Residual Shear Strength of Clays in Landslides in Southern Britain

Seyyed Mahdi Hosseyni

A Thesis submitted in partial fulfilment of the requirements of

Kingston University London

for the degree of Doctoral of Philosophy of Civil Engineering

School of Civil Engineering and Construction

YЯ	AXO	inesterevinversitzeand	KIN
		Acc. No.	SOA
		September 2012	·

Title: Residual shear strength of clays in landslides in southern Britain Seyyed Mahdi Hosseyni Kingston University 2012 Ethos persistent id: uk.bl.ethos.587376

Please do not digitise the following figures:

.

.

 Figure 1-2
 p.3

 Figure 7-5
 p.96

,

Abstract

Systematic back analyses of cross sections through landslipped slopes in Barton Clay have been carried out, based on the both published (cross sections) and unpublished (piezometer) data. The results of these support earlier (Barton, 1973) rather than later (Barton and Garvey, 2011) interpretations. Ring shear tests on clay samples from these landslides show broad agreement with the back analyses.

Further back analyses on landslide elements at Herne Bay in the London Clay throw additional light on the behaviour of landslides there. The remaining coastal landslide case histories in London Clay are reviewed.

The body of case records compiled by James (1970) for infrastructure (railway) cutting failures in London Clay is reviewed, with new back analyses. These show clearly the deficiencies in that set of analyses on which several important papers were based. Further reinterpretation and analysis goes some way to resolving questions arising from the review.

It is concluded that the back analysis technique is a useful one, and when applied correctly provides excellent general agreement with equally careful laboratory testing on appropriately selected samples.

A development in the back analysis technique for extracting the shear strength parameters for a weak bed forming the bedding-controlled basal shear of a compound landslide is presented and used.

The analyses and tests are supported by a review of published residual strength properties for British Clays.

İ

Acknowledgments

I would like to express my gratitude to the people and organizations without whom this thesis would not have been possible.

Firstly, to my director of studies, Professor Eddie Bromhead, for his never ending stream of ideas and his generous help, guidance and enormous support throughout my work on this thesis. Thanks also go to my other supervisor Dr Alan Dykes for his help and support.

To my friends Dr Nobuyuki Torii from Kobe City College of Technology, Japan and Dr Alireza Zolfaghari from Lloyd's for their encouragement and support.

At Kingston University, I have been helped enormously by Mr Chris Somerfield, Ms Jackie Deacon and Dr Akbar Aboutorabi and their technician, administrative and staff colleagues.

I would also like to acknowledge the financial support of Azad University, Azadshahr Branch.

To my lovely wife, Souri, for all her never ending encouragement, unstinting support, patience and time.

To my mother, sisters, brothers, and in-laws.

To all my friends in Iran and in the UK for their support, especially Hossein Ameri, Hossein Alibeiki and Shahram Asadi.

Finally to anyone who I missed.

CONTENTS

Abstract i				
A	Acknowledgments ii			
1	lr	Introduction		
_				
	1.1	Introduction: the technical problem – behaviour of landslides	1	
	1.2	Landslides in south east England	2	
	1.3	Residual shear strength	4	
	1.4	Essential terms and definitions	5	
	1.5	Research objectives	8	
	1.6	Research scope	9	
2	A	review of the previous research on residual shear strength	.11	
	2.1	History of residual shear strength, general review	11	
	2.2	Milestones: The 4 th Rankine Lecture 1964 and Skempton's lecture in 1984	12	
	2.3	How residual strength is measured in the field and the laboratory	14	
	2.4	Development of laboratory measurement and apparatus- historical review	16	
	2.5	Ring shear apparatus:	18	
	2.6	Bromhead Ring Shear Apparatus	19	
	2.7	Errors of Bromhead Ring Shear Machine	20	
	2.8	Summary	24	
3	G	eneral discussion on the factors which affect the residual shear strength	25	
	3.1	Level of total normal stress	25	
	3.2	Shear strain rate	26	
	3.3	Pore water pressure	28	
	3.4	Clay minerals	29	
	3.5	Clay fraction, plastic index and soil texture	31	
	3.6	Soil texture	33	
	3.7	Soil Chemistry	34	
	3.8	Consolidation, lateral stresses and general stress history	35	
	3.9	Discussion	36	
4	Ba	ckground and context of back analysis	37	

	4.1	Introduction	37
	4.2	Topographic shape	40
	4.3	Piezometric conditions	46
	4.4	Soil density	47
	4.5	Soil zonation	48
	4.6	Procedure of back analysis for a landslides	49
	4.7	Discussion	52
5	D	evelopments in back analysis review of previous work introduction to new methods	
-			
	5.1	The problem of missing back analyses	54
	5.2	Method of dealing with weak layers	54
	5.3	Third-party criticism of the back analyses approach	58
	5.4	Discuss when it is absolutely vital to use 3D	60
	5.5	Conclusions	61
6	R	esidual strengths in landslides in the Barton Beds (Upper Tertiary)	62
	6.1	Introduction: Geographical and geological setting	62
	La	DCATION	62
	6.2	Field investigation and observation:	68
	6.3	Reconstruction of piezometric conditions	71
	6.4	Back analyses	73
	6.5	Weak laver analyses	79
	6.6	Modified Cross Section of landslide at Cliff House	81
	6.7	Discussion of back analysis results:	
	6.8	Conclusion	84
_			
7	FI	eld investigation and experimental programme of Barton Clay	87
	7.1	General discussion on Identification of slip surface in the field	87
	7.2	Samples used in investigation	88
	7.3	Testing methodology	89
	7.4	Application of Bromhead Ring Shear Apparatus	91
	7.5	Sampling and Laboratory measurement of Barton Clay	95
	7.6	Previous research	100
	7.7	Conclusions	102
8	Th	e residual strength of clay in coastal landslides in the London Clay Formation	103
	8.1	Introduction: Geographical and geological setting	103

8.2	Previous work on coastal landslides	106
8.3	Field observations	109
8.4	Re-appraisal and further work on the Herne Bay landslides	109
8.5	Appraisal of Sheppey	114
8.6	Comparative discussion of back analysis results:	115
8.7	Previous research: (Ring shear results)	117
8.8	Conclusion	119
9 TI	he residual strength of London Clay in landslides along the infrastructure cuttings	122
9.1	Introduction: Geographical and geological setting	122
9.2	Previous work on Infrastructure cuttings	122
9.3	Re-appraisal of infrastructure cutting analyses	124
9.4	Discussion of back analysis results:	135
10 Di	scussion and conclusion	138
10.1	Introduction and summary of the thesis	138
10.2	Correlations	139
11 Co	onclusions	146
11.1	Conclusions	146
11.2	Future research	148
Referen	Ces	150
Append	ix A	164
Append	ix B	172
Append	ix C	174
Append	ix D	180
Append	ix E	188
Append	ix F	201
Appendi	x G	202
Appendi	х Н	206

FIGURES

Figure 1-1: A simplified Stratigraphic column for Britain. Most of the stiff clays in which deep-seated
landslides formed, are found in the beds of the Upper Carboniferous, the Triassic and especially in
Jurassic, Cretaceous and early Tertiary2
Figure 1-2: Simplified geological map of South East England showing of the locations of the major coastal
landslides and some the major case studies of this research (after British Geological Survey)3
Figure 1-3: Graph is idealised to show development of a residual shear strength envelope from a series
of individual tests on soil specimens and the equivalent peak strength
Figure 1-4: Definition of Brittleness and Sensitivity Index
Figure 1-5: An example of a shear surface from a shallow landslide at Barton on Sea (Barton D Zone) 6
Figure 1-6: The sketch shows shear surface and the plane of the separation of different parts of the
moved ground. It illustrates the relocation and re-alignment of the layers (after Henkel, 1957) 7
Figure 2-1: Simplified relation between normal and over consolidated clay (after Skempton 1964) 11
Figure 2-2: (a) Formation of multilayer slickenside after shear test (Skempton, 1964); (b) Variation of
shear strength of soil with displacement (Skempton, 1985)14
Figure 2-3: Various stages and their comparison in Multi-Stage ring shear test (after personal
communication with Bromhead, 2010)20
Figure 2-4: This picture shows soil extrusion during ring shear test. This particular test, shown in the
picture, was run for two days and under relatively high rate of shearing. The picture illustrates a
small amount of extrusion21
Figure 2-5: Sketch illustrates plan view of the upper platen, Centring Pin and Torque Arms of Bromhead
Ring Shear Apparatus
Figure 3-1: Curve shape of residual shear envelop and over estimate of residual c by linear interpretation
(after personal communication Bromhead, 2010)26
Figure 3-2: Variation of ϕ 'r with proportion of montmorillonite and kaolinite (after Yamasaki, 2000) 30
Figure 3-3: (a) Relationship between calip and residual friction angle using DSBT , (b) Relationship
between calip and residual friction angle using RST (after Colotta et al., 1989)
Figure 3-4: (a) relationship between residual friction angle and clay fraction (Skempton1985), (b) redrew
the curve relationship of clay-fraction residual ϕ' by Mesri and Cepeda-Diaz (1986)
Figure 3-5: The sketches illustrate changes of soil fabric as a result of shearing. The fabric of soil before
shearing, as it was deposited, and after shearing for different soils: granular soil material (single
shearing, as it was deposited, and after shearing for different soils: granular soil material (single size), poorly-sorted and clay particle are shown. Single size soil appears unchanged after shear, in
shearing, as it was deposited, and after shearing for different soils: granular soil material (single size), poorly-sorted and clay particle are shown. Single size soil appears unchanged after shear, in the poorly sorted granular soil, the larger particles are worked out of the shear band.

(
(bottom). On the left relationship between shear stress-displacement for different soils at various
stages illustrated (after Lupini et al., 1981).
Figure 3-6: Peak and residual strength of undisturbed and remoulded soils using various methods of
testing (after Bishop et al., 1971)35
Figure 4-1: A curved residual strength envelope
Figure 4-2: Example of different cliff profile after landsliding, Warden Point, Isle of Sheppey (after Dixon
& Bromhead, 2002)
Figure 4-3: Position of slip surface at Queen's Avenue Landslide, Herne Bay
Figure 4-4: Bowl shape of a landslide which should be analysed in 3D (after Bromhead and Martin, 2004)
Figure 4-5: Examples of error bars for piezometric levels (after The 12th Glossop Lecture Presentation;
Bromhead, 2011)
Figure 4-6: The error bars which caused by assumption of soil density (after The 12th Glossop Lecture
Presentation; Bromhead, 2011)
Figure 4-7: Bedding controlled compound landslide running along a weak bed (after Gregory, 1844) 49
Figure 4-8: The graph shows how the residual friction angle can be determined from the results of three
back analyses
Figure 5-1: shows a simplified landslide with a bedding controlled segment, roughly based around the
Barton cases. ϕ_{r2} : represents the slipping through the weak clay & ϕ_{r2} :represents the slipping
through the strong clay (the back slip)55
Figure 5-2: This graph shows results of ϕ'_{r1} vs. ϕ'_{r2} and the limitation and boundaries of the ϕ'_{r1} and ϕ'_{r2} .
Rule1 and Rule2, as explained in the text, are shown on the horizontal axis
Figure 5-3: Cross section of Grandview Lake Dam with assuming rupture surface and Piezometric line
(after Deschamps & Yankey, 2006)58
Figure 5-4: Results of triaxial tests on samples from Grandview Lake Dam. The Red and blue coloured
points show the results of analyses when the slope treated as two different materials
Figure 5-5: Results of analyses of the slope as e homogenous material and treating the back slip and
bedrock junction as two different materials60
Figure 6-1: Location of study area in the whole map of the UK as well as in outcrops of Barton Clay. The
area includes about 1400 m unprotected cliff line in frontage of Naish Farm Estate at Highcliff in
the west and to Cliff House Hotel in the east and about 1000 m of stabilised slopes from Cliff
House to Central Amenity Area including stable and unstable engineering works
Figure 6-2: The typical stratigraphical scheme of Barton Clay. The zonation was given by Burton (1933)
and thicknesses, in metres, have been amended from Barton (1973)

- Figure 6-5: The map shows the study area, from Naish Farm to eastern outcrop of Barton Clay. The location of four cross sections used in back analysis and the location of sampling are illustrated. Samples are collected from slip surface exposure in F horizon Tom's Garden and in D horizon down of Naish Farm (after Garvey, 2007).

- Figure 6-8: Shows how to define range of φ'r1 for cross section of landslides at CAA, HG, HGW, MCH using Average and Real piezometric conditions. The narrow red lines show the range of φ'r1.....80
- Figure 6-10: Geological section of Cliff House Landslide with cliff profile 2007. Barton et al. designated that deep movement taken place in Zone D. Evidence of the site and direction of displaced sheet pile indicate that the current topography of the area is the results of two separate slips in D horizon and F horizon. The former happens as result of toe erosion and continuous toward the cliff progressively along Zone D. This movement confirmed by the direction of displaced sheet piles. Subsequent movement take place in Zone F as a result of recession of the back scarp of the lower slips.
 82
 Figure 7-1: Direction of the D horizon is shown by the dash line and can clearly be recognised from the
 - viii

Figure 7-2: The polished slip surface which includes with brown staining is shown in the photograph. The
slip surface is located along the D Zone and includes fossils8
Figure 7-3: Bromhead ring shear apparatus, showing different parts, including: load hanger, proving
rings, dial gauges, transducers, torque arms9
Figure 7-4: Shows link between the ring shear, transducer and the data logger
Figure 7-5: The location of sampling from the exposure of slip surface in the F and D horizons in 2010
and 2011 are shown in the picture. Two samples from the slip surface in the weak bed in the D
horizon in 2010 and 2011 are made, and one sample from the F horizon (Bing Maps)
Figure 7-6: This graph shows σ'_{n} - τ for results of samples from the exposure of basal shear surface in the
D horizon of Baron Bed. Four tests are carried out on the samples in 2011 and three on the
samples in 2010. Locations of sampling are shown in Figure 7.5
Figure 7-7: This graph shows σ'_n - τ for results of samples from the exposure of basal shear surface in the
F horizon of Baron Bed. Three tests are carried out on the samples in 2011. Locations of sampling
are shown in Figure 7-5
Figure 7-8: Results of the Bromhead ring shear tests plotted as values of effective normal stress versus
shear stress for the dark chocolate brown seam and the immediately underlying D Zone of Barton
Bed (after Barton et al., 2006)
Figure 7-9: Results of the Bromhead ring shear tests on the samples from the landslides at Bartopn Sea
are compared with published results using same apparatus.
Figure 8-1: Location studied sites in the London Basin and Hampshire Basin. Each point shows the
location of the cross section for back analysis (after British Geological Survey)
Figure 8-2: Lithostratigraphy of the London Clay Formation in the Hampshire Basin, the Lithology highly
generalised (after King, 1991). It illustrates the major lithofacies and the relationship of the named
lithostratigraphic units to the deposition sequences105
Figure 8-3: Section through the Miramar Landslide showing erosion and regrading (after Bromhead,
1978). The front slip and ground profile in 1956 and 1966 & 1970 has been taken for the back
analysis
Figure 8-4: Section of Mudslide in London Clay at Beltinge, showing flow II (after Hutchinson, 1970). The
slip surface and piezometric level were recognised by instrumentation. The ground profile in 1966
has taken for back analysis113
Figure 8-5: Results of residual friction angle calculation using back analysis technique for the landslides
in London Clay at Beltinge. These results are compared with published data.
Figure 8-6: Results of residual friction angle measurement by ring shear apparatus (both Bromhead and
Bishop devices) for the landslides in London Clay. The data come from the published data or
extracted from published graphs

Figure 8-7: Result of back 2D and 3D back analysis of landslides at Herne Bay including the new analyses
of this research. The red coloured arrow shows the drop of stresses of 2D analysis to the 3D
analysis in Beacon Hill landslide
Figure 9-1: Section of landslide in London Clay cutting at Dedham (after James, 1970). The section
illustrates the original ground profile after the cutting in 1840 and the ground surface profile
before and after the slip in 1952. Water level was recorded in stand pipes after failure but slip
surface was assumed from failure profile and boring results.
Figure 9-2: Section through Northolt cutting (after Henkel, 1957). The original ground surface profile
after cutting 1938, the ground profile after slipping, location of piezometers and slip surface 128
Figure 9-3: Cross section of landslide in London Clay at Upper Holloway (after James, 1970). The section
illustrates original profile of ground cutting, ground profile after slipping, location of retaining wall
before and after of slipping, location of assumed slip surfaces and combination of piezometric
data to provide four different ground water condition. GWL2 is assumed piezometric level before
failure
Figure 9-4: Cross section through landslide in London Clay at Wembley Hill (after James, 1970). The
sketch shows ground profile before and after slip. Location of analysed slip surface and applied
ground water condition are drawn
Figure 9-5: Section of landslide in the cutting in London Clay at Fareham in Hampshire Basin (after
James, 1970)
Figure 9-6: The graph show results of back analysis of landslides in London Clay
Figure 10-1: Results of the Bromhead ring shear tests on the samples from the landslides at Barton on
Sea are compared with published results using same apparatus
Figure 10-2: Results of back analyses show the average residual friction angle for landslides at Barton on
Sea showing average ϕ 'rav for each case study
Figure 10-3: The graph compares results of back analysis and Ring Shear test of Landslides at Barton on
Sea (This research and Literature)
Figure 10-4: Results of published ring shear tests using Bromhead device and Bishop apparatus on the
samples from London Clay
Figure 10-5: The graphs shows results of published back analyses on the cross sections of landslides and
results of the acceptable cross section of this research (as stated in the previous chapters) 141
Figure 10-6: The graph compares results of back analysis and Ring Shear tests of landslides in London
Clay. The data includes published papers and results of this research
Figure 10-7: The graph shows results of back analyses and Bromhead shear results from the literature
and the results of this research. It is also included comparison of those results and results of
Bishop ring shear device from the literature

-

TABLES

Table 3-1: Landslide velocity classes by Cruden and Varnes(after Cruden and Varnes, 1996)
Table 6-1: Results of back analysis of cross section in Barton Bed for the ground profile of 1967 prior to
stabilisation work. Average value of ϕ'_r for different ground water scenario are listed
Table 6-2: Results of back analyses of the cross section of landslides at Barton on Sea for ground profile
of the post stabilisation failures75
Table 6-3: Results of back analysis of modified cross section of the landslide at Cliff House
Table 6-4: Date of major stabilisation works and significant landslide reactivation for in Barton on Sea. 77
Table 6-5: Location of basal slip surfaces for the landslides in Barton on Sea by B&G and modified slip
surfaces with the relevant average φ'_r 84
Table 7-1: Results of laboratory testing on the samples from F and D horizon of Barton Bed from the
landslides at Barton on Sea
Table 8-1: Shows results of back analysis of front slip considering the ground profile in 1956 and
1966&1970 separately, as shown in Figure 8-3112
Table 8-2: Shows results of back analysis of mudslide at Beltinge considering the main slip of the flow II
in the front and ground profile in 1966 as shown in the Figure 8-3
Table 9-1: Sections of landslides in London Clay which investigated by James. A number of sections re-
analysed by Hosseyni in order to calculate residual shear strength of London Clay
Table 9-2: Results of back analysis of landslide in the cutting at Dedham for main slip considering the
ground surface after slipping in 1952 and front slip surface considering the modified ground
profile as shown in Figure 9-1. The analysis repeated for different ratio of $ \varphi'_r$ in the case of
exciting weak bed127
Table 9-3: Results of back analysis for the cross sections of landslide at Northolt. The table show results
of analysis for the main slip in the cross section by James and Henkel separately. Results of
repeated analysis for lower ϕ 'r for the weak bed are shown in the second row129
Table 9-4: Results of back analysis for slip A of landslide at Upper Holloway using the ground profile after
slip 1953 at different ground water level extrapolated from the piezometers in the slipped and
intact areas as shown in Figure 9-3130
Table 9-5: Results of back analysis for slip A of landslide at Upper Holloway using the ground profile after
slip 1953 at different ground water level lower that GWL4 as explained
Table 9-6: Results of back analysis for the cross section in Figure 9-4, considering modified slip surface A
at different ground water condition as listed133
Table 9-7: Results of back analysis of two sections A and B of landslide at Fareham for different ground
water condition at different ratio of ϕ'_r 134

SYMBOLS

•:	Degree
μ:	Coefficient Friction
c':	Drained Cohesion of soil
c:	Cohesion of soil
c'r:	Drained Residual Cohesion
kPa:	Kilo Pascal
r:	Radius
r _i :	Internal Radius
r _o :	External Radius
r _u :	The ratio of pore water pressure
S:	Residual Strength
Т:	Torsion
W:	Water content
δΤ:	Differentiation of Torsion
σ _n :	Normal Stress
σ́n:	Effective Normal Stress
τ:	Shear Stress
φ.	Drained Internal Friction angle
ф:	Internal Friction angle of soil
φ ['] r:	Drained Residual of Internal Friction angle
φ'r1:	Drained Residual of Internal Friction angle of soil 1
φ' _{r2} :	Drained Residual of Internal Friction angle of soil 2
φ' _{rav} :	Drained Residual of Internal Friction angle of soil 2

ABBREVIATIONS

- 2D Two Dimensional Analysis
- 3D Three Dimensional Analysis
- 3-D Three Dimensional
- av Average
- BS British Standard
- CAA Central Amenity Area
- CF Clay-Size Particle Fraction
- CH Cliff House
- csc 1/sin
- F Factor of Safety
- Fig. Figure
- FS Factor of Safety
- GWL Ground Water Level
- HG Hoskin's Gap
- HGW Hoskin's Gap West
- Ibid. Previous Cited Reference
- IC-NGI Imperial College–Norwegian Geotechnical Institute
- ISL International Symposium on Landslides
- LL Liquid Limit
- m meter
- MCH Modified Cliff House
- min minute
- mm millimetre

- N.G.R National Grid References
- NaCl Sodium Chloride
- O.D. Ordnance Datum
- PI Plasticity Index
- Piez. Piezometric
- PL Plastic Limit
- R. S. Ring Shear Test
- SE South East
- SF Factor of Safety
- F Factor of Safety
- sic Thus it was written
- vs. Versus
- WL Liquid Limit

1 Introduction

1.1 Introduction: the technical problem – behaviour of landslides

This thesis describes a comprehensive study of the *drained residual shear strength*¹ of a number of British clays from sites in South East England. For reasons of accessibility for sampling, the majority of the sites which have provided samples for testing are coastal landslides, although the literature provides numerous examples of inland landslides, commonly failures of natural slopes and infrastructure cuttings.

When a landslide occurs in a soil with significant clay content, the soil mass is ruptured, and relative displacement occurs along one or more surfaces in the soil mass. These surfaces tend to be planar when sliding occurs along bedding planes, or concave upwards when the rupture surface crosses the bedding. During the process of rupture, the soil fabric is modified, and it has been shown (Lupini et al. 1981) that this is due to the realignment of clay mineral particles in the direction of relative movement. Along the surface of rupture, much of the original strength of the soil is lost, and what remains is termed the *residual shear strength*. Full definitions are given in Section 1.3 and a review of previous researches on the residual shear strength are discussed in Chapter 3.

An initial failure of a slope is usually characterised by large² displacement, followed by long periods of small magnitude slow movement. This movement can damage or distort infrastructure or buildings, and commonly requires the relic landslide mass to be stabilised. All issues of stabilisation rely on an appropriate assessment of the residual shear strength. The historical development of understanding this process is outlined in Section 1.3 below, with amplification of particular issues in Chapter 2 and Chapter 3.

¹ To be defined later

² Large displacements are large relative to the size of the slope. Displacements of landslides in Britain are usually small relative to (say) landslides in mountainous areas.

Chapter 1: Introduction

A review of the literature on residual shear strength in clay soils shows that what might at first seem a wealth of data is in fact rather sparse. In this thesis, the understanding of residual shear strength is improved via two approaches. One of these is laboratory based, and relates to the measurement of residual shear strength in a ring shear machine (Bromhead, 1979). The other technique known as *back analysis* (Chandler, 1977) is used to obtain an approximate value for the residual shear strength operating in the field.

1.2 Landslides in south east England

Geological sequences of Britain are remarkably varied within a relatively small country. They encompass strata from late Precambrian ranging to Holocene (Fig. 1-1). This complex underlying geological sequence and structure has been folded and uplifted, eroded but also repeatedly over geological time, producing new sediments so that later sediments often incorporate materials derived from the erosion of earlier ones.

In the SE sector of England, the geology comprises a sequence of mainly Mesozoic and Tertiary sediments that have been generally subject to only gentle folding, giving at most a few degrees of dip, although there are some local areas (such as the Weymouth-Portland anticline and the Isle of Wight monocline) where the strata are highly contorted.



Figure 1-1: A simplified Stratigraphic column for Britain. Most of the stiff clays in which deepseated landslides formed, are found in the beds of the Upper Carboniferous, the Triassic and especially in Jurassic, Cretaceous and early Tertiary. Britain generally, and south east England in particular, has very low relief. In south east England, the gently folded strata create simple structures which include two basins, the London Basin and the Hampshire Basin separated by a dome known as the Weald.

While Britain is often thought to be a wet place, the rainfall in the SE of England is not particularly high. There is generally a differentiation from summer (drier) to winter (wetter), the climate does not have the arid summers of (say) the Mediterranean part of Europe, and as a result, the groundwater table is usually high.

Figure 1-2: Simplified geological map of South East England showing of the locations of the major coastal landslides and some the major case studies of this research (after British Geological Survey).

This research has concentrated on particular parts of the geological sequence, notably in the Jurassic, Cretaceous and particularly the Tertiary, largely for logistical reasons (see Figure 1-2). For length reasons in this thesis only two important geological units in the Tertiary have been presented (results of the other clays are presented in Appendices D and E). These sequences are characterized by deposits of overconsolidated clays interbedded with sands and sometimes limestones. Landslides are associated with the clay members in almost all geological units. Inland slopes are often gentle because of the influence of periglacial solifluction.

1.3 Residual shear strength

The **shear strength** of a soil is specified as the maximum shear stress that the soil can carry, and it is categorised by two states; *peak* and *residual*. In laboratory measurement, the **peak shear strength** is the shear strength of an undisturbed soil up to the point of rupture (at fairly small shear strains in the soil mass) and the **residual shear strength** (sometimes termed *ultimate shear strength*) is the strength exhibited when the soil sample passes the point of rupture and is continuously sheared to large displacements until a lower bound strength is exhibited (Fig. 1-3). The historical dimension of the concept of the residual strength is more fully developed in Chapter 2. This thesis only considers residual shear strength, and therefore, a range of factors related to the development of peak strength is of little relevance.



Figure 1-3: Graph is idealised to show development of a residual shear strength envelope from a series of individual tests on soil specimens and the equivalent peak strength.

Residual strength is normally expressed in a 'cohesive-friction' model (c' - ϕ ') and in terms of effective stress, i.e. in the simplest case:

$$S = c'_r + \sigma'_n \tan \phi'_r$$

With the subscription 'r' denoting residual. Described in this way, the pore water pressure needs to be known, and the problem is therefore 'drained' (Terzaghi et al., 1996 & see Section 1.4).

If the failure envelope is not linear, then a more complex relationship may be required, although the linear form, often with $c'_r = 0$, is adequate.

1.4 Essential terms and definitions

Drained and **undrained** conditions in soils represent a range of strength which may be obtained in the soil mass in response to pore water pressure generated by loading and unloading.



Figure 1-4: Definition of Brittleness and Sensitivity Index

When a soil sheared beyond the strain in which its peak strength is mobilised, the load carrying capacity will decline. At enough large strain, residual strength is achieved. This process and its result are known as sensitivity and brittleness. For undrained states, it is classified as **sensitivity** and for the drained strength it is termed **brittleness** (Figure 1-4). These two phrases are determined as:

Sensitivity index = a/b

Brittleness index = (a-b)/a

Where:

a= Peak strength

b= Residual strength

Sensitivity and brittleness depend on many complex factors but in the case of simplicity it can be argued that they grow when the over-consolidation value, clay content (CF) and the plasticity index (IP) increase.

Chapter 1: Introduction

Soil fabric is a phrase to describe the range of soil particles, their arrangement, type and size of a specific soil feature. The fabric of a soil is affected by the conditions in which the soil was formed. History of stress and weathering are two other major factors which affect the soil fabric.

The **shear surface**, which also sometimes termed a slip surface or basal shear, is a phrase which describes a thin zone in which differential movement in the ground takes place, as a result the residual strength is established. Hutchinson (1970) discussed the process and locations in which the rupture surface or slip surface of landslides in British Clays take place. Figure 1-5 shows an example of the slip surface which explored in the field at Barton on Sea in Hampshire.



Figure 1-5: An example of a shear surface from a shallow landslide at Barton on Sea (Barton D Zone).

The shear surface consists of the plane of separation of two different parts of the moved-ground as well as the thin layer in which the remoulded clay can be found. In fact this is the location of existing soil fabric that it is replaced by a new fabric which indicates the effect of shearing. A shear zone may include two or more slip surfaces which may be sub-parallel or formed over another slip.

The clay particles which are mostly sheet type (platey) are orientated as result of shearing. In this case the shear zone contains a very large proportion of the clay particles which are oriented with their long axis in the direction of displacement and as a result a slip surface in the soil mass is formed. Therefore it is obvious that a shear

Chapter 1: Introduction

zone normally has a fluted, polished surface. Fluting is believed to be caused by harder lumps in the clay.

The Identification of slip surfaces is described by Hutchinson (1983). It can be done by visual identification in samples, visual identification in the walls of pits, or by instruments such as inclinometers. A variety of techniques exist to confirm these identifications (Fig. 1-6).





In the low dip angle of strata of south east England, all or part of sliding surface often follow a single weak horizon. This occurs most strongly where the weak horizon is at the toe of the slope, but can happen when the weak horizon is below the toe, or outcrop in the slope face. The resulting landslide shape is referred to as 'compound' (Fig. 1-6).

Where the sliding surface is controlled by the location and orientation of a weak bed in the sequence, it is termed a **bedding-controlled landslide**. Sometimes, these slides break out at a higher level within the strata and create progressive landslides which can be recognised as benches from distance (Bromhead & Ibsen, 2004).

Hutchinson (1970) discovered that many flow-like mass movements were, in fact, slides taking place along a basal slip surface. As slip surfaces only form in soil drier than the liquid limit, the Atterberg Limits do not relate to particular mechanical properties of soil. In a landslide, however, they remain an important method of classifying soil.

In this thesis Atterberg Limits are used to classify soil with tests done according to BS 1377 Part 2 1990.

1.5 Research objectives

The thesis arises out of a chance comment, made in a discussion with my supervisor, Professor E. N. Bromhead, who observed that when he was reading Hutchinson's 1969 paper on the Folkestone Warren landslides. He noticed that about half of the sections presented in that paper had not been analysed. He offered to do this work. Moreover, Hutchinson was keen to undertake residual strength determinations using the new Imperial College ring shear machine (Bishop et al., 1971) and the opportunity was taken to cross-correlate the results with a newly-developed small ring shear machine (Bromhead, 1979) as well as with the back analyses. During the course of preparing the results of the Folkestone Warren case for publication, it was discovered that some of the mechanics assumed in the 1969 analysis were misleading, and that was also corrected in the paper that ensued (Hutchinson et al., 1980).

Reflecting that original process, the Author of this thesis has set out to discover published landslide cross sections, and to back-analyse them. In this way, the body of results for field mobilised residual strength has been enlarged. Where it has been possible to sample material close to the slip surface of a landslide, ring shear tests have been carried out, supported by a range of classification tests. During this sequence of activities, it has been found that a significant proportion of the published sections lack good piezometric data, and indeed, some of the slip surface positions can be described as, at best, conjectural. Where it has been possible to provide alternative or better interpretations, they have been made. The thesis therefore reflects that original model for research.

The Folkestone Warren landslide complex is a series of compound landslides, with a common basal shear surface following a weak bed (sometimes called the 'high liquid limit' bed) near to the base of the 45m thick Gault Clay. Such landslide types are common in the literature for landslides in SE England. Many of the cases analysed or re-analysed in this thesis conform to that general type. The residual strength of a weak

bed is self-evidently lower along the flat portion of the basal slip surface than along the remainder of the slip surface where it curves up from the weak bed at the head of the slide³, and possibly at the toe. This has required some manipulation of the data to provide insights into the properties of weak beds generally, leading to observations about the back analysis procedure and to some developments of it.

For reasons largely of space, analyses and text related to landslides in only two strata have been considered: the Barton Clay and the London Clay. Space in this thesis precludes the inclusion of equivalent completed studies of Gault and Lias Clays which will be published elsewhere. A related task must be to continue to extend the datasets in these materials, and to undertake analogous reviews for other clay strata, at present less well represented in the literature.

1.6 Research scope

Chapter 2 contains a review of previous research on residual strength, organised largely in a chronological order.

In Chapter 3, factors that affect residual strength are reviewed and discussed in general terms. In this thesis, the residual strength determinations are made (in ring shear tests and back analyses) taking the sites 'as found', and not attempting to see how the residual strength may vary with environmental conditions.

Chapter 4 is a reflection on the limitations of the back analysis procedure as carried out hitherto, including the sources of uncertainty. In the following chapter (Chapter 5) some innovative extensions to the back analysis technique and its practical application are given.

The discussion of residual strengths for the Barton Clay is given in Chapter 6 and Chapter 7. Chapter 6 covers back analysis and Chapter 7 covers ring shear and the laboratory tests of Barton Clay.

³ Called the 'back slip' in this thesis

A review of the field and experimentation programme is given in Chapter 7. Sites were visited to confirm, as far as possible, the nature of the landslides, and where possible to take samples for laboratory testing. Clearly, some stabilised landslides do not have the necessary exposures for sampling, and their morphology does not reflect the state prior to stabilisation. Infrastructure cuttings have been, by and large, ignored for access and safety reasons.

The following two chapters consider the residual strength of the London Clay which determined by back analysis. Chapter 8 discuses some additional work on coastal landslides at Herne Bay and Chapter 9 is a critical review and re-analyses of the dataset for failures in infrastructure cuttings.

Chapter 10 compares and contrasts the results culled from the literature with those derived in this study. The final Chapter (11) concludes the thesis with observations and recommendations for future work.

2 A review of the previous research on residual shear strength

2.1 History of residual shear strength, general review

The stress-strain behaviour of clay soils has been one of the main concerns of geotechnical professions since the early days of soil mechanics as a science. Haefeli (1951) stated that clay soils exhibit brittle stress-strain behaviour: when clay reaches its peak load carrying capacity, then with further deformation this capacity decreases, until a residual or ultimate carrying capacity is reached (Ibid.). Perhaps the first time the term "residual strength" was introduced into English was in 1950 (sic) by Haefeli (Skempton, 1964). Its origins are even older: to the work of Hvorslev, (1936, 1937 & 1939) and the others who searched for *true friction angle* (Grüner & Haefeli, 1934; Cooling & Smith, 1935).

However, as geotechnical engineering was prior to that concerned with problems of the stability of unfailed slopes and foundations, research emphasis lay on the determination of peak strength properties. Henkel (1957) discovered that when the drained heavily over-consolidated clays fail, the shear strength diminishes fairly rapidly with increasing strain while the strain at failure is comparatively small (Fig. 2-1).



Figure 2-1: Simplified relation between normal and over consolidated clay (after Skempton 1964).

The broad understanding in geotechnical practice of the residual shear strength problem starts with Skempton's *Fourth Rankine Lecture* (1964). The concept of residual

shear strength is commonly thought (in Britain) to have originated with Skempton's lecture, however, this topic appeared earlier (see above) and was later developed by Skempton himself and others. He fully articulates the issue in his 1985 paper (Skempton, 1985). Today, it is fully understood that the limiting lower value of shear strength is the residual shear strength, and as this applies on the slip surface of a landslide, all problems of the stability of failed slopes and landslides must be a function of this strength component.

