Guidance

G0054B A5 Earth Structures - Guide for slope stability analysis

Contents

1 Purpose

The purpose of this document is to support LU standard S1054 'Civil Engineering – Earth Structures' and provide guidance for Geotechnical Engineers assessing the stability of earth structures on the London Underground (LU) network.

2 Scope

This guidance applies to both cuttings and embankments formed in all materials typically encountered on the network.

This guide is intended for earth structures up to approximately 8m in height – earth structures larger than this may require more detailed site-specific investigation, monitoring and assessment to determine appropriate parameters.

Note:	A section is included relating to chalk cuttings. However, these should be assessed on a site-specific basis and designers should refer to Chalk Cutting Stability Assessment Report and LU report "Metropolitan Line Chalk Cuttings, Technical Review".

3 Guidance for Slope Stability Analysis

3.1 Introduction

3.1.1 Previous Research

Mott MacDonald (MM) has over the past decade undertaken applied research on the engineering performance of historic railway embankments. Much of this research has been in relation to the earth structures on the LU network, including research into the properties of clay fill and the properties to be used in the assessment of embankments and cuttings, and studies on the impact of vegetation on the stability of slopes along with proposals for the management of vegetation. In addition to LU Applied Research, MM has undertaken research on Network Rail (NR) earth structures, as detailed in the "Network Rail Seasonal Preparedness Earthworks (TSERV 567), Final Summary Report, Recommendations for Improvement to Practice" (3). MM has also undertaken a review of clay cutting slopes for the Highways Agency (HA), as reported in "HA Delayed deep-seated failure of over-consolidated clay cutting slopes" (4).

3.1.2 Review of Current Standards

A review has been undertaken of applicable standards for the assessment and design of earth structures. This includes the following:

- Eurocode 7
- BS 6031
- LU standards
- NR standards
- HA standards

It is noted that both the LU and NR Standards have now been modified to ensure compliance with Eurocode 7 (EC7). Designs under the HA shall be in accordance with BS6031, which itself has been updated to comply with Eurocode 7.

The main difference between LU and NR seems to be that whereas NR simply state that design should be in accordance with EC7, LU have been more detailed, repeating the partial factors from EC7 and specifying a reduced partial factor for shallow slips, akin to the reduced Factor of Safety (FoS) for such slips used previously.

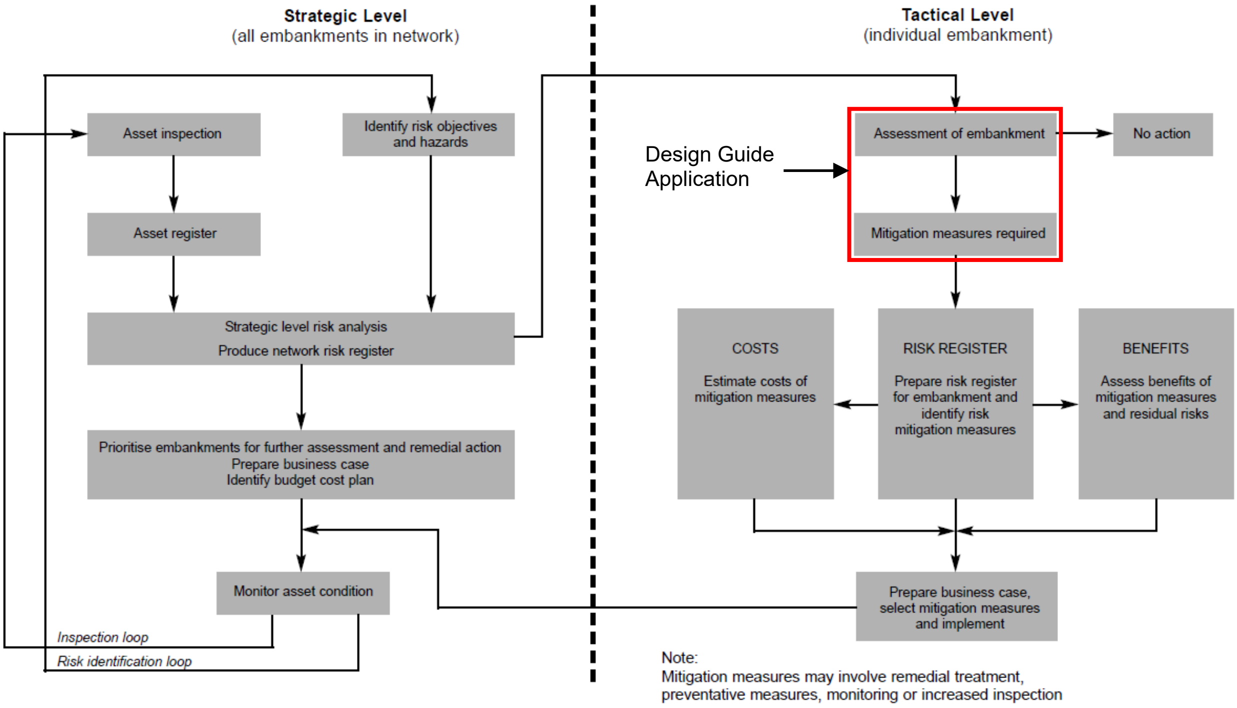
Although there is a common perception that EC7 relies on a partial factor approach, with the partial factors as specified in the National Annex, it is important to note that it does state that design values of geotechnical parameters can be ‘assessed directly’ (Cl. 2.4.6.2 (1)P), as an alternative to using a characteristic value with the appropriate partial factor applied. EC7 goes on to state that “If design values of geotechnical parameters are assessed directly, the values of the partial factors recommended in Annex A should be used as a guide to the required level of safety“. EC7 also acknowledges that when residual strength is used in design, it is appropriate to apply a lower partial factor than typically used.

3.1.3 Why Adopt a New Approach?

The current limit equilibrium approach utilised for the assessment of earth structures on the LU network has a number of pitfalls. Slope stability analyses are typically performed using slope stability software which will identify the critical failure surface based on uniform materials within an earth structure. This leads to many designers not fully considering the range of possible failure mechanisms and the implications of those failures should they occur (see Section 3.1.4) Specifically the current approach tends not to identify either: instability at the toe of earth structures where local softening is likely to occur first and can lead to failures downslope of remedial works over time; or the risk of translational failures once vegetation is removed from a slope.

The new design approach proposed in this guide encourages designers to consider all types of failure mechanism and promotes the use of directly derived material parameters in a worst credible failure mechanism, which may lead to more economic remedial works design or remove the need for works all together. A major benefit of the new approach is that it allows site specific observations to be integrated in a coherent framework for selection of appropriate geotechnical parameters. The new approach may also lead to significant benefits in prioritisation of earth structure assets for remediation in the future by considering which are most susceptible to progressive failure over time. Figure 3.1.1 below shows a typical flow chart for the strategic and tactical assessment of embankments from CIRIA C592. This design guide is intended to be applied on a tactical level but as shown below this feeds in to the overall strategic assessment of earth structures across the network.

Figure 3.1.1 Strategic and Tactical Risk Assessment Procedures (5)



3.1.4 Typical Failure Mechanisms

Many of the embankments on the London Underground network were constructed from clay fill, which in many instances is derived from weathered London Clay excavated from adjacent cuttings. This clay fill may also be derived from other in-situ materials, depending on the location of the earth structure, such as Glacial Till, Head Gravel/Clay with Flints etc. There are also many embankments constructed from more granular fill, derived from Terrace Deposits or Chalk.

Typically embankments have an Ash cap. This can vary in depth, with significant thicknesses or extents downslope often indicating previous instability, with the embankment having been built up with Ash.

Cuttings on the LU network can be found in London Clay, Chalk or Terrace Deposits depending on location.

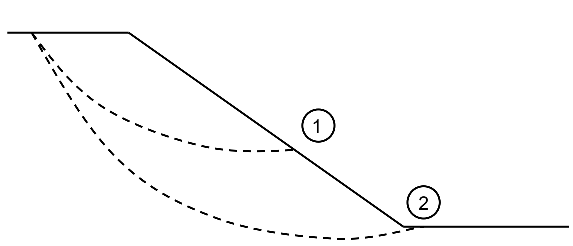
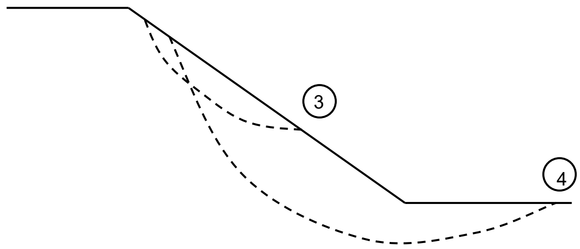
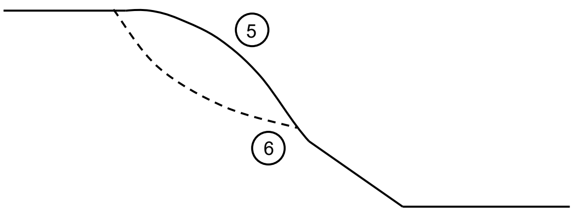
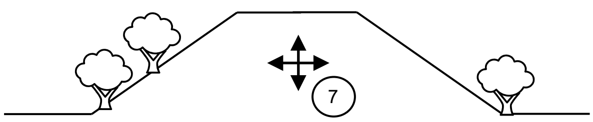
The failure mechanisms likely to be encountered will be dependent on the material the earth structure is constructed from, and for this purpose we will consider cohesive and granular earth structures separately within the document. Typical Failure Mechanisms include (see also Table 3.1.1):

- Deep-seated Instability – this can be due to progressive failure or reactivation of old shear surfaces within a cohesive material
- Shallow Instability – translational or planar type slips, typically up to 1.5m deep
- Deformation (serviceability)
- Ravelling of Ash, particularly during the Summer period as it dries.

Sections 3.4 and 3.5 discuss the first two of these mechanisms in more detail, with serviceability covered in Section 3.13 and ravelling considered in Section 3.12.

The failure mechanisms listed above should not necessarily be considered the same as any design criteria defined in LU standard S1054. For example, a shallow failure passing beneath the lineside services and affecting the track which is clearly a crest stability issue (for an embankment) should be considered as a shallow failure with reference to this guide. Designers should use judgement when considering the type of failure mechanism and the impact of that failure before selecting appropriate factors of safety. For the purpose of this guide, shallow or deep failures are defined in the context of their overall shape and depth, which are linked to mobilised soil strengths and maximum winter pore pressures which are relevant for a particular failure mechanism. LU standard S1054 in contrast defines shallow or deep in the context of the consequences of failure, with failures which impact on the track or lineside services requiring a higher Factor of Safety than those which daylight on the slope surface.

Table 3.1.1: Summary of Common Failure Mechanisms (3)

	Failure through the crest 1. Deep seated failure day-lighting through the slope. 2. Deep seated failure day-lighting through the toe.
	Failure through the slope 3. Shallow translational failure (thickness of slip ≤ 1.5m) 4. Deep seated failure day-lighting through the toe.
	Failure of the shoulder 5. Local ravelling due to over-steepening at the crest 6. Local crest instability
	Serviceability failure 7. Seasonal shrink-swell movements

3.1.5 Guide Layout

The design guide is split into three main sections. The first section discusses site investigation and back analysis for slopes (Sections 3.2 and 3.3) followed by a discussion on the progressive failure mechanism (Section 3.4), shallow stability (Section 3.5) and the proposed methodology for the application of train loading to embankments (Section 3.6)

The second section details the proposed design methodologies to be used for clay fill embankments (Section 3.7), clay cuttings (Section 3.8); granular embankments and cuttings (Section 3.9) and chalk fill embankments and cuttings (Sections 3.10 and 3.14). Shoulder instability is discussed in Section 3.11.

Section (Section 3.13) considers other issues such as serviceability.

3.1.6 Use of Design Guide

This document is intended to provide guidance to any geotechnical engineers assessing the stability of earth structures on the LU network. Individual engineers will still need to use engineering judgement both when applying this guidance, and when reviewing and interpreting the results of the analysis.

In addition users of this guide should be suitably qualified and competent to do so. The provisions of Eurocode 7 are based on the assumption that “data required for design are collected, recorded and interpreted by appropriately qualified personnel” and that “structures are designed by appropriately qualified and experienced personnel”. These statements equally apply to the use of this Design Guide. As a minimum, it is recommended that any organisation using this guide has an individual with a competency level of “Advisor”. The definition of “Advisor” is provided below. Further guidance on competency levels along with essential and desirable qualifications is included in Section 11.4.

Table 3.1.2: Definition of Competency Level for an Advisor

Competency Level	Definition: Demonstrate being able to…	Profiles: Other typical attributes may include…
Advisor	Apply proven competence to complex examples and situations in this skill area.	Has demonstrated successful delivery of this skill across a range of complex projects, Ability to think and deliver outside normal application of the codes. Providing Peer Assist on projects. Can lead Value Engineering and Risk Management Developing key contacts in specialist area across industry at national level.

3.2 Site Investigation

Eurocode 7 introduces the facility to classify a structure according to Geotechnical Category. Three categories have been introduced: Geotechnical Categories 1, 2 and 3. In some instances it may be appropriate to apply the procedures of higher categories to justify more economic designs for earth structures which fall into a lower category.

The three Geotechnical Categories are:

Geotechnical Category 1:

- Small and relatively simple structure.
- Negligible risk.
- Experience and qualitative geotechnical investigations.
- Routine design methods.

Geotechnical Category 2:

- Conventional types of structure and foundation e.g. spread foundations, embankments and earth structures, retaining walls.
- No exceptional risk or difficult soil/loading conditions.
- Quantitative geotechnical investigations.
- Quantitative analysis.

Geotechnical Category 3:

- Structures or parts of structures falling outside Categories 1 and 2.
- Very large or unusual structures.
- Highly seismic areas.
- Abnormal or high risk.
- Require bespoke parameters and analysis e.g. numerical modelling and specialist ground investigation.

Categories 1 and 2 will be where the majority of earth structures on the LU network fall, and the basic approach outlined in this design guide will be appropriate. This is underlined by Cl 3.6.1.1 in LU standard S1054, which states "In accordance with BS EN 1997-1 Section 2.1, the design requirements of a slope for Strengthening or Renewal works within the LU environment generally meets the classification of a Geotechnical Category 2 Structure".

If laboratory shear strength tests are carried out, then some care is needed in their specification and interpretation. For example, conventional triaxial tests may be used to determine peak effective stress parameters for clays. However, peak strengths may not be appropriate for use in subsequent analysis, since (depending on the problem being considered and site specific conditions) residual, or critical state strengths may be more relevant. The desiccated nature of some parts of the earthworks also needs to be recognised.

For example, near surface samples of clay may have higher peak effective stress parameters (notably high c´ values) than deeper samples below the central core of an embankment (typically "wetter" than below the slopes). Hence, sample locations, and sample reconsolidation procedures need to be appropriately specified. Mapping of natural moisture content profiles across an embankment cross-section can be a practical and cost-effective investigation technique, provided potential seasonal variations in moisture content are recognised and taken into account.

However, there may be instances where carrying out some more advanced specialist ground investigation will result in a more economic design. An example of this would be undertaking in-situ permeability testing – this would then allow a more appropriate groundwater regime to be derived for analysis, which may still be undertaken using routine limit equilibrium methods. Another example would be the use of monitoring of groundwater and earthworks deformation at a site over a period of more than 12 months, including a wet winter during the monitoring period. Again, the outcome could be a more appropriate groundwater regime and deformation mechanism to be used in routine analysis, which may significantly affect the scope and cost of remedial works.

There will however be some earth structures which will fall into Geotechnical Category 3, and for these more advanced ground investigation will be required in conjunction with the use of numerical modelling for some or all of the stability assessment and design. Examples of such earth structures would include very high, clay cuttings (in excess of 8m), where failure could directly impact the track and safe running of trains.

3.3 Back Analysis

A back analysis approach for deriving soil parameters has often been adopted when considering earth structures which have failed or are in poor condition. The parameters derived are then used in the design of the necessary remediation works on the failed/failing earth structure.

LU standard S1054, Civil Engineering – Earth Structures, prescribes a back analysis approach when considering earth structures with evidence of previous slope failures and notes that partial factors of unity should be applied to loads and material properties.

By applying partial factors of unity and establishing other stability defining variables for the earth structure condition at the time of failure the soil parameters should be fixed to give a factor of safety of 1.0 (100% utilisation factor). These characteristic soil parameters can then be factored accordingly for use in subsequent design stages. Section 3.3.1 lists the main factors that can be varied in a back analysis and Section 3.3.2 highlights potential pitfalls of the back analysis approach when considering the design life of the earth structure.

3.3.1 Variables

The back analysis approach is geared towards deriving realistic material parameters for the earth structure at the time of failure. To do so a combination of the following factors would normally be assessed to accurately define the earth structure condition at this time.

- Groundwater – The pore water pressure within the embankment or cutting is a key driver of slope stability. In terms of back analysing a failed earth structure, it is unlikely that extensive groundwater monitoring data will be available from which to deduce the groundwater regime at the time of failure. In this case a parametric approach is recommended whereby a realistic range of groundwater levels are proposed and the varying effect on slope stability assessed in the determination of appropriate soil parameters. Consideration of both the presence and condition of earth structure drainage should be made when assessing the groundwater level.

- Failure Mode – In slope stability analyses the shape of the failing mass has an effect on the factor of safety. Therefore, to perform an accurate back analysis the shape of the failure should be assessed. This is most likely to come from site observations where entry and exit points on the earth structure may be visible (e.g. a backscar and toe bulging respectively). A decision can then be made regarding the shape of the failure between these two points; i.e. circular or non-circular. In cases where a back analysis is being undertaken to verify parameters used in a completed design the failure may have been surveyed as part of the remedial works in which case a fully specified slip surface could be used in the back analysis giving a high degree of confidence in that particular variable.

- Vegetation – This can act to increase shallow stability on earth structure slopes by increasing the shearing resistance (see Section 3.5.1). Therefore, earth structure vegetation should be taken into account when performing a back analysis – if the slope is densely vegetated with trees, large values of root cohesion may be mobilised.

- Additional Resisting Forces – There may be additional forces acting on the failed material that are not immediately apparent and if omitted could result in unsafe soil parameters being derived. Examples may include old counterfort drains (see Section 3.8.3.2), grouted zones, or other historic stabilisation measures. For larger failures, greater than 3-4m in depth, the shear forces acting on the sides of the failure will be significant and should be included in any back analysis. Also, for non-linear earth structures or failures adjacent to structures (bridges, wingwalls etc.) there may be 3D effects on the failure which will create additional resisting forces. Again, these should be quantified and included in any back analysis. The analysis in a software program such as SLOPE/W will assume an infinitely long slope.

3.3.2 Long-term Considerations

Following derivation of parameters from a back analysis, designers should be aware that the conditions used for the back analysis may not be suitable for the design life of the earth structure (120 years for LU assets).

The worst credible long-term groundwater regime should be chosen for design (see Sections 3.7.3, 3.8.3, 3.9.3 and 3.10.3) with particular consideration of how pore water pressures will change in cuttings or embankments over time. The long-term performance of any drainage should also be considered and if it is to be relied upon to lower pore water pressures in the design this should be included in the maintenance plan. The same applies for earth structure vegetation, as this may change over the design life of the earth structure. In particular, the implementation of any remedial works to a failed slope is likely to change the vegetation type and pattern.

3.4 Progressive Failure (Deep Seated Instability)

3.4.1 Description of Progressive Failure Mechanism

As discussed by O'Brien (6), it is generally assumed that the strength of a soil layer is mobilised uniformly around a potential failure surface and that it remains constant with time, together with a water table level that is a uniform distance below the slope surface and is also constant with time.

Applied research has previously indicated that the strengths mobilised in 'first time' deep seated failures of clay slopes (both cuttings and embankments) of intermediate to high plasticity are primarily influenced by progressive failure. Progressive failure is a strain dependent mechanism, and is a result of the mobilised strength of medium to high plasticity clay fills degrading over time. This strength degradation commences at the toe and then propagates into the earth structure core.

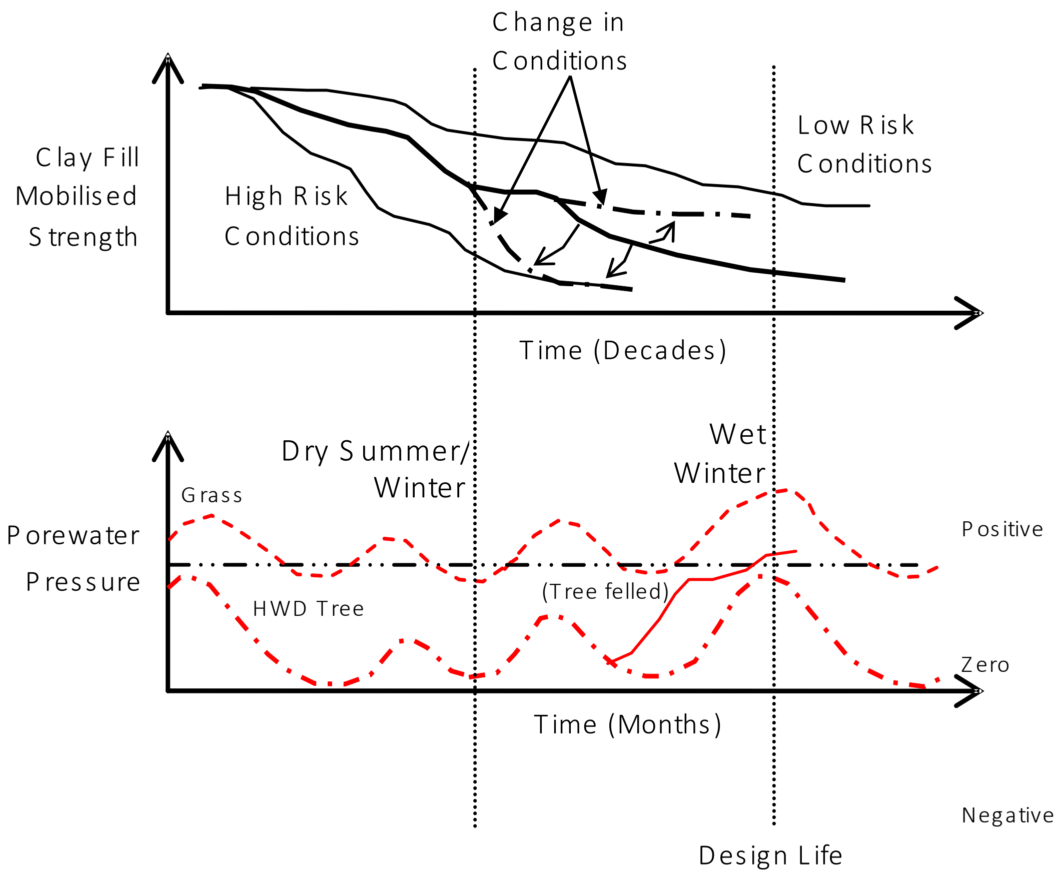
The following sections discuss the progressive failure mechanism for embankments and cuttings, along with appropriate soil parameters based on the risk of progressive failure occurring. Clays with low plasticity index or with a low clay fraction are not prone to this failure mechanism – evidence suggests that shallow slips are more of a concern. Hence the risk of progressive failure in such clays will tend to be low and appropriate parameters are recommended in Section 3.4.4 for analysis.

3.4.1.1 Embankments

Previous research regarding embankment stability (3) has demonstrated that the strain softening of plastic clay fills is initiated by seasonal variations in pore water pressure. Repeated shrink-swell cycles, caused by seasonal variations in climate, lead to a build-up of plastic strain initially at the embankment toe and then propagating into the embankment along the base. Numerical modelling undertaken as part of this research showed that just prior to failure, the main body of the embankment remains at peak strength, with a narrow basal layer reduced to residual values. The increase in pore pressure during the subsequent winter cycle causes the embankment to become unstable and the failure surface propagates up towards the crest. This modelling was carried out using the finite difference program FLAC (Fast Lagrangian Analysis of Continua) and was based on a methodology developed during previous LUL applied research, as summarised by O'Brien et al (7).

Background information on the strain softening model used and some of the results from the modelling are included in Appendix A. Figure 3.4.1 illustrates this degradation of strength and pore water pressure changes with time within old railway embankments. This emphasises the dynamic nature of the interactions between the clay fill, vegetation and climate contrasting with the conventional 'steady state' assumptions.

Figure 3.4.1: Schematic Diagram of Changes in Mobilised Strength and Pore Water Pressure (after O'Brien (2007))



The degree of degradation in strength of a clay fill over time can be estimated by assessing the risk of progressive failure of an earth structure. As can be seen schematically in Figure 3.4.1 the higher this risk, the greater the potential degradation and/or speed of degradation in strength of a clay fill. Therefore it is important to understand what factors govern this rate of deterioration. An assessment of these factors on a site-specific basis can then assist in determining the risk of progressive failure. This can then feed into an appropriate analytical model for the site to allow stability to be assessed and appropriate measures to be implemented to ensure the earth structure has the required design life.

The rate of deterioration is governed by a number of site specific factors:

- Groundwater profile and magnitude of seasonal variations
- Permeability of both the fill and the underlying material for an embankment
- Existence, condition and maintenance of toe drainage
- Fill properties
- Soil profile e.g. presence of weaker layers below the embankment, or presence of more permeable soil layers which will create under drainage
- Type and density of vegetation
- Slope geometry (height and angle of slope).

Strength parameters for clay fill should reflect the material's inherent characteristics, which in turn reflect the potential for progressive failure. Previous research also highlighted that this process is scale dependent and that the strength parameters for clay fill should also reflect the size of the embankment (8).

3.4.1.2 Cuttings

Progressive failure is also known to be a key factor when considering deep-seated failure mechanisms of cuttings in high plasticity clays, Chandler (11). This coupled with long-term recovery of pore pressures within the cutting can lead to delayed failure of cuttings.

Excavation of cuttings in stiff clay generates negative pore water pressures due to the unloading of the soil. These pore pressures will increase over time until long term values are attained – however, this process can take decades to occur. Post-excavation swelling of the clay causes strains to develop, initially at the toe of the cutting where there has been the greatest unloading of the soil. These strains lead to a reduction in shear strength, such that the average mobilised shear strength along a slip surface is part way between peak and residual.

Numerical modelling (4 & 9) has shown that strains initially develop at the toe and progress into the slope, increasing until a residual strength is reached. At this point the remainder of the cutting will be close to peak strength. The size of this residual zone will increase until the overall mobilised strength is such that failure occurs. Therefore the conceptual model for assessing progressive failure of cuttings is very similar to that outlined for embankments formed of a cohesive material. There is a basal layer at the toe to which reduced strength parameters will usually be assigned, with the remainder of the cutting being assigned conservative peak parameters.

As outlined by Ellis and O'Brien (9), the height of a cutting is also a key consideration in determining the likelihood of delayed failure. Numerical modelling indicated that cuttings with a height of 4m or less are unlikely to fail due to a progressive failure mechanism. Shallow translational failures are likely to be more of a concern for these cuttings. The modelling also indicated that for cuttings greater than 8m in height, mobilised strength parameters may be lower than those traditionally assigned, and which were based on back-analysis of slopes of, typically, between 5m and 7m in height.

As for an embankment, the rate of deterioration of the strength of the material at the toe of the cutting is linked to the risk of progressive failure occurring. Key factors are:

- Porewater pressure regime
- Existence, condition and maintenance of toe and slope drainage
- Soil properties
- Soil permeability
- Presence of any high permeability material above, within or below the cutting slope
- Type and density of vegetation
- Height of cutting
- Angle of slope

Strength parameters for the clay cutting should reflect the material's inherent characteristics, which in turn reflect the potential for progressive failure.

3.4.2 Conceptual Model

Conceptual models for progressive failure in both embankments and cuttings are outlined in the following sections. These models are supported by a number of published case studies and technical papers. The Selborne cutting stability experiment (10) between 1987 and 1989 performed a controlled failure of a 9m cut slope in Gault Clay by pore pressure surcharging. The test showed progressive failure to be the eventual cause of the collapse initiating from the toe of the slope. Chandler (11) also highlighted progressive failure as the dominant process involved in the failure of high plasticity fissured clay slopes. Take and Bolton (12) experimentally showed using centrifuge modelling that seasonal variations in pore pressures within embankments can lead to progressive failure with irrecoverable seasonal deformations at the toe.

It is considered that these conceptual models, when combined with the strength and groundwater parameters outlined in Sections 3.4.4 and 3.7.3/3.8.3 for embankments/cuttings respectively, represent a 'worst credible' failure mechanism. Therefore, the partial factor for material properties is 1.1. Section 3.4.5 outlines how this model should be applied when considering compliance with both the current LU standards and Eurocode 7.

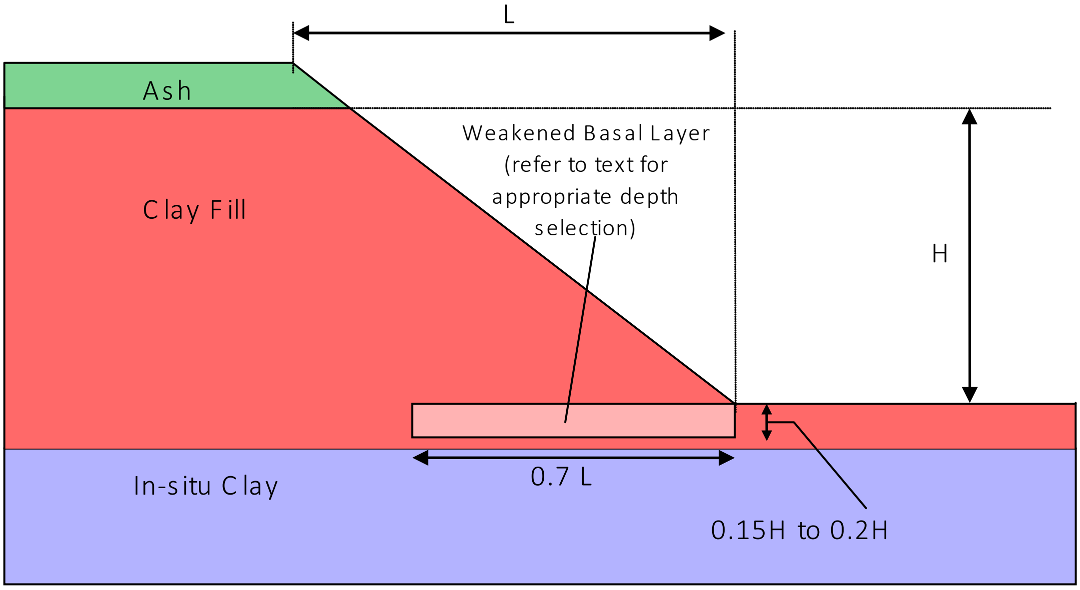
It should be noted that factors of safety obtained from these models may be lower than 1.0. This is to be expected. This conceptual model is based on the long term groundwater conditions and properties of the clay, which allow for future degradation of strength and wet winter conditions, and therefore do not represent existing conditions, where traditionally if the earth structure has not failed one would expect to achieve a FoS equal to or greater than 1.0. Referring back to Figure 3.4.1 clearly demonstrates this – the mobilised strength of the clay fill reduces with time and groundwater pressures may significantly increase depending upon climatic conditions. This conceptual model is attempting to determine these long term conditions. The aim is then to ensure adequate measures are implemented to ensure that even with this degradation over time, the embankment will not fail e.g. by the use of reinforcing elements such as piles, retaining walls, soil nails or a drainage management strategy.

Section 3.4.4 outlines parameter selection for the models shown in Figure 3.4.2 and Figure 3.4.3. In particular it is important to note that the strength adopted for the weakened basal layer is dependent on the risk of progressive failure, and this classification of risk is discussed in detail in Section 3.4.3. It depends on careful site specific observations. Sections 3.7.3 and 3.8.3 discuss groundwater pressures for design, for embankments and cuttings respectively. It should be emphasised that although considerable attention is often given to soil parameters, the assumed groundwater pressures have a greater impact on overall stability, and they also exhibit more significant site specific variations.

3.4.2.1 Embankments

As discussed in Section 3.4.1.1, research indicated that just prior to failure, the main body of the embankment remains at peak strength, with a narrow basal layer at reduced strength. It is therefore proposed that a more appropriate model would be to use cautious peak strength for the clay fill, and to include a weakened basal layer extending from the toe, as shown in Figure 3.4.2.

Figure 3.4.2: Conceptual Embankment Model



The dimensions of the weakened basal layer in Figure 3.4.2 are based on calibration of limit equilibrium slope stability analyses with numerical modelling of the progressive failure mechanism (3).

For first time failures, the thickness of the shear zone is likely to be much less than 10% of the embankment height. It is apparent from observations of shear zones, slope failures and appropriate numerical modelling that the overall shape of the progressive failure mechanism is non- circular. In order that simple and robust limit equilibrium analyses can be used, it is recommended that a thicker zone is adopted for the weakened basal layer, of between 15 and 20% H (where H is the height of the clay core), as shown on the model above for initial stability analysis at the conceptual design stage. This thicker zone enables circular failure surfaces to be used, which generally calculate similar Factors of Safety to a non-circular failure surface with a thin (less than or equal to 10%H) weakened basal layer. For detailed design the use of non-circular slips should be considered during the design process, to ensure that the design is robust. In particular, there may be some site-specific instances, such as knowledge of previous instability, or long term sub-surface deformation monitoring, which would merit the use of the non-circular slip analysis even during preliminary design stages.

The location of the basal layer is shown on Figure 3.4.2 to be wholly beneath the toe of the embankment. However, this location is dependent on the relative stiffness of the soils, and will sit above the stiffer material, e.g. in-situ overconsolidated clays, such as weathered London Clay. This may dictate that the basal layer will be located around the toe, rather than beneath. This should be assessed on a site by site basis. Conversely, the presence of softer layers beneath the embankment (e.g. soft bands of alluvial clay) may mean that the basal layer is deeper than shown on Figure 3.4.2 (refer to Section 3.4.2.1.1).

3.4.2.1.1 Weaker Layers

Previous research has shown that the inclusion of a “soft“ alluvial layer at the base of an embankment had a significant impact, reducing the number of seasonal cycles to failure i.e. decreasing the time to progressive failure occurring.

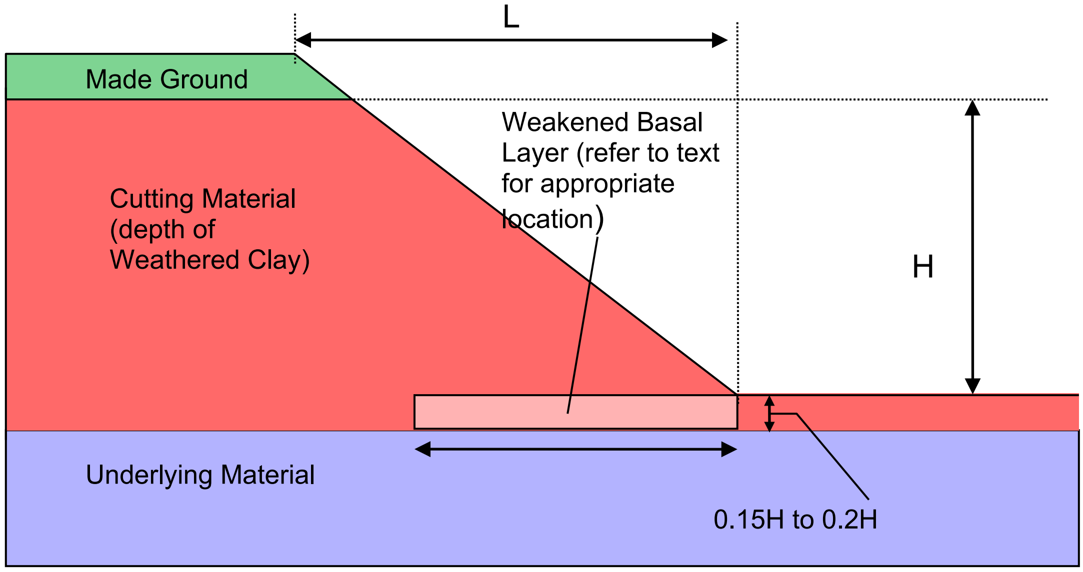
Modelling undertaken indicated that plastic straining was concentrated within the Alluvium leading to a progressive failure. It was also shown that once straining is initiated at the toe within the Alluvium, failure is more rapid and the total displacement before the model becomes unstable is less than when the Alluvium was not present. This is believed to be due to the Alluvium being less able to redistribute the shear stress associated with the localised plastic straining due to its reduced stiffness.

Therefore, it is particularly important to identify the presence of Alluvium or other ‘soft’ layers under the toe/lower third of a slope, as it is here where the strains start to develop. If a layer of softer material is present, this will potentially govern the location of the weakened basal layer, as discussed in the previous section.

3.4.2.2 Cuttings

As discussed in Section 3.4.1.2, research has indicated that just prior to failure, the main body of the cutting remains at or close to peak strength, with a narrow basal zone at reduced strength. It is therefore proposed that a more appropriate model would be to use cautious peak strength for the main body of the cutting, and to include a weakened basal layer extending from the toe, as shown in Figure 3.4.3

Figure 3.4.3: Conceptual Cutting Model



It should be noted that for cuttings formed in Weathered London Clay, the boundary between the Weathered and Unweathered London Clay will have a significant influence on the formation of the progressive failure mechanism. As discussed by Skempton (13), failures have been observed to pass along this interface. Therefore, high cuttings, which may be partially formed in both the weathered and unweathered material, may be more prone to failures daylighting at the location of this interface, rather than at the toe of the cutting. As for embankments, the weakened basal layer will occur in the lower stiffness material, and therefore will not always be wholly beneath the toe of the cutting as shown on Figure 3.4.3.

3.4.3 Risk Classification

The derivation of appropriate soil parameters for the weakened basal layer in the conceptual models proposed in Figure 3.4.2 and Figure 3.4.3 is dependent on the risk of progressive failure for a particular earth structure. The greater the reduction in the strength of the weakened basal layer, the greater the potential risk of progressive failure. To assess the risk of progressive failure, there are a number of factors to consider:

i. Soil Properties such as Liquid Limit and Clay Fraction
ii. Permeability of the fill/cutting material and the underlying strata
iii. Drainage conditions
iv. Earth structure geometry e.g. height of clay core
v. Evidence of instability i.e. site observations
vi. TRV Data
vii. Vegetation.

It should be noted when considering the above factors, that along a particular site these factors may vary, potentially leading to a change in risk classification and therefore a change in appropriate soil and groundwater parameters.

The following sub-sections discuss the above factors and how they can impact on the potential for progressive failure.

3.4.3.1 Soil Properties

Shrink-swell induced progressive failure requires:

- Clay particle reorientation and large post-peak strain softening towards low residual friction angles
- Significant shrinkage/swelling and associated volumetric strain with changes in moisture content

These two factors can be considered by assessing the clay fraction and liquid limit of the embankment fill or cutting material. The following 3 by 3 matrix indicates the risk of progressive failure based on these factors alone. This table is based on data presented by Wesley (14) and Stark and Eid (15 and 16). As will be seen in the following sections, other factors also have an influence on the risk of progressive failure, and this matrix should not be considered in isolation.

Table 3.4.1: Risk of Progressive Failure, Related to Index Properties

Clay Fraction	Liquid Limit		
	> 60%	40% to 60%	< 40%
< 25%	L	L	L
25% to 45%	H	M	L
> 45%	H	H/M	L

3.4.3.2 Permeability

The permeability of both the embankment fill/cutting material and the foundation material are key factors in determining the porewater pressure changes likely in the slope, and therefore will have an influence on the likelihood of progressive failure occurring.

Briggs et al (19), show that a lower permeability fill restricts both surface infiltration of rainfall into the soil, and drying of the soil. As a result porewater pressures within the fill are affected less by seasonal rainfall than if the fill had a higher permeability. Modelling has shown that as a result lower values of porewater pressure are attained in the lower permeability fill. This is supported by Nyambayo et al (18) who report that seasonal pore water pressure change in embankment fill due to vegetation increases with increases in permeability.

With regards to assessing the permeability of the embankment fill, it is important to understand its macro fabric (17). Typically, the old railway embankments were constructed from dumped clay fill without compaction – this clay fill often contained varying amounts of silt, sand and gravel. Therefore its structure and macro fabric is quite different to that of the natural clay, potentially leading to higher values of mass permeability (18).

O'Brien (7) reports that the permeability of a "dumped" (or end tipped) clay fill derived from London Clay is between 6 x 10-7 and 2 x 10-9 m/s based on in-situ tests. This is more than one order of magnitude higher than that reported for the natural London Clay.

On the LU network the embankments are formed from a range of materials, dependent on the natural material in the vicinity of the earth structure. Therefore, as well as understanding the permeability of London Clay fill, it is also necessary to understand the likely range of permeabilities for other clay fills commonly encountered. This would need to be assessed on a site-specific basis by the use of testing such as in-situ permeability tests and continuous sampling (with split, air-dry and log/photograph) to allow more detailed descriptions of the soil fabric. This is an example where undertaking more specialist ground investigation may be economically beneficial as it will allow a more site-specific groundwater regime to be derived. However, designers should be aware that it may be difficult to determine the permeability of clay fill with sufficient confidence in certain areas of the LU network given the constrained environment around the railway. This may limit the amount of testing that can be carried out which may then be insufficient to assess the likely variability in clay fill permeability.

The permeability of a cutting will influence the time to failure, and is therefore a factor when considering the risk of progressive failure. Pore pressures will take longer to recover to long-term equilibrium values in a cutting formed from a low permeability material than in one with a higher permeability (11).

3.4.3.3 Drainage Conditions

A well drained slope will serve to reduce the risk of progressive failure in that it increases the time to failure i.e. there is a reduction in the rate of degradation of strength along the base of the embankment or cutting. In particular, functioning toe drainage is beneficial as it reduces the water pressure at the toe in the winter months, resulting in greater mobilised shear strength.

However, a poorly maintained toe drain will be detrimental, and can be more detrimental than no drainage at all. Ponding of water at the toe will lead to an increase in water pressures at the toe, and therefore softening of the surrounding in-situ materials. This will result in an increased rate of strength degradation propagating from the toe. It is important to note that repair/reinstatement of the drainage system will reduce this rate of degradation, but it cannot ‘undo’ the weakening of the soil within the shear zone that has already taken place.

Slope drainage present on a cutting such as counterfort drains are beneficial as they reduce the porewater pressures near the surface of the cutting. Where such drains are present, consideration should be given to their ability to function as intended over the design life of 120 years. Normally it should be assumed that the effective design life of drainage is less than 20 years, unless there is effective maintenance. The design life can be extended but this needs regular maintenance, via piped drains and closely spaced manholes. Designers should refer to LU standard S1052 (Civil Engineering – Gravity Drainage Systems) for guidance on design life and maintenance of LU drainage assets.

The presence of an underlying more permeable layer (such as alluvial sand/gravels, River Terrace Deposits or Chalk) beneath the embankment will be extremely beneficial, as it results in the clay fill being underdrained. Recent studies (19) have shown that an underlying layer which is one order of magnitude more permeable than the overlying soil can lead to effective underdrainage. This can lead to a significant reduction in the risk of progressive failure.

3.4.3.4 Earth Structure Geometry

The height of the clay core of an embankment or the height of clay within a cutting also influences the risk of progressive failure. The height of the clay core or cutting clay height is not equivalent to earth structure height where there is a cap of Ash or Made Ground present - refer to Figure 3.4.2 and Figure 3.4.3 for a definition.

As reported by O'Brien (6), research has shown that if an embankment is less than 4m high the risk of deep seated, progressive failure is reduced, refer to Figure 3.4.5.

Ellis and O'Brien (9) state that the effect of progressive failure tends to increase quite significantly with changes in cutting height between about 4m and 7m. Above this height the effect of progressive failure is still significant but further increase in height does not show a comparable increase in the effect of progressive failure. For low height cuttings (below 4m), the displacements which occur at the toe (and the associated strength degradation) are reduced, and therefore a clay height of 4m within a cutting is proposed to distinguish between cuttings deemed to be at high or intermediate risk of progressive failure (see Figure 3.4.6 in Section 3.4.3.8).

As cutting height increases, there is however an increasing likelihood that the cut is not entirely within a weathered material, and it has been previously shown that the interface between the weathered and unweathered clay plays a significant role in the failure of cuttings, with failures likely to daylight at this point on the slope. As noted by Chandler (20), depths of weathering greater than 8-10m in London Clay are relatively rare, and will often be shallower. Therefore in large cuttings the location of the interface between the weathered and unweathered clay should be identified. This is most easily done during site investigation (see Section 2) where a combination of trial pits and continuously sampled boreholes should highlight the interface. An experienced geologist should be available to log the sampled materials.

It has also been shown that time to failure of a cutting is related to slope angle i.e. there is a general increase in time to failure with a reduction in slope angle (11).

For low height earth structures, i.e. with a clay core height of less than 4m (embankments or cuttings) the primary failure mechanism is unlikely to be deep seated instability due to progressive failure. For these earth structures alternative deformation/failure mechanisms (refer to Table 3.1.1), such as crest instability, local ravelling, shallow translational, or vegetation induced shrink-swell, are more likely. An exception is if the earthwork has been constructed across areas affected by periglacial processes, such as solifluction induced shear zones (occasionally observed beneath embankments built on sidelong ground). Hence, the parameters selected, and associated analytical methods, should reflect the change in risk, and likely failure mechanisms.

Retaining walls are commonly encountered at the toe of LU embankments and cuttings, although the type, scale and condition of these walls can vary considerably. Consideration needs to be given as to how the geometry and robustness of any retaining structure may modify the failure mechanisms which may affect the earth slope (see Section 3.8.6 for guidance).

3.4.3.5 Evidence of Deep Seated Instability

The importance of site observations should not be forgotten when reviewing the risk of progressive failure of an earth structure. Even if the other factors outlined are all pointing to a low risk of progressive failure, if there is clear evidence of deep seated instability on a site, whether it be from monitoring, or the observed profile of the slope (tension cracks at the crest and/or bulging at the toe), or from evidence reported in the desk study and/or principle inspection reports, then this needs to be taken into account when determining appropriate parameters for analysis. However, localised deformations of the earth structure should not be considered as evidence of instability. For example, leaning cable posts and trees may not necessarily indicate deep seated stability issues and careful consideration should be given before designating an earth structure as unstable in the context of the progressive failure mechanism described as this will have significant implications for design. Examples of some evidence of deep seated instability and more localised earth structure deformation are shown in Appendix B.

There may also be occasions where many of the factors outlined suggest that an earth structure has a high risk of progressive failure, when in fact there would appear to be no obvious signs of movement i.e. the slope profile is uniform, etc. Consideration should then be given to what is assisting the slope in terms of its stability e.g. is the toe heavily vegetated with high water demand trees which are maintaining suctions within the embankment core? Removal of these beneficial factors could lead to failure in the future.

3.4.3.6 Track Recording Vehicle Data

Track Recording Vehicle (TRV) data is a measure of the alignment of the tracks, and can be used to assess track quality. If an improvement in quality is shown by the data it may be a sign that maintenance such as reballasting has been undertaken.

TRV data is often available for cuttings and embankments, and can be an indication of movement of the slope. However, care needs to be taken reviewing this data, as the movement indicated by the TRV data may in some instances be related to the seasonal shrink-swell of the clay fill only. For embankments composed of medium to high plasticity clay fill, seasonal shrink-swell movements will be significant if mature high water demand trees are close to the track.

It should be noted that the track movement or deterioration shown by the TRV data may be completely unrelated to the earth structure and related to other factors such as condition of ballast, trackbed formation and track drainage.

3.4.3.7 Vegetation

Vegetation plays an important role in deep-seated stability primarily due to its influence on the groundwater regime, and, for embankments, on track deformation. The presence of high water demand (HWD) trees on slopes can lead to suctions being maintained in the clay even following a winter period. This will significantly increase the stability of the slope. However, the adverse impact of HWD trees is that they will cause large seasonal deformations within their zone of influence – settlement during summer and swelling during winter. Severe track serviceability problems have been observed along embankments composed of medium to high plasticity clay fills, especially during relatively dry summers. The primary cause is large increases in suction (hence increasing effective stress) induced by adjacent high water demand trees, causing large differential track settlements. In this condition the earth structure actually has a high Factor of Safety against slope instability. Hence slope or track deformation is not necessarily proof of a low factor of safety, however seasonal movements may lead to strength degradation and long term instability after a number of shrink-swell cycles, particularly if the trees are removed or die. Table 3.13.1 lists the water demand of many common tree species.

Root reinforcement can also lead to significant increases in slope stability, although this beneficial effect is usually restricted to shallow depths (typically less than 2m). Therefore, the presence of vegetation may in some instances be a significant factor in contributing to the existing stability of a slope, especially when mature HWD trees are located in the lower third of a slope.

3.4.3.8 Summary

In order to facilitate a practical and consistent approach to assessing the risk of deep seated instability due to progressive failure, flow charts have been developed to assist with the derivation of appropriate soil parameters and partial factors for the weakened basal layer. The flow charts shown in Figures 3.4.4, 3.4.5 and 3.4.6 should not be used in isolation, but must be considered in the context of the overall site conditions, and the factors discussed in Sections 3.4.3.1 to 3.4.3.7 inclusive. Table 3.4.2 provides a summary of the main risk factors, and their potential influence on deep seated instability and seasonal deformation, for embankment assessment (also refer to Figures 3.4.4 and 3.4.5). Table 3.4.3 provides a summary of the risk factors and their influence on deep seated instability for cuttings (also refer to Figures 3.4.4 and 3.4.5) . Further discussion on soil parameters and partial factors presented on these flow charts (Figures 3.4.4, 3.4.5 and 3.4.6) is contained within Sections 3.4.4 and 3.4.5.

When using Figure 3.4.5 and Table 3.4.2, it is important to distinguish between two different scenarios, i.e. firstly the conditions which may lead to seasonal deformation and track serviceability problems (usually due to a combination of high or very high water demand trees and high plasticity index embankment clay fill), and secondly the conditions which may lead to a high risk of deep seated instability. Evidence of track deformation, is not necessarily evidence of slope instability. Seasonal deformation, and track serviceability issues are discussed in Chapter 12. In general, serviceability issues are more common than deep seated instability issues across the LU network.

A factor which requires considerable judgement is "evidence of poor drainage". Firstly a global assessment of the embankment and adjacent site topography needs to be made in order to identify low spots along either the embankment toe, or the surface of the embankment clay fill (i.e. over-deep areas of ballast which could act as water traps). In these low spots can water drain or will it be trapped, (leading to locally high groundwater pressures, or "perched" water table effects)? Will the adjacent topography lead to surface water drainage towards low spots or not? Within low spots is there effective drainage, or evidence of poor drainage (marshy ground, hydrophilic vegetation, etc.). Other scenarios which experience indicates can lead to poor drainage are: embankments which have been built on side-long ground, where the "uphill" side of the embankment can act as a "dam" during periods of high rainfall (unless there is effective toe drainage). However the "uphill" embankment slope often has a relatively low risk of deep seated instability because the overall ground geometry is beneficial; embankments which have been widened can be vulnerable because the interface between the original and widened parts of the embankment often acts as a route for infiltration of rain and surface water into the embankment fill, which then can lead to local "perched" water tables and relatively high groundwater pressures, leading to a higher risk of instability. Hence the "evidence of poor drainage" needs to include a thoughtful review of a wide range of factors which could lead to poor drainage, and not just a local patch of wet ground observed during a site visit or ground investigation.

Figure 3.4.6 and Table 3.4.3 provide a summary of factors which need to be considered when assessing the risk of deep seated instability in clay cuttings. In contrast to embankments, very few problems have been reported of seasonal deformation in cuttings affecting the track. High water demand trees along the lower half of a cutting slope will be beneficial for cutting stability (similar to embankments), both in terms of root reinforcement effects (across the upper metre or two), and due to reducing wet winter pore water pressures in the influence zone of tree roots.

A factor which has led to slope stability problems across the LU network are services conduits/pipework which effectively enable concentrated water flows to be fed into a slope (either on a cutting or an embankment), leading to locally high groundwater pressures being created. These can include a very wide range of services, varying from old blocked land drains, leaking water mains, surface water drains from adjacent infrastructure (e.g. car parks), leaking track drainage, poorly backfilled trenches for other services, etc. A careful review of old maps, services information, etc., may identify these sources of water, but services records are not always comprehensive or reliable.

Based on the risk classification assigned to the earth structure, parameters can then be determined for the basal layer, as summarised in Section 3.4.4.

