

REPORT

Biffa Waste Services Ltd Eye Eastern, Extension Landfill

Stability Risk Assessment

Submitted to:

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21453458.634/A.0

11 May 2022

Distribution List

Biffa Waste Services Ltd - 1 pdf Environment Agency - 1 pdf

Golder, member of WSP UK Ltd - 1 pdf



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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.

SITE DEVELOPMENT AND SETTING 2.0 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 Mm³ of waste (pre-settlement, pre-restoration) or 2.43 Mm³ 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.

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

Table SRA1: Summary of Regional Geology

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):2(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 aguifer) 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 x 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 x 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 x 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 guality 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.



STABILITY RISK ASSESSMENT 3.0

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%.

Justification for Modelling Approach and Software 3.4

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/m³)
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 SRAS. Summary of Parameters used in the Sub-grade in the Side Side Analyse	Table SRA3: Summary	v of Parameters u	sed in the Sub-grad	le in the Side	Slope Analyses
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Materials	Unit Weight γ (kN/m³)	Undrained Shear Strength c _u (kPa)	Cohesion c' (kPa)	Friction Angle ∳′ (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/m³)	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.



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

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

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, ru=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, ru=0.1	1.24

Waste Analyses 3.6.4

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, ru=0.1	1.26
Section C_Temporary Waste_5	Section C, 1v:2h slope, circular failure, 1m leachate level, ru=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, ru=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, ru=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 Geo	membrane Capping	Stability Analyses
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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

Leachate Extraction System Analyses 3.6.6

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	Oettienient (min)	
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.

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	

Table SRA15: Summary of Leachate Pipe Work Deflection Calculations

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 clav 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 ru 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**

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Signature Page

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Drawings







APPENDIX SRA1

Basal Heave Analyses





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

2. Basal liner st	ability	y with o	clay												
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Factor of Safety	= (1	$F-D\gamma_m$	$+1.0\gamma_{c}$	=	1.33	3									
		(G-D)	γ_w												
3. Basal liner st	ability	y when	comp	lete	<u> </u>						_				
Factor of Safety	again	st basa	l heave	e after	placen	nent of	clay	liner	and g	grave	el:				
			1.0	0.5.		4.00									
Factor of Safety	= (P -	$-D_{\gamma_m} + \frac{1}{(C_{\gamma_m})}$	$\frac{1.0\gamma_c}{D}$	$-0.5\gamma_g$	=	1.38									
		(6-	D_{γ_w}	1 1											
References:															
Environment Agenc	y, 200)3													
Stability of Landfill L	inina	System	s: Rep	ort No.	1 Lite	rature l	Revie	w							
R&D Technical Rep	ort P1	-385/TI	R1												

APPENDIX SRA2

Side Slope Sub-Grade Analyses






Side Slope Liner Analyses

















Temporary Waste Analyses

















Final Waste Analyses









Geomembrane Capping Analyses



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cap	pin	g sy	sten	n. A	Anal	ysis	has	bee	en c	arrie	ed o	ut f	or se	elect	ted s	steep	best	and	hei	ghe	st se	ectio	on.										
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(19	98)	. Tł	he n	orm	nal o	pera	ting	g co	ndit	ions	s ha	ve b	een	mo	dell	ed u	sing	g dry	v cov	ver	soil	s an	d th	e w	orst	case	e co	ndit	ions	s of	full	у	
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Ge	osyı	the	ic i	inter	face	e shea	ar st	trei	ngth	s:																							
	Co	ver S	Soil	ls/G	eoco	ompo	site	e fri	ictio	n a	ngle	ε, δ ₁					24		Deg	•													
	Co	ver S	Soil	ls/G	eoco	ompo	site	e co	ohesi	ion	inte	ercej	ot, c	ι_1			0		kPa														
	Ge	ocor	npo	osite	/GN	A fric	tior	ı ar	ngle	, δ ₂							26		Deg	•													
	Ge	ocor	npo	osite	/GN	/I coh	esio	on	inter	rcep	ot, o	ι ₂					0		kPa														
	GN	1/B1	ind	ing	laye	\mathbf{r}, δ_3											24		kPa												\square		
	GM	1/B1	ind	ing	laye	r, α_3											0		Deg												$\mid \downarrow \downarrow$		
<u> </u>												-	<u> </u>																		\mid		
Par	alle	l sul	ome	erge	nce	ratio	, PS	R				-	<u> </u>			(0.00)													\mid		
Ge	osyı	the	tic t	tens	ile s	treng	ths:					-																_			$\mid \mid \mid$		
	Ge	otex	tile									-		-			10		kN/ı	m											\mid		
	1m	m L	LD	PE	Geo I	mem	brai	ne				-	-				11		kN/1	m											\vdash		
							-+					-	<u> </u>															-			$\mid - \mid$		

	PROJEC	T Biffa Eye	East	ern Ex	xtensi	on Sta	bility	As	sessm	ent	,	
Golder	Job No.	21453458		Made E	By: WY	Ή			Date:	22	2/01	/2022
Associates	Ref.	Appendix 6		Checke	d: BZ				Sheet:		3	
				Review	ed: BZ				of:		9	
										-		
1. Stability of Cover Soils												
										_		
Calculated Parameters												
Length of slope, L	+++-	24.80)139	m								
Thickness of water, h _w		0		m								
Weight of active wedge, W_A		408.0	0841	kN								
Weight of passive wedge, W _P		38.34	098	KN 1 N				_		_		
Pore pressure perp. to slope, U_n		0		KIN L-N								
Fore pressure in interwedge surface,	U _h	205.0	672	KIN L-NI								
Force normal to active wedge, N_A		393.9	023	KIN L-N				-				
vert pp on passive wedge, O_V	+ + +	05.70	104	KIN								
a h		93.79	072									
		-200.	073 877					-				
		19.0	0//									
		Factor of	' Safe	tv agai	nst cov	er soils	slidin	σ			1	98
								5				
2. Integrity of Geosynthetics												
(i) Geocomposite												
Mobilised shear stress at upper in	nterface	97.20	753	kN								
Shear strength at lower interface		211.2	.684	kN								
Tension developed in the GT		0		kN								
Tensile strength of the GT		10)	kN								
		Facto	r of S	Safety a	against	ruptur	·e				Infi	nite
(ii) GCL	+ $+$ $+$ $+$							+				
	+ $+$ $+$ $+$			1 3 7				+		-		
Shear strength at upper surface	+ $+$ $+$ $+$:084	kN				+				
Mobilized share stress stress i		07.00	752	1-NT			$\left - \right $	+				\vdash
moonsed snear stress at upper in		97.20	135	KIN		_		+				
Shear strength at lower interface	+ $+$ $+$ $+$	102 9	572	ĿN				+		-		
	+ $+$ $+$ $+$		512	NIN				+				
Tension developed in the GM	+ $+$ $+$			kN				+				
	+++			N1 1				+				
Tensile strength of the GM	+++		1	kN				\square		1		
	\uparrow									1		
		Facto	r of S	Safety a	against	ruptur	·e				Infi	nite