Although triaxial apparatus was found a convenient device to measure peak strength, for measurement of residual shear strength, several different methods using different apparatus were devised by many researchers, independently. Reversal direct shear box and triaxial tests on pre-sheared samples were common within the literature. To reach the residual shear strength, large displacement must take place, therefore a rotary device would be appropriate. Bishop et al. (1971) reviewed the earlier works on the application of torsion and ring shear apparatus on the residual shear strength measurement. They invented a ring shear device which is known as the IC-NGI device (Imperial College–Norwegian Geotechnical Institute) (Ibid.).

Up to the late 1970s, although a number of ring shear devices had been developed, however, mostly were expensive and complicated to employ for residual strength measurement. In 1979 Bromhead introduced his simple ring shear device which is easy to operate. Description of features of slip surfaces by Hutchinson (1970) and the invention of the new ring shear apparatus helped the landslide specialist to make a great progress in understanding of landslide mechanisms and residual shear strength definition.

2.2 Milestones: The 4th Rankine Lecture 1964 and Skempton's lecture in 1984

Although, Henkel (1957) saw reduction of the post peak strength of clay soils, he did not articulate the definition of residual shear strength. However, the concepts within the literature were pulled together to form the theory of residual strength and its application in geotechnical engineering by Skempton in 1964. In fact, the 4th Rankine Lecture can be considered as a turning point to understanding of landslides. Consequently the concepts within the literature can be divided in two major periods, before the 4th Rankine Lecture and after that. Prior researchers had been looking for the fundamental principles of shear strength of clay soils. They had not precisely known that what residual shear strength means and what are its applications in geotechnical engineering.

Afterward, the geotechnical specialists focused on how to find slip surfaces in the field and new methods of measuring residual shear strength. Since then, several researches have been conducted to find the factors which can affect properties of each slip surface and understand their relationship with geological structure, history of geology, loading and unloading. They are trying to find the factors which influence the residual shear strength measurement.

In 1985 Skempton consolidated his theory of residual shear strength far more clearly benefiting from 20 years additional research by him and others. He concluded that increase in water content and the orientation of clay particles parallel to the direction of slipping, at shear surface are two major factor of the post-peak fall in shear strength of overconsolidated clays. He clarified the relationship between clay fraction and residual ϕ' (Fig. 2-2).

Skempton discussed the post-peak shear strength behaviour of clay in the drained condition. The post-peak drop in drained shear strength of overconsolidated clays occurs in two stages. First, at relatively small strain, the strength diminishes to the *fully softened* or *critical state value*, due to an increase in water content. Then, after much larger strain, the strength drops to the residual value, due to reorientation of platy clay minerals parallel to the direction of shearing. In normally consolidated clay, the post-peak drop occurs only due to particle reorientation (Ibid.) (Fig 2-1).

Chapter3: A review of the previous research on residual shear strength



Figure 2-2: (a) Formation of multilayer slickenside after shear test (Skempton, 1964); (b) Variation of shear strength of soil with displacement (Skempton, 1985).

2.3 How residual strength is measured in the field and the laboratory

There are several techniques for measurement of residual shear strength. The selected method depends on various factors in which can affect the obtained results. In general there are four alternative approaches to residual strength measurement:

- Field methods
- Laboratory methods
- Analytical numerical methods
- Using the published correlation factors

For field measurement of residual shear strength, the sample tested must contain the shear surface. It may be a natural shear surface from a landslip or one created artificially.

In laboratory measurement the sample disturbance and the sample size are critical and have significant effects on the results. In order to cope with such difficulties, a field shear box can be applied to measure in-situ residual shear strength of clays. This apparatus was utilized by Chandler et al. (1981). Other types of field shear boxes in order to measure in situ strength of rock joint and intact rock mass were provided by Hoek and Bray (1974) and residual soils (Cross, 2010). Although field shear boxes resolve some difficulties in terms of sample disturbance and size, pore water pressure cannot be under strict control. Therefore, drained tests for the clays are impossible. Considering that the strain control while loading is also too difficult.

Available samples have a significant effect on the selected experimental method in the laboratories. If sample contains a natural formed shear surface a direct shear test can be conducted, if not a multiple reversal shear box can be applied (Chandler, 1966 &, 1969; Skempton, 1985). Obtaining a representative specimen and selecting an appropriate strain rate have a significant effect on the results.

The triaxial apparatus is an appropriate device for measurement of peak strength which can satisfy drain and undrained tests (Bishop and Henkel, 1957). The triaxial apparatus enables us to control pore water pressure and strain rate. Pre-formed shear plane samples were commonly used in triaxial tests in order to measure residual shear strength (Hutchinson, 1967; Wood, 1955).

Shear box and triaxial apparatuses can provide a small strain; on the other hand the specimen is not subjected to continuous shear strain in one direction. Therefore, orientation of clay particles, which is necessary to provide the formation of shear surface in terms of clay fabric, is not completely satisfied. In order to cope with these limitations torsional ring shear devices were developed (Bishop et al., 1971; Bromhead, 1979). A torsional ring shear apparatus has the ability to shear the specimens continuously in one direction for different strain rate.

Shear strength tests on natural shear surfaces are cost effective. On other hand, obtaining an adequate number of representative specimens for the laboratory test purposes is not so easy and sometime expensive. Considering the size of samples as compared to the landslides volume to search for all alternatives reliable technique to measure residual shear strength. Back analysis (e.g. Chandler, 1977; Hutchinson, 1969; Bromhead, 1978) is a technique which employs numerical analysis and the factor of safety to calculate the value of residual shear strength. The shape of a landslide in plan

Chapter3: A review of the previous research on residual shear strength

and cross section and the reliability of surface and subsurface detail, including the location of the slip surface and the pore water pressure information, have a significant effect on the calculated results. Comprehensive results on landsliding in London Clay using this methodology are provided by Bromhead and Dixon (1986). The back analysis of a single landslide provides just one result for a particular average normal effective stress therefore, to produce a full shear strength envelope, this single result must be supplemented by further back analyses of similar cross sections. At least three points are needed to plot an envelope of shear strength (see Figure 4-8).

Generally for British Clays, the back analysis results indicate excellent agreement with test results on natural shear surfaces and angle shearing resistance is appointed as 1-1.5 degree higher than ring shear results (Skempton, 1985; also see the last chapter of this thesis).

In the absence of test results, residual shear strength parameters are generally determined using the published correlations with other properties such as plasticity (Tiwari and Ajmera, 2011b & 2012; Stark and Hussain, 2010).

2.4 Development of laboratory measurement and apparatus- historical review

Laboratory measurement is a common way to determine soil properties. Samples from boreholes, trenches or slip surface exposures make the measurement of residual shear strength and index properties of shear surfaces possible. Apart from ring shear apparatuses, some of the significant research projects, particularly in Britain, on residual shear strength measurement in the laboratory are chronologically reviewed as follow.

Wood (1955) used unconfined compression tests for soft clay and triaxial tests for firmer samples to determine shear strength of samples, from boreholes at Folkestone Warren landslides.

Skempton (1964) quoted the results of multiple reversal shear box tests and drained triaxial tests using sheared specimens on the samples from London Clay.

Borowicka (1965) used reversal shear box tests on samples prepared from pure clay minerals.

The residual strength of fine-grained minerals and mineral mixture was investigated by Kenny (1966) using reversal direct shear tests.

Chandler (1966, 1969) applied reversal direct shear box and triaxial tests along pre-cut planes to study residual shear strength of low plasticity Keuper marl. Then in 1989 he used a thin-sample technique in reversal shear box test to compare residual strength of London Clay and two low plasticity glacial tills. He compared these results with the results from other techniques. He concluded that this method provides reliable results when a ring shear apparatus is not available (Chandler and Hardie, 1989).

Hutchinson (1967) performed triaxial tests on pre-formed shear planes to study effect of changes in the cross sectional areas, and restraint of the rubber membrane. Then, in 1969 he compared the residual shear strength of Gault Clay, from the landslide at Folkestone Warren by three methods; direct shear test, plane cut shear test and back analysis.

Laboratory results of residual shear strength from direct shear box and triaxial devices have been compared many times and it has been reported that results of direct shear box and triaxial are higher than the results from ring shear apparatus (e.g. Bromhead and Curtis, 1978; Skempton, 1985; Hawkins and Privett, 1985; Stark and Eid, 1992).

Bromhead compared residual shear strength of British Clays from alternative laboratory methods and back analysis (Bromhead and Curtis, 1983; Bromhead and Dixon, 1986; Bromhead et al., 1999).

2.5 Ring shear apparatus:

Residual shear strength has insightfully been investigated since the 1930s using rotary shear machines (e.g. Grüner & Haefeli, 1934; Cooling & Smith, 1935; Hvorslev, 1936, 1937 & 1939). Considerable researches on measurement of large deformation shear strength of cohesive soil were done during 1933-1939 by Hvorslev. In 1951 Haefeli spotted that residual strength is related to the development of a slip surface and its corresponding loss of cohesion. The research by Hvorslev and Haefeli provided the basis of understanding of residual shear strength.

The earliest generation of ring shear apparatus were very basic and composed of a laterally confined disk or cylinder which is normally loaded through the top platen. The machines were designed so that either the top or bottom platen is twisted in order to provide large displacement. Those apparatuses had ability to reach infinite shearing without any changes in the sheared surface. However, variation in shear stress across the radius of the sample could be a source of uncertainties in those machines. The main disadvantage of those machines was that the shear surface was generated between the main body of the sample and the platen, whereas Ghani (1966) considers that ideally it should be placed within the body of the sample.

In order to resolve this shortcoming an improved design using split confining ring to allow shear surface formation in the middle of the sample was proposed by Langer in 1938 (cited in Clark, 2005). This design did not solve the problem of uneven stress distribution so that more even stress distribution can be achieved using an annular sample. Bishop recognised that shear stress transmission would also be problematic.

Further to the Fourth Rankine Lecture, Casagrande at Harvard University and Bishop at Imperial College encouraged to develop a new ring shear machine. The Harvard design was introduced in 1969 (Sembenelli and Ramirez, 1969) and then fully developed in 1970 by La Gatta.

An excellent summary of early ring shear apparatus was provided by Bishop et al. in 1971. The new ring shear apparatus, which is now known as the IC-NGI ring shear

Chapter3: A review of the previous research on residual shear strength

apparatus, was a result of a collaborative work between Imperial College and the Norwegian Geotechnical Institute. This machine can be considered as the turning point in the field of residual shear strength investigation in the UK. This apparatus has been used for example by Lupini et al. (1981); Tika et al. (1996); Tiwari and Marui (2005 & 2003).

Although a body of data of correct measurement of residual strength was produced using the IC-NGI apparatus, however, it was too complicated for industrial use. There is no doubt that this is an appropriate instrument for research purposes.

Many other ring shears were invented, simultaneously, for research purposes around the globe. Among these for example and ring shear device RS-NL (Fleischer and Scheffler, 1972) and a ring shear at US Army Engineer Waterways Experiment Station based on the Hvorslev design by Townsend and Gilbert (1973).

The next ring shear apparatus was developed by Bromhead (1979) who introduced a compact design for ring shear test which was aimed for academic proposes and commercial markets. He name this apparatus *simple ring shear*. Features of this apparatus will be explained in more detail in Section 3.6.

Stark and colleagues modified Bromhead ring shear apparatus in order to measure the drained residual strength of cohesive soil and geosynthetic and soil interface (e.g. Stark and Vettel, 1992; Stark and Eid, 1993; Stark and Poeppel, 1994).

2.6 Bromhead Ring Shear Apparatus

Since Bromhead established his simple shear apparatus in 1979, it has been widely used for research and commercial purposes. The key success of this apparatus is simplicity in use and the depth of the sample. As it is only 5 mm deep, drainage is rapid therefore, reducing consolidation times and allowing use of faster rates of drained shearing. This apparatus significantly reduced the test time in comparison with the other devices. Many researchers have used the original apparatus and the Kingston Procedure or trying to modify the machine and the existing methods and procedures for their own purposes (i.e. Hawkins and Privett, 1985; Stark and Vettel, 1992).
In 1990 British Standard 1377 part 7 for Shear Stress Test was introduced. The test procedure given in this standard to determine the residual shear strength of a soil using the Bromhead ring shear apparatus, contains a number of procedures that make the test both difficult and time consuming. Therefore in 1997 Harris and Watson developed a test producer at Kingston University to optimise the test procedure. This procedure in the literature is recognised as *Kingston Procedure*. The procedure is in effect a multi-stage test and results in a value for the drained residual shear strength obtained quickly and with least effort.

Since Bromhead introduced his simple ring shear apparatus in 1979, he examined a huge number of samples of clays from basal shear surfaces of landsliding in Britain (Bromhead, 1979). He compared residual shear strength of London Clay from alternative laboratory methods and back analysis (Bromhead and Curtis, 1983; Bromhead and Dixon, 1986; Bromhead et al., 1999). He stated that multistage ring shear test is ideal for the shear strength measurement (see Figure 2-3). He spotted that there are some movements of fine particles at shear zone and basal shear areas which provide low residual strength (Bromhead, 1992).



Figure 2-3: Various stages and their comparison in Multi-Stage ring shear test (after personal communication with Bromhead, 2010).

2.7 Errors of Bromhead Ring Shear Machine

Bromhead realised that the peak strength could not be accurately measured owing the progressive nature of ring shear failures. The potential sources of errors are as follow:

friction transmitted between confining rings and top platen by extruded soil, side friction between the sample and confining rings, tilting of the top platen to nonuniform stress distribution, soil extrusion and inaccuracies in force measurement.

If the test is run carefully then some the inaccuracies can be controled and have minor effect on the records or even can be ignored.

There are two ways of looking at the effects of soil extrusion from the ring; firstly it must be noted that the strain rate effect vanishes at low laboratory strain rates while the field strain rate of landslides in the UK are much smaller than the ring shear apparatus (see Table 3.1), therefore, it should not be an issue if the test is run at very low shear rates. Figure 2-4 show an example of soil extrusion in ring shear test.

Secondly, the residual strength is achieved often as little as 25 millimetres (The 12th Glossop Lecture Presentation, Bromhead, 2011) which relatively is not shearing for long time. Therefore, the soil extrusion which is resulted by shearing for long time can be under control during the testing.

In addition to those, the extrusion is highly depends on the texture of the ring and the platens and also the texture of soil sample. For example, the IC-NGI apparatus should not have any extrusion in principle, however, there is extrusion in practice.



Figure 2-4: This picture shows soil extrusion during ring shear test. This particular test, shown in the picture, was run for two days and under relatively high rate of shearing. The picture illustrates a small amount of extrusion.

Another example of the traditional worries is the imbalance between the load rings. If this happens there is a friction on the central pin. A sketch of upper platen of the small ring shear device (Bromhead device, 1979) is shown in Figure 2-5. The following equations show that the typical errors are very small.



Figure 2-5: Sketch illustrates plan view of the upper platen, Centring Pin and Torque Arms of Bromhead Ring Shear Apparatus.

If the shear stress (τ) is uniform, then the Torque is: $T = \int_{ri}^{ro} \tau 2\pi r^2 dr$

$$T = \frac{2}{3}\pi\tau [r_o^3 - r_i^3]$$

The torque is also is:

$$T = p_1 l_1 + p_2 l_2 = (p_1 + p_2) l_2$$
 in which $l = l_1 + l_2$

If $p_1 \neq p_2$, if suppose that $p_1 > p_2$

One of the sources of errors is the normal load which acts on the middle pin at its perifery of the pin. For equilibrium this is equal $p_1 - p_2$

Error in Torque = $(p_1 - p_2)\mu r_p$

Where μ is the friction coefficient of platen to the pin, which for steel on steel can be considered between 0.1 and 0.35. As there is some lubricant in the apparatus therefore it can be considered $\mu = 0.1$ for this case.

rp is radius of centring pin which is equal 5mm

If suppose that p_1 and p_2 are similar then we suppose that: $p_1 - p_2 = \frac{p_1}{2}$ then if:

$$T = (p_1 + p_2) \frac{l}{2}$$

 $T = (1.9 p_1 \times \frac{150}{2}) = 1.9 p_1 \times 75$

Then the error in torque is: $\Delta T = 0.1 p_1 \times 0.1 \times 5 = 0.05 p_1$

Therefore the error of the friction on centring pin, as the forces must be in equilibrium, is equal to $\frac{\Delta T}{T}$ which for this case would be:

$$\frac{\Delta T}{T} = 0.05 / (1.9 \times 75) = 0.00035$$

It shows that the error in the torque in the way that it is measured is very small. Even if the coefficient friction of steel on steel considered its maximum value (i.e 0.35) the error is still tiny and can be ignored.

The worst case of friction on the centring pin happens when the entire applied load P (which is $P = P_1 + P_2$) is carried on the one arm of the load ring. In this case, the centring pin must carry the same load; therefore, the error of torque due to friction is then:

$$\Delta T = P \mu r_p$$

So the error is $\frac{\Delta T}{T} = (P\mu r_p)/PL$

With r_p = 5mm, L= 75 mm and the coefficient friction of greased pin μ = 0.1 the error less than one percent.

2.8 Summary

If it is concluded that the measurement of residual strength in the laboratory is feasible and satisfactory in the ring shear test. Frictions are small in the apparatus and deemed acceptable by most workers. Errors due to calibration are comparable with those in other soil testing devices that utilise load rings. Rather more precision is gained where the load rings are read using linear displacement transducers, as they are more precise than dial gauges and respond better when the rings relax.

3 General discussion on the factors which affect the residual shear strength

3.1 Level of total normal stress

In laboratory measurements, both the peak and residual strength are dependent on the level of normal stresses, at which the sample is tested. When peak shear strength is measured over a range of normal stresses, at higher normal stress a higher value for peak strength will be obtained. This also happens at the same stage as for the residual shear strength. Commonly the relationship between the shear strength and normal strength is assumed to be linear. For limited ranges of normal stress, it is likely be adequate to consider it to be a straight line with intercept cohesion and slope the tangent of friction angle (see Figure 1-3).

For residual shear strength measurement Skempton (1964) stated that the residual cohesion is negligible. Many researchers employed this linearity while determining residual shear strength at the particular effective stress levels from the parameters of residual stress-strain envelope (Skempton, 1964; Bishop et al., 1971; Lupini et al., 1980).

Although Bishop et al. (1971) showed the value of ϕ'_r for low σ'_n stresses was determined under rebound conditions, Townsend and Gilbert (1973) argued that the uniqueness of the τ/σ_n' curves was questionable. They mentioned that it is possible to measure the same ϕ'_r under either increasing or decreasing σ_n' conditions. Therefore for the range of σ_n' values shown, ϕ'_r is independent of loading sequence. (Townsend and Gilbert, 1973). Bromhead compared residual shear strength of London Clay from alternative laboratory methods and back analysis (Bromhead and Curtis; Bromhead and Dixon, 1986; Bromhead et al 1999). Bromhead suggested that the residual shear envelope has a curved shape for the lower normal stress range (see Figure 3-1) (Ibid.) although this may be questionable (see chapter 8). Therefore there is an error when c is estimated from linear extrapolation.

25





Skempton (1985) also stated that the strain-stress envelope is non-linear and recommended that due to variation in the degree of curvature for different clays, the best-fit linear envelope can be used for design purposes. Stark and Eid (1994) also concluded that the drained residual strength failure envelope is nonlinear and this nonlinearity is significant for cohesive soils with a clay size fraction greater than 50% and a liquid limit between 60 and 220. Maksimovic (1996) concluded that the non-linear stress-strain envelope should be considered as the preferred method.

The analyses in this research follow the Skempton (1985) recommendation.

3.2 Shear strain rate

There are alternative views on the effects of shear strain rate on residual shear strength. These are summarised by many researchers (Lupini et al 1981; Tika et al., 1996; Taylor, 1998; Clarke, 2005). Lupini et al. (1981) showed that any increase in shear strain rate will lead to an increase in shear strength of the sample then when shear strain rate are reduced, some brittleness appears in the soil sample. The higher strain rates produce a particle oriented zone. When the strain rate is decreased viscous reaction forces and tension forces disrupt this oriented zone and lead to brittleness (Ibid.). Tika et al. (1996) investigated the state of the strain rate in high strain rate residual shear strength tests. They tested the natural behaviour, on the effect of strain rate and both positive and negative rate effects. Tika and Hutchinson (1999)

conducted ring shear ring tests to study the slip surface of Vaiont landslide. They applied a strain rate above 100 mm/min and the minimum fast strength measured was 60% below the slow residual strength. This demonstrated the existence of a negative rate effect.

Sassa (1995) designed a dynamic ring shear to study the residual strength under very fast strain rates. Sassa studied the dynamics of large-scale, rapid-motion of large landslide which triggered by seismic activity.

While using the small ring shear apparatus, there is threshold strain rate in which at strain rates below that the influence of strain rate is negligible. In a 100 mm diameter apparatus, the strain rate threshold was explained to correspond with the speed of 1° per minute. A much slower strain rate of 0.048° per minute is a convenient strain rate for the laboratory testing programmes, as stated by Tika et al. (1996). At a lower strain rate, most of cohesive soils are not sensitive. With reference to the three modes of shearing by Lupini et al. (1981), they spotted that, in the transition zone, there is an increase in residual shear strength when shearing speed increases. They noted that this is caused as result of distortion in the clay particles due to the fast shearing speed. Fearon et al. (2004) conducted some ring shear tests in which the water bath was full and some tests with no water in the water bath. The tests showed different answers, because in one case when the pore pressure reduces, in fact a new material is under shearing. Then water is used to swell the sample while in the other cases this effect does not occur. Therefore different strain rate effects were observed.

Residual strength is a remoulded soil property, measured under drained conditions. Moreover, it is a normally consolidated property of the soil, and a constant value property. As a result, it is not necessary or relevant to consider an undrained residual strength.

Table 3-1 shows Cruden and Varnes (1996) classification of landslides based on their velocities. As many landslides in the field are slow moving slides therefore, in terms of effective strain rate they are always several magnitudes slower that the laboratory apparatuses. For example, for testing a sample from a landslide which moves

27

25mm/year, acceptable results will be gained only while using a ring shear apparatus which has 25 mm/week strain rate, 50 times faster than the landslide if strain rate effects are negligible at this speed. Even the slowest ring shear tests are categorised in the middle of the moderate landslide velocity class. Consequently, this probably means that the strain rate is not a problem in laboratory measurements. Although, using a ring shear apparatus to measure residual shear strength enables us to gain full drainage at different shear rates and to study the influence of strain rate on the shear strength.

Velocity Class	Description	Velocity (mm/sec)	Typical Velocity	Example
7	Extremely Rapid		And Car	Vaiont Landslide
		5 x 10 ³	5 m/sec	
6	Very Rapid			
		5 x 10 ¹	3 m/min	
5	Rapid			
		5 x 10 ⁻¹	1.8 m/hr	
4	Moderate			First-time failure, Kent coast &
		5 x 10 ⁻³	13m/month	Typical slow speed ring shear test
3	Slow			Typical coastal landslide in Britain,
		5 x 10 ⁻⁵	1.6 m/year	The Roughs, Hythe
2	Very Slow			Undercliff, Isle of Wight
+	ļ	5 x 10 ⁻⁷	16 m/year	
1	Extremely Slow		and the second s	Sandgate

Table 3-1: Landslide velocity classes by Cruden and Varnes(after Cruden and Varnes, 1996).

Overall, it must be considered that even where the rate effect is important, it is usually effective at comparatively fast strain rate. All of the tests in this research project are applied at low laboratory strain rate.

3.3 Pore water pressure

Pore water pressure plays an important role on slope instability. The variation of pore water pressure can cause slope instability therefore in order to determine the factor of safety or the design of stabilisation projects, accurate pore pressure information is needed. In the shearing process it plays a very active role and can control the stress

path. The shear strain rate has a significant effect on pore water pressure reaction. At fast shear displacement the void ratio changes consequently the water content is changed. Any changes in water content directly affect the pore water pressure. Higher moisture content in the soil mass leads to more water molecules which expand the void space in between the soil particles as result this pushes the particles away from each other therefore. Considering that the air in void space of soil body is replaced by water which is less compressible. Therefore it is enabled to change the soil strength. It must be mentioned that the chemical composition of clays and water affects the obtained residual shear strength.

In back analysis, pore water pressure strongly affects the obtained results. Estimation of pore water pressure, or the piezometer line, can cause big errors. In high slope angle the errors are significant but in the low slope angles sometime are negligible. This will be discussed more in this thesis later.

The effect of pore water pressure on residual shear strength has been investigated by many researchers (Kenny, 1966; Idriss, 1985; Sassa et al., 1992; Shoaie and Sassa, 1994; Parathiras, 1994. Dixon and Bromhead (2002) studied landslides in London Clay. They installed a number of piezometers on coastal cliffs where to understand the role of pore water pressure which is caused by undrained unloading from slope formation. They performed a simple model to determine pore water pressures which is caused by degrading of slide mass. As a result, back-analyses can be conducted and residual shear strength of London Clay calculated. Thus, residual strength envelope at higher normal stress was defined using data from back analysis at different stages of degradation. As they put the piezometer tips at different depths, a real and accurate condition of pore water pressure was acquired. This data enabled them to perform one of the most reliable datasets published back analysis results for finding residual shear strength of clays.

3.4 Clay minerals

Clay size particles and mineral mixture plays an important role in the strength properties of soils. The effect of clay mineral mixture on the residual strength has been

29

broadly investigated since Skempton (1964) presented his theory of the post-peak drop shear strength of clays.

Moore (1991) conducted ring shear test on samples made of pure montmorillonite and kaolinite, to study the effects of clay mineralogy upon the residual shear strength of pure and natural clays. He stated that if sodium and calcium exist in the fluid in the soil body, the residual friction angle, ϕ'_{r} , varies by up 3 degrees. Moore concluded that residual strength should be considered as a dynamic property of clay because it is changed in response to environmental change (Fig. 3-2).



Figure 3-2: Variation of ϕ 'r with proportion of montmorillonite and kaolinite (after Yamasaki, 2000).

Many of these researches investigated the effect of one or two pure clay minerals in the soil mixture. In order to study the effect of mineralogical compositions on slope stability and estimate ϕ'_r while mineralogical compositions are known, Tiwari and Marui (2005 & 2003) tested more than 80 samples. They stated that smectite, kaolinite, mica, feldspar, and quartz are the major constituent minerals of shear surface. As clay at sliding surface soil is composed of the heterogeneous mixture of various minerals, therefore varieties of mixture of clay minerals with various proportions were examined. They concluded that this method provides less deviation from the determined ϕ'_r , as well as minimises the range of estimation error as compare to the other methods. Influence of Sodium Chloride on index properties and residual shear strength of natural soil, pure minerals and mudstone were investigated by Tiwari et al. (2004 & 2005). In 2011, Tiwari and Ajmera (2011a) conducted some laboratory test to find a correlation between *fully softened shear strength* and index properties in order to estimate residual friction angle of clays.

3.5 Clay fraction, plastic index and soil texture

It is generally observed that the residual shear strength decreases at higher plasticities. Plasticity itself may be the results of the clay activity and the clay size particles. As they are linked together, therefore it is not so easy to investigate their effect on the residual strength separately. Many investigations have been conducted to indicative correlation between clay fraction, and plasticity index on the residual shear strength. Skempton (1964) intended to find a correlation between residual shear strength and clay fraction. He postulated that there is a certain tendency for residual friction angle (ϕ'_r) to decrease when clay fraction, percentage of particles smaller than 2μ , increased. Since then it has been widely investigated by many researcher such as: Borowicka (1965); Chandler (1966); Kenny (1967); Fleischer and Scheffler (1972). They showed that when clay fraction increases residual friction angle decreases while brittleness increases. Kenny (1967) concluded that there was no satisfactory link between residual friction angle (ϕ'_r) and plastic index. Other researchers reported the discontinuous relationship between Ip and ϕ'_r (Vaughan and Walbancke, 1975, Maksimovic, 1989). Lupini et al. (1981) reviewed and summarised the previous research in this regard (Voight, 1973; Kanji, 1974; Seyček, 1978). They concluded that there is a rough correlation between ϕ'_r and Ip, an increase in Ip leads to decrease in ϕ'_r . Colotta et al. (1989) employed a new function which was called CALIP to correlate residual shear angle (CALIP stand for CF2*LL*IP*10-5). They emphasised the importance of clay fraction by squaring its value while calculating CALIP (Fig. 3-3).

Skempton (1985) concluded all the previous investigations in this concept and pointed out that when the clay fraction (in over-consolidated clays) is less than 25% the clay behaves much like sand and silt. Conversely when the proportion of clay particles is about 50% the residual strength controlled by sliding friction of clay minerals while further increase in clay fraction has little effect. Mesri and Cepeda-Diaz (1986) extended Skempton's research and stated that the residual ϕ' and liquid limit, and clay

31

fraction for pure clay minerals and natural samples present similar relationships to that shown by Skempton. They redrew the curve relationship of clay-fraction residual ϕ' and specified a zone on the graph which presents this relation more clear than on the chart by Skempton (1985) (Fig. 3-4).



Figure 3-3: (a) Relationship between calip and residual friction angle using DSBT , (b) Relationship between calip and residual friction angle using RST (after Colotta et al., 1989).

Stark and Eid (1994) aimed to find a correlation between Liquid Limit and csc friction angle ($1/\sin\varphi$). Tika and Hutchinson (1999) tested samples from Vaiont landslide and concluded that at the slow shear rate decreasing IP leads to increase in φ'_r and for the case fast shear rate the residual strength was founded to be independent of plastic index.



Figure 3-4: (a) relationship between residual friction angle and clay fraction (Skempton1985), (b) redrew the curve relationship of clay-fraction residual ϕ' by Mesri and Cepeda-Diaz (1986).

3.6 Soil texture

Chandler (1966) identified that the clay fraction along slip surface was higher than in adjacent soil, representing the breaking down of particles during the slip. In order to investigate the effect of soil structure and texture on residual shear strength, a number of soil samples from varying proportions of soil, which were rounded and platy grains, were tested by Lupini et al. (1981).

They showed that three types of behaviour are possible. Soil exhibits a comparatively little brittleness when the large proportion of soil mass is rounded grain. In oppose to this where the vast proportion of particles in soil body are platy, the soils displays comparatively high brittleness and therefore a lower residual strength. The main reason for this behaviour is the strong particle realignment of platy grains under shearing procedure (see Figure 3-5).



Figure 3-5: The sketches illustrate changes of soil fabric as a result of shearing. The fabric of soil before shearing, as it was deposited, and after shearing for different soils: granular soil material (single size), poorly-sorted and clay particle are shown. Single size soil appears unchanged after shear, in the poorly sorted granular soil, the larger particles are worked out of the shear band. Concentration and alignment of platey soil particles in shear zone for clayey soil are shown (bottom). On the left relationship between shear stress-displacement for different soils at various stages illustrated (after Lupini et al., 1981).

Lupini et al. (1981), presented a transitional behaviour for a range of proportions between these two extremes. It must be taken into account that these ranges of behaviour, which were presented by Lupini and his colleagues, may not be demonstrated in the natural soils as clearly as their tested samples. In general, the soil fabric changing and is a result of shearing therefore it is obvious that soils from preexisting exhibits different behaviour than the non-sheared soils.

3.7 Soil Chemistry

Orientation of particle at shear surface increase surface activity of clay minerals therefore many of them can be alerted by ion-exchange. For instance, the calcium and sodium of smectite minerals have fundamentally different residual shear strength. However, when sodium is replaced by calcium a significant change happens in the shear strength. On the other hand, many of the soil mineral components are chemically inactive deposits from the chemical weathering of the mother rock. The primary products of these procedures are silt and sand which almost show no response to the changing of the pore water chemistry. However, some of the resulted components demonstrate responses to the pore water chemistry such as clay minerals, salts, pyrites and calcite. Clay minerals are very sensitive to pore water chemistry. Many overconsolidated clays contain different minerals such as pyrites. They can be weathered and oxidised when the soil is exposed. Therefore any changes in pore water chemistry can cause changes in residual strength. Another mineral in the clay soils is calcite. Existing calcite in a soil leads to a high peak strength and therefore the soil exhibit high brittleness. Weathering destroys the calcite, then further drops in strength result (personal communication, Bromhead, 2010).

Kenny (1966) and Ramiah (1970) concluded that the effect of pore water chemistry is insignificant and the residual strength is less dependent on chemistry. Moore (1991) stated that if sodium and calcium exist in the fluid in soil body, the residual friction angle, ϕ'_r , varies by 3 degrees. He discussed the influence of exchangeable ions on residual shear strength. Moore and Brunsden (1996) studied the effect of pore water chemistry on residual shear strength of costal landslides. They concluded that the chemical properties of clay soils should be considered in study of mass movement. Their particular examples are described in costal landslides where the sea spray plays an important role in causing changes in pore water salinity. They concluded that low concentrations of ion pore pressure decreases the residual shear strength and vice

34

versa. However, that they did not consider in their study was, the effect of sewage of the houses at the upper part of the landslide.

The influence of sodium chloride on index properties and residual shear strength of natural soil, pure minerals and mudstone were investigated by Tiwari et al. (2004 & 2005). They concluded that smectite are more sensitive to NaCl rich pore water and shows high residual shear strength as compared to the other minerals. Tiwari (2007) reviewed the previous research on the concept. He studied the influence of chemical, mineralogical, and mechanical properties on residual shear strength of Tertiary mudstone.

3.8 Consolidation, lateral stresses and general stress history

In 1971, Bishop et al. introduced a new ring shear apparatus, at Imperial College of London. This machine can be considered as turning point in the field of residual shear strength investigation in the UK. Bishop et al. (1971) reported results of tests on the samples from five different soils. They tested many undisturbed and remoulded samples using the new ring shear apparatus and reported that residual shear strength are not depended on stress history of soil samples (Fig.3-6).



Figure 3-6: Peak and residual strength of undisturbed and remoulded soils using various methods of testing (after Bishop et al., 1971).

It is independent of sample disturbance as it is affected by the parallel orientation of clay platelets which is forced by remoulding in shear. There is a possibility of cementation bonding of clay particles at the surface of sand and silts, which demands existence of water and any other energy to breakdown this bond, they stated.

3.9 Discussion

Naturally any changes in soil fabric causes changes in the strength properties of the soil. When a shear surface is formed, in the field or in the laboratory, the natural soil fabric, which is represent deposition or weathering and other factors on the soil structure initially, is changed. The soil fabric is replaced by a new fabric which is dominated by effect of shearing. For example, in a clay soil, the shear zone contains a large proportion of platy clay particles which are oriented with their long axis in the direction of shearing. The water content in this zone is higher than in the adjacent soil. These processes have a strongly negative effect on the shear strength properties of the soil. A number of factors affect these processes such as: minerals and clay fraction percentage, rate of shear strain, level of normal stress and soil and water chemical composition are presented. These factors have more effect than other mentioned factors on the residual shear strength, however, other influencing factors such as machine design, sample size, disturbance of samples should be considered while discussing the residual shear strength in detail.

In general, factors that lead to a low laboratory ϕ'_r are:

- Smectite content (e.g. monmorillonite)
- Certain chemistries

and factors that lead to a higher laboratory ϕ'_r are:

- Kaolinite content (illite is 'middle band')
- Silt, sand and gravel content
- other chemistries

4 Background and context of back analysis

4.1 Introduction

This Chapter is based on a paper to the 11th ISL in Banff, Canada (Hosseyni et al., 2012b). The concept of factor of safety in reverse slope stability analysis was developed by Taylor (1948). Whereas a **customary analysis** starts from defined shear strengths and results in the computation of a factor of safety for a particular slope, a **back analysis** starts from the assumption that the slope is failing or has failed, so that its factor of safety may safely be assumed to be one. Then, from this assumption and an equilibrium analysis of a slide, it is possible to determine the average shear strength of the soil involved in the failure. The method treats a landslide as though it was a large field shear strength test, from which an estimate of the shear strength can be obtained, so that field-mobilized residual shear strengths have been determined and reported in the literature.