Figure 3.4.4: Hazard Classification Flow Chart, Deep Seated Instability

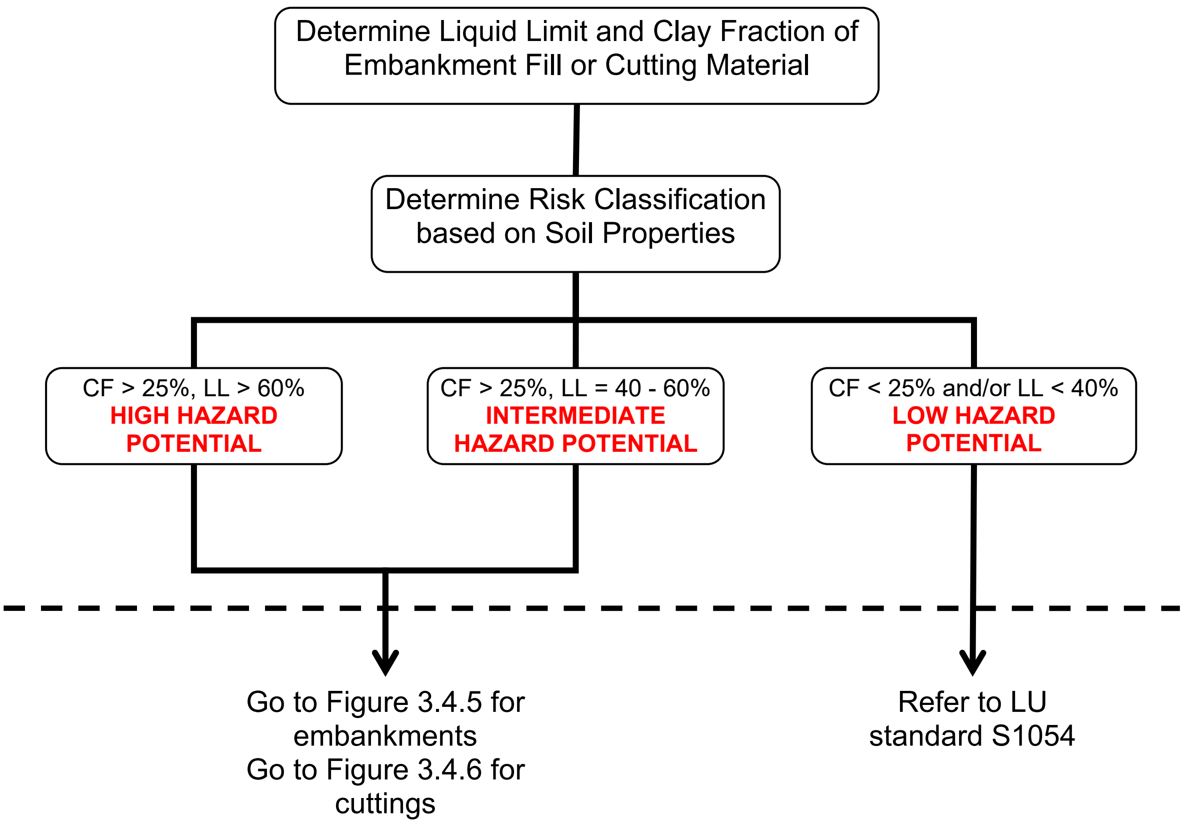
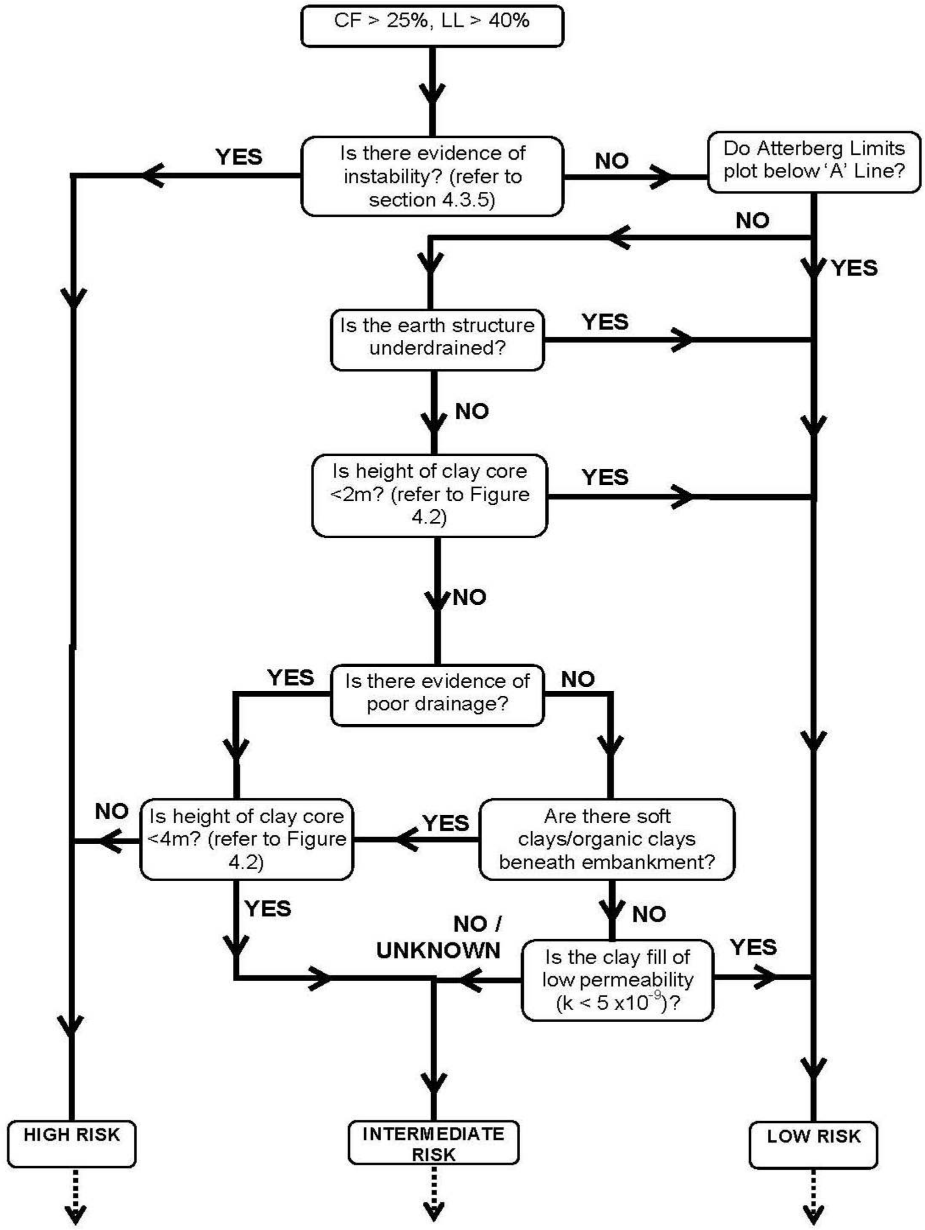


Figure 3.4.5: Risk Classification Flow Chart, Deep Seated Instability – Embankments with High and Intermediate Hazard Potential



ϕ'_{res} $\gamma_m = 1.1$	ϕ'_{int} (see Section 3.4.4.1) $\gamma_m = 1.1$	ϕ'_{cv} $\gamma_m = 1.1$
Design ϕ' and γ_m for weakened basal layer (for parameters and partial factors for other materials see Sections 3.4.4 and 3.4.5)		

Note: 1) Flow chart has been developed for assessing long term deep seated instability, the current condition may be better depending on the historic rate of deterioration (and associated site specific risk factors), refer to Figure 3.4.1 and 3.4.1.1 for embankments. Location of weakened basal layer depends upon relative stiffness of soils within and below slope, refer to Figure 3.4.2.

2) For low height embankments (clay core height <4m), alternative failure/deformation mechanisms more likely (refer to Section 3.4.3.4).

3) When assessing current earthworks condition, also consider likely impact of vegetation on earthworks performance (refer to Section 3.4.3.7).

Table 3.4.2: Embankments, Risk of Deep Seated Instability and Seasonal Deformation – Influence of Various Factors

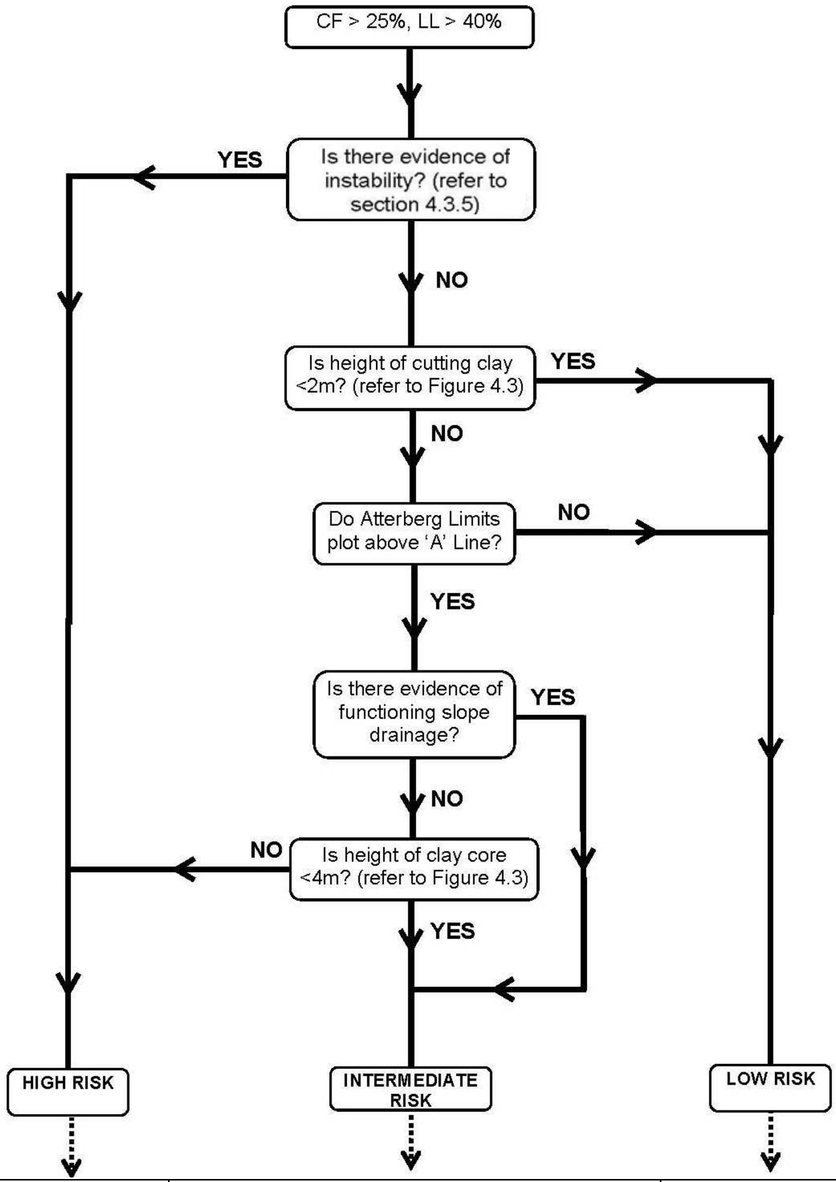
Factor	Deep seated instability	Seasonal deformation	Comments
Clay fill. Liquid limit/clay fraction (refer to 3.4.3.1)	+ + +	+ + +	Low plasticity or low clay fraction or silts (plot below 'A' line) = low risk of instability and seasonal deformation
Evidence of historic instability (refer to 3.4.3.5)	+ + +	+	Need to consider multiple sources of information, refer to text. Needs to be strong evidence. Do not confuse instability with seasonal deformation, they are different mechanisms.
Is earth structure under drained (refer to 3.4.3.3)	- - -	-	Earth structures with under drainage (e.g. by chalk or river terrace deposits) are low risk for deep seated instability; may still be prone to seasonal deformation, shoulder or shallow instability.
Height/thickness of clay core (refer to 3.4.3.4)	+ +	+ +	If height or thickness of clay core is less than 4m, then likely to be low or intermediate risk of instability. See flow chart.
Evidence of poor drainage (refer to 3.4.3.3)	+ +	+	If embankment is well drained and no soft/organic clay layers, then will be low or intermediate risk of instability. Need to consider a variety of indicators. Drainage has limited influence on seasonal deformation.
Soft or organic clay layers beneath embankment	+ +	+	The presence of soft/organic clay layers (e.g. alluvial channels, in valleys) will increase the risk of instability, but has limited influence on seasonal deformation.
Clay fill of low permeability (refer to 3.4.3.2)	- -	+	Modern compacted embankments are not prone to deep seated progressive failure (due to low permeability of clay fill). For old rail embankments can be challenging to reliably determine permeability; nevertheless low permeability areas will be less prone to instability.
High/very high water demand trees on slope	- - -	+ + +	A key factor. If slope has a dense cover of HWD trees, then likely to be stable, but (if high plasticity clay fill) will exhibit significant seasonal deformation.
Low water demand trees on slope	+	- -	A coverage of LWD trees on slope is beneficial, both in reducing risk of instability and reducing seasonal deformation.
Grass covered slope, desiccated clay at slope surface	+ + +	- - -	A key factor. If trees are absent, then high wet winter pore water pressures are more likely to develop, hence increased risk of both deep seated and shallow instability. Winter rainfall can readily infiltrate a desiccated clay exposed at the slope surface.

Notes: (1) +++ risk of instability increased

(2) - - - risk of instability decreased

(3) +++ major effect, + minor effect

Figure 3.4.6: Risk Classification Flow Chart, Deep Seated Instability – Cuttings with High and Intermediate Hazard Potential



ϕ'_{res} $\gamma_m = 1.1$	ϕ'_{int} (see Section 3.4.4.1) $\gamma_m = 1.1$	ϕ'_{cv} $\gamma_m = 1.1$
Design ϕ' and γ_m for weakened basal layer (for parameters and partial factors for other materials see Sections 3.4.4 and 3.4.5)		

Note: 1) Flow chart has been developed for assessing long term deep seated instability, the current condition may be better depending on the historic rate of deterioration (and associated site specific risk factors), refer to Figure 3.4.1 and 3.4.1.2 for cuttings. Location of weakened basal layer depends upon relative stiffness of soils within and below slope, refer to Figure 3.4.3.

2) For low height cuttings (clay core height <4m), alternative failure/deformation mechanisms more likely refer to Section 4.3.4.

3) When assessing current earthworks condition, also consider likely impact of vegetation on earthworks performance (refer to Section 3.4.3.7).

Table 3.4.3: Cuttings, Risk of Deep Seated Instability – Influence of Various Factors

Factor	Deep seated instability	Comments
Claycutting, liquid limit/clay fraction (refer to 4.3.1)	+ + +	Low plasticity or low clay fraction or silts (plot below 'A' line) = low risk of instability and seasonal deformation
Evidence of historic instability (refer to 3.4.3.5)	+ + +	Need to consider multiple sources of information, refer to text. Needs to be strong evidence
Evidence of functioning slope drainage (refer to 3.4.3.3)	- -	If cutting has effective crest and/or toe drainage, then slope will be at intermediate or low risk of instability. The maintenance plan for any reinstated earth structure must allow for all drainage to be maintained for the 120 year design life.
Height of clay, exposed in cutting (refer to 3.4.3.4)	+ +	If height of clay in cutting is less than 4m, then likely to be low or intermediate risk of deep seated progressive failure.
Evidence of local water ingress into slope	+ + +	Local water inflows, due to leaky services, or adversely located land drains from adjacent properties, can significantly increase risk, due to increase in groundwater pressures.

Notes: (1) +++ risk of instability increased

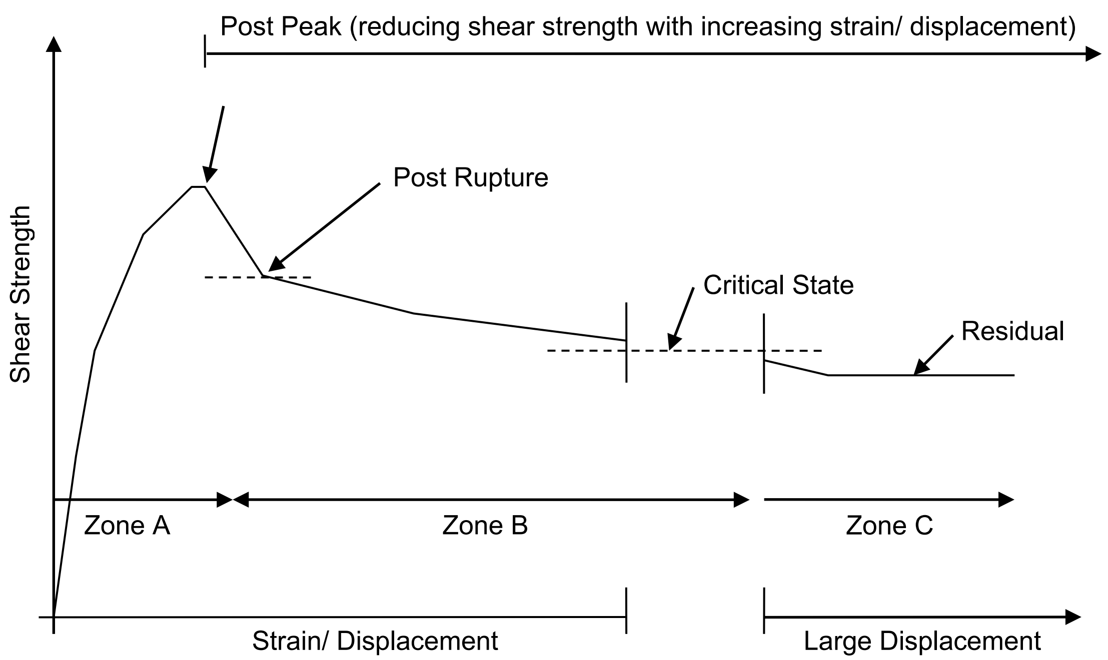
(2) - - - risk of instability decreased

(3) +++ major effect, + minor effect

3.4.4 Soil Parameters for Deep Seated Stability Analysis

Figure 3.4.7 shows the idealised stress-strain behaviour which is typically observed for a fissured, high plasticity, overconsolidated clay or clay fill.

Figure 3.4.7: Schematic Stress-Strain Response of High Plasticity Clay Fill



Under initial loading (Zone A) the sample mobilises a peak strength which decays rapidly with increasing strain (i.e. in a brittle manner) to a near constant value. The value reached after this initial rupture is referred to as the post rupture strength, as described by Burland (21). The value of peak strength is dependent on structure, stress history etc.

With increasing displacement (Zone B to C) the ″plate″ like clay particles will slowly become aligned, creating a smooth shear surface. The shear stress mobilised after large displacement is referred to as residual strength.

For conventional analyses soil strength is usually described using Mohr-Coulomb parameters, consisting of an internal angle of friction (ϕ´) and effective cohesion (c´). Whilst “real” failure envelopes tend to be curved, it is conventional to assume tangent, straight line, parameters over the range of effective stress relevant to the problem being analysed (Figure 3.4.8). Figure 3.4.9 shows the parameters defined in Figure 3.4.7 above plotted in s´-t space. Note that the principal difference between peak, post rupture and critical state strength is in the effective cohesion, as:

$$\phi'_{peak} \approx \phi'_{post\ rupture} \approx \phi'_{fully\ softened}\left(high\ stress\right) \approx \phi'_{critical\ state}$$

Note:	The φ´$_{peak}$ may be 2 or 3 degrees higher than critical state depending on stress level, hence the above assumption is conservative when selecting cautious peak strength parameters. The use of higher strength would require site-specific testing to determine the parameters. The residual friction angle will be considerably lower.

Figure 3.4.8: Idealisation of Tangent Parameters (s´-t space)

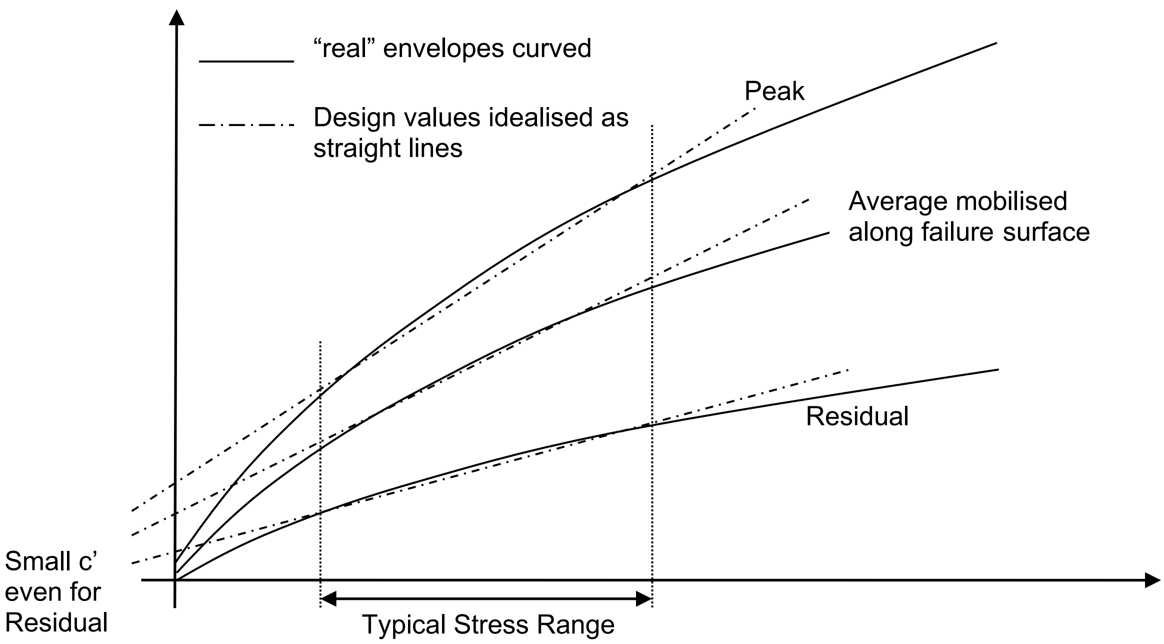
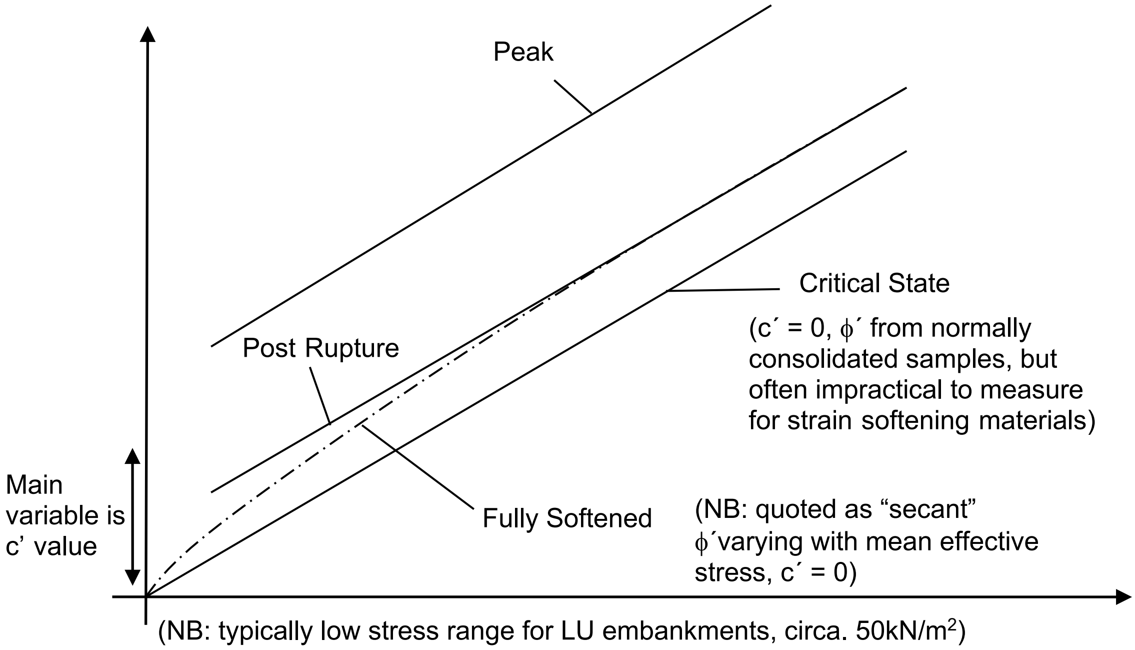


Figure 3.4.9: Comparison of Various Strength Definitions in s´-t Space



When using the following sections to derive both parameters for the basal layer and the cautious estimate of peak soil parameters, careful consideration to the index properties used to derive these strengths needs to be ensured. The progressive failure mechanism postulated is considered to be a ″worst credible″ model, and therefore a cautious estimate of properties such as the liquid limit and moisture content should be made to derive appropriate parameters for the stability analysis. It is also worth noting that the clay fill in the core of the embankment is usually wetter than that in the side slopes, and therefore depending on the location of the ground investigation holes, values recorded in testing may not be for the wettest material. O'Brien (6) reports typical values for index properties of high plasticity dumped clay fill, which can be used as a guide to values likely to be encountered. O'Brien et al (7) provide a typical cross-section of moisture content variation across an embankment, which was lightly vegetated and had soft alluvial clay beneath the embankment.

Note that the following sections regarding derivation of soil parameters can be applied to both embankment fill and the cutting material. However, due to its intrinsic structure, natural overconsolidated clays will be inherently stronger than a dumped clay fill, and therefore these parameters are conservative for the analysis of cuttings. They are considered suitable for preliminary analysis. For detailed design it may be considered appropriate by the Designer to undertake some ground investigation to justify the use of higher strengths – i.e. apply some of the principles of a Geotechnical Category 3 structure to a Category 2 structure to realise a more economical solution for remedial works (see Section 3.2).

3.4.4.1 Basal Layer

Low Hazard Potential

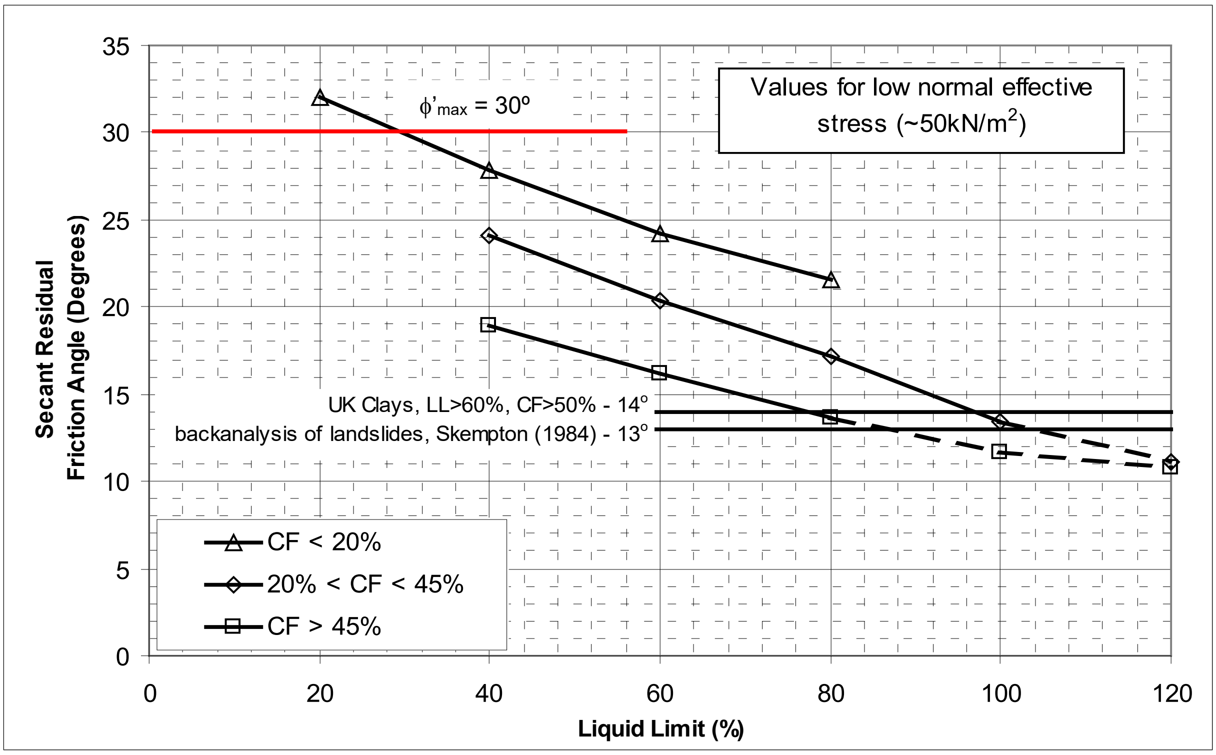
Where the hazard potential is identified as low from Figure 3.4.4 progressive failure is unlikely to be the dominant failure mechanism and as such designers should refer back to LU standard S1054 for deep-seated stability analyses. It should be noted that in these circumstances shallow translational failures or ″washout″ failures (due to high surface or groundwater flows) are invariably likely to be the more critical failure mechanisms, rather than deep seated failure. For these sites the risk of shallow translational and “washout” failures need to be considered carefully during site specific assessments, and, if necessary, risk mitigation methods designed.

Intermediate and High Hazard Potential

Where the hazard potential is intermediate or high, and the site specific risk of progressive failure is identified as High, then it is appropriate to use residual strength parameters in the basal layer. A value of residual friction angle can be obtained from Figure 3.4.10 and is dependent on both Liquid Limit and Clay Fraction. This chart is based on research by Stark and Eid (145) and is discussed in more detail in Appendix C. The residual friction angle obtained from this chart should be used in conjunction with a small cohesion of 1kPa (22).

If the site specific risk is low, then a critical state friction angle should be used (Figure 3.4.11). When the site specific risk is judged to be intermediate, then the friction angle for the basal layer should be the average of the critical state and residual friction angles, i.e. $\phi'_{int} = (\phi'_{cv} + \phi'_{res})/2$.

Figure 3.4.10: Residual Friction Angle (after Stark and Eid, (15))

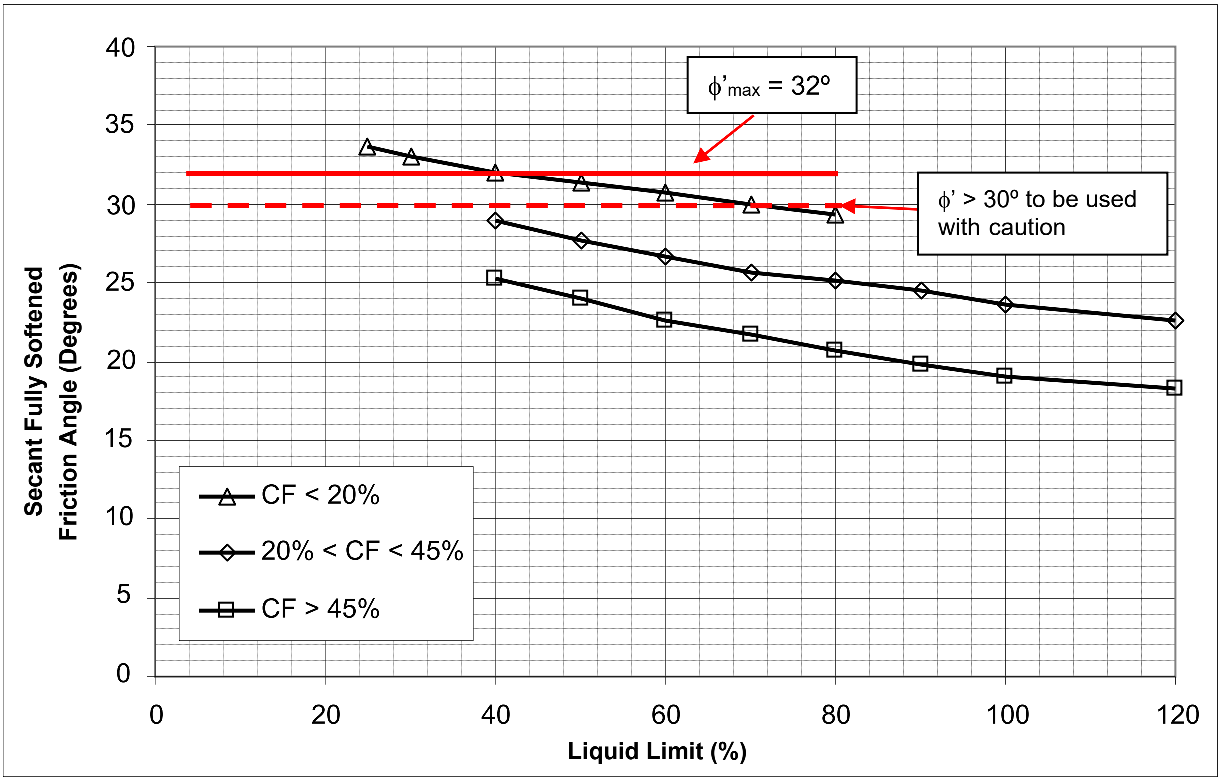


3.4.4.2 Embankment Fill (Beyond Basal Layer)

As discussed in Sections 3.4.1.1 and 3.4.1.2, it is considered appropriate to adopt cautious peak strength for the embankment fill beyond the weakened basal layer. Discussion regarding the derivation of these parameters is included in more detail in Appendix C. Below is a summary of this, including the key figures from which parameters can be derived based only on index testing.

With regards to the internal angle of friction of the fill, a conservative value for ϕ'_{peak} can be derived from the Stark and Eid (16) relationships of $\phi'_{fully\ softened}$ at high stress for appropriate values of Liquid Limit and Clay Fraction. The appropriate relationships are presented in Figure 3.4.11 below. Note that values of ϕ' greater than 30º should be used with caution.

Figure 3.4.11: Fully Softened Friction Angle (after Stark and Eid, (16))



Estimates of cohesion can be made from Figure 3.4.12 and Figure 3.4.13 below, with a judgement then made regarding the most appropriate value (refer to Appendix C for information on the derivation of these). The line designated "LU Clay Fill (MM, 1999)" is based on testing of London Clay fill and reported in the MM document "LUL Research Stage II, Assessment of Clay Fill, Doc. No: 51683/F&G/REP/100/B dated November 1999" (24).

When selecting design parameters following the approach presented herein consideration should be given to the impact of climate change and the effect on strength degradation (See Section 3.9).

3.4.4.3 Cuttings (Beyond Basal Layer)

For cuttings in natural clays a similar approach to that outlined above for embankments can be used, for those sites with an intermediate or high hazard potential (Figure 3.4.4). The strength of the weakened basal layer is assessed as discussed in 3.4.4.1 above. Beyond the weakened basal layer, it is appropriate to assume a cautious peak strength when carrying out deep seated stability analyses. Across many parts of the LU network, the cuttings will be in London Clay. The properties of London Clay are well known, and have been the subject to numerous technical papers. Failure of cuttings in London Clay have been backanalysed(13), and the derived mobilised strength properties (and, importantly, the associated assumed pore water pressures) have been widely used for slope stability assessments (and form the basis for the parameters published in the LU standards). However, it is important to emphasise that these were for cuttings in weathered London Clay, and were based on cuttings within a height range of between 5m and 7m. Experience indicates that indiscrimate use of these parameters can be overconservative for low height cuttings (less than 4m clay core height) and may be unsafe for very deep cuttings (in excess of 8m clay core height). The use of a site specific risk based approach for cutting stability assessments, as outlined above for embankments, can avoid the pitfalls associated with previous guidance.

Appendix E summarises published data for the peak strength of London Clay, measured during laboratory tests on high quality samples. For routine assessments of cutting stability, it is recommended that the empirical relationships outlined in Figure 3.4.11 are used to derive a cautious estimate of "peak" friction angle, and Figures 3.4.12 and 3.4.13 can be used to derive a cautious estimate of effective cohesion. As discussed in Appendix E, the cautious peak strength derived from Figures 3.4.11 and 3.4.12/3.4.13, will usually be lower than previously published London Clay parameters. As noted in Appendix E, the available partial factors on the proposed peak strength parameters (based on Figures 3.4.11 to 3.4.13 inclusive) exceed the requirements of EC7 when compared with data published in the technical literature. Previous experience has shown that low height cuttings are often relatively well drained, as a result natural moisture contents tend to be relatively low (although this may vary with the time of year). This will usually result in slightly higher c' values being derived from Figures 3.4.12 and 3.4.13 (although the cautionary advice in Appendix E, Section 5, should be noted, to avoid excessively high c' values being selected) for estimates of peak strength. Hence, this will result in higher factors of safety against deep seated instability for low height cuttings.

Clay cuttings across many parts of the LU network are in London Clay. However cuttings in other natural clay materials will require additional research by the Designer (i.e. technical papers, case histories, back analysis of existing and failed slopes), including specifying additional ground investigation works, to establish appropriate soil parameters. However, if the cutting is within a high plasticity fissured clay (such as the Upper Mottled Beds in the Lambeth Group) then these can experience progressive failure mechanism similar to London Clay.

When selecting design parameters following the approach presented herein consideration should be given to the impact of climate change and the effect on strength degradation (See Section 3.9).

Figure 3.4.12: Variation of Peak Effective Cohesion with Moisture Content (24)

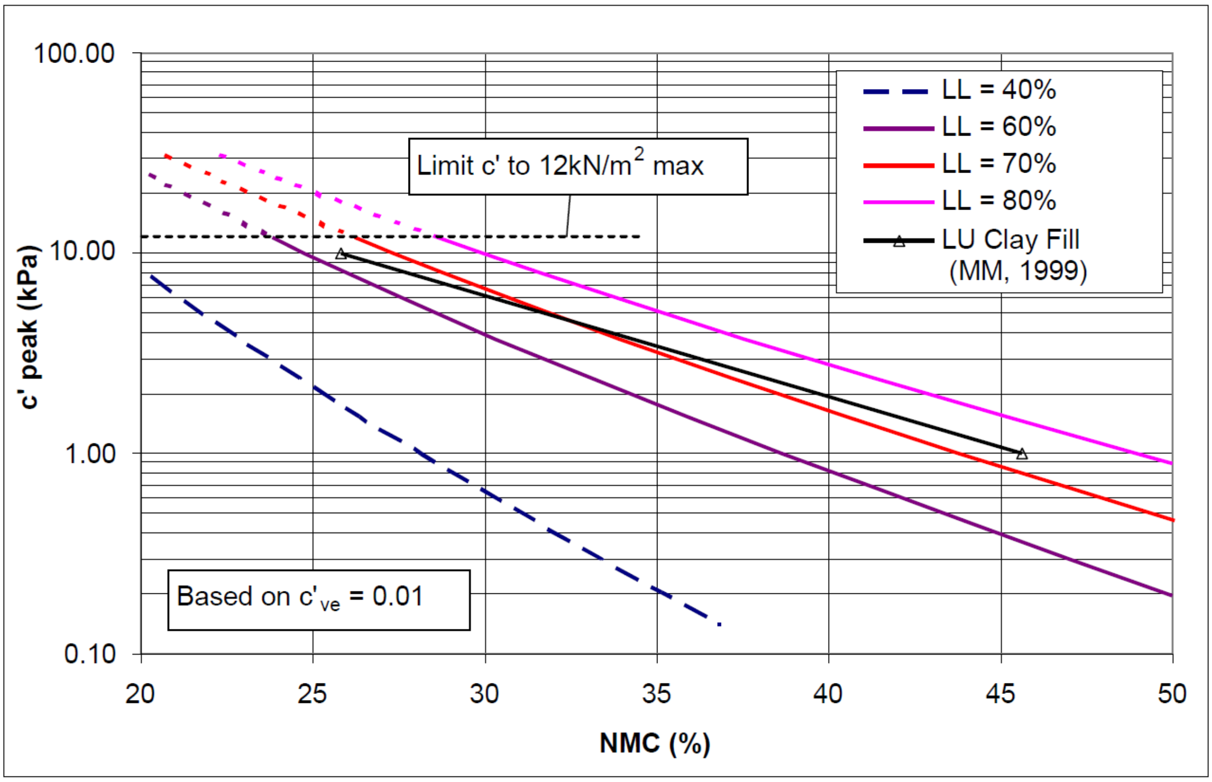
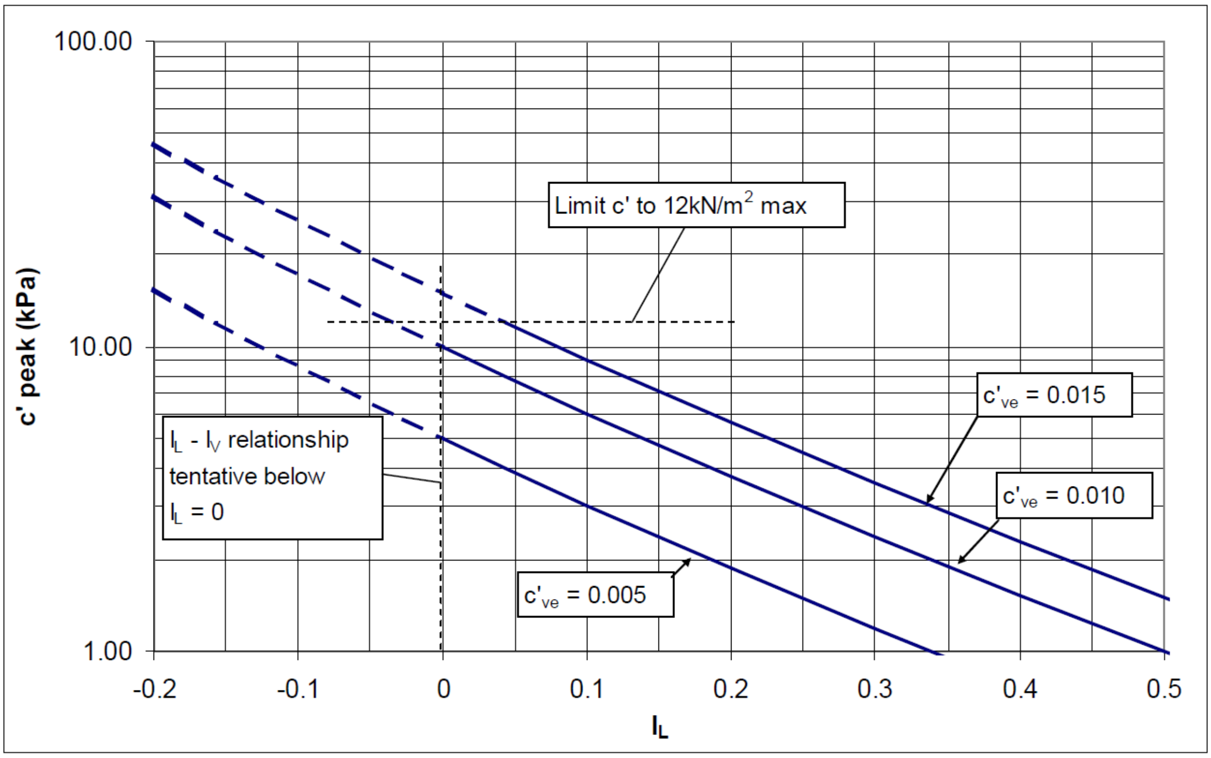


Figure 3.4.13: Variation of Peak Effective Cohesion with Liquidity Index (3)



(Refer to Appendices C and E for further details of the parameters shown in Figure 3.4.13.)

Clay cuttings across many parts of the LU network are in London Clay. However cuttings in other natural clay materials will require additional research by the Designer (i.e. technical papers, case histories, back analysis of existing and failed slopes), including specifying additional ground investigation works, to establish appropriate soil parameters. However, if the cutting is within a high plasticity fissured clay (such as the Upper Mottled Beds in the Lambeth Group) then these can experience progressive failure mechanism similar to London Clay.

3.4.4.4 In-situ Soil (Beneath Embankment)

When assessing the stability of an embankment based on the progressive failure model outlined in Section 3.4.2, the properties of the in-situ soil beneath the embankment will only be of significance when designing remedial measures. The critical slip surface will always pass through the basal layer as this is the weakest zone. However, deriving appropriate soil parameters for the in-situ material is important to enable an economical remedial solution to be developed.

Where the material beneath the clay embankment is either gravel or chalk, reference should be made to Sections 3.10 (Granular Embankments and Cuttings) and 3.11 (Chalk Fill Embankments), respectively.

In the absence of site-specific testing information, where the in-situ material beneath the embankment is clay, the following conservative parameters may be adopted:

- Where the surface of the in-situ material is close to the original ground surface i.e. at embankment toe level, the top 1m of this material should be assumed to have the same properties as the embankment fill, to allow for likely reworking by geological processes and/or historical activities by man.
- Where the in-situ material is London Clay, an angle of friction typically in the order of $\phi' = 20\text{-}22^{\circ}$ should be used in conjunction with a cohesion based on Figure 3.4.12 and Figure 3.4.13. It should be noted that values of c´ from these figures will be conservative for natural clay as they are based on reconstituted soil. Typically c´ = 3-6kPa for Weathered London Clay, and 6-12kPa for Unweathered London Clay.
- Figures 3.4.12 and 3.4.13 can also be used to derive c’ values for other fine grained deposits underlying the embankment.

These parameters are intended as a point of reference when no other information is available. However, designers should be aware that much of the published technical data on in-situ London Clay parameters may not be appropriate for use in slope stability analyses given that the majority of the data is for unweathered clay (whereas the near surface in-situ clay on the LU network may often have some degree of weathering) and the published parameters are often discussed in the context of deep excavations and retaining wall design.

The parameters proposed above are for undisturbed in-situ material. Where the in-situ clay has been disturbed by man-made or geological processes the strength parameters may be much lower. These include relic shear surfaces and clays subject to periglacial freeze thaw processes when strengths in weakened zones may be close to residual (and certainly no higher than critical state values).

3.4.4.5 Unweathered London Clay (Beneath Cutting)

The Unweathered London Clay beneath Weathered London Clay, will be stronger, and soil parameters should be chosen to reflect this. As above, Figure 3.4.12 and Figure 3.4.13 can be used to derive appropriate values of c´.

3.4.5 Compliance with EC7 and LU Standards

The updated LU standard 'Civil Engineering – Earth structures, S1054 states that the design of earth structure assets, including strengthening and renewal of these assets, shall be carried out in accordance with BS EN 1997-1 (Eurocode 7 Geotechnical Design Rules) and the corresponding National Annex. For many design situations, overall stability should be checked against Design Approach 1 Combination 2. Following this methodology requires partial factors to be applied to characteristic soil properties to determine design values, and a partial factor to be applied to live loading. Eurocode 7 Combination 1 needs to be checked only if the designer considers that the loading applied to the slope (other than the mass of ground in the slope) might control the failure mechanism rather than the ground strength parameters.

However, there is an alternative Eurocode 7 compliant method to derive design values of soil properties that can be used. As discussed in Section 3.1.2 Eurocode 7 does state that design values of geotechnical parameters can be ″assessed directly″ (Cl. 2.4.6.2 (1)P), rather than from a characteristic value with the appropriate partial factor applied. The relevant clauses are repeated below:

2.4.6.2 (1)P Design values of geotechnical parameters (X_d) shall either be derived from characteristic values using the following equation:

$$X_d = X_k / \gamma_M$$

or shall be assessed directly.

2.4.6.2 (3) If design values of geotechnical parameters are assessed directly, the values of the partial factors recommended in Annex A should be used as a guide to the required level of safety.

Eurocode 7 also highlights the importance of the nature of the failure mechanism being assessed when deriving soil parameters:

2.4.5.2 (2)P The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state.

Based on the risk classification, the parameters for the basal layer can be derived, along with appropriate material partial factors. Appropriate friction angles and partial factors are indicated on the flow charts in Section 3.4.3.8. These are summarised in Table 3.4.4, along with the corresponding values of c´ to be adopted in the basal layer.

Table 3.4.4: Strength Parameters for Weakened Basal Layer

Risk Level	Strength Parameters in Weakened Basal Layer			Partial Factor
High Risk	Residual:	c´ = 1kPa	ϕ'_{res} from Figure 3.4.10	1.1
Intermediate Risk – CF >25% and LL >40%	Average of Critical State and Residual:	c´ = 1kPa	ϕ'_{int} from Figure 3.4.10 and 3.4.11	1.1
Low Risk – CF >25% and LL >40%	Critical State/ Fully Softened:ma	c´ = 1kPa	ϕ'_{cv} from Figure 3.4.11	1.1
Low Risk – CF <25% and/or LL <40%	Progressive failure unlikely to be governing mechanism. Refer to LU standard S1054. Shallow failure likely to be dominant mechanism (see Section 3.5).			

The method outlined in the previous sections for considering progressive failure of clay fill embankments and clay cuttings is considered to represent a ″worst credible″ failure mechanism. Therefore, application of the partial factors on characteristic soil parameters as per EC7 and LU standard S1054 would be unduly onerous. It is therefore proposed that in general a partial factor of 1.1 be applied to these directly assessed parameters in order to derive "design values for the assessment of deep-seated instability – this is to be applied to all soil layers within the model.

The partial factors recommended in this Design Guide are for use in slope stability analyses only. All subsequent remedial works design (e.g. discrete bored piles, retaining walls etc) should be designed based on the partial factors given in EC7 and characteristic soil and groundwater parameters. The proposed new methodology for deep seated stability analysis should result in improved prioritisation of works across the network as well as reducing the amount of deep seated remediation works carried out where deep seated failure is not a significant issue.

Eurocode 7 requires all relevant deformation/failure mechanisms to be checked, e.g. refer to Table 3.1.1. The proposed conceptual model outlined in this Chapter deals solely with deep seated instability, i.e. mechanisms 2 and 4 in Table 3.1.1. Other Chapters in this report discuss other failure/deformation mechanisms. The risk based approach for deep seated instability leads to worst credible soil strength and groundwater (refer to Chapter 7.3) parameters being derived. For example, for high risk scenarios, residual soil strengths will be derived for the weakened basal layer, and hydrostatic pore water pressures. Because worst credible soil and groundwater parameters are selected, then low partial factors are appropriate (1.1 for the proposed conceptual model). This approach is consistent with historical UK approaches when assessing the stability of slopes affected by landslips (and other similar "worst credible" applications, such as BS8002); and it facilitates cost-effective remedial works design. Because parameter selection is intrinsically cautious, the actual factor of safety will be higher than the nominal value of 1.1 on strength parameters. In particular, the groundwater pressures are based on extremely wet winter conditions. Noting the comments above, regarding Clause 2.4.6 of EC7, the proposed approach is compliant with EC7 requirements.

3.5 Shallow Instability of Clay Cuttings and Embankments

This can occur as a translational or planar type slip, with a failure surface close to parallel with the slope surface over much of its length. These slips are typically 1-1.5m deep. The shallow stability design methodology described in this section is intended for these translational type failures and should not be misinterpreted as the shallow failure criteria defined in LU standard S1054 (which allows for a slip depth of up to 1.5m through the sloping portion of the earth structure not affecting the track or lineside services).

A desiccated clay slope surface subjected to winter rainfall causing cracks to fill with water would be a common trigger for such a failure. There is a higher risk of shallower instability if the slope is bare or grass covered i.e. not heavily vegetated. A granular surface layer (which acts as a capillary break) can significantly reduce the risk of shallow instability.

Another form of shallow instability is downward creep or ratcheting deformation. This can occur over time, and is evident downslope of some bored pile walls installed as part of a remedial solution on some LU earth structures.

The guidance in this section applies to both cuttings and embankments.

3.5.1 Role of Vegetation

It is well documented that vegetation on a slope can act to increase the stability of the earth structure in terms of shallow failure. The roots of the vegetation can serve to reinforce slopes by increasing the shearing resistance of the slope.

The magnitude of the increase in shear strength of the soil due to the presence of roots depends on several factors: strength and stiffness of the roots, root architecture (orientation, distribution etc) and the characteristics of the surrounding soil. For the purposes of shallow slope stability problems, where normal stresses will be low, the simplified approach using an additional cohesion, cr, can be adopted. This is discussed further in Section 3.5.2.1.

Quantification of cr is not straightforward, and is dependent on both plant species and root diameter and density. Once a value is determined then the benefit of this additional strength can be considered in stability analysis (see Section 3.5.2.1). This can be simply done using the infinite slope equation as shown in Figure 3.5.1 below.

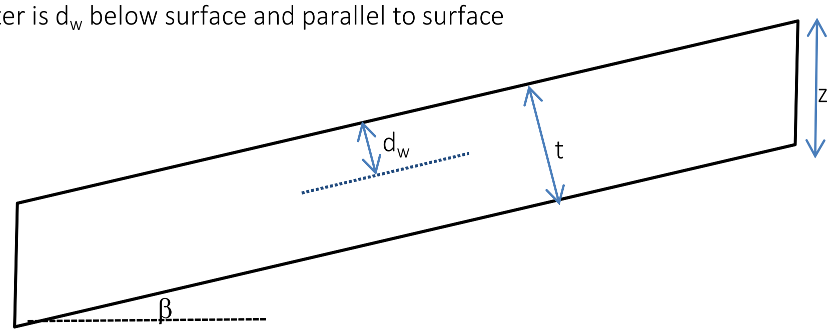
However, inclusion of a beneficial effect from vegetation on the slope when assessing shallow slope stability needs careful consideration of the following issues:

- Type of vegetation e.g. grass covered slopes will provide little or no benefit due to the presence of roots
- Tree species and appropriate value of cr
- Allowance of cr for long term design has implications with regards to maintenance of the vegetation on an earth structure over the design life
- Clearance of vegetation leads to a rapid reduction in cr (even if roots are left in place), and this will take a number of years to redevelop following replanting.

