REPORT

Biffa Waste Services Ltd

Eye Eastern, Extension Landfill

Stability Risk Assessment

Submitted to:

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1.0 INTRODUCTION

1.1 General

Biffa Waste Services Ltd (Biffa) would like to extend its existing landfill operations at Eye Landfill, Eyebury Road, Eye, Peterborough PE6 7TH (the 'Site') by the development of an Eastern Extension. The Site currently consists of four main areas compromising the Central Area, Northern Extension, Northeastern Extension and Southern Extension.

Willow Hall Farm Quarry and Inert Landfill is located immediately to the east of Eye Landfill and is operated by PJ Thory Ltd (Thory). It is an active sand and gravel quarry which is being restored to a low level, flat lying restoration through the progressive importation of inert waste. Biffa and Thory have agreed the feasibility of Biffa utilising void space at Willow Hall Farm Quarry and Inert Landfill for the disposal of non-hazardous waste and have been working collaboratively to this effect. In doing so, Biffa recognises the need for this permit variation application to include transfer of operations from Thory to Biffa and to include the necessary adjustments to the existing scheme. Re-development as a non-hazardous waste landfill requires a new scheme for the excavation and movement of underlying clay materials, excavation and relocation of inert waste already deposited, and changes to the site layout, infrastructure, approved phasing and restoration contours.

Biffa has requested Golder, member of WSP in UK (Golder), to prepare a Stability Risk Assessment (SRA) for the development of parts of Willow Hall Farm Quarry as a non-hazardous landfill (to be called the Eastern Extension) for continuous and uninterrupted landfilling operations after filling in the current Southern Extension at Eye Landfill is ceases in March 2023. The Eastern Extension filling is expected to commence in April 2023 be complete in approximately 2038. The Eastern Extension will be filled in ten cells, progressing in numerical order, from Cell 9 to Cell 18.

Inert waste already placed by Thory at the north end of its Inert Landfill would be excavated by Biffa and re-deposited in dedicated inert Cells 19 and 20 between the transmission line and the Cat's Water Drain. Inert waste would be placed to flat-lying surrounding ground levels and restored to provide an extension to Biffa's existing Wildlife Corridor.

1.2 Report Methodology

This document provides a Stability Risk Assessment (SRA) to support an Eastern Extension permit variation application. The SRA aims to assess the stability of the basal lining system, the sidewall lining system, the waste mass, the capping system, and the leachate extraction and monitoring system. The SRA has been prepared in accordance with the stability assessment methodology as outlined in the Environment Agency's guidance document released in March 2003 and entitled "Stability of Landfill Lining Systems: Report No. 2 Guidance" (Reference 1).

The assistance of Biffa in the provision of data for this work is gratefully acknowledged. Golder has not independently verified any of the information supplied by Biffa to support this risk assessment.

2.0 SITE DEVELOPMENT AND SETTING

2.1 Eye Landfill

Eye Landfill has been progressively developed as a quarry for the extraction of sand and gravel with restoration by landfill under a series of planning permissions since 1966. The different areas of the landfill, the Central Area, the Northern Extension, the Northeastern Extension, and the Southern Extension have been filled successively (**Figure SRA1)** since 1982. The Southern Extension is expected to cease filling waste for non-hazardous in March 2023. Stable non-reactive hazardous waste (asbestos waste) will continue to be accepted in the Southern Extension until end December 2025 but will not be taken in the Eastern Extension.

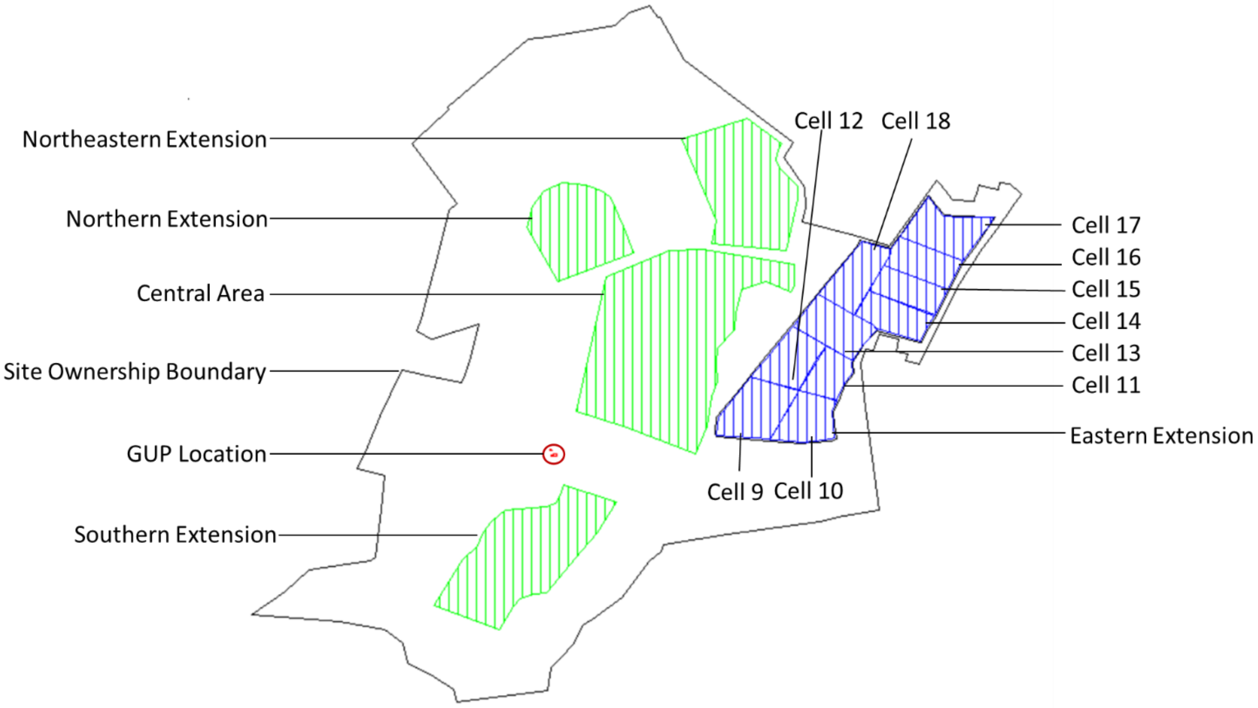

Figure SRA1: Site Layout

Landfilling in the Eastern Extension would commence in the southwestern corner and move anticlockwise and then progressively northwards. The site would receive some 3.23 Mm^3 of waste (pre-settlement, pre-restoration) or 2.43 Mm^3 of waste (post-settlement, pre-restoration) over the period from 2023 to 2038 followed by completion of restoration. The non-hazardous waste accepted at the landfill is expected to continue that already received at the Southern Extension and to consist of general industrial and commercial waste, inert materials and cover materials, contaminated soils and difficult wastes in line with the current waste composition at the Southern Extension.

Two additional cells (Cells 19 and 20) will be located between the Cat's Water Drain and transmission wires and will receive inert waste already deposited at the site by Thory.

2.2 Site Surroundings

The Eastern Extension is approximately 1.1 km southeast of the village of Eye and 2.3 km east of Peterborough. It is in a predominantly rural area, surrounded by agricultural fields and isolated dwellings. The A47 road is 1,150 m to the north, Eyebury Road is 1,400 m to the west, Oxney Road is 400 m to the southwest (of the Site Reception) and Willow Hall Lane passes down the eastern boundary.

The Eastern Extension lies on flat ground with an elevation ranging from approximately 3.5 m AOD to 4.5 m AOD. Spot heights provided by the Ordnance Survey on Willow Hall Lane show 4.0 m AOD at the north end of the Eastern Extension and 3.7 m AOD at the southern end.

Bar Pastures Scheduled Ancient Monument (SAM) is located immediately north of Willow Hall Farm Quarry and Inert Landfill around Bar Pastures Farm. It is part of a settlement of Iron Age and Roman date, with a drove and associated ditches, rectilinear yards and other enclosures, some of which contain the remains of buildings. It is located on a gravel terrace about 1 km west of what was, formerly, the edge of the peat fen. Archaeological features are visible as low earthworks and as buried features within the underlying gravel below the depth of ploughing. The stand-off of 50 m from the Bar Pastures Scheduled Ancient Monument (planning permission 12/01008/MMFUL) was reduced to 12 m (planning permission 17/00279/WCMM); however, Biffa proposes to revert to the 50 m stand-off originally proposed.

Bridleway/Footpath Eye 3 runs in an east to west direction across Eye Landfill and across the Willow Hall Farm Quarry, but is proposed to be wholly south of the Eastern Extension. It forms part of the Peterborough Green Wheel - a recreational route around the city with 'spokes' out from the centre.

2.3 Description of existing Willow Hall Farm Quarry and Inert Landfill

Willow Hall Farm Quarry and Inert Landfill is an active sand and gravel quarry operated by Thory. The site is being restored to a low level, flat lying restoration through the progressive importation of inert waste.

Pedestrian access to the site can be gained via Willow Hall Lane which runs south-westwards from the A47 trunk road; however, access for the export of sand and gravel and the import of inert waste is via a long, separate haul road from the east. Planning permission (Reference: 12/01008/MMFUL) was obtained in 2013 and an Environmental Permit (now EPR/FB3204MX/T001) in 2016.

Thory is systematically extracting mineral and filling with inert waste behind in a continuous operation from north to south. The sand and gravel is a shallow deposit of variable thickness and typically less than 6 to 8 m. It occurs below the top soil and a silty overburden, and overlies clay. To date, the site has progressed as follows:

- 'Restored Area' (north end of site). Sand and gravel has been extracted and the void backfilled with inert waste. Prior to infilling, clay excavated from the base of the quarry has been placed against the sidewalls to provide a geological barrier and to manage groundwater. The Area has been filled, graded and restored to a flat lying low level restoration, about 1 m below surrounding ground level.
- 'Active Filling Area'. Sand and gravel has been extracted, clay placed, and inert waste is currently being deposited. Waste exposed in the tipping face comprises primarily a brown soil-like material.
- 'Active Extraction Area'. Sand and gravel has been extracted down to the top of clay. The haul road for dump trucks passes across this area to the mineral extraction face that extends west to east and defines the southern edge. All sand and gravel has been removed but all top soil and overburden remains on site in areas already restored, in screening bunds, edge protection bunds, and in stockpiles on the quarry bottom.
- 'Soil Stripping Area'. Topsoil has been stripped in advance of the working face and archaeological survey takes place in accordance with the planning permission.
- 'Unworked Area' (southern end of the site). The Unworked Area remains in agricultural use for the time being. The Green Wheel footpath passes across the Unworked Area but in time will be subject to diversion and then reinstatement as a bridleway on its original route, in accordance with the planning permission.

Thory estimates that mineral extraction will be completed at end 2025. Consequently, if Biffa enters the Eastern Extension in April 2023, mineral extraction will have advanced to about the line of the Green Wheel path, and not wholly complete.

Mineral extraction is described by Thory in terms of three phases i.e. Northern, Central and Southern. The boundary between the Central Phase and the Southern Phase occurs, west to east, just north of the Green Wheel footpath, where the base of the sand and gravel shallows. The recoverable mineral reserve tonnage was identified in the planning application to be 2.25 Mt.

2.4 Geology

The British Geological Survey, sheet 158 for Peterborough, indicates that the Eastern Extension is underlain by Quaternary drift deposits which overlie the Jurassic Oxford Clay Formation and Kellaway Sands. It is noted that the Quaternary drift deposits have been targeted and excavated by the quarry operation but remain present around the sides of the excavation. A summary of the regional geology is presented in **Table SRA1**.

Table SRA1: Summary of Regional Geology

Age	Formation	Description	Approximate Thickness (m)
Quaternary	River Terrace Deposits	Sand and gravel with some silt	Variable
Jurassic	Oxford Clay	Olive grey fossiliferous, bituminous shale and blocky mudstone	63 – 76 m
	Kellaway Sands	Grey clayey silt and mud	1.9 – 6.4 m
	Kellaway Clay	Grey fissile mudstone	1.4 – 5.8 m
	Cornbrash	Fine grained shell-detrital limestone	1.2 – 4.3 m
	Blisworth Clay	Grey/Green mudstone with thin limestone	3.0 – 6.0 m
	Blisworth Limestone	Shell-detrital to micritic limestone with marl and mudstone	1.9 – 5.1 m

The Kellaway Sands and Oxford Clay underlie the whole of the Eastern Extension Area. The Oxford Clay is a well consolidated, calcareous clay which may be silty or sandy with thin cemented siltstone or mudstone. The Oxford Clay has been proven at the Eastern Extension by five boreholes is known locally to be 12.30 m to 17.50 m in thickness. It is typically described as stiff, very closely fissured, dark grey clay with frequent disseminated shell fragments.

The top of the Oxford Clay varies in elevation from 1.67 m AOD in the south to (-3.38) m AOD in the north. The base of the Oxford Clay varies in elevation from (-14.35) m AOD in the southwest to (-20.21) m AOD in the east.

2.5 Hydrogeology

The near surface River Terrace Deposits and the Kellaway Sands are the principal water bearing strata at the site. They are separated by the low permeability Oxford Clay which is an aquitard (i.e. does not transmit water at a significant rate).

The presence of dewatering operations associated with the ongoing mineral extraction and inert landfilling, together with groundwater management at Eye Landfill and previously at Cemex's adjacent operations further to the west means that the water table in the River Terrace Deposits is variable. The site investigation carried out in 2011 prior to development reported the groundwater to be between 1.25 to 2.35 m AOD and that there is hydraulic continuity between groundwater in the River Terrace Deposits and the Cat's Water Drain.

Groundwater is confined within the Kellaway Sands such that the piezometric level is at an elevation within the River Terrace Deposits. The high groundwater pressures developed within the Kellaway Sands mean that excavation into the Oxford Clay is constrained by the requirement to maintain a satisfactory factor of safety against basal heave.

2.6 Life Cycle Phases

2.6.1 General

The Eastern Extension will be divided into ten landfill cells for non-hazardous waste (Cells 9 to 18) and development shall proceed from the south towards the north. Progressive capping, restoration and landfill gas management within the Eastern Extension will be carried out as each cell is completed.

2.6.2 Waste Mass Geometry

As the waste is to be filled cell-by-cell, it will be necessary to form temporary waste slopes. The maximum temporary waste slope will be approximately 1(v):2(h) and the maximum permanent waste slope will be approximately 1(v):4(h) (pre-settlement).

2.6.3 Groundwater Management

In preparation of the Site to formation level and prior to placement of the engineered clay liner, a semi-perforated pipe drain will be installed, as required, behind the liner to collect and intercept groundwater in the shallow sand and gravel deposits. This drain will be progressively installed around the perimeter of the engineered area draining under gravity to engineered sumps.

Groundwater will be pumped from the sumps using a submersible pump, with groundwater being discharged into internal site drains or to the existing surface water pond for discharge to the Cat's Water Drain. Control of groundwater will be undertaken throughout the period of landfill development until waste has been placed across the whole site to an agreed level to ensure the stability of the perimeter side slopes, after which control of groundwater will cease.

2.6.4 Leachate Management

Leachate will be managed in Cells 9 to 18. The principles of leachate management have been established at the Southern Extension and are controlled through the Environmental Permit. Leachate management is not required in Cells 19 and 20.

For protection of the groundwater environment and in accordance with the Environmental Permit, the Site will be hydraulically contained such that the level of leachate in the base of each cell is maintained at a level lower than the surrounding groundwater level in the Kellaways Sand and River Terrace Deposits (once rebound occurs following cessation of groundwater management). Cells 9 to 18 will have infrastructure installed to manage leachate. Leachate may also be re-applied to the waste mass to aid degradation.

A leachate collection and removal system will be installed in each Cell 9 to 18. Leachate will be extracted from leachate sumps in the bottom of each cell by means of a vertical or side slope leachate extraction well extending to the surface of the landfill. The wells accommodate automatic pumping equipment (eductor or submersible pumps) to extract leachate.

The Eastern Extension Landfill will be hydraulically separated from its immediate surroundings by the engineered lining system and leachate levels across the base will be managed in accordance with the Environmental Permit i.e. Cells 9 to 18 will be hydraulically separated from each other by lined bunds, approximately 2 m high and from Cells 19 and 20 by a full height bund. The use of the inter-cell bunds will ensure that surface water collecting in non-operational sections of the Eastern Extension will remain uncontaminated by leachate. In addition, the bunds would assist in the control, containment and collection of leachate generated by landfilling operations.

Two leachate monitoring wells and one leachate abstraction well will be used to monitor, control, and remove leachate from each cell for re-circulation and/or treatment and disposal. The wells will be hydraulically connected to the leachate drainage system to optimise leachate control.

Excess leachate will be removed from the low point in the basal drainage system, by means of a leachate extraction well which will extend up to the surface. Leachate will be extracted from the cells to maintain leachate heads within each cell below the leachate head compliance level. Leachate will be transferred by surface pipework from the abstraction wells to the leachate holding tank at the Site Reception for removal by road tanker to an appropriately authorised water treatment works

Leachate generated within the inert waste landfill will by definition not be contaminated and will be allowed to infiltrate to groundwater without collection, treatment or disposal

2.6.5 Gas Management

Landfill gas will be managed in Cells 9 to 18. The principles of landfill gas management have been established at the Southern Extension and are controlled through the Environmental Permit. Landfill gas management is not required in Cells 19 and 20.

An active gas extraction system comprising gas extraction wells at approximately 40 m centres will be progressively installed across Cells 9 to 18 in the Eastern Extension and commenced within 12 months of the completion of each cell to pre-settlement, pre-restoration levels or at the earliest opportunity in the event that elevated gas levels are detected.

Landfill gas wells will be connected by a system of carrier pipes, valves, manifolds and condensate knock-out pots to a large diameter ring or branch main, that will divert gas to the crossing point over the Cat's Water Drain. From there, landfill gas from the Eastern Extension will connect with the existing gas collection system for Eye Landfill:

- Gas is collected from gas wells, generally at 40 m spacing, across the Central, Northern and North-eastern Extensions. These areas are now restored and the gas collection pipework is mostly buried.
- Gas extraction wells and pipework are currently being progressively installed in the Southern Extension.

All gas is piped to the existing Gas Utilisation Plant (GUP).

Landfill gases will be monitored and actively controlled and managed across the Eastern Extension throughout its operational life and during its post closure and Aftercare Period.

By definition, landfill gases will not be generated within areas of inert landfill such that gas will not need to be collected from Cells 19 and 20.

2.7 Conceptual Stability Site Model

2.7.1 Basal Sub-grade Model

The published geological maps indicate that the whole Site is underlain by River Terrace Deposits which comprise sand and gravel and which has been removed by quarry operations. The Oxford Clay formation lies beneath, which consists of well consolidated, calcareous clay which may be silty or sandy with thin cemented siltstone or mudstone and forms the basal sub-grade to the landfill. The Oxford Clay will provide a natural geological barrier over the base and lower sideslopes.

Below the Oxford Clay is the Kellaway Sands (main aquifer) which consists dominantly of silty sands and clayey silts with siltstone and mudstone. The strata are underlain by the Cornbrash and Blisworth Limstone.

Prior to commencement of landfilling activities, the base of the Site will be excavated down to approximately between (-5) m AOD and (-4) m AOD which is subject to this basal heave assessment in the conceptual model. The basal level of each individual cell will be determined and submitted to the EA as part of the Construction Quality Assurance (CQA) Plan for each cell prior to construction and in accordance with prevailing groundwater level conditions.

2.7.2 Side Slopes Sub-Grade Model

The side slopes sub-grade comprises the River Terrace Deposits and Oxford Clay. The side slopes are expected to form an angle of 1(v):2.5(h) before construction of the clay liner system.

The base of the sand and gravel is c. 6 to 7 m bgl in the current Restored Area, Active Filling Area, Active Extraction Area and Soil Stripping Area. The thickest sand and gravel so far encountered appears to be at the western end of the current working face.

With time, mineral extraction will proceed southwards into the current Unworked Area and the base of the sand and gravel is expected to rise to 4 to 6 m bgl. Further south, towards the Green Wheel path and beyond, the base of the sand and gravel rises further to c. 2.5 to 4 m bgl and in the far southwest corner of the Site, the sand and gravel is thin or absent.

The ‘top of side slope’ for the non-hazardous landfill along its southern boundary will be 20 m north of the Green Wheel path. The side-slope will be supported on a full height engineered clay bund. Areas to the south of the Green Wheel Path will be re-instated with backfilled clay.

For non-hazardous landfill, where River Terrace Deposits are exposed in the upper side slopes, the geological barrier will be artificially established and comprise 0.5 m of engineered clay with a maximum permeability of 1×10^{-9} m/s.

For inert landfill, where River Terrace Deposits are exposed in the upper side slopes, the geological barrier will be artificially established and comprise 1.0 m of engineered clay with a maximum permeability of 1×10^{-7} m/s.

2.7.3 Basal Lining System Model

For non-hazardous landfill, the artificial sealing liner for the basal and lower sidewall lining system will comprise 1.0 m of engineered clay with a maximum permeability of 1×10^{-9} m/s placed on the natural geological barrier.

2.7.4 Intercell Bunds

Each cell will be hydraulically separated from adjacent cells by an intercell bund constructed using low permeability engineered clay. Bunds will be a minimum of 2.0 m high and 2.0 m wide at their crest with a side slope gradient of 1v:2h.

2.7.5 Leachate Drainage System

Leachate will be extracted from leachate sumps in each cell for non-hazardous waste by means of a vertical or side slope leachate extraction well extending to the surface of the landfill. The wells accommodate automatic pumping equipment (eductor or submersible pumps) to extract leachate.

The base of each cell will be profiled to provide a fall of approximately 1:100 towards a leachate collection point. A pipe system will be placed on the surface of the basal clay that comprises a central HDPE slotted pipe with secondary drains comprising HDPE slotted pipe connected at regular intervals in a herringbone pattern. The central pipe will be connected to the leachate extraction point.

The drainage blanket may comprise aggregate, recycled aggregate, shredded tyres, or baled tyres. The leachate drainage system will conform to the choice of material and the specification contained within a CQA Plan submitted to the EA prior to construction. Installation and construction quality assurance procedures for the leachate drainage system will be defined within the CQA Plan.

2.7.6 Side Slope Lining System Model

For non-hazardous landfill, the artificial sealing liner for the upper sidewall lining system will comprise 0.5 m of engineered clay with a maximum permeability of 1 x 10^{-9} m/s placed on the artificially established geological barrier.

The engineered clay will conform to the specification contained within a Construction Quality Assurance (CQA) plan submitted to the Agency prior to construction.

An artificial sealing layer is not required for the disposal of inert waste.

2.7.7 Waste Mass Model

The permitted waste list for the Eastern Extension will be the same as that currently approved for the Southern Extension excluding stable non-reactive hazardous waste. Non-hazardous waste is proposed to be accepted at a constant rate of 220,000 tpa for all years, pro rata during the last year.

Assuming a post-settlement landfill waste mass with its base at (-4.30) m AOD and top at (+12.5) m AOD (allowing for 1 m thickness capping and restoration), the maximum waste thickness will be 16.80 m (post-settlement) or 22.40 m (pre-settlement, assuming 25% settlement). The maximum temporary waste slope will be approximately 1(v): 2(h), and the maximum permanent final waste slope will be approximately 1(v): 4(h).

Inert waste already deposited by Thory will be excavated and re-deposited in dedicated areas (Cells 19 and 20). The classification of these areas will be Inert. These areas will be restored to pre-existing ground level (~4 m AOD).

2.7.8 Capping System Model

2.7.8.1 General

To reduce the amount of precipitation that can infiltrate the waste, a low permeability cap will be constructed as waste deposition in each cell is completed to final pre-settlement levels. The principles of engineered capping and restoration have been established at the Site and are controlled through the Environmental Permit. They will continue in the development of Cells 9 to 18 in the Eastern Extension and are described below. A sealing layer is not required for inert Cells 19 and 20. The specification of the cap is outlined in the following sections.

2.7.8.2 Regulation Layer

Prior to the placement of the regulation layer the waste will be thoroughly compacted and smoothed so that sharp objects do not protrude excessively, and the thickness of the regulation layer may be controlled. A nominal 200 mm layer of sand, clay, or similar inert waste material will be laid over the waste in Cells 9 to 18 as a regulation layer. The regulation layer will be spread and compacted over the waste and will be smooth and free from debris, roots, angular or sharp gravel, boulders or any materials considered to be capable of causing damage to the sealing layer.

