

Lower Thames Crossing Tunnels & Approaches Tilbury Landfill Stability Risk Assessment 2026

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Lower Thames Crossing

Tilbury Landfill Stability Risk Assessment 2026

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1 Introduction

1.1 Background

- 1.1.1 National Highways (NH) is seeking approval for a new Environmental Permit to continue landfilling and PFA recovery at the former Tilbury Ash Disposal site (also known as Tilbury Ashfields) formerly Environmental Permit EPR/GP3733DZ. National Highways wishes to create the required development levels for the Lower Thames Crossing scheme using waste derived from the adjacent Goshems Farm Deposit for Recovery site (formerly Environmental Permit EPR/WP3094EP).
- 1.1.2 The site was formerly permitted to operate as a non-hazardous landfill under the conditions of an Environmental Permit, with the most recent variation (EPR/GP3733DZ/V007) issued on 09 December 2025.
- 1.1.3 In accordance with the approach agreed with the Environment Agency, this document provides an updated Stability Risk Assessment for the new Tilbury Landfill permit, reflecting the intention to continue the activities authorised under the former permit.

1.2 Report context

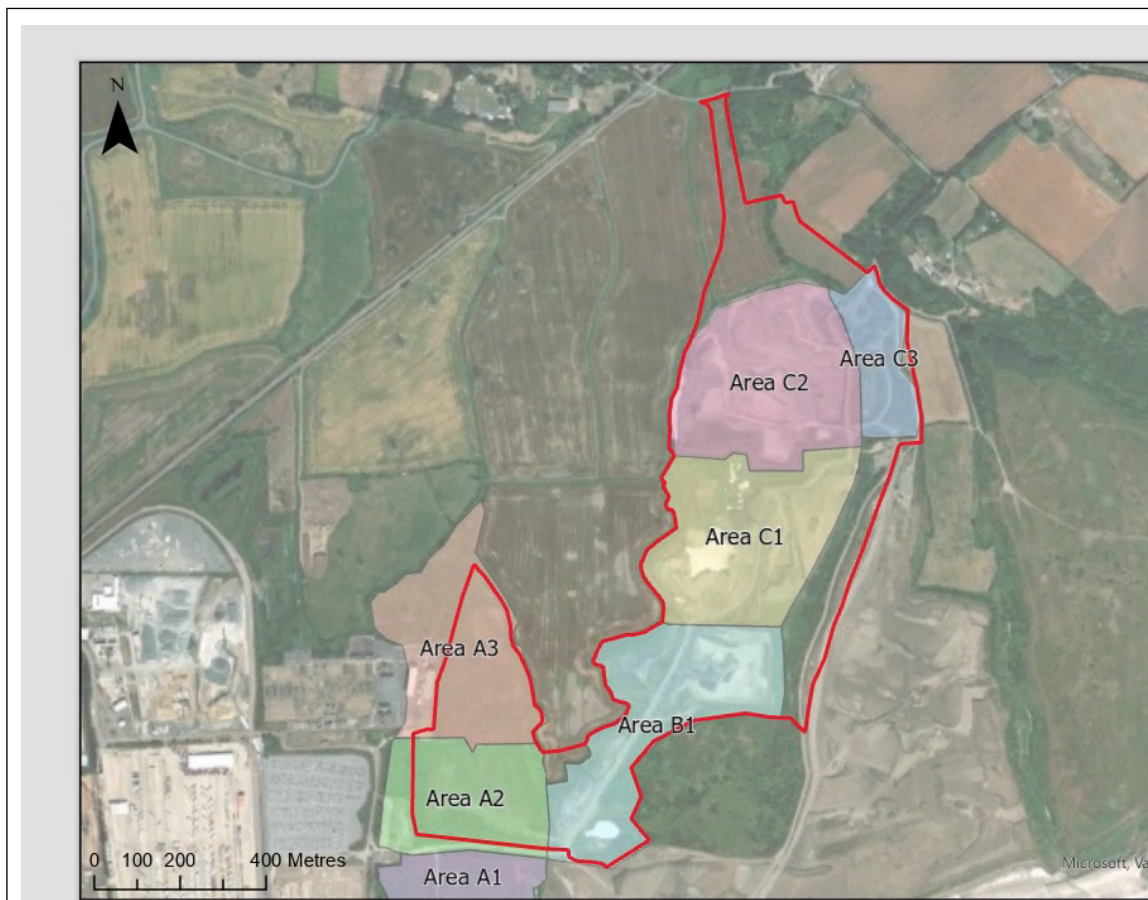
- 1.2.1 This document has been prepared by Arup Mott MacDonald Joint Venture (AMJV) on behalf of the Permit Holder, National Highways, to support an application for an environmental permit for Tilbury landfill (the site), which is located near Tilbury in Essex.
- 1.2.2 AMJV has been appointed by Bouygues-Murphy Joint Venture (BMJV) to prepare a Stability Risk Assessment in support of this new application.
- 1.2.3 This Stability Risk Assessment (SRA) is limited to a qualitative assessment of slope stability on the site as a result of the works.
- 1.2.4 The physical characteristics and geotechnical behaviour of the new proposed works align with that which has previously been allowed under the permit and therefore this Stability Risk Assessment builds upon the existing stability risk assessment completed by SLR in May 2021 on behalf of Ingrebourne Valley Limited (IVL) which can be found in the report Tilbury Ash Fields Inert Landfill Stability Risk Assessment, ref 416.01526.00076 [1].

1.3 Site Details

- 1.3.1 Tilbury Ash Disposal Site is a non-hazardous landfill east of Tilbury, Essex, centred on national grid reference TQ671763. The 80ha site was spilt into seven areas A1 to A3, B1 and C1 to C3.
- 1.3.2 This new permit application covers some of the area included in the previous permit boundary, but also includes additional areas to the north and east which form part of the LTC project. The previous environmental permit boundary (the site) and the proposed boundary for the new permit are shown on Figure 1-1.

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Figure 1-1 Former Tilbury Ash Disposal Site permit boundary and proposed new Tilbury Landfill permit boundary



The red line represents the proposed new permit boundary of Tilbury Landfill
The coloured areas represent the area formerly covered by Tilbury Ash Disposal Site permit

1.3.3 The site is bounded by agricultural land to the north and west, with Port of Tilbury land also bounding the site to the west. The land to the east comprises a mix of agricultural land and Goshems Farm Deposit for Recovery site. The land to the south comprises Goshems Farm Deposit for Recovery site and a wildlife area created from former landfill.

1.4 Permitting history

1.4.1 The disposal of Pulverised Fuel Ash (PFA) to Tilbury Ash Disposal Site commenced shortly after the construction of the power station in 1968 and continued until its closure in 2013. The site was historically covered by four separate Waste Management Licences (WML), as follows:

- License EA/WML/71185 for Area A3 (Jan 1978);
- License EA/WML/71186 for Area C2 (Nov 1991);
- License 193/91 for Areas A1, A2 and C1 (June 2001); and;
- License 38/78 for Area B (June 2001)

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1.4.2 Subsequent permitting history has been summarised in Table 1-1.