The following section outlines a proposed methodology to be followed in design.

Figure 3.5.1: Infinite Slope Model including Root Cohesion

Consider infinite slope at gradient β.
Unstable surface layer of thickness t, and depth z.
Water is d_w below surface and parallel to surface



For shear strength of soil c' and ϕ', then:

$$F = \frac{c' + [\gamma t\cos\beta - \gamma_w(t - d_w)\cos\beta]\tan\phi'}{\gamma t\sin\beta}$$

where γ and γ_w are the unit weights of the soil and water respectively

If soil is reinforced by roots then can consider additional cohesion effect, cr, such that:

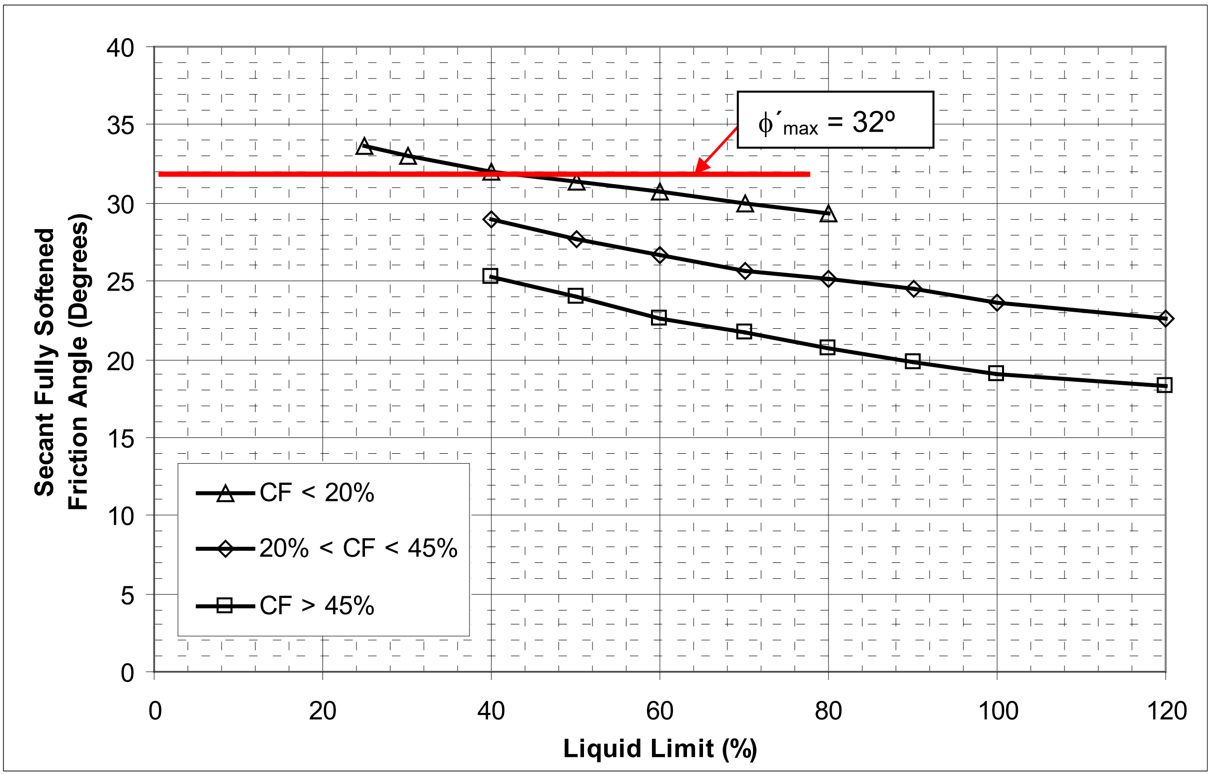
$$F = \frac{cr + c' + [\gamma t\cos\beta - \gamma_w(t - d_w)\cos\beta]\tan\phi'}{\gamma t\sin\beta}$$

3.5.2 Design Methodology

When assessing shallow slope stability it is considered appropriate to adopt critical state parameters in the analysis. This mechanism of failure should be considered separately to that of the deep-seated progressive failure, and can be simply assessed using the infinite slope model outlined in the previous section or by using appropriate analysis software.

For cohesive soils, it is considered that an estimate of the critical state angle of friction can be made using the relationships in Stark and Eid (16) for ϕ'fully softened at high stress levels (400kPa). This is the same relationship as is used to derive a cautious ϕ'peak in Section 3.4.4.2 and is repeated in Figure 3.5.2 below. Note that values of ϕ' greater than 30° should be used with caution. A cohesion of 1kPa for the soil should be used in conjunction with this value.

Figure 3.5.2: Fully Softened Friction Angle (after Stark and Eid, (16))

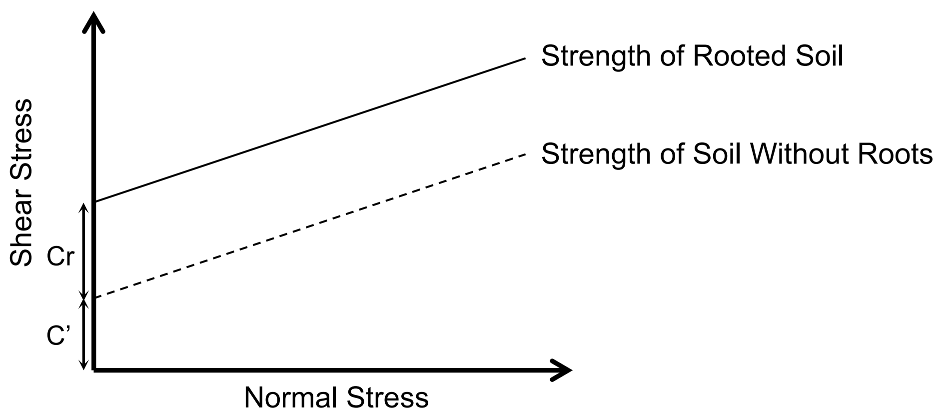


For granular or Chalk fills, reference should be made to Sections 3.9 and 3.10 respectively.

3.5.2.1 Assessing Root Cohesion

As outlined previously, the shear strength of a rooted soil mass is enhanced due to the presence of a root matrix. For the purpose of shallow stability problems the simplified approach as shown in Figure 3.5.3 will be used to assess the increase in strength due to the roots.

Figure 3.5.3: Simple Approach for Modifying Shear Strength in Rooted Soil (after Coppin and Richards, 2007 (26))



Attempts to quantify the root cohesion, c_r have been made by a number of researchers using models of varying complexity. The most common and simplest approach is to assume a perpendicular root model where all roots cross the shear plane at 90 degrees, and the contribution to shear strength arises from the tension developed in the individual roots as they are extended when the soil shears. However, this model may overestimate the contribution of the roots to the shear strength.

Published values of root cohesion, c_r, determined either from analytical models or from testing of the roots themselves typically vary between 1 and 20kPa. The root cohesion is known to be a function of:

- Root tensile strength (varies with tree species)
- Root area ratio i.e. area of roots crossing the shear plane divided by total area of shear plane
- Root diameter
- Depth beneath slope surface – decreases with depth

It is also important to note that the removal of vegetation will lead to a drop in root cohesion with time. Even if this vegetation is replanted, it may take several years for full beneficial impact of the root cohesion to be realised.

Root cohesion, $c_r = \alpha\ \sigma_t\ A_r$ Equation 3.5.1 (45)

α = empirical factor allowing for root orientation and root-soil interface strength, typically between 0.4 and 0.6

σ_t = root tensile strength, between 5MN/m^2 and 32MN/m^2, for example, for Poplar, Oak and Birch

A_r = root area ratio, depends on root diameter and density, for vegetated slopes may vary between 0.1% and 1.0%

Based on equ'n 3.5.1 and the potential range in values for α, σ_t and A_r quoted in the literature. c_r may vary across a wide range from about 2kN/m^2 to > 100kN/m^2. Back analysis of landslides (46) indicate c_r values of up to about 30kN/m^2 for tree covered slopes, and increases in Factor of Safety of up to 25% (47). Hence, root cohesion effects can be highly significant. Although a cautious approach is normally adopted to assessing root cohesion effects for design (in terms of future slope stability), e.g. Table 3.5.1, the potential for much higher root cohesion effects should be taken into account when assessing the current condition of vegetated slopes.

It is also important to note that the removal of vegetation will lead to a drop in root cohesion with time. Even if this vegetation is replanted, it may take several years for the full beneficial impact of the root cohesion to be realised.

Table 3.5.1 summarises cautious values of root cohesion which may be used for design purposes when checking shallow stability. However, as noted above, root cohesion can be five to six times higher. It is important to allow for potentially higher root cohesion values when carrying out back analysis of existing slopes. The following table summarises proposed values of root cohesion to be used in the assessment of shallow stability.

Table 3.5.1: Values of Root Cohesion to be used in Design

Type of Vegetation	Root Cohesion, c_r (kPa)	Comment
Lightly vegetated grass covered slope	0	Depth of any beneficial impact will be very shallow and so should not be considered in stability assessment
Moderately vegetated slope, including shrubs and some trees	1 - 2	Higher values of root cohesion may be feasible, however would need to be justified based on an assessment of tree species, root diameter and density etc.
Densely vegetated slope, including significant number of trees	2 - 5	Higher values of root cohesion may be feasible, however would need to be justified based on an assessment of tree species, root diameter and density etc.

Note: Actual c_r may be significantly higher, and should be considered carefully if back analysis is attempted.

The use of the above table requires knowledge of the vegetation cover across an earth structure. The depth over which the root cohesion acts will depend on vegetation type and density (3 & 26) although as discussed in Section 3.4.3.7 the effects of root reinforcement will typically be restricted to the top 2m of material. It may be appropriate to apply different values of root cohesion in different zones along the earth structure. This will require input from an arboriculturist or equivalent, who is able to identify the type of vegetation along the earth structure. One approach is to undertake a vegetation survey plan, zoning the earth structure dependent on both density and type of vegetation. Such a plan would also assist with identifying the locations of HWD trees, and therefore the appropriate groundwater regime to be used in stability analysis.

If a value of root cohesion greater than zero is to be used in design, to demonstrate that shallow stability is achieved, then this needs to be conveyed as part of the design solution. It will be necessary to ensure that the maintenance plan for the earth structure allows for the vegetation cover assumed in design (or condition assessment) to be maintained for the 120 year design life for a remedial works scheme, or for an agreed period (linked to future condition assessment) approved by the LU Head of Earth Structures. If the vegetation cover changes from the assessment assumption, the possibility of shallow instability would need to be reassessed.

Where remedial works are required for deep seated instability, it will be necessary to assume a root cohesion equal to zero, as vegetation is likely to be removed during implementation of the remedial solution. Even if vegetation is not removed, significant beneficial effects of vegetation should not normally be relied upon for a 120 year design life where, in the absence of root cohesion, deep seated instability would result. In these instances the inclusion of a granular capping layer as part of the remedial solution will probably need to be considered. This assists in preventing shallow slips in that it lowers groundwater levels (when combined with effective site drainage) and reduces the potential for long-term creep of the slope.

3.5.2.2 Groundwater

As previously discussed, the surface of a clay slope is usually desiccated – this leads to a near-surface zone with a higher permeability, even in the presence of HWD trees. A large number of factors can affect near surface groundwater pressures, including: intrinsic permeability of soil layers; climate conditions (duration and intensity of rainfall); exposure to sunlight (drying and desiccation effects); clay fraction and mineralogy (vulnerability to shrinkage and cracking of surface); slope topography (run-off of rainfall, risk of ponding water), etc. Hence, there is a need for site specific judgement. Table 3.5.2 provides a summary of the risk factors which should be considered in the assessment.

For the purposes of assessing shallow stability of clay slopes, the groundwater condition should be selected on an overall appraisal of the risk factors for the site under consideration. A high risk site should use the upper bound condition indicated in Table 3.5.3, whereas a low risk site should use the lower bound condition. Groundwater conditions for intermediate sites can be selected by interpolation between the upper/lower bounds. There is a need for experienced professional judgement; site specific factors may have a dominant influence, e.g. a leaking water main located on a slope, or effective drainage at the slope toe or via counterfort drains on the slope.

Table 3.5.2: Risk Factors for Shallow Groundwater Conditions

Factor	High Risk Factors	Low Risk Factors
Slope Topography	Irregular, hummocky, profile – higher risk of water ponding.	Smooth, regular profile – rainfall run-off improved.
Topography beyond the LU boundary	Ground sloping towards the LU boundary, potentially leading to increased water run-off towards the asset.	Level topography, or ground sloping away from the LU asset.
Clay exposed on slope surface	High plasticity clay is vulnerable to desiccation cracking during hot summers; slope can then readily become saturated during wet winters. Lack of vegetation coverage potentially increases near surface infiltration of water into slope. However vegetation, on the slope surface, may affect the seasonal movements of track (depending on the water demand of vegetation).	Low plasticity clay or silts, less likely to be subject to desiccation cracking. Surface capping (such as compacted HA Class 1A) will potentially improve surface water run-off and prevent infiltration into the clay core; and limit surface shrinkage/swell by providing a capillary break.
Ash/Ballast on slope surface (embankment)	"Dirty" ash/ballast with high fines content will readily saturate, but unlikely to be free draining. If ash/ballast in irregular "pockets" then water can become trapped, i.e. perched water.	"Clean" ash/ballast with low fines content likely to be free draining, especially if a continuous layer on a regular slope. Ash/ballast will protect underlying clay from desiccation cracking, even if high fines content.
Slope Orientation	North facing slopes tend to be wetter.	South facing slopes tend to be drier.
Crest Drainage	Crest/track drainage (embankment) in poor condition, and likely to be leaking. Crest (cuttings) drainage in poor condition, and likely to be leaking.	Crest/track (embankment) drainage in good conditions, will potentially reduce trapped water within from track ballast. Cutting drainage at crest limiting surface water run-off down slope.
Toe Drainage	Toe/track drainage in poor condition, and likely to be leaking. Ballast/clay (cuttings) interface topography is adverse, ("bath-tub" effect) likely to be a water source in wet weather. Toe drainage (embankments) may be inadequate leading to ponding.	Toe/track (cuttings) drainage in good conditions, will limit potentially trapped water from within the track ballast.
Mid Slope Drainage (cuttings)	Are counterfort drains, if present, functioning? Counterfort drainage may be blocked (ingress of fines and silt), or ongoing drainage may be damaged.	Functioning counterfort drains, improving near surface drainage.

Note: For guidance, site specific factors beyond those outlined may have a dominant influence. Underlying stratigraphy relates to deep instability and is not considered relevant for shallow instability.

Table 3.5.3: Groundwater Conditions for Shallow Stability Checks

Risk Classification	Groundwater Conditions
High	60% Hydrostatic from slope surface, or 100% hydrostatic below clay surface, whichever is greater.
Low	30% Hydrostatic from slope surface, or 50% hydrostatic below clay surface, whichever is greater.

Note: (1) Refer to Table 3.5.2 for risk factors to be considered.
(2) Groundwater conditions to be assumed for shallow slope stability check over upper 1.5m of slope surface.
(3) Interpolate groundwater condition for intermediate risk sites.

If the slope surface is remediated, with benching of the clay and placement of a capillary break capping layer (Class 1A well graded granular fill), then porewater pressures may be assumed to be 50% hydrostatic from the clay surface (provided the overall slope topography prevents any concentration of surface water or rainfall infiltration, maximises rainfall run-off, and is protected by provision of low water demand vegetation and grass cover).

Consideration needs to be given to the ability of slope drainage to function and to be maintained adequately over a 120 year design life.

When assessing shallow stability of granular or Chalk fill slopes, appropriate groundwater regimes should be derived following the guidance in Sections 3.9 and 3.10 respectively.

3.5.2.3 Method of Analysis

As outlined in Section 3.5.1, a simple approach to assessing shallow stability is to use the infinite slope equation, taking into account the root cohesion where appropriate. However, for some slopes this will not be appropriate, particularly those which are less than 6m in height. This method is therefore considered to be most appropriate for initial analysis, when a quick assessment of shallow stability is required.

For detailed design it is recommended that shallow stability be checked using a software program such as Slope/W, with the groundwater modelled as outlined in the previous section, and root cohesion allowed for where appropriate. Slip depths should typically be limited to approximately 1.5m based on the slip depths recorded on embankment and cutting failures in cohesive and granular materials across the UK motorway network (27). Both circular and non-circular analysis should be undertaken to ensure the critical failure mechanism is found.

3.5.2.4 Summary

Cohesive slopes on embankments and cuttings should be assessed for shallow stability as follows:

- Determine appropriate use of root cohesion. For long-term design this should be avoided and should be used for assessment/back-analysis purposes only. If design relies on vegetation consider applicability of this following renewal works and/or over the 120 year design life.

- Derive critical state soil parameters.
- Apply hydrostatic groundwater from surface of cohesive material (allow for influence of slope drainage if present and functioning).
- Circular and non-circular slips to be assessed over top 1.5m of slope.
- Appropriate partial factors are summarised in Section 3.5.3.

3.5.3 Compliance with EC7 and LU Standards

When considering slips downslope of the track/lineside services i.e. shallow slips, as defined by LU, the LU standard S1054 states that “a Partial Factor of 1.15 on the Angle of Shearing Resistance and the Effective Soil Cohesion parameters shall be adopted“. This is reduced from the value of 1.25 stipulated in EC7.

As stated in Section 3.4.5, EC7 allows parameters to be directly assessed and appropriate levels of safety to be determined.

It is considered that for embankments, the partial factor of 1.15 in LU standard S1054 is appropriate to be used in conjunction with the model above.

When assessing shallow slips for cuttings e.g. slips downslope of a proposed bored pile remedial solution, although these slips may not be daylighting at the track or lineside services, the consequences of such a slip need to be carefully considered by the designer. These slips inherently pose more of a risk to the operation and safety of the LU network than those downslope of lineside services on an embankment. Therefore, dependent on the geometry of the cutting, and the distance from the toe to the track, the Designer may consider that it is appropriate to apply a higher partial factor of safety of 1.25 in the analysis to reflect this increased risk. This is consistent with EC7 and LU standard S1054, which states “The designer should ensure the risk and consequence of failure have been adequately considered during the design. BS EN 1990 permits the variation of the relevant Partial Factors where the consequence of failure is either higher or lower than normal“.

Consideration of the appropriate partial factor for use in design may also be required where lineside services are located at the toe of an embankment, or cross the slope. This will require an understanding of the services present to assess the consequence of failure, as well as discussions with LU.

3.6 Application of Train Loading (Embankments)

The failure mechanism outlined in Section 3.4 is that of progressive failure, i.e. failure caused by time dependent (over decades) degradation of strength. Train loading is a transient load, and therefore is not compatible with such an approach.

However, it is recognised that the application of train loading to stability analyses is required under the LU standard, and that it has been applied historically to all stability analyses. Clause 3.5.7.12 in LU standard ″Civil Engineering – Earth Structures, No.: S1054, Issue no.: A5″ states the following:

″Variable (Live) and permanent Actions (loadings) shall be included in the required effective stress analysis to establish the long term Assessment of stability″.

The standard goes on to say:

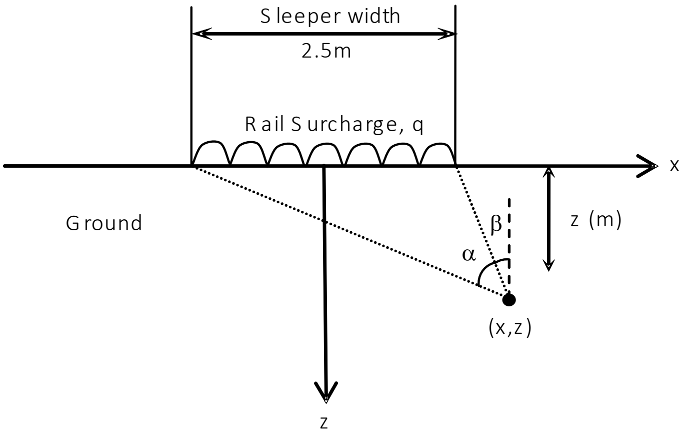
″Although there may be an apparent logic for using undrained soil parameters in conjunction with Live Loading, it is considered that using drained parameters will not be unduly conservative in the long term. Moreover, the investigation techniques suitable for use along the railway are not suited to producing consistent undrained strength results, and determining how these will be affected by cyclic loads in the long term. This stipulation applies both to Earth Structure slopes and to loadings on earth Retaining Structures formed within slopes″.

The loading to be applied to embankments, as defined in the Standard is either 30kPa or 50kPa across the width of the sleeper, dependent on the location of the embankment on the LU network.

Analysis has indicated that for many embankment geometries the inclusion of train loading results in a disproportionate change in the stabilising force required to obtain the target factor of safety compared with the change in the existing factor of safety. This adverse effect is particularly severe for relatively low height earth structures, which are normally relatively stable. Therefore a review of the application of train loading has been undertaken, given its relevance to the design of renewal works, and the costs associated with these.

It is important to understand how surface surcharges are applied to stability analysis within software such as SLOPE/W. Within any soil beneath a surcharge, the increase in vertical stress will be a fraction of that applied at the surface, dependent on depth beneath the surface and horizontal offset from the centre of the surcharge. It is straightforward to assess this increase in vertical stress assuming a simple Boussinesq solution.

Figure 3.6.1: Stresses due to Uniform Strip Loading



The increase in vertical stress at a point due to the uniform loading shown in Figure 3.6.1 can be calculated using the Boussinesq equation for distributed loading on an infinite strip:

$$\sigma_z = q/\pi[\alpha + \sin\alpha\cos(\alpha + 2\beta)]$$

Where: σ_z is the increase in vertical stress;

q is the vertical load in kN/m^2; and

α, β are defined in Figure 3.6.1

However, within software such as SLOPE/W, the full surface surcharge is applied at the slip surface independent of slip surface depth. Some preliminary analysis has been undertaken using the Boussinesq solution for strip loading to determine the increase in vertical stress beneath the train loading for typical slip depths. The following table indicates the increase in vertical stress as a proportion of the applied train loading, q for typical slip depths (depth indicated in the table is that beneath the centre of the train loading).

Table 3.6.1: Vertical Stress Increase due to Train Loading

Depth of Slip (m)	Average Increase in Vertical Stress Beneath Surcharge
1.0	0.75q
1.5	0.63q
2.0	0.56q

In addition to how the train loading is applied, the application of partial factors to this load needs to be reviewed. It is clearly stated in the LU standard S1054, that this load should be treated as a uniform variable unfavourable action, and as such in a DA1-2 analysis a partial factor of 1.3 should be applied.

It is considered that additional investigation should be undertaken to review the application of 'depth factors' to the train loading.

It is proposed that a modified approach to the application of train loading in stability analysis would be appropriate with the use of a depth factor:

$q_{applied} = q_{rail} \times DF \times \gamma_f$ where:

q_{rail} = 30 or 50kPa as per LU standard

DF = depth factor

γ_f = partial factor of 1.3 as per LU standard S1054

As can be seen in Table 3.6.1, the increase in vertical stress at the slip surface will decrease with increasing slip depth. Therefore, a conservative value of the depth factor, DF, would be 0.75, as for the majority of slips this factor will actually be lower, and will rarely be higher. Note that the depth factor should only be applied when assessing deep seated instability. For shallow instability and for cases where existing/proposed structures are affected by the rail loading no depth factor should be applied (i.e. DF=1). It is recommended that designers consider slope stability analyses with and without the depth factor proposed applied to the surcharge loading in order to appreciate the effect that the surcharge is having on the slope stability, and, in particular, the magnitude of the stabilising force which needs to be applied to achieve a target Factor of Safety.

Work undertaken as part of the RSSB research for NR (28) considered impacts of rail loading due to heavy freight trains with axle loads of 22.5 tonnes. This was considered to have an equivalent surcharge over the length of the sleeper of 32kPa. Passenger trains on the NR network have typical axle loads of up to 15 tonnes. Trains using the LU network are lighter than both these heavy freight trains, and NR passenger trains. It is recognised that the design loading of either 30 or 50kPa allows for the Ballast trains which run on parts of the network and Engineering trains, which are heavier than the LU passenger trains. However, based on the RSSB work, values of greater than 30kPa seem excessive, and must include additional factors to take into account surcharge dynamic loading etc, which is normally only relevant for bridge assessments.

3.7 Deep-seated Instability – Clay Fill Embankments

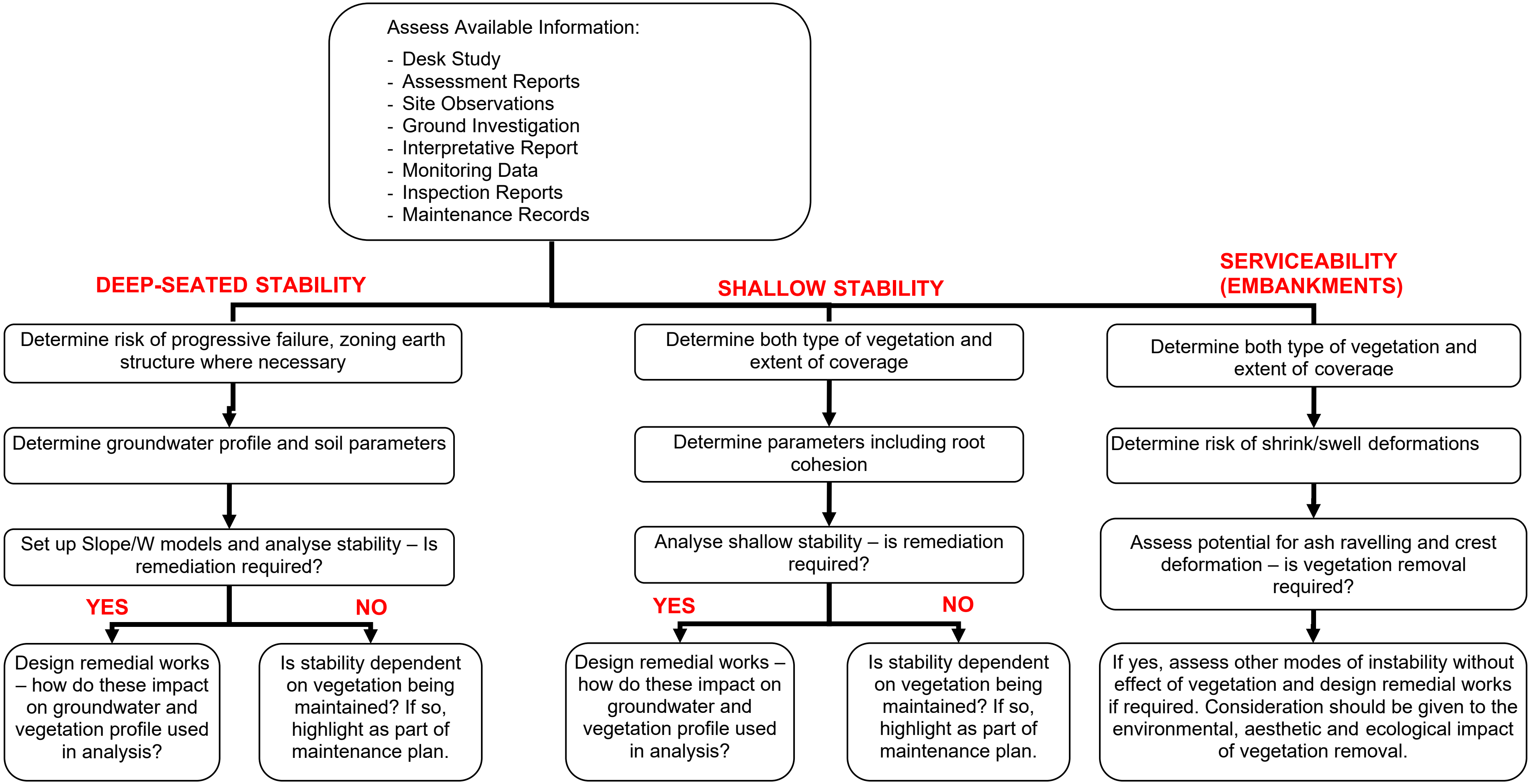
This section outlines the proposed method for deep-seated stability analysis for embankments constructed from clay fill, including giving guidance on the assessment of appropriate groundwater regimes for typical LU embankments. For the impact of climate change on the stability of the clay embankments please refer to Section 3.9

3.7.1 Method of Analysis

Where the embankments are formed of clay fill, it is necessary to assess the earth structure in terms of both deep and shallow failure, as outlined in Sections 3.4 and 3.5. Due to the methodologies proposed in these sections for determining appropriate soil parameters, it will be necessary to consider these two mechanisms in separate analytical models in parallel with each other. Figure 3.7.1 summarises the process to be followed.

In addition to the derivation of soil parameters given in Section 3.4.4, Section 3.7.3 outlines appropriate groundwater regimes to be applied in analysis.

Figure 3.7.1: Earth Structure Design Methodology



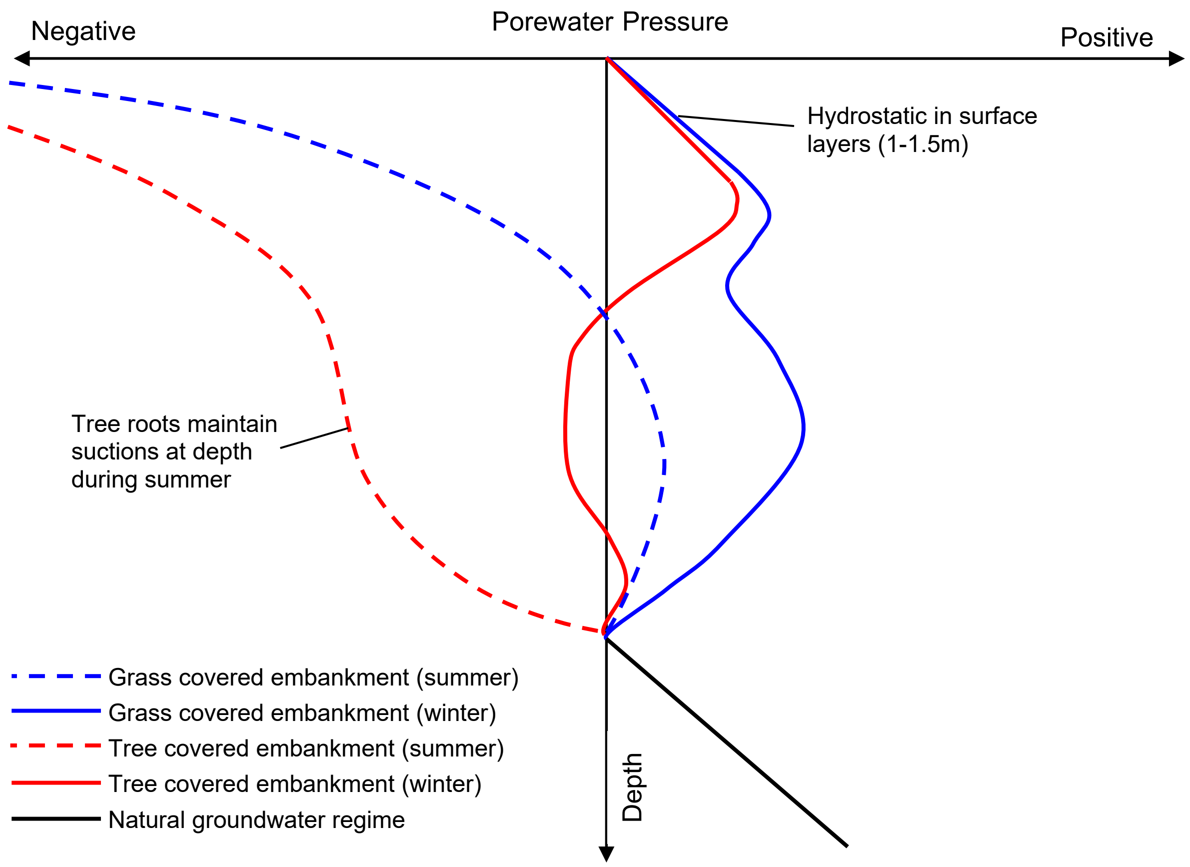
3.7.2 Soil Parameters

Material parameters and partial factors should be based on the worst credible progressive failure mechanism described in Section 3.4.

3.7.3 Groundwater

The most critical input parameter into slope stability analysis is the assumed porewater pressures. However, these can be difficult to assess, particularly in cohesive fill embankments. This is partly because the groundwater regime is constantly varying throughout the year (hence the seasonal shrink/swell cycles generally observed) and from year to year making it difficult to select a long term worst credible groundwater model for use in design. Figure 3.7.2 below conceptually shows the variability of pore water pressures with depth, time and vegetation type. With sparsely vegetated embankments suctions will develop in the surface layers during the dry summer months whereas in wetter periods these surface layers will show near-hydrostatic pressures. Sub-hydrostatic pore pressures to varying degrees will tend to be present at depth above the natural water table in the underlying material. Embankments that are vegetated with high water high water demand trees will demonstrate lower pore pressures above the natural water table with much larger suctions during the summer which may still be present (albeit lower) during the winter.

Figure 3.7.2: Seasonal Variation in Embankment Groundwater Regimes (Including the Effects of Vegetation)



The following sections summarise the existing groundwater models given in current LU standards and suggest modified pore pressures for assessing deep-seated stability based on groundwater monitoring of embankments across the LU network during the winter of 2000/2001, which is considered to be an extremely wet winter and therefore represents the worst credible pore pressures likely to develop over the design life of an embankment.

3.7.3.1 Traditional Approach

Traditional models have often assumed that the groundwater profile is either hydrostatic from the clay fill surface for poorly drained conditions, or consistently 1m below clay fill surface and hydrostatic for situations where the overall drainage conditions are more favourable.

The models described in the LU standard follow a similar pattern, with some allowance for the type of vegetation present on the embankment. These are repeated below:

Table 3.7.1: Design Pore Pressure for Embankment Fill Cohesive Material (after LU standard S1054)

Condition		Design pore pressures
A	Bare or grass-covered slope and no ash	Hydrostatic along the normal to the zero pressure line which is assumed to be 1m below the slope surface.
B	Bare or grass-covered slope and an ash surface layer	Hydrostatic along the normal to the zero pressure line which is assumed to be at the ash/clay interface.
C	Mature tree covered slope where the desiccation effect is to be relied upon. No ash.	Hydrostatic along the normal to the zero pressure line which is assumed to be 2m below the slope surface.
D	Mature tree covered slope where the desiccation effect is to be relied upon. Ash surface layer present.	Hydrostatic along the normal to the zero pressure line which is assumed to be 1m below the ash/clay interface

LU standard S1054 recognises that if an embankment is underdrained the pore pressures in the embankment will be reduced, and allows the Designer to assess an appropriate partial factor to apply to the hydrostatic pressure below the anticipated zero pressure line. No specific guidance on the value of this factor is provided.

In addition to the above, LU standard S1054 historically stated pore-pressures in Embankment fill cohesive materials within 2m of the ground surface shall be assessed for an assumed zero pressure line 1m higher than indicated“. This increase in porewater pressures within the top 2m from the slope surface has been discussed in Section 5.2.2 and is an important consideration for assessing shallow instability.

3.7.3.2 Modified Approach

There are a number of factors which will influence the porewater pressure regime in any given embankment:

- Type and extent of vegetation
- Permeability of fill
- Permeability of underlying material
- Presence of any drainage features
- Surrounding topography
- Presence of any leaking or damaged drains, or water mains (which may cause local "wet" spots)

The first three of these factors are discussed in detail below.

As well as considering pore pressures at depth within the embankment fill, there will also be pore pressures associated with the near surface clay soils, which is often desiccated (see Section 3.5.2.2) and of higher permeability. Pore pressures within the first 1-1.5m of the embankment can often be near hydrostatic during wet winters, with those at depth sub-hydrostatic.

3.7.3.2.1 Vegetation

It has been reported in many publications (e.g. Scott et al (29)) that the presence of high water demand (HWD) trees on the embankment can assist in maintaining low pore pressures, even suctions, throughout the winter when the highest rainfall is expected. Therefore, removal of these trees can lead to an increase in pore pressures within the embankment which needs to be considered in design. It therefore follows that grass covered slopes (for the purposes of this design guide taken to also include slopes vegetated with low water demand trees) will tend to exhibit higher pore pressures than those covered in HWD trees. It is therefore often important to determine not only whether an embankment is tree covered or not, but also to determine what type of trees are present on the slope. Table 3.13.1 lists the water demand of many common tree species.

3.7.3.2.2 Permeability of Fill

The permeability of the fill itself can also have an influence on the pore pressures developed as discussed in Section 3.4.3.2.

Lower permeability fill is still susceptible to desiccation cracking and as such remedial works should be engineered to avoid this (e.g. by utilising a capillary break layer or fibre reinforced fill).

3.7.3.2.3 Permeability of Underlying Material

The influence of permeability of the underlying material can be demonstrated both by the results of modelling and monitoring data (19). If an embankment constructed from cohesive fill sits on Chalk or Terrace Deposits or other more permeable strata, there will be underdrainage which will greatly reduce the pore pressures within the embankment fill itself. Numerical modelling has shown that two orders of magnitude increase in permeability is sufficient to create underdrainage.

3.7.3.2.4 Applied Research

When assessing monitoring data and considering groundwater regimes within embankment fill it is important to note the transient nature of the groundwater within the embankment. If plotting a set of porewater pressure data for a given period of time, then it is possible to deduce a maximum pressure envelope. However, at any particular point in time the actual groundwater regime will be different – at one depth a pressure may lie on this envelope, but at other depths pressures will be lower than the envelope. This is an important concept when deriving appropriate porewater pressures for design.

There is a dataset of monitoring of pore pressures in embankments on the LU network from the winter of 2000/2001, which is considered to be an extremely wet winter. For embankments founded on London Clay this data indicates near hydrostatic pore pressures in the top 2m below the clay surface (19). Beneath this, the bounding rate of increase in porewater pressure with depth reduces to about 60% of hydrostatic, creating a bilinear profile. Relatively low porewater pressures were recorded at many locations, with about 20% of the readings at zero pressure, and more than half at pressure heads of 2m or less. Embankments founded on Chalk or the Terrace Deposits showed the pore pressures to be sub-hydrostatic at nearly all points with pore pressures less than 20kPa even at depth within the embankment.

Figure 3.7.3 and Figure 3.7.4 show plots of the data described above for embankments founded on London Clay, and those founded on either Chalk or the Terrace Deposits respectively. A hydrostatic pressure line from the clay surface is shown on each figure to aid the graphical representation of the data.

Figure 3.7.3: Measured Porewater Pressures for Embankments founded on London Clay (after Briggs et al, (19))

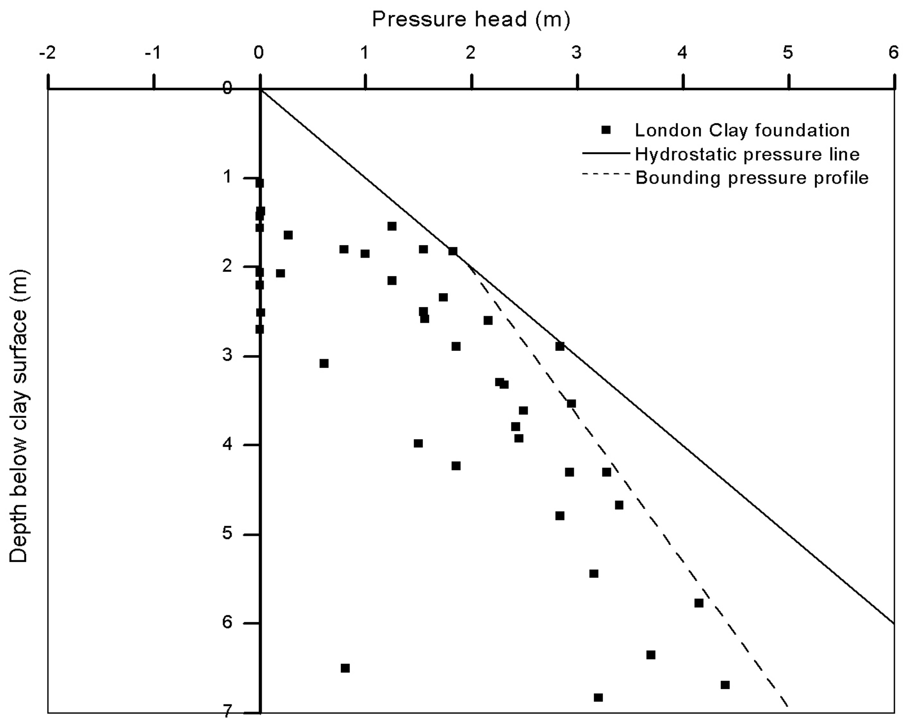
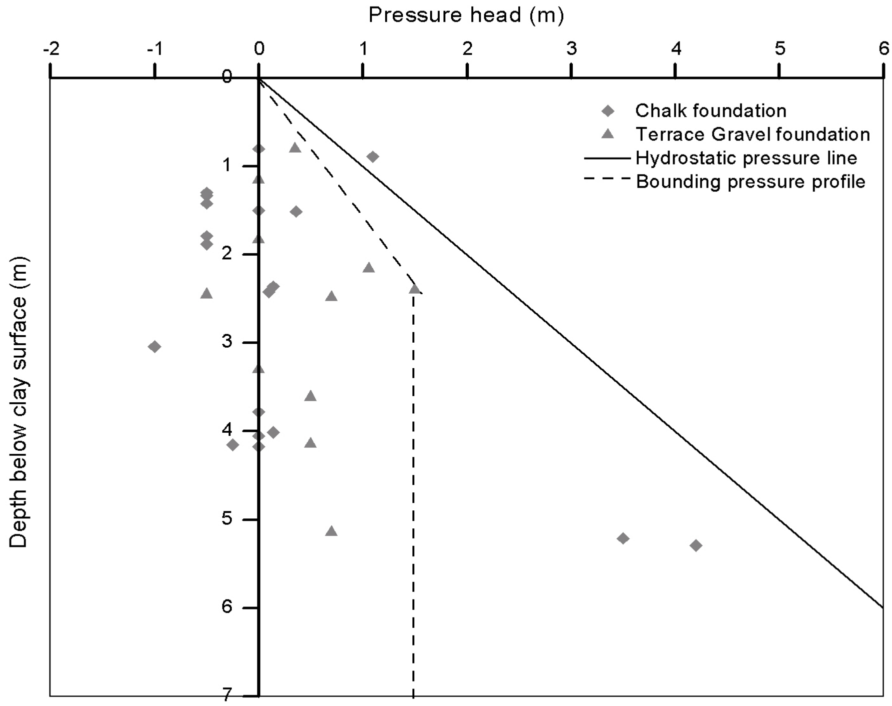


Figure 3.7.3: Measured Porewater Pressures for Embankments founded on Chalk or River Terrace Deposits (after Briggs et al, (19))



Field monitoring data and numerical modelling has shown that the variation of permeability with depth has a major effect on the end of winter porewater pressures following a period of extreme autumn and winter rainfall. The field monitoring data showed that even during an extreme wet winter, relatively low (less than 15kPa below 2.5m depth) porewater pressures can be sustained at depth within an embankment founded on London Clay. This is probably due to the influence of high water demand trees. The higher permeability of a near surface zone of cracked clay fill is clearly important and full hydrostatic porewater pressures can rapidly become established in this zone.

When considering the databank of rainfall totals for the SE region of England, which exists for the past 138 years, it is clearly shown that 2000/2001 is the winter with the highest rainfall total, and was characterised by a large number of moderately wet days, which would maximise rainfall infiltration into the slope (rather than a small number of extremely wet days, which would lead to surface run-off of rainfall). Therefore, the winter of 2000/2001 should be considered as an extreme event in terms of rainfall, and to base groundwater regimes wholly on this data for embankments on the LU network would be overly-conservative.

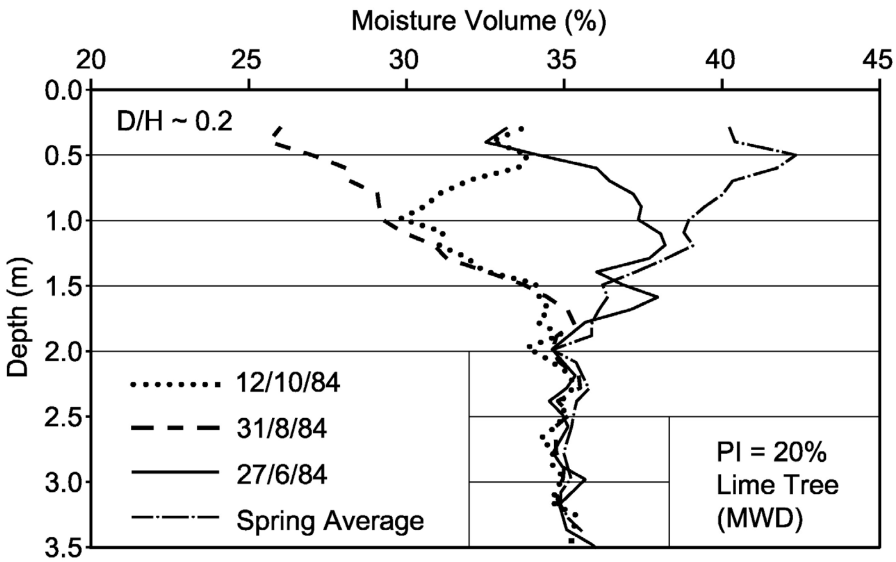
The influence of slope vegetation, and in particular different types of tree, on groundwater conditions within an embankment is an extremely important consideration. Figures 3.7.5, 3.7.6 and 3.7.7 (Note - the PI differs between figures) illustrate some important concepts:

i. Seasonal Changes, Figure 3.7.5 (44): during winter there is no photosynthesis and no water loss by transpiration, hence no water uptake by roots. Winter rainfall is therefore effective in allowing replenishment of soil moisture content, so soil is wettest (and groundwater pressures a maximum) just before deciduous trees start to photosynthesise (typically late March to late April, depending on temperatures). In late spring/early summer progressive soil drying begins from the surface and moves downwards as water uptake from tree roots exceeds the effective rainfall (and groundwater pressures decrease, typically becoming negative). In autumn, photosynthesis stops and rainfall becomes effective in rehydration of the soil. Rewetting starts from the surface and moves downwards (similar monitoring has shown that soil drying moves outwards during summer, and wetting moves progressively inwards towards the tree during winter; i.e. a tree acts as a "dewatering pump", switched on during the summer, and switched off during the winter).

ii. Persistent and Semi-Persistent Moisture Deficit, Figure 3.7.6 (44): Persistent – during initial tree growth progressive desiccation occurs (winter rewetting is less than summer drying). A permanent deficit is maintained (hence winter groundwater pressures are less than those beneath equivalent grassed slopes) throughout the time period when the tree remains healthy. Once the tree dies (or is cut down) the clay will progressively wet up, and swell, at a rate dictated by effective rainfall infiltration and clay permeability.

iii. Influence of Tree Water Demand, compare Figures 3.7.6 and 3.7.7 (44): the depth and magnitude of the influence of a HWD tree is far larger than that for a LWD tree, in particular the persistent moisture deficit is practically negligible for a LWD tree. Therefore, LWD trees are unlikely to maintain low pore-water pressures during a wet winter, whereas persistent deficits (and low pore-water pressures) are commonly observed for high water demand trees.

Figure 3.7.5: Seasonal Change in Soil Moisture Volume



Note:
D = Lateral distance from tree
H = Tree height
PI = Plasticity Index

Figure 3.7.6: Seasonal, Semi-persistent and Persistent Soil Moisture Volume Changes – High Water Demand Tree

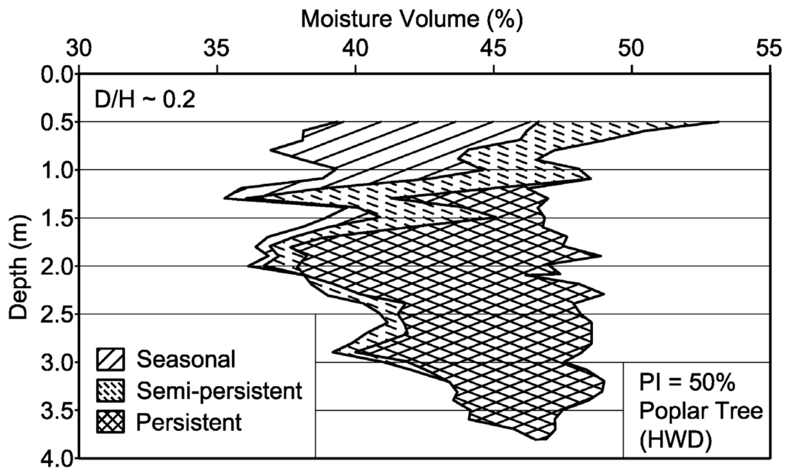


Figure 3.7.7: Seasonal, Semi-persistent and Persistent Soil Moisture Volume Changes – Low Water Demand Tree

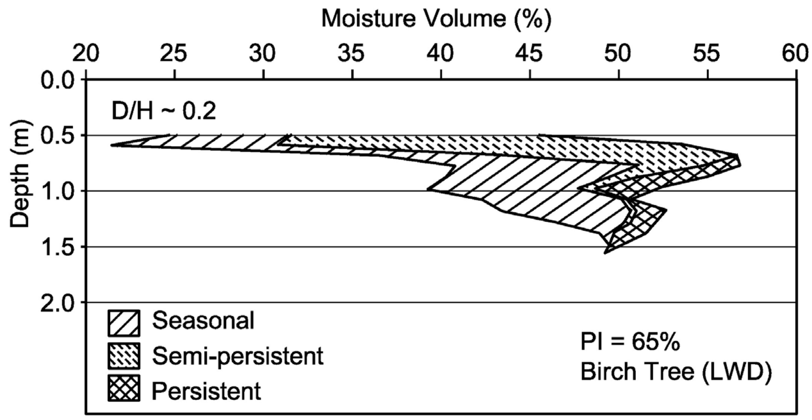
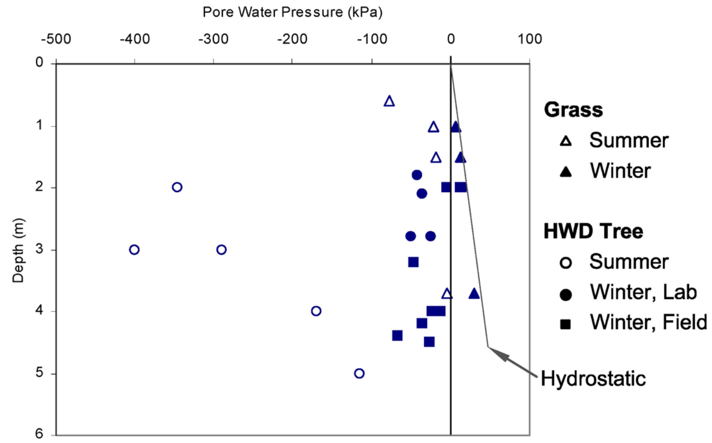


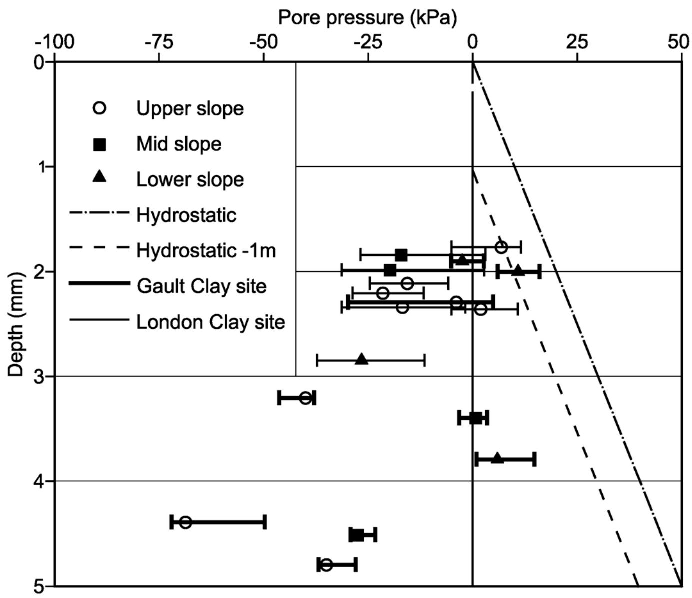
Figure 3.7.8 (44) collates pore-water pressure measurements which illustrate the differences between grassed slopes and slopes covered by HWD trees. For grass covered slopes, pore-water can increase towards hydrostatic, with seasonal average pressure changes of the order of 50kN/m^2 within the upper metre or so. In contrast, for HWD tree covered slopes suctions (negative pore-water pressures) are maintained during winter/spring at depths in excess of two metres, and seasonal pressure changes are of the order of 300 to 400kN/m^2 at depths of three to four metres. Seasonal pore-water pressure changes are strongly influenced by prevailing weather conditions. Figure 3.7.9 (44) plots data for a period when a wet summer was followed by a dry winter, with a maximum soil moisture deficit, SMD (for deciduous trees) of only 70mm (compared with a value of 279mm for the dry summer of 2003). During this period small suctions were generally recorded. Typically, pore-water pressures in the lower slope area were higher than those towards the crest.