2.7.8.3 Sealing Layer

The upper sealing layer will be provided over the waste by placement of:

- 1.0 m of engineered clay with a maximum permeability of 1 x 10^{-9} m/s; or
- Fully welded flexible membrane liner (FML).

It will be placed as approved in accordance with the specification contained within a CQA Plan submitted to the EA prior to construction.

2.7.9 Restoration Soils

Restoration soils will be placed above the capping system (Cells 9 to 18) and above inert waste (Cells 19 to 20) to promote the regeneration of the landform in accordance with the restoration scheme including agricultural use, Wildlife Corridor, Green Wheel Path and landscaping areas. Subsoil (0.7 m) and topsoil (0.3 m) will be spread evenly to achieve the final pre-settlement, post-restoration profile.

3.0 STABILITY RISK ASSESSMENT

3.1 Risk Screening

3.1.1 Basal Sub-Grade and Lining Screening

The basal lining system will be constructed on natural ground consisting of Oxford Clay. Following excavation of the landfill, a minimum of 10 m of Oxford Clay will remain between the base of the Site and the top of the Kellaways Sand. This foundation is stable and not subject to any significant settlement, either total or differential, that would lead to a breach of the lining system.

The near surface River Terrace Deposits and the Kellaway Sands are the principal water bearing strata at the site. They are separated by the low permeability Oxford Clay which is an aquitard (i.e. does not transmit water at a significant rate). The Kellaways Sand is a permeable formation usually with a known or probable presence of significant fracturing. The Kellaways Sand is confined by the overlying Oxford Clay, meaning that basal heave at the Site is a potential hazard. As such, basal heave calculations are required to be undertaken as part of the Stability Risk Assessment.

3.1.2 Side Slope Sub-Grade and Lining System Screening

Side slopes are established within the River Terrace Gravel and the Oxford Clay to a gradient of 1(v):2.5(h) prior to clay lining construction. The stability of the side slope sub-grade will be assessed.

The side slope lining systems are extensions of the basal lining system, extended up the face of the cell sidewalls. The stability of the side slope liner (pre-waste placement) shall be assessed. It is considered that if the unconfined slope is stable then it would not be necessary to assess the stability of the slope post-waste placement.

Two cross sections have been used to assess the side slope subgrade and lining stability. The locations of the analysed cross sections A and B are shown on **Drawing SRA1**.

3.1.3 Waste Mass Screening

The maximum temporary waste slope angle on site will be approximately 1v:2h. Analysis is required in terms of stability of the temporary waste slopes. The final waste slopes will also be analysed. The analysed temporary and final waste cross sections C and D are shown on **Drawing SRA2**.

3.1.4 Capping System Screening

The stability of the cap and cover soils shall be considered. An LLDPE geomembrane cap, GCL cap and a clay cap have been analysed along the steepest and highest cross section D shown on **Drawing SRA2**.

3.1.5 Leachate Extraction System Screening

The foundation of the leachate extraction and monitoring points will be analysed. The pipe deflections for the leachate drainage pipework will also be analysed.

3.2 Data Summary

3.2.1 General

Various phases of site investigation have been carried out at Willow Hall Farm Quarry and Inert Landfill proposed to become Biffa's Eastern Extension Landfill. The site investigations have comprised both shallow and deep shell and auger boreholes. Data for input into the stability of the sub-grade, lining system and capping system has been sourced from the site investigation data, available literature, and experience.

3.2.2 Groundwater Levels

Detailed information about groundwater levels can be found within the Hydrological Risk Assessment for the Eastern Extension (Reference 2).

A summary of groundwater monitoring of the River Terrace Deposits is shown in **Figure SRA2** and for the Kellaways Sand in **Figure SRA3**, below.

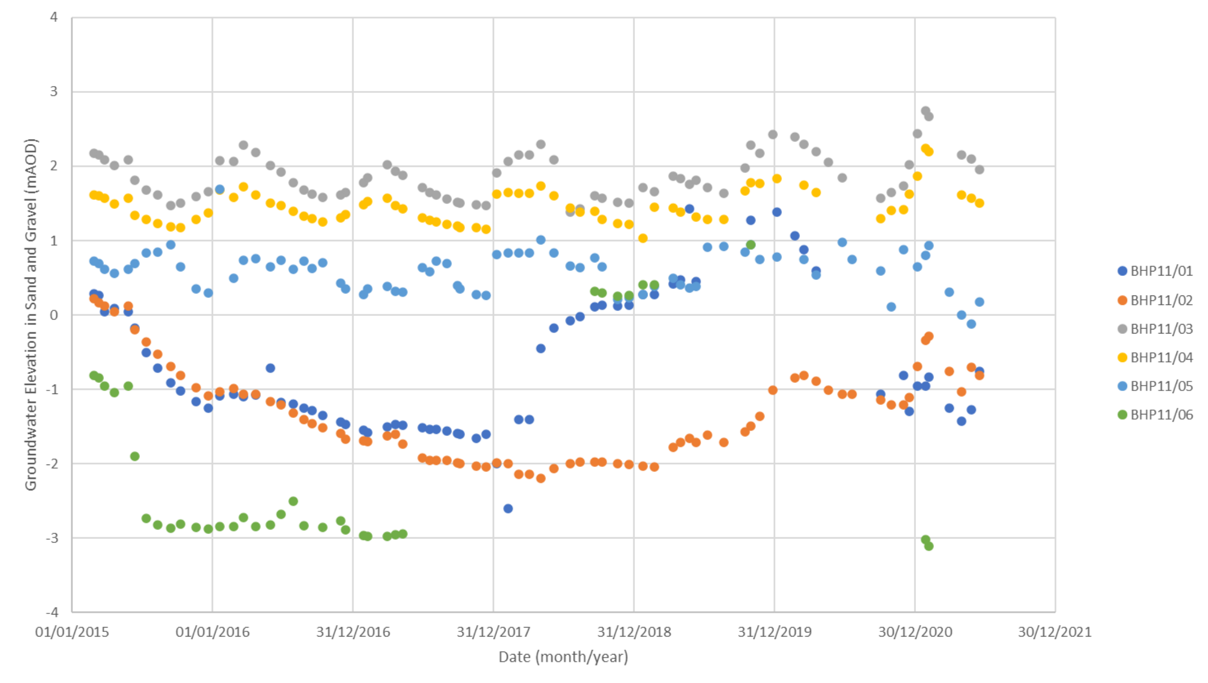

Figure SRA2: Groundwater Levels in River Terrace Gravel

A characteristic groundwater level of 2.5 mAOD in River Terrace Deposits has been adopted in the side slope sub-grade and liner stability analyses.

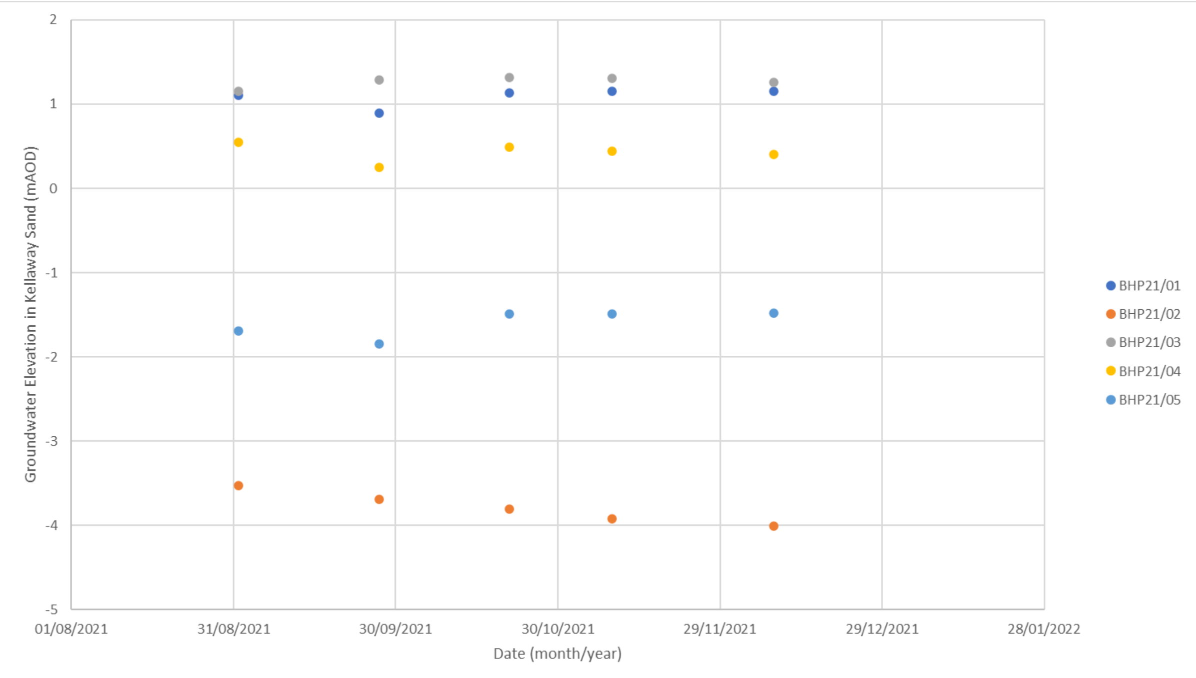

Figure SRA3: Groundwater Levels in Kellaways Sand

A characteristic groundwater level of 1.3 mAOD in Kellaways Sand has been adopted in the basal heave assessment.

3.3 Selection of Appropriate Factors of Safety

3.3.1 Factor of Safety for Basal Sub-Grade and the Basal Lining System

A minimum factor of safety of 1.2 and 1.3 against short-term and long-term basal heave respectively will be considered acceptable providing reasonably conservative parameters have been used.

3.3.2 Factor of Safety for Side Slopes Sub-Grade

A minimum factor of safety of 1.3 will be considered acceptable for the stability of the side slopes sub-grade providing reasonably conservative parameters have been used. At a factor of safety less than 1.3, although the slope may not be approaching failure, experience indicates the integrity of the lining system may be impaired.

3.3.3 Factor of Safety for Side Slope Lining System

A minimum factor of safety of 1.3 will be considered acceptable for overall stability providing reasonably conservative parameters have been used. At factors of safety less than 1.3, although the slope may not be approaching failure, experience indicates that the structure may become impaired by deformations, leading to increased permeability of the lining system.

Factors of safety of greater than 1.3 on the stability are usually considered sufficient to ensure the integrity of the lining system is not affected.

3.3.4 Factor of Safety for Waste Mass

A minimum factor of safety of 1.3 will be considered acceptable for overall stability providing reasonably conservative parameters have been used.

3.3.5 Factor of Safety for Capping System

A minimum factor of safety of 1.3 would typically be required for overall stability providing reasonably conservative parameters have been used.

3.3.6 Factor of Safety for Leachate Extraction System

A minimum factor of safety of 1.5 will be required for leachate well foundation and maximum deflection in the horizontal leachate pipework of 5%.

3.4 Justification for Modelling Approach and Software

The overall stability of the lining system prior to and post waste placement has been assessed using the slope stability programme Slope/W. Circular failure surfaces were analysed using the Morgenstern-Price method.

To summarise, stability assessments have been carried out to assess the following:

- **Stability of Side Slope Liner Pre-Waste Placement**

  The stability of the Side Slope Lining System has been assessed using the Slope/W for a range of circular failures.

- **Integrity of Side Slope Liner Pre-Waste Placement**

  The mode of integrity failure is the same as stability failure (long term) and therefore no additional calculations are required.

- **Stability of Temporary and Final Waste Slopes**

  The analysis of the temporary and final waste slopes has been carried out using the Slope/W for a range of circular failures.

- **Stability of Capping System**

  The stability of the capping system has been carried out for the steepest cross section taken through the proposed pre-settlement restoration levels. The stability of the cover soils has been assessed using the method proposed by Jones & Dixon,1998 (Reference 4) for a geomembrane cap.

- **Stability of Leachate and Drainage Extraction Systems**

  Calculations have been carried out to assess the stability of the leachate well foundation and deflection of the leachate pipework.

In all cases the worst-case scenario has been modelled. This includes the highest and steepest side slopes.

Methods of analysis are those described in the draft Agency Guidelines 'Stability of Landfill Lining Systems' (Reference 1). These represent best available techniques at the time of this report.

3.5 Justification of Geotechnical Parameters Selected for Analyses

This section describes the parameters used in the stability assessment. Parameter values have been selected based on a combination of the site specific and non-site-specific data. At all stages in the analysis conservative parameters have been selected, and where practicable, ultimate limit state parameters checked to ensure that failure is not likely with extreme conditions.

3.5.1 Parameters Selected for Basal Sub-Grade and the Basal Liner Analyses

The parameters selected for use in the basal heave analysis are presented in **Table SRA2**

Table SRA2: Summary of Parameters used in the Basal Heave Analyses

Materials	Unit Weight γ (kN/m^3)
Oxford Clay	20
Water	9.81

3.5.2 Parameters Selected for Side Slopes Sub-Grade and Liner Analyses

The material parameters used in the analysis of the side slopes are presented in **Table SRA3**. The parameters used for inert fill are considered typical.

Table SRA3: Summary of Parameters used in the Sub-grade in the Side Slope Analyses

Materials	Unit Weight γ (kN/m^3)	Undrained Shear Strength c_u (kPa)	Cohesion c' (kPa)	Friction Angle ϕ' (degrees)
Made Ground	18	-	0	30
Oxford Clay	20	-	30	26
River Terrace Deposits	20	-	0	30
Clay Liner	19	50	2	26

3.5.3 Parameters Selected for Waste Analyses

The material parameters used in the analysis of the waste slopes are presented in **Table SRA4**. The parameters for the analysis of the temporary and final waste slopes have been obtained from Reference 3).

Table SRA4: Summary of the Parameters used in the Waste Slope Analyses

Material	Unit Weight γ (kN/m^3)	Cohesion c' (kPa)	Friction Angle ϕ' (degrees)
Waste	10	5	25

3.5.4 Parameters Selected for Capping Analyses

The material parameters used in the analysis of the capping system are presented in **Table SRA5**.

Table SRA5: Summary of the Parameters used in the Capping Analyses

Material	Adhesion α' (kPa)	Friction Angle ϕ' (degrees)
Cover soil internal strength	0	25
Cover soil/Geotextile	0	24
Geotextile/Geomembrane	0	26
Geomembrane/Regulation layer	0	24
Cover soil/GCL	0	24
GCL/Regulation layer	0	24
Cover Soil/Clay Cap	0	22

3.6 Analyses

3.6.1 Basal Heave Analyses

Basal heave calculations have been undertaken in accordance with the methodology suggested in Reference 1). The detailed calculations sheets are presented in **Appendix SRA1**. A summary of the basal heave calculations is presented in **Table SRA6** below.

Table SRA6: Summary of Basal Heave Calculations

Scenarios	Factor of Safety
Formation Level at (-5) mAOD	1.20
Placement of Clay Liner	1.33
Placement of Clay Liner and Gravel	1.38

3.6.2 Side Slope Sub-Grade Analyses

A summary of the Slope/W runs for the sub-grade stability are presented in **Table SRA7** and the output files are given in **Appendix SRA2**.

Table SRA7: Summary of Slope/W Runs for Side Slope Sub-Grade Analyses

Analysis Reference	Description	Factor of Safety
Section A _Subgrade	Section A, 1v:2.5h slope	1.36
Section B _ Subgrade	Section B, 1v:2.5h slope	1.37

3.6.3 Side Slope Liner Analyses

A summary of the Slope/W runs for the side slopes liner stability are presented in **Table SRA8** and the output files are given in **Appendix SRA3**.

Table SRA8: Summary of Slope/W Runs for Side Slope Liner Analyses

Analysis Reference	Description	Factor of Safety
Section A_Liner_1	Section A, 1v:2.5h slope, fully functional back drain, undrained condition	2.02
Section A_Liner_2	Section A, 1v:2.5h slope, fully functional back drain, dry	1.59
Section A_Liner_3	Section A, 1v:2.5h slope, fully functional back drain, r_u=0.1	1.56
Section A_Liner_4	Section B, 1v:2.5h slope, dysfunctional back-drain, r_u=0.1	1.24
Section B_Liner_1	Section B, 1v:2.5h slope, fully functional back drain, undrained condition	1.93
Section B_Liner_2	Section B, 1v:2.5h slope, fully functional back drain, dry	1.57
Section B_Liner_3	Section B, 1v:2.5h slope, fully functional back drain, r_u=0.1	1.55
Section B_Liner_4	Section B, 1v:2.5h slope, dysfunctional back-drain, r_u=0.1	1.24

3.6.4 Waste Analyses

Temporary Waste Slopes

A summary of the Slope/W runs for the analyses of the temporary waste slopes are presented in **Table SRA9** and the output files are presented in **Appendix SRA4**.

Table SRA9: Summary of Slope/W Runs for Temporary Waste Analyses

Analysis Reference	Description	Factor of Safety
Section C_Temporary Waste_1	Section C, 1v:2h slope, circular failure, dry	1.39
Section C_Temporary Waste_2	Section C, 1v:2h slope, circular failure, 1m leachate level	1.39
Section C_Temporary Waste_3	Section C, 1v:2h slope, circular failure, 2m leachate level	1.35
Section C_Temporary Waste_4	Section C, 1v:2h slope, circular failure, 1m leachate level, r_u=0.1	1.26
Section C_Temporary Waste_5	Section C, 1v:2h slope, circular failure, 1m leachate level, r_u=0.2	1.13

Analysis Reference	Description	Factor of Safety
Section C_Temporary Waste_6	Section C, 1v:2h slope, circular failure, 1m leachate level, r_u=0.2, dry waste in the outer 10m of waste slope	1.18
Section C_Temporary Waste_7	Section C, 1v:2h slope, circular failure, 1m leachate level, r_u=0.2, dry waste in the outer 20m of waste slope	1.33

Final Waste Slopes

A summary of the Slope/W runs for the final waste slopes is presented in **Table SRA10** and the output files are given in **Appendix SRA5**.

Table SRA10: Summary of Slope/W Runs for Final Waste Analyses

Analysis Reference	Description	Factor of Safety
Section D_Final Waste_1	Section D, 1v:4h slope, circular failure, 2m leachate	3.04
Section D_Final Waste_2	Section D, 1v:4h slope, circular failure, 2m leachate, r_u=0.1	2.82
Section D_Final Waste_3	Section D, 1v:4h slope, circular failure, 2m leachate, r_u=0.2	2.59

3.6.5 Capping Analyses

The analyses carried out on the LLDPE geomembrane and GCL capping systems to calculate the stability of the restoration soils and the integrity of the geosynthetics were proposed by Jones and Dixon (1998), utilising a finite slope length for the selected critical capping slope cross section.

LLDPE Geomembrane Capping

A summary of the factor of safety calculated for the finite slope analyses is presented in **Table SRA11** and the output files are given in **Appendix SRA6**.

Table SRA11: Summary of Geomembrane Capping Stability Analyses

Description		Factor of Safety		
		Slippage of Restoration Soil	Tensile Failure of Geotextile	Tensile Failure of Geomembrane
Section D, 1v:4h slope, 6m high	PSR = 0	1.98	Infinite	Infinite
	PSR = 0.5	1.48	Infinite	Infinite
	PSR = 1.0	1.05	Infinite	Infinite
	PSR = 0.65	1.34	Infinite	Infinite

PSR represents Parallel Submergence Ratio

GCL Capping

A summary of the factors of safety calculated for the finite slope analyses is presented in **Table SRA12** and the output files are given in **Appendix SRA7**.

Table SRA12: Summary of GCL Capping Stability Analyses

Description		Factor of Safety	
		Slippage of Restoration Soil	Tensile Failure of GCL
Section D, 1v:4h slope, 6m high	PSR = 0	1.98	Infinite
	PSR = 0.5	1.48	Infinite
	PSR = 1.0	1.05	Infinite
	PSR = 0.65	1.34	Infinite

PSR represents Parallel Submergence Ratio

Clay Capping

A summary of the factors of safety calculated for the finite slope analyses is presented in **Table SRA13** and the output files are given in **Appendix SRA8**.

Table SRA13: Summary of Clay Capping Stability Analyses

Description		Factor of Safety against Slippage of Restoration Soil
Section D, 1v:4h slope, 6m high	PSR = 0	1.82
	PSR = 0.5	1.36
	PSR = 1.0	0.96

PSR represents Parallel Submergence Ratio

3.6.6 Leachate Extraction System Analyses

Extraction of Well Foundation

A summary of the foundation bearing capacity analysis and differential settlement calculated for the leachate extraction well is presented in **Table SRA14** and the calculations sheets are given in **Appendix SRA9**.

Table SRA14: Summary of Leachate Extraction Well Foundation Analyses

Description	Factor of Safety		Differential Settlement (mm)
	Total Stress	Effective Stress	
Leachate extraction wells with 3 x 3 x 0.3 m concrete base and 23m total height	1.5	23.9	3.3

Leachate Pipework Deflection

A summary of the leachate pipe work deflection calculations is presented in **Table SRA15** and the calculation sheets are given in **Appendix SRA10**.

Table SRA15: Summary of Leachate Pipe Work Deflection Calculations

Description	Pipe Deflection	
	(mm)	(%)
Leachate pipe with an internal diameter of 120mm	3.36	2.8
Leachate pipe with an internal diameter of 160mm	4.48	2.8

3.7 Assessment

3.7.1 Basal Heave Assessment

The basal heave analysis considers the worst-case scenario to be the basal excavation elevation of -5 m AOD and the characteristic groundwater table of 1.3 m AOD. The factor of safety calculated for this worst-case scenario is 1.20 which is considered acceptable for a short-term scenario before the placement of the engineered clay liner.

The factor of safety improves to 1.33 after the placement of clay liner and 1.38 after the placement of clay liner and drainage gravel. This is considered satisfactory.

3.7.2 Side Slope Sub-Grade Assessment

There are two cross sections considered for the side slope sub-grade analyses. Both sections are using a slope angle of 1(v):2.5(h) and a piezometric surface within the River Terrace Deposits.

The analysis of the Section A side slope sub-grade shows that the factor of safety against circular failure is 1.36. The analysis result also suggests that there will be no effect on the residential properties adjacent to the proposed extension.

For Section B, the factor of safety against circular failure is 1.37 which is greater than the minimum required 1.3. This is considered satisfactory.

3.7.3 Side Slopes Liner Assessment

The analysis of the side slope lining system for Sections A and B using undrained shear strength for clay liner with a slope gradient of 1(v):2.5(h) give factors of safety of 2.02 and 1.93 respectively. Its stability in the short-term is therefore considered satisfactory.

The analysis of the side slope lining system for Section A indicates that the factor of safety against circular failure with a fully functioning back-drainage system in the River Terrace Gravel is 1.59. When the side slope liner is analysed with a dysfunctional back-drainage system, the factor of safety reduces to 1.24 which is below the minimum required 1.3 and therefore could be considered unsatisfactory.

The analysis of the side slope lining system for Section B indicates that the factor of safety against circular failure with a fully functioning back-drainage system in the River Terrace Gravel is 1.57. When the side slope liner is analysed with a dysfunctional back-drainage system, the factor of safety reduces to 1.24 which is below the minimum required 1.3 and therefore could be considered unsatisfactory.

3.7.4 Waste Assessment

Temporary Waste Slopes

For the proposed 1(v):2(h) temporary waste slope in the extension cells, the factor of safety against circular failure is calculated as 1.39 for dry condition. The factor of safety remains unchanged with 1 m leachate level, and it will reduce to 1.35 with 2 m leachate level which is still satisfactory and is the height of the intercell bund. However, the factor of safety will drop to 1.26 and 1.13 respectively when the pore water pressure build-up is equivalent to r_u values of 0.1 and 0.2. This is considered unsatisfactory (see below).

Therefore, a dry waste mass with no leachate re-circulation is introduced into the analyses as 10 m and 20 m layers running parallel to the temporary waste slope. The factors of safety will increase to 1.18 and 1.33 respectively. The factor of safety of 10 m of dry waste slope is still unsatisfactory while 20 m of dry waste slope gives a satisfactory factor of safety. Therefore, leachate re-circulation shall only be carried out outside of 20 m of any open waste face.

Final Waste Slopes

The factor of safety against circular failure is calculated as 3.04 for dry condition with 2 m leachate level for the steepest and highest final waste slope. The factor of safety will slightly reduce to 2.82 and 2.59 respectively with pore water pressure build-up equivalent to r_u values of 0.1 and 0.2. Therefore, it is considered satisfactory.

