Weather-driven clay cut slope behaviour in a changing climate

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Weather-Driven Clay Cut Slope Behaviour in a Changing Climate

by

Harry Postill

Doctoral Thesis

Submitted in partial fulfilment of the requirements for the award of Doctor of Philosophy of Loughborough University

(July 2018)

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Supervisors: Prof. Neil Dixon and Dr. Ashraf El-Hamalawi

(Dr. Gary Fowmes – University of Warwick)

Geotechnical Engineering
School of Architecture, Building and Civil Engineering
Abstract

Long linear earthwork assets constructed in high-plasticity overconsolidated clay are known to be deteriorating due to long-term effects of wetting and drying stress cycles as a result of seasonal weather patterns. These stress cycles can lead to shallow first-time failures due to the mobilisation of post-peak strength and progressive failure. Design requirements of new earthworks and management of existing assets requires improved understanding of this critical mechanism; seasonal ratcheting.

Incremental model development and validation to allow investigation of multiple inter-related strength deterioration mechanisms of cut slope behaviour in high-plasticity overconsolidated clay slopes has been presented. Initially, the mechanism of seasonal ratcheting has been considered independently and a numerical modelling approach considering unsaturated behaviour has been validated against physical modelling data. Using the validated model, the effects of slope geometry, design parameter selection and design life have been considered. Following this, an approach to allow undrained unloading of soil, stress relief, excess pore water pressure dissipation, seasonal ratcheting and progressive failure with wetting and drying boundary conditions has been considered. Hydrogeological property deterioration and the potential implications of climate change have been explored using the model. In both cases the serviceable life of cut slopes is shown to reduce significantly in the numerical analyses. Finally, a model capable of capturing hydrogeological behaviour of a real cut slope in London Clay has been developed and validated against long-term field monitored data. Using the validated model, a climate change impact assessment for the case study slope has been performed.

The numerical analyses performed have indicated that seasonal ratcheting can explain shallow first-time failures in high-plasticity overconsolidated clay slopes and that the rate of deterioration of such assets will accelerate if current climate change projections are representative of future weather.

Keywords

Numerical Modelling; Seasonal Ratcheting; Progressive Failure; Cut Slope; Clay; Cyclic Wetting and Drying; Climate Change.
Firstly, I would like to acknowledge the help and support that I have received from my supervisors, Professor Neil Dixon, Dr. Ashraf, El-Hamalawi and Dr. Gary Fowmes throughout the course of my PhD studies and for making the experience so enjoyable. I would also like to thank faculty members across the School of Architecture, Building and Civil Engineering at Loughborough University for taking the time to help and discuss ideas, with particular mention to Dr. Tom Dijkstra, Dr. Alister Smith and Lewis Darwin.

The work conducted within this thesis would not have been possible without the help and support of academics and industrial partners across the iSMART project (EPSRC project EP/K027050/1) and through data provided by Professor Andy Take and Dr. Joel Smethurst to whom I am extremely grateful.

I would also like to thank my friends and family for being there when I needed them and for listening to me when I talked about slope stability for the hundredth time! In particular, I owe a significant debt of gratitude to the unwavering support and love that I have received from my beautiful fiancée Louise Flanagan, this thesis is dedicated to you.
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Chapter 1

Introduction

1.1 Introduction

The UK’s infrastructure network experiences some of the highest traffic volumes in the world. It is estimated that 20,000km of road, rail and canal are constructed either on embankments or in cuttings (Loveridge, et al., 2010). Approximately £20 billion of major highway infrastructure assets are slopes; this is one third of total asset value (Perry, et al., 2001). A nation’s economic future is highly dependent on the reliability, safety and resilience of its infrastructure network (Glendinning, et al., 2009a).

Currently, the nation’s infrastructure network is facing greater demand than ever, and expectations are changing. Greater traffic volumes at higher speeds with minimal delays, coupled with reduced environmental impact to create a more sustainable network is required under economic constraint (Glendinning, et al., 2009a).

Infrastructure slopes are used to achieve acceptable vertical alignment along transportation routes. Within the UK infrastructure slopes are an ageing asset, the majority of railway and highway infrastructure slopes were constructed more than 100 years ago and 60 years ago respectively (Glendinning, et al., 2015). In general, little is known about the construction, geological formation and current condition of infrastructure slopes (Glendinning, et al., 2009a).

Infrastructure earthwork behaviour across the UK has been described as an ‘asset time bomb’ scenario by Glendinning, et al. (2015). This is where a considerable percentage of an asset base nears the end of its serviceable life...
within a short timeframe (Thurlby, 2013). Unlike other sectors adopting the ‘asset time bomb’ ideology, earthworks cannot be replaced, and alternative transportation routes are often not economically viable, making maintenance strategies critical to long-term performance (Glendinning, et al., 2015).

Figure 1.1 shows hypothetical deterioration of earthwork performance with age. Problems arise when slopes are in the degenerating and unreliable phases. As deterioration progresses, the cost of maintenance increases dramatically (Ridley, et al., 2004).

![Figure 1.1 Deterioration model for infrastructure earthworks (after, Glendinning, et al., 2015)](image)

For effective asset management, owners must understand which sections of the network are most at risk of failure and the potential implications if a failure occurred. Incidents, such as slope failures, can cause complete closure of a transport corridor, so it is crucial to determine ‘weak links’ or critical sections within the network (Dijkstra, et al., 2014). It has been reported that unplanned remediation of earthworks can cost ten times that of routine maintenance, when all costs, works, delays, compensation etc. are included (Glendinning, et al., 2009a).

It is estimated that 7% of major infrastructure routes (railway lines, motorways and A-roads) are located within regions of moderate to significant land instability (Dijkstra & Dixon, 2010). In the winter of 2000-2001 there were approximately 160 slope failures recorded across the highways and railway network (Turner, 2001); climatic extremes have led to an increase in the
frequency of both natural and engineered slope failures (Glendinning, et al., 2015). Between 2012 and 2014, numerous failures across the railway network were recorded, a total of 186 and 156 failures for 2012/13 and 2013/14 respectively (ARUP, 2014). From forensic investigation of these slope failures, it was found that twice the number of soil cut slope failures occurred than embankment failures (ARUP, 2014). It was also established that the most common failure type for soil cut slopes was translational in nature.

Two types of infrastructure / engineered slopes exist;

- cut slopes – excavations within existing ground, used to lower existing ground to an acceptable level;
- embankments – constructed on top of existing ground, used to raise existing ground to an acceptable level.

These two earthworks are fundamentally different; embankments are constructed and compacted in a specified manner, whereas cut slopes are natural ground formations that have been unconfined. This means that embankments are constructed of, theoretically, known relatively homogeneous soils, although this is not entirely true and large variability exists. Cut slopes however, have natural variability due to geological formation and problems associated with stress history and in-situ structure.

Understanding the infrastructure slopes at risk of failure to allow formulation of robust asset management strategies can only be achieved with a better understanding of the mechanisms and processes leading to failure. In particular, shallow failures (i.e. failures less than 2m deep) in high-plasticity overconsolidated clay slopes (i.e. soils with large swelling potential) driven by repeated seasonal stress cycles causing accumulation of plastic strains and progressive failure need to be better understood. This mechanism of failure, known as seasonal ratcheting, has been shown conclusively through centrifuge experimentation presented by Take and Bolton (2011) and has been observed in aging infrastructure slopes (Briggs, et al., 2017). The aims of this research project are structured around enhancing understanding of these shallow failures driven by seasonal wetting and drying stress cycles.
1.2 Aim

1. Establish a numerical model approach for the assessment of high-plasticity overconsolidated clay infrastructure cut slopes under current and projected climatic conditions, incorporating inter-related deterioration mechanisms and provide design recommendations for shallow first-time failures.

1.3 Objectives

To achieve the aim of this project the following objectives were considered;

1. Evaluate, understand and review time-dependent construction and long-term weather-driven mechanisms attributed to the strength deterioration of high-plasticity overconsolidated clay cut slopes.

2. Develop a model for the analyses and investigation of weather-driven strength deterioration mechanisms for high-plasticity overconsolidated clay slopes and provide design recommendations for this failure mechanism.

3. Develop a model for the analyses of inter-related time-dependent strength deterioration mechanisms, including weather-driven strength deterioration, for high-plasticity overconsolidated clay cut slopes; furthering the understanding of long-term cut slope behaviour and significance of particular material parameters.

4. Enhance understanding of the implications of climate change on high-plasticity overconsolidated clay cut slopes.

1.4 Outline Methodology

The following section presents a brief overview of the methodology developed to achieve the aims and objectives presented in Sections 1.2 and 1.3. Model development has been considered within different phases to allow incremental development and validation of the modelling approaches. The phases of modelling have been clearly aligned with the Objectives presented in 1.3 and the flow of the research conducted is shown along with the Chapters in which modelling methodology and results are presented within a research map (see Figure 1.2). The phases of modelling are described in detail within Section 3.2
and have been summarised briefly below along with how each Objective is addressed:

- **Objective 1** – to develop understanding and evaluate different deterioration mechanisms associated with long-term cut slope failure a literature review considering mechanisms, process and current state-of-the-art numerical modelling has been undertaken and is presented in Chapter 2.

- **Phase 1** – using a saturated numerical framework, construction effects due to excavation in low saturated hydraulic conductivity soils has been modelled. This has been done to capture undrained unloading, stress relief, pore water pressure dissipation and progressive failure under constant boundary conditions. Within this work, localisation and mesh dependency have been investigated considering local and nonlocal strain-softening models. The methodology and results of this modelling are presented in Chapter 4. Results of modelling have been considered against previous modelling of these mechanisms presented by Potts, *et al.*, (1997).

- **Objective 2 / Phase 2** – a model capable of modelling seasonal ratcheting due to wetting and drying (i.e. weather-driven) stress cycles and progressive failure has been developed in a two-phase flow numerical modelling framework. This allows critical unsaturated behaviour to be modelled. Modelling results have been validated against physical modelling data of these mechanism presented by Take and Bolton (2011). Methodology and results of the validation of the numerical model undertaken and results of a geometric study considering the effects of strain-softening behaviour, slope angle and height on the failure mechanism are presented in Chapter 5.

- **Objective 3 / Phase 3 & 4** – a method for the inclusion of critical strength deterioration mechanisms presented within Figure 1.2 has been developed to allow modelling of long-term behaviour of cut slopes. Critical parameters have been considered to understand behaviour and boundary conditions to consider real slope behaviour and further soil-water-atmosphere interactions have been developed. Boundary
conditions and corresponding near-surface stress cycles have been validated against monitored data of a real cut slope presented by Smethurst, et al., (2012). The methodology and results of this work are covered within Chapter 6 and 7.

- **Objective 4 / Phase 4** – this work considers the implications of climate change on clay cut slope behaviour. Synthetic climate change boundary conditions have been created using statistical down-scaling methodology and have allowed stress testing of a case study slope to consider climate change effects. The methodology and results of this work are presented in Chapter 7.

**1.5 Thesis Structure**

This thesis has been presented in nine chapters.

**Chapter 2** considers current literature and state of knowledge of long-term behaviour of high-plasticity overconsolidated clay cut slopes and state-of-the-art modelling approaches.

**Chapter 3** describes an overview of the development of numerical modelling approaches along with generic methodology used throughout this work.

**Chapter 4** presents numerical analyses methodology and results of simplified cut slope behaviour within a saturated framework compared with previous
modelling in this area, along with the implementation of a nonlocal strain-
softening model.

Chapter 5 details methodology and results of numerical analyses of weather-
driven behaviour conducted within an unsaturated framework, validated
against centrifuge experimentation. Using the validated model, the effects of
slope geometry, design parameter selection and design life of slopes
subjected to seasonal ratcheting have been investigated.

Chapter 6 builds on the work presented in Chapter 4 and 5, by developing a
methodology to incorporate multiple inter-related mechanisms into a single
boundary-value problem. Mechanisms include, undrained unloading, stress
relief, excess pore water pressure dissipation, weather-driven behaviour and
progressive failure. The effect of hydrogeological property deterioration,
potential implication of climate change and effect of initial stress conditions
have been investigated.

Chapter 7 presents results of numerical analysis of a real cut slope in London
Clay. Hydrogeological behaviour and boundary conditions developed have
been validated against six years of monitored data for seventeen locations
across the slope. Following validation of hydrogeological behaviour, a climate
change assessment using a statistical down-scaling model to create synthetic
weather data is presented for the case study slope.

Chapter 8 brings together the discussions of all the analyses conducted in one
place considering limitations and uncertainties to ascertain the significance of
the results. The discussions focus on the numerical approaches used, key
results obtained from the work conducted and have been considered in the
wider context of clay cut slope deterioration and the potential implications of
climate change on slope behaviour.

Chapter 9 summarises the conclusions drawn throughout the work and
highlights areas for further work.
Chapter 2

Literature Review

2.1 Chapter Scope
This Chapter presents the findings of a literature review conducted in the context of the aims and objectives presented in Chapter 1. This literature review begins with a summary of the mechanisms attributed to first-time failures within clay cut slopes along with a summary of the key interactions between soil and the atmosphere. Following this, the approaches available for the numerical representation of soil behaviour have been discussed and key governing equations are presented. Finally, the current state-of-the art numerical modelling approaches used to consider the problem have been critiqued, methods for the assessment of climate change considered and gaps in knowledge presented.

2.2 First-Time Failure of Cut Slopes
In line with Objective 1, this section considers long-term strength deterioration mechanisms for first-time failures of cut slopes in high-plasticity overconsolidated clay.

Key mechanisms and processes attributed to strength deterioration of high-plasticity overconsolidated clay cut slopes are as follows;

1. Progressive failure;
2. Post-excavation pore water pressure equilibration;
3. Stress relief;
4. Shrink-swell (seasonal ratcheting);
5. Weathering;

2.2.1 Progressive Failure

The principal of progressive failure has been explained by many researchers, for example Skempton (1964), Vaughan (1994), Potts, et al., (1997), Petley, et al., (2005), Leroueil (2001). The mechanism is simple in principal; at a point in a soil mass stresses are such that the peak shear strength is exceeded and a soil element ‘fails’ causing the strength of that particular element to reduce and excess stresses are redistributed within the soil. The element of soil with reduced strength acts as a stress concentrator and if the stresses applied do not reduce, the strength of the soil element will continue to reduce, and other elements of soil will also begin to exceed their peak strength, thus the failure of successive soil elements progresses, and a shear surface develops.

As failure progresses, the overall strength available for mobilisation reduces, in some cases the strength across the shear surface will reduce from peak to residual at high strains (Skempton, 1964). In others, equilibrium will be reached and the shear strength available will be reduced but complete failure may not occur (Leroueil, et al., 2012). Progressive failure can explain both deep-seated and shallow failures in all types of slopes (Vaughan, et al., 2004).

To understand progressive failure, strain-softening behaviour should be considered. Physical behaviour of soil under loading can be shown by a typical stress-strain curve shown in Figure 2.1, the curve illustrates elastic and plastic behaviour of overconsolidated clay.

![Figure 2.1 Typical overconsolidated clay stress-strain curve](image)

Initially, under small strains soil behaves elastically, as stresses increase strains also increase. If the yield criterion is exceeded, permanent
irrecoverable plastic deformations will occur. If stresses within an element of soil reach a level where the material’s peak strength is exceeded then that soil element effectively fails, and its strength quickly reduces with significant straining happening at stresses well below peak strength, as shown in Figure 2.1. This is known as strain-softening behaviour.

The reduction in post-peak strength for overconsolidated clays can be attributed to two physical processes related to shearing (Skempton, 1964; Lupini, et al., 1981):

1. Increased soil water content due to dilation during shearing; this is described as turbulent shearing;
2. Reorientation of ‘plate-like’ clay particles parallel to the direction of shearing, described as sliding shearing.

### 2.2.2 Post-Excavation Pore Water Pressure Equilibration

It is well-established that the pore water pressure regime in an overconsolidated clay cut slope changes with time. Stress changes due to excavation cause undrained unloading and negative pore water pressures that equilibrate to steady state seepage pore water pressures in the long-term (Vaughan & Walbancke, 1973; Chandler & Skempton, 1974). The rate at which pore water pressures equilibrate can be considered to obey Terzaghi’s theory of consolidation, making the process a function of the soil’s saturated hydraulic conductivity and external boundary conditions (Vaughan & Walbancke, 1973). Pore water pressure equilibration is noted as the principal mechanism attributed to delayed failure of cut slopes in overconsolidated clays. It is suggested that this failure mechanism should be considered before any other (Vaughan & Walbancke, 1973).
Figure 2.2 shows the variation of pore water pressures due to undrained unloading with time for a cut slope. Negative pore water pressure suctions are beneficial to slope stability and in the short-term increase the stability. With time, excess pore water pressure suctions dissipate and conditions move towards steady state, effective stress reduces and thus the shear strength decreases and the overall stability of the system reduces.

The loss of excavation-induced suctions with time due to pore water pressure equilibration is accompanied by an additional swelling process, as suctions dissipate the soil water content increases and the soil swells (Vaughan & Walbancke, 1973; Dixon & Bromhead, 2002). This changes the hydraulic conductivity of the soil as the void ratio increases and as the soil water content increases, the soil strength decreases.
2.2.3 Stress Relief
Coupled with pore water pressure equilibration post-excavation, internal confined stresses exist within soils that are a function of the soil's stress history. Unconfinement of these internal stresses causes swelling. Horizontal stress change due to excavation in an overconsolidated material is far greater than the vertical stress change. Therefore, horizontal displacements due to unloading are also greater (Burland, et al., 1977). It is given that;

\[
\sigma_H' = K_0 \cdot \sigma_V'
\]

Where:
- \(\sigma_H'\) = horizontal effective stress;
- \(\sigma_V'\) = vertical effective stress; and
- \(K_0\) = coefficient of earth pressure at rest (function of overconsolidation ratio and stress history).

The displacements which occur due to stress relief can cause strain-softening leading to progressive failure (see Section 2.2.1) post-excavation. Stress relief is a much more significant mechanism when structural discontinuities exist within the cut slope. Structural discontinuities act as stress concentrators, displacements from unconfinement only need to be relatively small to cause significant strength reduction along discontinuities (Skempton & Petley, 1967; Burland, et al., 1977; Dixon & Bromhead, 2002). Stress relief is a dominant process causing strain accumulation and progressive failure in overconsolidated clay cuttings (Potts, et al., 1997).

Due to the in-situ structure of stiff fissured clays, swelling due to stress relief will open fissures and joints. This will cause a change in both the hydraulic conductivity and shear strength in the near-surface of the newly exposed material (Terzaghi, 1950; Vaughan & Walbancke, 1973). However, the increase in the saturated hydraulic conductivity post-excavation can be considered negligible as the time taken for excess pore water pressures to dissipate is so long. With time, there will be an increase in the near-surface saturated hydraulic conductivity due to stress relief and dissipation of excess pore water pressures but also due to weathering (see Section 2.2.5).
Using numerical models, it has been shown that the greater the level of overconsolidation, the further into a cut slope a horizontal softened zone produced by stress relief and strain-softening will propagate (Potts, et al., 1997), the results from numerical analyses showing this are presented in Figure 2.3. Development of this basal shear surface and a principal rupture surface due to stress relief, pore pressure equilibration and strain-softening has also been observed during the Selborne cutting experiment (Cooper, et al., 1998). The Selborne cutting experiment was a monitored experimental failure of a 9m deep cut slope constructed within Gault Clay driven to failure through pore water pressure recharge (Cooper, et al., 1998). Within the Selborne cutting experiment, it is likely that structural discontinuities and bedding features played a role in the basal softened zone and that the difference in the numerical model results presented by Potts, et al., (1997) is an artefact of the numerical modelling approach due to localisation problems and mesh dependency when implementing a local strain-softening model (Potts, et al., 2000).

Regardless, the softened zone presented numerically by Potts, et al. (1997) and observed experimentally by Cooper, et al. (1998), shows the significance of the strain energy released during excavation and the effects it has on the
overall condition of a cut slope. The softened zone resulting from stress relief will most likely be involved in any further earth movements later in the life of a cut slope (Cooper, et al., 1998).

2.2.4 Shrink-Swell Behaviour

High-plasticity fine grained soils experience volumetric change due to soil water content variation. Plasticity is a measure of the volume of water taken for a soil to change from a plastic state to a liquid state. The volume of water change required is dependent on the soil particle size and mineralogy. The greater the plasticity of a soil, the greater the shrink-swell potential of the soil. During wetting, the ground experiences heave; conversely, during drying soil water volume decreases and thus, the ground shrinks. The most noticeable shrink-swell cycles are driven by seasonal variation in soil water content. The magnitude of movement experienced due to cycles of water content depends on the materials plasticity and reduces with depth (Barnes, 2010).

Plasticity describes a soil's ability to be remoulded without breaking under constant soil water content. The plasticity of a soil is dependent on the proportion of fine grained material and its mineralogy, the soil water content governs the behaviour or state of the material along with the volume it occupies (Barnes, 2010). The volume change potential of a clay soil is dependent on the magnitude of change in soil water content achievable and thus relates to plasticity. Plasticity and soil state can be described using soil moisture content, forming consistency limits (or Atterberg limits) (Barnes, 2010);

- liquid limit, \( w_L \);
- plastic limit, \( w_p \);
- shrinkage limit, \( w_S \);
- plasticity index, \( I_P = w_L - w_p \).

The generalised relationship between soil moisture content, soil state and volume is shown in Figure 2.4.
Soils are defined as low, medium or high plasticity under the following conditions (Barnes, 2010):

<table>
<thead>
<tr>
<th>Plasticity Level</th>
<th>Liquid Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low Plasticity</td>
<td>20-35</td>
</tr>
<tr>
<td>Medium Plasticity</td>
<td>35-50</td>
</tr>
<tr>
<td>High Plasticity</td>
<td>50-70</td>
</tr>
<tr>
<td>Very High Plasticity</td>
<td>70-90</td>
</tr>
<tr>
<td>Extremely High Plasticity</td>
<td>&gt;90</td>
</tr>
</tbody>
</table>

Changes in soil water content can cause large volumetric change in high-plasticity soils. Below the plastic limit, the water available to maintain plasticity within the soil becomes insufficient and desiccation cracking can occur, affecting the hydrogeological properties of the soil. In addition, it has been suggested that cycles of shrinkage and swelling will cause diagenetic bonds within a soil to break and degradation of material strength and a change in the saturated hydraulic conductivity of the near-surface zone (Nyambayo, et al., 2004).

Field observations from the early 1990s considering embankments (ash fill over clay fill) on the London Underground Ltd. (LUL) network showed considerable differences in slope crest position between summer and winter due to shrink-swell behaviour. Horizontal movements where recorded between 5 to 10mm and vertical movements ±30mm (Kovacevic, et al., 2001). This has also been observed by Scott, et al., (2007) for London Clay embankments, where light vegetation was present, movements of 5 to 8mm were recorded and for high water demand trees, movements of 50 to 55mm seasonally were observed.
2.2.4.1 Seasonal Ratcheting
Slopes formed of high-plasticity soils subjected to wetting and drying stress cycles, and thus shrinkage and swelling, can experience a component of downslope movement with each cycle of volume change. Repeated wetting and drying stress cycles can lead to the accumulation of irrecoverable strains and progressive failure. This mechanism is known as seasonal ratcheting and has been shown through centrifuge experimentation (Take & Bolton, 2004; Take & Bolton, 2011) and has been observed through field measurements (Ridley, 2017). Due to the transient nature of the stress cycles imposed on slopes experiencing seasonal ratcheting, the failure mechanism is due to progressive failure and mobilisation of post-peak strength and not due to long-term creep under constant stress conditions.

The principal of shrink-swell due to seasonal water content variation leading to strength reduction in the near-surface region has been mentioned previously by Skempton (1964) with conclusive evidence presented by Take and Bolton (2004; 2011) through centrifuge experiments of high-plasticity clay. Take and Bolton (2004; 2011) presented the following conclusions;

1. seasonal soil water content fluctuation leads to stress cycles which can cause strength reduction and progressive failure;
2. shrinkage displacements due to drying tend to be approximately normal to the slope;
3. swelling displacements are not normal to the slope and include a component of downslope movement that increases towards the toe of the slope.

A schematic diagram of displacements on a slope surface due to wetting and drying stress cycles driving seasonal ratcheting is shown in Figure 2.5. A detailed account of the work conducted by Take and Bolton (2011) can be found in Section 5.2.1 with the results of their work are shown in Section 5.5.
Chapter 2 – Literature Review

Centrifuge experimentation was also undertaken to consider an intermediate plasticity soil under wetting and drying stress cycles (Hudacsek, et al., 2009). The volumetric change observed was far smaller than that observed by Take and Bolton (2004). Downward slope movement occurred but progressive failure due to strain accumulation was not witnessed, suggesting seasonal ratcheting is more prominent in high-plasticity soils (Hudacsek, et al., 2009).

2.2.5 Weathering

Soils experience numerous physical processes during formation creating inter-particle bonds and attractions within the material, diagenetic bonds. Weathering causes the destruction of these bonds due to mechanical and chemical processes, having a direct impact on material strength and hydraulic conductivity (Terzaghi, 1950; Chandler, 1972; Leroueil, 2001). Weathering is a non-uniform process that can cause particle size reduction, a change in mineralogy and reduction in inter-particle bond strength (Leroueil, 2001).

Two categories of weathering that can be considered (Chandler, 1972);

1. mechanical processes – the destruction of diagenetic bonds through; the variation of seasonal soil water content; frost action; and movement of soil mass;

2. chemical processes – the destruction of diagenetic bonds through; oxidisation; and leaching of carbonates.

Weathering causes an increase in soil water content as the soil matrix is broken down and the void ratio increases (Chandler, 1972). Chandler (1972)
observed a clear relationship between the level of oxidisation of a soil and the level of weathering through the consideration of ferric oxide (Fe$_2$O$_3$) and iron oxide (FeO). The study carried out by Chandler (1972) showed the influence of weathering of cut slopes is related to the original ground profile rather than the cut slope profile and reduces with depth, this was concluded for cut slope failures in Lias formation approximately 60 years after excavation.

### 2.2.6 Structural Discontinuities

Structural discontinuities have significant influence on the strength and hydraulic conductivity of a soil. Within a soil matrix, discontinuities act as stress concentrators and play a key role in progressive failure (Skempton, 1964). Skempton and Petley (1967) investigated the strength properties of different discontinuities, these are summarised in the following table (Skempton & Petley, 1967);

<table>
<thead>
<tr>
<th>Group</th>
<th>Type</th>
<th>Occurrence</th>
<th>Relative movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depositional or</td>
<td>Bedding Surfaces</td>
<td>Bedding planes, Laminations, Partings.</td>
<td>Zero</td>
</tr>
<tr>
<td>Diagenetic</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Structural</td>
<td>Joints (brittle fracture surface)</td>
<td>Systematic joints, Fissures.</td>
<td>Practically zero</td>
</tr>
<tr>
<td></td>
<td>Minor Shears (non-planar, slickensides)</td>
<td>Small-displacement-shears Riedel and thrust shears</td>
<td>Less than 1cm</td>
</tr>
<tr>
<td></td>
<td>Principal Displacement Shears</td>
<td>Principal slip surfaces in; landslides; faults; bedding planes; slips.</td>
<td>More than 10cm</td>
</tr>
<tr>
<td></td>
<td>(subplanar, polished)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2.2 shows discontinuities found within fine-grained soils. They are categorised by the processes which created them, either during formation or due to loading. Skempton and Petley (1967) concluded structural discontinuities are points of weakness which reduce the overall soil strength well below the idealised peak strength of the intact soil mass.

In addition to shear discontinuities, joints and fissures exist within stiff clays. They are described as surfaces with a ‘brittle fracture’ texture and vary in length considerably, from a couple to tens of metres (Skempton & Petley, 1967).
1967). As these features are not created through shearing, no (or very little) displacement or particle orientation will have occurred across the surface. Also, these features are effectively cracks, therefore the cohesion across the interface must have been destroyed during formation, $c' = 0$ kN/m$^2$ (Skempton & Petley, 1967).

Skempton and Petley (1967) studied joints in Blue London Clay and made the following observations (Skempton & Petley, 1967):

- the features where approximately normal to bedding planes;
- they occurred at 1-2m intervals;
- the length and profile of each feature varied considerably, on average they could be considered to be 1m long.

Tests carried out by Skempton and Petley (1967) showed shear strength along joints is greater than residual strength. However, only small displacements (less than 5mm) are required for strength to reduce to residual. These findings are based on the results of four tests (Skempton & Petley, 1967).

Burland et al. (1977) and Dixon and Bromhead (2002) presented cases where structural discontinuities played fundamental roles in slope failures, strains from unconfinement causing residual strength along bedding features leading to progressive failure. Structural discontinuities are an important aspect of clay behaviour due to stress concentration exacerbating progressive failure and the addition of preferential flow routes changing the hydraulic conductivity of soil.

### 2.2.7 Inter-Related Nature of Mechanisms

The mechanisms leading to cut slope failures presented are inter-related. Key mechanisms affect multiple facets of behaviour, changes in strength and hydrogeological behaviour occur concurrently. Understanding the complexities of these mechanisms and their inter-related nature is vital to the understanding of the complete problem.

Progressive failure and strain-softening occurs within cut slopes due to pore water pressure equilibration, stress relief, seasonal ratcheting and is affected by the presence of structural discontinuities exacerbating the problem. In
addition, weathering, both mechanical, from shrink-swell cycles, and chemical, cause strength reduction in the near-surface. Of these mechanisms, pore water pressure equilibration and seasonal ratcheting are dependent on the hydrogeological properties of the soil which are in turn affected by swelling from unconfinement, structural discontinuities acting as preferential flow routes, weathering and cycles of shrink-swell due to wetting and drying stress cycles. The boundary conditions which a slope experiences influences the rate at which post-excitation pore water pressures dissipate and the magnitude of seasonal shrink-swell behaviour, and therefore the strength and hydrogeological implications of this mechanism. Shrink-swell behaviour, and therefore seasonal ratcheting is driven by soil water content variation, it is dependent on climatic conditions, vegetation type and the material properties.

The mechanisms presented thus far and their inter-related nature, have been focused mainly on mechanical strength deterioration but clear dependency on the hydrogeological behaviour has been shown. Developing on this, the following section looks at hydrogeological behaviour with an emphasis on near-surface (i.e. within the upper 2m of the slope surface) processes within clay slopes.

### 2.3 Soil-Water-Atmosphere Interaction

Hydrogeological influences are fundamental to the failure of many slopes. A key observation from the mechanisms discussed in Section 2.2 is the inter-related nature of failure mechanisms along with the significant influence of hydrogeology on behaviour. This section considers soil-water-atmosphere interactions to further understand cut slope deterioration. Fundamental soil behaviour is driven by transient water movement, volume and strength variation, and pore water pressure fluctuations are critical to long-term slope stability.

High-plasticity soils experience shrink-swell cycles due to seasonal water cycles leading to volumetric change and can result in the accumulation of irreversible strains and progressive failure (Take & Bolton, 2011). Understanding the drivers of this mechanism is imperative for the understanding of complete cut slope behaviour.
Soil-water-atmosphere interactions are based on the external climate, the hydrological properties of the soil, slope geometry, vegetation and antecedent conditions (i.e. water availability). Primarily the response of soil to atmospheric inputs, such as rainfall, must be understood.

### 2.3.1 Slope Hydrological Cycle

The soil water content at any one time is dependent on a combination of processes. Essentially, soil water content and the resulting effect on soil properties, is based on the balance between the input and removal of water along with water movement through the soil. This is illustrated in Figure 2.6, via a simplified slope hydrological cycle.

![Figure 2.6 Simplified infrastructure slope hydrological cycle](image.png)

Movement of water within the unsaturated (or vadose) zone of a slope is fundamental to performance. Pore water pressure suctions in the near-surface play a significant role in stabilising a slope. Understanding water movement is therefore essential. Surface water from precipitation moves into the soil through infiltration and is then redistributed within the unsaturated zone. Redistribution is influenced by the inflow and outflow of water at the boundary. Water that has infiltrated into the unsaturated zone subsequently moves vertically down through percolation, eventually it will move into groundwater causing groundwater recharge. Conversely, when water is removed from soil through root uptake and evapotranspiration (both from the ground surface and vegetation), pore water pressure suctions occur leading to capillary rise from ground water into the unsaturated zone. As the ground is sloped, surface run-off and interflow should also be considered (Dingman, 2002).
2.3.2 Hydrogeological Properties

The following section addresses the material properties and processes that influence the movement of water through a soil. Understanding transient water movement, saturated and unsaturated, in slopes is critical for understanding stability.

2.3.2.1 Hydraulic Conductivity

Hydraulic conductivity describes the rate at which a fluid, normally water, can pass through a porous medium. It is governed by the soil matrix and macrostructure. The hydraulic conductivity of a material is dependent on a combination of factors that vary with time and space meaning it is non-homogeneous and anisotropic. To accurately determine a representative hydraulic conductivity for a fine-grained soil is therefore extremely difficult (Glendinning, et al., 2014).

Hydraulic conductivity differs considerably depending on the nature of the water flow and the state of the material, i.e. saturated (saturated hydraulic conductivity) or unsaturated (relative hydraulic conductivity).

Saturated hydraulic conductivity is a function of void ratio, any change in void ratio will change the hydraulic characteristics of the material. There are numerous processes which cause volume change within soils, thus altering hydraulic conductivity. Dilation causes an increase in volume, therefore, under shearing the saturated hydraulic conductivity across a rupture surface will increase. Stress relief causes soil water content change and swelling, the increased void ratio from swelling will increase the saturated hydraulic conductivity. With stress relief existing in-situ macro features can also open altering the materials saturated hydraulic conductivity (Vaughan & Walbancke, 1973). It is well-established that with depth, the void ratio of a soil decreases, and as such, the saturated hydraulic conductivity will also reduce. This has been shown through field observations by Dixon and Bromhead (1999).

For unsaturated flow, the addition of an air phase within the porous medium changes the rate at which water can move within the soil matrix due to the addition of surface tension, capillary rise and pore water pressure suctions. In
an ideal material, in which the structure does not change during desaturation, the lower the saturation (and the higher the pore water pressure suctions) the slower the rate of movement of water within the soil (i.e. the relative hydraulic conductivity will reduce). This has been shown experimentally and has had relationships fitted against the behaviour (van Genuchten, 1980).

In contrast to the relationship of relative hydraulic conductivity reducing as saturation reduces, within clays there is a point at which desiccation cracking will occur altering the void ratio and adding preferential flow route within the near-surface and the resulting relative hydraulic conductivity will in fact increase (Vaughan, 1994). Desiccation cracking is discussed further in Section 2.3.2.3.

2.3.2.2 Macropores and Preferential Flow
Macropores are large pores, or voids, that exist within soil and have significant implications on the hydraulic conductivity of the soil. Preferential flow, or bypass flow, is where fluid moves within a soil through defined, preferred, routes increasing the rate of water movement considerably (Beven & Germann, 2013).

Macropores and preferential flow routes within a soil matrix are caused by desiccation cracking, root growth and decomposition, and burrowing animals (such as worms) (Beven & Germann, 2013). The existence of such features can greatly change the behaviour of a soil in relation to water infiltration and extraction. Rapid inundation of slopes following wet periods is a known problem with many agreeing preferential flow is of major significant to slope hydrogeology and therefore stability (Beven & Germann, 2013).

2.3.2.3 Desiccation Cracking
Desiccation cracking of high-plasticity soils is related to the removal of water from the surface causing cracking due to drying. When tension forces from suctions exceed confining pressures, cracking occurs (Fredlund & Rahardjo, 1993). Thus, it is related to the vegetation and its ability to remove water from the soil, i.e. the more water removed the greater the cracking (Smethurst, et al., 2006). Increases in macro-features, such as cracks and fissures, increase
the near-surface hydraulic conductivity of fine-grained materials (Hughes, et al., 2009). This changes the material response to atmospheric inputs.

### 2.3.3 Vegetation

Historically, vegetation along infrastructure routes has been perceived as problematic; sighting issues, fallen trees closing routes, leaves on railway lines, fire hazard from steam trains, these led to vegetation management where trees where periodically removed from infrastructure slopes until the 1960s (Glendinning, et al., 2009b; Smethurst, et al., 2015). Post 1968, steam locomotives within the British railway became rare and many vegetation management programs ceased (Smethurst, et al., 2015). However, more recently the benefits of trees and their reduced hazardous implications, given advances in electric trains and signalling, means trees are a common occurrence on the UK’s infrastructure network earthworks.

Vegetation plays a significant role in the soil water content cycle a slope experiences and has an influence on the near-surface strength due to root reinforcement. The implications of vegetation, on the stability of slopes, can be broken down into two categories (Glendinning, et al., 2009b);

1. hydrogeological effects;
2. mechanical effects.

The hydrogeological benefits of vegetation on slope stability have long been discussed. Water removal reduces pore water pressures and creates suctions, thus increasing shear strength and overall slope stability. The development of suctions due to vegetation depend on the type of vegetation, the water demand of that vegetation, the rooting depth, the root density, the hydraulic conductivity of the soil, and the availability of water within the soil (Smethurst, et al., 2006; Smethurst, et al., 2015).

The suctions developed by vegetation during summer drying can be critical for slope stability. Suctions from high water demand trees can be maintained during wet winters maintaining a slope’s stability (Briggs, et al., 2013). However, it is suggested that the suctions developed by high water demand trees cannot be relied upon during particularly wet periods (Ridley, et al., 2004).
Grass covered slope will develop suctions in the summer that will most likely be lost during wet winters (Smethurst, et al., 2006). Vegetation type has a large impact on pore water pressure suctions a soil experiences during summer. For grass and shrubs, suctions of approximately 50kPa at 1m have been observed in a clay cut slopes (Smethurst, et al., 2006). However, for high water demand trees suctions of over 500kPa have been recorded (Glendinning, et al., 2009b).

Vegetation plays a significant role in dictating the magnitude of summer-winter pore water pressures cycles and therefore the magnitude of shrink-swell behaviour, seasonal ratcheting, and corresponding strength and hydrogeological property deterioration. Vegetation induced shrink-swell cycles are also a major cause of alignment issues for engineered slopes (Loveridge, et al., 2010), shrink-swell displacement cycles of 50 to 55mm for high water demand trees and 5 to 8mm for grass covered slopes have been observed in clay embankments (Scott, et al., 2007).

Vegetation rooting depth has a direct impact on the nature of failure experienced by a slope, deeper rooting zones cause cyclic changes in stress due to seasonal wetting and drying to a greater depth and deeper-seated failures can be anticipated (Vaughan, et al., 2004). The depth of roots depends on the vegetation type and material stiffness, rooting depths will be less in highly compacted materials (Hughes, et al., 2009). Through detailed monitoring of soil water content and pore water pressures to investigate the effect of tree removal, Smethurst, et al. (2015) showed the removal of mature trees from the crest of a clay fill embankment reduced seasonal volumetric change but also caused the loss of suctions at depth, reducing the overall stability of the slope. Following the removal of trees persistent deep-seated suctions dissipate quickly, within approximately 2 to 4 years (Smethurst, et al., 2015).

Vegetation on slopes, regardless of type, provides a lag between rainfall, infiltration, loss of suctions and strength reduction and means that critical pore water pressures are not reached as frequently as would be the case for a non-vegetated slope (Smethurst, et al., 2006). However, root paths can alter the
hydraulic conductivity of the near-surface zone by introducing preferential flow routes (Glendinning, et al., 2009b).

In addition to the benefits of pore water pressure suctions due to vegetation increasing slope stability, root reinforcement and buttressing, mechanical effects, increase the near-surface shear strength of a soil (Smethurst, et al., 2006; Glendinning, et al., 2009b).

### 2.3.4 Seasonal Soil Water Content Variation

Weather drives near-surface soil water content variation influencing key slope deterioration mechanisms. Soil-water content conditions are transient and change significantly between seasons (Blight, 2003); the following section looks at observations of seasonal behaviour in clay slopes.

For a grass covered cut slope, Smethurst, et al., (2012) studied seasonal variation of soil water content and pore water pressures to investigate seasonal soil water content cycles and shrink-swell behaviour. Using extensive monitoring, it was found that pore water pressures return to near hydrostatic at ground level for an average winter. It is also suggested that the level of summer rainfall, June-August has the largest impact on the magnitude of seasonal pore water pressure cycles (Smethurst, et al., 2012). Figure 2.7 presents soil water content variation in the near-surface of a grass covered London Clay cut slope and Figure 2.8 presents suction measurements (Smethurst, et al., 2012). It shows the inter-seasonal variation in pore water pressures and therefore stresses that the slope experiences.
From Figure 2.8, it can be seen that suctions develop in a progressive manner with increasing depth during summer drying. The behaviour of recharge is much quicker, resulting in the rapid increase of pore water pressures (Smethurst, et al., 2012). The response to seasonal variation occurs much quicker in shallow regions taking more time to act at depth (Smethurst, et al., 2012; Glendinning, et al., 2014).
Smethurst, et al., (2006) presented pore water pressure profiles with depth for different times during the year, illustrating the magnitude of seasonal pore water pressure variation with depth, see Figure 2.9. It also shows the limiting condition of near hydrostatic conditions during wet winters.

![Figure 2.9 Pore water pressure distribution with depth, grass covered clay cut slope (after, Smethurst, et al., 2006)](image)

### 2.3.5 UK Climate and Engineered Slopes

In early soil mechanics it was accepted that changes in seasonal conditions affected slope behaviour. Ground movement could be attributed to seasonal changes in soil moisture content and temperature fluctuation. Temperature fluctuation leading to thermal expansion and contractions, soil water content fluctuation leading to shrinkage and swelling and freeze thaw mechanisms leading to strength reductions (Terzaghi, 1950).

Engineered slopes within the UK experience most slope failures during winter months (Loveridge, et al., 2010), as shown by the 160 infrastructure slope failures recorded during the winter of 2000-2001 (Turner, 2001). Increased rainfall causes elevated pore water pressures thus reducing effective stress and shear strength, which can trigger failures. Landslides in the UK are primarily triggered by intense rainfall events following prolonged low-medium intensity rainfall (Dijkstra, et al., 2015). In general, the UK experiences frequent
low intensity rainfall events. However, intense rainfall events do occur such as that of winter 2000-2001 and winter 2013 (Dijkstra, et al., 2015).

Although wet winters are considered critical for slope failures, Loveridge, et al. (2010) showed earthworks constructed in high-plasticity clay soils are also susceptible to instability and serviceability failure during summer months. Seasonal pore water pressure variation causes cyclic straining within slopes, the extent of the straining is dependent on the magnitude of seasonal variation, the vegetation present and the soil characteristics. Along with shrink-swell, repeated stress cycling has the potential to cause strength reduction through fatigue. This mechanism will likely be a contributing factor to failure, but triggering will be due to increased pore water pressures, making it difficult to establish the significance of climatic driven shrink-swell effects on failure (Loveridge, et al., 2010).

For low hydraulic conductivity soils the movement of water is at such a slow rate that the slopes’ hydraulic condition at depth is dependent on past climatic cycles (Loveridge, et al., 2010; Dijkstra, et al., 2015). The pore water pressure state leading into winter is fundamental to the stability of the slope (i.e. a wet summer followed by a wet winter is far more likely to cause failure than a dry summer followed by a wet winter). Antecedent / proprietary factors are crucial for understanding slope stability (Dijkstra, et al., 2015).

### 2.3.6 Climate Change

Climate change is defined by the Intergovernmental Panel on Climate Change (IPCC) as follows;

“Climate change refers to a change in the state of the climate that can be identified (e.g., by using statistical tests) by changes in the mean and/or the variability of its properties and that persists for an extended period, typically decades or longer. Climate change may be due to natural internal processes or external forcings such as modulations of the solar cycles, volcanic eruptions and persistent anthropogenic changes in the composition of the atmosphere or in land use.”

(IPCC, 2014)
The concept of climate change is accepted globally, it is known that even if emission levels are reduced significant changes to the world’s climate will still occur. Figure 2.10 shows potential global temperature and precipitation change for the average of the projection models carried out for the period 1986-2005 to 2081-2100 (IPCC, 2014).

![Climate change projection models](image)

**Figure 2.10** Climate change, temperature and precipitation, average of projection models available (1986-2005 to 2081-2100) (after, IPCC, 2014)

### 2.3.6.1 UK Climate Change Projections (UKCP09)

UKCP09 is the most sophisticated and statistically sound set of climate projections available for the UK, compiled to allow scenario-based assessment of potential future climatic conditions for research and guidance. Three emission cases are considered low, medium and high, the projection data is based on the climate during 1961 to 1990, which forms the baseline period for any analysis. This period of weather data used to for the baseline of analyses has been selected due to the availability of weather data during the period and the fact that effects of external forcings (i.e. level of emissions from manmade processes) are much less than present day.
Climate change projections, UKCP09, provide clear trends for anticipated future variation through probability distributions and emission scenarios for particular aspects of climate, for example the anticipated percentage change in temperature or rainfall can be easily established. However, current projections do not provide information on the likely increase in frequency or duration of climatic extremes, such as storms (Kilsby, et al., 2009). Therefore, the application of climate change projections carries significant uncertainties and limitations.

Climate projections are given in the form of probability distributions as percentage confidence levels. For example, 10% probability level is the change that is perceived as extremely likely to be exceeded whereas the 90% level is very unlikely to be exceeded (Dijkstra, et al., 2015). The headline trends for climate change projections in the UK are wetter winters with higher frequency extreme events and drier summers (Wilby, et al., 2008).

### 2.3.6.2 Climate Change and Slope Stability

The general outcome of UKCP09 has indicated drier summers and wetter winters for the UK. This has been generally accepted within the engineering industry but the effect that this will have on slope stability are still debatable.

Changing climate will lead to alteration of the soil water balance (i.e. the infiltration-evapotranspiration balance). This hydrological balance is an important aspect of slope stability. A considerable shift in this balance could bring about a change in the mode, distribution and frequency at which slopes fail (Dixon & Brook, 2007; Dixon, et al., 2007; Dijkstra & Dixon, 2010). Wetter winters suggest a concentration of rainfall over a shorter period, potentially higher intensity rainfall events occurring more frequently compared to current trends. This would suggest an increase in the frequency at which triggering events occur (Dijkstra, et al., 2015).

Drier summers could result in an overall increase in slope stability with higher suctions occurring. However, high suctions and reduced soil water content may cause vegetation to change or die, altering the soil-water-vegetation system and introducing other problems such as surface erosion and washout.
Vegetation is extremely sensitive to climate variation, a change of 1 to 2°C can result in a change in the water usage and rooting characteristics (Hughes, et al., 2009; Kilsby, et al., 2009). Climate change projects for the UK suggest such changes could occur, which means vegetation demands and characteristics are likely to change.

Clarke and Smethurst (2010) showed that the likely frequency of dry summer conditions, using soil moisture deficit and climate change projection data, will increase dramatically suggesting current extremes will be a normal occurrence in the future. Clarke and Smethurst (2010) highlight the fact that dry conditions will lead to increased suctions but that they also lead to volumetric change in high-plasticity clays, seasonal ratcheting and desiccation cracking that will affect a greater depth of material increasing the hydraulic conductivity of the near-surface soil. Even with the additional benefit of increased suctions due to drier summers these other implications could reduce the overall stability of a slope, larger cycles of seasonal ratcheting and rapid inundation from significant rainfall events may increase the rate at which progressive failure occurs or even trigger failure (Dijkstra & Dixon, 2010). However, rainfall events are suggested to become more intense, this will decrease the time that infiltration can occur, potentially increasing surface run-off due to rapid saturation of the near-surface. In addition, dry top soil prior to a rain event may be too dry and hard to allow maximum infiltration, causing more runoff (Loveridge, et al., 2010).

Climate change is anticipated to change the frequency and nature of slope failures in the future, increasing uncertainty and risk associated with slope instability (Lacasse, et al., 2004; Dijkstra & Dixon, 2010; Loveridge, et al., 2010). This means that the infrastructure network is vulnerable to poor earthwork asset performance. Ageing infrastructure slopes within high-plasticity clays have been identified as the most susceptible slopes to failure due to climate change (Dijkstra, et al., 2015). It has also been suggested that slope instability across the UK will increase regardless of the anticipated climate change effects purely due to strength deterioration associated with fine-grained soils and age (Glendinning, et al., 2009a).
The full extent of climate change on engineered slope behaviour is currently unknown. Not conducting further research using current knowledge and climate change projections regarding slope stability and anticipated negative implications of climate change will carry considerable financial risk to the economy (Dijkstra & Dixon, 2010).

2.4 Numerical Representation of Soil Behaviour

Mechanical and hydrogeological behaviour of cut slopes along with climatic conditions and projections for the UK have been presented and the interrelated nature of behaviour and dependencies of different processes have been discussed. Given an understanding of the behaviours that are fundamental to long-term stability of clay cut slopes, methodologies available for the assessment of this complex inter-related behaviour have been considered. This has been done by considering the different strands of soil mechanics before critiquing current state-of-the-art modelling approaches for weather-driven behaviour and progressive failure.

Broadly, soil mechanics considers the physical behaviour and properties of soils (Fredlund & Rahardjo, 1993). It can be broken down into two strands; one considering saturated soils and the other considering unsaturated soils. When all pore spaces within a soil are filled with water, the soil is saturated. In this instance the pore fluid can be considered incompressible. When air and water phases are present within the pore matrix, the soil is unsaturated (also referred to as partially saturated within literature) and the pore fluid is compressible; in this case classic saturated soil mechanics cannot accurately describe behaviour (Fredlund & Rahardjo, 1993). The division of soil mechanics is illustrated in Figure 2.11, where $u_w$ is the pore water pressure;
Soil models illustrating the phases for saturated and unsaturated soils are shown in Figure 2.12, the mass and volume relationships are also displayed schematically.

Where:
- \( V \) = total volume;
- \( V_s \) = volume of solids;
- \( V_w \) = volume of water;
- \( V_a \) = volume of air;
- \( M \) = total mass;
- \( M_s \) = mass of solids;
- \( M_w \) = mass of water;
- \( M_a \) = mass of air.

Mechanical behaviour, such as shear strength and volume change, are dependent on soil state, i.e. saturated or unsaturated. The development of soil mechanics in the past has focused on saturated soils due to its relative simplicity compared to unsaturated soil behaviour. Unsaturated soil problems
involve pore water pressure suctions affecting strength, volume and the hydraulic conductivity of the soil (Fredlund & Rahardjo, 1993).

### 2.4.1 Stresses and Strains in Soil

Stress states give a relationship of external stresses to internal stresses that can be used to formulate constitutive models of physical behaviour. To understand the formulation of stress state variables it is necessary to establish the stresses and strains that act upon an element of soil.

The stresses acting on an element of soil can be illustrated in Figure 2.13;

![Figure 2.13 Stresses on an element of soil](image)

The forces acting on an element of soil can be considered as body and surface forces. Body forces are forces acting through the centroid of the element and surface forces act on the element boundary. The stresses shown in Figure 2.13 are;

- normal stress ($\sigma$) - stress component that acts perpendicular to a plane (body stress);
- shear stress ($\tau$) - stress component that acts parallel to a plane (surface stress).

In the same way that strains on an element of soil can be considered;
normal strain ($\varepsilon$) – strain component that acts perpendicular to a plane;

- shear strain ($\gamma$) – strain component that acts parallel to a plane.

In a two-dimensional plane-strain formulation for small strain, it is given that;

\[
\begin{align*}
\varepsilon_{xx} &= \frac{\partial x}{l_x}; \quad \varepsilon_{yy} = \frac{\partial y}{l_y}; \quad \varepsilon_{zz} = 0 \\
\gamma_{xy} &= \frac{\partial x}{l_y} + \frac{\partial y}{l_x}; \quad \gamma_{xz} = 0; \quad \gamma_{yz} = 0
\end{align*}
\]

\[\text{Equation 2.2}\]

\[\text{Equation 2.3}\]

### 2.4.2 Numerical Modelling of Soil

Numerical methods include many methods, with the most popular being finite element or finite difference models. Within finite element and finite difference analysis, problems are represented by a mesh of smaller discrete elements and/or nodes. These approaches are capable of modelling realistic geotechnical behaviour of soils. A model’s capability of accurately describing
physical processes is dependent on the constitutive model used and its ability
to emulate soil behaviour along with the engineer’s ability to apply
representative material properties and boundary conditions throughout the
analysis (Potts, 2003). Numerical modelling allows the analysis of the full life
cycle of a problem under transient conditions, and failure mechanisms are
determined and therefore do not need to be postulated (Potts, 2003). As with
any numerical representation of a physical process, the assumptions made,
and limitations of approaches must be understood.

The theoretical requirements that must be considered for numerical analysis
are as follows;

- Equilibrium;
- Compatibility;
- Constitutive model;
- Boundary conditions.

All forces and stresses within any model must be in equilibrium for the model
to be valid. Internal equilibrium, and corresponding stress fields, must satisfy
the following partial differential equations (expressed in terms of total stress
and Cartesian co-ordinates);

\[
\begin{align*}
\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{yx}}{\partial y} + \frac{\partial \tau_{zx}}{\partial z} + \gamma_x &= 0 \\
\frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{zy}}{\partial z} + \gamma_y &= 0 \\
\frac{\partial \tau_{zx}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \sigma_z}{\partial z} + \gamma_z &= 0
\end{align*}
\]

Where:
- \( \sigma_i \) = total stress;
- \( \tau_i \) = shear stress;
- \( \gamma_y \) = self-weight (assumed to act vertically down in line with sign
  convention in Figure 2.13) assuming no dynamic body forces are acting;
- \( \gamma_x = \gamma_z = 0 \) assuming no dynamic body forces are acting.

Compatibility considers the kinematic conditions of a model. Under
displacements the model moves. However, the geometry and mesh must
remain intact and representative of the problem. The geometry, displacements and strains that exist within the model must be compatible. Under small strains the following must be satisfied;

\[
\begin{align*}
\varepsilon_x &= \frac{\partial u}{\partial x}; \quad \varepsilon_y = \frac{\partial v}{\partial y}; \quad \varepsilon_z = \frac{\partial w}{\partial z} \\
\gamma_{xy} &= \frac{\partial v}{\partial x} + \frac{\partial u}{\partial y}; \quad \gamma_{yz} = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z}; \quad \gamma_{xz} = \frac{\partial w}{\partial x} + \frac{\partial u}{\partial z}
\end{align*}
\]

Where;
- \( u \) = displacement in x direction;
- \( v \) = displacement in y direction;
- \( w \) = displacement in z direction;
- \( \varepsilon_i \) = normal strain;
- \( \gamma_i \) = shear strain.

The constitutive model links the stress-strain-volume behaviour of the soil. Essential the constitutive model describes soil behaviour, which links the stresses and strains. In its simplest form the constitutive relationship can be expressed as follows;

\[
\{\Delta \sigma\} = [D]\{\Delta \varepsilon\}
\]

Many constitutive models exist that can be used for different materials. The selection of a constitutive model should be based on the accurate description of the principle soil behaviour or mechanism under investigation and material in question.

Boundary conditions consider the sequencing of modelling, the initial stress conditions, construction or excavation effects along with the physical constraints of the problem and any loading be it mechanical or hydrogeological. These factors are defined by the modeller and should be selected to ensure that the model accurately replicates the situation being considered.

### 2.4.3 Stress States

To allow the development of numerical models, stress states and stress state variables used within constitutive models must be defined to describe mechanical behaviour, relating observed physical behaviour to internal stress.
conditions. Stress state variables are fundamental to the numerical representation of geotechnical principles. Stress states should be independent of soil characteristics and therefore, the relationship is applicable for all soil types.

2.4.3.1 Saturated Stress State

Classic soil mechanics based on saturated soils has been successful in its attempt to replicate physical behaviours numerically because of the simple relationship between physical behaviour and internal stress conditions that has been formulated (Fredlund & Rahardjo, 1993).

The stress state variable for saturated soil is known as effective stress, it is independent of soil properties and was proposed by Terzaghi (1925), assuming the fluid phase, filling pores, is incompressible. Effective stress is given as:

\[ \sigma' = \sigma - u_w \]

Where;
- \( \sigma \) = total stress;
- \( \sigma' \) = effective stress;
- \( u_w \) = pore water pressure.

