Cyclic seasonal effects on infrastructure earthworks

by

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CYCLIC SEASONAL EFFECTS ON INFRASTRUCTURE EARTHWORKS

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Slope failures may cause substantial loss of life and damage to infrastructure. In UK much of the rail network was constructed over 150 years ago, and the earth structures were not built to modern standards. As these structures get older, they have become a potential threat to the safety of transport operations. Climate conditions directly influence the behaviour and failure of slopes. The presence of trees increases the depth and extent of desiccation and cracking and may increase the permeability to greater depths, leading to greater changes in seasonal pore water pressures. Seasonal pore water pressure changes lead to corresponding cyclic changes in effective stress. The fatigue of the clay brought about by seasonal effective stress cycles is a possibility though not well established. Climate change is expected to bring extreme weather patterns to the UK in which wetter winters and drier summers will prevail, potentially giving larger cycles of stress.

Field investigations of cracks and other macro pores are carried out as part of the current study. To establish appropriate values for the change in near surface permeability of infrastructure cut slopes caused by opening and closing of cracks, field permeability experiments on a cut slope in Newbury were carried out during different seasons of the year. Appropriate near surface permeability was established based on this. A mathematical and numerical study of the influence of a single crack on permeability is presented.

To investigate the influence of cycles of effective stress associated with pore pressure changes brought about by seasonal variations in climate on the development of accumulated strain and on the strength of stiff over consolidated clay materials, a series of cyclic triaxial experiments were carried out on soil samples from railway embankments. The results from cyclic triaxial tests on undisturbed and reconstituted Gault and Lias Clay embankment fill materials are presented and analysed.

The near surface vertical permeability of the cut slope in Newbury (10^8 m/s - 10^6 m/s) is found to be at least two orders of magnitude higher than that of intact London clay (10^10 m/s) and the end of summer vertical permeability (~10^6 m/s) two orders of magnitude higher than that of end of winter (~10^8 m/s). Near surface vertical permeability varies between summer and winter possibly mainly due to cracks opening and closing. The undrained shear strength of the reconstituted Lias Clay is consistent between 42.5 kPa to 47.5 kPa. Critical friction angle of reconstituted Lias Clay varies between 24° and 27°. For Lias Clay triaxial samples, permanent axial strains are accumulated with cycles of pore water pressure. The rate of accumulated strain decreases with the number of cycles when the effective stress ratio is below that corresponding to critical state line. The stress conditions under which the sample failed (\(\phi'_{mob} = 34°\)) when pore pressure is cycled is within the range of strength obtained for monotonic tests on the undisturbed fill material (which was between 28° and 37°).
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Nomenclature

\( c' \)  Effective cohesion

\( \phi' \)  Effective shearing angle of friction

\( \phi'_{\text{crit}} \)  Critical effective shearing angle of friction

\( \phi'_{\text{mob}} \)  Mobilised effective shearing angle of friction

\( q \)  Deviator stress

\( p' \)  Mean effective stress

\( u \)  Pore pressure

\( \sigma \)  Normal total stress

\( \sigma'_1 \)  Major principal effective stress

\( \sigma'_3 \)  Minor principal effective stress

\( \varepsilon \)  Strain

\( e \)  Void ratio

\( n \)  Porosity

\( \theta \)  Volumetric water content

\( S_r \)  Saturation ratio

\( \gamma \)  Unit weight

\( \gamma_{\text{sat}} \)  Saturated unit weight

\( G_s \)  Specific gravity

\( s \)  Matric or relative suction

\( v \)  Specific volume

\( \rho \)  Mass density
\( T \)  Surface tension

\( w \)  Water content

\( k \)  Hydraulic conductivity

\( i \)  Hydraulic gradient

\( q \)  Flow rate

\( A_{cr} \)  Crack area

\( E \)  Stiffness

\( M \)  Critical state parameter

\( t \)  Maximum shear stress

\( s' \)  Average effective stress

\( \tau_p \)  Peak shear strength

\( \tau_r \)  Residual shear strength

\( \tau_{mob} \)  Mobilised post-peak shear stress

\( I_{ca} \)  Brittleness index

\( \sigma_{ij}^{net} \)  Net stress component

\( \delta_{ij} \)  Kroenecker’s delta: \( \delta_{ii} = 1 \), \( \delta_{ij} = 0 \).

\( d\varepsilon_{ij} \)  Total strain increment

\( d\varepsilon_{ij}^m \)  Mechanical strain increment

\( d\varepsilon_{ij}^h \)  Shrinkage strain increment

\( D_{ijkl} \)  Stiffness matrix component
DECLARATION OF AUTHORSHIP

I, Aingaran Sellaiya declare that the thesis entitled Cyclic seasonal effects on infrastructure earthworks and the work presented in the thesis are both my own, and have been generated by me as the result of my own original research. I confirm that:

- this work was done wholly or mainly while in candidature for a research degree at this University;

- where any part of this thesis has previously been submitted for a degree or any other qualification at this University or any other institution, this has been clearly stated;

- where I have consulted the published work of others, this is always clearly attributed;

- where I have quoted from the work of others, the source is always given. With the exception of such quotations, this thesis is entirely my own work;

- I have acknowledged all main sources of help;

- where the thesis is based on work done by myself jointly with others, I have made clear exactly what was done by others and what I have contributed myself;

Signed: ……………………………………………………………………………………

Date:…………………………………………………………………………………

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1. Introduction

1.1 Background

Slope failures may cause substantial loss of life and damage to infrastructure. Depending on the size and extent of the slipping mass effects could range from a major landslide to localised settlements along the slope. Irrespective of the causes and the scale, the results are often detrimental. Transport infrastructure has often been significantly affected by slope failures as much of it is constructed on or through earth embankments and cuttings. In the UK, there are about 30,000 km of earth structures supporting the canal, rail and road networks (Perry et al., 2001, Perry et al., 2003). Much of the rail network was constructed over 150 years ago, and the earth structures were not built to modern standards. As these structures get older, they have become a potential threat to the safety of transport operations.

Several deep seated failures in clay slopes of both railway and highway cuttings and embankments have been reported (E.g:- Skempton, 1964, Hughes et al., 2007). A significant number of modern embankments constructed using proper compaction techniques also have shown signs of less severe, shallower failures (Perry, 1989, 1991). The Highways Agency reported about 60 sudden slope failures, and in excess of 100 railway failures were reported both in embankments and cuttings in the UK following prolonged periods of heavy rain during the winter of 2000/2001 (Turner, 2001). Disruption on the UK’s railways following the Hatfield accident followed by the large number of slope failures associated with storms and floods, caused major rail routes to be closed for several days at a time while emergency repair work was carried out (Webster, 2000). Over all, failures in embankments may undermine roads and railways. Slips in cuttings may cause material to obstruct transport routes, posing risks to drivers and causing derailment of trains (Abbott et al., 2014).

Deformation of the railway lines running on top of clay embankments can also be caused by seasonal shrink and swell movements of the clay. Seasonal deformations are amplified by the presence of mature vegetation on many of these earthworks. Track deformations often require speed limits to be imposed, in addition to costing millions of pounds to remedy. Considering the financial costs and safety risks, it is necessary to understand the complex interaction of trees, climate and slopes made of sensitive clays.
Climate conditions directly influence the behaviour and failure of slopes. Rapid infiltration of water into embankment slopes during wet winters leads to high pore water pressures and slope failures (Perry, 1991). Fluctuations in pore water pressure occur in slopes in response to seasonal variations in climate (Smethurst et al., 2006). The magnitude of the annual seasonal pore water pressure change is influenced by the permeability of the slope (Nayambayo et al., 2004). If the permeability of the clay is low, seasonal changes in pore water pressure will be small, even at moderate depths. Anderson et al (1982) observed extensive desiccation cracking in motorway embankments due to evapotranspiration during the summer periods. The cracking leads to increased permeability near the surface, potentially allowing much larger changes in pore water pressure to occur.

The presence of trees increases the depth and extent of desiccation and cracking and may increase the permeability to greater depths (Anderson et al., 1982). The interaction between climate, plant roots and shrinkage cracking needs to be better understood in order to be able to determine reliably the increase in permeability and its effect on cycles of pore water pressure. As part of this thesis the effect of shrinkage cracking on the increase in permeability is investigated.

Seasonal pore water pressure changes lead to corresponding cyclic changes in effective stress. Over a number of years these cycles of stress could lead to accumulated down slope movements and ultimately to the failure of the slopes (Take and Bolton, 2011). Failure mechanisms have been suggested in which cycles of stress lead to accumulation of shear strains, particularly starting at the toe of the slope. With sufficient strain, softening of the clay occurs, eventually leading to progressive collapse. These failures require the peak strength to be mobilised before failure and the material to strain soften (Potts et al., 1997, Kovacevic et al., 2001, Potts et al., 2005). However in contrast to mechanisms which suggest failure can only occur at the end of an extreme wet season when the factor of safety will be at a minimum, slopes often fail under pore pressures that are not extreme events (Picarelli et al., 2001). This suggests the possibility that cyclic seasonal stress variations weaken the strength of the soil.

An alternative explanation borrowed from material technology could be the fatigue of the clay brought about by seasonal effective stress cycles. Fatigue may occur in any soil material irrespective of its strain softening nature. In the context of rock mechanics researchers have shown that the peak strength of the rock material is reduced by fatigue.
resulting from the cyclic axial loading of the material (Yoshinaka & Osada, 1995, 1998). As most soils are micro structured with frictional bonds between particles, aggregates, or lumps, cycling loading may lead to progressive damage of these bonds. However there has been very little work carried out in the past regarding fatigue of soil materials resulting from cyclic effective stress changes associated with seasonal changes in pore water pressure.

Climate change is expected to bring extreme weather patterns to the UK in which wetter winters and drier summers will prevail (Hulme et al., 2002, IPCC, 2018). The UK Climate Change Risk Assessment 2017 Evidence Report (CCRA 2017) presents compelling evidence that climate change may lead to increases in heavy rainfall and significantly increased risks from fluvial and surface flooding by mid-century (HM_Government, 2017). Extreme weather may increase the severity of the issues mentioned in the previous paragraphs. Greater drying will cause a greater extent of cracking as well as larger and deeper variations in stress changes. UKCIP (UK Climate Impacts Programme)’s ‘Adaptation Wizard’ is a 5-step process to help organisations adapt to climate change (Figure 1:1). It’s also guides the user to useful information, tools and resources.

![Figure 1:1: UKCIP's Adaptation Wizard (UKCIP, 2018)](image)

This thesis makes a contribution to the improved understanding of the influence of cyclic seasonal climate on the permeability and the strength of the soil in clay slopes. Winter and summer field permeability measurements at an instrumented cut slope have
been used to show the effect of cracking on the depth and extent of seasonal pore water pressure variations. A triaxial testing programme has been carried out to investigate the effects of the cyclic seasonal stress variations on the strength and accumulation of strain in clay materials under different stress cycles.

1.2 Aims & Objectives

The aim of the project was to understand the cyclic seasonal effects of climate on the cracking, permeability, deterioration and loss of strength in infrastructure earthworks.

The objectives of the project are to:

1. Critically appraise the literature on deterioration, progressive failure and fatigue of infrastructure slopes and how this is linked to permeability changes due to cracking.
2. Carry out field permeability experiments during different seasons of the year to establish appropriate values for the change in near surface permeability of infrastructure cut slopes caused by opening and closing of cracks.
3. Carry out mathematical and numerical modelling to further quantify and understand the effects of clay cracking on permeability.
4. Carry out laboratory triaxial experiments to investigate the influence of cycles of pore pressure caused by seasonal variations in climate on the development of accumulation of strain and on the strength of stiff over consolidated clay material from railway embankments.
5. To draw experimental results together and draw conclusions for future practice.

1.3 Organisation of the thesis

Chapter 2 reviews prior relevant research. It describes the field evidence for seasonal pore water pressure changes, and for increased permeability near the surface. It then discusses the effect of permeability on slope failures and describes seasonally induced slope failures. The mechanism of strain softening and progressive failure is discussed in detail. The concept of fatigue of clay material due to cyclic seasonal changes in stress is discussed.

Chapter 3 reviews previous research on the effect of cracking on permeability.
Chapter 4 describes the field investigations of cracks and other macro pores carried out as part of the current study. The investigation of the effect of cyclic seasonal changes on clay slopes especially the influence of cracking on permeability are also described.

A series of field permeability measurements carried out on a cut slope in Newbury is described and analysed. A mathematical and numerical study of the influence of a single crack on permeability is presented.

Chapter 5 describes the investigation carried out into the effect of cycles of pore water pressure on the strength of clay material. A description of the triaxial testing programme and the experimental set up is given first. Then the results from a series of monotonic tests carried out on Lias Clay is presented. The results from cyclic triaxial tests on Gault and Lias Clay embankment fill materials are presented and analysed in the final section.

Chapter 6 draws attention to synthesis of literature review and discusses the key findings from the chapter 4 and chapter 5.

Chapter 7 summarises the conclusions achieved from this study programme. Recommendations for future research programmes are also presented.

Chapter 8 lists the Bibliography of the references made in this thesis.
2. Cyclic seasonal effects on slopes

2.1 Strain softening and progressive failure

London Clay is predominantly overconsolidated because of its history of loading. In addition, it could be asserted that any well compacted infrastructure slopes of clayey material are overconsolidated and brittle. Failure of brittle material occurs in a progressive manner (Terzaghi and Peck, 1948, Taylor, 1948). A background to the failure mechanism is given in detail in the following paragraphs.

When a soil continues to undergo shear deformation after reaching its peak strength, the resistance rapidly drops (Taylor, 1937). The lower bound of strength is defined as the residual strength (Skempton, 1964). Residual strength is applicable to slope stability analyses of natural slopes and excavations in stiff fissured overconsolidated clays that had experienced previous failures (Skempton, 1964).

In slopes, shear stresses first locally reach the peak shear strength of the material, typically at the toe of the slope. This will lead to a localised failure before the slope fails as a whole. This local shear failure can progress into the slope in long term, ultimately leading to a total collapse of the slope. This failure mechanism was recognised by Terzaghi & Peck (1948) and Taylor (1948). It was not, however, until the 1960s that it was clearly understood and discussed in the context of overconsolidated clays and clay shale (Skempton, 1964, Bjerrum, 1967, Bishop, 1967).

Progressive failure refers to the non-uniform mobilization of shear strength along a potential rupture surface. If a brittle soil is loaded non-uniformly, some elements of the soil will reach peak strength before the rest, and a rupture surface will develop. With further loading, the post-peak strains within the failure surface increase, and strength reduces from the peak towards residual. Movement continues until equilibrium between shear stresses and strains is reached. If such equilibrium cannot be obtained, the process will continue. Final collapse of the soil mass occurs before the rupture surface has fully developed. At collapse, part of the rupture surface has formed and lost strength post-peak, and a part has not yet formed. Thus, the average strength of the soil mass at collapse (the operational strength) must be less than the peak strength and greater than the residual strength.
Conditions necessary for Progressive failure

The conditions necessary for the development of progressive failure are briefly summarised below (Leroueil, 2001, Bishop, 1971):

(a) Brittleness of soil.
(b) Non-uniformity in the distribution of shear stresses.
(c) Local shear stresses that exceed the peak strength of the soil.
(d) Boundary conditions causing strains exceeding that needed to cause failure.

Brittleness is a characteristic of most natural soils in their over consolidated range. In addition, owing to the geometry of the problem, shear stresses are generally not uniform in a slope, and in particular along a potential failure surface. As a result, progressive failure plays a major role in the stages before the ultimate failure of slopes.

Development of the concept of Brittleness

Bjerrum (1967) particularly emphasised the importance of bonds and weathering, and of the possible release of stored energy, in the process of progressive failure. Brittleness, which constitutes a major factor in the development of progressive failure, is characterised by the brittleness index, IB, as follows (Bishop, 1967):

\[ I_B = \frac{\tau_p - \tau_r}{\tau_p}, \% \]

where, \( \tau_p \) and \( \tau_r \) are the peak and residual strengths respectively, defined under the same effective normal stress.

To characterise the vulnerability of a soil to progressive failure, the rate at which the strength decreases from peak strength to ultimate strength is important. A generalised brittleness index, \( I_{GB} \), is defined as follows (D'Elia et al., 1998):

\[ I_{GB} = \frac{\tau_p - \tau_{mob}}{\tau_p}, \% \]
where, \( \tau_{\text{mob}} \) is the mobilised post-peak shear stress at the considered strain or displacement.

Hence, with respect to slope failure, \( I_{GB} \) becomes associated with stress paths that are representative of those followed in situ, and not to be seen as a fundamental characteristic of a soil. With this extended definition, not only overconsolidated clays, clay shale, sensitive clays, residual soils and loess can be seen as brittle, but also cohesionless soils such as loose sands in undrained conditions (as suggested by Sladen et al., 1985).

**Fully softened strength**

The rate of the post peak fall in strength influences the failure behaviour of a soil. Skempton (1985) proposed that the drained shear strength drop from peak to residual for over consolidated clay may occur in two stages: (i) At relatively small displacements, the strength decreases to a ‘fully softened’ value, owing to an increase in water content (dilatancy); (ii) after significantly larger displacements, the strength falls to the residual value, owing to reorientation of platy clay minerals parallel to the direction of shearing (Figure 2:1). The post-peak drop in strength of normally consolidated clay is due only to particle reorientation.

Originally, the fully softened shear strength was considered to apply primarily to slope failures in stiff fissured overconsolidated natural clay and shale deposits as a result of monotonic shear strain. However, subsequent research suggested that repeated wetting and drying can also reduce the strength of compacted high plasticity clays and shale to the normally consolidated, or fully softened, state (Gullà et al., 2006, Wright et al., 2007).
Potts et al (1990) suggested that for progressive failure the most influential part of the post-peak curve is the initial section. In a finite element analysis, they assumed a post peak stress path for the Yellow Clay (a clay head deposit) from Carsington. It was based on the results of shear box tests carried out on intact clay. Tests on specimens of different thickness indicated that specimen thickness had an insignificant effect on the curve. In the finite element analysis, the stress path was assumed to be linear from peak to residual strength. Parametric finite element studies indicated that decreasing the rate at which the yellow clay softens with post-peak strain, decreases the amount of progressive failure and increases stability to a significant degree. Strain softening behaviour was introduced into the numerical model by allowing the angle of shearing resistance and the cohesion intercept to vary with the deviatoric plastic strain.

Later numerical analyses adapted the above model (Potts et al., 1997), and the parametric study carried out by Kovacevic (1994) indicated that the rate at which strain-softening occurs, when varied over a realistic range, does not have a major influence on the results obtained. But alternative independent analyses contradict this conclusion (Ellis and O’Brien, 2007).

Ellis and O’Brien (2007) used a model that allowed the strain-softening behaviour to be defined by peak, ‘post-rupture’ and residual strengths (Figure 2:2). This provides a more realistic initial rapid reduction from the peak to the post-rupture strength, followed by a more gradual reduction towards residual strength. Increasing the rate of strain-softening reduced the failure slope height as the soil became more brittle. Progressive failure increases with an increasing rate of strain-softening. This fits with the first observation by Potts et al (1990).
2.2 Influence of progressive failure in delayed collapse of cuttings

The average operational strength observed at collapse in several deep seated delayed cutting failures have been reported to be significantly less than the peak strength (Potts et al., 1997), and progressive failure has been postulated as the probable cause of this. Numerical modelling carried out by Potts et al. (1997) found out these slides exhibit progressive failure with the largest shear strains first forming at the toe of the slope and then moving horizontally inward as the pore pressures equilibrate (Figure 2:3). A series of coupled finite element analyses was conducted assuming strain-softening soil with properties based on the brown London Clay. The model slope was created through excavation of a cutting and pore pressure then allowed to recover leading to delayed progressive collapse. The results of these analyses show that progressive failure is considerable, and fully explain observed field behaviour. The delays experienced in the field are also reproduced in the analyses.
Progressive failure in cuttings in London Clay is generated primarily by the high lateral stresses in the soil prior to excavation. The rupture surface spreads horizontally from the toe as the soil swells, and differs significantly from the critical circular slip surface obtained by limit equilibrium analysis. The average strength on the slip surface at failure reduces as the initial lateral stress increases, but the effect is compensated by the increasing depth of rupture (Potts et al., 1997). The time to failure is dominated by the permeability of the clay and by the magnitude of the initial in-situ lateral stresses. The hydraulic surface boundary condition, which is controlled by climate, has an important influence on collapse.

Deep seated progressive collapse is also affected by many other factors beyond pore water pressure, such as cutting height, slope angle, strain-softening and pre-yield
stiffness (Ellis and O'Brien, 2007). Ellis & O'Brien (2007) incorporated brittle material behaviour in their numerical analysis, and demonstrated the effects of different factors on the failure height of progressive deep-seated failure of cuttings in weathered London Clay. The failure height for a given slope angle is affected not only by the strength parameters, but also by the initial earth pressure coefficient, the pre-yield stiffness and rate of post-peak strain-softening.

In cuttings, a higher lateral stress in the ground prior to excavation increases passive yielding of the soil below the toe of the cutting, and hence reduces stability. Increased pre-yield stiffness enables the peak strength to be more readily mobilised throughout the soil mass before there is significant strain-softening in the areas that yield first. This reduces the effects of progressive failure. The rate of post-peak strain-softening is positively correlated with progressive failure; hence a cutting of a given height in a material with a high strain softening rate is more vulnerable to failure than in a material with a lower rate of strain softening.

The process of progressive failure has been confirmed by direct field observations, in particular by Burland et al (1977), who observed the propagation of a horizontal shear band near the base of an excavation in over consolidated Oxford Clay, and by Cooper et al (1998), who brought to failure a well-instrumented experimental cut.

The site experiment carried out by Cooper et al (1998) was undertaken in a cutting at Selborne. Failure of the cutting was created by increasing pore pressures within the slope. Observations drawn from the experiment suggested that the mechanism of failure was progressive, started rapidly from the toe and progressed into the slope with further increases in pore pressure. A similar simultaneous initiation from the top was also noted. Interestingly during the pre-collapse stage a decrease in the pore pressure in the vicinity of the slip surface occurred, irrespective of the increase in the driving pressure, and probably due to the dilation. Despite the decrease in pore pressure, a progressive reduction in average mobilised shear strength along the slip surface was apparent through an increased rate of displacement up to failure. Dilation would increase negative pore water pressure (or a tendency to dilation). Increased permeability would enable them to dissipate.
2.3 Seasonal pore water pressure changes

The pore water pressure response observed in a grass covered cut highway slope indicated seasonal variations resulting from precipitation and evapo-transpirations (Smethurst et al., 2006). The interaction between climate and pore water pressures has been observed over several years by Smethurst et al. (2012, 2006). The pore pressure and moisture content were measured within the surface drying zone in a cut slope in overconsolidated London Clay, near Newbury in England. To capture the seasonal variation, pore water pressures were measured at eight depths ranging from 0.3m to 3.5m (Figure 2:4). The observations clearly indicate that the suctions created during the dry period are fully dissipated in the subsequent winter and early spring.

![Piezometer readings at different depths](image)

Figure 2:4: Piezometer readings at different depths (Smethurst et al., 2006)

During dry summer periods, the top three tensiometers at 0.3m, 0.6m and 0.9m depths indicates that at least 90 kPa suction is reached; the maximum measurable limit of the tensiometer. However the equitensiometer (with an error of ± 5%) reading at a 0.3m depth indicated a maximum of 440kPa suction. Considering the sensitivity of the equitensiometer to the calibrated water content–suction relationship, the exact quantitative reading should be treated with caution as also mentioned by Smethurst et al. (2006). However, it indicates a suction value in the range of several hundred kPa near the surface in the summer months. As would be expected, the seasonal pore pressure variation was observed to be greatest at the soil surface, with the amplitude of
variation decreasing with depth. A similar observation was reported in a residual soil (Gullà and Sorbino, 1996), where the observed pore pressures indicated a growing phase lag of seasonal variation with depth; this is consistent with the longer seepage pathway from the drainage/infiltration boundary. Further studies have been carried out in the recent past to investigate the importance of seasonal variations of pore pressure in the context of large landslides (Leonardo Cascini, 2006, Calvello et al., 2008).

2.4 Increased permeability

Numerical modelling of a London Clay embankment by Nayambayo et al. (2004) found that the laboratory measured permeability of London Clay, typically in the order of $10^{-9}$ m/s, prevents dissipation of the suction created during the summer. In Nayambayo’s model, 6 months of summer boundary condition was induced prior to 6 months of winter boundary conditions. This study also found that to obtain an annual cyclic change in pore water pressure permeability in the order of $10^{-8}$ m/s or more is required.

The hydrological performance of an infrastructure cut slope in London Clay at Newbury, England has been investigated over several years (Smethurst et al., 2006). The measured pore water pressure variation indicated a faster seasonal response than might be expected from the typical laboratory determined permeability of intact stiff London Clay. When back analysed using numerical modelling techniques, increased soil permeability of up to three orders near the surface was found necessary to give the observed field behaviour (Rouainia et al., 2009). Shrinkage cracking of clay layers and the opening of fissures near the surface could be a possible cause for this.

A recent study identified a permeability in the order of $5 \times 10^{-7}$ m/s or greater as the minimum needed to induce seasonal pore water pressure changes on a typical railway embankment, considering the precipitation for southern England (Loveridge et al., 2010)(Figure 2:5 & Figure 2:6).
Figure 2.5: Typical cross-section used for hydrological modelling by Loveridge et al (2010)

Figure 2.6: Pore water pressure changes for range of permeabilities for slope model shown in Figure 2.5 (Loveridge et al (2010))

The change in pore water pressure monitored by Anderson & Kneale (1980) in a clay motorway embankment, from the summer of 1979 through the following winter,
indicated a faster seasonal response to storms than would be expected from the permeability typical of the intact clay. Several shallow slips were reported in well compacted modern motorway embankments (Perry, 1991). The infiltration of rainfall into the slope surface aided by clay cracking can be considered as a possible triggering factor for these shallow slides.

Following the abnormally dry summer of 1976 (Al-shaikh-ali, 1978), extensive shrinkage cracking was reported on clay motorway embankments in various parts of the UK. Vaughan et al (1979) and Symons (1979) referred to the increased pore pressure at shallow depths resulting from the ingress of water into these cracks during subsequent wet weather. Increased pore pressure leads to slope instability. Subsequently, a critical scenario for slope failure is when a long dry period is followed by heavy rainfall. This would be expected to lead to an increase in the frequency of superficial slip failures in embankments (Symons, 1979).

Observations taken within grassed embankment and cutting slopes have shown that maximum winter pore pressures in the unstable zone are equivalent to a water table at the surface and flow parallel to it (Walbancke, 1976). Numerical analyses (Vaughan et al., 2004) indicated that a higher permeability layer close to the slope surface (e.g. due to desiccation cracks) may be a contributory factor, resulting in near-hydrostatic pressure over this depth when there is significant rainfall.

Further numerical studies by Briggs et al (2003) showed only embankments founded on impermeable layers such as London Clay, and therefore not underdrained, showed higher pore water pressures, during wet winter. Embankments underdrained by a more permeable layer, such as chalk or river terrace deposits, maintained very low pore water pressures throughout the soil profile despite the wet winter and a high rate of water infiltration at the soil surface.

It is evident that desiccation cracks directly influence the hydrological performance of slopes. Considering the influence of near surface permeability on the performance and stability of the slopes, it is necessary to investigate further the vertical permeability of the near surface London Clay.
2.5 Effect of permeability on slope failures

The recovery of pore pressure following excavation is the main driving factor in progressive failure of cut slopes (Cooper et al., 1998, Potts et al., 1997). Depending on the permeability, the time taken for recovery of pore water pressure will be either long or short and will govern the time to failure (Vaughan and Walbancke, 1973). The failure of newly constructed Victorian embankments of consolidated clay was also associated with suction dissipation aided by rain water infiltration (Squire, 1880). The effect of pore water pressure has also been identified as the most influential parameter affecting recent embankment instabilities (Ridley et al., 2004). The influence of variations in hydraulic boundary conditions is not generally taken into consideration in design practice, despite acknowledgement that pore pressure changes drive the mechanism of progressive failure in slopes.

In slopes originally cut very steep, the equilibration of pore pressures results in delayed failure (Vaughan and Walbancke, 1973). Vaughan and Walbancke (1973) suggested that delayed failure of cut slopes in London Clay is primarily controlled by the rate of pore pressure equilibration which depends in turn on the permeability of the material. The permeability is influenced by the presence of fissures and discontinuities. Unloading by excavation may lead to the opening of fissures, resulting in a superficial bulk permeability much greater than the intact clay (Skempton and Henkel, 1960). Similarly, shrinkage cracks also can play a major role in increasing the bulk permeability. Bonding in the clay may help keep the fissures open even during the winter by holding the soil together (Vaughan and Walbancke, 1973).

The pore water pressure changes in many slopes are dynamic, as a consequence of being subjected to seasonal variations of climate. Both field and numerical experiments carried out to study progressive failure have simplified the experiment by isolating the slope from climate (Cooper et al., 1998), or applying a constant surface boundary condition (Potts et al., 1997). The behaviour of real slopes is more complex, as it includes the interaction between soil and climate. A detailed explanation of the interaction between soil and atmosphere is given in Blight (1997), which clearly demonstrates the importance of the seasonal boundary conditions on the behaviour of soil.
Figure 2:7: Schematic view of the soil vegetation–atmosphere interactions (after A.M.Tang et al 2017)
The complex patterns and interactions driven by atmosphere–vegetation–soil interactions play an important role in the stability of the slopes. Complexities are already significant in terms of atmosphere–soil interactions, and vegetation effects create a further dimension. Figure 2:7 summarises the interaction between atmosphere, vegetation and soil. Figure 2:8 summarises the effect of predicted climate change in the European Union on stability of slopes.

The complex interaction between climate, vegetation and soil in the context of transport infrastructure slopes is still not clearly understood. With predicted climate change suggesting dryer summers and wetter winters for the UK, it is necessary to understand this interaction, so that reliable predictions can be made about future climate impacts on slope stability.
Figure 2.8: European Union regional climate change and the potential consequences for slope failure (A.M.Tang et al 2017).
2.6 Seasonally induced slope movements

Seasonal variations of rainfall, evaporation and transpiration lead to fluctuations in pore water pressure and the water table. This will cause effective stress changes in the soil in addition to resulting shrink and swell. Cyclic effective stress changes vary dramatically in magnitude and extent depending on the severity of seasonal climate.

