

UNIVERSITY OF SOUTHAMPTON

FACULTY OF ENGINEERING, SCIENCE AND MATHEMATICS

School of Civil Engineering and the Environment

**THE USE OF DISCRETE PILES FOR INFRASTRUCTURE SLOPE
STABILISATION**

by

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ABSTRACT

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A critical review of the existing case histories and design methods for discrete pile stabilised slopes has been carried out. A review of case histories has shown that there are few fully instrumented or comprehensively detailed case histories in the literature. The complex nature of pile-slope interaction means that there is a large number of design methods proposed in the literature, but in practice simple limit equilibrium calculations are commonly used. Some of the existing design methods were found to have significant limitations. The pile embedment length was found to have a large impact on the way that the pile bends. Soil arching was reviewed and found to occur in slopes through the formation of a major principal stress arch upslope of the piles.

Real piles used to stabilise a railway embankment near Tonbridge, Kent, have been instrumented with strain gauges and inclinometer tubes to measure pile bending moment and displacement. Pile bending and displacement was monitored over a period of two years. Lateral soil displacements of 8mm were measured to a depth of 4m. Much of the pile loading occurred as a result of regrading of the granular rockfill originally placed on the embankment surface as a construction platform, although early loading occurred as the result of the instability of the slope. The top of the pile bent in an upslope direction as it was restrained within the failing mass of soil. This bending behaviour is contrary to that assumed during design of the piles.

Analysis of the piles was carried out using both the elastic analysis program ALP and the finite difference program FLAC3D. The elastic analyses were used to back analyse the measured pile behaviour, and to investigate the stiffness properties of the soil materials. The finite difference analysis was able to incorporate the construction sequence and demonstrate the impact of the slope regrading on pile behaviour. The finite difference analysis did however calculate smaller bending moment than measured in the pile, mainly because it was found to be difficult to incorporate the change in pore pressures in the slope due to rainfall, and to model the exact instability of the slope (the real pore pressures were not measured).

The use of elastic analysis was studied as a means of pile design. There is often no site specific soil stiffness data available, but despite this it was found to be more satisfactory than the limit equilibrium calculations commonly used at present.

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Notation

AOD	above ordnance datum
B	distance between piles, pile edge to pile edge
c'	effective cohesion (intersection with the τ -axis of extrapolated straight line joining peak strength states in τ, σ' space)
c_u	undrained shear strength
d	pile diameter
E'	Young's modulus
EI	pile bending stiffness
F_p	restoring force (force that the stabilising pile applies to failing mass of soil)
F_s	factor of safety for slope stability
G	shear modulus
G_r	reduced shear modulus close to the pile
I	second moment of cross sectional area
k	(with subscript to denote direction) permeability
K	earth pressure coefficient
K_0	initial <i>in situ</i> (at rest) earth pressure coefficient
K_a	active earth pressure coefficient
K_p	passive earth pressure coefficient
K_q	pile-soil limiting pressure coefficient for the frictional component of the soil strength
K_c	pile-soil limiting pressure coefficient for the cohesion component of the soil strength
L	length of slope, length of pile
M	pile bending moment
mE'	increase in Young's modulus with depth
N_p	pile-soil limiting pressure coefficient
P	pressure acting on the pile
p_u	pile-soil limiting pressure
p- δ	pressure displacement curve for passive pile loading
p-y	pressure displacement curve for active pile loading
q	surcharge load
s	pile spacing, pile centre to pile centre
V	pile shear force
z	depth from the ground surface
δu_s	lateral soil displacement
δu_p	lateral pile displacement
ϕ'	strength or angle of shearing resistance
ϕ'_{crit}	critical state angle of shearing resistance

ϕ'_{mob}	mobilised angle of shearing resistance
τ	shear stress
σ, σ'	(with subscript to denote direction) total and effective stress, respectively
γ	bulk unit weight
γ_w	unit weight of water
ψ	dilation angle
ν'	Poisson's ratio
β	angle of slope
ε	strain

Subscripts

v	vertical
h	horizontal

Chapter 1. Introduction

1.1 Introduction

A modern transport system providing efficient access to goods, services, people and places is essential for the economic and social health of the nation. However, the ever-increasing growth in demand for travel has resulted in significant congestion problems. Increasing investment is being made to improve the transport system in the UK, with money being spent on new roads and railways, widening of existing transport corridors and maintenance to improve the often deteriorating condition of existing infrastructure.

Earth structures, particularly embankments and cuttings, form an important component of inland surface transportation infrastructure throughout the world. In the UK, there are about 30,000 km of earth structures supporting the canal, rail and road networks. Much of the canal and rail network was constructed over 150 years ago, and the earth structures were not built to modern standards. As these structures get older, it has become increasingly apparent that they potentially threaten the safe operation of transport systems, especially rail. Serviceability problems include the gradual outward slump and settlement of embankments, and failure of cutting and embankment side slopes. Deep seated slope failures are common and can damage the road, railway or canal located on the top of an embankment or at the base of a cutting, with serious safety implications. Increasingly, discrete piles are installed to stabilise infrastructure slopes to prevent serious slope failure from occurring (for example, New Civil Engineer, 2001; Ground Engineering, 2001).

Prolonged periods of heavy rain during the winter of 2000/2001 caused a large number of sudden embankment and cutting slope failures across the road and rail network in the UK. Over this period, the Highways Agency reported about 60 slope failures and Railtrack in excess of

100 (Turner, 2001). Disruption on the UK's railways following the Hatfield accident was severely exacerbated by the large number of slope failures associated with storms and floods, which caused major rail routes to be closed for several days at a time while emergency repair work was carried out (Webster, 2000). The forced closure of transport corridors caused the complete breakdown of parts of the road and rail networks that were already overloaded.

Climate change is expected to bring wetter winters to the UK, and more 'energetic' weather patterns in which both summer and winter rainfall will arrive in short heavy storm events that are more likely to trigger slope failures (Hulme *et al*, 2002). The wet winter of 2000/2001 has demonstrated the travel chaos that will occur in future years unless urgent action is taken to improve the geotechnical road and rail infrastructure in the UK. The threat of climate change coupled with the massive financial cost of disruption has proved the economic case for the more rapid upgrading of the UK's road and rail system to create an 'all weather' transport network. The UK's major transport infrastructure owners have already started to accelerate earth structures improvement programmes, with Network Rail budgeting to spend £250m over a 5 year period on surveying slopes and taking action to prevent serious slope failures (Turner, 2001).

Discrete piles are an important means of stabilising infrastructure slopes, where there is usually insufficient additional land available to allow construction of stabilising measures such as toe berms or a complete regrading of the slope profile. Whole life costing carried out for infrastructure owners in connection with specific sites often favours hard structural methods of stabilisation (such as discrete piles) over softer methods (such as soil nails, grouting or lime piles) because, while more expensive to construct, the structural methods have a much longer design life and repairs have to be carried out less often (O'Brien, 2002). In particular, it is the high costs associated with gaining access to remote sites or having to cone off sections of carriageway to carry out repair works that makes the long design life options more economical. Discrete piles tend to be more cost effective in the remediation of larger infrastructure slope failures, and soil nails for shallower slips (Rose, 2003).

Investment in infrastructure is also increasingly being targeted at projects that will improve the capacity, speed and reliability of largely existing transport links. This often necessitates widening existing transport corridors to accommodate additional road lanes and railway tracks. In urban areas where land costs are high, there is likely to be demand for cutting and embankment slopes to be steepened in order to reduce the land-take required for widening. The additional loading developed at the toe of the slope by steepening may in some cases be accommodated by installing a row of discrete piles.

Discrete pile stabilisation schemes are currently over-designed owing to limitations in our understanding of the factors affecting pile behaviour. With the requirement for remediation or prevention of slope failures likely to become more frequent, there is an urgent need to reduce the high costs associated with this type of structural slope stabilisation (typically over £3000/m in 2001).

Conventional limit equilibrium design does not take into account either the soil or pile stiffness, both of which are thought to influence significantly soil arching and pile bending behaviour. Conventional calculations also generally ignore the three dimensional nature of the problem. There is therefore considerable uncertainty as to the optimal pile spacing, and the maximum length along the slope that can safely be left unsupported if large diameter piles are used. Current design methods are based on plane strain analyses, with the piles placed conservatively close together to ensure that arching will prevent any flow of soil between them. There are also uncertainties concerning the development of lateral load on the top portion of the pile with increased slope displacements, and the effect of slope loading and displacement on the pile bending behaviour. A better understanding of the behaviour of discrete piles in infrastructure slope stabilisation applications is required before less conservative design assumptions can be used.

This thesis aims to make a valid contribution to the improved understanding of discrete pile behaviour, by reviewing previous work and the current design practice, and by back analysing the results from a recent case history in which a pile stabilised slope has been instrumented to record real pile behaviour.

1.2 Objectives

The objectives of this thesis are to:

- Provide a critical review of the current design methods and case studies presented in the literature, and to review some of the fundamental pile stabilisation mechanisms, such as soil arching;
- Present the results from the instrumentation of a real embankment slope stabilised using discrete piles;
- Back analyse the behaviour of the discrete piles used to stabilise the rail embankment (above) using elasticity and finite difference methods;
- Consider the implications for discrete pile design of the results from the instrumented study.

1.3 Layout of the thesis

The layout of the thesis is described below:

Chapter 2: Review of previous work.

A review of case studies and design methods currently used for pile stabilised slopes. The review also considers the existing work carried out to understand better the fundamental mechanisms of pile stabilisation, and hence the impact of pile embedment length and soil arching.

Chapter 3: Instrumentation and monitoring of discrete piles at Hildenborough, Kent.

Presentation and discussion of the results from field instrumentation installed in discrete piles designed to stabilise a railway embankment at Hildenborough, Kent.

Chapter 4: Back analysis of the piles at Hildenborough using pseudo-finite element analysis.

Back analysis of the pile displacement and bending behaviour using the pseudo-finite element program ALP, which models the soil as a series of elastic-plastic springs. The calculated pile behaviour using these methods is compared with the monitoring results and the pile behaviour assumed in the original limit equilibrium design.

Chapter 5: Back analysis of the piles at Hildenborough using finite difference analysis.

Back analysis of the pile displacement and bending behaviour using the finite difference program FLAC 3D. The full equilibrium of the slope and the construction stages are modelled to replicate the loading experienced by the real piles.

Chapter 6: Conclusions and further work.

The main conclusions drawn from the findings presented in this thesis are summarised and recommendations for further work are made.

Chapter 2. Review of previous work

2.1 Introduction

Over recent years, pile stabilisation of slopes has come into fairly widespread use. In particular, this form of stabilisation has an important role to play as a means of quickly improving the stability of a failing slope, whereas alternatives such as drainage, (particularly in clay slopes) often take some time to act. Piles are also often favoured due to their long design life and low maintenance costs.

Practical use has demonstrated piles to be an effective means of preventing excessive slope movement. However, the methods of design and analysis for discrete piles vary widely, due to a lack of sufficient information on stabilisation mechanisms and pile-soil interaction.

Discrete piles are designed and installed to penetrate through the moving portion of the slope into firm, non-yielding soil strata underneath. Displacement of the soil mass over the sliding surface then induces significant bending and shear forces within the piles, particularly around the location of the sliding surface. In this way, the earth pressure developed on the piles due to a potential sliding soil mass is transferred to stable strata beneath the sliding surface. The purpose of the piles is in effect to redistribute the stresses within the soil mass, forcing a larger mass of soil to resist the potential failure. Excessive slope movements are therefore reduced by means of the piles 'pinning' the sliding mass of soil.

Discrete piles are individual stabilising elements, and are usually installed some distance apart from each other. Soil arching has been demonstrated experimentally to take place between piles used to stabilise slopes (Bosscher and Gray, 1986; Adachi *et al*, 1989). The piles must be placed sufficiently close for the soil arching mechanism to prevent localised failure adjacent to the

piles, which would lead to a flow of soil between the piles. Engineering practice has demonstrated that piles placed into a firm, non-yielding base and designed to meet the conditions for soil arching will successfully provide significant additional stability to a failing slope (Guest *et al*, 2000; Carder *et al*, 2003).

In most cases piles have been designed to stabilise pre-existing failures. However, Poulos (1995) describes the design of piles to stabilise a newly cut but over-steep highway slope, where the piles are installed prior to the cutting of the slope – such that they act in a preventive way (i.e. the piles prevent failure from occurring at all). Design of piles in a preventative role is complicated by the need of many design methods for a failure surface on which to base design.

The current design approaches consist basically of three stages, as described by Viggiani (1981). These stages are:

- Selecting the diameter, spacing, depth of the piles and their position in the slope
- Evaluating the restoring shear force required from the piles to give a suitable factor of safety for the failing mass
- Evaluating the shear forces and bending moment within each pile

These stages may be carried out in any order, but for site specific reasons, pile dimensions and location (e.g. position in the slope) are most likely to be determined first. The design of discrete piles in this way is usually an iterative process, with pile spacing and penetration altered until an efficient design is reached. This chapter considers each of these design stages, although division of the pile-slope interaction mechanisms and design methods into a number of stages is difficult, as many are inter-related.

While there have been previous literature reviews on discrete piles (Rose *et al*, 1998; Carder and Temporal, 2000), none have been entirely comprehensive. There is, therefore, the need for a more extensive review of published case histories and design methodologies. This chapter considers in detail the work that has already been done to understand the basic mechanisms of pile-slope stabilisation, and relates this to both current design practice and proposed design methods. Pile-slope interaction is complex, and not fully understood, and subsequently it is of no surprise that there are a wide range of methods proposed for pile-slope analysis and design. Many of these have substantial limitations, and these are highlighted and discussed in detail in the subsequent sections.

There are also a large number of case studies in the literature. Many of these concern the installation of piles to stabilise landsides in natural slopes, rather than cut slopes and embankments. There are a few cases where instrumentation has been inserted into piles to record their bending and displacement behaviour, although generally there is a paucity of

information on real pile behaviour. The final section of this chapter studies the most important of these case histories.

2.1.1 Layout of chapter

Table 2.1 shows the section headings for the remainder of Chapter 2.

Section	Content
2.2	Slope failure
2.3	Calculating the force acting on the pile head from the failing soil mass
2.4	Pile spacing and soil arching
2.5	Evaluating the shear force and bending moment in each pile
2.6	Three dimensional finite element analyses
2.7	Case histories
2.8	Summary of review of previous work

Table 2.1. Section headings for Chapter 2.

2.2 Slope failure

This section aims to cover briefly the basic forms and mechanisms and terminology of slope failure. It is not intended to be an exhaustive guide, but simply an aid to understanding the forms and types of failure that piles have been used to stabilise.

2.2.1 Slope failure in embankments and cuttings

There are a number of mechanisms of slope failure that occur in infrastructure slopes. It is important to understand the difference between these as they are likely to influence the behaviour of stabilising piles installed in the slope.

For the purposes of this thesis, and to distinguish between some of the types of failure common to infrastructure slopes, a distinction is made between shallow and deep failures (based on Coppin and Richards, 1990):

- shallow failures, which are up to 2.0m deep and largely contained within the embankment or cutting slopes. Failures tend to be translational in shape.
- deep seated failures, in which a soil mass more than 2m deep fails in a rotational slip. The failure surface often penetrates outside the slope, damaging infrastructure or neighbouring property.

Deep seated failures occur in both cutting and embankment slopes. They are of considerable concern as they may undermine or bury infrastructure creating a danger to the public and causing serious disruption and economic loss.

Many embankments suffer serviceability failure. Serviceability failure is characterised by shallow failure or steady embankment settlement and is associated with excessive movement rather than total instability. While serviceability failure will not prevent use of the infrastructure, it can cause an escalation in maintenance requirements and cost (Perry *et al*, 2001). If serviceability failure is not addressed, it can lead to deterioration and ultimately instability of the slope.

In clay infrastructure slopes the main trigger for slope failure is a change in the pore water pressure regime (Greenwood, 1998; Vaughan and Walbancke, 1973). Mechanisms such as weathering and seasonal clay shrinkage and heave also contribute to failure, but are less well understood (Greenwood, 1998).

2.2.2 Deep failure in cuttings

There has been considerable attention paid to the delayed collapse of cutting slopes over the last 40 years (Henkel, 1957; Skempton, 1964, 1970, 1977). As the cutting is excavated, the clay at the base is unloaded and will tend to heave, generating lower pore pressures (Bishop and Bjerrum, 1960). These pore pressures will dissipate at a rate dependent on the permeability of the clay and the bounding pore pressure conditions until equilibrium pore water pressures are reached. For highly impermeable clays such as intact London Clay this may take of the order of 30 – 100 years. In cutting slopes originally cut too steep, the equilibration of pore pressures results in delayed failure (Vaughan and Walbancke, 1973).

In stiff brittle clays, strain softening may occur. Potts *et al* (1997) demonstrate using numerical analyses that this results in progressive failure of the slope, with the largest shear strains first forming at the toe of the slope and then moving horizontally inward as the pore pressures equilibrate (Figure 2.1). The numerical analyses run by Potts *et al* (1997) show that the time to failure is dominated by the permeability of the clay and by the magnitude of the initial *in-situ* lateral stresses.

2.2.3 Deep failure in embankments

The majority of railway embankments in the UK are well over 100 years old and were constructed before the availability of modern construction machinery. The material they were constructed from was usually sourced from the neighbouring areas of cut for the railway line, and tipped with little or no compaction taking place (Perry *et al*, 2001). Overconsolidated clays

when freshly excavated have suctions or reduced pore water pressures. Embankments constructed with these materials remained stable in the short term, but with the onset of prolonged rainfall and percolation of water into the fill material (aided by poor compaction) the suctions dissipated, often causing deep seated failure to occur in the newly finished embankment (Squire, 1880). Inadequate repair has left rupture surfaces and zones of failed material in the embankment where the strength of the material may be considerably reduced. Movement of the rupture surfaces can be re-activated by a rise in pore pressures due to heavy rainfall (for example, Ground Engineering, 1995). Vaughan (1994) has noted that most clay railway embankments over 6m high show some evidence of failure.

The effects of embankment settlement have mainly been countered in the past by adding ash or ballast to the top of the embankment. The presence of the stronger ash material on the crests and side slopes often drives the failure surface deeper into the embankment (Perry *et al*, 2001).

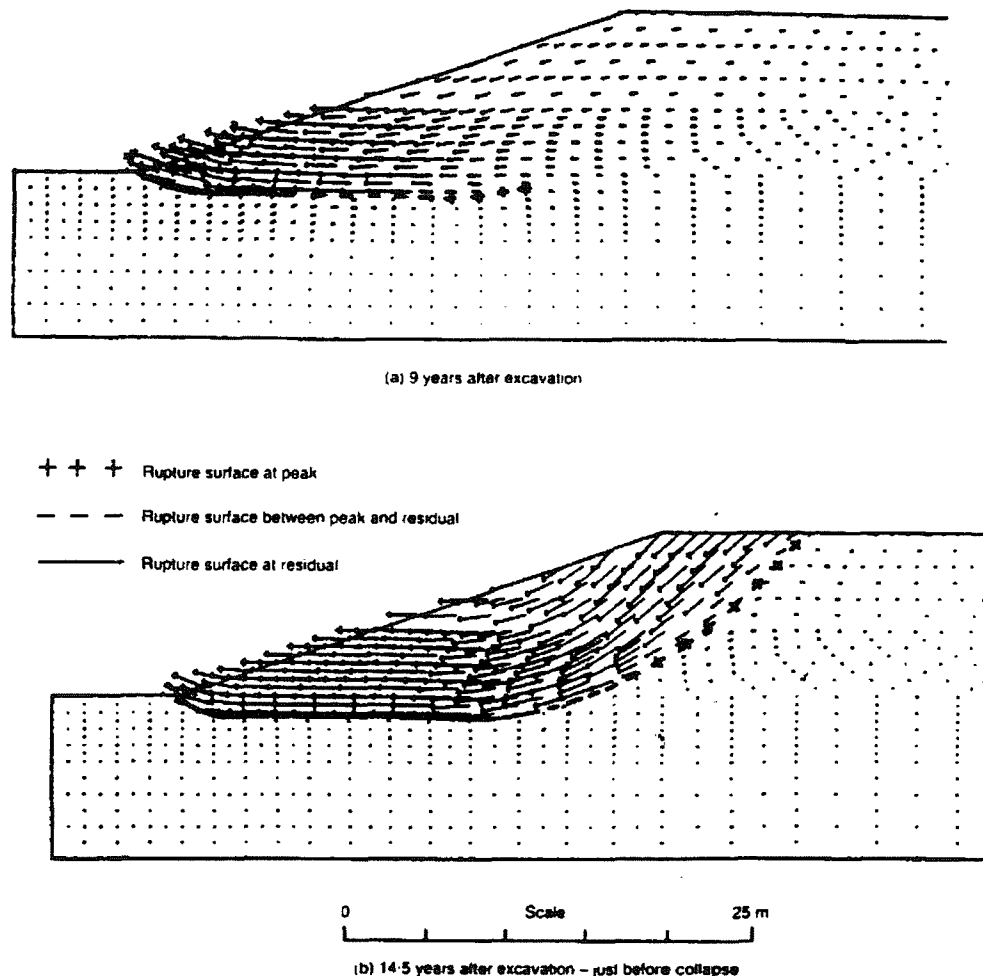


Figure 2.1. Results from a numerical analysis of progressive failure in a London Clay cut slope (from Potts *et al*, 1997). The analyses show how the failure surface starts at the toe of the slope and works progressively inward.

2.2.4 Serviceability failure in earthworks

Shallow translational slides have been widely reported in highway embankments made of overconsolidated clay (for example Crabb *et al*, 1987; Perry, 1989; Atkinson and Farrar, 1985). Most shallow slides are thought to be triggered by infiltration of rainfall into the slope surface (Fourie, 1996). During the winter months increased rainfall and reduced evapotranspiration from surface vegetation means that the clay at the surface of the slope has an increased water content and higher pore pressures. The UK summer can bring extremely dry periods, during which clay cracks to depths of up to 2m can form in the slope surface (Greenwood, 1998; Anderson and Kneale, 1980). These allow a rapid infiltration of water during heavy rainfall. Subsequently, a critical time for slope failure is when a long dry period is followed by heavy rainfall.

Surface failures may not prevent the use of areas above or below a slope, but can cause considerable inconvenience and expense when repairs have to be made. Slipped material often causes damage to drains and fences and is unsightly. Many motorway slopes prone to shallow failures may now be reaching a critical age for deep failure to occur (Carder and Temporal, 2000).

In clays that have poor volume stability, the change in moisture content with the seasons causes shrinkage and heave of the slope surface. The demand for water during the summer months from deciduous trees and shrubs growing on the slope surface exacerbates the magnitude of the clay shrink/heave cycle. Seasonal displacements of several tens of millimetres have been recorded in railway embankments (Andrei, 1999; Greenwood, 1998). Deformations can cause localised displacement of the rail level and aggravate slope failures.

Granular materials, particularly ash and ballast placed at the top of rail embankments can suffer ravelling, which is a loosely defined term for the downslope movement of individual particles. Downslope particle movement may be triggered by shrink/heave displacements and dynamic vibrations from passing trains (Perry *et al*, 2001). The movement of the granular material from the crest often causes cables and services to be undermined.

Some of the failure mechanisms and signs of failure typical to embankments are illustrated in Figure 2.2.

2.2.5 Natural slopes

In many parts of the country there is geomorphological evidence for extensive areas of relict landsliding, caused by valley rebound and glacial phenomena such as solifluction. Historic

landsliding activity, particularly in clay slopes, leaves relict failure surfaces along which soil shear strengths are close to residual. Many slopes are metastable and small changes within the slope can cause movements to restart. Common triggers for the re-occurrence of instabilities include periods of exceptional rainfall raising pore pressures on the failure surface, sea or river erosion of the toe of the slope (Guest *et al*, 2000) or excavations for construction works (New Civil Engineer, 2002). The failure of earthworks built on sidelong ground often occurs due to the reactivation of all or part of an existing *in situ* landslide (Chandler *et al*, 1973). Other factors also impact on the stability of natural slopes, such as a progressive weathering causing a change in material properties. Bromhead (1992) gives case histories for a large number of natural slope failures.

The size and extent of natural slope failures varies widely, and many large landslides affect built up areas, often encompassing property as well as road and rail infrastructure. While some very large landslide complexes (such as Ventnor, on the Isle of Wight) are too large to be stabilised with piles, other smaller, localised slope failures can be readily stabilised (for example, Allison *et al*, 1991; New Civil Engineer, 2002).

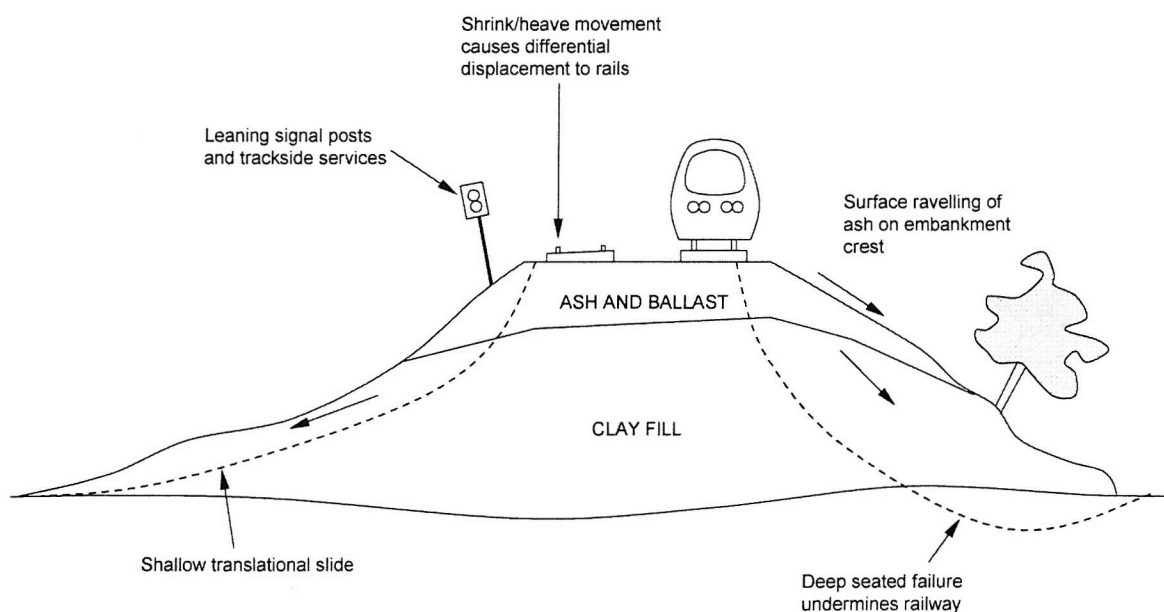


Figure 2.2. Failure mechanisms within embankments (after McGinnity *et al*, 1998).

2.3 Calculating the force acting on the pile head from the failing soil mass

2.3.1 The restoring force

The insertion of piles into the slope is designed to apply a force to the failing mass to reduce or stop slope movement. Such a force may be termed the 'restoring force'. As all forces are equal and opposite, it is also the force that the failing soil applies to the pile head, and which the piles must be designed to resist. The restoring force and its distribution over the top of the pile is a

function of a large number of variables, including slope geometry, slope displacements, soil strength, pile and soil stiffness and the pile spacing.

The presence of piles within the slope will limit the slope displacements that can occur. Initially, the pressure acting on the pile will increase with slope displacement. However, the increased pile-soil pressure will gradually inhibit further movement of the slope until an equilibrium point is found where the pile soil pressure is sufficient to stop slope movement. This equilibrium pile-soil pressure cannot exceed the ultimate lateral pile-soil pressure (taken from laterally loaded foundation pile theory, for example, Broms 1964).

In some cases, a good estimation of pile head load and its distribution on the pile is only likely to be obtained by using a 3D finite element model of the slope, in which 3D effects such as soil arching and full pile and slope equilibrium can be assessed. Such an analysis can also incorporate an accurate assessment the factor of safety of the pile stabilised slope. Three dimensional finite element models are discussed in Section 2.6. In practice, 3D finite element models are too costly and the parameters needed are rarely known with sufficient confidence, and simpler limit equilibrium and elastic stiffness models are used to estimate the restoring force. These methods are discussed in the remainder of Section 2.3.

2.3.2 Position of the piles within the slope

The piles are usually positioned in the slope where they are best able to resist the movement of any actual or potential failing masses of soil. The optimal location for the piles is dictated by the slope geometry, the slope failure mechanism, and the location of the infrastructure to be protected. The location of the piles will often affect the magnitude and distribution of the force that the failing mass applies to the piles.

Embankment stabilisation schemes carried out by London Underground Ltd have used piles placed near to the crest of the embankment slope, to provide direct support to the railway on top of the embankment (Andrei, 2000). Placement of the piles at the crest does not however preclude slope failure occurring downslope of the piles, although any serious failure will be reduced in size and would not damage the railway infrastructure. In cases where piles are constructed at the crest of the slope, additional stabilising methods are often used to prevent failure downslope of the piles (Andrei, 2000).

Positioning of the row of piles close to the toe of the slope is often adopted in the case of landslides, where placement of the piles at the toe ensures that the whole sliding mass is stabilised (for example, Carder *et al*, 2003). In steeper slopes, the largest stresses and shear strains in the slope material are at the toe, even if the slope is stable, illustrated by the tendency

for progressive failure surfaces to develop from the toe of the slope (Potts *et al*, 1997). Placement of the piles close to the toe to prevent shear does not prevent failure of the slope above the row of piles. While both the crest and toe arguments are valid, it is likely that the best position for the piles in steep, short slopes is around midslope, both where the circular rupture surface of the unsupported slope is deepest and where the row of piles reduces the size of any failure occurring above or below the pile row. Cai and Ugai (2001) demonstrate using a three dimensional finite element model of a 10m high 1v:1.5h slope that the largest factor of safety is found for the slope when the piles are placed at midslope (Figure 2.3). In long or tall slopes, it is possible that more than one row of piles will be required to prevent failure either above or below the piles causing damage to infrastructure either at the crest or toe of the slope.

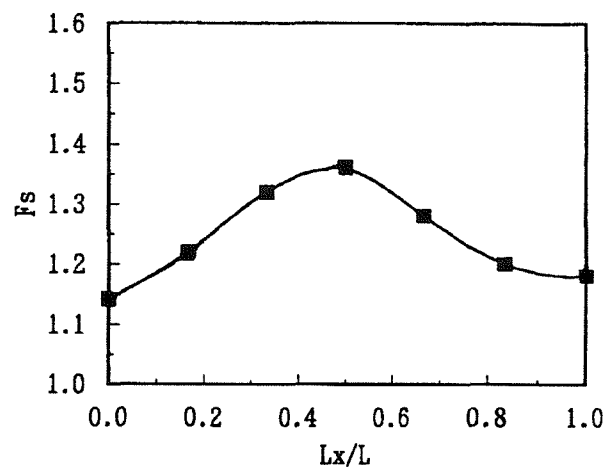


Figure 2.3. Factor of safety against the location of the pile in the slope, from a three-dimensional finite element model of a pile stabilised slope (Cai and Ugai, 2001). L is the length of the slope and Lx the distance from the toe of the slope to the pile. F_s is the factor of safety against any significant failure taking place in the slope. The soil strength parameters used throughout the slope were $\phi = 20^\circ$ and $c' = 10\text{kPa}$.

2.3.3 The limiting lateral pile-soil pressure

A maximum pressure that the displacing soil mass can exert on the pile is given by the lateral pile-soil capacity p_u , which is the stress at which the soil starts to fail plastically around the pile. This value is simply likely to be an upper limit, and takes no account of the stability of the slope.

Randolph and Houlsby (1984) propose an analytical solution for the undrained flow of soil around a pile. The solution is exact for the rigid plastic soil model, and allows for the friction between the pile and soil to be expressed as a proportion of the soil undrained shear strength c_u . The limiting pressure $p_u = N_p c_u d$ (per unit length of pile) is found to vary between $N_p = 9.14$ for a perfectly smooth pile and $N_p = 11.9$ for a perfectly rough pile (where d is the pile diameter). The solution assumes that the soil remains in contact with the pile, and that no gap occurs between them (Bransby and Springman, 1999).

The distributions of p_u with depth commonly used are based on empirical results from laterally loaded pile tests. The most commonly used distributions for p_u with depth are those presented by Broms (1964a, 1964b) and Fleming *et al* (1985), for both drained and undrained soil conditions. The recommended values for p_u are commonly reduced over the top length of the pile because of the tendency for a gap to occur at the ground surface in active loading.

For undrained conditions, the distributions of p_u with depth proposed by Broms (1964a) and Fleming *et al* (1985) are given in equations 2.1 and 2.2 respectively:

$$p_u = 0 \quad \text{at depths } z < 1.5 d, \quad p_u = 9 c_u d \quad \text{at depths } z > 1.5 d \quad \text{Equation 2.1}$$

$$p_u = 2 c_u d \quad \text{at depths } z < 3 d, \quad p_u = 9 c_u d \quad \text{at depths } z > 3 d \quad \text{Equation 2.2}$$

where d is the pile diameter and c_u the undrained shear strength. In slopes, the long term condition is probably more appropriate for design and the effective stress distributions given by Broms (1964b) and Fleming *et al* (1985) are given in equations 2.3 and 2.4 respectively:

$$p_u = 3 K_p \sigma'_v d \quad \text{Equation 2.3}$$

$$p_u = K_p \sigma'_v d \quad \text{at depths } z < 1.5 d, \quad p_u = K_p^2 \sigma'_v d \quad \text{at depths } z > 1.5 d \quad \text{Equation 2.4}$$

Chen and Poulos (1997) point out that the value of p_u must reduce if the piles are loaded downslope or towards an excavation. They propose a very approximate means of calculating the reduced p_u , which is probably only applicable in cases where piles are placed at the top of the slope:

$$p_u = \frac{2x}{d} c_u (\max 9c_u) \quad \text{Equation 2.5}$$

where x is the distance between the pile centre and edge of the slope (or excavation), and d is the pile diameter.

When the piles are close enough to interact (i.e. soil arching can occur), p_u will vary with pile spacing, but the magnitude of variations depends on soil strength and type. Little difference in ultimate load is usually observed in undrained (c_u) conditions over a range of pile spacings (Bransby and Springman, 1999; Chen and Martin, 2002). However, in effective stress terms, a greater variation in p_u with spacing is apparent (Chen and Poulos, 1997; Chen and Martin, 2002; Liang and Zeng, 2002). Chen and Poulos (1997) give the results from model tests on passively

loaded piles in sand, and show that for piles placed in a row the ultimate lateral pressure is reduced as the pile spacing decreases (a 30% reduction in N_p for piles placed at a spacing of 2.5 diameters, when compared to a single pile). Chen and Martin (2002) and Liang and Zeng (2002) ran 2D finite element analyses of passive soil loading of pile rows, and found a similar effect, with N_p substantially reduced at smaller pile spacings.

Viggiani (1981) and De Beer (1977) both suggest that lower values of lateral pressure coefficient N_p should apply to the piles in the failing soil mass, with the full values of N_p (as above) applying below the shear surface. They suggest a range of values ($N_p = 2.8 - 4$) for the moving soil mass that are less than half the values of N_p given above. However, other than for the near surface effects (such as the opening of a gap) there appears to be no reason why these values should be reduced above the shear surface (Poulos, 1999).

The limiting pile-soil pressure is used in a number of simple limit equilibrium calculations for stabilising piles (see Section 2.5.3). The limiting pile-soil pressure represents the point of plastic failure of the soil around the pile, and is unlikely to be reached in any pile stabilisation scheme that acts successfully to stabilise the slope.

2.3.4 Active earth pressure

Use of piles in a preventive role has been described by Poulos (1995) and investigated by means of finite element analyses by Carder and Easton (2001). Installation of piles to act in a preventive role could be into slopes required to be steepened or slopes thought likely to suffer failure at some point in the future (due to weathering of slope materials or an increase in pore pressure).

In designs for preventive pile-slope stabilisation (where the piles are installed in a slope already having a factor of safety larger than unity, and no pre-existing shear surface) it may be assumed that the displacements of the now pile stabilised slope will not be sufficient to generate loads close to the passive or limiting pile soil pressure condition. This is not unreasonable, as if sufficiently well fixed, the piles should prevent formation of a failure surface over which the upper soil mass can readily slide. Indeed, if the slope were to be re-profiled using a cantilever retaining wall, its design would probably proceed using active pressures on the upslope side, and passive pressures on the downslope side of the wall (derived from the earth pressure coefficients K_a and K_p respectively). In cases where the piles are spaced sufficiently close together to ensure that they can be designed using plane strain conditions (i.e. arching phenomena are less important), the active state could be used to calculate the pressure acting behind the piles.

Complete design for the stability of the piles could be based on active and passive pressure distributions running the length of the pile. To ensure that the design is not overly conservative, the active and passive pressure distributions must take into account the exact ground profile in front of and behind the wall. Daly and Powrie (2001) propose a numerical method using multiple Coulomb wedges to calculate the pressure from any slope surface geometry.

2.3.5 The required restoring force obtained from limit equilibrium slope stability

The active pressure and the limiting soil-pile pressure are likely to be the extreme pile loading conditions. In reality, if the piles are to stabilise the slope, then they must provide a force of sufficient magnitude to restore equilibrium.

The force required to restore equilibrium to the slope can be calculated using limit equilibrium slope stability analysis. In cases of an existing slope failure, the critical surface for the analysis can be determined from field measurements. In other cases where early signs of failure are evident, speculative analyses can be run to find the potential failure surface with the lowest factor of safety. The affect of inserting piles into the slope is modelled by introducing a point force at the proposed location of the row of piles into the critical failure mechanism for the unreinforced slope (some limit equilibrium slope stability packages allow this, including SLOPE/W (GEO-SLOPE, 1998)). The magnitude of this force is then increased until the desired factor of safety against slope failure is obtained. While the point force is applied in the analysis as the force that the piles apply to the failing mass, it is of course equal and opposite, and also the force that the failing mass will apply to the head of the piles (Figure 2.4). The point force can then be redistributed as a pressure over the length of the pile above the critical failure surface (Figure 2.4 assumes a triangular distribution).

The point force should be inserted into the limit equilibrium analysis at the point in the failing mass consistent with the assumed pressure distribution, e.g. for a triangular pressure distribution the point force should be placed at two-thirds the depth of the failing mass.

The point force will often be increased sufficiently to give a factor of safety for slope stability of about 1.25 – this will allow for any future reductions in soil strength (or increases in pore pressure) after the piles are installed in the slope that will result in further pile loading. Limit equilibrium slope stability is used to assess the restoring force in the design methods for discrete piles described by Hassiotis *et al* (1997) and Poulos (1995).

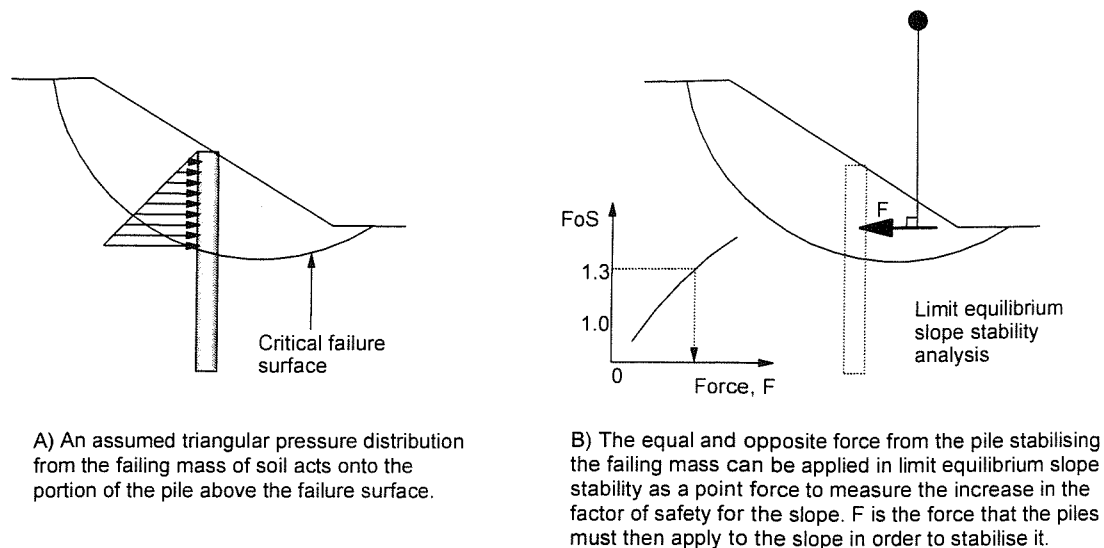


Figure 2.4. Insertion of point force into limit equilibrium slope stability analysis to calculate increased factor of safety from insertion of the pile into the slope (based on Poulos, 1995).

If limit equilibrium slope stability is being used to obtain speculative failure surfaces, Hassiotis *et al* (1997) point out that the addition of a force from the row of stabilising piles will alter the position of the critical failure surface, and they propose a modified version of the friction circle method (Taylor, 1937). Hassiotis *et al* (1997) derive new equations of slope stability to include the force F_p that the pile is exerting onto the failing mass. Every time that a new rupture surface is selected, the length of the pile above the failure surface changes, and the force F_p has to be recalculated. The limiting pile-soil pressure distribution used to obtain the force F_p was calculated using the method presented by Ito and Matsui (1975). As in standard limit equilibrium slope stability, the rupture surface with the lowest factor of safety is found.

The modified critical surface is found to be consistently shallower than the critical surface for the unreinforced slope (Hassiotis *et al*, 1997). This is because the load from the piles is entered into the analysis as an almost uniform distribution (down the length of the pile) and subsequently if the failure surface is shallower, less load from the pile acts on the failing mass and a smaller factor of safety is found. Hence this gives a force F_p and factor of safety for the modified surface that is smaller than the equivalent values obtained when using the critical surface from the unreinforced slope (Hassiotis *et al*, 1997). Presumably, if the pile pressure is large enough, the limit equilibrium slope stability will find a critical failure surface that is just a slither of the slope, and small value for the force F_p . This may not be a realistic model of the impact that the piles have on the slope – in practice they are providing additional stabilising support beneath the new critical surface that prevents a deeper failure from forming, and this is no longer accounted for in the value of pile loading F_p calculated. Hassiotis *et al* (1997) also seem to force all their failure surfaces through the pile row, when presumably given a sufficiently large pile force in the analysis, the critical failure will ultimately form upslope or

downslope of the row of piles. The assertion by Hassiotis *et al* (1997) that in slopes without a pre-existing failure surface the piles alter the critical failure mechanism is probably correct – however in such cases care must be taken when trying to use limit equilibrium slope stability to calculate the pile restoring force.

The limit equilibrium slope stability method is two dimensional, and can be assumed to represent a slice of the slope 1m thick. The load on each pile is normally then calculated by factoring the limit equilibrium slope stability design load to incorporate the pile spacing. The piles are often assumed to be sufficiently close together that the soil arches between them, and no flow of soil between the piles can occur. Limit equilibrium slope stability is the most commonly used means of calculating the pile head loading, because fundamentally the restoring force provided by the piles must be more than sufficient to provide stability to the slope.

2.3.6 Calculation of the restoring force using elastic methods

Numerical methods which link slope displacements to the restoring force offered by the pile have been widely used (Poulos, 1995; Hassiotis *et al*, 1997; Poulos, 1999). They are based on modelling the pile as an elastic beam either embedded in an elastic continuum or supported by a series of empirically derived non-linear springs or p-y curves (as in Figure 2.5). Such methods assume the soil to be displacing into the pile, and they allow calculation of the restoring force at any stage of soil movement. The more complex of these numerical methods, often called boundary element or pseudo-finite element analysis (Poulos, 1999; Gaba *et al*, 2003), take into account the lateral limiting pile-soil pressure allowing localised failure and a redistribution of soil pressure onto adjacent sections of the pile. A pile of finite length can be modelled, and the overall pile stability taken into account (more details are given in Section 2.5.5).

The accuracy of such solutions is dependent on good characterisation of the pile-soil interaction and measurement or prediction of slope displacements, which are also required for input into analyses. Analysis of a row of piles either ignores the interaction effects of adjacent piles, assuming that the piles are spaced sufficiently far apart, or uses empirical interaction coefficients to adjust the pile behaviour.

The limitation of the boundary element and elastic continuum methods for calculating the restoring force is that slope displacements adjacent to the pile have to be predicted. The need to predict slope displacements can be removed by gradually increasing the soil displacements until the ultimate pile-soil pressure is found and the design can proceed around this condition, although this is likely to lead to a conservative pile design. Careful analysis of the pressures generated on the head of the pile by the soil movement could also be compared with the force required to stabilise the slope obtained from limit equilibrium slope stability (Poulos, 1999).

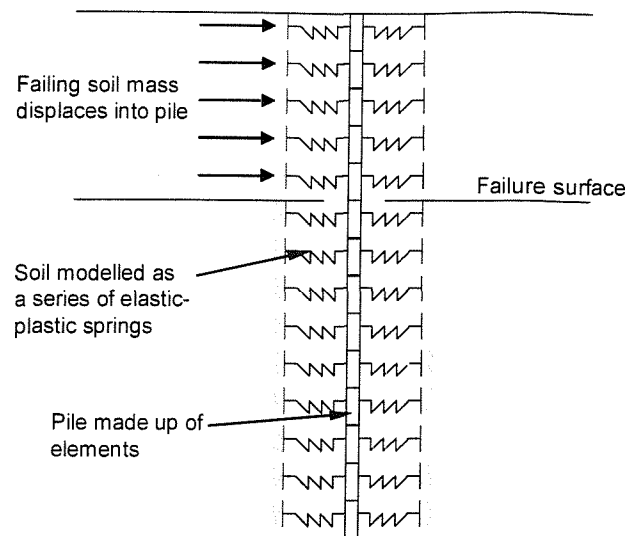


Figure 2.5. A laterally loaded pile model, representing the soil as a series of non linear springs (after Fleming et al, 1985)

2.4 Pile spacing and soil arching

2.4.1 Pile spacing

The spacing of the piles is chosen on two main criteria. The first of these is the limit on maximum pile spacing. If the piles are spaced too far apart, soil arching will be ineffective and localised failure with a flow of soil between the piles is likely to occur. In practice, this maximum pile spacing is difficult to assess, and is likely to depend on a large range of variables including soil strength and stiffness, type of mass movement and the pile stiffness and diameter. The soil arching mechanism and its application to pile stabilised slopes is discussed further in Sections 2.4.3 – 2.4.10.

The second criterion is that the row of piles must provide sufficient overall bending resistance, and the spacing will therefore depend on the restoring force required to stabilise the slope and the bending capacity of the piles to be installed. In cases where the bending resistance that the piles must provide (per m run of the slope) is large, the piles will often be spaced close together (almost as a wall, for example, De Beer and Wallays, 1970) and soil arching effects become less important.

2.4.2 Pile diameter

The pile diameter used will impact upon the bending capacity of the piles and the dimensionless pile spacing adopted. However, the pile diameter is often dictated by construction considerations, in particular the site access. Many rail and canal embankments have poor site access and this often dictates the size of the piling rig that can be used. Advances in piling

technology mean that minipiles of up to 300mm in diameter can be constructed with fairly compact and lightweight drilling rigs of less than 5 tonnes. The term bored pile typically refers to piles of greater than 300mm (Perry *et al*, 2001) and these can be constructed using tripod winches or hydraulically powered tracked machines. Most piling rigs normally require a horizontal platform on which to operate, and this is normally constructed by reprofiling the slope or by constructing a berm from imported materials (Figure 2.6). Scaffold platforms have been used on some steep slopes (Perry *et al*, 2001). Examples in the literature (Table 2.3, Section 2.7) cite pile diameters between 300 and 3000mm.



Figure 2.6. Rockfill berm constructed at the toe of a railway embankment at Hildenborough, Kent, to provide a platform from which to operate piling plant.

2.4.3 Soil arching

Soil arching is the mechanism by which soil stresses from a yielding mass of soil are transferred onto adjacent stationary or stiffer zones. It commonly occurs in geotechnical engineering where moving or settling soil is adjacent to fixed or stationary structures or masses of soil, and is often associated with pipes, tunnels, slopes, pile walls, underpinning, fills and backfills (Kellogg, 1987). Arching was first investigated by Terzaghi (1943), who studied a mass of soil above a yielding trapdoor, and observed that as the door displaced downward, the soil pressure transferred away from the yielding door onto the adjacent stationary frame. Terzaghi (1943) explained this by the development of shearing stresses between the yielding soil mass and that resting on the stationary frame trying to resist its downward movement.

An understanding of the soil arching phenomenon is important to the design of discrete piles. It dictates the maximum pile spacing that can be achieved before failure occurs adjacent to the piles and the soil is able to move between them, giving unacceptable slope displacements. Initially as the pile spacing increases, and the piles continue to successfully stabilise the slope, the piles will carry a load proportional to the pile spacing. As the piles become widely spaced, the arching mechanism starts to break down, and the load carried by an individual pile may well reduce as the soil fails between the piles (Bosscher and Gray, 1986).

Recent physical and numerical model studies have demonstrated that soil arching occurs between passively loaded discrete piles used to stabilise a failing slope (Bosscher and Gray, 1986; Chen and Martin, 2002; Liang and Zeng, 2002). The arching occurs due to the development of shear stresses in the soil resulting from the soil trying to move around and between the piles. The arching effectiveness is therefore significantly influenced by the soil strength (or limiting shear stress between individual soil particles) in turn affected by the soil type and soil particle size. The soil particle size in relation to the size of the piles will also influence the arching – soils with a large particle size (such as a rockfill) are likely arch more effectively than soils with a small particle size.

Wang and Yen (1974) developed an analytical solution based on infinite slope stability and Terzaghi's arching theory to calculate the transfer of load away from the soil between the piles and onto the piles in the slope at different pile spacings. Ito and Matsui (1975) also developed an analytical model to study pile loading at different pile spacings. Both methods are further discussed in Section 2.4.9.

Sections 2.4.5 – 2.4.8 review the results from some of the physical and numerical analyses of soil arching between piles, and the implications for discrete pile design.

2.4.4 Active and passive pile loading

The term 'passive pile loading' is used in this thesis to describe when the soil is pushing laterally onto a pile (as in discrete pile-slope stabilisation), whereas 'active pile loading' is used to describe situations where the piles are pushed into the soil (for example by loading to the pile head from a building superstructure). This terminology is commonly used to describe pile loading, where the terms 'active' and 'passive' are used to refer to the state of the soil in front of the pile (Bransby and Springman, 1999; Chen and Martin, 2002). In passive pile loading, the relative displacement between the free field soil and pile are used to calculate the p - δ curves (where δ = relative soil movement, p = additional stress acting on the pile) whereas in active pile loading, the full pile displacement is used to calculate p - y curves (where y = full pile displacement). p - δ curves (suitable for passive loading) are not therefore equivalent to p - y

curves (Bransby and Sringman, 1999; Chen and Martin, 2002). While for a given soil, the load at which the soil yields or fails around the pile is the same in both forms of loading, the $p-\delta$ curves are much stiffer (for example, Figure 2.7). Bransby (1996) demonstrates that, for a row of piles, $p-y$ curves may soften with reduced pile spacing even whilst the $p-\delta$ curves stiffen. There is also evidence that arching is more effective in passive loading (Chen and Martin, 2002).

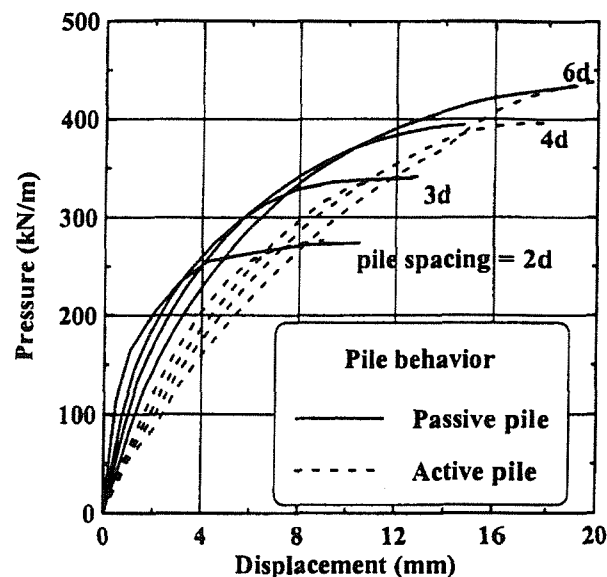


Figure 2.7. Active and passive loading for a row of piles at different pile spacings. The $p-\delta$ curves for passive pile loading can be seen to be stiffer than the $p-y$ curves in active loading (for a drained soil of strength $\phi = 30^\circ$) (taken from Chen and Martin, 2002). d is the pile spacing.

2.4.5 Studies of arch development

Handy (1985) studied soil arching within fill material settling between two rough walls. Sufficient vertical settlement of the fill was assumed to occur for its weight to become partly supported by wall friction. A consideration of the Mohr circle of stresses gives a rotation of the principal stresses at the shear boundary, and this leads to a downward arch defined by the direction of the minor principal stress between the shear boundaries. Handy (1985) calculates that such a supported arch (i.e. supported by the material below it so that it is not free standing) should take the form of a catenary.

An 'open arch' may be defined as an arch that is free standing, i.e., it will stand without the support of material directly beneath it. Hewlett and Randolph (1988) describe model tests in which a box was filled with damp sand. The sand at the base of the box was then allowed to fall away so that the remaining sand in the top of the box arched upward between ridges protruding from two sides of the box. The resulting 'open arches' were found to be approximately semi-circular in shape. Other open arch shapes have been observed but these are normally attributed to fracture geometry or the ability of rock materials to carry some tensile force (Kellogg, 1987).

Analysis of the open arches shows that static equilibrium requires the tangential stress is always the major principal stress (Hewlett and Randolph, 1988). There appear to be two categories of arching, and these are defined by Handy (Table 2.2, reproduced from Kellogg, 1987).

Major principal stress arching is defined by elastic behaviour, and the minor principal stress arching by plastic failure. The minor principal stress arching is therefore only relevant to the stress condition at the point of plastic flow (Wang and Yen, 1974; McKelvey, 1994). It seems certain that for arching in slopes, the initial arching behaviour will be elastic – when sufficient displacements have taken place to allow the formation of shear bands will only the arching behaviour take the plastic minor principal stress form. Table 2.2 defines major and minor principal stress arching. In contradiction to the definition made by Handy in Kellogg (1987), there is ample evidence from arching in slopes and soldier pile walls (Chen and Martin, 2002; Vermier *et al*, 2001) that the major principal stress arching does not have to be ‘open’ and will initially form in a concealed environment (although self supporting arches must always remain elastic). A combination of soil strength, elastic modulus and soil displacement will dictate when elastic arching breaks down into the plastic form (McKelvey, 1994). While design based on plastic, minor principal stress arching will be conservative, in cases where displacements between non-yielding supports are sufficiently constrained to ensure continued elastic arching (such as in soldier pile wall retaining a stiff soil, Vermier *et al*, 2001), design could probably proceed around this condition.

Property	Soil arching action	Fully supported arch or bridging
Continuous principal stress	Minor	Major
Behavioural mode	Plastic	Elastic
Shape	Downward catenary with spring angle (the angle between the arch and the shear boundary) determined by shear boundary friction	Variable, depending on soil and rock strength; may approach an upward semi-circle in homogenous soils
Architecture	Concealed	Open or concealed

Table 2.2. Minor and major principal stress arching mechanisms (modified from Kellogg, 1987).

Analyses run by Chen and Martin (2002) show that the elastic arching phase is particularly efficient within undrained soil slopes, with the effectiveness of the arching decreasing with further slope displacement as yield starts to occur (their results are discussed in more detail below). Caution should be given to relying on elastic arching within undrained soil slopes, due to the propensity for pore water pressures in the undrained state to dissipate (Liang and Zeng, 2002).

2.4.6 Physical studies of arching in slopes

Bosscher and Gray (1986) carried out a series of model experiments with a sand slope supported at the base by different sized yielding gates. The gates were allowed to pivot at their base point, as shown in Figure 2.8. Loose and dense samples of the same sand were used; the dense samples applied less load to the gates due to a higher degree of particle interlocking and strength mobilisation (Bosscher and Gray, 1986). Both the yielding and fixed gates were instrumented with load cells in order to assess the proportion of the residual slope load being transferred onto the fixed gates. Measurement of the load on the yielding gates showed that beyond a certain degree of rotation, the load acting on the gate drops away, as it is transferred to the adjacent fixed gates by soil arching (Bosscher and Gray, 1986).

The arching mechanism was most effective when the spacing between the fixed gates was small (two gate widths, centre-to-centre, or 50% of the total gates yielding), and the yielding gates carried a lower (30%) proportion of the overall residual slope loading (Figure 2.9). The figure also shows that as the fixed gate spacing is increased, the proportion of the total residual load on the yielding gates also increases (but this is to be expected as the percentage of the total gates now yielding is increased). Figure 2.10 shows the curve in Figure 2.9 re-plotted (as a best fit through the experimental data points) along with a second curve representing the proportion of the gates that are yielding (i.e. if arching did not occur, and the slope acted in plane strain, the proportion of load that the yielding gates should be carrying). The difference between these two curves therefore represents the proportion of overall slope loading to transfer away from the yielding gates, and this is plotted as a third curve in Figure 2.10. The curve shows a drop in the proportion of the load that is transferred through arching, indicating that arching is becoming less efficient with increased gate spacing. This does still equate to an initial increase in loading to individual fixed gates with gate spacing, as shown in the experimental results (Figure 2.11), as each gate is now supporting a wider section of the slope. However, the fixed gate loading seems to reach a peak at 4 gate widths centre-to-centre, after which it drops slightly because the proportion of load transferred from the yield gates to the fixed gates is getting very small as the efficiency of the arching deteriorates.

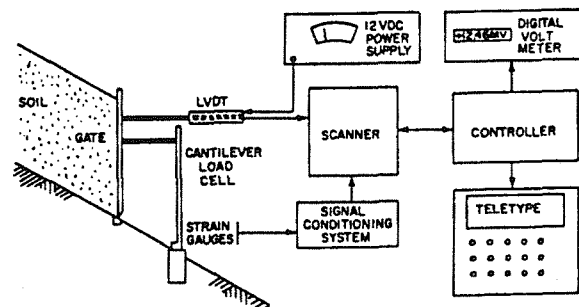


Figure 2.8. Cross section through the model set-up (from Bosscher and Gray, 1986).

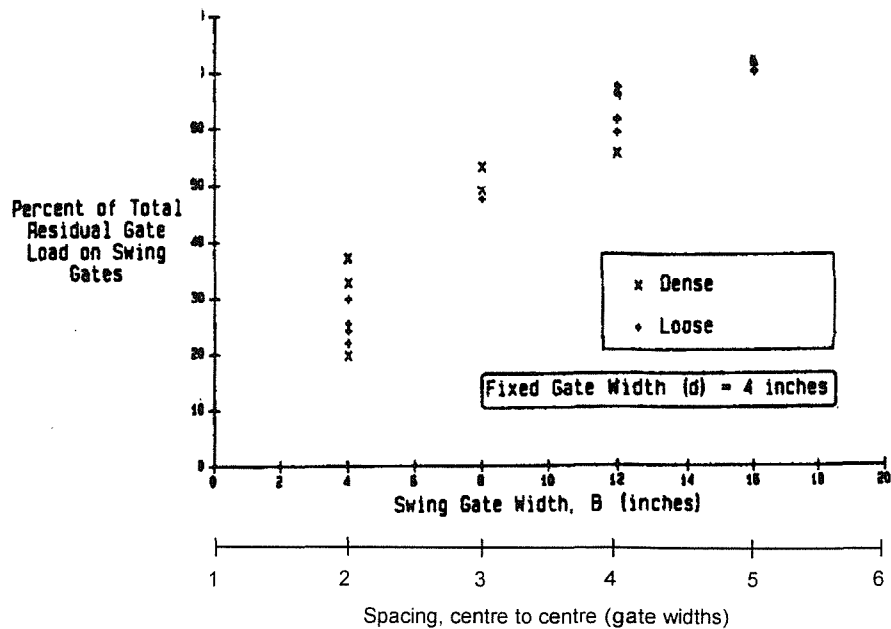


Figure 2.9. Graph of the percent of total residual load transferred to the swing gates against the swing gate width, for loose and dense sand slopes (from Bosscher and Gray, 1986)

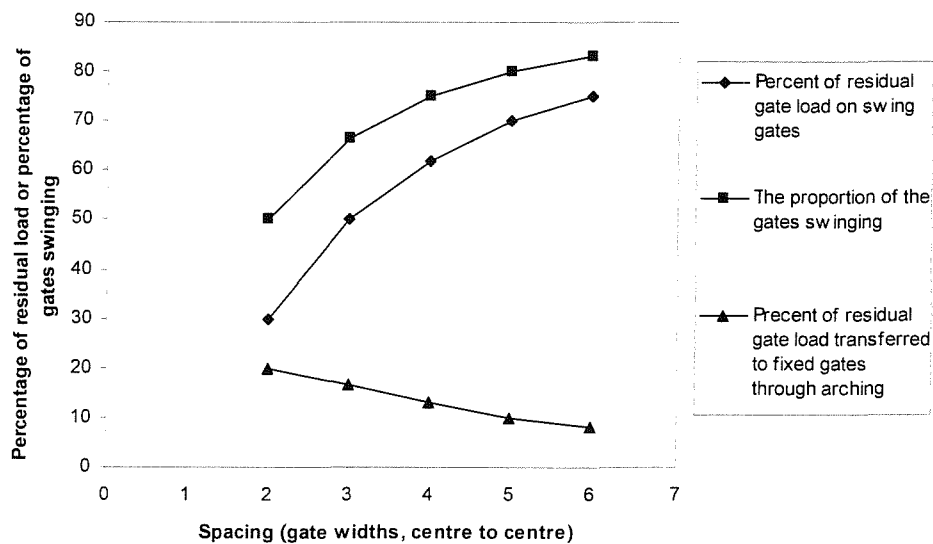


Figure 2.10. Graph of the percent of total residual load transferred to the swing gates, against the swing gate width (from Bosscher and Gray, 1986). Two further curves are plotted: i) a curve showing the percentage proportion of the gates that are yielding (or if the arching did not occur, and the slope acted in plane strain, the proportion of load that the yielding gates should be carrying); and ii) the proportion of residual load transferred to the fixed gates by soil arching (which is the difference between the first two curves).

The load displacement plots given by Bosscher and Gray (1986) show that the load transfers away from the yielding gates onto the fixed gates at relatively small gate rotations, and that further rotations make little difference to the gate loads. Figure 2.12 shows a photograph of the soil displacements behind the gates at the maximum gate displacement. The photographs clearly show an almost semicircular section of the sand slope having moved with the yielding gates, above which the sand arches onto the fixed gates.

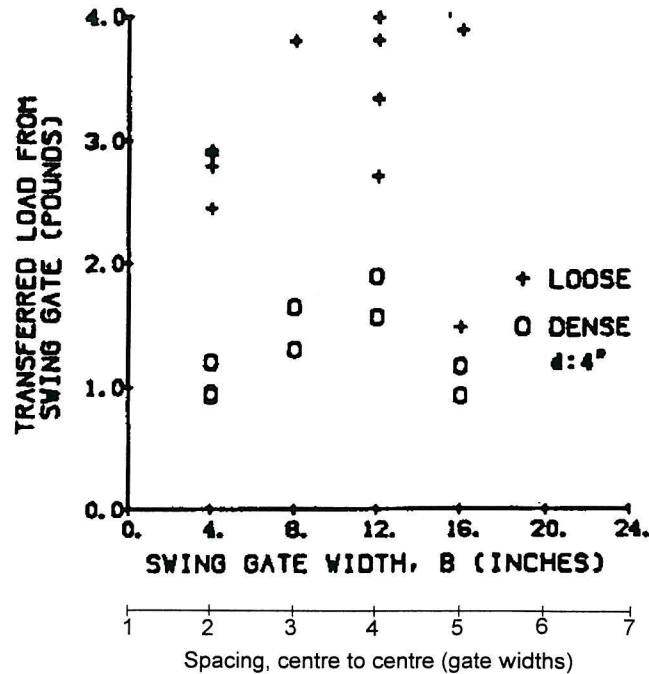


Figure 2.11. Graph of the load transferred from the swing gates onto the fixed gates, at different gate spacings (reproduced from Bosscher and Gray, 1986).

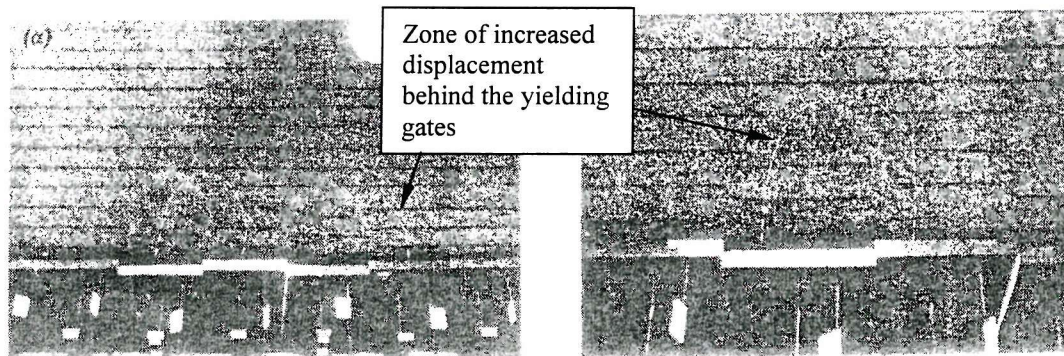


Figure 2.12. The displacement patterns in the sand behind the yielded gates (reproduced from Bosscher and Gray, 1986).

Adachi *et al* (1989) carried out a series of model tests to study the mechanism of soil arching. They used a series of fixed piles, strain gauged to measure load, and aligned horizontally in a vertical frame surrounded by a model soil made of aluminium rods (Figure 2.13). The aluminium rods were representative of a loose sandy soil having $\phi = 30^\circ$ (Adachi *et al*, 1989). The base plate of the model was then uniformly displaced downward, forcing the soil to move around the piles and form an arching mechanism above them. The displacement of some of the aluminium rods was tracked, to try to understand the arching mechanism. Adachi *et al* (1989) found that the arching initially took the form of an approximately equilateral triangular arch, defined by rod displacements of equal size to the base plate displacement below the arch (B, C and D in Figure 2.14), and much smaller displacements above this, where for most pile spacings the soil was supported by arching (A in Figure 2.14). Adachi *et al* (1989) found that the load

acting on the piles was equivalent to the weight of the area above the arch up to the soil surface (the vertically shaded section of Figure 2.14). The shape of the arching mechanism is similar to that in the model tests carried out by Bosscher and Gray (1986) (and shown in Figure 2.12). Tests at multiple pile spacings found that the load acting on the piles was larger at bigger pile spacings (Figure 2.15), and that load transfer onto the piles was about 15% more effective for square piles than for circular ones.

Adachi *et al* (1989) carried out 2D finite element analyses of the model tests, but the numerical model did not seem to replicate the model tests particularly well at large pile spacings (Adachi *et al*, 1989). The numerical model at smaller pile spacings did however show the development of an elastic arch, with a rotation of principal stresses above the pile. Further displacements within the numerical model seemed to cause plastic failure within the zones between and just above the piles, and this caused a slight drop in pile load (Adachi *et al*, 1989).

Hayward *et al* (2000) present the results from a series of centrifuge model tests on pile stabilised slopes. Tests were carried out on identical kaolin model slopes supported by piles at spacings of 3, 4 and 6 pile diameters (Figure 2.16). The model slope was designed to be unstable at the equilibrium pore pressure condition, and a test with no stabilising piles was run to confirm this. The model piles were instrumented with miniature strain gauges to measure bending moment, and slope displacements were tracked using a camera through the glass side of the model box and cameras mounted above the slope. The tests with piles at 3 and 4 diameters spacing successfully stabilised the slope. Shallow failure of the slope occurred with piles at 6 diameters spacing, although piles at this spacing appeared to prevent deep seated failure (Hayward *et al*, 2000). Analysis of displacements on the slope surface demonstrate the slope to be acting in plane strain, with little evidence of additional displacement of the soil between the piles.

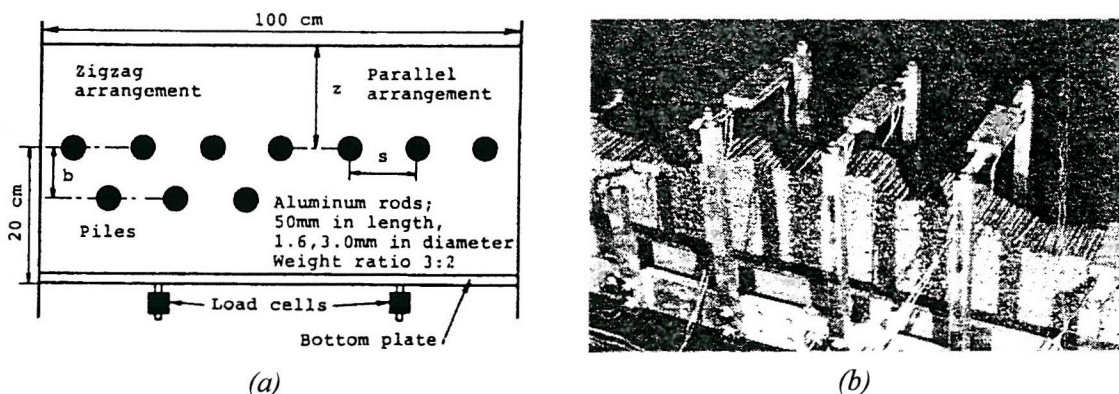


Figure 2.13. The model set-up: (a) a sketch of the experimental apparatus; and (b) a photograph showing the vertical frame with the exposed piles (rectangular in this case) mounted horizontally to measure vertical pressure, and where the rods representing the soil have not yet been built up to surround the piles (taken from Adachi *et al*, 1989).

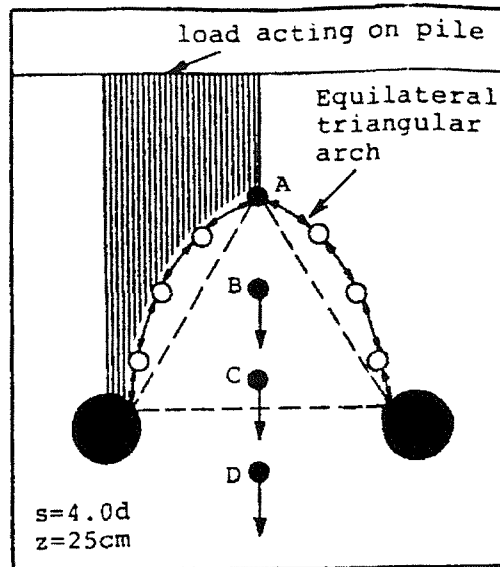


Figure 2.14. Arching effect observed in the model tests by Adachi *et al* (1989). s = the spacing of the piles, d = the diameter and z = the depth of the piles from the top surface of the aluminium rods. A, B, C and D are the initial locations of rods tracked during the tests.