		A	7						PR	OJ	EC	Г	Bi	ffa	Eye	e E	ast	tern	E	xte	nsi	on	Sta	bil	ity	As	ses	sm	ent			
		7		Ga	ələ	er			Joł	o No).	214	4534	458				Mac	le I	By:	WY	ζH					Da	te:	22	2/01	/202	22
	V	D	A	SS	OC	iates	5		Re	f.		Ap	pen	dix	6			Che	cke	ed:	ΒZ						She	eet:		4		
																		Rev	iew	ved:	ΒZ						of:			9		
Sec	tio	n	A				PS	R	=	0.	.50																					
A :		Г <u>а</u> а				- le : 1 : 4		:			414 4			1																		
Ain	1: 1	lo a	sses	ss th	e sta		and	inte	grity	OI	the	geos	synt	neti	c caj	ppii	ng s	yster	n.													
A ni	ro	ach	• 11	ce tl	10.91	pproac	h pr	onos	ed 1	JV L	one	s &r	Div	on	1009	2																
ΔP	10		• 0			pproae				<u> </u>		s a				5.																
Geo	ome	etrv	:																													
			-																													
						II												1														
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														/	A.	tivo	Wa	daa	/	1												
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\square									1074	/	/			WA	_	1			\"	6	لر		٦									
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					/			Wp		/				+	Ca	V		Ge	om	emb	oran	Ð										
			V	P	assiv	A		1	1	Ер		NA	tano	X																		
				W	edge	e >	/	EA	*	~	_	/		`	NA																	
				/		<u>_</u>	N	ptan	٩ -	/	TA	8			/																	
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							^	P				1	'h) F	Tinit	e sla	ne																
																spe																
Inn	nt Ì	Par	am	eter																												
	er :	soil	ann ann	it u	s veigt	nt (drv) γ,									18		kN/i	m ³													
Cov	ver	soil	s un	iit w	eigh	nt (satu	rate	y d). γ								21		kN/i	m ³													
Cov	er	soil	in:	tern	al sh	near sti	eng	<u>ա), լ</u> h տ	sat							25		Deg	r													
Cov	ver	soil	5 CO	hes	ion.	c		, ,								0		kPa	, .													
Thi	ckn	ess	of c	cove	er so	ils, h									-	1		m														
Hei	ght	ofs	lop	e, H	ł											6		m														
Slo	je a	angl	e, β													14		Deg	ŗ.													
Geo	syı	nthe	tic i	inter	rface	e shear	stre	ngth	s:																							
	Co	ver	Soi	ls/G	eoco	ompos	ite fi	rictio	on a	ngle	ϵ, δ_1					24		Deg	ŗ.													
	Co	ver	Soil	ls/G	eoco	ompos	ite co	ohes	ion	inte	rce	ot, o	ι,			0		kPa														
	Ge	ocoi	npo	osite	e/GN	/ fricti	on a	ngle	, δ ₂							26		Deg	ç.													
	Ge	ocoi	npo	osite	e/GN	1 cohe	sion	inte	rcep	pt, o	ι ₂					0		kPa														
	GM	//B1	ind	ing	laye	r, δ ₃	_									24		kPa														
\square	GN	1/B1	ind	ing	laye	r, α_3										0		Deg	; .									<u> </u>				
									_	_	_	_	_																			
Para	alle	l su	ome	erge	nce	ratio, l	PSR		-	-		-	-).50)											<u> </u>				
Geo	syı	nthe	tic 1	tens	ile s	trengtl	ns:		-	-		-	-			1.0		1										<u> </u>				
\vdash	Ge	otex	tile						_	_						10		k N/1	m									-				
\vdash	Im	m L	LD	PE	Geo	memb	rane		-	-	-	-	-			11		KIN /1	m									-				
							_	-		-	-	-	-		$\left \right $			$\left - \right $														
				1																												

	PROJEC	T Biffa Eye	East	ern E	xtensi	on Sta	bility	y As	sessi	nent	t	
Golder	Job No.	21453458		Made E	By: WY	Ή			Date	2	2/01	/2022
Associates	Ref.	Appendix 6		Checke	ed: BZ				Shee	t:	5	
				Review	ed: BZ				of:		9	
1. Stability of Cover Soils												
Calculated Parameters												
Length of slope, L		24.8	80139	m								
Thickness of water, h _w		().5	m								
Weight of active wedge, W _A		443	.6886	kN								
Weight of passive wedge, W _P		39.9	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		5.01	3476	kN								
a		104	.2228									
b		-164	4.622									
c l		15.8	85418									
		Factor of	of Safe	ty agai	inst cov	er soils	s slidin	g			1.	48
								Ĭ				
2. Integrity of Geosynthetics												
(i) Geotextile												
Mobilised shear stress at upper i	nterface	141	5033	kN								
			.5055	KI								
Shear strength at lower interface		228	87/1	ĿN								
		220	.0741	KIN								
Tension developed in the GT			0	ĿN								
Tension developed in the GT				NIN.				-		_		
Tansila strangth of the GT			10	ĿN						-		
Tensne strengti of the GT			10	KIN						_		
				C . C . 4				-			T (*	
		Faci		Salety a	against	ruptui	e	-			InII	mte
											-	
(II) Geomembrane		+ $+$ $+$ $+$ $+$						-		+		
			07.41	1 3 7						+		
Snear strength at upper surface	+ $+$ $+$ $+$.8/41	КN					\vdash	+		
			5022	1				+	\vdash	+		
Mobilised shear stress at upper i	nterface		.5033	kN								
	+ $+$ $+$ $+$	+ $+$ $+$ $+$ $+$						-		_	-	
Shear strength at lower interface		208	.9286	kN						+	<u> </u>	
		+ $+$ $+$ $+$ $+$							$ \vdash \downarrow $	\square	<u> </u>	
Tension developed in the GM		+ $+$ $+$ $+$ $+$ $+$ $+$	0	kN					$ \vdash \downarrow $	+	<u> </u>	
								-		_	-	
Tensile strength of the GM			11	kN								
		Fact	tor of S	Safety a	against	ruptui	e				Infi	nite
										\perp	<u> </u>	
		\downarrow \downarrow \downarrow \downarrow \downarrow \downarrow										