The seasonal movements of overconsolidated clay embankments were measured as a part of a long-term monitoring programme (Kovacevic et al., 2001). Vertical movements of the embankment top and the soil moisture deficit (SMD) are shown in Figure 2:9. The typical section of the monitored old ash and clay fill railway embankment moves in response to seasonal trends in the surface boundary condition, as indicated by the soil moisture deficit (SMD). These data indicate that the embankment is responsive to seasonal pore pressures.

Figure 2:9: Seasonal moisture cycles and track movements on an embankment at Cannon Park, London (after Kovacevic et al., 2001).
2.7 Seasonally driven progressive failure

A finite element analysis of seasonally driven progressive failure mechanism was carried out by Kovacevic et al (2001), building on the earlier work on cut slopes (Potts et al., 1997). First, embankment construction was simulated with zero pore water pressures, then the top surface boundary condition was defined to represent the end-of-summer conditions corresponding to a vegetated slope. The slope was then subjected to seasonal pore pressure cycles by alternately changing the top hydraulic boundary condition to a prescribed end-of-winter value and end of summer value, allowing time for full equilibrium with the applied surface boundary condition to be achieved by the embankment.

The slope responded to the pore pressure changes by swelling and shrinking, which continued until a global slope failure was reached during the fifteenth wet season. The displacement vectors from the fourteenth cycle of the analysis are shown Figure 2:10. The net slope movement during the cycle indicates that irrecoverable deformation occurs during the cycle of soil shrinkage and swelling. Further, as shown by the instability in the following wet season, these irrecoverable deformations can initiate a progressive failure. The reduction in strength from the peak rupture envelope due to strain softening was modelled by allowing the strength parameters $c'$ and $\phi'$ to be a function of the deviatoric plastic strain invariant. These analyses do not make any allowance for material softening driven by either fatigue or creep. Considering the complexity of embankment behaviour, further studies need to be carried out to fully understand the real life condition.
Figure 2: Finite element analysis of a) seasonal shrinkage, b) swelling, and c) resultant annual movement (after Kovacevic et al., 2001).
2.8 Fatigue of clay material due to cyclic stress changes

A possible failure mechanism for progressive failure in clay slopes under the action of seasonal pore pressure variations can be postulated in terms of the effective stress failure criterion (Picarelli et al., 2001). Wet and dry seasons characterised by periods of extended rainfall and evaporation will generate pore pressure variations during the year. Typically, not every season is an extreme event. Thus pore pressures within the slope will mainly fluctuate within the bounds of the pore pressures observed in the wettest winter on record and the driest summer. As a result, the mean effective stress experienced by an element of soil along the potential failure surface travels horizontally back and forth in \( q \) vs \( p' \) space as seasons pass (Figure 2:11). Failure can only occur at the end of an extreme wet season when the stress path is pushed sufficiently to the left to touch the rupture envelope. However as noted by Picarelli (2000), slopes often fail under pore pressures that are not extreme events.

![Stress path associated with cycling pore pressure](image)

Figure 2:11: Stress path associated with cycling pore pressure

A possible explanation for this is the loss in the strength of the soil material due to different causes. Picarelli and Di Maio (2010) state that the reasons and types of this
weakening are not yet well established and may be a result of mechanical and/or physical and/or chemical process. From a geo-mechanics point of view, the concept of fatigue of the soil material as a result of the cyclic effective stress induced on the soil element could give a better explanation.

Fatigue is a well-known process in continuum materials, where it is associated with accumulated damage due to the repetitive application of loads that may be well below the yield point. Continuous weakening of the material leads to development and propagation of cracks. Empirical means of quantifying the fatigue process and designing against it were developed by engineers in the form of S-N (stress-number of cycles) diagrams well before a microstructural understanding had been developed. A constant cyclic stress amplitude S is applied to a specimen and the number of loading cycles N until the specimen fails is determined and plotted. Figure 2:12 shows that in general at low stresses it might take millions of cycles to cause failure. In some materials, notably ferrous alloys, the S – N curve flattens out eventually, so that below a certain endurance limit $\sigma_e$ failure does not occur no matter how many times the loads are cycled.
In rock mechanics, researchers have shown that the peak strength of a rock is reduced by fatigue resulting from cyclic axial loading of the material (Yoshinaka & Osada, 1995, 1998). As most soils are micro structured with frictional contact points between particles, aggregates, or lumps, cycling loading may lead to progressive damage of these contacts.

Geomechanical applications involving clays undergoing a variety of cyclic loading regimes notably of mechanical and thermal loads, has increased in the past decades. Due to this a significant body of research has been carried out to understand the cyclic behaviour of clays. Foundations on deep soft clays, deposits underlying offshore structures and piles in soft clays are subjected to cyclic loading due to earthquakes and ocean waves.

Pore pressure changes resulting from seasonal variations in rainfall and evapotranspiration also impose effective stress cycles on the soil. However little
experimental research has been done for this cyclic phenomenon, compared with the areas mentioned in the previous paragraph.

Cyclic axial loading tests in triaxial cells leading to failure have been carried out by many researchers (Sharma and Fahey, 2003, Frost et al., 2004, Prakasha and Chandrasekaran, 2005, Ghionna and Porcino, 2006, Donohue et al., 2009, Dash and Sitharam, 2009, Eakelen and Potts, 1978). Where the test was continued to failure, pore water pressure build up caused by cyclic axial loading led to that failure. This is quite different from the proposed study programme in which the cause of failure (if it occurs) will be cycles of pore water pressure at constant deviator stress.

A hypothesis that cyclic variations of pore pressures in slopes could lead to some fatigue of the material involved and a lowering of its strength envelope was papered by Lacerda (1989). He suggested that changing the pore water pressure in cyclic tests in a way that the effective stress conditions would vary, at constant deviator stress, would simulate the field fluctuations corresponding to low and high pore pressure (Figure 2:13).

![Conceptual fatigue model](image)

Figure 2:13: Conceptual fatigue model suggested by Lacerda (1989)
The pore pressure $u_l$ is that needed to fail the soil by a monotonic increase in pore pressure, $u_o$ is the reference minimum pore pressure in the cycle, and $u_{\text{max}}$ is the maximum value of the applied cyclic pore pressure in the cyclic triaxial test. The ratio $U_c=(u_{\text{max}}-u_o) / (u_l-u_o)$ gives an idea of the degree of intensity of pore pressure cyclic loading. $U_c=1$ means rupture in just one cycle.

Santos Jr et al (1997) carried out triaxial tests as suggested by Lacerda (1989) on undisturbed samples of residual soil. The back pressure was cycled in steps with high and low values maintained alternately for 10 minutes. Pore pressure was measured at the bottom of the sample where the back pressure also applied, and owing to the limited facilities available only the global axial displacement was measured. A correlation between $U_c$ (%) and the number of cycles for failure showed a similar pattern to that of S-N curves of metals.

Figure 2.14: Relationship between $U_c$ and Number of cycles (from Santos Jr et al, 1997)
Figure 2:14 shows that the number of cycles needed to reach failure in these cyclic tests increases as $U_c$ decreases, which implies a lowering of the peak strength envelope with cyclic loading.

Permanent axial strains resulting from cycles of pore water pressure at low stresses were reported by Eigenbrod et al. (1987) from a triaxial test on a proglacial clay. Axial strain increased with the number of cycles while the strain rate decreased with the number of cycles and eventually achieved a constant value, i.e. a constant increase in axial strain with each further cycle. In the majority of tests, the number of cycles was limited to between 50 and 80. The stabilized strain rate/cycle was proportional to the ratio of the amplitude of pore water pressure cycle to the additional pore pressure required to fail the sample, relative to the initial stage. As the test apparatus had some limitations, such as the pore pressure being measured at the same end of the sample as the back pressure was applied and no local axial displacement measurement, the results could only be interpreted qualitatively.

Further studies (Eigenbrod et al., 1992) cycling the pore pressure within natural clay specimens investigated the relationships of axial and volumetric strain with deviator stress, maximum pore water pressure and stress ratio. The axial strain rates reduced linearly with time on a logarithmic scale and the deformation reached a stable value. Total plastic deformation was associated with the total length of time of loading, thereby implying a creep effect. Linear relationships were reported between the logarithm of the strain rate, and the applied deviator stress and the principle effective stress ratio during consolidation. There was no comment regarding the reduction in the peak strength envelope.

Fatigue behaviour of soil between critical and peak stress conditions was apparent in a centrifuge experiment carried out at Cambridge University (Take, 2003, Take and Bolton, 2011). The full process of the seasonally driven progressive failure of an undamaged embankment was replicated using a physical model in the centrifuge. Models of overconsolidated clay slopes were subjected to seasonal moisture changes imposed by varying the humidity in an atmospheric chamber. This boundary condition created seasonal pore water pressure variations. Digital photography and PIV was used to observe the ground displacements on the slope cross-section.
The seasonal variations in effective stress were observed to cause the soil to undergo strain cycles. In the dry season, the embankment shrunk under the action of soil suction, whereas swelling was driven by the elevated pore water pressures of the wet season. This experiment showed that seasonal pore pressure cycles produce irrecoverable downslope movements. Furthermore, average stress paths calculated by a modified Spencer analysis indicated that above-critical mobilisation of strength during the wet season was always accompanied by irrecoverable damage to the slope. Repeated excursions within this region of above-critical effective stress ratios caused the peak strength envelope to reduce until catastrophic failure occurred.

A similar experiment by Hudesk & Bransby (2009) on glacial till reported no signs of failure of this sort. Though not explained clearly, it is possible that in this experiment the above-critical strength was not mobilised. Although the current state-of-knowledge recognises the contributions of stress changes, fatigue, creep and strain-softening in the seasonally driven mechanism of progressive failure, the exact nature of these relationships remains to be determined.

2.9 Summary

Site experiments and numerical models have demonstrated that transport infrastructure slopes can fail progressively (Terzaghi and Peck, 1948, Taylor, 1948, Potts et al., 1997). It is also evident from numerical studies that cycles of pore water pressure cause the accumulation of strains and help to cause progressive failure (Nayambayo et al., 2004). Seasonal movements as a result of the wetting and drying helps to drive the progressive failure mechanism.

Field measurements suggest there are cyclic seasonal variations in pore water pressure with in the slope throughout the year (Smethurst et al., 2006, Smethurst et al., 2012). Near surface permeability of the clay slope should be significantly high for the cyclic variations in pore water pressure deep within the slope to occur (Nayambayo et al., 2004, Rouainia et al., 2009). Cracking is likely to be the cause of increased
permeability, and provide increases high enough for significant cycles of pore water pressure to occur.

Fatigue is a well-established mechanism in continuum material technology such as metals. Researchers in the past have tried to investigate the effect of cycles of stress on the strength of the soils but the methodology was quite limited (Eigenbrod et al., 1987, Eigenbrod et al., 1992, Santos Jr, 1996).

Based on the review presented in this chapter the following knowledge gaps are identified.

- There is not a clear understanding as of how the cracking influences the near surface permeability. This is investigated in chapter 3 and chapter 4 of this thesis.

- As identified above there is no clear understanding about the fatigue of soil material brought about by cycles of stress induced by seasonal changes in pore water pressure. This is further investigated in chapter 5 of this thesis.
3. Macro pores and the process of shrinkage cracking

Highly plastic clays lose moisture during the summer period due to evaporation (under the influence of dry, hot air and sunshine). This leads to volumetric shrinkage. Cracks form between adjacent shrinking volumes. Trees and the vegetation growing on slopes also absorb entrained water accelerating the shrinking process.

The following will be considered in the paragraphs below,

   a) Different types of macro pores in clay soil
   b) Mechanisms of crack formation
   c) Typical crack geometry (length, width and connectivity etc...)
   d) Effects of cracks on the bulk permeability of clay

3.1 Different types of macro pores

It was stated as early as the mid-19th century by Schumacher (1864) that “the permeability of a soil during infiltration is mainly controlled by big pores, in which the water is not held under the influence of capillary forces”. The International Union of Pure and Applied Chemistry classifies any pores with a diameter larger than 50 nm as macro pores (Rouquerol et al., 1994). However, in the context of soil mechanics anything other than the voids present within the normal soil matrix could be described as a macro pore as it is these that will have the differentiating effect on the soil structure. Macro pores can be categorised as follows (Beven and Germann, 1982):

   a) Cracks and fissures.

This type of macro pore results from volumetric shrinkage of clays due to tension. The depth and width of these cracks varies depending on the type of soil and the climatic conditions. The measured length of visible cracks has been reported as ranging between 200 - 750mm depth and 5-75mm width in motorway embankments (Anderson et al., 1982, Al-shaikh-ali, 1978), and as up to 2 m depth and 5-25 mm width in flood embankments (Dyer et al., 2009). Opening and closing of cracks occurs throughout the year with seasonal variations. However, cracks are not sealed fully and provide a preferential pathway for flow of water even at the end of winter (Li, 2009, Anderson et al., 1982).
b) Pores formed by plant roots.

These are tubular shaped pores formed by tree roots in the top soil, which also influence the permeability. They may be associated with either live or decayed roots. New roots tend to follow the channels formed by previous roots (Beven and Germann, 1982). The structure of macro pore systems associated with roots will depend on the plant species and the conditions of growth. Even in unsaturated soils, micro pore systems may be very effective in channelling water through the soil (Aubertin, 1971, Mosley, 1982). A funnelling system could be created by hollows resulting from decaying tree stumps and windblown trees. These may channel water into a network of macro pores formed by decaying roots. A system of relatively equal sized macro pores may result from grass roots (Beven and Germann, 1982). These pores have been found to be 35% of the volume of forest soil within the rooted zone, where annual dying and growing of the grass occurs (Aubertin, 1971).

c) Pores formed by soil fauna.

These include the tubular shaped channels formed by earth worms and ants, and holes formed by burrowing animals such as moles, gophers (found in North America), and wombats (found in Australia) concentrated very near to the surface. Earthworm channels of 2-10mm diameter with a frequency of 100 channels/m² at the surface were reported by Omoti and Wild (1979), of which at least 10% penetrated to a depth of 1.0 m. The frequency was found to vary between 300 to 900/m² in grassland in Netherlands (M. Hoogerkamp et al., 1983). Channels by ants are reported to be of 2-50 mm diameter and up to 1 m depth (Green and Askew, 1965). Holes formed by moles could form tubular shapes of over 50 mm diameter (Beven and Germann, 1982). Earthworm populations observed in forestry soils in different climates showed almost all soils had a population up to several hundred per square meter (Satchell, 1983). However the depth to which they penetrated was not reported.

d) Natural soil pipes.

When highly permeable poorly graded granular soils are subjected to a high hydraulic gradient, soil particles may be washed out. This is because of weaker contact forces compared with the hydraulic forces, mainly due to a lack of compaction (Zaslavsky and Kassiff, 1964). The space left by these eroded particles forms a pipe like channel.
The present study focuses mainly on the macro pores relevant to cut infrastructure slopes in stiff overconsolidated London, Clay namely shrinkage cracks.

### 3.2 Shrinkage and Crack formation

It is usually assumed in conventional soil mechanics theory that the soil is either saturated or dry. In practice, there are situations where the soil is unsaturated: that is, some of the voids are filled with water and some with air. The drying process of saturated soil is considered in the following paragraphs.

#### Shrinkage

The shrinkage characteristics of a soil can be defined in many different ways. Shrinkage is in essence the relationship between soil volume and soil water content. The properties of the clay matrix, the ability of the clay to settle or compact, the thickness of the clay bed, the load applied to the top of the soil, and the ambient temperature are some of the factors influencing the shrinkage of clay (Plummer and Gostin, 1981).

The shrinkage process can be considered in three different phases (Haines, 1923). These are:

1. **Normal Shrinkage.**
   
   In this case the total volume decrease of the clay is equal to the volume of moisture loss. The soil remains fully saturated. A saturated soil is a two-phase material, comprising solid soil particles and water. Its behaviour is controlled by the effective stress, defined as $\sigma - u$ where $\sigma$ is the total stress and $u$ is the pore water pressure.

2. **Residual shrinkage.**
   
   On further drying, the total volume of the soil still decreases, but the volume of moisture loss is greater than the volume decrease. At this stage, air begins to enter pores of the aggregates; hence the system becomes three phase (unsaturated). An unsaturated soil comprises soil particles, water and air. Unsaturated soil behaviour is controlled by two stress parameters, $(\sigma-u_a)$ and $(\sigma-u_w)$, where $\sigma$ is the total stress, $u_w$ is the pore water pressure and $u_a$ is the pore air pressure.
(3) Zero shrinkage.

At the beginning of this stage, the soil particles reach their densest configuration. On further moisture extraction, the total volume of the soil remains constant. Moisture loss is equal to increase of air volume in aggregates.

Figure 3:1: General form of the shrinkage characteristic with three shrinkage phases (after Bronswijk, 1988).

The relative ranges of the shrinkage phases vary with soil type. Clay soils can exhibit a range of shrinkage behaviours differing markedly from the idealized form given in Figure 3:1. In general heavy clay soils show normal shrinkage over a wide range of moisture contents, while light clay soils show mostly residual shrinkage.
The water content at which zero shrinkage starts is called the shrinkage limit (SL) of a soil. Below the shrinkage limit, further loss in moisture will not cause a decrease in its volume. Above the shrinkage limit, a ratio called the shrinkage ratio (SR or R) is used to represent the sensitivity of the volume of soil to a change in water content.

\[
\text{Shrinkage Ratio (SR)} = \frac{\text{Change in total volume}}{\text{Change in water content}}
\]

One of the most-used forms of the shrinkage characteristic is the relation between volumetric moisture ratio and the void ratio of the soil aggregates.

Volumetric moisture ratio and void ratio are defined as:

\[
\begin{align*}
\text{Volumetric Moisture Ratio (VMR)} & = \frac{\text{Volume of water } (V_w)}{\text{Volume of solids } (V_s)} & 3:1 \\
\text{Void ratio } (e) & = \frac{\text{Volume of voids } (V_v)}{\text{Volume of solids } (V_s)} & 3:2 \\
\text{Volumetric water content } (\theta) & = \frac{\text{Volume of water } (V_w)}{\text{Total Volume } (V_T)} & 3:3
\end{align*}
\]
Porosity \( (n) = \frac{\text{Volume of voids} \ (V_v)}{\text{Total Volume} \ (V_T)} \)  

The use of volumetric moisture ratio and void ratio is preferred to volumetric water content \( (\Theta) \) and porosity \( (n) \), because they allow for volume change of the aggregate. VMR and \( e \) can simply be converted to \( \theta \) and \( n \):

\[
\theta = \frac{VMR}{1 + e} \quad 3:5
\]

\[
n = \frac{e}{1 + e} \quad 3:6
\]

It’s possible to relate VMR to saturation ratio \( S_r \) and void ratio as follows:

\[
VMR = S_r \ e \quad 3:7
\]

**Simplified shrinkage stress and strain phenomena**

The current understanding of the shrinkage stress phenomena is summarised in the following paragraphs (after Laloui, 2010, Konrad and Ayad, 1997b). At the micro-scale the basic process behind shrinkage can be considered as a decrease in liquid pressure (an increase in suction), caused by evaporation at the interphase menisci. This suction acts as an attractive force between the components of the matrix. In the initial, saturated stage of the process the menisci are located at the external boundary of the soil body. At the macro-scale, this translates into application of suction at the boundary, a resulting increase of effective stress compression throughout the soil matrix and sample shrinkage.

For fine soils with no swelling minerals, capillary processes predominate over the mechanisms related to the adsorbed water at least over a large range of water content and associated shrinkage strains (Mitchell and Soga, 2005). Shrinkage strains then can be seen as the consequence of an effective stress increase.

The lateral tensile stress profile experienced by soil during shrinkage depends on the initial stress state, the tensile strength of the soil and the suction profile created in the soil. Based on the understanding of unsaturated soil mechanics, until air enters into the soil, the suction
(denoted by $\psi$ in Figure 3:2a) could be assumed to vary in a pattern similar to that shown in Figure 3:2a. Hence the effective tensile stress $\sigma_t$ developed would be as shown in Figure 3:2b. It should be noted that depth $D$ is not necessarily equal to $P$.

![Figure 3:2](image-url)

**Figure 3:2**: Variation of a) suction b) tensile stress with depth from the surface (modified after Konrad and Ayad, 1997b)

For unsaturated soils, the effective stress could be related to suction ($s$) and saturation ratio ($S_r$) using the following relationship (Bishop’s generalized effective stress (UKCIP, 2018, Houlsby, 1979, and Nuth and Laloui, 2008)), whose component is expressed as follows:

$$
\sigma_{ij}' = \sigma_{ij}^{net} + S_r s \delta_{ij}
$$

where $\sigma_{ij}^{net}$ is the net stress component (the difference between the total stress component and the air pressure), and $\delta_{ij}$ the Kroenecker’s delta: $\delta_{ii} = 1$, $\delta_{i\neq j} = 0$.

During drying, changes occur in the rheological properties of the soil. Because of the liquid loss the soil loses its ability to relieve the tensile forces generated by the restraint. Any additional build-up of stress is finally relieved by tensile failure and cracking (Lachenbruch, 1961). Water content and suctions are some of non-mechanical variables related to the shrinkage deformation. It is better to split the total strain increment $d\varepsilon_{ij}$ that occurs during the process of shrinkage into mechanical strain increment $d\varepsilon_{ij}^m$ and shrinkage strain increment
$d\epsilon_{ij}^h$ and analyse them separately (Peron et al., 2009b). Many studies aimed at explaining the origin of tensile stress generation during desiccation use this approach (Kowalski, 2003). This is similar to the approach used to compute the thermal stresses.

$$d\epsilon_{ij} = d\epsilon_{ij}^m + d\epsilon_{ij}^h \delta_{ij}$$  

Any soil element will develop an internal stress increment in reaction to any induced external restraint stress increment.

$$d\sigma_{ij} = D_{ijkl} d\epsilon_{kl}^m = D_{ijkl} (d\epsilon_{kl} - d\epsilon_{kl}^h)$$

Where $D_{ijkl}$ is the stiffness matrix component.

Studies considering the water retention curves of different soils obtained through wetting and drying cycles consider that drying shrinkage occurs in two different domains (Peron et al., 2009a). The first domain exists at a saturation ratio close to one, with mostly irreversible deformation occurring in this domain. The second domain exists at a decreasing degree of saturation, with a much smaller deformation, which is mainly reversible. Free desiccation tests on initially saturated slurries confirmed these two different characteristics (Peron et al., 2009b). Depending on the type of soils, (i.e. sand or clay) the amount of shrinkage in the above mentioned two domains would vary.

**Advanced shrinkage stress and strain phenomena**

The approach suggested by Bishop and Donald (2018) mentioned in earlier paragraphs was later shown not to account for the volumetric compression (often termed collapse) that can occur on increasing the water content of an unsaturated soil (wetting) at constant total stress, if the initial water content is below a certain critical degree of saturation. This critical degree of saturation depends on the grain characteristics and may be as low as 20% for coarse granular soils, 40-50% for silts and as high as 90% for clays (Google, 2018). An increase in the water content implies an increase in $u_w$, hence a reduction in $\sigma_i^\prime$, which if $\sigma_i^\prime$ really were an effective stress should result in volumetric expansion (swelling) rather than the volumetric compression (collapse) actually observed. Thus, $\sigma_i^\prime$ cannot be viewed as an effective stress, in the sense that an effective stress controls completely both the shear and volumetric behaviour of a soil; consideration must also be given to the matric or relative suction, $s = u_w - u_w$ (See
residual shrinkage mentioned under section 3.2).

The phenomena was further explained by Houlsby (1987) and Toll (1990) and later summarised by Powrie (2004). The explanation is reproduced in the paragraphs below for clarity.

The collapse of soils on wetting occurs because when the degree of saturation is low the water remaining in the soil retreats to the smallest voids, generally at the interparticle contacts. The curvature of the menisci and hence the pore water suctions are therefore high, which combined with their location at the particle contacts enables a very open structure to be maintained. As the water content is increased the water moves out into the larger pores, the curvature of the menisci is reduced and the bonding effect is lost. The open structure cannot be sustained and the soil collapses. This is an illustration of the importance of the distribution of the pore water within a soil, which - especially at low degrees of saturation - is likely to be non-uniform. Another is the tendency of compacted clay soils to aggregate, forming packets or lumps of relatively high water content with air voids in between.

Toll (Toll, 1990) presents a critical state framework for the behaviour of unsaturated soils in terms of five state variables. These are the deviator stress \( q \), the specific volume \( v \), the saturation ratio \( S_r \), and the isotropic stress parameters \((p-u_a)\) and \((u_a-u_w)\). Critical states are defined by the Equations,

\[
q = M_a(p-u_a) + M_w(u_a-u_w) \tag{3:11}
\]

and

\[
v = \Gamma_{aw} - \lambda_a \ln(p - u_a) - \lambda_w \ln(u_a - u_w) \tag{3:12}
\]

where the five soil parameters \( M_a, M_w, \Gamma_{aw}, \lambda_a \) and \( \lambda_w \) depend on the saturation ratio, \( S_r \). The dependence of \( M_a, M_w, \Gamma_{aw}, \lambda_a \) and \( \lambda_w \) on \( S_r \) must be determined experimentally. Wheeler (1991) points out that Toll’s (1990) critical state framework for unsaturated soils
cannot be used as a predictive tool, because $S_r$ (and hence the values of the parameters $M_a$, $M_w$, $\Gamma_{aw}$, $\lambda_a$ and $\lambda_w$) will change as the soil is sheared. Wheeler (1991) proposes an alternative critical state framework that overcomes this shortcoming, but with some loss of simplicity.

Houlsby (1997) shows that a stress parameter similar to $\sigma_i^*$, given by Equation 3:8, does have a physical significance in unsaturated soils as the work conjugate to the relevant strain (i.e. when the two are multiplied together, the result is the work done during an increment of strain). An additional component of work is given by a modified suction, $nS$ (where $n$ is the porosity and $s = u_a - u_w$) multiplied by the negative of the change in saturation ratio, $S_r$. Thus, a simple and appropriate form of stress parameter for an unsaturated soil is given by

$$\sigma_i^* = \left(\sigma - u_w\right) + S_r \left(u_a - u_w\right)$$  \hspace{1cm} 3:13

which is work conjugate to the strain, together with a modified suction

$$s^* = n(u_a - u_w)$$  \hspace{1cm} 3:14

which is work conjugate to the negative of the change in the saturation ratio $S_r$.

A review of the concepts appropriate to unsaturated soils is given by Fredlund (1979).

**Cracking**

The process of shrinkage and crack formation of a remoulded clay with an initial water content higher than the liquid limit is described below (after Peron et al., 2009b, Shin and Santamarina, 2011, and Shin and Santamarina, 2012). At the beginning, the supernatant water evaporates at an approximately constant rate. The effective stress in the soil is constant and the soil experiences no strain. Once the air–water interface reaches the soil surface, both continue moving downwards together. The soil skeleton consolidates owing to the increase in effective stress that equals the increase in suction ($u_c$). The instantaneous void ratio $e$ can be related to the suction (denoted by $u_c$ hereafter) as prescribed in the one-dimensional normal consolidation line NCL: $e = e_{1kPa} - \ln(u_c/1 \text{ kPa})$. Cracks initiate during this constant rate of evaporation stage. The soil is still fully saturated (degree of saturation is 100%, Peron et al.,
2009b) at crack initiation; that is the air–water interface and the apparent soil surface coincide. Hence cracking corresponds to the end of the saturated stage of drying.

At the particle-level, a lower air-entry suction $u_{c}^{AE}$ which is critical local suction for the interfacial membrane to invade into the saturated media corresponds to a higher local void ratio $e$. This can be estimated assuming parallel platy particles of thickness $t$ separated at a distance $d$ (Figure 3:4): the void ratio is $e = d/t$, and the specific surface is $S=2/(\rho t)$. Then, the air entry suction $u_{c}^{AE} = 2T/d$ (Laplace equation) can be estimated as following:

$$u_{c}^{AE} = \frac{\rho T s S_s}{e} \tag{3:15}$$

where $\rho$ [g/cm$^3$] is the mass density of the particle and $T=0.072$ N/m is surface tension of water.

During increasing suction within the sediment, the $e-\sigma$' path at the tip gradually deviates from the $e-\sigma$' path in the far field, which follows the normal consolidation line NCL (Figure 3:4). The void ratio at the tip reaches the air entry line AEL first, and air invades the soil mass at the tip while the rest of the soil remains saturated (Figure 3:4, Figure 3:3). This is the moment when the crack initiates (Childs, 1969).

Analogous to crack initiation, the void ratio increases and the air-entry value decreases at the crack tip. The air-water membrane preferentially invades the soil at the tip and the crack propagates (this crack propagation requires particles to displace). This sequence of events is in agreement with all the experimental observations carried out by Shin and Santamarina (2012) that place crack initiation at the tip of defects in soft sediments.

![Figure 3:3: Processes of crack initiation from a particle scale view (Shin and Santamarina, 2011)](image)

Figure 3:3: Processes of crack initiation from a particle scale view (Shin and Santamarina, 2011)
Figure 3: Variation of void ratio with suction during shrinkage cracking (modified from Shin and Santamarina, 2011)

In practical conditions crack initiation first occurs in the areas where drying shrinkage is constrained (Corte and Higashi, 1960). Typical causes of this constraint could be categorised as follows (Hueckel 1992):

a) Any Eigen-stress concentrations within the soil sample
b) A frictional or any other traction or displacement boundary conditions
c) Intrinsic soil inhomogeneity factors, such as soil texture and soil structure.

Eigen-stress concentrations within the soil sample occurs from soil moisture gradients, which do not respect the strain compatibility conditions (Kowalski, 2003).

Defects such as inclusions, large aggregate anomalies or small topographic features (e.g. indentations) and air-filled pores act as crack initiators in soils (Frydman, 1967, Morris et al., 1992, Shin and Santamarina, 2011, Towner, 1988, Weinberger, 1999). The increased local
void ratio at the tip of a defect facilitates air–water membrane invasion into the soil mass as suction increases in a drying soil. Air invasion is the starting point for desiccation crack formation in saturated soft soils.

Dry zones may appear directly inside water-filled pores due to cavitation (Peron et al., 2009b). When the water pressure at a point falls below the water vapour pressure at the local ambient temperature, water undergoes a phase change. This creates vapour-filled voids, termed cavitation bubbles. The saturation vapour pressure of water at ambient temperature is very low (approximately 3 kPa). Cavitation is likely to occur at a suction value greater than about 100 kPa.

Also, changes in ground water level brought about by seepage and drainage especially in the case of clay slopes could initiate cracking. Hydrostatic capillary suction is created in the unsaturated region of the clay by a drop in the water table. Tensile forces can be created by this process, which would lead to cracking of a clay.