3.7.5 Capping Assessment

Geomembrane Capping System

The geomembrane cap stability is analysed with different PSR values. The factors of safety against soil slippage for PSR values of 0 and 0.5 are 1.98 and 1.48, respectively. When a PSR value of 1.0 is applied, the factor of safety reduces to 1.05 which is less than 1.3 and considered unsatisfactory.

A further analysis is carried out to find out the threshold value of PSR which gives a satisfactory factor of safety for a geomembrane cap. The analysis result suggests that the maximum PSR value of 0.65 which gives a satisfactory factor of safety of 1.34. Therefore, the PSR value with the restoration soils should be kept below 0.65 to achieve a satisfactory factor of safety for the geomembrane capping system.

GCL Capping System

The GCL cap stability is analysed with different PSR values. The factors of safety against soil slippage for PSR values of 0 and 0.5 are 1.98 and 1.48, respectively. When a PSR value of 1.0 is applied, the factor of safety reduces to 1.05 which is less than 1.3 and considered unsatisfactory.

A further analysis is carried out to find out the threshold value of PSR which gives a satisfactory factor of safety for a GCL cap. The analysis result suggests that the maximum PSR value of 0.65 which gives a satisfactory factor of safety of 1.34. Therefore, the PSR value with the restoration soils should be kept below 0.65 to achieve a satisfactory factor of safety for the GCL capping system.

Clay Capping System

The clay cap stability is analysed with different PSR values. The factors of safety against soil slippage for PSR values of 0 and 0.5 are 1.82 and 1.36, respectively. When a PSR value of 1.0 is applied, the factor of safety reduces to 0.96. Therefore, the PSR value with the restoration soils should be kept below 0.5 to achieve a satisfactory factor of safety for the clay capping system.

3.7.6 Leachate Extraction System Assessment

Leachate Extraction Well Foundation

Calculations carried out to assess the bearing capacity of the clay liner beneath the leachate extraction well concrete bases indicate that the factors of safety for both total and effective stress are no less than 1.5, which are considered satisfactory. The calculated differential settlement for the leachate extraction well is 3.3 mm which is considered satisfactory.

Leachate Pipework Deflection

Calculations carried out to assess both 160 mm and 120 mm internal diameter for primary and secondary leachate pipework indicated the maximum deflections are 2.8% for both diameter pipes which are less than the maximum allowable deflection of 5% and therefore, it is considered satisfactory.

4.0 MONITORING

4.1 The Risk Based Monitoring Scheme

4.1.1 Basal Sub-grade and Liner Monitoring

The basal sub-grade and basal lining system shall be monitored during construction for any signs of water ingress. Basal heave calculation shall be reviewed on a cell-by-cell basis ahead of cell construction using cell specific groundwater levels.

4.1.2 Side Slopes Sub-grade and Liner Monitoring

The side slopes sub-grade system shall be monitored during construction for any signs of groundwater ingress. Site specific shear strength testing should be undertaken to obtain shear strength parameters for Made ground, River Terrace Deposits, Oxford Clay, and the clay liner verify that the materials on-site are in accordance with the parameters used within this assessment.

The back drain should be provided and monitored and maintained in a fully functioning condition.

4.1.3 Waste Mass Monitoring

It is recommended that all future temporary waste slopes are constructed at gradients of no steeper than 1(v):2(h). The waste slopes shall be monitored for any signs of instability immediately after any rainfall event.

Leachate levels shall be regularly monitored to ensure levels do not reach a point where the stability of the waste mass is threatened. The leachate level within each cell shall be maintained below 1.4 m above the base of the cell.

Leachate recirculation shall not be carried out within 20 m of any open waste face.

4.1.4 Capping System Monitoring

The capping system shall be monitored for signs of slumping in the restoration sols. Site specific restoration soil and interface shear strength should be undertaken to verify that the materials on site are in accordance with the parameters used within this assessment.

5.0 REFERENCES

1) Environment Agency (2003), 'Stability of Landfill Lining Systems', R & D Technical Report, Ref. P2-385/TR1

2) Golder Associates (UK) Ltd, 2022. Hydrogeological Risk Assessment - Eye Eastern Extension Landfill

3) Jones, DRV, Taylor D & Dixon N (1997), Shear Strength of Waste and its use in Landfill Stability Analysis. *Proc. Geoenvironmental Engineering conf.*, Yong & Thomas (eds.), Thomas Telford, London, pp. 343-350

4) Jones, DRV & Dixon N (1998), The stability of geosynthetic landfill lining systems, *Geotechnical Engineering of Landfills*, Thomas Telford, pp. 99-117

5) Golder Associates (UK) Ltd, 2008. Stability Risk Assessment - Eye Landfill Southern Extension

6) Environment Agency (2020), Landfill Operators Environmental Permits. Gov.uk. Design and build your landfill site - Landfill operators: environmental permits - Guidance - GOV.UK (www.gov.uk)

Signature Page

Golder WSP

Dr Bo Zhang
Senior Geotechnical Engineer

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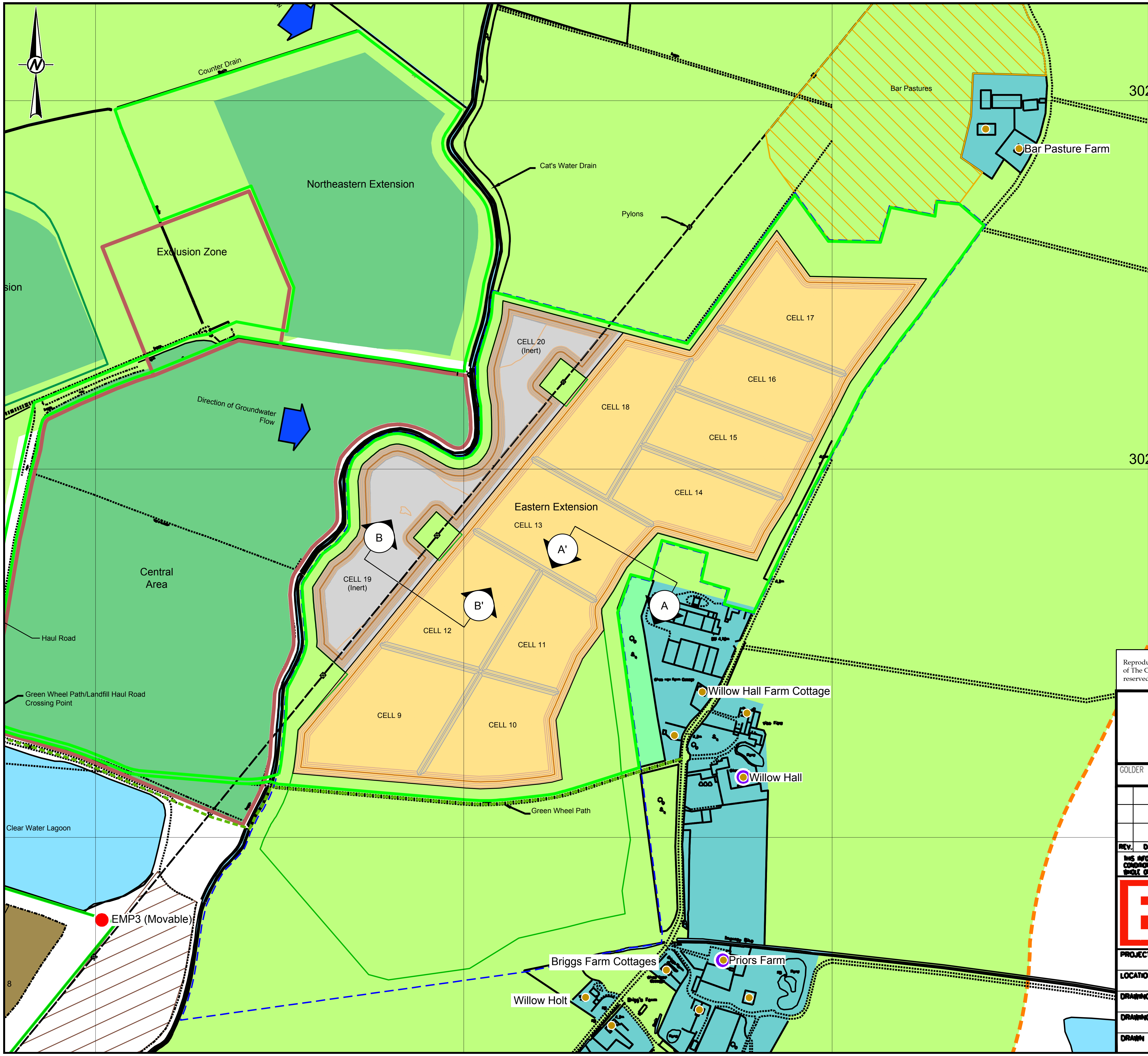
Drawings



Note:

1. All levels relative toNewlyn Ordnance Datum.
2. EMP1 South of the Site
3. EMP2 Northwest of the Site
4. EMP3 A non fixed location on the Site boundary which is downwind of the prevailing wind direction of the operational area at the time of sampling
5. EMP4 Entrance from highway

Reference:

1. Site survey provided by A&B Surveys, drawing ref.APA2003, dated January 2003, manually amended to east from Ordnance Survey Opendata.

Key:

SITE BOUNDARY, EYE LANDFILL
BIFFA REGULATED FACILITIES
THORY INERT LANDFILL
EYE LANDFILL, NORTHEASTERN EXTENSION AND SOUTHERN EXTENSION, EPR/BP3537PP/V010
AREA EXCLUDED FROM REGULATED FACILITIES
PYLONS AND OVERHEAD ELECTRICITY TRANSMISSION WIRES
WILDLIFE CORRIDOR
500m OFFSET FROM PROPOSED EASTERN EXTENSION
ENVIRONMENTAL MONITORING POINT

Sources:

CLOSED AND RESTORED LANDFILL
FILLED AND CAPPED LANDFILL
OPERATIONAL LANDFILL
FUTURE LANDFILL
LG GAS UTILISATION PLANT
LT LEACHATE STORAGE LAGOON FOR MISCANTHUS BEDS

Receptors:

DOMESTIC PROPERTY
COMMERCIAL/INDUSTRIAL PROPERTY
FARMLAND (ARABLE OR LIVESTOCK)
FOOTPATH, TRACK OR BRIDLEWAY
SCHEDULED ANCIENT MONUMENTS
LISTED BUILDINGS
RESIDENTIAL PROPERTIES

Pathways:

WATERCOURSES AND DRAINS
GROUNDWATER FLOW

wsp GOLDER

GOLDER FILE REF 1004-SR-0002	ENGINEER NW	REVIEWED BY CMcD

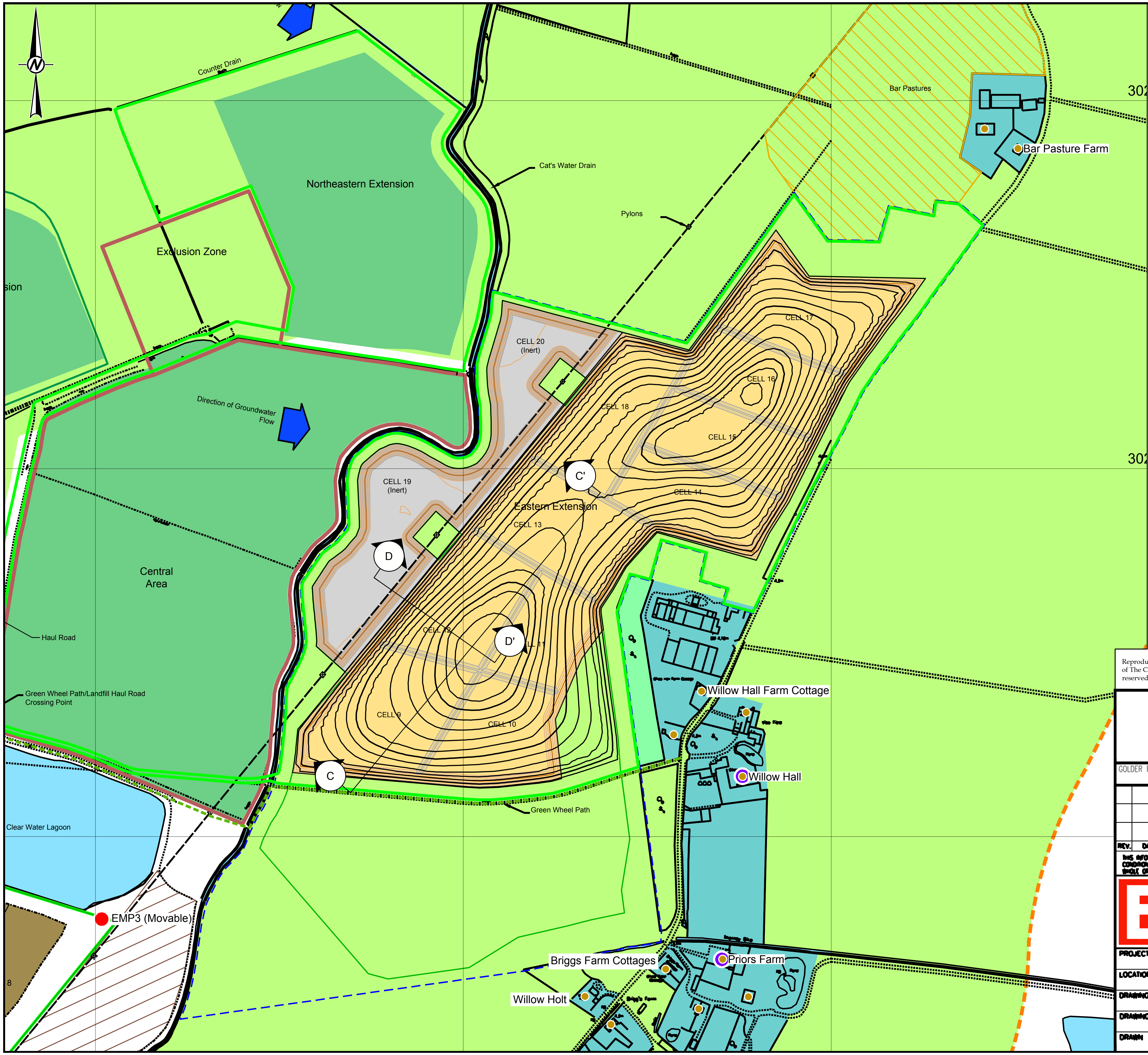
REV.	DATE	DRAWN	DESCRIPTION

Biffa

Biffa Waste Services Ltd
Redhill Landfill Site
Cormongers Lane,
Nutfield, Redhill
RH1 4ER
Tel. 01737 765042
Mob: 07921 386021

E-mail phil.mumford@biffa.co.uk

PROJECT	Eye Eastern Extension
LOCATION	Eye Landfill, Peterborough
DRAWING TITLE	Section A & B Location Plan
DRAWING No.	SRA1
COMPUTER REF.	E5239900
DRAWN	TS
DATE	21/01/2022
SCALE(S)	1:5,000



Note:

1. All levels relative toNewlyn Ordnance Datum.
2. EMP1 South of the Site
3. EMP2 Northwest of the Site
4. EMP3 A non fixed location on the Site boundary which is downwind of the prevailing wind direction of the operational area at the time of sampling
5. EMP4 Entrance from highway

Reference:

1. Site survey provided by A&B Surveys, drawing ref.APA2003, dated January 2003, manually amended to east from Ordnance Survey Opendata.

Key:

- SITE BOUNDARY, EYE LANDFILL
- BIFFA REGULATED FACILITIES
- THORY INERT LANDFILL
- EYE LANDFILL, NORTHEASTERN EXTENSION AND SOUTHERN EXTENSION, EPR/BP3537PP/V010
- AREA EXCLUDED FROM REGULATED FACILITIES
- PYLONS AND OVERHEAD ELECTRICITY TRANSMISSION WIRES
- WILDLIFE CORRIDOR
- 500m OFFSET FROM PROPOSED EASTERN EXTENSION
- ENVIRONMENTAL MONITORING POINT
- PRE-SETTLEMENT, PRE-RESTORATION CONTOURS

Sources:

- CLOSED AND RESTORED LANDFILL
- FILLED AND CAPPED LANDFILL
- OPERATIONAL LANDFILL
- FUTURE LANDFILL
- LG GAS UTILISATION PLANT
- LT LEACHATE STORAGE LAGOON FOR MISCANTHUS BEDS

Receptors:

- DOMESTIC PROPERTY
- COMMERCIAL/INDUSTRIAL PROPERTY
- FARMLAND (ARABLE OR LIVESTOCK)
- FOOTPATH, TRACK OR BRIDLEWAY
- SCHEDULED ANCIENT MONUMENTS
- LISTED BUILDINGS
- RESIDENTIAL PROPERTIES

Pathways:

- WATERCOURSES AND DRAINS
- GROUNDWATER FLOW

0 100 200
1:5,000 METRES

wsp GOLDER

GOLDER FILE REF 1004-SR-0003	ENGINEER NW	REVIEWED BY CMcD

REV.	DATE	DRAWN	DESCRIPTION

Biffa

Biffa Waste Services Ltd
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Cormongers Lane,
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RH1 4ER
Tel. 01737 765042
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E-mail phil.mumford@biffa.co.uk

PROJECT	Eye Eastern Extension
LOCATION	Eye Landfill, Peterborough
DRAWING TITLE	Section C & D Location Plan
DRAWING No.	SRA2
COMPUTER REF.	E52310000
DRAWN	TS
DATE	21/01/2022
SCALE(S)	1:5,000

APPENDIX SRA1

Basal Heave Analyses

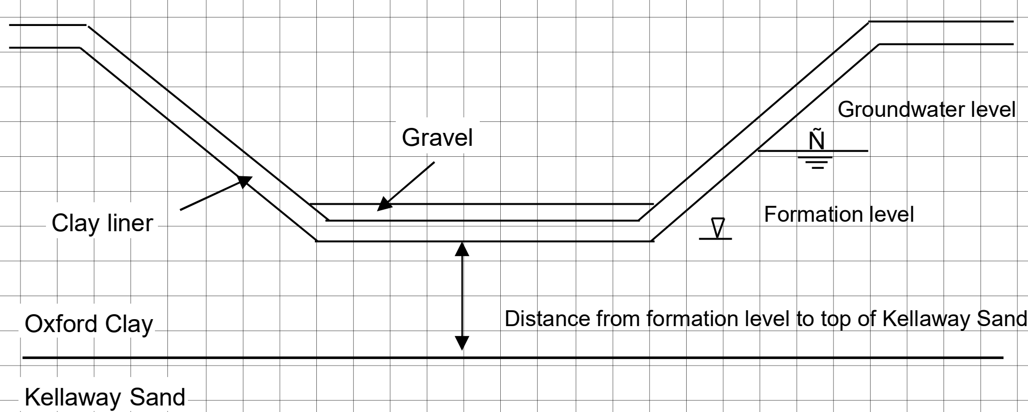
PROJECT	Biffa Eye Eastern Extension Stability Risk Assessment		
Job No. 21453458		Made By: WYH	Date: 20/01/2022
Ref.	Basal Heave	Checked: BZ	Sheet: 1
	Appendix 1	Reviewed BZ	of: 2

Basal Heave Assessment - Sitewide Assessment

Aim: To assess the potential for basal heave of the sub-grade.

Approach: Calculation of basal heave described by the Environment Agency R&D Technical Report P1-385/TR1.

Geometry:



Factor of safety against basal heave (FoS) calculated using formula FoS = s_v/u

where:

s_v = total vertical stress

u = pore water pressure

Assumptions:

Thickness of gravel layer =	0.3	m	Bottom of Oxford Clay level, D =	-14	m AOD
Thickness of clay liner =	1.0	m	Unit weight of clay, g_c =	20	kN/m^3
Formation level, F =	-5.0	m AOD	Unit weight of gravel, g_g =	22	kN/m^3
Groundwater level, G =	1.3	m AOD	Unit weight of water, g_{water} =	9.81	kN/m^3

1. Sub-grade stability

Factor of Safety against basal heave prior to liner placement:

$$\text{Factor of Safety} = \frac{(F - D)\gamma_c}{(G - D)\gamma_w} = 1.20$$

PROJECT	Biffa Eye Eastern Extension Stability Risk Assessment		
Job No.	21453458	Made By: WYH	Date: 20/01/2022
Ref.	Basal Heave	Checked: BZ	Sheet: 1
	Appendix 1	Reviewed BZ	of: 2

2. Basal liner stability with clay

Factor of Safety against basal heave after placement of clay liner:

Factor of Safety = $\frac{(F-D)\gamma_m + 1.0\gamma_c}{(G-D)\gamma_w}$ = 1.33

3. Basal liner stability when complete

Factor of Safety against basal heave after placement of clay liner and gravel:

Factor of Safety = $\frac{(F-D)\gamma_m + 1.0\gamma_c + 0.5\gamma_g}{(G-D)\gamma_w}$ = 1.38

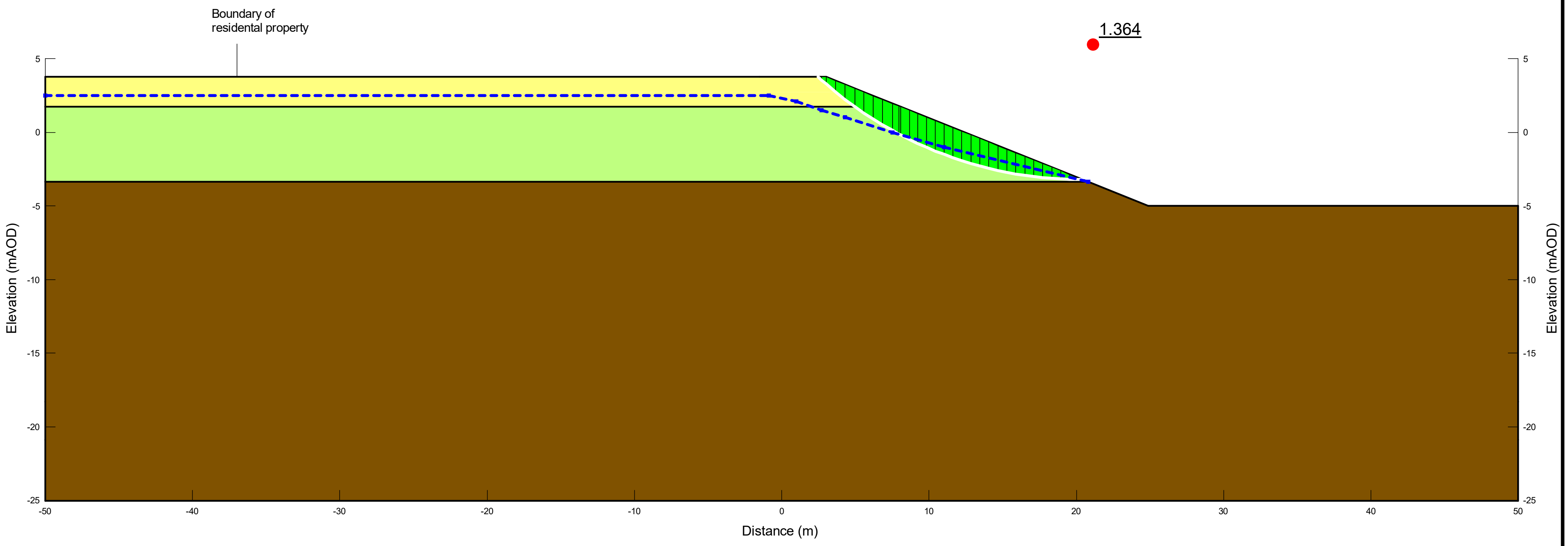
References:

Environment Agency, 2003
Stability of Landfill Lining Systems: Report No. 1 Literature Review
R&D Technical Report P1-385/TR1