										PR	OJ	EC	Г	Bi	ffa	Ey	e E	ast	ern	E	xte	nsi	on	Sta	bil	ity	As	ses	sm	ent			
		71		Ga	4	er				Job	o No).	214	4534	458				Mac	de I	By:	WY	ΥH					Da	te:	22	2/01	/202	22
	V	D	Á	SS	OC	iat	es			Ret	f.		Ap	pen	dix	6			Che	cke	ed:	ΒZ						Sh	eet:		6		
																			Rev	view	ved:	ΒZ						of:			9		
Sec	tio	n	A					PS	R	=	1.	00																					
Ain	n:]	Го а	sses	ss th	e sta	abili	ty a	nd i	integ	grity	v of	the	geos	synt	heti	c ca	ppi	ng s	ystei	m.													
Ap	pro	ach	: U	se tl	ne aj	ppro	ach	pro	pos	ed ł	oy J	one	s &	Dix	on,	199	8.																
Geo	ome	etry	:																														
																			1														
																		/	*														
			1														/			/													
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			1			/	/					/				Ca	V			/													
			\mathbf{N}	/	/			ŀ	N _P	/				and	*	/			Ge	om	emt	oran	e										
			X	Pa	issiv	e			/	1	EP		NA	la	×	N																	
			-	M	edge	e \	×	_	EA	-		/				/	•																
					~	_	C	N	tan	\$		1 P	_			`																	
			-					1	/																								
			-				/																										
				_				N	P				7	'h) F	Tinit	te el	one														_	_	
														0)1	m		ope				1												
-				<u> </u>																													
Inp	ut	Par	am	eter	s 												10		1.5.7/	3													
Cov	/er	soil	s ur	iit w	eigh	nt (d	ry),	γ_{dry}	r								18		KIN/	m 3											_		
Сол	/er	soil	s ur	it w	reigh	nt (sa	atur	ateo	ł), γ	sat							21		kN/	m													
Сол	/er	soil	s in	tern	al sł	near	stre	ngt	h,								25		Deg	g.													
Cov	/er	soil	s co	hes	ion,	c											0		kPa														
Thi	ckn	ess	of c	cove	r so	ils, ł	1										1		m														
Hei	ght	of	slop	e, H	[6		m												$ \square$		
Slo	pe a	angl	e, f														14		Deg	<u>z</u> .													
Geo	osyı	nthe	tic	inter	face	e she	ear s	stre	ngth	s:																							-
	Co	ver	Soi	ls/G	eoco	omp	osit	e fr	ictic	n ai	ngle	έ, δ ₁					24		Deg	g.													
	Co	ver	Soi	ls/G	eoco	omp	osit	e co	hes	ion	inte	rce	ot, o	ι			0		kPa														
	Ge	oco	mpo	osite	/GN	1 fri	ctio	n ai	ngle	, δ2							26		Deg	g.													
	Ge	oco	mpo	osite	/GN	I co	hes	ion	inte	rcep	ot, o	2					0		kPa														
	GN	//B1	ind	ing	laye	r, δ3											24		kPa														
	GN	1/B1	ind	ing	laye	r, α_1	;										0		Deg	<u>χ</u> .									1				
				3		,																											
Par	alle	1 511	hm ⁱ	eroe	nce	ratic). P	SR									1.00)															
Ger)SVI	1the	tic	enc	ile s	tren	othe										1.00																
	Ge	otev	tile				5412				-						10		kN/	m								-					_
⊢┤	1m	mI		DE .	l Geo	men	h.	ine	-		-	+	-				11		LNI/	m		-		-					-	-	-+		-
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\vdash		-		-	-							\vdash			-						-												

	PROJEC	T Biffa Eye Eas	tern Extension Stability As	sessment
Golder	Job No.	21453458	Made By: WYH	Date: 22/01/2022
Associates	Ref.	Appendix 6	Checked: BZ	Sheet: 7
			Reviewed: BZ	of: 9
			•	
1. Stability of Cover Soils				
Calculated Parameters				
Length of slope, L		24.80139	m	
Thickness of water, h _w			m	
Weight of active wedge, W_A		4/6.0981		
weight of passive wedge, W_P		44./3114		
Pore pressure perp. to slope, U_n	I	219.979		
Fore pressure in interwedge surface,	Uh	J 242 1965		
Vort pp op passive wedge, N _A		243.1803		
\sim		112 0400		
a h		120.011		
		12 21/35		
		12.21433		
		Factor of Saf	ety against cover soils sliding	1.05
2. Integrity of Geosynthetics				
(i) Geotextile				
Mobilised shear stress at upper in	nterface	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	$\left \right $			
	$\left \right $			
Shear strength at upper surface	+ $+$ $+$ $+$	246.4798		
Mohilized share stress of an	toufast	014.9401		
Noomsed snear stress at upper in	neriace	214.8421		
Shear strength at lower interface	+ $+$ $+$ $+$	225.0001	kN	
		225.0001		
Tension developed in the GM	+ $+$ $+$		kN kN	
Tensile strength of the GM			kN kN	
		Factor of	Safety against rupture	Infinite

							PR	OJ	EC	Г	Bi	ffa	Ey	e E	ast	tern l	Ext	ter	nsion	Sta	abil	ity	As	ses	sm	ent			
		G	J 4	er			Job	o No).	214	4534	458				Made	By	y: `	WYH					Da	te:	22	2/01	/202	22
	7 ľ	lss	OC	iates	.		Re	f.		Ap	pen	dix	6			Checl	ked	l:]	ΒZ					She	eet:		8		
										-	-					Revie	we	d:	ΒZ					of:			9		
																•													
Section	A	1			PS	R	=	0.	.65																				
Aim: To	asse	ess th	e sta	ability a	und :	integ	grity	of	the	geo	synt	heti	c ca	ppiı	ng s	ystem				-	1								
														_															
Approac	ch: U	Jse tl	ne aj	pproach	n pro	opos	ed l	oy J	one	s &	Dix	on,	199	8.				_			1								
																	_	_											
Geometi	ry:																_	_											
																$\mathbf{\lambda}$													
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												/	/				/	-											
										/	/		Ac	tive	We	dge		1											
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					/						/	T				\checkmark													
									/				Ca	V		Gas	\ 	mb											
	\backslash	-	/				1	_			tano	ź				Geol	ner	mbi	ane										
		₹ Pa W	assiv edg	e	/	E	1	EP.		NA		1	NA																
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		-			^	IP																							
					1				1	((b) I	ini	te sl	ope						1	1								
Input Pa	aran	neter	s 											10		1.57/	3												
Cover so	11s u	init w	veigh	nt (dry)	γ_{dr}	y I								18		KIN/M	3												
Cover so	11s u	init w	eigh	nt (satu	rate	$(1), \gamma_{s}$	sat							21		KIN/III		_											
Cover so	115 11	ntern	ai sr	iear stre	engt	.n, φ								25		Deg.	+	-											
Cover so	ns c	ones	ion,	C ile h										1		кра	_	_											
Height o		ne F	1 SO 1											6		m	+												
Slope an	ole	B												14		Deg	+												
Geosynth	netic	inte	face	e shear	stre	nøth	s:							17		Deg.	+												
Cove	r So	ils/G	eoco	omposi	te fr	rictio	n a	ngle	, δ1					24		Deg.	+			1									
Cove	r So	ils/G	eoc	omposi	te co	ohesi	ion	inte	erce	ot, o	ί1			0		kPa													
Geoc	omp	osite	e/GN	1 frictio	on a	ngle.	, δ ₂							26		Deg.													
Geoc	omp	oosite	/GN	1 cohes	ion	inter	rcep	ot, o	l ₂					0		kPa													
GM/	Blin	ding	laye	r, δ ₃										24		kPa													
GM/I	Blin	ding	laye	r, α ₃										0		Deg.													
Parallel s	subn	nerge	nce	ratio, P	SR								(0.65	5														
Geosynth	netic	tens	ile s	trength	s:																								
Geot	extil	e												10		kN/m													
1mm	LLI	DPE	Geo	membr	ane									11	_	kN/m													