Table 1-1 Summary of Tilbury Ash Disposal Site permit history

Permit Number	Date	Description
RWE Npower plc		
GP3739BQ/A001	05/04/2007	Application determined for non-hazardous landfill accepting solely Pulverised Fuel Ash (PFA) produced by Tilbury B Power Station.
XP3236UR/V002	01/06/2009	Variation to GP3739BQ (PPC permit) for the extension of the landfill site to include an extension in Area C3.
GP3739BQ/V003	30/03/2010	Variation to increase final restoration levels from 6m AOD to 9m AOD and increase material accepted to allow an extra 1 million tonnes of PFA.
RWE Generation UK Plc		
GP3739BQ/V004	02/12/2014	Holder varied from RWE Npower plc to RWE Generation UK Plc.
Ingrebourne Valley Limited		
GP3733DZ/T001	19/05/2017	Transfer of the environmental permit from RWE Generation UK Plc to Ingrebourne Valley Limited. Permit reissued as EPR/GP3733DZ.
GP3733DZ/V002	13/10/2017	Expanded authorised waste types from PFA to include inert waste and updated the permit to modern conditions. Incorporated revised risk assessments and management plans.
GP3733DZ/V003	18/04/2019	Increased maximum annual throughput to 1,500,000 tonnes per annum, amended groundwater compliance limits, approved the restoration plan, and additional waste codes related to Thames Tideway Tunnelling project additives.
GP3733DZ/V004	27/01/2020	Updated the permit to modern conditions and revised waste acceptance criteria and associated procedures.
GP3733DZ/V005	13/08/2020	Added a new waste code 17 05 04, associated with exceedances identified during verification testing of Thames Tideway Tunnelling project waste stream, and updated waste acceptance procedures; included information on engineering and capping materials.
GP3733DZ/V006	09/08/2021	Added additional waste codes, 19 12 12 and 19 02 03 - treated chalk waste arising and introduced amendments to the site's improvement programme.
GP3733DZ/V007	09/12/2025	Update to the Monitoring Management Plan with the removal of groundwater compliance limits for BHC5D and BHC3D, with both reclassified to control levels, removed in-waste gas compliance limits.

1.5 Previous Stability Risk Assessment

- 1.5.1 A SRA was completed in May 2021 by SLR [1] on behalf of IVL as part of the permit variation to add additional waste codes.
- 1.5.2 Works covered by previous permits and variations allowed the placement of PFA and other inert waste materials on the site up to an allowable height of 9mAOD. The variation to allow the height to be increased to 9mAOD was granted in 2010. Between 2010 and 2021, some PFA material was excavated for recovery from the site whilst other areas underwent restoration. The landfill was also subject to shallow land raising but with other inert waste material.
- 1.5.3 The purpose of the 2021 SRA was to allow all areas to be raised to 9mAOD using inert waste comprising predominantly chalk arisings from the Thames Tideway Tunnelling project along with clay derived from tunnelling and infrastructure projects in the London area.
- 1.5.4 Ongoing groundwater monitoring in the area at the time showed that the deep groundwater level varied between +0.4m and +0.7mAOD and the shallow groundwater (screened in the alluvium) was recorded between +0.5m and +6.0mAOD.

Conceptual site model

Basal subgrade model

- 1.5.5 The basal sub-grade model was noted to be formed of in-situ Alluvium up to 20m thick and comprising clays and occasional sand lenses and peat layers.
- 1.5.6 Proposed works covered by the SRA required the stripping of topsoil to form the Alluvium subgrade in area C3 and the removal of restoration topsoil in areas B, C1 and C2 (shown on Figure 1-1) to expose PFA prior to further landfilling. The latter is considered as part of the waste mass. Subsequent to stripping of topsoil, the basal material was graded to achieve a consistent base.

Side slope subgrade model

- 1.5.7 The works required limited excavation for preparation of the site for filling from removal of the topsoil or grading of existing PFA. However, the site is a land raise and therefore no additional side slopes are formed as a result of the development.

Basal geological barrier model

- 1.5.8 The in-situ alluvium has been considered to form a natural hydro-geological barrier with a low permeability between 10^{-9} and 10^{-12} m/s. Alluvium comprises the natural superficial deposits underlying the site which is covered by variable thicknesses of PFA. Alluvium forms the base of the existing land raise with a thickness of at least 15m under the site.
- 1.5.9 Although considered part of the waste mass, the existing PFA was considered to also act as an in-situ geological barrier on multiple phases of the site. Compacted PFA generally has a low permeability approximately between 10^{-7} and 10^{-8} m/s. This is lower or equal to the minimum required by the Environment Agency for inert landfill engineering purposes. The hydraulic properties of the

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in-situ alluvium and PFA were therefore considered to be adequate to form a basal geological barrier.

Side slope geological barrier model

- 1.5.10 Due to the method of waste placement on site, as a land raise, no side slopes of in-situ material were to be formed. As such, no side slope geological barrier was required.

Waste mass model

- 1.5.11 Tilbury Ash Disposal Site historically accepted PFA from the adjacent power station which was placed and compacted in 200mm thick layers. Consequently, at the time of preparing the SRA, areas had variable heights of PFA up to 6mAOD. Previous assessments had found that the existing perimeter of the ash mound was stable at waste slope gradients of 1v:3h constructed to an elevation of 6mAOD which was considered a stable long term slope angle.
- 1.5.12 Waste slope heights were expected to reach 4m with surrounding topographic levels typically 2mAOD. In the worst case, waste slope heights may occasionally extend to 4.5 to 5m with topographic levels of 1.0m to 1.5mAOD in the northeast of the site.
- 1.5.13 The proposed waste covered by the SRA was expected to comprise suitable selected inert imported soils that included chalk arisings from the Thames Tideway Tunnelling project. Waste materials would meet strict Waste Acceptance Criteria and would raise each area to 6mAOD prior to capping. The SRA stated that a portion of Area A3 was dedicated for the disposal of 34,000m³ of asbestos contaminated soil material, although this material is not thought to have been deposited as asbestos waste was not a permitted waste type in later permit variations.

Capping system model

- 1.5.14 As part of the proposed works in the 2021 variation, the site was to be restored with a 1m thick engineered clay seal and 1m of restoration soils. The capping system covers the side slopes of the waste mass due to the nature of the development as a raised landfill. Restoration drawings indicate a maximum waste and restoration material slope height of 7m when considering the surrounding topographic levels of 2mAOD.

Stability risk assessment

- 1.5.15 The Stability Risk Assessment undertaken in 2021 [1] completed a screening of the six principal components of the works against the criteria outlined in the EA guidance document *Stability of Landfill Lining Systems: Report No. 2 Guidance (Ref P1-385/TR2)*[3] to determine whether or not additional assessment was necessary. A summary of the findings of the assessment is presented in Table 1-2.

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Table 1-2 Summary of screening of the principal components for the 2021 assessment

Component	Description	Outcome of screening
Basal subgrade	<p>In line with the requirements of P1-385/TR2, this was assessed against the following criteria:</p> <ul style="list-style-type: none"> • Compressible subgrade – alluvium very compressible however not considered to pose a threat to the landfill stability or performance • Basal heave – no excavation and therefore no heave expected • Cavities in subgrade – no record of cavities in the subgrade • Compressible waste – waste material has been compacted on placement and therefore was considered incompressible • Cavities in waste – as above, compaction of material eliminates risk of voids 	No further consideration
Side slope subgrade	No excavation is required for the works so no further consideration of this was deemed necessary	No further consideration
Basal geological barrier system	<p>In line with the requirements of P1-385/TR2, this was assessed against the following criteria for mineral only system:</p> <ul style="list-style-type: none"> • Stability and integrity – low permeability of the alluvium and the existing waste mass resulting in a natural barrier. Potential movements along the interface with the waste mass however should be considered. • Compressible subgrade – alluvium is highly compressible however is not considered to pose a threat to the landfill stability or performance • Cavities – no record of cavities in the subgrade • Basal heave - no excavation and therefore no heave expected 	Considered only in the context of the stability of the waste mass
Side slope geological barrier system	No excavation is required for the works so no further consideration of this was deemed necessary	No further consideration
Waste mass	<p>In line with the requirements of P1-385/TR2, this was assessed against the following criteria:</p> <ul style="list-style-type: none"> • Failure wholly in waste – the existing waste is stable however further assessment of the chalk 	Further assessment required

Component	Description	Outcome of screening
	<p>arising is required to define the allowable gradient of the slopes.</p> <ul style="list-style-type: none"> Failure involving geological barrier and waste – potential for slip surfaces to extend into the alluvium and therefore further assessment required 	
Capping system	<p>In line with the requirements of P1-385/TR2, this was assessed against the following criteria for a mineral only system:</p> <ul style="list-style-type: none"> Stability – increase in height of the waste slopes requires assessment for stability and to determine allowable gradient Integrity – deformations are not considered likely due to compressibility and cavities 	Further assessment for stability required

1.5.16 Due to the nature of the inert waste deposited at Tilbury Ash Disposal site, a significant volume of leachate would not be generated and consequently a specific leachate collection system was not deemed necessary.

