Geotechnical Properties of Chalk Putties

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Abstract

Chalk putties are common in Southern England, occurring naturally and as a result of earthworks in intact Chalk. The ease and readiness with which intact Chalk breaks down into putty makes establishing a good geotechnical profile of the material necessary.

A review of literature indicates that previous laboratory studies on chalk putties are limited, and that testing of the material has proven difficult using standard procedures. This study initially quantifies index parameters such as Atterberg limits, thixotropy and particle size distribution before considering susceptibility to age-related strength gains and shear strength-strain dependency so that subsequent shear test data can be normalised.

Contrary to literature, age-related strength gains were found to be minor, whilst shear strength-strain dependencies were found to be significant. Large strain tests in ring shear apparatus (following recommended test procedures) found non linearity in the drained shear failure envelopes, with effective friction angles (Ø') *increasing* with strain. This non linearity is explained by sample grading evolution.

Using these findings the study develops new preparation and testing methodologies to create 'identical soils' of known stress history. Testing in a computer governed stress path cell (using 'Triax' software) found that chalk putty behaves as a contractive material, 'wet' of its critical state, exhibiting failure by liquefaction for mean effective stresses (p') lower than 200kPa. Pre and post yield permeability values in the range 2.5-13 x 10^{-9}m/s were recorded with state parameters indicating a constant reduction in sample void ratio during shear tests conducted at a pre shear p' of between 0 and 400kPa.

Key words: chalk putty; critical state; identical soils; geotechnical index parameters; liquefaction; non linear drained shear failure envelope; permeability; ring shear apparatus; stress path cell; stress paths; 'Triax'; state parameters; void ratio.

Content

Abstract	ii
Contents	iii-ix
List of Figures	x-xiii
List of Tables	xiv
Acknowledgements	xv
Personal Acknowledgement and Reflection	xvi
Chapter One - Introduction	1
1.1 Introduction and Aims	1
1.2 Layout of thesis	2
Chapter Two - Chalk Putty; the Material	4
2.0 Overview	4
2.1 Definition of chalk putty	4
2.2 Sedimentology and lithology of Southern England Chalks	5
2.2.1 Composition	5
2.2.2 Depositional environment	6
2.2.3 Diagenesis and the formation of Chalk	7
2.3 Stratigraphic and regional trends in intact Chalk strength	10
2.4 Engineering classification of intact Chalk	13
2.5 Occurrence of chalk putty	14
2.5.1 Putties that are found in areas which have under gone shear stress	14
2.5.2 Putties formed from weathering	14
2.5.3 Putties formed during civil engineering projects	14
2.6 Geotechnical index properties of chalk putty	18
2.6.1 Particle size distribution	18
2.6.2 Atterberg Limits	18
2.6.3 Thixotropic properties	19

2.7 Shear strengths in chalk putties 19
2.7.1 Published remoulded samples 19
2.7.1.1 Undrained condition 19
2.7.1.2 Drained condition 20
2.7.2 Illustrating shear strength in chalk putties 21
2.7.2.1 Coulomb failure envelope 22
2.7.2.2 Mohr – Coulomb failure envelope 22
2.7.2.3 Failure defined in p' and q space 23
2.7.2.4 The use of Critical State Soil Mechanics in defining chalk putty behaviour 23
2.7.2.5 Non-linearity of failure envelopes 26
2.8 Effects of ageing on chalk putties 28
2.8.1 Compaction and creep 28
2.8.2 Cementation 28
2.8.3 Weathering 29
2.9 Chalk putties and engineering classification system 30
2.10 Objectives 31

Chapter Three - Experimental Procedure and Methodology 33
3.0 Overview 33
3.1 Sampling procedure 33
3.2 Shear failure envelopes of parent intact Chalk material 34
3.3 Density and porosity calculations of parent intact Chalk material 34
3.4 Determination of non-calcium carbonate material in the chalk putties 35
3.5 Particle size distribution (PSD) 36
3.6 Atterberg limits and Linear shrinkage tests 36
3.7 Shear strength tests 37
3.7.1 Shear strength analysis using the conventional shear box 37

3.7.2 Shear strength analysis using the ring shear apparatus 37

3.8 Triaxial shear strength testing 40

3.8.1 'Quick', unconsolidated, undrained, partially, saturated triaxial tests 40

3.8.2 Advanced triaxial testing 40

3.8.2.1 Equipment overview 40

3.8.2.2 System hardware 41

3.8.2.3 System software 44

3.8.2.4 Calibration of Imperial College stress path system 46

3.8.2.5 Test procedure 46

3.8.2.5.1 Sample Preparation 47

3.8.2.5.2 Flushing reformed chalk putty 51

3.8.2.5.3 Saturation 52

3.8.2.5.4 Verification of saturation using Skempton's 'B' test 52

3.8.2.5.5 Required effective stress by consolidation 52

3.8.2.5.6 Drained shear 53

3.8.2.5.7 Pre and post shear permeability stage 54

3.8.2.5.8 Treatment and corrections to triaxial data 55

Chapter Four - Presentation of Data 57

4.0 Overview 57

4.1 Parent material of the studies chalk putties 57

4.1.1 Porosity and dry density 57

4.1.2 Failure envelopes and crushing of intact Chalk used to form test putties 59

4.2 Composition of the chalk putties reviewed 59

4.3 Particle size distribution of chalk putty 62

4.4 Geotechnical index properties of chalk putty 62

4.4.1 Atterberg values, linear shrinkage and thixotropy 62

4.4.2 Shear strength test using the cone penetrometer 68

4.5 Conventional direct shear box test 69

4.5.1 Shear strength tests with the conventional direct shear box 69

4.5.2 The effect of time dependent volume changes on test samples 70

4.6 Results of residual shear tests on chalk putty using the ring shear apparatus 71

4.6.1 Standard shear strength tests 71

4.6.2 Further investigations by the ring shear apparatus 76

4.7 Results of the undrained triaxial tests 77

4.8 Results of the advanced triaxial tests 80

4.8.1 Introduction 80

4.8.2 Back pressure 'B' value tests 80

4.8.3 Results of isotropic, consolidated drained stress path tests 82

4.8.3.1 The Mohr-Coulomb failure envelope 82

4.8.3.2 Vertical permeability 82

4.8.3.3 Sample volume change 82

4.9 Summary 85

Chapter Five - Discussion of Results 86

5.0 Overview 86

5.1 Analysis and characterisation of the chalk putties studied 86

5.1.1 Porosity and dry density of Chalk used to form putties 86

5.1.2 Comparison and grading 86

5.1.3 Atterberg values 87

5.1.4 Linear shrinkage 88

5.1.5 Thixotropy 88

5.2 Isolating variables which could affect the results of shear strength tests 88
5.2.1 Assessing the fabric and structure of chalk putty 88
5.2.2 Assessment of time-dependent volume change on chalk putty test samples 90
5.2.3 Assessment of chalk putty ageing using the cone penetrometer 90
5.2.4 Assessment of chalk putty ageing using the direct shear test 91
5.3 Discussion of the ring shear simple shear results 93
5.3.1 Ø' angles and comparison to literature 93
5.3.2 Non linear failure envelopes in chalk putty 94
5.3.2.1 Influence of test machine 94
5.3.2.2 Influence of sample material 96
5.3.2.3 Influence of particle grading on Ø′ angle 99
5.3.2.4 Direct sampling of sample to determine whether uniformity or grading changes during ring shear testing. 99
5.3.2.5 Discussion of RSPG 1-2 100
5.3.2.6 Influence of particle angularity on Ø' angle 101
5.4 Discussion of the advanced triaxial results 101
5.4.1 Discussions on the development of the 'dry press' technique 101
5.4.2 Discussion of the back pressure 'B' value tests 103
5.4.3 General permeability values 104
5.4.4 Permeability reduction with increased confining effective stress prior to shear 105
5.4.5 Permeability reduction on yield 108
5.4.6 Susceptibility to liquefaction 108
5.5 Summary 109

Chapter Six - Conclusions and Recommendations for Further Work 111

6.0 Overview 111

6.1 Geotechnical properties of chalk putty 111

6.2 Permeability 112

6.3 Liquefaction 112

6.4 Shear strength-strain dependency 113

6.5 Grading and fabric evolution 113

6.6 Developing testing procedure 114

6.7 Limitation of this study 115

6.7.1 The effect of back pressures on liquefaction susceptibility 115

6.7.2 The effect of back pressures in altering sample fabric 115

6.8 Future research 116

6.8.1 Investigation of the effects of back pressure 116

6.8.2 Re-inflation tests 116

List of References 118-135

Glossary of Terms 136-139

Principal symbols 140

Appendix One - Triaxial Sample Preparation Techniques 141

A1.1 Triaxial sample preparation techniques for granular materials 141

A1.2 Lateral shaking during sedimentation 141

A1.3 Consolidation Method 143

Appendix Two - Calibration of Porous Plates for their use in Permeability Tests 145

Appendix Three - Laser Particle Size Analysis of Chalk Soils 146

A3.1 Use of dispersants 146

A3.2 Analysis of the data from the Malvern Mastersizer 2000 laser granulometer 146

Appendix Four - Observations using an Electron Microscope 148

Appendix Five - The 'Triax ini.' File 151

List of Figures

Figure 2.1	Structure of coccolithophores. (After Brasier 1992)	6
Figure 2.2	Channel outcrops of pre-Upper Cretaceous rocks either on sea – floor or inferred beneath the sub – Upper Cretaceous unconformity (Owen, 1976 based on Dingwall 1971)	9
Figure 2.3	Occurrence of chalk putties	15-17
Figure 2.3a	i) Culver Cliff (Isle of Wight) fall of 19th May 2007,	15
	ii) Structured Chalk, Mundford grade III/IV, has chalk putty infilling joints and fractures	15
	iii) Chalk putty formed along the shear surface which ultimately caused the cliff to collapse	15
Figure 2.3b	i) Cliffs at Wheelers Bay (Isle of Wight),	16
	ii) and iii) Repeated landslips have resulted in structureless Chalk (Mundford V and VI), where the matrix is primarily chalk putty	16
Figure 2.3c	i) Cliffs at Freshwater (Isle of Wight) ii) Upper Chalk has undergone freeze-thaw action to form soliflucted Coomb deposits, where a melange of cobbles and boulders are suspended in chalk putty	17
Figure 2.4	Typical unconsolidated undrained test data for partially saturated soil (after Lambe and Whitman 1969)	20
Figure 2.5	The Mohr-Coulomb failure envelope	23
Figure 2.6	Cam Clay Model Yield Surface. (Schofield and Wroth, 1968)	25
Figure 2.7	Particle locking in sands	27
Figure 2.7a	Barton Sand, shows tangential contacts with low numbers of grain contacts per grains	27

Figure 2.7b	Grantham Sand, shows an interlocked fabric with features suggestive of pressure solution. The concavo/convex contacts give a high grain to grain contact area, increased density and porosity reduction. After Palmer and Barton, (1987)	27
Figure 2.8	A scanning electron photomicrograph of clearly defined aragonite crystals in fill material of Senonian Chalk. Argued as evidence of recementing by Clayton and Matthews (1987)	29
Figure 3.1	Tema Mill will Chalk lumps loaded	34
Figure 3.2	The Wykeham Farrance 'Torshear' ring shear apparatus used in the research. a) A schematic diagram of the ring shear apparatus b) Sample cell, c) Plan view of shear measuring transducers and sample cell and d) General overview	39
Figure 3.3	Schematic diagram of stress path cell equipment used. Modified diagram from Toll (2002)	41
Figure 3.4a	Schematic diagram of hydraulic triaxial cell developed by Bishop and Wesley (1975) from Craig (2004).	42
Figure 3.4b	The London Imperial stress path cell used in this research	43
Figure 3.5	Imperial College 50cc volume change unit. Leddra (1989)	44
Figure 3.6	The 'dry press' apparatus	48
Figure 3.7a	Arrangement of suction cap	50
Figure 3.7b	Sample placement on base pedestal of stress path cell using suction split former	50
Figure 3.8	Graph showing membrane correction in consolidated drained shear triaxial tests	56
Figure 4.1	Mohr-Coulomb failure envelopes for three Hampshire Basin Chalks considered for the study	60

Figure 4.2	Particle size distribution of non calcareous content of Chalks of the study	61
Figure 4.3	Particle size distribution of Culver chalk putty	63-64
Figure 4.4	Particle size distribution of Newhaven chalk putty	65-66
Figure 4.5	Plasticity Chart (BS 5930) with plasticity characteristics plotted for the putties studied	67
Figure 4.6	Investigation of ageing effects on chalk putty using the cone penetrometer on Newhaven chalk putty	68
Figure 4.7	Consolidation curve for direct shear box sample under 100kPa instantaneous normal loading	71
Figure 4.8	Results from the ring shear apparatus	74-75
Figure 4.9	Particle grading changes in ring shear tests,RSPG1-2	77
Figure 4.10	Undrained (‘Quick’) triaxial tests on Newhaven and Culver chalk putties	79
Figure 4.11	Screen shots of Triax software. Graphs plot top pressure (blue) and back pressure (pink) response in undrained conditions against a 50kPa ramp in cell pressure (red). Sample tested, Culver chalk putty ground for 2 minutes in the Tema mill. This response was typical across all chalks putties reviewed using the ‘B’ test	81
Figure 4.12	Mohr-Coulomb failure envelope for advanced triaxial tests	83
Figure 4.13	Permeability results	83
Figure 4.14	The interrelationship between sample volume and pore-water pressure during the shear stage (time) for advanced triaxial test RCL1 (data sampling at 1minute intervals)	84
Figure 5.1	Sample collected from material subjected to two weeks ring shear grinding at 100kPa and 800kPa	100
Figure 5.2	General permeability and permeability reduction	105

Figure 5.3	One dimensional consolidation tests on remoulded chalks with differing initial porosities (Clayton 1990)	106
Figure 6.1	Contact changes after application of back pressure. a) Idealized soil particles with point contacts before back pressure increase, b) Interstices reduced and c) Interstices increased. After Xia and Hu (1991)	116
FigureA1-1	A sketch of the apparatus for producing lateral vibration. Sand is pluviated into the ‘shaker pot” partly filled with water, during the vibration, after Barton and Brookes (1989)	142
FigureA1-2	Cross-section of the Consolidometer developed by Razoaki (2000)	143
FigureA2-1	Calibration of p_c value for loss of head due to porous discs (I, III) BS 1377-6:1990 section 6.4.3. Average flow rate used during permeability tests was 0.05ml/minute	145
FigureA4-1a	Electron micrograph of intact Newhaven Chalk	149
FigureA4-1b	Electron micrograph of Newhaven chalk putty milled for 2 minutes in Tema mill	149
FigureA4-2	Occluded porosity created by the complex morphology of the coccolithopores that make up chalk putty	150
FigureA4-3	Newhaven Chalk milled for 2 minutes. Cement infilling can be seen in centre of frame	150

List of Tables

Table 2.1	Mechanism of diagenesis in English Chalks after Clayton (1990)	7
Table 2.2	Comparison of the main Chalk nomenclature schemes in current use for Southern England Chalks, after table 11, in Booth (2011)	11
Table 2.3	Effective peak and effective residual strengths for intact Chalk	12
Table 2.4	Structured Chalk grading, after Spink (2001)	14
Table 2.5	Table of published effective strength parameters for remoulded Chalk, based on Lord et al. (2002)	21
Table 4.1	Parent intact Chalks reviewed in the study	58
Table 4.2	Investigation of ageing effects on chalk putty strength, using direct shear box apparatus on Portsdown chalk putty	70
Table 4.3a-d	Assessment of shear strength deviation developed in (400-800kPa) ring shear tests, compared with the best fit line through the 50-300kPa data set	73
Table 4.4	Void ratio changes at different stages of the advance triaxial tests	82
Table 5.1	Ø' values for different loading stages of the ring shear, illustrating non-linearity of failure envelopes	94
Table 5.2	Inherent soil mass properties influencing susceptibility to particle crushing (after Sadrekarimi, 2009)	98

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Mrs Rose

“Of all the rocks with which I am acquainted, there is none whose formation seems to tax the ingenuity of theorists so severely, as the White Limestone or Chalk, in whatever respect we may think fit to consider it”.

Thomas Allan .1823. Transactions of the Royal Society of Edinburgh. Volume 9.

Chapter One
Introduction

1.1 Introduction and aims

Chalk putty is as ubiquitous as the intact Chalk from which it is derived. Although Cretaceous chalks occur over much of Europe, the Middle East and America, this study concentrates on samples of Upper Cretaceous Chalks from Southern England. Famously a white, pure limestone exhibiting little macrostructure, chalk putties are formed naturally and during earthworks.

Nearly all occurrences of putty can be problematic for the geotechnical engineer, Lord et al. (2002), and yet laboratory studies on the material remain infrequent. Minor but key references and review are made in Construction Industry Research and Information Association (CIRIA) publications (Foundations in chalk, Lord et al. (1993) and Engineering in chalk, Lord et al. (1993), and an International Chalk Symposium (Burland et al. 1990). Limited index parameters are listed, but omissions include studies of permeability and void ratio change on shear.

A review in Chapter Two highlights the difficulties a geotechnical engineer would have in seeking mechanical behavioural parameters for his work. Shear strength parameters are available in these publications but data are limited and varied. It is now commonly accepted that soils are influenced by preparation, testing conditions, stress environment and stress history, yet the conditions in which the parameters in these publications have been attained are not described. The value and quality of the studies from which the data have been sourced need to be questioned, as the parameters obtained in earlier studies have generally been given too much credit, and used beyond the scope envisaged by the authors of the studies.

As an example, consider Lord et al. (2002) tabulated reference to shear parameters derived by Twine and Wright (1991), and Clayton (1978), determining Ø' as 39° and 31°-33° respectively (Table 2.5). The testing method is correctly quoted as undertaken by triaxial and shear box but what is omitted is that the Twine and Wright results are from a large (300 x 300 mm) shear box where strains are higher than those present in triaxial tests. This presents difficulties for the reader in ascertaining reasonable values for Ø'. The discrepancy could be owing to a strain dependency or the samples may have been prepared differently or the stress ranges at which the tests were conducted may have been different.

Similarly Clayton (1977) investigated whether triaxial samples age and strengthen with time. A series of undrained triaxial tests were carried out over a 100 day period, one test being conducted after each time period. Cohesion was found to increase with time, when calculated from individual tests using extrapolation to p' = 0 by assuming a constant Ø' value. Since 1977, however, studies have shown that Ø' is influenced significantly by soil fabric and is unlikely to be a constant with time. Clayton’s work remains heavily cited but its limitations are not.

Other tests have similar limitations. Razoaki (2000) continued the study of ageing and strengthening in chalk putty. Strength parameters readily cited from Razoaki (2000) are from effective triaxial tests following a drained stress path under a constant confinement. Aiming to review the variable ageing and extend earlier work, samples were diligently formed using the consolidation of reconstituted ground Chalk. A limited starting p' range was chosen for the tests which unfortunately gave rise to an over-consolidation ratio which varied between tests. Known to be fundamentally important in clay studies, the influence of over-consolidation ratio (stress history) is unknown on chalk putties but is likely to have influenced the findings.

Having evaluated earlier work, *this* study aims to address the paucity of geotechnical data and to develop testing methodologies for a material which because of testing infrequence does not benefit from good representation in testing standards. The overarching aim can thus be thought of as supplementing existing codes and approaches in the geotechnical testing of chalk putties.

Two reconstituted chalk putties from different local sources provide the material for the study, where consideration of test procedure is viewed as important as the results obtained. Attempts to remove unwanted variables in testing are made, for example the manufacture of ‘identical’ triaxial test samples to control variables such as stress history and fabric. Where this is not possible or where a discrepancy was not expected, an assessment of a variable’s influence is made.

1.2 Layout of thesis

Following this introductory chapter, the thesis follows a conventional format: a review of current knowledge, a laboratory testing programme, presentation of data, discussion and conclusions. A glossary, for terms not defined within the text along with a list of principal symbol is included prior to the Appendices.

Chapter Two forms a critical review of current knowledge of the geotechnical behaviour of chalk putty. The lack of experimental provenance for published shear strength parameters is highlighted, as is its paucity. This questions whether direct comparisons can be made between previous studies without consideration of fabric, ageing and the stress environment at which testing has taken place. Origins, composition and classification of Chalk are also included to provide the reader with a reference to subsequent discussions and conclusions, where the material's microstructure and composition are considered important in explaining its geotechnical behaviour.

Chapter Three describes a testing program addressing the paucity in knowledge identified in Chapter Two. A review of literature and initial findings of conventional tests (notably the undrained triaxial test) highlight the difficulties of testing the material and the necessity to develop testing methodologies specific to chalk putties which are not represented well in testing standards. In order to rationalise and normalise advanced shear test results, testing is undertaken to create 'identical test samples', assess ageing and strain dependency.

Chapter Four provides presentation of the results with initial sections documenting index parameters. These add to the limited data already available and enable comparison and characterisation of material studied with that published. Subsequent sections then detail the findings of the normalisation tests prior to presentation of the shear strength results. Monitoring of pore water pressure lines in the advanced triaxial tests provide details of permeability and void ratio change during shear.

Chapter Five discusses the results presented in Chapter Four, with a development in understanding of the principal findings. The form of chalk putty failure envelopes are explained based on changes in micro fabric, which is assessed using particle size analysis in a laser analyser and scanning electron microscope micrographs. These are discussed in the appendices which supplement the main text without interrupting theme and flow.

The concluding Chapter Six reviews the main findings of the study. Summary is made of the testing methodologies developed to adequately test chalk putties of an 'identical' nature. Normalisation against the effects of stress history, fabric evolution and ageing by adaptations of test procedure or quantitative assessment is defended to provide a better understanding of shear failure. The nature of failure is explained with reference to permeability and p' values.

Constraints of the study are then considered regarding the unavoidable use of back pressures in advanced triaxials, before a review of further work. Suggested development of the testing procedure by using more relevant field stress paths introduces the 're-inflation' stress path, whereby deviator stress (q) is held constant whilst pore pressure is raised from a given drained shear state.

Chapter Two
Chalk Putty: the Material

2.0 Overview:-

2.1 Definition of chalk putty.

2.2 – 2.4 Review of intact Chalk from which chalk putty is derived.

2.5 – 2.9 Occurrence and geotechnical properties of chalk putty.

2.10 Objectives of the study.

2.1 Definition of chalk putty

The term ‘chalk putty’ encompasses a range of materials that are often referred to using the terms; ‘chalk slurry’, ‘remoulded chalk’ and ‘reconstituted chalk’. All forms have in common the loss of cement bonding between grains that is present in the parent material, intact Chalk. Intact Chalk obtains its strength from three components, namely cementation, inter-granular friction and inter-granular molecular bonding. Whenever intact Chalk is subjected to energy it breaks down into finer constituents and the cementation resistance is lost. The resultant material is known as chalk putty.

Lord et al. (1993, p.177) after Burland (1990) defined reconstituted chalk as “completely reworked chalk to which water is added to form a slurry before it is compressed”, remoulded chalk as “chalk reworked in its natural state without a change of moisture content” and chalk slurry “as completely reworked chalk that has a water content in excess of its liquid limit”. Razoaki (2000) alternatively describes chalk slurries as any reworked chalk, the moisture content and degree of compression being undefined.

In this study, the generic term ‘chalk putty’ is used, with information as to how it was formed provided as required. Following convention, chalk is spelt with a small ‘c’ when considered as a material; whilst Chalk with a capital ‘C’ refers specifically to geological formations deposited during the Creteacous period. Putties derived from Southern England Chalks form the study’s primary focus. The research may be expanded to include material derived from Chalks found elsewhere but, being organic in origin (see Section 2.2.1), it is important to note that there may be differences owing to species variation.

2.2 Sedimentology and lithology of Southern England Chalks

2.2.1 Composition

Chalk is a pure biomicritic limestone formed almost entirely of calcite. Calcium carbonate content is in the region of 95% for Middle and Upper Chalk. Although some calcite is derived from macro-organisms such as echinoids, the vast majority is present in the form of calcite plates or laths known as coccolith plates in the particle size range 0.5 – 4 µm. These plates once formed the protective exoskeleton (coccolithosphere) of coccolithophores. Coccolithophores are unicellular planktonic protests (a group of organisms neither plant nor animal). They are generally spherical or oval unicells < 20µm in diameter. The unicell has two golden brown pigment spots (hence inclusion in Chrysophta golden alga division) with a nucleus in between. Coccolithophores are typically autotrophic* nanoplanktonic; they utilise sunlight energy to photosynthesise organic materials into inorganic ones. Hence coccolithophores largely live in the top 0 - 200m of water, that is, in the photic zone. As a result, Chalk deposits exhibit a notably high initial porosity as coccoliths laths settle out of suspension. In present day tropical seas their abundancy can reach 100,000 cells / litre water.

*glossary

The coccolithosphere is formed of shields comprising coccolith plates created within vesicles inside the unicell. These coccolith plates move to the outside of the cell replacing others, which are shed. Chalk therefore is formed of not only partially intact coccolithosphore shields but also a multitude of coccolith plates. An individual algal organism may create considerable numbers of coccolith plates over its life span. Sometimes the coccolith plates are held together by the cell membrane (motile), at other times calcification of the membrane gives a more rigid structure. Coccolith morphology is therefore broken down into holococcoliths and heterococcoliths. Holococcoliths are formed of submicroscopic, usually rhombohedral, calcite crystals arranged in a regular order. Heterococcoliths are larger than the holococcoliths formed of submicroscopic elements like plates, rods and grains, which are held together in a somewhat more rigid structure.

The marine transgression during the Upper Cretaceous provided the ideal conditions necessary for an explosion in the species of cooccolithophores, shown in Figure 2.1 (Brazier, 1992). Cretaceous evolution saw more complex forms with cross bars (Figure 2.1f, g) and a variety of shapes again changing the morphology of the building blocks of Chalk. It is unknown how coccolithophore morphology affects the macro strength of Chalk or chalk putty.

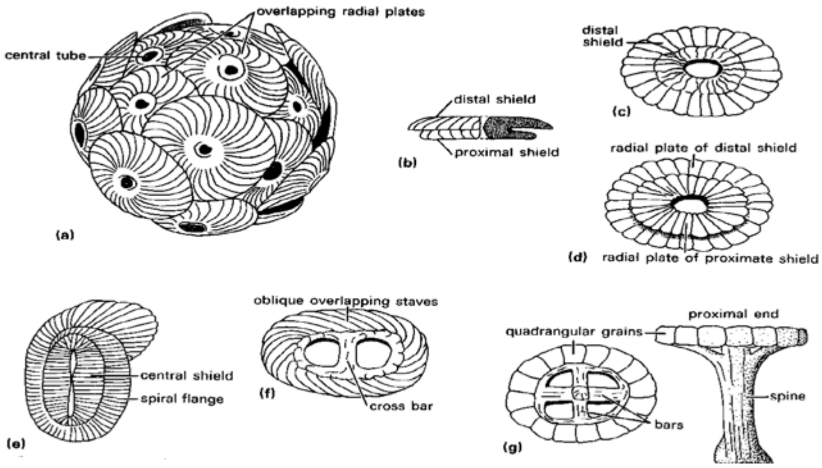


Figure 2.1 Structure of coccolithophores a) recent coccolithophore *Cyclococcolithina* x2870; b) side view of *Cyclococcolithina* coccolith shield with cross section; c) Pseudoemiliania distal view x3600; d) same from the proximal side; e) *Helicopontosphaera* x2930; f) *Zygodiscus* x5340; g) *Prediscosphaera* proximal and side view x4000. After Brasier (1992).

2.2.2 Depositional environment

Chalk was formed in Cretaceous seas, estimated to be 200 - 300m deep, approximately 100 - 60 million years ago. An extensive marine incursion had occurred over Southern England and beyond, giving ideal conditions for a deposit of entirely marine origin to be laid down. The lack of terrigenous material in Chalk is a palaeo-geographic indicator that the nearest land mass was some way off, probably low lying and under desert conditions. Only the Lower and Middle Chalks exhibit any significant amounts of terrigenous material. Lord et al. (2002) give a calcium carbonate content of 55-65% at the base of the Lower Chalk increasing to 90-95% at its top. Middle Chalks are typically 95-99% with Upper Chalks seldom having a calcium carbonate content of below 97.5%. Palaeo-climates would have been warm, encouraging prevalent algal growth within the photic zone. The final thickness of Chalk deposited and the extent of the marine transgression remain a matter of contention.

2.2.3 Diagenesis and the formation of Chalk

Chalk deposits are formed under gravity initially as coccolith debris descends to the sea bed, giving a highly porous material. In Table 2.1, Clayton (1990) tabulates the subsequent diagenetic processes that may or may not have taken place. Chalk is initially a granular material whose strength is reliant on the interlocking of grains. Early seafloor compaction and cementation in many situations provides sufficient strength for the Chalk to withstand later overburden without collapse and loss of porosity. The original porosity is maintained until effective stress levels are sufficiently high to induce grain crushing and breakage.

Mechanism	Influence in increasing Chalk density	Geographical Extent	Stratigraphic Extent
Intrinsic diagenesis (early lithification, prolonged seafloor exposure and bioturbation)	Potentially very strong	Very unpredictable	Very unpredictable
Consolidation (syn. Gravitational compaction)	Minor, except for high porosity Chalks	Everywhere	Greater for more deeply buried Chalks
Tectonism	Modest to strong	Around and including areas of steeply dipping Chalk	All levels
Late-stage solution	Strong to very strong	Lincolnshire and Yorkshire	All levels

Table 2.1 Mechanism of diagenesis in English Chalks after Clayton (1990)

Diagenesis significantly reduced the porosity from initial values of the order of 60% down to 15%. Carter and Mallard (1974) estimate that UK Chalks had a uniform diagenetic history. Buried beneath approximately 400m of over-burden, Chalks exhibited increased strength and density with depth, prior to post depositional processes such as tectonic activity and re-deposition of material through solution processes.

Pre-burial diagensis often occurred when the Chalk sediments suffered prolonged seafloor exposure and bioturbation. This is thought to reduce porosity as the initial porous fabric is broken down and reworked. A notably durable Chalk, The Chalk Rock, is thought to be a result of this diagenetic process. Prediction of the extent of such hardgrounds is extremely difficult due to the complexity of the paleao-sea floor topography. Addis (1989), for example, observes that chalk strata, in which material is highly cemented, can lie juxtaposed to strata in which the material is poorly cemented. He argues that although the degree of cementation can be attributed to depositional factors, it may also be as a result of re-cementation during yield and compaction due to burial. It is suggested that this re-cementation may be less important in Hampshire Basin Chalks than in North Sea Chalks where burial is deeper. There is general consensus on the collapse of the fabric under a given effective stress under isotropic loading.

The traditional view is that the onset of the Alpine orogeny stressed the Chalks in Southern England into plastic folds with occasional brittle faults. Tectonic activity was widespread after the Cenomanian (Table 2.2). The Alpine orogeny was thought to have been a phased event and different mountain regions were created during differing times (for example the Pyrenees formed in the Palaeocene to Eocene, the Carpathians during the mid Cretaceous and the Juras during the Pliocene) (Owen, 1976). In Southern England, Mid Oligocene sediments were involved in Alpine folding, which took place in the Miocene and was most intense along a line Isle of Wight-Purbeck-Weymouth. These echelon folds all have steep to inverted vertical limbs (see Figure 2.2).

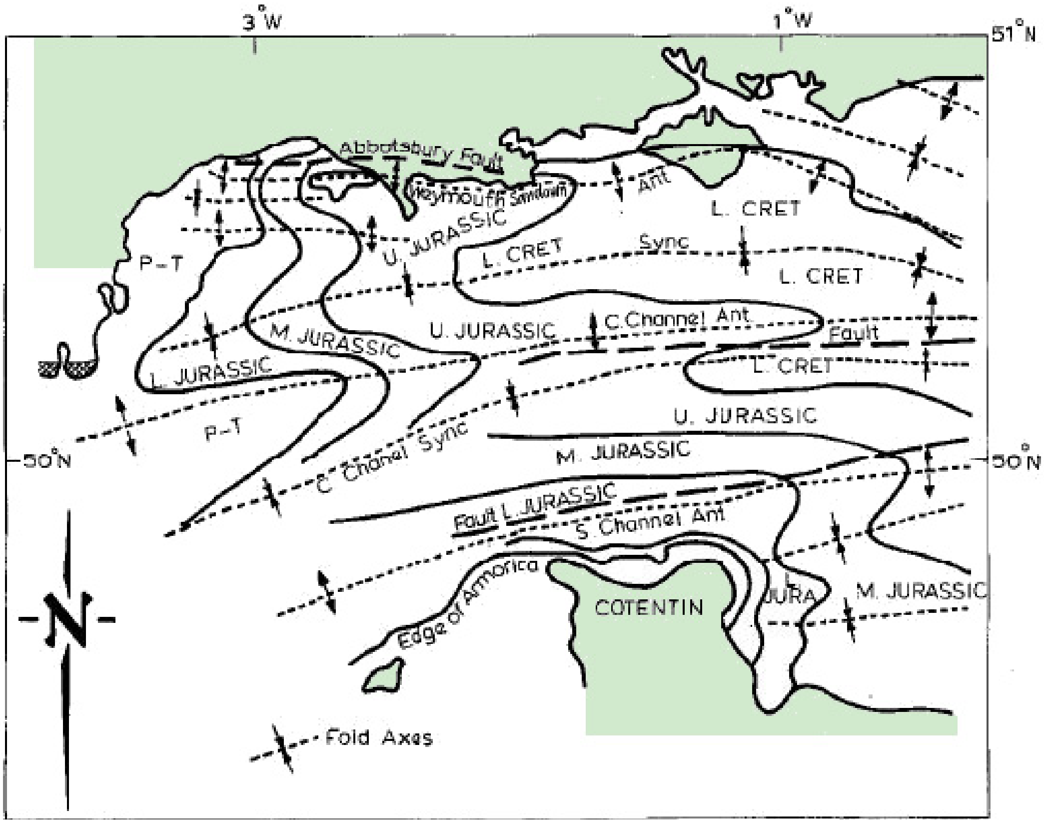


Figure 2.2 Channel outcrops of pre-Upper Cretaceous rocks either on sea – floor or inferred beneath the sub – Upper Cretaceous unconformity (Owen, 1976 based on Dingwall 1971)

Jones et al. (1984) revisited a phenomenon observed by Miram (1975) on the Isle of Purbeck and possibly elsewhere, where Chalk densities increase with dip angle of the bedding planes. After tests on several Chalk sites across the Purbeck monocline, they concluded that an increase in strength, hardening and cementation could occur much more locally and laterally (in the section) rather than vertically, as would be explained by traditional diagenetic burial processes.

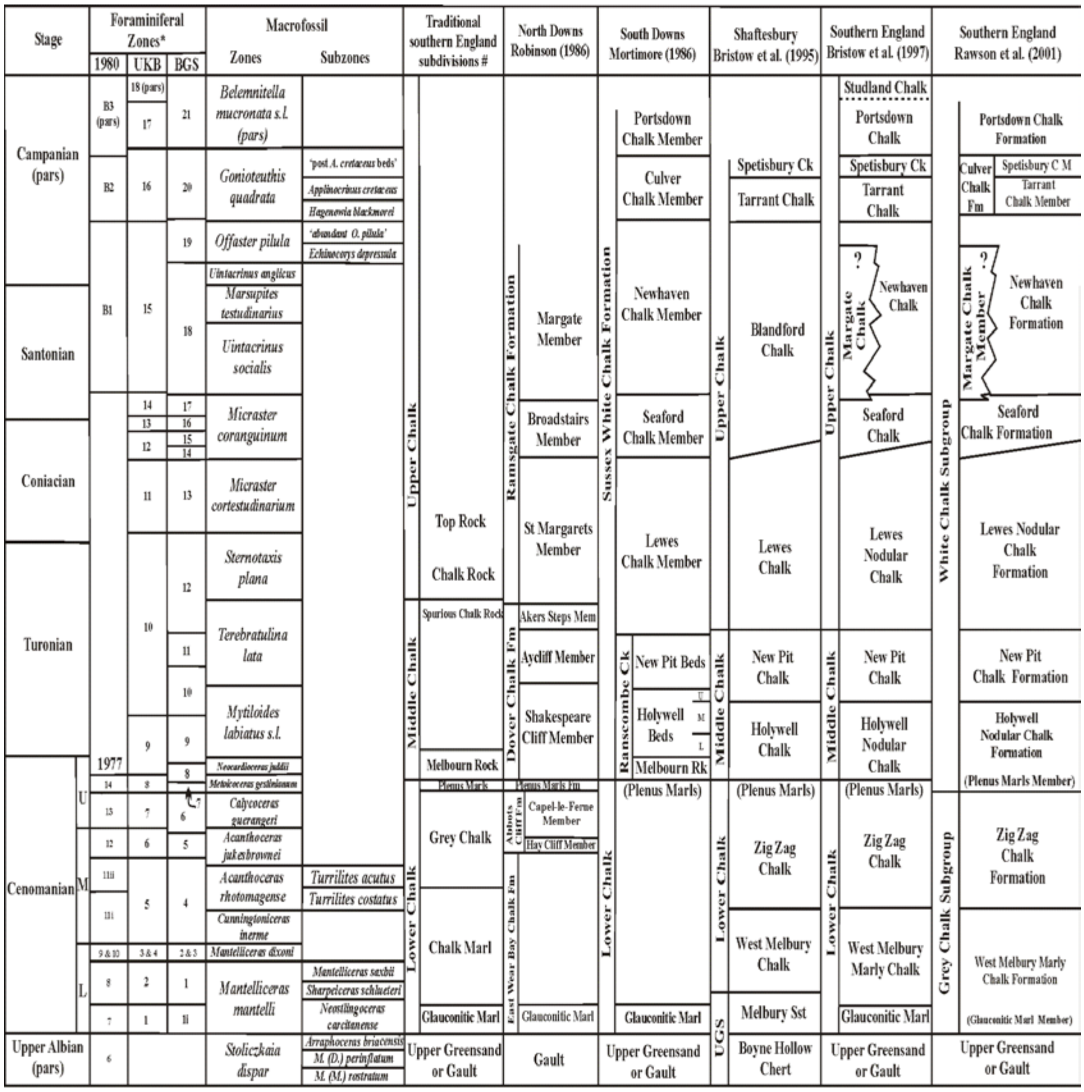
Jones et al. (1984) also emphasised the interlinking of consolidation (the time dependent volume reduction described by Terzaghi, 1923), cementation, and permeability. They describe how consolidation increases the differential effective stress (σ_v / σ_h) to a point where pressure solution is initiated. For cementation to be prolific, pore waters must migrate through a network of channels. Any lack of permeability not only reduces migration but also the differential effective stress that drives the system in the first instance. Tectonic activity influences all three parameters (consolidation, cementation and permeability) simultaneously. Jones et al. (1984) conclude that because of the importance of consolidation at low stresses and the high porosities of many Chalks, they are inherently sensitive to changes in their stress environment.

2.3 Stratigraphic and regional trends in Intact Chalk Strength

Bloomfield et al. (1995) and Carter and Mallard (1974) have reviewed the dry densities and porosities of intact Chalk in England. When the English Chalks are subdivided in terms of gross stratigraphy, i.e. Lower, Middle, and Upper Chalk, and on the basis of the four geographical areas of Northern England, East Anglia, Thames and Chilterns, and Southern England, it is seen that in all regions there is a decrease in porosity commensurate with age from Upper to Middle to Lower Chalk. A regional trend is also seen with increasing porosity in the order Northern England, Southern England, Thames and Chilterns, and East Anglia. Lord et al. (2002) give a range of dry densities for Chalk of 1.29 – 2.46 Mg/m^3 and a range of porosities between 9 – 52%.

The tendency of different types of Chalk to form putty on exposure to external stress is therefore, to some extent predictable as it will follow the trends in dry densities or porosities and is dependent on cementation. Post depositional fracturing, weathering and handling, however, will modify the material's behaviour significantly.

A more detailed stratigraphy exists for the Chalk but it is extensive, complex and beyond the scope of this thesis. The author echoes the sentiments of Spink (2002, p.367) on this issue, "The accurate determination of stratigraphic units in the Chalk is a specialized process relying on the correlation of flint and marl bands and identification of macro fossils and diagnostic Chalk textures". For the purposes of cross correlation of the "parent" intact Chalk used to make some of the test chalk putties in this thesis, Table 2.2 defines the correlation of Upper Cretaceous rocks with biozonial and Chalk nomenclature. For a more in depth review of Chalk stratigraphy, see Mortimore (2010) or Mortimore and Duperret (2004), although the often very detailed stratigraphic subdivision discussed therein may have only local importance.



Not to scale

#Traditional Chalk subdivisions after Jukes-Browne and Hill (1903,1904, for example). UGS = Upper Greensand; s.l. = sensu lato.

*Foraminiferal zones after Cater and Hart (1977), Swiecicki (1980), Hart et.al. (1989) UKB zones) and Wilkinson (2000) (BGS zones).

n.b. The Senonian stage (based originally on Chalk from Sens in the Yonne, France) is considered a former name to the Coniacian, Santonian and Campanian stages.

Table 2.2 Comparison of the main Chalk Nomenclature Schemes in current use for Southern England Chalks, after table 11, in Booth (2011)

Published data for Chalk strengths are scarce; a few are shown in Table 2.3. Peak strengths are achieved for first time shear, whilst residual values are those obtained post shear once a shear plane has developed within the intact rock. As test methods may vary, it is difficult to assess the degree of mechanical disintegration and the loss of cementation along the shear plane between the tests of Table 2.3. For example, the triaxial values provided by Leddra and Jones (1990) are critical state values obtained after the application of a high effective confining pressures (i.e. 15-18MPa). This is likely to have resulted in a de-structuring of the Chalk prior to shearing. Peak (Ø′) and post yield residual ($Ø_r′$) values for intact Chalk are observed to be similar to those discussed for remoulded Chalk, in Section 2.7.