APPENDIX SRA2

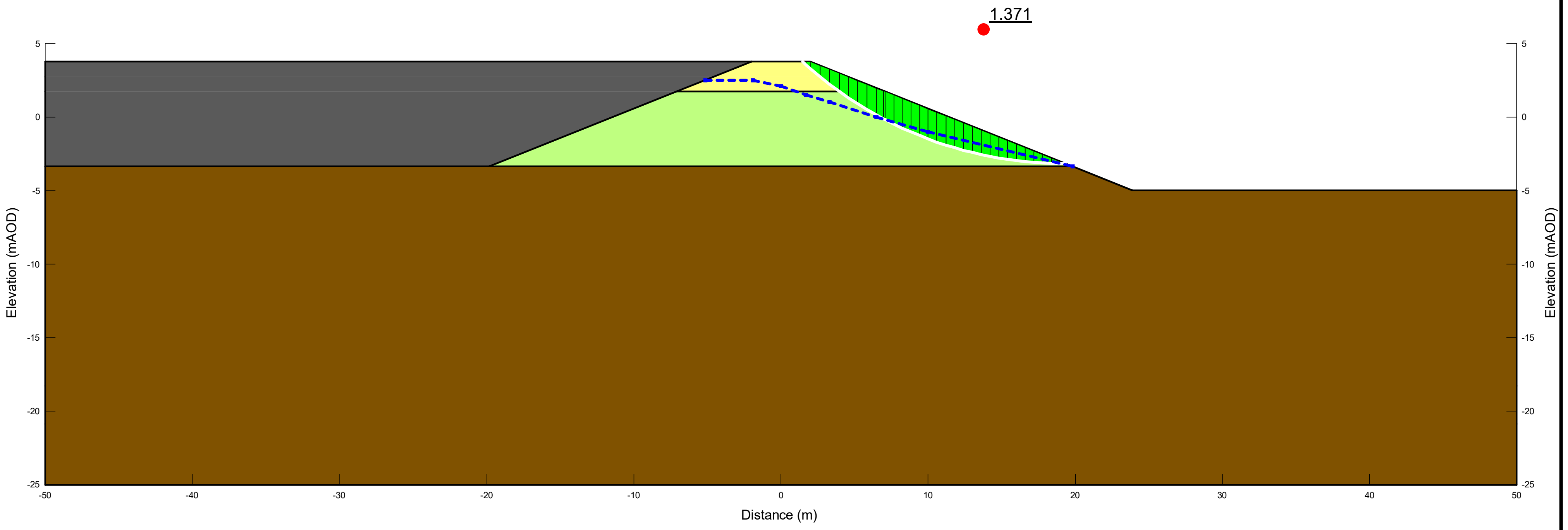
Side Slope Sub-Grade Analyses

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Made Ground	Mohr-Coulomb	18	0	30	1
	Oxford Clay	Mohr-Coulomb	20	30	26	1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1



Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Sub-Grade Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section A - Sub-grade	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Inert Fill	Mohr-Coulomb	16	0	28	1
	Made Ground	Mohr-Coulomb	18	0	30	1
	Oxford Clay	Mohr-Coulomb	20	30	26	1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1

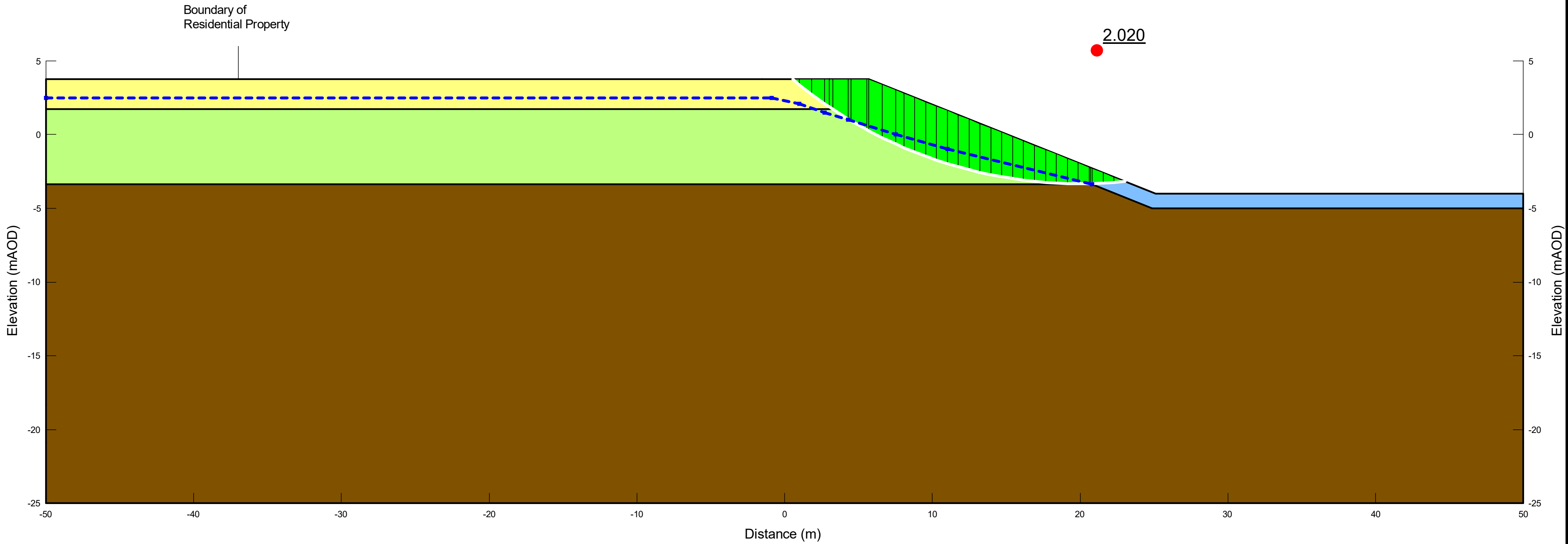


Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Sub-Grade Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section B - Sub-grade	Project Manager:	N White

APPENDIX SRA3

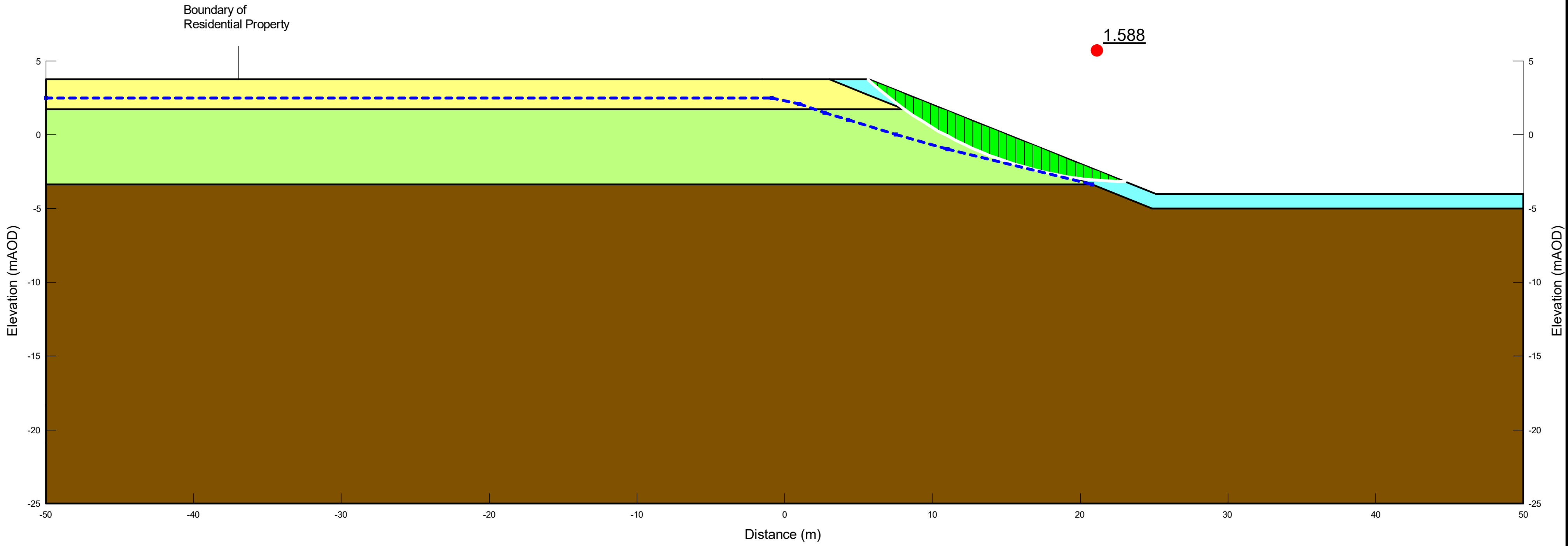
Side Slope Liner Analyses

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Cohesion (kPa)	Piezometric Line
	Clay Liner (Undrained)	Undrained (Phi=0)	19			50	1
	Made Ground	Mohr-Coulomb	18	0	30		1
	Oxford Clay	Mohr-Coulomb	20	30	26		1
	River Terrace Gravel	Mohr-Coulomb	20	0	30		1



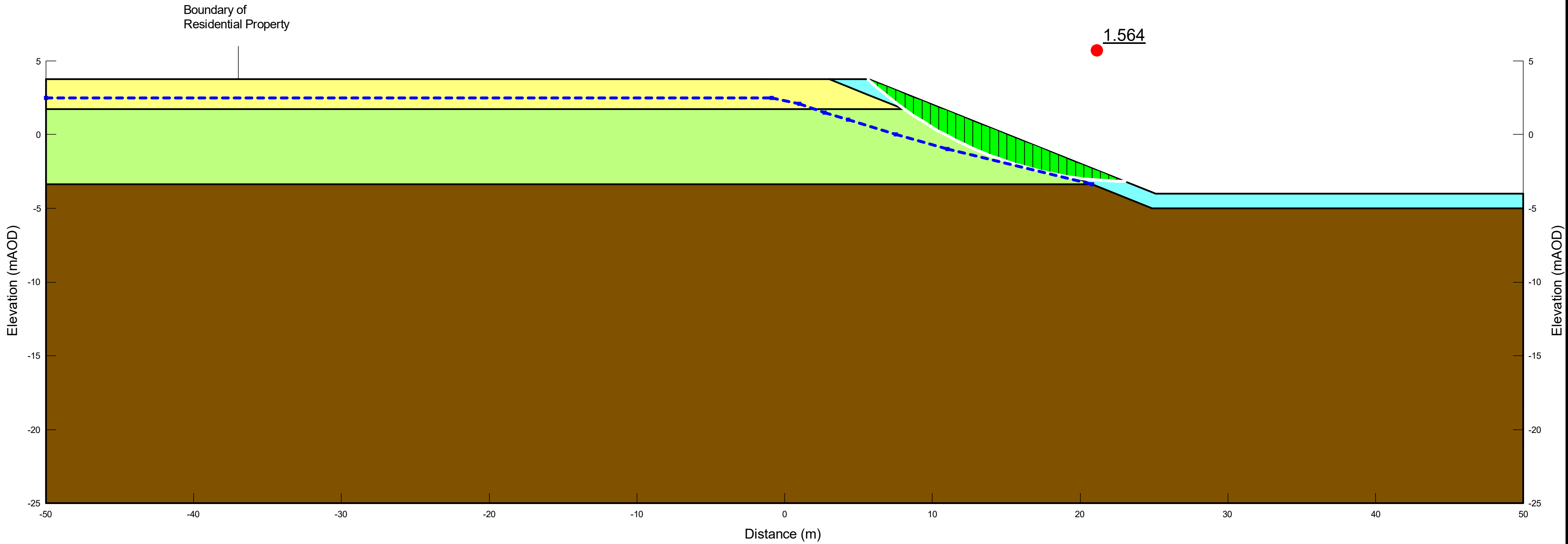
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section A - Liner 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26	1
	Made Ground	Mohr-Coulomb	18	0	30	1
	Oxford Clay	Mohr-Coulomb	20	30	26	1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1



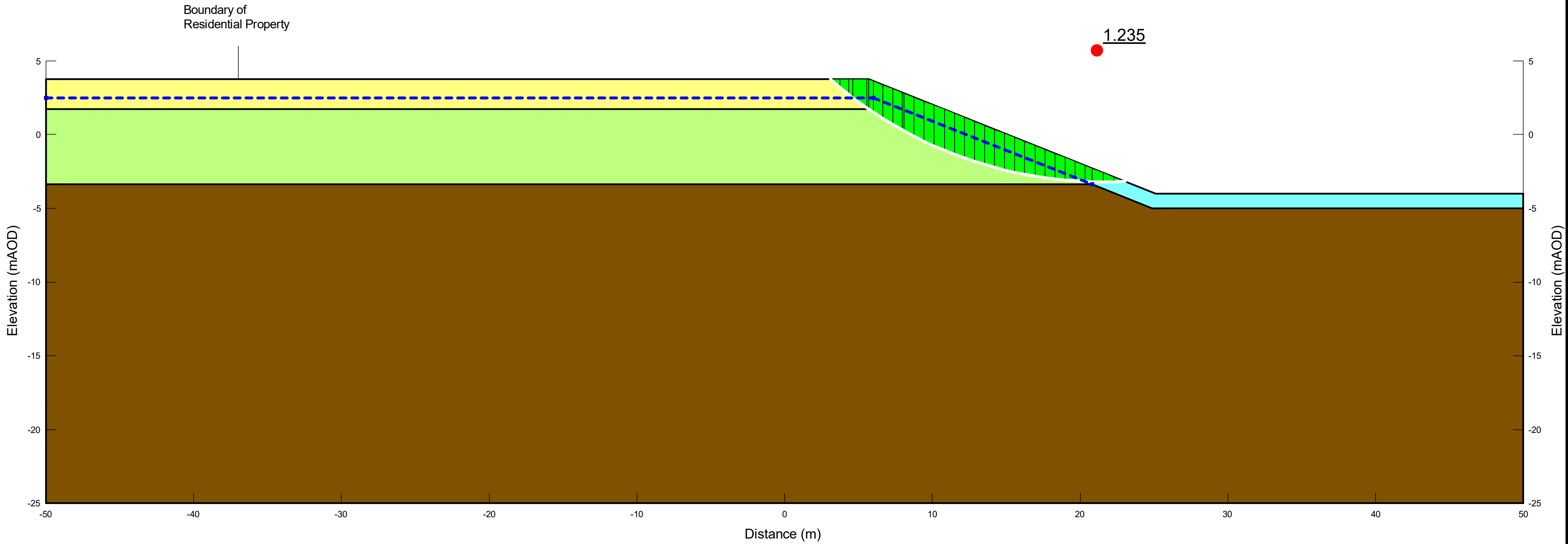
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section A - Liner 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26		0.1	Yes
	Made Ground	Mohr-Coulomb	18	0	30	1		No
	Oxford Clay	Mohr-Coulomb	20	30	26	1		No
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1		No



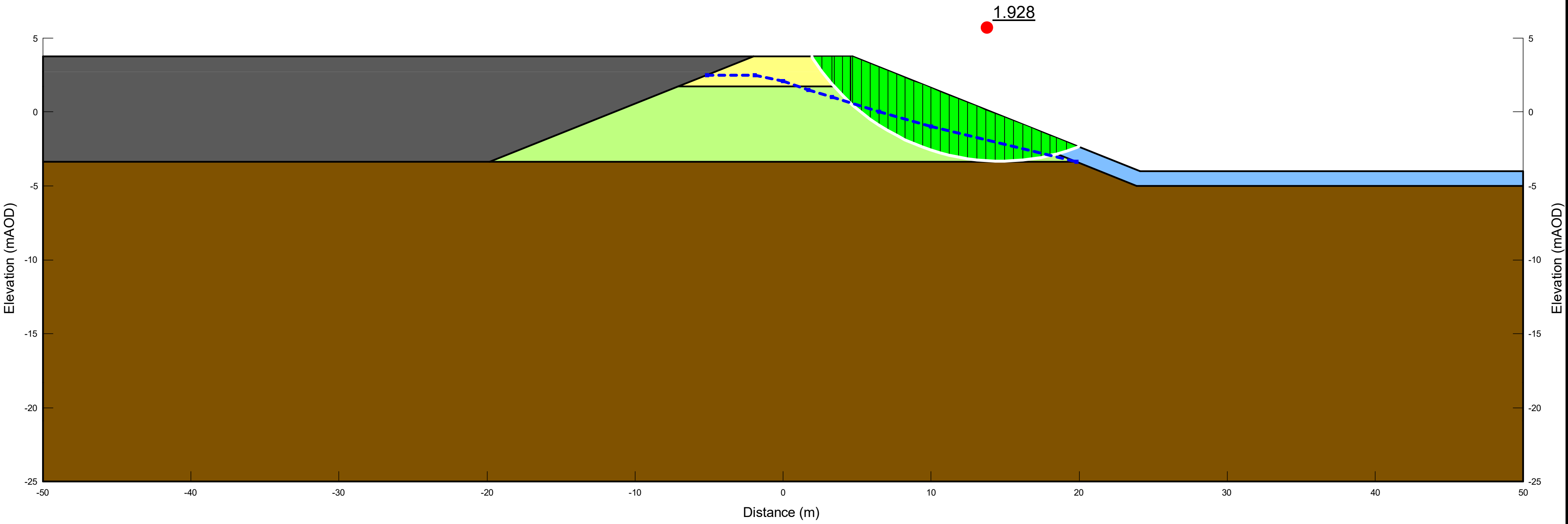
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section A - Liner 3	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26		0.1	Yes
	Made Ground	Mohr-Coulomb	18	0	30	1		No
	Oxford Clay	Mohr-Coulomb	20	30	26	1		No
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1		No



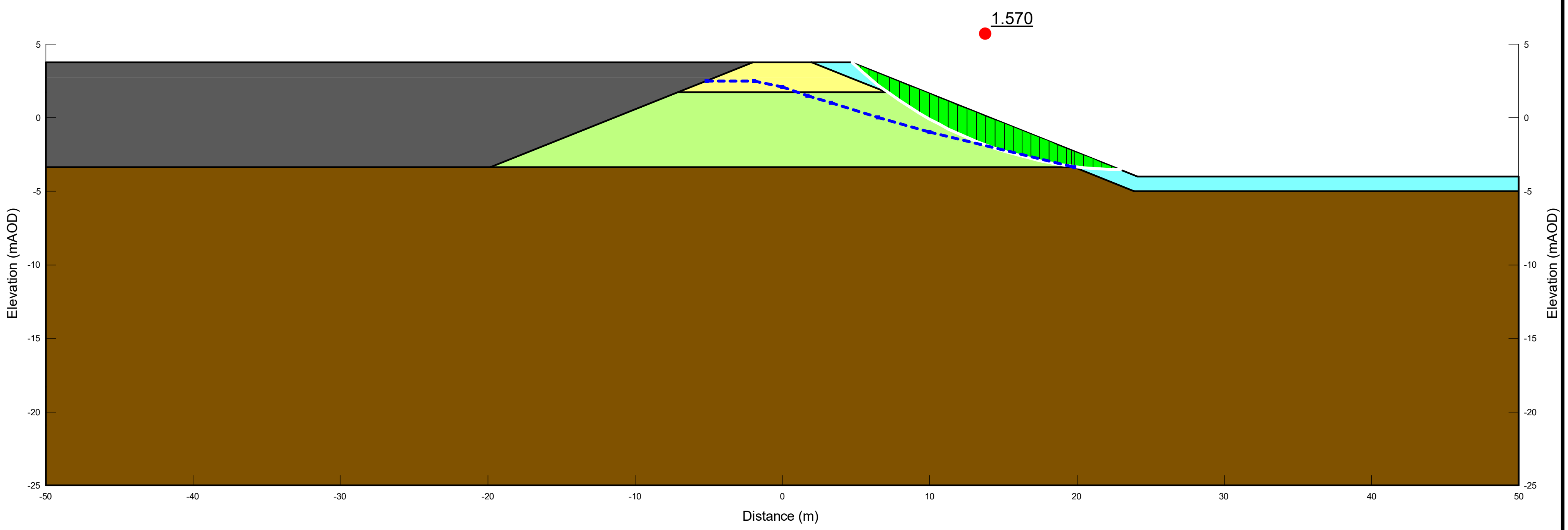
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section A - Liner 4	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Cohesion (kPa)	Piezometric Line
	Clay Liner (Undrained)	Undrained (Phi=0)	19				50	1
	Inert Fill	Mohr-Coulomb	16	0	28	0		1
	Made Ground	Mohr-Coulomb	18	0	30	0		1
	Oxford Clay	Mohr-Coulomb	20	30	26	0		1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	0		1



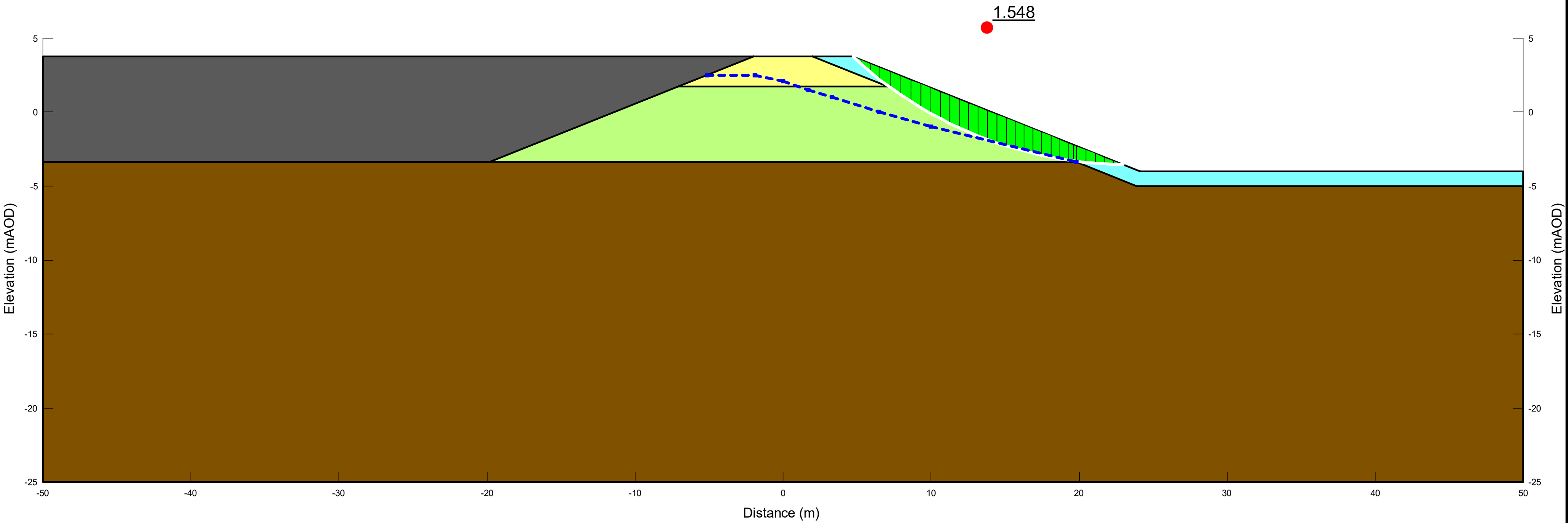
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section B - Liner 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26	0	1
	Inert Fill	Mohr-Coulomb	16	0	28	0	1
	Made Ground	Mohr-Coulomb	18	0	30	0	1
	Oxford Clay	Mohr-Coulomb	20	30	26	0	1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	0	1



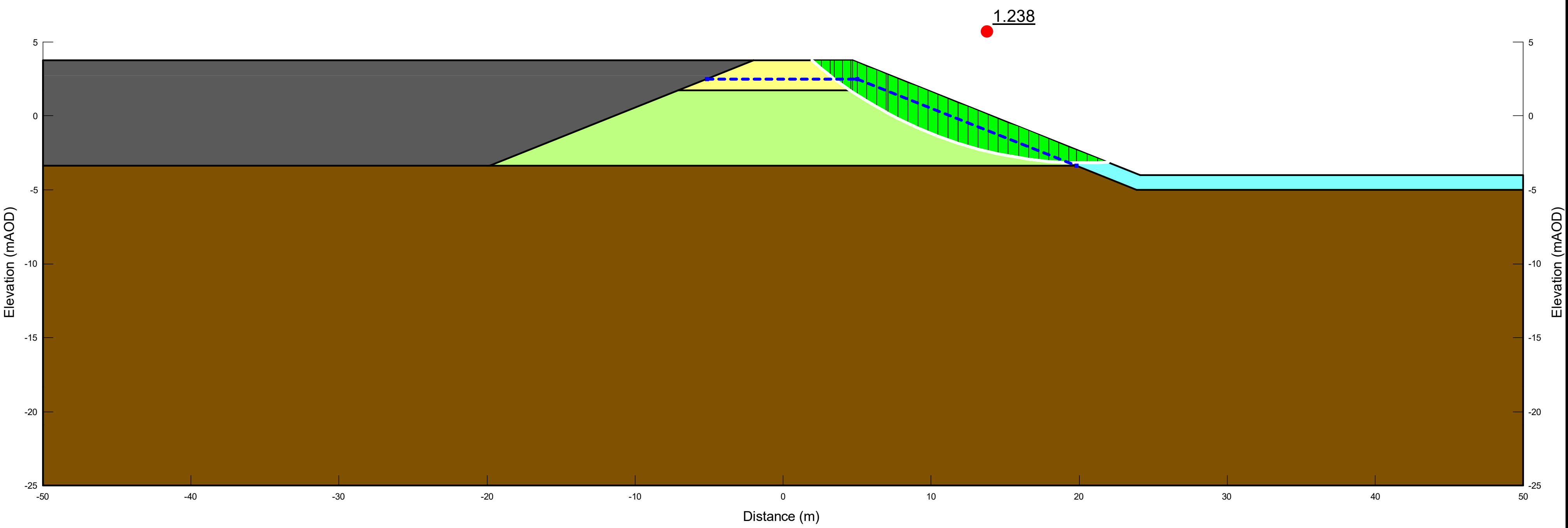
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section B - Liner 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26	0		0.1	Yes
	Inert Fill	Mohr-Coulomb	16	0	28	0	1		No
	Made Ground	Mohr-Coulomb	18	0	30	0	1		No
	Oxford Clay	Mohr-Coulomb	20	30	26	0	1		No
	River Terrace Gravel	Mohr-Coulomb	20	0	30	0	1		No



Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section B - Liner 3	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26	0		0.1	Yes
	Inert Fill	Mohr-Coulomb	16	0	28	0	1		No
	Made Ground	Mohr-Coulomb	18	0	30	0	1		No
	Oxford Clay	Mohr-Coulomb	20	30	26	0	1		No
	River Terrace Gravel	Mohr-Coulomb	20	0	30	0	1		No

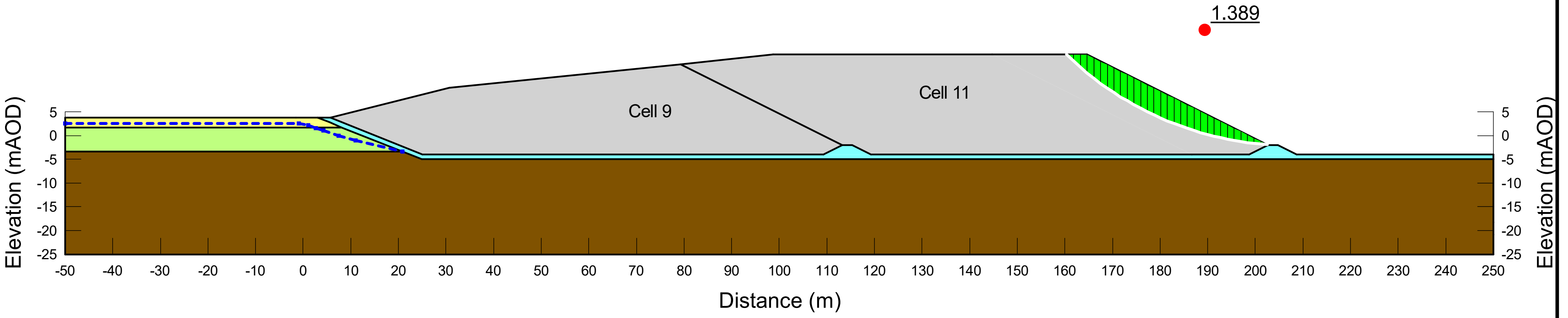


Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section B - Liner 4	Project Manager:	N White

APPENDIX SRA4

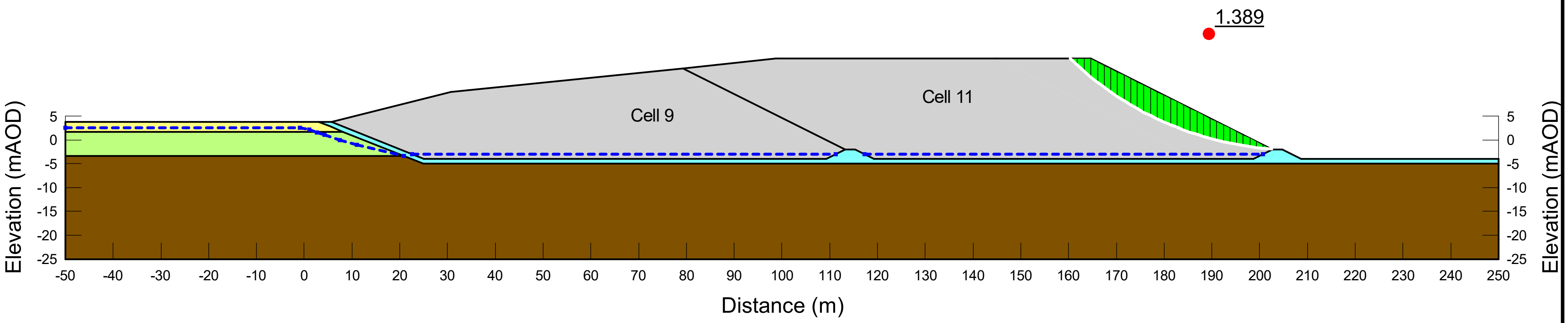
Temporary Waste Analyses

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26	1
	Made Ground	Mohr-Coulomb	18	0	30	1
	Oxford Clay	Mohr-Coulomb	20	30	26	1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1
	Waste_Cell 11	Mohr-Coulomb	10	5	25	1
	Waste_Cell 9	Mohr-Coulomb	10	5	25	1



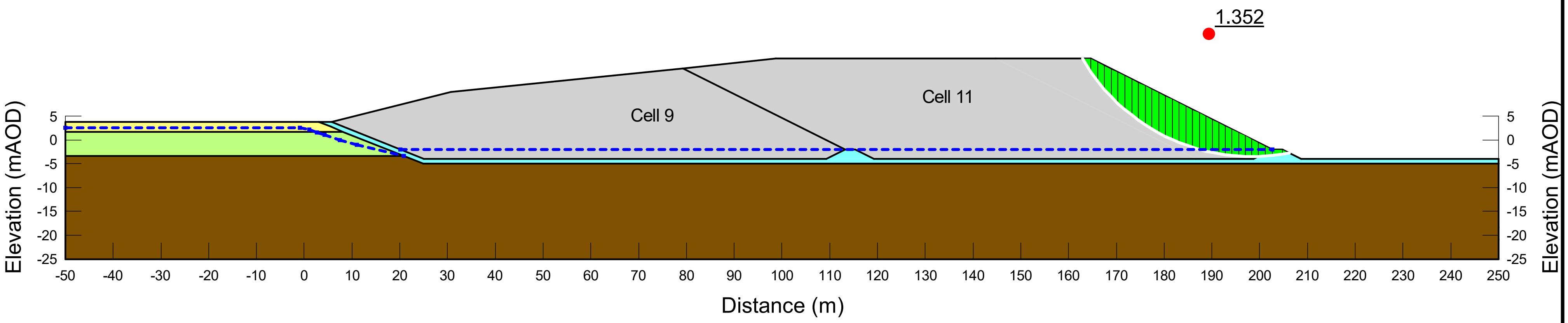
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section C - Temporary Waste 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26	1
	Made Ground	Mohr-Coulomb	18	0	30	1
	Oxford Clay	Mohr-Coulomb	20	30	26	1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1
	Waste_Cell 11	Mohr-Coulomb	10	5	25	3
	Waste_Cell 9	Mohr-Coulomb	10	5	25	2



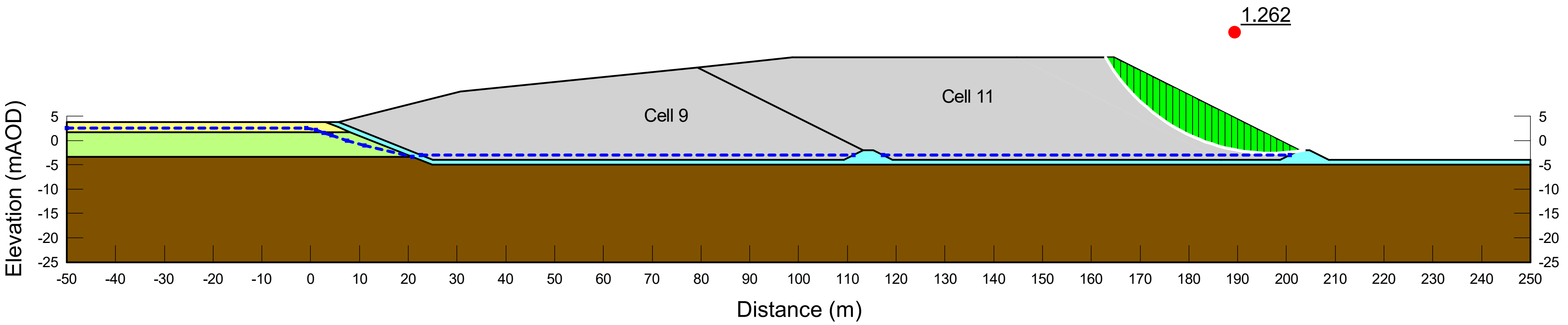
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section C - Temporary Waste 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26	1
	Made Ground	Mohr-Coulomb	18	0	30	1
	Oxford Clay	Mohr-Coulomb	20	30	26	1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1
	Waste_Cell 11	Mohr-Coulomb	10	5	25	2
	Waste_Cell 9	Mohr-Coulomb	10	5	25	1



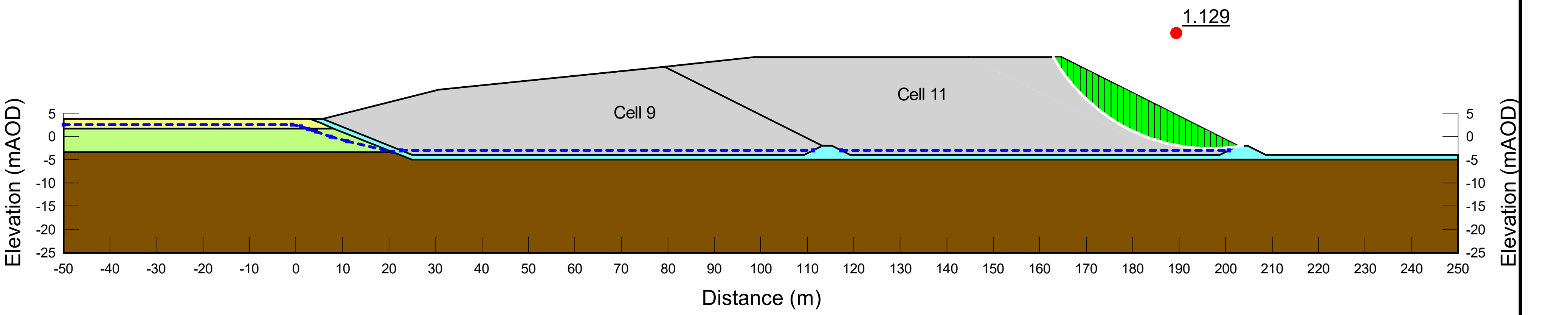
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section C - Temporary Waste 3	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26		1	No
	Made Ground	Mohr-Coulomb	18	0	30		1	No
	Oxford Clay	Mohr-Coulomb	20	30	26		1	No
	River Terrace Gravel	Mohr-Coulomb	20	0	30		1	No
	Waste_Cell 11	Mohr-Coulomb	10	5	25	0.1		Yes
	Waste_Cell 9	Mohr-Coulomb	10	5	25	0.1		Yes



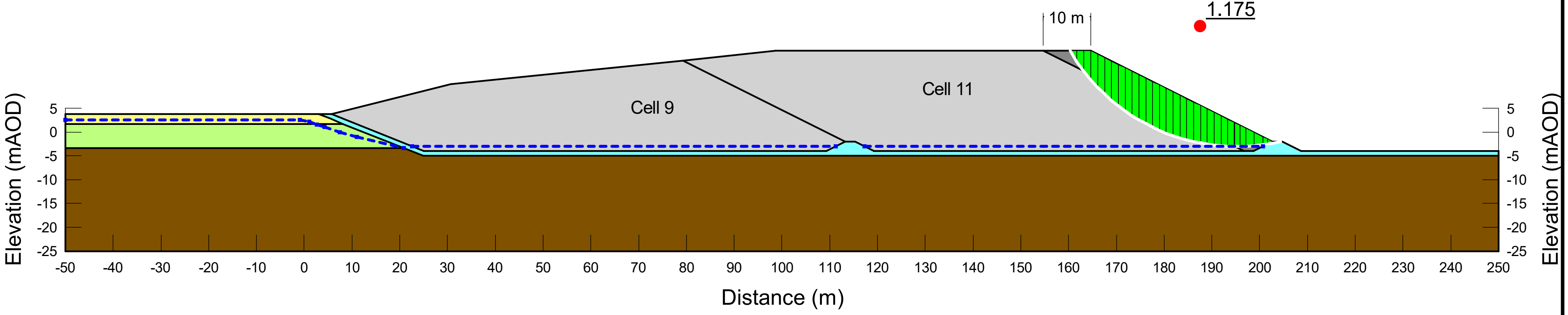
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section C - Temporary Waste 4	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26		1	No
	Made Ground	Mohr-Coulomb	18	0	30		1	No
	Oxford Clay	Mohr-Coulomb	20	30	26		1	No
	River Terrace Gravel	Mohr-Coulomb	20	0	30		1	No
	Waste_Cell 11	Mohr-Coulomb	10	5	25	0.2		Yes
	Waste_Cell 9	Mohr-Coulomb	10	5	25	0.2		Yes



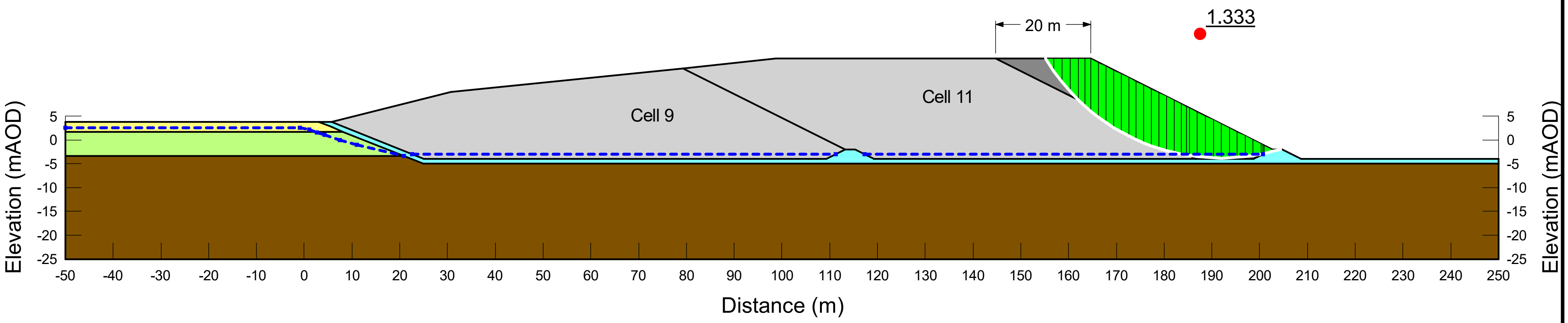
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section C - Temporary Waste 5	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26		1	No
	Dry Waste	Mohr-Coulomb	10	5	25		1	No
	Made Ground	Mohr-Coulomb	18	0	30		1	No
	Oxford Clay	Mohr-Coulomb	20	30	26		1	No
	River Terrace Gravel	Mohr-Coulomb	20	0	30		1	No
	Waste_Cell 11	Mohr-Coulomb	10	5	25	0.2		Yes
	Waste_Cell 9	Mohr-Coulomb	10	5	25	0.2		Yes



Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section C - Tempoaray Waste 6	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26		1	No
	Dry Waste	Mohr-Coulomb	10	5	25		1	No
	Made Ground	Mohr-Coulomb	18	0	30		1	No
	Oxford Clay	Mohr-Coulomb	20	30	26		1	No
	River Terrace Gravel	Mohr-Coulomb	20	0	30		1	No
	Waste_Cell 11	Mohr-Coulomb	10	5	25	0.2		Yes
	Waste_Cell 9	Mohr-Coulomb	10	5	25	0.2		Yes

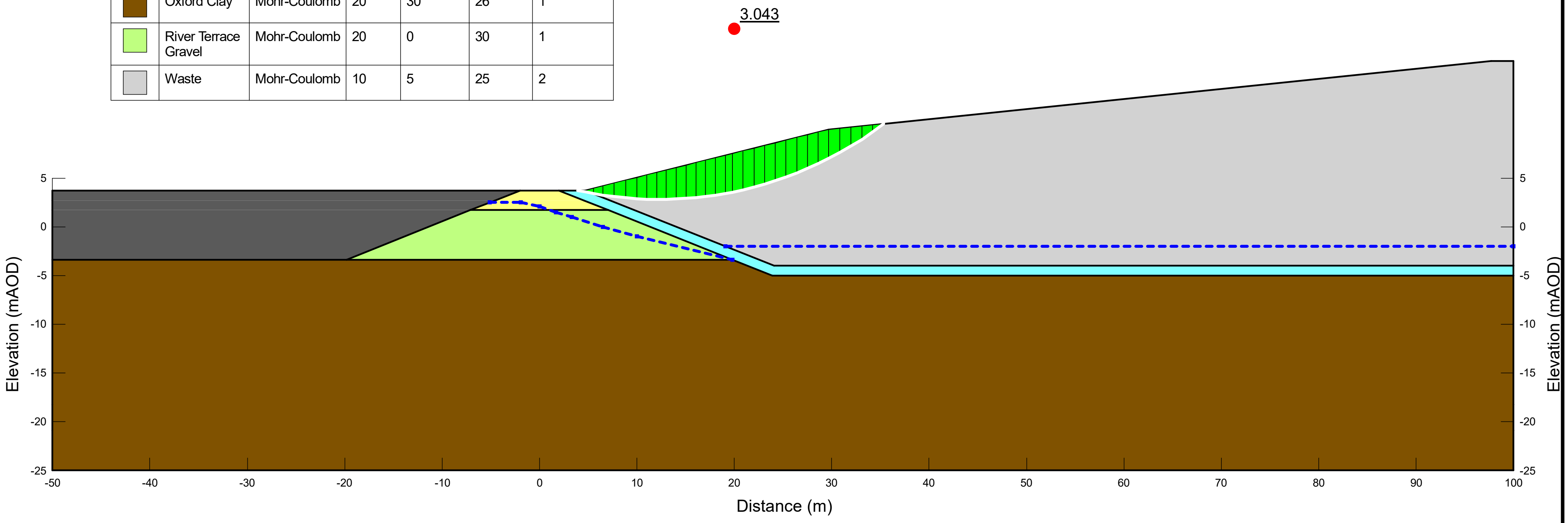


Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section C - Tempoaray Waste 7	Project Manager:	N White

APPENDIX SRA5

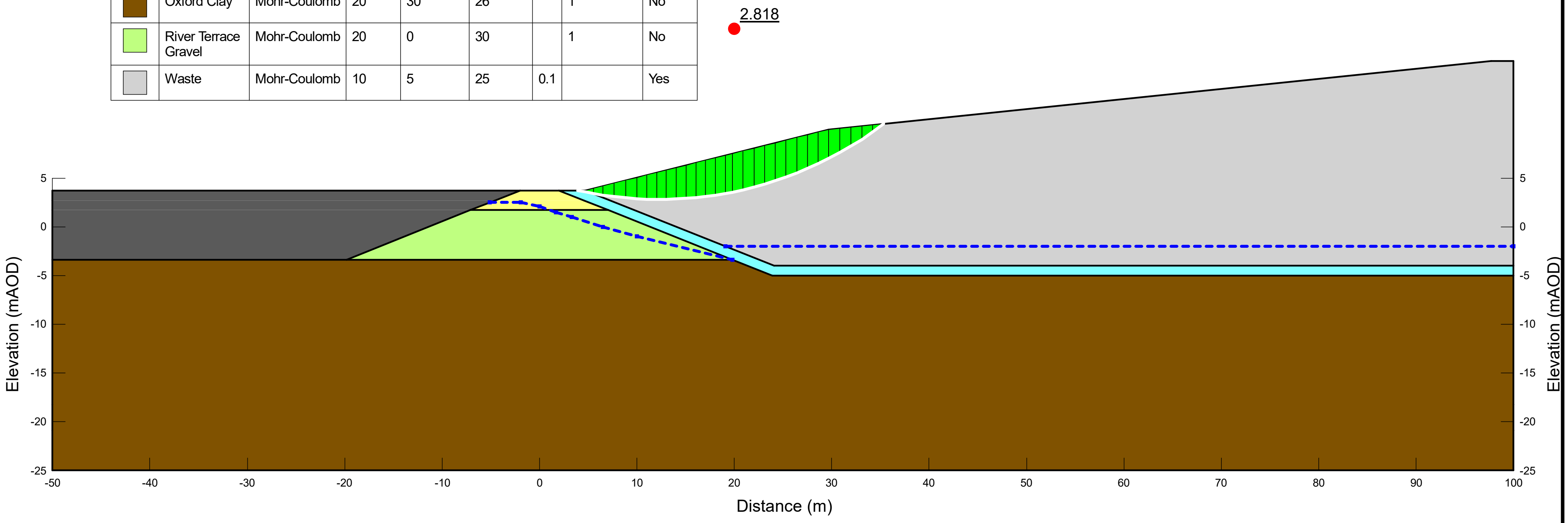
Final Waste Analyses

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26	1
	Inert Fill	Mohr-Coulomb	16	0	28	1
	Made Ground	Mohr-Coulomb	18	0	30	1
	Oxford Clay	Mohr-Coulomb	20	30	26	1
	River Terrace Gravel	Mohr-Coulomb	20	0	30	1
	Waste	Mohr-Coulomb	10	5	25	2



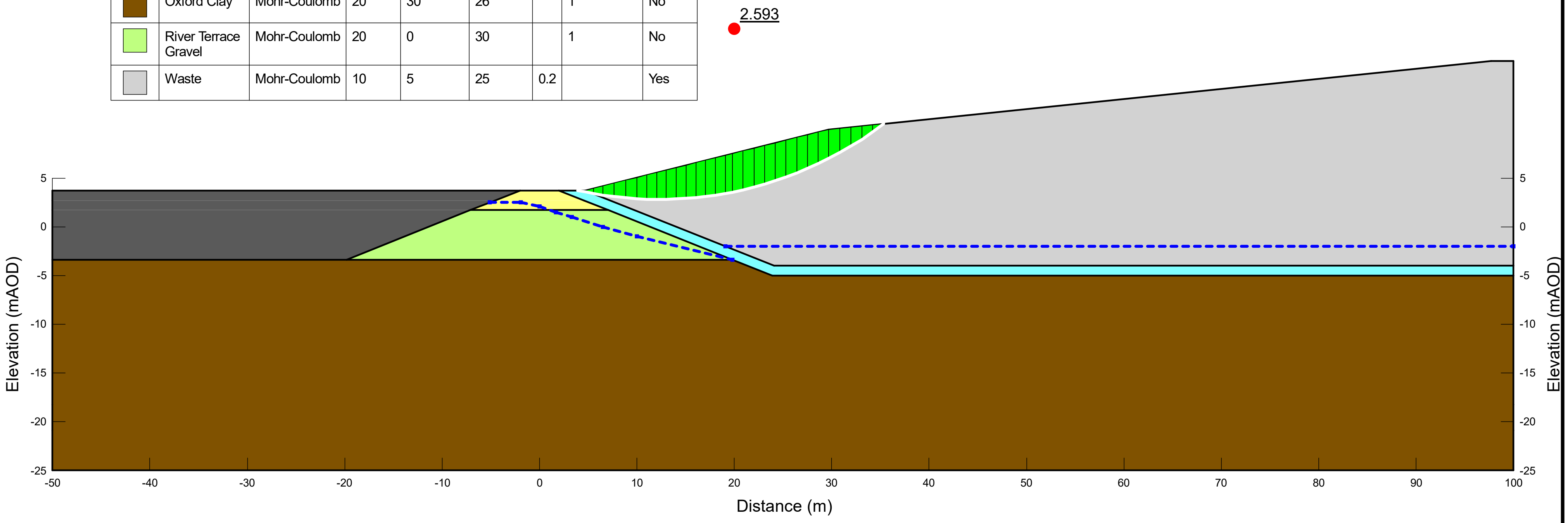
Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Final Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section D - Final Waste 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26		1	No
	Inert Fill	Mohr-Coulomb	16	0	28		1	No
	Made Ground	Mohr-Coulomb	18	0	30		1	No
	Oxford Clay	Mohr-Coulomb	20	30	26		1	No
	River Terrace Gravel	Mohr-Coulomb	20	0	30		1	No
	Waste	Mohr-Coulomb	10	5	25	0.1		Yes



Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Final Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section D - Final Waste 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Clay Liner (Drained)	Mohr-Coulomb	19	2	26		1	No
	Inert Fill	Mohr-Coulomb	16	0	28		1	No
	Made Ground	Mohr-Coulomb	18	0	30		1	No
	Oxford Clay	Mohr-Coulomb	20	30	26		1	No
	River Terrace Gravel	Mohr-Coulomb	20	0	30		1	No
	Waste	Mohr-Coulomb	10	5	25	0.2		Yes



Client:	Biffa Waste Services Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Project:	Biffa Eye Eastern Extension	Filename:	Final Waste Analysis.gsz	Reviewer:	Dr B Zhang
Report Title:	Stability Risk Assessment	Analysis Ref:	Section D - Final Waste 3	Project Manager:	N White

APPENDIX SRA6

Geomembrane Capping Analyses

INTRODUCTION

The stability of the cover soils and the integrity of the geosynthetic layers has been assessed for the LLDPE geomembrane capping system. Analysis has been carried out for selected steepest and heighest section.