The back analysis technique was described by Chandler (1977). Although he did not originate the method, he stated some of the more important principles. The analysis itself is usually undertaken with the aid of a computer software package employing one of the numerous available limit equilibrium methods of analysis. In the vast majority of reported cases the analysis is undertaken on a single cross section through the landslide. Bromhead (1986) recommended the use of Spencer's (1967) and Morgenstern-Price method (1967), i.e. methods that take into account inclined interslice forces. Some techniques involved in the reduction of stresses from the analysis are given by Bromhead (2005).

In the following sections a variety of factors which affect the accuracy of the estimates of shear strength that are made using this technique are described. In extreme cases may call into question the validity of the analysis entirely. The method employs a single cross section through a landslide to determine operative soil properties or even ground water condition. In most cases there is only sufficient subsurface investigation data for one cross section through the landslide with pits or boreholes passing through the basal shear surface. The reliability of the outputs strongly depends on the accuracy of the cross section. Indeed, the delineation of the slip surface is often done with very few boreholes. Ground water pressure conditions for back analysis come from a variety of methods, including field measurement with piezometers, or various forms of modelling. Another important factor for back analysis is the shape of the topographic surface. As the analysis itself is usually undertaken with the aid of a computer software package employing one of the numerous available limit equilibrium methods of analysis, therefore the computer software package used can in principle affect the obtained results.

Different authors write with satisfaction about the results from their analyses, and another group writes with dissatisfaction or cautions in the use of the method. There are clear reasons behind these different attitudes to the back-analysis approach and equally clear scenarios where satisfaction is likely or unlikely. Sometimes the problem results from lack of clarity as to what a back analysis can actually do.

Here are some critical questions on application of this technique to measure residual shear strength:

- a) Is the back analysis of a single case or of a collection of cases in the same material? In the latter situation, ideally the landslides span a range of effective stresses?
- b) Is the back analysis for a landslide in a single material (more than one material but where the range of properties is small, is equivalent), or does the failure involve two or more materials with significantly different properties?
- c) Is it required to correlate the results with small-scale laboratory testing?

When back calculation applied to a single case (a. above), which is possibly unique, the inaccuracies in the analysis predominate, and make the interpretation difficult. In the case of question c., it is of course important that the laboratory testing is appropriate, correctly done and correctly-interpreted.

A related problem to this; is to determine the operative shear strength at first rupture of a soil mass in a landslide. Skempton and Chandler (1975) were able to do this successfully for two types of British stiff clays (London Clay and Lias clay). However, in general, the first time failure is a complex problem involving progressive failure and multiple geotechnical materials, and their success was largely because they were dealing with a restricted range of soils. An important lesson to learn from this is not to attempt to obtain ϕ'_r from any single case: to do so could give a value from 14° at low normal effective stress to nearer 10° at high normal effective stress⁴.



Figure 4-1: A curved residual strength envelope.

Stretching the effective stress range at first seems to show a curved envelope, but as more points are added, some of the curvature melts out of sight in the 'error bars' and known inadequacies of individual data points. Curved residual strength envelopes are well-known (Fig. 4-1) (Tiwari et al., 2005; Stark & Eid, 1994).

In the following sections a variety of factors which affect the accuracy of the estimates of mobilised shear strength are described. The accuracy of these factors calls into

⁴ See section on London Clay landslides where this finding is criticised.

question the validity of the analysis entirely. Discussion here is limited to the back analysis of landslides, so that difficulty involving soil structure interaction is removed.

4.2 Topographic shape

There are at least three elements to this factor:

- a) The shape of the topographic surface
- b) The shape of the slip surface
- c) The shape of any internal interfaces between materials

The following discussion of factor (a) relates equally to factor (c).

Topographic surface

The shape of the topographic surface is usually well defined from terrestrial or aerial surveys but ground verification is essential as tree cover may lead to determination of the wrong levels. Reconstruction of the topography pre-failure can sometimes be approximate, especially for natural slopes with ancient slides, but is a lesser problem with the geometrically simpler shapes of infrastructure earthwork. In limit equilibrium based stability analyses, the vertical distance between ground level and slip surface is a critical factor in determining stresses, and errors in topography lead to errors throughout the analysis related directly to depth.

Two-dimensional analyses are usually done on a *principal cross section*, for which the position is chosen in the middle of a slide, and where it is orientated as far as possible in the direction of movement. Less commonly, multiple sections or 3D analysis are chosen. Clearly, the topography on the principal cross section needs to be representative of the slide as a whole, and the orientation of any cross-sections chosen for analysis relative to movement is critical in 2D analysis. Figure 4-2 shows the changes of cliff profile of Warden Point through the years of sliding.



Figure 4-2: Example of different cliff profile after landsliding, Warden Point, Isle of Sheppey (after Dixon & Bromhead, 2002).

Michalowski (1989) has considered the effect of curvature in plan on the three dimensional stability of the slope. An inward curvature provides lateral support but may constrain outward seepage and thus increase pore water pressures, resulting in lower stability. Conversely, an outward curvature may set off decreased lateral support with improved drainage of the sides of the landslide. A considerable literature exists on 3D methods, but the number of useful case records where 3D methods have been used in back analysis is almost zero (Bromhead and Martin, 2004).

Tension cracks at the head of a slip surface

Deschamps and Yankey (2006) raise the issue of tension cracks in analysis, and point out that in soils with high cohesive strength components a water-filled tension crack in the analysis can influence the results. Fell et al. (2010) voice the same opinion. However, where the soil is largely frictional, the calculated thrust in a tension crack may not be significantly different from the calculated active thrust from the uppermost part of the slip surface.

Slip surface – overall shape in 2D

The shape of the sliding surface is often determined from visual identification of slip surfaces in samples recovered from boreholes which penetrate into solid ground

Chapter4: Background and context of back analysis

underneath the landslide. In very shallow landslides, the visual identification is sometimes done in the walls of trial pits. Much more rarely, visual identification in-situ of slip surfaces is done in shafts and large diameter drill holes. These, and other methods of locating slip surfaces, are discussed by Hutchinson (1983). Commonly, there is very little positive identification for any landslide. When possible, multiple boreholes at any location make it possible to determine the local slope of a slip surface as well as its position (Bromhead, 1978) (see Figure 4-3). When not only the position, but also the local inclination, of the slip surface have been determined, this is rather better than just having a single fix on position. Ideally, boreholes need to be positioned along cross sections.

A good record of slip surface shape is given for the Selborne controlled slope failure by Cooper et al. (1998), where the landslide was sufficiently small for about half of it to be excavated away by means of wide trenches, and a continuous trace of slip surface location was established on the trench wall. Such large scale intrusive investigations in trenches were also done by Henkel (1956 & 1957) for failures in railway cutting slopes. It is quite possible that the section reported by Gregory in the 1844s (see Bromhead, 2004a) was dug out and logged in a similar way over a century before.



Cost pressures lead to having fewer boreholes and other excavations than is desirable.

Figure 4-3: Position of slip surface at Queen's Avenue Landslide, Herne Bay.

Slip surface – scarp, toe and in between

In a fresh landslide, the position of the head scarp clearly defines the extreme position of the slip surface. However, as a head scarp degrades, the initial breakout position and slope are lost. Then further failures may eventually hide the slip surface outcrop altogether. In contrast, the breakout of the slip surface at the toe of a slide is almost always hidden in a thrust zone unless the slip surface breaks out in the face of a slope (*i.e.* it is 'perched') in which case toe debris falls away and leaves it exposed. However, in some case the toe of slides is located under the sea, e.g. Beacon Hill landslide at Herne Bay (Bromhead, 1978).

Even where the visual identification of slip surface positions has not been possible, then the use of inclinometer instruments may permit the identification of these positions when the subsurface investigation has been completed. This depend on, there being sufficient movement taking place to deform the inclinometer access tube. If the movements are relatively rapid, then the inclinometer access tubing may shear off in between measurements before the position of the slip surface has been adequately defined, although an inclinometer tube can be plumbed to find the depth of blockage.

Projection of slip surface onto a principal cross section

In the case of investigation when the boreholes have not been located exactly along the principal cross section, it may be necessary to project the slip surface on to the section line. In such cases, although, a 3D analysis resolves the problem, rarely there is enough information available for 3D analysis. A reported exception is the Queen's Avenue landslide at Herne Bay (Bromhead et al., 2001). Investigations by borehole were done over a period of around 30 years. In this study position of the boreholes were located to be sufficient to enable the three dimensional shape of the slip surface for 3D analysis. This happy result was simply a matter of good luck, and the availability of suitable borehole location plans. In this time, the topography had varied considerably, but the slip surface position remained unchanged.

Where significant proportions of the slip surface are controlled in position by a weak bed then it is necessary only to identify this position by direct visual or instrumental means in a few of the boreholes as the existence of the slip surface can be reasonably deduced from the stratigraphic succession. It is then difficult to define precisely the curved rising part of the slip surface between the bedding controlled basal shear and the head of the landslide. The shape of this part of the slip surface has a great bearing on the magnitude of the active thrust that drives the landslide, but it occupies only a small proportion of the footprint of the landslide. Consequently, its effect and significance in a back- analysis needs to be determined by repeated trial analyses. The accuracy of a ground model may be compromised if it is not possible to locate boreholes and pits precisely along the principal cross section. It is then necessary to decide whether to project boreholes simply at right angles to the principal cross section, or whether there is sufficient information on the geological structure to project this information along the maximum movement. Projecting along the strike often appears rather random despite its being in principle more accurate, because the ground levels for the boreholes are not the ground levels on the cross section. A variety of factors may lead to this inability to locate boreholes precisely along the principal cross section. An example is where inclinometers of great depth need to be installed. The weight of the inclinometer cable requires vehicular access to the top of the borehole, or perhaps for reasons of land ownership and permission for access, boreholes simply cannot be installed in the most desirable positions.

Slip surface – 3D shape

It is commonly stated that neglect of end effects – *i.e.* the transverse curvature at the sides of a slip surface or 'ends' if one considers the slip to be similar to a cylinder – raises the factor of safety (Fig. 4-4). Therefore it leads to lower back-calculated shear strength from 5 to 30%. Skempton (1985) offers similar advice. However, almost

44

always such advice considers that the piezometric line is similar for these 'ends' as in the main body of the slip. If the ends do not have the same piezometric line, this conclusion can be misleading.



Figure 4-4: Bowl shape of a landslide which should be analysed in 3D (after Bromhead and Martin, 2004)

As in nature, bowl-shaped and transversely-curved slip surfaces are common, then it must either considered that the assumptions to be mistaken, or for nature to operate unconservatively.

To a large extent errors associated with the representativeness of the principal cross section and in the movement direction are removed if the back analysis is done in 3D (Bromhead et al., 2001; Bromhead, 2004a). In the case reported by Chandler (1979), an excavation made in a river bank released a large pre-existing slide which was unknown before it moved. The footprint of the landslide was larger than that of the excavation. The principal cross section included the excavation, and thus would have been unrepresentative. A sketch of this scenario is presented by Bromhead (2004b) (Fig. 4-4). Clearly, only 3-D analysis can handle this case. Similar localized excavation may be the result of gulley formation in the toe of a landslide, or irregular coastal retreat.

Slip surface – Multiple sections

Hutchinson (1969) suggested that the 3D effects could be accommodated by taking a weighted average of results from different 2D sections through a landslide. It later

became evident that some of the constituent analyses in his case were not of similar type (Hutchinson et al., 1980). In the absence of an effective 3-D analysis at the time, and in other places, the averaging of sections may be valid.

4.3 Piezometric conditions

For back analysis of a single slip surface, the *piezometric line* method of defining pore pressures is usually adequate. Clearly, the position of this must be based on field data: if it is not, the result could be worthless. Using the pore pressure r_u tends to damp out both highs and lows in the pore pressures, so that this method is usually a poor choice.

In 2D, it is important for the piezometers to be located close to the principal cross sections or the projection method might influence the result. Now there is no geological strike to provide an indication of the way to do this projection. Instead, the projection must be done at right angles to the direction of flow. It will only be an extremely rare circumstance where this is possible with any certainty.

Worse still, piezometers may have to be located outside the active area of slipping, with some doubt as to whether pore pressures are the same inside as they are outside of the slip. Piezometers are often destroyed by slide movements. Similarly, some parts of a landslide may simply be inaccessible, e.g. in the Beacon Hill landslide at Herne Bay, nearly half of the landslide was situated under the beach and therefore under the sea. Emphasising that the pore pressures in this area were probably overestimated, as a hydrostatic distribution of pore pressures under the sea was assumed, and later investigations at Sheppey in the foreshore (Dixon and Bromhead, 2002) showed extremely low water pressures in this area due to erosion and undrained unloading.

It is inevitable that pore pressure information is incomplete, and some assumptions need to be made, if only to fill in between the measurements. In the back analysis of the Sheppey landslides, the pore pressures were found to be strongly influenced by undrained unloading. It was then possible to reconstruct approximately the pore pressures by reference to the amount of undrained unloading that had occurred. This was done both for sections dated before the instrumentation was installed, and also after it had ceased to function as a result of the activity of the landslide (Dixon and Bromhead, 2002). This reconstruction of pore pressure leads to some errors in back analysis (Fig. 4-5). Reconstructed pore pressures on the basis of undrained unloading were also used by Bromhead (2004a).





Sometimes pore pressures are controlled by a nearby permeable bed which provides limits to pore pressure e.g. Bromhead (1978) and Hutchinson (1969).

An overestimate of the pore water pressures (i.e. piezometric elevation) underestimates the true Factor of Safety F, and this reflects in an overestimate of the shear strength in back-analysis and *vice versa* (Fig. 4-5).

4.4 Soil density

The density of soil is rather easy to measure. However, in many site investigations this property is not a priority. Figure 4-6 shows the error bars of the different soil density. The graph indicate that various density does not change the value residual friction angle.

Why this should be results from the common observation that many soils are approximately twice as dense as water, and the exceptions are normally extremely easy to detect, for example in the case of peat. Indeed, for some geotechnical purposes the density of soil is not particularly important. It is important in the stability of slopes. Where soils are lighter than the notional twice the density of water, the sensitivity to pore water pressures is increased and *vice versa*, and this reflects in the back analyses to some errors. Where the soil is 'frictional' in character (*i.e.* c' is negligible) and devoid of water pressures, the soil density for a single soil case is immaterial in the determination of ϕ'_r .





4.5 Soil zonation

A standard assumption of limit equilibrium analysis is that there is a uniform mobilization of shear strength around the entirety of the slip surface. This assumption is reasonably well justified if the landslide is moving slowly. Such an assumption does not mean that the shear strength or even the shear strength parameters are uniform around the whole slip surface. Consider the case of a bedding controlled compound landslide with a significant proportion of the slip surface running along a weak bed in the geological sequence (Fig. 4-7).

In such a case it is likely that the residual shear strength parameters operative along the weak bed are less than the parameters operative along the rising curved section of the slip surface as it approaches the head of the landslide.



Figure 4-7: Bedding controlled compound landslide running along a weak bed (after Gregory, 1844).

The difference between these two sectors may be small, for example in a comparatively uniform deposit where the weak bed is only slightly weaker than the surrounding material, or it may be large. An example where the difference is large comes from Barton on Sea (Barton & Garvey, 2011), where the basal bedding controlled shear surface is formed in plastic clays, and the heel zone developed through sandy gravels. A method for dealing with this effect is given in Section 5.2.

Deschamps and Yankey (2006) adduce related cases of a dam with a different foundation, and a fill on a different foundation. In the latter case, they discuss alternate toe positions – always a problem when slip circles are used, as this shows limitations of the analyst more than of the method.

4.6 Procedure of back analysis for a landslides

The back analysis technique has been employed with particular success in the determination of the residual strength of a number of landslides in what is thought to be the same general geology and that are slowly moving along pre-existing shear surfaces. Application of the back analysis technique is a method of taking the landslides cross sections and finding the operative shear strength for the factor of safety equal to one. Back analyses of landslides require deliberation on the following factors: the shape of the topographic surface (2D/3D), the shape of the sliding surface (in 2D or 3D), pore water pressures and groundwater level, soil unit weight or density

of soils, zonation into different materials, and the method of analysis and computer software used and finally; is F actually = 1 for the case considered?

The general approach here is to treat each analysis as a point τ - σ' graph (i.e. it is a single 'test specimen') – a best-fit line through the points lies well within estimates of error. While it is recognized that the data and results in individual cases are flawed, the value of the collection is greater than the sum of its parts. (see Bromhead, 2005, for methods of extracting average τ and σ' values).

A related problem is to determine the shear strength operative at first rupture of a soil mass in a landslide. An important lesson to learn from this is not to attempt to obtain ϕ'_r from any single case.

According to the uncertainties, which may happen in a back-analysis as described in the following sections, comprehensive back-analysis can be conducted according to the following procedure. The back-analysis procedure for pre-existing (progressive) landslides includes:

1) Visit the site and investigate to understand the surface and the subsurface conditions such as: type of soils and thickness of layers in order to have an idea about their shear strength of each layer, ground water level/pore water pressures, slope geometry, tension cracks, slip surface or features of slip surface and direction of maximum movement.

2) Find a representative cross-section which is located at the direction of maximum movement. The cross-section must present all the relevant information which mentioned in step (1).

3) Define the location of the weak layer based on the field investigation, slip surface and subsurface features.

4) Decide on the stability method and software for the back-analysis.

50

5) The value of mobilized shear intercept (c') is neglected during stability calculations to keep the design in the safe side

6) In order to determine the average shear strength of back analysis; consider the same shear strength for all the layers and run the stability analysis until the factor of safety equals approximately unity (FS \approx 1.0).

7) In the case of a bedding controlled compound landslide which the slip surface running along a weak bed in the geological sequence, the residual shear strength parameters along the weak bed are less than the average parameters operative on the slip as a whole. As this thin slide prone horizon is weaker than other parts of the slip surface, the whole slip back-analysis is an over-estimate. Therefore it is recommended that to consider smaller shear strength for the weak layer and run the stability analysis for variety of shear strength of the problematic/weak layer until the factor of safety equals approximately unity (FS \approx 1.0). The difference between these two sectors may be small, for example in a comparatively uniform deposit where the weak bed is only slightly weaker than the surrounding material, or it may be large. This issue discussed in Section 4.7.

8) In order to ensure the agreement between the back-calculated and laboratory results, compare the back analysis shear strength parameter (ϕ'_r) of weak layer and the upper layer with the results of laboratory strength testing on representative samples.

9) In the case of such reactivated landslides, compare the back analysis results with empirical correlations, if there are any.

10) If the back analysis results of residual shear strength are not in agreement with the laboratory measurement from the right place (or appropriate empirical correlation), this means there is an error in the back calculation. Therefore the whole process of back calculation must be repeated by checking all the input parameters.

11) The general approach here is to treat each analysis as a point on a τ - σ ' graph, when there are enough analysis (as least three points on the graph, i.e. three cross sections) best-fit line through the points to determine ϕ'_r is drawn (Fig. 4-8).



Figure 4-8: The graph shows how the residual friction angle can be determined from the results of three back analyses.

4.7 Discussion

While applying back calculation to measure shear strength properties of soils, there is a question that; is F=1 globally or locally? Occasionally, this problem presents itself. In a first time failure, the onset of detectable movements occurs with lower pore pressures than collapse for example; at Selborne, (Cooper et al., 1998); and at the Carsington Dam, (Kennard and Bromhead, 2000). At the instant of collapse, instrumentation ceases to operate in most cases, even if it is present in the first place. Further complexities result from no-uniform mobilization of shear strength as parts of the slip surface pass peak and the strength decreases to residual at different rates.

Clearly when a landslide occurs as a series of failures it is wrong and misleading to analyse it as though the whole failure occurred all at once. Some landslides 'creep' because of irregular and changing pore pressures in the landslide body or because slow erosion at its toe occurs. In the former case, the landslide does not move all at the same time, and back analysis assuming that it does would be incorrect. Hutchinson's (1987) exposition of the causes of large displacement, rapid, movement on pre-existing shears mostly concern landslides that have been reduced to F<1 by external agents. In most cases of reported back analysis, the arguments as to why F should be considered to be equal to 1 are essential.

The assumption of (any) cohesive behaviour also reduces the sensitivity to pore pressure change as a remedial measure.

While most modern limit equilibrium computer codes are found to give closely similar results, there may be slight differences. Computers are occasionally found to have faults, but for practical cases such errors can usually be ignored. Some of the factors above, if misinterpreted, will clearly lead to the 'wrong answer' as pointed out on several occasions, for example, by Duncan & Stark (1992).

Over all, the back analysis remains a valuable tool. While it has uses in forensic engineering, the corpus of data from systematic analyses of landslides occurring in a single geological unit remains the best way of identifying and understanding field residual shear strength behaviour. Most of the published data sets available come from the UK, and this technique could and should be adopted more widely. Analyses of single slides cannot resolve the balance of c' and ϕ' , and never will. Much dissatisfaction with the method can be resolved by (a) applying it correctly, and (b) comparing the results to lab strengths only when the latter have been correctly executed and interpreted.

5 Developments in back analysis, review of previous work, introduction to new methods

5.1 The problem of missing back analyses

There are a variety of reasons why published cross sections through landslides with detailed geology and position for the slip surface are then not analysed. Some of the reasons are as follows:

- The sections may pre-date development in soil mechanics so that the back analysis was not possible, for example the case of Gregory (1844).
- Sometimes the source paper is not about mechanics but is about geological interpretation. This may be the rationale of Barton (1973 2011)
- While the geology is known, pore pressure data is lacking, so that the analysis cannot be done in effective stress although some approximations may be made (e.g. James 1970).
- Reasons of commercial confidentiality or litigation in which case the section may come into the public domain later when a report is disclosed, but without any interpretation.
- A single back analysis of a relatively unique case may not be of great value to the wider geotechnical community for example the cover photograph for the 8th ISL (Bromhead, Dixon & Ibsen, 2000) is of a landslide in the only sizeable coastal outcrop of the Nothe Clay (Corallian) in whole of the UK.

5.2 Method of dealing with weak layers

When a landslide occurs where a part of the failure occurs along a weak bed, that part of the slip surface post-failure consists of the upper section of the weak bed sliding over its lower section. Some material, not originally part of the weak bed, slides over

Chapter5: Developments in back analysis and introduction to new methods

the lower part of the weak bed. In this location, a smear of the weak bed may be carried by the landslide mass, (as in a ring shear test), and the properties of that contact are likely also to reflect the residual strength of the weak bed. In the back analysis, a residual angle of shearing resistance ϕ'_{r1} is assigned to all of this material, representing the strength of the weak bed. The remainder of the slip surface, or 'back slip', may be composed of a variety of different materials sliding over each other (see Figure 5-1). It is impossible to resolve many components of this, so a lumped parameter ϕ'_{r2} is assigned to this material. In the analyses of the Folkestone Warren landslides (Hutchinson, 1969; Hutchinson et al., 1980) the 'back slip' part was considered to have the properties of remoulded chalk.



Figure 5-1: shows a simplified landslide with a bedding controlled segment, roughly based around the Barton cases. ϕ'_{r1} : represents the slipping through the weak clay & ϕ'_{r2} : represents the slipping through the strong clay (the back slip).

When a back analysis is conducted with the above two parameter assumption, the following conditions are possible:

If $\phi'_{r2} = \phi'_{r1}$ then the landslides is treated as homogenous and the calculation produces ϕ'_{rav} .

Similarly, if $\phi'_{r2} = \phi'_{r1} + \varepsilon$, (when ε represents a small angular difference), as it is in many of the London Clay slides (and some of Chandler's Lias slides), then the analysis can also be conducted assuming a single material without significant loss of accuracy
$(\phi'_{r2} \approx \phi'_{r1} = \phi'_{rav})$. Up to a point, this explains why the London Clay dataset has proved a good one, as although $\phi'_{r2} > \phi'_{r1}$, the difference is small.

However if $\phi'_{r2} >> \phi'_{r1}$ (by an appreciable amount as it will be if the geological sequence contains sand, gravel or limestone) then the average mobilised angle of shearing resistance (ϕ'_{av}) determined for the slide as a whole will be misleading. For example landslides in Barton Beds and the landslides in Gault and Chalk at Folkestone.



Figure 5-2: This graph shows results of ϕ'_{r1} vs. ϕ'_{r2} and the limitation and boundaries of the ϕ'_{r1} and ϕ'_{r2} . Rule1 and Rule2, as explained in the text, are shown on the horizontal axis.

To determine the likely range of ϕ'_{r1} , first $_{av}$ (= ϕ'_{r1} = ϕ'_{r2}) is calculated. Then the analysis is run by considering a higher value for ϕ'_{r2} and calculating the residual value of ϕ'_{r1} that gives F=1. The different values of ϕ'_1 are plotted versus ϕ'_2 as shown in Figure 5.2. When results of back analyses for at least three cross sections are plotted on the same graph, then the range of residual friction angle of the weak layer ($\dot{\phi}_1$) can be defined. The procedure is described in the following (two will do but three or more is better).

Typical results of this type are shown diagrammatically as a graph of ϕ'_{r1} VS. ϕ'_{r2} for three cases presented in Figure 5-2. The points defining each line have been obtained as several $\phi'_{r1} - \phi'_{r2}$ pairs that yield F=1 as explained above. These were found by trial and error.

Three exclusion zones are shaded in Figure 5-2. The first exclusion zone indicates the case of $\dot{\phi'_{r1}} < \dot{\phi'_{r2}}$ which is irrelevant because the solution cannot lie within this zone as it would make $\dot{\phi'_{r1}} > \dot{\phi'_{r2}}$ which means the basal shear plane is stronger than the back slip (improbable).

The next exclusion zone, which is named Zone 2, is the zone in which the back slip cannot be stronger than loose granular material. The location of this band depends on the cases. For example it is identified $\phi'_{r2} > 30^\circ$ (or perhaps 35°) as an indicator for sandy gravel in the back slip materials of landslides in the Barton Beds.

Moreover, some low value of ϕ'_{r1} will not solve with the Morgenstern-Price procedure. The exclusion Zone 3 indicates this band in which the Morgenstern-Price will not solve. This is easiest to understand if the basal shear surface plane dips toward the toe of the slope with $\alpha \ge \phi'_{r1}$ (α is the slope angle) when it is not possible to have a limit equilibrium solution without internal tension (Bromhead and Hosseyni, 2012).

In order to define range of residual friction angle of the weak bed, two equivalent rules to these exclusion zones must be followed. Firstly, the maximum possible value for ϕ'_{r1} is the lowest value of ϕ'_{rav} which found among the back analyses. The ϕ'_{r1} cannot be more than that ϕ'_{rav} otherwise it would make some analyses fall into the exclusion Zone 1 (see Figure 5-2).

The other limit for the value of ϕ'_{r1} is the highest value of ϕ'_{r1} which makes ϕ'_{r2} unacceptable. In other word, the plot in Figure 5-2 goes through the edge of the

exclusion Zone 2. If ϕ'_{r1} lower than this value then ϕ'_{r2} for some of the sections has to be too high which falls into the exclusion Zone2.

The indication of these two bands on the horizontal axis of the $\phi_{r1} - \phi_{r2}$ graph shows the minimum and maximum value of ϕ_{r1} for a set of back analyses.

5.3 Third-party criticism of the back analyses approach

The literature contains examples where the back analysis technique has been used inappropriately or interpreted incorrectly. For example Gould (1960) back analysed slides along the Californian coast, mixing up old slides, new slides, slides in different materials etc. As this preceded Skempton's Rankine Lecture, perhaps he cannot be blamed for this muddle.

Deschamps and Yankey (2006) described two cases, one in an earth dam on a foundation with different properties. They drew a cross section of the Grandview Lake Dam with good data of the location of the slip surface and the ground water measurement in the compacted glacial till (see Figure 5-3).



Figure 5-3: Cross section of Grandview Lake Dam with assuming rupture surface and Piezometric line (after Deschamps & Yankey, 2006).

They criticised the results of back analysis when compared the results with the laboratory test results (see Figure 5-4). The laboratory tests were done by two laboratories. It is not known whether the samples were truly representative, whether

Chapter5: Developments in back analysis and introduction to new methods

the test procedures were correctly executed, or even whether the results were correctly determined. Moreover, the stress range and 'goodness of fit' of the $c - \phi$ lines are unknown, with the results plotted not as points, but as envelopes.

If the Deschamps and Yankey dam is re-analysed and the τ - σ' pair plotted on the graph of the laboratory test results, it fall close to the middle of the laboratory data lines. However, treating the problem as having two zones, and calculating a τ - σ' pair for each zone. It is found that the horizontal part of the slip surface has higher stress than the 'back slip'. As it is improbable that the slip surface would follow the bedrock junction (with ϕ'_{r1}) unless $\phi'_{r1} = \langle \phi'_{r2}$, then the equivalent to Figure 5-2 is to raise the τ for the back slip (increasing ϕ'_{r2}) or to lower τ for the bedrock junction (lowering ϕ'_{r1}) (see Figure 5-5). The σ' values hardly change at all.



Normal effective stress (o') -----

Figure 5-4: Results of triaxial tests on samples from Grandview Lake Dam. The Red and blue coloured points show the results of analyses when the slope treated as two different materials.

Clearly, there is a lower bound for $\dot{\phi}_{r1}$ defined by its residual strength, and all the upper bound for $\dot{\phi}_{r2}$ based on $\dot{\phi}_{r2} = 30^\circ$, and thus a feasible range of values for each can be determined, although there is not enough data to determine precise values for either. Analysis of multiple sections might have benefited this interpretation.

What is clear, however, is that it is not the back analysis that is at fault because even excluding that from consideration it is impossible to see which of the test results is relevant.



Figure 5-5: Results of analyses of the slope as e homogenous material and treating the back slip and bedrock junction as two different materials.

5.4 Discuss when it is absolutely vital to use 3D

As described above (Section 5.2), two-dimensional analyses are the norm, and they are usually done on a cross section more or less through the centreline of a landslide, on what may best be described as the *principal cross section*. The main problem in using this section is by definition it is likely to be the deepest section possible through the slide, and certainly it is not representative of the sides or flanks of the slide where the normal stresses may be very much lower. As a result, the stresses indicated from a back analysis will be overestimates.

In order to minimise such effects, three dimensional analyses can be employed, but there is a paucity of validated computer software, and the techniques are uncommon. Three dimensional analysis is certainly called for if the movement direction of a landslide is not straightforward (see, for example, Bromhead and Martin, 2004), or if the piezometric conditions vary laterally in the slide making the principal cross section very unrepresentative. Similarly, if the slip surface shape is highly irregular, in principle only 3D analysis can resolve the stresses. It is arguable that the Herne Bay coastal landslides show 3D effects that might be interpreted as a two zone problem. This is discussed in Sections 8.4 and 8.8.

WinSlideXtra is a software application that performs slope stability analyses and is configured for the Windows family of 32-bit operating systems. This work has only used a subset of its facilities, primarily the analysis of factor of safety in multi-zoned soil slopes using the Morgenstern-Price (1967) procedure on general slip surfaces. WinSlide (Bromhead, pers. comm., 2009) computes the mobilized shear and normal stresses acting in each soil that the slip surface cuts through from the equilibrium equations for each slice. This software is essentially the same as was used by inter alia Bromhead (1978), Hutchinson et al. (1980) and Dixon and Bromhead (2002) for back analysis, although then it was implemented in a non-graphical, mainframe or DOS PC version.

The Morgenstern-Price procedure used for the back analysis permits the interslice forces to be varied between analyses by means of a user-supplied function (known as f(x)) and the computation of a parameter known as λ . Taking f(x)=1 usually produces good results for 'real' slip surfaces and implies inclined, but parallel, resultant interslice forces (Bromhead, 1992). The WinSlide program permits the f(x) function to be varied, and alternative solutions to be obtained. It has not been found necessary to do this to obtain convergence, and it provides an additional complicating factor for very little gain, as the global stresses are altered only very slightly between equally acceptable solutions with different f(x) distributions (Bromhead, 1992).

5.5 Conclusions

The success of the London Clay back analyses is more than anything due to the simplicity of the geology in London Clay slopes. Where more than one soil is involved, then the complications develop rapidly. Use of the procedures in Section 5.2 has been applied in the following chapters.

61

6 Residual strengths in landslides in the Barton Beds (Upper Tertiary)

6.1 Introduction: Geographical and geological setting

Coastal cliffs formed in the Barton Beds (Upper Tertiary) occur on the UK mainland in Christchurch Bay, (Hampshire Basin), and on the Isle of Wight, most notably in the northern parts of Alum Bay and Whitecliff Bay, but not in the London Basin. Some landslipping may have taken place inland of the coastal cliffs.

It is believed that there have been no scientific or engineering studies of coastal (or other) landslips in Barton Beds on the Isle of Wight, but the Christchurch Bay coastal cliffs have been studied in relation to coastal protection and stabilisation works, and as part of a long-term research project undertaken by Dr M. E. Barton of Southampton University with students and co-workers. Related inland slopes on the mainland are believed to have been studied, but the details are unpublished.

Dr Barton's investigations have been published at various times since 1973. They have produced a number of surveyed and investigated cross sections of the cliffs together with small amounts of field and laboratory data, including shear strength and classification tests. No systematic back analyses of these sections have been published to date.

Location

In view of the ready availability of Dr Barton's data, and the absence of data from elsewhere, this study considers only the coastal outcrop in part of Christchurch Bay. This outcrop is divided into three: west of the stream-cut valley of Chewton Bunny, where the cliffs were stabilised in the 1960s, a description of the landsliding was published by Barton (1973). East of Chewton Bunny, extending eastwards about 2.4km to the limit of the Barton Clay coastal outcrop, the cliffs can be further subdivided into

a 1.4 km length of unprotected cliff line seaward of the Naish Farm Holiday Estate (Barton & Coles, 1984), with the remainder again stabilised by regrading and drainage extending across the eastern end of the Barton Clay coastal outcrop (see Figure 6-1). Investigations for this thesis have been made east of Chewton Bunny, with sampling and testing of exposed slip surfaces in the unprotected section of cliff (best represented by the 'D zone' of Barton et al., 2006), and back analyses done for four critical failed locations in the stabilised section described by Barton & Garvey (2011).

A sketch map, showing important place names, is given in Figure 6-1. The length of the selected area is between National Grid References (N.G.R.) 421750 to 424000.



Figure 6-1: Location of study area in the whole map of the UK as well as in outcrops of Barton Clay. The area includes about 1400 m unprotected cliff line in frontage of Naish Farm Estate at Highcliff in the west and to Cliff House Hotel in the east and about 1000 m of stabilised slopes from Cliff House to Central Amenity Area including stable and unstable engineering works.

Geology

The Barton Clay is an over-consolidated, stiff-fissured clay in the Eocene sequence. It has a 46.4 metre thickness and in the study area is a continuous unfaulted sequence which is overlain by Barton Sand and Plateau Gravels (Barton, 1973). It dips at approximately 0.75° to the ENE (Barton and Garvey, 2011). The angle of this dip is enough shallow to be considered approximately horizontal in a cross section normal to the coastline, but with a readily-discernable dip along the outcrop from west to east.

Burton (1933) established zonal sequences of Barton Clay Formation according to their included fossils, labelled A-G and described in Figure 6-2. This zonation is the one used by Barton, and it will also be followed here. There are several important strata in the sequence which act as marker beds.