Author	Provenance	Peak Effective Cohesion kPa	Peak Effective Friction Angle °	Residual Effective Cohesion kPa	Residual Effective Friction Angle °
		c′	Ø′	c′	$Ø_r′$
Hutchinson 1972 (Shearbox)	Upper Chalk Micraster Coranguinum, Joss Bay, Kent	131	42	0	30
Twine and Wright 1991(Triaxial)	Upper Chalk, Caister St. Edmunds	>20	39	0	36
Twine and Wright 1991 (Shear Box)	Upper Chalk Caister St. Edmunds	70	39	0	39
Hoek and Bray 1981(Back Analysis)	Upper Chalk, Caister St. Edmunds	20 - 52	39	/	/
Leddra and Jones 1990 (Triaxial)	Lower Chalk, Butser Hill Quarry	/	/	0	31

Table 2.3 Effective Peak and Effective Residual Strengths for Intact Chalk

2.4 Engineering classification of intact Chalk

There are generally considered to be two systems for the systematic description of Chalk in the United Kingdom (UK), the Mundford scheme and the CIRIA system. Both are summarised by Spink (2002) but it is worth noting that Spink makes the observation that "Chalk grading by either the Mundford or CIRIA schemes should not be considered as a replacement for the full description of the Chalk" (Spink, 2002, p.363).

The earliest classification was developed for the Middle Chalks of Mundford, Norfolk by Ward et al. (1968). It is generally known as the Mundford system. The Chalk is divided into structured Chalk exhibiting bedding and jointing, and structure less material often occurring near the surface as a product of weathering. Initially five grades were made; these were increased to six by Wakeling (1970). The classification utilises discontinuity spacing and discontinuity aperture. It is widely adopted for all Chalk types in the UK and is no longer reserved for the purposes of the Middle Chalk.

The second system of Lord et al. (1994) was developed for the Construction Industry Research and Information Association (CIRIA). The system links the dry density of structured Chalk to the grade, treating dry density as one of the most significant Chalk properties. Even in the CIRIA description, it is a requirement to repeat the dry density term (low, medium or high) in the sample description. Although praised for its simplicity, and demonstrating a strong link between engineering behaviour and grade, Spink (2002) expands on the observation by Bowden et al. (2002), that Chalk dry densities as field observations, are often significantly different to those measured in the laboratory.

Like the Mundford system, the Chalk is subdivided principally into intact and structured Chalk and structureless Chalk, with the structured Chalk grading being based on discontinuity spacing and aperture. The author is critical of the CIRIA system in that if a range of discontinuity spacings are visible, then the description varies depending on the final purpose of the description. For example, smaller spacing would be cited for grade purposes when the determination is for earthwork projects, whilst horizontal discontinuities would be used for a vertically loaded foundation. Therefore it is to be expected that the same Chalk could have two different descriptions depending on the final purpose of the description.

A comparison of the two systems is given in Table 2.4. The main difference between the two systems is the inclusion of additional grades in the CIRIA system. These equate to undefined composite grades that had been developed in the Mundford system in order to expand its applicability.

Typical discontinuity spacing and BS5930 terminology	Typical discontinuity aperture/infill thickness			Grade
	Open or infilled > 3 mm	Open or infilled <=3 mm	Closed or clean	
< 20 mm Extremely close	IV C5	NA (B5)	NA (A5)	Mundford CIRIA
20–60 mm Very close	IV C4	III/IV* B4	NA (A4)	Mundford CIRIA
60–200 mm Close	III/IV* C3	III B3	II/III* A3	Mundford CIRIA
200–600 mm Medium	NA (C2)	[II/III*] or {NA} B2	[II] or {I} A2	Mundford CIRIA
> 600 mm Wide	NA (C1)	NA (B1)	[II] or {I} A1	Mundford CIRIA

Key
* = Mundford undefined extended Grading; NA = Mundford grade not applicable; ( ) = Not common; [ ] = Low, medium and high density; { } = High and very high density

Table 2.4 Structured Chalk Grading, after Spink (2001)

2.5 Occurrence of chalk putty

The transition from intact Chalk to chalk putty takes place in any environment where energy is allowed to break down inter-particulate cement bonds. This energy may be as a result of shearing, vibration, crushing or degradation of the cementation. Some putties are therefore a result of natural processes, but many are a result of handling of intact Chalk during civil engineering projects. Putty occurrence is therefore extensive, the primary occurrences are however:-

2.5.1 *Putties that are found in areas that have undergone shear stress*, commonly occurring as a fill material in the discontinuity apertures of structured Chalk, Mundford Grades IV and III (Figure 2.3a). Where repeated involvement in landslips has occurred material can quickly degrade into the structureless Chalk grades of the Mundford System, V and VI, (Figure 2.3b).

2.5.2 *Putties formed by weathering*. Weathering of intact Chalk was particularly prevalent during the Quaternary Ice Age where freeze/thaw resulted in large deposits of soliflucted deposits. These deposits are common in the Southern England Chalks as freeze-thaw was greatest at the front of the palaeo-ice sheets that lay across the region in Quarternary times. The material is typically a melange of boulders and cobbles suspended in fines of chalk putty, (Figure 2.3c).

2.5.3 *Putties formed during civil engineering projects*. Civil engineering projects, such as foundations, cut and fill earthworks and tunnelling, are commonplace in Chalk, which has an outcrop of nearly 15% of England's surface area. Most modern systems of excavation, transportation and placement involve vibration, grinding or crushing increasing the likelihood of

i)

(photograph courtesy of Mark Birch, Sandown Life Boat)

ii) iii)

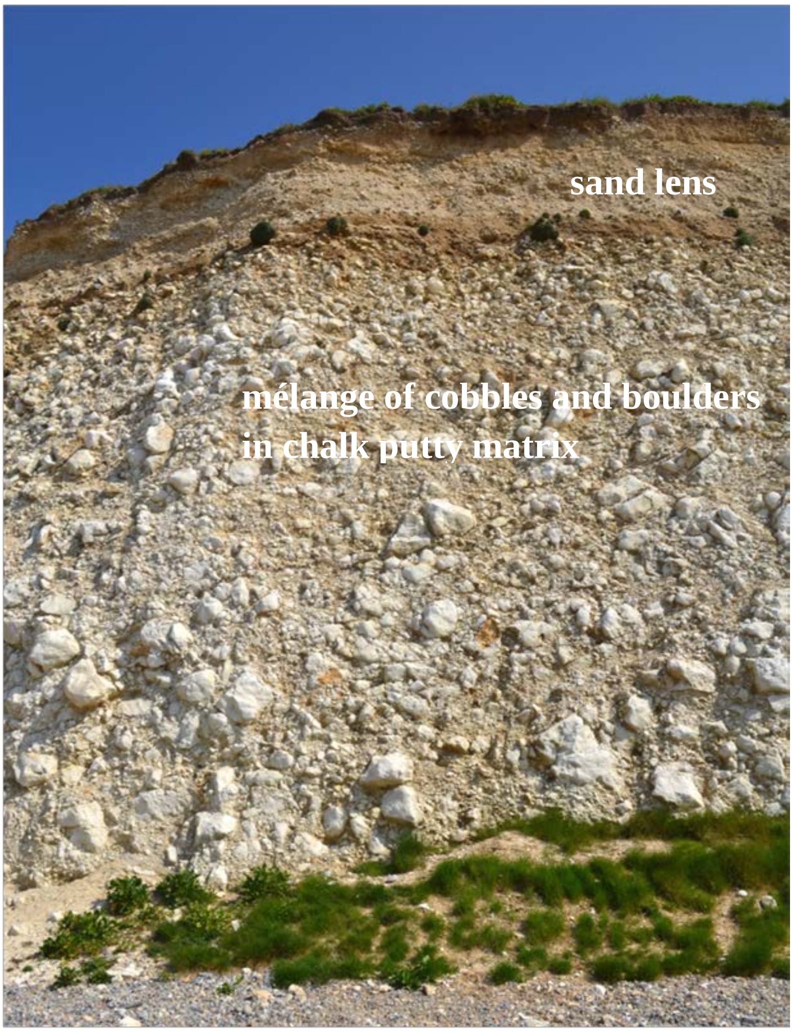
Figure 2.3a) i) Culver Cliff (Isle of Wight) fall of 19th May 2007, ii) Structured Chalk, Mundford grade III/IV, has chalk putty infilling joints and fractures. iii) Chalk putty formed along the shear surface which ultimately caused the cliff to collapse.

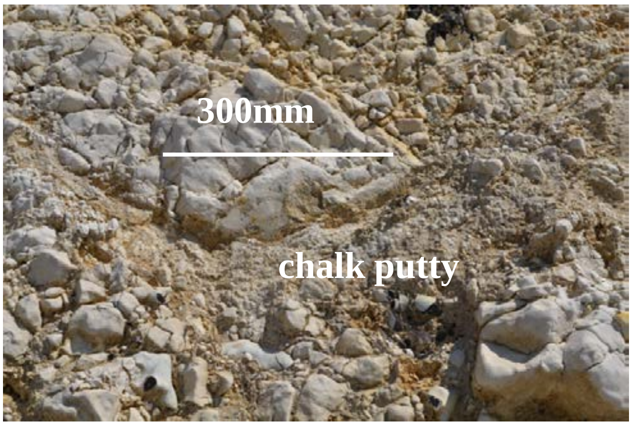
i)

ii) iii)

Figure 2.3b) i) Cliffs at Wheelers Bay (Isle of Wight), ii) and iii) Repeated landslips have resulted in structureless Chalk (Mundford V and VI), where the matrix is primarily chalk putty.

i)





ii)

Figure 2.3c) i) Cliffs at Freshwater, (Isle of Wight), approximate height 7m. ii) Upper Chalk has undergone freeze-thaw action to form soliflucted Coombe Deposits, where a melange of cobbles and boulders are suspended in chalk putty.

chalk putty as a common, but normally unwanted by product. Indeed modern compaction practices, which attempt to achieve fills with low air voids, can exacerbate the problem and are not a prerequisite of a satisfactory chalk fill. Many Victorian embankments built from end-tipped Chalk fill have proven very satisfactory, relying only on limited point contact of a high air voids material. Modern practices tend to produce more unwanted chalk putty because of the greater compaction and energy used in its placement. To counter the many difficulties of working with Chalk, CIRIA published the two key reports: *Foundations in chalk*, Lord et al. (1993) and *Engineering in chalk*, Lord et al. (2002).

2.6 Geotechnical index properties of chalk putty

2.6.1 Particle size distribution

Most mechanical action on intact Chalk results in an increase in the percentage of fines through crushing. Puig (1973) and Rat and Schaeffner (1990) observed that when the percentage of fines reaches 15-20% by mass, the whole material behaviour becomes dependent on the behaviour of the fines matrix. Few authors have further defined the nature (i.e. particle size distribution and angularity) of the fines, other than it is considered to be in the 0 - 400µm clay - silt range (Rat and Schaeffner 1990). Where studies on chalk putties have taken place they have only included a particle size distribution for the given material studied. For example, Puig (1973) reviewed artificially created putties from six French Chalks using vibration disintegration techniques. The chalk putties were created from the disintegration of intact Chalk under various moisture states, namely: dry, saturated, natural field moisture, and after water inundation for one hour. Clayton (1978) reviewed Puig's work and suggested that all of the fines created in the laboratory correlated well with site materials.

2.6.2 Atterberg Limits

Lord et al. (2002) give plasticity indices between 4 - 30% and liquid limits between 18 - 53%. High liquid limits of 30 – 34% are particularly common in putties derived from porous Upper Chalks, which Clayton (1978) considers to have a very uniform particle size distribution. The behaviour of chalk putties in partially saturated Atterberg tests differs markedly from that of standard phylosilicate* silts of the same silt size grading. Whereas standard phylosilicate silts would be expected to plot below the A-line, chalk putties are typically seen to plot above it in a similar zone to clays. This can be explained by the ionic charge of the composite particles. For chalk putties, plasticity indices tend to be much lower than those of clays. Chalk putties readily change from a plastic to liquid state with the addition of only small volumes of water.

The plasticities of clay / chalk mixes are shown by Perry (1979) to be governed by the equation: PI = 0.80 (W_L - 16.5), where PI = plasticity index and W_L = liquid limit. This relationship is very similar to PI = 0.80 (W_L - 16) derived by Clare (1948) for clay material alone. In Clare's work the clay originated from the Wealden beds.

2.6.3 Thixotropic properties

Many soils exhibit thixotropic behaviour when the moisture content reaches a certain value. This behaviour is common in materials that comprise high proportions of solid particles in the range 1-10µm (Freundlich and Jones 1936). When a soil behaves thixotropically, it changes from a gel to a sol. Boswell (1949) defines a gel to be a state in which a material will flow after an applied shear force reaches a given value and a sol as a state in which a material will flow under any applied shear force. Thixotropic materials, change state with the addition of energy not moisture. Chalk pastes (with impurities of 1.23%) are considered by Freundlich and Jones (1936) to be thixotropic. The degree of thixotropy may be quantified by establishing the void ratio (N value) at the point the material behaves thixotropically in an inverted test tube. The energy supplied to the system is through a finger tap to the side of the test tube. For chalk pastes Boswell (1949) states N values of 1.5 – 2.6, similar to those for Reading Beds (N = 1.4 – 2.7) and well below the classic thixotropic material bentonite (N = 14). From the limited Senonian Chalks pastes tested by Boswell, it would appear that they are as thixotropic as other silt/clay materials commonly tested in the laboratory.

2.7 Shear strengths in chalk putties

2.7.1 Published Remoulded strengths

Like other soils, putties may be tested under undrained or drained conditions.

2.7.1.1 Undrained Condition

Undrained test results suggest that true undrained conditions may be difficult to achieve in laboratory tests until full saturation is attained. 'Quick' unconsolidated partially saturated tests conducted by Clayton (1977) showed that unlike clays, putties show a large increase in compressive strength with increases in confining pressure, a behaviour explained by Lambe and Whitman, (1969) in Figure 2.4.

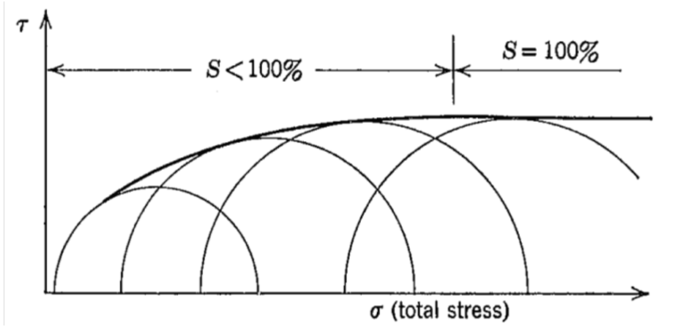


Figure 2.4 Typical unconsolidated undrained test data for partially saturated soil (after Lambe and Whitman 1969)

When partially saturated soils are subjected to confinement during triaxial testing, pore waters migrate to available voids and the material densifies. The rearrangement of particles to a denser state results in a strength gain and the effective stress on the soil skeleton increases. This effective stress increases until all pores become fully saturated at which point the true undrained state occurs during testing. Any gaseous state originally within the pores will either compress (gases compress infinitely more readily than liquids) or pass into solution. As a result, partially saturated, unconsolidated, undrained test data are dependent on the interelationship which exists between the drainage condition and saturation (a variable controlled by permeability).

2.7.1.2 Drained Condition

Effective strength tests by Clayton (1977, 1978), suggest effective angles of friction of between 29° and 34° with most in the range 31°- 33° (assuming c′ = 0 in all cases). It is assumed that the specimens tested were completely un-aged and that no effective stress history had been established in the making of the samples. Jenner and Burfitt (1974) also reported generally consistent values with a typical design value of 35° and an absolute minimum of 30°. Values of apparent cohesion (c′) ranged from 0 - 30kPa with 10kPa adopted as a typical design value. Jenner and Burfitt indicated that effective stress parameters for chalk fill are virtually independent of chalk type, moisture content and initial density. Clayton (1978) notes a general lack of variation with calcium carbonate content. Conversely, Perry (1979) used California Bearing Ratio (CBR) tests to show that even small quantities of clay impurities reduced CBR values considerably to levels that impeded trafficability. Lord et al. (2002) suggested that CBR values were commensurate with undrained strength. Other published results of shearing tests are given in Table 2.5.

Test Type	c′ (kPa)	Source	Ø°
Undrained triaxial	0	Lake (1975)	33° (critical state)
Drained triaxial	0	Clayton (1978)	31° - 33°
Drained triaxial*	0	Fletcher and Mizon (1983)	39°
Shear Box+	0	Twine and Wright (1991)	39°

*Drained triaxial tests on 8 remoulded samples at existing moisture content. Tests were carried out for the purposes pile design for the Orwell Bridge (National Grid Reference: TM 17020 41346). Fletcher and Mizon (1983).

+ Upper Chalk, sample size 300mm x 300mm tested in 'large' shearbox apparatus.

Table 2.5 Table of published Effective Strength Parameters for Remoulded Chalk, based on Lord et al. (2002). * + additional information

2.7.2 Illustrating shear strength in Chalk putties

For the purposes of reviewing how shear strength data are presented in this thesis, a short description on how shear strength is visualized follows. More detailed explanations, can be found in Craig (2004), Lamb and Whitman (1969), etc. Only the illustration of effective shear strength is reviewed, but shear strength in the undrained case may be illustrated in a similar fashion. Historically published shear strength data for chalk putties have tended to be expressed in the drained effective condition. It has long been observed that soil behaviour is most directly linked to effective strength and in many civil engineering projects involving chalk putty geotechnical design has required a drained analysis. It could be inferred from this, that chalk putty has sufficiently high permeability characteristics to meet the drainage needs of most designs, although no published work has been found to support this.

The shear strength of a soil sample is that strength that can be mobilised when a sample is subjected to shear forces. The stress at failure under different stress states is conventionally illustrated using a failure envelope. This may be a line or surface that demarks stress states in which a sample can exist or stress states where a sample would be in failure. Mathematically defining a failure envelope depends on the stress parameters used.

2.7.2.1 Coulomb failure envelope

This envelope is governed by the linear equation

$$\tau_f = c' + \sigma_f{}' \tan Ø'$$

where c′ and Ø′ are known as the effective shear strength parameters of a soil with c′ termed the cohesion intercept and Ø′ known as the angle of resistance. Stress points on the failure envelope are defined by the coordinates τ and σ_n , and so graphically the failure is illustrated by a straight line in τ and σ_n space. It is important to note that shearing resistance can only be developed by interparticle friction as no shear resistance can be developed by any interstitial liquids or gases. When the effective stress is zero, therefore, the shear strength (c′) must also be zero unless there is cementation between the particles. The Coulomb envelope is particularly useful in illustrating the results from the direct shear apparatus (Section 3.7.1) and the ring shear apparatus (Section 3.7.2) because the parameters of shear stress (τ) and normal stress σ_n are readily measured across a pre-defined failure plane in plane strain.

2.7.2.2 Mohr – Coulomb failure envelope

The Mohr-Coulomb envelope is a line (often curved) that can be drawn tangentially to the points of failure stress that will occur on the circumference of a series of Mohr circles as shown in Figure 2.5. Each Mohr circle is a circle defined by the effective parameters of principal stress (σ_1' and σ_3') applied to a test sample of soil. The effective principal stresses of σ_1' and σ_3' are known as the major and minor effective principal stresses respectively. By definition at any stressed point with in a material (termed here as an element of soil), the stresses that exist can be expressed by Cauchy's second order stress tensor* as:-

$$\begin{Bmatrix} \boldsymbol{\sigma_{xx}} & \boldsymbol{\sigma_{xy}} & \boldsymbol{\sigma_{xz}} \\ \boldsymbol{\sigma_{yx}} & \boldsymbol{\sigma_{yy}} & \boldsymbol{\sigma_{yz}} \\ \boldsymbol{\sigma_{zx}} & \boldsymbol{\sigma_{zy}} & \boldsymbol{\sigma_{zz}} \end{Bmatrix}$$

This three by three matrix simplifies when the soil specimen is considered to be in a state of equilibrium, as $\sigma_{xy} = \sigma_{yx}$, and $\sigma_{xz} = \sigma_{zx}$, and $\sigma_{yz} = \sigma_{zy}$ and the matrix can be reduced to six independent stress parameters of three principle stresses and their associated principal directions.

*glossary

For soil in a triaxial cell further simplification comes by the reduction of the three orthogonal principal stresses into two, because of axial symmetry whereby $\sigma_2 = \sigma_3$, owing to the isotropic nature of the confining medium, water. Scaling up the soil element approximates closely to a triaxial test sample (Section 3.8.2) where σ_3' equates to effective cell pressure and σ_1' is readily calculated from the recorded deviator stress as the test sample is vertically compressed.

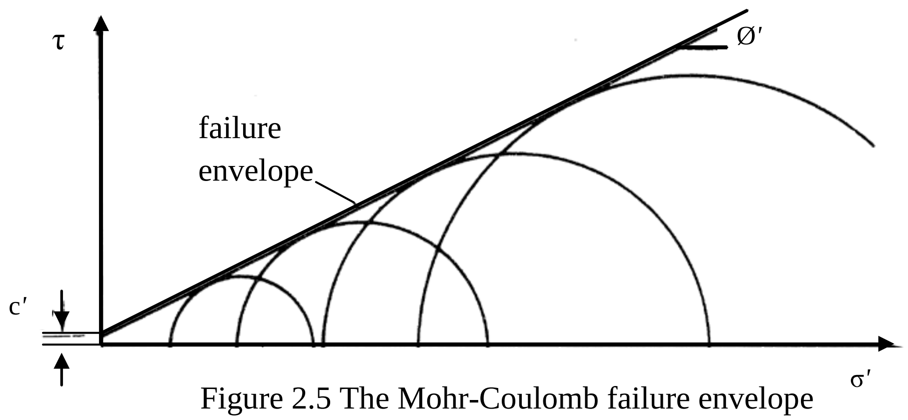


Figure 2.5 The Mohr-Coulomb failure envelope

2.7.2.3 Failure defined in p′ and q space

Because the use of major and minor effective principal stresses alone still requires an expression of directional orientation in a given stress field, it is beneficial to consider the eigenvalues of Cauchy's second order stress tensor matrix. These are known as the stress invariants p′ and q and importantly possess the property of remaining constant regardless of the rotation of the reference axis. They can be defined as

q = deviator stress = $\sigma_1 - \sigma_3$

p = mean stress 1/3 ($\sigma_1 + 2\sigma_3$)

p′ = effective equivalent

Due to the simplicity of illustrating failure and the property of the values not being directional, p′ and q space has been widely adopted to illustrate soil failure. The terms are extensively used in the topic of critical state soil mechanics.

2.7.2.4 The use of Critical State Soil Mechanics in defining chalk putty behaviour

Introduced by Schofield and Wroth under the mentoring of Roscoe, Critical State Soil Mechanics is eloquently explained in the eponymous book of 1968.

The concept suggests that soil and other granular materials, if continuously distorted until they flow as a frictional fluid, will come into a well-defined critical state determined by two equations:-

$q = Mp$ ………..equation 2.1

$\Gamma = \upsilon + \lambda \ln p'$ ……….equation 2.2*

The constants M, Γ and λ represent basic soil material properties that are intrinsic to a particular soil and depend only on the nature of the soil, i.e. on its grading, mineralogy, shape and texture of grains. The tests used to describe the nature of the soil are grading tests and Atterberg limits as discussed in Sections 3.5 and 3.6. The parameters q, υ and p' are deviator stress, specific volume and effective mean stress respectively. q and p' are as described previously in 2.7.2.3 and υ is known as the specific volume as defined in the glossary.

Two idealized isotropic plastic models were developed and named after the hypothetical materials they describe. The two materials, Granta-gravel and Cam-clay, were named after different reaches of a local Cambridge river. Of the two models, the Cam-clay model was subsequently the more successful in describing the observed behaviour of saturated remoulded clays in triaxial compression. The Granta gravel model failed to support the results seen with triaxial compression of undrained samples, as no distortion was predicted prior to reaching the critical state line; i.e. the model was a rigid / perfectly plastic model. Indeed many samples were seen to proceed to failure after only slight deformation. The Cam-clay model alternatively was a flexible / plastic model that highlighted the need for samples to yield and approach the Critical State Line through either dilation and reduced pore pressures (i.e. on the dry side of the critical state line) or through structural collapse and the establishment of a passive pore water pressure (i.e. on the ‘wet’ side of the critical state line). The terms ‘dry’ and ‘wet’ of the critical state line were introduced to explain these yield states whereby test samples are seen to increase or decrease in volume respectively.

The Cam clay model is represented in Figure 2.6. Each yield surface (or intrinsic state boundary surface) is individual to the material being considered and the material normally exists at a state below the surface unless it is cemented or re-cemented to some degree. The normal compression line (NCL) is also shown marking a materials specific volume response on virgin loading to a given mean stress p'. The line is curved in υ - p' space but straight in υ - $\ln p'$ space*. Wet of critical state is demarked by the surface between the NCL and Critical State Line (CSL), and is sometimes referred to as the Roscoe surface. On the dry side of the critical state line is the Hvorslev surface.

*$\ln p'$ is the natural logarithm of p' where the base is $e \approx 2.71828$. Equation 2.2 may be re-written in any log base and was first seen in the form $e = e_0 -$

$C_c \log_{10} \sigma'/\sigma'_0$ eg. Terzaghi et al. (1996) to represent virgin compression in consolidation experiments (where e and e_0 denotes the void ratio at pressures σ' and σ'_0 respectively, and C_c is known as the compression index). The change in log base is purely for mathematical convenience in the development of critical state theory. It should be remembered that void ratio/specific volume (or a dependant variable) approximate to a linear form when plotted against pressure in any log base. This will have further significance in Section 5.4.4 where the reader is presented with the juxtaposition of Figure 5.3 and equation 5.1.

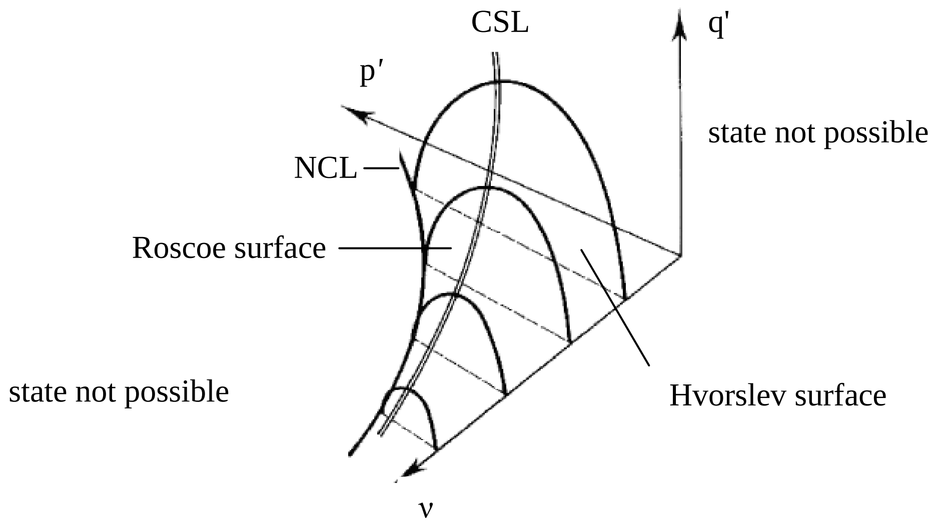


Figure 2.6 Cam Clay Model Yield Surface. (Schofield and Wroth, 1968)

Leddra (1989) convincingly reviews the deformation of intact Chalk and its transition to a less cemented material within a critical state framework with a considerable amount of supporting test data. Leddra explains that with intact Chalk at low stresses, the initial response is as an elastic material. When subjected to uniaxial or isotropic loading the skeletal structure of the Chalk forms much of the total strength of a test sample. In the elastic stage pore pressures are raised only slightly and volume changes are small. It is necessary for the samples to yield at the elastic envelope (higher than the Critical State Line) before undergoing structural changes to become a more destructured material. If the critical state is viewed as a residual strength and elastic yield as peak strength, they are seen to move closer together with increasing confining pressure. The confining pressures of 1 - 50MPa, used in Leddra’s tests, were well above those used in the present study. Many of the samples showed a distinct p' / q space linearity, indicative that a critical state had been reached after elastic yield.

2.7.2.5 Non-linearity of failure envelopes

The non-linearity of failure envelopes in saturated soils has long been established (de Mello, 1977; Fredlund, 1989; Coop et al. 2004; Sadrekarimi, 2009). Both granular soils and clays are seen to exhibit non-linear failure envelopes, dependent on the effective mean stress at which the shear strength has been studied. It may only be for simplicity that the shear data in Table 2.5 is presented as linear, with constant c′ and Ø′. The p′ stress range at which tests are conducted is seldom, if ever, quoted along with the strength parameters attained.

de Mello (1977) argues the case for a tri-linear failure envelope for saturated compacted fine grained cohesive material. To understand the different angles of the failure envelopes (described by de Mello as ‘zones’ or ‘universes’) it is considered important to understand the different compaction densities of a sample. A transition between ‘universes’ would take place when the testing mean effective stress range passes from a pre-consolidated pressure to one where virgin compression is taking place (viz. the over consolidated to normally consolidated boundary). For saturated granular soils de Mello found a series of ‘power law’ curves could be used for soil samples derived from different rock types.

Other studies concentrate on the fabric of soils as being fundamental to values of Ø′. Work by Palmer and Barton (1987) focussed on a study of shear strength failure envelopes of several sands, all of which were uncemented and matrix free. Any cohesion was as a result of locking of the grains. It was observed that the samples gave approximately linear envelopes over the stress range 50 - 900kPa and showed increased values of c′ and Ø′ with the age of the sand, correlating with porosity reduction and an increase in the number of grain contacts per grain. It was argued that the most likely cause of reduced density and particle interlocking could be explained not by mechanical compaction but by chemical compaction. Chemical compaction is defined as a process whereby inter-granular pressure solution occurs between particles as illustrated in Figure 2.7. Where micro-pitting and contact hollows consistent with solution occur, c′ and Ø′ are seen to increase, as greater interlocking of particles takes place. It is important to note that the inferred assumption is that micro pitting and solutions hollows have occurred at high pressures (MPa) and over many millions of years.

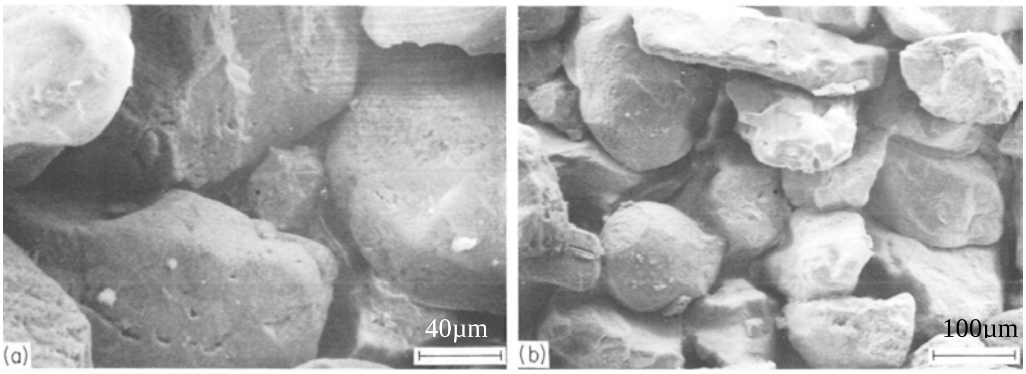


Figure 2.7 Particle locking in sands

2.7a) Barton Sand, shows tangential contacts with low numbers of grain contacts per grains b) Grantham Sand, shows an interlocked fabric with features suggestive of pressure solution. The concavo / convex contacts give a high grain to grain contact area, increased density and porosity reduction. After Palmer and Barton, (1987)

These features were not observed developing in the sands during the time scale of laboratory testing, but may be seen only between sands of different geological age. The dissolution of calcite grains in Chalks and its re cementing is reviewed in 2.8.2. Unlike silica sands, the assessment of calcite cements are considered more problematic, since observed deposits of super soluble calcite cement are extremely rare.

Developing the understanding of the relationship between Ø′ and sample fabric, Sadrekarimi (2009), reviewed how initial void ratio (degree of consolidation), grain hardness, shape, and size may all be interlinked with the value of Ø′. During shear tests, granular materials are expected to suffer particle breakage and should show a greater volume reduction at higher confining pressures, (Alva-Hurtado et al. (1981) and Razoaki (1994)) to give non - linear failure envelopes.

Nearly all studies on the relationship between Ø′ and fabric have been conducted on sands, with the exception of Razoaki (2000). Contrary to observations in sands, when conducting four drained triaxial tests on chalk putty, Razoaki found volumetric strain changes owing to repacking and crushing of particles to be smaller at higher confining pressures. It was inferred that chalk slurries must suffer minimal particle breakage on shear deformation in contrast with other granular materials.

2.8 Effects of ageing on chalk putties

Atkinson (1993), considered the principal ageing processes of soils to be compaction, creep, cementing, weathering and changes in pore water salinity. These processes are independent of primary consolidation, a process whereby loading or unloading of a soil causes changes in effective stress. As effective stress changes, pore water pressures also change, leading to drainage and volume change. A short review follows.

2.8.1 Compaction and creep

Both compaction and creep involve volume changes without a change in effective stress. With compaction the volume change is fairly instantaneous, whilst creep occurs slowly and at a rate that decreases with time. Since compaction is fairly instantaneous only creep may or may not be important in laboratory shear testing. Lord et al. (1993) suggest creep is significant in chalk CIRIA Grades B, C and D (see Section 2.4 and Table 2.4). Field observations using plate load tests exerting a constant stress of 250, 445 and 665kPa suggest a rate of creep typically lying between 0 and 0.9 mm / log.cycle of time for a Grade D chalk. Creep rate is seen to increase with applied stress and decrease with improved grade of Chalk.

The terms compaction and creep have developed wide meanings and it is considered appropriate to restrict their meaning to initial compression and secondary consolidation respectively within this study. The terms primary and secondary consolidation are discussed further in Section 4.5.2.

2.8.2 Cementation

The critical states of cemented and uncemented material should be about the same, because the influence of cementation will principally be on yielding and at small strain stiffness. General cementation in intact Chalks is believed to be by calcite overgrowths of the original coccolith grains, but this is not readily seen under the electron microscope*. In putties which have lost these calcite overgrowths, cementing involves accumulation of additional material, usually calcite or aragonite, in pores. This reduces specific volume (i.e. porosity), with the skeletal structural change causing a shift in the state boundary surface. The degree with which recementing occurs in chalk putty is not well understood and like the intact Chalk cement, is seldom seen. Clayton and Matthews (1987) have described redeposition of aragonite in compacted Senonion chalk fills forming part of an embankment in the south-eastern sector of the M25 motorway. Figure 2.8 shows a scanning electron photomicrograph of clearly defined aragonite crystals which were not seen to be present prior to excavation. However since aragonite is a high pressure polymorph of calcite it is surprising to see it at near surface temperature and pressures where it should quickly degrade to calcite again. Because the calcite coccoliths of Chalk are chemically stable, it is unlikely that any chemical recementing occurs. If cementing does occur, it is by a physical process of dissolution and recrystallisation of calcite. The process of recementing in chalk putties should not be confused with the chemical processes that occur when quicklime reacts with CO_2 to form a mortar, or that seen in other pozzolanic materials where cementation occurs in a siliceous (or siliceous and aluminous) material.

*Occasionally localised cementing, whereby localised voids have been in filled with calcite, can be observed as shown in Appendix Four, Figure A4-3.

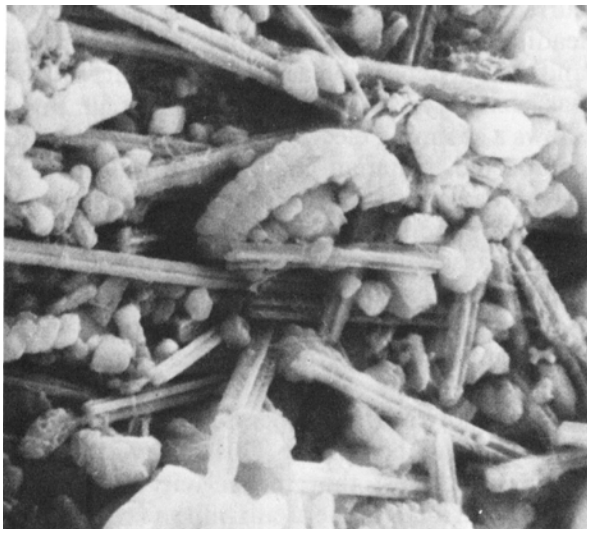


Figure 2.8 A scanning electron photomicrograph of clearly defined needle - like aragonite crystals in fill material of Senonian Chalk, argued as evidence of recementing by Clayton and Matthews (1987). No scale given

2.8.3 Weathering

Weathering causes physical and chemical alterations in soil at constant effective stress. Unlike cementing, both the intrinsic state boundary surface and the critical state line move as the grading and mineralogy change. The most common cause of weathering in Chalks is through the process of frost shattering, also known as freeze / thaw action. Soft (low dry density) Upper Chalks have the greatest frost susceptibility amongst the different Chalk types. Lord et al. (2002) notes that frost susceptibility increases with porosity and decreases with permeability.

For chalk putty both Clayton (1977) and Lord (2002) consider the main strength / time dependency to be an increase in effective strength with excess pore water dissipation, when there is no loading or unloading. Clayton studied a series of remoulded chalks under drained triaxial conditions, after different time periods from sample preparation. Test sample moisture loss was reported as minimal and an average Ø′ was assumed when calculating c′. An effective strength gain was observed up to 89 days before levelling off.

Perry (1979) also records an indirect time dependent strength gain in his work using CBR tests. In a series of CBR tests on Chalk / clay mixes he found a significant increase in CBR values over the first 8 days from sample preparation. The increase was greatest where the Chalk to clay ratio was highest. CBR tests are recorded to follow trends in undrained tests (Clayton 1978), a logical observation since CBR sample loading is relatively instantaneous. Razoaki (2000) and Clayton (1977) also consider recemention to be possible, but Lord (2002) argues that Chalk is unlikely to recement in fully saturated conditions.

2.9 Chalk putties and engineering classification system, Mundford and CIRIA

Chalk putties are associated with Grade V and VI of the Mundford system. Grade V is defined to be structureless remoulded Chalk containing small lumps of intact Chalk. Grade VI (as added to the Mundford system by Wakeingly 1970) is defined to be extremely soft, structureless Chalk containing small lumps of intact Chalk.

Under the CIRIA system chalk putties are associated with grade D materials, which are further subdivided into D_m and D_c grades. Lord et al. (2002) defines the D_m grade to be a fine soil where the matrix dominates. The Chalk matrix is greater than 35% with coarser fragments under 65%. The D_c grade is defined to be a coarse soil dominated by clasts, where the matrix is less than 35% and the coarse fragments are over 65%.

Rasoaki (2000) highlights the view of Puig (1973) and Rat and Schaeffner (1990) that significant portions of D_c graded Chalks have fines greater than 15%, considered to be the critical percentage of fines that controls the behaviour of the whole material.

2.10 Objectives

The following objectives will be investigated by this thesis.

i) From a review of chalk putty literature it is evident that there is a paucity of shear test data on chalk putty. Drained (effective stress paths) and undrained data sets exist (Table 2.3 and 2.5), but there is little clarity as to sample stress history prior to testing. Generally tests are not conducted under strict un-aged conditions, i.e. the starting stress history of test specimens has not been recorded and as a result comparison may not be possible between sets of data.

ii) It is unclear why the limited published data suggest a variation of effective friction angles for putties formed from material of similar provenance. De-structuring, remoulding or reconstitution of rock or soils traditionally results in a greater uniformity of shear strength parameters. Tables 2.3 and 2.5, however, suggest a more consistent friction angle for intact Chalk than in subsequent de-structured forms.

iii) The shape of the failure envelope for chalk putty is not defined in literature. Many soils are now considered to exhibit a curved failure envelope (both in undrained and drained conditions) with lower friction angles at higher confinements. The data reviewed have assumed a constant Ø′ across all p′ conditions for chalk putty probably because results are limited and testing has been conducted at infrequent loading stages.

iv) It is accepted that many of the behavioural issues of chalk putty arise because of its low permeability, requiring time for pore water pressures to equilibrate. To the knowledge of the author, no data are available on how permeability changes with p′, or how failure affects permeability. It is not clear whether chalk putty approaches its critical state by dilation or volume reduction. The response of porewater pressures to any volume change during failure is unknown.

v) There is little published data on the evolution of chalk putty particle size with grinding. Although Lord et al. (2002) state that particle size has little effect on shear values, this has not been extensively explored. The microscopic changes with grinding require further investigation to help explain macroscopic behavioural responses.

vi) Existing work by Clayton (1977) on ageing effects on remoulded Chalk samples under drained consolidated triaxial test conditions is fundamentally flawed. The study is frequently cited as key research, but test analysis that assumes a constant effective angle of friction (Ø′) should be questioned. It is probable that effective forces between grains will change (with time) as a result of pore pressures changes, fabric changes and re-cementing*. These changes would corrupt the argument that a constant Ø′ can be assumed.

*(re-cementing suggested by Clayton 1977, 1987)

vii) There remains a lack of clarity on whether chalk putties behave as a granular material like sands (where issues of cementing, grain interlocking and particle breakage are important) or a cohesive material such as clay (where particle alignment and over consolidation history are important).

