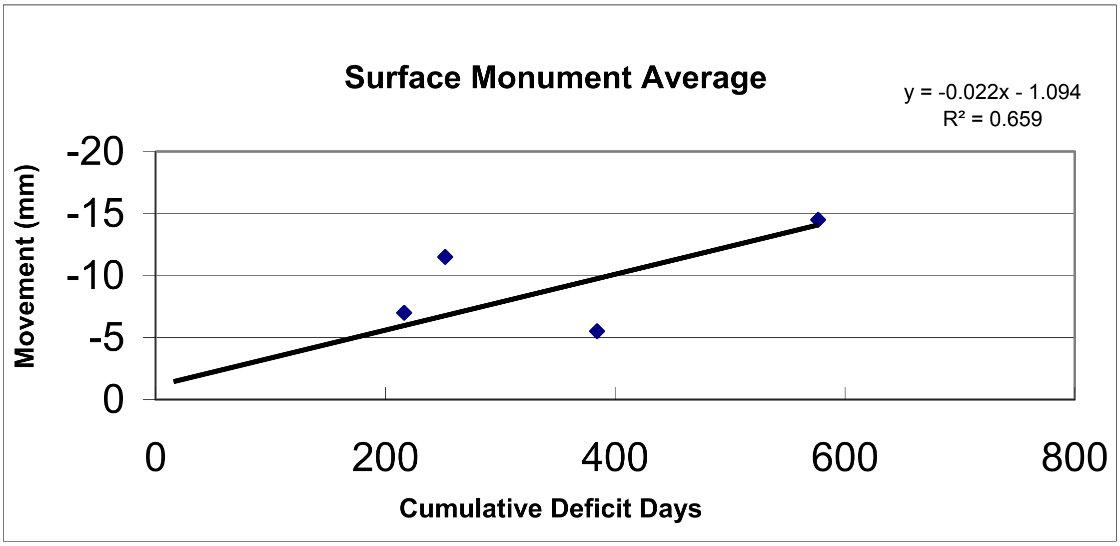

Otara
Water Content (%)
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
Depth (m)
0
0.5
1
1.5
2
2.5
3
W1
S1
S2
S3
W2
S4
S5

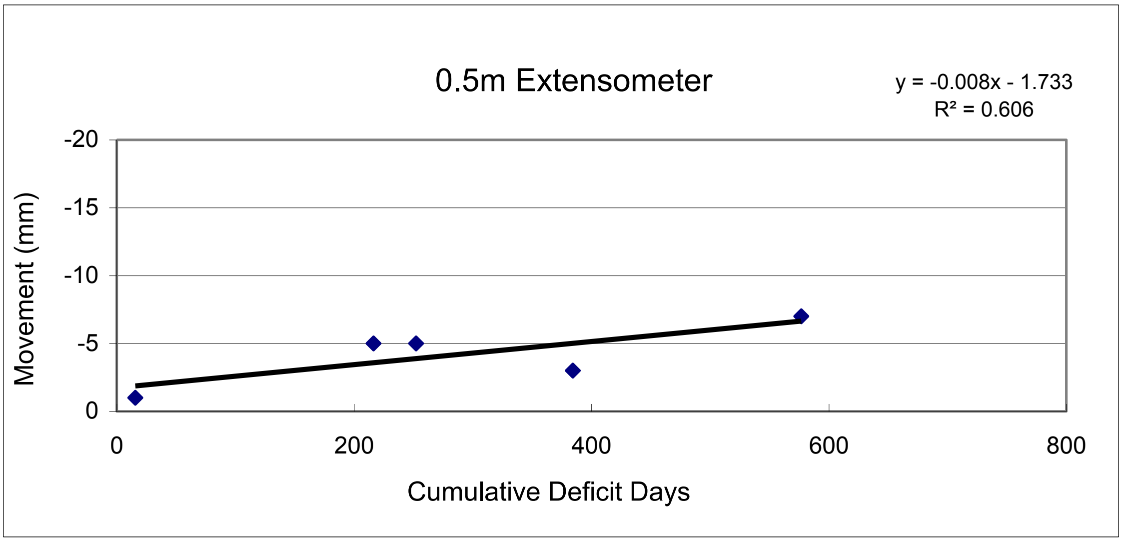

Otara
Soil suction change (pF)
-0.20
0.00
0.20
0.40
0.60
0.80
Depth (m)
0
0.5
1
1.5
2
2.5
3
S1
S2
S3
S4
S5

4.0 EXTENSOMETER READINGS - SITE 2B: OTARA

	Summer 04/05			Summer 05/06	
Date of Sampling	28.01.05	17.03.05	28.04.05	24.02.06	25.03.06
Deficit Value at Reading (mm)	-76.3	-81.3	-72.2	-85.4	-77.4
Cumulative Deficit Days	15.6	216.3	384.2	252.1	576.7
Surface Monument at 0.0 mbgl	4	-6	-5	-13	-17
Surface Monument at 0.2 mbgl	-3	-8	-6	-10	-12
Rod at 0.5 mbgl	-1	-5	-3	-5	-7
Rod at 1.0 mbgl	-2	-2	-1	-2	-2
Rod at 1.5 mbgl	-1	-1	0	-1	-1
Rod at 2.0 mbgl	-1	-1	0	-1	-1
Rod at 2.5 mbgl	-1	-2	0	-1	-1

NOTES: 1. Extensometer readings in mm compared to Winter 05 readings
2. NR = no reading due to damage of surface monument
3. Cumulative Deficit Days = sum of values below -70 mm threshold





SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference 2C
Location Howick
Main Soil Type Waitemata Group

1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

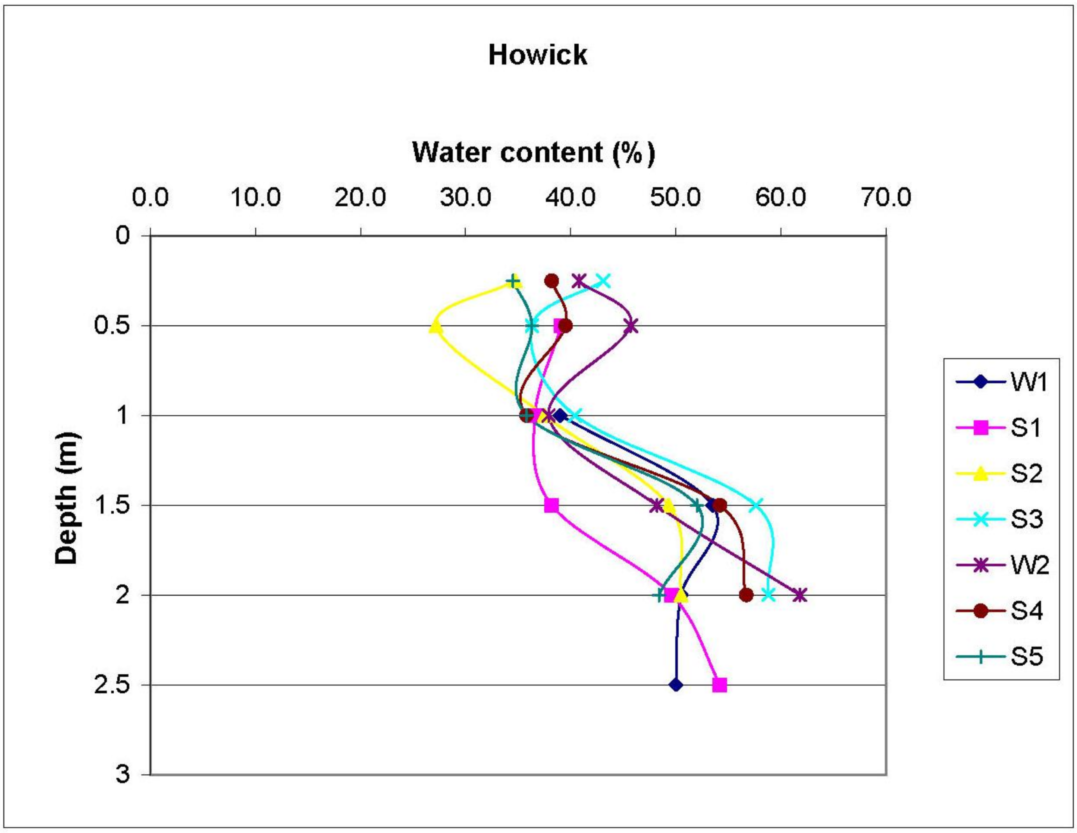
	Winter 04		Summer 04/05						Winter 05		Summer 05/06			
Date of Sampling	14.09.04		01.02.05		23.03.05		29.04.05		19.10.05		24.02.06		25.03.06	
Deficit Reading at Sampling (mm)	-1.9		-80.1		-85.6		-73.0		-14.1		-85.4		-77.4	
Date of Testing	05.10.04		21.04.05		27.04.05		12.05.06		10.11.05		30.05.06		24.05.06	
Depth to Groundwater Level (m)	Not encountered		Not encountered		Not encountered		Not encountered		Not encountered		Not encountered		Not encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.2/0.3m below GL	-	-	-	-	34.7	3.80	43.1	3.4	40.8	3.6	38.2	3.75	34.5	3.80
Sample at 0.5m below GL	-	-	39.1	3.6	27.2	3.5	36.3	3.4	45.7	3.4	39.5	3.5	36.3	3.6
Sample at 1.0m below GL	39.0	3.3	36.6	3.4	37.5	3.3	40.4	3.2	37.9	3.3	35.8	3.5	35.8	3.3
Sample at 1.5m below GL	53.5	3.4	38.2	3.4	49.3	3.2	57.6	3.2	48.2	3.3	54.2	3.3	52.0	3.4
Sample at 2.0m below GL	50.5	3.2	49.6	3.2	50.5	3.2	58.8	3.2	61.8	3.2	56.7	3.2	48.4	3.2
Sample at 2.5m below GL	50.0	3.2	54.2	3.2	-	-	-	-	-	-	-	-	-	-

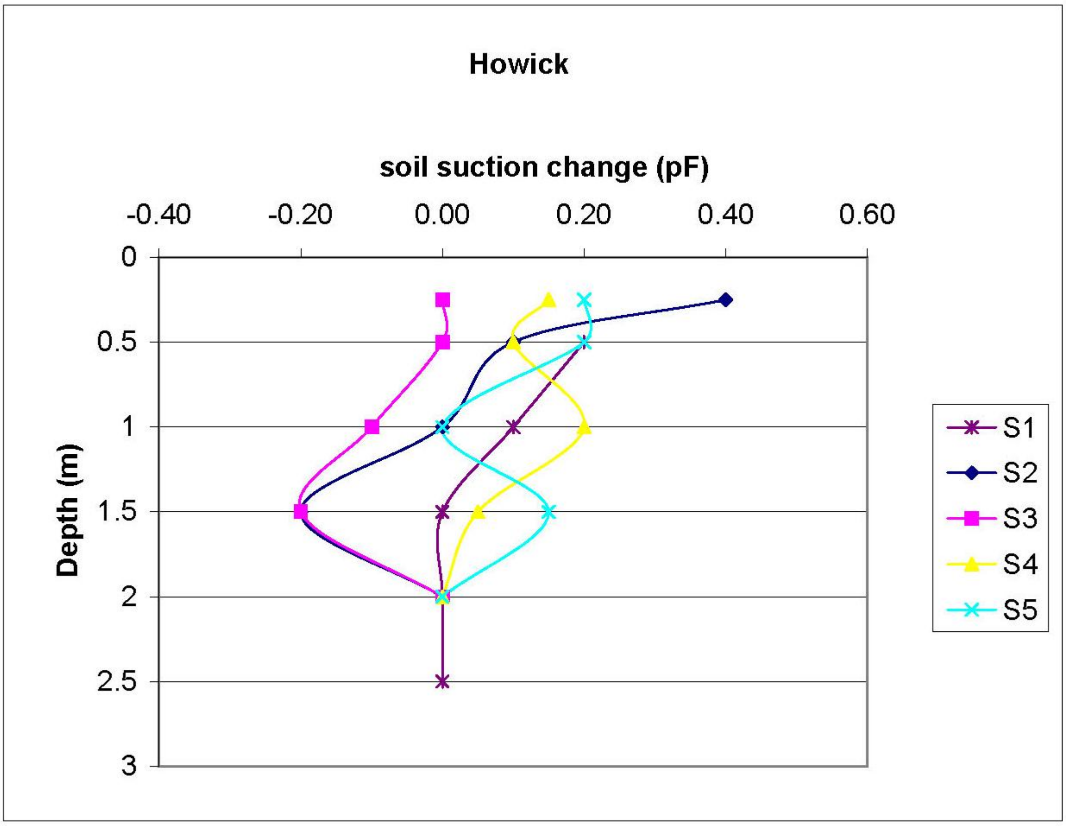
2.0 SOIL CLASSIFICATION TESTS - WINTER 2005 SAMPLES

	Atterburg Limits			
	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
Sample at 0.5m below GL	102	38	63	23
Sample at 1.0m below GL	93	34	59	23
Sample at 1.5m below GL	72	30	42	19