	H G	Chama Sand Formation
57	ħ	Mainly dars grey clay
5,9	Fi	Dark grey day with small shell rich lenses
2	0,0	Dark grey, very silty day: the "earthy bed"
7.5	D	Upper part: green glauconitic, very sandy, silly clay with scattered fine grave!
>		D Horizon dark chocolate brown seam
38	000	Green glauconitic, very sandy, silty clay with scattered fine gravel
15	В	Lenses of fine sand in grey slity clay
2.7	A ₁	Grey clay with numerous thin sand seams
92	Az	Greenish grey glauconitic, laminated fine sandy, silty clay
2.9	A ₁	Brownish grey laminated silty clay
3.4	A _o	Green glauconitic, sandy, silty clay with scattered fine gravel
1.8	P.E.	Glauconitic very sandy clay with scattered pebble
		Bracklesham Group-Muderford Sands

Figure 6-2: The typical stratigraphical scheme of Barton Clay. The zonation was given by Burton (1933) and thicknesses, in metres, have been amended from Barton (1973).

Dr Barton established that the predominant mode of landsliding involves compound landslides with bedding-controlled basal slip surfaces in particular sub-horizontal beds. The most notable of these is near the base of the D Zone, and also towards the top of the F Zone (see Figure 6-2). However, west of the study area, bed A3 occurs above sea level, and it also contains a bedding controlled slip-surface. At the top of the sequence, the H Zone or Chama Bed has a high permeability with a spring line in the cliffs at its base.

Although extensive research in the Barton Clay has been performed since early 1930s, most of the detailed landslide investigation in the Barton Beds has been carried out by Barton since 1973 (viz. Barton, 1973; Barton & Coles, 1984; Barton et al. 2006; Barton & Garvey, 2011). Generally, low vegetation cover and well defined stratigraphy, together with a marked difference between the Tertiary beds and overlying drift, assist investigation of the various mass movement processes of the landslides at Barton cliffs.

The compound form of the bench shape landslides is sketched in Figure 6.3.

Stabilisation works

During the last century, marine erosion of the toes of slopes along the coastline of Hampshire and Dorset has been the cause of landsliding and coastal retreat, resulting in the loss of many houses and causing problems for local residents. Burton (1925), Robinson (1955) and Stopher and Wise (1966) investigated the complex sequence of changes in beach at Barton on Sea and Christchurch Harbour. The erosion of the undercliff toe was continuous until slowed by the construction of the engineering works in 1930s with timber groynes (Barton & Garvey, 2011). The shoreline protection may have been initially provided in the 1930s, but the major engineering stabilization was undertaken in the 1960s (Stopher & Wise, 1966). Engineering works in both the shoreline (protection of the cliff toe from erosion by the sea) and stabilization of the upper parts of slope by regrading and drainage were intended to avoid further failure at the top of the cliff (Wood, 1967 & 1971; Summers & Maddrell, 1978).

This involved some grading of the slopes, and the installation of around 1000m run of sheet piles, backed with a cut-off drain which were in the area of active landslide the scarp slopes contain exposures of solid and drift strata with 80-90° range in the Plateau Gravel scarps. The fronting height of cliff is about 35 metres O.D.

In recent years, four landslides have occurred which have destroyed the sheet pile walls and cut-off drains. These four landslides have reactivated parts of the former landslide system which was stabilized, and in combination, their total length of cliff amounts to approximately 46% of the total length of the filter drain (Barton and Garvey, 2011), representing a significant failure of the whole system.

Barton's studies

In 1973 Barton studied a bench profile at Highcliffe. Several shell-and-auger borehole and trial pits were performed in order to describe sub-layers of the Barton Beds using Burton's (1933) sequences. Approximate cross sections of the landslide system were provided and analysed. These showed bench-like landslide features with beddingcontrolled sub-horizontal basal shear locations in bedding zones A3 and D. Barton stated that deep seated rotational slips involving the complete cliff failure are not present.

A further cross section of landslide at Naish Farm area was provided by Barton and Coles (1984). The following conclusions were drawn; mud slides and debris slides are of minor importance, instead bench sliding is the most important degradation process, causing cliff top recession of 1.9m/year. As this area is East of that studied by Barton (1973), the A3 zone has disappeared beneath the beach, and a higher zone, F, containing a weak layer that is followed by the basal shear surface of a higher bench, is present in the cliffs.

An attempt to describe the effect of groundwater conditions and lithological boundaries on instability of Barton frontage was made by Fort et al. (2000). They concluded that, although construction of coastal defence had arrested the coastal erosion, pore water pressure is still recovering from stress relief in the clay members of the sequence. Thus, they concluded that if the remedial actions are not made, the recovered pore water pressure will result in progressive deep-seated failure. The nature of the weakness in the base of the D Zone was identified and discussed by Barton et al. (2006). They performed mineralogical and chemical analysis, scanning electron microscopic study of the microfabric and also ring shear tests for the slip surface near the base of the D Zone. This dark zone is slightly more clay-rich and has a marginally lower value of residual shear strength (ϕ'_r) as compared with the remainder of D Zone clay.

Garvey (2007) collected monitoring information over the past sixty years and produced a large and diverse dataset for landsliding at Barton on Sea, including results from: inclinometers, piezometers, rainfall records, topographic surveying, aerial and ground photography, boreholes and exposures logs and reports provided by consulting engineers together with newspaper articles.

In their paper Barton and Garvey (2011) describe four reactivated landslides in the eastern end of Barton Clay coastal outcrop. They examined the relation between the landslides and the stratigraphy. The presence of the known preferred shear surfaces, and the characteristic geomorphological modes of degradation were taken into account. Cross sections of these landslides were drawn, but not analysed. They stated that although the original drainage design reflected the influence of the stratigraphy, there was insufficient consideration of the hazards posed by the natural patterns of degradation and their geomorphological expression (Ibid.). Barton and Garvey attribute these failures to sliding along weak layers in the D, F and possibly H zones.

The cross sections by Barton and Garvey (Ibid.) are employed for back analysis in this study. The groundwater information was extracted from Garvey's MSc dissertation at University of Southampton (Garvey, 2007).

The unprotected cliff below Naish Farm Holiday Estate, was chosen for sampling in order to use in laboratory measurements. In this area the exposure of slip surfaces in both D and F Zones make the sampling possible.

67

6.2 Field investigation and observation:

This site is readily accessible from Kingston, and was visited in May and November 2010 and subsequently in July 2011 to:

- Confirm, where possible, that the author agreed with Barton's geomorphological interpretation
- Attempt to resolve some of the issues raised in Barton's papers
- Take samples from the basal shear zones in D and F for ring shear testing

The investigation mainly concentrated on failed stabilization works, from fronting of the Cliff House Hotel to Central Amity Area, and the unprotected coastline from Naish Farm to Tom's Garden which is known as the *Naish Farm Geological Conservation Area*.

Compound landslides provide a bench shape for the cliffs (Fig. 6-3) as stated by Barton (1973), with basal shear surfaces traceable along the cliff in very definite stratigraphic horizons.

The prominent basal shear surfaces lie in horizons D and F. Although horizon D is mainly below beach level in the eastern part of the study area, it is still shallow enough to be followed by basal shear surfaces in the western part of the area, i.e. Naish Farm Estate (Fig. 6-3). The identification of the D Zone is confirmed by the presence of C Zone nodules in the slope below the slip surface outcrop. Horizon F is the next higher horizon which has potential to act as a basal of shear surface. This horizon becomes more important in central and eastern parts of the study area. While Barton and Garvey (2011) state that the Zone H1 and the boundary of Zone H1-H2 are other horizons in which basal shear surfaces are likely to occur, this could not be confirmed in the field. The F Bench is relatively narrow in the Naish Farm area, whereas the D Bench is very wide (Barton & Coles, 1984), but east of the Cliff House Section the F Bench widens and the D bench narrows and disappears.



Figure 6-3: The photograph shows the view looking eastwards toward Cliff House, and shows the undercliff fronting the Naish Farm Holiday Estate in left and Tom's Garden in the middle and in the far right of photograph is landslide at Cliff House. The sketch illustrates the back scarp three benches, the upper and lower weak horizons, Zone F, Zone D and Zone A3, which dipping west to east are hosting the progressive transitional part of slips. The basal shear surface of the compound slides which is located along one of the preferred weak horizon is illustrated. It also shows slumping of the overlying scarp forms a colluvial bench above the preferred shear surface which is the characteristic mode of degradation of landsliding in Barton bed. The scarps are separated by colluvium which overlies the benches.

Barton & Garvey (2011) present the cross section of four landslides in the study area, which they named Cliff House, Hoskin's Gap West, Hoskin's Gap, and Central Area, from west to east (Fig. 6-5). Field reconnaissance confirms the identification of these, although the Central Amenity Area slide was subsequently regraded, and traces of it are not very clear. As the other three landslides have burst through the sheet pile wall, and the Central Amenity Area slide did the same, the traces of sheet piles still exposed at ground surface mark the extent of that slide.

The slip surface in the cliffs west of the Cliff House landslide also follows the same stratigraphic horizon, F bed, although Barton and Garvey (Ibid.) stated this shear surface follows the D horizon. The cross section of landslide at Cliff House are discussed and modified in Section 6.6.

The basal shear surfaces of the Hoskin's Gap, Hoskin's Gap West and Central Amenity Area landslides have occurred along bedding planes situated in the F and H horizons and boundary of Zone H1-H2 of the Barton Clay, but at different although closely spaced, stratigraphic positions.



Figure 6-4: Barton on Sea Hoskin's Gap West area undercliffs, overturning and failure of sheet piles and drainage system resulting from sliding along shear surface within Zone F. The slide combined with loss of support seaward as result of toe erosion and caused reactivation of mudsliding.

At the unprotected area of study there are two major causal factors for landsliding. The surface water, i.e from precipitation, penetrates through the upper more permeable layers, Plateau Gravel and sand, and accumulates over less permeable stratum, therefore the pore water pressure in the underlying layers increases. Hence developing pore water pressure increase instability of the slopes. On the other hand, sea waves attack and erode the undercliff and erode the toes of landslides which leads to unloading and allows further slipping and slumping.

At the eastern part of the study area stabilization works still operate well but in the central area, between Cliff House and Central Amenity Area, the drainage system and sheet piles have failed. Three very similar recent failures have disrupted the drains and sheet piles. Then the drains lost their functions therefore allowing a local increase in seepage, raised pore pressure which caused further bulging in sheet piles. When the clutches in the sheet piles failed, the resulting issue of water caused the soils down slope to turn to mudslides, and remove the remaining support from the sheet piles and cause further bulging (Fig. 6-4).

6.3 Reconstruction of piezometric conditions

Although Barton (1973) and Barton and Coles (1984) show cross sections through the cliffs of Christchurch Bay in the vicinity of Highcliff, which show 'bench failures' associated with weak beds in the Barton Clay in A3 and D Zones, Barton et al. (2006), in a study specifically of the weak layer in the D Zone, do not show any investigated sections, and it is therefore concluded that the latter work was based on the outcrop of the slip surface (which can still be seen in the field) rather than specifically on borehole investigations.

As a result, the only published section of a slide associated with either of these two weak layers is that of at Cliff House. The remaining published cross sections all cover slipping in the Zone F and higher zones.

Moreover, cross sections in Barton (1973) and Barton & Coles (1984) cannot be analysed because they lack piezometric information, and there are no nearby piezometers from which information can be extrapolated.

The best available cross sections were published by Barton & Garvey (2011) in which the location of shear surfaces are shown at Cliff House, Hoskin's Gap, Hoskin's Gap West and Central Amenity Area but they also lack piezometric data (see Figure 6-5). These sections also have no direct piezometric information at any date, but Garvey's dissertation has piezometric information for the slope as a whole that can be extrapolated to the failed sections, and therefore they can be back-analysed.



Figure 6-5: The map shows the study area, from Naish Farm to eastern outcrop of Barton Clay. The location of four cross sections used in back analysis and the location of sampling are illustrated. Samples are collected from slip surface exposure in F horizon Tom's Garden and in D horizon down of Naish Farm (after Garvey, 2007).

Garvey's (2007) piezometer records are available for two main sets of piezometers, one on each side of the Cliff House landslide. One set contains three piezometers, and the other four. The piezometer levels recorded in October 1993 were selected as the best case. All of the piezometer levels were projected onto the Cliff House section. In view of the highly approximate values obtained in this way, the back analyses have also been done using upper and lower piezometric conditions, respectively 1m higher and lower than the reconstructed piezometric lines.

Subsequently, the piezometric line was constructed for the other landslides by assuming that its position at the toe of the slope was the same for all sections, and that the position of the piezometric line was also the same beneath the crest of the cliff. This makes the piezometric line steeper for the narrower undercliffs. This piezometric line is named the *Average Piezometric Line*. The upper and lower piezometric lines were derived in the same way. These limits represent 'error bar limits' for the piezometric lines.

6.4 Back analyses

In each of the four slip sections shown by Barton and Garvey (2011), a ground profile labelled 1967 and representing ground levels before the stabilisation works, is shown. This, in combination with the indicated slip surface and the piezometric lines reconstructed as above, forms the basic dataset for the first series of back analyses, carried out using WinSlide (Bromhead, pers. comm., 2009) and the method of Morgenstern and Price (1967) using parallel inclined interslice forces. Analyses were repeated for the average, upper and lower piezometric lines.

Output from this program includes stresses along different parts of the slip surface. The analyses were done using various combinations of properties. In the first combination, all parts of the slip surface have identical properties, in the second combination the basal slip surface is assumed to be weaker than the rest of the slip surface, as discussed in Section 5.2.

With the reconstructed piezometric lines, and Barton and Garvey's section for 1967, estimates have been made by back analysis for the average residual strength of clay when the materials are considered uniform. Results are listed in Table 6-1.

After regrading and installation of the sheet wall piles and the drains, it should be possible to re-analyse the slopes, using the regraded profiles, because after all the stabilisation works the regraded profile did fail again. However, it is not possible to estimate the effects on stability of the sheet piles. Because in some places it is known that they do not penetrate into the basal shear surface but in the others they do. Under these conditions, the Factor of Safety is also greater than 1.0 to an indeterminate degree, so the back analyses are impossible. Also it is unknown how effective the drain was installed behind the sheet piles. Therefore no back analysis has been conducted for these regraded four cases.

Cross	Selected Ground Profile for back	d Ground Condition of GWL		Average Piez. Line	Upper Piez Line	
Section	analysis	Selected Slip Surface	φ'rav (°)	φ'rav (*)	φ'rav (°)	
Central Amenity Area	Cliff Profile of 1967	Slip surface of 1974	11.50	12.19	14.7	
Hoskin's Gap	Cliff Profile 1967	Slip Surface by Barton	11.23	11.75	13.6	
Hoskin's		Slip Surface by Barton	10.0	10.28	12.08	
Gap West	Cliff Profile 1967	Modified Slip Surface	9.8	10.8	12.6	

Table 6-1: Results of back analysis of cross section in Barton Bed for the ground profile of 1967 prior to stabilisation work. Average value of ϕ'_r for different ground water scenario are listed.

Ultimately, all four sections failed. After failure, it is clear that the sheet piles no longer contribute to the stability of the sections. In addition, drains are proved inoperative in those sections. Therefore the interpolated pore pressure, on the basis of Garvey's thesis (2007), seems to be relevant again.

The landslide at Central Amenity Area was stabilised in 1974-75. No movement has been reported since then therefore, the cross section of stabilised landslide cannot be analysed because at this stage as it is not known that the factor of safety is equal to unity.

The cross sections at Hoskin's Gap and Hoskin's Gap West and Cliff House for the landslides after stabilisation works (in 1960s) are considered for the new back analyses.

It also seemed from the inclinometer data in Garvey (Ibid.) that the slip surface position indicated for HGW was perhaps incorrect as it lies in H1 (Chama Sand), and this section was modified to a more understandable location. However, in the resulting analysis, results showed little difference. The analyses for the section at Hoskin's Gap West are repeated for the modified cross sections. Results of these analyses are listed in Table 6-2. The cross sections for back analyses are presented in Appendix A.

Table 6-2	: Results of	back analyses	of the	cross	section	of la	andslides	at Barto	n on	Sea f	org	ground	profile
of the pos	st stabilisat	ion failures.											

Cross	Selected Ground	Condition of GWL	Lower Piez. Line	Real Piez. Line	Average Piez. Line	Upper Piez. Line	
Section	Profile for back analysis	Selected Slip Surface	ф'rav (°)	ф'rav (*)	ф'rav (*)	ф'rav (°)	
Hoskin's Gap	Cliff Profile 2006	Slip Surface by Barton	12.3		12.9	14.8	
Hoskin's Gap	Cliff Profile 2006	Modified Slip Surface 1 (Upper)	9.4	•	10.7	12.7	
West		Modified Slip Surface 2 (Lower)	10		11.4	12.8	
	Cliff Profile Oct. 93	Slip Surface 93	19.1	20.6	20.7	22.5	
Cliff House	Cliff Profile Dec. 93	Slip Surface 93	15.2	16.4	16.4	17.2	
By B&G	Cliff Profile 2007	Slip Surface 93	14.8	15.8	15.8	16.4	
		Slip Surface 2007	15.5	16.8	16.8	17.6	

It was found that there is broad agreement between the indicated values of ϕ'_{rav} for three of the sections: CAA, HG and HGW. However, the cross section which differs most greatly from the others is that of the Cliff House with Barton and Garvey's indicated slip surface. This indicates that the slip surface by Barton & Garvey (2011) is unlikely. Since on the basis of site reconnaissance it was already rather clear than Barton & Garvey's (2011) interpretation of the slip surface at this location was most probably incorrect. As a result this section is reinterpreted in terms of two slips, one seated in the D horizon and the other seated at a higher level in Zone F. This is discussed further in Section 6.6 below.

Once these modifications were in place, further back analyses, summarised in Table 6-3 showed much closer agreement with the other 3 sections even the slip surface through the D horizon showed similar results. Details of failures of these four landslides have been described by Barton (with and without co-authors) since 1973. Date of failures and remediation works are summarised in Table 6-4 and a very brief summery is given as follows:

Cross Section	Selected Ground Profile for back	Condition of GWL	Lower Piez. Line	Real Piez. Line	Average Piez. Line	Upper Piez. Line	
	analysis	Selected Slip Surface	ф'rav (*)	φ'rav (°)	ф'rav (*)	ф'rav (°)	
Modified Cliff House	Cliff Profile October 93	Slip Surface 2007 - 1	10.3	12.3	11.3	13.5	
	Cliff Profile December 93	Slip Surface 93	12	12.8	12.8	13.4	
	Cliff Profile 2007	Slip Surface 93	9.5	9.6	10	10.7	
		Slip Surface 2007 - 2	9.9	13.3	12.2	13.3	

Table 6-3: Results of back analysis of modified cross section of the landslide at Cliff House

Central Amenity Area

The failure at Central Amenity Area occurred in 1974. This is a sensitive area and used to be under high toe erosion until the main stabilization works in 1968. In the cross section of this landslide, by Barton & Garvey (2011) the cliff profile in 1967 and 2006 and the shear surface of landslides in 1974 and 1975 are shown. For the back analysis of this landslide, the cliff profile prior to the stabilization works which is of 1967, corresponding shear surface is chosen (see Appendix A).

Hoskin's Gap

At Hoskin's Gap, Barton & Garvey (2011) presented two cliff profiles and one slip surface. The employed cross section for back analyses is shown in Appendix A. Although stabilisation constructed in 1964, movement are reported in 1980s. New piles with ground anchors were installed in 1989, however, subsequent movement were reported in 1990s. Detail of major stabilisation works and significant reactivation are presented in Table 6-4.

Hoskin's Gap West

The Cross section of Hoskin's Gap West by Barton & Garvey (2011) shows four small compound slip surfaces and the exposure of a mudslide channel to east line of the

Chapter6: Residual strength in the Barton Beds

section. The movements for drawing the slip surfaces are, the records of the I2 inclinometer and the sheet piles movements, so that one of the basal shear surfaces was recorded by I2 inclinometer and the other was drawn from the movement of the sheet piles. These data and the geometry of the cliff profile are used to modify the slip surfaces. The two modified slip surfaces and the cliff profile for the back analyses are illustrated in Appendix A. The stabilisation works and major movement are listed in Table 6-4.

Cliff House

For Cliff House landslide, Barton and Garvey (Ibid.) presented four cliff profiles of 1967, October 1993, December 1993 and 2007. Two deep seated slip surfaces for the landslides in 1993 and 2007 were drawn which extend down to the D horizon. Different cliff profiles and slip surfaces are considered for back analyses. Detail of the cross sections is presented in Appendix A.

Clearly, the reconstructed piezometric lines are probably most accurate for the case of Cliff House. It is almost certain that the drainage system is inoperative in the failed sections, as is clear from surface water seepages seen in the field. Therefore, the same level of piezometric line is used for these sections.

Location	Date of stabilisation works	Date of post- stabilisation failure	Date of re- stabilisation works	More information
CAA	1967	1974-75	1991 & 1994	No movement reported since 1994
HG	1964	1989	1990 & 1994	Significant reactivation 1996-96 & 2006
HGW	1967-68	1988	1990 & 2009	Significant reactivation 1993 & 1997- 98 & 2001 & 2005 & 2006
СН	1967	1973-74 & 1992-93	1990 & 1994	2000-2001 & 2007

Table 6-4: Date of major stabilisation works and significant lar	andslide reactivation for in Barton on Sea
--	--

Back analyses have been carried out for all of these failed post-stabilisation sections. Once again, using Barton and Garvey's (2011) indicated slip surfaces, CH is the outlier result, but using the modified slip surface positions yields a much better fit. A summary of all the back analyses results for different condition is given in Figure 6-6.



Figure 6-6: Results of back analyses show the average residual friction angle for different piezometric line as stated in Section 6.3.

In order to cover lack of piezometric data of the cross sections of the landslides in the Barton Bed. As stated previously, back analyses have been carried out with the variation in piezometric head. The analyses conducted at different piezometric levels, then the residual strength calculated. These piezometric heads were considered as Average, Upper and Lower as stated previously. Using different piezometric heads lead to different residual strength. Therefore the error bars of variation of residual friction angle were calculated.

The error bars calculated from the average residual friction angle which rely to the average piezomtric level. The error bars are based on +/- 1 meter change in piezomtric head. The error bars on the results are shown in the Figure 6-7.

Chapter6: Residual strength in the Barton Beds



Figure 6-7: The error bars of the results of back analyses for different piezomtric level are shown in this graph. The error bars show the errors from the average residual friction angle which calculated for average piezomtric level.

6.5 Weak layer analyses

When there is a number of evidence that the failure is a bedding controlled landslide then the existing of a weak bed in the slope must be considered in the analysis.

In fact for the case of Barton Bed, the basal slip and the back slip have very different properties. Because the back slip is made of a lot of sand and gravel, this is even shown in the Barton 1973 paper where the trial pit which he demonstrates a lot of slip mass at the higher levels is derived from the head deposits or Barton Sand.

In this case, for sandy and silty parts of the slip surface (back slip), residual friction angle is higher than the average, shown by ϕ'_{r2} , and for the lower part of slip surface, which are mainly clay, smaller residual friction angle is expected, shown by ϕ'_{r1} .

Firstly, assuming all the material has uniform properties, ϕ'_{rav} is calculated. Then a set of adjustments to try and gain a weak bed and stronger back slip are run. After that, by considering higher value for ϕ'_{r2} the residual strength of weak layer (ϕ'_{r1}) is calculated. Results of back analyses of four cross sections for every ground water condition are plotted separately. Results of back analyses for the average piezometric line is presented in Figure 6-8. From this graph, the range of residual friction angle of weak layer for the recent piezometric line is identified 8.9-10° as described in Section 5.2. Equivalent results for the other piezometric conditions are presented in Appendix A.



Figure 6-8: Shows how to define range of ϕ 'r1 for cross section of landslides at CAA, HG, HGW, MCH using Average and Real piezometric conditions. The narrow red lines show the range of ϕ 'r1.

With this proposal then, the results of HG and HGW and CAA come out looking very similar and Barton and Garvey's interpretation of Cliff House comes out looking again

different. The recent cross section is modified as stated in Section 6.6 and new back analyses conducted. The results are plotted in Figure 6-9.



Figure 6-9: Results of back analysis for the cross section of landslides at Barton on Sea. The graph shows average ϕ'_r and value of ϕ'_{r1} of the weak layer for higher friction angle of back slip (ϕ'_{r2}).

6.6 Modified Cross Section of landslide at Cliff House

As mentioned, the basal shear surfaces of the HG, HGW, and CAA are located along bedding planes which is situated in the F and H horizons (sic) of the Barton Clay, but at different although closely spaced, stratigraphic positions.

Barton and Garvey (2011) illustrated that the landslide at Cliff House is a deep-seated slide with its basal shear in the D horizon of the Barton Clay. Not only it is found that it would be difficult to agree with the Barton and Garvey (Ibid.) interpretation via back analysis (as shown previously), on geomorphological grounds the landslide does not look like a single deep-seated slide. Instead, it looks like a small upper compound landslide perched on the F horizon, combined with a lower small compound slide with a basal shear in the D horizon. This landslide is situated where the D horizon is just

below beach level and this makes the lower bench more active than east of the site, where the weak bed in the D horizon disappears below the foreshore, and more active than to the west, where the D horizon emerges from the beach and forms its own bench (see Figure 6-3). On the other hand, the graben feature in the Sea Lane pavement is about 10m or so wide, and this (via Cruden et al.'s very approximate 1991 correlation of graben width with slide depth) is more compatible with a slide seated on the F horizon, as H is rather shallow and includes Chama Sand.

In addition, the mode of failure in the sheet piling is identical with the Hoskin's Gap and Central Amenity Area slides (particularly the former), and the rotation direction of the piles (see Figure 6-10), together with their eventual displaced position, is incompatible with the slip surface shown by Barton and Garvey (Ibid.). The present position of the displaced sheet piles is compatible with a double-bench system.



Figure 6-10: Geological section of Cliff House Landslide with cliff profile 2007. Barton et al. designated that deep movement taken place in Zone D. Evidence of the site and direction of displaced sheet pile indicate that the current topography of the area is the results of two separate slips in D horizon and F horizon. The former happens as result of toe erosion and continuous toward the cliff progressively along Zone D. This movement confirmed by the direction of displaced sheet piles. Subsequent movement take place in Zone F as a result of recession of the back scarp of the lower slips.

While it was shown in the previous section that the back analysis does not produce sensible residual shear strengths for their deep-seated model, it does for a doublebench system with quasi-independent compound slides on the F and D horizons (see Figure 6-10).

In positioning a slip surface for a conjectured upper bench several difficulties are faced. Firstly, west of the Cliff House landslide, there is a clear bench feature that appears to be related to the F horizon. This is easier to see from a distance than close up (see Figure 6-3), as the outcrop of the slip surface is nowhere near as clear as the corresponding outcrop in the D horizon. The more sandy nature of the slip debris also helps obscure the outcrop. Nevertheless, it appears as though the basal shear for the whole upper bench is contained within one horizon. However, Barton and Garvey (Ibid.) show separate weak beds for the remaining 3 slides in the stabilised cliffs east of the Cliff House Section. Given the previous observations by Barton (1973) and others relating to the A3 and D horizons, it seems likely that their interpretation is not an accurate reflection of what has occurred in the ground.

6.7 Discussion of back analysis results:

Barton has surveyed profiles, slip surface locations and the shapes established from instruments and direct observations – sometimes during remediation and stabilisation projects. However, the piezometric levels are usually obtained from very few instruments within the landslides. In addition, the slides commonly are very laterally-extensive, and a three-dimensional effect must be present.

In the Barton landslides, there is quite clear evidence that the basal shear surface is much weaker than the back shear. The back shear clearly is influenced by the fact that there is Barton Sand and other granular materials, and as a result it has higher ϕ'_r .

Back analysis on the landslide at Cliff House by Barton & Garvey indicate some anomalous results, therefore this analysis is down played. The four datasets can be analysed independently or together. The presented results of back analysis for the

Chapter6: Residual strength in the Barton Beds

average ϕ'_r and the different ratios of ϕ'_{r1} and ϕ'_{r2} (determined in accordance with the procedure in Section 5.2 are plotted in Figure 6-6 and Figure 6-8, respectively. The best fit straight lines through the origin for each set have the slopes as shown in the figures.

The graphs show that when the back analyses are handled reasonably sensibly, the average residual friction angle for the landslide as a uniform material in Barton Clay is 11-12 degrees and for the weak bed is 9-10 degrees.

The results indicate that it is unlikely that the basal slip is in a variety of beds as stated by Barton and Garvey (2011). However, Barton and Garvey's interpretation of basal slip surface position 'wander' around in the F and H horizon. It seems likely that this identification some vagueness in location. Therefore it can be concluded that there are only two weak beds which are in the F and D horizons.

	Location basal shear	Slip surfaces by Barton	Location basal	Modified Slip surfaces by Hosseyni	
Cross Section	surface by Barton and Garvey (2011)	φ'r _{av} (°) at Average Piez. Lin	shear surface by S Hosseyni	φ'r _{av} (°) at Average Piez. Lin	
Central Amenity Area	F2	12.2	F	12.2	
Hoskin's Gap	F2/H1	11.7	F	11.7	
Hoskin's Gap West	H1/H2&F1/E	10.5	F	11	
Cliff House	Deep Slip through D	18	F&D	11.5	

Table 6-5: Location of basal slip surfaces for the landslides in Barton on Sea by B&G and modified slip surfaces with the relevant average ϕ'_r .

6.8 Conclusion

The coastal landslides in the cliffs of Christchurch Bay do occur along specific bedding horizons as noted by Barton et al. (2006), and in the Naish Farm frontage, the dominant one of these is in the D horizon. A less important horizon is in Zone F, but east of Cliff House the D horizon weak band plunges below sea level, and the landslides take place on the F or H zone weak beds. The weak beds are more closely spaced here.

In 1973, Barton stated that these cliffs do not contain deep-seated landslides, only the bedding-controlled "bench failures". Barton and Garvey (2011) do show a cross section that contains a deep-seated landslide. On the basis of field examination, as discussed and also described above, this latter interpretation appears to be incorrect. Moreover, the back-analyses of the indicated cross section result in inconsistently higher residual strengths for this section than any other back analyses. This is taken as supporting evidence for the opinion above. It is concluded that Barton was more correct in 1973 than latterly.

Barton and Garvey (2011) describe four failures of the 'stabilised' section east of Cliff House. All four appear to have followed a similar sequence, starting from a small failure in the rear scarp. This has pushed the slide along the weak bed in the F horizon, causing deformations in the sheet piles, and stressing the interlocks ('clutches') – to the point that they have opened. Simultaneously, the sheet piles which are not always firmly bedded in the underlying in-situ clay have rotated, allowing the cut-off drain to settle. This drain contains a galvanised pipe, which forms a low point and opens at its joints, collecting water that escapes through the opened sheet pile clutches, and which in turn destabilizes the steep slopes to seaward (see Figure 6-4). Lack of support compounds the failure in the sheet piles.

It appears that leaving the steep slope in the rear scarp (to maintain the grassy area at the cliff top) was a design error (Bromhead et al., 2012). The sheet piles might have performed better if they had a waling beam. However, the lack of any intervention to secure the system when it began to show distress is the reason why the initial failure has developed, in recent cases over a number of years, into such a systematic collapse.

The Cliff House section has the additional complication that there is also a failure on the weak bed in the D zone.

Unlike the cases in the London Clay, where any weak horizons in the clay are only slightly different from surrounding clay material, a consistent set of results are only obtainable for the Christchurch Bay sections by assuming different properties in different parts of each landslide. The details of the assumptions made, and properties used (including the range of values where appropriate) are discussed in the foregoing detailed treatment of each section.

As in so many cases, the back analyses are rendered somewhat imprecise due to uncertainties with the pore water pressures. These have been largely determined from Garvey's instrumentation results and observations, but it should be noted that his piezometers are mostly outside the area of the active slide, and piezometric heads have had to be extrapolated, interpolated or scaled to fit the failed sections. However, the locations of the main slip surface is to a very great extent dominated by the weak strata in the D and F zones, and this makes the errors due to incorrect slip surface shape and position rather smaller than in other cases in the literature (e.g. in James, 1970).

It proved difficult to sample slip surface material for laboratory testing from the sheared weak band in the D zone that was free from the underlying gritty layer. Inclusion of the gritty material in a ring shear test would significantly raise the measured residual angle of shearing resistance.

The weak band in the F horizon, although clearly visible at a distance, proved less easy to sample than that in the D zone.

Notwithstanding these difficulties, the range of residual shear strengths obtained in the ring shear tests, and from appropriately configured back analyses cover a broadly comparable range.

86

7 Field investigation and experimental programme of Barton Clay

7.1 General discussion on Identification of slip surface in the field

Hutchinson (1983) discusses methods of finding slip surfaces in landslides. Among the techniques which he discussed are those appropriate to boreholes, shafts, inspection in pits, instruments and using the geomorphology to indicate of the shape of the slip surface and the nature of the direction of the ground movement. However, the most important one of these is visual identification, where the slip surface emerges or daylights in the slope face. It is possible by means of visual inspection to identify where the slip surface is. A good example of this is the Barton's D Zone in the Barton Clay where during the summer month it is possible to walk around the site and observing the bench nature of the cliffs to identify an approximate position for the slip surface break out point. It is possible to do this in the distance and it is equally possible to do it close-up.



Figure 7-1: Direction of the D horizon is shown by the dash line and can clearly be recognised from the west of the cliffs.

The photograph of Figure 7-1 shows a middle distance photograph of outcrop of the slip surface in the D Zone. The photograph in Figure 7-2 shows a close-up when soil above the slip surface has been removed which showing the polished nature of slip

surface and brown staining on it which Barton et al. (2006) identified a being an important indicator of this particular slip surface.



Figure 7-2: The polished slip surface which includes with brown staining is shown in the photograph. The slip surface is located along the D Zone and includes fossils.

7.2 Samples used in investigation

On close inspection of the slip surface in the D Zone, it was found that the description by Barton et al. (2006) is a good one. The slip surface is very planar, the slip surface is polished, and it has brown staining rather than the brown uniform colour. In the Figure 7-2 it can clearly be seen where the fossil has been incorporated in the slip surface materials. There is a very slight fluting in the slip surface which is caused by dragging of hard particles like fossils or lumps of clay along the slip surface.

When taking a sample, a small disturbed block was taken, because it was only intended to perform disturbed tests on the clay. The sample was cut out with a mattock, or spade and knifes and was preserved by being wrapped in multiple layer of plastic bags for transport to Kingston University Geo-Laboratories.

Chapter7: Field investigation and experimental programme of Barton Clay

A variety of tests were done and it was observed that in fact immediately underneath the polished and remoulded sheared zone there was rather gritty layer. When the particle size distribution was done for the material in slip surface zone it was seen to contain a greenish coloured silt as well as the clay fraction.

Sampling was undertaken at a number of location described later in Section 7.5, and a suite of the tests were done where practical to the standard in BS 1377: 1990. Although as noted later, a variety of testing methodology were used for the ring shear tests, finally setting on a modified method of Kingston University procedure (see Section 7.4).

An attempt was made to discover the location of the slip surface in the F Zone. This was sampled in a like manner but it is nowhere near as well developed above as the slip surface in D.

Similar exercises were done in the Gault at Gore Cliff in the Isle of Wight and Folkestone Warren in the high liquid band at Copt Point, and also in the Lias above the Fish Bed at Lyme Regis. The results of these two current cases will be published elsewhere.

