Docksway Disposal Site, Newport

Revised Area 2 Stability Assessment

For Permit Revision Application

On behalf of **Newport City Council**

Project Ref: 14739/155 | RevA - | Date: April 2015

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1 Introduction

1.1 General

1.1.1 Docksway Disposal Site is a non-hazardous waste disposal site located approximately 3km south of Newport City Centre, Gwent and is centred on National Grid Reference ST 305 853. The site is operated by Newport City Council under Environmental Permit No. EPRDP3733BK, and is split into a northern half (Area 1) and a southern half (Area 2). Area 1 of the site is a closed 'dilute and disperse' landfill and Area 2 of the site is engineered and constructed as a full containment landfill. The site operator is applying for a substantial variation to the existing Environmental Permit that includes a Stable Non-Reactive Hazardous Waste (SNRHW) cell for the disposal of asbestos, and to increase the side slope angles and final restored height of the landfill across Area 2.

1.1.2 This report has been prepared by Peter Brett Associates LLP (PBA) on the instruction of Newport City Council to address the proposed changes to the landfill geometry and waste properties proposed as part of the permit revision. This Report should be read in conjunction with the following PBA reports:

1.1.3 Stability Assessment Report for Area 2 – Landfill Extension. Report (PBA, 2005)

1.1.4 Area 2 Stability Assessment Addendum Report (PBA, 2006)

1.1.5 However for ease of reference appropriate sections from the two reports referred to above have been incorporated into this report.

1.1.6 The work has been carried out generally in accordance with published Environment Agency (EA) guidance Stability of Landfill Lining Systems Report No 2 Guidance R&D Technical Report P1-385/TR2 (EA, 2003a).

1.1.7 Details of the stability assessment carried out together with the stability calculations are also described in this report and the stability calculations are contained in Appendices.

1.1.8 Reference should be made to section 16 of this report for guidance essential to all readers of this report.

2 Current Proposal for Area 2

2.1 General

2.1.1 The location of the site is indicated on Figure 1 this also indicates the location of Area 2 relative to Area 1. The current proposals include a revised landfill geometry and include an asbestos waste cell within one of the permitted five cells as indicated on Figure 2. The proposals include for steeper side slopes of the landfill from the original 1v: 5.5h up to 1v:4h around the perimeter where capping has not already been undertaken. In addition the final finished level of the top of the landfill is to be 40mOD.

2.1.2 At the current time, two (Cells 1 and 2) of the five permitted cells in Area 2 have been developed. In the area of the currently undeveloped Cells 3 and 4 it is intended that the soft alluvial clay soils within the former river channel alignment will be stabilised to increase the strength of these soils to support construction plant in order that the engineered clay liner can be placed and the landfilling progressed. The stabilisation will be restricted to the upper 2m of the soft alluvial soils so as not to extend too close to the underlying River Gravels. In later parts of this report reference is made to a stabilised layer and it is this stabilisation of the upper surface of the alluvial soils to which it refers.

3 The Site

3.1 Site Location

3.1.1 The site is located at the Docksway Disposal Site, approximately 3km south of Newport City Centre, Gwent. The approximate National Grid Reference for the centre of the site is ST 307 855 (Figure 1).

3.1.2 The A48 Docks Way/Usk Way road lies to the north of the site. The area to the north of the A48 is largely residential but includes the Maes Glas Industrial Estate. The area to the west of the site (across the River Ebbw) is grassland and is known to have been used historically for landfilling. The Newport Docks are located adjacent to the eastern boundary of the site and the land (owned by Associated British Ports) is used by a variety of different industries including a timber treatment works, a fertiliser manufacturer and a waste transfer station. The River Ebbw forms the southern boundary of the site. There is a ditch/stream present within part of the north eastern boundary of the site that takes runoff from the docks and is culverted beneath the site before discharging to the River Ebbw to the west of the site.

3.2 Site Description

3.2.1 The site lies within the flood plain of the River Severn Estuary and is bound to the west by the meandering channel of the River Ebbw and to the east by Newport Docks and the River Usk beyond. Prior to landfilling operations at the site the area would have had a fairly flat, low-lying relief. Historical information indicates that original ground level in the area of the subject site would have been approximately 7mOD with the bed of the River Ebbw at approximately 2mOD.

3.2.2 For licensing and IPPC Permit application purposes the site is divided into 2 areas;

3.2.3 Area 1 comprises the northern part of the site and is occupied by an older closed landfill.

3.2.4 Area 2 is the `southern part of the site where the extension to the landfill has commenced and two cells are already in the process of being filled.

3.2.5 The extreme northern part of Area 1 is occupied by the original Disposal Site (approximately 5 hectares). This comprises an area where landfilling operations have now ceased and that has been restored to final restoration contours at approximately 15mOD. In addition there is a further area of landfill which has been raised to an elevation of approximately 36mOD. Landfilling operations at the Area 1 part of the site are now complete. The final maximum elevation for the current landfill in Area 2 is forecasted to be approximately 32mOD following the settlement of the waste.

3.2.6 The Area 2 landfill occupies the southern part of the site. The total area that will be used for landfilling in Area 2 is approximately 15.4 ha and the current permit allows for landfilling in five cells. The surface of this area is currently very undulatory and is cut by an oxbow of the former course of the River Ebbw. The area was reclaimed in 1990 following straightening of the River Ebbw when dams were formed at either end of the river meander to cut off the oxbow and the course of the River Ebbw was artificially straightened. The locations of the dams are indicated on Figure 2. In addition internal dams used to be present crossing the former river channel alignment which were constructed to allow access across the river channel. These internal dams have been replaced by the engineered clay bunds that form the northern ends of Cells 1 and 2. The general ground levels around the perimeter of the site range between elevations of about 6.5mOD and 9.5mOD.

3.2.7 Original survey information indicated that ground levels in the area of the former river meanders ranging between about 1mOD and 3.5mOD. However more recent survey data indicates that in the area of Cell 1 the ground levels in the former river meanders range between about 2.3mOD and 3.2mOD. The undulatory nature of this part of the site is partly due to clay removal for use in the existing landfill, and the old river channels. The site was flooded within the former river meanders until landfilling operations commenced in Area 2 with relatively good quality surface run-off water forming three separate lakes. Since construction of the first two cells the area of these water bodies reduced leaving two smaller lakes in the north eastern part of Area 2 in which water levels have been regulated over the past few years by pumping the water out in preparation for engineering the remaining part of Area 2.

3.2.8 The older Landfill in Area 1 is now closed. The boundary of the permitted area for Area 2 is shown in Figure 2. The two areas of landfill are completely separate and there is a cut-off in place along the perimeter of Area 1 as indicated on Figure 2.

4 Geology and Hydrogeology

4.1 Published Geology

4.1.1 The geology of the Newport area as shown on the British Geological Survey 1:50,000 Scale Sheet 249 (BGS, 1997) indicates the site to be covered by Marine Beach Deposits or Tidal Flat overlying Marine or Estuarine Alluvium overlying Mercia Mudstone.

4.1.2 Marine Beach Deposits predominately comprise sands and gravels and the Alluvium consists mainly of alluvial mud, silts and sands with a little gravel (BGS, 1948).

4.1.3 The Mercia Mudstone bedrock comprises red, brownish red or purplish red mudstone or silty mudstone. Locally the Mercia Mudstone contains thin bands of siltstone and silty limestones

4.1.4 Made Ground is present overlying the natural superficial deposits over some of the site due to raising of ground levels around the perimeter and alongside the River Ebbw, together with construction of bunds, roadways and dams .The natural topography and man-made changes to the topography is such that the thickness of each strata vary in different parts of the site.

4.2 Hydrogeology and Hydrology

4.2.1 The specific details on the hydrogeology and hydrology at the site are discussed in separate reports prepared by Peter Brett Associates (PBA, 2004a 2004b and PBA 2015) and hence are only briefly discussed in this report for completeness.

4.2.2 The regional hydrogeology for the site is described in outline on the 1:125,000 hydrogeology map for South Wales (BGS, 1986). The site is shown to be underlain by ‘alluvial deposits and peat’ beneath which is ‘sand and gravel’.

4.2.3 Close to the main rivers the natural groundwater quality in these deposits is described as similar to that of the river with which they are in hydraulic continuity (i.e. the estuarine Ebbw in the case of gravels beneath the site). Iron and manganese concentrations are also reported as undesirably high (BGS, 1986).

4.2.4 The Mercia Mudstone Group is described as approximately 190m thick across the map area comprising mudstones over a conglomerate layer at the base which is up to 60m thick. The map states that the mudstones act as an impermeable top to the basal conglomerate. This description is echoed in the Policy and Practice for the Protection of Groundwater, Welsh Regional Appendix, (EA, 1995).

4.2.5 The Maes Glas Pill, a stream to the east of the Area 1 Disposal Site, was formerly culverted along the southern edge of the Area 1 Disposal Site to emerge as an outfall into the River Ebbw immediately south of Area 1. It has since been diverted into a new culvert running further north through Area 1.

4.3 Ground Investigation Data

4.3.1 Various ground investigations have been undertaken at the site and these are reported in the reports listed below:

- Structural Soils Limited, 1994
- Exploration Associates Limited, 1996
- LG Mouchel and Partners Ltd, 1996

- Gwent Consultancy, 1999
- CJ Associates Geotechnical Limited, 2003
- CJ Associates Geotechnical Limited, 2004
- PBA, 2004c
- PBA, 2004d
- PBA 2012

4.3.2 It should be noted that the ground conditions identified by the various ground investigations indicate that the underlying geology is not exactly as indicated by the BGS. It has generally been identified that the nature of the natural succession beneath the site comprises Alluvium overlying River Gravels which in turn overlie the Mercia Mudstone.

5 Ground Conditions

5.1 General

5.1.1 This section summarises the geology encountered in all the previous ground investigations undertaken at the site associated with Area 2. Reference should be made to the various factual reports which are listed in section 4.3 for factual information on the ground conditions. A cross section across the site indicating the general geological conditions is presented as Figure 3.

5.2 Made Ground

5.2.1 Made Ground was encountered in many of the exploratory holes within Area 2 and typically ranged in thickness between about 0.5m and about 5.5m. The greater depths of Made Ground were at the locations of the northern and southern dams the positions of which are indicated on Figure 2.

5.2.2 The Made Ground in Area 2 generally comprises a mixture of reworked natural deposits with building rubble for example concrete and occasionally ash and coal.

5.3 Alluvium

5.3.1 The Alluvium beneath the site generally comprises a firm to stiff grey clay/silt between 1m and 2m thick, probably being a desiccated crust. The firm crust has however been removed from a large part of the area of Area 2 for use in the Area 1 landfilling. Beneath the firm upper crust and in the base of the excavated area and original course of the river there is very soft to firm grey silt or clay present with sand and gravel at the base.

5.3.2 The Alluvium varies in thickness ranging from about 2m to about 9m with an elevation at the top of the stratum varying from about 8mOD to about 1mOD.

5.4 River Gravels

5.4.1 The River Gravels were encountered below the Alluvium comprising medium dense to very dense sandy gravels containing clay, cobbles and boulders. There is a slight trend for the relative densities as indicated by the results of SPT N values to increase with depth from medium dense to very dense.

5.4.2 The thickness of the River Gravels was found to vary from about 4m to 9m thick, though the thickness was not proven in all the exploratory holes. The elevation at which the top of the stratum was encountered varies from about -0.5mOD to about -3.5mOD.

5.5 Mercia Mudstone

5.5.1 Mercia Mudstone was encountered at depth below the River Gravels and was found to comprise a weathered very stiff red brown clay or silt.

5.5.2 The full thickness was not proven and the upper surface was encountered at an elevation of between about -5mOD and -11mOD.

5.6 Groundwater

5.6.1 Details of the groundwater regime are given in the original Hydrogeological Risk Assessment (PBA, 2004a), and the revision prepared to support the permit variation application (PBA, 2015). The groundwater levels monitored on site have varied between 0.5mOD and 4,15mOD this is based on Figures 8A to 8J of the Conceptual Model in the Hydrogeological Assessment Report for Area 2 Landfill Extension Report (PBA 2004a).

5.6.2 The pore water pressures within the Alluvium beneath the landfill in Cell 2 of Area 2 have been monitored using vibrating wire piezometers over the period from their installation in October 2010 to January 2015.This monitoring data has been reviewed within the assessment presented in this report to determine appropriate criteria in respect of piezometric pressure to be used in the stability assessment.

6 Geotechnical Parameters

6.1 General

6.1.1 The geotechnical parameters adopted for the stability analysis presented in this report are based on the results of in situ and laboratory testing provided in the various ground investigation reports as listed in section 4.3.

6.1.2 The reports listed should be referred to in conjunction with this report for full details on the in situ and laboratory analyses undertaken.

6.2 Made Ground

6.2.1 The Made Ground around the perimeter and forming the north and south dams along the southern side of the site the positons of which are indicated on **Figure 2** generally comprise a mixture of building rubble with some natural clay soils and occasional traces of refuse type material. No laboratory testing of the Made Ground was carried out on any samples from the ground investigations due to its very variable nature and composition and its general unsuitability as a formation or engineering material.

6.2.2 In respect of the Made Ground forming the dams, based on visual observations, it generally includes a greater percentage of granular type material including large fragments of concrete and other building and demolition type material. Based on guidance in BS 8002:1994 and assuming the Made Ground is mostly granular with little cohesive matrix the drained angle of friction (ϕ') can be estimated as 32° based on the soil particles being sub-angular and of poor grading. As there is likely to be some cohesive matrix it could be expected that the Made Ground would have a nominal cohesion (c') and a value of 4kN/m² is considered appropriate for design.

6.2.3 The above values for the Made Ground forming the bulk of the dams are considered conservative estimates appropriate for the design in the absence of any specific laboratory test data. In addition it is likely that in the un-drained condition values of ϕ_u and c_u could be greater than the estimated conservative values of ϕ' and c'. However in the absence of any test data for the Made Ground it is considered that un-drained parameters equal to the drained values would be the most appropriate for the design and have therefore been used in this assessment.

6.2.4 In respect of the Made Ground comprising non-hazardous waste materials, the published information (Jones et al., 1997) indicates measured effective shear strength parameters ranging from zero to 28kN/m² for effective cohesion (c') and 15° to 42° for effective angle of shearing resistance (ϕ'). For old refuse c' values range from 16kN/m² to 19kN/m² with a ϕ' ranging from 33° to 42°.

6.2.5 Bulk densities of landfill material generally range from 3kN/m³ to 15kN/m³ dependent on the degree of compaction, age of the landfill and composition (Fassett et al, 1994). Based on observation of the nature of the landfill materials being placed in Cells 1 and 2 of Area 2 there is a significant percentage of soil within the waste and in consideration of this the bulk density of the landfill mass for this site may tend to be towards the higher end of the range indicated above.

6.2.6 In summary, for design, the following parameters have been used for the Made Ground materials. It is considered appropriate and to simplify the stability assessment to only use one set of parameters for non-hazardous waste materials? and relatively conservative values of drained and un-drained shear strength values have therefore been adopted based on the above.

Stratum	Bulk Density (kN/m³)	Undrained Shear Strength		Drained Shear Strength	
		c_u (kN/m²)	ϕ (°)	c' (kN/m²)	ϕ' (°)
Non-hazardous waste materials	12	5	25	5	25
Made Ground	20	4	32	4	32

Table 6 - 1: Design Parameters for Made Ground

Note: The bulk density is higher than the value of 10kN/m^3 used in the previous stability assessment, as based on observation the waste arriving at the site has a high percentage of soil and is also being well compacted and based on guidance in (Fassett et al 1994) a value of 12kN/m^3 is considered appropriate. In addition the review of piezometer data as presented in section 9 also identifies that the density of waste is likely to be greater than used in previous assessments.

6.3 Alluvium

6.3.1 Laboratory testing carried out on the Alluvium shows a large variation in moisture content from 10% to 71% with Plastic Limits and Liquid Limits ranging from 25% and 50% and 40% and 106%, respectively. These results indicate that the Alluvium is generally an inorganic clay or silt of high to very high plasticity.

6.3.2 The upper 1m to 2m where the desiccated crust can be found shows an undrained shear strength ranging from 15kN/m² to 75kN/m², below which the undrained shear strength reduces to 7kN/m² to 45kN/m². Effective shear strength parameters for the Alluvium have been recorded as 16.5° to 31.5° for angles of shearing resistance (ϕ') with an effective cohesion (c') of zero to 4.5kN/m². A plot of all the available results of effective stress testing for the Alluvium is presented as **Figure 4**.

6.3.3 Based on laboratory testing of the alluvial soils the dry densities vary from 0.95Mg/m³ to 1.56Mg/m³ and the bulk densities from 1.50Mg/m³ to 2.0Mg/m³. Comparing all the laboratory test results for the densities of the Alluvium and based on the mean value of bulk density a value of 17.5kN/m³ is used in the stability assessment.

6.3.4 In consideration of the generally soft nature of the Alluvium it has p[previously been agreed with Natural Resources Wales (formerly the Environment Agency) that an approach applying strength gain to the Alluvium can be used and this is included within the current Permit as condition 1.6.1.6.

6.3.5 The Permit condition states the following:

"A comprehensive assessment on strength gain due to clay consolidation has been made for the entire installation, submitted to the EA and the EA has agreed it in writing. The assessment shall consider drainage path lengths in the underlying strata and shall conclude with a table of maximum filling height during each operational year including a full height/time graph."

6.3.6 The following presents the approach adopted for the assessment of strength gain.

6.3.7 This assessment of strength gain considers the strength gain of the soft alluvial soils under consolidation and uses a critical state soil mechanics approach. In order to determine the change in effective stress conditions due to loading, the first step is to determine excess pore water pressure conditions for different stages in loading. This was undertaken in the Stability

Assessment Report (PBA, 2006). The appendix from (PBA, 2006) has been reproduced as **Appendix 1** in this report. This assessment of excess pore water pressures has then been used in the assessment of effective stress for the various load stages considered in the stability assessment. The strength gain has been reviewed to take account of actual excess pore water pressures as monitored in vibrating wire piezometers and this review is presented in later sections of this report.

6.3.8 In order to determine the strength gain, the approach used is based on determining a critical state line for the alluvial soils. This is based on guidance given in 'An Introduction to the Mechanics of Soils and Foundations' (Atkinson, 1993).

6.3.9 The critical state line can be determined from the results of effective stress triaxial testing by plotting a graph of the functions deviator stress p' versus yield stress q'.

Where $p' = \dfrac{(\sigma_1' + 2\sigma_3')}{3}$

and $q' = (\sigma_1' - \sigma_3')$

The slope of this graph is denoted as "M".

6.3.10 This has been done for all available triaxial stress testing results for the alluvial soils and is presented in **Appendix 1**. This gives a conservative value of M = 1.

6.3.11 At any point during the consolidation process, as the excess pore water pressure dissipates the effective value of p' will be reduced by an amount equivalent to the excess pore water pressure (u). Based on this, it is possible to determine a point on the critical state line for various values of p' and u and from this, the effective value of q' can be determined. q' represents the shear strength which is equivalent to a value of un-drained cohesion (c_u), for the particular values of p' and u. This is presented in **Appendix 1**.

6.3.12 Considering a point 0.5m below the underside of the stabilised alluvium, values of c_u for the differing heights of landfilling have been determined. A plot of c_u versus height of landfill is presented in **Appendix 1**. The lower bound value of c_u is limited to 10kN/m². This is considered a conservative value based on all the tests results.

6.3.13 The values gained from the assessment on strength gain have been used in the reappraisal of stability. This is a conservative approach as the higher bound pore water pressure levels that have been used are greater than the monitored actual pore water pressure measurements from the vibrating wire piezometers.

6.3.14 In respect of the strength gain that has already occurred beneath the northern and southern dams, a similar approach has been used. However, as these have been in place for several years, it is assumed that excess pore water pressures have dissipated to 70% and based on this, an undrained shear strength, cu, of 73kN/m² has been adopted for clay beneath the dams. This is presented in **Appendix 1**. The 70% degree of consolidation is based on the age of the dams being at least 5 years and with values of coefficient of vertical consolidation (c_v) = 0.5m²/year then from:

$Tv = (c_v \times t)/d^2$

Where d = H/2 = 5/2 = 2.5

Then $Tv = (0.5 \times 5)/2.5^2 = 0.4$

Therefore % consolidation U from Curves Based on Terzaghi theory of one dimensional consolidation = 70% (R Craig, 1987)

6.4 River Gravels

6.4.1 Ground investigations undertaken in the River Gravels recorded Standard Penetration Test (SPT) "N" values ranging between 6 to >50 (refusal) indicating the deposit to be loose to very dense. The majority of results however indicate the relative density to be medium dense to dense with a general trend of increasing density with depth. The high number of SPT refusals and many of the higher "N" values recorded may reflect the presence of cobbles and boulders.

6.4.2 The range of SPT results gives a range for angle of shearing resistance (ϕ') of between 29° and 41°. Based the results a value of 35° has been adopted as a conservative value for design.

6.5 Mercia Mudstone

6.5.1 Mercia Mudstone is present beneath the River Gravels and the shear strength parameters used in the assessment are as indicated below in section 6.9 are based on typical published values for these materials (CIRIA 2001).

6.6 Clay Liner

6.6.1 The proposed clay liner will be formed from low permeability clays. Within the two cells previously constructed this was constructed from material derived from two sources. One source was from similar clay to that of the alluvial deposits from a location close to the site. The second source was from clays from a source comprising Lias Clays. It is the intention that for the proposed Cell 3 that the source of clay for the liner will be the Lias Clay materials. However for acceptability and suitability criteria the material will need to be at a moisture content approximately equal to the plastic limit. Based on data from the placement and compaction of this source of materials in the construction of Cells 1 and 2 the following parameters are derived to be used in this assessment. Based on a comparison of undrained shear strength (c_u) against moisture content and results of plastic limit tests an undrained shear strength for the clay liner a conservative value of 40kN/m² would be appropriate for design with a drained angle of friction of 22° drained cohesion of 0kN/m² and a bulk density of 18kN/m².

6.6.2 It will be necessary to approve the suitability of the materials to be used for the clay liner and as part of this process some engineering testing will need to be undertaken to demonstrate that appropriate acceptance criteria are met prior to and during construction.

6.6.3 Similar materials will be placed in the former meanders to make up levels within Cell 3 Area.

6.7 Cement Stabilised Soil

6.7.1 A specialist contractor has reviewed the available ground investigation data and laboratory testing and has also conducted his own testing in order to design appropriate stabilisation works for the soft alluvial formation soils. The proposed system of stabilisation will comprise mixing of the soft alluvial soils with the addition of cement and bentonite to form a mass stabilised layer no greater than 2m thick. A summary of the specialist contractor's method is presented in **Appendix 2**. Based on the specialist contractor's assessment of the conditions and the proposed treatment of the very soft alluvial and the specified minimum undrained shear strength of 50kN/m^2 for the stabilised clay together with actual measured shear strengths using a hand shear vane on section where stabilisation trials have taken place which are all greater than 150kN/m^2 a value of undrained cohesion of 65kN/m^2 is used. For the long term drained situation a value of the drained angle of friction of 22° and a drained cohesion of 0kN/m^2 is used as this is the same as the non-stabilised Alluvium. This is considered conservative as it would be expected that the stabilising works would improve the long term shear strength of the Alluvium.

6.7.2 Based on all the available information the following table summarises the geotechnical parameters adopted in the stability analysis.

Stratum	Bulk Density (kN/m³)	Undrained Shear Strength		Drained Shear Strength	
		c_u (kN/m²)	ϕ (°)	c' (kN/m²)	ϕ' (°)
Non-hazardous waste materials	12	5	25	5	25
Clay Liner**	18	40	-	-	22
Cement and bentonite Stabilised Alluvium***	20	65	-	0	22
Made Ground	20	4	32	4	32
Alluvium	17.5	*	-	0	22
River Gravels	19	-	-	0	35
Mercia Mudstone	20	75	0	5	27

Table 6 - 2: Geotechnical Parameters

* *The un-drained shear strength is a factor of the strength gain as presented in Appendix 1*

** *The density and un-drained shear strength parameters for the clay liner are based on the minimum as specified in the liner construction specification and the drained angle of friction is taken as equal to that used for Alluvium this is based on the Plasticity Index of the Clays used in the Liner Construction which typically range between 30% and 40% and based on guidance in (BS8002:1994) the drained angle of friction for such clays would be in the range 20° to 25°*

6.7.3 The parameters within the above table are all considered to represent conservative values for the assessment of stability.

7 Revised Stability Assessment

7.1 General

7.1.1 This section of this report provides an outline of the areas where a revised Stability Assessment has been undertaken including the reasons for the revised assessment. Within this revised Stability Assessment the geotechnical design parameters used are as discussed in the preceding parts of this report. The assessment presented in the following sections does however include a review of excess pore water pressures based on piezometric data obtained over a period of time between October 2010 and February 2015 and this has been used in the assessment of short term stability with particular reference to rate of filling and need for rest periods.

- The Stability Assessment presented below includes the following:
- Landfill capping stability for 1v:4h side slopes.
- Review of excess pore water pressure based on piezometric data in order to assess strength gain and excess pore water pressure in the Alluvium for differing stages of landfilling.
- Northern dam stability.
- Southern dam stability.
- Cell 3 new bund stability.
- Assessment of the potential for the soft alluvial soils to extrude from beneath the landfill and result in instability.
- Asbestos sub-cell stability assessment.

7.1.2 The cross sections for the northern and southern dams remain the same as the original assessment within the Addendum Report (PBA, 2006) with the exception of including for a 1v:4h landfill slope. Reference should therefore be made to the previous assessments (PBA 2005 and PBA 2006)for discussions on each sections however justification of parameters and general assumptions are discussed in earlier parts of this report.

7.1.3 The Stability Assessment presented in Appendices 3 to 9 and which includes the assessments listed above in 7.1.2 are summarised in sections 8 to 14 of this report.

7.1.4 Within this report the assessment of stability has been undertaken in accordance with BS EN 1997-1 2004 Eurocode 7 Geotechnical Design.

8 Landfill Capping Stability

8.1 General

8.1.1 It is understood that the intention is to increase the gradient of the landfill side slopes, therefore an assessment of the capping stability on 1v:4h side slopes has been assessed. This capping stability assessment is based on use of a LLDPE artificial capping. The stability has been assessed using the method proposed by Jones and Dixon (EA, 2003b). The analysis has been modified to apply partial factors in accordance with BS EN 1997-1 2004 Eurocode 7 Geotechnical Design.

8.1.2 The capping construction is assumed to consist of a 0.6m thick restoration soil layer over a 0.2m thick protection soil layer. Below the soil cover the artificial capping will comprise a 1mm thick LLDPE textured liner sandwiched between two drainage composite layers (Pozidrain G6SD). The artificial capping will be placed over a regulation soil layer on the landfill.

8.1.3 The capping stability assessment is presented in **Appendix 3** as calculation sheets Cap 0 to Cap 11. The design parameters for the textured LLDPE and drainage capabilities are based on data provided by ABG Ltd though some parameters are derived from other published information. Relevant information provided by ABG Ltd is contained within **Appendix 3.**

8.1.4 The calculations have considered Parallel Submergence Ratio (PSR) of 0 0.25 and 0.5. This is the ratio of the thickness of submerged cover soil to the total thickness of cover soil ie where the PSR is 0.5 then the cover soil is fully saturated .From this it is found that as PSR increases from 0 to 0.5 the Over Design Factor (ODF) for sliding of cover soil reduces from 2.28 to 1.7. The ODF is effectively a factor of safety taking into consideration partial material and load factors and therefore for safe design the ODF should be greater than 1.so therefore the capping is safe in terms of sliding.

8.1.5 An assessment of the capping liner integrity indicates that theoretically, tensions in the geosynthetics are negative and that the shear strength along the lower interface is greater than the interface above. In this respect shear strength mobilised in the capping layers will be transferred into the regulation layer with no effective tension in the capping. This is the situation for all three cases of PSR considered. Therefore there is an adequate factor of safety against rupture of the geosynthetics analysed.

8.1.6 An assessment of strains within the LLDPE capping has been undertaken based on an approach given by Jones and Dixon (EA, 2003a). This indicates that for an LLDPE with a tensile strength of 11kN/m, the maximum strain in the LLDPE would be 6.2%. The assessment of stain is based on a void in the landfill and does not take into consideration any tensile strength in other layers of the capping makeup and is therefore conservative. In addition LLDPE liner materials can tolerate very large strains without failure and therefore an LLDPE capping is safe in terms of strain.

8.1.7 An assessment of the possible effects of landfill gas pressure on the stability of the capping system has been undertaken based on Thiel (Thiel, 1999). This indicates that for a factor of safety of at least 1.2 a maximum gas pressure of 6.5kN/m^2 on the geosynthetic capping should not be exceeded. The active gas extraction system at the site will need to be managed to ensure this pressure is not exceeded.

9 Review of Excess Pore Water Pressure

9.1 General

9.1.1 Vibrating wire piezometers were installed in the alluvial clay beneath the landfill Cell 2 in October 2010. The data from these instruments has been reviewed in order to provide a more robust approach to the predicted excess pore water pressure against landfill thickness as used in the assessment of the landfill slope stability.

9.1.2 In general the review presented within this report is largely based on Peter Brett Associates Report: Docksway Disposal Site Area 2 Cell 2 Vibrating Wire Piezometer Data Review Report Ref. 14739/139 dated November 2011 (PBA, 2011). This is included as **Appendix 4**. Based on this together with subsequent vibrating wire piezometer results, the level of piezometric head at each piezometer position (mOD) has been calculated from the results of the vibrating wire piezometer data. In addition the approximate levels of the landfill have also been estimated corresponding to the piezometric data and a graph plotted to show landfill thickness against height of pore water as measured from the base of the landfill. This has then been compared to the original estimate of the level of pore water pressure above the base of the landfill as used in the stability assessment presented in the Area 2 Stability Assessment Addendum Report.

9.1.3 The above assessment including graphs is presented in **Appendix 5.** At the time of the assessment within this report the survey data available indicated that there is a significant degree of scatter in the piezometric head. However, it does generally indicate that the actual piezometric head is above what was originally predicted for the early stages of filling but that as landfilling has progressed the actual piezometric head appears to fall below the predicted levels. There may however be some lag in respect between the time that piezometric readings were taken and the time at which the landfill reached the levels as recorded on available survey data. Based on the piezometric data a conservative fit for the design line for the piezometric head has been assessed that is 2.5m above the original design line. This new design line is used within the analysis of stability within this report.

9.1.4 The variance between the original assessed excess pore water pressure and the current evaluated design line is most likely due to the density of the waste being greater than the 10kN/m³ originally assessed. It is apparent from observation of the type of waste at the site, that a significant amount of soil is included within the current non-hazardous waste received at the site. In consideration of this and based on typical values (Fassett et al 1994& Zekkes 2006) it is considered appropriate to adopt a slightly higher value of density for the waste, particularly as the density of waste within the landfill mass is likely to increase with depth of burial. Typical values for waste density can vary between 10kN/m³ at the surface where normal compaction is applied to in excess of 15kN/m³ at depth. In consideration of this and in particular the amount of soil observed within the waste as a value of 12kN/m³ is considered appropriate for the bulk density of waste and has been applied within the assessment presented here.

10 Northern Dam Stability

10.1 General

10.1.1 The Northern Dam is the northern most dam on the former River Ebbw channel on the western boundary of Area 2 as indicated on Figure 2. The stability of this has been reviewed within this report as it is now proposed to increase the gradient of the landfill to 1v:4h. The ground model for this remains as per the original assessment with the exception of the 1v:4h landfill slope and increased height of landfill to an elevation of 40mOD.

10.1.2 This revised stability assessment is presented in **Appendix 6** and summarised below.

10.1.3 In accordance with Eurocode 7, Design Approach 1 has been used for the stability assessment and based on experience it is found that for this type of stability assessment particularly where there are no significant unfavourable permanent or variable actions that Combination 2 gives the most conservative assessment and therefore only Combination 2 has been applied.

10.1.4 The stability assessment for temporary stability as presented in the Addendum Report (PBA, 2006), has not been revised as the landfilling has been commenced in these areas and the temporary situation no longer applies. All that has been considered is the short term stability for the completed landfill including for a porewater pressure 2.5m above the original level used and the long term stability. For each case circular and wedge stability analysis has been undertaken.

10.1.5 The results of the analysis are summarised below:

Analysis	Short Term Max Utilisation Factor	Long Term Max Utilisation Factor
Large Circular Through Landfill	0.89	0.72
Small Circular Through Toe of Dam	0.95	0.98
Wedge	0.87	0.67

Table 10- 1: Results of Northern Dam Stability Analysis

Note: Utilisation Factor is the reciprocal of the Over Design Factor (ODF) where the ODF is a factor of safety taking into account partial material and load factors and provided the utilisation factor is below 1, the slope is deemed to be stable.

10.1.6 It should also be noted that this is a conservative assessment as in reality there is a hold period during the landfilling operations to allow pore water pressures to dissipate and therefore the actual pressures used in this assessment are most probably higher than will be present in the alluvial soils beneath the landfill in the short term.

11 Southern Dam Stability

11.1 General

11.1.1 The Southern Dam is the southernmost dam on the former River Ebbw channel on the western boundary of Area 2 as indicated on Figure 2. The stability of this has been reviewed within this Report as it is now proposed to increase the gradient of the landfill to 1v:4h. The ground model for this remains as per the original assessment with the exception of the 1v:4h landfill slope and increased height of landfill to an elevation of 40mOD.

11.1.2 This revised stability assessment is presented in **Appendix 6** and summarised below.

11.1.3 In accordance with Eurocode 7, Design Approach 1 has been used for the stability assessment and based on experience it is found that for this type of stability assessment particularly where there are no significant unfavourable permanent or variable actions that combination 2 gives the most conservative assessment and therefore only combination 2 has been applied.

11.1.4 The stability assessment for temporary stability as presented in the Addendum Report (PBA, 2006), has not been revised as the landfilling has been commenced in these areas and the temporary situation no longer applies and has not been revised. All that has been considered is the short term stability for the completed landfill including for a pore water pressure 2.5m above the original level used and the long term stability. For each case circular and wedge stability analysis has been undertaken.

11.1.5 The results of the analysis are summarised below:

Analysis	Short Term Max Utilisation Factor	Long Term Max Utilisation Factor
Large Circular Through Landfill	0.87	0.82
Small Circular Through Toe of Dam	0.99	0.95
Wedge	0.46	0.60

Table 11 - 1: Results of Southern Dam Stability Analysis

Note: Utilisation Factor is the reciprocal of the Over Design Factor (ODF) where the ODF is a factor of safety taking into account partial material and load factors and provided the utilisation factor is below 1, the slope is deemed to be stable.

11.1.6 It should also be noted that this is a conservative assessment as in reality there is a hold period during the landfilling operations to allow porewater pressures to dissipate and therefore the actual pressures used in this assessment are most probably greater than will be present in the alluvial soils beneath the landfill in the short term.

12 Cell 3 and 4 End Bund Stability

12.1 General

12.1.1 The revised layout of landfill necessitates construction of 2 bunds across the former channel of the River Ebbw at the north eastern end of Area 2. The locations of these are indicated on Figure 2.

12.1.2 Both of these bunds are going to have a very similar profile, so one typical profile has been considered within this assessment. This profile is presented in Figure 5. The gradient of the outer edge of the dam profile is considered within the analysis and the analysis includes the following cases:

- Short term stability of bund itself with no landfill;
- Long term stability of completed landfill with 1v:3h outer bund slope;
- Long term stability of completed landfill with 1v:6h outer bund slope;
- Long term stability of completed landfill with 1v:6h outer bund slope formed with granular fill;
- Short term stability for differing heights of landfill considering strength gain and hold periods.

12.1.3 The analysis of the new bunds is presented in **Appendix 7** and summarised below.

12.2 Short Term Stability

12.2.1 This assessment is for the stability of the bund immediately following construction and is based on a bund profile constructed with the same clay as the main clay liner and that it will be integrated with the liner. The geotechnical parameters used in this are as used in all other analysis of the un-drained situation and make no allowance for any strength gain due to the likely short duration of bund construction. The analysis does however include for a raised phreatic surface to model potential excess pore water pressures that are likely to build up during construction of the bund. The bund profile used includes for 1v:3h slopes on both sides with a minimum 6m wide crest. In addition the thickness of the stabilised Alluvium is assumed as 1m thick; in reality the thickness of stabilisation will be up to a maximum of 2m thick and therefore by using only 1m thickness this is a conservative approach. It has been assumed in the analysis that the stabilised clay will extend a minimum of 3m beyond the outer toe of the bunds.

12.2.2 The short term stability assessment for the bund alone indicates that for a circular failure affecting the whole bund, the maximum utilisation factor is 0.88 and for a wedge failure is 0.58.

12.3 Long Term Stability

12.3.1 This assessment includes for analysis of the long term stability of the completed landfill above the bund including for landfill side slopes of 1v:4h and a completed landfill level of 40mOD.

12.3.2 This analysis indicates that the landfill slope is stable; however the outer face of the bund would potentially be unstable with potential circular failure surfaces with utilisation factors up to about 1.3. A wedge stability check has therefore not been undertaken.

12.3.3 To mitigate the risk of failure of the outside face of the bund, an assessment has been undertaken for a bund with an outer slope of 1:6. This analysis indicates a maximum utilisation factor of 0.9 for circular failure and a maximum utilisation factor of 0.63 for wedge failure.

12.3.4 In addition to a bund formed entirely of clay an analysis has been undertaken for a composite bund construction where the outside shoulder is constructed in a granular fill formed at 1:6 slope. This analysis indicates a maximum utilisation factor of 0.88 for circular failure and a maximum utilisation factor of 0.73 for wedge failure.

12.4 Short Term Stability during Landfilling

12.4.1 This analysis includes for assessment of the short term stability of the bund and landfill slope as landfilling is progressed. The analysis includes for strength gain as in previous assessment presented in the Stability Assessment Addendum Report (PBA, 2006). In addition an allowance is made for strength gain immediately beneath the bund. The excess pore water pressures used in this assessment is as discussed earlier in this report and includes for a 2.5m increase, above what was previously analysed within the Stability Assessment Addendum Report (PBA, 2006).

12.4.2 The short term filling assessment includes for assessment of the landfill when it is at elevations of 13.5mOD, 18.5mOD, 23.5mOD and 28.5mOD. In addition within this assessment the ground model includes for the soft Alluvium to be a consolidation layer, Within the GGU stability software used for the analysis, a consolidation layer is effectively a layer of strata in which it is possible to set an excess pore water pressure. Incorporating a consolidation layer is appropriate in this situation to model the excess pore water pressure due to the landfilling above the soft Alluvium. This assessment of short term stability is presented on sheets New Bund 6 to New Bund 9 of **Appendix 7.**

12.4.3 It is identified in the short term stability assessment that once the landfill reaches an elevation of 28.5mOD that the utilisation factor could approach 1 and would therefore be potentially unstable. In view of this consideration is given to having a hold period when the landfill reaches an elevation of 23.5mOD to allow the excess pore water to dissipate and some strength gain to occur. The assessment of this is presented on sheets New Bund 11 to New Bund 14 of **Appendix 7** and this assessment considers a hold period of 12 months allowing for both a dissipation of pore water pressures and corresponding gain in un-drained shear strength. This assessment indicates that at 28.5mOD the utilisation factor would be 0.91, at 33.5mOD the utilisation factor would be 0.93 and at 40mOD would be 1.04. Though the assessment presented in **Appendix 7** is based on a time period it will be the pore water pressure that is the factor that will determine when landfilling can recommence after a hold period and based on this assessment it would be appropriate for any hold period at the point when the landfill reaches a level of 23.5mAOD to continue until the pore water pressure within the Alluvium reduces to a pressure equivalent to a piezometric level below 10.88mOD.

12.4.4 In consideration of the potential for utilisation factors to be greater than unity when the landfill is at an elevation of 40mOD an assessment has been undertaken for a second hold period of 6 months when the landfill is at an elevation of 33.5m. This assessment is presented on sheets New Bund 15 and New Bund 16 in **Appendix 7**. This assessment indicates that after the 6 month hold period at 33.5mOD elevation the utilisation factor at 33.5mOD would be 0.96 and at an elevation of 40mOD the utilisation factor would still remain just above 1. As discussed above it will be the pore water pressure that is the critical factor in determining the point when landfilling above the elevation of 33.5 could recommence and for this the pore water pressure would need to reduce to an equivalent elevation of below 12.6mOD. It is also apparent from the analysis of the landfill above a level of 28.5mOD that if the un-drained shear strength of the Mercia Mudstone is constant with depth that the analysis indicates potential slip surfaces with high utilisation factors approaching unity. This is however unlikely as the un-drained shear strength of the Mercia Mudstone will increase with depth as indicated by SPT N values. A plot of SPT N Value –v- Elevation is included in **Appendix 7** and based on this an un-drained cohesion of 100kN/m2 is used for the Mercia Mudstone below an elevation of -20mOD. This approach then mitigates the unrealistic potential deep seated slip surfaces.

12.4.5 Based on both this short term assessment of stability and the previous assessment of long term stability at the sections where bunds are to be constructed there may be some potential for the landfill to have limiting stability if raised to an elevation of 40mOD at these locations. In reality at the locations of the new bunds in the former river channel the maximum elevation of the landfill for section lines drawn perpendicular to the bunds will only get to a maximum of just over 30mOD as indicated on Figure 2.

12.4.6 The stability assessment presented in **Appendix 7** utilising GGU Stability software generally has been undertaken using circular analysis. However wedge analysis checks have been undertaken and where the wedge analysis indicates a worst case these are presented in **Appendix 7**.

13 Extrusion Assessment

13.1 General

13.1.1 Given that the new bund sections along the edge of Cells 3 and 4 will be formed on top of very soft alluvial soils there is a risk that extrusion could occur. Extrusion is a situation where particularly soft soils are present beneath earthworks such that the loading on these soft soils from the earthworks would induce outward stresses that could result in the soft soils spreading laterally or extruding. In this respect an assessment of the potential for extrusion to occur has been undertaken using guidance in (BS 8006: 2010). For this assessment it is identified that if the maximum elevation of the landfill is limited to 35mOD based on the fact that at the locations of the new bunds for sections drawn perpendicular to the bunds the maximum elevation of the landfill will be a maximum of just over 30m as indicated on Figure 2 extrusion should not be an issue. Within the extrusion assessment it is assumed that the thickness of stabilised soil is 2m. The assessment does not, however, allow for any strength gain and is therefore considered to be a conservative analysis. Also in the assessment the sensitivity to the thickness of Soft Alluvium is considered to determine the maximum thickness of very soft Alluvium that could be below the landfill without the risk of extrusion.

13.2 Extrusion Assessment

13.2.1 The extrusion assessment is presented in **Appendix 8.** This is inrelation to a section through the proposed landfill at the positions of the bunds in the former river channels. This assessment indicates that there is no risk of extrusion provided the following criteria is met:

- The final upper level of the landfill does not exceed an elevation of 35mOD.
- The thickness of the stabilisation should be at least 2m.
- Also in the assessment the sensitivity to the thickness of Soft Alluvium is considered and it is identified that the soft alluvium beneath the stabilised layer would need to be at least 5.7m thick for extrusion to occur and at the location of the bunds the thickness of soft Alluvium is likely to be much less than this as demonstrated by ground investigation information in this area.
- The extent of stabilisation extends at least 3m beyond the outer edge of the bund.

14 Asbestos Sub Cell Stability Assessment

14.1 General

14.1.1 The current proposals include for an asbestos sub cell to be formed within Cell 3 and this will require a liner to be formed between the asbestos waste and cover materials and the non-hazardous waste. The stability of the interface between the Asbestos Sub Cell and the remainder of Cell 3 needs to be considered and a recommendation made regarding an appropriate slope angle and maximum height for the interface and the liner between these two parts of the Cell. The following is a summary of the findings of this stability assessment as presented in **Appendix 9.**

14.1.2 The stability assessment presented in **Appendix 9** indicates three scenarios of filling that could potentially occur, and these are as follows:

- Scenario 1: The non-hazardous waste input is greater than the asbestos waste and therefore the asbestos waste will be formed overlying the non-hazardous waste at the interface between the two wastes.

