REPORT

Sandsfield Gravel Company Ltd

Milegate Eastern Extension Quarry and Landfill

Stability Risk Assessment

Submitted to:

Sandsfield Gravel Company Ltd.

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1.0 INTRODUCTION

1.1 Report Context

Sandsfield Gravel Company Ltd (‘Sandsfield’) has requested that Golder, member of WSP UK Ltd (‘Golder’) prepares an Environmental Permit Variation Application (hereafter referred to as the ‘variation application’) for its Milegate Extension landfill at Catwick Lane, Brandesburton, Driffield, East Yorkshire, YO25 8SA (the ‘Site’).

The landfill is currently authorised by Environmental Permit (EP) EPR/BX1942IX which was issued by the Environment Agency (EA) in 2006 and last varied and consolidated by the EA in February 2020 (Variation Notice V003). The EP allows Sandsfield to dispose of non-hazardous waste under the listed activity of Section 5.2 Part A(1)(a) of the Environmental Permitting (England and Wales) Regulations and the Site can accept up to 75,000 tonnes of waste per year for landfilling.

Sandsfield proposes to extend the existing Site into the neighbouring field to the east (the ‘Eastern Extension’) which is currently in agricultural use. This report details the Stability Risk Assessment (SRA) undertaken for the proposed Eastern Extension. The report will address issues relating to the stability of the basal lining system, the sidewall lining system, the waste mass and the capping system. The stability assessment 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” (Environment Agency, 2003). A detailed description of the installation is presented in **Environmental Setting and Installation Design** (ref. 20148978.632), but a brief description is given here.

1.1.1 Outline of Installation

The Site is located approximately 1 km southeast of the village of Brandesburton, East Yorkshire and is centred on National Grid Reference (NGR) TA 131 472. The Site is bound to the north by open fields and the Moor Main Drain, to the south and east by the Milldam Beck, and to the west by another landfill, Milegate landfill (closed). Access to the Site is obtained from Catwick Lane. Much of the surrounding area has been worked for the extraction of sand and gravel, and this has resulted in a number of pits that have been restored to ponds or have been utilised as landfill sites.

The Site lies in an area of relatively flat land, with ground elevations varying from 5 to 15 m AOD. Ground levels across the Site typically fall gently to the south and west towards the Milldam Beck, which lies at an approximate elevation of 5 m AOD.

1.1.2 Site Setting

1.1.2.1 Regional Geology

An indication of the regional geology has been obtained from the following published sources:

- 1:50,000 scale British Geological Survey geological map Sheet 72 for Beverley; and
- 1:50,000 scale British Geological Survey geological map Sheet 73 for Hornsea.

The maps indicate that the drift deposits in the region are dominated by post-glacial and glacial deposits consisting of estuarine clay and silt, alluvial clay and silt, peat, dry valley gravel, windblown sand, glacial lake deposits, glaciofluvial sand and gravel, and glacial Till. The drift deposits overlie the Cretaceous Chalk Group, which comprises the Flamborough Chalk Formation (white flintless chalk with thin marl beds) and the Welton and Burnham Chalk Formations (white flinty chalk with thin marl beds).

A summary of the regional stratigraphic sequence presented on the geological maps is given in Table SRA1.

Table SRA1: Regional Stratigraphic Sequence

Age	Formation	Description
Quaternary	Post-glacial deposits	Estuarine clay and silt, alluvial clay and silt, and peat.
	Glacial and post-glacial deposits	Dry valley gravels and blown sand.
	Glacial deposits	Vale of York glacial lake deposits including the 25-Foot and 100-Foot Drift, comprising clay and silt, sand and gravel, underlain by glacio-fluvial sand and gravel.
	Glacial deposits	Undifferentiated sand and gravel overlying stony clay Till.
Cretaceous	Flamborough Chalk Formation	White flintless chalk with thin marl beds. Thickness indicated on the geological map as approximately 200 m.
	Welton and Burnham Chalk Formations	White flinty chalk with thin marl bands. Thickness indicated on the geological map as approximately 180 m.
	Ferriby Chalk Formation	Grey to red marly chalk. Thickness indicated on the geological map as approximately 25 m.
	Hunstanton Chalk Formation	Brick red chalk. Thickness indicated on the geological map as approximately 5 m.

1.1.2.2 Local Geology

The published geological maps indicate that the southern part of the Site is underlain by drift deposits comprising undifferentiated glaciofluvial sand and gravel. The maps indicate that the sand and gravel are replaced by Till along the northern edge of the current Site, and within the northwest and northeast corner of the Eastern Extension.

A review of the borehole logs from intrusive investigations at the Site (see Section 0) has been undertaken to confirm and refine the geology indicated on the published geological maps. The borehole logs indicate that the geology beneath the current Site corresponds well with that indicated on the geological maps. The southern part of the Site is immediately underlain by sand and gravel, then Till, and Chalk is present at depth. Boreholes located to the northeast of the current site did not intercept any sand and gravel, with topsoil being immediately underlain by Till. These findings had been further confirmed by inspection of the geological exposures resulting from the quarrying works and operational experience. Borehole logs from the Eastern Extension indicate geological continuity between the currently operating site and the planned extension. It is underlain by an identical sedimentary sequence with the Chalk present at depth and the mineral absent in the northern peripheries.

Within the footprint of currently operating Site, the sand and gravel deposits have been mostly removed as part of the quarrying works that have taken place and the landfill therefore lies directly on the Till. An analogous approach will be applied to the Eastern Extension.

1.1.2.3 Previous Site Investigations

Several intrusive investigations have been historically completed at the site, and these are summarised below.

- Site Investigation Services completed an investigation in 1988. Six boreholes were advanced to a maximum depth of 15 m. All borings were in the western area of the Site. The locations were originally referred to as borings ‘1’ to ‘6’, but these have since been re-named as SI 1 to SI 6.
- Site Investigation Services completed a second investigation in 1998. Seven borings were advanced to a maximum depth of 15 m. The borings were located to north of the Site (borings ‘1a’ and ‘2a’) and in the eastern area of the existing Site (borings ‘3a’ to ‘7a’). The investigation locations have since been re-named as SI 7 to SI 13.
- A soils resource survey was completed in 1999 by an unknown contractor. Thirteen borings were advanced to a maximum depth of 1 m in order to characterise the soil covering the Site and assess its suitability for use as construction materials for the proposed landfill development. The borings were located in a grid with 100 m spacing.
- Site Investigation Services completed an investigation in 1999. Six borings were advanced to a maximum depth of 30 m (MB1 to MB6). All borings were installed with HDPE pipework such that groundwater could be monitored. Boreholes MB1 and MB6 were screened in the Chalk, MB2 was screened into waste in the adjacent Milegate Landfill, and the remaining boreholes were screened in the Upper and Lower Sand deposits.
- Golder Associates (UK) Ltd installed four boreholes at the site during 2004 for the purpose of monitoring groundwater levels in the Upper and Lower Sand units. The borings were located in pairs at two locations along the southern boundary of the Site (i.e. adjacent to the Milldam Beck). One borehole in each pair was screened in the Upper Sand and the other was screened in the Lower Sand.
- Sandsfield-commissioned investigation took place in autumn 2019 which involved installation of six investigation boreholes. Four of these were within the bounds of the planned Eastern Extension (BH01 to BH04) and two were drilled on the eastern bank of Milldam Beck (initially named UBH01 and UBH02 and subsequently renamed to BH05 and BH06). The maximum drilled depth reached -22.49 m AOD and two boreholes located across the Milldam Beck proved the entire thickness of the Till.

Borehole logs relevant to the Eastern Extension site investigation are illustrated on **Drawing ESID9B – Eastern Extension Site Investigation Infrastructure**.

1.1.2.4 Description of Strata

Sand and Gravel

The sand and gravel unit that immediately underlies the Site comprises three distinct layers: an upper sand unit (locally referred to as the ‘Upper Sand’) and a lower sand unit (locally referred to as the ‘Lower Sand’) that are separated by a thin discontinuous clay layer (locally referred to as the ‘Middle Clay’).

The Upper Sand unit is generally described in the borehole logs as being fine brown clayey or silty sand, with some traces of fine gravel reported nearer the upper part of the unit. Within the footprint of the currently operating Site, the base of the unit lies at elevations ranging from 3.1 to 7.5 m AOD, with an average elevation of 5.5 m AOD. Adjacent to the Milldam Beck, the unit has been shown to be less than 1.5 m thick. Similar base depths were observed in boreholes proven in the Eastern Extension with elevations ranging from 1.79 m AOD to 7.08 m AOD.

The Middle Clay is described in the borehole logs as being a soft or firm orange-brown and dark brown silty clay. Some of the logs describe the clay as being laminated. Borehole logs from the currently operating Site indicate that the base of the clay lies at elevations ranging from 0.2 to 5.8 m AOD, with an average elevation across the site of 4.0 m AOD. Where present, the clay ranges in thickness from 0.5 to 4.5 m, with an average thickness of 1.5 m. The clay is thickest in the eastern part of the Site and thins towards the west. In some investigation locations outside the Eastern Extension, it was found to be absent.

The Lower Sand unit is described as being a fine to coarse sand with fine to medium gravel. In some locations it is reported as being silty or containing cobbles. At the currently operating site, the base of the unit lies at elevations ranging from -4.56 to 3.9 m AOD, with an average elevation of -0.11 m AOD. The thickness ranges from 2.9 to 6.8 m, with an average thickness of 4.8 m. Where reached, by the recently drilled investigation boreholes in the Eastern Extension, the base of the Lower Sand was found at depths ranging from -3.08 m AOD to -0.42 m AOD.

Till

The Till unit underlies the Lower Sand unit beneath the Site, and outcrops to the north of the Site. It is described as being soft to stiff grey silty slightly sandy clay mixed with some assorted gravel. In some locations, the logs indicate that the clay becomes sandier, however these sandy units are not continuous across the Site. The full thickness of the Till has been proven in several boreholes at the current Site, and ranges from 13.1 m in the northwest of the Site to 17.2 m in the southeast of the Site. The two boreholes drilled across the Milldam Beck as a part of the Eastern Extension site investigation indicate Till thicknesses of approximately 14 m.

Chalk

The Chalk beneath the existing landfill has been proven in six boreholes, at approximate elevations between -15 m AOD and -17 m AOD (depths of between 22 and 28 metres below ground level). As part of the Eastern Extension site investigation, the Chalk was proven in BH05 and BH06 at elevations of -16.39 m AOD and -14.42 m AOD, respectively, consistent with the existing conceptual model.

The Chalk is described as being a soft to firm greyish-white or white putty chalk with occasional flints. The Chalk penetrated by the investigation boreholes is not fractured.

The geological description of the Chalk at the Site is in accordance with the description provided in the Aquifer Properties Manual (BGS, 1997), which describes the Flamborough Chalk as being soft white chalk with thin marl beds and negligible flint.

1.1.2.5 Hydrogeology

Groundwater elevations are monitored regularly. The water levels measured at the Site are provided as a part of the **Hydrogeological Risk Assessment** (ref. 20148978.633).

There are three distinct groundwater units at the site. These are as follows:

Sands and Gravels

The sand and gravel unit at the Site comprises an Upper Sand layer, a Middle Clay layer, and a Lower Sand layer. During drilling into the Upper Sand in 2004, perched groundwater was present immediately above the Middle Clay layer in the southeast corner of the site. The Lower Sand unit was found to be dry immediately beneath the Middle Clay, confirming that the water in the Upper Sand is perched above the Middle Clay. Perched water in the Upper Sand is unlikely to be extensive in its vertical or lateral extent. Rainfall that infiltrates into the Upper Sand will move vertically downwards forming the perched water seen in some boreholes during drilling. This perched water will then move through the Middle Clay to recharge the underlying Lower Sand aquifer.

As expected, and because of the ongoing dewatering as landfill progresses, groundwater levels in the sand and gravel are variable. In general, water levels in the Middle Clay and Upper Sand unit lie at between 6.7 and 7.8 m AOD, while the water levels in the Lower Sand unit are generally much lower.

Till

The Boulder Clay present between the sand and gravel unit and the Chalk. It is classified by the Environmental Agency as a Non-Aquifer.

Chalk

Groundwater levels in the Chalk have been systematically measured at the current Site over the past two decades in monthly or quarterly intervals. The data collected from six monitoring wells shows that the groundwater elevation does not change significantly over time with only minor deviations typically not exceeding 0.25 m in each well. Chalk has been encountered in four boreholes adjacent to the Eastern Extension; two previously drilled for the purposes of groundwater monitoring at the currently operating site and two more drilled as a part of the site investigation for the Eastern Extension along its eastern edge across Milldam Beck. The groundwater in the Chalk was identified as being confined beneath the Till as at the current Site.

1.2 Lifecycle Phases

The existing landfill is divided into ten cells, Cells 1 to 8 (Cells 2 and 4 are split into A and B). Landfilling at the Site has taken place continuously since waste acceptance commenced in 2007. Filling began in Cell 1 and proceeded in a westerly direction through Cells 3, 5 and 7. Cell 8 was constructed to the north of Cell 7 in 2016, and subsequently landfilling has continued in an easterly direction into Cell 6, Cell 4A and Cell 4B, with Cells 2A and 2B to follow. Cells 1, 3, 5, 7 and 8 are filled and restored. Cell 6 awaits restoration; Cell 4A has recently been filled and awaits restoration and Cell 4B is the currently operational cell.

The Site will be extended by about 200 m towards the east in line with Sandsfield’s ownership boundary. Landfilling at the Site will be undertaken in a phased manner in order to optimise the use of the minerals and void space on the Site. The Eastern Extension will comprise a further six landfill cells (Cells 9 to 14). Cells 1, 3, 4A, 4B, 5, 6, 7 and 8 have already been developed and their footprint will remain unchanged. Cells previously designated as Cell 2A and 2B have been redesigned and will now be split north-south instead of east-west like Cells 4A and 4B. Appropriate buffer space (approximately 10 m wide) will be preserved east of Cell 1 between the already filled and restored part of the landfill and the planned excavations to avoid disturbance of the already restored part of the existing landfill. Cell 2B will extend from Cell 2A eastwards followed by Cells 9 and 10.

Progressive capping, restoration, and installation of landfill gas and leachate management systems will be carried out as each cell is completed.

Waste Mass Geometry

The maximum temporary waste slope angle on site will be approximately 1v: 2h.

Groundwater Management

Groundwater present in the Lower Sand discharges into the excavation. As such, groundwater in the Lower Sand has been managed during the construction of the Site by the use of a back-drain system behind the side slope of each cell.

The groundwater drainage system already installed behind the sidewalls of existing cells will be extended behind Cells 2A and 2B, and Cells 9, 10, 11, 12, 13, and 14 in the Eastern Extension. Groundwater management is required whilst each cell is under development. As the Site moves towards completion, it may be possible to ‘turn off’ the drain behind some completed cells to minimise the groundwater discharging to the Milldam Beck.

In development of Cell 14 (the final cell), the back drain may be accessed by a temporary manhole with submersible pump until such a time that waste levels in the cell are high enough that the pump can be withdrawn.

1.2.1 Leachate Management

Leachate sumps will be located within each of the cells to extract leachate. Leachate will be removed from the leachate sumps by means of 600 mm (internal diameter) vertical telescopic leachate extraction wells extending to the surface of the landfill.

Leachate will be extracted from the cells to maintain the level of leachate within each cell at or below 1.0 m above the base of the cell.

During the early stages of waste infilling, and when required, leachate will be re-circulated after collection in the extraction wells onto the waste mass in the active cell by pumping below the working face using temporary pipework or a vacuum tanker. Excess leachate will be returned to the waste mass to fully utilise the absorptive capacity of the waste.

1.2.2 Gas Management

Details relating to the expected production of landfill gas from the Site were presented in the Landfill Gas Generation and Risk Assessment (GRA) provided as part of the original PPC permit application. At that time, given the low quantities of readily biodegradable waste that were to be disposed at the Site, it was not expected that sufficient quantities of gas would be generated at the Site to allow the gas to be used to generate power. However, it was expected that landfill gas would be generated from the waste in sufficient quantity for flaring after approximately two years of landfilling. This turned out to be the case and up until now landfill gas has been a subject to flaring. The **Landfill Gas Risk Assessment** (ref. 20148978.635) included within this application finds the quantities of gas generated at the Site to be sufficient for energy generation via two micro generator gas engines. These will be located in the northwest corner of the Site, within a new gas compound along with the relocated flare.

1.3 Conceptual Stability Site Model

1.3.1 Basal Sub-grade Model

The published geological maps indicate that most of the Site is underlain by drift deposits that comprise undifferentiated glaciofluvial Sand and Gravel. At the extreme northeastern boundary of the Site, the maps indicate that the Sand and Gravel are absent, and the Site is underlain by Boulder Clay (glacial till). Prior to commencement of landfilling activities, the base of the Site will be excavated down from approximately -5 mAOD to -4 mAOD subject to the outcome of the basal heave assessment.

The basal lining system will be constructed directly onto the *in situ* Boulder Clay. The full thickness of the Till has been proven in several boreholes at the current Site, and ranges from 13.1 m in the northwest of the Site to 17.2 m in the southeast of the Site. The two boreholes drilled across the Milldam Beck as a part of the Eastern Extension site investigation indicate Till thicknesses of approximately 14 m. Following excavation of the Site, a minimum of 10 m of Boulder Clay will remain between the base of the Site and the top of the Chalk.

1.3.1.1 Water Pressures in the Basal Sub-Grade

Records of the groundwater level monitoring date back to October 2006. The water levels measured at the Site are provided as a part of the **Hydrogeological Risk Assessment** (HRA) (ref. 20148978.633).

There are six monitoring wells that specifically target the groundwater within the Chalk. Two of these monitoring wells (GWC01 and GWC06) are located on the boundary between the existing landfill and the proposed eastern extension. Groundwater levels from GWC01, GWC06 and the Site wide monitoring wells have been used to assess the potential for the base of the excavation to be subject to basal heave.

1.3.2 Side Slopes Sub-grade Model

The side slopes sub-grade comprise the Glacial Sand and Gravel unit and the Till as described in Section 1.1.2.4 above. The side slopes are expected to be regraded to an angle of 1v:1h before construction of the engineered fill and lining system.

After the excavation of the Eastern Extension, the spoil will be used as an engineered fill for the sidewall lining. The engineered fill is anticipated to dominantly consist predominantly of silty Clay similar to that of the Middle Clay formation. For this reason, the engineered fill is considered to have similar parameters to that of the Middle Clay, see Table SRA3.

1.3.3 Basal Lining System Model

The artificial sealing liner for the basal and lower sidewall lining system will comprise 1.0 m of engineered clay with a maximum permeability of 1×10^{-9} m/s placed on the natural geological barrier. If necessary, use of on-site clay may be substituted by fully welded geomembrane or geosynthetic clay liner (GCL) with approval of the EA in accordance with the EP.

1.3.3.1 Intercell Bunds

Each cell will be hydraulically separated from adjacent cells by an intercell bund constructed using low permeability engineered. 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.

1.3.3.2 Leachate Drainage

Leachate in Cells 1, 3, 4A, 4B, 5, 6, 7, and 8 is managed at the Site in accordance with the EP. Leachate in Cells 2A and 2B and in Cells 9 to 14 will be managed by continuation of the existing design.

For protection of the groundwater environment and in accordance with the EP, 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. Therefore, each cell has infrastructure installed to manage leachate.

A leachate collection system is provided in each cell, as follows:

- Cell 1 – Blanket of recycled brick aggregate and 20 mm virgin gravel with drainage pipes leading to a sump;
- Cells 3, 5, 7, 8, 6 – Blanket of shredded tyres with drainage pipes leading to a sump in each cell; and
- Cells 4A and 4B – Blanket of aggregate composed of recycled aggregate and granite with drainage pipes leading to a sump in each cell.

Leachate will be collected in Cells 2A, 2B, 9, 10, 11, 12, 13, and 14 by continuation of the existing design or as approved in accordance with the EP.

Leachate will be extracted from leachate sumps in each cell by means of a vertical 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. The collection point at the Eastern Extension will be located at the lowest point along the northern boundary of Cells 2A and B, 9, and 10 and the western boundary of Cells 11, 12, 13, and 14. 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 collection point, which will be constructed from the base of the cell to the surface of the site to enable the extraction of the collected leachate.

The leachate drainage system will conform to the specification contained within a CQA Plan submitted to the EA prior to construction. Installation and construction quality assurance procedures for the leachate drainage system will be defined within the CQA Plan.

Leachate extraction at the Eastern Extension will follow the procedures applied at the current Site. As such it will take place from the leachate sumps, one of which is to be located within each cell. Leachate will be removed from the leachate collection points by means of vertical leachate extraction wells extending to the surface of the landfill. The wells will be able to accommodate automatic pumping equipment (eductors or submersible pumps) to extract leachate.