3.0 SOIL INDEX TESTS

	Swell-Shrink Index (%)		Core Shrinkage Index (%)	
	Oct-05	Mar-06	Mar-05	Mar-06
Sample at 0.5m below GL	2.9	2.2	2.71	4.99
Sample at 1.0m below GL	2.6	3.0	2.14	4.86
Sample at 1.5m below GL	-	-	-	3.86

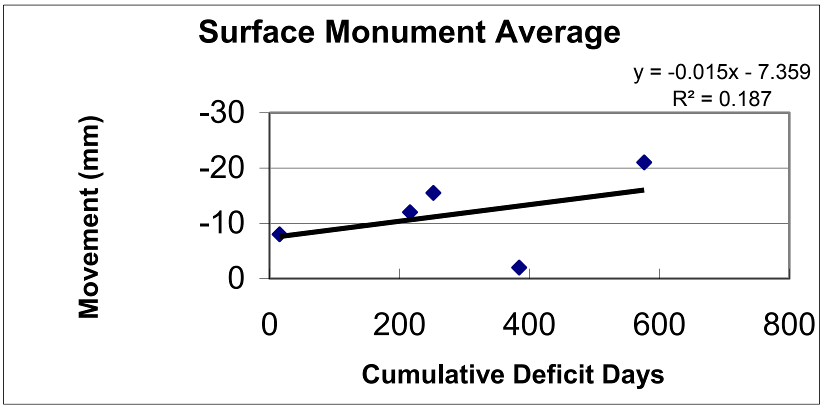

Howick
Water content (%)
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
Depth (m)
0
0.5
1
1.5
2
2.5
3
W1
S1
S2
S3
W2
S4
S5

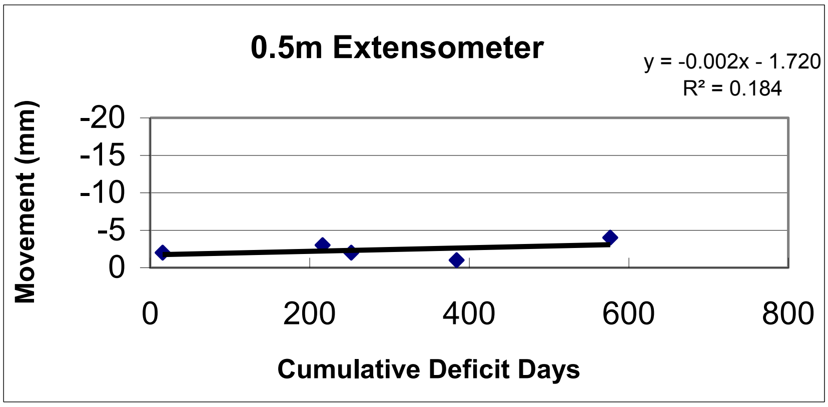

Howick
soil suction change (pF)
-0.40
-0.20
0.00
0.20
0.40
0.60
Depth (m)
0
0.5
1
1.5
2
2.5
3
S1
S2
S3
S4
S5

4.0 EXTENSOMETER READINGS - SITE 2C: HOWICK

	Summer 04/05			Summer 05/06	
Date of Sampling	01.02.05	17.03.05	28.04.05	24.02.06	25.03.06
Deficit Value at Reading (mm)	-76.3	-81.3	-72.2	-85.4	-77.4
Cumulative Deficit Days	15.6	216.3	384.2	252.1	576.7
Surface Monument at 0.0 mbgl	-8	-12	-2	-21	-28
Surface Monument at 0.2 mbgl	NR	NR	-2	-10	-14
Rod at 0.5 mbgl	-2	-3	-1	-2	-4
Rod at 1.0 mbgl	-1	0	-1	-2	-1
Rod at 1.5 mbgl	0	1	-1	0	0
Rod at 2.0 mbgl	0	1	0	0	0
Rod at 2.5 mbgl	0	1	1	0	0

NOTES: 1. Extensometer readings in mm compared to Winter 05 readings
2. NR = no reading due to damage of surface monument
3. Cumulative Deficit Days = sum of values below -70 mm threshold





SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference 2D
Location Mairangi Bay
Main Soil Type Waitemata Group

1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

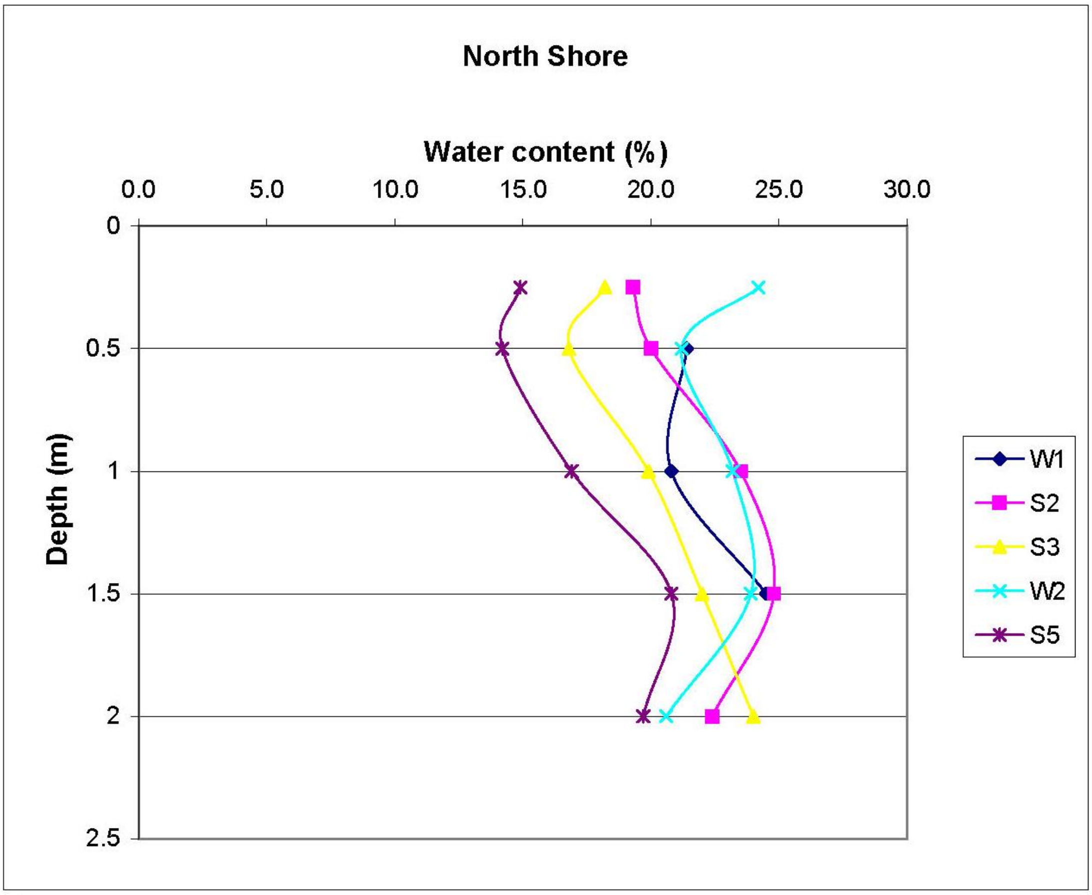
	Winter 04		Summer 04/05						Winter 05		Summer 05/06			
Date of Sampling	14.09.04		01.02.05		23.03.05		29.04.05		19.10.05		27.02.06		24.03.06	
Deficit Reading at Sampling (mm)	-1.9		-80.1		-85.6		-73.0		-14.1		-87.7		-78.0	
Date of Testing	06.10.04		-		29.04.05		12.05.05		10.11.05		-		26.05.06	
Depth to Groundwater Level (m)	Not encountered		-		Not encountered		Not encountered		Not encountered		-		Not encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.2/0.3m below GL	-	-	-	-	19.3	3.80	18.2	3.4	24.2	3.2	-	-	14.9	4.00
Sample at 0.5m below GL	21.4	3.3	-	-	20.0	3.6	16.8	3.3	21.2	3.2	-	-	14.2	4.05
Sample at 1.0m below GL	20.8	3.4	-	-	23.5	3.4	19.9	3.4	23.2	3.2	-	-	16.9	3.90
Sample at 1.5m below GL	24.5	3.5	-	-	24.8	3.3	22.0	3.4	23.9	3.3	-	-	20.8	3.6
Sample at 2.0m below GL	-	-	-	-	22.4	3.3	24.0	3.2	20.6	3.5	-	-	19.7	3.6
Sample at 2.5m below GL	-	-	-	-	-	-	-	-	-	-	-	-	-	-

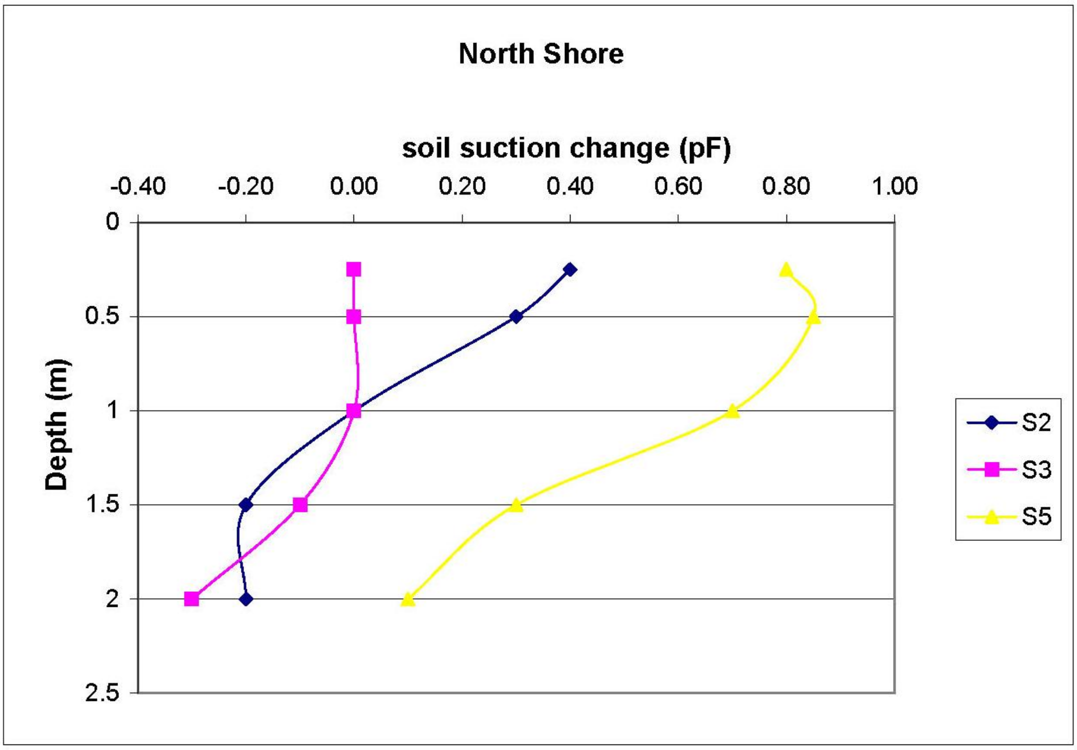
2.0 SOIL CLASSIFICATION TESTS - WINTER 2005 SAMPLES

	Atterburg Limits			
	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
Sample at 0.5m below GL	49	19	30	14
Sample at 1.0m below GL	69	20	49	18
Sample at 1.5m below GL	57	23	34	14

3.0 SOIL INDEX TESTS

	Swell-Shrink Index (%)		Core Shrinkage Index (%)	
	Oct-05	Mar-06	Mar-05	Mar-06
Sample at 0.5m below GL	1.1	0.5	1.92	0.12
Sample at 1.0m below GL	-	0.9	2.40	3.90
Sample at 1.5m below GL	-	-	-	4.20