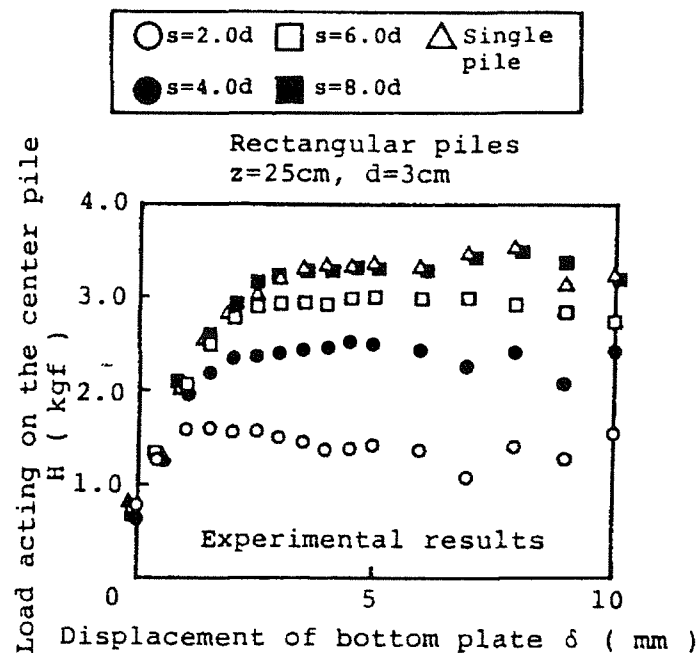


Figure 2.15. Load displacement curves for the piles at different spacings (taken from Adachi *et al*, 1989); s = the spacing of the piles, d = the diameter and z = the depth of the piles from the top surface of the aluminium rods.

The piles in all tests bent uniformly downslope, with approximately the top 2m of the pile apparently loaded by the downslope movement of the slope. Hayward *et al* (2000) determined the steady long term pressure acting on the head of the piles by differentiating the measured bending moments. The pile head load is plotted against pile spacing in Figure 2.17. The pressure was found to be much smaller than the Broms (1964a) and Fleming *et al* (1985) limiting lateral pressures, and smaller than the load calculated from a limit equilibrium slope stability analysis of the model slope. The piles appeared to have provided sufficient support to the slope to

prevent the soil from reaching the yield condition around the head of the piles. However, the piles in the centrifuge test were probably not fully fixed (due to their short length and the absence of a stiffer underlying stratum), and it is possible that the reduced pressure was due to pile movement/rotation (Hayward *et al*, 2000). Pile head movement and rotation was not measured in the tests.

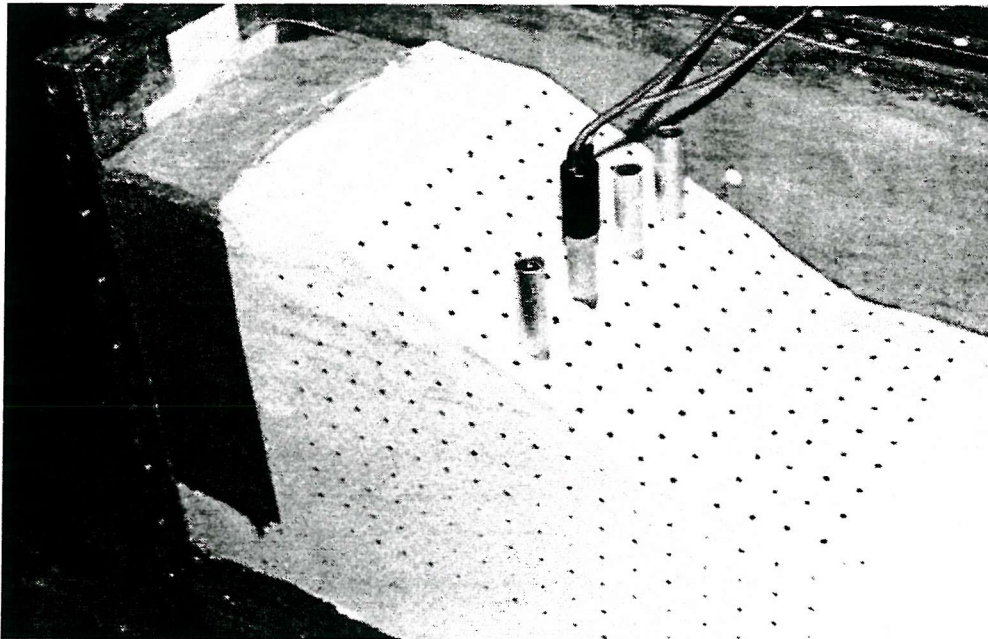


Figure 2.16. The model clay slope (from Hayward *et al*, 2000).

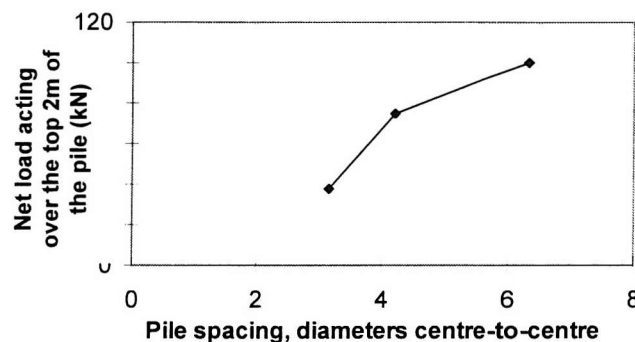


Figure 2.17. Plot of pile head load against pile spacing for the model piles in the centrifuge tests (the head load is assumed to be that acting on the top two metres of the pile, at prototype scale) (Hayward *et al*, 2000)

2.4.7 Numerical studies of arching in slopes

Liang and Zeng (2002) used the experiments by Bosscher and Gray (1986) and Adachi *et al* (1989) to verify a two dimensional finite element model of arching behaviour. They modelled a horizontal slice of soil of unit thickness through the pile-soil system (Figure 2.18). The model is two dimensional, and so assumes that soil strain is confined within one plane. This is probably not unreasonable, as vertical displacements are likely to be small in relation to horizontal displacements. Bransby and Springman (1996) carried out a 3-D finite element analysis of a pile group under passive lateral loading and found that the vertical strains were significantly lower

than those in the horizontal plane. Similar conclusions were drawn by Ellis (1997) and Chen and Poulos (1993) for piles in passive loading. However, it is uncertain whether this is true of a steep slope in which the piles are inserted vertically.

The finite element mesh used by Liang and Zeng (2002) modelled only the piles and soil upslope of the pile centreline (Figure 2.18). Interface elements were placed between the pile and soil. The passive pile loading was obtained by fixing the piles in the mesh and by applying a fixed displacement to the edge of the mesh representing the soil between the piles. Liang and Zeng (2002) show the mesh and system of displacements (Figure 2.18) that they used in order to model the physical tests carried out by Adachi *et al* (1989), in which the piles were horizontal within a vertical frame (Figure 2.13). Figure 2.19 shows the displacements and principal stresses for the analysis in which Liang and Zeng (2002) modelled the Adachi *et al* (1989) test at 2 pile spacings. This analysis modelled a vertical section through the soil-pile system, and hence the soil stresses increase with depth down the mesh. Liang and Zeng (2002) do not make clear for the later analyses and parametric study that they carried out (to represent landslide stabilising piles) whether they rotated the mesh to make it horizontal, or whether they continued to use a vertical section with increasing soil pressures with depth. Chen and Martin (2002) modelled a horizontal section with both the soil above and below the pile (their finite difference mesh is shown in Figure 2.20) and applied a fixed displacement to the boundaries AA and BB. While this is more rigorous, the system used by Liang and Zeng (2002) seems to produce sufficiently similar results to suggest that the basic arching mechanisms are the same. Liang and Zeng (2002) successfully replicated the model arching studies carried out by Bosscher *et al* (1986) using their finite element model.

The development of an arching mechanism in the analyses by Liang and Zeng (2002) is illustrated by the principal stress rotation obtained in the soil between and upslope of the piles (Figure 2.19). A clear arch is seen in the direction of the major stresses upslope of the piles, while directly between the piles the stresses remain small (illustrating the transfer of load away from the slope and onto the piles). The mode of arching is clearly by means of a major principal stress arch.

Limiting the model to two dimensions allowed Liang and Zeng (2002) to run a large number of analyses to investigate the effect on soil arching and load transfer from a number of parameters, including pile spacing, pile shape, pile diameter, and soil strength. They have tabulated the results from the analyses in a series of tables, one of which is shown in Figure 2.21. The table shows the proportion of overall residual load acting on the soil directly between the piles for a number of different soil strengths and pile spacings. Presumably, this proportion was calculated from the pressure acting on the boundary of the mesh between the piles and the pressure acting

on the piles themselves, although Liang and Zeng (2002) do not make this totally clear. Liang and Zeng (2002) also do not say whether the load redistribution figures they give (as in Figure 2.21) apply to the ultimate pile loading, the point at which the arching is most effective, or a fixed displacement. Figure 2.22 plots some of the data from Figure 2.21 and shows the effect of effective soil friction angle and pile spacing on the arching effect. As would be expected, arching was much more effective for the stronger soils, and for the smallest pile spacing. Any true cohesive strength in the soils was also found to have a significant effect on the arching mechanism, with high values of c' giving little reduction in the amount of load transfer on to the piles with increased pile spacing.

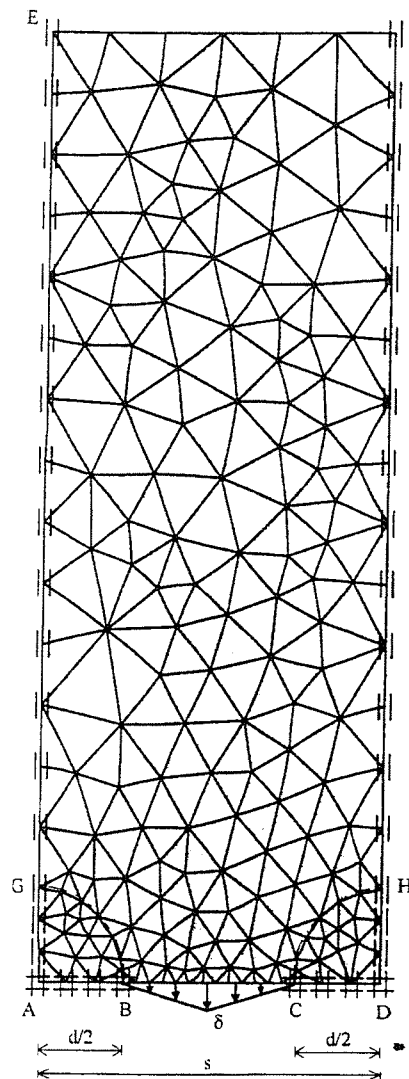


Figure 2.18. The 2-D finite element model (from Liang and Zeng, 2002)

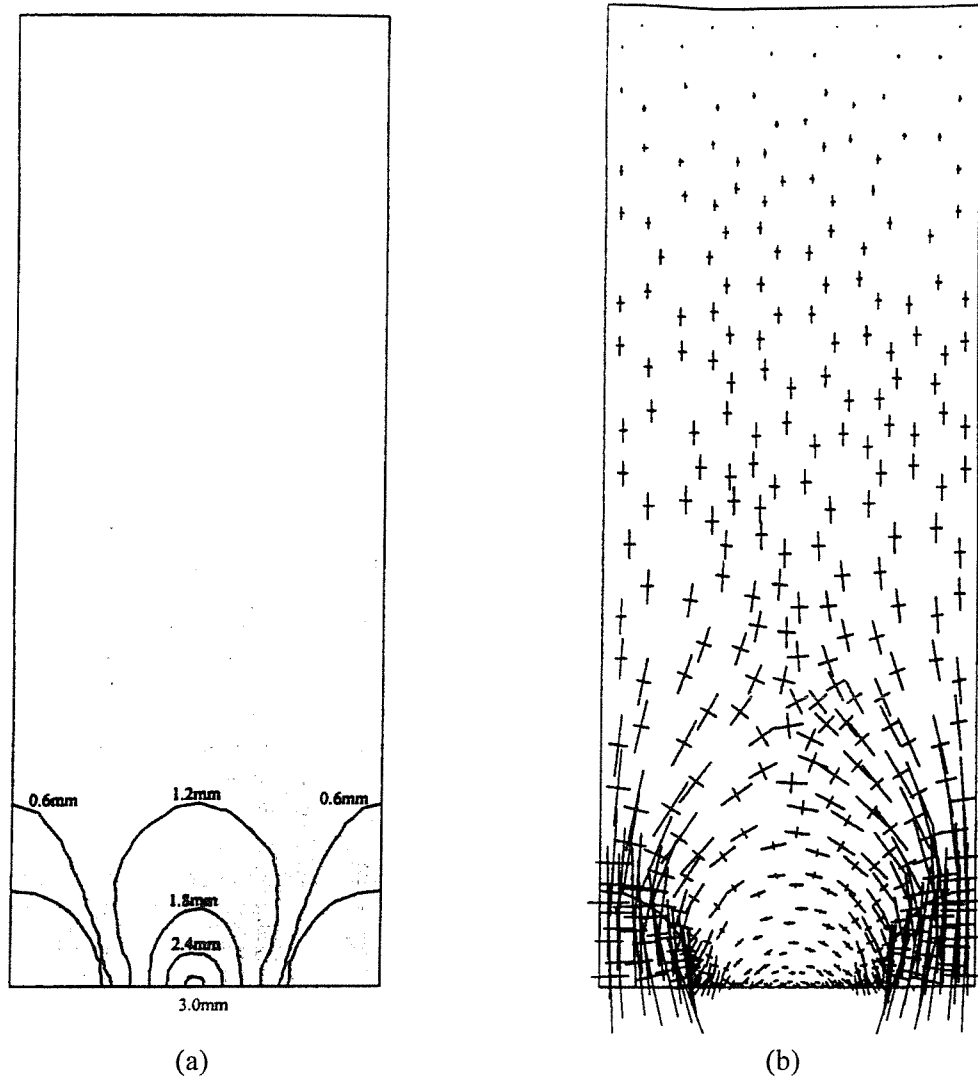


Figure 2.19. (a) contours of the downslope displacement of soil around the piles; and (b) the principal stress rotation around the piles, for a pile spacing of 2 diameters centre to centre and soil strength $\phi = 30^\circ$ (from Liang and Zeng, 2002).

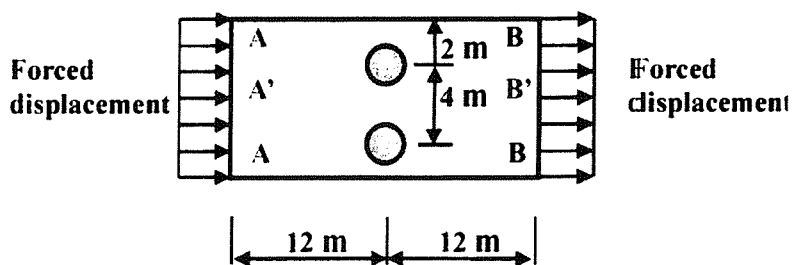


Figure 2.20. Finite difference mesh and boundaries used by Chen and Martin (2002). The piles are fixed while the soil is pushed around them using fixed displacements on the upper and lower boundaries.

		$c=0$ (kPa)	$c=6.9$ (kPa)	$c=13.8$ (kPa)	$c=27.6$ (kPa)	$c=41.4$ (kPa)
$\phi=0^\circ$	$s/d=2$	100.00	39.10	10.64	10.47	10.46
	$s/d=3$	100.00	61.55	21.66	14.21	13.44
	$s/d=4$	100.00	71.73	39.21	21.02	15.35
$\phi=10^\circ$	$s/d=2$	64.09	11.25	10.44	10.36	10.36
	$s/d=3$	76.72	37.71	14.27	14.11	13.41
	$s/d=4$	81.26	54.85	20.61	15.32	15.01
$\phi=20^\circ$	$s/d=2$	34.92	10.83	10.68	10.31	10.31
	$s/d=3$	56.86	21.77	14.07	13.93	13.37
	$s/d=4$	64.72	40.72	14.88	14.21	13.35
$\phi=30^\circ$	$s/d=2$	16.48	10.56	10.50	10.44	10.35
	$s/d=3$	47.71	15.03	14.06	13.91	13.34
	$s/d=4$	59.29	27.69	14.81	14.14	13.33
$\phi=40^\circ$	$s/d=2$	16.32	10.47	10.41	10.34	10.31
	$s/d=3$	37.50	15.79	14.03	13.89	13.31
	$s/d=4$	54.51	29.32	14.74	14.02	13.27

Figure 2.21. Table of results from finite element analyses of a passively loaded row of piles (taken from Liang and Zeng, 2002). The values in the table are the percentage proportion of residual load carried by the soil between two piles. Low values indicate a high transfer of load to the piles by soil arching.

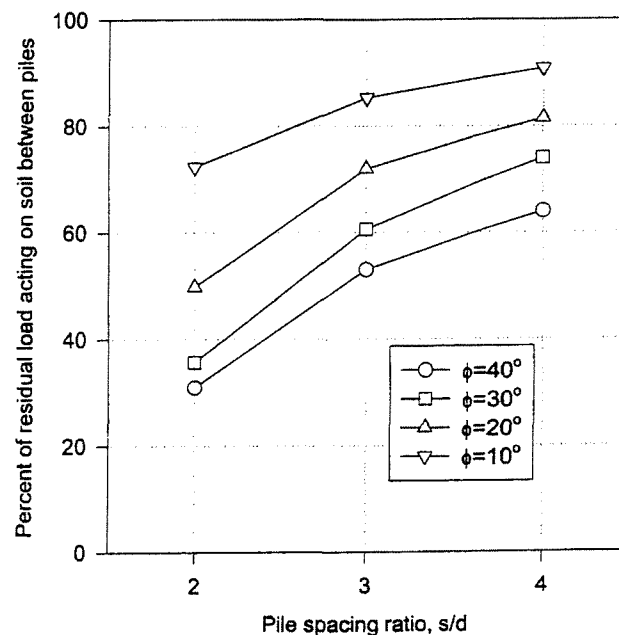


Figure 2.22. A plot of residual load acting on the soil between the piles against pile spacing, for drained soils of different frictional strength (taken from Liang and Zeng, 2002).

Liang and Zeng (2002) show that the effectiveness of the arching is also dependent on the pile diameter, with the smallest diameter pile modelled (0.3m) having about a 15% increase in the transfer from the soil to pile when compared with the largest diameter pile modelled (0.9m) for the same dimensionless pile spacing. Transfer is also increased if the piles used are square, as opposed to circular, as the square piles offer a better support for the arching foothold (Liang and Zeng, 2002).

Chen and Martin (2002) also carried out finite difference analyses to understand the arching mechanism occurring in the soil between spaced piles, and how it develops with increased soil displacement. They ran an analysis where a horizontal slice through the pile-soil system was modelled (a schematic of the finite difference mesh is shown in Figure 2.20). Chen and Martin (2002) modelled both the soil above and below the pile, and applied a fixed displacement to the end boundaries AA and BB, while the piles remained fixed in position.

Chen and Martin (2002) studied the development of soil arching by considering the stress state of the soil around the piles at increasing soil displacements. Figure 2.23 shows their study of the development of soil yield with increasing soil displacement for a drained soil of strength $\phi = 30^\circ$. It shows that plastic yielding begins from the sides of the pile shaft, where the soil is being squeezed between the piles. With increased displacement, the plastic zone then works to the front of the pile, except for a small elastic zone which is protected, and is termed the 'arching foothold' (Atkinson and Potts, 1977). The arching mechanism forms in a semi-circular shape upslope of the piles, as demonstrated by the principal stress rotations (Figure 2.25(a)). With sufficient displacements, the increased stress in the arch causes plastic failure of arch elements (Figure 2.23(c)). However, the reduced stresses in the zone directly below the arch means that these elements remain elastic, and this is termed the 'elastic arching zone' by Chen and Martin (2002).

Chen and Martin (2002) also studied the development of arching in an undrained soil of shear strength 25kPa. The development of the plastic zone around the piles is shown in Figure 2.24. In the undrained soil, there is no evidence of the elastic arching foothold forming upslope of the piles. The number of zones reaching plastic yield behind the piles is also reduced when compared with the drained analysis, and this may be explained by the formation of a gap between the rear of the pile and the soil (Chen and Martin, 2002). The existence of an arching effect is again demonstrated by principal stress rotation; however, the more uniform principal stresses (Figure 2.25(b)) give an indication that the arching is not as effective as for the drained soil strength parameters used. It is evident from Figures 2.23 and 2.24 that the development of the plastic failure mechanism is different for drained and undrained soils. This yielding behaviour is a progression toward plastic flow of the soil, and Chen and Martin (2002) go on to study the development of plastic yield (their analyses are discussed shortly).

Chen and Martin (2002) studied the effect of varying the soil stiffness, the initial *in situ* lateral stress, soil dilatancy and soil-pile friction on the development of arching. They concluded that soil stiffness and *in situ* stress made little difference to the arching behaviour (although they do obviously alter the p - δ curves obtained). Increasing the soil dilatancy caused volumetric dilation

to occur, increasing the soil squeeze between the piles and reinforcing the arching mechanism to give a higher ultimate yield load (Chen and Martin, 2002). As may be expected, a rough pile interface was found to give a larger yield load than a smooth interface.

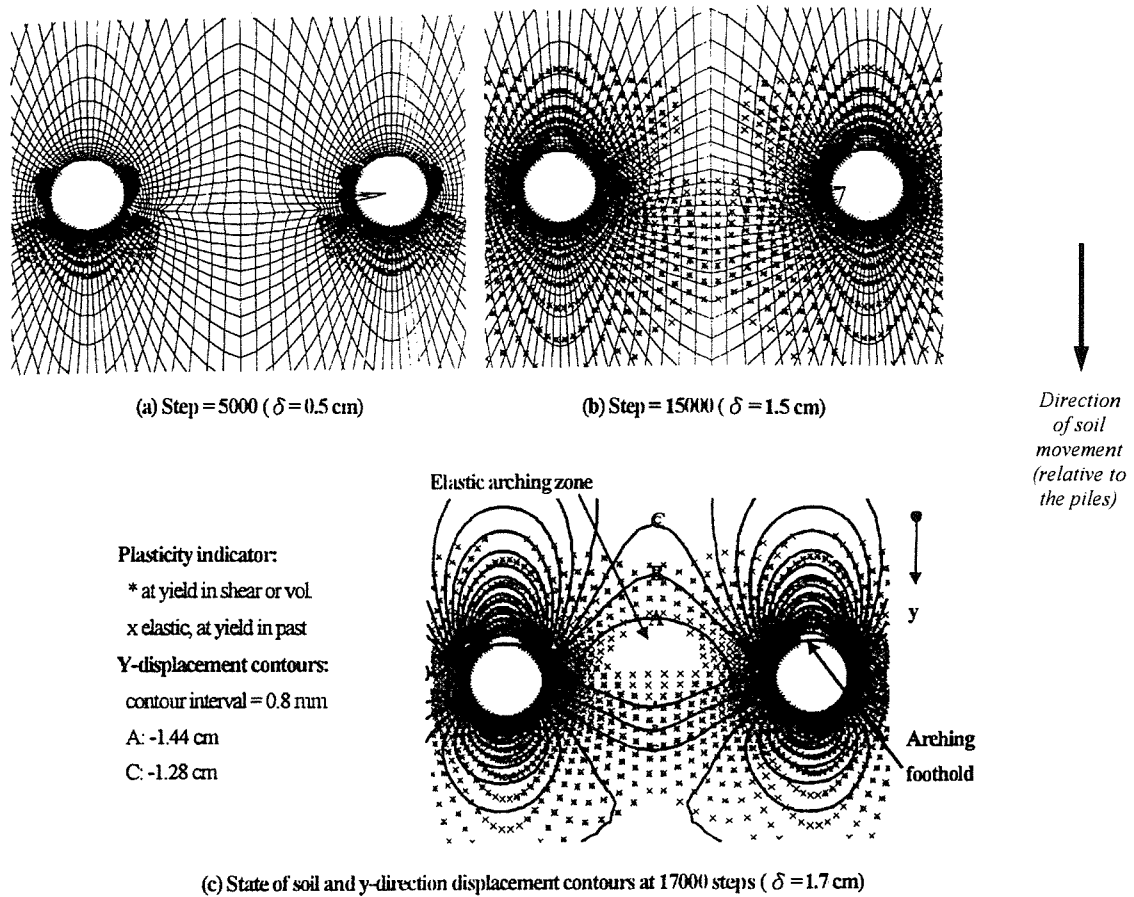


Figure 2.23. y-displacement contours and the development of the yield zone for piles at a spacing of 4 diameters centre to centre and soil strength $\phi = 30^\circ$ (from Chen and Martin, 2002).

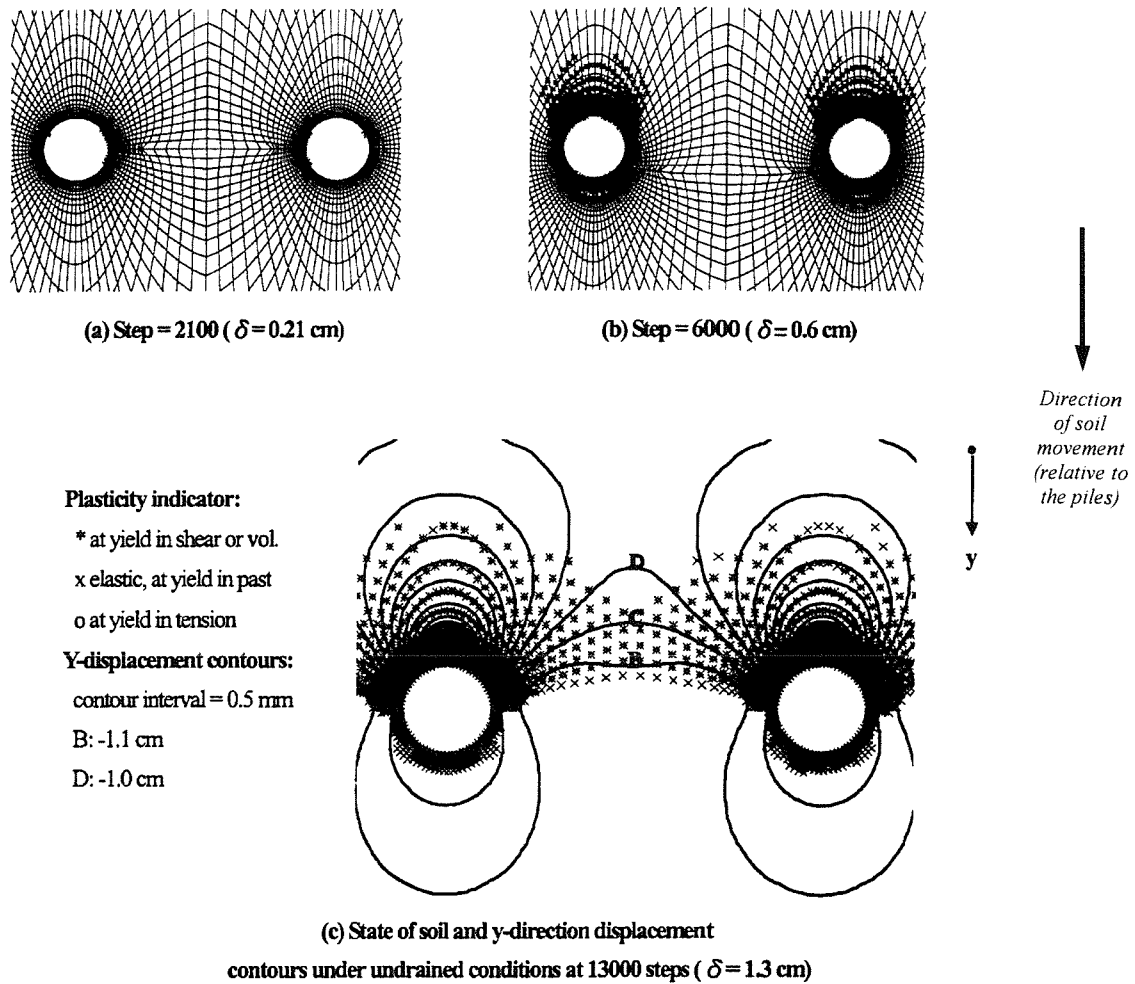


Figure 2.24. y-displacement contours and the development of the yield zone for piles at a spacing of 4 diameters centre to centre and an undrained shear strength of $c_u = 25$ kPa (from Chen and Martin, 2002).

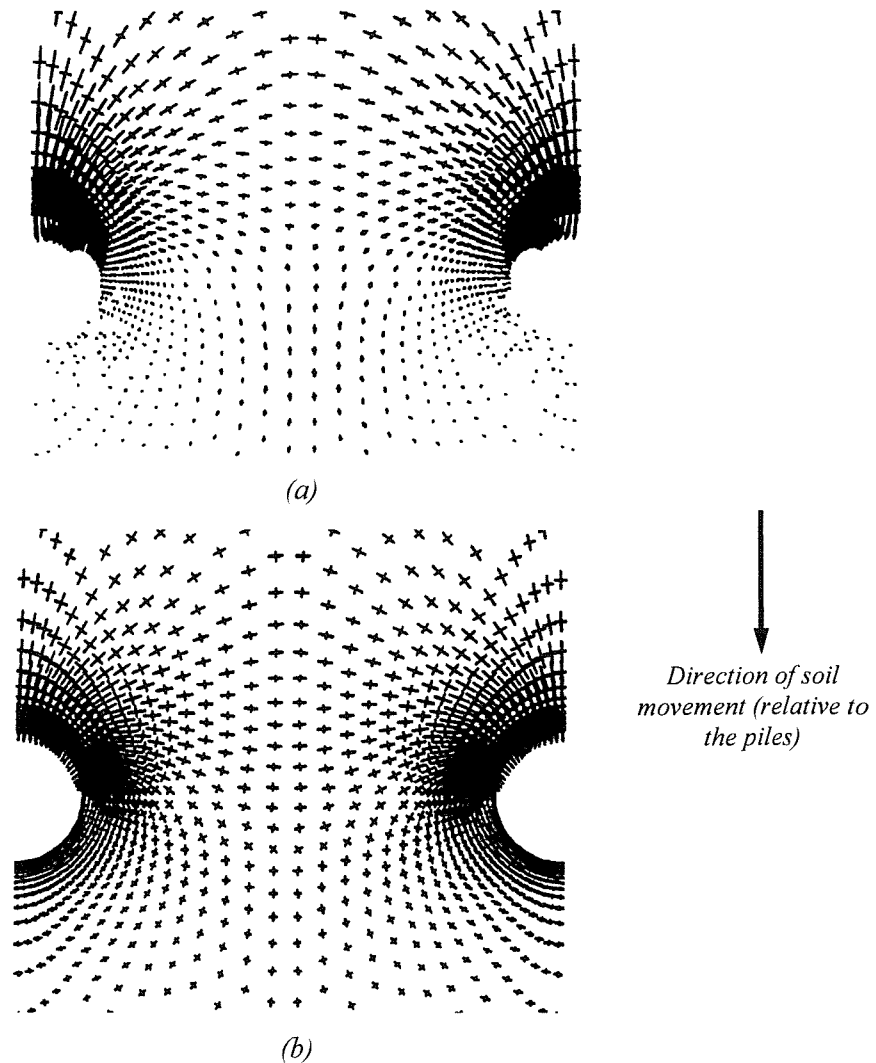


Figure 2.25. Principal stress rotations for analyses with piles spaced at 4 pile diameters: (a) drained soil of strength $\phi = 30^\circ$; and (b) undrained soil of shear strength $c = 25\text{kPa}$. (taken from Chen and Martin, 2002).

Also of interest is the ultimate pile load. Both Bransby and Springman (1999) and Chen and Martin (2002) show that for an undrained soil, the arching effect increases with soil displacement until the soil starts to yield around the pile, after which the arching effect then diminishes, and regardless of pile spacing the final ultimate pile loading is almost the same. This is shown in Figure 2.26(c) for a soil with an undrained shear strength of 25kPa. Figure 2.26(d) shows the pressure at the point B' on the base boundary of the mesh (as shown on Figure 2.20) against the free-field soil displacement, for the analyses with piles placed at spacings of 2 and 4 pile diameters. During the elastic loading of the piles, soil arching occurs, and this dramatically reduces the load transferred to the base boundary of the mesh (Figure 2.26(d)). The arching at the smaller (2d) pile spacing is clearly more efficient, as a larger drop in base boundary pressure is seen. As the soil starts to fail around the piles, and the pile load remains constant, the additional pressure applied to the analysis as a result of increased displacement then transfers to the base boundary of the analysis.

The yield behaviour in drained soils is different. The yield load for the piles is dependant on the pile spacing, with a smallest yield load for the piles placed closest together (Figure 2.26(a)).

This result may initially seem odd, as the smallest pile spacing would be expected to put up the most resistance to flow of soil around the piles. However, this result is also found by Chen and Poulos (1997) and Adachi *et al* (1989). The difference in ultimate pile load with pile spacing is a clear indication that arching or interaction effects continue to have an influence even at the point of soil flow around the pile. Chen and Martin (2002) investigated the extent of the plastic zone around the piles at the ultimate pile load for piles spaced at 2 and 3 pile diameters, and the result is shown in Figure 2.27. This shows that at a pile spacing of two diameters, all the soil elements between the piles are at yield, and there is no elastic arching zone. The high number of elements at yield may be the cause of the lower ultimate lateral load (Chen and Martin, 2002). However, the arching effect at two pile diameters is still the most efficient as evidenced by the larger drop in the pressure acting on the base boundary of the mesh (Figure 2.26(b)).

Cai and Ugai (2001) present the results from a series of three-dimensional finite element analyses on a stiff soil slope ($E = 200 \text{ MPa}$, $\phi = 30^\circ$ and $c' = 10 \text{ kPa}$) with stabilising piles placed at 2, 3, 4 and 6 pile spacings (their analyses are discussed in more detail in Section 2.6). The strength reduction method is used, and the results (Figure 2.28) are shown for the ultimate case, where the soil is on the verge of plastic failure around the piles (Cai and Ugai, 2001). The results show that the pile spacing alters the pile behaviour, with the ultimate pile load over the top 4m or so of the pile increasing with pile spacing. It can be seen that the pile displacements and bending moments are largest at the smallest pile spacing. However, care must be taken in interpreting these results, as the strength reduction method is likely to result in a lower slope strength when the piles are placed closer together than for when they are spaced further apart, with the lower soil strength resulting in a larger overall slope loading to the pile.

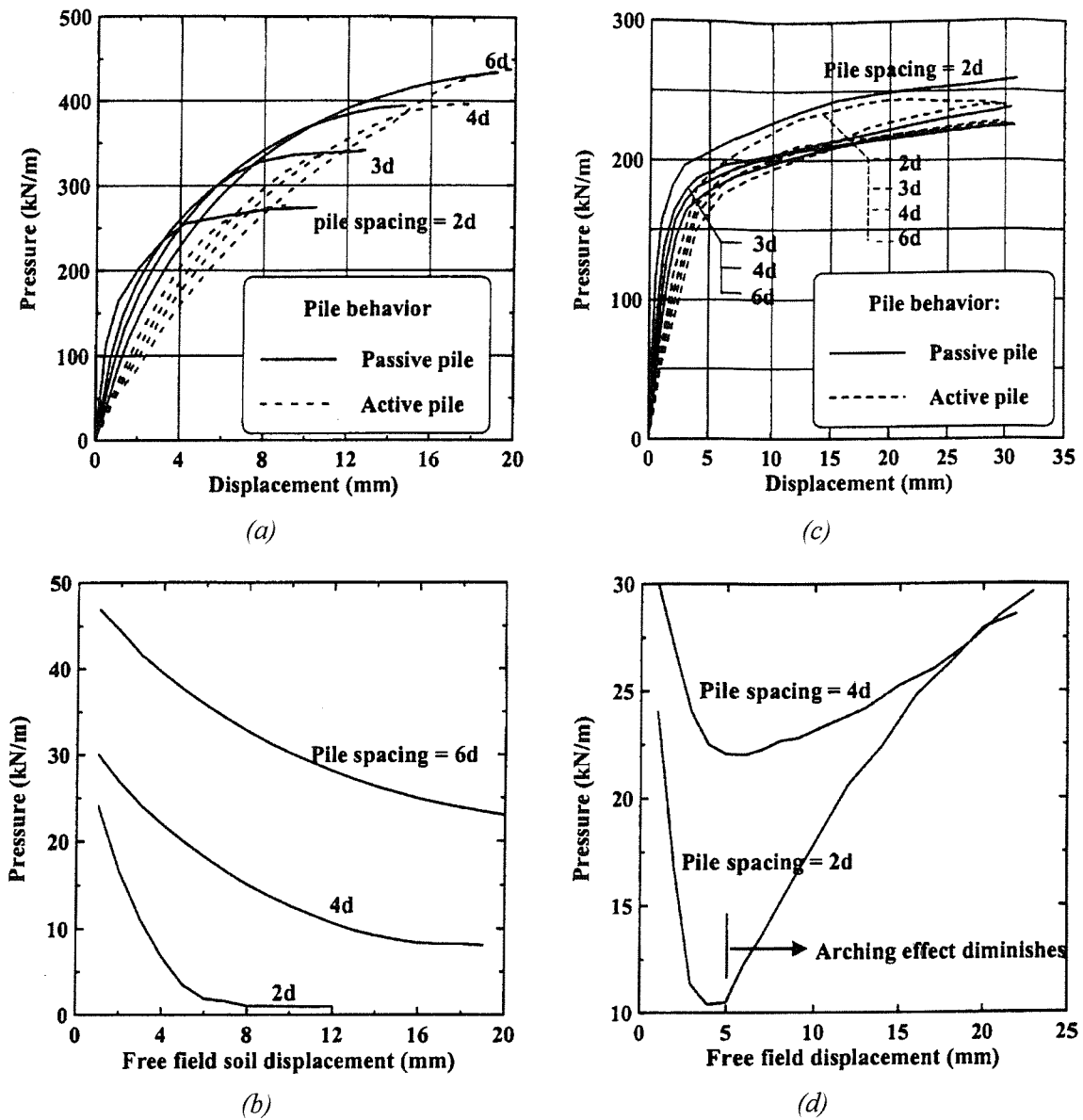


Figure 2.26. (a) p - δ curves for piles at different spacings in a drained soil ($\phi = 30^\circ$); (b) the pressure at the base of the mesh (B-B' in Figure 2.23) against the free field displacements for the same drained soil; (c) p - δ curves for piles at different spacings in an undrained soil ($c = 25$ kPa); (d) the pressure at the base of the mesh against the free field displacements for the same undrained soil (all taken from Chen and Martin, 2002). d = the pile diameter.

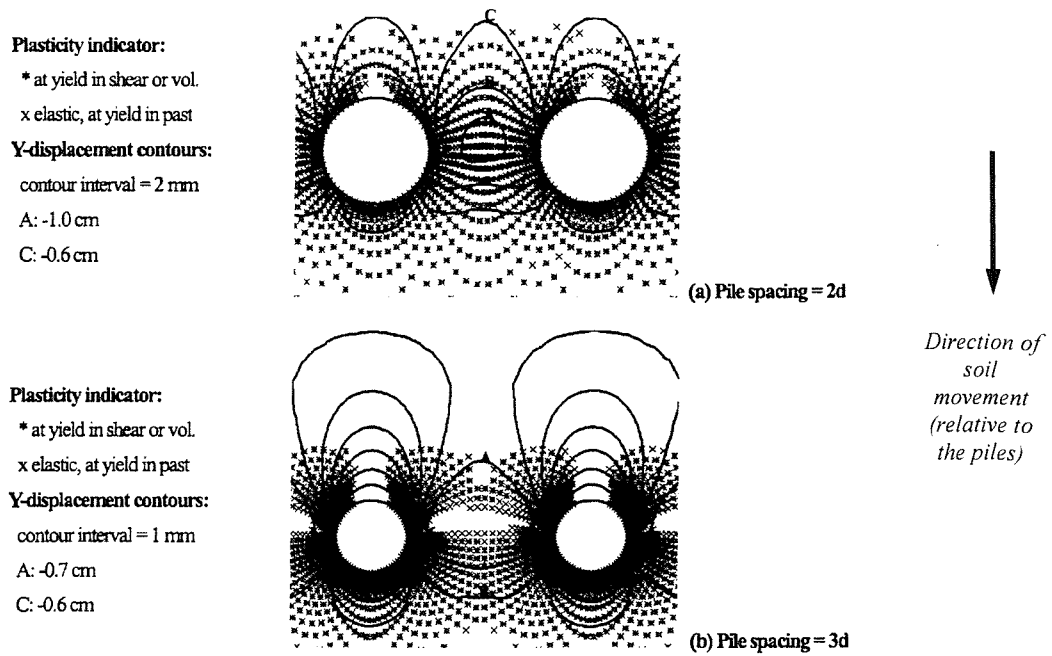


Figure 2.27. Yield zone around the piles at spacings of 2 and 3 pile diameters, at the ultimate lateral load. Drained soil of strength $\phi = 30^\circ$ (from Chen and Martin, 2002).

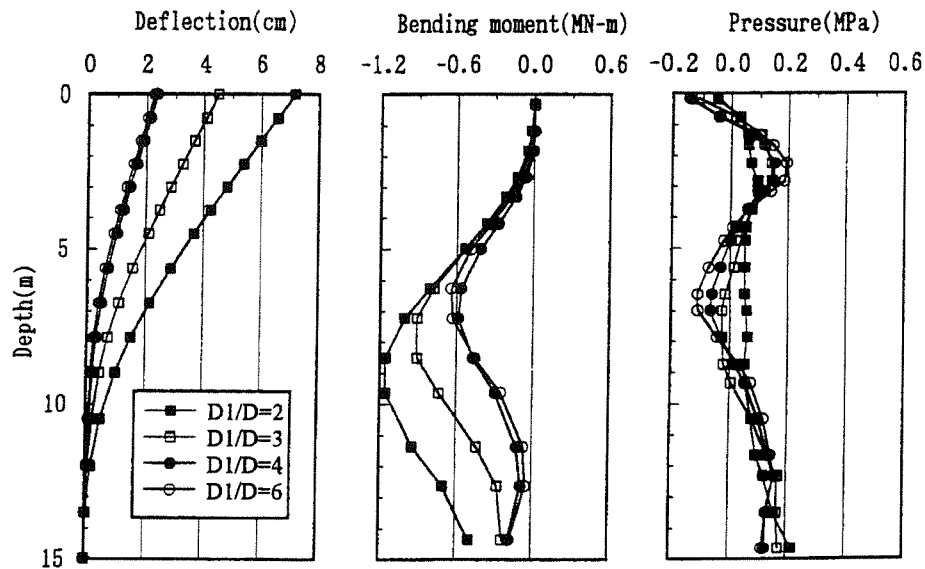


Figure 2.28. Pile displacement, pressure and bending results from finite element analyses (from Cai and Ugai, 2001). $D1$ is the distance between the piles, centre-to-centre, and D is the pile diameter.

2.4.8 Analytical studies of arching in slopes

Analytical approaches that incorporate the pile spacing when calculating the load on the head of the pile have been presented by Wang and Yen (1974) and Ito and Matsui (1975). Neither method addresses the equilibrium of the slope before or after pile installation, or allows the direct calculation of the increase in factor of safety after the piles have been inserted into the slope. To assess the factor of safety of the slope, the lateral pile loading calculated by these methods can be put into limit equilibrium slope stability analysis (Hassiotis *et al*, 1997; Poulos, 1995).

Wang and Yen (1974) method

Wang and Yen (1974) analysed an infinite slope of soil sliding over a firm underlying stratum into which the piles are embedded (Figure 2.29). They combined the infinite slope theory with classical soil arching theory developed by Terzaghi (1943), and additional terms were added to the Terzaghi (1943) force equilibrium equation to take into account the basal friction of the slice as it slides over the slope failure surface. The theory allows for a reduced soil strength along the failure plane. A limitation of the Wang and Yen (1974) method is that it is based on the analysis of an infinite slope, and so is not ideal for analysis of infrastructure slopes of finite length and height that are more likely to exhibit short curved failure surfaces.

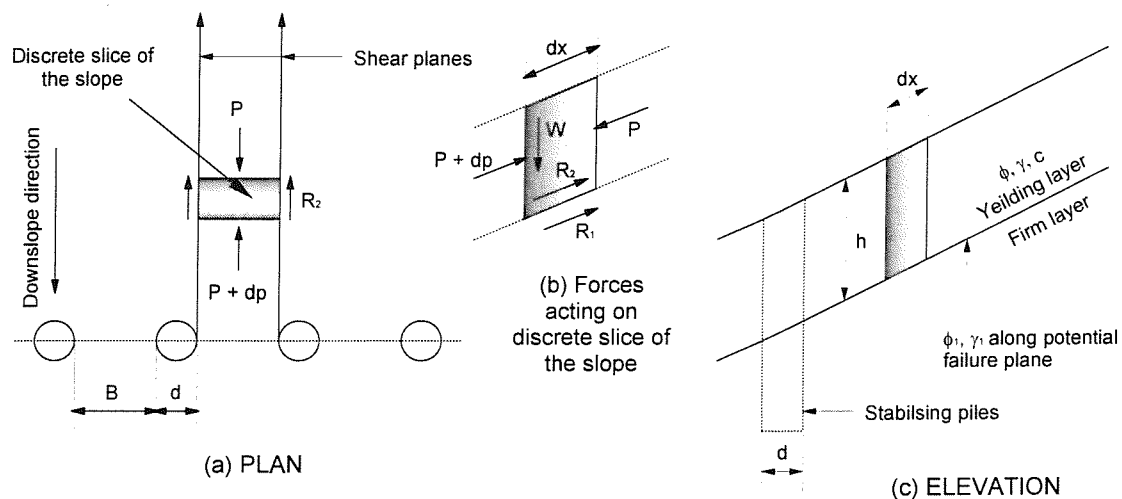


Figure 2.29. Diagram of the Wang and Yen pile-soil system (redrawn from Wang and Yen, 1974)

The analysis assumes a minor principal stress arching mechanism (see Section 2.4.5). Shear bands are assumed to form either side of the piles (as shown in Figure 2.29a), and the shear bands transfer the stress from the soil elements between them onto the piles (which are assumed to be rigid). Bosscher and Gray (1986) point out that in many cases these shear bands may not be a reasonable approximation of the actual failure mechanism. The formation of these shear bands certainly seems unlikely to be the critical mechanism at very big and very small pile spacings.

The Wang and Yen (1974) method calculates the proportion of the lateral pressure in the slope transferred from the soil between the piles onto the piles via the vertical shear bands, by dividing the section between the shear bands into a series of discrete slices, and by considering the reduction in horizontal downslope normal stress across each slice (Figure 2.29a and 2.29b). The method then calculates the total load acting on the pile by adding the 'at rest' lateral stress from the thin strip of stationary soil directly behind each pile to the load transferred onto the

piles via the shear bands. The use of the lateral stress as the loading on the upslope side of the pile within the total pile load calculation is equivalent to assuming that a tension crack or gap forms directly behind the piles, i.e. that no stress acts on the downslope side of the pile. The method takes no account of the stability of or the earth pressure that may result from the section of the slope downslope of the pile row.

The lateral pressures acting on the upslope side of the pile are calculated using the earth pressure at rest, K_0 , as are the lateral earth pressures at the top of the shear bands (which are of a specified finite length) (Wang and Yen, 1974). K_0 represents the 'at rest' condition, where the wall (or in this case, the soil) does not move – this may not be considered very realistic in a failing slope (Bosscher and Gray, 1986).

Figure 2.30 shows a graph of pile spacing against the load applied to the piles calculated using the Wang and Yen (1974) method. Also given in figure 2.30 are the ground conditions used as input to the calculations. Note that the parameters used make the slope just unstable.

The Wang and Yen (1974) curve shows some variation in loading of the pile head with change in pile spacing. The curve initially shows an increase in load on the piles as the pile spacing increases, which would be expected as each pile is supporting a larger width of the slope.

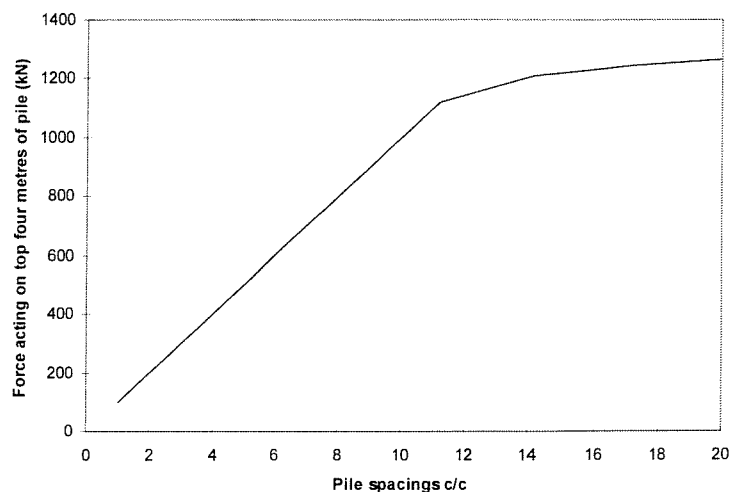


Figure 2.30. Comparison of the change in the force acting on the head of the pile with pile spacing, calculated using the Wang and Yen (1976) method. The parameters used to calculate this curve were as follows: the pile diameter = 1m; the depth to the failure surface = 4m; the length of slope above the row of the piles (or the length of the shear bands) = 12m; the angle of the slope = 14°; the intact soil strength = 20°; the reduced strength of the basal shear surface = 13.8°; and the soil unit weight = 19 kN/m³. The pore water pressures throughout the slope are assumed to be zero.

The curve starts as a straight line, because the system is 100% efficient, i.e. the shear bands support all of the downslope loading from the section of the slope spanning between them, and transfer all of this load to the piles. Each pile therefore carries a load approximately equal to one pile spacing width of the slope. In the example shown, this continues up to a pile spacing of 11 diameters, at which point the load transfer (or soil arching) is no longer totally efficient, and not all of the downslope load nominally acting between the piles is transferred onto them. Given a sufficiently large pile spacing, the proportion of the residual slope load transferred by the shear bands onto the piles reduces towards zero (Wang and Yen, 1974).

The load calculated to be acting on the piles is large, because as discussed above, the load is calculated using the lateral earth pressure distribution, and because the portion of the slope downslope of the piles is ignored by Wang and Yen (1974). If the same slope geometry is put into a limit equilibrium slope stability analysis, the additional force required as input to the analysis to stabilise a 4m wide strip of slope of the same length and depth is 12.5 kN (Figure 2.31).

Bossher and Gray (1986) carried out a series of model experiments with a sand slope supported by different sized yielding gates (see Section 2.4.6), and they compared their experimental loading results with the pile loads calculated using the Wang and Yen (1974) method. They found the experimental values of pile load to be much smaller than the values calculated by the Wang and Yen (1974) theory.

The equations of infinite slope stability can be re-written to include a force S that the piles provide to stabilise the slope.

Considering horizontal and vertical equilibrium on the failure surface, and the mobilised friction along the failure surface, the following equation can be derived:

$$F_s = \frac{\tan \phi_{crit}}{\tan \phi_{mob}} = \frac{\tan \phi_{crit} (\gamma z \cos^2 \beta + S \sin \beta)}{\gamma z \cos \beta \sin \beta - S \cos \beta}$$

Inserting the following parameters into the equation: the depth to the failure surface, $z = 4\text{m}$; the angle of the slope $\beta = 14^\circ$; the reduced strength of the basal shear surface $\phi_{crit} = 13.8^\circ$; the soil unit weight, $\gamma = 19 \text{ kN/m}^3$; and the factor of safety for the slope, $F_s = 1.0$, yields a value of $S = 0.26 \text{ kPa}$

Assuming as in the Wang and Yen (1974) method the length of slope above the row of the piles = 12m, and the width of slope supported by each pile is 4m, then the force S that the piles must add to the slope to stabilise it is $S = 12.48 \text{ kN}$

Figure 2.31. Modification of the equations for infinite slope stability to include a point force to represent the stabilising action of the piles.

Ito and Matsui (1975) method

By considering the state of plastic deformation in the ground around a pair of piles (Figure 2.32), Ito and Matsui (1975) determine the limiting lateral pile-soil pressure acting on the pile above a presumed slip surface in terms of the soil properties and the pile geometry. A number of equations and design charts are developed to allow calculation of the limiting lateral pile-soil load on the piles in a range of soil types and strengths.

Figure 2.33 shows the relationship between the force on the piles and the pile spacing calculated using the Ito and Matsui (1975) theory for the soil and pile geometry also given in Figure 2.33. The curve in Figure 2.33 shows that for a given diameter of pile, the force on the pile increases rapidly as the spacing between the piles becomes narrower. This is based on the soil wedge adjacent to the piles finding it harder to squeeze between them and therefore increasing the load on the pile. The Ito and Matsui (1975) method basically allows calculation of the maximum limiting pile-soil pressure. It therefore takes no account of the equilibrium of slope or the development of load with increased slope displacements (where in practice as the pile spacing decreases, the load transferred to each pile would be expected to decrease as each pile is supporting a narrower strip of the slope). The Ito and Matsui (1975) method also fails to take account of the sloping ground surface.

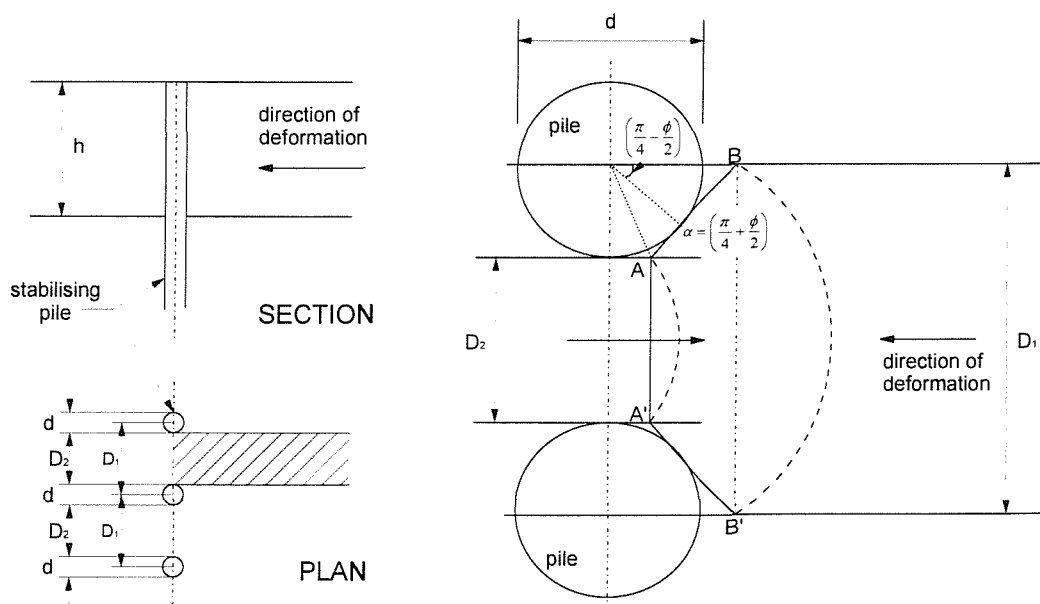


Figure 2.32. Diagram of the assumed pile-soil failure mechanism (redrawn from Ito and Matsui, 1975).

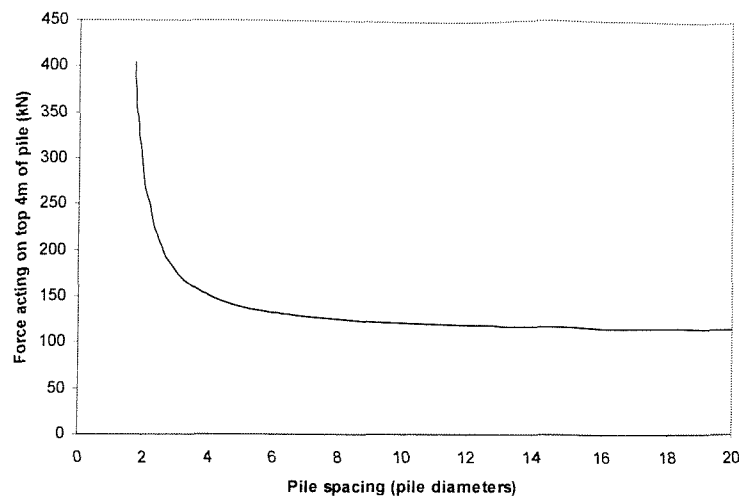


Figure 2.33. Relationship between pile loading and pile spacing, using the method presented by Ito and Matsui (1975). The parameters used to calculate this curve are as follows: pile diameter = 1m; depth to the failure surface = 4m; intact soil strength = 20° ; and soil unit weight = 19 kN/m^3 . The pore water pressures throughout the slope are assumed to be zero.

The plastic deformation mechanism assumed by Ito and Matsui (1975) is unlikely to be the critical mechanism at small pile spacings, evidenced by the impossibly large increase in pile load at these spacings. In many respects the Ito and Matsui (1975) analysis has similarities to the finite element analyses run by Chen and Martin (2002), where the soil is being pushed inexorably between the piles. However, for a drained soil Chen and Martin (2002) and Chen and Poulos (1997) find considerable variation in the limiting lateral pile-soil pressure at different pile spacings, with the lowest limiting pressure when the piles are closest together (see Figure 2.26a). The Ito and Matsui (1975) method calculates a reverse trend to this, with the largest limiting pressure when the piles are closest together (Figure 2.33).

Bosscher and Gray (1986) and De Beer and Carpentier (1977) point out a number of simplifying assumptions that Ito and Matsui make in their analysis, but which leave the analysis in error. Ito and Matsui (1975) assumed the planes AB and A'B' (Figure 2.32) to be principal stress planes; this is incorrect as the planes are not orthogonal to one another. De Beer and Carpentier (1977) assumed corrected principal stress directions and re-derived the solution to see the effect on the results. For drained soils, a large difference was found between the pile loading results obtained by Ito and Matsui (1975) and De Beer and Carpentier (1977). For undrained soils, the effect of the principal stress directions was less significant - this may be expected considering that this strength condition more closely represents the behaviour of a metal, and the Ito and Matsui (1975) method is taken from metal extruding processes (Bosscher and Gray, 1986). De Beer and Carpentier (1977) bracket the range of pile spacings over which they felt the re-derived theory can be applied. They give the range of pile spacings as 4-6 (pile diameters, centre to centre). Bosscher and Gray (1986) modified the Ito and Matsui (1975) method to correct for both the

errors in the assumptions of principal stress, and for the effect of a sloping ground surface. However, they do not explain how they do this in any detail, or present any re-derived equations.

Ito and Matsui (1975) compare the pile loading calculated using their design method to the behaviour of real piles placed into a number of landslides in Japan (see Section 2.7.2 of this thesis), and find that the method is a reasonable approximation to the pressures found to be acting on the top section of the real piles used to stabilise the slopes. In the case of large flowslides in which the pile stabilising force required will be large, an approximation of the limiting lateral pressure may be a reasonable estimate of the working pile-soil pressure.

The Ito and Matsui (1975) method has been used for the design of stabilising piles and back analysis of experimental work by Bosscher and Gray (1986), Allison *et al* (1991), Anagnostopoulos *et al* (1991), Popescu (1991), Poulos (1995) and Hassiotis *et al* (1997). Most of the above recognise that the Ito and Matsui (1975) method should be used as an estimate of the limiting lateral pile-soil pressure, however only some of the above (Bosscher and Gray, 1986; Anagnostopoulos *et al*, 1991; Poulos, 1995) make clear that they are aware of the inherent limitations (such as the error in the principal stress assumptions). The Ito and Matsui (1975) method is best avoided, particularly because the trend of limiting lateral pressure with pile spacing in a drained soil is the reverse of the trend found in finite element studies and model tests by Chen and Martin (2002) and Chen and Poulos (1997).

2.4.9 Arching between passively loaded piles in other geotechnical constructions

Parallels are often drawn between arching in slopes and soil arching in other geotechnical constructions, such as piles laterally loaded by an adjacent surcharge or spill-through bridge abutments. Care has to be taken in trying to infer too much from some of this work, as in many cases (such as the piled bridge abutments studied by Ellis and Springman, 2001) the boundary and kinematic constraints are somewhat different than for pile loading in slopes.

Springman (1989) carried out a series of centrifuge tests to look at the effect of embankment construction on the lateral pressure acting on an adjacent row of piles. The piles were loaded by the lateral displacement of a soft surface clay layer through which the piles were embedded and on to which the embankment was constructed. Springman and Bolton (1990) go on to develop a simplified means of calculating the pile behaviour, based on a geo-structural mechanism to estimate the pressure acting on the piles, and an elastic analysis to calculate the pile bending response to this pressure. The pile pressure was essentially defined by Springman and Bolton (1990) as a function of the shear stiffness of the soil and the relative pile-soil displacement:

$$p = 5.33 G (\delta u_s - \delta u_p)/d$$

Equation 2.6

where G = the local shear modulus, d = the pile diameter and δu_s and δu_p are the soil and pile displacements. The geo-structural mechanism was then developed to give the pile pressure in terms of the embankment surcharge, pile and soil stiffnesses and the pile spacing:

$$p_m = \frac{q}{\left[\frac{3G_m d}{G_r h} + \frac{d}{s} + \frac{0.71G_m d h^3}{EI} \right]}$$

Equation 2.7

where q = surcharge load, EI = the pile bending rigidity, h = the depth of the soft layer, G_r = the reduced shear modulus around the pile, G_m = the full shear modulus of the layer and s = the pile spacing. As the pile spacing increases, so does the load acting on the pile. The pressure distribution p_m is assumed to be constant over the depth of the clay layer. However, Springman and Bolton (1990) point out the adjustments that need to be made to the lateral pressure distribution due to the shear on the underside of the pile cap and for the reduced effective depth of surcharge in a deep soft clay layer.

Ellis and Springman (2001) develop this work by carrying out centrifuge tests and numerical modelling of full height piled bridge abutments, with the base of the abutment piled through a soft clay layer that deforms laterally due to embankment construction. The model abutment had two rows of piles, connected with a pile cap. Boundary effects due to embankment construction influenced the behaviour of the piles, with the granular embankment fill arching vertically onto the pile cap. Subsequently, the first row of piles acted in passive pile loading, but the second set were displaced actively into the soil, and the bending behaviour of each of the rows of piles is different.

Stewart (1992) carried out centrifuge model tests on pile groups laterally loaded by construction of an adjacent embankment, and found that a pile spacing of 5 diameters was sufficiently large to minimise the interaction between piles (Carder and Temporal, 2000).

Carder and Temporal (2000) review the design guidance for full spill through bridge abutments. Such bridge abutments are essentially a row of piles placed into an embankment slope to support the bridge deck, and many of the issues regarding arching and flow of soil between the piles apply. However, many of the design approaches are simplistic, and make basic assumptions about how close the piles need to be for arching to occur (Carder and Temporal, 2000). Carder and Temporal (2000) point out that further development in the design for bridge

abutments has not taken place because it is generally accepted that the soil stresses acting on the bridge abutments are heavily site specific, and will depend on the seasonal variation in lateral pile loading from the bridge deck.

Vermier *et al* (2001) describe the use of soldier piles used to retain a 16m deep excavation in sandy gravel in Cologne, Germany. The steel piles were installed at 4m centres with timber lagging spanning between them to retain the soil. 3D and 2D numerical modelling was carried out, in order to try and understand the soil arching that occurs behind the wall. 2D analyses modelled a horizontal plane through the excavation, with the steel piles fixed in position and the timber lagging modelled as beam elements allowed to deflect into the excavation. As the timber lagging deflects, the soil arches onto the fixed steel piles, as evidenced by the principal stress rotation (Figure 2.34). To try to back up these analyses in the field, some of the timber laggings were loosened to see if there was any additional transfer of load to those still fixed in place, or whether the arching mechanism transferred the additional load to the piles. On the whole, the increase in pressure on the fixed laggings was found to be small, confirming the effectiveness of the arching mechanism onto the piles (Vermier *et al*, 2001).

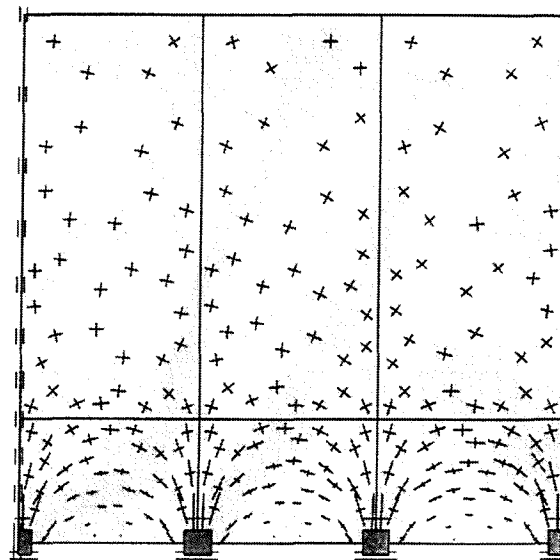


Figure 2.34. Principal stress rotation behind a soldier pile wall showing the soil arching onto the steel piles (from Vermier *et al*, 2001)

2.4.10 Discussion of arching in slopes

The results of the finite element analyses run by Liang and Zeng (2002) and Chen and Martin (2002) demonstrate the complexity of the arching behaviour, and how difficult this makes it to determine the likely maximum achievable pile spacing. It can be seen that arching effectiveness depends on the following factors: the equilibrium of the slope (and therefore the displacements obtained), dimensionless pile spacing, pile diameter, pile shape, soil frictional strength, soil cohesion, the pile-soil interface, a gap forming behind the piles, soil dilation and relative pile-

soil displacement. The main mode of arching in slopes occurs by means of a major principal stress arching mechanism upslope of the piles.

The finite element analyses run by Liang and Zeng (2002) and Chen and Martin (2002) did not model the equilibrium of the slope, but they did demonstrate the variation in limiting pile-soil pressure with pile spacing and give an indication of the overall effectiveness of arching for a range of pile spacings and soil strengths. Liang and Zeng (2002) show that arching becomes more effective when the piles are placed closer together and with an increase in soil strength. In particular, the effective cohesion component of soil strength has a large impact. However, they only consider a maximum pile spacing of 4 pile diameters, and it would be interesting to see how the arching is effected at larger spacings. Liang and Zeng (2002) suggest that the effectiveness of the arching is reduced when large diameter piles are used at the same dimensionless pile spacing – this is an indication that arching effectiveness will not only be based on dimensionless spacing but also the physical gap in metres between individual piles. Chen and Martin (2002) study the development of plastic yield around the pile, and show that the formation of plastic yield in drained and undrained soils is clearly different. The load transfer of the arching mechanism is expressed by the proportion of residual slope load carried by the soil between the piles, and this seems to be a suitable method of expressing arching effectiveness.

The physical model tests do have limitations – the tests carried out by Bosscher and Gray (1986) will not be at realistic soil stress levels – but they do have the advantage of the pile load being in equilibrium with the slope above the piles, and therefore may give a better idea about soil arching and pile loading before the limiting soil-pile pressure is reached. All of the model tests show an increase in pile loading with pile spacing as piles carry a larger width of the slope. In the tests by Adachi *et al* (1989) and Bosscher and Gray (1986) the arching takes place as semicircular major principal stress arch above the slope. The tests by Bosscher and Gray (1986) and Hayward *et al* (2000) seem to show the arching becoming less efficient with spacings beyond about 4 pile/gate diameters.

Loose guidance must be that for lower soil strengths, arching between the piles rapidly becomes less efficient with increasing pile spacing, and the maximum pile spacing will be limited to possibly 3 pile diameters. With much stronger soils (especially those with a high effective cohesion), the arching still appears to be highly effective at 5 or even 6 pile spacings. However, the effects of cyclic loading due to cyclic changes in pore pressure on long term arching effectiveness are uncertain. In practice it may be necessary to run finite element analyses to understand the soil arching behaviour for a given design situation.

Ito and Matsui (1975) and Wang and Yen (1974) present analytical methods of calculating the load acting on the piles as a result of soil arching. Both of the methods have significant limitations, including that the failure mechanism they use is only likely to be relevant over a small range of pile spacings.

2.5 Evaluating the shear force and bending moment in each pile

2.5.1 Penetration of the piles into the slope

The penetration of the piles into the slope must be sufficient to satisfy two conditions. The first is the equilibrium of the piles within the slope, so that the resisting forces and moments acting at the base of the pile can balance the overturning forces and moments from the failing mass. The fixity of the toes of the piles within a firmer underlying stratum is particularly important, as the larger the restoring force they can carry, the smaller the slope displacements are likely to be. The position of the sliding surface in relation to the ends of the pile has a significant influence on the mechanism of stabilisation and subsequently the pile movements and bending moments obtained. Poulos (1995) identifies three different mechanisms of stabilisation as shown and discussed in Figure 2.35. The overall pile stabilisation mechanism is further discussed in sections 2.5.2 and 2.5.3.

The second of the conditions for stability is the requirement for the piles to extend below any likely increase in the zone of instability – if the failure zone increases to encompass the toe of the piles they will no longer stabilise the slope. Design should ensure an adequate factor of safety for global slope stability as illustrated in Figure 2.36. In steep slopes the piles have to be long to prevent global failure beneath the pile system, and the mechanism of stabilisation tends to be for the majority of the pile to be beneath the sliding surface (mode (a) in figure 2.35). However, in the case of existing failures, and where global deep seated failure does not pose a problem, better economy would be achievable through use of shorter piles penetrating just below the failure surface (mode (c) in Figure 2.35). A factor of safety of 1.3 – 1.5 is typically required for global slope stability in infrastructure slopes, and this usually dictates the pile length (Ellis, 2002).