Leachate will be extracted from the cells to maintain the level of leachate within each cell in accordance with the EP.

1.3.4 Side Slope Lining System Model

The *in situ* Boulder Clay at Milegate Extension will form the geological barrier component of the lower sidewall system. The engineered liner will comprise reworked Boulder Clay to achieve a hydraulic conductivity of 1×10^{-9} m/s or less, placed on the natural geological barrier. The Boulder Clay will be placed to a minimum thickness of 1 m both at the base and on the slope. The engineered clay will conform to the specification contained within a Construction Quality Assurance (CQA) plan submitted to the Agency prior to construction.

The upper sidewall subgrade will comprise the Upper and Lower Sand and Gravel and the Silty Clay. The upper side walls will be buttressed with engineered fill. The engineered fill will be composed of spoil and clays recovered during the excavation of the Eastern Extension. Side slopes will be engineered at the Site to a gradient of 1v: 2.5h.

A sidewall drainage system will be installed between the *in situ* ground and the engineered fill to ensure that natural groundwater is intercepted before it percolates into the Engineered fill. Cells will also be designed to ensure that the side slopes will not remain exposed unnecessarily. These systems should ensure that engineered fill sidewall remain dominantly dry throughout the design life of the landfill.

1.3.5 Waste Mass Model

The site is classified as a non-hazardous landfill and will continue to only accept non-hazardous waste. The Site can accept up to 75,000 tonnes of waste per year for landfilling.

Cells 1, 3, 4A, 4B, 5, 6, 7 and 8 have already been developed and their footprint will remain unchanged.

The Site will be extended by about 200 m towards the east in line with Sandsfield's ownership boundary. Cells previously designated as Cell 2A and 2B have been redesigned and will now be split north-south instead of east-west like Cells 4A and 4B. Appropriate buffer space (approximately 10 m wide) will be preserved east of Cell 1 between the already filled and restored part of the landfill and the planned excavations to avoid disturbance of the already restored part of the existing landfill. Cell 2B will extend from Cell 2A eastwards followed by Cells 9 and 10.

The remaining part of the Eastern Extension will be split east-west into Cells 11, 12, 13, and 14 from north to south. The size of each operational cell will be designed to minimise the area open to rainfall whilst maintaining overall operational efficiency. During filling of each cell, effective infiltration into the Site should not form free leachate in the base of the cell.

Progressive capping, restoration, and installation of landfill gas and leachate management systems will be carried out as each cell is completed.

The steepest temporary waste slope gradient will be approximately 1v:2h. The steepest final waste slope gradient will be approximately 1v:6h.

1.3.6 Capping System Model

1.3.6.1 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 specification of the cap is outlined in the following sections.

1.3.6.2 Blinding Layer

Prior to the placement of the sealing layer, the waste will be thoroughly compacted and smoothed so that sharp objects do not protrude excessively. A blinding layer typically comprising up to 300 mm subsoil will be placed if deemed necessary.

1.3.6.3 Sealing Layer

The upper sealing layer for each cell will comprise either a 1 mm fully welded geomembrane liner or a Geosynthetic Clay Liner (GCL) as approved in accordance with the specification contained within a CQA Plan submitted to the EA prior to construction.

1.3.6.4 Drainage Layer

A geocomposite drainage layer (if required) will be placed above the capping liner to provide both protection and drainage. The drainage layer will typically comprise a non-woven geotextile bonded to a cuspated HDPE geomembrane on the top side. The requirement will be assessed at the detailed capping design stage and included as part of the CQA Plan submitted to the EA prior to construction.

1.3.6.5 Restoration Soils

Restoration cover soils will be placed above the capping system to promote the regeneration of the landform for agricultural use. Following placement of the cap, subsoil and topsoil will be spread evenly to achieve the final pre-settlement, post-restoration profile.

The final cap will be placed within 12 months of cell completion of filling to pre-settlement restoration levels.

2.0 STABILITY RISK ASSESSMENT

2.1 Risk Screening

2.1.1 Basal Sub-grade and Lining Screening

The site investigation data indicates that there are no cavities beneath the Site. Any locally softened compressible materials will be removed prior to the construction of the cells and will be replaced with suitable fill material. The basal lining system will be constructed on natural ground consisting of Boulder Clay. Following excavation of the landfill, a minimum of 10 m of Boulder Clay will remain between the base of the Site and the top of the Chalk. This foundation is considered to be stable and not subject to any significant settlement, either total or differential, that would lead to a breach of the lining system.

The Site is situated on a Secondary A and Secondary Undifferentiated Aquifer, as indicated by the information published on Defra's magic website. The groundwater in the sand and gravel deposits tends to converge towards the southeast of the Site. However, this is of minor importance due to the extraction of sand and gravel to create the void space.

Underlying the Site at depth, the Chalk has been classified as a Principal Aquifer. Boreholes drilled into the Chalk indicate an elevation beneath the Site of around -16 m AOD. The Chalk is a highly permeable formation usually with a known or probable presence of significant fracturing. The Chalk is confined by the overlying Glacial Till, 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.

2.1.2 Side Slope Sub-grade and Lining System Screening

Side slopes are excavated within the Glacial Sand and Gravel and the Boulder Clay at a gradient of 1v: 1h. The side slopes will be buttress by engineering fill material to a gradient of 1v:2.5h 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) should be assessed. It is considered that if the unconfined slope is stable then it is not 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.

2.1.3 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 profiles are shallow and vary between 6 and 10 degrees. The analysed temporary and final waste cross sections C and D are shown on Drawing SRA2.

2.1.4 Capping System Screening

The stability of the cap and cover soils should be considered. Both geomembrane cap and GCL cap have been analysed along the steepest and highest cross section D shown on Drawing SRA2.

2.2 Data Summary

Various phases of site investigation have been carried out at Milegate 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.

Groundwater Levels

Detailed information about groundwater levels can be found within the **Hydrogeological Risk Assessment** (ref. 20148978.633).

A summary of groundwater monitoring of the Lower Sands and gravels is shown in Figure SRA1, below.

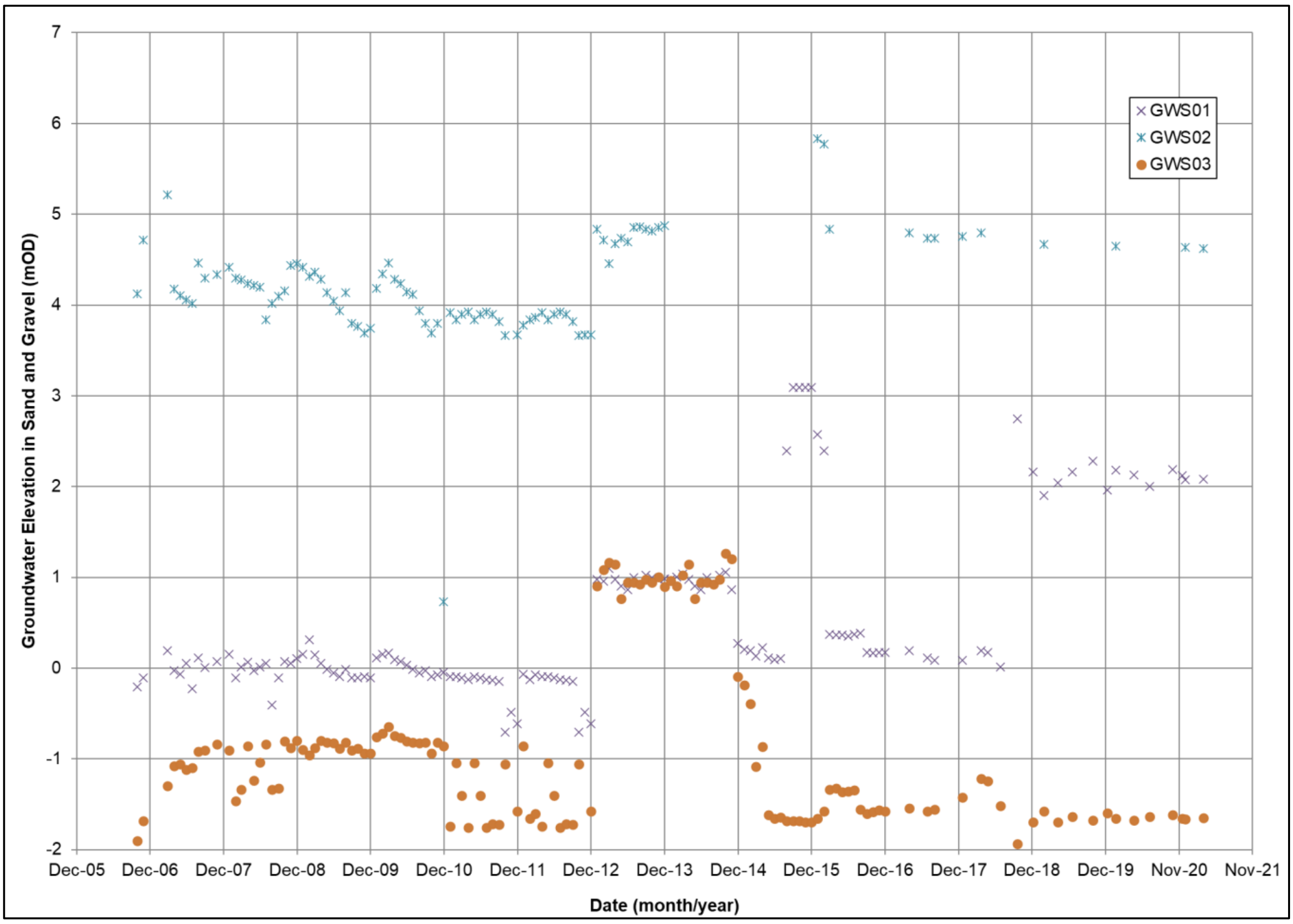

Figure SRA1: Groundwater levels in Lower Sands and gravels within the footprint of the Eastern Extension

A summary of groundwater monitoring within the Chalk in the footprint of the Eastern Extension is shown in Figure SRA2, below. A groundwater level of 2.5 mAOD has been chosen as the characteristic value to be adopted in the basal heave assessment.

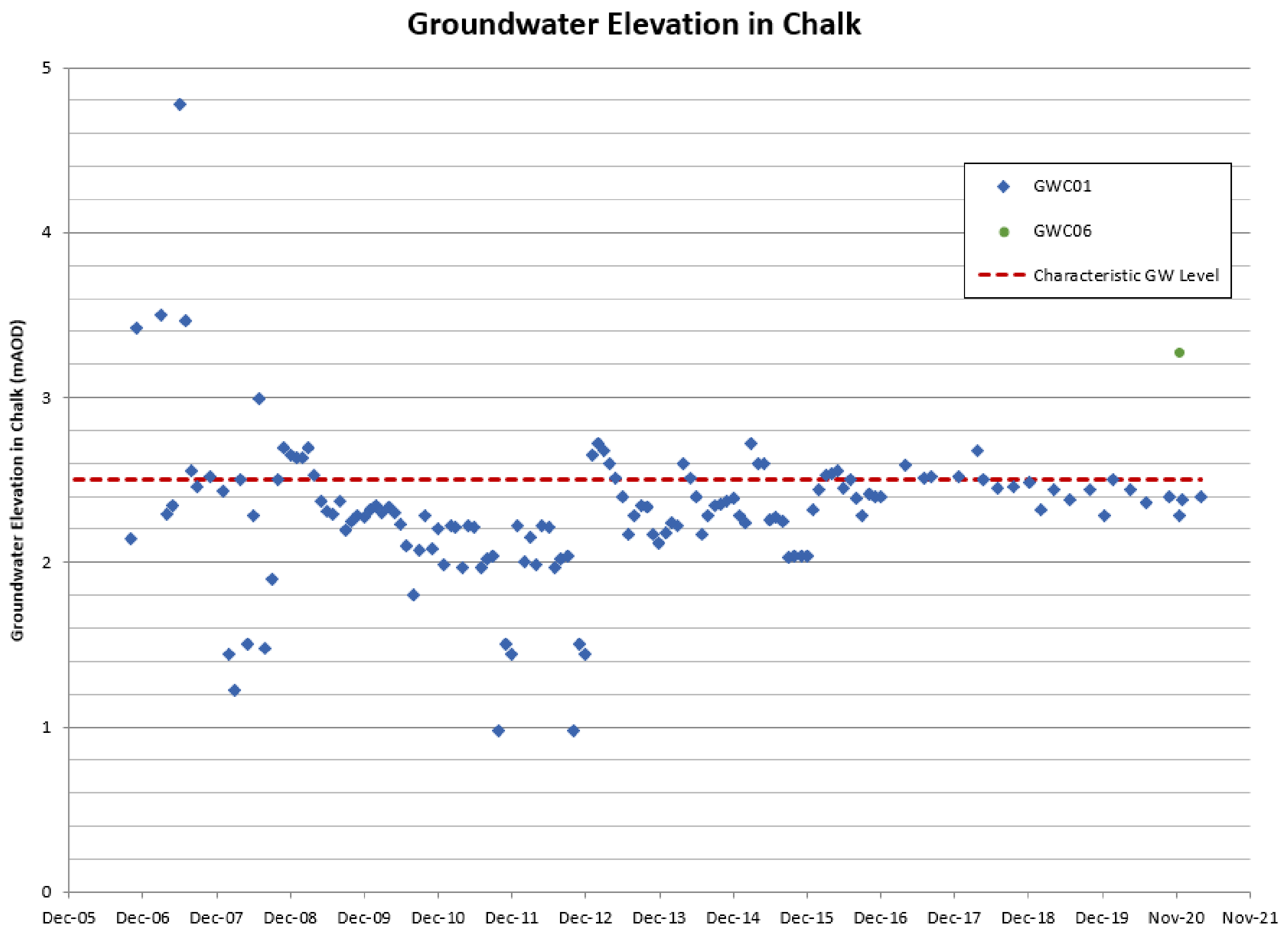

Figure SRA2: Groundwater levels in the Chalk within the footprint of the Eastern Extension

2.3 Selection of Appropriate Factors of Safety

2.3.1 Factor of Safety for Basal Sub-grade and the Basal Lining System

A minimum factor of safety of 1.3 against basal heave will be considered acceptable providing reasonably conservative parameters have been used.

2.3.2 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.

2.3.3 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.

2.3.4 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.

2.3.5 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.

2.4 Justification for Modelling Approach and Software

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

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

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

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

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

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

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

  The analysis of the temporary and final waste slopes have 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 for geomembrane cap.

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' (Environment Agency, 2003). These represent best available techniques at the time of this report.

2.5 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 available data, Golder's in-house experience and the technical literature. 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.

2.5.1 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

Material	Unit Weight, γ (kN/m^3)
Boulder Clay	20
Water	9.81

Note that conservative value of unit weight for boulder clay has been taken for the basal heave analyses.

2.5.2 Parameters Selected for Side Slopes Engineered Fill Analyses

The material parameters used in the analysis of the side slopes are presented in Table SRA3. There is no site-specific shear strength data available for the material used to be used to make up the side walls. Therefore, conservative shear strength values have been selected.

Table SRA3: Summary of Parameters Used in the Sub-grade in the Side slopes Analyses

Material	Unit Weight (kN/m^3)	Cohesion c′ (kPa)	Friction angle ϕ′ (degrees)
Topsoil	19	3	23
Engineered Fill	20	3	27
Upper Sands and Gravels	19	0	35
Middle Clay	20	3	27
Lower Sand and Gravels	19	0	35
Boulder Clay	20	5	27

2.5.3 Parameters Selected for Waste Analyses

The material parameters used in the analysis of the temporary waste slopes are presented Table SRA4. The parameters for the analysis of the temporary waste slopes have been obtained from Jones, Taylor & Dixon, 1997.

Table SRA4: Summary of Parameters Used in the Waste Liner Analyses

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

2.5.4 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	Cohesion (kPa)	Friction Angle (degrees)
Cover soil internal strength	0	25
Cover soil/Geotextile	0	24
Geotextile/Geomembrane	0	26
Geomembrane/Blinding layer	0	24
Cover soil/GCL	0	24
GCL/Blinding layer	0	24

2.6 Analyses

2.6.1 Basal Heave Analyses

Basal heave calculations have been undertaken in accordance with the methodology suggested in Environment Agency, 2003. The detailed calculation sheets are presented in Appendix SRA1. A summary of the basal heave calculations is presented in Table SRA3 below.

Table SRA6: Summary of Basal Heave Calculations

Scenarios	Factor of Safety	
	Formation Level @ -5 mAOD	Formation Level @ -4 mAOD
Prior to Clay Liner Placement	1.22	1.33
Post Clay Liner Placement	1.33	1.44
Post Drainage Blanket Placement	1.36	1.47

2.6.2 Side Slope Sub-grade Analyses

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

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

Analysis Reference	Description	Factor of Safety
Section A_Subgrade_1	Section A, 1v:2.5h slope, fully functional back-drain, dry	1.48
Section A_Subgrade_2	Section A, 1v:2.5h slope, fully functional back-drain, r_u=0.1	1.33
Section A_Subgrade_3	Section A, 1v:2.5h slope, partially functional back-drain, r_u=0.1	1.33
Section A_Subgrade_4	Section A, 1v:2.5h slope, dysfunctional back-drain, r_u=0.1	1.10
Section B_Subgrade_1	Section B, 1v:2.5h slope, fully functional back-drain, dry	1.48
Section B_Subgrade_2	Section B, 1v:2.5h slope, fully functional back-drain, r_u=0.1	1.33
Section B_Subgrade_3	Section B, 1v:2.5h slope, partially functional back-drain, r_u=0.1	1.33
Section B_Subgrade_4	Section B, 1v:2.5h slope, dysfunctional back-drain, r_u=0.1	1.09

2.6.3 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, dry	1.49
Section A_Liner_2	Section A, 1v:2.5h slope, fully functional back-drain, r_u=0.1	1.34
Section A_Liner_3	Section A, 1v:2.5h slope, partially functional back-drain, r_u=0.1	1.34
Section A_Liner_4	Section A, 1v:2.5h slope, dysfunctional back-drain, r_u=0.1	1.18
Section B_Liner_1	Section B, 1v:2.5h slope, fully functional back-drain, dry	1.49
Section B_Liner_2	Section B, 1v:2.5h slope, fully functional back-drain, r_u=0.1	1.34
Section B_Liner_3	Section B, 1v:2.5h slope, partially functional back-drain, r_u=0.1	1.34
Section B_Liner_4	Section B, 1v:2.5h slope, dysfunctional back-drain, r_u=0.1	1.19

2.6.4 Waste Analyses

Temporary Waste Slopes

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

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

File Ref	Description	Factor of Safety
Temp Waste Slope_1	Section C, 1v:2h slope, circular failure, dry	1.35
Temp Waste Slope_2	Section C, 1v:2h slope, circular failure, 1m leachate level	1.35
Temp Waste Slope_3	Section C, 1v:2h slope, circular failure, 2m leachate	1.34
Temp Waste Slope_4	Section C, 1v:2h slope, circular failure, 1m leachate, r_u=0.1	1.23
Temp Waste Slope_5	Section C, 1v:2h slope, circular failure, 1m leachate, r_u=0.2	1.10
Temp Waste Slope_6	Section C, 1v:2h slope, circular failure, 1m leachate, r_u=0.2, dry waste in the outer 10m of waste slope	1.15
Temp Waste Slope_7	Section C, 1v:2h slope, circular failure, 1m leachate, r_u=0.2, dry waste in the outer 20m of waste slope	1.35

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

File Ref	Description	Factor of Safety
Final Waste Slope_1	Section D, 1v:6h slope, circular failure, 1m leachate	4.01
Final Waste Slope_2	Section D, 1v:6h slope, circular failure, 1m leachate, r_u=0.1	3.69
Final Waste Slope_3	Section D, 1v:6h slope, circular failure, 1m leachate, r_u=0.2	3.36

2.6.5 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 factors of safety calculated for the finite slope analyses is presented in Table SRA11, and the output files are given in Appendix SRA6.

Table SRA11: Summary of Geomembrane Capping Stability Analyses

Description		Factor of Safety		
		Slippage of Restoration Soil	Tensile Failure of Geotextile	Tensile Failure of Geomembrane
Section D, 1v:6h slope, 7 m high	PSR = 0	2.89	Infinite	Infinite
	PSR = 0.5	2.13	Infinite	Infinite
	PSR = 1.0	1.45	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:6h slope, 7 m high	PSR = 0	2.89	Infinite
	PSR = 0.5	2.13	Infinite
	PSR = 1.0	1.45	Infinite

PSR represents Parallel Submergence Ratio

2.6.6 Leachate Extraction System Analyses

Extraction Well Foundation

A summary of the foundation bearing capacity analysis and differential settlement calculated for the leachate extraction well is presented in Table 636/13, and the calculations sheets are given in Appendix SRA8.