- Scenario 2: The asbestos waste input is greater than the non-hazardous waste and therefore the asbestos waste will be below the non-hazardous waste at the interface between the two wastes.

- Scenario 3: Asbestos waste and non-hazardous waste inputs are at similar rates such that the interface between the two wastes will be near vertical.

14.1.3 Diagrams indicating the three scenarios are included in **Appendix 9.**

14.1.4 The asbestos waste is likely to have somewhat different geotechnical parameters to the non-hazardous waste. In particular the asbestos waste will not be compacted and as such will have a relatively low bulk density. However this is complicated by the requirement for the asbestos waste to have greater daily soil cover than non-hazardous waste and the inclusion of this daily cover will impact on the en-masse geotechnical parameters. In consideration of this it is considered sensible to adopt a low value of un-drained shear strength cu = 25kN/m2 based on a conservative value for the cover soils and ignoring any benefit from the fibrous nature of the asbestos materials. The intention is that the asbestos waste cell will be split into smaller sub-cells to keep the ratio of asbestos waste thickness to soil cover thickness to a maximum and this ration is likely to be 2.5:1. Based on (Department of Environment Industry Profile for Asbestos 1995) the density of asbestos materials can range between 7kN/m3 for Insulation board to 16kN/m3 for fully compressed sheeting. In consideration of this and the fact that the asbestos waste will not be compacted a conservative bulk density of 13.5kN/m3 is appropriate for the asbestos materials. For the cover soil a bulk density of 18kN/m2 is considered appropriate. Based on a ratio of thickness of 2.5:1 (asbestos: soil) this would give an en-masse bulk density of 14.8kN/m3 therefore a bulk density of 15kN/m2 has been used in the stability assessment presented in **Appendix 9**. Should operational procedures change and the filling of the asbestos cell differ from the assumptions made it will be necessary to review the stability assessment.

14.1.5 During the landfilling there is potential that surface water will penetrate both the non-hazardous waste and the asbestos waste and become perched within either waste mass. In consideration of this, within the analysis presented in **Appendix 9**, a phreatic ground water surface equivalent to one third of the height difference has been modelled for each case analysed. This is considered a very conservative approach as the levels of leachate in the landfill and water levels in the asbestos waste will be controlled to be well below this. In addition the conservative assessment of the bulk density of the asbestos waste when coupled with the assumed phreatic surface will give a very conservative assessment of stability.

14.1.6 Only the short term un-drained stability is considered in the analysis undertaken as it is unlikely that fully drained conditions will be realised during the filling of the cells.

14.2 Summary of Assessment

14.2.1 In the assessment presented in **Appendix 9** it is apparent that for both Scenarios 1 and 2 that for an interface steeper than 1v: 1.5h and at a height difference of 7.5m that the interface between the non-hazardous waste and asbestos waste could be unstable. It is therefore recommended that the difference in height between the two wastes for both Scenarios 1 and 2 should be no greater than 7.5m and that the slope on the interface between the asbestos cell and the general waste cell should be no steeper than 1v: 1.5h. This recommendation is also relevant for Scenario 3.

15 Essential Guidance for Report Readers

1 This report has been prepared within an agreed timeframe and to an agreed budget that will necessarily apply some constraints on its content and usage. The remarks below are presented to assist the reader in understanding the context of this report and any general limitations or constraints. If there are any specific limitations and constraints they are described in the report text.

2 The opinions and recommendations expressed in this report are based on statute, guidance, and best practice current at the time of its publication. Peter Brett Associates LLP (PBA) does not accept any liability whatsoever for the consequences of any future legislative changes or the release of subsequent guidance documentation, etc. Such changes may render some of the opinions and advice in this report inappropriate or incorrect and we will be pleased to advise if any report requires revision due to changing circumstances, especially those over one year old. Following delivery of any report PBA has no obligation to advise the Client or any other party of such changes or their repercussions.

3 Some of the conclusions in this report may be based on third party data. No guarantee can be given for the accuracy or completeness of any of the third party data used. Historical maps and aerial photographs provide a “snap shot” in time about conditions or activities at the site and cannot be relied upon as indicators of any events or activities that may have taken place at other times.

4 The conclusions and recommendations made in this report and the opinions expressed are based on the information reviewed and/or the ground conditions encountered in exploratory holes and the results of any field or laboratory testing undertaken. There may be ground conditions at the site that have not been disclosed by the information reviewed or by the investigative work undertaken. Such undisclosed conditions cannot be taken into account in any analysis and reporting.

5 Unless specifically stated to the contrary, this report does not purport to be a “Geotechnical Design Report” as defined in Clause 2.8 of Eurocode 7 (Geotechnical Design BS EN 1997-1:2004). Some of the data contained herein and used to support any geotechnical assessment presented in this report may be historical or for other reasons not fully compliant with the requirements of that code.

6 It should be noted that groundwater levels, groundwater chemistry, surface water levels, surface water chemistry, soil gas concentrations and soil gas flow rates can vary due to seasonal, climatic, tidal and man-made effects.

7 If the report indicates that asbestos has been identified within the ground, any work that involves, or is likely to involve, contact with asbestos must be undertaken in accordance with the Control of Asbestos Regulations 2012, particularly in regard to risk assessment, licencing and training. Risk assessment should be carried out prior to any activities that could lead to the disturbance of asbestos materials, either buried or on the ground surface and should include appropriate mitigation measures, such as damping down to prevent the spread of asbestos, air monitoring and minimum PPE and/or RPE requirements for the work proposed.

8 This report has been written for the sole use of the Client stated at the front of the report in relation to a specific development or scheme. The conclusions and recommendations presented herein are only relevant to the scheme or the phase of project under consideration. This report shall not be relied upon or transferred to any other party without the express written authorisation of PBA. Any such party relies upon the report at its own risk.

9 The interpretation carried out in this report is based on scientific and engineering appraisal carried out by suitably experienced and qualified technical consultants based on the scope of

our engagement. We have not taken into account the perceptions of, for example, banks, insurers, other funders, lay people, etc., unless the report has been prepared specifically for that purpose. Advice from other specialists may be required such as the legal, planning and architecture professions, whether specifically recommended in our report or not.

10 Public or legal consultations or enquiries, or consultation with any Regulatory Bodies (such as the Environment Agency, Natural England or Local Authority) have taken place only as part of this work where specifically stated.

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Figures

Site Grid Reference: ST 309 852

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FIGURE 1

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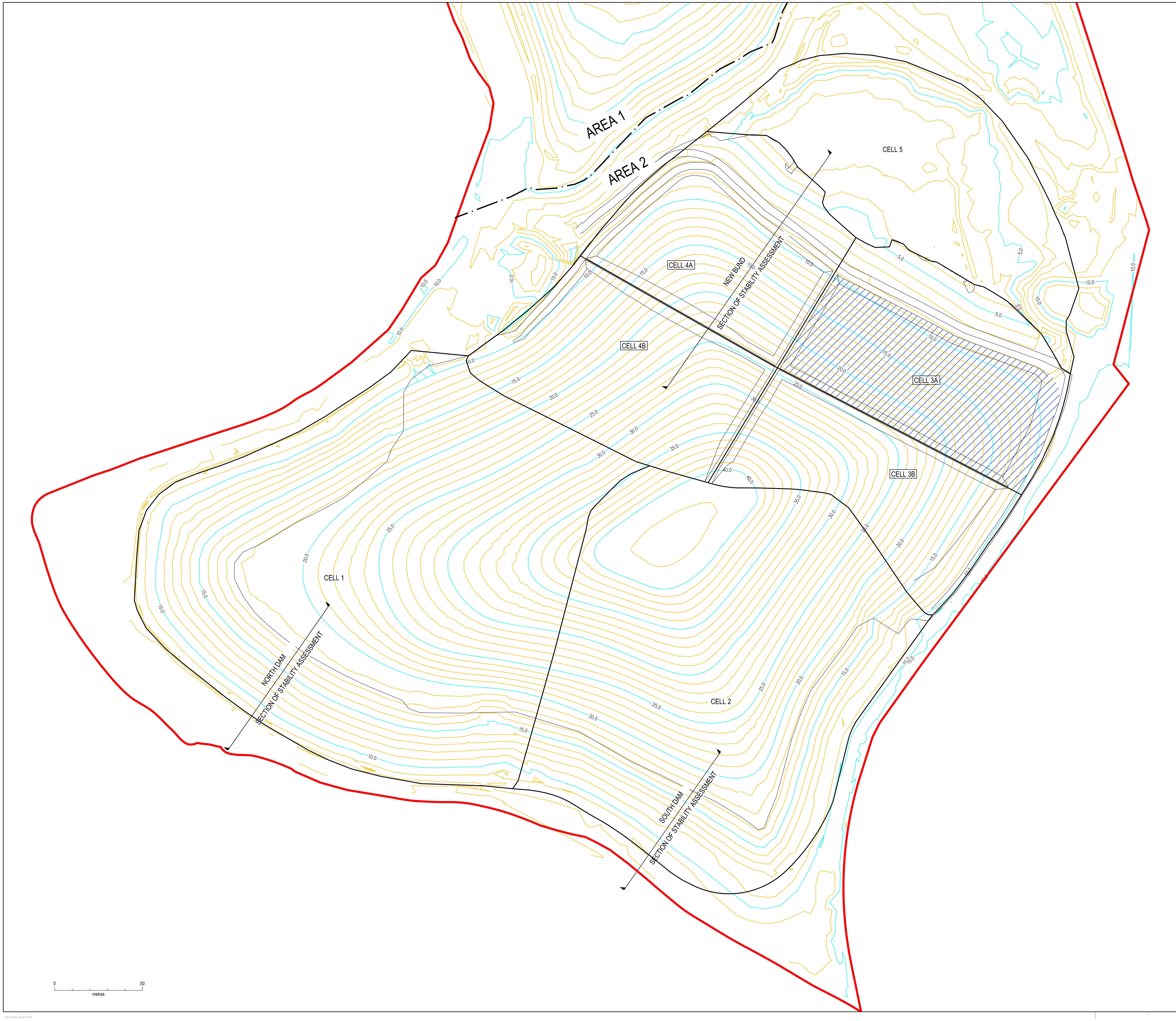

PERMIT AREA BOUNDARY

EXTENT OF ASBESTOS CELL

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DOCKSWAY LANDFILL AREA 2

LAYOUT

Client

NEWPORT CITY COUNCIL

Date of 1st Issue: 30.03.2015

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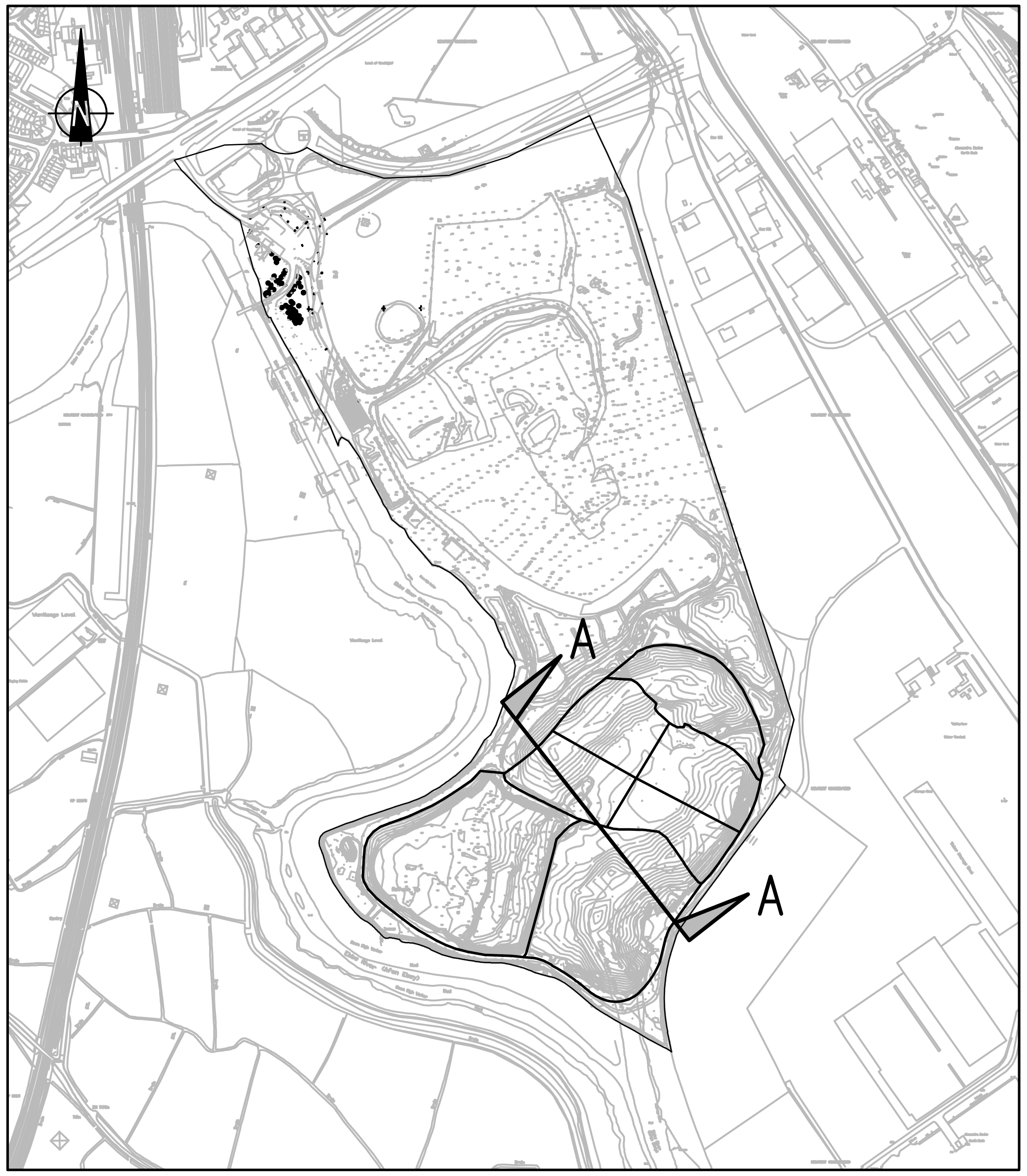
READING

Tel: 0118 950 0761

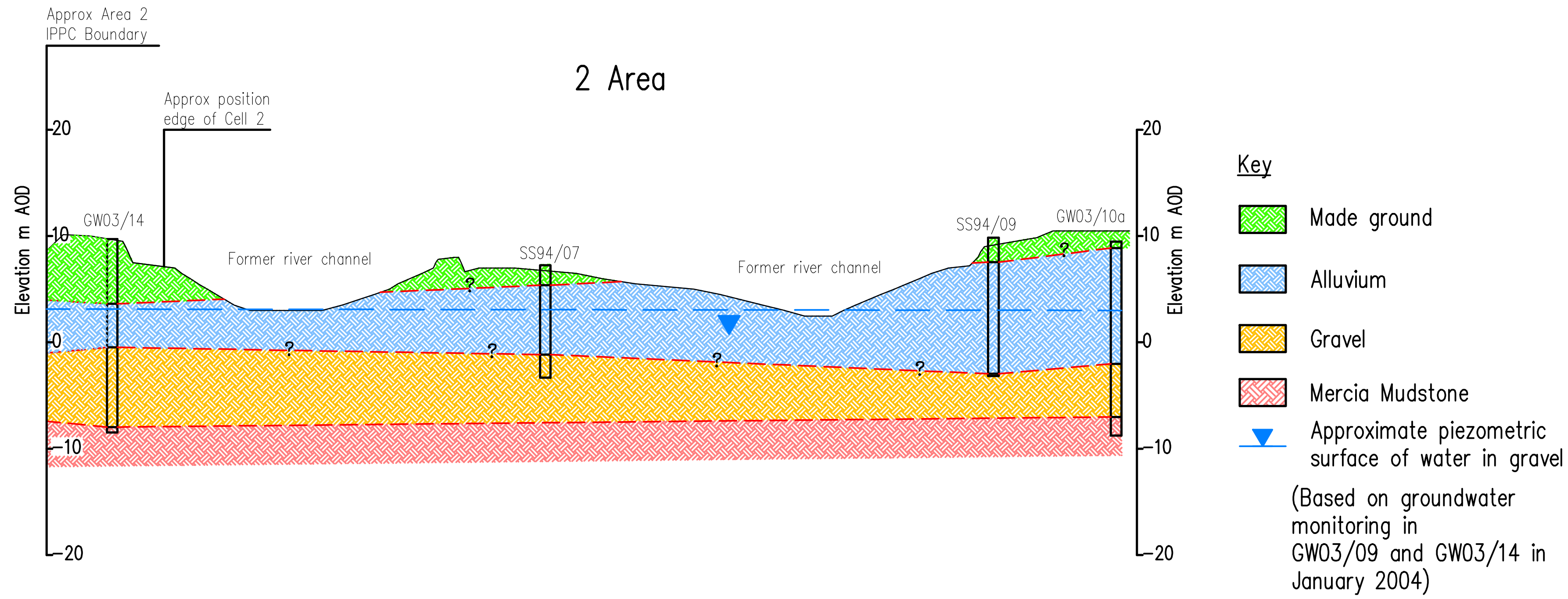
Notes

1. This drawing is to be read in conjunction with all other drawings.
2. Do not scale this drawing.
3. All levels are in metres above/below Ordnance Datum (AOD).



Location Plan

Scale 1:10,000



Section A–A

Scale H 1:2500, V 1:625

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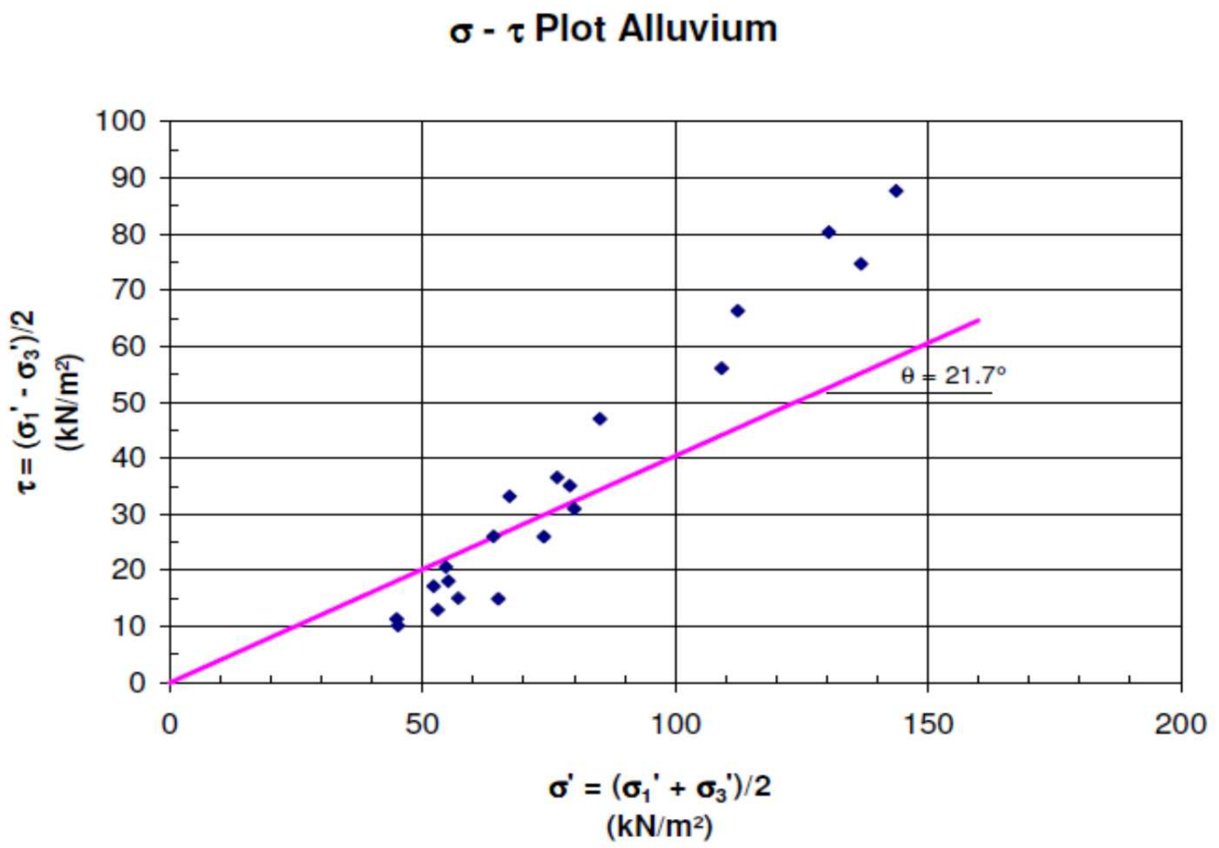
user name: david cotton

File Location: j:\14739\155 - permit revision 2015\03_ figures & drawings\cad\dwgs\l tomlin figures\figure 3.dwg



1. Based on the effective stress triaxial test results given in Exploration Associates Limited Factual Report 1996

2. σ_1' = major effective principal stress at failure
   σ_3' = minor effective principal stress at failure

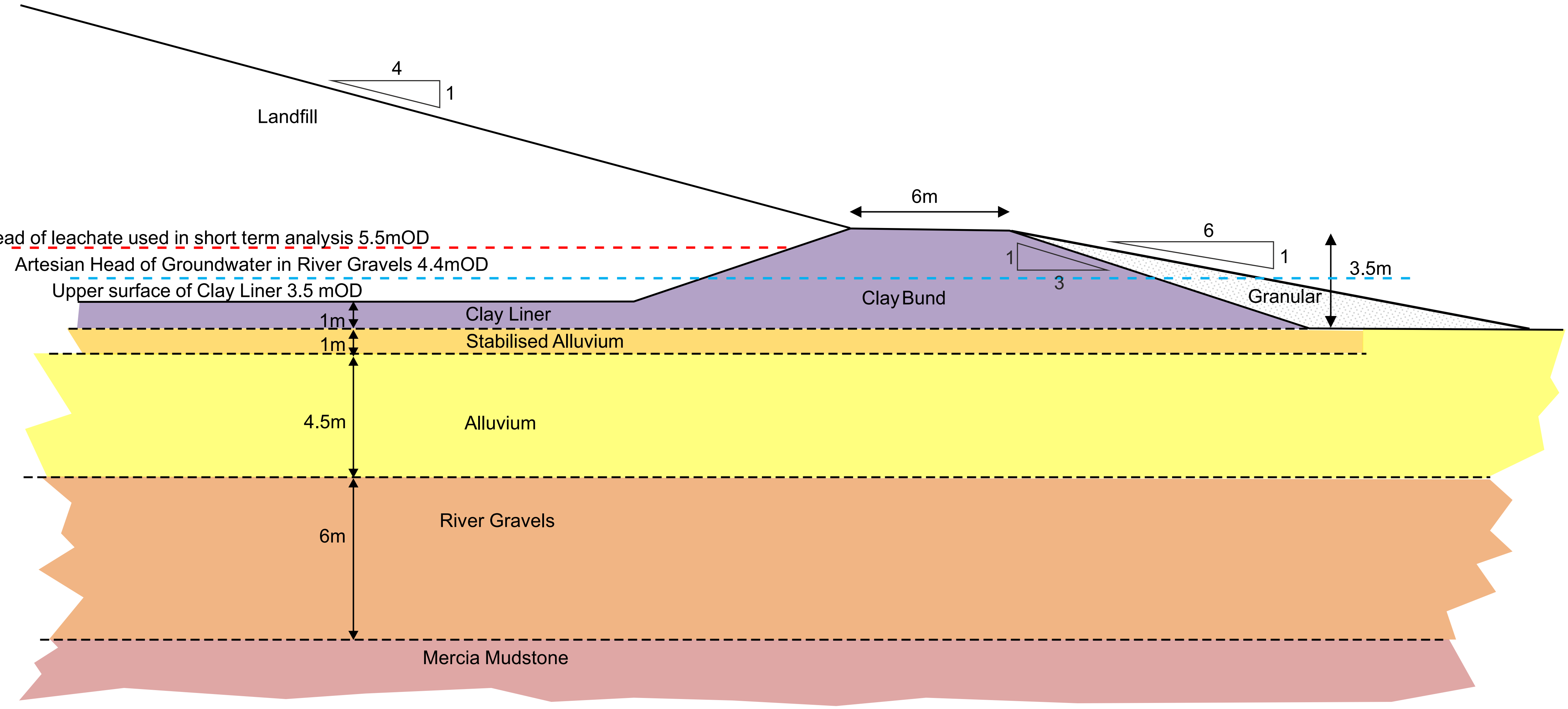
3. Worst case design line assuming c' = 0kN/m³, θ = 21.7°

   sin φ' = tan θ therefore φ' = 23.4°

Note: This has been reproduced from the Stability Assessment Report for Area 2 Landfill Extension Ref 14739/003/EXE/RHT/LT/AD/MDM Rev 21 Dated April 2005.

peterbrett	Client NEWPORT CITY COUNCIL	DOCKSWAY WASTE DISPOSAL SITE AREA 2 σ -τ PLOT ALLUVIUM	Date Sep-04 A4 Scale Drawn LJT Checked LJT Figure Number **4**

Lakeside House, Blackbrook Business Park, Taunton, TA1 2PX
Tel 0823 445150 Fax 01823 445151



Note: For Longterm analysis the maximum head of leachate in the landfill mass is taken as 25.0mOD

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Client

NEWPORT CITY COUNCIL

DOCKSWAY LANDFILL SITE AREA 2

SCHEMATIC SECTION THROUGH END BUND

Date	28.04.2015
A3 Scale	1:100
Drawn by	LJT/davco
Checked by	LT
Revision	0

FIGURE 5

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Appendix 1 Critical State Appraisal

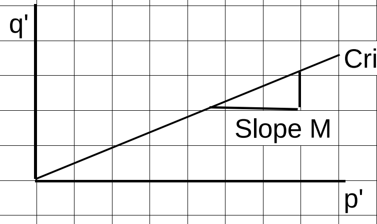
Calculations

Project Title	Docksway Disposal Site: Area 2											
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015		

The following sheets present the critical state assessment as undertaken for the Area 2 Stability Assessment Addendum Report Dated April 2006. This assessment remains unchanged since then and is presented here to indicate the theory applied to the assessment of strength gain. Within Appendix 7 the level of strength gain is revised to take account of the review of porewater pressures as measured in piezometers as presented in Appendix 5.

Consider the shear strength of the Alluvium by using the theory of critical state line soil mechanics to determine the strength gain of the Alluvium as load is applied.
This is based on guidance given in 'An Introduction to the Mechanics of Soils and Foundations' John Atkinson, 1993.

Using the following

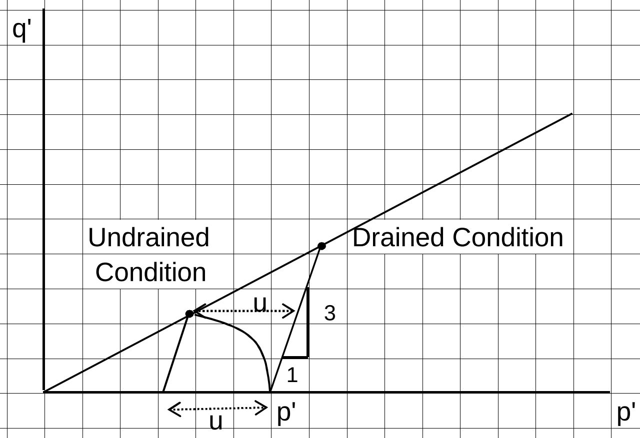


$$p' = \frac{(\sigma_1'+2\sigma_3')}{3}$$

$$q' = (\sigma_1'-\sigma_3')$$

using M = 1 See ADD2

For any values of σ_1' & σ_1' it is possible to determine value of p'
provided the porewater pressure (u) is knownfor any state of drainage
then the value of q' for any partially drained condition between undrained
and fully drained situation can be determined

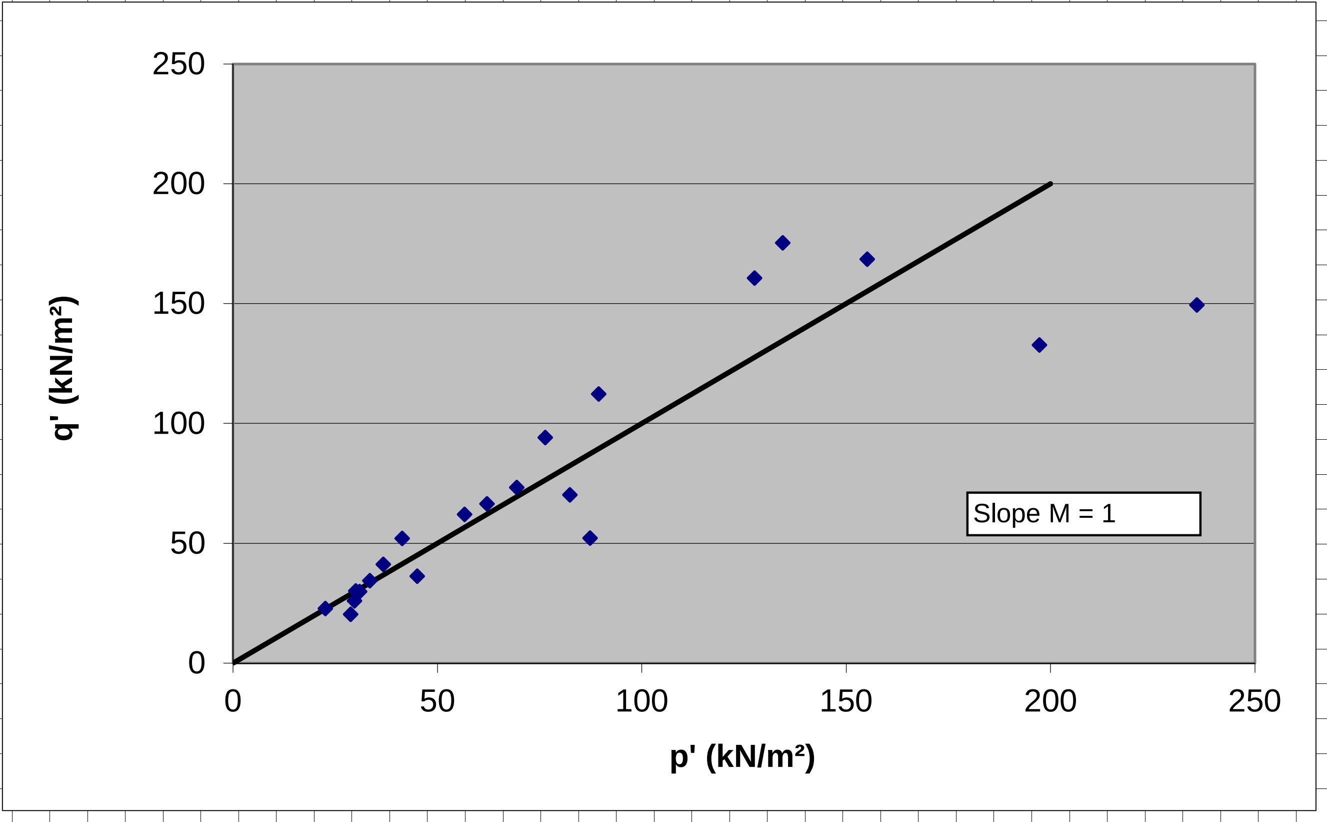


As shear strength $\tau = \frac{(\sigma_1'-\sigma_3')}{2}$ then $\tau = q'/2$

Therefore the undrained shear strength at any loading case can be determined

peter brett associates CONSULTING ENGINEERS	JOB TITLE: DOCKSWAY DISPOSAL SITE AREA 2 STABILITY ASSESSMENT ALLUVIUM CRITICAL STATE SHEAR STRENGTH	JOB No: 14739/061	SHEET: Appendix 1 ADD2
		DATE: 9/12/05 BY: LJT	CHECKED: AD

Plot of all available effective stress testing data as p' -v- q'
The design slope of M = 1 is considered a very conservative approach

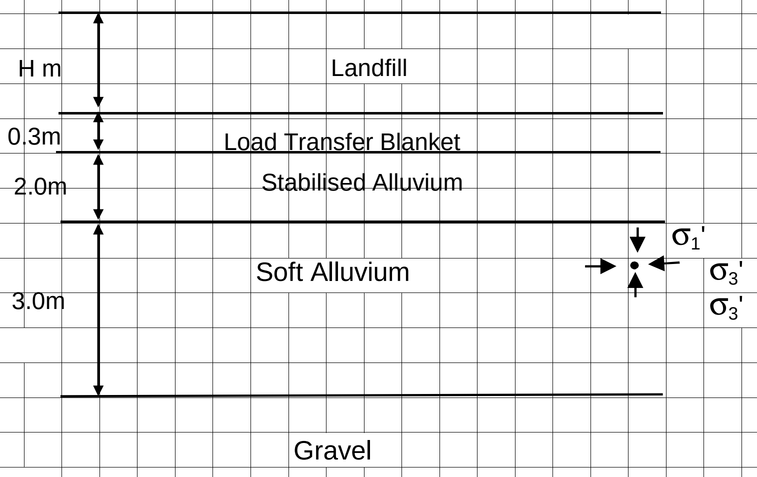


peter brett associates
CONSULTING ENGINEERS

JOB TITLE: DOCKSWAY DISPOSAL SITE
AREA 2 STABILITY ASSESSMENT
ALLUVIUM CRITICAL STATE SHEAR STRENGTH

JOB No:		SHEET:
14739/061		Appendix 1 ADD3
DATE: 9/12/05	BY: LJT	CHECKED: AD

Consider the load stages as used in the determination of porewater pressures for the following generalised section of the landfill site (See Also Sheets Pore 1 to Pore 3)



For a point 0.5m below the underside of the Stabilised Alluvium

σ_1 = $(H \times \gamma_{landfill}) + [(0.3 + 2 + 0.5) \times 20]$ - excess porewater pressure (u)
where the initial value of u = $H \times \gamma_w$ However this reduces with time as presented on Calculation Sheets Pore 1 to Pore 3

σ_3 = $\sigma_1' k_0$
Where k_0= $(1 - \sin\phi')$
= $(1 - \sin 22)$ = 0.625

For Alluvium ϕ'=22, see Table 5-3 in main SRA report

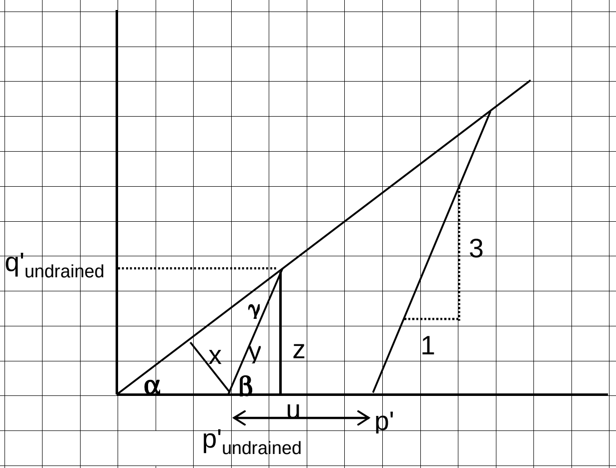
consider drained values of σ_1' i.e. u = 0

H	u*	σ_1'	σ_3'	p'	H	u*	σ_1'	σ_3'	p'
1.66	16.6	72.6	45.4	54.5	16.7	125.33	223	139	167
3.33	31.208	89.3	55.8	66.9	18.3	134.958	239	150	179
4.99	44.82	106	66.2	79.4	20	144.42	256	160	192
6.66	57.934	123	76.6	91.9	21.7	153.55	273	170	204
8.32	70.55	139	87	104	23.3	162.514	289	181	217
9.99	82.502	156	97.4	117	25	171.146	306	191	229
11.7	93.956	173	108	129	26.7	179.446	323	202	242
13.3	104.912	189	118	142	28.3	187.58	339	212	254
15	115.37	206	129	154	30	195.548	356	222	267

* This is the excess porewater pressure after filling is completed as determined from Graph on Pore 3

pba peter brett associates CONSULTING ENGINEERS	JOB TITLE: DOCKSWAY DISPOSAL SITE AREA 2 STABILITY ASSESSMENT ALLUVIUM CRITICAL STATE SHEAR STRENGTH	JOB No: 14739/061	SHEET: Appendix 1 ADD4
		DATE: 9/12/05 · BY: LJT	CHECKED: AD

To estimate q' for any value of p' and u use the following



Based on trigonometry the values for α, β and γ can be calculated as follows:

α = arctan M for M = 1 α = 45^o

β = arctan 3/1 = 71.5^o

γ = (90 - α) - (90 - β) = 26.5^o

$p'_{undrained}$ = p' - u

x = $p'_{undrained}$ sin α

y = x/sin γ

z = y/sin β = $q'_{undrained}$ = $[(p'_{undrained} \times \sin\alpha)/\sin\gamma]/\sin\beta$

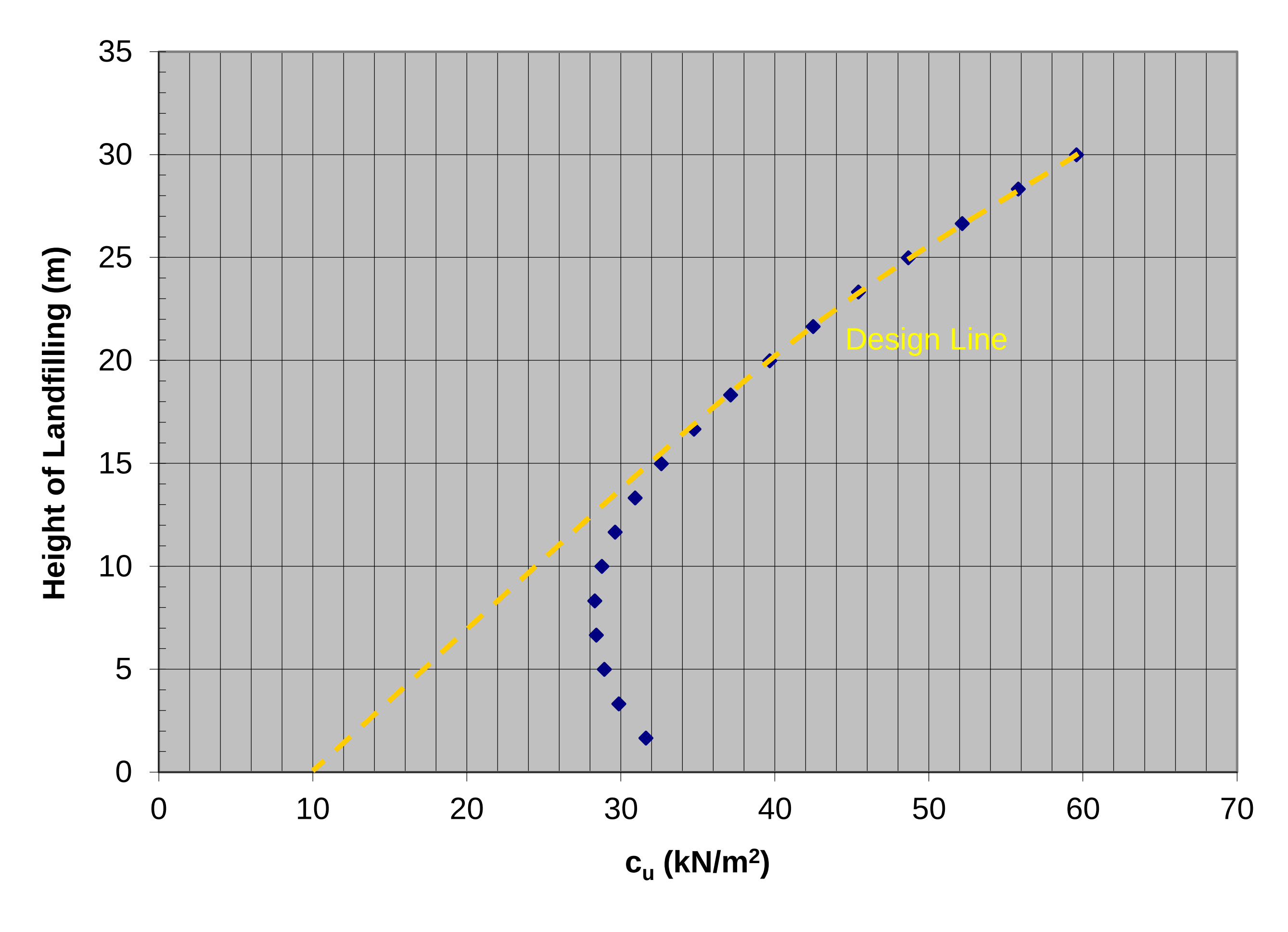
Based on this the values of p' and c_u have been estimated for the load increments during landfilling

See ADD5

peter brett associates CONSULTING ENGINEERS	JOB TITLE: DOCKSWAY DISPOSAL SITE AREA 2 STABILITY ASSESSMENT ALLUVIUM CRITICAL STATE SHEAR STRENGTH	JOB No: 14739/061		SHEET: Appendix 1 ADD5
		DATE: 9/12/05	BY: LJT	CHECKED: AD

H	u*	p'	p' -u	$q'_{undrained}$	c_u
1.66	16.6	54.5	37.9	63.25	32
3.33	31.208	66.9	35.7	59.72	30
4.99	44.82	79.4	34.6	57.85	29
6.66	57.934	91.9	34.0	56.82	28
8.32	70.55	104	33.9	56.62	28
9.99	82.502	117	34.4	57.52	29
11.7	93.956	129	35.5	59.26	30
13.3	104.912	142	37.0	61.84	31
15	115.37	154	39.0	65.24	33
16.7	125.33	167	41.6	69.48	35
18.3	134.958	179	44.4	74.27	37
20	144.42	192	47.5	79.34	40
21.7	153.55	204	50.8	84.96	42
23.3	162.514	217	54.4	90.86	45
25	171.146	229	58.2	97.31	49
26.7	179.446	242	62.4	104.32	52
28.3	187.58	254	66.8	111.61	56
30	195.548	267	71.3	119.18	60

peter brett associates CONSULTING ENGINEERS	JOB TITLE: DOCKSWAY DISPOSAL SITE AREA 2 STABILITY ASSESSMENT ALLUVIUM CRITICAL STATE SHEAR STRENGTH	JOB No: 14739/061		SHEET: Appendix 1 ADD6
		DATE: 9/12/05	BY: LJT	CHECKED: AD



Using a similar approach for the alluvium below the dams

For height of dam = 10m
and considering that the excess porewater pressure has dissipated to at least 70% of the excess porewater pressure

then for a point 0.5m below the underside of the dam

σ_1' = 20 x 10.5 = 210 kN/m²

σ_3' = 210 x 0.625 = 131.25 kN/m²

initial value of u = 10 x 10 = 100 kN/m²

Therefore 70% u = 70 kN/m²

p' = 157.5 p' - u = 87.5

Therefore q' = 146.22

and c_u = **73 kN/m²**

Also for the bunds between cell use a similar approach as follows

The height of the bund is 4m including the liner thickness

therefore σ'_1 = (4 x 20) + (0.3 + 2 + 0.5)x20 = 136 kN/m² Drained condition i.e. u = 0

Therefore σ'_3 = 0.625 x 136 = 85 kN/m²

For $p' = \frac{(\sigma_1'+2\sigma_3')}{3}$

p' = 102 kN/m²

Excess porewater pressure at construction of bund assuming no drainage

u = 4 x 20 = 80 kN/m²

Now using $q'_{undrained} = [(p'_{undrained} \times \sin\alpha)/\sin\gamma]/\sin\beta$

$q'_{undrained}$ = 37 kN/m²

Therefore c_u = **18.5 kN/m²**

Appendix 2 Deep Soil Mixing Method Statement

Project – Docksway Newport

Deep Soil Mixing General Site Proposals

DEEP SOIL MIXING LTD.

Project No.1134
Date: - 22/1/2015

Understanding of the Project

The works are essentially for design and construction of a working platform for the future construction of the new landfill development that is to be constructed in an area that was formally the course of the River Ebww. Some areas of the site have sufficient strength and low permeability cover to provide the required conditions to place land fill waste on top but the areas marked on the drawing indicate areas of special concern that need preparation treatment to both provide strength and maintain the low level of permeability for the base of the proposed site. Long term the importance of providing a barrier to mitigate leakage from the site is essential to the works.