Saturated effective stress incorporates external applied stresses and internal fluid pressure, effectively converting a two-phase porous medium, solid and water phases, into an equivalent mechanical single-phase continuum (Nuth & Laloui, 2008). Effective stress has been fundamental in the development of classic saturated soil mechanics;

“All the measurable effects of change in stress, such as compression, distortion, and changing in shearing resistance, are exclusively due to changes in the effective stress.” (Terzaghi, 1936)

Attempts have been made to overcome some of the assumptions of Terzaghi’s effective stress equation; all attempts have included the addition of soil properties, namely porosity and compressibility coefficients, with differing success (Skempton, 1960; Carroll & Katsube, 1983; Jardine, et al., 2004).
However, with consideration of the assumptions (i.e. pore fluid is incompressible), Terzaghi’s effective stress gives a stress for saturated soils that governs mechanical behaviour.

2.4.3.2 Unsaturated Stress States

Unsaturated soils (also referred to as partially saturated soils in the literature) include an additional air phase. Compared to the saturated formulation where the solid and water phase are considered incompressible, the gaseous, air-phase cannot be considered incompressible (Nuth & Laloui, 2008).

In natural soils, pore fluid is a combination of water and dissolved air. However, in the formulation of stress states for unsaturated soils the phases are assumed to be an idealised homogeneous liquid and gas (Nuth & Laloui, 2008). In this instance the soil is fully saturated by two fluids, a wetting and a non-wetting fluid, water and air.

Initial attempts to formulate an ‘effective stress’ style relationship for unsaturated soils mimicked the approach adopted by Terzaghi (1925; 1936). Converting a multi-phase medium with solid, liquid and gas phases into a single stress state variable that describes mechanical behaviour (Nuth & Laloui, 2008). Bishop’s generalised effective stress sometimes known as the soil skeleton stress is given as (Bishop, 1959);

\[
\sigma' = (\sigma - u_a) + \chi(u_a - u_w)
\]

Where:

- \(\sigma - u_a\) = net stress;
- \(u_a - u_w\) = matrix suction;
- \(\sigma\) = total stress;
- \(\sigma'\) = generalised effective stress;
- \(u_w\) = pore water pressure;
- \(u_a\) = pore air pressure;
- \(\chi\) = effective stress parameter or Bishop’s parameter; it is assumed that \(\chi = S_r\), degree of saturation in the generalised form.
The assumption that the effective stress parameter is the same as the degree of saturation is an approximation and it has been shown that the relationship is dependent on the micro structure of the soil (Jardine, et al., 2004). Relationships for $\chi$ and $S_r$ can be seen in Figure 2.16.

![Figure 2.16](image)

Figure 2.16 $\chi$ against degree of saturation ($S_r$) for different soils (after, Jardine, et al., 2004)

It was found that constitutive models based on Bishop’s generalised effective stress alone were incapable of describing complete physical behaviour of unsaturated soils, in particular wetting collapse (Nuth & Laloui, 2008). Numerous adaptations of the saturated ‘effective stress’ style equation have been proposed to describe unsaturated soil behaviour although extending saturated soil mechanics to unsaturated problems has proved difficult (Fredlund & Rahardjo, 1993).

Fredlund and Morgenstern (1977) found two independent stress state variables could be used to describe constitutive behaviour of unsaturated soils. Three combinations of possible stress state parameters have been considered (Fredlund & Rahardjo, 1993):

<table>
<thead>
<tr>
<th>Reference Pressure</th>
<th>Stress State Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air, $u_a$</td>
<td>$(\sigma - u_a)$ and $(u_a - u_w)$</td>
</tr>
<tr>
<td>Water, $u_w$</td>
<td>$(\sigma - u_w)$ and $(u_a - u_w)$</td>
</tr>
<tr>
<td>Total, $\sigma$</td>
<td>$(\sigma - u_a)$ and $(\sigma - u_w)$</td>
</tr>
</tbody>
</table>
The most commonly adopted independent stress variables for practical and physical purposes are the set with reference to air pressure (Nuth & Laloui, 2008). They can be considered the net stress \( \sigma_{net} = (\sigma - u_a) \) and matrix suction \( (u_a - u_w) \) (Fredlund & Rahardjo, 1993).

### 2.4.4 Elastic Constitutive Relationships

Given the stress states for saturated and unsaturated behaviour, the formulations of constitutive models for elastic behaviour for both saturated and unsaturated behaviour are presented.

#### 2.4.4.1 Saturated Elastic Constitutive Relationship

Changes to the soil skeleton, soil particle interactions, are reliant on changes in effective stress. The straining of the soil skeleton and thus the physical behaviour is a function of the effective stress and means that the following, relatively simple, constitutive model to describe elastic behaviour can be adopted;

Equation 2.9

\[
\dot{\varepsilon}^e_{ij} = C^e_{ijkl} \dot{\sigma}^I_{kl}
\]

Where;

- \( \dot{\varepsilon}^e_{ij} \) = rate of straining of soil skeleton;
- \( C^e_{ijkl} \) = drained elastic compliance matrix;
- \( \dot{\sigma}^I_{kl} \) = effective stress increment.

The effective stress concept has been shown to hold true experimentally, and numerical models are in acceptable agreement with observed physical behaviour, such as consolidation (Nuth & Laloui, 2008). As successful as the principal is, its application is limited to fully saturated soils.

#### 2.4.4.2 Unsaturated Elastic Constitutive Relationship (Independent Stress States)

The constitutive equation describing elastic behaviour considering independent stress state variables for unsaturated soils with reference to air pressure is given as;

Equation 2.10

\[
\dot{\varepsilon}^e_{ij} = C^e_{ijhk} (\dot{\sigma}^I_{hk} - \dot{u}_a) + C^s (\dot{u}_a - \dot{u}_w)
\]
Where:

- \( C_{ijk}^e \) = drained elastic compliance matrix;
- \( C^s \) = elastic hydric modulus (coefficient of proportionality between strain and suction);

Regardless of the set of stress state variables used the constitutive behaviour of unsaturated soils can be simplified into two components of strain (Nuth & Laloui, 2008).

Equation 2.11

\[
\dot{\varepsilon} = \dot{\varepsilon}_m + \dot{\varepsilon}_h
\]

Where:

- \( \dot{\varepsilon}_m \) = mechanical strain increment;
- \( \dot{\varepsilon}_h \) = hydraulic strain increment.

Although, independent stress state variables can formulate reasonable constitutive behaviours there are limitations. The use of independent stress state variables requires the addition of more material properties. Also, separation of mechanical stresses and hydrological stresses means hydraulic hysteresis and its effect on mechanical behaviour cannot be directly accounted for (Nuth & Laloui, 2008).

2.4.4.3 Unsaturated Constitutive Relationship (Complete Stress-Strain Framework)

A stress-strain framework including hydro-mechanical coupling adopting Bishop’s generalised effective stress with the addition of a second stress state to account for hydrogeological influences has been developed and the benefits discussed by Nuth and Laloui (2008). The stress state variables for a complete hydro-mechanical coupling of stress states are as follows;

Equation 2.12

\[
\begin{cases}
\sigma' = \sigma_{net} + \bar{\mu}(s, S_r) \\
\xi = \bar{\xi}(s, S_r)
\end{cases}
\]

Where:

- \( s \) = suction;
- \( S_r \) = degree of saturation;
- \( \sigma' \) = generalised effective stress, governs skeleton stresses;
- \( \xi \) = additional stress state to complete hydraulic behaviour description.
The additional variables describe the hydraulic state of the soil. This is through the inclusion of suction stress and degree of saturation. This approach provides smooth transition between unsaturated and saturated states and the presence of the function $\tilde{\mu}(s, S_r)$ allows the inclusion of hydraulic influence on the inter-particle stresses (Nuth & Laloui, 2008). The complete stress-strain framework is given as:

\begin{equation}
\sigma' = (\sigma - u_a) + S_r \cdot s
\end{equation}

\begin{equation}
s = u_a - u_w
\end{equation}

Where, generalised effective stress is linked to strains through a constitutive relationship (i.e. similar to Equation 2.9 but considering Bishop’s generalised effective stress rather than saturated effective stress) and suctions are linked to saturation through soil water retention properties. Due to the simple implementation of the framework shown in Equation 2.13 into existing numerical modelling codes, it is the most commonly adopted framework for modelling unsaturated soil behaviour (Nuth & Laloui, 2008).

Relationships between suction and degree of saturation are dependent on soil water retention properties which are also used to obtain relative hydraulic conductivity. The formulation of the relationships are shown in Section 2.4.6.

### 2.4.5 Strain-Softening Constitutive Model

One of the key behaviours attributed to failure of overconsolidated clay slopes is progressive failure. Within numerical models, this behaviour is represented using a strain-softening constitutive model as it allows post-peak strength reduction to be calculated as a function of plastic strains. For the consideration of progressive failure within overconsolidated clay slopes, Mohr-Coulomb elasto-plastic constitutive model has previously been employed (Potts, et al., 1997; Ellis & O’Brien, 2007).

Within the Mohr-Coulomb elasto-plastic constitutive model, the Mohr-Coulomb failure criteria is used to define the point of failure of the material and the theory of elasto-plasticity offers a framework for modelling strain-softening behaviour. Within elasto-plasticity, the soil behaviour is elastic up to the point when the
stress state of the soil reaches the yield locus (A) in Figure 2.17. At this point
the soil experiences plastic behaviour.

![Diagram of stress state and yield locus](image)

Figure 2.17 Yield locus and stress path for elasto-plastic strain-hardening and softening

Where,

- \( p' = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \) = mean normal stress;
- \( q = \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \) = deviatoric stress.

To establish the yield point of the soil, the point at which behaviour changes
from elastic to plastic, a yield function (the yield locus in Figure 2.17) must be
defined. The yield function \( F \) is a function of the stresses within the soil \( \{\sigma\} \) and
state parameters of the material \( \{k\} \). When the yield function is less than 0 the
material is elastic, when the yield function equals 0 the material is plastic and
plastic straining will occur. The direction of plastic straining is dependent on
the plastic potential function, flow rule and stress state. Like the yield function,
the plastic potential function is a function of the stress state and the state
parameters of the material. If the plastic potential function is assumed to be
equivalent to the yield function, the flow is associated. However, when the
plastic potential function and yield function differ, flow is non-associated.

As mentioned previously, the point of failure within the model is defined by the
Mohr-Coulomb failure criteria. This is given in Equation 2.14.

Equation 2.14 \[ \tau = c' + \sigma'Tan\varphi' \]

Where,

- \( \tau = \) shear stress;
• \( \sigma' \) = effective stress;
• \( c' \) = cohesion;
• \( \varphi' \) = internal angle of friction.

The yield surface of the Mohr-Coulomb model is obtained by plotting Mohr’s circle for stresses at failure considering the maximum and minimum principal stresses. The yield surface is represented by a hexagon in the \( \sigma_1, \sigma_2 \) and \( \sigma_3 \) space, as shown in Figure 2.18. The yield surface is defined by Equation 2.15.

![Figure 2.18 Mohr-Coulomb yield function](image)

Equation 2.15

\[
F(\sigma) = \frac{J}{(\frac{c'}{\sin\varphi'} - p') \cdot g(\theta)}
\]

Where,

- \( J = \frac{1}{\sqrt{6}} \sqrt{(\sigma_1' - \sigma_2')^2 + (\sigma_2' - \sigma_3')^2 + (\sigma_3' - \sigma_1')^2} \) = second invariant of deviatoic stress;
- \( \theta \) = Lode’s angle, gives orientation of stress state in deviatoric plane;
- \( g(\theta) = \frac{\sin\varphi'}{\cos\theta \cdot \frac{\sin\theta \cdot \sin\varphi'}{\sqrt{3}}} \);
- \( \tan\theta = \frac{2\sigma_1 - \sigma_2 - \sigma_3}{\sqrt{3}(\sigma_3 - \sigma_2)} \)

For the elastic portion of behaviour, Young’s modulus and Poisson’s ratio are required, behaviour post yielding (i.e. strain-hardening or strain-softening) is dictated by the state parameters specified for the material. A simple material softening relationship relative to plastic strains can be seen in Figure 2.19.
Local strain-softening models are models where material softening is dictated by plastic displacement or strain criteria with reference to a single element or calculation point.

This approach has been used very effectively (Potts, et al., 1997; Ellis & O’Brien, 2007; Rouainia, et al., 2009) but there has been growing concern regarding the mesh dependency of numerical models considering localisation problems such as progressive failure. When local strain-softening is employed, different element sizes and orientations can alter the location, time and thickness at which shear surfaces develop within a numerical model (Belytschko, et al., 1986; Galavi & Schweiger, 2010).

To overcome the problems associated with a conventional local strain-softening constitutive model, regularisation techniques have been explored (Galavi & Schweiger, 2010). The most promising of these techniques is nonlocal strain-softening.

Nonlocal strain-softening approaches consider the strains in adjacent elements, averaging local strains across a mesh to the calculation point where softening is being considered using a weighting function to obtain a nonlocal strain that dictates softening (Galavi & Schweiger, 2010; Summersgill, et al., 2017). Because the nonlocal approaches averages strains, the occurrence of unrealistic high strains, leading to rapid softening within a numerical model, is regulated (Summersgill, 2015).
One of the major benefits of the nonlocal regularisation technique is the governing equations are the same as the local model with the addition of only one extra variable to define the spatial averaging of local to nonlocal strains (Lu, et al., 2009; Summersgill, 2015). This makes the implementation of a nonlocal strain-softening model within an existing numerical analysis package straightforward (Summersgill, 2015).

It is common for the nonlocal regularisation technique to be partially implemented, considering the parameters associated with strain-softening only, i.e. calculating nonlocal strain but ignoring nonlocal stresses (Galavi & Schweiger, 2010). Galavi and Schweiger, (2010) showed nonlocal strain ($\varepsilon^*$) at a calculation point ($x_n$) can be calculated using the following equations;

\[ \varepsilon^*(x_n) = \frac{1}{V_w} \iiint \omega'(x_n') \varepsilon_d(x_n + x_n') dx_1' dx_2' dx_3' \]

Where:
- $V_w = \iiint \omega(x_n') dx_1' dx_2' dx_3' = \text{weighted volume}$;
- $\omega'(x_n') = \text{is the weighting function relating local strains of nearby elements to the calculation point to determine the nonlocal strain at that calculation point}$;
- $\varepsilon_d(x_n + x_n') = \text{local strain at different calculation points}$;
- $x_n = \text{global coordinate}$;
- $x_n' = \text{local coordinate}$.

Within literature numerous weighting functions are considered. Summersgill, et al., (2017) showed that for slope stability analysis considering nonlocal strain softening, the weighting function, proposed by Galavi and Schweiger, (2010) was the least mesh dependent of the weighting functions considered.

The weighting function proposed by Galavi and Schweiger, (2010) looks to consider local strains adjacent to a calculation point and ignores the local strain at the calculation point, ensuring excessive strains within elements due to numerical instability do not force rapid unrealistic softening. The weighting function is shown in Equation 2.17 with a graphical representation shown in Figure 2.20.
Equation 2.17  \[ \omega(r) = \frac{r}{l^2} e^{-\left(\frac{r}{l}\right)^2} \]

Where:
- \( r \) = distance between calculation point and nonlocal strain of surrounding elements;
- \( l \) = internal length or defined length parameter.

After obtaining the nonlocal strains, material softening is calculated in the same manner as local strain-softening, but nonlocal plastic strains rather than local plastic stains are used.

2.4.6 Fluid Flow

As discussed previously, the flow of water within soil has significant implications on the behaviour of a slope. Saturated flow of water following undrained unloading causes volume and stress changes as pore water pressures dissipate and the soil swells through consolidation. In addition, near-surface weather-driven behaviour causes seasonal cycles of pore water pressures and thus stress cycles that cause seasonal ratchetting.
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The movement of water within a soil is dependent on the hydraulic conductivity of the material and this is affected dramatically by the phases (i.e. solid, liquid and gas) present within the soil. The equations governing flow within saturated (including consolidation theory) and unsaturated soils have been presented.

### 2.4.6.1 Saturated Flow

One-dimensional fluid flow within a fully saturated soil can be described by Darcy’s law, first published in 1856;

\[
q = -k \frac{\partial H}{\partial y}
\]

**Equation 2.18**

Where;
- \( q \) = specific flux (m/s);
- \( k \) = saturated hydraulic conductivity (m/s);
- \( \frac{\partial H}{\partial y} \) = hydraulic gradient in the y.

For a two-dimensional flow problem, saturated flow can be described by Laplace’s equation, Equation 2.19.

\[
K_x \frac{\partial^2 H}{\partial x^2} + K_y \frac{\partial^2 H}{\partial y^2} = 0
\]

**Equation 2.19**

Where;
- \( K_x \) & \( K_y \) = saturated hydraulic conductivity in x and y respectively (m/s);
- \( H \) = total head (m).

### 2.4.6.2 Consolidation Theory

Post-excavation pore water pressure equilibration is modelled as a consolidation process, in most cases the Biot theory of consolidation is used in geotechnical models. Biot consolidation theory considers an elastic porous medium with saturated laminar pore fluid, it is given (after, Biot, 1941);

\[
\frac{K'}{\gamma_w} \left[ k_x \frac{\partial^2 u_w}{\partial x^2} + k_y \frac{\partial^2 u_w}{\partial y^2} + k_z \frac{\partial^2 u_w}{\partial z^2} \right] = \frac{\partial u_w}{\partial t} - \frac{\partial \sigma}{\partial t}
\]

**Equation 2.20**

Where;
- \( K' \) = bulk modulus;
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- \( \gamma_w \) = unit weight of water;
- \( k_x; k_y; k_z \) = hydraulic conductivity in x, y and z direction;
- \( u_w \) = pore water pressure;
- \( \sigma \) = mean total stress.

For a 2D plane-strain fully saturated soil, assumed to be incompressible the governing equation for consolidation theory is given as (after Biot, 1941);

\[
\frac{k_x}{\gamma_w} \frac{\partial^2 u_w}{\partial x^2} + \frac{k_y}{\gamma_w} \frac{\partial^2 u_w}{\partial y^2} + \frac{d}{dt} \left( \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} \right) = 0
\]

Where;
- \( u \) = displacement in the x direction;
- \( v \) = displacement in the y direction.

2.4.6.3 Unsaturated Flow

The most commonly adopted equation for describing unsaturated flow is Richard’s equation;

\[
- \frac{\partial K_r(\theta)}{\partial z'} + \frac{\partial}{\partial z'} \left[ K_r(\theta) \frac{\partial \psi(\theta)}{\partial z'} \right] = \frac{\partial \theta}{\partial t}
\]

Where;
- \( K_r \) = relative hydraulic conductivity (m/s);
- \( \theta \) = soil water content;
- \( \psi \) = pore water pressure suction (Pa);
- \( z' \) = depth vertically;
- \( K_r(\theta) \) = relative hydraulic conductivity as a function of soil water content;
- \( \psi(\theta) \) = pore water pressure suction as a function of soil water content.

Equation 2.22 gives vertical flow for infiltration into a uniform unsaturated soil. From the terms present it is clear that an understanding of the relative hydraulic conductivity and pore water pressure suctions with soil water content must be established. Simple closed form solutions for \( K_r(\theta) \) and \( \psi(\theta) \) have been proposed through the development of soil water retention curves (also referred
to as soil water characteristic curves within literature), these are discussed in Section 2.4.6.4.

With the existence of macropores and preferential flows there is scepticism about the validity of the Richard’s equation, which was formulated under the assumption that the soil is heterogeneous, to describe flow within an unsaturated soil (Beven & Germann, 2013).

### 2.4.6.4 Soil Water Retention Characteristics

Soil water retention curves (SWRC), sometimes referred to as the soil water characteristic curves (SWCC), are a function of a soil that relates pore water pressure suctions and soil water content. The relationship between suction and soil water content is fundamental to unsaturated behaviour. In particular, completing the stress-strain framework presented in Section 2.4.4.3 and for establishing the relative hydraulic conductivity of unsaturated soil.

Soil water retention curves describe the suctions that develop as water is removed from a soil, they are obtained from continuous monitoring of the material and the curve is fitted to the observed data. The curves fitted take a closed form relationship between the soil water content and pore water pressure suctions, as in Equation 2.23 presented by van Genuchten (1980);

\[
\Theta = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \left[\frac{1}{1 + (\alpha \psi)^n}\right]^m
\]

Where:
- \(\Theta\) = dimensionless water content (degree of saturation) \((S_r)\);
- \(\alpha\) = van Genuchten fitting parameter (kPa\(^{-1}\) or m\(^{-1}\));
- \(n\) = van Genuchten fitting parameter;
- \(m\) = van Genuchten fitting parameter;
- \(\psi\) = pore water pressure (pressure head) (kPa or m);
- \(\theta\) = soil water content;
- \(\theta_r\) = residual soil water content;
- \(\theta_s\) = saturated soil water content.

The curves allow categorisation of soil hydrological properties in the unsaturated zone; an example curve is shown in Figure 2.22.
For the modelling unsaturated flow, the relative hydraulic conductivity function is required. This is a very difficult parameter to obtain accurately so commonly the soil water retention properties are used to estimate hydraulic conductivity (van Genuchten, 1980; Durner, 1994). van Genuchten (1980) presented a closed-form solution for unsaturated relative hydraulic conductivity based on either the pore water pressure suctions ($\psi$) or soil-water content ($\theta$) shown in Equation 2.24.

\[
\begin{align*}
K_r (\psi) &= k_{sat} \frac{\{1 - (\alpha \psi)^{n-1}\} \{1 + (\alpha \psi)^n\}^{-m}}{[1 + (\alpha \psi)^n]^{m/2}} \\
K_r (\theta) &= k_{sat} \theta^{1/2} \left[1 - \left(1 - \theta^{1/m}\right)^{m}\right]^2
\end{align*}
\]

Equation 2.24 shows the relationship between relative hydraulic conductivity and pore water pressure and dimensionless water content (van Genuchten, 1980). The relative hydraulic conductivity function can be displayed graphically, an example for London Clay can be seen in Figure 2.23.
Figure 2.23 shows how relative hydraulic conductivity changes with soil water content when using the closed form solution proposed by van Genuchten (1980). The figure shows that when water is removed from the soil, the relative hydraulic conductivity reduces, this is due to the reduction in saturated flow routes through the soil matrix; as more capillaries become unsaturated the movement of water becomes harder. Physically this relationship can be considered representative for an idealised soil where the soil matrix does not suffer any significant deformation or change in void ratio due to drying. However, if a material experiences large shrinkage and the confining pressure is exceeded then desiccation cracking occurs, changing the pore structure of the material and the relationship presented by van Genuchten (1980) is not so representative.

Soil water retention curves have been further developed to incorporate pore heterogeneity to improve the approximation of soil hydraulic conductivity for unsaturated flow (Durner, 1994). Representation of a complex pore structure with a unimodal soil water retention curve can result in poor representation of the physical system. Durner (1994) proposed a method to combine multiple unimodal van Genuchten (1980) style soil water retention curves to better represent soil water characteristics of a soil with a heterogeneous pore structure. This allows macropores and desiccation cracking to be included with relative ease into the soil water retention characteristics for a material (Durner,
1994). Equation 2.25 shows the formulation for multimodal soil water retention characteristic curves (Durner, 1994).

Equation 2.25
\[ \Theta = S_e = \sum_{i=1}^{k} w_i \left( \frac{1}{1 + (\alpha_i \psi)^{n_i}} \right)^{m_i} \]

Where;
- \( k \) = number of unimodal retention curves being combined;
- \( w_i \) = weighting factor for each unimodal curve (\( 0 < w_i < 1 \) and \( \sum w_i = 1 \)).

In a similar way, the relative hydraulic conductivity can be estimated as follows;

Equation 2.26
\[ K_r (\psi) = \sum_{i=1}^{k} w_i k_{i, sat} \frac{1 - (\alpha_i \psi)^{n_i - 1} [1 + (\alpha_i \psi)^{n_i}]^{-m_i}^2}{[1 + (\alpha_i \psi)^{n_i}]^{m_i/2}} \]

Where;
- \( k_{i, sat} \) = is the saturated permeability of the unimodal curve i.

Studies have shown that hydraulic conductivity estimations are relatively good for course grained soils with small particle size distribution; often estimations for fine grained materials with large particle size distribution are not so representative (Durner, 1994). Along with consideration of multimodal soil water retention curves, an additional characteristic that should be included where possible is wetting and drying hysteresis. This hysteresis is shown schematically in Figure 2.24.
Wetting and drying curves describe the material behaviour more accurately capturing the movement of water within unsaturated soil (Likos, et al., 2014; Bordoni, et al., 2015). Hysteresis explains the lag between physical behaviour change behind the driver of the change, in this instance the soil water content. Hysteresis of soil water retention curves is required to adequately represent physical behaviour, the mechanical response of soil to wetting and drying, the volumetric change and changes in shear strength. If this phenomenon is omitted then predicted mechanisms and behaviour will not match observed behaviour (Likos, et al., 2014).

In addition to multi-modal soil water retention curves and hysteresis, recently it has been observed that the relationship between suctions and soil water content is not static. It is known that cycles of wetting and drying cause volume change, irreversible strains and the breaking of diagenetic bonds. This will change the internal structure of a soil matrix, especially on a micro scale. Changes in micro and macro pores within a soil will change the magnitude of suctions that can develop as water is removed and deterioration of the soil water retention properties will occur. This is another deterioration mechanism that will affect long-term slope stability.
2.5 Current State-of-the-Art Numerical Modelling

The previous sections present the physical mechanisms attributed to clay cut slope failure, the inter-related nature of these behaviours and presents the basis of soil mechanics and how numerical models are formulated along with key governing equations for the modelling of physical behaviour of interest. The literature discussed in this section considers the current state-of-the-art numerical models, the conclusions that have been obtained from their use and the limitations.

It is important to recall, long-term failure of clay cut slopes is due to numerous inter-related deterioration processes (Terzaghi, 1950; Rouainia, et al., 2009; Hughes, et al., 2009; Kilsby, et al., 2009). Numerical models have therefore been broken down into groups of models considering different mechanisms and also depending on the numerical modelling framework used for the modelling.

2.5.1 Simplified Cut Slope Behaviour (Saturated Framework)

The models discussed within this section consider, stress relief, post-excavation pore water pressure equilibration and progressive failure with constant uniform boundary conditions. It is these, static boundary conditions which render the behaviour modelled as ‘simplified’.

Potts, et al., (1997) presented numerical modelling of delayed failures in overconsolidated clay cut slopes considering deep-seated failure. The model was undertaken in the Imperial College Finite Element Program (ICFEP) with coupled consolidation and included a local Mohr-Coulomb strain-softening constitutive model with material softening allowing progressive failure to be modelled. The model results showed that progressive failure triggered by swelling due to post-excavation pore water pressure equilibration and stress relief could be captured and the failure surfaces predicted resembled failures observed in reality. The dissipation of post-excavation pore water pressures and progressive failure behaviour observed in the numerical analyses are shown in Figure 2.25.
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Figure 2.25 Numerical results of post-excavation pore water pressure dissipation and progressive failure; a) deviatoric plastic strain 9 years after excavation; b) deviatoric plastic strain 14.5 years after excavation; c) average pore water pressure ratio against time; d) mid-slope horizontal displacement against time (after, Potts, et al., 1997)

The failure surfaces produced from the numerical models showed the development of horizontal basal softened zones that propagated different amounts into the slope depending on the overconsolidation ratio employed in the analyses (Potts, et al., 1997), these results can be seen in Figure 2.3. This basal shear zone was observed within the Selborne cutting experiment by Cooper, et al., (1998). Although it was concluded that the value of overconsolidation ratio ($K_0$) had little effect on the probability of slope failure, it does have significant implications on the time taken for failure to occur and the location of the failure surface obtained by Potts, et al., (1997).

Developing on the work presented by Potts, et al., (1997), Ellis and O’Brien (2007) conducted a study looking at the effects of geometry on deep-seated failures in brittle clay cut slopes. The work was conducted in FLAC (Fast Lagrangian Analysis of Continua) and included consolidation and a local Mohr-Coulomb strain-softening model which considered plastic displacements normalised to element thickness. Ellis and O’Brien (2007) included post-peak strength to account for initial rapid softening, showing the significance of understanding the softening behaviour of a material and the implications it can
have on a numerical model. It was shown that progressive failure is less significant for lower height cut slopes (lower than 5m) and that deep-seated failures within low height cut slopes could be an unrealistic failure mechanism and shallow failures are of greater concern (Ellis & O'Brien, 2007).

The work presented by Potts, et al., (1997) and Ellis and O'Brien (2007) showed the importance of understanding complex inter-related failure mechanisms and highlighted the slopes that are at risk of these deep-seated failure mechanisms. However, the significance of seasonal wetting and drying cycles were omitted from the models and the conclusions regarding the depth of shear surfaces (i.e. deep-seated or shallow) are debatable due to our improved understanding of mesh dependency when local strain-softening models are employed.

Summersgill, et al., (2017) showed the benefits of using a nonlocal strain-softening regulatory model when considering simplified clay cut slope behaviour (omitting seasonal wetting and drying). Although nonlocal strain-softening is not entirely mesh-independent, it was shown that displacements observed and time to failures are in much closer agreement between different meshes than when a local strain-softening model is employed (Summersgill, et al., 2017). For the meshes considered, the weighting function proposed by Galavi and Schweiger (2010) was shown to be the least mesh dependent for simplified cut slope behaviour (Summersgill, 2015). The difference in mid-slope displacements and times to failure for six different mesh configurations using local and nonlocal strain-softening models is shown in Figure 2.26.
Finally, Summersgill, *et al.*, (2017) showed that the internal length parameter used in nonlocal strain-softening affected the rate of strain-softening of the material: the greater the internal length the slower the softening.
2.5.2 Weather-Driven Behaviour (Saturated Framework)

The following models consider slope behaviour with the inclusion of transient boundary conditions. However, there is the assumption that the soil does not become unsaturated and behaviour can be modelled in a saturated framework. This assumption is made on the premise that certain soils can withstand very high suctions prior to desaturation. Therefore, the magnitude of shrink-swell cycles will be predicted through consolidation.

“As London Clay can withstand very high values of suction before desaturation occurs (air-entry values of suction as high as 1MPa have been reported in the literature, e.g. Croney and Coleman 1954), the effect of desaturation has not been modelled here.” (Tsiampousi, et al., 2016)

However, field monitoring of soil water content by Smethurst, et al., (2012) showed the near-surface of a London Clay cut slope desaturating at significantly lower suctions. The assumption that the soil can be assumes fully saturated is highly debatable and saturated theory cannot explain volume and strength change due to the removal of water and addition of air. Whilst the ability of the numerical models to replicate physical behaviour correctly is questionable, the use of seasonal boundary conditions within a saturated framework does allow cycles of stresses due to seasonal wetting and drying on a slope to be considered and therefore findings should not be ignored.

Early work considering the numerical modelling of seasonal cycles of wetting and drying was conducted in a diagnostic manner, Kovacevic, et al., (2001) showed that cycles of effective stress could lead to strain accumulation and progressive failure. The work was conducted in ICFEP looking at an embankment. Shrinkage and swelling were modelled as a consolidation process (i.e. unsaturated behaviour was not included) and seasonal wetting and drying were simulated through the application of 5, 15, and 20kPa suction boundary conditions for summer and 0kPa boundary conditions for winter. Following application of the boundary conditions, the model was allowed to reach full equilibrium prior to the application of the next boundary pore water pressure conditions. There were several limitations in the work presented but
the main outcome, that cycles of effective stress can lead to progressive failure was a significant step forward.

Using the same premise of seasonal pore water pressure boundary conditions, Nyambayo, et al., (2004) developed on the work presented by Kovacevic, et al., (2001) to consider 25kPa and 10kPa suction boundary conditions for summer and winter respectively that were solved for 6 months each. Nyambayo, et al., (2004) investigated the effects of saturated hydraulic conductivity on behaviour, showing the displacement trends differ considerably depending on the rate at which water flows within the model. Three uniform hydraulic conductivity profiles of $1 \times 10^{-7}$ m/s, $1 \times 10^{-8}$ m/s and $1 \times 10^{-9}$ m/s were considered. Figure 2.27 shows the accumulation of displacement with seasonal cycles of pore water pressures and the significance saturated hydraulic conductivity has on the rate of failure of a slope with the $1 \times 10^{-9}$ m/s slope remaining stable more than 100 cycles after the $1 \times 10^{-8}$ m/s slope.

![Figure 2.27 Influence of hydraulic conductivity on horizontal movement due to seasonal boundary conditions (after, Nyambayo, et al., 2004)](image)

O’Brien et al., (2004) also used seasonal summer and winter boundary conditions to drive cycles of effective stress and did this for grass covered slopes and for a slope with a high-water demand tree – showing the effect that vegetation has on behaviour. O’Brien et al., (2004) are very open about the limitation of their work recognising that a more detailed soil-vegetation
interaction model is needed for the modelling of transient water supply and demand.

A more recent study looking at vegetation management and the effects on cut slope performance with time, considering monthly boundary conditions by Tsiampousi, et al., (2016), showed the difference in high and low water demand vegetation and the importance of understanding vegetation management strategies. Again, this work was conducted in a saturated framework.

2.5.3 Weather-Driven Behaviour (Unsaturated Framework)

To fully understand the implication of weather on slope stability it is necessary to model behaviour using real weather data and an unsaturated numerical framework. The work within this area can be separated into multiple areas;

1. weather-driven movement of reactive landslides;
2. weather triggered landslides;
3. weather-driven deterioration (and mobilisation of post-peak strength) of high-plasticity clay slopes.

The current state-of-the-art modelling approach for weather-driven deterioration behaviour in overconsolidated clay, considers unsaturated soil mechanics and employs two finite difference software packages to couple hydrological and geotechnical behaviour - SHETRAN and FLAC-TP (two-phase flow) respectively (Davies, et al., 2008a; Davies, et al., 2008b). Unsaturated behaviour is modelled in FLAC-TP considering Bishop’s generalised effective stress coupled with a van Genuchten (1980) style soil water retention curve to allow unsaturated flow (relative hydraulic conductivity) and pore water pressures suctions due to desaturation to be obtained. Surface pore water pressures are obtained on a daily basis from a SHETRAN model, which is run using hourly weather data and applied to a matching finite difference grid in FLAC-TP, the model is solved for one day and the process repeated (Davies, et al., 2008a; Davies, et al., 2008b). Within FLAC-TP Davies, et al., (2008a; 2008b) used the inbuilt Mohr-Coulomb strain-softening constitutive model to allow material softening due to weather-driven behaviour.
to be modelled for London Clay. The method described is computationally intensive and complex, this means there are limitations in the number of simulations that can be run. This approach was used by Rouainia, et al., (2009) to investigate the effects of different climatic conditions on a diagnostic embankment. It was shown that the rate of strength deterioration of the slope was greater under present climate than future climate. In the future climate scenario rainfall intensity was greater but for a shorter time period with greater run-off occurring resulting in lower infiltration and increased suctions maintained. The mid-slope horizontal displacements for the different analyses (i.e. present and future climate) are shown in Figure 2.28.

![Figure 2.28 Mid-slope horizontal displacement for future and present climate scenarios (after, Rouainia, et al., 2009)](image)

The conclusions drawn by Rouainia, et al., (2009) have been contradicted by more recent work done by the group presented in a paper by Elia, et al., (2017); within this study the same diagnostic slope was considered, and deterioration
was far greater under future climate than current, this can be seen in Figure 2.29.

![Figure 2.29 Pore water pressure and mid-slope horizontal displacement for future and present climate scenarios (after, Elia, et al., 2017)](image)

It should be noted, the work does not consider the effect of significant material characteristics, such as variable saturated hydraulic conductivity with depth and provides little validation that the model captures physical behaviour correctly.

Elia, et al., (2017) presented an overview of numerous numerical modelling approaches considering slope-vegetation-atmosphere interactions. The work considered a range of different approaches from limit equilibrium methods to detailed finite element and finite difference approached. The majority of work considered looked at the modelling of infiltration processes causing triggering of landslides and an appreciation of the importance of using real weather data to drive behaviour along with the need to develop better ways to understand future climate scenarios that can be included within these complex models.

Numerical models considering unsaturated behaviour have been shown to be able to replicate observed pore water pressure and therefore stress cycles in slopes. This study presented by Conte, et al., (2016) considered a reactive
Given the landslide is reactive, the shear surface and strength along the failure surface are relatively well-established and therefore, the movements predicted from numerical analyses are in good agreement with those observed in reality (Conte, et al., 2016). This is shown in Figure 2.30, it can be seen that it is possible to model the correct stress cycles within a slope due to real weather boundary conditions and that displacements in the right order of magnitude can be predicted.

Figure 2.30 Fosso San Martino landslide; a) cross-section; b) pore water pressures; c) displacements (after, Conte, et al., 2016)

### 2.5.4 Modelling Climate Change

Environmental resilience of long-term assets is becoming a key focus in policy making, maintenance strategies and asset management. Currently, there is sufficient understanding and information available to consider climate change implications on physical systems. However, there is no unified methodology for the inclusion of climate change projections into physical models (Brown & Wilby, 2012). Relating outputs of Global Climate Models (GCMs) to local impact assessments is extremely difficult due to the difference in scales of the two (Auandhi, et al., 2011). This section addresses some of the current methods available for climate change adaptation strategies.

One of the most commonly adopted methodologies for climate change impact studies, due to its ease of use, is the change factor (CF) method. In this method local climate data series are factored in line with the relative change calculated.
by GCMs (Auandhi, et al., 2011). This method requires a base period which will be used to initialise the GCMs and to which the change factors are associated with. This is the method adopted by UCKP09 but the change factors are available on a probabilistic basis. Commonly, the base period of weather data used for these studies is 1961 to 1990 as it can be assumed that the weather data within this period is relatively stable and comprehensive weather data is available.

The majority of work done considering climate change is carried out considering flooding; this is because the data available from projections facilitate easy adaptation of storm events that can be factored, and the consequences considered with multiple simulations of the physical process being carried out quickly. This allows analysis to be probabilistic making full use of UKCP09 climate change projections.

It has been proposed by Brown and Wilby (2012) that vulnerability analysis in the form of scenario-based ‘stress testing’ is a practical approach when considering climate change information for resource management. Three steps are outlined for the approach (Brown & Wilby, 2012);

1. Identification of physical system / problem, define objectives and key performance indicators;
2. Use scenario based ‘stress testing’ to understand and identify hazards; using a considerable range of variables non-climatic and climatic, evaluate performance;
3. Evaluate the risk considering climate change information and model projections.

This ‘stress testing’ approach has been further developed in the form of the Statistical Down-Scaling Model – Decision Centric (SDSM-DC) (Wilby, et al., 2014); as a tool SDSM-DC is a weather generator capable of creating daily data series guided by, but not explicitly based on, Global Climate Model (GCM) outputs. It is known that there are large uncertainties associated with GCMs these can be attributed to (Jenkins, et al., 2009);

- The natural variability of climate;
• Lack of knowledge and understanding regarding climate system;
• Uncertainty in future emissions.

Down-scaling of GCMs to site and regional scales has been previously employed but there is an appreciation that the level of uncertainty ‘cascades’ during this process making decision making extremely difficult (Wilby, et al., 2014). Therefore, a movement away from direct dependency on GCMs for scenario-led analysis to scenario-neutral approaches has become more common; in this case sensitivity analysis is effectively used to vary weather data series and the impact on the model of the physical system is observed (Wilby, et al., 2014).

SDSM-DC methodology allows the user to manipulate observed climate time series to represent plausible future time series. These manipulations are based on a range of factors and functions defined by the user. Changes can be made to the occurrence, variance mean and trend of the climate variable in question (Wilby, et al., 2014). The factors and functions considered should be based on GCM outputs, expert knowledge and plausible scenarios, but be broad enough to ‘stress test’ the system sufficiently (Wilby, et al., 2014).

An application of SDSM-DC ‘stress testing’ is presented by Wilby, et al., (2014) considering flood risk management; the following steps were outlined (Wilby, et al., 2014);

1. Adaptation scenario is outlined;
2. Calibration of model to represent physical system and assess impact;
3. Construct weather data series inputs for impact model using SDSM-DC methodology;
4. Sensitivity surface is plotted relative to the climate variable change used in the analysis;
5. Map climate change model projections onto sensitivity surface plots to understand likelihood of climate change projection causing model performance to exceed acceptable level.

One of the aspects highlighted as requiring further development is the ability to create data series capable of allowing multi-seasonal processes to be
modelled, also there is a need for guidelines regarding the range of climate
that weather generators should be used for within the sensitivity analysis
‘stress testing’ framework (Wilby, et al., 2014). It should be noted that the
model application example was for a flood risk analysis for a 1 in 100 year
flood making the climate data series required fairly short and the climate
change variables are limited to occurrence of rainfall event (wet-days) and
mean rainfall.

Down-scaling of climate data can be carried out on any site where
observations of climatic variables has been undertaken historically and the
variables can be considered as either unconditional (i.e. air temperature; wet-
day occurrence) or conditional (i.e. precipitation volume) (Wilby, et al., 2014).

Currently, there is no definitive method for modelling long-term inter-seasonal
climate change data series especially when considering multiple variables.
This is a problem when considering slope stability due to reliance on
antecedent conditions and the process of deterioration due to weather driven
stress cycles.

2.6 Gaps in Knowledge
From the literature review carried out, the following research gaps have been
identified and are summarised within this section.

Slope stability is being treated as a transient problem; there is an appreciation
that understanding of behaviour through modelling can only be improved if
realistic climatic boundary conditions are used (Davies, et al., 2008a; Bordoni,
et al., 2015; Conte, et al., 2016; Elia, et al., 2017). Numerical models capable
of modelling weather-driven behaviour must represent both saturated and
unsaturated behaviour. Current ability to model unsaturated behaviour is
improving but is by no means perfect; coupling hydrological behaviour and
geotechnical behaviour requires approximations and simplifications. Capturing
the hydrological properties of soil for use in these models is complex. In
instances where macro pores, preferential flows and desiccation cracking exist,
there are few available data sets. There are also questions regarding the
validity of the Richard’s equation to capture unsaturated flow adequately in
such instances (Beven & Germann, 2013). Further understanding of unsaturated behaviour is required. Although this is a critical area of research, the work presented here does not look to improve current understanding of unsaturated behaviour but to use the currently accepted models to focus on modelling weather-driven slope behaviour in order to understand strength deterioration and first-time failure of slopes due to mobilisation of post-peak strength.

Slope stability deterioration is commonly associated mostly with strength reduction, in particular when considering progressive failure and implementation of strain-softening constitutive models. When considering weather-driven deterioration there is another dimension that should be considered; with time and cycles of wetting and drying the matrix structure of near-surface material changes. Soil water content variation causes volume change and breaks inter-particle bonds; chemical weathering and vegetation growth introduces roots creating preferential flow routes. All these processes change the saturated hydraulic conductivity with time and will also change the soil water retention properties of the near-surface zone. With time the effects of this hydrological deterioration will penetrate deeper and will thus have a cascading effect on strength deterioration. The inter-related nature of these behaviours means that the influence of such deterioration must be considered alongside strength deterioration.

Modelling a complex physical system, such as a slope, is computationally intensive. Numerous approaches have been used to understand the effects of weather on slope stability as a transient problem. The most advanced work considering deterioration of high-plasticity overconsolidated clay slopes due to cycles of weather is that of Davies, et al., (2008a). The approach involves two independent finite difference codes used to couple hydrological and geotechnical behaviour. This process is very time-consuming and difficult, limiting the number of models that can be run so probabilistic analysis of future climate projections is not possible. As previously discussed, numerical models and the current state of knowledge mean it is not possible to capture complete deterioration; however, coupled hydrological-geotechnical modelling can give
an indication of relative rates of deterioration between different materials and slope geometries and displacements for reactive landslides driven by weather cycles have shown very promising results (Conte, et al., 2016). The modelling approach presented by Davies, et al., (2008a) is currently the most advanced available for modelling weather-driven deterioration of high-plasticity overconsolidated clay slopes. However, a less computationally intensive approach is required with the anticipation that hydrological deterioration will be included when there is data available.

Infrastructure earthworks are an ageing asset and in many instances relatively little is known about the slopes. Targeting of maintenance strategies is difficult with such limited information. An approach is needed to assess the current condition of a slope and identify high risk sections on a network scale.

There are very few quantitative investigations of the implications of climate change on infrastructure slope stability, especially explicitly considering cut slopes. Of those quantitative investigations carried out, analysis is done considering worst-case future climate projection. This only considers one climate scenario vs another, making the analysis scenario-based; climate change projections for the UK (UKCP09) are designed to be considered in a probabilistic manner. Valuable information can be gained from a probabilistic climate change impact assessment for a slope. Currently, no definitive methods are available for modelling long-term inter-seasonal climate change data series, especially when considering multiple variables. This is a problem when considering slope stability due to reliance on antecedent conditions and the process of deterioration due to cycles of weather. Regardless of the implications of climate change on slope stability, in the future there will be a change in the slopes that are most at risk of instability, and hence an increase in the frequency and nature of failures (Manning, et al., 2008; Loveridge, et al., 2010; Dijkstra & Dixon, 2010). Failure to carry out further research using our current knowledge and climate change projections regarding slope stability and anticipated negative implications of climate change will result in considerable financial risk to the economy (Dijkstra & Dixon, 2010).
2.7 Chapter Summary

This Chapter presents the findings of a literature review conducted to summarise key mechanisms and inter-related processes attributed to first-time failures in high-plasticity overconsolidated clay cut slopes. It is clear that seasonal wetting and drying stress cycles can cause accumulation of plastic strain and mobilisation of post-peak strength driving shallow first-time failures in these slope assets. The mechanism is currently not well understood and there is significant uncertainty regarding the implications of climate change on this mechanism.

Understanding of the inter-related deterioration mechanisms identified, can only be improved through accelerated physical modelling or numerical simulations. Currently, there are limitations in the numerical modelling approaches used to model key failure mechanisms (i.e. not including unsaturated behaviour and a lack of validation of numerical models against experimental and field data).
Chapter 3

Numerical Model Development and Methodology

3.1 Scope

From the literature presented in Chapter 2, it is clear that long-term failure of cut slopes in high-plasticity overconsolidated clay cannot be explained by isolated mechanisms. It is therefore imperative that work looking to develop understanding of the full life-cycle of these earthwork assets must consider the effects of inter-related mechanisms and in particular the effects of weather on the soil-water-atmosphere relationship and corresponding slope behaviour.

Infrastructure asset managers face difficult decisions regarding where to invest in maintenance over thousands of kilometres of linear infrastructure earthwork assets across the UK. Greater understanding of earthwork deterioration and the potential implications of climate change on deterioration must be developed. The time-frame for deterioration within these assets is decades to centuries; further investigation into this problem can therefore only be made considering accelerated physical modelling or numerical modelling. Whilst physical modelling provides conclusive evidence of mechanisms, the number of models that can be considered within the time-scale of a PhD project is very small in comparison to that of numerical models. For this reason, the work presented here focuses purely on numerical modelling.

This Chapter presents an overview of the approach adopted in developing, validating and stress testing numerical models to capture and understand time-dependent strength deterioration of high-plasticity overconsolidated clay cut slopes. Following this, the choice of software package used is presented and
generic methodology used throughout the work, with detailed methodology (i.e. material properties, model geometries and mesh configurations) linked to specific modelling approaches presented within relevant Chapters in line with the modelling phases outlined in the Section 3.2. The focus of the work is the validation of numerical modelling approaches developed to improve confidence in the model’s ability to capture relevant and realistic behaviour. Following validation of modelling approaches, parametric analyses and forecasting behaviour, including climate change, have been explored. Regardless of the level of validation of the model, there are still limitations in the methods adopted and these are clearly highlighted throughout and further discussed within the context of the results presented.

3.2 Overview of Approach

To date, there is no single model capable of capturing all mechanisms associated with overconsolidated clay cut slope deterioration as identified and discussed within the literature review presented in Chapter 2. Significant developments in the modelling of weather-driven behaviour have been made by Davies (2011) in which a hydrological and mechanical model have been coupled to capture behaviour and material softening has been included through the use of a strain-softening constitutive model. However, this method does not correctly capture the time-dependency of undrained unloading due to excavation. The work presented by Davies (2011) also omitted significant material characteristics, such as variable saturated hydraulic conductivity with depth and provided little validation of the model capturing physical behaviour correctly. As discussed previously, the method developed by Davies (2011) is very computationally intensive and restrictive due to the coupling of the two different software codes.

Weather-driven behaviour is of paramount significance within this work, in line with Objective 2, a numerical modelling approach capable of capturing weather-driven behaviour (i.e. seasonal ratcheting – outward and downward movements due to wetting and drying stress cycles) is required. To facilitate this, a numerical modelling approach has been validated against known seasonal stress cycles and displacements obtained from physical modelling
conducted by Take (2003) and presented by Take and Bolton (2011). The numerical modelling approach developed has then been used to investigate the mechanism of seasonal ratchetting to understand rates of strength deterioration due to different material softening behaviour and slope geometries.

In line with Objective 3, an approach has been developed for the long-term modelling of high-plasticity clay cut slopes in which both excavation processes (i.e. undrained unloading, dissipation of excess pore water pressures, stress relief and progressive failure) and weather-driven behaviour are included within a single boundary-value problem model. The modelling approach designed to include all these mechanisms has been developed incrementally to ensure all behaviours are captured sufficiently. Initially, simplified cut slope behaviour (i.e. undrained unloading, dissipation of excess pore water pressures, stress relief and progressive failure under constant boundary conditions) in a saturated framework is considered and finally a real slope with extensive monitoring has been considered in an unsaturated numerical modelling framework with real weather daily boundary conditions.

Deterioration within this work is primarily focused on strength reduction due to strain-softening leading to progressive failure. Modelling strain-softening behaviour is complex; the size and configuration of elements within a mesh influence both the time to failure and failure surface obtained within a localisation problem. For this reason, a local strain-softening model and nonlocal strain-softening regulatory model have been implemented, validated and compared for different numerical analyses in this study. Correctly capturing strength deterioration within a pragmatic framework is extremely important in the justification of decisions made during model development.

The model development undertaken within this project, to address the deterioration mechanisms explored within the literature review, has been presented schematically in Figure 3.1. Work has been separated into different phases to allow clear referencing throughout the methodology, results and discussion sections of this document. The different model development phases are linked to the Objectives of this work and are summarised below;
Chapter 3 – Numerical Model Development and Methodology

**Phase 1: Modelling Simplified Cut Slope Behaviour**

Phase 1 looks at developing a model capable of capturing simplified cut slope behaviour – undrained unloading, stress relief, pore water pressure equilibration and progressive failure – in a saturated framework with constant boundary conditions. The behaviour of models has been compared and validated against results of pioneering numerical analysis presented by Potts., *et al.*, (1997). Using the numerical model developed for simplified cut slope behaviour, local and nonlocal strain-softening models have been considered and mesh dependency explored.

**Phase 2: Modelling Seasonal Ratcheting: A Validation Against Physical Modelling and Geometric Study**

Phase 2 addresses Objective 2 regarding weather-driven strength deterioration and can be considered in two sections. Firstly, phase 2 considers the development and validation of a numerical modelling approach capable of capturing stress cycles due to wetting and drying leading to seasonal ratcheting displacements, mobilisation of post-peak strength and progressive failure. An unsaturated framework has been used to model behaviour and the model has been validated against physical modelling conducted by Take and Bolton (2011) considering Speswhite Kaolin Clay slopes.

Secondly, using the validated model the effect different strain-softening relationships for Kaolin and slope geometries on the behaviour of seasonal ratcheting has been investigated considering mobilised strength at serviceability failure and serviceable design life. Within this work, a method to assess slope deterioration and obtain the point of serviceability failure for a slope experiencing strength reduction due to seasonal ratcheting is presented.

**Phase 3: Modelling Cut Slope Behaviour with Seasonal Boundary Conditions**

Phase 3 combines the mechanisms of the two previous phases into a single numerical model in line with Objective 3 (i.e. a model for the analyses of inter-related time-dependent strength deterioration mechanisms). Modelling is undertaken in a two-phase flow numerical model allowing saturated and
unsaturated behaviour to be captured in a single model. This allows stress history of a cut slope including undrained unloading, stress relief and pore water pressure equilibration to be captured alongside unsaturated weather-driven behaviour within the near-surface of a slope leading to progressive failure. To allow this, initial conditions are set, and excavation carried out within a saturated framework (i.e. including Biot consolidation theory) and post-exavcation conditions are transferred to a two-phase flow model to allow application of seasonal boundary conditions. In this phase, weather has been modelled considering simple summer (7 months) and winter (5 months) boundary conditions, timings are indicative of seasonal behaviour in the UK. The modelling undertaken considers idealised London Clay cut slopes. In this phase hydrogeological deterioration and the potential implications of climate change considering general trends highlighted in UKCP09 have been investigated.

**Phase 4: Modelling Long-Term Behaviour of a Real Cut Slope: A Validation against Field Monitoring and Climate Change Stress Testing**

Phase 4 is a further development of the work in phase 3 (and therefore Objective 3) and includes real daily weather boundary conditions calculated using a soil water balance approach, applied and solved on a daily basis. The results of the models have been validated against a case study cut slope in London Clay. The case study slope has been summarised in Table 3.1.

<table>
<thead>
<tr>
<th>Name</th>
<th>Soil</th>
<th>Constructed</th>
<th>Time Monitored</th>
<th>Data</th>
<th>Slope Height</th>
<th>Slope Angle</th>
</tr>
</thead>
</table>

Source: (Smethurst, et al., 2012)

Following this, the numerical modelling approach used and validated against the case study slope has been subjected to a number of different climate change scenarios, obtained using statistical down-scaling methodology to stress test these assets and assess the implications of climate change, as highlighted in Objective 4.
To allow the model development shown in Figure 3.1, a numerical modelling software package is required that is capable of modelling both saturated and unsaturated behaviour, has the flexibility to implement a nonlocal strain-softening model and to add relationships for material properties to allow...
parameters to be updated while-stepping. FLAC 7 (Fast Lagrangian Analysis of Continua) by Itasca Consulting Group, Inc. (2011) has been chosen for this project. There are a number of reasons for this selection;

- FLAC is a commercially available software package with a customer support facility;
- Both saturated (single-phase flow) and unsaturated (two-phase flow) numerical frameworks are available within FLAC;
- FLAC has an inbuilt strain-softening constitutive model;
- FISH in FLAC – FISH is an inbuilt coding language that allows users access to material properties and current stress / strain conditions to implement material relationships and gives the user flexibility in defining additional parameters or conditions;

A summary of FLAC and the governing equations is presented in Appendix A.

### 3.3 Implementation of Strain-Softening Models

Post-peak strength reduction is a fundamental behaviour exhibited by high-plasticity overconsolidated clays. As discussed in Section 2.2.1, strain accumulation, softening and thus progressive failure is a prominent mechanism in the long-term stability of cut slopes. Strain-softening behaviour must therefore be incorporated into any numerical model developed to consider time-dependent mechanisms leading to cut slope failure.

There are two approaches that can be adopted for modelling strain-softening behaviour, material and structural softening (Galavi, 2007). This work considers material softening (intrinsic softening), in which strength reduction occurs as a function of the material properties due to changing stress conditions (as presented in Appendix A for FLAC).

Significant work has been conducted considering strain-softening constitutive models (see Section 2.4.5). It is known that local strain-softening models exhibit mesh dependency, altering the time to failure and failure surface
geometry, due to the shape, size and orientation of elements used within a mesh (Galavi & Schweiger, 2010; Summersgill, et al., 2017). Several regularisation techniques have been developed to reduce mesh dependency when strain-softening models are implemented.

This section describes how a local plastic displacement softening model and a nonlocal strain-softening regularisation model have been implemented within FLAC. Both strain-softening models adopt the Mohr-Coulomb strain-softening constitutive model with material softening and non-associated shear flow rule in FLAC. The local model considers plastic displacement criteria standardised against element thickness similar to the work by Ellis and O’Brien (2007). The nonlocal regulatory model softens as a function of the nonlocal plastic strains calculated by averaging local plastic strains using a weighting function (Galavi & Schweiger, 2010). These strain-softening models are used throughout later work and the material properties used are presented within the relevant method sections.

### 3.3.1 Implementation of Local Strain-Softening Model

Local strain-softening models consider plastic strains at calculation points with respect to softening criteria, the displacements are considered in isolation of adjacent nodal displacements (Summersgill, 2015). Within FLAC, there is an inbuilt local strain-softening constitutive model, the model is a material softening model in which cohesion, friction angle, angle of dilation and tensile strength of the material can either harden or soften as a result of plastic strain (Itasca Consulting Group, Inc., 2011). To determine yielding Mohr-Coulomb failure criteria is used, as shown in Appendix A, and plastic strains are calculated as shown in Equation 3.1:

\[
\Delta e_{ps}^{ps} = \left\{ \frac{1}{2} \left( \Delta e_{1}^{ps} - \Delta e_{m}^{ps} \right)^{2} + \frac{1}{2} \left( \Delta e_{m}^{ps} \right)^{2} + \frac{1}{2} \left( \Delta e_{3}^{ps} - \Delta e_{m}^{ps} \right)^{2} \right\}^{1/2}
\]

Where;
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- \( \Delta e_{m}^{ps} = \frac{1}{3} (\Delta e_{1}^{ps} + \Delta e_{3}^{ps}) \); \( \Delta e_{1}^{ps} \) and \( \Delta e_{3}^{ps} \) = principal plastic shear strain increments.