An experimental programme carried out by Peron et al (2009a) to measure the degree of saturation and suction during the shrinkage of the clay found that in a homogeneous mix, the water content corresponding to air entry value is very close to the shrinkage limit for many soils (Peron et al., 2009a). The degree of saturation varied between 0.90 & 1.0 at the point of crack initiation (eg:- see Figure 3:5) and was near 1.0 for Ca-montmorillonite in a similar experiment by Shin & Santamarina (2011) (Figure 3:6). However as the degree of saturation is a large-scale parameter, its utilisation requires careful interpretation in the context of desiccation cracks in fine-grained soils where soil blocks between cracks are expected to remain fully saturated long after crack formation (Figure 3:3) for the case of high-specific-surface soils (Fine grained soil).
There is a critical suction value corresponding to the crack initiation differs between clay types, but could be considered unique for a given clay when the material is uniform (Peron et al., 2009a, Shin and Santamarina, 2011). This critical suction value is near 400kPa for La Frasse Clay (Figure 3:5) and also for Ca-montmorillonite (Figure 3:6).
The development of desiccation cracks has been investigated in the laboratory by some researchers through casting of clay beams and plates by pouring a small thickness (usually several centimetres) of slurry or compacting clay into a container (Lau, 1987, Shorlin et al., 2000, Nahlawi and Kodikara, 2006). In these experiments the adhesion between the clay and the container below provides restraint to the movement of clay in addition to moisture gradients (Vallejo, 2009, Costa et al., 2008). This could possibly increase the number of cracks formed.

From an idealised mechanical point of view, a crack occurs at the soil surface only when the total horizontal stress σ₃ acting on the soil surface reaches the tensile strength σₜ of the soil, (Figure 3:7c). At crack initiation, the suction acting on a soil element adjacent to the surface has reached a critical value referred to as ψᶜʳ (Konrad and Ayad, 1997b). This critical suction value depends on the tensile strength of the soil, which is very small or none for many soils. The initial stress state and the total and effective stress paths followed by a soil element at the surface during the shrinkage-consolidation process and the stress history of the soil (Abu-Hejleh and Znidarcic, 1995). The depth of crack propagation A (Figure 3:7d) for the existing total stress conditions is governed by the actual suction profile and the effective stress path.
During desiccation. A fracture toughness criterion can be used to evaluate this depth (Konrad and Ayad, 1997b).

Figure 3:7: Schematic illustration of cracking (modified from Konrad and Ayad, 1997b)

During evaporation, suction progressively increases and the soil consolidates under a condition of zero lateral strain. The total stress state moves along the $\sigma_y = 0$ line while the effective stress path follows the $K_0$ line (Figure 3:8). Surface cracks are initiated when the minor total stress $\sigma_3$ reaches the tensile strength of the soil, $\sigma_t$ (point F). The critical suction $\psi_{cr}$ is then given by the distance FF’ in Figure 3:8.

Figure 3:8: Stress path during cracking process

It is evident from experimental results that desiccation cracking mainly occurs in opening mode (i.e. mode I) (Thusyanthan et al., 2007, Peron et al., 2009b). This also proves that cracking can be considered as a result of soil tensile strength mobilization, as commonly
acknowledged. The tensile strength of the clay is substantially low (Parry, 1960). The yield and failure criteria for soils relevant for extension behaviour are very few.

The failure points of saturated Weald Clay in both drained extension tests and undrained extension tests (Parry, 1960) satisfy Hvorslev’s failure criteria (Hvorslev, 1937). In some other investigations, principles from fracture mechanics, modified Griffith theory and rock mechanics principles based on ‘true cohesion’ were used to investigate the tensile strength of clay (Bishop and Garga, 1969, Morris et al., 1992). A criterion for crack initiation based on a modification of the Mohr-Coulomb criterion in the tensile range was suggested by Morris et al (1992). In this method it is assumed that the soil experiences tensile strength and apparent cohesion (denoted $C_{app}$) increases, related to the increase of suction.

An investigation of crack initiation in clay observed in beam bending was carried out by Thusyanthan et al (2007). This study investigates the stress-strain criteria for cracking in clays by performing four-point bending tests on consolidated kaolin clay beams. Beam bending tests were equipped with embedded high-capacity tensiometers and external digital-image-based strain measurement to revisit the possibility of deriving an strength of materials (SOM) criterion for crack initiation in clay by obtaining a full record of total and effective stresses and strains leading to failure. This study concentrates on the stress–strain behaviour of the mid-span of the beam, where the change of bending moment induced by the loading was uniform at its maximum value.

In the study by Thusyanthan et al (2007), The clay to be used in the experimental programme of bending tests was chosen to be in a remoulded state in order to permit the creation of a large number of identical specimens, and to evaluate the zero tension line effective stress criterion often quoted for these materials. During consolidation in the strong box, prior to the final unloading increment from approximately 30 kPa to 0 kPa, the specimen was isolated from all sources of water to eliminate the possibility of swelling. On unloading, the clay block was therefore forced into an initial negative pore pressure of slightly less than 30 kPa. This modest initial suction was imposed on the clay to ensure sufficient stiffness to permit undisturbed beam specimens to be cut from the large block of clay. The longitudinal direction of the beam (defined as the z-direction), was chosen to be transverse to the direction of consolidation loading (Figure 3:9).
Figure 3: Specimen preparation: (a) large block of consolidated clay; (b) 320mm x 80mm x 80 mm clay beam (reproduced from Thusyanthan et al, 2007)

Two types of high-capacity tensiometer were used in the work by Thusyanthan et al (2007): the miniature stainless steel tensiometers developed by Take & Bolton (2003), and a newly updated version of the Druck PDCR-81, which had been modified to the authors’ specifications by the manufacturer to make the device more robust under tensile water pressures. The tensiometers used were about 6 mm in diameter and 10 mm long, with very flexible and slim (3 mm diameter) connecting wires. The inclusion of the tensiometers was essential to the research. The ceramic tip of a tensiometer that had just been saturated using the procedures of Take & Bolton (2003), was coated with a small amount of kaolin paste and inserted into the drilled hole, backfilled with a clay slurry behind the device, and allowed to set.
The evolution of the magnitude and distribution of the longitudinal bending strain $\varepsilon_{zz}$ at this location was precisely measured using the non-contact digital image correlation technique of particle image velocimetry (PIV). This technique is described by White (2002), Take (2003), and White et al. (2003). As PIV operates on image texture rather than predefined target markers, the displacement of any location throughout the beam could be measured.

The stress and strain criterion for cracking in E-grade kaolin clay was investigated for beams of varying pre-consolidation pressure, of varying initial negative pore pressure over the range 15–102 kPa, and tested in both load and strain control. In the simpler conditions used here the clay remained saturated throughout, permitting the application of Terzaghi’s effective stress principle in the analysis of the results.

The beams were once again covered with a protective film for an additional moisture equilibration period prior to testing. All clay beams experienced brittle failure by cracking on the tensile face in the mid-span region of the beam. The load, and hence the bending moment experienced by the beams, increased up to the onset of cracking. Shortly after the initiation of a crack on the tensile face, complete collapse occurred in the load-controlled tests, whereas in the strain-controlled tests the load was observed to decrease while the crack propagated.
vertically upwards into the beams. Complete collapse of the beams in the strain-controlled tests was observed once the crack had propagated through about two-thirds of the beam thickness.

The measured tensile strain at the instant of crack initiation in each of the 17 beam tests is presented in Figure 3:11. An equation that may be used to predict the tensile strain at which a crack initiates in clay is also given in Figure 3:11 ($\varepsilon_{\text{crack}} = 14s_i^{0.5}$). The results indicate that the tensile strain required for crack initiation in clays reduces with increasing mean effective stress.

Figure 3:11 : Variation in tensile strain to cracking with initial (reproduced from Thusyanthan et al, 2007)
Strain plots obtained in both load-controlled tests and strain-controlled tests showed linear variation with depth until near the initiation of a crack. The strain plots also showed that the neutral axis remained at or close to the mid-depth of the beam until the initiation of a crack. The strain to crack initiation was shown to be a strong function of initial mean effective stress in the clay beam.

The stress states of the extreme fibre in tension of all the beams at initiation of a crack showed that cracks initiated when the effective stress state reached either the tension cutoff line for low mean effective stresses or the ‘apparent failure line’ for higher mean effective stresses corresponding to Hvorslev’s normalisation of the Mohr–Coulomb failure envelope. These effective stress criteria for crack initiation have been derived only from tests on remoulded saturated clays. Cracks were seen to open either as pure tension cracks at zero minor principal effective stress, or as mixed-mode shear-tension cracks when an effective stress path reaches the Hvorslev criterion of brittle shear rupture, but only if the minor total stress is also tensile.
An idealized framework for the analysis of cohesive soils undergoing desiccation was proposed by Konrad and Ayad (1997b). A simplified model referred to as “CRACK” was developed as a result. CRACK enables the prediction of the average spacing between primary cracks for a homogeneous soil deposit subjected to a given surface evaporation flux. The response of the Saint-Alban clay during desiccation up to the initiation of cracks was analysed using the CRACK model and the relevant properties of soil obtained from the field experiment (Konrad and Ayad, 1997a). To assess the sensitivity of the analysis with respect to variation in soil properties and surface evaporation flux a parametric study was performed.

There remains a shortage of models describing desiccation phenomena in clayey soils with a rational and complete approach that should ultimately lead to the prediction of the average spacing between primary shrinkage cracks given a set of boundary conditions and soil properties.

3.3 Crack pattern, geometry and connectivity

The cracking process, from the initiation of a crack onwards, can be divided into four stages based on general lab scale testing (Vallejo, 2009). These stages are categorised based on visual observations (see Figure 3:13) and are distinct from the shrinkage stages discussed earlier in this section.

a. Formation of 1st generation cracks which are long and isolated affecting a small region.

b. Formation of shorter 2nd generation cracks in an orthogonal direction to the 1st generation cracks that connect them. This process continues until the cracks are connected as a network enabling fluid flow.

c. Formation of more and more shorter, diversely oriented cracks which greatly increases the connectivity.

d. Extensive fragmentation.

Once a crack initiates, subsequent subdivision of the layer takes place rather rapidly until a number of cracks in equilibrium (force and moisture) with the test conditions have emerged (Kodikara and Choi, 2006, Péron, 2008, Moe et al., 2003).
Figure 3:13: Crack development in clay sample after desiccation time of: (a) 110 h (water content w =120% ) (b) 114 (w = 80%) (c) 123 h (w =30 %); (d) 155 h (w =15%) after Vallejo (2009)

Different complex patterns of cracks can be formed depending on three dimensional drying conditions, soil types and restraints. Cracks are formed parallel to each other if the specimen is restrained at two opposite ends (Costa et al., 2008, Peron et al., 2009b). This is most likely to occur in linear shrinkage conditions i.e. the specimen shape tends towards a beam. For the two dimensional case, i.e. plates, the crack formed makeing shapes that are either rectangles (if the cracks meet at 90°; (Vallejo, 2009) or hexagons (if the cracks meet at 120°,(Konrad and Ayad, 1997a, Li, 2009, Dyer et al., 2009), or a combination of both rectangles and hexagons (Dyer et al., 2009, Peron et al., 2009b).

Crack morphology is influenced by isotropic drying and directional drying (Shorlin et al., 2000). In isotropic drying, the entire layer is dried uniformly. Isotropic drying results in ‘arching’ cracks as shown in Figure 3:13. In directional drying, the layer is dried from one
end. In this case, a one-dimensional pattern of cracks propagates due to a moving drying front, resulting in a ‘laddering’ crack pattern (Figure 3:14).

![Figure 3:14: Laddering mode of cracking (after Peron et al., 2009b)](image)

For soft cohesive soils, if a fast evaporation rate is induced by high temperatures and moisture gradients between the ground and the atmosphere, the surface of the clay will dry out quickly. This leads to creation of a thin crust, that evaporating water cannot pass through easily (Abu-Hejleh and Znidarcic, 1995). The dried crust has much lower permeability, which protects the clay underneath from further shrinkage and only shallow cracks are formed whose walls also could be considered impermeable.

In contrast, if slow evaporation is induced by relatively mild changes in the atmosphere, the clay is able to drain more extensively prior to formation of a relatively impermeable crust. This leads to the formation of wider, deeper cracks. By the end of the process a much thicker crust would be formed in clays under low evaporation rates (Abu-Hejleh and Znidarcic, 1995). The clay surface will show a polygonal pattern, with the cracks forming the borders around the soil columns.

**Crack surveys**

Geometrical parameters of cracks such as width, depth and connectivity are important for better understanding of the influence of cracks on hydrology of the soil. In the field, these
parameters are measured at different planes; beginning at the surface level and moving downwards. Some of the methods used to measure these parameters are:

1. Digital imaging

Surface photographs are taken and analysed to quantify the surface geometry. This is preferable for measuring surface soil cracks as it causes minimal disturbance to the sample and yields very detailed data. This method also offers a possibility for determining crack-geometry in a semi-automatic or automatic process. The automated procedures can reduce the errors associated with gathering field fracture data by eliminating human bias and by standardizing the sampling procedure. Konrad and Ayad (1997a) and Li (2009) used this method for observing cracks in clay and Kemeny and Post (2003) for assessing rock fractures.

2. Scan line surveys

This is a traditional method for obtaining discontinuity data in the field. The fracture information is collected along a line at a cracked face. Scan line surveys provide detailed information on individual fractures in each set that can be used in probabilistic design. This method is difficult in the case of rock surveys and prone to errors. Sampling difficulties, the choice of sampling method, human bias, safety risks, instrument error, and other problems can lead to significant errors in gathering field data (Kemeny and Post, 2003). Scan line surveys were used to observe crack geometries in the field by Dasog et al (1988).

Cracks up to a depth of 750mm and to a width of 75mm were reported on motorway embankments made of lodgement till in the Cheshire basin by Al-shaikh-ali (Al-shaikh-ali, 1978). The magnitude of the cracks that developed in the boulder clay motorway embankments was related to the linear shrinkage and plasticity index of the soil. The average spacing and opening for cracks initiated in intact clay at several time steps are shown in Table 3:1 (Konrad and Ayad, 1997a). The water content of the soil was a crucial factor for crack formation.
<table>
<thead>
<tr>
<th>Time from excavation (hour)</th>
<th>Crack opening (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At surface</td>
<td>Below shear plane</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>Initiation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>2-2.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>2.5-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>4-6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>8.5-11.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>155</td>
<td>Initiation</td>
<td></td>
<td></td>
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<td>12.5-15</td>
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</tr>
<tr>
<td>200</td>
<td>13-16.5</td>
<td>2-3</td>
<td></td>
</tr>
<tr>
<td>240</td>
<td>17-22</td>
<td>4-5.5</td>
<td></td>
</tr>
</tbody>
</table>

Table 3:1 : Development of crack width with time in intact clay at the ground surface and below the shear plane (after Konrad and Ayad, 1997a).

Figure 3:15: Stages and zones of shrinkage cracking in clay (Konrad and Ayad, 1997a)

Under severe environmental conditions, horizontal cracks referred to as shear planes are formed below vertical cracks (1st zone) due to differential shrinkage (Figure 3:15d). This would lead to the formation of large blocks. Cracking continues to propagate downward below the shear plane to form a 2nd zone of cracks. These zones were observed in both normal ground (Konrad and Ayad, 1997a) and flood embankments (Dyer et al., 2009) (Figure 3:16).

During extremely dry summer periods, some clays crack to depths of up to 2m (Greenwood, 1998). Large vertical desiccation cracks up to a depth of 2 m have also been found in flood
embankments (Dyer et al., 2009). Wetting and drying processes brought about by seasonal variations and changes in water level cause severe swelling and shrinking in flood embankments. In flood embankments, blocks created by primary cracks are gradually moved upward and outward when ground water fills the cracks, due to the lateral thrust on the blocks and buoyancy effects.

Figure 3:16: Crack zones observed at a trial trench in a flood embankment (After Dyer et al., 2009)

3.4 Infrastructure slopes: Cracking, vegetation and permeability

Irreversible fabric changes are caused in clay by shrinkage during the first drying cycle (Yong and Warkentin, 1975). Cracks appear at the same locations in the 2nd and 3rd drying cycles (Yesiller et al., 2000). Cracks that develop in the first drying cycle are nearly closed by subsequent wetting; however, these cracks remain as potential failure zones. A field study by Li (2009) confirmed the weaker bonds between cracked soil blocks (aggregation) compared with the bond inside the block (aggregation).
Permeability is increased by several orders of magnitude even after closure of cracks (i.e., at the end of wetting cycle) compared with un-cracked clay (Rayhani et al., 2007, Anderson et al., 1982). Cracks persist as micro-discontinuities even after the end of winter (Anderson et al., 1982). Thus shrinkage cracks influence the permeability of slope surfaces throughout the year and cause a faster than expected response of the pore water pressure to seasonal rewetting and storms. This could lead to elevated water levels within the embankments. In very inactive soil (non-expansive soils), only small changes in hydraulic conductivity result from an increasing number of wet-dry cycles (Eigenbrod, 1996).

The type of vegetation cover directly influences the cracking of clay slopes in motorway embankments (Anderson et al., 1982). Few visible cracks are formed in the vegetated part of the embankment compared with the areas where the topsoil and vegetation cover are less. However, cracks in areas with vegetation cover persist longest after the onset of wet weather, whereas few or none were visible to the naked eye in areas with less vegetation and topsoil cover.

When cracking is severe, it is possible for cracks to be connected and form a network. This network could channel the water laterally. In the context of agricultural science, it was reported that the flow of water through a crack network both under experimental and natural rain flows were in a good agreement with equations derived from Darcy’s law (Inoue, 1993). Hydraulic conductivities of the agricultural field with cracks were estimated to be of the order of $10^{-4}$ m/s in spite of the very low permeability of the soil matrix of the field ($10^{-8}$ to $10^{-9}$ m/s).

Reasonably good closure of tension cracks (to the naked eye) is possible via several mechanisms, which may depend on the type of soil and climatic conditions (Eigenbrod, 2003). Three main causes for closure of fractures in fine-grained soils are (Eigenbrod, 2003):

1. Increase of effective stress (loss of suction) leading to a reduced undrained shear strength of the intact soil;
2. Clogging of fractures by particles eroded from the fracture surface during permeation in non- or low-plastic soils;
(3) Swelling of the clay particles near the fracture surfaces in highly volume sensitive clay.

However, these loosely closed cracks would still act as a preferential flow path for fluid. Overall, the cracking significantly affects the hydraulic properties of the cracked soil during drying and wetting cycles.

3.5 Summary

The field tests conducted so far by researchers have mainly focused on surveying crack geometric characteristics (Konrad and Ayad, 1997a, Li, 2009, Dyer et al., 2009), because a survey of crack development is significantly time consuming and expensive. Little information can be found on interaction of roots with shrinkage cracks in natural soils exposed to the atmosphere and on the corresponding hydrological response of cracks. In particular, there is not very much evidence relating to how the cracks form in clay infrastructure embankments and cuttings, for example the pattern and dimensions of the cracks are not clear.

Such information is necessary for numerical modelling of pore water pressure response and slope stability. The study of crack development and interaction of roots in the field under natural atmospheric conditions is of practical significance, and should be pursued.

It is evident that shrinkage cracks have a significant influence on the permeability of the slope surface (Rayhani et al., 2007, Anderson et al., 1982). However, it is not clear how the opening and closing of the crack during different seasons of the year are influencing the near surface permeability.

The following section describes and presents the results of a field study carried out to observe the macro pores and cracks and the effect of these on the permeability in the field.
4. Effects on near surface soil structure and permeability

Numerical modelling of a London Clay embankment by Nayambayo et al. (2004) found that the laboratory measured permeability of London Clay, typically in the order of $10^{-9}$ m/s, prevents depletion of the suction created during the summer. In Nayambayo’s model, 6 months of summer boundary condition was induced prior to 6 months of winter boundary conditions. This study also found that to obtain an annual cyclic change in pore water pressure permeability in the order of $10^{-8}$ m/s or more is required.

The hydrological performance of an infrastructure cut slope in London Clay at Newbury, England has been investigated over several years (Smethurst et al., 2006). The measured pore water pressure variation indicated a faster seasonal response than might be expected from the typical laboratory determined permeability of intact stiff London Clay. When back analysed using numerical modelling techniques, increased soil permeability of up to three orders near the surface (in the weathering zone) was found necessary to give the observed field behaviour (Rouainia et al., 2009). A recent study identified a permeability in the order of $5 \times 10^{-7}$ m/s or greater as the minimum needed to induce seasonal pore water pressure changes on a typical railway embankment, considering the precipitation for southern England (Loveridge et al., 2010).

The aim of this chapter is to test the hypothesis that assumption of increased permeability near the surface in numerical simulation is reasonable based on the effects of cracks observed in the field and numerical study.

A series of field investigations were conducted to investigate the influence of cyclic seasonal changes on the near surface soil structure and on the near surface clay slope permeability. A cut slope located in Newbury was used for this investigation. The following sub sections describe the experimental tests and numerical modelling conducted in this research.
4.1 Site description

This study was undertaken on the cutting of the A34 Newbury bypass in southern England (Figure 4:2). The cutting is formed of London Clay. This site has been monitored by Smethurst et al (2006, 2012) using instrumentation to measure pore water pressures, water content and rainfall.

The site was selected for this study due to its accessibility, relatively uniform soil conditions and vegetation characteristics (mainly grass), as well as the known recent climate and weather history for the area. The slope is east facing, 8 m high and 28 m long (Figure 4:3). The cutting was constructed in 1997, and is entirely within the London Clay. The London Clay at the site is about 20 m thick, highly weathered to a depth of about 2.5 m below original ground level, and underlain by Lambeth Group deposits and the Upper Chalk. After the cutting was excavated, up to about 0.4 m of topsoil was placed over the cut London Clay surface to facilitate the vegetation growth on the slope. A gravel fin drain approximately 600 mm deep, 4 m from the toe of the slope, was installed to drain the road subbase. The fin drain connects at intervals into a sealed carrier drain that outfalls to the south of the cutting, and to which the road gullies are also connected. A plan view of the slope is shown in Figure 4:1. A cross-section through the slope is shown in Figure 4:4.
Figure 4:1: Plan view of the Newbury cut slope (Original plan from Smethurst et al (2006))
Figure 4:2: Location of the Newbury site from where the sample of clay was excavated (Google, 2018)
Figure 4:3: View of the slope from a nearby pedestrian bridge

Figure 4:4: Cross section of the site: (Smethurst et al., 2006)
The current groundwater regime in the slope is effectively hydrostatic below the groundwater level. This indicates that, within the near-surface zone being monitored, any negative excess pore pressures induced by unloading as a result of excavating the cutting have substantially dissipated.

The vegetation is primarily rough grass and herbs with a few small shrubs less than 1.5 m high. The grass and herbs were initially mowed periodically to help the development of shrubs planted on the slope, but have remained uncut since October 2002. Mature beech, oak and silver birch trees fringe the top of the slope.

The London Clay is predominantly a stiff grey clay, but contains several bands of silty clay up to 50 mm thick and bands of large flints. The weathered London Clay is spatially very variable, changing from a stiff orange brown clay to a clayey silt over small distances and depths, similar to the description given by Perry et al (2000). The permeability, unit weight and plasticity index of the London Clay at the site are summarised in Table 4:1. The saturated vertical permeability was obtained from tests on undisturbed samples from depths of 0.5, 1.0, 1.5, 2.0 and 3.0 m carried out at effective confining pressures of 10, 15, 20, 25 and 35 kPa in the triaxial apparatus. Estimates of the in situ permeability, obtained from bailing out tests carried out in April 2003 in hand-augered boreholes 3 m deep, were typically one to two orders of magnitude larger owing to the effects of anisotropy and fabric (including silt partings and fissures) not fully captured in the triaxial samples. The dry unit weight was measured from undisturbed samples obtained from 0.5 m depth for both the weathered and unweathered clays.

<table>
<thead>
<tr>
<th>Property</th>
<th>Grey London Clay</th>
<th>Weathered London Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturated permeability from triaxial tests: m/s</td>
<td>$3.9 \times 10^{-11}$ to $6.6 \times 10^{-10}$</td>
<td>$2.3 \times 10^{-10}$</td>
</tr>
<tr>
<td>Saturated permeability from borehole bail-out tests: m/s</td>
<td>$2.3 \times 10^{-9}$ to $4.4 \times 10^{-9}$</td>
<td>$3.7 \times 10^{-9}$</td>
</tr>
<tr>
<td>Dry unit weight, $\gamma_d$: kN/m$^3$</td>
<td>13.2 to 15.2</td>
<td>146</td>
</tr>
<tr>
<td>Plasticity index, $I_p$: %</td>
<td>32.5 to 36.4</td>
<td>34.8</td>
</tr>
</tbody>
</table>

*Only one of five samples tested for the weathered London Clay exhibited plasticity, the remainder were a silt.

Table 4:1 Permeability, unit weight and plasticity index of grey and weathered London Clay at the Newbury test site (from Smethurst et al (2006)).
The forthcoming sections discuss the following:

a) Field observations of cracks and macro pores carried out on Newbury cut slope.

b) Field permeability measurements carried out on the Newbury cut slope.

c) Mathematical modelling and finite element simulations carried out to study the hydrological effects of cracks and other macro pores associated with the case study.

4.2 Field investigation of cracks and other macro pores

Field observations were carried out at the Newbury site during the summer of 2010 to observe the soil cracking. Geometrical properties of cracks were measured at depths close to the surface. Some of the photographs taken in the field have been included below (Figure 4:5).

A number of groups of interconnected cracks were observed at the surface level (in top soil), which were not connected to other groups. It was observed that some secondary cracks branched out from a primary crack within the group.

A small trench was dug in an area with visible surface cracks to investigate the geometrical properties in detail. The soil was dry near the surface. Moisture content measurements taken at the same site using neutron probes on 10th Aug 2010 up to a depth of 2.5m are shown in Figure 4:6 which show soil moisture level at significantly below saturated equilibrium water content of about 0.4 m³/m³ to a depth of 0.75m below ground level. A nearly vertical crack having a width of approximately 20mm at the top was observed up to a depth of 0.5m where its width was 11mm. The crack clearly continued further down (Figure 4:7). The very dry nature of the soil during the particular time of the year made the manual digging process difficult.
Figure 4:5: a) & b) Desiccation cracks observed at the surface level in two different places on 19th of August 2010.
Figure 4:6: Volumetric moisture content readings from the site

The volumetric water content reported in Figure 4:6 was measured using neutron probes.
An extended investigation was carried out two weeks later on 02\textsuperscript{nd} of September 2010 by carefully excavating a trench of approximately 1m x 1m in plan. Intense rainfall in the week before had rewetted the surface compared with the measurements in early August (see Figure 4:6). This didn’t affect the results of the investigation to a great extent, and it helped to hold the cracked blocks together and prevent them breaking up on excavation.
The clay at a depth of 300 mm was found to be heavily fragmented into small clay blocks divided by very fine cracks (Figure 4:8). The crack width was in the order of 0.1mm. Blocks were still intact and densely packed. This is just within the intact Grey London Clay below more organic surface top soils. Upon unearthing, the small blocks all naturally fell apart along the clear very fine cracks in the clay. The length of the blocks varied from about 30mm to 80mm (Figure 4:9).
Grass roots were observed to have penetrated in a random pattern in the top soil, which was originally placed to facilitate plant growth (Figure 4:10). Once roots enter the intact clay zone they were found to follow the crack paths and in places were packed in a line quite closely as observed at 200 mm from surface level (Figure 4:11, Figure 4:12 and Figure 4:13).
Figure 4:10: Random growth of grass roots in the top soil at about 100 mm from surface level.
Figure 4:11: Densely packed grass roots following the path of cracks from the topsoil/intact clay interface approximately at 200 mm from surface level

Figure 4:12: Close look of roots following cracks approximately at 200 mm from surface level
Figure 4:13: Roots found between two blocks (From Figure 4:8)
4.3 Field permeability measurements

As part of the investigation on the effect of cracking effects on permeability, the near surface permeability of the cut slope in Newbury was measured. A double ring infiltrometer was used to measure the vertical infiltration rate of the soil near the surface. In UK, the two main types of hydraulic conductivity test are: (i) borehole bailout test (ii) Guelph permeameter test. But
neither of these is able to measure purely vertical permeability. When the vertical permeability of a cut slope is concerned, the ring infiltrometer is the best option as it is capable of measuring the vertical permeability. Ring infiltrometers have been widely used in agricultural science and in the recent years in the applications of geotechnical engineering (Li et al., 2011, Al-Yaqout, 2016). An advantage of the double ring infiltrometer is that the lateral spreading of water from the inner ring is minimised by infiltration from the outer ring.

4.3.1 Principle of double-ring infiltrometer

The double-ring infiltrometer used in this project was purchased from Durham Geo Slope Indicator Company and this unit consists of two stainless steel rings measuring 12 and 24 in diameter x 20 in high. The rings incorporate a double thickness, welded top edge for increased stability when driving into the soil.

The double-ring infiltrometer method consists of driving two open cylinders, one inside the other, into the ground, partially filling the rings with water, and maintaining the water at a constant level. The volume of water added to the inner ring to maintain a constant water level is the measure of the water that infiltrates into the soil. The volume infiltrated during timed intervals is converted to an incremental infiltration velocity, and plotted against elapsed time. The maximum-steady state or average incremental infiltration velocity (depending on the application of the test) is equivalent to the infiltration rate.

4.3.2 Experimental method

The double-ring infiltration tests were conducted following ASTM D3385-09 standard (ASTM, 2009). The vegetation at the test site was removed, and the test ground was levelled by excavating a shallow bench into the slope. Great care was taken during the excavation to keep the disturbance to the soil to a minimum. The outer ring was driven into the soil to a depth of 150 mm with blows from a heavy sledge hammer. This is to prevent the test fluid from leaking outwards to the ground surface surrounding the ring. The inner ring was driven to a depth of 75mm in a similar manner. The Mariotte tubes used to maintain the water level in the rings were filled with water. Pieces of rubber sheets were placed on the soil surface used as splash guards in both the inner ring and the annular space between the inner ring and
the outer ring to avoid soil erosion as water was introduced into the rings. Water was introduced into the inner ring and the outer ring, to a level of 100 mm. Throughout the test, the water level inside the Mariotte tubes was recorded. The Mariotte tubes can be refilled with water as necessary during the test. The infiltration rate was obtained from the records of the water level inside the Mariotte tubes.

Samples of soil were taken using a hand auger in order to obtain moisture content measurements in the laboratory. Samples were taken from the middle of the inner ring after the test and from about 2m away from the test site representing the moisture content profile before the test.

![Cross section diagram](image)

Figure 4:15: Schematic diagram of the cross section of the experiment
Figure 4:16: Plan view of the rings after installation
Figure 4:17: Complete experimental set up

Figure 4:18: Flow paths within the double ring infiltrometer
4.3.3 Preliminary Test

The cumulative volume of water infiltrated from the inner ring plotted against time is shown in Figure 4:19.