	PROJEC	T Biffa Eye	Eastern E	xtension Stabi	lity Assessm	ent
Golder	Job No.	21453458	Made I	By: WYH	Date:	22/01/2022
Associates	Ref.	Appendix 6	Checke	ed: BZ	Sheet:	9
			Review	ved: BZ	of:	9
1. Stability of Cover Soils						
Calculated Parameters						
Length of slope, L		24.80	139 m			
Thickness of water, h _w		0.6	5 m			
Weight of active wedge, W _A		453.	747 kN			
Weight of passive wedge, W _P		41.04	-082 kN			
Pore pressure perp. to slope, U _n		147.6	883 kN			
Pore pressure in interwedge surface,	, U _h	2.11	25 kN			
Force normal to active wedge, N_A		293.0	915 kN			
Vert pp on passive wedge, U _V		8.472	2775 kN			
a		106.6	343			
b		-153.	955			
c c		14.72	.089			
		Factor of	Safety aga	inst cover soils sli	ding	1.34
2. Integrity of Geosynthetics						
(i) Geotextile						
Mobilised shear stress at upper i	nterface	159.4	187 kN			
Shear strength at lower interface		23/ 1	558 LN			
blical strength at lower interface		234.1				
Tension developed in the GT			kN			
Tensile strength of the GT		1() LN			
		E a a fa				Tufinita
		Facto	r of Safety	against rupture		Iminite
(ii) Coomombrono	+ $+$ $+$ $+$	+ $+$ $+$ $+$ $+$		+ + + + +		
		+ $+$ $+$ $+$ $+$				
Shoor strongth at your an and	+ $+$ $+$ $+$		550 1-NT			
snear suengur at upper sufface	+ $+$ $+$ $+$	234.1				
Mobilized cheer stores at a second	ntorfoss	150 4	107 1.1		+ $+$ $+$ $+$	
woomsed snear stress at upper in	merrace	159.4	10/ KN			
Shear strength at lower interface		213.7	501 kN			
	+ $+$ $+$ $+$				+ $+$ $+$ $+$	
Tension developed in the GM	+ $+$ $+$ $+$		kN		+ $+$ $+$ $+$	
	+ $+$ $+$ $+$	+ $+$ $+$ $+$ $+$ $+$				
Tensile strength of the GM	+ $+$ $+$ $+$		l kN			
	+ $+$ $+$ $+$			• • •		
		Facto	r of Safety :	against rupture		Infinite
		+ $+$ $+$ $+$ $+$ $+$				
		+ $+$ $+$ $+$ $+$ $+$				

GCL Capping Analyses



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	The to	ensil ferer	e str ice <u>p</u>	bage	-																										
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Golder	Job N	o.	2145	3458			Μ	[ade]	By:	WY	Ή					Dat	te:	22	2/01	/202	22
Associates	Ref.		Appo	endix	7		C	heck	ed:	ΒZ						She	eet:		2		
							R	eviev	ved:	ΒZ						of:			11		
Section A PSR	=	0																			
Aim: To assess the stability and inte	grity of	the	geosy	ntheti	c cap	ping	syst	em.		1 1											
Approach: Use the approach propos	sed by .	lones	s & D	ixon,	1998				1	1											
Geometry:		_																	-		
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			/		Acti	ive W	edg	~													
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	1-		NATA	no t				10011	em	nane	,										
Vedge F	EP		NA		NA																
C Notar		1 F	3		\backslash																
N _P			(1-		1																
	1 1		(0) Fini	te sio	pe			1	1 1											
Input Parameters							1.2	T 3													
Cover soils unit weight (dry), γ_{dry}		_				18	kí	N/m^2													
Cover soils unit weight (saturated), γ	sat			_		21	KI D	N/m													
Cover soils internal shear strength, ϕ						25	D	eg.	-												
Cover soils cohesion, c						0	kl	2a													
I hickness of cover soils, h		_				1	m														
Slang angle 8		_				0	m														
Geosynthetic interface shear strength						14		eg.													
Cover Soils/Geotevtile friction at	ngle A		+	+	,	24	п	eg													
Cover Soils/Geotextile cohesion	interce	nt α				0	k	ο _β .													
GCL/Blinding laver friction angl	e. δ ₂	p., o.				24	D	eg.													
GCL/Blinding cohesion intercept	α_2					0	kl	Pa													
Parallel submergence ratio, PSR						0															
Geosynthetic tensile strengths:																					
GCL						12	kľ	N/m													
	1 1	1	1		1				1	1							1			1	
	PROJEC	T Bi	ffa I	Eye E	Cast	tern	Ex	ktei	nsio	n S	Sta	bil	ity /	As	ses	sme	ent				
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Golder	Job No.	214534	458			Mao	de B	By:	WY	Н					Dat	te:	22	2/01	/2022	2	
Associates	Ref.	Appen	dix 7			Che	cke	d:	ΒZ						She	et:		3			
						Rev	view	ed:	ΒZ						of:			11			
	<u></u>																				
1. Stability of Cover Soils																					
Calculated Parameters																					
Length of slope, L				24.80	139		m														
Thickness of water, h _w				0			m														
Weight of active wedge, W _A				408.0	841		kN														
Weight of passive wedge, W _P				38.34	098		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				395.9	623		kN														
Vert pp on passive wedge, U _V				0			kN														
a				95.79	194																
b				-200.0)73																
с				19.88	77																
			Fact	tor of	Safe	ety a	igai	nst	cove	er se	oils	slic	ling					1.	98		
2. Integrity of Geosynthetics																					
(i) Geosynthetic Layer No.1																					
				07.00	150		1 3 7														
Mobilised shear stress at upper in	iterface			97.20	/53		kΝ														
				102.9	570		1-NI														
Shear strength at lower interface				192.8.	572		KIN														
Tension developed in the geosyth	atio			0			ĿΝ														
Tension developed in the geosyd				0			KIN.														
Tensile strength of the geosytheti				12			ĿΝ														
				12			KI V														
			Fact	tor of	Safe	etv a	gai	nst	run	ure								Infi	nite		
									<u> p</u>												
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	PROJ	ЕСТ	r Bi	ffa	Eye E	ast	ern F	Cxte	nsic	n	Sta	bili	ty A	sse	ssm	ent			
Golder	Job No).	214534	458	•		Made	By:	WY	Ή			•	Da	ate:	22	2/01	/202	22
Associates	Ref.		Appen	dix	7		Check	ed:	ΒZ					Sh	eet:		4		
							Review	wed:	ΒZ					of			11		
Section A PSR	= 0	.5																	
Aim: To assess the stability and inte	grity of	the g	geosynt	heti	c cappin	ng sy	stem.		1										
Approach: Use the approach propos	ed by Jo	ones	& Dix	on,	1998.			-						_					
														_					
Geometry:														_					
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		/			Ca		Geor	home	rana										
	1		Natano	Ń			0000	ienn.	ane										
Vedge F	EP		NC		NA														
C Notar		Ţβ			\backslash														
			(h) T									_							
			(6)	ini	te stope			1	1										
														_					
Input Parameters					10		1 1 1 3	3						_					
Cover soils unit weight (dry), γ_{dry}					18		$\frac{KIN}{m}$;						_					
Cover soils unit weight (saturated), γ	sat				21		$\frac{KIN/m}{D}$							_					
Cover soils internal shear strength, ϕ					25		Deg.							-					
Cover soils conesion, c					0		кРа							+					
Height of slope U					1		m							_					
Slope angle 8					14		Deg							+					
Geosynthetic interface shear strength	e.				14		Deg.												
Cover Soils/Geotextile friction at	ngle. δι				24		Deg.							-					
Cover Soils/Geotextile cohesion	intercen	ot. α.			0		kPa							+					
GCL/Blinding layer friction angl	$e. \delta_2$., .,			24		Deg.												
GCL/Blinding cohesion intercept	α_2				0		kPa												
Parallel submergence ratio, PSR					0.5														_
Geosynthetic tensile strengths:																			
					12		kN/m												
										_									
$\mathbf{I} $		1																	