Table SRA13: Summary of Leachate Extraction Well Foundation Analyses

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

Leachate Pipework Deflection

A summary of the leachate pipe work deflection calculations is presented in Table SRA14, and the calculation sheets are given in Appendix SRA9.

Table SRA14: Summary of Leachate Pipe work Deflection Calculations

Description	Pipe Deflection	
	(mm)	(%)
Primary pipe with an internal diameter of 160 mm	4.67	2.9
Secondary pipe with an internal diameter of 120 mm	2.45	2.9

2.7 Assessment

2.7.1 Basal Sub-grade and Liner Assessment

When the analysis is carried out for a basal excavation elevation of -5 mAOD, the factors of safety calculated for pre clay liner placement, post clay liner placement and post drainage blanket placement are 1.22, 1.33 and 1.36. Whilst the factors of safety for post clay liner placement and post drainage blanket placement are greater than the minimum required 1.3, the factor of safety calculated for pre clay liner placement is unsatisfactory.

When the analysis is carried out for a basal excavation elevation of -4 mAOD, the factors of safety calculated for pre clay liner placement, post clay liner placement and post drainage blanket placement are 1.33, 1.44 and 1.47. These factors of safety are greater than the minimum required 1.3 and therefore are satisfactory.

2.7.2 Side Slopes Sub-grade Assessment

The analyses of the side slope sub-grade for Section A show that the factors of safety against circular failure for a fully functioning back-drainage layer installed beneath the engineered fill material are 1.48 and 1.33. When the side slope sub-grade is analysed with a partially functioning back-drainage layer, the factor of safety remains 1.33. This is considered satisfactory. When the side slope sub-grade is analysed with a dysfunctional back-drainage system, the factor of safety reduces to 1.10. This is below the minimum required 1.3 and therefore considered unsatisfactory.

The analyses of the side slope sub-grade for Section B show that the factors of safety against circular failure for a fully functioning back-drainage layer installed beneath the engineered fill material are 1.48 and 1.33. When the side slope sub-grade is analysed with a partially functioning back-drainage layer, the factor of safety remains 1.33. This is considered satisfactory. When the side slope sub-grade is analysed with a dysfunctional back-drainage system, the factor of safety reduces to 1.09. This is below the minimum required 1.3 and therefore considered unsatisfactory.

2.7.3 Side Slopes Liner Assessment

The analysis of the side slope liner using for Section A indicates that factors of safety against circular failure with a fully functioning back-drainage layer in the engineered fill material are 1.49 and 1.34. When the side slope liner is analysed with a partially functioning back-drainage layer, the factor of safety remains 1.34. This is considered satisfactory. When the side slope liner is analysed with a dysfunctional back-drainage system, the factor of safety reduces to 1.18. This is below the minimum required 1.3 and therefore considered unsatisfactory.

The analysis of the side slope liner using for Section B indicates that factors of safety against circular failure with a fully functioning back-drainage layer in the engineered fill material are 1.49 and 1.34. When the side slope liner is analysed with a partially functioning back-drainage layer, the factor of safety remains 1.34. This is considered satisfactory. When the side slope liner is analysed with a dysfunctional back-drainage system, the factor of safety reduces to 1.19. This is below the minimum required 1.3 and therefore considered unsatisfactory.

2.7.4 Waste Assessment

Temporary Waste Slopes

For the proposed 1v:2h temporary waste slope in the extension cells, the factor of safety against circular failure is calculated as 1.35 for a dry condition. With a 1 m leachate level which is the compliance level, the calculated factor of safety remains largely unchanged at 1.35. With a 2 m leachate level, the calculated factor of safety slightly reduces to 1.34. This is satisfactory. With pore water pressure build-up equivalent to r_u values of 0.1 and 0.2, the factors of safety reduce to 1.23 and 1.10 respectively which are considered unsatisfactory.

When a 10 m layer (running parallel to the temporary waste slope) is introduced with no leachate re-circulation (i.e. dry waste) the factor of safety increases to 1.15 which is still below the required 1.3. When a 20 m layer (running parallel to the temporary waste slope) is introduced with no leachate re-circulation (i.e. dry waste) the factor of safety increases to 1.35 which is considered satisfactory. Leachate recirculation should therefore not be carried out within 20 m of any open waste face.

Final Waste Slopes

For the proposed steepest and highest final waste slope, the factor of safety against circular failure is calculated as 4.01 for a dry condition with 1 m leachate level at the base. With pore water pressure build-up equivalent to r_u values of 0.1 and 0.2, the factors of safety reduce to 3.69 and 3.36 respectively. These factors of safety are all significantly greater than the required 1.3 and therefore considered satisfactory.

2.7.5 Capping Assessment

Geomembrane Capping System

The geomembrane cap stability analysis results show that the factors of safety against cover soil slippage for a LLDEP geomembrane caps are all above a value of 1.3, which would typically be considered appropriate. There will be no tension developed within both the geotextile and geomembrane layer. This is therefore considered satisfactory.

GCL Capping System

The GCL cap stability analysis results show that the factors of safety against cover soil slippage for a GCL cap are all above a value of 1.3, which would typically be considered appropriate. There will be no tension developed within the GCL layer. This is therefore considered satisfactory.

2.7.6 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 to assess the 160 mm internal diameter primary and 120 mm internal diameter secondary leachate pipework, indicate that the maximum deflections (2.9% for both 160 mm diameter and 120 mm diameter pipe) are less than the maximum allowable deflection of 4.2% and therefore considered satisfactory.

3.0 THE RISK BASED MONITORING SCHEME

3.1.1 Basal Sub-grade and Liner Monitoring

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

3.1.2 Side Slopes Sub-grade and Liner Monitoring

The side slopes should be monitored for any sign of ground water ingress during construction. If local slumping of the Glacial Sand and Gravel occurs after a particularly heavy rainfall event, then the material should be replaced to a suitable specification.

Site specific shear strength testing should be undertaken to obtain shear strength parameters for the Sand and Gravel, the Till, the engineered fill, and the clay liner to confirm the assumptions made in the stability assessment.

3.1.3 Waste Mass Monitoring

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

Leachate levels should 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 should be maintained below 1 m above the base of the cell.

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

3.1.4 Capping System Monitoring

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

4.0 REFERENCES

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Signature Page

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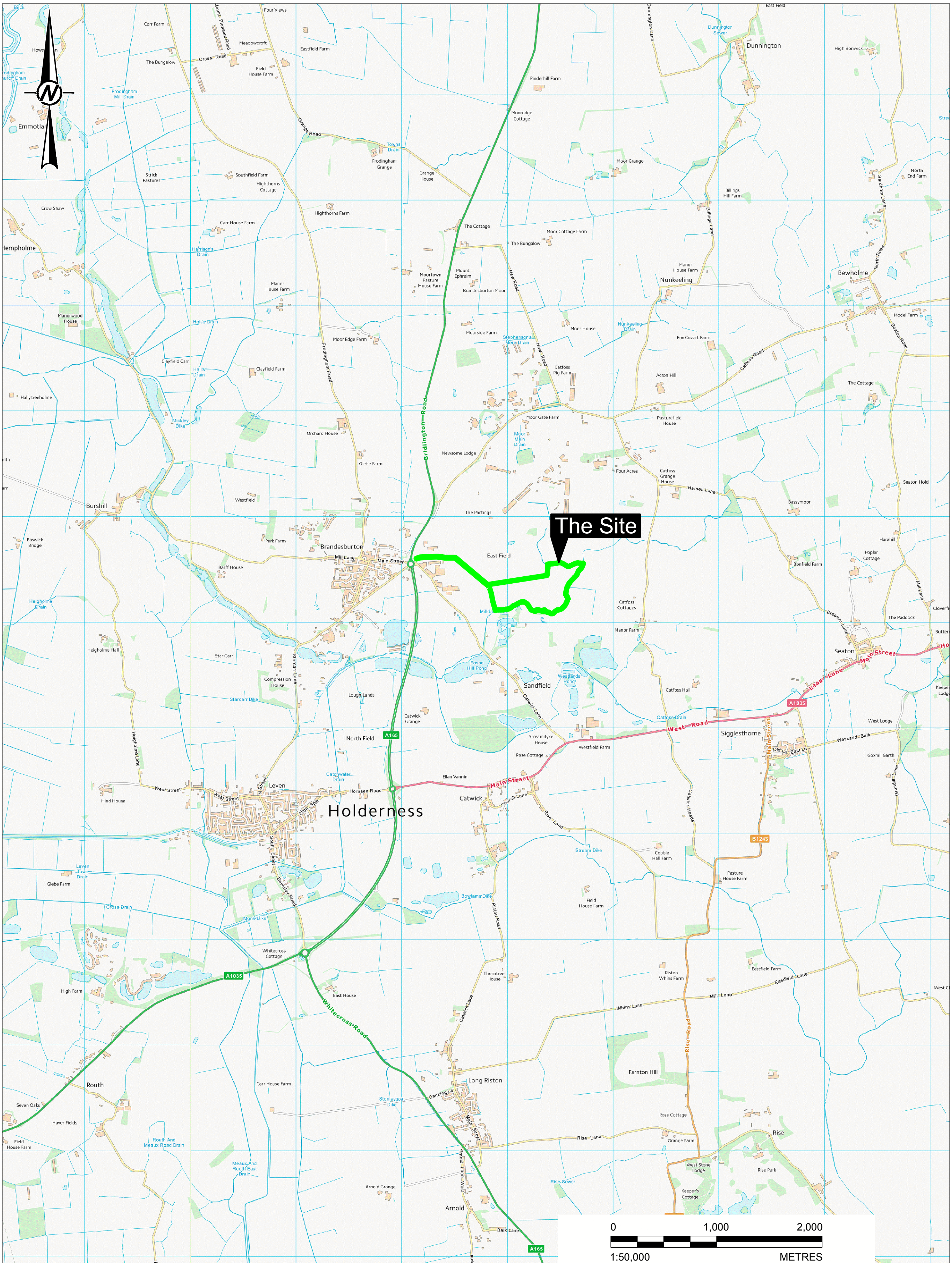
Drawings



CLIENT
SANDSFIELD GRAVEL COMPANY LTD

PROJECT
EASTERN EXTENSION SRA

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-12-16
DESIGNED	BZ
PREPARED	ECS
REVIEWED	BZ
APPROVED	BZ

TITLE
SITE LOCATION PLAN

PROJECT NO.	CONTROL	REV.	DRAWING
20148978	1006-SR-0001	A	SRA1

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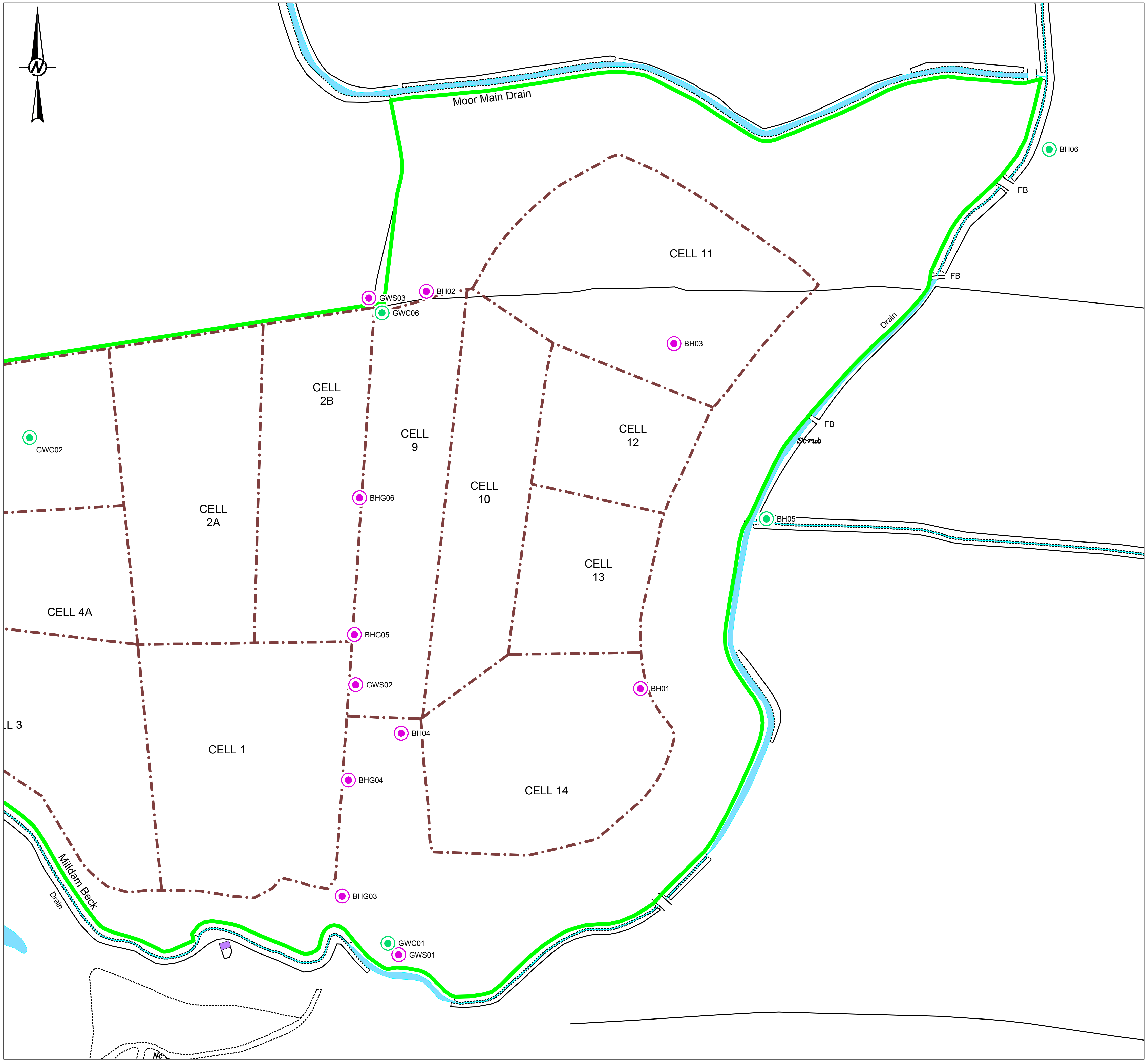

LEGEND

- PERMIT BOUNDARY
- CELL BOUNDARY
- INVESTIGATION LOCATIONS IN THE LOWER SAND (SHALLOW)
- INVESTIGATION LOCATIONS IN THE CHALK (DEEP)

REFERENCE(S)

SURROUNDING TOPOGRAPHY REPRODUCED FROM ORDNANCE SURVEY® DIGITAL MAP DATA © CROWN COPYRIGHT 2003. ALL RIGHTS RESERVED.

EXISTING FINAL RESTORATION SCHEME PROVIDED BY DJM WASTE MANAGEMENT CONSULTANCY, DRAWING REF: 02/01/005, DATED 08.05.02. EXTENSION BY GOLDER ASSOCIATES.

0 50 100

1:2,000 METRES

CLIENT
SANDSFIELD GRAVEL COMPANY LTD

PROJECT
EASTERN EXTENSION SRA

TITLE
EASTERN EXTENSION SITE INVESTIGATION INFRASTRUCTURE

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-12-16
DESIGNED	BZ
PREPARED	ECS
REVIEWED	BZ
APPROVED	BZ

PROJECT NO.	CONTROL	REV.	DRAWING
20148978	1006-SR-0002	A	SRA2

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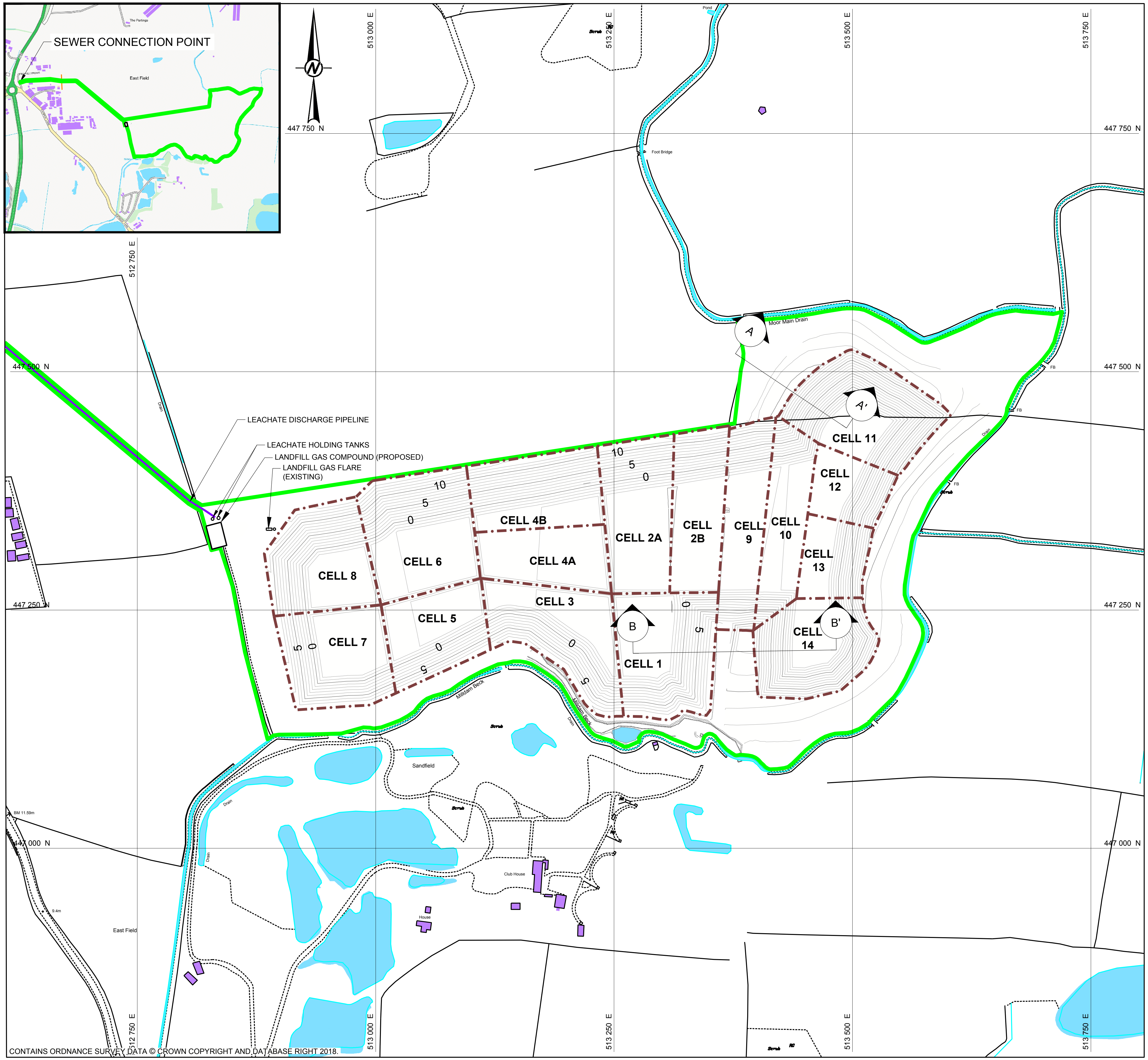

LEGEND

- PERMIT BOUNDARY
- CELL BOUNDARY
- CLAY LINER CONTOUR

NOTE(S)

1. ALL LEVELS RELATIVE TO NEWLYN ORDNANCE DATUM.

REFERENCE(S)