Figure 3.7.8: Seasonal Changes in Pore Water Pressure. Grass Covered versus Tree Covered Slope (Mature High Water Demand Trees)



Note:
Data for Grass slope, Smethurst et al (2006). Data for HWD Trees, Chandler et al (1992); O'Brien et al (2004); Crilly & Driscoll (2000); James (2006)

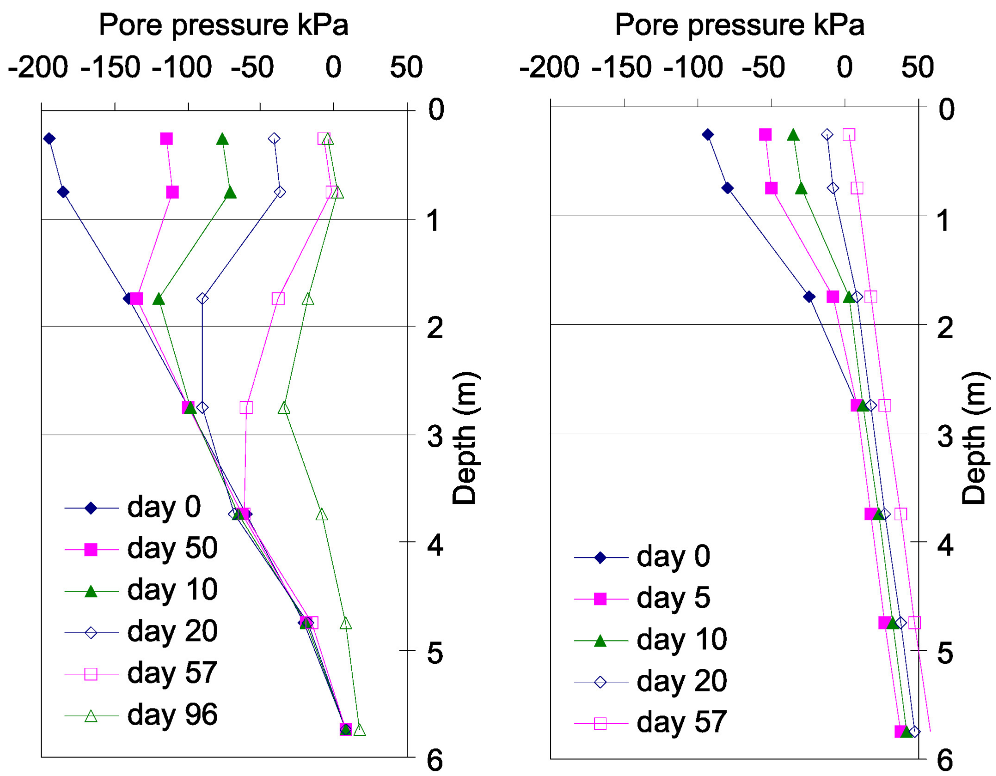
Figure 3.7.9: Measured Pore Water Pressure during Wet Summer/Dry Winter Period



Notes:
1. Gault Clay site, monitoring between January 2007 and January 2008.
2. London Clay site, monitoring between April and December 2008.

Numerical simulations of climate/pore-water pressure/vegetation interactions have captured these physical processes, and are summarised on Figures 3.7.10 (44), 3.7.11 (48) and 3.7.12 (48). Figure 3.7.10 illustrates the effects of progressive re-wetting from the surface (e.g. refer to Figure 3.7.5), and the maintenance of small suctions below an HWD tree covered slope, compared with the development of near hydrostatic pressures below a grass covered slope. Figure 3.7.11 illustrates the complex variation of pore-water pressures which may develop during a winter-summer cycle. Figure 3.7.12 illustrates the potential influence of HWD tree removal from the upper part of an embankment slope (for example to reduce seasonal track deformation). In this example if trees had been removed from the toe and lower slope, then a deep seated failure of the embankment slope would have been triggered.

Figure 3.7.10: Hydrogeological Modelling of Pore Water Pressure Change During Winter Rainfall



a) HWD Tree cover b) Grass cover

Notes:
1. Clay fill permeability = 3x10-8m/s.
2. 3 month "wet" winter (rainfall= 34mm/week).
3. At day 0. end of summer conditions.

Figure 3.7.11: Hydrogeological Modelling of Seasonal Changes in Pore Water Pressure (Tree Covered Slope)

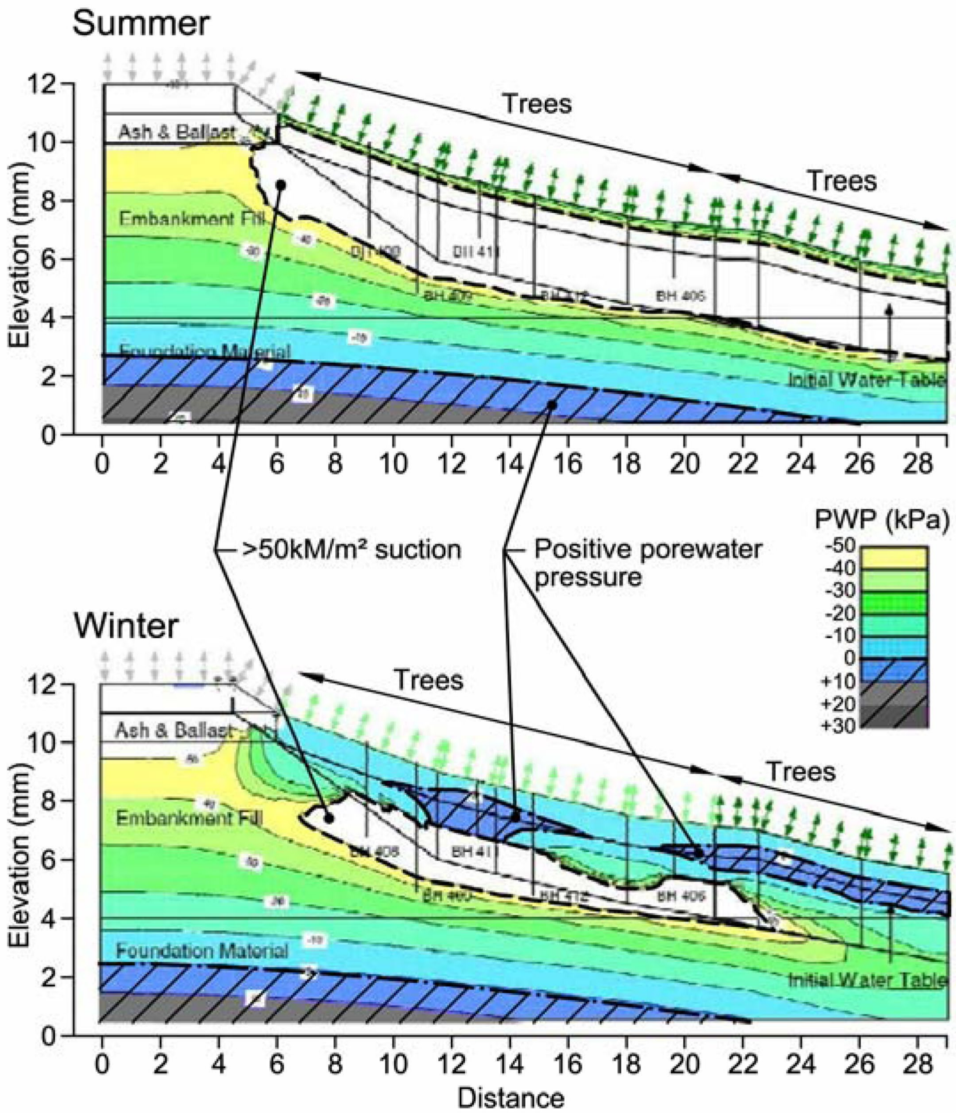
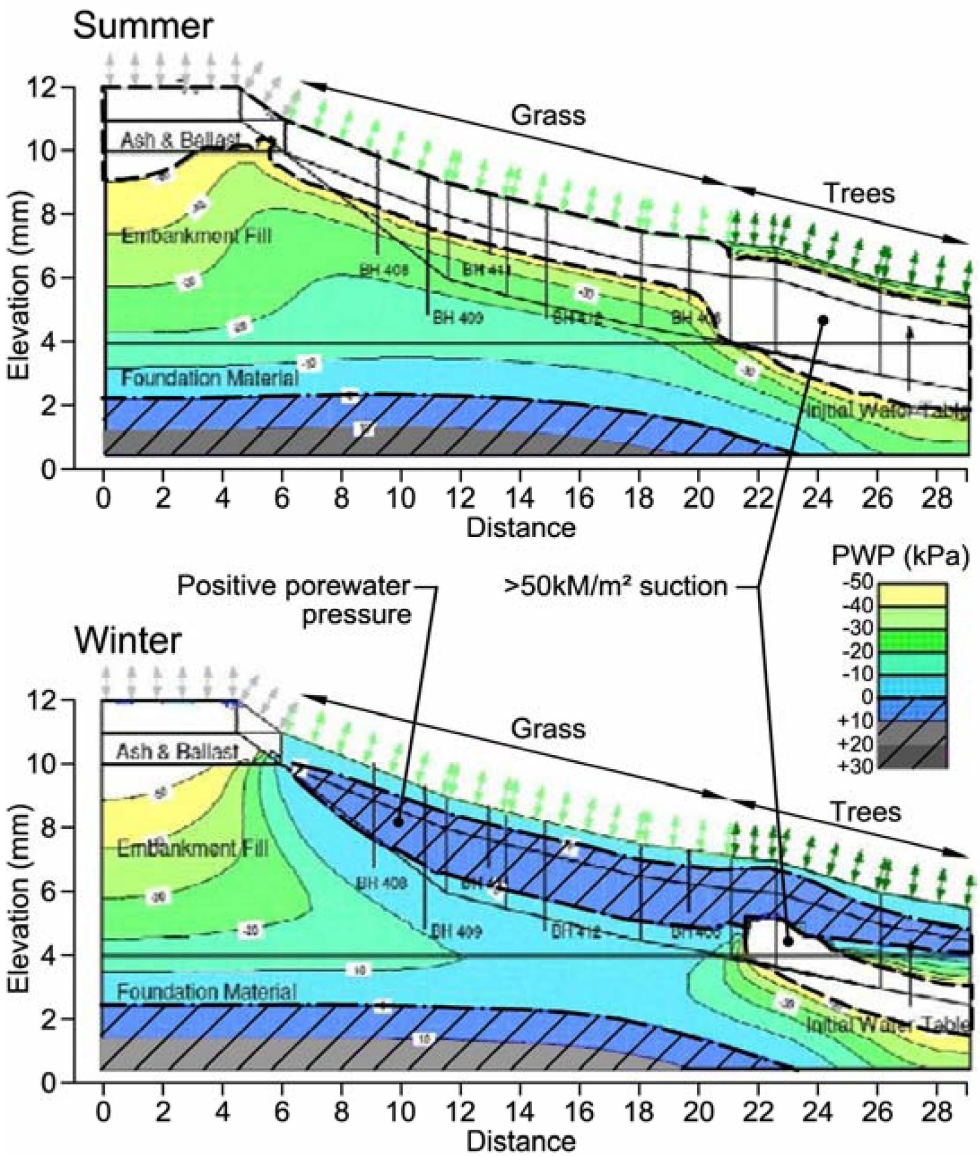


Figure 3.7.12: Hydrogeological Modelling of Seasonal Changes in Pore Water Pressure (Trees removed from Upper Slope)



Because of the above complex processes and interactions the interpretation of piezometer monitoring over a limited time period can be challenging. SMD (as published by Met Office) has often been used to assess the risk of slope instability (e.g. Ridley, et al, 2004), excessive track deformation, and to assess the representativeness of monitoring data. SMD is a useful index parameter, but some care is required in its use, and preceding weather patterns (rainfall, temperature, etc.) also need to be assessed. For example, during the period between December 2007 and May 2008 there were 6 months when SMD was close to zero (SMD <5mm), which is just one month less than the historically wet winter of 2000/2001. However, during 2007/2008 winter rainfall was low and slope stability problems were not as bad as those during 2000/2001. SMD is a measure of moisture conditions in the near-surface soil zone (and therefore not necessarily representative of pore pressure conditions at depth). The apparent inconsistency between 2007/2008 and 2000/2001, can be explained by the fact that the 2007 summer was very wet, hence the soil surface zone requires less rainfall infiltration to return to field capacity. SMD is published for a 40km MORECS square, and therefore changes in weather patterns across the MORECS square may lead to variations in earthworks risks across the area. The use of local weather station data can facilitate interpretation, especially when a MORECS square crosses a major topographical feature (e.g. North Downs) which will often create local variations in weather.

3.7.3.2.5 Design Pore Pressure

Modelling has been undertaken using the software package VADOSE/W to analyse porewater pressures that develop in clay fill embankments using actual rainfall data. Based on this, and a review of the available monitoring data, modified groundwater models are proposed for deep seated stability analyses. These models are intended as worst credible groundwater regimes where reliable monitoring data is not available and may be modified by designers if there is sufficient evidence and appropriate analysis to do so.

The modified groundwater model for embankments on a clay formation with grass covered slopes or where the benefits of HWD trees cannot be relied upon is shown in Figure 3.7.13 and Figure 3.7.14 shows the design pore pressures at the crest and toe of the embankment based on this modified groundwater model compared against the 2000/2001 LU monitoring data (see Section 3.7.3.2.4). Note that this modified groundwater model shows the water table at the toe at the surface of the in-situ clay. Its precise location will depend on a number of factors such as the depth and nature of any topsoil and/or Made Ground at the toe (shown as 0.5m thick in Figure 3.7.13), the presence of any existing functioning drainage and the surrounding topography. For typical conditions a 75% hydrostatic groundwater regime is suggested for design, whereas for poorly drained embankments fully hydrostatic conditions should be used. The decision to use the typical or poorly drained design lines should be carefully considered by designers as this will have a large impact on the earth structure stability. Poorly drained embankments may be indicated by the presence of hydrophytic vegetation, slope erosion features, evidence of ponding/boggy ground at the toe and poorly functioning or no toe drainage. Drainage conditions will usually vary along the length of an embankment. The ease with which rainfall can infiltrate into the slope is an important consideration; slopes with a “hummocky” or “coat hanger” profile will often facilitate ponding of water on the slope. Hence, higher groundwater pressures will tend to develop within these slopes, than those within slopes with a more regular profile, where rainfall can run-off more easily. Embankments, at a natural low point (often associated with an old valley feature) can be poorly drained, whereas adjacent slopes further away from the low point can be better drained (typically these are lower height earthworks as well). General slope instability can often be linked to poor drainage whilst the surrounding topography (and local infrastructure drainage, i.e toe drainage or drainage from adjacent wing walls and abutments) can also play a part in the drainage characteristics of embankments.

The modified groundwater model for embankments on a permeable formation is shown in Figure 3.7.15. For the purpose of this report, “permeable” formations are those with a permeability which is more than two orders of magnitude more permeable than the clay fill. Given the typical permeability of clay fills of between 5 x 10-9 m/s and 5 x 10-8 m/s, then a permeable formation would need to have a permeability of 5 x 10-6 m/s or higher to be “permeable” (i.e. silty or clayey sand or sandy silt).

Figure 3.7.16 shows the design pore pressures at the crest and toe of the embankment based on this modified groundwater model compared against the existing LU monitoring data (see Section 3.7.3.2.4). Due to the underdrainage effect of the underlying material a 50% hydrostatic groundwater regime is suggested for design. Again, the precise location of the groundwater table at the toe will depend on site specific factors which should be considered by the designer before selecting an appropriate worst credible design groundwater model. For the purposes of this guide the example shown in Figure 3.7.15 sets the water table 0.5m below the toe level.

Figure 3.7.13: Modified Groundwater Profile for "Poorly" and "Typically" Drained Grass-Covered Clay Fill Embankments on In-Situ Clay (Deep Seated Instability)

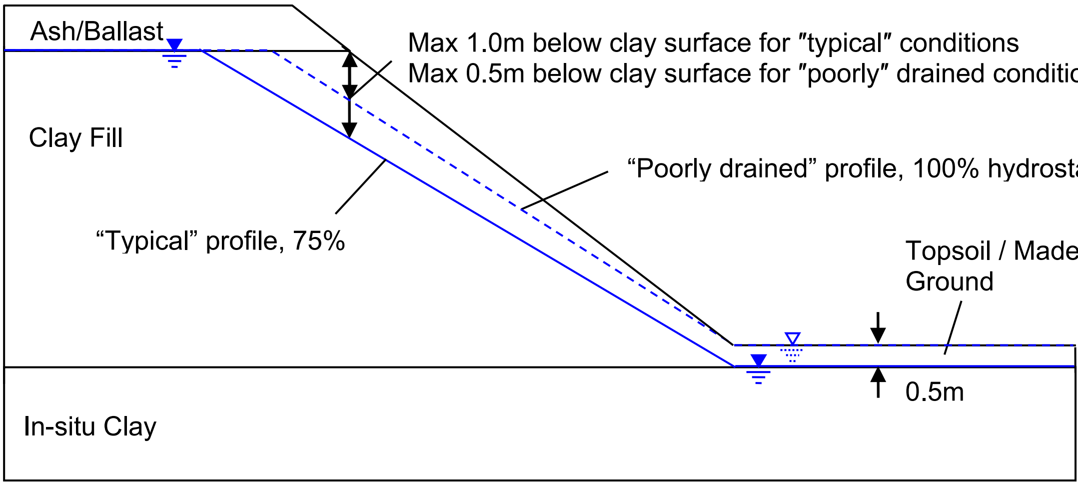
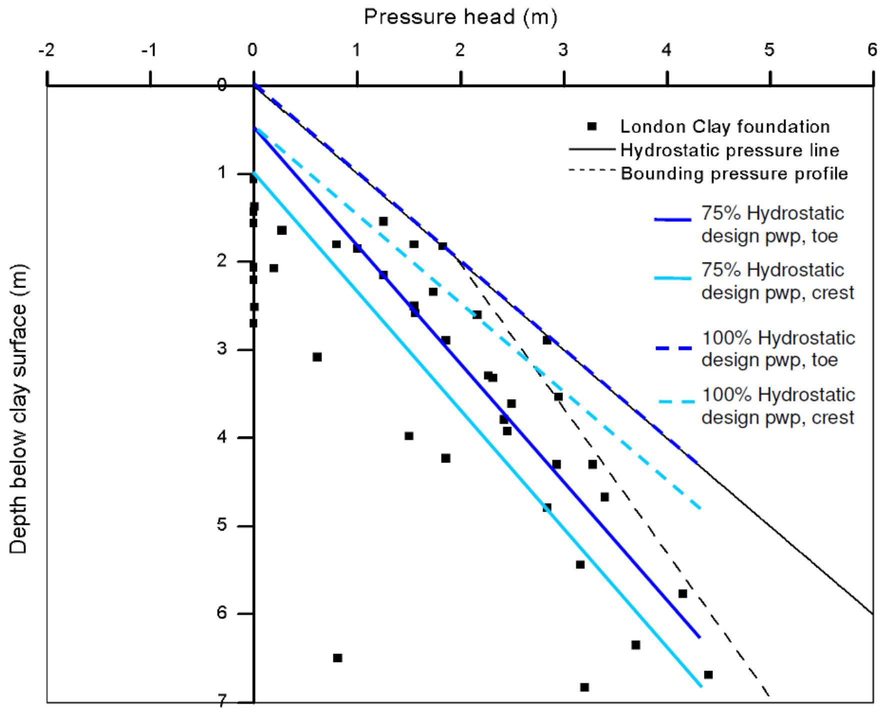


Figure 3.7.14: Deep Seated Instability, Comparison of Modified Design Porewater Pressures and Monitoring Data for "Typically" and "Poorly" Drained Clay Fill Embankments on In-situ Clay



Note: For shallow instability, depths ≤ 1.5m below slope surface, assume hydrostatic conditions. See Section 3.5.2.2

Figure 3.7.15: Modified Groundwater Profile for "Underdrained" Grass-Covered Clay Fill Embankments on Permeable Formation (Deep Seated Instability)

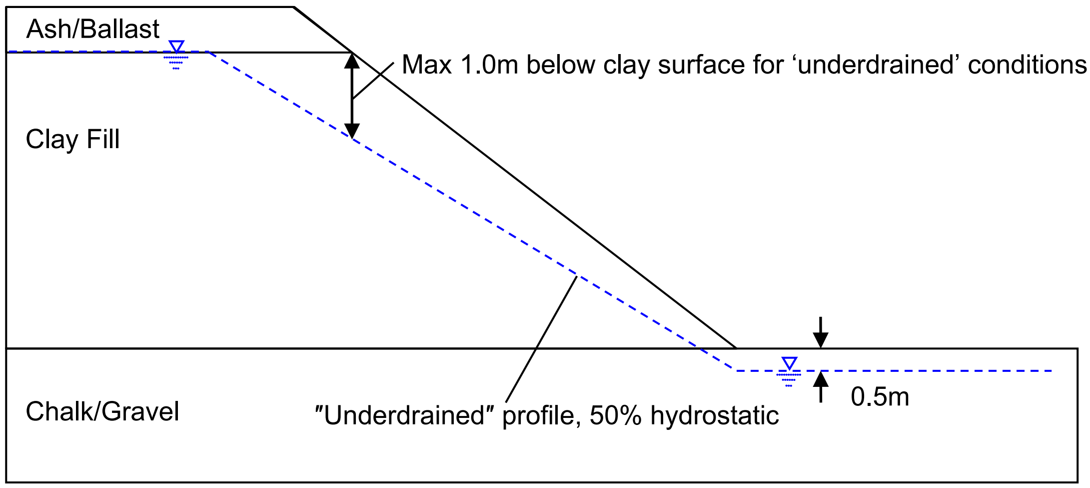
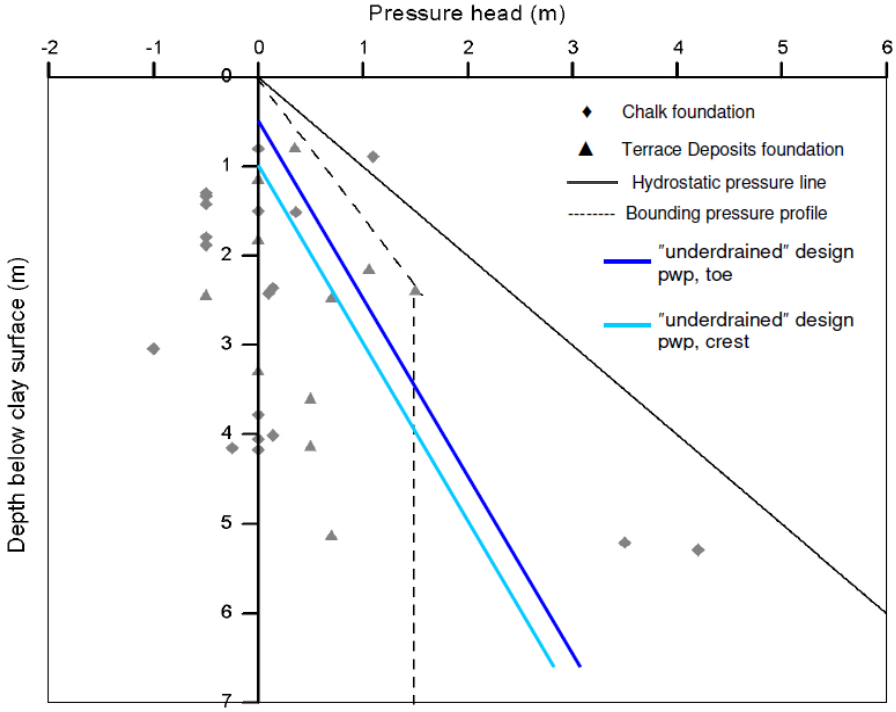


Figure 3.7.16: Deep Seated Instability, Comparison of Modified Design Porewater Pressures and Monitoring Data for "underdrained" Clay Fill Embankments on Permeable Formation

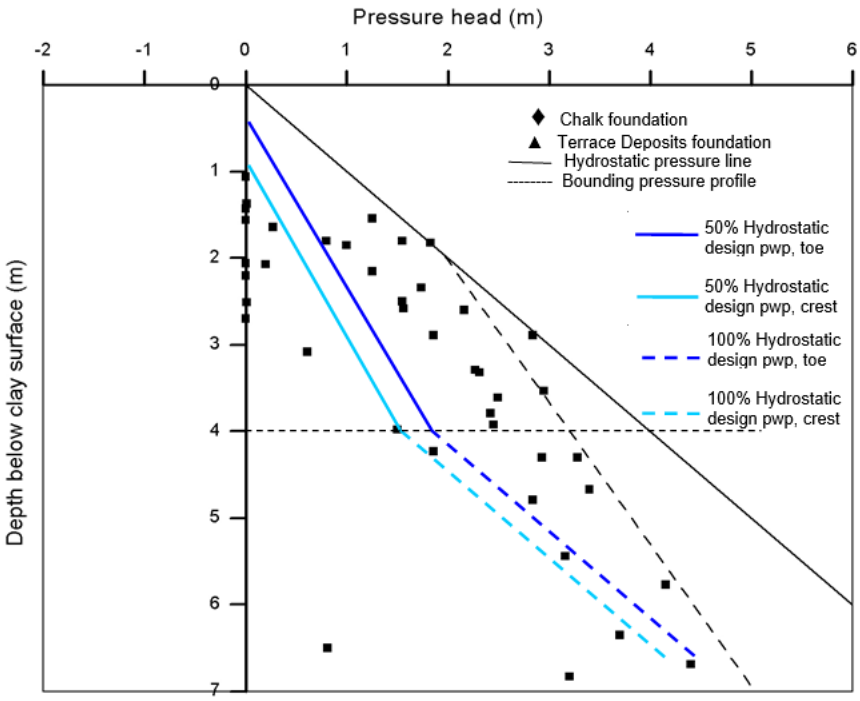


Note: For shallow instability, depths ≤ 1.5m below slope surface, assume hydrostatic conditions. See Section 3.5.2.2.

For embankments greater than 5m in height which are underdrained, applying the porewater pressure profiles shown on Figure 3.7.16 would result in porewater pressures at depth much higher than could realistically be expected. Therefore, for those embankments it may be appropriate to apply a more complex profile with porewater pressures at depth capped at a maximum of 20kPa.

For embankments covered in HWD trees, which are not underdrained, porewater pressures within the top 4m of the clay fill will be lower than for grass-covered slopes due to the influence of the trees – modelling has shown that even at the end of winter suctions can remain in the slopes, especially at depths of between 2 and 4m. Therefore it is proposed that the modified porewater pressures shown in Figure 3.7.17 are applied in deep seated stability analysis, in comparison with the typically drained scenario in Figure 3.7.14. As previously stated within this guide, the implementation of renewal works may require the removal of vegetation, and if this is the case the groundwater model in the stability analysis will need to be modified accordingly. Also, if the earth structure is determined to be stable, and this stability relies upon the influence of HWD trees on the groundwater regime, this will need to be highlighted in the maintenance plan.

Figure 3.7.17: Deep Seated Instability, Comparison of Modified Porewater Pressures for HWD Tree-Covered Clay Fill Embankments on In-Situ Clay and Monitoring Data



Notwithstanding all of the above, when deriving a groundwater regime for a particular embankment, any site specific monitoring or observations should be used, and if they indicate an alternative to that discussed within this report, then this should be considered. However, when considering the monitoring data available, the time of year and length of monitoring needs to be assessed – typically groundwater will be at its highest in March at the end of winter, and at its lowest in September at the end of summer. The year in which the monitoring was undertaken should also be taken into consideration, as some years may have had a dry winter and therefore not provide representative data. Published soil moisture deficit (SMD) data may be useful in this respect. The designer should consider how the data from monitoring should be extrapolated to allow for potential increases in pore pressures during future wet winters. This may require calibrated numerical hydrogeological modelling which can simulate transient interactions between climate and slope pore pressures. Relevant input parameters will include in-situ permeability, unsaturated hydraulic properties and weather data.

3.8 Deep-seated Instability – Clay Cuttings

This section outlines the proposed method of analysis for cuttings constructed in clay soils including giving guidance on the assessment of appropriate groundwater regimes for typical LU cuttings. For the impact of climate change on the stability of the clay cuttings please refer to Section 3.9

3.8.1 Method of Analysis

Where a cutting is formed in a medium to high plasticity clay (e.g. London Clay), it is necessary to assess the earth structure in terms of both deep-seated progressive failure mechanisms and shallow failures, as outlined in Sections 3.4 and 3.5. Due to the methodologies proposed in these sections for determining appropriate soil parameters, it will be necessary to consider these two mechanisms in separate analytical models in parallel with each other. Figure 3.7.1 summarises the process to be followed.

In addition to the derivation of soil parameters outlined earlier in the guide, it is necessary to determine appropriate groundwater regimes and assess any surcharge loading to be applied in the analysis. This is discussed in Sections 3.8.3 and 3.8.4 respectively.

3.8.2 Soil Parameters

Material parameters and partial factors should be based on the worst credible progressive failure mechanism described in Section 3.4.

3.8.3 Groundwater

3.8.3.1 Traditional Approach

Traditionally, r_u values have been used to represent porewater pressures in cuttings when assessing stability. Following the LU standard S1054, "the value of r_u in London Clay Cuttings shall generally be 0.25 for Cuttings which include mature trees over a clay foundation". This value can be modified dependent on site conditions – this information is repeated below in Table 8.1 from the LU standard S1054.

Table 3.8.1: Modifications to r_u = 0.25 in London Clay Cuttings for Conditions other than Mature Trees over Clay Foundations (after LU standard S1054)

Condition	Modification to r_u
Bare or grass-covered Cutting slopes	Add 0.1
Drainage in good condition at crest or down slope on Earth Structure	Reduce by 0.05
Underdrainage by gravel layer	Reduce by 0.1

Within the LU standard there is scope to use alternative representations of porewater pressure providing this can be justified. These values agree in general with Chandler and Skempton (30) who quote typical long-term r_u values of between 0.25 and 0.35, based on slips in cuttings typically in excess of 5m high.

3.8.3.2 Modified Approach

Although porewater pressures represented by r_u values as stated in the LU standard S1054 have been commonly used for London Clay cuttings, it is proposed that stability analysis should be carried out using groundwater tables. The use of r_u values has a number of problems, including: errors can easily be made if the cutting is composed of several different layers e.g. Made Ground over clay; difficult to correlate r_u values with monitoring data, or to use more sophisticated modelling.

Table 3.8.2 summarises the porewater pressures to be applied for deep-seated stability analysis, which are equivalent to the r_u values in the LU standard. All piezometric lines are to be applied at the slope surface or the interface of the granular soil/Made Ground and cutting material. For shallow slips, the guidance in Section 3.5.2.2 should be followed.

Table 3.8.2: Modified Porewater Pressures for Cuttings

Condition	Piezometric Line
Bare or grass-covered Cutting slopes	70% hydrostatic
Bare or grass-covered Cutting slopes with drainage in good condition at crest or down slope	60% hydrostatic
Bare or grass-covered Cutting slopes with Underdrainage by permeable layer	50% hydrostatic
Tree-covered (HWD trees) Cutting slopes	50% hydrostatic
Tree-covered (HWD trees) Cutting slopes with Underdrainage by permeable layer	30% hydrostatic

Drainage conditions will usually vary along the length of a cutting. The ease with which rainfall can infiltrate into the slope is an important consideration; slopes with a “hummocky” or “coat-hanger” profile will often facilitate ponding of water on the slope. Hence, higher groundwater pressures will tend to develop within these slopes, than those within slopes with a more regular profile, where rainfall can run-off more easily.

Low height cuttings (less than 4m clay core height) can exhibit lower pore water pressures than deep cuttings, because crest and toe drainage (or track ballast occasionally) tend to provide greater overall benefits (since a drainage feature will have a finite zone of influence). Also vegetation can provide a more significant beneficial effect. When considered appropriate, these effects can be simulated by applying a lower percentage (typically 10% reduction for each one metre height reduction below 4m clay core height) of hydrostatic groundwater pressure, below that outlined in Table 3.8.2 above. The potential beneficial effects for low height cuttings should not be applied if the slope is bare, i.e. non-vegetated and clay, which may be vulnerable to desiccation cracking, is exposed at the surface. Irrespective of the combination of conditions which may apply, a minimum pore water pressure condition of 30% hydrostatic should always be applied.

As for embankments, consideration needs to be given to the changes in groundwater that may happen over a design life of 120 years. The existing condition may not be maintained over this period.

3.8.3.2.1 Counterfor Drains

Many LU clay cuttings have counterfort drainage systems which may be decades old, and in varying condition. Desk studies may identify the age and as-built geometry of the counterforts. These are often apparent on the slope surface and can be located by topographic surveys/mapping. Historical aerial photographs (if available) can also be useful in identifying these features depending on the vegetation coverage across the site.

In addition, site specific ground investigations should be carried out to confirm:

i. The geometry of the counterforts – depth (especially across the lower third of the cutting), width, length, and spacing.

ii. Nature of counterfort fill – typically coarse grained material, including angular rockfill was used to backfill counterforts

iii. Effectiveness as drains – this will depend on connections (and their condition) to an effective drainage outfall.

Trial pits, or trial trenches, are usually required to investigate and assess the effectiveness of counterforts, and therefore ground investigation costs will be largely dependent on access to the cutting toe and associated health and safety requirements. The location and condition of any lineside services will also be a consideration.

Counterforts may be effective in stabilising a cutting due to two different mechanisms:

a) "Buttress Effect" – the strength of the granular fill will provide enhanced shear resistance along those potential failure planes which cut through the counterforts. This is often the most significant residual benefit of old counterforts, but reliance on this benefit requires knowledge of the as-built geometry, especially in the critical toe area of the cutting. Enhanced shear resistance can be assessed through equation 3.8.1 below, based on first principles for a weighted average:

$$c'_{av} + \tan \phi'_{av} = (A_r)(c'_{cb} + \tan \phi_{cb}) + (I-A_r)(c'_{clay} + \tan \phi'_{clay}) \qquad \text{Equation 3.8.1}$$

Where:

c'_{av}	= effective cohesion of composite clay + counterfort, per metre run of cutting
ϕ'_{av}	= effective angle of shearing resistance of composite clay + counterfort, per meter run of cutting.
A_r	= area ratio of counterforts, i.e. plan area of a segment of the failure plane occupied by counterforts, expressed as a fraction of the overall cutting area.
c'_{cb}	= effective cohesion of counterfort backfill, typically this will be zero.
ϕ'_{cv}	= effective angle of shearing resistance of counterfort backfill. Typically this will be relatively high due to angular coarse grained nature of fill, refer to Chapter 9 for guidance on assessing the shear strength of granular materials (Note – coarse grained angular fill may have $\phi'_{cv} \geq 40°$, refer to Table 9.1).
c'_{clay}, ϕ'_{clay}	= effective cohesion, and angle of shearing resistance for clay in cutting (with typical values dependent on the hazard potential of the slope).

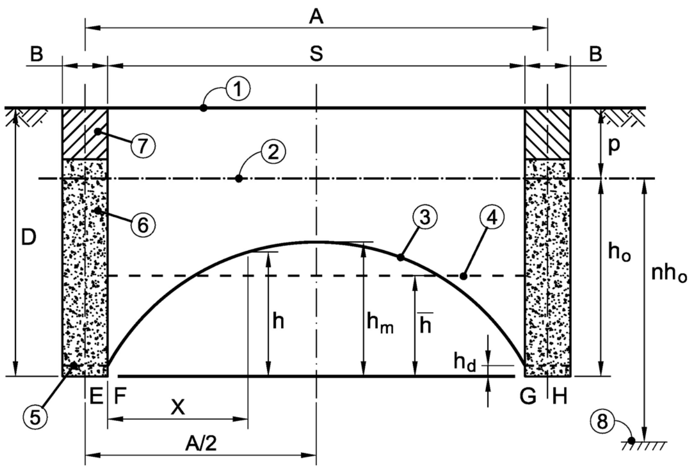
This provides an initial basis for estimating the effective cohesion and shearing resistance for a composite cutting (clay and counterfort – per m run). However any assessment must also consider the risk of localised shallow failures occurring between the counterfort drains, where the slope strength remains unchanged.

b) "Drainage Effect" – if the counterforts are judged to be effective, then the piezometric line should be lowered. Hutchinson (1981) provides guidance on the effectiveness of counterforts as drains, and the drainage effect will depend on the spacing of the counterforts (assuming they have an effective drainage outfall, e.g. to track drainage. Hutchison's method is summarised on Figures 3.8.1 and 3.8.2, with Figure 3.8.2 plotting η versus S/h_o, where:

$\eta = (h_o - h')/h_o$ Equation 3.8.2

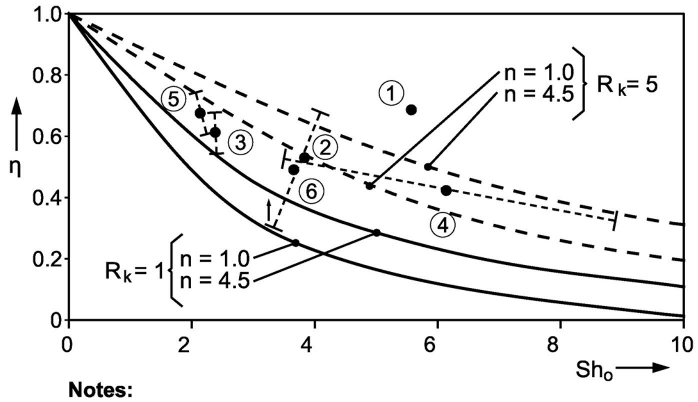
h_o = height of original piezometric line above drain invert
h' = mean piezometric line, above drain invert, after drain installed
s = horizontal distance between counterfort drains (see Figure 3.8.2)
R_k = anisotropy of permeability, for London Clay $R_k \geq 2.0$ (see Figure 3.8.2)

Figure 3.8.1: Cross-section of Typical Counterfort



Notes:
1. Ground surface.
2. Original piezometric level on plane EH.
3. Piezometric levels on plane FG after drainage.
4. Mean piezometric level on plane FG after drainage.
5. Mean piezometric levels on drain inverts after drainage.
6. Trench or counterfort drains.
7. Clay seal.
8. Impermeable boundary at depth.

Figure 3.8.2: Comparison of Approximate Theoretical Predictions for the Effect of Trench Drains with Field Data



Notes:
1. Sevenoaks.
2. Guildford.
3. Hodson, near Swindon.
4. Burderop Wood, near Swindon.
5. Boulby.
6. Barnstable.

The charts assume the drainage material is free-draining, which will in practice mean that the drainage material needs to have a permeability of more than two orders of magnitude larger than adjacent clay. At shallow depth, the horizontal permeability of London Clay is typically of the order of 10-8m/s to 10-9m/s, hence the drainage material would need to have a mass permeability of more than 10-6m/s (and also to have an effective drainage outlet). Hence, many old counterforts may be made more effective as drains by ensuring the counterforts are connected to an effective outfall along the slope toe. If the drainage effect is to be relied upon then an assessment should be made on how they can be maintained over the design life of the asset. The counterforts typically do not have drainage pipes, just granular backfill. Therefore during ground investigations an assessment should be made of any siltation of the granular fill whist also confirming the geometry of the counterfort. Any groundwater regime must consider the functionality, or otherwise, of all existing site drainage and how this may be modified by installing additional counterforts and pipes. If new counterforts are built then the trenches should be lined with a geotextile filter, and a rodable drainage pipe should be installed, to facilitate future cleaning and maintenance.

3.8.4 Surcharge Loading

The LU standard S1054 states that “A live load (uniform variable unfavourable action) of 10kN/m^2 shall be applied to the crest of all cuttings where vehicle access or further development is deemed possible by the Designer“ (Cl 3.5.6.1). Where appropriate greater loadings should be applied e.g. due to nearby buildings. Based on this, a partial factor of 1.3 should be applied to the 10kN/m^2 loading when undertaking a DA1-2 analysis.

BS6031:2009 states “A minimum surcharge of 10kN/m2 should be applied to the surface at the top of embankments and cuttings where the external action might adversely affect the stability of the slope“ and that “The minimum surcharge should be considered as a permanent load and appropriate partial factors should be applied to the action". BS6031:2009 also states that additional loading should be applied where appropriate e.g. live loads such as vehicle loading.

The National Annex to EC7 does not give specific values of partial factors to be used for actions in the case of earth structures, but instead refers to the partial factors for buildings.

For each site the Designer is required to assess whether it is appropriate to apply a 10kPa surcharge to the crest of the cuttings, inside and outside of the LU boundary fence.

For typical cutting heights the application of this surcharge, with a Partial Factor of 1.3, will not impact the existing stability significantly, but may impact the remedial works, particularly if a solution such as piles is implemented. For low height cuttings, less than 4m in height, the application of this surcharge will have more of an influence when considering the stability of the slope, and the design of any remedial works.

3.8.5 Vegetation

The comments made earlier in this report, in terms of the effect of vegetation (especially trees) on pore water pressures within the slope, apply to both embankments and cuttings. However, experience indicates that trees on cutting slopes (in particular, high water demand trees), seldom cause track serviceability problems. Additional risks which should be considered is the risk of trees, or tree stumps on cutting slopes collapsing onto the tracks or leaf fall onto the tracks during autumn/winter period.

3.8.6 Retaining Wall at Toe of Cutting

Retaining walls are commonly encountered at the toe of LU cuttings, although the type, scale and condition of these walls and their potential beneficial effects can vary enormously. Some questions which need to be considered during an assessment are outlined in Table 3.8.3. From Table 3.8.3 it can be seen that a good understanding of the overall size and condition of the retaining wall is an important part of the assessment process.

Table 3.8.3: Potential Influence of retaining Walls at the Toe of a Cutting

Factor	Questions	Comments
Age of Retaining Wall	Was wall built when cutting formed, or shortly after?	If wall built when cutting formed, then less risk of progressive failure.
Condition of Retaining Wall	Evidence of wall cracking, or tilting/bulging of wall?	Wall in good condition, then less risk of deep seated instability; vice-versa if in poor condition.
Depth of Foundations	Are wall foundations, built well below (say ≥1.0m) toe of cutting?	If wall foundations are shallow, then they will be of little value in reducing risk of deep seated instability.
Drainage behind or through Wall	Is there permeable backfill or drainage behind wall? Are there effective drainage outfalls? Can groundwater flow or seep through wall?	If there is effective drainage at toe, then this reduces the risk of progressive failure, refer to Figure 4.6, and Table 4.3.
Risk of Slope Failure above Retaining Wall	What is relative height of retaining wall compared with height of cutting?	A large and massive retaining wall may have a dominant influence on slope stability.
Risk of Slope Failure below Retaining Wall	Consider age, condition, foundations, drainage, slope and underlying stratigraphy, geometry etc. in assessing deep seated failures.	Retaining wall may have a dominant influence on slope stability.

Massive masonry retaining walls were commonly built as part of bridge works across the cuttings. In addition there are large retaining walls located at the toe of some cuttings which are not associated with wing walls. Provided these walls are judged to be in good condition, then the risk of deep seated failure below or through the walls is likely to be low. However, it is important to assess the risk of cutting failures occurring above the walls, Figure 3.8.3. For wing walls, Because of the kinematic constraints provided by an adjacent bridge, the orientation of critical earthworks failure mechanisms may be at less than 90° to the track alignment, Figure 3.8.4.

The effectiveness of drainage, both behind toe retaining walls, and at the crest of the cutting is always an important consideration; if drainage is effective then the risk of instability is substantially reduced. Similarly if drainage is poor, or if services facilitate leakage of water into the cutting (e.g. leaking water/sewer mains, or car park outfalls) then instability risks will be substantially increased. Any ingress of water into the cutting should be remediated at source.

Although, as suggested, the risk of deep seated failures beneath the wall may be low; the risk of failure below the wall must still be assessed – considering the slope and wall geometry, stratigraphy, historical performance of the assets (both wall and slope) and the groundwater conditions etc. In addition to global stability checks for cutting failure beneath/above the wall, local ULS checks should be made for the retaining wall itself (i.e. sliding, bearing capacity, etc.).

Figure 3.8.3: Retaining Walls at Toe of Cutting

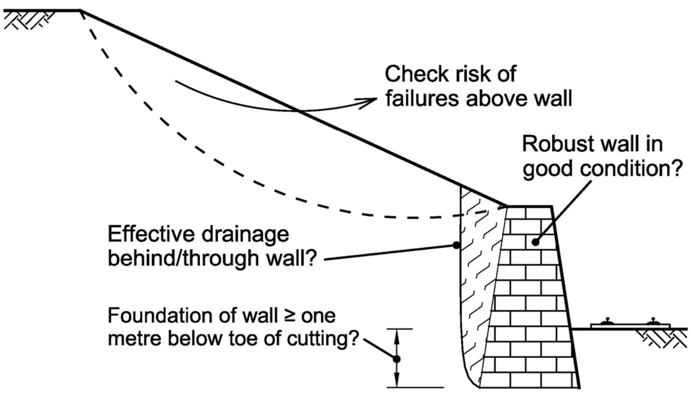
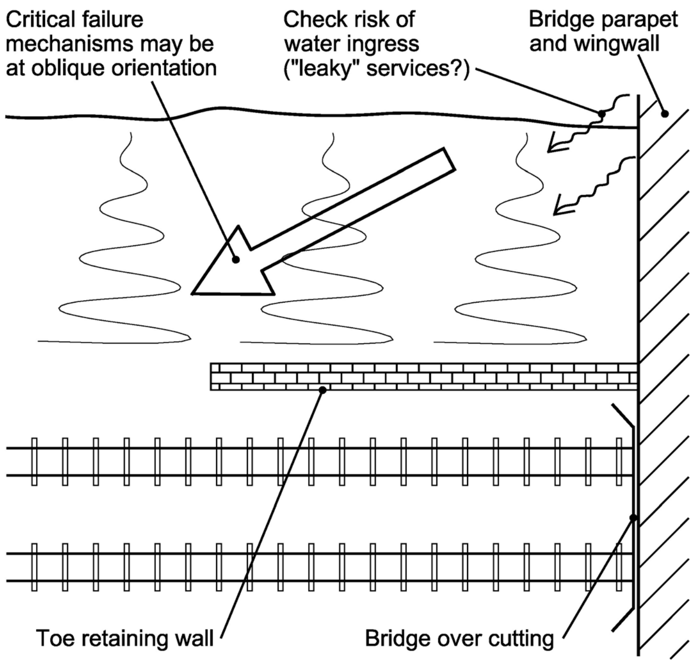


Figure 3.8.4: Retaining Walls, adjacent to Bridge (Plan View)



3.9 Impact of Climate Change on Earth Structures

3.9.1 General

The impact of climate change and inclement weather on LU earth structures on the BCV (Bakerloo, Central and Victoria lines) and SSL (Subsurface lines; District, Circle, Metropolitan and Hammersmith & City lines) network was studied in two phases between 2009 and 2010. The results of this study was reported in “The Effects of Inclement Weather on Earth Structures – Phase II Report”, LU (49).

In this study four main failure mechanisms following adverse weather events were identified for earth structures as follows:

1. Deep-seated or shallow rotational failure from prolonged rainfall and low Soil Moisture Deficit
2. Flow failures resulting from storm events
3. Frost shattering of chalk
4. Flooding and scour.

Assets susceptible to each mode of instability were identified and classified into High, Medium and Low risk categories. This study was later extended to cover the JNP (Jubilee, Northern and Piccadilly lines) network. The results of this study was reported in “Effects of Inclement Weather on JNP Earth Structures”, LU (50).

For further details on the above studies and the list of assets at risk from the effects of adverse wethaer events reference should be made to the above reports.

For the purpose of this design guide, Items 2 and 3 cannot be analysed by conventional stability methods and, hence, are only briefly covered in Sections 3.10.1 and 3.15.

Flooding and scour applies to cases where rivers and streams which are running parallel or perpendicular to the toe of embankments are flooded during storm events. This will potentially impact the stability of the embankments by increasing the pore pressures within the embankment and/or by scouring and erodin the toe of embankments. These stability modes can be analysed by the methodology described in this report by considering appropriate piezometric levels reflecting the high flood water level and geometry for the embankment.

However, Item 1 which is mainly related to the effects of inclement weather on clay cuttings and embankments is covered in more detail in Section 3.9.2 below.

3.9.2 Impact of Climate Change on Clay Cuttings and Embankments

The impact of climate change on pore-water pressure regimes for the design and assessment of clay earthworks, has been studied by Wengui Huang, et al. (51). The climate data used in the study, were taken from UK Climate Projections 2018 (UKCP18), which is the latest national set of climate projections for the UK, Murphy et al. (50). The projected climate shows that there will be more potential evapotranspiration and less rainfall in summer, and more rainfall in winter.

This study was carried out for earthworks made of clay fill and/or in-situ clay, with a relatively low permeability (5×10-8 m/s ~ 5×10-9 m/s), using climate projections for the London area and one-dimensional seepage analyses.

Based on the results of extensive one-dimensional seepage analyses the following conclusions were drawn:

- The projected climate change is not expected to require higher design PWPs for analysis of deep-seated slips. A localised perched water table at shallow depth (due to weathering and desiccation cracking, etc.) is expected, even with the current climate (Smethurst et al.) (52 and 53).

- Climate change will lead to increases in the magnitude of dry-wet cycles. This will drive greater shrink-swell behaviour and may increase desiccation cracking. This also means that the rate of weather driven deterioration of soil strength is likely to increase.
- For clay slopes of low permeability, the infiltration rate is governed by the soil permeability. Therefore, the increase in rainfall intensity leads to significantly increased runoff. This may bring challenges to drainage management and potentially cause more flooding or erosional failures such as washout, in both clays and other materials.
- The projected increase of potential evapotranspiration will have a greater impact for slopes with tree cover than grass cover, as the former has deeper roots and can transpire water even in the late summer when the availability of soil water is limited. Therefore, the vegetation management strategy of earthworks (Briggs et al. (53); Smethurst et al. (54)) needs to be reviewed in the context of climate change.

Hence, the pore pressure regime specified in sections 3.7.3 and 3.8.3 will remain valid at present.

The increase in the amplitude of the swell-shrink movements will cause increased serviceability concerns for the clay embankments, as well as increasing the rate of strength deterioration. This needs to be considered carefully when deriving soil strength parameters following the procedure in Sections 3.4 and 3.5.

The predicted increase in surface runoff may require provision of drainage at certain locations depending on the catchment size, topography, and boundary conditions.

The increased intensity of rainfall could surcharge the local drainage network and direct flow towards the adjacent earthworks, leading to flooding and surface erosion. A catchment study should be carried out during the design to assess the susceptibility of the slope to this effect, so that appropriate mitigation measures can be considered as part of the design.

Vegetation continues to play an important role in controlling the pore pressure regime in the clay slopes, as well as the swell-shrink behaviour and the shallow stability (See also Section 3.4.3.7). Hence, vegetation on the earth structures should be managed in accordance with LU Standards S1165 and G0058, in a manner to maximise the beneficial effects of reducing the pore pressures and increasing the near surface shear strength through root reinforcements, as well as minimising the detrimental impact of swell-shrink behaviour.

3.10 Granular Embankments and Cuttings

This section outlines the proposed method of analysis for embankments and cuttings constructed in granular material, including giving guidance on the derivation of soil parameters, and the assessment of appropriate groundwater regimes. Appendix F provides guidance on the shear strength of recycled Class 1A fill.

3.10.1 Method of Analysis and Failure Mechanisms

For granular soils, the most common failure mechanism is erosion and loss of fines, or washout (due to concentrated water flows), which cannot be analysed by conventional stability methods. This mechanism of failure is likely to become more frequent due to impact of climate change. According to the climate change projection UKCP18 (50), it is likely that in the UK and London Area the frequency and intensity of intense rainfall events will increase. This will lead to increases in surface run-off which could trigger washouts and debris flows. This risk needs to be considered carefully during the assessment and design of earthworks in granular materials. For granular slopes, careful detailing of drainage (especially along the crest), and erosion control matting over exposed slopes will be important as well as careful management of slope vegetation. For steep granular cuttings, there may be a risk of trees or tree stumps falling on the track and lineside services., and this should be carefully considered.