	PROJEC	T Bi	ffa E	ye E	last	tern	Ex	tens	ion	Sta	bil	ity A	Ass	ess	me	ent			
Golder	Job No.	214534	58			Mac	le B	y: W	ΥH				Ι	Date	e:	22	2/01	/202	22
Associates	Ref.	Append	lix 7			Che	cked	- 1: B2	Ζ				S	Shee	et:		5		
						Rev	iewe	ed: B	Z				c	of:			11		
1. Stability of Cover Soils																			
Calculated Parameters																			
Length of slope, L			2	4.801	139		m												
Thickness of water, h _w				0.5			m												
Weight of active wedge, W _A			4	43.68	886		kN												
Weight of passive wedge, W _P			3	9.938	352		kN												
Pore pressure perp. to slope, U _n			1	15.15	565		kN												
Pore pressure in interwedge surface,	U _h			1.25	5		kN												
Force normal to active wedge, N _A			3	15.65	551		kN		_										
Vert pp on passive wedge, U _V			5	.0134	476		kN												
a			1	04.22	228				_										
b				164.6	522				-										
c			1	5.854	418			_	_										
															_				
			Facto	or of (Safe	ety a	gain	ist co	ver s	soils	slio	ling			_		1.4	18	
													_						
2. Integrity of Geosynthetics									_										
(i) Geosynthetic Layer No.1									_										
			1	41.50			1 3 1		_										
Mobilised shear stress at upper in	nterface		1	41.50)33		KIN												
Shaan strongth at lawar interface			2		106		I-NI								_				
Shear strength at lower interface			2	.08.92	280		KIN		_										
Tension developed in the geosyt	atio			0			ĿN						_		_				
Tension developed in the geosyd							KIN.		_										
Tensile strength of the geosythet	ic			12			ĿΝ												
				12			NIN.												
			Facto	or of :	Safe	etv a	gain	nst ru	ntur	.е							[nfi	nite	
			1 4000				5		Prui						T				
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Golder	Job 1	Jo.	21453	458	•		Mad	le B	y:	WY	Ή			•		Da	te:	2	2/01	/202	22
Associates	Ref.		Apper	ndix	7		Chee	cked	d:	ΒZ						She	eet:		6		
							Revi	iewe	ed:	ΒZ						of:			11		
Section A PSR	=	1																			
Aim: To assess the stability and inte	grity c	f the	geosyn	theti	c capp	ing s	ystem	n.													
Approach: Use the approach propos	sed by	Jone	s & Diy	kon,	1998.																
Geometry:																					
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					Activ	e We /	dge		1												
		5		WA		Į		h		لر		-									
				L				×				I									
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W		/		Ļ	Ca		Geo	ome	mb	rane	•										
Passive	1 E	,	NAtans	X																	
Wedge	1 And I				NA																
C Nptai	10		3		\backslash																
			(b)	Fini	te slop	e															
			(b)	Fini	te slop	e															
Input Parameters			(b)	Fini	te slop	e															
Input Parameters			(b)	Fini	te slop	e	kN/r	m ³													
Input Parameters Cover soils unit weight (dry), γ _{dry} Cover soils unit weight (saturated), γ	, sat		(b)	Fini		e	kN/r kN/r	$\frac{m^3}{m^3}$													
Input Parameters Cover soils unit weight (dry), γ _{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, φ	sat		(b)	Fini	18	e	kN/r kN/r Deg.	m^3 m^3													
Input Parameters Cover soils unit weight (dry), γ _{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, φ Cover soils cohesion, c	, sat		(b)	Fini	18 18 22 0	e	kN/r kN/r Deg. kPa	$\frac{m^3}{m^3}$.													
Input Parameters Cover soils unit weight (dry), γ _{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, φ Cover soils cohesion, c Thickness of cover soils, h	sat		(b)	Fini	te slop	e	kN/r kN/r Deg. kPa m	$\frac{m^3}{m^3}$.													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H	sat		(b)		18 22 22 00 11 6	e	kN/r kN/r Deg. kPa m m	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β	sat		(b)		te slop	e	kN/r kN/r Deg. kPa m m Deg.	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength	sat				te slop 18 22 23 00 11 66 14	e	kN/r kN/r Deg. kPa m Deg.	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a	sat				te slop 18 22 00 11 66 14 24	e	kN/r kN/r beg. m Deg. Deg.	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion	is: ngle, &				te slop	e	kN/r kN/r Deg. kPa Deg. Deg. kPa	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angle	sat sat sst ngle, δ interce e, δ_2				te slop	e	kN/r kN/r beg. kPa Deg. kPa Deg.	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angle GCL/Blinding cohesion intercep	r_{sat} r_{sat}				te slop 18 22 23 00 11 66 14 24 00 24 00 24 00	e	kN/r kN/r Deg. kPa Deg. kPa Deg. kPa	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angle GCL/Blinding cohesion intercep	sat sat sst ngle, δ interce e, δ_2 t, α_2	1 1 1 1 1 1 1 1 1 1 1 1 1 1			te slop 18 22 00 11 66 14 24 00 24 00 24 00 24 00 24 00 24 00 24 00 14 15 15 15 15 15 15 15 15 15 15	e 33 1 5 4 4 4 4	kN/r kN/r beg. kPa Deg. kPa Deg. kPa	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angle GCL/Blinding cohesion intercep Parallel submergence ratio, PSR Gaosynthetic torsile stress of the	r_{sat}	1 1 1 1			te slop	e	kN/r kN/r Deg. kPa Deg. kPa Deg. kPa	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angle GCL/Blinding cohesion intercep Parallel submergence ratio, PSR Geosynthetic tensile strengths:	$rac{sat}{sat}$				te slop	e	kN/r kN/r beg. kPa m Deg. kPa Deg. kPa beg. kPa	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angle Geosynthetic tensile strengths: Geosynthetic tensile strengths:	$rac{sat}{sat}$	1 ppt, 0			te slop	e	kN/r kN/r Deg. kPa Deg. kPa Deg. kPa kPa kN/r	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angl GCL/Blinding cohesion intercep Parallel submergence ratio, PSR Geosynthetic tensile strengths: GCL	r_{sat}				te slop 18 2 2 2 0 1 1 2 2 0 1 2 2 0 1 2 2 0 1 1 2 2 1 1 2 2 2 2 2 2 2 2 2 2 2 2 2	e	kN/rr kN/rr beg. kPa m Deg. kPa Deg. kPa kPa kN/r	m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angle GCL/Blinding cohesion intercep Parallel submergence ratio, PSR Geosynthetic tensile strengths: GCL	$rac{sat}{sat}$	1 ppt, 0			te slop	e	kN/r kN/r Deg. kPa Deg. kPa Deg. kPa kPa kN/r	m ³ m ³													
Input Parameters Cover soils unit weight (dry), γ_{dry} Cover soils unit weight (saturated), γ Cover soils internal shear strength, ϕ Cover soils cohesion, c Thickness of cover soils, h Height of slope, H Slope angle, β Geosynthetic interface shear strength Cover Soils/Geotextile friction a Cover Soils/Geotextile cohesion GCL/Blinding layer friction angl GCL/Blinding cohesion intercep A Parallel submergence ratio, PSR Geosynthetic tensile strengths: GCL	$rac{1}{3}$ sat				te slop	e	kN/rr kN/rr kPa m Deg. kPa Deg. kPa kPa kPa kN/r	m ³													