Material strength reduction within the models used in this work can follow any softening profile allowing different softening relationships to be compared. A simplistic softening profile from peak to residual strength for the local strain-softening model is shown in Figure 3.2.

![Figure 3.2 Material softening profile – local plastic strain](image)

There are two approaches for the allocation of strain criteria for the local softening models; these are percentage plastic strain-softening criteria (Potts, et al., 1997; Kovacevic, et al., 2001; Nyambayo, et al., 2004) and plastic displacement softening criteria (O'Brien, et al., 2004; Ellis & O'Brien, 2007; Summersgill, 2015).

Percentage plastic strain-softening criteria are commonly calibrated against the mesh used but suffer heavily from mesh dependency. When considering a problem with variable element size, the plastic displacement required to cause 5% plastic strain for a smaller element is less than that of the displacement needed for a much larger element. For the plastic displacement softening criteria, however, the displacement at which softening occurs is the same for each element, but the level of plastic strain stipulated for each element differs. Displacement criteria are changed into plastic strains for each element as a function of element or reference length.

In this work plastic displacement softening criteria have been adopted and normalised against element thickness (Ellis & O'Brien, 2007) in an attempt to
reduce mesh dependency within this approach. Equation 3.2 shows how plastic strain criteria are calculated to be input within FLAC.

\[
\text{Equation 3.2} \quad \text{plastic strain criteria} = \frac{\text{plastic displacement criteria}}{\text{element thickness}}
\]

### 3.3.2 Implementation of Nonlocal Strain-Softening Regulatory Model

Strain-softening regulatory techniques have been shown to minimise mesh dependency within localisation problems (Galavi & Schweiger, 2010; Summersgill, et al., 2017) as discussed in Section 2.4.5.

In this work a partial nonlocal strain-softening model has been implemented. Local plastic strains across a mesh are averaged relative to each stress point using a weighting function to obtain the nonlocal plastic strain. The nonlocal plastic strain calculated then dictates material softening within the model. The implementation procedure adopted here is similar to that described by Galavi and Schweiger, (2010). However, a Mohr-Coulomb strain-softening constitutive model with material softening criteria has been employed rather than a multilaminate constitutive model. Nonlocal plastic strain (\(\varepsilon^*\)) at stress point \((x_n)\) is calculated using Equation 3.3.

\[
\text{Equation 3.3} \quad \varepsilon^*(x_n) = \frac{1}{V_w} \iiint \omega'(x_n') \varepsilon_d(x_n + x_n') dx_1' dx_2' dx_3'
\]

Where;

- \(V_w = \iiint \omega(x_n') dx_1' dx_2' dx_3'\) = weighted volume;
- \(\omega'(x_n') = \) is the weighting function relating local strains of nearby elements to the calculation point to determine the nonlocal strain at that calculation point;
- \(\varepsilon_d(x_n + x_n') = \) local plastic strain at different calculation points;
- \(x_n = \) global coordinate;
- \(x_n' = \) local coordinate.

The weighting function adopted in this work takes the form of the distribution proposed by Galavi and Schweiger (2010) and is given in Equation 3.4 and
represented visually in Figure 3.3. Other weighting functions have been proposed for calculating nonlocal plastic strains. However, Summersgill (2015) showed that for a numerical model considering a cut slope the weighting function proposed by Galavi and Schweiger (2010) produces the least mesh dependent results. The centre of the weighting function is located at the stress point at which the nonlocal plastic strain is being calculated.

Equation 3.4

\[ \omega'(r) = \frac{r}{l} e^{-\left(\frac{r}{l}\right)^2} \]

Where:

- \( l \) = internal or defined length;
- \( r \) = distance from the calculation (stress) point to adjacent calculation points.

Figure 3.3 Generic weighting function for nonlocal strain-softening model (after, Galavi & Schweiger, 2010)

Equation 3.3 can be simplified to allow easy implementation within existing finite difference or finite element codes, the simplification presented here has been adapted to consider local plastic strains rather than damage strains as initially used by Schadlich (2012).

Equation 3.5

\[ \varepsilon_p^* = \frac{1}{V_w} \sum_{m=1}^{m_{sp}} \omega'_m \cdot \varepsilon_{p,m} \cdot V_m \]

Where:

- \( V_w = \sum_{m=1}^{m_{sp}} \omega'_m \cdot V_m \) = weighted volume;
- \( m_{sp} \) = number of stress points in area of integration;
- \( \omega'_m \) = is the weighting function;
- $\varepsilon_{p,m} =$ plastic strain at stress point;
- $V_m =$ volume of influence of stress point.

Within FLAC, local plastic strains are calculated at the centre of each element using Equation 3.1. The centre of any element is therefore the calculation / stress point at which nonlocal plastic strains are obtained by averaging local plastic strains. It is therefore necessary to establish the centroid of all elements so that the centre of the weighting function for each stress point can be located and the distance from the stress point to all other stress points within the mesh can be determined. In addition to this it is necessary to calculate the volume of influence of each stress point and the element volume. Following this, all terms within Equation 3.5 are determined and the nonlocal plastic strain can be obtained. A generic element is presented in Figure 3.4 and the general form to calculate the centroid and volume are shown.

![Figure 3.4 Generic element centroid and volume](image)

Nonlocal plastic strains are calculated at each time-step in FLAC with stress points in a range of three times the internal length included in the averaging process. This area of integration was shown to be sufficient due to the rapid reduction in weighting function with distance from stress point by Summersgill (2015). The material softening follows the same profile to that of the local stain-softening models but is associated with the nonlocal plastic strain at the stress point rather than the local plastic strain, see Figure 3.5.
As the nonlocal strain-softening framework already considers the element size and has the additional length parameter, the internal length, it is not necessary to use plastic displacement softening criteria. For this reason, material softening criteria for nonlocal strain-softening models are given as absolute nonlocal plastic strains.

### 3.4 Calibrating Strain-Softening Model Parameters

When implementing strain-softening models, it is essential that realistic plastic strain (or displacement) criteria are established for the soil being modelled. This behaviour, the stress-strain curve of the material, can be obtained from simple shear and triaxial test experiments and numerical models replicating the experiments can be used to calibrate the required strain criteria to ensure mechanical behaviour of the modelled soil is representative. This method was presented by Potts, et al., (1997), where a single element numerical model of a simple shear test and undrained triaxial tests were conducted to calibrate strain-softening criteria against experimental data.

The approaches presented in Sections 3.4.1 and 3.4.2, can be used to obtain local strain-softening criteria (Potts, et al., 1997). However, as single element numerical models are used it is not possible to average strains as required in the nonlocal framework. It is not possible to match local and nonlocal strain-softening properties exactly as, within this work, the local approach considers plastic displacement criteria and the nonlocal model considers absolute nonlocal plastic strains.
Local plastic displacement softening criteria are obtained through the calibration of single element numerical models against experimental data, as described in Sections 3.4.1 and 3.4.2. These displacement criteria have also been considered as the absolute nonlocal plastic strains for the nonlocal strain-softening models; i.e. if local plastic displacement for peak strength is 10mm then the absolute plastic strain for the nonlocal model is 0.01. The internal length parameter is then used to dictate the rate of softening in models using nonlocal strain-softening, this relationship was presented by Summersgill (2015). The effect of different internal length parameters has been investigated to assess the rate of softening as a function of the internal length, see Section 4.2.2.

3.4.1 Simple Shear Numerical Model

Where simple shear test data is available for the soil of interest a single element numerical model can be used to replicate the stress-strain curve obtained for the material and the strain-softening criteria calibrated. The numerical modelling approach depends on the soil being tested and the rate of shearing applied during the test as this will determine if the test is drained or undrained.

The boundary conditions of the experiment are replicated within the single element numerical model by fixing the base of the element vertically and horizontally and fixing the top of the element vertically. A compressive force matching that applied within the experiment is applied to the element and a steady horizontal velocity is applied to the top of the element to emulate loading during the test. The model is shown schematically in Figure 3.6.

When the test is drained, volume change due to drainage is allowed (i.e. soil remains drained, no excess pore water pressures are generated) and the rate of shearing (i.e. applied velocity in Figure 3.6) is slow. Conversely, when the test is undrained, no volume change is allowed to occur due to drainage and excess pore water pressures during shearing can develop.

Stress-strain data from the numerical analyses can be obtained and the results compared against experimental data. Where the numerical model does not
match the physical behaviour, adjustments can be made to the plastic strain and strength criteria until behaviour is representative.

![Figure 3.6 Simple shear model](image)

**3.4.2 Triaxial Test Numerical Model**

As with the simple shear test, when triaxial test data is available for the soil of interest, a single element numerical model can be used to calibrate strain-softening behaviour. Again, the modelling approach is dependent on whether the test is drained or undrained.

A triaxial test is an axisymmetric problem. Therefore, it is only necessary to model a quadrant of the cross-section of a sample. A single element is used for this model; the base of the element and one side are on roller boundaries.

For a drained triaxial test, initially the element is fully saturated, and the confining pressure set. The horizontal confining pressure is fixed throughout the analysis, and a steady vertical velocity applied to the top of the element to emulate loading in a strain-controlled test. Volume change due to drainage is permitted (i.e. soil remains drained, no excess pore water pressures are generated) throughout the analysis.

For an undrained triaxial test, the element is fully saturated, confining pressures (total stress = initial pore pressure + effective confining stress, \( \sigma_3' \)) is stipulated along with the pore water pressures at the start of the test. Again, the horizontal confining pressure is fixed, and a steady vertical velocity applied. Volume change due to drainage is not permitted (i.e. flow is turned off).

During analysis, shear stress, axial strain and pore water pressures (for undrained case only) are monitored, so that a comparison can be made with
experimental results allowing the strain-softening criteria to be calibrated. A schematic of the single element numerical model is shown in Figure 3.7.

![Figure 3.7 Single element axisymmetric triaxial test of a single quadrant](image)

### 3.5 Slope Model Mesh Generation and Configuration

As discussed in Sections 2.4.5 and 3.3, mesh dependency is a significant problem when modelling localisation problems. Therefore, local and nonlocal strain-softening models implemented within FLAC are to be investigated to assess the significance of mesh dependency. This requires multiple mesh configurations to be generated. While generating meshes, effort has been made to ensure elements are as uniform as possible especially in critical areas. Meshes are based on approximately uniform 1.0x1.0m and 0.5x0.5m elements. Model extents for the crest length, toe length and depth of the model are all based on the geometries presented by Ellis and O’Brien (2007).

![Figure 3.8 Slope numerical model boundary lengths (after, Ellis & O’Brien, 2007)](image)

Where multiple models are considered for a specific problem, the models are all run with the same mesh configuration to ensure any difference is due to
parameter or geometry variation and not a function of different meshes used. Meshes have been given a reference letter that will be used throughout this work and have been presented in Figure 3.9; Meshes A to D have elements of approximately 1.0x1.0m and Meshes E to H 0.5x0.5m. The elements excavated to form cut slopes are not shown in Figure 3.9.

![Mesh A and Mesh E](image1)
![Mesh B and Mesh F](image2)
![Mesh C and Mesh G](image3)
![Mesh D and Mesh H](image4)

Figure 3.9 Typical slope mesh configurations used and references

### 3.6 Chapter Summary
Within this Chapter, an overview of the numerical modelling undertaken has been presented along with the choice of software package to be used. Generic methodology used throughout the work has been presented with detailed methodology of specific modelling phases presented within the relevant Chapters. The modelling being undertaken has been structured around the aims and objectives of this research project and gaps in knowledge identified in Chapter 2.
Chapter 4

Modelling Simplified Cut Slope Behaviour

4.1 Chapter Scope

This Chapter presents the methodology and results of work looking at the modelling of simplified cut slope behaviour, considering undrained unloading, pore water pressure equilibration (see Section 2.2.2), stress relief (see Section 2.2.3) and progressive failure (see Section 2.2.1) under constant boundary conditions. The numerical model approach developed replicates the work of Potts, et al., (1997), summarised in Section 2.5.1, and the results obtained have been compared. Following the validation of the modelling approach used, mesh dependency, and the effect of local and nonlocal strain-softening models have been considered, similar to the work presented by Summersgill, et al., (2017) which is summarised in Section 2.5.1. The analyses within this section all consider London Clay cut slopes and modelling has been conducted within a saturated framework. For clarity, Figure 4.1 shows model development and outputs of modelling for this phase of analysis.

Figure 4.1 Phase 1 model development and outputs
4.2 Methodology (Phase 1) – Simplified Cut Slope Behaviour

As stated with Section 3.2, within this work simplified cut slope behaviour incorporates undrained unloading, stress relief, pore water pressure equilibration and progressive failure under constant fixed boundary conditions. Potts, et al., (1997) presented pioneering work capturing these mechanisms within a numerical model, developing understanding of delayed failure of cut slopes in London Clay. A model within FLAC has been developed to replicate the behaviour and results presented by Potts, et al., (1997). This has been done to validate the behaviour being modelling in FLAC, and to validate the implementation of a nonlocal strain-softening model and consider mesh dependency of the problem.

This section presents the method adopted in the analyses along with the material properties used, the results of the analyses conducted are presented within Sections 4.3 and 4.4.

4.2.1 Replicating Results of Potts, et al., (1997)

To replicate the work by Potts, et al., (1997), modelling has been performed in FLAC single-phase flow. In this framework time-dependent consolidation due to stress change is modelled using Biot consolidation theory and the soil is assumed to be either fully saturated or unsaturated (partial saturation is not included). The details of the soil-water interaction and mathematical formulation within FLAC are summarised in Appendix A.

In line with the work presented by Potts, et al., (1997), a local strain-softening constitutive model has been calibrated considering a single element numerical model for a simple shear test (see Section 3.4.1) and an undrained triaxial tests (see Section 3.4.2). This strain-softening model is then employed within the slope models being compared against Potts, et al., (1997) work. The parameters used are summarised in Table 4.2 alongside the values used by Potts, et al., (1997). The local stain-softening model employed within FLAC considers plastic displacement softening criteria normalised relative to the element thickness as suggested by Ellis and O’Brien (2007) (see Section 3.3.1).
The modelling procedure is summarised in the following steps;

1. Mesh generated – including material to be excavated (models use Mesh B (see Section 3.5) for the validation against Potts, et al., (1997));
2. Boundary conditions set (10kPa suction along initial ground profile), base fixed in both horizontal and vertical direction, sides of model fixed in horizontal only. All boundaries, except for the surface, are modelled as impermeable (see Figure 4.2);
3. Material properties applied to mesh (see Section 4.2.1.2);
4. Pore water pressures are initiated, vertical stresses calculated considering gravity (9.81m/s$^2$), material density and pore water pressure and horizontal stresses initiated considering coefficient of earth pressure at rest ($k_0$) value;
5. Model is solved to ensure that it is equilibrium and initial conditions are correct;
6. Excavation is carried out at the same rate as Potts, et al., (1997), 1m increments, 10m in 3 months;
7. On completion of excavation boundary pore water pressure condition of 10kPa suction are applied along the crest, slope surface and toe of the model (see Table 4.1);
8. Models are run until mid-slope horizontal displacement exceeds 1.5m, it is accepted that failure will have occurred prior to this point;
9. During running multiple history points are monitored to allow comparison with the results presented by Potts, et al., (1997), and to allow simple independent checks to ensure the correct behaviour is being observed.
Three models have been considered in the comparison of the model developed here and the work of Potts, et al., (1997). For convenience the models have retained the references adopted by Potts, et al., (1997). The analyses considered are summarised in Table 4.1.

Table 4.1 Summary of comparison models (after, Potts, et al., 1997)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Height (m)</th>
<th>Length (m)</th>
<th>Angle (º)</th>
<th>$k_0$</th>
<th>Surface Suction (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>10</td>
<td>30</td>
<td>≈18.5</td>
<td>1.0</td>
<td>10</td>
</tr>
<tr>
<td>S3</td>
<td>10</td>
<td>30</td>
<td>≈18.5</td>
<td>1.5</td>
<td>10</td>
</tr>
<tr>
<td>S5</td>
<td>10</td>
<td>30</td>
<td>≈18.5</td>
<td>2.0</td>
<td>10</td>
</tr>
</tbody>
</table>

4.2.1.2 Material Properties

This section presents the material properties used to model simplified London Clay cut slope behaviour and how they have been incorporated into the model developed within FLAC. As mentioned previously, plastic displacement softening criteria normalised against element thickness has been used within the models considered in FLAC. Results of the models used to calibrate the plastic displacement criteria from single element triaxial and simple shear models against experimental data presented in Potts, et al., (1997) work can be seen in Section 4.3.1. The strain-softening properties are summarised in Table 4.2.
Within this work, stiffness refers to the bulk and shear moduli that are calculated considering material relationships or obtained through consideration of the Young’s modulus of the material. The stiffness relationship, considering Young’s modulus, adopted within Potts, et al., (1997) modelling of delayed failure of cut slopes in London Clay has been commonly cited as shown in Equation 4.1 where $\sigma’$ is in kPa;

\[
E = 25(\sigma' + 100) \text{ kPa}
\]

However, on closer inspection of the work, it is stated that;

“Young’s modulus, $E$, varies with mean effective stress, $p'$, but not with shear stress level. No distinction is made between unloading and loading. Consolidation and swelling follow approximately the same modulus in stiff clays therefore the Young’s modulus used in the present analysis is based on the appropriate swelling modulus.” (Potts, et al., 1997)

It is not entirely clear how a swelling modulus is included within the modelling conducted by Potts, et al., (1997). However, there is no explicit way to include this behaviour within FLAC unless the stiffness relationship is adjusted to the relevant swelling profile depending on the stress history of the soil. Potts, et al., (1997) present different swelling profiles for London Clay dependent on the coefficient of earth pressure at rest, these have been derived considering measured swelling behaviour from laboratory oedometer tests (Apted, 1977)
and field measurements (Skempton & Henkel, 1957). The swelling profiles can be seen in Figure 4.3.

From the swelling behaviour presented in Figure 4.3, the stiffness relationship for different coefficient of earth pressures at rest can be obtained. It is given that Young’s modulus, $E$ can be calculated as:

$$E = \frac{\Delta \sigma}{\Delta \varepsilon}$$

It is also given that the Young’s modulus relationship within the analysis is of the form:

$$E = \lambda (\sigma' + 100) ; \text{min} 4000kPa$$

Using Figure 4.3 and Equation 4.2, stiffness’s for different ranges of effective stresses can be obtained and through knowledge of the stiffness relationship being used in the numerical analysis, Equation 4.3, a value of $\lambda$ that accounts for the swelling behaviour can be found for different overconsolidation ratios (i.e. different coefficient of earth pressures at rest). The stiffness multipliers ($\lambda$) and effective stress ranges determined from Figure 4.3 are summarised in Table 4.3.
Table 4.3 Summary of fitted stiffness multipliers

<table>
<thead>
<tr>
<th>Coefficient of earth pressure at rest, $k_0$</th>
<th>Effective Stress Range, $p'$ (kPa)</th>
<th>Stiffness Multiplier, $\lambda$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0 to 70</td>
<td>11.0</td>
</tr>
<tr>
<td></td>
<td>&gt;70</td>
<td>30.0</td>
</tr>
<tr>
<td>1.5</td>
<td>0 to 115</td>
<td>11.5</td>
</tr>
<tr>
<td></td>
<td>&gt;115</td>
<td>25.0</td>
</tr>
<tr>
<td>2.0</td>
<td>0 to 100</td>
<td>14.0</td>
</tr>
<tr>
<td></td>
<td>&gt;100</td>
<td>11.0</td>
</tr>
</tbody>
</table>

Given the Young’s Modulus relationship presented in Equation 4.3 and stiffness multiplier values presented in Table 4.3, the bulk and shear moduli can be obtained while-stepping using Equation 4.4 and Equation 4.5 respectively.

Equation 4.4 \[ K = \frac{E}{3(1 - 2v')} \]

Equation 4.5 \[ G = \frac{E}{2(1 + v')} \]

Where:
- $E$ = Young’s modulus (kPa);
- $K$ = bulk modulus (kPa);
- $G$ = shear modulus (kPa);
- $v'$ = Poisson’s ratio (0.2 for London Clay after, Potts, et al., (1997)).

Finally, hydrogeological properties (i.e. saturated hydraulic conductivity) can be considered. Figure 4.4 shows the depth dependent saturated hydraulic conductivity profile used within the analyses taken from the work by Potts, et al., (1997).

![Figure 4.4 Depth dependent saturated hydraulic conductivity profile (after, Potts, et al., 1997)](image-url)
4.2.1.3 Comparison of Numerical Analyses

To facilitate a comparison of the results obtained from the modelling approach developed within FLAC and the results presented by Potts, et al., (1997), and to achieve the outputs presented in Figure 4.1, a number of history points within the numerical models have been monitored throughout the analysis of the slope models.

Firstly, the stress changes 1.0m below the toe of the slope have been monitored and can be compared to a quick calculation of change in total stress following excavation. As the model developed is initially capturing undrained unloading, the change in pore water pressures after excavation will be equal to the change in total stress. With time the excess pore water pressures will dissipate (Vaughan & Walbancke, 1973; Chandler & Skempton, 1974). By monitoring, pore water pressure, total vertical stress and effective vertical stress at a single point, the behaviour of undrained unloading and pore water pressure dissipation can be clearly illustrated.

In addition, direct comparison of model outputs against the results presented by Potts, et al., (1997) have been considered. For the three slope models being analysed, the following comparisons have been undertaken;

- horizontal mid-slope displacement against time;
- and time to failure.

The differences in the model results have been quantified and discussed in Section 4.3.

4.2.2 Local and Nonlocal Strain-Softening Model Comparison

To investigate mesh dependency when implementing local and nonlocal stain-softening models, slope model S3 (see Table 4.1) has been considered for eight different mesh configurations (see Figure 3.9). The results of the analyses considering local and nonlocal strain-softening models have been compared in Section 4.4.

The slope geometry and all material properties with the exception of the saturated hydraulic conductivity are as presented in Section 4.2.1. To ensure
that the mesh dependency observed within these analyses is a function of only
the mesh and strain-softening models, a uniform saturated hydraulic
conductivity has been used within the slope analyses, $5 \times 10^{-10}$ m/s.

Mid-slope horizontal displacement and time to failure have been used to
facilitate a comparison of the strain-softening models. Local strain-softening
displacement criteria are summarised in Table 4.2 and nonlocal plastic strain
criteria for softening of London Clay are summarised in Table 4.4.

<table>
<thead>
<tr>
<th>Nonlocal Plastic Strain</th>
<th>0.012</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Plastic Strain</td>
<td></td>
</tr>
<tr>
<td>Residual Plastic Strain</td>
<td>0.055</td>
</tr>
</tbody>
</table>

Four different internal length parameters have been considered for the
nonlocal strain-softening model. This has been done to explore the influence
of the internal length and check conclusions regarding rate of softening and
internal length presented by others (i.e. Summersgill, et al., (2017)). The
different internal lengths considered are 0.5m, 1.0m, 1.5m and 2.0m. Results
of this comparison can be seen in Section 4.4.2.1.

4.2.3 Key Reasons for Modelling and Model Requirements
The significance of this modelling phase is to show that construction effects
due to excavation and undrained unloading can be captured within the
numerical modelling method developed. This is important as it is essential for
the modelling of realistic stress conditions and time dependency of critical
physical behaviours within cut slopes in low saturated hydraulic conductivity
soils.

The model developed must capture undrained unloading, stress relief,
dissipation of excess pore water pressure and progressive failure. To allow
stress relief to be investigated models with different initial stress conditions
prior to excavation are considered. Progressive failure is being modelled
through the use of local and nonlocal strain-softening models and the effect of
mesh dependency investigated – reducing mesh dependency is vital to
increase the repeatability of this work for other researchers in the future and to
obtain a robust method for modelling progressive failure moving forward within this research.

4.3 Validation of model against Potts, et al., (1997)

Since the publishing of the pioneering work by Potts, et al., (1997), there have been advances in software and changes in governing equations. It should be noted that there are inherent differences between finite element formulation, in which Potts, et al., (1997) modelled and the finite difference formulation used here. For these reasons, it is not anticipated that the results of the models compared are going to be the same, but it is important that the mechanisms being considered in the numerical analyses are captured correctly. The methodology, material properties used and approach for comparison of the models is presented in Section 4.2.1.

4.3.1 Calibration of Local Plastic Displacement Softening Criteria

At this stage of model development, a local strain-softening model has been implemented considering plastic displacement criteria normalised against element thickness in the same way as Ellis and O’Brien (2007). The displacement criteria require calibration against the strain-softening model used by Potts, et al., (1997). This was done through a comparison of single element numerical models of a simple shear test and triaxial test, the method adopted for these can be seen in Section 3.4 and the results, compared to Potts, et al., (1997), are presented in Figure 4.5 and Figure 4.6. The results of a single element simple shear test numerical model under a normal compressive force of 137kPa can be seen in Figure 4.5.
Sandroni (1977) presented two unconsolidated undrained triaxial tests with pore water pressure measurements for London Clay under different confining pressure, 112kPa and 137kPa, which have been adopted by Potts, et al., (1997) in order to validate the strain-softening model. Again, the results of the calibrated plastic displacement model within FLAC have been plotted against the results of Potts, et al., (1997) and the experimental data, see Figure 4.6.
The plastic displacement criteria normalised against element thickness used within the models compared with the strain-softening model presented by Potts, *et al.*, (1997) can be seen to be in good agreement. The plastic displacement criteria adopted are summarised in Table 4.5.

**Table 4.5** Calibrated strain-softening displacement thresholds for London Clay, normalised by element thickness

<table>
<thead>
<tr>
<th></th>
<th>12.0 mm</th>
<th>55.0 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Plastic Displacement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Residual Plastic Displacement</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.5 and Figure 4.6 show that the calibrated strain-softening model in FLAC appears to be marginally conservative when considering the triaxial test response as fully softened strength is reached at an axial strain of approximately 0.9% less than the relationship presented by Potts, *et al.*, (1997) for the 137kPa scenario. Given the closeness of the fit for the triaxial test model,
the plastic displacement criteria presented in Table 4.5 are deemed to be acceptable for modelling strain-softening behaviour of London Clay in line with the relationship presented by Potts, et al., (1997).

In both Potts, et al., (1997) and the FLAC model, pore water pressures generated by the models during loading of the triaxial model do not match the pore water pressures observed experimentally. This is due to the limitations of modelling the test as a single element. However, the pore water pressure results of the FLAC model and Potts, et al., (1997) work are alike. The significance of this calibration is the capturing of realist mechanical behaviour through obtaining strength and plastic strain criteria to model strain-softening representatively. The fact that the model underestimates pore water pressures generated during shearing in the numerical model has little implication on the mechanical model calibrated for the modelling of London Clay – especially given the work is a comparison against Potts, et al., (1997) work and the results of the numerical models of the triaxial tests match well.

4.3.2 Capturing Undrained Unloading / Stress Changes with Time

One of the most important mechanisms known to cause delayed failure of cut slopes in high-plasticity overconsolidated clay with low hydraulic conductivity is the dissipation of post-excision excess pore water pressure following undrained unloading (see Section 2.2.2) (Vaughan & Walbancke, 1973; Chandler & Skempton, 1974). To demonstrate the mechanism is being sufficiently captured within the model approach developed, pore water pressure, total vertical stress and effective vertical stress have been monitored at a point 1m below the toe for slope model S3. The results of the stress changes with time are shown in Figure 4.7.
Figure 4.7 Stress changes with time – 1m below toe of slope (analysis S3); a) 0 to 17 years; b) 0 to 0.5 years

Figure 4.7 shows that the total stress change, due to excavation, is initially carried by the pore water pressures and then with time the pore water pressures dissipate and the effective stress, and therefore strength of the soil in the slope reduces. In the formation of this cut slope, 10m of material has been excavated above the toe of the slope, assuming the upper 1m is unsaturated (10kPa suction boundary conditions), then the change in total stress due to excavation is; $\Delta\sigma_{yy} = (9.0 \times 18.8) + (1.0 \times 15.8) = 185\, kPa$. The changes in total stress and pore water pressure at the end of excavation at the monitored point 1m below the toe of the slope are summarised in Table 4.6.

Table 4.6 Summary of total stress and pore water pressure change after excavation – 1m below toe of slope (analysis S3)

<table>
<thead>
<tr>
<th>$\Delta$Total Stress (kPa)</th>
<th>$\Delta$Pore Water Pressure (kPa)</th>
<th>Difference (kPa)</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td>176.0</td>
<td>171.8</td>
<td>4.2</td>
<td>2.4%</td>
</tr>
</tbody>
</table>
The total stress change observed in the model is slightly less than that of the rough calculation (9kPa – 5% difference), however the values are relatively close. In addition, the differences in the changes in total stress and pore water pressure after excavation have a difference of only 2.4%. The difference can be accounted for by the fact that excavation of the cut slope is not modelled as instantaneous (see Section 4.2.1), and dissipation of pore water pressures can occur during excavation. The results show that the numerical model approach developed captures undrained unloading.

4.3.3 Comparison of Models against Potts, et al. (1997)

Models have been developed in FLAC to replicate the behaviour observed by Potts, *et al.* (1997) and seen in reality (Vaughan & Walbancke, 1973; Cooper, *et al.*, 1998). Using the method presented in Section 4.2.1, three scenarios, considering different coefficients of earth pressure at rest, have been analysed, and the results compared with the work presented by Potts, *et al.* (1997). The results for mid-slope horizontal displacement with time can be seen in Figure 4.8 and the time to failure for the different models have been summarised in Table 4.7.

![Figure 4.8 Mid-slope horizontal displacement for FLAC models compared against results presented by Potts, *et al.* (1997)](image-url)
Table 4.7 Summary of failure time for FLAC models compared against results presented by Potts, et al., (1997)

<table>
<thead>
<tr>
<th>Slope Model</th>
<th>Time to Failure (years)</th>
<th>Potts, et al., (1997)</th>
<th>FLAC</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>23.8</td>
<td>23.4</td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>14.5</td>
<td>16.2</td>
<td></td>
</tr>
<tr>
<td>S5</td>
<td>31.6</td>
<td>38.4</td>
<td></td>
</tr>
</tbody>
</table>

It can be seen from Figure 4.8 and Table 4.7 that the times to failure for the models developed in FLAC are within a few years of those presented by Potts, et al., (1997). To further support this, the trend in behaviour due to different initial stress conditions (i.e. coefficients of earth pressure at rest) observed also match the trends observed by Potts, et al., (1997).

Considering the inherent differences between finite difference (i.e. FLAC) and finite element (i.e. ICFEP – used by Potts, et al., (1997)) and given that the problem in question considers local strain-softening behaviour which is known to be mesh-dependent and the meshes used differ considerably, a difference of 1.7 years for a numerical analysis that failed in 14.5 years is not a significant difference. The difference in the meshes used in the analyses are shown in Figure 4.9. The difference in mesh configurations explains why trends in accumulation of displacement with time, shown in Figure 4.8, are so different between the two approaches.

![Comparison of meshes](image)

Figure 4.9 Comparison of meshes; a) mesh used by Potts, et al., (1997); b) mesh used in FLAC

The results presented are in acceptable agreement given the differences in the mesh and software used. The profiles of the displacements accumulated with time differ between the analyses, but this can again be attributed to the differences in mesh used within the analyses.
From the results presented, it has been shown that the numerical modelling approach developed within FLAC captures undrained unloading, dissipation of post excavation pore water pressures and progressive failure. In additions, trends in behaviour due to different initial conditions (coefficients of earth pressure at rest) have shown similar trends to the results presented by Potts, et al., (1997), with times to failure being comparable.

4.4 Investigating Mesh Dependency

The following section presents the results of modelling slope S3 with eight different meshes (see Section 3.5) to understand the mesh dependency of the problem, as discussed in Section 4.2.2. First, the implication of local plastic displacement softening criteria (see Section 3.3.1) on mesh dependency is investigated. Following this, the same models have been run considering a nonlocal strain-softening regulatory model (see Section 3.3.2) employing a weighting function proposed by Galavi and Schweiger (2010).

4.4.1 Local Plastic Displacement Softening Criteria

Considering the eight meshes presented in Figure 3.9, simplified cut slope behaviour has been modelled as described in Section 4.2 using local plastic displacement criteria normalised against element thickness (see Section 3.3.1). The results of mid-slope horizontal displacement with time and shear surfaces at failure for the different meshes are shown in Figure 4.10.
Figure 4.10 Local plastic displacement normalised against element thickness softening criteria results for different meshes; a) mid-slope horizontal displacement against time; b) shear surfaces at large strains

The results displayed in Figure 4.10 have been summarised in Table 4.8. From Figure 4.10 and Table 4.8, it can be see that the local plastic displacement softening model is highly mesh dependent, there is a range in failure times of 18.7 years and an average failure time of 14.4 years. Figure 4.10a shows different trends in accumulation of horizontal displacements and different mid-slope horizontal displacements for when failure occurs, with Mesh D failing at approximately 175mm and Mesh F failing at a horizontal displacement of greater than 400mm. The failure surfaces shown in Figure 4.10b also show a high variability in the size and depth of the failure mass observed for the different meshes. As shown by Summersgill, et al., (2017) (see Section 2.5.1), the results show that for the same problem, different mesh configurations can yield very different times to failure and failure surfaces when a local strain-softening model is implemented.
Table 4.8 Summary of local plastic displacement softening models for different meshes

<table>
<thead>
<tr>
<th></th>
<th>Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>14.4</td>
</tr>
<tr>
<td>Minimum</td>
<td>8.6</td>
</tr>
<tr>
<td>Maximum</td>
<td>27.3</td>
</tr>
<tr>
<td>Range</td>
<td>18.7</td>
</tr>
</tbody>
</table>

4.4.2 Nonlocal Strain-Softening Regulatory Model

Given the significant scatter in time to failure of the different meshes considering the same slope problem when local strain-softening is implemented and the current state of knowledge regarding regularisation techniques (see Section 2.4.5), a nonlocal strain-softening model, as described in Section 3.3.2, has been implemented. Again, the meshes presented in Figure 3.9, considering slope model S3 with a uniform saturated hydraulic conductivity have been modelled (see Section 4.2.2). The effects of varying internal length have also been investigated. Summersgill (2015) concluded that the internal length affected the rate of softening and the larger the internal length the greater the time to failure for cut slopes – this conclusion has been tested here.

Mid-slope horizontal displacements for the meshes have been plotted against time and shear surfaces at large strains presented. This has been done for the models with an internal length of 1.0m and can be seen in Figure 4.11.
Figure 4.11 Nonlocal strain-softening results for different meshes (internal length 1.0m); a) mid-slope horizontal displacement against time; b) shear surfaces at large strains

Table 4.9 Summary of nonlocal strain-softening models for different meshes

<table>
<thead>
<tr>
<th></th>
<th>Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>15.7</td>
</tr>
<tr>
<td>Minimum</td>
<td>11.6</td>
</tr>
<tr>
<td>Maximum</td>
<td>17.3</td>
</tr>
<tr>
<td>Range</td>
<td>5.7</td>
</tr>
</tbody>
</table>

Figure 4.11 and Table 4.9 show that the nonlocal strain-softening regulatory model implemented reduces mesh dependency significantly. For the eight meshes considered, the range in failure times has reduced from 18.7 years for the local models to 5.7 years for the nonlocal strain-softening models. For clarity, a comparison of the failure times for the local and nonlocal strain-softening models are shown in Figure 4.12. It can be seen the time to failure for the nonlocal strain-softening models are in much closer agreement and are therefore less mesh dependent.
The results presented also validate the successful implementation of the regulatory model within FLAC. Comparing Figure 4.10 and Figure 4.11, it can be seen that the variability in the size and depth of the failure mass is much less when the nonlocal strain-softening model is employed. The nonlocal strain-softening model is not mesh independent but does yield results that are considerably less mesh dependent than using a local strain-softening model. In instances where localisation within numerical analyses, due to strain-softening, is a problem, a nonlocal strain-softening model should be considered rather than a local model.

4.4.2.1 Effect of Internal Length Parameter

As mentioned previously, the conclusion drawn by Summersgill (2015) regarding the rate of softening being a function of the internal length parameter when using a nonlocal strain-softening regulatory model is to be tested. This has been done by considering the eight meshes presented in Figure 3.9 and the same model properties presented in Section 4.2.2 for four different internal length parameters; 0.5m, 1.0m, 1.5m and 2.0m. The results of the mid-slope horizontal displacement with time for these models are shown in Figure 4.13.
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Figure 4.13 Mid-slope horizontal displacement against time for different meshes using a nonlocal strain-softening model; a) internal length = 0.5m; b) internal length = 1.0m; c) internal length = 1.5m; d) internal length = 2.0m

The failure times for each slope model have been summarised in Figure 4.14 for easy interpretation of the results.
In line with the conclusion drawn by Summersgill (2015), it can be seen that as the internal length increases the average time to failure also increases. Therefore, the internal length used within an analysis has a bearing on the rate of softening of the material being modelled. There is a greater spread in the results when larger internal lengths are used. From Figure 4.14, it can be seen that the least mesh dependent model is when an internal length of 1.0m is used. A nonlocal strain-softening model has been implemented successfully and the findings further confirm the conclusions drawn by other researchers.

4.5 Chapter Summary
This Chapter has shown that key overconsolidated clay cut slope behaviours, undrained unloading, dissipation of excess pore water pressures, stress relief
and progressive failure can be captured within a modelling approach developed in a saturated framework including Biot’s consolidation theory. With calibration of strain-softening material properties, it has been possible to recreate behaviour, and trends in behaviour due to different initial stress conditions, presented by previous researchers. Providing confidence in the modelling approach developed.

The significance of mesh dependency on localisation problems employing strain-softening constitutive models has been confirmed. The most significant output within this Chapter is the successful implementation of a nonlocal strain-softening model and an appreciation of the benefits that using the regulatory technique has. Using a nonlocal strain-softening regulatory model gives much less mesh dependent results relative to the use of a local strain-softening modelling approach. This work has by no means been revolutionary but is a vital element on which following modelling decisions and development have been made.

Given the findings regarding mesh dependency, all further models include the nonlocal strain-softening regulatory model to reduce mesh dependency.
Chapter 5

Modelling Seasonal Ratcheting: A Validation Against Physical Modelling and Geometric Study

5.1 Chapter Scope

This Chapter presents the methodology and results of numerical modelling conducted to replicate weather-driven behaviour driving progressive failure observed in physical modelling conducted by Take (2003). The modelling approach developed, and material properties used are presented in Section 5.2.2. The modelling has been undertaken to validate the numerical modelling approach developed can replicate physical behaviour. Following the validation of the numerical analysis methodology against experimental data, the implication of continued uniform cyclic boundary conditions on different stiffness material is explored (see Section 5.6). Finally, a parametric study looking at the effect of strain-softening behaviour, slope angle and height has been undertaken. Phase 2 of the model development strategy presented in Section 3.2 is shown in Figure 5.1.

![Figure 5.1 Phase 2 model development and outputs](image)
5.2 Methodology (Phase 2a) – Validation Against Physical Modelling

Conclusive evidence of seasonal ratcheting driving progressive failure within expansive, high-plasticity clay slopes has been presented by Take and Bolton (2011) (see Section 2.2.4.1). Wetting and drying can drive stress cycles in the near-surface of high-plasticity clay slopes that cause cyclic volumetric change which can lead to the accumulation of irrecoverable deformation, strain-softening and progressive failure (Take & Bolton, 2004; 2011). It is therefore vital that numerical models developed to model weather-driven behaviour in high-plasticity clays capture this behaviour. This section presents the method developed to validate a coupled hydro-mechanical model against physical modelling results of seasonal ratcheting and progressive failure presented by Take and Bolton (2011).

Validating numerical models against physical models is a necessity in geotechnics. Physical models provide controlled conditions limiting the unknowns within the model and therefore the number of assumptions required for the development of a numerical model that can replicate the measured behaviour. In addition, the material used is uniform, the properties well-known, the stress history has been dictated and finally, the boundary conditions are well-established.

Firstly, a summary of the physical modelling undertaken by Take (2003) is presented followed by a description of the model developed and material properties used. Results of the numerical analyses have been validated against two different slope models. The hydrological response of the numerical model and boundary conditions applied, are compared with pore water pressure measurements obtained during the physical modelling using high-capacity tensiometers (Take & Bolton, 2003); and the mechanical response of the numerical model is compared with displacements measured during physical modelling using particle image velocimetry (White, et al., 2003). Results of the physical modelling have been provided by Professor Andy Take of Queen’s University, Ontario to allow this work to be undertaken. Two slopes have been considered within the validation of the numerical modelling to
illustrate the approach developed can replicate generic behaviour and not a single specific scenario.

Following validation, the numerical model developed has been used to investigate the effects of different strain-softening behaviours and the effects of slope height and angle allowing design recommendations regarding serviceable design life to be discussed. The scenarios considered have been summarised in Section 5.7.

### 5.2.1 Summary of Physical Modelling (after, Take, 2003)

Take (2003) conducted numerous centrifuge experiments considering the seasonal effects of soil moisture content and the corresponding stress cycles on the stability of high-plasticity clay slopes. The work here focuses on two of these models constructed in Speswhite Kaolin at 36° and 140mm high at 1/60th scale, corresponding to a slope height of 8.4m at full scale. The centrifuge modelling was run under centripetal acceleration of 60g. Speswhite Kaolin is a high-plasticity clay, the index properties of this soil are summarised in Table 5.1.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Liquid Limit</strong></td>
<td>69</td>
</tr>
<tr>
<td><strong>Plastic Limit</strong></td>
<td>38</td>
</tr>
<tr>
<td><strong>Plasticity Index</strong></td>
<td>31</td>
</tr>
</tbody>
</table>

(Take, 2003)

The difference between the slopes considered is the level of overconsolidation prior to forming of the slope and application of wetting and drying boundary conditions. One model was initially consolidated to 500kPa (ref – WAT7a) and the other 200kPa (ref – WAT8a). WAT7a was subjected to three complete cycles of wetting and drying to replicate seasonal influences, seven months of summer and five months of winter (at full scale), followed by prolonged wetting. WAT8a was subjected to seven seasonal wetting and drying cycles.

The slope models were formed through the following processes, after Take (2003). Speswhite Kaolin was mixed to 120% moisture content and consolidated within a consolidometer to form a one-dimensional consolidated block of material. The loading history within the consolidometer, incremental
increasing vertical stress was as follows; 2, 10, 15, 30, 60, 120, 250 and 500 kPa for WAT7a. WAT8a followed the same loading path but stopped at a maximum vertical load of 200kPa. Primary consolidation was allowed to occur prior to the increasing of load. Unloading was also staged, this was done in 75 kPa increments; 500, 425, 350, 275, 200, 125 and 60 kPa for WAT7a and the same for WAT8a but beginning at 200kPa. Prior to the final unloading stage, the Kaolin block was isolated from external water sources to limit elastic swelling due to the unloading, base drainage within the consolidometer was closed and free surface water removed. This resulted in the clay block having an initial suction of approximately 60 kPa throughout for WAT7a and 45kPa for WAT8a (Take, 2003).

Following consolidation and unloading of the Speswhite Kaolin block, the slopes were formed through cutting the consolidated block using a mould and a sharp aluminium cutting blade (Take, 2003). The slope model was then loaded into a climate chamber and onto the centrifuge and spun to equilibrium.

Figure 5.2 shows the different slope model geometries at 1/60th scale, the location of high-capacity tensiometers for measuring pore water pressures and the area of focus of different cameras to allow displacements to be obtained through particle image velocimetry.

Boundary conditions were controlled using a climate chamber developed by Take and Bolton (2002), in which rainfall was simulated using suspended misting nozzles and the relative humidity was fluctuated between 100% (winter) and approximately 40% without simulated rainfall (summer). The relative humidity boundary conditions applied are shown in Figure 5.3.
During the formation of WAT8a, part of the slope near the crest was damaged (see Figure 5.2) and became dislodged during the first seasonal cycle. However, it became reattached during drying and subsequently remained attached throughout the modelling (Take, 2003).

The physical modelling provides pore water pressure and displacement data for slopes subjected to repeated simplified uniform seasonal boundary conditions for a relatively homogeneous material. Although this is an idealised problem it is the only data set that the author is aware of where the mechanism of seasonal ratcheting and progressive failure is observed and both hydrological and mechanical data is available for known seasonal stress cycles. Results from the centrifuge experimentation are presented alongside results from a numerical model in Section 5.5.
5.2.2 Numerical Model Development and Validation

Using FLAC two-phase flow (see Appendix A), a numerical model capable of replicating physical behaviour due to known stress cycles driven by wetting and drying has been developed. The two-phase flow option has been utilised as the problem is partially saturated / unsaturated and therefore it is important that unsaturated soil behaviour is modelled.

The following sections present a summary of the numerical framework used for the modelling, the material properties and justifications for the decisions made in initialising the models and applying the boundary conditions.

It should be noted that the work presented within this phase of modelling does not look to advance current modelling capacities for soil-water-atmosphere interaction. The focus of the work is on the validation that the physical movements of seasonal ratcheting due to wetting and drying stress cycles, the accumulation of displacements and progressive failure (i.e. strain-softening) can be captured within a numerical model.

5.2.2.1 Numerical Model Framework

Seasonal ratcheting behaviour is driven by soil water content variation due to cycles of wetting and drying, making the problem unsaturated. As discussed in Section 2.4.4.3, to allow modelling of an unsaturated problem hydrogeological and mechanical behaviour must be coupled. Within the code used the equations governing behaviour are shown in detail in Appendix A. In summary, mechanical behaviour is described through a constitutive model linking Bishop’s generalised effective stress and strains with hydrogeological factors included through a relationship involving matrix suctions and degree of saturation (i.e. a soil water retention curve). This stress-strain framework is presented in Equation 5.1.

\[
\sigma' = (\sigma - u_a) + S_e s \\
s = u_a - u_w \\
\varepsilon = S_e
\]

Where:

- \( \sigma' = (\sigma - u_a) + S_e(u_a - u_w) \) = Bishop’s generalised effective stress;
- \( \sigma = \) total stress;
• $u_a =$ pore air pressure;
• $u_w =$ pore water pressure;
• $(\sigma - u_a) =$ net stress;
• $s = (u_a - u_w) =$ matrix suction;
• $S_e =$ effective saturation;
• $\varepsilon =$ strain.

Kaolin exhibits post-peak strength reduction and seasonal ratchetting can cause plastic strain accumulation and progressive failure (Take & Bolton, 2011). For this reason, a nonlocal strain-softening model using a Mohr-Coulombs constitutive model, as described in Section 3.3.2, has been implemented within the coupled hydro-mechanical framework to describe the mechanical behaviour of the soil.

### 5.2.2.2 Model Geometry and Scaling

As mentioned in Section 5.2.1, the physical models were constructed at 1/60th scale, see Figure 5.2, and run at 60g in the centrifuge experimentation. The numerical analyses replicating these slopes have been modelled as full scale and 1g. It is therefore necessary to scale the lengths and time of the physical model so that results from the physical and numerical models are comparable. The scaling factors from centrifuge 1/nth and ng to full scale and 1g are;

• time, $n^2$;
• length, $n$.

The full scale numerical model cross-section and the locations at which pore water pressures and displacements have been monitored during the numerical analyses for comparison with the physical modelling results are shown in Figure 5.4.
The mesh used in these analyses is Mesh E (see Section 3.5); this mesh has been adopted due to the increased run speed in comparison of the other meshes; along with its uniformity and the size of elements (0.5x0.5m) being reasonably small. The mesh is shown in Figure 5.5.

**5.2.2.3 Material Properties**

The following section presents the material properties used within the numerical analyses of Speswhite Kaolin.

**5.2.2.4 Strain-Softening Parameters**

The strength parameters and strain-softening criteria to model Speswhite Kaolin have been obtained from calibration of single element axisymmetric drained triaxial test numerical models against experimental results of overconsolidated drained triaxial tests performed by Cekerevac and Laloui (2004). This has been done using the method described in Section 3.4.2 to obtain local plastic displacement criteria and strength parameters.

Local plastic displacement softening criteria have, as described in Section 3.4, been taken as the absolute nonlocal plastic strain criteria. The nonlocal strain-
softening regulatory model has been implemented as described in Section 3.3.2. The results of the comparison between single element numerical model and triaxial test data are presented in Section 5.3 and a sensitivity analysis for mechanical behaviour where strain-softening is omitted is presented. A summary of the material strengths and strain-softening criteria can be seen in Table 5.2.

Table 5.2 Kaolin strain-softening parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Local Plastic Displacement (mm)</td>
<td>5</td>
</tr>
<tr>
<td>Critical State Local Plastic Displacement (mm)</td>
<td>15</td>
</tr>
<tr>
<td>Peak Nonlocal Strain</td>
<td>0.05</td>
</tr>
<tr>
<td>Critical State Nonlocal Strain</td>
<td>0.15</td>
</tr>
<tr>
<td>Internal Length Parameter (m)</td>
<td>0.5</td>
</tr>
<tr>
<td>Peak Cohesion (kPa)</td>
<td>6.25</td>
</tr>
<tr>
<td>Critical State Cohesion (kPa)</td>
<td>0.00</td>
</tr>
<tr>
<td>Peak Friction (°)</td>
<td>24.0</td>
</tr>
<tr>
<td>Critical State Friction (°)</td>
<td>24.0</td>
</tr>
<tr>
<td>Dilation Angle (°)</td>
<td>0.0</td>
</tr>
<tr>
<td>Unit Weight (kN/m³)</td>
<td>17.9</td>
</tr>
</tbody>
</table>

### 5.2.2.5 Stiffness

The stiffness relationships adopted for modelling Kaolin have been taken from past numerical modelling of centrifuge experimentation of a Kaolin embankment (Almeida, et al., 1986) in which the bulk and shear moduli are a function of the specific volume, mean effective stress and are updated at each time-step (Schofield & Wroth, 1968);

\[
K = \frac{v, \sigma'}{\kappa} \cdot \min(2000 \text{ kPa})
\]

\[
G = \frac{3(1 - 2v')}{2(1 + v')} \cdot K
\]

Where;

- \(K\) = bulk modulus;
- \(G\) = shear modulus;
- \(v\) = specific volume (\(v = 1 + e\));
- \(e\) = void ratio;
- \(\kappa\) = gradient of the swelling line;
• \( v' \) = Poisson’s ratio;
• \( \sigma' \) = mean effective stress.

The specific volume of a soil can be obtained from knowledge of the consolidation and swelling properties, the stress history, and the current stress state (Schofield & Wroth, 1968). The specific volume and thus stiffness of the soil will change due to stress cycles driven by wetting and drying. Wetting and drying effectively act as mechanical loading and unloading which can be considered in the \( v : \ln(\sigma') \) space in line with Equation 5.4 and Equation 5.5.

\[
\text{Equation 5.4} \quad v = v_\lambda - \lambda \cdot \ln \frac{\sigma'}{\sigma_{\text{ref}}} ; \text{ normal consolidation line}
\]

\[
\text{Equation 5.5} \quad v = v_\kappa - \kappa \cdot \ln \frac{\sigma'}{\sigma_{\text{ref}}} ; \text{ swelling lines}
\]

Where;

• \( \lambda \) = gradient of the virgin compression line;
• \( \kappa \) = gradient of the swelling line;
• \( \sigma' \) = current mean effective stress;
• \( \sigma_{\text{ref}} \) = reference pressure;
• \( v_\lambda \) = original specific volume at reference pressure;
• \( v_\kappa \) = specific volume at reference pressure following swelling line.

From work considering Kaolin the gradient of the consolidation line can be taken as \( \lambda = 0.25 \) and the swelling line can be taken as \( \kappa = 0.05 \) (Clegg, 1981; Take, 2003). For Kaolin consolidated to 500 kPa the void ratio is approximately \( e = 1.0 \) (Al-Tabbaa & Wood, 1987). Therefore, the specific volume at 500kPa has been taken as \( v = 2.00 \) and the corresponding specific volume for a reference pressure of 1kPa is \( v_\lambda = 3.55 \).

Considering the stress history of the Kaolin slopes in the physical model (consolidated to 200kPa and 500kPa, unloaded, slope formed, spun to equilibrium and subjected to wetting and drying cycles at 60g) the change in specific volume due to the cyclic seasonal boundary conditions imposed can be explained solely by the swelling line after unloading (i.e. stresses do not exceed initial consolidation pressure during modelling). Therefore, for the
different overconsolidation pressures; WAT7a, $v_{k,500} = 2.31$ and WAT8a, $v_{k,200} = 2.49$. The sequence of stresses described, and corresponding specific volume are presented in Figure 5.6.

The parameters used to obtain specific volume and therefore stiffness are summarised in Table 5.3.

![Figure 5.6 Specific volume relationships; a) WAT7a; b) WAT8a](image)

<table>
<thead>
<tr>
<th>Consolidation line Gradient, $\lambda$</th>
<th>0.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Swelling line Gradient, $k$</td>
<td>0.05</td>
</tr>
<tr>
<td>Poisson’s Ratio, $v'$</td>
<td>0.35</td>
</tr>
<tr>
<td>Reference Pressure, $\sigma_{ref}$ (kPa)</td>
<td>1.00</td>
</tr>
<tr>
<td>Specific Volume at Reference Pressure – Consolidation, $v_\lambda$</td>
<td>3.55</td>
</tr>
<tr>
<td>Specific Volume at Reference Pressure – Swelling WAT7a, $v_{k,500}$</td>
<td>2.31</td>
</tr>
<tr>
<td>Specific Volume at Reference Pressure – Swelling WAT8a, $v_{k,200}$</td>
<td>2.49</td>
</tr>
</tbody>
</table>

Contrary to the modelling of the seasonal stress cycles, the specific volume relationship for the single element axisymmetric triaxial test numerical models is solely dependent on the normal consolidation line (Equation 5.4) in which $v_\lambda = 3.55$ for a reference pressure of 1kPa.

Within numerical analyses, mean effective stress within each element is calculated at each time-step and depending on the relationship required (consolidation or swelling line) the specific volume of the element is calculated. With this information the bulk modulus is obtained as is the shear modulus using the equations presented.

A sensitivity analysis of the inclusion of time-stepping stiffness as a function of specific volume and stress state compared to a static stiffness modulus has
been presented in Section 5.3. This has been done to show the importance of this relationship for correctly capturing mechanical behaviour of Kaolin.

### 5.2.2.6 Hydrological Properties

As discussed in Section 2.4.4.3 and 2.4.6.4, soil water retention properties are fundamental to the modelling of unsaturated behaviour and establishing pore water pressure suction and relative hydraulic conductivity. Relative hydraulic conductivity is a function of the saturated hydraulic conductivity and the soil water retention properties. Within FLAC-TP, soil water retention properties take the form of a van Genuchten (1980) style soil water retention curve (see Appendix A).

For Kaolin, the saturated hydraulic conductivity has been presented as a function of the void ratio of the material (Al-Tabbaa & Wood, 1987). Thus, having a dependency on the stress history and current stress level. In the same way that specific volume has been calculated at each time-step within the model, the void ratio can also be obtained given that void ratio, \( e = v - 1 \). The saturated hydraulic conductivity can then be updated for each time-step. Vertical and horizontal saturated hydraulic conductivity have been given as a function of void ratio by Al-Tabbaa and Wood (1987);

\[
\text{Equation 5.6} \quad k_{\text{sat,vertical}} = 0.53e^{3.16} \times 10^{-9} \text{ m/s}
\]

\[
\text{Equation 5.7} \quad k_{\text{sat,horizontal}} = 1.49e^{2.03} \times 10^{-9} \text{ m/s}
\]

To complete the hydrological model, the soil water retention properties are required. There is an acceptance that there are more complex and representative soil water retention models that can be used that take into account the change in characteristics with mechanical loading and changing void ratio (Hu, et al., 2013). However, there is limited data for the use of such a model for Kaolin and limitations regarding its implementation within FLAC-TP. For these reasons a simple van Genuchten (1980) style soil water retention curve has been fitted against experimental data through visual inspection.