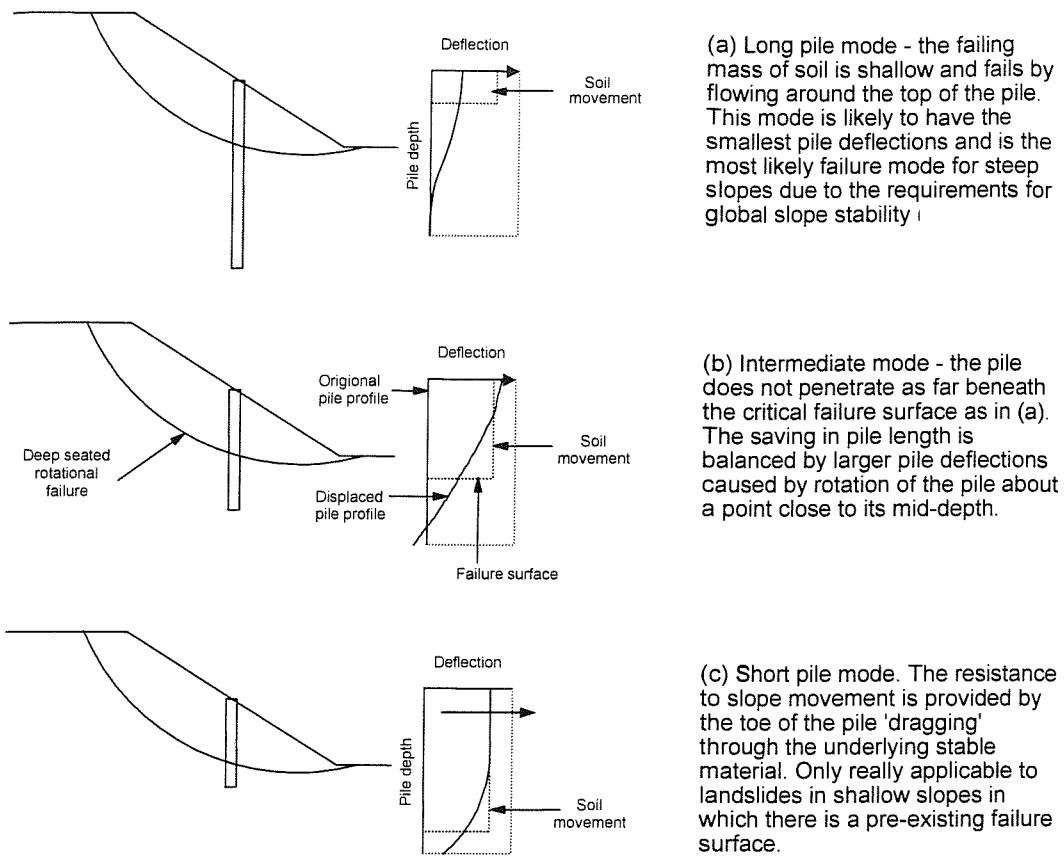


Figure 2.35. Possible modes of stabilisation (based on Poulos, 1995).

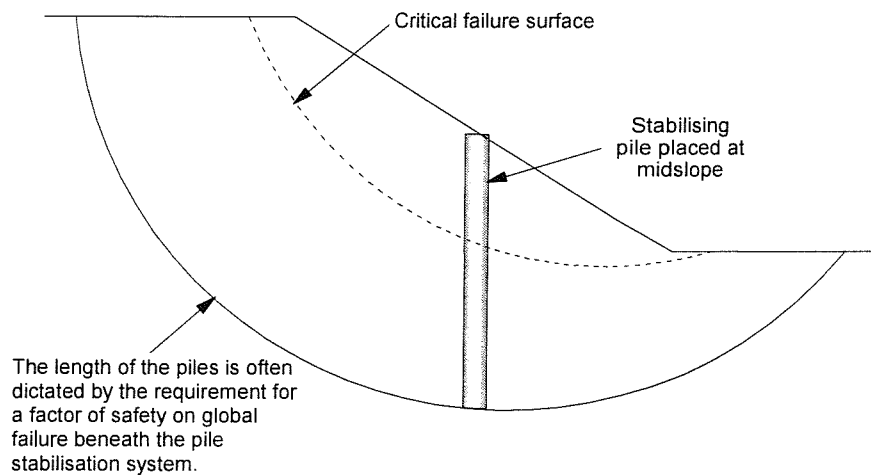


Figure 2.36. Global stability of a pile system.

2.5.2 Overall pile stability

An important stage of pile design is to ensure that the pile has sufficient capacity in shear and bending to withstand the forces applied to it by the moving mass of soil. This involves understanding how the pile behaves beneath the failure surface, where the largest bending moments often occur. The length of pile beneath the failure surface is laterally loaded at the head by the force applied by the failing soil mass. Assuming that the pile acts in 'long' mode (see Figure 2.35), active resistance is formed in front of the pile in response to the lateral loading, and ensures the overall stability of the pile ((a) in Figure 2.37). In many cases, the bending and displacement behaviour of the pile below the failure surface will be both influenced by and have an influence on the soil-pile behaviour above the failure surface.

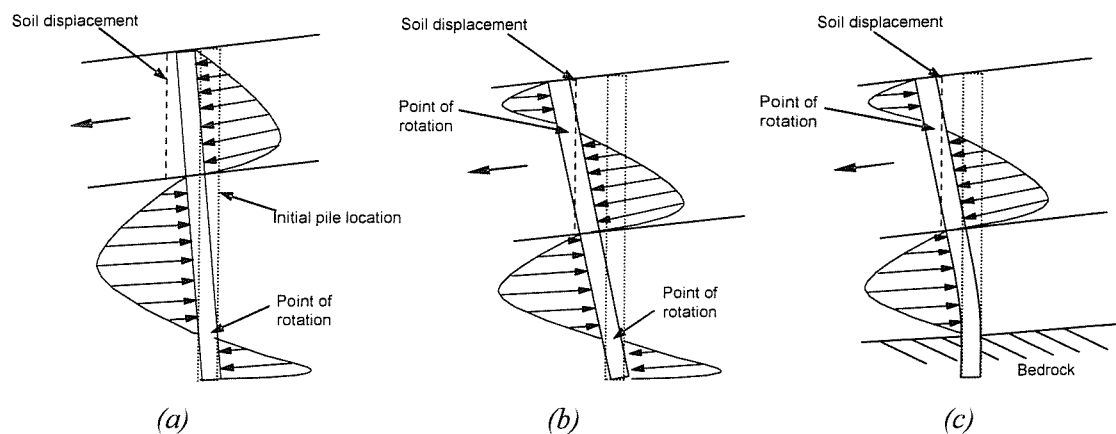


Figure 2.37. Diagrams showing net pile-soil pressure and displacement of piles for: (a) pile stability in 'long' pile mode; (b) pile stability in 'intermediate' pile mode; and (c) pile fixed into a stiff underlying bedrock.

Pile bending behaviour is heavily dependant on the rotation of the pile and the pile base fixity. If there is no stiff underlying stratum for the pile to toe into, the 'long' mode pile is likely to act under fixed earth conditions, in which the pile rotates about a point a short distance above its base ((a) in Figure 2.37). In cases where piles are installed in 'intermediate' mode ((b) in Figure 2.37, or where only a short pile length penetrates below the failure surface and there is a lack of pile base fixity), it is not uncommon to see a point of inflexion with the head of the pile bending back on itself as the top of the pile starts to displace further than the moving soil (for example, as reported by Carder *et al*, 2003).

The presence of a very much stiffer soil stratum for the pile to toe into may largely prevent the base of the pile from rotating within the soil, and this is likely to result in much larger pile bending moments ((c) in figure 2.37). This is illustrated by Carder and Easton (2001) who run three-dimensional finite element models of a cutting slope (which are described in more detail in Section 2.6.2). Carder and Easton (2001) ran an analysis with a 20m long pile in a cut slope

made up entirely of a soft clay material, and under serviceability conditions found a maximum bending moment of 250 kNm. In a further analysis of an otherwise identical slope, the bottom ten metres of the pile was embedded in a soil that was approximately 6 times stiffer. The result was a large bending moment at the interface between the soft and stiff soils – with the maximum bending moment of 1800 kNm about 7 times larger than that in the first analysis.

2.5.3 Limit equilibrium analyses

It is common design practice to calculate the bending moments and shear forces within the pile using straightforward limit equilibrium calculations. In many cases, soil stiffness data are not available for running elasticity based analyses. The limitation of limit equilibrium analyses is that in trying to assess the serviceability behaviour the effect of overall pile rotation is not well modelled (assumptions have to be made about the location of points of rotation) and the effect of soil layers of difference stiffness on bending behaviour is not modelled at all.

Using a limit equilibrium calculation, the calculated length of pile beneath the failure surface required for equilibrium will often be quite short (Ellis, 2002; Caroli, 2003; section 3.3 of this thesis). There is then a tendency to extend the pile length to ensure an adequate length of embedment to limit the slope displacements that can occur, and also to prevent a global slope failure occurring beneath the base of the piles (Ellis, 2002). The limit equilibrium calculation then will not calculate a bending moment for the extended length of piles, and increasing the length of the piles may also alter the mode of pile behaviour (and the way it rotates).

There are a number of simple limit equilibrium pile stability calculation methods presented in the literature. Viggiani (1981) has developed dimensionless design charts for piles placed in a two layer, undrained soil, where the upper soil layer is displacing over the lower layer. The limit equilibrium condition is considered for a series of soil failure mechanisms based on the relative lengths of the pile above and below the failure surface, and the soil strength (Figure 2.38). The solutions assume that full pile-soil limiting pressure is developed along the lengths of the pile around which the soil will fail, and therefore represent the ultimate condition only, with no consideration of how the restoring force develops with increased slope movements. The solutions only allow for a soil-pile limiting pressure that is constant with depth.

Figure 2.38 shows three failure mechanisms for a rigid pile: failure mode A has the lower soil layer failing around base of the pile (which is assumed to be dragged through the lower soil); B fails in rotation (in a similar mechanism to that shown in Figure 2.37b); and failure mode C assumes the upper soil layer fails around the top of the pile. Note that it is only possible to calculate the maximum pile bending moment for failure mode B. Viggiani (1981) also derives a

set of conditions that can be used to find the mode of failure based on the ratios of the length of pile and soil strengths above and below the failure surface.

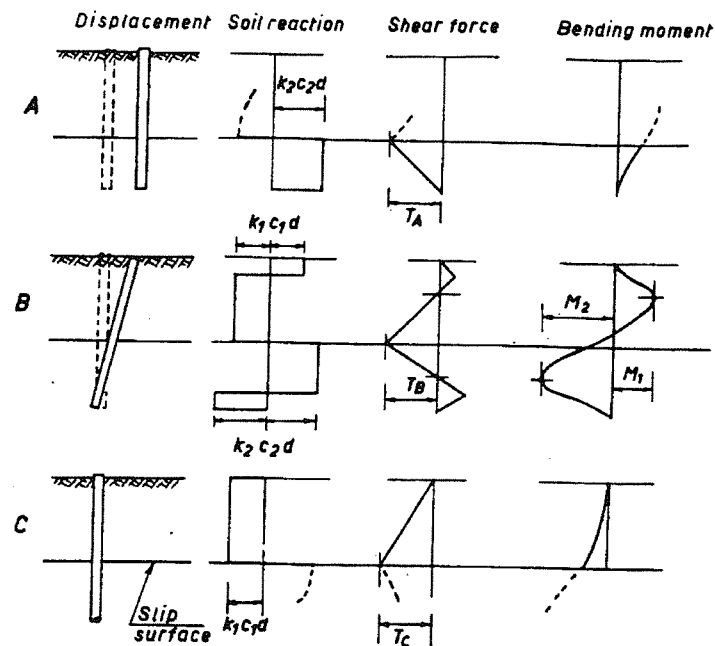


Figure 2.38. Soil pressure distributions and bending and shear diagrams for different failure modes for a rigid pile (taken from Viggiani, 1981). Notations as follows: d = the pile diameter, c = undrained shear strength of the soil, k = limiting lateral pressure coefficient, where the subscripts 1 and 2 denote the upper and lower soil layers respectively.

Viggiani (1981) also considered a set of failure mechanisms that assume the pile to fail in bending with the development of a plastic hinge before the ultimate pile soil pressure is reached (Figure 2.39). Viggiani (1981) points out that usually the bending capacity of the pile will be the limiting condition, rather than the failure of the soil around the pile. This is probably correct, as design based upon the limiting lateral pile pressure would result in the specification of piles with a very large bending capacity. The use of p_u to calculate pile bending and shear force for piles used to stabilise small slopes is likely to be unrealistic. Using the assumption that plastic hinges form in the pile, it is also possible to calculate much more of the pile bending distribution (Figure 2.39).

Design charts based on the Viggiani (1981) solutions are given by Poulos (1999) for fixed ratios of soil strength above and below the failure surface (see Figure 2.40). The Viggiani (1981) solutions are used by Allison *et al* (1991) in the design of a pile stabilisation scheme for a landslide in South London, and in the stabilisation of a railway embankment reported by Carder and Temporal (2000). Guest *et al* (2000) briefly describe the design of landslide stabilising piles by assuming that the piles develop a plastic hinge.

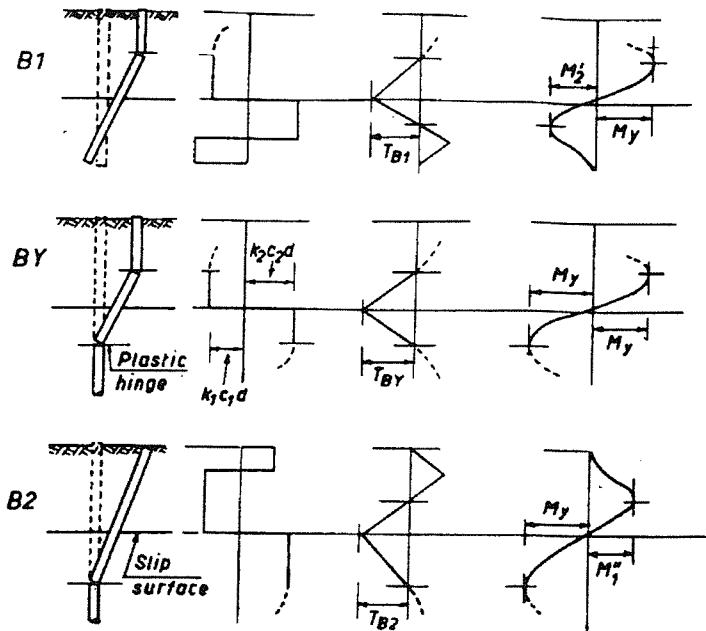


Figure 2.39. Soil pressure distributions and bending and shear diagrams for different failure modes assuming the formation of plastic hinges in the pile (taken from Viggiani, 1981). Notations as follows: d = the pile diameter, c = undrained shear strength of the soil, k = limiting lateral pressure coefficient, where the subscripts 1 and 2 denote the upper and lower soil layers respectively.

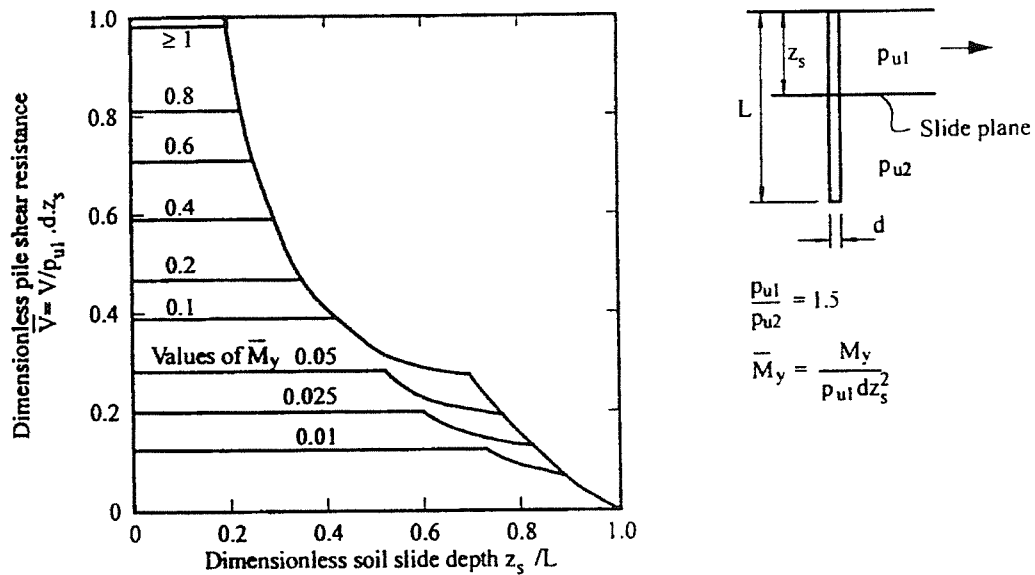


Figure 2.40. An example of a design chart based on the Viggiani (1981) solutions to pile stability (taken from Poulos, 1999). The chart incorporates the three failure modes shown in Figure 2.38. p_{u1} = the limiting lateral soil-pile pressure above the failure surface, p_{u2} = the limiting lateral soil-pile pressure beneath the failure surface plane, V = the shear resistance from the pile, M_y = the yield bending moment of the pile.

Limit equilibrium design methods that incorporate limit equilibrium slope stability often model the section of the pile beneath the failure surface as a laterally loaded pile (Ellis, 2002). The restoring force required to increase the factor of safety of the slope is applied to the pile head, which is assumed for the purpose of the calculation to be at the intersection of the failure surface and the pile. The Broms (1964a, 1964b) method of analysing laterally loaded piles is used. This is based on the limit equilibrium behaviour of the pile assuming that the pressure distribution along the entire length of the pile is the limiting lateral pile-soil pressure p_u . A check is carried out to determine the mode of failure for the pile - for long piles this tends to be through the formation of a plastic hinge within the pile section. Design charts linking the moment capacity of the pile section to the horizontal loading for different pile geometries and different pile failure mechanisms are presented (Broms, 1964a and 1964b). Determination of the pile failure mode requires knowledge of the soil and pile stiffness. The method takes no account of the portion of the pile in the failing soil mass, for which it is not possible to calculate the bending moments.

The pile-soil limiting pressure p_u effectively represents the condition of plastic failure, where the soil is squeezed around an individual pile. Neither calculation method above directly considers the spacing of the piles, but empirical interaction factors can be used to adjust the pile-soil limiting pressure p_u to account for pile interaction when the piles are spaced sufficiently close together (Chen and Poulos, 1997). Adjustments to the value of p_u could also be made to take into account the sloping ground surface (Chen and Poulos, 1997). However, as already discussed, the use of p_u to calculate pile bending and shear force is likely to result in large diameter piles with a large bending capacity spaced a significant distance apart.

Care has to be taken with the method of design where the portion of the pile passing through the failing mass of soil is ignored. The design assumes that the biggest bending moment occurs beneath the failure surface. However, the biggest bending moment can only be certain to occur beneath the failure surface if the pile is bending uniformly downslope and if the length of pile beneath the failure surface is longer than the length above.

2.5.4 Elastic design methods

Elastic methods have the advantage of linking the pressures acting on the pile to pile and slope displacements. A number of analytical and numerical pseudo-finite element methods can model the pile and slope displacement behaviour.

The more simple of these methods assume the portion of the pile beneath the failure surface to be embedded in an purely elastic continuum, and loading applied to the upper portion of the pile by the failing mass allows the calculation of pile deflections and hence the calculation of the bending moments and shear forces from the soil and pile stiffness.

Fleming *et al* (1985) present a method of calculating pile length and deflection for laterally loaded piles based on the results of a series of finite element analyses. The restoring force required to increase the factor of safety of the slope is applied to the pile head (which is assumed for the purpose of the calculation to be at the failure surface). Dimensionless charts can be used to find the bending moments and shear force in the lower portion of the pile (Figure 2.41). The key limitation of this method is that it makes no account of the behaviour of the section of the pile in the failing soil mass, which could generate a moment at the top of the lower portion of the pile.

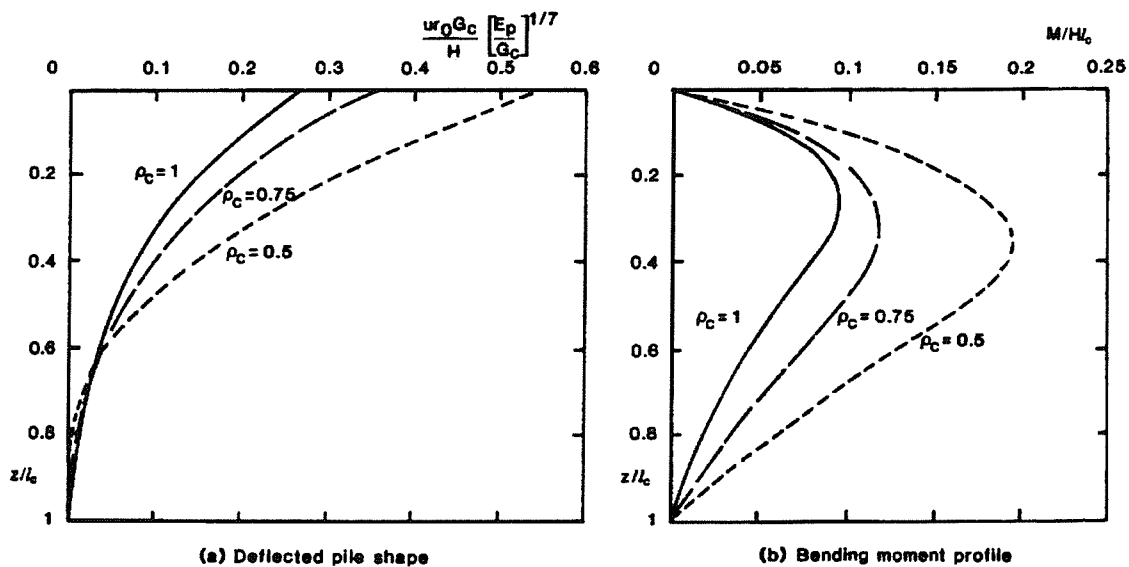


Figure 2.41. Charts for the design of laterally loaded piles (taken from Fleming *et al*, 1985). G_c is the average value of soil shear modulus over the length of the pile, ρ_c represents the degree of homogeneity in the soil stiffness (see Fleming *et al*, 1985), u is the lateral deflection, M is the moment in the pile, l_c is the critical length of the pile, E_p is the Young's Modulus of the pile, H is the lateral pile loading, r_0 is the radius of the pile.

Commercially available software for the analysis of laterally loaded foundation piles (such as STAAD Laterally Loaded Pile; STAAD, 2001) can be used in a similar way to the analysis method discussed above, with only the portion of the pile beneath the failure surface being modelled. Commercial analysis packages use either elastic continuum or p-y curves to calculate the reaction of the pile to being laterally loaded, but have the advantage of being able to deal with multiple soil layers and varied profiles of soil stiffness.

Analytical solutions presented by Ito *et al* (1981) assume that an infinitely long pile is embedded in an elastic continuum and that the layer of sliding soil applies a uniformly distributed load to the top portion of the pile. The analytical solutions provide the deflection, bending moment, shear force and net pressure diagram on the pile, and the equations given are easily programmed into a spreadsheet. However, the analytical solutions are for an infinitely long pile only, and the solutions do not extend to piles of finite length for which global stability considerations (such as force and moment equilibrium) are required. The method therefore makes no account of how the pile is likely to rotate. The profile of soil stiffness modulus E has to be constant with depth. Solutions are given for different pile head fixities. Hassiotis *et al* (1997) use a similar approach to calculate pile bending moments, but solve the equation for the pile section beneath the failure surface using a finite difference method to allow for variation in the elastic modulus with depth.

2.5.5 Pseudo-finite element approach

In this thesis, pseudo-finite element method is used to describe approaches in which a pile of finite length is modelled as a series of finite elements attached to a series of p-y curves (often simply defined as elastic-perfectly plastic) that represent the soil. The specification of p-y curves allows a plastic limit to be defined, at which the load on the portion of the pile reaches a maximum and then starts to re-distribute onto neighbouring elements. The modelling of the pile of a finite length by a series of elements also allows the global pile stability to be taken into account. The method will also model displacement of the upper layer of soil into the pile, much as happens in a real slope failure. The full pile behaviour is modelled, with the length of pile above the failure surface fully influenced by the restoring forces below. As a result, the approach allows calculation of the bending moment and deflections for the whole length of the pile. A full description of the pseudo-finite element method is given in Poulos and Davis (1980) (although they call it 'boundary element analysis').

There are a number of commercially available pseudo finite element software packages (for example ALP; Oasys, 2001). The limitation of such an approach is that for the purpose of design, the soil displacements pushing into the top of the pile have to be estimated (although in some packages, such as ALP, a uniformly distributed load can be applied to the pile head

instead). The advantage of applying soil displacements is that the true soil-pile relative displacement and loading distribution is modelled. Caroli (2003) shows that in the back analysis of instrumented piles on the M25 near Cobham (described in Section 2.7.2), pile behaviour is back analysed much more closely by applying the soil displacement profile in the pseudo finite element analyses rather than by a pressure distribution obtained from limit equilibrium slope stability (the difference being that in the first case passive loading is generated, and in the second case the loading is active, as the head of the pile is pushed into the surrounding soil). Chen and Poulos (1997) and Poulos (1999) also back analyse several instrumented piles using the pseudo-finite element method (by estimating the slope displacements), and find a close correlation with the measured pile bending moment and displacement (see Section 2.7.2 of this thesis). Poulos (1999) also used pseudo finite element analysis to demonstrate the basic pile bending modes shown in Figure 2.35.

Although in a design situation it will be almost impossible to predict the slope displacements likely to occur adjacent to the pile, the pseudo finite element method could be used to obtain a better understanding of how the pile will behave (such as the location of points of rotation, and the location of the maximum bending moment). By progressively increasing the soil displacements until the pile pressure reaches a maximum (at which the soil will flow around the pile), the ultimate pile bending condition may be found. Poulos (1999) demonstrates that for simple cases with uniform soil stiffness and soil shear strength above and below the failure surface, pushing the analysis to the ultimate condition replicates the limit equilibrium solutions found by Viggiani (1981) (Figure 2.38). Careful analysis of the pressures generated on the head of the pile by the soil movement could also be compared with the force required to stabilise the slope obtained from limit equilibrium slope stability analysis (Poulos, 1999). In the case of a landslide it is common to assume that the failing mass of soil has a uniform displacement profile with depth (Anagnostopoulos, 1991; Poulos, 1999).

Analysis of a row of piles either ignores the interaction effects of adjacent piles, assuming that the piles are spaced sufficiently far apart, or uses empirical interaction coefficients to adjust the pile behaviour.

Design charts are presented by Chen and Poulos (1997) based on results from a series of pseudo-finite element analyses. The dimensionless charts allow calculation of the maximum bending moment and pile head displacement from the pile geometry and pile and soil stiffnesses. Charts are presented for soil displacements at the head of the pile that are uniform and linear with depth. The charts assume fully elastic conditions that do not take into account plastic flow of soil around the pile, and in comparison with results from model tests and case studies considerably overestimate the bending moment at soil displacements larger than 10% of

the pile diameter (Chen and Poulos, 1997). The use of the charts is still limited by the need to input slope displacements.

2.5.6 Head condition of pile

The results of analytical and numerical parametric studies (Ito *et al*, 1981; Hassiotis *et al*, 1997; Cai and Ugai, 2001) show that piles restrained at the head against rotation and deflection exhibit substantially smaller bending moments (for example, Figure 2.42). The reduced bending moments often allow the use of smaller diameter and/or shorter piles. Rail embankment stabilisation schemes using spaced piles have tended to use pile caps restrained either by raking tension piles or ties passing between pile walls constructed on both sides of an embankment (Perry *et al*, 2001). Raking tension piles have proved popular because they provide a high degree of fixity to the piles. Ground anchors may also be used to restrain the pile head (for example, Ground Engineering, 1995; Brown *et al*, 1995).

Restraining the pile head will require connections between the vertical piles and the raking piles (or ground anchors) by means of a pile cap. In many cases, the cost of constructing the pile cap will offset any savings obtained from the use of smaller or shorter vertical piles. In the case of rail embankments cited earlier, the pile cap often has to be constructed to act as a small retaining wall to give better lateral support to the rail tracks and provide a safe walkway for railway workers (Andrei, 2000; Perry *et al*, 2001).

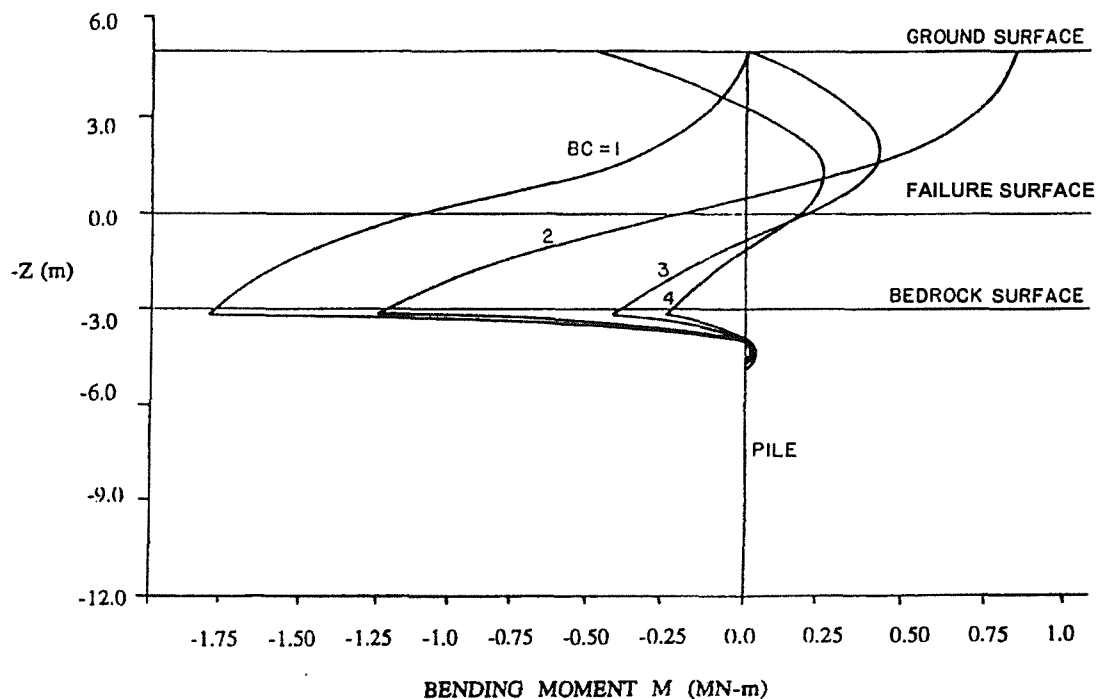


Figure 2.42. Bending moment results with and without fixed head restraints (taken from Hassiotis *et al*, 1997). The boundary conditions (BC) are: (1) free head; (2) unrotated head; (3) hinged head; and (4) fixed head.

Stabilising piles are often installed through granular berms built on embankment or cutting slopes to provide a working platform for large piling plant (Figure 2.6). The granular berm is then either left or regraded to cover the whole slope surface, and may provide a degree of deflection and rotational restraint to the pile head (Smethurst, 2002).

2.6 Three-dimensional finite element analysis

Discrete piles are an inherently three dimensional problem, and it is likely that three dimensional finite element analyses could be used in future pile design. The complex nature of the discrete pile problem, and the large number of variables that effect pile behaviour, mean that if more economic designs are to be achieved three-dimensional finite element analysis may be essential.

Currently, three dimensional finite element analyses require considerable computing time, often taking several days to arrive at a solution. Construction of the models and analysis of the results can be time consuming, and require experienced numerical modellers. The advantage of finite element analysis is that it may be used to assess the serviceability state of a proposed solution as well as the ultimate limit state. In addition, complex models can be used which incorporate long term pore pressure equilibration, small strain stiffness and stress history, all of which would be very difficult to incorporate into a simpler analysis.

It is evident that with further advances in computer technology finite element analyses could be used as a design tool for discrete piles. The flexibility of the method should make it possible to overcome many of the limitations inherent in other design approaches - particularly the difficulty of incorporating the effects of pile spacing.

This section describes four studies in which discrete pile stabilised slopes have been modelled using the 3D finite element method. All of the studies were undertaken as academic and parametric studies to understand pile and slope behaviour under a series of slope and pile geometries and soil conditions (except Hayward *et al*, 2000, who attempted to back analyse model tests). There have as yet been no attempts to back analyse the behaviour of a real pile stabilised slope using 3D finite elements – yet such analyses are probably essential if sufficient confidence is to be gained in the use of 3D finite elements as a feasible design tool.

2.6.1 The use of finite element analysis for assessing slope stability

It has now been widely demonstrated that finite element analysis can be used to accurately model slope stability (Potts *et al*, 1990; Matsui and San, 1992; Potts *et al*, 1997; Griffiths and Lane, 1999).

Furthermore, the use of finite element analysis in the analysis of slopes has several advantages over conventional limit equilibrium slope stability – and these have been described by Griffiths and Lane (1999):

- no assumption needs to be made in advance about the location or shape of the failure surface;
- assumptions about interslice forces required in limit equilibrium methods are not necessary;
- if realistic soil stiffness data is available, the finite element data will give deformations at working stress levels; and
- the finite element method can model time dependent progressive slope failure.

The shape of the critical failure surface and the time to reach failure is heavily influenced by stress history, and the *in situ* stress ratio K_0 (Potts *et al*, 1997). The *in situ* stresses are markedly different for embankments, cut slopes and natural slopes. Limit equilibrium slope stability methods ignore stress history, whereas finite element analyses are able to incorporate it and will give different failure mechanisms for the three types of slope.

The ‘strength reduction method’ is commonly used in finite elements analysis to obtain a factor of safety for the slope (Matsui and San, 1992; Griffiths and Lane, 1999; Cai and Ugai, 2000). The factor of safety of the slope may be defined as the number by which the original shear strength parameters must be divided to just cause failure of the slope (the same as for limit equilibrium slope stability). The factored strength parameters are therefore:

$$c'_f = \frac{c'}{FOS} \quad \text{Equation 2.8}$$

$$\phi'_f = \tan^{-1} \left(\frac{\tan \phi'}{FOS} \right) \quad \text{Equation 2.9}$$

The finite element analysis can then be run repeatedly, with the soil strengths incrementally reduced until the factor of safety at failure is found.

A slight difficulty of using finite element analysis can be defining exactly what constitutes slope failure, although failure is normally associated with numerical non-convergence and will be accompanied by a dramatic increase in nodal displacements (as demonstrated by Griffith and Lane, 1999).

2.6.2 Studies of discrete pile stabilised slopes using 3D finite element analysis

Hayward *et al* (2000)

Hayward *et al* (2000) carried out 3D finite element analyses using the program CRISP to try and replicate the behaviour of piles placed in a model slope tested in a geotechnical centrifuge. Some problems were encountered with these analyses, including the presence of a stiffer upper layer (possibly caused by negative pore pressures) that caused reverse curvature at the pile head not seen in the model tests. Furthermore, the pile spacing seemed to have little effect on the slope displacements or the bending moments in the pile, which is contrary to the behaviour in the centrifuge model tests, in which both slope displacements and pile bending moments increased with pile spacing (Hayward *et al*, 2000). Displacement of the piles in the numerical analyses was also much larger than those observed in the centrifuge, and the piles in the CRISP analyses appeared to be increasing the depth of the failing mass of soil. The reasons why the numerical models failed to replicate the behaviour of the model piles in the centrifuge is unclear.

Carder and Easton (2001)

3D finite elements have been used by Carder and Easton (2001) to model piles used to stabilise cutting and embankment slopes within overconsolidated clays typical to the UK.

The slope geometry used by Carder and Easton is given in Figure 2.43, and is typical of highway infrastructure slopes in the UK. The soil strength, stiffness and permeability parameters are given in Figure 2.44, and are typical of commonly occurring over-consolidated clays, such as the London Clay.

The mesh used in the finite element model is shown in Figure 2.45. The authors make no mention of slip elements being used to allow flow of soil material around the pile, and it is assumed that the whole model was therefore a continuum in which the soil elements were attached to the pile elements. The analyses carried out by Carder and Easton (2001) investigated the serviceability behaviour of the piles, where relative soil-pile displacements are likely to be small, and the incorporation of slip elements would probably have made little difference to the result of the analyses.

Carder and Easton (2001) modelled the behaviour of piles installed within a slope in a preventive role – i.e. where piles are installed because the slope is thought likely to fail at some stage in the future. They analysed a cut slope in which excavation and heave of the clay material is modelled, and the pore pressures allowed to return towards the equilibrium condition over a

period of 60 years. The piles were installed at the end of the cutting excavation stage. Analyses were run for both piles placed into the slope at 3 and 6 diameters centre to centre. Two different stiffnesses (soil types A and B, Figure 2.50) have been used for the cutting slope materials.

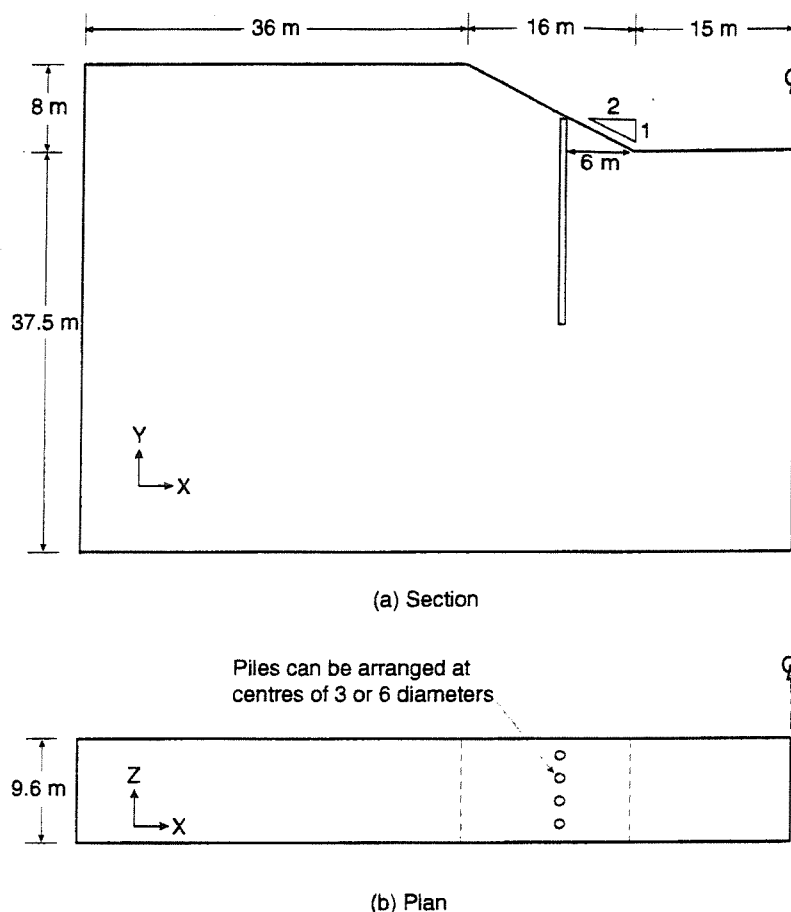


Figure 2.43. Slope geometry used by Carder and Easton (2001)

Description	Soil type A (stiff clay)	Soil type B (soft clay)
Elastic modulus, E' (MPa)	6.6	4
Rate of increase with depth, mE' (MPa/m)	3.2	0.5
Angle of friction, ϕ' ($^\circ$)	20	20
Cohesion, c' (kPa)	12	12
Poisson's ratio, ν	0.2	0.2
Horizontal permeability, k_x (m/year)	0.015	0.015
Vertical permeability, k_y (m/year)	0.003	0.003

Figure 2.44. Soil strength, stiffness and permeability parameters used by Carder and Easton (2001)

The bending moments in the piles inserted into the stiff soil were found to be smaller than those in the soft soil, as the latter gave much larger slope displacements after excavation. For both the stiff and soft clays, the bending moments were almost the same for the piles spaced at 3 pile spacings and 6 pile spacings after a period of 60 years (Figure 2.46). The reason that the bending moments were similar is because the piles have been inserted into a slope which

remains stable (Carder and Easton give a factor of safety for limit equilibrium slope stability for the unreinforced slope of 1.32). The piles were therefore carrying load due to movement of the slope post excavation (the movements were quite large as the soil stiffness was relatively low) rather than because the slope was actually failing. The piles are shown to have very little effect on the lateral displacement of the slope when compared with the unreinforced slope (Carder and Easton, 2001).

Piles were also modelled in a steeper cutting slope of 1v:1.5h. This also remained stable, having a factor of safety of 1.17 calculated using limit equilibrium slope stability analysis. After 60 years its factor of safety may be closer to 1.0, and there is a bigger difference between the bending moments in the piles at 3 and 6 pile spacings. The effect of toeing the piles into a stiff underlying stratum was also considered by Carder and Easton (2001) and this acted to reduce the displacements of the pile, but led to a very large bending moment occurring at the interface between the soft and stiff layers.

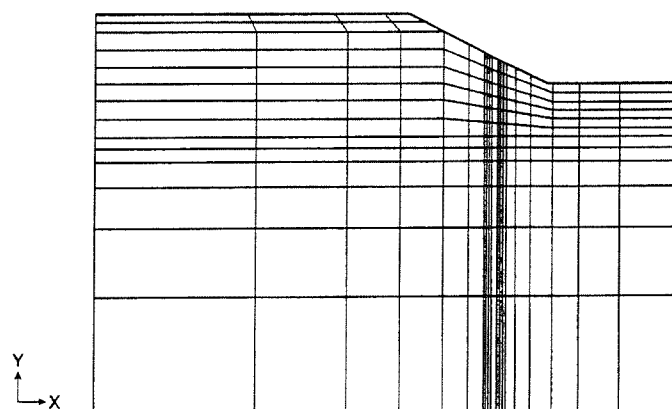


Figure 2.45. Cross-section through the mesh used in the finite element analyses of pile stabilised slopes by Carder and Easton (2001)

Carder and Easton (2001) also modelled piles installed into an embankment slope. The embankment fill material was the soft soil (B) and the underlying ground the stiff soil (A). The piles placed into the embankment slope appear to have had a larger effect in reducing slope movements than the piles placed in the cutting slope. The piles placed at a spacing of 6 diameters centre to centre exhibited a substantially larger bending moment and a larger movement of soil between the piles than those placed at 3 diameters centre to centre.

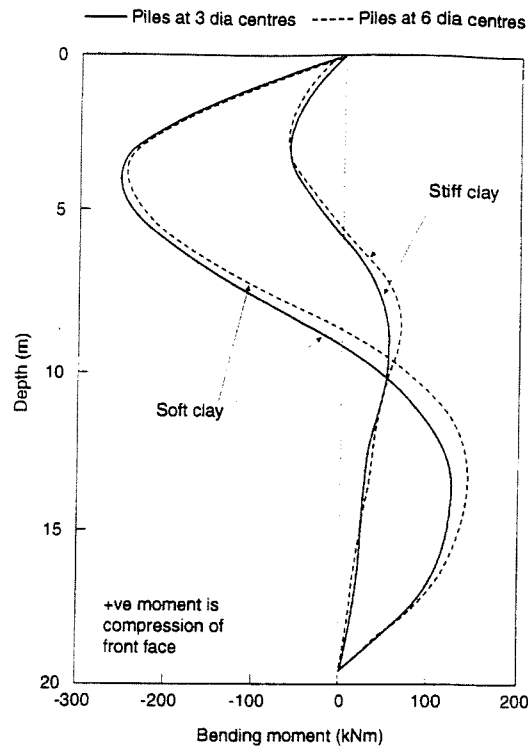


Figure 2.46. Comparison of bending moments from analysis on stiff and soft cut clay slopes (taken from Carder and Easton, 2001)

The analyses run by Carder and Easton (2001) show the impact that soil stiffness and a stiff layer at depth have on the bending behaviour of the piles placed within typical infrastructure slopes. However, Carder and Easton (2001) investigated only the pre-failure behaviour of the piles, and did not extend the study toward understanding pile behaviour under strain softening and slope failure (where the bending moments would be expected to be larger, and the pile spacing effects more critical). The analyses are of interest, but Carder and Easton (2001) investigated a fairly specialist condition (pre-failure for piles used in a preventative role).

Oakland and Chameau (1989)

Oakland and Chameau (1989) present the results of a parametric study using 3D finite element analyses of bored piles used to stabilise slopes. They consider piles placed both at mid-slope and at the crest in 2h:1v and 1h:1v slopes. The slope geometry and mesh used is shown in Figure 2.47. The 3D mesh used by Oakland and Chameau (1989) made optimal use of symmetry such that the slice of the slope used in the analyses comprises half a pile and half of the soil between the piles. In all of the analyses the 1.22m diameter piles were spaced at 2.44m centre-to-centre. The pile was assumed to be embedded in to a rigid underlying bedrock 18m below the crest of the slope, which was reproduced in the analysis by fixing the pile at the base of the finite element mesh.

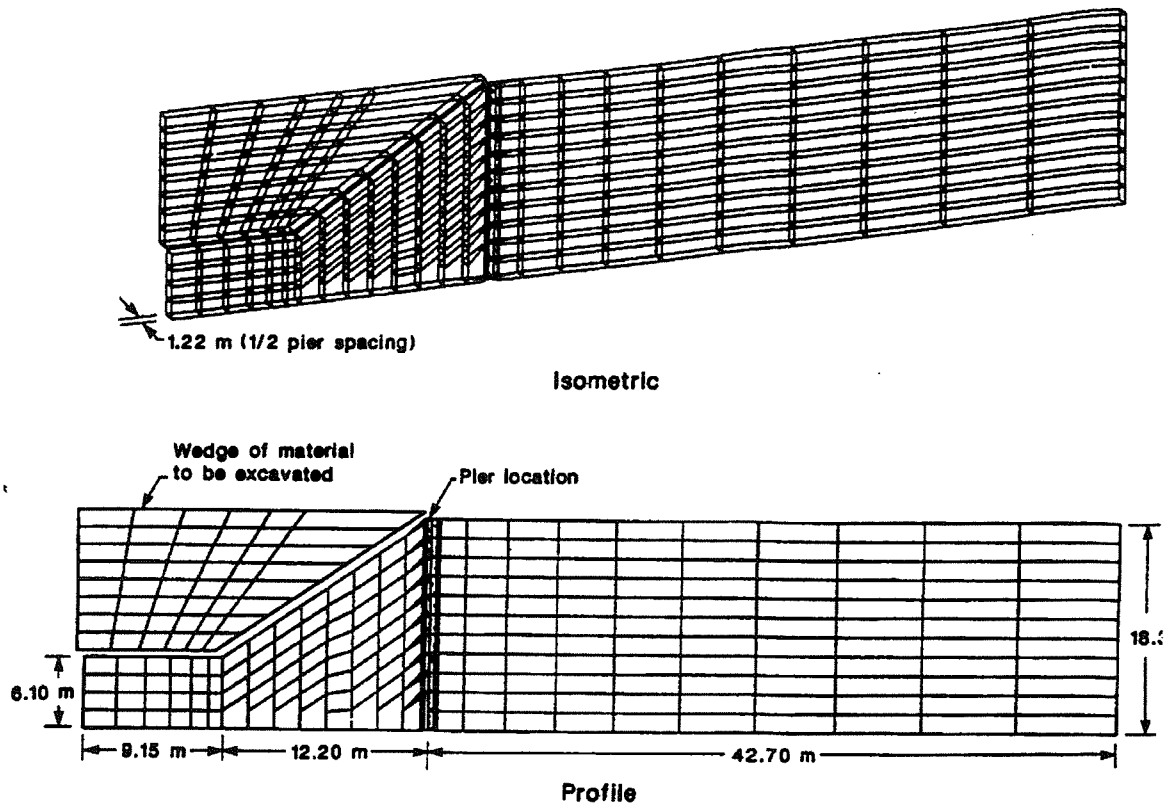


Figure 2.47. The slope geometry and mesh used by Oakland and Chameau (1989)

In the first set of analyses presented by Oakland and Chameau (1989), the pile was positioned at either the crest or the mid-slope in a 1h:1v slope 12m high and in an undrained soil. In both cases the pile was found to stabilise the slope and reduce slope movements considerably. The pile placed at mid-slope was substantially more effective, with the pile placed at the crest failing to prevent large movements downslope. Shear stresses were reduced significantly in the stabilised slopes. Element nodal loads were used to calculate the pressure acting on the piles. For the pile placed at the crest of the slope, the pressure acts in a net downslope direction along the entire length of pile extending from the bedrock.

The increase in the factor of safety of the pile stabilised slope was assessed using shear stress distributions from the finite element analysis to carry out limit equilibrium slope stability analyses on the reinforced and unreinforced slopes. This demonstrated that addition of the piles to the slope increased its factor of safety from 1.0 to 1.7.

This study demonstrates that piles can stabilise a slope when placed at midslope. However, fixing the piles at the base of the mesh models quite a specific case - a firm underlying bedrock into which the piles can be embedded is often not present in reality.

Cai and Ugai (2000)

Cai and Ugai (2000) compare the results of three-dimensional finite elements with results from limit equilibrium slope stability analyses. The mesh and slope geometry used in the analysis is shown in Figure 2.48. Soil strength and stiffness parameters are shown in Figure 2.49. The piles were assumed to be toed into a bedrock at depth, and the model pile was fixed in all directions at the base of the mesh. The mesh made maximum use of symmetry and only half a pile and half a gap between adjacent piles was modelled. Interface elements were placed between the soil and pile to allow slip and opening of a tension crack to occur (Cai and Ugai, 2000).

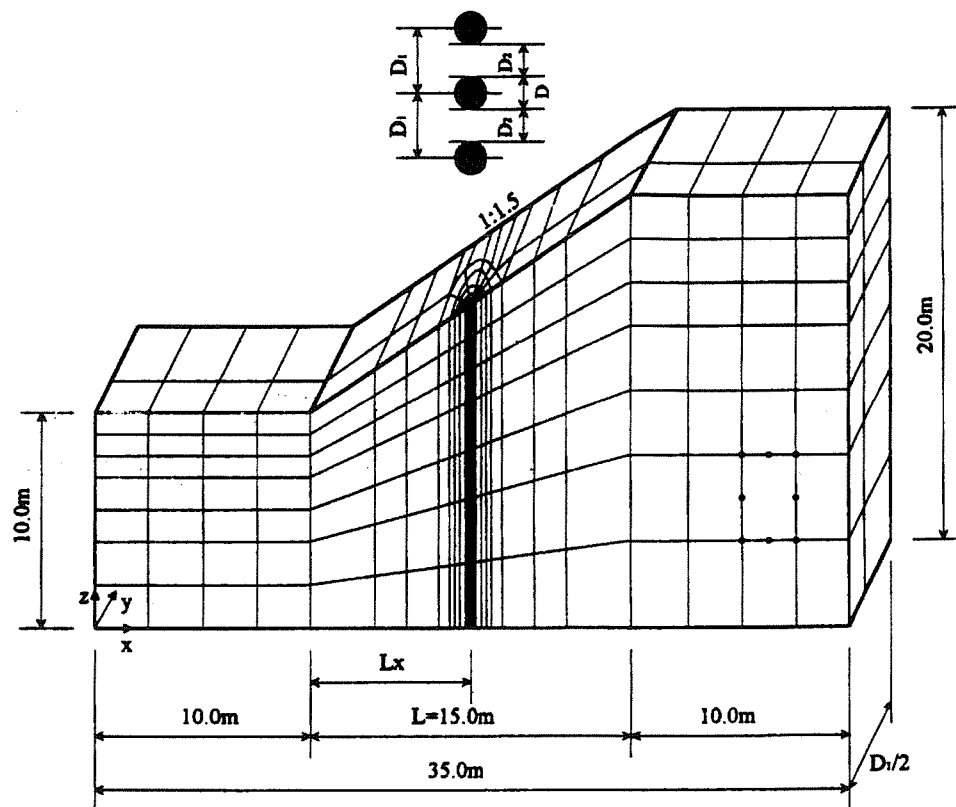


Figure 2.48. The slope geometry and mesh used by Cai and Ugai (2000)

Parameter	Soil	Interface	Pile
Young's modulus, E (MPa)	200	200	60000
Poisson's ratio, ν	0.25	0.25	0.20
Unit weight, γ (kN/m ³)	20.0		
Cohesion, c (kPa)	10.0	10.0	
Friction angle, ϕ (°)	20.0	20.0	
Dilatancy angle, ψ (°)	0.0	0.0	

Figure 2.49. Soil strength and stiffness parameters used by Cai and Ugai (2000)

A uniform soil stiffness of 200 MPa was used in the analysis. The soil strength and stiffness was representative of a heavily overconsolidated mudstone material or fractured soft rock mass. The stiffness profile used in the analyses by Cai and Ugai (2000) is plotted in Figure 2.50 with the two stiffness profiles used by Carder and Easton (2001).

Analyses were run with piles spaced at 2,3,4 and 6 pile diameters. Different pile head fixities were used, and analyses were run with piles free, hinged, unrotated and fully fixed at the head. Cai and Ugai (2000) ran the analyses until they reached an initial equilibrium, at which point the soil strength was progressively reduced until each of the pile stabilised slopes suffered failure. Figure 2.28 shows the results from the free head pile at different pile spacings just before failure of the pile-stabilised slope occurs. The piles spaced closest together (2 pile spacings) underwent the most deflection and have the largest pile bending moments (this is probably a reflection of the lower soil strength at pile-slope failure in the model at 2 pile spacings, when compared with those at 3 and 4 pile spacings). The piles at 4 and 6 pile spacings had the least deflection and smallest bending moments.

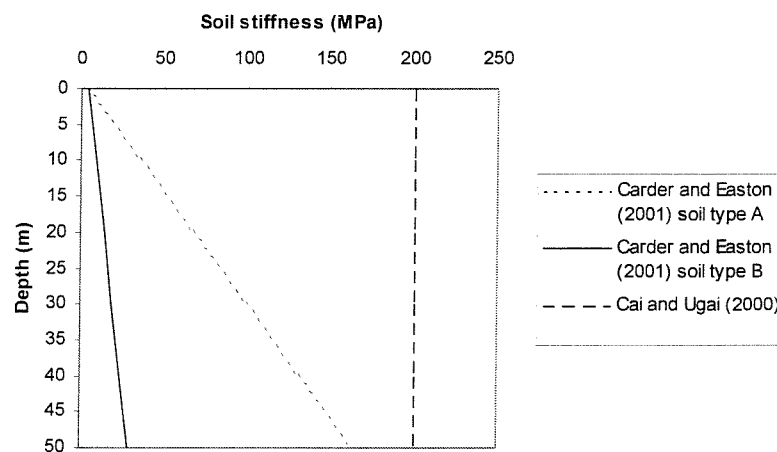


Figure 2.50. Plots of soil stiffness used in finite element analyses by Cai and Ugai (2000) and Carder and Easton (2001)

Pile deflections were much smaller than those reported by Carder and Easton (2001) in less stiff soils, and the bending moments much larger - this reflects the huge differences in the soil stiffness used and the pile base fixity used by Cai and Ugai (2000). Cai and Ugai (2000) compare the results from their finite element analyses to limit equilibrium analyses in which the load acting on the slope from the pile is calculated using the Ito and Matsui (1975) method, and then put into limit equilibrium slope stability to find the factor of safety of the pile stabilised slope.

Cai and Ugai (2000) plot graphs of the pile location within the slope (placed at intervals from the toe to the crest) against the factor of safety of the slope obtained by reducing the soil

strength parameters until slope failure occurs (Figure 2.51). For piles placed towards the top of the slope, there is some difference between the factor of safety calculated using limit equilibrium and that calculated using finite element analysis. This is probably to be expected, as the limit equilibrium analysis does not incorporate the full pile-soil interaction and the 3D nature of the problem included in the finite element analyses (Cai and Ugai, 2000). The highest factor of safety for slope stability was found when the pile is placed at midslope.

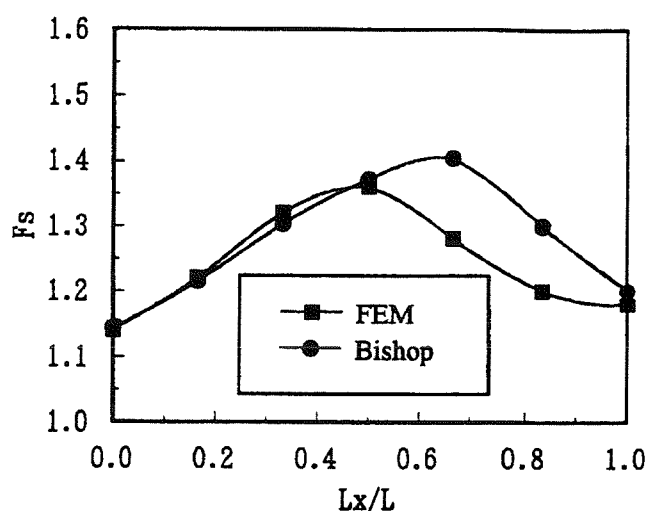


Figure 2.51. Factor of safety against the location of the pile in the slope: results for a three-dimensional finite element model and limit equilibrium calculation for the pile stabilised slope (Cai and Ugai, 2001). L is the length of the slope and L_x the distance from the toe of the slope to the pile. F_s is the factor of safety against any significant failure taking place in the slope. The soil strength parameters used throughout the slope were $\phi = 20^\circ$ and $c' = 10\text{kPa}$.

2.7 Case histories

The uncertainties and difficulties of discrete pile design make the study of as constructed pile behaviour an important guide on how future pile designs are likely to perform. Indeed, much pile design is currently based on simple mechanics and a process of observation of the behaviour of existing pile stabilised slopes to give a continuous design improvement. Of considerable interest therefore are i) the design methods used by other practitioners in the design of piles, and ii) the results from instrumentation installed in pile stabilised slopes to assess pile and slope behaviour post construction.

Detailed case histories of piles used to stabilise cutting and embankment slopes are infrequent, but those that do occur in the literature are described below. Many only describe the procedure used for the design of the piles, with no record of how the piles behaved post construction. Of those that do describe the instrumentation of pile stabilised slopes, the instrumentation is often limited (probably by financial constraints), and the full picture of pile and slope behaviour is not

obtained. However, most do provide basic information on pile bending and deflection behaviour in relation to the size and type of mass movement.

This review of case histories is divided in two sections, first those case histories that describe basic pile design, and second those that go on to describe the post construction behaviour and monitoring results.

Table 2.3 lists all of the case histories reviewed, together with key parameters including pile diameter, length and spacing, and the size of mass movement.

2.7.1. Case histories describing pile design methods

Case 1. Huy, Belgium. De Beer and Wallays (1970)

De Beer and Wallays (1970) consider the design of piles for a failing railway cutting slope, in Huy, Belgium. The cutting was to be widened to accommodate an additional railway track, and the new slope was cut through schists parallel to the existing cutting slope. The new cut triggered movements along a series of pre-existing shear surfaces that were caused by slope movement during the original cutting construction. Stabilisation of the slope was to proceed using two rows of discretely spaced bored concrete piles, the first placed in a row at the toe of the slope and second placed in a row just below midslope (Figure 2.52). The upper row of piles were designed to be 1.5m in diameter, 20m long and installed at a spacing of 1.34 pile diameters (which is close and may almost be considered as a wall). The row of piles placed at the toe to help retain the steeper lower portion of the slope were to be either 1.07m or 1.28m diameter depending on the moment resistance required, 10m long and placed at a spacing of 3m (2.8 or 2.3 pile diameters).

Limit equilibrium slope stability analysis was used to obtain the restoring force that the piles needed to provide to the failing mass of soil. The pile stability and required bending moment capacity were calculated using a simple limit equilibrium calculation based on the lateral limiting pile-soil pressure distribution proposed by Brinch Hanson (1961) and by assuming the pile to rotate about a point close to its base. As a check, design was also carried out using lateral pressure distributions assuming the piles to act as a solid wall (a modification of the method proposed by Brinch Hansen, 1961) and a pressure distribution based on the soil having a rupture surface around the pile as proposed by Jaky (1948). The method proposed by Jaky (1948) assumes that the piles are sufficiently far apart not to interact, which is unlikely at the pile spacing proposed. Factors of safety against pile instability were applied to the shear strength parameters. There is no record of how the piles or slope behaved post construction.

Location of scheme and paper from which case history originates	Type of slope	Depth to failure surface (m)	Type of pile	Diameter of pile (m)	Spacing of piles centre to centre (pile dia)	Length of pile (m)	Data on pile performance after construction
Huy, Belgium. De Beer and Wallays (1970)	Cut	15	Reinforced concrete	1.07 - 1.50	1.34-2.80	9.9 - 20.0	No
Italy, Esu and D'Elia (1974) and Chen and Poulos (1997)	Natural	7.5	Reinforced concrete	0.79	Single pile	30	Yes
Katamachi, Japan. Ito and Matsui (1975)	Natural	8.2	Reinforced concrete tube	0.30	6.66	> 12	Yes
Higashitono, Japan. Ito and Matsui (1975)	Natural	6.0	Tubular steel pipe	0.32	6.28	> 12	Yes
Kamiyama, Japan. Ito and Matsui (1975)	Natural	6.2	Tubular steel pipe	0.32	6.28	> 12	Yes
Stahlberg, Germany. Sommer (1979)	Natural	22	Reinforced concrete tube	3.00	3.00	27	Yes
Geislingen, Germany. Winter <i>et al</i> (1983) and Gudehus and Schwarz (1985)	Cut	3.5	Reinforced concrete	0.42	9.52	8	No
Dautenhiem, Germany. Gudehus and Schwarz (1985)	Embankment	8	-	1.50	2.53††	13	No
Italy, Carrubba <i>et al</i> (1989) and Chen and Poulos (1997)	Natural	9.5	Reinforced concrete	1.22	Single pile	22	Yes
Kuala Lumpur, Ting <i>et al</i> (1990)	Natural	15	Reinforced concrete	0.25	3.00 [†]	20	Yes
Gypsy Hill, London. Allison <i>et al</i> (1991)	Natural	4.5	Reinforced concrete	0.60	4.67	10	Yes
Greece, Kalteziotis <i>et al</i> (1993)	Natural	4	Reinforced concrete	1.00	2.50	12	Yes
Florence, Alabama, USA. Brown <i>et al</i> (1995).	Cut	-	Steel tube filled with reinforced concrete	0.15	2.80	-	No

Table 2.3. Summary of key parameters taken from pile stabilised slope schemes described in the literature (table and annotations continue over).

Location of scheme and paper from which case history originates	Type of slope	Depth to failure surface (m)	Type of pile	Diameter of pile (m)	Spacing of piles centre to centre (pile dia)	Length of pile (m)	Data on pile performance after construction
Newcastle, Australia. Poulos (1995)	Cut	-	Reinforced concrete	1.20	2.66 - 4.16	6 – 12	No
Todmorden, West Yorkshire, UK. Carder and Temporal (2000)	Embankment	-	Reinforced concrete	0.90	4.44 ^{†‡}	15	No
Runswick Bay, Yorkshire. Guest <i>et al</i> (2000)	Natural	10 - 20	Grouted steel tubular pipe	0.32 – 0.40	11.25 [§]	15 - 25	No
Istanbul, Turkey. Ergun (2000)	Natural	6	Reinforced concrete	1.20	2.00	16	No
Pusan, Korea. Hong and Park (2000)	Cut	10 - 20	Reinforced concrete	0.60	2.50 – 5.00	-	No
Cobham, Surrey, UK. Carder <i>et al</i> (2003)	Cut	10	Reinforced concrete	1.05	2.38	16	Yes
Hildenborough, Kent, UK. Chapter 3 of this thesis	Embankment	-	Reinforced concrete	0.60	4.00	10.0	Yes
[†] Two rows of piles in the slope, staggered so that the lower row piles are placed midway between the upper row [‡] Spacing between staggered piles [¶] This is the spacing between clusters of 2 or 3 piles [§] Several rows of piles placed in a diamond grid layout							

Table 2.3 (cont.) Summary of key parameters taken from pile stabilised slope schemes described in the literature

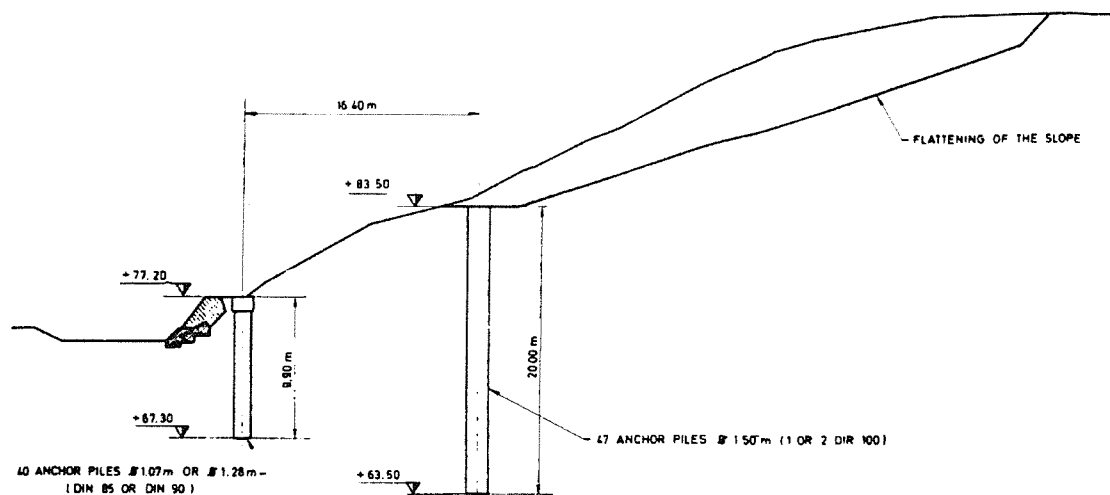


Figure 2.52. Layout of the stabilising piles in cutting slope at Huy, Belgium (taken from De Beer and Wallays, 1970)

Case 2. Newcastle, Australia. Poulos (1995)

Poulos (1995) describes the design of piles for slopes at a road interchange in Newcastle, Australia. Ramps for an overbridge were constructed on top of slopes forming the main cutting for the highway. Spaced piles were needed to stabilise the slopes as constraints on landtake meant they needed to be steeper than the available soil strengths allowed for (all of the slopes had a factor of safety less than the required 1.5). The geology through a typical cross section of the site is illustrated in Figure 2.53, along with the pile position and design failure surfaces. The final design used 1.2m diameter piles in a single row at midslope, with the pile spacing varying between 3.2m and 5m, or 2.67 and 4.17 diameters centre to centre, depending on the restoring force required. The piles were designed in two stages, the first using limit equilibrium slope stability to obtain the critical failure surface in the slope and to determine the restoring force that the piles needed to provide. The second stage determined the bending moment and shear force acting in the piles using a simplified pseudo finite element analysis that considered the effect of soil moving past the pile. A trial and error method was used to determine the best pile location, diameter and spacing. There is no record of how the piles or slopes behaved post construction.

Case 3. Yorkshire, UK. Carder and Temporal (2000)

Carder and Temporal (2000) document the discrete pile stabilisation of a railway embankment at Kitson Wood, Yorkshire, UK. The embankment is made up of 12m depth of ash fill overlying reworked mudstone fill, which in turn overlies intact weathered mudstone. Geotechnical investigation indicated that the embankment was only marginally stable, with a danger of deep seated failure. Two rows of 900mm diameter, 15m long discrete piles were constructed towards the toe of the slope, and reinforced with universal steel columns (Figure 2.54). The two rows of piles were 3.5m apart with the piles staggered at a spacing of 4 pile diameters. The piles pass through the mudstone fill to toe into the intact mudstone. The pile design used the method

presented by Viggiani (1981) (see Section 2.5.3) with the piles designed to apply sufficient force to the slope to increase the factor of safety found using limit equilibrium stability analysis.

A minipile wall was also constructed at the crest of the embankment to provide additional support to the tracks and lineside services.

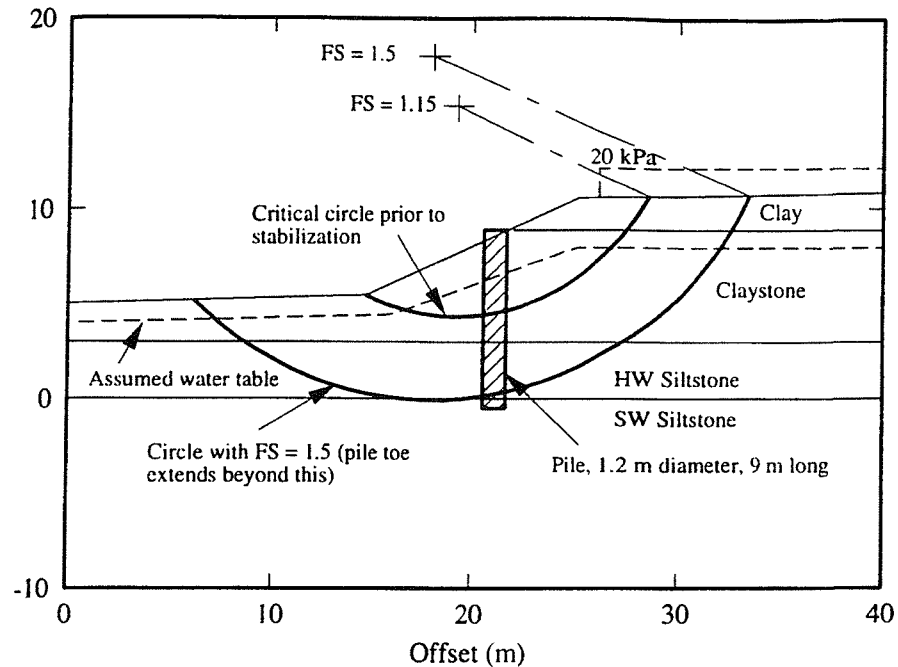


Figure 2.53. A typical cross section through the pile stabilisation scheme at Newcastle, Australia (taken from Poulos, 1995).

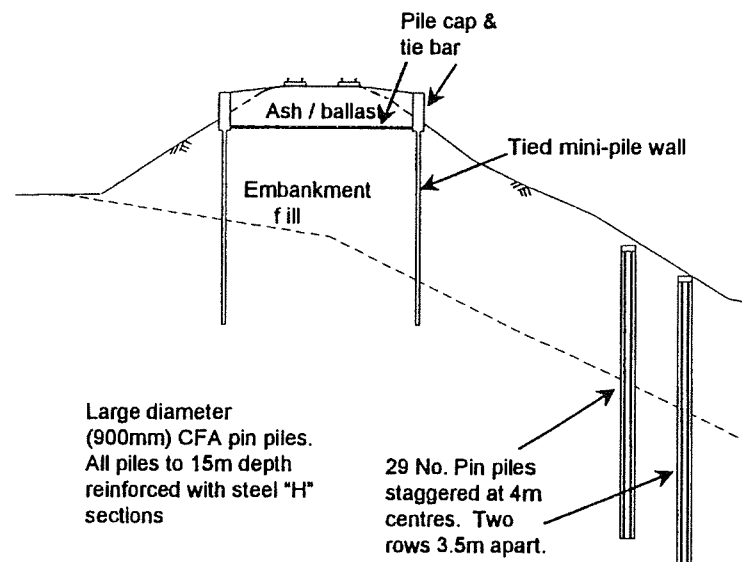


Figure 2.54. Cross section through the pile stabilised embankment at Kitson Wood, Yorkshire (taken from Carder and Temporal, 2000).