Chapter Three
Experimental Procedure and Methodology

3.0 Overview:-

3.2 - 3.4 Characterisation of the intact parent Chalk material, in terms of its composition and likelihood to develop chalk putty.

3.5 – 3.7 Characterisation of the studies of chalk putty in terms of its particle size, liquid / plastic response and susceptibility to soil ageing.

3.7 Shear strength tests using the ring shear apparatus and conventional direct shear box.

3.8 Triaxial testing of chalk putty, incorporating the development of an advanced triaxial testing methodology that eliminates the variables of ageing and effective stress history that result from sample preparation.

3.1 Sampling procedure

Chalk putties were formed from intact Chalk samples collected from three main sites: Longlands Quarry, Portsdown Quarry and Ballard Down. Further details of the sites are shown in Table 4.1 but each Chalk and its derived putty are subsequently (in this thesis) referred to as Culver, Newhaven and Middle Cenonanian; these being the statigraphic horizons from which samples were collected. Sampling took place over several visits, with great care being taken to remove material from the same location at any given outcrop. All sampling used light pick action to remove the Chalk in a blocky form. For intact Chalk tests, field orientation was marked and replicated in the laboratory.

Chalk putties were created using a grinding process within a Tema mill. The mill in Figure 3.1 uses a ring and cylinder which eccentrically mills when vigorously vibrated. To achieve consistency between milling, all intact samples were air dried to a constant weight, gently broken into gravel-size with diameters less than 30mm, and loaded into the mill at a set mass of 170g per mill cycle. Variation in particle size was achieved by milling for different periods. Distilled de-aired water was then added to the resultant milled material to form putty with the required moisture content.

Figure 3.1 Tema Mill, with Chalk prior to milling

3.2 Shear failure envelopes of parent intact Chalk material

The failure strength envelopes of the intact Chalk, which were subsequently milled to form a putty, were established using either a Hoek cell (for confinements above 600kPa) or a standard soil triaxial cell. Tests in both cells were completed in accordance with the procedure outlined in The International Society for Rock Mechanics (ISRM), *Suggested Methods for Rock Characterization, Testing and Monitoring*: 1974 - 2006, Hudson and Ulusay (2007). Strain rates were such that tests were conducted over the recommended 5-15 minutes in drained conditions.

3.3 Density and porosity calculations of parent intact Chalk material

Porosity calculations were made in accordance with the ISRM Suggested Methods (Hudson and Ulusay, 2007) using specimens of a regular geometry, with masses greater than 50g. Cylindrical samples, of regular geometry, were prepared using water flushed diamond tipped core barrels in a pillar drill. Samples were cored to 38mm diameter for Culver and Newhaven Chalk with smaller 25mm diameter samples made for Middle Cenonanian Chalk, because its more fractured nature prevented larger samples being cut. Each end of the cylinder was first cut and then ground flat perpendicular to the sides using carborundum (silicon carbide) on a lapping wheel. The test procedure relies on the use of water ingression to calculate the volume of pore-space. It is more accurate on samples having a high effective porosity with negligible occluded porosity. Bloomfield et al. (1995) concluded from the clear link between porosity and dry density data that chalk matrix is essentially interconnected with negligible occluded porosity.

3.4 Determination of non-calcium carbonate material in the chalk putties tested.

As reviewed in Section 2.2.2, most Chalks, and therefore their putties, contain impurities of clay and silt, in addition to the calcium carbonate from which they are formed. Although silt is not recorded to have any significant effect, Sections 2.6.2 and 2.7.1 indicate that the presence of small quantities of clay impurities can affect the geotechnical behaviour of putties. Assessment of non-calcium carbonate content is therefore necessary to determine any influence of clay.

No standard procedure was found for the calculation of non-calcium carbonate content. Leddra (1991) used dissolution of Chalk in 10% hydrochloric acid to separate the silica and clay content from chalks. The research described here used 5% acetic acid to dissolve the calcium carbonate. An abridged experimental procedure follows:

Experimental procedure:

i) Chalk sample was powdered in a pestle and motar.

ii) The sample was oven dried at 105°C for 24 hours and again for another 24 hours until equilibrium of weight.

iii) 140g of chalk powder was dissolved with 5% acetic acid (percentage by volume v/v) until no further reaction was seen. This process took 2 to 3 days.

iv) After 4 to 5 days it was deemed that no further reaction was likely and the acidity was checked (as acid) by litmus paper. The liquid remaining now contained the non-calcium carbonate material in suspension. A centrifuge was then used to separate the liquid and solid fractions. Earlier attempts to separate the fractions using drying only, proved unsuccessful as drying also caused acetic acetone salts to come out of solution, so altering the calculation of solid mass.

v) The solid fraction was washed with distilled water and again centrifuged several times, until clean of acetic acid, and air dried on a 30 °C hot plate in a watch glass to avoid baking. The solid fraction was weighed and calculated as a percentage of the original mass.

3.5 Particle size distribution (PSD)

Analysis of particle size distribution for soils is traditionally carried out in accordance with BS1377-2, clause 9 (1990). The method describes a process of either wet or dry sieving followed by analysis of the sub 63µm size using a pipette or hydrometer method. All procedures prove lengthy and involve the shaking, mixing or disaggregation of material into constituent sizes. For clay and silt soils the process has proven accurate, but caution should be exercised when reviewing chalk soils because of the readiness in which Chalk breaks down into putties. Particle evolution (the process whereby a soil disintegrates to an ever smaller grain size whenever worked) can be reduced by conducting PSD analysis in a laser granulometer. In this study the laser granulometer used was a Malvern Mastersizer 2000 which was capable of testing material with a grain size below 2mm.

Testing procedure was carried out in line with the manufacturer's recommendation and with reference to BS ISO 13320:2009 and BS ISO 14887:2000. The chalk putties tested in the Mastersizer underwent dispersion prior to testing in a solution of sodium hexametaphosphate prepared in accordance with section 9.4.3.2, BS1377-2:1990. This solution was then diluted with de-ionised water to the concentrations that would have been present had the hydrometer method (section 9.5, BS 1377-2:1990) been followed. There is some discussion on the use and suitability of dispersants with chalk materials in Appendix Three of this thesis. Appendix Three also discusses how data from laser analysis may need to be interpreted slightly differently than that obtained when testing silt and clay materials using traditional particle size techniques.

3.6 Atterberg limits and Linear shrinkage tests

Atterberg tests were conducted in accordance with BS 1377-2:1990. The cone penetrometer definitive method (section 4.3 of this standard) was used to determine the liquid limit. The method was repeated five times (section 4.3.3.10, BS 1377-2:1990) for improved accuracy.

The linear shrinkage test was conducted in linear shrinkage moulds (section 6.5, BS 1377-2:1990) on chalk putties prepared at their individual liquid limits.

After preparation, the shrinkage moulds were air dried, then oven dried below 65°C and then at 104°C according to section 6.5.4.5. Contrary to BS 1377-2 the average of two tests per sample were taken and recorded to the first decimal place.

The apparatus outlined in BS1377-2:1990 (cone penetrometer) was further used as an indicator of strength gain; the greater the penetration of the falling cone, the weaker the material. A chalk putty of Newhaven parentage was prepared using material milled for 2 minutes. 15 cone penetration pots were made at the same time and sealed to limit further moisture loss. The moisture and penetration of the sample in each pot was then recorded after increasing periods up to 3 months from the time of preparation.

3.7 Shear strength tests

3.7.1 Shear strength analysis using the conventional shear box

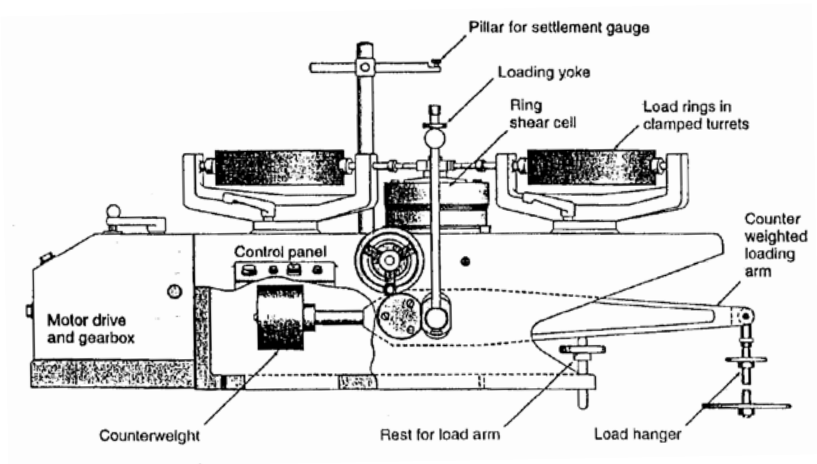
The conventional shear box was used to review the effect of the changing strength of chalk putties with time, using samples of 60mm x 60mm cross sectional area and height approximately 25mm (drainage path of 25/2 mm). The procedure outlined in BS1377-7:1990 clause 4, was used for a series of tests on Newhaven Chalk which had been ground for 2 minutes in the Tema mill and then aged for 3, 25, and 50 days prior to first testing. After preparation, the putty was placed, whilst at its plastic limit, into shear box cutting shoes in accordance with the placement of a remoulded soil section 4.4.3.5. The procedure primarily involves tamping the putty evenly into the cutter in three approximately equal layers. The cutting shoes and soil were then sealed to reduce moisture loss. After the requisite period of time had elapsed, the samples were extruded into the shear box test apparatus. The same test machine was used on all tests to reduce possible machine effects between tests. Results for both initial shear (pseudo peak) and residual shear after multiple reversals (BS1377-7:1990 section 4.5.5), were obtained for one loading stage of 100kPa normal stress. The test was considered as a determination of effective stress, which is equivalent to total stress when the machine speed was set in accordance with that calculated in Section 4.5.2 using T_{100} determined from consolidation plots. The pore water pressures developed in shearing are assumed to be zero when the test speed is sufficiently slow.

3.7.2 Shear strength analysis using the ring shear apparatus

The equipment and a test procedure for the ring shear test are specified in the BS 1377-7:1990 clause 6 and shown in Figure 3.2a-d. The equipment was designed initially to allow test samples to be subjected to large strains. The equipment is able to accommodate large deformations because the sample material, at its plastic limit, is placed in an annular mould and may be subjected to *significant* rotation. This annular displacement gives much large strains than can be achieved in either the conventional ring shear or standard triaxial cell. Harris and Watson (1997) argued that the larger strains enable a more realistic residual value to be obtained as changes in soil sample fabric occur around the formation of a shear surface or shear zone during the test itself. In standard triaxial or conventional shear box tests only pre-formed shear surfaces can sensibly be tested to give residual values of friction angle. For triaxial tests this requires the creation of an artificial shear plane, careful alignment and sample installation. In the conventional shear box alignment is so difficult that a method of multiple reversals (BS 1377-7:1990 section 4.5.5) may alternatively be used. Neither method replicates unidirectional residual conditions as successfully as the ring shear apparatus.

Developments of the apparatus (Figure 3.2) by Bishop et al. (1971) and Bromhead (1979) lead to the use of only a thin sample of remoulded soil. This has the advantage in this study, of any ageing effects due to raised pore water pressures during sample preparation being quickly lost. Harris and Watson (1997) suggest that the test drainage path maybe as little as 1mm, as the shear surface often forms within 1mm of the upper platen. The work of Harris and Watson (1997), also details an alternative ring shear testing procedure to that outlined in BS 1377-7:1990 clause 6. This ‘optimal procedure’, as adopted in this study, is now considered an industry standard and the preferred procedure used in the development of the Bromhead ring shear apparatus. The procedure differs from the British Standard primarily in its consideration of the consolidation of the test sample. Because the drainage paths are typically 1mm, and sample thicknesses generally are small, BS 1377-7:1990 presents an over estimation of the T_{100} (time to 100% primary consolidation) used to calculate the shear rate, BS 1377-7:1990 section 6.4.2.2.

In these tests the ‘optimal procedure’ recommended that a shear rate of 0.048 degrees / minute was used. After each load stage the torque readings were viewed in accordance with the ‘optimal procedure’ and confirmed as remaining constant. A drop in the torque values would have indicated that shear rate was too fast and that the test should be repeated. Nine load stages were conducted on each chalk putty sample tested, in what is effectively a multistage test. In accordance with the ‘optimal procedure’ it was not considered necessary to allow for a consolidation stage prior to each shear stage. This was further supported by the findings of Section 4.5.2, of a near instantaneous primary consolidation for chalk putty samples.



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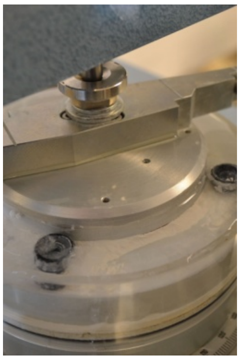

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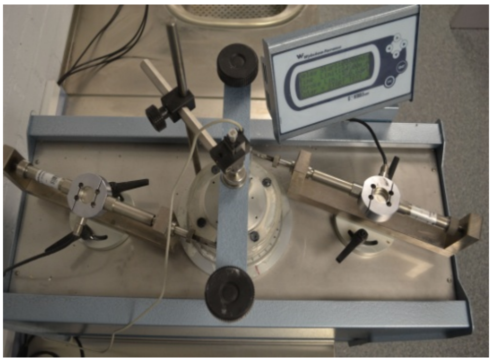

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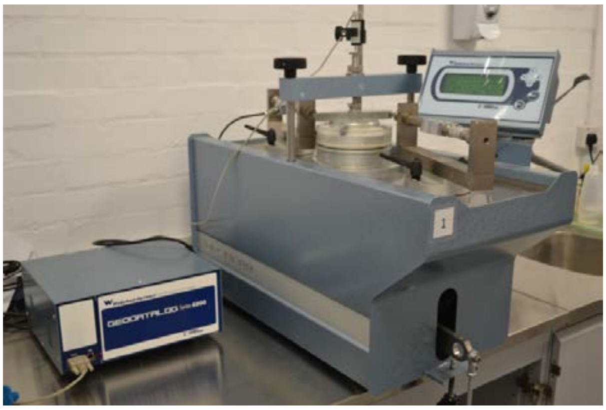


d)

Figure 3.2 The Wykeham Farrance 'Torshear' ring shear apparatus used in the research. a) Schematic diagram of the ring shear apparatus b) Sample cell showing chalk putty extrusion, c) Plan view of shear measuring transducers and sample cell d) General overview.

3.8 Triaxial shear strength testing

3.8.1 'Quick', unconsolidated, undrained partially, saturated triaxial tests.

Prior to the advanced triaxial testing (3.8.2), 'quick' (unconsolidated, undrained) triaxial tests were conducted on Culver and Newhaven putties, reconstituted from material milled for 2 minutes. The procedure adopted followed that outlined in BS1377-7 clause 8 (1990), with 38mm diameter samples tested under partially saturated conditions. As with the ring shear tests, samples were reconstituted to their plastic limits with moisture content checked after testing to confirm. Specimens were formed at the start of each test using the procedure outlined in BS1377-7 section 8.3.3.1c (1990), whereby material reconstituted from milled material was compacted into three layers into a split former using a tamping rod. This procedure complies with BS1377-1 7.7.3 criteria b (1990) in creating samples of the same dry density. A minor departure from the British Standards was that the rubber membrane (used to isolate the sample from the cell waters) was placed inside the split former *prior* to the material being placed. This is contrary to BS 1377-7 section 8.4.1.4 (1990).

Having conducted two sets of 'quick' triaxial tests, in which the sample was prepared and then immediately tested, an additional set of tests was conducted on Newhaven putty. These tests used samples that were subjected to the test confining pressure for a 24hr period prior to the shearing stage. The 24 hour period was included to facilitate the dispersion of differential pore pressures, which may have formed during sample preparation.

All of the 'quick' triaxial tests were considered as unconsolidated and undrained tests, despite the partially saturated nature of samples enabling some internal drainage. This internal drainage would have caused both drained and consolidation conditions to develop, although by definition the tests remain unconsolidated and undrained.

3.8.2 Advanced triaxial testing.

Advanced triaxial testing took the form of isotropic, consolidated, drained shear tests, with sample volume change and pore water pressure monitoring under stress controlled compression.

3.8.2.1 Equipment overview

Although BS1377-8:1990 describes the equipment and procedure for soils testing in triaxial cells, the apparatus used in this study is sufficiently different to merit a more detailed description. Drained shear tests were conducted in an

Imperial College, London stress path cell, controlled by 'Triax' computer software. The equipment is schematically shown in Figure 3.3.

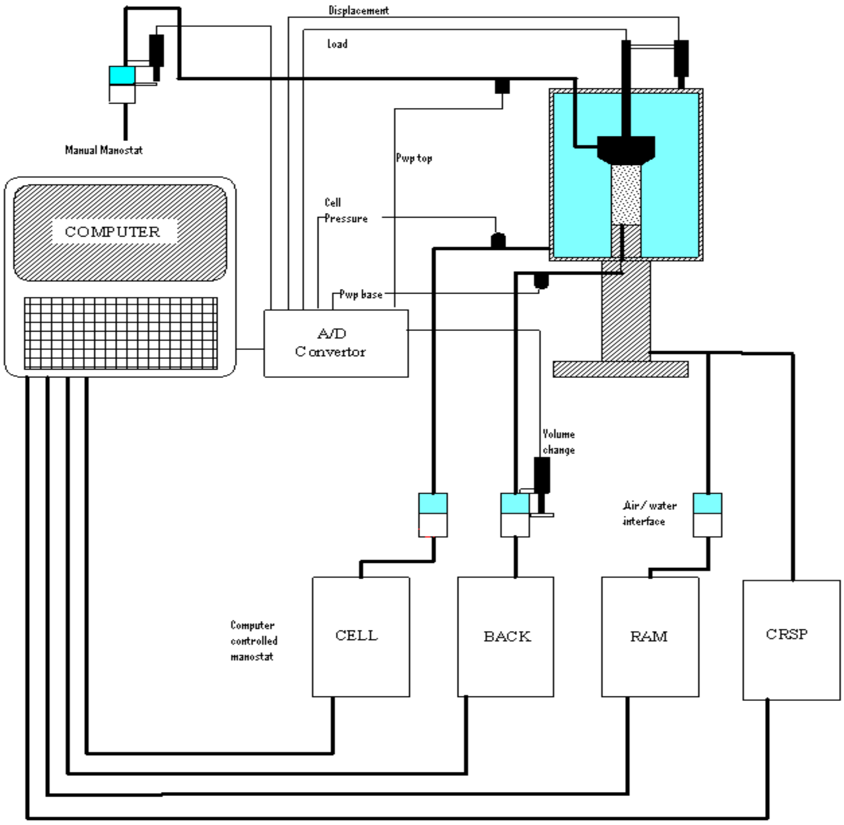


Figure 3.3 Schematic diagram of stress path cell equipment used. Modified diagram from Toll (2002).

3.8.2.2 System hardware

The system hardware comprises a test cell, computer controlled air manostats, air water interfaces, sample state sensors (volume change, pore pressure and vertical displacement), valve board and associated cabling and logging equipment.

The test cell was manufactured following the design of the hydraulic triaxial cell (Figure 3.4a-b) developed by Bishop and Wesley (1975). The vertical deformation load is applied by a hydraulic ram which raises or lowers the lower sample platen, using a system of two rolling bellofram diaphragms by means of changes in water pressures below the lower diaphragm. There is some mechanical advantage as the vertical pressures acting on the lower bellofram diaphragm are increased on the sample by the ratio of the area of the lower bellofram diaphragm divided by the area of the sample platen. The water pressure in the lower chamber is created by changes in air pressure entering the connected air / water interface. The air / water interface consists of a rubber membrane dividing a pressure cylinder, such that changes of air pressure on one side are transmitted to changes in water pressure on the other.

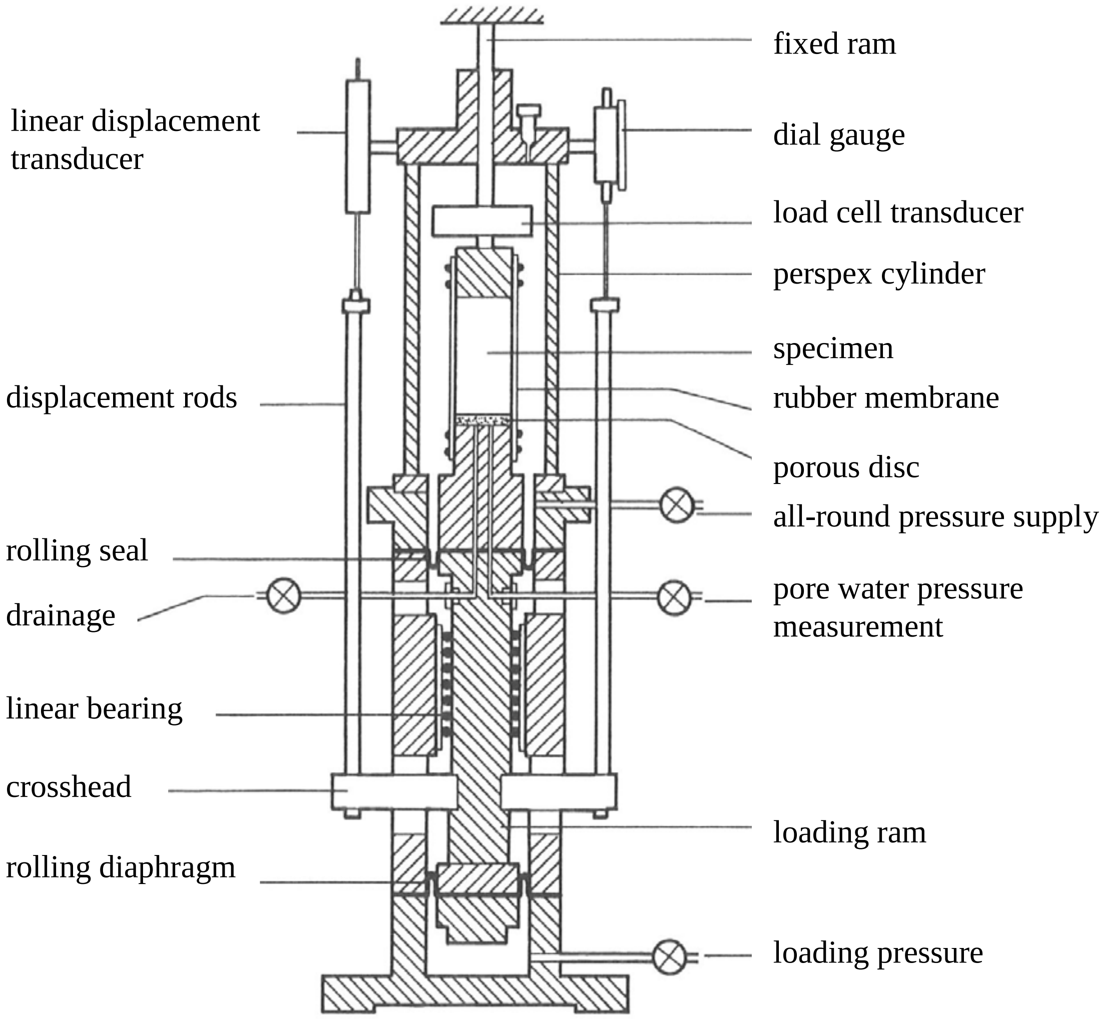


Figure 3.4a) Schematic diagram of hydraulic triaxial cell developed by Bishop and Wesley (1975) after Craig (2004). Top drainage and suction cap apparatus are not shown (see Figure 3.7).

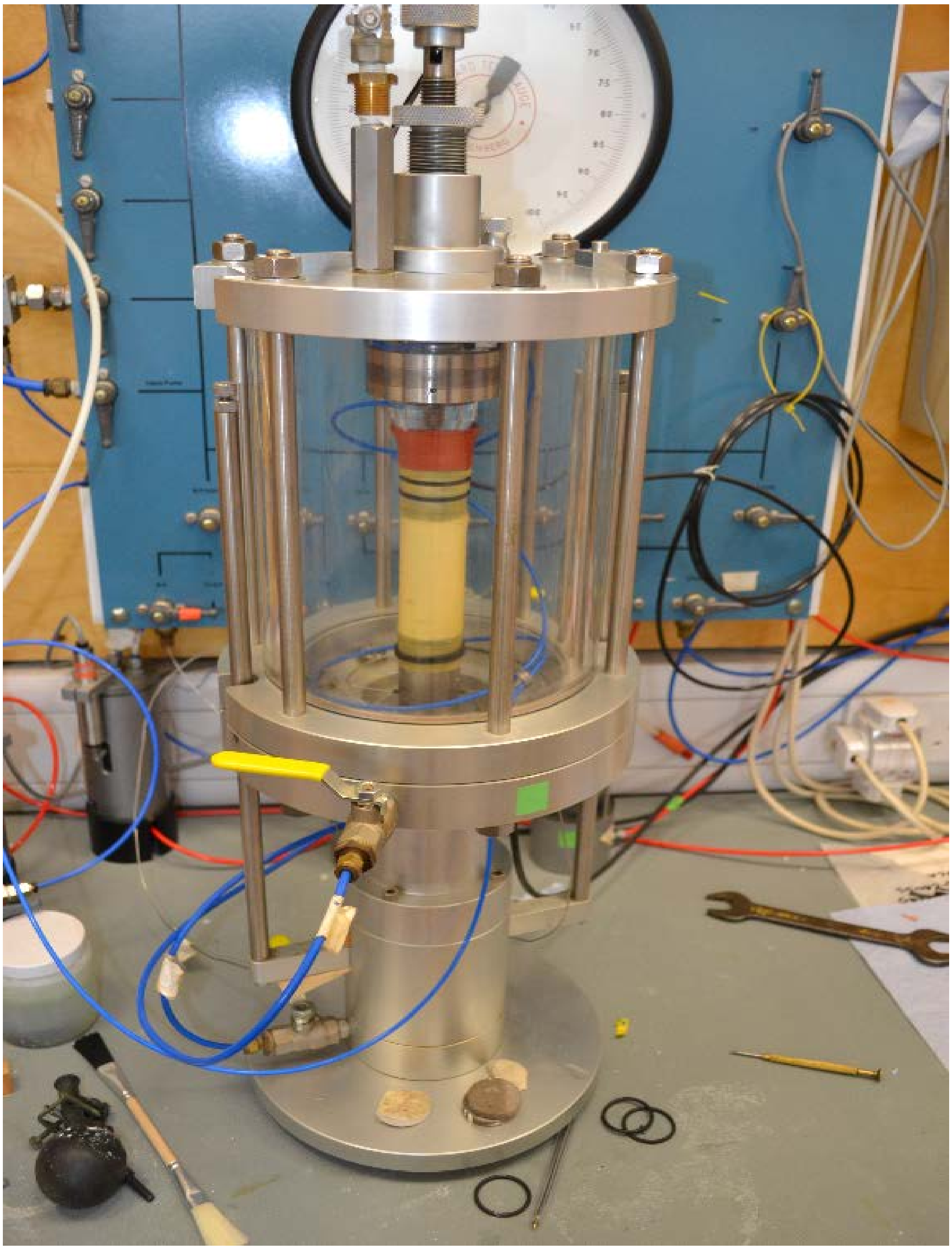


Figure 3.4b The London Imperial Stress Path (IC) Cell used in this research

Three computer controlled air manostats control the cell pressure, back pressure and ram pressure. A fourth manually operated manostat was incorporated to apply a separate pore pressure to the sample top cap, so enabling hydraulic gradients to be established in the sample for permeability studies (discussed in Section 3.8.2.5.7) Each manostat had a maximum input pressure of 1000kPa and output pressure of 850kPa. All were continuous bleed so that pressure could also be reduced down line. The air supply was at a continuous 1000kPa working pressure, dried and cleaned in a refrigerant dryer so that manostat bleed glands did not become blocked.

Pressures achievable in the cell are more limited than in a conventional triaxial cell. The hydraulic cell is designed so that the ram travel is resisted by both cell pressure and the test sample. If, for example, a sample is subjected to a back pressure of 400kPa and is consolidated to an effective stress of 300kPa then a cell pressure of 700kPa is required. As a result the ram manstat must reach 700kPa and above before it begins to apply a force on the sample. For this reason there is also the option to use a constant rate of strain pump (CRSP). This device uses a stepper motor to directly actuate a piston so creating hydraulic pressures, without the need for an air supply. Like the air manostats, Triax is able to operate CRSP under stress control. Maximum pressures of 2MPa are achievable in the lower chamber of the cell.

All sample measuring devices were external of the cell. Volume change units were fitted to the pore pressure lines at the top and base of the sample. Figure 3.5 illustrates a volume change unit which measure water flow into or out of a sample during testing.

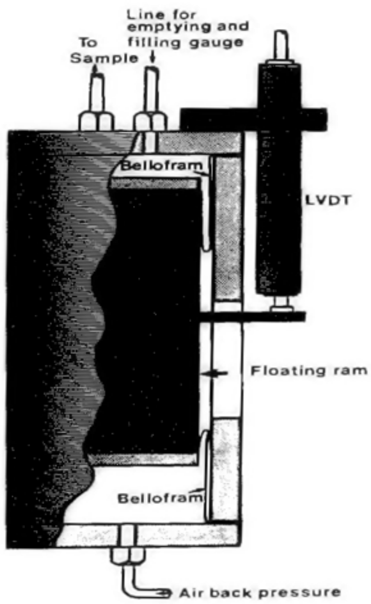


Figure 3.5 Imperial College 50cc volume change unit. Leddra (1989).

3.8.2.3 System Software

The computer control of the pressure systems was achieved by Triax Version 5.1 software developed at Durham University (Toll, 2002). An input / output loop set up exists so that any control outputs are monitored by cell pressure transducer, back / top cap pressure transducers and load cell. Feedback information from these transducers is in turn processed so that pre-assigned control output profiles can be preformed. The return loop is augmented with a

MSL (Measuring System Ltd) 16 channel Datascan 7020. This enables channels to be specified at 100mV range for devices such as load cell, pressure transducer and 10V range for devices such as LVDTs. Communication through the computer is via a serial com port with a matching handshake.

Data are recorded in a comma separated file, with the time period between scanning of system variables selected by the operator. Variables denote the physical conditions within the cell or of the sample itself. Variables may be those recorded from transducers, those calculated or those inputted at the start of the test by the operator. An example of a recorded variable would be variable 5 "cell", recorded using the cell pressure transducer. Calculated variables are derived from a variable equation again inputted by the operator. Variable 20 "rstrain" denoting sample radial strain would represent a calculated variable using the equation (d_0 – diameter)/d_0*100 where diameter is again a calculated variable 19 and d_0 is a user input variable denoting initial sample diameter. System variables are held in the Triax "ini" file described further in Apppendix Five.

The output side of the loop is through the computer control of the Imperial College (IC) controllers. Each controller consists of a Watson Smith manostat regulated using a stepper motor. Stepper motor control is via a PCI 836 card (Peripheral Component Interconnect*) installed on one of the computer bus slots and wire linked to a component board inside the IC controller. It is important that the digital I/O card* (input / output card) is matched to the component board with the correct resistor configuration as earlier 8255 I/O cards are common, and use a different IC controller component board. The computer is equipped with the necessary PCI hardware driver. The PCI card option as used here is compliant with Windows 2000.

*(Please see glossary)

The Triax software enables fine control of the stress parameters via the stepper motor controlled manostats. Each step is equivalent to a 0.07kPa pressure change. The program enables the user to denote the maximum number of steps that can be taken in one adjustment on each controller. Choice of steps taken was made after some initial test runs indicated the maximum that could be made without pressure overshooting or hunting

Triax Version 5.1 used in this research, gives the operator the choice of using pre-configured test sequences, eg. saturation stage, consolidation stage or stages created using the operators own control equations. A control equation would be a formula that governs the response of the system controllers in a pre-defined manner. Stages 12-21 (except 19) were created for the purposes of testing chalk putty (Appendix Five). Alarm equations are created when defined criterion have been met to enable full automation. Should, for example, a sample reach a strain of 20%, an alarm equation may be set to stop the stage and enable a different stage to begin.

3.8.2.4 Calibration of Imperial College Stress Path System

Back pressure, cell pressure and top pressure transducers were calibrated in situ and in unison using the Budenberg pressure gauge mounted in the valve panel (background, Figure 3.4b), using the calibration option in the Triax software. The maximum percentage error from the calculated best fit line was 0.96 % (equivalent to 6.2kPa), although values typically had an error of 0.20%.

Volume gauge units were calibrated by filling the gauges (air supply off) and measuring the expelled water (air supply on) against the transducer voltages. To enable flow output to be slowed to a recordable rate an outlet pipe was fitted with a temporary end valve so that flow rate was reduced. Water was expelled because of the residual air supply from manostats were set to their lowest value. This calibration technique produced worst case percentage errors of less than 1.67% (equivalent to 0.75cc), although these were typically 0.5% from the best fit line through the calibration data set. Top and bottom sample lines were narrow gauge and zero volume change. Water used in the lines was de-aired and distilled before use in the system to reduce risk of air pockets.

The submersible load cell (Applied Measurement Stalc3, 5kN) was calibrated whilst removed from the cell, in a known traceable calibrated loading frame, (BS EN ISO 7500-1:2004) using the Triax calibration option. During data collection, the converted output (Toll, 1992) was seen to fluctuate by +/- 2 N. This necessitated high tolerances of 1.5 in Triax's 'stage' set up to avoid control oscillations, the process whereby a controller continually increases then decreases to achieve a target value. The readings from the internal load cell are independent of cell pressure.

3.8.2.5 Test procedure

Samples where put through the following stages; preparation, placement in the cell, sample flushing, saturation, degree of saturation check, consolidation, pre-shear permeability, drained shear and post shear permeability. Reference is made to Sections 3.8.1 and 4.7 which highlight general difficulties in chalk putty triaxial testing.

3.8.2.5.1 Sample Preparation

Samples were prepared using the apparatus shown in Figure 3.6. The purpose of the preparation technique was to produce cylindrical samples of known size and density which could then be hydrated to a fully saturated condition under a minimal known effective stress (i.e. a true un-aged condition). Two alternative techniques were reviewed prior to adopting this technique and they are explained more fully in Appendix One. Barton and Brookes (1989) developed a ‘shaker’ method to produce sand samples of a specific density; sand was pluviated slowly into a water filled sample preparation tube subjected to lateral shaking. Razoaki (2000) developed the ‘consolidatometer’, a simple device were pre reconstituted chalk putty was placed into a cylinder lined with a standard triaxial testing membrane and was then subjected to vertical loading by weights to give a reformed sample of known consolidation history. A discussion of why neither method was adopted can be found in Section 5.4.1.

The ‘dry press’ technique used in this work proceeds as follow:-

Intact Chalk was milled as in Section 3.1 to create a dry powder of the required milling time.

A ‘suction’ split former was manufactured by taping a quick release pipe fitting (capable of taking a 4.7mm outside diameter pipe) at the mid height position of one half of a conventional triaxial former. The two halves of the former were then secured together with a small amount of non-setting gasket compound to improve air tightness between the two halves. A standard triaxial membrane was placed inside the split former along with three strips of filter paper 10mm wide by 76mm long. The filter paper was positioned between the membrane and former to improve a uniform suction of the membrane to the sides of the former once a vacuum pump had been connected to the pipe taping.

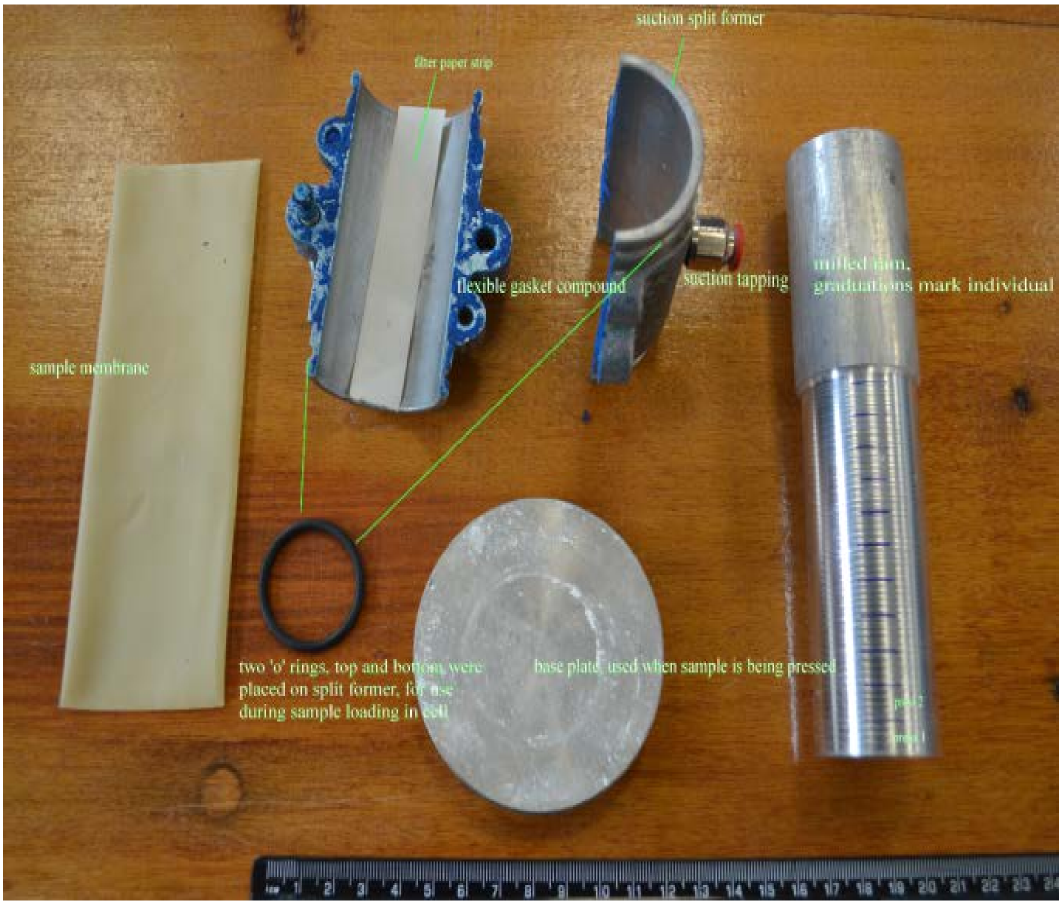


Figure 3.6 The ‘dry press’ apparatus

With a vacuum applied to the suction former, the apparatus was placed in a load frame. For samples of the dry density of Culver Chalk, 15g of milled chalk was placed into the former and pressed to 1/10th of the final sample length of 76mm using a machined plunger. The plunger was then removed and the surface of the compacted material scarified using a screwdriver. The next press was then preformed in the same way and so on until 10 layers gave a sample of the required 76mm length and 38mm diameter. The vacuum pump was then removed and the prepared sample and vacuum former taken to the stress path cell for loading.

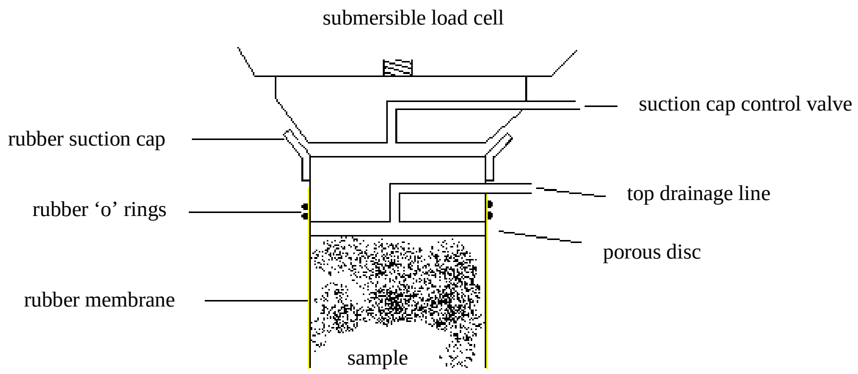
By keeping the sample in the split former, the membrane could be rolled over the base pedestal and top cap without sample disturbance. Two ‘0’ rings were then rolled off each end of the former to seal the sample within the membrane.

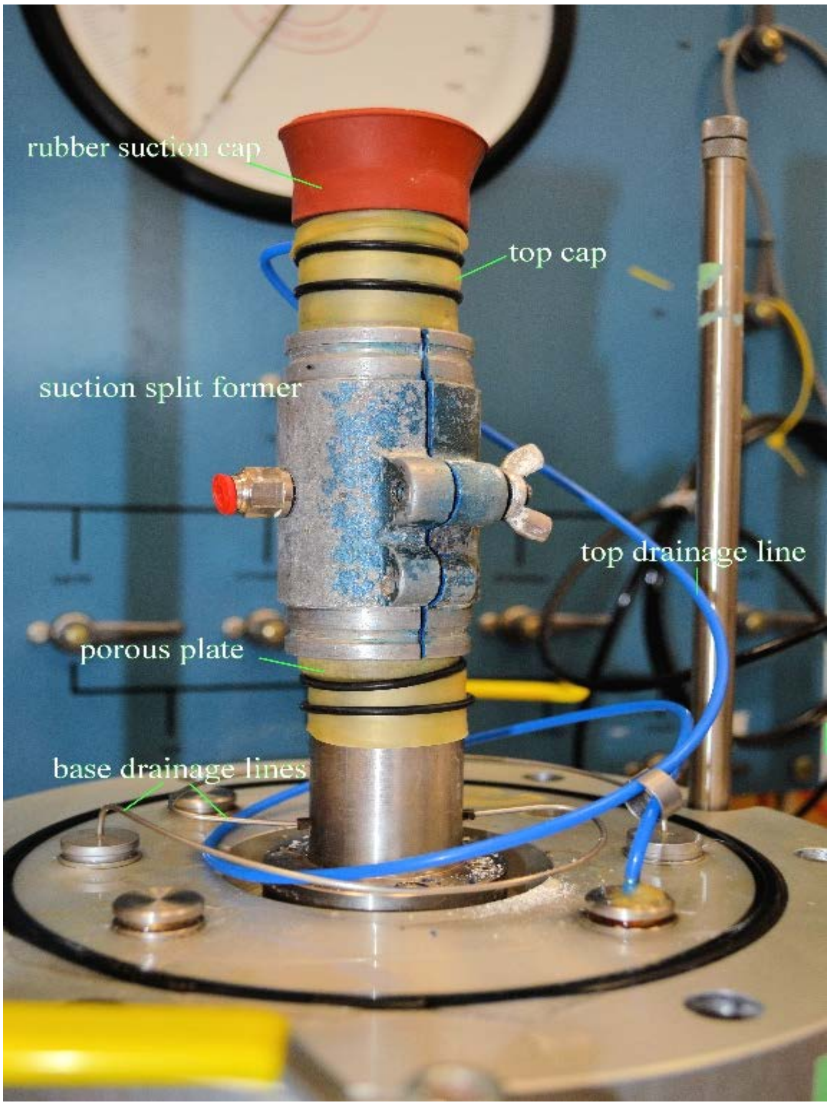
A suction cap was used during the experiments to aid sample integrity during set up. The suction cap was originally designed to allow extension tests to be conducted within the hydraulic cell. Situated over the top end of the top cap

(Figure 3.7a-b), it enables the top cap to be linked to the load cell once the sample is submersed in the triaxial cell. Atkinson and Evans (1985) review of Jardine et al. (1984) comments on the advantages of using a suction cap to eliminate seating and tilting between the load cell and top platen at the very start of a test before the initial stresses due to saturation and consolidation are applied. Conventionally, the top cap would be brought into contact with the load cell after saturation and consolidation. Both of these stages however would make the sample stiffer and more difficult to seat.