7.3 Testing methodology

Samples were taken provided from the shear surface and adjacent layers as described above. In order to check that the samples were good enough to merit the tests, the samples were evaluated in the laboratory. All of the routine sample testing was carried out by the Author and according to the British Standards, in the Geo-laboratory of Kingston University. The tests consist of moisture content determination, particle size determination, index property tests and ring shear tests. A brief on each test is given in the following:

Moisture content

The moisture content of in situ samples was measured by oven drying and weighing. The natural moisture content of samples from the F horizon in the Barton Bed indicates a higher value than the D horizon as this horizon is close to the spring zone.

Harris and Watson (1997) suggested that the ring shear test is stated at Plastic Limit water content. So, in order to know the moisture content of the sample and its changes during the ring shear test, it was measured before and after ring shear tests according to BS 1377 Part: 2 1990. The results are listed in Table 7-5.

Particle size distribution

In order to find percentage of fines, (clay, slit and sand), the particle size distribution determinations of the samples are carried out in accordance with BS 1377 Part 2 1990. The results of these tests are used to determine the Clay Size Fractions (CF) in the soil samples from the exposure of slip surfaces. The full results of the particle size distribution tests are not included in this thesis, however, value of CF for each sample is listed in the Table 7-5.

Index properties

Index properties of the samples were determined in accordance with BS 1377 Part 2 1990. These give the Liquid Limit (LL or WL) Plastic Limit (PL) and as a result the Plastic Index (PI) can be calculated. Index properties indicate the plasticity of the soils, hence their potential and susceptibility to develop low residual strength. See Section 3.4 on mineralogy effects. The results summarised in Table 7-5.

Residual strength measurement

Ring shear tests were conducted to determine the residual shear strength of soil which is sampled from the shear surface exposure.

The Bromhead ring shear apparatus was chosen for the ring shear tests as the Kingston University laboratory has several machines. Because of its simplicity, availability and possibility of simulation of landslides in the UK in terms of speed rate and particle size of material from in the slipped zone.

Bromhead (1979) recommended a test procedure when he introduced the simple ring apparatus. Stark and Vettel (1992) recommended four procedures for the Bromhead ring shear machine. Ring shear test procedure is also given in the BS 1377: Part 2:1990 for the small ring shear apparatus. This procedure is time consuming and difficult as well as some confusion in terms of the test is a drained or undrained test which is related to the BS Code number. Harris and Watson (1997) discussed the BS procedure and recommended a new simple procedure for the ring shear test. This procedure is known as the *Kingston University Procedure* in the literature. Testing procedures used in this research are based on this recommendation, however, it is modified according to the experience of the Author. This testing method is explained in the following section.

7.4 Application of Bromhead Ring Shear Apparatus

The ring shear tests are carried out to the modified simplified Kingston University procedure as per Harris and Watson (1997) technical note. The stages of the modified test procedures are as follows:

Sample preparation:

The first step is to remould the soil sample with distilled water until the complete destructuration of the original fabric occurs. When a soil sample is remoulded then it is adequate for residual strength measurement. If the specimen is sampled from the vicinity of the shear surface it represents the residual strength of that shear surface.

Sample preparation for the ring shear test has to be completed at a moisture content of the plastic limit or lower, because shear surface formation is a result of soil
Chapter7: Field investigation and experimental programme of Barton Clay

brittleness. The brittleness is more likely at moisture contents lower than the plastic limit. Plastic limit defined at the water content that shows the onset of brittleness, although a different definition of brittleness is need in the different cases.

The remoulded soil is then packed into the lower ring, which is a thin ring shape mould, with the fingers or provided plate knife. Then the excess clay has to be trimmed off from the container by using a plate knife. The sample is located in the sample container (empty water bath) and the top loading platen is placed on the sample. After that the lever loading arm adjusted to the upper ring on this. Then by using the coarse adjustment, the cross arm is brought into contact with the proving rings, so that both proving rings meet the cross arms simultaneously. The bath is then filled with water and the sample allowed to saturate fully. When the test set up, the water bath filled with distilled water. It must be noted that settlement readings may be taken but squeeze effects tend to make the readings valueless.



Figure 7-3: Bromhead ring shear apparatus, showing different parts, including: load hanger, proving rings, dial gauges, transducers, torque arms.

At this stage load is applied on the hanger. The hanger gives 10:1 lever arm on the loading beam. Before shearing starts, sample must be left for consolidation according

to the calculated consolidation time. Considering thickness and the soil samples the consolidation time is 15-20 minutes, however, the sample is left under consolidation for at least one hour.

Initial Shearing

It is necessary to select an appropriate rate of shear, before shearing can take place. A slow shearing rate, 0.048 degree/min, is chosen to complete the formation of this feature correctly. It is then sheared under a series of normal load. Figure 7-3 shows Bromhead ring shear device and its gear cog wheels which provide different shearing rates.

Once consolidation is complete, the machine is switched on for rotating the lower ring. It must be reminded that taking up 'Slack' in the system may take a considerable time which can result in a lag between starting the motor and readings being recorded. Careful setting up reduces this lag significantly. When constant readings are obtained, the motor is then stopped, and the gauges observed for a further 15 min. If the readings are found to drop substantially the shearing rate is too fast. In this case, it would be necessary to repeat the shearing at a slower rate.

Subsequent load stages

The load is increased by a further nominal amount. It is not necessary to allow the sample to consolidate under the new load as pore pressure dissipation is rapid and torque readings indicate whether there are excess pore pressures present or not. Then they would change as dissipation occurred.

The test carried out for more several normal loads. Amount of normal load depends on the size of landslides and low depth of slip surface.

The sample is unloaded and the apparatus dismantled carefully. The shear surface formed in the sample can then be examined.

Calibration of apparatus

The test can be run manually by reading the dial-gauges or collecting the data by data logger. In order to improve the accuracy of the tests results, all the test are data logged. Therefore, all the linear transducers have to be calibrated.

The data logger is includes software which allows data collection from each channel. Every channel is linked to a transducer which allows the data collection. Each transducer can be set for a range of 1000 points (intervals) using the software. Each one of the transducer is then put into the calibration procedure of the software. The software package automatically finds the range for the particular channel corresponding to a particular transducer. After that the calibration factor is identified using the gauges.

Data logger

The software package allows for each channel to be read simultaneously and recorded the data at the set intervals during the test procedure. The tests can be set up by choosing a particular channel for recording the data. The channel can be set on zero or any other numbers at the beginning of the test. The interval time can be set for any that the test might need to be recorded but the software can only record 1000 points at each time. Once the test is set up and channels are calibrated, then the test can be run and the procedure of calibration and the set up of the test do not need to be done again for next test. When the set up is completed, the test will start in 30 second and continue to data log the reading at the set interval time. When test is run the data logged for each channel can be viewed as a graph which allows monitoring the test procedure. The data logger is stopped when the test finishes. Then the collected data is downloaded from the memory of each channel separately. Figure 7-4 shows the ring shear, transducer, data logger, computer for the software package and their relation in the laboratory.



Figure 7-4: Shows link between the ring shear, transducer and the data logger.

The collected data is inputted into an Excel spreadsheet. The data is analysed and the graph of normal effective stress versus shear stress in order to determine residual friction angle of the soil sample under particular normal stress.

7.5 Sampling and Laboratory measurement of Barton Clay

It is difficult to make samples from the sheared weak band in order to laboratory measurement of slip surface material from the underlying layers. However, sampling from the landslides in Barton Clay at Naish Farm Geological Conservation Area is not that much difficult as the other sites. At this site, the exposure of slip surfaces from the frontage of compound landslides can clearly be recognised. The weak band in the F horizon, although clearly visible at a distance in the west, sampling is less easy than the D zone. A band of nodules was used as a guide to find the slip surface. The stratum has a dip from west to east so that the D Zone is beneath the sea level in the east, however, it is accessible in the west at frontage of Glenside Road and Bayview Road (see Figure 7-5). Despite the fact that the sampling from the basal weak horizon is

difficult, samples were taken twice in 2010 and 2011. Detail and locations from which each sample was taken, are presented as following:

Figure 7-5: The location of sampling from the exposure of slip surface in the F and D horizons in 2010 and 2011 are shown in the picture. Two samples from the slip surface in the weak bed in the D horizon in 2010 and 2011 are made, and one sample from the F horizon (Bing Maps).

Sampling from D horizon (The lower weak horizon)

Date: 17th May 2010

Location: Fronting of Naish Farm Geological Conservation Area; down of Glenside Rd

National Grid Reference: Eastings (X): 421927

Northings (Y): 093193

Latitude: 50.737939

Longitude: -1.6906393

NGR: SZ 21927, 93193

Sampling from D horizon (The lower weak horizon)

Date: 22nd July 2011

Location: Fronting of Naish Farm Geological Conservation Area; down of Bayview Rd

National Grid Reference: Eastings (X): 422025	Northings (Y): 093181
Latitude: 50.737830	Longitude: -1.6892445

NGR: SZ 22025, 93181

Sampling from F horizon (The upper weak horizon)

Date: 22nd July 2011

Location: Fronting of Naish Farm Geological Conservation Area; Down of Seaview Rd

National Grid Reference: Eastings (X): 422548	Northings (Y): 093153
Latitude: 50.737552	Longitude: -1.6818309
NGR: SZ 22548, 93153	

Experimental programme results

In order to check the samples are good enough to merit the tests, firstly the samples are evaluated. All the routine sample testing is carried out according to British Standards, as described in Section 7.3, in the Geo-laboratory of Kingston University. The test consist of moisture content determination, particle size determination, index property tests. Moisture content of in situ samples as well as before and after ring shear tests is measured. The results are shown in Table 7.5.

Chapter7: Field investigation and experimental programme of Barton Clay

In addition to the aforementioned tests, in order to determine residual shear strength of the samples, a number of ring shear tests using Bromhead ring shear apparatus are also carried out. The ring shear tests carried out as described in Section 7-4.

The two samples from the D horizon are tested in 2010 and 2011, separately. In order to see the variability of ϕ'_r for different normal effective stress, ring shear tests have conducted for different values of σ'_n . The results are presented in Appendix B. Examples of graphs of ring shear tests are shown in Appendix G. The graphs in Appendix G illustrate different stages of ring shear test using Bromhead ring shear apparatus including: consolidation and multi stage shearing under different normal stresses. The entire tests are carried out by the Author.

Table 7-1: Results of laboratory testing on the samples from F and D horizon of Barton Bed from the landslides at Barton on Sea.

Index properties of samples from Barton on Sea in 2010 and 2011											
Year	sample	Ring shear test	Location of sampling	Date of sampling	WL%	PL%	PI%	CF%	w % (Natura 1)	w % (Before R. S. test)	w % (After R. S. test)
2010		1	Fronting of Naish		64.1	15.6	48.5	62	38		•
	1	2	Conservation Area;	17 May 2010	63.7	15.2	48.5	61.7	38.1		•
	2	1	(Zone D)		65.1	15.5	49.6	60.8	38.5	•	•
		2	(NGR: SZ 21927, 93193)		68.9	16.3	52.6	59.5	37.8		•
	1	1	Fronting of Naish	22 July 2011	62.3	18.4	43.9	48.2	26.1	26.3	30.2
		2	Conservation Area;		64.5	20.3	44.2	47.9	26.06	23.5	28.4
		3	Rd		60.8	19.6	41.2	46.1	25.9	23	27.2
		(Zone D) 4 (NGR: SZ 22025, 93181)		63.7	18.9	44.8	46.5	25.8	26.35	30.25	
2011	2	1	Fronting of Naish Farm Geological		70	22.9	47	55.6	39.6	28.7	35.6
		2	Conservation Area;	22 July 2011	689	22.3	46.6	50.2	39	25.9	30.6
		3	Rd (Zone F) (NGR: SZ 22548, 93153)		69.4	21.9	47.5	52.1	40	28.8	32.97

Chapter7: Field investigation and experimental programme of Barton Clay

In order to measure the residual shear strength a graph of normal effective stress versus shear stress is plotted (Fig. 7-6). The result of each test is shown as a single test on the graph. Slope of the trend line of the data points on the σ'_{n} - τ graph represent the residual friction angle of the samples. It is expected that the power trend line demonstrates a carve shape which shows higher slope for the small normal effective stress and lower slope for larger σ'_{n} . The trend line in the Figure 7-6 proves such a tendency.

In order to compare the graphs effectively with results of back analyses, regardless of the value of the data in the graphs, the same scale has been used for the graphs in the following sections.



Figure 7-6: This graph shows σ'_n - τ for results of samples from the exposure of basal shear surface in the D horizon of Baron Bed. Four tests are carried out on the samples in 2011 and three on the samples in 2010. Locations of sampling are shown in Figure 7.5.

F horizon which is the upper weak bed in the Barton Bed is not visible at the far west of Naish Farm, however, it is clearly visible at the Tom's Garden. Samples are made from the weak bed near the Cliff House as shown in Figure 7-7. At this point zone of spring and fossils are the signs to find the F weak layer. Ring shear test are carried out the same as the samples from the D zone using Bromhead ring shear apparatus and results are presented in Appendix B.

Results of three tests on the samples from weak bed in the F horizon provide a data sets which allows a σ'_n - τ graph. The result are plotted in a σ'_n vs. τ graph and the power trend line is drawn (Fig. 7-7).



Figure 7-7: This graph shows σ'_n - τ for results of samples from the exposure of basal shear surface in the F horizon of Baron Bed. Three tests are carried out on the samples in 2011. Locations of sampling are shown in Figure 7-5.

7.6 Previous research

Barton et al. (2006) studied the D Zone of the Barton Clay. They collected the samples from this horizon and tested using Bromhead ring shear apparatus. The tests were

Chapter7: Field investigation and experimental programme of Barton Clay

carried out using the standard procedure while the rotation rate was 0.048 ° per minutes. The results extracted from the $\phi'_r vs \sigma'_n$ from the original paper and plotted in a $\sigma'_n - \tau$ graph as shown in Figure 7-8.

When the trend line is drawn through the data point the slope of the trend line indicates residual strength of the clay resulted from the tests. The result show higher value for the residual ϕ'_r when compare to the results of this research. The first reason for that is the testing procedure which is a complicated and according to the Author's experience it shows a higher value for the ϕ'_r . The normal stress value, water content before testing and time for rotation of the ring shear machine are the factors which affect the results.



Figure 7-8: Results of the Bromhead ring shear tests plotted as values of effective normal stress versus shear stress for the dark chocolate brown seam and the immediately underlying D Zone of Barton Bed (after Barton et al., 2006).

7.7 Conclusions

The bedding-controlled basal shear locations identified by Barton et al. (2006), are readily identified and sampled. Properties of the slip surfaces have been determined and are reported in the above sections.

Residual shear strength determinations have been made in the small ring shear machine. They correlate well with the results appropriately interpreted back analyses, notwithstanding that the latter relate to F beds, and the former to D and F beds. The residual strength of the D Horizon is too low to permit Barton and Garvey's (2011) interpretation of the Cliff House landslide, but it is supportive of the alternative interpretation made in Section 6.6.



Figure 7-9: Results of the Bromhead ring shear tests on the samples from the landslides at Bartopn Sea are compared with published results using same apparatus.

In review of this result, it is concluded that in this case, ring shear test have made it possible to correct a misinterpretation of the mechanics of failure in the affected cliffs. Moreover, there is a good degree of agreement between test and back analyses that must increase confidence in both.

8 The residual strength of clay in coastal landslides in the London Clay Formation

8.1 Introduction: Geographical and geological setting

The London Clay Formation is found in both the London and Hampshire Basins in southern England, with corresponding strata in northern France. Its coastal outcrop in the Hampshire Basin is very small (Whitecliff Bay and Alum Bay on the Isle of Wight) but it is much larger in the London Basin, especially along both sides of the Thames Estuary, where landslides are common.

There have been numerous scientific and engineering studies of the coastal landslides in the Thames Estuary (e.g. Dixon & Bromhead, 2002; Hutchinson, 1970; Bromhead, 1978). Coastal outcrops in the Hampshire Basin are very small.

In this chapter the coastal landslides are discussed. The inland landslides in the London Clay are presented in the Chapter 9.

Location(s)

The coastal landslides investigated are mainly located in Kent at Isle of Sheppey, Herne Bay and Beltinge. The sites are shown in Figure 8-1. This figure also shows all the considered sites in the next chapter. If the sites could not be precisely located, they are approximately within about 5km positioned on the map.

Geology

The outcrop of the London Clay Formation is shown in Figure 8-2, extends offshore from London Basin into the North Sea. Both basins are synclinal structures and their present configuration is due to mid-Tertiary compressional tectonics. The maximum thickness of London Clay is approximately 150 meters (King, 1991). In the Hampshire Basin, the London Clay Formation comprises predominantly clays and silts, with subordinate sand units. The base of London Clay Formation is defined by a sharp contact with the underlying non-marine sediments of Reading Formation. The London Clay Formation is overlain by the sediments of Wittering Formation.



Figure 8-1: Location studied sites in the London Basin and Hampshire Basin. Each point shows the location of the cross section for back analysis (after British Geological Survey).

The correct stratigraphic relationships of the Palaeogene outcrops in the London Basin and Hampshire Basin and were established by Prestwich (1846 & 1850 cited in King, 1991) who realised that the complete Eocene sections were exposed at Alum Bay and Whitecliff Bay, Isle of Wight. King (1981) defined several named members within the London Clay Formation in the Hampshire Basin. The lithostratgraphic classification of the London Clay Formation is summarised in Figure 8-2.

Stabilization works

Investigations of landslides are commonly an early stage in a slope stabilization scheme, for example, connected to a coastal protection and cliff stabilization,

stabilization of a slip in an infrastructure cutting, stabilization of an inland landslide for development.



Figure 8-2: Lithostratigraphy of the London Clay Formation in the Hampshire Basin, the Lithology highly generalised (after King, 1991). It illustrates the major lithofacies and the relationship of the named lithostratigraphic units to the deposition sequences.

The classic large coastal slope stabilization schemes in London Clay are those at Herne Bay and Minster in the Isle of Sheppey. They involved the certain amount of regrading and the installation of herringbone pattern shallow drains. Regrading was targeted at the deep slips, and herring bone drainage to shallow failures. These schemes are not complete without seawall construction, and sometimes formation of a beach to eliminate toe erosion. Smaller schemes were executed at various time at the Whitstable, Swalcliff and Westcliff to southend on the Essex side of Thames Estuary.

It is clearly impractical to re-investigate remediated slopes, and any deficiencies in those investigations cannot be reanalysed.

8.2 Previous work on coastal landslides

The coastal erosion along the North coast of Kent is more acute than that of South Essex, because of the different exposure to North Sea weather.

Landslides along the South Essex coast are noted by Hutchinson (1963), but apart from Hutchinson & Gostelow's work on the 'abandoned cliff' at Hadleigh, (Hutchinson & Gostelow, 1976) little or nothing has been published.

Herne Bay

East of Herne Bay, passing seaward of the village of Beltinge, the coastal cliffs rise to a height of around 40m (Hutchinson, 1970). A section of the cliffs adjacent to the town was defended from the sea by a sea wall and graded and drained, as described by Duvivier (1940). Over the next few decades, this stretch of cliffs was subjected to two forms of instability: shallow slippages at a high elevation in the cliffs, and slow deformation of part of the sea wall. It transpired that the latter was an ancient rotational landslide now known as the Beacon Hill landslide. In 1957, an MSc student (Wise, 1957) observed the head scarp crack of this slide and Hutchinson (1963) initiated a deformation monitoring programme for the slide. The slide was investigated with boreholes in 1969-70 (Bromhead, 1978), and a deep drainage system was installed some years later (Berkeley-Thorne & Roberts, 1981). The date of first failure of this slide is unknown.

Some distance to the east of the Beacon Hill landslide, another, slightly smaller landslide happened in 1896 seaward of the end of the Queen's Avenue, this giving its name to the slide. As it was intended to extend the seawall and grading through this landslide at various times after 1940, several sets of boreholes were drilled through it. It was investigated systematically with boreholes in 1969-70 (Bromhead, 1978). It was graded and stabilised by drainage at around that time.

Even further to the East, a larger landslide occurred in 1953 in the tea gardens of the Miramar Hotel (now a residential care home) in Herne Bay. This landslide was studied by Hutchinson (1970) and systematically in 1969-70 (Bromhead, 1978). Although it was graded and drained in 1969-70, it was not fully stabilised, and further investigation and sea defence work was undertaken subsequently (McGown et al., 1987). Old map evidence was adduced by Bromhead (1978) to support the hypothesis that there had been an earlier slide seaward of this location in 1883.

All the major landslides were found to have bedding-controlled basal shears in what is probably Zone A or B of the London Clay Formation (King, 1981 & 1991). As this is at a depth of 32m below sea level at the Beacon Hill landslide, its influence is slight, but it is significant for the Queen's Avenue and Miramar landslides, in the case of the latter giving its first failure a pronounced graben feature, and, where the base of the London Clay Formation rises above sea level, the character of a 'bench slide' (Barton, 1973).

Between the Beacon Hill and Queen's Avenue landslides, the sea cliffs were occupied by a series of large, full cliff height mudslides. These do not appear to be stratigraphically controlled in any way, and indeed, may have been caused by land (or other) drainage. East of the Miramar landslide, the cliffs are again occupied by mudslides, but these penetrate almost to the base of the London Clay Formation. One of these mudslides, investigated by Hutchinson (1970) appears to follow the weak bed thought to be in the A2 unit. A small failure recorded during constructions works may also follow this weak bed.

Bromhead (1978) made back analyses of the three main slides at Herne Bay, publishing not only the mobilised residual angle of shearing resistance, but also mean stresses in each slide.

Isle of Sheppey

Investigations of coastal cliff in the Isle of Sheppey at Warden Point were made by Dixon (1986). Sections were drawn down through the 1971 landslide at Warden Point

Chapter8: The residual strength of clay in coastal landslides in the London Clay

instrumented primarily with piezometers and also an inclinometer. A further section was instrumented in the same way at the west of that location. It was taken through a section of coastal cliff where the previous coastal slide had been washed away.

Dixon's study included long term measurement of pore pressure in which stand pipe piezometers equilibrated and showed a depressed pore pressure regime on the slope. Using the piezometer tubes as a slip indicator proved useful to find the slip surface location in the first section. The 1971 landslide was fully delineated with the aid of some surveys that have been done over a number of years by undergraduate students at Kingston University. It was possible to interpret the 1971 landslide development through the time and therefore to provide cliff profiles at the different dates (see also Figure 4-2).

The pore pressure information was interpreted into a model from which the pore pressure of the earlier dates were produced. On the basis of this, back analyses were made of the 1971 landslide as it developed.

The important output of this in preliminarily terms which was reported by Bromhead and Dixon (1985) in response to Skemton's lecture (1985) of that year. A full account of the investigation and its results were published by Dixon and Bromhead (2002).

The primary outcome of the back analyses was to show that as the landslide evolved it in fact wasted away and so the stress is reduced through the time and the location of the effective stress-shear points moved down the residual strength envelope towards the origin. Because of this the Sheppey Warden Point landslide back analyses demonstrate an important effect noted only in passing by Bromhead (1978) which is that the stress point representing in the back analysis migrates down toward the origin along the residual strength envelope.

The Sheppey coastal cliffs provide many more examples of the landslides but only in the vicinity of the Warden Point they have been fully investigated.

The cliffs at Warden Point are very accessible, particularly during the summer and provide materials for test programmes including Dixon's and provided the London Clay samples for Bromhead et al. (1999) and Bromhead and Curtis (1983).

8.3 Field observations

Field visits were made to the active landslides in north Kent at Warden Point, Isle of Sheppey, and to the stabilized landslides at Herne Bay. At Sheppey where the upper part of the London Clay is exposed, there is a series of active landslides and observation which has been made of over the last 50 years. Generally, after a landslide, there is a period of marine erosion and shallow slides activity until eventually the whole slide is eaten away and the new one takes place.

While the slopes at Sheppey are actively eroding, and the process are very clear; at Herne Bay there is little to see apart from grassy slopes with the signs of the drains. In the Beacon Hill landslide the slight deformation in the sea wall still visible, the heads of deep drainage shafts (Berkeley-Thorn and Roberts, 1981) are still in evidence. The shape of the regraded Miramar slope reflects the ridge and graben shape of the slide. An unsuccessful attempt was made to find the basal shear in the cliffs east of Beltinge.

8.4 Re-appraisal and further work on the Herne Bay landslides

Beacon Hill landslide

The Beacon Hill landslide is dish-shaped, and as Bromhead's (1978) analyses were done on the principal cross section through the slide they must have over-estimated the stress levels in the slide. Moreover, a significant fraction of the slide mass appears to lie below high-water, and in Bromhead's analyses, he took a piezometric line at 'seabed' level offshore. This position provides the same answer at low tide and high tide, as the weight of seawater is counteracted by pore pressure changes in the soil at high tide. In retrospect, and with a better understanding of pore pressures in the soil, gained from work in the foreshore at Sheppey (Dixon & Bromhead, 2002), it is clear that this assumption over-estimates the pore pressures, with a consequent underestimation of normal effective stress and over-estimation of residual angle of shearing resistance.

The over-estimation of pore water pressures seaward of the seawall is indeterminate, but would reduce the residual angle of shearing resistance calculated.

A 3D analysis of this slide is inhibited by lack of knowledge of the transverse curvature of the slip surface but the 3D results are: $\sigma'_n = 70.8$ and $\tau = 22$ while for the 2D analysis the normal effective stress is 106.6 and the shear stress is equal to 28.5 (Personal communication, Bromhead 2012).

This demonstrates the importance of 3D shape in reducing the general stresses, but also because this landslide is predominantly not along the weak bed of the slip surface. It should be expected to generate a higher residual angle of resistance.

Indeed at Herne Bay the lowest residual strength was predicted by the landslide with the biggest area of basal slip surface along the weak bed (see also Section 8.8).

Queen's Avenue landslide

The Queen's Avenue landslide is understood to have occurred in about 1896 (Bromhead, 1978). From the 1930's onwards it was investigated several times with a view to extending the stabilization works eastwards. The scattered boreholes were reinterpreted into the 3D shape of the slip surface, showing that this landslide is also strongly 3 dimensional. Piezometers installed in 1969-70 are much better distributed from head to toe of this landslide in comparison to the piezometers in the Beacon Hill landslide, so that the piezometric line is more reliable. Also, the basal shear surface is somewhat higher relative to sea level.

A 3D analysis of this landslide shows that the 3D effect alone reduces the average normal effective stress from around 103kPa to 86kPa, and the average shear stress from 23kpa to 19kPa, so that the residual angle of shearing resistance reduces by 0.5°

(Bromhead et al., 2001) relative to the 2D analyses on the principal cross section reported earlier (Bromhead, 1978).

Miramar landslide

Although the Miramar landslide is a 'graben slide' or a 'block slide', it has many characteristics in common with the 'bench slides' described by Barton (1973). 3D effects are small as a result. The basal weak bed does dip across the site (from E to W), leading to faster movements of the western end of the Miramar landslide's 'ridge', and so there ought, in principle, to be detectable differences in the stability across the slide, but the data are not available to do 3D analyses.

Taking published data for the three landslides at Herne Bay and at its face value, it is then obvious that the analysis of these landslides could be criticised on the number of ground. Firstly the Beacon Hill and Queen's Avenue landslides are really three dimensional in character rather than two dimensional. An attempt has been made by Bromhead et al. (2001) to analyse those in 3D. This also produced a lower friction angle.

The main slip surface of the Miramar landslide was analysed by Bromhead (1978), however, he did not analyse the front slip surface, which is the early stage of landslide. Bromhead (1978) described that as the remnant of an earlier landslide at the same coastal location, so that this is an opportunity for further back analysis which is not being done. In the reconstruction of the early stages of the Miramar landslide (viz. Bromhead, 1978) a 'seaward slide' is shown, which is formed from debris from the supposed 1883 landslide.

A simple back analysis of the frontal slide as reconstructed by Bromhead has been undertaken. Two different ground profiles, 1956 and 1966-1970 considering the front slip surface, as shown in the Figure 8-3, have been analysed. Chapter8: The residual strength of clay in coastal landslides in the London Clay



Figure 8-3: Section through the Miramar Landslide showing erosion and regrading (after Bromhead, 1978). The front slip and ground profile in 1956 and 1966 & 1970 has been taken for the back analysis.

The analysis has been conducted in accordance to the method explained in Section 5.2. The software which has been used is the Windows version of the same software as Bromhead used for analysing the main slip. The equivalent results are presented. The results are listed in Table 8-3.

Table 8-1: Shows results of back analysis of front slip considering the ground profile in 1956 and 1966&1970 separately, as shown in Figure 8-3.

	Herne Bay			
Selected Slip Surface for back analysis	Condition of GWL	GWL as presented by Bromhead 1978		
	φ', (°)	φ' _{r1} (°)	φ',2 (°)	
	φ' _{rav}	15	15	
Ground Profile 1956 & Front Slip	First Ratio of ¢',	14.6	16	
	Second Ratio of ϕ'_r	14.2	17	
Ground Profile 1966-1970 & Front Slip	φ' _{rav}	19.2	19.2	

In the cross section of Miramar Landslide, the both slip surfaces are going through the weak bedding, as shown by Bromhead (1978). However, it is believed that the results which are listed in Table 8-1 indicate that the reconstruction of this part of the Miramar landslide is probably incomplete and inaccurate. Because the indicated

residual angle of shearing resistance is far too high and it is not in accordance with any of the other observations.

It appears probably that the landward part of the slip surface is in the wrong place, or has the wrong shape. That may mean the piezometric conditions are wrong and may not have the same pattern as the main slip. It may arise because; the front slip surface is just not connected with the weak bed. The front slip surface probably not entirely along the bedding plane and it may be the failure in the debris and is not driving along the weak bedding plane. Possibly, there are genuinely differences between mudslide and landslide which are bedding controlled in nature. In this case, the analysis has been done and the bedding plane seems not very important.

Beltinge landslide

In order to test this theory a bit further, Beltinge landslide (Hutchinson, 1970) is considered. This particular mudslide and all the mudslide at Herne Bay are associated with weak bed. So that the results of back analysis for this cross section would show the weak bed residual strength and lead to an averageed London Clay residual strength which is presented in Table 8-2.



Figure 8-4: Section of Mudslide in London Clay at Beltinge, showing flow II (after Hutchinson, 1970). The slip surface and piezometric level were recognised by instrumentation. The ground profile in 1966 has taken for back analysis.

Hutchinson (1970) shows a cross section through his 'Beltinge mudflow' (mudslide). This is complicated by undrained loading at its head, and movement in a channel. Analysis of this mudslide, which appears to run along the A2 weak layer, gives the following results in Table 8-2.

It has not been possible to analyse the small slide that occurred during construction works (as shown on the upper part of the section in Figure 8-4), although the reconstruction of the section (Arup, 1970) shows a bedding-controlled basal shear surface location, possibly in A2.

Table 8-2: Shows results of back analysis of mudslide at Beltinge considering the main slip of the flow II in the front and ground profile in 1966 as shown in the Figure 8-3.

		Beltinge				
Selected Slip Surface for back analysis	Condition of GWL	GW at Max P Flov by Hutchin	iezo. within v II son 1970	GW at Min Piezo. within Flow II by Hutchinson 1970		
	φ', (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	
Ground Profile Dec 1963	φ' _{rav}	11.3	11.3	10.4	10.4	
Ground Profile Sep 1964	Φ' _{rav}	11.9	11.9	10.9	10.9	

Other than these remarks (As in the case for Herne Bay), the published back analysis data are taken as probably being the best reasonable values obtainable for the source data to hand. Bromhead made a 2-zone analysis of the first-time failure of the Miramar landslide, but 2-zone analyses have not been carried out for residual strength cases. Indeed, as the London Clay formation plastic clay strata away from the weak layers are probably only a degree or so different in residual angle of shearing resistance terms, it is not possible to easily distinguish the two in any back analyses.

8.5 Appraisal of Sheppey

Although Gostelow made a survey of the Warden Point landslide after failure in November 1971, serious scientific work on this landslide and the adjacent cliffs to the

west were not done until Dixon's (1986) study. Pore pressures from this study were found to be lowered by the effects of stress relief through landsliding and coastal erosion (Bromhead and Dixon, 1986), and attempts were made to understand later landslides in the area by Dixon and Bromhead (1991), but a full set of back analyses were not published until much later (Dixon and Bromhead, 2002). Some early results were given in a Technical Note (Bromhead & Dixon, 1986).

Due to the very low pore water pressures in this slope, and its height (significantly higher than even the Herne Bay slopes), the normal effective stress levels are significantly more than in the other London Clay case records.

8.6 Comparative discussion of back analysis results:

Results of Beltings mudslide for the ground profile of December 1963 and September 1964 are presented in Figure 8-5. The data set include back analyses results for maximum and minimum piezometric condition within the mudslide. The best fit straight lines through the origin of the data set have the slopes as shown in the figure. The results are compared with the result published back analyses.

The back analysis results in the literature fall into several classes. The best data values for the purpose of this research are listed numerically in the source papers, and are published with an account of the on-site investigations into the geometry of the slip, and the corresponding piezometric conditions. Some accounts simply list the equivalent ϕ'_r value, without the corresponding mean stresses, or show the points on a low-resolution graph (e.g. Chandler 1982b). These results have been extracted as accurately as possible.

The dataset does contain a few results from shallow *solifluction* type landslides, notably at Hadleigh Castle (Hutchinson and Gostelow, 1976; Skempton, 1978).

The Herne Bay analyses (Bromhead, 1978) provided at one time the main corpus of high stress-level data for the London Clay (Dunbaven et al., 1980). The analyses are of

115

Chapter8: The residual strength of clay in coastal landslides in the London Clay

3 coastal landslides. Of these, the Miramar landslide is the one with the most "bedding control". The topography for all three landslides is good, and the slip surfaces are wellidentified, and the piezometric conditions are not only determined through multiple piezometers, but they are to some extent (especially in the case of the Miramar landslide) controlled by the proximity of the underlying Oldhaven Beds (Lambeth Group). However, the Queen's Avenue and Beacon Hill landslides are not laterallyextensive, and there is a possible 3-D effect there (explored briefly by Bromhead et al. 2001). Furthermore, the Beacon Hill landslide extends significantly under the foreshore, and in retrospect, the pore pressures used for this zone are probably overestimated.



Figure 8-5: Results of residual friction angle calculation using back analysis technique for the landslides in London Clay at Beltinge. These results are compared with published data.

The stability analyses for the landslides at Warden Point, Isle of Sheppey, have even better piezometric data, good topography and slip surface positioning. Like a few of the Miramar slide analyses, an effort has been made to back-figure the pore pressures through time, and thus to provide data for stages in the evolution of the landslide for which a profile, but no directly measured piezometric conditions, were available. Some of the residual strength data appears in the graph that does not, at first sight seem to relate to residual strength. An example of this is Skempton and Petley, 1967.

76 data points are plotted on a graph and the best fit straight line through the origin of the data set has been drawn (Fig.8-5). The best estimate for slope of trend line of these data indicates that $\phi'_r = 10.8^\circ$ with $R^2 = 0.97$. The slope of best fit straight line for Beltinge analyses similar ϕ'_r with these results.

8.7 Previous research: (Ring shear results)

For the other case studies, in addition to the back analyses, laboratory tests are undertaken on those soils properties which the relevant information is lacking. However, the laboratory testing in London Clay properties are very well known and just the results from the literature survey reviews are presented.

London Clay is first clay which has been tested in the laboratory. Since Skempton presented his theory of residual strength, researchers focused to find the methods to measure the residual strength in the laboratory. Results of a number of methods in order to measure residual strength are presented in the literature, among those, results of ring shear apparatus particularly Bishops ring and Bromhead ring shear apparatus are collected are plot in Figure 8-6.

For brevity in this research, the only results considered which are listed in Appendix C. Any of the published results have not been knowingly omitted for this clay, but are aware of missing data. The repeated data in the literature are followed and referenced from the original works.