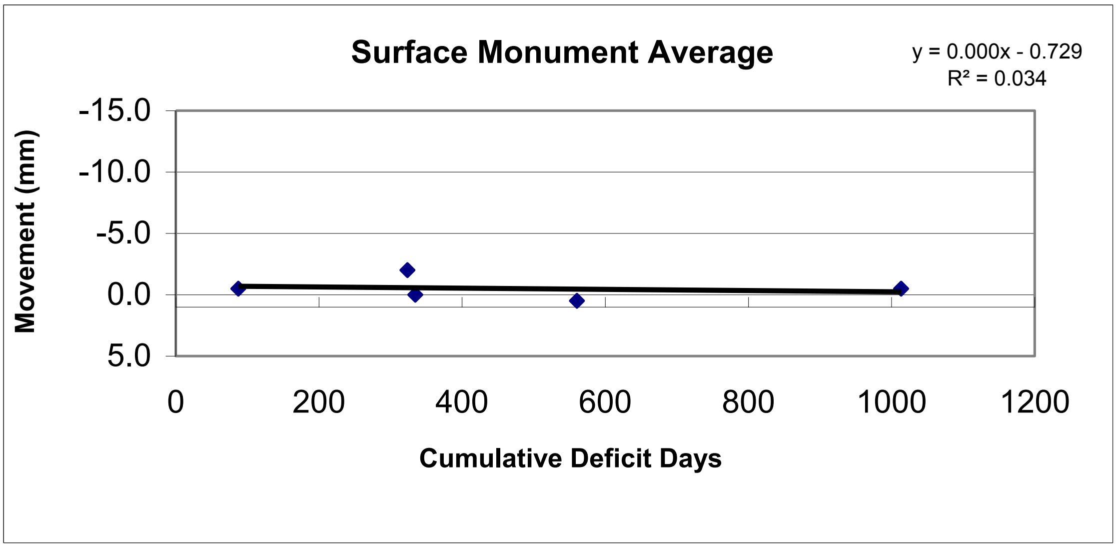

North Shore
Water content (%)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Depth (m)
0
0.5
1
1.5
2
2.5
W1
S2
S3
W2
S5

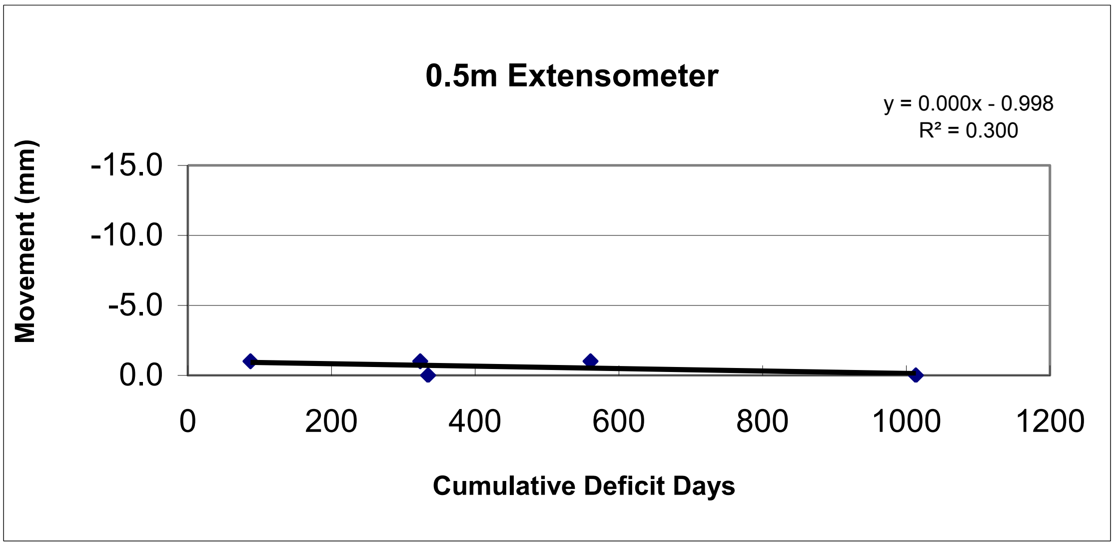

North Shore
soil suction change (pF)
-0.40
-0.20
0.00
0.20
0.40
0.60
0.80
1.00
Depth (m)
0
0.5
1
1.5
2
2.5
S2
S3
S5

4.0 EXTENSOMETER READINGS - SITE 2D: MAIRANGI BAY

	Summer 04/05			Summer 05/06	
Date of Sampling	01.02.05	17.03.05	29.04.05	27.02.06	24.03.06
Deficit Value at Reading (mm)	-76.3	-81.3	-73.0	-87.7	-78.0
Cumulative Deficit Days	87.3	334.6	1013.4	323.6	560.5
Surface Monument at 0.0 mbgl	0	0	-1	-2	1
Surface Monument at 0.2 mbgl	-1	0	0	-2	0
Rod at 0.5 mbgl	-1	0	0	-1	-1
Rod at 1.0 mbgl	0	0	0	0	1
Rod at 1.5 mbgl	-3	-1	-3	-3	-2
Rod at 2.0 mbgl	-1	1	-1	-1	0
Rod at 2.5 mbgl	0	1	-1	-2	1

NOTES: 1. Extensometer readings in mm compared to Winter 05 readings
2. NR = no reading due to damage of surface monument
3. Cumulative Deficit Days = sum of values below -70 mm threshold





SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference 2E
Location Red Beach
Main Soil Type Onerahi Chaos

1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

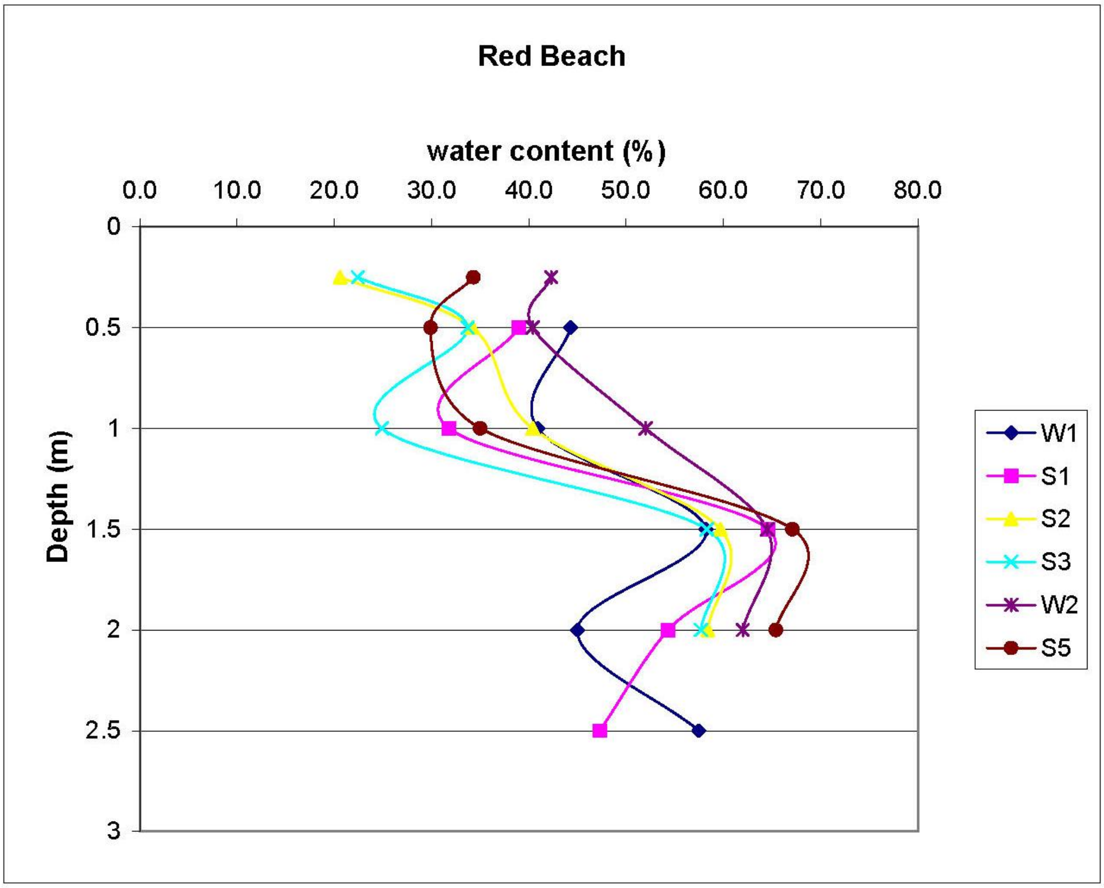
	Winter 04		Summer 04/05						Winter 05		Summer 05/06			
Date of Sampling	13.09.04		01.02.05		23.03.05		28.04.05		19.10.05		27.02.06		24.03.06	
Deficit Reading at Sampling (mm)	-5.3		-80.1		-85.6		-72.2		-14.1		-87.7		-78.0	
Date of Testing	07.10.04		22.04.05		02.05.05		18.05.05		17.11.05		-		23.05.06	
Depth to Groundwater Level (m)	Not encountered		2.4		Not encountered		Not encountered		1.5		-		Not encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.2/0.3m below GL	-	-	-	-	20.6	3.6	22.4	3.6	42.3	3.3	-	-	34.3	3.90
Sample at 0.5m below GL	44.3	3.5	39.0	3.2	34.2	3.5	33.7	3.4	40.4	3.2	-	-	29.9	3.75
Sample at 1.0m below GL	40.9	3.5	31.8	3.6	40.4	3.5	24.9	3.5	52.0	3.3	-	-	35.0	3.70
Sample at 1.5m below GL	58.2	3.4	64.6	3.4	59.7	3.4	58.3	3.4	64.5	3.2	-	-	67.1	3.5
Sample at 2.0m below GL	45.0	3.3	54.3	3.2	58.4	3.2	57.7	3.3	62.0	3.2	-	-	65.4	3.4
Sample at 2.5m below GL	57.5	3.2	47.3	3.2	-	-	-	-	-	-	-	-	-	-

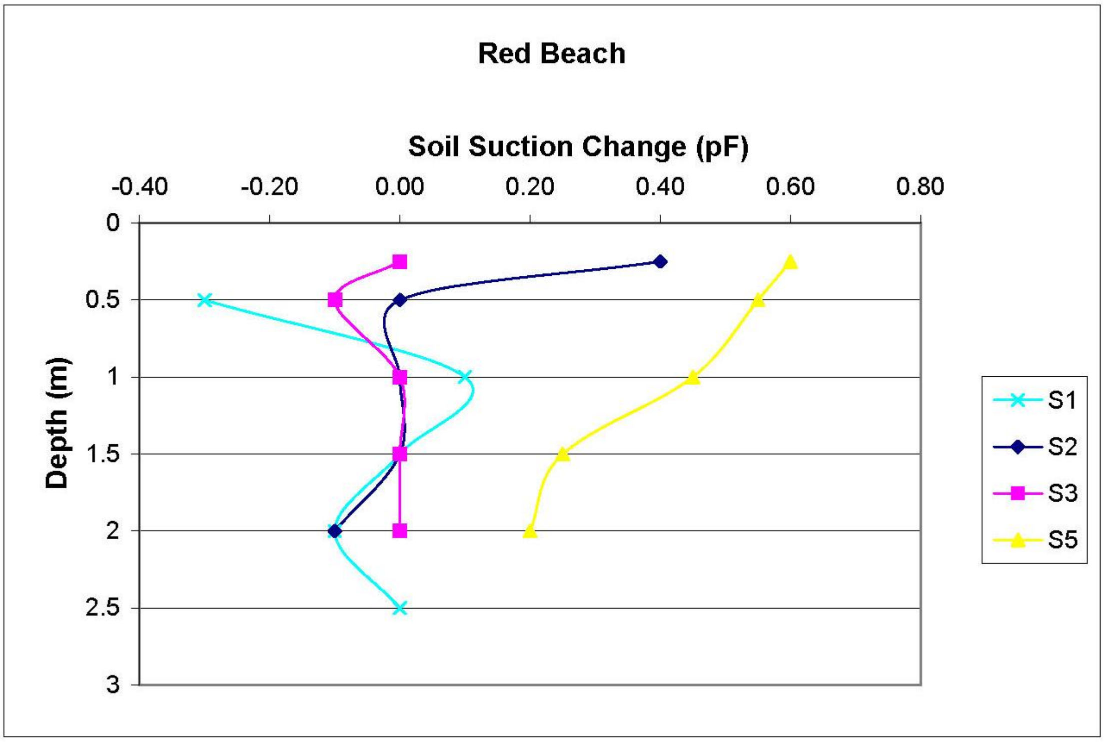
2.0 SOIL CLASSIFICATION TESTS - WINTER 2005 SAMPLES

	Atterburg Limits			
	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
Sample at 0.5m below GL	99	39	60	22
Sample at 1.0m below GL	106	37	69	24
Sample at 1.5m below GL	116	43	73	22