As for all other earth structures, both deep-seated and shallow failure mechanisms need to be assessed. Although for this type of soils deep-seated instability is not generally a major concern and is generally easier to predict. The instability in this type of material mostly manifests itself as surface ravelling of materials downslope, leading to maintenance problems (where the angle of friction of the soil is close or less than the slope angle, leading to factors of safety close to or below unity). The designer should make a judgment based on the slope stability analysis as to what constitutes a deep-seated failure, taking into consideration the specific slope characteristics and the potential impact and the risk to the railway infrastructure.

Soil parameters for both failure mechanisms should be derived following the guidance in Section 3.10.2, taking into account any variations in the nature of the near-surface material which may affect its strength. Porewater pressures should be derived for analysis following the guidance in Section 3.10.3. When assessing shallow failure mechanisms, as for cohesive earth structures, the contribution of the vegetation to stability should be considered, however if it is to be relied upon over a 120 year design life this needs to be highlighted as part of any maintenance plan. If the vegetation is to be removed, then its contribution to shallow stability should be ignored. Shallow stability should be assessed for both circular and non-circular slips, typically 1.5m depth.

3.10.2 Parameter Derivation

When considering the strength of a granular material, a simple and coherent approach to assessing this strength has been described by Bolton (31):

$\phi'_{peak} = \phi'_{cv} + \varphi$

Where:

ϕ'_{peak} = mobilised angle of friction

ϕ'_{cv} = critical state or constant volume angle of friction

φ = dilatancy angle

The critical state angle of friction is an intrinsic property, mainly dependent on particle shape, mineralogy, and grading. The dilatancy of a material is mainly related to relative density and effective stress level. In order to determine appropriate values of friction angle to apply in a slope stability analysis, $\phi'cv$ and φ should be independently assessed.

Values of the critical state angle of friction for various Sands and Gravels are given in Bolton (31) and Stroud (32). Some of these are included in Table 3.10.1, and can be used to assist with deriving appropriate values for granular soil based on knowledge of the particle shape, grading and mineralogy.

Table 3.10.1: Critical State Angle of Friction, ϕ'_{cv}, for some Sands and Gravels

Sand Type	Particle Size	Mineralogy	Particle Shape	D_{50} (mm)	D_{10} (mm)	Unif. Coeff	E_{max}	e_{min}	ϕ'_{cv}
Ottawa	c	q	well rnd	0.75	0.65	1.2	0.8	0.49	29.5
Ottawa	m	q	rnd	0.53	0.35	1.7	0.79	0.49	30.0
Chattahoochee	m	q	s ang	0.37	0.17	2.5	1.10	0.61	32.5
Mol	f-m	q	s rnd	0.19	0.14	1.5	0.89	0.56	32.5
Ticino	c	q	s rnd	0.53	0.36	1.6	0.89	0.6	31.0
Sacramento	f-m	q+f	s ang/ s rnd	0.22	0.15	1.5	1.03	0.61	33.3
Reid Bedford	f-m	q+sf	s ang	0.24	0.16	1.6	0.87	0.55	32.0
Hokksund	c	q+f	s ang	0.39	0.21	2.0	0.91	0.55	32.0
Toyoura	f	q	s ang	0.16	0.11	1.5	0.98	0.61	32.0
Mersey	f-m	q	s ang/ s rnd		0.1	2.0	0.82	0.49	32.0
Milton Mines	f-m	q+f	ang	0.2	0.11	2.0	1.05	0.62	35.0
Southport	f-m		ang	0.2	0.12	1.8	0.88	0.53	35.0
Crushed quartz	f	q	v ang	0.12	0.07	2.0	1.15	0.55	36.4
Crushed feldspar	f	f	v ang	0.12	0.07	2.0	1.21	0.49	38.7
River sand & gravel	37mm-f sand	f+q	s ang / s rnd	4.8	0.6	8			35.0
Glacial outwash sand	f-c		s ang	0.75	0.15	6	0.84	0.41	37.0
San Francisco	50mm-fines	basalt	ang						38.0
Furnas Dam	10mm-fines	quartzite							39.0
Granite gneiss	37mm-4mm		ang						40.8

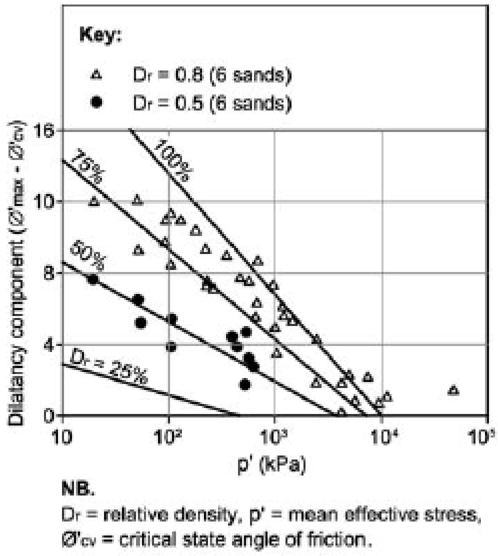
Source: Based on Stroud (32) and Bolton (31)

Key:

f = fine	q = quartz	rnd = rounded
m = medium	f = feldspar	s and = sub angular
c = coarse	s = some	s rnd = sub rounded

Dilatancy can be assessed from a knowledge of the relative density of the granular soil, and the appropriate effective stress level from Figure 3.10.1. In the past effective cohesion has been applied in analyses using granular materials (typically 1 kPa). Effective cohesion may be present in granular materials with fines contents greater than 15% but designers should use judgement when assessing the applicability of applying any cohesion in analyses based on classification tests of the material. It is recommended that for non plastic and low plasticity materials no cohesion is applied as these materials may have a high proportion of fines that are towards the coarse end of the fine particle range but will not necessarily exhibit cohesion.

Figure 3.10.1: Variation of Dilation (ϕ'_{max} - ϕ'_{cv}) versus Relative Density and Mean Effective Stress for Triaxial Shear (after Bolton, (31))



It is necessary to consider how ″clean″ the granular soil is, particularly when considering granular embankment fill which may have been contaminated during excavation and placement. The clay content of the fill should be reviewed to determine what proportion of the dilatancy should be used when deriving appropriate values of ϕ'_{peak}. If no specific strength testing (shear box or triaxial) of the soil has been carried out, the following relationships can be used:

- If clay fraction < 5%, full dilatancy can be assumed
- If clay fraction > 15%, no dilatancy should be considered; the Atterberg limits and soil classification should be checked to assess if it behaves as a coarse grained or fine grained soil (e.g. a low plasticity clay)
- For clay fractions of between 5 and 15%, linear interpolation should be used to assess the contribution of dilatancy to ϕ'peak.

Given the large number of different granular materials across the LU network provision of parameters for these materials is not deemed appropriate for the guide. For example, the British Geological Society (BGS) lists 8 different River Terrace Deposits that may be found across London (33). These are:

i. Sub-alluvial gravel

ii. Kempton Park Gravel

iii. Taplow Gravel

iv. Hackney Gravel

v. Lynch Hill Gravel

vi. Finsbury Gravel

vii. Boyn Hill Gravel

viii. Black Park Gravel

Given the variability of material between and within these deposits parameters should be derived on a site specific basis following the approach outlined in this section.

3.10.3 Groundwater

The current LU standard S1054 does not provide any specific guidance for porewater pressures to be used in the assessment of the stability of granular embankments or cuttings.

When deriving an appropriate groundwater regime for the site, a key consideration will be whether the granular soil can be relied upon to be of relatively high permeability (in excess of 5x10-6 m/s) i.e. has the granular material become contaminated with clay and/or silt.

However, even if the granular material is contaminated with clay and/or silt, the groundwater regime is unlikely to ever be more onerous than that for a clay fill embankment which is underdrained – see Figure 3.7.15 and Figure 3.7.16. Historically, an r_u of 0.1 has sometimes been applied in stability analysis of granular slopes if the clay/silt content is relatively low (say <10%), which equates to a piezometric line at the slope surface which is 20% hydrostatic.

Therefore, the appropriate groundwater regime will need to be assessed on a site by site basis, with it tending towards that for an underdrained clay fill embankment as contamination of the granular soil with clay and/or silt increases. Consideration should also be given to the nature of the soil within the top few metres of the slope surface – if this is more heavily contaminated then it may be more appropriate to apply higher porewater pressures in this zone when assessing shallow stability.

Unlike cohesive materials, high water demand trees will NOT provide a beneficial effect for winter groundwater pressures within granular slopes (embankments or cuttings), because the permeability of the soil is sufficiently high to allow rapid surface infiltration of rainfall. Hence, there is then reliance on gravity drainage of this infiltrated water, through the slope, to maintain low pore pressure. This should be considered, if there are any low permeability zones, towards the base of the granular soils which may lead to groundwater being "trapped", causing local increases in groundwater pressure.

3.10.4 Application of Surcharge Loading

The approaches outlined in Sections 3.6 and 3.8.4 should be followed for embankments and cuttings respectively.

3.10.5 Compliance with EC7 and LU Standards

The methodology outlined in Section 3.10.2 to derive soil parameters will result in characteristic or moderately conservative values. Therefore, partial factors on material properties should be applied in accordance with EC7 and LU standard

S1054, and do not require any modification i.e. a Partial Factor of 1.25 is applied for deep-seated slips, and a Partial Factor of 1.15 to shallow slips. As per Section 3.5.3, for granular cuttings in particular, consideration should be given to whether the partial factor for shallow slips should be increased depending on the perceived level of risk should such a slip occur; with a higher partial factor if there is a potential impact on lineside services or track, and a lower partial factor if not.

A partial factor of 1.3 shall be applied to both the train loading and surcharge applied to the crest of cuttings in accordance with LU standard S1054.

3.11 Chalk Fill Embankments

This section outlines the proposed method of analysis for embankments constructed from chalk fill, including giving guidance on the derivation of soil parameters, and the assessment of appropriate groundwater regimes.

There are several locations on the LU network where the embankments are constructed largely from Chalk Fill. These would have been constructed by end-tipping Chalk excavated from adjacent cuttings, with no significant compaction undertaken (34). More recent Chalk Fill embankments will have been constructed using compaction in accordance with either a method or performance specification.

Chalk is variable in composition, ranging from structureless Chalk (CIRIA Grade D) to strong intact white Chalk (CIRIA Grade A), and hence Chalk Fill will also be variable dependent on its source. Therefore in Section 3.11.2, guidance on the derivation of appropriate strength properties is provided, rather than the provision of absolute values. It will still be necessary to assess the site-specific nature of the Chalk Fill to determine appropriate parameters for analysis.

3.11.1 Method of Analysis

As for all other earth structures, both deep-seated and shallow failure mechanisms need to be assessed. Soil parameters for both should be derived following the guidance in Section 3.11.2, taking into account any variations in the nature of the near-surface material which may affect its strength. Porewater pressures should be derived for analysis following the guidance in Section 3.11.3. When assessing shallow failure mechanisms, as for clay earth structures, the contribution of the vegetation to stability should be considered, however if it is to be relied upon over a 120 year design life this needs to be highlighted as part of any maintenance plan. If the vegetation is to be removed then its contribution to shallow stability should be ignored. Shallow stability should be assessed for both circular and non-circular slips, typically 1.5m depth.

3.11.2 Chalk Fill Properties

In order to determine appropriate soil parameters it is necessary to review the likely source of the fill material as parameters will depend greatly on the source of the chalk – this will assist in determining the nature of the fabric of the Chalk fill. It is important to note that ground investigation methods such as window sampling, which are commonly used on the LU network, will not allow an accurate description of the Chalk Fill due to the remoulded nature of the samples. Trial pits will provide a better description of the Chalk Fill fabric as well as allowing visual inspection of the in-situ material.

The following should therefore form part of the process to classify the Chalk Fill:

- Review adjacent cuttings from which it is likely the Chalk Fill was derived. Can the grade of Chalk be reasonably estimated from a visual inspection of these?
- Trial pits should form part of the ground investigation to allow a better description of the Chalk Fill – this will assist in informing the Engineer about its possible source and therefore soil parameters.
- A walkover of the site may reveal some of the Chalk Fill characteristics from observations at the slope surface.

Based on CIRIA Report C574, angles of friction of remoulded Chalk (assuming c´ = 0kPa) have been found to vary between 29º and 34º, typically falling in the range 31-33º.

As discussed in CIRIA Report C574, the strength of remoulded Chalk has been seen to increase with time. This is considered to be due to the effects of drainage and evaporation after compaction leading to a decrease in moisture content, and also as a result of cementation which can occur due to the process of compaction releasing calcium carbonate. It is therefore considered that Chalk Fill will have a small cohesion as a minimum.

In the absence of site specific data the following conservative parameters are recommended for design:

- Angle of friction, ϕ' = 31 – 33º, dependent on source of Chalk Fill i.e. if it is believed to be from structureless Chalk then ϕ' = 31º, otherwise it will tend towards the higher end of the given range.
- Cohesion, c´ = 1-2kPa, again dependent on source of Chalk Fill i.e. if it is derived from structureless Chalk, c´ = 1kPa, otherwise it will tend towards the higher end of the given range.

Use of higher parameters than those outlined above may often be justifiable, however they would need to be proved by ground investigation and associated material testing (see Section 3.2 or other relevant evidence.

3.11.3 Groundwater

It is reported in CIRIA Report C574 that the permeability of remoulded Chalk can be low due to the significant proportion of fines generated during compaction. Therefore, an assessment of the clay content of the Chalk Fill may assist in understanding the likely groundwater regime. However, it is considered that this is only likely to impact the top 2m of the Chalk Fill embankment only, with porewater pressures at depth being very low. Therefore for shallow slips, if there is known to be significant clay and/or silt size fraction then porewater pressures should be taken as varying between 30% and 50% of hydrostatic over the top 1.5m below the slope surface depending on a visual assessment of the chalk fill (may vary from “blocky” to a chalk “putty”). Sensitivity analysis should be undertaken for shallow stability to ensure a robust design is proposed.

As for granular embankments and cuttings, when deriving an appropriate groundwater regime for assessing deep seated stability, a key consideration will be whether the Chalk fill can be relied upon to be of high permeability. Even if the Chalk fill is contaminated with clay and/or silt, the groundwater regime is unlikely to be more onerous than that for a clay fill embankment which is underdrained – see Figure 3.7.16 and Figure 3.7.16. At depth many chalk fill embankments will have near zero pore water pressures. Therefore, the appropriate groundwater regime will need to be assessed on a site by site basis, with it tending towards that for an underdrained clay fill embankment as contamination of the Chalk fill with clay and/or silt increases.

3.11.4 Application of Train Loading

The approaches outlined in Sections 3.6 and 3.8.4 should be followed for embankments and cuttings respectively.

3.11.5 Compliance with EC7 and LU Standards

The methodology outlined in Section 3.11.2 will result in the derivation of characteristic or moderately conservative soil parameters. Therefore partial factors should be applied to these as outlined in EC7 and LU standard S1054 for deep-seated analysis i.e. 1.25, and the modified partial factor of 1.15 for shallow slips in accordance with LU standard S1054. A partial factor of 1.3 shall be applied to the train loading in accordance with LU standard S1054.

3.12 Recycled Class 1A Fill

3.12.1 Introduction

Class 1A fill material is used across London Underground sites for slope re-profiling and construction of berms. It is composed of recycled material comprising natural gravel, natural sand, crushed gravel, crushed concrete and crushed rock. This document describes an approach by which the strength of Class 1A material at a known relative density and stress state may be determined. The strength framework proposed is based on the model described by Bolton (31), making use of shear box test data for Class 1A fill and strength data for granular materials from published literature.

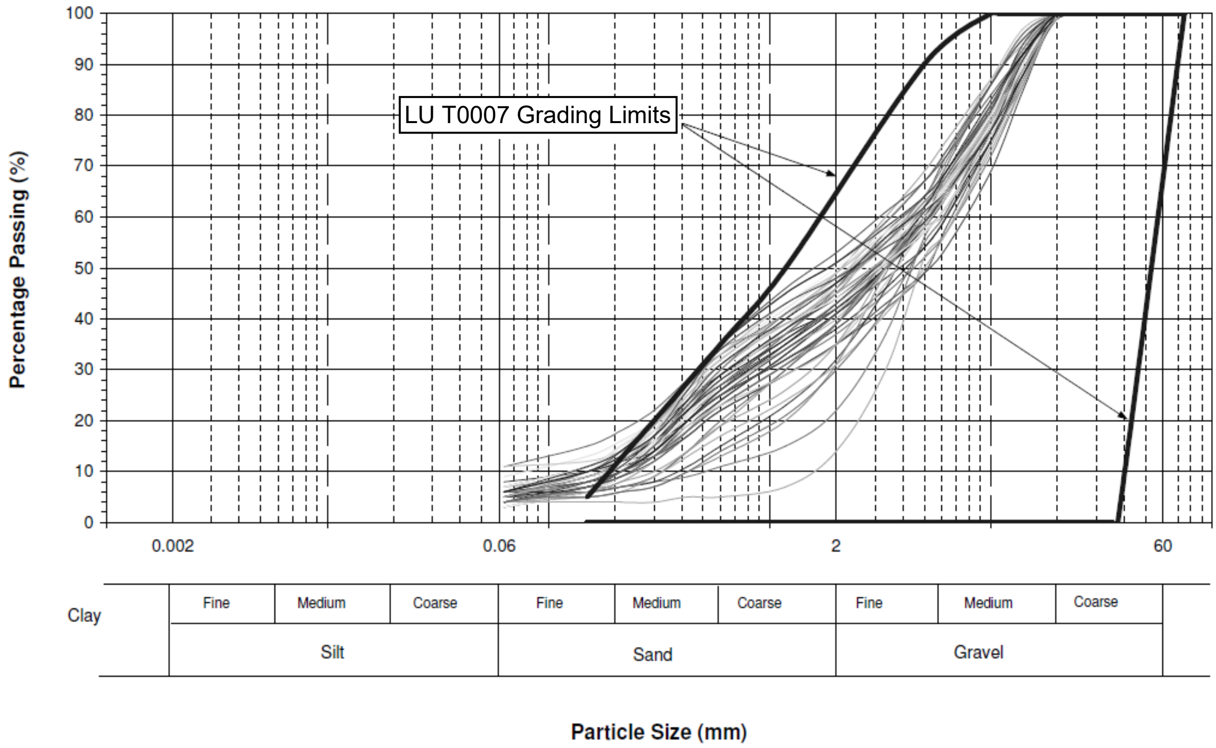
3.12.2 Index Properties and Compaction Characteristics

Class 1A fill can be described as Greyish brown sub-angular very sandy fine to medium GRAVEL of concrete, brick and rock (see Figure 3.12.1).

Figure 3.12.1: Photographs of Class 1A Fill Stockpile taken at Wimbledon Park Site

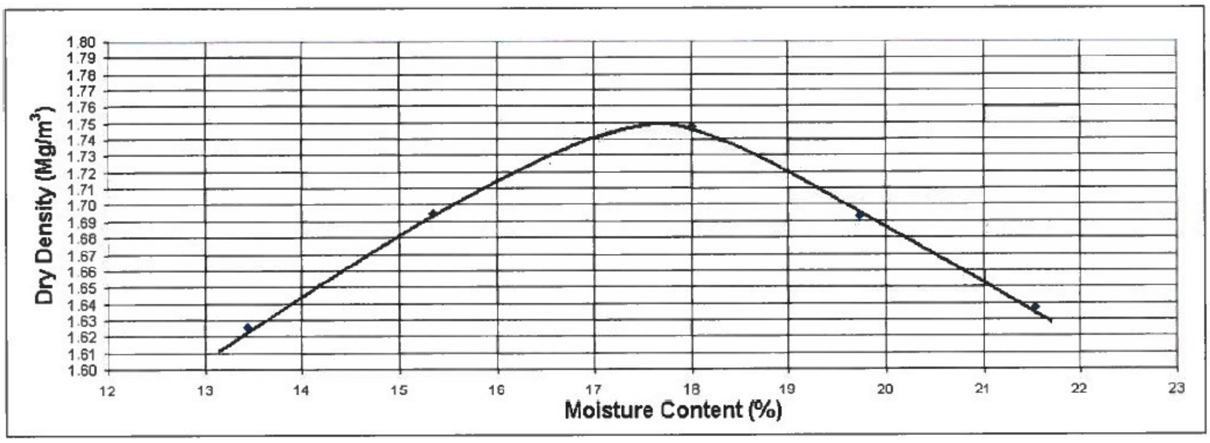
Grading curves obtained from samples taken from site stockpiles at Chalfont and Latimer to Amersham (shown in Figure 3.12.2) indicate a maximum particle size of 20mm, and a uniformity coefficient of between approximately 8 and 35. London Underground Technical Specification T0007 "Earth Structures Materials and Workmanship" (45) specifies grading requirements for Class 1A fill, shown as solid lines on Figure 3.12.2 For the majority of samples tested, the percentage of material falling within the "medium sand" to "coarse gravel" range complies with LU requirements. However, the samples generally contain a higher percentage of "fine sand" than specified in T0007 (up to 16% in the samples compared to a specified limit of 5%).

Figure 3.12.2: Class 1A Fill Grading Curves



The maximum dry density and optimum moisture content of Class 1A fill, obtained using the vibrating hammer method, are between 1.75 Mg/m3 and 1.84 Mg/m3 and 13% and 18% respectively. Results of a compaction test on Class 1A fill taken from the stockpile at the Wimbledon Park site are shown in Figure 3.12.3.

Figure 3.12.3: Results of Compaction Test on Class 1A Fill taken from Wimbledon Park Stockpile



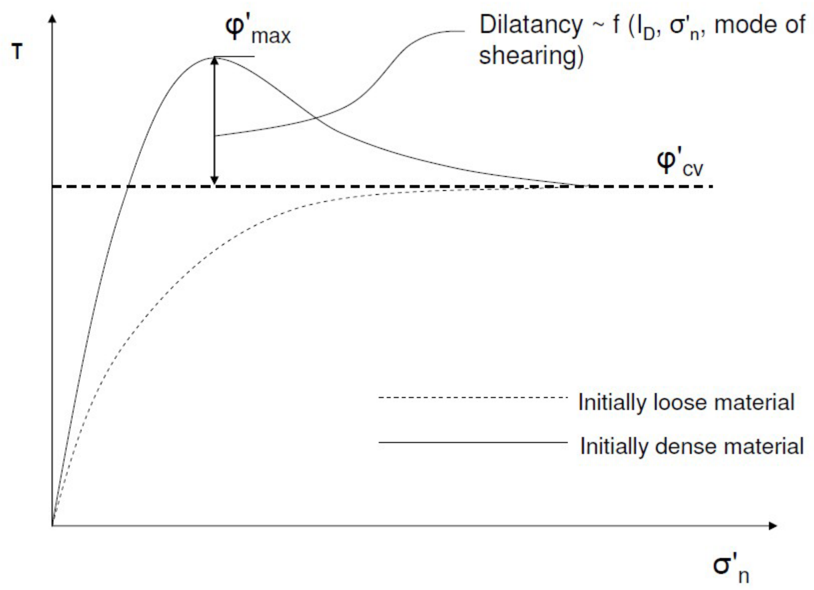
Optimum Moisture Content (%): 18 **Maximum Dry Density (Mg/m³):** 1.75

Particle density testing performed on two samples taken from the Wimbledon Park site stockpile indicate an average particle density for the Class 1A fill of 2.53 Mg/m^3.

3.12.3 Shear Strength Framework for Coarse Grained Materials

A strength framework for cohesionless soils was proposed by Bolton in his 1986 paper “The strength and dilatancy of sands.” It can be considered that the shear strength of a granular material is composed of two elements – the strength resulting from friction between individual particles moving parallel to and in contact with one another, and the strength arising due to particle rearrangement when densely-packed particles are forced to override one another. It is known that a loosely compacted sample, which is able to deform without particle overriding, is likely to contract on shearing, deriving its shear strength from particle-particle friction only. In contrast, a densely compacted sample is likely to experience dilation on shearing, deriving shear strength from a combination of particle-particle friction and particle overriding. The two types of shear behaviour are shown schematically in Figure 3.12.4 for initially loose and initially dense samples.

Figure 3.12.4: Shear behaviour for Contractile (Initially Loose) and Dilative (Initially Dense) Samples



3.11.3.1 Critical State Friction Angle

The component of shear strength resulting from particle-particle friction is known as the critical state friction angle, φ'_{cv}. The critical state friction angle is the minimum shear strength of a given material, irrespective of the density or stress state of the sample. Φ'_{cv} is related to intrinsic properties of the material including uniformity, particle angularity and mineralogy. For uniformly graded fine sands, typical critical state friction angles for various mineralogies are given in Table 3.1 of Randolph, Jamiolkowski and Zdrakovic (47).

Table 3.12.1: Uniformly Graded Fine Sands. Critical State Friction Angles, Influence of Different Mineralogy (Randolph, Jamiolkowski and Zdrakovic, 2004)

Mineralogy	Range of φ'_{cv}
Siliceous	33° – 34°
Quartz	32° – 34°
Calcareous	40° – 42°

Guidance is also given in BS 8002:2015 “Code of Practice for Earth Retaining Structures” for estimation of critical state friction angle for siliceous sands and gravels based on angularity and grading. Equation (1) is suggested, with constants “A” and “B” as given in Table 3.12.2. Use of Equation (1) suggests a critical state friction angle of 36° for sub-angular, well graded sands and gravels.

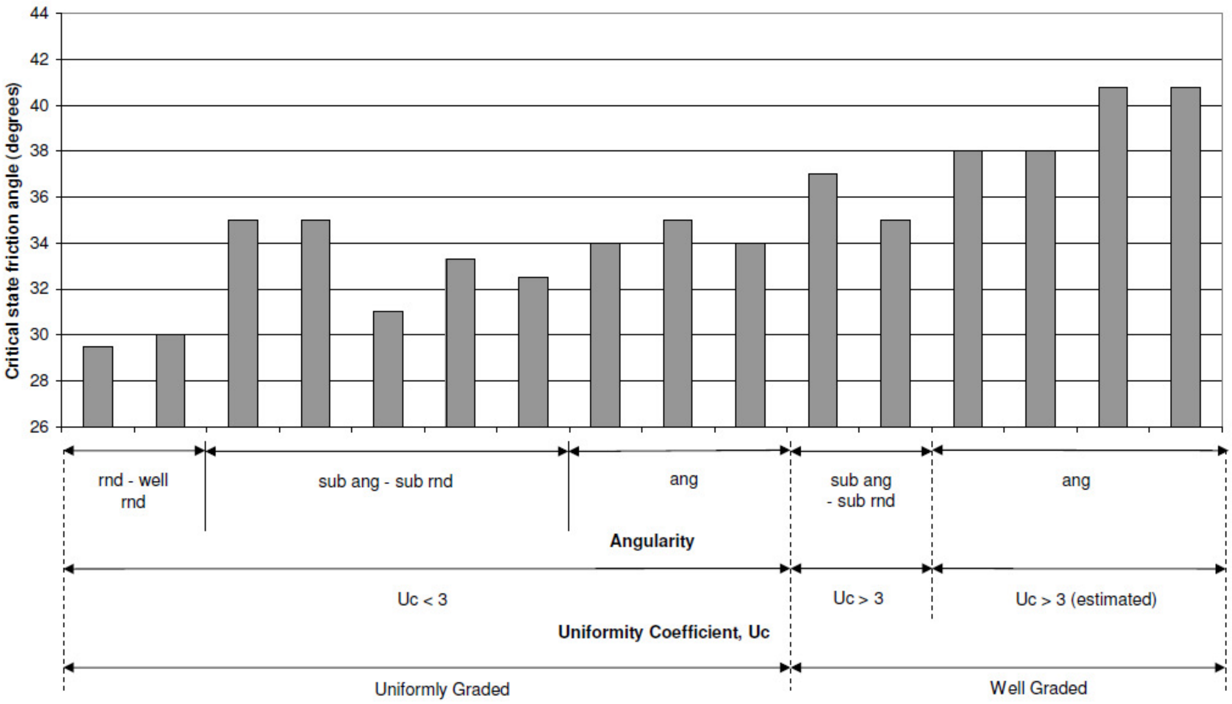
$\Phi'_{cv} = (30 + A + B)°$ - Equation (1)

Table 3.12.2: Guidance for Estimation of φ'cv Based on Angularity and Grading (after BS 8002:2015 "Code of Practice for Earth Retaining Structures")

A – Angularity	A (degrees)
Rounded	0
Sub-angular	2
Angular	4
B – Grading of Soil	**B (degrees)**
Uniform (U_C < 2)	0
Moderate grading (2 < U_C < 6)	2
Well graded (U_C > 6)	4

Study of published literature reveals that φ'cv for a granular material is rarely less than 30°and can be over 40°for angular, well-graded materials. Figure 3.12.5 presents $φ'_{cv}$ values from published literature, Bolton (31) & Stroud (48) for 16 materials in terms of grain angularity and uniformity coefficient. The test data for 3 uniformly graded very angular sands is not plotted, but these gave $φ'_{cv}$ values of between 36°and 42°. It can be seen that $φ'_{cv}$ tends to increase significantly (by about 5°) with increasing particle angularity. The well-graded sands and gravels tended to exhibit higher values of φ'cv than the uniformly graded sands; typically with $φ'_{cv}$ values being 2°to 4°higher if well-graded.

Figure 3.12.5: Variation of Critical State Friction Angle with Uniformity Coefficient and Grain Angularity from Published Literature (rnd = Rounded, ang = Angular)



3.11.3.2 Peak Friction Angle and Angle of Dilation

The component of strength resulting from particle rearrangement and overriding is known as the angle of dilation, ψ and can be considered as the difference between the peak friction angle and the critical state friction angle ($\varphi'_{max} - \varphi'_{cv}$). As indicated on Figure 3.12.6 and Figure 3.12.7 a loosely compacted sample may have angle of dilation equal to zero, whereas a densely compacted sample of the same material may have high angle of dilation, in excess of 10°at low effective confining stresses (less than 100kN/m2).

Conversely, the same densely compacted sample may experience no dilation, and exhibit shearing behaviour similar to that of the loose sample, if the vertical stress applied to the sample during shearing is sufficiently high that overriding of particles is prevented. (Although confining stresses in excess of 5000 kN/m^2 are required to minimise dilation – stresses of this magnitude are not relevant for the envisaged slope engineering applications). It follows that the angle of dilation is dependent on the compaction, stress state and mode of shearing of the sample.

Figure 3.12.6 after Bolton (31) shows the angle of dilation (= $\varphi'_{max} - \varphi'_{crit}$) obtained during triaxial and plane strain shear testing of sands compacted to different relative densities. Samples were tested at effective stresses of between 150 and 600kPa. Figure 3.12.7 (after Bolton, 1986) shows the angle of dilation (= $\varphi'_{max} - \varphi'_{cv}$) obtained during triaxial testing of sands at different mean effective stresses. Figures 3.12.6 and 3.12.7 demonstrate that the angle of dilation increases with increasing relative density and decreases with increasing mean effective stress.

Figure 3.12.6: Angle of Dilation vs Relative Density (after Bolton, 1986) at a Mean Effective Stress of about 300kN/m2

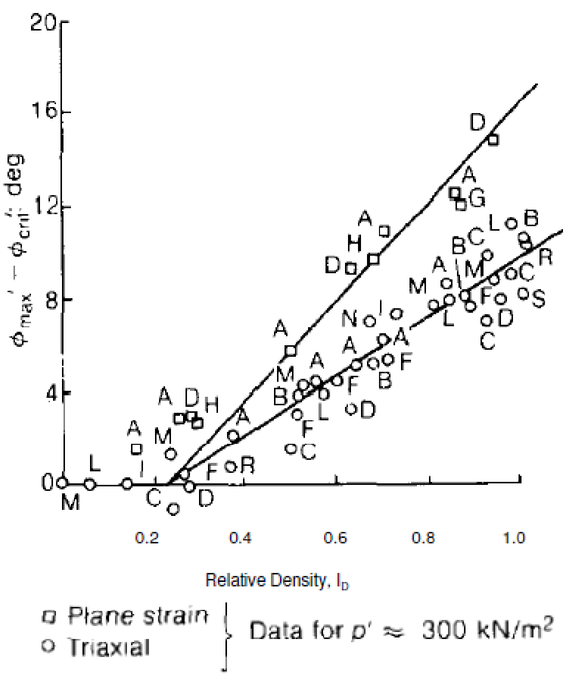


Figure 3.12.7: Angle of Dilation vs Mean Effective Stress (after Bolton, 1986) for Triaxial Tests

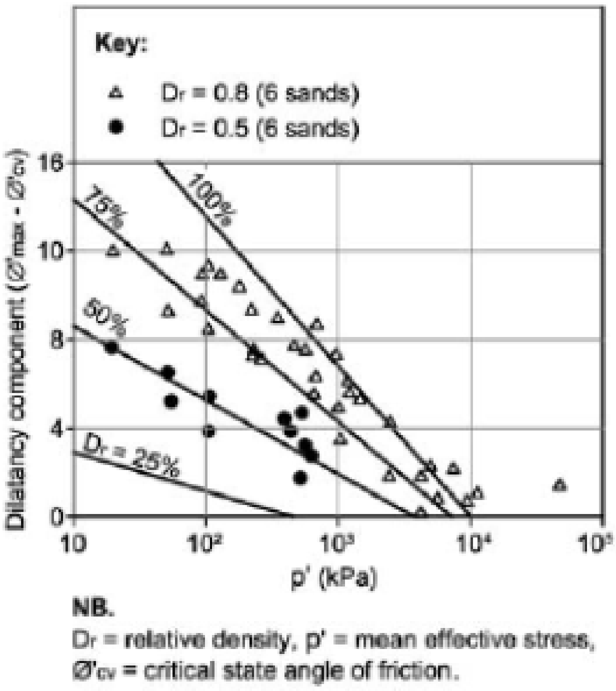


Figure 3.12.6 indicates that at relative densities, ID, in excess of 0.8, plane strain shearing exhibits dilation angles about 4°larger than those for triaxial shear. The data in Figure 3.12.7 shows that for triaxial shear, dilation in excess of 8°would be anticipated at mean effective stresses lower than 100kN/m^2, and for relative densities in excess of 0.8.

3.12.4 Shear Box Test Results

Six no. shear box tests were performed on samples of Class 1A fill taken from three London Underground sites. Samples were tested at 90-94% maximum dry density (where "maximum" is based on vibrating hammer compaction tests) and optimum water content under effective normal stresses of 50, 100 and 200kPa.

Table 3.12.3: Results of Class 1A Fill Shear Box Testing

Sample Description	Max Dry Density, ρmax	Optimum Moisture Content	Initial Bulk Density, ρ	Initial Moisture Content, w_o	Initial Dry Density, ρd	Relative Density*, ID	Normal Applied Stress, σ'n	Peak Shear Stress, τ	Secant Friction Angle. Φ'max
	Mg/m³	%	Mg/m³	%	Mg/m³		kN/m²	kN/m²	Degree
Crushed Rock (Crushed Concrete) Upney to Becontree	1.79	13	1.85	12.9	1.64	69	50	58.6	50
			1.85	12.9	1.64	69	100	101.3	45
			1.85	13.1	1.64	69	200	179.4	42
Brown Sand & Gravel(Crushed Concrete) Chalfont & Latimer to Amersham	1.84	15	1.94	15.1	1.68	65	50	62.1	51
			1.94	15.2	1.68	65	100	127.8	52
			1.94	15.2	1.68	65	200	191.7	44
Brown Sand & Gravel(Crushed Concrete) Chalfont & Latimer to Amersham	1.79	16	1.90	15.8	1.64	69	50	68.9	54
			1.91	15.9	1.64	69	100	120.5	50
			1.91	15.9	1.64	69	200	190.6	44

Sample Description	Max Dry Density, ρmax	Optimum Moisture Content	Initial Bulk Density, ρ	Initial Moisture Content, w_o	Initial Dry Density, ρd	Relative Density*, ID	Normal Applied Stress, σ'n	Peak Shear Stress, τ	Secant Friction Angle. Φ'max
	Mg/m^3	%	Mg/m^3	%	Mg/m^3		kN/m^2	kN/m^2	Degree
Brown Sand & Gravel(Crushed Concrete) Chalfont & Latimer to Amersham	1.76	17	1.89	17.1	1.61	69	50	61.6	51
			1.89	16.9	1.62	72	100	110.4	48
			1.89	16.9	1.62	72	200	177.2	42
Crushed Concrete & Brick (Crushed Concrete) Wimbledon Park	1.75	18	1.89	18.3	1.60	70	50	73.1	56
			1.89	17.3	1.61	72	100	121.3	50
			1.89	18.7	1.60	70	200	194.5	44
Crushed Concrete & Brick (Crushed Concrete) Wimbledon Park	1.75	18	1.89	17.8	1.61	72	50	67.5	53
			1.89	18.0	1.61	72	100	113.8	49
			1.89	17.9	1.61	72	200	183.6	43

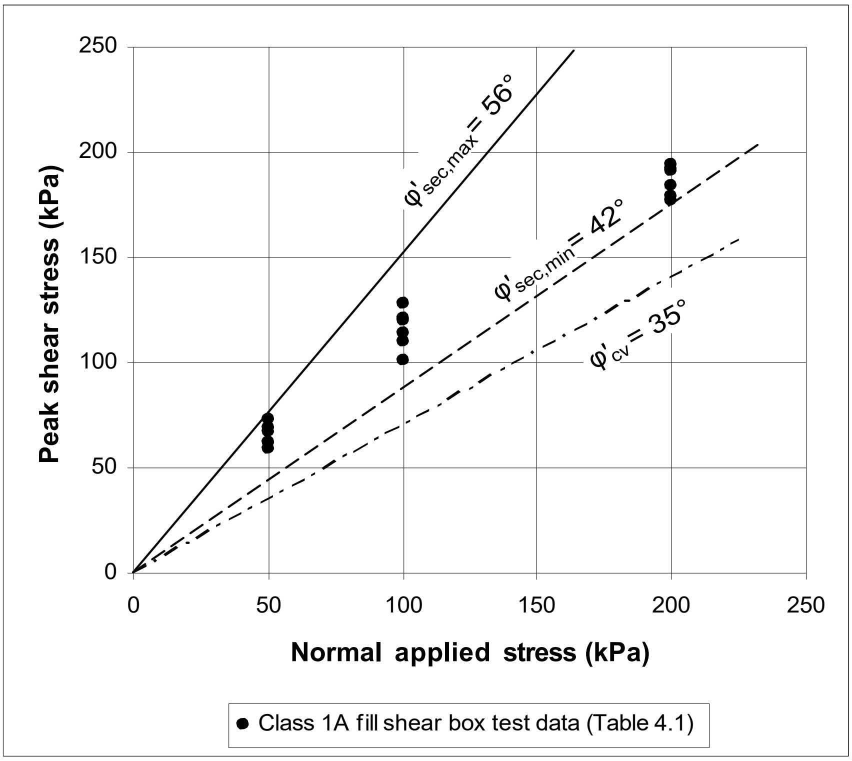
* Relative Density, $I_D = (e_{max} - e_0)/(e_{max} - e_{min})$

Where: e_0: initial voids ratio = $((\rho_s / \rho_d) - 1)$, where ρ_s is the particle density, equal to 2. 53 Mg/m^3 result from tests on 2 No. samples taken from Wimbledon Park site stockpile)

e_{min}: minimum voids ratio = $((\rho_s / \rho_{max}) - 1)$; and e_{max}: is the maximum voids ratio, assumed = 2 x e_{min} (Youd (49)).

Peak shear stresses obtained from the shear box tests are plotted against applied normal stress in Figure 3.12.8. The maximum and minimum peak secant friction angles are 56°and 42° respectively. A representative critical state friction angle of 35°is also shown for comparison.

Figure 3.12.8: Peak Shear Stresses Obtained during Shear Box Testing of Class 1A Fill



Published peak strengths obtained for gravels during triaxial and direct shear tests (Herle et al (46)) are presented in Figure 3.12.9a for comparison with the Class 1A fill shear box test results. Figure 3.12.9b presents the same data, limited to tests conducted under normal applied stress of less than 200kPa, and showing the maximum and minimum secant peak friction angles (72°and 42°respectively). The results for Class 1A fill appear to compare reasonably well with the published data.

Figure 3.12.9a: Peak Shear Stresses Obtained During Shear Box Testing of Class 1A Fill and Published Peak Shear Stresses for Gravel (Herle et al (46))

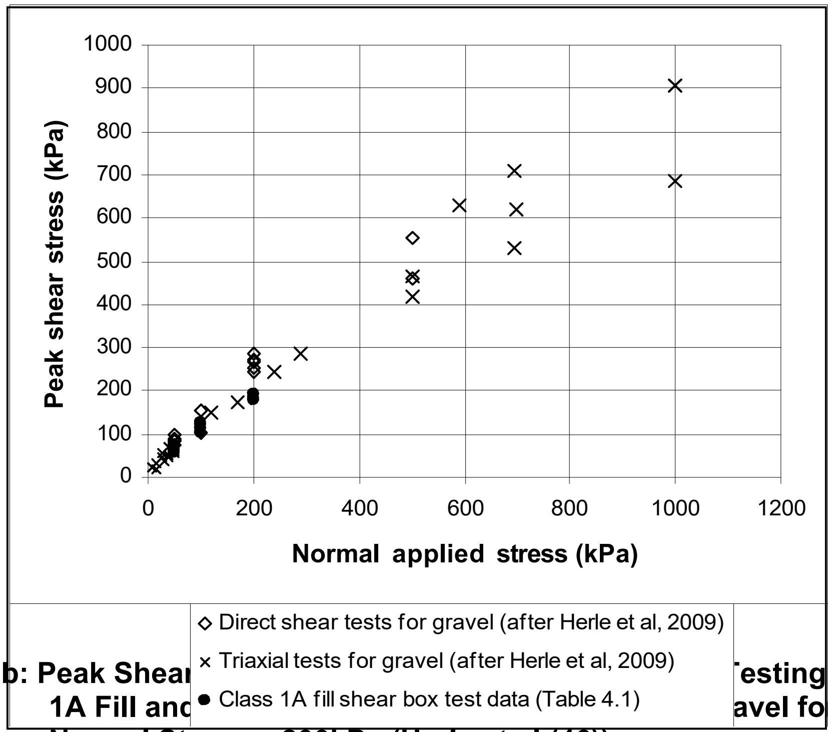
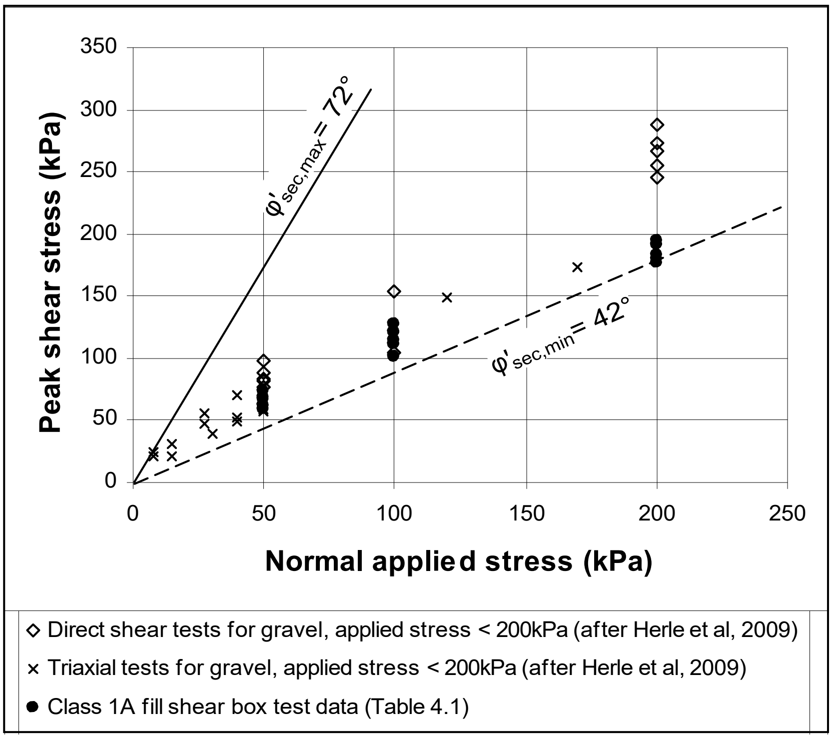


Figure 3.12.9b: Peak Shear [...] Testing of Class 1A Fill and [...]avel for Applied Normal Stress < 200kPa (Herle et al (46))



3.12.5 Data Interpretation

Bolton (31) proposed the following empirical relationships linking dilative strength with compaction and stress state:

$IR = ID . (Q – ln(p'))$ - Equation (2)

$\varphi'_{max} – \varphi'_{cv} = m.IR$ - Equation (3) Where,

IR: relative density index; ID: relative density;

p': mean effective stress (conservatively assumed equal to vertical effective stress for shear box tests);

Q: constant, typically between 8 and 11;

φ'_{max}: peak friction angle;

φ'_{cv}: critical state friction angle; and

m: constant equal to 5 for plane strain conditions and 3 for triaxial conditions. Equations (2) and (3) are valid over the range $0 < IR < 4$.

Q is dependent on the "crushability" of the soil grains and is lower for crushable materials such as calcareous sands or chalk and high for strong grains such as quartz. Q becomes more important at high stress levels, since grain crushing will suppress dilation.

A critical state friction angle equal to 35°has been adopted for the Class 1A fill, based on the data presented in Figure 3.12.5. It is believed that this is a conservative estimate of the likely critical state friction angle of a well-graded, angular material such as Class 1A fill. The true critical state friction angle of the fill should be verified by shear box tests on a set of loose samples.

Figure 3.12.10 and Figure 3.12.11 present the shear box test results for Class 1A fill, assuming a critical state friction angle of 35°, in addition to published data for granular materials tested in direct shear (Bolton (31)). The relationships defined by Equations (2) and (3) are plotted twice for m = 3 and m = 5 over the range 0 < IR < 4. The test data plotted was obtained under plane strain conditions and should be compared with the relationships presented for m = 5. The relationships presented for m = 3 are suitable for triaxial conditions and are shown for comparison purposes only.

Figure 3.12.10: Dilative Strength vs Vertical Effective Stress for Constant Initial Relative Density, ID = 0.8 (Densely Compacted)

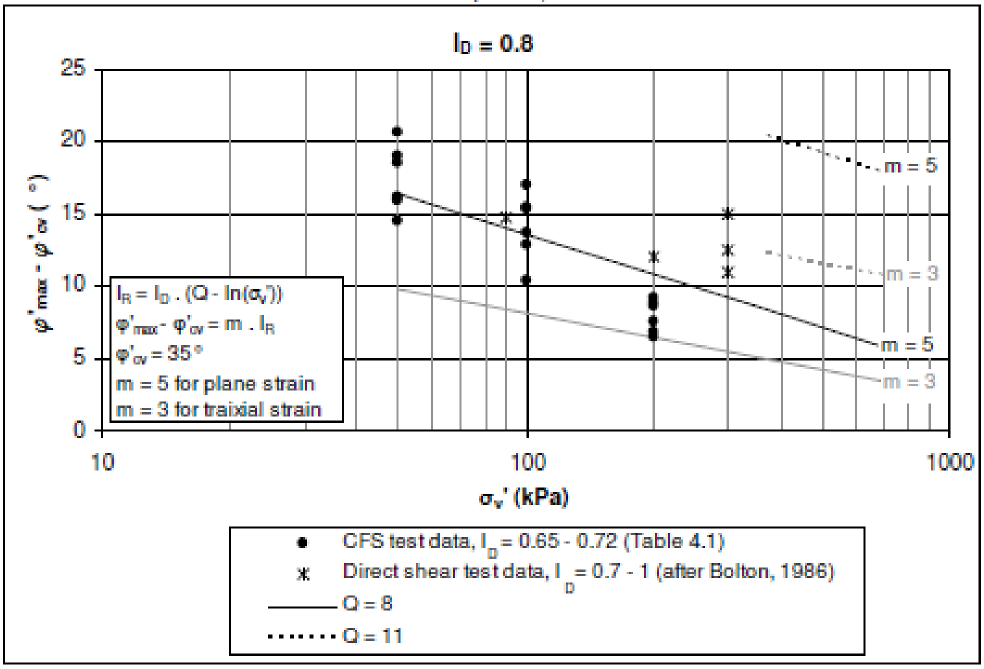
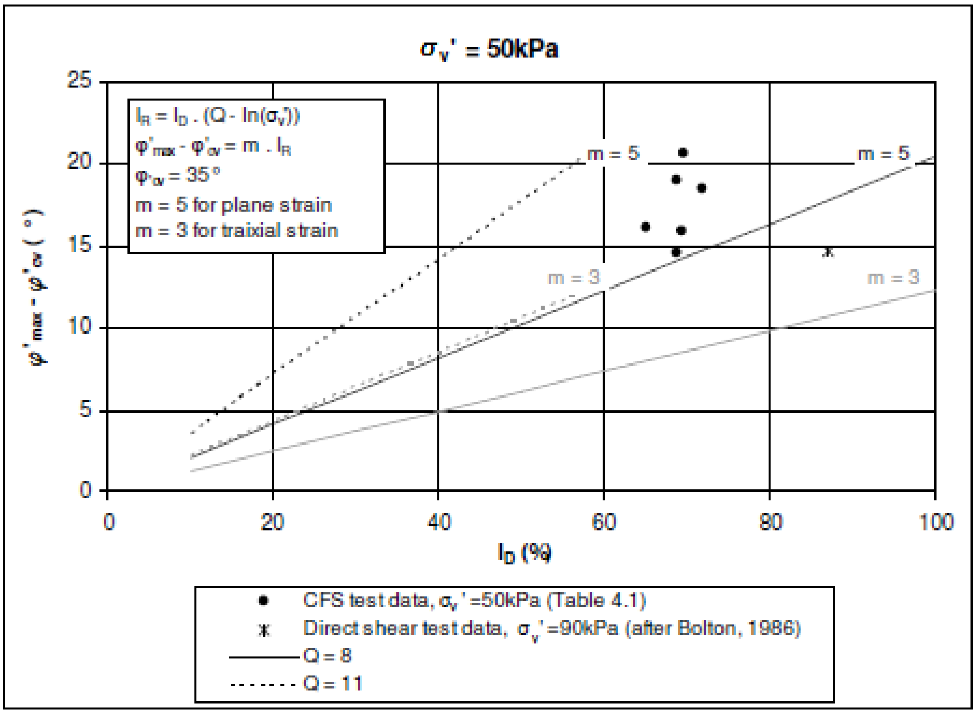


Figure 3.12.11: Dilative Strength vs Initial Relative Density for Constant Vertical Effective Stress, σv' = 50kPa (Approximate Maximum Working Stress of Class 1A Fill)



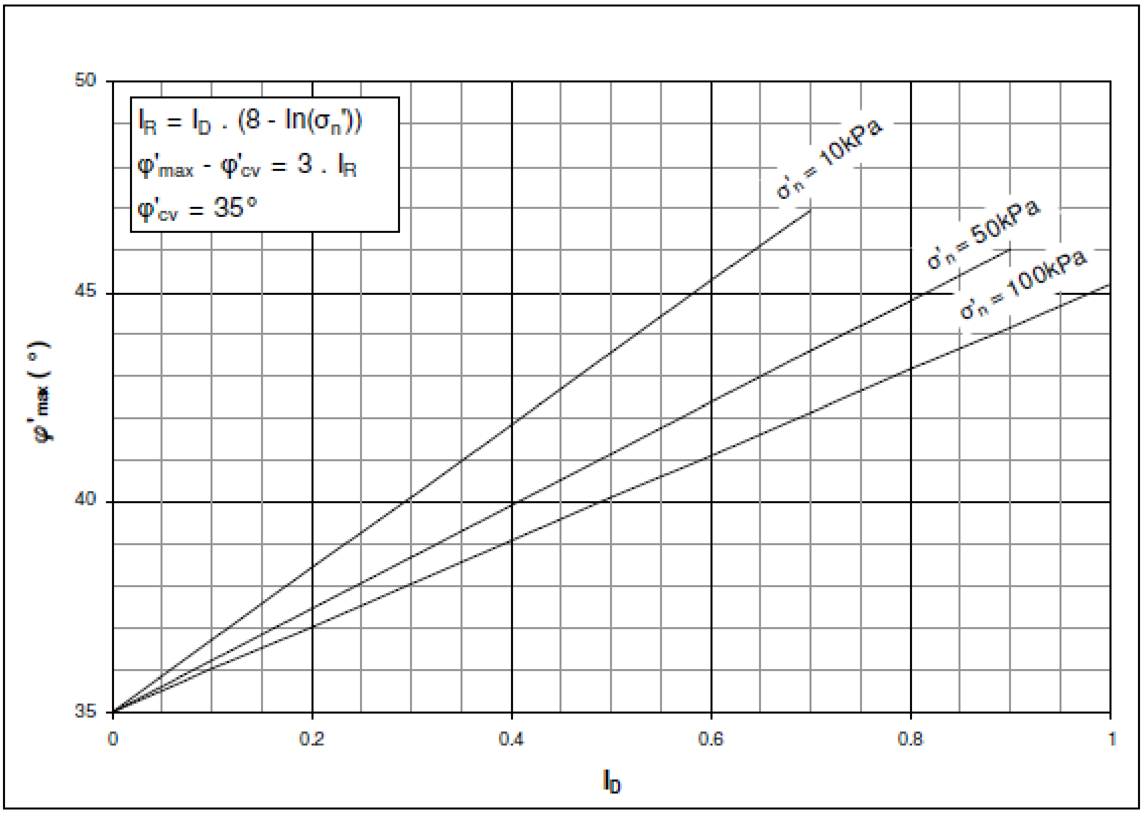
3.12.6 Recommendation for Design

As indicated above, m = 5 is recommended for plane strain failure. Therefore, m = 5 is appropriate for comparison with the results of shear box testing. Slope failure can also often be considered a plane strain failure mode, however, for design it is recommended that m = 3 be conservatively adopted, which is suitable for triaxial shear conditions.