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0 100 200
1:4,000 METRES

CLIENT
SANDSFIELD GRAVEL COMPANY LTD

PROJECT
EASTERN EXTENSION SRA

TITLE
INSTALLATION INFRASTRUCTURE

CONSULTANT
WSP GOLDER

YYYY-MM-DD	2022-06-14
DESIGNED	BZ
PREPARED	ECS
REVIEWED	BZ
APPROVED	BZ

PROJECT NO.	CONTROL	REV.	DRAWING
20148978	1006-SR-0003	B	3

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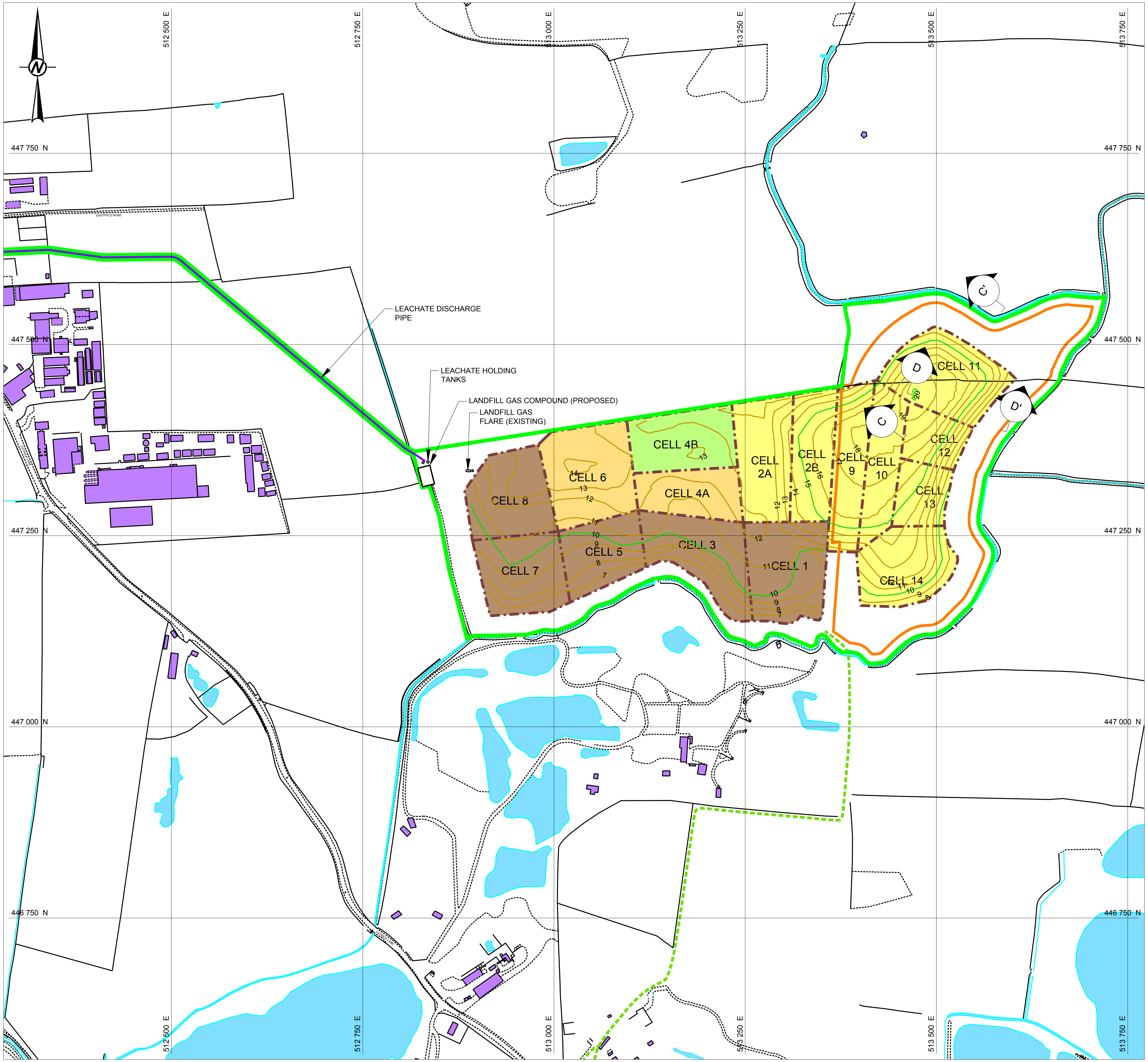

LEGEND

- PERMIT BOUNDARY
- CELL BOUNDARY
- MINERAL EXTRACTION BOUNDARY
- FILLED AND RESTORED LANDFILL
- FILLED AND UNRESTORED LANDFILL
- ACTIVE LANDFILL
- FUTURE LANDFILL
- ACCESS ROUTE FROM SITE RECEPTION TO LANDFILL
- PRE-SETTLEMENT, PRE-RESTORATION CONTOURS

REFERENCE(S)

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CONTOUR DATA FROM PRE SETTLEMENT PRE RESTORATION CONTOURS.DWG AND E_EXT RESTORATION PRE SETTLE PRE RESTORE.DWG.

0 100 200

1:5,000 METRES

CLIENT
SANDSFIELD GRAVEL COMPANY LTD

PROJECT
EASTERN EXTENSION SRA

TITLE
SITE LAYOUT AND WASTE DEPOSITION

CONSULTANT		
WSP GOLDER	YYYY-MM-DD	2022-06-14
	DESIGNED	NW
	PREPARED	ECS
	REVIEWED	BZ
	APPROVED	BZ

PROJECT NO.	CONTROL	REV.	DRAWING
20148978	1006-SR-0004	B	4

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APPENDIX SRA1

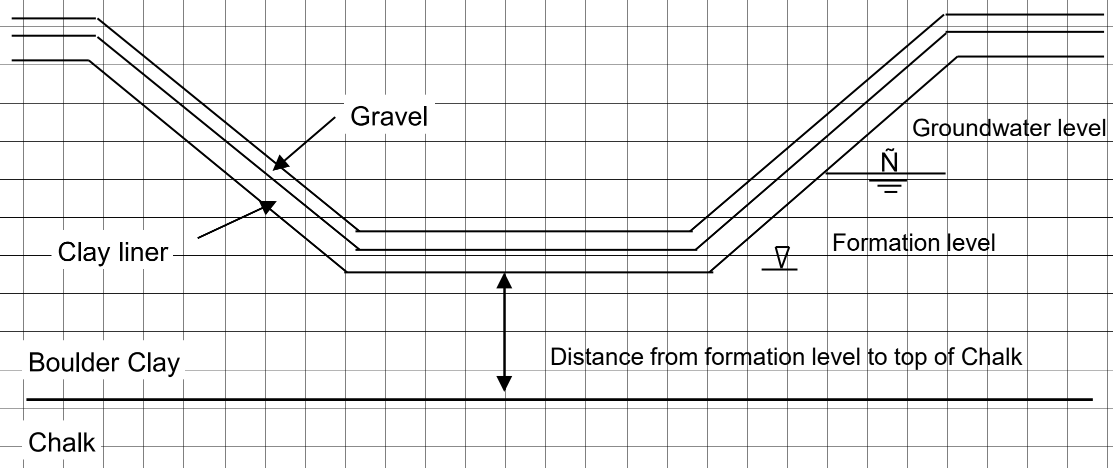
Basal Heave Calculations

Basal Heave Assessment - Sidewide Assessment

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

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

Geometry:



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

where:

s_v = total vertical stress

u = pore water pressure

Assumptions:

Thickness of gravel layer =	0.3	m	Bottom of Boulder Clay level, D =	-16	m AOD
Thickness of clay liner =	1.0	m	Unit weight of clay, g_c =	20	kN/m^3
Formation level, F =	-5.0	m AOD	Unit weight of gravel, g_g =	22	kN/m^3
Groundwater level, G =	2.5	m AOD	Unit weight of water, g_{water} =	9.81	kN/m^3

1. Sub-grade stability

Factor of Safety against basal heave prior to liner placement:

Factor of Safety = $\frac{(F-D)\gamma_c}{(G-D)\gamma_w}$ = 1.22

2. Basal liner stability with clay

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

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

3. Basal liner stability when complete

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

Factor of Safety = $\dfrac{(F-D)\gamma_m + 1.0\gamma_c + 0.5\gamma_g}{(G-D)\gamma_w}$ = 1.36

References:

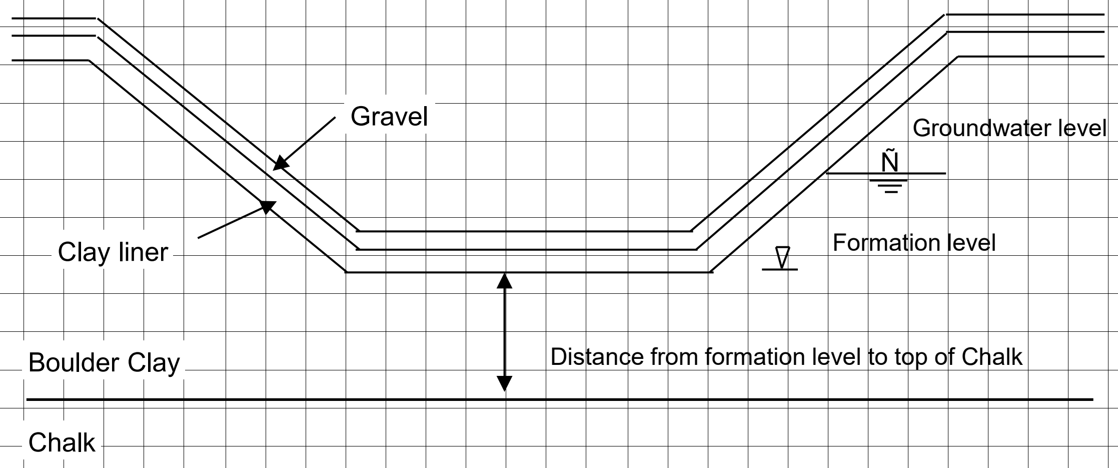
Environment Agency, 2003
Stability of Landfill Lining Systems: Report No. 1 Literature Review
R&D Technical Report P1-385/TR1

Basal Heave Assessment - Sidewide Assessment

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

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

Geometry:



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

where:

s_v = total vertical stress

u = pore water pressure

Assumptions:

Thickness of gravel layer =	0.3	m	Bottom of Boulder Clay level, D =	-16	m AOD
Thickness of clay liner =	1.0	m	Unit weight of clay, g_c =	20	kN/m^3
Formation level, F =	-4.0	m AOD	Unit weight of gravel, g_g =	22	kN/m^3
Groundwater level, G =	2.5	m AOD	Unit weight of water, g_{water} =	9.81	kN/m^3

1. Sub-grade stability

Factor of Safety against basal heave prior to liner placement:

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

2. Basal liner stability with clay

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

Factor of Safety = $\frac{(F-D)\gamma_m + 1.0\gamma_c}{(G-D)\gamma_w}$ = 1.44

3. Basal liner stability when complete

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

Factor of Safety = $\frac{(F-D)\gamma_m + 1.0\gamma_c + 0.5\gamma_g}{(G-D)\gamma_w}$ = 1.47

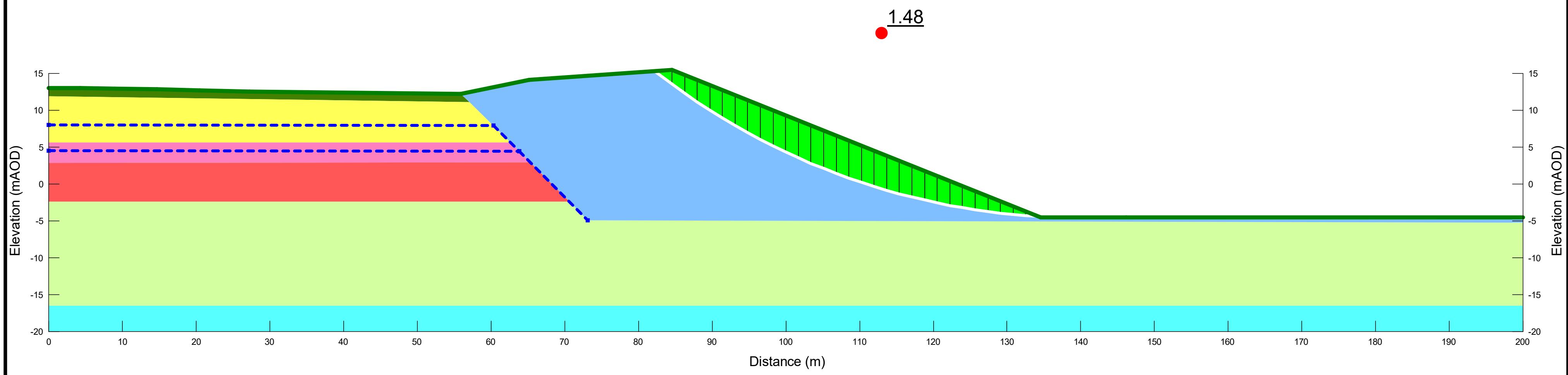
References:

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

APPENDIX SRA2

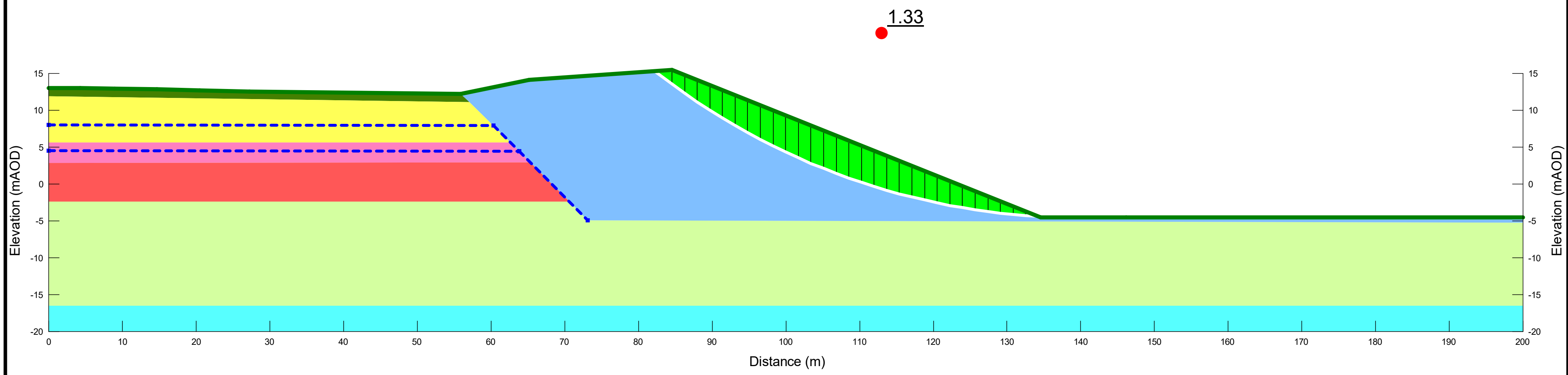
Side Slope Sub-Grade Analyses

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Boulder Clay	Mohr-Coulomb	20	5	27	1
	Chalk	Bedrock (Impenetrable)				1
	Engineered Fill	Mohr-Coulomb	20	3	27	1
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1
	Middle Clay	Mohr-Coulomb	20	3	27	
	Topsoil	Mohr-Coulomb	20	2	26	
	Upper Sands	Mohr-Coulomb	19	0	35	2



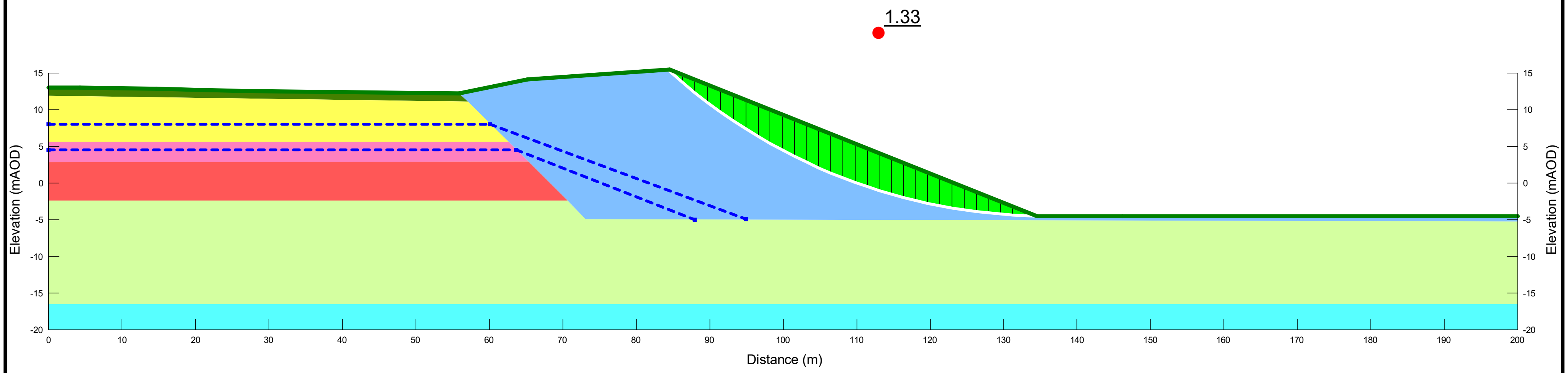
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Subgrade Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Subgrade 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	1		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No



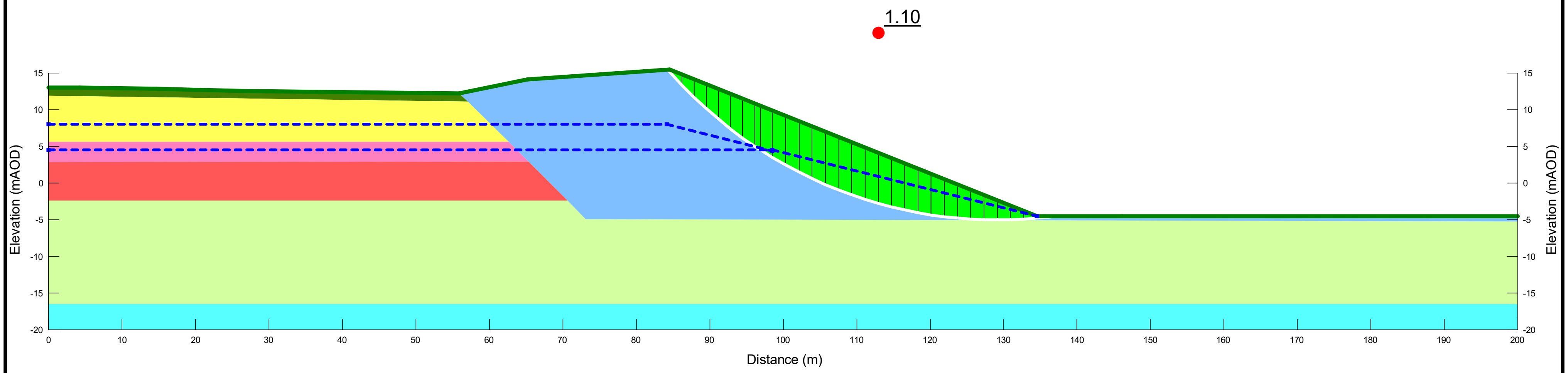
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Subgrade Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Subgrade 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	1		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No



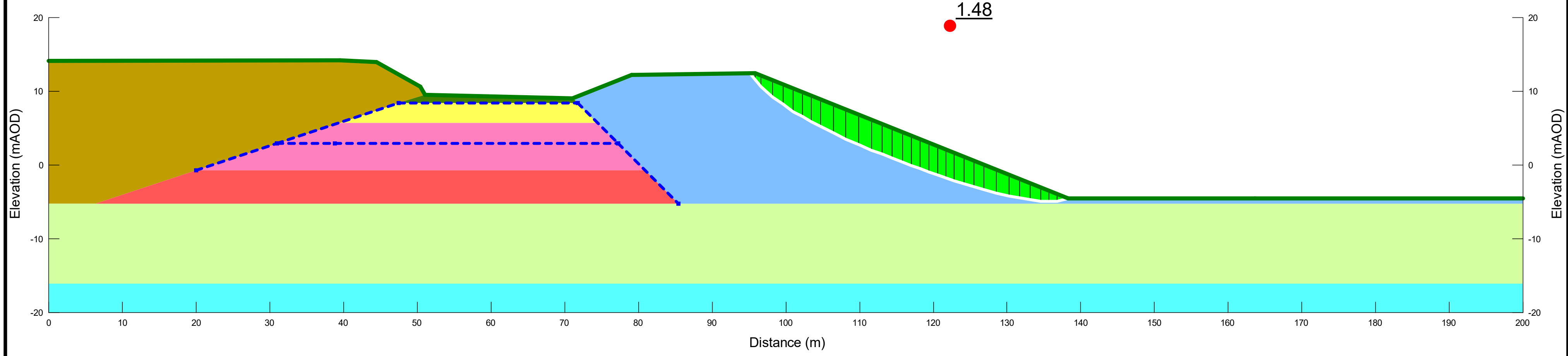
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Subgrade Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Subgrade 3	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	1		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No