	PROJECT	Biffa Eye Eastern Extension Stability Assessme	ent
Golder	Job No.	21453458 Made By: WYH Date:	22/01/2022
Associates	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	
		112.0499	
b		-129.011	
c 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 in	nterface	214.8421 kN	
Shear strength at lower interface		225.0001 kN	
Tension developed in the geosyth	netic	0 kN	
Tensile strength of the geosytheti	c	12 kN	
		Factor of Safety against rupture	Infinite
+ + + + + + + + + + + + + + + + + + +			
+ + + + + + + + + + + + + + + + + + +			
+ + + + + + + + + + + + + + + + + + +			

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Golder	Job No).	214534	458		N	Made	By:	WY	Н					Dat	e:	22	2/01	/202	22
Associates	Ref.		Appen	dix	7	C	Check	ed:	ΒZ						She	et:		8		
						F	Review	wed:	BZ						of:			11		
Section A PSR	= 0.	65																		
Aim: To assess the stability and inte	grity of	the g	geosynt	heti	c capping	g sys	stem.		1 1											
Approach: Use the approach propos	sed by Jo	ones	& Dix	on,	1998.															
Geometry:																				
							$\mathbf{\lambda}$													
								2	_											
					Active W	/edg														
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Wa		/		↓	Ca		Geon	hemb	orane											
Pagging	En		NAtano	X								_								
Wedge EA					NA							_								
C Nptai	10	Ţβ	<u> </u>		\backslash							-								
												-								
NP NP			(b) F	Finit	te slone							-								
			(-)-																	
Innut Doxomotors																				
Cover soils unit weight (dry) y.					18	k	N/m ³													
Cover soils unit weight (ary), _{fdry}					21	k	$\frac{N}{m^3}$													
Cover soils internal shear strength d	sat				21	Г	Deg													
Cover soils cohesion, c	·				0	k	Pa													
Thickness of cover soils, h					1	n	n													
Height of slope, H					6	n	n													
Slope angle, β					14	Ι	Deg.													
Geosynthetic interface shear strength	ns:																			
Cover Soils/Geotextile friction a	ngle, δ_1				24	Ι	Deg.													
Cover Soils/Geotextile cohesion	intercep	ot, α	1		0	k	Pa													
GCL/Blinding layer friction angl	e, δ ₂				24	Ι	Deg.													
GCL/Blinding cohesion intercep	t, α_2				0	k	Pa													
Parallel submergence ratio, PSR					0.65									_						
Geosynthetic tensile strengths:																				
GCL					12	k	N/m							-				$ \rightarrow $		
				-	$\left \right $	_	_		$\left \right $					-				-+		_
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	PROJECT	Biffa Eye Easte	rn Extension Stability As	ssessment
Golder	Job No.	21453458 N	Made By: WYH	Date: 22/01/2022
Associates	Ref.	Appendix 7	Checked: BZ	Sheet: 9
		R	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		
с		14.72089		+ + + + + + + + + + + + + + + + + + +
		Factor of Safet	y against cover soils sliding	1.34
2. Integrity of Geosynthetics				
(1) Geosynthetic Layer No.1				
Mobilized sheer stress at upper i	tarfaco	150 /197	1-NI	
Moonised shear stress at upper in	lierrace	139.4187	KIN	
Shear strength at lower interface		213 7501	kN	
Shear strength at lower interface		215.7501		
Tension developed in the geosyt	netic	0	kN	
Tensile strength of the geosythet	ic	12	kN	
		Factor of Safet	y against rupture	Infinite
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PROJECT	Biffa Eye Eastern Extension Stability Assessment	
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Job No. Ref.

21453458 Made By: WYH Checked: BZ Appendix 7 Reviewed: BZ

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interface shear strength corresponding to $\delta = 10.7^{\circ}$ and $\alpha = 26.7$ 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.

no data terreta con con	n na dosin	Interfa	ace shear st	rength para	meters	e to data
Interface	Re-	Peak	and to each		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	aptrate to	apin <u>r</u> acte	ng kadang in

 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 $\delta = 13.1^{\circ}$ and $\alpha = 17.9$ kPa with an R² 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 $\delta = 15.4^{\circ}$ and $\alpha = 4.1$ 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 $\delta = 19.9^{\circ}$ and $\alpha = 30.1$ 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



Job No.	21453458
Ref.	Appendix 7

Made By:	WYH
Checked:	BZ
Reviewed:	BZ

Fibre-reinforced Geosynthetic Clay Liner (GCL)





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Phone: +49 5743 41-0 · Fax: +49 5743 41-240 E-Mail: info@naue.com · Internet: www.naue.com

Bentofix® NSP 4900

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 pov	vder):		
Mass per unit area	EN 14196 (<i>P</i> 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 (p 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***	≥ 60
		N/m	≥ 360
Static puncture strength	EN ISO 12236 / ASTM D 6241	N	2,000
Permeability / Hydraulic Conductivity	DIN 18130 / ASTM D 5887	m/s	2 x 10 ⁻¹¹
Index Flux	DIN 18130 / ASTM D 5887	(m³/m²)/s	5 x 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