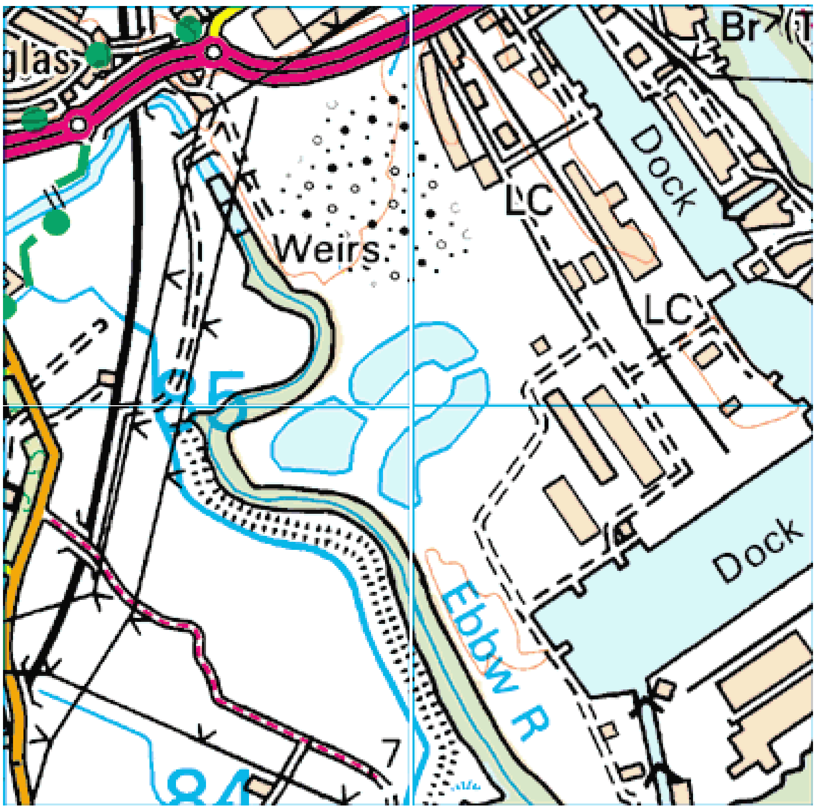
On the site there presently exists a band of very soft alluvium silts with low permeability properties that range in depths that vary between 8.0m in the east of the site and down to 2.0m in the riverbed. While these silts do have a low permeability level there is an importance to treat the top surface to form the working platform but it is considered essential that the bottom 2.0m of the silt must remain intact to provide a constant barrier to the underling terrace gravel.

In order that the site is best understood for the depths of alluvial overlying the gravels there is a need to carry out a grid survey of the site and probe the silts down to establish their depth, sections then need to be plotted and an assessment of treatment established. This will identify areas that do not have sufficient depth of alluvium to carry out the soil mixing treatment and the areas that can have material redistributed from to enable those areas with only the 2.0m of undisturbed alluvium to have newly treated material placed on top to provide the working platform with low permeability properties.

In addition to the survey work and assessment of the above DSML shall also look to utilize the early period of the project to trial some mixing operations with the proposed plant in order to demonstrate the binders and system of mixing is providing the required specification again to the client's engineer, the EA and DEFRA.

Project: **Docksway Newport**

General Site Location



General Working Methodology

The mass mixing would be performed using the Allu mixing head. By treating all of the soft silt material within the top 1.5-2.0m, the ground is transformed into a firm block of suitable strength to carry the proposed loading.

The ALLU PM Power Mix is a hydraulic mixing tool for excavators. The mixing power is based on the inclined locating of the drums and the unique structure of the mixing parts. The drums move simultaneously in three ways at the same time mixing the material in a controlled way. ALLU PMX Power Mix can process different materials all the way up to a depth of five meters depending on the chosen ALLU PMX Power Mix model, the excavator's reach and the quality of the material. The binder is feed by compressed air right inside the ground through a tube to a selected location near the drums. The amount of binder per square can range from only a few percent up to tens of percents. ALLU Power Mix is mounted as an accessory onto a standard excavator by a pin mounting or quick hitch adapter plate.

To maintain control of the mass mixing operation, the work needs to be carried out in cellular blocks ranging from approximately 70m3 to 100m3 at a time. Pre-construction layout of grids, individually identified, will be established on the construction drawings. Grid sizes and depth will be clarified on site prior to the mass mixing taking place and with the known volume to be mixed shall be calculated on a cell-by-cell bases. Monitoring equipment to record the binder volume used in each cell keeps the operator informed of how much is in the cell and how much is still needed to go in. Each cell has its unique number identified on the site plan with a print out of the volume used within that cell.

Treatment of the silts shall be by a wet mix system; dry binders shall be delivered to site in sealed containers and transferred into site-based silos through sealed pipelines. Then utilising the self contained batching plant that draws the powder/binder from the silos, mixing these with predetermined water: binder ratio the resultant grout is held in suspension in an agitating vessel until called off by the mixing rig operator.

Each batch that is mixed in automatic mode is electronically recorded for source of binders, with two binders being mixed on site records are downloaded on a daily bases to show the percentages of each binder being used in a batch, the weight of binder, volume of water, mixing times both period of mixing and time of day.

DSML propose to use Mass Stabilisation as the method for achieving the requirements of the project at Docksway Disposal Site. Mass stabilisation is a method to stabilise soft soils by adding binders to improve the shear strength. It is proposed to use blended bentonite and cement powder as the binder at Docksway this will be batched in ratios determined by the site trials and introduced to the soils requiring stabilisation using a computerised automated batching plant and an Allu mixing head attached to a 35T CAT 330 excavator. (See Below Sketches and Photo.)

It is proposed to use a wet mass mixing solution, in order to produce a consistent grout a specialised batching plant shall be erected with feeds from two silos that contain the dry powders. To produce the wet grout, a predetermined quantity of water is dropped into the mixing tank of the batching plant, followed by the programmed weight of Bentonite and Cement Binders, these shall be blended to the required consistency within the mixing pan before being discharged as a grout to the holding/agitator hopper where the grout is stored before being pumped to the soil mixing head. The speed of the grout being pumped is controlled by the Allu soil mixing operator who is mixing a particular cell. Each cell shall be individually identified and the actual volume of each cell shall be calculated and the required quantity of binder known in advance. The grout shall be continually supplied until the predetermined quantity has been batched for that particular cell size.

DSML consider that the top 1.5 – 2.0m of silt can be mass mixed to meet both the required soil strength and the permeability factors.

Full method statement shall be provided for all these activities prior to the work commencing but it is clear to say that the client's engineer, Newport CC the EA and DEFRA shall all be satisfied with the survey information and the working proposals prior to commencing any full scale stabilization operations.

As mentioned previously for installation of the actual works we shall be bringing in our own ALLU mixing equipment and our own batching plant for the mixing and pumping of the binders as a grout.

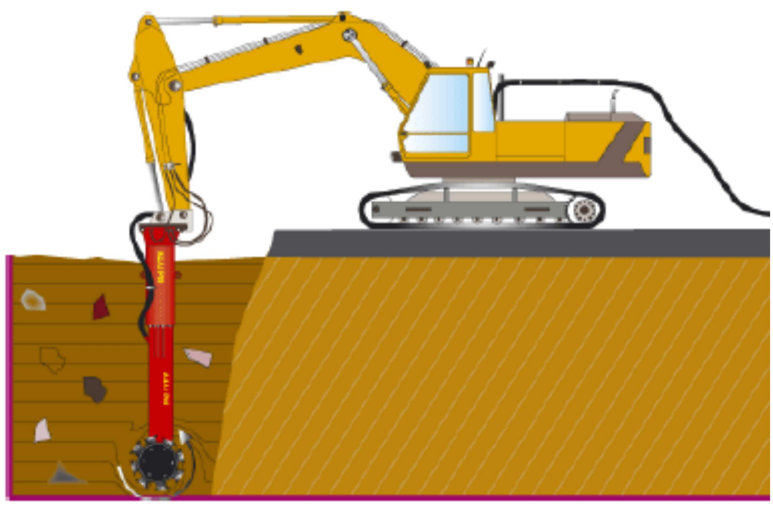
We recognise the importance of probing the alluvium layer to identify the depth to the gravels and this shall be carried out with a small dynamic probe rig that we shall again sub-contract in but this process we consider is not needed for the complete 6m by 6m grid.

We propose initially carrying out cross-sections of the site at about 20 m spacing with the probing being concentrated nearer the old riverbed thus identifying the risk areas for depth of alluvium and potential areas that need material transferred to/built up.

For this work, we envisage placing large timber rafts on top of the softer silts to provide a safe working platform for the probing rig.

For the main work, all men on site shall be directly employed by DSML. With an experience soil mixing foreman working full time on the project the upper tier management shall be either Colin Critchlow or Robert McGall who shall be onsite a minimum of 90% of the working week but who shall always be available on mobile phone. Full time site Engineer/manager shall be George Olney.

Mass Mixi



Mass mixing with the ALLU mixing arm and head can enable the mixing operations to reach depths of 5.0m but as seen in the photograph a simple mark to represent the desired depth keeps the operator aware and the engineer in tune with the correct depth.

<u>Sketch showing typical Mass Mixing Operation using the ALLU Mixing Head</u>

2) Site Programme

Short Programme

	Description	Months								
		Weeks	Jan	Feb	Mar	Apr	May	Jun	July	Aug
	Mobilisation of Plant site compound fencing	**2**								
	Field Trials with ALLU mixing head.	**1**								
	Site Survey - Probing of site and lay out of cells	**3**								
	Testing of site trial samples and assessment of results	**1**								
	Anticipated main works start date Week Commencing 16th Feb 2015									
	Installation of soil mixed blanket over site 15000m2	**15**								
	Testing procedures and validation works									
	Decommission and move off site									
	Design Validation report and Warrantees issued	**4**								

While this is a simple programme, there will be a need to expand this once the site survey and plan of mixing cells has been established with ID's, plan sizes and depths.

With this information recorded, the anticipated progress programme shall be established and used to monitor weekly working progress reports. Testing regime and designer reporting schedule shall also be plotted onto the contract programme with monthly site meetings.

Appendix 3 Capping Stability Assessment

Calculations

Artificial Capping Layer Stability Assessment

Assumes 1 in 4 slope for final restoration profile max elevation 40mOD

Project Title	Docksway Disposal Site: Area 2								
Project No	14739	156	By	AD	Checked	LJT	Date	24	04	2015

1.0 Introduction

This spreadsheet has been developed based on the information and references contained within:

- Jones DRV and Dixon N. Stability of Landfill Lining Systems: Report No. 1, Literature Review. R&D Technical Report P1-385/TR1, Environment Agency 2003.
- Jones DRV and Dixon N. Stability of Landfill Lining Systems: Report No. 2, Guidance. R&D Technical Report P1-385/TR2, Environment Agency 2003.

It has been modified to use partial factors for overall stability from BS EN 1997-1:2004 Eurocode 7: Geotechnical design - Part 1: General Rules and the UK National Annex to this document.

UK practice has adopted Design Approach 1 (DA-1) which requires design to be undertaken using two different Combinations of partial factors. Although not explicitly stated in BS EN 1997-1, in DA-1, Combination 2 is normally the recommended method for overall stability checking in problems where ground is the main element providing resistance; in such cases, Combination 1 is not relevant.

Treatment of permanent loads due to gravity loads and water is often difficult in DA-1 Combination 1, since these loads are usually unfavourable in part of the sliding mass but favourable in another part.

Therefore this spreadsheet only uses the partial factors for soil parameters for overall stability for the design case DA-1 Combination 2, as follows:

		Partial Factors
Coefficient of shearing resistance (tan ϕ)	γ_ϕ	1.25
Effective cohesion (c')	$\gamma_{c'}$	1.25
Weight density (γ_d and γ_{sat})	γ_γ	1.00

Note: Over Design Factors (ODF) of 1.00 or greater are required for a safe design.

Calculations

Artificial Capping Layer Stability Assessment

Assumes 1 in 4 slope for final restoration profile max elevation 40mOD

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	AD	Checked	LJT	Date	24	04	2015

2.0 Capping Liner Stability

Assumed Profile Capping Layer

Restoration Layer h_1 = 0.6 m

Upper Protection Layer h_2 = 0.2 m — L_1

Pozidrain (G6SD) — L_2

LLDPE 1mm textured — L_3

Pozidrain (G6SD) — L_4

Existing Regulation Layer

Use the method proposed by Jones and Dixon (1998).

The following peak interface shear strength parameters have been assumed and will require confirmation:

Restoration/Upper Protection layer	c'_k =	0	kN/m^2	ϕ'_k =	35	deg	
Upper Protection layer/Pozidrain	α_1 =	0	kN/m^2	δ_1 =	35	deg	Data based on
Pozidrain/Textured LLDPE	α_2 =	0	kN/m^2	δ_2 =	25	deg	laboratory results
Textured LLDPE/Pozidrain	α_3 =	0	kN/m^2	δ_3 =	25	deg	by Geospec
Pozidrain/existing Regulation layer	α_4 =	0	kN/m^2	δ_4 =	35	deg	Oct/Nov 13

Characteristic Input Parameters

Upper protection layer unit weight dry	γ_k	=	18	kN/m^3
Upper protection layer unit weight saturated	$\gamma_{sat,k}$	=	21	kN/m^3
Upper protection layer angle of internal friction	ϕ'_k	=	35	deg
Upper protection layer effective cohesion	c'_k	=	0	kN/m^2

Calculations

Artificial Capping Layer Stability Assessment

Assumes 1 in 4 slope for final restoration profile max elevation 40mOD

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	AD	Checked	LJT	Date	24	04	2015

Design Parameters (Factored)

Restoration/Upper Protection layer	c'_d	=	0	kN/m^2	ϕ'_d	=	29.3	deg
Upper Protection layer/Pozidrain	α_1	=	0	kN/m^2	δ_1	=	29.3	deg
Pozidrain/Textured LLDPE	α_2	=	0	kN/m^2	δ_2	=	20.5	deg
Textured LLDPE/Pozidrain	α_3	=	0	kN/m^2	δ_3	=	20.5	deg
Pozidrain/existing Regulation layer	α_4	=	0	kN/m^2	δ_4	=	29.3	deg

Upper protection layer unit weight dry	γ_d	=	18	kN/m^3
Upper protection layer unit weight saturated	$\gamma_{sat,d}$	=	21	kN/m^3
Upper protection layer angle of internal friction	ϕ'_d	=	29.3	deg
Upper protection layer effective cohesion	c'_d	=	0	kN/m^2

Additional Input Parameters

Unit weight of water	γ_w	=	9.81	kN/m^3	
Thickness of upper protection layer	$h = h_1 + h_2$	=	0.8	m	
Height of slope	H	=	35	m	
Slope angle	β	=	14	deg	(1v:4h)

Length of slope $L = \dfrac{H}{\sin\beta}$ = 145 m

Geosynthetic tensile strength:

Pozidrain (G6SD data sheet)	=	18	kN/m
LLDPE (1mm textured, GRI GM17)	=	11	kN/m
Pozidrain (G6SD data sheet)	=	18	kN/m

Calculations

Artificial Capping Layer Stability Assessment

Assumes 1 in 4 slope for final restoration profile max elevation 40mOD

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	AD	Checked	LJT	Date	24	04	2015

Parallel Submergence Ratio PSR = 0

Height of water $h_w = PSR \times h$ = 0 m

Total weight of active wedge $W_A = \dfrac{[\gamma_d(h-h_w)(2H\cos\beta - (h+h_w)) + \gamma_{sat,d}h_w(2H\cos\beta - h_w)]}{\sin(2\beta)}$ = 2058.78 kN

Total weight of passive wedge $W_P = \dfrac{[\gamma_d(h^2-h_w^2)+(\gamma_{sat,d}h_w^2)]}{\sin(2\beta)}$ = 24.54 kN

Resultant of the pore pressures acting perpendicular to slope $U_n = \dfrac{[\gamma_w h_w \cos\beta(2H\cos\beta - h_w)]}{\sin(2\beta)}$ = 0.00 kN/m

Resultant of pore pressures acting on the interwedge surfaces $U_h = \dfrac{\gamma_w h_w^2}{2}$ = 0.00 kN/m

Resultant of the vertical pore pressures acting on the passive wedge $U_v = \dfrac{U_h}{\tan\beta}$ = 0.00 kN/m

Effective force normal to the failure plane of the active wedge $N_A = W_A\cos\beta + U_h\sin\beta - U_n$ = 1997.62 kN/m

Over Design Factor $ODF = \dfrac{-b + \text{SQRT}(b^2 - 4ac)}{2a}$

Where $a = W_A\sin\beta\cos\beta - U_h\cos^2\beta + U_h$ = 483.27

$b = - W_A\sin^2\beta\tan\phi_d + U_h\sin\beta\cos\beta\tan\phi_d - \cos\beta((\alpha_1 L)+N_A(\tan\delta_1))$
$- (W_P-Uv)\tan\phi_d - c'h/\sin\beta$ = -1167.00

$c = \sin\beta\tan\phi_d(\alpha_1 L + N_A\tan\delta_1)$ = 151.64

Therefore Over Design Factor **ODF = 2.28**

Calculations

Artificial Capping Layer Stability Assessment	
Assumes 1 in 4 slope for final restoration profile max elevation 40mOD	

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	AD	Checked	LJT	Date	24	04	2015

Parallel Submergence Ratio PSR = 0.25

Height of water $h_w = PSR \times h$ = 0.2 m

Total weight of active wedge $W_A = \dfrac{[\gamma_d(h-h_w)(2H\cos\beta - (h+h_w)) + \gamma_{sat,d}h_w(2H\cos\beta - h_w)]}{\sin(2\beta)}$ = 2145.33 kN

Total weight of passive wedge $W_P = \dfrac{[\gamma_d(h^2-h_w^2)+(\gamma_{sat,d}h_w^2)]}{\sin(2\beta)}$ = 24.79 kN

Resultant of the pore pressures acting perpendicular to slope $U_n = \dfrac{[\gamma_w h_w \cos\beta(2H\cos\beta - h_w)]}{\sin(2\beta)}$ = 274.61 kN/m

Resultant of pore pressures acting on the interwedge surfaces $U_h = \dfrac{\gamma_w h_w^2}{2}$ = 0.20 kN/m

Resultant of the vertical pore pressures acting on the passive wedge $U_v = \dfrac{U_h}{\tan\beta}$ = 0.79 kN/m

Effective force normal to the failure plane of the active wedge $N_A = W_A\cos\beta + U_h\sin\beta - U_n$ = 1807.04 kN/m

Over Design Factor $ODF = \dfrac{-b + SQRT(b^2 - 4ac)}{2a}$

Where $a = W_A\sin\beta\cos\beta - U_h\cos^2\beta + U_h$ = 503.60

$b = - W_A\sin^2\beta\tan\phi_d + U_h\sin\beta\cos\beta\tan\phi_d - \cos\beta((\alpha_l L)+N_A(\tan\delta_l))$
$- (W_P-Uv)\tan\phi_d - c'h/\sin\beta$ = -1065.93

$c = \sin\beta\tan\phi_d(\alpha_l L + N_A\tan\delta_l)$ = 137.18

Therefore Over Design Factor **ODF = 1.98**

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Calculations

Artificial Capping Layer Stability Assessment

Assumes 1 in 4 slope for final restoration profile max elevation 40mOD

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	AD	Checked	LJT	Date	24	04	2015

Parallel Submergence Ratio PSR = 0.5

Height of water $h_w = PSR \times h$ = 0.4 m

Total weight of active wedge $W_A = \dfrac{[\gamma_d(h-h_w)(2H\cos\beta - (h+h_w)) + \gamma_{sat,d}h_w(2H\cos\beta - h_w)]}{\sin(2\beta)}$ = 2231.37 kN

Total weight of passive wedge $W_P = \dfrac{[\gamma_d(h^2-h_w^2)+(\gamma_{sat,d}h_w^2)]}{\sin(2\beta)}$ = 25.56 kN

Resultant of the pore pressures acting perpendicular to slope $U_n = \dfrac{[\gamma_w h_w \cos\beta(2H\cos\beta - h_w)]}{\sin(2\beta)}$ = 547.60 kN/m

Resultant of pore pressures acting on the interwedge surfaces $U_h = \dfrac{\gamma_w h_w^2}{2}$ = 0.78 kN/m

Resultant of the vertical pore pressures acting on the passive wedge $U_v = \dfrac{U_h}{\tan\beta}$ = 3.15 kN/m

Effective force normal to the failure plane of the active wedge $N_A = W_A\cos\beta + U_h\sin\beta - U_n$ = 1617.68 kN/m

Over Design Factor $ODF = \dfrac{-b + SQRT(b^2 - 4ac)}{2a}$

Where $a = W_A\sin\beta\cos\beta - U_h\cos^2\beta + U_h$ = 523.83

$b = - W_A\sin^2\beta\tan\phi_d + U_h\sin\beta\cos\beta\tan\phi_d - \cos\beta((\alpha_1 L)+N_A(\tan\delta_1))$
$- (W_P-Uv)\tan\phi_d - c'h/\sin\beta$ = -964.86

$c = \sin\beta\tan\phi_d(\alpha_1 L + N_A\tan\delta_1)$ = 122.80

Therefore Over Design Factor **ODF = 1.70**

Sheet Cap5 of

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3.0 Capping Liner Integrity

3.1 Slope Deformation

Integrity have been carried out for the geosynthetic liners in accordance with the recommendations given by Bourdeau et al (1993) as modified by Jones and Dixon (1998). The same interfriction strengths to those quoted above for the capping liner have been adopted.

Where PSR = 0

Tension in Pozidrain = $\left[\left(\frac{\alpha_1}{F}-\alpha_2\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_1}{F}-\tan\delta_2\right)\right]L$ = -256.79 kN/m

ODF = **Infinite**

Tension in LLDPE = $\left[\left(\frac{\alpha_2}{F}-\alpha_3\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_2}{F}-\tan\delta_3\right)\right]L$ = -422.91 kN/m

ODF = **Infinite**

Tension in Pozidrain = $\left[\left(\frac{\alpha_3}{F}-\alpha_4\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_3}{F}-\tan\delta_4\right)\right]L$ = -801.16 kN/m

ODF = **Infinite**

Since the tension in the geosynthetics are negative, the shear strength of the lower interface is greater than the mobilised shear stress on the preceding interface and there is no tension developed in the geosynthetics. The mobilised shear stress is thus transferred into the existing regulation layer without any tension.

Calculations

Artificial Capping Layer Stability Assessment

Assumes 1 in 4 slope for final restoration profile max elevation 40mOD

Project Title	Docksway Disposal Site: Area 2								
Project No	14739	156	By	AD	Checked	LJT	Date	24	04	2015

Where PSR = 0.25

Tension in Pozidrain = $\left[\left(\frac{\alpha_1}{F}-\alpha_2\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_1}{F}-\tan\delta_2\right)\right]L$ = -189.49 kN/m

ODF = Infinite

Tension in LLDPE = $\left[\left(\frac{\alpha_2}{F}-\alpha_3\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_2}{F}-\tan\delta_3\right)\right]L$ = -388.59 kN/m

ODF = Infinite

Tension in Pozidrain = $\left[\left(\frac{\alpha_3}{F}-\alpha_4\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_3}{F}-\tan\delta_4\right)\right]L$ = -782.6 kN/m

ODF = Infinite

Since the tension in the geosynthetics are negative, the shear strength of the lower interface is greater than the mobilised shear stress on the preceding interface and there is no tension developed in the geosynthetics. The mobilised shear stress is thus transferred into the existing regulation layer without any tension.

Where PSR = 0.5

Tension in Pozidrain = $\left[\left(\frac{\alpha_1}{F}-\alpha_2\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_1}{F}-\tan\delta_2\right)\right]L$ = -97.2 kN/m

ODF = Infinite

Tension in LLDPE = $\left[\left(\frac{\alpha_2}{F}-\alpha_3\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_2}{F}-\tan\delta_3\right)\right]L$ = -337.62 kN/m

ODF = Infinite

Tension in Pozidrain = $\left[\left(\frac{\alpha_3}{F}-\alpha_4\right)+\left(\gamma_{sat}h_w+\gamma_d(h-h_w)\right)\cos\beta\left(\frac{\tan\delta_3}{F}-\tan\delta_4\right)\right]L$ = -747.39 kN/m

ODF = Infinite

Since the tension in the geosynthetics are negative, the shear strength of the lower interface is greater than the mobilised shear stress on the preceding interface and there is no tension developed in the geosynthetics. The mobilised shear stress is thus transferred into the existing regulation layer without any tension.

Calculations

Artificial Capping Layer Stability Assessment

Assumes 1 in 4 slope for final restoration profile max elevation 40mOD

Project Title	Docksway Disposal Site: Area 2								
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Summary

For a 1(v):4(h) slope the calculations show that the over design factor obtained for sliding of the cover soil on the capping liner decreases from 2.28 to 1.70 as the parallel submergence ratio (PSR) increases from 0 to 0.5. The PSR is the ratio of the height of water in the capping layer to the total thickness of the capping layer i.e. when PSR = 0.5 the capping layer is fully saturated. Therefore there is an adequate factor of safety against sliding of the cover soil.

The calculations show that for all parallel submergence ratios (including a fully saturated capping layer) no tension is developed in any of the geosynthetics that make up the capping layer. Therefore there is an adequate factor of safety against rupture failure of the proposed geosynthetics.

3.2 Compressible Waste

The multi-axial performance of the LLDPE is better than the HDPE geomembrane and is considered to be suitable for the proposed capping system.

3.3 Cavities

Calculate the strain imposed on the LLDPE using the approach given by Jones and Pine (2001).

<u>Input parameters</u>

Max allowable tension in LLDPE (T_d)		=	11	kN/m (from GRI GM17)
Unit weight of cover soil (γ_d)		=	18	kN/m³
Thickness of cover soil (h)		=	0.8	m
Surcharge above LLDPE (q)	= γ_d x h	=	14.4	kN/m²
Width of void (a)		=	0.6	m (typical value given in EA Guidance)
Capping layer angle (β)		=	14	deg

Calculate the reduced load (ω) on the geomembrane due to arching in the soil layer:

$$\omega = 2\gamma_d a(1 - e^{-0.5h/a}) + qe^{-0.5h/a} = 17.90 \text{ kN/m}^2$$

Calculate the horizontal force at the edges of the void (H):

where

$A' = 1 + \tan^2\beta$ = 1.06

$B' = \omega.a.\tan\beta$ = 2.68

$C' = (\omega.a)^2/4 - T_d^2$ = -92.2

Therefore

$$H = \frac{-B' + \text{SQRT}(B'^2 - 4A'C')}{2A'} = 8.14 \text{ kN}$$

Calculations

Artificial Capping Layer Stability Assessment

Assumes 1 in 4 slope for final restoration profile max elevation 40mOD

Project Title	Docksway Disposal Site: Area 2									
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Calculate the deformed length (L) of the geomembrane. First calculate the vertical force at the lower side of the void (R_1) and constants A and B:

$R_1 = (\omega.a/2) - H\tan\beta$	=	3.34
$A = \omega/(2H)$	=	1.1
$B = R_1/H$	=	0.41

Calculate L:

$J = 2Aa - B$	=	0.91
$K = -B$	=	-0.41
$j = \text{arcsinh}(J) + J(1+J^2)^{0.5}$	=	2.04
$k = \text{arcsinh}(K) + K(1+K^2)^{0.5}$	=	-0.84
$L = (j-k)/4A$	=	0.66

Calculate the strain in the geogrid:

$D = a/\cos\beta$	=	0.62
$\varepsilon = (L-D)/D$	=	**0.06 or 6.17%**

Comment

The maximum permissible strain of the 1mm textured LLDPE geomembrane to be used in the capping layer must be greater than 6.2%.

Calculations

Artificial Capping Layer Stability Assessment	
Assumes 1 in 4 slope for final restoration profile max elevation 40mOD	

Project Title	Docksway Disposal Site: Area 2									
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4.0 Landfill Gas Pressure

An assessment of the possible affects of landfill gas pressure on the stability of the capping system has been carried out.

From Thiel (1999), the factor of safety for an infinite slope with gas pressure is given by:

$$FoS = \frac{\alpha + (h.\gamma_d.\cos\beta - u_g)\tan\delta}{h.\gamma_d.\sin\beta}$$

where

α is the cohesion intercept of the lower geomembrane interface	0	kN/m²
δ is the friction angle of the lower geomembrane interface	29.3	deg
h is the thickness of the cover soil above the membrane	0.8	m
β is the slope angle	14	deg
u_g is the gas pressure beneath the geomembrane	?	kN/m²
γ_d is the unit weight of the cover soil	18	kN/m³

Assuming a minimum factor of safety of 1.2

the maximum allowable gas pressure will be

$$u_g = h.\gamma_d.\cos\beta - \frac{FoS.h.\gamma_d.\sin\beta - \alpha}{\tan\delta} = \mathbf{6.51}\ \text{kN/m}^2$$

For a factor of safety of 1.2, a maximum gas pressure of 6.5 kN/m² on the geomembrane is permissible and this pressure should not be exceeded. The active gas extraction system at the site should be managed to ensure this does not occur.

Appendix 4 Area 2 Cell 2 Vibrating Wire Piezometer Data Review Report

Newport City Council

Docksway Disposal Site - Area 2 Cell 2. Vibrating Wire Piezometers

Data Review Report

Project Ref: 14739/139

Doc Ref: R001/rev01

November 2011

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Document Control Sheet

Project Name: **Docksway Disposal Site - Area 2 Cell 2. Vibrating Wire Piezometers**
Project Ref: **14739/13939**
Report Title: **Data Review Report**
Doc Ref: **R001/rev01**
Date: **November 2011**

	Name	Position	Signature	Date
Prepared by:	Kate Riley	Senior Geotechnical Engineer	[signature]	23/11/2011
Reviewed and Approved by:	Richard Thomas	LLP Director	[signature]	23/11/2011
For and on behalf of Peter Brett Associates LLP				

Revision	Date	Description	Prepared	Reviewed	Approved
001	23/11/11	Issue	VKR	RHT	RHT

Contents

Figures

Appendices

1 Introduction

Peter Brett Associates LLP (PBA) have been retained by Newport City Council (NCC) to provide a preliminary data review report for 5 No. vibrating wire piezometers which were installed in Area 2 Cell 2 at Docksway Disposal Site, Newport during September 2010. The purpose of the vibrating wire piezometers is to monitor groundwater pore pressures in the 'soft' alluvium (present beneath the site) as landfilling proceeds to check that any build up of pore pressure due to the infill load does not place the landfill at risk of failure, thus compromising the stability of the landfill. This will enable filling to take place at the most appropriate rate and for filling to slow or cease as factors of safety approach the limiting design value.

1.1 General

The Construction Quality Assurance (CQA) Validation Report for the installation of the vibrating wire piezometers (hereafter referred to as 'the piezometers') recommended that the piezometers were monitored weekly for a period of 2 months from installation and then monthly for a period of 6 months when the data should be reviewed to determine the ongoing monitoring programme. In addition the report recommended that the piezometers were monitored over a complete tidal cycle for a period of 14 hours, with readings taken every hour.

This report summarises the baseline readings obtained since installation, including the tidal cycle monitoring, and provides recommendations for ongoing monitoring and filling based on interpretation of the data.

1.2 Parties

Docksway Disposal Site at Newport is operated by Newport City Council (the Client). The monitoring of the piezometers is carried out by an Engineering Design Assistant from Newport City Council.

1.3 Scope of Works

The scope of works undertaken comprised the following;

- Review of the tidal cycle monitoring data to determine if there are any significant obvious tidal effects on the piezometers.
- Review and interpretation of the monitoring data available against the 'Stability Assessment Report for Area 2 – Landfill Extension' (PBA 2005) to determine the current groundwater pore pressures within the alluvium.
- Preparation of this report to include recommendations for ongoing monitoring and landfilling in Cell 2.

2 Site Description

2.1 Site Location and Setting

The Docksway Disposal Site is situated approximately 3km south of Newport City Centre between the River Ebbw and Newport Docks (see Figure 1). The site is centred on NGR ST305853 and occupies a total area of around 60 hectares. The site is maintained and operated by Newport City Council (NCC).The A48 (Docksway) Road lies adjacent to the north of the site. The area to the north of the A48 comprises predominantly residential development, but includes the Maes Glas Industrial Estate. The area to the west of the site (on the western bank of the River Ebbw) comprises grassland currently used for the grazing of livestock and is known to have been used historically for landfilling. The northern part of the eastern boundary of the site is formed by a stream (the Maes Glas Pill). East of the Maes Glas Pill the land is owned and occupied by Associated British Ports (ABP). The River Ebbw also forms the southern boundary of the site. Arable land is largely present to the south of the River Ebbw. Entrance to the site is gained via an access road located off a roundabout on the A48 Docksway Trunk Road. The Civic Amenity Areas, Weighbridge and Site Offices are all located off this access road.

2.2 Current Land Use

The site can be divided into two halves. The northern half (Area 1) comprises a capped but unlined dilute and disperse landfill operated under waste management license number EAWML30058. Area 1 has been used for waste disposal since approximately 1939. Waste deposition was completed in Area 1 during 2007.

The southern half (Area 2) comprises an engineered contained landfill currently comprising two cells with other undeveloped areas allocated for further cell construction. Filling operations commenced in Area 2 Cell 1 in April 2007 and ceased in August 2010. Filling then commenced in a small area of Cell 2 prior to installation of the vibrating wire piezometers. Area 2 is centred on National Grid Reference ST 305 853 and covers an area of approximately 15.4 hectares. Area 2 of Docksway Landfill is classed as a non-hazardous landfill (under the Landfill Regulations 2002). According to Pollution Prevention Control (PPC) Permit no. DP3733BK the operator is permitted to receive, handle, store and dispose of non-hazardous waste only.

3 Geoenvironmental Setting and Ground Conditions

3.1 Published Geology

According to the regional geology map (BGS, 1986) the subject site is underlain by Mercia Mudstones of the Triassic period, which are in-turn overlain by recent Estuarine Alluvium and River Terrace Gravels. Around the perimeter of the site the natural deposits are overlain by Made Ground, that is understood to have been placed to reclaim low-lying ground and form flood protection banks and site roadways. Within parts of the site area there is a substantial thickness of domestic refuse materials.

3.2 Ground Conditions

A number of previous investigations have been carried out across Area 2 and a summary of the ground conditions encountered is presented below. Generally the information from previous investigations confirms the published geology.

The Alluvium beneath the site generally comprises a firm to stiff grey clay/silt between 1m and 2m thick overlying a very soft to firm grey silt or clay with sand and gravel at the base. The top 1m to 2m probably represent a dessicated crust and in Area 2, this has been removed from large areas for use in the Area 1 landfilling. The Alluvium has been partially cut through by former river channels in places and ranges in thickness between about 2m and 9m with an elevation at the top of the stratum between about 8m OD and 1m OD.

The River Gravels were encountered below the Alluvium comprising medium dense to very dense sandy gravels containing clay, cobbles and boulders. The thickness of the River Gravels was found to vary from about 4m to 9m thick, though the thickness was not proven in all the exploratory holes. The elevation at which the deposits were encountered varies from about -0.5mOD to about -3.5mOD.

Mercia Mudstone was encountered at depth below the River Gravels and was found to comprise a weathered very stiff red brown clay or silt. The full thickness was not proven and the upper surface was encountered at an elevation of between about -5mOD and -11m OD.

Beneath Cells 1 and 2 the top of the Alluvium has previously been ‘stabilised’ by in-situ mixing with cement to create 2m long columns. The cement stabilised columns were installed in a grid pattern to provide some increase in strength of the soils between the columns. Following stabilisation of the Alluvium, an approximately 300mm thick load transfer blanket was installed over the top, on top of which a minimum 1.2m thick engineered imported clay liner was constructed. Finally, an approximately 500mm thick granular drainage blanket was constructed on top of the engineered clay barrier. Filling in Cell 2 commenced in 2010 on top of the granular drainage blanket.

3.3 Installation of Piezometers

The works to prepare and install the vibrating wire piezometers were carried out during September 2010. Five piezometers were installed in Cell 2, targeted at areas where the Alluvium is thickest.

The installation details of the piezometers and the ground conditions encountered during installation are contained within the CQA report (PBA, 2010)[1]. The locations of the piezometers are shown on Figure 2.

4 Monitoring Data Review

4.1 Tidal Monitoring

The monitoring over a tidal cycle took place on the 28th September 2011, commencing at 07.30am and finishing at 21.30pm. High water was at 08.22am and 20.41pm with low water at 14.07pm. Atmospheric pressure was recorded as 1023mb at 07.30am, falling to 1020mb at 18.30pm and rising to 1021mb for the last 2 readings.

4.2 Data Processing

The data recorded on site is in frequency based units and is processed to obtain engineering units using the calculations provided in the piezometer user manual (contained in Appendix 1). This includes for corrections for atmospheric pressure. The engineering units calculated are then used to derive an equivalent mH2O above the base of the landfill.

4.3 Tidal Effects

Data obtained during the monitoring over a full tidal cycle indicates that the piezometers are not affected by any tidal influence. Figure 3 shows that none of the piezometers recorded any significant changes in frequency and therefore equivalent porewater pressure during the tidal cycle monitoring. Therefore it can be concluded that no correction is required for readings taken at different stages during the tidal cycle in the ongoing monitoring.

4.4 Survey

NCC undertook the survey to provide the precise locations of each of the instruments and the precise ground level elevations at each of the instrument locations, from which the precise piezometer tip elevations have been calculated as detailed in Table 4.1 below.

Table 4.1 Survey Details

Exploratory Hole	Easting	Northing	Ground Level (mOD)	Calculated Tip Elevation (mOD)
BH1	184894.310	331010.501	4.911	-0.609
BH2	184884.141	330913.707	6.094	0.394
BH3	184845.439	330965.705	4.886	-0.744
BH4	184791.490	330919.419	5.449	-0.351
BH5	184766.053	330965.533	5.243	-0.557

4.5 Monitoring Data

The piezometers have been monitored by NCC on seventeen occasions post connection to October 2011, at approximately weekly or bi monthly intervals for 2 months and then monthly. The readings have been taken during a range of atmospheric pressures and also during the initial filling stages of some of the subcells.

4.6 Current Rate of Filling

The current rate of filling within Cell 2 is variable but in the worst case (most rapid) has been 4m per month. Generally the rate of filling has been approximately 1.5 to 2m per month.

4.7 Data Review

The raw monitoring data obtained from each of the piezometers has been processed as discussed in Section 4.2 and plotted. Figures 4 to 8 show the equivalent porewater pressure in mH20 as reduced levels with reference to the base of the landfill.

Figures 4 (BH1) and 5 (BH2) appear to indicate that there is a noticeable increase in equivalent mH20 and therefore porewater pressure in the alluvium during or shortly after waste is placed and compacted in the relevant sub cells. Figure 6 indicates that this effect may also be starting to occur following placement and compaction of waste during July 2011 over BH3. The sub cells that contain BHs 4 and 5 had not received any waste following the blinding layer at the time of writing and the data for these piezometers was not indicating any noticeable effects, as shown on Figures 7 and 8.

The Stability Assessment Report for Area 2 – Landfill Extension (PBA 2005)[2] indicates that for waste slopes up to 8m in height, with side slopes no greater than 1:5.5, there is a Factor of Safety (FOS) of 3 assuming porewater pressure is no greater than 3m above the base of the landfill. The monitoring data available indicates that in the worst case (BH1), porewater pressure is currently no greater than 1.5m above the base of the landfill.

The Stability Assessment Report (PBA 2005)[2] also indicates that for internal bunds with waste no greater than 5m in height and side slopes no greater than 1:3, the FOS is greater than 2 assuming the porewater pressure is 0.5m above the base of the landfill. The FOS reduces to 1.77 when porewater pressure is 4m above the base of the landfill. In the worst case, the monitoring data indicates that in one area of Cell 2, porewater pressures are approximately 1.4m above the base of the landfill. This tends to suggest the FOS is between 1.77 and 2.

5 Conclusions & Recommendations

The piezometer monitoring over a tidal cycle does not indicate any tidal influence on the piezometers and therefore the monitoring can continue to be undertaken at times to suit the monitor, and not scheduled for the same stage of the tidal cycle each time.

The data available at the time of writing indicates that the porewater pressures within the Alluvium are currently well below those assumed in the Stability Assessment Report (PBA 2005)[2] for acceptable Factors of Safety in the current conditions. The data indicates (as shown on Figures 4 to 8) that equivalent porewater pressures rise following a period of filling, and are slower to dissipate. Therefore it is recommended that monitoring can reduce to every other month for the next two years, after which the data should be reviewed again.

It is recommended that the maximum rate of filling does not exceed 2.5m/month and should be limited to a maximum increase in landfill height of 5m within any 2 month period for any area of filling, in accordance with the Stability Assessment Report (PBA, 2005)[2].

REFERENCES

1) Peter Brett Associates, Docksway Disposal Site – Area 2 Cell 2. Installation of Vibrating Wire Piezometers. Construction Quality Assurance Validation Report. November 2010.

2) Peter Brett Associates, Docksway Disposal Site, Newport. Stability Assessment Report for Area 2 – Landfill Extension. Revision 1 April 2005.

Essential Guidance for Report Readers

This report has been prepared within an agreed timeframe and to an agreed budget that will necessarily apply some constraints on its content and usage. The remarks below are presented to assist the reader in understanding the context of this report and any general limitations or constraints. If there are any specific limitations and constraints they are described in the report text.

1 The opinions and recommendations expressed in this report are based on statute, guidance, and best practice current at the time of its publication. Peter Brett Associates LLP (PBA) does not accept any liability whatsoever for the consequences of any future legislative changes or the release of subsequent guidance documentation, etc. Such changes may render some of the opinions and advice in this report inappropriate or incorrect and we will be pleased to advise if any report requires revision due to changing circumstances, especially those over one year old. Following delivery of any report PBA has no obligation to advise the Client or any other party of such changes or their repercussions.

2 Some of the conclusions in this report may be based on third party data. No guarantee can be given for the accuracy or completeness of any of the third party data used. Historical maps and aerial photographs provide a "snap shot" in time about conditions or activities at the site and cannot be relied upon as indicators of any events or activities that may have taken place at other times.

3 The conclusions and recommendations made in this report and the opinions expressed are based on the information reviewed and/or the ground conditions encountered in exploratory holes and the results of any field or laboratory testing undertaken. There may be ground conditions at the site that have not been disclosed by the information reviewed or by the investigative work undertaken. Such undisclosed conditions cannot be taken into account in any analysis and reporting.

4 It should be noted that groundwater levels, groundwater chemistry, surface water levels, surface water chemistry, soil gas concentrations and soil gas flow rates can vary due to seasonal, climatic, tidal and man made effects.

5 This report has been written for the sole use of the Client stated at the front of the report in relation to a specific development or scheme. The conclusions and recommendations presented herein are only relevant to the scheme or the phase of project under consideration. This report shall not be relied upon or transferred to any other party without the express written authorisation of PBA. Any such party relies upon the report at its own risk.

6 The interpretation carried out in this report is based on scientific and engineering appraisal carried out by suitably experienced and qualified technical consultants based on the scope of our engagement. We have not taken into account the perceptions of, for example, banks, insurers, other funders, lay people, etc, unless the report has been prepared specifically for that purpose. Advice from other specialists may be required such as the legal, planning and architecture professions, whether specifically recommended in our report or not.

7 Public or legal consultations or enquiries, or consultation with any Regulatory Bodies (such as the Environment Agency, Natural England or Local Authority) have taken place only as part of this work where specifically stated.