Case 4. Dautenheim, Germany. Gudehus and Schwarz (1985)

Gudehus and Schwarz (1985) describe the stabilisation of an embankment slope at Dautenheim, Germany, using two rows of discretely spaced piles. The 8m high embankment suffered slowly creeping deep seated failure. Inclinator measurements identified the position of the failure surface, which is shown in Figure 2.55. The pile layout, diameter and spacing are also shown in Figure 2.55.

The piles were designed using a method in which the soil is assumed to be a very viscous fluid and the piles are designed to bring about a reduction in the creep rate of the slope (this is explained further in Winter *et al*, 1983). The piles at Dautenheim were found to reduce the creep rate from 0.15cm/day to almost zero.

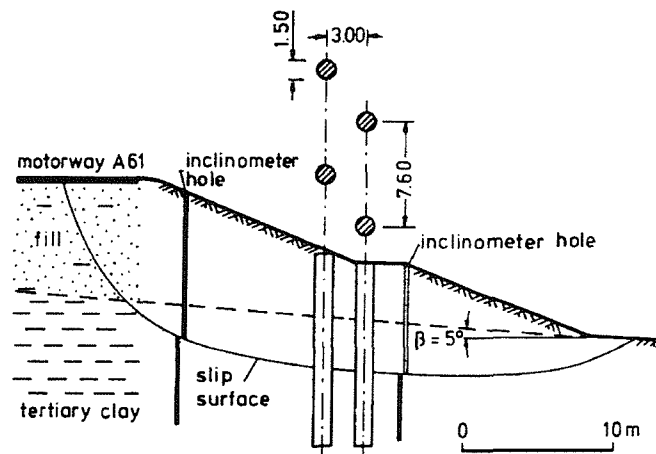


Figure 2.55. Cross section through the pile stabilised embankment at Dautenheim, Germany (taken from Gudehus and Schwarz, 1985).

Case 5. Pusan, Korea. Hong and Park (2000)

Hong and Park (2000) describe the design of stabilising measures for cut slopes constructed as part of the excavation for the Hwangryung mountain youth hostel. Parts of the slopes had collapsed, and there was concern that others might be on the verge of failure.

The geology was a mix of sandstone, mudstone and calcareous shale, highly weathered close to the surface, with randomly orientated joints and fissures spaced at 7-15cm and mostly infilled with soft weathered soils. The soft soil filled fissures were thought to be the cause of most of the instability. Geotechnical testing was used to obtain soil strength and stiffness parameters.

Stability analyses found many of the slopes to be close to failure, with projected failure surfaces running on the boundaries between soft weathered rock materials and more competent underlying geology.

One of the methods used to stabilise the slopes was the installation of discretely spaced piles, 600mm diameter and made of concrete reinforced with a universal steel column. Depending on the resistance required, the piles were spaced between 1.5 and 3.0m apart (2.5 – 5 pile diameters centre-to-centre). The piles were designed to penetrate into the hard rock by at least 3m, and increase the factor of safety of the slope to at least 1.3. Sections of the slopes within the vicinity of the piles were treated using shotcrete held down with rock bolts, with the intention that this would protect the slopes from further weathering and the weathered material from being washed out during heavy rainfall.

A plan of the site showing the location of the stabilising piles is given in Figure 2.56. Hong and Park (2000) state that some of the piles were instrumented to measure bending and displacement, but unfortunately do not go on to describe any results.

Case 6. Runswick Bay, UK. Guest *et al* (2000)

Guest *et al* (2000) describe the stabilisation of a large coastal landslide at Runswick Bay in North Yorkshire. The section of the north Yorkshire coastline around Whitby is actively eroding and made up of shales overlain by sandstones and Glacial Till. The village of Runswick Bay is situated in embayment formed as a result of historic coastal erosion.

The seawall to the south of the village had been moving as a result of beach erosion and landslide activity at a rate of 10-30mm/month, until winter 1998, when an increase in the rate of movement to as much as 100mm/month was observed.

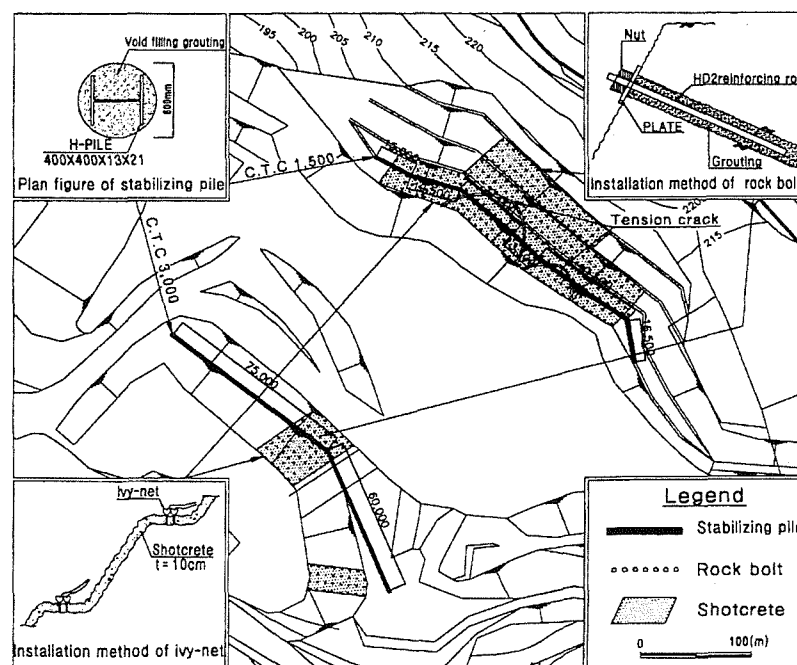


Figure 2.56. A plan of the Hwangryung mountain site showing the location of pile rows and shotcreting (taken from Hong and Park, 2000)

The landslide movements extended back 300m from the sea wall, and were causing damage to an area of car parking and a section of public highway. Investigation of the landslide showed Glacial Tills sliding over fractured shale materials, with the failure plane at a depth between 10 and 20 m. There was concern that if the seawall completely failed, the landslide would retrogress and extend even further back than the 300m already moving (Guest *et al*, 2000).

Remediation works included construction of a new seawall protected from the sea by a rock armour revetment. The landslide was stabilized using a combination of slope re-profiling, drainage works including french and counterfort drains, and 500 bored piles.

The piles were augured using a continuous flight rig. They were constructed between 10 and 25m in length (depending on the resistance required) and reinforced by either 406 mm diameter or 324 mm diameter mild steel tubular sections, vibrated into a borehole filled with C20 sand/cement grout. The piles were spaced at 4.5m centres on a diamond grid layout. Each pile was constructed to penetrate at least 5m into the intact underlying shale bedrock (Guest *et al*, 2000). The piles were designed to give a maximum lateral resistance to landslide movement on the formation of a plastic hinge above and below the failure surface.

The regrading of the slope into a series of benches increased the factor of safety of the slope failure by 5%. The trench and counterfort drains were designed to reduce groundwater levels across parts of the site by up to 1.5 m, and contribute an additional 5% increase to the factor of safety. The piles were installed to increase the factor of safety by an additional 15%, so as to give a global factor of safety of 1.25 (Guest *et al*, 2000).

The performance of the stabilizing measures have been subsequently monitored using piezometers, inclinometer tubes installed in piles and the slope, and precise survey techniques to measure displacements. However, Guest *et al* (2000) give no details of the performance of the slope post construction.

2.7.2. Case histories in which piles have been instrumented to record behaviour

Case 7. Godstone, Surrey. Ground Engineering (2001), Carder *et al*, (2003)

A cutting slope failure on the M25 near Godstone, Surrey, UK, was reported by Ground Engineering (2001) and Carder *et al* (2003). The slope failure occurred after a particularly heavy period of rainfall in December 2000. Geotechnical investigation of the site found that the failure surface was 10m below ground level, and that the failure extended 80m up the slope from the hard shoulder of the M25 (Figure 2.57). As a result of the initial slope movement, the hard shoulder suffered a heave of 25mm (Carder *et al*, 2003). The slope comprises stiff fissured Gault Clay overlain by Head Deposits. The lower slopes of the North Downs escarpment have a history of slope movement, and geological and geomorphological mapping found various fossil landslide features. The slope suffered a deep seated failure even though the cut slope angle was only about 14° .

Slope stabilisation measures comprised a row of reinforced concrete piles installed about one-third of the way up the slope failure. Counterfort drains were installed at the toes to prevent this section below the piles from continuing to move. A cut-off drain was also installed at the crest of the slope to intercept surface water. The piles are reinforced concrete 16m long and 1.05m diameter, spaced 2.50m apart.

Two of the piles were instrumented with strain gauges to measure the bending acting in the piles. Inclinator tubes were installed in piles adjacent to those that had been fitted with strain gauges, and also midway between the piles to try to measure relative pile-slope displacements. One further inclinometer was installed 20m upslope of the pile row. Piezometers were also installed in the slope to measure pore water pressures (Figure 2.58).

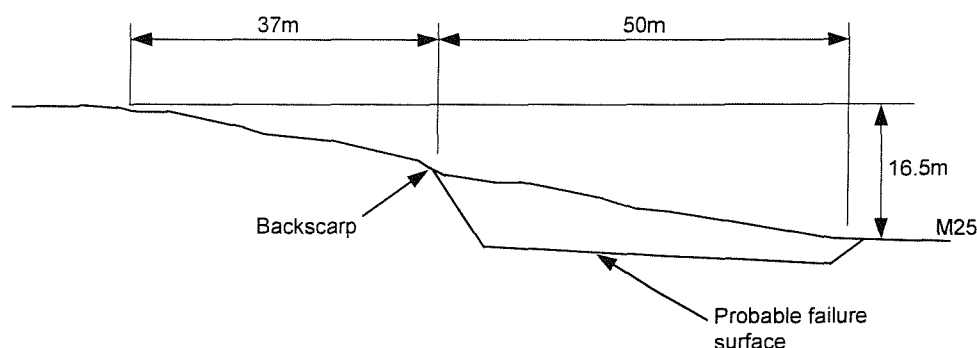


Figure 2.57. Cross section through the pile stabilised cutting slope on the M25 at Godstone, Surrey (redrawn from Ground Engineering, 2001).

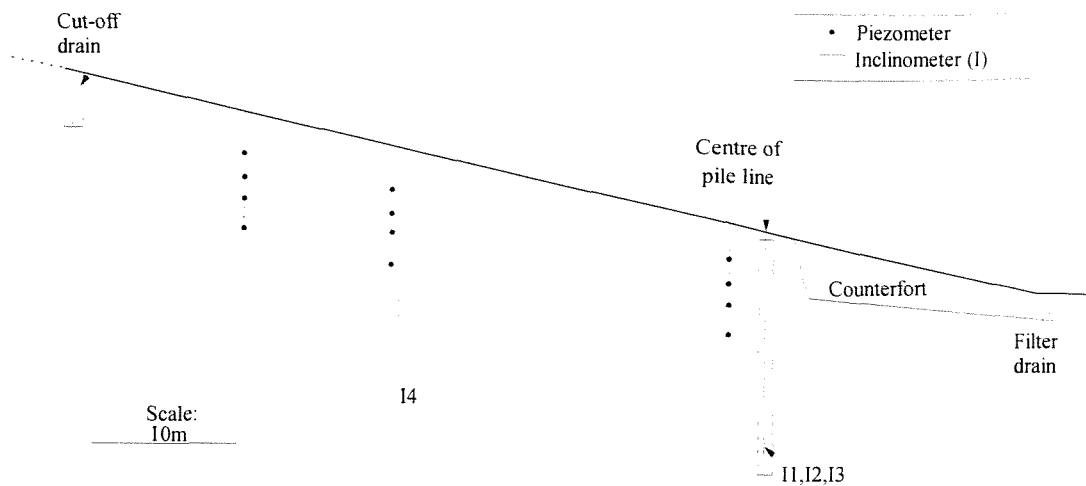


Figure 2.58. Section through the instrumented area (taken from Carder *et al*, 2003)

Figure 2.59 shows the development of lateral displacements with time, measured using the inclinometer tube in pile 21, and the development of bending moments in the adjacent pile 22. The majority of the displacements and bending moment developed after heavy rainfall in early October 2001, after which further movements of the pile have been limited. Results from the inclinometer tube installed midway between the piles showed no evidence of soil flow between them. Displacements in the inclinometer tube 20m upslope of the pile row were slightly larger than those observed within the pile row, with a maximum displacement of 25mm. The maximum bending moment was found to occur in the piles just beneath the failure surface (Carder *et al*, 2003). The inflexion in the bending moment diagram is indicative of the head of the pile being restrained by the failing mass and being bent upslope.

Case 8. Esu and D'Elia (1974), Chen and Poulos (1997)

Chen and Poulos (1997) back analyse a single instrumented pile inserted into a moving clay slope (as originally described by Esu and D'Elia, 1974). The shear surface over which the upper portion of the slope was sliding was 7.5m below the ground surface. The 30m long, 0.79m diameter reinforced concrete pile was instrumented with pressure cells at 5, 10 and 15m depth, both on its upslope and downslope sides. An inclinometer tube was also cast into the centre of the pile.

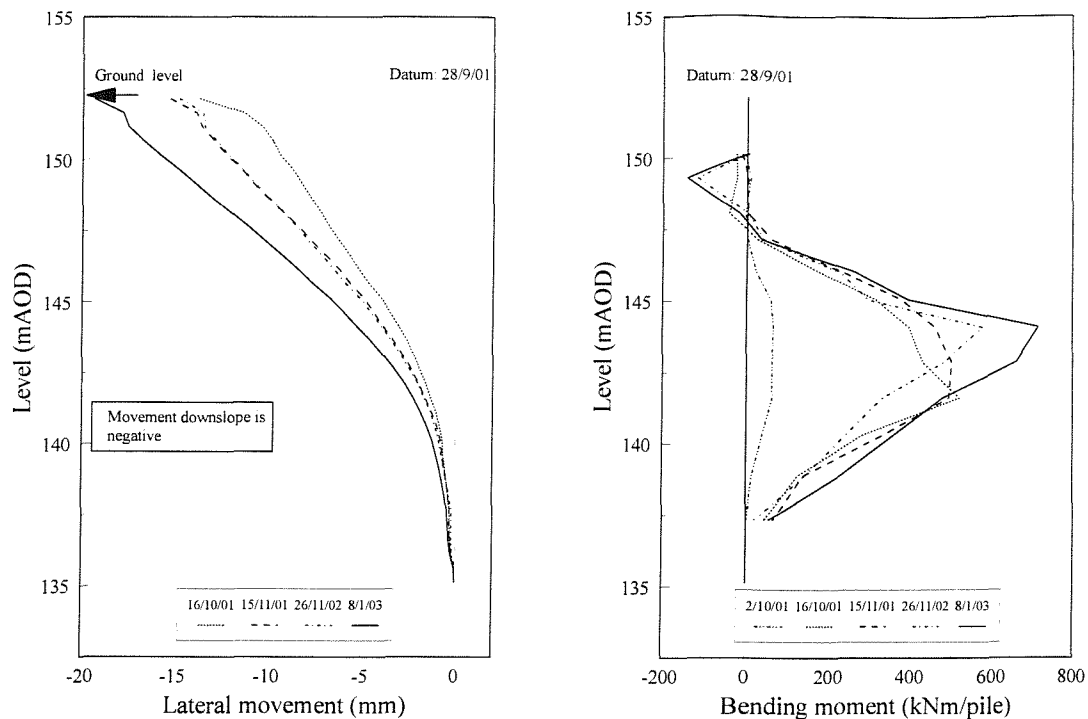


Figure 2.59. Development of pile movements and bending moments (taken from Carder et al, 2003)

The stress on the pile gradually increased until the pile failed with a plastic hinge at 11m depth, and the ultimate bending and deflection of the pile are shown in Figure 2.60. This figure also shows the bending and deflection behaviour calculated by Chen and Poulos (1997) using pseudo finite element analysis. In order to get a matching response from the analysis, Chen and Poulos (1997) had to make a number of assumptions about soil strength and stiffness, and the displacement of the upper layer (which are presumably not reported by Esu and D'Elia, 1974). The assumptions were that the undrained soil strength $c_u = 40$ kPa, the soil stiffness E_s varies linearly from zero at the ground surface to 16 MPa at the base of the pile, and the displacement of the upper layer was assumed to be a uniform 110mm. Pile bending can be seen to be predominantly in the direction of soil movement, with a slight bending back of the pile head where it is restrained within the moving layer. The pile is clearly longer than it needs to be - the bottom 10m of the pile carried none of the load. The bending and displacement of the pile represent the ultimate condition, and behaviour of the pile may have been different if it had been part of a pile row acting to stabilise the slope.

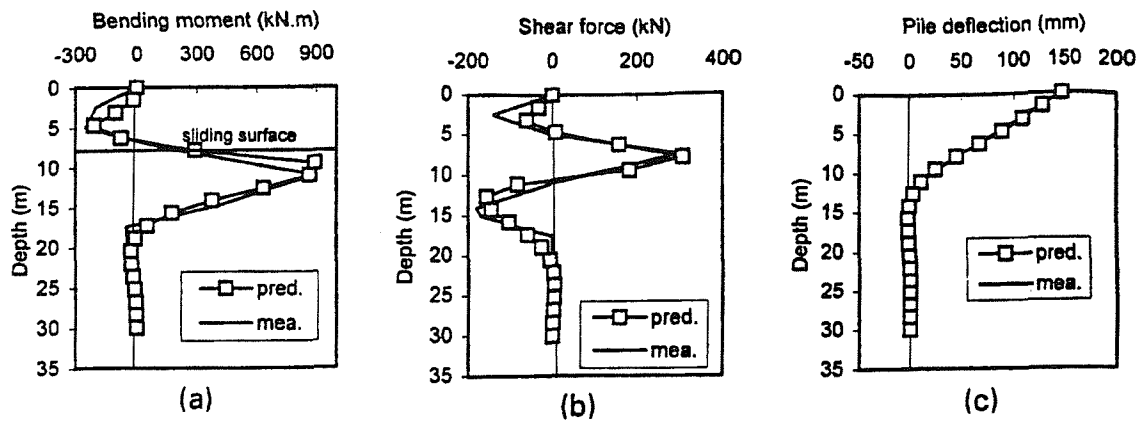


Figure 2.60. Measured and calculated pile response for Case 8 (taken from Chen and Poulos, 1997).

Case 9, Carrubba et al (1989), Chen and Poulos (1997)

Carrubba *et al* (1989) also instrumented a single pile located in a moving slope with pressure cells and an inclinometer tube. The reinforced concrete pile was 22m long, 1.22m in diameter and the failure surface was at 9.5m depth. The undrained shear strength of the soil was found from triaxial testing to be a uniform 30 kPa. The pressures acting on the pile gradually increased until a plastic hinge formed in the pile at 12m depth. As in Case 8, there are no data on soil stiffness or displacement, and Chen and Poulos (1997) have assumed values of stiffness and displacement in their back analysis of the pile to get the measured and calculated profiles to match (Figure 2.61). This pile acts in much the same way as the pile in Case 8, with the maximum bending moment (and plastic hinge) occurring 2-3m below the failure surface. Restraint from the sliding layer to direct downslope displacement of the pile head causes a reversal of the bending moment in the pile section above the failure surface.

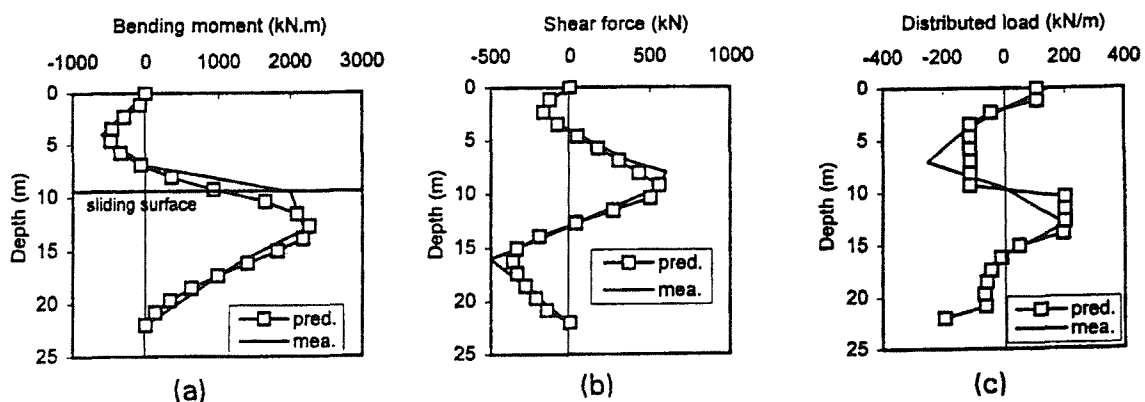


Figure 2.61. Measured and calculated pile response for Case 9 (taken from Chen and Poulos, 1997).

Case 10, Kalteziotis *et al*, 1993

Kalteziotis *et al* (1993) describe the installation of two rows of discrete piles to stabilise a steep slope undergoing landslide movements. Movement of the slide was causing damage to a major highway, part of which was built out of the slope on semi-bridge structure (Figure 2.62). The ground conditions were alternating layers of clayey marl and conglomerate with some thin lenses of sandstone, and depth to the failure surface at the location of the piles was about 4m.

Two rows of piles were installed, located just downslope of the bridge structure. Pile design appears to have been done using some kind of pile-soil interaction analysis, but full details are not given in the paper. The piles were 12m long, 1m diameter reinforced concrete and spaced at 2.5 pile diameters. Two of these piles were replaced by steel tubular piles of equal bending stiffness instrumented with strain gauges and inclinometers. The location of the instrumented piles (AM1 and AV2) is shown in Figure 2.62. The inclinometer N2 was placed between two piles to measure any difference in the displacement of the soil slope and the piles. Inclinometers were also placed upslope and downslope of the pile row (the upslope inclinometers, G12 and G13, are shown on Figure 2.62).

After installation of the piles, slope movements were observed to slow considerably. Over the two years of recorded data, pile and slope displacements were found to be very small. Maximum pile displacement (AM1) was 1.5mm, and maximum displacement for inclinometer N2 was 2mm. Movement of the inclinometers placed upslope of the pile row showed maximum movement of almost 1cm, indicating a possible compression of the soil between the two inclinometer tubes. Figure 2.63 shows the bending and deflection behaviour of pile AM1. Pile bending is uniformly downslope, and the head of the pile does not seem to be restrained in any way by the sliding mass. Of note is that the maximum bending moment in pile AV2 (downslope row) is consistently about twice the maximum moment measured in pile AM1 (upslope row). More load from the sliding mass is being taken by the lower pile row, and this may be explained by the crack running through the slope just in front of the lower pile row (see Figure 2.62). This tension crack in the slope would reduce the pressure acting on the downslope side of the pile, forcing it to act more like a true cantilever. The piles were designed to protect the bridge structure and no changes were made to the slope below the piles to prevent it from continuing to move.

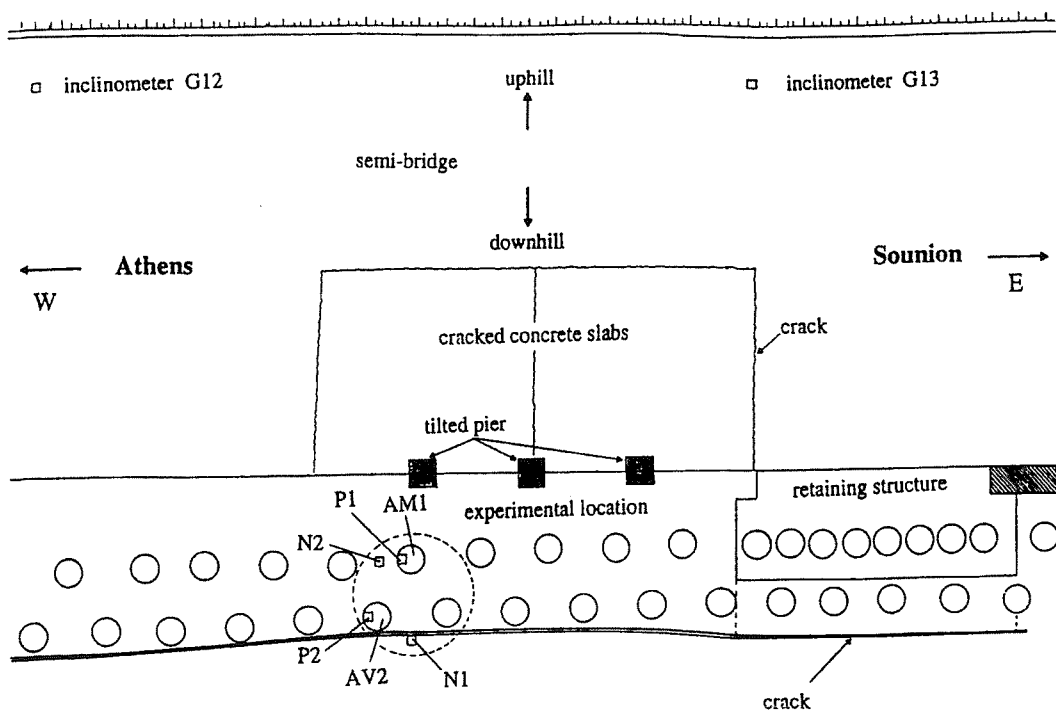


Figure 2.62. Plan view showing the location of the instrumented piles (taken from Kalteziotis et al, 1993)

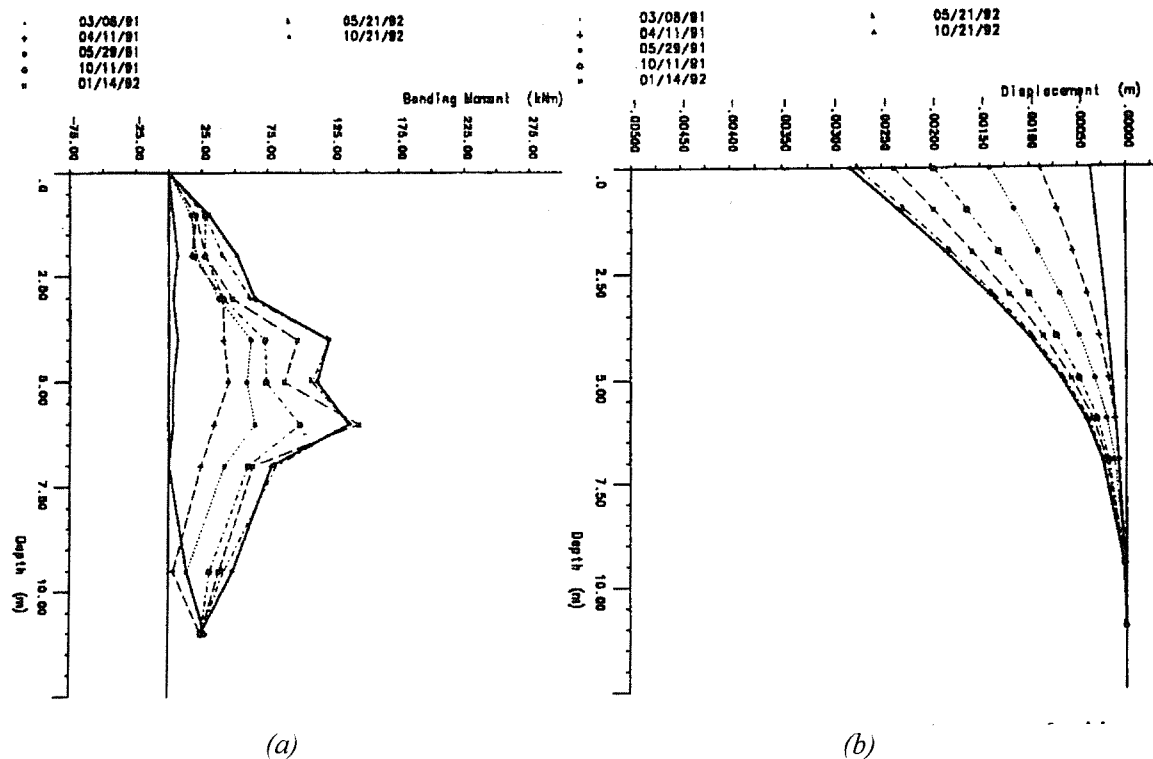
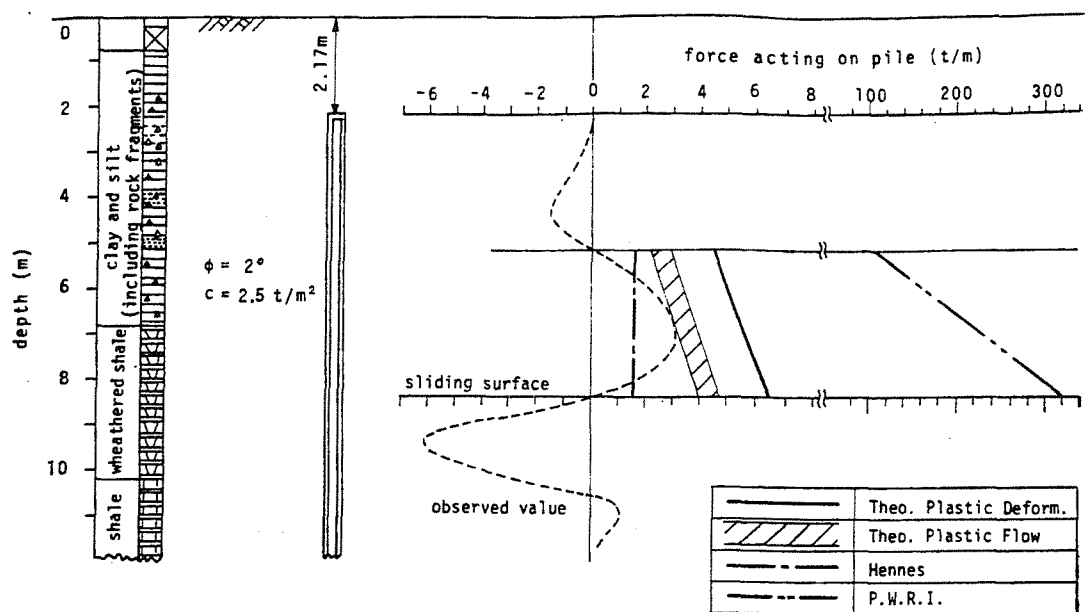


Figure 2.63. Diagram showing (a) bending, and (b) deflection of pile AM1 (taken from Kalteziotis et al, 1993).

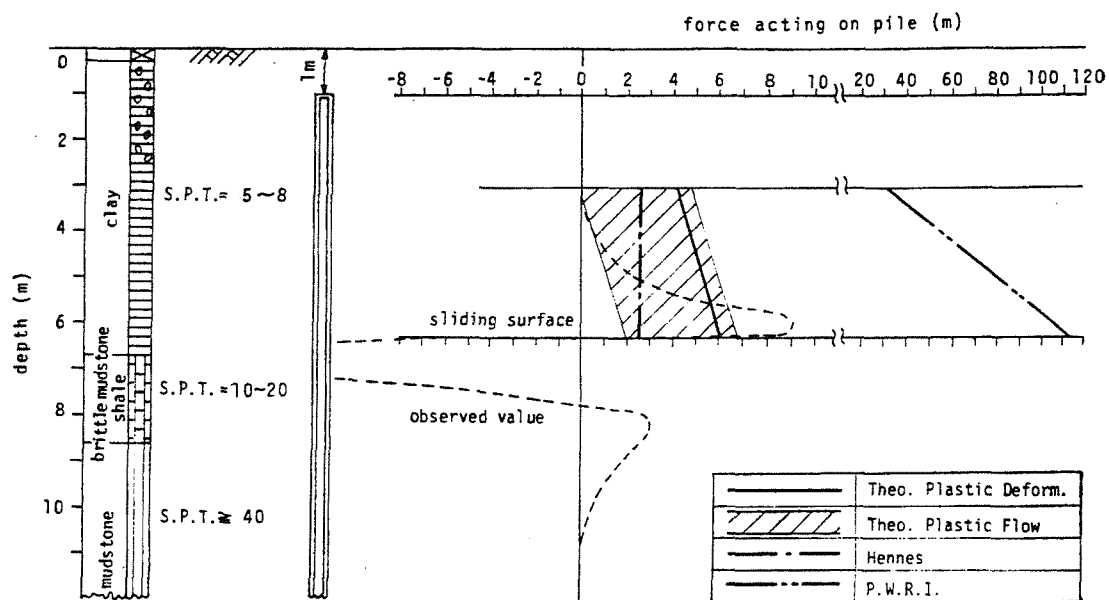
Case 11. Niigata, Japan. Ito and Matsui (1975)

Ito and Matsui (1975) describe the pile stabilisation of several slopes within the landslide area at Niigata, Japan. Individual landslide areas are named as Katamachi, Higashitono and Kamiyama. These landslides all comprise slow sliding clay and mudstone layers several metres thick. In the Katamachi landslide area the piles are 300mm diameter hollow reinforced concrete and in the other areas the piles used are 300mm diameter hollow steel pipes. In all of the landslide areas the piles are placed in two rows 2m apart and the piles are zigzagged with the piles in each row spaced 4m apart. The lower SPT counts show that the strength of the sliding material is in all cases less than the intact material below the shear plane.

Strain gauges were placed into some of the piles to measure bending strain. The load acting on the piles was calculated from the strain measurements, and plotted for each of the piles in Figures 2.64 and 2.65. The measured load on the head of the piles is compared with theoretical values calculated using the theories of plastic flow and plastic deformation (Ito and Matsui, 1975) and the Hennes and PWRI methods (which are neither explained or formally referenced in the paper). It can be seen from the figures that the theory of plastic deformation is the best estimate of the measured load acting on the piles. The largest loads on the piles occur close the shear surface, with the load resisting pile movement acting over only about 2m length of pile below the shear surface. No indication is given by Ito and Matsui (1975) of the pile length (Figures 2.64 and 2.65 show the piles extending below the base of the pressure diagrams), and the piles appear to be considerably longer than necessary. In three of the piles the load distribution reverses within the failing mass, showing that the very head of the pile is displacing into the sliding soil, and that this portion of the pile is being restrained against rotation.

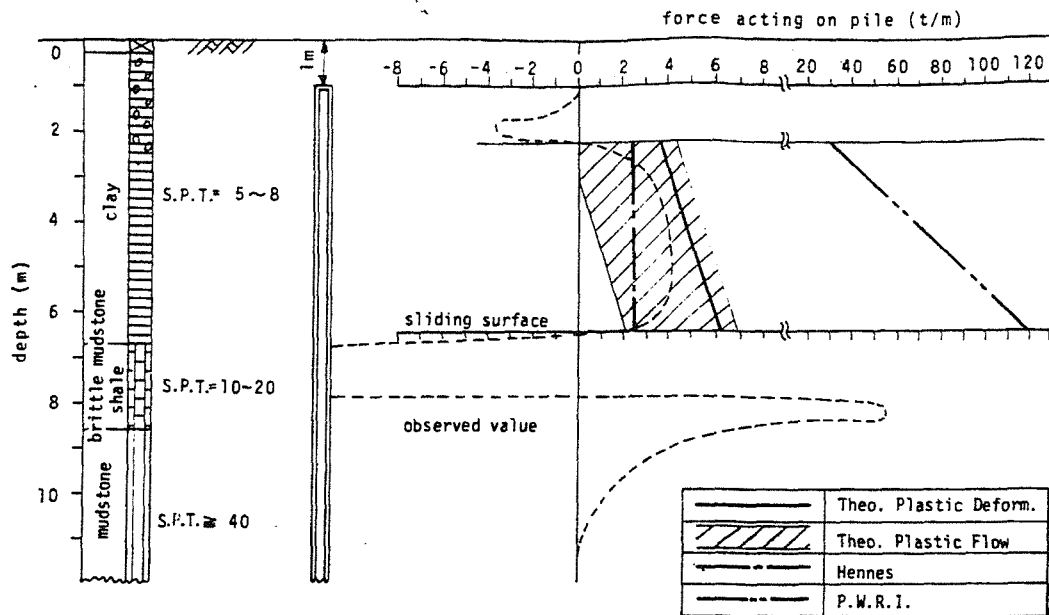


(a) Katamachi B pile

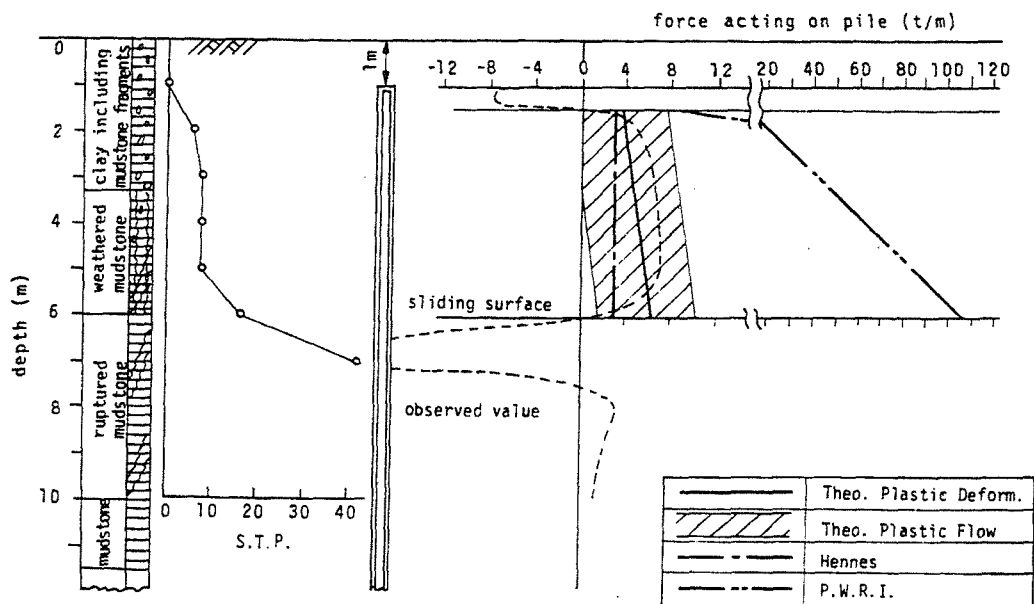


(b) Kamiyama No. 1 pile

Figure 2.64. Pressure distributions acting on instrumented piles in the Niigata landslide area, Japan (taken from Ito and Matsui, 1975).



(c) Kamiyama No. 2 pile



(d) Higashitono No. 2 pile

Figure 2.65. Pressure distributions acting on instrumented piles in the Niigata landslide area, Japan (taken from Ito and Matsui, 1975).

Case 12, Allison *et al* (1991)

Allison *et al* (1991) describe the use of discrete piles to stabilise a slope failure in south London. The slip was triggered by heavy rainfall several months after removal of a retaining wall and regrading of the site to allow the construction of new houses at the toe of the slope. The slip was about 40m wide and extended 30m up the slope. The failure plane was found to be 4-5m below ground level and at the interface between intact London Clay and overlying colluvium. Due to concerns about further slope movements causing damage to the newly built properties at the toe of the slope, it was decided to stabilise the slope using discrete piles. The piles were designed using the Viggiani (1981) method of analysis (see Section 2.5.3). Thirty reinforced concrete piles, 0.6m in diameter and 10m long were installed in three rows, each about 5m apart. Fourteen piles were placed in the bottom row, spaced 2.8m apart, with the spacing of the piles in the upper rows somewhat larger than this. The factor of safety of the slope after pile installation was designed to be 1.24. The factor of safety was further increased by the addition of drainage measures to the slope at a later stage. A cross section through the stabilisation scheme is shown in Figure 2.66. About 1 year after the piling work was finished, some further movement took place in the lowest part of the slope. Localised slip surfaces formed in between the piles in the bottom row, exposing cusp shaped soil surfaces 0.5m – 0.75m high arching between the piles.

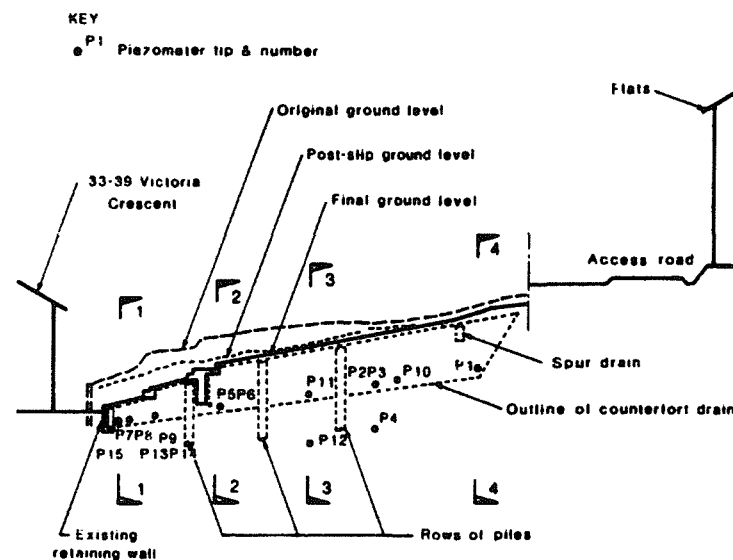


Figure 2.66. Cross section through the slide in south London, Case 12 (taken from Allison *et al*, 1991).

Monitoring of pore water pressures using piezometers in the slope showed that installation of deep drains stabilised the water table within the slope to about 2.5m below ground level. Ten of the piles in the slope were instrumented with an inclinometer tube, and the results from the monitoring of pile displacements are reported for a five year period after pile installation. Figure 2.67 shows a plot of the pile head displacements against time. Pile head displacements initially

increased rapidly but levelled out over the five year period. The largest pile displacements occurred in the top row of piles (I2, I4 and I8 on Figure 2.67). Bending moment distributions calculated from the inclinometer displacement profiles show that the largest bending moments occur in the upper row of piles, which are operating at 45% of their ultimate bending capacity. In comparison, the lower rows of piles are found to be operating at about 20% of their ultimate bending capacity. The pattern of displacement given by the inclinometer readings indicate that the piles are being uniformly loaded over a cantilever length of 6m.

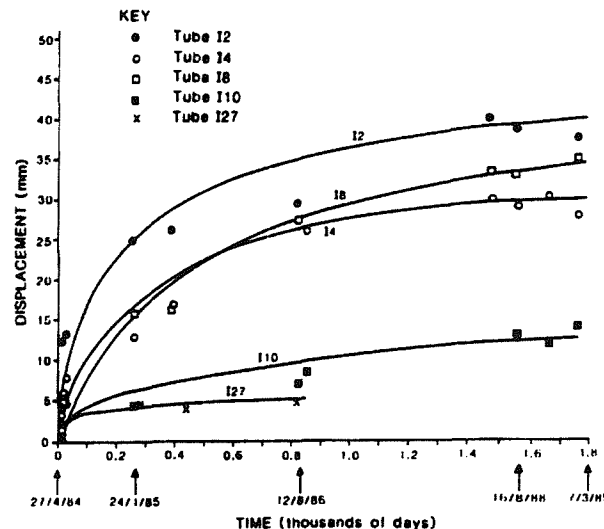


Figure 2.67. Pile head displacements against time for instrumented piles, Case 12 (taken from Allison *et al*, 1991)

Case 13. Stahlberg, Germany. Sommer (1979)

Sommer (1979) describes the stabilisation of large landslide in Stahlberg, Germany. A 12m high road embankment was constructed across a natural slope with an inclination of 5-8° and a geology comprising a highly plastic overconsolidated clay. As the embankment approached a height of 10m, the slope started to move along a failure surface running through the rear of the embankment and extending 180m downslope. The failure surface ran on the boundary between the intact and disturbed clay materials, at about 15m depth. Geotechnical investigation demonstrated the residual strength on the shear surface to be about 8°.

Initial toe weighting measures reduced the rate of landslide movement, but failed to stop it completely. Subsequently, a row of 3m diameter piles were constructed midway up the length of the slide, spaced with centres 9m apart, and penetrating 5m into the intact underlying geology.

To prevent failure of the piles, they were designed to resist pressures at the full ultimate lateral pile load. The piles were designed assuming that only the bottom 10m of the pile would resist slope movement (5m above the failure surface and 5m below it), the rest of the pile was

assumed to be redundant as shown in Figure 2.68. It was intended that the piles increased the factor of safety of the slope to 1.1.

Installation of the piles was found to reduce the rate of slope movement from 14mm per month to 1.4mm per month. Pressure cells were installed on one of the piles to measure the lateral soil pressure acting on the pile close to the failure surface. The measured pressure distribution is compared with the design distribution in Figure 2.69. The figure shows that the pressures acting on the pile are about one-third of the ultimate lateral pressures used in design. The measured pressures do not seem to be in equilibrium, but this may be the result of having to infer the pressure distribution between the measured points. There also appears to be no pressure acting on the upper part of the pile, but it is uncertain whether this is because the measured pressure was small or because there were no pressure cells on this part of the pile. Sommer (1979) does not discuss the type of pressure cell used, or the reliability of the measurements obtained.

Most load acting on the piles developed with the first 4 months after installation, beyond which there was only some small additional increases in load. Soil displacements measured midway between piles show that the maximum soil displacement is only 5mm larger than the displacement of the piles.

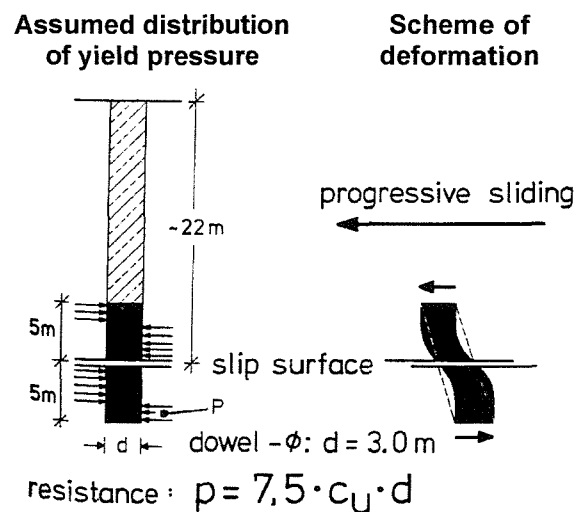


Figure 2.68. Assumed design pressure distribution for the piles (taken from Sommer, 1979)

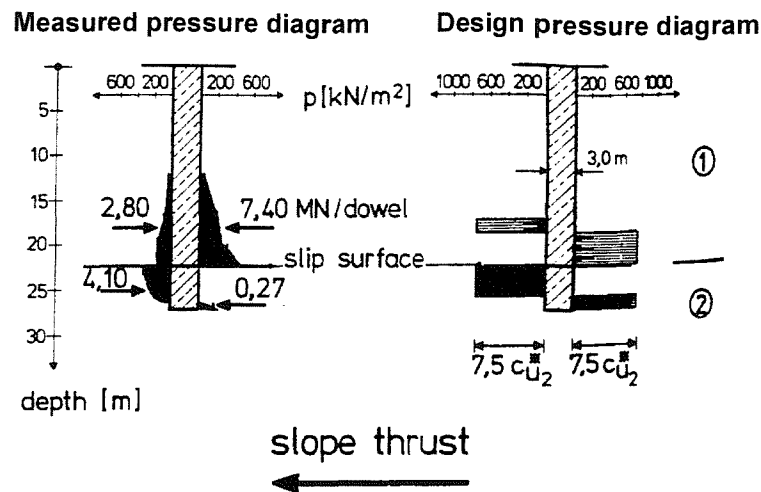


Figure 2.69. Measured and design pressure distributions (taken from Sommer, 1979)

Case 14 Kuala Lumpur, Ting *et al* (1990)

Ting *et al* (1990) describe the use of piles to stabilise a fill material placed onto a slope to facilitate the construction of several rows of houses. The houses were built on timber piles driven into the intact underlying ground, but there was considerable concern that downslope lateral movement of the fill material would excessively load the timber piles. Vertical and horizontal slope movements were measured using precise levelling and inclinometers, and were found to be excessive. As a result, it was decided to install concrete piles to help stabilise the slope.

The 250mm diameter piles were principally designed to provide resistance to the slope in bending, but the piles were constructed at 30° to the vertical in an attempt to try to mobilise their tensile strength, as this was assumed likely to provide additional resistance to slope movements (Ting *et al*, 1990). The small diameter of the piles meant that they were built in clusters of two or three, with each cluster spaced about 3m apart, depending on the force the piles were required to apply to different sections of the slope (Figure 2.70). The purpose of the piles was to reduce slope movements, rather than to stabilise the slope directly as there was no evidence of a failure mechanism (Ting *et al*, 1990).

The piles were designed to increase slope stability by 15%, with the interface between the fill material and natural ground taken as the boundary above which movements are assumed to occur. The design bending moment was calculated using lateral pile pressures calculated using methods proposed by Brinch Hansen (1960) and Tschebotarioff (1973). Brinch Hansen's method uses ultimate pile-soil pressures, whereas Tschebotarioff's method gives pressures closer to working load conditions, and subsequently much smaller bending moments. The

bending moments calculated using Tschebotarioff's (1973) method were used in the final pile design.

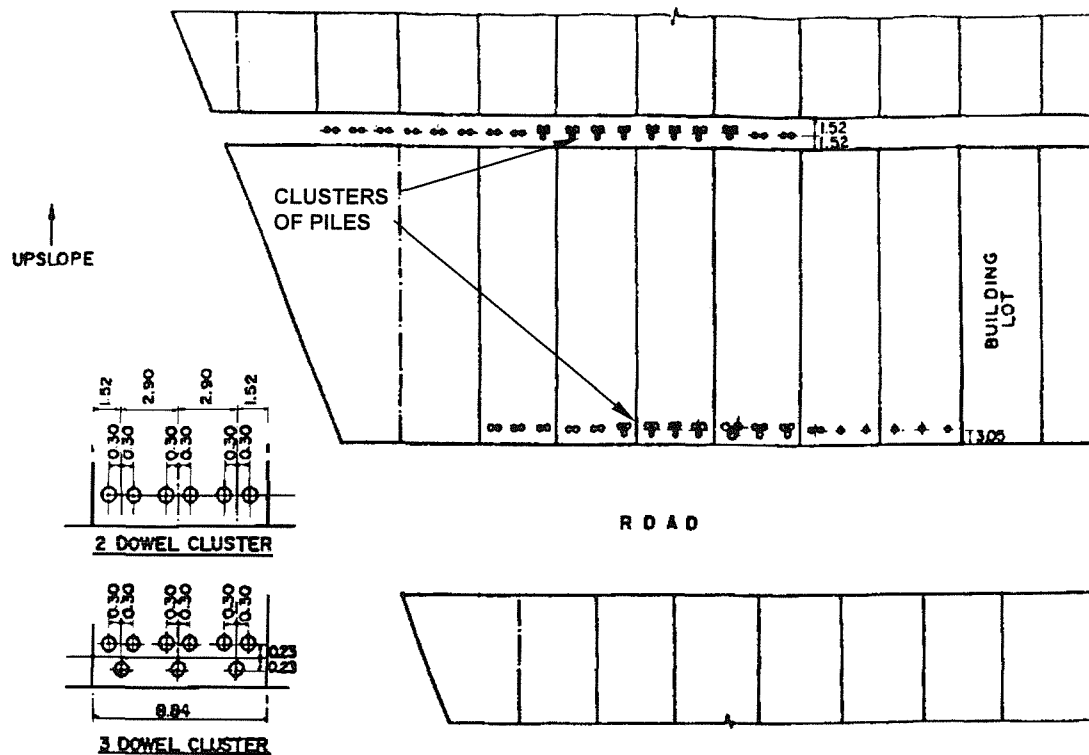


Figure 2.70. Plan of the pile layout, showing a detail of the pile cluster arrangement (modified from Ting et al, 1990). Dimensions in metres.

A test pile was constructed and instrumented with strain gauges to measure the bending moment generated by slope movement. The vibrating wire strain gauges were attached in pairs to the universal column reinforcement placed into the pile, as shown in Figure 2.71. The test pile was initially subject to a pull out test, in order to mobilise tension resistance in the pile, and the strains were then measured in the pile over the following two years as the pile acted to resist slope movements. Lateral slope movements were measured by inclinometers installed into the slope.

Measurements from the strain gauges after one year show that the pile is acting in compression, rather than the tension hoped for. However, the maximum bending moment measured at 23 kNm is close to that calculated by the Tschebotarioff (1973) method of 20 kNm. The maximum bending moment calculated for Hanson's method using the ultimate pile-soil pressure was 103 kNm.

Transfer of load to the piles was initially quite rapid, with the majority of load taken up in the first ten days or so. There was a gentle increase in load after that, except for around day 400, when construction work commenced close to the instrumented pile, and a rapid increase in load

was seen (Ting *et al*, 1990). Over the period of measurement, the rate of slope movements has decreased from 0.04mm/day before pile installation to 0.01mm/day two years after pile installation.

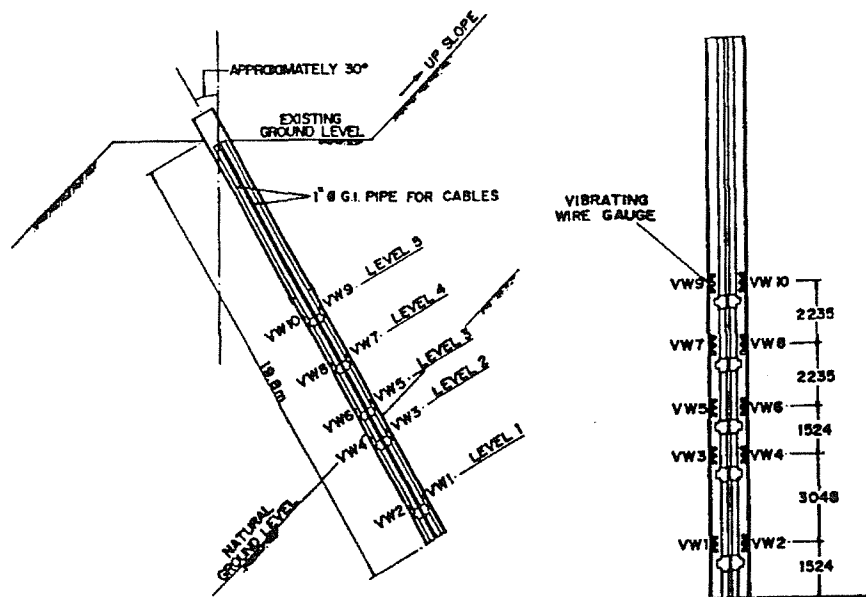


Figure 2.71. Location of the strain gauges in the instrumented pile (taken from Ting *et al*, 1990).

2.7.3 Conclusions from instrumented studies

While there are a number of case histories presented here in which piles are instrumented, many of the studies are limited in scope, or are not described in detail. There is a paucity of comprehensive information on how real piles behave, and more instrumented case studies are required to obtain a better understanding of slope and pile interaction. In particular, there is a need to compare pile behaviour with the way in which the piles were designed, and their structural design capacity – this is not done in many of the studies reviewed here.

The pile behaviour in the cases reported is heavily dependant on the site conditions, including soil stiffness, type and size of mass movement and pile length. Behaviour of the single piles which develop a plastic hinge (Cases 8 and 9) are likely to reflect the ultimate behaviour of the piles, and not necessarily their working behaviour were they to be placed in a row to fully stabilise the slope.

There are a number of broad conclusions that are of interest to pile design:

- The maximum bending moment measured in most of the piles occurs beneath the failure surface, and that in most cases the piles were designed in 'long' or 'intermediate' mode (see Figure 2.35). The exception are Cases 6, 13 and 14, in which the piles were designed in 'short' pile mode. In Cases 6 and 13 (Guest *et al*, 2000; Sommer, 1979) the piles are designed in short mode due to the large depth of the failing mass, and much of the upper

part of the pile ends up being structurally redundant. In Case 14 (Ting *et al*, 1990) the particularly competent intact slope material allows a short length of pile beneath the plane of movement. In some of the cases (Cases 8 and 11) the piles installed are clearly longer than they need to be (as the bottom length of the piles is carrying no load).

- In Cases 7, 8 and 9, and some of the examples in Case 11, the depth to the failure surface is several metres and the bending moment reverses across the pile within the failing mass. When the depth to the failure surface is quite large, it is clear that the head of the pile could be restrained from bending uniformly downslope by the failing mass. The development of a significant bending moment at the head of the pile must be considered when specifying pile bending capacity.
- Where more than one row of piles is used (Cases 10 and 12) it is clear that piles in different rows will not always carry equal load, and this should also be carefully considered in design. In case 10, (where there are two rows of piles 2.5m apart) the lower row of piles is carrying more load than the upper row. This can probably be explained by the tension crack that has opened in the slope just below the lower row of piles, and which causes the lower row to act like a cantilever, carrying more of the load. In case 12, (where there are three rows of piles 6m apart) the difference in loading is more difficult to explain, but may simply be a result of differences in the displacement of parts of the slip, or the difference in the pile spacing used in the rows.
- It is evident from a number of case histories (Cases 7, 10, 12 and 14) that pile loading is developed slowly. Case 12 shows the steady increase in pile displacements only tailing off after about 4 years. Other case histories (such as 7 and 10) only present data for an initial period of monitoring (about 18 months in both cases). It is evident particularly in Case 7 that movements are ongoing, and the data may not show the final pile loading. It is evident from this that the monitoring period for any future pile stabilised slope may have to extend over several years if a full set of pile loading data is to be obtained. Only Carder *et al* (2003) (Case 7) have apparently attempted to link increases in pile displacement and bending with rainfall events.
- Pile spacings vary between 1.34 and 11.25 pile diameters centre-to-centre, although biggest dimensionless pile spacings occur with the use of small diameter piles (Cases 6 and 11). The majority have a dimensionless spacing in the range 3 – 5 diameters centre-to-centre. In most cases, the piles are not more than a distance of 4m apart (centre-to-centre), the exception being Case 13 in which the piles are 9m apart (but are also 3m in diameter).
- Some of the case studies considered do not state how the piles were designed, and others do not give very much detail. Only Ting *et al* (1990) (Case 14) attempt to compare the design pile bending with the bending moment actually measured in the pile. Limit equilibrium slope stability analysis was used to find the pressure applied to the pile by the failing mass in Cases 1 and 2. In quite a number of the cases (Cases 1, 3, 6, 12 and 13) design was

carried out using the limiting pile-soil pressure within a limit equilibrium calculation and by assuming the mode of failure to be similar to those presented by Viggiani (1981) or Brinch Hansen (1961). Case 14 showed that the limiting soil-pile pressure resulted in a design bending moment far in excess of that measured in the pile. Limit equilibrium methods are favoured in design, and only Cases 2 and 10 used an elastic based analysis to assess the bending and displacement behaviour of the pile (although Chen and Poulos (1977) back analyse the piles in Cases 8 and 9 using an elastic analysis).

2.8 Summary of the review of previous work

Case histories and methods of design for discrete pile stabilised slopes have been reviewed in this chapter. The major findings are summarised as follows:

- i. Methods of calculating the load acting on the pile from the failing soil mass were considered. Limit equilibrium slope stability analysis is commonly used in practice and is probably the best means of estimating the pile head load as it takes into account the equilibrium of the slope. There are uncertainties as to how limit equilibrium slope stability should be used in a preventative role, in which the piles are likely to affect the shape and depth of the critical mechanism. When piles are used to stabilise large failures the pile head loading is often assumed to be the maximum pile-soil limiting pressure.
- ii. The position of the piles within the slope dictates how the piles act to stabilise the failing mass. For short steep slopes, the piles are likely to be most effective when placed around midslope, where they penetrate the deepest part of the slip. In the case of embankments, piles are often placed at the crest of the slope to directly protect the infrastructure on the top of the embankment.
- iii. Physical and numerical studies investigating pile spacing and soil arching between piles were reviewed. The studies indicated that soil arching in slopes occurred by means of a semi-circular shaped major principal stress arch upslope of the piles. The studies showed that the maximum pile spacing achievable is a complex soil structure interaction problem, dependant on the pile spacing, pile shape, pile diameter, soil frictional strength, soil cohesion, the pile-soil interface, a gap forming behind the piles and soil dilation. The numerical studies demonstrated that arching effectiveness is likely to be part dependant on the physical distance between the piles, and not just on the dimensionless pile spacing. Review of a number of studies suggested that for lower soil

strengths, arching between the piles rapidly becomes less efficient with increasing pile spacing, and the maximum pile spacing will be limited to possibly 3 pile diameters. With much stronger soils (especially those with a high effective cohesion), the arching still appears to be highly effective at 5 or even 6 pile spacings. The range of pile spacings used in the case studies varied considerably, but the maximum spacing tended to be used with small diameter piles and where multiple rows of piles were being used. Piles were rarely placed more than 4m apart.

- iv. The pile penetration is particularly important, and was found to dictate the mechanism by which the pile acts to stabilise the slope. This in turn was shown to have a significant effect on the pile bending behaviour. Pile penetration and the associated bending behaviour has been broadly categorised into 3 groups: long, intermediate and short. Most of the case studies had been designed with piles acting in 'long' or 'intermediate' mode, and the maximum bending moment tended to occur beneath the failure surface. 'Short' piles were used in two cases where the depth of the failing mass was large. In a number of cases the pile head was found to be restrained within the failing mass, causing the head of the pile to bend back upslope, and causing the measured bending moment distribution to reverse across the pile.
- v. Pile design within short steep slopes typical to infrastructure cuttings and embankments tends to require piles to be designed in long or intermediate mode to prevent a failure surface forming beneath the toes of the piles. Limit equilibrium pile design calculations tend to give piles that are fairly short, and the length of the piles then have to be increased to account for global stability. This makes calculating the bending moment over the base of the piles difficult. Limit equilibrium design also requires assumptions about the pile bending and rotation to be made. The use of the limiting pile-soil pressure in design of piles used to stabilise short steep infrastructure slopes is likely to result in piles of large diameter spaced a significant distance apart. Pile failure is more likely to occur by means of plastic hinge formation within the pile, rather than failure of the soil around the pile section.
- vi. A better means of modelling the pile bending and rotational behaviour is to use pseudo finite element analysis, which takes account of the stiffness of the soil and pile. Such analyses have been used to accurately back analyse the behaviour of instrumented piles. The soil displacements required as input to the analysis can be increased until the desired restoring force is found.

- vii. The pile head fixity has been demonstrated by several studies to have a significant impact on the pile bending behaviour. Bending moments in the piles tend to be smaller when the pile is restrained from displacement and rotation.
- viii. A number of academic studies have been carried out using three-dimensional finite element analyses to investigate the use of discrete piles to stabilise slopes. The studies demonstrate that three-dimensional finite element analysis is a powerful tool, and is able to model the full equilibrium of the slope and piles. None of the studies try to back analyse the behaviour of a real pile stabilised slope.
- ix. A number of instrumented pile studies were reviewed to assess the pile bending behaviour in a range of types of geology and mass movement. Some of the points of summary from the case study data have already been mentioned above. In many of the studies the pile loading was found to develop over several years, with implications for the duration of pile and slope monitoring projects. In only one case pile movements and bending were linked to climatic data. Many of the case histories did not describe the design of the piles, but for those that did, design using the limiting soil-pile pressure incorporated within a limit equilibrium calculation was most common.

Chapter 3. Instrumentation of a pile stabilised rail embankment at Hildenborough, Kent

3.1. Background

Discrete piles have been used to stabilise a railway embankment at Hildenborough, Kent. The embankment forms part of the geotechnical infrastructure on the line between London and Hastings, and is part of the route currently used by European rail services between London and Paris.

Remediation of the embankment was required to solve long term maintenance problems, including excessive side slope displacements and track settlements. Slope stability calculations carried out by Mott MacDonald after an initial site investigation demonstrated the north slopes of the embankment to be close to failure. Vegetation was removed from the embankment slopes in May 2000 in order to carry out a further investigation of the embankment with a view to constructing remedial works. Removal of the vegetation may have contributed to a number of shallow rotational slope failures during a period of prolonged rainfall in October and November 2000. To help stabilise these failures, the embankment surface was covered in a layer of granular rockfill material.

Two hundred piles were installed along two lengths of the embankment to further increase the factor of safety of these sections to the required value of 1.3. A plan showing remedial works is given in Figure 3.1.

Design and build contractor for the project was Cementation Foundations Skanska/Mott MacDonald Ltd. Cementation Foundations Skanska paid for the instrumentation put into the piles, which was specified and monitored as part of this research project.

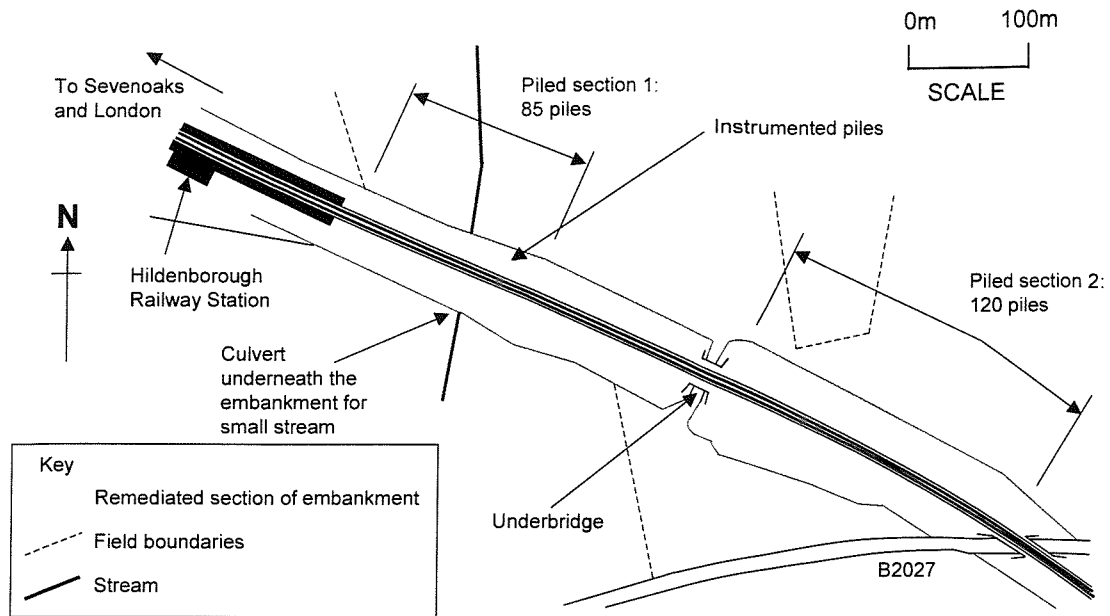


Figure 3.1. Location sketch showing the remediated sections of the embankment.

3.2 The instrumented embankment section

3.2.1 Soil properties

The railway embankment at the instrumented section comprises tipped Weald Clay. The Weald Clay also underlies the site, and in the Tonbridge area the stratum is about 250m thick (Gallois and Edmunds; 1965). The deposit is dark grey stiff thinly laminated closely fissured CLAY, with occasional clay ironstones. The top metre is weathered as evidenced by a change in colour to mottled brown and yellow. The embankment fill material is entirely weathered.

Imported granular rockfill material was placed onto the embankment slopes in January and February 2001 to act as toe weighting to existing slips and to provide a platform on which to operate piling plant.

The soil strength parameters used in design, which are based on data from the site investigation and associated triaxial tests, are given in Table 3.1. At the time of analysing the Hildenborough piles for this thesis, the full triaxial test laboratory report, from which the stiffness of the triaxial samples could have been determined, was not available.

Soil type	Unit weight γ , kN/m ³	Effective friction angle ϕ' , degrees	Effective stress cohesion c' , kPa
Weald Clay embankment fill	19	25	1
Softened Weald Clay embankment fill	19	19	1
Weathered Weald Clay	19	25	1
Weald Clay	20	30	5
Granular embankment fill	19	35	0

Table 3.1 Design soil parameters

3.2.2 The embankment geometry

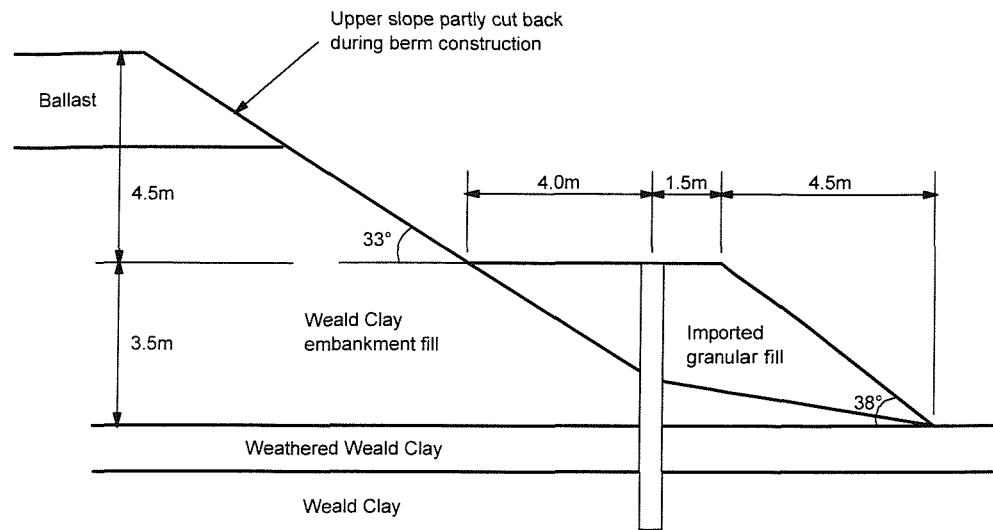
The embankment at the instrumented pile location is 8m high, and originally comprised side slopes averaging an angle of about 30°, but with the upper slopes made up largely of rail ballast standing at a steeper angle and the lower Weald Clay embankment fill slopes at an angle shallower than 30°.

The section of the slope that has been instrumented did not suffer a slope failure after the vegetation was removed from the slope surface, but an adjacent section did fail, with a failure surface approximately 2m below the original slope surface (Duncan, 2002).