3.0 SOIL INDEX TESTS

	Swell-Shrink Index (%)		Core Shrinkage Index (%)	
	Oct-05	Mar-06	Mar-05	Mar-06
Sample at 0.5m below GL	3.6	2.1	3.19	4.94
Sample at 1.0m below GL	6.7	2.8	4.77	4.74
Sample at 1.5m below GL	-	-	-	7.96

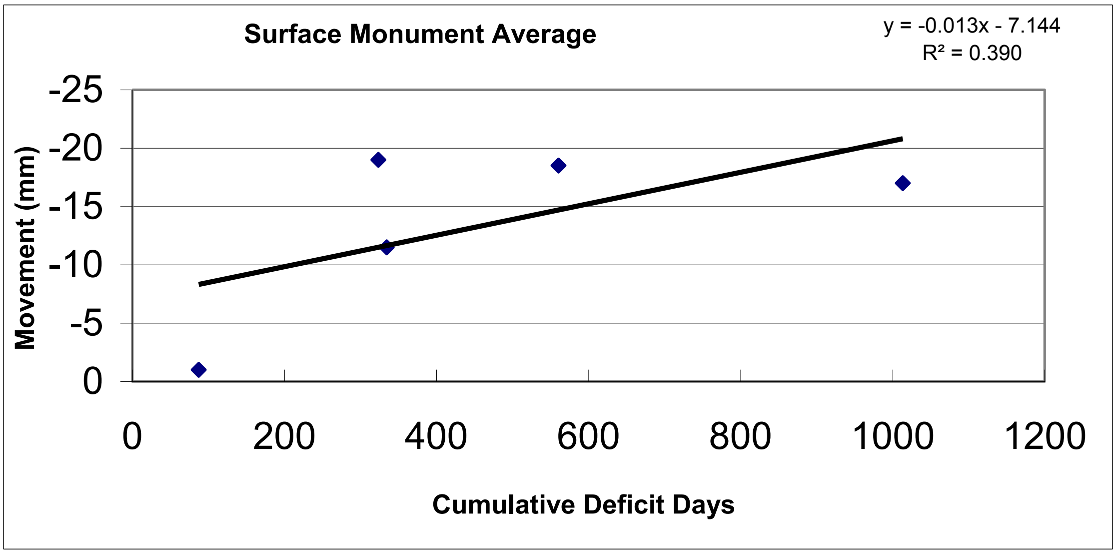

Red Beach
water content (%)
0.0 10.0 20.0 30.0 40.0 50.0 60.0 70.0 80.0
Depth (m)
0 0.5 1 1.5 2 2.5 3
W1
S1
S2
S3
W2
S5

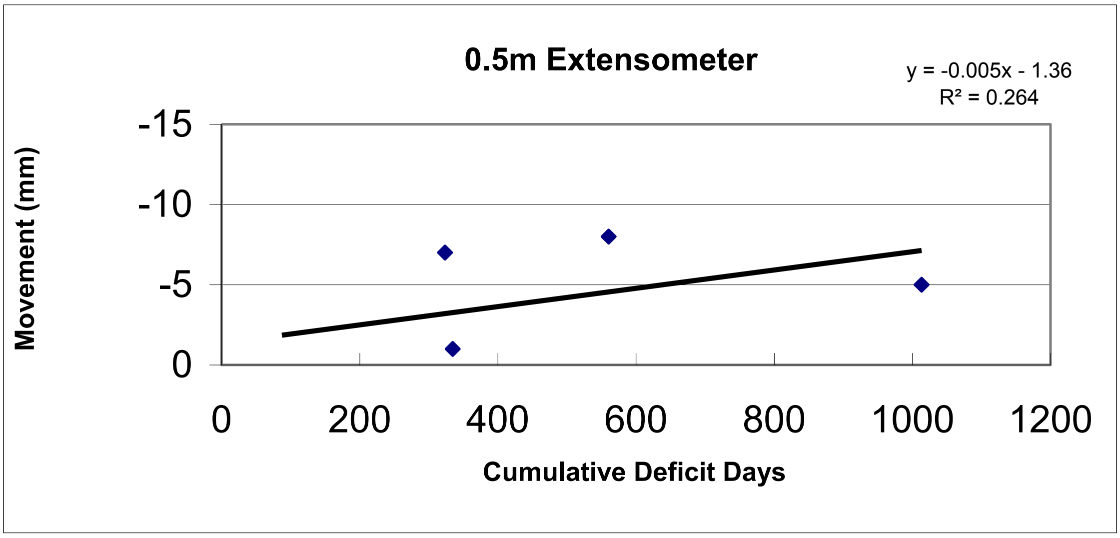

Red Beach
Soil Suction Change (pF)
-0.40 -0.20 0.00 0.20 0.40 0.60 0.80
Depth (m)
0 0.5 1 1.5 2 2.5 3
S1
S2
S3
S5

4.0 EXTENSOMETER READINGS - SITE 2E: RED BEACH

	Summer 04/05			Summer 05/06	
Date of Sampling	01.02.05	17.03.05	29.04.05	27.02.06	24.03.06
Deficit Value at Reading (mm)	-76.3	-85.6	-73.0	-87.7	-78.0
Cumulative Deficit Days	87.3	334.6	1013.4	323.6	560.5
Surface Monument at 0.0 mbgl	-1	-9	-14	-16	-15
Surface Monument at 0.2 mbgl	NR	-14	-20	-22	-22
Rod at 0.5 mbgl	1	-1	-5	-7	-8
Rod at 1.0 mbgl	0	-1	0	-1	-1
Rod at 1.5 mbgl	0	-1	0	0	1
Rod at 2.0 mbgl	0	-1	0	-1	0
Rod at 2.5 mbgl	0	-2	-1	-1	-1

NOTES: 1. Extensometer readings in mm compared to Winter 05 readings
2. NR = no reading due to damage of surface monument
3. Cumulative Deficit Days = sum of values below -70 mm threshold





SITE REPORTS FOR EXPANSIVE SOILS TESTING

Site Reference 2F
Location Pukekohe
Main Soil Type Basaltic Ash

1.0 GENERAL TESTING INFORMATION & SOIL SUCTION RESULTS

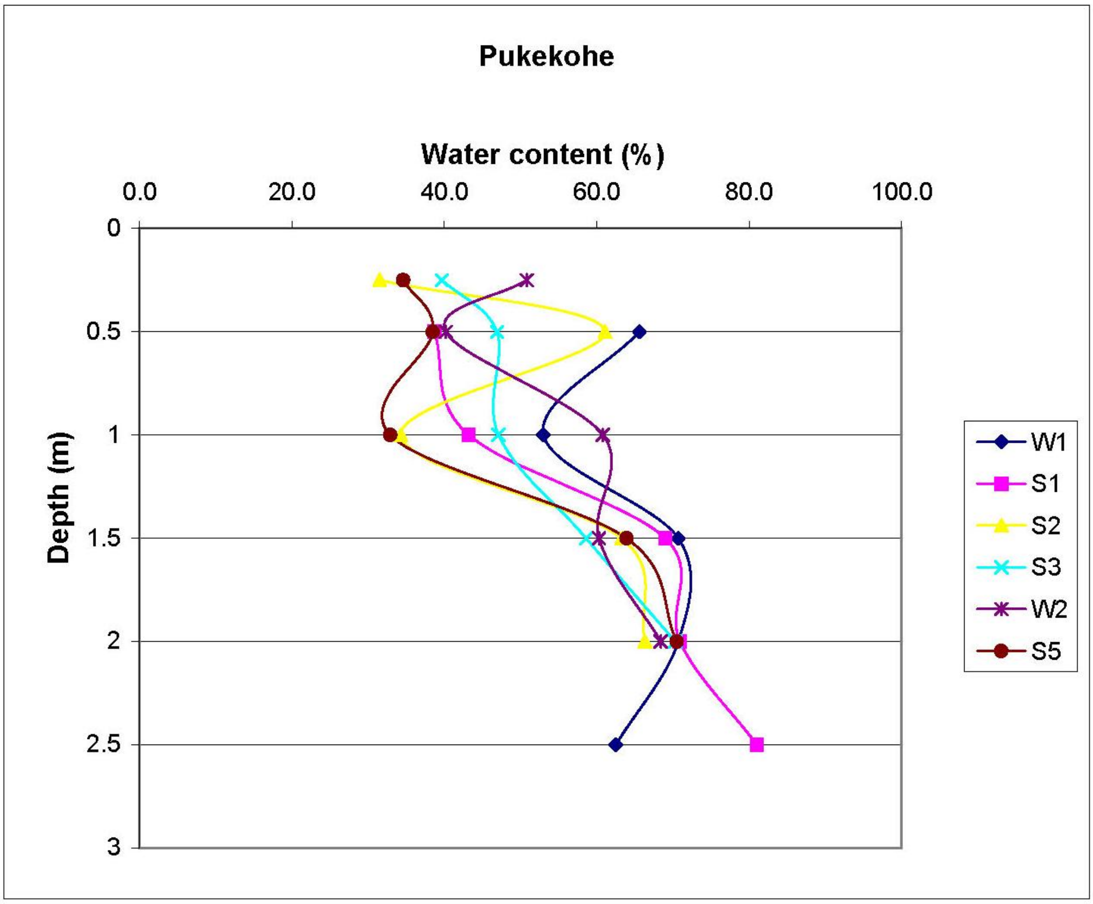
	Winter 04		Summer 04/05						Winter 05		Summer 05/06			
Date of Sampling	13.09.04		02.02.05		22.03.05		28.04.05		20.10.05		28.02.06		25.03.06	
Deficit Reading at Sampling (mm)	-5.3		-81.3		-85.3		-72.2		-14.1		-88.2		-77.4	
Date of Testing	14.10.04		26.04.05		02.05.05		18.05.05		18.11.05		-		23.05.06	
Depth to Groundwater Level (m)	Not encountered		1.7		1.9		Not encountered		1.1		-		Not encountered	
	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)	Water Content (%)	Soil Suction (pF)
Sample at 0.2/0.3m below GL	-	-	-	-	31.5	3.5	39.7	3.3	50.8	3.2	-	-	34.6	3.5
Sample at 0.5m below GL	65.6	3.2	38.7	3.70	61.1	3.4	46.9	3.3	40.2	3.2	-	-	38.5	3.4
Sample at 1.0m below GL	53.0	3.2	43.2	3.4	34.3	3.2	47.1	3.2	60.8	3.2	-	-	32.9	3.2
Sample at 1.5m below GL	70.7	3.2	69.0	3.4	63.3	3.3	58.6	3.2	60.3	3.2	-	-	63.9	3.2
Sample at 2.0m below GL	70.7	3.2	70.9	3.3	66.3	3.2	70.3	3.2	68.4	3.2	-	-	70.5	3.2
Sample at 2.5m below GL	62.5	3.2	81.0	3.2	-	-	-	-	-	-	-	-	-	-

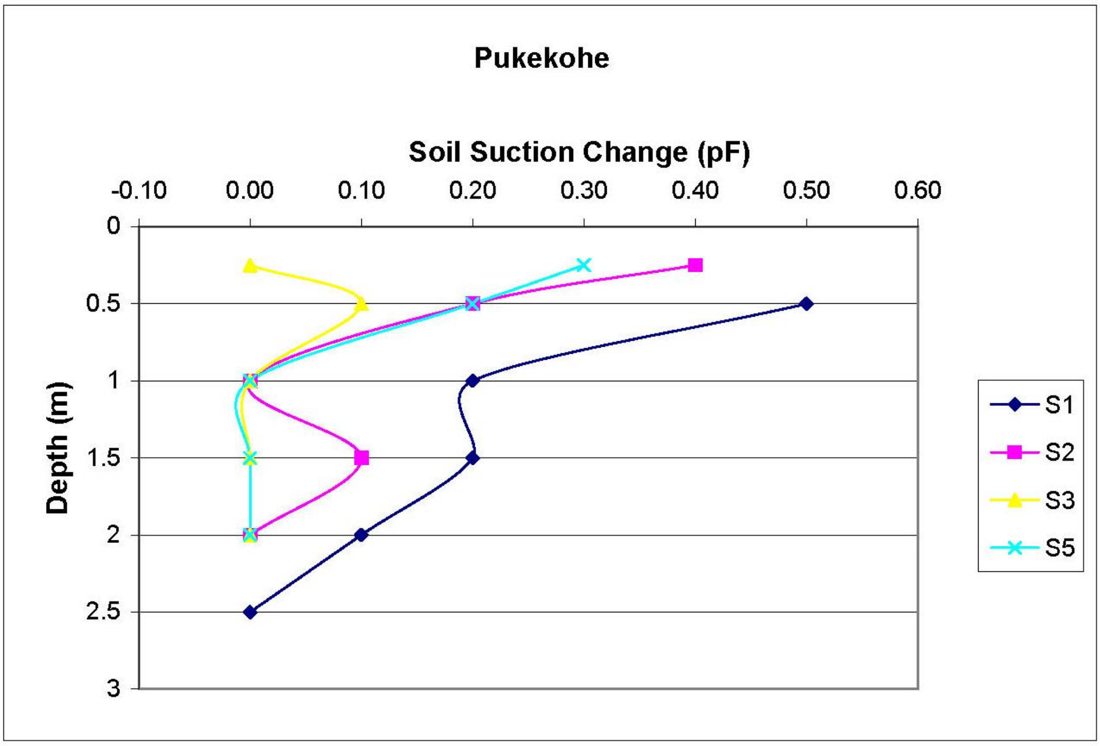
2.0 SOIL CLASSIFICATION TESTS - WINTER 2005 SAMPLES