Based on Figure 3.12.10 and Figure 3.12.11, it is recommended that Q = 8 be adopted for design, with m = 3 as discussed above. Φ'cv should be determined by testing on a representative set of samples, however, for initial design a value of 35° may be assumed.

Figure 3.12.12 presents proposed design lines for the peak strength of Class 1A fill for vertical effective stresses of 10kPa, 50kPa and 100kPa, assuming m = 3, Q = 8 and φ'cv = 35°. From Figure 3.12.12 it can be seen that peak friction angles in excess of 40°(the currently assumed value for design) can be readily mobilised for fills which are medium dense or denser (i.e. ID > 0.5). For Class 1A fills which meet the compaction criteria (i.e. ID > 0.8), friction angles in excess of 43°would be anticipated.

Figure 3.12.12: Proposed Design Lines for Peak Strength of Class 1A Fill



3.12.7 Limitations of Shear Strength Model

The following limitations apply to the proposed shear strength framework:

1. Equations (2) and (3) (presented in Figure 3.12.12) are valid for $0 < I_R < 4$ only. This condition may not be satisfied for materials with very high relative density and very low applied stress. For densely placed Class 1A fill under low applied vertical effective stress, where $I_R > 4$, it is recommended that the peak friction angle is calculated assuming $I_R = 4$, corresponding to an upper-bound dilation equal to 12°. This approach will result in a conservative estimate of φ'_{max}, the true value of which will be very high (> 45°) under conditions of low stress and high density.
2. The proposed model conservatively neglects the increase in cohesive strength of Class 1A fill that may occur over time due to cementitious fines present in the material.
3. Further testing is recommended to confirm:
   - Critical state friction angle, φ'_{cv} ; and
   - Maximum and minimum void ratio, e_{max} and e_{min}

3.12.8 Conclusions

Published research (Bolton, 1986) demonstrates that the peak strength of a granular material depends not only on the critical state friction angle, which is related to material properties such as uniformity and grain angularity, but also on the angle of dilation, which depends on the compaction state of the material, the mean effective stress at failure and the mode of shearing. The angle of dilation has been shown to increase with increasing relative density and with decreasing effective stress.

Across London Underground sites, Class 1A fill is used for slope re-profiling and construction of berms and as such is unlikely to be subject to large overburden pressures. Furthermore, it is a London Underground requirement that the fill be compacted to at least 95% of maximum dry density (Table 5.3, London Underground Technical Specification T0007 (45). Therefore, Class 1A material is likely to mobilise shear strengths in excess of 45°in many London Underground fill applications. Conservatively an angle of friction of 43°could be used for well-compacted, well-graded sub-angular to angular sands and gravels (typical of crushed concrete fill). The current assumed angle of friction of 40°should be readily mobilised with only moderate compaction. The above angles of friction only apply for fills at low confining stresses (typically 50kN/m2 or less) which are typical of fills behind low-height retaining walls and for slope regrading.

3.13 Shoulder Instability (Embankments)

This section discusses shoulder instability, which can affect many LU embankments. The mechanism of deformation/failure can be very different to deeper seated failure mechanisms.

Commonly shoulder instability is dependent on the behaviour of the near surface soils adjacent to the crest, which typically comprise Ash or Ballast, together with variable quantities of fines which have "contaminated" the ash/ballast layers. Often the ash/ballast is in a loose to very loose state.

3.13.1 Ash Fill

A review of the strength and dilatancy of railway Ash was undertaken by MM in the Report ″LUL Research II, Strength and Dilatancy of Railway Ash, Doc. No: 51683/REP/F&G/400/B, dated July 1999″. This review covered the following aspects:

- Review of crushable soils
- Review of shear strength of Railway Ash, including data from a number of sites on the LU network
- Recommendations for testing and strength parameters for design
- A key point is the recognition that at low effective stresses sands can exhibit strengths higher than those at critical state due to the effects of dilatancy (31).

The key conclusions/recommendations from the report relevant to this design guide are:

- Critical state strengths should generally be used in slope stability analysis
- If no site-specific laboratory testing is available, a value of 39º is recommended
- As the value recommended is based on critical state strength, lower values should not occur. This should be considered when deriving appropriate partial factors.

Historically, MM have adopted the approach of applying an angle of friction equal to 40º to the top 1m of Ash due to the low effective stresses present, and the LU standard value of 35º below this depth.

It is recommended that a friction angle of 39° is applied to the Ash in stability analysis. Where the depth of Ash is greater than 2m, consideration should be given to the impact of applying a friction angle of 35° to the Ash below this depth on the stability analysis.

The partial factor to be applied to the strength of the Ash is governed by the failure mechanism being considered as discussed in previous sections of this design guide.

3.13.2 Deformation/Failure Mechanisms

Several different mechanisms can adversely affect embankment shoulders, e.g.:

(i) Ravelling

(ii) Wash Out

(iii) Composite Failure

(iv) Animal Burrowing

(i) Ravelling – typically this failure mechanism is most prevalent during hot/dry summers, when vibration (from passing trains) leads to the loose coarse grained soils moving progressively downslope. This mechanism cannot be reliably analysed by using conventional slope stability checks; fundamentally “ravelling” is due to the combination of the cohesionless particulate nature of the soil, very low confining stress (less than 10kN/m^2) and train induced vibrations “shaking” the particles downslope. These initially minor downslope movements can incrementally, over time, lead to serious undermining of lineside services, and a loss of lateral support to the track. The fines content within the ash/ballast can facilitate beneficial suctions within the soil (over winter/spring/autumn periods, and relatively wet summers) when it is moist. These suctions provide a temporary “cohesion”, which maintains stability. During relatively dry summers, the ballast/ash can dry out completely which then leads to a loss of suction (or “cohesion”) and ravelling downslope. Remedial measures to reduce the risk of ravelling, normally involve providing a combination of compaction and confinement to the ash/ballast. Experience indicates that the provision of a granular capping layer, which is benched and compacted over and adjacent to the ash/ballast, can be effective in preventing future ravelling.

(ii) Wash Out – this is mainly a risk for embankments on side-long ground and where poor surface drainage enables concentrated water flows to occur across the embankment surface. This can lead to wash-out failure of poorly graded granular soils, during/immediately after heavy rainfall. High risk locations include area in the vicinity of embankment/cutting interfaces. The construction of cut-off drains, hydraulic barriers and erosion control matting would normally need to be considered to remediate these slopes.

(iii) Composite Failure – “composite” failures, are failures through layers of coarse and fine-grained soils (typically ash/ballast over clay fill). Often these failure mechanisms would be non-circular, with an overall “wedge” shape; sub-horizontal through clay, and steep active wedge through ash/ballast. The failure through the clay tends to be shallow, and may be sensitive to the geometry of the ash/clay interface. This type of mechanism is amenable to limit equilibrium analyses, with parameter selection based on Chapter 3.5 and 3.9, for fine grained and coarse grained soils respectively, together with the groundwater guidance in Chapter 3.5.

(iv) Animal Burrowing – animal burrowing through ash can exacerbate loosening of granular layers, and lead to the site becoming more vulnerable to wash out and ravelling failure mechanisms.

3.13.3 Remedial Measures

The above failure mechanisms are very different to the deep seated failure mechanisms which are commonly investigated in slope stability analyses.

Remedial measures will need to be bespoke, specifically addressing site specific observations. Engineering judgement is important. The creation of a footpath along the crest may also be required. Although specific circumstances may vary, some common features are:

(i) Providing a compacted cover layer – many of the shoulder support problems are associated with very loose granular layers. Hence, compaction, and creation of a dense well graded granular capping over loose layers may be effective.

(ii) Confinement – stress levels close to the surface of ash/ballast layers tend to be very low (less than 10kN/m2). This means the cohesionless soils are more vulnerable to vibration. Capping by a granular layer, can assist by increasing stress levels in the existing ash/ballast. Alternatively crest retaining walls (provided they can be founded on more competent layers at depth), may also be effective.

(iii) Drainage/Erosion Protection – if the surface soils are silt or sand size and uniformly or gap graded, they can be very vulnerable to erosion by surface water flows (during/after intense rainfall events). Hence, erosion matting and/or crest drainage may be effective. An assessment of existing drainage systems – are they leaking? Or are they blocked? Is also an additional consideration. Hence, CCTV surveys, and remediation/repair of existing drains may be an important stabilisation measure. Outfalls may be inadequate, and may need to be upgraded.

3.14 Serviceability of Clay Embankments

3.14.1 Serviceability

The serviceability performance of LU earth structures can be adversely affected by vegetation induced seasonal shrink-swell movements. For embankments composed of medium or high plasticity clay fills and where there are high water demand trees close to the track seasonal movements may require regular track maintenance and reballasting. A series of simple guidelines for the management of vegetation adjacent to LU structures was produced by Mott MacDonald in 2008, entitled ″LUL Vegetation Management Guidelines, Preliminary Guidelines for Vegetation Management Adjacent to London Underground Track and Structures″.

3.14.1.1 Seasonal Shrink-Swell

Seasonal shrink-swell movements are a result of the seasonal changes in moisture contents of the soil. The term seasonal changes refer to those changes in moisture content that usually follow an annual cycle, with the soil drying in the summer (resulting in settlement) and rehydrating in the winter (resulting in heave). The magnitudes of these seasonal changes are controlled by a number of factors including the amount of precipitation, runoff/evaporation, wind speed, soil type, vegetation type and drainage conditions.

The removal of water from a soil by vegetation can create a zone of desiccation to a considerable depth, resulting in shrinkage of the soil and a general downward movement. The removal of a tree or the reduced water demand associated with a tree's dormant winter months can give rise to swelling and upward ground movements.

Climate change will lead to increases in the magnitude of dry-wet cycles. This will drive greater shrink-swell behaviour and may increase desiccation cracking (See Section 3.9), leading to greater serviceability concerns for clay embankments.

3.14.1.1.1 Influence of Vegetation Type

Different types of vegetation can significantly affect the magnitude of track and embankment deformation. This is illustrated by Scott (35) where instrumentation was installed in a high plasticity index clay fill in two parts of the slope – one area was a predominantly grass covered slope, whilst the other area was close to several high water demand (HWD) trees. Table 3.14.1 shows the monitoring data. It can be seen that embankment deformation was more than ten times higher in the area affected by HWD trees. As SMD increased during the summer, settlement occurred, while swelling occurred when SMD reduced during winter. Track movement follows a similar pattern to the crest deformation.

Figure 3.14.1: Embankment Deformation, Grass Covered versus Mature HWD Tree Covered Slope (after Scott, (35))

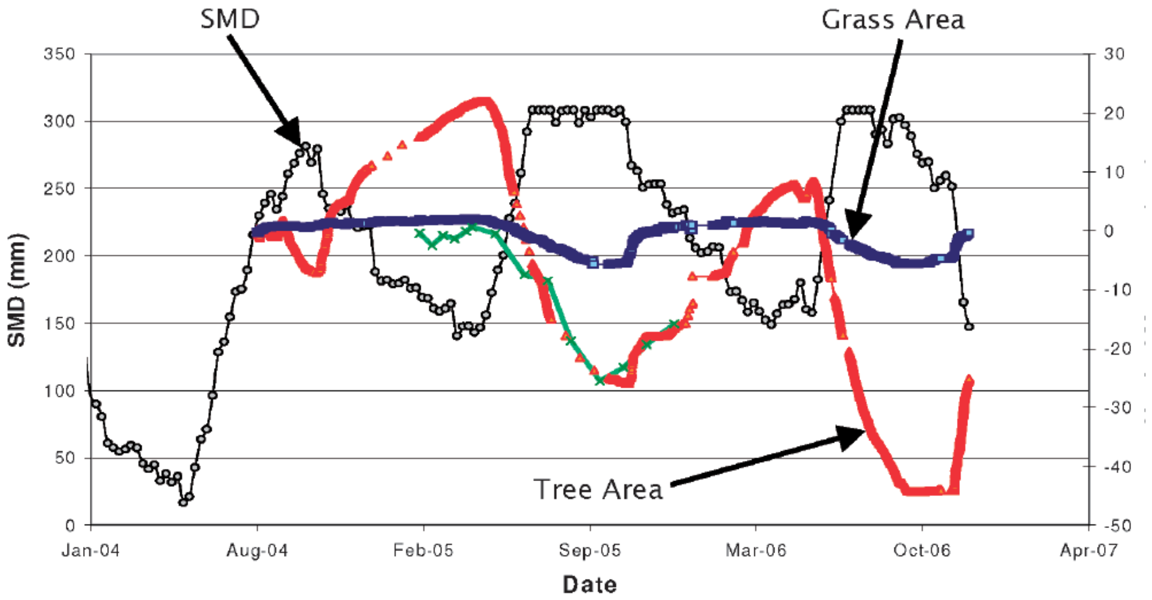


Table 3.14.1 lists the water demand and mature height of tree species based on the National House Building Council (NHBC) classification. The most significant serviceability problems across the LU network have been caused by Oaks and Poplars (and are sometimes referred to as “very high water demand”).

Table 3.14.1: NHBC Classification of Water Demand for Various Tree Species (NHBC, (41))

Broad Leafed Trees		
Water Demand	**Species**	**Mature Height (m)**
High	Elm	
	English	24
	Wheatley	22
	Wych	18
	Eucalyptus	18
	Hawthorn	10
	Oak	
	English	20
	Holm	16
	Red	24
	Turkey	24
	Poplar	
	Hybrid Black	28
	Lombardy	25
	White	15
	Willow	
	Crack	24
	Weeping	16
	White	24
Moderate	Acacia false	18
	Alder	18
	Apple	10
	Ash	23
	Bay Laurel	10
	Beech	20
	Blackthorn	8
	Cherry	
	Japanese	9
	Laurel	9
	Orchard	12
	Wild	17
	Chestnut	
	Horse	20
	Sweet	24
	Lime	22
	Maple	
	Japanese	8
	Norway	18
	Mountain Ash	11
	Pear	12
	Plane	26
	Plum	10
	Sycamore	22
	Tree of Heaven	20
	Walnut	18
	Whitebeam	12
Low	Birch	14
	Elder	10
	Fig	8
	Hazel	8
	Holly	12
	Honey Locust	14
	Hornbeam	17
	Laburnum	12
	Magnolia	9
	Mulberry	9
	Tulip tree	20

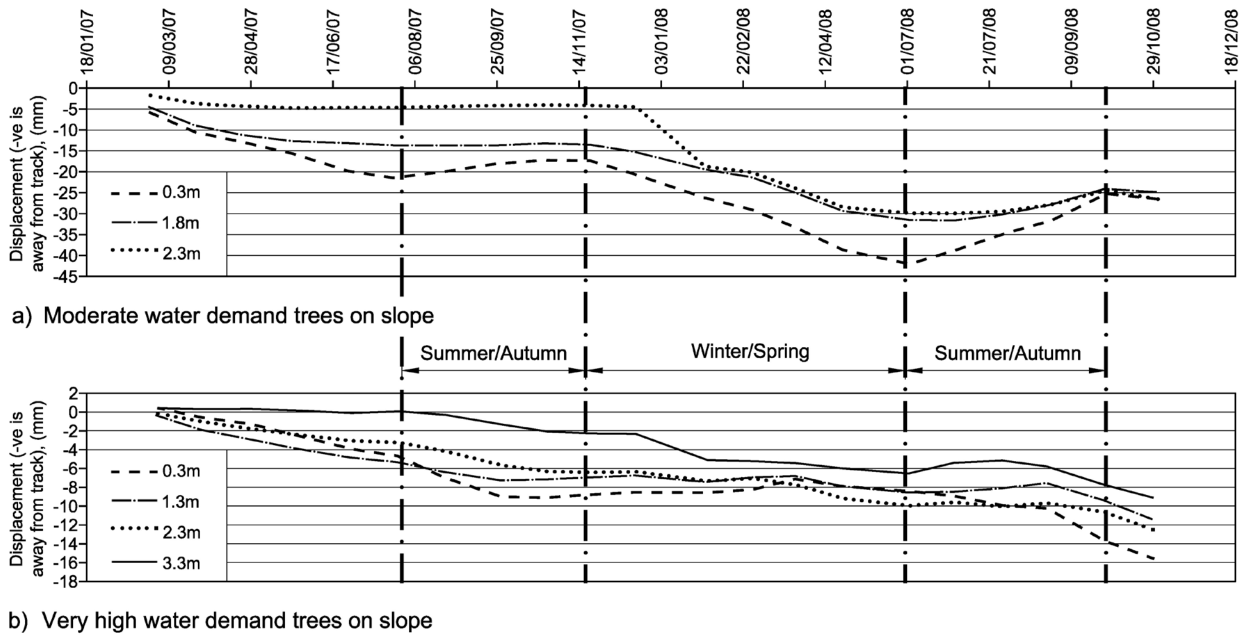
Coniferous Trees		
Water Demand	**Species**	**Mature Height (m)**
High	Cypress	
	Lawon's	18
	Leyland	20
	Monterey	20
Moderate	Cedar	20
	Douglas fir	20
	Larch	20
	Monkey Puzzle	18
	Pine	20
	Spruce	18
	Wellingtonia	30
	Yew	12

Note:

1.Where hedgerows contain trees, their effects should be assessed separately. In hedgerows, the height of the species likely to have the greatest effect should be used.
2.Within the classes of water demand, species are listed alphabetically; the order does not signify any gradation in water demand.
3.When the species is known but the sub-species is not, the greatest height listed for the species should be assumed.
4.Further information regarding trees may be obtained from the Arboricultural Association or the Arboricultural Advisory and information Service (see Section 4.2.4 of NHBC Standards (2016)).

Figures 3.14.2 and 3.14.3 summarise embankment monitoring reported by O'Brien (2013), for Network Rail embankments (but which are of similar age and composition as many LUL embankments). Figure 3.14.2 plots horizontal displacement versus time (from inclinometers located in the mid-slope area). Outward displacement is observed between February and July, the displacement then moves inwards between August and November, followed by another outward/inward cycle during the subsequent winter/summer. Inclinometers located within an area of High Water Demand (HWD) trees, i.e. Oak trees, showed smaller seasonal horizontal displacements than those located in areas of less dense and lower water demand vegetation. Typically outward movements coincided with periods when SMD was less than about 10mm. Figure 3.14.3 plots vertical and horizontal deformation against time, during a period when HWD trees were removed. Vertical displacements (Figure 3.14.3a) exhibits initial settlement during the first summer, followed by sustained swelling which is deep seated (to a depth of 4m). Horizontal movement (during first summer) is negligible, followed by large outward movement during the following winter. The movements are consistent with a gradual wetting up of the slope, following HWD tree removal and rainfall during the autumn/winter period.

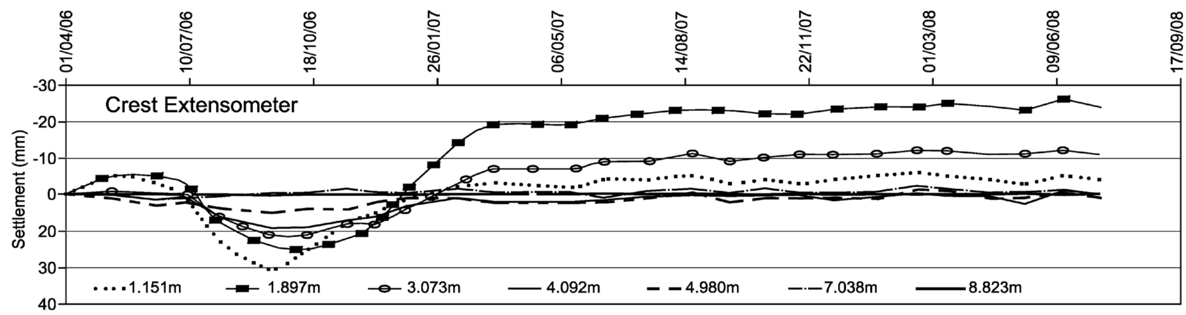
Figure 3.14.2: Horizontal Deformation Versus Time. Tree Covered Embankment



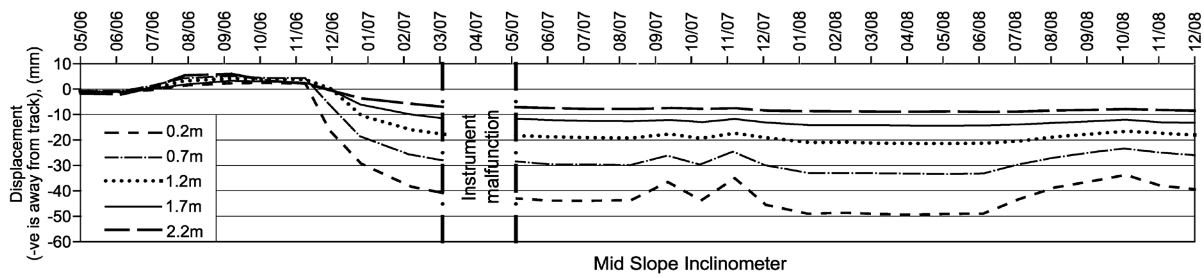
Note:

1. Vegetation on embankment slope classified in accordance with NHBC (2003), except Oak and Poplar trees classified as Very High Water Demand (VHWD).

Figure 3.14.3: Deformation Versus Time. Tree Covered Embankment



a) Vertical deformation versus Time



b) Horizontal deformation versus Time

Notes:
1. Tree removal during mid-March, 2006.
2. Depths quoted are depth below slope surface.

The potential of vegetation to adversely affect track performance is dependent on several factors:

- Volume change potential of the clay fill.
- Water demand and mature height of specific tree species on the slope.
- Distance of tree away from track (D = offset of tree from running track).

The volume change potential of a soil in the vicinity of a tree can be classified based on its modified plasticity index (refer to Table 3.Table 3.14.2). Clay fills of moderate to high volume change potential will be susceptible to serviceability deformations. The modified plasticity index (PI) is defined as:

Modified PI = PI x % particles less than 425μm

Table 3.14.2: Volume Change Potential (after NHBC, (41))

Modified Plasticity Index	Volume Change Potential	Typical Soils
> 40%	High	In-situ London Clay, embankment fill derived from London Clay
20 – 40%	Moderate	Glacial Till, embankment fill derived from Glacial Till or derived from a combination of London Clay and Head Deposits
< 20%	Low	Terrace Deposits or embankment fill derived from Terrace Deposits

Clay fill embankments with a modified PI greater than 40% populated with HWD trees within D/H = 0.75 of the track have the potential to experience significant track serviceability deformations (> 20mm). Moderate water demand trees may also be problematic if particularly close to the track, for clay fills with a modified PI > 40%.

The NHBC has provided guidance on appropriate distance of trees from building foundations. These are overly conservative for use on railway embankments. Shrink-swell effects for embankment fills of low volume change potential (or low shrinkability) would generally be considered to be low risk in terms of track deformation. In addition, low water demand trees would also be considered to be low risk. For the remaining categories (high/medium water demand trees, and moderate/high soil volume change potential), the minimum offset distances when trees may cause significant track deformation are about half to two thirds of those given by NHBC for building foundations.

However, the position of the trees on the slope i.e. upper, middle or lower third, is usually a more critical criterion. Generally, the removal of HWD trees from the upper third of the slope is likely to be beneficial in terms of reducing track serviceability problems, whilst the influence on stability is likely to be modest. In contrast, removal of trees from the lower third is likely to significantly reduce deep-seated stability. Hence, if large HWD trees becomes established on a slope, the engineer can face a “Catch 22” situation. If the tree is left on the slope, the track will experience excessive track deformation (especially during hot dry summers). However, if the tree is removed, the slope may be at high risk of collapse, during wet winters. The guidance provided in Chapters 4, 5 and 7 should allow the engineer to assess the change in deep seated stability induced by tree removal. Additional stabilisation measures may be needed to compensate for tree removal.

3.14.1.1.2 Soil Moisture Deficit

The soil moisture deficit (SMD) may be defined as the amount of water required to be added to the soil, over the full depth of its root zone, to bring it to its normal moisture content at its field capacity i.e. to effectively saturate the root zone. If evaporation and/or evapotranspiration exceed the rainfall then the amount of water in the soil will fall below its maximum holding state, the field capacity. Therefore, the SMD can be used as a means to relate the embankment porewater regime to potential track movements. For example, as the SMD increases the soil moisture content reduces, hence there is a greater potential for the soil to shrink resulting in settlement of the tracks. Similarly, as the SMD reduces towards zero, there is a greater potential for heave due to the soil swelling in response to the increase in moisture content.

Figure 3.14.1 shows that greater embankment deformations were measured in the years where there was the greatest change in SMD. SMD for deciduous trees has also been used as an indicator of stability problems, and to assess how representative pore pressure monitoring data is, compared with wet winter conditions.

Pore water pressure monitoring data should be compared against SMD data for the monitoring period, and against historical SMD data, including the 2000/2001 winter. A judgement can then be made if the pore water data is representative of wet winter conditions. Prolonged periods (more than a month) of zero SMD will be more critical for deep seated instability of clay earthworks, than a day or two of intensely high rainfall. Short periods of high rainfall will tend to be more critical for granular embankments (washout/erosion), or shallow instability. In contrast SMD values in excess of 250mm, represents relatively dry conditions; if these conditions persist for a prolonged period then track serviceability problems may develop if HWD tress are located in the vicinity of the track within high PI clays.

Therefore, knowledge of the SMD at a site, may assist in determining the likelihood of any potential track serviceability issues.

3.14.1.2 Alternative Causes of Poor Track Performance

It is important to note that poor track performance may be due to other factors than trees in the vicinity of the track. Other causes include:

- Lack of crest support or over steepened crest leading to ravelling of the Ash/Ballast
- Localised shoulder instability
- Deep seated slope instability
- Poor track bed drainage
- Transition hard spot between an earth structure and a bridge structure
- Culvert beneath the earth structure in poor condition
- Other services crossings beneath the earth structure in poor condition

3.15 Chalk Cutting

3.15.1 Chalk Cutting – Generic Assessment Strategy

Williams (1990), and Phipps and McGinnity (2001) (refs 42 and 43 respectively), together with CIRIA report C574 provide a comprehensive review of the factors which may affect the stability of cuttings in chalk.

For chalk cuttings assessment is mainly based on an understanding of a site's geology, its geomorphology, and the adjacent hydrology of the area. Geological and geomorphological mapping of the slope and the areas adjacent to the slope crest play a vital role. Theoretical soil and rock mechanics is usually of little value.

As discussed by Phipps and McGinnity there have been few reported problems with the stability of unweathered chalk slopes. The main instability risks are associated with overlying superficial deposits, the presence of destructured re-worked chalk, and the presence of dissolution features (in-filled with metastable deposits).

The assessment strategy would normally include:

(i) Desk study to collate and interpret existing geological, hydrogeological and topographical data (including maps, memoirs, old investigations, condition assessments, aerial photographs, etc.);
(ii) Preliminary site walkover;
(iii) Local vegetation removal across selected areas of cutting (if deemed necessary);
(iv) Detailed geological and geomorphological mapping of slope.

The assessment of the chalk cuttings will inform and advise on the condition of the slope, and guide the design process and remediation options. It should be emphasised that vegetation removal may not be necessary. The geomorphological mapping reported by Phipps (2001) was carried out without any vegetation removal.

LU has a system of formalised inspections and assessment procedures which are used across the network as detailed in – 'Civil Engineering – Earth Structures,' S1054 (36), which considers the whole lifecycle for Earth Structure assets. The recommendations for chalk cutting assessment discussed in this section should be used in conjunction with and compliment the LU guidance.

3.15.2 Unweathered Chalk

Experience of cutting performance has been summarised in CIRIA C574, Williams (1990) and Phipps (2001). The available evidence for inland cuttings within the high porosity Middle and Upper Chalk in Southern England, summarised on Figure 3.15.1, indicates that:

(i) cuttings in structured chalk of at least Mundford Grade III (or Grades A to C as per CIRIA grading) formed at an angle of 45° rarely exhibit significant spalling, provided the slope is vegetated;

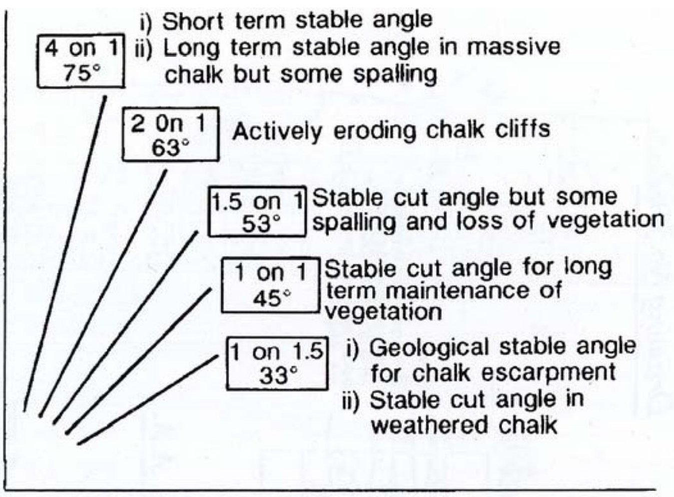
(ii) cuttings formed at 53° may exhibit some spalling, particularly during severe winters (due to freeze/thaw effects);

(iii) there is insufficient evidence on the influence of slope aspect on the degree of spalling;

(iv) evidence of major falls (>10m3) is rare, even for very steep cuttings, circa 70°, but spalling would present a hazard and a long term maintenance problem;

(v) instability within weathered Chalk slopes formed at a slope of 33° is rare, although washout of in-filled solution features can be a significant risk.

Figure 3.15.1: Relationship between Chalk Slope Angle and Stability Conditions



As noted above, spalling is the most common instability hazard, when blocks of chalk are detached from the slope face by frost action. Vegetation provides some protection. When severe spalling has occurred on steep slopes (>53°), the Chalk has been protected by the installation of netting, rock bolts, and then hydroseeded. The length of the bolts needs some consideration to ensure they are long enough to be anchored in Chalk which will be unaffected by future frost action. The use of bolts or nails with a lower thermal conductivity (e.g. glass-reinforced plastic) has also been considered (CIRIA C574).

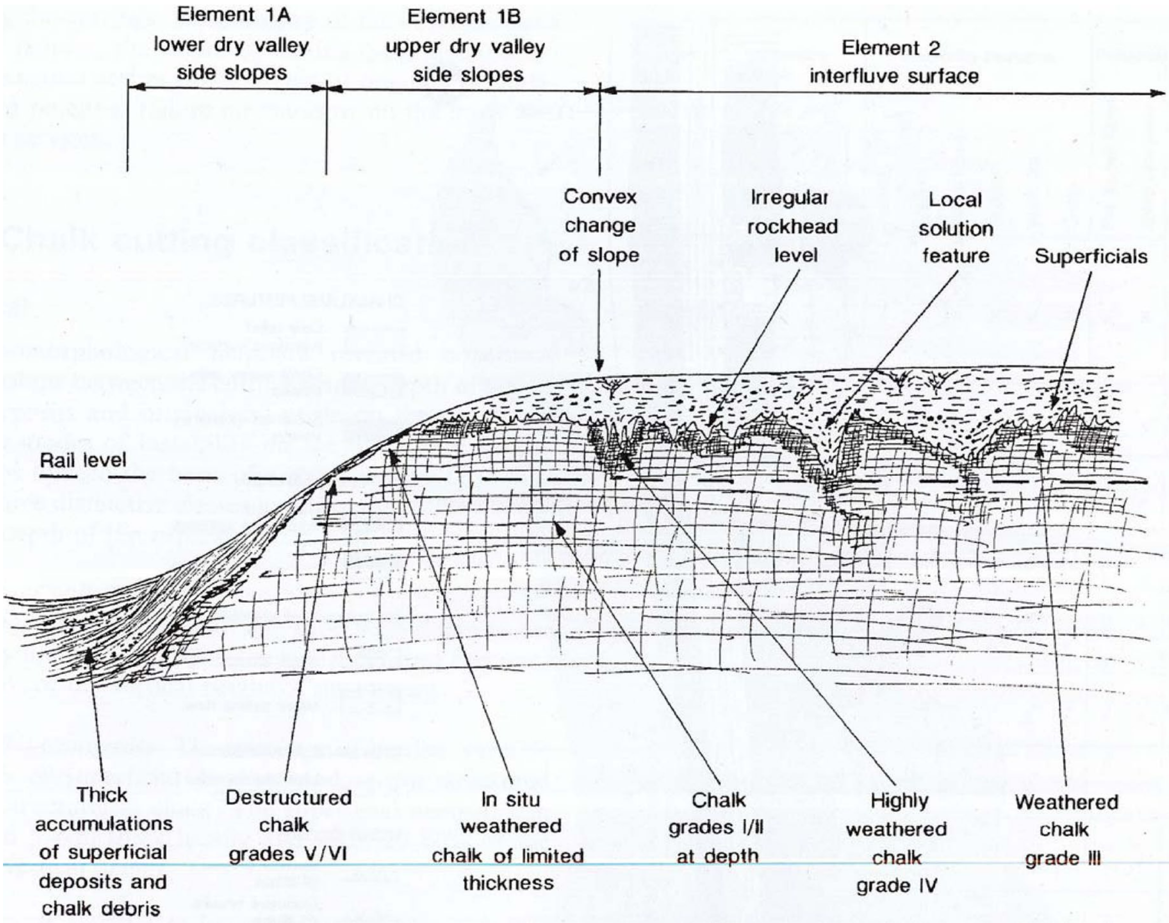
The Superficial deposits, based on a limited number of historical projects on the Metropolitan Line around Watford, Rickmansworth and Chorleywood, broadly categorise the Head deposits as firm slightly sandy Clay with rare Chalk fragments and flint. However, broader experience indicates a greater variability of the Superficial deposits, which must be verified on a site by site basis through ground investigations.

The Superficial deposits are also sensitive to changes in porewater pressure associated with rainfall intensity and whether a particular year/season is wet or dry. Groundwater conditions must be verified on site, and may vary depending on the thickness of Superficial deposits, any drainage located at the cutting's crest (or toe) and site specific ground water monitoring.

3.15.3 Chalk Slopes Capped by Superficial Deposits

Many Chalk cuttings can be characterised as unweathered Chalk towards toe, overlain by weathered destructured Chalk and Superficial deposits. The interface between unweathered and weathered Chalk, and between weathered Chalk and Superficial deposits can be highly irregular, as illustrated on Figure 3.15.2.

Figure 3.15.2: Schematic Long-section of a Typical Chalk Cutting



For LUL Chalk cuttings a classification system and framework for assessing instability hazards has been developed. The classification system is based on comprehensive site specific geomorphological mapping, and is summarised on Figure 3.15.3. Type B class comprises variable amounts of superficial deposits and in situ structured and/or structureless Chalk. The superficial deposits can be up to 5m to 6m thick locally and form 20% to 60% of the cutting vertical height. Across the LUL network Type B1 cuttings (with slope angles in excess of 40°, and heights in excess of 5m) represented the most common instability hazard. The superficial deposits comprise a hetereogenous sequence of clays, silts, sands and gravels; geological mapping indicates three main types: Clay with Flints; Pebbly clay and sand; Glacial outwash deposits. The engineering properties of these deposits are variable, and difficult to characterise reliably. The potential failure mechanisms for B1 are debris flows or washouts of the superficial deposits, triggered by high rainfall events. Areas up to 15m to 20m wide and 10m deep have been affected historically. Geomorphological mapping has identified poorly defined dry valleys upslope of most previously failed cuttings. These features probably lead to a concentration of surface water flows, especially after low frequency high magnitude rainfall events. Type B1 cuttings are only normally found on interfluve surfaces, which tend to support thick superficial deposits. In general, superficial deposits which are at a slope angle of steeper than 26° (1 vertical: 2 horizontal) should be assumed to be potentially unstable.

Figure 3.15.3: Classification Scheme Adopted for Chalk Cuttings (43)

Class		Geometry	Instability Features						Position		Hazard
Geology	Classification Class	α, h (m)	Rockfall	Spalling	Debris Flows	Slumps	Wash Out	Creep	Flat Topped Ridge	Valley Side-Slope	Hazard Class
Chalk <20% profile superficials	A1	>53°, >10	√	√	x	x	√	x	√	x	High
	A2	45-60°, 8-18	√	√	x	√	√	√	√	x	Moderate
									x	√	Low
	A3	30-45°, 5-12	x	√	x	√	√	√	√	√	Low
	A4	30-40°, <6	x	x	x	x	x	√	√	x	Very Low
									x	√	Negligable
Composite 60-20%	B1	40-47°, 5-15	x	x	√	√	√	√	√	x	Moderate
	B2	30-45°, 4-12	x	x	x	√	√	√	√	x	Low
									x	√	Very Low
	B3	30-40°, <5	x	x	x	x	x	√	x	√	Very Low
Superficials >60%	C1	30-40°, 2-6	x	x	x	√	x	√	√	√	Low
	C2	25-35°, 1-4	x	x	x	x	x	√	√	√	Very Low
Retained	D		x	x	x	x	x	√	x	√	Very Low

3.15.3.1 Remedial Options

Remedial measures for a Chalk cutting, capped with Superficial deposits, would typically include one or a combination of the following – depending on the cutting's geometry and the available space at the crest of the slope:

- Regrade the cutting to a flatter slope;
- Depending on space restrictions, defined by the LU boundary – regrading the Superficial deposits may adequately stabilise the slope.
- Soil nailing and appropriate meshing around the nails;

- Hydraulic barriers (such as concrete cut-off walls) and drainage measures to reduce the risk of concentrated surface and groundwater flow;
- Face protection (mesh and bolts), considering the final aesthetics of the slope, constructability, long term maintenance and cost.

CIRIA C574 provides further guidance on potential remedial options.

3.15.3.2 Slope Maintenance

From a maintenance perspective, future vegetation growth is to be controlled, for the following reasons:

- Inspections and testing of soil nails/bolts is more difficult to undertake amidst vegetation cover due to access issues and locating the nail/bolt locations;
- Vegetation growth may promote Chalk degradation (i.e. through root jacking);
- Vegetation growth underneath the face protection/facing may cause bulging (i.e. 'tenting') between the bolts and soil nails; inducing additional loads on the system.
- Vegetation growth on top of the face protection/facing would require maintenance to prevent vegetation becoming too large, potentially becoming uprooted and toppling onto the track or lineside services at the slope toe.

Vegetation coverage of the Head deposits are broadly considered beneficial, however growth should be controlled to prevent the establishment of larger species whose root system may penetrate the underlying Chalk. Larger trees may also be prone to toppling in extreme weather. According to the climate change projection UKCP18 (50), it is likely that in the UK and London Area the frequency and intensity of intense rainfall events will increase. This will lead to increases in surface run-off which could trigger washouts and debris flows. This risk needs to be considered carefully during the assessment and design of earthworks in granular materials.

3.15.4 Chalk Slope and Face Mapping

Face mapping is important to assess the risk of slope instability. The accuracy of the face mapping will be dependent upon:

- the expertise of the geomorphologist carrying out the mapping, this is a specialist activity and should only be undertaken by professionals with the appropriate training and experience;
- the opportunity to inspect and observe the slope, often this may be severely restricted by vegetation. During winter/spring when deciduous vegetation dies back mapping may be more viable than during summer/autumn.

If the slope is so heavily vegetated that mapping is not viable, then some vegetation removal will need to be considered. Currently two options are considered:

i. Local vegetation removal and mapping (possibly via abseil if slope is steep) – vegetation clearance is carried out locally, to create vertical and/or horizontal strips of exposed slope, by manual trimming and removal of trees/shrubs and top soil. The geomorphologist then maps the slope at set intervals. The main disadvantage of this option is that the interface between chalk and superficial deposits may not be accurately mapped along the entire slope (i.e. local deep channels or solution features may be missed).

ii. Complete face exposure and 3D survey – manual excavation plant (e.g. long reach excavator) can remove all vegetation and top soil to expose all the slope face. This does allow for clear identification and mapping of all the key geological and geomorphological features along the slope. The major disadvantage of this option is that LUL would then be committed to installing face protection (e.g. nailing and netting) plus revegetation, in order to reduce risk of excessive erosion and spalling of surface materials. Hence this option is usually only appropriate after LUL have decided to carry out extension stabilisation works on the slope.

For condition assessments or investigations of a cutting, option (ii) above would not be appropriate. For condition assessments, provided the geomorphologist is suitably experienced, vegetation removal will often be unnecessary.

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4 Responsibilities

4.1 The requirements of this guidance document shall be incorporated in any contract to which it is relevant and shall stipulate that a programme of audits are implemented which ensures that these requirements are complied with.

5 Supporting Information

5.1 Background

5.1.1 This guidance document is one of a suite of Standards and Guidance Documents which cover the whole life cycle of Civil Engineering assets. Other Standards in this suite have a bearing on the activities covered by this Standard. In many cases a direct reference to another Standard is given; in other instances the need to refer to another Standard is implied.

5.1.2 The complete suite of Civil LU standards comprises the following documents.

Number	Title
S1050	Civil Engineering – Common Requirements
S1051	Civil Engineering – Bridge Structures
S1052	Civil Engineering – Gravity Drainage Systems
S1053	Civil Engineering – Building and Station Structures
S1054	Civil Engineering – Earth Structures
S1055	Civil Engineering – Deep Tube Tunnels and Shafts
S1056	Civil Engineering – Pumping Systems
S1057	Civil Engineering – Miscellaneous Assets
S1062	Civil Engineering – Temporary Works

5.1.3 The following Guidance Documents have also been prepared to give guidance and explanation for each of the above Standards:

Number	Title
G-050	Civil Engineering – Common Requirements
G0051	Civil Engineering – Bridge Structures
G0052	Civil Engineering – Gravity Drainage Systems
G0053	Civil Engineering – Building and Station Structures
G0054A	Civil Engineering – Earth Structures
G0054B	Civil Engineering – Earth Structures: Guide for Slope Stability Analysis
G0055	Civil Engineering – Deep Tube Tunnels and Shafts
G0056	Civil Engineering – Pumping Systems
G0057	Civil Engineering – Miscellaneous Assets
G0058	Civil Engineering – Technical Advice Notes

5.2 Safety Considerations

5.2.1 Safety aspects shall be considered throughout the design process and due account taken of the Construction (Design and Management) Regulations.

5.2.2 Earth Structures deteriorate with time under cyclic loading and the influence of seasonal effects such as high winter water levels and water demand of trees, as well as vandalism and the effect of burrowing animals,

5.2.3 Many of the existing Earth Structures have already passed a 120 year Service Life, and were constructed without known standards and with primitive methods. Many failed during and soon after construction and have been the subject of phases of repair.

5.2.4 In view of these considerations this Guidance requires a structured approach to Earth Structures management based on standard procedures of Inspection and Assessment of the existing Earth Structures to reduce the safety risks in accordance with ALARP principles.

5.3 Environmental Considerations

5.3.1 All activities including planning, design, procurement, construction, installation, testing, commissioning, operation, Maintenance, decommissioning and disposal must comply with current environmental legislation, approved Codes of Practice and authoritative guidance literature issued by relevant statutory bodies.

5.4 Customer Considerations

5.4.1 The Earth Structures shall provide effective support to the track formation, service posts, etc, and maintain the structure gauge requirements, all as described in this Guidance, so as to allow uninterrupted and smooth operation of the railway to meet the needs of Customers.

6 Person Accountable for the Document

Name	Job title
Nader Saffari	Profession Head - Earth Structures and Geotechnical

7 References

Document no.	Title or URL
Please see Section 3.15 of this guidance document for references.	

8 Document history

Issue no.	Date	Changes	Author
R5	August 2007	Standard 5-01304-005 re-formatted and re-numbered to G-054, no technical changes have been made to the content other than changing references to other Standards where their numbers have changed.	
A1	October 2007	Authorised for use. Previous authorisation is valid	
A2	December 2014	G-054 re-named to a Guidance Document and split into G0054A “Earth Structures” and G0054B “Earth Structures – Guide for Slope Stability Analysis”.	Nader Saffari
A3	June 2018	Updated as per DRACCT 05243 General update and formatting.	Nader Saffari
A4	May 2019	General update and formatting using the current template as per change request No. CR-11118.	Joseph Kennedy / Nader Saffari
A5	November 2023	Including the impact of climate change in Section 3.9 and other other sections as per Change Request No. CR-18323.	Nader Saffari

9 Attachments

9.1 Numerical Modelling of Progressive Failure

The following sections are intended to provide background information on the numerical modelling previously carried out by Mott MacDonald to investigate the progressive failure mechanism in cohesive earth structures. The modelling was carried out using FLAC (Fast Lagrangian Analysis of Continua) and was based on methodology developed during previous LUL applied research, as summarised by O'Brien et al (7). This appendix gives the basic strain softening model used including material parameters relevant to the model as well as an example of the output produced. It should be noted that during the research a number of different models and scenarios were analysed with a large number of permutations considered.

9.1.1 Model Geometry and Boundaries

The basic model geometry was a uniform 1:2.5 slope embankment, 8m high. The grid was restrained against horizontal movement at the side boundaries and both horizontal and vertical movement at the base. The crest was 6m wide to ensure the slips were not affected by the left hand boundary. The right hand boundary of the mesh was located approximately 47.5m from the toe of the slope in order to not affect the slope stability.

Ash capping was assumed to be 1m thick and it was assumed that the embankment fill extended to approximately 0.5m below the toe of the slope. The presence of alluvium under the embankment toe was considered in sensitivity studies. The thickness of the underlying clay layer was maintained as approximately twice the modelled embankment height.

The grid consisted of 2400 elements (referred to as ″zones″ in FLAC terminology) and their distribution was modified so that the elements in the embankment where approximately square (except near the slope surface where the grid was distorted by the line of the slope). The characteristic dimension derived (0.6m) was approximately 7.5% of the embankment height.

9.1.2 Model Properties

The strain softening model used in the numerical modelling is shown in Figure 9.1.

Table 9.1 shows the material parameters used in the modelling.