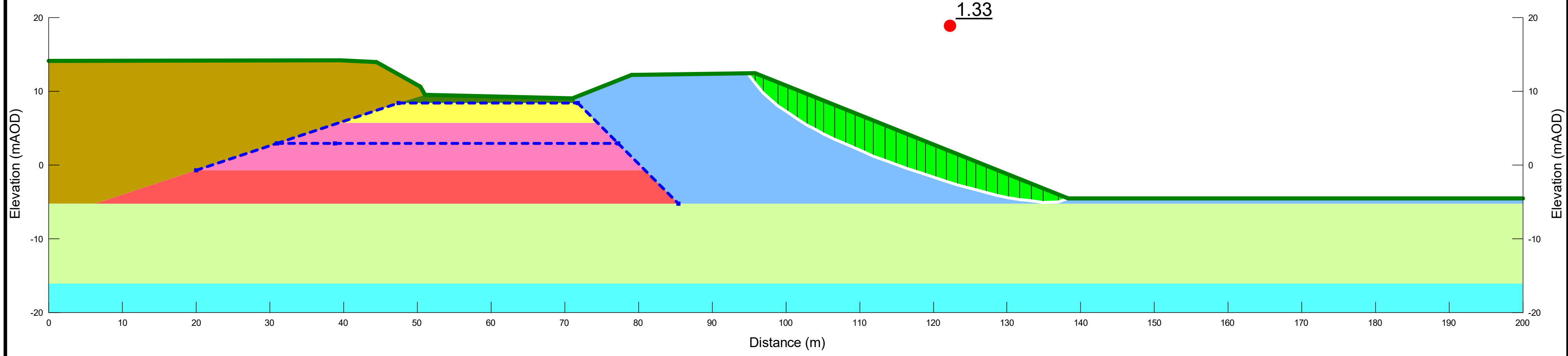
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Subgrade Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Subgrade 4	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Boulder Clay	Mohr-Coulomb	20	5	27	
	Chalk	Bedrock (Impenetrable)				
	Engineered Fill	Mohr-Coulomb	20	3	27	1
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1
	Middle Clay	Mohr-Coulomb	20	3	27	
	Topsoil	Mohr-Coulomb	20	2	26	
	Upper Sands	Mohr-Coulomb	19	0	35	2
	Waste	Mohr-Coulomb	10	5	25	1



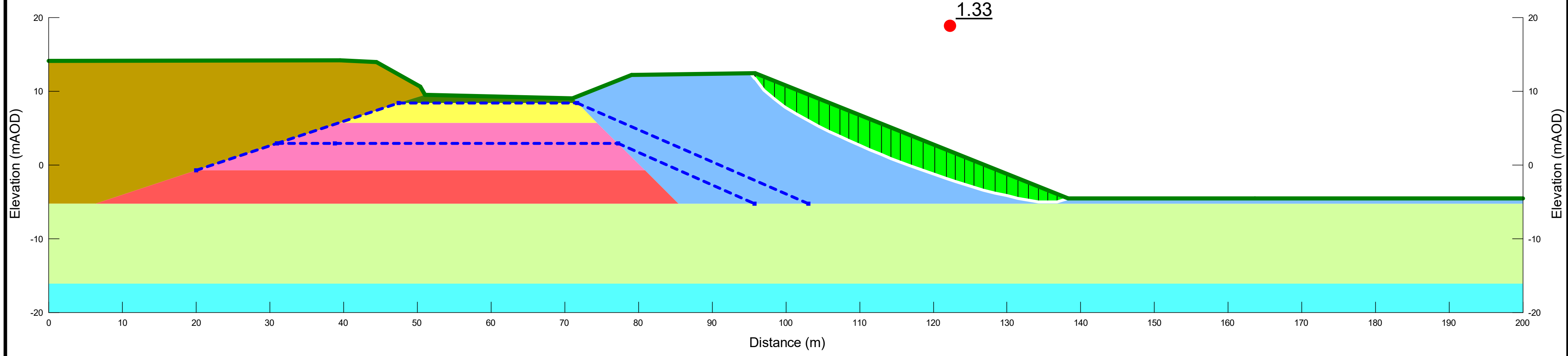
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Subgrade Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section B - Subgrade 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	2		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No
	Waste	Mohr-Coulomb	10	5	25	1		No



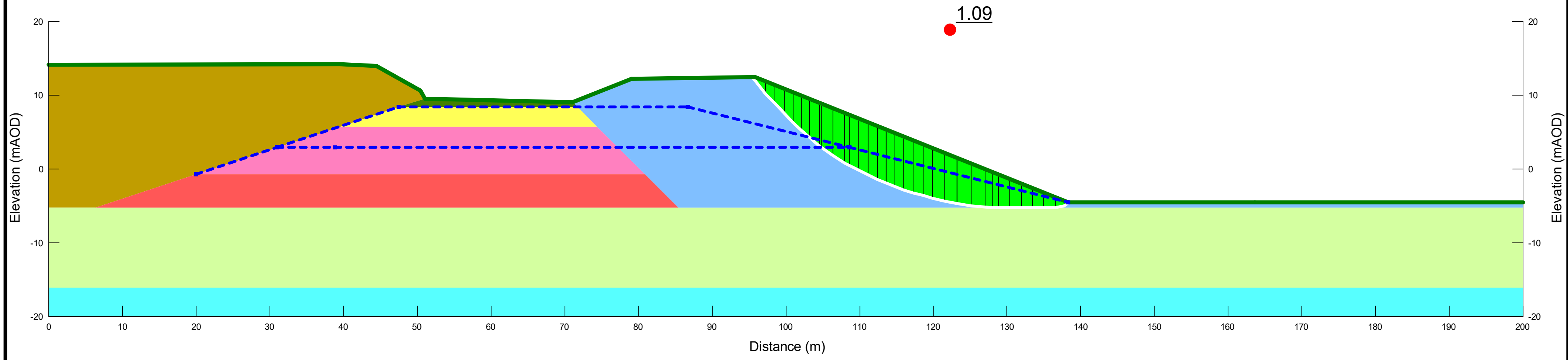
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Subgrade Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section B - Subgrade 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	2		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No
	Waste	Mohr-Coulomb	10	5	25	1		No



Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Subgrade Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section B - Subgrade 3	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	2		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No
	Waste	Mohr-Coulomb	10	5	25	1		No

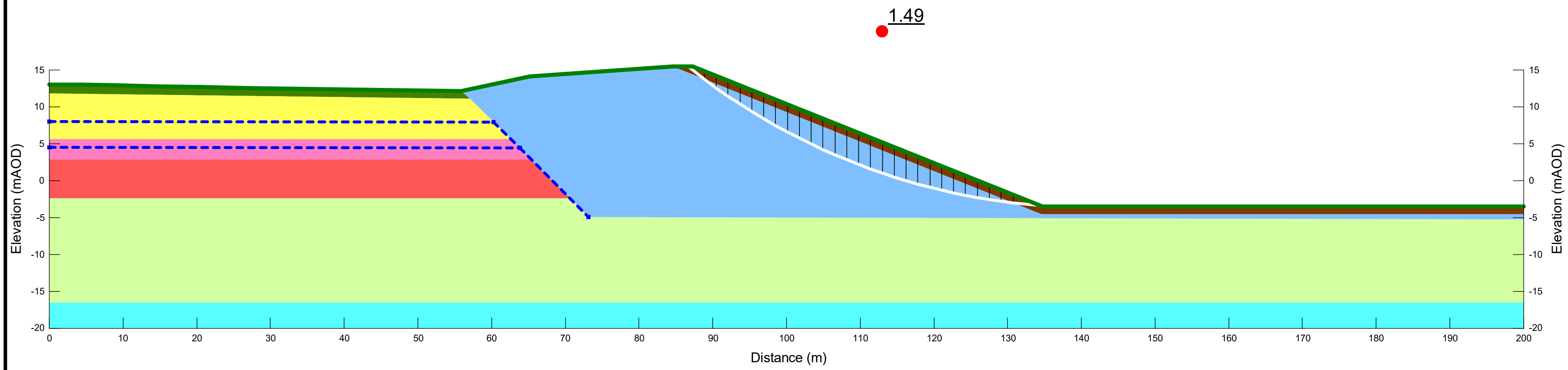


Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Subgrade Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section B - Subgrade 4	Project Manager:	N White

APPENDIX SRA3

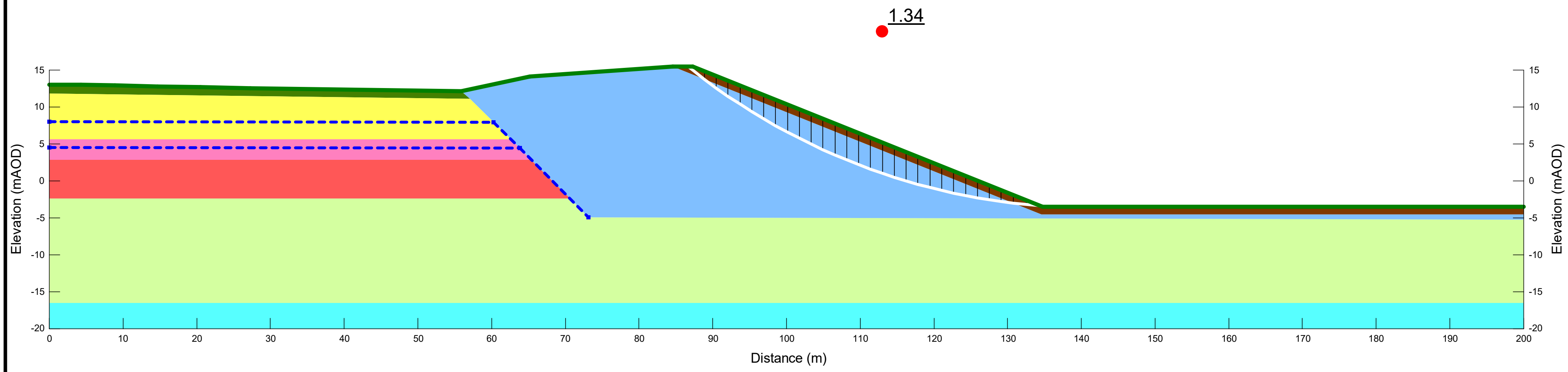
Side Slope Liner Analyses

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Boulder Clay	Mohr-Coulomb	20	5	27	1
	Chalk	Bedrock (Impenetrable)				1
	Engineered Clay	Mohr-Coulomb	20	3	27	1
	Engineered Fill	Mohr-Coulomb	20	3	27	1
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1
	Middle Clay	Mohr-Coulomb	20	3	27	
	Topsoil	Mohr-Coulomb	20	2	26	
	Upper Sands	Mohr-Coulomb	19	0	35	2



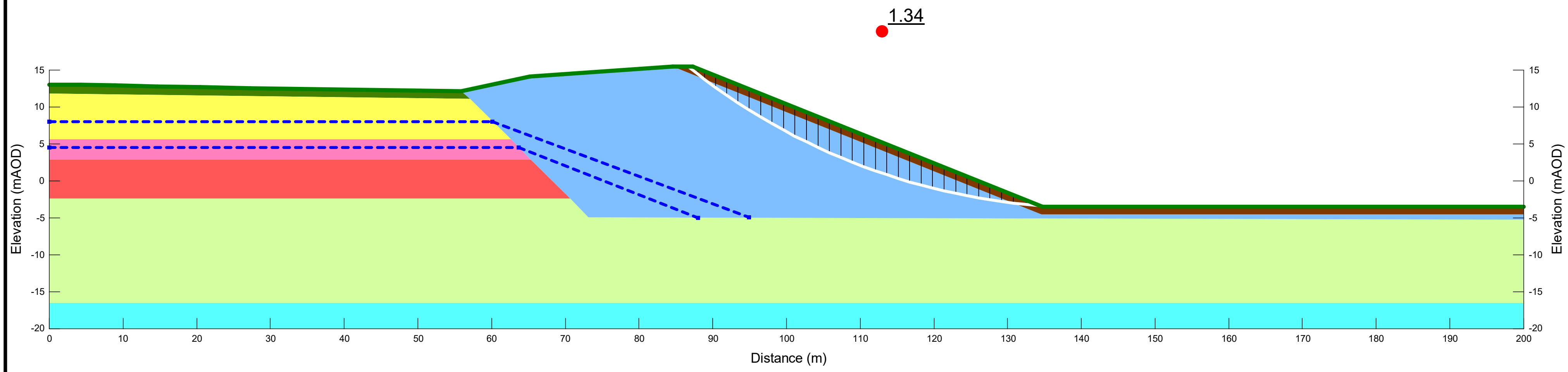
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Liner 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Clay	Mohr-Coulomb	20	3	27	1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	1		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No



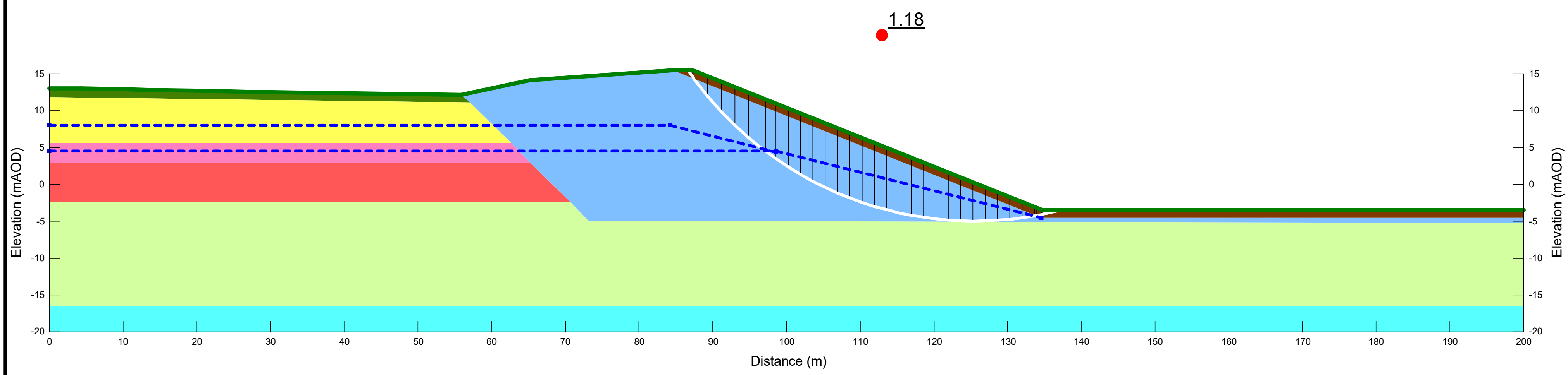
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Liner 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Clay	Mohr-Coulomb	20	3	27	1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	1		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No



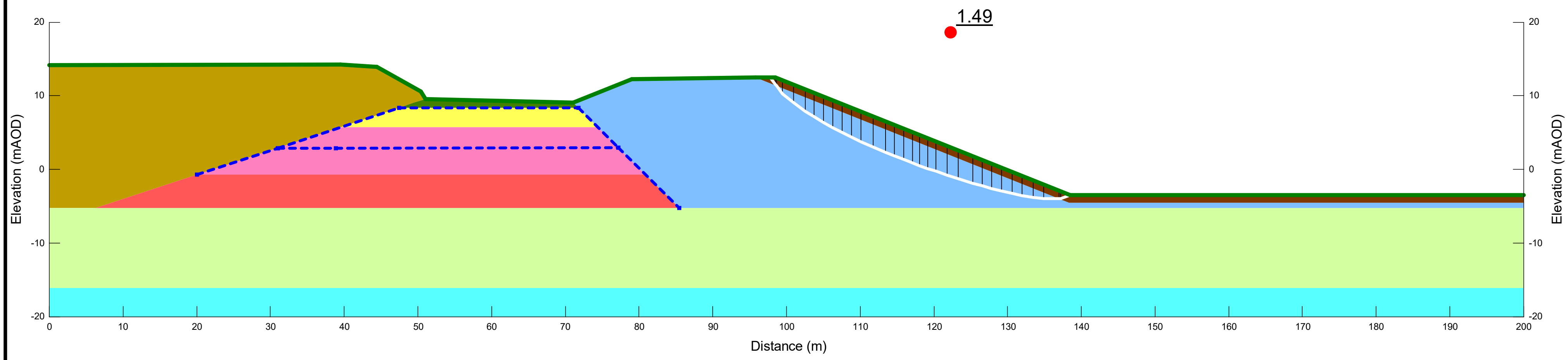
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Liner 3	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Clay	Mohr-Coulomb	20	3	27	1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1	0.1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27	1		No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No



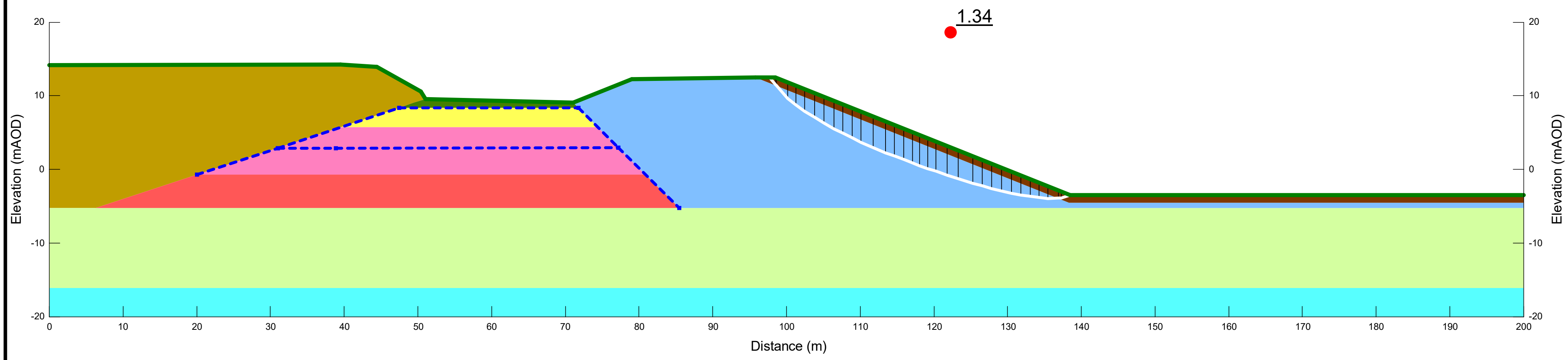
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Liner 4	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Boulder Clay	Mohr-Coulomb	20	5	27	
	Chalk	Bedrock (Impenetrable)				
	Engineered Clay	Mohr-Coulomb	20	3	27	
	Engineered Fill	Mohr-Coulomb	20	3	27	1
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1
	Middle Clay	Mohr-Coulomb	20	3	27	
	Topsoil	Mohr-Coulomb	20	2	26	
	Upper Sands	Mohr-Coulomb	19	0	35	2
	Waste	Mohr-Coulomb	10	5	25	1



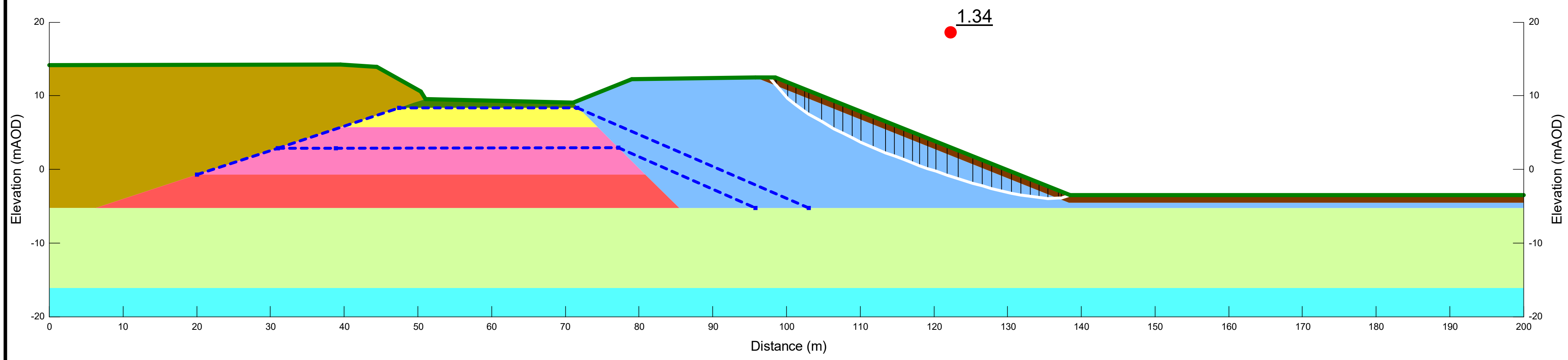
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section B - Liner 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27		1	No
	Chalk	Bedrock (Impenetrable)					1	No
	Engineered Clay	Mohr-Coulomb	20	3	27	0.2		Yes
	Engineered Fill	Mohr-Coulomb	20	3	27	0.1	1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35		1	No
	Middle Clay	Mohr-Coulomb	20	3	27		2	No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35		2	No
	Waste	Mohr-Coulomb	10	5	25		1	No