Lifecycle phases

1.5.17 The SRA addressed the key issues throughout the life of the landfill, the in-situ basal geological barrier (as part of the waste assessment), waste slopes and restoration capping system slope stability.

Data summary

1.5.18 The assessment undertaken for the SRA used the material unit weight and the drained shear strength of the soils and waste. In the absence of laboratory test data for the site for material shear strengths, the assessment relied upon published data and a previous stability assessment from the site to provide the data. Geotechnical laboratory testing was completed on the proposed chalk waste material.

Selection of appropriate factors of safety

1.5.19 A factor of safety of 1.3 was considered appropriate for the stability assessments.

Justification for modelling approach and software

1.5.20 A limit equilibrium stability assessment using the Slope/W 2007 software was adopted. The Bishop slip-circle and Morgenstern-Price non-circular methods of analysis were used.

Justification of geotechnical parameters selected for analysis

1.5.21 The geotechnical parameters adopted for the assessments undertaken are presented in Table 1-3.

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Table 1-3 Parameters adopted for assessment in 2021 Stability Risk Assessment

Material	Unit weight, g (kN/m ³)	Effective cohesion, c' (kPa)	Angle of shearing resistance, ϕ' (°)	Undrained shear strength (kPa)	Typical description
Restoration soils	16	0	30	-	Imported topsoil/subsoil
Engineered clay seal	18	0.5	28	-	Inert imported clay
Waste	16	0	29	14	Inert chalk arisings, derived from the Thames Tideway Tunnelling project
In-situ alluvium	16	0	23 (residual 20)	15+2kPa/m max 45kPa	Normally consolidated superficial alluvium basal subgrade

Analyses

Waste mass analyses

- 1.5.22 Stability assessments for the waste mass were undertaken for both Mode 1 (failure in the waste mass only) and Mode 2 (failure plane through the waste mass and underlying subgrade). Stability of the existing PFA waste mounds was not considered further as the mounds were known to be stable from previous experience on the site.
- 1.5.23 For Mode 1, a 4m high waste slope comprising chalk arisings with a gradient of 1v:3h was assessed for both drained and undrained conditions with and without pore water pressure in the waste mass. A summary of the resulting factors of safety are presented in Table 1-4.

Table 1-4 Summary of factors of safety for Mode 1 assessment

Assessment description	Method	Factor of safety
Failure within the waste mass, without pore pressure	Undrained, circular slip	1.675
Failure within the waste mass, without pore pressure	Drained, circular slip	1.586
Failure within the waste mass, with pore pressure	Drained, circular slip	1.442

- 1.5.24 For Mode 2, short term conditions were modelled in the subgrade with peak material parameters. The long term conditions considered post peak or residual shear strength conditions. For these cases, a factor of safety of 1.0 was considered acceptable.
- 1.5.25 As for Mode 1, the Mode 2 assessment assessed a 4m high slope with a 1v:3h gradient. Pore pressure in the waste mass was also considered for some assessments. A summary of the resulting factors of safety are presented in Table 1-5.

Table 1-5 Summary of factors of safety for Mode 2 assessment

Assessment description	Method	Subgrade ϕ' (°)	Subgrade c' (kPa)	Factor of safety
Failure involving the subgrade, without pore pressure	Undrained, non-circular	23	0	1.772
Failure involving the subgrade, without pore pressure	Drained, non-circular	23	0	1.579
Failure involving the subgrade, with pore pressure	Drained, non-circular	20	0	1.451
Failure involving the subgrade, without pore pressure	Undrained, non-circular	20	0	1.724
Failure involving the subgrade, without pore pressure	Drained, non-circular	20	0	1.499
Failure involving the subgrade, with pore pressure	Drained, non-circular	20	0	1.367

- 1.5.26 For both Mode 1 and Mode 2 assessments, factors of safety of greater than 1.3 were achieved.

Capping analyses

- 1.5.27 The capping analyses assumed a 1m thick engineered clay seal and 2m of restoration soils on Tilbury Ash Disposal Site. The restoration profile covered the top and side slopes of the land raise up to 9mAOD. For the capping analysis, a single cross section was considered as being representative of the typical worst case for the slope geometry: a 7m high slope with a gradient of 1v:3h. A summary of the results of the analyses is presented in Table 1-6.

Table 1-6 Summary of factors of safety for the capping layer analyses

Assessment description	Method	Factor of safety
Failure through the toe of the waste mass	Drained, non-circular	1.431

Assessment description	Method	Factor of safety
Failure through impermeable clay seal	Drained, non-circular	1.607
Failure along the interface between the waste mass and the seal. Restoration profile 50% saturated	Drained, non-circular	1.517

1.5.28 In all cases a factor of safety of greater than 1.3 was achieved.

Assessment

1.5.29 For all scenarios assessed, a minimum factor of safety of greater than 1.3 was achieved. This suggests that a waste mass slope up to 4m in height with a gradient of 1v:3h is stable.

Monitoring

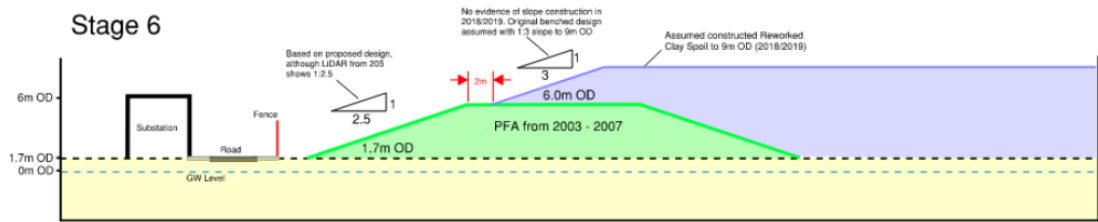
1.5.30 The monitoring requirements for the site were limited to the control of waste and restoration soils levels such that the final restoration contours are achieved. The site inspections and operational practices, adhered to as part of the Quarries Regulation 1999 regime, including the excavation and tipping rules, were deemed to be sufficient for the purposes of maintaining a safe working environment within the operation.

1.6 Slope instability

- 1.6.1 A slope failure was first detected in September 2019 along the boundary between the Tilbury Ash Disposal Site and the adjacent National Grid Tilbury Substation. To date, approximately 1.5m of heave and 3m of lateral movement westward (towards the substation) has been observed and this has caused damage to the substation's security fence, access road and associated infrastructure.
- 1.6.2 In 2024, COWI UK Ltd were commissioned by National Grid Electricity Transmission Plc (NGET) to investigate the cause of the failure. Their report *Root Cause of Ground Movement Expert Report* dated December 2025 (A123989-COWI-XX-XX-RP-GE-000002) [4] has been reviewed to understand the conditions surrounding the failure.
- 1.6.3 At the location of the failure, the existing PFA material had been extracted from the stockpile to sell for construction purposes until 2018. The majority of the stockpile was excavated to approximately 1.85mOD with a boundary bund left in situ. LiDAR data indicates that the top of the bund remained at 6mOD but its width reduced from 51m to 37m. The slope gradient is indicated to be at 1V:2.5H rather than the design gradient of 1V:3H.
- 1.6.4 Between April and July 2019, the ground level was raised from 6mOD to 9mOD. No LiDAR data is available for this ground level raise but the design proposal was for a 2m bench at the top of the bund and a 1V:3H slope from 6mOd to 9mOD. A section through the geometry of the slope is shown in Figure 1-2 taken from the COWI report.

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Figure 1-2 Cross section through the slope geometry (taken from the COWI 2025 report [4])



1.6.5 A ground investigation at the site of the failure was undertaken during 2025 and was interpreted by COWI in their report. This indicated that the generalised stratigraphy presented in Figure 1-3.