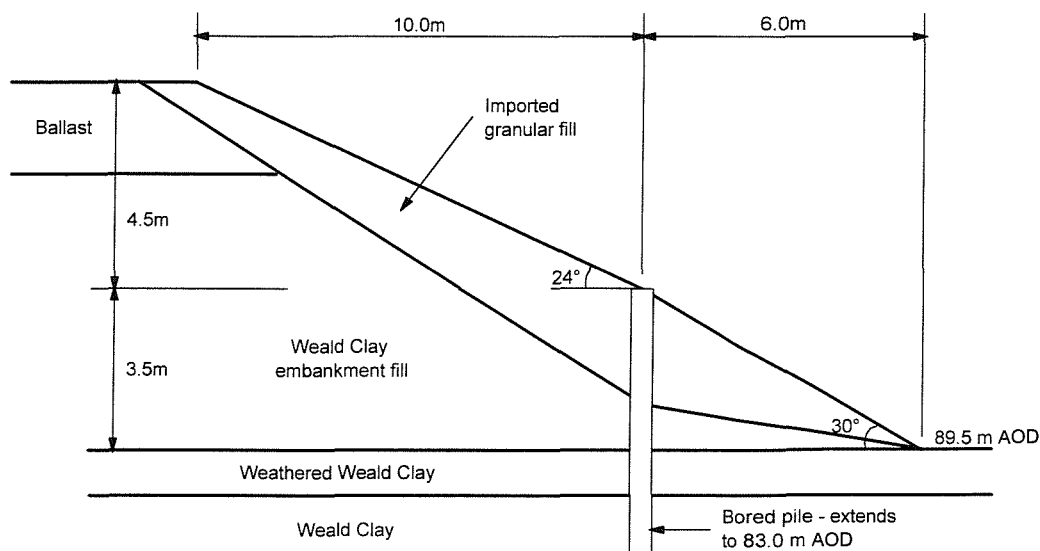
The instrumented piles were placed in a location where the berm changed in height and was slightly wider so that construction plant had space to traverse around the instrumented section, in which the inclinometer casings protruded from the top of the piles. Figure 3.2 shows the location of the piles within the slope and the embankment profile both during construction and after regrading of the construction platform had taken place. The embankment profile changes along the instrumented section, with an increase in width and slight drop in height from pile A towards C. Figure 3.3 shows the instrumented pile group shortly after pile construction, in early April 2001. On the 19 and 20 April 2001 (days 22 and 23 after pile installation), the granular rockfill material was regraded into a two stage slope, as shown in Figure 3.4. The final regrading profile was such that a small horizontal platform remained downslope of piles B and C (as can be seen in Figure 3.4).

The piles at the instrumented section are 10 m long, 0.6 m diameter bored C35 concrete piles constructed at a spacing of 2.4 m or 4 pile diameters centre to centre. The piles contain 6×T25 steel reinforcing bars over their full length, giving an estimated ultimate bending moment capacity of 250 kNm, and an additional 6×T32 rebars over their lower 7 m to increase the

ultimate bending capacity over this length to 520 kNm. The concrete piles were constructed using a continuous flight auger rig.



(a)



(b)

Figure 3.2. Embankment profile (a) from pile installation on 29 March 2001 to the start of regrading on the 19 April 2001, showing granular berm used for pile installation; (b) from the 20 April onwards.



Figure 3.3. The instrumented section of the embankment at Hildenborough shortly after pile construction.



Figure 3.4. The instrumented section of the embankment at Hildenborough after the berm had been re-graded to create a two stage slope

3.2.3 Instrumentation

Strain gauges were installed in three piles at Hildenborough to measure the bending moment induced in the pile by slope movements. Inclinator tubes were installed both inside the strain gauged piles and in the slope midway between piles to measure any difference between slope and pile movements, and in particular to measure any flow of soil between the piles. The three instrumented piles are located adjacent to one another, as shown in Figure 3.5.

Strain Gauges

Twelve strain gauges in 6 pairs were cast into each of the three instrumented piles to measure bending moments. The gauges were installed by ITM Ltd. The strain gauges are Slope Indicator vibrating wire gauges, which are installed by gluing the gauge to a small flat surface machined into the side of the reinforcement bars making up the pile cage. The vibrating wire plucking/reading unit is then clamped over the gauge and secured around the re-bar using plastic cable ties. The pairs of gauges were spaced at 1.6m centres down the pile, as shown in Figure 3.6. Care was taken to attach the gauges onto the inside of the reinforcement making up the pile cage, such that the planed surface to which the gauge was attached was perpendicular to the pile radius. The strain gauges were all attached to the T25 reinforcing bars in the pile. The gauges have a thermistor to measure the temperature.

The piles were finished by pushing the pile cage into the concrete filled auger hole produced by the continuous flight auger rig. As the cages were fairly flexible, additional steel ties were added to better secure the shear and main reinforcing bars making up the pile cage, and prevent relative movement of the gauges as the pile cage was lifted and then pushed into the concrete.

Pile C also contains twelve Gage Technique TES/5.5/T vibrating wire concrete embedment strain gauges, to compare the performance of the embedment type of gauges to those attached to the reinforcement cage. These gauges were installed by the University of Southampton and Structural Instrumentation Partnership Ltd. The embedment gauges measure the strain within the concrete, but for the purposes of pile construction are supported by steel holding bars spanning between adjacent pile reinforcement bars (Figure 3.7). There was considerable concern regarding the effect on the gauges of the cage being pushed into the concrete pile, and subsequently additional steel holding bars were attached about 20cm below the base of the gauges in an attempt to reduce the force applied to the gauge by the flow of concrete upward relative to the cage.

A Campbell Scientific CR10X datalogger was used to obtain a continuous record of strain gauge output.

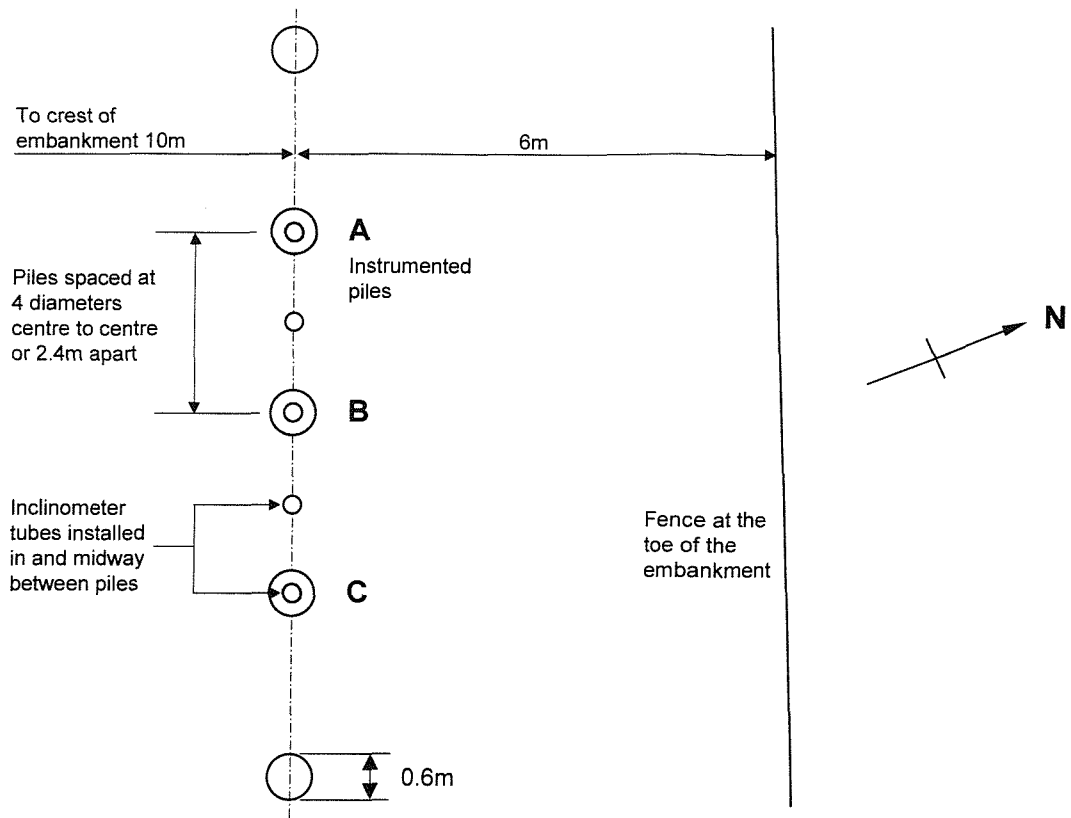


Figure 3.5. A large scale plan of the instrumented section of the slope, showing the layout of the instrumentation

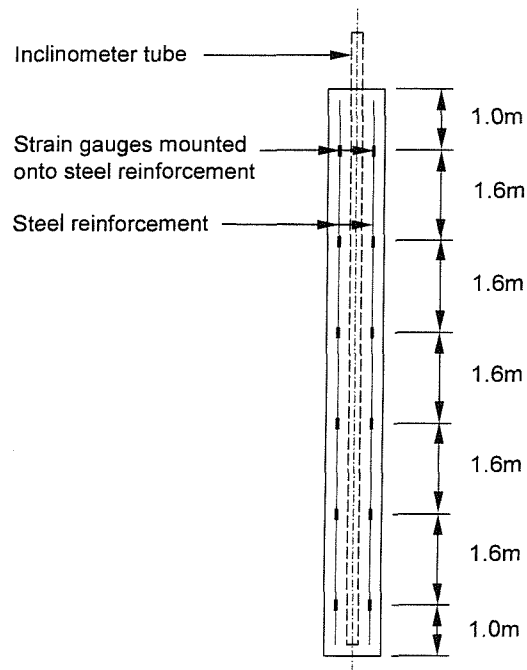


Figure 3.6 A cross section through the instrumented piles showing the location of the strain gauges and inclinometer tube

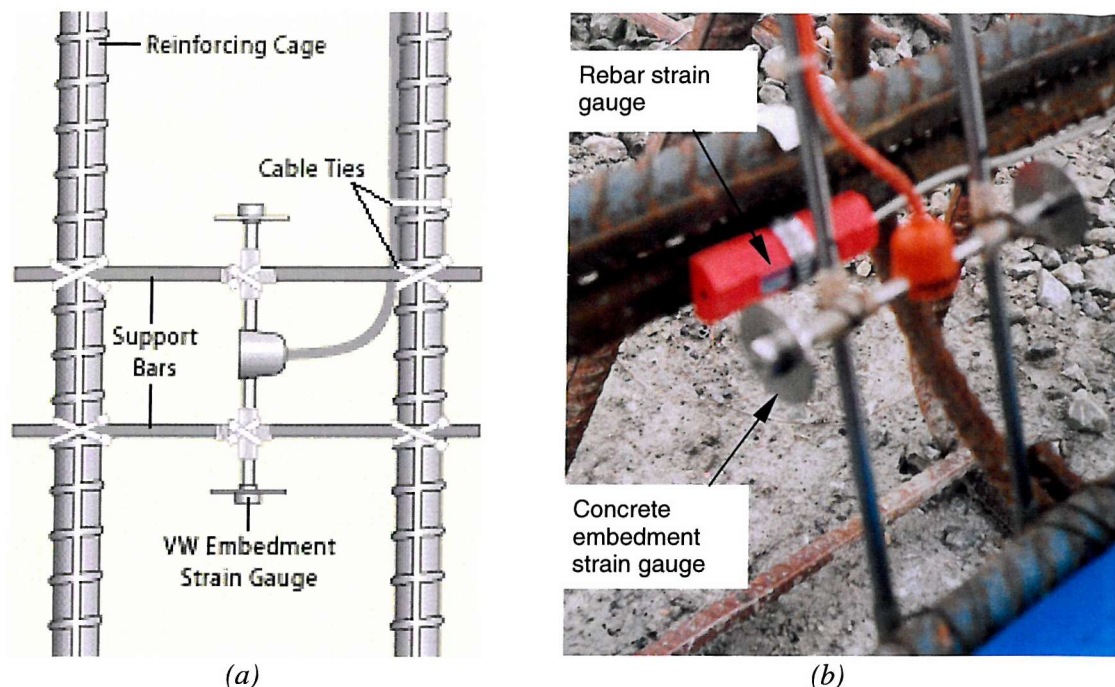


Figure 3.7. Installation of the Gage Technique concrete embedment gauges: (a) diagrammatic layout of the gauges showing the support steel and the placement of gauges between the pile reinforcement (taken from Gage Technique installation instructions); and (b) the gauges as installed in the piles at Hildenborough.

Inclinometers

The inclinometer tubes installed within the piles were attached to the pile reinforcement cage using steel brackets, but such that some flexibility was still afforded to the tubes so that they were not damaged during pile installation. All inclinometer tubes installed were 70mm diameter. The inclinometer tubes did not extend beyond the base of the base of the piles, and analysis of the results assumes the pile toe is fixed and does not move laterally.

The 100mm diameter holes for the inclinometer tubes installed midway between the piles were made using a shell and auger tripod rig. As this was unable to penetrate the granular rockfill material used to construct the berm, the granular rockfill in the zone between the piles was excavated out and replaced with Type 1 granular fill material, and the holes were then augered through the Type 1 material. Within the clay, the inclinometer tubes were grouted in using a cement/bentonite grout designed to give a stiffness similar to that for published values of Weald Clay soil stiffness. Where the tubes passed through the Type 1 granular material, the tubes were later concreted into the hole.

The slope inclinometer tubes were installed on days three and four after pile construction, with the holes dug in the rockfill material and refilled with Type 1 fill two days after pile construction. The influence on the behaviour of the piles and the installed slope inclinometer tubes of excavating a substantial hole within the rockfill and its replacement with a different

material is uncertain. Care was taken to keep the plan area of the holes as small as possible, and to ensure that on re-filling the holes the Type 1 material was well compacted.

Dummy pile

A short, 1.6m long, dummy pile was constructed within the granular rockfill material midway between two stabilising piles and 3.6m away from instrumented Pile C. The dummy pile contained two rebar strain gauges attached to a 1.6m long replica pile cage (made up of 6 no. T25 reinforcing bars). The dummy pile was designed to measure the strain in the rebar type gauges as a result of concrete shrinkage, so that the impact of concrete shrinkage on the bending strain caused by genuine pile loading could be assessed. The strain gauges were located within the vertical plane perpendicular to the cross section of the slope, and should therefore have been insensitive to bending induced by in-plane movements of the slope. While the positioning of the dummy pile within a slope that nonetheless had the potential to move was not ideal, the very restricted space available on site meant that it had to be constructed somewhere either on the embankment or close to the embankment toe.

3.3 Pile design

Pile design was carried out by Mott MacDonald Ltd. Design of the piles used limit equilibrium slope stability to assess the factor of safety of the slope. A point force, modelling the effect of the pile, was introduced into the analysis to increase the factor of safety of the slope.

As relatively little was known about the pore water pressures in the embankment, a pore pressure condition was specified in the analysis that gave a factor of safety of just above 1.0 (the slope was assumed to be close to failure based on the failure of adjacent sections of the embankment). The piezometric line put into the slope stability analysis ran at a depth of about 0.5 m below the lower embankment slope surface to the mid-height of the embankment, above which pore pressures are assumed to be zero and below which they are hydrostatic. The placement of the granular rockfill on the surface of the slope was then modelled in the slope stability analysis by the addition of a zone of higher strength material on the slope surface. The point force applied to model the piles was increased in magnitude until a factor of safety for the slope of 1.3 was obtained. The method assumed that the critical failure surface was unaltered by the insertion of the piles, and the point force was applied where the slip surface and the pile intersect (the design limit equilibrium slope stability analysis is shown in Figure 3.8). The soil strength parameters used in the analysis were those given in Table 3.1.

The point force of 144 kN (per pile, or per 2.4m width of the slope) obtained was then converted into an equivalent triangular pressure distribution acting over the 4.0 m length of the pile above the prospective slip surface. The depth of pile required to resist the lateral loading from the slipping mass was then determined using horizontal equilibrium, with the resistance to the pile head loading acting upslope as a uniform distribution beneath the failure surface (Figure 3.9a). This pressure distribution was calculated using the lateral limiting pile pressure p_u (Ellis, 2002). This gave a maximum bending moment of 230 kNm at 4.5 m depth (Figure 3.9b). The pile length was extended to 10m to ensure a good base fixity in the intact Weald Clay underlying the embankment, and to ensure an adequate factor of safety for global slope stability, which was checked by forcing prospective failure surfaces to run beneath the base of the pile (Ellis, 2002).

The point force was applied within the limit equilibrium slope stability analysis at the intersection between the pile and the failure surface. This same force was however applied to the pile bending moment calculation as a triangular pressure distribution, the resultant of which acts at a point two-thirds of the way up the section of the pile above the failure surface. It is desirable to represent the force applied to the pile as triangular pressure distribution as this allows the calculation of a bending moment profile within the upper section of the pile above and just below the failure surface. However, if the calculation was to be totally correct, the point force should have been applied within the limit equilibrium slope stability analysis at a point two-thirds the way up the upper section of pile. When circular failure surfaces are used, this will produce a larger point force because the moment lever arm in the Bishop limit equilibrium slope stability analysis is reduced in length.

The length of the pile is longer than required for horizontal and moment stability, which makes calculating the bending distribution for the lower 5.5m of the pile difficult. If it is assumed that the whole pile will act to resist the force applied by the failing mass of the slope, then the full bending moment distribution in Figure 3.9c could be inferred. As there is no calculation for the bending moment in the lower section of the pile and because there was additional steel available on site, the bending capacity over the bottom 7m section was increased with the addition of more steel.

It is important to note that the calculation is based on the pile restoring force required to increase the factor of safety of the slope from a metastable state with a factor of safety of just above 1.0, to a factor of safety of 1.3. The increased factor of safety is to allow for future deterioration of the slope, and the actual restoring force required to improve the stability of the piles at the current time is likely to be much smaller. Hence, the actual or measured bending moments would be expected to be significantly smaller than the pile bending capacity specified.

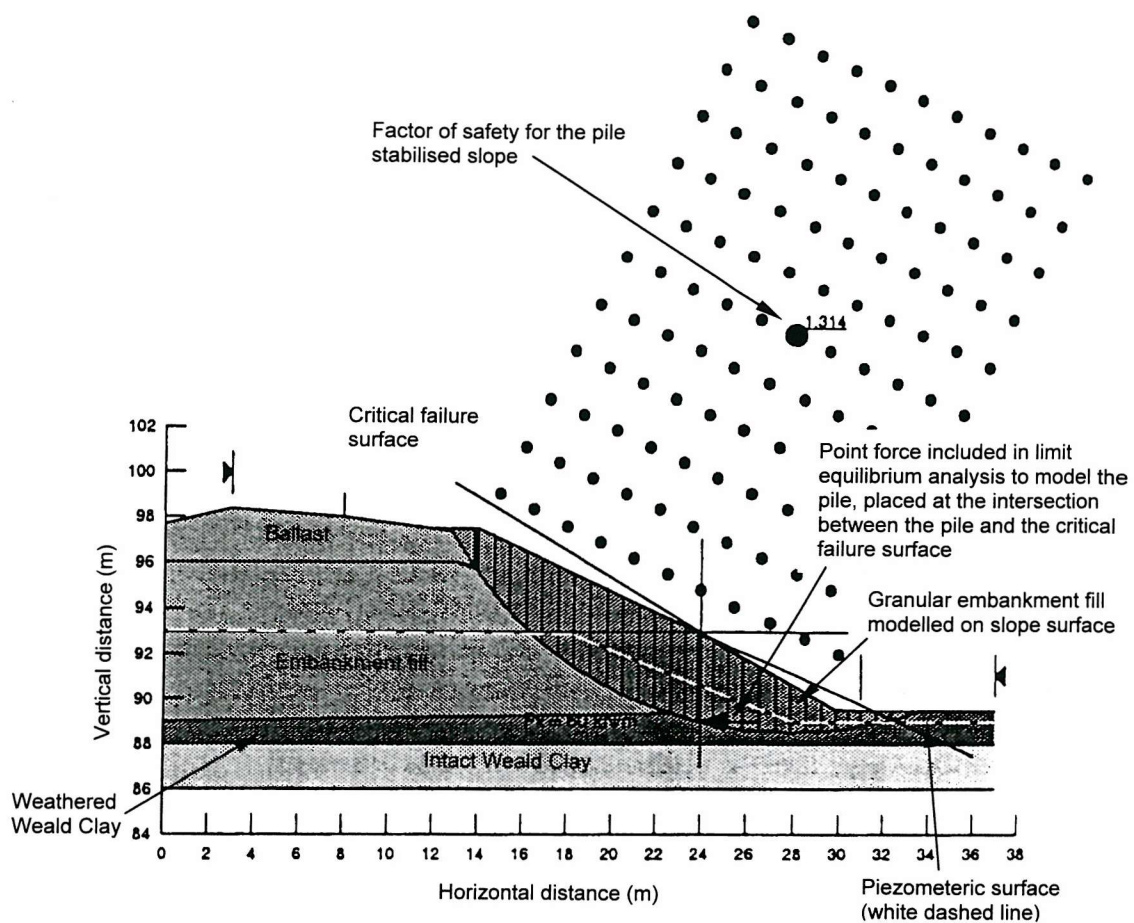


Figure 3.8. Limit equilibrium slope stability analysis of the slope at Hildenborough, with the pile modelled as a point force (taken from Mott MacDonald design calculations).

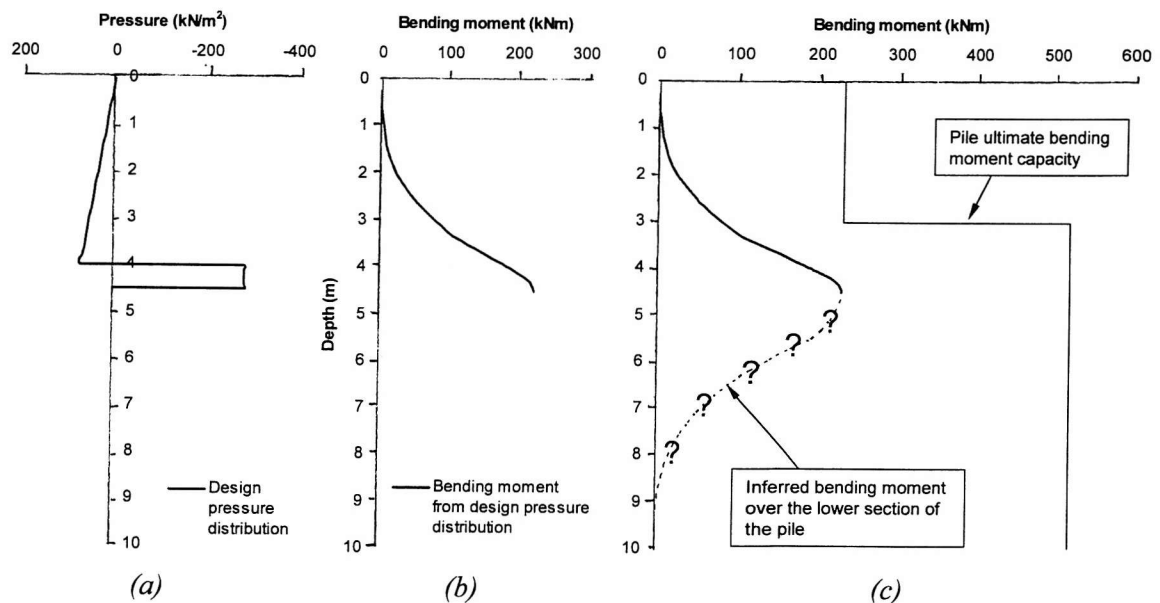


Figure 3.9. The design bending moment and the pile bending capacity specified.

3.4 Measurement of bending moment

3.4.1 Bending moment calculation

Standard engineering beam theory (Equation 3.1) is used to convert the longitudinal strains ε_1 and ε_2 measured by each pair of strain gauges into bending moment, M:

$$M = \frac{EI(\varepsilon_1 - \varepsilon_2)}{y} \quad \text{Equation 3.1}$$

where E is the Young's modulus of concrete, I is the second moment of area and y is the distance between the gauges in each pair. The composite flexural rigidity, EI, of $171 \times 10^3 \text{ kNm}^2$ was calculated using $E = 25 \times 10^6 \text{ kN/m}^2$ for concrete (British Standards Institution, 1985).

Estimating the bending moment using Equation 3.1 is not straightforward, because the flexural rigidity of the piles varies with bending moment when the concrete piles are in a partly cracked state. Uncracked behaviour is normally assumed in the analysis of field measurements of retaining walls (Tedd *et al*, 1984; Wood and Perrin, 1984), and has been used in the analysis of the results in this thesis.

3.4.2 Concrete shrinkage

A problem with measuring strain in concrete piles placed directly into a slope is distinguishing between the strain developed by loading of the pile and the strain developed by the shrinkage effects as the concrete cures. While the concrete is in a fluid or plastic state, initial shrinkage occurs due to a reduction in the system cement and water due to chemical reaction, and loss of water through evaporation and absorption by the formwork or ground. This shrinkage may be masked by the thermal expansion as the concrete temperature increases. A later drying shrinkage occurs as the concrete hardens and is allowed to dry, and this is normally accompanied by a thermal cooling shrinkage (Murdock *et al*, 1991).

A foundation that remains immersed in groundwater from casting will generally expand, with the expansion about one-quarter of concrete shrinkage (Murdock *et al*, 1991). However, on drying the concrete will shrink as before. The groundwater level at Hildenborough is about 1m below the intact Weald Clay on which the embankment is constructed, leaving approximately 5m or one-half of the pile beneath the water table.

The bending moment is calculated from the difference in the strain readings on either side of the pile. In theory, the shrinkage effects of the concrete should affect all of the strain gauges equally, but in reality concrete is not a perfectly homogenous material, and shrinkage strains

may not be uniform throughout the pile section. Differential concrete shrinkage may be interpreted as a bending moment in the pile. Estimates of final concrete shrinkage calculated using empirical formulae based on the properties of the concrete mix and the amount of steel reinforcement in the section gives values of 105-130 microstrain (see Appendix 1). These shrinkage strains are of almost the same size as the bending strains required to generate a bending moment of 200kNm in the pile section. Appendix 1 shows that a difference of 5% in the shrinkage strains acting on a pair of the embedment strain gauges will give a bending moment misread of about 5kNm. Because the concrete embedment gauges are closer to the centre of the pile section than the rebar gauges, the measured concrete strains at a given pile bending moment are smaller, and subsequently the reading from these gauges becomes more susceptible to differential concrete shrinkage.

Concrete shrinkage will continue over a period of several years in large concrete specimens (Atherton, 1963). However, much of the concrete shrinkage (at least a third of the total) will typically occur in the first few hours after casting (see Appendix 1).

3.4.3 Temperature effects and concrete cracking

The change in temperature as concrete heats and cools can affect the rate and magnitude of concrete shrinkage, but the change in temperature of the concrete also causes the vibrating wire in the gauges to expand and contract, which affects the strain reading obtained. In theory, as for shrinkage, the temperature should affect each of the pair of gauges used to measure bending moment equally, but there is a possibility that differences in temperature may affect the bending moment readings. The temperature of the concrete will also fluctuate with the average air temperature during the seasons. It is not common to adjust for the effects of temperature within concrete walls in which strain is being measured to calculate bending moment, and no adjustments have been made to the strain data in this thesis. Analysis of the data from the thermistors in the pairs of gauges down the pile show that most are reading within 1°C of each other.

Data from the gauge thermistors shows that the temperature of the piles appears to reach a peak of about 15°C approximately 24 hours after pile installation, cooling to a fairly steady temperature of 7°C at the base of the pile after about 4 months (the temperature at the top of the pile fluctuates with average air temperature through the seasons).

Additional factors affecting the measurement of strain in concrete include cracking in the tension side of the pile, and subsequent stress relief in the concrete adjacent to the cracks (Roesler and Barenberg, 1999). Stress has been shown to increase in the reinforcing bars passing through cracks in concrete (Scott and Gill, 1987; Daoud and Lovegrove, 1983). These

effects are very difficult to remove from the strain measurements, as there is no knowledge of whether or how the pile has cracked.

3.5 Bending moment results

3.5.1 Bending moments

The pile bending moments measured in each of the piles A, B and C are shown in Figures 3.10, 3.11, 3.12 and 3.13. In all of the figures, positive bending moment corresponds to tension on the upslope side of the pile (or curvature of the pile downslope). A comparison of the results from each of the piles shows some variation in the bending moment profile measured. Some of the bending moment values measured in piles A and B were considered to be unreliable; Pile C is therefore considered the most representative of the true pile behaviour and the results from Pile C are described in detail first.

3.5.2 Pile C

The data from Pile C (Figures 3.10 and 3.11) show that the bending distributions obtained from the concrete embedment and rebar strain gauges do not completely agree (example results are plotted together in Figure 3.14). While the distributions are broadly the same shape, in that there are negative moments at the top and base of the pile and a positive moment in the middle, the depth and magnitude of the peak moments varies considerably. Some of the possible explanations for this variation have been discussed in Section 3.4, and include the effect on the different types of strain gauges of concrete cracking and shrinkage, and the pile cage installation method. However, this presents a stark illustration of how difficult measuring bending strain in concrete can be, particularly when concrete shrinkage processes are happening at the same time as pile loading, and the measured bending moments are fairly small.

A study of the evolution of the bending moments with time shows that significant bending moments were developed in the first 40 days or so; beyond this some smaller changes in bending moment occurred up to day 200, after which changes are mostly small. By plotting the bending moments at individual gauge locations against time (Figures 3.15, 3.16, 3.17, 3.18, 3.19 and 3.20) it is possible to see that significant moments developed in the first few days, and also that there was a large jump in bending moment at some gauge locations on day 22 and 23, coinciding with the regrading of the rockfill on the embankment into a two stage slope. Days 20 and 21 were also particularly wet. Unfortunately, the logger failed after taking 5 initial sets of readings, and this was not detected for the next 4 days – consequently the initial jump in bending moment is not recorded (in Figures 3.15-3.20 the increase in bending moment from zero at the very start of the graphs is missing). A number of the pairs of gauges show a



progressive change in bending moment after the jump in the readings on days 21 and 22; this may be attributable to settlement of the embankment as result of a gradual reduction in the pore pressures generated by increasing the volume of rockfill on the slope above the pile row. After the initial loading over the first 25 days, the maximum negative moment at the head of the pile decreases with time for both types of gauge, and the maximum positive moment in the middle of the pile increases for both of the gauge types.

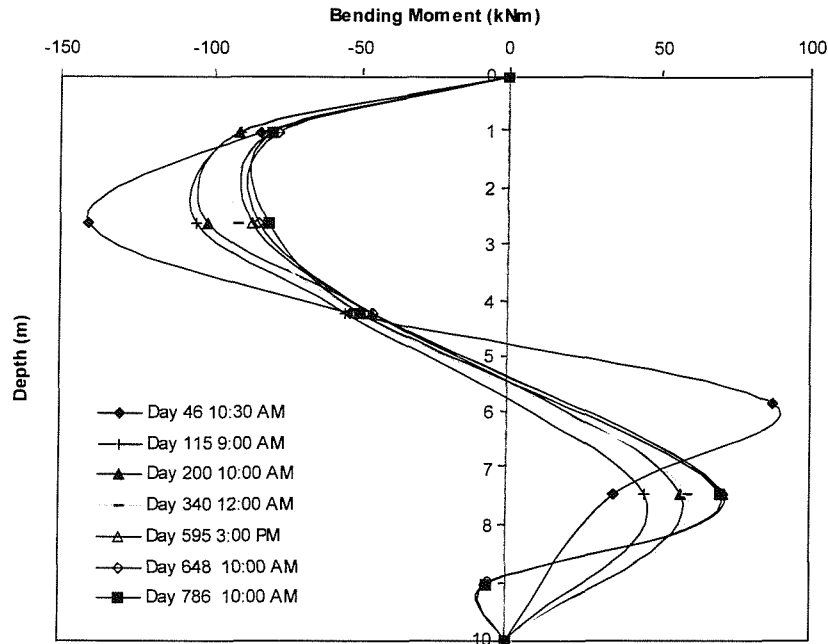


Figure 3.10. Bending moments measured in Pile C using Slope Indicator rebar strain gauges

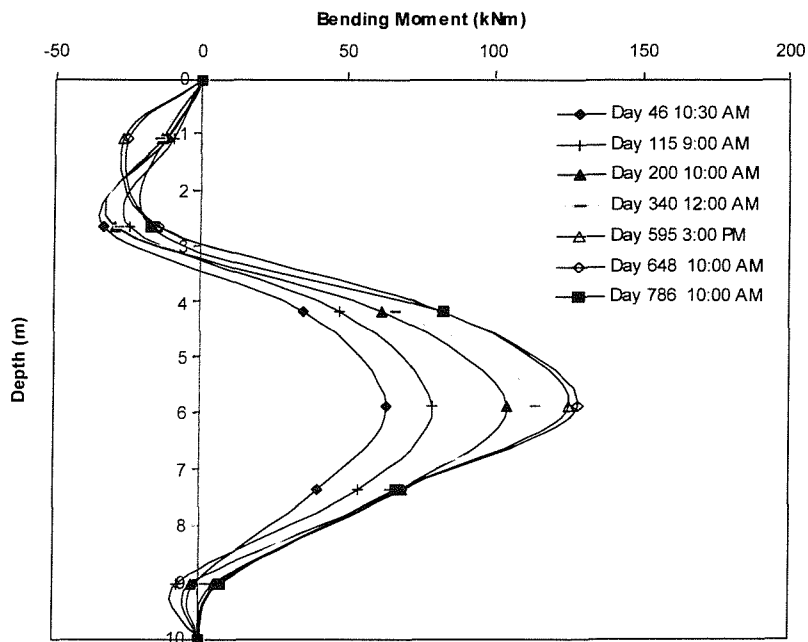


Figure 3.11. Bending moments measured in Pile C using Gage Technique concrete embedment strain gauges

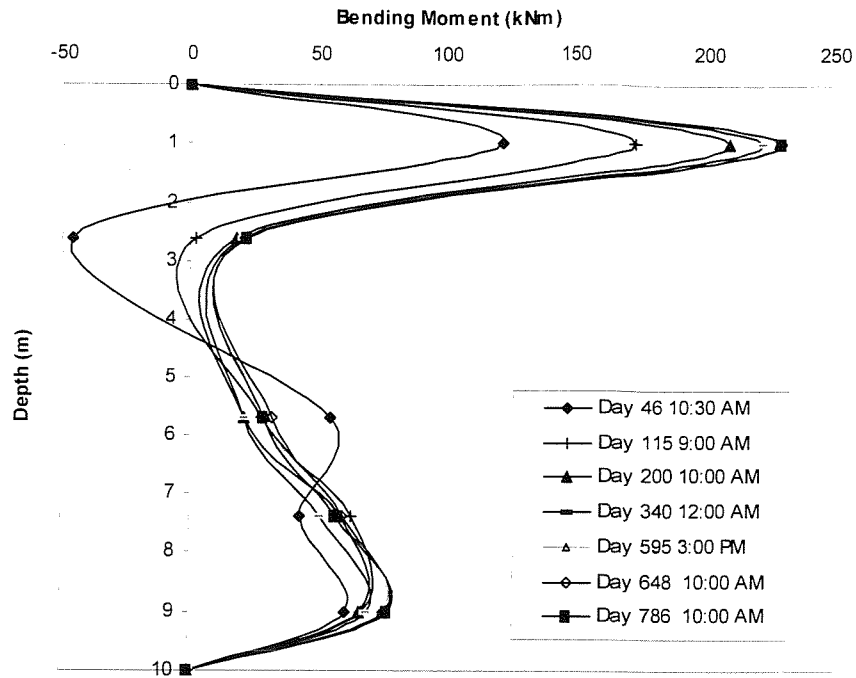


Figure 3.12. Bending moments measured in Pile A

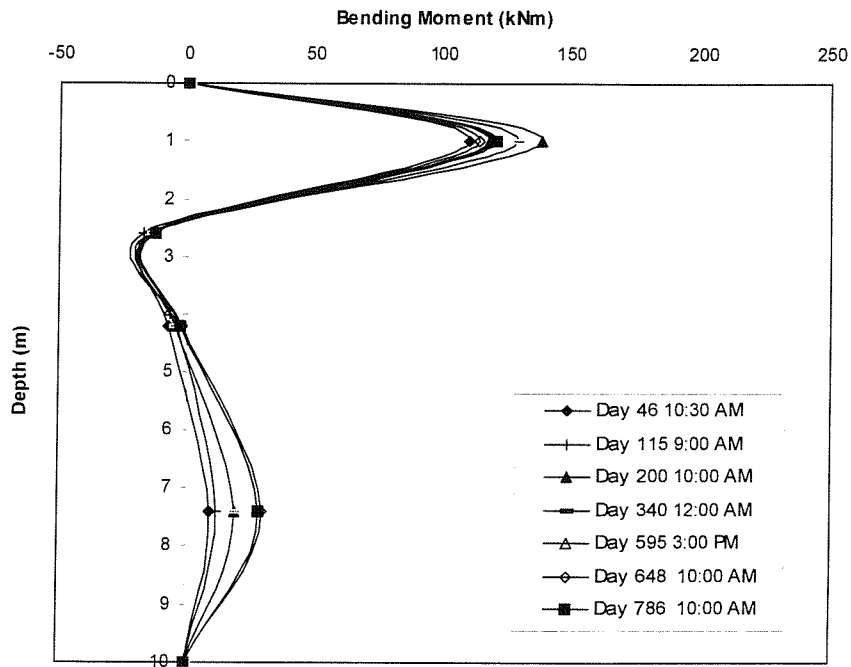


Figure 3.13. Bending moments measured in Pile B

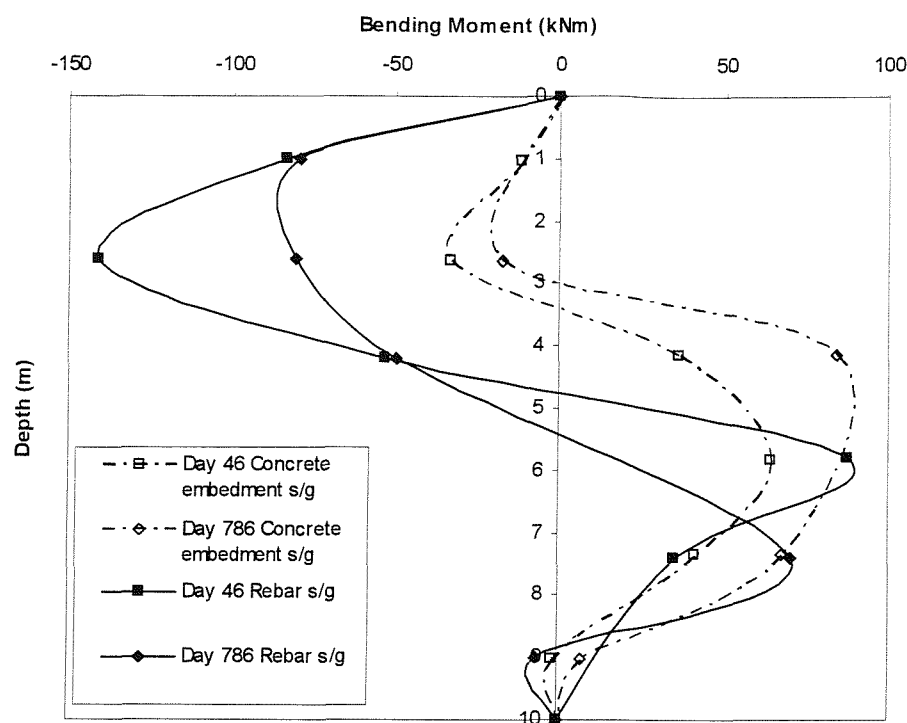


Figure 3.14. Bending moments in Pile C as measured by the concrete embedment and rebar type strain gauges.

It is clear that much of the pile loading occurred in the first 100 days or so, and it is possible to look more closely at the evolution of bending moment for both sets of gauges over this period (Figure 3.21). Figure 3.21 shows that the bending moment measured by the concrete embedment gauges on Pile C over the first 17 days is close to zero (or slightly positive) over much of the pile length, whereas the bending moment measured by the rebar gauges over the first 7 days are larger, and up to 80kNm in magnitude. It can be seen that the form of the bending moment profiles for the rebar gauges on the first few days after installation (days 7 and 17) echoes the later bending behaviour of the pile, while the profiles from the concrete embedment gauges for the early dates are rather different from those during the later stages of pile bending.

At the 2.6m deep gauge location on day 17 there is a difference in reading of almost 100kNm between the concrete embedment and rebar gauges. The calculation of the effect on the gauge readings from concrete shrinkage (Section 3.4.2 and Appendix 1) shows that the differential shrinkage could be significant (with a 5% difference in shrinkage causing a misread of 5kNm in the embedment gauges), but it is surprising that the differential shrinkage could cause a difference in reading of 100kNm between the gauge types. On the lower 5m of the pile below the water table where the presence of water should restrict concrete shrinkage, the readings from the different gauge types are much closer (see Figures 3.14 and 3.21).

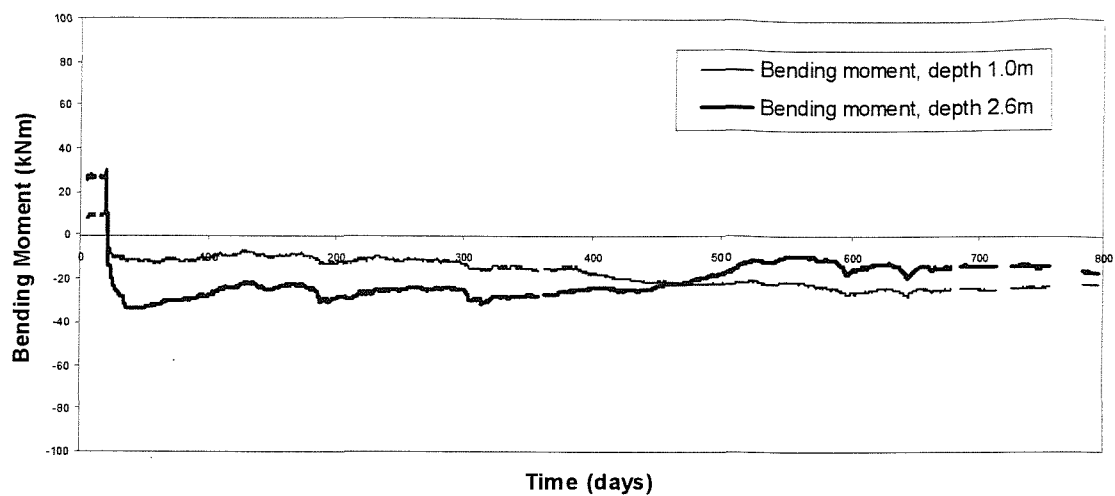


Figure 3.15. Plot of bending moment verses time, for the top 2 pairs of concrete embedment gauges in Pile C

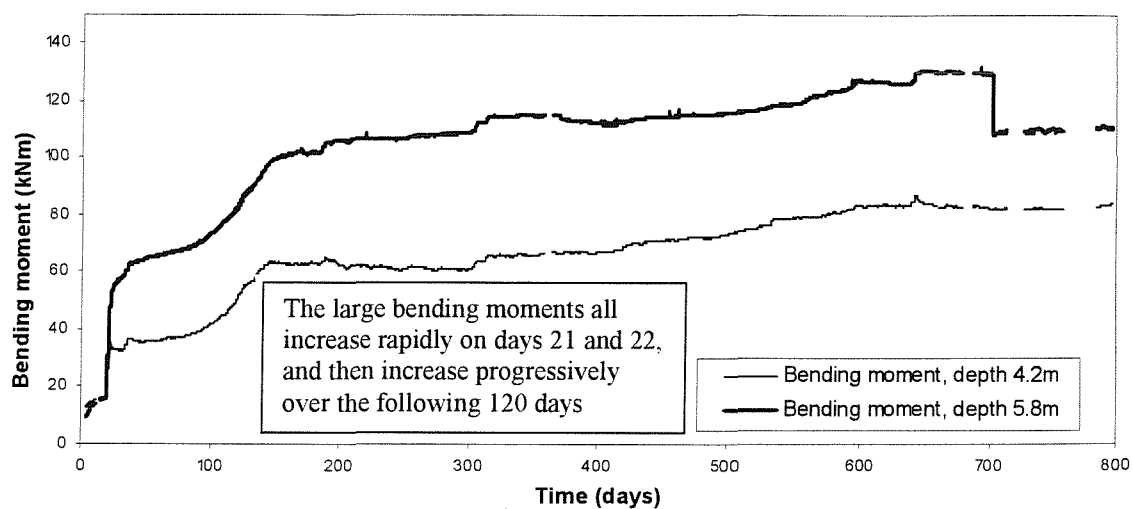


Figure 3.16. Plot of bending moment verses time, for the middle 2 pairs of concrete embedment gauges in Pile C

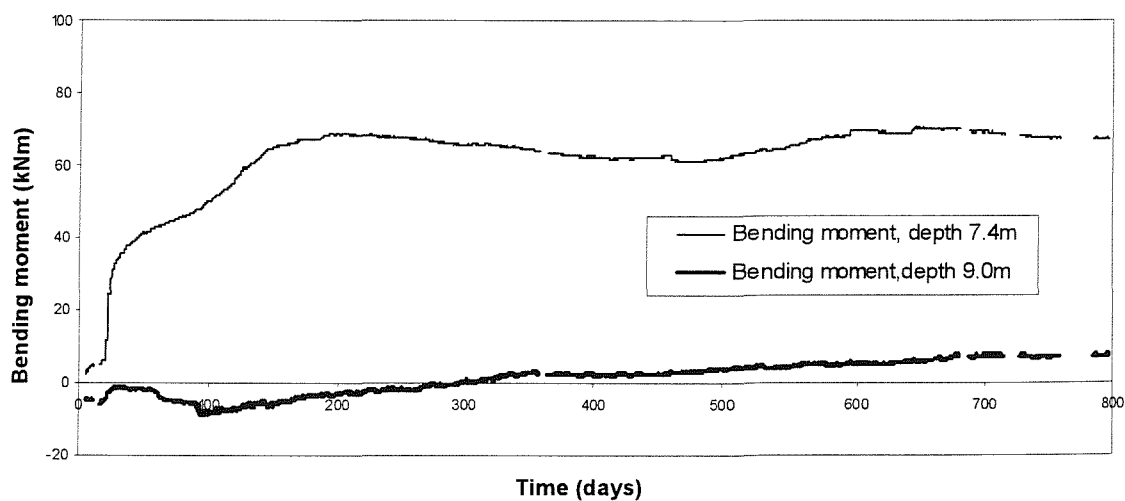


Figure 3.17. Plot of bending moment verses time, for the bottom 2 pairs of concrete embedment gauges in Pile C

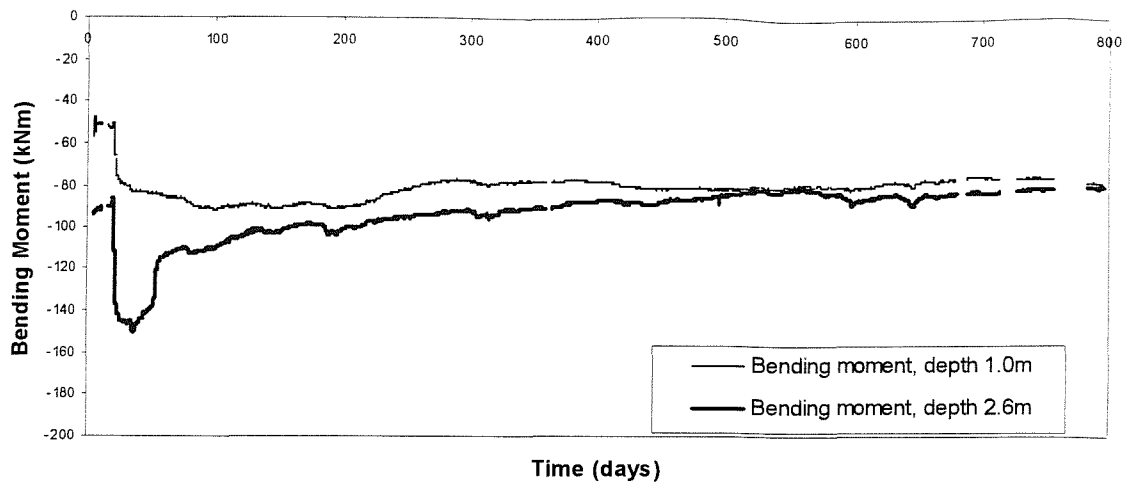


Figure 3.18. Plot of bending moment verses time, for the upper 2 pairs of rebar gauges in Pile C

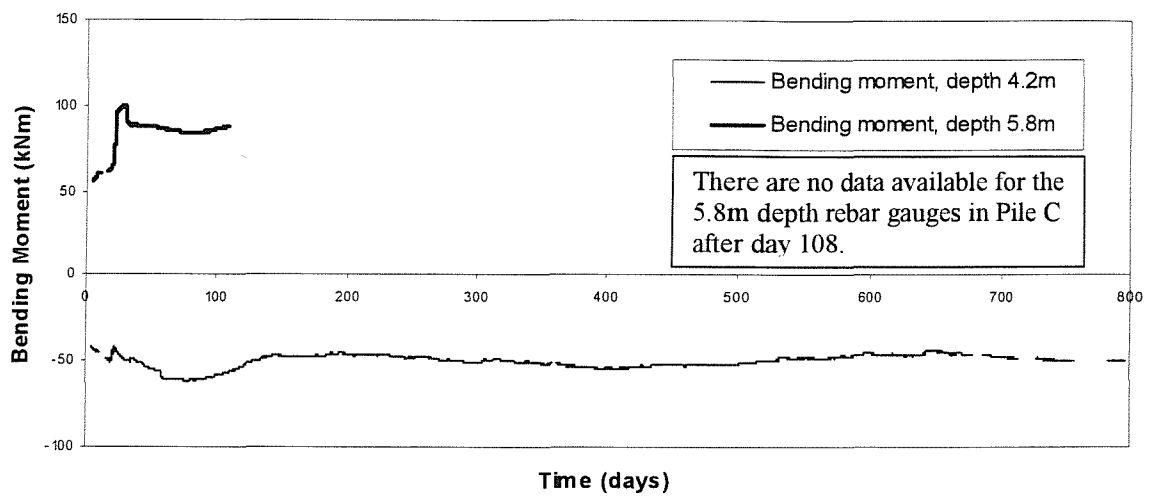


Figure 3.19. Plot of bending moment verses time, for the middle 2 pairs of rebar gauges in Pile C

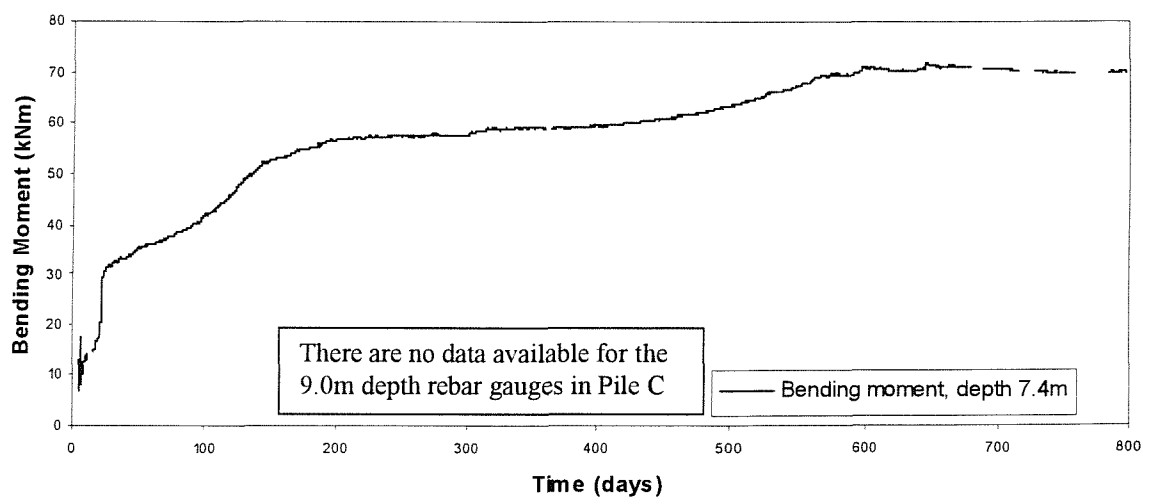


Figure 3.20. Plot of bending moment verses time, for the lower rebar gauges in Pile C

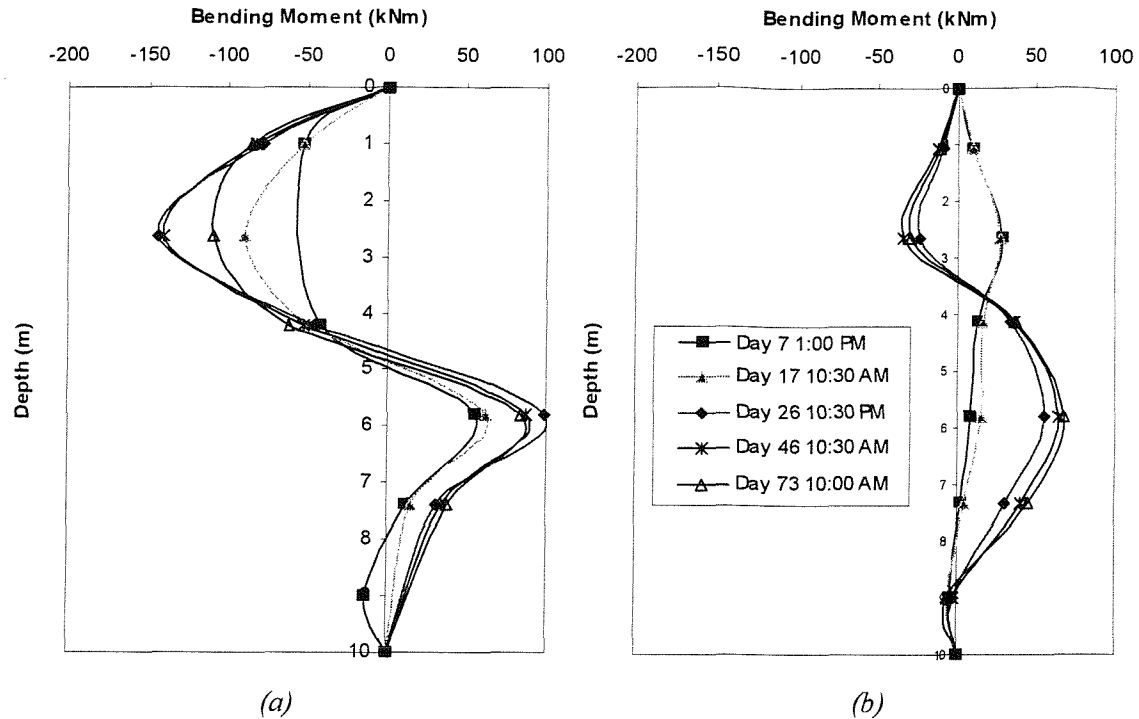


Figure 3.21. Early development of bending moment in Pile C as recorded by (a) rebar strain gauges and (b) concrete embedment gauges

Smethurst (2001) attempts to analyse the data from the two sets of gauges using later strain datum points, to try to remove any early age shrinkage effects from the data and to see whether there was a better correlation in the later bending moment development between the two sets of gauges. Use of a datum 5 days after pile installation does improve the correlation between the concrete and rebar embedment gauges (Figure 3.22). The negative moments at the head of the pile do seem closer, but this is difficult to fully assess as there is no reading for one of the rebar gauges at 2.6m depth on Day 5 and hence no point plotted for this pair of gauges on Figure 3.22a. The magnitude of the bending moments is substantially reduced, and it is uncertain whether use of this later strain datum may be removing genuine pile loading that has occurred in the first five days.

If the differences in the readings from the two gauge types can be attributed to concrete shrinkage effects, it could be argued that the concrete embedment gauges were affected by early concrete shrinkage that has distorted the genuine bending moment measured in the pile over the first 17 days. However, the big difference in the moment reading between the gauge types at 2.6m depth points towards the possibility that both sets of gauges are being affected by concrete shrinkage, or that other factors such as the impact of the pile construction method on the embedment gauges (discussed in Section 3.2.3) is having an effect on the readings.

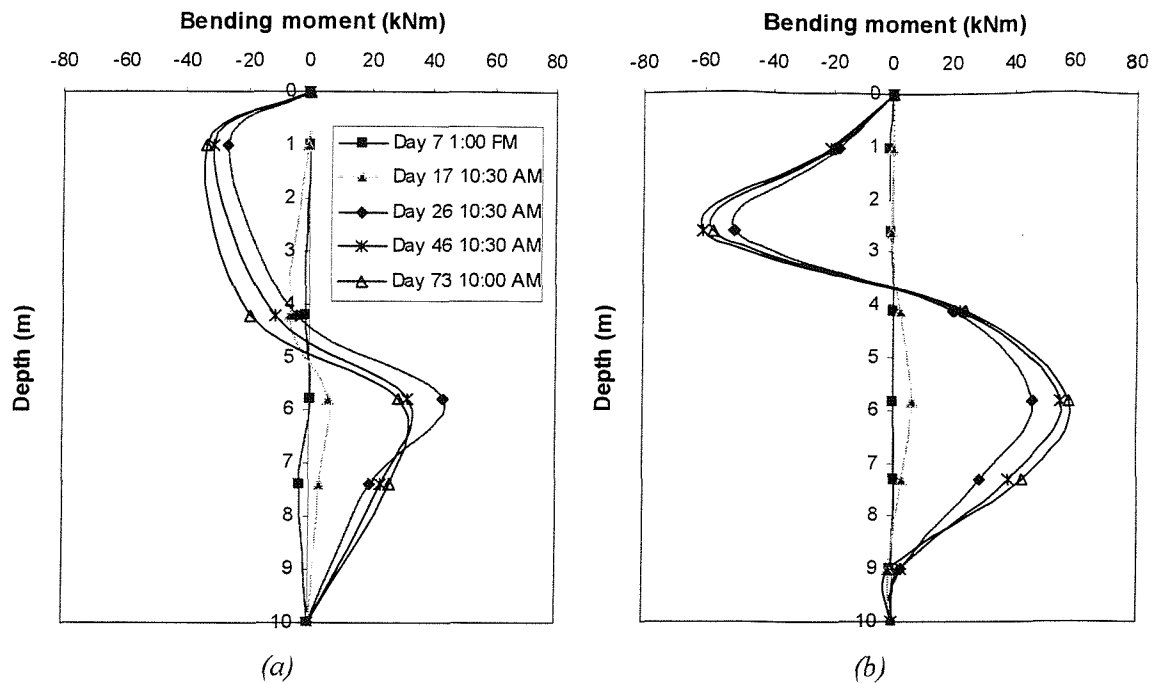


Figure 3.22. Pile C bending moment calculated using a strain datum 5 days after pile installation: (a) rebar strain gauges; and (b) concrete embedment gauges

Distribution of pile pressure

It is possible to fit curves between the bending moment data points in an attempt to derive the net soil pressure acting on the pile. This is not straightforward, as both rebar and embedment gauges have at some locations failed leaving fewer data points to which to fit the data, and when combined with a complex bending behaviour, assumptions have to be made about exactly how the bending moment distribution should behave between gauge locations. Spline fitting is difficult for a number of these distributions, not least due to the tendency for large bending moments very close to the heads of the piles. This makes it very difficult to get the fitted curve to increase from zero at the pile head as a second or third order distribution with depth – consistent with a rectangular or triangular pressure distribution acting over the head of the pile. Instead, the fitted curves tend to approach the zero bending moment point at the top of the pile as a straight line or linear distribution with pile depth – this is consistent with a large shear force acting the very head of the pile, a shear force which is not present in reality. Using a system of 4th order polynomial splines (Georgiadis *et al*, 1992) fitted between the bending moment data points, requires in several cases the addition of a significant shear force at the head of the pile in order to obtain a reasonable distribution of bending moment between the fitted data points. If this shear force is added to the very head of the pile, it then distorts the pressure diagram, which changes to counteract the action of the shear force (as the system of splines is in equilibrium). Another (and possibly better) solution is not to fit the bending moment spline directly through data points at the pile head, but to instead allow it to form the second or third order distribution with depth consistent with a rectangular or triangular distribution of pressure. The same

problems are encountered if high order polynomials are fitted through the bending moment data points.

An Excel spreadsheet was written to fit the 4th order polynomial splines (Georgiadis *et al*, 1992) to the bending moment data. Figures 3.23 and 3.24 show the splines fitted between the bending moment data points for Pile C, and the net pressure diagrams derived from these.

In Figure 3.23, where the gauge at 5.8m depth has failed, it is assumed that the bending moment reaches a peak above the data point at 7.4m – this is not unreasonable as before the gauge at this location failed, the bending moment recorded at 5.8m depth was larger (see Figure 3.14). When the splines were fitted directly to the measured data points, the distribution of bending required for the splines to be in equilibrium was unrealistic, with the bending moment zigzagging wildly between adjacent data points. To get the system of splines to fit a reasonable distribution between the data points (and remain in equilibrium), a shear force of magnitude 100 kN acting upslope can be applied to the head of the pile. As already discussed above, this distorts the pressure distribution and makes pile behaviour very difficult to interpret. As an alternative solution, the 1.0m gauge reading within the system of splines was reduced from -80 kNm to -25 kNm, to encourage development of a curving distribution of bending moment from the pile head. While it is not entirely desirable to adjust the measured data, this does appear to be the only way that a reasonable distribution of bending moment can be fitted. The resulting pressure distribution is shown in Figure 3.23b. There is a net downslope pressure acting between 1 and 4m depth, below which the pile resists this pressure with a fixed earth point of rotation. The restraint to pile movement in the failing mass is represented by a net upslope pressure on the top 1m of the pile. The pile bends downslope at the base, and upslope at the head. Note that this bending behaviour is contrary to the pile design, which assumed the piles to bend uniformly downslope.

Figure 3.24 shows the net pressure distribution as a result of fitting a spline between the bending moment data points derived from the concrete embedment strain gauges. The pair of gauges at 5.8m depth have recently failed, and the bending moment was assumed to reach a peak at this depth of about 130 kNm, which was the last bending moment recorded at this location (and shown by day 648 on Figure 3.11). In this case, there was no need to adjust bending moment data points to get a good fit from the spline. Considering the differences in the bending moment distributions for the concrete embedment and rebar gauges, the net pressure distributions are very similar in both shape and magnitude. Study of the Pile C displacements taken from the inclinometers (Figure 3.32) shows a displacement profile that broadly fits the bending and pressure distributions measured in the pile.

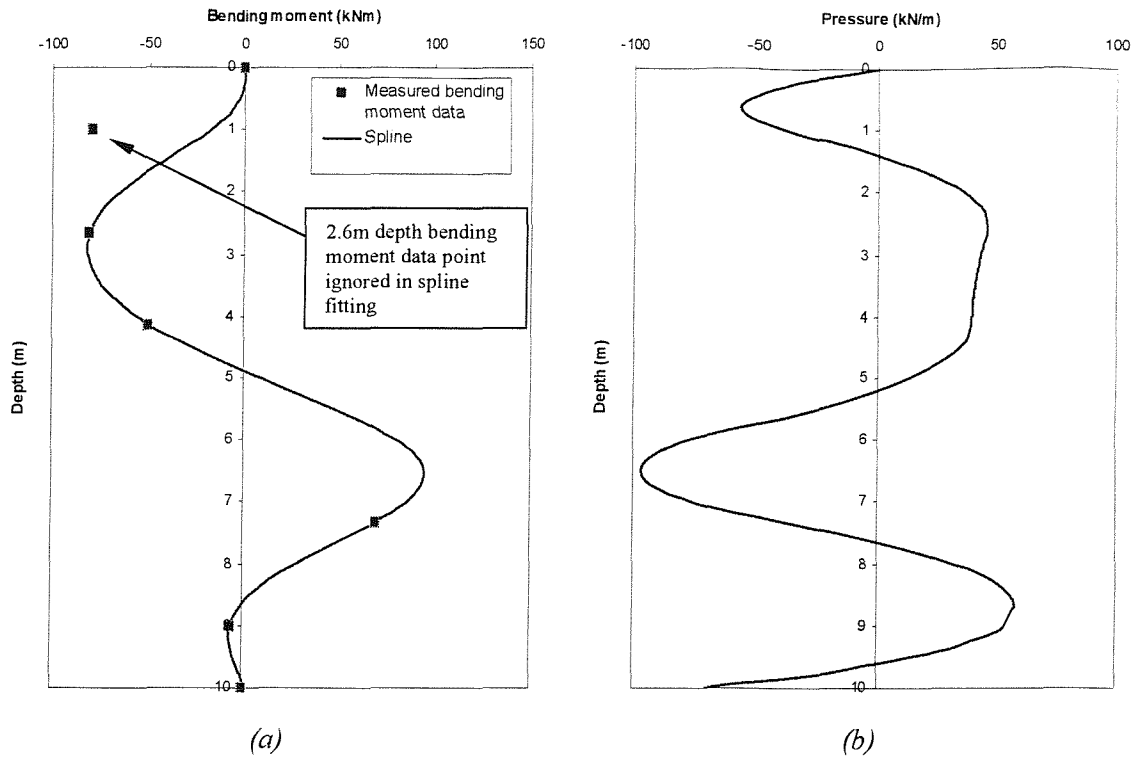


Figure 3.23. (a) The spline fitted to the bending moment data points (for Day 786) recorded by the rebar strain gauges in Pile C, and (b) the resulting pressure net distribution (in which a positive pressure acts downslope).

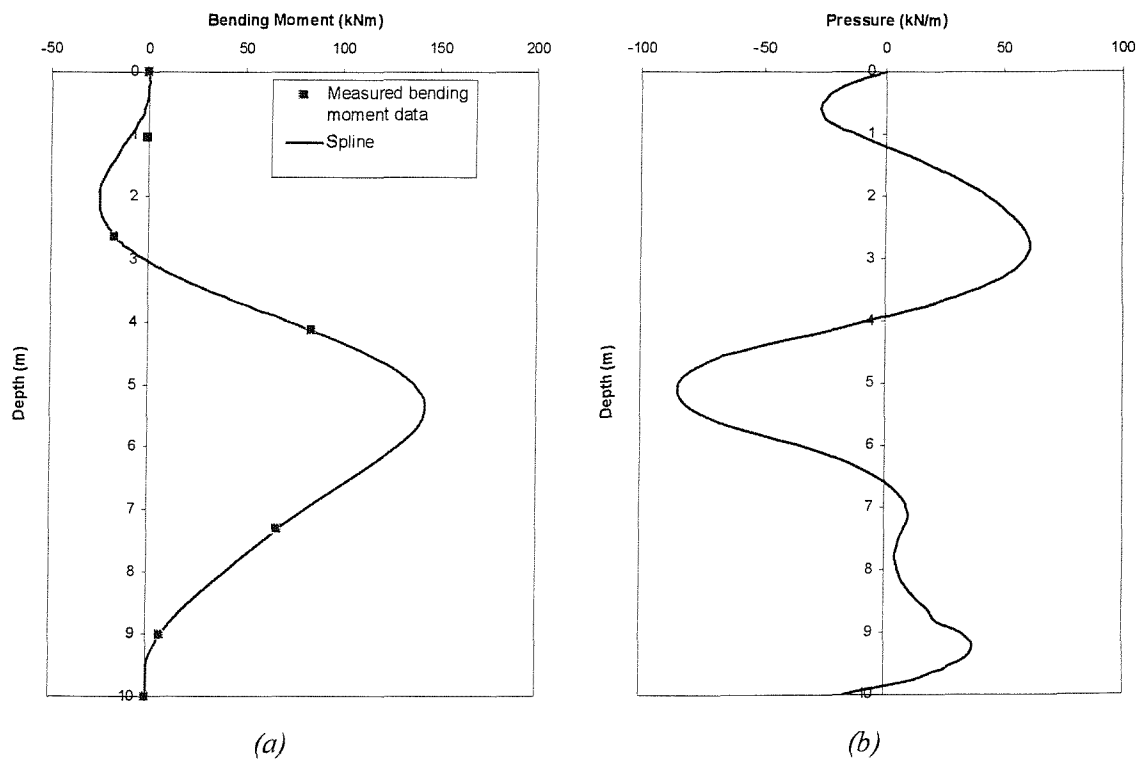


Figure 3.24. (a) The spline fitted to the bending moment data points (for Day 786) recorded by the concrete embedment strain gauges in Pile C, and (b) the resulting net pressure distribution (in which a positive pressure acts downslope).

3.5.3. Piles A and B

The bending moments in piles A and B (Figures 3.12 and 3.13) are similar in both profile and magnitude, with a large positive bending moment of 150-200 kNm acting close to the head. This is consistent with the very head of the pile bending downslope. However, a pressure of 666 kN/m² acting onto the top 1m length of the pile would be required to generate a moment of 200 kNm at the top gauge location. This pressure is very large and it is unlikely that the movement of the slope would generate such a large pile loading over the top metre of the pile. This loading is much larger than the design pile loading of 144 kN obtained from the limit equilibrium slope stability analysis of the slope. The inclinometer data also show that slope movements predominantly occur at 4m depth (Figures 3.28 and 3.29).

It has been suggested that the large positive bending moment reading at 1m depth on piles A and B may be caused by lack of steel bond length in the end of the pile. A calculation of the minimum steel bond length required using Eurocode 2 (British Standards Institution, 1993) gives 1070 mm, which just encompasses the top pair of strain gauges in each pile. However, bond length is the length required for the steel and concrete bond to develop stress, and slippage in the concrete/steel bond would cause the stress in the reinforcing bars to reduce rather than increase (as required to generate a large bending moment). Another possible explanation for the very large measured bending moments include the effect of plant traversing behind the instrumented piles shortly after construction. The piles were constructed without any casing, and there is the possibility that rockfill close to the slope surface could have been displaced into the wet concrete as a result of plant movement directly after construction, possibly loading or deforming the pile cage. However, while the development of the large bending moment in Pile B occurs from a very early stage (see Figure 3.25), the majority of the moment in Pile A develops on day 22, consistent with the regrading of the surface rockfill also seen to increase bending moments in Pile C.

As for Pile C, the bending moments measured in Piles A and B are generated in the first 200 days after pile installation, with a large increase seen on days 22 and 23 when the rockfill is regraded on the slope surface. The bending moments towards the toe of both piles A and B are positive, as in Pile C, although this moment is smaller in Pile B than in Pile A or C. A positive bending moment indicates that the pile is bending downslope, and that the toe of the piles is restrained within the intact Weald Clay (this is also borne out by the inclinometer displacement data, Figures 3.30 and 3.31). Over the upper half of Piles A and B the bending moment reduces to close to zero at 2.6 and 4.2m depth (and goes negative on Pile B at 2.6m depth) indicating that were it not for the large positive moment reading at the top gauge location, the bending moment would probably go negative over the top 4m of the piles, as in Pile C.