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Th	ickn	less	of v	vate	r h											2.	0			m													
We	eigh	t of	acti	ve v	ved	w De V	N.									40	8 08	841		kN													
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b								<u> </u>								-18	34.2	.43															
c																18	.047	25															
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	C				— –	Jo	b Nc).	214	4534	458				Ma	de l	By:	W	ΥH			·		Dat	e:	09)/02	2022	2
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Section	Α				PSR	=	0	.5																					
Aim: To	assess	the s	stabil	ity o	f rest	oratio	on so	oils p	olac	ed a	lbov	e the	e lov	w po	erme	eabi	ility	cla	y ca	p.									
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Cover soi	ile unit	ers	aht (d	1mz)	~								18		ĿΝ	$/m^3$													
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Cover soi	ils inte	rnal (shear	stre	noth	/sat							21																
Cover soi	ils coh	esion		Suc	ingui,	Ψ							0		kPa	<u>g.</u> 1													
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Height of	f slope	. H											6		m														
Slope and	gle, β	, -								1			14		Des	g.											\neg		
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Cover	r Soils	/Clay	/ Cap	fric	tion a	ngle,	δ_1						22		Deg	g.													
Cover	r Soils	/Clay	/ Cap	o coh	esion	inter	cept	, α ₁					0		kPa	ì													
Parallel s	ubmer	genc	e rati	o, P	SR								0.5																
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1. Sta	ıbilit	y of	Co	ver	Soil	S																										
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Lengt	h of	slop	e, L	,											24.	.801	39		m													
Thick	iness	of v	vate	r, h	N											0.5			m													
Weigl	ht of	acti	ve v	vedg	ge, V	VA									443	3.68	886		kN													
Weig	ht of	pas	sive	wee	dge,	W _P									39.	.938	352		kN													_
Pore p	press	ure	perp	o. to	slop	be, I	U _n								11:	5.15	65		kN	_												_
Pore p	press	ure	in ir	nterv	wedg	ge s	urfa	ce,	U _h					<u> </u>	!	1.25	5	<u> </u>	kN													_
Force	norr	nal	to ac	ctive	e we	dge	, N _A	\							315	5.65	51		kN	_												_
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Section	l	A					PS	R	=		1																					
Aim: T	o as	ses	s th	e sta	abili	ty o	f re	stora	atio	n so	ils p	olac	ed a	bov	e th	e lo	w p	erm	eabi	lity	clay	/ ca	p.	1								
Approa	ach:	Us	e th	ne aj	ppro	ach	pro	pos	ed b	y Jo	ones	&	Dix	on,	1998	8.	1				1				1					\vdash		
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Innut F	Para	me	ter	6																												
Cover s	oils	un	it w	s eigł	nt (d	rv).	γ									18		kN	m^{3}													
Cover s	oils	un	it w	eigh	$\frac{n}{s}$	atura	ated). γ.	at							21		kN/	m^3													
Cover s	oils	int	erna	al sh	near	stre	ngtl	<u>,,,,,</u> 1. φ	aı							25		De	σ.													
Cover s	oils	col	hesi	on	c			γ								0		kPa	⊃. I													
Thickne	ess o	ofc	ove	r so	ils. 1	h										1		m														
Height	of s	lon	e, H	[, 1	-										6		m												$ \uparrow $		
Slope a	ngle	e, β	,													14		Des	g.													
Interfac	e sh	iear	stre	engt	ths:													- 2														
Cov	ver S	Soil	s/Cl	lay (Cap	fric	tion	ang	jle,	δ_1						22	I	Deg	g.													
Cov	ver S	Soil	s/Cl	lay (Cap	coh	esic	on in	terc	ept	, α ₁					0		kPa	ı													
Parallel	sub	me	rge	nce	ratio	o, PS	SR									1																
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														Rev	iew	ed:	ΒZ						of:			7		
						L								1														
1. Stabili	ty of	Co	ver	Soils																								
Calculate	ed Pa	arar	nete	ers																								
Length of	slop	e, L	,								24	.801	139		m													
Thickness	s of v	vate	r, h _v	v								1			m													
Weight of	f acti	ve v	vedg	ge, W _A							47	6.09	981		kN													
Weight of	fpas	sive	wee	dge, W	P						44	.731	14		kN													
Pore pres	sure	perp	o. to	slope,	U_n						21	19.9	79		kN													
Pore pres	sure	in ir	nterv	wedge	surfa	ice, L	J _h					5			kN													
Force nor	mal	to ac	ctive	e wedg	e, N	A					24	3.18	365		kN													
Vert pp o	n pas	ssive	e we	dge, U	V						20).05	39		kN													
a											11	2.04	199															
b											-1	19.2	.88															
c											11	.084	401															
										Fa	ctor	• of	Safe	ety a	gai	nst	cov	er s	oils	slid	ing					0.9	96	
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APPENDIX SRA9

Leachate Extraction System Analyses



	PROJEC	T Biffa Eye East	ern Extension Stability As	sessme	nt
Golder	Job No.	21453458	Made By: WYH	Date:	22/01/2022
Associates	Ref.	Appendix 9	Checked: BZ	Sheet:	1
Associates			Reviewed: BZ	of:	7

Ain	n: Est	abl	ish the	e sta	bilit	y ar	nd s	ervi	cebi	ility	of t	the	eac	hate	ext	ract	ion	and	moi	nito	ring	wel	ls.						
Bac	kgro	unc	1: The	e lea	chat	te ex	ctrac	ctior	n we	ells	com	pris	se	0	.9	m i	nter	nal	diar	nete	er,								
rein	force	d co	oncret	te cł	namł	ber.																							
The	e base	co	mpri	ses a	a	30	00	mm	thi	ck,	30	000	mn	1 sq	uare	e cor	ncre	te sl	ab.										
The	leach	nate	well	will	be	built	t up	witl	n th	e wa	aste	, wi	th a	ma	xim	um l	neig	ht o	f 23	.0 n	n (in	clu	ling	1.0	m				
of d	lraina	ge g	gravel	on	top	of tł	ne sl	lab a	nd	1.0	m o	of re	stor	atio	n so	ils).													
Ар	proac	h:	Asses	ss th	e be	arin	ig ca	apac	ity	and	dif	fere	ntial	l set	tlen	nent	und	er lo	oadi	ing.									
							-																		-				
Ass	umpt	ion	IS:																1.5.7	, 3									
	Unit	wei	ght of	cor	ncret	e, γ	conc									=	2	4	kN/	$\frac{m}{\sqrt{3}}$									
	Unit	wei	ght of	cla	y , γ _c	lay										=	1	9	kΝ/	$\frac{m^2}{\sqrt{3}}$									
	Unit v	wei	ght of	f gra	vel,	γ _{grav}	vel									=	1	8	kΝ/	$\frac{m}{\sqrt{3}}$									
	Unit v	wei	ght of	res	tora	tion	SO1	ls, γ _r	est							=	1	8	KN/	$\frac{m}{\sqrt{3}}$									
	Unit v	wei	ght of	t wa	ste, '	Ywast	e 1.		. 1							=	1	0	KN/	m									
	Shear	' str	ength	oft	he c	lay	line	r (to	otal	stre	ss),	c _u				=	2	0	kPa 1 D	a									
	Shear	str	ength	of t	the c	lay	line	r (et	tec	tive	stre	ess),			c'	=	-	2	kPa 1	a									
	F · /·			1 /				1							φ′	=	2	6 2	deg	grees	5								
	Fricti Wast	$\frac{\text{on}}{2}$	angle	betv	keet K	$\frac{1}{6}$	$\frac{1}{5}$ ste	$\frac{1}{2}$	con	lcret	te, d	0=				_	1	2	deg	grees	5								
	wasu				IX _{W2}	ste	h /C	v)								_	U	.4							-				
Cal	aulat	ion																											
Cal	$\frac{culat}{1 \text{ Lo}}$	odi	S: ng fr	0.000	vori	0116	0.01	nno	non	te																			
	1. LU	aui	ing in		v a 1 1	ous	COL	npo	nen	15															-				
	(a) Se	lf v	veigh	t of	con	rete	e ch	amh	er																				
	Interr	nal (diame	ter	=	neu	0.9	amo	m																				
	Wall	thic	ckness	5	=		0.1		m																				
	Exter	nal	diam	eter	=		1.1		m																				
	Final	hei	ght		=		21.5	5	m																				
	Waste	еH	eight		=		23		m																				
	Unit	wei	ght of	fcor	ncret	e	=	2	4	kN	$/m^3$																		
		T		_																									
	Load	= (π/4)h	(D^2_e)	- D	² _i)γ _c	onc																						
	Load	=		162.	1	kN																							
	(b) C	onc	rete s	lab	load	ing																							
	3	;	х		3	-	m																						
	Thick	nes	ss =		0.3		m																						
	Unit v	wei	ght of	f cor	ncret	e	=	2	4	kN	$/m^3$																		
	Load	= 1	/olum	ne x	γ _{conc}	;																							
	Load	=		64.8	8	kN																						 	