	Atterburg Limits			
	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
Sample at 0.5m below GL	88	40	48	22
Sample at 1.0m below GL	95	56	39	22
Sample at 1.5m below GL	84	46	38	18

3.0 SOIL INDEX TESTS

	Swell-Shrink Index (%)		Core Shrinkage Index (%)	
	Oct-05	Mar-06	Mar-05	Mar-06
Sample at 0.5m below GL	2.8	2.5	2.75	2.30
Sample at 1.0m below GL	3.7	2.7	2.30	-
Sample at 1.5m below GL	-	-	-	5.49

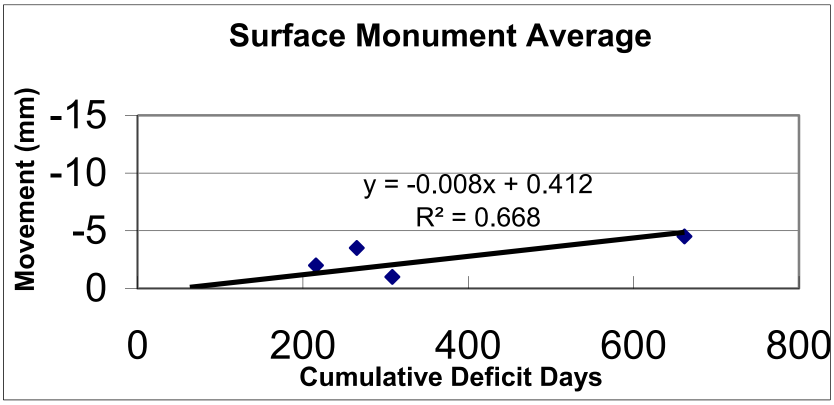

Pukekohe
Water content (%)
0.0
20.0
40.0
60.0
80.0
100.0
0
0.5
1
1.5
2
2.5
3
Depth (m)
W1
S1
S2
S3
W2
S5

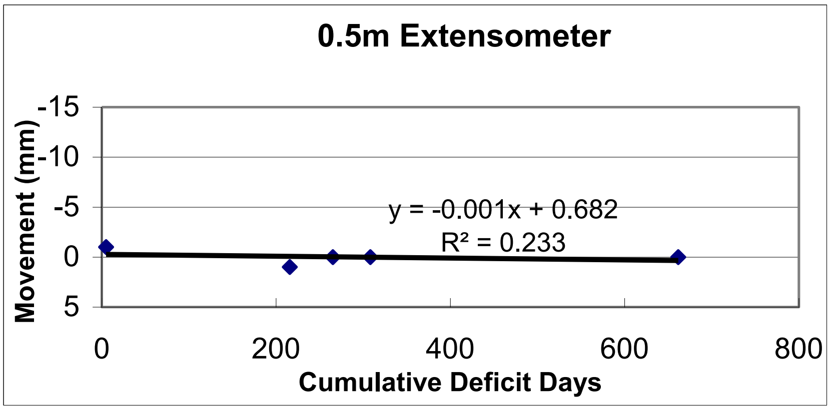

Pukekohe
Soil Suction Change (pF)
-0.10
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0
0.5
1
1.5
2
2.5
3
Depth (m)
S1
S2
S3
S5

4.0 EXTENSOMETER READINGS - SITE 2F: PUKEKOHE

	Summer 04/05			Summer 05/06	
Date of Sampling	02.02.05	17.03.05	28.04.05	28.02.06	25.03.06
Deficit Value at Reading (mm)	-76.3	-81.3	-72.2	-88.2	-77.4
Cumulative Deficit Days	63.3	215.9	308.3	265.3	661.5
Surface Monument at 0.0 mbgl	NR	-4	-2	-6	-7
Surface Monument at 0.2 mbgl	1	0	0	-1	-2
Rod at 0.5 mbgl	1	0	0	0	-1
Rod at 1.0 mbgl	2	0	1	0	-1
Rod at 1.5 mbgl	1	0	0	-1	-1
Rod at 2.0 mbgl	1	0	0	0	0
Rod at 2.5 mbgl	2	1	1	1	0

NOTES: 1. Extensometer readings in mm compared to Winter 05 readings
2. NR = no reading due to damage of surface monument
3. Cumulative Deficit Days = sum of values below -70 mm threshold