![Graph showing the cumulative volume of water infiltrated with time](image)

**Figure 4:19:** Graph showing the cumulative volume of water infiltrated with time

The infiltration rate in the inner ring can be calculated following ASTM D 3385 (ASTM, 2009). The infiltration rate is

\[
v_{IR} = \frac{\Delta V_{IR}}{A_{IR} \Delta t}
\]

4:1

\(v_{IR}\) = inner ring incremental infiltration velocity, cm/min,

\(\Delta V_{IR}\) = volume of liquid used during time interval to maintain constant head in the inner ring, cm³,

\(A_{IR}\) = internal area of inner ring, cm², = 706.8583 cm²

\(\Delta t\) = time interval, min.
Infiltration Velocity from Graph = \(\frac{0.6145}{706.8583}\) cm/min = \(1.4489 \times 10^{-7}\) m/s

Although the units of infiltration rate and permeability of soil are similar, there is a distinct difference between these two quantities. They cannot be directly related unless the hydraulic boundary conditions are known, such as hydraulic gradient and the extent of lateral flow of water can be reliably estimated. One way of calculating the hydraulic gradient is by determining the saturated depth \(z_w\) of the soil.

Samples up to a depth of 40 cm were taken in two locations and sealed in sandwich bags, with depths marked on them individually. One set of samples were taken 0.5 m away (adjacent) to the outer ring to represent the moisture content conditions before the infiltration experiment (taken before the experiment). Another set of samples were taken from within the inner ring immediately after the test to represent the moisture content conditions after the infiltration experiment.

Gravimetric water content measurements were measured by drying the sample with known weight at each depth and obtaining the dry weight. These values were converted to volumetric water contents using the relationship between two. Volumetric water content measurements changing with depth are shown in the Figure 4:20.
Figure 4:20: Volumetric moisture content before and after the test varying with depth

The saturated depth \((z_w)\) from the graph (Figure 4:20) is 65 mm.

\[ i = \frac{(z_w + h_w)}{z_w} = \frac{(65 + 100)}{65} = 2.53 \]

\(z_w\) – Saturated depth of soil

\(h_w\) - Level of water in the ring above the surface of the test ground

Hydraulic conductivity \(k = \frac{\nu R}{i}\)

\[ k = 1.4489 \times 10^{-07} / 2.53 \text{ m/s} = 5.72688 \times 10^{-08} \text{ m/s} \]
4.3.4 Modified double-ring infiltration test method

In this method, which is more accurate than the previous, the hydraulic gradient was calculated by measuring the pore pressure at different depths under the rings.

Pressure transducers connected to extension tubes with ceramic tips were used to measure the pore water pressure at the test site. Extension tubes are glass tubes of approximately 1 inch diameter and are available at heights (length) ranging from 30 cm to 150 cm. These were inserted into the augured holes made by drill heads. These are very useful in translating the pore pressure at depths under the ground to the surface level where a tensiometer can be connected to measure the pore pressure. A pressure transducer, which is sensitive to the change of the pressure in the tube, is connected on the top of the tube. The water pressure is measured and logged by a data logging unit recording the signals automatically at regular intervals. GP1 Data Logger from Delta - T devices was used for this purpose. It can automatically record the pore water pressure in every 10 minutes. The pressure transducer can measure pressures in the range of +100 kPa to -100 kPa linearly. They were calibrated using water column between -10 kPa and +10 kPa for this experiment purpose.

Hydraulic gradient:

The hydraulic gradient can be obtained through the head difference between the test ground level and the soil below the test ground level where pore pressure is measured as shown in Figure 4:21.
4.3.5 Experimental Location and the Series

A series of three experiments were carried out using the modified double ring infiltrometer method described above. First experiment was carried out to measure the near surface permeability during the middle of the winter in Nov/Dec 2011 (Test 1). The second experiment was carried out during March 2012 to measure the near surface permeability during the end of winter (Test 2). Final experiment was carried out to measure the near surface permeability during the end of summer in September 2012 (Test 3). Figure 4:22 shows the locations where the tests were carried out.
4.3.6 Experiment carried out in Nov-Dec 2011 (Test 1)

An experiment was carried out using the modified Double ring infiltrometer method in the same Newbury site mentioned from 25\textsuperscript{th} of Nov to 3\textsuperscript{rd} Dec 2011. Same methodology described in 4.3.2 is used to set up the experiment at 0.1m depth from the surface (after removing the grass root zone).

In addition to this, 4 tensiometers were installed to a designated depth by hand auger in a hole of 20mm diameter (see Figure 4:23). The middle of the ceramic tip was set in the calculated depth used in pore water pressure calculations. For seating of the ceramic
a hole was made using a 16 mm diameter drill head (The size of the hole should be slightly smaller than the ceramic tip to ensure good connectivity between the tip and the surrounding soil). Then the pipe was inserted into it. After the tensiometers were installed, the hole was backfilled with moist soil. Then, for the vertical tensiometer, cement-bentonite mix was used to seal the hole to avoid flow along the tensiometer. For the horizontal tensiometers, kaolin clay was used to fill around and was sealed well at the surface level. A period of 72 hours was allowed for the seal to settle and the pore pressure to equalise. Schematic diagram of the experiment set up is shown in Figure 4:23. Figure 4:24 and Figure 4:25 show the set up in field.

Figure 4:23: Cross section of the experimental set up marking the locations of tension meters
Figure 4:24: View of the experimental set up – seen from down slope
Figure 4:25: View of the experiment set up – side view
Table 4.2: Readings and Infiltration rate calculations for inner ring

Marriott tube readings and instantaneous rate of infiltration for inner ring are shown in Table 4.2.

The corresponding volume infiltrated is shown in Figure 4.26.
Figure 4.26: Cumulative volume infiltrated in the inner ring changing with time

Infiltration rate from latter part of the graph = 3.5 cm$^3$/hr

\[
= \frac{3.5}{((22/7) \times (6 \times 2.54)^2)} \times 100 \times 60 \times 60
\]

m/Sec

\[
= 1.34E-08 \text{ m/s}
\]
The infiltration rate in the inner ring (Figure 4:27) is used to calculate the vertical hydraulic conductivity. The infiltration rate reduced over time, and became stable as the soil became fully saturated.

**Pore pressure response**

Horizontal Tensiometer H3 at the bottom failed due to a disturbance to the pressure transducer causing loss of water from tensiometers. Vertical tensiometer (Vi) did not respond possibly due to the cement bentonite seal fill blocking and sealing the ceramic tip. The layout of the transducers that responded and the corresponding readings are shown in Figure 4:28 and Figure 4:25 respectively.
Figure 4:28: Cross section of the experimental set up indicating the locations of tensiometers that responded

Figure 4:29: Readings recorded from transducers H1 and H2
Considering the test ground level as the datum, the total head at the test ground level was estimated to be 0.15 m, because the water level in the ring was 0.15 m. The total head at 0.20 m depth can be determined using the pore water pressure values measured by the tensiometer at the corresponding depth. The stabilised pore water pressure from the tensiometer H1 is 2.36 kPa. The total head at 0.20 m was calculated as: \((-0.20 + 2.36/9.8) = 0.036\) m.

The average hydraulic gradient is: \((0.15-0.036)/0.20 = 0.57\)

<table>
<thead>
<tr>
<th>Tensiometer</th>
<th>Depth from ground surface (cm)</th>
<th>Elevation head (kPa)</th>
<th>Pressur e Head (kPa)</th>
<th>Total Head (m)</th>
<th>Hydraulic Gradient (Between ZPL &amp; Tensiometer)</th>
<th>Infiltration rate (I) m/sec</th>
<th>Permeability k(m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1</td>
<td>20</td>
<td>-2.00</td>
<td>2.36</td>
<td>0.036</td>
<td>0.57</td>
<td>1.28E-08</td>
<td>2.24207E-08</td>
</tr>
<tr>
<td>H2</td>
<td>32</td>
<td>-3.20</td>
<td>1.20</td>
<td>-0.200</td>
<td>1.09</td>
<td>1.28E-08</td>
<td>1.17029E-08</td>
</tr>
</tbody>
</table>

Table 4.3: Hydraulic conductivity calculations

Hydraulic conductivity

The steady state infiltration rate divided by the stabilised hydraulic gradient gives the vertical hydraulic conductivity.

The average hydraulic conductivity values between two points are given in Table 4.3. The average hydraulic conductivity is in the order of \(10^{-8}\) m/s. The average hydraulic conductivity between the surface and 0.32 m depth is slightly smaller than between the surface and 0.2 m depth, indicating a reduction in hydraulic conductivity over the depth.

Figure 4.30 shows the volumetric water content readings taken before and after the test. There is a clear increase in volumetric water content up to 30 cm depth and the ground is saturated up to this level after the test.
Figure 4:30: volumetric water content readings taken before and after the test

4.3.7 Experiment carried out in March 2012 (Test 2)

In order to measure the permeability of the near surface at the end of the winter, another experiment was carried out using the modified Double ring infiltrometer method in the same Newbury site mentioned in 4.1 between 7 and 15 March 2012. Same methodology mentioned in section 4.3.2 was used to set up the experiment at a 0.1m depth from the surface (after removing the grass root zone).

Schematic diagram of the experimental set up is shown Figure 4:31. Figure 4:32 show the set up in field.
Figure 4:31: Cross section of the experiment set up indicating the locations of tensiometers
Tensiometer T4 did not respond due to a connection issue.
Table 4:4: Volume and infiltration rate calculations for inner ring

<table>
<thead>
<tr>
<th>Time Elapsed (hr)</th>
<th>Volume infiltrated (cm³)</th>
<th>Instantaneous Rate of infiltration (cm³/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>0.33</td>
<td>2.676</td>
<td>4.36609E-08</td>
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<td>2.08</td>
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<td>4.07501E-08</td>
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<td>22.58</td>
<td>74.928</td>
<td>1.29208E-08</td>
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<td>169.58</td>
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</tr>
<tr>
<td>263.58</td>
<td>845.616</td>
<td>1.21383E-08</td>
</tr>
</tbody>
</table>

Marriotte tube readings and instantaneous rate of infiltration for inner ring are shown in Table 4:4. And the corresponding volume infiltrated is shown in Figure 4:26.

Figure 4:33: Readings recorded from transducers T1, T2 and T3 (See discussion in summary section 4.5)
Figure 4:34: Cumulative volume infiltrated with time

Figure 4:35: Infiltration rate of the inner ring changing with time
Infiltration Rate from latter part of the graph = 1.31E-08 m/s

<table>
<thead>
<tr>
<th>Tensiometer readings</th>
<th>Depth from ground surface (cm)</th>
<th>Elevation head (kPa)</th>
<th>Pressure Head (kPa)</th>
<th>Total Head (m)</th>
<th>Hydraulic Gradient (Between ZPL &amp; Tensiometer)</th>
<th>Infiltration rate (I)</th>
<th>Permeability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
<td>0</td>
<td>1.5</td>
<td>0.15</td>
<td></td>
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<tr>
<td>T1</td>
<td>9</td>
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<td>0.121</td>
<td>0.32</td>
<td>1.31E-08</td>
<td>4.05E-08</td>
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<tr>
<td>T2</td>
<td>19.5</td>
<td>-1.95</td>
<td>2.26</td>
<td>0.031</td>
<td>0.61</td>
<td>1.31E-08</td>
<td>2.14E-08</td>
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</tbody>
</table>

Table 4:5: Hydraulic conductivity calculations

The average hydraulic conductivity values between two points are given in Table 4:5. The average hydraulic conductivity is in the order of 10^{-8} m/s. The average hydraulic conductivity between the surface and the depth of 19.5 cm is slightly smaller than between surface and the depth of 9.0 cm, indicating a reduction in the hydraulic conductivity over the depth.

**4.3.8 Experiment carried out in September 2012 (Test 3)**

Another experiment was carried out using the modified double ring infiltrometer method at the Newbury site introduced in section 4.1. The same methodology described in section 4.3.2 is used to set up the experiment at 0.1m depth from the surface (after removing the grass root zone). A schematic diagram of the experimental set up is shown in Figure 4:36. Figure 4:37 shows the set up in field.
Figure 4:36: Cross section of the Experiment set up indicating the locations of tensiometers.

The marriott tube readings and instantaneous rate of infiltration for the inner ring are shown in Table 4:4.

The corresponding volume infiltrated is shown in Figure 4:38.
Figure 4:37: Experiment Set up in the field
Table 4.6: Volume and infiltration rate calculations for the inner ring

<table>
<thead>
<tr>
<th>Time Elapsed (hr)</th>
<th>Volume infiltrated (m$^3$)</th>
<th>Instantaneous Rate of infiltration (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
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<tr>
<td>0.67</td>
<td>5.89E-05</td>
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<td>3.05512E-07</td>
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<tr>
<td>2.67</td>
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<td>4.81E-05</td>
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</tr>
<tr>
<td>5.33</td>
<td>4.81E-05</td>
<td>2.74961E-07</td>
</tr>
</tbody>
</table>

Figure 4.38: Cumulative volume of water infiltrated with time for experiment carried out in September 2012
Figure 4:39: Infiltration rate changing with time for experiment carried out in September 2012

The volume infiltrated into the soil and instantaneous rate of infiltration for the inner ring are shown at various times in Table 4:6. The corresponding plots are shown in Figure 4:38 and Figure 4:39.
Figure 4:40: Readings from Tensiometer A & C located in inner ring for experiment carried out in September 2012

Figure 4:40 shows the pore water pressure readings from the tensiometers A & C. Tensiometers B & D did not respond, and on unearthing the tensiometers at the end of the test it was found that the ceramic tips were blocked by cement bentonite mix.
Figure 4.41: Hydraulic gradient between surface and point A against time for experiment carried out in September 2012

Table 4.7: Hydraulic conductivity calculations calculated between ground surface and tensiometer location for experiment carried out in September 2012

<table>
<thead>
<tr>
<th>Tensiometer readings</th>
<th>Depth from ground surface (cm)</th>
<th>Elevation head (kPa)</th>
<th>Pressure Head (kPa)</th>
<th>Total Head (m)</th>
<th>Hydraulic Gradient (Between ZPL &amp; Tensiometer)</th>
<th>Infiltration rate (m/s)</th>
<th>Permeability (m/s)</th>
</tr>
</thead>
<tbody>
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<td>0</td>
<td>1.6</td>
<td>0.16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
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<td>1.48E-06</td>
</tr>
<tr>
<td>C</td>
<td>19.5</td>
<td>-1.95</td>
<td>3.05</td>
<td>0.11</td>
<td>0.256</td>
<td>2.75E-07</td>
<td>1.07E-06</td>
</tr>
</tbody>
</table>

The average hydraulic conductivity values between ground surface and tensiometer location are given in Table 4.7. The average hydraulic conductivity is in the order of $10^{-6}$ m/s. The average hydraulic conductivity between the surface and a depth of 19.5 cm is slightly smaller than between surface and a depth of 10.2 cm, indicating a reduction in the hydraulic conductivity over the depth.
4.4 Mathematical and Numerical analysis of influence of cracking on permeability

This section introduces the theory for the flow through parallel plates and pipes. These theories are used to link through observations of cracks and observations of permeability changes, observed in the field which are reported in the previous section. This forms a base for the numerical modelling of a single crack described later in this section.

While the methodology used here has been applied for cracks in rocks, it’s not yet applied to cracks in clay of infrastructure slopes.

4.4.1 Permeability of a single crack

Water flow through a saturated planar crack can be considered to be laminar. A law of flow under saturated conditions for a crack with an aperture \( b \) can be derived considering two parallel plates of width \( l \).

> A

> B

Figure 4:42: Flow through two parallel plates is given by the cubic law

\[
q = \left( \frac{\gamma w g}{12 \mu} \right) b J
\]

4.2

\[
\left( \frac{\gamma w g}{12 \mu} b^3 \right) l J = \left( \frac{\gamma w g}{12 \mu} b^2 \right) J (bl)
\]

4.3

\[
q = \left( \frac{\gamma w g}{12 \mu} b^2 \right) JA
\]

4.4

\( J \) – Hydraulic gradient
Comparing it with Darcy’s equation;

\[ q = kAi \] 4.5

The permeability \( k \) of the crack could be considered to be

\[ k = \frac{\gamma_w g b^2}{12\mu} \] 4.6

Snow (1969) showed that the flow in a smooth planar crack can be described by the above cubic law.

Consider a clay layer of height \( H \) with a cracked area ratio of \( A_{cr} \) where

\[ A_{cr} = \frac{A_{crack}}{A_{total}} \] 4.7

The following equations can be derived from Darcy’s law and the continuity principle.

Figure 4:43: Parallel cracks assumed in a solid soil mass
\[ A_{C1} + A_{C2} + A_{C3} + A_{C4} + A_{C5} + A_{C6} = A_C \]  
\[ A_{\text{total}} = A_{\text{crack}} + A_{\text{soil}} \]  
\[ \frac{A_{\text{soil}}}{A_{\text{total}}} = 1 - \frac{A_{\text{crack}}}{A_{\text{total}}} \]  
\[ q = A_k i \]  
\[ q_{\text{total}} = q_{\text{soil}} + q_{\text{crack}} \]  
\[ A_{\text{total}} k_{\text{bulk}} i = A_s k_s i + A_c k_c i \]  
\[ k_{\text{bulk}} = (1 - A_{cr}) k_s + A_{cr} k_c \]  
\[ k_{\text{bulk}} = k_s + A_{cr} (k_c - k_s) \]

The intact London Clay at Newbury has a mean permeability of \(8.7 \times 10^{-10} \text{ ms}^{-1}\) (Smethurst et al., 2006). If a permeability value in the order of \(10^{-7} \text{ ms}^{-1}\) considered for the cracks, the variation of bulk permeability with these two for a constant cracked area ratio of 0.1 will be as seen in the following figure.

Figure 4:44 shows the variation in bulk permeability given by Equation 4:15 for soil permeabilities in the order of \(10^{-9} \text{ ms}^{-1}\) and crack permeabilities in the order of \(10^{-7} \text{ ms}^{-1}\) for a constant crack area ratio \(A_{cr} = 0.1\).
Figure 4.44: The relationship between crack permeability, soil permeability and overall bulk permeability for a cracked area ratio of 0.1.

For a fixed crack ratio, the variation between crack, soil permeability and the resulting bulk permeability is linear.

For a mean crack width of 0.01 mm, the following values are derived for the permeability of crack under saturated conditions.

\[ b = 0.01 \text{mm} = 0.00001 \text{ m} = 1 \times 10^{-5} \text{ m} \]

At 20°C for water;
\( \gamma_w = 1000 \text{ kg/m}^3 \)

\( \mu = 1 \times 10^{-3} \text{ kg/ms} \)

\[ K_c = \frac{\gamma_w gb^2}{12 \mu} = 1000 \times 9.81 \times (1 \times 10^{-5})^2 / (12 \times 1 \times 10^{-3}) = 8.17 \times 10^{-4} \text{ m/s} \]

Table 4.8 shows the permeabilities for values of \( b \) ranging for 0.1 mm to 100 mm.

<table>
<thead>
<tr>
<th>Crack width (mm)</th>
<th>Crack Permeability ( K_c ) (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>8.17 \times 10^{-5}</td>
</tr>
<tr>
<td>0.1</td>
<td>8.17 \times 10^{-3}</td>
</tr>
<tr>
<td>1</td>
<td>8.17 \times 10^{-1}</td>
</tr>
<tr>
<td>10</td>
<td>8.17 \times 10^{1}</td>
</tr>
<tr>
<td>100</td>
<td>8.17 \times 10^{3}</td>
</tr>
</tbody>
</table>

Table 4.8: Crack permeability changing with crack width

Figure 4.45: Permeability of an individual crack for different crack widths
A crack width of just 0.01 mm would have a reasonably high permeability, is in the order of $10^{-4}$ m/s. In saturated conditions, the permeability of the crack is about 6 orders higher than the intact clay. The Figure 4:46 shows the variation of crack ratio, permeability and bulk permeability for an intact clay permeability of $8.57 \times 10^{-10}$ ms$^{-1}$.

![Figure 4:46: Relationship between crack permeability, cracked area ratio and overall bulk permeability](image)

Figure 4:46 demonstrates that the crack permeability dominates the overall permeability of the soil even at a crack density less than 1%.

In saturated conditions, the flow through cracks could be considered as a flow between two smooth plates. Although the cracked surfaces are irregular and rough, it is possible that the water wets the surfaces of the clay walls, and with ponding of water in micro pits of the wall,
a smooth surface on a micro scale will be formed. In this case the effective width between plates may be less than the aperture of the cracks. Therefore the effective width is that which could correspond to “b” in Figure 4:42. In real cracks, the surfaces could be curvilinear parallel, rather than straight parallel.

CASE STUDY 1:

Anderson et al. (1982) reported the permeability of cracked clay from field measurements taken during February 1980, using a constant head permeability measurement method. The permeability was measured at a depth of 0.25 m at a time when the cracks had closed. The mean value of measured permeability was $5.19 \times 10^{-5} \text{ms}^{-1}$, which was two to three orders of magnitude higher than similarly obtained values for un-fractured locations at the surface of slopes ($4 \times 10^{-7} \text{ms}^{-1}$). A mean value of 5 mm for the crack width at the surface level was reported. The measurements were made using callipers, rules and probes. Hence it is not clear what the minimum measurable crack width was. It may be reasonable to assume that the number of cracks with a width of less than 0.1 mm would be many times greater than the number of cracks in the measurable range with the above mentioned tools. In other words, 0.1 mm is a conservative value for the crack width.

Assuming a 0.1 mm for crack width and using the Equation 4:6, the permeability of an individual crack can be calculated to be $8.17 \times 10^{-2} \text{ms}^{-1}$.

Using Equation 4:15;

$$K_{bulk} = K_s + A_{cr}(K_c - K_s)$$

$$A_{cr} = (K_{bulk} - K_s)/(K_c - K_s) = (5.19 \times 10^{-5} - 4 \times 10^{-7})/(8.17 \times 10^{-2} - 4 \times 10^{-7}) = 6.35 \times 10^{-4}$$

This shows that an extremely small cracked area (0.0635% on a planar area) is needed to induce a significantly higher permeability. However, it should be noted that the permeability measured in the field by Anderson et al. (1982) is not absolute vertical. The vertical permeability may be higher than the value measured in the field as the horizontal permeability, which is likely to be lower than vertical permeability, also plays a role in the field experiments. This will in turn point to a higher cracked area ratio, which is consistent with the large cracks observed in the field.
The permeability values measured by Anderson et al (1982) in sites classified as “un-fractured areas” showed a variation with depth. The values measured at the surface (4 x 10^{-7} ms^{-1}) were two orders of magnitude greater than that measured at a depth of 1 m (1 x 10^{-9} ms^{-1}). This suggests to the possibility of existence of cracks invisible to the naked eye at the surface level.

CASE STUDY 2:

The field modified double ring infiltrometer measurements carried out as part of the present study found out the near surface permeability of cracked clay taken during March 2012, to be 3 x 10^{-08} m/s. The permeability was measured at a depth of 0.1 m at a time when the cracks had closed. The mean value of measured permeability is two to three orders of magnitude higher than obtained values for intact clay immediately after construction of the slope at the surface of slopes (from triaxial experiments (Smethurst et al., 2006)). A mean value of 20 mm for the crack width at the surface level is reported. The measurements were made using callipers, rules and probes. It may be reasonable to assume that the number of cracks with a width of less than 0.1mm would be many times greater than the number of cracks in the measurable range with the above mentioned tools. In other words, 0.1mm is a conservative value for the crack width.

Assuming a 0.1 mm for crack width and using the Equation 4:6, the permeability of an individual crack can be calculated to be 8.17 x10^{-2} ms^{-1}.

Using Equation 4:15;

\[ K_{bult} = K_s + A_{cr}(K_c - K_s) \]  \hspace{1cm} (4:17)

\[ A_{cr} = (K_{bult} - K_s)/(K_c - K_s) = (3 \times 10^{-08} - 2.3 \times 10^{-10})/(8.17 \times 10^{-2} - 2.3 \times 10^{-10}) = 3.67 \times 10^{-7} \]

This shows that an extremely small cracked area (0.00000367% on a planar area) is needed to induce a significantly higher permeability.

From the both case studies above, It could be concluded that the presence of very fine cracks itself would induce a very high permeability increase even the crack width is very small. Considering the sensitivity of the permeability to crack width, it is wise to carry out the slope
stability analysis considering an increased permeability value for the surface layer of up to 0.5m for design purposes.

4.4.2 Permeability of holes and channels made by soil fauna

The channels and pores created by earthworms and moles are of a tubular rather than planar. The water flow through these tubes can be considered as being similar to the flow of viscous Newtonian fluid through pipes.

Figure 4.47: Flow velocity change over the diameter

Poiseuille’s equation gives

\[
q = \frac{dQ}{dt} = \frac{\pi}{8} \left( \frac{R^4}{\mu} \right) \left( \frac{p_1 - p_2}{L} \right) \tag{4.18}
\]

\[
q = \pi R^2 \left( \frac{\rho_w g R^2}{8 \mu} \right) \left( \frac{p_1 - p_2}{\rho_w g \frac{\rho_w g}{L}} \right) \tag{4.19}
\]

Comparing to Darcy’s law

\[
q = A \ k \ i \tag{4.20}
\]

The equivalent permeability (in m/s) is
\[ k = \left( \frac{\rho_v g R^2}{8\mu} \right) \]  \hspace{1cm} 4.21

If a 2mm diameter for an earthworm channel is assumed, permeability of the channel at 20°C will be,

\[ k = 1000 \times 9.81 \times 0.002^2/(8 \times 1 \times 10^{-3}) \text{ ms}^{-1} = 4.905 \text{ ms}^{-1} \]

At a density of one channel/m², \( A_{cr} = \pi \times 0.002^2/1 = 1.257 \times 10^{-5} \)

\[ K_{bulk} = K_s + A_{cr}(K_c + K_s) \]  \hspace{1cm} 4.22

\[ K_{bulk} = 8.7 \times 10^{-10} + 1.257 \times 10^{-5} (4.905 - 8.7 \times 10^{-10}) = 6.17 \times 10^{-5} \]

The influence of an individual tube of 2 mm diameter on the overall permeability of the top layer is quite large as it gives a resultant bulk permeability that is 5 orders higher than the permeability of the soil. Hence, earthworm holes at even a very low frequency will have a large impact on the hydrological response.

### 4.4.3 Numerical study of the influence of a single crack

In order to understand the drying and wetting of the clay around the crack and to replicate the transient boundary conditions a numerical study of a single crack was carried out.

A steady state analysis was first carried out to understand the effect of a high permeability crack on the hydraulic head distribution, flow path and flow volume of water infiltrating the soil.

**Steady state**

The influence of a single crack on the hydraulic properties of the soil was studied using the finite element software Seep/W (GEO-SLOPE, 2007). Boundary conditions were applied to a 2 dimensional soil layer 10 m thick and 15 m long. A crack of 1 mm width and 2 m depth was modelled, extending from the surface of the soil at the middle of the length. The crack was modelled as a rectangular region. The permeability of the soil layer modelled was 8.7 ×
$10^{-10}$ m/s. Both sides of the soil layer were modelled with a 100 mm thick layer with a very low permeability of the order of -200 to impose a near impermeable layer.

Heads of 10 m were applied to the top of the soil layer and a 0 m to the bottom. The resulting head distribution is shown in Figure 4:48. Figure 4:49 show the flow vector through and around the crack, and Figure 4:50 the flow path for the given geometry.

Figure 4:48: Effect of a single crack on the total head within the ground.

Crack influences the way the total head is distributed in the soil geometry modelled. High permeability inclusion allows the head applied at the top to be transmitted further down into the soil layer. Figure 4:49 and Figure 4:50 shows that the water flows down into the crack (funnelled) and flow down and laterally out of the crack.

The analysis also demonstrates that the seep/w is capable of modelling single cracks successfully. This approach is developed into transient analysis in the next section.
Figure 4.49: Flow vectors through and around the crack (At a magnification of $5 \times 10^7$)
Figure 4:50: Patterns of flow paths for the given geometry
**Transient Analysis**

Numerical studies to analyse the hydrological processes of slopes often use an increased vertical permeability near the surface of the slope to account for the influence of cracks (Rouainia et al., 2009, Briggs, 2011). However, with the knowledge of threshold suction (Shin and Santamarina, 2011) at which the cracks are initiated, it is possible to modify the hydraulic conductivity curve for the soil to represent the conductivity of cracked soil below the threshold suction.

**Finite-element modelling**

This numerical study on cracking was carried out using the finite-element software SEEP/W (GEO-SLOPE, 2007). It is possible to impose climate data collected from a weather station near the cut slope in Newbury to the surface of the model. This is the first time SEEP/W has been used to modify the hydraulic conductivity function of a continuum region to represent opening and closing of the crack.

SEEP/W calculates saturated and unsaturated water flow in response to applied boundary conditions. A notable feature of SEEP/W is that a transient climate boundary condition can be applied (this differs from most groundwater modelling reported in the literature). As described later, the climate boundary condition uses hourly weather data (rainfall, air temperature, humidity, wind speed and solar radiation) to calculate water infiltration and water removal from the surface of the soil. Variations in pore water pressure with time, in response to weather patterns of different duration and intensity, may then be investigated.

**Mesh geometry**

Models representing one-dimensional vertical flow through a column of soil were used to provide an insight into pressure head variation within soil profiles of different saturated hydraulic conductivity (Figure 5), in response to drying and wetting weather scenarios (Table 2). The models explore the extent to which a simple one dimensional model can provide useful comparisons with in assessing the influence of a single crack on the permeability of the soil (the pore water pressure response to long-term weather conditions). The models neglect the influence of downslope flow. Comparison between the one-dimensional column and a two-dimensional embankment model (using Vadose/W (Briggs et al., 2013)) shows that
the one-dimensional soil column approximates the midslope condition in a uniform embankment slope, subject to uniform downslope flow.

**Material properties**

As the soil becomes unsaturated, both its water content and its hydraulic conductivity decrease. This causes liquid and vapour flow rates, and rates of evaporation and transpiration, to reduce as the soil dries. Relationships between soil water content and soil suction, and soil water content and hydraulic conductivity, need to be obtained experimentally, but may for analytical convenience be represented by formulae such as those proposed by van Genuchten (1980) and Mualem (1976) respectively. The relationship between water content and suction for the in situ London Clay was based on the soil water retention curve (SWRC) measured for LondonClay by Croney (1977), is shown in Figure 4:51.