Figure 1-3 Generalised stratigraphy from the 2025 GI (from the COWI report [4])

Table 8-1: Generalised stratigraphy of the natural ground based on LiDAR and Leapfrog modelling development using available 2025 GI information.

Stratum	Thickness (m)	Top Level (m OD)
Natural Ground Conditions		
Alluvial Crust	1.0	1.7
Upper Alluvium	7.7	0.7
Peat	0.5	-6
Lower Alluvium	7.8	-6.5
River Terrace Deposits	3	-14.3
LSN Chalk	Unproven (Anticipated to be >30m)	-17.2

1.6.6 The parameters for design interpreted by COWI are presented in Figure 1-4. These broadly align with those adopted in the 2021 assessment (Table 1-3).

Figure 1-4 Calculated parameters from COWI GI

Table 8-2: Summary table of the derived geotechnical parameters from COWI's Ground Investigation Report [9].

Parameter	PFA	Reworked Clay	Alluvium	Peat
Type	Cohesive	Cohesive	Cohesive	Cohesive
Bulk Density (Mg/m ³)	0.98*	1.97	1.60	1.04
Moisture Content (%)	55	27	64	269
SPT N value	7	6	3	-
Cu (kPa)**	15	30	15	10
Φ' (°)	29	27	28	
Φ'cv (°)	-	22	22	-

1.6.7 Slope stability assessments undertaken by COWI investigated a number of different failure mechanisms:

- Shear failure of soil in bearing
- Heave due to changes in pore water pressures (leading to zero effective stress conditions)
- Deep seated rotational failure
- Translational failure through weak strata (peat layer)
- Combined slope/bearing failure

1.6.8 Several sensitivity analyses were undertaken within these assessments to understand the influence of various factors on the stability:

- Rate of load placement
- Groundwater levels
- Variation in undrained shear strength of the alluvium
- Reworked clay fill unit weight
- Strength of peat layer
- Surcharge distribution and geometry i.e. variation in slope gradient for the existing PFA (to 1V:3H) and reduced height of fill placed

1.6.9 The analyses undertaken by COWI demonstrated the following:

- Short term undrained conditions during fill placement are critical in controlling slope stability
- Weak horizons exert a significant influence on failure mechanisms

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- Reductions in applied load improve stability
- Geometric modifications alone provide limited benefit

1.6.10 COWI further concluded that the cause of the failure was the rapid placement of reworked clay fill over soft alluvial deposits. This rapid loading caused undrained conditions in the alluvium resulting in the generation of excess pore water pressures which could not dissipate quickly due to the material's low permeability. This resulted in loss of effective stress, significantly reducing the shear strength of the soils and triggering slope instability.

1.6.11 COWI also state *“The presence of a weak peat horizon significantly exacerbated the failure, acting as a critical slip plane and triggering a compound translational-rotational failure consistent with the observed displacements and patterns and asset damage”*.

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2 Conceptual stability site model

2.1 Proposed works covered by this assessment

- 2.1.1 Proposed activities associated with the LTC project at the site include:
- Disposal of inert waste for site earthworks and leveling.
 - Removal and storage of any encountered PFA in accordance with the relevant Quality Protocol.
 - Replacement of excavated PFA with inert material.
- 2.1.2 The new permit application proposes a continuation of current site activities, namely:
- the excavation and recovery of PFA, which will be stockpiled in a designated area within the site boundary for reuse in the creation of Open Mosaic Habitat at a later date.
 - the import of inert material for placement in the landfill to support LTC construction.
- 2.1.3 Inert material may be moved from one part of the permit area to another during earthworks activity, or transferred from the adjacent Goshems Farm site, which operated under an Environmental Permit for Deposit for Recovery and was permitted to accept inert wastes for recovery, including chalk arisings from tunnelling and clay from infrastructure projects in the London area. The material placement at the site is therefore considered consistent with the waste types previously authorised for the Tilbury Ash Disposal Site.
- 2.1.4 The proposed works covered by this SRA are deemed to fall within the nature of works previously covered by the existing permit. Site levelling and placement of material will be from elsewhere on the site and will be placed in slopes no steeper than 1v:3h.

2.2 General

- 2.2.1 This conceptual stability site model has been developed from information contained in the 2021 SRA [1] and information gained through ongoing site investigations undertaken for the LTC project. No substantial changes to the conceptual site model from the 2021 model have been made, beyond the nature of the works which are planned to be undertaken on the site.

2.3 Basal Subgrade model

- 2.3.1 The basal subgrade is formed from in situ alluvium up to 20m thick. This material comprises clay becoming silty clay with depth. Layers of peat are present throughout the stratum.

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2.4 Side slope subgrade model

- 2.4.1 There will be limited excavation in the existing landfill material to support the temporary and permanent earthworks. As material will be placed on top of the existing landfill material and no excavation in the subgrade is proposed, there will be no side slopes to consider in the subgrade.

2.5 Basal engineered system

- 2.5.1 No basal engineered system exists nor is proposed. The material will be placed directly on the existing ground surface or existing waste surface, taking benefit from the natural geological barrier provided by the underlying low permeability alluvium.

2.6 Side slope engineered system

- 2.6.1 No side slope engineered system exists nor is proposed. The material will be placed as a land raise directly on the existing ground surface and hence no excavation requiring a liner is proposed.

2.7 Waste mass

- 2.7.1 Tilbury Ash Disposal Site historically accepted PFA from the adjacent power station which was placed and compacted in 200mm thick layers. Consequently, areas have historically had variable heights of PFA up to 6mAOD. Previous assessments found that the existing perimeter of the ash mound was stable at slope gradients of 1v:3h. Slopes extending to 6mAOD resulted in slope heights up to 5m depending on the existing ground level.
- 2.7.2 Prior to any disposal being undertaken, topsoil was stripped and stockpiled away from any tipping operations.
- 2.7.3 Planning permission (2013) from Thurrock Council (12/00497/FUL) for recovery of beneficial use of pulverised fuel ash deposited on Tilbury Power Station ash disposal areas A2, A3 and B was granted together with an amendment to the existing permit to allow recovery of the PFA.
- 2.7.4 Excavation of the PFA has been undertaken by the current permit holder, IVL since 2017. PFA has been removed in Areas A2 and A3 and filling has taken place with selected inert imported soil including chalk arising from the Thames Tideway Tunnelling project. These materials met strict Waste Acceptance Criteria and raised each area to 6mAOD prior to capping.
- 2.7.5 In Areas B and C however, PFA has been partially removed and existing material has been moved around the site, but excavations are not backfilled. Existing ground levels in Areas B and C are therefore variable with elevations between 2mAOD and 6mAOD present.
- 2.7.6 With the exception of Area A1 which had already been completed in 2021, it was proposed that the completion works would include an engineered 1m cap comprising a low permeability mineral layer. Once the final contours were reached (maximum elevation of 9mAOD) the site would be restored for agricultural use using the topsoil previously stripped from the landfilled areas.

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- 2.7.7 Future works will comprise the movement of existing material present on the site around to accommodate the LTC development. This will include
- Disposal of inert waste for site earthworks and levelling
 - Removal and storage of any encountered PFA in accordance with the relevant Quality Protocol
 - Replacement of any excavated PFA with inert material

2.8 Pore fluid pressures

- 2.8.1 The existing (PFA and inert) and proposed inert waste types do not generate significant quantities of leachate, therefore, no leachate management systems have been used historically within the landfill. Consequently, the generation and storage of leachate from the Tilbury Ash Disposal Site is not considered to be a significant issue and water balance calculations have not been undertaken for the purposes of this report.
- 2.8.2 The proposed activity aligns with activities under previous waste licences. The movement and placement of inert waste undertaken as part of the LTC project is not considered to materially change the likelihood of leachate generation at the site. On this basis, the provision of leachate management systems is not considered necessary.
- 2.8.3 The Alluvium which underlies the site is highly compressible with low permeability. As a result, excess pore water pressures in this material are generated by the placement of fill material on the existing ground surface. Monitoring of pore water pressures across the site has been undertaken since 2018. This has indicated that pore water pressure equivalent to a groundwater level at 6m to 8mAOD is present where material has been placed on the Alluvium. This aligns with the groundwater pressures which were assessed by SLR in their Stability Risk Assessment in 2021. This is not considered to be of concern from a slope stability perspective.