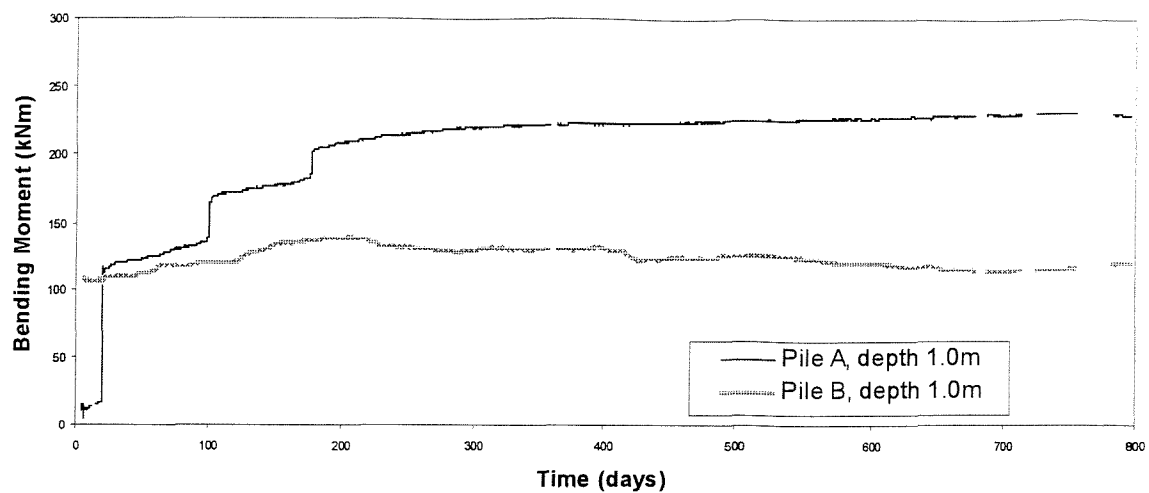


Figure 3.25. Development of bending moment with time for the top (1.0m depth) set of strain gauges on Piles A and B.

While it is not clear exactly what causes the large positive moment to occur at the top gauge location on Piles A and B, it is clear that this moment is probably not the result of genuine pile loading, and instead is caused by near surface pile construction effects.

Distribution of pile pressure

Fourth order polynomial splines have been fitted to the bending moment data points to find the net soil pressure distribution acting on Piles A and B. It was not possible to fit a realistic bending moment distribution to the data with the 1.0m depth bending moment data point included, as the system of splines became unstable with the bending moment zigzagging wildly between adjacent data points. Instead the large positive bending moments at 1.0m depth on Piles A and B were removed and the splines fitted to the remaining data points. Removal of the large bending moment at 1m depth is reasonable because it is clearly related to construction effects and is not the result of genuine pile loading.

The splines fitted to the bending moment data points and the resulting net pressure distributions are shown in Figures 3.26 and 3.27. Comparison of the net pressure for Pile B (Figure 3.27b) with that acting on Pile C (Figure 3.23b) shows a similar distribution, although the magnitude of the pressures on Pile B are considerably smaller. The positive bending moment measured on Pile A at 2.6m depth means that removal of the large positive bending moment at 1m depth gives a spline fitted to the bending moment data that does not reverse across the pile. This leads to a pressure distribution that has large pressures acting over the bottom two metres of the pile, and a pressure distribution over the top three metres that is in the reverse sense compared with the pressure distributions for the other piles (i.e. the pressure over the top 1m of the pile acts downslope, not upslope). This pressure distribution for Pile A is likely to be incorrect, with the

modification of the bending spline to exclude the top item of bending data substantially altering the fitted pile bending behaviour.

On Piles A and B there are probably insufficient data points to interpret a complex bending behaviour fully, and the errors involved in curve fitting (such as having to assume a certain bending behaviour through the sections of the pile where gauges have failed) mean that interpretations of net pressure and pile behaviour have to be made with some caution. Further analysis of pile behaviour is given in Chapter 4.

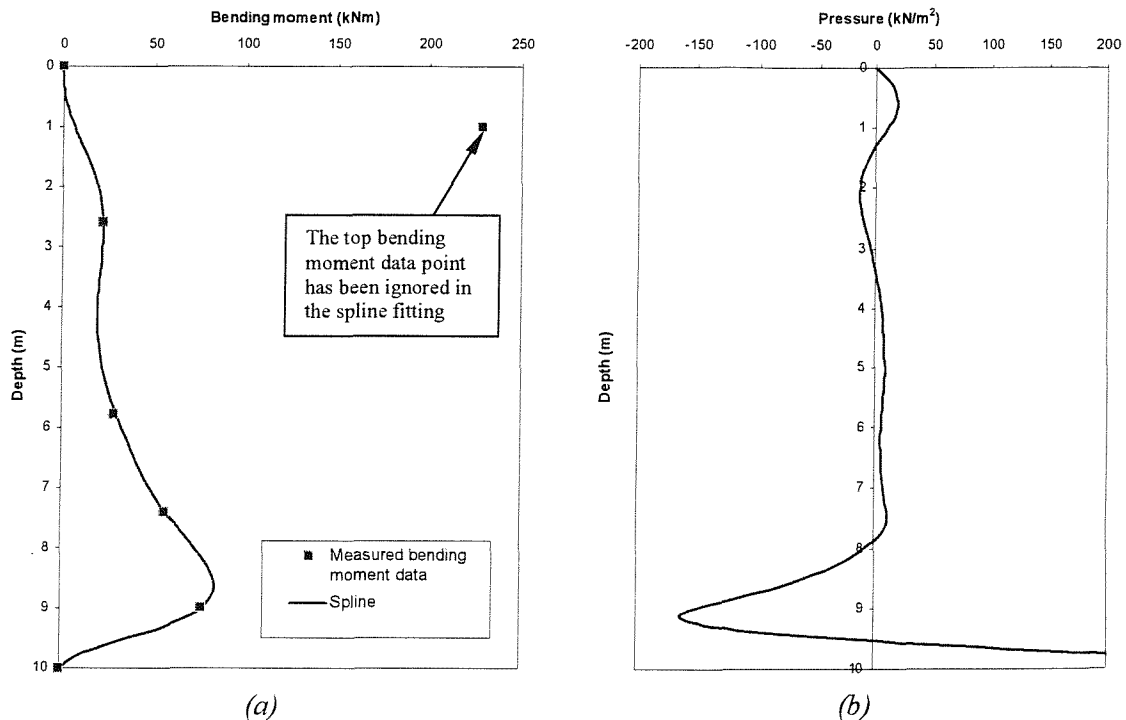


Figure 3.26. (a) The spline fitted to the bending moment data points (for Day 786) recorded by the rebar strain gauges in Pile A, and (b) the resulting pressure distribution (in which a positive pressure acts downslope).

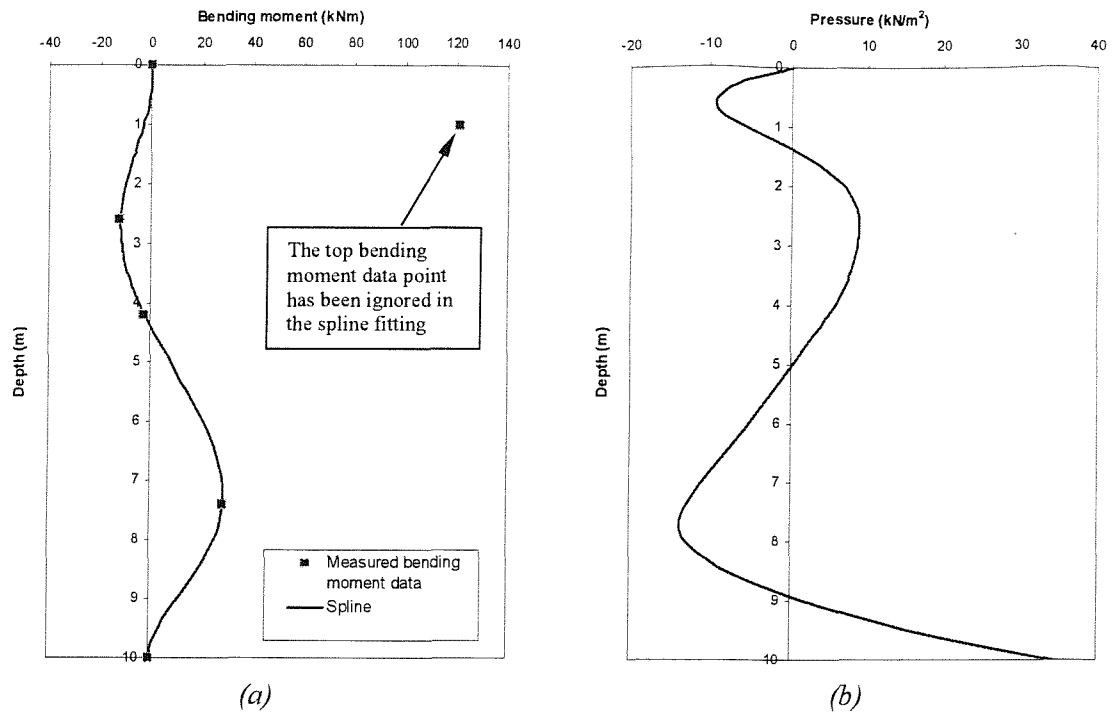


Figure 3.27. (a) The spline fitted to the bending moment data points (for Day 786) recorded by the rebar strain gauges in Pile B, and (b) the resulting pressure distribution (in which a positive pressure acts downslope).

3.6 Pile and slope displacement results

3.6.1 Datum readings

The datum inclinometer readings for the tubes installed in the piles were taken on day 3 after pile installation, and the datum readings for the tubes in the slope midway between the piles on day 5 (as these tubes were installed after the piles on days 3 and 4). Several further sets of inclinometer readings were taken over the 2 years after pile installation to measure the relative horizontal displacement of the piles and the slope. Unfortunately, a set of inclinometer displacements immediately prior to the regrading of the slope surface was not taken, and consequently the exact impact of regrading the slope on the pile and slope displacements was not measured.

The analysis of the inclinometer data here assumes that the base of the piles have not displaced laterally. The inclinometer used gives readings to a resolution of 0.1mm of lateral displacement for every 0.5m depth. The repeatability of the measurements was better than ± 1.0 mm.

3.6.2 Slope displacements

The results from the inclinometer tubes placed in the slope midway between the piles are shown in Figures 3.28 and 3.29. They show a clear movement of 7mm over the top 4m of the slope. The majority of this movement had occurred by day 42, with only a gradual increase in some locations after this date. There is a significant increase in the displacement profiles at about 4m depth, which corresponds to the depth of the critical failure surface found in design and the depth of the slope failures recorded on adjacent sections of the un-remediated embankment. While the largest displacements occur above 4m, there are still displacements of up to 3mm below this depth. Displacements in the longitudinal plane seem to indicate a small movement to the right, when looking downslope.

The slope inclinometer tubes protrude by about 70cm from the slope surface, and hence the two uppermost readings in Figures 3.28 and 3.29 are within the section of tube above the slope surface, and should be ignored.

3.6.3 Pile displacements

The displacement results from the inclinometer tubes in Piles A, B and C are shown in Figures 3.30, 3.31 and 3.32. Overall, the piles show a downslope displacement of about 8mm at the head, which corresponds to the ground displacements midway between the piles. The piles all show a broadly similar bending behaviour, with a downslope bending at the base of the pile, and a bend back upslope above this. As with the ground movements, the majority of the displacement occurs in the first 42 days. In the longitudinal plane, the pile heads have moved about 2mm to the right when looking downslope.

There is no significant difference in magnitude between the readings of the inclinometers installed in the slope between the piles and those installed directly in the piles, indicating that no significant soil movement is taking place between the piles, although the pile displacement profiles are much smoother than the ground displacements as a result of the pile bending stiffness.

3.6.4 Pile C displacements

The bending displacements of Pile C (Figure 3.32) show a fairly smooth downslope and then upslope bending behaviour - this seems to correspond with the bending moments measured and indicates that the pile has good fixity within both the intact Weald Clay at depth and within the moving mass (the top 4m), which restrains the pile from bending uniformly downslope.

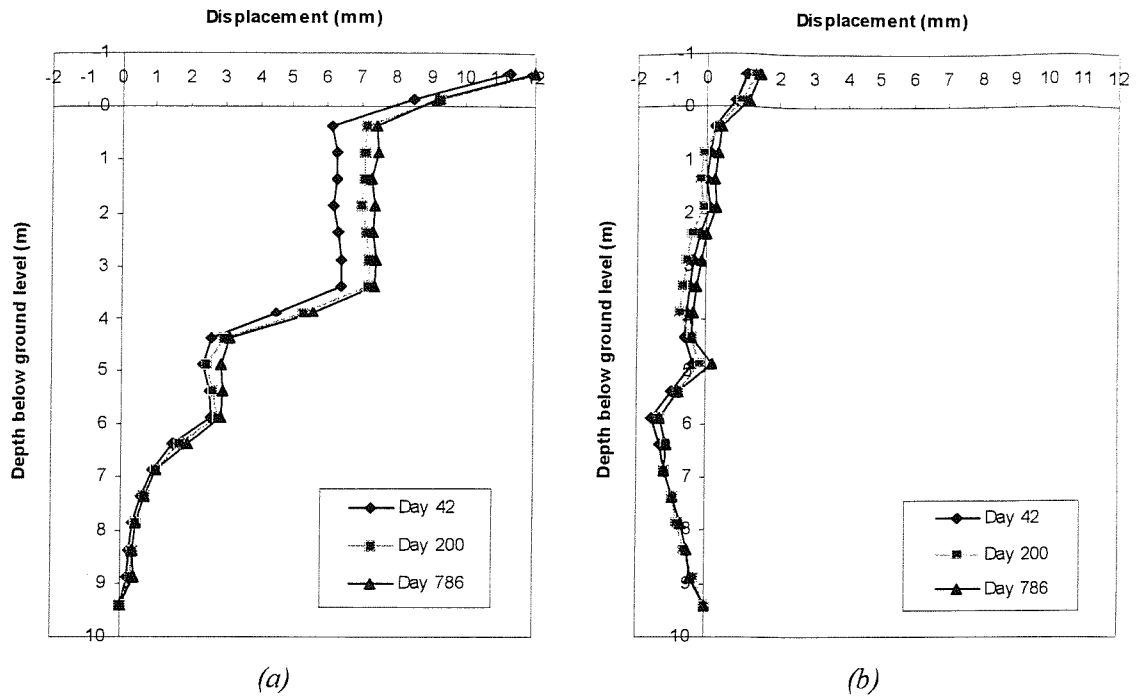


Figure 3.28. Displacements for the inclinometer tube in the slope between piles B and C: (a) upslope-downslope displacements, with downslope displacements denoted positive; and (b) left right displacements, with movements to the right (when looking downslope) denoted positive.

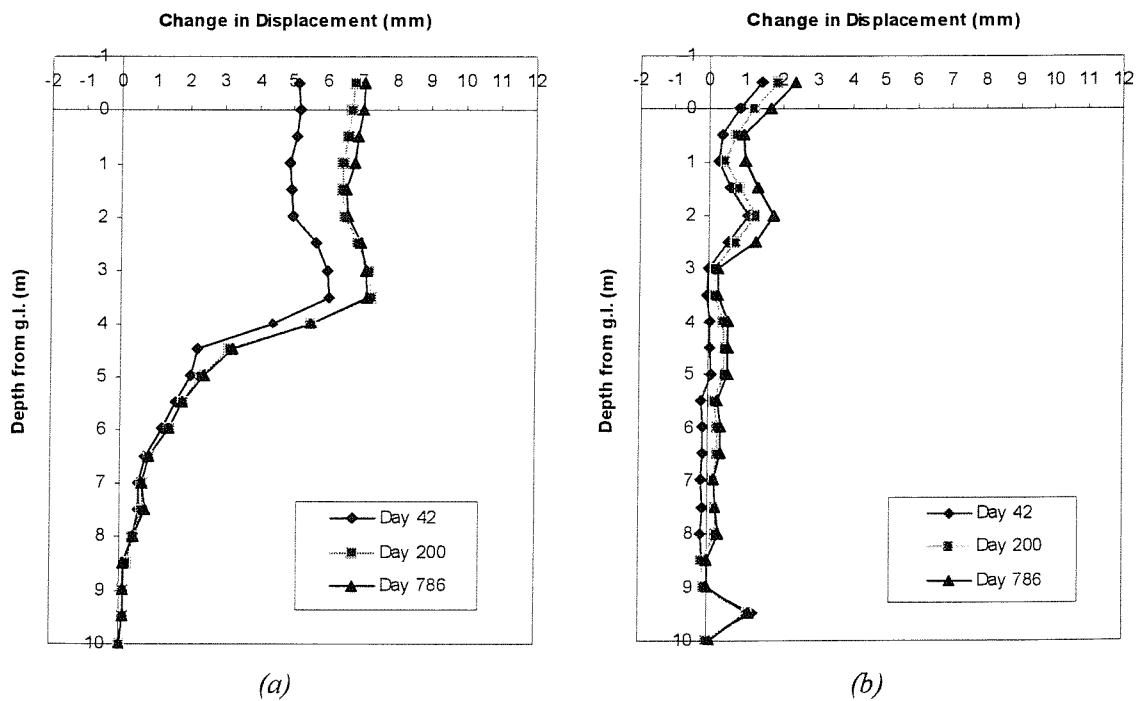


Figure 3.29. Displacements for the inclinometer tube in the slope between piles A and B: (a) upslope-downslope displacements, with downslope displacements denoted positive; and (b) left right displacements, with movements to the right (when looking downslope) denoted positive.

The splines fitted to the bending moment data points from both the concrete embedment and rebar strain gauges were integrated twice to obtain the pile displacement profile. The spline displacement profiles for Pile C are plotted against the inclinometer displacement profile in Figure 3.33. Integration of the splines only gives the bent pile shape, and any rotation of the pile has to be estimated (in this case by comparison with the inclinometer data). In Figure 3.33, a rotation of 0.00022 radians has been applied to the base of the rebar strain gauge profile to give a total displacement of 6.3mm at the ground surface (the same as the surface inclinometer displacement). There is a very close match between the inclinometer displacement profile and the spline profile from the rebar strain gauges. The embedment gauge spline profile is not such a good fit to the inclinometer data, with the small bending moments measured on the top 4m of Pile C by the embedment gauges causing little upslope curvature of the spline over this section of the pile. This seems to indicate that the rebar strain gauges are giving a more reliable measurement of bending moment in Pile C.

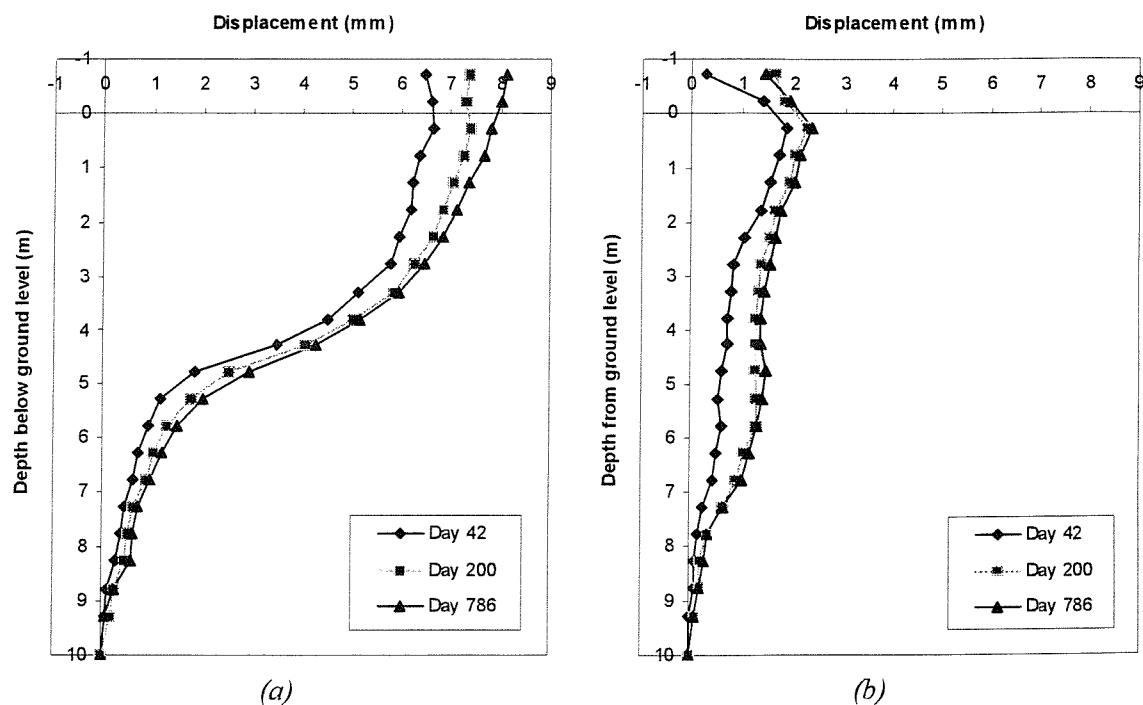


Figure 3.30. Inclinometer displacements for Pile A: (a) upslope-downslope displacements, with downslope displacements denoted positive; and (b) left right displacements, with movements to the right (when looking downslope) denoted positive.

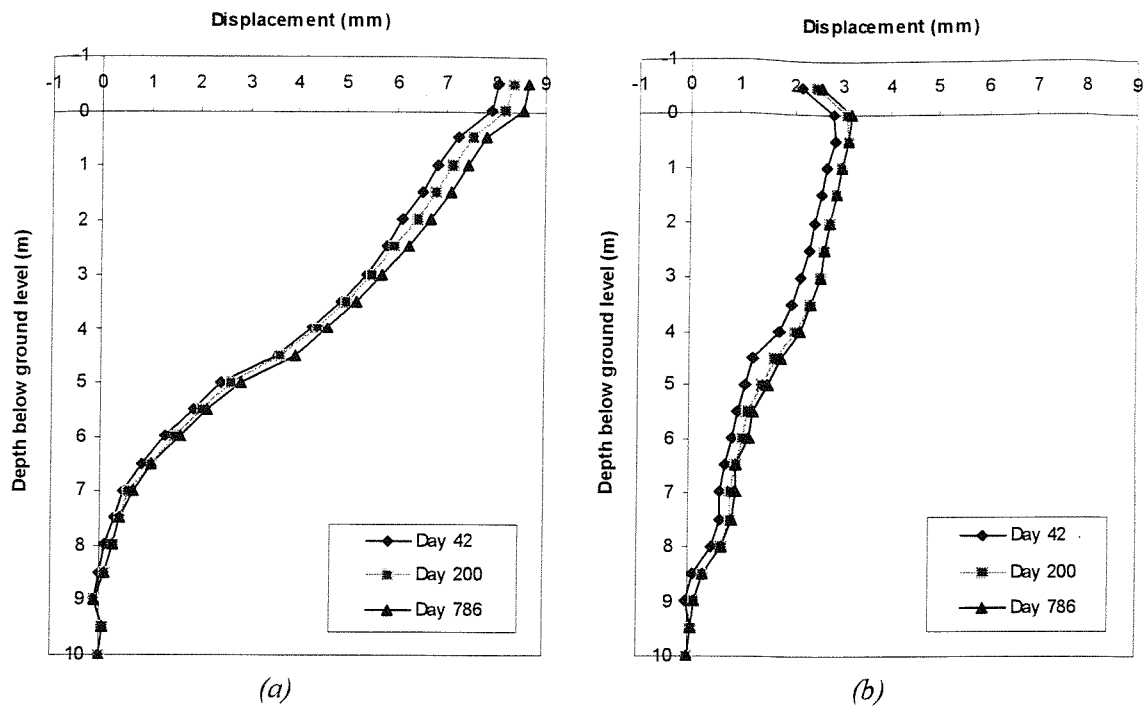


Figure 3.31. Inclinator displacements for Pile B: (a) upslope-downslope displacements, with downslope displacements denoted positive; and (b) left right displacements, with movements to the right (when looking downslope) denoted positive.

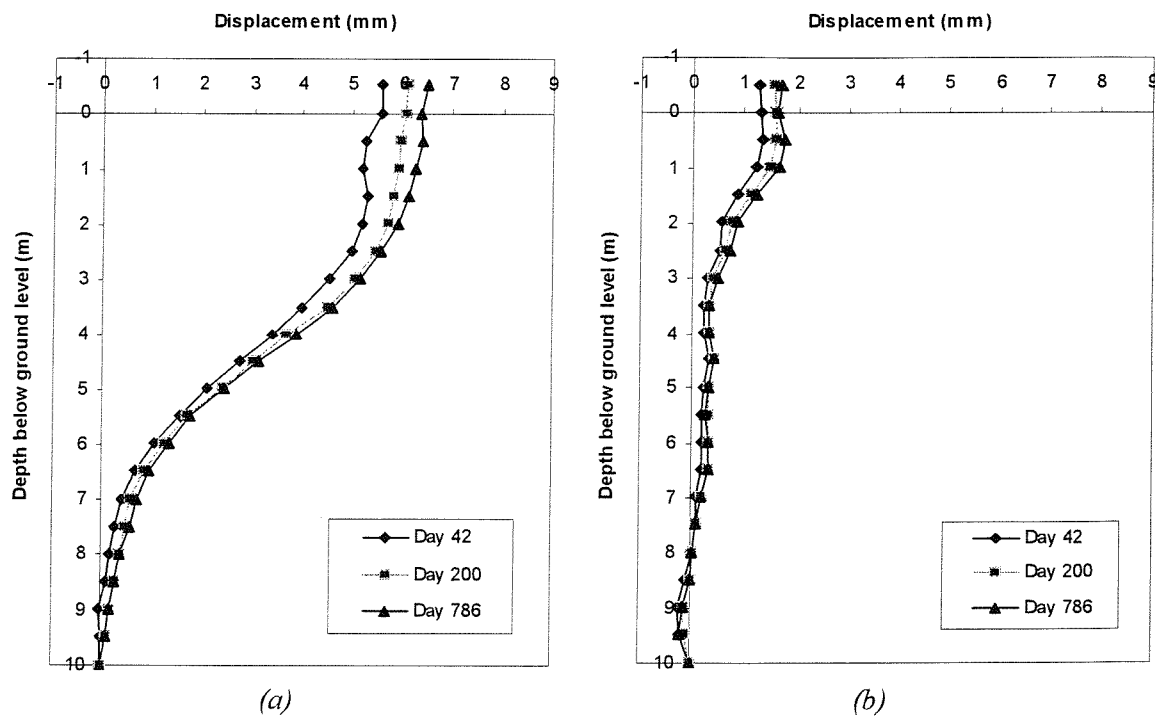


Figure 3.32. Inclinator displacements for Pile C: (a) upslope-downslope displacements, with downslope displacements denoted positive; and (b) left right displacements, with movements to the right (when looking downslope) denoted positive.

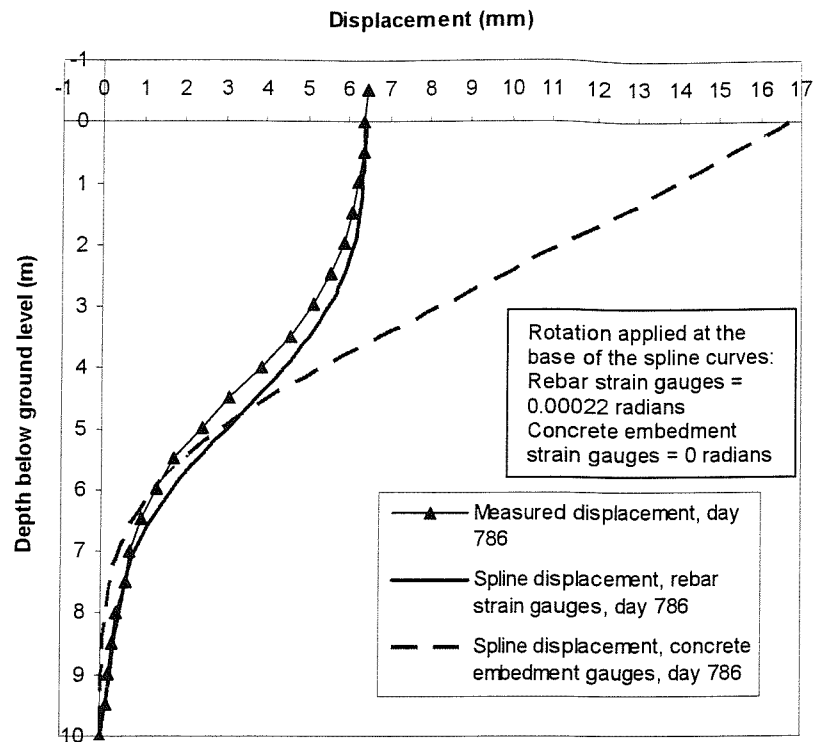


Figure 3.33. Pile C inclinometer displacements and spline displacements for day 786.

3.6.5 Pile A and B displacements

The displacement readings at the base (6.0-10.0m depth) of Piles A and B (Figures 3.30 and 3.31) are similar, although the base of Pile A seems to be slightly better restrained and this may account for the bigger bending moment measured at a depth of 7.4m on Pile A. The head of Pile B is not as well restrained in the failing soil mass, as indicated by the smaller upslope bend over the top four metres of the pile and a larger overall displacement at the very head of the pile. It is not clear why Pile B is not as well restrained by the moving upper 4m of the slope, but it may be related to the excavation of the rockfill material and its replacement with Type 1 fill directly on either side of Pile B. The Pile B deformation behaviour corresponds with the smaller bending moments measured on Pile B, compared to Pile C.

There is nothing in the displacement plots for either Pile A or Pile B that indicates why a bending moment of 150-250kNm was recorded at 1.0m depth – there appears to be no significant downslope bending of the pile measured at 1.0m depth by the inclinometer.

Figure 3.34 shows the displacement profile obtained by double integration of the spline fitted to the Pile A bending moment data. The spline profile does not correlate well with the inclinometer data, with no upslope curvature occurring over the top 4m of the spline profile. The poor correlation occurs as a result of the large positive and unexplained bending moment at 1m depth. This moment is removed from the Pile A bending moment spline (Figure 3.26a),

leaving a spline that remains positive along the entire pile length. This gives a displacement spline that bends downslope over its complete length.

The Pile B displacement spline (Figure 3.35) does not have very much curvature as a result of the small bending moments measured in Pile B. A significant rotation has to be applied to the base of the spline curve to get it to echo the total pile displacement measured by the inclinometer. However, its correlation with the inclinometer data is reasonable, particularly over the top 4m of the pile where little bending occurs in either profile.

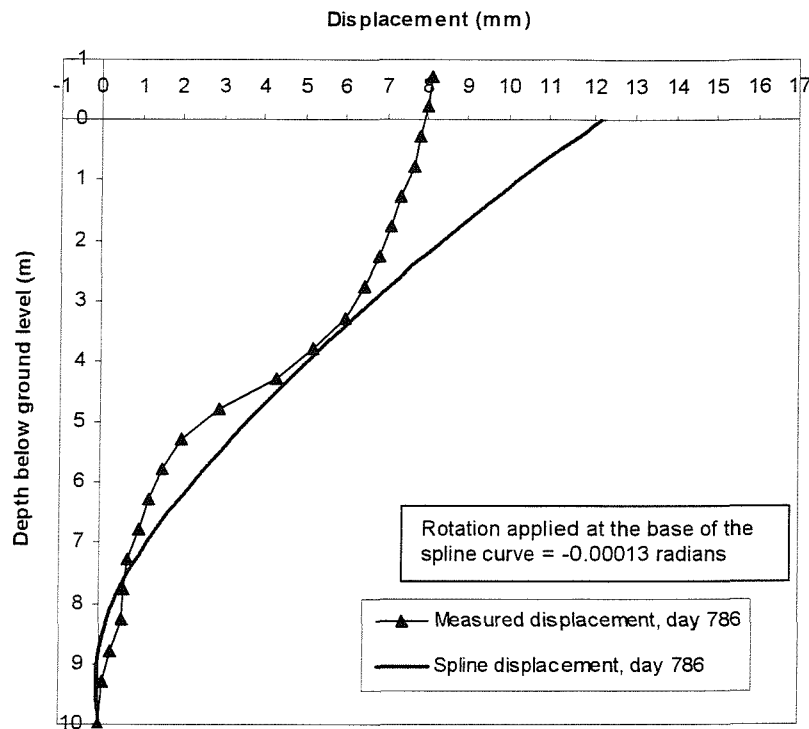


Figure 3.34. Pile A inclinometer displacements and spline displacements for day 786.

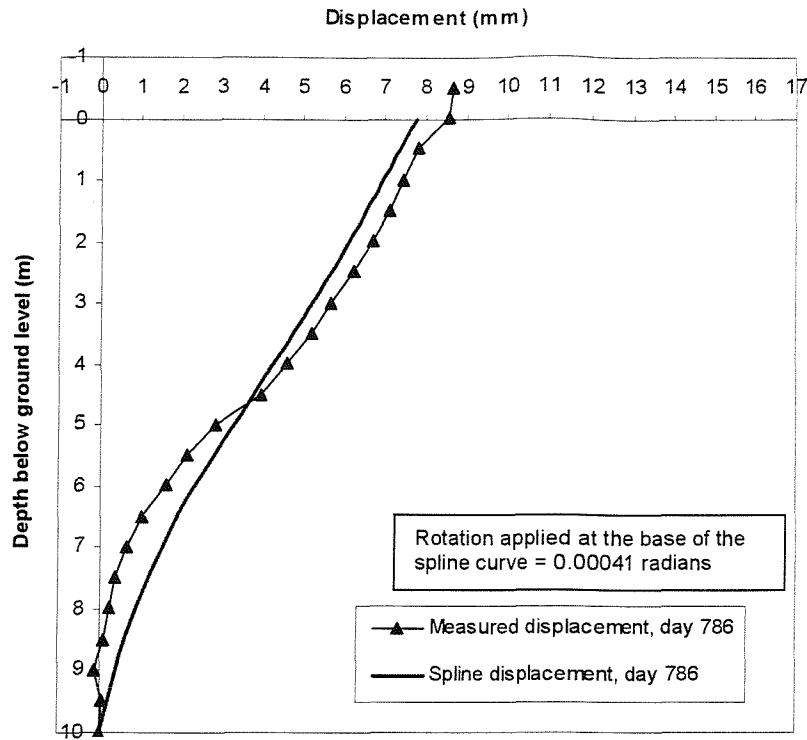


Figure 3.35. Pile B inclinometer displacements and spline displacements for day 786.

3.7 Axial strain results

3.7.1 Axial strain measurement

The effect of concrete shrinkage on the strain gauges can be explored by considering the axial force acting in each of the piles. Figures 3.36 and 3.37 show the axial strain in Piles A and B, and the axial strains in Pile C according to both the reinforcement bar and embedment gauges. The axial strain ε_a is calculated as in Equation 3.2:

$$\varepsilon_a = \frac{\varepsilon_1 - \varepsilon_2}{2} \quad \text{Equation 3.2.}$$

where ε_1 and ε_2 are the strains measured by each of a pair of strain gauges.

The axial strain acting on each of the gauges in a pair should be the same. The axial strains are studied here to see whether the pairs of strain gauges are being pulled into net compression by concrete shrinkage.

3.7.2 Axial strain in Piles A, B and C

The results from the steel reinforcement bar strain gauges in Piles A, B and C (Figures 3.36 and 3.37b) show that some pairs of gauges are in net axial tension, with others in net compression. If the results for Day 786 are considered, there is a tendency for the top pairs of gauges to be in net compression, and the lower pairs to be in net tension. This may reflect the expansion of the concrete within the lower section of the pile that remains submerged in groundwater, and the shrinkage of the section of pile that is allowed to dry.

The concrete embedment strain gauges (Figure 3.37a) show a steady increase in compressive strains with time. The compressive strains are fairly uniform down the pile, and show a steady increase in strain tailing off with time in a similar fashion to the concrete shrinkage observed in lab specimens (Bakoss *et al*, 1977; Murdock *et al*, 1991). The axial strains recorded by the concrete embedment gauges are not significantly larger than for the reinforcement bar strain gauges.

Interestingly, the net compressive strains in the embedment gauges increase fairly uniformly with time, in comparison with the more random change with time of the net axial strains indicated by the rebar gauges in Piles A, B and C. The concrete embedment gauges appear to be more consistently affected by concrete shrinkage.

3.7.3 Dummy pile

The strain gauges in the dummy pile appeared to indicate the development of a bending moment shortly after construction, with one strain gauge giving a large negative strain reading and the other a large positive strain reading. The strain readings with time are shown in Figure 3.38, and correspond to a bending moment of about 60 kNm. The development of the bending moment in the dummy pile mirrors the development of large moments at the heads of Pile A and B, and may be the result of plant moving on the embankment. Ignoring the large increase in bending moment on day 1, there is a general increase in compressive strain measured in each of the gauges, of about 10 microstrain. However, the development of the bending moment at such an early age unfortunately means that the dummy pile reliably tells us little about the effect of concrete shrinkage on the strain readings.

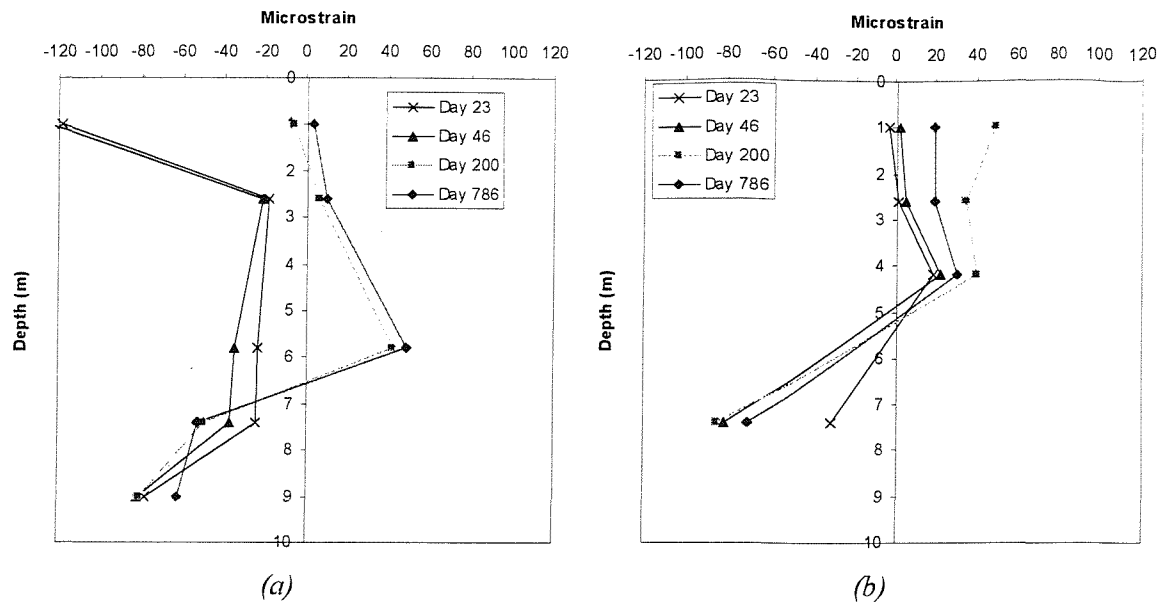


Figure 3.36. Axial strain, as measured in (a) Pile A; and (b) Pile B. Compressive strain is positive.

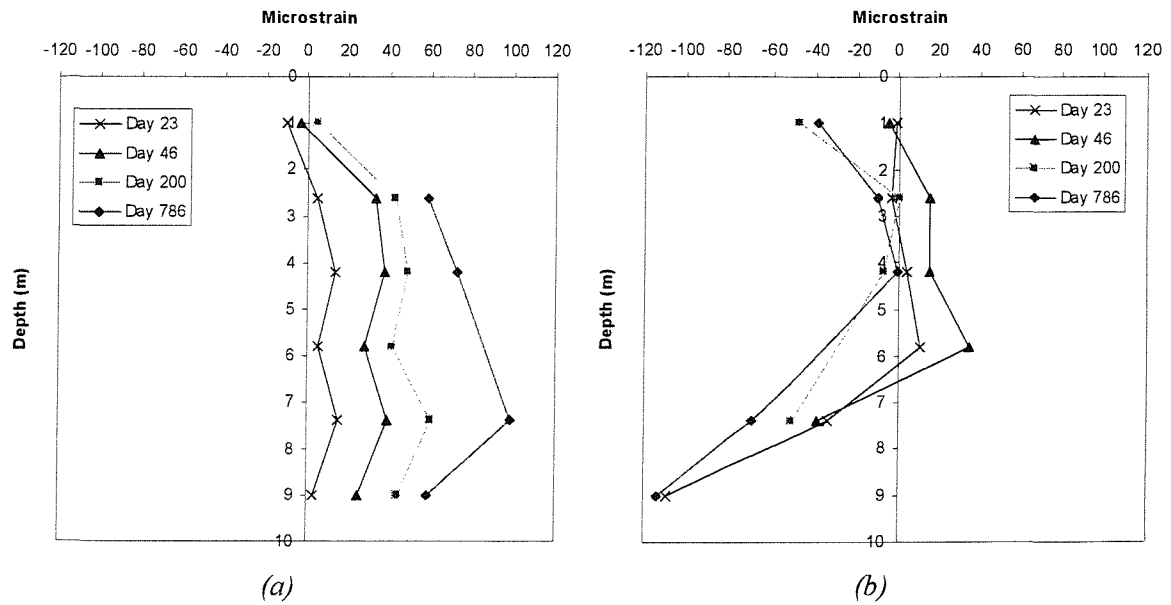


Figure 3.37. Axial strain measured in Pile C: (a) concrete embedment strain gauges; and (b) steel reinforcement strain gauges. Compressive strain is positive.

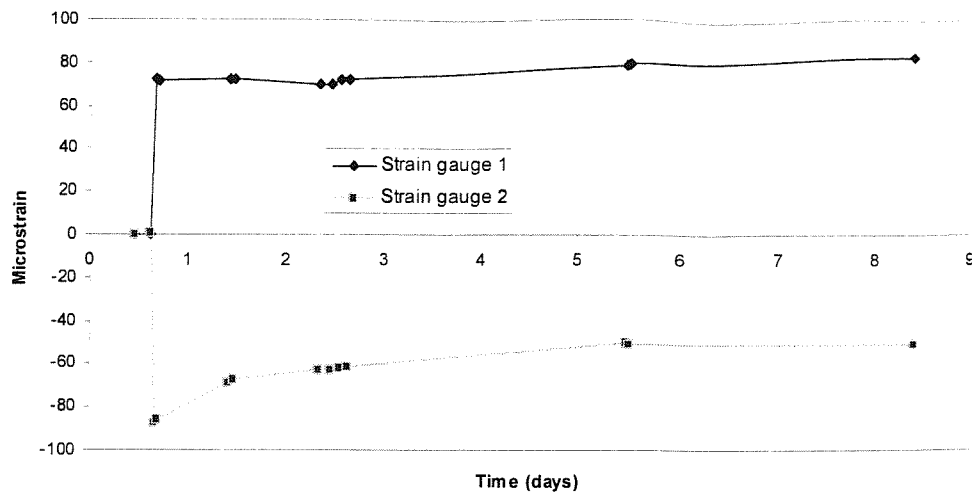


Figure 3.38. Axial strains as measured by the steel rebar gauges in the dummy pile.

3.8 Summary of overall pile behaviour

The measurement of bending moment and displacement within three piles installed to stabilise a rail embankment slope at Hildenborough, Kent, has been carried out over a period of 2 years.

The following key features were observed:

- i. Pile design was carried out by Mott MacDonald using simple limit equilibrium calculations. The restoring force required from the piles to give a factor of safety of 1.3 for the slope was obtained by putting a point force into limit equilibrium slope stability. A simple limit equilibrium calculation was then used to assess the design pile bending moment, with the restoring force applied to the back of the pile a triangular pressure distribution acting over the top 4m depth (the depth of pile to the critical failure surface).
- ii. Substantial bending moment was measured at some strain gauge locations after 4 days. Increases in bending moment also occurred on days 22 and 23, when the granular rockfill on the slope surface was regraded to form a two stage slope. At some gauge locations the bending moment further increased over the next 150 days or so; this may have been a result of settlement and dissipation of pore pressures within the embankment due to the mass of the granular rockfill placed upslope of the piles. After the first 200 days, there has been little observed change in either the bending moment or the displacement of the piles. A significant part of the pile bending and movement is attributable to the regrading of the slope.

- iii. Ignoring the very large bending moment at the head of Pile A (the cause of which has proved difficult to explain, but is possibly due to surface construction effects), the largest bending moment was 150 kNm in Pile C, substantially below the minimum ultimate pile bending capacity of 230 kNm.
- iv. Measurement of pile displacements using inclinometer tubes cast into the piles showed that they displaced by a maximum of 8mm at the pile head. Measurement of slope displacements using inclinometer tubes placed midway between the piles indicated a movement of the upper 4m of the slope by about 7mm. There is therefore no indication of any significant movement of soil between the piles (although the soil does displace further than the piles just above 4m depth, as the piles have a smoother profile of displacement due to their inherent bending stiffness). The increase in soil displacements at 4m depth correlates well with the critical failure surface used in design and the depth of failure surface observed in adjacent sections of the embankment prior to remediation.
- v. Splines fitted to the bending moment data points have been differentiated twice to obtain the distribution of net pressure acting on the pile, and integrated twice to obtain the bent pile shape. In fitting the bending splines to Piles A and B, the large positive bending moment measured by the top set of gauges in each of the piles was ignored, as it was thought that this bending moment was an artefact of surface construction processes, and not caused by genuine slope movement. The pressure distributions showed a net downslope pressure acting between 1 and 4-5m depth, below which the pile resists this pressure with a fixed earth point of rotation. The restraint to pile movement in the failing mass is represented by a net upslope pressure on the top 1m of the pile.
- vi. The pile inclinometer displacements and the displacement splines obtained from integration of the bending moment data indicate that the lower half of the piles bends downslope. The upper half of the pile clearly bends back upslope, as it is restrained from bending uniformly downslope by the moving mass (possibly aided by the granular rockfill on the slope surface). This behaviour is contrary to that assumed in pile design, which assumes the pile to bend uniformly downslope.
- vii. The insertion of concrete embedment type strain gauges into Pile C alongside the rebar type strain gauges has demonstrated the difficulties of measuring bending strain while concrete shrinkage is taking place. The concrete embedment and reinforcement bar gauges put into Pile C showed broad agreement in the shape of the bending moment distribution acting in the pile, but the magnitudes of the bending moments were

different. The displacement profile obtained by twice integrating the rebar gauge bending moment spline is a good fit to the Pile C inclinometer displacements, whereas the displacement spline from the embedment gauges is a poor fit to the inclinometer displacements. There is evidence from the axial strain plots that the embedment type gauges are more consistently affected by concrete shrinkage. Construction of a dummy pile within the embankment to assess concrete shrinkage strains did not yield any useful information, as the pile apparently developed a large bending moment in the same way as the head of Piles A and B.

- viii. The real pile behaviour is quite varied, and the measured bending moments are still significantly smaller than the pile bending capacity. The slope has probably not been sufficiently close to failure for significant slope (and therefore pile) movements to take place. It is possible that were the UK to receive a very wet winter (similar to 2000/2001 that caused the original embankment stability problem) the piles would carry more load.
- ix. The monitoring has reaffirmed
- the desirability of datalogging the output from strain gauges to allow changes in bending moment to be correlated to specific construction events.
 - the need for pore water pressure data to understand the influence that pore pressure changes within the embankment have on pile movement. This is perhaps the missing link in this study, and it is unfortunate that budgetary limitations meant that pore water pressure transducers could not be installed.
 - the difficulties inherent in making reliable strain measurements within reinforced concrete structures, particularly when loading is able to occur at the same time as concrete shrinkage processes are taking place.

Chapter 4. Back analysis of the Hildenborough embankment using elastic analysis

4.1 Introduction

The observed pile behaviour at Hildenborough has been back analysed using the program ALP (OASYS, 2001), which models the pile as a series of finite beam elements attached to elastic-plastic springs which model the pile-soil interaction behaviour. This type of analysis is described in Chapter 2 as ‘pseudo finite element analysis’.

The aim of the analyses using ALP was to replicate the measured pile behaviour, and in doing so to assess the unknown or uncertain properties required for such an analysis, including the soil stiffness and pore water pressures. This method of elastic analysis has also been assessed for its more general use in the design of piles for slope stabilisation.

4.2 Analyses using ALP

4.2.1 The program ALP

The program ALP calculates the pressures, horizontal movements, bending moments and shear forces induced in a pile when it is laterally loaded by either lateral forces, bending moments or imposed soil displacements. The pile is modelled as a series of elastic beam elements. ALP models a pile of finite length and is able to take into account the pile rotation and its global stability. However, the vertical pile-soil frictional interface is not modelled (the soil is assumed to be attached to the pile). The soil is represented as a series of elastic-plastic springs or fully specified load-deflection (p-y) curves. The solution scheme creates two stiffness matrices that relate nodal forces to displacements, one for the pile and another for the soil. In each iteration

that the program makes, the soil pressures are calculated, and if these infringe the plastic yield point the nodal forces are re-distributed onto adjacent pile beam elements. The program assumes that the soil surface and soil layers are horizontal, and it is unable to model the equilibrium of the slope.

4.2.2 Inputs to the analyses

Required as input to the program are the pile geometry, the pile stiffness, the soil geometry, the soil yield criterion and the soil stiffness. Soil displacements are applied to the model as the pile loading.

Pile geometry and stiffness

The pile was modelled as a series of 40 elastic beam elements, each 0.25m long. The pile stiffness properties were applied to each of the nodes joining the beam elements together. The pile stiffness (EI) was calculated as 170880 kNm². This stiffness assumes that the concrete in the pile is uncracked.

An initial pile model using 20 elements was used, but it was found that increasing the number of elements to 40 allowed the boundaries between the soil layers to be slightly more accurately defined. Running an analysis with 20 and an analysis with 40 elements also acted as a check to ensure that sufficient elements were used to get an accurate bending response.

Soil and groundwater geometry

The soil geometry used in the analysis of the piles was based on Figure 3.2 and is shown in Table 4.1. In the absence of more detailed pore water pressure data, the pore pressures are assumed to be hydrostatic from 3.5m depth based on the ponding of water that occurred at the toe of the embankment slope in April 2001, and zero above 3.5m depth.

Soil yield criterion

The passive yield load (F_p) for the different soil layers was calculated as follows (OASYS, 2001):

$$F_p = [K_q \sigma'_v + c' K_c] \quad \text{Equation 4.1}$$

where σ'_v = the vertical effective soil stress, c' = the effective soil cohesion, K_q = the passive resistance coefficient for the frictional component of the soil strength, and K_c = the passive resistance coefficient for the effective cohesion component of the soil.

The vertical effective stress was calculated using a unit weight for all soil materials of $\gamma = 19\text{kN/m}^3$. The value of K_q was calculated using the limiting pressure for laterally loaded piles as defined by Broms (1964b), so that $K_q = 3 K_p$, where K_p is the coefficient of passive earth pressure $(1+\sin\phi'/1-\sin\phi')$. The effective cohesion was assumed to give a component of limiting lateral pressure where $K_c = 9$, based on the limiting lateral pressure solution for a smooth pile in an undrained soil given by Randolph and Houlsby (1984). The values of soil strength, and K_q and K_c used for each of the soil layers is given in Table 4.1.

The limiting lateral pile-soil pressure (or yield load) would be expected to be lower on the downslope side of the pile where the ground profile drops away from the top of the pile. However, no changes were made to the limiting lateral pile-soil pressures used in the analyses to account for the sloping ground profile. The reasoning for this was that most of the upslope resistance to pile movement takes place below 3.5m depth (the height of the rockfill berm), and except for the very top of the pile, the lateral limiting pressure is significantly larger than the pressures acting on the pile and the elastic-plastic springs representing the soil remain in the elastic zone. The pile spacing of 4 pile diameters centre-to-centre is assumed to be sufficiently large that the pile interaction also has no affect on the limiting lateral pressure.

Soil stiffness

As mentioned in Chapter 3, the stiffness data from the triaxial tests carried out as part of the geotechnical investigation could not be obtained. In order to carry out an elastic analysis of the pile, the stiffness parameters had to be estimated from existing data for the Weald Clay in the literature. Comparing the results of the ALP analysis using this initial data against the real pile behaviour allowed an assessment of the initial soil stiffness data. This was then modified to try to reproduce the real pile behaviour more closely. The initial stiffness data used for the soil components making up the embankment are shown in Table 4.2.

Soil layer	Depth range	Frictional soil strength	Effective cohesion	Passive resistance coefficient, frictional component of the soil	Passive resistance coefficient, effective cohesion component of the soil
	(m)	ϕ' (degrees)	c' (kPa)	K_q	K_c
Rockfill	0 – 2.5	35	0	11.07	9.00
Embankment fill	2.5 – 3.5	25	1	7.38	9.00
Weathered Weald Clay	3.5 – 4.5	25	1	7.38	9.00
Intact Weald Clay	4.5 - 10	30	5	9.00	9.00

Table 4.1. Parameters used to calculate the limiting lateral soil pressure.

Model section	Elastic modulus at top surface of layer E' (MPa)	Increase in elastic modulus with depth mE' (MPa/m)
Weald Clay embankment fill	8	0
Weathered Weald Clay	8	1.0
Weald Clay	9	3.2
Rockfill	20	0

Table 4.2. Stiffness data used in the initial analysis of the Hildenborough piles.

The underlying intact Weald Clay stiffness is based on the stiffness properties of Weald Clay presented in both Loveridge (2001) and O'Brien *et al* (1992). Loveridge (2001) gives design stiffnesses for the Weald Clay determined from laboratory tests and used in finite element analyses for the design of the cut and cover tunnel for the CTRL at Ashford, Kent. At the site in Ashford, the Weald Clay is found at approximately 19 m depth, and the soil stiffness profile used in the analysis of the retaining structure was $E' = 18.2$ MPa and $mE' = 3.2$ MPa/m. O'Brien *et al* (1992) presents stiffness data derived from self boring pressuremeter and laboratory tests on the Weald Clay. The data from the pressuremeter gives a shear modulus $G = 75$ MPa at 10m depth, based on a cavity strain of 0.2% (O'Brien *et al*, 1992). This can be converted into a gradient of Young's modulus with depth, $mE' = 19.5$ MPa/m, from 0 MPa at the ground surface (using Poisson's ratio $\nu = 0.3$). Undrained triaxial tests on the Weald Clay (O'Brien *et al*, 1992) showed a large variation of undrained stiffness with axial strain, with the stiffness on average 10 times smaller at 1.0% strain than at 0.01% strain. It is difficult to estimate the strain likely to occur in the soil adjacent to the pile, but assuming that a zone of soil half a pile diameter (300mm) is affected, displacement of the pile of about 6mm into this zone (which is similar to the pile displacement measured) gives a strain of about 2%. Therefore initially a lower Weald Clay stiffness was used, with a profile similar to that reported by Loveridge (2001) adopted in the first analysis (Table 4.2). The weathered Weald Clay was given a slightly lower value of increase in stiffness, $mE' = 1.0$ MPa/m.

Considerable investigation of old tipped London Clay embankments has been carried out (Mott MacDonald, 1999a and 1999b), and it is expected that the tipped Weald Clay fill will be similar in nature. The Weald Clay embankment fill is likely to comprise intact clods of Weald Clay (randomly orientated lumps of clay 50-300mm in size as dug and tipped to form the embankment) surrounded by a softened matrix of Weald Clay. The matrix is likely to be material that has swollen to fill the original voids space, and the lumps themselves will have experienced some swelling that will decrease with distance from the initial outer surface. The matrix material is likely to be weaker, more permeable and less stiff than the intact lumps, and this will affect the bulk properties of the material to some extent (Mott MacDonald, 1999a).

Advanced laboratory testing of the London Clay fills recovered from a number of LUL embankments has shown the stiffness to be similar to that of intact London Clay (Mott MacDonald, 1999a). However, the fill material tends to have a greater range of stiffnesses (at the same axial strain) than the intact material, and this probably reflects the varying proportions of clod and matrix material within individual samples. In the absence of any data on Weald Clay embankment fill, it can probably be assumed that the stiffness properties of the Weald Clay fill and intact Weald Clay are also broadly similar. Initially, the embankment fill was assumed to be slightly less stiff than its intact parent material, and given a uniform stiffness $E = 8.0 \text{ MPa}$.

While there is no data on the stiffness of the rockfill material used at Hildenborough, there is data in the literature on the stiffness of other granular materials, including rockfill materials used in embankment dam construction. Values of normalised shear modulus $G/p' = 200$ and Poisson's ratio $\nu' = 0.25$ may be taken as typical of medium-dense sand at a shear strain increment of 0.2% (based on pressuremeter tests, Bellotti *et al*, 1989). Using the average effective stress p' at the centre of the rockfill adjacent to the pile, the Young's modulus for the layer is calculated as 12 MPa. Oldecop and Alonso (2001) present the results of a one-dimensional compression test on a dry slate rockfill of particle size 10-40mm, and show that in virgin compression the rockfill has Young's modulus 30 MPa at low vertical effective stress (30kPa). In unloading and reloading, the same rockfill has a stiffness of about 200 MPa. One dimensional compression data for a number of rockfill materials are given by Marsal (1973). The materials include sands and gravels, conglomerates, diorites, slates, basalts and granite. The Young's modulus of the materials in virgin compression and at an axial stress of 200kPa range from 10-50 MPa, although the unload-reload lines give stiffnesses much larger than this. The rockfill stiffness is quite dependant on the initial void ratio of the rockfill, with the tests starting at a low initial void ratio (tending to relate to a small average particle size) giving a higher stiffness than those with a high initial void ratio.

In the absence of any stiffness data for the actual rockfill material placed onto the slope surface, a value of 20MPa has been used, reflecting the fact that the granular rockfill is likely to be stiffer than the Weald Clay embankment fill. However, from the evidence above, it is possible that the rockfill stiffness could be higher than this initial value.

Soil displacements

The measured slope displacements are imposed on the pile head. The soil displacements input into the model are shown in Figure 4.1, together with the measured soil displacements. Two profiles were used, the first displacement profile (D1) has a soil displacement of 7.5mm acting over the top 4m of the pile. The second profile (D2) has a displacement of 7.5mm over the top 4m and a linear distribution of displacement directly below this, starting at 4mm at 4m depth

and returning to 0mm at 7m depth, to reflect more closely the real soil displacements measured at this depth in the embankment.

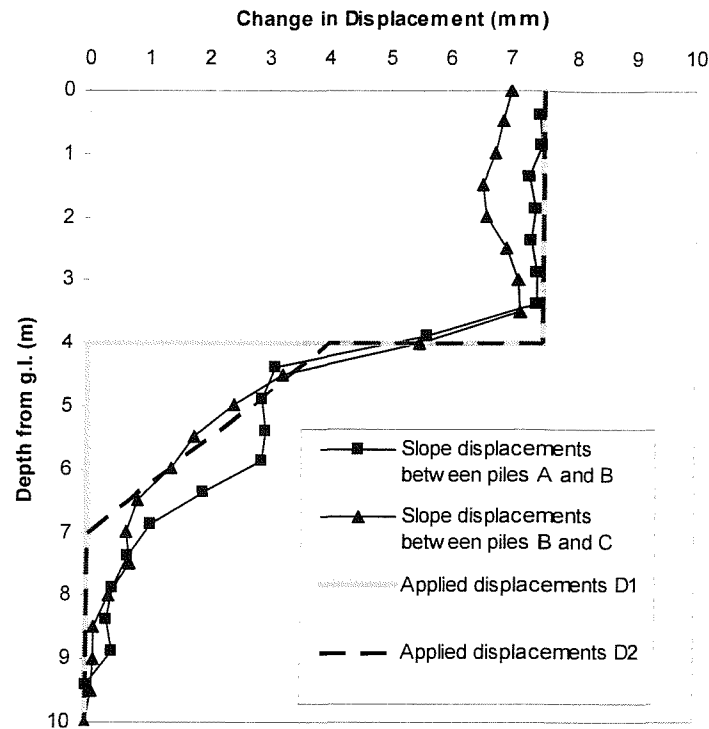


Figure 4.1. Profiles of applied soil displacement used in the model plotted with the measured slope displacements from the inclinometer tubes midway between Piles A and B and Piles B and C.

4.2.3 Analysis results

A series of analyses was run, as listed in Table 4.3. The results of the individual analyses are presented and discussed sequentially in the following section. All analysis results were aimed at replicating the most recent pile bending behaviour (as measured on day 786).

Analysis	Number of pile elements	Displacement profile (see Figure 4.1)	Soil stiffness
A1	20	D1	As Table 4.2
A2	40	D1	As Table 4.2
A3	40	D1	As Table 4.4
A4	40	D2	As Table 4.4
A5	40	D1	As table 4.4, but the the Rockfill stiffness increased to $E' = 150$ MPa
A6	40	D2	As table 4.4, but the the Rockfill stiffness increased to $E' = 150$ MPa

Table 4.3. Summary of analysis inputs

Analyses A1 and A2

The analyses A1 and A2 were both run with displacement profile D1 and the stiffnesses given in Table 4.2. Twenty elements were used to model the pile in A1 and forty in A2, to assess the number required to give numerically accurate bending behaviour. Broadly the bending and displacement results of the two analyses are the same, indicating that 20 elements are sufficient. However, the model with 40 elements has been used in the remainder of the analyses.

The ALP bending moment and displacement results are shown in Figure 4.2, together with the real pile bending and displacement data (for the bending data, the splines fitted to the data points in Chapter 3 are shown). The calculated pile behaviour is similar in overall bending shape to that measured in the piles, with the lower sections of the piles bending downslope, and the upper sections bending back upslope. The ALP bending moments fall approximately midway between the bending moments measured in Piles B and C. The calculated pile displacement behaviour is a reasonable match to the displacements measured in Pile B, although the base of the pile in the ALP analysis appears to be less well constrained by the Weald Clay than in the measured data. The ALP analysis fails to replicate the displacement behaviour of Pile C, where the pile bends upslope at 2.5m depth so that the top section of the pile is almost vertical. Subsequently, the positive bending moment calculated at 2.5m depth is significantly smaller than the bending moment measured by the rebar strain gauges on Pile C.

Figure 4.3 shows the pressure distribution calculated by ALP, compared with the pressure distributions obtained from splines fitted to the measured bending moment data. The ALP pressure distribution is in overall shape a reasonable match to the measured pressure distributions, although the calculated pile pressures are not quite as large as those measured on Pile C. The spike that occurs in the ALP pressure distribution at 2.5m depth is the result of the contrast in stiffness between the rockfill and embankment fill.

Analysis A3 and A4

The analysis A3 was run with displacement profile D1 and the soil stiffnesses given in Table 4.4, which are twice as stiff as those used in analyses A1 and A2. The increased stiffnesses are not unreasonable, as the initial stiffnesses adopted for A1 and A2 were at the low end of the range of Weald Clay data found in the literature. The results of the analysis are shown in Figures 4.4 and 4.5. The aim of the analysis was better to constrain the pile both within the Weald Clay toward the base of the pile, and within the rockfill material close to the top of the pile. The larger soil stiffnesses significantly increase the calculated bending moments in the pile, and close to the base of the pile the bending moments are similar in magnitude to those measured in Pile C by the rebar gauges, although the ALP bending moments are about half the magnitude of the measured Pile C rebar gauge bending moment at 2.5m depth. The ALP

bending moments also switch across the pile about 1m higher than the Pile B and Pile C rebar gauge bending moment distributions.

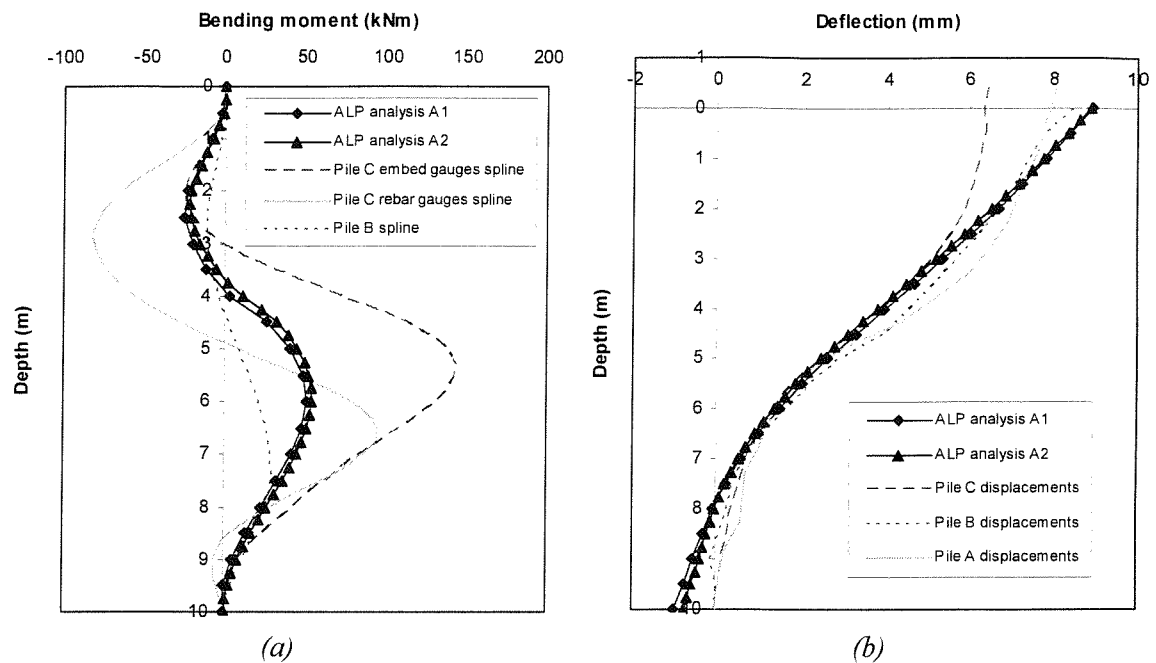


Figure 4.2. (a) Bending moment and (b) displacement results from analyses A1 and A2, plotted with measured data. Downslope displacements are positive.

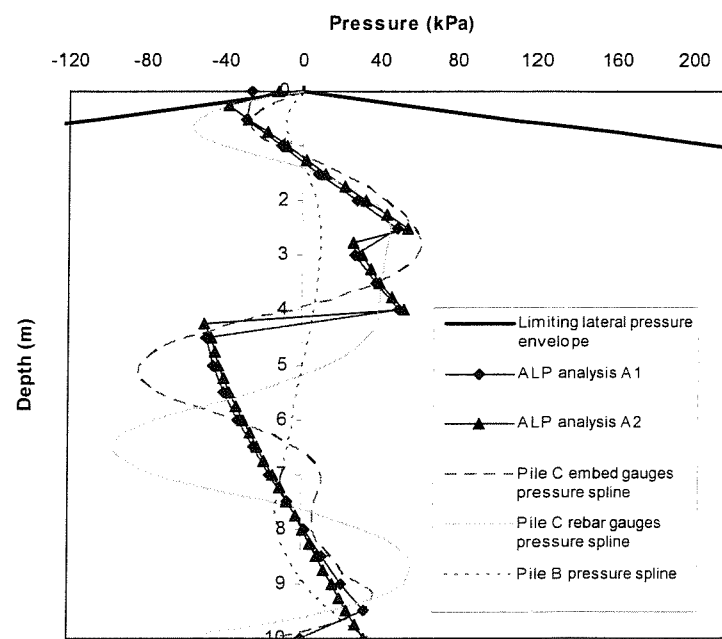


Figure 4.3. Net pile pressure results for analyses A1 and A2, plotted with the pressure results of splines fitted to the real bending moment data. Pressures acting downslope are positive.

Model section	Elastic modulus at top surface of layer E' (MPa)	Increase in elastic modulus with depth mE' (MPa/m)
Weald Clay embankment fill	16	0
Weathered Weald Clay	16	2.0
Weald Clay	18	6.4
Rockfill	40	0

Table 4.4. Modified stiffness data used in the elastic analysis (A3 and A4) of the Hildenborough piles.

Analysis A4 is run with the same increased values of soil stiffness, but with the soil displacement profile D2. The increased displacement over the section of pile between 4 and 7m acts to reduce the pressures on the pile over this depth (Figure 4.5), and this in turn slightly reduces the bending moments calculated in the lower section of the pile. The depth at which the crossover of the bending moment from negative to positive occurs is now closer to the depth measured in Pile B and in Pile C by the rebar strain gauges. However, the ALP pile displacement profile is also changed, with larger displacements occurring further down the pile.

Analysis A5 and A6

Analyses A5 and A6 are both run with the soil stiffnesses in Table 4.4 but with an increased Rockfill stiffness of 150 MPa (Figures 4.6 and 4.7). Analysis A5 is run with displacement profile D1 and A6 run with D2. The result of the much larger rockfill stiffness is to increase the magnitude of the negative bending moment at 2.5m depth to a similar magnitude to the bending moments measured in Pile C by the rebar strain gauges. The additional displacements in profile D2 reduce the bending moments slightly in A6 when compared to A5, particularly over the lower section of the pile.

In Chapter 3, the integration of the Pile C bending moment spline fitted to the rebar strain gauges gave a bent pile shape that closely matched the Pile C inclinometer displacements. It was generally felt that the rebar strain gauges were possibly giving the more reliable measurement of bending moment in Pile C, and hence it is of interest to see whether the ALP analyses are able to closely back analyse the bending moments given by the Pile C rebar gauges. Analysis A6 is a reasonably good match to the measured Pile C rebar strain gauge bending moments and pressures (Figures 4.6 and 4.7). However, the applied rockfill stiffness of 150MPa is large, and it is uncertain whether the rockfill really has such a large stiffness so close to the slope surface. It is possible that this high value of stiffness may in part be modelling the impacts on the pile from surface construction/compaction effects (that cannot be properly incorporated within the simple ALP analysis).

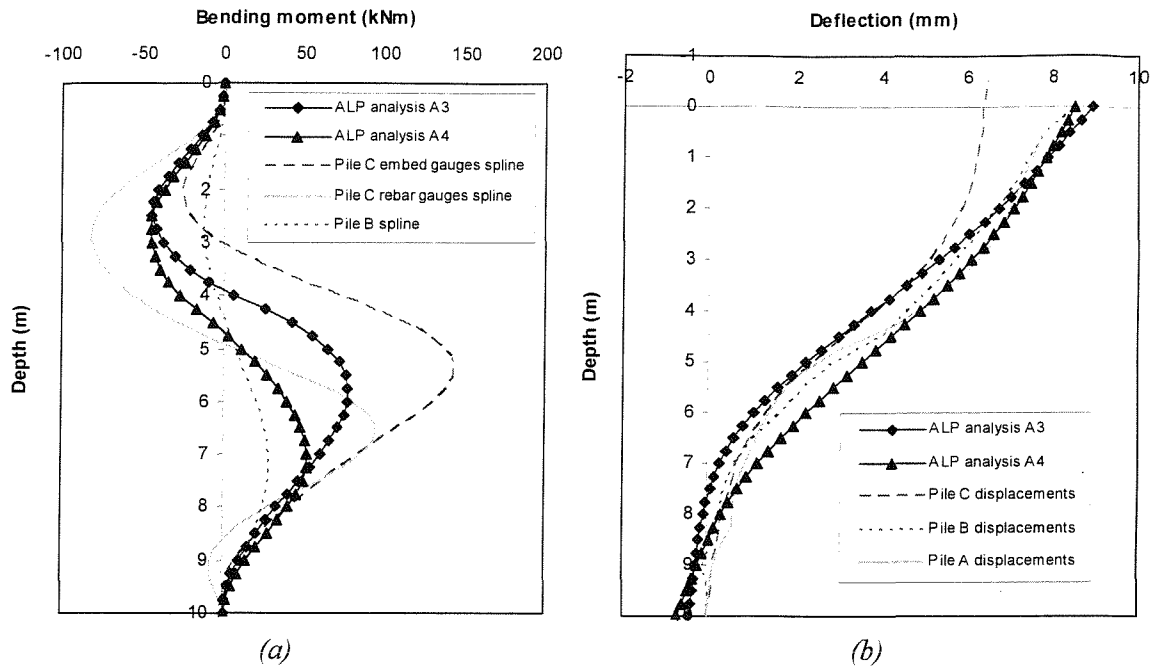


Figure 4.4. (a) Bending moment and (b) displacement results from analyses A3 and A4, plotted with measured data. Downslope displacements are positive.