This was modified to adjust for maximum suction in field conditions associated with the wilting point of plants and used for this modelling (Figure 4:52). The Van Genuchten curve fit was used to derive parameters from the Croney curve. The hydraulic conductivity function was then calculated using these parameters, with a saturated hydraulic conductivity value of 5 x10⁹ m/s (Figure 4:53).
Figure 4:51: Soil water retention curve for London Clay (After Croney, 1977)

Figure 4:52: Soil water retention curve after limiting for plant wilting point (Modified from Croney, 1977)
Figure 4:53: Estimated hydraulic conductivity function for London Clay from Van Genuchten curve fit to the Croney curve.

**Plant wilting point**

The minimal point of soil moisture the plant requires not to wilt is defined as wilting point. If moisture decreases any lower a plant wilts and can no longer recover its turgidity when placed in a saturated atmosphere for 12 hours. The matric suction of the soil at this soil moisture condition is commonly estimated at (-1500) kPa.

During construction of the slope, top soil was placed on top of the London Clay surface to facilitate vegetation growth on the slope (Smethurst et al., 2006). From field measurements, this layer of fill material with a heavy presence of grass roots (referred to as the root zone hereafter) was observed to be about 100mm deep on average (Figure 4:54).
Figure 4:54: Layer of fill material with heavy presence of grass roots

To replicate this top soil, a material with hydraulic conductivity two orders of magnitude higher than London clay was used. The hydraulic conductivity and volumetric water content functions are shown in Figure 4:55 and Figure 4:56 respectively.
Figure 4:55: Hydraulic conductivity curve of root zone after limiting for plant wilting point
A crack was represented in the model by a V shape element of 1 mm width and 2m depth. The properties of the material have been derived by modifying the London Clay so as to represent the increase in the hydraulic conductivity of the medium after a cracking suction (also called critical suction) is exceeded. The cracking suction reported by Peron et al (2009a) for La Frasse clay of around 350 kPa was used in the absence of published results for London Clay. To achieve numerical convergence, the change in permeability was carried out over a suction range of 100 kPa up to 1000kPa (Figure 4:57). Similarly the volumetric water content function was modified so as to represent the drying up of water after the opening of cracks (Figure 4:58). In both of the cases, the crack is assumed to close perfectly when rewetting occurs (i.e. it returns to the same permeability as the in-situ soil). It is recommended that in future studies, the model is modified to represent a partial closure which is more representative of what happens in the field.
Figure 4.57: Hydraulic conductivity function of the crack
Figure 4: Volumetric water content function of the crack
Figure 4:59: Seep model showing a) The whole soil column modelled, b) The region near the crack

**Boundary and initial pore water pressure conditions**

A climate boundary condition was applied at the soil surface to determine pore water pressure changes in response to weather scenarios.
Modelling results

The development of suction with depth by the end of a two month drying and wetting period at half that of the evaporation rate at Newbury cut slope during the summer months as calculated by Smethurst et al (Smethurst et al., 2006, Smethurst et al., 2012) is compared between a model with crack and a model without any cracks. The comparison of drying is discussed first.

Figure 4:60 shows the soil column model without crack. Figure 4:61 shows the pore water pressure contours within soil column at the end of the two months drying period.
Model without crack – Drying

Following section describes a soil model used in analysis of soil without any crack during the process of continuous drying for two months at the half of average Newbury summer months evaporation rate. It also describes the results from the above analysis.

Figure 4:60: The soil column model without a crack
Figure 4: The soil column showing the pore water pressure contours at the end of the two months drying period.
Figure 4:62: Volumetric water content with height from bottom of the soil column along a vertical line positioned in the middle from the left and right boundaries.

Figure 4:62 shows the volumetric water content with height from bottom of the soil column considered along a vertical line positioned in the middle from the left and right boundaries. It shows the soil dries within 2.2 m depth from the surface level and below the depth of 2.2 m the soil is in saturated condition. More water is extracted from the surface than further down.
Figure 4:63: Volumetric water content changing with time at 0.5 m depth at the right boundary

Figure 4:64: Volumetric water content changing with time at 0.5 m depth in the middle of soil column
Figure 4:63 and Figure 4:64 show the volumetric water content changes with time at 0.5 depth below the surface at the right boundary of soil column and middle of soil column respectively. The volumetric water content values are similar at the right boundary of soil column and middle of soil column. The volumetric water content in both cases reduce linearly with time reaches the value of 0.455 from saturated level.

Figure 4:65 shows the pore water pressure changes with time at the centre of top surface of soil column. The negative pore water pressure develops approximately linearly with time reached the value of -85 kPa in 60 days.

Figure 4:65: Pore water pressure changing with time at the middle of surface boundary
Figure 4:66: Pore water pressure changing with time at 2 m depth in the middle from left and right boundary

Figure 4:66 shows the pore water pressure changes with time at 2 m depth from the surface in the middle from left and right boundary of soil column. The pore water pressure reduces to -3 kPa from 20 kPa.
Figure 4:67: Volumetric water content changing with height from the bottom of the soil column at different time intervals in the middle from the left and right boundary.

Figure 4:67 shows the volumetric water content changing with height from the bottom of the soil column at different time intervals in the middle from the left and right boundary. It shows unsaturated front is very slowly propagating downward with time. The soil reached the volumetric water content of 0.452 over 1440 hours.
Figure 4:68: Pore water pressure changing with time at 5 m depth at middle from left and right boundary

Figure 4:68 shows the pore water pressure changes with time at 5 m depth from the surface in the middle from left and right boundary of soil column. The pore water pressure reduces to 28 kPa from 50 kPa.
Figure 4:69: Pore water pressure changing with height from the bottom of the soil column at different time intervals at middle from the left and right boundary.

Figure 4:69 shows the pore water pressure changes with height from the bottom of the soil column at different time intervals in the middle from the left and right boundary. The suction front is propagating with time. The suction developed is showing a non-linear relationship with depth and suction reduces with depth from the surface of soil column.

**Model with crack – Drying**

Following section describes a soil model used in analysis of soil with one crack during the process of continuous drying for two months at the half of average Newbury summer months evaporation rate. It also describes the results from the above analysis.
Figure 4:70: Soil column model with 2.2 m deep crack
Figure 4:71: Suction generated in top of the soil column near the surface mainly including the grass root zone

Figure 4:70 shows the soil column model with crack of 2.22 m deep. The Figure 4:71 shows the suction generated in top of the soil column near the surface including the grass root zone. The suction gradient is higher near the surface. Each contours on the above diagram is denoting an increase of 500 kPa suction, starting at (-500) kPa pore pressure at bottom and ending at (-6000) kPa at the top.
Figure 4.72: Pore water pressure changing with height from the bottom of the soil column at different time intervals at the left top boundary.

Figure 4.72 shows the pore water pressure changes with height (from the bottom of the soil column) at different time intervals at the top left boundary. The suction front is propagating downward with time. The suction developed is showing a non-linear relationship with depth and suction reduces with depth from the surface of soil column.
Figure 4:73: volumetric water content changes with height from the bottom of the soil column at different time intervals at the left side - furthest form the crack

Figure 4:73 shows the volumetric water content changes with height from the bottom of the soil column at different time intervals at the left side furthest form the crack. It shows unsaturated front is propagating downward with time. Only top 1.2 m becomes unsaturated during the drying process at the location furthest away from the crack.
Figure 4.74: Pore water pressure changing with height (from the bottom of the soil column) at different time intervals along the middle of the crack

Figure 4.74 shows the pore water pressure changes with height from the bottom of the soil column at different time intervals along the middle of the crack. Higher suction is generated along the middle of the crack compared to the same depth furthest from the crack (Figure 4.72). The suction front is propagating downward with time. The suction developed is showing a non-linear relationship with depth and suction reduces with depth from the surface of soil column.
Figure 4:75: volumetric water content changes with height from the bottom of the soil column at different time intervals along the middle of the crack.

Figure 4:75 shows the volumetric water content changes with height from the bottom of the soil column at different time intervals along the middle of the crack. It shows unsaturated front is propagating downward with time. Only top 1.2 m becomes unsaturated during the drying process along the middle of the crack, which is similar to the furthest location from the crack (Figure 4:73). This shows same moisture gradient exist in these two locations at similar depth.
Comparison of wetting stage by a model of no crack and a model with one crack is discussed below.

Initial suction value of 500 kPa was assigned to the top of the soil column before the start of this part of analysis. This is to make sure there is sufficient soil moisture deficit in the soil column so that whole amount of water will infiltrate without run off.

Figure 4:76 shows the soil column modelled with no crack and a wetting boundary flux.

**Model without crack – Wetting**

Following section describes a soil model used in analysis of soil without any crack during the process of continuous wetting for two months at the half of average Newbury winter months precipitation rate. It also describes the results from the above analysis.
Figure 4:76: Soil column modelled with no crack and a wetting boundary flux
Figure 4:77 shows the suction contours generated with distance for the top portion of soil column. Each contour in the Figure 4:77 represents 50 kPa change and it starts at (-450) kPa from the bottom and the pore pressure reduces to (-100) kPa at the bottom of the red shaded region.
Figure 4:78: Pore water pressure changing with height (from the bottom of the soil column) at different time intervals at the left boundary of the soil column.

Figure 4:78 shows the pore water pressure changes with height (from the bottom of the soil column) at different time intervals at the left boundary of the soil column. The suction depletes with time at a given depth. Higher quantity of suction deplete nearer to the surface compared to further down at the left boundary of soil column.
Figure 4.79: Pore water pressure changing with height (from the bottom of the soil column) at different time intervals at the middle (centre) top of the soil column.

Figure 4.79 shows the pore water pressure changes with height from the bottom of the soil column at different time intervals at the middle top of the soil column. There is no change in the behaviour of suction depletion with water infiltration between ‘in the middle’ and ‘in the left boundary’ of soil column.
Figure 4:80: volumetric water content changes with height (from the bottom of the soil column) at different time intervals at the middle (centre) top of the soil column.

Figure 4:80 shows the volumetric water content changes with height (from the bottom of the soil column) at different time intervals at the middle (centre) of soil column. Volumetric water content increases with time at a given depth along the middle of soil column which implies degree of saturation also increases. The volumetric water content and thus the degree of saturation is high closer to the surface than at depth.
Figure 4:81: Pore water pressure obtained with time during drying and wetting stages both together.

Figure 4:81 shows the Pore water pressure obtained with time during drying and wetting stages. Suction is developed during the drying process to a value of -80 kPa at the end of 60 days (1440 hours). Suction development is almost linear with time. It should be noted that pore water pressure values were set to (-500) kPa at the top boundary of the model just before the wetting stage started. For the wetting stage the sudden depletion of suction at the beginning decreases with time and pore water pressure reaches the value of (-50) kPa in 60 days (1440 hours).

**Model with crack – Wetting**

Following section describes a soil model used in analysis of soil with one crack during the process of continuous wetting for two months at the half of average Newbury winter months precipitation rate. It also describes the results from the above analysis.
Figure 4:82: Soil model with crack for wetting stage showing the suction contours.
Figure 4:83: Top portion of Soil model with crack showing suction contours for wetting stage

Figure 4:82 shows the whole of soil model with crack for wetting stage. Figure 4:83 shows the suction contours in the top portion of soil model with crack for wetting stage. Each contours in this diagram are denoting a change of 50 kPa. The contour just below
the bottom of crack is -450 kPa, and this increases to -50 kPa at the bottom of the red shaded area near surface of the soil.

![Suction - Left side Top](image)

Figure 4:84: Pore water pressure changing with height (from the bottom of the soil column) at different time intervals at the left boundary (furthest from the crack) of the soil column.

Figure 4:84 shows the pore water pressure changes with height (from the bottom of the soil column) at different time intervals at the furthest from the crack. Wetting stage started just after 1440 hours at which the pore water pressure at the top was set to -490 kPa (-50 m water column). The suction depletes with time at a given depth. Higher quantity of suction depletes nearer to the surface compared to further down at the left boundary of soil column. The suction depletes deeper (1.5 m more depth) for soil column with crack compared to the soil column without crack (Figure 4:78).
Figure 4:85: Pore water pressure changing with height (from the bottom of the soil column) at different time intervals at the middle (centre) top of the soil column along the crack

Figure 4:85 shows the pore water pressure changes with height from the bottom of the soil column at different time intervals at the middle top of the soil column along the crack. More suction is depleted along the crack compared to the location furthest from the crack (Figure 4:84). Higher quantity of suction deplete nearer to the surface compared to further down along the crack.
Figure 4:86 shows the volumetric water content changes with height (from the bottom of the soil column) at different time intervals at the furthest location from the crack. Volumetric water content increases with time at a given depth which implies degree of saturation also increases. The volumetric water content and thus the degree of saturation is high closer to the surface than at depth. The degree of saturation is higher for the soil column with crack compared to soil column without crack (Figure 4:80).
Figure 4:87: volumetric water content changes with height (from the bottom of the soil column) at different time intervals at the middle (centre) top along the crack of the soil column.

Figure 4:87 shows the volumetric water content changes with height (from the bottom of the soil column) at different time intervals at the middle (centre) top along the crack. The material filled within the crack has different volumetric water content function with suction than that of London Clay thus the volumetric water content starts at approximately 0.18. In general, the volumetric water content increases with time at a given depth which implies degree of saturation also increases. The volumetric water content and thus the degree of saturation is high closer to the surface than at depth. The degree of saturation is higher along the crack than furthest from the crack (Figure 4:86).

Figure 4:88 shows the negative pore water pressure depletes with time in the location of crack 0.3 m depth below the surface.
Figure 4:88: Pore water pressure changing with time in the crack at 0.3 m depth from the surface.
Figure 4:89: Pore water pressure changing with time in the crack at 2 m depth from surface.

Figure 4:89 shows the negative pore water pressure depletes with time in the location of crack 2 m depth below the surface. The amount of suction depletion is very low around 80 kPa at the depth of 2 m compared to the value of 450 kPa approximately at the depth of 0.3 m along the crack (Figure 4:88).
Figure 4:90: Volumetric water content changing with time in the crack at 2 m depth from surface.

Figure 4:90 and Figure 4:91 show the volumetric water content with time at 2 m depth below the surface along the crack and furthest from the crack respectively. The volumetric water content increases with time but the amount of increment is very low which is approximately 0.006 m$^3$/m$^3$. The volumetric water content values at the furthest location is more than the values along the crack due to different properties of soils modelled.
Figure 4: Volumetric water content with time at the furthest from the crack at 2 m depth from surface

**Discussion on above four models**

During drying stage, the model with the crack generate suctions to a deeper level compared to the model without the crack (Figure 4:69, Figure 4:74). Near the surface level (at 11 m height) suction generated at the crack is about 10 times higher than the suction generated at the same level in the model without a crack.

Suction assigned is reduced to a greater depth in the model with crack than the model without crack for the wetting stage (Figure 4:85, Figure 4:78). For the model with the crack, Suction is reduced in the order of tens for the 0.5 m depth near the surface, whereas at 2 m depth it is still at -400 kPa level.
After the above mentioned numerical modelling a numerical simulation of the soil column model with crack was rerun with imposed real time transient resultant boundary flux.

Potential evapotranspiration for the Newbury slope was calculated on an hourly basis using the Penman-Monteith equation (4:23) for the period from 1st March 2006 to 6th July 2006.

\[
ET_0 = \frac{0.408 \Delta (R_n - G) + \gamma \frac{900}{T + 273} U_2 (e_s - e_a)}{\Delta + \gamma (1 + 0.34U_2)}
\]

where
- \(ET_0\) reference evapotranspiration [mm day\(^{-1}\)],
- \(R_n\) net radiation at the crop surface [MJ m\(^{-2}\) day\(^{-1}\)],
- \(G\) soil heat flux density [MJ m\(^{-2}\) day\(^{-1}\)],
- \(T\) mean daily air temperature at 2 m height [°C],
- \(U_2\) wind speed at 2 m height [m s\(^{-1}\)],
- \(e_s\) saturation vapour pressure [kPa],
- \(e_a\) actual vapour pressure [kPa],
- \(e_s - e_a\) saturation vapour pressure deficit [kPa],
- \(\Delta\) slope vapour pressure curve [kPa °C\(^{-1}\)],
- \(\gamma\) psychrometric constant [kPa °C\(^{-1}\)].

The equation uses standard climatological records of solar radiation (sunshine), air temperature, humidity and wind speed. To ensure the integrity of computations, the weather measurements should be made at 2 m (or converted to that height) above an extensive surface of green grass, shading the ground and not short of water.
Figure 4:92 and Figure 4:93 show the actual evapo-transpiration and resultant boundary flux changing with time respectively.

Figure 4:92: Actual Evapo Transpiration changing with time for the period considered
Figure 4:93: Resultant Boundary flux changing with time

Figure 4:94 shows the summary of resultant pore water pressure from pore pressure measurements at field and from model with and without cracks at the end of the period.
The trend of reduction in suction with depth is observed from the models and field data. The pattern (shape) of field data B well align with model results in the negative porewater pressure region. It should be noted that the field data is obtained for comparatively lower depths from the surface (less than 4 m). The lower suction is observed in the field compared to the predicted values from both the models (i.e with crack and without crack). Obtaining the field data for greater depths, (say 10 m) may be useful in analysing the correlation and validity of results from numerical modelling.

Figure 4:94: Variation of pore water pressure with depth derived from model and from field observations

- Model - No Crack
- Field Data A
- Field Data B
- Model Cracked
Figure 4:95: Variation of Vol. water content with time at 0.1 m depth from surface

Figure 4:95 shows the volumetric water content against time at 0.1 m depth below the surface. It shows that the soil column starts to dry and the volumetric water content drop dramatically to 0.18 at 400 hours then slowly increases to the saturation by 2000 hours followed by sudden drop to volumetric water content of 0.25 and then again reach the near saturation at 3100 hours.
Figure 4.96: Variation of Vol. water content with distance (depth) at different time intervals obtained from the model.

Figure 4.96 shows the volumetric water content with depth at different time intervals. The reduction in volumetric water content due to soil drying reduces with depth from surface.
Figure 4: Variation of pore-water pressure with depth for different time intervals obtained from the numerical model.

Figure 4 shows the pore water pressure with depth observed at different time intervals. It shows the trend of reduction in pore pressure values with increasing time. Matrix suction developed progressively up to 4m from the surface level. Suction developed within the soil column approximately up to 1.5m from surface level at 500 hours and progressed to 3.6 m at 3065 hours.
Figure 4: Variation of pore-water pressure with time at 0.3 m depth in the London Clay soil material

Figure 4:98 shows the pore water pressure changes with time at 0.3 m depth from surface level. The pore water pressure drops dramatically from hydrostatic level to -2800 kPa suction at 500 hours and then reaches the hydrostatic level by 2000 hours followed by a slight development of suction of -200 kPa. This shows the soil elements near the surface are highly sensitive to boundary flux.
Figure 4:99: Variation of total head with time at 1 m depth from the surface in London Clay soil material

Figure 4:99 shows the variation of total head with time at left boundary of the soil column at 1 m depth from the surface. The total heads reduces to approximately 3.3 m from 10m at 2000 hours followed by a rapid increase to 5.5 m and then decreases to 3.75 m by 3000 hours.

Figure 4:100 and Figure 4:101 show the extent of range of points used from the various curves mentioned above (Figure 4:53 and Figure 4:57) in the numerical simulation carried out.
Figure 4: Range of points used for the numerical simulation in the Variation of hydraulic conductivity with suction curve for London clay soil.

Figure 4:100 shows the range of hydraulic conductivity and suction values used in the above numerical simulation. The hydraulic conductivity varies between $10^{-5}$ and $2 \times 10^{-7}$ for the suction values between 0.7 kPa and 200 kPa.
Figure 4:101: Range of points used for the numerical simulation in the Variation of hydraulic conductivity with suction for the crack region soil

Figure 4:101 shows the range of hydraulic conductivity and suction values used in the above numerical simulation for the crack region soil material. The hydraulic conductivity values between $5 \times 10^{-7}$ and $2 \times 10^{-5}$ were used in the simulation for the suction values of 20 kPa to 200 kPa as shown in the figure.
Figure 4:102: Variation of pore-water pressure with time at 6.5 m depth from the surface in London Clay soil material.

Figure 4:102 shows the pore water pressure changes with time at the left boundary of soil column at 6.5 m depth from the surface. The pore water pressure reduces to approximately 20 kPa at 3000 hours from 63 kPa.
Figure 4:103: Variation of pore-water pressure with time at furthest from crack and 0.3 m depth from the surface in London Clay soil material
Figure 4:104: Variation of pore-water pressure with time in the middle of the crack at 0.3m depth

Figure 4:103 and Figure 4:104 shows pore water pressure with time at 0.3 m depth below surface at furthest from crack and middle of crack respectively. It shows very high suction values develop in the middle of crack compared to furthest from crack. Figure 4:103 shows the pore water pressure drops from hydrostatic level to approximately -175 kPs at 1500 hours and then raises closer to hydrostatic level at 2100 hours followed by the reduction to -185 kPa and returns back to -150 kPa by 3065 hours. Figure 4:104 shows the pore water pressure dramatically reduces to -1000 kPa at 400 hours and returns to -200 kPa then gradually increases to hydrostatic level at 2000 hours followed by a rapid drop to -800 kPa and slowly increases to -200 kPa by 3000 hours.
Figure 4:105: Variation of pore-water pressure with time in the middle of the crack at 1 m depth
Figure 4:106: Variation of pore-water pressure with time at the edge of the crack at 1 m depth

Figure 4:105 and Figure 4:106 show the pore water pressure with time at 1 m depth from surface at the middle and edge of the crack. The pore water pressure in the middle and edge of crack are similar. The pore water pressure reduces rapidly from the hydrostatic level to -120 kPa and returns to -60 kPa at 1000 kPa and remains until 2000 hours, which is followed by small kink and then reduces gradually.

4.5 Summary of Chapter 4

The pore pressures recorded during the test carried out in March 2012 showed a daily fluctuation in reading (for T1 up to +/-0.2 kPa, see Figure 4:107). This fluctuation was found to correlate with the Perspex tube expanding in a higher rate than water and
thereby a reduction in pressure when heated. This effect was overcome by taking moving average values over 24 hr periods.

Figure 4:107: Actual reading recorded from transducer T1 for experiment carried out in September 2012

In the field, a number of groups of inter connected cracks were observed at the surface level (within the top soil). Intact London Clay is heavily fragmented into clay blocks divided by very fine cracks in the order of 0.1 mm. Densely packed grass roots follow the path of the cracks from the topsoil/intact clay interface. This will accelerate the cracking process as the roots tend to absorb more water as they grow in the summer.

<table>
<thead>
<tr>
<th>Measurement type</th>
<th>Measurement depth (m)</th>
<th>Measured during</th>
<th>Average permeability (ms⁻¹)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double Ring</td>
<td>0.1</td>
<td>Nov/Dec 2011</td>
<td>2.2 x 10⁻⁸</td>
<td>Table 4:3</td>
</tr>
<tr>
<td>Double Ring</td>
<td>0.1</td>
<td>Mar 2012</td>
<td>3. x 10⁻⁸</td>
<td>Table 4:5</td>
</tr>
<tr>
<td>Double Ring</td>
<td>0.18</td>
<td>Sep 2012</td>
<td>1.3 x 10⁻⁶</td>
<td>Table 4:7</td>
</tr>
<tr>
<td>Tri axial tests</td>
<td>0.5 - 3.0</td>
<td></td>
<td>2.3 x 10⁻¹⁰</td>
<td>(Smethurst et al., 2006)</td>
</tr>
</tbody>
</table>

Table 4:9: Comparison of permeability measured in the Newbury site
The near surface vertical permeability measured in the field during the middle of winter 2011/2012 and at the end of winter 2011/2012 were in the order of $10^{-8}$ m/s. Vertical permeability measured in the triaxial apparatus on intact samples taken from 0.5 m to 3.0 m immediately after the construction of the cut slope was reported by Smethurst et al. (2006) to be in the order of $10^{-10}$ m/s. Hence there is an increase in the saturated permeability near the surface by an order of 2, as seen in the end of winter permeability measurement.

Near surface vertical permeability measured in the field at the end of summer 2012 is in the order of $10^{-6}$ m/s. This is two orders of magnitude higher than that of end of winter permeability. This can be attributed to the cracks opening over the summer.

Zhange et al (2018)(2018)(2018)(2018)(2018) reported a two orders of magnitude difference in the infiltration rate between the areas where opened cracks are visible and areas where no opened cracks are visible, on a slope made of expansive clay in China measured during the same time of the year (Figure 4:108 and Figure 4:109).

![Cracks observed by Zhange et al](image)


Some mathematical models have been developed to observe the flow through cracks are presented together with numerical model of a single crack which was carried out using SEEP/W software. The model with cracks funnels water into the ground by allowing it to pond in the crack before it can infiltrate. Evaporation from the model with cracks was greater than from the un-cracked model as the effective drying surface area was greater, by three or four times. The funnelling of water through cracks is thousands of times greater compared to the model without the crack. Hence the model with the cracks at the end of the year was wetter than that without the crack. The funnelling effect of cracks is two to three orders larger than the drying effects on a volumetric scale.

This section has described the field investigation and numerical modelling of cracks and the near surface vertical permeability, on the Newbury cut slope.

Field observations of cracks are reported. A number of groups of inter connected cracks are observed at the surface level (at top soil). Intact London Clay is heavily fragmented into clay blocks divided by very fine cracks in the order of 0.1 mm. Densely packed grass roots follow the path of cracks from the topsoil/intact clay interface. This will accelerate the cracking process as the roots tend to absorb more and more water in the summer.
A series of field permeability measurements during 2011-2012 are reported. The double ring infiltrometer experiment setup with additional pore water pressure measurements was used to measure the permeability of the soil. One end of summer time measurement and two winter time measurements were presented to compare the values of permeability and the influences of crack opening and closing. Near surface vertical permeability varied between summer and winter, probably mainly due to cracks opening and closing. End of summer vertical permeability (~$10^6$ m/s) was two orders of magnitude higher than that of end of winter permeability (~$10^8$ m/s). Near surface vertical permeability of the cut slope in Newbury (London clay found fragmented/cracked, $10^8$ m/s - $10^6$ m/s) is at least two orders of magnitude higher than that of intact London clay ($10^{10}$ m/s).

Some mathematical models of flow through cracks are presented together with a Seep/W model to investigate the influence of a single crack on wetting and drying of the soil.
5. Effects on the strength of the clay material

A series of experimental programme were carried out to:

- See what strains may be accumulated by repeated cycling of pore pressure of the sample at stress levels below critical and peak strength of the clay material.
- See whether repeated cycles may cause failure of the sample at a strength lower than that in monotonic loading and in particular, whether it may be possible to cycle over the Critical State Line (CSL), but below peak, and induce failure.

5.1 Triaxial testing programme

The clay samples used in this study were excavated from a railway embankment of Victorian age at Tumpy Green to the north of Bristol on the Bristol to Gloucester railway line (ELR BGL). The Bristol to Gloucester railway line opened in 1844 and it is now part of the main line from the Northeast of England through Derby and Birmingham to Bristol and the Southwest. The published geological map covering the Tumpy Green Embankment site, British Geological Survey Map Sheet 234 (1975), shows the site to be underlain by Lower Jurassic strata of Lower Lias Clay, bounded by outcrops of Pleistocene and Recent drift deposits. The drift materials include alluvial deposits to the east, estuarine alluvium to the west and a small outcrop of Terrace Gravel (third stage) to the north. Engineering properties of lower lias clay is shown in Table 5:1.
<table>
<thead>
<tr>
<th>Property</th>
<th>Lower Lias Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Weathered</td>
</tr>
<tr>
<td>Liquid limit, $W_l$ (%)</td>
<td>59 - 62</td>
</tr>
<tr>
<td>Plastic limit, $W_p$ (%)</td>
<td>23 - 25</td>
</tr>
<tr>
<td>Plasticity Index, $I_p$ (%)</td>
<td>36</td>
</tr>
<tr>
<td>Clay fraction ≤2 µm (%)</td>
<td>53</td>
</tr>
<tr>
<td>Void ratio, $e$</td>
<td>0.7 – 1.01</td>
</tr>
<tr>
<td>Natural water content (wt. %)</td>
<td>20 - 32</td>
</tr>
<tr>
<td>Specific gravity of particles</td>
<td>-</td>
</tr>
<tr>
<td>Bulk density, $\gamma_b$ (Mg/m$^3$)</td>
<td>1.96 – 2.05</td>
</tr>
<tr>
<td>Undrained shear strength $s_u$ (kN/m$^2$)</td>
<td>-</td>
</tr>
<tr>
<td>Effective cohesion, $c'$ (kN/m$^2$)</td>
<td>0 - 5</td>
</tr>
<tr>
<td>Effective angle of friction, $\phi'$ (degree)</td>
<td>24 - 27</td>
</tr>
<tr>
<td>Residual shear strength, $\phi_r$ (degree)</td>
<td>12.5 - 16</td>
</tr>
<tr>
<td>Coefficient of volume compressibility, $m_v$ (m$^2$/MN)</td>
<td>0.2</td>
</tr>
<tr>
<td>Coefficient of consolidation, $c_v$ (m$^2$/yr)</td>
<td>100 (laboratory)</td>
</tr>
<tr>
<td>Modulus of elasticity $E$ (Mn/m$^2$)</td>
<td>10 GN/m$^2$</td>
</tr>
</tbody>
</table>

Table 5.1: Engineering properties of Lower Lias clay (After Cripps and Taylor, 1987)

The soil samples were tested in a triaxial apparatus, to understand the behaviour of clay when subjected to cyclic stress changes representative of seasonal variations in pore pressure within a slope. The tests involved drained changes in pore water pressure under different stress regimes. This test programme differs from the experiment conducted by Santos et al (1997) in that a different soil type was used and local (in addition to global) instrumentation was used.

### 5.1.1 Block sampling and sample preparation

Block samples were extracted based on the guidelines given by Clayton et al (1982), who describe, in detail, the tools and methods for block sampling. The Lias Clay samples were taken in February 2011, at which time the soil was in a very wet
condition. All samples were carefully sealed on return to the laboratory using 7 layers of cling film and 7 layers of kitchen foil in alternative layers. Samples were stored in an air-conditioned environment of 20°C.

5.2 Monotonic Tests

To establish the strength of Lias Clay material used in the cyclic testing, undrained monotonic tests were carried out on in situ and remoulded samples of 100 mm diameter and 200 mm height. All the samples were sheared at an effective cell pressure of 50 kPa. Tests were carried out according to BS EN ISO 17892-9 (British Standard Institution, 2018). The experimental setup used for these tests is described in the following section.