FIGURES

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Client

NEWPORT CITY COUNCIL

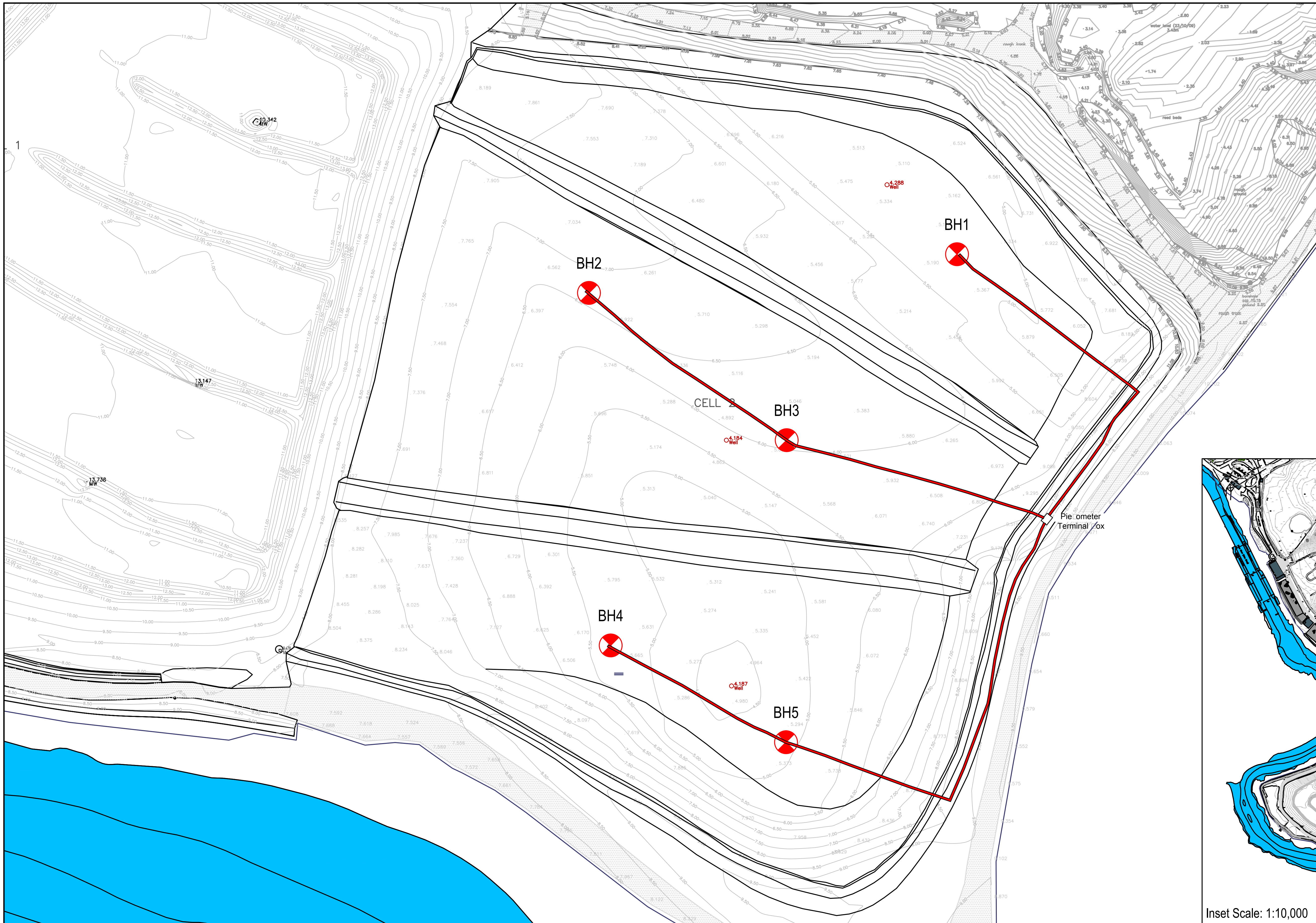
DOCKSWAY DISPOSAL SITE NEWPORT

SITE LOCATION PLAN

Date	17.11.2010
Scale	1:50,000
Drawn by	DJM
Checked by	VKR
Revision	-

FIGURE 1

J:\14739\123 CQA Cell 2 Vibs\001 Geo Brief Name\04 CAD & Graphics\Corel\...



NOTES:
Various coloured tape has been used to identify the Piezometer cables for future reference.

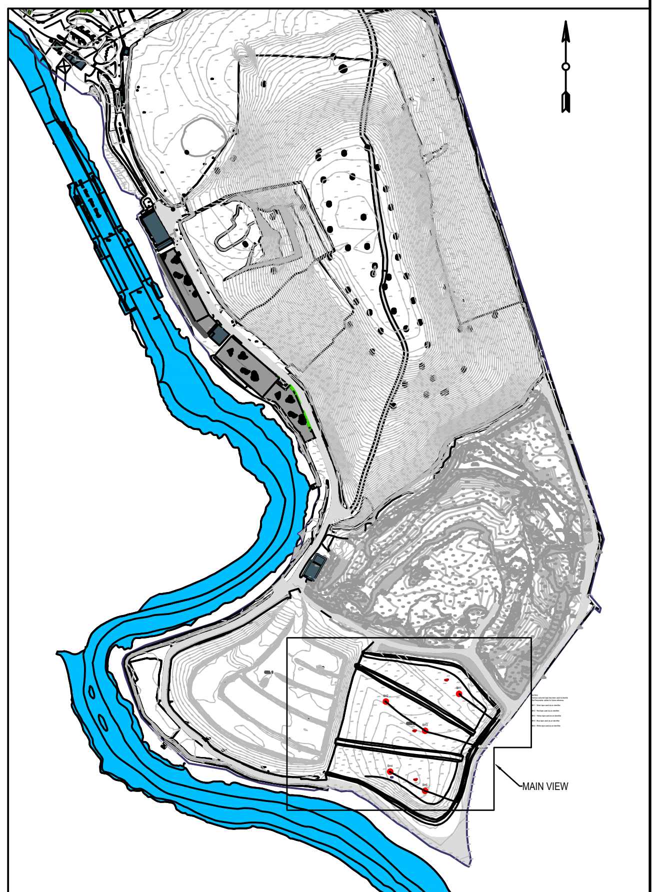
BH1 - Silver tape used as an identifier

BH2 - Red tape used as an identifier

BH3 - Yellow tape used as an identifier

BH4 - Blue tape used as an identifier

BH5 - White tape used as an identifier



Inset Scale: 1:10,000

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DOCKSWAY DISPOSAL SITE
NEWPORT

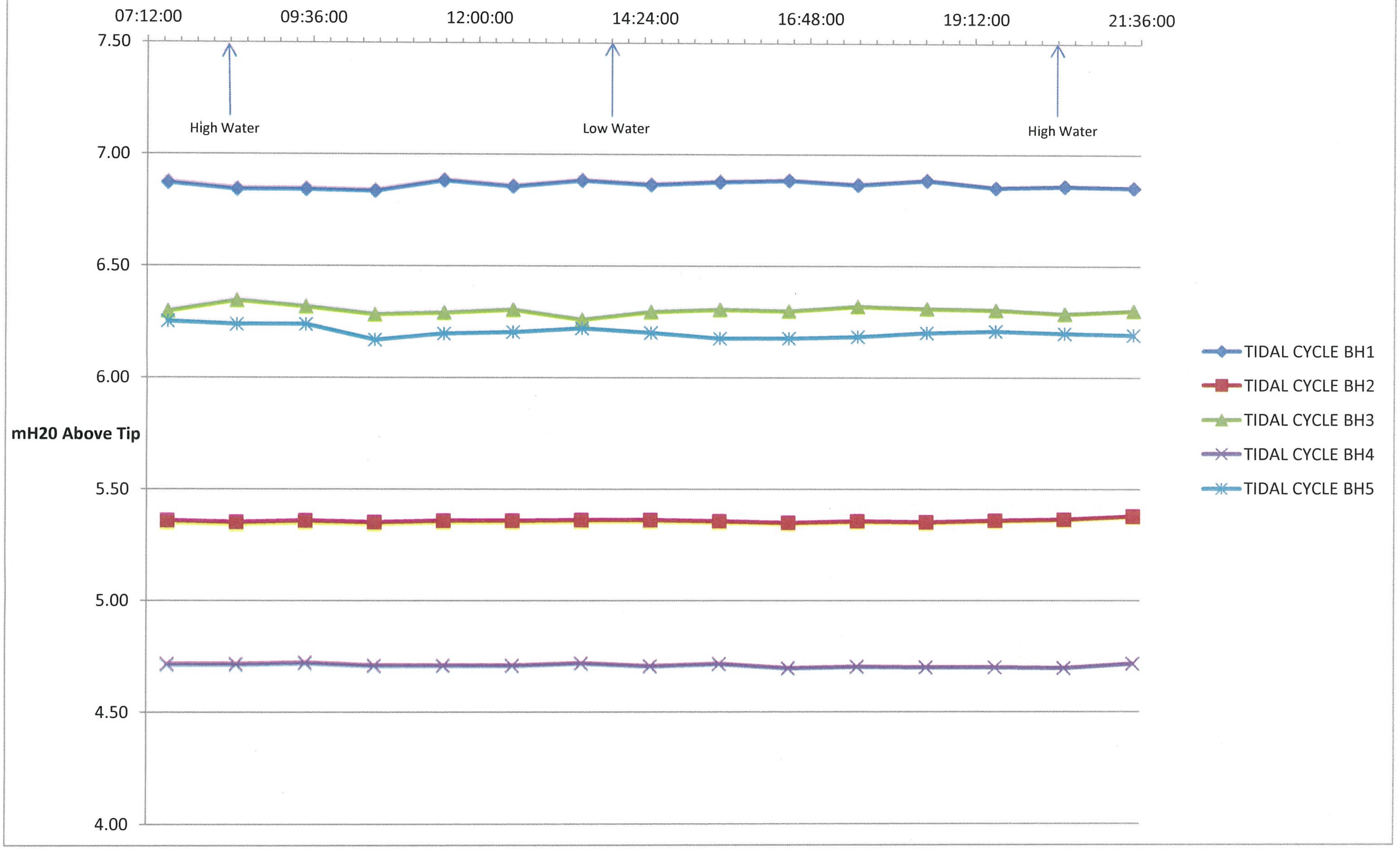
□I□RATING WIRE PIE□OMETERS
LAYOUT PLAN

Date	1□.11.2010
A3 Scale	1:1000 □Main □iew□
Drawn by	D□M
Checked by	□KR
Figure Number	2

user name: david maggs

File Location: j:\14739\123 cqa cell 2 vibs\001 geo brief name\04 cad & graphics\cad\dwgs\geo\figure 2 - piezo layout plan.dwg

Figure 3 - Vibrating Wire Piezometers Tidal Monitoring Chart



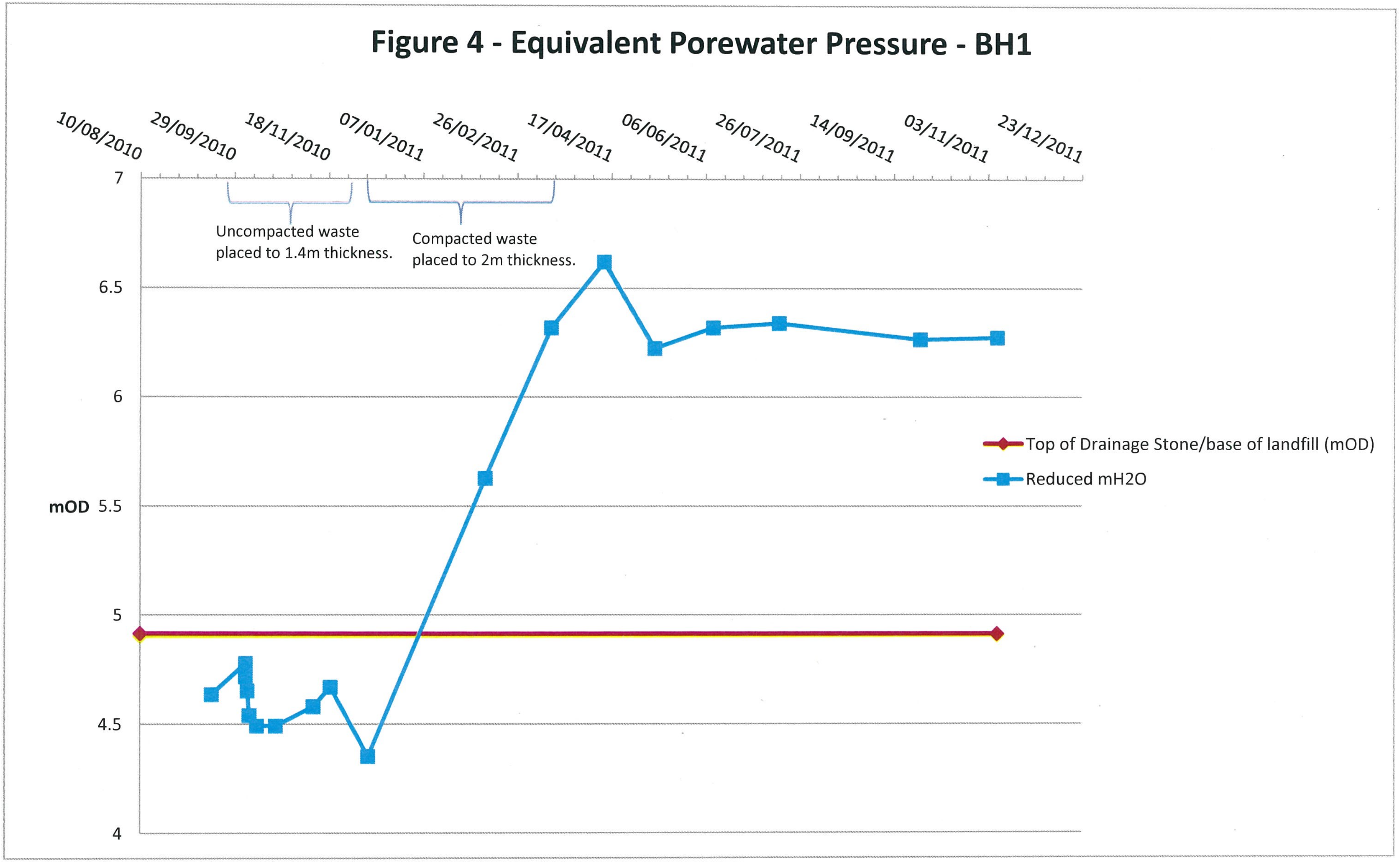


Figure 4 - Equivalent Porewater Pressure - BH1
10/08/2010
29/09/2010
18/11/2010
07/01/2011
26/02/2011
17/04/2011
06/06/2011
26/07/2011
14/09/2011
03/11/2011
23/12/2011
7
6.5
6
mOD 5.5
5
4.5
4
Uncompacted waste placed to 1.4m thickness.
Compacted waste placed to 2m thickness.
Top of Drainage Stone/base of landfill (mOD)
Reduced mH2O

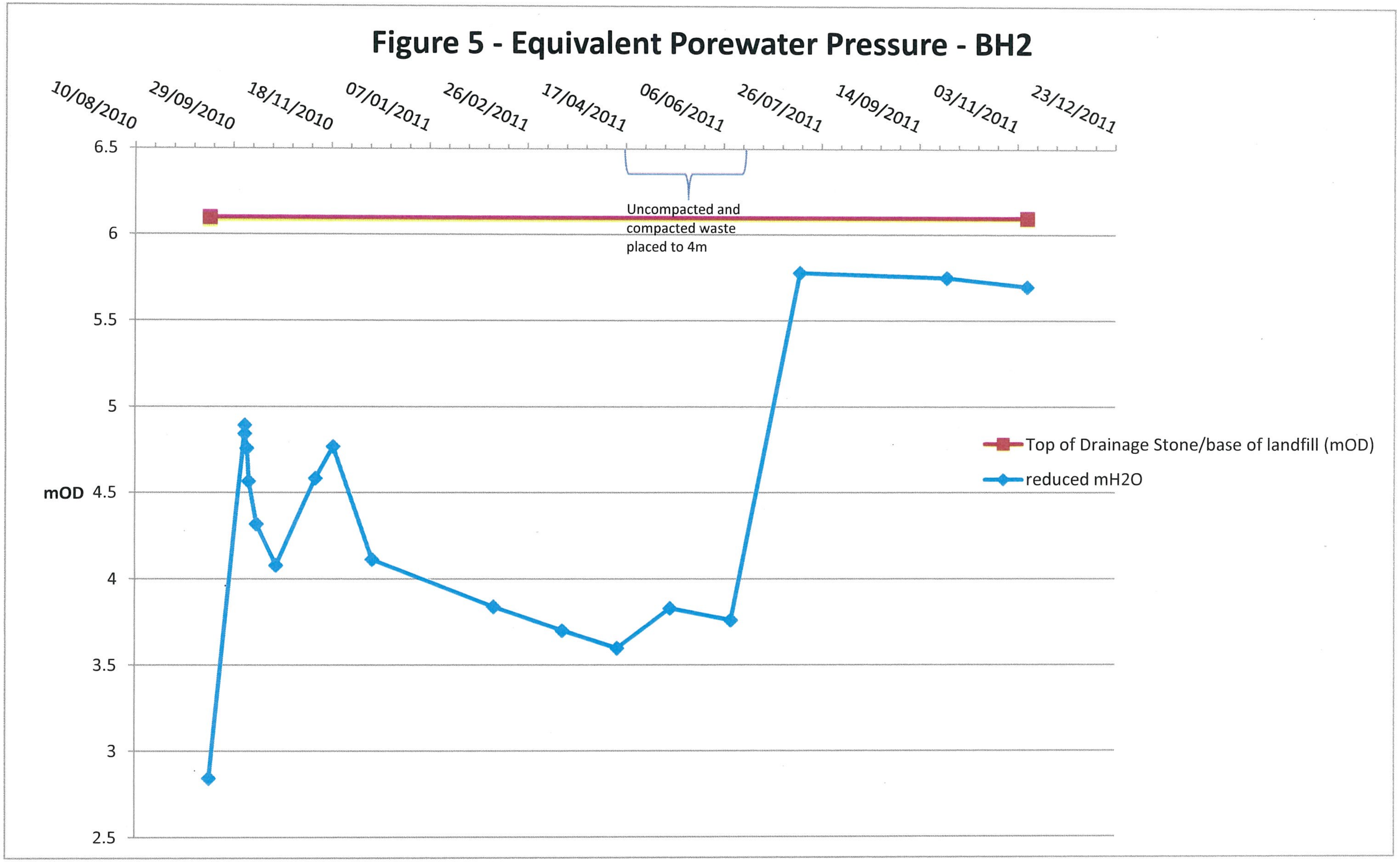


Figure 5 - Equivalent Porewater Pressure - BH2
10/08/2010
29/09/2010
18/11/2010
07/01/2011
26/02/2011
17/04/2011
06/06/2011
26/07/2011
14/09/2011
03/11/2011
23/12/2011
6.5
6
5.5
5
4.5
4
3.5
3
2.5
mOD
Uncompacted and compacted waste placed to 4m
Top of Drainage Stone/base of landfill (mOD)
reduced mH2O

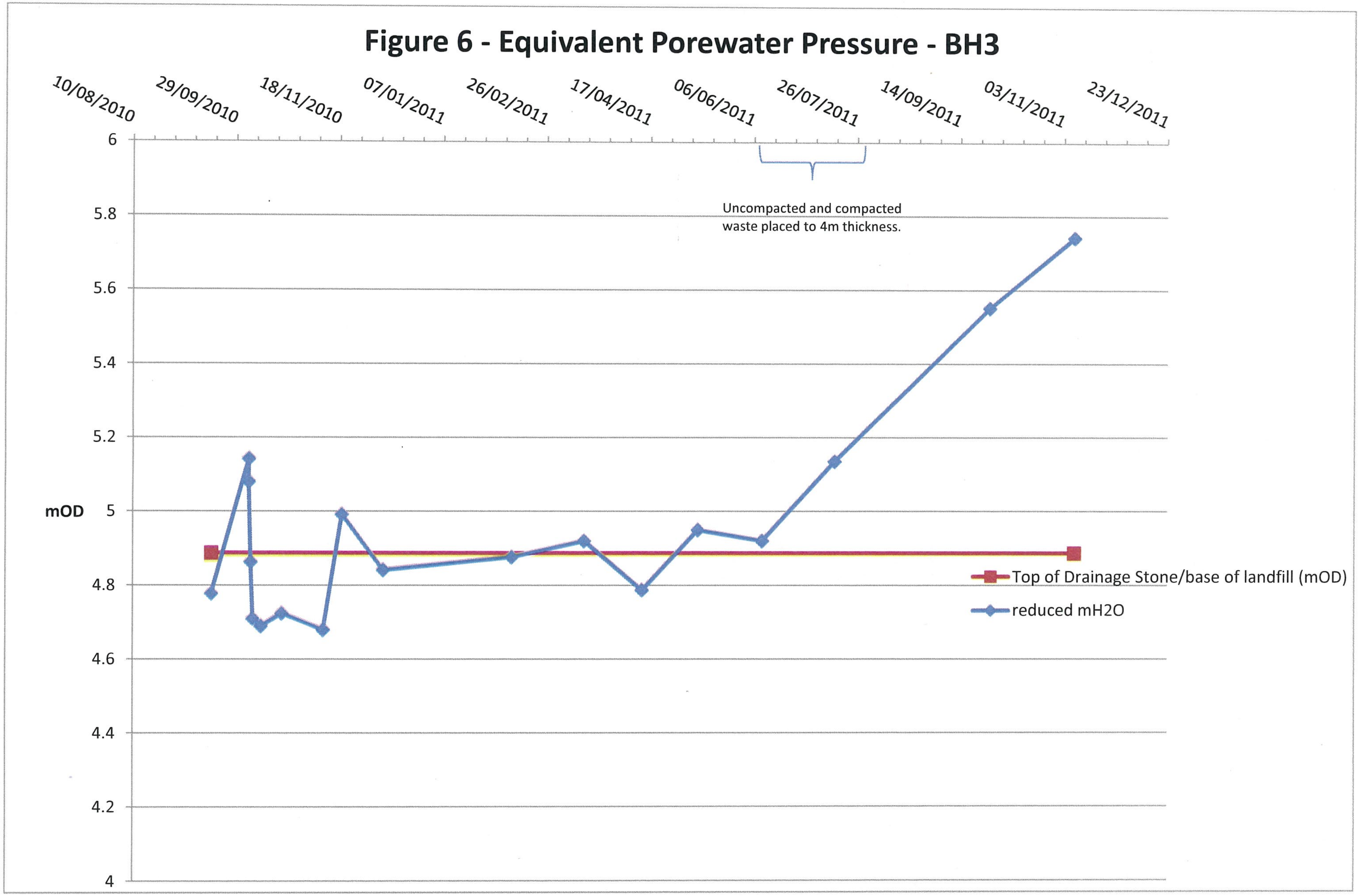


Figure 6 - Equivalent Porewater Pressure - BH3
10/08/2010
29/09/2010
18/11/2010
07/01/2011
26/02/2011
17/04/2011
06/06/2011
26/07/2011
14/09/2011
03/11/2011
23/12/2011
mOD
6
5.8
5.6
5.4
5.2
5
4.8
4.6
4.4
4.2
4
Uncompacted and compacted waste placed to 4m thickness.
Top of Drainage Stone/base of landfill (mOD)
reduced mH2O

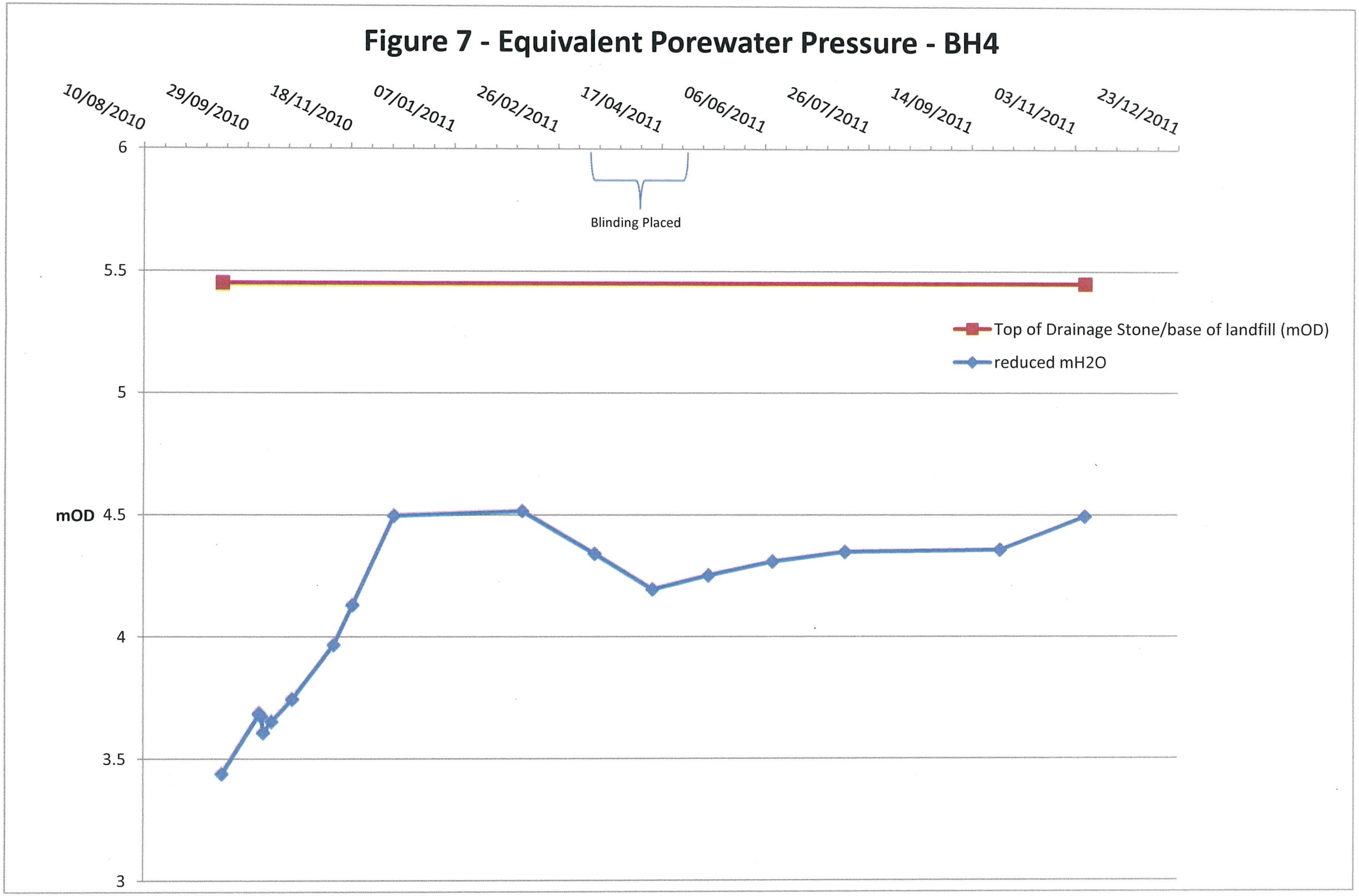


Figure 7 - Equivalent Porewater Pressure - BH4
10/08/2010
29/09/2010
18/11/2010
07/01/2011
26/02/2011
17/04/2011
06/06/2011
26/07/2011
14/09/2011
03/11/2011
23/12/2011
6
5.5
5
mOD 4.5
4
3.5
3
Blinding Placed
Top of Drainage Stone/base of landfill (mOD)
reduced mH2O

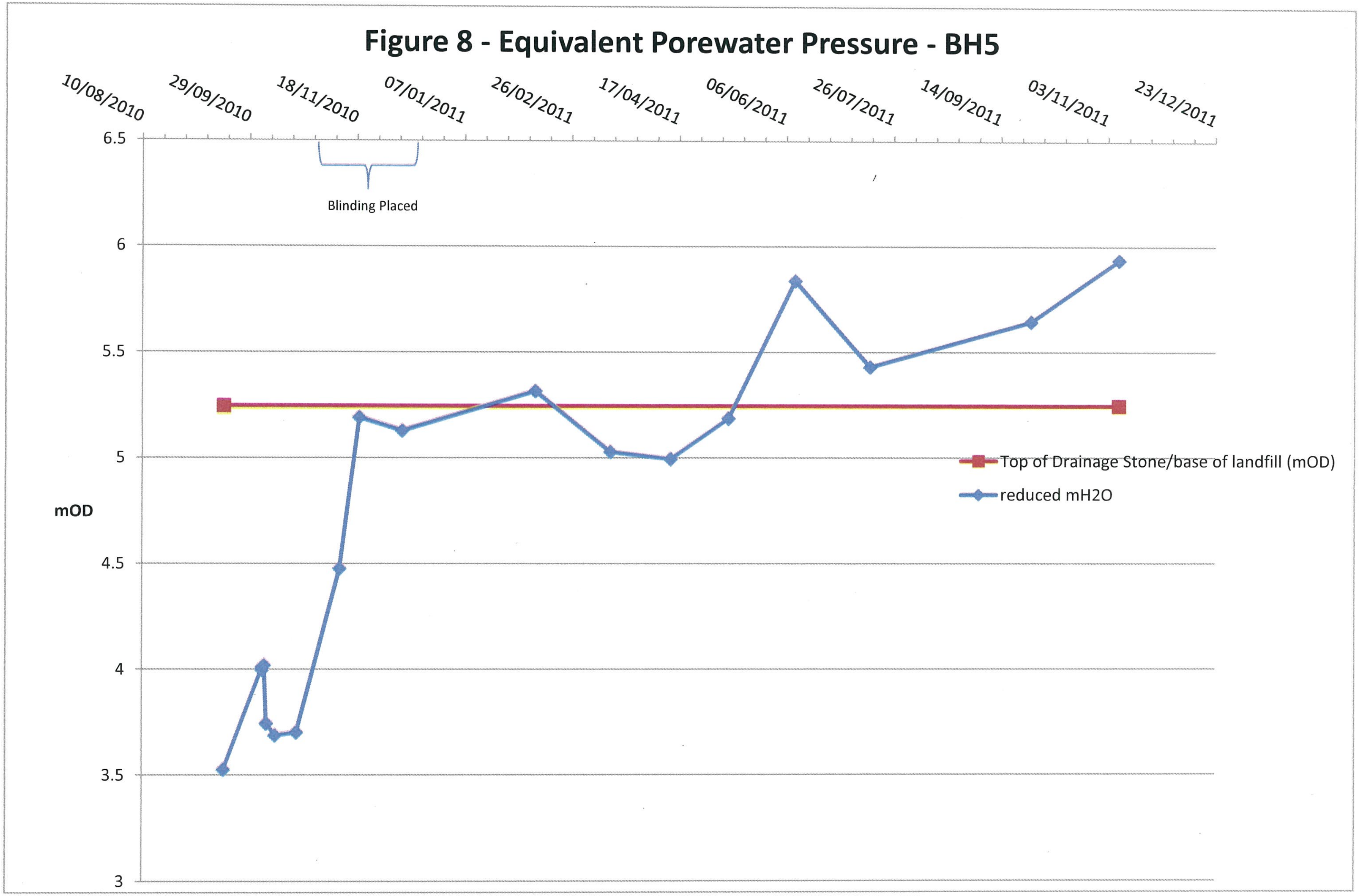


Figure 8 - Equivalent Porewater Pressure - BH5
10/08/2010
29/09/2010
18/11/2010
07/01/2011
26/02/2011
17/04/2011
06/06/2011
26/07/2011
14/09/2011
03/11/2011
23/12/2011
Blinding Placed
6.5
6
5.5
5
4.5
4
3.5
3
mOD
Top of Drainage Stone/base of landfill (mOD)
reduced mH2O

APPENDIX 1

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Soil Instruments Ltd

VIBRATING WIRE/WATER SETTLEMENT CELL USER MANUAL

Man 140	3(A5)	26/01/2007	L.Williams	P.Day	Chris Rasmussen
Manual No.	**Revision**	**Date**	**Originator**	**Checked**	**Authorised for Issue**

Contents

Section 1 : Introduction

This instruction manual describes the techniques required for borehole or trench installation of the water filled vibrating wire settlement cell. The operating principle and method of reading using commercially available vibrating wire readout equipment is briefly discussed. For a complete description of the use of the readout or datalogger used, please see the appropriate manual for that instrument.

It is essential that the equipment covered by this manual should be installed and operated by competent and suitably qualified staff.

The techniques described for installation of the cell are intended to serve as a general guide and will vary to suit particular site conditions.

1.01 General Information

Description

The vibrating wire/water settlement cell is installed at the required measuring horizon and is connected to the monitoring station by an electric cable for the vibrating wire signal and a pair of small bore nylon tubes for the water.

The instrument readout point must built on stable ground beyond the influence of any expected movement or surveyed accurately and regularly to quantify any movement it is suffering. The readout point provides a datum from which all readings are referenced. The cell consists of a vibrating wire pressure transducer with an integral liquid chamber. The twin nylon tubes are filled with water and connected at one end to the water level datum pot and at the other to the transducer liquid chamber.

Vertical movement of the cell relative to the datum pot at the control house results in a corresponding change in liquid pressure at the transducer.

Readings are taken using a vibrating wire readout unit connected to the cable via a suitable terminal unit. Alternatively, for multiple vibrating wire instrument installations cables may be terminated at automatic data logging equipment.

The system is suitable for either trench or borehole installation. The trench type settlement cell is 115mm diameter, 370mm high and mounted on a 300mm square base plate and the borehole type is 155mm diameter, 340mm high, with a left hand threaded connection to receive a special placing adaptor.

The system is reliable, simple to install and to read. Measurements are taken remotely from the readout point and because cells are buried they do not interfere with the construction progress after instrument installation. The liquid tubes allow for re-circulation of the water after installation and for in-situ instrument checks/calibration.

The settlement cell must be installed at a level lower than the liquid datum pot at the monitoring station and the difference in elevation between the two points must be within the cell operating range, with due allowance made for anticipated vertical ground movement.

The reading range and accuracy of the cell is determined by the range of the transducer fitted.

In installations that involve datum pot open to the atmosphere, a correction for atmospheric pressure will be needed. An accurate barometer is therefore necessary for this.

In installations where water lines, transducers or both will be subject to temperature variations, corrections for temperature variation will be required.

Installation Techniques

The installation procedures outlined in this text are intended as a general guide and flexibility in the interpretation of them is required to suit particular local site conditions.

The three main methods of settlement cell placement are trench installation, borehole installation and underwater installation. Each is described separately below.

1.02 Trench Installation

The settlement cell terminal/datum panel should be rigidly fixed to the wall of an instrument house or cabinet located on stable ground beyond the zone of any anticipated ground movement. The cable and water line tubing are cut to length and bound together at intervals along their length using adhesive tape, the colour of which identifies a particular instrument. A trench is excavated from the terminal location to the cell installation point and the cable and twin tubes laid in the trench, connected to the respective fittings within the cell and on the terminal/datum panel, and tested. The water level in the datum pot provides the datum to which all readings can be related. The cell is then placed horizontally on compacted material or, alternatively, cast inside a concrete block, and the trench carefully backfilled and compacted.

(a) Confirm with the Engineer the required installed locations of the cell and terminal point, in both plan and elevation, and ensure that the elevation difference between the two plus the anticipated settlement is within the instrument operating range.

Calculate the tubing and cable lengths required for proposed trench route. Agree and record the master colour code system to provide identification for each settlement cell.

(b) Construct the instrument house or cabinet on stable ground, preferably beyond the influence of any anticipated ground movement and rigidly fix the terminal/datum panel in a vertical position to one wall.

(c) Cut the cable and water line tubing to length. Up to 10% over the exact calculated length should be allowed to accommodate snaking the cable in the trench, ground movements, and cutting the cable back at the instrument and terminal points. Seal both ends of each tubing using adhesive tape to prevent entry of dirt, etc. Working to the previously agreed master colour code, individually identify the instrument by binding the cable and tubing together at intervals with the respective colour of adhesive tape supplied for the purpose. For positive identification at the tubing ends colour code at 1m intervals over the final 3m.

(d) The trench cross-sectional dimensions will be dependent upon such factors as the number of instruments involved and the type of parent material through which the trench will be excavated. Each particular site situation should be individually considered in these terms but with the common denominator that effective protection of the buried tubing is essential.

Excavate the trench from terminal location to cell installation point and ensure that the trench invert is smooth, with no undulations, and of constant gradient along its length. Place a layer of compacted sand or stone-free material in the trench to a depth to provide adequate tubing protection and compact.

(e) With reference to FIGURE 1, connect the cell to the cable and water line tubing as follows. Loosen the two gland nuts (3) to free the cable tail (1). Remove the two screws (4) and part the cell outer case tube (5) from the base plate (8). Withdraw cable tail (1) from inside the cover tube through gland. Remove screws (9) from top of cover tube to release the vibrating wire transducer (6).

Prepare the water line (2) by carefully removing the outer polythene sheathing to expose the two tubes within over sufficient length to reach comfortably from the gland nut (3) to the fittings (7). To prevent kinking ensure that when reassembled the tubes will describe gradual curves over their length within the cell. Connect the tubes each to one compression fitting (7). It does not matter which tube is connected to which fitting. Re-assemble the cell.

(f) The electrical cable and twin water line tubing should now be snaked along the base of the trench and into the instrument house. The trench should not be backfilled until the final zero reading has been established.

(g) With reference to FIGURE 3 connect the water line tubing to the terminal panel, carefully remove the outer polythene sheathing to expose the two tubes over a sufficient length, so that they reach comfortably between the tube ducting and the manifolds. From each settlement cell usually the clear tube is connected to the return manifold so the water can be seen returning through the line.

(h) Fix a permanent length of tubing to the supply manifold, long enough to reach the water circulating unit. Connect the de-aired water boiler, the vacuum pump and the pressure pump to their respective positions on the water circulating unit.

(i) Open the necessary valves and draw de-aired water from the boiler to completely fill the cylinder of the water circulating unit using the vacuum pump. Close all valves. Open the necessary valves and apply pressure to the bladder within the cylinder using the pressure pump. This in turn will pressurise the water in the circulating unit. Close all valves. For full details of how to use the De-aired Water boiler and Water Circulating Unit refer to Sections 3 and 4.

(j) Open the necessary valves to allow water to flow from the cylinder to the water supply manifold, through the respective manifold valve into the water line tubing to the cell, back to water return manifold and into the datum pot. Continue until the datum pot is filled to the overflow level. Close the valve at the top of the water return manifold.

(k) Water should now be circulated through the tubing before the zero reading is taken. A quantity of water equal to 5 times that contained in the tubes should be circulated around the system to comprehensively flush out any air prior to commissioning. A length of tubing should be attached to the return manifold bottom valve and as the water returns through this tube, it should be plunged to the bottom of a clear container partly filled with water. Doing this will allow any air that comes out of the tube to be identified as a bubble that rises from there turn tube. Also, the container can be used to measure the volume of water that has already been circulated.

(l) Zero Readings

A zero reading is now be obtained using the Vibrating Wire Readout/Logger. In the case of a trench installation, the cell should be covered over with a mound of material at least 1 metre high and left for at least two hours before the zero can be taken. This is to ensure the transducer is at a stable temperature and not exposed to the sun or cold winds. A change in temperature of the transducer will cause a change in period reading ,all other variables remaining constant. Before the final zero reading is established, it is preferable to monitor the cell over the course of a few hours to confirm zero period stability.

It is also essential now to accurately record the “base” atmospheric pressure(in cmH20) at the moment the base period is recorded. Most barometers measure the pressure in millibars which can be considered equal to cmH20 to a suitable degree of accuracy.

(m) With a zero reading established, the trench is now be backfilled with stone-free material and compacted.

(n) With the trench backfilled and compacted, check the zero reading.

1.03 Borehole Installation

The settlement cell terminal/datum panel should be rigidly fixed to the wall of an instrument house or cabinet located on stable ground beyond the zone of any anticipated ground movement. If this is not possible, the control house should be levelled regularly to quantify any movement occurring. The cable and water line tubing are cut to length and bound together at intervals along their length using adhesive tape, the colour of which identifies a particular instrument. The twin tubes are connected to the respective fittings within the cell and on the terminal/datum panel, and tested. The water level in the datum pot provides the datum to which all readings can be related. A borehole is formed to the depth at which the cell is to be placed and a trench excavated from the top of the borehole to the terminal location. Grout is placed at the borehole base to form a plug and the cell is pushed into the grout using a placing adaptor and placing tubes. The installing equipment is disconnected from the cell and the borehole backfilled with weak grout or granular material. The cable and tubing are laid in the trench which is carefully backfilled and compacted.

(a) Confirm with the Engineer the required installed locations of the cell and terminal point, in both plan and elevation, and ensure that the elevation difference between the two plus the anticipated settlement is within the instrument operating range.

Calculate the total tubing length required for proposed trench route and borehole depth. Agree and record the master colour code system to provide identification for each settlement cell.

(b) As described in 2(b) - (d) but referring to FIGURE 2.

(c) Form borehole, minimum diameter 150mm to required cell installation depth.

(d) With reference to FIGURE 2, as described in 2(d).

(e) With reference to FIGURE 3, as described in 2(e) - (g).

(f) With reference to FIGURE 2, as described in 2(h).

(g) The borehole depth is governed by the instrument operating range from 0.5m to the range of the vibrating wire transducer fitted and therefore it is necessary to select length of placing tubes required for installation. Connect placing adaptor to placing tube, and tighten. Using a measuring tape and starting from the top of the placing adaptor, mark level corresponding to ground level for the proposed installation depth with a band of coloured adhesive tape (after having taken due allowance for the distance from the placing adaptor to the pneumatic transducer).

(h) Place stiff grout to the borehole base over sufficient depth to cover the cell (minimum 400mm).

(i) The placing adaptor and receiving hole in the cell are fitted with left-hand threads. By turning in an anticlockwise direction screw the placing adaptor and attached placing tube into the cell. Carefully lower down the borehole, assembling placing tubes in the previously marked sequence. Push the cell into the grout plug until the previously measured ground level mark corresponds to actual ground level. Throughout ensure that no tension exists in the cable and tubing. Unscrew the placing adaptor from the cell by rotating the placing tubes in a clockwise direction and remove all placing tubes from the borehole.

(j) Make a final check to ensure cable continuity and water circuits are leak-free, as described above.

(k) Backfill borehole with weak grout or granular material to trench invert.

The procedures described in Section 2 should now be followed in order to carry out the following tasks:

1) Snaking of tubing and electrical lines along the base of the trench.

2) Circulation of de-aired water through the tubing prior to commissioning.

3) Establishment of zero reading.

4) Checking of zero reading after trench backfilled and compacted.

1.04 Underwater Installation

The settlement cell terminal/datum panel should be rigidly fixed to the wall of an instrument house or cabinet located on dry land beyond the zone of any anticipated ground movement. If this is not possible, the control house should be levelled regularly to quantify any movement occurring. The cable and water line tubing are cut to size and bound together at intervals along their length using adhesive tape, the colour of which identifies a particular instrument. The twin tubes are connected to the respective fittings within the cell and on the terminal/datum panel, and tested. The water level in the datum pot provides the datum to which all readings can be related. Cell installation is by lowering from a boat or direct hand-placement by divers. Finally, the cell cable and tubes (along their submerged length) are carefully anchored to the sea/river bed until fill is hydraulically placed to hold them permanently in position.

It is not realistic to attempt to state a precise underwater installation procedure due to the considerable variety of possible site conditions that influence the choice of method. However, the main points are discussed below and should be included in any adapted procedure used to suit the conditions of a particular site.

(a) Confirm with the Engineer the required installed locations of the cell and terminal point, in both plan and elevation, and ensure that the elevation difference between the two plus the anticipated settlement is within the instrument operating range.

Calculate the cable and tubing lengths required for the proposed route. Agree and record the master colour code system to provide identification for each settlement cell.

(b) Construct the instrument house or cabinet on dry land beyond the zone of any anticipated ground movement and rigidly fix the terminal/datum panel in a vertical position to one wall so that the future reduced level of mercury within the datum pots will be as specified by the Engineer.

(c) As described in 2(c).

(d) As described in 2(e) - (j).

(e) Install the cell. If there are soft sediments on the sea/river bed they should be removed by dredging down to a more competent foundation material before commencing instrument installation. While it is possible to install the cell remotely by lowering from a boat it is more convenient and thorough, and therefore recommended, to use a diver for direct hand placement.