Earlier ‘quick’ triaxial tests (Section 3.8.1) found that the seating of chalk putty samples (prior to the shear stage) to be problematic (see Section 4.7). Chalk putty under no effective stress in a fully saturated state (the purpose of the saturation stage) has an effective strength of zero (Clayton, 1977), and is unable to maintain the form of right cylinders prior to shear. Further, Burland et al. (1983) describes the tendency of chalk putty to liquefy during piling and suggests handling could easily cause deformation. Use of the suction cap was seen to improve sample integrity and avoid sample collapse or deformation under self-weight.

a)





b)

Figure 3.7 a) Arrangement of suction cap, b) Sample placement on base pedestal of stress path cell using suction split former.

n.b. With care, the split former could be prised apart to leave a 'right' cylinder of dry pressed ground chalk of known volume and mass.

3.8.2.5.2 Flushing chalk putty

Distilled, de-aired water was flushed through the samples from base to top cap so as to reduce the volume of air in pores. With clay / silt samples it is common to pass carbon dioxide through the sample prior to flushing with water. Carbon dioxide displaces air and then is more readily absorbed in the flushing water. Carbon dioxide, however, also creates acidic conditions with water on dissolution and it was decided that use of carbon dioxide with calcium carbonate samples was inappropriate because of possible chemical reaction.

Sample flushing was achieved by subjecting the reformed specimen to an upward flush of de-aired water from a header tank with its base at an approximate height of 1.5m above the sample. Typically pressures of 17kPa were achieved at the base inlet with the top cap exposed to a negative pressure (i.e. a vacuum) of -12kPa. The negative pressure was achieved using a Bernoli tap fitted vacuum device.

To protect both base and top volume gauges from negative pressures, bypass valves were fitted to isolate the gauges, as under suction the bellofram diaphragms (shown Figure 3.5) could invert and suffer damage (personal communication, S. Ackerley, 4/2012). At the end of the flushing stage, negative pressures were allowed to dissipate before top drainage valves were closed so that the mean effective stress (33-((17 – 12)/2 kPa) would migrate to (33-17kPa) over approximately 2 hours.

Computer control was not used during the transition from flushing to saturation stage as a number of premature failures occurred in initial tests due to ram pressure and cell pressure controllers struggling to adjust to an effective stress based on a sample pore pressure (within the control equation) which varied not only in respect to time and valve closures but was also differential along the length of the sample. For example, to reduce the impact of closing the base drain valve from the header tank and opening the back pressure from the pressure controller (to enable the saturation stage to proceed) it was necessary to first preset the top line to 17kPa after allowing the earlier suction to dissipate before switching the base lines once the sample was uniformly at 17kPa. Any deviation of the base back pressure from 17kPa was adjusted manually on Triax before the top line was again closed for the saturation stage. In this fashion rapid changes in mean effective stress were avoided.

To maintain sample integrity at the flushing stage, a cell pressure of 33kPa was applied with the variable load tracked at zero to give a general confinement of 33kPa and an average effective stress of 30.5kPa.

Once a volume of greater than 87000 mm^3 (i.e. the initial sample volume) was recorded passing from the top line, the flushing stage was considered complete. The volume was chosen arbitrarily, but was significantly above the air void percentage as suggested by porosity calculation. An intermediary jar to which the top line suction was applied was used to collect the flushed water before measurement. Two to three days was the typical time required to achieve successful flushing, without significant damage to sample through piping, or loss of integrity.

3.8.2.5.3 Saturation

Sample saturation was achieved by increments of cell pressure and back pressure. BS 1377-8: section 5.3 (1990) describes an equivalent manual method in a conventional triaxial cell. Effective cell pressure was set to a hold value of 15kPa. This effective stress was sufficiently high during the saturation stage to avoid hunting and oscillation of pressures and to maintain the effective stress status quo from the sample flushing stage. Ramping of back pressure at a rate of 0.2kPa/min was found to be satisfactory. The initial sample dimensions of h_o and d_o were entered into the Triax program at the start of testing. No volume change was assumed for the flush stage as the volume gauges could not be connected during flushing for reasons explained in Section 3.8.2.5.2.

3.8.2.5.4 Verification of saturation using Skempton's B test

A series of 'B' value tests were carried out to ascertain what back pressures were needed to achieve acceptable saturation values. It is generally regarded that Skempton's pore water coefficient should be greater or equal to 0.95 (BS 1377-8:1990 section 5.3.2.d). Difficulties in obtaining a value of 0.95 are explained and discussed in 4.8.2. After experimentation, pressures of between 400kPa- 450kPa were considered sufficient.

3.8.2.5.5 Required effective stress by consolidation

The cell pressure was increased at a controlled rate of 0.2kPa/min whilst the back pressure was held constant at 400kPa. The ram controller was set to track zero so that the sample was held without vertical force between top and bottom platens. In this manner the sample height could be recorded.

3.8.2.5.6 Drained shear

In this test stage, pore pressure is kept at a constant value 400kPa whilst deviator stress (q) on the sample is increased. In conventional triaxial tests q is increased by using a controlled rate of strain as the gap between sample top and base is closed. In this study stress control, not strain control, was used. Stress control is considered a more accurate representation of field conditions. In most field cases, failure of a soil occurs as dynamic equilibrium is lost on an increase of stress, not strain.

During the drained condition, it is important to avoid rising pore pressure during the test stage. As well as controlling back pressure, the tests were conducted slowly to help maintain the drained condition. BS 1377- 8 section 6.3.8, (1990), gives a suggested method for calculation of rate of axial displacement during drained shear, using equation 3.1.

$$d_r = \frac{\varepsilon f \ . \ Lc}{tf} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \text{equation 3.1}$$

Where:-

d_r = axial displacement rate in mm/min
ε_f = is the significant strain interval for the test specimen
t_f = is the significant testing time in minutes
L_c = is the length of the consolidated specimen in mm

It should be noted that whilst BS 1377, part 8 gives tables for calculating t_f for plastic non-sensitive soils, the author is wary of including reformed Chalk within this group. Furthermore, ε_f is an estimated value.

No literature was found on the recommended rates of increase of deviator stress for stress controlled drained shear tests on chalk putty. Observations of sample pore pressure, however, during this test stage found that a rate q = 0.2kPa/min caused no pore pressure increases until the point of failure. This was in accordance with British Standard recommendations that pore pressure readings should show that greater than 95% of excess pore pressure was dissipated during each elevation of deviator stress. The decision to use a rate of 0.2kPa/min was further supported by Ng's (Ng, 2007) successful use of 0.5kPa/min stepped increases of deviator stress on more impermeable samples with permeability values of 10^{-9} m/s compared to those of 10^{-8} m/s (Section 4.8.3.2) of reformed chalk putty as tested at varying mean effective stresses (0 - 400kPa).

3.8.2.5.7 Pre and post shear permeability stage

Permeability calculations of chalk putty samples formed using the 'dry press' technique in the drained shear tests under different mean effective confinements were carried out in accordance with BS1377-6: clause 6 (1990), (determination of permeability in a triaxial cell). A permeability stage was included before and after shear. Two volume change units were used, both on the top and base lines to verify that the flow of water entering the sample equalled that leaving (BS1377-6: section 6.3.7 (1990)). A pressure difference of 30kPa (415- 385kPa) was used to create the required head which produced a flow from sample top to base, under a hydraulic gradient of 40. The coefficient of permeability (k_v m/s) in the vertical direction at 20°C was calculated using:-

$$k_v = \frac{1.63\, q\, L}{A[(p_1 - p_2) - p_c]} \times R_t \times 10^{-4} \ \text{(m/s)}$$

Where:-

q is the mean rate of flow of water through the soil specimen (in mL/min);

L is the length of the specimen (in mm);

$(p_1 - p_2)$ is the difference between the pressure applied to the top and base pressure lines (in kPa);

p_c is the pressure loss in the system (in kPa) for the rate of flow q, derived from graph Appendix Two, Figure A2-1;

Rt is the temperature correction factor for the viscosity of water.

Sample area was recorded post consolidation as used in the pre-shear permeability calculations and a post-consolidation sample diameter (d_0) was calculated and re-entered into the Triax software after the permeability stage. This was to avoid volume changes during the permeability stage being treated as 'true' by the Triax software. No volume change would occur if the volume change in both volume changes units remained equal. No d_0 adjustment was needed in the post shear permeability stage as the subsequent stage was 'dismantle' sample.

p_c was measured as 0 kPa at the flow rates achieved during the permeability stage, typically q = 0.02ml/min. Head loss due to the porous discs was calculated in accordance with BS1377-6:1990 section 6.4.2. The calibration graph is given in Appendix Two.

3.8.2.5.8 Treatment and corrections to triaxial data

i) Area correction

As triaxial samples deform during drained shear, radial strain occurs which increases the area of the sample during compression. Without correction of this area a significant error would be recorded in the vertical stress calculations. The software Triax counters this by calculating and applying an area correction (using the variable ‘area’) which is then used in subsequent calculations.

The ‘area’ variable is calculate as

$$\text{area} = \frac{\text{A0}\,(1-\text{vstrain})}{(1-\text{strain})}$$

where:-
A_0 = initial area of triaxial sample
vstrain = volumetric strain
strain = linear vertical strain

ii) Membrane resistance and correction

During consolidation and shearing the rubber membrane, in which the sample is placed to isolate it from the ingression of cell waters, contributes to the measures forces exerted on the sample (BS 1377-part 8 section 8.4.f.). As can be seen from Figure 3.8 (BS 1377-part 8, Figure 4) for a membrane of 0.2mm thickness around a sample of 38mm diameter, the effect of the membrane is limited to 2kPa at 20% strain. This was considered negligible and so a membrane correction was not routinely applied to graphs presented in the thesis illustrating continuous data acquisition, but was applied in the calculation of the consolidated drained failure envelope to enable a direct comparison with plane shear envelopes from ring shear apparatus to be made.

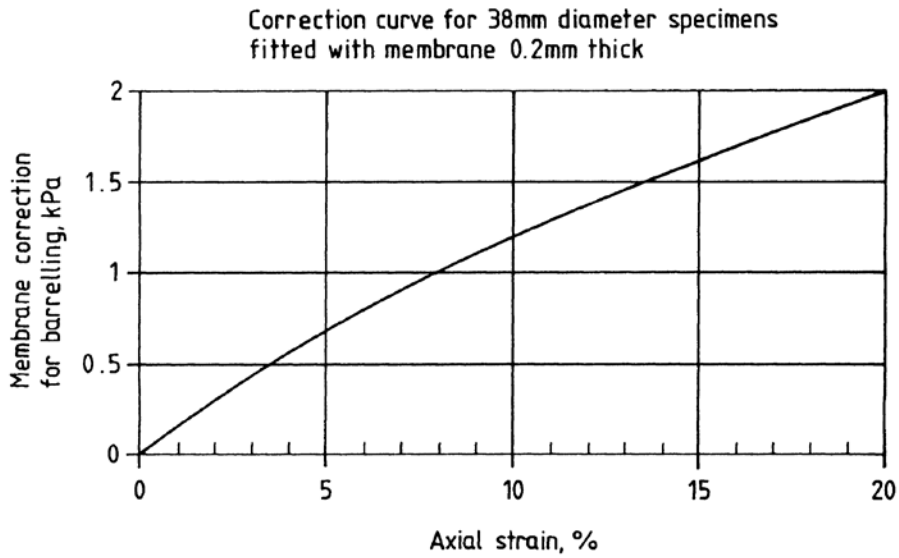


Figure 3.8 Graph showing membrane correction in consolidated drained shear triaxial tests

iii) Membrane penetration effects

This correction allows for the penetration of the membrane into the surface 'pits' in the sample as cell pressure is raised. As lateral forces cause the membrane to indent, a change in the recorded volume is seen. The volume reduction is dependent on the size of the 'pits'. The correction is commonly used in the testing of coarse granular material where 'pits' are represented by the mean particle size (D_{50}). This correction was not considered relevant for the reformed chalk samples which were later shown to have D_{50} in the range 1-10µm.

Chapter Four
Presentation of Data

4.0 Overview:-

4.1- 4.4 Results that characterise the chalk putties studied.

4.5- 4.7.2 Results necessary to develop a suitable chalk putty testing methodology.

4.7 - 4.8 Results that improve the knowledge of the mechanics of chalk putty.

4.9 Summary

4.1 Parent material of the chalk putties

4.1.1 Porosity and dry density

As described in Section 3.1, chalk putties were prepared from grinding intact Chalk from two locations, Culver Chalk from Longlands Quarry (SZ 62274 86394) and Newhaven Chalk from Portsdown Quarry (SU 63444 06662). The physical properties of the intact Chalks are shown in Table 4.1 along with those of a third Chalk of Middle Cenomanian age from Ballard Down (SZ 04044 81173). Each of the three Chalk types represents a separate density group as defined by Mortimore et al. (1990) and Mathews et al. (1993). Portsdown Chalk represents medium density Chalk (dry density 1.55-1.70 Mg/m^3), Culver Chalk high density (1.70-1.95 Mg/m^3) and Ballard Chalk very high density (above the threshold of 1.95$Mg/m^{3)}$. Selection of the Portsdown and Culver Chalks from an initial three Chalk members was primarily made on the basis of their porosity, (38% and 32% respectively). Both the Newhaven and Culver Chalks were considered to be the most likely Chalks to form putties if they were to be subjected to the processes described in Section 2.7.

Sample Location	Longlands Chalk Quarry, Culver	Portsdown Quarry, Paulsgrove	Ballard Down, Swanage
Map reference	SZ 62274 86394	SU 63444 06662	SZ 04044 81173
Stratigraphic Horizon †	Culver	Newhaven	Middle Cenomanian
Non calcium carbonate material (%)	1.39	0.66	3.27
Sample Porosity (%)	32.00	38.20	11.73
Structural Context	Isle of Wight Monocline	Portsdown Anticline*	Purbeck Monocline
Bedding dip angle (°)	65	10	80
Dry density (Mg/m^3)	1.78	1.65	2.37

† Mortimore (1986) South Downs Succession

* Portsdown anticline includes Newhaven, Tarrant and Portsdown Chalk members. Portsdown Quarry is situated in the Newhaven formation.

Culver and Newhaven porosity and dry density values were based on tests that exceeded ISRM Standards (Section 3.3). Values are an average of 4 samples with a mass approximately 3 times the minimum recommended.

Non calcium carbonate material tests were repeated once with 0.5% variation

Table 4.1 Parent intact Chalks reviewed in the study

4.1.2 Failure envelopes and crushing of intact Chalk parent material

Comparison of the compressive rock strengths (Mohr circle plots) for intact Chalk (Figure 4.1) shows that they correlate well with the porosities in Table 4.1. As expected, the strengths of the three Chalks are directly related to the porosity, the higher porosities resulting in weaker material. This is particularly marked between Ballard Down and Newhaven / Culver Chalks. From Figure 4.1 it is evident that initially all samples show an increase in strength with confinement (least notable in Ballard Down Chalk with the highest density). As confinement increases, it ultimately results in a reduction of strength. This can be explained by reference to the Hoek cell test procedure in Section 3.2. On the application of pressure in the Hoek cell, sample crushing and pore-collapse was observed above 15MPa for Ballard Down Chalk and 10MPa for Newhaven Chalk / Culver Chalks, causing a lowering of the failure envelopes.

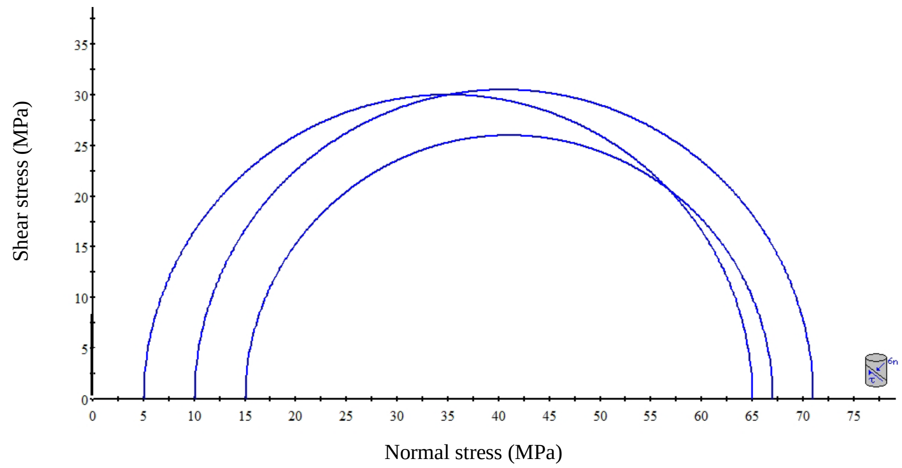
It should be noted that if the Chalks are considered in their structural context (as discussed in Section 2.2.3), the data partially supports the prediction by Jones et al. (1984) and observed by Miram (1975) that structural deformation and strength are related. Newhaven and Culver Chalk are the least structurally deformed and provide the study with the weakest material, whilst Ballard Down material is from a highly faulted exposure, with beds lying at 80° and provides a significantly stronger material.

4.2 Composition of the Chalk parent materials and subsequent putties

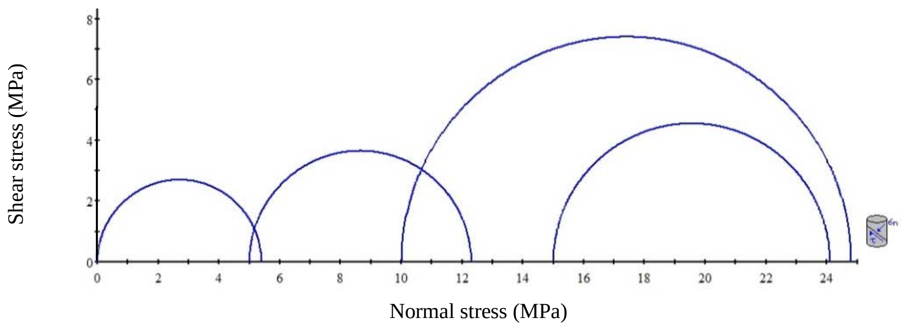
From Table 4.1 it is shown that all three Chalks are extremely pure calcium carbonate. Purity is seen to reduce slightly with age. The oldest, Middle Cenomanian (Lower Chalk from Ballard Down under the traditional Southern England Chalk subdivisions, Table 2.2) having the highest non calcium carbonate fraction at 3.27%. The remaining two Upper Chalks have only trace impurities.

The nature of the non calcareous fraction is investigated in Figure 4.2 using particle size analyses. In the laser diffractometer, Newhaven Chalk has silt size non calcite particles with an asymmetrical scatter around the 20µm medium/coarse silt grade*; Culver Chalk contains non calcareous impurities distributed asymmetrical around the 10µm medium silt grade* with a slight proliferation of the 1µm clay grade*; and Ballard Chalk is bi modal with peaks at the clay 0.2µm and silt 7µm grade*.

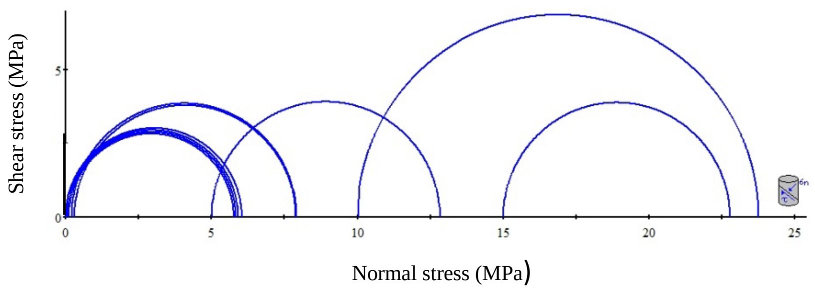
Extremely low impurities in the two Upper Chalks support their selection for the study as the influence of clay or silt contamination can be mitigated. Perry



a) Ballard Down Middle Cenomanian

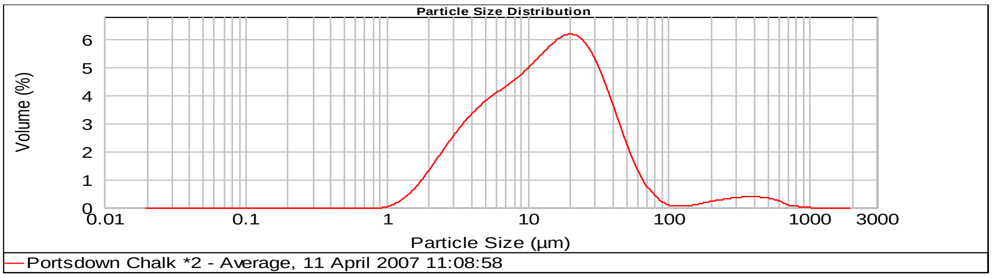


b) Longlands Quarry, Culver Chalk

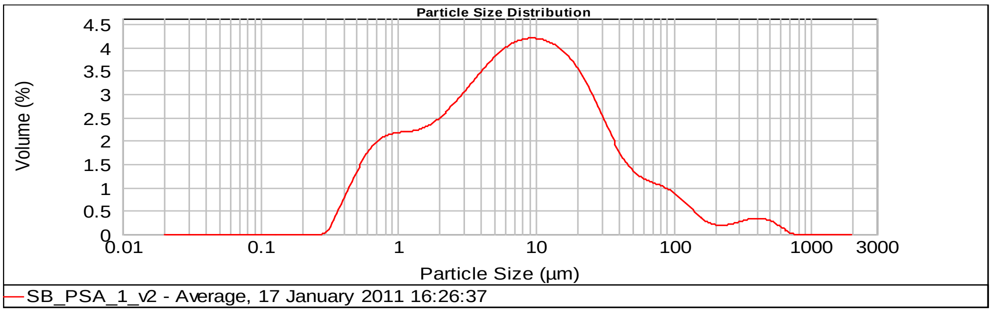


c) Portsdown Quarry, Newhaven Chalk

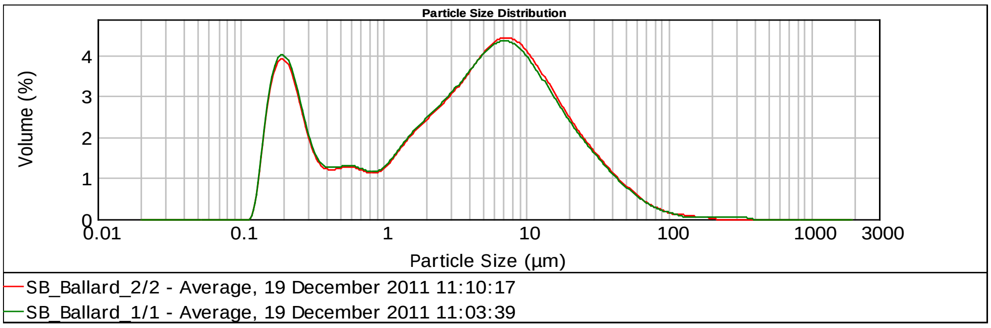
Figure 4.1 Uniaxial compression tests on intact Chalk



a) Portsdown Quarry, Newhaven Chalk



b) Longlands Quarry, Culver Chalk



c) Ballard Down, Middle Cenomanian Chalk

PSD curves are an average of 3 samplings. Variation in $d_{(50)}$ size between the 3 samplings were typically within 1% of the average value. To confirm repetition in sample preparation, repeat tests were conducted. This is demonstrated by the close alignment of the 2 PSD curves in Figure 4.2c.

Figure 4.2 Particle Size Distribution of the Non Calcareous Content of Chalks of the Study

(1979) has shown that chalk putties, when mixed with even small quantities of London clay, respond by a significant reduction in Californian Bearing Ratio (CBR). This suggested strength dependency with impurity content can be considered negligible with the Culver and Newhaven chalk putties. Consequently the Middle Cenomanian Chalk from Ballard Down, demonstrating the highest impurities, lowest porosity (Section 4.1.1) and limited putty formation (Section 4.1.2) is not reviewed further in this study.

The grading values as discussed in 4.2 and subsequently in the study are as recorded by the Mastersizer Malvern Instrument Laser Analyser (as discussed in Section 3.5). The interpretation of the grade is discussed more fully in Appendix Three.

4.3 Particle size distribution and structure of chalk putty

The particle size distributions of Culver and Newhaven Chalk are shown in Figures 4.3 and 4.4 respectively. In addition to the material ground in the Tema mill for 2 and 14 minute periods (the period at which many of the geotechnical tests were conducted), the results from other periods have also been included. This enables consideration of the evolution of particle size with grinding to be reviewed.

Alone, PSD analysis cannot fully define soil fabric;* structure is also important. To analyse the arrangement of the grains, use was made of a scanning electron microscope. The arrangement, size, shape, and frequency of the soil components and their relationship with voids are investigated in Appendix Four. The difficulties in assessing soil structure *(in terms of cluster theory) are highlighted in Appendix Four and a discussion of sample strengthening due to cluster formation, is discussed in 5.2.1.

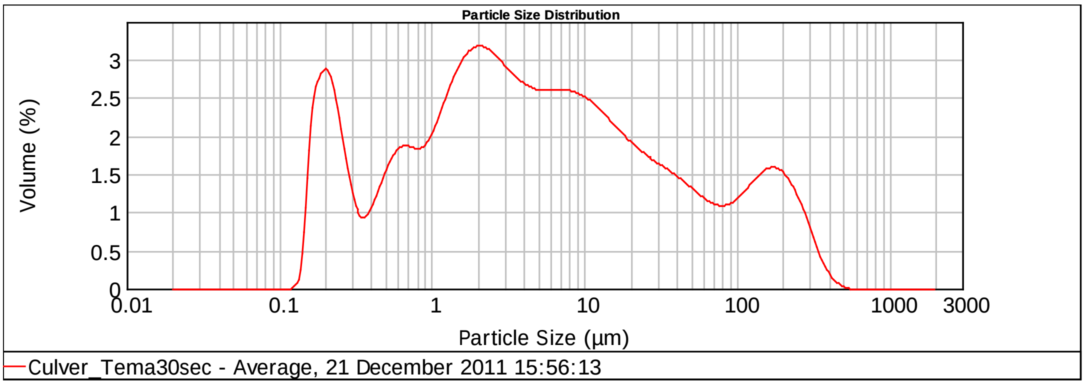
*See glossary

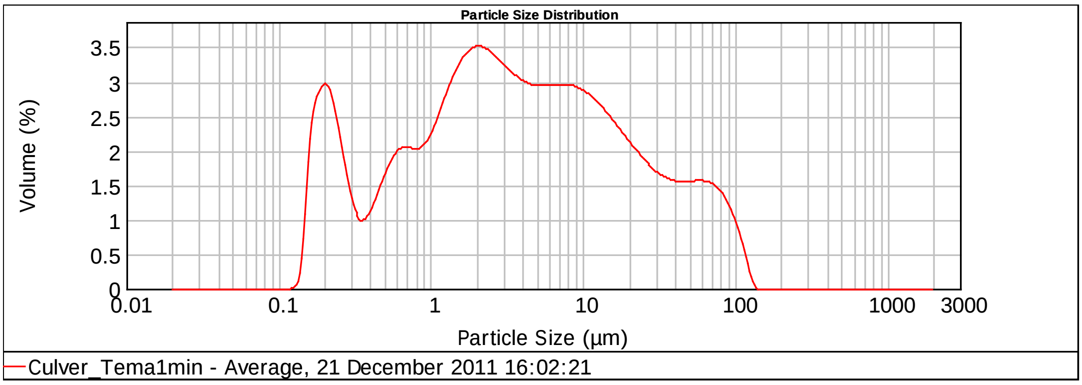
4.4 Geotechnical index properties of chalk putty

4.4.1 Atterberg values, linear shrinkage and thixotropy

Atterberg values, linear shrinkage and thixotropy were reviewed as a matter of course and are tabulated for Culver and Newhaven chalk in Figure 4.5. Data are presented for Culver and Newhaven chalk putty subjected to milling for 2 and 14 minute periods.

	d10	d50	d90	CLAY	CLAY	V. FINE SILT	FINE SILT	MED SILT	COURSE SILT	V. FINE SAND	FINE SAND	MED SAND	COURSE SAND	V. COURSE SAND
				0.02 - 0.1 µm	0.1 - 3.90 µm	3.90 - 7.81 µm	7.81 - 15.63 µm	15.63 - 31.25 µm	31.25 - 62.50 µm	62.50 - 125.00 µm	125.00 - 250.00 µm	250.00 - 500.00 µm	500.00 - 1000.00 µm	1000.00 - 2000.00 µm
Culver_Tema 30sec - Average	0.2730	4.3410	111.1370	0	48.1301	11.8532	10.9637	8.4013	6.4631	5.2006	6.7386	2.2495	0	0
Culver_Tema 1min - Average	0.2660	3.4680	39.4070	0	52.3611	13.4371	12.5633	9.1865	7.2368	5.1706	0.0446	0	0	0
Culver_Tema 2min - Average	0.2610	3.8310	82.1050	0	50.3118	11.6682	10.6436	8.3543	6.8103	6.1039	5.8973	0.2106	0	0
Culver_Tema 4min -Average	0.2690	3.5060	53.5420	0	51.9243	11.7076	11.0366	8.9785	8.1313	6.0301	2.1917	0	0	0
Culver_Tema 8min - Average	0.2620	2.5230	26.6000	0	59.5248	12.2901	11.1893	8.9238	6.6388	1.4332	0.0000	0	0	0
Culver_Tema 14min -Average	0.2340	1.3890	26.0230	0	72.6855	7.9100	5.7587	5.1126	5.5658	2.5899	0.3776	0	0	0





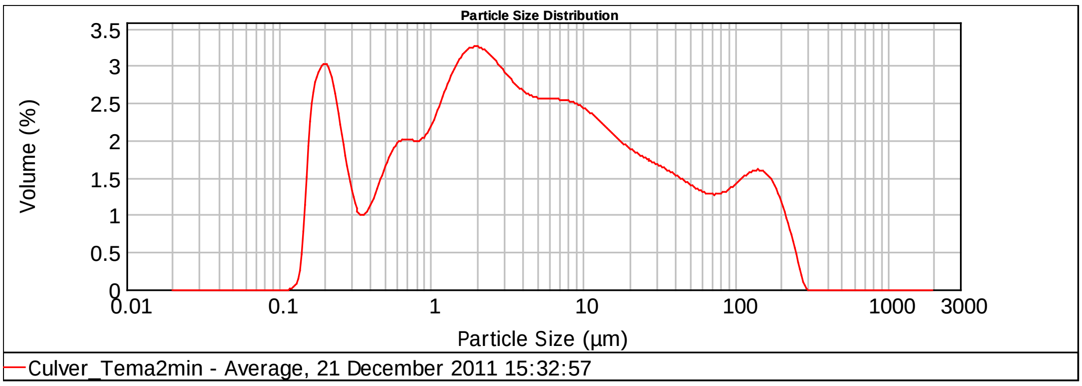
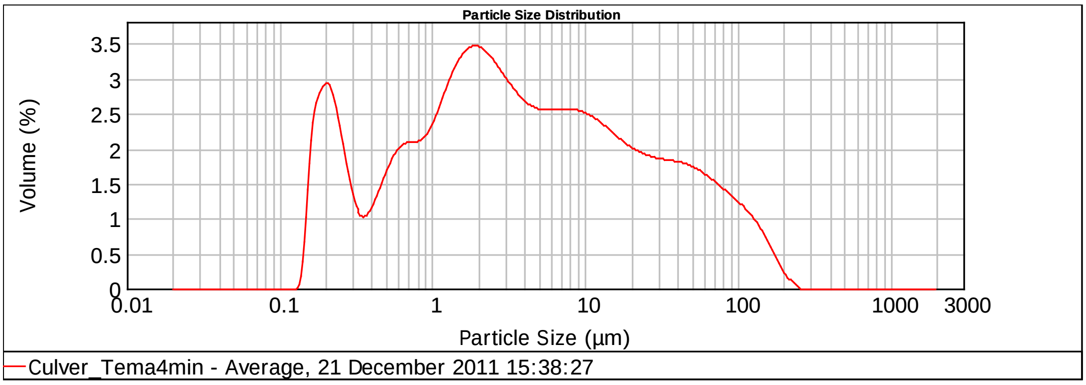
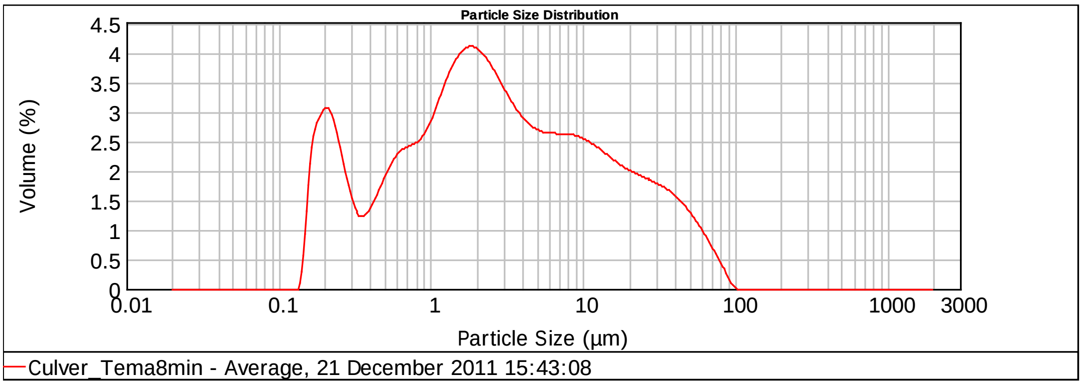
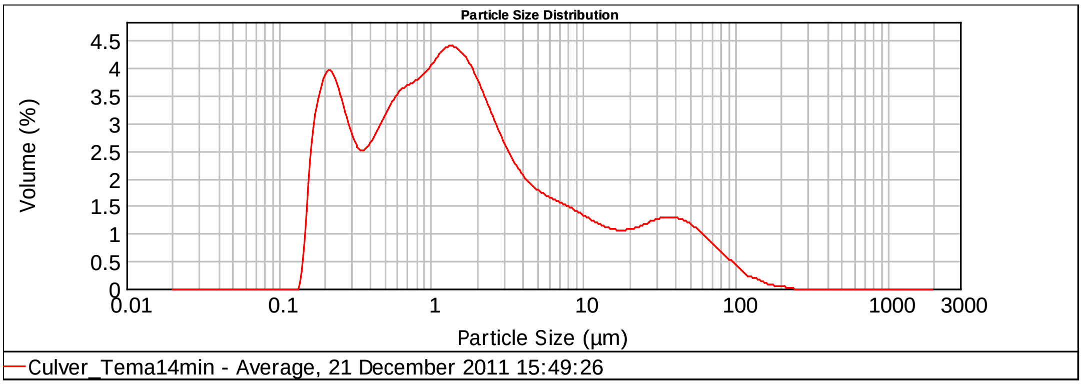


Figure 4.3 Particle size distribution of Culver Chalk for different grinding periods in the Tema Mill (continues overleaf)



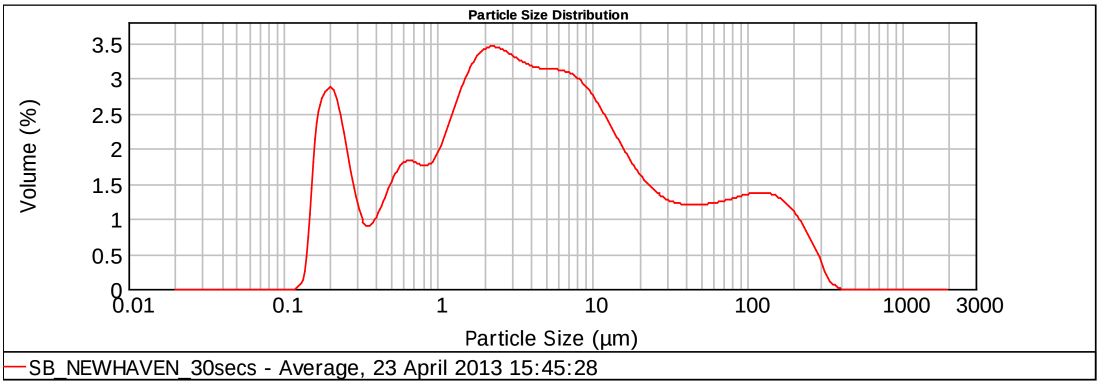


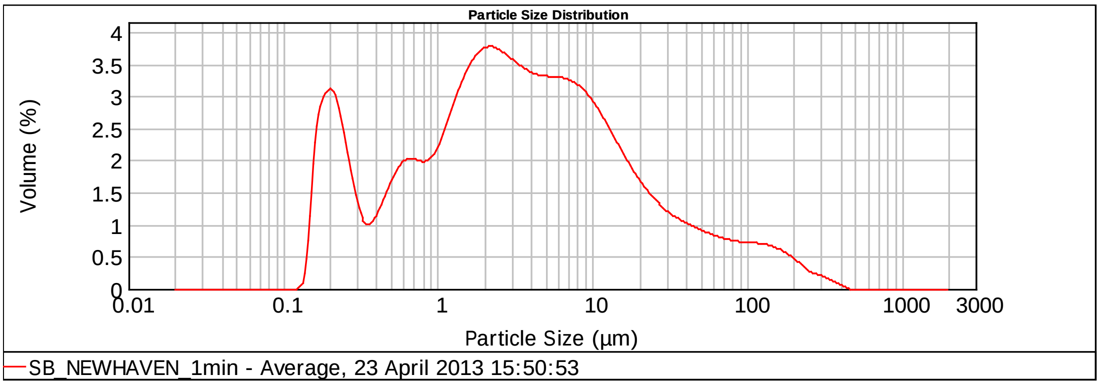


PSD curves are an average of 3 samplings. Variation in $d_{(50)}$ size between the 3 samplings were typically within 4% of the average value.