5.2.1 Experimental setup

The experimental setup was similar to that used by Dykes (2008), based on a triaxial apparatus comprising a transparent cell and a Tri-Scan 50 kN load frame (Figure 5:1). This cell has an internal diameter of 230 mm to provide enough clearance for local instrumentation (including a radial strain caliper) to be attached to the sample. The local instrumentation consisted of an internal load cell to measure axial force; two Linear Variable Differential Transformers (LVDT) mounted vertically on opposite sides (180° apart on plan view) over the middle 40 mm of the sample to determine local axial strains; and an LVDT monitored radial strain caliper (Figure 5:2). The apparatus has a velocity-control based loading system. By connecting the loading frame to a PC via a serial connection, a stress path system can be created whereby the GDSLAB software dynamically changes the velocity of the system based on feedback from the load cell to achieve and maintain a required stress value. The individual components of the apparatus are discussed in more detail below.
Figure 5:1: Triaxial apparatus
5.2.1.1 GDS Pressure Controllers

Standard GDS pressure/volume controllers having a precision of 1 kPa were used to control both cell pressure and back pressure. The volume change increment is 1 mm$^3$ for a total volume of 200 cm$^3$. The maximum pressure of 1.7MPa was more than sufficient for the pressure ranges used in testing. GDS standard pressure/volume controllers include a serial connection for control and data logging within GDSL LAB.

The back pressure controller was used to apply the back pressure required and to maintain a zero volume change during B-value tests and possible undrained shearing. The controllers were filled using water from a Nold Deaerator; the use of de-aired water for both the pore fluid and cell fluid facilitates the dissolution of any air within the apparatus and the sample at relatively low pressures.
5.2.1.2 LVDTs used to measure axial and radial displacements

Local strains were measured using RDP D5W Linear Variable Differential Transformers (LVDTs). The LVDTs were connected via RDP S7AC D.C. powered transducer amplifiers, with a +/- 15v D.C power supply. Cable lengths were kept as short as possible, with pre-amplification and a cable shielding connected throughout, to reduce interference and hence increase the signal-to-noise ratio (SNR) of the LVDT output.

Two LVDTs were mounted axially on opposite sides of the sample between aluminium blocks fixed to the sample membrane using superglue, to measure displacements over the middle third of the sample. At small strains (<5%) it was assumed that axial strains in the sample and the membrane would be the same. Calibration of the LVDTs was carried out at 0.1mm intervals over the linear range of the transducer using a micrometer mounted in a stand. The micrometer has a quoted accuracy of 0.01 mm.

Radial strain was measured using a brass radial strain calliper based on a modified ‘lateral strain indicator’ described in Bishop and Henkel (1957). Figure 5:3 shows a schematic drawing of the radial calliper. The design is largely unchanged from Bishop and Henkel (1957); however, it features an LVDT instead of the mercury indicator. The LVDT armature was not fixed to the armature stop on the calliper; it was instead lightly spring-loaded with a sufficient force to hold the armature end against the stop. This ensures free vertical movement of the armature up and down, compensating the effect of gravity. The armature stop was adjustable and was initially set to be perpendicular to the armature.
5.2.1.3 Submersible Load Cell

The submersible load cell was a GDS 4 kN load cell. Using a proving ring or Sbeam transducer external to the cell is not ideal due to the influence of piston friction on the measured axial stresses; an internal load cell is free from this problem. The load cell is connected to a GDS serial pad (described in a later section) which, in turn, is connected to a GDS multiplexer. The load cell was calibrated using a dead-weight calibration system (DR Budenberg Ltd., 580-series hydraulic dead-weight tester). A fine calibration in steps of 0.2 kN was carried out up to 1kN, as this was the range used in the testing. The maximum error associated to the calibration of load cell was found to be ± 0.004 kN.
5.2.1.4 Pore Pressure Transducers

Back pressure was applied from top surface of the sample, and the pore pressure was measured at the middle, and bottom i.e. the furthest distance from the drainage boundary. The transducers were calibrated using a DH-Budenburg hydraulic dead-weight tester connected to the saturation chamber (completely water filled). The calibration was carried out in 0.5 bar (50 kPa) steps over the full linear range of the transducer. The maximum error during the calibration of pressure transducers was found to be ± 0.6 kPa. The transducers were then kept submerged in de-aired water until use.

5.2.1.5 GDSLAB Software

GDSLAB was used as the control and data acquisition package, in view of its flexibility in allowing the control of non-GDS equipment and the ability to add custom transducers in a non-standard setup. All of the instruments were logged using a personal computer through the GDSLAB software and a GDS Ltd data logging system (16 bit data acquisition).

5.2.1.6 GDS Serial Pad

The GDS serial pad offers 8 channel 16-bit data acquisition. “Gain” settings are managed by GDSLAB, and +/- 5V D.C. excitation voltage was provided for each channel. In accordance with Head (1986), the Serial Pad averages 16 readings taken over a few milliseconds to reduce the effect of electrical noise. Transducers were connected via 5-pin DIN connectors; however, to allow transducers to be changed, LEMO interconnects were used in the cable as their size allows them to be passed through the cell without rewiring the transducer.

5.2.1.7 Offsetting the controllers and transducers

Owing to the differences in manufacturing, and the different elevations of controllers and the transducers, there is a possibility that pressure readings at the back pressure and cell pressure controllers on the wall differ slightly from the corresponding readings of the transducers within the cell. This was overcome by equating the controller reading to that of the calibrated transducers, at pressures approximately equal to the testing values,
by manually setting a soft offset in the GDS software reading. Pore pressure was measured at the bottom and in the mid plane of the sample.

5.2.2 Tests on Undisturbed Samples

5.2.2.1 Sample Preparation

When preparing and setting up the test specimens, great care was taken to avoid any disturbance and minimise moisture loss. After removing the clay block samples from the wooden sampling box, the cling film and foil wrapping was removed and the sample was inspected for any evidence of disturbance. Typically, there were small fissures at the edges of the block sample, but the middle was in good condition, and an appropriate length was carefully selected for coarse cutting. Up to 4 samples having a diameter of 100 mm were produced from each block. The sample preparation procedure is explained below.

- Sample preparation was conducted in a temperature-controlled environment (20°C) and carried out as quickly as possible to prevent drying of the samples.

- Once a block sample had been quartered, the parts not immediately required were rewrapped in aluminium foil and cling film to preserve the moisture content.

- The clay was gently shaped using a trimming knife to approximately 125 mm diameter and 230 mm height. Then the sample was trimmed in a GDS manual soil lathe to the required diameter of 100 mm.

- To prevent damage and disturbance, manual handling of the sample was kept to a minimum and as soon as the sample was small enough, it was transferred to the soil lathe for further trimming. This offered firm support to the sample by fixing it between a base and a top platen.

- To minimise the disturbance to the sample, a dental pick was used to remove impurities (e.g. plant roots) apparent on the outer surface of the samples. The voids formed by removal of roots were filled with a stiff paste mixed from the trimming waste.
Once the sample had been trimmed to the correct diameter, it was removed from the lathe. The end of the sample was trimmed to the correct height.

Trimming of the samples to length was done using a straight edge pulled over the sample end. When carrying this out, the sample was supported in a 100 × 200 mm metal triaxial sample former to reduce lateral stresses on the sample, such that the trimmed edge was perpendicular to the longitudinal axis of the sample.

The stiff nature of the cut material made the sample preparation difficult. Plant roots hindered the sample cutting process and prevented smooth trimming of the sample; in some cases severely damaging the sample so that it could no longer be used for testing.

Comparatively weaker parts of the sample such as silt/sandy pockets, fissures and shear surfaces also led to difficulties in high quality sample preparation.

In general, Lias Clay has a very low permeability. A side drainage system made of vertical filter paper strips was used in the tests to facilitate consolidation (or swelling). Filter paper sheets of 100 mm diameter were placed at the top and bottom of the sample.

The latex membrane was carefully checked for defects before the test, and then soaked in de-aired water for at least 24 hours. After drying both the inside and outside surfaces using paper towels, the membrane was placed over the specimen. Then the specimen was placed horizontally on a cradle to measure and mark the positions of the LVDTs.

A positive pressure of 30 kPa was applied to the pedestal at the bottom by temporarily connecting the back pressure line until a thin layer of water was formed on the bottom high air entry ceramic. Then the specimen was slid carefully onto the base pedestal. The sliding action minimized the risk of trapping air between the sample and the ceramic. Two O-rings were used to seal the membrane onto the pedestal. After this, the top cap was placed in position. Prior to two O-rings being placed to seal top of the membrane around the top cap, any air trapped between the sample and the membrane was removed by gentle stroking upwards using fingers.

The positions for the LVDTs were marked following the placement of the specimen on the pedestal. A ruler and a sprit level were used to mark the vertical and horizontal lines
on the membrane specimen. Careful attention was paid to achieve near perfect vertical and horizontal lines.

The two pairs of brackets for the LVDTs were fixed at the pre-marked positions on the membrane using superglue adhesive. The brackets were held in position by hand for at least 3 minutes immediately after they had been glued onto the membrane, to allow the adhesive to gain its initial strength. Brackets were held in place by elastic bands for a further period of 15 minutes, while the adhesive developed its full strength.

Each axial LVDT was clamped vertically to the top bracket, carefully positioning the armature approximately at the electrical zero. Additional allowance was made for the axial deformations during the consolidation (or swelling) stage, based on experience from previous trial tests. The aim was to have the LVDTs as close as possible to the electrical zero point, once the sample had reached the target initial effective stress. The bottom brackets permitted a slight adjustment of the armature, if required.

Before closing the cell, the internal load cell was raised to its highest position to leave enough space for the specimen. The cell chamber had enough space to avoid any vibrations or disturbance to the LVDTs, permitting it to be lifted manually.

5.2.2.2 Saturation and consolidation

Alternate cell and back pressure increments were applied to achieve saturation. This method is accepted as normal practice for effective stress triaxial tests in which measuring shear strength at failure is the main objective (Head, 1986 page 793). In this method, the initial negative pore pressure reading is noted and the cell pressure is increased either by 50 or 100 kPa. The corresponding pore pressure rise is recorded after stabilisation of the reading.

The initial pore pressure coefficient B is then calculated from Equation 5.1:

\[ B = \frac{\Delta u}{\Delta \sigma} = \frac{u_i - u_o}{\Delta \sigma}, \]  

Then the back pressure valve is opened and the back pressure is applied to a predetermined value. This value is normally calculated based on the minimum effective stress to which the sample will be subjected under the initial stress conditions. After equilibrium is reached, the back pressure valve is closed. The cell pressure is increased
again and the prescribed steps are carried out to calculate the new B value. This process is continued until a satisfactory B value is achieved.

Lias Clay is a medium-stiff clay. The B value required for medium-stiff materials to reach the saturation of 99.0% is 0.93 (Table 5:2).

<table>
<thead>
<tr>
<th>Soil category</th>
<th>Degree of saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100%</td>
</tr>
<tr>
<td>Soft</td>
<td>0.9998</td>
</tr>
<tr>
<td>Medium</td>
<td>0.9988</td>
</tr>
<tr>
<td>Stiff</td>
<td>0.9877</td>
</tr>
<tr>
<td>Very stiff</td>
<td>0.913</td>
</tr>
</tbody>
</table>

Table 5:2: Values of B for typical soils at and near-complete saturation (after Black and Lee, 1973)

5.2.2.3 Shearing

Samples were sheared under undrained conditions. A strain rate of 0.5%/hour was used, which was sufficiently slow to capture the pore pressure response at the mid plane of soil samples. The results for saturation and shearing are described for all of the tests carried out in section 5.2.2.4.

Relevant plots are displayed under each tests and the strength of the material (the parameter of interest) is discussed thereafter.

5.2.2.4 Results

Test 1:

Figure 5:4 to Figure 5:8 show the various parameters obtained from Test 1, and pictures of the deformed test sample are given in Figure 5:9. Figure 5:4 shows the cell pressure and pore pressure readings during the saturation stage of Lias Clay where the effective confining pressure of 50 kPa maintained. It is notable that mid height pore pressure and base pressure transducer readings are similar over the saturation period.
Saturation stage

Figure 5:4: Cell and Pore pressure variation with time for specimen of Lias Clay at saturation stage (Test1)

Shearing stage

Local axial strain is more representative of the deformation of the sample. Hence the change of parameters plotted against the local axial strain is shown below. Figure 5:5 shows the plot of deviator stress against local axial strain of Lias Clay during shearing at an effective confining pressure of 50 kPa. The deviator stress attains the residual strength with axial deformation without passing through the peak value. This is expected for a clay material.

Figure 5:6 shows the pore pressure variations observed during undrained shearing condition. The excess pore pressure of 23 kPa developed during undrained condition. The secant modulus values for Lias Clay is shown in Figure 5:7. The secant modulus values show the clear trend of reduction for local axial strain larger than 0.005%. For local axial strains less than 0.005% the local axial LVDT measurements may not be
accurate and need more precise techniques. The reduction in secant modulus values during the test indicates that the initial stiffness reduces during shearing.

The effective stress path under undrained monotonic loading for Lias Clay is shown in Figure 5:8. The visual examination of clay sample before and after the undrained shearing is shown in Figure 5:9.

Figure 5:5: Deviator stress variation with local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test1)
Figure 5:6: Pore pressure variation with local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test1)

Figure 5:7: Stiffness variation with local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test1)
Figure 5:8: Deviator stress variation with mean effective stress for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test1)

Figure 5:9: Sample before and after the testing for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test1)

**Test 2:**

Figure 5:10 to Figure 5:15 show the various parameters obtained during Test 2 and the photographs of the sample before and after the test.
Figure 5:10 shows the cell pressure and pore pressure readings during the saturation stage of Test 2. The cell pressure increased in the steps of 100 kPa during saturation and effective confining pressure of 50 kPa obtained with saturation.

**Saturation stage**

![Graph of cell and pore pressure variation with time for specimen of Lias Clay at saturation stage (Test 2)](image)

Figure 5:10: Cell and Pore Pressure variation with time for specimen of Lias Clay at saturation stage (Test2)

**Shearing stage**

Figure 5:11 shows the deviator stress against local axial strain for Test 2. The deviator stress continues to increase with local axial strain toward the end. Figure 5:12 shows the excess pore water pressure developed during the undrained monotonic shearing measured by base and mid height pressure transducers. The readings observed from both base and mid height transducers were similar.

Figure 5:13 show the secant modulus values decrease with local axial strain implies reduction in stiffness during shearing. The effective stress path of Lias Clay observed during Test 2 is shown in Figure 5:14.

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Figure 5:15 shows the Lias Clay specimen before the undrained shearing and deformed shape after shearing.

Figure 5:11: Deviator stress vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 2).
Figure 5.12: Excessive pore water pressure vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 2)

Figure 5.13: Eu local vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 2)
Figure 5:14: Deviator stress vs mean effective stress from mid height pore pressure for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 2)

Figure 5:15: Sample before and after testing for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 2)

**Test 3:**

Figure 5:16 to Figure 5:21 shows the various parameters obtained during Test 3 and the photographs of the sample before and after the test.
Saturation stage

Figure 5:16 shows the cell pressure and pore pressure increments during the saturation stage for Test 3.

Shearing stage

Figure 5:17 shows that deviator stress increases with local axial strain and attains residual strength. Figure 5:18 shows the pore pressure measured from base and mid height transducers during undrained shearing. Figure 5:19 shows secant modulus reduces during shearing and the initial secant modulus value observed in Test 3 lower than Test 1 and Test 2. Figure 5:20 show the effective stress path of Lias Clay observed during Test 3 and the shape of stress path is similar to Test 2. Figure 5:21 shows the Lias Clay specimen before and after the undrained shearing during Test 3.
Figure 5:17: Deviator stress vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 3)

Figure 5:18: Pore pressure vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 3)
Figure 5:19: $E_u$ local vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 3)

Figure 5:20: Deviator stress vs mean effective stress from height pore pressure for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 3)
Figure 5.21: Sample before and after testing for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 3)

Test 4:

Figure 5.22 to Figure 5.28 show various parameters obtained during test 4 and the photographs of the sample before and after the test.

Saturation stage

Figure 5.22 shows the cell pressure and pore pressure increments during saturation of Lias Clay in Test 4.
Figure 5:22: Cell and pore pressure vs elapsed time for specimen of Lias Clay at saturation stage (Test 4)

Shearing stage

Figure 5:23 shows the deviator stress against local axial strain during undrained shearing of Test 4. Following rapid increment of deviator stress against local axial stress, the deviator stress continues to increase in slow pace, which is similar to Test 2. Figure 5:24 shows the pore pressure measurements from base and mid height transducers and shows slight variation of 3 to 4 kPa between both measurements. Figure 5:25 and shows secant modulus reduces during the shearing and initial stiffness losses completely towards the end of test. The effective stress path during undrained shearing of Test 4 is shown in Figure 5:26 and Figure 5:27 calculated from base and mid height pore pressure transducer readings respectively and the shape of effective stress path is similar to Test 1. Figure 5:28 shows the specimen before shearing and the deformed specimen shape. The specimen deformed more in top part tilted and bulged at the top end.
Figure 5:23: Deviator stress vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 4)

Figure 5:24: Pore pressure vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 4)
Figure 5:25: Eu local vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 4)

Figure 5:26: Deviator stress vs mean effective stress from base pore pressure for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 4)
Figure 5:27: Deviator stress vs mean effective stress from mid height pore pressure for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 4)

Figure 5:28: Sample before and after testing for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 4)

5.2.2.5 Discussion

Figure 5:29 shows the development of deviator stress with shear strain for all four samples. It is evident from the plots that the undrained shear strength of the clay given
by half the deviator stress varies between 18.75 kPa to 52.5 kPa. The development of mobilised effective angle of friction with shear strain in all four tests is shown in Figure 5.30.

Figure 5.29: Deviator stress vs axial strain for all four undisturbed specimens of Lias Clay sheared undrained at an effective stress of 50kPa
Figure 5.30: Mobilized friction angle vs axial strain for all four undisturbed specimens of Lias Clay sheared undrained at an effective stress of 50kPa

Figure 5.30 shows that the maximum effective friction angle varies from $28^\circ$ to $38^\circ$. This can be attributed natural variations in the strength. In view of the wide range of strength obtained, further investigations into the soil fabric and particle size distribution was carried out.

Gravel particles were present in all four samples when dissected after testing. Particle size distribution analysis carried out on vertically quartered portions of Sample 3 and Sample 4 (by K4 Soils Ltd) are given in Figure 5:31 and Figure 5:32 respectively. Sieve analysis was carried out for particles of 0.063 mm and above and sedimentation analysis was carried out on particles below 0.063 mm. Samples 3 and 4 were chosen for PSD because they are opposing ends of strength range (with sample 3 giving $28^\circ$ and sample 4 giving $38^\circ$). Sample 3 has slightly more gravel than sample 4 although otherwise the PSD are very similar as shown in Figure 5:31 and Figure 5:32. The location of the larger particles within the sample might be important, with their presence adjacent to the plane of failure in the sample potentially affecting its strength.
However the presence of the coarser particles on the plane of failure determines the degree of influence of these particles on the measured strength. Therefore, this should be interpreted qualitatively.

Figure 5:31: Particle size distribution for sample 3

Figure 5:32: Particle size distribution for sample 4
Another possible reason for the significant variation in critical friction angle calculated could be the influence of orientation and the shape of failure planes as described by Wong (1999). For a given material the principal stress ratio ($\sigma_1/\sigma_3$) measured during the failure can vary significantly depending on the shape of failure plane (Figure 5:33, Table 5:3).

![Kinematically admissible failure modes](image)

Figure 5:33 Kinematically admissible failure modes: (a) single plane; (b) conical; and (c) multiple planes. $\beta$, intersecting angle; $\delta_1$ and $\delta_2$, dip angles. (Wong, 1999)

The “residual” state or strength is usually used to refer to the state in clay at which the mineral platelets are oriented in a preferential direction due to large relative displacement without any volume change. For granular soils, residual state is equivalent to the critical state.
<table>
<thead>
<tr>
<th>Failure mechanism</th>
<th>Relationship of principal stress ratio to angle of failure plane &amp; critical friction angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single-plane</td>
<td>$\frac{\sigma_1}{\sigma_3} = \frac{1 + \tan \phi \tan \alpha}{\sin \alpha (1 - \tan \phi \cot \alpha)}$</td>
</tr>
<tr>
<td>Conical</td>
<td>$\frac{\sigma_1}{\sigma_3} = \frac{2 \tan \alpha}{\tan(\alpha - \phi)}$</td>
</tr>
<tr>
<td>Multiple shear-band (wedge)</td>
<td>$\frac{\sigma_1}{\sigma_3} = \frac{\sigma_1}{\sigma_3}(\delta_1, \delta_2, \beta, \phi)$</td>
</tr>
</tbody>
</table>

Table 5:3: Relationship between stress ratio ($\sigma_1/ \sigma_3$) and the relevant parameters for different failure shapes

Figure 5:34 shows the postulated failure patterns and orientation of the failure planes. However the shapes of failure planes could not be clearly established in these clay samples. The orientation and the shape of failure planes of different samples are different. Also, it is possible a combination of more than one of the failure mechanisms illustrated in Figure 5:33 and Table 5:3 is involved in these tests. In the absence of techniques such as CT scanning for these samples (unlike by other researchers (Perry et al., 2001)), quantification of this influence is not possible in this instance.
Figure 5:34: Postulated failure patterns of the all four undisturbed specimens of Lias Clay sheared undrained at an effective stress of 50kPa
5.2.3 Tests on Reconstituted samples

5.2.3.1 Sample Preparation

The reconstituted samples of embankment clay fill tested in the triaxial apparatus were prepared as follows:

1. The material (Lias Clay) used in the reconstitution process was obtained from the trimmings from the block samples used to produce the intact specimens for triaxial testing, and bagged clay obtained on site during excavation of the original block samples.

2. The clay was coarsely grated to prepare it for mixing. As a part of this process, all stones and organic material (such as plant roots) were removed.

3. The grated clay was placed in a drum clay mixer and mixed with de-aired water; water was added steadily, creating a reconstituted soil at a moisture content close to the liquid limit for this material.

4. After mixing; the material was left in the mixer for at least 48 hours to allow any remaining lumps of clay to absorb water and soften (the same principle as used for remoulding soil for liquid and plastic limit tests).

5. The soil was further mixed for several hours, with water added as necessary.

6. The mixed clay was placed into an aluminium strong box (internal dimensions of 550 mm x 200 mm in plan, and 250 mm deep) with sand and filter paper drains at both the bottom of the box and on the top surface of the clay to allow two-way drainage.

7. The soil in the strong box was vertically loaded using a hydraulic press, in a series of increments starting at 5 kPa and ending at 50 kPa, to achieve consolidation of the sample. The time taken to achieve consolidation was determined by measuring the vertical displacement of the soil (from the lid of the press), following the application of each load increment.

After consolidation, the strong box was removed from the press, the box was dismantled and the clay block was taken out. A similar procedure to that used for intact samples
described in section 5.2.2 was used to prepare the individual 100 mm, diameter 200 mm high samples for testing.

5.2.3.2 Saturation and consolidation

The method described in section 5.2.2.2 was used to saturate the samples. A B value greater than 0.93 was achieved in all cases, corresponding to 99% saturation.

5.2.3.3 Shearing

Samples were sheared at a strain rate of 0.5%/ hour which was sufficiently slow to capture the pore pressure response at the mid plane of soil samples. The results for saturation and shearing are described for all tests in section 5.2.3.4.

5.2.3.4 Results

Test 5:

Figure 5:35 to Figure 5:41 show the various parameters obtained during test 5 and photographs of the sample before and after the test. The strength of the material (the parameter of mean interest) is discussed later.
Saturation stage

Figure 5:35 shows the cell pressure increments and pore pressure changes during the saturation of Lias Clay for Test 5.

![Cell pressure and mid height pore pressure vs elapsed time](image)

Figure 5:35: Cell and pore pressure vs elapsed time for specimen of Lias Clay at saturation stage (Test5)

Shearing stage

Figure 5:36 shows the deviator stress against local axial strain during undrained shearing of Test 5. Following rapid increment of deviator stress against local axial strain, the deviator stress attains residual strength. Figure 5:37 shows the pore pressure measurements from base and mid height transducers and shows differences between both measurements unlike the tests on undisturbed samples.

Figure 5:38 shows the reduction of secant modulus during the shearing and initial stiffness losses. The effective stress path during undrained shearing of Test 5 is shown in Figure 5:39 and Figure 5:40 calculated from base and mid height pore pressure
transducer readings respectively. Figure 5:41 shows the specimen before shearing and the deformed specimen shape.

Figure 5:36: Deviator stress vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 5)

Figure 5:37: Pore pressure vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 5)
Figure 5:38: Eu local vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 5)

Figure 5:39: Deviator stress vs mean effective stress from base pore pressure for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 5)
Figure 5:40: Deviator stress vs mean effective stress from mid-height pore pressure for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 5)

Figure 5:41: Sample before and after testing for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 5)
Test 6:

Figure 5:42 to Figure 5:47 show the various parameters obtained during test 6 and the photographs of the sample before and after the test.

Saturation stage

Figure 5:42 shows the cell pressure and pore pressure increments during saturation of the reconstituted clay sample for Test 6.

![Graph showing cell pressure and pore pressure vs elapsed time](image)

Figure 5:42: Cell and pore pressure vs elapsed time for specimen of Lias Clay at saturation stage (Test6)

Shearing stage

Figure 5:43 shows the deviator stress against local axial strain during undrained shearing of Test 6. Following rapid increment of deviator stress against local axial stress, the deviator stress attains residual strength. Figure 5:44 shows the pore pressure
measurements from base and mid height transducers and shows differences between both measurements similar to that in Test 5.

Figure 5:45 shows the reduction of secant modulus during the shearing and initial stiffness losses. The effective stress path calculated from mid height pore pressure transducer during undrained shearing of Test 6 is shown in Figure 5:46. The specimen deformation in Test 6 is shown in Figure 5:47. It shows that specimen top was tilted during shearing.

![Deviator stress vs local axial strain](image)

**Figure 5:43:** Deviator stress vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 6)
Figure 5:44: Pore pressure vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 6)

Figure 5:45: Eu local vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 6)
Figure 5:46: Deviator stress vs mean effective stress from mid-height pore pressure for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 6)

Figure 5:47: Sample before and after testing for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 6)

**Test 7:**

Figure 5:48 to Figure 5:53 show various parameters obtained during Test7 and the photographs of the sample before and after the test.
Saturation stage

Figure 5:48 shows the cell pressure and pore pressure readings during Test 7.

![Pressure vs time graph](image)

Figure 5:48: Cell and pore pressure vs elapsed time for specimen of Lias Clay at saturation stage (Test7)

Shearing stage

Figure 5:49 shows the deviator stress against local axial strain during undrained shearing of Test 7 and deviator stress continues to increase in slow pace without reaching clear residual strength. Figure 5:50 shows the pore pressure measurements from base and mid height transducers and slight differences between both readings.

Figure 5:51 and shows the reduction of secant modulus during the shearing and initial stiffness losses.

Figure 5:52 shows the effective stress path of reconstituted Lias Clay calculated from mid height pore pressure transducer during undrained shearing of Test 7. The sample before and after the Test 7 is shown in Figure 5:53.
Figure 5:49: Deviator stress vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 7)

Figure 5:50: Pore pressure vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 7)
Figure 5:51: Eu local vs local axial strain for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 7)

Figure 5:52: Deviator stress vs mean effective stress from mid-height pore pressure for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 7)
Figure 5:53: Sample before and after testing for specimen of Lias Clay sheared undrained at an effective stress of 50kPa (Test 7)

5.2.3.5 Discussion

The development of shear stress against axial strain for all three reconstituted samples is shown in Figure 5:54. The undrained shear strength of the clay is consistent between 22.5 kPa and 25 kPa. Figure 5:55 indicates an average value of 25\(^0\) for the critical state friction angle for reconstituted Lias clay. Sample 7 doesn’t display a clear peak strength and it continues to rise which is possibly due to barrelling of sample (Figure 5:56). Normally in calculations the exact shape of the sample deformation is not accounted when adjusting for change in the cross-section area.
Figure 5.54: Undrained shear strength vs external axial strain for all three reconstituted specimens of Lias Clay sheared undrained at an effective stress of 50kPa.

Figure 5.55: Mobilised friction angle vs external axial strain for all three reconstituted specimens of Lias Clay sheared undrained at an effective stress of 50kPa.
Table 5:4 summarises the undrained shear strength and friction angle values obtained for Lias Clay undisturbed and reconstituted samples. The undisturbed samples show variations in undrained shear strength and friction angles while reconstituted samples show similar values. The variations observed in undisturbed samples could be due to presence of coarser particles in failure plane and influence of orientation and the shape of failure plan.

Table 5:4: Summary of monotonic tests carried out on Lias Clay and their strength

<table>
<thead>
<tr>
<th>Sample Name</th>
<th>Type</th>
<th>Undrained Shear Strength (kPa)</th>
<th>Friction Angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>Undisturbed</td>
<td>44</td>
<td>38</td>
</tr>
<tr>
<td>Sample 2</td>
<td>Undisturbed</td>
<td>26</td>
<td>27</td>
</tr>
<tr>
<td>Sample 3</td>
<td>Undisturbed</td>
<td>18</td>
<td>25</td>
</tr>
<tr>
<td>Sample 4</td>
<td>Undisturbed</td>
<td>53</td>
<td>37</td>
</tr>
<tr>
<td>Sample 5</td>
<td>Reconstituted</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>Sample 6</td>
<td>Reconstituted</td>
<td>23</td>
<td>25</td>
</tr>
<tr>
<td>Sample 7</td>
<td>Reconstituted</td>
<td>22</td>
<td>30</td>
</tr>
</tbody>
</table>

Figure 5:56: Barrelling of sample
5.3 Analysis of cyclic triaxial tests on Gault Clay fill

5.3.1 Introduction

To understand the behaviour of the embankment fill material subjected to cyclic changes of pore water pressure, a laboratory experiment was undertaken by Dykes (2008). An experimental set up similar to that described in section 5.2.1 was used in these tests.

Dyke’s (2008) tests were carried out on Gault clay fill from Charing embankment (Briggs, 2010). The data were never analysed. Native GDS files from the tests were retrieved and analysed as part of the current study. This section presents the processed data from these experiments and discusses the behaviour of the material.

5.3.2 Results

Three samples (named A, B & C hereafter) were tested in a triaxial cell. The samples were saturated and consolidated under a constant deviator stress prior to cycles of back pressure being applied.