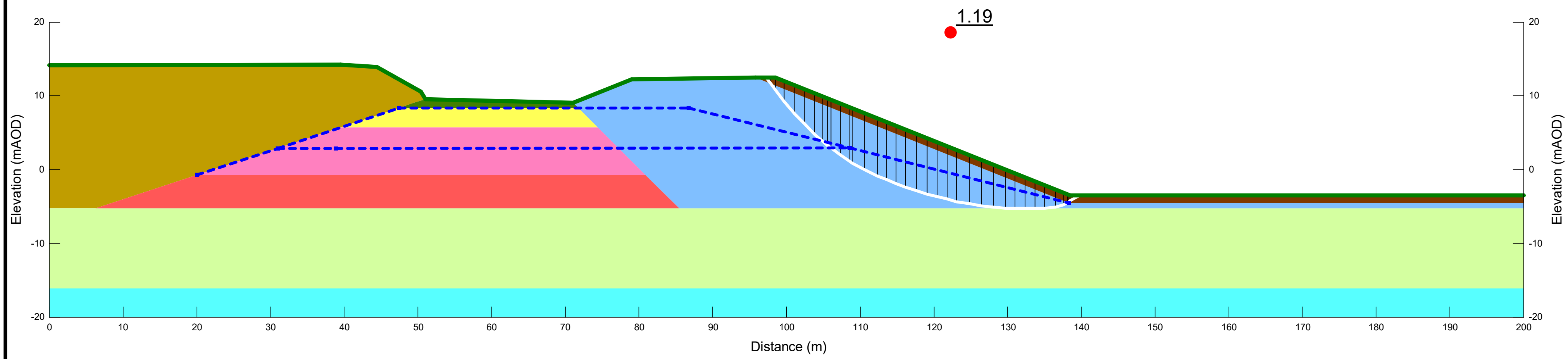
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section B - Liner 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27		1	No
	Chalk	Bedrock (Impenetrable)					1	No
	Engineered Clay	Mohr-Coulomb	20	3	27	0.2		Yes
	Engineered Fill	Mohr-Coulomb	20	3	27	0.1	1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35		1	No
	Middle Clay	Mohr-Coulomb	20	3	27		2	No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35		2	No
	Waste	Mohr-Coulomb	10	5	25		1	No



Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section B - Liner 3	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27		1	No
	Chalk	Bedrock (Impenetrable)					1	No
	Engineered Clay	Mohr-Coulomb	20	3	27	0.2		Yes
	Engineered Fill	Mohr-Coulomb	20	3	27	0.1	1	Yes
	Lower sands and gravels	Mohr-Coulomb	19	0	35		1	No
	Middle Clay	Mohr-Coulomb	20	3	27		2	No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35		2	No
	Waste	Mohr-Coulomb	10	5	25		1	No

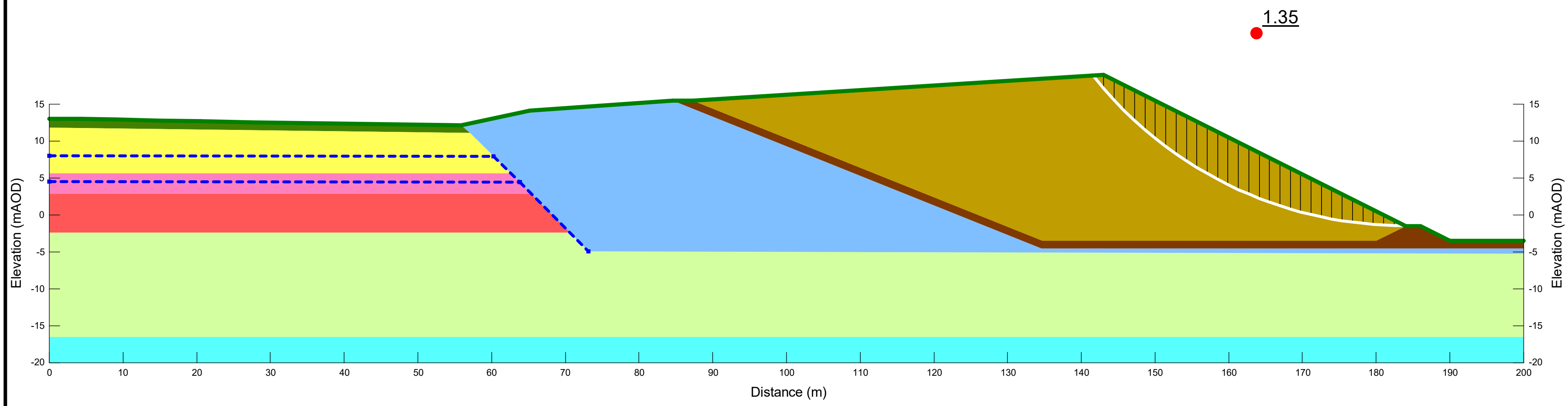


Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Liner Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section B - Liner 4	Project Manager:	N White

APPENDIX SRA4

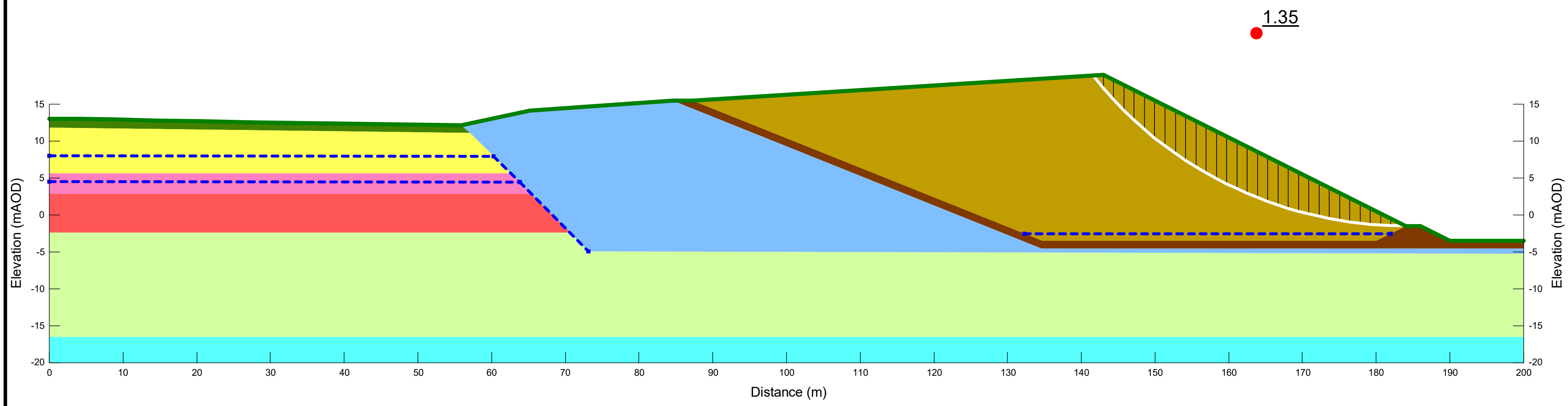
Temporary Waste Analyses

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Boulder Clay	Mohr-Coulomb	20	5	27	1
	Chalk	Bedrock (Impenetrable)				1
	Engineered Clay	Mohr-Coulomb	20	3	27	1
	Engineered Fill	Mohr-Coulomb	20	3	27	1
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1
	Middle Clay	Mohr-Coulomb	20	3	27	
	Topsoil	Mohr-Coulomb	20	2	26	
	Upper Sands	Mohr-Coulomb	19	0	35	2
	Waste	Mohr-Coulomb	10	5	25	1



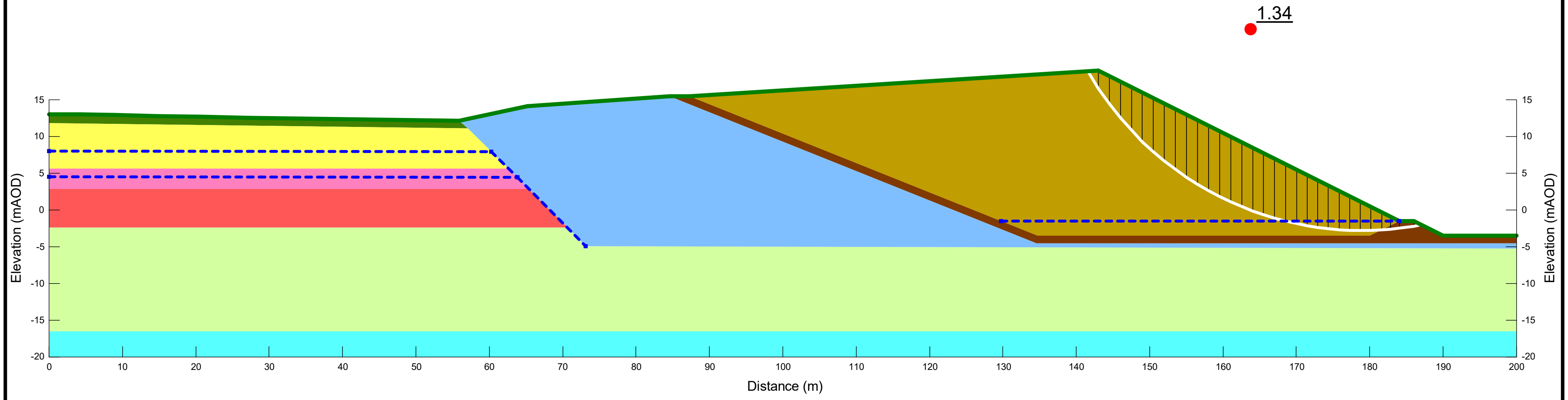
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Temporary Waste 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Boulder Clay	Mohr-Coulomb	20	5	27	1
	Chalk	Bedrock (Impenetrable)				1
	Engineered Clay	Mohr-Coulomb	20	3	27	1
	Engineered Fill	Mohr-Coulomb	20	3	27	1
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1
	Middle Clay	Mohr-Coulomb	20	3	27	
	Topsoil	Mohr-Coulomb	20	2	26	
	Upper Sands	Mohr-Coulomb	19	0	35	2
	Waste	Mohr-Coulomb	10	5	25	3



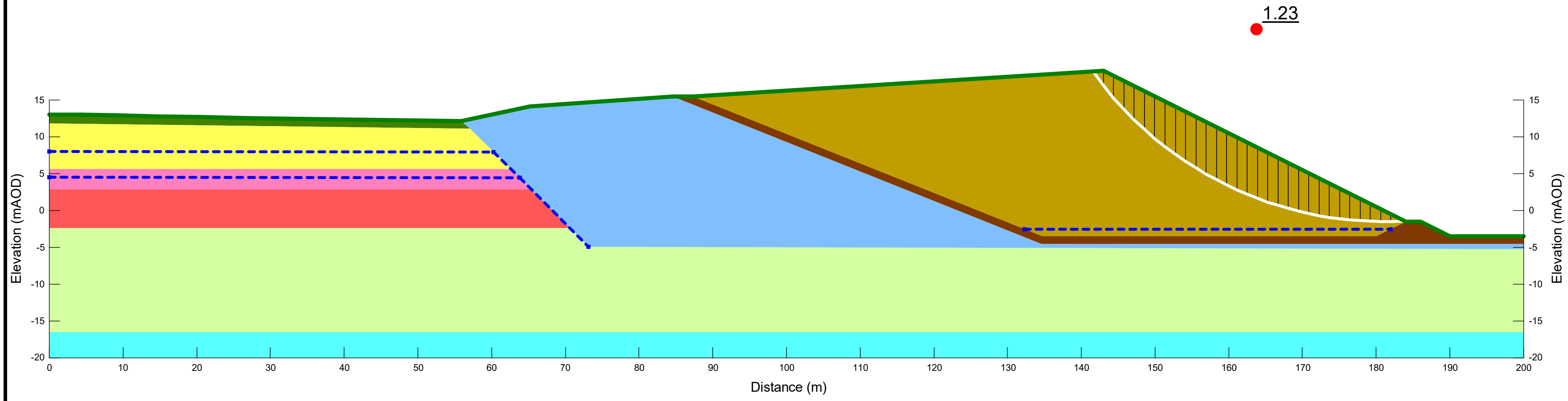
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Temporary Waste 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Boulder Clay	Mohr-Coulomb	20	5	27	1
	Chalk	Bedrock (Impenetrable)				1
	Engineered Clay	Mohr-Coulomb	20	3	27	1
	Engineered Fill	Mohr-Coulomb	20	3	27	1
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1
	Middle Clay	Mohr-Coulomb	20	3	27	
	Topsoil	Mohr-Coulomb	20	2	26	
	Upper Sands	Mohr-Coulomb	19	0	35	2
	Waste	Mohr-Coulomb	10	5	25	3



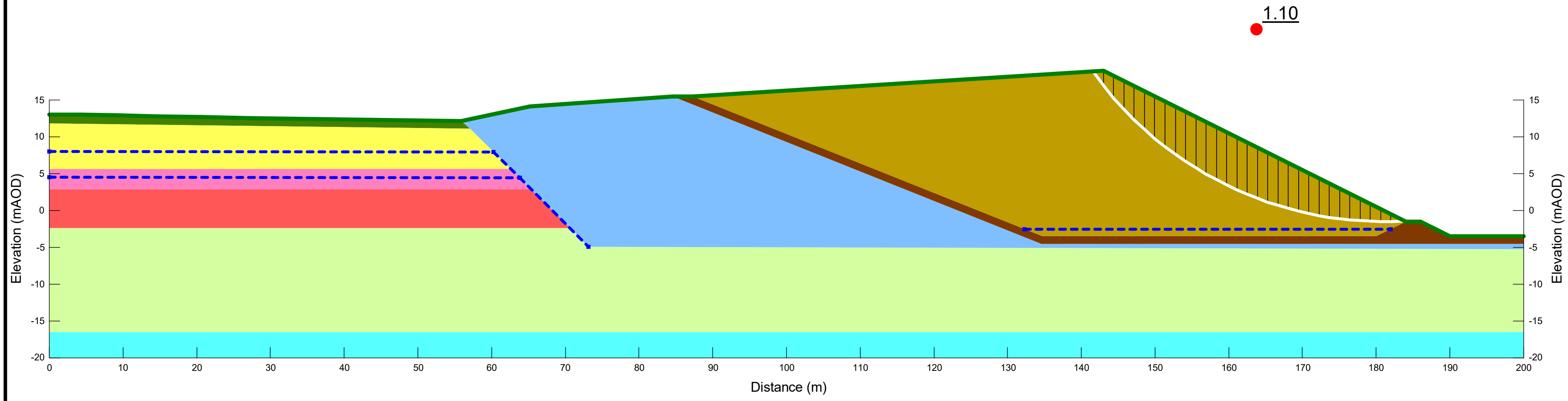
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Temporary Waste 3	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27		1	No
	Chalk	Bedrock (Impenetrable)					1	No
	Engineered Clay	Mohr-Coulomb	20	3	27		1	No
	Engineered Fill	Mohr-Coulomb	20	3	27		1	No
	Lower sands and gravels	Mohr-Coulomb	19	0	35		1	No
	Middle Clay	Mohr-Coulomb	20	3	27			No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35		2	No
	Waste	Mohr-Coulomb	10	5	25	0.1		Yes



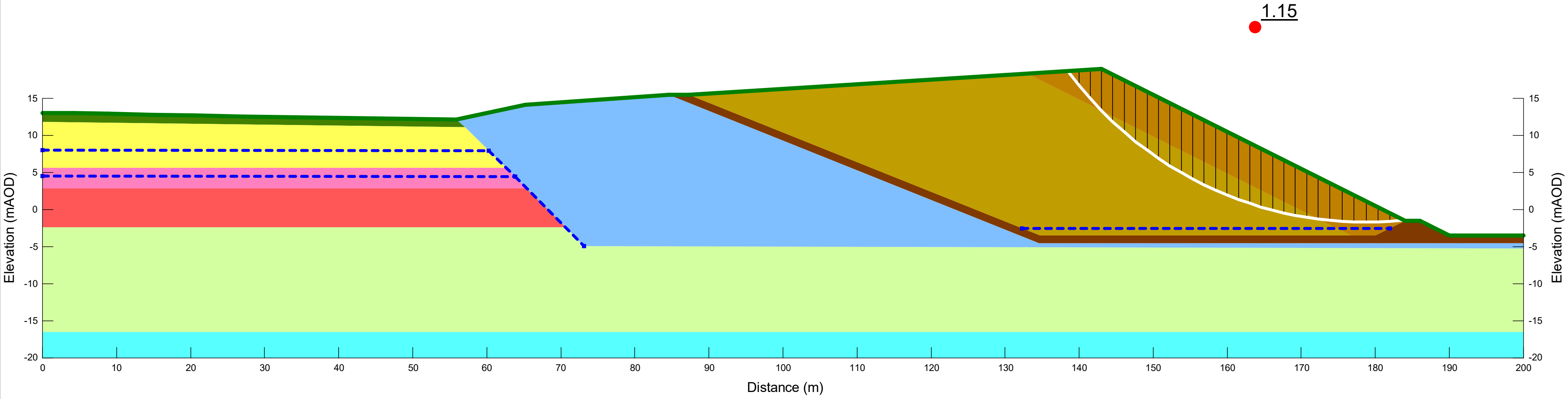
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Temporary Waste 4	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27		1	No
	Chalk	Bedrock (Impenetrable)					1	No
	Engineered Clay	Mohr-Coulomb	20	3	27		1	No
	Engineered Fill	Mohr-Coulomb	20	3	27		1	No
	Lower sands and gravels	Mohr-Coulomb	19	0	35		1	No
	Middle Clay	Mohr-Coulomb	20	3	27			No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35		2	No
	Waste	Mohr-Coulomb	10	5	25	0.2		Yes



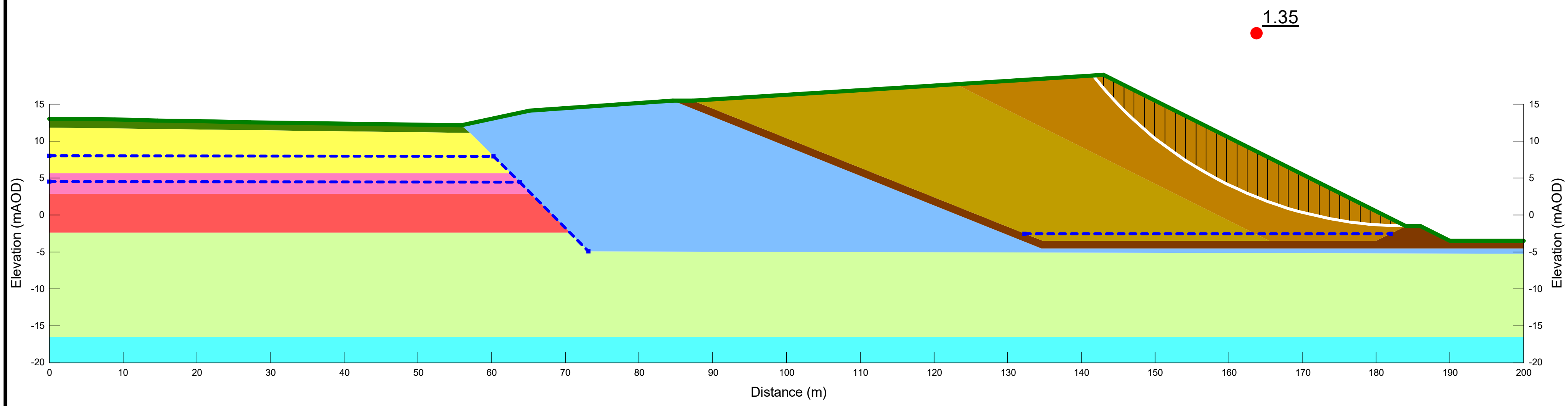
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Temporary Waste 5	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27		1	No
	Chalk	Bedrock (Impenetrable)					1	No
	Dry Waste	Mohr-Coulomb	10	5	25		1	No
	Engineered Clay	Mohr-Coulomb	20	3	27		1	No
	Engineered Fill	Mohr-Coulomb	20	3	27		1	No
	Lower sands and gravels	Mohr-Coulomb	19	0	35		1	No
	Middle Clay	Mohr-Coulomb	20	3	27			No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35		2	No
	Waste	Mohr-Coulomb	10	5	25	0.2		Yes



Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Temporary Waste 6	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Line	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27		1	No
	Chalk	Bedrock (Impenetrable)					1	No
	Dry Waste	Mohr-Coulomb	10	5	25		1	No
	Engineered Clay	Mohr-Coulomb	20	3	27		1	No
	Engineered Fill	Mohr-Coulomb	20	3	27		1	No
	Lower sands and gravels	Mohr-Coulomb	19	0	35		1	No
	Middle Clay	Mohr-Coulomb	20	3	27			No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35		2	No
	Waste	Mohr-Coulomb	10	5	25	0.2		Yes