Figure 4.3 Particle size distribution of Culver Chalk for different grinding periods in the Tema Mill

Sample Name	d 10	d 50	d 90	CLAY	CLAY	V. FINE SILT	FINE SILT	MED SILT	COARSE SILT	V. FINE SAND	FINE SAND	MED SAND	COARSE SAND	V. COARSE SAND
				0.02 - 0.1 µm	**0.1 - 3.90 µm**	**3.90 - 7.81 µm**	**7.81 - 15.63 µm**	**15.63 - 31.25 µm**	**31.25 - 62.50 µm**	**62.50 - 125.00 µm**	**125.00 - 250.00 µm**	**250.00 - 500.00 µm**	**500.00 - 1000.00 µm**	**1000.00 - 2000.00 µm**
SB_NEWHAVEN _30secs - Average	0.274	3.988	81.583	0	49.5359	14.1310	11.6340	7.0375	5.5461	5.8485	5.3301	0.9369	0	0
SB_NEWHAVEN _1min- Average	0.257	3.185	35.365	0	54.5727	14.9753	12.3518	7.1708	4.5441	3.3524	2.4576	0.5753	0	0
SB_NEWHAVEN _2min - Average	0.257	3.453	76.045	0	52.4330	13.1645	11.0907	6.9684	5.1073	4.6186	4.8358	1.6318	0.1499	0
SB_NEWHAVEN _4min - Average	0.255	3.379	74.469	0	52.5842	11.3642	10.1620	7.2741	6.7777	7.8754	3.9624	0	0	0
SB_NEWHAVEN _8min- Average	0.245	2.235	27.249	0	62.8393	11.8231	9.8957	6.5851	5.2085	3.0495	0.5988	0	0	0
SB_NEWHAVEN _14min- Average	0.249	2.039	15.474	0	64.7459	13.7134	11.6530	4.5990	3.3265	1.8744	0.08789	0	0	0





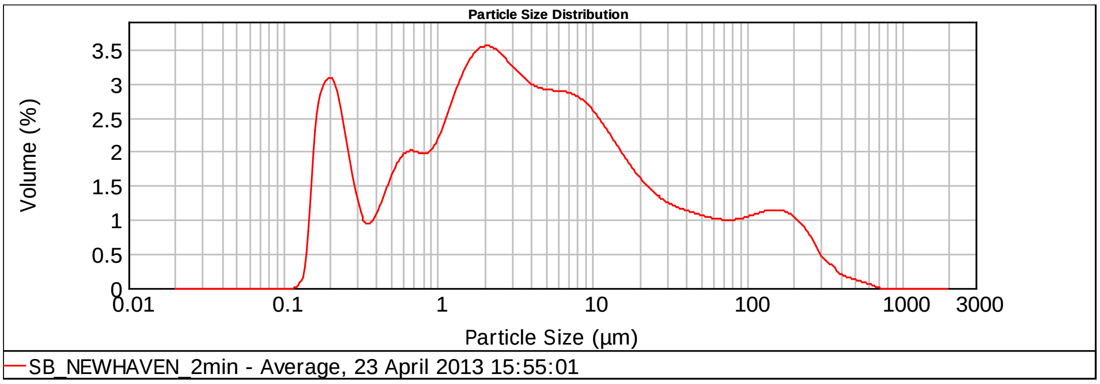
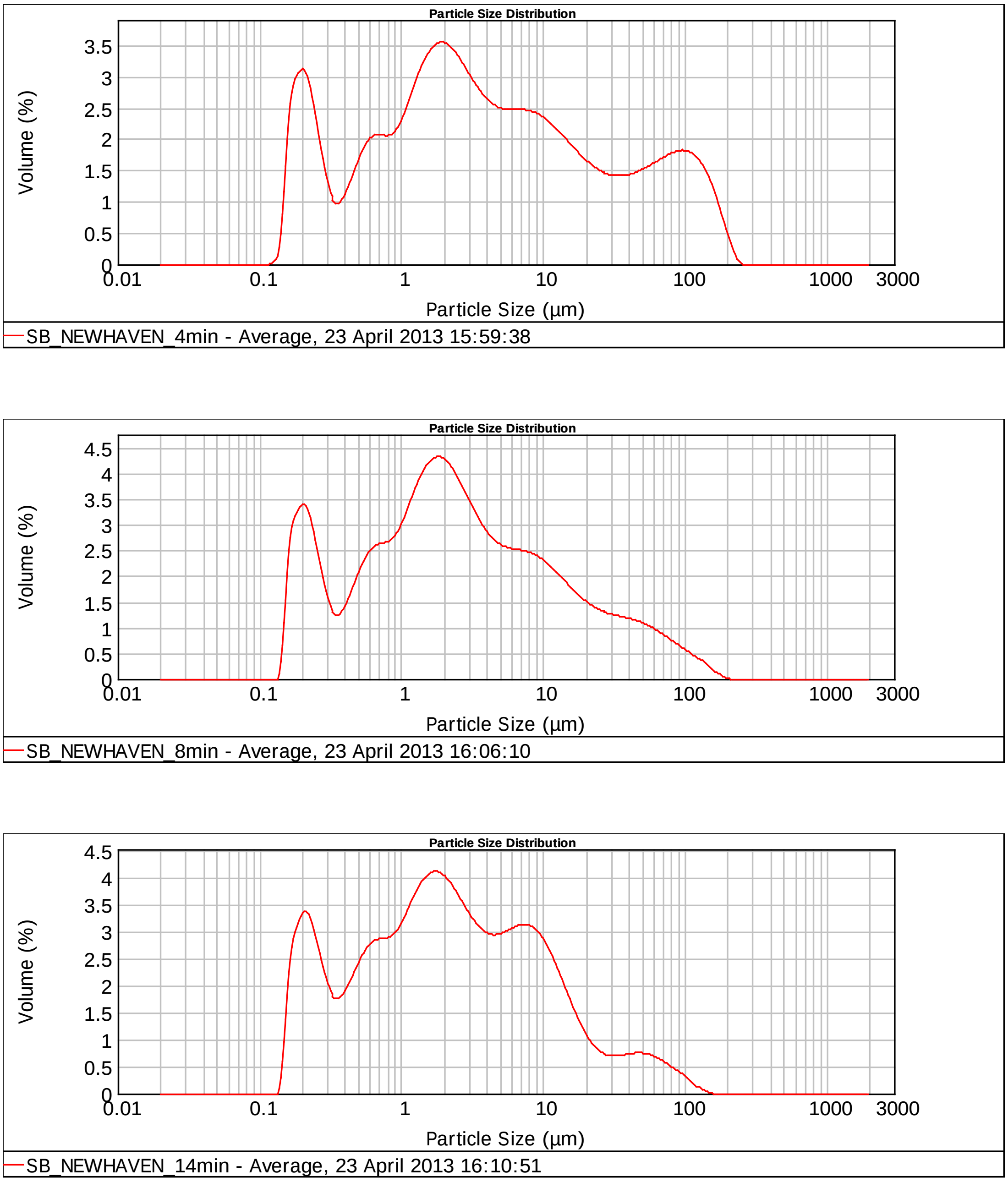
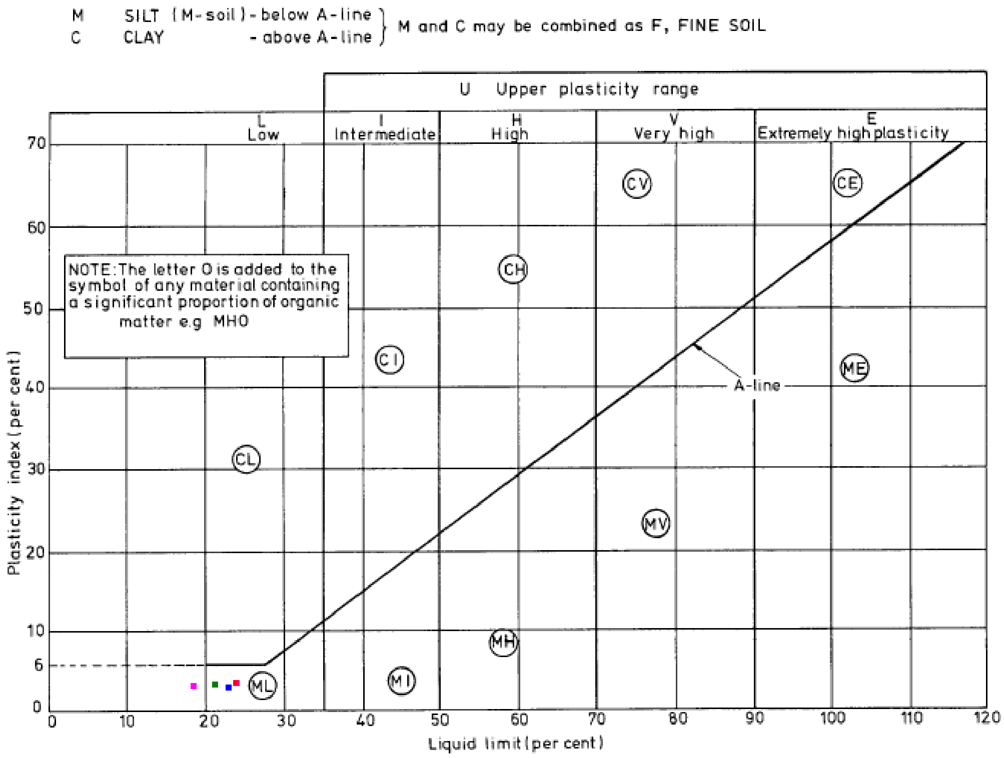


Figure 4.4 Particle size distribution of Newhaven Chalk for different grinding periods in the Tema Mill (continues overleaf)



PSD curves are an average of 3 samplings. Variation in $d_{(50)}$ size between samplings were typically within 4% of the average value.

Figure 4.4 Particle size distribution of Newhaven Chalk for different grinding periods in the Tema Mill



Culver Chalk, Longlands, 2 minutes milling

Newhaven Chalk, Portsdown, 2 minutes milling

Culver Chalk, Longlands, 14 minutes milling

Newhaven Chalk, Portsdown, 14 minutes milling

Figure 4.5 Plasticity Chart (BS 5930) with plasticity characteristics plotted for the putties studied

Sample	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index	Linear Shrinkage (%)	Thixotropic values
Culver chalk putty (2 minutes milling)	18.3	22.1	3.9	2.0	2.1
Culver chalk putty (14 minutes milling)	20.8	24.7	3.9	2.1	2.1
Newhaven chalk putty (2 minutes milling)	22.9	26.8	3.8	1.4	2.2
Newhaven chalk putty (14 minutes milling)	21.9	25.3	3.4	1.6	2.3

To improve repeatability, liquid limits were calculated using the cone penetrometer definitive method, (BS 1377-2:1990, section 4.3.1 note). Results are expressed to 1 decimal place for research purposes and test stages exceed the minimum recommended 4 points in section 4.3.3.9 of the standard.

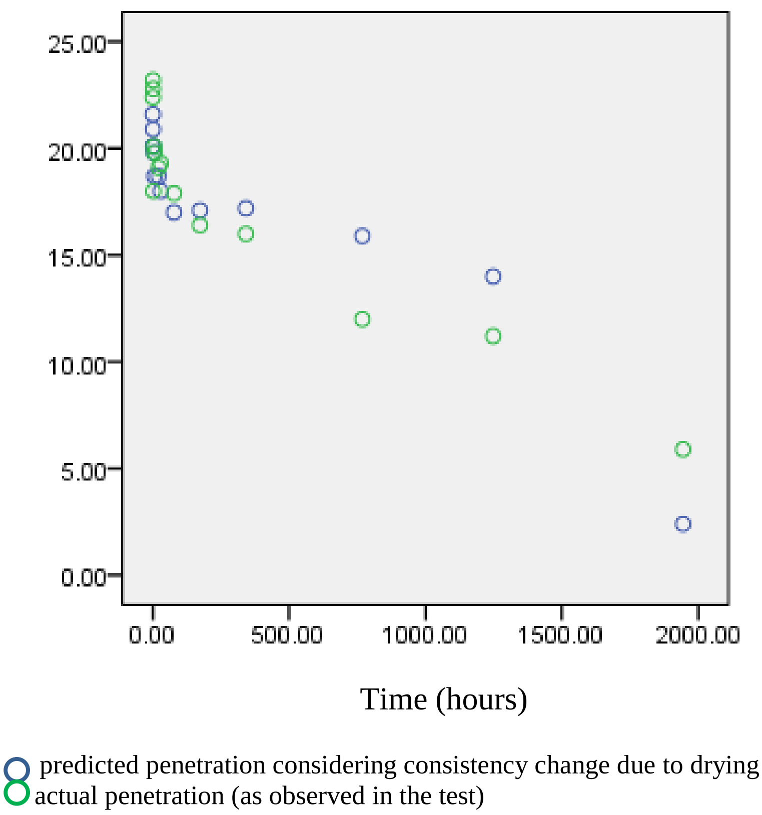
Repeatability of linear shrinkage values was confirmed by a repetition of the test for each of the putties. An average of the two tests is tabulated.

Tabulated Atterberg limits, linear shrinkage and thixotropic values

4.4.2 Shear strength tests using the cone penetrometer

Cone penetrometer tests and conventional direct shear box tests (Section 4.5.1) were conducted primarily to assess the susceptibility of chalk putty to time dependent strengthening (Sections 2.8, 3.6 and 3.7.1). Subsequent analysis of chalk putty could then be made acknowledging any possible changes in strength with time. Newhaven chalk putty, created from material ground for 2 minutes, was considered to be representative of the studied putties in general and selected for the tests.

Figure 4.6 illustrates the results of the cone penetrometer for samples aged for increasing time periods. Cone penetration was found to be inversely proportional to shear strength. Ergo: higher shear strengths are indicated by a lower penetration of the cone. Some drying was observed in the samples aged for longer. To attempt to normalise the data (by isolating consistency changes due to sample drying) Figure 4.6 includes a plot of predicted penetration based on sample moisture.



Data points are an average of 3 penetrations, with a variation of less than 0.5mm. BS1377-2 section 4.3.3.8)

Figure 4.6 Investigation of ageing effects on Chalk putty using the cone penetrometer on Portsdown Chalk.

It is important to note that strength gains owing to drying are considered a change in consistency, not a strength gain as defined by Clayton (1977), Atkinson (1993) and Razoaki (2000). The consequences of drying had not been considered by Clayton (1977) beyond wrapping samples to avoid moisture loss. Although samples were wrapped in these tests moisture content analysis after the penetration test showed a reduction in moisture despite significant wrapping.

To calculate changes in consistency, use was made of the liquid limit tests of Section 4.4.1. In the liquid limit definitive method BS1377-2:1990 section 4.3, a straight line graph is plotted through values of cone penetration against corresponding sample moisture. Linear extrapolation of this line graph (i.e. penetration verses moisture) enables the predicted penetration graph in Figure 4.6, to be plotted based on the recorded moistures at increased periods.

The vertical difference between green and blue data points at a given time, thus illustrating the resultant change in strength owing to time as defined by Clayton (1977), Atkinson (1993) and Razoaki (2000). The graph is used in Section 5.2.3 to demonstrate that strengthening could equally be explained by a change in consistency, as opposed to an age related strengthening reliant on an effective stress change, re-cementing, or fabric change, as discussed in Section 2.8.

(Nb. The linearity of penetration verses moisture beyond the 15-25mm range is not discussed in BS1377-2:1990 section 4.3.4.2 to 4.3.4.4)

4.5 Conventional direct shear box test

4.5.1 Shear strength tests with the conventional direct shear box

The purpose of the tests was to assess whether a shear strength gain occurred with time in chalk putty. The findings are presented in Table 4.2. Although the terms peak and residual have been used to denote shear strength, it is important to emphasise that samples are reconstituted. Peak values, therefore, are only representative for the samples as prepared here under BS1377-7:1990 clause 4. Residual values are shear strengths obtained after multiple reversals of the box halves within the apparatus. Although the peak and residual values may not be indicative of field samples, (which are likely to have been subjected to different stress histories, strains and could not be considered as ‘identical’), the tests would identify (if present) strength gains with time for idealised chalk putty.

Test period (days)	peak (kPa)	residual (kPa)
3	75.1	69.4
25	73.1	68.3
50	72.8	67.5

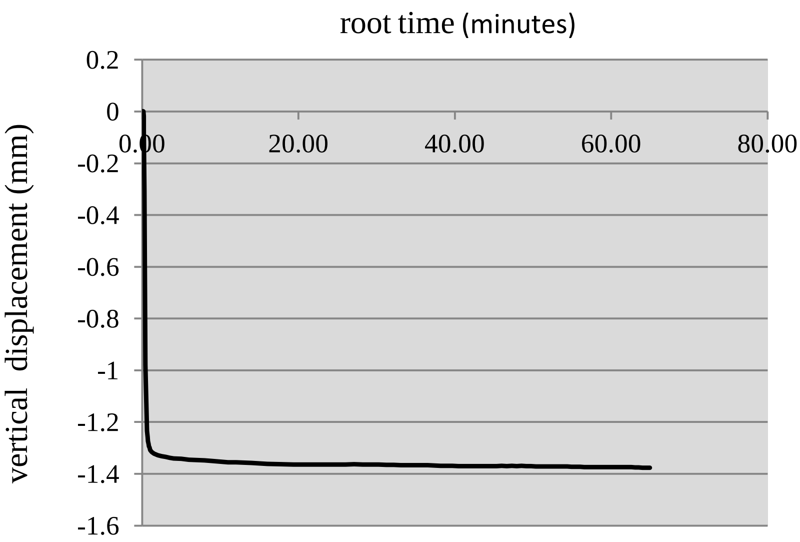
Test speeds:-
Peak 0.0015mm/min
Residual 0.0056mm/min

The test was repeated using a separate machine. Results were replicated, with an average repeat test result of 75kPa.

Table 4.2 Investigation of ageing effects on chalk putty strength, using direct shear box apparatus on Portsdown chalk putty

4.5.2 The effect of time dependent volume changes on test samples

Field observations discussed in Section 2.8.1 suggest that chalk putties may be susceptible to time dependent volume changes (when subjected to continuous loading), which could affect test results. Secondary consolidation (commonly referred to as creep) is considered to be significant in Grade D chalk. To assess the possible effects on testing, volume change was reviewed in the direct shear, ring shear and advanced triaxial consolidation (loading) stages. Figure 4.7 shows a standard consolidation curve for a direct shear box sample subjected to a normal load of 100kPa. This consolidation curve typifies consolidation of the putties tested. By using the curve fitting Taylor root time method, primary consolidation is seen to be rapid, with a coefficient of consolidation (C_v) calculated at 75m^2/year. Post $\sqrt{t_{90}}$ (root time to achieve 90% consolidation) the curve is seen to flatten, adopting a zero gradient indicative of minimal secondary consolidation (C_α).



Rapid primary consolidation was observed in all 6 conventional shear box tests

Figure 4.7 Consolidation curve for direct shear box sample under 100kPa instantaneous normal loading

4.6 Results of residual shear strength tests on chalk putty using the ring shear apparatus

4.6.1 Standard shear strength tests

Test stages were conducted at increasing loading intervals of 50kPa up to 300kPa and then subsequently at 400, 600 and 800kPa. Samples of both Culver and Newhaven chalk putty were tested, formed from material that had been ground in the Tema mill for 2 and 14 minutes. The results are show in Figure 4.8a-d. Individual data points were obtained from averaging the maximum shear strength developed during 6 hour duration of shearing stage. Conventionally plotted here as residual shear strength, Section 5.3.1 challenges the use of this terminology. Each complete test (of nine stages) was conducted over a two week period with final angular displacements of 155°.*

*Unlike the triaxial test or the conventional direct shear box test, it is not possible to simply calculate values of strain as a percentage in the ring shear apparatus. To assess how much strain a sample sustains during testing, it is necessary to know the sample height, shear zone thickness and displacement of the rotating platens. Sadrekarimi and Olson (2007) estimate that a 29mm high sample with a 5mm wide shear zone that has undergone an angular displacement of 265mm would have sustained a shear strain of 53000%. It is understood that strains are significantly higher than the 20-30% strain achieved in the other test, and convention is to represent the magnitude of strain by using either angular displacement in degrees or millimetres, or using time and a shearing rate. Strain is not indicated as an estimated percentage.

No line denoting the failure envelope has been included in Figure 4.8a-d. A linear Coulomb failure envelope (see Section 2.7.2.1) would normally be prescribed as a single best fit line for all data points. Observations suggest, however, that all four materials show an envelope that becomes less linear at higher normal loadings, possibly resolving to a bi linear line with a gradient change at 400kPa normal load. To illustrate this observation, Tables 4.3a-d are provided, showing analysis of the data using a least squares statistical procedure*. Each group of ring shear stages could be considered to comprise two data sets: above and below 400kPa normal loads. To give summative assessment to the non linearity between the two groups, the percentage error of data points above 400kPa, from an extrapolation of a linear regression through the set below 400kPa, is provided in Table 4.3a-d.

*please see glossary

The errors of the 400-800kPa normal loading data set are seen to be consistently larger than, and positive of, the 50-300kPa normal loading linear regression. Coefficients of regression (a measure of the compliance of data points to a regression) are found to reduce on the amalgamation of the two data sets.

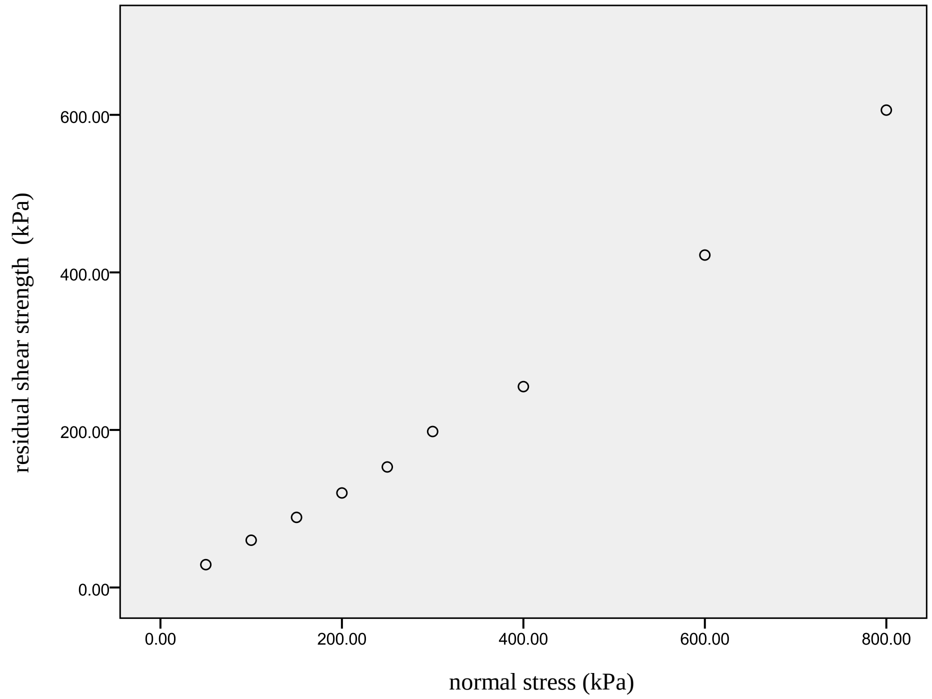
From this statistical approach and an empirical approach based on recent observations in sands (see Section 5.4.1.3), a bi linearity should be entertained for the ring shear failure envelopes.

Sample	Normal pressure (kPa)	Shear strength developed (kPa)	Predicted value (kPa)	Error (kPa)
4.3a)	401	258	242.27	15.74
Newhaven, Portsdown, 2minutes tema mill	600	436	362.58	73.42
	800	614	483.50	130.50
4.3b)	400	226	225.24	0.76
Newhaven, Portsdown, 14minutes tema mill	600	354	337.80	16.20
	800	516	450.35	65.65
4.3c)	400	255	257.56	-2.63
Culver, Longlands, 2minutes tema mill	600	422	390.56	31.32
	800	606	523.55	82.27
4.3d)	400	243	243.66	-0.66
Culver, Longlands, 14minutes tema mill	600	383	364.91	18.09
	800	596	486.16	109.83

Errors are calculated as the vertical deviation from an idealised least square line as predicted by the 50-300kPa data set

Table 4.3a-d. Assessment of shear strength deviation developed in (400-800kPa) ring shear tests, compared with the best fit line through the 50-300kPa data set

a) Culver chalk putty, 2 minutes Tema mill



b) Culver chalk putty, 14 minutes Tema mill

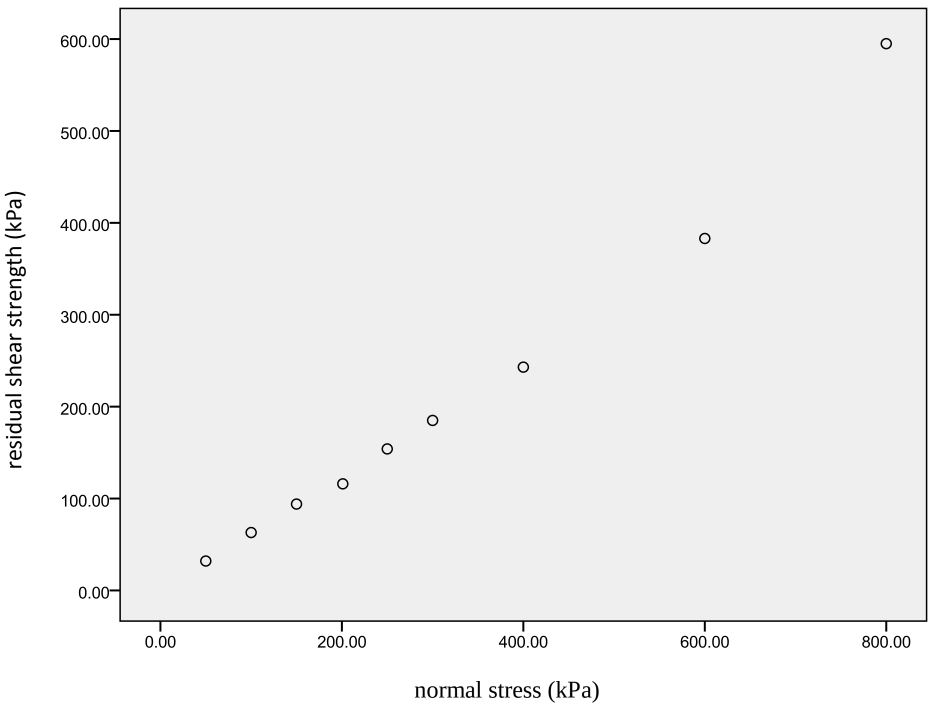
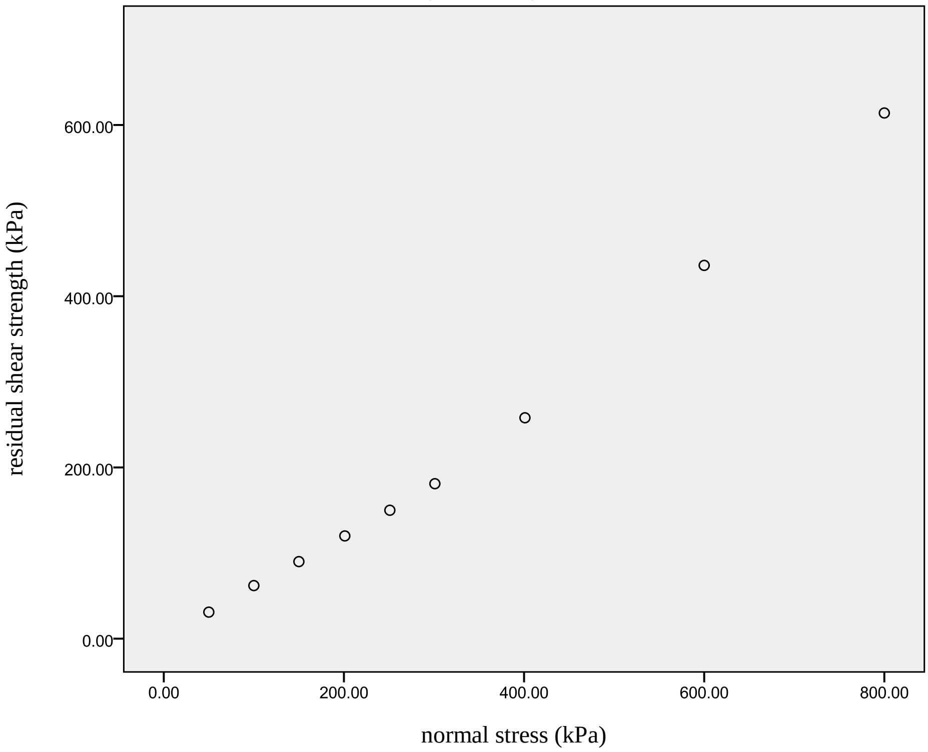


Figure 4.8a-b Results from the ring shear apparatus for Culver chalk putty

c) Newhaven chalk putty, 2 minutes Tema mill



d) Newhaven chalk putty, 14 minutes Tema mill

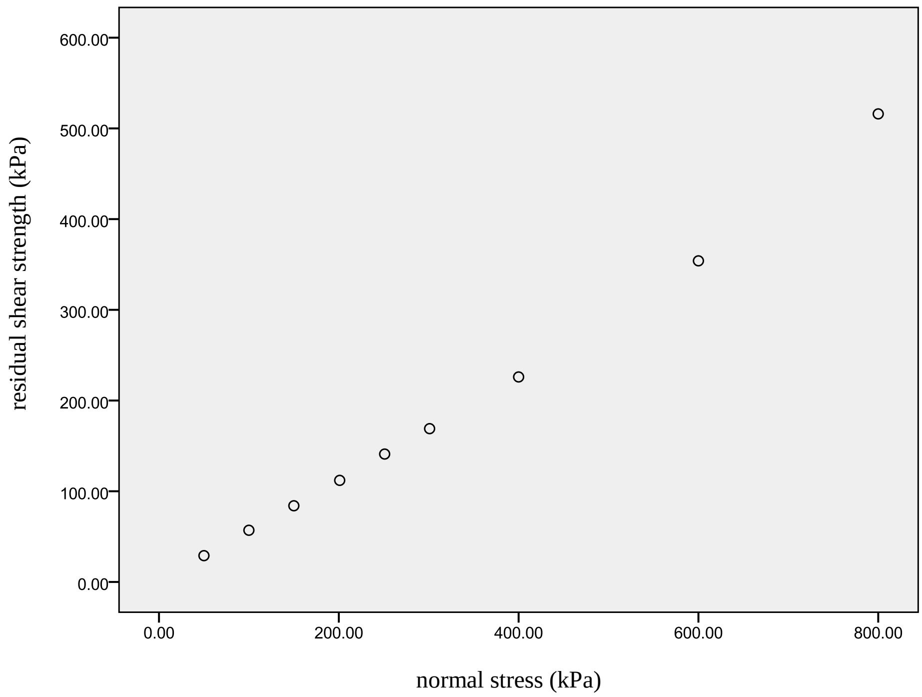


Figure 4.8c-d Results from the ring shear apparatus for Newhaven chalk putty

4.6.2 Further investigation by ring shear tests

The bi linear failure envelope observed in Figure 4.8a-d was an unexpected finding. Section 5.3 discusses and correlates this bi linearity with similar findings occurring in sand specimens in ring shear apparatus. One explanation proposed in 5.3, is that the variation in $Ø'$ is attributable to change in grain size generated by the grinding action on test material. Ergo: $Ø'$ is dependent on sample strain. To investigate this behaviour, further additional tests RSPG1-2 were conducted to establish whether:-

i) Shear strength varied at a constant loading with increased strain.
ii) Any change in shear strength was dependent on the degree of normal loading.
iii) Evidence of grain size and grading changes could be determined from material taken from the shear zone, or from material extruded from between the platens.

The tests RSPG1-2 were conducted over a two week period at constant normal stresses of 100kPa and 800kPa in separate machines. The sample tested was Culver Chalk milled for 2 minutes in the Tema mill. A period of two weeks was chosen to represent the approximate length of the tests carried out using the ‘optimal procedure’ multistage method (section 3.7.2). Laser particle size analyses were carried out on samples taken from both the material extruded from the outside of the plates and from the estimated location of the shear plane within the top 1mm of the sample undergoing shear. The machine speed was set at 0.048°/min as in the ‘optimal procedure’ tests; both machines were run for an identical period.

The findings of residual shear strength variation over the two week period are shown in Figure 4.9 and discussed in Section 5.1.3. Shear stress was calculated by averaging the calibrated outputs of two load cells acting on the top platen of the apparatus (see Figure 3.2c). The automated logging data values of shear force, developed by the chalk putty samples, were ‘weeded’ prior to conversion to stress values before their graphical presentation.

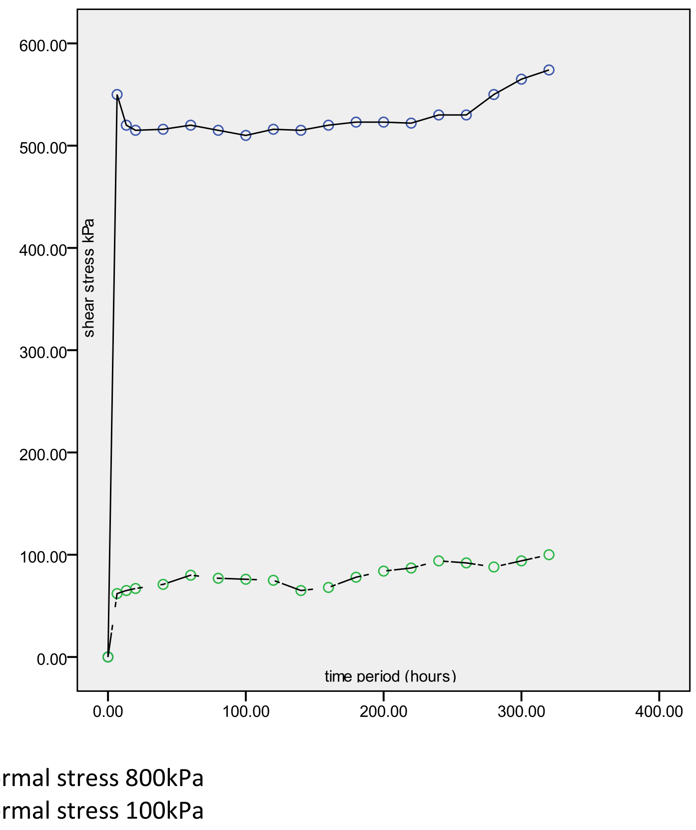


Figure 4.9 Residual shear strength changes with time in ring shear tests, RSPG1-2

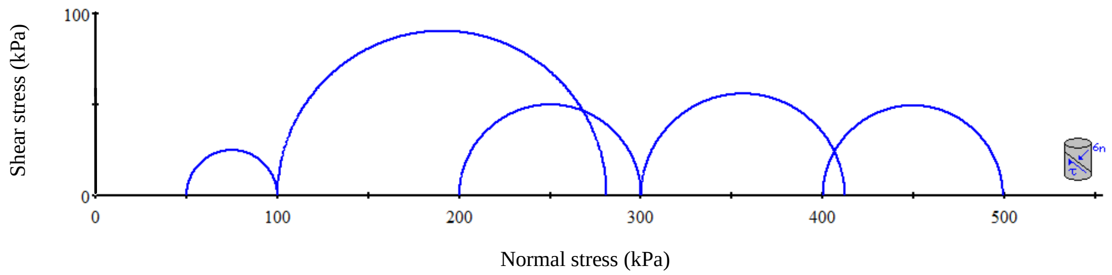
4.7 Results of the undrained triaxial tests

Results of shear strength analysis on partially saturated undrained triaxial samples using conventional procedures are shown in Figure 4.10a-b. Clayton (1977) predicts that the failure envelopes of chalk putty should be of a similar form to the hypothetical examples given in Figure 2.4 for partially saturated granular materials. In this study, however, both Culver and Newhaven data sets exhibit failure strengths that varied with confinement, and randomly between tests. This random variation is not seen in Clayton's work and is greater than would normally be expected in tests on clays or sandy soils. An explanation of the poor fitting of the undrained failure envelopes to the hypothetical curve, Figure 2.4, could be explained by sample preparation. In many tests, placement of the reconstituted sample on the base platen proved difficult prior to testing. With plasticity indices of 3.4 -3.9 (Figure 4.5), samples were seen to deform from a right cylinder. Pre test barrelling and loss of vertical alignment were observed as putty samples slumped under self weight (explored further in Huang (2012)). Other sample defects may also have been important. It was found that despite meticulous control of tamping, pockets of low density material were not uncommon. Further, saturation conditions were found to vary vertically across the sample. Pore waters draining to the lower portions of the sample meant basal material was much closer to its liquid limit than at the top of the sample.

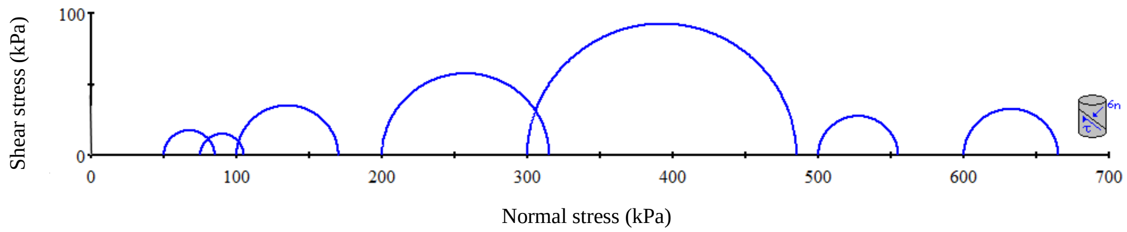
An additional set of undrained tests was performed on Newhaven chalk putty, with samples subjected to a 24 hour period of undrained confinement prior to shear. Results for this set of test are shown in Figure 4.10c. Greater approximation to Figure 2.4 is evident, suggesting a better consistency of sample shape prior to shear. Observations indicated that samples consolidated through internal drainage and adopt a more consistent form. Internal drainage may also have enabled dissipation of porewater pressures, although it was not possible to investigate this as pore pressure readings were not recorded.

It is likely that the scatter was not readily seen in Clayton's (1977) study as only three tests per sample were conducted; if a greater number had been conducted (as here) it is predicted that a random scatter between tests would have been manifest.

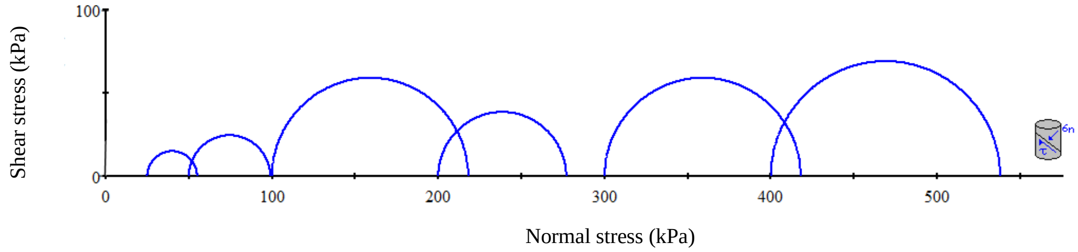
Experiences from these conventional tests demonstrate the difficulties in routinely applying British Standard procedures to the triaxial testing of chalk putties. The difficulties of sample preparation can be used to defend the 'dry press' technique used in the advanced triaxial tests of Section 4.8. The use of a suction top cap enabled self weight deformation to be eliminated as q could be maintained at zero in all pre-shear test stages.



Undrained test, Culver chalk putty



Undrained test, Newhaven chalk putty



Undrained test, sample confined for 24 hours, Newhaven chalk putty

Typical Mohr circle failure envelope plots for soil consist of 3-4 test stages, (eg. Craig, 2004). Tests at additional confining pressures are provided to emphasize the variability of tests results.

Figure 4.10 Undrained ('Quick') triaxial tests on Newhaven and Culver chalk putties

4.8 Results of the advanced triaxial tests

4.8.1 Introduction

Results of the advanced triaxial tests can be divided into two sub-groups:

Section 4.8.2 presents the back pressure data required to develop the methodology outlined in 3.8.2, describing isotropic, consolidated drained shear tests under back pressure.

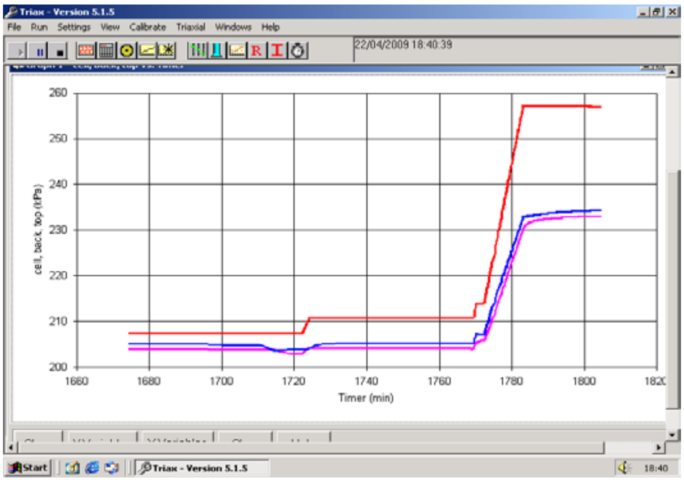
Section 4.8.3 presents the results of the isotropic, consolidated drained shear tests. These results provide information of chalk putty behaviour under repeated isotropic, consolidated drained stress paths at differing confining pressures; giving information on the isotropic, consolidated failure envelope (4.8.3.1), permeability change (4.8.3.2), and void ratio evolution (4.8.3.3) during the different stages of the tests outlined in Sections 3.8.2.5. The chalk putty tested in the advanced triaxial tests was Longlands, Culver Chalk milled for 2 minutes in the Tema mill.

4.8.2 Back pressure 'B' value tests

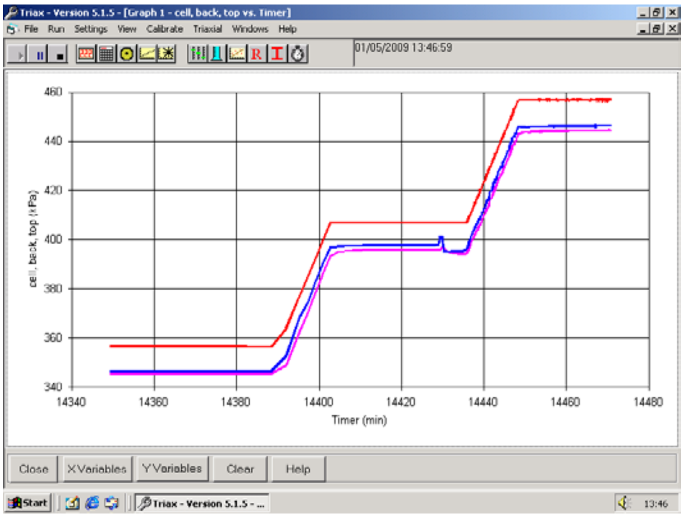
A series of 'B' value tests was carried out to ascertain what back pressures were needed to achieve acceptable saturation values. It is generally regarded that Skempton's pore water coefficient should be greater or equal to 0.95 (BS 1377 - 8:1990). A series of screen shots of the Triax software, Figure 4.11a-b illustrates that for reformed chalk putty samples using the 'dry press' technique, back pressures of at least 400kPa were required to achieve saturations greater than 90%.

It was observed that up to back pressures of 400kPa, 'B' test values of 0.90-0.93 were regularly recorded. In all tests, 'B' values improved by 0.02-0.03 after monitoring the back pressure until equilibrium was reached, BS 1377-8:1990 5.3.2.b,c,i. This often required a period of one hour and is indicative of the impermeable nature of the reconstituted sample. Equipment limitations meant that the use of higher back pressures of greater than 400kPa (i.e. 820kPa maximum manostat pressure minus 400kPa cell pressure) could not be explored when the test stages proceeded to consolidations of more than 400kPa (σ_3'). A back pressure of 400kPa was therefore used in all the tests. Discussion of the cause of reduced 'B' values can be found in Section 5.4.2, with the relevance of using back pressures on clay-size inert particulate materials (as opposed to conventional cohesive clays) noted in Section 6.7.

a)



b)



This behaviour was repeated in B tests on Newhaven chalk putty.