5.3.2.1 Saturation

Alternate cell and back pressure increments were applied to achieve saturation. Figure 5:57 to Figure 5:59 show the cell pressure and pore pressure variations during the saturation of the samples A, B and C respectively.
Figure 5.57: Cell pressure and back pressure variation during the saturation of sample A

Figure 5.58: Cell pressure and back pressure variation during the saturation of sample B
For all three samples a B value of 0.93 or above was achieved. This corresponds to 99.5% saturation (Black and Lee, 1973).

### 5.3.2.2 Consolidation and Behaviour under cycles

For all three samples, a deviator stress of 30 kPa was applied. The calculations of stresses that are acting on an element of soil in an infinite slope are shown in section 5.4.1. However the geometric properties used by Dykes (2008) to arrive at the above mentioned values and the cycling there after is unknown, as the calculation is lost. The back pressures were brought to the datum point of the cycling program, the cell pressure was brought to the value shown in Table 5:5 and the samples were left to consolidate. Then cycles of back pressure were applied from the top and the pore pressure responses at 1/3rd depth, 2/3rd depth and at the bottom of the sample were measured. The back pressure was cycled as indicated in Table 5:5. Cycle time was long enough for the pore pressure measured at the far end to be close enough to the applied back pressure.
Table 5.5: Pore pressure cycles applied to samples

<table>
<thead>
<tr>
<th>Sample Name</th>
<th>Deviator Stress</th>
<th>Cycle PP Pressure Range (kPa)</th>
<th>Datum (kPa)</th>
<th>Cell Pressure (kPa)</th>
<th>Effective Stress range (kPa)</th>
<th>Cycle Time (hr)</th>
<th>No of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>30</td>
<td>205 to 285</td>
<td>245</td>
<td>320</td>
<td>115 to 35</td>
<td>5</td>
<td>24</td>
</tr>
<tr>
<td>B</td>
<td>30</td>
<td>220 to 280</td>
<td>250</td>
<td>320</td>
<td>100 to 40</td>
<td>24</td>
<td>18</td>
</tr>
<tr>
<td>C</td>
<td>30</td>
<td>200 to 260</td>
<td>230</td>
<td>300</td>
<td>100 to 40</td>
<td>12</td>
<td>19</td>
</tr>
</tbody>
</table>

**Sample A**

Figure 5:60 to Figure 5:64 show the variation of the different parameters recorded and calculated from sample A. Average values of two mid plane pore pressure readings were used as the pore pressure in calculations of effective stress. The parameters of interest are discussed in section 5.3.3.

Figure 5:60 show the deviator stress against axial strain during the consolidation of sample A of Gault clay fill. The deviator stress of 30 kPa applied during consolidation stage while cell pressure and pore pressure were brought to 320 kPa and 245 kPa respectively.

Figure 5:61 shows the applied back pressure and the measured pore pressure during the cyclic stage of sample A with a duration of 5 hours/ cycle where the back pressure cycled between 205 kPa to 285 kPa. The pore pressure reading used for analysis is the average of the pore pressure responses at 1/3 depth called mid plane pore pressure – near end, and the pore pressure responses at 2/3 depth called mid plane pore pressure – far end.

Figure 5:62 shows the global axial strain accumulation with time during the 24 cycles of effective radial stress range between 115 kPa and 35 kPa. The rate of axial strain accumulation reduces with time. At start of cycle, the effective radial stress was brought to 35 kPa from 75 kPa datum value, thus negative axial strain seen in Figure 5:62 attributed to radial expansion of specimen or axial shortening as a result of this stress stage.
Figure 5:63 shows the mean effective pressure against the axial strain during cyclic stage. Figure 5:64 shows the axial strain accumulation with time from local LVDT readings. It can be seen from Figure 5:62 and Figure 5:64 that the axial strain accumulation is 0.1 % more when measured using local axial LVDT.

Figure 5:60: Variation of deviator stress with axial strain for consolidation stage
Figure 5.61: Variation of back pressure and pore pressure with time

Figure 5.62: Variation of global axial strain with time
Figure 5.63: Variation of mean effective stress with axial strain

Figure 5.64: Variation of local axial strain with time

Note: One of the Local Axial LVDT (LA1) failed after few cycles
Sample B

Figure 5:65 to Figure 5:70 show the parameters recorded and calculated from sample B. Figure 5:65 shows deviator stress against axial strain for the consolidation stage of sample B. The deviator stress of 30 kPa applied during consolidation stage while cell pressure and pore pressure were brought to 320 kPa and 250 kPa respectively.

Figure 5:66 shows the applied back pressure and the measured pore pressures from three locations during the cyclic stage of sample B with a duration of 24 hours/cycle.

Figure 5:67 shows the global axial strain accumulation with time during the 18 cycles of effective radial stress range between 100 kPa and 40 kPa. The rate of axial strain accumulation reduces with time. At start of cycle, the effective radial stress was brought to 40 kPa from 70 kPa datum value, thus negative axial strain attributed to radial expansion of specimen or axial shortening as a result of this stress stage.

Figure 5:68 shows the variation of mobilised friction angle with mean effective stress during the cyclic stage. The mobilised friction values varies between $7.5^0$ and $14.5^0$ approximately with varying mean effective stress.

Figure 5:69 shows the axial strain accumulation with time from two local LVDT readings. It can be seen from Figure 5:67 and Figure 5:69 that the global axial strain accumulation are closer to the local axial strain 1 measurements.

Figure 5:70 shows the local radial strain variations with time during the cyclic stage. The direction of radial strain accumulation is not clear and a polynomial curve is fitted to see the radial strain accumulation pattern.

Further parameters of interest are discussed in the discussion section in detail.
Figure 5.65: Variation of deviator stress with axial strain for consolidation stage

Figure 5.66: Variation of back and pore pressures with time for Sample B
Figure 5.67: Variation of global axial strain with time

Figure 5.68: Variation of mobilised friction angle with mean effective stress
Figure 5:69: Variation of local axial strain with time

Figure 5:70: Variation of local radial strain with time
Sample C

Figure 5:71 to Figure 5:84 show the variation of different parameters recorded and calculated from sample C.

Figure 5:71 and Figure 5:72 show deviator stress against axial strain and deviator stress against time for the consolidation stage of sample C. The deviator stress of 30 kPa applied when specimen underwent 0.6% axial strain during consolidation stage while cell pressure and pore pressure were brought to 300 kPa and 230 kPa respectively.

Figure 5:73 shows the applied back pressure and the measured pore pressures from three locations during the cyclic stage of sample C with a duration of 12 hours/cycle.

Figure 5:74 shows the effective axial stress calculated during cyclic stage of specimen C. As the measured pore pressures 10 kPa less than applied back pressures (Figure 5:73), the calculated effective axial stress fall below the targeted values and cycles between 80 kPa and 120 kPa.

Figure 5:75 shows the global axial strain accumulation with time during the 19 cycles of effective radial stress range between 40 kPa and 100 kPa. The rate of axial strain accumulation reduces with time. At start of cycle, the effective radial stress was brought to 40 kPa from 70 kPa datum value, thus negative axial strain attributed to radial expansion of specimen or axial shortening as a result of this stress stage.

Figure 5:76 shows the axial strain accumulation with time from two local LVDT readings. It can be seen from Figure 5:75 and Figure 5:76 that the global axial strain accumulation varies between local axial strain 1 and local axial 2 measurements where permanent axial deformation is closer to local axial strain 1 and resilient axial deformation (amplitude) is closer to local axial strain 2.

Figure 5:77 shows the variation of mobilised friction angle with mean effective stress during the cyclic stage. The mobilised friction values varies between $8^\circ$ and $14^\circ$ approximately with varying mean effective stress.

Figure 5:78 shows the mean effective stress with axial strain during cyclic stage where mean effective stress varies between 60 kPa and 100 kPa.
Figure 5:71: Variation of deviator stress with axial strain

Figure 5:72: Variation of deviator stress with time
Figure 5:73: Variation of pressure with time

Figure 5:74: Variation of effective axial stress with time
Figure 5:75: Variation of global axial strain with time

Figure 5:76: Variation of local axial strain with time
Figure 5.77: Variation of mobilised friction angle with mean effective pressure

Figure 5.78: Variation of mean effective stress with axial strain
Comparison of results for all three samples

Figure 5:79 shows effective axial stress against number of cycles for all three samples. Figure 5:80 and Figure 5:81 show axial strain variation with time and cycles. Sample B accumulated more strain compared to sample A and C. Table 5:6 shows the strain accumulation of all three samples at 10th and 15th cycle. The sample C shows less resilient strain (amplitude) compared to sample A and B and this can be attributed to the comparatively lower axial stress cyclic range experienced by sample C as shown in Figure 5:79.

![Figure 5:79: Variation of effective axial stress with cycle no for all three samples.](image)

Table 5:6 shows the strain accumulation of all three samples at 10th and 15th cycle.
Figure 5:80: Variation of axial strain with time for all three samples.

Figure 5:81: Variation of axial strain with cycle No for all three samples.
In all samples, the axial strain (Figure 5:81) indicated a cyclic response in line with the applied changes in pore pressure. The axial strain increased with the number of cycles, but the rate of increase gradually reduced as cycling continued and eventually tended towards a stable state. Pore pressure cycling led to the compaction and stabilisation of each sample.

The cycles of pore water pressure

The amplitude of the axial strain cycle reduced with the number of cycles. For sample B there is a sudden increase in the amplitude of cycle after the 7th which was a result of sudden increase in the pore water pressure measured in the other end (Figure 5:66). This increase in pore pressure remote from the controlled boundary may have been due to fissures opening with in the sample.

Relative position of cycles to critical state line

The critical state parameter M is a measure of the ratio of shear to the normal effective stress at failure. It is, therefore, related to the effective soil friction angle \( \phi'_{\text{crit}} \) (Powrie, 2004).

For triaxial tests in compression, relationship between M and \( \phi'_{\text{crit}} \) is:

\[
M = \frac{6 \sin \phi'_{\text{crit}}}{(3 - \sin \phi'_{\text{crit}})}
\]

For Gault Clay, \( \phi'_{\text{crit}} = 23^\circ \) is used and using equation 5:2

<table>
<thead>
<tr>
<th>Sample</th>
<th>Strain accumulated after 10 cycles</th>
<th>Strain accumulated after 15 cycles</th>
<th>Strain accumulated after 23 cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.11</td>
<td>0.14</td>
<td>0.16</td>
</tr>
<tr>
<td>B</td>
<td>0.28</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.15</td>
<td>0.175</td>
<td></td>
</tr>
</tbody>
</table>

Table 5:6: Summary of strain accumulated with no of cycles
\[ M = \frac{6 \sin 23^\circ}{(3 - \sin 23^\circ)} = 0.90 \]  

Critical state line for soils can be described as

\[ q = Mp' \]

where, \( q \) – deviatoric stress

\( p' \) – effective mean stress

\( M \) – critical state parameter.

Figure 5: Variation of deviator stress with mean effective stress for sample A
The variation in deviator stress with mean effective stress is shown in Figure 5:82 to Figure 5:84. The critical state line is also indicated. It is evident from these graphs that the stress state experienced by the sample during the cycles of pore water pressure was
below the critical state line, even at its closest possible point. Therefore, the repeated cyclic effective stress loading acted so as to compact rather than shear the samples.

5.3.2.3 Shearing and Peak strength

Samples were sheared at the end of the cycling stage at a rate of 0.15 %/hour. The results of the shearing stage for samples A, B & C are presented below.

Sample A

Figure 5:85 shows the deviator stress against axial strain during post-cyclic shearing of sample A. Figure 5:86 shows the effective stress ratio with axial strain. Both deviator stress and effective stress ratio continue to increase with axial strain but at decreasing rate.

Figure 5:87 shows the effective stress ratio against mean effective stress. Figure 5:88 shows the mobilised friction angle against mean effective stress. The effective stress ratio and mobilised friction angles show similar trend with mean effective stress.

Figure 5:85: Variation of deviator stress with axial strain for sample A
Figure 5:86: Variation of effective stress ratio with axial strain for sample A

Figure 5:87: Variation of effective stress ratio with mean effective stress for sample A
Figure 5:88: Variation of mobilised friction angle with mean effective stress for sample A

Sample B

Figure 5:89 shows the effective stress ratio with axial strain. The effective stress ratio increases with axial strain up to 2 % then follows by slight reduction with axial strain.

Figure 5:90 shows the $t$ against $s'$ for sample B. Figure 5:91 shows the mobilised friction angle against mean effective stress. It shows increasing mobilised friction angle with reducing mean effective stress. Figure 5:92 shows the deviator stress against mean effective stress.
Figure 5:89: Variation of effective stress ratio with axial strain during shearing for sample B

Figure 5:90: Variation of parameter t changing with s’ for sample B
Figure 5:91: Variation of mobilised friction angle with mean effective stress for sample B

Figure 5:92: Variation of deviator stress with mid height mean effective stress for sample B
Sample C

Figure 5:93 shows the deviator stress against axial strain and it shows deviator stress increases with axial strain and then stabilises for specimen C during post-cyclic shearing. Figure 5:94 shows the pore pressure measurements obtained in three locations with axial strain. It shows the differences in pore pressure readings, which were measured at different locations, but the differences is only within approximately 6 kPa.

Figure 5:95 shows the deviator stress against mean effective stress. Figure 5:96 shows the effective stress ratio against mean effective stress. Figure 5:97 shows the mobilised friction angle against mean effective stress. It shows an overall trend of increasing mobilised friction angle with reducing mean effective stress.

Figure 5:98 shows effective stress ratio against axial strain. It shows effective stress ratio increases with axial strain until 3.5 % then reduces gradually and then stabilises. Figure 5:99 shows t against s’ for sample C.

![Figure 5:93: Variation of deviator stress with external axial strain for sample C](image-url)
Figure 5:94: Variation of pore pressure with mid height mean effective stress for sample C

Figure 5:95: Variation of deviator stress with mid height mean effective stress for sample C
Figure 5:96: Variation of effective stress ratio with mid height mean effective stress for sample C

Figure 5:97: Variation of Mobilised friction angle with mean effective stress for sample C
Figure 5:98: Variation of effective stress ratio with axial strain for sample C

Figure 5:99: Variation of parameter t changing with $s'$ for sample C
Comparing all three samples

Fig 5:100 shows $t$ against $s'$ for sample A, B, and C. It shows sample B and sample C intercept the path in $t$ against $s'$ plot while sample A follows separate path.

Figure 5:100: Variation of parameter $t$ changing with $s'$ for all three samples

Figure 5:101: Variation of effective stress ratio with axial strain for all three samples
The stress ratio $q/p'$ is plotted against axial strain in Figure 5:101. A $q/p'$ ratio of about 1.30 is achieved at the peak state for samples B and C. This corresponds to an effective friction angle of $33.0^\circ$ as shown in Figure 5:102.

![Figure 5:102](image)

Figure 5:102: Variation of mobilised friction angle with axial strain for all three samples

5.3.3 Discussion & Implications

Permanent axial strains accumulated with cycles of pore water pressure. The rate of strain accumulation decreases with the number of cycles when the effective stress ratio is below that corresponding to the critical state line.

From the pore water pressure variation plots during the cyclic stage (Figure 5:61, Figure 5:66, and Figure 5:73) it is visible there is an attenuation and a phase lack in the pore water pressure response when moving away from the applied pore water pressure boundary. If a sinusoidal back pressure change is imposed it is suggested a longer period should be allowed such that the far end of the sample will reach the full level in pore pressure change.

For embankment fill materials, it is possible that fissures act as preferential paths for water flow. Clay samples taken from embankment fill material should be investigated for occurrence of fissures using advance Computer Topography (CT) scanning. In the
triaxial tests, pore water pressures measured at the far end may not be the actual pore pressure experienced in the sample throughout the clay because of preferential pathways due to fissures, cracks and other micropores. This is not the case in the intact materials from cuttings taken at depth, as they tend to be homogeneous and free from macro pores.

Other researchers have found out the anisotrophy in strength in triaxial tests (Nishimura et al., 2007) and also the dependency of strength on structure of the clay material (Gasparre, 2005, Gasparre et al., 2007). This is something that should be taken into account when preparing the soil samples for testing on triaxial apparatus.

For triaxial tests on embankment clay fill materials an improved method would be to use the stabilisation of back volume as a check to decide that the entire sample has reached the intended back pressure. For this, a step change method of cycling the pore pressure should be adapted.
5.4 Cyclic testing programme on Lias Clay

Cyclic triaxial tests were carried out on the Upper Lias Clay samples from a railway embankment. This clay was selected for testing owing to its higher permeability than intact London Clay. In addition, the Lias Clay is more prone to strain softening.

5.4.1 Calculation of the stresses to be applied to the sample

Vaughan and Walbancke (1973) suggested that a cut slope tends to lose some of its horizontal in situ stress following excavation as the clay material swells. A series of subsequent numerical studies (Potts et al., 1997) found that cut slopes with a lower $K_0$ take a relatively short time to dissipate the suction generated by excavation, and consequently fail more quickly than slopes with a high $K_0$.

Field monitoring carried out at the Newbury site (Smethurst et al., 2006) indicated pore water pressures near hydrostatic during the winter of 2002/2003. This shows that the suction generated due to excavation within the slope has already dissipated, and it could be assumed that the release of horizontal stress is almost complete. A back analysis of the slope demonstrated that a reasonably high permeability, particularly near the surface, is required to model the seasonal changes observed in the site (Rouainia et al., 2009). A relatively high permeability reinforces the likelihood that any suctions generated by excavation have already dissipated, leading to a considerable amount of horizontal stress release.

A research study by the Transport Research laboratory (Carder and Easton, 2001, Carder and Temporal, 2000) indicated that cut slopes on motorways can undergo greater swelling movements; this is a possible indication of significant stress release. Stress reduction is likely to be aided over time with cycles of seasonal effective stress, close to the slope surface. In this study the earth pressure coefficient $k$ was assumed to be equal to unity ($K_0=1$).

The stress conditions were calculated for a depth of 2 m below the existing surface.
Figure 5:103: Forces acting on an imaginary slice of soil considered in an infinite slope

The long term stability of slopes in clay can be investigated by idealising the slope as infinite and uniform, and considering the total stress acting on a plane parallel to the surface of the slope at a depth of \( z \) from the slope surface as shown in Figure 5:103. For a unit length along the slope at an angle \( \beta \) to the horizontal, the weight \( W \) of the block of soil ABCD is \( W = \gamma z \cos \beta \). The side forces \( X \) are, by symmetry, equal and opposite, so that the shear stress \( \tau \) and the normal total stress \( \sigma \) acting on the plane at a depth \( z \) below the soil surface may be found by resolving forces in parallel and in perpendicular to the slope:

\[
T = W \sin \beta = \gamma z \cos \beta \sin \beta \\
\sigma = W \cos \beta = \gamma z \cos^2 \beta
\]

These forces are represented in Figure 5:104 by a Mohr circle of stress.
Figure 5:104: Mohr circle of stress for plane at depth Z parallel to slope surface

\[ s = \frac{(\sigma_1 + \sigma_3)}{2} \] \hspace{1cm} 5:7

\[ t = \frac{(\sigma_1 - \sigma_3)}{2} \] \hspace{1cm} 5:8

\[ \sin \beta = \frac{(s - \gamma z \cos^2 \beta)}{t} \] \hspace{1cm} 5:9

\[ \cos \beta = \frac{\gamma z \cos \beta \sin \beta}{t} \] \hspace{1cm} 5:10

(5:8, 5:10) \Rightarrow

\[ t = \gamma z \sin \beta = \frac{\sigma_1 - \sigma_3}{2} \] \hspace{1cm} 5:11

(5:9, 5:11) \Rightarrow

\[ t \sin \beta = \gamma z \sin^2 \beta = s - \gamma z \cos^2 \beta \] \hspace{1cm} 5:12
\[ s = \gamma z (\sin^2 \beta + \cos^2 \beta) = \gamma z \]  

\[ s = \gamma z = \frac{(\sigma_1 + \sigma_3)}{2} \]

\((5:11, 5:13) \rightarrow \)

\[ \sigma_3 = s - t = \gamma z (1 - \sin \beta) = \text{cell pressure (CP)} \]

\[ q = \sigma_1 - \sigma_3 = 2\gamma z \sin \beta \]

\[ \sigma_1 = \sigma_3 + 2\gamma z \sin \beta = \gamma z (1 + \sin \beta) \]

For the Newbury slope \(\beta = 14^0\); however, for a better representation of a steep slope a value of \(20^0\) is assumed. Considering a soil sample located 2m below the slope surface, the following values of the in-situ stress state can be driven.

For total stress conditions, \(\gamma = \gamma_{sat}\)

For London Clay:

\(G_s = 2.65\), Saturated volumetric water content = 0.45.

\[ \gamma_{sat} = (2.65 \times 0.55 + 1.0 \times 0.45) \times 9.81 \text{ kN/m}^3 \]

\[ = 18.713 \text{ kN/m}^3 \]

Cell Pressure \(\sigma_3 = \gamma z (1 - \sin \beta) = 18.713 \times 2 \times (1 - \sin 20^0) = 24.626 \text{ kPa}\)

\[ q = 2\gamma z \sin \beta = 2 \times 18.713 \times 2 \times \sin 20^0 = 25.6 \text{ kPa}\]

Field monitoring by Smethurst et al (2006) for the grass covered cut slope at Newbury in the London Clay indicates that the pore water pressure fluctuations at a depth of 0.4 m have an upper boundary of 4 kPa in winter and a lower boundary of less than (-90kPa) in summer. Hence the range of the pore pressure cycle is more than 94 kPa. An approximated range of 100 kPa was used in the testing programme. To mimic this process in the triaxial cell the following cell pressure and back pressure values were selected.
Cell pressure 600 kPa, back Pressure 575 kPa $\rightarrow$ Effective Pressure 25 kPa

Cell pressure 600 kPa, back Pressure 475 kPa $\rightarrow$ Effective Pressure 125 kPa

5.4.2 Specimen Set up

A similar experiment set up used for monotonic tests (described in section 5.2.1) was used for this test (Figure 5:105), except that a different sample size 70 mm in diameter and 140 mm in height was used.
Figure 5:105: Triaxial apparatus set up used for this test

The sample preparation method described in Section 5.2.2.1 was also used in this test.
5.4.3 Saturation & consolidation

The same method described in section 5.2.2.2 was used to saturate the samples and a B value of above 0.94 was achieved corresponding to above 99% saturation (Black and Lee, 1973). Figure 5:106 shows the variation of pressure with time during the saturation process of the sample.

![Graph showing variation of pressure with time during saturation process](image)

Figure 5:106: Variation of pressure with time

After saturation, the cell pressure was kept at 600 kPa and the back pressure was brought to 525 kPa which was the datum for the cycling programme. After this, a deviator stress of 25.4 kPa was applied and the sample was left to consolidate. Figure 5:107 and Figure 5:108 show the variation of deviator stress with axial strain and axial strains with time respectively. Figure 5:108 shows that the axial strain obtained by external measurement is as twice as much of average of local axial LVDT measurements.
Consolidation

Figure 5:107: Deviator stress vs. Axial strain

Figure 5:108: Axial strains vs. Time
The variation of deviator stress with mean effective stress for the consolidation stage is shown in Figure 5.109. The variation of stress ratio with mean effective stress for the consolidation stage is shown in Figure 5.110.

Figure 5.109: Variation of deviator stress with mean effective stress
Figure 5:110: Variation of effective stress ratio with mean effective stress

5.4.4 Cycling the pore pressure

In this test programme, the pore pressure was switched between high and low points leaving enough time to consolidate and swell. Back pressure was applied at the top and bottom of the specimen and the volume change was used as the controlling factor. To determine a reasonable half-cycle time, the following steps were followed.

The back pressure was reduced from the datum (525 kPa) to the low point of the cycle (475 kPa) and the sample was allowed to consolidate for three days. After consolidation, the back pressure was brought to the high point of the cycle (575 kPa) and the sample was left to swell for three days. Figure 5:111 shows the volume change calculated using the readings from the back pressure controller. In this graph, a red line is drawn to show the potential stabilising line for volume change. Another line corresponding to 95% value of the volume change was drawn and the time corresponding to this was noted. This time is approximately 172000 seconds (48 hrs). A half cycle time of 48 hours was therefore used in this testing programme.
Stage 1:

The effective stress ratio mobilised in the “low”, “datum” and “high” conditions is shown in Table 5:7. In stage 1, 26.5 cycles were applied on this sample (Figure 5:112). The response of the sample is shown in Figure 5:115 to Figure 5:120.

<table>
<thead>
<tr>
<th>Radial Pressure (kPa)</th>
<th>Pore Pressure (kPa)</th>
<th>Deviator Stress (kPa)</th>
<th>Eff. Radial Stress (kPa)</th>
<th>$\delta_i'$</th>
<th>Axial Stress (kPa)</th>
<th>$P'=(\delta_1'+2\delta_3')/3$ (kPa)</th>
<th>$\sin \phi'$ mob</th>
<th>$\phi'$ mob = $\frac{1}{2}(\delta_i'-\delta_3')/(\delta_1'+\delta_3')$ (Degrees)</th>
<th>$q / p'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>575</td>
<td>25.6</td>
<td>25</td>
<td>50.6</td>
<td>33.53</td>
<td>0.34</td>
<td>19.79</td>
<td>0.76</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>525</td>
<td>25.6</td>
<td>75</td>
<td>100.6</td>
<td>83.53</td>
<td>0.15</td>
<td>8.38</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>475</td>
<td>25.6</td>
<td>125</td>
<td>150.6</td>
<td>133.53</td>
<td>0.09</td>
<td>5.33</td>
<td>0.19</td>
<td></td>
</tr>
</tbody>
</table>

Table 5:7: Stress ratios for the cycle
The variation of the cell and pore pressure with time and the corresponding variation of stress ratio are shown in Figure 5:112 and Figure 5:113 respectively. The corresponding variation of stress ratio with cumulative global strain is shown in Figure 5:114.
Figure 5.113: Variation of stress ratio with Time

Figure 5.114: Variation of stress ratio with global axial strain
Cumulative global and local axial strains for the Stage1 stress cycles are shown in Figure 5:115 and Figure 5:116 respectively. It shows that global axial measurement is
larger than local axial measurement. These graphs show that permanent axial strains accumulated with an increasing number of cycles of pore water pressure. The rate of accumulation of strain decreases with the number of cycles.

Figure 5:117: Variation of volume change with time (Water out +ve)

Figure 5:117 shows the volume change of sample with time. It shows the overall trend of volumetric compression of sample with pore water pressure cycles. Figure 5:118 shows the volume change with time for each step. Figure 5:119 shows the volumetric strain of sample with time. Figure 5:120 shows the volume change of sample against mean effective stress where the mean effective stress varies approximately between 30 kPa to 130 kPa.
Figure 5:118: Variation of volume change with time for each step

Figure 5:119: Variation of volumetric strain (%) with time
At the end of 26.5 cycles, the back pressure was increased to 580kPa and 48 hours were allowed for the swelling of the sample. Based on the observed response of the sample and the reading tolerance of the controller and transducers, it was decided to increase the back pressure to 585kPa. A further 24 hours were allowed for the back volume to stabilise. The response of the sample during these two steps is given in Figure 5:121 to Figure 5:127.
Figure 5:121: Variation of cell and pore pressures with time

Figure 5:121 shows the cell pressure and pore pressure readings during the pore water pressure cycles, and then during above mentioned two steps. The back pressure increment during the end of cyclic stage to let the specimen swell is clearly shown in by the highlighted circle.

Figure 5:122 shows the stress ratio with time. Following the cyclic stage, the stress ratio increased in two steps to 1.1 during the back pressure increment steps which can be clearly differentiated.

Figure 5:123 shows the stress ratio against global axial strain. The sample allowed to swell over 48 hours soon after cyclic stage by increasing the back pressure to 580 kPa and then to 585 kPa. The axial shortening of sample during the swelling can be clearly seen.

Figure 5:124 and Figure 5:125 shows the global axial strain and local axial strain with time respectively. Axial strain measurement during the above two steps after the pore pressure cyclic stage is highlighted and it shows the reducing axial strain with time.
Figure 5:122: Variation of stress ratio with time

Figure 5:123: Variation of stress ratio with global axial strain
Figure 5.124: Variation of Global Axial strain with time

Figure 5.125: Variation of Local Axial strain with time
Figure 5.126: Variation of volume change with time

Figure 5.127: Variation of volumetric strain (%) with time
Figure 5:126 and Figure 5:127 shows the volume change and volumetric strain with time respectively. The highlighted region shows the water moves into the sample during the above-mentioned two steps, which implies sample swelling.

At the end of this stage, the load cell was found to have drifted with time, by approximately 6 kPa (i.e reading was 6kPa when undocked). Using the volumetric strain vs time plot (Figure 5:128 ), the point, at which the load cell started to drift, was identified; just after 16\textsuperscript{th} cycle. To avoid this issue in future tests, it was decided to undock the load cell after each cycle (normally 4 days) and re-set the reading to zero (done within 30 Sec).
Stage 2:

The back pressure was cycled between 485kPa and 585kPa with the cycle times given in Table 5:8. Effective stress ratios corresponding to this condition are shown in Table 5:9.

<table>
<thead>
<tr>
<th>Cycle Time (hr)</th>
<th>No of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>96</td>
<td>4</td>
</tr>
<tr>
<td>144</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 5:8: Cycle Time and corresponding number of cycles for stage 2

<table>
<thead>
<tr>
<th>Radial Pressure (kPa)</th>
<th>Pore Pressure (kPa)</th>
<th>Deviator stress (kPa)</th>
<th>Eff. Radial Stress (kPa)</th>
<th>$\delta_1'$</th>
<th>$\delta_3'$</th>
<th>$P'_*=(\delta_1'+2\delta_3')/3$ (kPa)</th>
<th>$\sin\phi'_*$ mob</th>
<th>$\phi'_*$ mob = $\sin^{-1}(\delta_1'-\delta_3')/(\delta_1'+\delta_3')$ (Degrees)</th>
<th>$q / p'_*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>585</td>
<td>25.6</td>
<td>15</td>
<td>40.6</td>
<td>23.53</td>
<td>0.46</td>
<td>27.41</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>535</td>
<td>25.6</td>
<td>65</td>
<td>90.6</td>
<td>73.53</td>
<td>0.16</td>
<td>9.47</td>
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<tr>
<td>600</td>
<td>485</td>
<td>25.6</td>
<td>115</td>
<td>140.6</td>
<td>123.53</td>
<td>0.10</td>
<td>5.75</td>
<td>0.21</td>
<td></td>
</tr>
</tbody>
</table>

Table 5:9: Effective stress ratio corresponding to the "high", "datum" and "low" points of this cycling stage

The response of the sample is shown in Figure 5:129 to Figure 5:138.

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Figure 5:129 shows the cell pressure and pore pressure measurements with time during stage 2 where cycle time of first 4 cycles and next 2 cycles are twice and thrice as much as stage 1.