Experimental data from work conducted by Tarantino (2009), Hu, et al., (2013) and Tripathy, et al., (2014) have been presented alongside the soil water
retention curve fitted to the data, see Figure 5.7a, and the parameters used are summarised in Table 5.4. Figure 5.7b shows the effect of void ratio change on the relative hydraulic conductivity function for Kaolin.

![Figure 5.7 Kaolin soil water retention properties; a) soil water retention curve fitted against experimental data (experimental data after, Tarantino, 2009; Hu, et al., 2013; Tripathy, et al., 2014); b) vertical relative hydraulic conductivity function for different void ratios](image)

<table>
<thead>
<tr>
<th>Table 5.4 Fitted van Genuchten parameters for Kaolin</th>
</tr>
</thead>
<tbody>
<tr>
<td>van Genuchten fitting parameter (kPa)</td>
</tr>
<tr>
<td>van Genuchten fitting parameter</td>
</tr>
<tr>
<td>van Genuchten fitting parameter</td>
</tr>
</tbody>
</table>

### 5.2.2.7 Initial Conditions

To replicate the method undertaken to consolidate the block of Kaolin by Take (2003), the numerical models were initially saturated and subjected to one-dimensional consolidation. Incremental increasing compressive stress was applied in line with the loading history applied within the consolidometer (i.e. 2, 10, 15, 30, 60, 120, 250 and 500 kPa for WAT7a, and stopping at 200kPa for WAT8a). Within the physical models, primary consolidation was allowed to occur prior to the increasing of the load, within the numerical model complete pore water pressure equilibration has been reached between each loading increment. Unloading was also staged in the numerical model to match the physical modelling (i.e. 500, 425, 350, 275, 200, 125 and 60 kPa for WAT7a, and starting at 200kPa for WAT8a). Prior to the final unloading stage, the consolidated blocks were isolated from external water sources to limit elastic
swelling due to the unloading. This resulted in the clay block having an initial suction of approximately 60 kPa throughout for WAT7a and 45 kPa for WAT8a (Take, 2003). At this point, the consolidated Kaolin blocks were cut to the desired slope angle using a mould and cutting blade. This sequencing was replicated within the numerical models and the slope formed through the removal of elements to create the desired slope profile. For the physical modelling, shaping of the slope was conducted at 1g so the stress relief due to unconfinement will have been very small.

The slope models were then attached to the centrifuge arm and swung to equilibrium at 60g. At this point, horizontal stresses have been initiated within the numerical model by considering the overconsolidation ratio and determining a coefficient of earth pressure at rest ($k_0$). The overconsolidation ratio is determined using Equation 5.8 and the coefficient of earth pressure at rest for reconstituted clays has been shown to fit the empirical relationship given in Equation 5.9;

\[
OCR = \frac{\sigma_{v, maximum}'}{\sigma_{v, current}'}
\]

Equation 5.9

\[
K_0 = K_{ nc}(OCR)^{\phi'}
\]

Where;

- $K_{ nc} = 1 - \sin \phi'$ ($\phi'$ is in radians).

The overconsolidation ratio and coefficient of earth pressure at rest with depth prior to application of seasonal boundary conditions calculated using Equation 5.8 and Equation 5.9 and scaled from model to full scale for both WAT7a and WAT8a numerical analyses are shown in Figure 5.8.
Within any centrifuge experiment there is a variation in the centripetal acceleration across the depth of the model being considered due to the distance from the centroid of rotation (Wood, 2004). If the model height is less than 0.1 times the radius of the centrifuge arm then the variation in the centripetal acceleration can be assumed to be negligible (Wood, 2004). The sample used in the physical model is 260mm high (Take & Bolton, 2011) and the nominal radius of the centrifuge used, the Turner Beam at the University of Cambridge, is 4.125m (Wood, 2004) and therefore it has been assumed that the effect of this stress field variation is negligible.

5.2.2.8 Seasonal Boundary Conditions

From the physical modelling it is very difficult to ascertain the exact boundary conditions applied due to the imposed relative humidity and mist sprinkler system (see Figure 5.3). It can however be assumed that the boundary conditions are relatively uniform across the toe, slope surface and crest of the model. It has therefore been decided that three different types of boundary condition will be applied to slope model WAT7a to assess the different hydrogeological and mechanical behaviour modelled. The different boundary conditions considered are as follows:

- summer and winter discharge boundary conditions;
- summer and winter pore water pressure boundary conditions;
• summer discharge boundary condition and winter pore water pressure boundary conditions (combined boundary conditions).

The hydrological and mechanical behaviour obtained from the numerical analyses considering different boundary conditions have been compared against the physical modelling to establish the most representative boundary conditions for numerical analysis within the numerical framework being used. In the development of the model boundary conditions the magnitude of the summer and winter boundary conditions have been chosen by comparing the pore water pressure response of the numerical model against the physical modelling results at HCT1 (see Figure 5.4). This point was chosen as it is closest to the slope surface and as such the most significant pore water pressure record for the principal mechanism under investigation, seasonal ratcheting. It also ensures that the near-surface stress cycles within the numerical analyses are representative of the physical model.

Summer boundary conditions are applied, and the model solved for approximately seven months and winter boundary conditions solved for approximately five months, timings of boundary condition application have been matched to full scale timings of the physical modelling. The results of the different boundary conditions for WAT7a have been presented in Section 5.4.

Once the most representative boundary conditions have been obtained the results of numerical analyses of WAT7a and WAT8a have been compared against the physical modelling results in detail. Comparison of the numerical and physical models are presented in Section 5.5.

5.2.3 Key Reasons for Modelling and Model Requirements

The focus of this phase of model development is to validate a numerical modelling approach against physical modelling data of weather-driven stress cycles, the deformation that occurred and progressive failure, validating that the mechanism of seasonal ratcheting can be captured. To the authors knowledge, this is the first time that this mechanism has been modelled numerically and validated against physical data.
5.3 Calibration of Kaolin Strain-Softening Model

In line with the method presented in Section 3.4.2, a single element axisymmetric numerical model for overconsolidated drained triaxial tests on Kaolin have been modelled and behaviour compared against experimental data to allow calibration of the strain-softening model to be employed in model analyses. Experimental data of overconsolidated drained triaxial tests on Kaolin presented by Cekerevac and Laloui (2004) show post-peak strength reduction (i.e. strain-softening behaviour) and have been used within this calibration.

The single element axisymmetric numerical model of the triaxial test was initially fully saturated and volume change due to flow was permitted (i.e. the sample remained drained throughout). In line with the experimental procedure, the samples were initially consolidated to 600kPa and then unloaded to 100kPa (OCR=6) and 50kPa (OCR=12) (Cekerevac & Laloui, 2004). Confining pressure was set along the boundary of the model and a constant velocity applied to the top of the element to emulate a strain controlled test. Within the numerical model, stiffness is a function of specific volume which is dependent on the swelling line from 600kPa following the unloading of the sample following initial consolidation. This gives a specific volume of $v_{k,ref} = 2.27$ at a reference pressure of 1kPa, the specific volume relationship for these triaxial tests is shown in Figure 5.9. Throughout the analysis, the deviator stress, axial strain and stress path of the model have been monitored and compared against experimental data (Cekerevac & Laloui, 2004). This can be seen in Figure 5.10.

For the different overconsolidation ratios Figure 5.10a shows that peak strengths mobilised in the numerical model and the strains at which peak strength is mobilised are in close agreement to the experimental data. Figure 5.10 shows that the stiffness relationship and peak strength parameters stipulated are adequate to describe the mechanical behaviour of overconsolidated Kaolin. In addition, post-peak softening to critical state at large strains can be seen in both the stress-strain curve and stress paths presented.
To assess the sensitivity of the mechanical model being considered, the numerical models of the triaxial tests have been analysed without strain-softening behaviour and without stiffness as a function of specific volume. The results of these analyses can be seen in Figure 5.11 and show that omitting these key relationships results in unrepresentative mechanical behaviour.
5.4 Boundary Condition Selection

In line with the approach described within Section 5.2.2.8, the following boundary conditions scenarios have been considered for analysis of WAT7a (see Section 5.2.2);

1. Summer and winter discharge boundary conditions;
2. Summer and winter pore water pressure boundary conditions;
3. Summer discharge boundary condition and winter pore water pressure boundary conditions (combined boundary conditions).

In all instances, the boundary conditions stipulated have been selected by comparing the hydrogeological response of the numerical analyses with the physical modelling results of HCT1 and calibrating the boundary conditions to drive the correct pore water pressure cycle in the near-surface of the slope.

Figure 5.12 shows the comparison of the hydrogeological response of the different numerical analyses and the physical modelling results for four pore water pressure records. In all instances, the responses of the numerical models are more representative of the physical modelling results in the near-surface of the model, i.e. HCT1 and HCT2. At depth the numerical models do not predict the seasonal pore water pressure cycles as effectively – this discrepancy is discussed later as the focus of this section is establishing the most appropriate boundary conditions to drive near-surface stress cycles and not the validation of the numerical analysis approach.
When discharge boundary conditions are considered, the drying phase of analysis replicates the drying observed in the physical modelling well. However, the wetting within the physical modelling is at a quicker rate to that of the numerical analysis. Conversely, the wetting phase is modelled well when considering pore water pressure boundary conditions in the numerical analysis, but the drying phase is not. Finally, combining the two boundary conditions, discharge boundary conditions drying and pore water pressure boundary conditions wetting, it has been possible to replicate both the drying and wetting phases observed in the physical modelling. It should be noted that all boundary conditions used within the numerical analyses can adequately drive the seasonal stress cycles in the near-surface considering the pore water pressure cycles observed at HCT1. The magnitude of pore water pressures at different waypoints (i.e. end of summer and end of winter) match the physical modelling results. Therefore, at each waypoint, all numerical analyses are at the same stress state but have taken different stress paths to get there.
Figure 5.12 Comparison of physical and numerical modelling pore water pressures for different boundary conditions – WAT7a; a) HCT 1; b) HCT 2; c) HCT 3; d) HCT 4

The mechanical behaviour of the different analyses are shown in Figure 5.13.
Figure 5.13 Comparison of physical and numerical modelling displacements for different boundary conditions – WAT7a; a) crest; b) mid-slope; c) toe

The mechanical behaviour of the numerical analyses of different boundary conditions compared to physical modelling results show that for all analyses, the general trend of behaviour observed by Take and Bolton (2011) is being captured;

- drying causes primarily vertical displacement;
- wetting causes vertical and horizontal (up and outward) displacements;
- crest displacements experience very little horizontal movement;
- toe displacements experience primarily horizontal movement.

For the analysis considering discharge boundary conditions, the magnitude and directions of displacements of the mid-slope and toe are in very close agreement with the physical modelling. However, the horizontal displacements at the crest of the model do not match the physical model results as well. For the pore water pressure and combined boundary conditions, there are much
greater outward and downward movements observed in the mid-slope and toe displacements than in the physical modelling.

As stated previously, at the different waypoints, the stress states in all the numerical analyses will be comparable but the stress paths to get to the same stresses will differ. For this reason, the displacements at the waypoints have been plotted and the differences between the numerical analyses and physical modelling data have been established. The toe is the most critical location as it is from here that post-peak strength will initial be mobilised leading to progressive failure that will propagate back through the slope. Therefore, the mid-slope and toe displacements have been considered, these can be seen in Figure 5.14.

![Figure 5.14 Comparison of physical and numerical modelling displacements at waypoints for different boundary conditions – WAT7a; a) mid-slope; b) toe](image)

From Figure 5.14 it can be seen that the displacements associated with the numerical analysis with pore water pressure boundary conditions experience too much vertical displacement with drying and do not experience the inward movement seen in the physical modelling. The movement associated with the combined boundary conditions numerical analysis do not experience the same vertical swelling that the discharge boundary condition and physical modelling show. To quantify the fit of the different numerical analyses, the difference in the displacements at different waypoints for the mid-slope and toe have been calculated and are shown in Figure 5.15.
Figure 5.15 shows that for the mid-slope displacement the results from the pore water pressure boundary condition analysis is the closest match to the physical modelling and for toe displacements, it is the analysis considering discharge boundary conditions. Considering the displacement path of the mid-slope and toe, shown in Figure 5.13 and Figure 5.14, the path shown by the analysis with discharge boundary conditions is a better match to the displacement path observed within the physical modelling.

Therefore, it has been concluded that discharge boundary conditions produce the most representative hydrogeological and mechanical behaviour when considering the physical modelling. These boundary conditions will be used to drive seasonal weather behaviour in all subsequent numerical analysis.

5.5 Validation of Numerical Model against Physical Modelling

The following section presents the results of numerical models replicating physical models WAT7a and WAT8a. The numerical modelling approach developed, and material properties used are presented in Section 5.2.2. The
strain-softening model used has been validated against experimental data in Section 5.3, this has been done for a local strain-softening model. The displacement strain criteria obtained have been adopted in the nonlocal strain-softening model as absolute nonlocal plastic criteria as done previously for analysis of simplified cut slope behaviour considering London Clay in Chapter 4.

It has been shown that discharge boundary conditions are the most appropriate boundary condition within this modelling approach to drive seasonal pore water pressure cycles, and therefore stress cycles, to produce the most realistic displacements associated with seasonal wetting and drying. To allow the transient pore water pressures driven within the physical modelling to be reproduced in the numerical model, uniform steady discharge boundary conditions (i.e. constant for summer and winter and applied to full slope surface) have been applied to the numerical model boundary to drive the pore water pressure cycles recorded at HCT1 and HCT D3 for WAT7a and WAT8a respectively. This has been done as it is the near-surface behaviour that is the most significant behaviour being modelled. The tensiometers used to dictate the boundary conditions are approximately 1.0m below the slope surface and the closest to the slope surface. Therefore, allowing the closest match of boundary conditions to drive the appropriate hydrogeological cycles, and therefore stress cycles, within the numerical analyses.

5.5.1 WAT7a Results

Firstly, the hydrogeological behaviour of the numerical and physical models for WAT7a have been compared. Following this, the mechanical response of the models at three locations along the slope surface have been compared.

Figure 5.16 shows the hydrogeological behaviour of the physical and numerical models. The discharge boundary conditions used within the numerical model have been calibrated, through trial and error, to ensure the numerical model results matched the seasonal pore water pressure cycles, and therefore the stress cycles, observed in the near-surface of the physical modelling. The discharge boundary conditions applied to model WAT7a,
obtained through trial and error to drive the appropriate near-surface stress cycles are summarised in Table 5.5.

Table 5.5 WAT7a numerical model boundary conditions

<table>
<thead>
<tr>
<th>Waypoint</th>
<th>Discharge (mm/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B to C</td>
<td>-0.25</td>
</tr>
<tr>
<td>C to D</td>
<td>0.23</td>
</tr>
<tr>
<td>D to E</td>
<td>-0.17</td>
</tr>
<tr>
<td>E to F</td>
<td>0.25</td>
</tr>
<tr>
<td>F to G</td>
<td>-0.18</td>
</tr>
<tr>
<td>G to H</td>
<td>0.21</td>
</tr>
</tbody>
</table>

Figure 5.16 Comparison of physical and numerical modelling pore water pressures – WAT7a; a) HCT 1; b) HCT 2; c) HCT 3; d) HCT 4
From Figure 5.16 it can be seen that the hydrogeological behaviour observed in the physical modelling has been replicated within the near-surface of the numerical model. The magnitudes of seasonal pore water pressures at HCT1 are very similar. Therefore, the magnitude of stress cycles experienced by the numerical model in the near-surface is representative of the physical model.

At depth the pore water pressure cycles are not quite a good match. This is not surprising as within the numerical model it is assumed the soil is homogeneous and preferential flow has not been included but will exist within the physical model, in particular between the edge of the slope and the container during shrinkage due to drying. Other assumptions made within the modelling such as idealised liquid and gas phases and a single soil water retention curve to represent both drying and wetting will have influenced the pore water pressure trends obtained.

The results shown in Figure 5.16 are point record and allow transient records to be compared. To further validate the hydrogeological behaviour of the numerical analysis, a comparison of pore water pressure contours at the end of a wetting period (waypoint D) have been presented in Figure 5.17.

From visual inspection of Figure 5.17, it can be seen that the position of the zero pore water pressure line and 10kPa suction line are in similar positions for both models. However, it is difficult to directly compare the two as the amount of data available for the numerical model is significantly more that the number of tensiometers used in the physical modelling to obtain the contour plot.
Chapter 5 – Modelling Seasonal Ratcheting

Considering Figure 5.16 has shown the stress cycles in the near-surface of the numerical model are representative of the physical modelling, the displacements along the slope surface for the physical and numerical analyses have been compared in Figure 5.18. The mechanical behaviour observed in the numerical analysis replicates the key findings by Take and Bolton (2011). Of particular significance is the capturing of outward and downward movement and the fact that more outward movement is observed at the toe of the slope and more vertical movement observed at the crest.

Following a complete cycle of wetting and drying (i.e. D-F), the magnitudes of displacements obtained in the numerical analysis at the toe and mid-slope are close to those observed experimentally. The shrink-swell behaviour shown in Figure 5.18 is extremely important in the modelling of seasonal ratcheting and for the material relationships used within the numerical model analysis as stiffness and saturated hydraulic conductivity are a function of specific volume and void ratio respectively (void ratio = specific volume – 1). To illustrate the change in volume of the near-surface due to the stress cycles experienced, Figure 5.19 shows the void ratio from the numerical analysis at the end of a wetting period and a drying period.
Figure 5.18 Comparison of physical and numerical modelling mechanical behaviour – WAT7a; a) crest displacements; b) mid-slope displacements; c) toe displacements

Figure 5.19 Comparison of void ratios within numerical model at different times – WAT7a; a) after wetting – waypoint F; b) after drying – waypoint G

It has been shown that given the correct stress cycles, the numerical modelling approach developed can replicate the movements of seasonal ratcheting. A significant part of the mechanism of seasonal ratcheting is mobilisation of post-peak strength due to the stress cycles imposed. To show this process, shear
strain contour plots from the physical and numerical modelling have been presented in Figure 5.20 for waypoints D and G.

![Figure 5.20 Comparison of physical and numerical modelling shear strains – WAT7a; a) physical modelling waypoint D; b) numerical modelling waypoint D; c) physical modelling waypoint G; d) numerical modelling waypoint G](image)

The magnitude of shear strains and area over which shear strain accumulation has occurred in the physical and numerical models are shown to be comparable within Figure 5.20 for two different times in the analysis. Greater shear strains are present within the numerical analysis near the toe of the slope, this in part can be explained by the far greater number of calculation points within the numerical model compared to the physical model.

The accumulation of shear strains, a part of which are elastic and a part plastic, propagating into the slope with number of seasonal cycles (i.e. waypoint D compared to G) shows the onset of progressive failure within the models due to the stress cycles experienced by the slope models.

Within the physical modelling of WAT7a, a short shallow shear surface at the toe of the slope was formed following prolonged wetting from waypoint G to H and the formation of a tension crack part way up the slope. Within the numerical modelling, a deeper longer failure mechanism was observed, the
difference can be explained by the fact that the numerical model assumes the soil acts as a continuum and tension cracks cannot be included. However, the effect of prolonged wetting on slope performance can be observed. This is shown by considering shear strains within the numerical model at waypoints G and H, see Figure 5.21.

![Figure 5.21 Shear strain contours at different points in numerical analysis – WAT7a; a) waypoint G; b) waypoint H](image)

Figure 5.21a shows that strength reduction at the toe of the slope has occurred following seasonal wetting and drying and Figure 5.21b indicates a dramatic increase in the shear strains within the slope and therefore softening of the soil due to prolonged wetting.

### 5.5.2 WAT8a Results

As done for the results of WAT7a, firstly the hydrogeological behaviour then the mechanical behaviour of the numerical model for WAT8a has been compared with the results of physical modelling (Take & Bolton, 2011). Again, the discharge boundary conditions applied to drive the near-surface stress cycles for model WAT8a are summarised in Table 5.6.
Table 5.6 WAT8a numerical model boundary conditions

<table>
<thead>
<tr>
<th>Waypoint</th>
<th>Discharge (mm/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B to C</td>
<td>-0.25</td>
</tr>
<tr>
<td>C to D</td>
<td>0.18</td>
</tr>
<tr>
<td>D to E</td>
<td>-0.23</td>
</tr>
<tr>
<td>E to F</td>
<td>0.17</td>
</tr>
<tr>
<td>F to G</td>
<td>-0.21</td>
</tr>
<tr>
<td>G to H</td>
<td>0.20</td>
</tr>
<tr>
<td>H to I</td>
<td>-0.20</td>
</tr>
<tr>
<td>I to J</td>
<td>0.22</td>
</tr>
<tr>
<td>J to K</td>
<td>-0.17</td>
</tr>
<tr>
<td>K to L</td>
<td>0.19</td>
</tr>
<tr>
<td>L to M</td>
<td>-0.18</td>
</tr>
<tr>
<td>M to N</td>
<td>0.17</td>
</tr>
</tbody>
</table>

Figure 5.22 Comparison of physical and numerical modelling pore water pressures – WAT8a; a) HCT D3; b) HCT D2

As with the previous model, boundary conditions have been imposed to drive the seasonal pore water pressures, and therefore stress cycles, observed in the shallowest high capacity tensiometer, in this case HCT D3.
Figure 5.22 and Figure 5.23 show that behaviour observed in physical modelling can be successfully replicated within the numerical modelling approach developed allowing seasonal ratcheting movements to be captured. In additions, the displacements observed are of same order of magnitude and direction, increasing confidence in the numerical model’s ability to model real behaviour.

Sections 5.5.1 and 5.5.2 have shown that the modelling approach developed, the framework and material relationships adopted, can model seasonal ratcheting movements and progressive failure due to established stress cycles. As far as the author is aware, this is the first time that such behaviour, strength deterioration of an overconsolidated clay slope due to cyclic wetting and drying, has been modelled numerically and validated against real data.

### 5.6 Effect of Material Stiffness on Seasonal Ratcheting

Using the numerical model of WAT7a, the effect of continued seasonal cycles of wetting and drying, rather than prolonged wetting, have been investigated for two slopes formed in Kaolin with different stiffness relationships. Discharge boundary conditions have been used to apply continuous seasonal cycles of 0kPa at the end of winter and 40kPa suctions at the end of summer at HCT1 up to failure of the slope models. Analyses have been run considering the material stiffness used to replicate the physical modelling (Material A) and for a stiffer material (Material B). The stiffness of both materials is calculated
while stepping within the analyses in line with the method described in Section 5.2.2.5. The different stiffness parameters used are summarised in Table 5.7 and are shown graphically in Figure 5.24.

Table 5.7 Stiffness parameters for different materials

<table>
<thead>
<tr>
<th>Material</th>
<th>$\Gamma$</th>
<th>$\lambda$</th>
<th>$\kappa$</th>
<th>$K_{\text{min}}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material A</td>
<td>3.55</td>
<td>0.25</td>
<td>0.05</td>
<td>2000</td>
</tr>
<tr>
<td>Material B</td>
<td>2.65</td>
<td>0.124</td>
<td>0.02</td>
<td>3500</td>
</tr>
</tbody>
</table>

The seasonal pore water pressure cycles observed at HCT1 in the models are shown in Figure 5.25a, it can be seen that the two models were subjected to the same seasonal stress cycles during the analyses. Figure 5.25b shows that the mechanical behaviour of the two models differ considerably. The analysis considering Material A fails after 14 seasonal cycles whereas the stiffer material, Material B, remains stable for 33 seasonal cycles. In addition, the failure of analysis with Material A is much more sudden than that of the gradual deterioration of the slope in Material B. For the same seasonal cycle of pore water pressures and thus stress cycles, the seasonal movement driven within the analysis considering Material B is much less than the scenario with Material A, this is shown by the smaller ratcheting cycles shown in Figure 5.26a. Therefore, the magnitude of plastic strain accumulation for a single seasonal cycle is smaller for a stiffer material and the number of seasonal cycles taken for failure to occur is greater. Finally, both shear surfaces obtained from these analyses are short and shallow, incorporating only the toe of the slope, see Figure 5.26b. Compared to failure surface following prolonged wetting, shown in Figure 5.21b, it can be concluded that the stress changes due to seasonal wetting and drying influence a shallower volume of material than the stress.
changes due to prolonged wetting. It also shows the significance a prolonged wet period can have on slope deterioration, Material A is the same as the material used in the analysis of WAT7a, under continued seasonal cycles the slope remained stable for an additional 11 seasonal cycles compared to prolonged wetting.

The results presented in this section, along with the previous results of the validation of the numerical modelling approach and the findings presented by Take and Bolton (2011) provide an explanation for why a slope that appears
stable during one wet period may fail a number of years later under the same or a less significant wet period. It is this mechanism, seasonal ratcheting, that explains the failures being observed in high-plasticity overconsolidated clay ageing infrastructure slopes.

### 5.7 Methodology (Phase 2b) – Geometric Study

Section 5.5 shows the numerical modelling approach developed captures both hydrogeological and mechanical behaviour of a slope subjected to wetting and drying stress cycles leading to seasonal ratcheting and progressive failure. The strength deterioration experienced by repeated wetting and drying can explain shallow first-time failures due to repeated stress cycles. For this reason, the mechanism has been further investigated through a geometric study looking at the effect of slope geometry and strain-softening behaviour to allow design recommendations to be proposed.

Design of slopes is conventionally carried out using limit equilibrium analysis and design parameters reflecting the desired design life / time scale for which the asset is in use. Within clays, design parameters for temporary slopes can consider peak strength whereas long-term (i.e. perpetually stable) slopes should consider residual strength (Skempton, 1964). However, the use of residual strength parameters in infrastructure slope design will yield slopes with design-lives far in excess of requirement. For deep-seated failures in overconsolidated clay Chandler and Skempton (1974) and Skempton (1977) discuss the use of ‘fully softened’ material properties for design. For assets experiencing seasonal ratcheting, Take and Bolton (2011) discuss the use of critical state strength to provide sufficient design-lives. This geometric study aims to test the hypothesis presented by Take and Bolton (2011) and provide design recommendations for material properties and serviceable design life of clay slopes that experience shallow first-time failures due to seasonal ratcheting driven by wetting and drying stress cycles.

Firstly, the material properties, initial stress conditions and boundary conditions used within the numerical analyses are presented followed by the method developed for determining the point of serviceability failure. A summary of the numerical analyses run for different strain-softening behaviour and slope
heights considered is shown. Finally, the method for interpreting results through a residual factor (after, Skempton, 1964) and mobilised friction angle to communicate strength deterioration and inform design parameter selection is presented.

### 5.7.1 Model Properties

All slope models considered have the same initial stress conditions and material properties of model WAT7a (see Section 5.2.2), with the exception of different strain-softening relationships used (see Section 5.7.2). As with the models considering different material stiffness (see Section 5.6), slope models have been subject to seasonal pore water pressure cycles of 40kPa; 40kPa suction at the end of summer (7 months) and 0kPa at the end of winter (5 months). Boundary conditions are monitored throughout analysis at the mid-slope surface of the model and discharge boundary conditions adjusted at the end of each seasonal cycle to ensure the 40kPa cycle is maintained, this is shown in Figure 5.27 for a typical slope mesh.

![Figure 5.27 Typical mesh and boundary conditions](image)

The boundary conditions imposed have been selected to ensure that long-term strength deterioration is captured for numerous slope geometries in the same way. In addition, within Section 5.5, the numerical model behaviour has been proven to replicate physical behaviour within these pore water pressure ranges increasing confidence in the numerical analyses outputs. The effects of extreme wet events have not been included as they can trigger sudden failure,
masking the trends in behaviour of rate of strength deterioration for different slope geometries, which is of primary interest.

Within the analysis the nonlocal strain-softening model described in Section 3.3.2 has been used. To avoid very shallow failures due to material softening to a cohesion of 0kPa and low effective stress in the near-surface, the upper 1m of slope surface for all slope models has been limited to a minimum cohesion of 2kPa (i.e. the material softens in the same way, but the cohesion does not reduce below 2kPa in the upper 1m of the slope), this is shown in Figure 5.27 for a typical slope mesh. For completeness, material properties are summarised in Table 5.8 and Table 5.9. The residual friction angle for Kaolin has been taken from ring shear experimental data to be 12º (Dobbie, 1992).

Table 5.8 Mechanical properties for Kaolin

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Nonlocal Plastic Strain</td>
<td>0.05</td>
</tr>
<tr>
<td>Critical State Nonlocal Plastic Strain</td>
<td>0.15</td>
</tr>
<tr>
<td>Residual Nonlocal Plastic Strain (Option B)</td>
<td>0.80</td>
</tr>
<tr>
<td>Residual Nonlocal Plastic Strain (Option C)</td>
<td>0.40</td>
</tr>
<tr>
<td>Internal Length Parameter (m)</td>
<td>0.5</td>
</tr>
<tr>
<td>Peak Cohesion (kPa)</td>
<td>6.25</td>
</tr>
<tr>
<td>Critical State Cohesion (kPa)</td>
<td>0.00</td>
</tr>
<tr>
<td>Residual Cohesion (kPa)</td>
<td>0.00</td>
</tr>
<tr>
<td>Peak Friction (º)</td>
<td>24.0</td>
</tr>
<tr>
<td>Critical State Friction (º)</td>
<td>22.0</td>
</tr>
<tr>
<td>Residual Friction (º)</td>
<td>12.0</td>
</tr>
<tr>
<td>Dilation Angle (º)</td>
<td>0.0</td>
</tr>
<tr>
<td>Unit weight (kN/m²)</td>
<td>17.9</td>
</tr>
<tr>
<td>λ</td>
<td>0.25</td>
</tr>
<tr>
<td>κ</td>
<td>0.05</td>
</tr>
<tr>
<td>Γ</td>
<td>3.55</td>
</tr>
<tr>
<td>Poisson’s Ratio, ν’</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Bulk modulus, K (kN/m²)*
\[ \frac{
u, \sigma’}{\kappa} \text{; min } 2000 \text{kPa} \]

Shear modulus, G (kN/m²)
\[ \frac{3(1 - 2\nu’)}{2(1 + \nu’)} . K \]

*ν = specific volume and \( \sigma’ \) = effective stress (kPa)

Table 5.9 Hydrogeological parameters for Kaolin (after, Al-Tabbaa & Wood, 1987)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>van Genuchten fitting parameter (kPa)</td>
<td>α</td>
</tr>
<tr>
<td>van Genuchten fitting parameter</td>
<td>n</td>
</tr>
<tr>
<td>van Genuchten fitting parameter</td>
<td>m</td>
</tr>
<tr>
<td>Vertical saturated hydraulic conductivity</td>
<td>( k_{\text{sat,vert}} = 0.53e^{3.16} \times 10^{-9} )</td>
</tr>
<tr>
<td>Horizontal saturated hydraulic conductivity</td>
<td>( k_{\text{sat,hor}} = 1.49e^{2.03} \times 10^{-9} )</td>
</tr>
</tbody>
</table>

*\( e = \text{void ratio} \)
5.7.2 Effect of Strain-Softening Behaviour

Within this study, the effect of slope angle on 8.65m slopes (i.e. same height as WAT7a at full scale) has been investigated considering three different strain-softening relationships. The strain-softening relationships considered for Kaolinite have been proposed to allow assessment of different softening behaviours exhibited by soils. The strength and nonlocal plastic strain criteria for the different relationships are summarised in Table 5.8 and shown graphically in Figure 5.28 from single element numerical models of an overconsolidated drained triaxial test (see Section 3.4.2).

The strain-softening relationships proposed have been selected to develop understanding of design parameters for slopes experiencing seasonal ratcheting with different shear behaviours. Option (A) has been taken as a baseline for the numerical analyses, considering a material that experiences strength reduction to critical state due to dilation and concurrent destruction of cohesion and does not experience further strength reduction due to particle reorientation. Option (A) has been used to conduct preliminary analysis and explore behaviour prior to the modelling of more realistic strain-softening behaviour of options (B) and (C). Options (B) and (C) are more indicative of overconsolidated clays exhibiting turbulent shear post-peak due to dilation to critical state strength followed by further strength reduction at a reduced rate due to particle reorientation (i.e. sliding shear (Lupini, et al., 1981)). Two rates of softening from critical state to residual strength have been proposed to assess the influence of the rate of softening due to particle reorientation.
A summary of the models run within this study for the different softening relationships and different slope geometries are shown in Table 5.10, with the results of the analyses presented in Section 5.8.1.

Table 5.10 Summary of model analyses considering different strain-softening relationships

<table>
<thead>
<tr>
<th>Slope Height (m)</th>
<th>Slope Angle (°)</th>
<th>Softening (A)</th>
<th>Softening (B)</th>
<th>Softening (C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.65</td>
<td>40.9</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>38.2</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>35.8</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>33.6</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>31.7</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>30.0</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>28.4</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>27.0</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>25.7</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>24.5</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>23.4</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>21.5</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>19.8</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>18.4</td>
<td>✓</td>
<td>✓</td>
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</tr>
<tr>
<td>8.65</td>
<td>17.2</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8.65</td>
<td>15.1</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

5.7.3 Effect of Slope Height

As a second part of this study, the effect of slope height on rate of strength deterioration due to seasonal ratcheting has been considered. This has been done considering strain-softening option (B) only for a range of different angled slopes at 5m, 8.65m and 12m high. Again, all material properties, initial stress conditions and boundary conditions remain the same for all models. A summary of the different geometries for the slopes considered are shown in Table 5.11 and results are presented in Section 5.8.2. Strain-softening option (B) has been used within the study as the behaviour is the most representative of a real over-consolidated clay with quick softening to critical state strength and then large strains required to reach residual.
Table 5.11 Summary of model results considering different slope heights for strain-softening option (B)

<table>
<thead>
<tr>
<th>Slope Angle (°)</th>
<th>5m High Slopes</th>
<th>Slope Angle (°)</th>
<th>8.65m High Slopes</th>
<th>Slope Angle (°)</th>
<th>12m High Slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td>59.0</td>
<td>✓</td>
<td>40.9</td>
<td>✓</td>
<td>33.7</td>
<td>✓</td>
</tr>
<tr>
<td>51.3</td>
<td>✓</td>
<td>38.2</td>
<td>✓</td>
<td>32.3</td>
<td>✓</td>
</tr>
<tr>
<td>45.0</td>
<td>✓</td>
<td>35.8</td>
<td>✓</td>
<td>31.0</td>
<td>✓</td>
</tr>
<tr>
<td>39.8</td>
<td>✓</td>
<td>33.6</td>
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<td>28.6</td>
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</table>

5.7.4 Defining Serviceability Failure

Given the focus of this parametric study, looking at strength deterioration with number of stress cycles due to controlled wetting and drying boundary conditions (i.e. no extreme wet events). It is important that a robust, repeatable methodology be developed for the assessment of time of failure for a slope model. This is particularly important for very shallow slope angles as the annual displacements observed due to cyclic wetting and drying can be very small and as such it is very difficult to ascertain the point at which the slope becomes unstable. Within this study, serviceability failure is defined as the point at which the slope becomes unstable and the failing mass begins to accelerate to ultimate state failure and deformations become excessively large.

It has been shown that a negative linear relationship between the reciprocal of velocity and time indicates first-time failures of slopes formed within strain-softening soils (Saito, 1980; Petley, et al., 2005). As the reciprocal of velocity tends towards zero, ultimate limit state failure is observed. Within this work, it is the point in time at which the slope becomes unstable (i.e. the point of onset of failure) and significant strength deterioration occurs within a single seasonal cycle that is of more interest as this is where the slope experiences serviceability failure.
Within all slope analyses, a number of points along the slope surface are monitored during the running of the numerical analyses to obtain displacement records. At ultimate failure (i.e. large strains) the critical shear surface for the model is obtained and the displacement records along the slope surface corresponding to the mid-point of the critical shear surface are extracted. These are used to calculate the reciprocal of velocity for an annual seasonal cycle when outward and downward movement, indicative of seasonal ratcheting, is observed. To obtain the point of serviceability failure, a curve is fitted against the annual reciprocal of velocity for the failed slope and the point of inflection of the curve (i.e. the point of greatest acceleration of the failing mass) is taken as the serviceability failure point. The fitted curve is optimised (using an inbuilt function ‘Solver’ within Microsoft EXCEL) by varying parameters used to create the fitted curve to obtain the best $R^2$ value possible between the fitted curve and the reciprocal of velocity. The ‘Solver’ function uses the ‘Generalized Reduced Gradient’ algorithm to obtain optimised values. The process of curve fitting and obtaining point of serviceability failure can be seen for one of the slopes in the geometric study (25.7\degree and 8.65m high slope strain-softening option A) in Figure 5.29.

![Figure 5.29 Deterioration curve and point of serviceability failure – reciprocal of velocity and fitted curve](image)

To further illustrate that the method developed allows the point of onset of failure to be captured (i.e. serviceability failure), the point of serviceability failure defined using the method illustrated in Figure 5.29 has been plotted on graphs of displacement in an annual cycle and cumulative displacement against number of seasonal cycles for the point on the slope surface...
corresponding to the centre of the failed mass at large strain, this can be seen in Figure 5.30.

![Figure 5.30 Serviceability failure criteria; a) displacement in an annual cycle; b) cumulative displacement](image)

Figure 5.30 shows that the serviceability failure criteria plotted on both graphs is at a tipping point in behaviour. In Figure 5.30a, after the point of serviceability failure, there is an increase in the magnitude of displacement occurring annually indicating the acceleration of the failing mass. Similarly, in Figure 5.30b, the gradient of the cumulative displacement plot increases after the point of serviceability failure.

For shallow first-time failures in clay slopes due to seasonal ratcheting, this method provides an approach to obtain the point in time of serviceability failure that is consistent and reproducible. This method has been used to obtain the point of serviceability failure for all slopes considered in the geometric study.
5.7.5 Determining Residual Factor / Mobilised Friction Angle at Serviceability Failure

Using the point of serviceability failure, it is possible to determine the mobilised strength at the point where the slope becomes unstable. Knowing the mobilised strength at serviceability failure allows design parameter selection and serviceable life for shallow first-time failures in clay slopes due to seasonal wetting and drying stress cycles to be discussed. Within this study two approaches for the consideration of strength deterioration at serviceability failure have been used:

- residual factor;
- mobilised internal friction angle.

The residual factor, as presented by Skempton (1964), gives an easily understandable indication of the strength reduction experienced along the critical shear surface relative to the peak and residual strength of the material. The mobilised internal friction angle at serviceability failure also provides an indication of the level of deterioration experienced and allows findings to be explored relative to the material softening behaviour and identify design recommendations in an easier way than the residual factor.

For the calculation of the residual factor at the point of serviceability failure, the peak, current and residual shear strength for each element within the mesh of a slope model are calculated using Equation 5.10.

\[
\tau_i = c'_i + \sigma'_{\text{current}} \cdot \tan(\phi'_i)
\]

Where:

- \( i \) = peak, current or residual strength parameters.

Using the three shear strengths calculated (i.e. peak, current and residual strength), the residual factor for each element can be obtained using Equation 5.11.

\[
\text{Residual factor}, R = \frac{\tau_{\text{peak}} - \tau_{\text{current}}}{\tau_{\text{peak}} - \tau_{\text{residual}}}
\]
Considering the failure surface at large strains, the residual factors of the elements that the shear surface passes through can be extracted and averaged providing an average residual factor for the shear surface. The average residual factor gives an indication of the strength deterioration required for the particular slope geometry to fail. This method has been used to obtain the average residual factor at the point of serviceability failure for all slopes modelled in the geometric study.

Similarly, the mobilised internal friction angle at serviceability failure is obtained by extracting the current friction angles from the numerical model which the critical shear surface passes through. The values are then averaged to produce a single average value of friction angle mobilised along the critical shear surface. Considering average mobilised friction angle allows strength properties to be considered in terms of slope angle and design life for the investigation of design recommendations for limit equilibrium analyses.

5.7.6 Key Reasons for Modelling and Model Requirements

Given the validation of the numerical modelling approach against physical data showing that seasonal ratcheting can be captured numerically, a parametric study using the approach to better understand the mechanism of failure is to be undertaken. The key behaviours being investigated within this parametric study are strain-softening behaviour and slope geometry.

5.8 Geometric Study Results

As outlined in Section 5.7, an investigation into the effect of slope geometry on seasonal ratcheting due to repeated wetting and drying has been undertaken. With the exception of the different strain-softening relationships, highlighted in Figure 5.28, all slopes within this study have the same initial conditions, material properties and have been subjected to the same seasonal pore water pressure cycles, as summarised in Section 5.7.1.

Within this section, firstly, the results of numerical analyses considering different strain-softening behaviours (as described in Section 5.7.2) considering slope movement, failure surfaces and obtaining the point of serviceability failure (see Section 5.7.4) are presented. This is followed by the
results for an investigation into the effect of slope height on the seasonal ratcheting mechanism (as described in Section 5.7.3). Finally, the residual factors and mobilised friction angle along the critical shear surface at serviceability failure (see Section 5.7.5) have been determined for all analyses and the results presented.

It should be noted that this work considers deterioration rates and trends in behaviour for high-plasticity overconsolidated clay slopes subjected to uniform seasonal stress cycles to further understanding of the mechanism of seasonal ratcheting and progressive failure.

**5.8.1 Effect of Strain-Softening Behaviour**

The different strain-softening relationships adopted within this study are shown in Figure 5.28. The displacements of all slope analyses considering the different strain-softening relationships and the large strain shear surfaces are shown in Figure 5.31 to Figure 5.36.

![Figure 5.31 Displacements of slope surface corresponding to mid-shear surface against number of seasonal cycles for different slope angles, strain-softening option (A); a) horizontal displacement; b) vertical displacement](image_url)
Chapter 5 – Modelling Seasonal Ratcheting

Figure 5.31 shows the displacements of the slope surface corresponding to the mid-point of the large strain failure surface. The numerical analyses results show the shallower the slope angle the longer it takes for the accumulation of outward and downward movement (i.e. seasonal ratcheting). The results presented in Figure 5.31 can be considered in three categories;

1. 40.9° to 33.6° slopes – these slopes are relatively steep for the material in which they are formed and the failures that occur are sudden, shown by the sharp increase in horizontal and vertical displacement. Considering Figure 5.32, the shear surfaces of these slopes are deep-seated and include the entire length of the slope;

2. 31.7° to 24.5° slopes – these slopes fail in a much more gradual manner, as the slope angle decreases, the gradient of the accumulated horizontal displacement with time decreases, indicating a reduction in the amount of displacement occurring within an annual cycle due to wetting and drying for shallower angled slopes. Again, considering the failure surfaces for these slopes, all the failures are shallow, translational in nature and only affect the lower half of the slope length.

3. 23.4° slope – the behaviour of this slope deviates from the trends shown for the other numerical analyses. No acceleration to failure is shown within either the horizontal or vertical direction even though the accumulation of displacements with number of seasonal cycles is observed. Investigating the numerical analysis after over 200 seasonal cycles, the majority of the material within the slope model is at the minimum strength for this particular analysis but ultimate failure is not observed.

The results presented in Figure 5.31 indicate that the minimum friction angle of a soil limits the geometry of a slope that is at risk of failure due to seasonal ratcheting and progressive failure. For the numerical analyses results presented in Figure 5.31, the minimum friction angle of the material is 22.0° and a slope of 23.4° did not fail due to repeated wetting and drying. The difference in the slope angle being slightly greater than the friction angle can
be explained by the additional surface cohesion used within the numerical analyses as discussed in Section 5.7.1.

For strain-softening options (B) and (C) the minimum friction angle of the material is less than for option (A) (i.e. 12.0° compared to 22.0°). Within the analysis with the lower minimum friction angle, the behaviour of slopes shallower the 23.4° do not show the same divergence in behaviour (see Figure 5.33 and Figure 5.35). It can therefore be concluded that the behaviour of the 23.4° slope in Figure 5.31 is an artefact of the numerical analyses (i.e. convergence issues) rather than real behaviour and that the minimum friction angle of a soil limits the slope geometries that are at risk of first-time shallow failure due to seasonal wetting and drying stress cycles, seasonal ratcheting and progressive failure.

Figure 5.32 Failure surfaces for different slope angles – strain-softening option (A)
As with the results for the numerical analyses considering strain-softening option (A), the results of the analyses of strain-softening option (B) and (C) have been presented.

Figure 5.33 Displacements of slope surface corresponding to mid-shear surface against number of seasonal cycles for different slope angles, strain-softening option (B); a) horizontal displacement; b) vertical displacement
Figure 5.34 Failure surfaces for different slope angles – strain-softening option (B)
Figure 5.35 Displacements of slope surface corresponding to mid-shear surface against number of seasonal cycles for different slope angles, strain-softening option (C); a) horizontal displacement; b) vertical displacement
The behaviours shown for the numerical analyses considering strain-softening option (B) and (C) show similar trends as that of option (A). The shallower the slope angle the higher the number of seasonal cycles the slope remains stable.
and the shorter and shallower the failure surface. The rate of strength reduction post-critical state in option (C) is greater than option (B), the point at which the slope displacements accelerate to failure is earlier for the option (C) showing that strain-softening behaviour has an influence on the rate of deterioration due to seasonal ratcheting.

The main observation regarding behaviour shown in Figure 5.33 and Figure 5.35 is the change in behaviour of slopes less than 23.4°. For strain-softening option (A) this angled slope did not fail as it was so close to the minimum frictional angle of the material. For strain-softening options (B) and (C), the behaviour of the slope movements changes at this slope angle, the horizontal displacement accumulation becomes more gradual and the tipping point to accelerated failure is greater than slope angle steeper than 23.4°. This difference in behaviour can be attributed to the change in the rate of strength reduction with level of straining depending on the strength mobilised at failure (i.e. for slopes greater than critical state friction angle, strength reduction from peak to critical state strength is rapid and failure is sudden; for slopes shallower than critical state friction angle strength reduction from critical state to residual is more gradual and therefore failure is also more gradual).

The results of the numerical analyses presented thus far show the significance of the minimum friction angle of the material and how understanding the strain-softening behaviour of a material is vital to understanding the slopes at risk of shallow first-time failure due to seasonal ratcheting.

Using the method presented in Section 5.7.4, the point of serviceability failure for all analyses considering the different strain-softening relationships have been established and the deterioration curves showing the point of serviceability failure presented in Figure 5.37 to Figure 5.39. The $R^2$ values of the fitted curves and the point of serviceability failure have been summarised in Table 5.12 and Table 5.13 respectively.
Figure 5.37 Deterioration curves and points of serviceability failure for different slope geometries – strain-softening option (A); a) reciprocal of velocity of annual cycle; b) fitted curve; c) displacement in an annual cycle; d) cumulative displacement
Figure 5.38 Deterioration curves and points of serviceability failure for different slope geometries – strain-softening option (B); a) reciprocal of velocity of annual cycle; b) fitted curve; c) displacement in an annual cycle; d) cumulative displacement
Figure 5.39 Deterioration curves and points of serviceability failure for different slope geometries – strain-softening option (C): a) reciprocal of velocity of annual cycle; b) fitted curve; c) displacement in an annual cycle; d) cumulative displacement
Table 5.12 Summary of $R^2$ values for fitted curve against reciprocal of velocity associated with seasonal ratcheting considering different strain-softening relationships

<table>
<thead>
<tr>
<th>Slope Height (m)</th>
<th>Slope Angle (°)</th>
<th>Softening (A)</th>
<th>Softening (B)</th>
<th>Softening (C)</th>
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<tbody>
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Table 5.13 Summary of model results for number of cycles to serviceability failure considering different strain-softening relationships

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<th>Slope Height (m)</th>
<th>Slope Angle (°)</th>
<th># of Seasonal Cycles to Failure</th>
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</thead>
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Figure 5.37 to Figure 5.39 show the deterioration curves for all the slopes considered within this section of the geometric study. The points of serviceability failure determined using the inverse velocity and fitted curve can be seen to capture the tipping point of magnitude of annual displacement for all the numerical analyses indicating the method developed is robust and repeatable.
The results summarised in Table 5.13 have been plotted in Figure 5.40 to show slope angle against number of seasonal cycles to serviceability failure and compare the effect of different strain-softening relationships.

As discussed previously, from the results of strain-softening option (A), Figure 5.40 shows that the minimum friction angle of a material limits the slope angle that is at risk to failure due to seasonal ratcheting. The same trends are shown for options (B) and (C) but tending towards residual strength rather than critical state strength. Interestingly, Figure 5.40 indicates that the number of seasonal cycles to serviceability failure increases for option (B) and (C) quite considerably for slopes shallower than the critical state friction angle. This shows the significance of post-peak and post-critical state strain-softening of a material and its implications on seasonal ratcheting behaviour. The difference in the rate of post-critical state strength reduction for options (B) and (C) can be seen to have a significant effect on the number of seasonal cycles to serviceability failure for the same geometry slope. With slopes considering strain-softening option (C) failing a lot earlier than the same slopes using option (B). To illustrate this difference more clearly, the numbers of seasonal cycles to reach serviceability failure for particular slope angles are shown in Figure 5.41 for the three strain-softening options.
Figure 5.41 Difference in number of seasonal cycles to serviceability failure for different slope angles and strain-softening relationships

Figure 5.41 shows the shallower the slope angle, the greater the difference in the serviceable life of a slope for the different strain-softening relationships considered. For steep slopes within this material (i.e. > 30°), there is very little difference in the serviceable life for different strain-softening options. This shows that behaviour in these steep slopes is controlled by post-peak strength reduction.

The strain-softening behaviour of a soil has been shown to have significant influence on the slope geometries that are at risk of shallow first-time failure due to repeated seasonal stress cycles. Slopes at risk of failure are limited to slope angles greater than the residual friction angle of the material. The importance of softening behaviour, post-peak and post-critical state on the serviceable life of slopes due to repeated seasonal stress cycles has also been shown.

5.8.2 Effect of Slope Height

In the same way that results have been presented for the three strain-softening relationships, the results of numerical analyses considering the effect of slope height on seasonal ratcheting are summarised in the following section. Slope heights of 5m, 8.65m and 12m have been considered with all numerical analyses including strain-softening option (B). The methodology for this study can be found in Section 5.7.3. Within Figure 5.42 to Figure 5.47, the
displacements of the different analyses and large strain shear surfaces for the different height slopes are shown.

Figure 5.42 Displacements of slope surface corresponding to mid-shear surface against number of seasonal cycles for different slope angles, 5m high slopes; a) horizontal displacement; b) vertical displacement
Figure 5.43 Failure surfaces for different slope angles – 5m high slopes
Figure 5.44 Displacements of slope surface corresponding to mid-shear surface against number of seasonal cycles for different slope angles, 8.65m high slopes; a) horizontal displacement; b) vertical displacement
Figure 5.45 Failure surfaces for different slope angles – 8.65m high slopes
Figure 5.46 Displacements of slope surface corresponding to mid-shear surface against number of seasonal cycles for different slope angles, 12m high slopes; a) horizontal displacement; b) vertical displacement.
From the figures presented so far considering the effect of slope height on seasonal ratcheting, it can be seen that lower height slopes of the same angle remain stable for a greater number of seasonal stress cycles than greater height slopes. However, when failure does occur it is more sudden, and the majority of the slope length is involved in the failure. This is shown in Figure 5.42 by the sudden increase in displacements at failure relative to the displacements shown in Figure 5.44 and Figure 5.46. The shear surfaces shown in Figure 5.43 for the lower height slopes of 5m include large proportions of the slope length within the shear surface even at shallow slope angles.

For higher slopes, the failure surfaces observed tend to include only a small portion of the slope length and the displacements accumulated prior to accelerated failure are much greater than the lower height slope models.
The deterioration curves for the slopes allowing the point of serviceability failure to be defined are presented in Figure 5.48 to Figure 5.50, in line with the method presented in Section 5.7.4. With the $R^2$ values of the fitted curves compared to the reciprocal of velocity and the number of seasonal cycles to serviceability failure summarised in Table 5.14 and Table 5.15 respectively.
Figure 5.48 Deterioration curves and points of serviceability failure for different slope geometries – 5m high slopes; a) reciprocal of velocity of annual cycle; b) fitted curve; c) displacement in an annual cycle; d) cumulative displacement
Figure 5.49 Deterioration curves and points of serviceability failure for different slope geometries – 8.65m high slopes: a) reciprocal of velocity of annual cycle; b) fitted curve; c) displacement in an annual cycle; d) cumulative displacement
Figure 5.50 Deterioration curves and points of serviceability failure for different slope geometries – 12m high slopes; a) reciprocal of velocity of annual cycle; b) fitted curve; c) displacement in an annual cycle; d) cumulative displacement
Table 5.14 Summary of $R^2$ values for fitted curve against reciprocal of velocity associated with seasonal ratcheting considering different slope heights

<table>
<thead>
<tr>
<th>Slope Angle (°)</th>
<th>5m High Slopes</th>
<th>Slope Angle (°)</th>
<th>8.65m High Slopes</th>
<th>Slope Angle (°)</th>
<th>12m High Slopes</th>
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</thead>
<tbody>
<tr>
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<td>0.950</td>
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<td></td>
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</tr>
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<td></td>
<td></td>
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<td>0.911</td>
</tr>
</tbody>
</table>

Table 5.15 Summary of model results for number of cycles to serviceability failure considering different slope heights

<table>
<thead>
<tr>
<th>Slope Angle (°)</th>
<th>5m High Slopes</th>
<th>Slope Angle (°)</th>
<th>8.65m High Slopes</th>
<th>Slope Angle (°)</th>
<th>12m High Slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td># of Seasonal Cycles to Failure</td>
<td></td>
<td># of Seasonal Cycles to Failure</td>
<td></td>
<td># of Seasonal Cycles to Failure</td>
</tr>
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<td>33.7</td>
<td>8</td>
</tr>
<tr>
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<td>15</td>
<td>32.3</td>
<td>11</td>
</tr>
<tr>
<td>45.0</td>
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<td>18</td>
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<td>13</td>
</tr>
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<td>39.8</td>
<td>40</td>
<td>33.6</td>
<td>22</td>
<td>28.6</td>
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<td>31.7</td>
<td>28</td>
<td>26.6</td>
<td>22</td>
</tr>
<tr>
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<td>81</td>
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<td>33</td>
</tr>
<tr>
<td>29.1</td>
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<td>50</td>
</tr>
<tr>
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<td>27.0</td>
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<td>-</td>
</tr>
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<td></td>
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<td></td>
<td>15.1</td>
<td>305</td>
</tr>
</tbody>
</table>

The results summarised in Table 5.15 have been plotted in Figure 5.51 to show the influence of slope height on the serviceable life for different slope angles.
Figure 5.51 shows that slope height has significant influence on the number of seasonal cycles to serviceability failure for the same angled slope. This is shown more clearly in Figure 5.52, where number of seasonal cycles to serviceability failure have been compared to slope angle for the different slope heights. There is a much larger difference between the number of seasonal cycles to failure for the 5m to the 8.65m high slopes than compared with the 8.65m and 12m high slopes.
Chapter 5 – *Modelling Seasonal Ratcheting*

The results presented in this section show the significance that slope height has on the rate of deterioration of high-plasticity overconsolidated clay slopes experiencing seasonal ratcheting and progressive failure. The design requirements of low height slopes will differ considerably to higher slopes of the same angle.

Considering the mechanism being modelled, it is not surprising that lower height slopes of the same angle as higher slope remain stable for a greater number of seasonal cycles. Failure occurs due to the ratcheting of the near-surface of the slopes, strain accumulation and progressive failure. Within a lower height slope the slope face is much shorter and therefore there is a smaller mass of material ratcheting with each stress cycle and less weight / soil mass effectively pushing along and down the slope face compared to a higher slope.