The best data values of ring shear results in the literature are listed numerically in the source papers (see Appendix C). Some papers simply list the equivalent ϕ'_r value, without the corresponding mean stresses, or show the points on a low-resolution

graph (e.g. Chandler, 1972). These results have been extracted as accurately as possible.

Indeed, the main outlier data points seem to be those for the manufactured samples by mixing Happisburgh Clay and London Clay (Lupini et al., 1981), using different percentage of London Clay in the mixture. The higher values above the trend line show the lower percentage of London Clay and the lower points indicate higher percentage. Without those outliers, the data set indicate $\phi'_r = 10.0^\circ$ with R² equal to 0.97.



Figure 8-6: Results of residual friction angle measurement by ring shear apparatus (both Bromhead and Bishop devices) for the landslides in London Clay. The data come from the published data or extracted from published graphs.

The best fit straight line through the origin of the data set has been drawn (Fig. 8-6). The best estimate for ϕ'_r in 50 data points is 10.8 with an acceptable R-squared.

8.8 Conclusion

Three landslides at Herne Bay were analysed by Bromhead (1978). If the results of analyses of three landslides at Herne Bay is plotted on the same graph, the Miramar landslides analyses indicate lower value of ϕ'_r than the Queen's Avenue, and the results of Queen's Avenue is lower than the Beacon Hill landslide (see Figure 8-7). This effect also continues in 3D analyses.

The Queen's Avenue landslide in 3D analysis has lower stress then the 2D analysis, but in the case of Beacon Hill landslide, there is much more difference between 2D and 3D analyses. This is a function of the changing level of the weak bed, because at Beacon Hill the slip surface goes down the sea level and only a small part of the bowl shape landslide runs along the weak bed.

The weak bed in the Queen's Avenue landslide is just below the sea level and the proportion of that in the landslide is rather more significant. However, the Miramar landslide is at or above the sea level (more or less, relatively to the others) and a big fraction of slip surface is governed by weak bed. Therefore, the Miramar landslide comes out with lower ϕ'_r then the two others.

On the other hand, the Sheppey case is very analogise to the Miramar landslide. Landslide at Sheppey is a very big (relatively to the ones at Herne Bay) and it is very wide and there is a very little 3D effect. The Warden Point landslide is different which does not actually have all that bigger proportion running along the weak bed. So it gives a slightly higher residual friction angle (comparable) than the Miramar landslides.

The other thing which comes out generally from the Herne Bay analyses is that, when the Queen's Avenue was regraded, in fact it is not changed the required ϕ for stability. In other word, the factor of safety is barely changed.



Figure 8-7: Result of back 2D and 3D back analysis of landslides at Herne Bay including the new analyses of this research. The red coloured arrow shows the drop of stresses of 2D analysis to the 3D analysis in Beacon Hill landslide.

Similarly in the Miramar landslide, it is much more obvious that what was done in Miramar. The soils were moved from the ridge to somewhere else over the flat are of the slip surface. In fact there was not too much change.

As the landslide changes its shape and evolves then the stress comes back down along the failure envelope. That is what which well developed in the landslides at Sheppey. Figure 4-2 shows the ground profile of the Warden Point landslide through the years. As the shape of cross section is changing the results of analyses fall down along the failure envelope.

There are two new analyses; the front part of the Miramar landslide and the Hutchinson's Beltinge mudslide.

Analysis of the front slip of Miramar landslide gives a much higher friction angle than the main Miramar slip. There are several possibilities for that; firstly there are very

Chapter8: The residual strength of clay in coastal landslides in the London Clay

different materials with a lot of incorporation of gravel. Secondly, the pore pressure assumed locally to be higher or thirdly, the ground profile is wrong. Because it was measured from the air photographs by surveying specialist at Arup who used to use parallax bar. So any one of the above would be the case of uncertainty.

The Beltinge mudslide is a different category as it was where Hutchinson discovered that there is a slip surface at the base of the mudslide. Following that he made the reference to mudslide rather than the mudflow. When this section was analysed, a low residual ϕ' was obtained (10-11 degrees). The reason for that is, because it runs along the weak bed. Although, it is very three dimensional (as it runs down a channel) but much of its base is on the weak bed which give the residual results.

If these results plotted on the same graph as the result of Bromhead (1978). It appears at the low stress on the graph (see Figure 8-7). On this graph the Beacon Hill 2D and 3D analyses are shown. The 3D analysis has the same pore pressure assumption as the 2D which indicates that is too high offshore. If the pore pressure is reduced it will move down along the envelope on the graph.

9 The residual strength of London Clay in landslides along the infrastructure cuttings

9.1 Introduction: Geographical and geological setting

The large landslides in the London Clay are located in the coastal line of south and south east England. However, small landslides in London Clay are also found inland (mainly in the London Basin) and in numerous infrastructure cuttings.

There have been numerous investigations of failures inland in the London Clay formation. In the London Basin they are located along the infrastructure cuttings (e.g. Gregory, 1844; Skempton, 1948 & 1977, Henkel, 1956). A large inland landslide at Stagg Hill in Guildford lies in the Hampshire Basin (Skempton and Petley, 1967). The only published infrastructure cutting failure in the Hampshire Basin is the one at Fareham, described by James (1970).

The location of outcrop of the London Clay is Figure 8-1.

Stabilization works

An example of inland slope stabilization is at Surrey University in Guildford (Skempton and Petley, 1967). This was stabilised by herringbone pattern drainage and by localised regrading. Shallow drainage is extremely widely used where the depths of sliding material are small.

9.2 Previous work on Infrastructure cuttings

Failures in infrastructure cuttings were reported in the mid-nineteenth Century (e.g. Gregory, 1844), and have continued to plague the railways, and latterly roads, since then. Data on railway cutting failures assembled by Skempton (analysed in total stresses) were presented in 1948 (Skempton, 1948) in support of a hypothesis that the undrained shear strength of stiff clays diminished with time. An effective stress variant

Chapter9: Residual strength of London Clay in landslides in infrastructure cuttings

of this was presented in 1970 (Skempton, 1970), modified later by Skempton and Chandler (1975).

Data on the residual strength of London Clay was also presented by Skempton in 1985. These presentations depended on a series of analyses done by James in his PhD thesis (James, 1970). James had taken a number of sections from the archives of British Rail (now Network Rail) and by Henkel (1957), and De Lory (1957). However, James's dataset has some notable omissions, for example, the Uxbridge cutting (Watson, 1956; Henkel, 1956) and of course the noted New Cross landslide (Gregory, 1844; Bromhead, 2004a).

James (1970) carried out back analyses on 19 failed railway cutting slopes in London Clay. These are listed in Table 9.1. Back analyses were generally done to establish both first-time failure strengths and (where relevant) residual strength. Unfortunately, few (6) of the sections have precisely located slip surfaces, and even fewer (5) have piezometric data with observations in standpipes or boreholes. Some of the reconstructions of slip surface position do not seem particularly accurate or even correct, and in a small number of cases, the deformations are very slight. This may indicate that instead of a new slide developing in London Clay which is brittle (Bishop et al., 1965; & Petley, 1994) and would be expected to have larger deformations, what may have been recorded could be further deformations in some unrecorded early remedial works.

A small number of the cases appeared worthy of re-analysis, partly as a check on the accuracy of the original analyses, and partly to investigate new interpretations. James used the original computer program written by Morgenstern and Price (1967).

Generally, however, these slides are small, and all the results cluster at the low normal effective stress end of the scale. Taken on its own, the best fit line through James's dataset indicates a higher residual angle of shearing resistance than when the coastal cases such as Herne Bay and Sheppey data is added. This may indicate a slight upwards convex curvature of the residual shear strength envelope (see Figure 4-1 and also

results of Chapter 8), or it might simply reflect the highly approximate nature of some of the analyses.

Table 9-1: Sections of landslides in London Clay which investigated by James. A number of sections reanalysed by Hosseyni in order to calculate residual shear strength of London Clay

Location	No of sections	Date of Slip	Slip surface	Piezometric Line	Ground profile before slip	Ground profile after slip	Re- analysed by Hosseyni
Northolt	5	1955	Observed	Estimated	Estimated	Observed	Y
Wembley Hill	1	1918	one slip Assumed & one slip observed	Adjusted to before slip condition	Observed	Observed	Y
Upper Holloway	1	1951-3	Two slips Assumed & one slip "Worst Circle De Lory (1957)"	observed	Observed	Observed	Y
Farham (Hampshire Basin)	1	1961	one slip assumed & one slip observed	Estimated	Observed	Observed	Y
Dedham	1	1910-11 & 1952	Observed	Observed in standpipes	Observed	Observed	Y
West Acton	2	1966	Assumed	No data	Observed	Observed	N
Tulse Hill	1	1968	Estimated	Observed in standpipes	Observed	Observed	N
Grove Park	1	1962	Assumed	Observed in standpipes	Observed	Observed	N
Kingsbury	2	1947 & 1968	Assumed	No data	Observed	Observed	N
St. Helier	1	1952	Assumed	Observed in Boreholes	Observed	Observed	N
Grews Hill	1	1956	Assumed	No data	Observed	Observed	N
Althorne	1	1957	Assumed	No data	Estimated	Observed	N
Cuffley	1	1951	Observed in boreholes	No data	Observed	Observed	N
Hadley Wood	1	1947 & 1951-2	Assumed	No data	Observed	Observed	N
Whitstable	1	1959	One slip observed & two slips assumed	No data	Assumed	Observed	N
Grange Hill	1	1950-1	Assumed	No data	Observed	Observed	N
Isle of Sheppey	2	Not mentioned	Assumed	Estimated	Observed	Observed	N
Bradwell	1	Not mentioned	Assumed	Estimated	Observed	Observed	N
Wood Green	1	1948	Assumed	Estimated	Observed	Observed	N

9.3 Re-appraisal of infrastructure cutting analyses

The set of cases analysed by James (1970) includes the list of cases in Table 9.1. Three of the cases have multiple cross sections: cases at Northolt, West Acton and Isle of

Chapter9: Residual strength of London Clay in landslides in infrastructure cuttings

Sheppey (failures in a small pip, since infilled). By 1970 this represented an unparallelled body of results, Skempton (1985), Skempton (1978) and Bromhead (1978). However, now some four decades later it is time to be critical about these data. In the following sections a number of new interpretations of the data are made.

Dedham

The cutting at Dedham was made in 1840 and in the winter of 1910-11 there was a landslide at this site. Then the slope was stabilised by remedial measures and a deep trench drain. Further slipping occurred again in 1952 over a length of 45m in the direction of general ground slope (James, 1970).

The presented cross section by James (1970) shows ground profile before and after slip in 1952 (see Appendix C). In this cross section the water level was recorded in stand pipes after failure and slip surface assumed from the failure profile and few borehole results. The water level and the ground profile before slipping were assumed and reconstructed. Two slip surfaces were drawn in the sections, namely slip A and slip B. James conducted two analyses for slip B considering r_u =0.28 and r_u =0.29 before and after the slide respectively. He found $\phi' \approx 14^\circ$ for the residual case and $\phi' \approx 24^\circ$ for the peak case when c'=0. r_u was assumed equal to 0.15 for slip A and ϕ' calculated about 21° when cohesion was neglected with no residual calculation.

When the ground profiles before and after slipping are scrutinized, it is found that the slip A is unreliable and is not proved by the slope geometry. The observed slip plane in the bore holes may possibly be part of a pre-slipping in the front of the slope as shown in Figure 9-1. The trimmed toe also indicates that this slope profile is unlikely. Therefore, according to slip geometry, the ground profile of after failure 1952 is modified as shown in Figure 9-1.

Whereas James tried to analyse the cutting for the first time failure parameters, clearly the piezometric condition were questionable relevant to residual strength calculation.

James took $r_u = 0.3$ as an average in the absence of water level recording, however, the stand pipe condition is used for analyses in this research.



Figure 9-1: Section of landslide in London Clay cutting at Dedham (after James, 1970). The section illustrates the original ground profile after the cutting in 1840 and the ground surface profile before and after the slip in 1952. Water level was recorded in stand pipes after failure but slip surface was assumed from failure profile and boring results.

The back analysis of this section has been conducted for two set of topographies. First for the main slip surface (Shown by James) and ground profile of after slip 1952. Then for the front slip surface and modified ground profile, separately. The analysis is carried out according to the method explained in Section 6.4. As on the cross section James noted that the water level is assumed, in order to consider different ground water conditions the analysis is done for lower ground water condition at 0.5m and 1.0m below the level is shown in the section. The results are listed in Table 9-2.

The calculated average residual strength for the main slip agrees with James result for the same slip surface. However, results of modified slip ground profile indicate ϕ'_{rav} = 9.7° which is consistent with the average of residual shear strength of data from the literature.

Table 9-2: Results of back analysis of landslide in the cutting at Dedham for main slip considering the ground surface after slipping in 1952 and front slip surface considering the modified ground profile as shown in Figure 9-1. The analysis repeated for different ratio of ϕ'_r in the case of exciting weak bed.

		Dedh	am				
Selected Slip Surface	Condition of GWL	Assumed	piez. Line	Lower p	oiez. Line	Lower piez. Line	
for back analysis		by P.	lames	at -0.5 m		at -1.0 m	
	φ', (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)
Mala Clin	φ' _{rav}	14.3	14.3	*	*	13	13
Iviain Siip	First Ratio of \$\$\phi'\$r	13.1	15		*	11.5	14
Front Slip (using	φ' _{rav}	9.7	9.7	8.6	8.6	8	8
profile)	First Ratio of ϕ'_r	8.6	11		*	•	

James noted that a thin capping of sand and gravel is presented on the hill at the top, therefore ϕ'_r of the back slip could have higher values than the average. In this case residual angle of bedding plane is resulted 11.5° when water is at lower level.

Northolt

The cutting at Northolt was made in 1936; however, the slip occurred in autumn to spring of 1955 along a considerable length of railway (Henkel, 1955).

Although James noted that the slip at Northolt was well documented and boreholes and piezometer made the accurate cross sections possible, none of the five presented cross sections contain observed ground water conditions. The cross sections included observed slip surface, soil stratum, and ground profile before and after slipping, however, the piezometric level is estimated in all the sections. One of the cross sections, shown in the Appendix C, is selected and back analysis is done. For the selected section James reported that ϕ' is about 18 degrees while c'=0, although this research shows 15.6° for the same condition.

Therefore, for accuracy, the cross section of landslide at Northolt which was drawn by Henkel (1957) is used as another case for back analysis (see Figure 9-2). This cross section shows the location of slip surface which is found in trench, soil stratum and
ground surface profile before and after slipping as shown in Figure 9-2. The ground water level measured by Casagrande type piezometers in December 1955 as shown in the section (Henkel, 1957). Henkel used this water table data to analyse a circular arc slip surface which occurred early in 1955. He applied the after slip piezometric data in Bishops method of analysis.



Figure 9-2: Section through Northolt cutting (after Henkel, 1957). The original ground surface profile after cutting 1938, the ground profile after slipping, location of piezometers and slip surface.

In the cross section by James the water level was assumed and in the cross section by Henkel after slip piezometric data was employed for the analysis. In order to survey how the ground water level changes influence the results, the analyses are repeated for different ground water condition parallel to what is shown in the sections at +/- 0.5 and +/-1.0 m (above and below). The analyses are conducted in accordance to the method explained in Section 6.4. The results are presented in Table 9-3.

By the time when the Henkel studied the Northolt landslide (i.e. 1957), the definition of residual strength had not been presented. The results for this section shows higher value for average ϕ'_r even when according to the method of analysis a higher residual strength considered for the back slip the value of ϕ'_r is 13.4° at minimum. It must be reminded that the slip occurred in boundary of Brown and Blue London Clay which are the same material (more or less) therefore this difference of the ϕ'_r 's could not be too much.

Table 9-3: Results of back analysis for the cross sections of landslide at Northolt. The table show results of analysis for the main slip in the cross section by James and Henkel separately. Results of repeated analysis for lower ϕ 'r for the weak bed are shown in the second row.

			No	rtholt							
Selected Slip Surface for back	Condition of GWL	Estimated piez. Line by P James		Lower piez. Line at -0.5 m		Lower piez. Line at -1.0 m		Upper piez. Line at -+0.5 m		Upper piez Line at +1.0 m	
analysis	φ', (°)	φ' _{r1} (°)	Φ'r2 (°)	φ' _{r1} (°)	Φ'r2 (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ',1 (°)	φ' _{r2} (°)
Main Slip by P	Φ' _{rav}	15.6	15.6	14.8	14.8	14.2	14.2	16.8	16.8	18.5	18.5
James	First Ratio of ϕ'_r	15.4	16	14.1	16	13.8	15	16.4	18	•	•
Main Slip by David	φ' _{rav}	17.4	17.4	15.8	15.8	14.8	14.8		•		
Henkel	First Ratio of Φ'_r	16.8	18	14.5	17	13.4	16	•	•	•	•

James reported that the residual strength for this case is about 20° however the result of these research show lower value for ϕ'_r even when the water level in 1.0 m higher than what is shown in the cross section.

Upper Holloway

The cutting and retaining wall were constructed in brown London Clay in 1870. The cutting slipped in 1951-3 and pushed the retaining wall forward.

The cross section was drawn by James with reference to De Lory is investigation in 1957 (see Appendix C). It shows the original ground profile of the cutting and the ground profile after slipping in 1953. For the analysis James employed two slip surfaces, A and B which both were assumed from the failure profile. He also used a circle slip surface C which is named 'worst circle' by De Lory. When the after failure ground profile is scrutinized, it is found that the slip surfaces B and C seem unreliable and the assumptions were wrong. Therefore they are neglected for the back analysis in this research. James showed two piezometers in the slipped area and two in the intact slope, however, piezometric level after slip was assumed.



Chapter9: Residual strength of London Clay in landslides in infrastructure cuttings

Figure 9-3: Cross section of landslide in London Clay at Upper Holloway (after James, 1970). The section illustrates original profile of ground cutting, ground profile after slipping, location of retaining wall before and after of slipping, location of assumed slip surfaces and combination of piezometric data to provide four different ground water condition. GWL2 is assumed piezometric level before failure.

In order to utilise ground water condition for the back analysis the data of four piezometers have been used. Four different ground water conditions have been utilised namely ground water condition 1 (showed GWL1) and GWL2, GWL3 and GWL4 (see Figure 9-3). The back analysis has been carried out for the slip A using the above ground water condition and the results are listed in Table 9-4.

Table 9-4:Results of back analysis for slip A of landslide at Upper Holloway using the ground profile after slip 1953 at different ground water level extrapolated from the piezometers in the slipped and intact areas as shown in Figure 9-3.

			Upper	Holloway	/S				
Selected Slip Surface for back	Condition of GWL	. 1 as own ig9.3	GWL sho in F	. 2 as own ig9.3	GWL sho in Fi	. 3 as own ig9.3	GWL 4 as shown in Fig9.3		
analysis	φ', (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ',1 (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)
Ground profile after failure & Slip A	φ' _{rav}	17.2	17.2	16	16	14.9	14.9	13.9	13.9

James reported 20° for the residual friction angle of the landslides which is much higher than the results of these analyses. The GWL4 is the lowest ground water condition in these analyses and indicates lower value for ϕ'_r , therefore back analysis repeated for more two ground water condition at 0.5 and 1.0 meter lower than GWL4 and results are presented in Table 9-5.

Table 9-5: Results of back analysis for slip A of landslide at Upper Holloway using the ground profile after slip 1953 at different ground water level lower that GWL4 as explained.

		Uppe	r Holloways					
Selected Slip Surface for back	Condition of GWL	GWL 4 a	is shown	GWL 4 - shown	0.5 m as in Fig9.3	GWL 4 -1.0 m as shown in Fig9.3		
analysis	φ', (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	
Ground profile after failure & SlipA	φ' _{rav}	13.9	13.9	12.5	12.5	11.5	11.5	

The results are comparable with results from the literature when the ground water is one meter lower than GWL4. Of course, the record of the area shows that it still high relatively to the basal shear surface.

Wembley Hill

At Wembley Hill, the cutting and retaining wall were constructed in 1905 and the slope slipped first time in 1918 in the length of about one mile (sic). James referred to De Lory (1957) studies in order to draw the cross section of this landslide (see Appendix C). James noted that slipping occurred largely within the blue London Clay and perhaps concentrated along certain bedding planes.

The cross section shows the ground profile before and after slipping and ground stratum. The slope contains pebble gravel at the top and brown and blue London Clay at the lower layers. Two slip surfaces were drawn, slip A which is assumed from the failure profile and observed at the toe of slip, and slip B is the worst slip surface by De-Lory in 1957. This recent slip surface is neglected in this research. There were three piezometers on the landslides area. The data of the piezometers have been used in order to provide ground water information for back analysis, however, piezometric level adjusted to before slip condition (see Figure 9-4).



Chapter9: Residual strength of London Clay in landslides in infrastructure cuttings

Figure 9-4: Cross section through landslide in London Clay at Wembley Hill (after James, 1970). The sketch shows ground profile before and after slip. Location of analysed slip surface and applied ground water condition are drawn.

Two separate back analyses have been carried out for the main slip (Slip A) considering the adjusted piezometric condition and the available piezometric information as shown in Figure 9-4. The slip surface A is modified in accordance to the failure profile geometry. In order to see how the ground water condition affect the calculated average ϕ'_r , the analysis is repeated for more ground water condition assuming it is at 0.5, 1.0, 1.5 and 2.0 meter below the piezometric line. The back slip is situated in different soil and partly (a bit) in pebble gravel, therefore in order to emphasis existing of the weaker bed, the analysis is done for different ratio of ϕ'_r .

The calculated average residual friction angle for adjusted piezometric condition prior to landsliding is, more or less, equal of what James reported. However, the results of back analysis for available piezometric information show very lower residual strength and is not consistent with the data from the literature because the shear surface is assumed. These results (as reported in Table 9-6) indicate that the assumptions are questionable. Table 9-6: Results of back analysis for the cross section in Figure 9-4, considering modified slip surface A at different ground water condition as listed.

					We	mbley							
Selected Slip	Condition of GWL	Piez. adjust befo	Line ed to pre	Assu piez.	med Line	Lower	r piez. ne	Lowe	piez. ne	Lowe	r piez. ne	Lower	r piez. ne
Surface for back		slip t	by P les	by P J	ames	at -0	.5 m	at -1	.0 m	at -1	.5 m	at-2	.0 m
analysi	φ', (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ'r2 (°)	φ' _{r1} (°)	φ'r2 (°)	φ'r1 (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)
	φ' _{rav}	21	21	18.1	18.1	17.7	17.7	17.2	17.2	16.8	16.8	16.4	16.4
Main Slip (Modified)	First Ratio of ¢'r		•	17.3	19	16.3	19	16.4	18	15.6	18	15.8	17
(wound)	Second Ratio of ϕ'_r			16.4	20	15.4	20	15.5	19	14.6	19	14.8	18

Fareham

The cutting at Fareham was made in 1900. The first time slip occurred in 1961, however, there was doubt that it is possibly a reactivation of an earlier slip. According to a site investigation by National Rail, the cross section was drawn in the two slip planes which are shown Section A and B (see Appendix C). Each section includes the slip surface which is observed by Alkathene Tubes.



Figure 9-5: Section of landslide in the cutting in London Clay at Fareham in Hampshire Basin (after James, 1970).

The cross section in Figure 9-5 shows two different ground profiles after slipping in 1961 and location of filed data for drawing the slip planes. It also contains estimated water level in the slope. The back analyses have been conducted for two sections separately at assumed piezometric line. In order to investigate effect of changing ground water condition on the calculated residual friction angle, the analyses are repeated for piezometric line at 0.5, 1.0, 1.5, 2.0 m lower and results are presented in Table 9-7. James noted that there are sand and gravel at the top of slope so that it may mix up with soil body at the lower level of slope and affect the mobilised residual strength after slipping which is not the interest of this research. Therefore the analysis is repeated for different ratio of ϕ'_r and the results reported in the following table.

Table 9-7	: Results o	f back an	alysis of tw	o sections	A and B o	f landslide i	at Fareham	for differe	ent ground
water con	ndition at o	different r	atio of ϕ'_r						

				Fare	ham						
Selected Slip	Condition of GWL	Assumed piez. Line by P James		Lower piez. Lower piez. Line Line at -0.5 m at -1.0 m		Lower piez. Line		Lower piez. Line at -1.5 m		Lower piez. Line at -2.0 m	
Surface for						.0 m					
back analysis	φ', (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ'n (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)
	φ' _{rav}	18.7	18.7	18.1	18.1	17,3	17.3	16.4	16.4	15.6	15.6
Section A	First Ratio of ϕ'_r	•	•	16.7	19	16.4	18	15.5	17	15	16
	φ' _{rav}	19.2	19.2	18.6	18.6	17.7	17.7	16.8	16.8	16	16
Section B	First Ratio	18.2	20	18	19	16.1	19	15.2	18	Lower Lin at -2. ¢'r1 (°) 15.6 15 16 14.6	17

It is not expected that the results of Fareham to fit particularly with the data set, because it is located in Hampshire Basin which is more silty and less clayey. Although the average residual friction angles are the same as what is reported by James, however, is it not still consistant with the data in the literature.

In the previous sub-sections the results of back analysis for the selected section of landslides in London Clay are presented. The result by James are reported and compared with results of this research. The real problem in these cross sections is the piezometric line which is sometimes is very high. Of course, measurement of pore pressure in London Clay cutting show the pore pressure has vanished and they may not be fully equilibrated by the time that these failures occurred. Therefore, the pore

Chapter9: Residual strength of London Clay in landslides in infrastructure cuttings

pressure could easily be a lot lower. It is well known that if the pore pressure is lower the ϕ'_r is going to be lower so that means the lower estimated pore pressure by analyst leads to overestimate ϕ'_r resulted from back analysis and vice versa. On the other hand, if the ground profile is steeper than really it is so the back analysis ϕ'_r is higher and vice versa.

In the cases from railway cuttings, most of the time it is not known where the slip surface is and how the pore pressure is, so that having some piezometers does not mean to say the pore pressure is right on the slip surface. Therefore it must be noted that the Skempton's data, particularly 1985 paper is not found the unsurpassed very much. Even so, Bromhead and Dixon (1986) technical notes includes more useful data.

9.4 Discussion of back analysis results:

James's (1970) analyses of slides in railway cuttings have surveyed profiles, and occasionally slip surface locations and shapes established from instruments and direct observations – sometimes during remediation. However, the piezometric levels are usually obtained from very few instruments, of the "standpipe" type, and where the piezometric data were lacking, an average pore pressure ratio (r_u) equal to 0.3 was assumed. Undoubtedly, the assumption of a high pore pressure ratio leads to high back analysed ϕ'_r , and where the slides occurred before full equalisation of initially undrained piezometric conditions has occurred, then this is the source of some error.

In addition, in many of the section the slip surface location are documented and only observed at the toe (or lower part of the slip), and at the top of the slope. There are very few pit observations and rarely instrumentation records. Whereas the error bar of the soil density is negligible, the error bar of the slip surface position is vital and leads to very different values.

Furthermore, the slides commonly are not very laterally-extensive, and a threedimensional effect (where in particular the analyses are done on a principal, or centreline, section), again leads to a small overestimate of ϕ'_r . Results of selected cross sections are compared in Figure 9-6. The three datasets can be analysed independently or together. The best fit straight lines through the origin for each set have the slopes as shown in Figure 9-6.



Figure 9-6: The graph show results of back analysis of landslides in London Clay.

Three different data sets are presented. First trend line presents average of ϕ'_r for all the back analysis which are presented in the previous sections. It shows a higher value for average residual shear strength. In some cases existence of silt, sand and gravel leads to considering higher value for back slip surface. Of course for landsliding in London Clay the difference of ϕ'_r of bedding plane and the back slip is not very high whereas that is much higher for back analyses in Barton Clay. The second trend line shows the probable value of ϕ'_r for the weak bed at the base of the slides.

As discussed previously, in a few number of cross sections by James, slip surface and piezometric line are located precisely and the result confirmed this fact. Therefore, trend line for those sections (including Dedham and Beltinge) are presented as the *Acceptable Cases* (the brown colour trend line in Figure 9-6). It represents that the

average residual friction angle for London Clay is around 11°, and not the higher value of around 14° which comes from the set as whole.

However, it is clear that James's dataset does not meet the 'gold standard'.

10 Discussion and conclusion

10.1 Introduction and summary of the thesis

The research was started by looking at the residual shear strength of four British clays (see Chapter 6,7,8 9 and Appendices D and E). The available values of the ϕ'_r resulted from back analysis and ring shear apparatus (Bishop and Bromhead types) were collected and presented in graphs in Figures 10-1 to 10-7). The information came from published tables, and even sometimes extracted from published graphs or calculated from the published stresses.

An analysis of the data from these published back analyses, which are mostly supplemented with some of the more reliable laboratory testing, has been presented and compared. Laboratory results which have been specifically recorded as being on samples from bedding-controlled basal shear surfaces are included in the datasets. Datasets were selected from a number of stiff fissured over-consolidated clays of very different ages: Eocene, Cretaceous and Jurassic including Barton Clay, London Clay, Gault and Lias.

For this data analysis, it was carefully considered that some of the information is duplicated (for example, the "Misc Cut Slopes" of (Bromhead, 2004b) are known to be a subset of James's data (1970), and where possible, it has been "weeded out" the known duplicates. Hosseyni et al. (2011) presented a data analysis of results of back analysis for three different British clays.

For reason of largely of space, landslides in only two strata are considered in this thesis: the Barton Clay and the London Clay, the other two important clays (Gault and Lias) presented fewer or worse opportunities for new discoveries. However, the graphs of results of these two clays are presented in Appendix D (Gault Clay) and Appendix E (Lias Clay).

10.2 Correlations

The two datasets can be analysed independently or together. The best fit straight lines through the origin for each set have the following slopes which indicating the best estimate for ϕ'_{r} as shown in the following figures.



Figure 10-1: Results of the Bromhead ring shear tests on the samples from the landslides at Barton on Sea are compared with published results using same apparatus.



Figure 10-2: Results of back analyses show the average residual friction angle for landslides at Barton on Sea showing average ϕ 'rav for each case study.



Figure 10-3: The graph compares results of back analysis and Ring Shear test of Landslides at Barton on Sea (This research and Literature).



Figure 10-4: Results of published ring shear tests using Bromhead device and Bishop apparatus on the samples from London Clay.



Figure 10-5: The graphs shows results of published back analyses on the cross sections of landslides and results of the acceptable cross section of this research (as stated in the previous chapters).



Figure 10-6: The graph compares results of back analysis and Ring Shear tests of landslides in London Clay. The data includes published papers and results of this research.



Figure 10-7: The graph shows results of back analyses and Bromhead shear results from the literature and the results of this research. It is also included comparison of those results and results of Bishop ring shear device from the literature.

The main outlier data points seem to be those for the mixture of Happisburgh (which is silt) and London Clay by Lupini et al. (1981) using Bishop's ring shear apparatus. Without those outliers, the two datasets are indistinguishable to the unaided eye when plotted on the same graph.

Through the review of the investigations of residual ϕ' , it is found that ϕ'_r is absolutely critically dependent on the constitute minerals of the clays (e.g. Kenny, 1966 & 1967; Moore, 1991 & 1996), however, from the back analysis the same answer is obtained. Because, firstly, the whole London Clay is not the same as the whole Barton Clay and not the same as the Gault and the Lias Clay. But the weak bed in the London Clay is very similar with the weak bed in other clays. From this similarity, it can be argued that those weak beds in stiff clays have been sedimented in a very similar environment conditions. So that has to mean that they all have a common origin.

In the bedding-controlled landslides, the property of those weak beds dominates the behaviour of the landslide. Because in these cases the back slip does not play an important role in the landsliding. For example, the back slip of the landslides in the Barton Clay which is sand and gravel. There is a measurable difference between residual friction angle of the back slip and the weak bed. However, this difference for the landslides at Herne Bay is only a few degrees or less. The back slip is more important at Beacon Hill and Queen's Avenue landslides as they are strongly three dimensional, but it is not at Miramar landslides.

If all of these weak layers have pretty much the same origin, if they can be found in the geological records through the time of sedimentation, periodically. Therefore, it can argued that there is a thin layer in the sequence of the all clays which has similar properties.

For interpretation of this similarity, it can be said that the British clays are dominated by illitic and chloritic minerals, with small and variable contents of smectite and kaolinite. In a few cases, the beds containing the basal shear surfaces are observed to be a subtly different colour, or noticeably different plasticity to adjacent material. For example colour differences were note in the Barton Clay (Barton and Garvey, 2011) and in the Atherfield Clay in south Kent but at Folkestone Warren (Bromhead et al., 1998), or at Folkestone Warren (Hutchinson et al., 1980). However, the 'slide prone horizon' (Hutchinson and Bromhead, 2002) often is indistinguishable except by very careful sampling and testing. Nature, however, finds these slide prone horizons easy to detect.

Bromhead (2007 & Presentation of The 12th Glossop Lecture, 2011) speculated publicly that they may well be the results of small additions of volcanic ash at the time of deposition: these ashes weathered to smectites, and could be a mechanism for producing very localised effects, but given the distance to likely ash sources, they are unlikely to produce thick deposits (although one such is known from Walton on the Naze).

Generally, however, smectites evolve into illite and chlorite, so there is little difficulty in seeing why those minerals dominate: not only they are maturation products, but also there is some evidence of "recycling", where the erosion of some older strata creates the sediment supply for younger. This process today is putting all of the British clays into new sediments, much as Italian clays are being deposited, for example, in the Adriatic, where Niedoroda et al. (2005) has reported the accumulation of 50m of clayey sediments in places since the recovery of sea level after the LGM (i.e. in the past 22ka, and most probably, in the past 10ka or so).

Figure 10-1 for the two datasets is likely to be the effective lower bound for the residual shear strength of a dominantly illitic clay, but containing sufficient smectite to distinguish it from the adjacent material, although insufficient to give it smectite properties. The rather stronger adjacent materials demonstrate the (unquantifiable) effects of silt and sand content, and of the various cementitious minerals such as calcite, gypsum and various iron compounds.

It is found that, at a practical engineering level, the residual shear strengths of these two strata appear to be the same. This leads the author to conclude that the hypothesis above is likely to be correct.

While British clays are dominated by illite, a small number of strata are dominated by smectites. This may even be true of the Fuller's Earth of Jurassic age, but certainly includes examples from the Panama Canal (Lutton and Banks, 1970), from western Canada (Cruden et al., 1991) and Japan (Gibo et al., 2002). Preliminary indications are that the latter class also cluster closely around a single value range for ϕ 'r.

This hypothesis is developed and discussed in more detail by Bromhead in presentation of the 12th Glossop Lecture (2011). A graph which indicates this hypothesis for different British Clay is presented in Appendix D.

Ring shear testing on the Barton Clay exhibits lower values than those suggested by Barton et al. (2006). Back analysis shows that this residual strength comes out of the

144

analysis only if the 'back slip' is considered to be stronger and the procedure in Section 5.2 is used. These analyses provide confirmation that the morphological interpretation of the Cliff House slide as separate slides on D and F horizon is more likely to be correct than Barton and Garvey (2011) interpretation as a single slide on the D horizon. This view was expressed in a discussion letter by Hosseyni et al. (2012a).



Figure 10-8: The graph shows the mean value and standard deviation of the published data from back analysis on the cross section of landslides in the London Clay.

Figure 10-2 shows a histogram of results from back analyses of London Clay landslides. The range of values obtained shows a degree of natural variability combined with the uncertainties in back analyses. However, to take a higher value e.g. 14 °, from this population would be to risk failure of any residual scheme, or at least, poor performance.