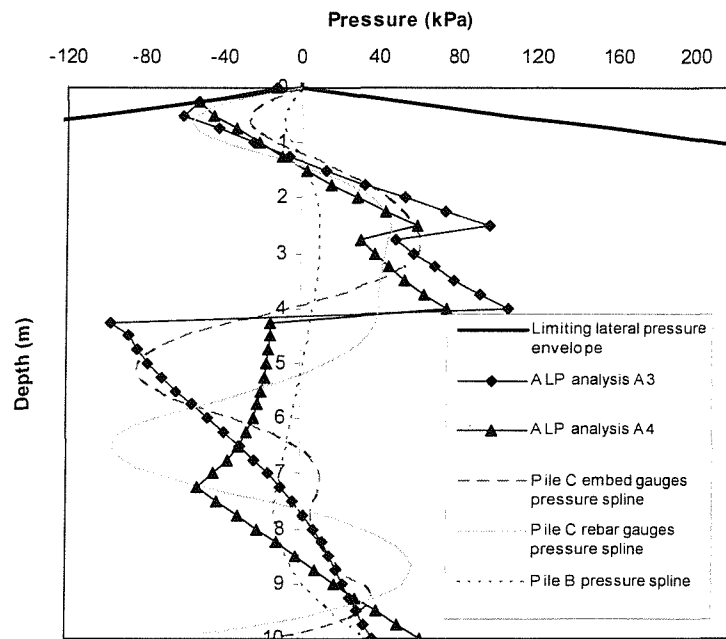


Figure 4.5. Net pile pressure results for analyses A3 and A4, plotted with the pressure results of splines fitted to the real bending moment data. Pressures acting downslope are positive.

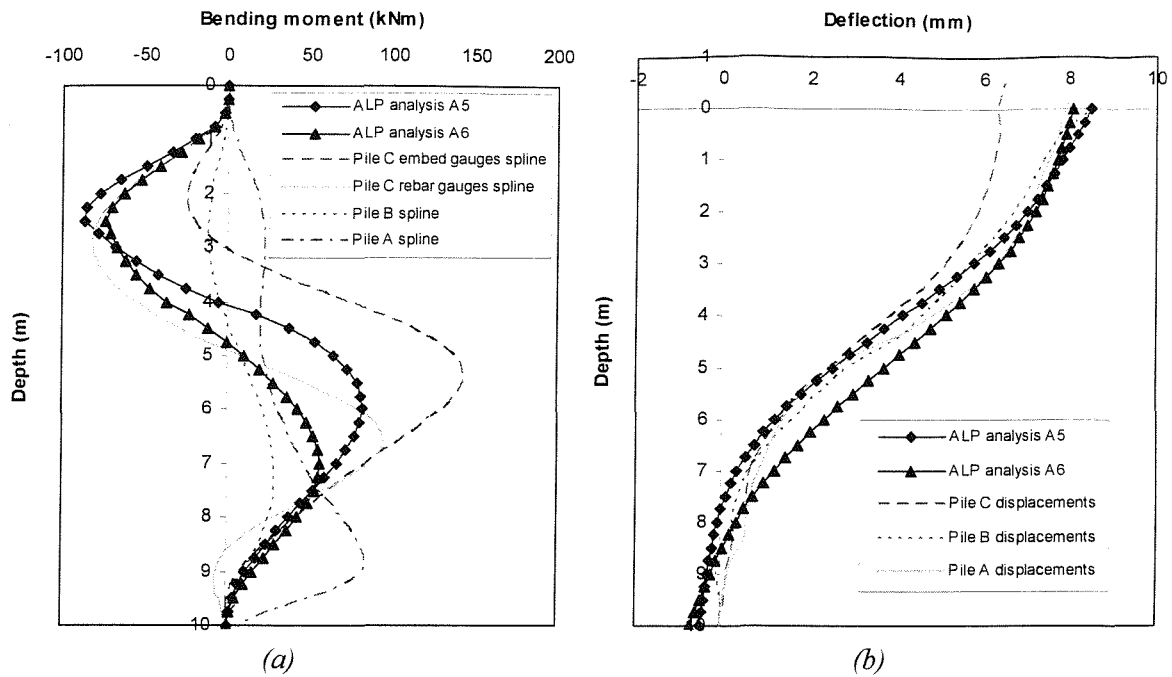


Figure 4.6. (a) Bending moment and (b) displacement results from analyses A5 and A6, plotted with measured data. Downslope displacements are positive.

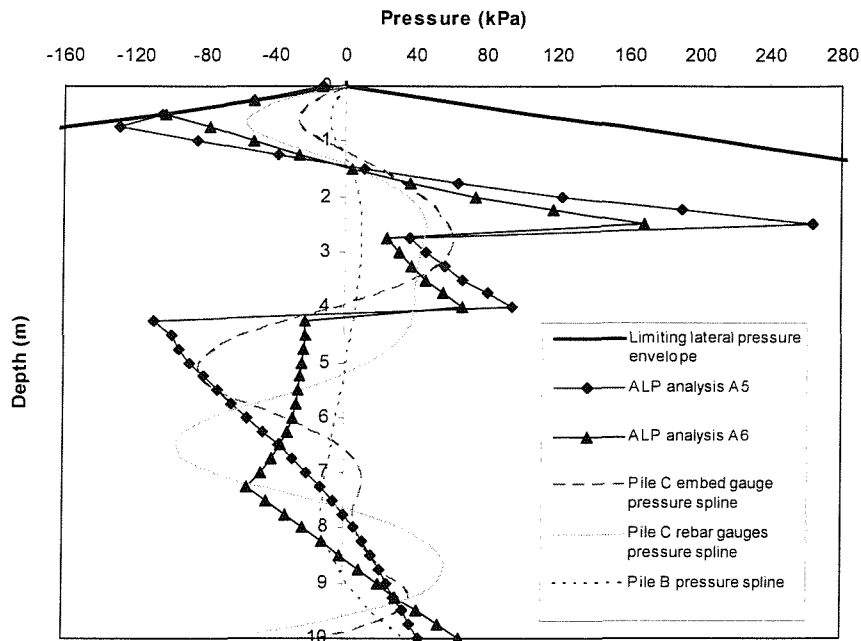


Figure 4.7. Net pile pressure results for analyses A5 and A6, plotted with the pressure results of splines fitted to the real bending moment data. Pressures acting downslope are positive.

While the overall bending behaviour of Pile C is better replicated by analyses A5 and A6, the much more curved inclinometer displacement profiles are not matched exactly (Figure 4.6b). The ALP pressure distributions now seem to be of about the same magnitude as those obtained from the system of splines fitted to the real bending moment data (Figure 4.7).

Slope loading

The force from the failing or displacing mass that is laterally loading the pile may be determined from the net pile pressures calculated using ALP. The net pile pressure distribution was exported into Excel, divided into strips 0.25m deep, and the force applied to the pile as the result of the pressure acting from each 0.25m strip was calculated. These forces were then summed over the 4m length of pile above the failure surface and the 6m length below the failure surface, to create a net horizontal downslope force acting above 4m depth and a net horizontal upslope force below 4m depth. As the pile must be in horizontal force equilibrium, the upslope and downslope forces are equal, and equivalent to the force that the failing soil mass applies to the pile.

The upslope and downslope horizontal forces calculated from analyses A2-A6 are shown in Table 4.5. The values of force should be compared to the design pile loading of 144kN per pile from the limit equilibrium slope stability analysis (Chapter 3, Section 3.3). As expected, the upslope and downslope forces are in equilibrium. The forces are all smaller than the design pile loading – but this would be expected as the pile stabilised slope was designed with a factor of safety of 1.3. Notably, the ALP analyses using the closer-to-measured pile displacement profile D2 have significantly lower values of force than the corresponding analyses using the displacement profile D1, as the additional displacements over the depth 4 to 7m reduce the pressure acting on the pile over this depth.

The analysis A6, which replicates closely the large bending moments measured on Pile C, gives a value of slope loading of 51.7kN, or 36% of the design value.

Analysis	Displacement profile	Net downslope pressure acting on the top 4m of the pile (kN)	Net upslope pressure acting on the lower 6m of the pile (kN)
A2	D1	44.6	44.6
A3	D1	78.9	79.0
A4	D2	42.0	42.1
A5	D1	92.8	92.8
A6	D2	51.7	51.7

Table 4.5. The net upslope and downslope forces acting on the pile calculated from the ALP pile pressure distribution.

4.3 The use of elastic analysis in pile design

4.3.1 Elastic analysis in pile design

The method used in these analyses of applying a fixed soil displacement to the pile is considered difficult to use in design because the slope displacements after pile installation are likely to be impossible to predict. There also tends to be an absence of site specific soil stiffness data available for use in such analyses. However, it has been demonstrated here that such an analysis can reproduce the overall bending behaviour better than the assumptions made in a limit equilibrium calculation, even when estimated soil stiffness data from the literature is used. Furthermore, the ability to calculate from the ALP net pressure distribution the applied pile force means that the soil can be displaced into the pile until the pile loading from limit equilibrium slope stability is found. This is likely to give an estimate of the maximum soil and pile displacements that will occur in the real pile.

4.3.2 Investigation of the use of ALP in pile design

To understand better whether an ALP analysis can be realistically used in design, analysis A7 used the originally estimated soil stiffnesses (Table 4.2), and assumed that soil displacements only occurred above 4m and were uniform over this depth (the depth of 4m is assumed in this case to come from the limit equilibrium slope stability calculation). The soil displacements were steadily increased until the upslope restoring force calculated from the ALP pile pressure distribution equalled that from the limit equilibrium slope stability analysis (of 144kN, calculated using a factor of safety of 1.3 on slope stability, and a pile spacing of 2.4m). The pile bending, pressure and displacement results are shown for various soil displacements in Figures 4.8, 4.9 and 4.10.

At the soil displacement of 24.5mm, the upslope pressure acting on the top of the pile is equal to the pile load calculated from limit equilibrium slope stability of 144kN. The maximum bending moment calculated is 180kNm at 6m depth, 50 kNm smaller than the maximum bending moment calculated for design using a limit equilibrium calculation (Chapter 3, Section 3.3). The real pattern of pile bending behaviour is far better predicted by the ALP analysis than by the limit equilibrium calculation in which assumptions have to be made about the distribution of pile head loading and the way the pile bends (in the Hildenborough limit equilibrium pile design the piles are assumed to bend uniformly downslope). The ALP analysis correctly predicts the negative bending moments acting in the upper 3.5m of the pile where the upper section of pile is restrained from bending downslope by the failing mass. The ALP design calculation overestimates the magnitude of the bending moments in the pile (compared with the measured data) because the force from the limit equilibrium slope stability analysis will be larger at the design factor of safety of 1.3 than for the current pile-slope equilibrium.

The effect on the analysis of the magnitude of the soil stiffnesses used was tested by running three analyses (A7, A8 and A9) with 3 different profiles of soil stiffness, and displacing the soil into the pile until the pile force was equal to 144kN. The profiles of soil stiffness in Tables 4.2, 4.4 and 4.6 were applied to the analyses A7, A8 and A9 respectively, with the values of soil stiffness (E') and the stiffness gradient (mE') twice and three times as large in analyses A8 and A9 than for analysis A7. The pressure distributions from these analyses (Figure 4.12) are all much the same – this is to be expected as the limiting displacement condition for the analyses is a pile restoring force of 144kN. The ALP bending moments (Figure 4.11) are similar, but not quite the same – this is the result of the slight differences in the pile base behaviour with the different Weald Clay stiffnesses profiles. The pile displacements vary – as would be expected with the different values of soil stiffness.

The limiting lateral pile pressure (p_u) envelope used in all of the analyses is shown plotted with the net pressure distributions for analyses A5 and A6 in Figure 4.13. It can be seen that the limiting soil-pile pressure is very much larger than the pressures acting on the pile, and the only point where it impacts on the calculated pile pressure distributions is over the top 0.5m depth. Analysis A10 was run to calculate the pile bending moment distribution with the full limiting lateral pressures acting along the length of the pile. Large soil displacements ($> 700\text{mm}$) were applied to the analysis to push the pressures along almost the full pile length to the limiting lateral pressure p_u (Figure 4.14). This generates a maximum pile bending moment of 2250kNm (Figure 4.15), about 15 times larger than the biggest moment measured in the Hildenborough piles, and at least 4 times larger than the maximum design ultimate pile bending capacity of 520kNm. At these very large soil displacements, the pile bends uniformly downslope, and there are no negative bending moments acting in the pile (Figure 4.15). The Hildenborough pile would fail with the formation of a plastic hinge long before the full length of the pile reaches the limiting lateral pile soil pressure p_u . This reinforces the argument on failure mechanisms put forward by Viggiani (1981) (see Section 2.5.3), who points out that most piles are likely to fail in bending, and that the use of p_u in design will result in piles of large bending capacity spaced a long distance apart.

Model section	Elastic modulus at top surface of layer E (MPa)	Increase in elastic modulus with depth mE (MPa/m)
Weald Clay embankment fill	24	0
Weathered Weald Clay	24	3.0
Weald Clay	27	9.6
Rockfill	60	0

Table 4.6. Soil stiffness data used in analysis A9.

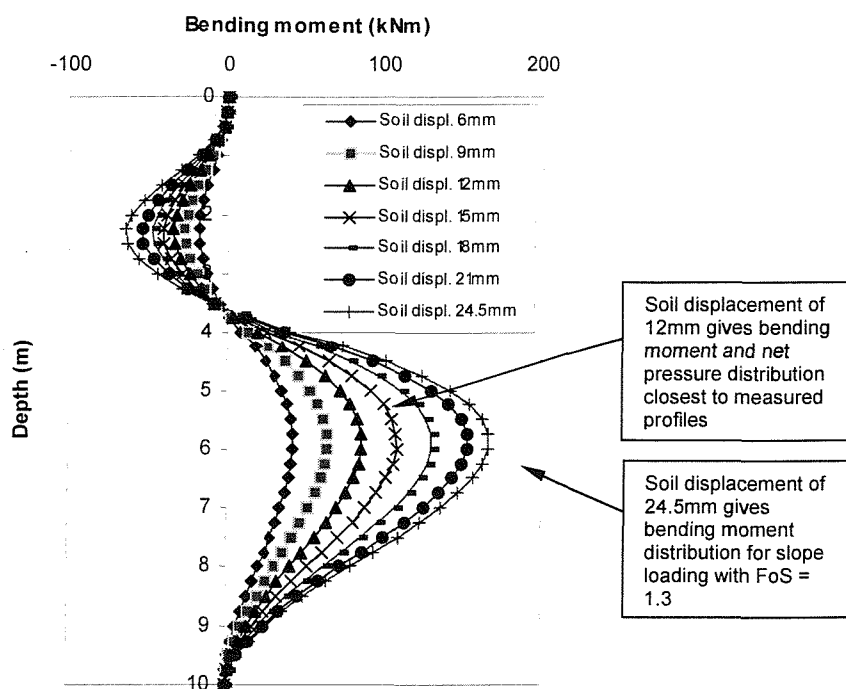


Figure 4.8. Bending moment results from analysis A7.

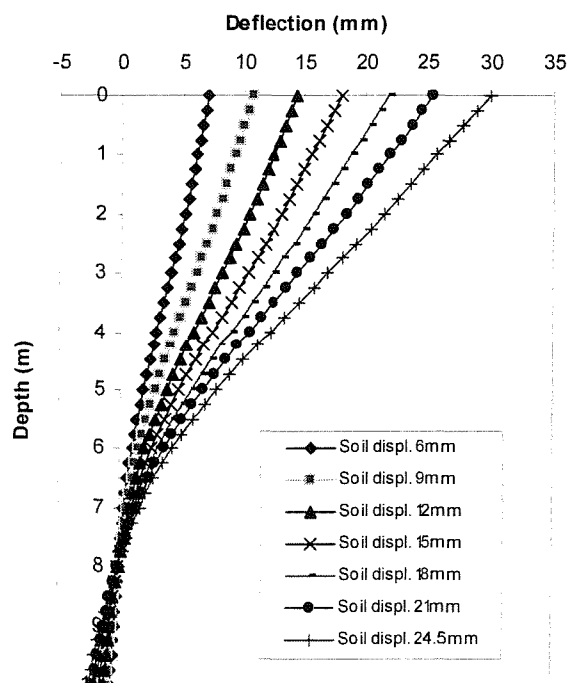


Figure 4.9. Displacement results from analysis A7. Downslope displacements are positive.

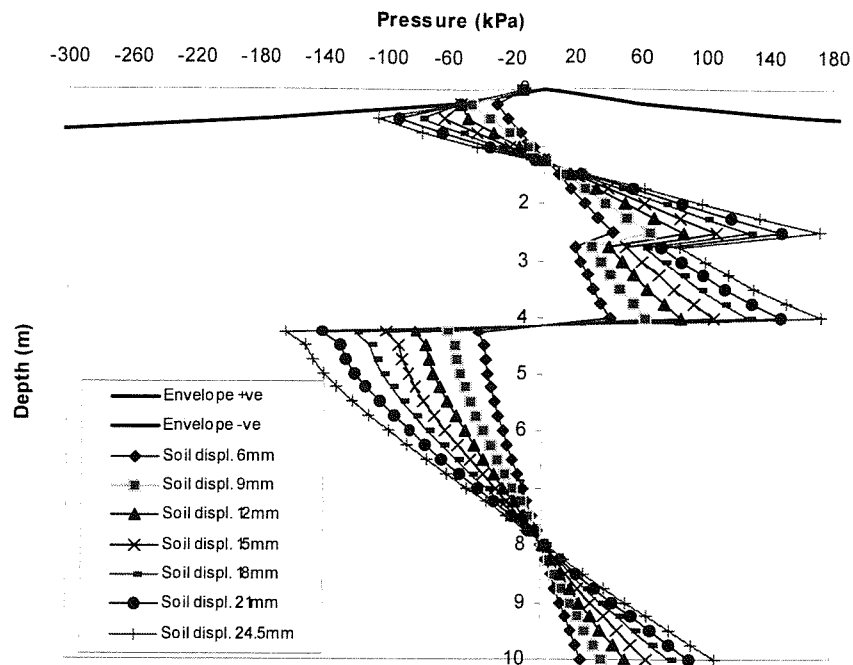


Figure 4.10. Net pile pressure results for analysis A7. Pressures acting downslope are positive.

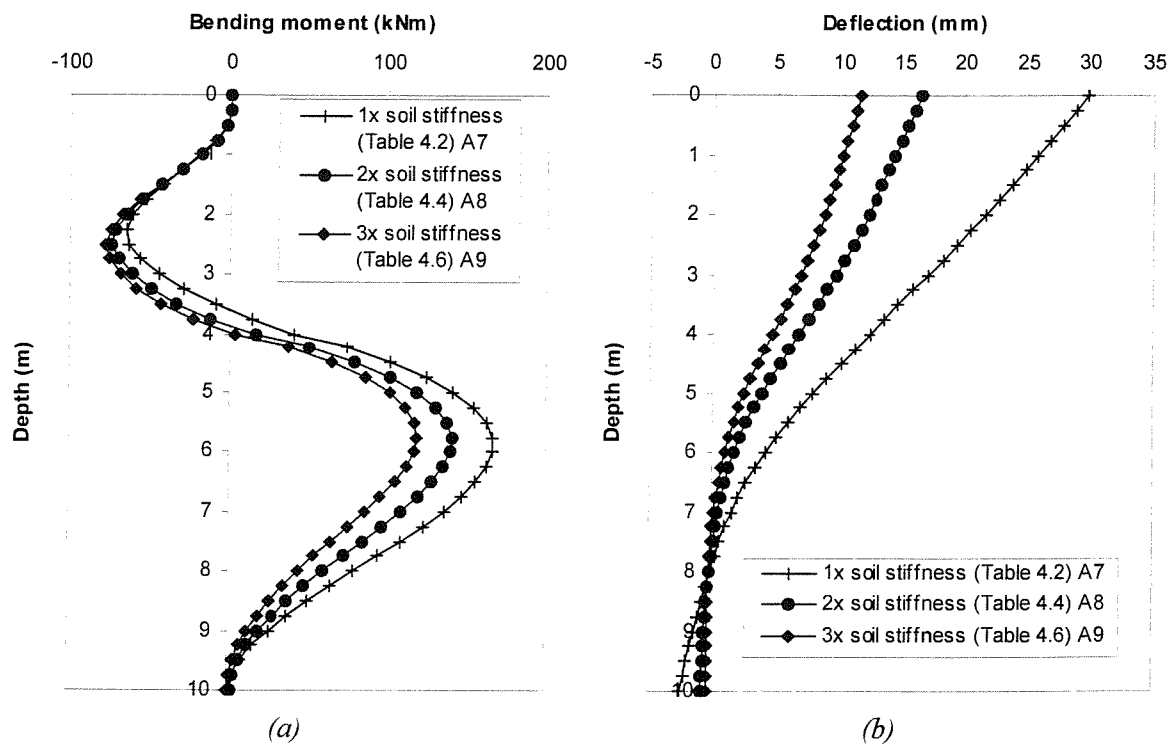


Figure 4.11. (a) Bending moment and (b) displacement results from analysis A8, A9 and A10, all at a slope load of 144kN. Downslope displacements are positive.

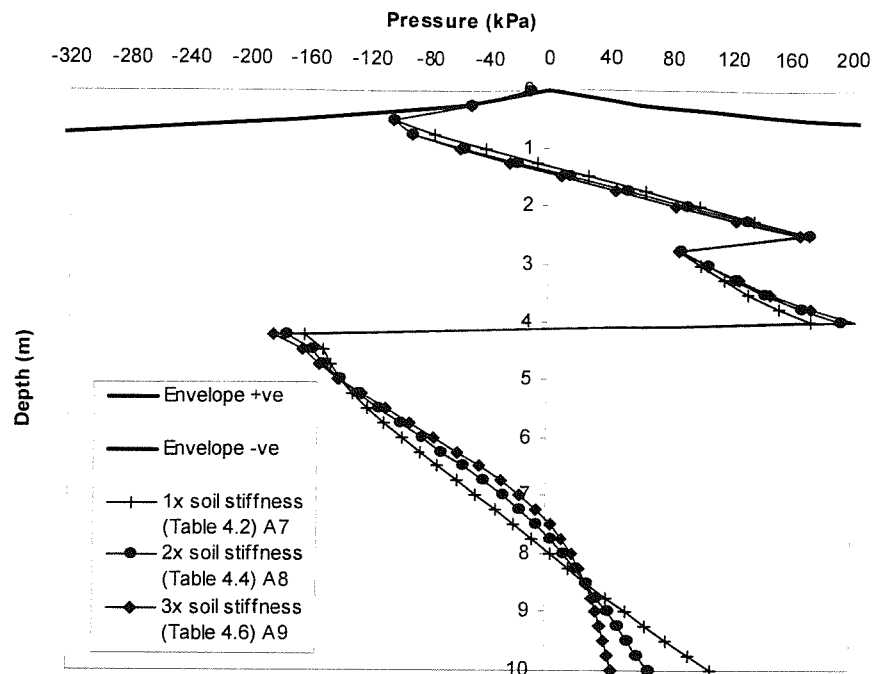


Figure 4.12. Net pile pressure results for analysis A8, A9 and A10, all at a slope load of 144kN. Pressures acting downslope are positive.

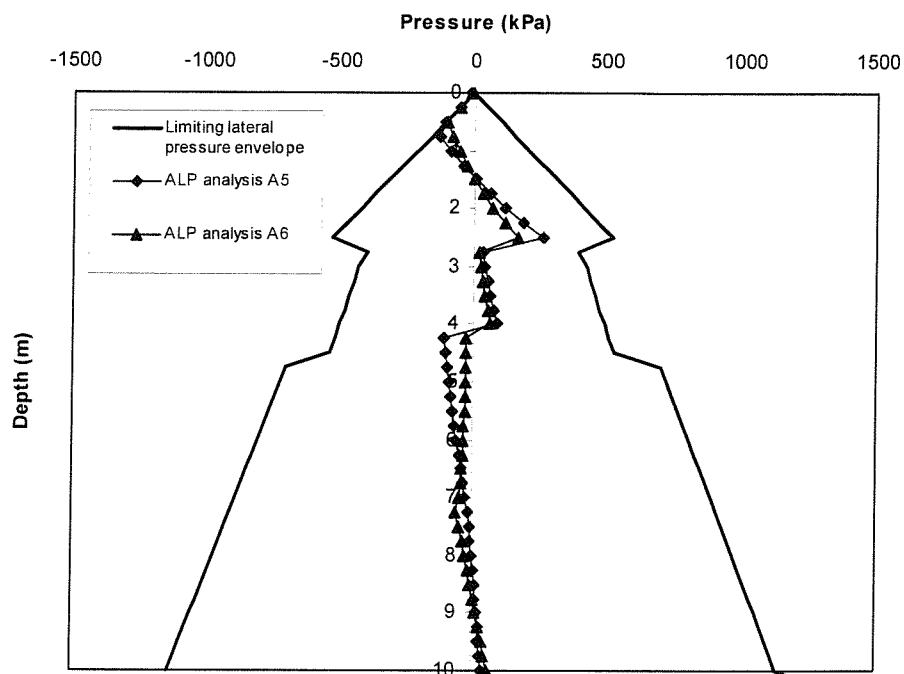


Figure 4.13. Limiting lateral pressure envelope plotted with the pressure distributions from analyses A5 and A6.

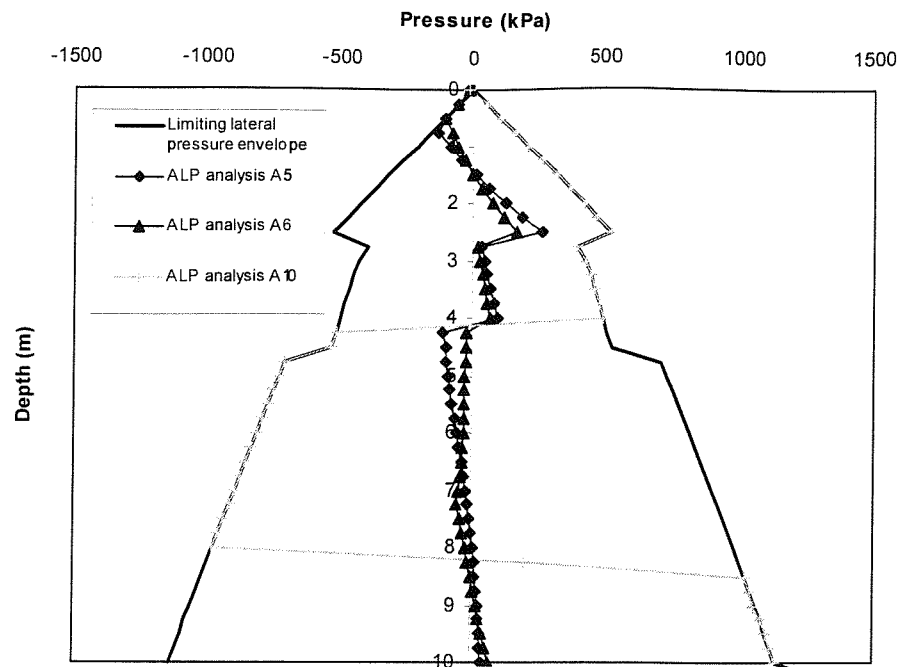


Figure 4.14. Limiting lateral pressure envelope plotted with the pressure distributions from analyses A5, A6 and A10.

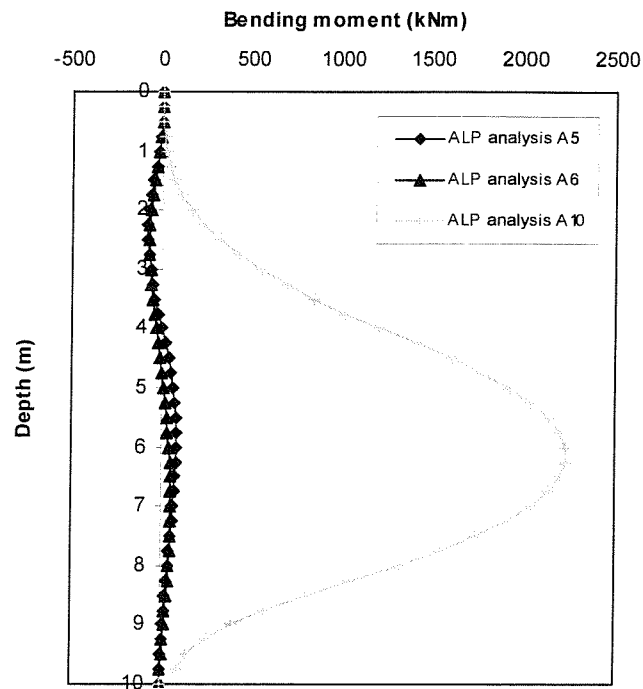


Figure 4.15. Distributions of bending moment acting in the pile for analyses A5, A6 and A10.

4.3.3 Implications for design

Although in a design situation it will be almost impossible to predict the equilibrium slope displacements likely to occur adjacent to the pile, it was shown here that the net pile pressures can be compared with the restoring force from limit equilibrium slope stability analysis to get the elastic pile analysis to give a reasonable calculation of pile bending moment and displacement. Elastic pile design has the advantage over limit equilibrium pile design of being able to better predict the pile bending behaviour. However, assumptions will often need to be made about the soil stiffness and about the depth and distribution of soil displacements.

Analyses A7, A8 and A9 demonstrate that the magnitude of the soil stiffness does not matter that much, and a good approximation of pile behaviour can be found by estimating the soil stiffness values from data in the literature. In the application of piles to stabilise overconsolidated stiff clay cut slopes and embankments, the soil stiffness and its variation with depth should be relatively easy to estimate to within 2 or 3 times the actual stiffness, and differences within this range are unlikely to have a big effect on the pile bending moments calculated. It may however be more difficult to use such a method if there are multiple layers of heavily contrasting stiffness, where the differences in the magnitude of the stiffness within individual layers may have a greater effect on pile bending behaviour.

Analyses A5 and A6 demonstrate that the soil displacement profile can have a significant impact on the pile bending behaviour, with the slight differences in soil displacement profiles D1 and D2 giving a marked difference in the bending and pressure distributions for these analyses. In practice, inclinometer tubes are installed into many failing slopes to measure slope displacements, and the distribution of soil displacement input into the elastic analysis can be based on data from these.

In all the analyses run, the limiting soil-pile pressure (p_u) distribution is very much larger than the pressures acting on the pile, and the only point where it impacts on the calculated pile pressure distributions is over the top 0.5m depth. This is an indication that the design of piles using the limiting lateral pile pressure is likely to result in the calculation of structural forces that are much larger than those ever likely to occur in reality, particularly in short steep infrastructure slopes where the force from the failing mass of soil is limited by the size of the slope (although the use of p_u may be more realistic in large landslides). The soil displacements required to push the full length (or even parts of the length) of the pile to the limiting lateral pile pressure would also be unacceptable in the case of most infrastructure slopes. The most critical mode of failure is therefore likely to be formation of a plastic hinge in the pile section rather than a flow of soil around the pile.

4.4 Summary of elastic analyses

Elastic analyses carried out using the program ALP have been used to back analyse the behaviour of the Hildenborough piles. The major findings were as follows:

- i. The Hildenborough piles were modelled using the program ALP. The values of soil stiffness required for input to the model were estimated from data in the literature. The pile was loaded within the ALP model by applying the measured soil displacements (maximum of 7.5mm) to the top of the pile. The method was able to reproduce reasonably closely the overall bending behaviour of the pile, with the pile displacements a good match to the measured displacement plots. The ALP bending moments were found to be smaller than those measured in Pile C.
- ii. The profile of soil stiffness required to re-create the measured pile bending behaviour was explored. The majority of the pile was surrounded by the intact Weald Clay and the rockfill material used to construct the berm at the toe of the slope. It was found that to calculate more accurately the Pile C rebar gauge bending moments the stiffness of the Weald Clay needed to be increased to better constrain the base of the pile from backward rotation, and the rockfill stiffness needed to be increased to better constrain the head of the pile from bending downslope. The results from analysis A6 were a good match to the bending moments measured in Pile C by the rebar strain gauges. However, the rockfill stiffness of 150MPa in analysis A6 is large, and it is uncertain whether the rockfill really has such a large stiffness so close to the slope surface.
- iii. The back analysis of the Hildenborough piles demonstrated that elastic analysis may be a better means of design than a limit equilibrium calculation, even if the soil stiffnesses have to be estimated from values obtained from the literature. In practice, it should be possible to estimate the soil stiffnesses to within 2 or 3 times the actual operational stiffness, and it was demonstrated that differences within this range are unlikely to have a major effect on the pile bending moments calculated. In particular, the elastic analysis much better predicts the overall bending behaviour of the pile, and removes the need (as required within limit equilibrium pile design) to make assumptions about the way in which the pile will bend. Pile bending moments and displacements were calculated by comparing the net lateral pressure distribution from the elastic pile analysis with the force required to stabilise the slope obtained from limit equilibrium slope stability analysis. If a factor of safety of 1.3 is used in the slope stability analysis, the calculated restoring force will be larger than the force acting on the piles under working conditions.

- iv. In all the analyses run, the limiting soil-pile pressure distribution was found to be much larger than the pressures acting on the pile. This is an indication that the use of the limiting lateral pile pressure for the design of piles to stabilise infrastructure slopes is likely to result in structural forces that are much larger than those likely to occur in practice. The critical mode of failure is likely to be via the formation of plastic hinge in the pile section.

Chapter 5. Back analysis of the Hildenborough embankment using 3D finite difference analysis

5.1 Introduction

The aim of the finite difference study of the Hildenborough instrumented section is to back analyse the behaviour of the full slope and pile system, and in particular to try to model the effect of the construction sequence and embankment slope regrading on the pile behaviour. To do this, the finite difference program FLAC 3D has been used.

The advantage of 3D finite difference/element methods over conventional limit equilibrium or elastic analysis methods is fully discussed in Chapter 2, but includes the ability to model:

- 3D effects, including soil arching;
- the *in situ* soil stress state into the model; and
- the stability of the slope and the effect that the piles have on slope displacements.

However, analysis of the slope is made difficult by the lack of site specific stiffness data for the Weald Clay and rockfill, and lack of information on pore pressures and the change in pore pressures with time within the embankment. In particular, the absence of pore water pressure data makes it very difficult to model exactly how close to instability the embankment slope is during various stages of the embankment re-grading. The stiffness parameters needed to replicate the measured pile behaviour were investigated in Chapter 4, and a set of parameters that were adequate for use in the finite difference analyses determined.

5.2 Finite difference analyses

5.2.1 FLAC 3D

The program FLAC3D (ITASCA, 1997) is an explicit finite difference code that models soil structures that undergo plastic flow when their yield limit is reached. Materials are represented by three dimensional eight-noded mesh elements that follow linear or non-linear stress-strain laws in response to changes in stress and displacement boundary conditions. If stresses are high enough to cause materials to yield and flow, the mesh elements deform and move with the failing material. The Lagrangian calculation scheme is able to model large deformations with no numerical distress. FLAC3D uses low order constant stress elements, in which the stress in each element is uniform. This requires the use of many more mesh elements than is typical in other finite element based programs, but allows failure planes or bands to be more accurately represented by the displacement or failure over a much narrower section of the mesh. Each element in a FLAC3D mesh may have a different constitutive model or property, and a continuous gradient or statistical distribution of any property can be specified.

The calculation scheme used by FLAC3D takes a large number of calculation steps, each one progressively redistributing the out of balance force (OBF), caused by changes to stress or displacement boundaries, through the mesh. A typical analysis will make a large number of calculation steps until numerical equilibrium is achieved, although the exact number of steps will depend on the size of the stress or displacement changes applied to the mesh (the magnitude of the initial 'out of balance force') as well as material stiffnesses and constitutive models. Complex analyses incorporating groundwater, such as those presented here, take several million steps to solve. The Lagrangian calculation scheme solves the full equations of motion for each step the analysis takes, and this allows the examination of the development of yield or failure as it occurs. This is particularly useful when analysing the development of progressive failure. After each calculation step, the 'OBF' (or size of the external force applied to the boundary of the mesh that has not yet been distributed through the mesh by the calculation steps taken thus far) is displayed. By monitoring the behaviour of this out of balance force it is possible to see whether the analysis is converging towards equilibrium (shown by a reduction in OBF) or total instability (shown by an increase in OBF). Each mechanical calculation step does not represent any real time, and mechanical steps are assumed to occur instantaneously.

Groundwater, including pore pressure generation and water flow, can be coupled into the model to analyse consolidation problems. If the groundwater module is in use, the soil is modelled as saturated and mechanical deformation of the soil material generates pore pressures. An explicit groundwater solution scheme allows groundwater calculation steps to be made that represent a flow of water through the mesh in accordance with the isotropic form of Darcy's law. Each

groundwater calculation step represents a period of model consolidation time, with the maximum consolidation time for each groundwater calculation step calculated by FLAC3D and controlled by a series of factors that affect numerical stability.

5.2.2 Finite difference mesh

The 3D mesh used in the analyses is shown in Figure 5.1. Maximum use of symmetry was made, such that only half a pile and half the space between adjacent piles was modelled in the analysis. The mesh represents a section through the embankment 39.5m deep (z), 45m long (x) and 1.2m wide (y). The stabilising piles were 10 m long and 0.6 m in diameter. The mesh modelled the geometry of the real embankment as closely as possible, although there were some slight compromises close to the pile due to the complex element shapes required to surround the semi-circular pile section. The mesh contained a total of about 5000 elements.

The vertical mesh boundaries were restrained on roller supports, and free to move in the z (vertical) direction. The boundaries through the section of the embankment were only restrained in the y-direction, and were free to move in the x and z-directions – this is obviously necessary to allow slope failure to occur. The bottom boundary of the mesh was fixed in all directions. The left hand boundary of the mesh (as viewed in Figure 5.2) was positioned to represent a plane of symmetry through the centreline of the embankment (the top of which was assumed to be 6m wide). The right hand boundary was placed 1.5 embankment lengths away from the toe of the slope, considered to be sufficiently far that it would not influence the major embankment slope and pile movements, in a compromise with the number of elements in the mesh (with the number of elements having a major impact on solution time).

The mesh was designed to allow the addition of elements to model the installation of the rockfill berm, as well as the subsequent removal of some of the berm and the addition of further fill on the upper embankment surface as the slope is regraded into a two stage slope. The elements making up the semicircular pile section were incorporated in the mesh from the start, but were initially given soil properties. These sections of the mesh are shown in Figure 5.2.

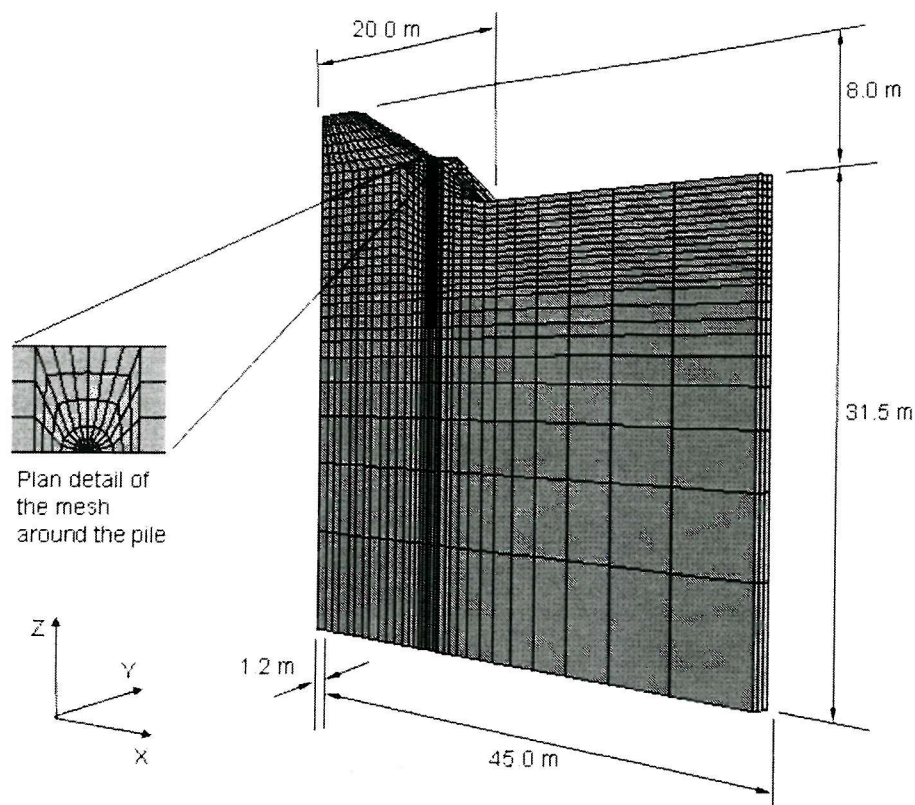


Figure 5.1. The embankment mesh.

5.2.3 Soil and groundwater properties

The embankment was modelled as uniform saturated clay and given the strength and stiffness properties of a typical tipped embankment fill (see section 4.2.2). The underlying ground was modelled as a uniform stratum of saturated overconsolidated Weald Clay. Consolidation elements were used throughout to allow time dependant stress and strain to be modelled. The soil was modelled as an isotropic linear elastic - Mohr-Coulomb plastic model, with a non-associated flow rule and a Young's modulus increasing with depth.

Soil stiffness

As the original pile design was carried out using limit equilibrium methods, no stiffness data were available from the limited geotechnical investigation. The stiffnesses of all the materials used in these analyses were therefore based on stiffness data for the Weald Clay presented in the literature, and the result of the back analysis of the pile behaviour using ALP in Chapter 4.

The soil stiffnesses used in ALP analysis A5 and A6 (section 4.2.3) were felt generally to give results closest to the actual pile behaviour at Hildenborough, with the higher values of stiffness for the Weald Clay and rockfill better constraining the base and head of the pile. The A5/A6 analysis stiffnesses were adopted for the finite difference analysis, and are shown in Table 5.1. In Chapter 4 it was felt that the rockfill stiffness of 150 MPa was rather large and that the high stiffness required to replicate the pile bending may result from the inability of ALP to model

surface construction effects and soil compaction (vertical soil displacements). However, the case for this is not totally clear, and a stiffness of 150 MPa was also used for the rockfill in the FLAC3D analyses.

Figure 5.2 shows the areas of the mesh that were represented by the different soil properties. It should be noted that the upper section and higher surface side slopes of the embankment were in part built up of railway ballast, but because the exact depth and spread of ballast at the instrumented section is uncertain, it has been modelled in the analyses as a horizontal band 1.5m deep. The ballast was assigned the same stiffness as the rockfill material.

Soil strength

The soil strength parameters were derived from the basic ground investigation of the embankment at Hildenborough (Table 5.2). The strength of the Weald Clay compares well with the strength data for the Weald Clay given by Loveridge (2001) of $\phi = 25^\circ$ and $c' = 0$ kPa.

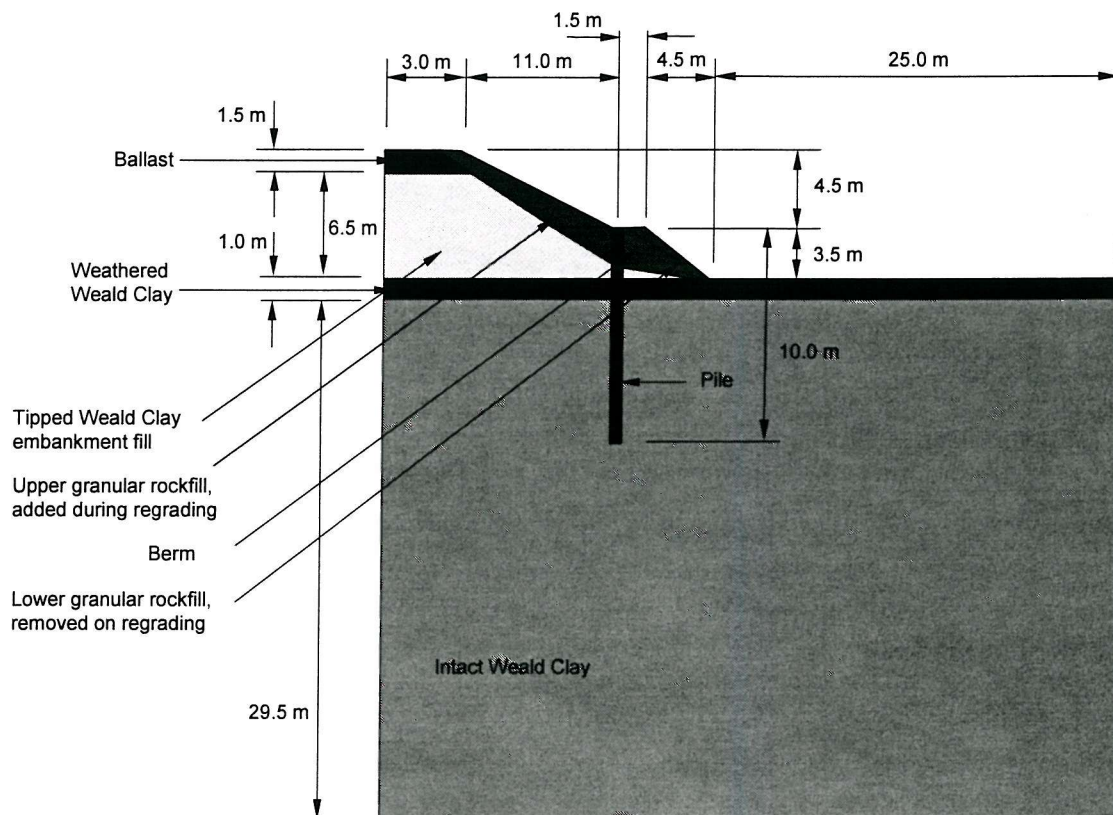


Figure 5.2. Cross section through the embankment model showing the zones of different geomaterials

Model section	Elastic modulus at surface of clay material E' (MPa)	Increase in elastic modulus with depth mE' (MPa/m)	Poisson's ratio ν'
Weald Clay embankment fill	16	0	0.3
Weathered Weald Clay	16	2.0	0.3
Weald Clay	18	6.4	0.3
Rockfill	120	0	0.3

Table 5.1. Profiles of Soil stiffness used in the finite difference analyses.

Parameters	Friction angle, ϕ' (degrees)	Effective cohesion, c' (kPa)	Unit weight, γ_s (kN/m ³)
Weald Clay embankment fill	25	1	19
Weathered Weald Clay	25	1	19
Weald Clay	30	5	19
Granular rockfill	35	0	19

Table 5.2. Design soil strength parameters taken from the Hildenborough embankment geotechnical investigation report.

Dilation

The dilation angle ψ effects the volume change of the soil during yielding. The actual volume change exhibited by a real soil during yielding is quite variable (for example, a loose soil will show a decrease in volume before dilation), but the elastic - Mohr-Coulomb plastic model implies a constant value of dilation. If $\psi = \phi$ then the plasticity flow rule is associated and the stress and velocity characteristics coincide (Griffiths and Lane, 1999). The use of the associated flow rule in combination with frictional soil models tends to predict far greater dilation than is observed in reality. This tends to result in an over stiff response in the short term due to the generation of negative excess pore water pressures at undrained yield (Griffiths, 1982). In reality, dilation would be restricted to a failure zone much thinner than that predicted by a finite difference analysis, in which any failure surface or zone will be at least one element thick. The dissipation of negative pore pressures due to dilation in the field would therefore be much more rapid than in the finite difference analysis. In the analyses run here, the non-associated flow rule ($\psi = 0$) was adopted. The zones of failure should be limited, as the slope is being stabilised by the piles.

Initial soil stress

The *in situ* stresses prior to berm construction were prescribed using an earth pressure coefficient, $K_0 = \sigma'_h / \sigma'_v$ (where σ'_h and σ'_v are the horizontal and vertical effective stresses

respectively). O'Brien *et al* (1992) present lateral stress data taken from pressuremeter and laboratory tests on the Weald Clay, and show the material at their test site to be heavily overconsolidated close to the slope surface. *In situ* lateral stress data taken from O'Brien *et al* (1992) were converted into a distribution of K_0 with depth, which is shown in Figure 5.3. The passive coefficient of earth pressure for the weathered Weald Clay (K_p) is 2.46 and so the maximum value of K_0 used in model was limited to 2.0 to prevent passive failure of elements close to the Weald Clay ground surface.

The excavated Weald Clay material used to construct the embankment would have lost any overconsolidation stress in the process of excavation and tipping, and an initial $K_0 = (1 - \sin\phi) = 0.6$ was applied to this material.

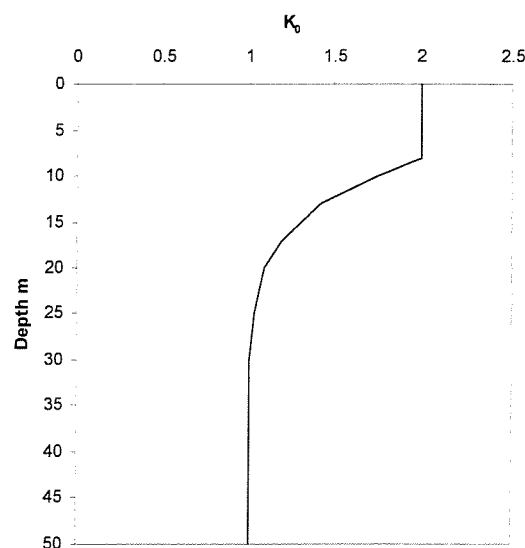


Figure 5.3. The Weald Clay *in-situ* stress used in the model (interpreted from data given by O'Brien *et al*, 1990).

Permeability

The permeability is important as it affects the time that it takes the pore water pressures as a result of berm construction and regrading to dissipate. This will have a differing impact on the pile depending on whether the time it takes the pore pressures to dissipate is longer or shorter than individual construction stages (for example, whether the pore pressures resulting from berm construction fully dissipate before pile installation).

There are no site data for the permeability of either the intact Weald Clay or the embankment fill. It is typical for the Weald Clay to contain a lot of silt partings and fissured siltstone bands which give highly anisotropic permeability (Gallois and Edmunds, 1965; Loveridge, 2001). However, FLAC 3D does not allow an anisotropic permeability to be modelled. In the absence of any permeability data for the site, a uniform permeability of 10^{-10} m/s was considered to be

sufficiently representative of the intact Weald Clay, representing a compromise between the less permeable stiff clay and more permeable silty bands.

The permeability of the embankment fill is likely to be similar to that of stiff clay. However, the permeability will be increased by the matrix structure of the fill and the tendency for fissures and cracking close to the slope surface (Mott MacDonald, 1999a). In the analyses, the embankment fill was given a slightly larger permeability of 2×10^{-10} m/s.

Initial pore water pressures

The values of pore water pressure at the model boundaries remained fixed during the analyses, and this held the pore water pressures within the embankment in a fairly steady state. Additional pore water pressures were generated in the slope during construction events such as berm installation, and these were then allowed to dissipate by running the analyses in consolidation mode. In reality, the pore water pressures in the embankment also changed as a result of rainfall infiltration and summer drying at the embankment and ground surface. However, owing to the lack of measured pore pressure data, this is not modelled, and the pore water pressure boundaries remain unchanged during the analysis.

The pore water pressures applied to the model were those thought likely to best represent the real pore pressure condition in the period from December 2000 to May 2001, when the berm and pile construction works were taking place. Perry *et al* (2001) give typical distributions and ranges of pore water pressure measured in old London Underground clay embankments. Within the core of the embankment, the maximum winter pore pressures are typically hydrostatic from 1.0-2.0m depth. This is consistent with pore water pressure measurements from piezometers installed into the core of a London Clay rail embankment at Wimbledon (Penn, 2000). The side slopes are typically drier than this, and there may be significant suctions maintained throughout the year by the presence of vegetation on the slopes (Perry *et al*, 2001). At Hildenborough, vegetation was stripped from the embankment prior to summer 2000, and so was unable to cause an extensive desiccation zone over the summer period. October, November and December 2000 were particularly wet, and at the end of December 2000 the pore pressures may be expected to be fairly close to the maximum. In late March/early April, when the piles were being installed, water was visibly ponding at the toe of the embankment, indicating that the foundation was close to total saturation. To reflect the likely 'close to winter maximum' pore pressures, the starting point for the embankment pore pressures was taken as a hydrostatic distribution of pore pressure from the ground surface, with the horizontal ground surface boundary and the clay embankment slope surface at -10kPa . However, this distribution, when modelled in SlopeW, was found to cause the embankment (without the granular berm) to fail. Consequently it was felt that the embankment must have contained a larger zone of negative

pore water pressures that were acting to maintain its stability. In the FLAC3D model the initial pore water pressure distribution is modified from that above by fixing a horizontal band of the clay embankment 2.0m deep directly below the ballast to -10kPa. The rockfill material and railway ballast were modelled without groundwater, i.e. they were assumed to remain dry and no pore pressures were able to generate in these materials. The pore pressures on the upper, left and right boundaries of the model (when the model is viewed in section) remained fixed throughout the analyses.

5.2.4 The piles

The circular pile used to stabilise the slope was 10m long and 0.6m in diameter. The pile (of which only half is modelled in the analyses) was constructed of 480 brick elements, arranged in cross section as shown in Fig 5.4. The bending moment acting in the pile was calculated from the z direction element stresses. As stated in Section 5.2.1, the uniform stress elements used in FLAC3D are of a much lower order than those commonly used in finite element solutions, and subsequently a large number elements are required to model accurate bending behaviour. The bending moment was calculated using the stresses (σ_a and σ_b) from the elements shown on the pile cross section (Figure 5.4), and by assuming the stresses in these edge elements to apply across the section of the pile. The bending moment calculated using these stresses can be shown theoretically to be almost equivalent to the bending moment calculated by a uniform stress distribution through the pile (of stress σ_1 at the far boundary).

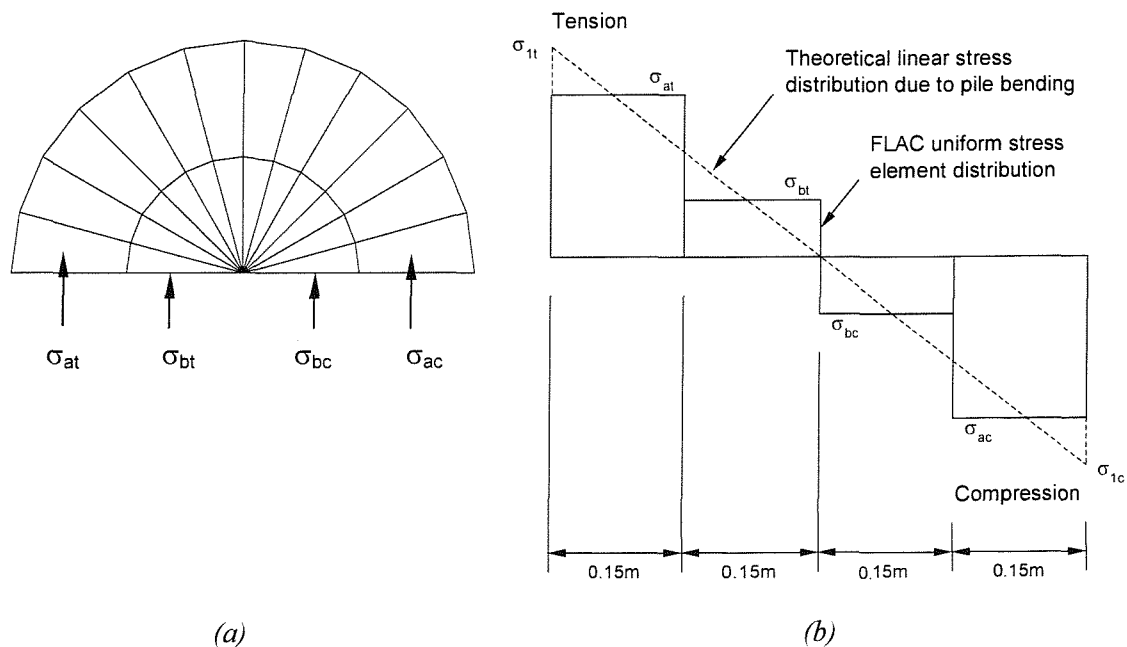


Figure 5.4. The pile used in the analyses; (a) a cross section through the half pile showing the arrangement of elements, and the elements used to calculate the bending moment; and (b) the distribution of stress given by the uniform stress elements in FLAC 3D in the direction of bending.

The pile was given concrete/steel stiffness properties with Young's modulus $E = 26.8 \times 10^6$ kN/m² and Poisson's ratio $\nu = 0.15$. The pile stiffness was not reduced to account for concrete cracking. The pile elements were attached directly to the soil elements. While it is more desirable to model the interface between the pile and the soil, it was found to be difficult to incorporate interface elements between the soil and the pile when sections of the pile were not modelled in the initial mesh.

Careful consideration has to be given in all finite element analyses to the number of elements required in portions of the mesh in which there are large gradients of stress, to ensure that sufficient elements are used to accurately represent the stress gradient. In these analyses, the largest gradient of stress results from the bending stress generated in the pile. A series of analyses were run to assess the number of elements required within the pile to give sufficiently accurate values of stress from which to calculate the bending moment. In the course of this investigation it was considered necessary to model only the pile, which was completely fixed at its base, and uniformly distributed load of 20kN/m was applied down one side (Figure 5.5). Only half a pile was modelled (as in the full slope analyses), and the section through the pile was restrained from moving in the y direction, to prevent any torsion within the pile due to the eccentric loading (this boundary within the full slope analyses was also restrained from moving in the y direction). The bending moment distributions calculated from the internal stresses for piles 20 and 40 elements long (the cross section through the pile, Fig 5.4, remained the same in both cases) are compared with the theoretical bending distribution in Figure 5.6. The figure shows that with more elements in the pile, the bending moment calculated becomes progressively closer to the theoretical curve, so that the pile 40 elements long gives an almost identical bending distribution to the theoretical solution. Note that while the 20 element pile gives a bending moment that is 80% of the theoretical value at 9.0m depth (where the bending moment is around 1500kNm), the 20 element pile moment is almost 100% of the theoretical value at 0-2.5 m depth (where the bending moment is around 100kNm). In the full slope analyses, a pile 20 elements long was used, as there had to be some trade off between the accuracy of the bending moment calculated and the run time of the analyses.

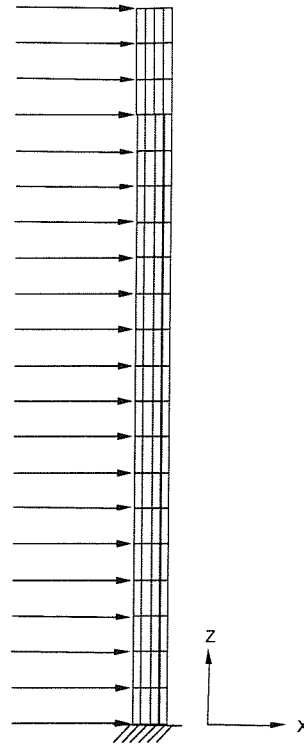


Figure 5.5. The pile mesh 20 elements deep, showing the base fixity and the uniformly distributed load applied as a series of nodal forces down one edge of the pile.

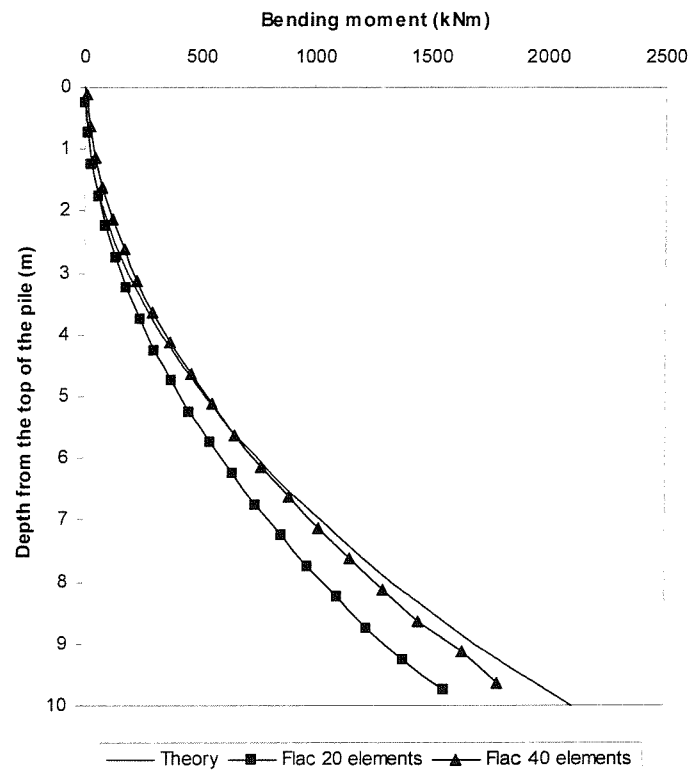


Figure 5.6. The bending moment calculated from element stresses for piles 20 and 40 elements long.

5.2.5 Analysis sequence

The analysis steps and time intervals between the steps are summarised in Table 5.3. Further discussion about each of the stages is given below.

Stage 1 – embankment generation

The initial mesh comprised the embankment and the intact Weald Clay substratum. *In situ* stresses were applied to the mesh to account for the weight of the clay embankment on the substratum. In effect, the embankment and substratum were ‘wished’ into place. The pile elements were given soil strength and stiffness properties. The analysis was stepped in mechanical mode only until mechanical equilibrium was reached. No flow of groundwater was allowed, but pore water pressures were generated as a result of soil displacement. As the embankment was wished into place in this initial stage, the strength parameters of the embankment slope were increased to prevent the initially large out of balance force causing failure to occur in the slope. As mechanical equilibrium was reached, the strength parameters were reduced to the expected values, and the model run again to ensure continued equilibrium. The analysis was then run in coupled mode (mechanical and groundwater flow) until steady state equilibrium groundwater pressures were achieved.

Stage 2 – berm installation

The rockfill berm at the toe of the slope was installed. The analysis was initially run in mechanical mode to redistribute stresses and generate pore water pressures in the model. The analysis was then run in coupled mode for 4 months (the period between berm construction and pile installation).

Stage 3 – installation of the pile

The pile was given full pile strength and stiffness properties. The analysis was then run in coupled mode for a further month.

Stage 4 – re-grading of the embankment

Regrading of the rockfill on the embankment was carried out by the addition of elements upslope of the pile (with rockfill soil strength and stiffness properties) and the removal of some elements on the berm slope downslope of the pile. The analysis was initially stepped to equilibrium in mechanical mode only, so that the instantaneous result of adding the granular rockfill material on the pile bending behaviour was recorded, i.e. modelling undrained conditions. The analysis was then run in coupled mode for 2 years, so that the effect of dissipating pore pressures on pile behaviour was modelled.

Stage	Description	Mechanical stepping of analysis	Groundwater stepping of analysis	Consolidation time to next stage
Stage 1a	Embankment and substratum generation. Analysis is stepped to generate equilibrium <i>in situ</i> stress and pore water pressures. Embankment strength parameters are increased over this stage to prevent the out of balance force causing slope failure.	Yes	No	No elapsed time
Stage 1b	Groundwater flow turned on to allow initial dissipation of generated pore water pressures.	Yes	Yes	Until steady state pore pressure condition is reached
Stage 2a	Berm construction. Analysis is stepped to generate new <i>in situ</i> stress and pore water pressures as a result of berm construction.	Yes	No	No elapsed time
Stage 2b	Groundwater flow turned on to allow initial dissipation of generated pore water pressures.	Yes	Yes	Approximately 4 months (110 days)
Stage 3	Pile installation. The analysis continues to be stepped with groundwater flow on.	Yes	Yes	Approximately 1 month (22 days)
Stage 4a	Regrading of the embankment. Additional elements added to the model to construct the two stage slope. Analysis is stepped to generate <i>in situ</i> stress and pore water pressures as a result of berm regrading.	Yes	No	No elapsed time
Stage 4b	Groundwater flow turned on to allow initial dissipation of generated pore water pressures.	Yes	Yes	2 years (730 days)

Table 5.3. Basic analysis stages

5.2.6 Analyses run

A number of analyses were run to investigate different construction effects on the pile and to investigate the use of finite difference analysis in design. These analyses were labelled H1-H3. Table 5.4 lists the analyses.

To run the full set of analysis stages (which were completed in analyses H2 and H3) took about 1 week. The long run time reflects the small consolidation time step for each groundwater calculation carried out in the analyses. As a result several hundred thousand steps had to be taken to represent even one year of consolidation time. The consolidation time step is calculated by FLAC3D and controlled by a series of factors that affect numerical stability, including the shape of elements in the mesh, bulk modulus of the water and soil permeability.

Analysis	Description
H1	Analysis H1 was run to investigate the initial pore water pressure condition required for stability of the embankment slope (Stage 1).
H2	Analysis H2 was run to model the full berm construction, pile installation and regrading of the rockfill (Stages 1-4). This analysis was also used to investigate the pile behaviour under ultimate slope loading (with a factor of safety of 1.3 applied to the soil strength).
H3	Analysis H3 was run to model the full berm construction, pile installation and regrading of the rockfill (Stages 1-4), but with changes to stage 3 where the impact on the pile bending of excavating holes for the installation of the slope inclinometers was investigated.

Table 5.4. Description of analyses.

5.3 Results

5.3.1 Stage 1 results – embankment generation

Results are given for each of the stages in the analysis. The first stage modelled the instantaneous formation of the embankment and generated an equilibrium pore water pressure condition. The initial pore water pressure condition specified (Analysis H1) caused failure of the embankment slope, with a failure surface starting at the crest of the embankment and emerging at the toe of the slope (Figure 5.7). The initial pore water pressures in Analysis H1 were set as hydrostatic from -10kPa at the slope and ground surface, exempt a zone 2.0m deep at the top of the embankment directly beneath the ballast that was fixed to -10kPa.

To prevent failure of the embankment the pore water pressure boundaries were changed to slightly reduce the pore water pressures in the embankment. For the embankment to be just stable, the zone of the embankment in which the pore pressures were fixed to -10kPa had to be extended to cover the top 2.5m of the clay section of the embankment (Analysis H2). The pore water pressure boundaries on the left hand end of the mesh (as viewed in Figure 5.2) and slope surface were also adjusted accordingly. This gave a pore pressure condition not dissimilar to that used in the design limit equilibrium slope stability analysis (Figure 3.8).

The embankment remained stable in Analysis H2. The equilibrium pore water pressures at the end of Stage 1b are shown in Figure 5.8.

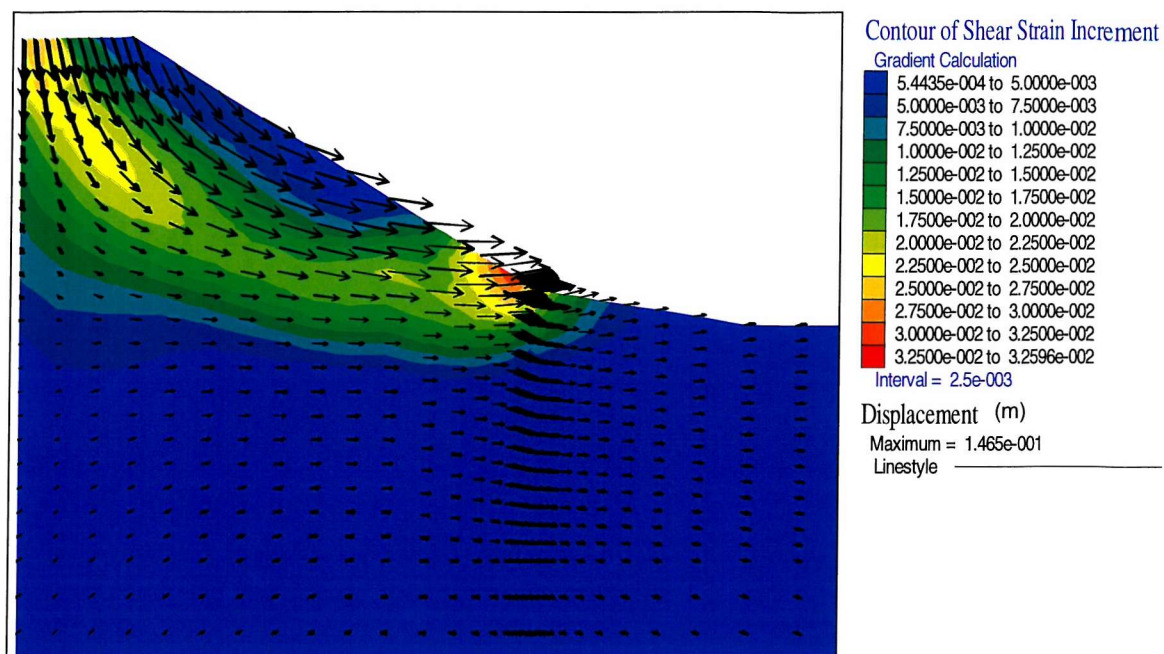


Figure 5.7. Contours of accumulated shear strain plotted with vectors of displacement, showing failure of the embankment along a surface from the crest of the embankment through to its toe (Analysis H1). The contours are plotted for the front surface of the model, i.e. for the section through the pile.