2.9 Settlement and strains

- 2.9.1 Settlement of the alluvial subgrade is anticipated with up to 1m of settlement expected over a period of 30 to 100 years. This is not considered to be a concern for the stability of the proposed earthworks.

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3 Revised Stability Risk Assessment

3.1.1 Each of the five principal components of the conceptual site model has been considered and the various elements of that component have been assessed with regard to stability.

3.1.2 The principal components considered are:

- The basal subgrade
- The side slope subgrade
- The basal geological barrier
- The side slope geological barrier
- The waste mass

3.2 Risk screening

3.2.1 In line with the guidance in P1-385/TR2, issues relating to stability and integrity for each principal component of the proposed development have been subject to preliminary review to determine if they are applicable to the assessment. The outcome of the screening is summarised in Table 3-1.

Basal subgrade

Excessive Deformations

Compressible sub-grade

3.2.2 The basal subgrade is formed by the existing Alluvium on the site. Based on ground investigation undertaken as part of the LTC works, this stratum is understood to be up to 17m in thickness and comprises predominantly clay material over the top 10m becoming siltier with depth. Several layers of peat have also been identified in this material. The Alluvium is highly compressible with low permeability. Settlements of up to 1m are anticipated as a result of the proposed works, taking 30 years to complete primary consolidation with creep movements expected to continue for up to 70 further years.

3.2.3 This component is not considered to pose a threat to the stability of the proposed earthworks; however, the extent of settlement will be taken into account in assessment of the performance of the stability of the waste mass.

Cavities

3.2.4 No cavities have been reported in the Alluvium, either from site specific data or from available literature and as such this aspect will not be considered further in this Stability Risk Assessment.

Basal heave

3.2.5 The proposed works comprise the placement of material on the subgrade and no excavation is proposed. As such heave of the underlying Alluvium is not anticipated and this aspect will not be considered further in this Stability Risk Assessment.

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Filling on Waste

Compressible Waste

- 3.2.6 The site has been raised with variable levels of PFA and imported inert material. While this material has been placed and compacted in relatively thin layers, this material may consolidate further upon loading. While this is not considered to pose a threat to the stability of the proposed works, this will be considered in the assessment of their performance.

Cavities in waste

- 3.2.7 The nature of the material which has been placed and compacted as waste material on the site is not considered to be prone to the presence of voids and as such this aspect is not considered further in this Stability Risk Assessment.

Side slope subgrade

- 3.2.8 No excavation in the subgrade is proposed and therefore this is not considered further in this Stability Risk Assessment

Basal geological barrier

- 3.2.9 In line with the previous assessment, the naturally occurring, low permeability Alluvium will form the basal geological barrier and as such this aspect is not considered further in this Stability Risk Assessment.

Sides slope geological barrier

- 3.2.10 No excavation in the subgrade is proposed and therefore this is not considered further in this Stability Risk Assessment

Waste mass

Failure wholly in waste

Stability

- 3.2.11 The proposed works comprise the movement of waste material from around the site including the recovery of any PFA encountered and the import of inert waste from the adjacent Goshems Farm Deposit for Recovery site to form a level working platform. This aligns with the works which are already underway which are covered by the previous SRA undertaken in 2021. Further consideration of the applicability of the previous assessment will be undertaken.

Failure involving geological barrier waste

Stability

- 3.2.12 As for the previously assessed waste mass, there is a risk of a slip surface forming through the waste mass and the underlying subgrade. Further consideration of the applicability of the previous assessment to the works will be undertaken.

Integrity

- 3.2.13 No basal engineered barrier is proposed due to the low permeability of the alluvium and this is therefore not considered further here.

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Summary

3.2.14 Table 3-1 provides a summary of the outcome of the risk screening assessment.

Table 3-1 Summary of outcome of the risk screening

Component		Outcome of screening
Basal subgrade	Compressible subgrade	To be considered only in the context of the performance of the waste mass
	Cavities	No further consideration
	Basal heave	No further consideration
	Compressible waste	To be considered only in the context of the performance of the waste mass
	Cavities in waste	No further consideration
Side slope subgrade		No further consideration
Basal geological barrier system		No further consideration
Side slope geological barrier system		No further consideration
Waste mass	Failure wholly in waste	Further assessment required
	Failure involving geological barrier	Further assessment required

3.3 Lifecycle phases

3.3.1 The area will be developed in one phase with slopes in line with those which have previously been agreed under the existing permit. Maximum slope gradients of 1v:3h will be maintained with maximum slope heights expected to be around 8mAOD.

3.4 Data Summary

Published geology

- 3.4.1 The geological setting of the site has been reviewed based on the 1:50,000 scale and 1:10,000 scale geological mapping available from the British Geological Survey (BGS).
- 3.4.2 A summary of the stratigraphy at the site is provided in Table 3-2. The 1:50,000 geological map indicates the presence of Thanet Formation towards the north of the site. However, it is likely that this will not be encountered at the site location and therefore details of the Thanet Formation have not been included in Table 3-2.

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Table 3-2 Summary of expected stratigraphy at the site

System / Period	Group	Formation	Description	Anticipated Thickness (m)
Quaternary	-	Artificial Ground	<p>Areas where the ground surface has been modified by anthropogenic processes of material removal, deposition or redistribution. The site is likely to include artificial ground:</p> <ul style="list-style-type: none"> Made ground: where artificial material has been deposited on the ground surface. This type of artificial ground is present across the site due to historical and current day landfilling activities. <p>Artificial ground comprising made ground across the site contain both inert and contaminated material due to the presence of waste and landfill material. The thickness of the stratum varies depending on the area of the site. PFA is known to be present in areas of the site.</p>	Up to 9m
		Alluvium	Soft organic-rich silt and clay which may have a firm surface crust 1 to 2m thick. Peat is present within this stratum and may have thicknesses up to 2m (most commonly present in the top 8m).	Up to 17m
	Thames Catchments Subgroup	Maidenhead Formation (Shepperton Gravel, Kempton Park Gravel, Lynch Hill Gravel and Taplow Gravel Members)	<p>River terrace deposits and terrace gravels comprising sand and gravel with local deposits of sandy silt.</p> <p>Interglacial deposits consisting of fine-grained material (i.e. silt and clay) are occasionally encountered.</p>	Up to 5m
Cretaceous	White Chalk Subgroup	Seaford Chalk Formation	A white chalk with conspicuous and semi-continuous nodular and tabular flint seams (Hopson 2005, Mortimore 2014). The Seaford Chalk Formation comprises three beds, the Haven Brow, the Cuckmere and the Belle Tout Beds. Thin marls are known to be present in the Belle Tout Beds. Key marker bands include the Bedwell's Columnar Flint, the Michel Dean Flint and the Seven Sisters Flint.	Up to 20m

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System / Period	Group	Formation	Description	Anticipated Thickness (m)
		Lewes Nodular Chalk Formation	Hard to very hard nodular chalks and hardgrounds with interbedded soft to medium hard chalks and marls (Hopson 2005). Key marker beds include the Shoreham Marls (1 and 2) and the Shoreham Tubular Flint.	>27.5m
<p>Notes:</p> <ol style="list-style-type: none"> 1. The Thanet Formation is present towards the north of the site however is not anticipated to be encountered within the site location. 2. The anticipated thicknesses are generally based on the ground model produced for the North Portal caterpillar structure, the thicknesses are likely to be variable across the site. 3. The 1:50,000 Geological map (Dartford, Sheet 271) indicates the Lewes Nodular Chalk Formation typically ranges from 20m to 40m, however an approximate thickness from a borehole (OH07023) in the site boundary has been included. 				