11 Conclusions

11.1 Conclusions

The ring shear apparatus has proved to be an effective way to determine the residual strength of a clay soil specimen. In the literature various types of ring shear machine have been developed and the results of ring shear tests have been compared with results from other procedures. If the test is done carefully enough then many phenomena such as rate effects, soil extrusion, and friction between the platens can be managed so that a meaningful residual strength will be obtained.

In order to understand the residual strength of a landslide as distinct from the residual strength of sample clay or residual strength for a geological unit as a whole, the samples must be taken from the right place.

The major disadvantage of ring shear test is the perception that somehow it is underestimates residual strength and that it requires the correct soil to be sampled and tested, as a site may contain a range of materials. The stronger materials do not form the slip surfaces even if they are in corporated in slide mass.

Techniques (covered in Chapter 7) have been found to make testing simpler and less error-prone.

Back analysis proves to be an even more effective method of determining of residual strength, when applied correctly, which is when a series of landslides are analysed, and each has correctly observed slip surface positions and piezometric levels. In this way the back analyses in principle have to be at least as good as doing tests. If the slope is made of more or less same material, then the back analysis can be conducted for whole slide, considering ϕ'_r average. Where there are weak beds in the geological sequences, procedures have been put in place to provide bounds for the residual angle of shearing resistance, taking into account the possible strength of other materials

(Section 5.2). For many of the London Clay cases this is unnecessary⁵ but for the Barton on Sea analyses it is vital.

Back analyses occasionally throw up anomalous results. Careful inspection shows that these cases always have simple explanation such as:

- Slip surface in the wrong place (e.g. section of landslide at Upper Holloway)
- Pore water pressure is in error (i.e. section landslide at Fareham)
- Wrong model used

In particular, the railway cutting cases produced by James (1970) show all of these defects. Moreover, several of the cases are reviewed movement in remediated slopes. These cases may involve shearing through or around counterfort drains, spent ballast replacement fill or even grout. So that the back-calculated residual strength is not that of previously undisturb London Clay. While it was wrong in principle to rely on James's results (Skempton, 1985; Bromhead, 1978), the fact that they all fall into the low effective stress of the plot, means that their influence on the slope of the best-fit-line through the results is small. However, where these data are studied carefully, it is realised that the results are not very good, because many of them do not have pore pressure information and a certain number of them do not have any sensible slip surface. This is the same as analysing the stabilised slope at Barton on Sea. In these stabilised slopes there is no information about the resistance and contribution of piles and drain on the stability of the slopes.

Further more, in the railway cuttings, the deformation is small, and is an indicative of low brittelness. Therefore, that indicates the slope materials is not the London Clay any more.

Throughout the literature, cases are presented in the form of cross section without corresponding back analyses. A number of these have been analysed, but it becomes

⁵ For particular purposes, but also see sections 8.8 and 9.4.

clear that they are all lacking in value because of missing data. The gold standard cases remain:

- a) Herne Bay (Bromhead, 1978) notwithstanding some deficiences
- b) Sheppey (Dixon and Bromhead, 2002)
- c) Folkestone Warren (Hutchinson, 1969; Hutchinson & Bromhead and Lupini, 1980)
- d) Lias clay shallow slides (Chandler various years, e.g. 1977 & 1982b)

It is found that back analysis and ring shear results only agree when testing is appropriate.

Overall, if the back analysis is done corretcly on the a big population of small, medium and large slips and results compare with the residual strength of right material in a ring shear apparatus then we pretty much understand the residual strength of these landslides.

11.2 Future research

The residual shear strength of British Clays from four strata have been studied and compared in this research. For the reason of space and word limit just two of them are presented in detail in this thesis.

None of the other strata for which data points and case records are available are as comprehensive as for these four strata dealt with in this research, nor are they so consistent internally or with each other. However, for the future works suggestion are listed below.

This research is concerned with a small number of research questions, and it attempts to provide answers not only by the collection of new, primary, data but instead by reviewing a body of already published data.

- To apply the back analysis technique to every published landslide cross section in every significant stratum in the UK geological records. Listing the results in an appropriate dataset to complement the UK National Landslide Dataset. (The word limit of a Kingston University Thesis has prevented a full description of work done on Gault and Lias)
- To explore in more detail the soil fabrics in bedding-controlled shear surfaces, in terms of mineralogy, moisture content, strength etc.
- To explore in more detail about the mode of formation of what are taken to be sedimentologically-controlled weak horizons occurring throughout the geological record.
- While British clays are dominated by illite, a small number of strata are dominated by smectites. This may even be true of the Fuller's Earth of Jurassic age, but certainly includes examples from the Panama Canal (Lutton and Banks 1970), from western Canada (Cruden et al. 1991) and Japan (Gibo et al. 2002). Preliminary indications are that the latter class also cluster closely around a single value for φ'r.
- To extend these techniques to soils encountered outside the UK.

References

- Barton, M.E. (1973) 'The degradation of the Barton Cliffs of Hampshire'. *Quarterly Journal of Engineering Geology*, 6, pp. 423-40.
- Barton, M.E. (1984) ' The preferred path of landslide shear surface in over-consolidated clays and soft rocks', *Proc of the 4th international symposium on landslide*, Toronto, Canada, 3, pp. 75-79.
- Barton, M.E. and Coles, B.J. (1984) 'The characteristics and rates of the various slope degradation processes in the Barton Clay Cliffs of Hampshire', *Quarterly Journal of Engineering Geology and Hydrogeology*, 17, pp. 117-136.
- Barton, M.E., Hillier, S. and Watson, G.V.R. (2006) 'The Slip surface in the D zone of the Barton clay', *Quarterly Journal of Engineering Geology and Hydrogeology*, 39(4), pp. 357-370.
- Barton, M.E. and Garvey, P.M. (2011) 'Reactivation of landsliding following partial cliff stabilization a Barton-on-Sea, Hampshire, UK', Quarterly Journal of Engineering Geology and Hydrogeology, 44(2), pp. 233-248
- Berkeley Thorn, R. and Roberts, A.C. (1981) *Sea defence and coast protection works*, London: Thomas Telford Ltd.
- Bishop, A.W. and Henkel, D.J. (1957) *The measurement of soil properties in the triaxial test*. London: Edward Arnold.
- Bishop A.W., Webb D.L. and Lewin P.I. (1965) 'Undisturbed samples of London Clay from the Ashford Common shaft: strength-effective stress relationships', *Géotechnique*, 15, No.1, pp. 1-31.
- Bishop, A.W., Green, G.E., Garga, V.K., Andresen, A. and Brown, J.D. (1971) 'A new ring shear apparatus and its application to the measurement of residual strength', *Géotechnique*, 21(4), pp. 273-328.

Borowicka, H. (1965) 'The influence of the colloidal content on the shear strength of clay', Proceedings of 6th International Conference on Soil Mechanics, Rock Mechanics & Waterways, pp. 175-178.

Bromhead, E.N. (1972) ' A large coastal landslide at Herne Bay, Kent', Arup J, 7, pp. 12-14.

- Bromhead, E.N. (1978)'Large landslides in London clay at Herne Bay, Kent', *Quarterly Journal of Engineering Geology and Hydrogeology*, **11**, pp. 291-304.
- Bromhead, E.N. (1979) 'A simple ring shear apparatus', Ground Engineering, 12(5), pp. 40-44.
- Bromhead, E.N. and Curtis, R.D. (1983)'A comparison of alternative methods of measuring the residual strength of London clay', *Ground Engineering*, 16(3), pp. 39-41.
- Bromhead, E.N. (1986) 'Shear strength of discontinuities', *The stability of slopes*. Blackie A &P, pp. 60-72.
- Bromhead, E.N. and Dixon, N. (1986) 'The field residual strength of London clay and its correlations with laboratory measurements, especially ring shear test', *Géotechnique*, 36(3), pp.449-452.

Bromhead, E.N. (1992) The Stability of Slopes. 2nd edn. Glasgow: Blackie.

- Bromhead, E.N., Harris, A.J. and Ibsen, M.L. (1999) 'Statistical variability of ring shear test results on a shear zone in London Clay', *International Symposium on Slope Stability Engineering: Geotechnical and Geo-environmental aspect.*
- Bromhead, E.N., Dixon, N. and Ibsen, M.L. (2000) Landslide in research, theory and practice: proceedings of the 8th International Symposium on Landslides, Cardiff 26-30 June. Thomas Telford.
- Bromhead, E.N., Fourlas, G., Zemichael, A.A. (2001) 'Applications of 3D Slope Stability Analysis', Geotechnical Engineering: Meeting Societies' Needs, Hong Kong, pp. 685-690.
- Bromhead, E.N., Ibsen, M.L. (2004) 'Bedding-controlled coastal landslides in Southeast Britain between Axmouth and the Thames Estuary', *Landslides*, 1 (2), pp. 131-141.

- Bromhead, E.N. and Martin, P.L. (2004) 'Three-dimensional limit equilibrium analysis of the Taren landslide', *Advances in Geotechnical Engineering: The Skempton Conference (Eds R. J. Jardine, D. M. Potts, K. G. Higgins)*, London: Thomas Telford, 2, pp. 789-802.
- Bromhead, E.N. (2004a) 'Reflections on C. H. Gregory's New Cross landslide of 1841', Advances in Geotechnical Engineering: The Skempton Conference (Eds R. J. Jardine, D. M. Potts, K. G. Higgins), Thomas Telford: London, 2, pp. 803-814.
- Bromhead, E.N. (2004b)'Landslide slip surfaces: their origins, behaviour and geometry', *Proceedings of. 9th Int. Symp. on Landslides*, Amsterdam: Balkema, London, 1, pp. 3-22.
- Bromhead,E.N. (2005)'Geotechnical structures for landslide risk reduction', In: Glade T, Malcolm G A, J CM (eds.) *Landslide Hazard and Risk*, John Wiley & Sons, New York, pp. 549-594.

Bromhead, E.N. (2010) Personal communication with S. Hosseyni, June.

Bromhead, E.N. (2011) The 12th Glossop Lecture. 13th November.

Bromhead, E.N. (2012) Personal communication with S. Hosseyni, May.

- Bromhead, E.N. and Hosseyni, S. (2012) 'When the analysis won't converge why, and what to do about it', *Proceedings of 11th (ISL and the 2nd North American Symposium on Landslides, Landslides and Engineering Slopes: Protecting Society through Improved Understanding (ed. E. Eberhardt & C. Froese & K. Turner and S. Leroueil,*, June 2-8, 2012, Banff, Alberta, Canada.
- Bromhead, E.N., Hosseyni, S. and Torii, N. (2012) 'Soil slope stabilization', in Clague, J.J and Stead, D. (eds.) *Landslides Types*. Mechanisms and Modeling . Cambridge University Press, pp. 252-266
- British Standard Institution (1990) BS1377-2: 1990. Methods of test for soils for civil engineering purposes. Part 2: Classification tests. London: British Standard Institute.
- British Standard Institution (1990) BS1377-7: 1990. Methods of test for soils for civil engineering purposes. Part 7: shear strength tests. London: British Standard Institute.

- Burton, E.S.T.J. (1925) The Barton Beds of Hampshire. British Association for the Advancement of Science (Southampton), pp. 312-314.
- Burton, E.S.T.J. (1933) 'Faunal horizons of the Barton Beds in Hampshire', *The Geologists'* Association, 44, pp. 131-167.
- Chandler, R.J. (1966) 'The measurement of strength in triaxial compression', *Geotechnique*, 16 (3), pp. 181-186.
- Chandler, R.J. (1972b)' Periglacial mudslides in Vestspitsbergen and their bearing on the originof fossil 'solifluction' shears in low angled clay slopes', *Quarterly Journal of Engineering Geology and Hydrogeology*, 5, pp. 223-241.
- Chandler, R.J. (1977) 'Back analysis technique for slope stabilization works: a case record', *Géotechnique*, 27 (4), pp. 479-495.
- Chandler, R.J., and Skempton, A.W. (1974) 'The design of permanent cutting slopes in stiff fissured clays'. *Géotechnique*, 24, pp. 457-466.
- Chandler, R.J. (1979) 'Stability of a structure constructed in a landslide', *Proceedings of.7th Eur. Conf. on Soil Mechanics & Foundation Eng. Brighton,* 3, pp. 175-182.
- Chandler, M.P., Parker, D.C. and Selby, M.J. (1981) 'An open-sided field direct shear box with applications in geomorphology', *Technical Bulletin, 27, British Geomorphological Research Group*, Geo Abstracts Ltd., Norwich.
- Chandler, R.J. (1982b) 'Lias Clay Slope Sections and Their Implication for the Prediction of Limiting or Threshold slope Angles', *Earth Surface Processes and Landforms*, 7 (5), pp. 427-438.
- Chandler, R.J. and Hardie, T.N. (1989) 'Thin-sample technique of residual strength measurement', *Géotechnique*, 39(3), pp. 527-531.
- Clarke, K. (2005) *Residual shear strength with special reference to rate effects*. PhD thesis, Kinston University London.

- Collotta, T., Cantoni, R., Pavesi, U., Ruberl, E. and Moretti, P.C. (1989) 'A correlation between residual friction angle, gradation and the index properties of cohesive soils', *Geotechnique*, 39(2), pp. 343-346.
- Coolling, A. and Smith, D.B. (1935) 'The shearing resistance of soils', *Journal of the Institution of Civil Engineers*, 3(7), pp. 333-343.
- Cooper, M.R., Bromhead, E.N., Petley, D.J., Grant, D.I. (1998) 'The Selborne cutting stability experiment', *Géotechnique*, 48 (1), pp. 83-101.
- Cross, M. (2010) 'The use of a field open-sided direct shear box for the determination of the shear strength of shallow residual and colluvial soils on hillslopes in the south', *North West Geography*, 10(2), pp. 8-18.
- Cruden, D.M., Thomson, S., Hoffmann, B.A. (1991) 'Observation of Graben geometry in landslides', *The international conference on slope stability*, Isle of Wight, pp.33-36.
- Cruden, D.M and Varnes, D.J. (1996) Landslide types and processes. In Special Report 247: Landslides: Investigation and Mitigation, Transportation Research Board, Washington D.C., pp. 36-75.
- Deschamps, R. and Yankey, G. (2006) 'Limitations in the back analysis of strength from failures', J. Geotech and Geoenvir. Eng, ASCE, 132, pp. 532-536.
- Dixon, N. (1986) The mechanics of coastal landslides in London clay at Warden Point, Isle of Sheppey. PhD thesis, Kingston Polytechnic.
- Dixon, N., Bromhead, E.N. (1991) 'The mechanics of first time slides in the London Clay cliff at the Isle of Sheppey, England', *Proceedings, International Conference on Slope Stability Engineering: Developments and Applications*, Isle of Wight, UK, April 15-18, pp. 277-282.
- Dixon, N., Bromhead, E.N. (2002) 'Landsliding in London clay coastal cliffs'. *Quarterly Journal of Engineering Geology and Hydrogeology*, 35 (4), pp.327-343.
- Dunbaven, M. (1980) 'Discussion on Large landslides in London Clay at Herne Bay', *Quarterly Journal of Engineering Geology and Hydrogeology*, 13 (February), pp.63-65.

Duncan, J.M. and Stark, T.D. (1992) 'Soil strengths from back analysis of slope failures', *Proc* Specialty Conf. Stability and Performance of Slopes and Embankments II, ASCE, Berkeley, 1, pp. 890-894.

Duvivier, J. (1940) 'Cliff stabilisation works in London Clay', J. Instn cir. Engrs, 14, paper 5233.

- Fearon, R.E. (1998) *The behaviour of structurally complex clay from an Italian landslide (BL)*. PhD thesis. London, City University.
- Fearon, R.E., Chandler, R.J. and Bommer, J.J.(2004) 'An investigation of the mechanisms which control soil behaviour at fast rates of displacement', *The Skempton conference*, London, 1, pp. 441-452.
- Fell, R., MacGregor, P., Stapledon, D. and Bell, G. (2005) *Geotechnical engineering of dams*. Leiden, New York : A.A. Balkema.
- Fleischer, S. and Scheffler, H. (1972) 'Problem oriented shearing technology for the determination of the drained shear strength of cohesive soils', *Proceeding of 7th European Conference on Soil Mechanics*, Brighton 2, pp. 41-48.
- Fort, D.S., Clark, A.R. and Cliffe, D.G. (2000) 'The investigation and monitoring of coastal landslides at Barton-on-Sea', *Landslides in Research*, Theory and Practice: the 8th ISL, Cardiff, London: Thomas Telford, pp. 567-572.
- Garvey, P. (2007) A study of the reactivation of landsliding at Barton-on-Sea, Hampshire, following stabilisation works in the 1960s. MSc dissertation, University of Southampton [Online]. Available at: http://eprints.soton.ac.uk/69767/ (Accessed: March 2011).
- Gould, J. P. (1960) A study of shear failures in certain marine sediments. *Proc. ASCE Research Conference on Shear Strength of Cohesive Soils*, Boulder, Colorado. pp. 615-641.
- Gregory, C.H. (1844) 'On railway cutting and embankments, with an account of some slips in the London clay', *Min.Proc.Inst.Civ. Eng.*, 3, pp. 135-145.
- Gruner, H.E. and Haefeli, R. (1934) 'Contribution to the investigation of the phisical and static behaviour of cohesive soils', *Schweiz Bauzig*, 103, pp. 171-174 and 185-188.

- Haefeil, R.(1951)' Investigation and measurements of the shear strengths of saturated cohesive soils', *Geotechnique*, 2(3), pp. 186-208.
- Harris, A. J. and Watson, P.D.J. (1997), 'Optimal procedure for the ring shear test', *Ground Engineering*, 30(6), pp. 26-28.
- Hawkins, A.B. and Privett, K.D. (1985), 'Measurement and use of residual shear strength of cohesive soils', *Ground Engineering*, 18(8), pp. 22-29.
- Henkel, D.J. and Skempton A.W. (1955) 'A landslide at Jackfield, Shropshire, in a heavily overconsolidated clay', *Géotechnique*, 5(2), pp. 131-137.
- Henkel, D.J. (1956) 'Discussion on Watson (1956), Proc Inst. Civ. Engrs, 5, pp. 320-323.
- Henkel, D.J. (1957) 'Investigations of two long term failures in London Clay slopes at Wood Green and Northolt', *Proceedings of. 4th Int. Conf. on Soil Mechanics & Foundation Eng.*, London, 2, pp. 315-320.
- Hoek, E. and Bray, J.W. (1974). *Rock Slope Engineerine 3rd. Edition*. london: Institute of Mining and Metallurgy.
- Hosseyni, S., Torii, N. and Bromhead, E.N. (2011) 'Residual strengths of some British clays', *Proceedings of 2nd World Landslide Forum, "PUTTING Science into Practice"*, October 3-9, Rome.
- Hosseyni, S., Torii, N. and Bromhead, E. N. (2012) 'Discussion on 'Reactivation of landsliding following partial cliff stabilization at Barton-on-Sea, Hampshire' by M. E. Barton & P. Garvey', *Quarterly Journal of Engineering Geology and Hydrogeology*, 45, pp. 125-128.
- Hosseyni, S., Torii, N. and Bromhead, E.N. (2012) 'Limitations of, and sources of inaccuracy in, back analysis', Proceedings of 11th (ISL and the 2nd North American Symposium on Landslides, Landslides and Engineering Slopes: Protecting Society through Improved Understanding (ed. E. Eberhardt & C. Froese & K. Turner and S. Lerouei, June 2-8, 2012, Banff, Alberta, Canada.
- Hutchinson, J.N. (1963) 'Introductory Notes for the Informal Discussion on Landslips', *The* ISSMFE Meeting, 9th October1963 at the ICE, London, pp. 15-18.

- Hutchinson, J.N. (1967) 'The free degradation of London Clay cliffs', *Proc.Geotech. Conf. Oslo*, 1, pp. 113-8.
- Hutchinson, J.N. (1969) 'A reconsideration of the coastal landslides at Folkestone Warren, Kent', *Géotechnique*, 19 (1), pp. 6-38.
- Hutchinson, J.N. (1970) 'A Coastal Mudflow on the London Clay Cliffs at Beltinge, North Kent', Geotechnique, 20(4), 412-438.
- Hutchinson, J.N. and Gostelow, T.P. (1976) 'The development of an abandoned cliff in London Clay at Hadleigh, Essex', *Philosophical Transactions of the Royal Society of London Series A*, 283, pp. 557-604.
- Hutchinson, J.N., Bromhead, E.N. and Lupini, J.F. (1980) 'Additional observation on the Folkestone Warren landslides', *Quarterly Journal of Engineering Geology and Hydrogeology*, 13 (1), pp. 1-31.
- Hutchinson, J.N. (1983) 'Methods of locating slip surfaces in landslides', *Bull. Assoc. Eng. Geol*, 20, pp. 235-252.
- Hvorslev, M.J. (1936) 'A ring shearing apparatus for the determination of shearing resistance and plastic flow of soils', *Proceedings of 1st International Conference of Soil Mechanics*, Boston 2, pp. 125-129.
- Hvorslev, M.J. (1937) 'Physical properties of remoulded cohesive soils (Uber Die Festigkeitseigenschaften Gestorter Bindiger Boden)', *Danmarks Naturvidensk Samfund*. English translation (1969), U.S. Waterways Expriment Station, Vicksburg, Mississippi.
- Hvorslev, M.J. (1939) 'Torsion shear tests and their place in the determination of shearing resistance of soils', *Proceedings of American Society of Testing Material*, 39, pp. 999-1022.
- Idriss, I.M. (1985) 'Evaluating seismic risk in engineering practice', *Proceedings of the 11th International Conference of Soil Mechanics and Foundation Engineering*, San Francisco, CA , 1, pp. 255-320.
- James, P.M. (1970) *Time effects and progressive failure in clay slopes*. PhD thesis. Imperial College, London University.

- Kanji, M.A. (1974) 'The relationship between drained friction angles and Atterberg limits of natural soils', *Geotechnique*, 24(4), pp. 671-674.
- Kennard, M.F. and Bromhead. E.N. (2000)' Carsington Dam-The Near-Miss Which Became a Bulls-Eye', *Proceedings of. of the 2nd Forensic Congress, Puerto Rico*, May 21-23. Eds K. L. Rens, O. Rendon-Herrero, and P. A. Bosela, ASCE, Reston Va, pp. 102-111.
- Kenney, T.C. (1966) 'Residual strength of fine-grained minerals and mineral mixture', Norwegian geotechnical institute publication, 68, pp. 53-57.
- Kenney, T.C. (1967) 'The influence of mineral composition on the residual strength of natural soils', *Preoceeding of geotechnical conference*, Oslo, 1, pp. 123-129.
- King C. (1981), 'The stratigraphy of the London Basin and associated deposits', *Tertiary Research Special Paper*, 6, Backhuys, Rotterdam.
- King C. (1991) Stratigraphy of the London Clay (Early Eocene) in the Hampshire Basin. PhD Thesis, Kingston Polytechnic.
- Lagatta, D.P. (1970) Design of the rotation shaer machine, Harvard Soil Machine, [Report], 86, pp. 10-55.
- Lupini, J. F, Skinner, A. E., and Vaughan, P. R. (1981) 'The drained residual strength of cohesive soils', *Géotechnique*, 31(2), pp. 181–213.

Maksimovic, M. (1989) 'On the residual strength of clays', Géotechnique, 39(2), pp. 347-351.

- Maksimovic, M. (1996) A family of nonlinear failure envelopes for non-cemented soils and rock discontinuities', *Electron. J. Geotech. Engrg.*, 1, pp. 1-36.
- McGown, A., Roberts, A.G. and Woodrow, L.K.R. (1987) 'Long term pore pressure variations within coastal cliffs in North Kent, UK', *The 9th European Conference Soil Mechanics and Foundation Engineering*, Dublin, pp. 455-460.
- Mesri, G. and Cepeda-Diaz, A.F.(1986) 'Residual shear strength of clays and shales', *Géotechnique*, 36(2), pp. 269-274.

- Michalowski, R.L. (1989) 'Three-dimensional analysis of locally loaded slopes', *Géotechnique*, 39(1), pp. 27-38.
- Moore, R. (1991) 'The chemical and mineralogical controls upon the residual strength of pure and natural clays', *Geotechnique*, 41(4), pp. 35-47.
- Moore, R. and Brunsden, D. (1996) 'Physico-chemical effects on the behaviour of a coastal mudslide', *Geotechnique*, 46(2), pp. 259-278.
- Morgenstern, N.R. and Price, V.E. (1967) 'A Numerical Method for Solving the Equations of Stability of General Slip Surfaces', *The Computer Journal*, 9(4), pp. 388-393.
- Parathiras, A.N. (1994) Displacement rate effects on the residual strength of soils. PhD thesis, Imperial College London (University of London).
- Petley, D.N. (1994) The deformation of mudrocks. Ph.D. thesis, University of London.
- Ramiah, B.K., Dayalu, N.K. and Prurshotharmaraj, P. (1970) 'Influence of chemicals on residual strength of silty clay', *Soils and foundations*, 10, pp. 25-36.
- Robinson , A. W. H. 1955. 'The Harbour Entrances of Poole, Christchurch and Pagham', *Geology Journal*, 121, pp. 33-50.
- Sassa,K., Fukuoka, H., Lee, J.H. and Zhang, D.X. (1992) 'Measurement of the apparent friction angle during rapid loading by the high-speed high-stress ring shear apparatusinterpretation of the relationship between landslide volume and the apparent friction during motion', *Proceedings of. 6th International Symposium on Landslide*, 1, pp. 545-552.
- Sassa, K. (1995) 'Access to the dynamics of landslides during earthquakes by a new cyclic loading ring-shear apparatus', *Proceedings 6th International Symposium on Landslide*,3, Rotterdam: Balkema, pp. 1919-1939.
- Sembenelli, P., and Ramirez, A. L. (1969) 'Measurement of Residual Strength of Clay with a Rotating Shear Machine', *Proceedings of the 7th International Conference on Soil Mechanics, Mexico*, 3, pp. 528-529.

- Seyček, J. (1978) 'Residual shear strength of soils', *Bulletin of Engineering Geology and the Environment*, 17(1), pp. 73-75.
- Shoaei, Z. and Sassa, K. (1994) 'Basic Study on the Shear Behavior of Landslides During Earthquakes', *Bulletin of the Disaster Prevention Research Institute*, Kyoto University, Kyoto, 44, pp. 1-43.
- Skempton, A.W. (1948) 'The rate of softening in stiff-fissured clays, with special reference to London clay', *The 2nd International Conference on Soil Mechanics and Foundation Engineering*, 2, pp. 50-53.

Skempton, A.W. (1964) 'Long-term stability of clay slopes', Géotechnique, 14 (2), pp. 77-102.

- Skempton, A.W., Petley, D.J. (1967) 'The strength along structural discontinuities in stiff clay'. The Geotechnical Conference, Oslo, pp. 29-46.
- Skempton, A.W. (1970) 'First-time slides in over-consolidated clays', Géotechnique, 20, pp. 1-5.
- Skempton, A.W. and Chandler, R.J. (1975) 'The design of per-manent cutting slopes in stiff fissured clays', *Géotechnique*, 25, pp. 425-427.
- Skempton, A.W. (1978) 'Slope stability of cuttings in brown London clay', *Proceedings of 9th International Conference on Soil Mechanics and Foundation Engineering*, pp. 261-270
- Skempton, A.W. (1985) 'Residual strength of clay in landslides, folded strata and laboratory', *Géotechnique*, 35 (1), pp. 3-18.
- Spencer, E. (1967) 'A method of analysis of the stability of embankments assuming parallel inter-slice forces', *Geotechnique*, 17(1), pp. 11-26.
- Stark, T.D. and Eid, H.T. (1992) 'Comparison of Field and Laboratory Residual Strengths', ASCE Specialty Conference Stability and Performance of Slopes and Embankments-II, University of California, Berkeley, CA, ASCE, New York.
- Stark, T.D., Vettel, J.J. (1992) 'Bromhead Ring Shear Test Procedure', Geotechnical testing journal, 15(1), pp. 24-32.

- Stark, T.D. and Eid, H.T. (1993) 'Modified Bromhead ring shear apparatus', *Geotechnical testing journal*, 16(1), pp. 100-107.
- Stark, T.D. and Eid, H. (1994) 'Drained residual shear strength of cohesive soils', J. Geotech. Eng. ASCE, 120, pp. 856-871.
- Stark, T.D. and Poeppel, A.R. (1994) ' Landfill Liner Interface Strengths from Torsional-Ring-Shear Tests', *Journal of Geotechnical Engineering*, 120(3), pp. 597-615.
- Stark, T.D. and Hussain, M. (2010) 'Drained Residual Strength for Landslides', *Proceedings of Specialty Conference GeoFlorida 2010*, ASCE, Orlando, pp. 3217-3226.
- Stopher, H.E. and Wise, E.B. (1966) 'Coast erosion problems in Christchurch Bay', *Journal of the Institution of Municipal Engineers*, 93, pp. 328-332.
- Summers, L. and Maddrell, R.J. (1978) 'Barton-on-Sea: development of an alternative approach to coast protection', *Unpublished paper distributed at the Geological Society*, Engineering Group Regional Meeting at Southampton [Online]. Available at: <u>http://www.maddrell.org.uk/publications.php</u> (Accessed: March 2011).
- Taylor, D.W. (1948) 'Stability of earth slopes', *Journal of the Boston Society of Civil Engineers*, 24(3), pp. 197-246.
- Taylor, D.P. (1998) Fast shearing of cohesive soil using ring shear apparatus. PhD thesis: University of Warwick.
- Terzaghi, K., Ralph, B., Peck, G., Mesri, G., (1996) Soil Mechanics in Engineering Practice, John Wiley & Sons Inc.
- Tika, T. and Hutchinson, J. N. (1999) 'Ring shear tests on soil from the Vaiont slip surface', *Géotechnique*, 49, pp. 59-74.
- Tika, T.E., Vaughan, P.R. and Lemos, L.J. (1996) 'Fast shearing of pre-existing shear zones in soil', *Géotechnique*, 46(2), pp. 197-233.
- Tiwari, B., and Marui, H. (2003) 'Estimation of residual shear strength of Toyoura sand-Bentonite-Kaolin Mixture', *Journal of Japan Landslide Society*, 40(2), pp. 124-133.

- Tiwari, B., Marui, H., and Bhattarai, P. (2004) 'Mechanism of slow moving active landslides along mudstone formations', *Proceedings of IX International Symposium on Landslides*, Rio De Generio, Taylor and Francis Group, London, 2, pp. 1177-1182.
- Tiwari, B., and Marui, H. (2005) 'A new method for the correlation of residual shear strength of the soil with mineralogical composition', *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 131(9), pp. 1139-1150.
- Tiwari, B., Brandon, T.L., Marui, H. and Tuladhar, G.R. (2005) 'Comparison of residual shear strengths from back analysis and ring shear tests on undisturbed and remolded specimens', *J. Geotech.& Geoenvir. Eng, ASCE*, 131, pp. 1071-1079.
- Tiwari, B. (2007) 'Residual shear strength of tertiary mudstone and influencing factors', Progress in Landslide Science, Springer Special publication, pp. 135 – 145.
- Tiwari, B. and Ajmera, B. (2011 a) 'A new correlation relating the shear strength of reconstituted soil to the proportions of clay minerals and plasticity characteristics', *Applied Clay Science*, 53, pp. 48-57.
- Tiwari, B. and Ajmera, B. (2011 b)'Estimation of fully softened shear strength with index properties', *Proceedings of 14th Asian Regional Conference of ISSMGE*, Hong Kong, 432, pp. 1-6.
- Tiwari, B. and Ajmera, B. (2012)'New Correlations Equations for Compression Index of Remolded Clays', *Journal of Geotechnical and Geoenvironmental Engineering*, 136(6), pp. 757-762.
- Townsend, F.C. and Gilbert, P.A. (1973) 'Tests to measure residual strengths of some clay shales', *Géotechnique*, 23(2), pp. 267-271.
- Vaughan, P.R. and Walbancke, H.J. (1975) 'The Stability of Cut and Fill Slopes in Boulder Clay', Symposium Engineering Behaviour of Glacial Material, Birmingham, 209-219.
- Voight, B. (1973) 'Correlation between Atterberg plasticity limits and residual shear strength of natural soils', *Geotechnique*, 23(2), pp. 265-267.

- Watson, J.D. (1956) 'Earth movements affecting LTE railway in deep cutting east of Uxbridge', *Proc. Inst. Civ. Engrs.*, 11, pp. 320-323.
- Wise, E.V.A. (1957) The stability of natural slopes in overconsolidated fissured clay. MSc dissertation: Imperial College, London University.
- Stopher, H.E. and Wise, E.B. (1966) 'Coast erosion problems in Christchurch Bay', Journal of the Institution of Municipal Engineers, 93, 328-332.
- Wood A. M. (1955) ' Folkestone warren landslips: investigations, 1948-1950 (Residual strength by triaxial test)', *Railway Engineering Division Meeting 22 Feb. 1955*, 56, pp. 410 428.
- Wood A. M. (1967) 'Coastal Stabilisation at Barton-on-Sea', *Cir. Eng. Public Works Review*, 62, pp. 1401-2.
- Wood, A.M. (1971) 'Engineering aspects of coastal landslides', *Institution of Civil Engineers*, 50, pp. 257-276.
- Yamasaki, T., Mayumi, T. (2000) 'Ring shear characteristics of high purity clay mineralscorrelation with natural slip surface clay', *Landslides (Journal of Japan Landslide Society)*, 37(2), pp. 30-39.
Appendix A



Cross sections and results of back analyses of landslides at Barton on Sea

Figure A1: Show cross section of landslide at Central Area with the cliff profiles in 1974 and the corresponding slip surface which has been employed for back analysis.

Table A1: Results of back analysis of the cross section of landslide at Hoskin's Gap. The average ϕ'_{r} and different residual friction angle of the weak layer (ϕ'_{r1}) for various ratio of ϕ'_{r1} and ϕ'_{r2} are presented.

Cross Section:	Tit	1000	and the	Centr	al Area		23	131
Condition of GWL	Real pi	ez. Line	Upper	piez. ne	Averag	e piez. ne	Lower	piez. ne
Name	7-R		7-U		7-A		7-L	
φ', (°)	φ' _{r1} (°)	φ'r2 (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)
φ' _{rav}	*	*	14.72	14.72	12.19	12.19	11.50	11.50
First Ratio of ϕ'_r	*	*	13.99	16.00	11.62	13.00	10.26	13.00
2	*	*	13.42	17.00	10.90	14.00	9.43	14.00
3	*		12.85	18.00	10.19	15.00	8.61	15.00
4	*	*	12.28	19.00	9.47	16.00	7.78	16.00
5	*		11.71	20.00	8.76	17.00		



Figure A2: Cross section of the Hoskin's Gap Landslide with the cliff profiles of 1967 and 2006. The slip surface in 1960s which has been employed for back analysis is also shown.

Table A2: Results of back analysis landslide at Hoskin's Gap show the average ϕ 'r and different residual friction angle of the weak layer (ϕ 'r1) for various value for ϕ 'r2.

Cross Section:				Hoski	in's Gap		1	19
Condition of GWL	Real pi	ez. Line	Uppe Li	r piez. ne	Averag	e piez. ne	Lower	r piez. ne
Name	9-R		9-U		9.	A	9-L	
φ', (°)	φ'r1 (°)	φ'r2 (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' ₁₁ (°)	φ' _{r2} (°)	φ'r1 (°)	φ'r2 (°)
φ' _{rav}	*	*	13.60	13.60	11.75	11.75	11.23	11.23
First Ratio of ϕ'_r	*	*	12.91	15.00	10.90	13.00	10.64	12.00
2	*	*	12.41	16.00	10.22	14.00	9.88	13.00
3	*	*	11.91	17.00	9.55	15.00	9.13	14.00
4	*	*	11.42	18.00	8.871	16.00	8.37	15.00
5	*	*	10.92	19.00	8.20	17.00	7.61	16.00





Figure A3: The cross section of the Hoskin's Gap West landslide by Barton and Garvey (2011) (the above figure. The below shows the selected slip surface of this cross section for back analysis. This section includes cliff profile of 1967 and slip slip surface prior to the stabilization works.