Stability

The effect of a partially and fully saturated cover soil layer has been assessed using the method proposed by Jones & Dixon (1998). The normal operating conditions have been modelled using dry cover soils and the worst case conditions of fully saturated cover soils have been analysed. The water pressures acting on the system have been modelled using a Parallel Submergence Ratio (PSR). PSR = 0 for dry conditions, PSR = 0.5 for a partially saturated condtions and PSR = 1 for a fully saturated cover soil with seepage flow.

Integrity

The integrity of the geosynthetic liner have been assessed by considering the shear strength developed above and below the geosynthetic, and comparing this to the material strength.

Geosynthetic

Analyses has been carried out assuming the lining system comprises a 1 mm LLDPE geomembrane with an overlying geocomposite drainage layer, and 1.0 m of restoration soil.

The parameters used in the analysis have been derived from a summary of the technical literature on interface shear strengths reported by Jones & Dixon (1998) in conjunction with Golder's in-house expeprience. Based on this and our experience of geosynthetic interfaces, a conservative assessment of the interface shear strength parameters is:

•	Cover soils	/	Geotextile	$\alpha_{p'}$	=	0	kPa	δ_p'	=	24	Deg.
•	Geotextile	/	Textured GM	$\alpha_{p'}$	=	0	kPa	δ_p'	=	26	Deg.
•	Textured GM	/	Blinding Layer	$\alpha_{p'}$	=	0	kPa	δ_p'	=	24	Deg.

These values should be confirmed by site-specific shear strength testing. The values given above are all peak shear strengh values.

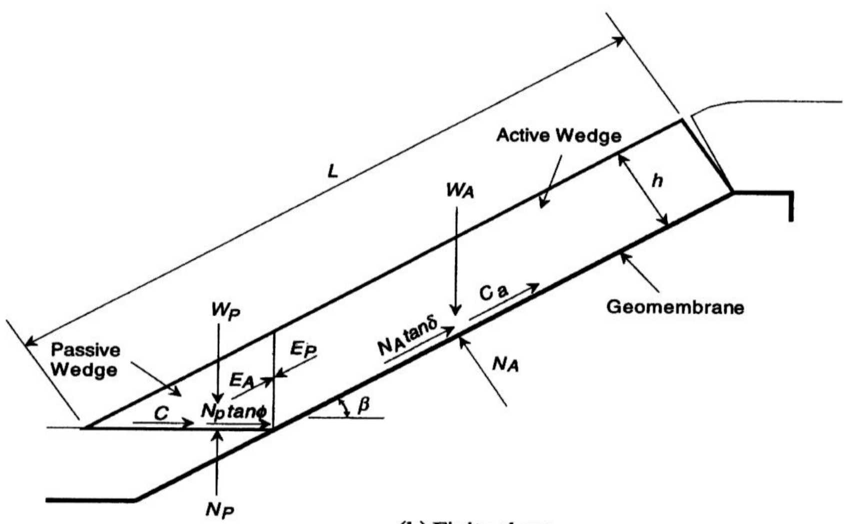
PROJECT	Biffa Eye Eastern Extension Stability Assessment		
Job No.	21453458	Made By: WYH	Date: 22/01/2022
Ref.	Appendix 6	Checked: BZ	Sheet: 2
		Reviewed: BZ	of: 9

Section A **PSR** = 0.00

Aim: To assess the stability and integrity of the geosynthetic capping system.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geocomposite friction angle, δ_1	24	Deg.
Cover Soils/Geocomposite cohesion intercept, α_1	0	kPa
Geocomposite/GM friction angle, δ_2	26	Deg.
Geocomposite/GM cohesion intercept, α_2	0	kPa
GM/Blinding layer, δ_3	24	kPa
GM/Blinding layer, α_3	0	Deg.
Parallel submergence ratio, PSR	0.00	
Geosynthetic tensile strengths:		
Geotextile	10	kN/m
1mm LLDPE Geomembrane	11	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m	
Thickness of water, h_w	0	m	
Weight of active wedge, W_A	408.0841	kN	
Weight of passive wedge, W_P	38.34098	kN	
Pore pressure perp. to slope, U_n	0	kN	
Pore pressure in interwedge surface, U_h	0	kN	
Force normal to active wedge, N_A	395.9623	kN	
Vert pp on passive wedge, U_V	0	kN	
a	95.79194		
b	-200.073		
c	19.8877		
Factor of Safety against cover soils sliding			**1.98**

2. Integrity of Geosynthetics

(i) Geocomposite

Mobilised shear stress at upper interface	97.20753	kN	
Shear strength at lower interface	211.2684	kN	
Tension developed in the GT	0	kN	
Tensile strength of the GT	10	kN	
Factor of Safety against rupture			**Infinite**

(ii) GCL

Shear strength at upper surface	211.2684	kN	
Mobilised shear stress at upper interface	97.20753	kN	
Shear strength at lower interface	192.8572	kN	
Tension developed in the GM	0	kN	
Tensile strength of the GM	11	kN	
Factor of Safety against rupture			**Infinite**

PROJECT	Biffa Eye Eastern Extension Stability Assessment		
Job No.	21453458	Made By: WYH	Date: 22/01/2022
Ref.	Appendix 6	Checked: BZ	Sheet: 4
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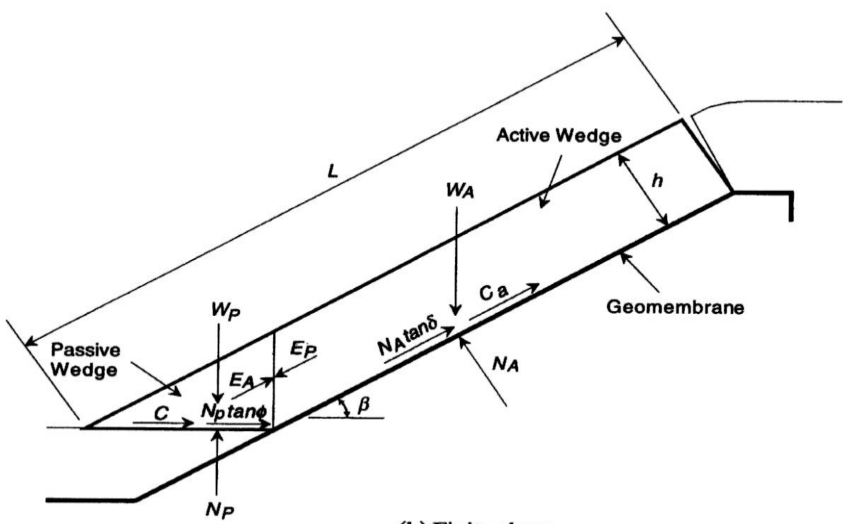
Section **A** **PSR** = 0.50

Aim: To assess the stability and integrity of the geosynthetic capping system.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geocomposite friction angle, δ_1	24	Deg.
Cover Soils/Geocomposite cohesion intercept, α_1	0	kPa
Geocomposite/GM friction angle, δ_2	26	Deg.
Geocomposite/GM cohesion intercept, α_2	0	kPa
GM/Blinding layer, δ_3	24	kPa
GM/Blinding layer, α_3	0	Deg.
Parallel submergence ratio, PSR	0.50	
Geosynthetic tensile strengths:		
Geotextile	10	kN/m
1mm LLDPE Geomembrane	11	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m
Thickness of water, h_w	0.5	m
Weight of active wedge, W_A	443.6886	kN
Weight of passive wedge, W_P	39.93852	kN
Pore pressure perp. to slope, U_n	115.1565	kN
Pore pressure in interwedge surface, U_h	1.25	kN
Force normal to active wedge, N_A	315.6551	kN
Vert pp on passive wedge, U_V	5.013476	kN
a	104.2228	
b	-164.622	
c	15.85418	

Factor of Safety against cover soils sliding **1.48**

2. Integrity of Geosynthetics

(i) Geotextile

Mobilised shear stress at upper interface	141.5033	kN
Shear strength at lower interface	228.8741	kN
Tension developed in the GT	0	kN
Tensile strength of the GT	10	kN

Factor of Safety against rupture **Infinite**

(ii) Geomembrane

Shear strength at upper surface	228.8741	kN
Mobilised shear stress at upper interface	141.5033	kN
Shear strength at lower interface	208.9286	kN
Tension developed in the GM	0	kN
Tensile strength of the GM	11	kN

Factor of Safety against rupture **Infinite**

PROJECT	Biffa Eye Eastern Extension Stability Assessment				
Job No.	21453458	Made By: WYH		Date:	22/01/2022
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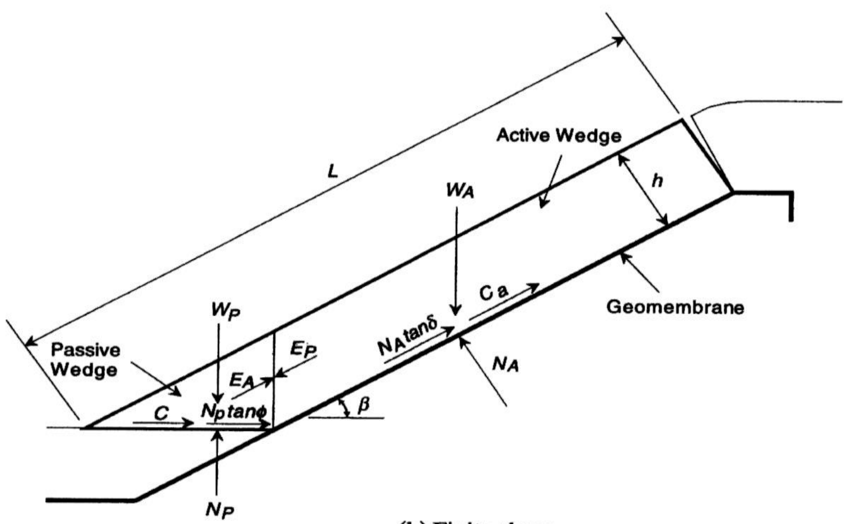
Section **A** **PSR** = 1.00

Aim: To assess the stability and integrity of the geosynthetic capping system.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geocomposite friction angle, δ_1	24	Deg.
Cover Soils/Geocomposite cohesion intercept, α_1	0	kPa
Geocomposite/GM friction angle, δ_2	26	Deg.
Geocomposite/GM cohesion intercept, α_2	0	kPa
GM/Blinding layer, δ_3	24	kPa
GM/Blinding layer, α_3	0	Deg.
Parallel submergence ratio, PSR	1.00	
Geosynthetic tensile strengths:		
Geotextile	10	kN/m
1mm LLDPE Geomembrane	11	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m	
Thickness of water, h_w	1	m	
Weight of active wedge, W_A	476.0981	kN	
Weight of passive wedge, W_P	44.73114	kN	
Pore pressure perp. to slope, U_n	219.979	kN	
Pore pressure in interwedge surface, U_h	5	kN	
Force normal to active wedge, N_A	243.1865	kN	
Vert pp on passive wedge, U_V	20.0539	kN	
a	112.0499		
b	-129.011		
c	12.21435		
	Factor of Safety against cover soils sliding		**1.05**

2. Integrity of Geosynthetics

(i) Geotextile

Mobilised shear stress at upper interface	214.8421	kN	
Shear strength at lower interface	246.4798	kN	
Tension developed in the GT	0	kN	
Tensile strength of the GT	10	kN	
	Factor of Safety against rupture		**Infinite**

(ii) Geomembrane

Shear strength at upper surface	246.4798	kN	
Mobilised shear stress at upper interface	214.8421	kN	
Shear strength at lower interface	225.0001	kN	
Tension developed in the GM	0	kN	
Tensile strength of the GM	11	kN	
	Factor of Safety against rupture		**Infinite**

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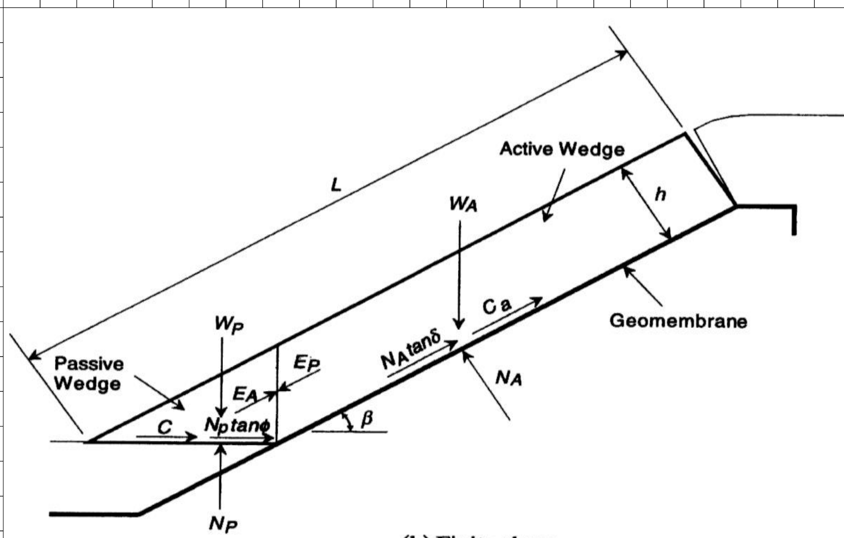
Section **A** **PSR** = 0.65

Aim: To assess the stability and integrity of the geosynthetic capping system.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geocomposite friction angle, δ_1	24	Deg.
Cover Soils/Geocomposite cohesion intercept, α_1	0	kPa
Geocomposite/GM friction angle, δ_2	26	Deg.
Geocomposite/GM cohesion intercept, α_2	0	kPa
GM/Blinding layer, δ_3	24	kPa
GM/Blinding layer, α_3	0	Deg.
Parallel submergence ratio, PSR	0.65	
Geosynthetic tensile strengths:		
Geotextile	10	kN/m
1mm LLDPE Geomembrane	11	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m	
Thickness of water, h_w	0.65	m	
Weight of active wedge, W_A	453.747	kN	
Weight of passive wedge, W_P	41.04082	kN	
Pore pressure perp. to slope, U_n	147.6883	kN	
Pore pressure in interwedge surface, U_h	2.1125	kN	
Force normal to active wedge, N_A	293.0915	kN	
Vert pp on passive wedge, U_V	8.472775	kN	
a	106.6343		
b	-153.955		
c	14.72089		
	Factor of Safety against cover soils sliding		**1.34**

2. Integrity of Geosynthetics

(i) Geotextile

Mobilised shear stress at upper interface	159.4187	kN	
Shear strength at lower interface	234.1558	kN	
Tension developed in the GT	0	kN	
Tensile strength of the GT	10	kN	
	Factor of Safety against rupture		**Infinite**

(ii) Geomembrane

Shear strength at upper surface	234.1558	kN	
Mobilised shear stress at upper interface	159.4187	kN	
Shear strength at lower interface	213.7501	kN	
Tension developed in the GM	0	kN	
Tensile strength of the GM	11	kN	
	Factor of Safety against rupture		**Infinite**

APPENDIX SRA7

GCL Capping Analyses

INTRODUCTION

The stability of the cover soils and the integrity of the geosynthetic layers has been assessed for the GCL capping system. Analysis has been carried out for the selected steepest and highest section.

Stability

The effect of a partially and fully saturated cover soil layer has been assessed using the method proposed by Jones & Dixon (1998). The normal operating conditions have been modelled using dry cover soils and the worst case conditions of fully saturated cover soils have been analysed. The water pressures acting on the system have been modelled using a Parallel Submergence Ratio (PSR). PSR = 0 for dry conditions, PSR = 0.5 for a partially saturated condtions and PSR = 1 for a fully saturated cover soil with seepage flow.

Integrity

The integrity of the geosynthetic liner have been assessed by considering the shear strength developed above and below the geosynthetic, and comparing this to the material strength.

Geosynthetic

Analyses has been carried out assuming the lining system comprises a GCL liner with 1.0 m of restoration soil.

The parameters used in the analysis have been derived from a summary of the technical literature on interface shear strengths reported by Jones & Dixon (1998). A summary of the geotextile interfaces is given in the reference pages. Based on this and our experience of geosynthetic interfaces, a conservative assessment of the interface shear strength parameters is:

•	Cover soils	/	GCL	$\alpha_{p'}$	=	0	kPa	δ_p'	=	24	Deg.
•	GCL	/	Blinding layer	$\alpha_{p'}$	=	0	kPa	δ_p'	=	24	Deg.

These values should be confirmed by site-specific shear strength testing. The values given above are all peak shear strengh values.

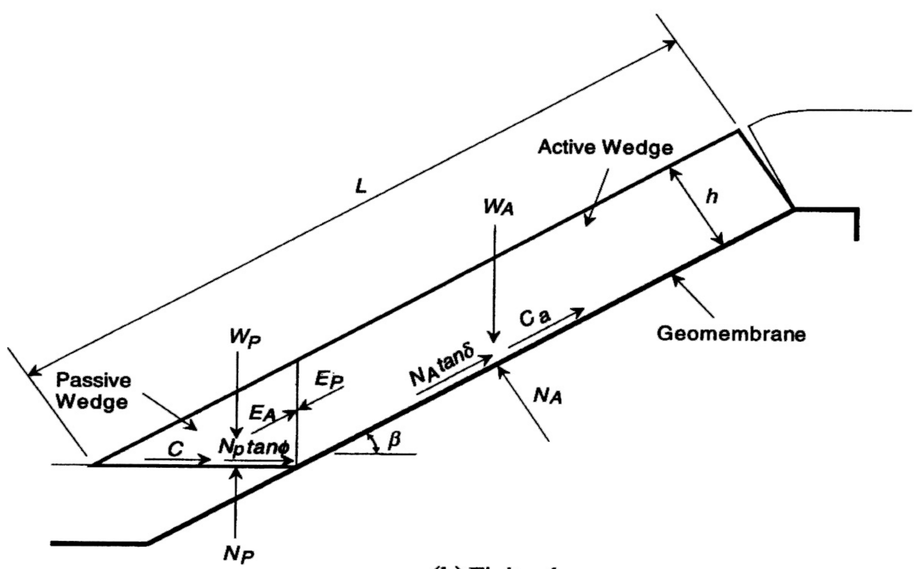
The tensile strength of the GCL has been taken from the a tpyical GCL cap product. A copy of the relevant data sheet is givn in the reference page.

Section **A** **PSR** = 0

Aim: To assess the stability and integrity of the geosynthetic capping system.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geotextile friction angle, δ_1	24	Deg.
Cover Soils/Geotextile cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	0	
Geosynthetic tensile strengths:		
GCL	12	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m
Thickness of water, h_w	0	m
Weight of active wedge, W_A	408.0841	kN
Weight of passive wedge, W_P	38.34098	kN
Pore pressure perp. to slope, U_n	0	kN
Pore pressure in interwedge surface, U_h	0	kN
Force normal to active wedge, N_A	395.9623	kN
Vert pp on passive wedge, U_V	0	kN
a	95.79194	
b	-200.073	
c	19.8877	

Factor of Safety against cover soils sliding **1.98**

2. Integrity of Geosynthetics

(i) Geosynthetic Layer No.1

Mobilised shear stress at upper interface	97.20753	kN
Shear strength at lower interface	192.8572	kN
Tension developed in the geosythetic	0	kN
Tensile strength of the geosythetic	12	kN

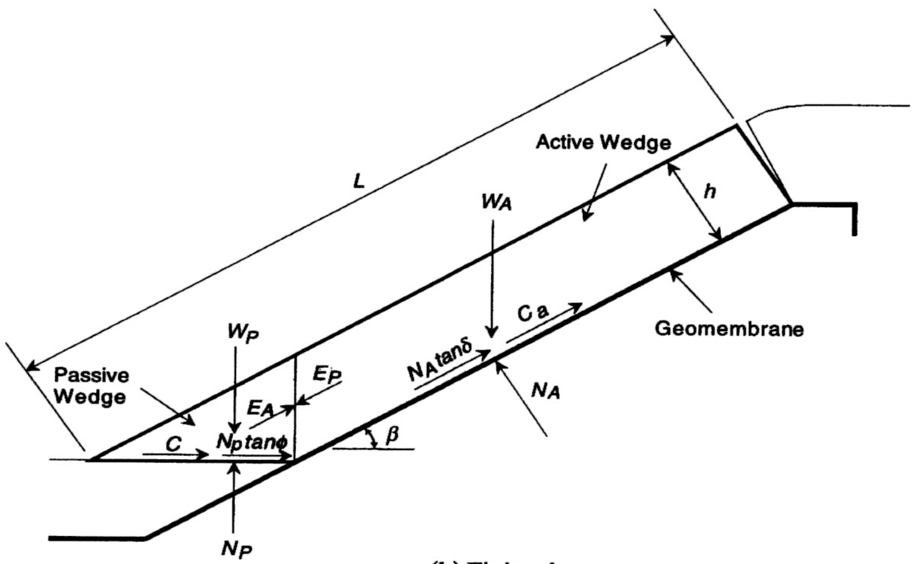
Factor of Safety against rupture **Infinite**

Section **A** **PSR** = 0.5

Aim: To assess the stability and integrity of the geosynthetic capping system.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geotextile friction angle, δ_1	24	Deg.
Cover Soils/Geotextile cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	0.5	
Geosynthetic tensile strengths:		
GCL	12	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m
Thickness of water, h_w	0.5	m
Weight of active wedge, W_A	443.6886	kN
Weight of passive wedge, W_P	39.93852	kN
Pore pressure perp. to slope, U_n	115.1565	kN
Pore pressure in interwedge surface, U_h	1.25	kN
Force normal to active wedge, N_A	315.6551	kN
Vert pp on passive wedge, U_V	5.013476	kN
a	104.2228	
b	-164.622	
c	15.85418	

Factor of Safety against cover soils sliding **1.48**

2. Integrity of Geosynthetics

(i) Geosynthetic Layer No.1

Mobilised shear stress at upper interface	141.5033	kN
Shear strength at lower interface	208.9286	kN
Tension developed in the geosythetic	0	kN
Tensile strength of the geosythetic	12	kN

Factor of Safety against rupture **Infinite**

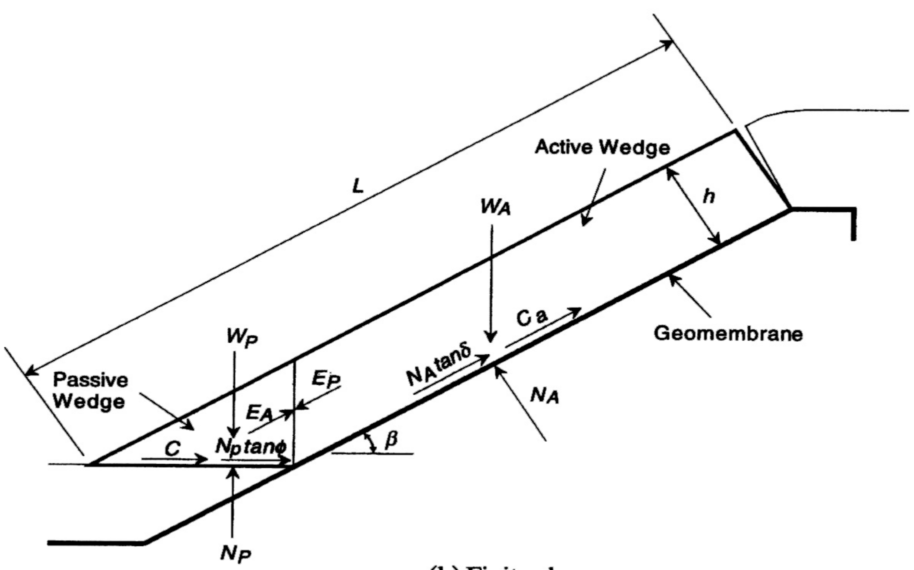
PROJECT	Biffa Eye Eastern Extension Stability Assessment		
Job No. 21453458		Made By: WYH	Date: 22/01/2022
Ref. Appendix 7		Checked: BZ	Sheet: 6
		Reviewed: BZ	of: 11