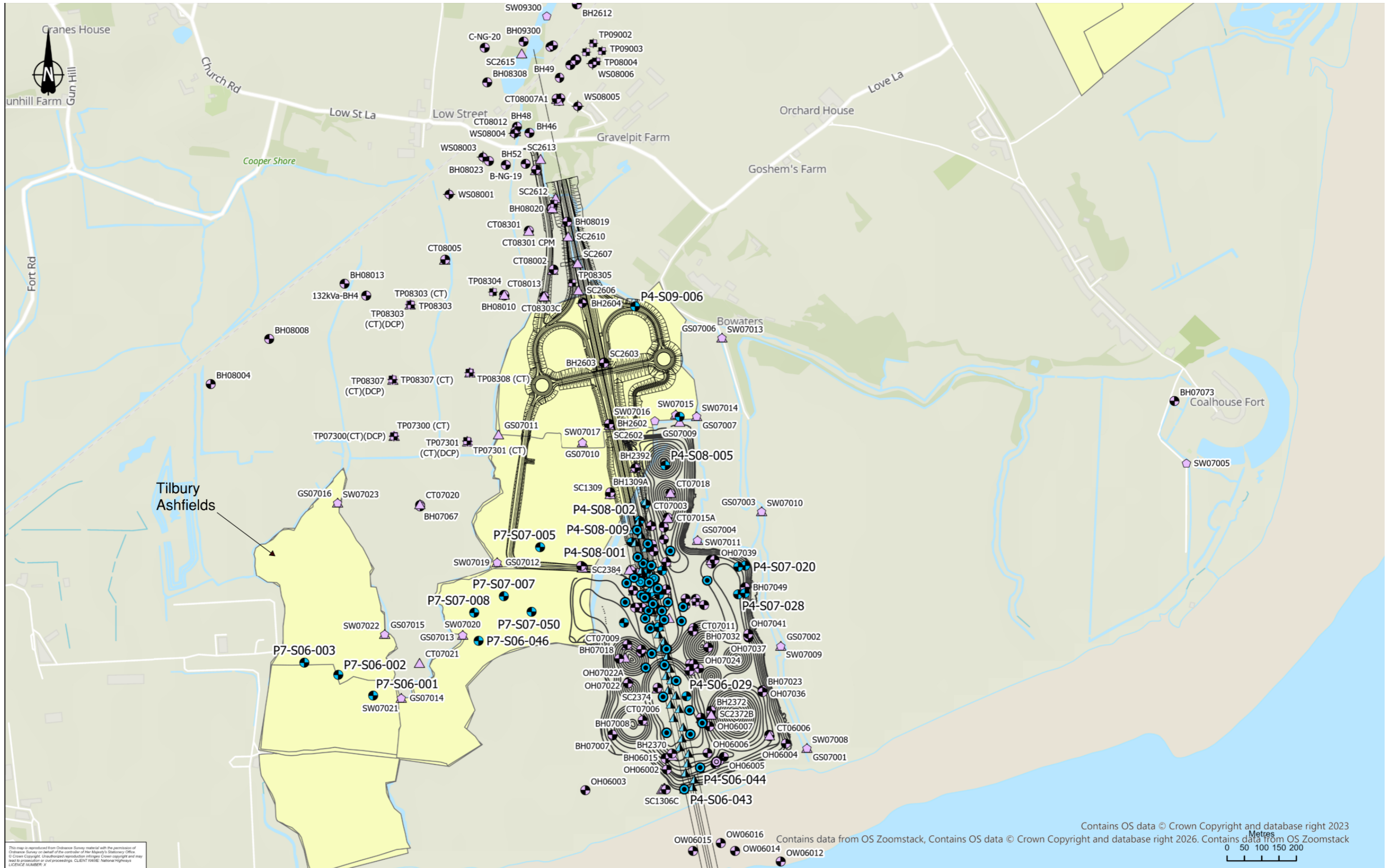
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Site specific data

- 3.4.3 Four phases of ground investigation (GI) have been undertaken across the area for the LTC project. This has included CPTs, boreholes, groundwater and ground gas monitoring, geotechnical and geo-environmental laboratory testing and in-situ testing. A plan showing the extent of exploratory holes in the area is presented in Figure 3-1.

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Figure 3-1 Plan of exploratory holes present on the site and in the surrounding area



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3.4.4 Based on the site specific investigation, the design stratigraphy presented in Table 3-3 is considered valid for the assessment. This aligns with the materials considered by SLR in their assessment.

Table 3-3 Summary of design stratigraphy for the site

Stratum		Thickness (m)	Elevation at top of stratum (mAOD)
Made ground	Deposit for Recovery Material	5.6	8.8
	Historic Fill	2.8	3.2
Alluvium ¹	Silty Clay / Clayey Silt	16.2	0.4
River terrace deposits ²		4.8	-15.8
Notes:			
1 4 distinct peat bands determined through analysis of the geology are present at -0.4mAOD, -26mAOD, -4.7mAOD and -11,8mAOD. The bands have thicknesses between 0.4m and 0.7m.			
2 Chalk is present below the river terrace deposits but is ignored for the purposes of this assessment.			

3.4.5 An interpretation of all the laboratory and field data has been undertaken to determine the parameter ranges for assessment of the proposed earthworks. For the purposes of the assessment, only the stratigraphy to the River Terrace Deposits has been considered. The interpreted parameters are presented in Table 3-4.

Table 3-4 Summary of ranges of parameters for the materials encountered during the site specific GI

Parameter	Unit	Made ground	Alluvium		River terrace deposits
			Clay	Peat	
Bulk Density	kN/m ³	16	10.6 to 19.6 ^b	10.1 to 19.1 ^b	16 to 20 ^c
Plasticity Index	%	8 to 63	10 to 119	12 to 209	-
Undrained shear strength	kPa	5 to 138 ^d	9 to 83 ^e	20 to 83 ^e	-
Unconfined Compressive Strength	MPa	-	-	-	-
Drained (effective stress) shear strength parameter (c')	kPa	0 ^f	0 ^f	0 ^f	0 ^f
Drained (effective stress) shear strength parameter (φ')	°	20 to 42	16 to 35	13 to 29	28 to 40
Young's Modulus	MPa	Eu = 300 to 750cu ⁱ E' = 0.87E _u ^j	Eu = 200 to 600cu ⁱ E' = 0.87E _u ^j		

Parameter	Unit	Made ground	Alluvium		River terrace
Poisson's ratio	-	0.3 ^k	0.3 ^k	0.3 ^k	0.2 ^k
Hydraulic Conductivity	m/s	Cohesive 1x10 ⁻⁹ to 1x10 ⁻⁷ Granular 1x10 ⁻⁷ to 1x10 ⁻⁵	Horizontal 1x10 ⁻⁹ to 1x10 ⁻⁷ Vertical 5x10 ⁻¹¹ to 5x10 ⁻⁹		5x10 ⁻⁶ to 5x10 ⁻³
<p>^a The values reported from the existing GI data have a wider range however based on the material descriptions and the presence of PFA, the values reported here are considered appropriate</p> <p>^b The ranges reported here have been derived from the test data however a value of 16kN/m³ for the clay alluvium and 12.5kN/m³ for peat are considered to be appropriate values</p> <p>^c No laboratory test data is available for this material. This range is recommended based on the type of material encountered and previous experience of this type of material.</p> <p>^d The higher values presented here are likely to be in the PFA, with lower values being more representative of the made ground generally</p> <p>^e The higher values are likely to be in material near the top of the stratum where a crust is present however the undrained shear strength of the alluvium increases with depth and a design value of 10+1.7z is considered appropriate.</p> <p>^f Recommended values based on CIRIA C760 Section 5.5 p108 and experience in these materials</p> <p>ⁱ 0.5 to 1% strain range</p> <p>^j 0.5 to 1% strain range</p> <p>^k Values based on engineering judgement and reference to CIRIA C760</p>					

3.5 Justification for modelling approach and software

3.5.1 Analysis has been undertaken using the 2D finite element analysis software Plaxis. A soft soil creep constitutive model has been used so that yielding and creep are modelled as well as failure. This also allows for excess porewater pressures within the underlying alluvium to be comprehensively considered for all stages of the slope's design life.