	PROJE	CT J	Biffa	Eye	e Ea	sterr	E E	xte	nsion	Sta	bili	ty .	As	sessn	ner	nt		
Golder	Job No.	2145	53458			Ma	de I	By:	WYH					Date:		22/01	1/202	22
Associates	Ref.	App	endix	8		Ch	ecke	ed:	ΒZ					Sheet		2		
Associates						Re	view	ved:	ΒZ					of:		7		
Calculations:																		

Ca	lcula	ntio	ns:																									
	Loa	din	ig fi	rom	va	riou	s co	mn	one	ents	(Co	ont'	1.)															
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	(c) ¹	Was	ste 1	load	on	extr	acti	on s	slab																			
	Slak	n ar	ea =	=			aon	(9	m^2																		
	Pine	- an	ea =	= π γ	хD	² / 4	=	0	95	m^2																		
	1 Ip	2 11	cu	10 2	D_e	7 -		0.	//																			
	Loa	d =	(s]	ah a	rea	nir	A 11	rea)	v h	eigh	nt v	24																
	Lua	u –	(514	aU a	ica ·	- piţ		1	А II 051	a digit	11 A	Y wast	e															
	Loa	a =						1,	,851	.4	KIN														_			
-	(1)	~		1	1				1.1																 _			
	(d) ($\frac{Gra}{d}$	vel	loa	d on	ext	ract	10n	slat) niale	200																	
		u –	(512			- piţ		iea)	x u	пск	nes	sχγ	grave	1											_			
-	Thi	ckn	ess	of C	irav	rel			1	m																		
-	Loa	d =						1	144.	9	kN																	
	(e) (Сар	ano	d Re	esto	ratic	n lo	oad o	on e	extra	actio	on sl	ab															
	Loa	d =	(sla	ab a	rea	- pip	be an	rea)	x ((cap	thi	ckne	ess 2	κ γ _{caj}	p) +	(res	tora	tion	thic	ckne	ess x	γ_{res}	_{st}))					
	Min	era	l Ca	ap tł	nick	ness		=	-	0	m																	
	Res	tora	itioi	n Tł	nick	ness		=		1	m																	
	Loa	d						=]	144.	9	kN																
	(f) N	Veg	ativ	ve sk	cin f	ricti	on l	load	ling	on	con	crete	e pip	be														
	NSI	F is	giv	en b	yσ	h'tar	ıδ, v	when	re σ	' =	Kw	aste.C																
	NSF	7 =	(K	waste	$\cdot \sigma_{vn}$		tanδ	5)/2		=	1	1.3	-	kPa	a													
				waste		lux																						
	Loa	d =	NS	F x	sur	face	are	a																				
	Loa	d =	NS	F x	πх	Ext	erna	al di	ame	eter	x to	tal l	neio	ht														
	Loa	d =	115	5	840	<u>ראת</u> ז	ι ν																					
	Loa	u			540.		KI V																					
	(m)]		din	a of		ste	can	rec	tors	tion	50	ile a	nd c	trave	el or	nlv												
	(g)	LUa	um	g 01	wa	sic,	cap,	, 103			1 30	115 a		51410		шy												
	Loa	d =	(he	ight	tvv) + ((thic	- - kne	Dec v	~ ~)+	(thi	ckne	000 1	~ ~) +	(th	ickn	000	v V							
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	Date: Sheet: of:

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Su	mm	ary	of loa	adir	ıgs																				
Ele	me	nt					<u> </u>		E	xtrac	tion	point													
							<u> </u>																		
Co	ncre	te cl	namb	er s	elf we	eigh	t				162.1	kN	[
Co	ncre	te sl	ab								64.8	kN	[
Wa	iste	on sl	ab							1	,851. 4	4 kN	[
Gra	avel	on s	lab								144.9	kN	[
Caj	p an	d Re	stora	tior	1 soils	on	slab				144.9	kN	[
Ne	gati	ve sk	in fr	ictic	on					1	840.2	kN	[
Tot	tal l	oad								3	,208.3	3 kN													
Exp	pres	sed a	is a p	oress	sure						356.5	kP	a												
2.1	Bea	ring	capa	icity	y																				
(i)	Tot	al st	ress																						
The	e be	aring	g cap	acit	y (qf)	oft	the (Clay	liner l	senea	th the	e squa	re sl	lab i	n to	tal s	stres	s te	rms	can	be				
exp	oress	sed a	s:																						
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APPENDIX SRA10

Leachate Pipework Deflection Analyses





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	PROJECT	Biffa Eye East	ern Extension Stability As	sessme	nt
	Job No.	21453458	Made By: WYH	Date:	22/01/2022
	Ref.	Appendix 10	Checked: BZ	Sheet:	4
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Job No. 21453458 Ref. Appendix 10

Made By: WYH Checked: BZ Reviewed: BZ
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 Job No.
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		PROJEC	г Biffa	Eye Eas	tern Ex	xtension Stability	Assessme	ent
Gold	ler	Job No. Ref	2145345	58	Made B	By: WYH	Date:	22/0
SSOC	iates	Kei.	Appendix	10	Review	ved: BZ	of:	5
High > 95% Std. Proctor >70% Rel. Den.	21,000	21,000	14,000	14,000 <i>E</i> NR		fill; NR = Not recom-	amic loads, or beneath	
Moderate 85–95% Std. Proctor 40–70% Rel. Den.	21,000	14,000	7,000	$7,000^{E}$ NR		nmended for initial back ³ (ASTM D-698)	ler heavy dead loads, dyn cal Engineer before using	
Slight < 85% Std. Proctor ^c < 40% Rel. Den. ^D	21,000	7,000	NR	NR NR		large rocks are not recor). g about 598,000 joules/m	They are not suitable und ım. Consult a Geotechni	
Dumped	7,000	NR	NR	NR NR		arth, debris, and l GM, GC, GC-SC est standards usin	ry initial backfill. ⁷ htly dry of optimu	
Soil type for pipe bedding material (Unified Classification System ⁴)	Crushed rock: manufactured angular, granular material with little or no fines (6 to 38 mm)	Coarse-grained soils with little or no fines: GW, GP, SW, SP ^B containing less than 12 percent fines (max. particle size 38 mm) Coarse-grained soils with fines:	GM, GC, SM, SC ^B containing more than 12 percent fines (max. particle size 38 mm) Fine-grained soil (LL < 50): Soils with medium to no plasticity CL, ML-CL, with more	than 25 percent coarse-grained particles Fine-grained soils (LL > 50): Soils with high plasticity CH, MH, CH-MH Fine-crained soils (T1 < 50). Soils with	medium to no plasticity CL, ML, ML-CL with less than 25 percent coarse-grained particles	ils OL, OM, and PT as well as soils containing frozen e r use per ASTM D-2321; LL = Liquid Limit. esignation D-2487 rederline soil beginning with some of these symbols (i.e., roctor based on laboratory maximum dry density from to Density per ASTM D-2049.	ne circumstances Class IV(a) soils are suitable as prima able. Compact with moisture content at optimum or slig ter Howard [14].	
Class ASTM D-2321	Ι	II II	IV(a)	IV(b)		Organic soi mended for ^A ASTM De ^B Or any boi ^C Percent Pr ^D Relative D	^E Under son the water ta Source: Aft	

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