Figure 5:130 shows the stress ratio against global axial strain. The arrow shows the direction of movement between high and low points of axial strain within cycles. Figure 5:131 and Figure 5:132 shows the axial strain from global measurement and local measurement respectively. Axial strain from global measurement is higher than local measurement.
Figure 5:130: Variation of stress ratio with global axial strain

Figure 5:131: Variation of global axial strain with time
Figure 5:132: Variation of local axial strain with time

Figure 5:133: Variation of volume change with time (Water out +ve)

Figure 5:133 and Figure 5:134 shows the volume change and volumetric strain with time respectively. These show a trend of slight swelling following the initial
compression. Whereas Figure 5:135 and Figure 5:136 show volumetric strain and volume change with time relative to each step respectively.

Figure 5:137 and Figure 5:138 show the volumetric strain and volume change against mean effective stress respectively. The arrows indicate the direction of movement between high and low points of cyclic stage.

Figure 5:134: Variation of volumetric strain with time
Figure 5:135: Variation of volumetric strain with time for each step

Figure 5:136: Variation of volume change with time for each step
Figure 5:137: Variation of volumetric strain with mean effective stress (kPa)

Figure 5:138: Variation of volume change with mean effective stress (kPa)
Stage 3:

After the cyclic programme described in stage 2, it was decided to raise the cell and back pressure, to 900 kPa and 885 kPa respectively (with the effective stress remaining unchanged), to eliminate any doubt about undissolved air. After this step, a further cycling program was carried out at the following stress conditions.

<table>
<thead>
<tr>
<th>Cell Pressure (kPa)</th>
<th>High Pore Pressure (kPa)</th>
<th>Low Pore Pressure (kPa)</th>
<th>No of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>900</td>
<td>885</td>
<td>785</td>
<td>8</td>
</tr>
<tr>
<td>900</td>
<td>887</td>
<td>787</td>
<td>4</td>
</tr>
<tr>
<td>900</td>
<td>890</td>
<td>790</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 5:10: Cycle range and corresponding number of cycles for stage 3
<table>
<thead>
<tr>
<th>Radial Pressure (kPa)</th>
<th>Pore Pressure (kPa)</th>
<th>Deviator Stress (kPa)</th>
<th>Eff. Radial Stress (kPa)</th>
<th>61'</th>
<th>Axial Stress (kPa)</th>
<th>P&quot;=(61'+263')/3 (kPa)</th>
<th>sinφ'</th>
<th>mob</th>
<th>φ'mob=sin⁻¹(61'-63')/(61'+63') (Degrees)</th>
<th>q / p'</th>
</tr>
</thead>
<tbody>
<tr>
<td>900</td>
<td>885</td>
<td>25.6</td>
<td>15</td>
<td>40.6</td>
<td>23.53</td>
<td>0.46</td>
<td>27.41</td>
<td>1.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>835</td>
<td>25.6</td>
<td>65</td>
<td>90.6</td>
<td>73.53</td>
<td>0.16</td>
<td>9.47</td>
<td>0.35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>785</td>
<td>25.6</td>
<td>115</td>
<td>140.6</td>
<td>123.53</td>
<td>0.10</td>
<td>5.75</td>
<td>0.21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Radial Pressure (kPa)</td>
<td>Pore Pressure (kPa)</td>
<td>Deviator Stress (kPa)</td>
<td>Eff. Radial Stress (kPa)</td>
<td>61'</td>
<td>Axial Stress (kPa)</td>
<td>P&quot;=(61'+263')/3 (kPa)</td>
<td>sinφ'</td>
<td>mob</td>
<td>φ'mob=sin⁻¹(61'-63')/(61'+63') (Degrees)</td>
<td>q / p'</td>
</tr>
<tr>
<td>900</td>
<td>887</td>
<td>25.6</td>
<td>13</td>
<td>38.6</td>
<td>21.53</td>
<td>0.50</td>
<td>29.74</td>
<td>1.19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>837</td>
<td>25.6</td>
<td>63</td>
<td>88.6</td>
<td>71.53</td>
<td>0.17</td>
<td>9.72</td>
<td>0.36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>787</td>
<td>25.6</td>
<td>113</td>
<td>138.6</td>
<td>121.53</td>
<td>0.10</td>
<td>5.84</td>
<td>0.21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Radial Pressure (kPa)</td>
<td>Pore Pressure (kPa)</td>
<td>Deviator Stress (kPa)</td>
<td>Eff. Radial Stress (kPa)</td>
<td>61'</td>
<td>Axial Stress (kPa)</td>
<td>P&quot;=(61'+263')/3 (kPa)</td>
<td>sinφ'</td>
<td>mob</td>
<td>φ'mob=sin⁻¹(61'-63')/(61'+63') (Degrees)</td>
<td>q / p'</td>
</tr>
<tr>
<td>900</td>
<td>890</td>
<td>25.6</td>
<td>10</td>
<td>35.6</td>
<td>18.53</td>
<td>0.56</td>
<td>34.15</td>
<td>1.38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>840</td>
<td>25.6</td>
<td>60</td>
<td>85.6</td>
<td>68.53</td>
<td>0.18</td>
<td>10.13</td>
<td>0.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>790</td>
<td>25.6</td>
<td>110</td>
<td>135.6</td>
<td>118.53</td>
<td>0.10</td>
<td>5.98</td>
<td>0.22</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5:11: Effective stress ratio corresponding to the "high", "datum" and "low" points of this cycling stage.

Cycling programme and the response of the sample are shown in Figure 5:139 to Figure 5:147.
Figure 5:139: Variation of pore pressure with time (Different sub-stages are highlighted with arrow brackets)

Figure 5:139 shows pore pressure against time during stage 3 where pore water pressure is cycled at elevated cell pressure and back pressure values compared to stage 1 and stage 2. Sample failed at the end of 16\textsuperscript{th} cycle (i.e.: 4\textsuperscript{th} cycle at 890 kPa stress condition).

Figure 5:140 and Figure 5:141 shows the axial strain calculated from global measurement and local LVDT measurement respectively. At the end of 16\textsuperscript{th} cycle axial strain continues to increase rapidly which indicates failure.
Figure 5:140: Variation of global axial strain with time

Figure 5:141: Variation of local axial strain (2) with time
Figure 5:142: Variation of stress ratio with global axial strain

Figure 5:142 shows the stress ratio against global axial strain. Sample failure indicated by continuous increase in global axial strain at stress ratio 1.4.

Figure 5:143 and Figure 5:144 show the volumetric strain and volume change with time relative to each step respectively. The volume behaviour of specimen during the sixteen cycles and failure is shown in Figure 5:145 and Figure 5:146 in terms of volumetric strain and volume change respectively. Sample failure is indicated by sudden larger quantity of water flow in to the specimen as shown in Figure 5:146.

Figure 5:147 shows the volume change against mean effective stress.
Figure 5:143: Variation of volumetric strain with time for each step

Figure 5:144: Variation of volume change with time for each step
Figure 5.145: Variation of volumetric strain with time

Figure 5.146: Variation of volume change with Time
5.4.5 Summary of Cyclic Testing Programme on Lias Clay

Permanent axial strains are accumulated with cycles of pore water pressure, although the rate of accumulation of strain decreases with an increasing number of cycles.

The following plots (Figure 5:148 to Figure 5:162) show the merged and combined plots for all three stages.
Figure 5:148: Variation of pore pressure with time

Figure 5:148 shows the variation of pore pressure with time for all three stages.

Figure 5:149 shows the axial strain of sample during the all three stages until failure. As an overall trend, the axial strain increases with time and the strain accumulation rate is reduces. The sample settled 1.96 mm (1.2 % axial strain) in vertical direction with cycled pore pressure until it experienced failure at stage 3.
Figure 5.149: Variation of axial strain with time
Figure 5:150 shows the axial strain with time of all three stages separately. Axial strain accumulates at a higher rate for stage 1 than stage 2 than stage 3. The amplitude of the cycles decreases with successive stages. It is possible that the compaction induced stiffness increase plays a role in the reduction of amplitude of the cycles.
Figure 5:151 shows the local axial strain with time for all three stages. The axial strain increases with time in decreasing rate. The axial strain calculated from local LVDT measurements is lower than the global measurement in all three stages (Figure 5:149 and Figure 5:151). The local axial strain variation with time for all the stages is shown separately in Figure 5:152. It shows that axial strain accumulation is more for stage 1 than stage 3 than stage 2. Anyhow, the axial strain difference between stage 2 and stage 3 are very low compared to stage 1.

Figure 5:153 and Figure 5:154 show the volumetric strain with time for all three stages together and separately in a plot respectively. It shows that sample compresses in stage 1 and then follow the trend of swelling with subsequent cycles. The amplitude of volumetric strain is higher for stage 1 than stage 2 than stage 3. Thus, resilient volumetric stain reduces with stages.
Figure 5:152: Variation of local axial strain with time - all three stages separately

Figure 5:153: Variation of volumetric strain with time
Figure 5:154: Variation of volumetric strain with time - all three stages separately

Figure 5:155: Variation of stress ratio with time - all three stages separately
Figure 5:155 and Figure 5:157 show the stress ratio experience by sample in all three stages separately and together against time respectively. The sample underwent more stress ratio in subsequent stages. In stage 3, the stress ratio is progressively increased in three sub stages and sample failed at stress ratio of approximately 1.38. Figure 5:156 show the stress ratio against axial strain for all three stages. It shows that axial strain accumulates with increasing stress ratio.

Figure 5:159 shows the stress ratio accumulated in all three stages in separate plots against axial strain accumulation. Figure 5:160 show the volumetric strain against mean effective stress in all three stages in separate plots. Figure 5:161 and Figure 5:162 show volumetric strain and volume change related to each step with time respectively for all three stages separately.

Figure 5:156: Variation of stress ratio with axial strain - all three stages separately
Figure 5.157: Variation of stress ratio with time

Figure 5.158: Variation of stress ratio with axial strain - all three stages separately
Figure 5: Variation of stress ratio with axial strain; (a) Stage 1 (b) Stage 2 (c) Stage 3
Figure 5: Variation of volumetric strain with mean effective stress; (a) Stage 1 (b) Stage 2 (c) Stage 3
Figure 5.161: Variation of volumetric strain with time - all three stages separately

Figure 5.162: Variation of volume change with time - all three stages separately
Failure occurred in the form of slip at the end of the 4\textsuperscript{th} cycle at 890 kPa. The back pressure was kept at 890 kPa for a further 96 hours. However, the sample slipped once again and the failure was not continuous. Hence, it was decided to apply one more cycle, after which a gradual slide occurred becoming very active afterwards. This additional cycle and the response of the sample are shown in Figure 5:163 to Figure 5:165.

Figure 5:163 shows the pore pressure and cell pressure readings with time during the one extra cycle applied. The sample failure in the form of gradual slide is indicated by the massive axial settlement occurred after the one extra cycle as shown in Figure 5:164 and Figure 5:165.

![Variation of cell pressure and pore pressure with time](image)

Figure 5:163: Variation of cell pressure and pore pressure with time
Figure 5.164: Variation of global axial strain with time
Figure 5:165: Variation of local axial strain with time
The post-failure sample is shown in Figure 5:166 to Figure 5:171. A clear failure plane is evident (Figure 5:167). In addition to the number of stones of about 5 – 15 mm diameter, several stiff roots of approximately 1-3 mm diameter were found perpendicular to the failure plane (Figure 5:170 to Figure 5:171.)
Figure 5:167: Sample after test - Failure plane marked
Figure 5:168: Divided sample along failure plane – Side elevation
Figure 5:169: Divided sample along failure plane – Top down view
Figure 5: Location of roots and stone particles found in the top surface of the failure plane
Stress condition under which sample failed ($\phi'_{mob}=34^\circ$) is in between high and low stress condition experienced (i.e. mobilized friction angles observed between 28° and 37°) during the monotonic tests on the fill material samples of Lias Clay which are reported at chapter 5.2.

The higher than average strength of the sample obtained is due to the influence of stone particles and stiff roots found in the failure plan.

Monotonic tests were carried out at 50 kPa, but cyclic test closer to failure stages were carried out on much lower effective stresses (5 kPa, 10 kPa). At lower effective stresses peak failure envelop of clay tends to curve (Figure 5:172) and is above critical state line. This may be another reason for higher than average stress conditions for being sustained by the sample.
5.5 Summary of Chapter 5

Tests carried out on Gault Clay by Dykes (2008) were cycled too far away from critical state line. In other words, use of 23° as an initial guess for critical state line is too low. At failure, fill samples of Gault clay have developed friction angles much higher than this when failed under monotonic shearing (Section 5.3.2.3).

Stress condition under which Lias Clay cyclic sample failed (\(\phi'_{mob}=34^\circ\)) is in between high and low stress condition experienced (i.e. mobilized friction angles observed between 28° and 37°) during the monotonic tests on the fill material samples of Lias Clay which are reported at chapter 5.2.

The higher than average strength of the Lias Clay cyclic sample obtained is due to the influence of stone particles and stiff roots found in the failure plan.

Monotonic tests were carried out at 50 kPa, but cyclic tests closer to failure stages were carried out on much lower effective stresses (5 kPa, 10 kPa). At lower effective stresses
peak failure envelop of clay tends to curve and is above critical state line. This may be another reason for higher than average stress conditions for being sustained by the sample.

A laboratory study investigating the effect of pore water pressure cycles are presented in Chapter 5. Analysis carried out from the Cyclic Triaxial testing on Gault Clay fill is presented. A series of undrained monotonic tests on 100 mm undisturbed and reconstituted Lias Clay samples are presented. A series of tests on an undisturbed Lias Clay sample by means of cycling the pore pressure are presented in later part of the chapter.
6. Discussions

6.1 Introduction to discussion

An extensive review of the literature has been presented in the area of potential fatigue failure of soil due to cycles of effective stress change and some increase in near surface permeability long after the construction of cutting of slopes made of clay.

Fluctuations in pore water pressure occur in slopes in response to seasonal variations in climate (Smethurst et al., 2006). The magnitude of the annual seasonal pore water pressure change is influenced by the permeability of the slope (Nayambayo et al., 2004). Seasonal pore water pressure changes lead to corresponding cyclic changes in effective stress. Over a number of years these cycles of stress could lead to accumulated down slope movements and ultimately to the failure of the slopes (Take and Bolton, 2011). However in contrast to mechanisms which suggest failure can only occur at the end of an extreme wet season when the factor of safety will be at a minimum, slopes often fail under pore pressures that are not extreme events (Picarelli et al., 2001). This suggests the possibility that cyclic seasonal stress variations weaken the strength of the soil.

Fatigue may occur in any soil material irrespective of its strain softening nature. As most soils are micro structured with frictional bonds between particles, aggregates, or lumps, cycling loading may lead to progressive damage of these bonds. However there has been very little work carried out in the past regarding fatigue of soil materials resulting from cyclic effective stress changes associated with seasonal changes in pore water pressure.

The current study has made a contribution to the improved understanding of the influence of cyclic seasonal climate on the permeability and the strength of the soil in clay slopes. Winter and summer field permeability measurements at an instrumented cut slope have been used to show the effect of cracking on the depth and extent of seasonal pore water pressure variations. A triaxial testing programme has been carried out to show the effects of the cyclic seasonal stress variations on the strength and accumulation of strain in clay materials under different stress cycles.
6.2 The effects of cracking on near surface soil structure and permeability

The statement by Schumacher (1864) that “the permeability of a soil during infiltration is mainly controlled by big pores, in which the water is not held under the influence of capillary forces” seems to be very much valid. Different types of macro pores in clay soil have been observed in the present study too. Cracks and fissures, pores formed by plant roots and by soil fauna such as earth worms were all present in the current study site.

Cracks result from volumetric shrinkage of clays due to pore water tension. In the field, a number of groups of inter connected cracks were observed at the surface level (within the top soil). The intact London Clay was heavily fragmented into clay blocks divided by very fine cracks in the order of 0.1 mm. Densely packed grass roots follow the path of the cracks from the topsoil/intact clay interface. This will accelerate the cracking process as the roots tend to absorb more water as they grow in the summer.

The depth and width of dessication cracks varies depending on the type of soil and the climatic conditions. The measured dimensions of visible cracks has been reported by other researchers as ranging between 200-750mm depth and 5-75mm width in motorway embankments (Anderson et al., 1982, Al-shaikh-ali, 1978), and as ranging up to 2 m depth and 5-25 mm width in flood embankments (Dyer et al., 2009).

The cracking process, from the initiation of a crack onwards, can be divided in to four stages based on general lab scale testing (Vallejo, 2009). 1st generation cracks and 2nd generation cracks are visible in the field observations made in Newbury reported above in section 4.2 (Figure 4:8). When cracking is severe, it is possible for cracks to be connected and form a network. This network could channel the water laterally.

Opening and closing of cracks occurs throughout the year with seasonal variations. However, cracks are not sealed fully and provide a preferential pathway for flow of water even at the end of winter (Li, 2009, Anderson et al., 1982). Irreversible fabric changes are caused in clay by shrinkage during the cracking. Cracks persist as micro-discontinuities even after the end of winter (Anderson et al., 1982). According to previous researchers in the lab scale, cracks appear at the same locations in the 2nd and
3rd drying cycles (Yesiller et al., 2000) when altered between wetting and drying. However further investigations are require to establish this in the field.

When the vertical permeability of a cut slope is concerned, the ring infiltrometer is the best option as it is capable of measuring the vertical permeability. An advantage of the double ring infiltrometer is that the lateral spreading of water from the inner ring is minimised by infiltration from the outer ring. The near surface vertical permeability measured in the field during the middle and end of winter 2011/2012 were in the order of $10^{-8}$ m/s. Vertical permeability measured in the triaxial apparatus on intact samples taken from 0.5 m to 3.0 m depth immediately after the construction of the cut slope was in the order of $10^{-10}$ m/s reported by Smethurst et al. (2006). Hence, there is an increase in the saturated permeability near the surface by an order of two, as seen in the end of winter permeability measurement. This is in line with reports by previous researchers who reported that permeability is increased by several orders of magnitude even after closure of cracks (i.e. at the end of wetting cycle) compared with un-cracked clay (Rayhani et al., 2007, Anderson et al., 1982). Thus, shrinkage cracks influence the permeability of slope surfaces throughout the year and cause a faster than expected response of the pore water pressure to seasonal rewetting and storms. This could lead to elevated water levels within the embankments and larger seasonal cycles.

The near surface vertical permeability measured in the field at Newbury at the end of summer 2012 is in the order of $10^{-6}$ m/s. This is two orders of magnitude higher than that of end of winter permeability. This can be attributed to the cracks opening over the summer. In the context of agricultural science, it was reported that the flow of water through a crack network both under experimental and natural rain flows were in a good agreement with equations derived from Darcy’s law (Inoue, 1993). Hydraulic conductivities of the agricultural field with cracks were estimated to be of the order of $10^{-4}$ m/s in spite of the very low permeability of the soil matrix of the field ($10^{-8}$ to $10^{-9}$ m/s).

Snow (1969) showed that the flow in a smooth planar crack can be described by the cubic law. Water flow through a saturated planar crack can be considered to be laminar. A law of flow under saturated conditions for a crack with an aperture $b$ was derived considering two parallel plates of width $l$. 

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Two case studies have been presented in section 4.4.1 to evaluate the influence of cracks on the increase in permeability. Both of these studies show that an extremely small cracked area is needed to induce a significantly higher permeability. The calculations are able to link together the observed crack frequency / dimensions with the observed changes in permeability.

The channels and pores created by earthworms and moles are tubular. The influence of an individual tube of 2 mm diameter on the overall permeability of the top layer is quite large as it gives a resultant bulk permeability that is five orders higher than the permeability of the soil. Hence, earthworm holes at even a very low frequency will have a large impact on the hydrological response.

Numerical studies investigating the effect of crack on infiltration and evaporation by means of wetting and drying respectively have been presented. During drying stage, the model with the crack generate suctions to a deeper level compared to the model without the crack. Near the surface level (at 11 m height) suction generated at the crack is about 10 times higher than the suction generated at the same level in the model without a crack. Suction assigned is reduced to a greater depth in the model with crack than the model without crack for the wetting stage. For the model with the crack, Suction is reduced in the order of tens for the 0.5 m depth near the surface, whereas at 2 m depth it is still at -400 kPa level. As a summary it can be said that the presence of crack increases the depth of both drying and wetting thereby having a translating effect of the boundary conditions.

6.3 **Cyclic seasonal effects on the strength of the clay material**

London Clay is predominantly overconsolidated because of its history of loading. In slopes, shear stresses first locally reach the peak shear strength of the material, typically at the toe of the slope. This leads to a localised shear failure that can progress into the slope in long term, ultimately leading to a total collapse of the slope. The pore water pressures observed in a grass covered cut highway slope indicated seasonal variations resulting from precipitation and evapo-transpiration (Smethurst et al., 2006). This will translate to corresponding effective cyclic stresses. From a geo-mechanics point of
view, the concept of fatigue of the soil material as a result of the cyclic effective stress induced on the soil element could give a better explanation for the loss of strength and hence failure.

The work in this thesis has investigated this cyclic phenomenon experimentally. The first hypothesis of the present study has been to see what strains may be accumulated by repeated cycling of pore pressure of the sample at stress levels below critical and peak strength of the clay material. The first stage of the present study used data from tests on Gault Clay samples. Table 6:1 and Table 6:2 show the summary of stress and strain cycles resulted from the experimental programme.

<table>
<thead>
<tr>
<th>Sample Name</th>
<th>Deviator Stress</th>
<th>Cycle PP Pressure Range (kPa)</th>
<th>Cell Pressure (kPa)</th>
<th>Effective Stress range (kPa)</th>
<th>Minimum P'</th>
<th>Maximum q/P'</th>
<th>Maximum q/p' at monotonic shear failure</th>
<th>Percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>30</td>
<td>205 to 285</td>
<td>320</td>
<td>115 to 35</td>
<td>45</td>
<td>0.67</td>
<td>1.3</td>
<td>51</td>
</tr>
<tr>
<td>B</td>
<td>30</td>
<td>220 to 280</td>
<td>320</td>
<td>100 to 40</td>
<td>50</td>
<td>0.6</td>
<td>1.3</td>
<td>46</td>
</tr>
<tr>
<td>C</td>
<td>30</td>
<td>200 to 260</td>
<td>300</td>
<td>100 to 40</td>
<td>50</td>
<td>0.6</td>
<td>1.3</td>
<td>46</td>
</tr>
</tbody>
</table>

Table 6:1: Summary of stress conditions and strength mobilised during the cycling stage of Gault Clay samples

<table>
<thead>
<tr>
<th>Sample</th>
<th>Strain accumulated after 10 cycles (%)</th>
<th>Strain accumulated after 15 cycles (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.11</td>
<td>0.14</td>
</tr>
<tr>
<td>B</td>
<td>0.28</td>
<td>0.3</td>
</tr>
<tr>
<td>C</td>
<td>0.15</td>
<td>0.175</td>
</tr>
</tbody>
</table>

Table 6:2: Strain accumulate after 10 and 15 cycles

The tests carried out on Gault Clay (by Alex Dykes) were cycled at approximately 51% and 46% of the peak strength, and the results in Table 6:2 show that the samples accumulated little strain during this process.
Gault Clay samples were sheared at the end of the cycling stage. The results of the shearing stage for samples showed that a $q/p'$ ratio of about 1.30 is achieved at the peak state for samples. This corresponds to an effective friction angle of $33.0^\circ$.

The second part of the cyclic triaxial programme aimed to see whether repeated cycles may cause failure of the sample at a strength lower than that in monotonic loading and in particular, whether it may be possible to cycle over the Critical State Line (CSL), but below peak, and induce failure.

To establish the strength of Lias Clay material used in the cyclic testing, undrained monotonic tests were carried out on insitu (embankment fill) and remoulded samples of 100 mm diameter and 200 mm height. For undisturbed samples, it is evident from the results that the undrainded shear strength of the clay given by half the deviator stress varies between 18.75 kPa to 52.5 kPa. For reconstituted samples, the undrained shear strength of the clay is consistent between 22.5 kPa and 25 kPa.

The stress conditions under which the Lias Clay cyclic sample failed ($\phi'_\text{mob}=34^\circ$) is in between high and low stress conditions at failure (i.e. peak friction angles observed between $28^\circ$ and $37^\circ$) during the monotonic tests on the fill material samples of Lias Clay.

The higher than average strength of the Lias Clay cyclic sample obtained was due to the influence of stone particles and stiff roots found in the failure plane.

Monotonic tests were carried out at 50 kPa, but cyclic tests closer to failure stages were carried out on much lower effective stresses (5 kPa, 10 kPa). At lower effective stresses peak failure envelop of clay tends to curve and is above critical state line. This may be another reason for higher than average stress conditions for being sustained by the sample.
7. Conclusions and recommendations

7.1 Effects on near surface soil structure and permeability

At the Newbury site, a number of groups of interconnected cracks we observed at the surface level (within the top soil). The Intact London Clay below was heavily fragmented into clay blocks divided by very fine cracks in the order of 0.1 mm. Densely packed grass roots followed the path of cracks from the topsoil/intact clay interface. This will accelerate the cracking process as the roots tend to abstract more and more water in the summer.

The near surface vertical permeability of the surface zone of the cut slope at Newbury ($10^{-8}$ m/s - $10^{-6}$ m/s) was found to be least two orders of magnitude higher than that of intact London Clay ($10^{-10}$ m/s). The end of summer vertical permeability ($\sim 10^{-6}$ m/s) was two orders of magnitude higher than that of end of winter vertical permeability ($\sim 10^{-8}$ m/s). The near surface vertical permeability varies between summer and winter mainly due to the opening and closing of the observed crack network.

Numerical studies investigating the effect of crack on infiltration and evaporation by means of wetting and drying respectively shows that the presence of crack increases the depth of both drying and wetting thereby having a translating effect of the boundary conditions.

7.2 Effects on the strength of the clay material

7.2.1 Monotonic Tests

Monotonic tests were carried out to characterise the shear strength of the Lias Clay fill later used in cyclic tests. The undrained shear strength of the undisturbed Lias Clay varied between 37.5 kPa to 105 kPa. The corresponding peak friction angle varied from $28^0$ to $38^0$. 

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The undrained shear strength of the reconstituted Lias Clay was consistently between 42.5 kPa to 47.5 kPa. The critical friction angle of the reconstituted Lias Clay varied between $24^0$ and $27^0$.

7.2.2 Analysis of cyclic tests on Gault Clay

Permanent axial strains were accumulated with cycles of pore water pressure. The rate of accumulation of strain was found to decrease with the number of cycles.

In the Tests carried out, pore pressures were cycled too far away from the Critical State Line (CSL). Use of $23^0$ as an initial guess for the critical friction angle was too low – fill samples were monotonically failed to obtain higher friction angles than this initial guess. The tests therefore indicated strain behaviours at low levels of $q/p'$, but showed no signs of cycling to failure.

7.2.3 Cyclic Tests on Lias Clay

Permanent axial strains were accumulated with cycles of pore water pressure. The rate of accumulated strain decreased with the number of cycles when the effective stress ratio $q/p'$ was below that corresponding to critical state line.

The stress conditions under which sample failed ($\phi_{mob}' = 34^0$) was within the range of strength obtained for monotonic tests on the undisturbed fill material (which was between $28^0$ and $37^0$).

Stone particles and stiff roots found in the sample may explain the higher than average strength obtained.

Monotonic tests were carried out at effective stress of 50 kPa, which is higher than that in cyclic tests. During the cyclic test, the sample eventually failed at a friction angle that was within the range of values obtained for the monotonic tests. Strength exhibited by the cyclic sample is towards the upper end of the range of values obtained from monotonic tests on undisturbed samples. Cycling the effective stress may have lead to compaction and this may have helped the sample to exhibit a strength toward the upper end of the range of values obtained.
7.2.4 Overall Conclusions on triaxial tests

Cycling of the Gault and Lias fill some way below the CSL seems to simply cause compaction of the sample to modest axial strains (0.2% to 0.3%), with diminishing increase in strains with time so that after the certain number of cycles the sample appears to almost reach a steady state.

In the Lias test which was cycled eventually closer to CSL, there was still no (or not much) apparent increase in axial strain.

The Lias cyclic test sample failed at pretty much same level as the monontonic samples. The variability of the fill samples made exact interpretation slighty tricky. But generally it is possible to conclude that cycling of the pore water pressures is not causing failure at a lower stress level that would be obtained monotonically.

Noteably there was no peak strength exhibited for monotonic Lias and only a little peak strength exhibited for monotonic failure in the Gault; so it was not really possible to cut off the peak strength through cycling in these tests. Fill material loses it overconsolidated behaviour during placement; hence the matrix rather than cold may dominate in strength terms.

7.3 Recommendations for future work

Further crack surveys are recommended through out the year to observe the opening and closing of cracks in the summer and winter respectively. Photogrametry techniques can also be utilised to survey the cracks. Undisturbed soil samples retrieved from field could also be carefully sealed and taken to lab for CT scanning to observe the occurrence of cracks. This will give further insight into crack initiation and interaction of cracks and plant roots. Moisture movements occuring within the soil during drying and wetting process can also be studied using the above technique.

The modified double ring infiltrometer can be utilised to measure the permeability of soil (especially clay) in a larger number of embankments and cuttings and at different
locations and at depths on the slope. This will give further spatial permeability data which will be of great use in numerical analyses of climate driven slope failure.

Further field measurements of pore water suctions near the top surface of slope are needed using sensors that can measure suctions in the orders of thousands of kPa. Current field tensiometers are limited in this ability and only measure the suctions in the range of couple of hundreds of kPa. Measurement techniques in this area need further research and development so as to invent novel instruments to be used in field measurements. These measurements will help understand the links between dessication and cracking.

Further numerical simulations using the Seep/W software are needed to study the influence of change in depth and spacing of cracks in order to understand the effect of cracking of cycles of pore water pressure. This study can be further extended to take into account the climate change scenarios and the effect of extreme weather patterns predicted for UK, on the changes in water pressures and consequently on the stability of slopes. This numerical experiment can be combined with other software such as Vadose/W to model the influence of trees in the drying and wetting of the slope.

Fatigue of soil by the cycles of effective stress changes brought about by changes is pore water pressure should be investigated further. If someone else wanted to continue the work utilising triaxial apparatus, it is recommended that:

a) Reconstituted samples are used, for greater consistency in behaviour.

b) The reconstituted Lias Clay is overconsolidated in the first stage of test so it develops a clear peak strength, as the cycling may influence the peak strength of the sample. Hence cycles above but close to the “critical state”, which are below peak need to be applied to see if it will cause failure at less than peak strength.
8. Bibliography


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