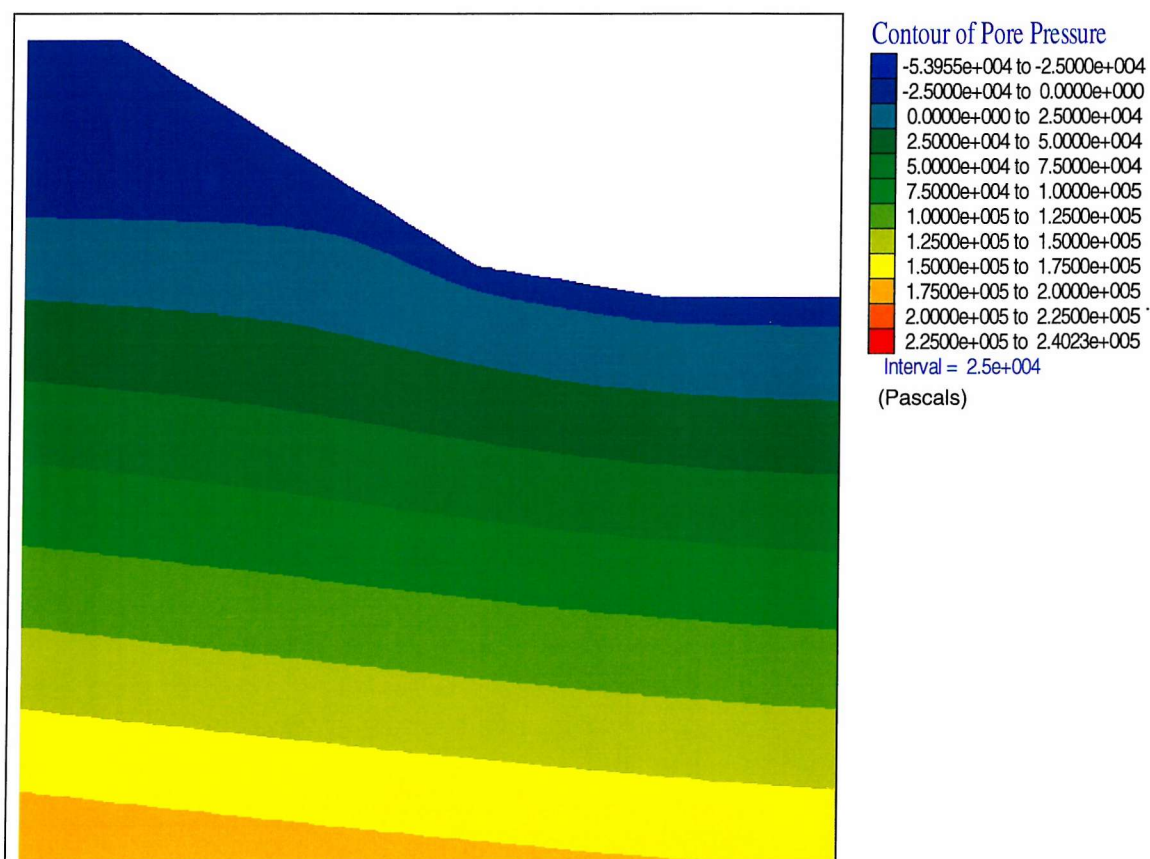


Figure 5.8. Contours of pore water pressure, at the end of Stage 1b (Analysis H2). The contours are plotted for the front surface of the model, i.e. for the section through the pile.

5.3.2 Stage 2 results – berm installation

Introduction of the rockfill berm into the analysis initially generated an increase in pore pressures directly beneath the berm, as shown in Figure 5.9, which shows contours of pore pressure at the end of Stage 2a (undrained berm installation). The pore water pressures generated by berm construction dissipated completely as the analysis was stepped in consolidation time for 110 days ahead of the installation of the pile (Stage 2b). The x-direction displacement down the pile centreline (Figure 5.10) shows that the rockfill displaced about 2.0mm laterally over the Weald Clay immediately after construction of the berm, and continued to move further as the pore water pressures generated by berm construction dissipated. However, at the end of the 110 day consolidation period lateral movement of the rockfill had stopped (Figure 5.10).

5.3.3 Stage 3 results – pile installation

Assigning soil properties to the pile elements prior to pile installation meant that some deformation of the pile elements took place before they were given concrete properties. The result was a pile that was initially slightly bent in shape, particularly at the interface between the Weald Clay and the rockfill berm. In order that only the displacement of the pile elements after installation of the pile were recorded, the nodal displacements throughout the mesh were set to zero at the beginning of Stage 3.

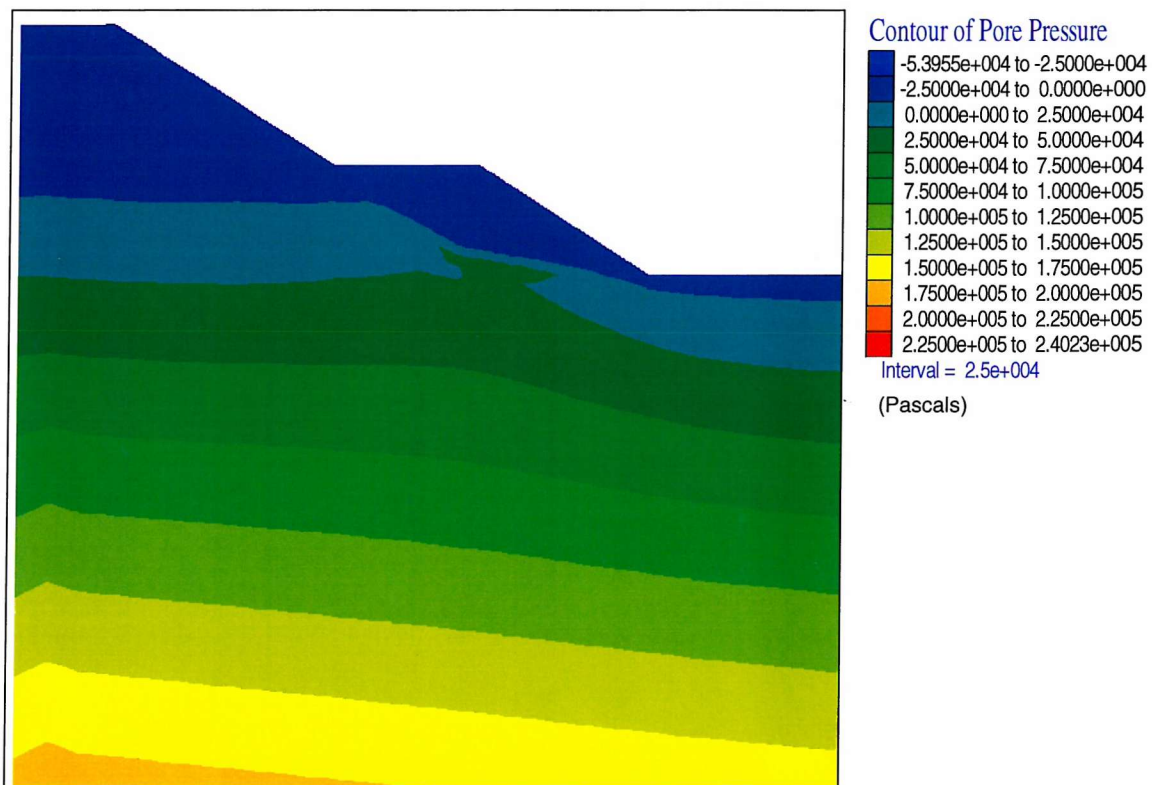


Figure 5.9. Contours of pore pressure, at the end of Stage 2 (Analysis H2). The contours are plotted for the front surface of the model, i.e. for the section through the pile.

The analysis was then run in consolidation mode for a further 30 days. The pile bending moments at the end of Stage 3 are shown in Figure 5.11. As the berm was essentially stable (by the end of stage 2b it had stopped moving), the bending moments acting in the pile were extremely small. This is not unexpected, as the embankment slope was initially just stable and the addition of the rockfill berm should have acted to increase the stability of the slope.

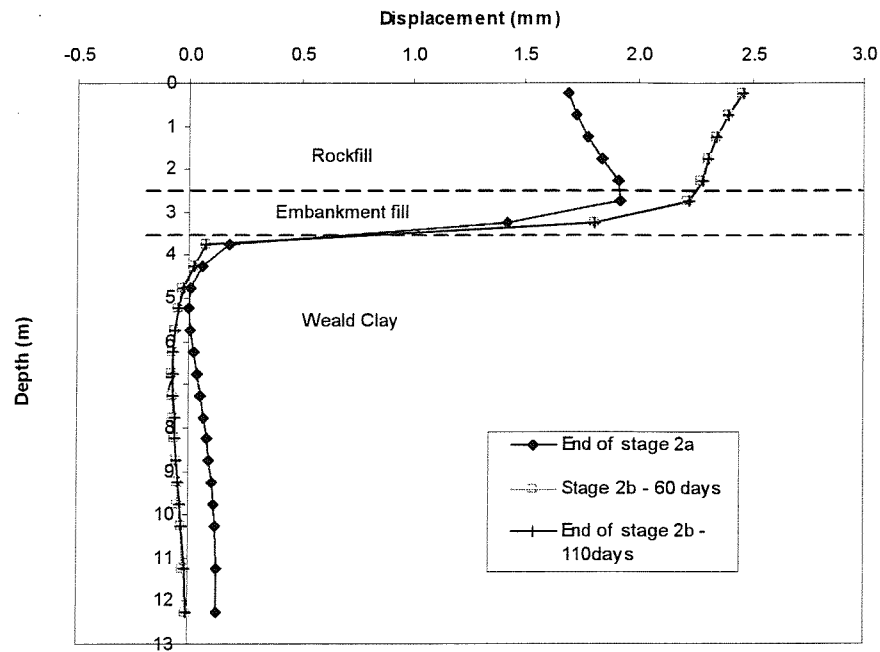


Figure 5.10. *x*-displacements at the pile centreline location measured from the beginning of stage 2a. The end of Stage 2a is after instantaneous berm construction, the end of Stage 2b after 110 days consolidation time. Positive displacement is downslope.

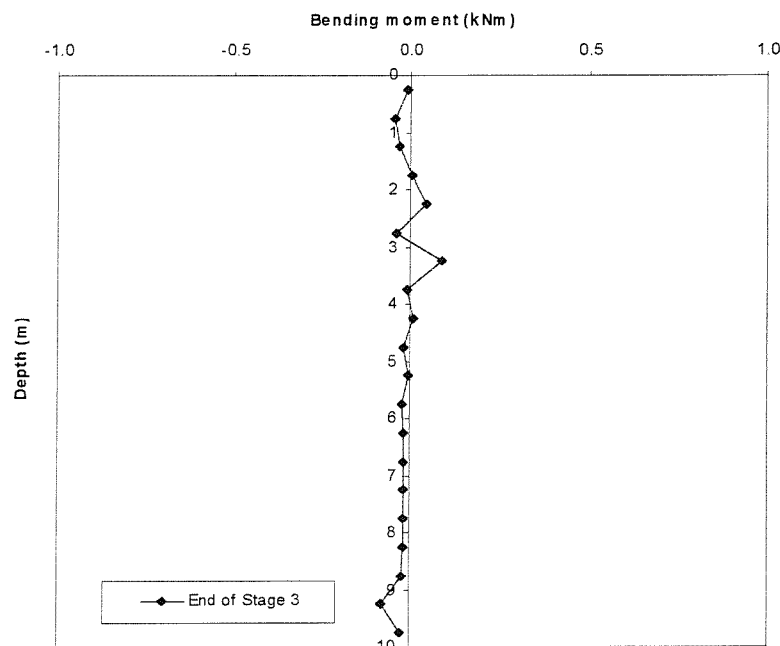


Figure 5.11. Bending moments acting in the pile at the end of Stage 3 (1 month after pile installation). Positive bending moment is the pile bending in a downslope direction.

5.3.4 Stage 4 results - re-grading of the embankment

The regrading of the embankment saw the addition of substantial amounts of rockfill placed upslope of the piles, and this was modelled by the addition of elements on the upper slope of the embankment. Introduction of the rockfill into the analysis initially generated an increase in pore pressures in the embankment. Analysis stage 4a was run using the FLAC3D mechanical solution scheme to generate the instantaneous (undrained) displacements and pore water pressures as a result of the addition of the rockfill upslope of the piles. Analysis stage 4b was run using the consolidation scheme to allow dissipation of the generated pore water pressures.

The bending moments calculated in the pile at the end of stage 4a and at various points during stage 4b are shown in Figure 5.12. The maximum pile bending moment is 30kNm, at about 2.5m depth. The bending moment diagram is of broadly the same shape as the bending moment measured in the real piles (Figure 5.15). The increase in pile bending moment of about 10-30 kNm as a result of the regrading of the rockfill is similar to but in some cases smaller than the increases measured in the real piles on day 22 and 23, when the rockfill on the real slope was regraded (see Figures 3.15-3.20). The dissipation of the pore water pressures generated as a result of the additional rockfill placement caused a slight reduction in bending moment with time (Figure 5.12). Note that the bending moments presented here may be slightly reduced due to the use of slightly too few elements in the pile (as discussed in section 4.2.4).

The calculated displacement of the pile is shown in Figure 5.13, together with the displacement of the soil midway between the piles. The maximum displacement of the pile at the end of stage 4a was about 2.75mm, with the displacement of the soil between the piles slightly larger. The base of the pile moved about 1mm downslope. As the pore water pressures generated by the addition of the rockfill were allowed to dissipate, the pile moved laterally back towards its position at the end of Stage 3, although its bent shape changed relatively little, as evidenced by the only small change in bending moment that occurred with time (Figure 5.12). As the pore water pressures dissipated, consolidation occurred in the embankment fill material with a downward soil displacement which seems to have dragged the pile back into the slope (this is shown in Figure 5.14, which plots the nodal velocities at the end of day 20 of stage 4b).

Figures 5.15 and 5.16 plot bending moment and displacement at the end of stage 4b against the measured bending moment and displacements in the real piles A, B and C. The load seen to develop on the real piles both before and after the rockfill regrading was not reproduced in the FLAC analyses because the embankment was not sufficiently unstable (no increase in pore pressures have been modelled in the FLAC analysis).

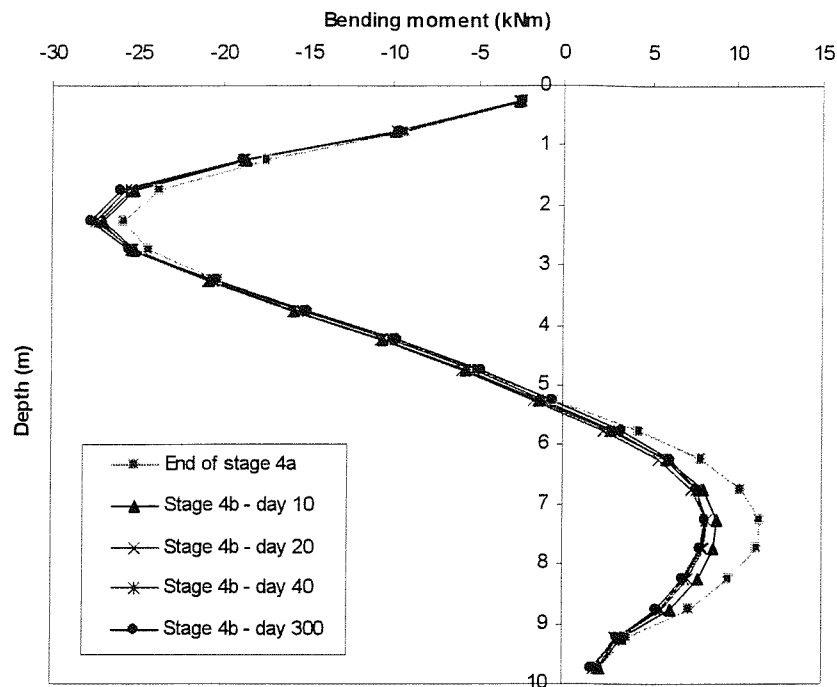


Figure 5.12. Bending moments acting in the pile at the end of Stage 4a and at various consolidation times through Stage 4b. Positive bending moment is the pile bending in a downslope direction.

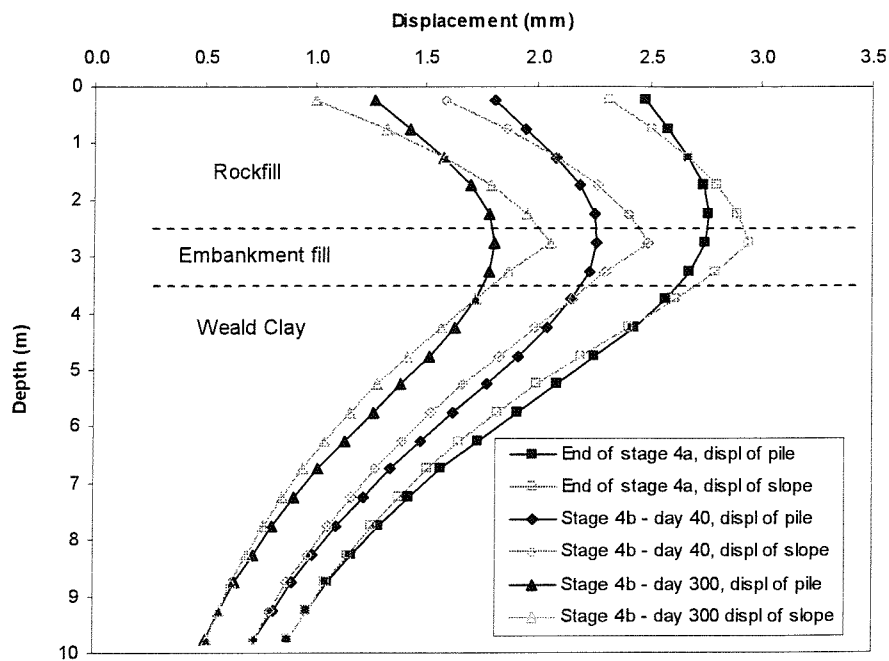


Figure 5.13. Displacement of the pile at the end of Stage 4a and at various consolidation times through Stage 4b. Positive displacement is downslope.

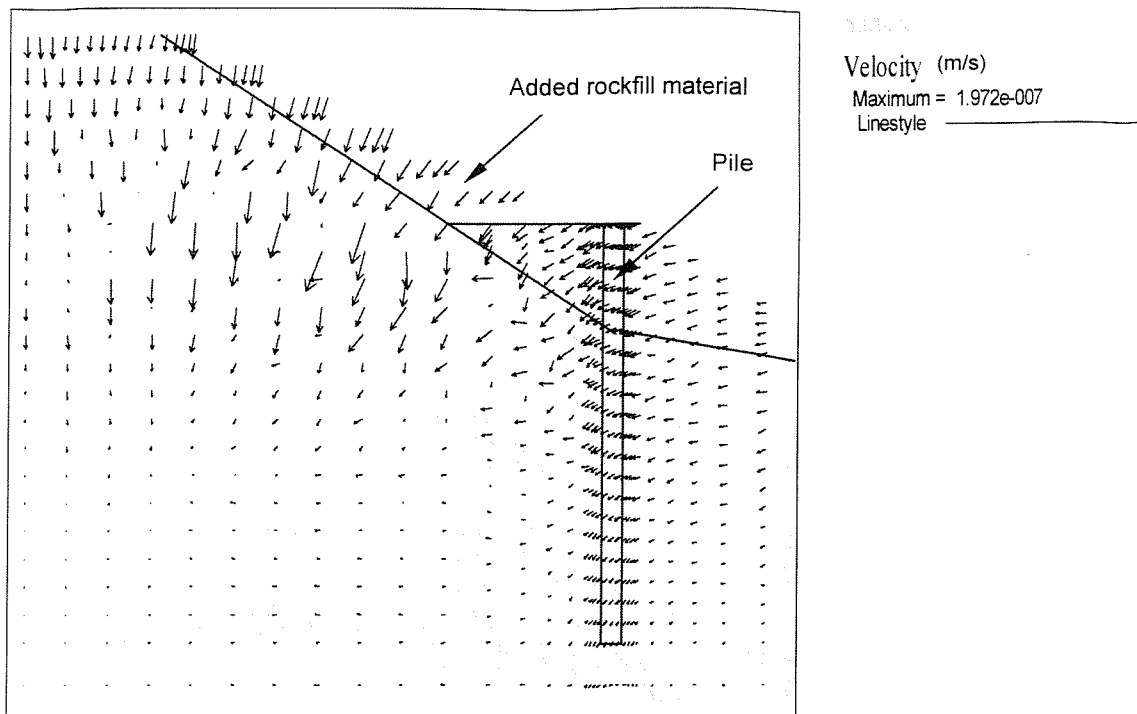


Figure 5.14. Vectors of nodal velocity for the upper section of the embankment and for Stage 4b, day 20. The vectors are plotted for the front surface of the model, i.e. for the section through the pile.

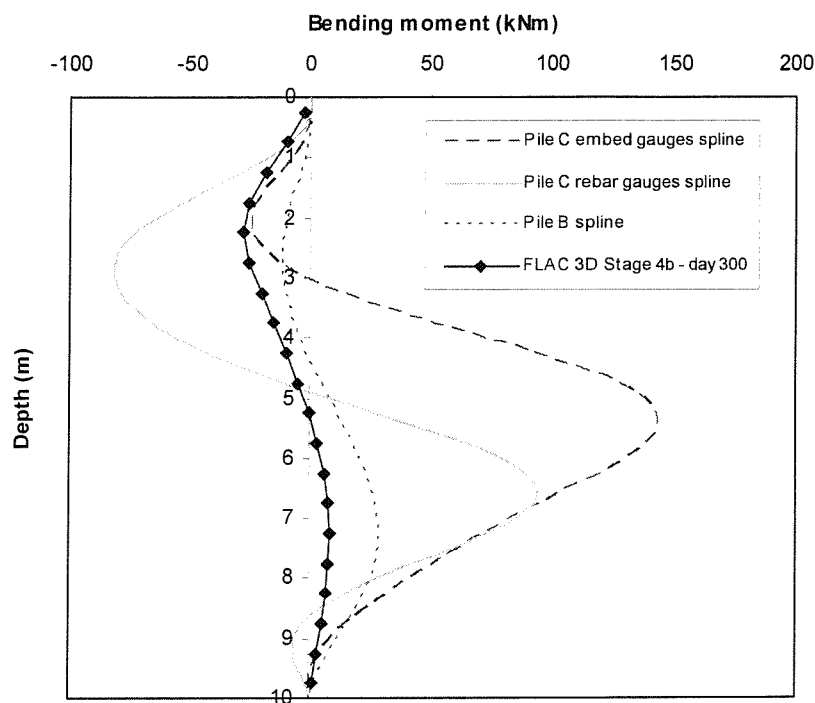


Figure 5.15. Bending moments acting in the pile at the end of Stage 4b plotted with the measured pile bending moments (the splines fitted to the measured bending moment data are shown). Positive bending moment is the pile bending in a downslope direction.

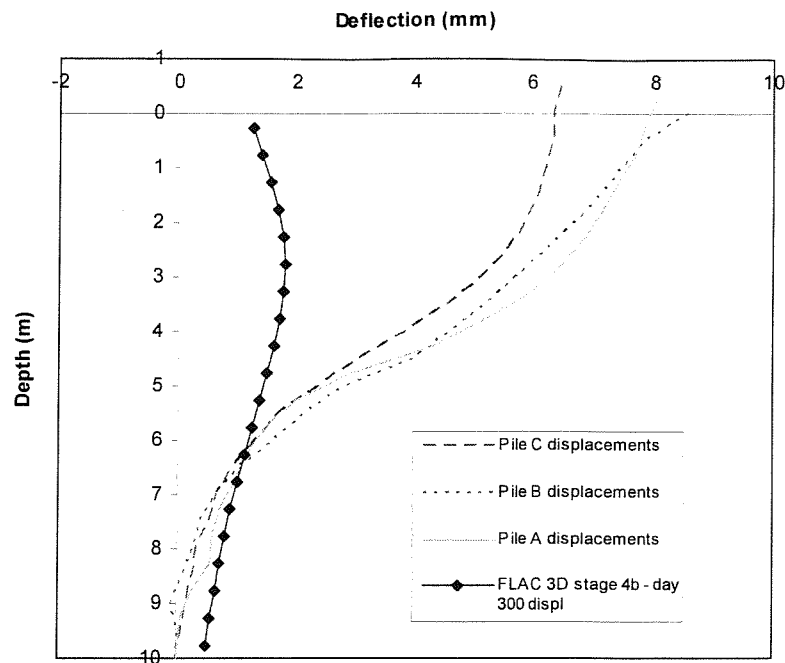


Figure 5.16. Displacement of the pile at the end of Stage 4b plotted with the measured pile displacement. Positive displacement is downslope.

The x-direction total soil stresses acting in the mesh elements immediately upslope and downslope of the pile are shown in Figure 5.17, with the difference or net stress loading to the pile shown in Figure 5.18. The net total stresses were largest at the zone around the interface between the rockfill and embankment fill materials. This indicates that the addition of the rockfill upslope of the piles caused the rockfill berm to continue to move relative to the clay slope. The stresses obtained from the mesh to plot Figure 5.17 and 5.18 are from the elements in the section of the model that runs through the centre of the pile, and that these may be larger than the overall lateral stress acting on the pile as a whole (as the elements in the pile centre section are most constrained from moving around and between the piles).

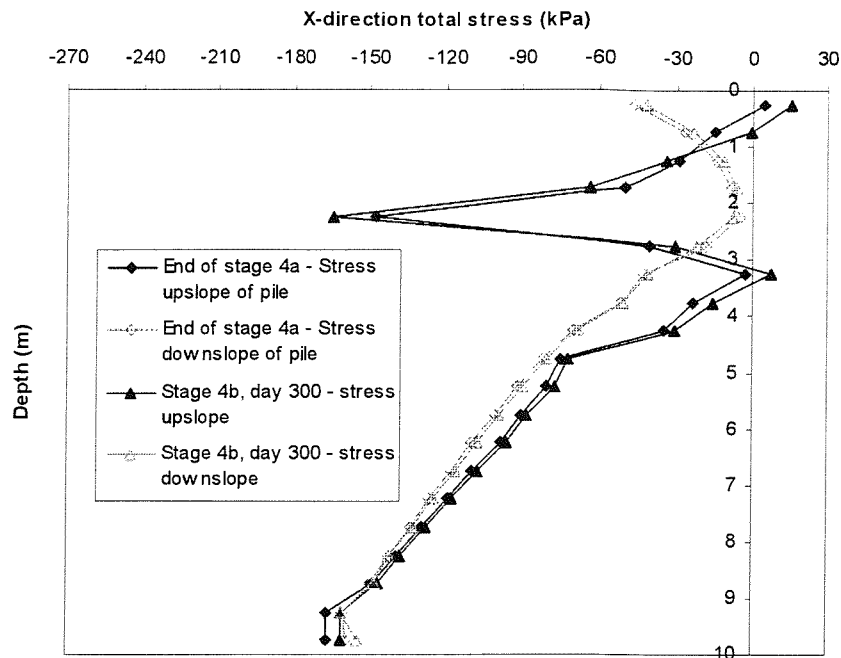


Figure 5.17. The lateral stress acting in the soil elements directly upslope and downslope of the pile at the end of stage 4b (where compressive stresses are negative).

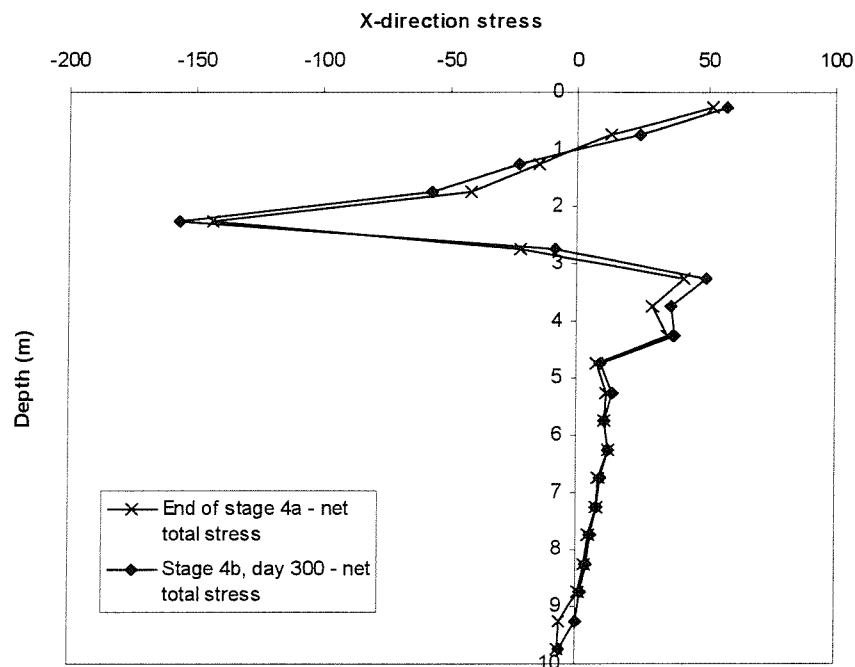


Figure 5.18. Net lateral stress acting on the pile at the end of Stage 4b as a result of slope movement (where negative stresses act downslope).

5.3.5. Analysis of the slope inclinometer construction.

In order to construct the slope inclinometers, the rockfill between the piles was excavated and replaced with Type 1 material. There was some uncertainty concerning the effect of this process on the pile bending moments. An additional analysis was run to assess the impact on the pile of the removal of the rockfill and its replacement with Type 1 fill (Analysis H3). Four days after pile installation, the analysis modelled the excavation of the rockfill by giving the berm

elements between the piles null properties (Figure 5.19). After the analysis had been run to achieve mechanical equilibrium, these elements were then replaced to model the filling of the hole with Type 1 fill material. As the Type 1 material used to infill the hole may have been less stiff than the rockfill, the replacement elements were given a reduced stiffness of 30MPa. Stages 3 and 4 of the analysis were then run in exactly the same way as Analysis H2. The hole is not exactly the same shape as the hole made in the real embankment, when much less of the pile sides were exposed, but the shape of the hole in the model was constrained by the shape of the elements in the section of the mesh surrounding the pile. The hole in the model remained stable despite having vertical sides, almost certainly as a result of the limited (about 1.4m diameter) size of the excavation.

The pile bending moments from Analyses H2 and H3 at the end of Stage 4a are shown in Figure 5.20. Figure 5.20 shows some difference in the bending moment, with a small reduction in bending moments in the lower half of the pile for analysis H3. The excavation of material caused the clay beneath the hole to heave slightly, and this put the middle section of the pile into tension. Addition of the Type 1 material back into the analysis reduced the pile tension, but the process seems to have caused the failure of some additional elements surrounding the pile, which may explain the lower bending moment over the bottom 5m of the pile. Overall, the immediate impact on the pile behaviour as a result of the slope inclinometer installation is small.

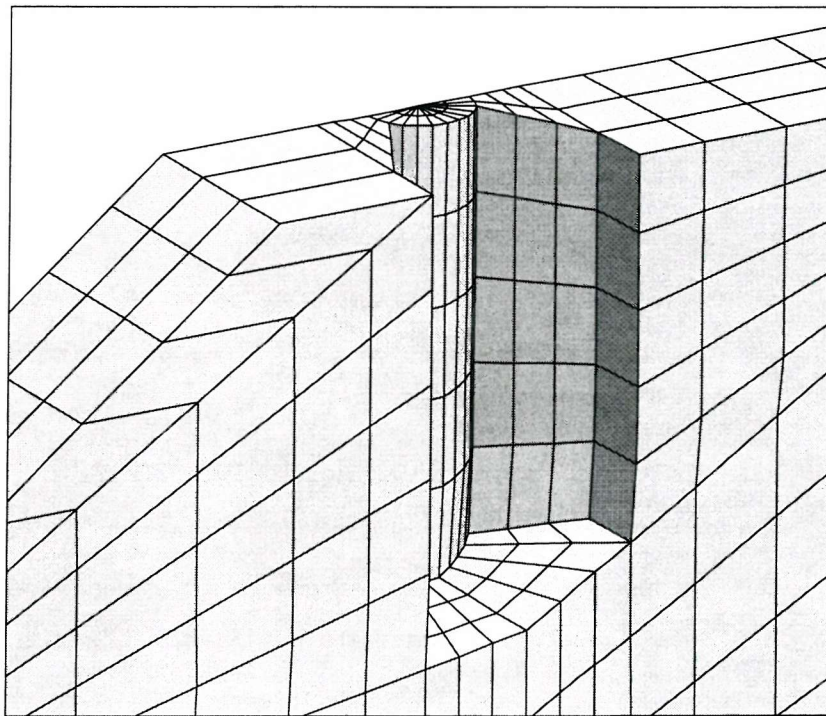


Figure 5.19. Excavation of a hole within the mesh to model the construction of the slope inclinometer tubes.

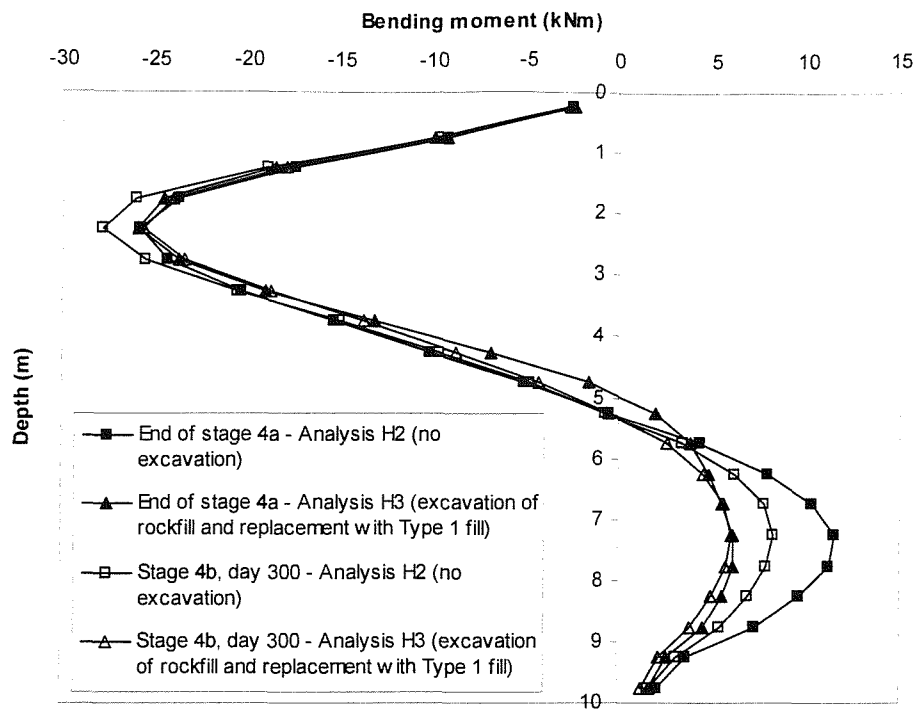


Figure 5.20. Pile bending moments at the end of Stage 4a and 300 days into Stage 4b, for analyses H2 and H3. Positive bending moment corresponds to the pile bending in a downslope direction.

5.3.6. Discussion of analyses H2 and H3

The results of analyses H2 and H3 show that the pile is loaded by the process of embankment regrading. However, the load seen to develop on the real piles both before and after the rockfill regrading was not reproduced in the analyses because the embankment was clearly not sufficiently unstable during these stages. In order to model the bending moments generated as a result of slope instability, changes would need to have been made to the pore water pressure condition in the embankment. In the absence of any real pore water pressure data, such changes would have been difficult to incorporate into the model with any confidence.

The lateral pile loading calculated in the analyses as a result of the rockfill regrading shows the development of load on the pile as a result of movement between the rockfill/embankment fill boundary, at 2.5m depth, and not movement at 4m depth as measured in the real slope. The real slope is possibly moving on a relic shear surface within the embankment, which would be difficult to incorporate within the analysis without attempting to model the embankment construction and strain-softening of the clay materials.

However despite these limitations, the analyses have clearly shown the influence that the construction events around the pile have on the pile bending. The analyses show that the effects of inclinometer installation on pile bending moments are small, but the effects of the rockfill regrading on the pile bending moments are significant. They also show that part of the bending

moment measured in the real piles (in particular Pile C) was probably caused by slope movement due to increases in pore pressure in the embankment slope after pile construction.

5.3.7. The use of the finite difference model in design

The FLAC3D model was used to assess pile behaviour under ultimate slope loading, with a strength reduction factor of 1.3 applied to the soil strength.

The factor of safety applied to the soil strength in the finite difference analysis is the same as the ratio of restoring to driving moments used to give the factor of safety in limit equilibrium slope stability (Griffiths and Lane, 1999). The derivation of the equations for limit equilibrium slope stability such as those given by Bishop (1955) and Janbu (1973) require the factor of safety to be applied to the soil strength, $\tan\phi$. Consequently, it is possible to compare the results from FLAC3D analyses in which the soil strength is reduced by a factor of safety of 1.3 (modelling the ultimate state) with the results from the elastic analyses in which the restoring force was added to the limit equilibrium slope stability analysis to give a factor of safety for the slope of 1.3.

After stage 4b of analysis H2 had finished, a factor of safety of 1.3 was applied to the soil strength. The analysis was first run in mechanical only mode, and then in the consolidation mode for a period of 2 years to allow pore water pressures generated by further slope movement to dissipate.

The bending and displacement results from the analysis are shown in Figure 5.21. The bending moments calculated from the FLAC3D model are probably slightly smaller than those actually measured in the pile, despite the factor of safety of 1.3 applied to the soil strengths. At 9mm the soil displacement is only slightly larger than measured in the real piles. Figure 5.21(b) shows that the slope and pile displaced by about 1mm at the pile base, and that for the lower 6m of the pile the soil displacement was almost identical to the pile displacement. The FLAC3D net lateral pile pressure distribution for the reduced strength slope is shown in Figure 5.22. The net downslope pressure seems to only act over the top 2.75m of the pile, indicative of the slope having been loaded through movement of the rockfill over the clay slope surface. However, study of the accumulated element shear strain and nodal displacements for this stage of the analysis (Figure 5.23) shows a band of increased shear strain running through the embankment well below the interface between the rockfill and the embankment fill, and which appears to intersect the pile at about 4m depth (although the pile acted to reduce the shear strain through the adjacent section of the slope).

It is clear that the bending moments and pile displacements calculated from the model in which a factor of 1.3 has been applied to the soil strengths seem small when compared with the real pile bending moments and displacements. This may be a result of the finite difference analysis being too simple. The rainfall in spring 2001 may have pushed the real slope closer to failure than modelled in FLAC3D. The sharp increase in displacements measured in the real pile at 4m depth is possibly indicative of movement on a relic shear surface that was also not modelled in the FLAC3D analysis, and it is also likely that the soil stiffness and *in situ* stress state are not correctly modelled.

Figure 5.24(a) compares the reduced strength FLAC3D bending moment with the bending moment calculated from an ALP analysis carried out using identical soil stiffness properties and a displacement that gives a restoring force to the slope of 144kN, which is the magnitude of the point force required in the limit equilibrium slope stability analysis to give a factor of safety of 1.3 for the embankment slope. The ALP bending moment is somewhat larger than the FLAC3D bending moment.

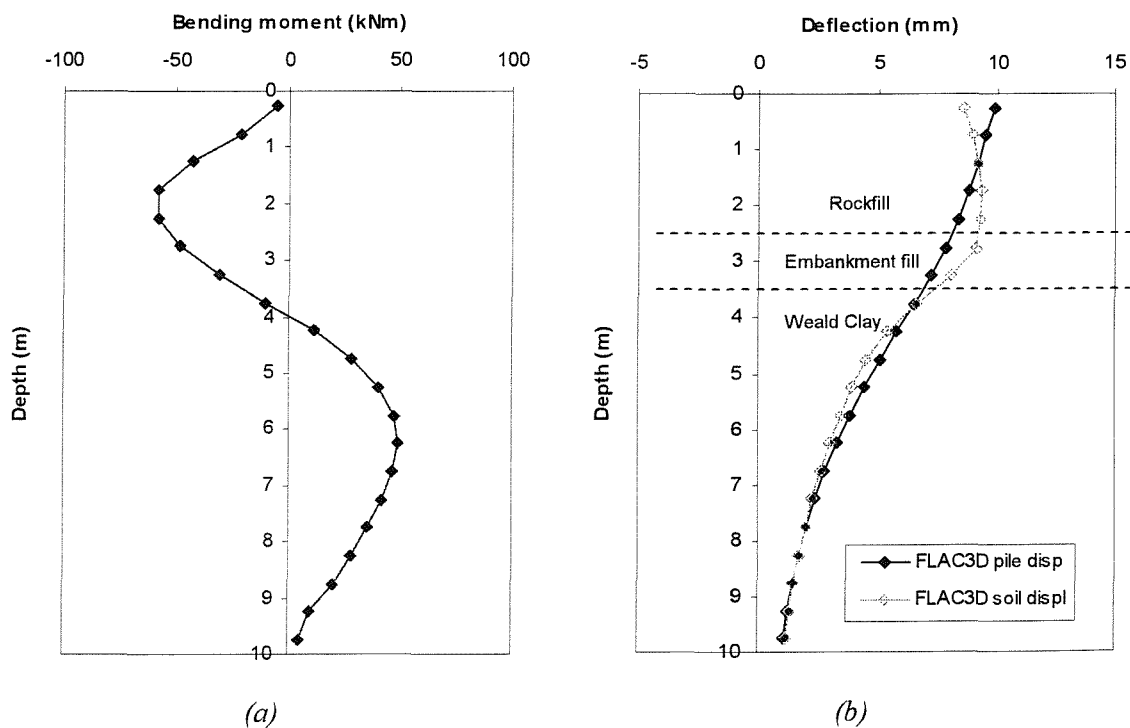


Figure 5.21. Results from the FLAC3D analysis with a factor of safety of 1.3 applied to the soil strength: (a) the bending moments calculated in the pile (where positive bending moment is the pile bending in a downslope direction); and (b) the lateral pile and soil displacements (where positive displacement is downslope). The soil displacement in the FLAC3D analysis is the displacement of the slope midway between adjacent piles.

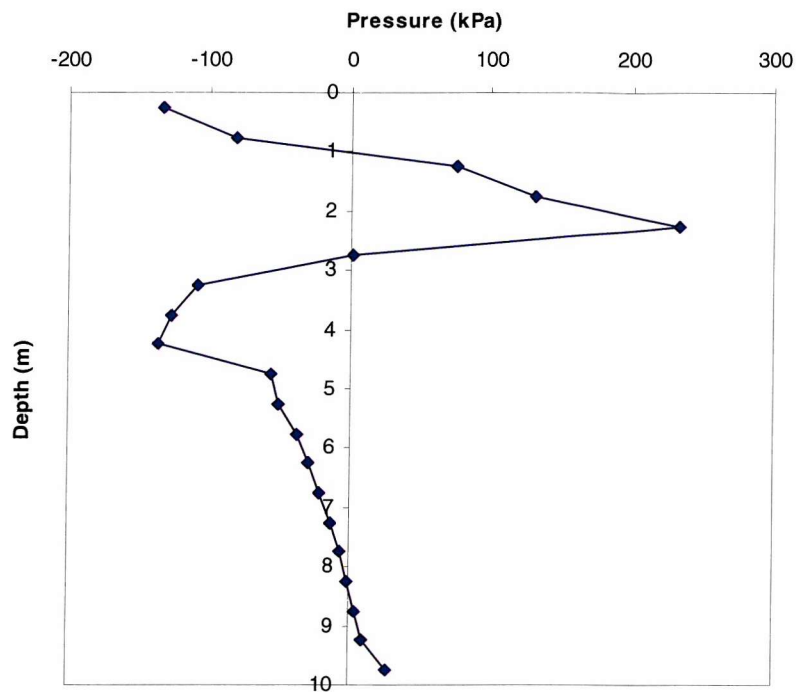


Figure 5.22. The net lateral earth pressure acting on the pile in the FLAC3D analysis with a factor of safety of 1.3 applied to the soil strength. Positive pressures are downslope pile loading.

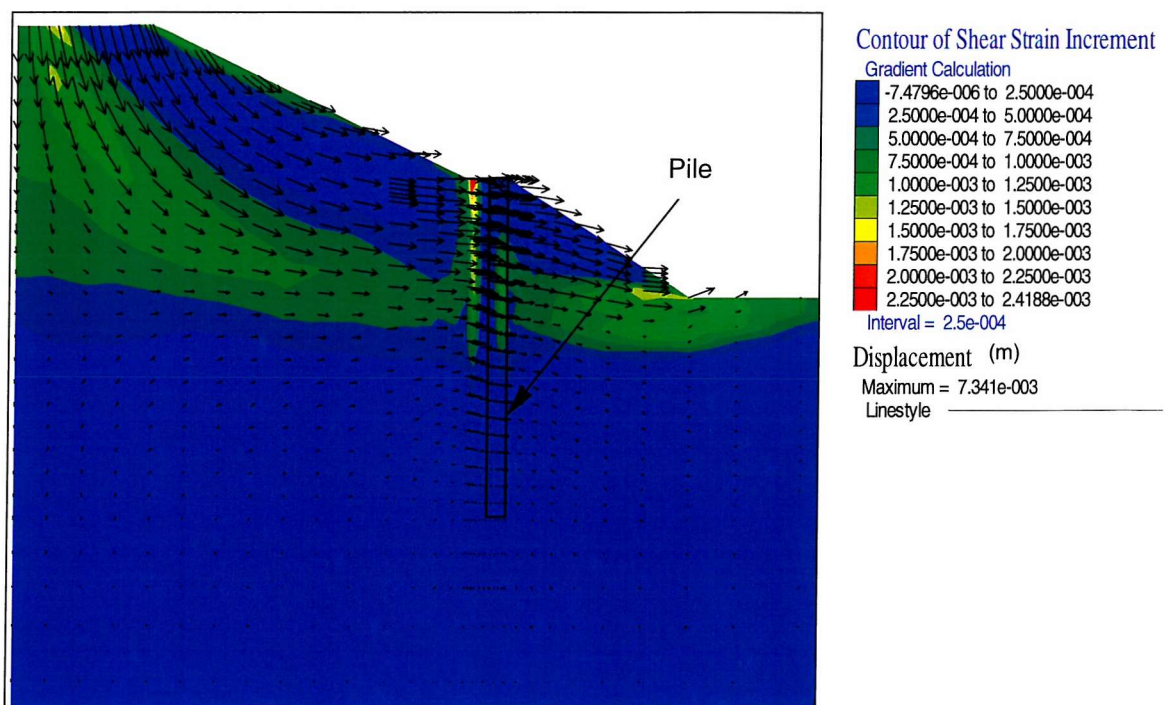


Figure 5.23. Plot of accumulated element shear strain and vectors of nodal displacement in the model after the soil strengths have been reduced by a factor of safety of 1.3. The contours and vectors are plotted for the front surface of the model, or the section through the pile.

Figure 5.24(b) shows that the slope and pile displacements were smaller in the FLAC3D analysis than in the ALP analysis. The pile toe was able to displace laterally within the FLAC3D analysis, which was able to model the displacement of the entire slope. The distributions of net lateral soil pressure found to be acting on the pile in the ALP and FLAC3D analyses are shown in Figure 5.25. The FLAC3D pressures were slightly smaller than those

calculated in the ALP analysis. The lateral soil pressures were calculated from the FLAC3D element stresses in the section of the model that runs through the centre of the pile, and as discussed earlier, these are likely to be larger than the overall lateral stress acting on the pile as a whole. The downslope loading in the FLAC3D model occurred only over the top 3m of the pile, rather than the 4m over which displacements were applied to the ALP analysis (where the 4m depth was obtained from the critical failure surface in the limit equilibrium slope stability analysis).

The differences in pile bending between ALP and FLAC3D are possibly the result of differences between the stability of the slope as modelled in the FLAC3D analysis and the limit equilibrium slope stability analysis. However, it must be remembered that there are limitations in the ALP/limit equilibrium slope stability analysis method, which in this case assumed the displacements to be zero below 4m depth, and assumed the failure surface (on which the depth of soil movement was based) to be circular, which may not be the case in reality. The FLAC3D analysis models more accurately the overall slope displacements (which extend the full depth of the pile) and also model the *in situ* stress state of the soil.

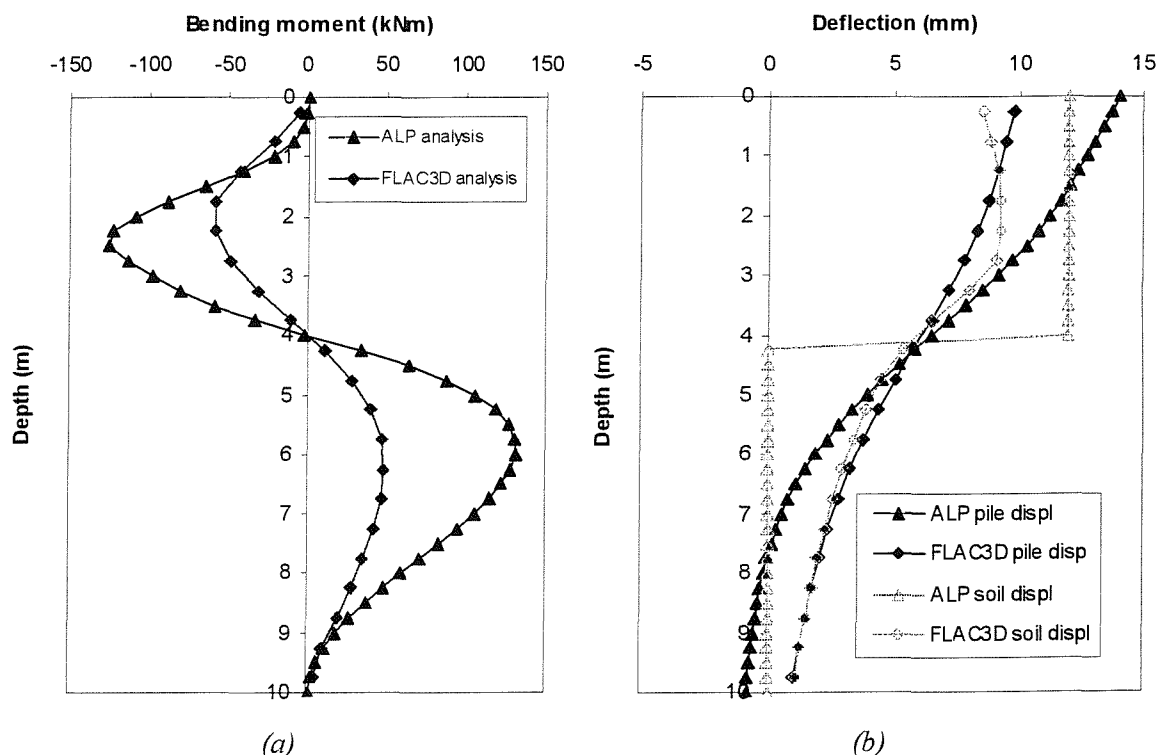


Figure 5.24. Comparison between the ALP analysis and FLAC3D analysis with a factor of safety of 1.3 applied: (a) the bending moments calculated in the pile; and (b) the lateral pile and soil displacements. The soil displacement in the FLAC3D analysis the displacement of the slope midway between piles.

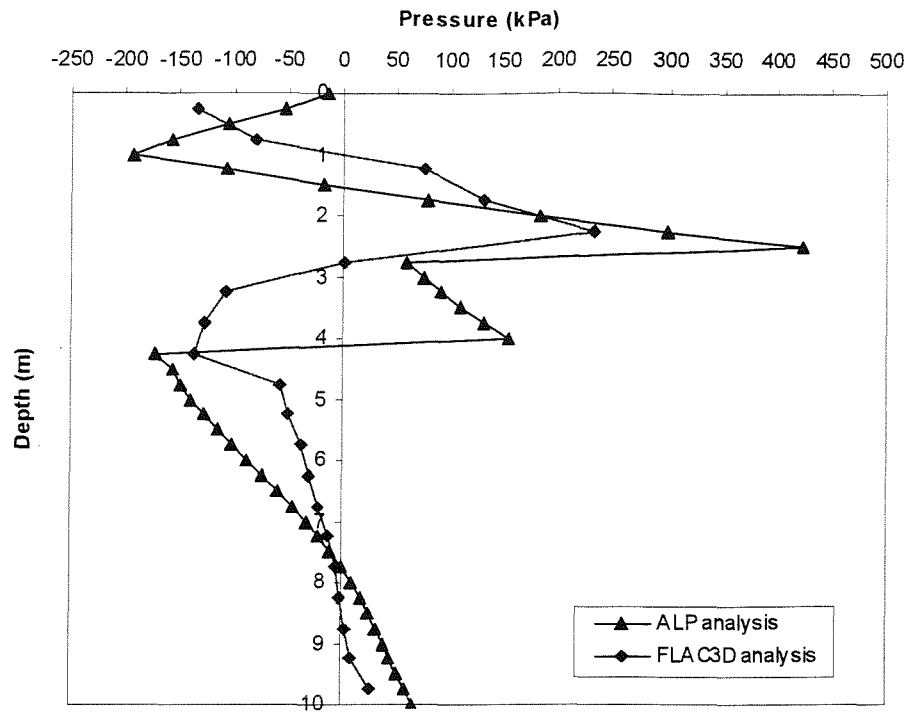


Figure 5.25. Comparison between the lateral earth pressure acting on the pile in the ALP analysis and FLAC3D analysis with a factor of safety of 1.3 applied. Positive pressures are downslope pile loading.

5.4 Summary of FLAC3D analyses

A FLAC3D model was used to investigate the effect on the pile of slope surface construction processes and to consider the pile behaviour under ultimate slope conditions. The major findings were as follows:

- i. The installation of additional rockfill upslope of the berm caused the pile to carry load, with a maximum calculated bending moment of about 30kNm. This compared reasonably well with the measured increase in bending moment on days 22 and 23, when the real slope was regraded at the instrumented section.
- ii. The bending moments calculated by the FLAC 3D model were significantly smaller than the bending moments measured in the real piles. The initial embankment slope was assumed to be stable in the FLAC3D model, and the pore pressure boundaries remained unchanged from this condition throughout the analysis, as there was no data on the real pore water pressures in the embankment or their change with time. Consequently, the bending moments calculated in the model resulted only from regrading of the rockfill on the slope surface. This demonstrated that some of the measured pile loading must have resulted from genuine instability of the embankment slope.

- iii. The excavation of holes in the rockfill for installation of the slope inclinometers was found to have little effect on the bending behaviour of the pile.
- iv. The clay permeability used in the analyses was found to give fairly rapid dissipation of the pore water pressures generated by surface construction changes. The pore pressures generated by berm construction dissipated before the piles were installed.
- v. The strength reduction method was used to apply a factor of safety of 1.3 to the soil strengths in the model in order to assess the increase in load that this produced on the pile. The calculated bending moments were slightly smaller than those measured in the field. This may be a result of the slope instability not being correctly modelled in FLAC3D (see i. above), or due to differences in stiffness and *in situ* stress between the model and the real slope. When the results from the FLAC3D analysis are compared with an equivalent analysis run in ALP, the bending moment, pile displacements and pressures are all larger in ALP. However, the ALP model is probably a simplification of real pile and slope behaviour.
- vi. To run the full set of analysis stages (which were completed in analyses H2 and H3) took about 1 week. The long run time in part reflects the small consolidation time step for each groundwater calculation carried out in the model, which is a limitation of the FLAC3D program. The long run time and complexity of 3D finite difference/element models makes their use in pile design difficult.

Chapter 6. Conclusions

6.1 Introduction

This research has investigated the existing case studies and design methods for pile stabilised slopes. The research has generated a case record of the performance of discrete piles used to stabilise a Weald Clay railway embankment at Hildenborough, Kent. The field data have been used to investigate suitable geotechnical parameters and models for use in the design of discrete piles used to stabilise infrastructure slopes. The major results and findings are summarised in this chapter, and recommendations for further work are suggested.

6.2 Objectives

The objectives of the research were as follows:

- i. To review the existing case studies and design methods for pile stabilised slopes presented in the literature.
- ii. To present the results from the instrumentation installed to monitor the behaviour of discrete piles installed to stabilise a real embankment slope.
- iii. To back analyse the measured behaviour of the embankment using elasticity and finite difference analyses.
- iv. To consider the implications for discrete pile design of the results from the instrumented study.

6.3 Review of case studies and existing design methods

Methods of design including limit equilibrium, pseudo finite element analysis and 3D finite element analysis were reviewed. Limit equilibrium slope stability was thought to be the best means of calculating the loading on the piles from the slope, as it takes into account the equilibrium of the slope. Some of the other design methods for calculating the pile restoring force were found to have substantial limitations. Evidence from existing studies showed that given the measured slope displacements as input, elasticity based analyses enabled a good calculation of the measured pile bending behaviour. Studies using three-dimensional finite element analysis demonstrated the potential of such analyses to model accurately both the equilibrium of the slope and pile, although none of the studies attempted to back analyse an existing case study.

Physical and numerical studies investigating pile spacing and soil arching between piles were reviewed. The studies indicated that soil arching in slopes occurred by means of a semi-circular shaped major principal stress arch upslope of the piles. The studies showed that the effectiveness of the arching and the maximum pile spacing achievable is a complex soil structure interaction problem. In practice, it is probable that piles can be safely placed at least 4 pile diameters apart. However, the numerical studies demonstrated that arching effectiveness is likely to be part dependant on the physical distance between the piles, and not just on the dimensionless pile spacing.

A number of instrumented pile studies were reviewed, and gave an insight into pile bending behaviour in a range of types of geology and mass movement. All of the case studies had been designed with piles acting in 'long' or 'intermediate' mode, and the maximum bending moment always occurred beneath the failure surface. It was not unusual for the pile head to be restrained within the failing mass, causing the head of the pile to bend back upslope. In many of the studies the pile loading was found to develop over several years.

6.4 Field monitoring

Records of bending moment and displacement within three piles installed to stabilise a rail embankment slope at Hildenborough, Kent, were obtained. These showed that there was an increase in the bending moment and displacements of the piles on regrading of the rockfill on the embankment surface into a two stage slope. Some additional pile bending occurred as a result of slope movement caused by rainfall in the spring of 2001. After the first 200 days, there was little change in the pile bending moments and displacements.

The bending moments and pile inclinometer displacements show that the lower half of the piles bent downslope, with the pile toe reasonably well fixed in the Weald Clay underlying the

embankment. The upper half of the pile bent back upslope, as it was restrained from bending uniformly downslope by the stiffness of the moving mass (possibly aided by the granular rockfill placed on the slope surface). This pile bending is contrary to the limit equilibrium pile design, which assumed that the pile should bend uniformly downslope.

Installation of duplicate sets of embedment and rebar type strain gauges in the same pile indicated difficulties in the reliable measurement of bending strain, particularly when concrete shrinkage processes are occurring.

The real pile behaviour is quite varied, and the measured bending moments are still significantly smaller than the pile bending capacity. The piles will only act to stabilise the slope if its factor of safety drops below 1.0. The slope may not yet have been sufficiently close to failure for significant slope (and therefore pile) movements to take place. It is possible that were the UK to receive a very wet winter (similar to 2000/2001 that caused many of the original embankment problems) the piles would carry more load.

6.5. Elastic analysis of the Hildenborough embankment

The program ALP was used to back-analyse the measured bending behaviour of the Hildenborough piles, with loading to the pile applied as a soil displacement. The analyses were used to find the stiffness of the soil surrounding the pile. Large stiffnesses were required for the rockfill and the intact Weald Clay to constrain the top and bottom of the pile and reproduce the real pile bending behaviour. The high stiffnesses for the rockfill may result from the inability of the ALP analysis to model the surface construction and compaction processes.

The elastic-plastic springs supporting the pile remained elastic, demonstrating that for piles used to stabilise infrastructure slopes, the ultimate lateral pile-soil limiting stress is much larger than the stresses likely to act on the pile. The most critical mode of failure is likely to be formation of a plastic hinge in the pile section rather than a flow of soil around the pile.

The ALP program was used to demonstrate that elastic analysis makes a better prediction of pile bending behaviour than a limit equilibrium calculation, in which it is necessary to make assumptions about the pile bending mechanism. If site specific soil stiffness data is not available, the pile behaviour can still be suitably predicted using soil stiffness data from the literature. However, assumptions do need to be made about the depth and profile of the applied displacements.

6.6. Finite difference analysis of the Hildenborough embankment

The finite difference analyses using FLAC3D modelled the full sequence of berm construction, pile installation and the regrading of the rockfill. In the absence of data showing the how the pore pressures changed with infiltration due to rainfall, it was found to be difficult to model the exact pile bending due to instability of the slope.

The analysis confirmed that pile bending occurred as a result of the regrading of the rockfill on the slope surface. Excavation of a hole in the rockfill for installation of the slope inclinometers had little effect on the pile. The whole pile was found to move laterally within the slope as a result of the addition of the rockfill upslope of the piles.

The strength reduction method was used to calculate the bending behaviour of the pile under the same factor of safety of 1.3 used in the limit equilibrium slope stability analyses. The calculated bending moments were smaller than those resulting from an equivalent ALP analysis.

The time taken to construct and run the FLAC 3D model illustrates the difficulty of using the finite difference/element method in design. In this case, there was a paucity of information on the slope pore pressures, soil stiffness, *in situ* stress, and permeability needed to construct the model.

6.7 Further work

Generally, when a research project is completed, it does not mean that the research into the problem investigated has been completed. Suggestions for further research can usually be made. On the basis of the research described in this thesis the following suggestions for further work might be made.

- The deterioration of geotechnical infrastructure assets such as the embankment at Hildenborough is a long term process. It would be desirable to continue to monitor the piles at Hildenborough, to see whether the piles gain load from any further slope movements due to continued deterioration of the embankment and wetter winters.
- There is a paucity of information on the behaviour of real pile stabilised slopes in a range of geologies and types of mass movement. Establishment of further field studies are required to measure the behaviour of real pile stabilised slopes. Further studies would benefit from the measurement of additional field parameters such as pore pressures (and suctions) and also weather parameters.

- Lab tests to measure concrete shrinkage strains within a typical pile section, and the effect that it has on bending moment measurement. In particular, assessment of different types of gauges should be carried out to see if certain types of gauge are less affected by concrete shrinkage.
- There is a requirement for more high quality published stiffness data for stiff clay soils that are prone to slope failure. Extensive work has already been carried out to investigate the properties of London Clay fill within old tipped rail embankments, but there is a need for further research into the strength and stiffness properties of old embankments made of other materials. There is also a requirement for a better understanding of the deterioration (such as that caused by cycles of wetting and drying) and failure mechanisms of embankment and cutting slopes, including the strain softening (post-peak) behaviour of high plasticity clays.
- There is still a need for a better understanding of fundamental pile stabilisation mechanisms, particularly soil arching, the placement of piles at large spacings and the effect that this has on the stabilised slope displacements. A parametric study run using 3D finite element analyses may help to answer some of these questions.
- Further work is required to demonstrate elastic analysis as an improved means of design over limit equilibrium calculations, and probably requires the back analysis of further instrumented pile studies. Further work should also be done to investigate the design of piles with the formation of plastic hinges.
- When discrete piles are used to stabilise railway embankment slopes, there is little understanding of how dynamic loads resulting from trains traversing the embankment impact on the piles.

Appendix 1

Calculation of concrete shrinkage strain

This appendix shows the calculation of concrete shrinkage strain for the Hildenborough pile section.

The magnitude and rate of concrete shrinkage typically differs within different parts of a foundation structure. The shrinkage will be significantly influenced by the atmospheric conditions, with the rate of concrete drying influenced by whether a foundation is buried in the ground or open to the atmosphere. A foundation that remains immersed in groundwater from casting will generally expand, with the expansion about one-quarter of concrete shrinkage (Murdock *et al*, 1991). However, on drying the concrete will shrink as before. Assuming that the groundwater level at Hildenborough is 1.5m below the intact Weald Clay on which the embankment is constructed, this gives 5m or one-half of the pile beneath the water table.

Murdock *et al* (1991) present an equation to calculate a rough assessment of the drying shrinkage (for unreinforced concrete):

$$S = \frac{5.07W - 400}{10000} \quad \text{Equation A1}$$

where S = the drying shrinkage, per cent reduction in length and W = mass water in kg per m³ of concrete. This formula does take account of the cement content of the concrete, because the water/cement ratio for concrete of medium workability remains pretty constant (at around 0.5). The formula is based on the shrinkage of small laboratory specimens, and significantly lower shrinkages are likely to occur in practice (Murdock *et al*, 1991).

Other factors that influence concrete shrinkage include the amount of reinforcement, the type of cement, the aggregate mineral type, the size and shape of the specimen and the surrounding atmospheric conditions. There are refinements to Equation A1 to take into account some of the above factors, but in practice the large number of variables affecting concrete shrinkage mean that accurate calculations are difficult (Murdock *et al*, 1991).

In normal reinforced concrete, drying shrinkage results in a compressive stress in the reinforcement bars, which act to resist the overall drying shrinkage. It is generally assumed that the drying shrinkage of reinforced concrete is half that of unreinforced concrete, although the exact shrinkage is dependant on the steel to concrete ratio. BS8110 (British Standards

Institution, 1985) advises that the proportion of reinforcement may be taken into account by multiplying the unreinforced concrete shrinkage by the equation:

$$\frac{1}{1 + \chi\rho}$$

Equation A2

where ρ is the area of steel relative to that of concrete (through the section of the pile), and χ is a coefficient: 25 for internal exposure and 15 for external exposure.

The properties of the concrete mix used in the piles at Hildenborough are given in Table A1. In addition to cement, aggregate and water, the concrete also contained a plasticizer that allowed the concrete to be pumped through the piling rig. Each pile contained 6×T25 reinforcement bars over its full length, and as explained in Section 3.2.2, a further 6×T32 bars over the bottom 7m.

Using Equation A1 and the value of 110kg as the mass of water per m³ of concrete, the unreinforced concrete shrinkage for the Hildenborough pile is calculated as 0.015% strain or 150 microstrain. Table A2 shows the calculation of the impact the steel reinforcement has on the pile.

Constituent	Mass in kg (or volume in ml) per m ³ of concrete
Portland cement	219kg
GGBS (Ground granulated blast furnace slag)	146kg
Sand	730kg
20mm gravel	993kg
Water	110kg
Plasticizer HP/PLA	870ml

Table A1. Proportions of materials in C35 pile concrete mix used at Hildenborough.

Steel in pile section	Unreinforced concrete shrinkage strain (microstrain)	Area of steel/area of concrete, ρ (%)	Coefficient, χ	$\frac{1}{1 + \chi\rho}$	Reinforced concrete shrinkage strain (microstrain)
6×T25	150	1.05	15	0.86	129
6×T25 + 6×T32	150	2.83	15	0.70	105

Table A2. Calculation of reduced concrete shrinkage due to the presence of steel.

The maximum bending moment measured in the piles is about 200kNm. For the pairs of rebar strain gauges (that are 370mm apart), this bending moment is generated by a strain of 233 microstrain acting on each of the gauges in the pair (with one gauge in tension and the other in compression). For the concrete embedment gauges, that are closer together at 230mm apart, this bending moment is generated by a smaller strain of 145 microstrain. This means that potentially the concrete shrinkage strain is of almost the same magnitude as the strain required to generate the maximum measured pile bending moment in the pairs of embedment gauges.

Assuming that the concrete shrinkage strain is not uniform in both of the gauges making up a pair, such that the shrinkage strain is 5% less in one than the other, this will generate a miss-read of 4.5kNm from the concrete embedment gauges. This is a significant impact on the bending measurement considering that the bending moment profile on most of piles A, B and C remains less than 100kNm.

Rate of concrete shrinkage with time

The above shrinkage calculations represent the maximum concrete shrinkage – in practice it may take a long time for the concrete to dry and for this concrete shrinkage to occur.

The exact rate of drying shrinkage depends on the atmosphere around the concrete, and will occur much faster in hot dry conditions than in damp humid conditions. Drying shrinkage commences from the exposed surfaces of the concrete, and small blocks of concrete will subsequently dry much faster than large masses of concrete. In large mass foundations, concrete shrinkage may still be occurring after 10 years (Atherton, 1963).

Figures A1 and A2 show concrete shrinkage curves for relatively small laboratory specimens of concrete. In the first few hours of curing, the shrinkage is initially quite rapid, but after this, shrinkage continues to occur over a period of several months and years.

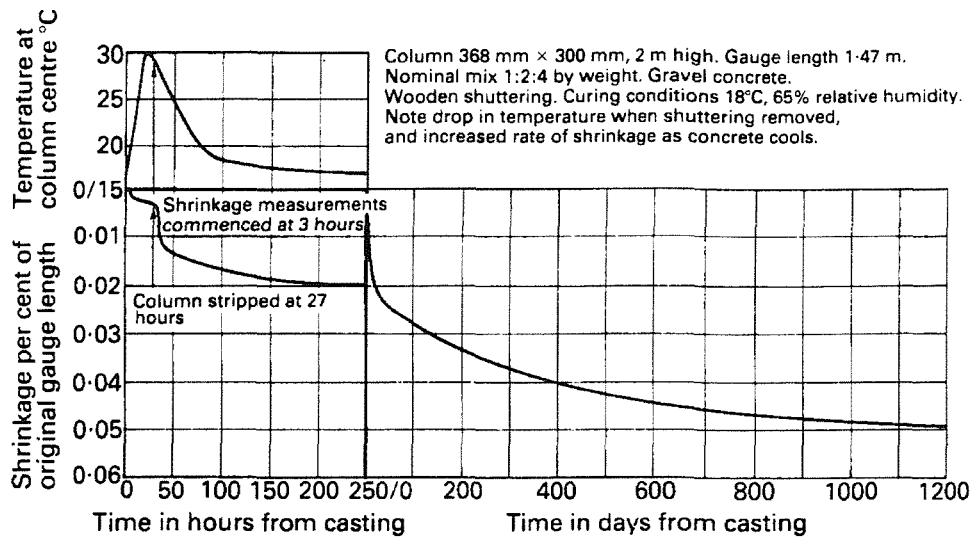


Figure A1. Concrete shrinkage in a concrete column 368mm x 300mm and 2m high (from Murdock et al, 1991). 0.06% shrinkage is equal to 600 microstrain.

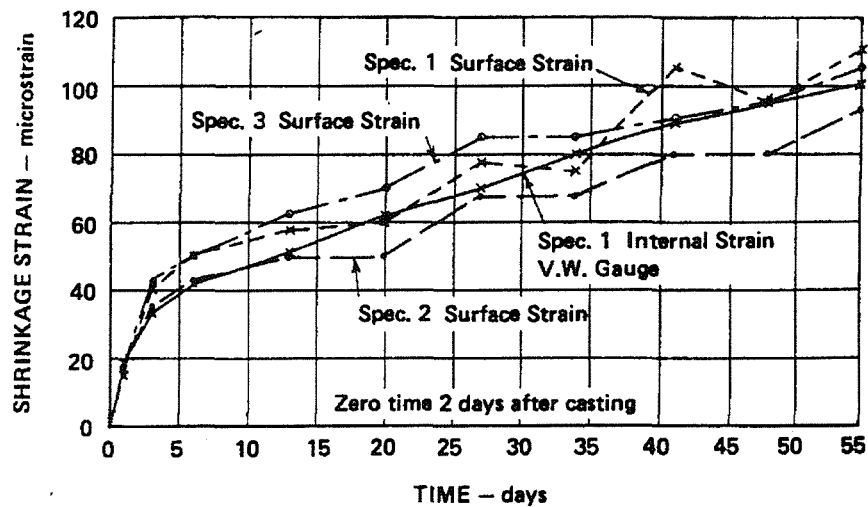


Figure A2. Concrete shrinkage in three concrete specimens 127mm x 127mm and 381mm long (from Bakoss et al, 1977).

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