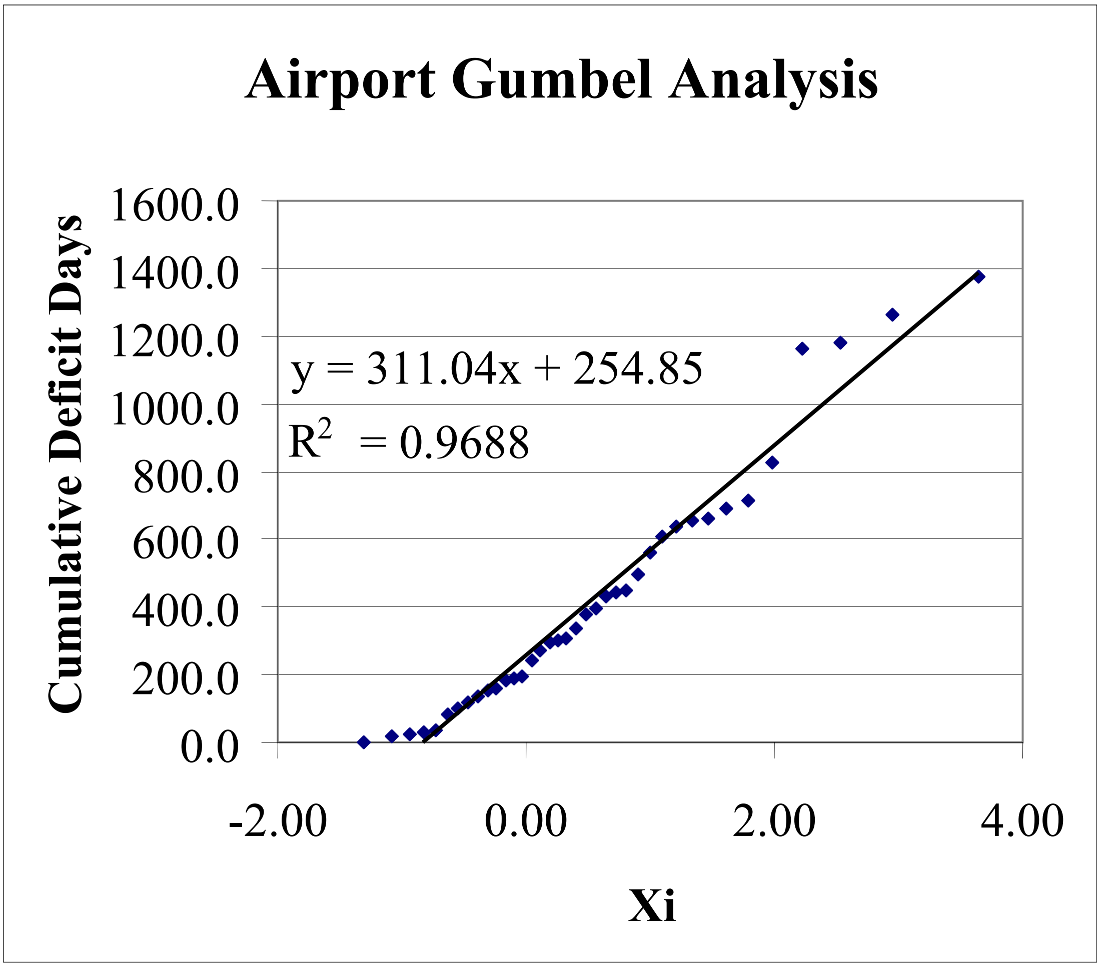


Appendix D

Cumulative Deficit Data and Gumbel Analyses

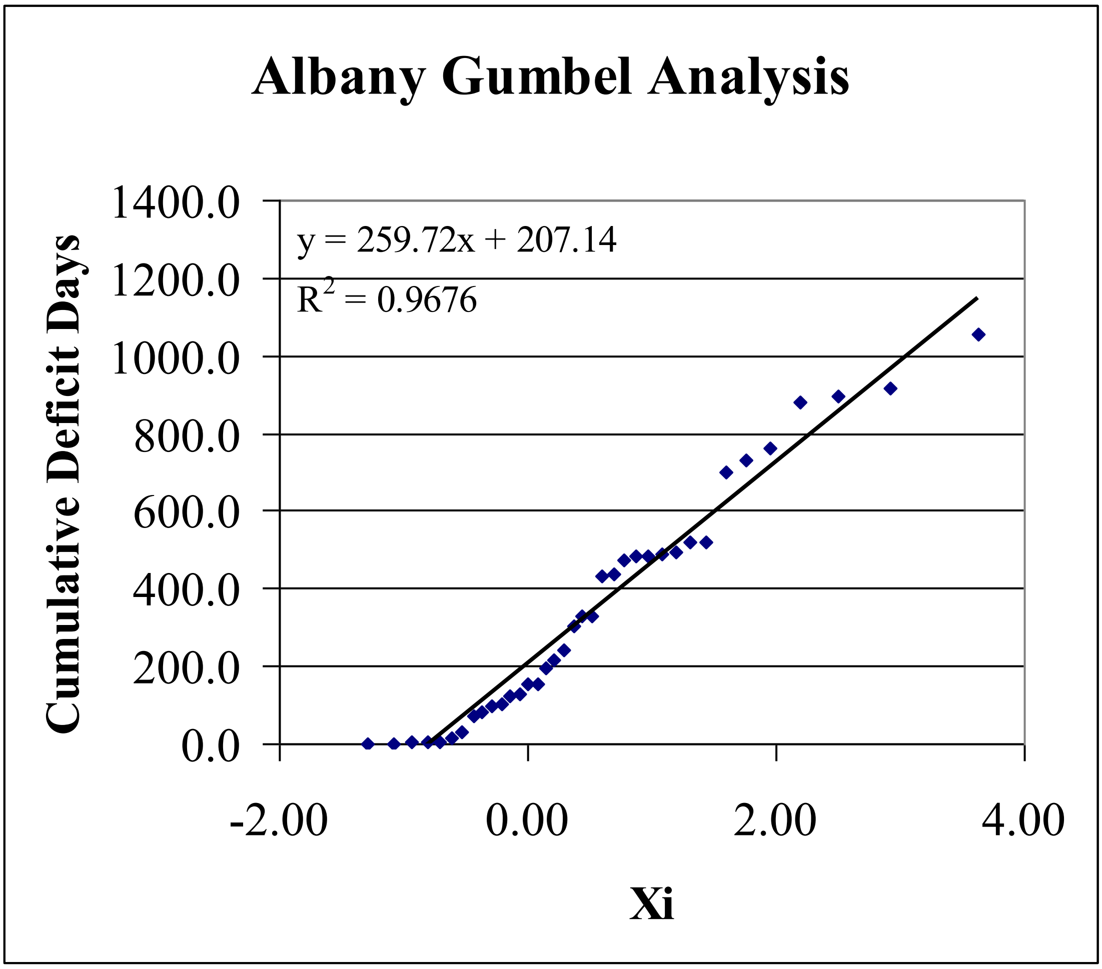
AIRPORT CUMULATIVE DEFICITY DAYS DATA AND GUMBEL ANALYSES

Year	CDD	Ordered by CDD		Rank	Pi	Xi
1963	117.7	1974	1376.3	38	0.97	3.65
1964	428.6	1978	1263.0	37	0.95	2.94
1965	0.0	1973	1180.7	36	0.92	2.53
1966	0.0	1998	1164.0	35	0.90	2.22
1967	0.0	1970	826.5	34	0.87	1.99
1968	158.1	1983	716.2	33	0.85	1.79
1969	26.6	1981	693.5	32	0.82	1.62
1970	826.5	1994	661.0	31	0.79	1.47
1971	447.8	2006	654.0	30	0.77	1.34
1972	0.0	1993	636.4	29	0.74	1.22
1973	1180.7	1982	608.3	28	0.72	1.10
1974	1376.3	1991	559.3	27	0.69	1.00
1975	137.3	1979	498.6	26	0.67	0.90
1976	306.2	1971	447.8	25	0.64	0.81
1977	297.1	1987	441.8	24	0.62	0.72
1978	1263.0	1964	428.6	23	0.59	0.64
1979	498.6	2005	394.0	22	0.56	0.56
1980	0.0	1990	377.3	21	0.54	0.48
1981	693.5	1995	334.5	20	0.51	0.40
1982	608.3	1976	306.2	19	0.49	0.33
1983	716.2	1997	302.7	18	0.46	0.26
1984	0.0	1977	297.1	17	0.44	0.19
1985	22.8	1999	271.6	16	0.41	0.12
1986	0.6	1989	244.3	15	0.38	0.05
1987	441.8	2001	195.5	14	0.36	-0.02
1988	180.6	2000	186.5	13	0.33	-0.09
1989	244.3	1988	180.6	12	0.31	-0.16
1990	377.3	1968	158.1	11	0.28	-0.24
1991	559.3	1996	153.2	10	0.26	-0.31
1992	99.1	1975	137.3	9	0.23	-0.38
1993	636.4	1963	117.7	8	0.21	-0.46
1994	661.0	1992	99.1	7	0.18	-0.54
1995	334.5	2004	80.1	6	0.15	-0.63
1996	153.2	2003	37.4	5	0.13	-0.72
1997	302.7	1969	26.6	4	0.10	-0.82
1998	1164.0	1985	22.8	3	0.08	-0.94
1999	271.6	2002	15.7	2	0.05	-1.09
2000	186.5	1986	0.6	1	0.03	-1.30
2001	195.5	1965	0.0			
2002	15.7	1966	0.0			
2003	37.4	1967	0.0			
2004	80.1	1972	0.0			
2005	394.0	1980	0.0			
2006	654.0	1984	0.0			


Airport Gumbel Analysis
Cumulative Deficit Days
Xi
y = 311.04x + 254.85
R^2 = 0.9688
1600.0
1400.0
1200.0
1000.0
800.0
600.0
400.0
200.0
0.0
-2.00
0.00
2.00
4.00

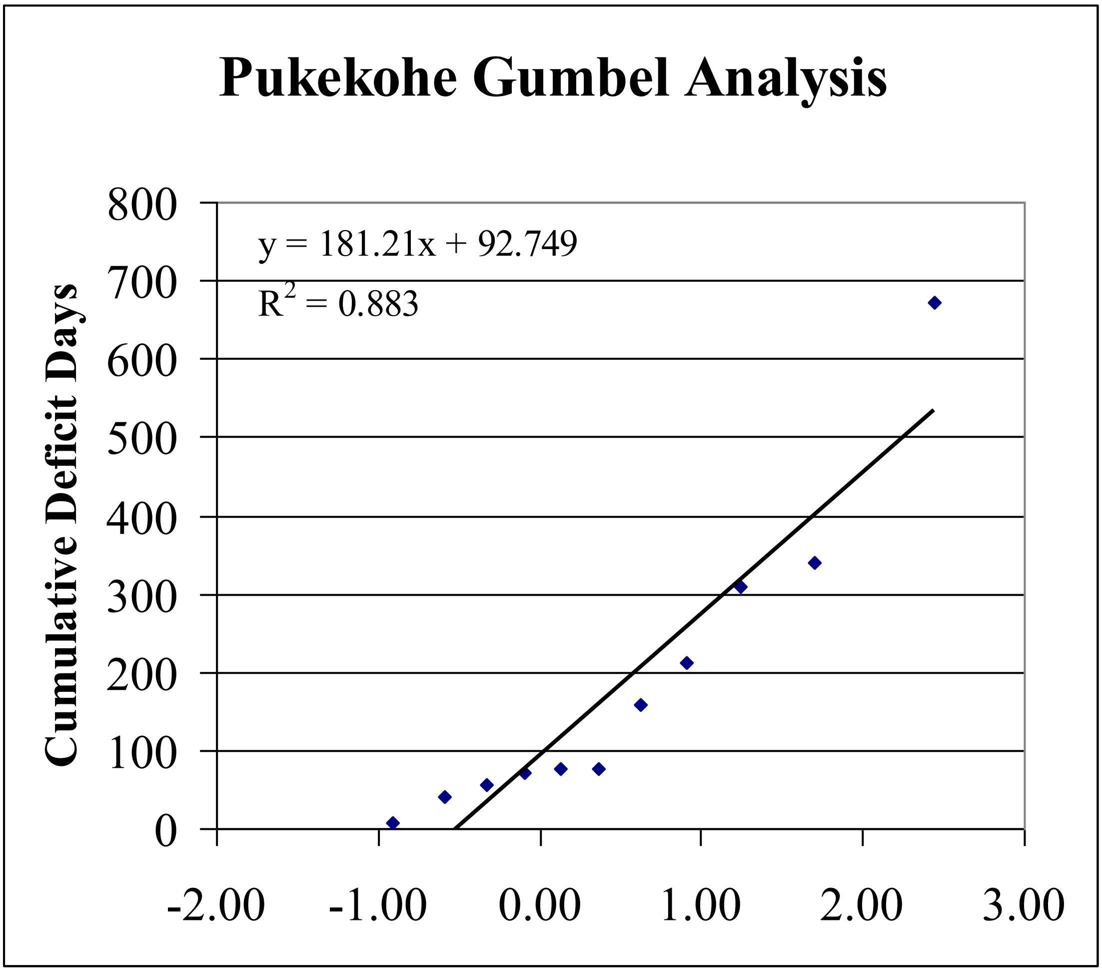
ALBANY CUMULATIVE DEFICIT DAYS DATA AND GUMBEL ANAYLSES

Year	CDD	Ordered by CDD		Rank	Pi	Xi
1967	0.0	2005	1053.2	37	0.97	3.62
1968	6.6	1974	917.8	36	0.95	2.92
1969	80.0	1973	896.0	35	0.92	2.50
1970	728.7	1978	882.7	34	0.89	2.20
1971	214.1	1994	760.7	33	0.87	1.96
1972	153.8	1970	728.7	32	0.84	1.76
1973	896.0	1998	701.6	31	0.82	1.59
1974	917.8	2000	522.4	30	0.79	1.44
1975	15.3	1977	518.8	29	0.76	1.31
1976	489.8	1983	494.1	28	0.74	1.19
1977	518.8	1976	489.8	27	0.71	1.07
1978	882.7	1990	482.8	26	0.68	0.97
1979	330.0	2006	481.7	25	0.66	0.87
1980	0.0	1987	473.3	24	0.63	0.78
1981	306.2	1995	436.1	23	0.61	0.69
1982	156.5	1993	430.9	22	0.58	0.60
1983	494.1	1979	330.0	21	0.55	0.52
1984	0.0	1991	328.9	20	0.53	0.44
1985	2.6	1981	306.2	19	0.50	0.37
1986	7.6	1989	239.4	18	0.47	0.29
1987	473.3	1971	214.1	17	0.45	0.22
1988	99.8	1999	198.1	16	0.42	0.15
1989	239.4	1982	156.5	15	0.39	0.07
1990	482.8	1972	153.8	14	0.37	0.00
1991	328.9	2003	129.2	13	0.34	-0.07
1992	124.2	1992	124.2	12	0.32	-0.14
1993	430.9	2004	102.3	11	0.29	-0.21
1994	760.7	1988	99.8	10	0.26	-0.29
1995	436.1	1969	80.0	9	0.24	-0.36
1996	2.3	1997	72.9	8	0.21	-0.44
1997	72.9	2001	28.5	7	0.18	-0.53
1998	701.6	1975	15.3	6	0.16	-0.61
1999	198.1	1986	7.6	5	0.13	-0.71
2000	522.4	1968	6.6	4	0.11	-0.81
2001	28.5	1985	2.6	3	0.08	-0.93
2002	1.2	1996	2.3	2	0.05	-1.08
2003	129.2	2002	1.2	1	0.03	-1.29
2004	102.3	1984	0.0			
2005	1053.2	1980	0.0			
2006	481.7	1967	0.0			


Albany Gumbel Analysis
y = 259.72x + 207.14
R2 = 0.9676
Cumulative Deficit Days
1400.0
1200.0
1000.0
800.0
600.0
400.0
200.0
0.0
-2.00
0.00
2.00
4.00
Xi

PUKEKOHE CUMULATIVE DEFICIT DAYS DATA AND GUMBEL ANAYLSES

Year	CDD	Ordered by CDD		Rank	Pi	Xi
1996	70.7	2006	671.7	11	0.92	2.44
1997	211.3	1998	339.3	10	0.83	1.70
1998	339.3	2005	308.3	9	0.75	1.25
1999	157.4	1997	211.3	8	0.67	0.90
2000	75.5	1999	157.4	7	0.58	0.62
2001	57.4	2004	77.7	6	0.50	0.37
2002	6.6	2000	75.5	5	0.42	0.13
2003	40.2	1996	70.7	4	0.33	-0.09
2004	77.7	2001	57.4	3	0.25	-0.33
2005	308.3	2003	40.2	2	0.17	-0.58
2006	671.7	2002	6.6	1	0.08	-0.91



Appendix E

Appendices B and C from AS 2870 Standard

APPENDIX B
PERFORMANCE CRITERIA AND FOUNDATION MAINTENANCE
(Informative)

BI GENERAL

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

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

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

A more extensive discussion of the material in Paragraphs B2 to B4 is contained in the CSIRO Pamphlet 10-91: *Guide to Home Owners on Foundation Maintenance and Footing Performance* and its recommendations should be followed.

B2 FOUNDATION MAINTENANCE

B2.1 Foundation soils

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

(a) A means stable (non-reactive)
(b) S means slightly reactive
(c) M means moderately reactive
(d) H means highly reactive
(e) E means extremely reactive.

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

B2.2 Class A and S sites

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

B2.3 Class M, II and E Sites

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

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

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

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

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

(i) 1½ x mature height for Class E sites
(ii) 1 x mature height for Class H sites
(iii) ¾ x mature height for Class M sites.

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

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

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

B3 PERFORMANCE REQUIREMENT FOR WALLS

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

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

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

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

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

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

B4 PERFORMANCE REQUIREMENTS FOR A CONCRETE FLOOR

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

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

Floor coverings and their adhesives can be damaged by moisture in the concrete and by the shrinkage that occurs as the concrete dries. Drying could take three months or more. The time of fixing of floor coverings and the selection of the adhesive should take these factors into account (see AS 3958.1).

APPENDIX C
CLASSIFICATION OF DAMAGE DUE TO FOUNDATION MOVEMENTS
(Normative)

TABLE Cl
CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

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

TABLE C2
CLASSIFICATION OF DAMAGE WITH REFERENCE TO CONCRETE FLOORS

Description of typical damage	**Approximate crack width limit in floor**	**Change in offset from a 3 m straight edge centred over defect (see Note 6)**	**Damage category**
Hairline cracks, insignificant movement of slab from level.	<0.3 mm	<8 mm	0
Fine but noticeable cracks. Slab reasonably level.	<1.0 mm	<10 mm	1
Distinct cracks. Slab noticeably curved or changed in level.	<2.0 mm	<15 mm	2
Wide cracks. Obvious curvature or change in level.	2 mm to 4 mm	15 mm to 25 mm	3
Gaps in slab. Disturbing curvature or change in level.	4 mm to 10 mm	>25 mm	4

Notes:

1. Crack width is the main factor by which damage to walls is categorised. The width may be supplemented by other factors, including serviceability, in assessing category of damage.
2. In assessing the degree of damage, account shall be taken of the location in the building or structure where it occurs, and also of the function of the building or structure.
3. Where the cracking occurs in easily repaired plasterboard or similar clad-framed partitions, the crack width limits may be increased by 50% for each damage category.
4. Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.
5. Account should be taken of the past history of damage in order to assess whether it is stable or likely to increase.
6. The straight edge is centred over the defect, usually, and supported at its ends by equal height spacers. The change in offset is then measured relative to this straight edge.

Appendix F

Foundation Analysis

FOUNDATION ANALYSIS
Review of AS 2870/HB 28:197 Design Approach

F1 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.

F2 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 2 m and with clad framing or masonry veneer.