(f) Snake the cable and tubing and carefully place stone-free material around and above to such a depth as to provide adequate protection. (The degree of snaking will be dependent upon the magnitude of the expected ground movement. Where the cable and tubing crosses the interface between two dissimilar fill materials, across which differential settlement is expected, it is important that the degree of snaking should be locally increased.) With reference to 2(d) and (m), the cable and tubing should be trenched up the shore to the monitoring point so that exposure to wave action is kept to a minimum.

(g) De-aired water can now be circulated and zero readings established as described in 2(k) and 2(l).

1.05 De-Aired Water Boiler

Refer to FIGURE 4.

Up to 50 litres of de-aired water can be prepared in the boiler at one filling. An immersion heater coil heats the water to boiling thus expelling any air or gases in solution. By sealing the boiler and allowing it to cool, the steam and water vapour above the water condense thus creating a partial vacuum inside the boiler. This prevents further solution of air or gases.

(a) Remove the central filler-cap using the special spanner provided. Open valves V and S.

(b) Pour in clean water to within 100mm of the top.

(c) Connect the electrical cable to a suitable mains supply (either 110V or 230V as indicated on the immersion heater coil cover). Switch on at the mains. Bring to the boil and allow to boil for fifteen minutes. Switch off at the mains supply and disconnect plug from wall socket.

(d) Close Valves V and S and replace the central filler-cap. Allow to cool.

WARNING: - Although the cap is fitted with a pressure relief valve, never boil with the filler-cap on.

NOTE: - With a partial vacuum inside the boiler, never open valve S before opening valve V, as this will draw air into the de-aired water.

SAFETY REQUIREMENTS: - The 4 psi Pressure Relief Valve in the centre of filler-cap must be checked at intervals not exceeding 12 months by a competent person.

Section 2 : Water Circulating Unit

Refer to FIGURE 3.

Operating instructions for drawing de-aired water from the boiler and filling water-line tubing and settlement cells are as follows:-

(a) Close all valves.

(b) Attach vacuum pump to valve B with black hose, open valve B and A, remove air from inside bladder.

(c) Close valve B and remove hose and fit on valve A.

(d) Fit red hose to valve D and place end of hose in water. Pump vacuum pump to draw water into circulating unit. Avoid drawing air.

(e) Close valve D and A. Fit foot pump to valve E and attach nylon tube to valve C and terminal panel left-hand side lower valve.

(f) Open valve C of water circulating unit. Open the bottom valves on the left and right valve panels - these are the inlet and outlet valves for the circulating water. Now open inlet and outlet valves for the cell to be de-aired. All other valves should be closed.

(g) Make sure top valves leading to the datum pot are closed. Start to pump water with the foot pump. Do not go above 2.5 bar or the range of the cell, whichever is the least. During circulation, the outlet valve should have a tube attached (which can be permanently left in place) which should be plunged to the bottom of a clear container partly water filled. Any air expelled from the tubes will appear as a bubble which rises from the tube.

(h) Repeat steps (a) to (g) as necessary to de-air/circulate for all cells.

Section 3 : Cable Jointing

Refer to FIGURES 5 and 6.

It is desirable to minimise cable joints, but where they are unavoidable a joint kit may be supplied. The effectiveness of this joint largely depends on the care with which the jointing operation is carried out.

(a) Thoroughly scrape all wax and dirt from cable end for approximately 150mm. Prepare the cable ends as shown. Stagger the individual conductor connections.

(b) Use the crimped connectors to join the conductors. Ensure electrical continuity of outer armoured screen is re-established across joint. Use the electrical insulation tape to wrap the connectors. Stretch the tape to half its original width and apply one layer half lapped over connector area only. Do not wrap the tape beyond the pencilled area.

(c) Trim the ends of the mould with a sharp knife to suit the diameter of the cable. Hold the mould halves in place centred over the splice. Snap both halves together and fit the pouring spouts in the holds. Ensure that both seams are completely snapped together. Tape the ends of the mould body to form a seal.

(d) Mix the resin thoroughly and maintaining the mould in a level position, spouts uppermost, pour the resin through one spout until both spouts are completely filled. When the resin has solidified and cooled remove the spouts.

N.B. In cold weather (below 15 degrees C) the resin becomes very viscous. It is therefore advisable to keep the resin in a warm place prior to mixing. Mix the compound until its temperature starts to rise, this decreases the viscosity.

Section 4 : Cable Termination

The cables are normally terminated in terminal boxes. The cables enter through waterproof glands.

(a) Unscrew the cover or open the lid as appropriate. Unscrew the fixing screws holding the terminal panel and carefully remove it without straining the internal connecting leads.

(b) Prepare the cables by stripping and cutting back 20mm approx. of the outer insulation and armour. Remove the rubber packing and strip back 5mm of the conductor insulation.

(c) Slacken the entry glands and insert the cables. Make connections to the contact block (details of colour coding supplied with each instrument). Retighten the glands to grip the cables.

(d) Replace the terminal panel and secure. Connect the Vibrating Wire Readout/Logger to each instrument in turn to check connections

Section 5 : Data Reduction

FIGURE 7 shows an example of the vibrating wire instrument calibration certificate as supplied with each vibrating wire instrument. The form is designed to provide complete site record facilities for a single instrument and its application is self explanatory.

5.01 Factors Affecting Reading Accuracy

The reading accuracy of vibrating wire/water settlement cells is influenced by the effect of temperature changes on water density. Where large seasonal or daily temperature fluctuations occur then due arithmetic allowance of this must be made when calculating elevation differences between the datum level and the installed settlement cell.

Consider the installed cell. It is connected to the remote monitoring point by the twin tubing, containing the continuous water column, with the tubing installed within trenches over their entire length to the terminal point. In all but exceptional circumstances the soil temperature will remain constant at the depth of the trenched tubing and therefore the temperature of the water contained within will also remain constant. There will be minimal reading errors. However, for maximum reading accuracy attention should be given to that portion of the tubing length which is exposed to ambient air temperature at the monitoring station, particularly when no means of temperature control is provided (eg. heating or air-conditioning). The worst effects will arise from large vertical heads subject to large temperature variations.

The effect of temperature on reading accuracy becomes more significant with the underwater installations required during causeway construction or land reclamation, for example. For shallow and sheltered water, the water temperature will vary considerably with weather or season, causing corresponding temperature fluctuations in the water column contained within the twin tubes lying on the bottom. Changes in temperature will alter the water density and consequently the pressure exerted on the diaphragm of the cell's transducer. Additionally in such situations the monitoring station is often constructed on a platform founded on piles extending above maximum water level. Normally it is not possible to equip the monitoring station with any form of temperature control. That part of the twin tubing length above water level will be subjected to ambient air temperature. Where single skin metal terminal enclosures are used or the tubing is exposed to direct sunlight, temperatures considerably higher than ambient shade temperatures can occur. The combined effect of the three factors on reading accuracy can be considerable.

The correction method required to obtain accurate data under the conditions described above is outlined in the Specimen Calculation below. It is important that no part of the twin tubing is exposed to direct sunlight. Tubing leaving the monitoring station platform to the installed instrument should be positioned within the natural shadow of the platform. It may be necessary to place the tubing within a freely-ventilated cover to provide a continuous shade. Also, to determine air, water and soil temperatures it is advisable to install a screened, perforated access tube vertically through the monitoring station floor to the lake or sea bed soil beneath. Temperature readings can then be taken at various depths as required.

Experience has also shown that to obtain accurate data it is absolutely essential that no part of the instrument system is allowed to move or vibrate during the reading operation. To eliminate this possibility, two precautions are necessary. Firstly, connecting tubing from the installed instrument to the monitoring point should be firmly anchored throughout its length. The most critical areas are where the tubing is subjected to wave and wind action - at and immediately above the water surface. Secondly, the monitoring station itself should be sufficiently strong to resist movement induced by wave and wind. If this is not possible then readings should only be taken when there is no such movement.

Water purity may sometimes be important. At 4°C the density of fresh water relative to distilled water is 1.0011. This represents a reading error of approximately 2cm on a 20m water head. However, as the water used in a practical installation is likely to be derived from the same source throughout its useful life, such an error will be constant and would therefore not affect the accuracy of reading differences, i.e. settlement. This assumption is valid when very small proportions of water treatment chemicals are present - it may not be valid in some tropical areas where chemical additives vary in both quantity and quality. Advice should be sought from the local water authority or, failing this, only distilled water used throughout the instrument system.

Finally, it may also be useful to take readings at the same time each day. Direct sunlight effects are minimal in the early morning. Also, much of the overnight temperature, which is relatively stable and predictable, will remain within that portion of the water column subject to ambient air temperature variations, thus reducing the possibility of significant temperature gradients in that portion of the tubing assumed to be at the same temperatures.

5.02 Magnitude of Errors

To illustrate this, refer to FIGURE 8. The graph shows the effect of water temperature variations on a 20m high water column related to a readout device assumed to be totally accurate over the considered temperature range (0 to 40°C).

Two important points are evident. Over the range of 0 to 10°C errors due to temperature variation are minimal and can normally be ignored.
The effect is considerable over the range 10 to 40°C and must be compensated for if meaningful data are to be obtained.

The graph tends to oversimplify the problem since it assumes that the entire 20m water column is subjected to the same temperature at any given time. This assumption is not valid if the water column is subjected to the differing soil, water and air temperature along its length.

Consider the case of an underwater installation as described earlier. Both the air temperature and the surrounding water temperature will affect the water within the tubing. The correction to the data must be applied in two parts corresponding to the effect of each particular temperature on the respective length of water column over which it has direct influence.

5.03 Specimen Calculation

Refer to FIGURE 8.

Total water head subject to air temperature = 4m.

Total water head subjected to water temperatures = 16m.

Total water head = 20m.

Mean air temperature = 25°C (Taken insider perforated/screened tube).

Mean water temperature = 21°C.

With reference to the graph and relating all corrections to 4°C.

Error due to air temperature = $\frac{\text{4m (column in the air) x (Error at 25°C)}}{\text{20m (test column)}}$

= 0.2 x 60

= 12mm

Error due to water temperature = $\frac{\text{16m (column in water) x (Error at 21°C)}}{\text{20m (test column)}}$

= 0.8 x 40

= 32mm

Total error on 20m head = 12 + 32

= 44mm

Therefore 44mm must be added to the reading to standardise at 4°C, i.e. to read the same head of water at 4°C you would need 44mm more water head added because the column would shrink at the lower temperature.

5.04 Correction for Atmospheric Pressure

Vibrating Wire Water Settlement Cells have a water datum pot open to the atmosphere. This means that they measure the following pressure:

Pressure Measured = Head of Water in twin tubing + Atmospheric Pressure acting on Datum Pot

Atmospheric pressure can easily change from 980 millibars to 1020 millibars over a 24 hour period. 1 millibar is approximately equal to 1 cmH20. Therefore a change in atmospheric pressure from 980 to 1020 millibars would register on the settlement cell as a 40cm settlement.

It can therefore be seen that a correction for atmospheric pressure is essential when using Vibrating Wire/Water Settlement Cells. It is recommended that for this purpose, two barometers be made available on site. Both barometers need to be accurate to 1 millibar (1 cmH20). Prior to using the barometers to derive correction factors, they should each be monitored for a few days to ensure that they give the same magnitude of pressure changes. (They need not both be giving the correct absolute atmospheric pressure or even both reading equal values).

Atmospheric pressure corrections are made in the following way:

1) Once each cell has been backfilled over and water tubes filled with de-aired water, a "BASE ATMOSPHERIC PRESSURE" (Bp) should be recorded at the same time the "BASE PERIOD" is measured.

2) One barometer (preferably the most rugged) is carried around the site and an atmospheric pressure reading taken in millibars (cmH20) at the same time a settlement reading is taken for each cell. Let us call this pressure "Mp".

3) The other barometer is kept in an "impact free" environment such as on the wal lin the site office. The "site barometer" should be checked against this "office barometer" both before and after site visits and at random intervals in between to ensure that both are measuring equal pressure changes.

4) Atmospheric Pressure Correction Factors are determined in the following fashion:

Correction (cm) = [Bp - Mp]

Where:

Bp = Base Atmospheric Pressure (in millibars or cmH20).

Mp = Measured Atmospheric Pressure at time of reading (in millibars or cmH20).

5) This correction is added onto the reading obtained from the vibrating wire readout/logger. Using a positive "K" factor in the logger, leads to settlement causing a positive increase in value on the logger. Similarly, an increase in atmospheric pressure causes an "apparent settlement" of the cell. For this reason, it can be seen that pressures higher than Base Pressure give rise to negative ("heave") corrections.

Section 6 : Taking Readings and Monitoring

6.01 Taking Readings - Theory

Vibrating Wire Settlement Cells can be read with Soil Instruments Vibrating Wire Readout/Logger or any other readout capable of reading a vibrating wire transducer. The wire of the sensor is excited to oscillate at its resonant frequency which is then accurately measured by counting the number of oscillation cycles. The reading can be obtained from the readout/logger as raw data in terms of normal reading (Period x 107) or linear reading ($f^2/1000$). If a readout/logger channel is dedicated for one particular instrument and a Channel Table set up (refer to logger user's manual) for that channel, the readings can be obtained and logged/stored in the readout/logger memory directly in cmH20. It is extremely important to establish a reference reading (or "base" reading) at the time of installation and initial setting up. Store this reading for zero displacement with gauge factor for a particular cell. The logger will display positive displacement for increasing settlement if a positive gauge factor is inputted into the logger.

6.02 Taking Readings - Correct Procedure

Following the procedure given below should mean random errors are avoided when taking measurements of the cells:

1) Select correct channel on readout/logger.

2) Select correct number on dial of terminal panel.

3) Close all valves of instruments NOT being used.

4) Check water level in datum pot - if low, top up with de-aired water.

5) During reading, the logger should be allowed to "ping" at least 3 or 4 times on each instrument to ensure stability.

6) Record Atmospheric Pressure at time of reading.

6.03 Frequency of Readings

The frequency with which readings of the Vibrating Wire/Water Settlement Cell are taken will depend on the stage of construction and the behaviour of the structure after completion. The following points are relevant:

1) Immediately after installation it is a good idea to take readings as frequently

as twice daily for a few days to confirm zero readings.

2) During the general construction phase of the project, a normal reading frequency would be once or twice a week.

3) After completion of the structure, the frequency with which readings are taken may be set at every two weeks or even less. If an increased rate of change in the readings is observed, then it may be necessary to increase the frequency with which readings are taken in order to properly record any movement taking place.

Section 7 : Data Interpretation

7.01 Calculation of Engineering units from frequency-based units.

The mathematical relationship between the frequency of vibration of a tensioned wire and the force applying the tension, is an approximate straight line relationship between the square of the measured frequency and the applied force.

Engineering units of measurement maybe derived from the frequency-based units measured by vibrating wire readouts, in 3 traditional ways:-

From 'Period' units and from 'Linear'(f^2/1000) units using two methods: a simple Linear equation or a Polynomial equation.

Calculation using 'Period' units.

The following formula is used for readings in 'Period' units.

$$E = K\,(10^7/P0^2 - 10^7/P1^2)$$

Where,

E is the Pressure in resultant Engineering units,

K is the Period Gauge Factor for units of calibration (from the calibration sheet), P0 is the Period 'base' or 'zero' reading

P1 is the current Period reading.

This method of calculation is used by the Soil Instruments Vibrating Wire loggers' (models RO-1-VW-1 or 2 and with serial numbers starting VL or TVL) internal processors', for calculating and displaying directly on the loggers' LCD screen, the required Engineering based units.

The loggers' require 'Period' base or zero reading units for entering into their channel tables, to calculate and display correctly the required engineering units.

If an Engineering-based unit is required other than the units of calibration, then the correct K factor will have to be calculated using the standard relationship between Engineering units.

For example, if the units of calculation required were in mH2O and the calibration units were kPa, we can find out that 1kPa is equal to 0.1022mH2O, so we would derive the K factor for mH2O by multiplying the K factor for kPa by 0.1022.

Please see conversion factors in Figure 10

Calculation using Linear units.

The following formula is used for readings in 'Linear' units.

$E = G (R0 – R1)$

Where,

E is the resultant Engineering unit,

G the linear Gauge factor for the units of calibartion (from the calibration sheet), R0 is the Linear 'base' or 'zero' reading

R1 is the current Linear reading.

Again the Linear gauge factor for units other than the units of calibration would need to be calculated using the same principles as stated in the last paragraph of the 'Period unit' section.

Linear unit calculation using a Polynomial equation.

Linear units maybe applied to the following polynomial equation, for calculation of Engineering units to a higher order of accuracy.

E = AR1^2 + BR1 + C

Where,

E is the resultant Engineering unit,

A, B and C the Polynomial Gauge factors A, B and C, from the instrument's calibration sheet,

R1 is the current Linear reading.

The value C is an offset value and relates to the atmospheric pressure experienced by the piezometer at the time of calibration. This figure will have changed at the time of installation due to changes in altitude or barometric pressure, so C should be re-calculated at the installation time as follows:

C = - (AR0^2 + BR0)

Where,

A and B are as above,

R0 is the Linear 'base' or 'zero' reading.

Please note that the sign of the re-calculated value of C, should be the same as the original value of C, so if the original is negative then the recalculated value should also be negative.

Conversion to engineering units other than the units of calibration, would best be done after conversion, using a factor calculated using the same principles as stated in the last paragraph of the 'Period unit' section.

Please see conversion factors in Figure 10

7.02 Use of Temperature Correction coefficients.

Soil instruments Ltd's vibrating wire transducers have been carefully developed over many years, so that the combination of material types in the instruments' build, its design and the manufacturing processes, results in instruments which are little affected by changes in temperature. Any changes are regarded as insignificant.

Therefore, as standard, Soil Instruments Ltd's vibrating wire instruments are not provided with temperature correction coefficients, even though they may be specified with a thermistor.

It should be noted that while an instrument is changing temperature, the stresses created across the transducer body by temperature gradients, will induce changes in the instruments reading, but once the instrument has stabilised at its new temperature, these differences will disappear.

The time for an instrument to fully stabilise at a new temperature will be in the order of 30 to 40 minutes.

At the time of ordering, instruments may be specified, calibrated with individual temperature correction coefficients.

If supplied with this coefficient, then the coefficient may be applied using the following formulae and example:

$$ET = E + TK\,(T1-T0)$$

Where,

ET is the temperature corrected Engineering reading,

TK the temperature coefficient,

T1 the current instrument temperature and

T0 the base or zero reading temperature (both in degrees C and recorded from the instruments internal thermistor).

The temperature coefficient, TK, will be in units of Engineering per degree c, as per the calibration certificate.

Ensure that the temperature coefficient is converted to the same units of Engineering as per the calculated E, via a conversion factor before applying the temperature correction.

Section 8 : Troubleshooting Guide

8.01 The Vibrating Wire Transducer

If a failure of any vibrating wire transducer or the electrical cable is suspected, the following steps can be followed. The transducers themselves are sealed and cannot be opened for inspection. The "Troubleshooting Flowchart" in FIGURE 9 should also be followed if any instrument failures are suspected.

The steps below and the Troubleshooting Flowchart are applicable generally to any vibrating wire instrument.

STEP 1

Before any of the following steps are followed, the readout unit should be used to verify the stability of the reading and the audio signal from the portable logger should be heard. An unstable (wildly fluctuating) reading from a transducer or an unsteady audio signal are both indications of possible problems with instruments or their related electrical cables.

* Important: If a portable data logger is giving faulty readings or audio signals from all transducers, a faulty readout unit must be suspected. Another readout unit should be used to check the readings from the transducers and Soil Instruments Ltd. should be consulted about the faulty readout unit.

STEP 2

The resistance across the two conductors of the electrical cable should be checked. This can be done using a multimeter device across the two exposed conductors if the cable has not been connected to a terminal cabinet, or can be done just as easily across the two conductors if the instrument has been connected to such a terminal (or data loggers).

The resistance across the two conductors should be approximately of the order of 120Ω to 180Ω. The majority of this resistance will come from the transducer (say approximately 130Ω) and the remainder from the electrical cable connected to the transducer (for 22 gauge copper), resistance is approximately 1Ω/15m.

Before proceeding to Steps 2 and 3, the continuity should be checked between conductors and earthing screen of the electrical cable. If continuity exists, a damaged cable is confirmed.

STEP 3

If the resistance across the two conductors is much higher than the values quoted in "STEP 1" (or is infinite), a severed cable must be suspected.

If the location on site of cable damage is found, the cable can be repaired using a joint sealing kit (or "splice").

STEP 4

If the resistance across the two conductors is much lower than the values quoted in "STEP 1" (say 80Ω or less) it is likely that cable damage has occurred causing a short in the circuit.

It is possible to calculate approximately how far from the cable end (or readout location) the suspected circuit break is. If the resistance of a known length of conducting cable is measured, a resistance/unit length can be found. This figure can be used to calculate the length of conductor cable in between readout location and the "short". It must be remembered that this method is only applicable if the "short" occurs between the two conductors of the electrical cable. Since cables are generally buried and hidden it is usually impossible to confirm the short is of this nature. It should therefore be remembered this method can be used as a rough guide only.

STEP 5

If the resistance is within the values quoted in "STEP 1" (i.e. 120Ω to 180Ω), AND no continuity exists between conductor and earth screen and on checking the reading from the transducer, it proves to be still unstable or wildly fluctuating, it must be assumed that the integrity of the circuit is good. A faulty transducer must be suspected and Soil Instruments Ltd. should be consulted.

8.01 Troubleshooting with Factors other than the transducer

The following are a few symptoms of common errors that can occur with Vibrating Wire/ Water Settlement Cells. Their most common cause is given along with suggestions on how to avoid the problem in the future.

1) Symptom: "Large", simultaneous, and equal magnitude heaves and settlements noticed from reading to reading for every cell on site. These would take the form of common peaks and troughs on settlement plots.

   Possible Causes: Incorrect barometer calibration. check the two barometers against each other. If errors found, change barometer(s).

2) Symptom: "Unrealistic" but equal heave of all the instruments connected into the same

datum panel.

Possible Causes:

a) Drop in water level in datum pot. Top up with water.

b) Settlement of the instrument house. Use survey techniques to monitor the level of the house and correct for any movement observed.

3) Symptom: Large random errors in readings on individual cells.

Possible Causes:

a) Choosing wrong dial number on terminal panel.
b) Choosing wrong channel on vibrating wire logger.
c) Opening wrong valves on terminal panel.
d) Channel table errors - check set up of channel table in logger memory.

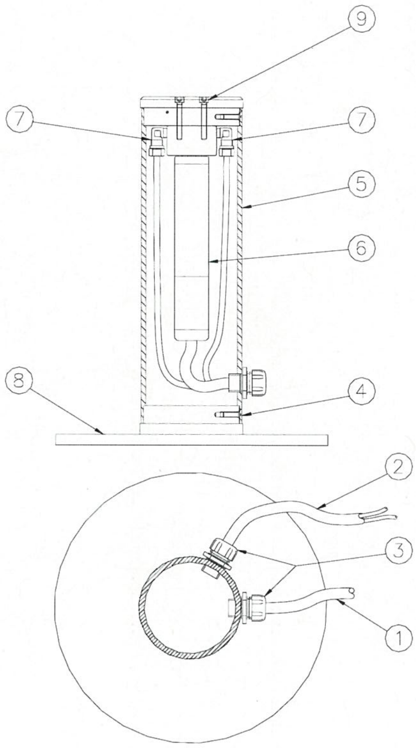
4) Symptom: Long term "unrealistic" heave of individual cells.

Possible Causes: Air creeping into the water filled tubes. Circulate de-aired water around the tubing again.

5) Symptom: Long term "unrealistic" heave of all cells on site.

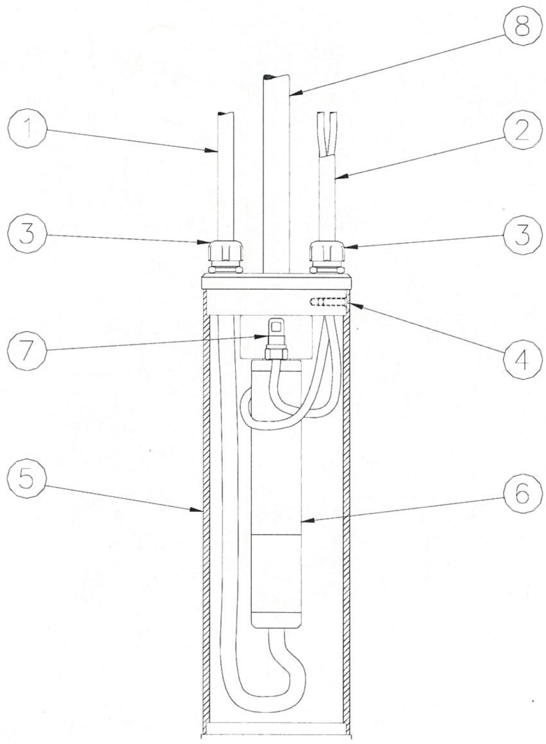
Possible Causes: If surveying techniques are being used to correct for settlement of the instrument sheds, possible settlement of the main survey bench mark.

Figure 1 - Trench Type Cell



(1) - Cable
(2) - Twin Nylon Tubes for water
(3) - Gland Nuts for cable and tubes
(4) - Screws
(5) - Cover Tube
(6) - Vibrating Wire Transducer
(7) - Fitting for water tubes
(8) - Base Plate
(9) - Screws

Figure 2 - Borehole Type Cell



(1) - Cable
(2) - Twin Nylon Tubes for water
(3) - Gland Nuts for cable and tubes
(4) - Screws
(5) - Cover Tube
(6) - Vibrating Wire Transducer
(7) - Fitting for water tubes
(8) - Placing adaptor

Figure 3 - Vibrating Wire Settlement Cell System Installation

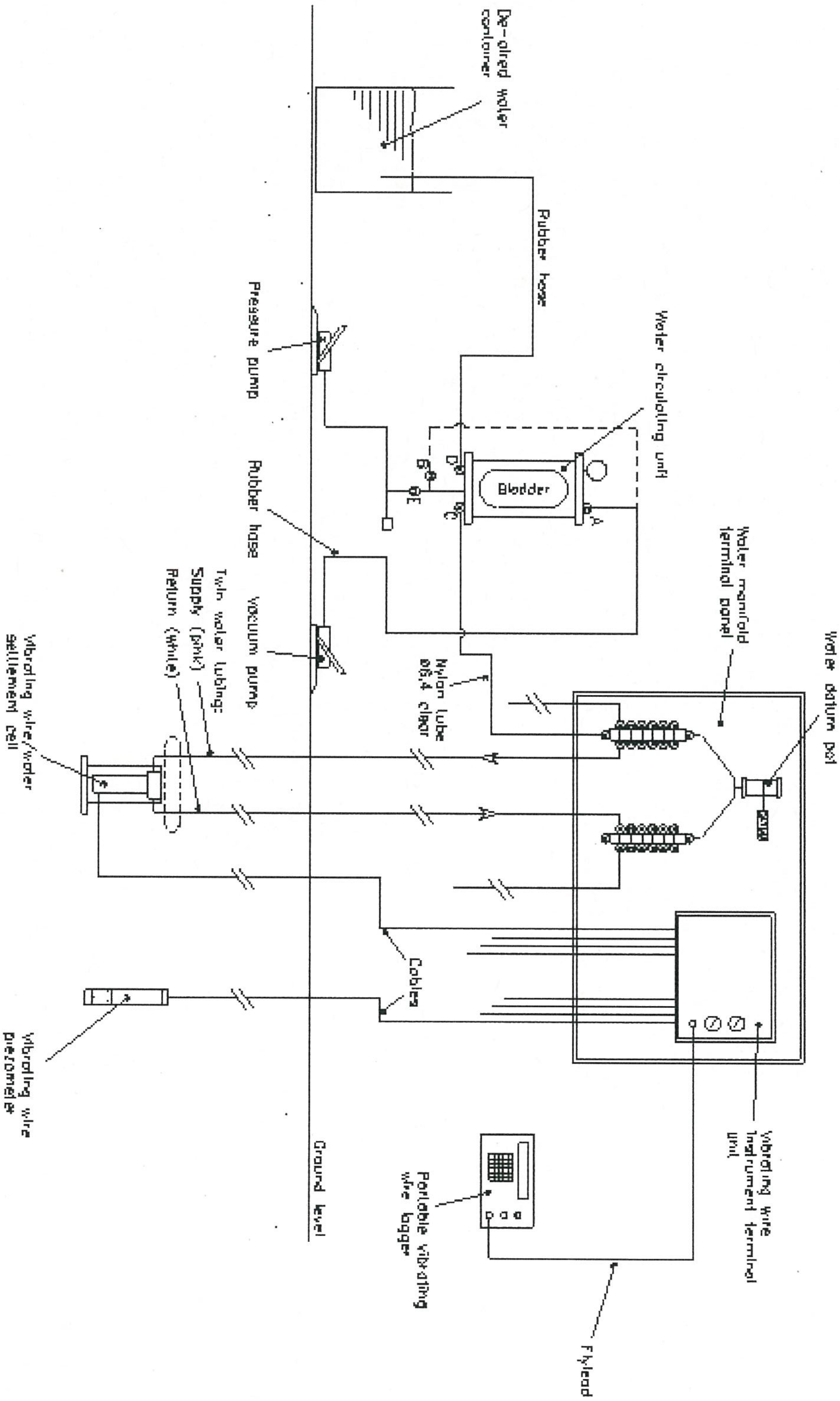


Figure 4 - De-Aired Water Boiler

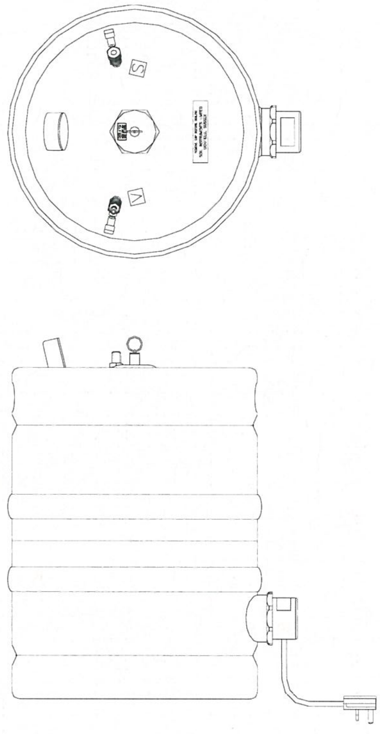
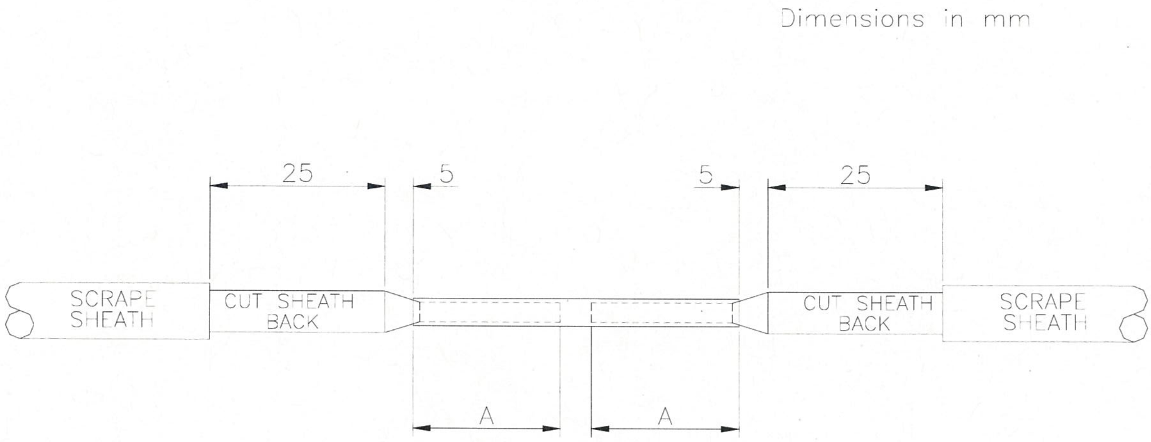


Figure 5 - Preparation Of Cable Joint



A	A	Connector
As required to fit connector		Compression type connector
Sheath opening not to exceed 115mm		

Figure 6 - Cable Jointing Kit Wrapping Electrical Connector & Fitting Mould

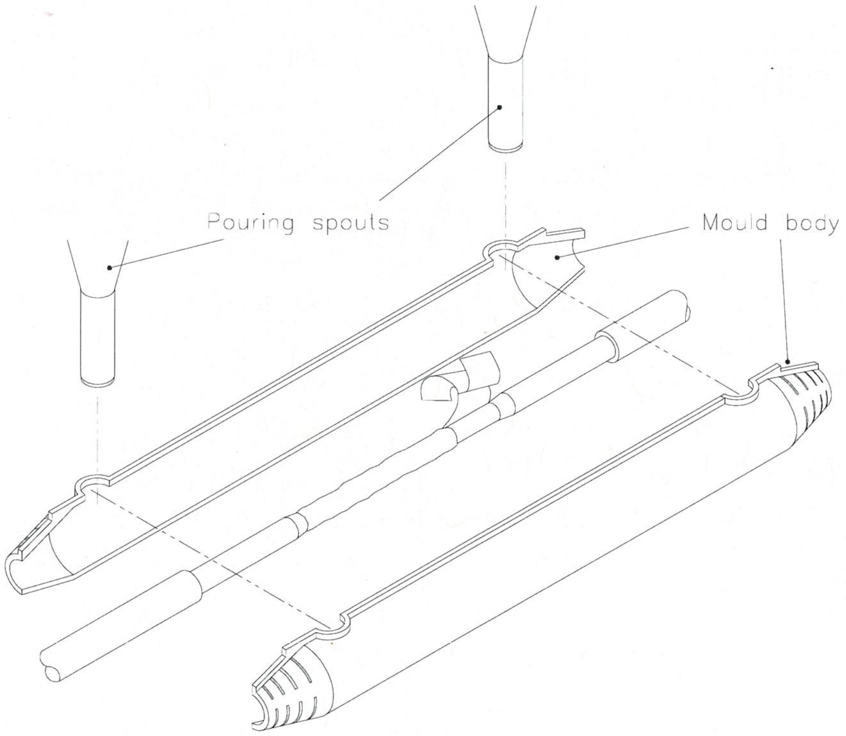


Figure 7 - Sample Calibration Certificate

Soil Instruments Limited
Bell Lane Uckfield East Sussex TN22 1QL
Telephone
+44 (0) 1825 765044
Facsimile +44 (0) 1825 761740
Email sales@soil.co.uk www.soil.co.uk

International Geotechnical Instrumentation Specialists

BS EN ISO 9001 FM 22368

INVESTOR IN PEOPLE

VIBRATING WIRE INSTRUMENTS CALIBRATION CERTIFICATE

Instrument Type : Vibrating Wire Settlement Cell — **Serial No.** : 025315

Instrument Range : 0.00 to 150.0 kPa — **Calibration Date** : 27/02/2006

Gauge Factors in kPa — **Ambient Temperature** : 22°C

Period Gauge Factor (K): 885.7146000 — **Barometric Pressure** : 1017 mbar

Linear Gauge Factor (G): (kPa/digit)0.0885700 — **Calibration Technician** : David Manville

Polynomial Gauge Factor A: 0.000000118766700

Calibration Equipment:
Air Dead-Weight Calibrator Serial No. 23152
Vibrating Wire Logger Serial No. 635

Polynomial Gauge Factor B: -0.0900006300

Polynomial Gauge Factor C:** 612.135400 — **Regression Zero** : 6863.0

Applied (kPa)	Reading (Period)	Reading $F^2/1000$	Calculated (Linear)	Error %FS (Linear)	Linear Increment	Calculated (Polynomial)	Error %FS (Polynomial)
0.00	3816.9	6864.0	-0.084	-0.06	0.0	-0.033	-0.02
15.00	3865.1	6694.0	14.973	-0.02	-170.0	14.993	0.00
30.00	3915.1	6524.0	30.030	0.02	-170.0	30.026	0.02
45.00	3967.1	6354.0	45.087	0.06	-170.0	45.067	0.04
60.00	4021.0	6185.0	60.056	0.04	-169.0	60.025	0.02
75.00	4077.1	6016.0	75.024	0.02	-169.0	74.990	-0.01
90.00	4135.2	5848.0	89.904	-0.06	-168.0	89.874	-0.08
105.00	4197.0	5677.0	105.050	0.03	-171.0	105.030	0.02
120.00	4260.9	5508.0	120.019	0.01	-169.0	120.015	0.01
135.00	4327.8	5339.0	134.987	-0.01	-169.0	135.008	0.01
150.00	4398.0	5170.0	149.956	-0.03	-169.0	150.007	0.00

Formulae: Linear* $E = G(R0 - R1)$

Polynomial** $E = AR1^2 + BR1 + C$

* The zero reading should be established on site by the user on installation.
** The site value of C must be calculated using the formula $C = -(AR0^2 + BR0)$

SOIL INSTRUMENTS LIMITED certifies that the instrument detailed hereon has, as applicable, been inspected, tested and calibrated in accordance with ISO 9001:2000 approved procedures and, unless otherwise indicated, performs within ± 0.10% as specified. Thus, the instrument conforms in all respects to our relevant specifications and drawings.

Certified: .... Line MANAGER

FOR SOIL INSTRUMENTS LIMITED

Figure 8 - Water Settlement Cells Temperature Effects

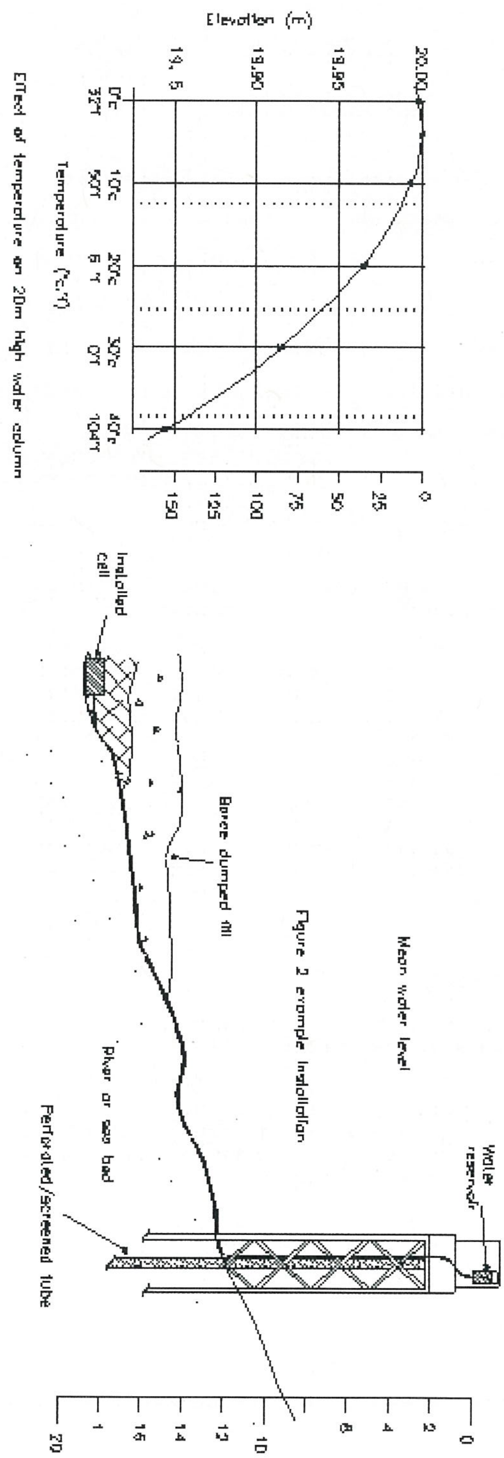


Figure 9 - Trouble Shooting Flowchart

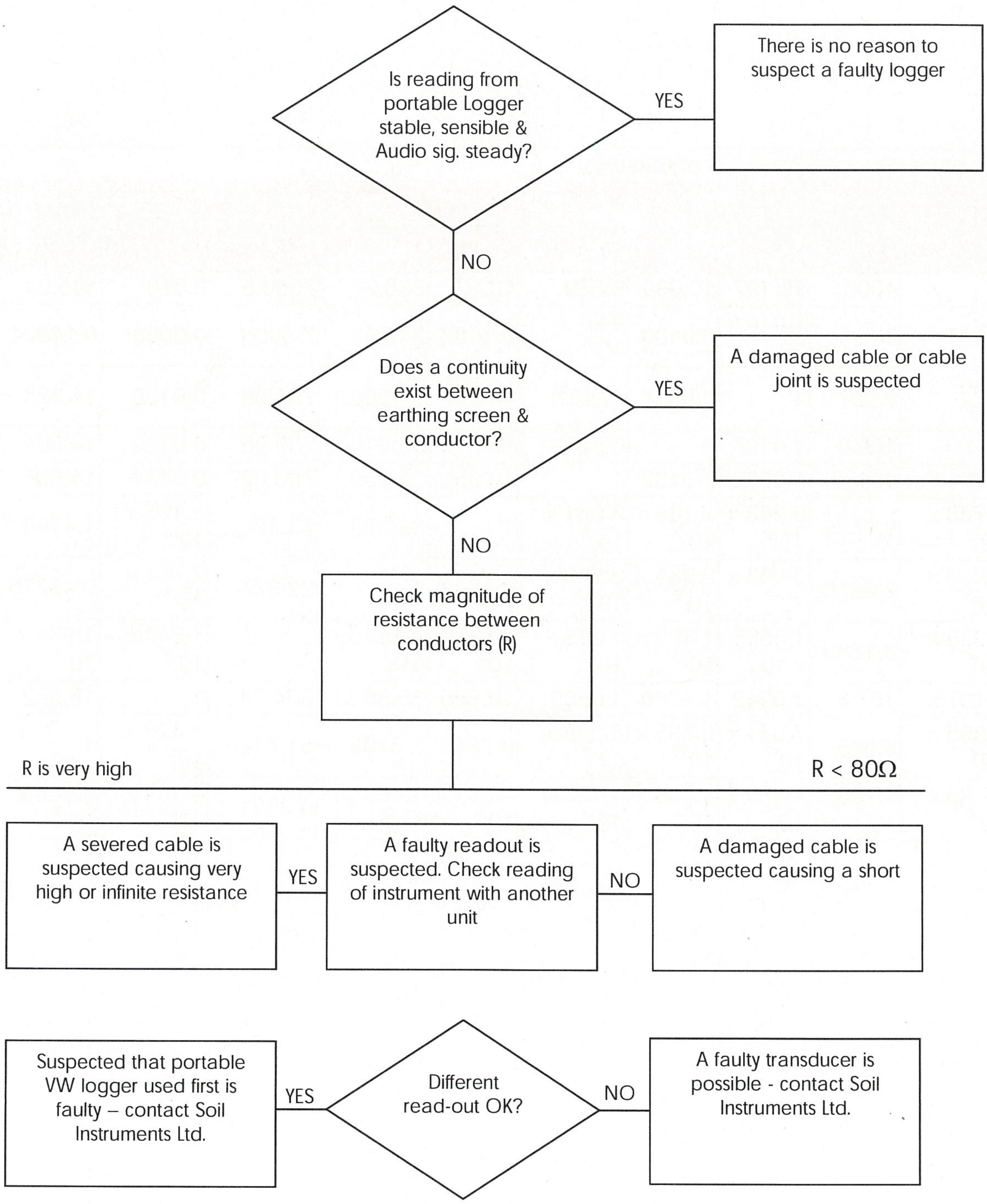


Figure 10 – Conversion Table

Pressure, Stress & Modulus of Elasticity										
MN/m^2 or MPa	**kN/m^2 or kPa**	**kp or kgf/cm^2**	**bar**	**atm**	**m H_2O**	**ft H_2O**	**mm Hg**	**$tonf/ft^2$**	**psi or lbf/in^2**	**lbf/ft^2**
1	1000	10.197	10.000	9.869	102.2	335.2	7500.6	9.320	145.04	20886
0.001	1	1.019×10^{-2}	0.0100	9.87×10^{-3}	0.1022	0.3352	7.5006	0.0093	0.14504	20.886
9.807×10^{-2}	98.07	1	0.9807	0.9678	10.017	32.866	735.56	0.9139	14.223	2048.1
0.100	100.0	1.0197	1	0.9869	10.215	33.515	750.06	0.9320	14.504	2088.6
0.1013	101.33	1.0332	1.0132	1	10.351	33.959	760.02	0.9444	14.696	2116.2
9.788×10^{-3}	9.7885	9.983×10^{-2}	9.789×10^{-2}	9.661×10^{-2}	1	3.2808	73.424	9.124×10^{-2}	1.4198	204.45
2.983×10^{-3}	2.9835	3.043×10^{-2}	2.984×10^{-2}	2.945×10^{-2}	0.3048	1	22.377	2.781×10^{-2}	0.43275	62.316
1.333×10^{-4}	0.1333	1.3595×10^{-3}	1.333×10^{-3}	1.315×10^{-3}	1.362×10^{-2}	4.469×10^{-2}	1	1.243×10^{-3}	1.934×10^{-2}	2.7846
0.1073	107.3	1.0942	1.0730	1.0589	10.960	35.960	804.78	1	15.562	2240.0
6.895×10^{-3}	6.895	7.031×10^{-2}	6.895×10^{-2}	6.805×10^{-2}	0.7043	2.3108	51.714	6.426×10^{-2}	1	144.00
4.788×10^{-5}	4.788×10^{-2}	4.883×10^{-4}	4.788×10^{-4}	4.725×10^{-4}	4.891×10^{-3}	1.605×10^{-2}	0.3591	4.464×10^{-4}	6.944×10^{-3}	1