Figure A4: The modified cross section of Hoskin's Gap West which shows cliff profile in 1967 and 2006. The slip surface is modified into F Zone.

Table A3: Results of back analysis cross section of landslide at Hoskin's Gap West, shows the average ϕ 'r and different residual friction angle of the weak layer (ϕ 'r1) for various ratio of ϕ 'r1 and ϕ 'r2.

Cross Section:		and the		Hoskin's	Gap We	st		
Condition of GWL	Real piez. Line		Upper Lir	Upper piez. Line		Average piez. Line		r piez. ne
Name	11	R	11	-U	11	-A	11	I-L
φ'r (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (*)
φ' _{rav}	*	*	12.08	12.08	10.28	10.28	10.04	10.04
First Ratio of ϕ'_r		*	11.45	13.00	9.60	11.00	9.09	11.00
2	*	*	10.75	14.00	8.67	12.00	8.10	12.00
3	*	*	10.06	15.00	7.73	13.00	7.12	13.00
4	*	*	9.37	16.00	6.80	14.00	6.13	14.00
5	*	*	8.671	17.00	5.87	15.00	5.15	15.00



Figure A4: Cross section of Cliff House Landslide with cliff profile 1993 and deep-seated shear surface in Zone D by Barton and Garvey (2011).

Table A4: Results of back analysis of un-modified cross section of landslide at Cliff House, shows the average ϕ 'r and the first ratio of ϕ 'r1 and ϕ 'r2.

Cross Section:		17.14		Cliff	House			
Condition of GWL	Real piez. Line 14-R		Upper	r piez. ne	Averag	ge piez. ne	Lower piez. Line	
Name			14-U		14-A		14-L	
φ', (°)	φ'r1 (°)	φ' _{r2} (°)	φ'r1 (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (°)
φ' _{rav}	20.60	20.60	22.45	22.45	20.60	20.60	19.11	19.11
First Ratio of ϕ'_r	15.94	25.00	20.62	25.00	16.90	25.00	18.32	20.00



Figure A5: Modified cross section of landslide at Cliff House, showing cliff profile of 1993 and 2007. Two separate shear surfaces in D horizon and F horizon as discussed in Chapter 6.

Table A5: Results of back analysis of modified cross section of landslide at Cliff House, shows the average ϕ 'r and different residual friction angle of the weak layer (ϕ 'r1) for various ratio of ϕ 'r1 and ϕ 'r2.

Cross Section:	Modified Cliff House								
Condition of GWL	Real pi	ez. Line	Upper p	iez. Line	Averag	e piez. ne	Lower piez. Line		
Name	14N-R		14N-U		14N-A		14N-L		
φ', (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ' _{r2} (°)	φ' _{r1} (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (°)	
φ' _{rav}	12.31	12.31	13.47	13.47	11.30	11.30	10.35	10.35	
First Ratio of	12.17	13.00	13.12	15.00	10.78	13.00	9.77	12.00	
2	11.96	14.00	12.89	16.00	10.48	14.00	9.43	13.00	
3	11.75	15.00	12.65	17.00	10.17	15.00	9.08	14.00	
4	11.55	16.00	12.42	18.00	9.87	16.00	8.74	15.00	
5	11.34	17.00	12.19	19.00	9.56	17.00	8.39	16.00	
6	11.13	18.00	*	*	*	*	*	*	



Figure A6: Shows how to define range of $\phi' r_1$ for the cross section of landslides at CAA, HG, HGW, MCH using Upper piezometric conditions (as explained in Section 5.2).

Appendix B

Experimental results of Barton Clay samples from landslides at Barton on Sea

Table B1: The results of Bromhead ring shear test on the samples from the exposure of slip surface in the D horizon of Barton Bed in the frontage of cliff at Naish Farm in Barton on Sea in May 2010.

13.823	Tes	Test 1		t 2	Tes	t 3	Tim?le
o' (kN/m2)	τ (KN/m2)	φ', (*)	τ (kN/m2)	φ', (*)	τ (kN/m2)	φ', (°)	Average φ', (*)
15.24	3.2	12.0	3.14	11.64	3.23	12.0	11.8
27.48	4.9	10.1	4.32	8.93	4.88	10.1	9.7
51.96	10.4	11.3	8.46	9.25	10.42	11.3	10.6
76.44	15.1	11.2	11.69	8.69	15.13	11.2	10.4
100.92	19.4	10.9	15.26	8.60	19.45	10.9	10.1
15.24	3.1	11.3	3.0	11.0	3.1	11.3	11.2

Table B2: The tests results on the samples from the exposure of slip surface in the D horizon of Barton Bed in the frontage of cliff at Naish Farm in Barton on Sea in July 2011 using Bromhead Ring Shear Apparatus.

	Test 1		Tes	Test 2		Test 3		t 4	1220	
O'kN/m ²)	τ (kN/m²)	φ', (*)	τ (kN/m²)	φ', (°)	τ (kN/m²)	φ' _r (°)	τ (kN/m²)	ф', (*)	Average ¢', (°)	
15.24	2.49	9.28	2.44	9.10	2.43	9.06	3.14	11.6	9.8	
27.48	3.58	7.42	4.14	8.57	3.88	8.04	4.32	8.9	8.2	
51.96	7.28	7.98	7.76	8.49	7.11	7.79	10.55	11.5	8.9	
76.44	9.11	6.80	9.59	7.15	9.38	7.00	12.12	9.0	7.5	
100.92	11.16	6.31	11.85	6.70	10.91	6.17	13.87	7.8	6.8	
125.4	13.47	6.13	14.43	6.56	12.56	5.72	15.52	7.1	6.4	

	Test 1		Tes	Test 2		it 3		
σ' (kN/m2)	τ (kN/m2)	φr(*)	τ (kN/m2)	фr(*)	τ (kN/m2)	фr(*)	Average or(*)	
15.24	2.75	10.22	2.92	10.85	2.83	10.52	10.5	
27.48	4.83	9.97	4.97	10.25	4.88	10.08	10.1	
51.96	9.29	10.13	9.55	10.41	9.55	10.41	10.3	
76.44	10.94	8.15	11.47	8.53	10.90	8.12	8.3	
100.92	12.99	7.34	13.12	7.41	12.86	7.26	7.3	
125.4	14.69	6.68	15.26	6.94	14.69	6.68	6.8	

Table B3: The results of Bromhead ring shear tests on samples from the exposure of slip surface in the F horizon of Barton Bed in the frontage of cliff at Naish Farm in Barton on Sea in July 2010.

Appendix C

Table C1: List of published results of laboratory testing on the samples from landslides in London Clay using Bromhead Ring Shear device. The results used for preparing the graphs in Chapter 8.

	London Clay (Bromhead Ring Shear Apparatus)										
No.	Paper	Location	σ' _n (kPa)	τ (kPa)	φ'r (Degrees)						
Lo-1	61	Warden Point, Isle of Sheppey (Grey London Clay)	40.4	11.7	16.2						
Lo-2	61	Warden Point, Isle of Sheppey (Grey London Clay)	68.2	15.0	12.4						
Lo-3	61	Warden Point, Isle of Sheppey (Grey London Clay)	92.7	21.9	13.3						
Lo-4	61	Warden Point, Isle of Sheppey (Grey London Clay)	113.9	25.3	12.5						
Lo-5	61	Warden Point, Isle of Sheppey (Grey London Clay)	149.7	33.0	12.4						
Lo-6	61	Warden Point, Isle of Sheppey (Grey London Clay)	204.6	38.6	10.7						
Lo-7	61	Warden Point, Isle of Sheppey (Grey London Clay)	260.9	47.8	10.4						
Lo-8	61	Warden Point, Isle of Sheppey (Grey London Clay)	314.6	52.4	9.5						
Lo-9	61	Warden Point, Isle of Sheppey (Grey London Clay)	372.2	59.5	9.1						
Lo-10	169	Herne Bay	170.0	28.1	9.4						
Lo-11	188	Unknown	252.1	42.6	9.6						
Lo-12	188	Unknown	249.1	49.3	11.2						
Lo-13	188	Unknown	151.0	36.3	13.6						
Lo-14	188	Unknown	50.0	16.7	17.4						

Table C2: List of published results of laboratory testing on the samples from landslides in London Clay using Bishop Ring Shear device. The results used for preparing the graphs in Chapter 8.

	Londo	on Clay (Bishops Brom	head Ring Shear A	Apparatus)	
No.	Paper	Location	σ' _n (kPa)	τ (kPa)	φ'r (Degrees)
Lo-1	See 59	Happisburgh Clay	185	30.5	9.4
Lo-2	See 59	Happisburgh Clay	200	33.0	9.4
Lo-6	See 59	Happisburgh Clay	221.6	27.0	7.0
Lo-7	See 59	Happisburgh Clay	400	51.2	7.3
Lo-8	See 59	Happisburgh Clay	400.0	64.0	9.1
Lo-9	See 59	Happisburgh Clay	401	71.0	10.0
Lo-10	See 59	Happisburgh Clay	401.0	75.4	10.6
Lo-11	See 59	Happisburgh Clay	402	78.4	11.0
Lo-12	See 59	Happisburgh Clay	250	48.3	10.9
Lo-13	See 59	Happisburgh Clay	250	49.3	11.1
Lo-14	See 59	Happisburgh Clay	250	50.0	11.3
Lo-15	See 59	Happisburgh Clay	250	49.3	11.1
Lo-16	See 59	Happisburgh Clay	250	48.8	11.0
Lo-17	See 59	Happisburgh Clay	220.0	46.2	11.9
Lo-18	See 7	Happisburgh Clay			
Lo-19	See 7	Happisburgh Clay	176.0	20.4	11.9
Lo-20	See 7	Happisburgh Clay	352.0	37.7	12.4
Lo-21	See 7	Happisburgh Clay	878.0	154.8	10.0
Lo-24	See 7	Happisburgh Clay	352.0	63.4	10.2
Lo-25	See 7	Happisburgh Clay	198.6	33.2	9.5

Table C2: List of published results of laboratory testing on the samples from landslides in London Clay using Bishop Ring Shear device. The results used for preparing the graphs in Chapter 8.

	Londo	n Clay (Bishops Brom	head Ring Shear A	Apparatus)	
No.	Paper	Location	σ' _n (kPa)	τ (kPa)	φ'r (Degrees)
Lo-26	See 2	Happisburgh Clay	199.9	33.1	9.4
Lo-27	See 2	Happisburgh Clay	82.0	13.6	9.4
Lo-28	See 2	Happisburgh Clay	40.7	6.6	9.2
Lo-29	See 2	Happisburgh Clay	279.9	45.8	9.3
Lo-30	See 2	Happisburgh Clay	213.7	32.7	8.7
Lo-31	See 2	Happisburgh Clay	110.3	17.3	8.9
Lo-32	See 2	Happisburgh Clay	59.3	10.0	9.6
Lo-33	See 2	Happisburgh Clay	34.5	6.3	10.4
Lo-34	See 2	Happisburgh Clay	163.4	24.1	8.4
Lo-35	See 2	Happisburgh Clay	250.3	35.2	8.0
Lo-36	See 2	Happisburgh Clay	31.0	5.8	10.6
Lo-37	See 2	Happisburgh Clay	15.9	3.4	12.2
Lo-38	See 2	Happisburgh Clay	6.9	1.7	13.8
Lo-39	See 2	Happisburgh Clay	90.3	15.4	9.7
Lo-40	See 2	Happisburgh Clay	65.5	11.5	10.0
Lo-41	See 2	Happisburgh Clay	93.1	15.7	9.6
Lo-43	See 94	Hadleigh, Essex	174.8	26.9	8.8
Lo-44	See 94	Hadleigh, Essex	63.8	11.2	10.0
Lo-45	See 94	Hadleigh, Essex	27.5	5.8	11.9
Lo-46	See 94	Hadleigh, Essex	9.8	2.6	14.8
Lo-48	See 142	Various sits	430.0	66.6	8.8
Lo-49	See 142	Various sits	150.0	28.3	10.7
Lo-50	See 169	Hadlieigh	180.0	26.6	8.4

Cross section of railway cutting by James (1970)



Figure C1: Cross section landslide at Dedham by James (1970)



Figure C2: Cross section landslide at Northolt by James (1970)



Figure C3: Cross section landslide at Upper Holloway by James (1970)



Figure C4: Cross section landslide at Wembley Hill by James (1970)



Figure C5: Cross section landslide at Fareham (in Hampshire Basin) by James (1970)

Appendix D

Residual strength of clay in landslides in the Gault Clay



Figure D1: The map shows the solid geology of south east England and northern France. The coastal sites exhibiting the descending sequence of Chalk, upper Greensand, Gault and Lower Greensand are indicated.



Figure D2: The simplified stratigraphical of Cretaceous



Figure D3: The location of sampling from the exposure of slip surface in the F and D horizons in 2010 and 2011 are shown in the picture. Two samples from the slip surface in the weak bed in the D horizon in 2010 and 2011 are made, and one samples from the F horizon.

Sampling from slip surface exposure in Gault at Folkestone, Kent:

Date: 11th June 2010

Location: South east of Folkestone Warren; down of Foreland Avenue (down of the Martello Tower, see Figure 3)

National Grid Reference: Eastings (X): 624293

Northings (Y): 136895

Latitude: 51.087492

Longitude: 1.2013426

NGR: TR 24293, 36895

Sampling from slip surface exposure in Gault at Blackgang, Isle of Wight

Date: 5th August 2011

Location: South east of Balckgang, Isle of Wight (see Figure)

National Grid Reference: Eastings (X): 448919

Northings (Y): 076316

Latitude: 50.584534

Longitude: -1.3103744

NGR: SZ 448919, 76316

Residual shear strength measurements in laboratory

Table D1: Results of laboratory testing on the samples from exposure of landslides in Gault atFolkestone Warren and Isle of Wight

		Lat	poratory Results	of samples from	Folkesto	ne & Isle	of Wight	in 2010) and 2011				
Year	sample	No of Ring shear test	Location of sampling	Date of sampling	WL%	PL%	P1%	CF%	w % (Natural)	w % (Before R. S. test)	w % (After R. S. test)		
		1	South east of Folkestone	11th June 2010	66.8	25.53	41.27	63.5	38.77				
2010	1	1	Warren; Down of Foreland Avenue	11th June 2010	67.3	26.5	40.8	64	37.8	•	•		
		1	(NGR: TR 24293, 36895)	11th June 2010	67.1	25.4	41.7	63	38.46				
		1		5th August 2011	56.8	21.4	35.4	54	40.8	25.9	29.7		
2011	1	1	South east of Balckgang, Isle of Wight (NGR: SZ	South east of Balckgang, Isle of Wight (NGR: SZ	South east of Balckgang, Isle of Wight (NGR: SZ	5th August 2011	56.9	21.5	35.4	55	41	26	27.2
		(NGR: SZ 448919, 76316) 1		5th August 2011	57.2	22	35.2	54.5	40.6	26.5	29.4		

Table D2: The results of Bromhead ring shear test on the samples from the exposure of slip surface at Folkestone Warren in 2010

1 6 5 1	Test	1	Test	2	Test	3	
σ' (kN/m²)	τ (kN/m²)	фr(*)	τ (kN/m²)	φr(*)	τ (kN/m²)	φr(*)	Average фr(*)
15.24	3.27	12.11	3.10	11.50	3.45	12.76	12.1
27.48	4.67	9.64	4.33	8.95	4.84	9.99	9.5
51.96	8.11	8.87	7.96	8.71	8.27	9.04	8.9
76.44	11.12	8.28	10.92	8.13	11.35	8.45	8.3
100.92	14.04	7.92	13.85	7.81	14.19	8.00	7.9
15.24	12 19 1	0.00	2.54	9.46	2.71	10.08	9.8

Table D3: The results of Bromhead ring shear test on the samples from the exposure of slip surface at Blackgang, IOW, August 2011

Welling and	Tes	t 1	Tes	t 2	Tes	t 3	
σ' (kN/m2)	τ (kN/m2)	φr(*)	τ (kN/m2)	φr(*)	τ (kN/m2)	φr(*)	Average ¢r(*)
15.24	2.9	10.7	2.83	10.54	3.23	12.0	11.1
27.48	5.2	10.8	4.97	10.25	5.28	10.9	10.6
51.96	9.5	10.3	9.55	10.41	10.20	11.1	10.6
76.44	12.3	9.2	12.21	9.07	12.60	9.4	9.2
100.92	15.8	8.9	15.31	8.62	16.09	9.1	8.9
125.4	18.9	8.6	18.2	8.3	19.2	8.7	8.5



Figure D4: This graph shows σ'_n - τ of Gault resulted from Bromhead ring shear tests on the samples from the exposure of basal shear surface at Folkestone Warren and Isle of Weight



Previous research: (Ring shear results)

Figure D5: Published results of Bishop and Bromhead ring shear apparatus on the sample from landslides in Gault



Previous research: (Back analysis results)

Figure D6: Results of published back analyses of the cross section of landslides in Gault

Appendix E

The residual strength of clay in landslides in Lias

Back analyses

Landslide at Lyme Regis 1962:



Figure E1: Cross section of the landslide in Lias Clay (Lower) in 1962 at Lyme Regis. Slip plane estimated from failure observation and boring results (After James, 1970).

Table E1: Results of back analyses of cross section of landslide in Lias at Lyme Regis in 1962 by James (1970) , the above cross section.

		L	yme Re	gis by P	James						
Selected Slip Surface	Condition of GWL	Lower Lir at -1	ne .0 m	Lowe Li at -0	r piez. ne).5 m	Estin piez. by P	hated Line James	Lowe Lii at -+(r piez. ne 0.5 m	Lowe Lin at +1	r piez. ne 1.0 m
	φ', (°)	φ'r1 (°)	φ'r2 (°)	φ'n (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (°)	φ'rı (°)	φ'r2 (°)
Main Slip by P. James	φ' _{rav}	9	9	9.3	9.3	9.7	9.7	10	10	10.4	10.4
Warr Sup by P James	First Ratio of φ'_r	10	8.5	10	9	11	9	11	9.4	12	9.5

Landslide at Lyme Regis 1978:



Figure E2: Cross section of the landslide in Lias Clay in 1978 at Lyme Regis. (After Section 12 by High-Point Rendel).

Table E2: Results of back analyses of cross section of landslide in Lias at Lyme Regis in 1978 by High-

Point Rendel, the above cross section.

		Lyme	Regis:	Section	12, Gr	ound P	rofile 1	978 by	High Po	int Rne	del				
Selected Slip	Condition of GWL	Lov piez.	ver Line	Lov	wer Line	Lov piez.	wer Line	Estin piez by l	nated . Line High	Lov piez.	wer Line	Lov piez.	wer Line	Lov piez.	Line
back analysis	φ', (°)	at -1	.5 m φ'r2	at -1	φ ¹ _{r2}	at -0	φ'12 (°)	Po Rei ¢'ri (°)	oint ndel Φ'r2	φ'r1	φ'r2	at +1 φ'r1 (°)	φ'r2	at +1	φ'r2
Main Slip Surface in Fish Bed (The Lower Slip by High Point)	φ' _{rav}	7.2	7.2	7.5	7.5	7.8	7.8	8.1	8.1	8.5	8.5	9	9	9.6	9.6

Landslide at Lyme Regis 1995:



FigureE3: Cross section of the landslide in Lias Clay in 1995 at Lyme Regis. (After Section 10 by High-Point Rendel).

Table E3: Results of back analyses of cross section of landslide in Lias at Lyme Regis in 1995 by High-Point Rendel , the above cross section.

		Lym	e Regis	: Sectio	on 10, 0	Ground	Profile	1995 by	High Po	int Rne	del				
Selected Slip Surface for	Condition of GWL	Lov piez.	wer . Line	Lov piez.	wer Line	Lov piez.	wer Line	Estin piez by Hig	hated Line h Point	Lov piez.	wer Line	Lov piez.	wer Line	Lov piez.	wer Line
back		at -1	1.5 m	at -1	.0 m	at -C).5 m	Ren	ndel	at -+	0.5 m	at +1	1.0 m	at +1	1.5 m
analysis	φ', (°)	φ'r1 (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (°)	¢'n (°)	φ'12 (°)	φ'11 (*)	¢'12 (°)	φ'r1 (°)	¢'12 (°)	Φ'r1 (*)	φ'r2 (°)
Slip Surface in Table Ledge (The Upper Slip by High Point)	φ' _{rav}	15	15	15	15	15	15	16	16	16	16	18	18	19	19
Slip Surface in Fish Bed (The Lower Slip by High Point)	φ' _{rav}	12	12	12	12	12	12	12	12	12	12	13	13	14	14

Landslide at Lyme Regis 1995:



FigureE4: Cross section of the landslide in Lias Clay in 1995 at Lyme Regis. (After Section 10 by High-Point Rendel).

Table E4: Results of back analyses of cross section of landslide in Lias at Lyme Regis in 1995 by High-

	Condition	Lov	ver	Lov	wer	Lo	wer	Estin	nated	Lov	wer	Lowe	r piez.	Lowe	r piez
Selected Slip Surface for	of GWL	piez.	Line	piez.	Line	piez	Line	piez. by I Po	Line ligh int	piez.	Line	Li	ne	Li	ne
back analysis		at -1	.5 m	at -1	.0 m	at -0).5 m	Rer	del	at -+(0.5 m	at +1	.0 m	at +1	1.5 m
	φ', (°)	φ' _{r1} (")	φ'12 (°)	φ'11 (°)	φ'r2 (°)	φ'r1 (*)	φ',2 (°)	¢'a (°)	¢'n2 (°)	φ'r1 (°)	φ' ₁₂ (*)	φ'r1 (°)	¢'12 (*)	Φ'r1 (°)	φ',2 (°)
Main Slip Surface in Fish Bed (The Lower Slip by High Point)	φ' _{rav}	8.3	8.3	8.6	8.6	8.9	8.9	9.3	9.3	9.7	9.7	10.3	10.3	10.9	10.9

Point Rendel , the above cross section.

Landslide at Lyme Regis 2006:



Figure E5: Cross section of the landslide in Lias Clay in 2006 at Lyme Regis. (After Section 12 by High-Point Rendel).

Table E5: Results of back analyses of cross section of landslide in Lias at Lyme Regis in 2006 by High-Point Rendel , the above cross section.

Selected Slip Surface	Condition of GWL	Lov piez.	ver Line	Lov piez.	wer Line	Lov piez.	wer Line	Estin piez.	Line	Lov piez.	ver Line	Lov piez.	wer Line	Lov piez.	ver Line
for back analysis	φ', (°)	at -1 \$\phi'r1 (°)	.5 m φ'r2 (°)	at -1 \$\phi'_{r1} (°)	.0 m φ'r2 (°)	at -0 \$\phi'_{r1} (°)	φ'r2 (°)	Point I ϕ'_{r1} (°)	Rendel ϕ'_{r2} (°)	at -+(\$\phi'_{r1} (°)	0.5 m φ'r2 (°)	at +1 \$\$'r1 (*)	4'r2 (°)	at +1 \$\phi'_{r1} (°)	φ'r2 (°)
Main Slip Surface in Fish Bed (The Lower Slip by High Point)	φ' _{rav}	7.3	7.3	7.6	7.6	7.9	7.9	8.2	8.2	8.7	8.7	9.2	9.2	9.8	9.8

Landslide at Lyme Regis 2007:



Figure E6: Cross section of the landslide in Lias Clay in 2007 at Lyme Regis. (After Section 10 by High-Point Rendel)

Table E6: Results of back analyses of cross section of landslide in Lias at Lyme Regis in 2007 by High-Point Rendel , the above cross section.

		Lym	e Regis	: Sectio	on 10, G	Fround	Profile	2007 by	High Poi	int Rnee	del				
Selected Slip	Condition of GWL	Lov piez.	wer Line	Lor piez	wer . Line	Lov piez.	wer Line	Estin piez. by I	Line High	Lon piez	wer Line	Lov piez	wer Line	Lov piez.	ver Line
back analysis		at -1	.5 m	at -1	.0 m	at -C).5 m	Point	Rendel	at -+	0.5 m	at +1	1.0 m	at +1	.5 m
Dack analysis	φ', (°)	φ'r1 (°)	φ'12 (°)	φ'r1 (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (°)	φ'n (°)	φ'rz (°)	φ' ₁ (°)	¢'r2 (°)	φ'r1 (°)	φ'12 (°)	φ'11 (°)	φ'12 (°)
Slip Surface in Table Ledge (The Upper Slip by High Point)	φ' _{rav}	14	14	14	14	14	14	14	14	14	14	14	14	15	15
Slip Surface in Fish Bed (The Lower Slip by High Point)	φ' _{rav}	13	13	13	13	13	13	13	13	13	13	14	14	15	15

Landslide at Toddington:



Figure E7: Cross section of the landslide in Lias Clay in 1960 at Toddington (After James, 1970).

Table E7: Results of back analyses of cross section of landslide in Lias at Toddington in 1960 by James (1970) , the above cross section.

			Т	oddingto	on						
Selected Slin Surface for	Condition of GWL	Lowe Li	r piez. ne	Lowe Li	r piez. ne	Estimat Li	ed piez. ne	Lowe Li	r piez. ne	Lowe Li	r piez. ne
back analysis		at -1	.0 m	at -C).5 m	by P.	lames	at -+	0.5 m	at +1	1.0 m
	φ', (°)	φ'r1 (°)	φ'r2 (°)	φ'r1 (°)	φ'r2 (*)	¢'n (*)	φ'a (°)	φ'r1 (°)	Φ'r2 (°)	Φ'r1 (°)	Φ'12 (°)
Slip A estimated from slip indicators and failure profile by P James	φ' _{rav}	15.2	15.2	15.2	15.2	15.2	15.2	16	16	19.4	19.4
Slip B estimated from slip indicators and failure profile by P James	φ' _{rav}	17.3	17.3	18.9	18.9	21.3	21.3	23.6	23.6	27.7	27.7
Slip B Circle estimated from slip indicators and failure profile by P James	φ' _{rav}	16.5	16.5	17.7	17.7	19.7	19.7	21.9	21.9	26.2	26.2
Modified Slip surface by HOSSEYNI estimated from slip indicators and failure profile	φ' _{rav}	•	•	•	•	18.5	18.5	•		•	

Results of back analyses:



Figure E8: The graph shows the results of back analyses of cross sections of the landslides in Lias Clay (the above cross sections).

Experimental programme results

Table E8: Results of laboratory testing on the samples from landsliding in Lias at Lyme Regis and Charmouth

		1	aboratory Results of s	amples from Charn	nouth &	Lyme R	egis in 20	010 and 3	2011		
Year	sample	No of Ring shear test	Location of sampling	Date of sampling	WL%	PL%	P1%	CF%	w % (Natural)	w % (Before R. S. test)	w % (After R. S. test)
		1		29th April 2011	61.3	34.1	27.2	52.5	34.1	33.3	38.4
	1	1	East Cliff Slip at Lyme Regis; Down of Car Park,	29th April 2011	61.8	34.5	27.3	63	34.5	36.3	36
	1	1	(NGR: SY	29th April 2011	62.1	35.1	27	63.5	35.1	32.7	31.5
2014		1	34552, 92483)	29th April 2011	60.8	34	26.8	54	34.6	33.8	32.3
2011		1		9th June 2011	60.4	27	33.4	47	33.7	33.3	38.4
	2	1	Lyme Regis; Down of Car Park,	9th June 2011	61.2	27.3	33.9	47.5	33.6	36.3	36
	2	1	(NGR: SY	9th June 2011	58.7	26.8	31.9	43.5	33.7	32.7	31.5
		1	34552, 92483)	9th June 2011	59.2	26.9	32.3	50	33.8	33.8	32.3
		1	Rock fall at South	1st August 2010	61.1	28.2	32.9	53	24.7	29.6	34.4
2010	1	1	east of Charmouth, Dorset	1st August 2010	60.5	27.9	32.6	55	24.9	31.3	33.1
		1	(NGR: SY 36875, 93001)	1st August 2010	61.9	28.3	33.6	58	24.8	29	32.7

Table E9: The results of Bromhead ring shear test on the samples from the exposure of slip surface inLyme Regis, Dorset, April 2011

	Test	1	Test	12	Test	3	Test	4	
σ' (kN/m²)	τ (kN/m²)	φr(°)	Average						
15.24	3.10	11.50	•	•			3.23	11.97	11.7
27.48	4.88	10.07	•	•	•	•	5.06	10.43	10.3
51.96	9.16	10.00	8.85	9.67	9.37	10.22	9.51	10.37	10.1
76.44	12.47	9.27	11.56	8.60	11.73	8.72	12.95	9.62	9.1
100.92	15.48	8.72	15.04	8.48	14.56	8.21	16.70	9.40	8.7
125.4	18.88	8.56	18.44	8.37	17.66	8.02	20.10	9.11	8.5
15.24	3.01	11.17	2.70	10.05	3.36	12.43	3.62	13.36	11.8
27.48	5.36	11.04	4.71	9.73	5.15	10.61	5.01	10.33	10.4
51.96	•		8.42	9.20	8.81	9.62	8.76	9.57	9.5
76.44	•			•			12.47	9.27	12.47

Table E10: The results of Bromhead ring shear test on the samples from the exposure of slip surface in Charmouth, Dorset, August 2010

	Test	1	Test	2	Test	3	
σ' (kN/m²)	τ (kN/m²)	φr(°)	τ (kN/m ²)	φr(°)	τ (kN/m²)	фr(°)	Average ¢r(°)
15.24	3.7	13.8	3.43	12.68	3.39	12.5	13.0
27.48	5.7	11.7	5.35	11.02	5.09	10.5	11.1
51.96	9.1	9.9	9.16	10.00	8.98	9.8	9.9
76.44	12.1	9.0	12.31	9.15	12.19	9.1	9.1
100.92	14.8	8.4	14.17	7.99	15.44	8.7	8.3
15.24	2.5	9.4	2.8	10.3	2.6	9.7	9.8



Results of Bromhead Ring Shear Test on samples from Gault

Figure E9: This graph shows σ'_{n} - τ for results of Bromhead ring shear apparatus on the samples from the exposure of basal shear surface Lyme Regis and Charmouth in June 2011 and August 2011.



Previous research: (Ring shear results)

Figure E10: This graph shows collaboration of the published results of ring shear test using Bishop's apparatus which are collected from the literature. Literature survey indicates that there is a lack of published results using Bromhead ring apparatus.


Previous research: (Back analysis results)

Figure E11: This graph shows results of published back analyses of the cross sections of landslides in Lias. The results are mainly illustrate back analyses results from the literature.

Appendix F



Figure F1: The graph compares the results of back analyses, Bromhead ring shear and Bishop ring shear apparatus of four British Clays (Barton Clay, London Clay, Gault and Lias). Value of residual friction angle of each clay is shown. The main outlier data points are those from the Folkestone Warren sections both (a) the minimum piezometric levels and (b) the after (or post) sliding profile (after the major displacement of 1915). Reflection on the groundwater conditions renders it highly improbable that this is a realistic interpretation of the piezometric levels operating. without those outliers, the four datasets are indistinguishable to the unaided eye on the graph.

Appendix G



Example of Ring Shear Test Results

Figure G1: The graph shows results of ring shear test on the sample from the exposure of landslide in Zone F at Barton on Sea. Sample collected in 2010 as mentioned in the text. The Graph illustrates different stages of ring shear test using Bromhead ring shear apparatus including: consolidation and multi stage shearing under different normal stresses. The graph also shows displacement of the sample under different normal stresses (Blue coloured graph).



Figure G2: The graph shows results of ring shear test on the sample from the exposure of landslide in Zone F at Barton on Sea. Sample collected in 2010 as mentioned in the text. The Graph illustrates different stages of ring shear test using Bromhead ring shear apparatus including: consolidation and multi stage shearing under different normal stresses. The graph also shows displacement of the sample under different normal stresses (Blue coloured graph).



Figure G3: The graph shows results of ring shear test on the sample from the exposure of landslide in Zone F at Barton on Sea. Sample collected in 2010 as mentioned in the text. The Graph illustrates different stages of ring shear test using Bromhead ring shear apparatus including: consolidation and multi stage shearing under different normal stresses.



Figure G4: The graph shows results of ring shear test on the sample from the exposure of landslide in Gault Clay on the Isle of Wight. The Graph illustrates different stages of ring shear test using Bromhead ring shear apparatus including: consolidation and multi stage shearing under different normal stresses.

Appendix H

List of relevant publications

Journals:

- Majrouhi Sardroud, J., Hosseyni, S., Bromhead, E.N. (2012) 'Automated Landslide Monitoring and Warning using RFID Technology', *International Journal of Safety and Security Engineering*, (Accepted).
- Hosseyni, S., Torii, N, Bromhead, E.N. (2012) Discussion of 'Reactivation of landsliding following partial cliff stabilisation at Barton-on-Sea, Hampshire by Barton and Garvey', *Quarterly Journal of Engineering Geology & Hydrogeology*, 45 (1).
- Hosseyni, S., Arabameri, M. (2011) 'Landslide; mechanics, causal factors and its components in North of Iran', *Journal of Namad Golestan* (in Persian), Iran.

Book chapters:

- Bromhead, E.N., Hosseyni, S. and Torii, N. (2012) 'Soil slope stabilization', in Clague, J.J and Stead, D. (eds.) *Landslides Types*. Mechanisms and Modeling . Cambridge University Press, pp. 252-266
- Hosseyni, S., Bromhead, E.N., Majrouhi Sardroud, J. and Limbachiya, M. C. (2011) 'Real-time landslides monitoring and warning using RFID technology for measuring ground water level', *Disaster Management and Human Health Risk*, Edited by Brebbia & Kassab and Divo, WIT Press.

Conference papers:

- Hosseyni, S., Torii, N., Bromhead, E.N. (2012) 'Limitations and sources of inaccuracy in back analysis', 11th International & 2nd N American Symposium on Landslides; ISL & NASL2012, Banff, Canada.
- Bromhead, E.N. and Hosseyni, S. (2012) 'When the analysis won't converge why, and what to do about it', *Proceedings of 11th (ISL and the 2nd North American Symposium on Landslides, Landslides and Engineering Slopes: Protecting Society through Improved Understanding (ed. E. Eberhardt & C. Froese & K. Turner and S. Leroueil,*, June 2-8, 2012, Banff, Alberta, Canada.
- Bromhead, E.N., *Winter, M.G., Floyd-Walker, H.,* Hosseyni, S. (2012) 'The Contribution of QJEGH to Landslide Knowledge: There and Back in 45 Volumes ', 11th International & 2nd North American Symposium on Landslides; ISL & NASL 2012, Banff, Canada.

Hosseyni, S., Torii, N., Bromhead, E.N. (2011)'Residual strength measurements for some British Clays', *The Second World Landslide Forum Conference*, Rome, Italy.

Hosseyni, S., Bromhead, E.N., Majrouhi Sardroud, J. (2011) 'Integrated RFID and sensor technologies for effective landslides monitoring and early warning', *ISEC-6, Zurich,* Switzerland.

Presentation:

Hosseyni, S. (2011) Residual strength measurements for three British Clays, Room 2003 John Galsworthy Building, Penrhyn Road, Kingston University, March.

Poster Presentation:

Hosseyni, S. (2009) *Research Students Competition* [Poster]. Exhibited at Kingston University London. Runner-up of the Research Poster Competition.

Hosseyni, S. (2010) *Research Students Competition* [Poster]. Exhibited at Kingston University London. Runner-up of the Research Poster Competition.