**5.8.3 Residual Factors at Serviceability Failure**

Using the method outlined in Section 5.7.5, the residual factor at the point of serviceability failure has been determined to ascertain the average mobilised strength along the critical shear surface at this time. The results have been summarised in Table 5.16 and Table 5.17 and are presented in Figure 5.53 considering the different strain-softening options and Figure 5.54 considering the different height slopes.
Table 5.16 Summary of model results for number of cycles to serviceability failure and residual factor considering different strain-softening relationships

<table>
<thead>
<tr>
<th>Slope Height (m)</th>
<th>Slope Angle (°)</th>
<th>Softening (A)</th>
<th>Softening (B)</th>
<th>Softening (C)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td># of Cycles</td>
<td>Residual Factor</td>
<td># of Cycles</td>
<td>Residual Factor</td>
</tr>
<tr>
<td>8.65</td>
<td>40.9</td>
<td>3</td>
<td>0.13</td>
<td>3</td>
</tr>
<tr>
<td>8.65</td>
<td>38.2</td>
<td>16</td>
<td>0.43</td>
<td>15</td>
</tr>
<tr>
<td>8.65</td>
<td>35.8</td>
<td>18</td>
<td>0.39</td>
<td>18</td>
</tr>
<tr>
<td>8.65</td>
<td>33.6</td>
<td>24</td>
<td>0.60</td>
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</tr>
<tr>
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<td>45</td>
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<tr>
<td>8.65</td>
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<td>87</td>
<td>0.88</td>
<td>64</td>
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</tbody>
</table>

Table 5.17 Summary of model results for number of cycles to serviceability failure and residual factor considering different slope heights

<table>
<thead>
<tr>
<th>5m High Slopes</th>
<th>8.65m High Slopes</th>
<th>12m High Slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope Angle (°)</td>
<td># of Cycles</td>
<td>Residual Factor</td>
</tr>
<tr>
<td>59.0</td>
<td>12</td>
<td>0.21</td>
</tr>
<tr>
<td>51.3</td>
<td>16</td>
<td>0.22</td>
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<tr>
<td>45.0</td>
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<tr>
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<td>35.5</td>
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<td>32.0</td>
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<td>0.78</td>
</tr>
<tr>
<td>15.1</td>
<td>305</td>
<td>0.913</td>
</tr>
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</table>
Figure 5.53 Residual factors for different slope geometries and strain-softening relationships; a) residual factor at serviceability failure against slope angle; b) residual factor at serviceability failure against number of seasonal cycles to serviceability failure

Figure 5.53 shows the residual factor (i.e. the level of strength reduction relative to peak and residual or minimum strength) at serviceability failure (i.e. prior to acceleration of the failing soil mass) against slope angle and number of seasonal cycles to serviceability failure for different strain-softening relationships. Considering Figure 5.53a, the numerical analyses show that at steep slope angles for this material, (i.e. > 35°) very little strength reduction is required for failure to occur and as such, the three strain-softening relationships considered exhibit similar behaviour. This indicates that the peak strength of the material (which is the same for all analyses) dictates the failure behaviour of steep slopes.
As the slope angle reduces, the level of strength reduction required for serviceability failure increases and behaviours of the different strain-softening models contrast. As shown previously, the geometry of slopes at risk of failing through seasonal ratcheting for strain-softening option (A), are limited by the critical state friction angle (i.e. the minimum friction angle of the material). This is further supported within this plot (Figure 5.53a) as it can be seen that the residual factor for option (A) tends towards unity at a slope angle equal to the critical state friction angle.

Given the different minimum strength of strain-softening options (B) and (C) of residual friction angle at large strains, as the slope angle decreases, the residual factor tends towards unity at the residual friction angle but at different rates. Slopes modelled with strain-softening option (C) fail at lower residual factors (i.e. higher mobilised strength) than the same angled slope with strain-softening option (B). This suggests that the greater rate of strength reduction included in option (C) post-critical state has significant implications on the serviceable life of a slope and that large strain behaviour is extremely important in the assessment of slope behaviour due to seasonal stress cycles and progressive failure.

Figure 5.53b shows the residual factors and number of seasonal cycles to serviceability failure, the trends shown are logarithmic. The trends presented in Figure 5.53b can be considered in two groups, the behaviour associated with strain-softening (A) and that of options (B) and (C). As the relationships are logarithmic, for options (B) and (C), slopes that fail because of a residual factor greater than 0.6 (i.e. slopes at angles less than 22° from Figure 5.53a, the critical state friction angle) small reductions in slope angle will increase the serviceable life of a slope considerably.

As for the different strain-softening relationships, the results of the residual factors have been plotted for the different height slope analyses in Figure 5.54.
Unlike the different strain-softening models, Figure 5.54a shows the behaviour of seasonal ratcheting on slopes at steep angles is very different depending on the height of the slope. At slope angles less than the critical state friction angle, the trends in behaviour shown are very similar – all tending towards a residual factor of unity at the residual friction angle of the material modelled. The results suggest that the residual factor at which a slope at an angle less than critical state will fail due to seasonal ratcheting is independent of slope height.

As mentioned previously, at greater slope angles slope behaviour is highly dependent on the height of the slope. The lower the height of a slope the more
stable it will be at higher angles. If the slope angle that will fail at zero residual factor is considered for the three different slope heights, a 12m high slope at 34.0°; an 8.65m high slope at 45.5°; and a 5m high slope at 65.5° will fail instantaneously after construction. This a range of more than 30° due to the different slope heights.

Again, the relationship of residual factor and number of seasonal cycles to serviceability failure is logarithmic as can be seen in Figure 5.54b. two very different trends can be observed for the different height slopes considered. The 8.65m and 12m high slopes show very similar behaviour within Figure 5.54b. However, the 5m high slopes are very different showing the significant increase in number of seasonal cycles required for serviceability failure presented previously in Figure 5.51 and Figure 5.52.

5.8.4 Mobilised Friction Angle at Failure

As presented in Section 5.7.5, along with the residual factor at serviceability failure, the average friction angle mobilised along the critical shear surface has been considered for the different analyses. In the same way as the residual factors, average mobilised friction angle has been plotted against slope angle and number of seasonal cycles to serviceability failure. Considering mobilised friction angle allows strength properties, to be used within limit equilibrium analyses, and slope angle to be compared so that physical behaviour can be interpreted with reference to material properties.

Figure 5.55 shows the average mobilised friction angles at serviceability failure for strain-softening option (B) and (C). Figure 5.56 shows the average mobilised friction angle at serviceability failure for different slope heights.
Figure 5.55 Mobilised internal angle of friction at serviceability failure for different strain-softening relationships and slope geometries; a) mobilised internal angle of friction at serviceability failure against slope angle; b) mobilised internal angle of friction at serviceability failure against number of seasonal cycles to serviceability failure
Figure 5.56 Mobilised internal angle of friction at serviceability failure for slope angles and heights; a) mobilised internal angle of friction at serviceability failure against slope angle; b) mobilised internal angle of friction at serviceability failure against number of seasonal cycles to serviceability failure.

Figure 5.55 and Figure 5.56 allow material properties to be related to design life allowing recommendations to be made regarding the selection of strength parameters for limit equilibrium analyses. It can be seen very clearly from Figure 5.55b and Figure 5.56b that the use of friction angle greater than critical state within limit equilibrium analysis will result in a slope design that will fail after a very small number of seasonal stress cycles. However, a small reduction in the friction angle used in limit equilibrium below critical state friction angle yields far greater design lives.
To fully understand the appropriate design parameters to be adopted within limit equilibrium analysis for a slope experiencing seasonal ratcheting and progressive failure, the strain-softening behaviour of the soil must be known and the design life of the asset being designed must be established. For slopes that are unconditionally stable, design should consider residual friction angle. This approach will produce overly conservative and uneconomically viable slope designs. Design recommends are discussed further in Section 5.9.

5.9 Discussion

Within this Chapter, a numerical model capable of capturing seasonal ratcheting and progressive failure due to summer/winter boundary conditions has been validated against hydrological and mechanical data from physical modelling of two slopes (Take, 2003). The pore water pressure cycles, and therefore stress cycles, within the numerical analyses have been shown to match the results of the physical modelling within the near-surface and general trends in displacements are indicative and in the same order of magnitude as the behaviour observed experimentally.

Modelling of seasonal ratcheting and progressive failure has been conducted in an unsaturated two-phase flow numerical model with a nonlocal strain-softening model implemented. Material stiffness and saturated hydraulic conductivity have been included as a function of specific volume and void ratio respectively – making these variables stress dependent. The variability of void ratio throughout the slope models after wetting and drying have been shown indicating the significance of this physical change in volume with transient stress conditions.

The following sections discuss the results presented in this Chapter looking at numerical modelling of seasonal ratcheting and the study considering the effect of slope geometry on the mechanism of seasonal ratcheting and slope deterioration. The discussion follows the same order as the results presented, firstly considering the validation of the numerical modelling approach against physical modelling, a discussion of the numerical framework used, followed by the investigation into the effect of stiffness, strain-softening behaviour and the effect of slope geometry.
5.9.1 Numerical Model Validation against Physical Modelling

Two physical slope models of different initial stress conditions subjected to repeated wetting and drying cycles (WAT7a and WAT8a) conducted by Take (2003) have been used to allow the validation of a numerical modelling approach developed within this work. The methodology is presented in Section 5.2 and the results of a comparison of the physical behaviour and the numerical models are shown in Section 5.5.

As discussed in Section 5.2, the boundary conditions applied to the numerical models, and therefore the hydrogeological behaviour of the numerical models, have been selected to drive the same pore water pressure cycles within the high capacity tensiometer closest to the slope surface. This ensures that the stress cycles within the near-surface of the numerical analysis are representative of the physical modelling experimentation. A comparison of the pore water pressures at different locations in WAT7a are shown in Figure 5.16 and for WAT8a in Figure 5.21. From these figures, it can be seen that hydrogeological behaviour of a complete cycle, and therefore stress cycle, of the physical model is being captured within the numerical analyses. Whilst a complete stress cycle is captured well within the numerical analyses, there is a discrepancy in the wetting stress path between the numerical and physical modelling results. In all instances, wetting within the physical modelling is more pronounced with the slope model returning to near hydrostatic very quickly after wetting. The wetting within the numerical analyses is more gradual. This can be explained by assumptions within the numerical modelling approach and the fact that physical behaviours such as preferential flow (which is likely to be a significant problem between the slope model and sides of climate chamber) have not been included within the numerical model. One of the key behaviours of unsaturated soil that has been omitted for the numerical analysis due to a lack of measured data in the area is soil water retention hysteresis. In the future, soil water retention hysteresis can be easily included within the modelling approach considering the change in pore water pressures at a calculation point and updating the soil water retention properties of the material (see Figure 2.24) while-stepping through the use of a FISH function within FLAC. Drying and wetting behaviour of soils is very different, as shown in Section 2.4.6.4.
However, the stress conditions at the end of wetting and drying within the numerical analyses are representative of the stress conditions in the physical modelling at the same times.

Given that representative pore water pressure cycles compared to the physical models are driven within the numerical analyses, it can be assumed that the stress conditions due to pore water pressure changes within the numerical model represent those experienced within the physical model. It can therefore be postulated that if the stress cycles within the numerical model are close to reality then if the numerical model framework and material properties are correct then the mechanical behaviour of the numerical models will match that of the physical modelling. Undertaking a comparison of the mechanical behaviours of the numerical and physical models is critical to demonstrate that the numerical modelling approach developed can successfully replicate seasonal driven cycles of shrink-swell displacements, seasonal ratcheting, mobilisation of post-peak strength and progressive failure. It is the first time that a numerical model considering this mechanism of slope deterioration, in particular stress cycles leading to mobilisation of post-peak strength and progressive failure, has been validated against both hydrogeological and mechanical behaviour of a monitored slope.

The mechanical behaviour of WAT7a is shown in Figure 5.18 and WAT8a in Figure 5.23. The results show that the numerical modelling approach developed captures key behaviour associated with seasonal ratcheting. In Figure 5.18, mainly vertical movement with a small amount of horizontal movement is observed at the crest of the slope and conversely, mainly horizontal with little horizontal movement at the toe of the slope. The magnitudes of the displacements observed in the numerical analyses are of the same magnitude as the physical modelling, especially if the displacements of a complete wetting and drying seasonal cycle are considered. However, it is apparent that there is greater vertical movement observed in the numerical models than the physical models. The difference can be explained by considering how the stress cycles within the numerical analyses have been obtained.
Considering Figure 5.16, the boundary conditions within the numerical analysis have been selected to drive the pore water pressure cycles at HCT1. Because of the difficulties associated with modelling an unsaturated permeable material, the pore water pressure cycles at depth in the numerical model do not match the physical modelling as closely as the near-surface. This can be attributed to spatial variability and preferential flow increasing the hydraulic conductivity within the real soil which has not been included within the numerical modelling along with soil water retention hysteresis.

Given this difference in hydraulic conductivity between the numerical and physical models, to achieve the pore water pressure suctions at HCT1 in the numerical model that match the physical modelling, the boundary conditions imposed will have to create a greater suction at the slope surface. As HCT1 is approximately 1.0m below the slope surface (at full scale) and the boundary conditions in the numerical model will produce greater suctions at the slope surface, where the displacement of the numerical and physical models are compared. Therefore, it is not surprising that greater vertical displacement is observed in the numerical modelling approach.

The shear strain contour plots for WAT7a at waypoint G and H are shown in Figure 5.21. At G there is very little shear strain accumulation, and therefore softening observed in the numerical analysis. However, following prolonged wetting (G-H) the amount of shear stain accumulation within the slope is considerable. This demonstrates the slow accumulation of shear strains due to repeated seasonal wetting and drying and seasonal ratcheting up to G and the significance of prolonged wet periods on the global condition of a slope. Figure 5.21 clearly illustrates progressive failure due to seasonal ratcheting and the onset of softening at the toe of the slope propagating through the slope (Skempton, 1964; Potts, et al., 1997; Leroueil, 2001).

From the results presented, for hydrogeological and mechanical behaviour of two different slopes, it can be concluded that the numerical modelling approach developed is sufficient for the modelling of weather-driven season ratcheting behaviour and progressive failure.
5.9.2 The Numerical Framework

Within this work, a two-phase flow numerical framework coupling Bishop’s generalised effective stress and van Genuchten (1980) style soil water retention properties has been utilised. It has been shown that when the correct material properties and boundary conditions are used, the hydrogeological and mechanical behaviour of high-plasticity overconsolidated clay slopes subjected to repeated wetting and drying stress cycles can be replicated within this framework.

As discussed in 2.4.3.2, there are alternate, more sophisticated methods for modelling unsaturated behaviour of soil considering multiple independent stress states (Fredlund & Morgensten, 1977; Alonso, et al., 1990). However, it has been shown that this framework can capture the desired behaviour and when making decisions regarding numerical modelling approaches, the user should aim to use the simplest approach that allows the critical mechanism to be captured. This framework will therefore be used in further work within this project considering seasonal ratcheting and weather-driven behaviour.

5.9.3 Effect of Material Stiffness on Seasonal Ratcheting

The results presented in Section 5.6 have shown that the stiffness of a slope greatly influences the magnitude of seasonal displacements for the same stress cycles. The numerical analyses show that annual displacements due to seasonal wetting and drying are greater for a less stiff material resulting in quicker accumulation of displacements and shear strains, strain-softening, and ultimately failure after fewer seasonal cycles.

Material stiffness is an interesting material property, within this study the stiffness within the numerical analyses is related to the generalised effective stress whilst-stepping and therefore is updated throughout the analysis. This is a major simplification of material stiffness in reality. It is known, that stiffness deterioration under repeated cyclic loading is a problem. It is therefore important that more is known about stiffness deterioration due to repeated wetting and drying as this will change the material properties of the near-surface zone, especially when the number of seasonal cycles a slope experiences becomes very high.
5.9.4 Geometric Study

Slopes of different slope angle, height and different strain-softening behaviours have been modelled considering uniform controlled boundary conditions, as described in Section 5.7. The majority of the failures, ignoring slopes that failed after very few seasonal cycles, observed in this study are shallow, short, and translational in nature indicative of failures seen in ageing infrastructure slopes. Briggs, et al., (2017) discusses failures occurring in high-plasticity overconsolidated clay slopes at a depth of approximately 1.5m due to repeated seasonal cycles of pore water pressures.

The boundary conditions applied in this study are the same for all slopes and omit the effects of extreme wet periods. Extreme wet periods have been previously shown in Section 5.5 (i.e. through prolonged wetting of the slope model) to be extremely influential on the overall condition of a slope. The results of the numerical analyses presented show that repeated cycles of wetting and drying cause seasonal ratcheting leading to strength deterioration. This mechanism provides an explanation to why a slope that appears stable will not fail during a particular wet period but after a number of years fails due to a wet period that is the same or not as extreme as previously experienced.

Within this study, a method for establishing the point at which a slope becomes unstable due to seasonal ratcheting and strength deterioration has been presented. It has allowed the serviceability failure point of all slopes to be obtained in a repeatable and robust way, this has been done by considering the reciprocal of velocity of the failing mass as described in Section 5.7.4. Coupling this, the point of serviceability failure, with the residual factor and average mobilised friction angle along the critical shear surface (see Section 5.7.5) it has been possible to consider mobilised strength at serviceability failure (i.e. level of strength deterioration) and design life of these slopes.

From the observations made throughout Section 5.8.1, it is clear that strain-softening behaviour and the rate of strength reduction post-peak and post-critical state has significant implications on the design life of slopes experiencing seasonal ratcheting.
A significant observation from this work was the fact that the numerical analyses predicted a slope at an angle slightly greater than the minimum friction angle for strain-softening option (A) did not fail due to seasonal ratcheting. As such it can be suggested that slopes constructed at angle less than the minimum friction angle of the soil are not at risk of failure due to seasonal ratcheting. Whilst this statement may seem obvious, the fact that the numerical analysis exhibits this behaviour improves confidence in the numerical modelling approach.

Figure 5.53 to Figure 5.56 show logarithmic relationships between residual factor / average mobilised friction angle and number of seasonal cycles to serviceability failure. For slope angles failing at average mobilised friction angles below critical state friction angle of the soil (i.e. high residual factors, > 0.7) small reductions in slope angle will lead to a considerable increase in serviceable design life of the slope. Conversely, slopes constructed at angles greater than the critical state friction angle fail at near peak strength with very little strength reduction and therefore the design life of such assets is short. Figure 5.40 and Figure 5.41 show that the rate of post-critical state strength reduction to residual strength has significant implications on slope design life for slopes constructed at angles less than critical state. At steeper angles relative to material strength, slope behaviour is a function of peak strength only and post-peak behaviour has little effect on the design life of the slope (see Figure 5.40). It has been shown that slope height has a large influence on the design life of slopes experiencing seasonal ratcheting, Figure 5.51 and Figure 5.52 show that for slopes of the same angle, a 5m high slope will remain stable for a considerably higher number of seasonal cycles than a 12m high slope.

5.9.4.1 Implications for Design

The results of this study show the significance of understanding the shear behaviour of soils (i.e. the rate of strength reduction post-peak and post-critical state) in which slopes are constructed and in which shallow first-time failures due to repeated seasonal wetting and drying will occur. Whilst design guidelines for limit equilibrium analysis suggest fully softened strength parameters are adequate for the assessment of deep-seated first-time failures
in overconsolidated clay slopes (Chandler & Skempton, 1974), the failures observed in this study are shallow and the use of fully softened strength parameters to assess these failures in limit equilibrium analysis can result in unconservative design leading to increased risk of failure prior to the desired design life.

For deep-seated failures in overconsolidated clays, Skempton (1964) concluded that slopes designed considering the minimum (i.e. residual) friction angle of the soil will not fail. The results of the numerical analyses presented here indicate that this is also true for shallow first-time failures in high-plasticity overconsolidated clay slopes.

The results presented in Figure 5.55 to Figure 5.56 have been summarised in Figure 5.57 to allow easy reference throughout this discussion.
Observations from Figure 5.57:

- there is a threshold for the level of strength reduction at which such little strength reduction is required that failure occurs after very few seasonal cycles and therefore, the slope geometry is unstable within the soil modelled;
- conversely, for shallower slope angles the level of strength reduction required for failure requires a large number of seasonal cycles such that the serviceable life of the slope will be far in excess of the desired design life. It can therefore be suggested that designing all slopes at
risk of shallow first-time failure due to seasonal ratcheting at an angle equal to or less than the residual friction angle of the soil is overly conservative;

- the difference in design life for the 8.65m high slopes strain-softening (B) and (C) is considerable. Therefore, the rate of post-critical state strength reduction is critical in the choice of design parameters to be used within limit equilibrium analysis;

- slope height is an important factor when assessing design life of slopes experiencing seasonal ratcheting and selecting design parameters. Lower height slopes are much more stable; and

- it has been established that residual friction angle limits the slopes that will fail due to seasonal ratcheting and progressive failure. However, Figure 5.57b shows that selecting residual friction angle for design purposes will result in a slope with a far greater design life than is required in practice (i.e. > 120 years);

Using the results presented, the selection of design parameters for limit equilibrium analysis have been considered in two parts, firstly through the consideration of design life and the effect that increasing design life has on the required geometry of a slope for stability and then in terms of mobilised strength to achieve the desired design life.

5.9.4.1.1 Effect of Slope Geometry on Design Life

Considering slopes in the context of the highways network in the UK, standard design lives of slope assets can be up to 120 years (The Highways Agency, 2001). Typically, 60 and 120 year design lives are used within design. Using Figure 5.57, it is possible to obtain the slope angle of slopes that will remain stable for these design lives for 8.65m and 12m high slopes and for a material that softens to residual friction angle at high strains (i.e. strain-softening option B). Using Figure 5.57b,

The average mobilised friction angle for 60 and 120 seasonal cycles considering 8.65m and 12m high slopes, are 20.1° and 19.0° respectively. Considering Figure 5.57a and these mobilised friction angles, the slope angles
corresponding to a serviceable life of 60 and 120 seasonal cycles can be obtained. These have been summarised in Table 5.18.

<table>
<thead>
<tr>
<th>Slope Angle (°)</th>
<th>8.65m High Slope</th>
<th>12m High Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>60 Seasonal Cycles</td>
<td>22.7</td>
<td>21.1</td>
</tr>
<tr>
<td>120 Seasonal Cycles</td>
<td>20.1</td>
<td>18.6</td>
</tr>
<tr>
<td>Difference</td>
<td>2.6</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The results shown in Table 5.18 illustrate the slope angle reduction required to extend the serviceable life of a slope constructed within this material from 60 seasonal cycles to 120. In terms of design of these hypothetical slopes, the difference in the land take required to extend the design life for 60 to 120 seasonal cycles would be 3.0m and 4.6m for a single slope (i.e. 6.0m and 9.2m for two slopes) for the 8.65m and 12m high slopes respectively.

An extra 5 to 10m of land take over the length of an infrastructure route that can be many kilometres will have significant economic and political implications for any project. Understanding how design life and slope angle are linked can facilitate more effective cost benefit analysis for long linear infrastructure projects.

**5.9.4.1.2 Mobilised Strength for Required Design Life**

Defining the relevant mobilised strength that should be considered in limit equilibrium analysis for a defined serviceable life is difficult due to the variability shown for strain-softening behaviour, slope height and the fact that the models considered here are subjected to very simplistic uniform seasonal boundary conditions. However, using the results obtained general guidance has been formulated.

As discussed previously, material property selection is highly dependent on the required design life. For temporary slopes (i.e. up to a couple of years) that are not particularly high, bulk peak strength can be considered as long as no structural discontinuities or softened zones exist. From the results presented, it is unlikely seasonal ratcheting will be the critical mechanism for consideration when designing temporary slopes as numerous stress cycles leading to shear strain accumulation and progressive failure will not have occurred.
Mechanisms such as undrained unloading and excess pore water pressure dissipation in cut slopes will be much more prominent mechanisms within the short timeframe of these slopes.

At the other end of the spectrum, slopes that require very long design lives (i.e. perpetually stable) or the consequence of failure is so high that all risk should be mitigated should be designed considering residual strength within limit equilibrium analysis.

The difficulty is in the selection of design parameters for slopes in between these criteria as there is a balance to be struck between the risk of failure, the cost of construction of an asset and the potential consequence of failure. Take and Bolton (2011) previously hypothesised that slopes constructed at an angle less than the critical state friction angle (i.e. critical state strength used in limit equilibrium analysis) of the soil will result in a slope with a reasonable design life considering failure due to seasonal ratcheting. For the material and scenarios considered these design parameters will produce a slope with a design life ranging between 8 and 42 seasonal cycles. For all analyses considered to produce design lives of greater than 60 seasonal cycles an average mobilised friction angle of 18.6° would be required. If results from models considering strain-softening (C) are omitted, the average mobilised friction angle would increase to 20.1°. These mobilised friction angles are 3.4° and 1.9° less than the critical state friction angle.

With an appreciation of the limitations in the models considered and modelling approach developed (see Section 5.9.4.2), it can be suggested that in general, for a design life of 60 seasonal cycles material strength parameters of less than critical state friction angle (i.e. minus 2 to 4°) but not residual should be considered in limit equilibrium analysis for a material that reaches residual strength at very high strains (i.e. > 100% strain indicative of strain-softening option B). For soils that soften much more rapidly to residual, friction angles of slightly more than residual friction angle should be considered (i.e. plus 2 to 4°).
Understanding the rate of strength reduction of a material, the swelling potential, the stiffness of the material and undertaking a similar geometric study as presented here will provide more accurate strength parameter suggestions for design. As with any design process, the material properties suggested here for limit equilibrium analysis will require factoring in line with the design approach being used to account for uncertainty in the material properties and the limit equilibrium analysis methodology.

5.9.4.2 Limitations within the Study and Outputs
The limitations of this work can be considered in two parts; the limitations associated with the numerical modelling approach and the limitations of the outputs (i.e. implications for design) presented on the design of slopes in other overconsolidated clay soils.

Within this study, an idealised homogeneous material and slopes with simple stress history have been considered. Seasonal ratcheting has been driven by uniform controlled wetting and drying boundary conditions without any extreme (i.e. prolonged) wet periods. Real engineered slopes have complex stress histories and spatial variability of material properties, in some instances (i.e. structural discontinuities, preferential flow routes, etc.) these variabilities in material properties will control failure of these slopes. In addition, real slopes experience weather-driven behaviour that is highly variable with frequent extreme wetting and drying events that have not been considered. The frequency and duration of these events are likely to change in the future as highlighted in climate change projections.

Decisions to omit the variability of boundary conditions and extreme events were taken to ensure that is study considered trends in strength deterioration considering slope geometry. However, the omission of extreme events does bring into question the reliability of the conclusions drawn regarding the time to failure and hence design parameters suggested for use in limit equilibrium analysis from this study.

As previously discussed, stiffness deterioration due to cyclic loading is a known problem but has not been included within this study again adding
uncertainty to the average mobilised strength suggested for particular design lives. The slopes modelled in this study are not vegetated, and in reality, vegetation introduces roots that change the hydrogeological and strength properties of the near-surface zone which will affect seasonal ratcheting behaviour.

For first-time deep-seated failures in cut slopes, Chandler and Skempton (1974) showed that slopes formed in London Clay and Lias Clay failed at similar pore water pressure conditions and fully softened strength, which are comparable (i.e. \( c' = 1.5 \text{kPa} \) \( \varphi' = 20^\circ \) for London Clay and \( c' = 1.5 \text{kPa} \) \( \varphi' = 23^\circ \) for brecciated Upper Lias Clay). Behaviour of one clay is similar to that of other clays so it can be suggested that for shallow first-time failures in high-plasticity overconsolidated clay slopes due to seasonal ratcheting, the lessons learnt from this study are applicable to other similar clays.

### 5.10 Key Findings

From the work presented within this Chapter, the following key findings have been obtained and summarised;

- The numerical modelling approach developed, consisting of unsaturated soil behaviour and nonlocal strain-softening, has enabled seasonal ratcheting deformations and progressive failure due to known near-surface stress cycles to be modelled numerically and validated against physical modelling data;
- Through parametric investigation and the validation performed, it has been shown that it is the frequency and duration at which near hydrostatic conditions are reached that dictate the rate of strength deterioration of high-plasticity clay slopes due to seasonal stress cycles;
- Material stiffness has been shown to be a significant parameter in slope behaviour due to seasonal stress cycles, the stiffer a material, the smaller the annual deformation and rate of accumulation of plastic strains and therefore softening;
- Understanding strain-softening behaviour of a material is important in terms of establishing the serviceable life of earthwork assets. Slopes constructed at an angle less than the residual friction angle of a material
will not suffer failure due to seasonal ratcheting. Slopes constructed in materials that soften to residual under relatively low strains are highly susceptible to failure due to seasonal ratcheting;

- Slope height is a critical factor in determining the serviceable life of slopes that suffer failure due to seasonal ratcheting, with lower height slopes remain stable for a considerably greater number of seasonal stress cycles than higher slopes of the same angle.

5.11 Chapter Summary

The numerical modelling approach and framework used (i.e. two-phase flow analysis considering Bishop’s generalised effective stress coupled with a van Genuchten (1980) style soil water retention curve with nonlocal strain-softening) has been validated against physical modelling behaviour of two slopes. The hydrogeological and mechanical behaviour of the numerical and physical models have been compared in detail.

The results of the numerical analyses presented provide clear evidence of strength deterioration with seasonal stress cycles driven by wetting and drying and why real slopes that appear stable following one extreme wet event fail a number of years later due to the same or a smaller magnitude wet event. The significance of prolonged wet events on the condition of a slope has been illustrated. It is the frequency and time spent at near hydrostatic conditions that dictates the rate of strength deterioration due to seasonal ratcheting.

The effect of material stiffness on seasonal ratcheting was considered and it was shown that stiffer materials will remain stable for more seasonal cycles. It also demonstrated the importance in understanding stiffness deterioration due to cyclic loading.

Conclusive evidence showing that the residual friction angle of a soil dictates the slope geometries that are at risk of failure due to seasonal ratcheting has been presented. The importance of understanding post-peak and post-critical state strength reduction to appreciate the most at-risk slopes of failure due to seasonal ratcheting to gain an appreciation of serviceable design life for a particular material has been demonstrated. The implications of the geometric
study on design have been presented and the limitation of the study discussed. Critically, strength parameter selection for limit equilibrium analysis is dependent on the required design life of a slope and the potential consequence of failure. Slopes constructed at angles at or less than the residual friction angle of the soil will not failure due to seasonal ratcheting. From the results presented it can be suggested that a friction angle of less than critical state but not residual should be used in limit equilibrium analysis of slopes for design lives of approximately 60 seasonal cycles. The more rapid the rate of post-critical state strength reduction with strain-softening the closer to residual strength the friction angle should be for design.
Chapter 6

Modelling Cut Slope Behaviour with Seasonal Boundary Conditions

6.1 Chapter Scope

The following Chapter presents the method developed to allow undrained unloading, stress relief, pore water pressure equilibration, concurrent weather-driven seasonal ratcheting and progressive failure to be captured within a single numerical model. The method is described in Section 6.2. For completeness the modelling development and outputs of this phase of modelling are shown in Figure 6.1.

![Figure 6.1 Phase 3 model development and outputs](image)

Results are presented to show the evolution of the modelling approach and validate the correct mechanisms are being captured within the numerical models. This has been done by first considering simplified cut slope behaviour (i.e. undrained unloading, stress relief, pore water pressure equilibration and progressive failure under constant boundary conditions) in a saturated framework and the same model in two-phase flow (i.e. unsaturated) framework as described in Section 6.2.2. Following this, simplified cut slope behaviour is
considered in a coupled modelling approach to allow undrained unloading and construction effects to be modelled adequately within the two-phase flow framework. After validating simplified cut slope behaviour modelled in an unsaturated framework, models with seasonal boundary conditions have been run and the implications of hydrogeological deterioration, climate change and initial stress conditions have been investigated.

6.2 Methodology (Phase 3) – Cut Slope Behaviour with Seasonal Boundary Conditions

To allow meaningful numerical analysis of inter-related strength deterioration mechanisms, it is imperative that all critical physical processes can be captured within a single numerical analysis. The stress history, construction effects along with seasonal wetting and drying are fundamental to the long-term performance of cut slopes in overconsolidated clay.

To capture time-dependent strength deterioration, processes associated with excavation and surface processes (i.e. weather-driven behaviour) must be considered simultaneously. This section presents the methodology developed to allow undrained unloading, stress relief, excess pore water pressure dissipation with weather-driven behaviour to be modelled within a single boundary-value problem.

6.2.1 Model Validation Approach

An approach that allows relevant strength deterioration mechanisms within a single numerical model has been developed. These mechanisms include; undrained unloading, stress relief, excess pore water pressure dissipation, seasonal ratcheting and progressive failure.

To model seasonal ratcheting due to wetting and drying, unsaturated soil behaviour must be included within the numerical framework. As discussed previously, the two-phase flow framework being used within this work allows unsaturated behaviour and saturated behaviour to be modelled. The single-phase flow code being used includes Biot’s consolidation theory allowing post-excavation pore pressures to be calculated. However, it is assumed that a phreatic surface exists within the model, below this the soil is fully saturated.
and above it is completely unsaturated, and suctions exist due to water tension within the model. Seasonal ratcheting cannot be modelled sufficiently within the saturated framework.

The modelling conducted within this phase, and validation of modelling approach developed has been broken into three stages;

1. Simplified cut slope behaviour – undrained unloading, stress relief, excess pore water pressure dissipation and progressive failure under constant boundary conditions modelled in single and two-phase flow framework;
2. Simplified cut slope behaviour – undrained unloading, stress relief, excess pore water pressure dissipation and progressive failure under constant boundary conditions modelled in single and coupled single and two-phase flow frameworks;
3. Cut slope behaviour with seasonal boundary conditions – using the coupled approach, undrained unloading, stress relief, excess pore water pressure dissipation, seasonal ratcheting and progressive failure have been modelled using simple summer and winter boundary conditions. Within this stage, hydrogeological deterioration, initial stress conditions and the potential implications of climate change have also been considered.

The analyses run within this phase of modelling are summarised in Table 6.1, details of the analyses, material properties and boundary conditions are discussed in the following section. All slope models consider London Clay and slopes 10m high at 1 in 3, mesh A (see Section 3.5) has been used throughout.
Initially, to assess the different modelling codes (i.e. single-phase and two-phase flow approaches) the same simplified cut slope model with constant boundary conditions has been considered in both frameworks. Following this, the differences in behaviour have been assessed. The results of this comparison are shown in Section 6.3. It was shown that the time to failure for the different modelling approaches were 24.3 years within the saturated model (SP-A1) and 30.7 years in the unsaturated model (TP-A1). Considering the same mesh, material properties (with the additional parameters required for an unsaturated model – air phase properties and soil water retention characteristics) and boundary conditions were used in both models a difference of 26.3% in the time to failure is considerable. The behaviour modelled in this exercise, undrained unloading, dissipation of excess pore water pressures and stress relief are saturated processes; therefore, the baseline results for this behaviour are the results of the saturated numerical analysis (SP-A1).

To overcome the problem of unrealistic time to failure when considering simplified cut slope behaviour within the two-phase flow framework; a coupling of a single and two-phase model has been undertaken. Within this approach, post-excavation pore water pressures, stresses and strains from a single-phase flow analysis have been transferred to a two-phase flow model of the same mesh configuration and properties (with the additional parameters

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Table 6.1 Summary of numerical analyses run – Phase 3

<table>
<thead>
<tr>
<th>Reference</th>
<th>Mechanisms</th>
<th>Numerical Framework</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-A1</td>
<td>Simplified cut slope behaviour</td>
<td>Saturated</td>
<td>(1)</td>
</tr>
<tr>
<td>TP-A1</td>
<td>Simplified cut slope behaviour</td>
<td>Two-Phase Flow</td>
<td>(1)</td>
</tr>
<tr>
<td>TP-A2</td>
<td>Simplified cut slope behaviour</td>
<td>Two-Phase Flow</td>
<td>(1) (2)</td>
</tr>
<tr>
<td>TP-B1</td>
<td>Cut slope behaviour with seasonal boundary conditions</td>
<td>Two-Phase Flow</td>
<td>(2) (3)</td>
</tr>
<tr>
<td>TP-B2</td>
<td>Cut slope behaviour with seasonal boundary conditions</td>
<td>Two-Phase Flow</td>
<td>(2) (3) (4)</td>
</tr>
<tr>
<td>TP-B2-CC</td>
<td>Cut slope behaviour with seasonal boundary conditions</td>
<td>Two-Phase Flow</td>
<td>(2) (4) (5)</td>
</tr>
</tbody>
</table>

(1) Constant 10kPa suction boundary conditions;
(2) Post-excavation saturated conditions transferred;
(3) 70kPa seasonal pore water pressure cycles at mid-slope surface boundary conditions;
(4) Inclusion of higher near-surface saturated hydraulic conductivity;
(5) Wetter winters and drier summers reflected in boundary conditions – as suggested by UKCP09.
required for an unsaturated model – air phase properties and soil water retention characteristics). Post-excavation pore water pressures, stresses and strains are then taken as the initial conditions for the two-phase flow analysis so that undrained unloading can be captured in line with the results of the saturated analysis. This is explained in more detail in Section 6.2.2 and 6.2.3.

6.2.2 Stage 1 – Simplified Cut Slope Behaviour

Adopting the same modelling approach as described in Section 4.2.1, a single-phase flow numerical model of cut slope behaviour including undrained unloading, stress relief, excess pore water pressure dissipation and progressive failure under constant boundary conditions has been considered (model reference: SP-A1). The modelling approach used has been validated against work done by Potts, et al., (1997) (see Section 4.3). Therefore, the results of this single-phase flow model have been taken as the baseline results for the first stage of this model validation.

The same modelling setup and sequencing as outline in Section 4.2.1 has been used to model a slope of the same geometry, mesh and material properties (with the addition of an air modulus and soil water retention properties) in an unsaturated framework (model reference TP-A1).

For the comparison of these modelling approaches, a London Clay cut slope 10m high at 1 in 3 has been considered with constant 10kPa suction boundary conditions. Mesh A (see Section 3.5) has been employed along with a nonlocal strain-softening model with an internal length of 1.0m (see Section 3.3.2). Material properties are summarised in Table 6.2 and the depth dependent saturated hydraulic conductivity profile used is shown in Figure 6.2.
Table 6.2 Summary of London Clay material properties

<table>
<thead>
<tr>
<th>London Clay Strain-Softening Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Nonlocal Plastic</td>
<td>0.012</td>
</tr>
<tr>
<td>Large strain Nonlocal Plastic Strain</td>
<td>0.055</td>
</tr>
<tr>
<td>Internal Length (m)</td>
<td>1.00</td>
</tr>
<tr>
<td>Peak Cohesion (kPa)</td>
<td>7 †</td>
</tr>
<tr>
<td>Large Strain Cohesion (kPa)</td>
<td>2 †</td>
</tr>
<tr>
<td>Peak Friction Angle (°)</td>
<td>20 †</td>
</tr>
<tr>
<td>Large Strain Friction Angle (°)</td>
<td>13 †</td>
</tr>
<tr>
<td>Dilation Angle (°)</td>
<td>0</td>
</tr>
</tbody>
</table>

London Clay Stiffness Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus, E (kPa)</td>
<td>25(σ'+100) * †; min 5000</td>
</tr>
<tr>
<td>Bulk Modulus, K (kPa)</td>
<td>$E/3(1-2\nu)$</td>
</tr>
<tr>
<td>Shear Modulus, G (kPa)</td>
<td>$E/2(1+\nu)$</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td>0.2 †</td>
</tr>
</tbody>
</table>

Other

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Density, $\gamma$ (kN/m³)</td>
<td>18.8 †</td>
</tr>
<tr>
<td>Coefficient of Earth Pressure at Rest, $K_0$</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Soil Water Retention Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>van Genuchten - n</td>
<td>1.18 †</td>
</tr>
<tr>
<td>van Genuchten - m</td>
<td>0.153 †</td>
</tr>
<tr>
<td>van Genuchten - $\alpha$ (kPa)</td>
<td>125 †</td>
</tr>
</tbody>
</table>

* $\sigma'$ = effective stress (kPa) (single-phase flow model)
* $\sigma'$ = Bishop’s generalised effective stress (kPa) (two-phase flow model)
† (Potts, et al., 1997)
‡ (Briggs, et al., 2013)

Figure 6.2 Depth dependent saturated hydraulic conductivity profile for London Clay


Stress changes due to pore water pressure equilibration at a point 1m below the toe of the slope and mid-slope horizontal displacement were monitored throughout the analyses and the results of SP-A1 and TP-A1 compared (these are presented in Section 6.3).
Stage 2 – Simplified Cut Slope Behaviour (Coupled Model)

As mentioned previously there were considerable discrepancies between the results of SP-A1 and TP-A1. Therefore, to overcome the differences in the models, a coupling of a single and a two-phase flow model has then been undertaken, where post-excavation conditions have been passed from the single to the two-phase flow model. A direct comparison of behaviour of the single and coupled model (model reference: TP-A2) with constant 10kPa suction boundary conditions has been conducted to ensure the coupled model is capturing stress relief, dissipation of post-excavation pore water pressures and progressive failure sufficiently.

Again, a 10m high London Clay cut slope at 1 in 3 has been considered and all material properties are as described in Section 6.2.2. Following the transfer of post-excavation conditions to the two-phase flow model, the model has been fixed as fully saturated to ensure the effects of undrained unloading (which is saturated behaviour) are modelled correctly. The modelling procedure is shown schematically in Figure 6.3.

Ideally, the coupled approach would not be required, and simplified cut slope behaviour would be captured within the two-phase flow analyses. However, this is not the case and the model cannot correctly replicate this behaviour adequately. Therefore, additional work to couple a saturated and unsaturated framework model has been undertaken to overcome this limitation.
As with the previous stage, stress changes and mid-slope displacements have been compared for SP-A1 and TP-A2, this can be seen in Section 6.4.

### 6.2.4 Stage 3 – Cut Slope Behaviour with Seasonal Boundary Conditions

Using the same slope geometry, mesh, material properties and coupling process described previously (shown in Figure 6.3), the post-excavation conditions have been taken as the initial conditions for the two-phase flow model but with the addition of wetting and drying boundary conditions applied (model reference TP-B1). For a grass-covered London Clay cut slope, Smethurst, et al. (2012) monitored seasonal cycles of soil water content and pore water pressures in the near-surface. It was shown that weathered London Clay desaturates at pore water pressure suctions significantly less than the cited air entry value for intact London Clay of 1MPa (Croney, 1977). At depth the soil remains saturated, but the near-surface desaturates during drying.
To accommodate this behaviour and ensure the correct modelling approach is being used, initially the model is set as fully saturated throughout. However, in this scenario, the near-surface is allowed to desaturate due to the boundary conditions imposed. This means that dissipation of excess pore water pressures can be modelled at depth within saturated soil and seasonal ratcheting of the near-surface can be modelled using an unsaturated framework as validated against physical modelling in Chapter 5.

In line with observations of seasonal pore water pressure cycles in a grass covered London Clay cut slope; where Smethurst, et al., (2012) monitored cycles of 0kPa in the winter and 50 to 70kPa suction in the summer in the near-surface of the slope (approximately 1.0m below slope surface). Summer and winter discharge boundary conditions have been applied to the model straight after the transfer process to drive 70kPa annual cycles along the slope model boundary. Summer boundary conditions have been applied for seven months and winter boundary conditions for five months, at the end of summer the pore water pressures at the mid-slope surface are 70kPa suctions and 0kPa for the end of winter. Due to changing conditions within the slope model due to excess pore water pressure dissipation and the region of the near-surface that desaturates, the discharge boundary conditions are adjusted after each annual cycle (i.e. if the pore water pressures at mid-slope surface do not reach 70kPa suctions at the end of summer the discharge boundary condition will be reduced). Extreme events have not been included within this modelling as the focus of this work is on long-term strength deterioration and not triggering behaviour. The boundary conditions for this model are shown in Figure 6.4.

Due to the complex stress conditions within the slope model changing with time, the uniform discharge boundary conditions can push parts of the slope model past hydrostatic conditions; i.e. water is forced into the slope even though it is fully saturated. To avoid this, limiting conditions of hydrostatic have been imposed along the boundary surface and when hydrostatic conditions are reached the inflow discharge boundary condition is fixed to a small flow rate to maintain hydrostatic whilst not imposing unrealistic conditions. To implement this limiting condition, the toe, slope and crest of the model have been broken into 19 sections and the pore water pressures along these
sections are monitored throughout the analysis. If the pore water pressures exceed 0kPa along any of the sections, then the boundary conditions along that section are changed to the limiting condition. Limiting behaviour to hydrostatic has been considered in previous modelling of partially saturated slopes (Smith, 2003). In reality at this point, the excess water not input into the slope due to the soil being fully saturated would runoff. As such, this limiting condition is valid.

Again, the results of this analysis (TP-B1) have been compared with the previous analysis of simplified cut slope behaviour in a single-phase flow model (SP-A1) to show the effect of seasonal boundary conditions on slope behaviour, see Section 6.5.

### 6.2.4.1 Considering Hydrogeological Deterioration

Developing on and addressing one of the main limitations of the previous analysis, TP-B1, another analysis, again using the coupled modelling approach considering the same London Clay cut slope with the same boundary conditions, has been run with increased near-surface saturated hydraulic conductivity (model reference: TP-B2). This has been done to assess the influence of hydrogeological deterioration of the surface of a cut slope following stress relief, opening fissures and due to repeated wetting and drying leading to weathering of the slope surface.

From *in-situ* measurements for a London Clay cut slope, it has been shown that there is a significant increase in the saturated hydraulic conductivity compared to what would have been expected post-excavation for a slope excavated just over 20 years ago (Dixon, et al., 2018). To account for this in analysis TP-B2, the saturated hydraulic conductivity of the upper 3.0m of the
The results of this model, TP-B2, have been compared against TP-B1 and SP-A1 in Section 6.5.1 to show the effect of near-surface hydrogeological deterioration.

### 6.2.4.2 Considering Climate Change

General trends in climate change projections for the UK (UK Climate Change Projects 2009 (UKCP09) (Murphy, et al., 2009)) suggest wetter winters and drier summers. Given the simple summer and winter boundary conditions currently being applied to the models, this trend of behaviour has been investigated to develop understanding of overconsolidated clay cut slope with climate change.

The method used to assess changing summer and winter behaviour does not fit into a conventional framework for climate change assessment, as this would require more sophisticated boundary conditions and for analyses to be run in a probabilistic manner. However, the potential significance of climate change considering headline trends in UKCP09 has been assessed.

To replicate general climate change projection trends for the UK model TP-B2 has been considered up to the end of the 15th seasonal cycle and then boundary conditions of 0kPa in the winter and 70kPa increasing to 90kPa suction in the summer, along with increased time spent at near hydrostatic conditions during winter have been modelled to replicate climate change (model reference TP-B2-CC). The change in boundary conditions at the end of the 15th seasonal cycle has been used to demonstrate the deviation of behaviour from a ‘steady climate’ (i.e. TP-B2) to a changing climate (i.e. TP-B2-CC).

The results of this analysis TP-B2-CC have been compared against the steady climate scenario of TP-B2 in Section 6.5.2.
6.2.5 Effect of Initial Stress Conditions

Potts, et al., (1997) showed that the initial stress conditions in a slope model has significant implications on the time to failure when simplified cut slope behaviour is modelled. The conclusions drawn were for deep-seated mechanisms due to undrained unloading, excess pore water pressure dissipation and stress relief. The work here builds on this work by including seasonal boundary conditions.

In order to assess the effect of initial stress conditions, numerical analyses considering the same slope and material properties have been run considering different coefficient of earth pressure at rest values 1.0, 1.5 and 2.0. The different values of stiffness used in the analyses are summarised in Table 6.3 as used in the models validated against Potts, et al., (1997) work.

<table>
<thead>
<tr>
<th>Coefficient of earth pressure at rest, $k_0$</th>
<th>Effective Stress Range, $p'$ (kPa)</th>
<th>Stiffness Multiplier, $\lambda$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0 to 70</td>
<td>11.0</td>
</tr>
<tr>
<td></td>
<td>&gt;70</td>
<td>30.0</td>
</tr>
<tr>
<td>1.5</td>
<td>0 to 115</td>
<td>11.5</td>
</tr>
<tr>
<td></td>
<td>&gt;115</td>
<td>25.0</td>
</tr>
<tr>
<td>2.0</td>
<td>0 to 100</td>
<td>14.0</td>
</tr>
<tr>
<td></td>
<td>&gt;100</td>
<td>11.0</td>
</tr>
</tbody>
</table>

Results are compared for models equivalent to SP-A1 and TP-B2 (i.e. with uniform 70kPa seasonal cycles and increased near-surface saturated hydraulic conductivity) for these different initial stress conditions. The results of these analyses are presented in Section 6.6.

6.2.6 Key Reasons for Modelling and Model Requirements

The model development presented within this Chapter has been undertaken to ensure critical inter-related strength deterioration mechanisms associated with first-time failures of cut slope are considered within a single boundary condition problem. The model developed is required to capture undrained unloading, pore water pressure dissipation, stress relief, seasonal stress cycles and progressive failure.
6.3 Simplified Cut Slope Behaviour Single and Two-Phase Flow Analysis – Stage 1

As described in Section 6.2.2, simplified cut slope behaviour (undrained unloading, excess pore water pressure dissipation, stress relief and progressive failure due to constant boundary conditions) has been modelled following the steps outlined in Section 4.2.1 within a saturated and unsaturated (two-phase flow) framework. The results of the two analyses considering stress changes 1m below the toe of the slope and mid-slope horizontal displacements are shown in Figure 6.5.

Figure 6.5 Comparison of simplified cut slope behaviour in a saturated (SP-A1) and two-phase flow framework (TP-A1); a) stress changes 1m below toe of slope; b) mid-slope horizontal displacement

The results in Figure 6.5a show that the mechanism of undrained unloading and dissipation of excess pore water pressures are captured within the two-phase flow analysis but Figure 6.5b shows that the time to failure is very different to that of the saturated model. The mechanisms being modelled, undrained unloading and dissipation of excess pore water pressures, are saturated processes so the results of the saturated analysis are the baseline
results. The two-phase flow analysis failed after 30.7 years and the saturated model after 24.3 years, this is a difference of 26.3%.

6.4 Simplified Cut Slope Behaviour a Coupled Model – Stage 2

To improve the consistency of the modelling approach and ensure saturated processes are captured adequately within the unsaturated (i.e. two-phase flow) framework being used, a coupling process, shown in Figure 6.3, has been developed.

To allow undrained unloading and pore water pressure dissipation to be modelled, and the time to failure match the saturated analysis, a saturated model has been coupled with the two-phase flow model so that all behaviours are captured accurately in a single boundary-value problem.

The following results are for numerical analysis of undrained unloading, stress relief, pore water pressure equilibration and progressive failure under constant boundary conditions. As discussed in Section 6.2.3, post-excavation conditions from a single-phase flow (i.e. saturated) analysis have been transferred to a two-phase flow model with the same material properties and mesh configurations. The two-phase flow model has been set as fully saturated which is the case for undrained unloading and 10kPa suction boundary conditions have been applied to both the single and two-phase flow models and the analyses run to failure. A comparison of stress changes 1m below the toe of the cut slope and mid-slope horizontal displacements for the two models are shown in Figure 6.6.
Figure 6.6 Comparison of simplified cut slope behaviour in a saturated (SP-A1) and coupled two-phase flow framework (TP-A2); a) stress changes 1m below toe of slope; b) mid-slope horizontal displacement

Figure 6.6 shows that the behaviour observed for simplified cut slope behaviour in a saturated framework, can be replicated within a two-phase flow analysis far better when post-excavation conditions from the saturated model are transferred and the model is fixed as saturated than modelling the behaviour in the two-phase flow framework only. The time to failure of the two analyses compared in Figure 6.6 are within 0.6 years, for a slope that failed after 24 years this difference can be assumed to be negligible. In addition, the stress changes observed in Figure 6.6a are very similar, the pore water pressures between the two analyses are a difference of 2.8, 0.5, and 0.3kPa at 2.0, 5.0, and 10.0 years respectively.

The results show that critical cut slope mechanisms can be captured adequately within a two-phase flow analysis using the coupled modelling approach developed.
6.5 Cut Slope Behaviour with Seasonal Boundary Conditions – Stage 3

Using the same coupling approach allowing simplified cut slope behaviour to be modelled in a two-phase flow numerical model, an analysis considering undrained unloading, stress relief, pore water pressure equilibration and progressive failure driven by seasonal wetting and drying boundary conditions has been considered. As described in Section 6.2.4, the slope has been subjected to uniform drying and wetting, 7 months and 5 months respectively, to simulate seasonal behaviour. With pore water pressures on the slope surface, at mid-slope, reaching 70kPa suctions at the end of summer drying and 0kPa at the end of winter wetting.

To allow seasonal ratcheting behaviour to be modelled as well as pore water pressure equilibration, the two-phase flow analysis has been initially fixed as fully saturated, however, the soil can desaturate due to the boundary conditions applied, at depth the model remains saturated to replicate behaviour in reality. Figure 6.7 shows a comparison of the two-phase flow analysis with seasonal boundary conditions (model reference TP-B1) and simplified cut slope behaviour in a saturated framework under constant 10kPa suction boundary conditions (model reference SP-A1).
Figure 6.7 Comparison of simplified cut slope behaviour (SP-A1) and cut slope behaviour with seasonal boundary conditions (TP-B1); a) stress changes 1m below toe of slope; b) pore water pressures below toe of slope; c) mid-slope horizontal displacements; d) shear surfaces

Figure 6.7 shows that the time to failure of the cut slope when subjected to seasonal wetting and drying in a two-phase flow analysis increases
dramatically compared to a saturated analysis where a constant conservative pore water pressure boundary condition is imposed, with the slope falling after 162.4 years compared to the 24.3 years. This can be explained by the desaturation of the near-surface of the slope experienced in the two-phase flow analysis during drying and the extra time required for pore water pressure equilibration to reach steady state under continued wetting and drying.

Within Figure 6.7b, the delayed recovery of post-excavation pore water pressures within the two-phase flow analysis can be seen clearly. The pore water pressures at 1.0m and 2.5m below the slope toe have dissipated by the time the slope fails but at greater depths the pore water pressures remain suppressed. It can also be seen that the failure surface of the two analyses differ considerably (see Figure 6.7d). The failure surface for the saturated model, in which pore water pressures at depth dissipate, is deeper than the failure surface when seasonal wetting and drying is included. This indicates that the stress changes driven within a slope by seasonal wetting and drying do not influence as greater depth of material as stress changes due to prolonged wetting (i.e. constant conservative boundary conditions).

The failure surface observed for TP-B1 is shallower in nature than the failures obtained considering pore water pressure equilibration alone and is more characteristic of failures observed in ageing infrastructure slopes as discussed by Briggs, et al. (2017).

The results presented demonstrate that inter-related deterioration mechanisms associated with overconsolidated clay cut slope behaviour such as undrained unloading, excess pore water pressure dissipation, stress relief, seasonal ratcheting, and progressive failure, can be modelled within a single boundary condition model capable of including unsaturated behaviour.

### 6.5.1 Considering Hydrogeological Deterioration of the Near-Surface

As described in Section 6.2.4.1, hydrogeological deterioration of the near-surface occurs following excavation. Stress relief causes fissures to open and repeated wetting and drying causing restructuring of the soil. This has been investigated by increasing the near-surface saturated hydraulic conductivity in
a zone parallel to the excavated slope profile, see Section 6.2.4.1. The increase in near-surface saturated hydraulic conductivity has been made following the transfer of post-excavation conditions to the two-phase flow model (model reference TP-B2). Except for the near-surface saturated hydraulic conductivity, TP-B2 has the same material properties, mesh, softening model and has been subjected to the same magnitude of seasonal pore water pressure cycles, and therefore stress cycles, as TP-B1. The mid-slope horizontal displacement of analysis TP-B2 have been compared with analyses SP-A1 and TP-B1 in Figure 6.8.

Figure 6.8 shows that there is a considerable reduction in the time to failure when the near-surface saturated hydraulic conductivity is increased within the two-phase flow analysis. The time to failure has reduced from 162.4 years (TP-B1) to 47.1 years (TP-B2). There are limitations within the method developed, the saturated hydraulic conductivity profile applied post-excavation is static and applied over a constant depth. Currently, there is insufficient data available to quantify the rate of change of saturated hydraulic conductivity with time / number of seasonal cycles of wetting and drying. The results presented show relative influence of increased saturated hydraulic conductivity and the significant effect it has on the mechanical behaviour of a slope.
6.5.2 Considering the Effects of Climate Change

In line with the methodology described in Section 6.2.4.2, the effects of general trends in climate change projections (UKCP09) have been investigated. This has been done by changing the boundary conditions to drive wetter winters and drier summers after the 15th seasonal cycle of model TP-B2. Drier summers have been modelled by increasing the suctions at the end of drying from 70kPa to 90kPa and wetter winters have been modelled by increasing the duration that the slope experiences near hydrostatic conditions at the end of winter.