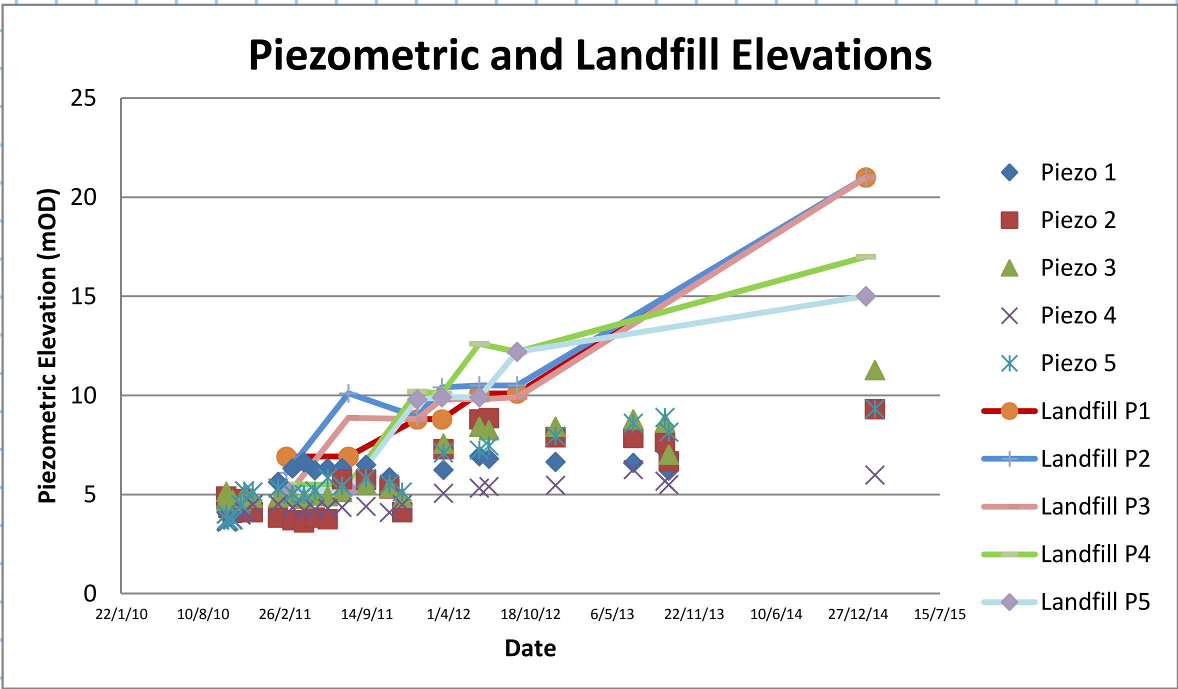
Appendix 5 Pore Water Pressure Review

Calculations

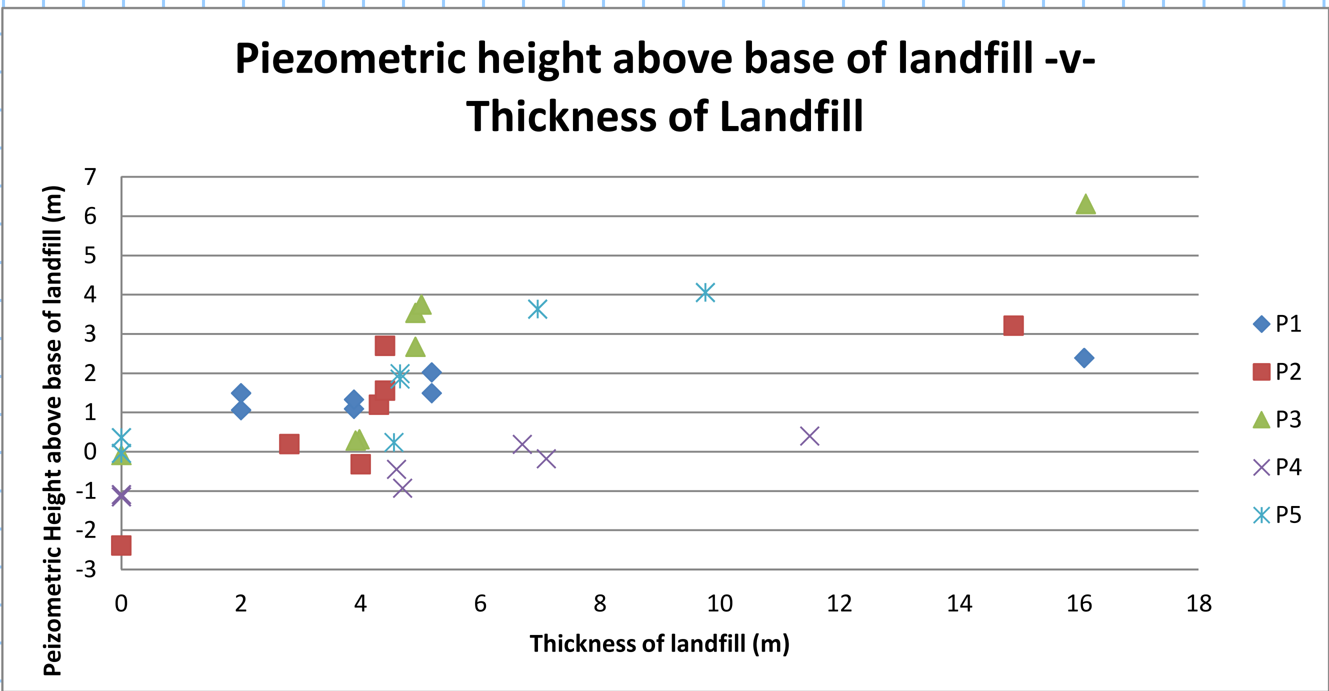
Review of Porewater pressures

Project Title	Docksway Landfill - Area 2								
Project No	14739	156	By	LJT	Checked	AD	Date	09 04	2015

Graph indicating the level of piezometric head as measured in the vibrating wire piezometers installed in Cell 2
The graph also includes a record of the elevation of the landfill at the location of each piezometer



Based on the above graph the graph below indicates the piezometric height relative to the thickness of landfill

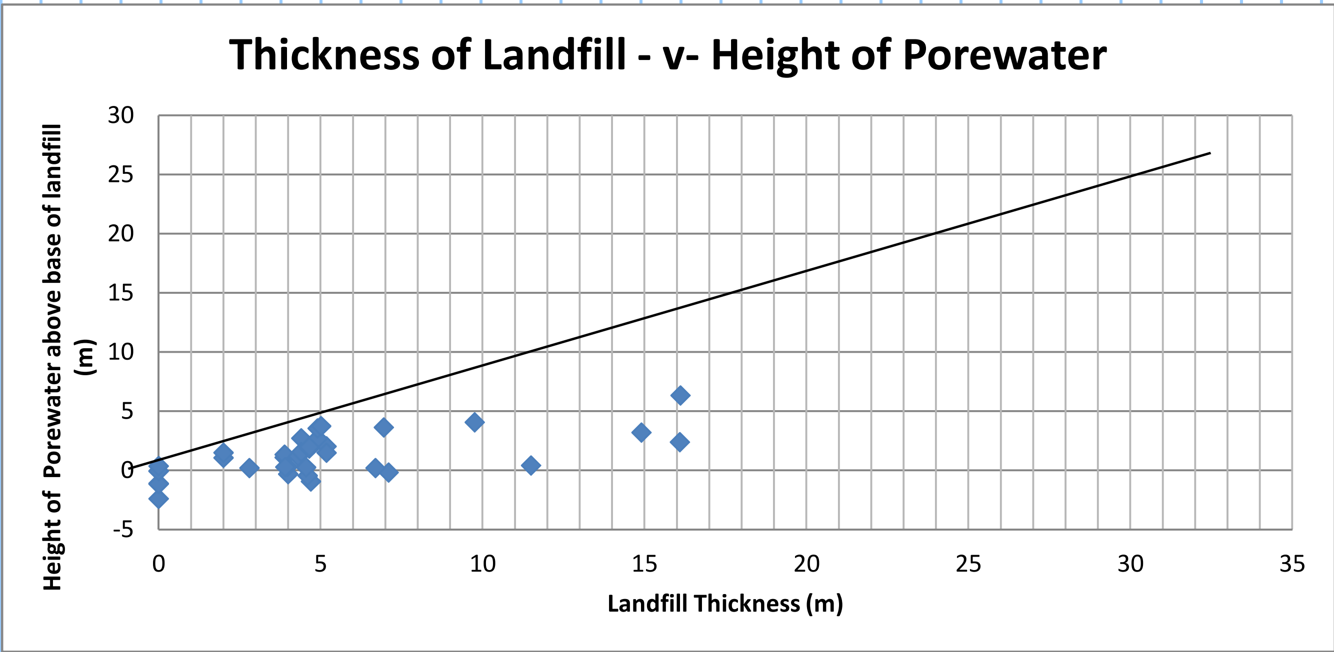


Calculations

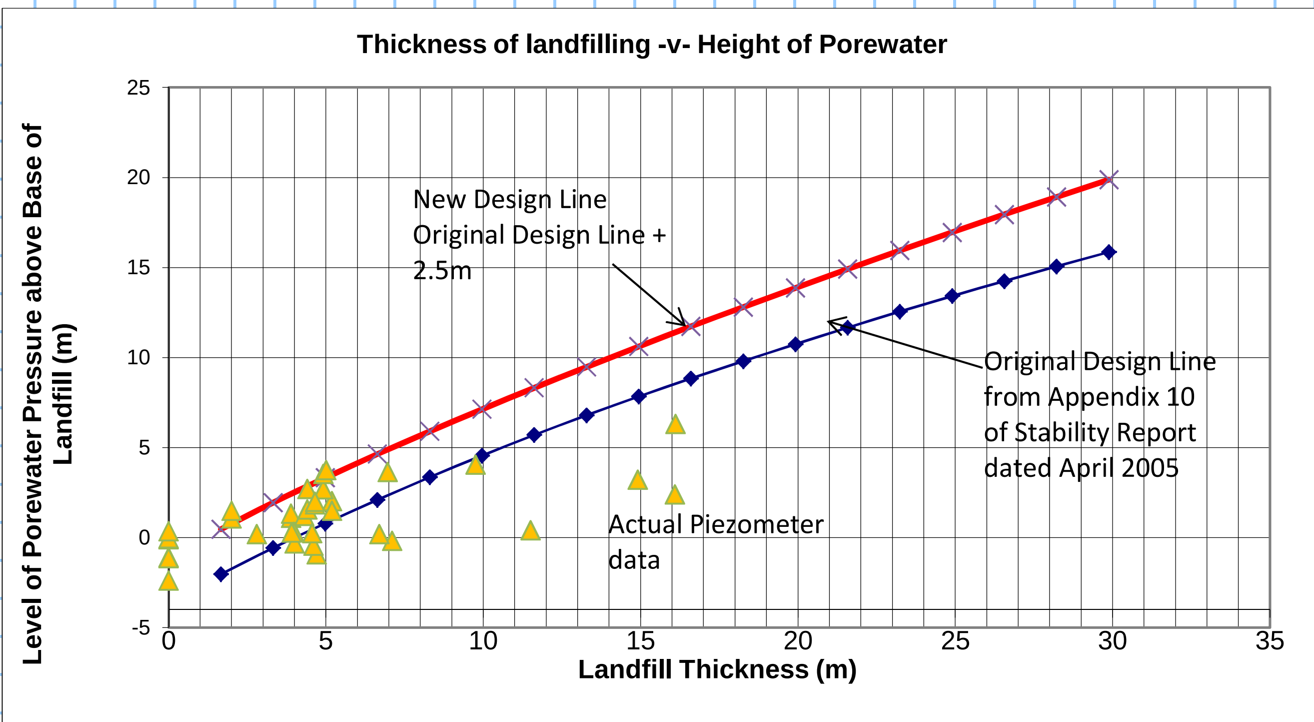
Review of Porewater pressures

Project Title	Docksway Landfill - Area 2								
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

From the previous graph a worst case scenarion of estimated height of porewater pressure relative to thickness of landfill has been poltted assuming a straight line as indicated below



A straight line fit is condidered too conservative and therefore an approach using a curve similar to the original assessment is adopted with a conservative line passing above all the data points at 2.5m above the original design line is proposed as presented below.

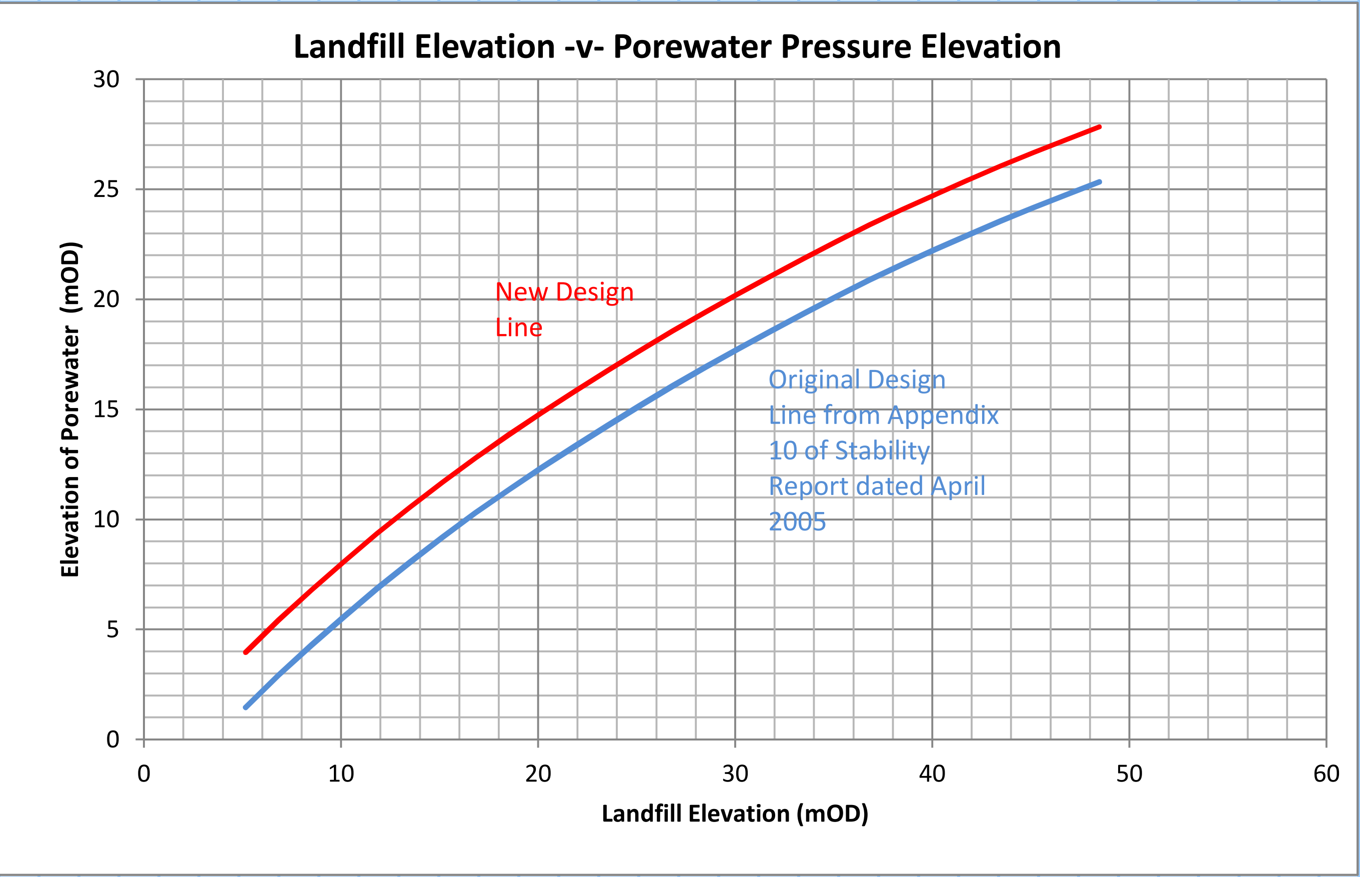


Calculations

Review of Porewater pressures

Project Title	Docksway Landfill - Area 2										
Project No	14739	156	By	LJT	Checked	AD	Date	09	06	2015	

The following graph has been used in the stability assement using the new design line as indicated in red



Appendix 6 Dam Stability Assessment

Calculations

Project Title	Docksway Disposal Site: Area 2

Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Existing Dam Stability

The Cross Section used for this revised stability assessment is the same as previously assessed in the Stability Assessment Addendum Report dated February 2006 and has been reproduced in the GGU stability runs presented on the following sheets

The design parameters used in this assessment are the same as used previously in the Stability Assessment Addendum Report dated February 2006 and as summarised below

		Undrained		Drained	
Material	γ_b	ϕ_u	c_u	ϕ'	c'
	kN/m^3	Degrees	kN/m^2	Degrees	kN/m^2
Made Ground	20	32	4	32	4
Landfill	12	25	5	25	5
Clay Liner	18	0	40	22	0
Alluvium	17.5	0	10	22	0
Strength Gain Alluvium beneath Dam	17.5	0	73	-	-
Strength Gain Alluvium beneath Landfill	17.5	0	60	-	-
Gravel	19	35	0	35	0
Mudstone	20	0	75	27	5

Note the exception to this is the bulk density of the Landfill which has been increased to 12kN/m^2 to reflect the nature of the waste that is actually being placed at the site as discussed in in the main report text.

Within this assessment the approach adopted for the analysis is in accordance with EC7 applying Design Approach 1. Combination 1 applies partial load factors of greater than unity to unfavourable external loads and material partial factors of unity. Combination 2 applies greater than unity material partial factors and partial load factors to unfavourable loads of unity. As no external loads are considered in this assessment Combination 2 will give the worst case and therefore only combination 2 is used in all analysis.

Calculations

Project Title	Docksway Disposal Site: Area 2

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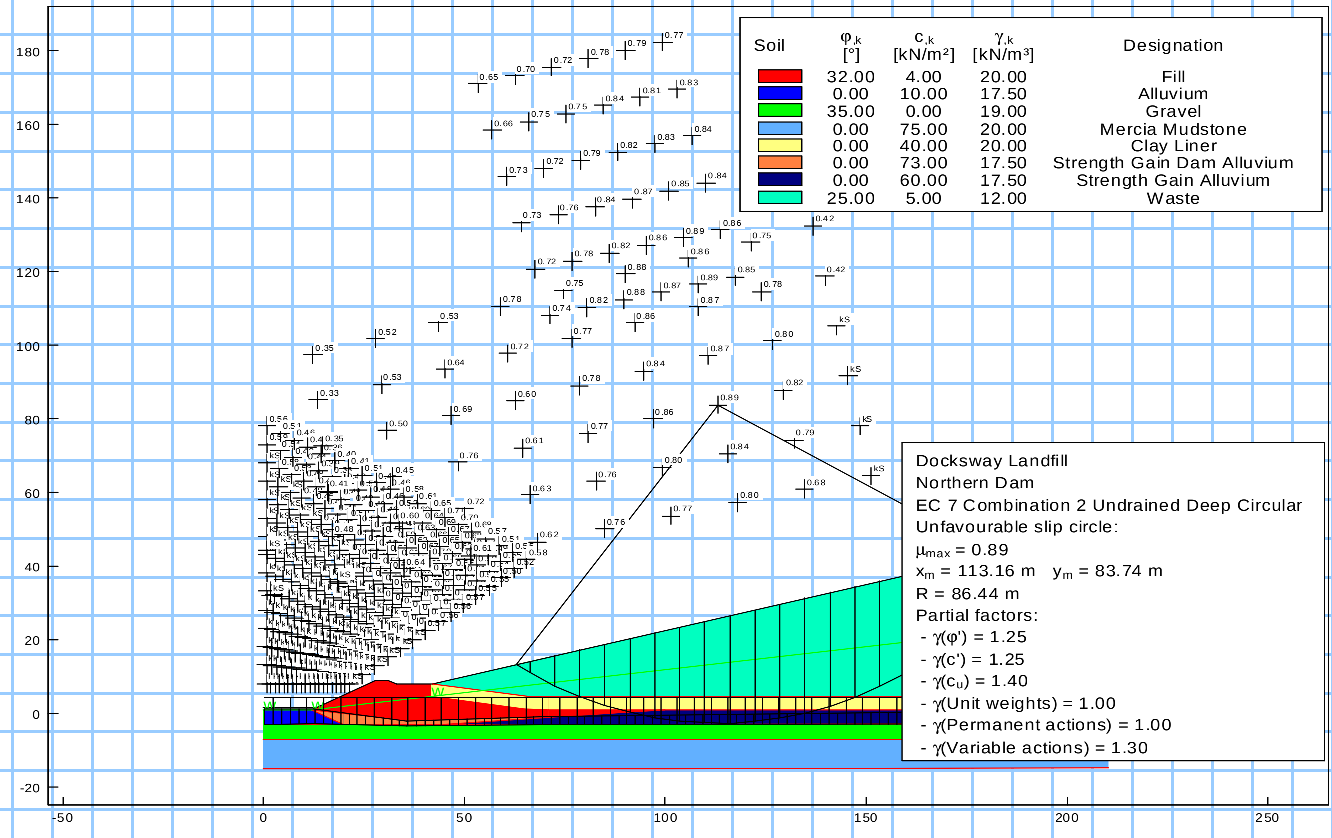
The analysis undertaken has included for the finished landfill profile with a 1:4 side slope and a final landfill upper surface level of 40mOD. The cases considered include the following for the Northern and Southern Dams:

- Undrained Stability for landfill just after completion deep circular failure
- Undrained Stability for landfill just after completion shallow circular failure
- Undrained Stability for landfill just after completion wedge failure
- Drained Stability for landfill just after completion deep circular failure
- Drained Stability for landfill just after completion shallow circular failure
- Drained Stability for landfill just after completion wedge failure

Northern Dam Undrained Deep Circular max Utilisation Factor 0.89



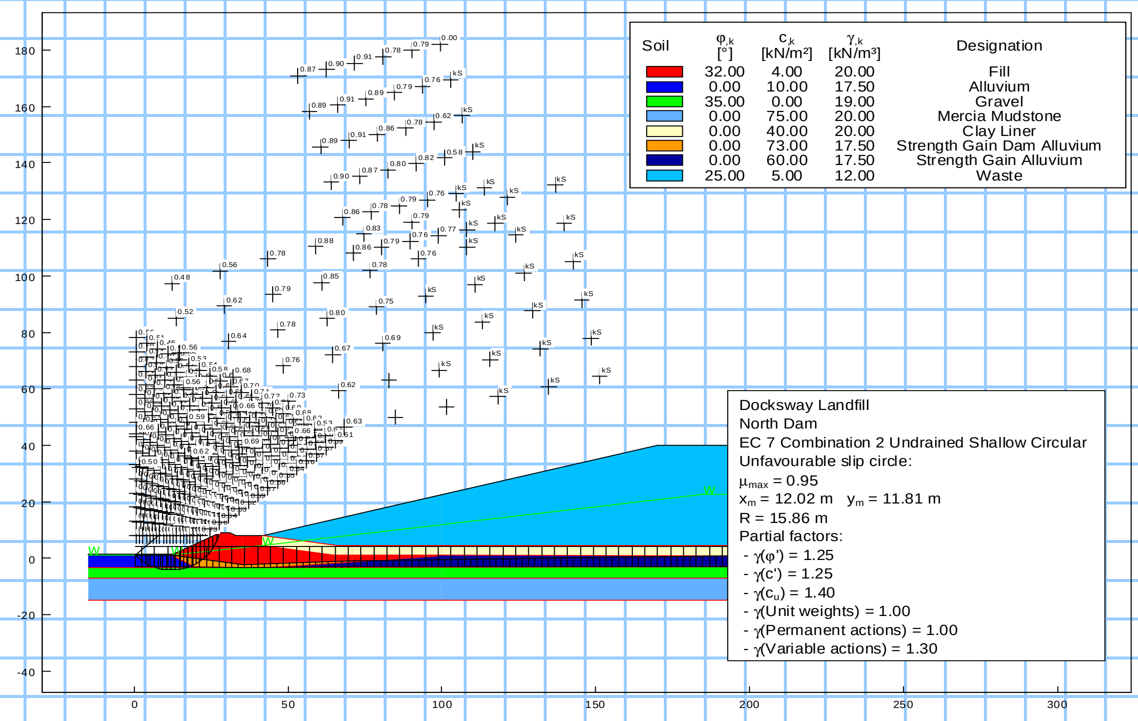
Sheet	Dam 2

Calculations

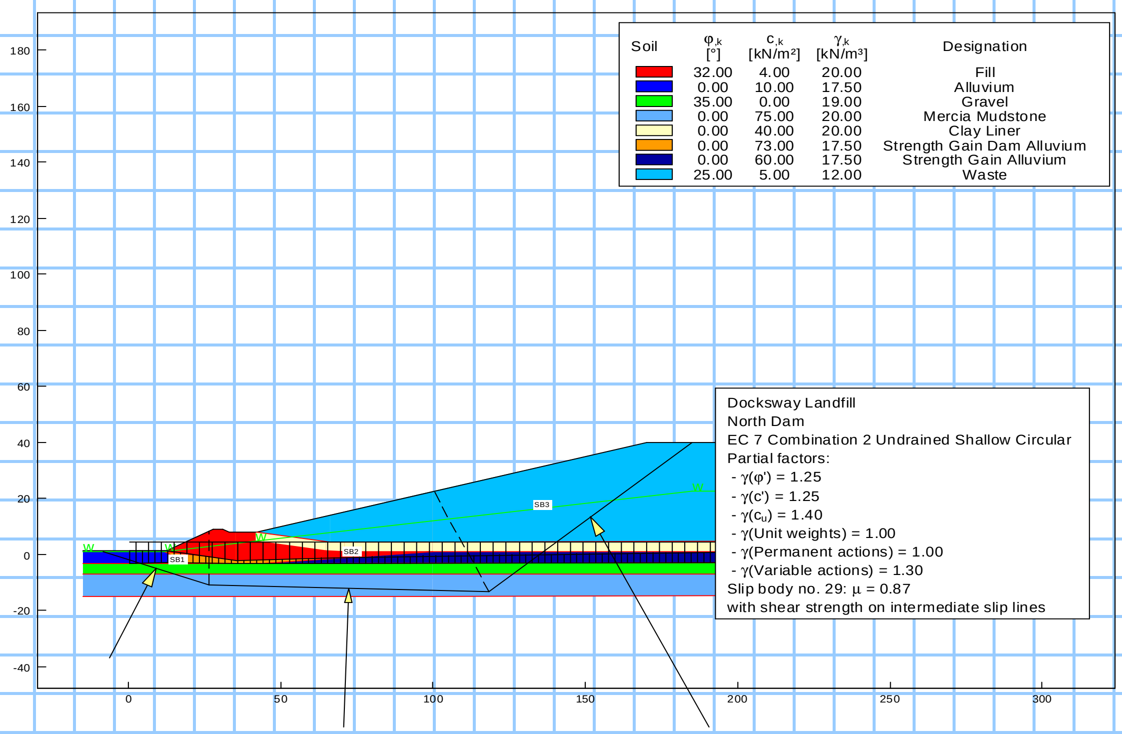
Project Title	Docksway Disposal Site: Area 2

Project No	14739	156	By	LJT	Checked		Date	09	04	2015

Northern Dam Undrained Shallow Circular max Utilisation Factor 0.95



Northern Dam Undrained Wedge max Utilisation Factor 0.87

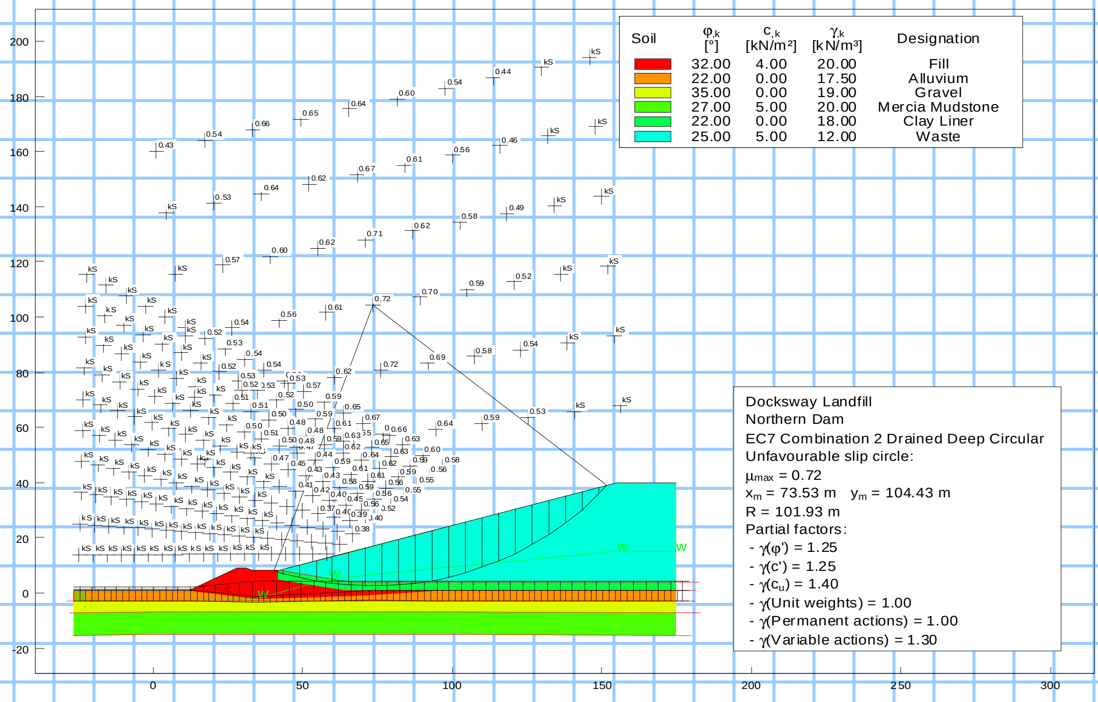


Sheet Dam 3

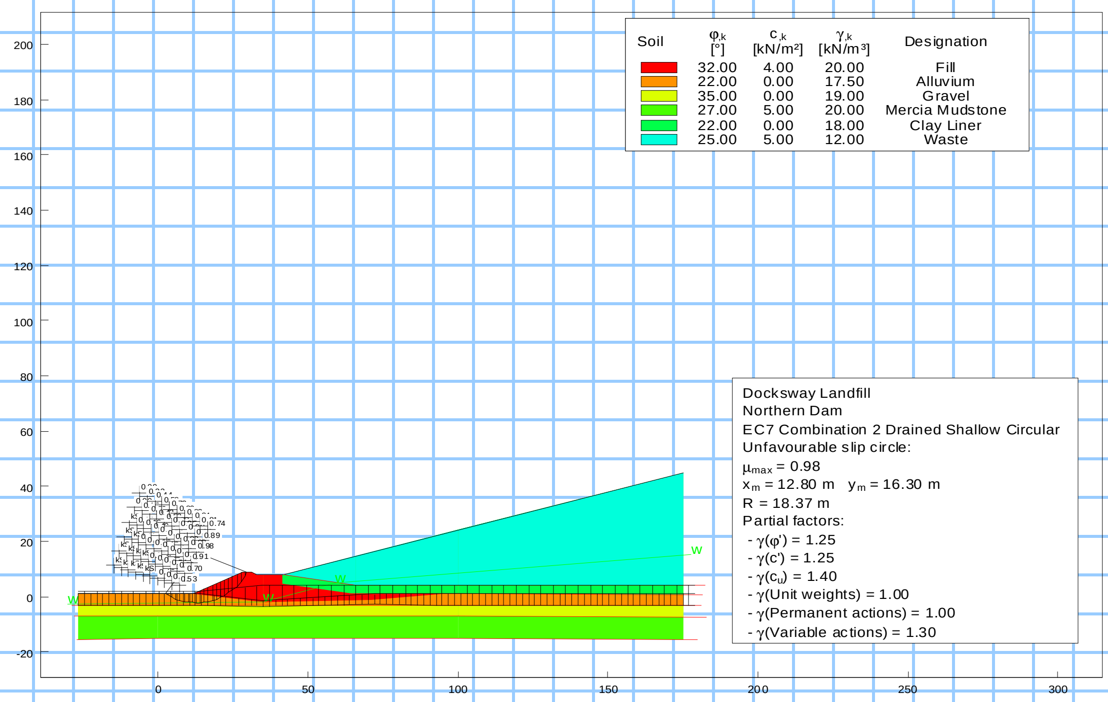
Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Northern Dam Drained Deep Circular max Utilisation Factor 0.72



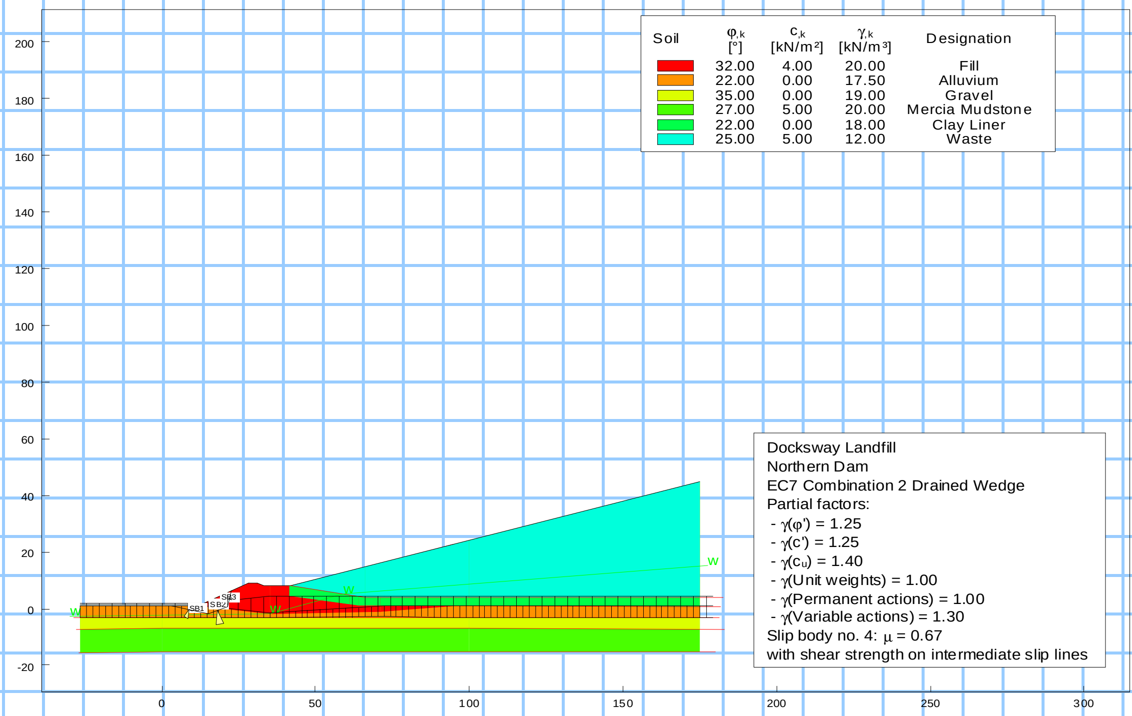
Northern Dam Drained Shallow Circular max Utilisation Factor 0.98



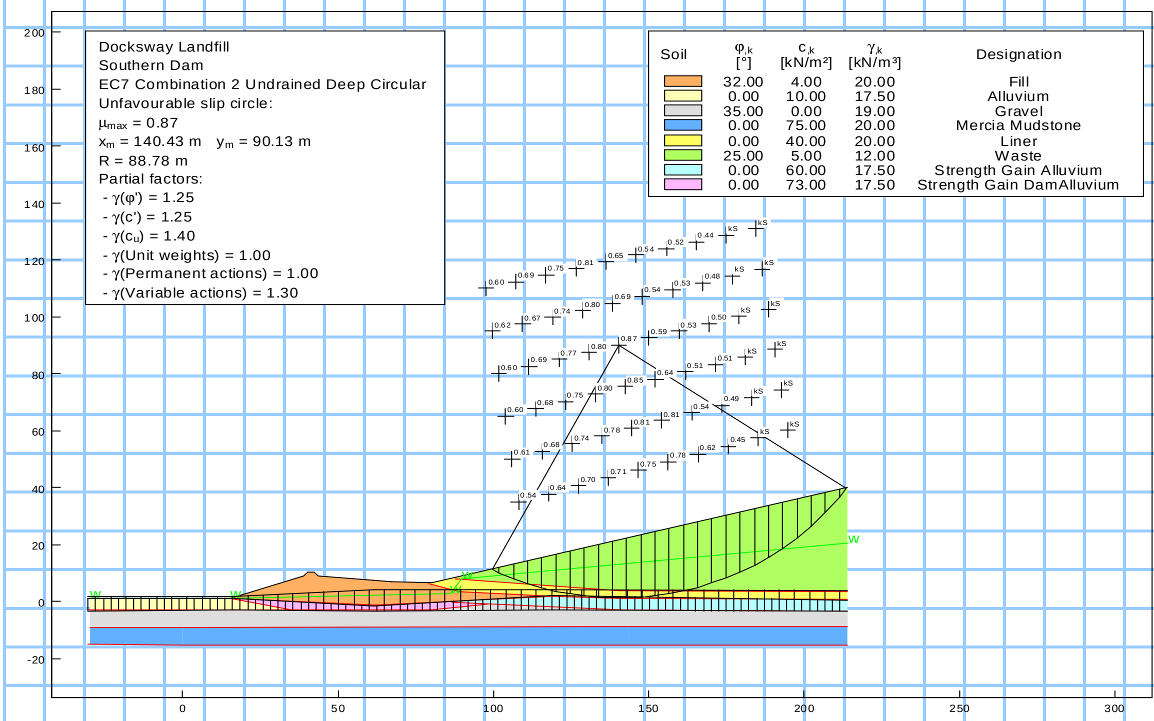
Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Northern Dam Drained Wedge max Utilisation Factor 0.67



Southern Dam undrained Deep Circular max Utilisation Factor 0.87

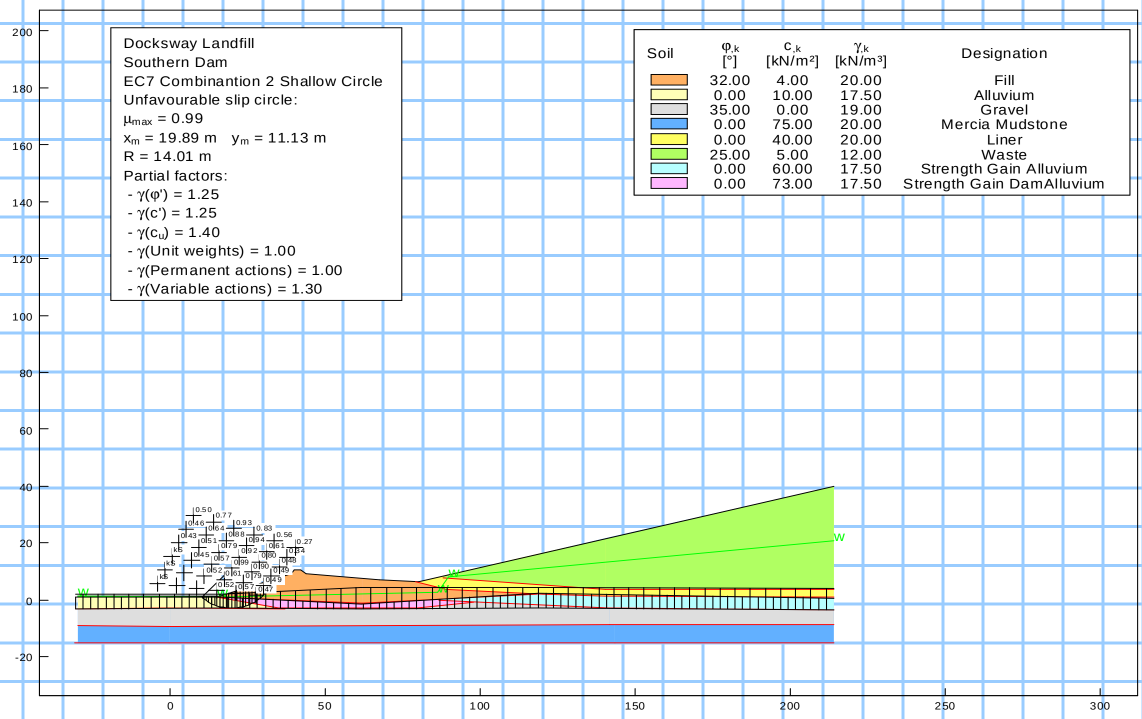


Calculations

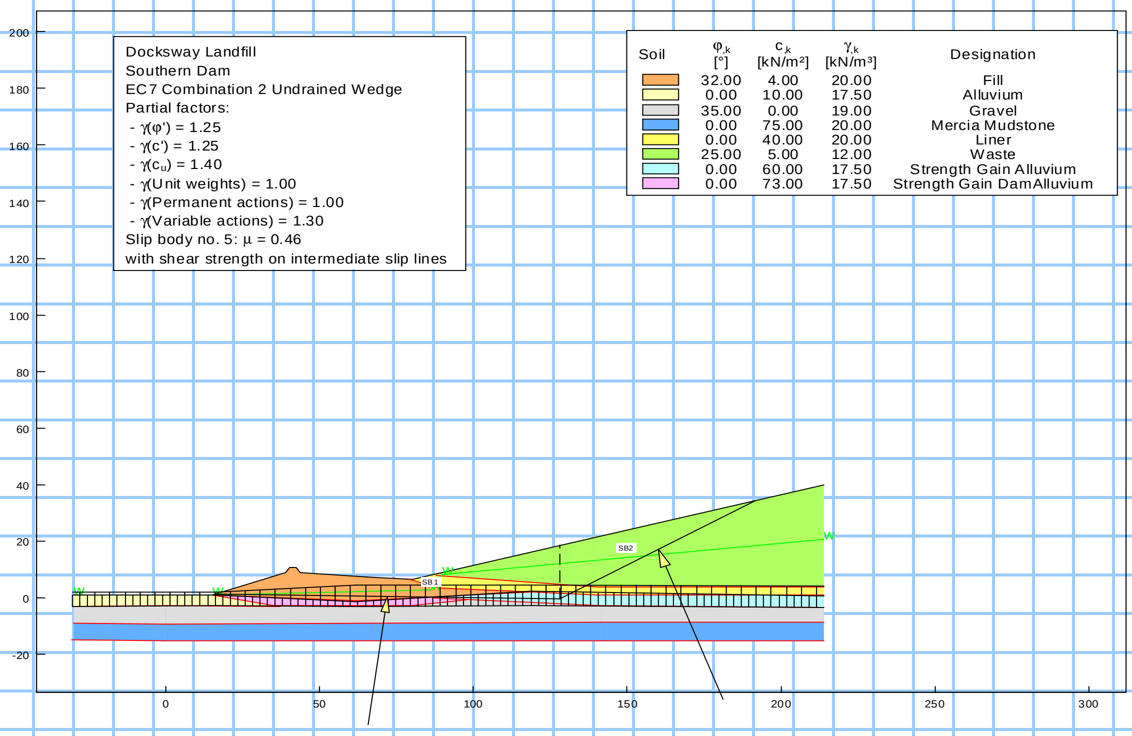
Project Title	Docksway Disposal Site: Area 2

Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Southern Dam undrained Shallow Circular max Utilisation Factor 0.99



Southern Dam undrained Wedge max Utilisation Factor 0.46

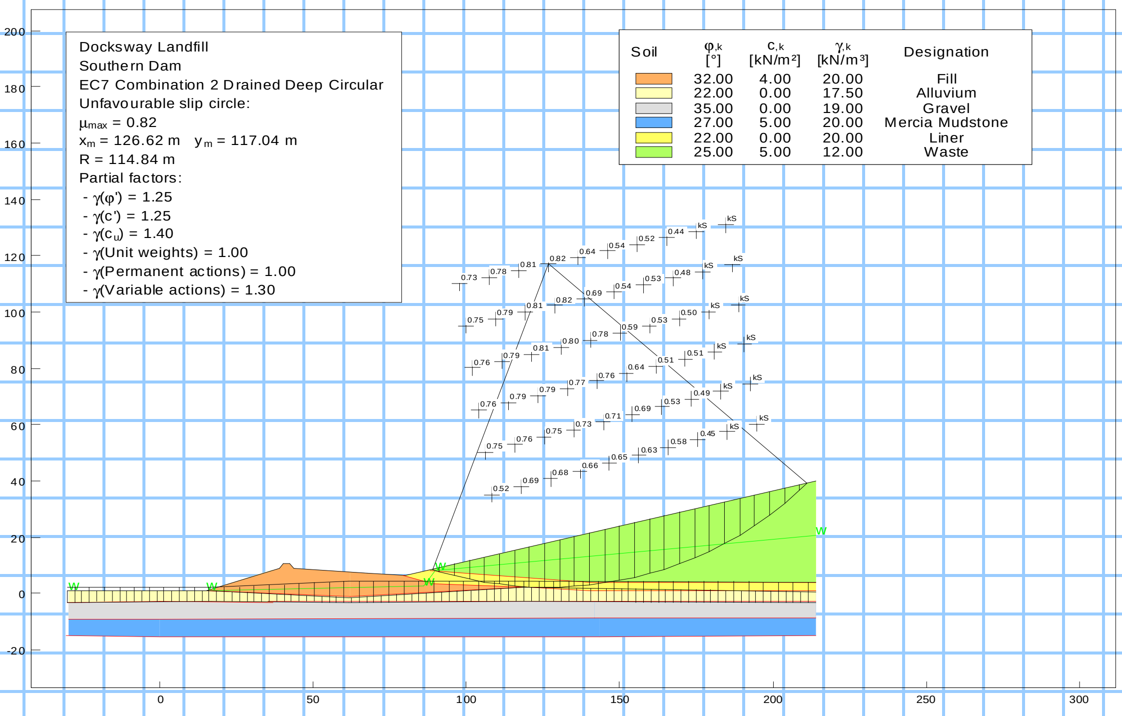


Sheet Dam 6

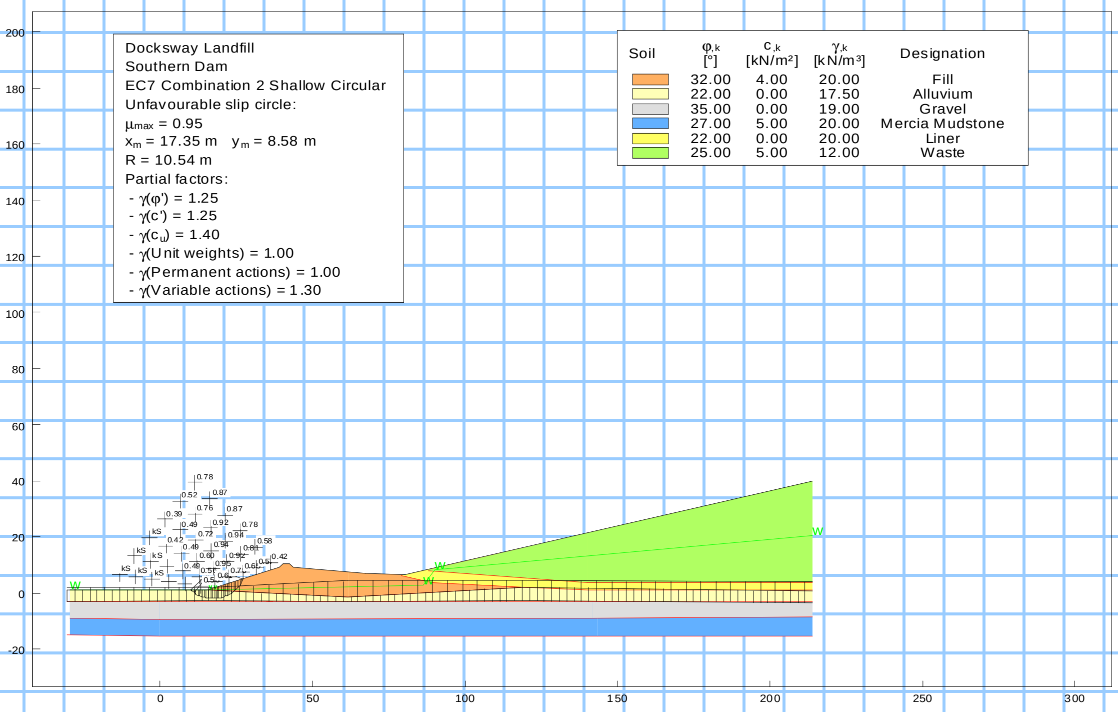
Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Southern Dam drained Deep Circular max Utilisation Factor 0.82



Southern Dam drained Shallow Circular max Utilisation Factor 0.95

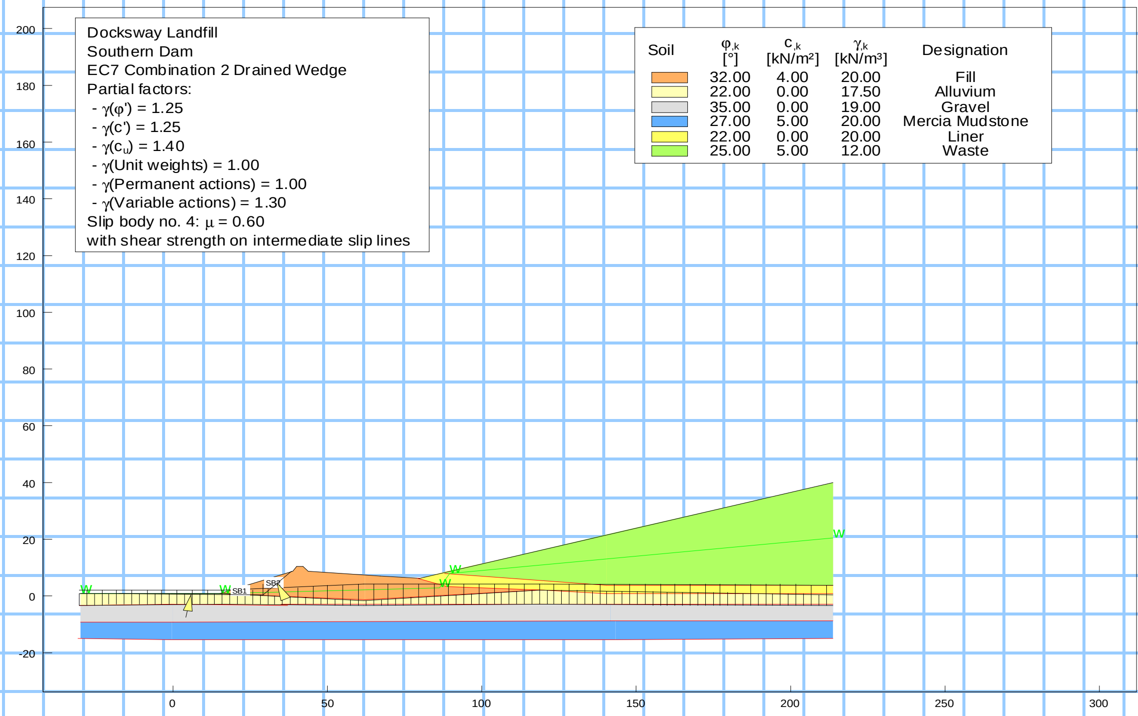


Calculations

Project Title	Docksway Disposal Site: Area 2

Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Southern Dam drained Wedge max Utilisation Factor 0.6



Sheet Dam 8

Appendix 7 New Bund Stability Assessment

Calculations

Project Title	Docksway Disposal Site: Area 2										
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015	

Cell 3 New Edge Bund Stability Assessment

This assessment considers the stability of new bund that will be required at the edge of Cell 3 in the former River Ebbw channel. The location of the section assessed is presented on Figure 2 and a schematic section is presented as Figure 5 . The assessment presented here includes the following analysis:

- Short term undrained stability of bund itself immediately on completion of the bund
- Long term drained stability of the completed landfill slope and bund
- Short term stability of landfill and bund for different stages of filling taking account of excess porewater pressures and strength gain.

The design parameters used in this assessment are the same as used previously in the Stability Assessment Addendum Report dated February 2006 and as summarised below

		Undrained		Drained	
Material	γ_b	ϕ_u	c_u	ϕ'	c'
	kN/m^3	Degrees	kN/m^2	Degrees	kN/m^2
Fill	20	32	4	32	4
Landfill	12	25	5	25	5
Clay Liner	18	0	40	22	0
Alluvium	17.5	0	10	22	0
Stabilised Alluvium	20	0	65	22	10
Gravel	19	35	0	35	0
Mudstone	20	0	75	27	5

Note the exception to this is the bulk density of the Landfill which has been increased to 12kN/m^2 to reflect the nature of the waste that is actually being placed at the site as discussed in the main report text.

Within this assessment the approach adopted for the analysis is in accordance with EC7 applying Design Approach 1. Combination 1 applies partial load factors of greater than unity to unfavourable external loads and material partial factors of unity. Combination 2 applies greater than unity material partial factors and partial load factors to unfavourable loads of unity. As no external loads are considered in this assessment Combination 2 will give the worst case and therefore only combination 2 is used in all analysis.

Calculations

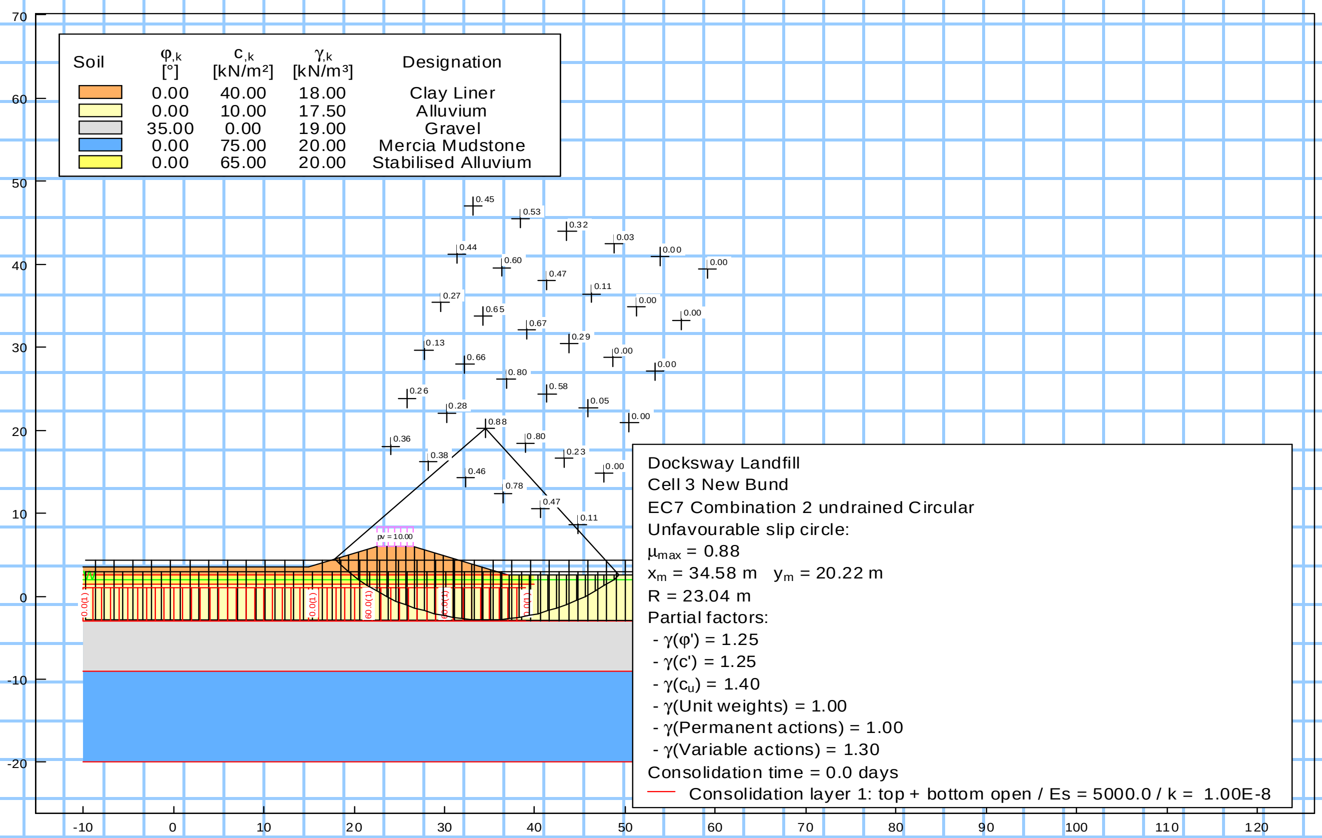
Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Short term stability of the bund immediately on completion of the bund. For this analysis the bund is assumed to be constructed with clay the same as the liner as it will be necessary for the bund to form the outer edge of the liner and that the stabilised alluvium extends at least 3m beyond the outer edge of the dam. The bund is assumed to have a crest elevation of 6m OD in order to provide sufficient freeboard above the maximum leachate level within the landfill and a crest width of 6m to provide room for access around the perimeter with 1:3 side slopes. In addition an artesian head is the included within the River Gravels beneath the Alluvium as applied in previous analysis. To take account of the excess porewater pressure that will be generated within the Alluvium as the bund constructed the Alluvium layer is analysed as a consolidation layer to include an excess porewater pressure equivalent to the pressure exerted by the bund ie 3m x 20kN/m3 = 60kN2

It should be noted that as all the parameters have been factored that the outcome of the assessment is very conservative.

The analysis includes assessment of circular and wedge stability checks and the results of these are as follows:

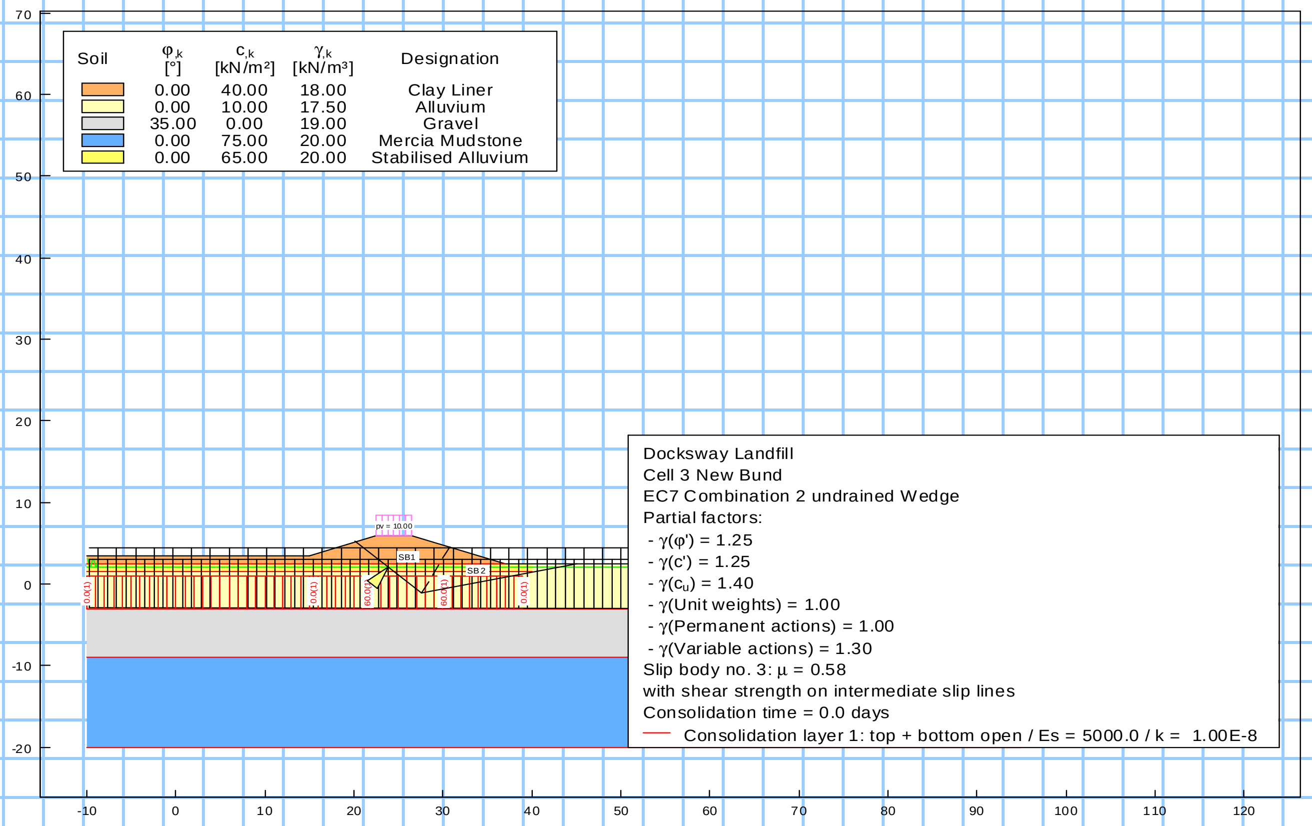
Cell 3 bund undrained circular maximum utilisation factor 0.88



Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Cell 3 Bund undrained wedge maximum utilisation factor 0.58



Now consider the long term stability of the completed landfill to include for a 1:4 landfill slope and a finished level of the landfill of 40mOD. For this analysis a circular and wedge stability check has been undertaken these are presented on the following sheet. For this assessment the ground water level within the landfill is taken as standing at an elevation of 25mOD, this is taken from the revised assessment of porewater pressure and for a landfill elevation of 40m OD.

Calculations

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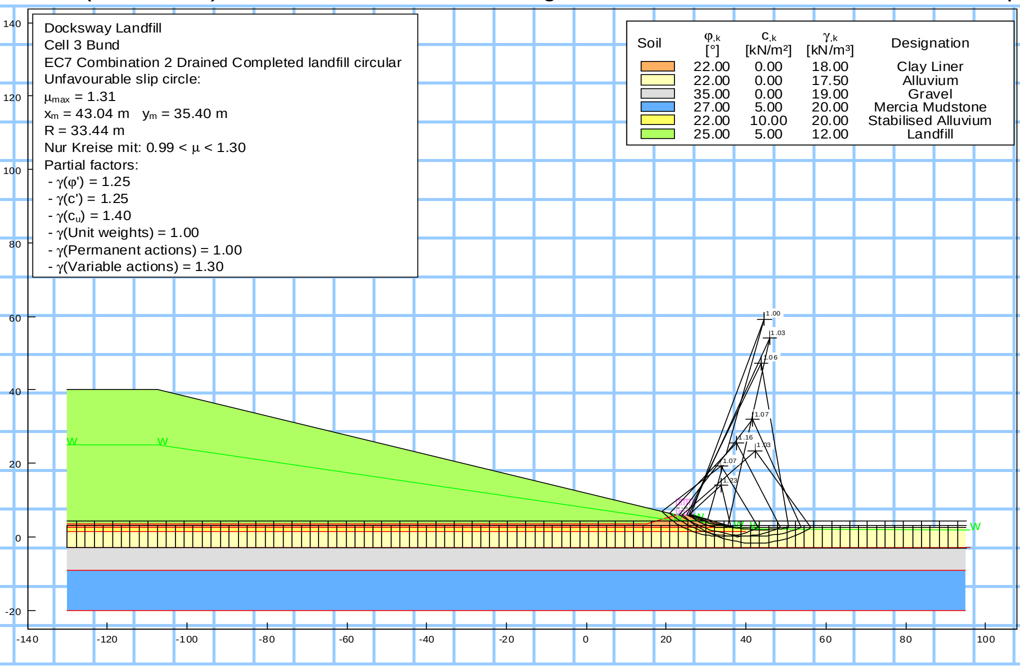
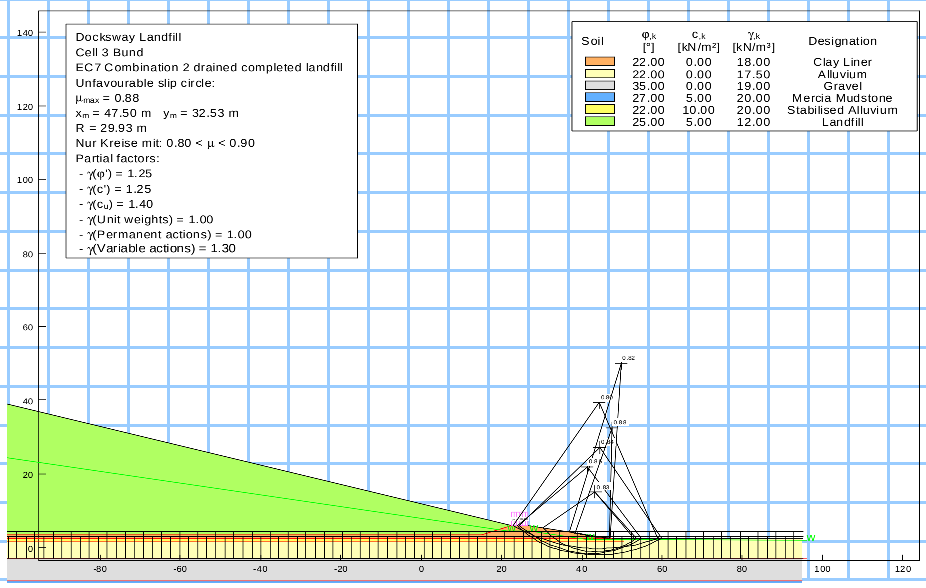
For the circular assessment of the completed landfill it is found that there are several potential failure surfaces with utilisation factors above 1 these include failures surfaces through the landfill and though the outer edge of the dam (see below). In view of the failures though the dam consider a slacker outer slope for the bund at 1:6



For a 1:6 outer slope on the bund it is found that for circular failure the maximum utilisation factor is 0.9. This however is for a circular failure though the landfill mass and not the bund. This assessment is however a very conservative assessment based on the water level assumed, as for the long term case condition the porewater pressures would need to have dissipated. In addition it would also be expected in the long term condition that the landfill density would also increase. These factors would all reduce the utilisation factor. It may be however that at an elevation of 40mOD that the landfill may be approaching limiting stability. If failure surfaces are limited to just the bund the maximum utilisation factor is 0.86

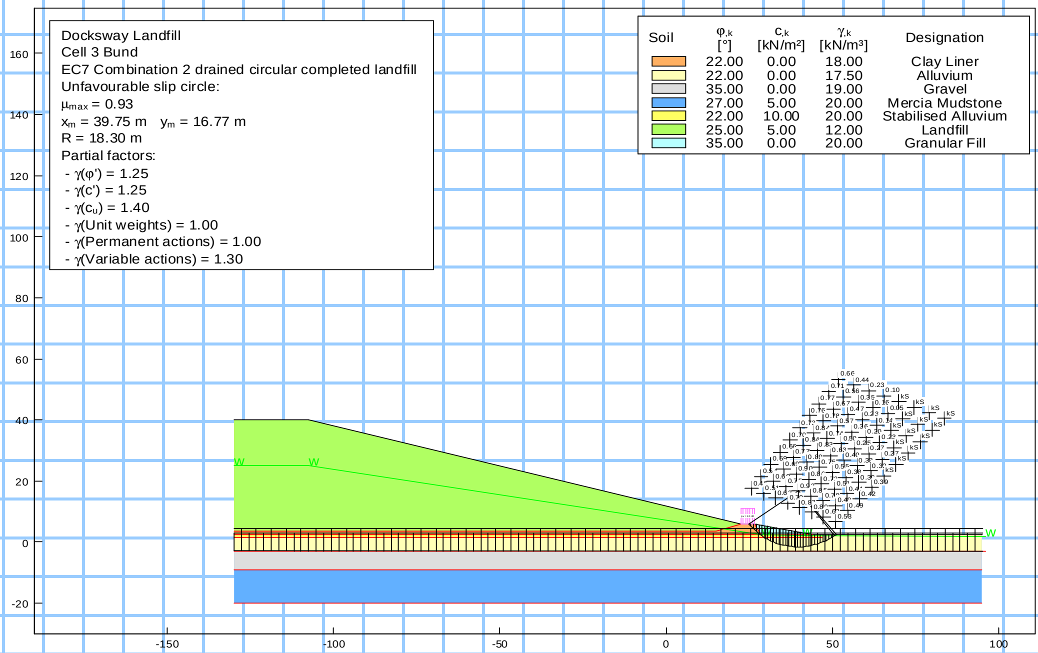
A wedge analysis for the bund indicates a maximum utilisation factor of 0.63



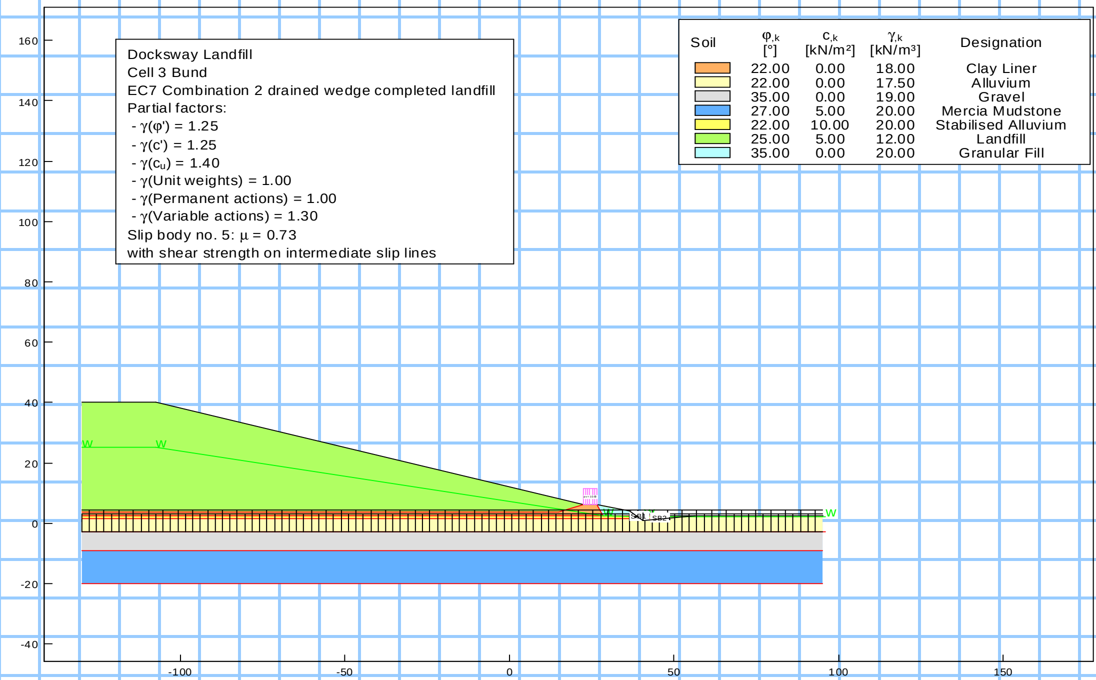
Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

It may be that to save on the quantity of clay that the outer edge of the bund could be constructed with a granular fill so an assessment of this has been undertaken with the granular fill taken as having a bulk density 20kN/m^3 and an angle of friction of 35^o . This assessment indicates a maximum utilisation factor of 0.88 Again this is for failure though the landfill mass and not in the bund as per the previous assessment. Limiting failure to just the bund gives a maximum utilisation factor of 0.93



For a wedge assessment based on a bund at 1:6 outer slope with a granular fill the maximum utilisation factor is 0.73 this is for a wedge failure though the very edge of the dam.



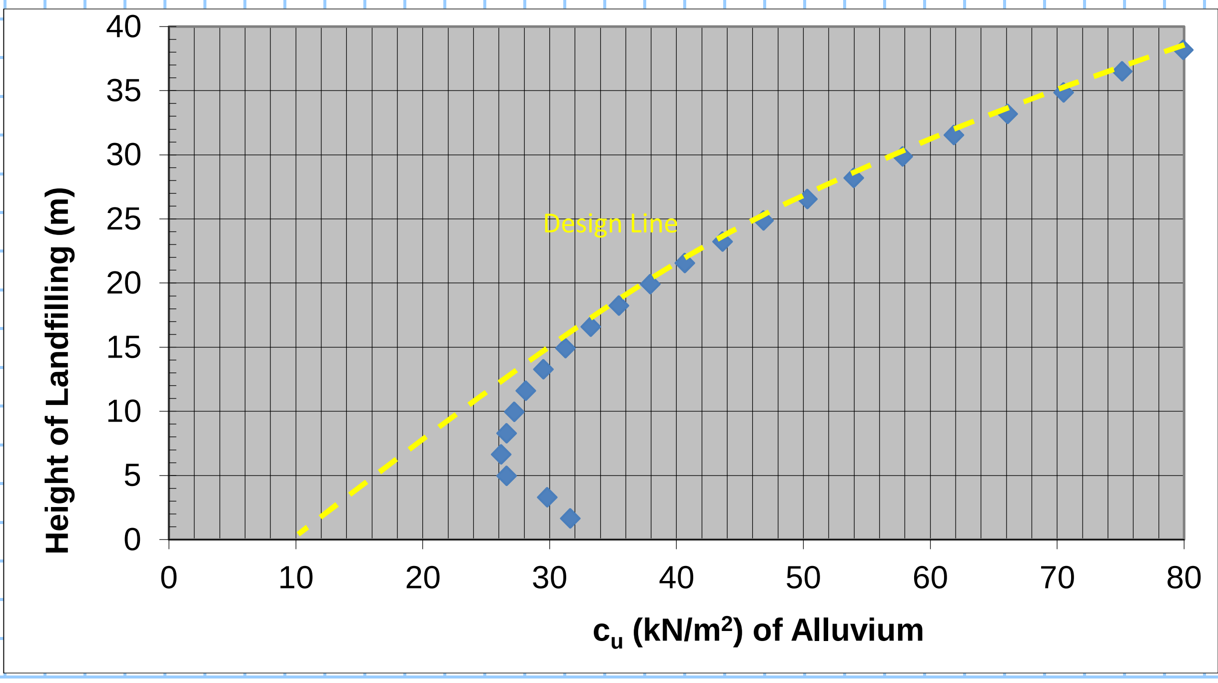
Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

To assess the stability as the landfilling is progressed and to take account of the build up of excess porewater pressure the analysis presented below has been takes account of the piezometric head relative to landfill elevation as presented in Appendix 5. In addition the strength gain in the soft Alluvium due to consolidation and dissipation of excess porewater pressure is taken into account as per the Stability Assessment Addendum Report dated February 2006.

A copy of the porewater elevation -v- landfill elevation is presented below

A copy of the strength gain -v- landfill thickness is included below Note this has been revised from the assessment undertaken in 2005 (Appendix 1) to take account of review of porewater pressure (Appendix 5)



Sheet	New Bund 6	of	17

Calculations

Project Title	Docksway Disposal Site: Area 2									
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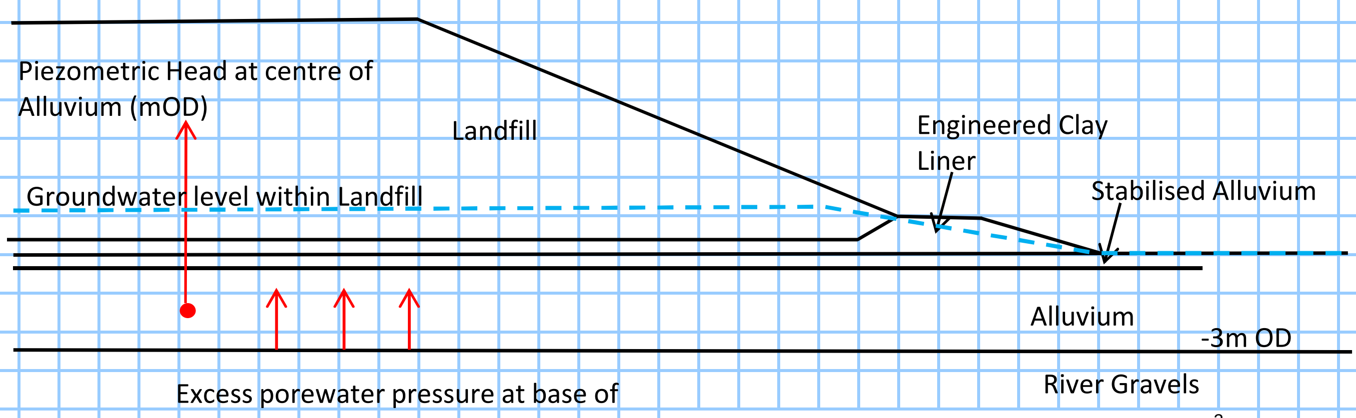
For the assessment of the short term stability it is appropriate to consider the alluvial clays beneath the stabilised surface as a consolidation layer as with this it is possible to input the porewater pressures into that layer based on the predicted porewater elevation - v - landfill elevation as presented on the previous sheet.

For this assessment the following is assumed:



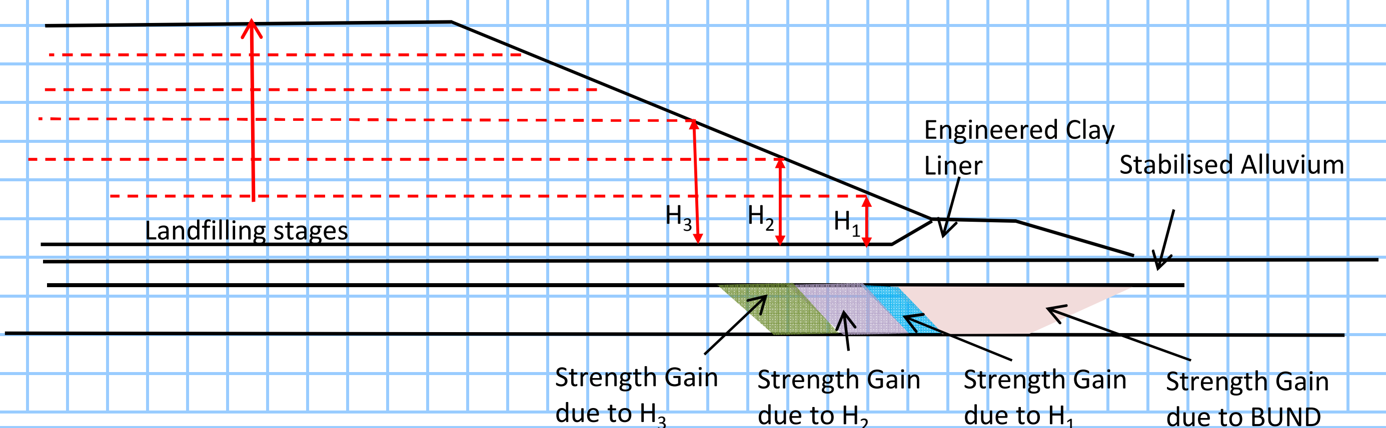
[Piezometric Head mOD - (-3m OD)] x 10kN/m^3

Consider the following Landfill Stages

Landfill Level (mOD)	Piezometric Level (mOD)	Excess PWP (kN/m^2)	Equivalent Landfill Ht (m)	Strength Gain c_u (kN/m^2)
13.5	10.5	135	10	24
18.5	14	170	15	31
23.5	16.5	195	20	38
28.5	20	230	25	47
33.5	21.5	245	30	58
40.0	25	280	36.5	75

NOTE

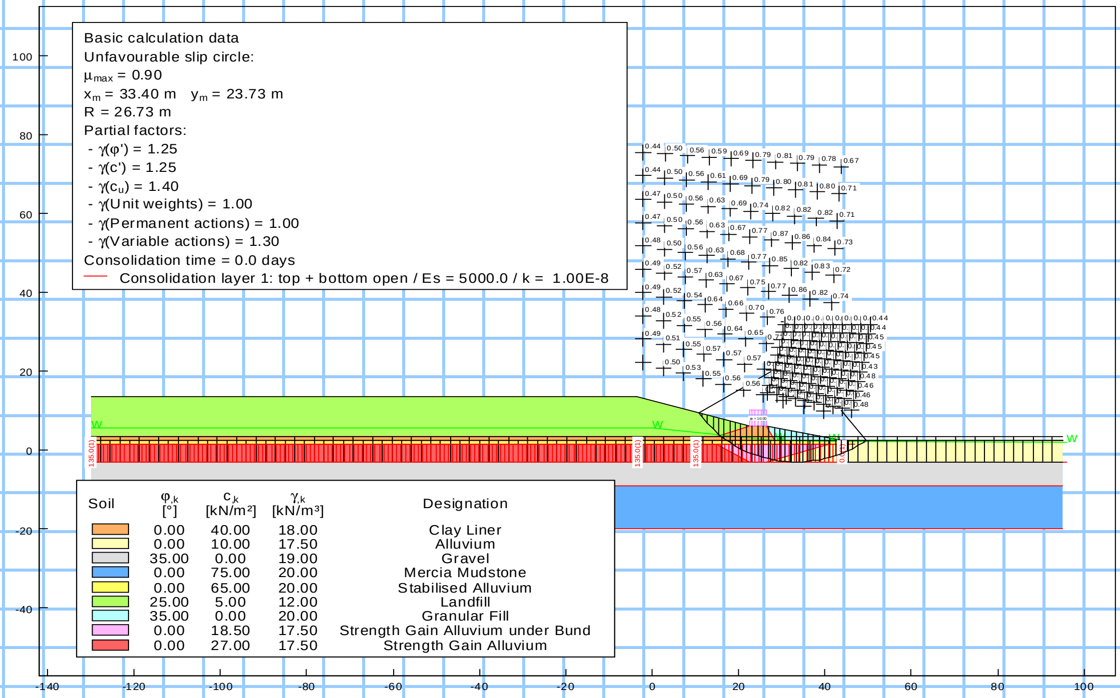
In this assessment the strength gain beneath the dam is assumed not to increase above 18.5kN/m^2 which is based on a bund 3m high which is equivalent to 5m of landfill (ie 3 x 20 = 5 x 12) so this is a conservative approach. In addition the strength gain beneath the landfill is applied as follows:



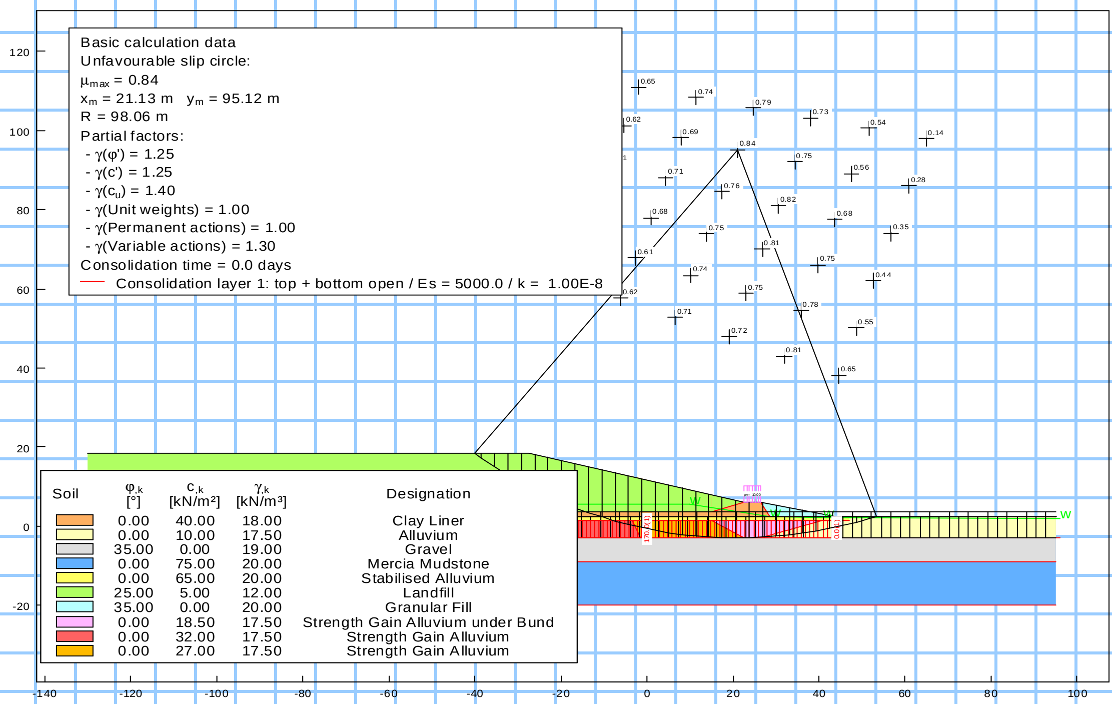
Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