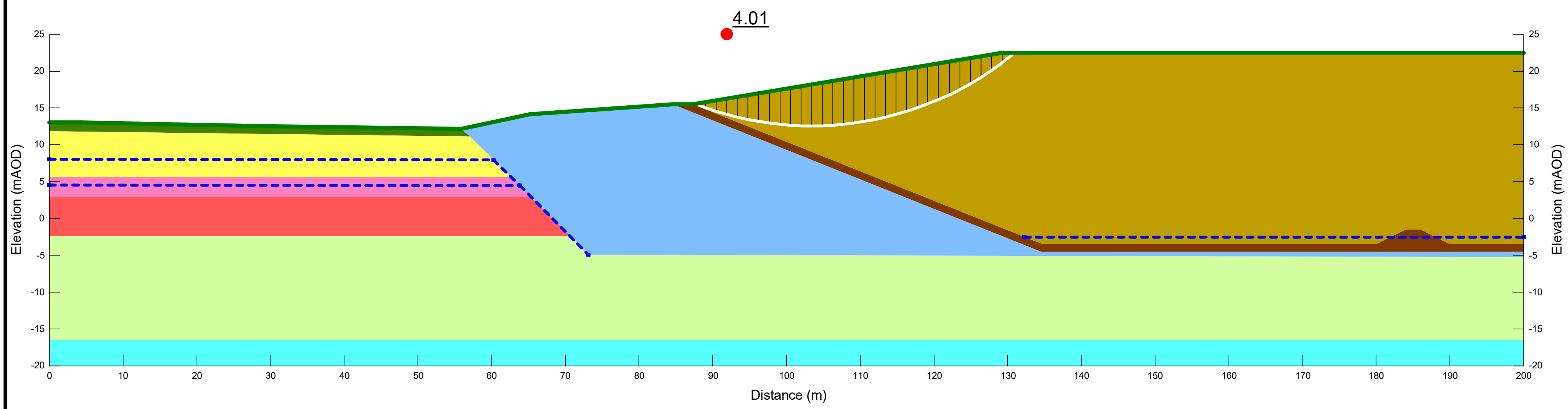


Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Temporary Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Temprory Waste 7	Project Manager:	N White

APPENDIX SRA5

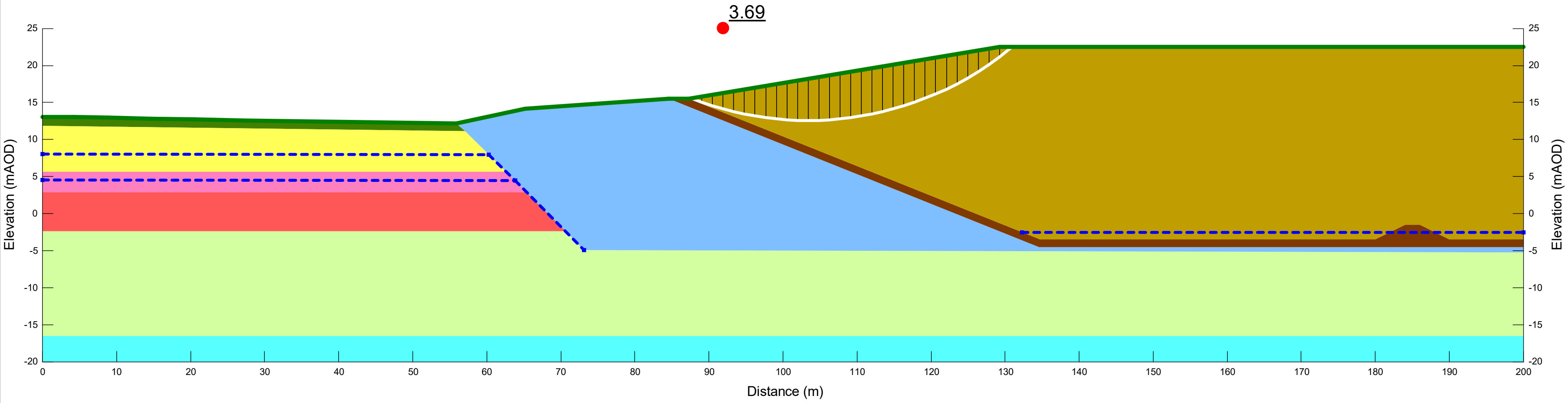
Final Waste Analyses

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line
	Boulder Clay	Mohr-Coulomb	20	5	27	1
	Chalk	Bedrock (Impenetrable)				1
	Engineered Clay	Mohr-Coulomb	20	3	27	1
	Engineered Fill	Mohr-Coulomb	20	3	27	1
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1
	Middle Clay	Mohr-Coulomb	20	3	27	
	Topsoil	Mohr-Coulomb	20	2	26	
	Upper Sands	Mohr-Coulomb	19	0	35	2
	Waste	Mohr-Coulomb	10	5	25	3



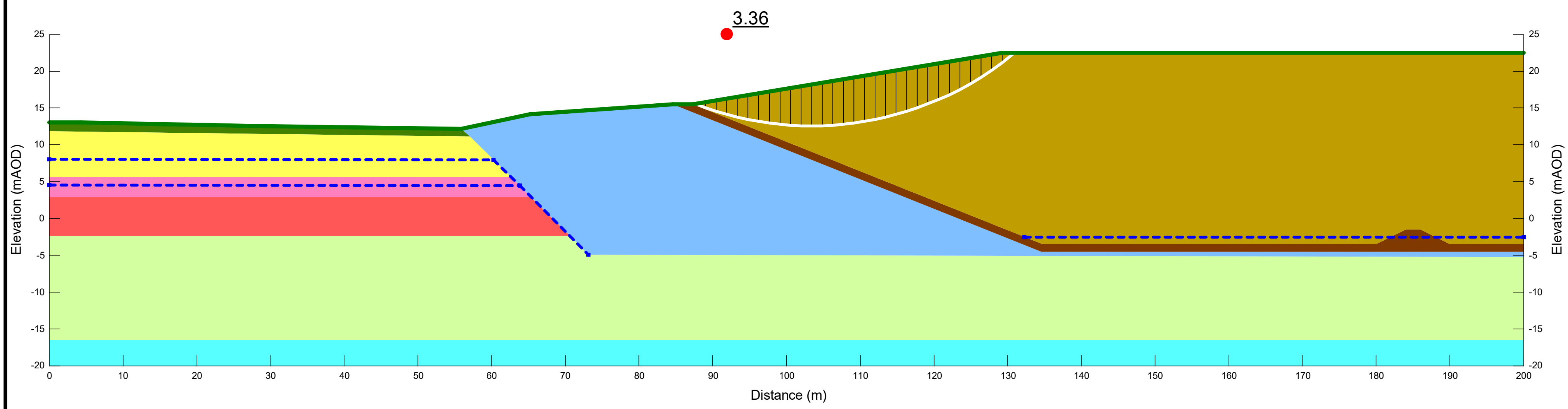
Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Final Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Final Waste 1	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Clay	Mohr-Coulomb	20	3	27	1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1		No
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27			No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No
	Waste	Mohr-Coulomb	10	5	25	3	0.1	Yes



Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Final Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Final Waste 2	Project Manager:	N White

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Line	Ru	Include Ru in PWP
	Boulder Clay	Mohr-Coulomb	20	5	27	1		No
	Chalk	Bedrock (Impenetrable)				1		No
	Engineered Clay	Mohr-Coulomb	20	3	27	1		No
	Engineered Fill	Mohr-Coulomb	20	3	27	1		No
	Lower sands and gravels	Mohr-Coulomb	19	0	35	1		No
	Middle Clay	Mohr-Coulomb	20	3	27			No
	Topsoil	Mohr-Coulomb	20	2	26			No
	Upper Sands	Mohr-Coulomb	19	0	35	2		No
	Waste	Mohr-Coulomb	10	5	25	3	0.2	Yes



Client:	Sandsfield Gravel Company Ltd	Consultant:	Golder Associates (UK) Ltd	Engineer:	W Y Htike
Site Name:	Milegate Eastern Extension Landfill	Filename:	Final Waste Analysis.gsz	Reviewer:	Dr B Zhang
Project Title:	Stability Risk Assessment	Analysis Ref:	Section A - Final Waste 3	Project Manager:	N White

APPENDIX SRA6

Geomembrane Capping Analyses

INTRODUCTION

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

Stability

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

Integrity

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

Geosynthetic

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

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

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

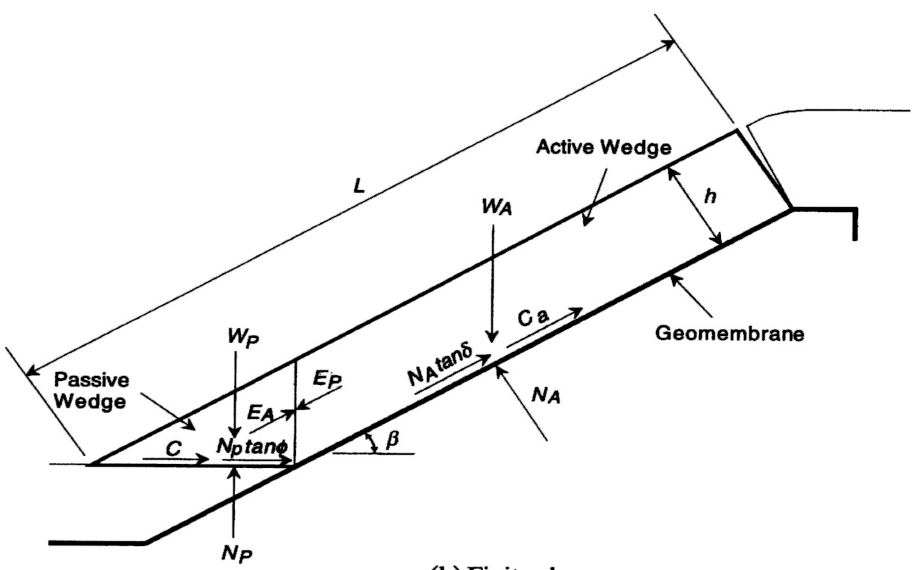
These values should be confirmed by site-specific shear strength testing. The values given above are all peak shear strengh values.

Section **A** **PSR** = 0.00

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

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

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	7	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geocomposite friction angle, δ_1	24	Deg.
Cover Soils/Geocomposite cohesion intercept, α_1	0	kPa
Geocomposite/GM friction angle, δ_2	26	Deg.
Geocomposite/GM cohesion intercept, α_2	0	kPa
GM/Blinding layer, δ_3	24	kPa
GM/Blinding layer, α_3	0	Deg.
Parallel submergence ratio, PSR	0.00	
Geosynthetic tensile strengths:		
Geotextile	10	kN/m
1mm LLDPE Geomembrane	11	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	42.41201	m	
Thickness of water, h_w	0	m	
Weight of active wedge, W_A	708.1281	kN	
Weight of passive wedge, W_P	55.28796	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	698.4166	kN	
Vert pp on passive wedge, U_V	0	kN	
a	115.272		
b	-341.467		
c	23.93203		
	Factor of Safety against cover soils sliding		**2.89**

2. Integrity of Geosynthetics

(i) Geocomposite

Mobilised shear stress at upper interface	115.9799	kN	
Shear strength at lower interface	367.2365	kN	
Tension developed in the GT	0	kN	
Tensile strength of the GT	10	kN	
	Factor of Safety against rupture		**Infinite**

(ii) GCL

Shear strength at upper surface	367.2365	kN	
Mobilised shear stress at upper interface	115.9799	kN	
Shear strength at lower interface	335.2333	kN	
Tension developed in the GM	0	kN	
Tensile strength of the GM	11	kN	
	Factor of Safety against rupture		**Infinite**

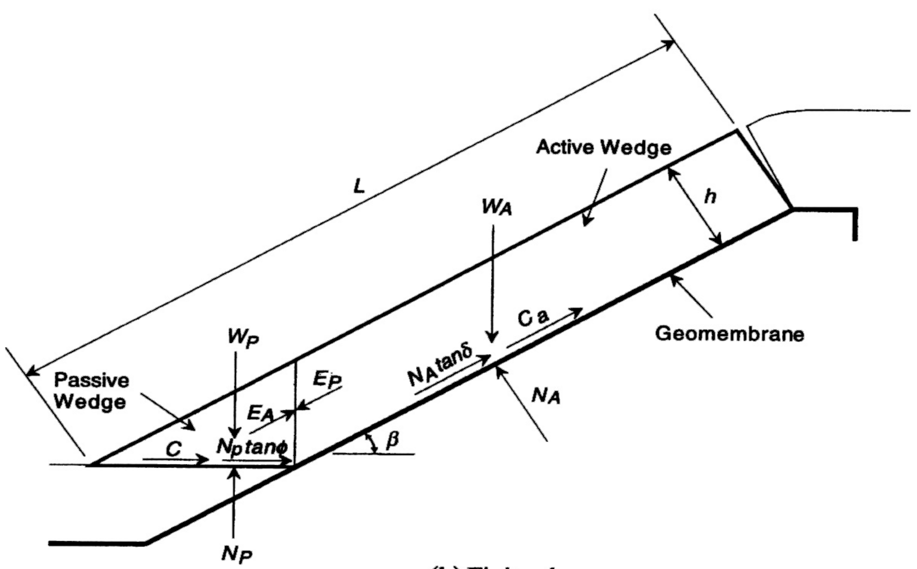
PROJECT	Milegate Eastern Extension Stability Assessment		
Job No.	20148978	Made By: DL	Date: 17/12/2021
Ref.	Appendix SRA6	Checked: BZ	Sheet: 4
		Reviewed: BZ	of: 7

Section **A** **PSR** = 0.50

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

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

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	7	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geocomposite friction angle, δ_1	24	Deg.
Cover Soils/Geocomposite cohesion intercept, α_1	0	kPa
Geocomposite/GM friction angle, δ_2	26	Deg.
Geocomposite/GM cohesion intercept, α_2	0	kPa
GM/Blinding layer, δ_3	24	kPa
GM/Blinding layer, α_3	0	Deg.
Parallel submergence ratio, PSR	0.50	
Geosynthetic tensile strengths:		
Geotextile	10	kN/m
1mm LLDPE Geomembrane	11	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	42.41201	m
Thickness of water, h_w	0.5	m
Weight of active wedge, W_A	749.0044	kN
Weight of passive wedge, W_P	56.82374	kN
Pore pressure perp. to slope, U_n	201.5782	kN
Pore pressure in interwedge surface, U_h	1.25	kN
Force normal to active wedge, N_A	537.3604	kN
Vert pp on passive wedge, U_V	7.469705	kN
a	121.96	
b	-268.401	
c	18.41325	

Factor of Safety against cover soils sliding **2.13**

2. Integrity of Geosynthetics

(i) Geotextile

Mobilised shear stress at upper interface	166.1428	kN
Shear strength at lower interface	387.6385	kN
Tension developed in the GT	0	kN
Tensile strength of the GT	10	kN

Factor of Safety against rupture **Infinite**

(ii) Geomembrane

Shear strength at upper surface	387.6385	kN
Mobilised shear stress at upper interface	166.1428	kN
Shear strength at lower interface	353.8574	kN
Tension developed in the GM	0	kN
Tensile strength of the GM	11	kN

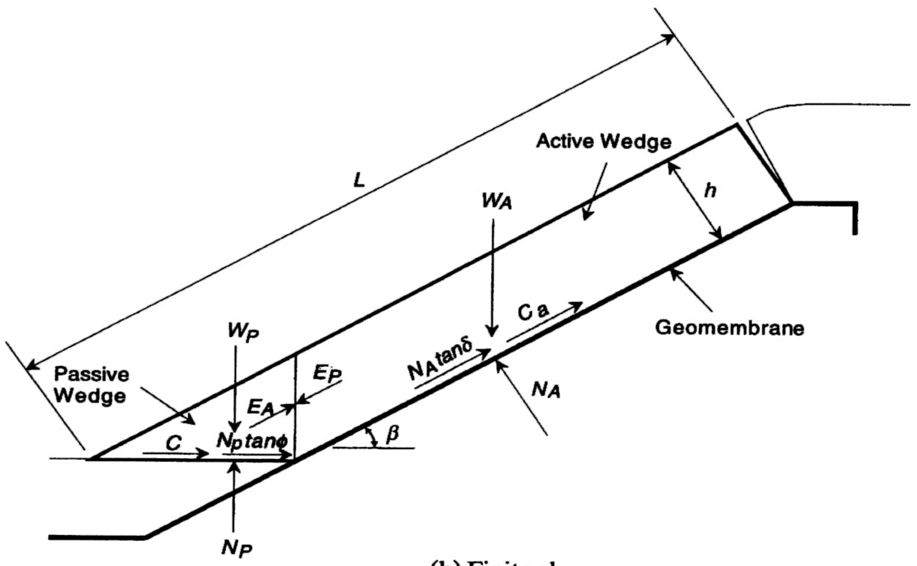
Factor of Safety against rupture **Infinite**

Section A **PSR** = 1.00

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

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

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	7	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geocomposite friction angle, δ_1	24	Deg.
Cover Soils/Geocomposite cohesion intercept, α_1	0	kPa
Geocomposite/GM friction angle, δ_2	26	Deg.
Geocomposite/GM cohesion intercept, α_2	0	kPa
GM/Blinding layer, δ_3	24	kPa
GM/Blinding layer, α_3	0	Deg.
Parallel submergence ratio, PSR	1.00	
Geosynthetic tensile strengths:		
Geotextile	10	kN/m
1mm LLDPE Geomembrane	11	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	42.41201	m	
Thickness of water, h_w	1	m	
Weight of active wedge, W_A	786.809	kN	
Weight of passive wedge, W_P	61.43107	kN	
Pore pressure perp. to slope, U_n	388.0092	kN	
Pore pressure in interwedge surface, U_h	5	kN	
Force normal to active wedge, N_A	388.8345	kN	
Vert pp on passive wedge, U_V	29.87882	kN	
a	128.2162		
b	-195.074		
c	13.32385		
Factor of Safety against cover soils sliding			**1.45**

2. Integrity of Geosynthetics

(i) Geotextile

Mobilised shear stress at upper interface	256.9249	kN	
Shear strength at lower interface	408.0405	kN	
Tension developed in the GT	0	kN	
Tensile strength of the GT	10	kN	
Factor of Safety against rupture			**Infinite**

(ii) Geomembrane

Shear strength at upper surface	408.0405	kN	
Mobilised shear stress at upper interface	256.9249	kN	
Shear strength at lower interface	372.4814	kN	
Tension developed in the GM	0	kN	
Tensile strength of the GM	11	kN	
Factor of Safety against rupture			**Infinite**

APPENDIX SRA7

GCL Capping Analyses

INTRODUCTION

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

Stability

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

Integrity

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

Geosynthetic

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

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

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

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

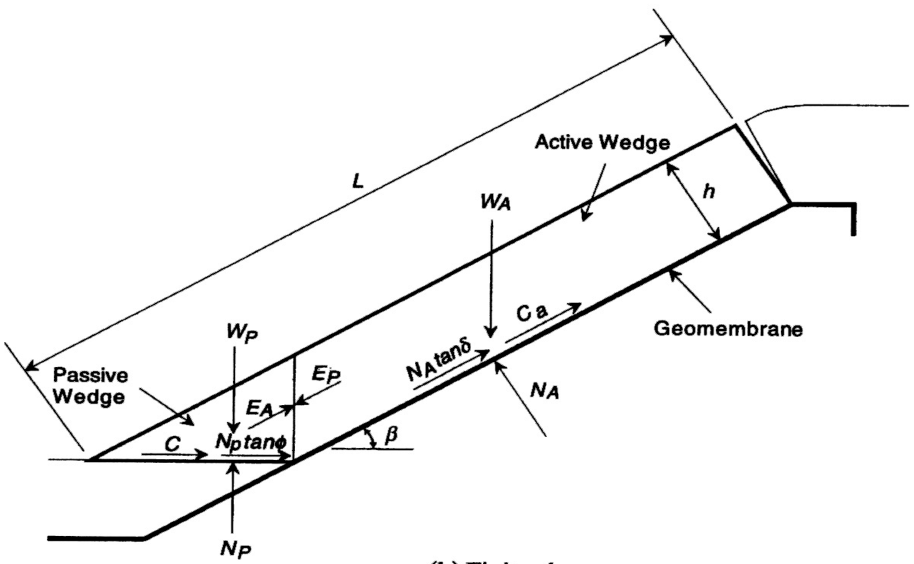
The tensile strength of the GCL has been taken from the a tpyical GCL cap product. A copy of the relevant data sheet is givn in the reference page.