The results of the analysis considering climate change (TP-B2-CC) have been plotted alongside the ‘steady climate’ scenario (TP-B2) in Figure 6.9. The different pore water pressure cycles at mid-slope surface due to the different boundary conditions imposed can be seen in Figure 6.9a and Figure 6.9b, with the stress changes 1m below the toe of the slope in Figure 6.9c. Finally, the mid-slope horizontal displacements against number of seasonal cycles for the two different models can be seen in Figure 6.9d.
Chapter 6 – Modelling Cut Slope Behaviour with Seasonal Ratcheting

Figure 6.9 Comparison of cut slope behaviour with different seasonal boundary conditions (TP-B2 - steady climate; TP-B2-CC – drier summers and wetter winters i.e. climate change); a) TP-B2 boundary conditions; b) TP-B2-CC boundary conditions; c) stress changes 1m below toe of slope; d) mid-slope horizontal displacements

From Figure 6.9d, it can be seen immediately after the seasonal pore water pressure cycles change there is a divergence in the behaviour of the two models. TP-B2-CC experiences greater displacement annually and this can be attributed to the longer time spent at near hydrostatic conditions during wetter winters. The model subjected to climate change boundary conditions failed 3.8 years before the steady state climate boundary condition model. This is a
reduction in the life of this slope of approximately 8% purely due to changes in the time spent at near hydrostatic conditions. The results show the potential impact of climate change reducing the serviceable life of earthwork assets in the future.

### 6.6 Effect of Different Initial Stress Conditions

One of the significant conclusions drawn from numerical analysis of early work considering cut slope behaviour (simplified cut slope behaviour in a saturated framework and constant boundary conditions) conducted by Potts, *et al.*, (1997) was the effect different initial stress conditions had on behaviour. Through considering different coefficients of earth pressure at rest the stiffness of the clay and the potential energy released during unconfinement differs and so does the time to failure. Numerical analyses following the method described in Section 6.2.5 has been conducted to investigate the influence of different coefficients of earth pressure at rest.

For clarity, results of analyses considering different coefficients of earth pressure at rest for simplified cut slope behaviour and constant 10kPa suction boundary conditions (like model SP-A1) have been presented alongside results where seasonal ratcheting boundary conditions have also been included (like model TP-B2) to investigate the influence of initial stress conditions. This has been done for three coefficients of earth pressure at rest, results of mid-slope horizontal displacement are shown in Figure 6.10.
Figure 6.10 Effect of initial stress conditions on cut slope behaviour; a) simplified cut slope behaviour; b) cut slope behaviour with seasonal boundary conditions

Figure 6.10a shows significant variation in the time to failure due to different initial stress conditions when simplified cut slope behaviour with constant boundary conditions are modelled. The behaviour shown is indicative of the trend observed by Potts, et al., (1997). Interestingly, when cut slope behaviour is modelled including seasonal boundary conditions the time to failure between the models with different initial stress conditions is much smaller. This can be explained by the mechanism of failure.

The results in Figure 6.10a are for deep-seated failure driven by pore water pressure equilibrium and progressive failure and the results in Figure 6.10b are shallow first-time failures due to seasonal stress cycles and progressive failure, a comparison of the shear surfaces obtained from the numerical analyses for the different $K_0$ values are shown in Figure 6.11. The increased stiffness due to higher stresses with increased coefficient of earth pressure at rest has less of an effect on the behaviour of failures driven by seasonal stress cycles as surface processes are more prominent, and therefore stresses are lower, than for deep-seated mechanisms where stresses are higher.
Figure 6.11 Comparison of failure surfaces for different initial stress conditions; a) simplified cut slope behaviour; b) cut slope behaviour with seasonal boundary conditions

6.7 Discussion

Within this Chapter, an approach for the numerical modelling of cut slope behaviour including construction effects and seasonal driven cycles of shrink-swell behaviour leading to progressive failure has been presented. A coupling of single and two-phase flow analyses has allowed multiple inter-related strength deterioration processes attributed to cut slope behaviour within overconsolidated clay to be incorporated within a single boundary-value problem; including, undrained unloading, excess pore water pressure dissipation, stress relief and seasonal ratcheting.

The physical behaviours being modelled have been considered systematically and results shown to prove that the relevant behaviours are captured sufficiently within the modelling approach developed. There are limitations in the modelling approach, however, trends in behaviour have illustrated the significance of hydrogeological deterioration of the near-surface and the impact this has on the number of seasonal cycles to failure. In addition, the potential implications of climate change on the performance of cut slopes in high-plasticity overconsolidated clays has been shown. The effects of initial stress conditions prior to excavation have been investigated and it has been shown that the initial stress conditions have a reduced impact on time to failure.
for scenarios where shallow first-time failure due to seasonal stress cycles are expected in comparison to deep-seated failure due to construction effects. The outcomes of this modelling are discussed in more detail in the following sections of this discussion.

The numerical analyses presented assume the material modelled is homogeneous and as such, like the previous study in Chapter 5, the effects of spatial variability of material properties have not been considered. In particular, bedding features and other structure discontinuities found in overconsolidated clays can influence material properties hugely and influence the failure surface and mechanism by which a slope might fail. Again, as with the previous study in Chapter 5, the effect of stiffness deterioration as a result of repeated cyclic stress has not been considered but will have an impact on slope behaviour.

The boundary conditions imposed in this study have been selected as they represent field observations of seasonal wetting and drying cycles in grass-covered London Clay slopes. Therefore, the stress cycles imposed on the near-surface of the slope models should in theory be representative of real slopes. There has also been an attempt to include spatial variability in hydrogeological properties (i.e. near-surface depth dependent saturated hydraulic conductivity profile) using observations of saturated hydraulic conductivity in the near-surface (see Figure 6.2). The measurements used to create the profiles used were for grass-covered London Clay slopes. This means that the effect of roots on hydrological behaviour has been partially included within the numerical analysis. The mechanical implications of roots changing the strength and stiffness of the near-surface material, however, have not been included in this work.

### 6.7.1 Hydrogeological Deterioration

The results of the numerical analyses shown in Section 6.5.1 show that changing the hydrogeological properties of the near-surface zone has a significant influence on the number of seasonal stress cycles for which a slope experiencing wetting and drying will remain stable. Hydrogeological deterioration has been shown to be extremely important for shallow first-time failures due to seasonal ratcheting. However, it is extremely difficult to
establish *in-situ* hydraulic conductivity due to spatial variability in the parameter, and temporal variability as the condition of the soil will differ depending on the antecedent conditions. If the complete extent of climate change is to be assessed for engineered slopes it is of paramount importance that the rate of deterioration of hydrogeological properties is established for the material being investigated.

Within this study, the method used to investigate hydrogeological deterioration of the near-surface is relatively simple. The near-surface hydraulic conductivity profile has been obtained from field measurements and is applied to the model instantaneously after excavation of the slope even though there will be an element of time for the material to change from intact post-excavation to weathered. The model only considers the deterioration of saturated hydraulic conductivity but undoubtedly, after numerous seasonal cycles of wetting and drying, the soil water retention characteristics of the soil will also deteriorate.

Regardless of the limitations, conclusive evidence is shown indicating that hydrogeological deterioration of the near-surface will shorten the life of high-plasticity overconsolidated clay slopes and more is needed to be known about the deterioration of near-surface hydrogeological properties.

### 6.7.2 Climate Change

Section 6.5.2 has shown that increased seasonal pore water pressure cycles and more prolonged wetting, which are the headline trends of major climate change projection models (UKCP09), will lead to a reduction in the serviceable life of cut slope assets constructed in high-plasticity overconsolidated clay. There are obvious limitations within the work presented for this assessment of climate change.

Simple wetting and drying, winter and summer, boundary conditions have been used and only the trend in magnitude of summer suction and time spent near hydrostatic conditions during winter wetting have been considered. The frequency of extreme events on behaviour have not been considered. In addition, the magnitude of the changes imposed have not been obtained from projection data but has been selected in a deterministic way to illustrate the
implications of greater pore water pressure and therefore stress cycles on slope behaviour.

Whist, the approach used to assess climate change does not fit within a conventional climate change assessment framework, the results presented clearly indicate that if the general trends of climate change projections are representative of the future climate then the risk associated with shallow first-time failures due to seasonal wetting and drying in high-plasticity overconsolidated clay slopes will increase significantly.

**6.7.3 Hydrogeological Deterioration and Climate Change**

Climate change projections (UKCP09) have indicated wetter winters and drier summers resulting in larger cycles of pore water pressures, and therefore stress cycles, in the near-surface of a slope. Numerical model results presented have predicted that the general trends in climate change projected will result in a reduction in the serviceable life of high-plasticity overconsolidated earthworks. In addition, it has been shown that increased saturated hydraulic conductivity of the near-surface of a slope shortens the serviceable life of a slope considerably.

It can be postulated that the change in hydrogeological properties of the near-surface of a cut slope change post-excavation due to stress relief and repeated stress cycles due to wetting and drying. Therefore, if stress cycles are projected to increase in magnitude, the rate of deterioration of hydrogeological properties and the depth of material effected will also increase.

Therefore, under future climate change projections it can be anticipated that these two inter-related processes (i.e. hydrological deterioration and increased stress cycle magnitude) will reduce the serviceable life of cut slopes in high-plasticity overconsolidated clay slopes. As the stress cycles due to climate change increase, the rate of hydrogeological deterioration will increase, increasing the magnitude and depth of influence of stress cycles as water can be added and removed more readily from the soil. Both these processes negatively impact one another, compounding their influence on reducing the serviceable life of these earthwork assets.
6.7.4 Initial Stress Conditions

Numerical analyses considering the effect of different initial stress conditions on slope behaviour for simplified cut slope behaviour and cut slope behaviour including seasonal boundary condition were undertaken and results presented in Section 6.6. It has been indicated that when seasonal boundary conditions are included within analyses for this slope geometry, failures are shallow and a result of the near-surface stress cycles imposed by the boundary conditions. These failures are indicative of failures observed in ageing infrastructure slopes (Briggs, et al., 2017).

As failure is driven by near-surface process, initial stress conditions prior to excavation of the slope have a significantly reduced effect on the times to failure for this failure mechanism than observed by Potts, et al., (1997) when considering numerical analyses of deep-seated failure mechanisms due to construction effects.

These results show that the critical unknowns in this modelling is the time-dependent change in near-surface properties and that initial conditions and material properties at depth have less influence on the failure mechanism.

6.8 Key Findings

From the work presented within this Chapter, the following key findings for high-plasticity clay cut slopes have been obtained;

- Deterioration of near-surface hydrogeological parameters has a significant impact on the time to failure of a slope subjected to seasonal stress cycles. Hydrogeological deterioration of the near-surface is a critical deterioration mechanism that very little is known about;
- Initial stress conditions prior to excavation of a cut slope subjected to repeated seasonal stress cycles has limited effect on behaviour due to the depth of influence of stress cycles (i.e. predominately within the near-surface of the slope) and the failure mechanism of seasonal ratcheting;
• If climate change projections are representative of future climate conditions high-plasticity clay cut slopes will experience an increased rate of deterioration in the future.

### 6.9 Chapter Summary

Within this Chapter, a method for the numerical modelling of multiple interrelated strength deterioration mechanisms has been presented. Undrained unloading, stress relief and dissipation of excess pore water pressures, due to excavation within a low hydraulic conductivity material, can be capture along with seasonal wetting and drying boundary conditions driving seasonal ratcheting within a two-phase flow (unsaturated) framework.

The results of the numerical analyses show that shallow first-time failures in high-plasticity overconsolidated clay cut slopes can be explained by repeated seasonal wetting and drying stress cycles causing strength reduction due to the accumulation of outward and downward movement. The significance of near-surface hydrogeological deterioration due to repeated seasonal wetting and drying cycles has been demonstrated and shown to change the time to failure of a cut slope dramatically.

The numerical modelling approach developed has been utilised to investigate the implications of drier summers and wetter winters in line with the general trends of climate change projections (UKCP09). If the trends are indicative of the future climate, the risk of failure of these earthwork assets due to seasonal wetting and drying will undoubtedly increase. As stress cycles increase, the rate of deterioration of hydrogeological properties will also increase, and as slopes experience near hydrostatic conditions for longer in the winter, greater strength deterioration will occur. All these factors will reduce the serviceable life of cut slopes in high-plasticity overconsolidated clay.
Chapter 7

Modelling Long-Term Behaviour of a Real Cut Slope: A Validation against Field Monitoring and Stress Testing

7.1 Chapter Scope

Using the modelling approach developed in Chapter 6, this Chapter presents and validates a method for modelling real cut slope hydrogeological behaviour through improved soil-water-atmosphere interaction boundary conditions, as outlined in Figure 7.1. Results of hydrogeological behaviour are validated against long-term monitoring of a London Clay cut slope. The approach developed for the modelling, the method used to obtain boundary conditions, and the material relationships used to model long-term cut slope behaviour, including unsaturated near-surface behaviour are presented in Section 7.2.

Figure 7.1 Phase 4 model development and outputs
7.2 Methodology (Phase 4) – A Validation Against Field Monitoring

Using the method presented previously for the coupling of a single and two-phase flow numerical model to allow undrained unloading, stress relief, excess pore water pressure dissipation, seasonal ratcheting and progressive failure to be modelled in a single boundary-value problem numerical model (see Chapter 6), realistic weather boundary conditions have been applied to a numerical model replicating a real slope. Soil-water-atmosphere interaction boundary conditions are considered, and the hydrogeological behaviour of the numerical analysis has been compared to long-term monitored data to allow validation of the hydrogeological properties used in the analysis and boundary conditions applied.

The approach to allow the modelling of undrained unloading, stress relief, dissipation of excess pore water pressures, seasonal ratcheting and progressive failure is the same as for the numerical analyses considered in Section 6.2.4. Initial excavation is carried out within a saturated framework which includes consolidation theory, post-excitation stresses and strains are then transferred to a two-phase flow model and taken as the initial conditions for the analysis. The two-phase flow model is initially fixed as saturated but the near-surface of the model is allowed to desaturate due to the boundary conditions imposed.

Firstly, the method used to obtain daily boundary conditions for the numerical model from real weather data is presented followed by description of material properties and modelling assumptions made for a case study slope used to validate the hydrogeological behaviour of the numerical modelling approach.

Following the presentation of results of the validation exercise, the effect of different water and air bulk moduli, used in the numerical analyses, on hydrogeological and mechanical response of the numerical model have been considered (see Section 7.4) followed by a parametric study looking at the potential implications of stiffness (see Section 7.5) and roots (see Section 7.6) on mechanical behaviour. Finally, a numerical model representative of the case study slope has been subjected to a long-time sequence of weather data
and a climate change impact assessment through stress testing using statistical down-scaling (Wilby, et al., 2014) as described in Section 2.5.4 has been conducted (see Section 7.7).

### 7.2.1 Boundary Conditions

Current state-of-the-art weather-driven modelling applies pore water pressures along the slope surface boundary and solves the model daily (Rouainia, et al., 2009; Davies, 2011). The approach developed by Davies (2011) uses pore water pressures obtained from a hydrological finite difference programme (SHETRAN) and applies them to a coupled hydrogeological-geotechnical model, this was done within FLAC-TP. SHETRAN is an in-house software developed at Newcastle University driven by weather input data. The coupling of these two finite difference software packages and the level of data required to drive SHETRAN makes the process very computationally intensive and constrictive as both meshes need to match and boundary conditions are only relevant for the given slope geometry analysed.

To reduce computational time and increase model flexibility, an alternative approach for modelling weather-driven behaviour is proposed. Developing on previous work presented in Chapters 5 and 6; boundary conditions are applied as a flux (discharge) directly to a coupled hydrogeological-geotechnical model and solved daily. As behaviour being modelled considers the life-cycle of slopes, which are decades, daily boundary conditions are sufficient to model hydrogeological behaviour and capture strength deterioration. The method for obtaining the daily discharge boundary conditions is based on a water balance calculation (Blighth, 2003). Figure 7.2 shows a conceptual hydrological cycle for a slope and a simplified slope boundary illustrating the soil-water-atmosphere interaction.
7.2.2 Water Balance Calculation

The water balance method is a simplistic hydrological model based on the conservation of mass, the approach is capable of calculating usable data for transient soil-water conditions (Blight, 2003). The method is dependent on topography, vegetation type, antecedent conditions and external climate. It is given that (Blight, 2003);

\[
\sum (R - RO) + S - \Sigma ET = RE + \text{losses}
\]

Where;

- \( R \) = rainfall (mm);
- \( RO \) = run-off (mm);
- \( S \) = stored water (mm);
- \( ET \) = evapotranspiration (mm);
- \( RE \) = recharge from ground water (mm);
- \( \text{losses} \) = inaccuracies (mm).

Within this calculation the following assumptions are made;

- a constant depth of soil, commonly adopted as the vegetation rooting depth, is considered – the implications of this assumption are discussed later;
- soil water content change is averaged over this constant depth, thus wetting and drying fronts are not directly account for;
- soil is assumed to be homogeneous and isotropic.
For soils with low hydraulic conductivities and considerable thickness, it can be assumed that recharge from ground water is negligible in comparison to the variation due to surface processes, evapotranspiration and rainfall (Smethurst, et al., 2006). Therefore, Equation 7.1 can be simplified to;

**Equation 7.2**

\[ S = \Sigma ET - \Sigma (R - RO) \]

Using Equation 7.2, the daily net change in soil water content within the rooting zone can be obtained and converted into a flux (m/s) which is applied directly to the boundary of the numerical model and solved for a single day (86400 seconds).

**Equation 7.3** *Daily Flux Boundary Condition (m/s)*

\[ \frac{\Delta ET - \Delta (R - RO)}{1000 \times 86400} \]

As the boundary conditions are calculated assuming a fixed depth for the unsaturated zone (i.e. the rooting depth) there will be instances when the inflow discharge creates unrealistic positive pore water pressures within the numerical analyses. In this case water is being effectively forced into the model when the soil is already fully saturated and creating a heave like reaction. As this behaviour is unrealistic a limiting condition of hydrostatic is applied to the numerical model if there is a case where inflow would cause positive pore water pressures at the surface. Hydrostatic conditions are maintained by a very low inflow discharge conditions. This condition can be further justified within FLAC-TP as once the hydrostatic condition is applied to the boundary water is allowed to continue flowing into the model to maintain the condition and thus act as a wetting front allowing infiltration to continue at a rate dictated by the saturated hydraulic conductivity of the soil.

To allow the maximum limiting condition of hydrostatic to be applied to the slope model, the pore water pressures across the slope surface in the numerical model have been split into small groups and are continuously monitored. If the pore water pressures exceed 0kPa with any of the monitored groups, the discharge applied to that group of elements is reduced ensuring the soil remains saturated.
The data and equations used to obtain rainfall and evapotranspiration are presented in the relevant section associated with the case study slope. Where regional data series have been used, the method used to obtain parameters has been calibrated against a control period of onsite monitored data.

### 7.2.3 Case Study Slope – Newbury Cutting (A34)

To validate the modelling approach and boundary conditions developed, a case study slopes with extensive monitoring has been obtained and the results of the numerical analyses compared to behaviour observed in reality.

There are few comprehensive data sets available for the validation of both hydrogeological and mechanical behaviour – at Newbury cutting there are numerous piezometers (17 in total) installed from 2003 but until recently (2017) there have been no displacement measurements. Newbury has also been the subject of a study considering near-surface hydraulic conductivity (Dixon, et al., 2018). Therefore, the case study site has a wealth of knowledge regarding the hydrogeological behaviour of this cut slope, which has been provided by Dr Joel Smethurst of The University of Southampton.

<table>
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<tr>
<th>Table 7.1 Summary of Newbury cutting case study</th>
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<td>Source: (Smethurst, et al., 2012)</td>
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Newbury cutting is a highway slope on the A34 constructed in 1997 within London Clay. Since 2003 the site has been monitored extensively to investigate seasonal cycles of soil water content and soil-water-vegetation interaction (Smethurst, et al., 2006; Smethurst, et al., 2012). The London Clay formation is approximately 20m thick across the site with the upper 2.5-3.0m being weathered in line with the original ground profile (Smethurst, et al., 2006). Newbury cutting is a grass covered slope 8m high at an approximate angle of 16°, a site map and cross section are shown in Figure 7.3 and Figure 7.4 respectively.
Figure 7.3 Newbury site location (after, Smethurst, et al., 2012) (coast outline reproduced from Ordnance Survey map data by permission of the Ordnance Survey © Crown copyright 2001)

Figure 7.4 Newbury cutting slope cross-section (after Smethurst, et al., 2006)

For further information on the slope and monitoring regime see Smethurst, et al., (2006; 2012).

7.2.3.1 Site-Specific Weather Data

During the monitoring of Newbury cutting, a weather station has been operational, and data has been monitored to allow evapotranspiration to be estimated at the site along with rainfall being monitored continuously (Smethurst, et al., 2006; Smethurst, et al., 2012). The data available has allowed potential evapotranspiration for the site to be estimated using the Penman–Monteith equation, which is given in Equation 7.4.

\[
PET = \frac{\Delta(R_n + G) + \rho \cdot cp \cdot \frac{e_s - e_a}{T_a}}{\lambda \left( \Delta + \gamma \left( 1 + \frac{r_s}{r_a} \right) \right)}
\]

Where:

- \( \Delta \) = rate of change of vapour pressure with temperature (hPa/K);
- \( R_n \) = net radiation (W/m²);
Chapter 7 – Modelling Long-Term Behaviour of Real Cut Slopes

- $G$ = soil heat flux (W/m²);
- $\rho$ = density of the air (kg/m³);
- $c_p$ = specific heat of the air (J/kg/K);
- $e_s$ = saturation vapour pressure (hPa);
- $e_a$ = actual vapour pressure (hPa);
- $r_s$ = minimum stomata resistance (s/m);
- $r_a$ = aerodynamic resistance (s/m);
- $\lambda$ = latent heat of vaporisation (MJ/kg);
- $\gamma$ = psychrometer constant (0.67 hPa/K).

The daily weather sequence for the site is available between the start of 2003 and the end of 2011. Allowing a water balance calculation to be undertaken to determine boundary conditions, in line with the method described in Section 7.2.2, using this site-specific weather data.

7.2.3.2 Regional Weather Data
As weather data is not available for the entire life of the cut slope (i.e. start of 1997 to end of 2017), an alternative approach for determining boundary conditions for periods that have not been monitored is required. This has been done using regional weather data to fill in the gaps in the weather sequence for the slope. To ensure that the regional data is representative of the site-specific data, the period of 2003 to 2011 has been used as a base period and the differences in the data sets considered, this is shown in Section 7.3.1.

Unfortunately, for past weather there is insufficient regional data to calculate potential evapotranspiration using the Penman–Monteith method. Therefore, a simpler temperature based method has been adopted for the calculation of the longer weather sequence, the Blaney-Criddle (1950) method, shown in Equation 7.5 has been used.

Equation 7.5

$$PET = a + b[p(0.46T + 8.13)]$$

Where:

- $a$ and $b$ = calibration factors;
- $p$ = mean daily percentage of annual daytime hours;
Chapter 7 – Modelling Long-Term Behaviour of Real Cut Slopes

- \( T \) = mean daily air temperature (°C).

From an investigation into the sensitivity of different methods for calculating potential evapotranspiration for temperate climate regions, Bormann (2011) showed that the Blaney-Criddle (1950) method estimated similar annual values of potential evaporation to the Penman-Monteith method. Ultimately, the method selected for calculating potential evapotranspiration will depend on the data available.

The calibration factors (i.e. \( a \) and \( b \)) take into account relative humidity and sunshine hours as a ratio of actual to possible hours of sunlight. Some assumptions have been made in the use of the Blaney-Criddle (1950) method; the slope is east facing and shaded from the mid-afternoon, also the prevailing wind for the UK is westerly therefore the slope is assumed to be mainly sheltered from the wind with high cloud cover to account for shade. Values of \( p \) and \( b \) adopted in the calculation have been taken from tables presented in crop water requirements by Doorenbos and Pruitt (1977). These are summarised in Table 7.2. Parameter \( a \) has been calculated using Equation 7.6.

Table 7.2 Site-specific calibration factors for Blaney-Criddle (1950) PET calculation
(after, Doorenbos and Pruitt, 1977)

<table>
<thead>
<tr>
<th>Month</th>
<th>Parameter</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( p )</td>
<td>( a )</td>
<td>( b )</td>
</tr>
<tr>
<td>January</td>
<td>0.19</td>
<td>-1.059</td>
<td>0.52</td>
</tr>
<tr>
<td>February</td>
<td>0.22</td>
<td>-1.098</td>
<td>0.52</td>
</tr>
<tr>
<td>March</td>
<td>0.27</td>
<td>-1.129</td>
<td>0.64</td>
</tr>
<tr>
<td>April</td>
<td>0.31</td>
<td>-1.151</td>
<td>0.64</td>
</tr>
<tr>
<td>May</td>
<td>0.35</td>
<td>-1.150</td>
<td>0.64</td>
</tr>
<tr>
<td>June</td>
<td>0.37</td>
<td>-1.129</td>
<td>0.64</td>
</tr>
<tr>
<td>July</td>
<td>0.36</td>
<td>-1.141</td>
<td>0.64</td>
</tr>
<tr>
<td>August</td>
<td>0.33</td>
<td>-1.151</td>
<td>0.64</td>
</tr>
<tr>
<td>September</td>
<td>0.28</td>
<td>-1.120</td>
<td>0.64</td>
</tr>
<tr>
<td>October</td>
<td>0.24</td>
<td>-1.076</td>
<td>0.52</td>
</tr>
<tr>
<td>November</td>
<td>0.2</td>
<td>-1.050</td>
<td>0.52</td>
</tr>
<tr>
<td>December</td>
<td>0.17</td>
<td>-1.059</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Equation 7.6

\[
a = 0.0043RH_{min} - \frac{n}{N} - 1.41
\]
Where:

- $R_{H_{\text{min}}}$ = minimum daily relative humidity (due to the data series available, this work considers minimum average monthly relative humidity for the time period in question);
- $n/N$ = ratio of actual sunlight hours to possible sunlight hours.

The regional weather data series used in this work to determine model boundary conditions and the time that they cover are summarised in Table 7.3 and the area that the different data set consider are shown in Figure 7.5. To calculate potential evapotranspiration using Equation 7.5, average mean daily temperatures were obtained from HadCET – Central England Temperature (Parker, et al., 1992) and monthly average relative humidity from HadISDH – Gridded Land Surface Humidity (Willett, et al., 2014). A comparison of cumulative potential evapotranspiration for site-specific weather data using the Penman-Monteith equation and regional weather data using the Blaney-Criddle equation and data sets discussed is shown in Section 7.3.1.

### Table 7.3 Regional weather data series - sources

<table>
<thead>
<tr>
<th>Dataset</th>
<th>Name</th>
<th>Variable</th>
<th>Frequency</th>
<th>Time Series</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>HadCET</td>
<td>Central England Temperature</td>
<td>Mean Temperature</td>
<td>Daily</td>
<td>1772 - present</td>
<td>(Parker, et al., 1992)</td>
</tr>
<tr>
<td>HadISDH</td>
<td>Gridded land surface humidity</td>
<td>Relative Humidity</td>
<td>Monthly</td>
<td>1973 - present</td>
<td>(Willett, et al., 2014)</td>
</tr>
</tbody>
</table>

Figure 7.5 Regional weather data series and area of coverage (map outline reproduced from Ordnance Survey map data by permission of the Ordnance Survey © Crown copyright 2001)
7.2.3.3 Calculating Actual Evapotranspiration

Once potential evapotranspiration has been calculated, the influence of vegetation and soil water availability must be considered to allow actual evapotranspiration to be calculated. Evapotranspiration has been calculated as shown in Equation 7.7 (Clarke, et al., 1998; Smethurst, et al., 2006);

\[
\begin{align*}
\Delta ET & = \Delta PET \times k_c \\
\Delta ET & = \Delta PET \times k_c \times \frac{TAW - SMD}{TAW - RAW} \\
\end{align*}
\]

for \( 0 \leq SMD \leq RAW \)

\[
\begin{align*}
\Delta ET & = \Delta PET \times k_c \times \frac{TAW - SMD}{TAW - RAW} \\
\end{align*}
\]

for \( SMD \geq RAW \)

Where;

- \( PET \) = potential evapotranspiration (mm);
- \( k_c \) = crop factor;
- \( TAW \) = total available water (mm);
- \( SMD \) = soil moisture deficit (mm);
- \( RAW \) = readily available water (mm).

This method of calculating evapotranspiration takes into account the stressing and wilting of vegetation and limits water removal to a realistic level for the soil being considered. A considerable proportion of water within fine-grained soil is held within capillaries and through chemical attraction. Significantly more energy is required to remove this water than vegetation is capable of producing (Smethurst, et al., 2006). Therefore, the total available water is much less than the total water in the soil.

Total available water is defined as the point at which vegetation wilts, corresponding to a suction of approximately 1500kPa; total available water is roughly 18% for London Clay (Smethurst, et al., 2006). Readily available water is the volume of water that defines the point at which vegetation becomes stressed, this is at suctions of approximately 40-100kPa (Smethurst, et al., 2006). Readily available water can be obtained by considering the SWRC of the soil and obtaining the soil water content variation necessary to cause a suction of 40-100kPa as a percentage of the total available water (where total available water is taken as 10-1500kPa on the SWRC). For London Clay, readily available water is approximately 40% of the total available water.
The crop factor accounts for different vegetation types. In this study, the slope is grass covered, thus, the crop factor can be taken as 1 and the rooting depth in London Clay taken as 800mm, giving TAW = 144mm and RAW = 58mm (Smethurst, et al., 2006).

The different weather data series used and the corresponding boundary conditions of the control period (i.e. 2003 to 2011) using site-specific data and regional weather data and a complete weather sequence for the life of Newbury cutting have been presented in detail in Section 7.3.1.

### 7.2.3.4 Mechanical Material Properties

From the previous work considering London Clay slopes, the same nonlocal strain-softening model has been implemented within this work. For clarity all mechanical material properties used within the numerical analyses presented in Chapter 7 are presented in Table 7.4, unless stated otherwise throughout this methodology section.

<table>
<thead>
<tr>
<th>London Clay strain-softening properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Nonlocal Plastic</td>
<td>0.012</td>
</tr>
<tr>
<td>Residual Nonlocal Plastic Strain</td>
<td>0.055</td>
</tr>
<tr>
<td>Internal Length (m)</td>
<td>1.00</td>
</tr>
<tr>
<td>Peak cohesion (kPa)</td>
<td>7</td>
</tr>
<tr>
<td>Residual cohesion (kPa)</td>
<td>2</td>
</tr>
<tr>
<td>Peak friction angle (°)</td>
<td>20</td>
</tr>
<tr>
<td>Residual friction angle (°)</td>
<td>13</td>
</tr>
<tr>
<td>Dilation Angle (°)</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>London Clay stiffness properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus, E (kPa)</td>
<td>25(σ’+100); min (4000)*</td>
</tr>
<tr>
<td>Bulk modulus, K (kPa)</td>
<td>E / (3(1 − 2ν))</td>
</tr>
<tr>
<td>Shear modulus, G (kPa)</td>
<td>E / 2(1 + ν)</td>
</tr>
<tr>
<td>Poisson’s ratio, ν’</td>
<td>0.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Other</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk density, γ (kN/m³)</td>
<td>18.8</td>
</tr>
<tr>
<td>Coefficient of earth pressure at rest, k₀</td>
<td>1.5</td>
</tr>
</tbody>
</table>

* σ’ = effective stress (kPa) (single-phase flow model)
* σ’ = Bishop’s generalised effective stress (kPa) (two-phase flow model)

### 7.2.3.5 Hydrological Material Properties

From previous studies, the importance of understanding near-surface hydrogeological material properties has been demonstrated, see Chapter 6. For Newbury cutting, the upper 3m of the original ground surface is considered
highly weathered and the slope surface will be partially weathered after 20 years of cyclic weather. Below this the London Clay can be assumed to be intact. For this reason, the slope has been split into a number of regions to represent the state of the material at the time of monitoring. These sections then have different hydrological properties depending on level of weathering. Representing the slope in this way does mean that the analysis is only representative for a certain period of the slopes life-cycle but there is currently no complete data set that can give a detailed rate of change of hydrological parameters with time for London Clay. The weathering profile of the slope is shown graphically in Figure 7.6.

Within the numerical model considering weather boundary conditions the soil water retention properties and saturated hydraulic conductivity have been stipulated to represent the current understanding of weathering at the site.

The values for the different soil water retention curves depending on the level of weathering of the material are summaries in Table 7.5 and shown graphically in Figure 7.7. The intact soil water retention properties have been taken from work by Briggs, et al., (2013), the air entry value of the curve has then been changed to represent the degradation of the soil water retention properties due to continued seasonal cycles of wetting and drying effectively weathering the soil and increasing the void ratio of the material.
Table 7.5 Model soil water retention properties

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturated Soil Water Content, $\theta_s$</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
</tr>
<tr>
<td>Residual Soil Water Content, $\theta_r$</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.1</td>
</tr>
<tr>
<td>van Genuchten - n</td>
<td>1.18</td>
<td>1.18</td>
<td>1.18</td>
<td>1.18</td>
</tr>
<tr>
<td>van Genuchten - m</td>
<td>0.153</td>
<td>0.153</td>
<td>0.153</td>
<td>0.153</td>
</tr>
<tr>
<td>van Genuchten – $\alpha$ (kPa)</td>
<td>125.0</td>
<td>62.5</td>
<td>30.3</td>
<td>15.15</td>
</tr>
</tbody>
</table>

* (Briggs, et al., 2013)

Finally, to complete the hydrogeological properties of the model, the saturated hydraulic conductivity profile adopted in the analyses is shown in Figure 7.8. The profile adopted has been obtained by considering numerous near-surface in-situ measurements from Newbury and other depth dependent London Clay saturated hydraulic conductivity measurements. In line with the work presented in Chapter 6, the hydraulic conductivity profile has been applied considering the original ground surface and post-excavation the upper zone of the slope has had an increased saturated hydraulic conductivity applied to it prior to the application of daily boundary conditions. The depth dependent saturated hydraulic conductivity profiles are shown on Figure 7.8, with Figure 7.8a showing the hydraulic conductivity profile associated with the original ground surface (i.e. prior to excavation) and Figure 7.8b showing the near-surface hydraulic conductivity profile (i.e. weathered). The near-surface profile
has been applied to the upper 4m of the slope surface and upper 5m associated with the original ground profile.

![Saturated Hydraulic Conductivity Profiles](image)

**Figure 7.8** Depth dependent saturated hydraulic conductivity profiles for Newbury cutting; a) profile associated with original material; b) profile associated with weathered near-surface zone

### 7.2.3.6 Analysis of Newbury Cutting

Using the boundary conditions and material properties described throughout this section an analysis of Newbury cutting from construction (i.e. start of 1997) to the end of 2017 has been conducted. The results of the numerical analysis have been presented in Section 7.3 where the boundary conditions developed are shown in detail and hydrogeological behaviour of the numerical analysis has been compared against pore water pressure records at Newbury for the monitoring period of 2003 to 2008. Finally, the mechanical response of the numerical analysis is shown.
Using the model developed, the behaviour of the numerical analysis has been stress tested considering different material behaviours and climate change scenarios. The different scenarios considered are presented in Sections 7.4 to 7.7.

### 7.2.4 Key Reasons for Modelling and Model Requirements

The effects of climate change on high-plasticity clay cut slopes have been debated within literature, see Section 2.3.6.2. A numerical modelling approach capable of modelling inter-related deterioration mechanisms including weather driven behaviour has been developed and presented within the previous Chapters. The work within this Chapter looks to develop soil-water-atmosphere interaction modelling capabilities to allow the effects of climate change to be explored (see Section 7.7). The model must be capable of replicating realistic seasonal stress cycles, to validate this, a case study slope with extensive monitoring has been considered and behaviour of the numerical model compared.

### 7.3 Newbury Cutting Case Study Results

The location of Newbury cutting, and the weather data used to fill the gaps in the monitored site-specific weather data are shown in Section 7.2.3. The locations of piezometers across the slope are shown in Figure 7.4 and the points at which displacements are considered in the numerical analysis are shown in Figure 7.9.

#### 7.3.1 Daily Weather Boundary Conditions

Newbury cutting has been monitored extensively since 2003 by Smethurst, et al., (2006; 2012) and a water balance calculation has been carried out between
2003 and 2012. The data for rainfall, evapotranspiration and the soil moisture deficit calculated has been used as ‘control period’ data to which regional weather data has been compared and calibrated to allow the synthesis of a long-term weather series from the construction of Newbury cutting in 1997 to the end of 2017. The following section presents the process of creating and calibrating daily boundary conditions for Newbury cutting considering the input parameters presented in Sections 7.2.2 and 7.2.3. For clarity, the schematic boundary conditions are presented again in Figure 7.10 and the equation to calculate daily discharge boundary condition in Equation 7.8.

![Figure 7.10 Weather boundary conditions](image)

Equation 7.8  

\[
\text{Daily Flux Boundary Condition (m/s)} = \frac{\Delta ET - \Delta (R - RO)}{1000 \times 86400}
\]

Rainfall (R) is simply taken as the daily measured value and run-off (RO) is assumed only to occur when the soil is at field capacity (soil moisture deficit (SMD) = 0); this assumption has been validated by field measurements (Smethurst, et al., 2006). Historical records of rainfall are relatively easy to obtain for regions of the UK. Regional potential evapotranspiration has been calculated as outlined in Section 7.2.3.2 using the Blaney and Criddle (1950) approach dependent on temperature and site-specific factors.

Figure 7.11a shows annual cumulative rainfall for Newbury cutting and South East England (HadUKP) (Alexander & Jones, 2001) over the control period. It can be seen that the annual cumulative rainfall from HadUKP data for South East England is similar to the rainfall monitored at the site during the control period. The HadUKP data does not appear to capture extreme wet events leading to much higher cumulative site-specific rainfall during 2007 and 2008, but in general the trend of data is of the same profile. In the same way, Figure
7.11b shows that the potential evapotranspiration estimated using the more simplistic Blaney and Criddle (1950) methodology produces the same magnitude of annual cumulative potential evapotranspiration as the monitored data using the Penman–Monteith approach.

Finally, the soil moisture deficit profile obtained using the Southern General weather data is a very close match to that created by Smethurst, et al., (2012). To assess the relationship between the site-specific and Southern General soil moisture deficit, the regression of the line $y = x$ for the two approaches has been considered, this can be seen in Figure 7.12.
Figure 7.11 Comparison of site-specific and regional weather data over control period 2003 to 2012 (monitored data after, Smethurst, et al., 2012): a) cumulative annual rainfall; b) cumulative annual potential evapotranspiration; c) soil moisture deficit
The $R^2$ value of 0.77 between the two data sets in Figure 7.12 shows that there is a significant correlation between the two data sets and indicates that the method and data sets used to create the Southern General SMD data sequence is appropriate to facilitate hindcasting and forecasting of the weather sequence.

Using the method described in Section 7.2.2 the following SMD sequence from the year of construction of Newbury cutting in 1997 to the end of 2017 has been created. In line with the method in Section 7.2.2, daily boundary conditions are obtained considering the daily change in soil moisture deficit and turning it into a flux for application within the numerical analysis.
7.3.2 Validation of Hydrological Model

In the following section, the hydrological behaviour of the numerical model for Newbury cutting has been validated against field monitored pore water pressure records for the period 2003 to 2008. All 17 locations shown in Figure 7.4 have been compared and presented. The modelled results presented employ a bulk water modulus of 1E8 Pa and air modulus 1E5 Pa. The numerical analysis considers the entire life-cycle of the slope from construction in 1997 (i.e. including undrained unloading and subsequent dissipation of negative excess pore water pressures) but results of hydrogeological behaviour are only shown for the monitored period (i.e. 2003 to 2008).
Figure 7.14 Comparison of monitored and modelled pore water pressures for Newbury cutting piezometers A (monitored data after, Smethurst, et al., 2012); a) 1.0m depth; b) 1.5m depth; c) 2.0m depth; d) 2.5m depth; e) 3.5m depth
Figure 7.15 Comparison of monitored and modelled pore water pressures for Newbury cutting piezometers B (monitored data after, Smethurst, et al., 2012); a) 1.0m depth; b) 1.5m depth; c) 2.0m depth; d) 2.5m depth; e) 3.5m depth
Figure 7.16 Comparison of monitored and modelled pore water pressures for Newbury cutting piezometers C (monitored data after, Smethurst, *et al.*, 2012); a) 1.0m depth; b) 1.5m depth; c) 2.0m depth; d) 2.5m depth
Figure 7.17 Comparison of monitored and modelled pore water pressures for Newbury cutting piezometers D (monitored data after, Smethurst, et al., 2012); a) 1.0m depth; b) 1.5m depth; c) 2.0m depth

From visual inspection it can be seen that the near-surface pore water pressure cycles, and therefore stress cycles, are similar. The magnitude of maximum and minimum pore water pressure within a seasonal cycle has been captured successfully. However, the timing of the maximum and minimum pore water pressures obtained from the numerical analyses do not necessarily coincide with the same times as those monitored. At depth the numerical model does not capture the magnitude of pore water pressure cycles as observed in reality. This behaviour is the same as for the validation of the centrifuge experimentation presented in Chapter 5. Clearly, physical behaviours are not being captured within the numerical analysis for the modelling of pore water pressure cycles at depth. This can be accounted for due to the fact that the numerical analysis does not take into account the effect of preferential flow routes within the soil increasing the hydraulic conductivity
of the soil mass, and therefore increasing the magnitude of stress cycles at depth. Assumptions must be made in the modelling of a real slope and the static soil water retention properties not including hysteresis and static saturated hydraulic profile can capture near surface stress cycles but does not perform as well at depth. However, seasonal ratcheting and failures observed across the infrastructure network (i.e. Briggs, et al., 2017) are shallow in nature and the numerical model replicates stress cycles within this near-surface region well. Correlation coefficients between the measured and modelled pore water pressure records have been presented in Table 7.6.

Table 7.6 Correlation coefficients for measured and modelled pore water pressures

<table>
<thead>
<tr>
<th>Piezometer Depth</th>
<th>Group A</th>
<th>Group B</th>
<th>Group C</th>
<th>Group D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0m</td>
<td>0.236</td>
<td>0.452</td>
<td>0.314</td>
<td>0.390</td>
</tr>
<tr>
<td>1.5m</td>
<td>0.373</td>
<td>0.338</td>
<td>0.305</td>
<td>0.198</td>
</tr>
<tr>
<td>2.0m</td>
<td>0.281</td>
<td>0.336</td>
<td>0.043</td>
<td>-0.149</td>
</tr>
<tr>
<td>2.5m</td>
<td>0.229</td>
<td>0.083</td>
<td>-0.181</td>
<td>-</td>
</tr>
<tr>
<td>3.5m</td>
<td>0.222</td>
<td>0.135</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The correlation coefficients presented in Table 7.6 are all relatively low with only a few demonstrating a slight correlation between the measured and modelled data. There are a number of uncertainties within this work from the spatial and temporal variability in weather, spatial and temporal variability in soil material properties along with the assumptions within the numerical modelling approach (i.e. wetting and drying hysteresis of soil water retention properties is not accounted for, water and air are assumed to be idealised, etc.). However, over the period for which data is available, the numerical model has been able to replicate the general trends in behaviour and the corresponding stress cycles occurring in the near-surface due to daily real weather boundary conditions. Therefore, if the mechanical model for the material being modelled is representative of the real soil then deterioration due to seasonal stress cycles and seasonal ratcheting will be captured within the numerical analysis.

It should be noted that the hydrogeological properties used within the numerical analysis were selected to reflect deterioration of the near-surface zone. This was done through the inclusion of a high saturated hydraulic conductivity profile and different soil water retention properties reflecting the
level of weathering of the London Clay at Newbury cutting, see Section 7.2.3.5. Including these behaviours has been critical to the successful replication of near-surface pore water pressure cycles. Of particular note are the difference in the results at piezometer group A, as these points are in weathered material and the hydrogeological properties in the numerical analysis reflect that. The pore water pressure cycles in piezometer group A are less than the other groups and this difference has been captured within the numerical analysis results. This indicates the importance in correctly modelling all soil deterioration processes and also the significance hydrogeological deterioration has on weather-driven stress cycles and slope behaviour.

7.3.3 Mechanical Model Results

As the hydrological results have been shown to reproduce similar stress cycles in the near-surface of the slope, the mechanical behaviour of the numerical analysis from construction (i.e. start of 1997) to the end of 2017 have been presented in Figure 7.18. For clarity, Figure 7.18a shows the pore water pressure cycles at a depth of 1.0m for Group B piezometers from construction to the end of 2017.

Mid-slope horizontal and vertical displacements are shown and it can be seen that there is a large amount of horizontal displacement accumulating over the 20 years of analysis and that the accumulation of displacement is gradual, but that failure has not been observed, which is consistent with the Newbury cutting being stable at the present time. The vertical movement indicates swelling, this can be attributed to the dissipation of excess pore water pressures post-excavation. However, it can be seen that the last few seasonal cycles have led to outward and downward movement indicative of seasonal ratcheting. Figure 7.18d shows the mid-slope movement and there is the distinctive shrink-swell behaviour associated with seasonal ratcheting.
Figure 7.18 Mechanical behaviour of Newbury cutting numerical analysis; a) piezometer B 1.0m depth (monitored data after, Smethurst, et al., 2012); b) mid-slope horizontal displacement; c) mid-slope vertical displacement; d) mid-slope displacement

There is no data to allow validation of the mechanical behaviour of the numerical model against measured data, only the fact that Newbury cutting appears to be stable and has not failed yet. For slopes at the angle of Newbury
cutting, Skempton (1977) observed first-time failures in London Clay between 46 to 65 years post-construction.

Scott, et al., (2007) observed seasonal shrink-swell movements in grass covered embankments in end-tipped London Clay to be around 5 to 8mm and Kovacevic, et al., (2001) indicated vertical movements in the order of ±30mm annually in monitored clay embankments. Obviously, the material characteristics of an embankment differ considerably compared to a cut slope, however, these displacement criteria can be used as a benchmark to assess the near-surface movements. Figure 7.18d shows that the shrink-swell cycles are considerably larger than ±30mm annually.

To allow consistent interpretation of the magnitude of shrink-swell movements, the vertical displacement of the crest of the slope from the numerical analysis have been shown Figure 7.19.

Figure 7.19 Crest of slope vertical displacement from Newbury cutting numerical analysis

Figure 7.19 shows that the displacements obtained from the numerical analysis are considerably greater, with seasonal movements of greater than three times what would be expected for a London Clay Fill (i.e. ±30mm). As the Newbury material is in situ London Clay the displacements would be anticipated to be even less. It can therefore be assumed that the stiffness relationship adopted for the modelling of London Clay subjected to repeated wetting and drying stress cycles is unrealistic. An investigation into the effect of stiffness relationship on behaviour is presented in Section 7.5.
7.4 Effect of Water and Air Bulk Moduli

Given the results presented within Section 7.3, it has been shown that it is possible to replicate realistic seasonal pore water pressure and therefore stress cycles within the near-surface of a high-plasticity clay cut slope, a parametric investigation into the influence of different water and air bulk moduli on the numerical models has been undertaken. This has been considered as within two-phase flow analyses high bulk modulus of water and air compared to the bulk stiffness of the soil will lead to very slow convergence of solution due to the time-stepping relationship where the minimum stable time-step is calculated as follows;

\[
\Delta t = L^2_z n. \text{min} \left( \frac{1}{k_w K_w}, \frac{1}{k_g K_g} \right)
\]

Where;

- \( L_z \) = smallest zone size within the analysis;
- \( n \) = porosity;
- \( k_w \) = saturated mobility function of water as defined in Equation A.9;
- \( k_g \) = saturated mobility function of air (\( k_g = k_w \mu w / \mu_g \));
- \( K_w \) = bulk modulus of water.

Therefore, there is an inverse relationship of time-step and bulk modulus of water and air. Within the FLAC manual it is suggested for numerical purposes it is unnecessary for the bulk modulus of water to exceed 20 times \((K + 4/3G)n\) and that the bulk air modulus (as a default) should be \(10^{-3}\times\) the water bulk modulus (Itasca Consulting Group, Inc., 2011).

To assess the effect of different bulk moduli of water and air, analyses with different combinations of the two parameters considering the slope geometry and material properties presented for Newbury cutting within Section 7.2.3 has been undertaken. The combinations of parameters considered are summarised in Table 7.7.
Table 7.7 Combinations of water and air bulk moduli considered

<table>
<thead>
<tr>
<th>Analysis Ref</th>
<th>Water Bulk Modulus (Pa)</th>
<th>Air Bulk Modulus (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0E7</td>
<td>1.0E7</td>
<td>1.0E4</td>
</tr>
<tr>
<td>5.0E7</td>
<td>5.0E7</td>
<td>5.0E4</td>
</tr>
<tr>
<td>1.0E8</td>
<td>1.0E8</td>
<td>1.0E5</td>
</tr>
<tr>
<td>2.5E8</td>
<td>2.5E8</td>
<td>1.0E5</td>
</tr>
</tbody>
</table>

The results and implications of these analyses are presented and discussed in Section 7.4.1.

### 7.4.1 Results

As discussed in Section 7.4, the bulk moduli of water and air have a significant implication on the time-step and therefore the time required for an analysis to be completed. To assess the effect of different moduli on numerical model behaviour four combinations of water and air bulk moduli, presented in Table 7.7, have been considered and the results presented. Initially, the hydrogeological behaviour of the numerical analyses are compared against the monitored data for Newbury cutting (i.e. 2003 to 2008), and then the mechanical behaviour of the numerical analyses are presented from construction to the end of 2017.
Figure 7.20 Comparison of numerical analyses hydrogeological behaviour for different water and air bulk moduli (monitored data after, Smethurst, et al., 2012); a) piezometer A 1.0m depth; b) piezometer B 1.0m depth; c) piezometer C 1.0m depth; d) piezometer D 1.0m depth

For the monitored period of pore water pressures for Newbury cutting, it can be seen that there is very little difference in the hydrological behaviour, and therefore stress cycles driven within the numerical analysis depending on the bulk moduli of water and air used. There is a difference of 6.2kPa between the 1.0E7 analysis and the 2.5E8 analyses on the 26/12/07 for piezometer C at 1.0m depth. In most instances, with the exception of the 1.0E7 analysis, the...
analyses cannot be distinguished through visual inspection. Mechanical behaviour of the different analyses is presented in Figure 7.21.

Figure 7.21 Comparison of numerical analyses mechanical behaviour for different water and air bulk moduli; a) mid-slope horizontal displacement; b) mid-slope vertical displacement; c) mid-slope displacement

Considering the little difference in hydrogeological behaviour when different bulk moduli for water and air were included within the numerical analyses (shown in Figure 7.20), the difference in mechanical behaviour is surprising. It
can be seen that the use of very low bulk modulus for water (i.e. 1.0E7Pa) has led to the slope failing prematurely within the numerical analysis. Considering Figure 7.21a, it appears that as the bulk modulus of water increases behaviour converges, shown by the small changes in accumulation of horizontal movement between 5.0E7, 1.0E8 and 2.5E8. However, when considering vertical displacement, the behaviour between the different analyses are all very different.

The results show the sensitivity of the numerical modelling framework to the effect of different water and air bulk moduli. In the best-case scenario, the real values of water and air bulk moduli should be used. However, there is a significant increase in computational time required for the use of these real parameter values that makes them problematic to use within numerical analyses. Therefore, a compromise must be made between model run-time and realistic nature of these parameters. From the results presented, due to the convergence of accumulation of horizontal displacements in Figure 7.21a, the bulk moduli for water and air have been taken as 1.0E8Pa and 1.0E5Pa respectfully throughout the numerical analyses considering the unsaturated framework. This has been done to reduce computational time whilst keeping the stiffness of water far greater than the stiffness of the soil.

7.5 Effect of Soil Stiffness

In a similar way to water and air bulk moduli, the effect of material stiffness relationship on the behaviour of numerical analyses of Newbury cutting has been investigated. Numerical analyses of the same time period and material properties for Newbury cutting presented previous, in Section 7.2.3, have been considered. A number of stiffness relationships for London Clay have been considered in this comparison and have been summarised in Equation 7.10 and Table 7.8.

\[ E = \lambda (\sigma' + 100) ; \min E_{\min} \text{kPa} \]

<table>
<thead>
<tr>
<th></th>
<th>Stiffness A</th>
<th>Stiffness B</th>
<th>Stiffness C</th>
<th>Stiffness D</th>
<th>Stiffness E</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \lambda )</td>
<td>25</td>
<td>25</td>
<td>12.5</td>
<td>12.5</td>
<td>6.25</td>
</tr>
<tr>
<td>( E_{\min} ) (kPa)</td>
<td>4000</td>
<td>500</td>
<td>2000</td>
<td>500</td>
<td>500</td>
</tr>
</tbody>
</table>
The relationships have been chosen to represent a fairly soft material to reflect the fact that numerous cycles of wetting and drying stress cycles are known to lead to stiffness deterioration. The rate of stiffness deterioration is unknown so arbitrary relationships were proposed and applied throughout the length of an analysis. The results of the analyses with different stiffness for Newbury cutting are presented and discussed in Section 7.5.1.

![Different stiffness relationships for London Clay](image)

**Figure 7.22** Different stiffness relationships for London Clay

### 7.5.1 Results

As described in Section 7.5, five different stiffness relationships have been considered to assess the mechanical behaviour of a numerical model of Newbury cutting. This has been done due to the significant vertical movement observed at the crest the numerical analysis of Newbury cutting (see Figure 7.19). As with the analyses considering different water and air bulk moduli, firstly the hydrogeological behaviour of the numerical analyses compared to monitored data from Newbury cutting has been considered (see Figure 7.23) followed by the mechanical behaviour of the numerical analyses (see Figure 7.24).
Figure 7.23 Comparison of numerical analyses hydrogeological behaviour for different stiffness relationships (monitored data after, Smethurst, et al., 2012); a) piezometer A 1.0m depth; b) piezometer B 1.0m depth; c) piezometer C 1.0m depth; d) piezometer D 1.0m depth

Figure 7.23 shows the near-surface pore water pressure cycles, and therefore stress cycles, at different locations for Newbury cutting assuming different stiffness relationships. It can be seen that the less stiff the material model (i.e. E compared to A) the smaller the magnitude of pore water pressure cycles generated within the numerical analysis.
What the results do show is that as the soil stiffness reduces, which will happen within a real slope subjected to wetting and drying stress cycles, the magnitude of pore water pressure cycles the slope experiences reduce. The mechanical behaviour of the different analyses is presented in Figure 7.24 and Figure 7.25.

Figure 7.24 Comparison of numerical analyses mechanical behaviour for different stiffness relationships; a) mid-slope horizontal displacement; b) mid-slope vertical displacement; c) mid-slope displacement
Figure 7.24 shows very little difference in the mechanical behaviour of the analyses considering different stiffness relationships. The amount of horizontal displacement at mid-slope for all models is within less than 100mm and the vertical displacement within less than 90mm. Considering the numerical analyses have been subjected to 20 years of weather boundary conditions, this difference is small. As done previously, the shrink-swell behaviour of the crest of the slope has been considered in line with the suggestion that ±30mm of vertical movement has been observed for clay embankments (see Section 7.3.3). These can be seen in Figure 7.25.

The mechanical behaviour at this point differs considerably depending on the stiffness relationship used in the numerical analysis. For stiffness A -363mm of vertical displacement are recorded at the end of 2016 and only -73mm for stiffness E. From the results, it can be assumed that stiffness A and B are not representative of the material as the displacements indicate the slope is nearing failure, which there is no evidence of. There is very little difference in the behaviour of the other stiffness relationships, the displacements are still larger than ±30mm annually but the overall trend in movement is shrink-swell only without the indication of failure.

7.6 Long-Term Behaviour of Hypothetical Cut Slopes in London Clay (Effect of Root Reinforcement)

Given the work conducted to reproduce hydrogeological behaviour and assess the effects of different stiffness relationships, the material properties used for the modelling of Newbury cutting should be indicative of other cut slopes in
London Clay. From records, many cut slopes across the railway network were constructed during the Victoria era (i.e. 1850 to 1900) therefore, these slopes have been subjected to a significantly higher number of seasonal stress cycles than Newbury cutting but many are currently still stable. To investigate this observed behaviour, a hypothetical slope of the same geometry and material properties (with the exception of the properties presented in this section) as Newbury cutting (see Section 7.2.3) has been subjected to a weather sequence from the beginning of 1931 to the end of 2017. The weather sequence has been created for this time period due to the availability of regional weather data from historical records being used within this work (see Section 7.2.3.2 and Table 7.3).

To facilitate this modelling the strain-softening behaviour used to model London Clay has been enhanced to include residual strength at very large strains in addition to post-peak strain-softening to critical state / fully softened. This has not been included in the London Clay models up to this point as the strain-softening behaviour had replicated previous work by others (e.g. Potts, et al., 1997). However, given the results presented in Chapter 5, the importance of rate of strength reduction post-peak and post-critical state for shallow first-time failures due to wetting and drying stress cycles has been shown.