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

F3 FOUNDATION DESIGN TO AS 2870

F3.1 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 further 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) Specified designs – these are to be based on engineering design principles and detail in Section 4 of AS 2870, which allow the designer to alter the standard designs for footing types 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 (themselves) 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 i.e. buildings beyond the limits prescribed in Section F3.1(a). However for standard draft designs, AS 2870 specifies acceptable ranges for design parameters in Clause 4.5.1 of the Standard.

Although Clause 1.1 of AS 2870 indicates that the Standard will generally be applied to "Class 1 and 10A Buildings", i.e. 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.

F3.2 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 30 m;

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

(d) Two-storey construction with a suspended concrete floor at the first floor level except in accordance with Clause 3.5 [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 m];

(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 the 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 3 m height.

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

F3.3 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 commonly used in Auckland come within the AS 2870 definitions 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 1.5 m 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" i.e. 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 F1.

Table F1: 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

F3.4 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” i.e. 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 F1. 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).

F3.5 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 400–1,500 kPa/m for beams in contact with swelling soil, with the further limitation of 100 q (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”.

(b) Soil heave – the free unloaded heave (y_m) is calculated using Equation F1 as:

$$y_m = 0.7\, y_s \qquad \textbf{Equation F1}$$

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 Section 5.4.2) the maximum in the range is always used for analysis to ensure conservatism.

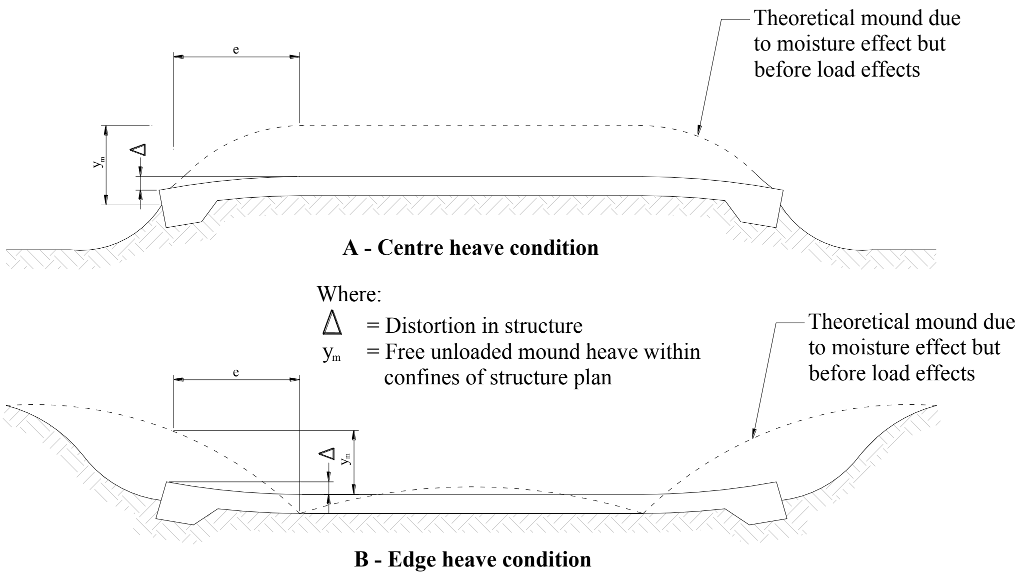


Figure F1: Soil structure interaction (from HB 28:1997 Figures 1.6 and 5.4)

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 pre-treated to maintain a high water content. This is further discussed in Section 15.0 *Residual swelling potential*.

(c) Edge distance

$e = (H_s/8 + y_m/36)$ for centre heave **Equation F2**

$e = 0.2$ slab length, or $(0.6 + y_m/25)$ for edge heave **Equation F3**

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 F4.

(e)

$$m = \frac{1.5\,L}{a}$$ **Equation F4**

where

L = span of footing (m)

a = D_{er} - D_e, where **Equation F5**

$$D_{er} = \frac{H_s}{7} + \frac{y_m}{25}$$ **Equation F6**

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

(f) 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 F2 and Figure F2.

Table F2: Maximum design differential footing 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.

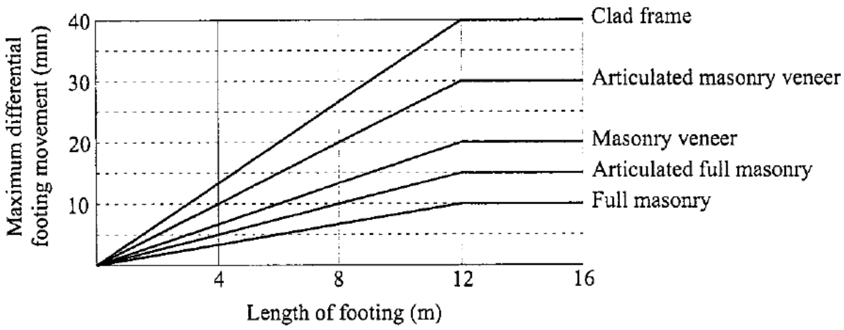


Figure F2: Acceptable footing movement as a function of length of footing

F3.6 AS 2870 gravity loads

Table F3 is reproduced from Clause 6.2.2 of HB 28:1997, 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 F3 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 F3: Typical stiffened slab loads for a single-storey building with tiled roof and trusses (taken from HB 28:1997 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 28:1997 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".

F3.7 Failure definition

In order to guard against unrealistic expectations of foundation performance AS 2870 includes some definitions of failure, including the implications that certain levels (of damage) should not be regarded as "significant failure". These are discussed further in Appendices B3 and B4 of AS 2870 reproduced in Appendix E.

This failure definition is noted in HB 28:1997 as "an honest, though at times unpopular, aspect of the AS 2870 Standard".

A major intention (of AS 2870) was:

(a) To discourage claims for failure for what are very minor levels of damage. If such claims persisted, they would force the cost of footing systems up far above community accepted levels.

(b) To provide a reasonable framework for builders, designers and house guarantee funds to operate. Minor cracking of brick structures on reactive clays is very difficult to avoid, even by use of good design. Without some reasonable failure definition, many very competent designers and builders would have found themselves involved in litigation.

Where significant damage (Category 3 or worse) has occurred, the extent of this is clearly defined in AS 2870 so that the home owner is protected from poor design or construction practices.

In addressing the design philosophy concerning load factors for strength, the authors of HB 28:1997 state:

> It would be ideal if house footing systems could be designed to cope with the worst conceivable loads and circumstances and perform so that even the most fastidious home owner had no cause for complaint. Unfortunately this is not possible nor would it be in the community's interest to waste resources in this manner. House footings have to be designed to give a balance between cost of construction and cost of minor repairs.
>
> House footings are not expected to perform in the same way as conventional concrete or geotechnical structures. In the event of failure there is not risk to life nor are the failures, generally, expensive to repair.

Thus it is appropriate to seek load factors and performance criteria that are specific to footing design. These factors should give designs consistent with known satisfactory past performance.

It is necessary to give guidance to home owners on methods they can use to minimise the risk of damage to the footing system of their homes, just as they protect the structural frame with preventative maintenance.

F3.8 Evaluation

Key elements of the approach adopted are summarised below:

(a) With regard to "performance expectation":

The AS 2870 Standard states that houses should be crack-free, but if ground movement is experienced a low incidence of minor damage may result.

Further, AS 2870 expects the owner to provide some control of moisture condition on the site so that extreme moisture changes (arising from, for example, leaking plumbing or large trees) do not have to be considered in design.

In order to ensure that these expectations can be reached, it is necessary to rationally consider the load factors and to calibrate designs as calculated with past practice.

(b) With regard to "assessment of load factor":

A rational method for assessing load factors for unusual design situations involves a process of "cost optimisation".

Using this procedure, the total cost of design plus an allowance for the cost of failure is minimised, leading to estimates of what are appropriate load factors.

(c) With respect to "serviceability optimisation":

Although the analysis carried out (by the optimisation method) is in terms of crack width, a conventional deflection/distortion limit expressed in terms of span/500 or span/1000, or the like, is sought.

If Δ is the crack width expected in the superstructure and Ω is the width at which repairs will be required, the lack of serviceability occurs when $\Delta > \Omega$. The design problem is to find the average crack width as estimated by the design method which can be a realistic target for design, given that both Δ and Ω are subject to statistical distributions which can (in theory) be defined.

(d) With respect to "analysis and data used":

(i) Cost definition – must be related to, for example, improvements in serviceability
(ii) "… for example, providing deeper beams to increase the stiffness to result in half the crack width under the same load (or deformation) would (by this model) add an extra 15% to the structural cost (i.e. $(0.5)^{-0.2} = 1.15$)"
(iii) Failure cost

(iv) "… actual repair costs to be (typically) no more than 1.5 x cost of original slab"
(v) Variability of design
(vi) "… design methods used to predict crack widths included in internal co-efficient of variation of 50%"
(vii) Compliant threshold
(viii) "… from records, it was found that average crack width beyond which repairs are required is 9 mm (being the level at which owners are prepared to undertake significant investigation, and potentially expensive repairs)"
(ix) Target crack width
(x) "… combining all the above factors, gives a 'nominal target' crack width of 4.7 mm, corresponding to a deflection/distortion limit of 1/500.

(e) With respect to "yield optimisation":

… taking the variability of the design process as 50%, and the cost of failure at three times the initial cost, it has been found (Walsh 1985) that to minimise the overall community cost, the average strength should be 2.7 x the average load effect. Converting this factor to one based on nominal or characteristic values leads to a total load factor of 1.05 (or less) on loads, giving a somewhat surprising result that load factor for strength can be taken as unity.

(f) With respect to "mound shape":

… in the above analysis, the mound shape (y_m) was defined so that the chance of being exceeded during the life of the structure is 5%.

(g) With respect to "individual against global optimum":

The above discussion leads to an overall "community optimum" position, but some consideration has to be given to the impact on, and consequences to, the individual who may suffer a failure, including:

- the individual benefits from cost savings which can be significant, even compared with failure costs;
- greater reliability in design from (the individuals or communities) improved understanding actually lessens the risk;
- advice to home owners being given to avoid or moderate the extent of damage which may arise from inadvertent actions by owners;
- more conservative designs being selected by owners if they wish to do so.

It must be appreciated that the failure rate of house footings is extremely low. Only one or two failures per thousand houses are expected, and even then the cost per individual house would typically be a few hundred to several thousand dollars. The failure rate for houses is therefore much lower than for other geotechnical structures e.g. large dams failing at a rate of 10 to 20 per thousand.

F4 AS 2870 PERFORMANCE STATEMENTS

F4.1 Engineering design principles

Paraphrasing AS 2870 requirements for "Design by Engineering Principles" (references to AS 2870 clauses e.g. Clause 4.2 as noted).

Cl.4.1 Slabs and footings designed in accordance with engineering principles shall satisfy:

(a) Clauses as referenced separately in AS 2870 Section 4.

(b) Requirement of NZS 3101:2006 *Concrete Structures Standard*.

Cl.4.2 Slabs and footings and associated superstructure are required to achieve the performance requirements set out in Cl.1.3 when subject to the loads noted therein.

This states:

The footing systems complying with [the AS 2870 Standard] are intended to achieve acceptable probabilities of serviceability and safety of the building during its design life. Buildings supported by footing systems designed and constructed in accordance with this Standard on a "normal site" (see Clause 1.3.2), which is:

(i) not subject to abnormal moisture conditions; and
(ii) maintained such that the original site classification remains valid and abnormal moisture conditions do not develop

are expected to experience usually no damage, a low incidence of damage category 1 and an occasional incidence of damage Category 2.

AS 2870 makes reference to Appendix B for information and guidance on the maintenance of foundation site condition, and to Appendix C for damage categories, viz Tables C1 and C2 for classification of damage with respect to walls and concrete floors respectively.

NB: Refer Appendix E reproducing Appendices B and C from the AS 2870 Standard.

Cl.4.3 Footing systems are required to be designed as either:

(a) Raft footing systems, supporting a superstructure that relies entirely on the raft cantilever to resist cracking.

(b) Footing systems for walls which are (when acting with the foundation system) able to cantilever without cracking.

F4.2 AS 2870 design philosophy for gravity loads

The Standard includes empirical and section analysis rules for deriving the required foundation strength and flexural stiffness, when the foundation system is analysed using the pre-qualified soil structure interaction methods and for the prescribed "design loads" as:

(a) Design Moment M* (being not less than øMu) of the footing system, where Mu is nominal strength calculated from NZS 3101 and ø is the strength reduction factor.

(b) Flexural Stiffness (EI) of the footing system, when both E_c = 15000 MPa (fc = 20 MPa) and I is determined from NZS 3101 Table G6.6.