Figure 4.11a-b Screen shots of Triax software. Graphs plot top pressure (blue) and back pressure (pink) response in undrained conditions against a 50kPa ramp in cell pressure (red). Sample tested, Culver chalk putty ground for 2 minutes in the Tema mill.

4.8.3 Results of isotropic, consolidated drained stress path tests

4.8.3.1 The Mohr-Coulomb failure envelope (described in Section 2.7.2.2) for Longland's Culver chalk putty is presented in Figure 4.12. The failure envelope determined from the tangent to the Mohr curves for confining pressures of 100-400 kPa is linear with an angle of 31°. This construction has been determined in accordance with BS1377-8 section 8.6.3, note 4. Testing during drained shear was automatically stopped using an 'alarm' condition (stop strain > 20%).

4.8.3.2 Vertical permeability testing of the triaxial samples was conducted both prior and post shear (procedure as in Section 3.8.2.5.7). The results for chalk putty permeability are presented in Figure 4.13, with an interpolation line included to assist assessment of the data.

4.8.3.3 Sample volume change was recorded throughout the saturation, consolidation and drained shear stages of the advanced triaxial tests. Using volume change readings, the void ratio was calculated. The results of the changes in void ratio during the different stages of the advanced triaxial tests are tabulated in Table 4.4.

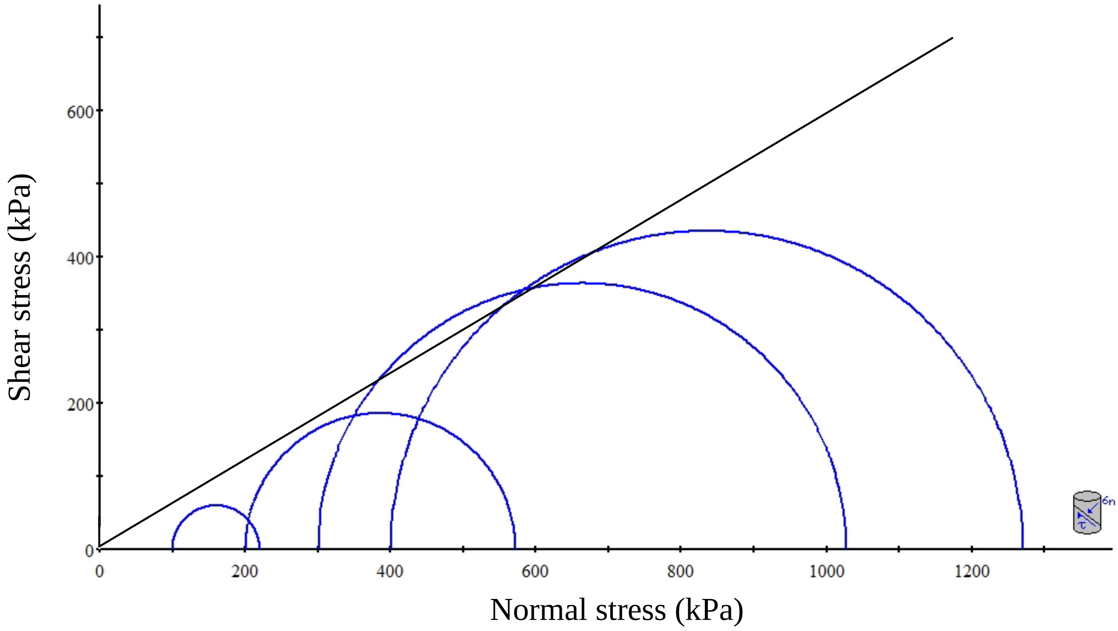
The initial void ratio of the sample was calculated using a particle density (G_s) of 2.71Mg/m^3 (the density of calcite). Subsequent void ratios were calculated by treating volume change gauge readings as indicative of void change and considering the volume of solid material as constant.

Test Sample/confining pressure kPa	e_{flush}	e_{sat}	$e_{sat, con}$	$e_{sat,con,drained\ shear}$	Modified state parameter (Ψ)
RCL2 / 100	0.565	0.646	0.629	0.589	0.040
RCL1 / 200	0.557	0.616	0.579	0.550	0.029
RCL4 / 300	0.576	0.635	0.596	0.551	0.045
RCL3 / 400	0.543	0.622	0.566	0.523	0.043

e_{flush} assumed to be equal to initial void ratio, see Section 3.8.2.5.2
$e_{sat,con\ drained\ shear}$ at 20% strain

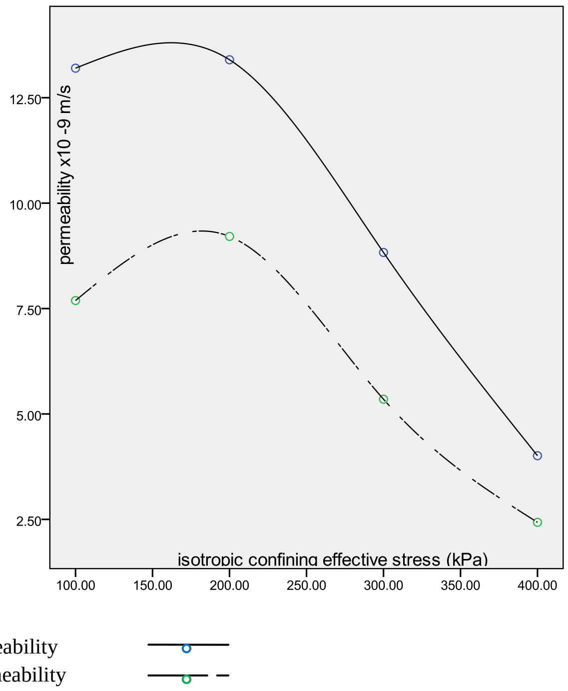
Table 4.4 Void ratio changes at different stages of the advance triaxial tests

Change in void ratio in the advanced triaxial tests is discussed in 5.4.6, but attention is drawn to the change in void ratio $e_{sat,con,drained\ shear}$ during shear stages which is represented by a modified state parameter Ψ in the final column of Table 4.4. State parameters were first defined for sands in the undrained condition by Been and Jeffries (1985) as the difference in void ratio between the initial and critical state void ratio; they have since been adopted more widely (Davies et al. no date) for silt materials in effective stress analysis.



Typical Mohr circle failure envelope plots consist of 3-4 test stages, (eg. Craig, 2004).

Figure 4.12 Mohr circles denoting the position of the Coulomb failure envelope for advance triaxial tests. Ø' is calculated at 31°



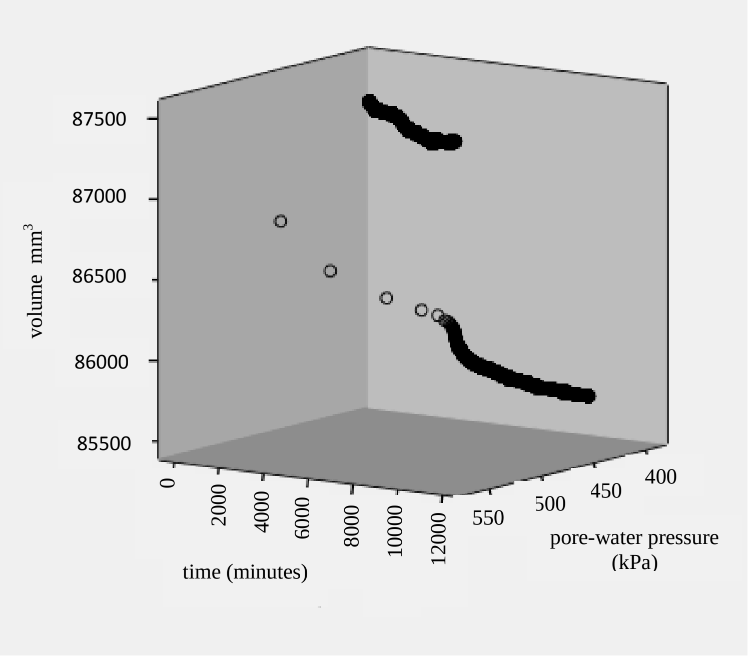
A spline* (glossary) interpolation line has been applied. Permeability results are an average of 3 permeability tests. Variations between calculated test values were typically less than 3%.

Figure 4.13 Vertical permeability in advanced triaxial tests, prior and post shear stage

The state parameter is considered 'modified' in Table 4.4 because it is based on the difference between the void ratio at the beginning of drained shear and the void ratio at 20% strain. Discussions continue in Section 5.4.6.

In order for volume change to occur during any stage of the advanced triaxial tests, pore pressures must increase or decrease within the test sample to generate a transient state of flow. By creating conditions of minimal pore water pressure change, invariably by slow ramping in the stress path cell (see Section 3.8.2.5.6), drained conditions can be achieved.

Monitoring of top and base sample pore water pressure lines during testing showed that in tests RCL3-4, drained conditions were maintained in all stages. For tests RCL1-2 however, a 'spike' in pore pressure was recorded at the point of failure during the shear stage. This failure (described as liquefaction in 5.4.6) was a near instantaneous event lasting less than two minutes. Figure 4.14 illustrates the interrelationship between volume and pore-water pressure for test RCL1 during the shear stage period. The failure event was seen to coincide with an abrupt change in pore water pressure, reaching a maximum increase of 167.5kPa. A near identical graphical form was affirmed for test RCL2, in which the maximum pore pressure rise was recorded as 162.0kPa. It should be noted that the data recording frequency for test RCL1 was at 1 minute intervals. This was increased to 10 second intervals for RCL2 once the rapidity of failure in RCL1 had been observed.



RCL2 replicated the findings of RCL1

Figure 4.14 The interrelationship between sample volume and pore-water pressure during the shear stage (time) for advanced triaxial test RCL1 (data sampling at 1minute intervals)

4.9 Summary

Geotechnical index properties for chalk putty following BS 5930 (1990) show test putties to be classed as ML soils* with limited thixotropy and plasticity. An understanding of chalk putties' composition, PSD and plasticity has been used in developing 'identical' soils for subsequent shear strength tests.

*glossary

Work by Clayton (1977), and Razoaki (2000) reviewed in Chapter Two, suggest that chalk putty strengthens with time. This ageing could influence shear strength tests, particularly the advanced triaxial tests carried out over a three week period. The effect of the parameter is explored in cone penetrometer tests and conventional shear box studies. From the limited results presented, no evidence of strengthening with time was observed. Chapter Five explores why earlier studies may have indicated strengthening, and justifies the discounting of this variable in subsequent shear strength tests.

Shear strength analysis in high strain ring shear apparatus indicates that failure envelopes have a bilinear nature. Ø' angles are seen to vary between 29° to 38°, similar to the range of published data in Table 2.5. Chapter 5 attempts to explain this bi linearity with reference to microfabric and possible changes that may occur at high strains.

Shear strength parameters for undrained triaxial tests following standard procedures BS1377-1 and BS1377-7 (1991) proved unsatisfactory. Results were contrary to those obtained by Clayton (1977), which indicated that a partially saturated curved failure envelope was readily identifiable using conventional tests to BS 1377-7 clause 8 (1991). Inconsistent data are explained by experiment practice, where samples deformed through self weight prior to shear stages. Adaptations for this were incorporated in advanced triaxial tests.

Under stress-controlled isotropic, consolidated drained conditions, a linear failure envelope was observed with Ø' = 31° from consolidated p' between 0 and 400kPa for strains measured at failure and at 20% (when no identifiable failure was observed). Permeability assessment pre and post shear gave values of the order of 10^{-9} m/s, indicative of an impermeable soil. The nature and form of the permeability against p' curve is explored in Chapter Five and used to explain the different mechanism of failure observed under different p'. Volume change measurements enable void ratio changes during experimentation to be tabulated indicating contraction during failure. The pattern of void ratio changes is explored in Chapter Five using modified state parameters (Ψ).

Chapter Five
Discussion of Results

5.0 Overview:-

5.1 Tests conducted to characterise the chalk putties of the study. It is argued that a micro understanding of chalk putty and its formation from intact Chalk is essential to an understanding of the macro geotechnical responses observed.

5.2 Isolating variables which could affect the results of shear strength tests. It is argued that without standardisation of material and testing methodology, geotechnical data have limited value.

5.3 Test data obtained from the ring shear apparatus. Comparison of test data with published data and explanation of bilinear failure envelopes.

5.4 Developing an advanced triaxial testing methodology. Explanations of the effective shear strength parameters, pre- and post-yield permeability, and void ratio changes obtained.

5.5 Summary.

5.1 Analysis and characterisation of the chalk putties studied

5.1.1 Porosity and dry density of ‘parent’ material

Results Sections 4.1 and 4.2 focus on the findings of characterisation of the intact parent Chalks, which were subsequently milled to form test samples. Three Chalks, from different stratigraphic horizons, were initially considered. Assessment of porosity, dry density, and intact failure envelope supported the use of Culver Chalk from Longlands Quarry and Newhaven Chalk from Portsdown Quarry as detailed in Table 4.1 for the purposes of this study. Both Chalks were considered weak enough to likely form putties in the event of civil engineering works.

5.1.2 Composition and grading

Once milled, chemical assessment confirmed a high purity which isolates possible influences of clay content on testing; see Section 4.2. Laser particle size analysis (Figure 4.3 - 4.4) indicates that both samples are generally formed of inert, non-cohesive, calcite particles between the ranges 0.1µm to 300µm when milled for two minutes in a Tema mill. The grading was observed to narrow on further milling to material of a more uniform grain size of 0.2 – 2 µm.

Tables included in Figure 4.3 and 4.4 offer tabulated assessment of the changes in particle size distribution curves with milling. Included in the tables are summative assessment of the data in the form of D_{10}, D_{50} and D_{90}, where D denotes the effective particle size and the numerical subscript denotes percentage passing that size.

Both samples show the migration of particle size from a bimodal to a more mono modal material. It can be argued that this grain size and its evolution are related to the building blocks of intact Chalk discussed in Section 2.2.1. Observation of both Culver and Newhaven chalk putty under an electron microscope (Appendix Four) indicates that the putties are formed from material which varies in size between the basic coccolithic laths and cemented collections of entire coccolithic shields and shields at various stages of disintegration. Particle size evolution is observed as the material breaks down to progressively finer material. After 14 minutes of milling D_{90} is seen to be 15.5µm for Newhaven Chalk and 26.0µm for Culver Chalk*. The predominant particle size is observed to be in the range 2-4µm, concurrent with the size of the individual coccolithic laths longest dimension. Particle size evolution appears to be towards this critical size with D_{50} showing only a small change from 4µm to 2µm in both samples during the 14 minute milling period. Once at 2µm-4µm further size reduction is not evident, the coccolithic lath tending to form the ultimate particle size. The peak at 0.2µm is likely to represent the laths' minimum dimension.

*There may be some discrepancy between particle sizes observed under the electron microscope and those of the laser analyser. Appendix Three considers a shift of the clay/silt boundary to compensate.

It was not expected that D_{90} should reduce significantly in the 1 minute milled samples of both Chalk types, prior to increasing in subsequent milled samples. A possible explanation may be found in the way that the Mastersizer 2000 software analysis data using the Mie model. Explained further in Appendix Three, the laser analyser utilises the obscurity of particles to enhance accuracy. Obscurity may be set depending on the material being tested. In tests in this study obscurity was set for calcitic particles. Section 4.2, however indicates that some of the material in the greater than 'very fine sand range', is non-calcitic, having a different obscurity than the major calcitic components of the Chalk. Inaccuracies in reading this coarser material may have resulted in variations in D_{90} values which are seen to vary to a greater extent than the other effective particle sizes.

5.1.3 Atterberg values

Atterberg values categorise the chalk putties as ML soils. Figure 4.5 illustrates that Atterberg values are slightly below the A line (Section 2.6.2). The low plasticity of the soil is indicative of a soil low in clay mineralogy without van der Waal forces between clay minerals to provide cohesion. Insufficient data is presented to relate grain size and particle size distribution to consistency, but as a generalisation the finer Newhaven chalk putties have slightly higher liquid limits. This is consistent with the logic that the liquid limit is related to the amount of water attracted to particle surfaces. The finer the grain size the greater the total surface area and therefore the greater the potential to attract water around individual particles resulting in a higher liquid limit.

The low plasticity index (an average of 4%) represents a material with a tendency to adopt a liquid consistency with the addition of only small amounts of water. This proved problematic in sample preparation of partially saturated samples for the 'quick' undrained tests (Section 4.7), with the collapse and loss of form through self weight occurring at a moisture content of 20% (average plastic limit). To attain higher saturations (needed in advanced triaxial testing to facilitate pore water pressure analysis), the liquid limit was reached well before full saturation.

5.1.4 Linear Shrinkage

Linear shrinkage was low across all samples (typically 2%), indicative of a low clay content. In soils of a clay mineralogy, water can be held at the atomic level (i.e. between the SiO_4 tetrahedral layers), causing significant shrinkage on drying. For soils with low shrinkage values, water is found interstitially, with little water-solid interaction at the atomic level. Gromko (1974) has related linear shrinkage to shrinkage limit and defines a linear shrinkage value of 0-5% as exhibiting a non-critical degree of expansion. For the chalk putties tested, the effect of shrinkage on subsequent tests is considered negligible, and significantly less than soils containing clay.

5.1.5 Thixotropy

Results of tests conducted to determine sample thixotropy were found to be in the range 2.1 - 2.3 as predicted by Boswell (1949). As values are comparable to those observed in silts and clay soils, it was considered unlikely that thixotropic behaviour would affect subsequent testing, and adaptations were therefore unnecessary.

5.2 Isolating variables which could affect the results of shear strength tests

5.2.1 Assessing the fabric and structure of chalk putty

To create identical soil samples for testing, it is necessary to understand the fabric and soil structure of chalk putty. Electron microscopy described in Appendix Four shows that the re-packing of constituent grains is not uniform, with the development of aggregations or clusters of grains in a non-uniform pattern. Such aggregation patterns have been observed in other granular soils, for example, Smith et al. (1929) investigated the distribution of voids in lead shot and noted how alternate regions of minimum and maximum density occurred with porosities of 47.6% and 25.9% respectively.

Others (Feda 1992, 1994) observed similar dense clusters in granular sands, and argued that these clusters controlled initial loading when the sand sample is subjected to compression. Loading is initially seen to move the clusters closer together until the dense clusters alone (not the intermediary low density material) provide a supporting framework.

Rowe (1971) developed the theory that dense clusters of particles (later commonly referred to as aggregations) could alter the friction angle. It was argued that for plane strain tests, more energy (indicated by the measurement of a higher Ø' value) was needed to cause shear than was observed in triaxial tests. Plane strain was found initially to encourage particle sliding in a number of directions, with the pre-determined shear plane rarely coinciding with the minimum friction angle. In triaxial tests however, particles commonly form into locked groups or clusters. At small strains, it is argued that these clusters of dense, locked granular material slide against neighbouring clusters. The system is then locked until failure and the creation of new cluster groups takes place.

From the test data presented in Sections 4.6.1 and 4.8.3.1 comparison may be made between effective friction angle (Ø') determined in plane strain (ring shear) and triaxial tests for Culver chalk putty milled for two minutes. Although the strain varies between tests, the reconstituted nature of samples would mean a residual value is reached rapidly in both tests. In plane strain, Ø' is 30.6° in the lower strain test stages, whilst under triaxial conditions Ø' reaches 31°. The similarity of angles suggests higher Ø' values have not formed owing to the absence of plane strain cluster formation.

Absence of clusters in test samples could be explained by their fully saturated nature. Aggregate, or cluster, theory has been extended to fine grained soils, silts (Delage et al*;* 1996) and clays (Sills, 1998). Delage et al. observed how particles within partially saturated silts (drier than optimum density or moisture) clumped into aggregates of 100μm diameter with large inter-aggregate pores of approximately 80μm diameter. With increased moisture the whole silt material was seen to become denser with a less distinct aggregation / pore pattern. The need for partially saturated conditions to assist aggregate formation is noted by both Delage et al. and Sills.

Although evidence of Ø' change with aggregation is not seen in the results of this study, an aggregation pattern was observed in electron microscopy of chalk slurries by Razoaki (Razoaki 2000) and in the electron micrograph (Figure A4-1b in Appendix Four). It is suggested that the conditions of mounting putty onto a stud for electron microscopy are very different from those present in test preparing samples. Sample preparation for electron microscopy is likely to be the cause of aggregation, as explained further in Appendix Four.

There is some evidence that aggregation in chalk putties is encouraged by vibration (Razoaki 1994, 2000), or as a natural process whereby material settles out of suspension from algal blooms (i.e. as an original sedimentary feature, Riebesell, 1991). Samples from this study however, were not subjected to vibration (see Section 3.8.2.5.1), nor able to exhibit a former sedimentary pattern, as this would have been destroyed by the milling process.

If aggregation does occur in chalk putties, only low p', partially saturated tests (such as the index tests) are likely to see its presence. It is unlikely to occur or have an influence in fully saturated shear strength tests.

5.2.2 Assessment of time-dependent volume change on chalk putty test samples.

The rapid primary consolidation and negligible secondary consolidation observed in Section 4.5.2 indicate that time-dependent volume changes would not be influential on subsequent laboratory testing. With test procedures such as the Watson Harris ‘optimal’ procedure (3.7.2) and advanced triaxial ‘ramped’ consolidation (3.8.2.5.5) designed to mitigate the effects of consolidation, further consideration of the time-dependent volume changes on test size samples was viewed as unnecessary. The discrepancy between secondary consolidation in laboratory samples and macro field observations (detailed in 2.8.1) maybe a result of laboratory samples exhibiting reduced heterogeneity, owing to the absence of field features such as rootlets and fissures.

5.2.3 Assessment of chalk putty ageing using cone penetrometer

Whilst the effective cohesion strength gain with time tests (cone penetrometer at intervals of over 2000 hours and shear box tests at 3, 25, 50 day periods) were not intended to be exhaustive, it was considered prudent to evaluate how susceptible the Culver and Newhaven chalk putties were to this phenomenon. The results of the cone penetrometer are shown in Figure 4.6. Although a decrease in penetration indicates an increase in strength with time, the process of strength gain with drying must be considered along with the penetration against time plot. The change of soil consistency with changes in soil moisture give rise to strength variations in addition to those suggested in Section 2.8.3. Consistency of a soil and the ability to change its physical state from a liquid to a solid is tested for, by using arbitrary Atterberg tests (Section 2.6.2). It is seen from the Atterberg data, presented in Figure 4.5, that all putty samples exhibit a low 3-4% plasticity index. This suggests that the study putties exhibit rapid state changes with minimal moisture content differences, as indicated by the steep line in the multi-point liquid limit plots of cone penetration against moisture described in BS1377-2 section 4.3, (1990), cone penetrometer method (definitive method).

Despite careful sample storage of the cone penetration pots to avoid moisture loss (Section 3.6), post-test moisture determination indicated that sample state change would be significant considering the putties' low plasticity indices. For this reason a second (blue line) of predicted cone penetration is presented in Figure 4.6, based on the linear extrapolation of a multi point liquid limit graph. The predicted strength increase is therefore somewhat reduced when compared with the normalised blue line, other than for the points at 1940 hours which are for penetrations of around 5mm.

Although the author accepts that this linear extrapolation of consistency with moisture is only a prediction, it does suggest that some of the strength increase with time may be explained in part by sample drying. Non-linearity in moisture verses penetration at the extremes of the liquid limit test (i.e. 5mm) may explain the discrepancy at the 1940 hour points.

It should be noted that for partially saturated tests (Atterberg limits, cone penetrometer tests to review ageing) the effect of cluster formation may have been greater. This was supported, in so far that, any ageing effects were only observed in the cone penetrometer tests, not in the direct shear box tests (discussed further in Section 5.2.4).

5.2.4 Assessment of chalk putty ageing using direct shear test

The direct small shear box tests were conducted to ascertain the influence of strength gain with time of chalk putty. This was predicted by Clayton (1977), and Razaoki (2000) as described in Section 2.8. The results are tabulated in Table 4.2. All tests were conducted on Newhaven chalk putty with an applied normal stress of 100kPa using the same test apparatus throughout.

From the data tabulated, there is no apparent increase in strength over the 50 day test period. Yet Clayton (1977, 1990) predicted a noticeable increase in strength up to 80 days. A difference in peak strength (indicative of sample 'cohesion') of 2kPa was observed between tests conducted at the different time periods. However, this minor variation was considered within the limits of the repeatability of the test (based on a ±0.5° Ø′ angle, BS1377-7:1990 section 4.7m) and so would not be treated as indicative of a strength change.

Residual values were consistently 5kPa lower than the peak values which suggests that any cohesion was minor, regardless of the time period from sample preparation. It should be remembered that all samples would be defined as ‘remoulded’ (BS1377-7:1990) and ‘reconstituted’ by Lord et al. (1993) (after Burland (1990), see Section 2.1 of this study). As such, peak values are first time shear or ‘pseudo peak’ values (Bundy, 1991) and are not indicative of a cohesion formed from a field fabric. If the modest 5kPa strength increase is not due to a fabric interlocking (field fabric) it may be attributed to apparent cohesion. Apparent cohesion is seen in partially saturated cohesionless soils, where matric suctions provide a surface tensional force between grains. Apparent cohesion can be more significant in fine cohesionless soils, where osmotic suctions are considered important (Terzaghi et al., 1996). Although the shear box test is considered to test for effective strength, a degree of partial saturation is likely prior to the multiple reversal stage and full drainage equalization.

Eight multiple reversals (BS1377-7:1990 section 4.5.5) gave a consistent displacement of 20cm prior to the residual strength tests. Unlike clay soils, where clay platelet particle alignment plays a significant part in peak / residual behaviour, there is no suggestion here that such a fabric has, or could have, developed within the putties during shear. Although coccoliths have some plate-like nature, many are rhombic (Section 2.2.1), and no alignment has been identified in electron microscope studies (Appendix Four). Kerry et al. (2010) suggests some calcitic plate alignment when analysing chalk soils in laser diffractometers, although this alignment is somewhat less than in clay samples, see Appendix Three.

No evidence was found that cementing or re-cementing was contributing to strength. If the strength difference between peak and residual was owing to cementing, an increase with time might logically be expected. Further, no fabric changes were observed between samples to support a re-cementing argument. Samples showed some dilation during peak shear, (indicating a lower sample density across the plane shear zone prior to residual testing) but sampling across the shear zone and observation under the electron microscope showed no fabric changes at different time intervals.

It can be argued that when compared to shear strength tests using the cone penetrometer (Section 5.2.3), sample drying is less likely to occur within the direct shear tests. It was recorded that all samples underwent some rehydration (based on calculations of moisture before and after testing) and that moisture content evaluation after shear tests indicated a greater uniformity than was observed in moisture content recorded after ageing in the aforementioned cone penetrometer tests. This is logical since the direct shear box test pertains to be an effective test in which water migration is permitted in and out of the sample during testing. Drainage/rehydration occurs because the test sample sits in a water bath during testing. When the moisture content was controlled between strength tests conducted with up to 50 days of ageing, no variation was recorded in the strength of samples in the shear apparatus.

5.3 Discussion of the ring shear simple shear results

5.3.1 Ø' angles and comparison to literature

Figure 4.9 typifies the stress-strain response of the putties tested in the Bromhead ring shear apparatus. All ring shear tests showed stress-strain behaviour whereby yield was approached by a linear path with post yield showing the adoption of constant failure strength or an ultimate strength condition within an individual loading stage (i.e. for the initial 6 hour period). Yield generally occurred at displacements of 1mm, with a more brittle response occurring at the higher effective stresses. A number of post yield strength parameters can be assigned to materials of a granular nature (namely ultimate strength, critical state strength, steady state deformation*). From Figure 4.9 it is evident that any post yield state is broadly of the same value for all putties tested. Roscoe et al. (1958), Schofield and Wroth (1968) argued that in order to achieve the critical state, the soil must continue to deform not only at a constant stress but also a constant volume (or void ratio) under drained conditions. Establishing the condition of zero volume change during shear at constant mean stress and void ratio can prove difficult in practice (Coop, 1990). It should have been possible to demark the point when no volume change occurs within the ring shear by using the vertical linear displacement transducer. Zero volume change is demarked by a constant vertical displacement. With the study tests however, extrusion of chalk putty occurred continuously during shear in all ring shear tests (Figure 5.1). Such soil extrusion (also observed by Osano, 2004 and Sadrekarimi, 2009 in ring shear tests with sand) results in the sample height (and hence volume) reducing relatively constantly during the tests and masking any “true” volume change.

*glossary

Without the ability to determine the point of zero volume change and with the post yield stress/strain behaviour a constant value (in a stage period of 6 hours), it is reasonable to assume that the critical state and the ultimate state are of a similar value. Unlike clay samples, no reduction in strength beyond the critical state to a lower residual value was seen. Although the ring shear apparatus is designed primarily to investigate residual values, it might be prudent to question whether it is appropriate to use the term $Ø'_r$ (effective residual friction angle) rather than Ø' for chalk putties. $Ø'_r$ has not been used in this section, as would typically have been used in values derived from the ring shear apparatus. Shear strength values for remoulded or reconstituted materials would ordinarily be expected to be equivalent to other (lower strain) shear strength tests (i.e. the direct shear box or triaxial tests). Furthermore, $Ø'_r$ could be confused with the same term (Ø'$_{r)}$ used to denote post failure values in intact Chalk.

As discussed in 5.2.4, in cohesive materials the residual state is associated with a reorientation of clay plate-like particles in a narrow shear band or zone. The absence of a stereotypical lower residual state would suggest an absence of particle alignment, although as with tests on sands (Sadrekarimi, 2009), a thin zone of failure would be expected. If failure is within a thin zone, this would be contrary to the critical state concept which envisages deformation throughout the sample as a whole.

Table 5.1, tabulates the Ø' angles shown in the graphs of Figure 4.8a-d. These values compare favourably with values presented in Table 2.5. Although only small differences in Ø' are seen between Culver and Newhaven putties a notable bi-linearity is seen in all failure envelopes.

normal load (kPa)	**50, 100, 150**	**200, 250, 300**	**400, 600, 800**
Culver chalk putty (Ø')			
2 minutes Tema mill	30.6	32.2	35.9
14 minutes Tema mill	31.1	31.3	34.7
Newhaven chalk putty(Ø')			
2 minutes Tema mill	31.1	31	36.8
14 minutes Tema mill	29.4	29.3	32.8

n.b. Ø' values are based on an assumed zero cohesion

Table 5.1 Ø' values for different loading stages of the ring shear, illustrating non-linearity of failure envelopes

5.3.2 Non-linear failure envelopes in chalk putties

The possible causes of the non-linearity observed in Figures 4.8a-d, are discussed hereunder.

5.3.2.1 Influence of test machine

In the light of the bi-linearity seen in the chalk putties of the study, consideration was initially given as to whether operational procedure could influence results in the ring shear apparatus. Earlier work by the author with Osano (2004) revealed a fundamental lack of understanding of the interconnectivity between data, operational procedure, and machine characteristics. Test data was considered 'unique' until Stark and Vettel (1992) suggested higher effective Ø' values might result in conditions of sample thinning which could cause greater frictional force at the wall of the shear chamber. As sample thinning was argued to be more prevalent under certain practices, it was argued that Ø' might vary with the test procedure adopted.

Four commonly used test methods were cited 1) single stage, 2) pre-shearing, 3) multistage and 4) 'flush' test procedure for the ring shear apparatus. Methods 1 and 2 need little explanation and are as described in BS 1377-7 clause 6, (1990); method 3 is as described in Section 3.7.2 under the Harris and Watson 'optimal procedure'. The fourth method, the 'flush' procedure however, was proposed by Stark and Vettel, (1992) as a new more accurate multistage test in which the top platen was not allowed to sink below the height of the adjacent retainer. Typically with procedures 1-3, the sample is extruded at the outer edge of the annulus and the sample thins. In the 'flush' method the sample height is maintained, and wall frictional forces are minimised because the shear plane is maintained close to the top of the retainer where less wall friction is observed. Tests performed on Pierre shale indicate that at normal effective loads above 100kPa the multistage test procedure over-estimates Ø' compared with the 'flush' procedure. The flush multistage values were also considered to be a better representation of values calculated from back analysis in the field. The additional strength gain, in the standard multistage tests, was attributed to increasing wall friction developing as the top platen settles into the specimen. However, it was considered unlikely that changes in wall friction alone could result in the magnitude of increase in Ø' in the chalk putties of the study for the following reasons:-

i) The multistage results for Pierre Shale are linear and not bilinear as with chalk putty. The chalk putties show a distinct upward trend above 400kPa. Stark and Vettel predicted that samples maybe weaker than that predicted using a linear projection of the failure envelope.

ii) A distinct strain dependency in Ø' is observed at effective normal loads as low as 100kPa, as illustrated in Figure 4.9.

iii) A steepening in Ø' occurs above 400kPa even though sample extrusion on the outer annulus ring occurs from the onset of shear.

iv) Consolidation of chalk putty was shown to be relatively instantaneous with typical C_v values calculated as $75m^2$/year (in the direct shear box under a normal load of 100kPa Section 4.5.2). Rapid consolidation was further verified during the ramp consolidation stages of the advanced triaxial test. Although sample extrusion was recorded, the consolidation of chalk putty would have been considerably more rapid than with clays, as tested by Stark and Vettel (1992). If the shear plane maintained a position at the top of the retainer more readily than with clays, a less pronounced Ø' change from a linear trend would have been seen.

v) A new modified top platen was used in the Wykeham Farrance ring shear of the study. Other studies Meehan (2006) have shown that adoption of this platen minimized side wall friction to give more accurate results than achieved by using modified test procedures such as the ‘flush’ method.

vi) The Watson and Harris ‘optimal procedure’ reduces strain by not using a pre-shear stage post consolidation (see Section 3.7.2).

5.3.2.2 Influence of sample material

De Mello (1977) was one of the first to discuss non-linearity in clays (albeit that the failure curves became shallower with increasing p') as reviewed in Section 2.7.3.5. De Mello suggested that consolidation history was the most important controlling parameter giving rise to non-linearity in clays. However, because all ring shear tests presented here followed a virgin compression path (having been created from reconstituted samples at their plastic limit in accordance with the “optimal procedure” methodology) it is unlikely that over-consolidation history could have influenced the four failure envelopes (Figure 4.8a-d).

Considering chalk putty as a non-cohesive granular material (Section 4.4), failure envelope non-linearity might be explained by considering how shear strength is mobilised at the microscopic level. Lee and Seed (1967) and Terzaghi et al. (1996) considered that for granular material, shear strength is formed from inter-particle sliding friction and geometrical interference.

Sadrekarimi and Olson (2011) describe inter-particle friction as dependent on particle surface roughness with a value ($Ø'_{\mu}$) independent of confining stress and density (Rowe (1962), Lee and Seed (1967)). A value of $Ø'_{\mu} = 30°$ has been established by Terzaghi et al. (1996) for chalk, but it is generally considered that $Ø'_{\mu}$ values are difficult to reproduce in practice (Negussey et al. (1988), and Sadrekarimi and Olson (2011)). This said, it is not unreasonable to suggest that $Ø'_{\mu}$ is constant between ring shear stages. Geometrical interference (synonymous with tangential locking in this thesis) is formed as particles push against, pass over, and damage adjacent particles. $Ø'_{\mu}$ maybe considered as that shear strength mobilised by dilation ($Ø'_d$) and that mobilised by damage and rearrangement ($Ø'_p$) of particles. As dilation was not observed in either the ring shear stages or advanced triaxial tests, it is reasonable to suggest that friction generated by damage and particle rearrangement ($Ø'_p$) is significant in explaining variations in Ø' between ring shear stages.

As discussed previously (Section 2.8; Atkinson, 1993), critical state envelopes can be seen to shift (i.e. are not constant) if the fabric (particle distribution, packing and grain shape) of a soil changes. Coop (1990), McDowell et al (1996) and Lade et al (1996) all describe grain crushing in sands during isotropic compression and shear stages of triaxial tests. If this phenomenon were to occur in chalk putties, it could be supposed that the critical state envelope could equally change. Mean effective stress (p') could be a parameter that changes both particle form (size and shape) and grading, with changes more prevalent at higher p'.

Nearly all studies on the effect of grain crushing have been conducted on sand. Experimental observation by Drescher and de Josselin de Jong (1972) and Mandl et al. (1977) have shown that forces between soil particles on the microscopic scale are considerably higher than the external macro applied forces. As a result, global stresses* on a sample below the crushing strength of the constituent material can result in significant grain crushing. Stress is found to initially concentrate on particle asperities where crushing initiates.

Once fracturing occurs, the micro stress rapidly reduces as increased grain numbers result in a reduction of the average stress at contact points. It is believed that shear stress, at least in sands, is the primary cause of crushing, in preference to confinement. Luzzani and Coop (2002) argued that to initiate crushing without the presence of shear requires global confining stresses several times greater than those of shear.

From the data presented here, the steepening envelopes appear most pronounced in the ring shear tests where shear strains are significant (see Section 4.6.1) at the end of a two week multi stage 'optimal procedure' test. A similar bi-linearity is not seen in the other effective stress tests conducted. Drained shear triaxial tests (presented in Figure 4.12) show no clear bi-linearity despite being over a similar p' range. Possibly the much lower strains of triaxial tests (a maximum of 20% if no clear failure is recorded, BS 1377-8:1990 section 7.2.11.), are insufficient to induce grading and particle crushing.

Sadrekarimi, (2009) summarises in Table 5.2 those soil properties which affect particle damage and crushing. Although tabulated as a review of particle crushing in sand, it seems logical that the table has relevance to chalk putty. In terms of grain properties for example, it is logical to assume that calcitic coccoliths would degrade readily. With a Mohs hardness of 3 (compared with 7 for quartz), crushing would be considered more likely than in sands even if a reduced grain size may reduce crushing forces.

*Please see glossary

Soil mass properties	
Grain size distribution	In a well-graded soil, more particles surround individual grains reducing the average contact stress and decreasingparticle damage (Lee and Farhoomand 1967; Lade et al 1996).
Initial void ratio	Decreasing void ratio (at a given confining stress) decreases particle damage because smaller void ratios generally yield a higher number of particle contacts (coordination number) and thus a better distribution of stresses (i.e., confinement) produced by neighboring particles (Hagerty et al 1993; Lade et al 1996; McDowell and Bolton, 1998; Tsoungui et al 1999; Nakataetal2001)
Grain properties	
Hardness	Increasing hardness decreases particle damage (Marsal 1967; Lade et al 1996; McDowell and Bolton 1998; Feda 2002)
Shape	Increasing angularity increases particle damage as a result of greater stress concentrations at asperities (Lee and Farhoomand 1967; Hagerty et al 1993; Lade et al 1996).
Size	Increased particle size generally increases crushing as a result of the increased probability of inherent flaws and defects occurring in the particles (Billam 1972) and the decrease of Brazilian tensile strength (Lee, 1992; Lade et al. 1996; Nakata et al. 1999). However, in a well graded sand, the coordination number for large particles surrounded by large numbers of finer particles is very high while the opposite is true for the finest particles. In this case, the tensile splitting stress for the large particles is relatively small while that of the small particles with a low coordination number is much larger. Thus the probability of splitting of the finer particles would be higher (McDowell et al. 1996; McDowell and Bolton 1998; Muir Wood and Maeda 2008)
External parameters	
Effective confining stress	Increasing effective confining stress increases particle damage(Lade et al. 1996).
Shear displacement	Increasing shear displacement increases particle damage(Agung et al. 2004; Coop et al. 2004; Lobo-Guerrero and Vallejo 2005).
Time	Some particle damage continues with time, resulting in creep(Lade et al 1996; Leung et al 1996; Takei et al 2001;McDowell and Khan 2003)
Mode of loading (or stress path)	More particle damage occurs during shearing than during isotropic compression (e.g., Hall and Gordon 1963; Bishop 1966)
Temperature	Temperature can affect the crushing susceptibility of some mineral constituents (Nakata et al 2003; Chester et al 2004)

Table 5.2 Inherent soil mass properties influencing susceptibility to particle crushing (after Sadrekarimi, 2009)

To assess how grading (and particle shape see Section 5.3.2.6) may evolve with the working of chalk putty, PSD analyses were conducted on Culver Chalk and Newhaven Chalk, ground for different periods of time in the Tema Mill. A series of PSD plots were produced using the Mastersizer 2000 laser analyser (described in Section 3.5 and Appendix Three) on material which has been ground for periods of 30 seconds to 12 minutes. The results are shown in Figure 4.3 and 4.4.

Although it is impossible to quantify how much crushing, as opposed to shearing, takes place in the Tema milling apparatus (Figure 3.1) it is evident that a change in grading and grain size begins to occur after 8 minutes. This finding is contrary to that of Clayton (1977) and Razaoki (1994), who argue that once milled (ground), Chalk quickly achieves a standard grain size after which further crushing becomes minimal. These findings were based on simple field observation with no quantification of the physical effort involved in the Chalk break down.

Initially (in the 30 seconds to 8 minute grinding periods) the samples appear well graded with grains in the range 0.5µm to 2mm. As grinding proceeds, however, increased uniformity occurs as particles of the order of 10µm break down to 1µm. Study of electron micrographs (Appendix Three) indicates that this may be explained by the break-up of coccolithospheres at 10µm into their constituent 1 µm coccolith laths.