In addition to the change in the strain-softening model being used, the effect of additional near-surface cohesion has been included to assess the influence of root reinforcement on slope behaviour. The analyses within this section use stiffness relationship C from Section 7.5.

### 7.6.1 Strain-Softening Model

Within the results presented in Chapter 5, the effect of strain-softening behaviour on the numerical analyses results was considered. It was shown that for shallow angled slopes, the rate of strength reduction from critical state to residual influences the time to failure of a slope considerably. In addition, Ellis and O'Brien (2007) showed the significance of including post-rupture strength into numerical analyses of strain-softening for London Clay. From laboratory studies, it has been shown that it can take hundreds of millimetres...
of displacements for residual shear strength to be achieved in overconsolidated clays for principle shear surfaces in intact clay (i.e. more than 10cm – see Section 2.2.6) (Skempton & Petley, 1967). All these behaviours have been included in the decision-making process for the appropriate strain-softening model for the hypothetical cut slope models. The strain-softening model used has been shown graphically in Figure 7.26.

![Figure 7.26 London Clay strain-softening behaviour for hypothetical cut slope analysis](image)

**7.6.2 Effect of Root Reinforcement**

One of the main benefits of roots is the additional strength that they provide to the near-surface of a slope. However, up to this point this benefit has not been included in the numerical analyses due to the complexity of modelling root reinforcement and the shortage of measured data providing meaningful parameters for the inclusion in continuum analysis of soil. However, to assess the potential influence of roots, additional cohesion has been included in the upper 2m of slope models. It is appreciated that this method of including root reinforcement is not representative of the physical problem (i.e. root breakage and pull-out are not modelled), but it allows additional benefits of roots to be included in the analyses.

As the exact strength increase due to grass roots in London Clay is not well-established, two analyses have been run for the hypothetical cut slope; the different analyses are summarised in Table 7.9.
Table 7.9 Summary of analyses considering root reinforcement and long-term behaviour of a hypothetical London Clay cut slope

<table>
<thead>
<tr>
<th>Reference</th>
<th>Root Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hypothetical Slope A</td>
<td>+2kPa Cohesion</td>
</tr>
<tr>
<td>Hypothetical Slope B</td>
<td>+5kPa Cohesion</td>
</tr>
</tbody>
</table>

7.6.3 Weather Sequence

Using the method described in Sections 7.2.2 and 7.2.3.2, regional weather data (see Table 7.3), calibrated to the control period weather data for the Newbury cutting site, has been used to create daily boundary conditions from the start of 1931 to the end of 2017. The soil moisture deficit for the weather sequence is presented in Figure 7.27.
Results

As discussed, two hypothetical cut slope analyses considering long-term behaviour and different root reinforcement criteria have been investigated. A different strain-softening model has been implemented to allow residual strength to be reached at very high plastic strains (see Figure 7.26) and the

Figure 7.27 Southern general soil moisture deficit profile: 1931 to 2017; a) 1931 to 1960; b) 1961 to 1990; c) 1991 to 2017

7.6.4 Results

As discussed, two hypothetical cut slope analyses considering long-term behaviour and different root reinforcement criteria have been investigated. A different strain-softening model has been implemented to allow residual strength to be reached at very high plastic strains (see Figure 7.26) and the
upper 2m of the slope models have increased cohesion to mimic root reinforcement. For the analyses considered, hydrogeological and mechanical behaviour are presented and compared in Figure 7.28.

Figure 7.28 Comparison of hydrogeological and mechanical behaviour of different hypothetical slope analyses for different root reinforcement conditions; a) pore water pressure cycles at piezometer A 1.0m depth b) mid-slope horizontal displacement; c) mid-slope vertical displacement; d) mid-slope displacement
Figure 7.28a shows that the hydrogeological behaviour of the two slope analyses are the same for the period up to failure of hypothetical slope A. Figure 7.28b and c show a considerable difference in the time and number of seasonal stress cycles for the different slopes to fail.

As expected for the slope with greater root reinforcement (i.e. higher increase in surface cohesion) remains stable for longer than the model with less root reinforcement, 58.9 years compared to 36.5 years. The results show the significance of the effect of root reinforcement and the additional life that a slope can have due to increased strength in the near-surface of the slope. Figure 7.28d indicates that within the numerical analyses there is a threshold displacement at which ultimate failure begins, at approximately 1.0m of horizontal displacement. This behaviour is due to the strain-softening criteria used within the analyses.

To investigate the influence of the weather sequence and the point of failure, cumulative rainfall and soil moisture deficit have been considered along with the conditions at failure of the different analyses in Figure 7.29 and Figure 7.30 respectively.
Figure 7.29 shows that the failures of the different slope analyses both occurred during years that were not overly wet. In fact, slope A experienced 9 individual years with greater rainfall and slope B experienced 37 years with greater rainfall prior to failure. This clearly demonstrates long-term deterioration of the slope and the effect on the magnitude of triggering events.

It can also be seen that the two years prior to failure of both slopes were particularly wet. This agrees with understanding that failures of slopes in low saturated hydraulic conductivity soils are dependent on long antecedent conditions prior to triggering events. Whilst annual rainfall indicates the slopes have experienced wet conditions, soil moisture deficit gives a more rigorous consideration of wetting and drying behaviour and has been considered for the different slope analyses conducted, presented in Figure 7.30.
As with Figure 7.29, Figure 7.30 shows that prior to failure both slopes experienced a number of wet years (i.e. lower soil moisture deficit) and that the conditions at failure were not the worst that the slope had experienced within its life. Slope A experienced 15 worse (i.e. wetter) years than the year of failure and slope B experienced 23 years. The results show the significance of seasonal stress cycles causing strength deterioration leading to shallow first-time failure.

7.7 Climate Change Assessment for Newbury Cutting

Bringing together the lessons learnt regarding material parameters, a single numerical model representative of Newbury cutting has been analysed with the only variable being the weather data applied to the numerical analyses past the end of 2017. This has been done to stress test the cut slope under potential future climate scenarios. Climate change stress testing scenarios
have been obtained using the statistical down-scaling model (Wilby, et al., 2014) as described in 2.5.4. The approach used to create synthetic weather sequences for present and future climate scenarios is presented in detail in Section 7.7.1.

Stiffness relationship C has been used along with the strain-softening model shown in Figure 7.26 and root reinforcement has been assumed to be an additional 2kPa cohesion within the upper 2m of the slope models.

7.7.1 Synthetic Weather Sequences (Weather Generator)
A statistical down-scaling model has been used to create a weather generator allowing synthetic weather data to be generated given known base period weather data. In line with global climate models and UKCP09, a base period of 1961 to 1990 has been used. Base period weather data uses the same regional weather data sets as previously used to generate the southern general weather sequence, presented in Table 7.3.

For the statistical down-scaling model, the trends investigated over the base period can be considered as unconditional;

- daily temperature (°C);
- number of wet days;

and conditional;

- daily rainfall amount (conditional on it being a wet day) (mm).

Using the base period data for number of wet days, it is possible to obtain the average number of wet days in each calendar month. The same can be done for daily temperature but rather than the average for a month a cumulative frequency distribution of all the daily temperatures in that month across all base period years can be obtained, an example is shown in Figure 7.31.
The same procedure can be taken for daily rainfall amount when a wet day occurs, this is shown in Figure 7.32.

This procedure has been done for all months of the year and the data then allows the random selection of temperature and rainfall values that are representative of the baseline data. A summary of all base period trends is presented in Section 7.7.4.1. The weather generator being used has been developed within Microsoft EXCEL and uses the random number generator function to select values of temperature, if it is a wet day and the daily rainfall value. The method takes into account the probability of occurrence of events in the generation of weather data, to illustrate the effectiveness of the method, the cumulative probability distribution of synthetic (i.e. generated) weather data for 500 random days in January assuming no change in the climate are shown.
in Figure 7.33. The results shown in Figure 7.33 show that the method developed generated weather data randomly that is representative of the base period data.

Using the method to determine daily temperature, if it is a wet day and the volume of daily rainfall randomly each day provides synthetic weather data that allows a sequence of soil moisture deficit to be generated (as described in Sections 7.2.1 and 7.2.2) and boundary conditions to be created for numerical analyses.

![Graphs showing cumulative probability of daily temperature and rainfall](image)

Figure 7.33 Synthetic weather generator validation for 500 random days – January 1961 to 1990; a) daily temperature; b) daily rainfall on a wet day

The method for creating synthetic weather allows very easy factoring of weather data for the inclusion of climate change scenarios to allow stress testing of the physical system being modelled. For example, if it is 30% wetter in the month of January than the base period data then the following cumulative distributions for the base period and climate change scenario will
be obtained, see Figure 7.34. In the same way that daily synthetic weather data was created using the base period cumulative probability distribution, random number generators including the new cumulative probability distribution are used to create daily weather data for climate change scenarios.

![Figure 7.34 Including climate change into weather generator](image)

### 7.7.2 Climate Change Projections (UKCP09)

One of the benefits of the statistical down-scaling approach for climate change assessments is the fact that the scenarios tested are not directly dependent on climate change projections obtained from global climate models. Instead, the physical system is stress tested to see how it responds to different synthetic weather sequences. However, for the stress testing to be realistic the scenarios considered should be representative of expected climate change projections. Table 7.10 to 7.13 summarise the headline changes projected by UKCP09 for the South East of England. These projected changes have been used in the decision-making process for the selection of climate scenarios for stress testing Newbury cutting.
Table 7.10 Change in mean winter temperature (after, UKCP09)

<table>
<thead>
<tr>
<th>Change in Mean Winter Temperature (°C)</th>
<th>South East England</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probability Level</td>
<td></td>
</tr>
<tr>
<td>Low Emissions Scenario 10% 50% 90%</td>
<td></td>
</tr>
<tr>
<td>2020s 0.5 1.3 2.1</td>
<td></td>
</tr>
<tr>
<td>2050s 0.9 2 3.1</td>
<td></td>
</tr>
<tr>
<td>2080s 2 2.6 4</td>
<td></td>
</tr>
<tr>
<td>Probability Level</td>
<td></td>
</tr>
<tr>
<td>Medium Emissions Scenario 10% 50% 90%</td>
<td></td>
</tr>
<tr>
<td>2020s 0.6 1.3 2.2</td>
<td></td>
</tr>
<tr>
<td>2050s 1.1 2.2 3.4</td>
<td></td>
</tr>
<tr>
<td>2080s 1.6 3 4.7</td>
<td></td>
</tr>
<tr>
<td>Probability Level</td>
<td></td>
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<tr>
<td>High Emissions Scenario 10% 50% 90%</td>
<td></td>
</tr>
<tr>
<td>2020s 0.6 1.4 2.2</td>
<td></td>
</tr>
<tr>
<td>2050s 1.4 2.5 3.8</td>
<td></td>
</tr>
<tr>
<td>2080s 2 3.7 5.7</td>
<td></td>
</tr>
</tbody>
</table>

Table 7.11 Change in mean summer temperature (after, UKCP09)

<table>
<thead>
<tr>
<th>Change in Mean Summer Temperature (°C)</th>
<th>South East England</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probability Level</td>
<td></td>
</tr>
<tr>
<td>Low Emissions Scenario 10% 50% 90%</td>
<td></td>
</tr>
<tr>
<td>2020s 0.5 1.7 2.8</td>
<td></td>
</tr>
<tr>
<td>2050s 1.1 2.5 4.1</td>
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<tr>
<td>2080s 1.4 3 5.1</td>
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<tr>
<td>Probability Level</td>
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<tr>
<td>Medium Emissions Scenario 10% 50% 90%</td>
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</tr>
<tr>
<td>2020s 0.6 1.6 2.7</td>
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<tr>
<td>2050s 1.3 2.8 4.6</td>
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</tr>
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<td>2080s 2 3.9 6.5</td>
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<td>2020s 0.5 1.5 2.7</td>
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<td>2050s 1.4 3.1 5.2</td>
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<tr>
<td>2080s 2.6 4.9 8.1</td>
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### Table 7.12 Change in mean winter precipitation (after, UKCP09)

<table>
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<th>Medium Emissions Scenario</th>
<th>High Emissions Scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10%</td>
<td>50%</td>
<td>90%</td>
</tr>
<tr>
<td>2020s</td>
<td>4</td>
<td>7</td>
<td>19</td>
</tr>
<tr>
<td>2050s</td>
<td>1</td>
<td>13</td>
<td>30</td>
</tr>
<tr>
<td>2080s</td>
<td>4</td>
<td>18</td>
<td>40</td>
</tr>
</tbody>
</table>

### Table 7.13 Change in mean summer precipitation (after, UKCP09)

<table>
<thead>
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<th>Probability Level</th>
<th>Low Emissions Scenario</th>
<th>Medium Emissions Scenario</th>
<th>High Emissions Scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10%</td>
<td>50%</td>
<td>90%</td>
</tr>
<tr>
<td>2020s</td>
<td>24</td>
<td>7</td>
<td>13</td>
</tr>
<tr>
<td>2050s</td>
<td>37</td>
<td>14</td>
<td>9</td>
</tr>
<tr>
<td>2080s</td>
<td>39</td>
<td>15</td>
<td>13</td>
</tr>
</tbody>
</table>
7.7.3 Climate Change Impact Assessment (Stress Testing)

Considering the change factors presented in Section 7.7.2, a number of different synthetic weather sequences have been created by factoring the base period weather data as shown previously in Section 7.7.1. The scenarios considered for stress testing are summarised in Table 7.14. The scenarios have been selected to investigate the effect of inter-seasonal variation due to climate change projections. The synthetic weather sequences created have been applied to numerical analyses representative of Newbury cutting from the end of 2017. This has been done in order to represent existing clay cut slopes that are currently stable but will experience different stress cycles in the future due to climate change.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Winter Precipitation (%)</th>
<th>Summer Precipitation (%)</th>
<th>Winter Temp. (°C)</th>
<th>Summer Temp. (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current Climate 1</td>
<td>0%</td>
<td>0%</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Climate Change 1</td>
<td>30%</td>
<td>0%</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Climate Change 2</td>
<td>30%</td>
<td>-15%</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Climate Change 3</td>
<td>30%</td>
<td>-30%</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Climate Change 4</td>
<td>30%</td>
<td>-30%</td>
<td>+3.5</td>
<td>+5.0</td>
</tr>
</tbody>
</table>

The scenarios considered have been chosen to test a range of behaviours indicative of wetter winters and drier summers. The first scenario, current climate 1, has been considered as the baseline analysis assuming there is no change in weather patterns due to climate change. Climate change 1 assumes that winters become wetter with no change to summer precipitation or seasonal temperature. Climate change 4 includes the most significant change to seasonal cycles with increased temperature, reduced rainfall in the summer and increased winter rainfall (representative of UKCP09 high emissions 50% probability level). The other scenarios are in-between climate change 1 and 4.

The behaviours of the different analyses have been compared and results presented in Section 7.7.4.
7.7.4 Results
As discussed, a numerical model representative of Newbury cutting has been analysed to the end of 2017 and then different synthetic weather sequences have been applied to the model. This approach has been adopted as it is representative of many existing infrastructure slopes that are currently standing but will be subjected to different stress cycles in the future due to climate change.

Synthetic weather sequences have been created using statistical down-scaling and considering the change factors given by UKCP09. Winter month precipitation, summer month precipitation, winter temperature and summer temperature have been varied to account for inter-seasonal variation due to climate change projections. Due to the computational intensity of the numerical analyses only five scenarios have been considered. Firstly, the different synthetic weather sequences created are presented followed by the results of the different analyses. The different analyses considered are summarised in Table 7.14.

The numerical analyses considering different climate scenarios have been run considering root reinforcement through the inclusion of a 2kPa increase in cohesion throughout the upper 2m of the slope surface. The numerical analyses results show relative behaviour for different climate scenarios but times to failure are not correct for Newbury cutting due to limitations in the modelling approach.

7.7.4.1 Synthetic Weather Sequences
The statistical down-scaling method adopted for creating synthetic weather sequences has been presented in detail in Section 7.7.1. The base period weather data used has created the following trends in monthly behaviour, see Figure 7.35. These trends can be used to randomly generate current weather data and allow climate change scenario data to be obtained by factoring the different cumulative probability distributions.
Figure 7.35 Base period climate trends for synthetic weather generator; a) wet day occurrence; b) daily rainfall on a wet day; c) daily temperature

The soil moisture deficit of synthetic weather sequences for current climate and for climate change scenarios are shown in Figure 7.36.
To illustrate the difference between the current climate and climate change scenarios, the cumulative probability function of the soil moisture deficit for the
complete synthetic weather sequences (i.e. 2018 to 2045) are shown in Figure 7.37.

![Soil moisture deficit cumulative probability for different synthetic weather sequences](image)

Figure 7.37 Soil moisture deficit cumulative probability for different synthetic weather sequences

Figure 7.37 shows that the climate change 1 scenario produces the wettest weather sequence with climate change 2 being very similar to current climate. Climate change 3 and 4 are much drier than the current climate. This range of weather allows a logical assessment of climate change impact on high-plasticity overconsolidated clay cut slopes due to inter-seasonal variation given an understanding of the limitation in the model approach and material properties.

### 7.7.4.2 Numerical Analyses Results

As explained previously, a numerical analysis has been run from construction to the end of 2017 prior to the application of synthetic weather sequences. To obtain the point of failure for these analyses the change in mid-slope displacements for each time-step of the numerical analyses results has been considered and when large displacements occur failure has been detected. This is essentially the same as assessing the reciprocal of velocity to obtain the timing of uncontrolled large magnitude displacement. This is shown in Figure 7.38.
Figure 7.38 Change in mid-slope displacement per time-step for different synthetic weather sequences; a) current climate 1; b) climate change 1; c) climate change 2; d) climate change 3; e) climate change 4

For all the different synthetic weather sequences, it can be seen clearly that there is a point in time where horizontal displacements accelerate, and failure
occurs, see Figure 7.38. The date / time to failure for the different analyses are summarised in Table 7.15.

| Current Climate 1 | 20/04/2027 | 30.3 | - |
| Climate Change 1  | 13/07/2026 | 29.6 | -2.3% |
| Climate Change 2  | 25/04/2027 | 30.3 | 0.0% |
| Climate Change 3  | 30/08/2025 | 28.7 | -5.3% |
| Climate Change 4  | 25/12/2025 | 29.0 | -4.3% |

The difference in time to failure for the different slope analyses is not large, with the largest difference being between climate change 3 and current climate 1, a difference of 1.6 years a 5.3% reduction in life span of the slope. The hydrogeological and mechanical responses of the different analyses are shown in Figure 7.39 and Figure 7.40 respectively.
Figure 7.39 Hydrogeological response of numerical analyses for different synthetic weather sequences – pore water pressure records at piezometer B 1.0m depth; a) current climate 1; b) climate change 1; c) climate change 2; d) climate change 3; e) climate change 4
Figure 7.39 shows the hydrogeological behaviour of the models driven by different synthetic weather data after the end of 2017. In nearly all instances, the pore water pressures reach near hydrostatic during winter months. It can be seen that the pore water pressure cycles and therefore stress cycles are largest within climate change 4. The stress cycles in climate change 4 are also relatively uniform and there does not appear to be an extreme wet year within the period of data shown. The stress cycles in the other models are considerably smaller with significant variability in the magnitude of the stress cycles. Visually, it appears that climate change 2 spends the most time with near hydrostatic conditions compared to the other slope analyses.
Figure 7.40 Mechanical response of numerical analyses for different synthetic weather sequences; a) mid-slope horizontal displacement; b) mid-slope vertical displacement; c) mid-slope displacement

The first observation from Table 7.15 and Figure 7.40, is the fact that the four climate change scenarios all fail prior to or at the same time as the current climate analysis. Following application of synthetic weather data (i.e. end of 2017) Figure 7.40a shows clear deviations in behaviour of mid-slope horizontal movement depending on the synthetic weather sequence applied. Climate change 4 scenario exhibits the largest change in rate of accumulation of horizontal displacement with time. There are subtle changes in the mid-slope vertical displacements of the different analyses with climate change 2 showing
the largest accumulation of vertical displacement (see Figure 7.40b) – climate change 2 also spent longest at near hydrostatic explaining this increased swelling behaviour observed relative to the other analyses.

Finally, the shear surfaces predicted by the numerical analyses for the different synthetic weather sequences are shown in Figure 7.41.

Figure 7.41 Shear surfaces from numerical analyses for different synthetic weather sequences

Figure 7.41 shows that the depth of root reinforcement used in the numerical analyses dictates the depth of the failure surface for all analyses. The failures predicted are translational in nature and shallow, below the root reinforcement. As with previous analyses, these failure surfaces are indicative of failures observed in ageing high-plasticity clay infrastructure slopes (Briggs, et al., 2017).

The results show that climate change has the potential to reduce the serviceable life of these earthwork assets if wetter winters and drier summers are representative of future weather conditions.

7.8 Discussion

The following section presents the discussion surrounding the results presented within this Chapter. Firstly, the validation of the numerical modelling approach against Newbury cutting has been considered along with the implications of the results. Then the results of modelling long-term behaviour, the effects of root reinforcement and climate change assessment have been discussed.

The numerical modelling framework being used is not discussed within this section as it develops on the work presented within Chapters 5 and 6.
7.8.1 Boundary Conditions

A method for obtaining daily boundary conditions using a water balance calculation for application within a numerical analysis of real cut slope behaviour has been presented. It has been shown that monitored pore water pressure trends can be replicated using the boundary conditions developed. In particular, the near-surface pore water pressure cycles generated using the boundary conditions developed have been able to replicate the seasonal magnitude of monitored pore water pressures in a number of locations across the case study slope.

The boundary conditions imposed are relatively simplistic in the data required for their generation making the use of this modelling approach very flexible. However, there are limitations in the method. As a water balance method is used, a constant depth of soil is assumed, soil water content variation is averaged over this constant depth and the soil is assumed to be homogeneous.

The boundary conditions applied have been solved daily, Smith (2003) showed for modelling of infiltration into unsaturated slopes boundary conditions should in fact be applied in hourly increments or less. This is because significant rainfall events can lead to considerable runoff that is not accounted for when daily boundary conditions are employed. However, the focus of the work throughout this project is deterioration rather than modelling the triggering of a failure. Therefore, the application of boundary conditions on a daily basis is acceptable for the problem being investigated within this project.

7.8.2 Hydrogeological Behaviour

Hydrogeological behaviour of a numerical analysis considering Newbury cutting has been validated against 17 piezometers for a six-year period. The near-surface pore water pressure records have been shown to be a very good match between the numerical analysis and the monitored data, with the magnitude of seasonal pore water pressures, and therefore stress cycles, being replicated well. At depth comparison of modelled and monitored pore water pressures are not a good match. This can be accounted for due to;

- preferential flow within fissured clay and due to roots has not been accounted for in numerical analysis;
• soil water retention hysteresis due to wetting and drying has not been included in numerical analysis due to a lack of data;
• a uniform saturated hydraulic conductivity profile was applied across the slope for the near-surface, the data used to create the profile was extremely variable with very few measurements below 3.0m.

Regardless of the limitation in the modelling approach, the behaviour of the numerical analysis reflected monitored data. Important lessons were learnt in this modelling;

• near-surface saturated hydraulic conductivity must be included to allow realistic pore water pressure cycles to be generated;
• soil water retention deterioration must be better understood – this behaviour was included by reducing the air entry value for regions within the slope that were more highly weathered. This proved important in replicating the pore water pressure cycles at piezometer group A.

The numerical model was validated against six years of monitored data, whilst this is a significant amount of data in the context of a cut slope life span 6 years is very small. Therefore, the hydrogeological properties validated are for a snap-shot of this slope and are likely unrepresentative of the material properties years later. It should be noted that the period validated is approximately 11 years after construction. If the hydrogeological properties used in the numerical analysis are representative of the real material properties in the slope, significant deterioration of the soil water retention properties and saturated hydraulic conductivity in the near-surface has occurred in a very short time.

Understanding the rate of deterioration of these properties is critical to developing more realistic numerical models considering shallow first-time failure due to wetting and drying stress cycles.

7.8.3 Mechanical Behaviour
The results of the mechanical response of Newbury cutting given the validation of seasonal pore water pressure and stress cycles of the near-surface has been presented. Currently, there is not data available for the explicit validation
of the displacements shown apart from the fact that Newbury cutting is currently stable and vertical seasonal movement of ±30mm has been observed in clay embankments (Kovacevic, et al., 2001).

In line with these facts, the results presented in Figure 7.18 and Figure 7.19 show seasonal shrink-swell behaviour, seasonal ratcheting and the slope has not failed. However, the annual displacement cycles are far greater than ±30mm vertically. The stiffness relationship adopted within the work is effective stress dependent and has been taken from previous numerical analysis of London Clay cut slopes conducted by Potts, et al., (1997). Their work considered deep-seated mechanisms so does raise questions about the validity of the stiffness model for modelling near-surface processes.

Stiffness of the near-surface of a slope requires significant investigation to improve numerical modelling ability to develop understanding of weather-driven deterioration. It is known that under cyclic stresses stiffness deterioration occurs but currently there is not enough data available for London Clay to establish such a model, especially considering the stress cycles are driven by suctions due to desaturation that will cause desiccation cracking and damage diagenetic bonds. In addition, the effect of roots on the stiffness of the upper zone of a slope should be better understood and included in numerical analyses.

There are significant limitations in the modelling conducted but it has been shown that seasonal movements due to validated near-surface stress cycles can be modelled within the numerical modelling approach developed and boundary conditions applied.

7.8.4 Effect of Root Reinforcement
The inclusion of root reinforcement through additional cohesion for the upper 2m (i.e. rooting depth) of the near-surface of the slope has been performed for two different cohesion values (i.e. different values of strength increase due to root reinforcement). It has been shown that additional cohesion within the upper zone of the slope surface greatly increases the time to failure predicted by the numerical analysis, with the 2kPa reinforcement model failing at 36.5 years and the 5kPa analysis at 58.9 years.
The results show that if numerical modelling capabilities for modelling real slope behaviour – particularly shallow first-time failures due to repeated seasonal stress cycles – are to be improved then more must be known about the effects of root reinforcement and how it can be modelled representatively within a continuum model. The approach for including root reinforcement used here is very crude and does not consider the mechanics of root reinforcement. The behaviour during shearing of roots experiencing pull-out and tensile failure through breakage is not included. There is the additional complication that annual displacements due to seasonal stress cycles are relatively small and will potentially not exceed the displacements for pull-out or tensile breakage of roots. If roots are broken and pulled out due to shearing, there is the potential that roots will re-grow and become as strong as before making the deterioration of root reinforcement a very complex problem to include in numerical analyses. Whilst there are fibre reinforcement models available for modelling root reinforcement (e.g. Cohen, et al., 2009) these consider ultimate failure and not the damage and recovery of roots during an annual cycle of wetting and drying.

7.8.5 Antecedent Conditions

Within the results from modelling root reinforcement, it was shown that failure within the numerical analyses was predicted to occur during wet years that were by no means extreme. The failures were predicted to occur after two to three successive wet years but the magnitude of the wet year causing failure had been exceeded many times previously. These results show the significance of long-term strength deterioration of high-plasticity overconsolidated clay slopes and the importance of antecedent conditions in triggering shallow first-time failures within these slopes.

7.8.6 Climate Change Assessment

A statistical down-scaling model to create synthetic weather sequences to allow stress testing of a numerical model considering Newbury cutting has been conducted with multiple analyses run to assess the effect of inter-seasonal variation due to wetter winters and drier summers. The synthetic weather sequences have been applied following the analysis of Newbury cutting to the end of 2017. This means that the analyses all begin at the same
time for a slope with the same level of deterioration. This is important as the majority of slopes along the infrastructure network within the UK are ageing and have experienced deterioration and are now facing changes in the magnitude of annual stress cycle due to climate change. There has been much debate about the implications of climate change on ageing infrastructure slopes. Some believe that larger seasonal cycles of wetting and drying will lead to more rapid deterioration (Dijkstra & Dixon, 2010) and others believe that prolonged summer drying will result in more stable slopes (Clarke & Smethurst, 2010).

The modelling conducted for the climate change impact assessment for Newbury cutting has shown that the time to failure of the slope is the same or less than the current climate scenario. In all cases, climate change has been shown to be detrimental to the serviceable life of the cut slope. In fact, the numerical model predicted that the analysis with the largest pore water pressure cycles (i.e. driest summers) failed 4.3% earlier than the current climate scenario. This supports the ideas that larger stress cycles increase the rate of strength deterioration of these ageing clay slopes. It should be noted that the times to failure for these analyses are unlikely to be accurate but relative behaviour between analysis show important trends.

The conclusions drawn from this climate change assessment should be considered in line with the limitation of the modelling approach. Root reinforcement has been included as an additional cohesion of 2kPa across the slope surface – as discussed previously this modelling approach for root reinforcement is questionable but must be included. As with other modelling results presented previously, stiffness and hydrogeological deteriorations are not included within the analysis due to lack of data available. The most significant limitation are the boundary conditions, the water balance calculation approach to determine discharge boundary conditions has been shown to work very effectively for generating pore water pressure cycles of a real slope (i.e. validation against monitored pore water pressure data). However, with cycles of stresses hydrogeological deterioration will occur and this will change the soil water retention properties of the soil, therefore affecting the total and readily available water in the rooting zone of the soil. As with hydrogeological
deterioration, changes in stiffness due to cyclic stress cycles will lead to a change in the depth and density of the roots within the near-surface which could change the crop factor coefficient in the water balance calculation. Finally, significant temperature change will result in vegetation change and a different crop factor for the water balance calculation. There are also questions around the implication of increased desiccation cracking due to larger stress cycles with climate change. Increased desiccation cracking will increase hydrogeological, strength and stiffness deterioration of the near-surface but has not been included within these analyses due to a lack of data and understanding of the mechanism.

Climate change weather scenarios have been synthesised considering change in summer and winter month temperature and precipitation. Change in the number of wet days for different months and frequency of extreme events (i.e. storms and droughts) for future scenarios have not been included. The work focuses on the rate of deterioration of the slope assets due to long-term weather patterns so omitting extremes that would likely cause sudden failure within the numerical analyses is a valid and likely conservative assumption.

The uncertainties outlined for obtaining boundary conditions through a water balance calculation and creating synthetic weather data means there are question about the validity of the boundary conditions for future climate scenarios. It should be appreciated that conducting a climate change assessment of such a complex physical problem is challenging and that there are many unknowns when considering the future. However, it is necessary to undertake work considering climate change impacts and the analyses conducted have shown that climate change will likely reduce the serviceable life of high-plasticity overconsolidated clay cut slopes.

7.9 Key Findings

From the work presented within this Chapter, the following key findings have been obtained:

- Daily boundary conditions obtained considering a water balance calculation can be used to drive realistic pore water pressure and
therefore stress cycles in the near-surface of clay slopes, improving the modelling of soil-water-interaction behaviour;

- The inclusion of different soil water retention properties representing different levels of weathering along the slope surface within the numerical analyses is important with respect to capturing realistic hydrogeological behaviour. Much more is needed to be known about how soil water retention properties deteriorate as they influence the magnitude of stress cycles and rate of deterioration of slopes;

- The inclusion of root reinforcement within numerical analyses is extremely important regarding the number of seasonal cycles to failure when considering shallow first-time failures and requires further investigation to improve modelling of root effects in continuum modelling;

- Antecedent conditions a number of years prior to failure are significant in triggering failure in these low saturated hydraulic conductivity slopes. However, the magnitude of trigger events is not the most extreme events experienced within the life-cycle of the slope analyses, illustrating the importance in understanding strength deterioration due to seasonal ratcheting;

- Through stress testing a slope model considering potential future climatic conditions in line with UCKP09, it has been shown that climate change reduces the serviceable life of these earthwork assets relative to current climatic conditions.

## 7.10 Chapter Summary

The results presented within this Chapter progress the previous work presented in Chapter 6 by including and validating more rigorous soil-water-atmosphere interaction boundary conditions. The approach for obtaining daily boundary conditions from real weather data has been presented and the hydrogeological behaviour of a numerical analysis validated against long-term monitoring of a real slope, Newbury cutting. There are uncertainties regarding the mechanical model used for London Clay, the strain-softening model not including strength reduction to residual strength is unrepresentative and the effective stress dependent stiffness relationship is questionable for the
modelling of seasonal ratcheting. Regardless, analyses have been conducted and different mechanical behaviours considered. The most significant observation is the necessity to include root reinforcement within analyses as the benefit greatly increases the serviceable life of the assets within the numerical analyses. More knowledge is needed about root reinforcement and how it can be included representatively within a continuum analysis.

Using a model of Newbury cutting a climate change impact assessment has been conducted including strength reduction to residual and root reinforcement, as an additional cohesion in the upper 2m of the slope. Synthetic weather has been generated using statistical down-scaling methodology to assess the impact of change in annual cycles of wetting and drying. Numerical analyses have predicted that wetter winters and drier summers will reduce the serviceable life of high-plasticity overconsolidated clay slopes.
Chapter 8

Discussion

8.1 Chapter Scope
The following Chapter brings together the uncertainties and limitations of the modelling conducted and the key results. Results and conclusions drawn have been discussed in the context of the limitations highlighted and with consideration of previous work highlighted within the literature review presented in Chapter 2. Initially, the uncertainties in the modelling approaches are discussed followed by key results and finally, the limitations in current knowledge that should be addressed if more meaningful analyses of weather-driven behaviour and climate change effects of overconsolidated clay cut slopes are to be conducted.

8.2 Uncertainty
To facilitate a discussion regarding the results presented in the previous Chapters and the implications of these results, the areas of uncertainty surrounding the results must first be discussed. Areas of uncertainty can be separated into the following categories;

- numerical model frameworks;
- fundamental mechanisms being modelled;
- material input parameters used in the modelling;
- boundary conditions used for modelling; and
- due to lack of knowledge or missing data.
Additional uncertainty can come from inaccuracies within the numerical solver being used (an example being mesh dependency of localisation problems) and inappropriate assumptions made during formulation of numerical analyses.

Uncertainties in the different modelling approaches used and therefore the uncertainty of the results presented have been considered in order of the results Chapters presented. The categories of uncertainty presented previously have been considered for each modelling phase.

8.2.1 Modelling Simplified Cut Slope Behaviour – Uncertainty

The modelling conducted within this section utilised the saturated framework within FLAC 7, see Appendix A. Within this framework, fully saturated flow and scenarios where a phreatic surface exists can be modelled (Itasca Consulting Group, Inc., 2011). In the case of a phreatic surface being modelled, above the phreatic surface the soil is assumed to be completely unsaturated and the air pressure taken as atmospheric. Within this modelling framework, it is assumed that capillary effects, due to desaturation increasing pore water pressure suctions, are negligible. This assumption is only valid for very coarse soils and is not representative of fine-grained soils. The framework includes consolidation theory based on the Biot theory of consolidation to allow hydro-mechanical coupling. It is assumed that the fluid within the numerical analysis is incompressible – this is achieved if the fluid bulk modulus is far greater than the stiffness of the soil.

The assumption regarding capillary effects and pore water pressure suctions in the numerical framework is not valid for modelling clay. However, the modelling being conducted considers saturated behaviour and a phreatic surface near the surface of the slope with very low pore water pressure suctions within the soil above the phreatic surface (i.e. 10kPa suction). The capillary effect due to such a small suction will have limited effect on the numerical results. The assumption regarding incompressibility of the fluid holds true if the soil is fully saturated with a fluid that does not include any dissolved air – which in reality is unlikely. However, the bulk modulus of water with some dissolved air is still significantly less compressible than the soil skeleton being modelled so this assumption is valid.
Regarding the mechanisms being modelled, behaviour is saturated and as such the framework is appropriate. Pore water pressure change due to excavation (i.e. undrained unloading) is modelled by consolidation theory and the rate of dissipation of excess pore water pressures is dictated by the saturated hydraulic conductivity profile and boundary conditions prescribed to the model. Following excavation within the numerical model a simple check calculation of stress changes 1m below the toe of a slope was conducted and compared with the results of the numerical analysis – the results were in close agreement satisfying that this mechanism is being captured within the numerical analysis. From results presented in Figure 4.7, it can be seen clearly that dissipation of excess pore water pressures post-excavation has been modelled successfully. Stress relief is modelled following excavation depending on the horizontal stresses initiated within the numerical analysis and progressive failure modelled using a Mohr-Coulomb strain-softening constitutive model. Three numerical analyses of cut slopes, with different initial stress conditions, replicating work conducted by Potts, et al., (1997) were compared against the results presented by Potts, et al., (1997). The strain-softening model used within the analyses was calibrated to match that of Potts, et al., (1997) and the models showed the same trends in behaviour with failure occurring within 0.4, 1.7 and 6.8 years of the results presented by Potts, et al., (1997). The mechanisms being modelled have been captured within the numerical modelling approach and behaviour validated through replication of previous modelling of these mechanisms.

The material models used within this work for London Clay has been taken from the work by Potts, et al., (1997), with the strain-softening model calibrated against simple shear and triaxial test data. An effective stress dependent stiffness relationship, taking into account the increase in stiffness with depth and change in stiffness as excess pore water pressures dissipate post-excavation has been adopted. Saturated hydraulic conductivity is depth dependent considering the initial ground surface and does not change due to swelling or stress changes post-excavation. More sophisticated stress dependent saturated hydraulic conductivity relationships have been used for modelling simplified cut slope behaviour (Summersgill, 2015) given the
understanding that the material property is depth, and therefore stress, dependent, see Section 2.3.2.1. The boundary conditions applied within this modelling are static and therefore the time to failure of the slope analyses do not reflect real slopes. Any conclusions must be drawn from relative behaviour and not absolute time to failure.

Regarding inaccuracies in the numerical solver, it is known that mesh dependency is an issue for localisation problems as discussed in Section 2.4.5. The use of a local plastic displacement strain-softening model and a nonlocal strain-softening regulatory model, both considering the same strain-softening behaviour (i.e. same peak and fully softened strength and rate of strength reduction) were investigated to assess mesh dependency within FLAC for eight different mesh configurations. It was shown that the local strain-softening model was highly mesh dependent, the times to failure and failure surfaces obtained from the analyses are very different for different mesh configurations. The nonlocal strain-softening model was much less mesh dependent; however, the regulatory model did not completely remove mesh dependency within the analysis. The work conducted considered the Galavi and Schweiger (2010) weighting function and internal length in line with the recommendations made by Summersgill, et al., (2017).

Mesh dependency presents a significant uncertainty for the results of numerical analyses considering strain-softening behaviour (i.e. where modelling progressive failure is important). The implementation of the nonlocal strain-softening model has reduced this uncertainty. To further reduce uncertainty due to mesh dependency, for analyses that are directly compared, analyses should (and have been for later analyses) be conducted within the same mesh configuration to minimise differences due to the mesh.

8.2.2 Modelling Seasonal Ratcheting – Uncertainty

Numerical analyses conducted in this phase of modelling were carried out in FLAC two-phase flow, an unsaturated numerical framework that accounts for hydro-mechanical coupling and capillary effects by considering two idealised immiscible fluids within a porous media (Itasca Consulting Group, Inc., 2011). Hydro-mechanical coupling is achieved through the use of Bishop’s
generalised effective stress including hydrogeological descriptors, saturation and matrix suction that are linked through a van Genuchten (1980) style soil water retention curve.

It is assumed that the porous media is fully saturated by the two fluids. The fluids are homogeneous and do not mix. The framework allows for easy transition between fully saturated behaviour and unsaturated behaviour. When unsaturated, the soil particles are assumed to be incompressible and the water and air phases (i.e. the fluid phases) are assumed to be slightly compressible.

Although, in reality air dissolves within water and the two fluids are not immiscible, formulation of two-phase flow with immiscible fluids assumes that the two fluids completely fill the void space of the porous media (i.e. fully saturated by the two fluids) and the pressures calculated (i.e. pore water and pore air pressure) are derived through consideration of surface tension between the two fluids (Klubertanz, et al., 2003). Miscible formulations should be used for instances where mixing of fluids is fundamental to the behaviour being considered (i.e. for pollutant transport modelling). The assumption of immiscible fluid phases is acceptable for the consideration of unsaturated soils and reservoir simulation.

The compressibility of the fluid phases and the fact that the soil matrix is assumed incompressible indicates that the soil particles are much stiffer than the fluids that fill the void spaces. This is representative of an unsaturated soil as in reality the air phase is much more compressible than the soil skeleton.

The critical mechanism of interest within this work is seasonal ratcheting, the behaviour is driven by wetting and drying stress cycles in high-plasticity clays, it is therefore an unsaturated problem. The stress cycles due to wetting and drying have been shown to cause shear strain accumulation and progressive failure. Previous numerical analyses (i.e. Kovacevic, et al., 2001; O’Brien, et al., 2004; Nyambayo, et al., 2004) have shown that stress cycles can lead to progressive failure. However, the seasonal ratcheting movements were not shown or validated against real slope behaviour. To illustrate that the numerical modelling approach developed within the framework discussed can capture seasonal ratcheting, behaviour of two numerical analyses were
compared to hydrogeological and mechanical behaviour of physical modelling conducted by Take (2003). The comparison has shown that the modelling approach developed adequately captures key features of seasonal ratcheting (i.e. outward and downward movement and accumulation of strains with stress cycles) and the displacements predicted by the numerical analyses are of a similar magnitude and direction to the physical modelling.

The material being modelled, Kaolin clay, is assumed to be homogeneous and uniform throughout the cross-section of the slope model. As the monitored data is from experimental physical models, the material properties of the soil within this modelling will be more uniform than natural soil. However, some heterogeneity will still exist. The soil being modelled exhibits post-peak strength reduction, to account for this, strain-softening behaviour has been calibrated against triaxial test data and a sensitivity analysis performed to assess the mechanical model presented. As with the previous work, to reduce mesh dependency, the nonlocal strain-softening regulatory model was used for all models within this work. Stiffness and saturated hydraulic conductivity are a function of the specific volume and void ratio respectively. Therefore, these parameters depend on the stress history and current stress state within the numerical model. This is representative of reality but deterioration of both material properties due to repeated stress cycles has not been accounted for – within the validation models this will not have a significant effect as the models are only subjected to a small number of seasonal stress cycles. For the geometric study this extra deterioration mechanism of material properties would influence the time to failure of the numerical analyses.

From the physical modelling, the stress history of the soil is well-known, and it has been possible to recreate the initial stress conditions of the two slopes within the numerical analyses. The boundary conditions applied to the numerical analyses are wetting and drying discharge boundary conditions that have been calibrated to ensure the stress cycles occurring within the near-surface of the numerical models are representative of the stress cycle observed in the physical modelling. The boundary conditions do not aim to improve current capabilities to model soil-water-atmosphere interactions but
focus on driving realistic stress cycles in the near-surface of the slope due to wetting and drying.

Considering the geometric study, all the material relationships were the same as the validated models (with the exception of the different strain-softening models used) and stress cycles imposed were within the range of stress cycles that were validated against. This has been done to increase confidence that the results are within the range of validated behaviour for the material modelled. Within the geometric study, uniform winter wetting and summer drying boundary conditions were applied to drive the same pore water pressure cycles, and therefore stress cycles, within the slope analyses. This means that the time to failure for these analyses are relative for different geometry slopes experiencing the same stress cycles, the analyses do not provide the absolute time to failure for these slope geometries. As mentioned previously, with high numbers of stress cycles additional deterioration of hydrogeological and stiffness parameters would occur affecting the number of stress cycles to failure but these behaviours have not been included due to a lack of data and knowledge of these behaviours for this material.

8.2.3 Modelling Cut Slope Behaviour with Seasonal Boundary Conditions – Uncertainty

The numerical frameworks used within this phase of modelling have been discussed in detail in the two previous sections. The focus of this work is the development of a numerical modelling approach that can capture multiple inter-related strength deterioration mechanisms. The largest uncertainty is therefore in the mechanisms being modelled. The results in Chapter 4 have shown that simplified cut slope behaviour (i.e. undrained unloading, dissipation of excess pore water pressures, stress relief and progressive failure) can be successfully modelled in a saturated framework. Results were compared with other numerical modelling (Potts, et al., 1997) and shown to capture the same behaviour. Therefore, the results of this modelling approach can be assumed to capture these mechanisms. Chapter 5 presented a modelling approach within a two-phase flow framework that allowed seasonal ratcheting due to wetting and drying stress cycles (i.e. weather-driven behaviour) to be modelled and behaviour has been validated against physical modelling (Take, 2003).
These mechanisms need to be considered in parallel for long-term modelling and assessment of high-plasticity overconsolidated clay cut slopes. When simplified cut slope behaviour was modelled in the two-phase flow framework, the time to failure of the model was considerably different to the equivalent saturated analysis. A coupling of the single and two-phase flow frameworks was developed, and simplified cut slope behaviour was modelled in the two-phase flow framework with the time to failure being very similar to the saturated analysis. This validated the fact that undrained unloading and excess pore water pressure dissipation, which are saturated processes, along with stress relief and progressive failure can be modelled in a numerical framework (i.e. the two-phase flow framework) that can also model weather-driven seasonal ratcheting. By including wetting and drying boundary conditions to the coupled analysis, all the physical mechanisms mentioned can be captured within a single boundary-value problem.

The results presented in Chapter 6 show that when seasonal boundary conditions are applied to the coupled model, post-excavation pore water pressure dissipation is observed within the model. However, the rate of pore water pressure dissipation under transient boundary conditions is at a much slower rate than when the model is subjected to constant static boundary conditions. The times to failure observed in the numerical analyses with seasonal wetting and drying boundary conditions are indicative of the observations made by Chandler and Skempton (1974) for delayed failures in cut slopes (i.e. between 40 to 65 years post-excavation for similar slope geometries in London Clay, not the 14.5 to 16.2 years predicted in the simplified cut slope models and by Potts, et al., (1997)). In addition, pore water pressure dissipation occurs preferentially at a shallower depth with suppressed pore water pressures remaining at depth within the slope. Finally, the failure surface observed within the analysis is much shallower than when static boundary conditions are applied. The failure surface predicted by the numerical analysis is indicative of shallow first-time failures observed in ageing infrastructure slopes (Briggs, et al., 2017).

Whilst there is no monitored data to validate this behaviour it is known that many cut slopes in high-plasticity overconsolidated clay are of an age where
pore water pressure dissipation alone cannot explain failure – especially, shallow failures. It was shown by Chandler (1974) that steady pore water pressure conditions were reached within around 60 years in un-brecciated clay. Therefore, shallow first-time failure within the slopes must occur due to additional mechanisms and can be explained by repeated stress cycles causing seasonal ratcheting and progressive failure. The results of the numerical analyses presented in Chapter 6 show this behaviour.

Given the incremental validation of the different modelling approaches (i.e. in Chapter 4 and Chapter 5) it can be assumed with some confidence that deep-seated mechanisms (i.e. pore water pressure dissipation) and near surface processes (i.e. seasonal ratcheting) are being captured within the numerical model presented. Although the mechanisms are captured, there are limitations in the material model and boundary conditions applied that should be considered.

The slope analyses in this phase consider London Clay, the mechanical behaviour is the same as used for the modelling of simplified cut slope behaviour. The soil is assumed to be homogeneous, the strain-softening behaviour has been calibrated against the model presented by Potts, et al., (1997) and an effective stress dependent stiffness has been adopted. The initial stress conditions within the analysis, prior to excavation, are assumed to be uniform for a single coefficient of earth pressure at rest increasing horizontal stresses relative to vertical stress to allow stress relief to be modelled. A depth dependent saturated hydraulic conductivity profile has been used considering measured values and the effect of increased near surface saturated hydraulic conductivity has been investigated. In both cases, the saturated hydraulic conductivity profiles used are unchanging, they are applied to the model and fixed throughout the analysis. It is known that changes in hydrogeological properties occur with time and stress cycles but currently there is insufficient data to allow this to be included in numerical analyses of London Clay. Also, a single constant wetting soil water retention curve for London Clay has been used in all analyses. Not including hysteresis and deterioration of soil water retention characteristic properties is a significant limitation of the modelling,
but again, there is currently insufficient information to allow this behaviour to be accounted for in a meaningful way.

As with the results of the geometric study presented in Chapter 5, the boundary conditions applied within this phase of modelling are simple winter wetting and summer drying discharge boundary conditions, designed to drive constant annual stress cycles representative of field observations in London Clay. Therefore, the time to failure for the different analyses is not exact and only relative behaviour should be considered.

8.2.4 Modelling Long-Term Behaviour of a Real Cut Slope – Uncertainty

The numerical frameworks and mechanisms being modelled have been discussed in the previous Section 8.2.1 to 8.2.3. Therefore, this section focused on the material models and the boundary conditions used for the modelling within this phase. Initially, the boundary conditions have been discussed and following this, the different strain-softening behaviour, stiffness models and how root reinforcement was included within the different numerical analyses have been addressed. To finish, the uncertainties associated with modelling climate change and the climate change impact assessment undertaken in Section 7.7 is discussed.

8.2.4.1 Boundary Conditions

The boundary conditions imposed within this phase of modelling have been obtained from a soil water balance calculation that allows the volume of water removed or added to the soil to be obtained daily (Blight, 2003) and are applied as a constant discharge throughout the day. This is a significant simplification as storm events can lead to very high rainfall over a very short period leading to significant runoff and much less infiltration than captured within the method used here. Smith (2003) suggested that weather-driven boundary conditions should be applied in hourly increments. The simplification to daily boundary conditions can be justified given the mechanisms being modelled in this work, deterioration and mobilisation of post-peak strength after decades of stress cycles is of interest with the time increment of the boundary conditions significantly smaller than the time to failure of the slope models. However, if
the modelling were considering the triggering of landslides then boundary conditions should be applied in smaller time increments as suggested by Smith (2003).

Regarding the boundary conditions, there are uncertainties due to the water balance calculation and the weather data used to facilitate the water balance calculation. Again, due to the time frame of the deterioration mechanisms being modelled, long-term data series are required and often there is insufficient site-specific weather data available to create boundary conditions for the full life-cycle of a slope being modelled.

The water balance calculation is based on conservation of mass and considers a uniform depth of material (i.e. rooting depth) and assumes that the change in water is uniform over the depth of soil considered (Blight, 2003). The total and readily available water within the constant depth of material considered within the calculation are a function of the soil water retention properties of the soil. This is problematic as it is known that hydrogeological deterioration occurs due to repeated stress cycles, this means if hydrogeological deterioration is included in a numerical analysis, the total and readily available water used in the water balance calculation should also change with repeated stress cycles.

The water balance calculation uses rainfall and potential evapotranspiration to calculate the soil moisture deficit. Potential evapotranspiration is a very difficult parameter to obtain and there are many assumptions within the different approaches available. Within this work, the Penman-Monteith (for site-specific weather data) and the Blaney-Criddle (1950) (for regional weather data to fill gaps in site-specific data series) methods have been used. To show that representative boundary conditions can be obtained using the two different methods of calculating potential evapotranspiration, regional and site-specific weather data over a base period have been compared and are shown to produce comparable soil moisture deficit trends. This has allowed missing weather data to be filled within the life-cycle of the slope being modelled.

Considering the limitations, assumptions and simplifications, the boundary conditions used have been shown to model representative hydrogeological
behaviour and therefore stress cycles for a real slope and as such progress the ability to model soil-water-atmosphere interactions.

**8.2.4.2 Strain-Softening Model**

For the initial models of Newbury cutting, the strain-softening model calibrated against the model presented by Potts, *et al.*, (1997) was employed. The results of the analyses showed that the cut slope was stable up to the end of 2017 but significant softening had occurred with the majority of the near-surface material mobilising minimum strength, in this case fully softened strength. From previous modelling, presented in Chapter 5, strain-softening behaviour has been shown to be significant for long-term behaviour, with the rate of strength reduction towards residual strength being critical. The strain-softening model calibrated against the model presented by Potts, *et al.*, (1997) does not include softening to residual strength as it stops at fully softened. This assumption is valid for deep-seated first-time failures due to post-excavation pore water pressure dissipation as shown by Chandler and Skempton (1974) and Skempton (1977), where failure of cut slopes occurred at an average mobilised strength of fully softened strength not an average mobilised strength of or close to residual. For shallow first-time failure due to repeated seasonal stress cycles, the behaviour of softening towards residual is extremely important regarding the time to failure and the geometry of slopes that are at risk of failure due to the mechanisms. Therefore, a new strain-softening model including post-peak softening to critical state followed by strength reduction to residual at very large plastic strains has been used. The model has not been validated / calibrated against experimental data, but it is known that it can take many tens even hundreds of millimetres of displacement for a complete residual shear surface to develop within overconsolidated clay (Skempton & Petley, 1967). The strain criteria used within the model reflect this fact.

**8.2.4.3 Stiffness Models**

As with previous models for London Clay, an effective stress dependent stiffness model has been used. The same model was initially used for the Newbury cutting model and then a sensitivity analysis of different stiffness relationships was conducted. There are limitations in the fact that stiffness deterioration due to repeated stress cycles has not been included – again there
is a lack of data available to create a realistic model that can be included within these numerical models. There are also questions regarding the most appropriate stiffness relationships for modelling London Clay. Should small strain stiffness be included? Is effective stress dependent stiffness appropriate? Should stiffness be related to specific volume as done for Kaolin previously?

Regardless, it has been shown that seasonal ratcheting behaviour can be captured using the effective stress dependent stiffness relationship. But more is needed to be known regarding post-excavation stiffness, long-term stiffness and stiffness deterioration due to repeated stress cycles for overconsolidated clays.

### 8.2.4.4 Root Reinforcement
The modelling considering root reinforcement is speculative. There has been considerable debate in the literature regarding the inclusion of root reinforcement through increased cohesion with the general consensus being that it is inappropriate. Root reinforcement has significant temporal and spatial variability and considering root reinforcement as a constant cohesion increase does not provide the real stress-strain behaviour of the roots (Cohen, et al., 2009).

However, the increased cohesion in the near-surface used to represent root-reinforcement has been shown to be extremely influential on the time to failure of slopes. Not including root reinforcement omits critical physical behaviour in the numerical analyses of shallow first-time failures due to repeated wetting and drying. The failure mechanism modelled generates very small annual displacements and therefore there will be limited breakage and pull out of roots due to shearing in an annual cycle. In addition, the time frame of the failure mechanism, being decades, means there will be some level of recovery in the strength provided by roots following large displacements. It can therefore be postulated that the inclusion of root reinforcement as a constant cohesion along the slope surface is not completely unrealistic for consideration of deterioration.

In addition to strength increase due to roots, the stiffness of the near-surface of the slope will also be affected by roots. This will have a significant implication
on the mechanical behaviour of the near-surface, the rate of accumulation of strains and therefore the rate of progressive failure. Finally, the root reinforcement modelled in this work, for grass, is wished in place post-excavation. In reality, vegetation will take a number of seasonal cycles to become established and it is likely that there will be changes in the vegetation along an infrastructure slope during its life (i.e. historically there could have been trees on a slope that have been removed). It is clear that much more knowledge is needed regarding the inclusion of root reinforcement in continuum numerical analyses.

8.2.4.5 Modelling Climate Change Scenarios
Climate change assessments are extremely complex given the variability in weather data and methods available to allow inclusion of climate change projections. Significant uncertainty exists within global climate change models so to move away from direct use of these, statistical down-scaling decision centric methodology has been adopted in this study (Wilby, et al., 2014). A base period (i.e. 1961 to 1990) has been used to create regional weather data trends. To account for climate change scenarios, winter and summer month precipitation and temperature have been factored to create synthetic weather sequences. Synthetic weather sequences, representative of potential future climatic conditions, have been used to allow stress testing of a slope system past the end of 2017.

One of the main difficulties with modelling climate change for long-term deterioration behaviour is the fact that inter-seasonal variation is of key importance so there are more variables than if the physical system only relied on one factor (e.g. such as volume of rain for flood modelling). This means that multiple analyses should be considered to investigate this variability. However, the numerical analyses are computationally expensive and as such only a handful (i.e. five) analyses have been conducted. Therefore, the modelling conducted here is still considered deterministic, but the scenarios modelled have been selected to consider the range of climate change scenarios that could be feasible in accordance with inter-seasonal trends projected by UKCP09. However, considered a range of different scenarios is an advancement on previous work within this area that considered a single worst-
case climate scenario to assess climate change impacts (Rouainia, et al., 2009; Elia, et al., 2017).