Section **A** **PSR** = 1

Aim: To assess the stability and integrity of the geosynthetic capping system.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geotextile friction angle, δ_1	24	Deg.
Cover Soils/Geotextile cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	1	
Geosynthetic tensile strengths:		
GCL	12	kN/m

PROJECT	Biffa Eye Eastern Extension Stability Assessment				
Job No.	21453458	Made By:	WYH	Date:	22/01/2022
Ref.	Appendix 7	Checked:	BZ	Sheet:	7
		Reviewed:	BZ	of:	11

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m	
Thickness of water, h_w	1	m	
Weight of active wedge, W_A	476.0981	kN	
Weight of passive wedge, W_P	44.73114	kN	
Pore pressure perp. to slope, U_n	219.979	kN	
Pore pressure in interwedge surface, U_h	5	kN	
Force normal to active wedge, N_A	243.1865	kN	
Vert pp on passive wedge, U_V	20.0539	kN	
a	112.0499		
b	-129.011		
c	12.21435		
	Factor of Safety against cover soils sliding		**1.05**

2. Integrity of Geosynthetics

(i) Geosynthetic Layer No.1

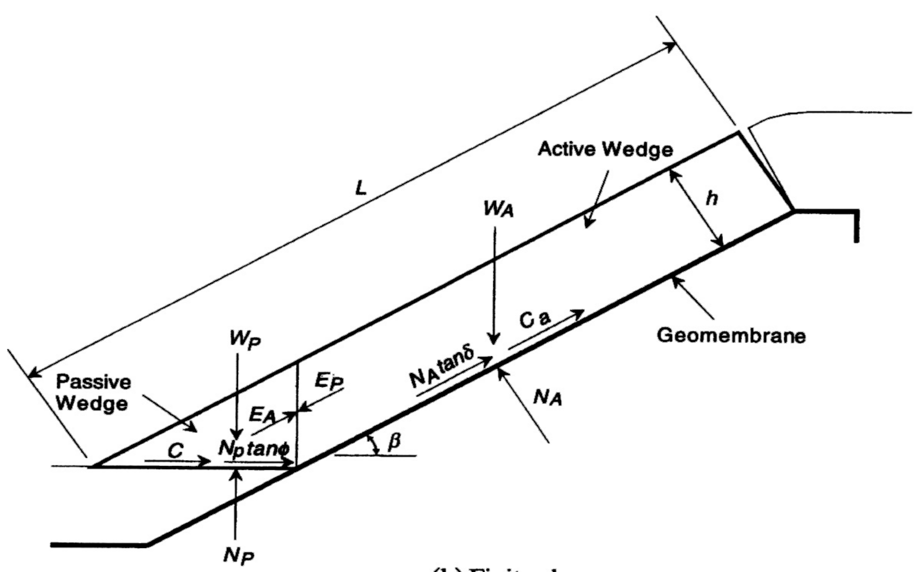
Mobilised shear stress at upper interface	214.8421	kN	
Shear strength at lower interface	225.0001	kN	
Tension developed in the geosythetic	0	kN	
Tensile strength of the geosythetic	12	kN	
	Factor of Safety against rupture		**Infinite**

Section **A** **PSR** = 0.65

Aim: To assess the stability and integrity of the geosynthetic capping system.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geotextile friction angle, δ_1	24	Deg.
Cover Soils/Geotextile cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	0.65	
Geosynthetic tensile strengths:		
GCL	12	kN/m

Golder Associates	**PROJECT**	**Biffa Eye Eastern Extension Stability Assessment**			
	Job No.	21453458	Made By: WYH	Date:	22/01/2022
	Ref.	Appendix 7	Checked: BZ	Sheet:	9
			Reviewed: BZ	of:	11

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m
Thickness of water, h_w	0.65	m
Weight of active wedge, W_A	453.747	kN
Weight of passive wedge, W_P	41.04082	kN
Pore pressure perp. to slope, U_n	147.6883	kN
Pore pressure in interwedge surface, U_h	2.1125	kN
Force normal to active wedge, N_A	293.0915	kN
Vert pp on passive wedge, U_V	8.472775	kN
a	106.6343	
b	-153.955	
c	14.72089	

Factor of Safety against cover soils sliding **1.34**

2. Integrity of Geosynthetics

(i) Geosynthetic Layer No.1

Mobilised shear stress at upper interface	159.4187	kN
Shear strength at lower interface	213.7501	kN
Tension developed in the geosythetic	0	kN
Tensile strength of the geosythetic	12	kN

Factor of Safety against rupture **Infinite**

interface shear strength corresponding to δ = 10.7° and α = 26.7 kPa. Closer inspection of the plot reveals that a non-linear fit may be more representative for the peak shear strength envelope, possibly curving downwards at lower normal stresses and passing through the origin. There is insufficient data to determine the residual shear strength for this interface, however, it is likely that the residual interface shear strength will be the residual shear strength of the clay. The asperities of the textured geomembrane are very similar to the upper sintered brass platten on the standard Bromhead ring shear apparatus (Bromhead 1979).

Non-woven geotextile

The results of testing on non-woven geotextiles are presented in Figure 3 and a summary is given in Table 3 below.

Interface	Interface shear strength parameters					
	Peak			Residual		
	δ (°)	α (kPa)	R^2	δ (°)	α (kPa)	R^2
Geonet	13.1	17.9	0.76	15.4	4.1	0.92
Gravel	35.0	-1.0	0.87	19.9	30.1	0.99
Sand	33.0	-1.3	0.93	28.7	7.7	0.92
Clay - undrained	25.3	5.3	0.91	17.7	55.6	0.98
Clay - drained	32.5	4.4	0.98	-	-	-

Table 3 Summary of results for non-woven geotextile

The results of shear strength testing on non-woven geotextile/geonet interfaces are plotted in Figure 3a and linear regression of all the data points give peak interface shear strengths of δ = 13.1° and α = 17.9 kPa with an R^2 value of 0.76. For the range of normal stresses considered, the residual envelope is similar to the peak in terms of its mobilised shear strength, however the friction angles and cohesion intercept are different. The best fit line through the residual data points is given by δ = 15.4° and α = 4.1 kPa, i.e. a higher friction angle but a lower cohesion intercept with a correlation coefficient of 0.92.

The non-woven geotextile/gravel interface has a high shear strength with some values in the literature reported as high as 48°. Most of the results available are for tests carried out at normal stresses less than 200 kPa (Figure 3b) and linear regression gives a friction angle of 35.0° with a cohesion intercept of -1.0 kPa. This reduces to a residual shear strength corresponding to δ = 19.9° and α = 30.1 kPa. The peak shear strength envelope shows a reasonable strong straight line fit with a correlation coefficient of 0.94, while the residual envelope has a very strong fit with R^2 = 0.99, however the residual is based on a small number of data points.

There is much more information available in the literature on the interface shear strength between sand and non-woven geotextiles, and this is also a high strength interface with a peak friction angle of 33.0° and a cohesion intercept of -1.3 kPa (Figure 3c). The residual shear strength for this interface is

Fibre-reinforced Geosynthetic Clay Liner (GCL)

Bentofix® NSP 4900

NAUE
0799-CPD-20
NAUE GmbH & Co. KG
Gewerbestrasse 2
32339 Espelkamp-Fiestel, Germany
Phone: +49 5743 41-0 · Fax: +49 5743 41-240
E-Mail: info@naue.com · Internet: www.naue.com

The following table lists properties of **Bentofix® NSP 4900**, a shear strength transmitting geosynthetic clay liner, continuously needle-punched through all components. Additional bentonite powder is impregnated into a 50 cm overlapping area on both longitudinal sides of the cover layer. The 30 cm longitudinal overlapping area is marked on the bottom side.

Property	Test method*	Unit	Values
Geotextile layers:			
Cover layer (polypropylene nonvowen):			
Mass per unit area	EN ISO 9864	g/m²	220
Carrier layer (polypropylene woven):			
Mass per unit area	EN ISO 9864	g/m²	110
Bentonite layer (sodium bentonite powder):			
Mass per unit area	EN 14196 (ρ_{CLAY})	g/m²	4,670
Swell index	ASTM D 5890	ml/2g	24
Fluid Loss	ASTM D 5891	ml	≤ 18
Water content	DIN 18121 / ISO 11465 (5hrs, 105 °C)	%	approx. 10
Geosynthetic Clay Liner:			
Mass per unit area	EN 14196 (ρ_{GBR-C})	g/m²	5,000
Thickness	EN ISO 9863-1	mm	6.0
Max. tensile strength, md/cmd**	EN ISO 10319 / ASTM D 4595	kN/m	12.0 / 12.0
Elongation at break, md/cmd**	EN ISO 10319 / ASTM D 4595	%	10.0 / 6.0
Peel strength	ASTM D 6496	N/10 cm*** N/m	≥ 60 ≥ 360
Static puncture strength	EN ISO 12236 / ASTM D 6241	N	2,000
Permeability / Hydraulic Conductivity	DIN 18130 / ASTM D 5887	m/s	2×10^{-11}
Index Flux	DIN 18130 / ASTM D 5887	(m³/m²)/s	5×10^{-9}
Roll dimensions:			
width x length, / diameter	-	m x m / m	5.00 x 40 / Ø 0.65

** = based on; **md = machine direction, cmd = cross machine direction; ***max. peak*

APPENDIX SRA8

Clay Capping Analyses

INTRODUCTION

The stability of the cover soils has been assessed for the clay capping system using the finite slope method. Analysis has been carried out for the selected steepest cross section, i.e. 1v:4h and 6 m high slope.

Stability

The effect of a partially and fully saturated cover soil layer has been assessed using the method proposed by Jones & Dixon (1998). The normal operating conditions have been modelled using dry cover soils and the worst case conditions of fully saturated cover soils have been analysed. The water pressures acting on the system have been modelled using a Parallel Submergence Ratio (PSR). PSR = 0 for dry conditions, PSR = 0.5 for partially saturated condtions and PSR = 1.0 for fully saturated conditons.

Capping

Analyses has been carried out assuming the lining system comprises a 1m thick Compacted Clay Liner (CCL) with a layer of 1.0m thick restoration soils above.

The parameters used in the analysis have been derived from a combination of the technical literature on interface shear strengths and previous project experience. Based on this and our experience of landfill capping assessment, a conservative assessment of the interface shear strength parameters is:

- Restoration soils / Clay Cap $\alpha_{p'}$ = 0 kPa δ_p' = 22 Deg.

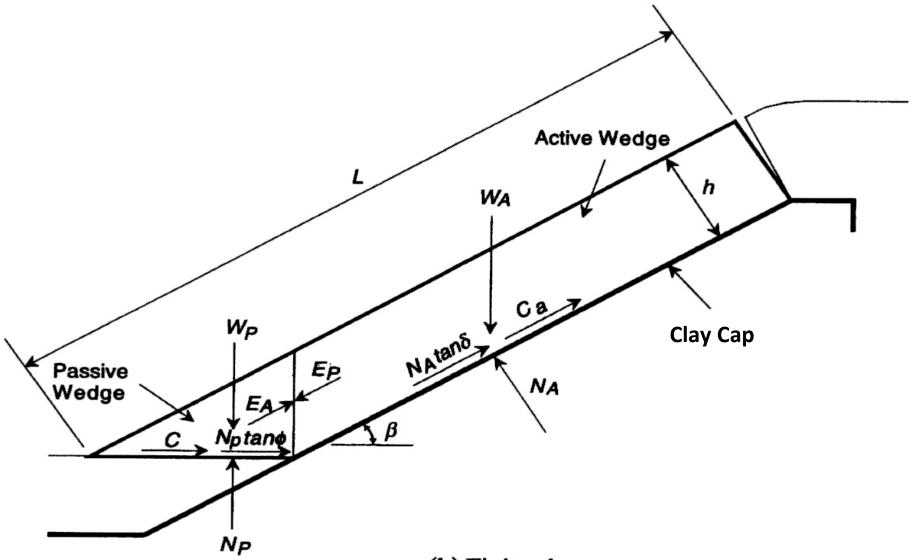
These values should be verified by site-specific shear strength testing during the construction stage. The values given above are all peak shear strengh values.

Section **A** **PSR** = 0

Aim: To assess the stability of restoration soils placed above the low permeability clay cap.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Interface shear strengths:		
Cover Soils/Clay Cap friction angle, δ_1	22	Deg.
Cover Soils/Clay Cap cohesion intercept, α_1	0	kPa

Parallel submergence ratio, PSR 0

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m
Thickness of water, h_w	0	m
Weight of active wedge, W_A	408.0841	kN
Weight of passive wedge, W_P	38.34098	kN
Pore pressure perp. to slope, U_n	0	kN
Pore pressure in interwedge surface, U_h	0	kN
Force normal to active wedge, N_A	395.9623	kN
Vert pp on passive wedge, U_V	0	kN
a	95.79194	
b	-184.243	
c	18.04725	

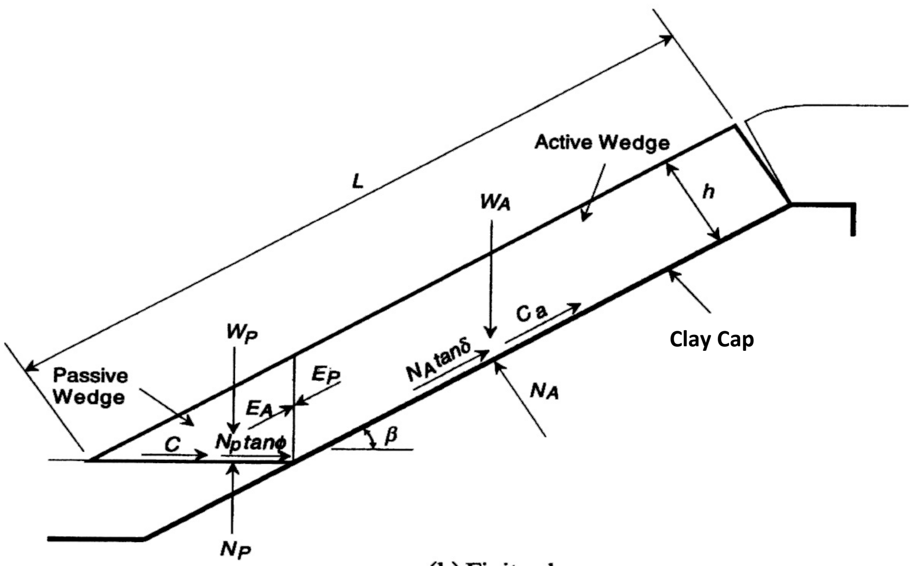
Factor of Safety against cover soils sliding **1.82**

Section **A** **PSR** = 0.5

Aim: To assess the stability of restoration soils placed above the low permeability clay cap.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Interface shear strengths:		
Cover Soils/Clay Cap friction angle, δ_1	22	Deg.
Cover Soils/Clay Cap cohesion intercept, α_1	0	kPa
Parallel submergence ratio, PSR	0.5	

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m
Thickness of water, h_w	0.5	m
Weight of active wedge, W_A	443.6886	kN
Weight of passive wedge, W_P	39.93852	kN
Pore pressure perp. to slope, U_n	115.1565	kN
Pore pressure in interwedge surface, U_h	1.25	kN
Force normal to active wedge, N_A	315.6551	kN
Vert pp on passive wedge, U_V	5.013476	kN
a	104.2228	
b	-152.002	
c	14.387	

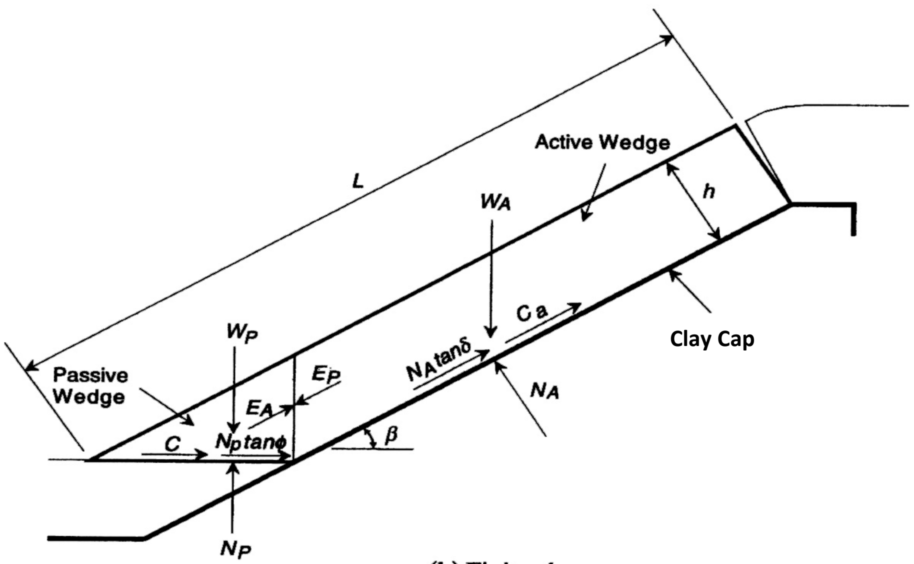
Factor of Safety against cover soils sliding **1.36**

Section A **PSR** = 1

Aim: To assess the stability of restoration soils placed above the low permeability clay cap.

Approach: Use the approach proposed by Jones & Dixon, 1998.

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	21	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	6	m
Slope angle, β	14	Deg.
Interface shear strengths:		
Cover Soils/Clay Cap friction angle, δ_1	22	Deg.
Cover Soils/Clay Cap cohesion intercept, α_1	0	kPa
Parallel submergence ratio, PSR	1	

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	24.80139	m
Thickness of water, h_w	1	m
Weight of active wedge, W_A	476.0981	kN
Weight of passive wedge, W_P	44.73114	kN
Pore pressure perp. to slope, U_n	219.979	kN
Pore pressure in interwedge surface, U_h	5	kN
Force normal to active wedge, N_A	243.1865	kN
Vert pp on passive wedge, U_V	20.0539	kN
a	112.0499	
b	-119.288	
c	11.08401	

Factor of Safety against cover soils sliding **0.96**

APPENDIX SRA9

Leachate Extraction System Analyses

Aim: Establish the stability and servicebility of the leachate extraction and monitoring wells.

Background: The leachate extraction wells comprise 0.9 m internal diameter, reinforced concrete chamber.

The base comprises a 300 mm thick, 3000 mm square concrete slab.

The leachate well will be built up with the waste, with a maximum height of 23.0 m (including 1.0 m of drainage gravel on top of the slab and 1.0 m of restoration soils).

Approach: Assess the bearing capacity and differential settlement under loading.

Assumptions:

Unit weight of concrete, γ_{conc}		=	24	kN/m^3
Unit weight of clay, γ_{Clay}		=	19	kN/m^3
Unit weight of gravel, γ_{gravel}		=	18	kN/m^3
Unit weight of restoration soils, γ_{rest}		=	18	kN/m^3
Unit weight of waste, γ_{waste}		=	10	kN/m^3
Shear strength of the clay liner (total stress), c_u		=	50	kPa
Shear strength of the clay liner (effective stress),	c'	=	2	kPa
	ϕ'	=	26	degrees
Friction angle between waste and concrete, δ=			12	degrees
Waste coefficient, $K_{waste}(\sigma_h'/\sigma_v')$		=	0.4	

Calculations:

1. Loading from various components

(a) Self weight of concrete chamber

Internal diameter	=	0.9	m
Wall thickness	=	0.1	m
External diameter	=	1.1	m
Final height	=	21.5	m
Waste Height	=	23	m
Unit weight of concrete	=	24	kN/m^3

Load = $(\pi/4)h(D^2_e - D^2_i)\gamma_{conc}$

Load = **162.1** kN

(b) Concrete slab loading

3 x **3** m

Thickness = **0.3** m

Unit weight of concrete = 24 kN/m^3

Load = Volume x γ_{conc}

Load = **64.8** kN

Calculations:

Loading from various components (Cont'd.)

(c) Waste load on extraction slab

Slab area = 9 m^2

Pipe area = π x D_e^2 / 4 = 0.95 m^2

Load = (slab area - pipe area) x height x γ_{waste}

Load = **1,851.4** kN

(d) Gravel load on extraction slab

Load = (slab area - pipe area) x thickness x γ_{gravel}

Thickness of Gravel $\boxed{1}$ m

Load = **144.9** kN

(e) Cap and Restoration load on extraction slab

Load = (slab area - pipe area) x ((cap thickness x $\gamma_{cap)}$ + (restoration thickness x γ_{rest}))

Mineral Cap thickness = $\boxed{0}$ m

Restoration Thickness = $\boxed{1}$ m

Load = **144.9** kN

(f) Negative skin friction loading on concrete pipe

NSF is given by $\sigma_h'\tan\delta$, where $\sigma_h' = K_{waste}.\sigma_v'$

NSF = $(K_{waste} \cdot \sigma_{vmax}' \cdot \tan\delta)/2$ = 11.3 kPa

Load = NSF x surface area

Load = NSF x π x External diameter x total height

Load = **840.2** kN

(g) Loading of waste, cap, restoration soils and gravel only

Load = (height x γ_{waste}) + (thickness x γ_{cap}) + (thickness x γ_{rest}) + (thickness x γ_{gravel})

Load = 266.0 kPa

Calculations:

Summary of loadings

Element	**Extraction point**	
Concrete chamber self weight	**162.1**	kN
Concrete slab	**64.8**	kN
Waste on slab	**1,851.4**	kN
Gravel on slab	**144.9**	kN
Cap and Restoration soils on slab	**144.9**	kN
Negative skin friction	**840.2**	kN
Total load	**3,208.3**	kN
Expressed as a pressure	**356.5**	kPa

2. Bearing capacity

(i) Total stress

The bearing capacity (qf) of the Clay liner beneath the square slab in total stress terms can be expressed as:

$q_f = c_u N_c + \sigma_v = c_u N_c + \gamma D$

where:

c_u is the undrained shear strength of the material within the bearing capacity failure zone

N_c is a bearing capacity factors = 5.14 obtained from page 6 (Skempton, 1951).

γD = (height x γ_{waste}) + (thickness x γ_{cap}) + (thickness x γ_{rest}) + (thickness x γ_{gravel})

γD = **266.0** kPa

For c_u = 50 kPa

q_f = **523.0** kPa

Factor of safety against shear failure is given by:

$F = q_f / q$

Factor of safety: **1.5**

Calculations:

(ii) Effective stress

The bearing capacity (q_f) of the Clay liner beneath the square slab in effective stress terms can be expressed as:

$q_f = 0.5\gamma_{Clay}BN_\gamma + 1.2cN_c + p_oN_q$

where:

γ_{Clay} is the unit weight of the Clay beneath the slab

B = width of slab

c = cohesion of the Clay

p_o = effective stress of overburden soil at foundation level

Assuming the maximum leachate head will be 3m (conservative),

p_o = 266.0 - (3 *10) **236.0** kPa

N_γ, N_c and N_q are bearing capcity factors given by:

$N_q = \exp\{\pi\tan\phi\} \times \tan^2(45+\phi/2)$

$N_q = \exp\{\pi * \tan\ 26\} \times \tan^2(45 + 26/2)$

$N_q =$ **11.85**

$N_\gamma = (N_q - 1) \times \tan(1.4\phi)$

$N_\gamma = (N_q - 1) \times \tan(1.4 \times 26)$

$N_\gamma =$ **8.00**

$N_c = (N_q - 1)\cot\phi$

$N_c = (N_q - 1) / \tan\ 26$

$N_c =$ **22.25**

Hence, q_f = **3117.8** kPa

Factor of safety against shear failure is given by:

$F = q_f / q$

Factor of safety: **23.9**

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Calculations:

3. Settlement

Using the Skempton-Bjerrum method for consolidation settlement:

$\rho_{consol} = m_v \times H \times \Delta p \times \mu$

where:

m_v = 0.1 m^2/MN

μ = 0.5

H = 0.5 m (thickness of Clay liner)

The increase in vertical stress under the centre of the slab, Δp, can be obtained from Janbu *et al.* , 1956 (see page 7)

z/B = 0.3 / 3 = 0.1 hence from Page 7 $\Delta p/q$ = 0.99

(a) Settlement under extraction slab

Maximum value of q = 356.5 hence Δ_p = 356.5 * 0.99 = **352.9** kPa

ρ_{consol} = 0.1 0.5 x 352.9 x 0.5

ρ_{consol} = 8.8 mm

Total settlement is typically no greater than 1.5 x ρ_{consol}

ρ_{tot} = 1.5 x 8.8 = **13.2** mm

(b) Settlement under waste only

Maximum value of q = 266.0

ρ_{consol} = 0.1 x 0.5 x 266 x 0.5

ρ_{consol} = 6.7 mm

Total settlement is typically no greater than 1.5 x ρ_{consol}

ρ_{tot} = 1.5 x 6.7 = **10.0** mm

(c) Differential settlement:

Settlement beneath slab = 13.2

Settlement beneath waste = 10.0

Differential settlment = 13.2 - 10.0 = **3.3** mm

Conclusions:

Both bearing capacity and anticipated settlement are considered satisfactory.