5.3.2.3 Influence of particle grading on Ø' angle

Several authors (Coop et al. 2004, Sadrekarimi, 2009) have suggested that changes in grading, grain form and size are significant in causing a curved failure envelope in granular material such as sands. The variation in Ø' was found to be complex, dependent on whether a sample was contracting or dilating or at its critical state (i.e. dependent on the stage of the phase tranformation*). Ø' was consistently defined as to whether it was referring to friction angle at yield, critical state or mobilised. Ring shear tests on three different sands, compacted by air-pluviation and moist tamping, (Sadrekarimi and Olson, 2011) showed an increasing yield Ø' with decreasing consolidation void ratio, increased mobilised friction angle with increased strain and an eventual constant critical state value once all volume changes have taken place.

* Glossary

That yield Ø' is dependent on initial density has been long understood (eg. Craig 2004). It is intuitive that dense sand will have higher shear strength than loose sand until the point that initial fabric densities are lost through phase transformation. In this respect an increase in friction angle could be accounted for in the ring shear results of this study if each stage of the tests is considered as commencing at a new lower consolidation void ratio, i.e. denser than before. With each stage representing increased strain, particle breakage and reduced void ratio. Particle breakage (as observed in Section 4.3) would cause densification as the open-structured coccolithospheres are broken down into their more closely packed component coccolithic laths.

5.3.2.4 Direct sampling of sample to determine whether uniformity or grading changes during ring shear testing.

To investigate if a change in grading takes place during the ring shear test, two ring shear tests were conducted over a two week period at constant normal stresses of 100kPa and 800kPa (tests RSPG 1-2). The sample tested was Culver chalk putty milled for 2 minutes in the Tema mill. A period of two weeks was chosen to represent the approximate length of the tests carried out using the 'optimal procedure' multistage method. Laser particle size analysis was carried out on samples taken from both the material extruded from the outside of the plates and from the estimated location of the shear plane within the top 1mm of the sample undergoing shear. The machine speed was set at 0.048°/min as in the 'optimal procedure' tests. Both machines were run for an identical period.

The results are shown in Figure 5.1. Very little difference in PSD is identifiable between any of the four samples and none seem to have changed dramatically with regard to grading or grain size from the original sample of Culver Chalk milled for two minutes in the Tema mill Figure 3.1. The reason could be that sampling was unable to extract material from the zone of shear and so the material has undergone no significant shear deformation, being outside of this zone. Although relatively easy to identify in sand (Sadrekarimi and Olson 2010) where it is both visible and of the same magnitude as the sand grains in which it is located, the shear zone is not visible in chalk putty. The shear zone could be as narrow as several microns across assuming it is related to grain size in the same way that it is in sand.

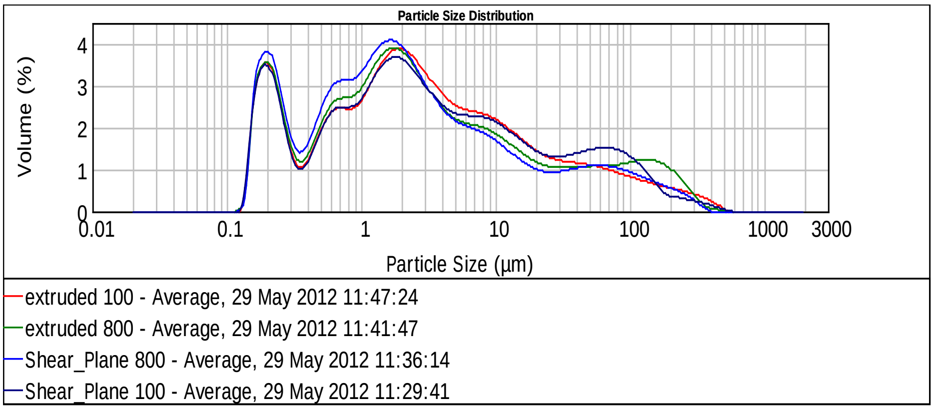


Figure 5.1 Sample collected from material subjected to two weeks ring shear grinding at 100kPa and 800kPa

5.3.2.5 Discussion of RSPG 1-2

Although no change of particle size distribution was detected, as described in the previous section, stress / time period graphs of the tests RSPG 1-2 (Figure 4.9) show a clear upward trend over the 12 days they were conducted. The greatest strength increase, of 50-60kPa was seen at the 800kPa normal loading, which compared with an increase of 35kPa for the 100kPa test. These values suggest that, like the studies of Coop, et al. (2004) on carbonate sands, higher confinements result in increased particle damage. Unlike carbonate sands however, particle crushing is considerably more prevalent at low normal loads such as 100kPa, a result reasonably explained by the weakness of coccolithospheres compared with carbonate sand grains.

Extrapolation of the graphs indicates that strength should increase because particle breakage should continue well beyond 12 days. The tests could not be extended beyond the 12 day period because of the excessive material lost from between the plates, as discussed 5.3.1.

The result of this finding is that volumetric compression must be occurring throughout the 'optimum procedure' ring shear tests (RS 1-4). The critical states of Ø' that had earlier been assumed in Table 5.1 are therefore not rigorously defined. The additional tests RSPG1-2 illustrate that Ø' values of tests RS 1-4 are only critical values at a transient state of 'constant volume'. It would be argued that all chalk putty tests undertaken at large strains (displacements) will produce strain-dependent failure strength parameters; a result that is contrary to the findings of Coop et al. (2004) on carbonate sands.

5.3.2.6 Influence of particle angularity on Ø' angle

Increased particle damage is reported (eg. Sadrekarimi and Olson 2011) to result in increased angularity of particles in sands and an increase in the $Ø'_p$ component of the geometrical interference or tangential locking between grains. This phenomenon is expected to be mirrored in chalk putty samples under shear tests, with some increase in Ø' a result of increased particle angularity.

5.4 Discussion of the advanced triaxial results

As in Section 4.8 this section can be similarly divided into a discussion of issues related to testing methodology (5.4.1 - 5.4.2) and discussions of the results of the subsequent isotropic, consolidated drained shear tests under back pressure (5.4.3 - 5.4.6).

5.4.1 Discussion on the development of dry press technique

Before the 'dry press' technique (3.8.2.5.1) was adopted as the chosen method for preparing triaxial samples, two other techniques of achieving suitable density samples from loose material, were considered. The techniques are fully reviewed in Appendix One. It was considered that:-

i) The 'horizontal shaker' (Barton and Brookes, 1989) was not used, because of the evidence of Feda (1994) and Razoaki (2000). Both Feda and Razoaki recorded a significant change in the microfabric of chalk putties when subjected to vibration. Vibration was observed to result in the aggregation of coccoliths whereby clumps of dense material co-existed amongst looser material. Razoaki (2000) considered the formation of aggregates as a major contributor to the strengthening of chalk slurries with time. The change of strength with a microfabric alteration would have affected the repeatability of testing.

ii) The 'consolidometer' (Razoaki, 2000) was not used, because using consolidation (as a means to reduce the density of loose samples) would have imparted an effective stress history onto the test samples. Razoaki (2000) found that suitable maximum densities where achieved at a vertical consolidation pressure of 86.5kPa. Such a pre-consolidation pressure indicates that in some tests the samples would be over-consolidated, whilst in others they would be normally consolidated. The role of over-consolidation ratio in the bi-linearity of the clay failure envelope has already been discussed in Section 2.7.2.5 and much work has been conducted on the effect of over-consolidation on clay shear strengths. Primarily over-consolidation is found to alter the behaviour of pore pressure in subsequent shear test on clays, and it is entirely logical that chalk putty (of comparable grain size) would behave similarly. Even non-cohesive materials are thought to have strengths dependent on over-consolidation history with numerous authors (Seed (1979), Nagase et al. (2000, 2004)) reporting increases in liquefaction strength for sands with over-consolidation. Any assessment of the importance of over-consolidation on chalk putties is beyond the scope of this work, but it can be argued that the variable is best avoided by using an alternative preparation technique.

In addition to stress history, using consolidation for densification introduces a time dependency. It is unclear what the role of time dependant densification is on Razoaki's (2000) work on chalk slurry ageing as the degree of consolidation (U)* could also be influential. It maybe be assumed that consolidation is fairly instantaneous (Section 4.5.2), but small time dependent changes in densification may have affected the results of the ageing studies unless the time from preparation to shearing was kept a constant.

*Glossary

Using the 'dry press' technique, negated both the proceeding problems of vibration and time dependant densification. Absence of vibration avoided fabric changes (such as aggregation) and densification by time dependent means avoided differing starting densities. Moreover, since in this study pore waters were not introduced at the preparation stage (only later in the flushing stage), the effective stress history could be more accurately controlled to give pre-consolidation stage triaxial samples a minimal and uniform stress history.

It is accepted that the compressive forces used could be high, but these were instantaneous as was the reorganisation of the particle grains within the putty on being pressed. Possible changes in grading and grain size were explored using electron microscopy. Under the electron microscope, although sample densification was visible, there was little evidence of grading and grain size change on the samples before and after pressing into the split form mould.

A further limitation of the 'dry press' technique may have been a density variation through the sample from top to base. By compacting into 10 layers, the density changes could be reduced compared with a sample compressed in a single action. Any shear plane developing at an angle θ to the horizontal would run oblique to the scarified joins (between layers) making failure 'blind' to the effects of inter layering.

5.4.2 Discussions of back pressure 'B' value tests

The recommended value of Skempton's pore water coefficient (B) to achieve acceptable saturation B > 0.95 is based on empirical studies carried out on clay and sands. No values were found in the literature for materials such as chalk putty. Section 4.8.2 draws attention to the difficulties in achieving the B > 0.95 condition. Reference to Skempton's equation (5.1) can be used to explain why B values are lower in chalk putties compared to other soils. Skempton establishes, through equation 5.1, that B is inversely proportional to sample porosity, i.e. soil samples with higher void ratios give lower B values.

$$B = \frac{\Delta u}{\Delta \sigma_3} = \frac{1}{1 + n\frac{Kaw}{Ks}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \text{equation 5.1}$$

Where:-

B = Skempton's pore pressure coefficient

n = porosity

K_{aw} = compressibility (Bulk modulus) of pore fluids water and air

K_s = compressibility (Bulk modulus) of soil skeleton

$\Delta\sigma_3$ = change in confining pressure

Δu = pore pressure

High porosities were found in both intact Culver and Newhaven Chalk (Table 4.1, 32.0% and 38.2% respectively). Using electron microscopy, little of this initial porosity was seen to be lost even after milling for 2 minutes in the Tema mill. Figures A4-1 to A4-3 (Appendix Four) show little degradation of occluded and connective porosity which is a function of the open skeletal coccolithic structure reviewed in 2.2.1. The ability of Chalk to keep this high porosity is further confirmed by initial void ratio calculations (e_{flush}). Values of 0.5% are consistent with the findings of Hough (1957) on sands, in Lamb and Whitman (1969). Unlike sands however, a significant portion of the porosity in chalk putties is occluded or becomes connective only at higher back pressures, when pore waters enter occluded cavities. Information on the use of back pressure in a triaxial testing procedure for chalk putties is limited. Recent studies (Xia and Hu (1991), Okamara and Soya (2006), and Raghunandan and Juneja (2011)) have suggested that back pressures (especially when high) are instrumental in changing soil fabric and often influence final strength parameters. Back pressures used in this study are twice those used in other similar studies on clay and silt soils (Ng, 2007), suggesting that the testing methodology developed here may need further modification with regards to the saturation stage procedure. This is discussed further in Chapter 6 under ‘Future research’.

5.4.3 General permeability values

Chalk putty permeabilities calculated during the permeability stages of the advanced triaxial tests are seen to be in the range 10^{-8}m/s to 10^{-9} m/s (Figure 4.13). These values are comparable to clay soils of a similar particle size. Figure 4.13 has been re-drawn below (Figure 5.2) with additional annotation. An additional line has been included to convey permeability reduction (pre-shear permeability minus post shear permeability) during the shear stages as well as visible observations of test specimen mode of failure.

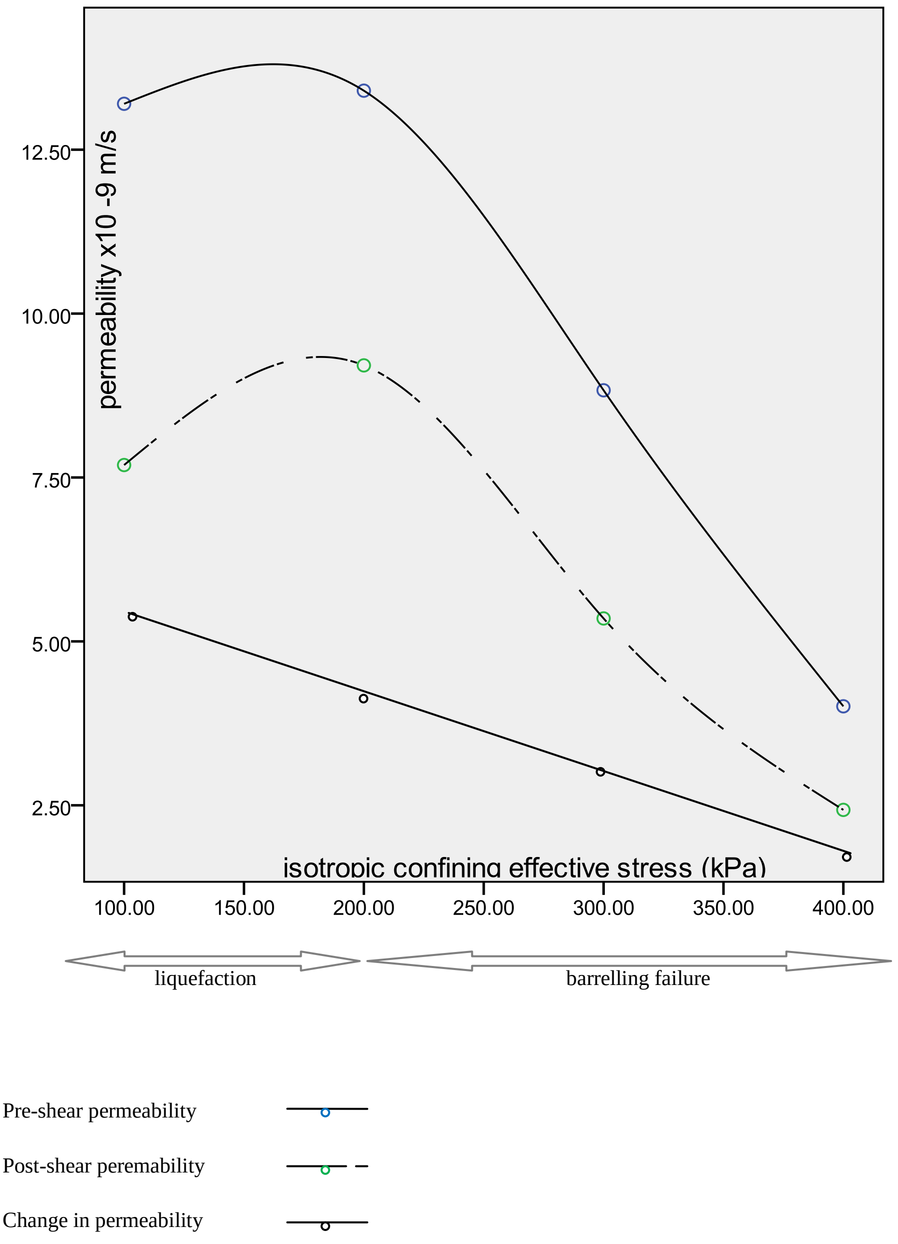


Figure 5.2 General permeability and permeability reduction

5.4.4 Permeability reduction with increased confining effective stress prior to shear

Figure 5.2 shows a clear trend of reduced permeability with confining effective stress. Traditionally reduced permeability values with increased confining effective stress were explained by the change in void ratio, porosity and / or specific volume that takes place as a sample dewaters (Lamb and Whitman, 1969), the assumption being that permeability was principally (if not solely) controlled by the drainage space or the effective porosity. The initial response of loose granular material was argued by Atkinson (1993) as being of a form of consolidation similar to that traditionally described by Terzaghi in cohesive soils. That is, initial response is controlled by dewatering of pore waters in a fashion controlled by the sample permeability.

In loose granular material Atkinson (1993) argues that irreversible volume reductions result from particle sliding as plastic deformation takes place. Studies by Clayton (1990) supported chalk putties' adoption of this behaviour. The result of repacking after particle sliding was found to result in a unique porosity which subsequently defines the compression path of the remoulded chalk putty in one dimensional consolidation as shown in Figure 5.3.

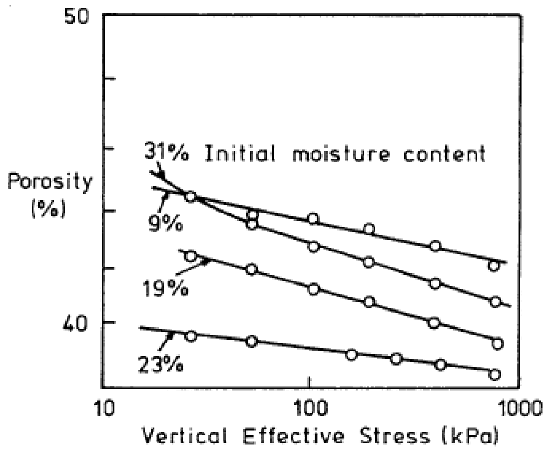


Figure 5.3 One dimensional consolidation tests on remoulded chalks with differing initial porosities (Clayton 1990)

As the Figure 5.3 shows the subsequent compression path, considered to be that of virgin compression, is linear when plotted against $\log_{10}$. Mathematically the compression paths of both cohesive soils and loose granular soils can be further expressed by the linear equation

$\nu = \nu_\lambda - \lambda \ln p'$ ………………………… equation 5.2*

where :-

λ = soil constant and represents the slopes of the compression line
ν = specific pore volume
ν_λ = specific pore volume at p' =1kPa

*After Atkinson and Bransbury (1978) and Bolton (1991). More recent studies, Sheng, et al. (2008) suggest compression paths in sands are better represented by double logarithmic curves in ln v – ln p' space.

Work by Razoaki (2000) on consolidated chalk slurries supported this linear relationship. These results, however, may have been influenced by a variation in ‘over consolidation ratios’ formed by use of a ‘consolidometer’ to form test samples (5.4.1). The linear compression path observed for chalk slurry was from a series of triaxial tests ranging from normal to overconsolidated conditions. The minimum confining pressure was 25kPa and maximum 150kPa. The ‘consolidatometer’ subjected the samples to a pre effective stress history of 87kPa.

Razoaki (2000) found dewatering during consolidation periods to be relatively rapid, suggesting primary consolidation was complete after 40 minutes (triaxial samples under effective confining pressures of 50kPa). These values were supported by typical coefficient of consolidation (C_v) values* of 75m^2/year obtained from the direct shear box test at a normal load of 100kPa. Once grain to grain contact is established, chalk putty becomes resistant to further volume reduction in a similar fashion to tangentially-locked sand, as described in the glossary. Clayton (1990) argues that any subsequent volume reduction under increasing effective confinement requires grain crushing or inter-grain bond breakage.

Numerous authors have identified that there is a range of normal compression lines in sands dependent on initial specific pore volume or sample density (for example, Jefferies and Been (2000), Sheng et al. (2008), Graham et al. (2004)). These tend to be relatively flat below confinements of 800kPa because of the resistance of quartz grains to crushing. With Chalk (made of calcite grains, 3 on the Mohs scale of hardness, compared to 7 for quartz sand grains) crushing occurs much more readily (as described in Section 5.3.2.2). This could be used to explain the form of the pre-shear permeability graph in Figure 4.13. From the figure, it is suggested that the isotropic effective stress of 200kPa, provides a boundary above which crushing and hence permeability begin to change as a result of crushing during consolidation. Triaxial samples subjected to consolidation pressures of greater 200kPa have a markedly lower permeability. The form of the pre and post-shear permeability graph also indicates that permeability, unlike porosity, is not linear to effective stress as described by Clayton (1990) and defended by Razoaki (2000). It is possible that this non-linearity is a function of non-proportionality between permeability and porosity. Permeability differs from porosity in that it is also influenced by particle size, composition, fabric and degree of saturation (Lambe and Whitman, 1969). Although composition and degree of saturation is controlled between triaxial tests, changes in particle size and fabric could well alter if crushing begins to occur.

*Calculated by use of Taylor root time method recommended for compression graphs covering short time periods (Craig, 2008). C_v values from triaxial consolidation stages could not be calculated as applied confining pressure was not applied instantaneously but applied using a rapid ramping technique (Toll, 2002).

5.4.5 Permeability reduction on yield

With all tests, permeability is seen to reduce on yield. This can be explained by a rearrangement of particles to a denser state, and grain crushing as deformation occurs. The amount of permeability reduction during yield is seen to decrease linearly with effective confining stress as shown in Figure 5.2. It would appear that less restructuring takes place on yield at the higher effective confining stresses. This would be logical, as the samples are already at a denser state because of increased confinement and so there is less scope for further densification. The fact that permeability reduction appears little changed (or even linear) with confining pressure suggests that the failure mode between liquefaction and barrelling may be transient. This is surprising as the process of liquefaction is presumed to be instantaneous (Davies et al. no date).

5.4.6 Susceptibility to liquefaction

Liquefaction is a term which describes the sudden loss of strength of a soil when loaded. Originally, dynamic forces were considered necessary and it was thought likely to occur only in clean sands. It was also considered a process that existed in continuous undrained conditions.

These assumptions, however, have now changed. Studies such as Rogers and Figuers (1991) and Ishihara (1985), show that much finer material (such as mine and quarry tailings), formed of non plastic clay-size crushed rock particles, are also susceptible. The association with dynamic forces is considered unnecessary with descriptions of static liquefaction, (where collapse of soil strength is triggered by static loads) by de Jager et al. 2008 and Davies et al. (no date).

It can be demonstrated that many of the properties of chalk putty suit the criteria for susceptibility to liquefaction. Liquefaction is seen to be more prevalent in soils with:-

i) Uniform grain size. In soils with wide grading, pores are filled (low void ratio) with smaller particles interstitial of bigger particles. The tendency for densification and pore water pressure development (liquefaction) is thus reduced. The chalk putties of this study, however, are seen to exhibit a relatively uniform grading Section 4.3, so increasing susceptibility to liquefaction.

ii) Low clay content. Clay soils are considered to behave plastically owing to van der Waal forces between platy clay particles, and thus resist liquefaction.

Seed et al. (1983) after studying case histories by Wang (1979) suggested clay contents below 15% are required for liquefaction. Impurities are extremely low in the chalk putties (Table 4.2), with particle size analysis of the impurities (Figure 4.1) suggesting the clay content is insignificant. The low clay content gives rise to extremely low plasticity indices of 3.4-3.9% which encourages liquefaction.

Susceptibility to liquefaction also depends on the stress environment of the soil (Kramer and Bolton Seed, 1988). At constant confining pressure, resistance to liquefaction increases with relative density. At constant relative density, resistance to liquefaction increases with confining pressure. This second case (constant relative density and increased confining pressure) is demonstrated in the advanced triaxial tests of this study. All advanced triaxial samples were prepared to the same initial void ratio / relative density. Whilst tests below 200kPa confining pressure demonstrated a liquefaction failure, those at a higher confinement did not. It is entirely logical that the looser the initial packing state, the greater the susceptible to volumetric strain, resulting in void reduction and pore water pressure increases.

It may appear illogical that in the higher permeability tests (confining pressure below 200kPa) there is a greater tendency to transfer from a drained to an undrained state. It is assumed that the inability of samples to drain is dependent on the volume of waters migrating, as well as the drainage capability of the sample. Moreover, tendency to liquefy is not expressed in the trends in the modified state parameter (Ψ) which in Table 4.4 appear relatively constant within the accuracy achieved for consistent sample preparation (demonstrated by e_{flush}). It is argued that liquefaction does not produce a higher densification of the sample during failure, rather the failure mechanism changes. It might be expected that pore collapse (during liquefaction) could lead to a greater densification; but greater densification in liquefaction is likely to be offset by a reduced effective stress as pore pressures become established during shear.

5.5 Summary

Having isolated a notable strain dependency whilst testing chalk putties under standard ring shear procedures in Chapter Four, Chapter Five has discussed possible causes of non-linearity in failure envelopes. The concluding Chapter Six highlights the impact this has in a wider geotechnical context with reference back to the abnormally wide Ø' range presented in Chapter Two.

Discussions have drawn on indicators of the causes of the transient nature of data, using particle size distribution and electron microscopy as presented in Appendix Three. A microscopic fundamental fabric change, dependent on the paleontological composition of chalk putty from coccolithospheres, (at differing stages of disintegration) has provided a basis for the discussions.

Greater linearity of the failure envelope was seen in triaxial testing under a new testing methodology, although it was considered that fabric change could still be occurring, albeit on a lesser scale. At the lower strains encountered in the triaxial test, permeability was found not to decrease linearly with isotropic confinement, indicating a change in particle size or fabric (Section 5.4.4).

Chapter Six returns to the discussion of developing an advanced triaxial testing procedure, and questions whether the use of conventional back pressures can be used on chalk putty. The next chapter questions whether the fundamental principle set out by Terzaghi (1923, 1936) can truly be applied to chalk putties.

Chapter Six
Conclusions and Recommendations for Further Work

6.0 Overview:-

6.1 Geotechnical properties of chalk putty.

6.2 Permeability

6.3 Liquefaction

6.4 Shear strength-strain dependency

6.5 An increased understanding of the transient nature of chalk putty's grading and fabric, through high strain ring shear apparatus.

6.6 Development of an advanced triaxial procedure to form test samples of known identity, fabric, grain size, void ratio and stress history.

6.7 Limitations of this study.

6.8 Future research. Progression of research into advanced triaxial re-inflation stress path studies and a review of the influence of back pressure in triaxial testing of chalk putty.

6.1 Geotechnical properties of chalk putty

The study has described a material that behaves as cohesionless inorganic ML soil*, exhibiting minimal shrinkage and low thixotropy. A narrow plasticity index enables rapid changes in consistency from plastic to liquid. Little to no dry strength typifies a low liquid limit, with dry chalk putties crumbling readily between fingers. The material, although consisting of silt to clay size particles, behaves like non-cohesive materials such as sand, maintaining form through weak tangential particle contacts, and suction under partially saturated conditions.

(*Please see glossary)

6.2 Permeability

Chalk putties of the study show low permeabilites with pre- and post- yield permeability values in the range 2.5-13 x 10^{-9}m/s at confining pressures of 0-400kPa. The amount of permeability reduction during yield was seen to decrease linearly with effective confining stress. Unlike porosity, permeability was not seen to form a linear relationship with the log of confinement. Fabric and grading changes have been identified as probable causes, suggesting that even during confinement (as opposed to shear) some particle breakage occurs.

6.3 Liquefaction

The study has shown that chalk putties may fail by either liquefaction or barrelling whist under triaxial test conditions. Like other materials susceptible to liquefaction, chalk putties exhibit a uniform grain size and low clay content. Although such a sudden loss of strength has been noted previously in field observations of chalk putties, no data have been found from analysis of the event under laboratory conditions.

The trigger for liquefaction remains little understood; it has been shown that it is not manifested in permeability reduction or change in void ratio (state parameters) over the selected confining pressures. It is suggested that the trigger is a transition from drained to undrained conditions occurring as a result of permeability changes which develop as grading and fabric evolve on shear. Davis et al. (no date) questions the narrow interpretation of the term ‘static loading’ and suggests it should be expanded to include other mechanisms that reduce effective stresses, such as periods of transient saturations. Lo and Li (2009) explain how static liquefaction (strictly an undrained event) has been observed under drained conditions in q - constant stress tests. The core of the triaxial specimen is thought to become undrained despite drained boundaries at the sample ends.

Chalk putty liquefaction is seen to be a rapid event (less than 2 minutes) coinciding with dramatic rises in pore water pressure (160kPa). This rapidity of the failure event is likely to be the reason why it has not been observed prior to this study. An advanced computer - controlled triaxial cell apparatus able to record specimen parameters at a *minimum* of 10-second intervals is necessary for its observation.

6.4 Shear strength-strain dependency

Large strain tests in the ring shear apparatus (following recommended test procedures) found non-linearity in the drained shear failure envelopes, with effective friction angles (Ø') increasing with strain. Increases of Ø' of up to 5° were recorded, with values in the range 29° - 37°. This non-linearity is explained by sample grading evolution.

6.5 Grading and fabric evolution

The ease with which Chalk breaks down into putty is demonstrated. Both the medium density Newhaven Chalk and the high density Culver Chalk readily form putties when worked, and are typical of many Southern England Chalks. Low uniaxial compressive strengths of 3.0MPa and 3.5MPa respectively (Figure 4.1) are indicative of low dry densities: 1.65Mg/m^3 and 1.78 Mg/m^3. The de-structuring of the Chalk from an intact rock to a soil-like putty is seen to be progressive and very much dependent on microstructure. Contrary to earlier findings, the continued de-structuring of the resultant putty is observed as the material is worked. Particle size distribution migrates to a mono-modial form with grading narrowing to an ultimate size of 1-2µm. Once milled (Tema mill of this study), the ultimate grain size was that of the individual laths that comprise the coccolithospheres of the coccolithophores.

Under laboratory conditions, this phenomenon presents the geotechnical engineer with the problem that the material has the potential to evolve in terms of its grading to a progressively finer material. Although index properties (BS 1377:2 (1990)) were noted to be unaffected by this grading evolution, as less working of the material occurs in the test procedure, high strain ring shear tests exhibited a distinct increase in effective strength with increased strain and normal loading.

Unlike cohesive soils, which adopt a well-defined critical state in the time period of standard geotechnical tests, this observation implies that there is no true critical state in the shear strength laboratory testing for chalk putties; it is merely transient. Such transience has similarly been recorded for granular sands which conversely move away from a unique soil fabric and critical state. Theoretically, a unique fabric within chalk putty is only achievable once all the material has broken down into its fundamental elements of 1-2µm coccoliths.

6.6 Developing testing procedures

Laboratory testing of chalk putties can prove problematic when following conventional soil testing procedures as outlined in BS1377 (1990). For example, conventional undrained, 'quick' triaxial test procedures were found to inadequately deal with test sample form, with low plasticity indices and with effective strengths that at high saturations caused irregularly shaped samples prior to shear.

As a result, the validity of published shear strength data must be questioned. The review of Chapter Two found Ø' values of between 30° and 40°. These values are wider than would normally be expected in reconstituted or remoulded material, where fabric and grading become homogenized.

To reduce possible causes of variation, an advanced triaxial testing methodology was determined. Earlier studies by Clayton (1977) and Razoaki (2000) suggest an age-dependent strengthening might in part explain some shear strength variation. Tests of this study however found no such effect, with efforts concentrating instead on lessening the effects of sample density (void ratio) variation, inconsistent saturation, grading, fabric, and stress history.

In this methodology, great care was taken to ensure that the initial void ratio and degree of saturation were controlled in samples formed from the "dry press technique" to provide so called 'identical' soil fabric specimens (de Mello 1977, Fredlund 1989, Toll 1991). It is believed, however, that chalk putties readily become 'different' soils (soils of different universes as defined by de Mello 1977 and Fredlund, 1989) during subsequent laboratory tests, when they involve the transition of energy that results in change or damage to sample fabric or particle size distribution.

For the geotechnical engineer, it becomes difficult to ascertain which shear strength parameters to use. The strength parameters reviewed in Chapter 2 have been conducted on chalk putties that are neither identical, nor defined in terms of the strain at which they have been obtained. This study has shown variations of several degrees in Ø' of two putties, formed from medium and high density intact Chalk, over large strains. Such a difference in the Ø' parameter would have significant economic and design implications emphasizing that a review of the geotechnical properties of chalk putties is long overdue. From advanced triaxial testing, a Ø' of 31° is advised for small strain conditions.

6.7 Limitation of this study

6.7.1 The effect of back pressures on liquefaction susceptibility.

Whilst the triaxial studies have been conducted using acceptable back pressure practices, as described in BS1377-7 (1990), there is increased debate as to the suitability of this approach, particularly with non-clay based soils. Xia, and Hu, (1991) conducted a series of tests to consider the effect of saturation and back pressure on sand liquefaction during dynamic cyclic triaxial loading. It was concluded that higher back pressures resulted in increased resistance to liquefaction, this being contrary to the traditional principles of effective stress as determined by Terzaghi in 1936. Although back pressures were kept constant between tests, possible effects of such back pressures on chalk putties (which readily demonstrated liquefaction at confinements below 200kPa) were considered outside of this study, principally because of equipment limitations, primarily the limited cell pressure with the Imperial College stress path cell.

Okamara and Soya (2006), and Raghunandan and Juneja (2011) further noted that the presence of air pockets and small degrees of partial saturation similarly affected the liquefaction of sands. With chalk putty saturations in the 92 - 96 % range, further errors are likely to have been incorporated in the testing methodology developed here. Assessment of the degree of error is beyond the scope of saturated soil equipment used here, but it is suggested that the quotation of shear strength parameters in literature for chalk putties should be accompanied by a saturation value. This saturation value should be offered in addition to the specification of initial void ratio (fabric and particle size distribution if available), strain and stress history, as discussed in Section 6.4.

6.7.2 The effect of back pressures in altering sample fabric

The measurement of volume expansion during saturation and contraction (Table 4.4) during consolidation indicates that the uniqueness of chalk putty fabric was not achieved as any volume change is indicative of fabric change. Xia and Hu (1991) argue that use of back pressure alters the micro-conditions of inter-particle forces. This may be visualised in Figure 6.1 in the example of particles touching at a point contact. The figure illustrates the application of a back pressure (P) on two hypothetical hemispherical particles. Since porewater at ‘x’ is not ‘free’ (being bound by particle contacts on one side) micro-forces are changed so that grains may move close together (6.1b) or as in Figure 6.1c.

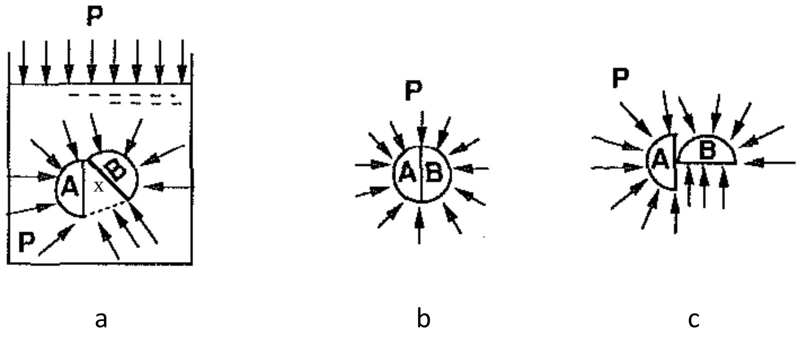


Figure 6.1 Contact changes after application of back pressure. a) Idealized soil particles with point contacts before back pressure increase, b) Interstices reduced and c) Interstices increased. After Xia and Hu (1991)

Terzaghi's effective stress principle (first presented in 1923, see also 1936) relies on water being 'free' and unbound between particles. This would only be found if all porosity was effective and not occluded (Appendix Four), and water not confined asymmetrically by irregular particle shape. Both of these conditions are improbable in chalk putties, and the void ratio changes (indicative of expansion) during saturation (Table 4.4) bear testament to the limitations of Terzaghi's effective stress principle in chalk putty testing.

6.8 Future research

6.8.1 Investigation of the effects of back pressure

The limitation of using back pressures is noted in Section 6.7. Studies show that high porosity in chalk putties even after milling is intrinsically linked to the necessity for high back pressures. Further studies should include a review of the effects of back pressures in the triaxial testing of chalk putties.

6.8.2 Re-inflation tests

The work carried out in this thesis has produced potentially significant findings within the framework of conventional geotechnical testing. Although the study presents a proven advanced triaxial testing methodology under stress control conditions, it may be prudent to reflect on the shortcomings of traditional consolidated-drained and consolidated-undrained stress paths. Just as stress control tests represent field conditions in a more realistic manner than traditional strain controlled tests (for example, Kalauger et al., 2000), it may be argued that the undrained and drained stress paths have limited field representation (Brand, (1981), Petley, et al., (2005), Ng, (2007), and Lo and Li (2009)).

Many failures (notably in slopes) occur when porewaters rise significantly after intense periods of rainfall. A rising water table may be replicated in advanced triaxial testing by a re-inflation stress path. The term Pore Pressure Re-inflation test was a term first used by Petley et al. (2005). Prior to this, Ng (2007) cites other approximations of this test methodology as “Constant-shear Drained Test (CSD)” (Anderson, 1998; Chu et al, 2003; Springman et al. 2003; Faroq et al., 2004 and Lourenco et al 2006) and “Constant Dead Load Test” (Gco, 1982; Brenner et al., 1985; Geo 1994a; Chen et al. 2000, 2004).

In a re-inflation triaxial test, the shear stage is in part replaced by a stage in which the sample is subjected to an increasing back pressure whilst deviator stress (q) and cell pressure are maintained at a constant value. Having developed suitable sample preparation, flush, saturation and consolidation stages, the shear stage is completed to a percentage (80% typically) of the full failure envelope as defined in this thesis, before proceeding to the re-inflation stage. If the foregoing procedure is followed, this would be an important step in ensuring that field conditions are replicated more accurately in laboratory testing.

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Glossary of Terms

Authigenic minerals – minerals that have crystallized in a sediment during or after its deposition.

Autotrophic organism – an organism capable of making nutritive organic molecules from inorganic sources via photosynthesis.

Biomicritic limestone - a microscopic-textured limestone composed of skeletal grains in a matrix of micrite; micrite is a finely crystalline carbonate sediment with the upper crystalline diameter being 4 microns.

Carbonate sands – material of sand grade formed commonly of fragments from carbonate secreting organisms. The magnesium content of the calcite from which they are principally made is found to vary with palaeo-seawater chemistry, Tucker et al. (1990).

Cauchy's stress tensor - a fundamental concept to the theory of linear elasticity and is applied to objects subjected to small deformations. The stress tensor represents the link between geometric vectors and a resultant stress. Used throughout many mechanical disciplines the stress tensor describes nine vectors acting on an element of soil (the minimum needed to define its stress state). These can then be reduced to three principal stresses (σ_1, σ_2, σ_3) which are sufficient alone to define the stress state. The display of the stress vectors in a two dimensional array (matrix) gives the tensor its "second order status".

Coefficient of Consolidation (C_v) – a mathematical expression used to estimate the rate of dissipation of pore water pressures or rate of settlement when a soil is permitted to drain in the vertical direction.

Degree of Consolidation (U) – the progress of consolidation in a soil subjected to a stress increment can be measured by

$$U = \frac{e_0 - e}{e_0 - e_1}$$

where e_0 is the void ratio before the start of consolidation, e_1 is the void ratio at the end of consolidation e is the void ratio at the time in question.

Diagenesis – changes which take place in a sedimentary rock at low temperatures and pressures after its deposition. They include compaction, cementation, recrystallization.

Effective stress – is equal to the total stress minus pore water pressure in soils. It is directly related to the forces between particles and intuitively will more closely correlate with soil behaviour than either total stress or pore water pressure. Terzaghi recognised the importance of effective stress in 1923 for fully saturated soils.

Formation – a geological formation is a collection of rock strata exhibiting a similar lithology.

Global stresses – those stresses read externally on a laboratory sample.

I/O card - an input output card that attaches peripheral devices to a computer. This is a predecessor to the PCI card used in this work.

Least squares fitting – a mathematical procedure for finding the best-fitting line to a given set of points so as to minimise the sum of the squares of the offset of the points to such a line. Vertical offsets (as opposed to perpendicular offsets) have been used in Section 4.6.1.

ML soil – under the British soil classification system for engineering purposes, (BS 5930), a silt soil of low plasticity.

Overconsolidation – a state achieved by a soil which has been normally consolidated to one effective stress and then re-eqilibrate to a lower effective stress. The ratio of these two stresses is known as the overconsolidated ratio.

Phase transformation – transition from compressive or contractive behaviour to dilative behaviour (in respect of volume change) gives rise to phase transformation. The line in p,q space that denotes this volume change is the phase transformation line. Some authors (eg. Lade and Ibsen, 1997) define the *phase transformation line* for undrained conditions and the *characteristic line* for drained conditions.

PCI card (Peripheral Component Interconnect) - a local computer bus that enables hardware devices (here IC air controllers) to be attached to a computer. The computer processor (under instruction from Triax) signals the PCI card to convert commands to 5 volt signals accepted by the IC controllers. The card is slotted into the mother board of the computer using a bus slot connection.

Phyllosilicates – these are sheet silicate minerals formed of parallel sheets of silicate tetrahedra, they include the clay mineral group and mica group and some other less common groups of minerals that have in common a platy form.

Pore – a term for a void used more commonly in sediments or rock, but used synonymously with void.

Pore throat – The aperture between voids, often being much smaller than the voids they connect. A controlling feature in permeability, see effective and occluded porosity.

Porosity- is the ratio of the volume of voids within a soil or rock, to its total volume. It is related to void ratio as follows:

$$e = \frac{n}{1-n}$$

where e = void ratio, n = porosity

Pozzolanic - a siliceous or siliceous and aluminous material which, in itself, possesses little or no cementitious value but which will, in finely divided form in the presence of moisture, react chemically with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties'.

Re-inflation stress path – a stress path where q is constant but u increases, simulating infiltration of soil and rising pore water pressures.

Soil fabric – a term often used synonymous with soil structure that has developed several broadly similar definitions. Fitzpatrick (1993) describes it as the arrangement, size, shape and frequency of the individual solid soil components within soil as a whole and within features themselves.

Soil structure – see soil fabric. More accurately applied to the geometric arrangement of clusters, aggregates of particles in which there is a soil fabric.

Specific volume (υ) - the total volume of soil which contains unit volume of solid soil (i.e. $\upsilon = 1 + e$). Where e is the void ratio.