To ensure post-rupture ductility, the flexural cross-section is required to be reinforced to provide a nominal strength 20% greater than the cracking movement capacity (M_{cr})

calculated in accordance with NZS 3101.

F4.3 Soil structure interaction

The following soil structure interaction analysis rules apply:

(a) Design load and load effects

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 e.g. NZS 3101 *Code of Practice for the Design of Concrete Structures*.

Clause 1.4.2 of AS 2870 specifies the following Equation F7 as a means of determining the "design load" for the calculation of settlement, while 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 F7

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* e.g. 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 for Stage I report).

(b) Foundation movement

Foundation movement is required to be assessed as the level which has less than a 5% chance of being exceeded over the life of the structure.

Design soil suction profiles (H_s Δu), e.g. Profile Gamma as derived earlier in this study using the aforementioned engineering principles, are considered to satisfy this requirement.

Soil parameters used for this analysis are taken as the mean values of available results for each soil horizon or particular soil.

(c) Load effects

The factored design loads for strength and serviceability for the specified ground movement corresponding to the 1000-year (extreme drought event) are:

(i) Strength and/or safety (i.e. ultimate limit state to B1.3.1) = 1.0
(ii) Deflection and crack control (i.e. serviceability limit state to B1.3.2) = 1.0

where B1.3.1 etc are performance statements within Clause B1 *Structure* of the New Zealand Building Code.

Notably the load factors for strength are significantly less than the values typically specified in the loading Standard (NZS 4203:1992 or AS/NZS 1170.0:2004) which typically would be around 1.35 to 1.40 for a design based on a more frequent (say 50 to 100-year) return period event, and around 1.2 for the "near drought" (say 100 to 500-year) return period event. This low value reflects the relatively low cost of failure as reported by Walsh (1985 – see F3.7), and are consistent with the performance requirements specified in 1.3.2 of the AS 2870 Standard.

F5 COMPARISON OF NZS 3604 TO AS 2870

F5.1 Introduction

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

F5.2 Comparison of the geometrical limitations in NZS 3604 and AS 2870

Table F4 summarises the building variables for which both AS 2870 and NZS 3604 provide geometrical limitations for the building structure above foundation level.

Table F4: 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	one and two-storeys – timber framed	–[1]	Unlimited
	two-storey – other forms of construction	–[1]	300 m^2
	three-storey – other forms of construction	No three-storey buildings	250 m^2

Note 1. A (–) indicates that the applicable Standard makes no reference to a limitation on the applicable factor.

F5.3 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 F5.

Table F5: Foundation design parameters for Australia and Auckland

Design parameter	Australian parameter range (from AS 2870 Clause 4.5.1)		Auckland parameter range
H_s	<3 m	>3 m	1.5 m [1]
y_s	10–70 mm	100 mm	15–85 mm [1]
y_m	70 mm	7–49 mm	10–60 mm [1]
Δ	5–50 mm	5–0 mm	5–50 mm [2]
Span	5–30 m	5–30 m	5–30 m [2]
Average load	to 15 kPa	to 15 kPa	to 15 kPa [2]
Edgeline 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 Gamma as discussed in Section 13.3.
2. These proposed parameters have been adopted from AS 2870 as they have not been a focus of this particular study.

F5.4 Foundation designs using SLOG software

F5.4.1 Introduction

A desktop 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.

F5.4.2 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 F7.

The "target depths" are the minimum beam depth that meets the requirements of NZS 3406 or AS 2870, and the analyses undertaken alter 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 Section F5.4.3.

The results of the analyses are reported in Section F5.4.5.

F5.4.3 Foundation design variables

(a) Geometry – a typical rectangular single-storey house of masonry veneer construction and of external dimensions, 16 x 8 m, was modelled with the following foundation layouts, as shown in Table F6:

(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.

Table F6: 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 kNhn
"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

(b) Loadings – the SLOG programme allows for seven different loading inputs. The load parameters and values used for the analyses reported herein are summarised in Table F6.

(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 20 mm– as indicated in Table 21 and Figure 23 for masonry veneer construction have been used.

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

(f) Youngs modulus of concrete – for the analyses reported herein, an E_c, value of 15000 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 Foundations 1A, 1B, and 1 – 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.

F5.4.4 Underlying SLOG theory

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 F5 and shown on Figure F1 are:

(a) The centre heave condition; and

(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.

F5.4.5 Results of analyses

Table F8 summarises the results of the analyses undertaken for the foundation systems discussed in Section F5.4.2 and Table F7.

F5.5 Back analyses of foundations used in the Auckland region

F5.5.1 Introduction

A foundation type similar to the Type 2A foundation from Table F7 and Figure F3 is commonly used in Auckland. As discussed in Section 4.2, minimum “embedment depths” of 300 mm and 450 mm, specified in NZS 3604:1999, and 600 mm, 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".

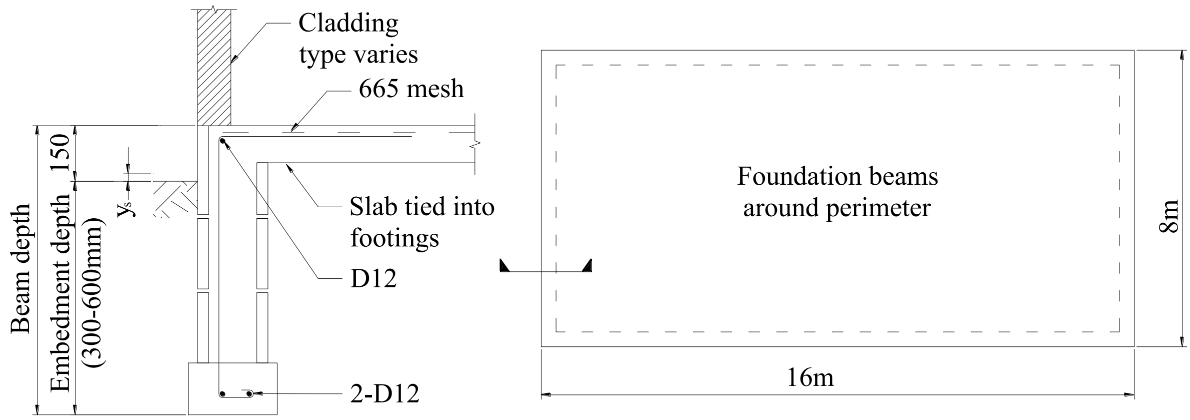


Figure F3: Typical Type 2A foundation

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.

F5.5.2 Methodology for back analyses of foundations

The foundations were analysed using the foundation design variables set out in Section F5.5.3 and the NZS 3604 specified reinforcement, as shown in Figure F3, 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.

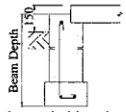
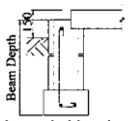
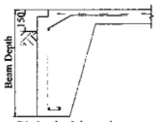
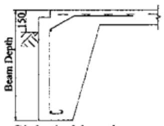
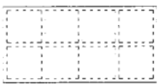
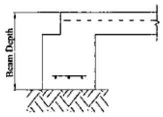
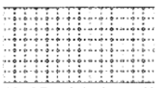
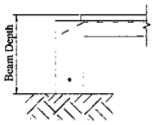
The design variables discussed in Sections F5.4.3 and F5.4.4 were used in these analyses. However, the edge loads specified in Section F5.4.4(b) were reduced from 8.0 kN/m to 3.3 kN/m for "clad frame" construction.

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 21 and Figure 23, for clad frames, articulated masonry veneer and masonry veneer construction respectively.

F5.5.3 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 F5.5.2, 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 16 x 8 m.

Table F7: 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 664 M* *Bottom 3-H12*	400 400
4	 15 x 8 Foundation beam grid	 Slab tied into beam	AS 2870 Fig 3.4	Slab SL72 Bottom 3-L11TM *NZ equivalent* *Slab 664 M* *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".

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.

Table F8: Results of analyses of foundations using SLOG programme (site expansivity Class M)

Foundation reference	Beam width (mm)	Slab thickness (mm)	Reinforcing steel in footings			Minimum target depth required
			Bottom	Slab	Top	
1 A	240	N/A	2-D12	NS [1]	D12	1725
1 B	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 [3]	350 [3]

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.

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.

3. 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 400 mm. An additional N12 bar at the top of the footing reduces this to 350 mm. However, increasing the bar size beyond 12 mm diameter has no advantage in decreasing beam depth.

Table F9: Results of back analyses of Type 2A foundations for a 16 x 8 m building using SLOG programme

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

F5.5.4 Interpretation of back analyses

Based on the back analyses results in Section F5.5.3, it can be concluded that the embedment depth of Type 2A foundations for a building with a footprint of 16 x 8 m could be determined for a specified site design characteristic surface movement, y_s, using a nomograph as in Figure F4, which reflects the cladding construction type adopted.

The historical performance of residential building foundations in Auckland indicates 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 600 mm, 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 Section F2, 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 Section 2.5, is carried out to validate the performance of residential building foundations in Auckland, and to provide designers and local TAs with design nomographs similar to that shown in Figure F4 for varying sizes of buildings.

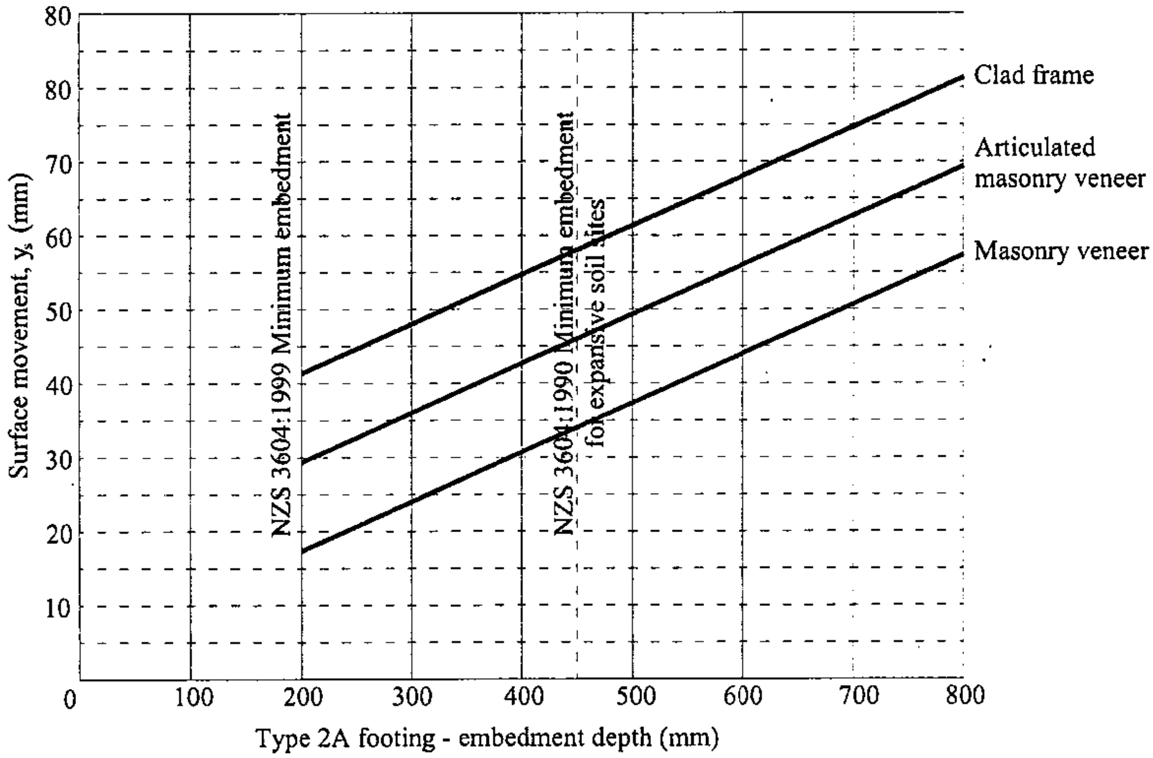


Figure F4: Surface movement against embedment depth

F6 APPLICATION OF AS 2870 IN AUCKLAND

F6.1 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?

F6.2 Conclusions

As discussed in Section F3.1, 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 750 mm. For reduced beam depth, additional steel reinforcement will be required as indicated in Table F7 for Foundation Type 2B.

(d) Based upon the geometry limitations for AS 2870 summarised in Table F4, 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 F7) are applicable to the following buildings:

  (i) Clad frame construction of up to two-storeys with concrete masonry blockwork foundation walls less than 1.5 m high.

  (ii) Masonry veneer construction of up to one-storey with concrete masonry blockwork foundation walls less than 1.5 m high.

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