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

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

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

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	7	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geotextile friction angle, δ_1	24	Deg.
Cover Soils/Geotextile cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	0	
Geosynthetic tensile strengths:		
GCL	12	kN/m

PROJECT	Milegate Eastern Extension Stability Assessment		
Job No. 20148978	Made By: DL	Date: 17/12/2021	
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	Reviewed: BZ	of: 9	

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	42.41201	m
Thickness of water, h_w	0	m
Weight of active wedge, W_A	708.1281	kN
Weight of passive wedge, W_P	55.28796	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	698.4166	kN
Vert pp on passive wedge, U_V	0	kN
a	115.272	
b	-341.467	
c	23.93203	

Factor of Safety against cover soils sliding **2.89**

2. Integrity of Geosynthetics

(i) Geosynthetic Layer No.1

Mobilised shear stress at upper interface	115.9799	kN
Shear strength at lower interface	335.2333	kN
Tension developed in the geosythetic	0	kN
Tensile strength of the geosythetic	12	kN

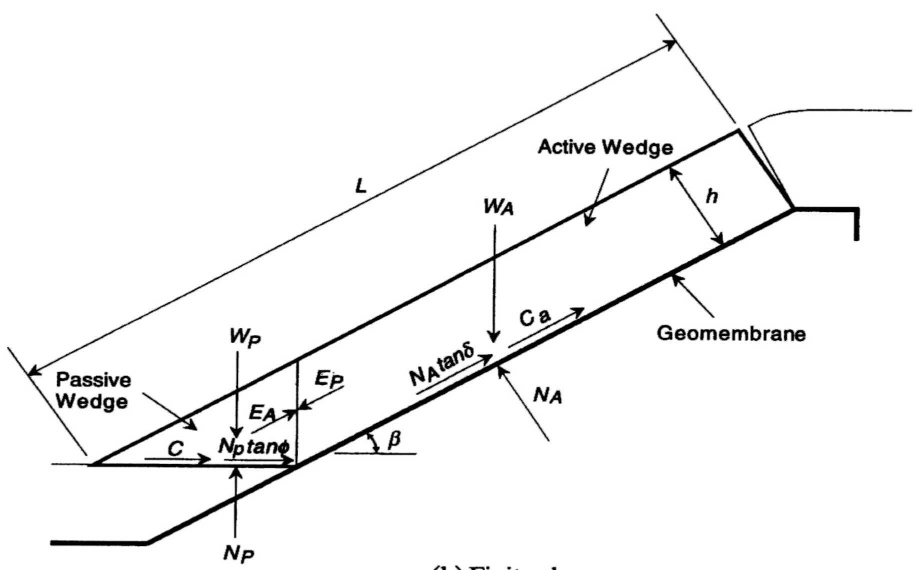
Factor of Safety against rupture **Infinite**

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

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

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

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	7	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geotextile friction angle, δ_1	24	Deg.
Cover Soils/Geotextile cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	0.5	
Geosynthetic tensile strengths:		
GCL	12	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	42.41201	m
Thickness of water, h_w	0.5	m
Weight of active wedge, W_A	749.0044	kN
Weight of passive wedge, W_P	56.82374	kN
Pore pressure perp. to slope, U_n	201.5782	kN
Pore pressure in interwedge surface, U_h	1.25	kN
Force normal to active wedge, N_A	537.3604	kN
Vert pp on passive wedge, U_V	7.469705	kN
a	121.96	
b	-268.401	
c	18.41325	

Factor of Safety against cover soils sliding **2.13**

2. Integrity of Geosynthetics

(i) Geosynthetic Layer No.1

Mobilised shear stress at upper interface	166.1428	kN
Shear strength at lower interface	353.8574	kN
Tension developed in the geosythetic	0	kN
Tensile strength of the geosythetic	12	kN

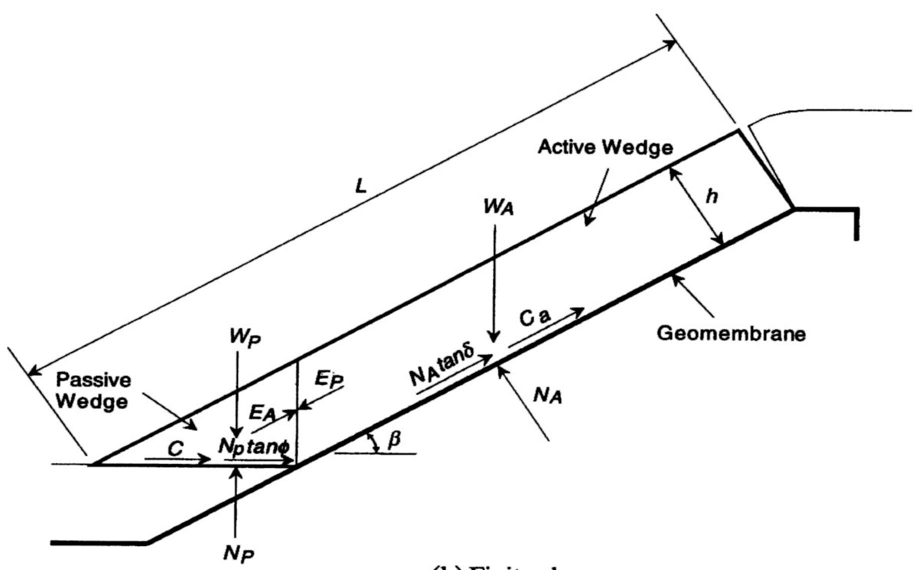
Factor of Safety against rupture **Infinite**

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

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

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

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1	m
Height of slope, H	7	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/Geotextile friction angle, δ_1	24	Deg.
Cover Soils/Geotextile cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	1	
Geosynthetic tensile strengths:		
GCL	12	kN/m

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	42.41201	m
Thickness of water, h_w	1	m
Weight of active wedge, W_A	786.809	kN
Weight of passive wedge, W_P	61.43107	kN
Pore pressure perp. to slope, U_n	388.0092	kN
Pore pressure in interwedge surface, U_h	5	kN
Force normal to active wedge, N_A	388.8345	kN
Vert pp on passive wedge, U_V	29.87882	kN
a	128.2162	
b	-195.074	
c	13.32385	

Factor of Safety against cover soils sliding **1.45**

2. Integrity of Geosynthetics

(i) Geosynthetic Layer No.1

Mobilised shear stress at upper interface	256.9249	kN
Shear strength at lower interface	372.4814	kN
Tension developed in the geosythetic	0	kN
Tensile strength of the geosythetic	12	kN

Factor of Safety against rupture **Infinite**

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

Non-woven geotextile

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

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

Table 3 Summary of results for non-woven geotextile

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

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

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

Fibre-reinforced Geosynthetic Clay Liner (GCL)

Bentofix® NSP 4900

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E-Mail: info@naue.com · Internet: www.naue.com

0799-CPD-20

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

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

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

APPENDIX SRA8

Leachate System Analyses

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

Background: The leachate extraction wells comprise 0.9 m internal diameter, reinforced concrete chamber.
The base comprises a 300 mm thick, 3000 mm square concrete slab.
The leachate well will be built up with the waste, with a maximum height of 23.0 m (including 1.0 m of drainage gravel on top of the slab and 1.0 m of restoration soils).

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

Assumptions:

Unit weight of concrete, γ_{conc}		=	24	kN/m^3
Unit weight of clay, γ_{Clay}		=	20	kN/m^3
Unit weight of gravel, γ_{gravel}		=	18	kN/m^3
Unit weight of restoration soils, γ_{rest}		=	18	kN/m^3
Unit weight of waste, γ_{waste}		=	10	kN/m^3
Shear strength of the clay liner (total stress), c_u		=	50	kPa
Shear strength of the clay liner (effective stress),	c'	=	3	kPa
	ϕ'	=	26	degrees
Friction angle between waste and concrete, δ=			12	degrees
Waste coefficient, $K_{waste}(\sigma_h'/\sigma_v')$		=	0.4	

Calculations:

1. Loading from various components

(a) Self weight of concrete chamber

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

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

Load = **162.1** kN

(b) Concrete slab loading

3 x **3** m

Thickness = **0.3** m

Unit weight of concrete = 24 kN/m^3

Load = Volume x γ_{conc}

Load = **64.8** kN

Calculations:

Loading from various components (Cont'd.)

(c) Waste load on extraction slab

Slab area = 9 m^2

Pipe area = π x D_e^2 / 4 = 0.95 m^2

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

Load = **1,851.4** kN

(d) Gravel load on extraction slab

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

Thickness of Gravel $\boxed{1}$ m

Load = **144.9** kN

(e) Cap and Restoration load on extraction slab

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

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

Restoration Thickness = $\boxed{1}$ m

Load = **144.9** kN

(f) Negative skin friction loading on concrete pipe

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

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

Load = NSF x surface area

Load = NSF x π x External diameter x total height

Load = **840.2** kN

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

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

Load = 266.0 kPa

Calculations:

Summary of loadings

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

2. Bearing capacity

(i) Total stress

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

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

where:

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

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

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

γD = **266.0** kPa

For c_u = 50 kPa

q_f = **523.0** kPa

Factor of safety against shear failure is given by:

$F = q_f / q$

Factor of safety: **1.5**

Calculations:

(ii) Effective stress

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

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

where:

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

B = width of slab

c = cohesion of the Clay

p_o = effective stress of overburden soil at foundation level

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

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

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

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

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

N_q = **11.85**

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

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

N_γ = **8.00**

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

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

N_c = **22.25**

Hence, q_f = **3117.8** kPa

Factor of safety against shear failure is given by:

$F = q_f / q$

Factor of safety: **23.9**

Calculations:

3. Settlement

Using the Skempton-Bjerrum method for consolidation settlement:

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

where:

m_v = 0.1 m^2/MN

μ = 0.5

H = 0.5 m (thickness of Clay liner)

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

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

(a) Settlement under extraction slab

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

ρ_{consol} = 0.1 0.5 x 352.9 x 0.5

ρ_{consol} = 8.8 mm

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

ρ_{tot} = 1.5 x 8.8 = **13.2** mm

(b) Settlement under waste only

Maximum value of q = 266.0

ρ_{consol} = 0.1 x 0.5 x 266 x 0.5

ρ_{consol} = 6.7 mm

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

ρ_{tot} = 1.5 x 6.7 = **10.0** mm

(c) Differential settlement:

Settlement beneath slab = 13.2

Settlement beneath waste = 10.0

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

Conclusions:

Both bearing capacity and anticipated settlement are considered satisfactory.

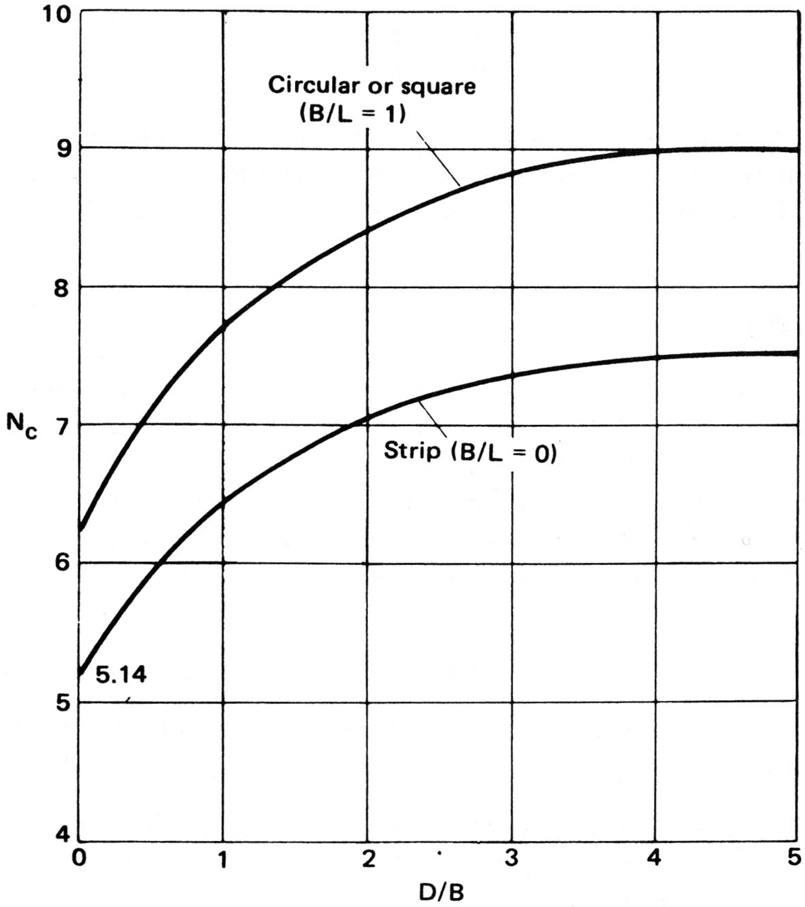


Fig. 8.5 Skempton's values of N_c for $\phi_u = 0$. (Reproduced from A.W. Skempton (1951) *Proceedings of the Building Research Congress*, Division 1, p. 181, by permission of the Building Research Establishment, © Crown copyright.)

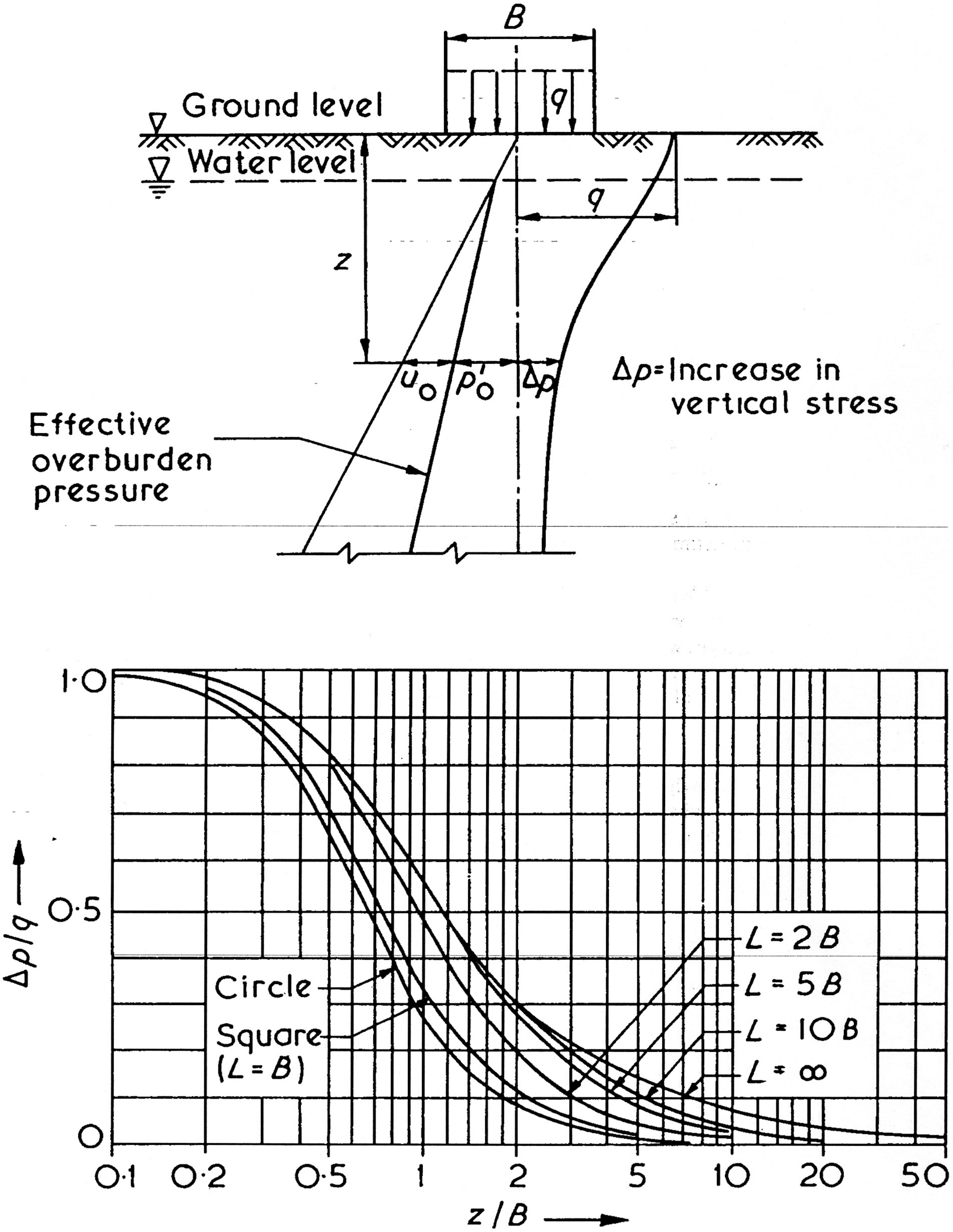


Fig. 3.3 Determination of increase in vertical stress under the centre of uniformly loaded flexible footings, after Janbu, Bjerrum and Kjaernsli (1956)

APPENDIX SRA9

Leachate Pipework Deflection Analyses

Leachate Pipework Strength Calculations

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

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

References:

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

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

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

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

Where:

W_c = Static Loading (simple prismatic loading is assumed)

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

= ((22.0 m x 10 kN/m^3) + (0.45 m x 18 kN/m^3) + (1 m x 18 kN/m^3)) x 0.16

= **39.376** kN/m

D_L = Deflection lag factor (dimensionless)

= **1.5** (assumed)

K_x = Bedding factor

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

r = Mean radius of pipe

= **80** mm

t = Wall thickness of pipe

= **9.412** mm

I = Moment of inertia of pipe wall per unit length

= **69.5** mm^3

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

= **150,000** kPa

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

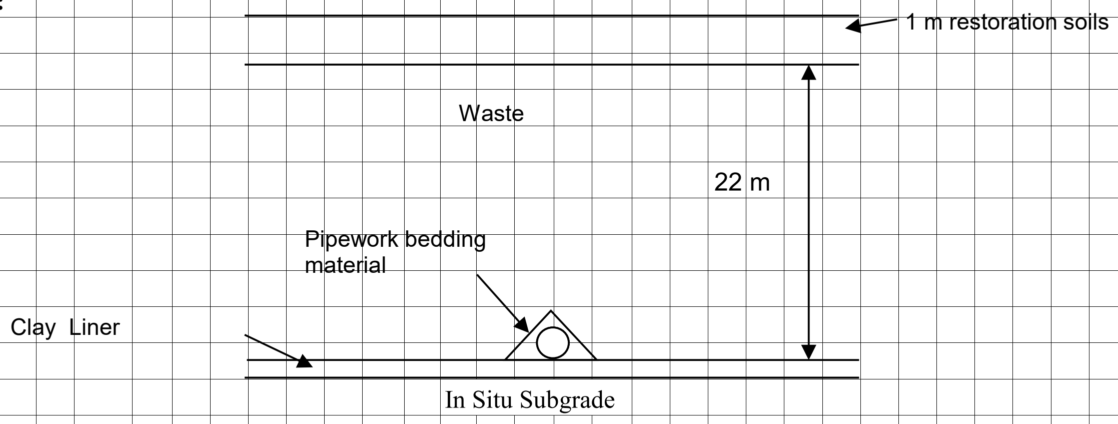
= **20.4** kPa

`

E' = Modulus of soil reaction

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

Geometry:



Calculation:

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

δ_v	=	**0.005**	m
	=	**4.67**	mm
	=	**2.9**	% of the nominal pipe inside diameter

The calculations indicate that once the waste has been placed, the leachate drainage pipe will deflect up to approximately 2.6%. It is envisaged that this amount of deflection will not result in intergrity failure of the pipe.

Leachate Pipework Strength Calculations

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

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

References:

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

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

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

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

Where:

W_c = Static Loading (simple prismatic loading is assumed)

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

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

= **29.078** kN/m

D_L = Deflection lag factor (dimensionless)

= **1.5** (assumed)

K_x = Bedding factor

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

r = Mean radius of pipe

= **60** mm

t = Wall thickness of pipe

= **7.1** mm

I = Moment of inertia of pipe wall per unit length

= **29.3** mm^3

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

= **150,000** kPa

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

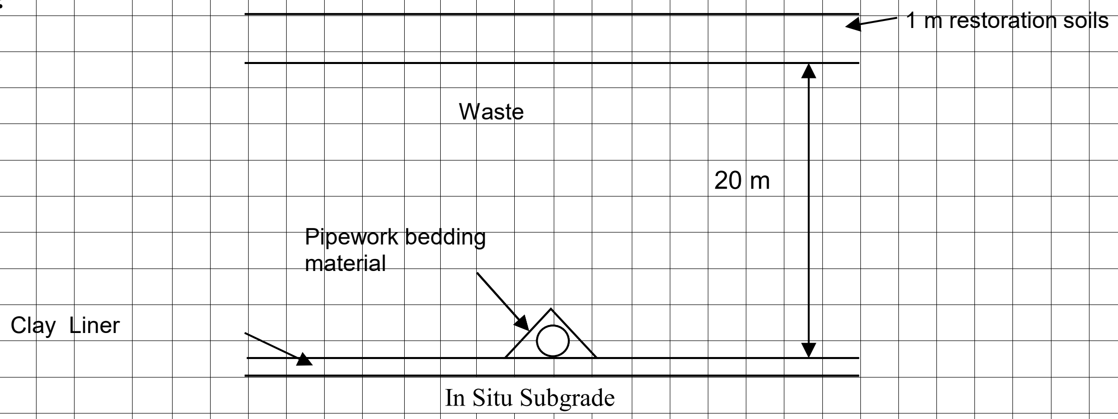
= **20.4** kPa

`

E' = Modulus of soil reaction

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

Geometry:



Calculation:

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

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

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

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

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

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

[A] ASTM Designation D-2487

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

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

[D] Relative Density per ASTM D-2049.

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

Source: After Howard [14].

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