Spline – used in the interpolation of data points, where use of a polynomial is considered appropriate. The subsequent curve maybe piecewise if the form of the polynomial is considered to change across the data set. A spline adopts a shape that minimizes the curvature or bend between data points.

Steady state deformation – a state in which a soil is continuously deforming at constant; volume, normal effective stress, shear stress and velocity. Poulos, (1981).

Stokes law – a mathematical expression linking the radius of a spherical particle settling through a viscous fluid with the velocity of that particle. It assumes laminar flow, that the particles are spherical with smooth surfaces and that particles do not interfere with each other when settling.

Stress path - the path followed by an element of soil in q / p' space during deformation. Used extensively in critical state theory they can be defined in terms of total or effective stresses. Numerous stress paths exist that define field scenarios, for example a σ_3 constant, q increasing stress path, simulates slope or foundation loading. See also re-inflation stress path.

Tangentially-locked sand – a sand deposit not locked by pressure solution hollows that have formed on a geological time scale, but one held by tangential forces so that a form of cohesion is exhibited.

Total stress – a stress that is shared by both a soil's skeleton and the pore fluid pressure, it is derived from the total load carried by an element of soil.

Void ratio - the ratio of the volume of voids in a soil to the total volume of solids (i.e. V_v/V_s)

Principal symbols

B	Skempton's pore water coefficient (B)
e	void ratio
G_s	specific gravity of soil particles
Ø'	effective friction angle
c'	effective cohesion, shear strength parameter
C_α	secondary consolidation
ν	specific volume
k, k_v	permeability and vertical equivalent
p'	mean effective stress
q	deviator stress
$\sigma_{1,}\sigma_3$	major and minor stresses
$\sigma_1'{}_{,}\sigma_3'$	major and minor effective stress equivalents
$\sigma_{n,}$ $\sigma_{n'}$	normal stress, effective normal stress equivalent
τ,$\tau_{f,}$	shear stress, shear stress at failure, effective equivalent
u	pore water pressure
Γ	soil constant critical state
M	ratio q'/p'
λ	soil constant, equivalent to the slope of a soil's normal consolidation line
Ψ	state parameter

nb. Symbols specific to equations follow in main text.

Appendix One
Triaxial Sample Preparation Techniques

A1.1 Triaxial sample preparation techniques for granular materials

Prior to the development of the ‘dry press’ technique for creating triaxial samples of a known density with no previous stress history, two alternative/related techniques were reviewed: a lateral shaking and sedimentation method by Barton and Brookes (1989), and a consolidation method by Razoaki (2000). Both procedures enable the preparation of dense samples of a particular grading for laboratory testing from loose material.

A1.2 Lateral shaking during sedimentation

The procedure was developed by Barton and Brookes (1989) whereby the maximum density of a granular soil could be achieved in the laboratory by using lateral shaking. Earlier attempts at creating samples using vertical vibration with the application of a surcharge (ASTM, 1964) and vibration hammers whilst sample was submersed in water (Kolbuszewski, 1948), were cited as creating samples where particle breakdown was significant. The equipment developed is shown schematically in Figure A1-1. It incorporates a D.C. motor for motion, a cam for vibration and a damping spring to smooth the operation. Amplitudes up to 0.3mm were recorded at the shaker pot with frequencies of 1-130Hz achieved using a thyristor speed controller connected to the motor. Sands primarily were studied and sedimentation was achieved by pluviation by using a funnel and pipe with a variable aperture. The funnel/pipe was supported above the apparatus using a stand.

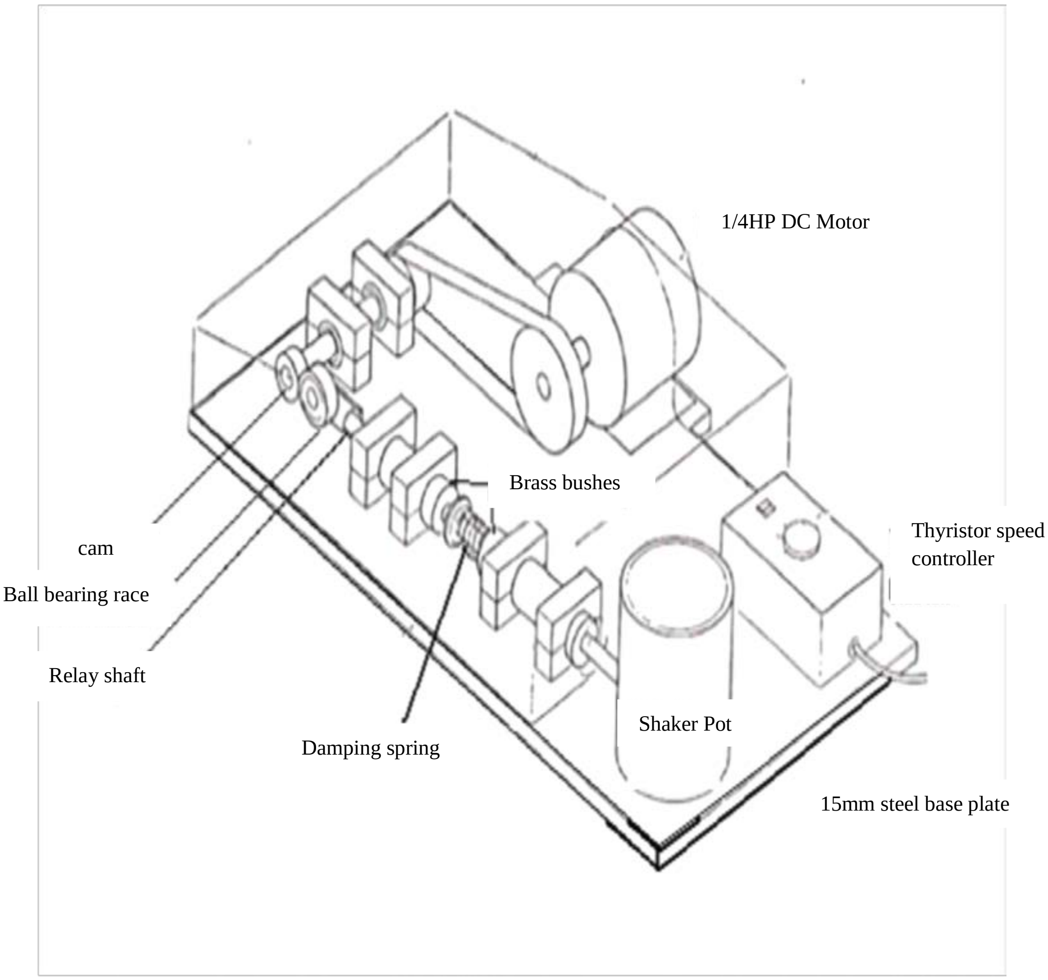


Figure A1-1 A sketch of the apparatus for producing lateral vibration. Sand is pluviated into the 'shaker pot" partly filled with water, during the vibration, after Barton and Brookes (1989)

Tests conducted on the efficiency of the technique to achieve maximum densities showed that sedimentation times of at least 10 minutes were required to achieve a maximum density. The density achieved also varied with the frequency of vibration of the shaker pot. 60Hz was considered the optimal frequency for the lateral vibration at given amplitude.

The procedure as a whole was found to be most successful on fine sands, particularly when the shaker pot was operated in a water bath, 'wet' shaking. No material finer than fine sand, was tested using the lateral shaker.

A1.3 Consolidation Method

Razoaki (2000) developed a procedure for creating chalk putty triaxial samples using the apparatus figured in A1-2. The device was named a consolidometer and consists of a perspex cylinder in which a wet (moisture content typically 28%) sample of chalk slurry was placed. The slurry was subsequently consolidated at an effective stress of 86.5kPa by the application of a 10Kg 'dead load' on the top platen. Sample drainage was allowed at the sample base and top with the whole apparatus being submerged in water. Repeatability tests determined that an average moisture content of 21% ± 0.3% was achieved after consolidation. Some difficulties with side friction were reported but these were considered insignificant, reducing the effective stress by 3kPa at the sample centre in comparison with stresses at the sample ends.

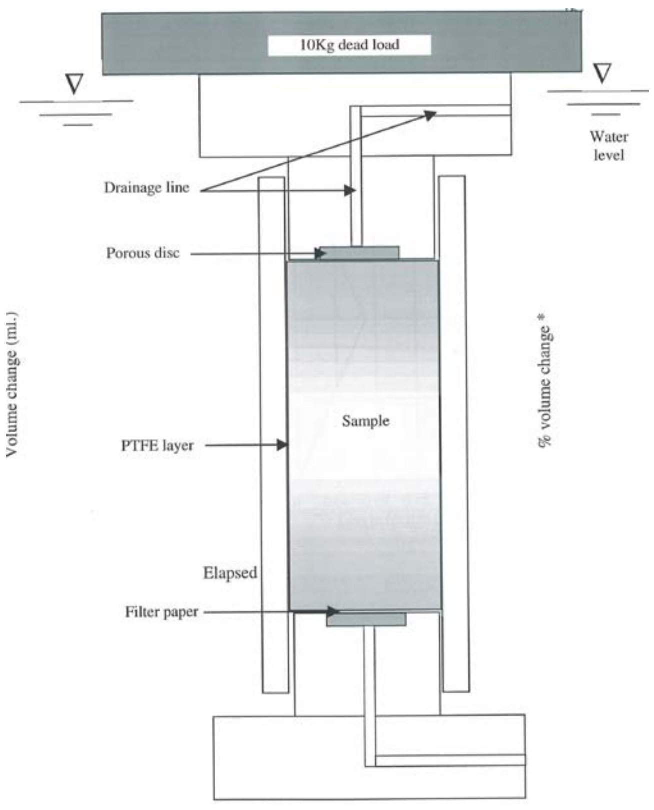


Figure A1-2 Cross-section of the Consolidometer developed by Razoaki (2000)

Although tests showed consistency between prepared sample dry densities, it was acknowledged that micro-fabrics may vary between samples. This meant that particle arrangements and interparticle contacts could be different between samples.

Appendix Two
Calibration of Porous Plates for their use in Permeability Tests

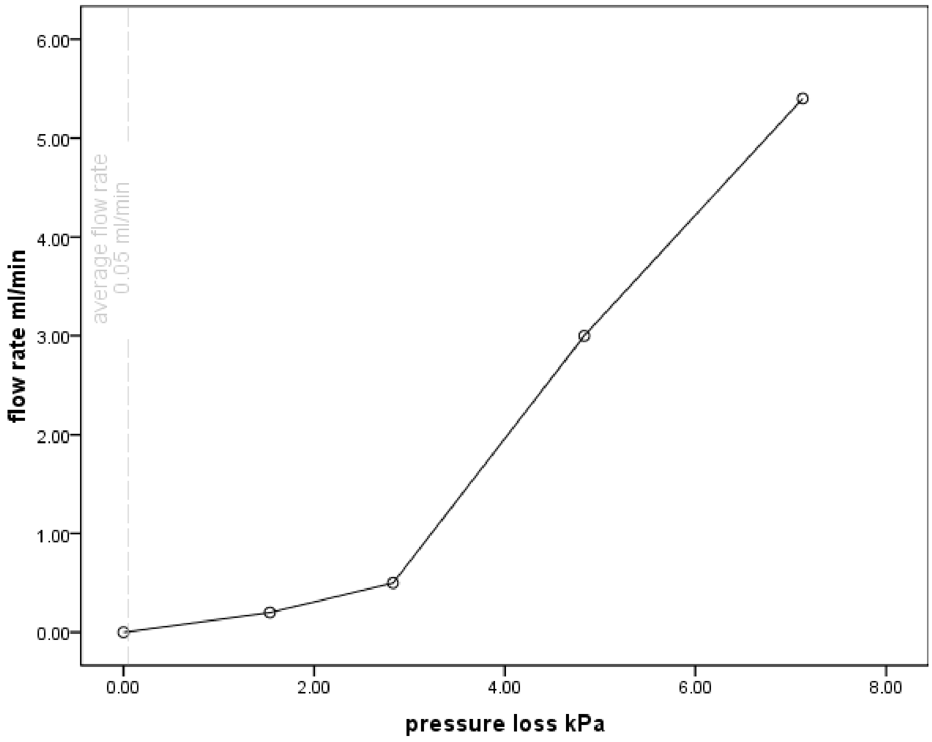


Figure A2-1 Calibration of p_c value for loss of head due to porous discs (I III) BS 1377-6:1990 section 6.4.3. Average flow rate used during permeability tests was 0.05ml/minute

Appendix Three
Laser Particle Size Analysis of Chalk Soils

A3.1 Use of suitable dispersants

A dispersing agent was used in this study in accordance with BS 1377-part 2, section 9.4.3.2**.** (1990). A dispersing agent is conventionally added to the soil prior to testing for particle size analyses in both the hydrometer and pipette method. The standard states that a solution of sodium hexametaphosphate has been found to be suitable for a very large number of soils occurring in the UK and abroad. Personal communications Kerry (1/2012) have also verified that in studies (eg. Kerry, et al. (2010)), sodium hexametaphosphate was reliably used on soil samples with a 18-35 % calcium carbonate content.

In personal communications with Michael Mörtenhumer (11/2011) of Malvern Instruments, however, it has been argued that using hexametaphosphate on calcium carbonate samples can hasten dissolution, and a more appropriated dispersant would be sodium pyrophosphate. Further, as calcium carbonate is normally partially soluble in water an additional 0.5Mol Na CO_3 solution can be added to the dispersant to suppress this dissolution.

A3.2 Analysis of the data from the Malvern Mastersizer 2000 laser granulometer

Traditionally when using the conventional pipette analysis method (BS 1377:2 clause 9, 1990), the clay/silt boundary is considered to be at the 2 µm size. For material below 63µm, particle size distributions are calculated based on the differential settlement of spherical particles settling through a liquid under gravity at different rates in accordance with Stokes' Law*. Platy shaped (phyllosilicate) minerals are seen to settle more slowly than spherical particles. This creates an over estimation of the phyllosilicates in hydrometer analysis, of which clays are the most predominant sub group.

Kerry, et al. (2010) suggest that 8µm may be a more appropriate boundary when testing clay and silt soils using modern lasers. Numerous authors are cited, Beuselinck et al. (1998), Loizeau et al. (1994), McCave et al. (1986) and Pierre et al. (2006), as concluding that laser analysis under-estimates the clay fraction significantly. The reason for this under estimation comes from the process that is used in the laser analysis. As material passes through a laser beam in the granulometer, it becomes diffracted and the scatter of light is recorded using detectors positioned at different angles. Most modern laser systems use either the Fraunhofer or Mie computer models to process the data recorded from the detectors. Both rely on the assumption of spherical particles and the random orientation of these particles as they pass through the laser beam. Clay platy particles are significantly non-spherical, and have a preferred orientation as they settle through the laser; causing the under estimation of the clay fraction in laser analysis.

Similarly Kerry et al. (2010) suggest shifting the clay / silt boundary for chalky soil analysed in a laser. It is suggested that the boundary of clay / silt size for chalky soil ought to be established at the 4 µm as opposed to the 8 µm as for phyllosillicate-rich soils.

The Mastersizer 2000 used for particle size analysis in this thesis uses Mie computer modelling. It is argued that Mie modelling improves on Fraunhofer modelling in that it takes into consideration absorption and diffusion of laser light on particles, in addition to diffraction in its analysis.

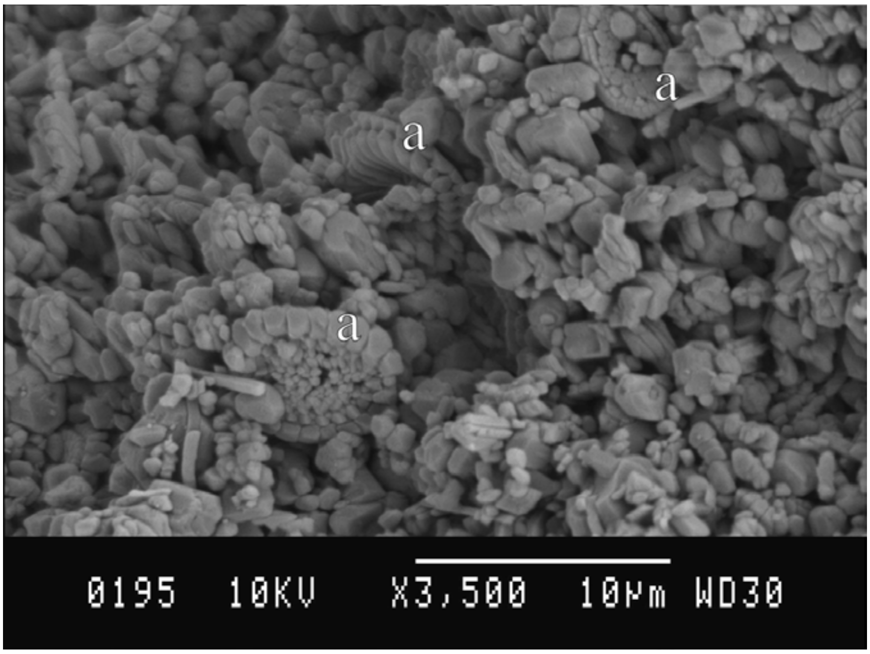
Appendix Four
Observations using an Electron Microscope

Scanning electron microscopy was carried out using a JEOL JSM-6100 scanning electron microscope on dry samples coated with gold palladium following best practices summarised in the machine’s Standard Procedure Manual. The micrographs obtained, prove useful in visualisation of fabric evolution and supporting discussions in Chapter 5.

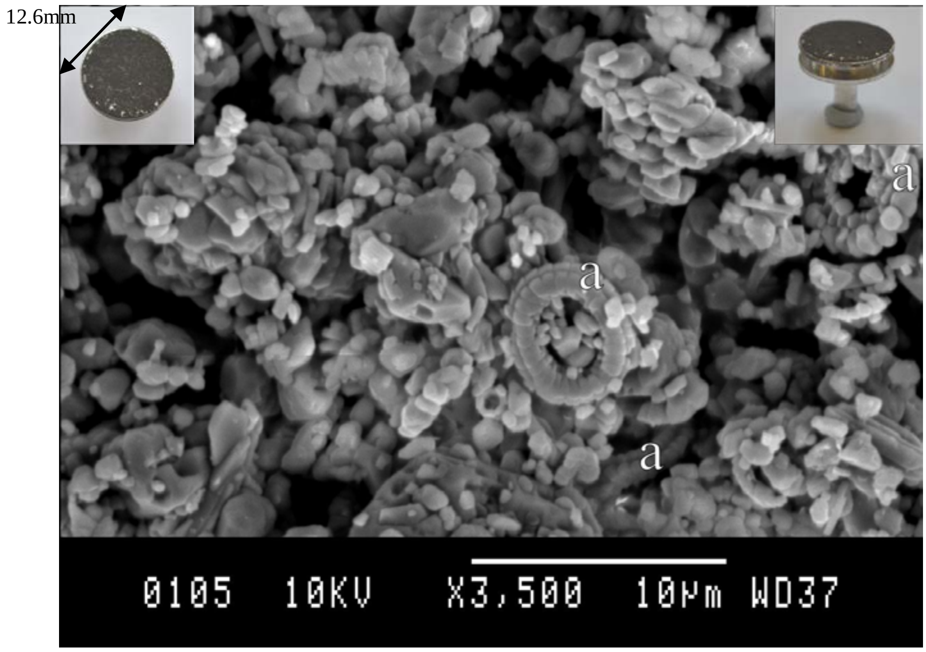
Figures A4-1a-b shows a typical comparison between intact Chalk and material milled for 2 minutes in the Tema mill. The sample shown is of Newhaven Chalk, although similar fabric was observed with Culver Chalk and putty. It is evident that in micrograph 0195 significant numbers of coccolithophore shields remain complete or partially complete in the intact sample of Chalk. The original depositional fabric of high porosity (reviewed in Section 2.2.2) is seen to remain despite subsequent diagenetic processes. After milling for 2 minutes in the Tema mill the same material is shown in micrograph 0105, again exhibiting high numbers of complete or partially complete coccolithophore shields.

Micrograph 0105 also demonstrates an aggregation fabric, found to be common in all samples of putties tested under the electron microscope. The clusters of denser material are self evident in micrograph 0105 as predicted by Rasoaki (2000) discussed in Section 5.2.1. It is important, however, to consider the sample preparation procedure used to acquire micrographs. An insert is included in figure A4-1b, to demonstrate sample disturbance during preparation. Samples are mounted on metal stubs prior to an application of a coating of gold palladium using an E5000 Bio Rad sputter coater before their insertion into the electron microscope. This sample preparation produces a fabric less dense than present in raised p' tests were grains are forced together. Further, micrographs are of material dried from a partially saturated condition. As a result fabric (in respect of packing) should be considered very different between electron microscope samples and that present in samples of raised p' tests.

Figure A4-2 shows the complexity of Chalk porosity. The coccolithophore, Prediscosphaera (Figure 2.1) is shown at its proximal end and as a side view of a detached spine (Figure A4-2). The genus (abundant in both Culver and Portsdown chalk putties) demonstrates a morphology supporting air voids that because of their size (less than 0.5μm) remain occluded to pore waters until high back pressures enable connectivity (5.4.2). Bars that cross the coccolith rosettes are seen. These fill voids further and strengthen the structure against degradation through milling and breakage.



a) micrograph 0195



b) micrograph 0105

Figure A4-1a) electron micrograph of intact Newhaven Chalk, b) electron micrograph of Newhaven chalk putty milled for 2 minutes in Tema mill

Examples of partially complete coccolithophore shields are marked ‘a’ and are typically 6-10µm in diameter. Individual coccoliths which form the coccolithophore shields are approximately 1µm in size.

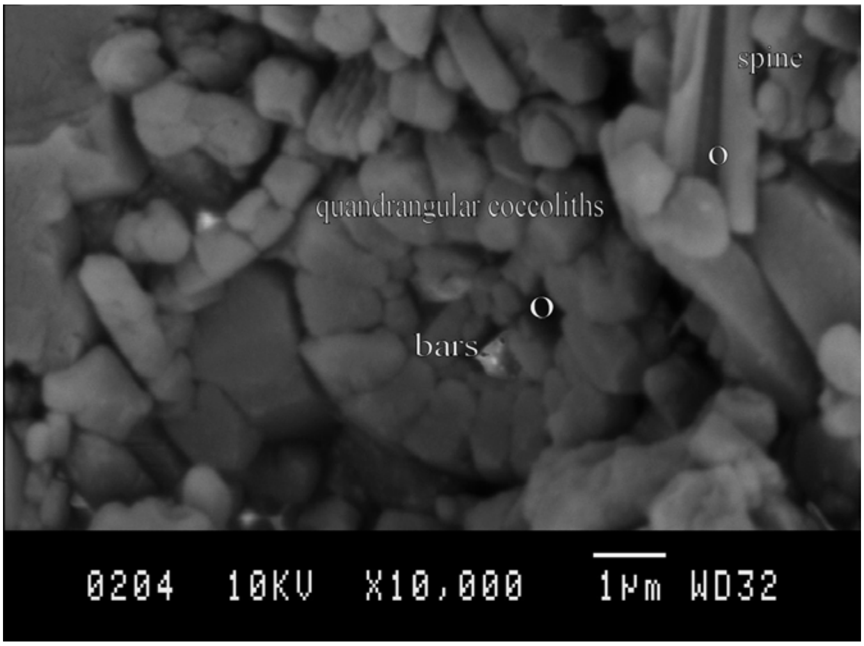


Figure A4-2 Occluded porosity created by the complex morphology of the coccolithopores that make up chalk putty.

Examples of occluded pore spaces are marked with an ‘o’

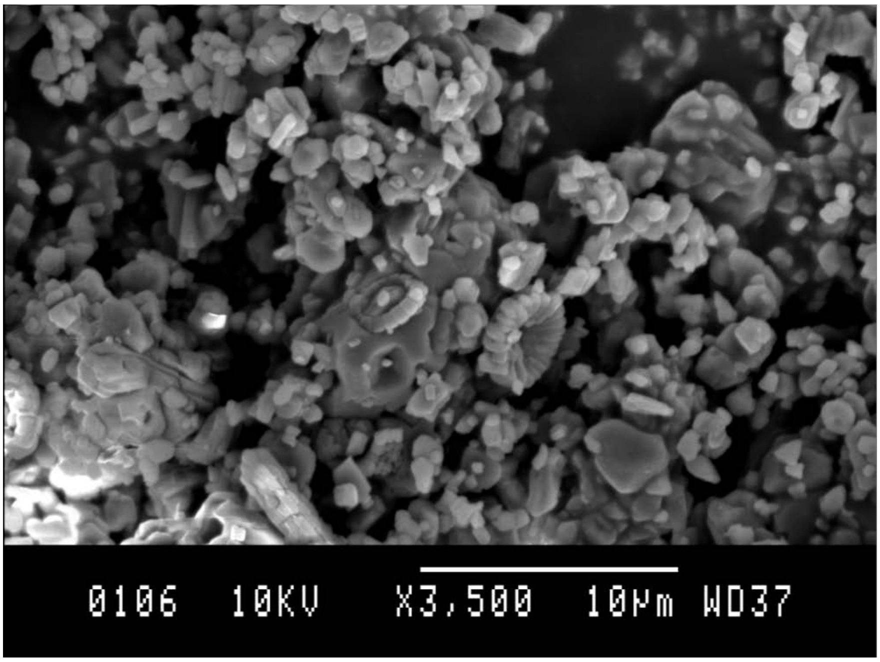


Figure A4-3 Newhaven Chalk milled for 2 minutes. Cement infilling can be seen in centre of frame

Appendix Five
The 'Triax ini.' File

The 'Triax ini.' file (below) stores all the relevant, regressions, variable definitions and stage set ups required for the stress path cell operating software to function. It is imperative that the 'ini.' file and the system set up have matching configurations for effective functioning of the stress path system. The 'ini.' or initation file is read by the operating system on start up.

```
[Main]
Version=5.1.5
Equipment="Durham Stress Path Cell"
WindowPosition=-60,-60,12120,8700
Maximised=2

[Devices]
Device1="MSL Datascan","MSL DataScan",0,"COM1",-
1,"9600,n,8,1",0,"",1,0,0,#TRUE#
Device2="Back Pressure","IC Air Valve Controller",1,"PCI-836
I/O",1,"1000007232",1,"",0,0,200,#TRUE#
Device3="Cell Pressure","IC Air Valve Controller",1,"PCI-836
I/O",3,"1000007232",1,"",.002,0,200,#TRUE#
Device4="Ram Pressure","IC Air Valve Controller",1,"PCI-836
I/O",4,"1000007232",1,"",.002,0,200,#TRUE#
Device5="CRSP","IC Piston Controller",1,"PCI-836
I/O",5,"1000007232",1,"",0,0,200,#TRUE#

[Channels]
Channel1=1,1,2,1,""
Channel2=1,2,2,1,""
Channel3=1,3,2,1,""
Channel4=1,5,2,1,""
Channel5=1,6,2,1,""
Channel6=1,8,2,1,""
Channel7=1,9,2,1,""

[Counters]
Counter3=91392068

[Regressions]
Regression1=1,10.68432,286.5826,0,0,0,0,0,"N",""
Regression2=2,-11.4126,20.2613,0,0,0,0,0,"kPa",""
```

Regression3=3,-9.49971,20.30135,0,0,0,0,0,"kPa",""
Regression4=4,-5.852786,-.7218041,0,0,0,0,0,"mm",""
Regression5=5,-.1027034,20.47246,0,0,0,0,0,"kPa",""
Regression6=6,2.065392,-1.420846,0,0,0,0,0,"cc",""
Regression7=7,2.065392,-1.420846,0,0,0,0,0,"cc",""

[Variables]
Variable1="Date",0,"Date","","dd/mm/yy",-2216,#FALSE#
Variable2="Time",0,"Time","","hh:mm:ss",0,#FALSE#
Variable3="Timer",0,"ElapsedTime","min","0.0#",59118361.2319661,#TRUE
#
Variable4="load",1,"MSL Datascan Channel 1","mV","0.0##",-
3.6450000000001,#TRUE#
Variable5="cell",2,"MSL Datascan Channel 2","mV","0.0",-1.1,#TRUE#
Variable6="back",3,"MSL Datascan Channel 3","mV","0.0",-.129,#TRUE#
Variable7="displacement",4,"MSL Datascan Channel 5","mV","0.00##",-
4.06270000000004,#TRUE#
Variable8="top",5,"MSL Datascan Channel 6","mV","0.0",6.203,#TRUE#
Variable9="volume_top ",6,"MSL Datascan Channel
8","mV","0.0##",10.619,#TRUE#
Variable10="volume_gauge",7,"MSL Datascan Channel
9","mV","0.0##",35.171,#TRUE#
Variable11="a0",-1,"(PI*d0^2)/4","mm2","0.0##",2.71,#TRUE#
Variable12="v0",-1,"a0*h0","mm3","0.0##",167.7,#FALSE#
Variable13="volume",-
1,"v0+volume_gauge*1000","mm3","0.0##",1.68,#FALSE#
Variable14="vstrain",-1,"((v0-volume)/v0)*100","%","0.0##",175,#FALSE#
Variable15="height",-1,"h0-displacement","mm","0.0##",2.65,#FALSE#
Variable16="strain",-1,"((h0-height)/h0)*100","%","0.0##",0,#FALSE#
Variable17="diameter",-1,"SQR(4*area/PI)","mm","0.0##",76.007,#FALSE#
Variable18="rstrain",-1,"((d0-
diameter)/d0)*100","%","0.0##",75.271,#FALSE#
Variable19="u",-1,"top","KPa","0.0##",.4,#FALSE#
Variable20="cell'",-1,"cell-back","kPa","0.0##",0,#FALSE#
Variable21="q",-1,"load*1000/area","kPa","0.0",38,#FALSE#
Variable22="sigv",-1,"q+cell","kPa","0.0",0,#FALSE#
Variable23="sigv'",-1,"sigv-u","kPa","0.0",.199999999999999,#TRUE#
Variable24="sigh",-1,"cell","kPa","0.0",0,#FALSE#
Variable25="sigh'",-1,"sigh-u","kPa","0.0",0,#FALSE#
Variable26="sig1",-1,"sigv","kPa","0.0",0,#FALSE#
Variable27="sig1'",-1,"sig1-u","kPa","0.0",0,#FALSE#
Variable28="sig3",-1,"sigh","kPa","0.0",0,#FALSE#
Variable29="sig3'",-1,"sig3-u","kPa","0.0",0,#FALSE#
Variable30="k",-1,"sigh'/sigv'","","0.0##",0,#FALSE#

```
Variable31="p",-1,"(sigv+2*sigh)/3","kPa","0.0",0,#FALSE#
Variable32="p'",-1,"p-u","kPa","0.0",0,#FALSE#
Variable33="t",-1,"q/2","kPa","0.0",0,#FALSE#
Variable34="s",-1,"(sigv+sigh)/2","kPa","0.0",0,#FALSE#
Variable35="s'",-1,"s-u","kPa","0.0",0,#FALSE#
Variable36="e",-1,"volume*Gs/Md/1000-1","","0.0##",0,#FALSE#
Variable37="w",-1,"((Mw-Md-
volume_gauge)/Md)*100","%","0.0",0,#FALSE#
Variable38="d0",-1,"UserInput","mm","0.0##",0,#FALSE#
Variable39="h0",-1,"UserInput","mm","0.0##",38.38,#TRUE#
Variable40="Md",-1,"UserInput","g","0.0##",76.326,#TRUE#
Variable41="Mw",-1,"UserInput","g","0.0##",150,#FALSE#
Variable42="area",-1,"a0*(1-vstrain/100)/(1-
strain/100)","mm2","0.0##",176,#TRUE#
Variable43="Gs",-1,"UserInput","","0.0##",0,#FALSE#

[Stage 1]
Description="Saturation"
Device2=#FALSE#,"back","390",390,.5,0,20,0,"0"
Device3=#FALSE#,"cell-back","10",10,.5,0,20,0,"0"

[Stage 2]
Description="Saturation hold"

[Stage 3]
Description="Isotropic Consolidation"
Device2=#FALSE#,"back","current",390.2,.5,0,20,0,"0"
Device3=#FALSE#,"cell-back","200",200,.5,0,20,0,"0"
Alarm1=#TRUE#,"vstrain>30","Stop","",""
Alarm2=#TRUE#,"vstrain<-30","Stop","",""

[Stage 4]
Description="Isotropic Consolidation Complete"
Device2=#FALSE#,"back","current",394.3,.5,0,20,0,""
Alarm1=#TRUE#,"vstrain>30","Stop","","0:0:0"

[Stage 5]
Description="Adjust ko (increase p')"

[Stage 6]
Description="Adjust ko (increase q)"

[Stage 7]
Description="Anisotropic consolidation completed"
```

```
[Stage 8]
Description="Drained Shear"
Device2=#FALSE#,"back","Current",16.8,.5,0,20,0,""
Device3=#FALSE#,"cell-back","Current",7.7,.5,0,20,0,"0"
Device4=#FALSE#,"q","current",-2.6,.5,.1,30,1,"0"
Alarm1=#TRUE#,"vstrain>30","Stop","","0:0:0"
Alarm2=#TRUE#,"vstrain<-30","Stop","",""
Alarm3=#TRUE#,"strain>30","Stop","",""
Alarm4=#TRUE#,"strain<-30","Stop","",""

[Stage 9]
Description="Ko CONSOLIDATION (direct)"

[Stage 10]
Description="Cyclic testing (sinusoidal)"
Device3=#FALSE#,"cell","Current",24.9,.5,0,20,0,"0"
Device4=#FALSE#,"q+100*(COS(2*PI*Timer/20)-1)","0",0,.5,0,30,0,""
Alarm1=#TRUE#,"vstrain>30","Stop","","0:0:0"
Alarm2=#TRUE#,"vstrain<-30","Stop","","0:0:0"
Alarm3=#TRUE#,"strain>30","Stop","",""
Alarm4=#TRUE#,"strain<-30","Stop","",""

[Stage 11]
Description=""

[Stage 12]
Description="saturation ramp controlled rate"
Device2=#TRUE#,"back","current",15.2,.5,.2,20,1,""
Device3=#TRUE#,"cell'","15",15,.5,0,30,0,""
Device4=#TRUE#,"load","5",5,1.5,0,20,0,""
Alarm1=#TRUE#,"back>=cell","Stop","13","0:0:0"
Alarm2=#TRUE#,"back>=400","Continue","13","0:0:0"

[Stage 13]
Description="saturation hold"
Device2=#TRUE#,"back","400",400,.5,0,20,1,"Steps"
Device3=#TRUE#,"cell'","15",15,.5,0,20,1,"Steps"
Device4=#TRUE#,"load","5",5,1.5,0,30,0,"Steps"
Alarm4=#FALSE#,">=","Stop","","0:0:0"

[Stage 14]
Description="B-value check by ramp"
Device2=#FALSE#,"back","400",200,.5,0,30,0,"Steps"
Device3=#TRUE#,"cell","Current",414.4,.5,.5,40,1,"Steps"
```

```
Device4=#TRUE#,"load","5",5,1.5,0,30,0,"Steps"
Alarm1=#TRUE#,"cell>=465","Stop","","0:0:0"

[Stage 15]
Description="Decreasing cell pressure to 415kPa after B-check"
Device2=#FALSE#,"back","400",200,.5,0,30,0,"Steps"
Device3=#TRUE#,"cell","Current",464.8,.5,-.5,40,1,"Steps"
Device4=#TRUE#,"load","5",5,1.5,0,30,0,"Steps"
Alarm1=#TRUE#,"cell<=415","Stop","","0:0:0"

[Stage 16]
Description="chalk isotropic consolidation"
Device2=#TRUE#,"back","400",400,.5,0,20,0,""
Device3=#TRUE#,"cell'","current",15.1,.5,.2,20,1,"Steps"
Device4=#TRUE#,"load","5",5,1.5,0,30,0,""
Alarm1=#TRUE#,"cell'>=300","Continue","17","0:0:0"
Alarm2=#TRUE#,"vstrain>=30","Stop","","0:0:0"
Alarm3=#TRUE#,"vstrain<=-30","Stop","","0:0:0"

[Stage 17]
Description="Chalk isotropic consolidation hold"
Device2=#TRUE#,"back","400",400,.5,0,20,0,""
Device3=#TRUE#,"cell'","300",300,.5,0,20,0,""
Device4=#TRUE#,"load","5",5,1.5,0,20,0,"Steps"

[Stage 18]
Description="Drained shear"
Device2=#TRUE#,"back","400",400,.5,0,20,0,""
Device3=#TRUE#,"cell'","300",300,.5,0,20,0,""
Device4=#FALSE#,"q","current",-92.8,1,.1,30,1,""
Device5=#TRUE#,"q","current",5,3,.2,100,1,""
Alarm1=#TRUE#,"vstrain>30","Stop","","0:0:0"
Alarm2=#TRUE#,"vstrain<-30","Stop","","0:0:0"
Alarm3=#TRUE#,"strain<-20","Stop","","0:0:0"
Alarm4=#TRUE#,"strain>20","Stop","","0:0:0"

[Stage 19]
Description="laterial relief"
Device2=#FALSE#,"back","40",40,.5,0,20,0,""
Device3=#FALSE#,"cell'","current",27,.5,-.5,30,1,"Steps"
Device4=#FALSE#,"sig1","1080",1080,.5,0,20,0,""
Device5=#FALSE#,"cell'","1080",1080,.5,0,10000,0,""
Alarm1=#TRUE#,"strain>3","Stop","","0:0:0"
Alarm2=#TRUE#,"strain<-3","Stop","",""
```

```
Alarm3=#TRUE#,"vstrain<-3","Stop","",""
Alarm4=#TRUE#,"vstrain>3","Stop","","0:0:0"
Alarm5=#TRUE#,"cell<100","Stop","","0:0:0"

[Stage 20]
Description="chalk permeability test"
Device2=#TRUE#,"back","385",385,.5,0,20,0,""
Device3=#FALSE#,"cell","700",700,.5,0,30,0,""
Device4=#FALSE#,"load","5",5,1.5,0,20,0,""

[Stage 21]
Description="dismantle"
Device2=#TRUE#,"back","-10",-10,.1,0,30,0,"Steps"
Device3=#TRUE#,"cell","-5",-5,.5,0,30,0,""
Device4=#TRUE#,"load","0",0,1.5,0,40,0,""
Device5=#FALSE#,"load","3",3,1.5,0,200,1,""

[Options]
DataFileExtension="*.dat"
DataSeparator="Tab"
Comments=#TRUE#
BackupStatus=#TRUE#
BackupFile="*.tmp"
BackupInterval=600
Monitor=3000
Scan=60000
Control=3000
Change=500
Calibrate=1000
Graphs=3000
Graph Maximum Points=500

[Monitor]
WindowPosition=-300,45,12060,9180
Maximised=0
View=0
Variables=1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,16,17,18,19,20,21,22,23,24,25,26,27,28,29,30,31,32,36,37,38,39,40,41,42,43
Conversion=True

[Scan]
WindowPosition=-1770,4140,12060,7800
Maximised=0
View=0
```

```
Filename="C:\Documents and Settings\triax\My Documents\RLC4.perm2.dat"
ScanInterval=600
Variables=1,2,3,4,5,6,7,8,43,43,43,11,12,13,14,15,16,17,18,19,20,21,22,23,24,
25,26,27,28,29,30,31,32,37,38,39,40,41,42

[Graph1]
WindowPosition=-60,-345,12060,7800
Maximised=2
View=0
XVariables=6
YVariables=7

[Graph2]
WindowPosition=-60,-345,12060,7800
Maximised=2
View=0
XVariables=3
YVariables=5,6,8,21

[Graph3]
WindowPosition=480,5190,5955,3625
Maximised=0
View=0
XVariables=3
YVariables=43,43

[Graph4]
WindowPosition=-60,-345,12060,7800
Maximised=2
View=0
XVariables=3
YVariables=43,43

[Control]
WindowPosition=-3660,-45,12060,7800
Maximised=0
View=0
ValveSettings=32
ActiveStage=21

[Communications]
WindowPosition=-60,-345,12060,7800
Maximised=2
View=0
```

```
[Triaxial]
Stages=2,2,3,3,3,3,3,4,4,5,5
Controllers=3,2,4,-1
Changes=20,20,30,0
ControlType=0,0,0,0
Variables=5,6,19,21,32,30,16,18,14
Tolerances=.5,.5,.5,.5,.5,.01,.005,.005,.005
```

Declaration

“Whilst registered as a candidate for the above degree, I have not been registered for any other research award. The results and conclusions embodied in this thesis are the work of the named candidate and have not been submitted for any other academic award.”

Word count: 44000

FORM UPR16

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University of Portsmouth

Please complete and return the form to Research Section, Quality Management Division, Academic Registry, University House, with your thesis, prior to examination

Postgraduate Research Student (PGRS) Information		**Student ID:**	**342676**
Student Name:	**Stephen Bundy**		
Department:	School of Civil Engineering and Surveying	**First Supervisor:**	**Dr. Paul Watson**
Start Date: (or progression date for Prof Doc students)			

Study Mode and Route:	Part-time ●	MPhil ❑	Integrated Doctorate (NewRoute) ❑
	Full-time ❑	MD ❑	Prof Doc (PD) ❑
		PhD ●	

Title of Thesis:	A Geotechnical Review of Chalk Putty
Thesis Word Count: (excluding ancillary data)	44000

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b) Have all contributions to knowledge been acknowledged?	**YES**
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Ethical review number(s) from Faculty Ethics Committee (or from NRES/SCREC):	
Signed: ***(Student)***	**Date:**
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Signed: Stephen Bundy ***(Student)***	**Date:17/7/2013**