Figure 9.1: Strain Softening Model (Defined in Terms of Plastic Strain (γ^{p}) or Displacement

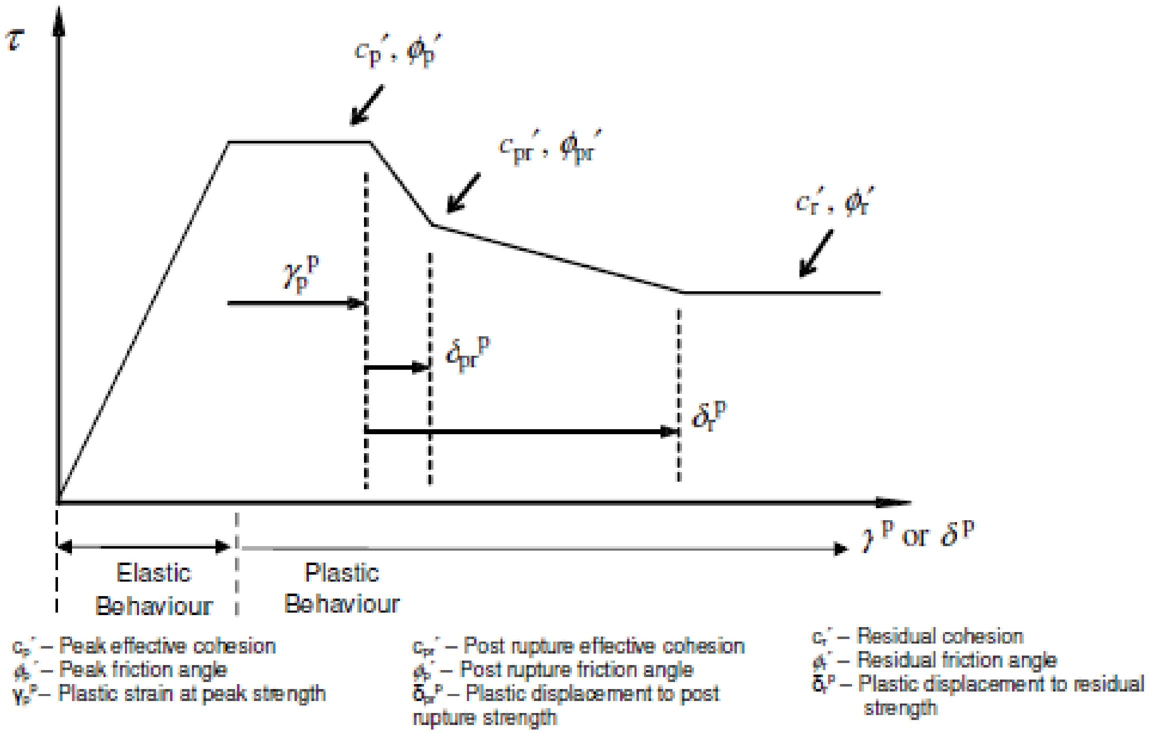


Table 9.1: Material Parameters used in FLAC Analyses

Parameter	High to Very High Plasticity (e.g. LUL London Clay Fill)		
	In situ London Clay	London Clay Fill	London Clay Fill (modified peak strength)
Soil Model No.	(1)	(2a)	(2b)
Bulk unit weight	18.8 kN/m3	18.1 kN/m3	18.1 kN/m3
Young's modulus kPa	75(p′+100), min. 5000 kPa	75(p′+100), min. 5000 kPa	75(p′+100), min. 5000 kPa
Poisson's ratio	0.2	0.3 (load), 0.2 (unload)	0.3 (load), 0.2 (unload)
Peak strength (Bulk)	c′ = 7.0 kPa, ϕ′ = 21.0°	c′ = 3.4 kPa, ϕ′ = 22.9°	c′ = 5 kPa, ϕ′ = 21°
Post-rupture strength	c′ = 2.0 kPa, ϕ′ = 21.0°	c′ = 2.0 kPa, ϕ′ = 21.0°	c′ = 2.0 kPa, ϕ′ = 21.0°
Residual strength	c′ = 2.0 kPa, ϕ′ = 13.0°	c′ = 2.0 kPa, ϕ′ = 13.0°	c′ = 1.0 kPa, ϕ′ = 13.0°
Plastic strain at peak strength, γ^{p}	3 %	6 %	6 %
Plastic displacement to post-rupture strength, δ□^{p}	5 mm	5 mm	5 mm
Plastic displacement to residual strength, δ^{p}	100 mm	100 mm	100 mm

Table 9.2: Material Parameters used in FLAC Analyses (continued)

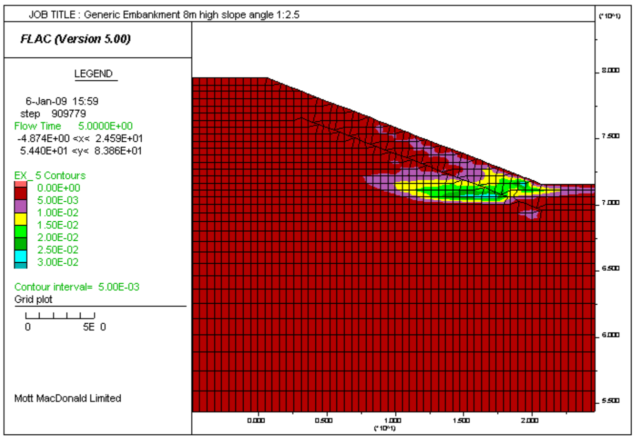
Parameter	Intermediate to High Plasticity (e.g. Pound Green London Clay Fill and Charing Gault Clay Fill)							
	In situ Gault Clay	Clay Fill	Clay Fill (increased stiffness)	Clay Fill (reduced peak strength)	Clay Fill (increased residual strength)	Clay Fill (increased residual strength)	Clay Fill (reduced peak strength)	Clay Fill (increased peak strength)
Soil Model No.	(3)	(4a)	(4b*)	(4c*)	(4d)	(4e*)	(4f)	(4g)
Bulk unit weight	19.6 kN/m3	18.8 kN/m3	18.8 kN/m3	18.8 kN/m3	18.8 kN/m3	18.8 kN/m3	18.8 kN/m3	18.8 kN/m3
Young's modulus kPa	135(p′+100), min. 7500 kPa	90(p′+100), min. 6000 kPa	110(p′+100), min. 6000 kPa	90(p′+100), min. 6000 kPa	90(p′+100), min. 6000 kPa	90(p′+100), min. 6000 kPa	90(p′+100), min. 6000 kPa	90(p′+100), min. 6000 kPa
Poisson's ratio	0.2	0.3 (load), 0.2 (unload)	0.3 (load), 0.2 (unload)	0.3 (load), 0.2 (unload)	0.3 (load), 0.2 (unload)	0.3 (load), 0.2 (unload)	0.3 (load), 0.2 (unload)	0.3 (load), 0.2 (unload)
Peak strength (Bulk)	c′ = 10.0 kPa, ϕ′ = 27.0°	c′ = 8.0 kPa, ϕ′ = 24.0°	c′ = 8.0 kPa, ϕ′ = 24.0°	c′ = 5.0 kPa, ϕ′ = 24.0°	c′ = 8.0 kPa, ϕ′ = 24.0°	c′ = 8.0 kPa, ϕ′ = 24.0°	c′ = 6.0 kPa, ϕ′ = 24.0°	c′ = 12.0 kPa, ϕ′ = 24.0°
Post-rupture strength	c′ = 3.0 kPa, ϕ′ = 24.0°	c′ = 2.0 kPa, ϕ′ = 22.5°	c′ = 2.0 kPa, ϕ′ = 22.5°	c′ = 2.0 kPa, ϕ′ = 22.5°	c′ = 2.0 kPa, ϕ′ = 22.5°	c′ = 2.0 kPa, ϕ′ = 22.5°	c′ = 2.0 kPa, ϕ′ = 22.5°	c′ = 2.0 kPa, ϕ′ = 22.5°
Residual strength	c′ = 1.0 kPa, ϕ′ = 13.0°	c′ = 1.0 kPa, ϕ′ = 13.0°	c′ = 1.0 kPa, ϕ′ = 13.0°	c′ = 1.0 kPa, ϕ′ = 13.0°	c′ = 1.0 kPa, ϕ′ = 17.0°	c′ = 1.0 kPa, ϕ′ = 15.0°	c′ = 1.0 kPa, ϕ′ = 13.0°	c′ = 1.0 kPa, ϕ′ = 13.0°
Plastic strain at peak strength, γ^{p}	3 %	7 %	7 %	7 %	7 %	7 %	7 %	7 %
Plastic displacement to post-rupture strength, δ^{p}	5 mm	5 mm	5 mm	5 mm	5 mm	5 mm	5 mm	5 mm
Plastic displacement to residual strength, δ^{p}	100 mm	100 mm	100 mm	100 mm	100 mm	100 mm	100 mm	100 mm

Table 9.3: Material parameters used in FLAC analyses (continued)

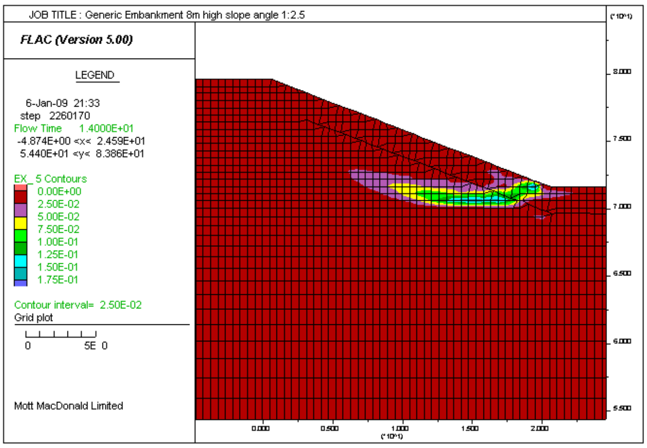
Parameter	Alluvium	Ash fill
Soil Model No.	(5)	(6)
Bulk unit weight	16.0 kN/m3	10.5 kN/m3
Young's modulus kPa	45(p′+100), min. 3000 kPa	1000 kPa
Poisson's ratio	0.3 (load), 0.2 (unload)	0.3
Peak strength (Bulk)	c′ = 3.0 kPa, ϕ′ = 24.0°	c′ = 2 .0 kPa, ϕ′ = 35.0°
Post-rupture strength	c′ = 2.0 kPa, ϕ′ = 22.5.0°	N.A.
Residual strength	c′ = 1.0 kPa, ϕ′ = 13.0°	N.A.
Plastic strain at peak strength, γ^p	7 %	N.A.
Plastic displacement to post-rupture strength, δ^p	5 mm	N.A
Plastic displacement to residual strength, δ^p	100 mm	N.A

9.1.3 Example Output

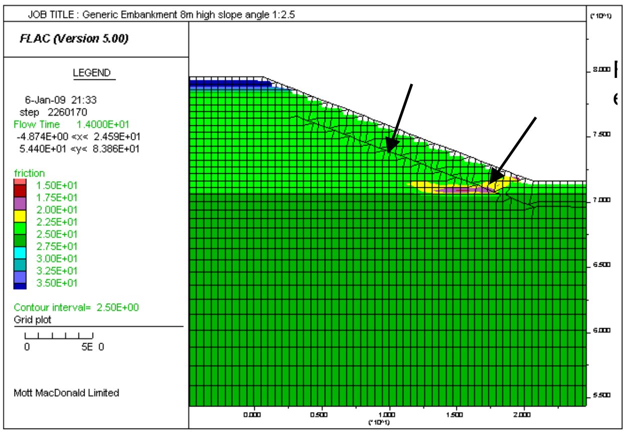
Figure 9.2: Example output from FLAC modelling showing progressive failure mechanism



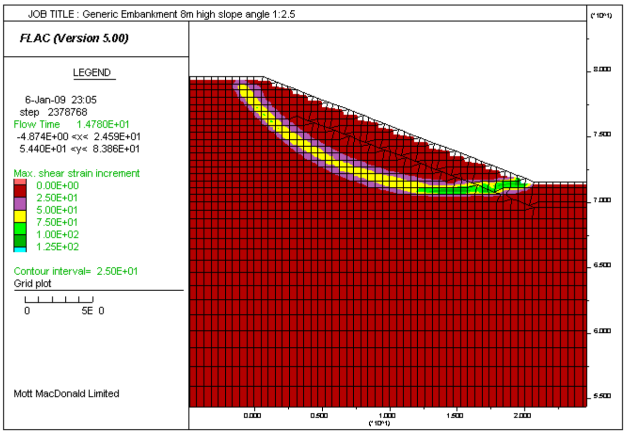
(a) Contours of plastic strain after 5 shrink/swell cycles



(b) Contours of plastic strain after 14 shrink/swell cycles



(c) Internal friction angle after 14 shrink/swell cycles (just prior to failure)



(d) Contours of shear strain after 15 shrink/swell cycles (failure)

9.2 Evidence of Instability

The photos included in this appendix are intended to provide additional guidance on what should be considered as evidence of instability relating to the progressive failure mechanism described in Section 3.4.

Photo 9.1: Large-scale deep seated failure of a cutting (North Acton)

Photo 9.2: Backscar towards the crest of a cutting indicating previous slope failures

Photo 9.3: Tension cracks at the crest of earth structures may be indicative of deep seated instability

Photo 9.4: Toe bulging may be indicative of deep seated stability

Photo 9.5: Leaning cable posts and vegetation may indicate shallow slope deformation and crest instability

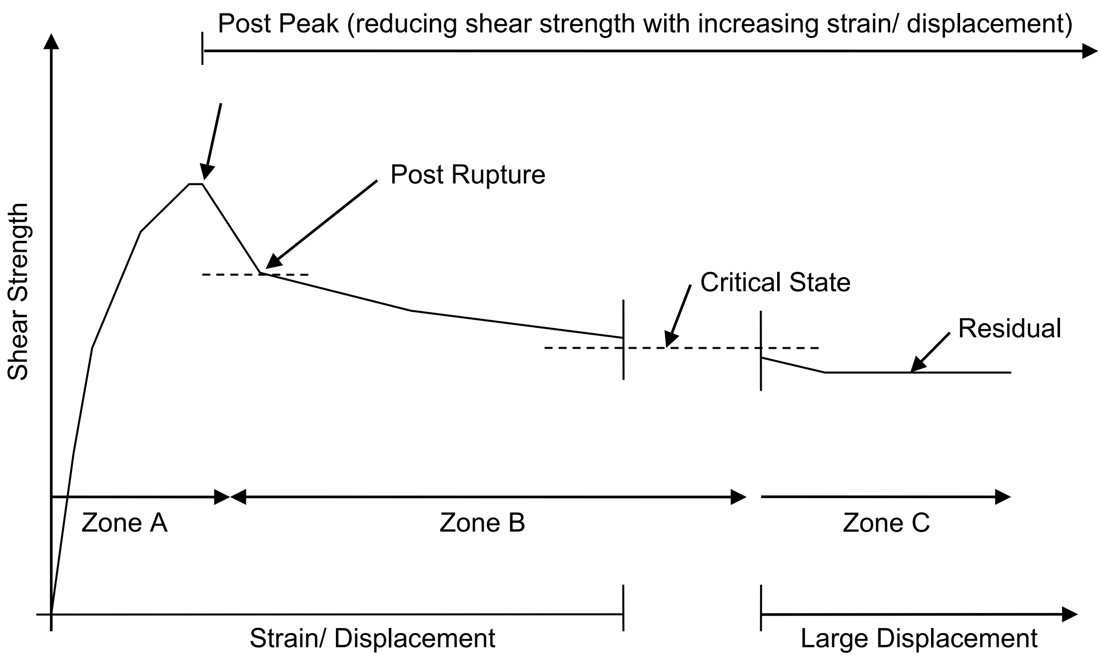
Photo 9.6: Localised crest instability such ad ravelling ballast should NOT be considered as evidence of instability in the risk classification flowcharts

Photo 9.7: Localised slope deformation (NOT related to deep seated instability)

9.3 Derivation of Soil Parameters – Background Information

The idealised stress-strain behaviour typically observed for a fissured high plasticity, overconsolidated clay or clay fill is shown in Figure 3.4.7 in Section 3.4.4, and repeated below in Figure 9.3.

Figure 9.3: Schematic Stress-Strain Response of Clay Fill



Under initial loading (Zone A) the sample mobilises a peak strength which decays rapidly with increasing strain (i.e. in a brittle manner) to a near constant value. The value reached after this initial rupture is referred to as the post rupture strength, as described by Burland (21). The value of peak strength will be highly dependent on structure, stress history (OCR), type/rate of loading etc.

With increasing displacement (Zone B to Zone C), the "plate" like clay particles will slowly become aligned, creating a smooth or "slickenside" shear surface. The shear stress mobilised after large displacement is referred to as residual strength.

Critical state strength describes the shear strength mobilised along a surface which is shearing at constant volume i.e. zero dilation. In this state particles may be visualised as "tumbling" over one another, with no significant granular interlock or sliding plane development affecting the resistance to shearing. Critical state strength is not affected by any inherited fabric or bonding of the soil. In high plasticity clays with "plate" like particles, particle alignment means that the transition between post rupture and residual strength is gradual but continuous with increasing displacement.

Therefore, it is often impractical to measure true critical state strength in the laboratory for strain softening materials.

Stark and Eid (16) present useful relationships between ″secant fully softened″ friction angle and liquid limit, clay fraction and mean effective stress. These are discussed in more detail below. The fully softened state describes a condition after initial failure when moisture content in the vicinity of the shear surface is increased due to dilation reducing the effect of structure and bonding, but before particles have started to re-orientate. In the tests summarised by Stark and Eid (16), the clays were reconstituted at high moisture contents (thereby eliminating structure/bonding effects) before reconsolidation to a variety of effective stress levels, hence the term fully softened. Shear strength is quoted as a secant friction angle (with c´ = 0) which varies with increasing mean effective stress. This contrasts with the conventional tangent fit approach commonly adopted.

9.3.1 Derivation of Mohr-Coulomb Parameters

As discussed in Section 3.4.4, for conventional analyses soil strength is usually described using Mohr-Coulomb parameters, consisting of an internal angle of friction (ϕ´) and an effective cohesion (c´), whilst ″real″ failure envelopes tend to be curved, it is conventional to assume tangent, straight line parameters over the range of effective stress relevant to the problem being analysed (Figure 9.4).

Figure 9.4: Idealisation of Tangent Parameters (s´-t space)

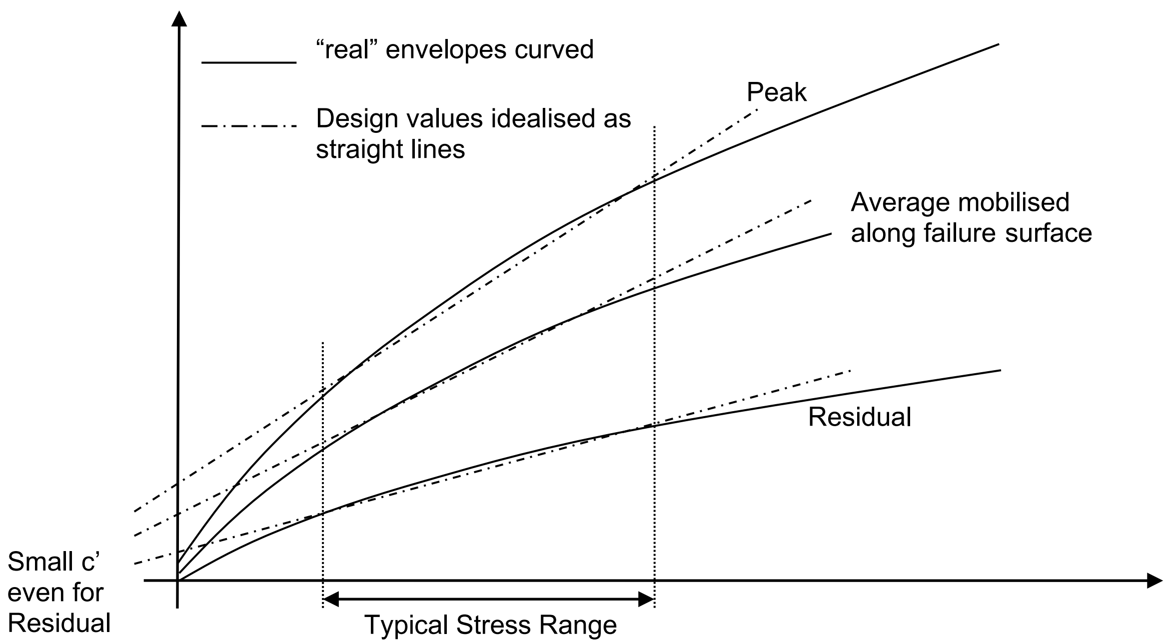
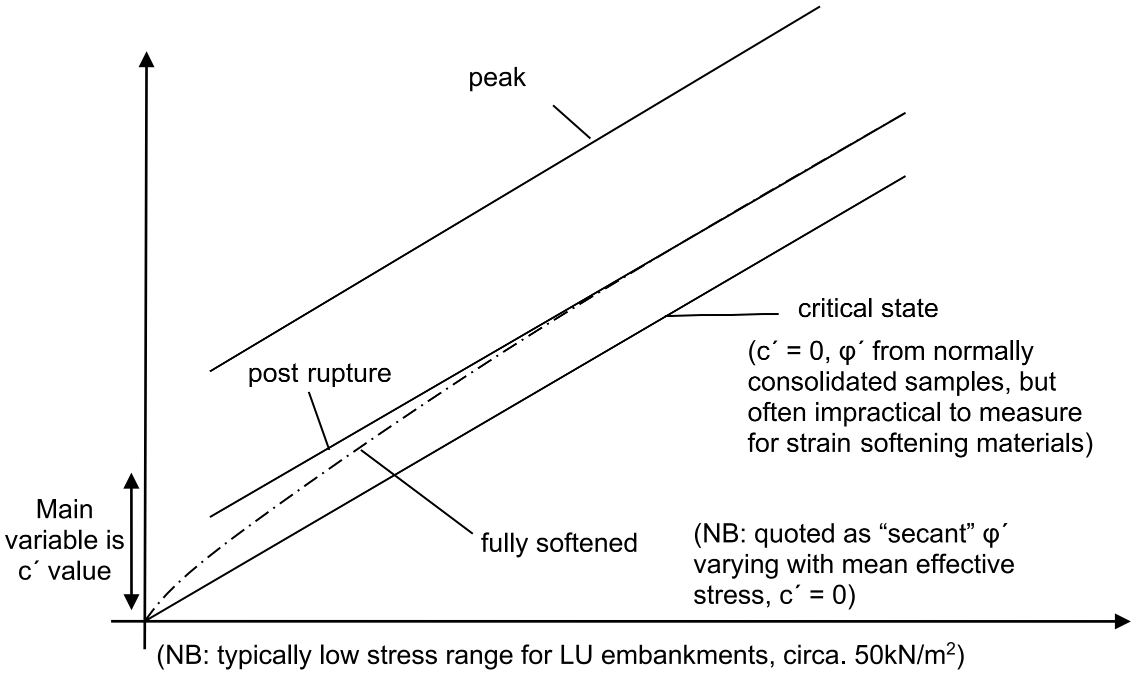


Figure 9.5 shows the parameters defined in Figure 9.3 above plotted in s'-t space. Note that the principal difference between peak, post rupture and critical state strength is in the effective cohesion, with;

$$\phi'_{peak} \approx \phi'_{post\ rupture} \approx \phi'_{fully\ softened}(high\ stress) \approx \phi'_{critical\ state}$$

Note that φ'_{peak} may be 2 or 3 degrees higher than critical state depending on stress level. The residual friction angle will be considerably lower.

Figure 9.5: Comparison of Various Strength Definitions in s´-t Space



9.3.1.1 Conservative Peak Strength

For preliminary analyses (e.g. for site comparison/prioritisation), a conservative value for φ´peak can be derived from the Stark and Eid (16) relationships of φ´fully softened at high stress with Liquid Limit and Clay Fraction. Site specific ground investigation and associated laboratory testing will be required to justify the use of higher peak strengths. The appropriate relationships are presented in Figure 3.4.11 in Section 3.4.4.2.

One of the main practical benefits of using the Stark and Eid approach as opposed to other published empirical relationships is that the separate effects of both Clay Fraction and plasticity (via Liquid Limit) are independently accounted for. This is particularly important for LU embankments since they are often contaminated by sand/gravel (due to the method of construction). It can be seen that for Clay Fractions less than 45%, the friction angle is significantly higher. The friction angles given in Figure 3.4.11 are to be used when a conventional "tangent" set of parameters are applied for stability assessment (i.e. c´, φ´), rather that secant values (i.e. c´= 0, φ´). The parameters given in Figure 4.11 have been derived from the secant values quoted by Stark and Eid (16) at high stress levels (circa 400kN/m2), since these will be more appropriate for a conservative assessment of peak strength (as c´, φ´); rather than the secant values quoted at low stress levels.

Peak strength is observed to vary with stress history (OCR) and structure/bonding which leads to significant variation in apparent effective cohesion (with no significant variation in friction angle), as shown schematically in Figure 9.6. The variable effects of stress history can be removed by normalising samples based on their void index (Hvorslev normalisation). Stresses are normalised by the equivalent pressure (σ*ve) on the intrinsic compression line (after Burland 1990), which is derived from the void ratio at the end of shearing using the recorded moisture content. Under

Hvorslev normalisation (s´-t) space becomes (s´/σ*ve – t/σ*ve) space and envelopes can be fitted around the stress paths using a Hvorslev surface of the form;

$(t/\sigma^*_{ve}) = c'_{ve} + (s'/\sigma^*_{ve}) \sin \varphi'_e$

This by comparison with the more standard Mohr-Coulomb failure criterion gives;

$c' = c'_{ve}\, \sigma^*_{ve}$

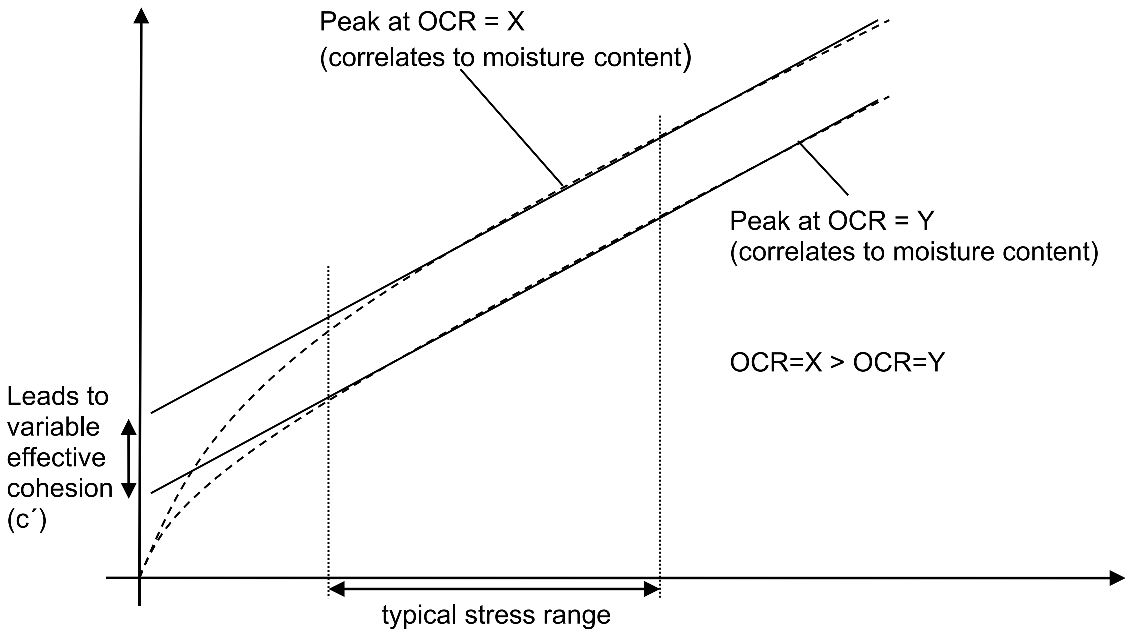
and $\varphi' = \varphi'_e$

Previous studies on LUL London Clay fill showed a lower bound Hvorslev intercept to be about 0.01. Using this value with the equations above peak effective cohesion can be related to void index and hence natural moisture content (Figure 3.4.12 in Section 3.44.2). Using Chandler's (20) relationship between IL and IV (IV=2IL+1) it is possible to plot the variation in effective cohesion with liquidity index. This will often be useful for routine design and the appropriate relationships are presented in Figure 3.4.12 in Section 3.44.2. Estimates of c' should be assessed from both Figure 3.4.12 and Figure 3.4.13, and a judgement made on the most appropriate value.

Using the above approach it is possible to obtain a conservative peak strength (c´ and φ´) for preliminary analyses based only on index testing. Therefore only relatively low cost GI is required at this stage. More advanced testing and GI would be required to justify higher strengths.

If the cost of remedial works is likely to justify it, additional site specific effective stress tests could be carried out prior to the detailed design stage. It is recommended that these tests are carried out on high quality samples (not U100) and are properly normalised (using the Hvorslev approach), so the variable effects of stress history (or moisture content) can be properly assessed.

Figure 9.6: Variation of Peak Strength with Stress History (OCR)



9.3.1.2 Residual Strength

There have been several studies relating residual strength to index properties (Clay Fraction and/or Liquid Limit/ Plastic Limit), e.g. Lupini et al (25), Wesley (14). Stark and Eid (15) have produced comprehensive correlations between residual angle of friction and Clay Fraction and Liquid Limit, and normal effective stress level (residual friction angle varies significantly with stress level).

Figure 3.4.10 in Section 3.4.4.1 plots residual friction angle versus Liquid Limit for varying clay Fractions (<20%, 20% - 45% and >45%), for a normal effective stress level of 50kN/m2 (appropriate for most NR embankment analyses). This is based on the research by Stark and Eid (15). This residual friction angle should be used in conjunction with a small effective cohesion, c´ = 1.0kN/m2.

Also shown on Figure 3.4.10 is the friction angle derived by Skempton (22); based on his back analysis of several landslides in a wide range of UK natural high plasticity clays. This forms a practical lower bound for residual strengths for LU earth structure analyses. It should be emphasised that residual friction angles much lower than those indicated on Figure 3.4.10 can be measured in ring shear tests, especially if they are carried out at high stress levels. Residual friction angles less than 13^{o} are not considered appropriate for LU earth structure analyses.

9.4 Competency

An example of the required competency level plus essential and desirable qualifications for the members of a design team assessing earth structure stability and designing suitable remedial works is set out below.

Table 9.4: Design Team Competency Levels and Qualifications

Position	Competency Level			Qualifications	
	Slope Stability & Earth Structures	Retaining Walls & Piles	Ground Reinforcement & Strengthening	Essential	Desirable
Project Director	5	5	5	A, B, C	
Project Manager	4	4	4	A, B, C	
Deputy Project Manager	3	3	3	A	B, C
Design Team Leader	2	2	2	A	B, C
Design Engineer	1	1	1	A	
Design Checker	3	3	3	A, C	B
Project Reviewer	4	4	4	A, C	B

Key:

Qualifications

A. Batchelor degree in Civil Engineering or Geology

B. Masters degree in Civil Engineering, Geology or Soil Mechanics/Geotechnics

C. Chartered Engineer or Geologist

Competency Level

1. Moderate Knowledge – is developing skills and knowledge, works under close supervision
2. Working Knowledge - has basic design knowledge and experience (works under some supervision)
3. Practitioner - has design knowledge and experience (proficient designer who can supervise and direct others)
4. Advisor - has specialist design knowledge and experience
5. Expert – has specialist design knowledge and experience and recognised as an industry expert

Further details regarding competency levels are set out in the table below:

Table 9.5: Determination of Competency Level

Competency Level	Definition: Demonstrate being able to……	Profiles: Other typical attributes may include…
1. Moderate Knowledge	Participate &/or contribute to a conversation about the skill area, but not in detail. Application of Standard Techniques and Procedures. Carry out basic tasks with close supervision.	Understanding of general issues of skill through working alongside practitioners, or those with working knowledge. Becoming reasonably current with industry developments. Gaining knowledge through self learning.
2. Working Knowledge	Lead a detailed discussion and communicate what needs to be done and how it is applied in practice Independently evaluates, selects and applies standard techniques, receives objectives from supervisor. Carry out work in this skill area (including complex tasks) with supervision	Understands key issues and best practice. Previous practitioner but no longer directly delivering hands on or would no longer feel appropriate to practice in this field. Ability to quickly assimilate issues and ask searching questions. Has hands-on experience in delivering part of this skill. Learning the codes, standards and regulations
3. Practitioner /Specialist	Be trusted to carry out work in this skill area with minimal supervision Fully competent in all conventional aspects of subject.	Has hands-on successful delivery of at least part of this skill with little re-work or need for supervision. Awareness of current developments and research in specialist area. Trusted to act as independent checker or Project Reviewer. Familiarity with and ability to implement codes. Leading technical teams in this skill Can contribute to Value Engineering and Risk Management. Delivering best practice
4. Advisor	Apply proven competence to complex examples and situations in this skill area.	Has demonstrated successful delivery of this skill across a range of complex projects, Ability to think and deliver outside normal application of the codes. Providing Peer Assist on projects. Can lead Value Engineering and Risk Management Developing key contacts in specialist area across industry at national level.
5. Expert	Represent the company using current and leading edge practice under scrutiny &/or close questioning under an adversarial environment.	Recognised as a technical expert in this skill both by peer group as well as by clients. Developing key contacts in specialist area across industry internationally. Delivering research papers/ presentations. Active involvement in industry working parties. A successful Innovator Developing and Establishing best practice.

9.5 London Clay and Dumped Clay Fills - A Comparison of Peak Strength

Content

9.5.1 Introduction

This Appendix provides a comparison between the factored peak shear strength of London Clay and of high plasticity dumped clay fill used extensively for the construction of London Underground railway embankments.

For the worst credible deep seated failure mechanism discussed in the Design Guide (refer to chapters 4, 7 and 8), the partial factor of safety applied to the cautious estimate of peak strength (for soils beyond a weakened basal layer) is equal to 1.1. This is used together with the weakened basal layer (with a strength varying between critical state and residual), with a partial factor of 1.1, and worst credible groundwater pressures).

The peak strength of dumped clay fill presented in this appendix is based on the recommendations provided in Chapter 4 of this Design Guide. The peak strength of the London Clay is obtained from published technical literature considering primarily high quality samples subjected to Triaxial Compression shearing. To facilitate simple cautious estimates of "peak" strength a set of design charts are provided, Figure 3.4.11, 3.4.12 and 3.4.13, to enable angles of friction and effective cohesion to be estimated from readily available site specific data (clay fraction, liquid limit, natural moisture content and liquidity index). The purpose of this Appendix is to summarise a series of comparisons between the peak strengths derived from the design charts, and published high quality laboratory test data for the peak strength of London Clay. Data is provided for unfactored and factored (i.e. peak strength divided by partial factor) strengths.

Solely for the purposes of comparison in this Appendix, the factored strength for the published London Clay data uses a partial factor of 1.25 (hence, is consistent with EC7 partial factors IF the peak strength is considered to the characteristic strength for the design scenario under consideration), and the factored strength for the dumped clay fill is 1.1 (consistent with the guidance in this report for use in the worst credible failure mechanism outlined above).

The comparison aims to demonstrate the conservative nature of the LU Earth Structure Design Guide recommendations on the selection of peak strength values and of the partial factor of safety equal to 1.1 for the derivation of peak strengths. It should be emphasised that this approach can only be considered to be conservative, when used in conjunction with the weakened basal layer, and associated groundwater pressures, for the assessment of deep seated instability. Other sets of parameters and failure mechanisms also must be considered, as outlined in the Design Guide. It is also worth noting that although considerable attention is usually given to soil strength parameters, the selection of appropriate groundwater pressures is usually far more critical and warrants more attention than it is usually given.

The following notation is used in this Appendix:

- φ_p´: effective angle of peak shearing resistance for Morh-Coulomb strength envelope definition
- c_p´: effective peak cohesion for strength for Morh-Coulomb strength envelope definition
- α_p´: angle of line of maximum shear stress at failure
- a_p´: intersect of line of maximum shear stress at failure with the shear stress axis

with,

$$\alpha_p' = \text{atan}(\sin(\varphi_p'))$$

and

$$a_p' = c_p' \cdot \cos(\varphi_p')$$

All peak strength values reported within this appendix are rounded to the nearest tenth digit unless otherwise shown (primarily for comparison purposes, rather than implying the accuracy of the derived shear strength).

9.5.2 Dumped Clay Fill Peak Strength

9.5.2.1 Index Properties for LU Clay Fills

The clay fills considered in this Design Guide have typically been formed by end-tipping the fill (often from adjacent London Clay cuttings) with little or no compaction; commonly known as “dumped clay fills”. For ease of reference, these are often referred to in this Appendix simply as “London Clay fill” or “Embankment fill”.

LU Clay Fill index properties obtained from railway embankments from Central and Metropolitan lines are reported by O’Brien (2004) and O’Brien (2007) and are given in Table 9.6 and in Section 9.5.8, supplementary data. Table 9.6 also shows the typical range of the mean index properties as well as the range if the standard deviation from the mean (SD) is considered. The clay fraction of the fills was reported to be between 50% and 60%, which is typical for fills of London Clay origin.

O’Brien (2004) shows the spatial distribution of moisture content through a Dumped Clay Fill LU embankment with a grass covered slope given in Figure 9.7. The clay core has relatively high moisture content whereas the side slopes have much lower moisture content.

Scott et al (2007) investigated the influence of climate and vegetation on railway embankments. Results for the moisture content distribution within an instrumented 5m high embankment, constructed from end-tipped London Clay and capped with 1m-2m of ash are shown on Figure 9.8. The presence of high water demand oak trees reduces the moisture content of the clay. However, the absence of high water demand trees, which is typical during embankment remediation works, resulted in significant increases in the moisture content of the fill.

A typical range of moisture content and liquid limit for the selection of the peak strength parameters for LU Dumped Clay Fill, for the worst credible deep seated failure mechanism discussed in the Design Guide, is given below:

Moisture Content (%): 35 to 45

Liquid Limit (%): 70 to 80

A typical range for the plasticity index is 45% to 55%.

Values of moisture content less than 33% should be used with caution (see also Section 9.5.5).

Table 9.6: Index Properties for LU Dumped Clay Fills obtained from Railway Embankments (after O'Brien, 2007)

Moisture Content (%)				Liquid Limit (%)				Plasticity Index (%)			
Mean	SD	+SD	+2SD	Mean	SD	-SD	+SD	Mean	SD	-SD	+SD
26	7	33	40	73	11	62	84	47	9	38	56
30	5	35	40	68	8	60	76	43	8	35	51
30	6	36	42	68	10	58	78	44	9	35	53
32	7	39	46	71	9	62	80	49	8	41	57
33	6	39	45	71	7	64	78	46	7	39	53
34	7	41	48	69	7	62	76	43	7	36	50
36	5	41	46	74	16	58	90	48	14	34	62
Range (rounded)											
30-34	33-41		40-48	68-73	58-62		76-84	43-47	35-39		51-57

Figure 9.7: Variation of Moisture Content Across a Dumped Clay Fill LU Embankment with a Grass Covered Slope (after O'Brien et al, 2004)

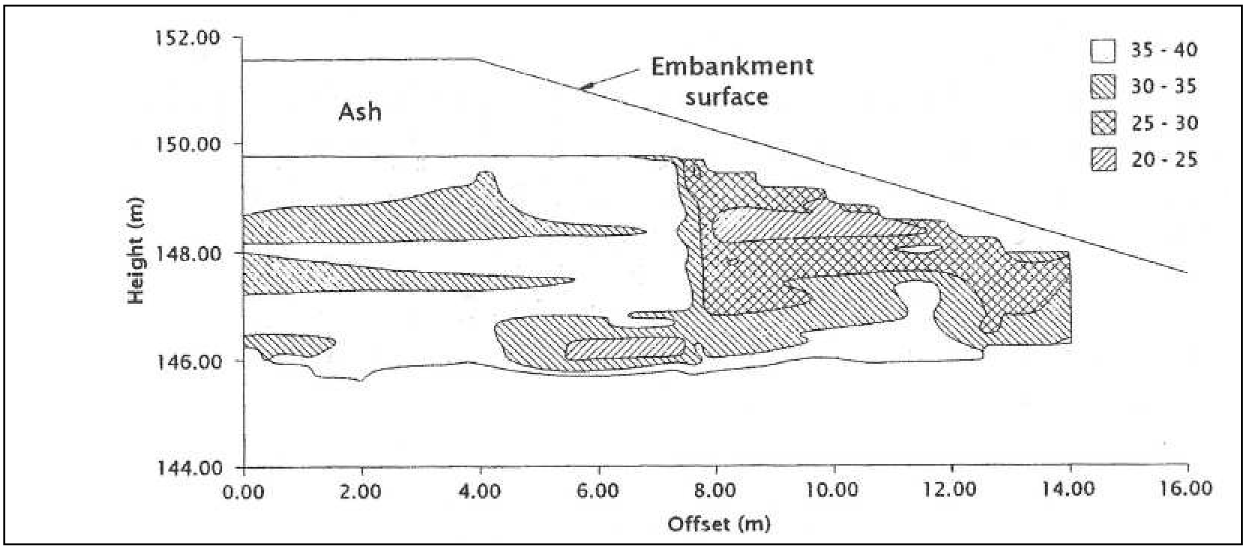
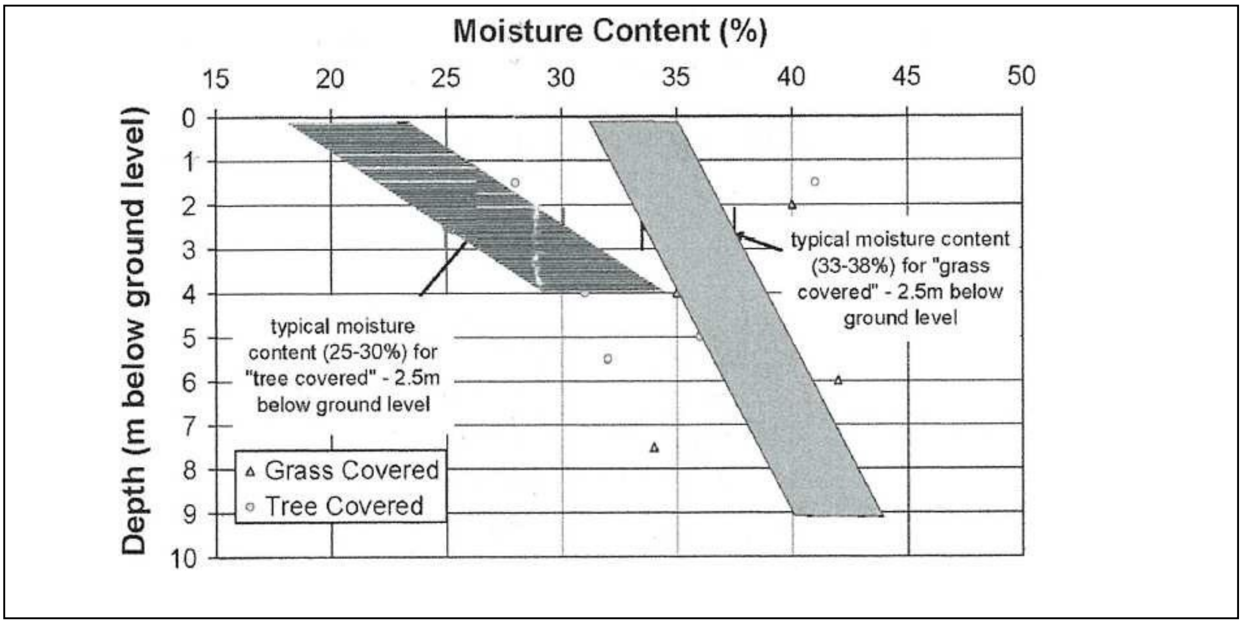


Figure 9.8: Typical Moisture Content Range Measured on "Tree-covered" and on "Grass-covered" Sections, on a 5m High Clay Fill LU Embankment, Capped with 1m-2m of Ash Slope (after Scott et al, 2007)



9.5.2.2 Peak Angle of Shearing Resistance

The recommendations of the LU Earth Structure Design Guide for the selection of the peak angle of shearing resistance are based on Stark and Eid (1997). Stark and Eid tested a series of remolded clay samples of various origins. The tests were carried out in ring shear apparatus at three vertical stress levels (50kPa, 100kPa, 400kPa). The secant fully softened angles of shearing resistance, at 400kPa, and the relation with the liquid limit and clay fraction are given in Table E 9.7.

Table 9.7: Secant Fully Softened Angles of Shearing Resistance at High Stress Levels (after Stark and Eid, 1997)

	φ_p´ (°)			α_p´ (°)		
LL (%)	CF<20%	20%<CF<45%	CF>45%	CF<20%	20%<CF<45%	CF>45%
25	34	-	-	29	-	-
30	33	-	-	29	-	-
40	32	29	25	28	26	23
50	31	28	24	27	25	22
60	31	**27**	**23**	27	**24**	**21**
70	30	**26**	**22**	27	**23**	**20**
80	29	**25**	**21**	26	**23**	**19**
90	-	24	20	-	22	19
100	-	24	19	-	22	18
120	-	23	18	-	21	17

Note:

1. The φ_p´ values most relevant for "typical" LUL Dumped Clay Fills are highlighted.
2. Values greater than φ_p´>30° be used with caution.
3. Angle of peak shearing resistance not to exceed a maximum value φ_p´$_{,max}$ =32°.
4. Liquid limit LL>80% is unusually high for LUL Dumped Clay Fills.

9.5.2.3 Peak Effective Cohesion

Table 9.8 provides the values of peak effective cohesion for a range of Natural Moisture Content (NMC) and Liquid Limit (LL), based on the recommendations of the LU Earth Structure Design Guide for normalised cohesion equal to cp´, ve=0.01 (Hvorslev intercept). The range of NMC and LL considered is typical for London clay fills. The corresponding ap´ values are given in Table 9.8.

Table 9.8: Peak Effective Cohesion Based on the Recommendations of the LU Earth Structure Design Guide for cp´, ve=0.01

LL (%)	c_p´ (kPa) NMC=30%	c_p´ (kPa) NMC=35%	c_p´ (kPa) NMC=40%	c_p´ (kPa) NMC=45%
60	4.0	1.8	0.8	0.4
70	6.7	3.3	1.6	0.9
80	9.9	5.3	2.8	1.6

Note:

1. Peak effective cohesion not to exceed a maximum value c_p´$_{,max}$ =12kPa.

Table 9.9: Corresponding ap´ Values for the Peak Effective Cohesion Values of Table 2.1

CF (%)	LL (%)	a_p´ (kPa) NMC=30%	a_p´ (kPa) NMC=35%	a_p´ (kPa) NMC=40%	a_p´ (kPa) NMC=45%
<20	60	3.4	1.5	0.7	0.3
	70	5.8	2.8	1.4	0.8
	80	8.7	4.6	2.5	1.4
20 to 45	60	3.6	1.6	0.7	0.4
	70	6.0	3.0	1.5	0.8
	80	9.0	4.8	2.6	1.4
>45	60	3.7	1.6	0.8	0.4
	70	6.2	3.1	1.5	0.8
	80	9.3	4.9	2.7	1.5

Note:

1. Maximum value of a_p´ based on maximum $c_p{}'_{,max}$ =12kPa.

9.5.2.4 O’Brien, 2007

O’ Brien (2007), reports peak strength values based on the results of a suite of laboratory tests on block samples of high plasticity dumped clay fill, with clay fractions between 50% and 60%, obtained from LUL railway embankments. The values reported vary between φp´= 22°, c p´=1kPa and φp´= 23°, c p´=10kPa (corresponding to αp´= 20°, a p´=1kPa and αp´= 21°, a p´=9kPa).

For clay fraction greater than 45% and liquid limits of between 70% and 80% the cp´ values correspond to natural moisture contents of between 29% (for cp´=10kPa) and 46% (for cp´=1kPa), and the measured φp' values are about 1° higher than those derived from the Stark and Eid (1997) charts.

The index test data for a range of sites from where the samples were cut is given in the Section 9.5.8 of this Appendix.

9.5.3 London Clay Peak Strength

9.5.3.1 Bishop et al, 1965 – Ashford Common, Surrey

Bishop et al (1965), reported London Clay peak strength results of drained (D) and consolidated-undrained (CU) triaxial tests, on horizontal (Hor.) and vertical (Vert.) specimens, obtained from bock samples cut from the Ashford Common shaft in Surrey. The results are shown in Table 7.10. Additional information regarding the clay properties is given in the Appendix.

It is important to note that these tests were carried out on 38mm diameter specimens. Hence, these tests are likely to measure the “intact” strengths of the clay, rather than the “bulk” or “mass” properties (i.e. the influence of fissures are unlikely to be measured).

Table 9.10: London Clay Peak Strength Data for all Levels at Confining Stress Ranging from 70kPa to 700kPa (after Bishop et al, 1965)

Level	Depth (m)	Test	φ_p' (°)	c_p' (kPa)	α_p' (°)	a_p' (kPa)
A (1)	9.14	D Vert.	28	17	25	12
(5)		D Vert.	26	119	23	107
B	15.24	D Vert.	25	0	23	0
		CU Vert.	25	15	23	14
C	20.12	D Vert.	25	130	23	118
		D Hor.	26	135	23	122
		CU Vert.	23	124	21	114
		CU Hor.	26	105	24	94
D	27.74	D Vert.	23	174	21	160
		CU Vert.	27	108	24	97
E	34.75	D Vert.	29	93	26	81
		D Hor.	27	134	25	119
		CU Vert.	26	142	23	128
		CU Hor.	29	94	26	82
F	42.06	D Vert.	29	178	26	155
		CU Vert.	28	252	25	223

Note:

1. A-level blocks 1 and 5 gave significantly different values.

All tests were carried out on unweathered London Clay. The deeper samples (levels D, E, F) were most probably carried out on what are nowadays classified as the Unit A of the London Clay, which is stronger than Unit B, or the weathered London Clay.

As reported by Bishop et al, at levels C, D, E and F a sufficient number of intact samples could be cut without too much difficulty, and the results of a very limited number which failed prematurely on an apparent fissure are not included in the data, which therefore approaches that of intact clay.

However, at A and B levels some apparently intact samples failed at strengths very much below the average, while others, for example Block A5 (Table 9.10), indicates values of the shear strength parameters little different from those of deeper samples. This suggests that a change in the nature and scale of the fissure pattern may account for the marked decrease in the c p´ value as the surface is approached.

Within the pressure range shown in Table 9.10 there appear to be no very marked differences between the “intact” strength parameters for vertical and horizontal samples, whether determined in drained or consolidated-undrained tests.

For levels C, D and E, the average peak strength reported by Bishop et al, is $\varphi p' = 26°$ and $c\ p' = 124$kPa (corresponding to $\alpha p' = 24°$ and $a\ p' = 111$kPa), for confining stress ranging from 70kPa to 700kPa.

9.5.3.2 Burland and Kalra, 1968 – Queen Elizabeth II Conference Center, Central London

These tests were carried out on 100mm diameter specimens obtained from push-in thin wall sampling tubes. A feature of these tests was that the were carried out at relatively low effective confining stresses, which were reasonably representative of the relatively low effective stresses behind a retaining wall, and close to the base of excavations in front of embedded retaining walls.

Burland and Kalra, 1968 report that the failure envelope is curved and gives a secant $\varphi'_{p,sec}$= 35°for confining pressures less than 100kPa, while at higher confining pressures the failure envelope flattens as the stresses approach the in-situ vertical effective stress (160kPa to 195kPa). For confining pressures values in excess of the in-situ vertical effective stress a secant $\varphi'_{p,sec}$= 22°is given. The test data and the secant peak strength envelops are shown in Table 9.11.

Table 9.11: London Clay Test Data and Secant Peak Strength Envelops (after Burland and Kalra, 1968)

London Clay Unit	Test	s´ (kPa)	t (kPa)
Unit B	Undrained	31.5	18.5
		36.9	20.0
		64.6	37.8
		76.9	47.4
		89.2	43.7
		144.6	67.4
		204.6	78.5
	Drained	7.7	8.1
		57.7	37.0
		101.5	60.7
		140.8	60.7

London Clay Unit	Confining Stress	$\varphi'_{p,sec}$ (°)	$a'_{p,sec}$ (kPa)
Unit B	<100kPa	35	30
	>100kPa	22	21

9.5.3.3 Chandler and Apted, 1988 – South Ockendon, Essex

Chandler and Apted (1988) reported peak strength results of unconsolidated-undrained triaxial tests at s’ = 60kPa to 130kPa (i.e. vertical effective stress ranges from 100kPa to 200kPa), on London Clay specimens (100mm) for various levels of clay weathering. The samples were obtained from U100 samples from South Ockendon in Essex. The results are summarized in Table 9.12. Additional information regarding the sample properties is given in the Appendix.

Chandler and Apted (1988), made the assumption that the effective angle of peak shearing resistance for weathered London Clay is equal to $\varphi p'$= 20°and adjusted the cohesion to fit the failure envelope to the data for the different weathering levels. However, they reported that different values of $\varphi p'$= 27°and cp'=12kPa are obtained if the best fit lines to the data are used (corresponding to $\alpha p'$= 24°and a p'=11kPa).

Table 9.12: London Clay Peak Strength Data at s´ = 60kPa - 130kPa (after Chandler and Apted, 1988)

Weathering	Zone	Depth (m)	φ_p´ (º)	c_p´ (kPa)	α_p´ (º)	a_p´ (kPa)
Unweathered	I	>4.6	20	28	19	26
Partially	II	4.2-4.6	20	28	19	26
Partially	III	2.8-4.2	20	18	19	17
Fully	IV	1.0-2.8	20	18	19	17
(lower bound)	III, IV	1.0-4.2	20	9	19	8

9.5.3.4 Hight and Jardine, 1993 – Central London

Hight and Jardine (1993), reported the peak strength values shown in Table 9.13, resulting from undrained triaxial compression tests, on London Clay specimens (100mm). The specimens were obtained from thin- wall tube samples form various sites in Central London. Additional information regarding the sample properties is given in the Appendix.

Table 9.13: London Clay secant peak strength envelopes (after Hight and Jardine, 1993

s´ (kPa)	Depth (m)	φ_p´ (º)	c_p´ (kPa)	α_p´ (º)	a_p´ (kPa)
0-165	8-12	34	0	29	0
	16-24	56	0	40	0
>165	8-12	12	71	12	69
	16-24	23	76	21	70

9.5.3.5 Hight et al, 2007 and Gasspare et al, 2007 – Heathrow Airport, Terminal 5

Hight et al (2007) reported London Clay peak strength envelopes for the different London Clay lithological units summarized in Table 9.14. Undrained triaxial compression (UTC) tests were carried out on specimens (100mm diameter) obtained from rotary-core samples and block samples from Terminal 5 (T5) at Heathrow Airport. The stress paths of the tests performed are given in the Appendix.

Gasspare et al (2007), plotted the peak strength results obtained from all T5 tests in a single τ-σ´ plot (given in the Appendix). For vertical effective stress up to 600kPa, a secant angle of peak shearing resistance equal to φp´= 33°(or αp´= 28.6°), with c´ = 0, can be derived if the best fit line to the ata is used. In the same graph Burland (1990) post-rupture envelope is also plotted, which gives a secant angle of shearing resistance equal to φp´= 23°(or αp´= 21.3°), with c´ = 0, for vertical effective stress up to 300kPa.

Table 9.14: London Clay Peak Strength Values from Terminal 5 at Heathrow Airport (after Hight et al, 2007)

LC Unit	Depth (m)	LL (%)	w (%)	Test	φ_p' (°)	c_p' (kPa)	α_p' (°)	a_p' (kPa)
Unit C	5-8	65-70	25-27.5	UTC	31	0	27	0
Unit B2(c)	11-12.5	65	22-25	UTC	34	0	29	0
Unit B2(b)	16-17	65-75	25-27.5	UTC	35	0	30	0
Unit B2(a)	24-28	65-75	24-27.5	UTC	31	64	27	55
Unit A3	31-40	60-65	22.5-27.5	UTC	32	88	28	75

9.5.4 Peak Strength Comparison

A comparison between the peak strength of natural London Clay and London Clay Fill is presented in the following figures. For the purposes of comparison, the Clay Fill is assumed to have a clay fraction of greater than 45% and a liquid limit of between 70% and 80%. The natural moisture content is assumed to be between 35% and 40%.

These index properties are believed to be representative of the majority of LU Dumped Clay Fills, derived from natural London Clay. The important influence of moisture content is discussed in Section E5. The above range of moisture contents are fairly typical of the central core of embankments, which are usually wetter than at shallow depths below side slopes, which can often be affected by vegetation induced desiccation.

Figure 9.9 shows peak strength envelopes for a wide range of index properties, for Embankment Clay Fill, calculated based on the recommendation of the LU Earth structure Design Guide. The values of c´ vary between 0.9kPa and 9.9kPa with φ´ values between 21°and 27°.

Figure 9.10 and 9.11 show the unfactored and factored peak strength envelopes respectively for clay fraction greater than 45%, which are representative of Dumped Fills with London Clay origin (as discussed above), calculated in accordance with the LU Earth structure Design Guide.

In Figures 9.12 to 9.16 the factored peak strength of natural London Clay, extracted form the technical literature, and of London Clay Fill given in Figure 9.11 are compared. For the purposes of the comparison in this Appendix, the partial factors of safety applied on the peak strength of natural London Clay data and Clay Fill are equal to 1.25 and 1.1 respectively.

In Figure 9.17 the factored strength envelopes shown in Figure 9.11 are plotted against the factored strength envelopes reported by O'Brien (2007) from tests carried out on Clay Fill samples obtained from London Underground railway embankments.

From the Figures in can be concluded that the factored peak strength envelopes of natural London Clay, obtained from high quality and well documented technical literature, is in general, higher than the range of peak strength envelopes calculated for London Clay Fill using the LU Design Guide.

This is also shown in Tables 9.15 and 9.16 where the natural London Clay mobilised factored peak strength (tmob) values obtained by the technical literature are divided by the factored London Clay Fill peak strength calculated using the LUL design Guide for a vertical effective stress range between 50kPa and 100kPa approximately. The natural London Clay mobilised factored peak strength (tmob) is greater than the factored London Clay Fill peak strength, calculated using the LUL design Guide, by 5% to 39% in Table 9.15 and 11% to 47% in Table 9.16. The reason why the natural London Clay typically has a much higher peak strength than London Clay fill, is that the natural clay possesses an additional component of strength due to the development (over geological timescales) of "structure". Strength due to "structure" is lost, during end-tipping of the clay fill and due to its clod/matrix macro-fabric, O'Brien (2007). Weathering of the natural clay, can also lead to a loss of "structure", usually known as "destructuring"; the data of Chandler and Apted (1988) given in 9.5.3 above clearly shows this.