3.5.2 Analyses were undertaken to ensure the existing slope remains stable and assess the proposed slopes as presented in section 3.9. Two analysis cases have been adopted:

- Existing PFA slope at 1v:3h up to 6.5mAOD with a Reworked London Clay slope at a gradient of 1v:3h and with 2m bench at the top of the PFA slope (considered as current day)
- All PFA excavated down to the top of the alluvium and reworked London Clay slope at 1v:3h placed up to 8.5mAOD (considered as proposed works)

3.6 Justification of geotechnical parameters selected for analyses

3.6.1 The geotechnical parameters provided in Table 3-4 have been derived from laboratory and field test data undertaken in the four phases of investigation which have been undertaken to date for the LTC project.

3.6.2 The soft soil creep constitutive model used in the analyses requires additional geotechnical parameters to be defined. These are outlined in Table 3-5. Compressibility and permeability parameters are based on oedometer testing

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and effective strength parameters on triaxial testing. Undrained A conditions were used, meaning effective strength parameters are required. The resulting shear stress available to be mobilised was compared to undrained strength profiles to ensure they were appropriate.

Table 3-5 Alluvium soft soil creep parameters

Parameter	Unit	Alluvium		
		Upper Clay	Peat	Lower Clay
Bulk Density	kN/m ³	16	12	16
Drained (effective stress) shear strength parameter (c')	kPa	0	0	0
Drained (effective stress) shear strength parameter (φ')	°	25.6	35	27.0
Compression Index (Cc)	-	0.89	4.29	0.4
Recompression Index (Cr)	-	0.15	0.83	0.05
Creep Index (Cα)	-	0.036	0.21	0.014
Initial voids ratio (e ₀)	-	1.99	6.61	1.02
Hydraulic Conductivity	m/s	k _x = 4x10 ⁻⁸ k _y = 2x10 ⁻⁹	k _x = 1.3x10 ⁻⁸ k _y = 6x10 ⁻¹⁰	k _x = 4x10 ⁻⁸ k _y = 2x10 ⁻⁹

3.7 Select appropriate factors of safety

3.7.1 In line with the guidance provided in P1-385/TR2 [3] and the previously agreed permit assessment [1], a global factor of safety on the slope stability of 1.3 is recommended.

3.8 Sensitivity analysis

3.8.1 As the proposed works will use site won material and the slopes are considered to be stable, no sensitivity analysis is considered necessary.

3.9 Assessment

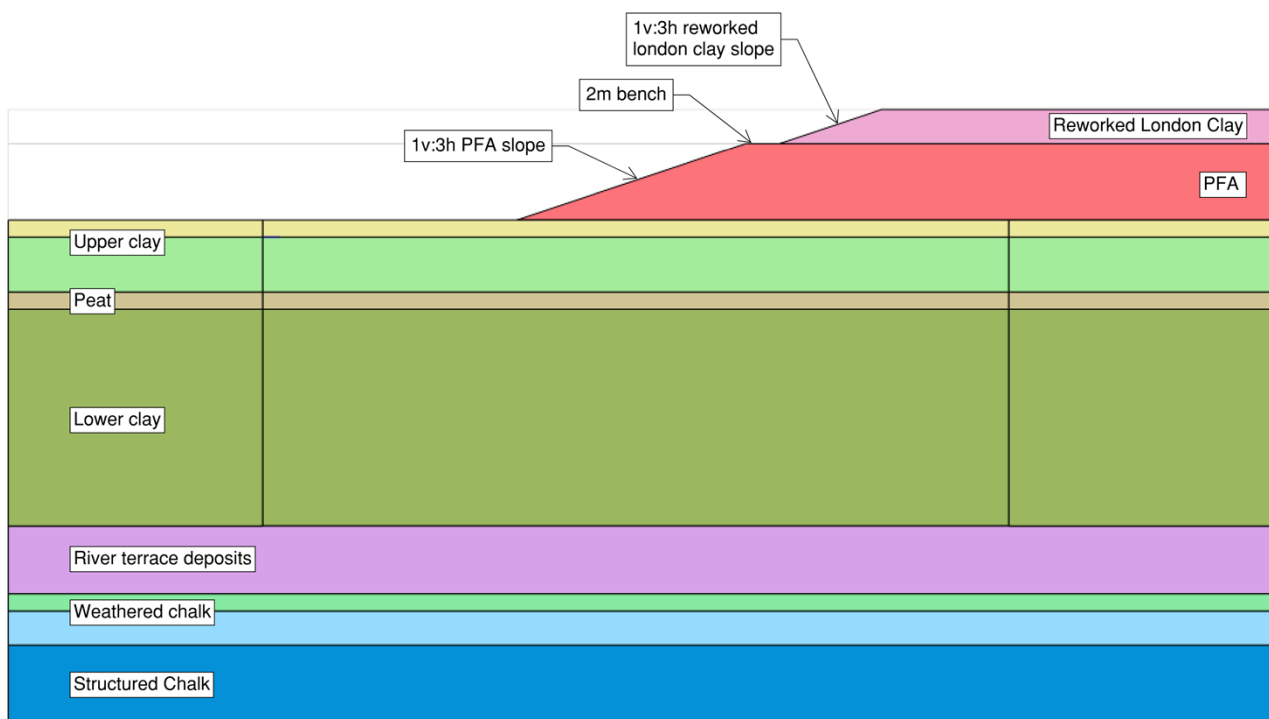
3.9.1 An assessment of the ground conditions at the site has been undertaken and considered against the existing assessment undertaken in 2021. This indicates that the parameters adopted for the assessment are appropriate and the parameters can be considered valid for assessment. It should also be noted that the parameters adopted for the assessment of the slope failure (Figure 1-4) also broadly align with these parameters.

- 3.9.2 The proposed works on the site will comprise earthworks which fit within the parameters outlined in the existing permit. As such, the assessment undertaken for the previous Stability Risk Assessment in 2021 [1] is considered to remain valid for the forthcoming works.
- 3.9.3 The previous assessment found that factors of safety of greater than 1.3 were achieved for all assessed slopes and modes of failure. This is therefore considered acceptable for the works.

Existing slope analysis

- 3.9.4 The purpose of this analysis was to assess the condition of the existing PFA slope. The geometry assessed is shown in Figure 3-2, this is based on the slope geometry which is understood to be the design case for the agreed permit.

Figure 3-2 Existing slope model geometry



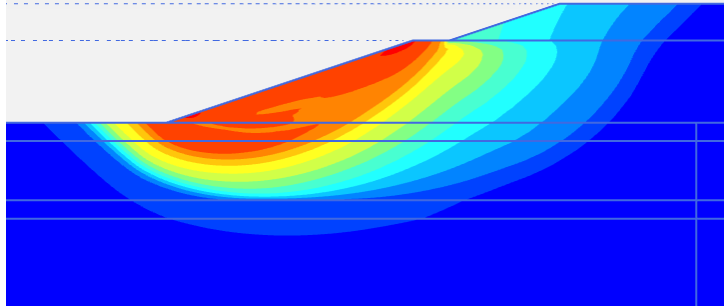
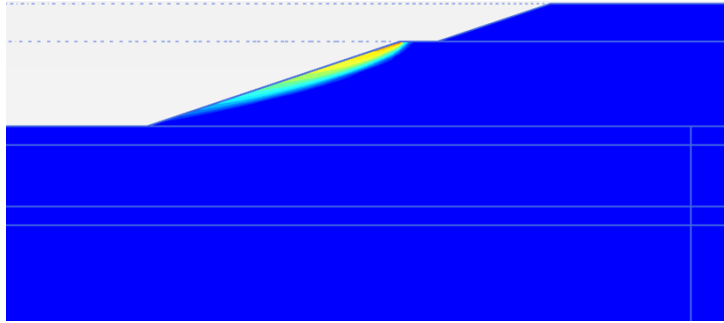
- 3.9.5 The stages used in the FEA model are presented in Table 3-6. Hold periods are included to allow for the dissipation of excess pore water pressures (ePWP). Time periods are based on the construction sequence which has been derived from available data. The end of the 9 year hold period represents the current conditions on site (2026), a dissipation stage was then used to allow ePWP to reduce to negligible values in order to assess the drained (long term) stability of the slope.