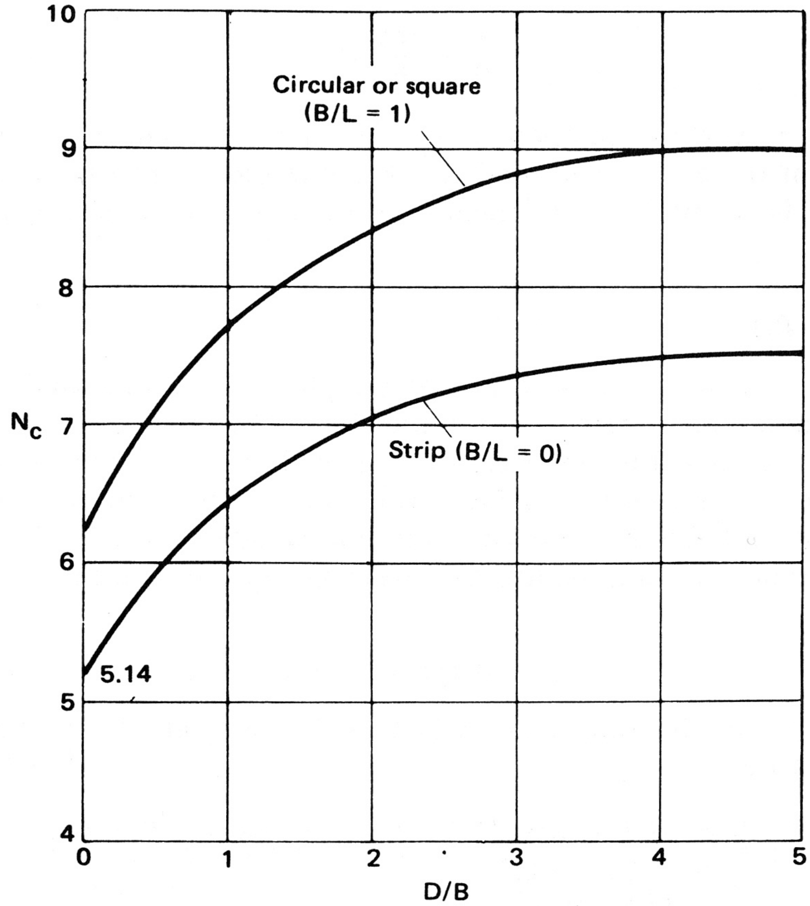

Fig. 8.5 Skempton's values of N_c for $\phi_u = 0$. (Reproduced from A.W. Skempton (1951) *Proceedings of the Building Research Congress*, Division 1, p. 181, by permission of the Building Research Establishment, © Crown copyright.)

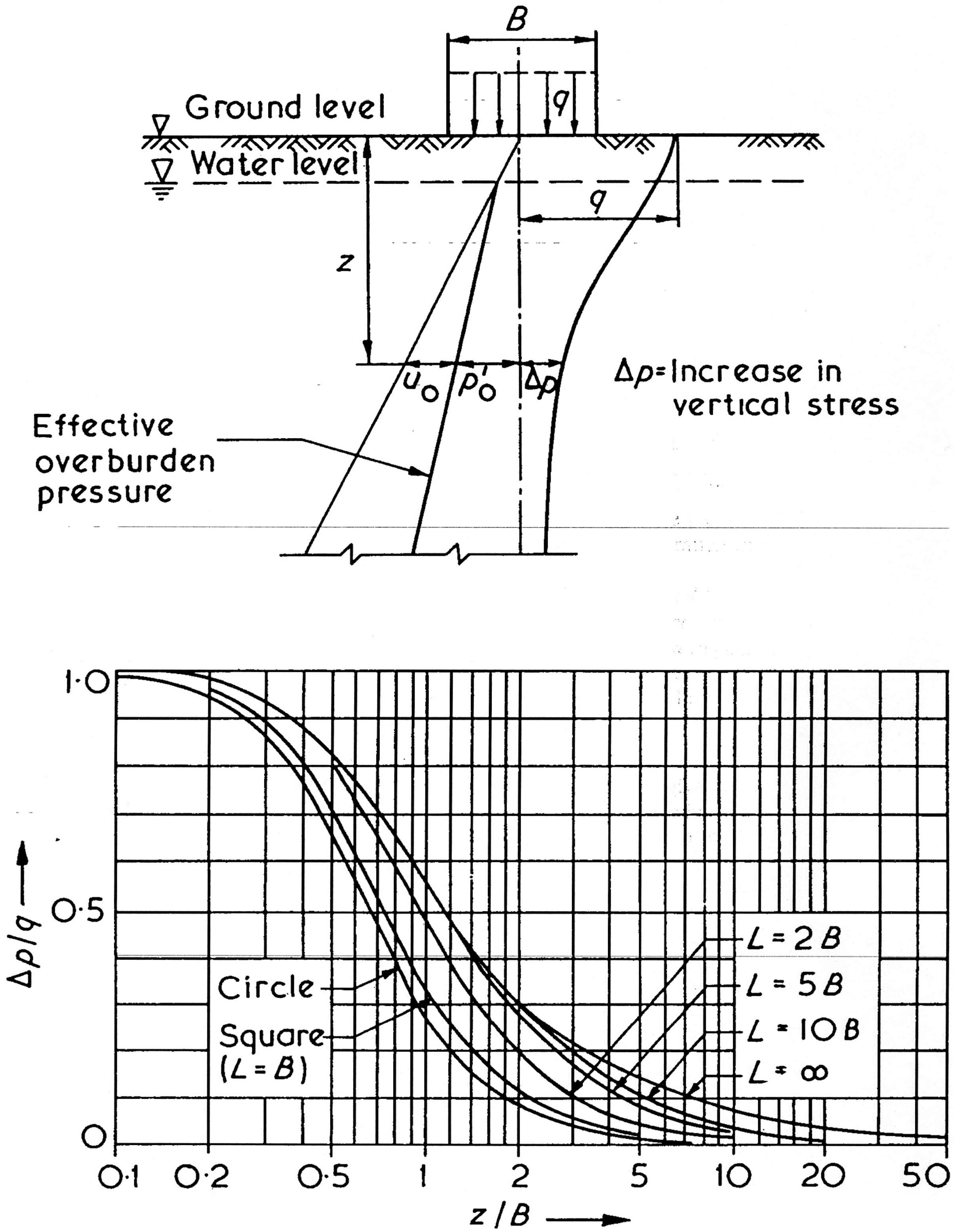

Fig. 3.3 Determination of increase in vertical stress under the centre of uniformly loaded flexible footings, after Janbu, Bjerrum and Kjaernsli (1956)

APPENDIX SRA10

Leachate Pipework Deflection Analyses

Leachate Pipework Strength Calculations

Aim: To assess strength of the primary leachate drainage pipe with an internal diameter of 120 mm.

Approach: To use the Iowa formula to predict the long term deformation of the leachate drainage pipe.

References:
1 Environment Agency, R&D Technical Report P1-397/TR, Landfill Engineering: Leachate Drainage, Collection and Extraction Systems, September 2002.

2 Qian X., Koerner R.M., and Gray D.H., Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

The Iowa Formulae can be used to predict the deformation of a pipeline at any stage in its life. The primary design limitation of long term deformation can be calculated using the following equation:

$$\boldsymbol{\delta_v} = \frac{D_L\, K_X\, W_c}{(EI/r^3)+(0.061\ E')}$$ Equation 1.

Where:

W_c = Static Loading (simple prismatic loading is assumed)
= ((depth to crown of pipe·γ_{waste})+(leachate drainage thickness·γ_{gravel})+(resto soil thickness·$\gamma_{restor\ soils}$))·OD of pipe
= ((21.0 m x 10 kN/m^3) + (0.45 m x 18 kN/m^3) + (1 m x 18 kN/m^3)) x 0.12
= **28.332** kN/m

D_L = Deflection lag factor (dimensionless)
= **1.5** (assumed)

K_x = Bedding factor
= **0.103** (value assumed is as recommended by the Water Research Centre)

r = Mean radius of pipe
= **60** mm

t = Wall thickness of pipe
= **7.059** mm

I = Moment of inertia of pipe wall per unit length
= **29.3** mm^3

E = Modulus of elasticity of the pipe material (long term)
= **150,000** kPa

S_L = (EI/r^3) = Long-term stiffness of pipe
= **20.4** kPa
`

E' = Modulus of soil reaction
= **21,000** kPa, (corresponding to a crushed rock with little or no fines compacted to 85-95% Standard Proctor density Ref. 2 Table 7.9 reproduced on page 3)

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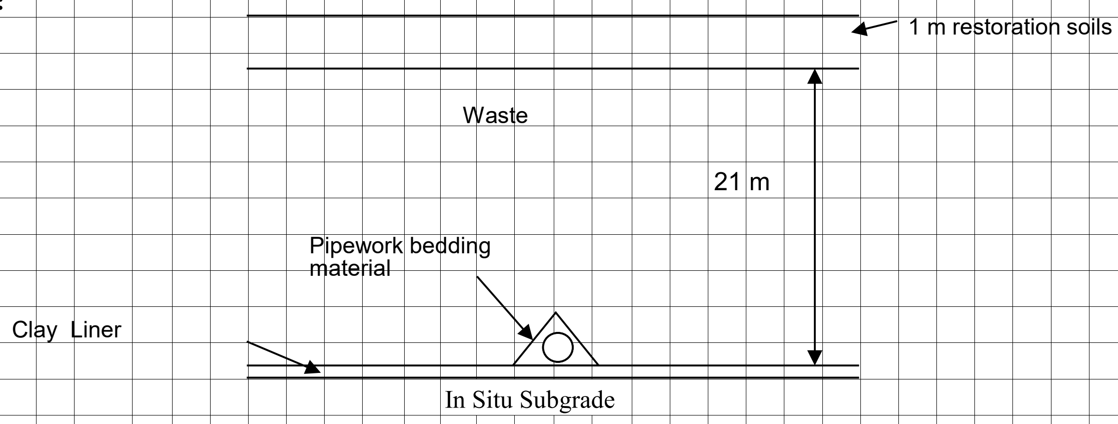
Geometry:



Calculation:

From Equation (1), the deflection of the pipe is given by:

δ_v = **0.003** m
= **3.36** mm
= **2.8** % of the nominal pipe inside diameter

The calculations indicate that once the waste has been placed, the leachate drainage pipe will deflect up to approximately 2.8%. It is envisaged that this amount of deflection will not result in intergrity failure of the pipe.

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Leachate Pipework Strength Calculations

Aim: To assess strength of the secondary leachate drainage pipe with an internal diameter of 160 mm.

Approach: To use the Iowa formula to predict the long term deformation of the leachate drainage pipe.

References:

1 Environment Agency, R&D Technical Report P1-397/TR, Landfill Engineering: Leachate Drainage, Collection and Extraction Systems, September 2002.

2 Qian X., Koerner R.M., and Gray D.H., Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

The Iowa Formulae can be used to predict the deformation of a pipeline at any stage in its life. The primary design limitation of long term deformation can be calculated using the following equation:

$$\boldsymbol{\delta_v} = \frac{D_L\, K_X\, W_c}{(EI/r^3)+(0.061\ E')}$$

Equation 1.

Where:

W_c = Static Loading (simple prismatic loading is assumed)

= ((depth to crown of pipe·γ_{waste})+(leachate drainage thickness·γ_{gravel})+(resto soil thickness·$\gamma_{restor\ soils}$))·OD of pipe

= ((22.0 m x 10 kN/m^3) + (0.24 m x 18 kN/m^3) + (1 m x 18 kN/m^3)) x 0.16

= **38.771** kN/m

D_L = Deflection lag factor (dimensionless)

= **1.5** (assumed)

K_x = Bedding factor

= **0.103** (value assumed is as recommended by the Water Research Centre)

r = Mean radius of pipe

= **80** mm

t = Wall thickness of pipe

= **9.4** mm

I = Moment of inertia of pipe wall per unit length

= **69.5** mm^3

E = Modulus of elasticity of the pipe material (long term)

= **150,000** kPa

S_L = (EI/r^3) = Long-term stiffness of pipe

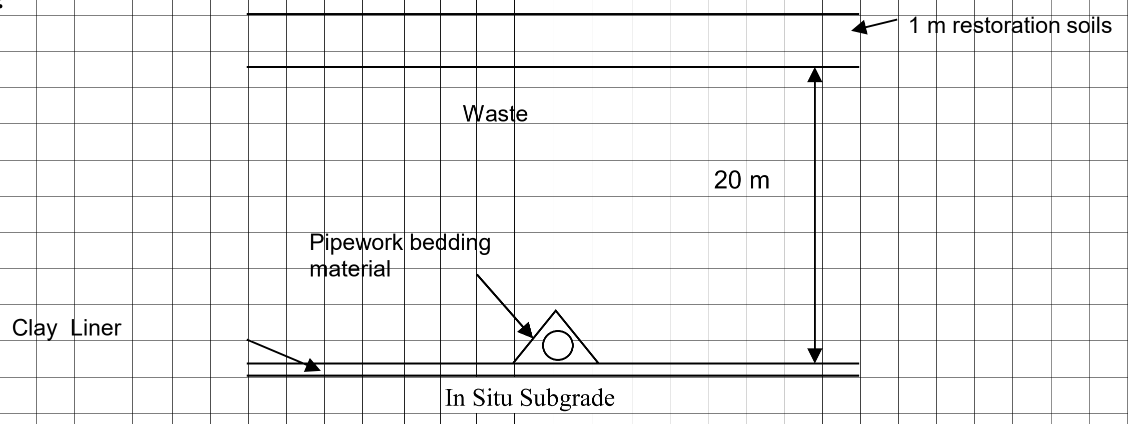
= **20.4** kPa

`

E' = Modulus of soil reaction

= **21,000** kPa, (corresponding to a crushed rock with little or no fines compacted to 85-95% Standard Proctor density Ref. 2 Table 7.9 reproduced on page 3)

Geometry:



Calculation:

From Equation (1), the deflection of the pipe is given by:

δ_v	=	**0.005**	m
	=	**4.60**	mm
	=	**2.9**	% of the nominal pipe inside diameter

The calculations indicate that once the waste has been placed, the leachate drainage pipe will deflect up to approximately 2.5%. It is envisaged that this amount of deflection will not result in intergrity failure of the pipe.

PROJECT	**Biffa Eye Eastern Extension Stability Assessment**		
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Leachate Pipework Strength Calculations

Aim: To assess strength of the primary leachate drainage pipe with an internal diameter of 160 mm.

Approach: To use the Iowa formula to predict the long term deformation of the leachate drainage pipe.

References:

1 Environment Agency, R&D Technical Report P1-397/TR, Landfill Engineering: Leachate Drainage, Collection and Extraction Systems, September 2002.

2 Qian X., Koerner R.M., and Gray D.H., Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

The Iowa Formulae can be used to predict the deformation of a pipeline at any stage in its life. The primary design limitation of long term deformation can be calculated using the following equation:

$$\boldsymbol{\delta_v} = \frac{D_L\, K_X\, W_c}{(EI/r^3)+(0.061\ E')}$$ Equation 1.

Where:

W_c = Static Loading (simple prismatic loading is assumed)

= ((depth to crown of pipe$\cdot\gamma_{waste}$)+(leachate drainage thickness$\cdot\gamma_{gravel}$)+(resto soil thickness$\cdot\gamma_{restor\ soils}$))$\cdot$OD of pipe

= ((21.0 m x 10 kN/m^3) + (0.45 m x 18 kN/m^3) + (1 m x 18 kN/m^3)) x 0.16

= **37.776** kN/m

D_L = Deflection lag factor (dimensionless)

= **1.5** (assumed)

K_x = Bedding factor

= **0.103** (value assumed is as recommended by the Water Research Centre)

r = Mean radius of pipe

= **80** mm

t = Wall thickness of pipe

= **9.412** mm

I = Moment of inertia of pipe wall per unit length

= **69.5** mm^3

E = Modulus of elasticity of the pipe material (long term)

= **150,000** kPa

S_L = (EI/r^3) = Long-term stiffness of pipe

= **20.4** kPa

`

E' = Modulus of soil reaction

= **21,000** kPa, (corresponding to a crushed rock with little or no fines compacted to 85-95% Standard Proctor density Ref. 2 Table 7.9 reproduced on page 3)

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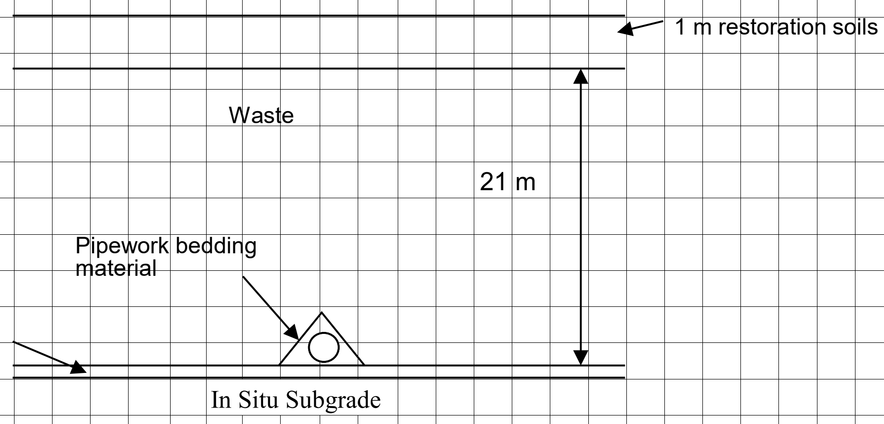
Geometry:



Calculation:

From Equation (1), the deflection of the pipe is given by:

δ_v = **0.004** m

= **4.48** mm

= **2.8** % of the nominal pipe inside diameter

The calculations indicate that once the waste has been placed, the leachate drainage pipe will deflect up to approximately 2.8%. It is envisaged that this amount of deflection will not result in intergrity failure of the pipe.

Leachate Pipework Strength Calculations

Aim: To assess strength of the secondary leachate drainage pipe with an internal diameter of 120 mm.

Approach: To use the Iowa formula to predict the long term deformation of the leachate drainage pipe.

References:

1. Environment Agency, R&D Technical Report P1-397/TR, Landfill Engineering: Leachate Drainage, Collection and Extraction Systems, September 2002.
2. Qian X., Koerner R.M., and Gray D.H., Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

The Iowa Formulae can be used to predict the deformation of a pipeline at any stage in its life. The primary design limitation of long term deformation can be calculated using the following equation:

$$\boldsymbol{\delta_v} = \frac{D_L K_X W_c}{(EI/r^3)+(0.061 E')}$$

Equation 1.

Where:

W_c = Static Loading (simple prismatic loading is assumed)

= ((depth to crown of pipe$\cdot\gamma_{waste}$)+(leachate drainage thickness$\cdot\gamma_{gravel}$)+(resto soil thickness$\cdot\gamma_{restor\ soils}$))$\cdot$OD of pipe

= ((22.0 m x 10 kN/m^3) + (0.24 m x 18 kN/m^3) + (1 m x 18 kN/m^3)) x 0.12

= **29.078** kN/m

D_L = Deflection lag factor (dimensionless)

= **1.5** (assumed)

K_x = Bedding factor

= **0.103** (value assumed is as recommended by the Water Research Centre)

r = Mean radius of pipe

= **60** mm

t = Wall thickness of pipe

= **7.1** mm

I = Moment of inertia of pipe wall per unit length

= **29.3** mm^3

E = Modulus of elasticity of the pipe material (long term)

= **150,000** kPa

S_L = (EI/r^3) = Long-term stiffness of pipe

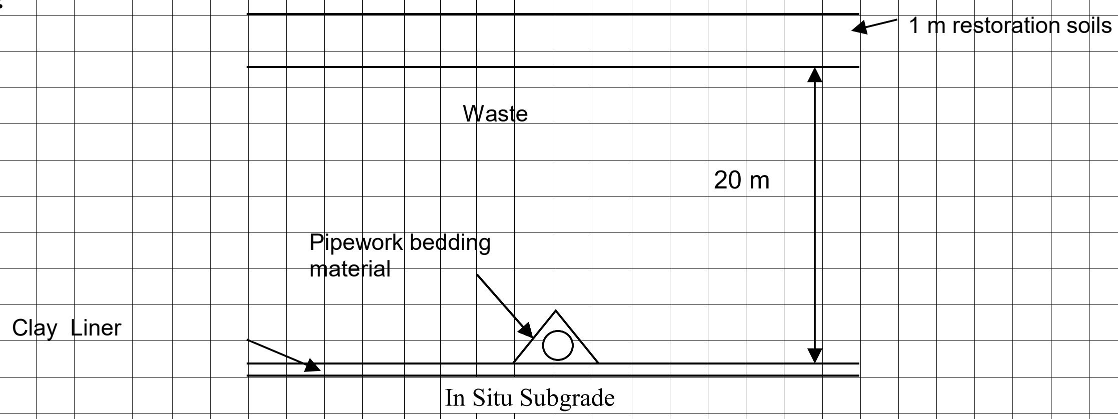
= **20.4** kPa

`

E' = Modulus of soil reaction

= **21,000** kPa, (corresponding to a crushed rock with little or no fines compacted to 85-95% Standard Proctor density Ref. 2 Table 7.9 reproduced on page 3)

Geometry:



Calculation:

From Equation (1), the deflection of the pipe is given by:

δ_v	=	**0.003**	m
	=	**3.45**	mm
	=	**2.9**	% of the nominal pipe inside diameter

The calculations indicate that once the waste has been placed, the leachate drainage pipe will deflect up to approximately 2.5%. It is envisaged that this amount of deflection will not result in intergrity failure of the pipe.

TABLE 7.9 U.S. BUREAU OF RECLAMATION VALUES OF MODULUS OF SOIL REACTION E' (kPa) FOR BURIED PIPELINES

Class ASTM D-2321	Soil type for pipe bedding material (Unified Classification System[A])	Dumped	Slight < 85% Std. Proctor[C] < 40% Rel. Den.[D]	Moderate 85–95% Std. Proctor 40–70% Rel. Den.	High > 95% Std. Proctor >70% Rel. Den.
I	Crushed rock: manufactured angular, granular material with little or no fines (6 to 38 mm)	7,000	21,000	21,000	21,000
II	Coarse-grained soils with little or no fines: GW, GP, SW, SP[B] containing less than 12 percent fines (max. particle size 38 mm)	NR	7,000	14,000	21,000
III	Coarse-grained soils with fines: GM, GC, SM, SC[B] containing more than 12 percent fines (max. particle size 38 mm)	NR	NR	7,000	14,000
IV(a)	Fine-grained soil (LL < 50): Soils with medium to no plasticity CL, ML, ML-CL, with more than 25 percent coarse-grained particles	NR	NR	7,000[E]	14,000[E]
IV(b)	Fine-grained soils (LL > 50): Soils with high plasticity CH, MH, CH-MH	NR	NR	NR	NR
	Fine-grained soils (LL < 50): Soils with medium to no plasticity CL, ML, ML-CL with less than 25 percent coarse-grained particles				

Organic soils OL, OM, and PT as well as soils containing frozen earth, debris, and large rocks are not recommended for initial backfill; NR = Not recommended for use per ASTM D-2321; LL = Liquid Limit.

[A] ASTM Designation D-2487

[B] Or any borderline soil beginning with some of these symbols (i.e., GM, GC, GC-SC).

[C] Percent Proctor based on laboratory maximum dry density from test standards using about 598,000 joules/m^3 (ASTM D-698)

[D] Relative Density per ASTM D-2049.

[E] Under some circumstances Class IV(a) soils are suitable as primary initial backfill. They are not suitable under heavy dead loads, dynamic loads, or beneath the water table. Compact with moisture content at optimum or slightly dry of optimum. Consult a Geotechnical Engineer before using.

Source: After Howard [14].

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