Table 9.15: Ratio of Natural London Clay Mobilised Factored Peak Strength and Factored Dumped London Clay Fill Peak Strength, Calculated Based on the LUL Design Guide, at 50kPa to 100kPa Vertical Effective Stress (LL=70%, NMC=35%, CF>45%)

Data Source:	LUL DG LL=70% MNC = 35%	Burland & Karla, 1968	Chandler & Apted, 1988 (<4m)	Chandler & Apted, 1988 (lower bound)	Hight et al, 2007 (Unit C)	Hight et al 2007 (Unit B2b)
Factored t_{mob} s'=50kPa	19.60	24.10	27.20	20.50	20.80	23.13
Ratio	1.00	1.46	1.39	1.05	1.06	1.18

Table 9.16: Ratio of Natural London Clay Mobilised Factored Peak Strength and Factored Dumped London Clay Fill Peak Strength, Calculated Based on the LUL Design Guide, at 50kPa to 100kPa Vertical Effective Stress (LL=80%, NMC=40%, CF>45%)

Data Source:	LUL DG LL=80% MNC = 40%	Burland & Karla, 1968	Chandler & Apted, 1988 (<4m)	Chandler & Apted, 1988 (lower bound)	Hight et al, 2007 (Unit C)	Hight et al 2007 (Unit B2b)
Factored t_{mob} s'=50kPa	18.50	24.10	27.20	20.50	20.80	23.13
Ratio	1.00	1.55	1.47	1.11	1.12	1.25

Figure 9.9: Unfactored Peak Strength Envelopes for a Range of Index Properties for Embankment Fill

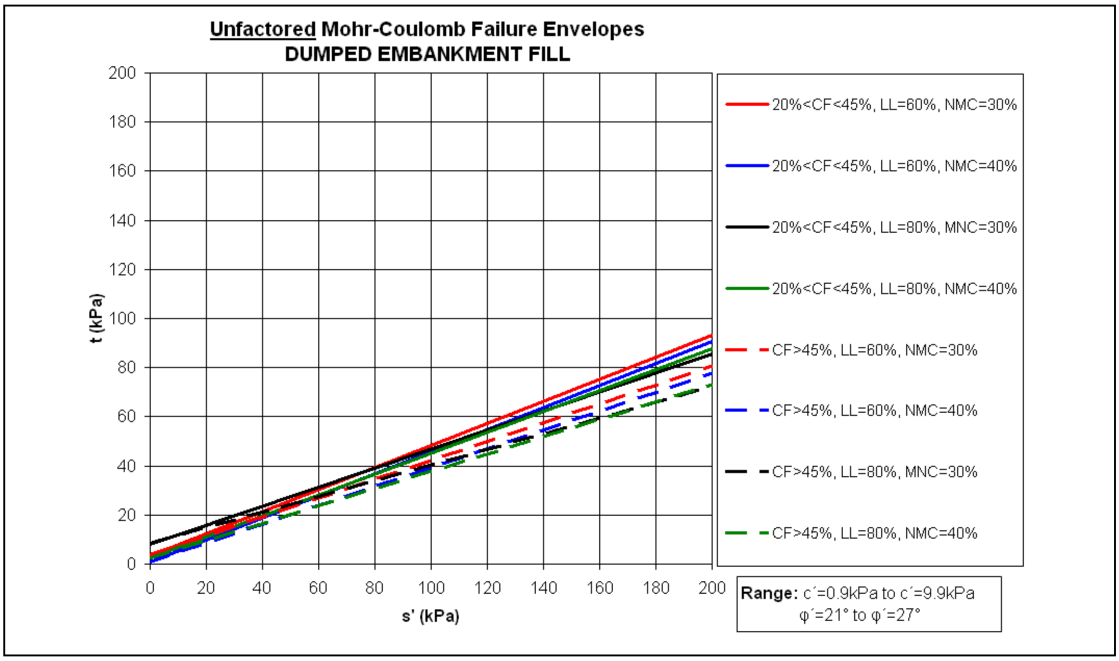


Figure 9.10: Unfactored Peak Strength Envelopes for a Range of Index Properties Typical for London Clay Fill

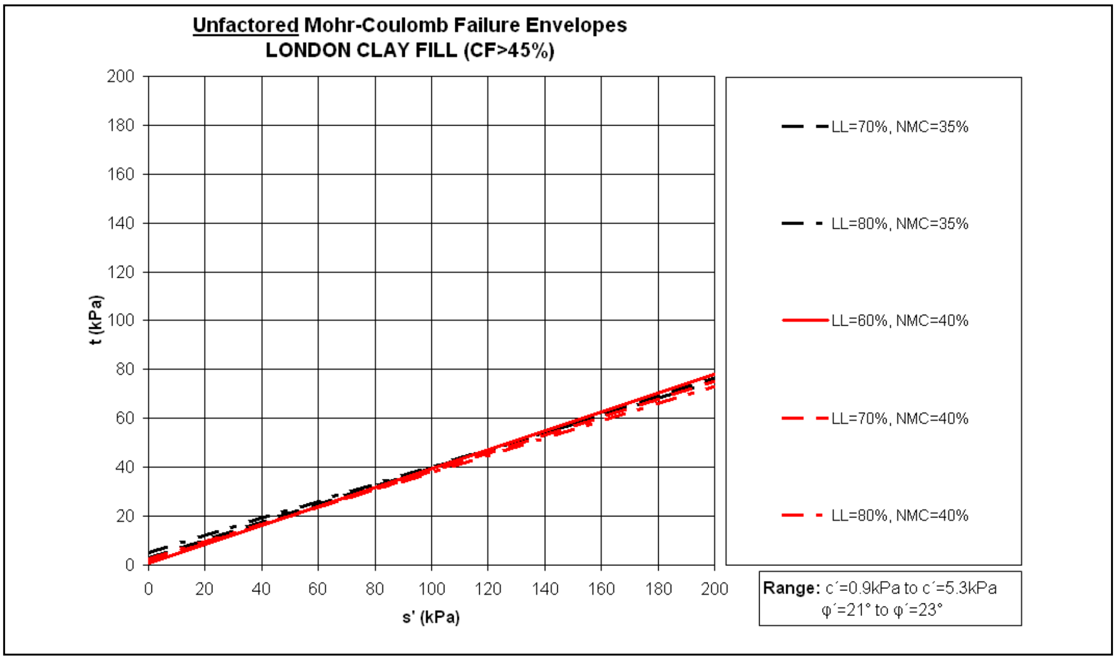


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Figure 9.11: Factored Peak Strength Envelopes for a Range of Index Properties Typical for London Clay Fill

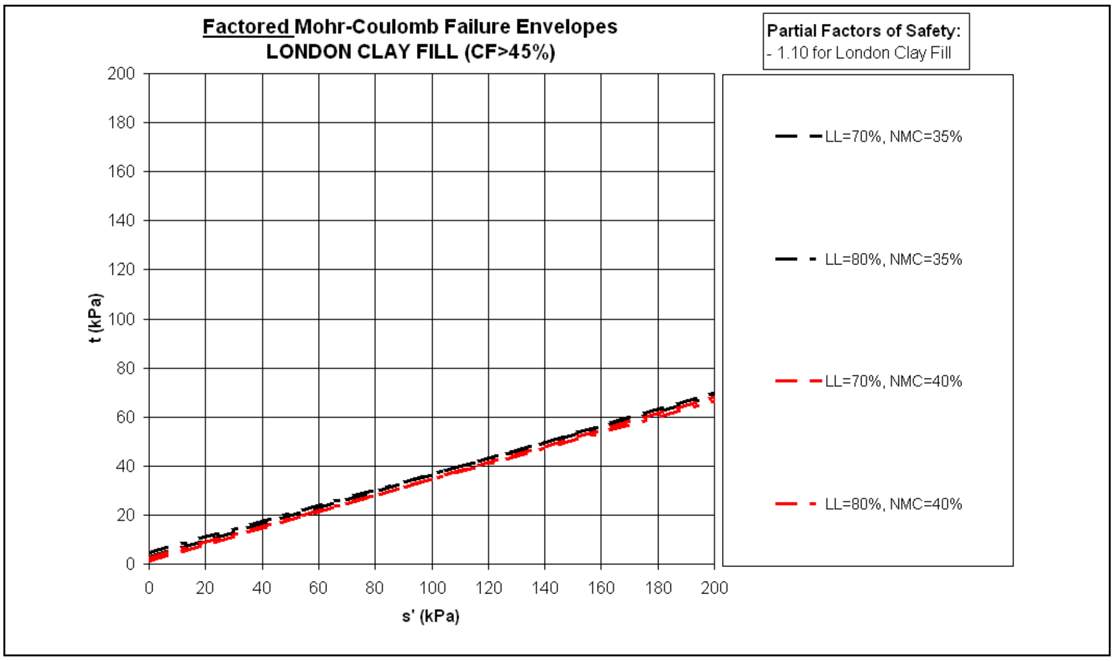

Figure 9.12: Comparison Between Factored Peak Strengths of Natural London Clay (after Bishop et al, 1965) and London Clay Fill

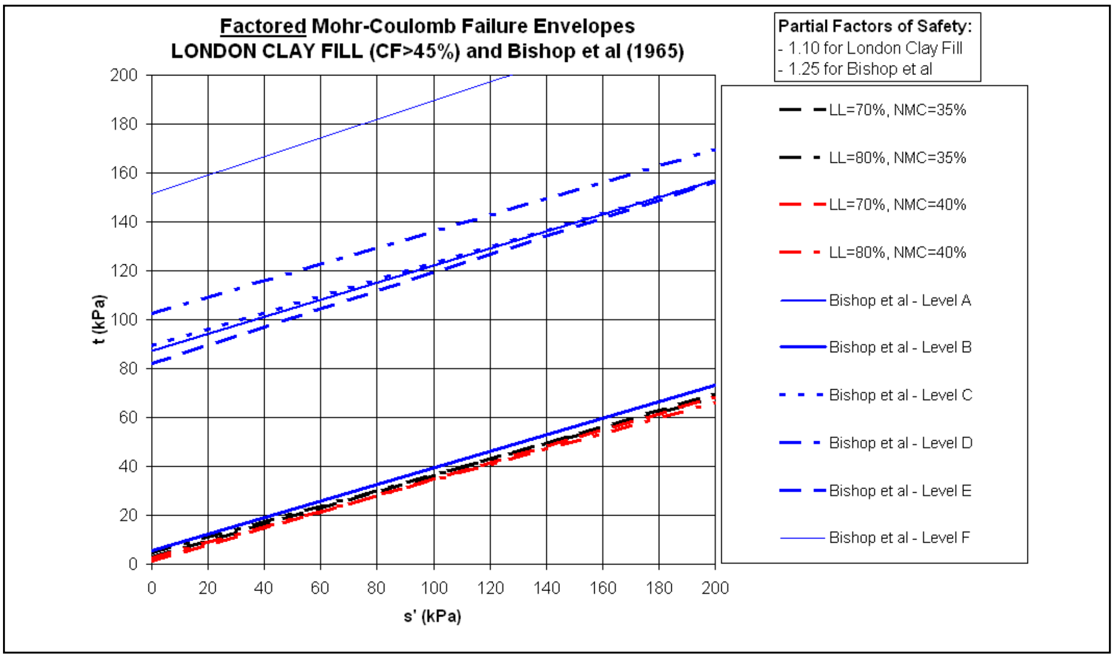

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Figure 9.13: Comparison Between Factored Peak Strengths of Natural London Clay (after Burland & Kalra, 1986) and London Clay Fill

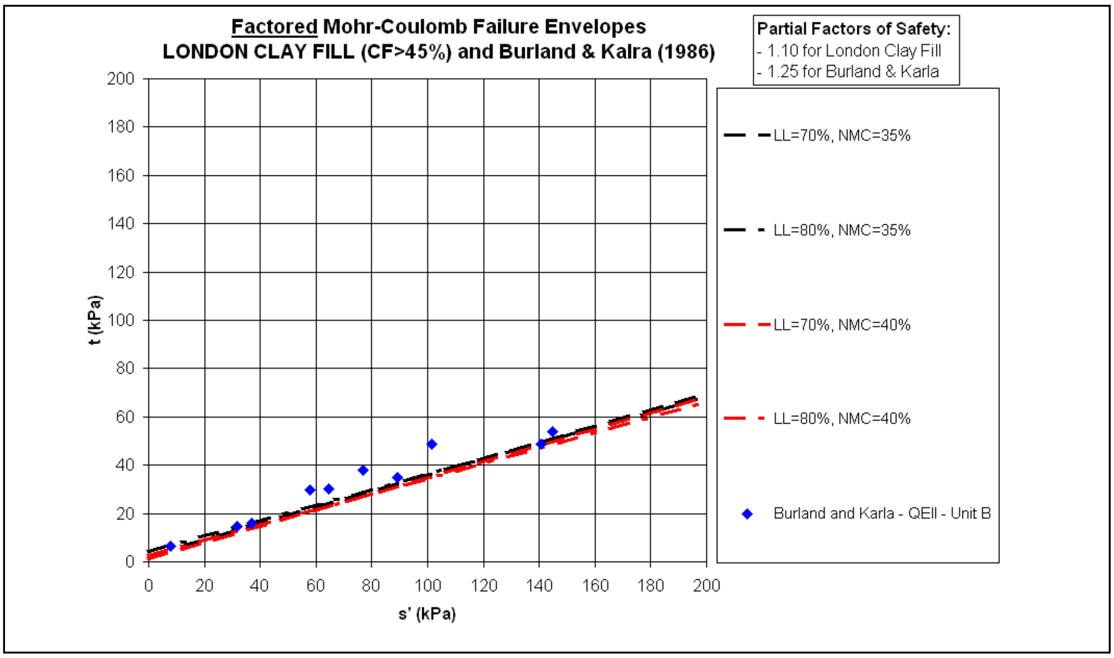

Figure 9.14: Comparison Between Factored Peak Strengths of Natural London Clay (after Chandler & Apted, 1988) and London Clay Fill

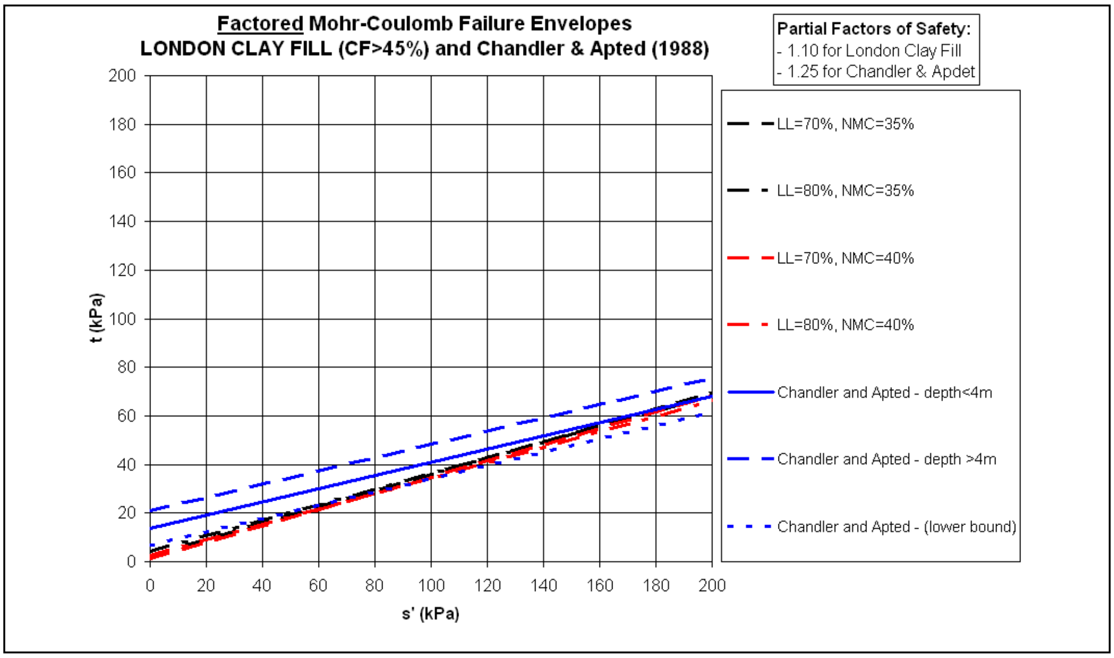

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Figure 9.15: Comparison between factored peak strengths of natural London Clay (after Hight & Jardine, 1993) and London Clay Fill

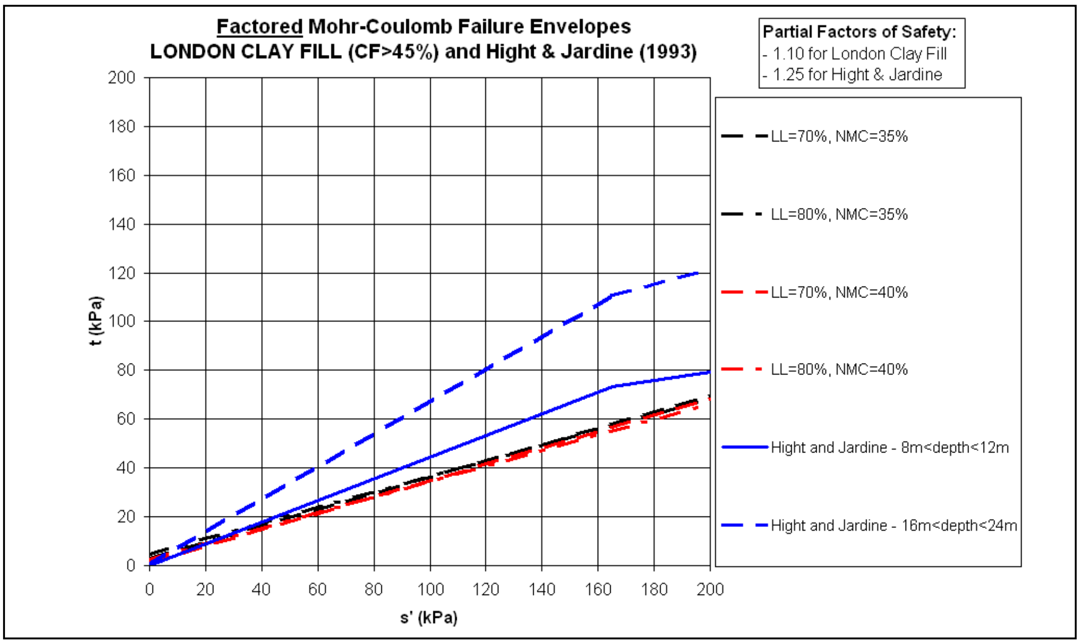

Figure 9.16: Comparison Between Factored Peak Strengths of Natural London Clay in Triaxial Compression (after Hight et al, 2007) and London Clay Fill

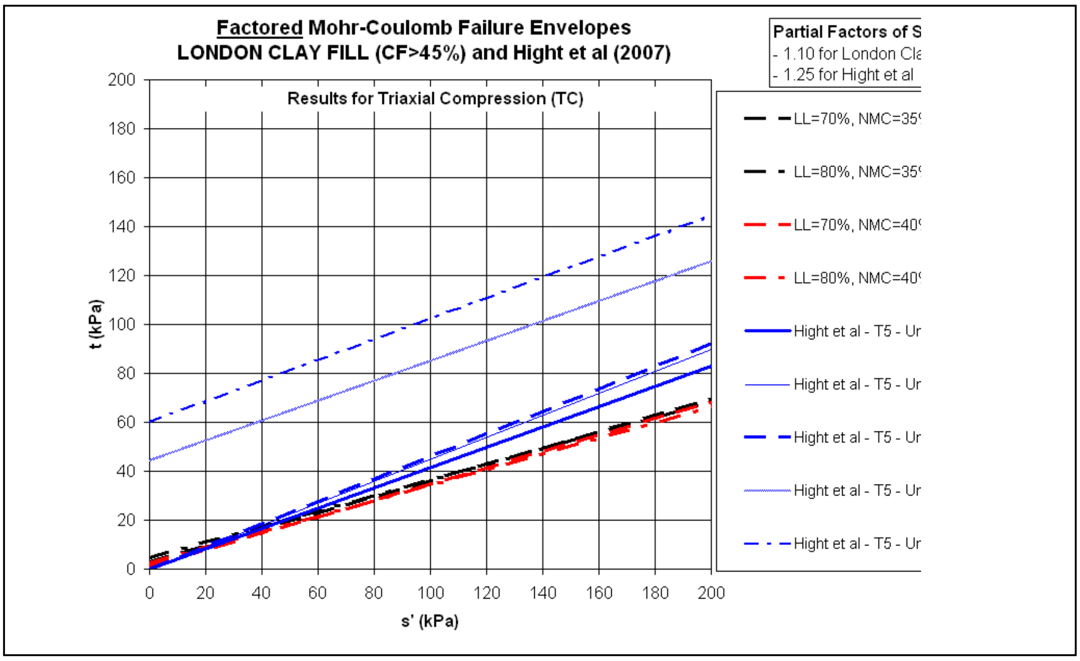

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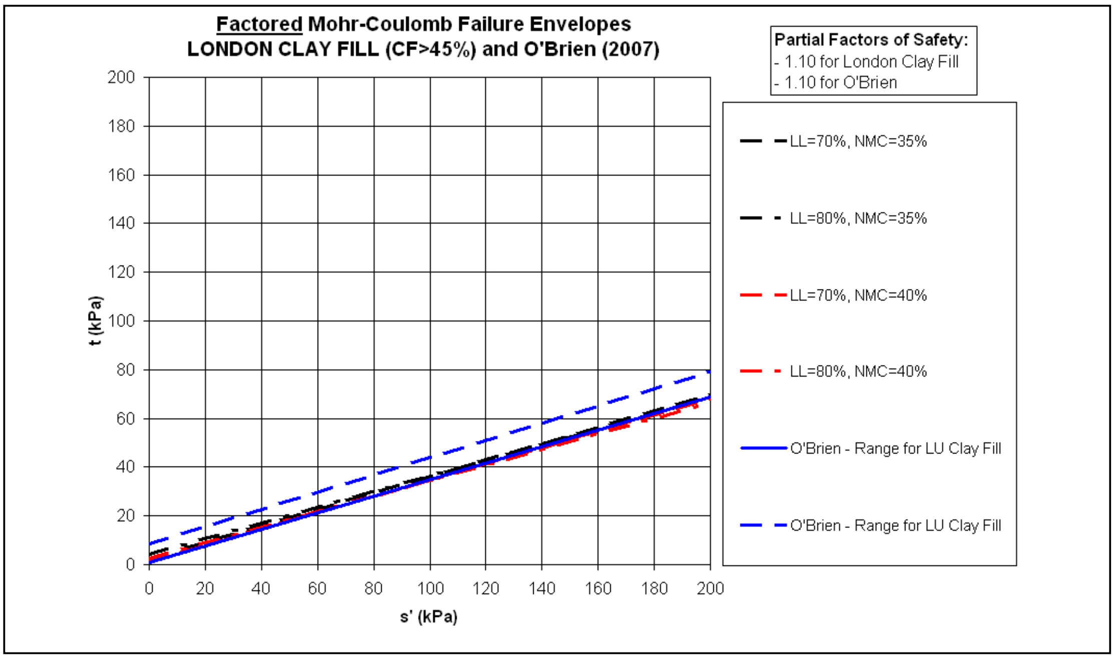
Figure 9.17: Comparison between Factored Peak Strengths of Clay Fill Obtained from Various London Underground Railway Embankments (after O'Brien, 2007) and London Clay Fill Peak Strength Parameters from Design Guide



9.5.5 Selection of Cautious Peak Strength - Influence of Natural Moisture Content

The LU design guide provides a series of charts to assess c´ and φ´ values from the basic index properties for the clay fill, namely clay fraction, liquid limit and natural moisture content.

The value of c´ is sensitive to the assumed natural moisture content, and care is required to avoid excessively high values being selected. At shallow depths below embankment side slopes, the clay fill can be desiccated due to the suctions mobilised by vegetation, especially mature high water demand trees during the summer months. At these locations, natural moisture contents of less than 25% could be recorded. Based on Figures 9.17 and 9.18 the influence of natural moisture content on c´ is summarised in Tables 9.17 and 9.18 and shown on Figures 9.18 and 9.19, for liquid limits of 70% and 80% respectively. Failure envelopes for moisture contents between 25% and 45% are shown. It should be noted that the combined values of effective cohesion and friction angle should be used for the natural moisture contents and liquid limits shown in Table 9.17 and 9.18.

From this is can be seen that the mobilised strength, at a moisture content of 25% is about 20% to 30% greater than that at a moisture content of 35% depending on the liquid limit. The design guide recommends values of c p´ greater than 12kPa are NOT used, irrespective of site specific data.

In general, it is recommended that a large number of natural moisture content and Atterberg limit determinations are made, in order to ensure that a representative cautious value is selected. For an embankment, say 250m long, more than 35 to 40 Atterberg limits, moisture content and clay fraction measurements should be made. The sampling locations should be considered, to ensure they are representative of the areas potentially affected by the deep seated failure mechanisms. The mean value should NOT be used, and values close to the mean plus one standard deviation will generally be more appropriate.

Based on previous LU embankment investigations, the above guide would usually lead to moisture contents of 33% or more, which would indicate c´ values of less than c p´=6.5kPa (assuming a liquid limit of 80% and a clay fraction of more than 45%). If moisture contents of less than 30% are deemed to be representative, then some consideration should be given to the clay fill permeability, the potential for seasonal changes in moisture content and the longevity of vegetation, in the context of the particular design check being carried out (eg. assessment of current condition vs 120 year design life for remedial works).

Table 9.17: Effect of Natural Moisture Content (NMC) on Effective Peak Cohesion c p´ for Liquid Limit LL=70%, Clay Fraction >45%, and Mobilised Shear Strength t´mob at s´=50kPa

NMC (%)	c_p´ (kPa)	φ_p´ (°)	a_p´ (kPa)	α_p´ (°)	$t´_{mob}$ (kPa) at s´=50kPa
25	12.0	22	11.1	20	30
30	6.7	22	6.2	20	26
35	3.3	22	3.0	20	23
40	1.7	22	1.5	20	22
45	0.9	22	0.8	20	21

Table 9.18: Effect of Natural Moisture Content (NMC) on Effective Peak Cohesion c p´ for Liquid Limit LL=80% and Mobilised Shear Strength t´mob at s´=50kPa

NMC (%)	c_p´ (kPa)	φ_p´ (°)	a_p´ (kPa)	α_p´ (°)	$t´_{mob}$ (kPa) at s´=50kPa
25	12.0	21	11.2	19	29
30	9.9	21	9.3	19	28
35	5.3	21	4.9	19	24
40	2.9	21	2.7	19	22
45	1.6	21	1.5	19	21

Figure 9.18: Effect of Natural Moisture Content (NMC) on Effective Peak cohesion c p´ for London Clay fill (for liquid limit LL=70%., clay fraction >45%)

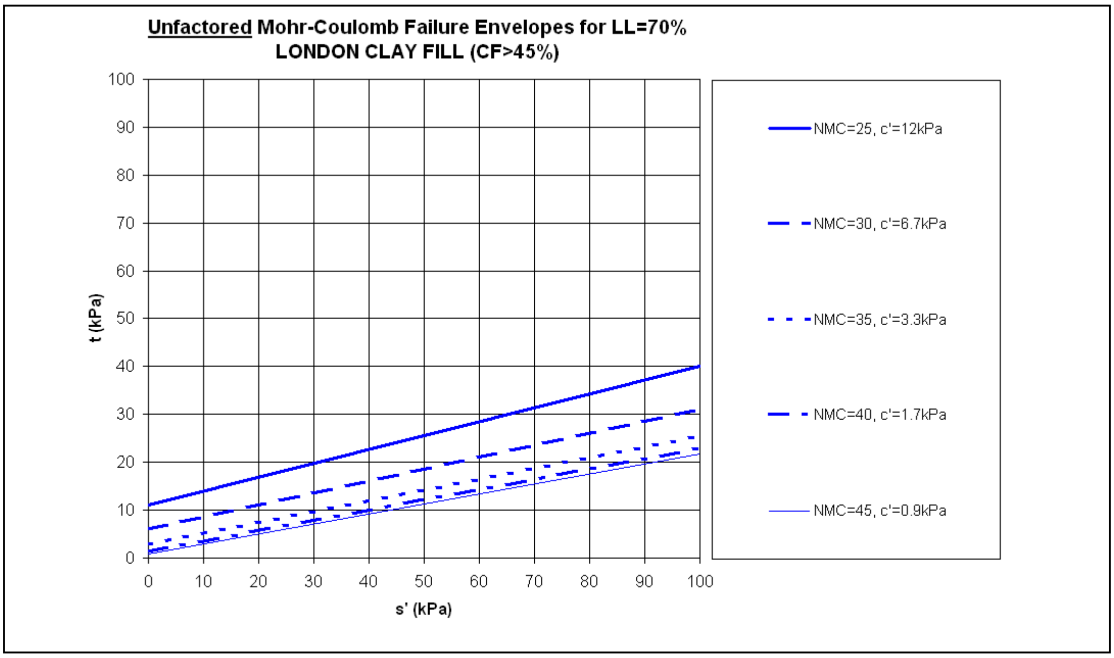
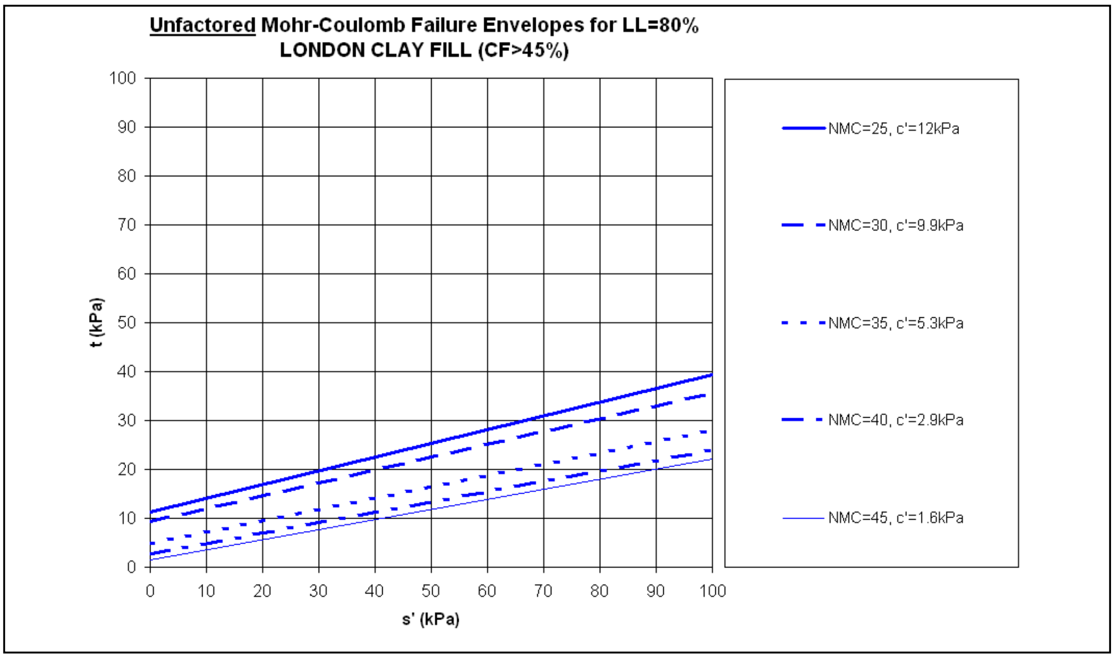


Figure 9.19: Effect of Natural Moisture Content (NMC) on Effective Peak Cohesion c p´ for London Clay fill (for Liquid Limit LL=80%., Clay Fraction >45%)



9.5.6 Conclusions

This Appendix provides a comparison between the peak strength of London Clay (based on high quality published data) and the peak strength of London Clay fill (derived from Figures 3.4.11 and 3.4.12 of this Design Guide). This shows that if the guidance is carefully implemented, then relatively conservative peak strength parameters can be derived for both London Clay fill and natural London Clay, based on appropriate site specific index test data and application of Figures 3.4.11 and 3.4.12. When partial factors of 1.25 are applied to natural London Clay peak strengths, the resulting factored strengths are higher than the factored strengths derived for London Clay fill (using a Partial Factor of 1.1). However, as noted in 9.5.5 above, care is required in selection of natural moisture content, and its effect on effective peak cohesion. The proposed approach to assessing progressive failure for deep seated instability in Chapters 4, 7 and 8 (with Partial Factors of 1.1.) is consistent with Eurocode 7 requirements, and specifically with Clause 2.4.6 and the direct assessment of geotechnical parameters based on an assessment of worst credible conditions.

9.5.7 References

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9.5.8 Supplementary Data

1. Bishop et al, 1965

Table 1

Index properties

Level	Depth below G.L.: ft	Unit weight: lb/cu. ft	Rel. density of grains: G_s	Water content: Field %	Water content: Lab. %	Liquid limit: %	Plastic limit: %	Plasticity index: %	Clay fraction: % $<2\mu$	Activity: $\frac{w_{PI}}{\%\ \text{clay}}$	Undrained strength vertical: c_u lb/sq. in.
A	30	129	2·73	22·9	22·4	60	24	36	42	0·86	37
B	50	127	2·75	26·9	25·8	69	29	40	59	0·68	33
C	66	127	2·77	25·6	24·8	71	29	42	53	0·79	44
D	91	129	2·72	23·4	22·8	63	26	37	47	0·79	54
E	114	128	2·75	25·5	24·2	70	27	43	57	0·75	60
F	138	128	2·74	24·2	23·6	69	29	40	60	0·67	81

Table 2

Site	w_{LL}	w_{PL}	w_{PI}	% $<2\mu$	Activity
Bradwell	95	30	65	52	1·25
Waterloo Bridge	77	27	50	53	0·95
Ashford Common shaft . .	67	27	40	54	0·76

Table 3

Effective stress shear parameters

Level	Range of σ_3': 10–100 lb/sq. in.								Least slope								Depth below G.L.: ft
	Drained tests				Consolidated-undrained tests				Drained tests				Consolidated-undrained tests				
	Vert.		Horiz.		Vert.		Horiz.		Vert.		Horiz.		Vert.		Horiz.		
	c' lb/sq. in.	ϕ'	c'	ϕ'	c'	ϕ'	c'	ϕ'	c'	ϕ'	c'	ϕ'	c'	ϕ'	c'	ϕ'	
A* (1) . . (5) . .	2·0 17·2	28·1 25·6	—	—	—	—	—	—	—	—	—	—	—	—	—	—	30
B . .	0	25·1	—	—	2·2	25·1	—	—	—	—	—	—	—	—	—	—	50
C . .	18·8	25·1	19·6	25·8	18·0	23·0	15·2	26·4	112	9·5	89	12·0	103	9·8	—	—	66
D . .	25·2	23·2	—	—	15·7	26·7	—	—	—	—	—	—	—	—	—	—	91
E . .	13·5	29·2	19·4	27·5	20·6	25·6	13·6	28·8	113	8·2	157	9·0	97	10·5	123	10·5	114
F . .	25·8	29·4	—	—	36·6	27·7	—	—	—	—	—	—	—	—	—	—	138

* A-level blocks 1 and 5 gave significantly different values

2. Burland and Kalra, 1968

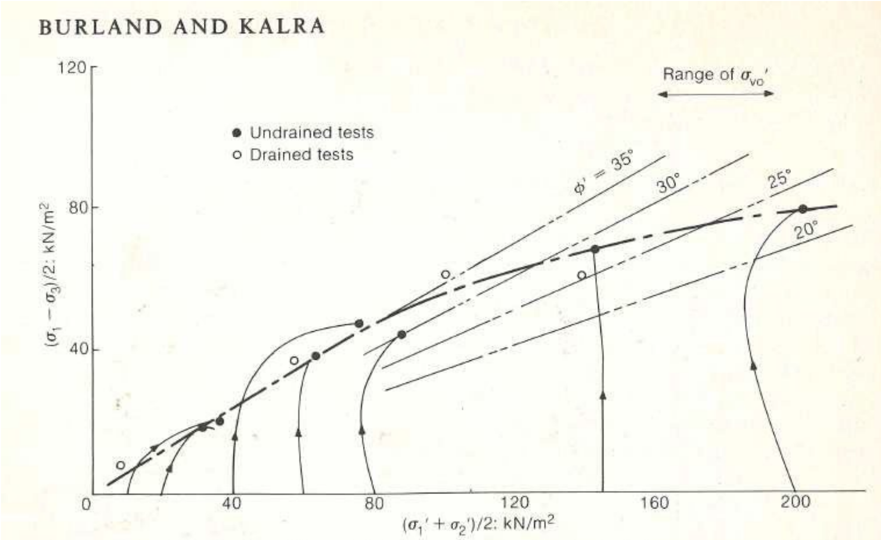

Fig. 5. Results of consolidated drained and undrained triaxial tests on samples from between 97 m and 92 m SD

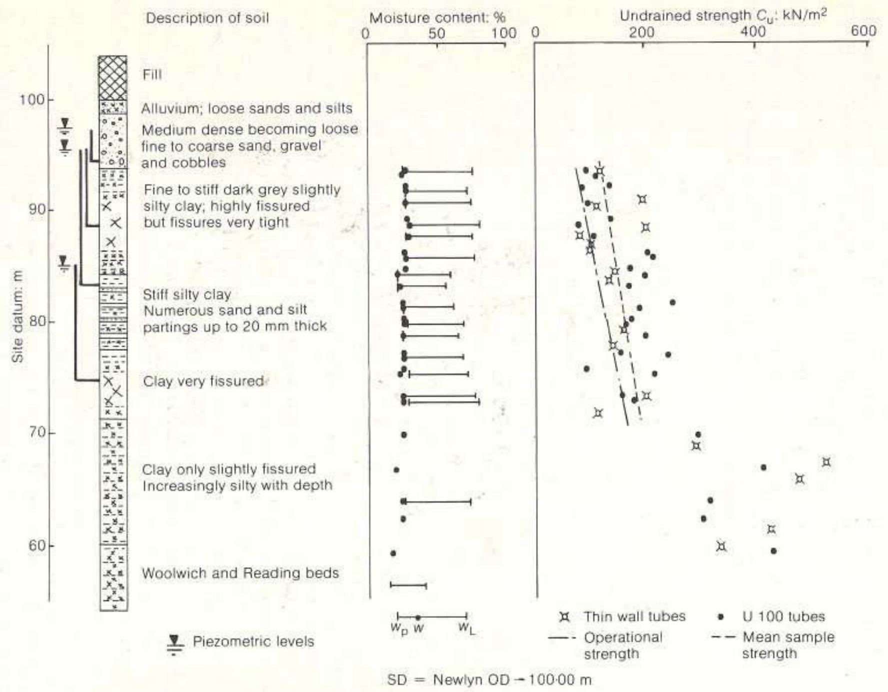

Fig. 3. Soil profile, piezometric levels and soil properties

3. Chandler and Apted, 1988

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TABLE 4. *Average index properties of the South Ockendon triaxial test specimens*

Weathering zone	*Depth (m)*	*LL (%)*	*PL (%)*	w_{nat} (%)	*LI (%)*	*CF (%)*	A_c	G_s
IV	1.0–2.8	73	29	32	0.07	52	0.85	2.73
III	2.8–4.2	80	30	32	0.04	56	0.88	2.73
II	4.2–4.6	78	30	32	0.04	58	0.83	2.73
I	>4.6	81	30	31	0.02	59	0.86	2.69

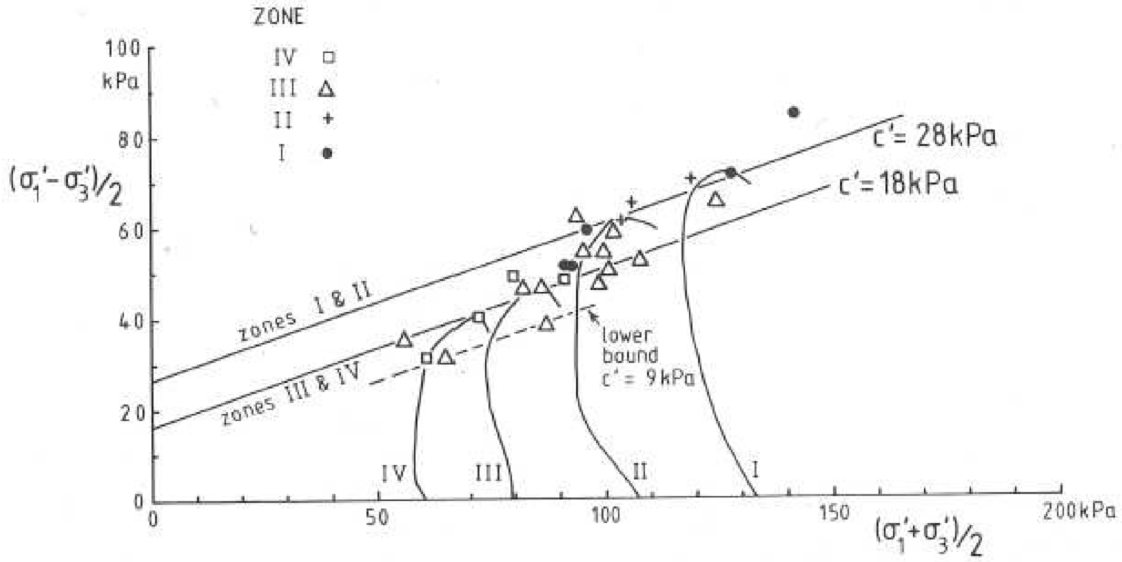

FIG. 4. Undrained triaxial compression tests, London Clay, South Ockendon; ϕ' is taken as 20°.

4. Hight and Jardine, 1993

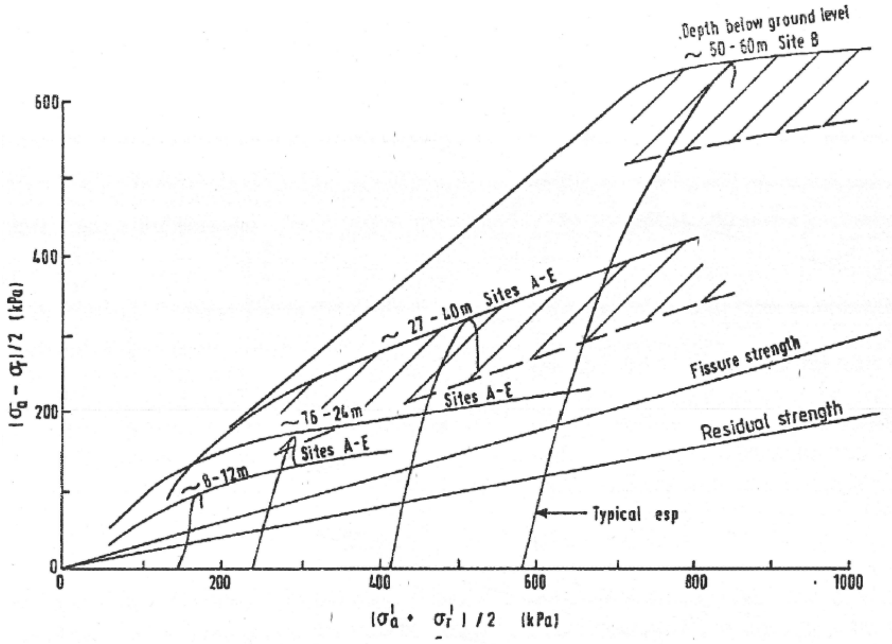

Fig. 11. Summary of upper and lower bound failure envelopes for London Clay.

5. Hight et al, 2007

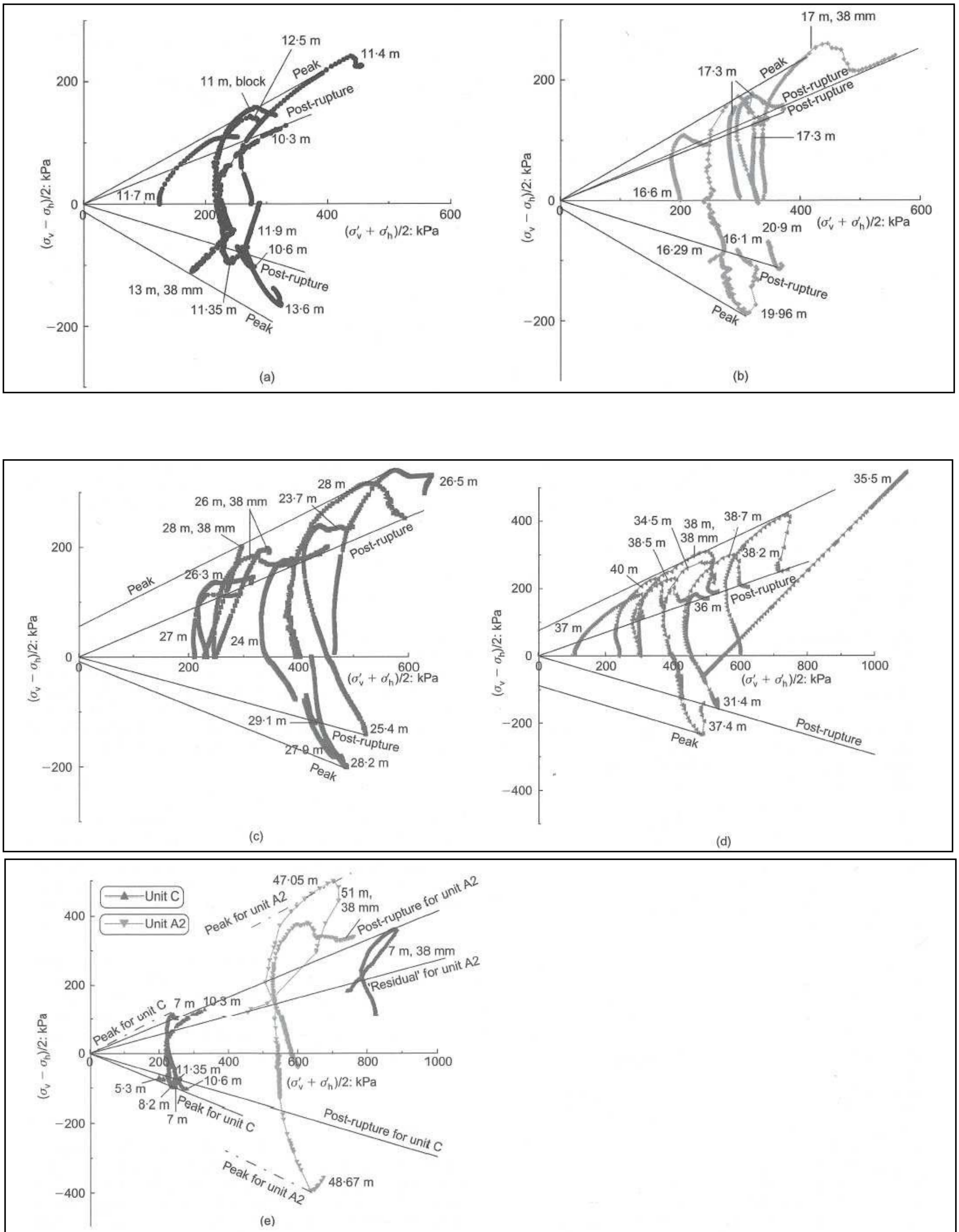

Fig. 17. Effective stress paths in undrained triaxial compression and extension (100 mm diameter rotary-cored samples except where noted): (a) unit B2(c); (b) unit B2(b); (c) unit B2(a); (d) unit A3; (e) units C and A2

6. Gasparre et al, 2007

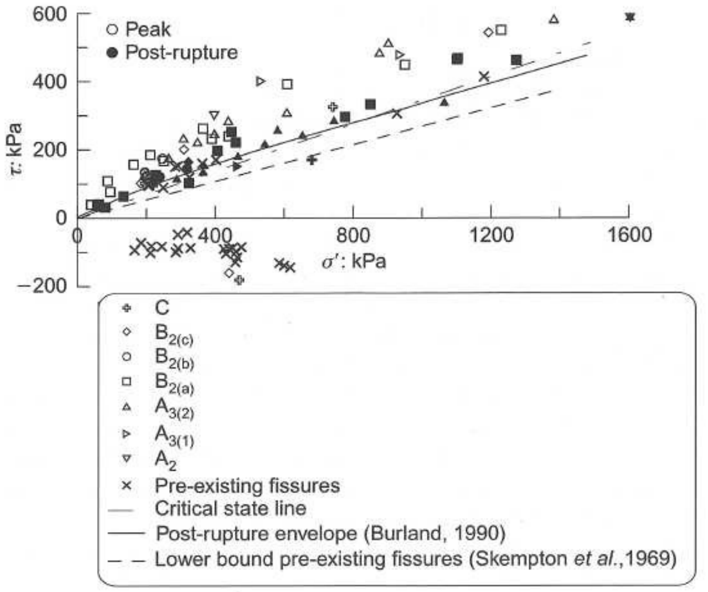

Fig. 11. Stresses mobilised both at peak and in the post-rupture state

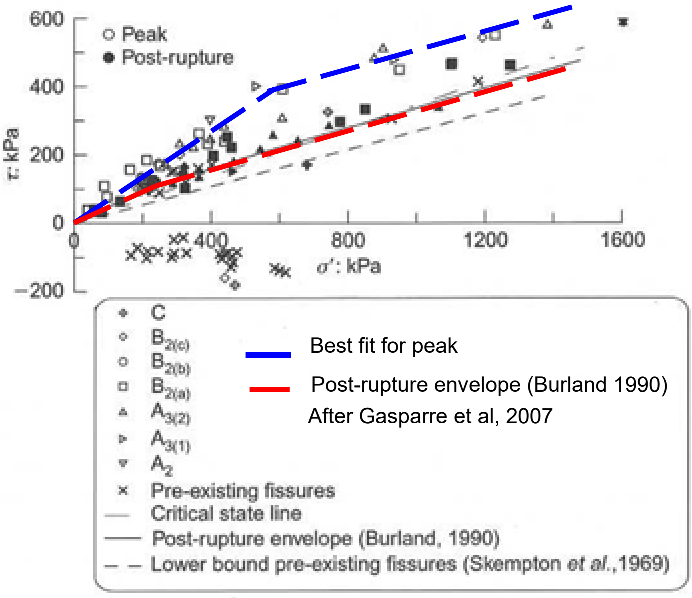

Fig. 11. Stresses mobilised both at peak and in the post-rupture state

7. O'Brien, 2007

Table 1. Index Properties for High Plasticity Dumped Clay Fill from a Range of Sites

Site	Moisture Content (%)		Liquid Limit (%)		Plasticity Index (%)	
	Mean	SD	Mean	SD	Mean	SD
C6	26.5	7.2	72.9	10.5	47.1	8.8
M10	29.5	5.3	67.7	8.3	42.7	8.0
C11	29.5	6.3	68.2	9.8	43.6	8.8
C9	32.2	7.4	71.4	8.7	49.4	7.8
C8	32.5	5.8	71	6.9	45.9	7.4
M14	34.3	6.9	69.4	6.7	43.4	6.5
C10	35.8	5.4	74.4	15.9	48.4	13.9

Note: SD = standard deviation

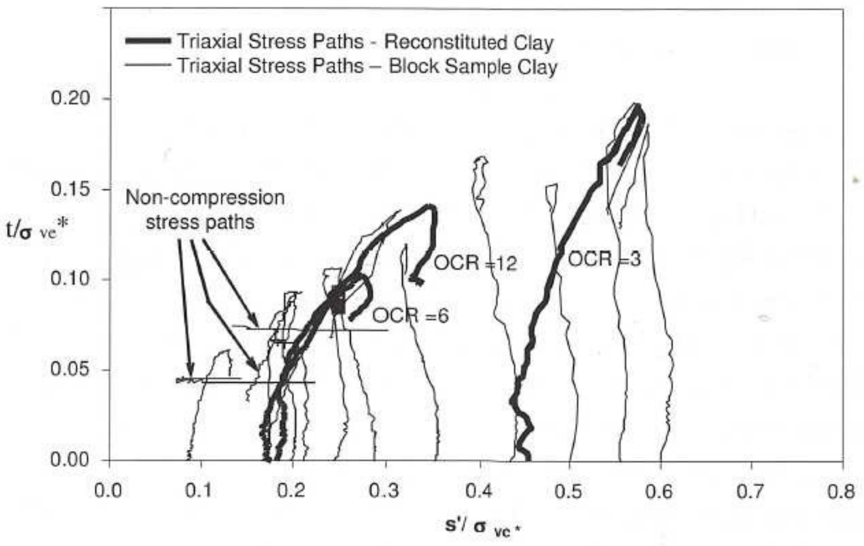

Figure 12. Normalised Shear Strength of High Plasticity Index Dumped Clay Fill. Behaviour of Undisturbed versus Reconstituted Samples