Table 3-6 Existing slope model stages

Stage	Changes	Duration (years)
Initial	-	-
100 year hold	-	100
PFA slope constructed	1v:3h PFA slope constructed to 6.5m AOD	5
10 year hold	-	10
London clay placed	1v:3h reworked London clay slope constructed to 8.5m AOD	5
9 year hold (current day)	-	9
Dissipation	Excess PWP allowed to dissipate to zero	~250

3.9.6 A series of c'-phi reduction stages were used to assess the global factor of safety of the slope at relevant stages, the results of which are shown in Table 3-7. Given the slope is already in place, only the current conditions and long-term stability of the slope were assessed. In both cases the global FoS was calculated to be above the 1.3 requirement.

Table 3-7 Global factors of safety of the existing embankment

Stage	Global factor of safety	Failure mechanism (incremental displacements from c-phi reduction stage)
London clay slope (current day)	1.71	
London clay slope (long term)	1.58	

Proposed reworked London clay slope

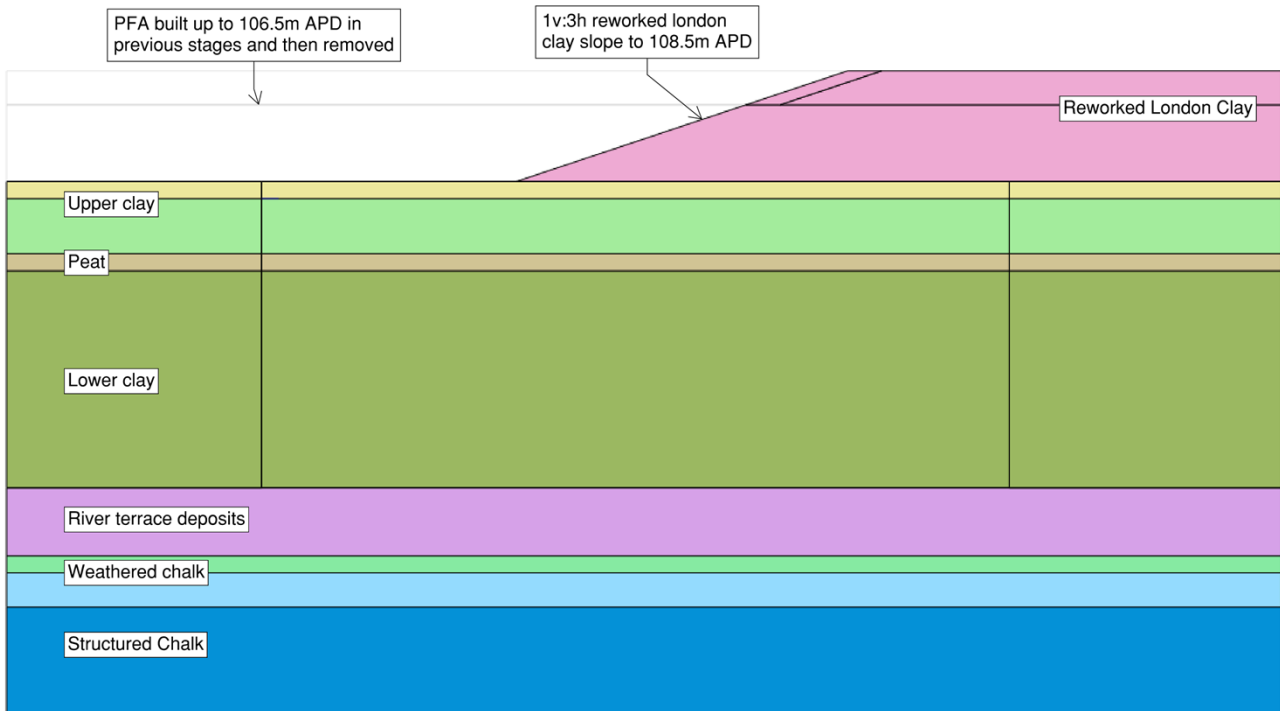
- 3.9.7 The purpose of this analysis is to assess the works outlined in section 2.1.2 for the removal of the existing PFA material and construction of a 1v:3h reworked London clay slope for LTC works.
- 3.9.8 The model stages are presented in Table 3-8. The existing PFA material and subsequent removal have been modelled so that the stress history and resulting strength in the Alluvium is representative of the existing condition. A 2 year hold period was included after the removal of the PFA. This is considered to be conservative as it allows for the suction within the Alluvium due to the removal of the PFA material to reduce. It has been estimated that the time period for the construction of the new slope will be 1 year.

Table 3-8 Proposed slope model stages

Stage	Changes	Duration (years)
Initial	-	-
100 year hold	-	100
PFA placed	PFA placed sitewide to 6.5m AOD	5
20 year hold (current day)	-	10
PFA removed	PFA material removed (ground level at 2m AOD at the top of Alluvium)	1
2 year hold	-	2
Build LC slope	1v:3h reworked London clay slope constructed to 8.5m AOD	1
Dissipation	Excess PWP allowed to dissipate to zero	~400

- 3.9.9 The proposed slope geometry is shown in Figure 3-3.

Figure 3-3 Proposed slope model geometry



3.9.10 A series of c'-phi reduction stages were used to assess the global factor of safety of the slope at relevant stages, the results of which are shown in Table 3-9. Global factors of safety remain above the 1.3 requirement for all stages assessed, hence the slope is considered stable.

Table 3-9 Proposed slope global factors of safety

Stage	Global factor of safety	Failure mechanism (incremental displacements from c-phi reduction stage)
London clay slope (short term)	1.46	
London clay slope (long term)	2.02	

3.10 Monitoring

- 3.10.1 The monitoring requirements for the site are essentially limited to the control of the waste and restoration soils levels such that the final restoration contours are achieved. However, ongoing monitoring as part of the LTC project will continue including the monitoring of groundwater on the site.

3.11 Conclusion

- 3.11.1 The proposed works for the site lie within the bounds of the works which have been agreed on the site for the existing permit. Existing material on the site will be moved to create a level site and slopes with a maximum gradient of 1v:3h will be made up to maximum heights of approximately 8mAOD.
- 3.11.2 Further ground investigation has been undertaken at the site which supplements existing data used on the assessment undertaken by SLR. The results of the additional GI indicate that the parameters adopted by SLR are reasonable and the ground model used remains valid.
- 3.11.3 An assessment has been undertaken of the existing condition on site and the proposed works. The assessment has been undertaken in the 2D FE programme, Plaxis 2D using a soft soil creep model. This allows not only the global stability to be assessed but allows for yielding and creep to also be modelled. Excess porewater pressures within the underlying alluvium have also been comprehensively considered for all stages of the slope's design life as part of the soft soil creep model.
- 3.11.4 In line with the previously assessed waste slopes, the minimum factor of safety for these works is expected to be 1.3 and therefore the slopes will remain stable. However, in light of the slope failure which occurred on the site in 2019, the rate of placement of fill material on site requires control and adequate hold periods to prevent a reduction in effective stress causing instability in the slope.

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4 Reference Documents

Document title	Document number /Link
[1] Tilbury Ash Fields Inert Landfill Stability Risk Assessment, SLR, 2021	ref 416.01526.00076.
[2] Stability of Landfill Lining Systems: Report No.1 Literature Review	P1-385/TR1
[3] Stability of Landfill Lining Systems: Report No.2 Guidance	P1-385/TR2
[4] Root Cause of Ground Movement Expert Report, COWI, 2025	A123989-COWI-XX-XX-RP-GE-000002