Within the synthesis of weather data for future climate scenarios, using statistical down-scaling, the numbers of wet days per month have not been altered (i.e. they are the same number as in the base period) and change in frequency of extreme events such as storms and droughts have not been included. This work focuses on long-term deterioration due to annual cycles of wetting and drying causing failure after decades of stress cycles. Therefore, extremes have been omitted to allow relative deterioration behaviour for different climate scenarios to be considered, with accelerated failure due to extremes not included. Other climate change behaviour such as increased freeze thaw causing damage and deterioration to the near-surface of a slope have not been considered. If these sorts of behaviour are to be considered, more information is needed about how they will change in the future to allow inclusion within soil water balance calculations and climate change assessments.

One of the main limitations in the synthesis of future weather data is the fact that factors used in the water balance calculation are not updated to represent the effects of climate change. As discussed previously, hydrogeological deterioration, due to repeated stress cycles, will result in changes to soil water retention properties which will change the total and readily available water contents in the water balance calculation. Significant changes in temperature will likely cause changes in vegetation type, root depth and root density. This will affect the crop factor used within the water balance calculation as well as the level of strength and stiffness increase due to root reinforcement. In addition, larger stress cycles will likely lead to increased desiccation cracking and more rapid stiffness and hydrogeological property deterioration as well as stressing vegetation more frequently.

There are many uncertainties with modelling climate change effects but not attempting to conduct meaningful analysis with an appreciation of the limitations will leave infrastructure routes vulnerable to failures due to a lack of understanding.
8.2.4.6 Climate Change Impact Assessment of Newbury Cutting

In line with Objective 4, a climate change impact assessment has been undertaken, and results presented in Section 7.7.4, to further understand climate change effects on clay cut slopes. The numerical analyses conducted have indicated that wetter winters and drier summers will reduce the serviceable life of high-plasticity overconsolidated clay cut slopes. Larger stress cycles due to projected climate change conditions have been shown to increase the rate of deterioration of clay cut slopes.

It should be noted that there are uncertainties and limitations in the modelling approach and material properties used within the climate change impact analyses. As discussed previously, there are assumptions within the numerical framework and a lack of current information to capture all deterioration mechanisms known to be problematic for overconsolidated clay cut slopes. In particular, current limitations include: the lack of hydrogeological and stiffness deterioration; no soil water retention hysteresis; and the simple inclusion of root reinforcement. Alongside these limitations, there is also the fact that critical physical behaviours that are known to affect the strength, stiffness and hydrogeological properties of the near surface have not been considered (e.g. desiccation cracking). It is appreciated that desiccation cracking will likely get worse with drier summers changing the fabric of the near-surface soil. There is currently very limited information regarding the effects of desiccation cracking on behaviour and as such this has not been included within the numerical analyses. The time to failure of Newbury cutting analyses for different climate scenarios are therefore not accurate but represent relative behaviour.

As discussed in Section 8.2.4.5, the boundary conditions are calculated assuming constant rooting depth, total and readily available water contents and crop factor that with climate change and repeated stress cycles would likely differ. The boundary conditions for future climates potentially do not capture the reality of future weather patterns. However, the boundary conditions allow the numerical analyses of a slope to be subjected to different stress cycles to stress test the general trends in climate change projections.
8.3 Results and Implication

The uncertainties and limitations of the different phases of modelling have been considered in detail in Section 8.2. With the uncertainty and limitations in mind, the significant results and their implications for current state of knowledge have been summarised and discussed.

8.3.1 Local vs Nonlocal Strain-Softening Model

Within Chapter 4 it was shown that the use of a nonlocal strain-softening regulatory model yields far less mesh dependent results than a local strain-softening model for both the time to failure and shear surfaces obtained at failure. Whilst this has been shown previously by Summersgill, et al., (2017), the results presented in Figure 8.1 demonstrate the successful implementation of the nonlocal strain-softening regulatory model within FLAC that has been used extensively throughout this project.
Mesh dependency is a significant problem for numerical analyses considering progressive failure so a method for reducing mesh dependency allowing analyses to be more repeatable / easily replicated by others is a promising
step forward. As mentioned previously, the nonlocal strain-softening regulatory model is not mesh independent, but the results presented further validate the conclusions drawn by Galaví and Schweiger (2010) and Summersgill, et al., (2017).

8.3.2 Validation of Numerical Modelling of Seasonal Ratcheting against Physical Modelling

One of the main advances within this work has been the validation of a coupled hydro-mechanical numerical model, capable of modelling unsaturated behaviour, against physical modelling data of seasonal ratcheting (Take, 2003). It has been shown that characteristic seasonal ratcheting displacements can be replicated within a numerical analysis subjected to known stress cycles (i.e. pore water pressure cycles) and progressive failure observed. This mechanism is critical to shallow first-time failures in clay slopes after repeated stress cycles and this is the first time that a numerical model has been validated against real data for this mechanism.
Figure 8.2 Pore water pressure and displacement comparisons for validation of numerical model against physical modelling for seasonal ratcheting (physical modelling data after, Take 2003); a) WAT7a – geometry; b) WAT8a – geometry; c) WAT7a – HCT1; d) WAT8a – HCT D3; e) WAT7a – mid-slope displacements; f) WAT8a – toe displacements

The results in Figure 8.2 show that general trends in movements and magnitude of displacements of seasonal ratcheting observed in the physical
modelling have been captured within the numerical analyses. It can therefore be concluded that the numerical framework and material relationships used within the numerical analyses are acceptable for the modelling of this mechanism. The validation of this behaviour being captured numerically means that a modelling approach for considering weather-driven behaviour and strength deterioration of clay slopes (i.e. Objective 2) has been achieved.

8.3.3 Geometric Study of Seasonal Ratcheting

Having validated a numerical model approach against physical modelling data a geometric study considering Kaolin Clay slopes subject to the same uniform seasonal stress cycles to failure was conducted. A method for assessing the point of serviceability failure was developed and the mobilised friction angle at serviceability failure compared for different slope heights and angles and strain-softening behaviour. The results of this are shown in Figure 8.3.
The headline observation from the geometric study is the fact that slopes at an angle equal to or less than the minimum friction angle of the soil do not fail due to seasonal ratcheting. However, designing slopes to this angle will likely result in slopes that are uneconomic and of a design-life far in excess of design requirements. The rate of strength reduction post-critical state has been shown to be fundamental to the number of seasonal cycles a slope can experience prior to serviceability failure. Simplistically, behaviour can be considered in two groups and the design parameters for use in limit equilibrium analysis for a reasonable serviceable design life (i.e. around 60 to 100 seasonal cycles) can be categorised as follows;

Figure 8.3 Mobilised internal angle of friction at serviceability failure for different strain-softening relationships, slope angles and heights; a) mobilised internal angle of friction at serviceability failure against slope angle; b) mobilised internal angle of friction at serviceability failure against number of seasonal cycles to failure
• for clays that soften to residual strength rapidly, residual strength should be considered in limit equilibrium analysis to achieve an acceptable design life;

• for clays that require significantly large displacements to reach residual strength, strength parameters less than critical state but greater than residual should be considered in limit equilibrium analysis to achieve a reasonable design life.

These two categories of softening behaviour and corresponding strength parameters for use in limit equilibrium analyses are shown within Figure 8.4.

These conclusions are for Kaolin clay subjected to simple wetting and drying stress cycles. Variability of weather, prolonged wetting and its influence on behaviour have not been considered. For more detailed recommendations of design parameters, modelling considering the soil of interest should be conducted.

The only previous attempt to suggest design parameters for the use in limit equilibrium analyses of shallow first-time failures due to seasonal ratcheting was made by Take and Bolton (2011). They hypothesised that the use of critical state friction angle in limit equilibrium analyses would yield design lives acceptable for infrastructure slopes. Whilst the conclusions drawn in this work suggest strength parameters less than critical state, ultimately it is the strain-softening relationship that dictates what the design parameter should be along with the required design life, the slope height and the implications if failure.
were to occur (i.e. slopes adjacent to high-speed rail will require a lower risk of failure than next to a minor road).

**8.3.4 Modelling Inter-Related Strength Deterioration Mechanisms for Clay Cut Slopes**

In line with Objective 3, a method for analysis of multiple inter-related strength deterioration mechanisms for high-plasticity overconsolidated clay cut slopes has been developed and results shown in Chapter 6. An approach capable of modelling undrained unloading, excess pore water pressure dissipation, stress relief, seasonal ratcheting and progressive failure has been presented. The results of a comparison of simplified cut slope behaviour and the model including seasonal ratcheting (for simple summer winter boundary conditions) are shown in Figure 8.5.
Figure 8.5 Comparison of simplified cut slope behaviour (SP-A1) and cut slope behaviour with seasonal boundary conditions (TP-B1); a) stress changes 1m below toe of slope; b) pore water pressures below toe of slope; c) mid-slope horizontal displacements; d) shear surfaces

The main difference in behaviour between the analyses is the time taken for the dissipation of excess pore water pressures at depth for the slope subjected
to transient wetting and drying boundary conditions compared with the static boundary conditions. Repeated stress cycles have led to a much shallower failure surface developing within the analysis including seasonal wetting and drying boundary conditions. The failure predicted within the numerical analyses is indicative of slope failures observed within ageing clay infrastructure slopes (Briggs, et al., 2017).

Except for the boundary conditions imposed being simplistic and lack of detailed material (i.e. hydrogeological and stiffness property) deterioration models, the results show that critical physical processes for clay cut slope behaviour can be adequately captured within a single boundary-value numerical analysis. This advances previous work considering near-surface stress cycles for clay cut slopes (Kovacevic, et al., 2001; Nyambayo, et al., 2004) by combining saturated and unsaturated processes in a single model and considering the wetting and drying stress cycles as unsaturated – which is representative of real behaviour shown by Smethurst, et al., (2012) for a London Clay cut slope.

### 8.3.5 Hydrogeological Deterioration

One of the most significant outcomes of the work presented in Chapter 6 is the effect that near-surface hydrogeological deterioration has on the time to failure of a slope. Two saturated hydraulic conductivity profiles were considered; one considering saturated hydraulic conductivity as it would be post-excavation and the other introducing a higher saturated hydraulic conductivity layer across the near-surface to a depth of 3.0m (i.e. representing deterioration of the saturated hydraulic conductivity). The numerical analyses were the same apart from this and subjected to the same magnitude of seasonal wetting and drying stress cycles, the results are presented in Figure 8.6.
The times to failure for the analyses are considerably different and show that hydrogeological property deterioration has a significant effect on slope performance. The field data used to create the near-surface high saturated hydraulic conductivity profile were measured across the surface of a London Clay cut slope approximately 20 years after excavation showing how quickly near-surface hydrogeological deterioration can occur. This rapid change in hydrogeological properties shows that deterioration is occurring at a very high rate and differs from the observations made by Chandler (1972), where weathering was shown to be mainly associated with the original ground profile and not the newly exposed cut slope profile 60 years after construction. It should be noted that Chandler (1972) was looking at cut slopes within the Lias formation not London Clay as considered here.

**8.3.6 Effect of Initial Stress Conditions**

The effect of initial stress conditions through the consideration of different coefficients of earth pressure at rest initiating different horizontal stress conditions within numerical analyses prior to excavation have been considered for simplified cut slope behaviour and cut slope behaviour including seasonal boundary conditions in the coupled approach. The results of these analyses are shown in Figure 8.7.
Figure 8.7 Effect of initial stress conditions on cut slope behaviour; a) simplified cut slope behaviour mid-slope horizontal displacement; b) cut slope behaviour with seasonal boundary conditions mid-slope horizontal displacement; c) simplified cut slope behaviour shear surfaces; d) cut slope behaviour with seasonal boundary conditions shear surfaces.

Figure 8.7 shows that initial stress conditions influence the times to failure for failures driven by seasonal stress cycles far less than models with constant
boundary conditions (i.e. simplified cut slope behaviour). This is shown in the difference in the spread of times to failure in Figure 8.7a and 8.7b. The difference can be attributed to the fact that the stress cycles due to seasonal boundary conditions are to a shallow depth, as shown by the failure surfaces in Figure 8.7d, and the effect of different initial stress conditions is minimal within the near-surface of the slope. Within the models considering simplified cut slope behaviour, deep-seated failures due to stress changes and pore water pressure equilibration post-excitation are observed. The effect of different initial stress conditions are much more significant at depth increasing the variability in time to failure considerably. The failure mechanisms are fundamentally different and have different levels of dependency on the initial stress conditions within the numerical analyses.

8.3.7 Modelling Real Slope Behaviour
A method for obtaining and applying real weather boundary conditions to a numerical model capable of modelling inter-related strength deterioration mechanisms has been presented and hydrogeological behaviour validated against long-term field monitoring. The validation shows that the boundary conditions developed advance current modelling capabilities of soil-water-atmosphere interactions. Results of hydrogeological behaviour for a number of locations within the slope cross-section and the corresponding mechanical behaviour due to the stress cycles imposed are shown in Figure 8.8.
Figure 8.8 Comparison of monitored and modelled pore water pressures for Newbury cutting and mechanical response of numerical analysis (monitored data after, Smethurst, et al., 2012); a) piezometers A 1.0m depth; b) piezometers B 1.0m depth; c) piezometers C 1.5m depth; d) mid-slope displacement - 1997 to 2017

Figure 8.8 shows that realistic pore water pressure cycles in the near-surface of the slope can be replicated using the numerical modelling approach developed. The magnitude of the pore water pressure cycles is similar in the 1.0m depth records, although at greater depths the numerical analyses do not tend to replicate pore water pressure records to the same accuracy. The
timings of maximum and minimum pore water pressures within a seasonal cycle are delayed within the numerical analyses. There are significant uncertainties in the weather data used to create boundary conditions, the material properties used within the numerical analyses and limitations in assumptions made. The main limitations are the fact that soil water retention hysteresis and hydrogeological property and stiffness deterioration are not included due to lack of data available to create representative models for these material properties with repeated stress cycles. However, it has been shown that realistic pore water pressure, and therefore stress cycles, can be created within this numerical modelling approach regardless.

To allow the modelling of Newbury cutting, and for the numerical analysis to replicate pore water pressures at different locations, the level of weathering of the London Clay was considered. It was known that the upper 3m of the original ground surface (i.e. prior to excavation) was highly weathered and that there would have been some weathering of the slope surface from excavation to the time of the monitoring. To account for different levels of weathering, different soil water retention properties were used across the slope (see Figure 7.6). The use of fully weathered soil water retention properties was fundamental in capturing the smaller pore water pressure cycles observed in group A piezometers, relative to the other piezometers in less weathered material. This shows the significance of better understanding hydrogeological material deterioration with repeated stress cycles.

8.3.8 Climate Change Impact Assessment
A climate change impact assessment for Newbury cutting has been undertaken, as discussed in Section 8.2.4.6, there are limitations with the material models and boundary conditions applied. However, it is still necessary to assess the implications of climate change on these assets. The limitations are the same for all models, so behaviour observed is relative and is a function of the stress cycles imposed due to different climate scenarios considered only. Numerical analyses have predicted that wetter winters and drier summers leading to larger stress cycles increase the rate of strength deterioration for high-plasticity overconsolidated clay slopes. The mid-slope horizontal
movement of the different climate scenarios considered are shown in Figure 8.9.

Figure 8.9 shows the variation in the rate of deterioration of Newbury cutting following the application of synthetic weather sequences. It can be seen clearly that the numerical analyses predict that the climate change scenarios considered will fail quicker than the model considering current climatic conditions. Although only 4 climate change scenarios have been considered, this has provided a range of potential weather sequences and their effects on a cut slope to be considered. Considering one climate scenario does not provide an appreciation of the variability of potential future scenarios. For example, Rouainia, *et al.*, (2009) showed that infrastructure embankments will likely be more stable under future climate conditions than present for a single analysis.

As discussed within Section 2.3.6.2, there are differences in opinion on the implications of climate change on overconsolidated clay slopes. Some studies have indicated that drier summers will increase the overall stability of slopes (Clarke & Smethurst, 2010). Whereas others believe that larger stress cycles, increasing the magnitude of seasonal ratcheting displacements and increased damage to the near surface from desiccation cracking in the summer could result in an increase in the rate of deterioration of these assets (Dijkstra & Dixon, 2010).

The analyses conducted do not allow conclusions to be drawn regarding the effect of increased deterioration from desiccation cracking or how stiffness and
hydrogeological properties will deteriorate with number of seasonal stress cycles. But these processes are likely to increase the rate of deterioration of a clay slope. The results do show that for an existing slope that has experienced some level of strength reduction, increased stress cycles lead to a quicker rate of deterioration. This means that if trends in climate change projections are representative of the future climate, existing high-plasticity overconsolidated clay cut slopes will experience greater annual deterioration and be at risk of failing much quicker than under current weather patterns. These conclusions cannot be extended to new cut slopes as increased summer suctions could potentially increase the time taken for post-excitation excess pore water pressure dissipation to increase. However, following dissipation of excess pore water pressures, deterioration due to projected climate change will increase relative to deterioration due to current weather patterns.

### 8.4 Limitations in Current Knowledge

There are limitations within the numerical framework used for the analyses presented within this work. However, it has been shown that physical mechanisms can be captured within the numerical analyses and therefore the frameworks used are appropriate. The most significant limitations within the work are the material properties and relationships used, which do not represent real behaviour and deterioration mechanisms that will undoubtedly affect the time to failure of these numerical analyses. These critical deterioration mechanisms have been investigated and shown to be significant but currently there is insufficient knowledge/data available to create meaningful relationships for these parameters that can be included within numerical analyses.

The different properties that require better understanding to advance numerical modelling capabilities within this area have been summarised in the following sections.

#### 8.4.1 Strain-Softening Behaviour

It has been shown in Chapter 5 and Chapter 7, that strain-softening behaviour is fundamental in establishing the number of seasonal stress cycles that lead to failure within numerical analyses considering seasonal ratcheting. Currently
for London Clay, the majority of strain-softening models consider strength reduction to fully softened and not residual strength (Potts, et al., 1997; Ellis & O'Brien, 2007; Summersgill, et al., 2017). The modelling conducted within these studies considered deep-seated first-time failure due to construction effects (i.e. dissipation of post-excavation pore water pressures).

Shallow first-time failure due to repeated stress cycles effects slopes that have not failed due to construction but are formed at an angle greater than the minimum friction angle of the soil. Failure is dependent on material softening to a lower strength than fully softened. Therefore, it is necessary to better understand strain-softening behaviour to residual strength at very large strains. The progression of knowledge in strain-softening behaviour required for better modelling capabilities is shown schematically in Figure 8.10.

![Simple Strain-Softening Behaviour](image.png)

Figure 8.10 Improved understanding of strain-softening behaviour required

### 8.4.2 Soil Water Retention Hysteresis

The simplification of a single soil water retention curve to model wetting and drying is a significant limitation of this work. Within Chapter 5 it was shown that drying behaviour could be replicated very well using discharge boundary conditions, but wetting behaviour could not be replicated as effectively. Soil water retention hysteresis has not been included within this work due to a lack in data available to create wetting, drying and scanning curves for the materials modelled. The progression of knowledge of soil water retention behaviour to be included in modelling to improve pore water pressure generation and therefore stress cycles within the near-surface of the slope models is shown schematically in Figure 8.11.
Improved understanding of soil water retention properties required

8.4.3 Hydrogeological Deterioration

Within Chapter 6 it was shown that saturated hydraulic conductivity deterioration has a major influence on the rate of strength deterioration of a slope subjected to repeated stress cycles. In addition, it was shown that soil water retention property deterioration is important to capture the correct pore water pressure and therefore stress cycles in the near-surface of a slope (see Chapter 7). Much more is needed to be known about how these properties change with time, and the number and magnitude of stress cycles to allow this fundamental deterioration behaviour to be included within numerical analyses. Hydrogeological deterioration has been shown schematically in Figure 8.12.

8.4.4 Desiccation Cracking

Desiccation cracking is one physical process that has received very little attention within this work but has the potential to cause significant changes in behaviour depending on the magnitude of seasonal wetting and drying cycles. Desiccation cracking in high-plasticity clays occurs when the suction stresses within the soil exceed the tensile strength of the soil. When cracks form, there is some recoverable and some irrecoverable damage to the soil and it effects the hydrogeological properties (i.e. deterioration and introduction of
preferential flow routes), strength and stiffness of the soil. Currently, very little is known regarding the extent of the damage that desiccation cracking causes or how to include this extra deterioration process within a continuum analysis. This lack of understanding has meant this behaviour has not been directly considered in this work, but it is an area that needs to be addressed for future work as it is likely to be critical for climate change assessments.

### 8.4.5 Stiffness and Stiffness Deterioration

Within this work two stiffness models have been considered, for Kaolin, stiffness has been calculated as a function of specific volume and takes into account stress history and the current stress state of the soil. For London Clay an effective stress dependent stiffness model has been adopted. Within Chapter 5 it was shown that similar magnitude of displacements were obtained when using the specific volume dependent stiffness model compared to physical modelling results. This was for a well-established material and known stress history. For a real slope in overconsolidated clay the stress history will be much more complex, and it could be necessary to include small strain stiffness within numerical analysis for post-excavation displacements. Therefore, more knowledge is required about the most appropriate stiffness model for real soils. In addition, it is necessary to establish how stiffness changes with time, number and magnitude of seasonal stress cycles. A potential approach for modelling cyclic stiffness deterioration through changing specific volume relationship with repeated stress cycles is shown schematically in Figure 8.13

![Stiffness Deterioration](image)

Figure 8.13 Improved understanding of stiffness deterioration required
8.4.6 Root Reinforcement

The inclusion of root reinforcement within continuum models is a complex problem. Often root reinforcement is included through an additional cohesion value although this is known to be a poor representation of the physical behaviour. In Chapter 7 the effect of different cohesion values for modelling root reinforcement were considered and it was shown that the additional strength from root reinforcement changes the time to failure of these assets considerably. In addition to strength increase due to roots, the stiffness properties of the rooted zone will also change and should be accounted for. It has been postulated that annual displacements due to wetting and drying stress cycles may not be significant enough to cause breakage or pull-out of roots and the processes are so slow that roots will have time to become re-established during an annual cycle. As such there is limited long-term deterioration of root strength during stress cycles and root strength reduction is critical for ultimate failure only. This would suggest that a uniform cohesion increase to model root reinforcement could be an appropriate assumption for numerical analyses of long-term weather-driven deterioration. Regardless, more is needed to be known about root reinforcement in overconsolidated clays for the inclusion in continuum numerical analyses.

![Figure 8.14 Improved understanding of root reinforcement required](image)

8.4.7 Effect of Climate Change on Slope Behaviour

Climate change is a complex problem to model and the difficulty increases when the physical processes being modelled are inter-related with the effects of climate change. Within this work, boundary conditions for climate change have been synthesised considering change from base period weather data to stress test physical behaviour. There are limitations with how boundary conditions have been calculated as the work does not take into account that
vegetation type, root depth and root density will likely change due to climate change. As such, the assumptions made in the water balance calculation could produce unrepresentative boundary conditions for real behaviour in the future.

In addition, if vegetation changes due to temperature change and larger magnitude of wetting and drying cycles (see Section 2.3.3), the strength and stiffness of the near-surface of a slope would also change. The magnitude of strength increase from root reinforcement has been shown to change the number of seasonal stress cycles that a slope experiences prior to failure considerably.

If meaningful climate change impact assessments are to be conducted the potential change in vegetation and the implications that this has on slope behaviour should be considered. Also, the potential change in vegetation and the influence this has on the boundary conditions obtained and the corresponding stress cycles that the slope experiences should be considered. It is also important that other deterioration mechanisms, particularly hydrogeological property deterioration, are included in the numerical analyses and the factors used in the water balance calculation for boundary conditions reflect the deterioration occurring within the numerical model.

The numerical analyses conducted considering climate change focus on inter-seasonal trends in climate behaviour and other potential mechanisms such as freeze thaw have not been considered but have a potential to occur in the future. More guidance is needed on the appropriate climate change scenarios to use and how to include these behaviours into a soil water balance calculation to create model boundary conditions.

### 8.5 Key Implications

Seasonal ratcheting, due to seasonal stress cycles, accumulation of deformations and progressive failure can explain shallow first-time failures in high-plasticity clay slopes. Seasonal ratcheting provides an explanation as to why a failure occurs during one wet event even though the slope has experienced the same or more extreme conditions previously. From the numerical analyses conducted within this work, it has been shown that the near-surface zone of high-plasticity clay slopes are deteriorating due to
repeated seasonal wetting and drying. Deterioration includes strength as well as hydrological properties. All high-plasticity clay slopes across the UK’s infrastructure network experience seasonal stress cycles. As such, the condition of high-plasticity clay earthwork assets on the transportation network are deteriorating with every annual cycle of wetting and drying.

Slopes constructed in high-plasticity clay at an angle greater than the critical state friction angle of the soil are at high risk of shallow first-time failure due to seasonal ratcheting. Slopes constructed at angles less than the critical state friction angle but greater than the residual friction angle of the soil are also at risk of shallow first-time failure due to seasonal ratcheting, but the serviceable life of such slopes may be sufficient for their desired application. For a slope to remain perpetually stable and at no risk of failure due to seasonal ratcheting, the slope should be constructed at an angle less than the residual friction angle of the soil. In the context of the UK’s infrastructure network, there is a large proportion of existing slopes in high-plasticity clay that are at risk of first-time failure due to seasonal ratcheting.

Through investigation of the effects of climate change on high-plasticity clay cut slope behaviour, it has been shown that climate change increases the rate of strength deterioration and reduces the serviceable life of slopes. This does not take into account the effect of increased deterioration of hydrogeological properties of the near surface that will also increase deterioration rates. Regardless, the results presented show that in the future the rate of deterioration of these earthwork assets will increase.

It has been shown that trigger events, causing failure within the numerical analyses were not the worst conditions experienced in the life-time of the slope and that strength deterioration due to seasonal stress cycles reduces the magnitude of triggering event required to cause failure. With climate change, increased stress cycles and quicker rates of deterioration, the magnitude of event required to trigger failures in high-plasticity clay slopes will be smaller. With anticipated climate change, the frequency at which triggering events occur will likely increase. This will affect the frequency at which shallow first-time failures in high-plasticity clay slopes are observed across the network. In
summary, it is anticipated that with climate change the frequency of shallow first-time failures in high-plasticity clay earthworks will increase across the UK’s transport network.

8.6 Chapter Summary

Significant work has been conducted to develop numerical modelling approaches to assess the effect of climate change on high-plasticity overconsolidated clay cut slopes. The results presented show considerable advances in modelling capabilities with key physical mechanisms validated (i.e. validation of seasonal ratcheting behaviour against physical modelling), inter-related strength deterioration processes captured within a single boundary-value model and boundary conditions considering soil-water-atmosphere interactions have been validated. Using statistical down-scaling, multiple climate change scenarios have been considered to stress test future behaviour of Newbury cutting. It has been shown that numerical analyses predict a reduction in the serviceable life of the slope due to larger stress cycles driven by wetter winters and drier summers.

Within this Chapter, the uncertainties, limitations and implications of key results have been presented and discussed. Many advances have been made in current understanding of overconsolidated clay cut slope deterioration and effects of climate change but there are still significant gaps in knowledge that must be better understood if more meaningful numerical analyses of climate change effects are to be undertaken in the future.
Chapter 9

Conclusions and Recommendations for Further Work

9.1 Chapter Scope
The following Chapter summarises the key findings and the conclusions drawn from this research project followed by recommendations for future work to progress the work further.

9.2 Key Findings
1. Seasonal ratcheting, driven by repeated seasonal wetting and drying stress cycles leading to progressive failure, can explain shallow first-time failures within high-plasticity clay infrastructure slopes, and why a slope can remain stable during one wet event but fail due to an equivalent or smaller magnitude wet event later in the slopes life. This deterioration behaviour can be modelled within an unsaturated couple hydro-mechanical numerical framework including a strain-softening constitutive model.

2. High-plasticity clay cut slopes constructed at an angle greater than the residual friction angle of the soil in which they are constructed are susceptible to shallow first-time failure due to seasonal ratcheting. It has been shown that the rate of strength deterioration due to seasonal stress cycles will be exacerbated in the future if current climate change projections are representative of future weather conditions.

3. Deterioration of high-plasticity clay cut slopes cannot solely consider strength deterioration. Hydrogeological deterioration has significant
implications on the magnitude of seasonal stress cycles and the rate of strength deterioration of a slope. These processes are inter-related, and must be considered in parallel, with much more needed to be understood regarding the rate of deterioration of hydrogeological properties within the near-surface, especially considering the effects of larger stress cycles predicted within climate change projections.

9.3 Conclusions

The aims of this research project were to understand time-dependent strength deterioration of high-plasticity overconsolidated clay cut slopes and establish a framework for the assessment of these slopes under current and projected climatic conditions. Numerical analyses conducted have indicated that larger near-surface stress cycles driven by wetter winters and drier summers will increase the rate of deterioration of existing high-plasticity clay cut slopes. The aims of the project have been achieved through accomplishing the four Objectives presented in Section 1.3, the conclusions presented below have been structured around these Objectives.

9.3.1 Objective 1

Through a literature review it was established that weather-driven wetting and drying stress cycles can drive progressive failure and mobilisation of post-peak strength in high-plasticity clay slopes, the mechanism was shown conclusively through centrifuge experimentation by Take and Bolton (2011). Previous numerical modelling of seasonal ratcheting considered simple stress cycles driven by summer and winter pore water pressure boundary conditions (Kovacevic, et al., 2001; Nyambayo, et al., 2004). Repeated stress cycles led to progressive failure, but modelling was conducted within saturated frameworks and no validation of the mechanical behaviour was shown. Attention of researchers then turned to the development of more realistic soil-water-atmosphere interactions and assessment of climate change in a deterministic, worst-case scenario basis (Rouainia, et al., 2009; Elia, et al., 2017), again using numerical models that lacked validation bringing into question the validity of the conclusions drawn.
Opinions on the effect of climate change on clay earthwork slope assets have been divided; some believe that larger stress cycles will result in quicker strength deterioration and others believe that drier summers could result in more stable slopes, due to increased summer pore water pressure suctions, and a slower rate of deterioration. There are significant gaps in knowledge surrounding weather-driven behaviour and the effect of climate change on high-plasticity clay slopes that have been the focus of this research.

9.3.2 Objective 2

A numerical modelling approach for the consideration of weather-driven behaviour for high-plasticity clay slopes has been developed and validated against physical modelling data. The physical modelling data set provides detailed experimental validation that weather-driven wetting and drying stress cycles can cause progressive failure of high-plasticity overconsolidated clay slopes. For prescribed stress cycles replicating the stress cycles in the physical modelling, the numerical modelling approach developed captures characteristic behaviour of seasonal ratcheting, notably outward swelling and downward movement during wetting and predominately vertical shrinkage during drying. Whilst general trends of behaviour have been captured within numerical analyses, the magnitudes of displacements have also been shown to be representative of the physical modelling data.

To successfully model and validate the numerical modelling approach for weather-driven behaviour, an unsaturated numerical framework coupling hydrogeological and mechanical behaviour through the use of a van Genuchten (1980) style soil water retention curve and Bishop’s generalised effective stress has been used. Progressive failure has been modelled using a Mohr-Coulomb strain-softening constitutive model with the addition of a nonlocal regulatory model to reduce mesh dependency.

The mechanism of seasonal ratcheting has been explored for different slope geometries and put into context for design considering mobilised strength at failure. Recommendations for assessment of slopes at risk of shallow first-time failure due to repeated weather-driven stress cycles have been made. Ultimately, the design parameters for the use in limit equilibrium analysis of
shallow first-time failures are dependent on the desired design-life, the strain-softening behaviour of the soil, the height of the slope and the potential implications of the slope failing. Numerical analyses have indicated that slopes in clay constructed at an angle less than or equal to the residual friction angle of the soil will not fail due to seasonal ratcheting. For soils with strength that reduces to residual strength at relatively small strains, residual strength should be considered in limit equilibrium analysis. However, for soils that require significant plastic strains to reach residual strength, strength greater than residual but less than critical state should be considered in limit equilibrium analysis. This will result in a design life appropriate for most applications considering infrastructure slopes. For high risk slopes (i.e. high-speed rail) further detailed modelling of the soil of interest should be undertaken to assess design parameter selection and design life requirements.

9.3.3 Objective 3

Multiple inter-related strength deterioration mechanisms attributed to clay cut slopes have been captured within a single numerical analysis. Coupling of a saturated and unsaturated numerical analysis has been performed to allow undrained unloading, dissipation of excess pore water pressures, stress relief, seasonal ratcheting (i.e. weather-driven behaviour) and progressive failure to be captured within a single numerical analysis. On comparison of numerical analyses considering simplified cut slope behaviour in a saturated analysis (i.e. omitting seasonal ratcheting and unsaturated behaviour) and all mechanisms mentioned in an unsaturated analysis, the failure mechanism and time to failure differ considerably. For the analysis including seasonal wetting and drying, the transient nature of boundary conditions means the time taken for dissipation of post-excavation pore water pressures is much longer than constant fixed boundary conditions, but more consistent with observed slope behaviour. Also, repeated stress cycles driven by wetting and drying result in a much shallow failure surface indicative of failures observed in ageing clay infrastructure slopes.

Using the coupled modelling approach developed, more rigorous soil-water-atmosphere boundary conditions have been applied to a numerical model using a soil water balance calculation considering site-specific and regional
weather data for the life of a case study slope. The hydrogeological response of the numerical analysis has been validated against monitored data and shown to reproduce stress cycles of comparable magnitudes within the near-surface of the slope. It has been demonstrated that hydrogeological property deterioration, saturated hydraulic conductivity and soil water retention characteristics, have a significant effect on the time to failure within the numerical analyses and much more is needed to be known about how these properties change with time, the number and the magnitude of stress cycles.

The effect of root reinforcement has been explored and has been shown to be fundamental in determining the time to failure of the numerical analyses and the depth of the shear surface. Again, more is needed to be known about root reinforcement and how it can be included within continuum analyses. From the analyses considering root reinforcement, the effect of long-term strength deterioration and the magnitude of event required to cause failure has been shown clearly. Slope analyses failed during wet events that had been exceeded a number of times previously in both scenarios, and failure followed two to three wet years indicating the significance of antecedent conditions on the triggering of shallow first-time failures in clay slopes.

9.3.4 Objective 4

The results of analyses presented have indicated that wetter winters and drier summers, representative of climate change projections for the UK (i.e. UKCP09) will lead to increased rates of strength deterioration for existing slopes. The numerical modelling approach used has been validated at a number of stages to show that critical physical mechanisms have been captured. The only difference in the analyses considering climate change are the boundary conditions applied and the corresponding stress cycles that the slope model experiences. Whilst the results showed significant trends in behaviour due to projected climate change, the modelling also raised many questions that form part of the conclusions.

Deterioration of soil water retention characteristics will change factors included within the soil water balance calculation used to determine the boundary conditions as will vegetation change. Vegetation change has been highlighted
as a potential risk of climate change, along with affecting the calculation of boundary conditions, change in vegetation, root depth and density will change the mechanical behaviour of root reinforcement. These additional inter-related processes are a result of climate change and better understanding is required if numerical modelling capabilities of climate change are to be further developed.

9.4 Implications for Practice

There must be an appreciation that with every seasonal stress cycle high-plasticity clay slopes across the UK’s infrastructure network experience deterioration. Wetter winters and drier summers, as predicted by climate change projections, will increase the rate of deterioration of these assets. As earthworks deteriorate, the magnitude of events required to trigger shallow first-time failures reduce. With climate change, the frequency of potential triggering events is likely to increase and as such, shallow first-time failures within high-plasticity clay slopes will become more frequent in the future. More needs to be done to establish the slopes most at risk of these failures to reduce the potential impacts of failures across the network.

Slopes constructed at angles greater than the critical state friction angle of the soil in which they are constructed are at high risk of shallow first-time failures due to seasonal ratcheting. While new slopes constructed at these angles will be stable, older slopes that have experienced high numbers of seasonal stress cycles should be the primary focus of asset owners to target maintenance and improve the condition of these assets.

With regards to design, it is recommended that no new slopes are constructed at angle greater than the critical state friction angle of the soil. Where failure would have severe consequences, detailed ground investigation and numerical modelling should be undertaken to establish serviceable design life or slopes should be designed considering the residual friction angle of the soil.
9.5 Recommendations for Further Work

In line with the limitations in current knowledge highlighted in Section 8.4, further understanding of a number of material properties and how to include them are required if numerical modelling capabilities are to improve. These are:

- large displacement stress-strain behaviour;
- soil water retention hysteresis;
- hydrogeological deterioration due to repeated stress cycles (including the effects of desiccation cracking);
- stiffness deterioration due to repeated stress cycles;
- root reinforcement; and
- climate change effects (including; effects on vegetation, frequency of extreme events, potential implication of freeze thaw).

Regardless of whether better understanding of these critical physical properties and processes can be attained, there is a need to conduct further climate change impact assessments for high-plasticity overconsolidated clay cut slopes. A better understanding of the slopes most at risk of failing under current and future climatic conditions is required to allow the development of more meaningful asset management strategies. Different clay materials should be considered along with slope geometries and heterogeneity, including presence of discontinuities such as bedding features and joints/fissures.

Finally, one of the main areas of work that should be advanced is the consideration of adaptation strategies to establish the most appropriate remediation schemes to prolong the serviceable life of these assets suffering strength deterioration due to repeated seasonal stress cycles. This should be done to provide asset owners with the knowledge of where and when maintenance should be undertaken and what should be done to achieve the required increase in serviceable life at the lowest cost.
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Appendix A

FLAC 7 (Fast Lagrangian Analysis of Continua)

A.1 Appendix Scope

The following sections summarise the theory and background of FLAC 7, a geotechnical finite difference software package adopted for the modelling in this work. For more information, the user manuals should be consulted (Itasca Consulting Group, Inc., 2011).

FLAC is an explicit finite difference software package capable of modelling soil behaviour. Within the finite difference software FLAC, the model is discretised into deformable elements in order to solve a set of differential equations governing behaviour. Stresses, strains and displacements for discrete points are then calculated using the resulting simultaneous equations from the solved differential equations. Deformations occur within the model as a function of the stress-strain relationship adopted and applied boundary conditions. FLAC finds a static solution to an unstable physical problem using an explicit, time-stepping approach, where the calculation cycle is shown graphically below (Itasca Consulting Group, Inc., 2011);

Figure A.9.1 Explicit calculation cycle (after, Itasca Consulting Group, Inc., 2011)
Appendix A

As shown in Figure A.9.1, initially the equations of motion are considered, which calculates new velocities and displacements in relation to the existing stresses in the model. Using the new velocities and displacements, the rate of straining and thus new stresses can be obtained (Itasca Consulting Group, Inc., 2011). It should be noted that during each boxed phase the new calculated values are 'frozen' and grid values are updated. For this to hold true, one full cycle of the system must be a time-step that is sufficiently smaller than the time taken for physical information to propagate across an element. The time-step is a function of the element size and the stiffness of the material being modelled. Following numerous cycles, stresses and deformations will be transmitted through the grid (Itasca Consulting Group, Inc., 2011).

A.2 FLAC – Equations of Motion and Equilibrium

In accordance with Newton’s law of motion; \( \text{force} = \text{mass} \times \text{acceleration} \);

Equation A.1

\[
\frac{d\ddot{u}}{dt} = F
\]

The relationship between force, displacement, velocity and acceleration can be shown simplistically as follows;

![Figure A.9.2 Acceleration \( \ddot{u} \), velocity \( \dot{u} \) and displacement \( u \) due to time varying force on a mass (after, Itasca Consulting Group, Inc., 2011)](image)

When acceleration tends to zero the static equilibrium of the problem will be obtained. For a continuous solid body, Equation A.1 takes the generalised form, (note – \( i \) and \( j \) are components relating to a Cartesian coordinate system);

Equation A.2

\[
\rho \frac{\partial \ddot{u}_i}{\partial t} = \frac{\partial \sigma_{ij}}{\partial x_j} + \rho g_i
\]

Where;

- \( \rho = \text{mass density} \);
• $x_i$ = components of coordinate vector;
• $g_i$ = components of gravitational acceleration (body forces);
• $\sigma_{ij}$ = components of stress tensor.

In combination with the equations of motion a constitutive relationship (stress-strain relationship) is required. Initially the strain rate from velocity gradients is derived.

\[
\dot{\varepsilon}_{ij} = \frac{1}{2} \left[ \frac{\partial \dot{u}_i}{\partial x_j} + \frac{\partial \dot{u}_j}{\partial x_i} \right]
\]

Where:
• $\dot{\varepsilon}_{ij}$ = strain-rate components;
• $\dot{u}_i$ = velocity components.

Mechanical constitutive laws within FLAC are in the form;

\[
\sigma_{ij} := M(\sigma_{ij}; \dot{\varepsilon}_{ij}; \kappa)
\]

Where:
• $M(\ )$ = the functional form of the constitutive law;
• $\kappa$ = history parameter, may or may not be present, depending on constitutive law;
• $\Rightarrow$ means to be defined as under specific assumptions

A.3 FLAC – The Grid

The formulation of a problem in FLAC requires the solid body to be discretised into a mesh of quadrilateral elements. Each quadrilateral element is internally subdivided into two sets of constant-strain triangles. This means for each quadrilateral element, there are four constant strain triangle sub-elements. Stress components are calculated independently for each triangle and the nodal values are then taken as the mean for that point. The overlaid sub-elements are illustrated in Figure A.9.3.
A.4 FLAC – Constitutive Models

Within FLAC there are multiple constitutive models available for the modelling of soil. Of the elasto-plastic constitutive models available, three have been considered for the modelling of clay;

**Drucker-Prager model** – this model is an elastic-perfectly plastic model that can be used to consider soft clays. Within the FLAC manual it is stated that the model is not recommend for modelling geological materials (Itasca Consulting Group, Inc., 2011). For this reason, it has not been considered further.

**Mohr-Coulomb Strain-Softening model** – this model allows the modelling of shear failure within soil and inclusion of nonlinear variation of Mohr-Coulomb properties (i.e. cohesion, friction, dilation and tensile strength) depending on plastic strain increment. The Mohr-Coulomb model is relatively simple to implement and only requires knowledge of cohesion and friction angle at different plastic stains, the model performs well considering strength behaviour.

**Modified Cam-Clay model** – this model is used to model behaviour of soils where volume change affects the materials stiffness and shear resistance properties. The model is formulated for the consideration of soft clays.

In terms of representation of soil behaviour, the Cam-Clay model captures more of the physical behaviour than the Mohr-Coulomb strain-softening model. Given the relationships of specific volume and stress state used in the Cam-Clay model volumetric behaviour is much more realistic than the Mohr-Coulomb model. However, the Mohr-Coulomb model requires less input variables and has been shown to capture strength behaviour of soils well. Strength and shear failure is the main focus of slope stability and so the Mohr-
Coulomb model is acceptable for the consideration of this. In addition, the Mohr-Coulomb model has an established record of modelling overconsolidated clay slope failure (Potts, et al., 1997; Kovacevic, et al., 2001; O’Brien, et al., 2004; Ellis & O’Brien, 2007; Rouainia, et al., 2009). Given this, the Mohr-Coulomb strain-softening model has been used in this work and is presented in detail in the following section.

A.4.1 FLAC – Strain-Softening Constitutive Model

The strain-hardening/softening constitutive model in FLAC is capable of modelling progressive failure, a fundamental failure mechanism in slopes. The model is an extension of the Mohr-Coulomb constitutive model, whereby cohesion, friction, dilation and tensile strength of the material can either harden or soften as a result of plastic yielding. The stress strain relationship within the elastic region of the Mohr-Coulomb model is given as Hooke’s law expressed incrementally using principal stresses and strains (Itasca Consulting Group, Inc., 2011);

\[
\begin{align*}
\Delta \sigma_1 &= \alpha_1 \Delta e_1^e + \alpha_2 (\Delta e_2^e + \Delta e_3^e) \\
\Delta \sigma_2 &= \alpha_1 \Delta e_2^e + \alpha_2 (\Delta e_1^e + \Delta e_3^e) \\
\Delta \sigma_3 &= \alpha_1 \Delta e_3^e + \alpha_2 (\Delta e_1^e + \Delta e_2^e)
\end{align*}
\]

Equation A.5

Where:

- \( e \) = elastic;
- \( \alpha_1 = K + 4G/3 \);
- \( \alpha_2 = K - 2G/3 \);
- \( K \) = bulk modulus;
- \( G \) = shear modulus.

The Mohr-Coulomb failure criterion within FLAC is given as;
Appendix A

The failure envelope is based on Mohr’s circles for failure, obtained by plotting the maximum and minimum principal stresses. Within FLAC the failure envelope is defined for $f_s = 0$ and $f^t = 0$;

\[
\begin{align*}
  f^s &= \sigma_1 - \sigma_3 N_\phi + 2c \sqrt{N_\phi} \\
  f^t &= \sigma^t - \sigma_3
\end{align*}
\]

Where;
- $\sigma_1$ = maximum principal stress;
- $\sigma_3$ = minimum principal stress;
- $N_\phi = \frac{1 + \sin \phi}{1 - \sin \phi}$; $\phi$ = internal angle of friction;
- $c$ = cohesion;
- $\sigma^t_{\text{max}} = \frac{c}{\tan \phi}$ = maximum tensile strength.

When considering the inclusion of plastic strains, the relationships shown in Equation A.5 can be updated to;

\[
\begin{align*}
  \Delta \sigma_1 &= \alpha_1 \Delta e_1^e + \alpha_2 (\Delta e_2^e + \Delta e_3^e) - \lambda^s (\alpha_1 - \alpha_2 N_\psi) \\
  \Delta \sigma_2 &= \alpha_1 \Delta e_2^e + \alpha_2 (\Delta e_1^e + \Delta e_3^e) - \lambda^s \alpha_2 (1 - N_\psi) \\
  \Delta \sigma_3 &= \alpha_1 \Delta e_3^e + \alpha_2 (\Delta e_1^e + \Delta e_2^e) - \lambda^s (-\alpha_1 N_\psi + \alpha_2)
\end{align*}
\]

Where;
- $N_\psi = \frac{1 + \sin \phi}{1 - \sin \phi}$;
Appendix A

- \( \psi = \) dilation angle.

When yielding occurs within the strain-hardening/softening constitutive model and plastic strains occur, material properties are updated as a function of the plastic portion of total strain (Itasca Consulting Group, Inc., 2011). An example stress-strain curve with yielding is shown below;

![Stress-strain curve with yielding](image)

Figure A.9.5 Stress-strain curve with yielding example (after, Itasca Consulting Group, Inc., 2011)

Plastic strains are calculated within FLAC using the following equation;

\[
\Delta e^{ps} = \left\{ \frac{1}{2} \left( \Delta e_1^{ps} - \Delta e_m^{ps} \right)^2 + \frac{1}{2} \left( \Delta e_m^{ps} \right)^2 + \frac{1}{2} \left( \Delta e_3^{ps} - \Delta e_m^{ps} \right)^2 \right\}^{1/2}
\]

Equation A.8

Where;

- \( \Delta e_m^{ps} = \frac{1}{3} \left( \Delta e_1^{ps} + \Delta e_3^{ps} \right) \);
- \( \Delta e_1^{ps} \) and \( \Delta e_3^{ps} = \) principal plastic shear strain increments.

A.5 FLAC – Fluid-Mechanical Interaction

FLAC has the capability to model fluid flow through a permeable medium. This can be carried out independently or in parallel with mechanical computation. Two options for fluid-mechanical interaction within FLAC exist (Itasca Consulting Group, Inc., 2011);

1. Single-phase flow – this flow scheme is capable of modelling fully saturated flow, consolidation and problems where a phreatic surface is
present (i.e. above the phreatic surface pore water pressures are zero, the air phase is passive and capillary effects are neglected);

2. Two-phase flow – this flow scheme is capable of modelling unsaturated flow where an air and liquid phase are considered within a permeable medium.

The formulation of these schemes in FLAC is presented in Sections A.5.1 and A.5.2. It should be noted that within FLAC a mobility coefficient is considered rather than saturated hydraulic conductivity, it is given that;

\[ k = \frac{k_h}{g \rho_w} \]

Where;

- \( k \) = mobility coefficient (m\(^2\)/(Pa.s));
- \( k_h \) = saturated hydraulic conductivity (m/s);
- \( g \) = acceleration due to gravity (m/s\(^2\));
- \( \rho_w \) = density of water (kg/m\(^3\)).

### A.5.1 FLAC – Saturated Framework (Single-Phase Flow)

Saturated fluid flow within FLAC is calculated considering Darcy’s law, with coupled fluid-mechanical interactions being calculated within the quasi-static Biot theory of consolidation framework (Itasca Consulting Group, Inc., 2011). Darcy’s law describes fluid transport in a permeable material, which is given as;

\[ q_i = -k_{ij} \hat{k}(s) \frac{\partial}{\partial x_i} (P - \rho_w g_k x_k) \]

Where;

- \( q_i \) = specific discharge vector;
- \( k_{ij} \) = FLAC mobility coefficient (see Equation A.9);
- \( \hat{k}(s) \) = relative permeability as a function of saturation;
- \( P \) = fluid pressure.

The fluid mass balance is given as;
Equation A.11
\[ \frac{\partial \zeta}{\partial t} = - \frac{\partial q_i}{\partial x_i} + q_v \]

Where:
- \( \zeta \) = variation of fluid content;
- \( q_v \) = volumetric fluid source intensity.

The balance of momentum is:

Equation A.12
\[ \frac{\partial \sigma_{ij}}{\partial x_j} + \rho g_i = \rho \frac{d\dot{u}_i}{dt} \]

Where:
- \( \rho = (1 - n)\rho_s + n\rho_w \) = solid bulk density

The constitutive behaviour of the pore fluid, the pore fluid pressure, is a function of the level of saturation; at full saturation \( (s = 1) \) it is given that:

Equation A.13
\[ \frac{\partial P}{\partial t} = M \left( \frac{\partial \zeta}{\partial t} - \alpha \frac{\partial \varepsilon}{\partial t} \right) \]

Where:
- \( M = \) Biot modulus \( \frac{K_w}{n} \) (if \( \alpha = 1 \));
- \( K_w \) = fluid bulk modulus;
- \( n \) = porosity;
- \( \alpha = \) Biot coefficient (for incompressible soil particles \( \alpha = 1 \));
- \( \varepsilon \) = volumetric strain.

When the porous medium is not fully saturated, \( s < 1 \), the saturation is calculated as:

Equation A.14
\[ \frac{\partial s}{\partial t} = \frac{1}{n} \left( \frac{\partial \zeta}{\partial t} - \alpha \frac{\partial \varepsilon}{\partial t} \right) \]

The relationship between saturation and fluid pressure is given by a retention curve, in the following relationship:

Equation A.15
\[ P = h(s) \]
In the single-phase flow option in FLAC it is given that $h(s) = 0$ within the unsaturated zone and thus suctions and capillary effects are not included within the flow regime. Flow within the unsaturated zone in single-phase flow is solely ruled by gravity. The relative hydraulic conductivity is calculated as a function of saturation as shown below;

$$\hat{k}(s) = s^2(3 - 2s)$$

This is used to factor the saturated hydraulic conductivity as shown in Equation A.10. Within single-phase flow, the permeable medium is considered to be either fully saturated or totally unsaturated, which means that it does not account for partial saturated behaviour. It can however have fluid tension within the soil to allow theoretical suctions to develop within the model.

**A.5.2 FLAC – Unsaturated Framework (Two-Phase Flow)**

Two-phase flow is an additional option in FLAC that can be used to model unsaturated soil behaviour. In the formulation, the soil is assumed to be fully saturated by two idealised phases, a wetting (water) and non-wetting (air or gas) phase. The code allows coupled fluid-mechanical behaviour, whereby volumetric deformations influence pore fluid pressures by changes in porosity, hydraulic conductivity and capillary pressure curve parameters (soil water retention curves) (Itasca Consulting Group, Inc., 2011). The two fluids are assumed to be immiscible and are slightly compressible. The porous medium is assumed to be incompressible.

Both the phases within the logic follow Darcy’s law, where the wetting fluid is given the subscript, w (water), and the non-wetting, g (gas);

$$q_i^w = -k_{ij}^w \kappa_r^w \frac{\partial}{\partial x_j}(P_w - \rho_w g_k x_k)$$

$$q_i^g = -k_{ij}^w \mu_w \kappa_r^g \frac{\partial}{\partial x_j}(P_g - \rho_g g_k x_k)$$

Where;

- $k_{ij}$ =FLAC saturated mobility coefficient;
Appendix A

- \( \kappa_r \) = relative hydraulic conductivity (as a function of the wetting saturation);
- \( \mu \) = dynamic viscosity.

Relative hydraulic conductivity within FLAC is calculated as a function of saturation given by the van Genuchten (1980) closed form solution.

Equation A.19
\[
\kappa^w_r = S^b_e \left[ 1 - \left( 1 - S_e^{1/a} \right)^a \right]^2
\]

Equation A.20
\[
\kappa^g_r = (1 - S_e)^c \left[ 1 - S_e^{1/a} \right]^{2a}
\]

Where:
- \( a \) = FLAC van Genuchten coefficient (m in van Genuchten (1980) paper assuming \( m = 1 - 1/n \));
- \( b \) = FLAC van Genuchten coefficient b (taken as 0.5);
- \( c \) = FLAC van Genuchten coefficient c (taken as 0.5);
- \( S_e \) = effective saturation.

The effective saturation is given as;

Equation A.21
\[
S_e = \frac{S_w - S^w_r}{1 - S^w_r}
\]

Where;
- \( S^w_r \) = residual wetting fluid saturation.

The capillary pressure law (or suctions) are found by considering the differences between the fluid pressures relative to saturation;

Equation A.22
\[
P_g - P_w = P_c(S_w)
\]

Where;
- \( P_g - P_w \) = can be considered as the matrix suction;
- \( P_c \) = capillary pressure.

Capillary pressures, like relative hydraulic conductivity, are calculated using the van Genuchten (1980) form;
Appendix A

Equation 9A.23
\[ P_c(S_w) = P_0 \left[ S_e^{-1/a} - 1 \right]^{1-a} \]

Where;

- \( P_0 = \frac{\rho_w g}{\alpha} \) = FLAC van Genuchten coefficient \( P_0 \);
- \( \alpha = \) van Genuchten parameter (\( m \) in 1980 paper).

The fluid balance, considering fluids that are slightly compressible, are given as;

Equation A.24
\[ \frac{\partial \zeta_w}{\partial t} = - \frac{\partial q^w_i}{\partial x_i} + q^w_v \]

Equation A.25
\[ \frac{\partial \zeta_g}{\partial t} = - \frac{\partial q^g_i}{\partial x_i} + q^g_v \]

Where;

- \( \zeta = \) variation of fluid content;
- \( q_v = \) volumetric fluid source intensity.

The constitutive behaviour of the fluids are;

Equation A.26
\[ S_w \frac{dP_w}{dt} = K_w \left[ \frac{\partial \zeta_w}{\partial t} - n \frac{\partial S_w}{\partial t} - S_w \frac{\partial \varepsilon}{\partial t} \right] \]

Equation A.27
\[ S_g \frac{dP_g}{dt} = K_g \left[ \frac{\partial \zeta_g}{\partial t} - n \frac{\partial S_g}{\partial t} - S_g \frac{\partial \varepsilon}{\partial t} \right] \]

The following equations concern the mechanical side of the coupled fluid-mechanical interaction. The balance of momentum is given as;

Equation A.28
\[ \frac{\partial \sigma_{ij}}{\partial x_j} + \rho g_i = \rho \frac{d\dot{u}_i}{dt} \]

Where;

- \( \rho = \rho_d + n(S_w \rho_w + S_g \rho_g) \) = bulk density;
- \( \rho_w, \rho_g = \) fluid densities;
- \( \rho_d = \) dry density of soil.

The mechanical constitutive behaviour is given as an incremental response for the medium;
Equation A.29 \[ \Delta \sigma'_{ij} = H(\sigma_{ij}, \Delta \epsilon_{ij}, \kappa) \]

Where;

- \( \Delta \sigma'_{ij} \) = change in effective stress;
- \( H \) = functional form of constitutive model.

Within FLAC two-phase flow, effective stress change is calculated as;

Equation A.30 \[ \Delta \sigma'_{ij} = \Delta \sigma_{ij} + (S_w \Delta P_w + S_g \Delta P_g) \]

When plasticity is included within a constitutive model, Bishop’s generalised effective stress is used to identify failure. It is given that Bishop’s generalised effective stress is;

Equation A.31 \[ \sigma'_{ij} = \sigma_{ij} + (S_w P_w + S_g P_g) \]

A.6 FLAC – FISH in FLAC

FISH is an embedded programming language within FLAC. It allows the user to define additional functions and variables within the programme. This extends the ability and scope of application of FLAC (Itasca Consulting Group, Inc., 2011).