For the landfill at 13.5m OD the maximum utilisation factor is 0.90



For the landfill at 18.5m OD the maximum utilisation factor is 0.84

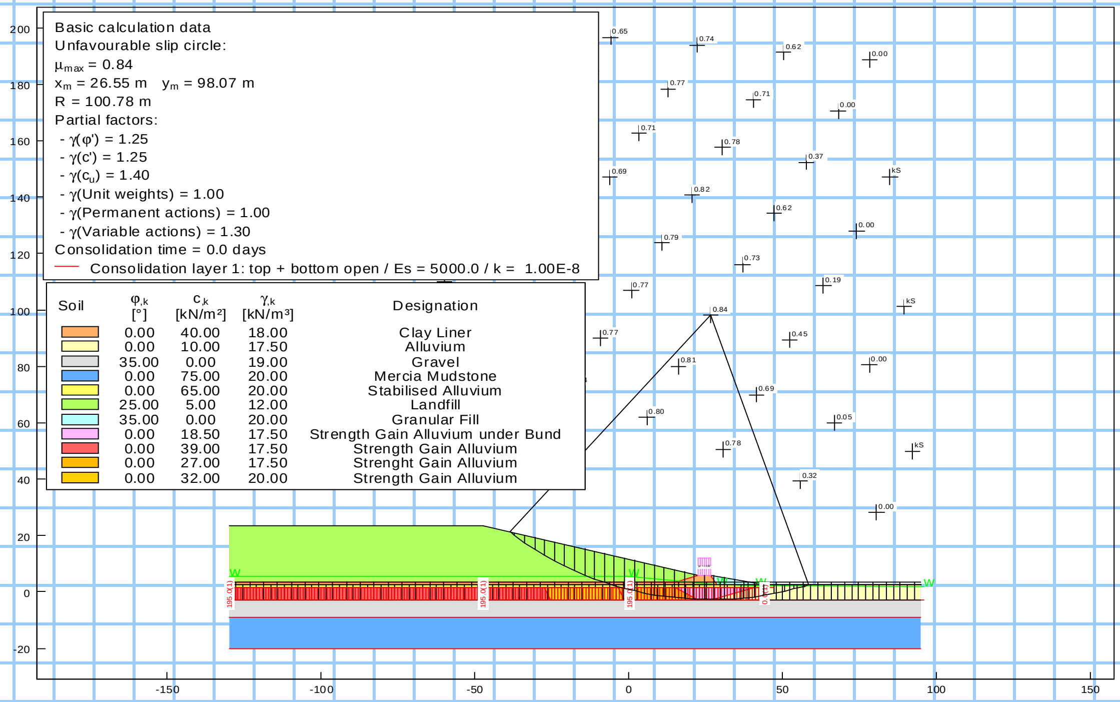


Calculations

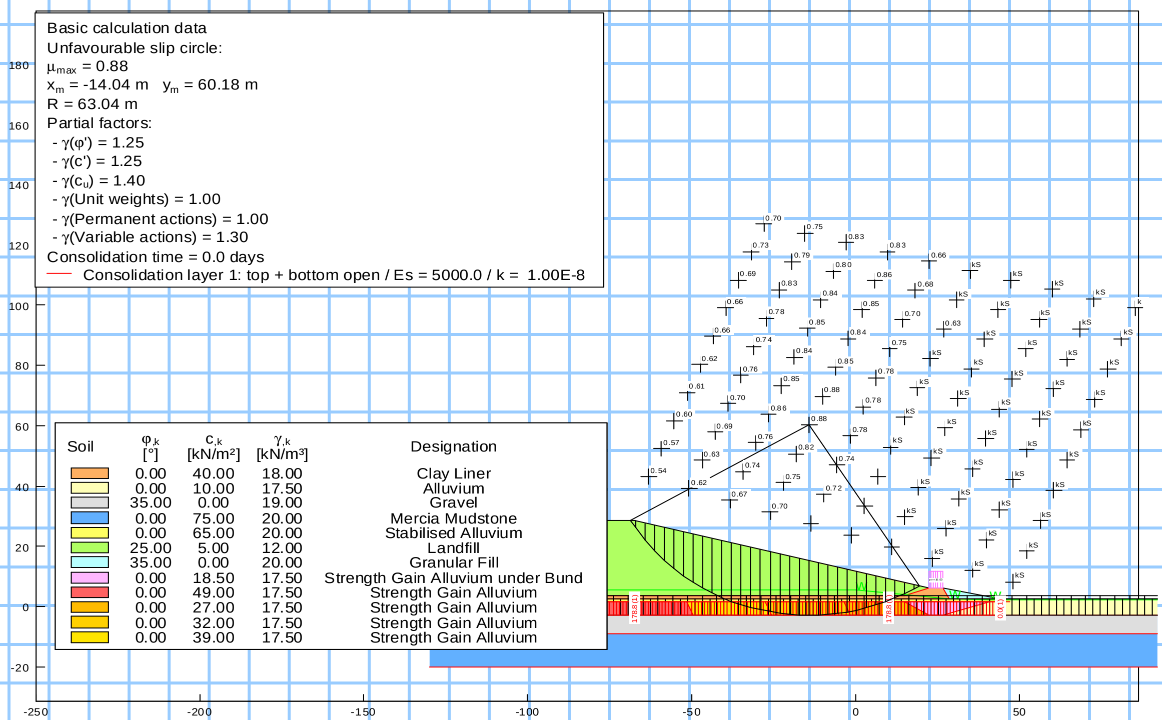
Project Title	Docksway Disposal Site: Area 2

Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

For the landfill at 23.5m OD the maximum utilisation factor is 0.84



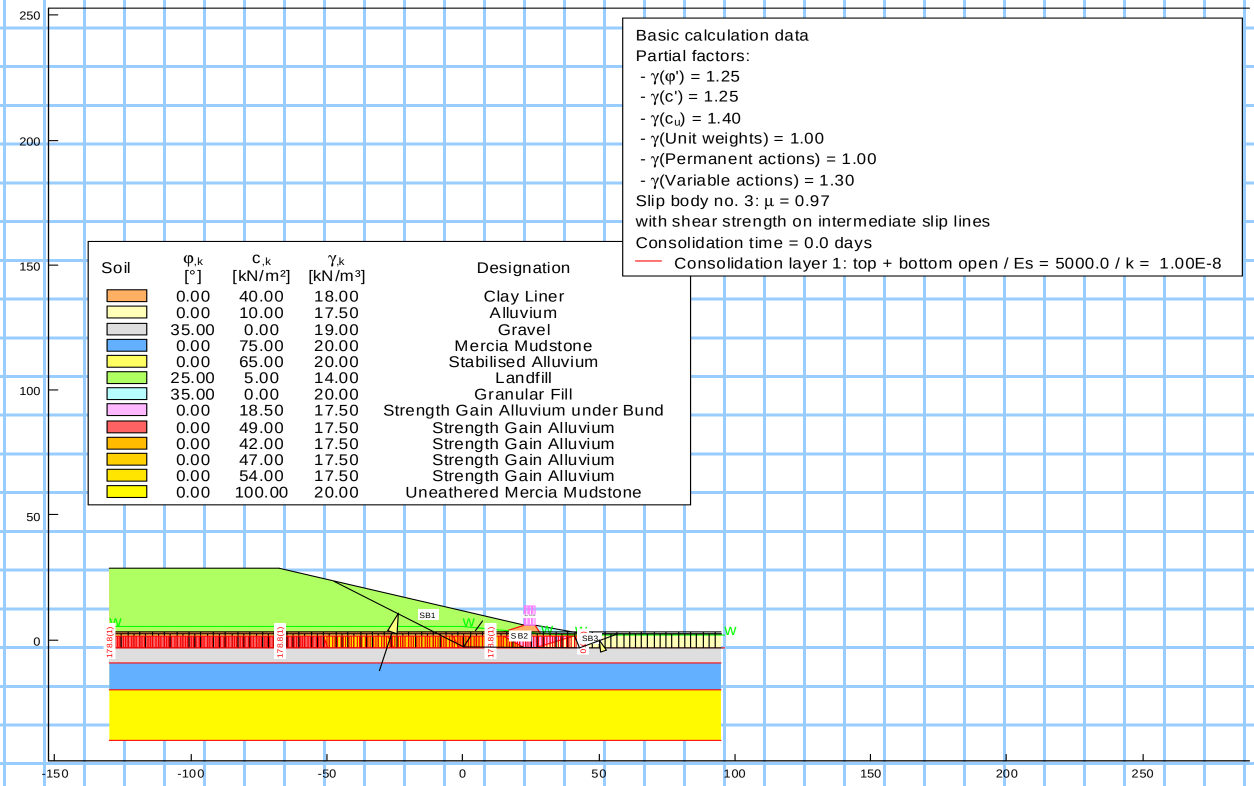
For the landfill at 28.5m OD the maximum utilisation factor is 0.88



Calculations

Project Title	Docksway Disposal Site: Area 2								
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

A check on the stability using a wedge analysis for the landfill at 28.5 m indicates that the utilisation factor for a wedge failure surface is 0.97



Sheet	New Bund 10	of	17

Calculations

Project Title	Docksway Disposal Site: Area 2

Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Based on the assessment of short term stability on the preceding pages there is potential that if landfilling commences at the current estimated rate then once the upper surface of the landfill in cell 3 has reached an elevation of 28.5m that for a 1:4 slope face utilisation factors are approaching unity and there is therefore a potential for failure of the landfill. On this basis it is recommended that landfilling is held in this area for a period before the landfill reaches an elevation of 28.5m OD to let porewater pressures dissipate and for the strength gain to improve.

The assessment presented on the following pages assumes that the dissipation in porewater pressure during a hold period will result in a corresponding strength gain

Now assess rate of consolidation and hence dissipation of excess porewater pressure

$$T_v = \frac{C_v t}{d^2}$$

C_v Taken as 0.5 m^2/year based on previous assessment.

Taking values of t as follows and based on a hold landfill level of 23.5m OD with a piezometric level* of 16mOD

t	T_v	Percentage Consolidation U%	Reduced Piezometric Level (mOD)	Difference in Piezometric Level
6 months	0.04	23%	12.32	3.68
12 months	0.08	32%	10.88	5.12
18 months	0.12	40%	9.60	6.40
24 months	0.16	47%	8.48	7.52

* This is the porewater pressure within the alluvium beneath the landfill and not a phreatic surface level

Note Reduced Piezometric Level = Piezometric Level from graph on New Bund 6 x (100-U%)

Also given that strength gain is a function of excess porewater pressure and overburden pressure based on the original assessment of strength gain presented in the original Stability Assessment. Then the equivalent increases in the vertical pressure to estimate the undrained shear strength from is equal to

(difference in piezemetric level x density of water/revised density of landfill) + height of landfill x increase in landfill density

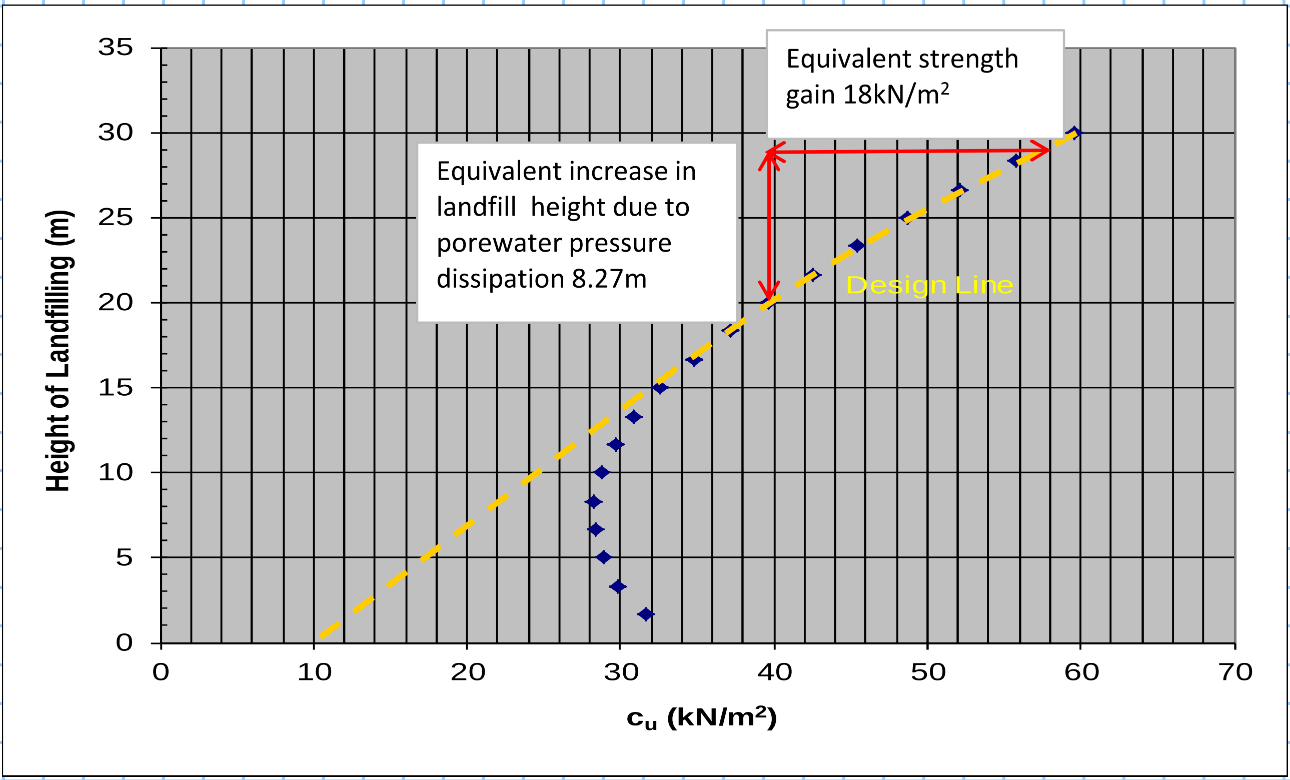
for the hold level at 23.5m OD the thickness of landfill is 20m therefore the equivalent increase in height used to calculate c_u after a 12 month hold period is

$$= \frac{5.12 \times 10}{12} + 20 \times 0.2$$

$$= 8.27 \text{ m}$$

Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015



For assessment use strength gain of 15kN/m^2 for each 5m lift of after the hold period which is less than the 18kN/m2 as indicated above and so conservative.

Now the porewater pressure will commence from a lower level so revise the excess porewater pressure for subsequent levels as follows:

For subsequent landfill levels use the following

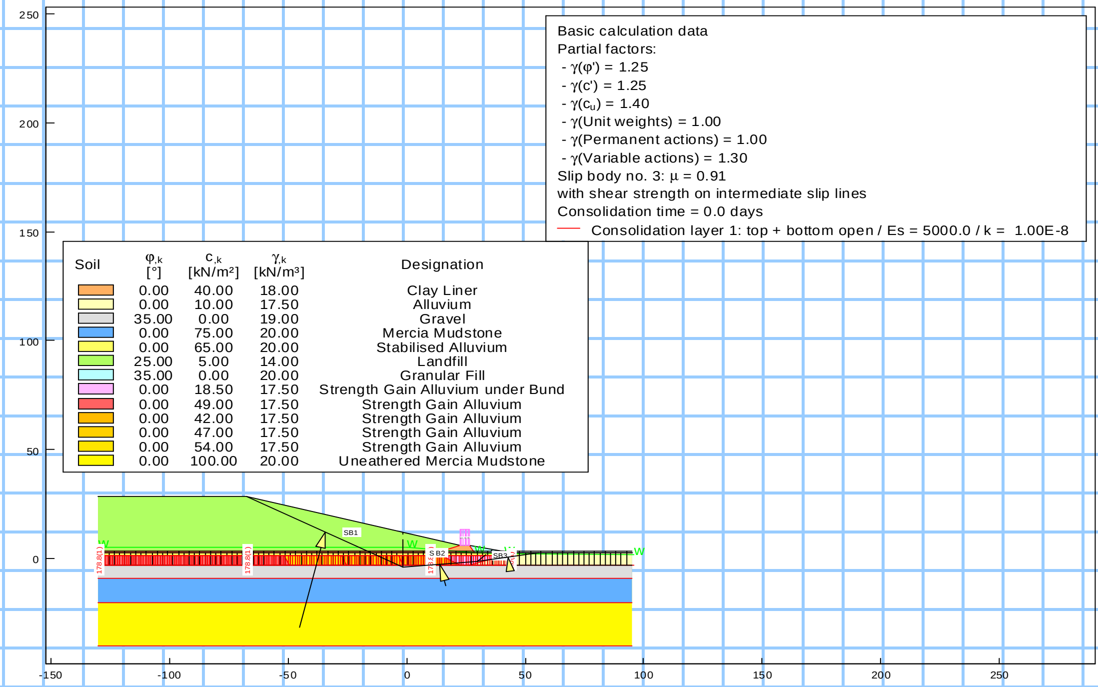
Landfill Level (mOD)	Piezometric Level (mOD)	Excess PWP (kN/m^2)	Equivalent Landfill Ht (m)	Strength Gain c_u (kN/m^2)
28.5	14.88	178.8	25	64
33.5	16.38	193.8	30	74
40.0	19.88	228.8	36.5	89

Calculations

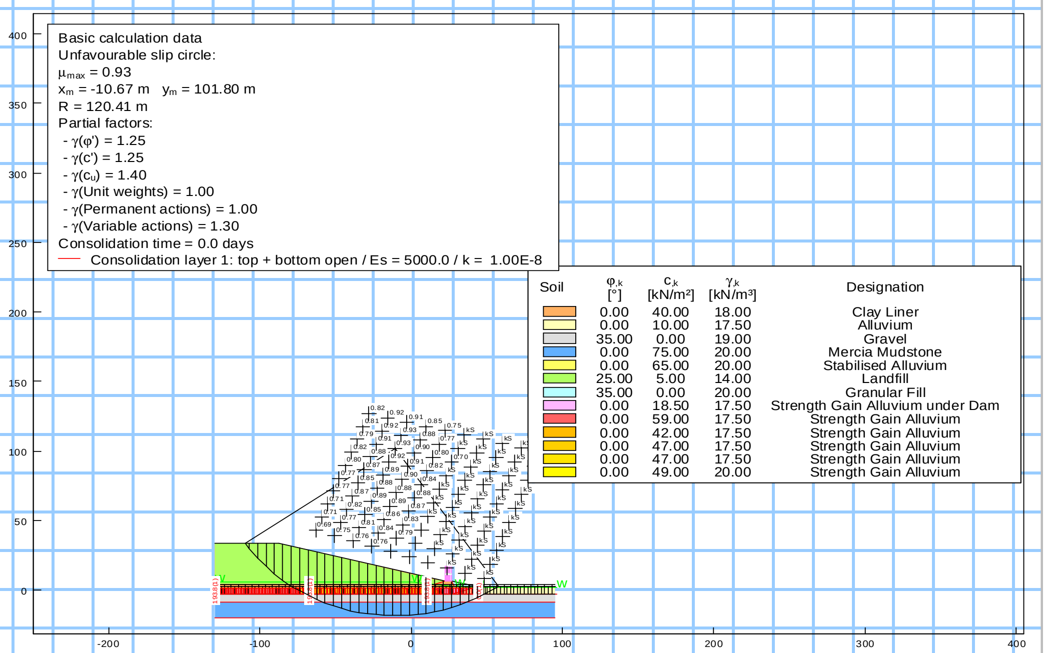
peterbrett

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

For the landfill at 28.5m OD after 12 month hold at 23.5m OD the maximum utilisation factor is 0.91 (Wedge)



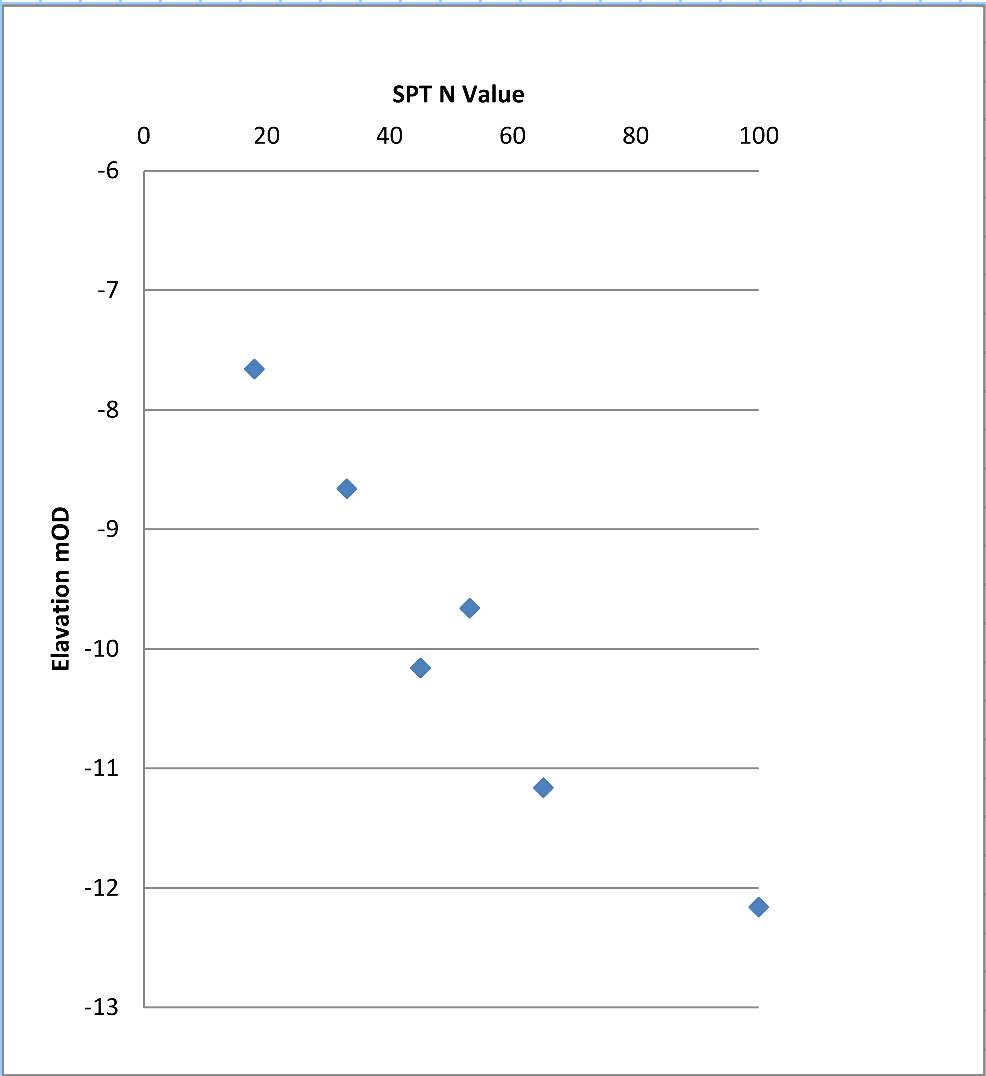
For the landfill at 33.5m OD after 12 month hold at 23.5m OD the maximum utilisation factor is 0.93



Note: As it is identified that deeper seated surfaces have the larger utilisation factors and therefore for further assessment it is assumed that the Mercia Mudstone undrained shear strength increases with depth as would be expected and as demonstrated by the increasing SPT values relative to elevation. see nex sheet. To model this the shear strength of the Mercia Mudstone is taken as 100kN/m² below an elevation of -20mOD. This is conservative based on SPT N values as the SPT values are all greater than 50 below and elevation of - 11mOD

Calculations

Project Title	Docksway Disposal Site: Area 2								
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015



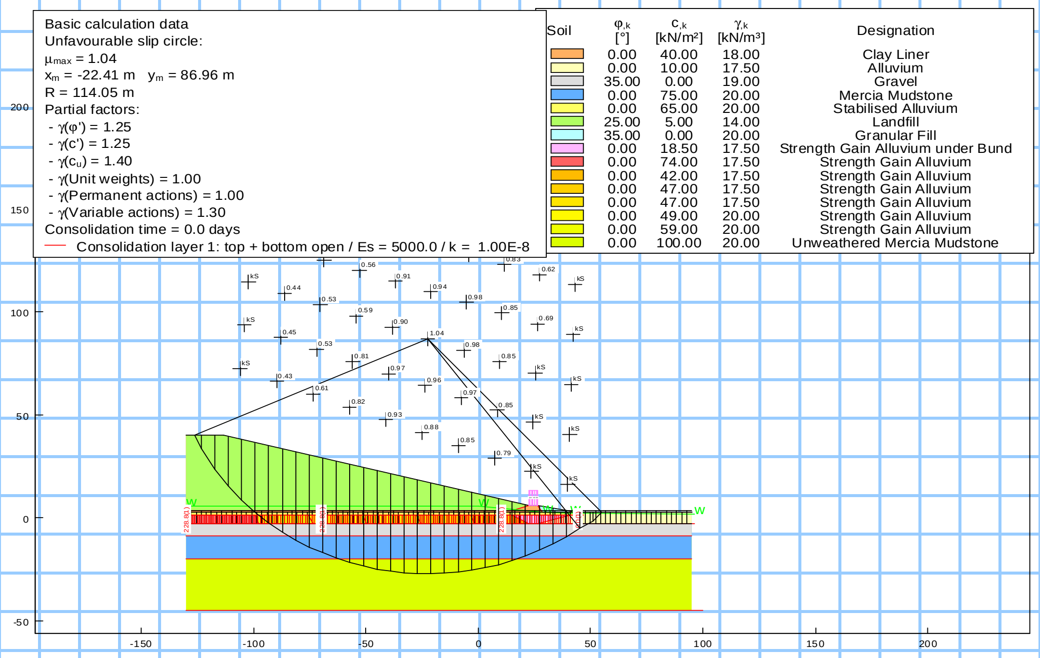
Graph of SPT -v- Elevation for Mercia Mudstone

Sheet	New Bund 14	of	17

Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

For the landfill at 40m OD after 12 month hold at 23.5m OD the maximum utilisation factor is 1.04



Given that the utilisation factor is 0.93 when the landfill level is 33.5mOD and increases to 1.04 when the landfill elevation is at 40m it would probably be sensible to consider another hold period to allow for a little more dissipation of porewater pressure and strength gain.

If a hold of 6 months was applied then the equivalent reduction in piezometric head would be 23% based on Sheet New Dam 10. If this hold is applied when the landfill is at an elevation of 33.5mOD the reduced piezometric level would be 16.38 x (1 - 0.23) = 12.6mOD

this is a reduction of 16.83 - 12.6 = 4.23m

Now applying the same logic as used on Sheets New Dam 10 and 11

the equivalent landfill height increase would be $= \frac{4.23 \times 10}{12} + 20 \times 0.2$

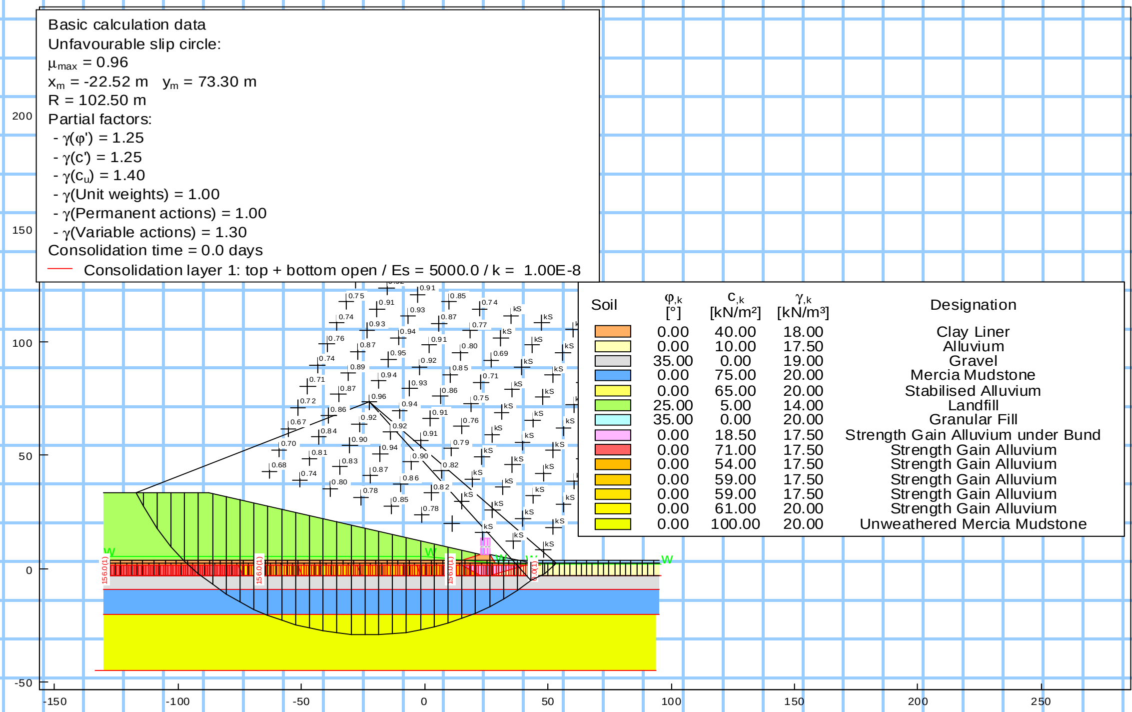
$= 7.53$ m

this equates to an increase in c_u of about 12kN/m^2 so use 10kN/m^2 for assessment and apply this to the 33.5mOD and 40mOD stability assessment.

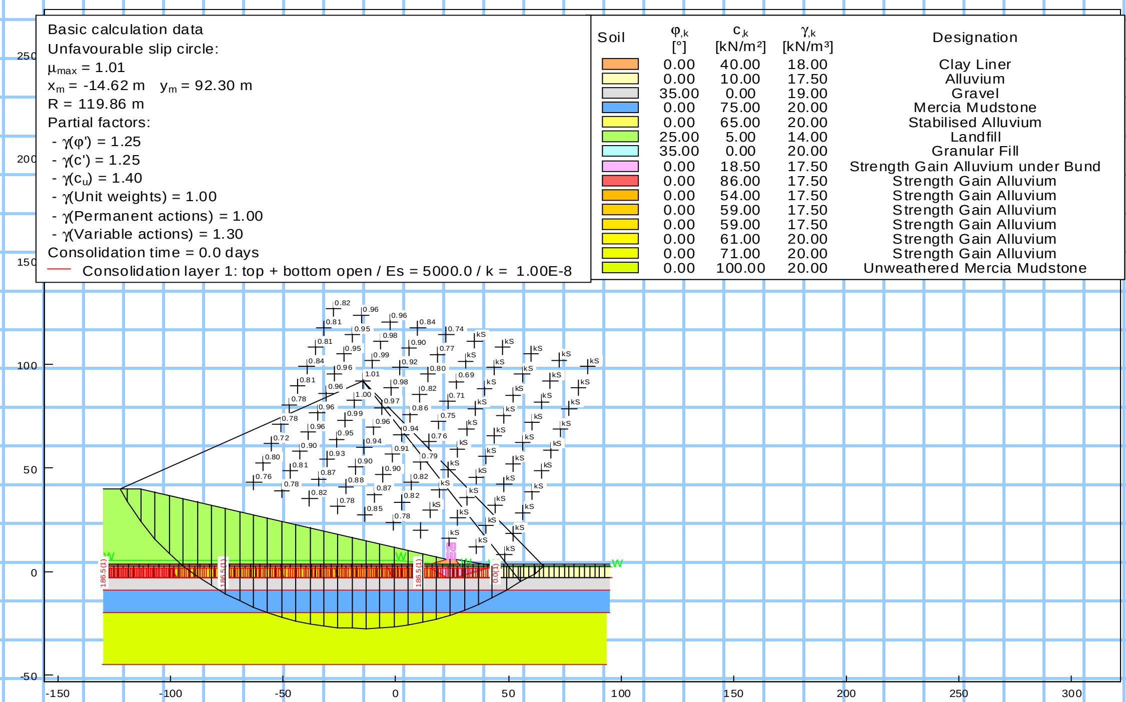
Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

For the landfill at 33.5m OD after two hold periods the maximum utilisation factor is 0.96



For the landfill at 40m OD after two hold periods the maximum utilisation factor is 1.01

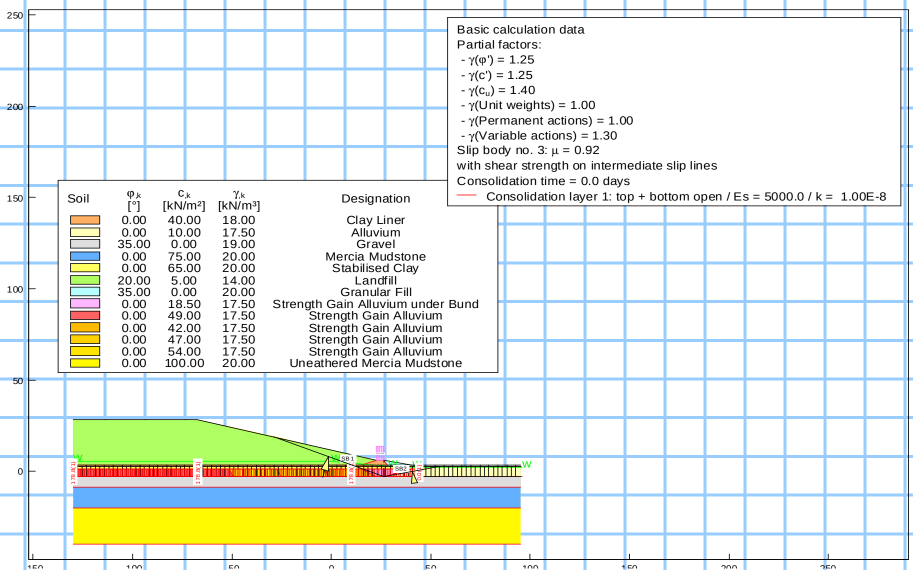


Calculations

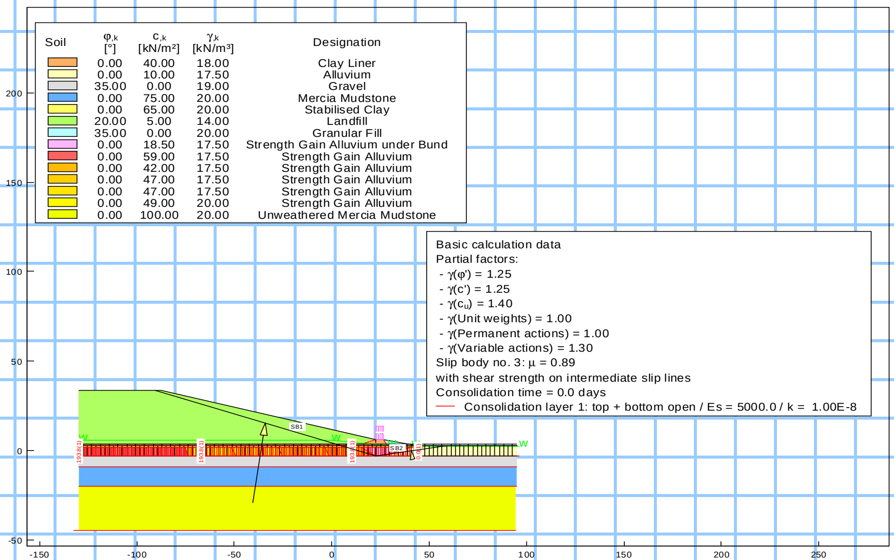
Project Title	Docksway Disposal Site: Area 2								
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

It is apparent from the preceding assessment that even if a second hold period is applied at 33.5mOD level that the utilisation factors for the landfill at 40m OD is still greater than unity. In addition it is observed that the critical failure surface is much deeper than for the lower height of landfill. In consideration of this it would be sensible to cease land filling at an elevation of around 33m OD in Cell 3 as above this level there is considered a risk to the landfill slopes where they pass through the new bunds on the edge of this cell

As a check on the circular stability assessment undertaken two wedge analysis have been undertaken for the landfill at 28.5m OD and for the landfill at 33.5m OD after a 12month hold period both these analysis indicate that the utilisation factors for the wedge assessment are lower than for the corresponding circular assessment and from this it can be assumed that the circular analysis is the more critical assessment.



Landfill at 28.5m OD wedge
Max utilisation factor = 0.92



Landfill at 28.5m OD wedge
Max utilisation factor = 0.89

Sheet	New Bund 17	of	17

Appendix 8 Extrusion Assessment

Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

The following sheets present an assessment of the risk of extrusion.

Where there are particular soft soils beneath any earthworks there is a risk that the soft soils could spread laterally outwards form beneath the earthworks ie. Extrude.

The geometry of an embankment induces outward shear stresses within the soft soils supporting the embankment and where the supporting soils are of limited strength and at limited depth the induced shear stresses can result in these supporting soils spreading laterally from beneath the embankment resulting in an extrusion failure.

To prevent the risk of extrusion failure the length of the embankment side slope L_s as indicated on the next sheet should be great enough to prevent mobilisation of the outward shear stresses. This length needs to be sufficiently long that adequate lateral confinement over a sufficient surface area at the underside of the embankment is achieved. To achieve the following conditions need to be satisfied where there is no reinforcement at the base of the embankment:

The overall shearing resistance on the underside of the embankment should be sufficient to resist the lateral loads developed in the foundation soil.

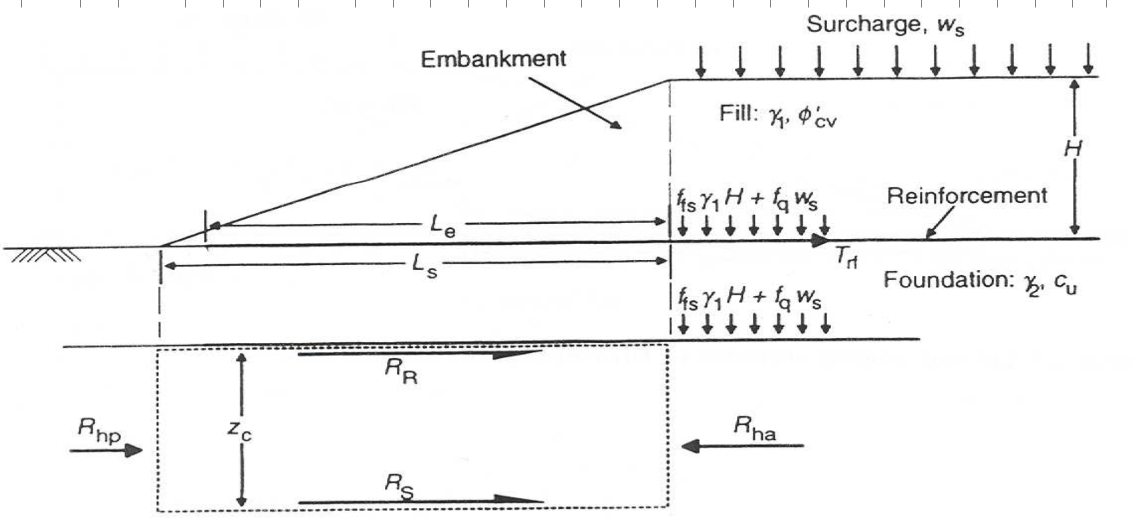
The figures on the following sheet indicate the various stresses such that

R_{ha} = Factored horizontal force causing foundation extrusion
R_{hp} = Factored horizontal force due to passive resistance of the foundation
R_S = Factored horizontal force due to the shear resistance of the foundation soil at depth z_c
R_R = Factored horizontal force due to the shear resistance of the foundation soil at the underside of the foundation

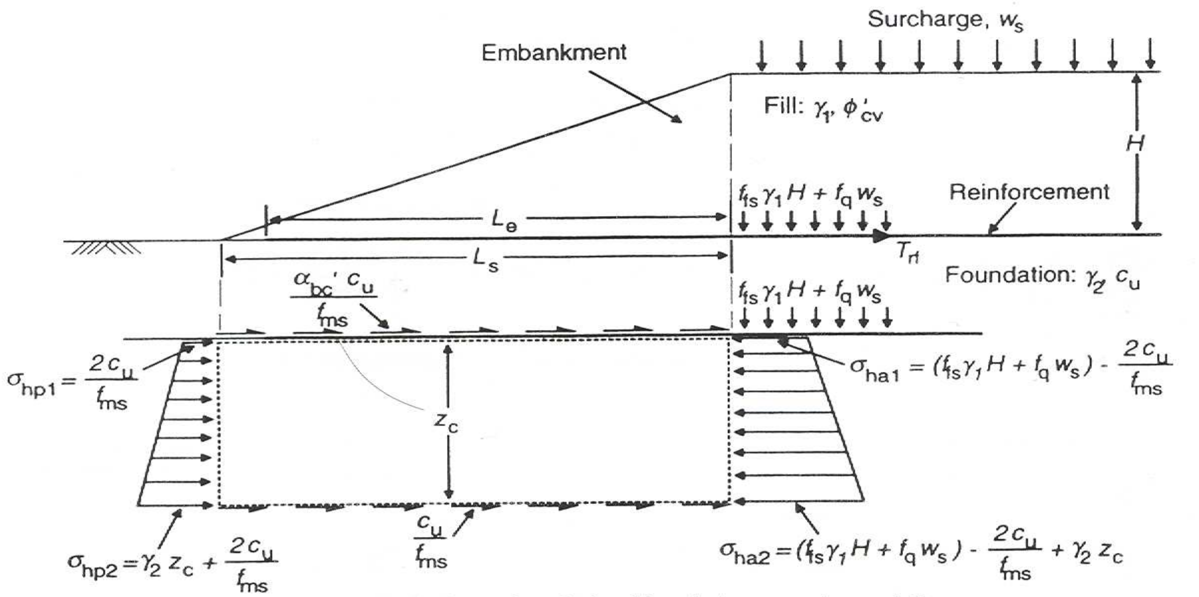
This is from BS 8006:2010 Part 1 Code of practice for strengthened/reinforced soils and other fills it should be noted in this assessment no reinforcement is included.

Calculations

Project Title	Docksway Disposal Site: Area 2								
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015



a) Force components in foundation extrusion stability analysis



b) Undrained analysis of foundation extrusion stability

Analysis of Foundation Extrusion Stability
Based on BS8006:1995 Section 8

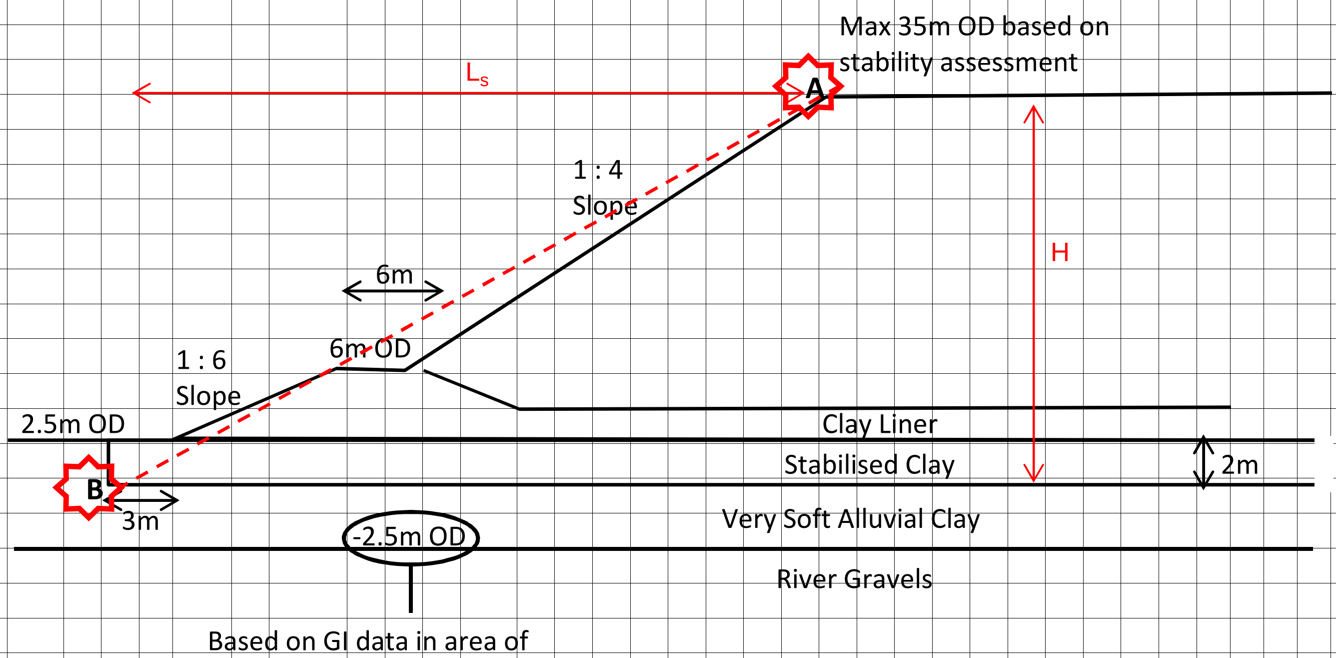
To prevent foundation extrusion R_{ha} < or = R_{hp}+R_s+RR

R_{ha}	=	Factored horizontal force causing foundation extrusion
R_{hp}	=	Factored horizontal force due to passive resistance of the foundation
R_S	=	Factored horizontal force due to the shear resistance of the foundation soil at depth z_c
R_R	=	Factored horizontal force due to the shear resistance of the foundation soil at the underside of the foundation

Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

For the new dam Sections check the potential for extrusion applying the following ground model



Make H = Vertical distance between points A and B = 34 m

Make L_s = Horizontal distance between points A and B = 155 m

Z_c = 3 m

For assessment use c_u = 10 kN/m^2

Calculations

Project Title	Docksway Disposal Site: Area 2									
Project No	14739	156	By	LJT	Checked	AD	Date	09	04	2015

Test for base extrusion — See Sheet Extrusion 1 for explanation of analysis

Using undrained shear strength of 10 kN/m^2

Variables

z_c	=	3			m					
L_s	=	155			m					
c_u	=	10			kN/m^2					
f_{ms}	=	1								
f_{fs}	=	1.3								
f_q	=	1.3								
γ_1	=	12			kN/m^3					
γ_2	=	17.5			kN/m^3					
H	=	34			m					
W_s	=	0			kN/m^2					
α_{bc}	=	0.8								
σ_{hp1}	=	20	σ_{ha1}	=	510	c_u/f_{ms}	=	10		
σ_{hp2}	=	72.5	σ_{ha2}	=	563	$\alpha_{bc}\ c_u/f_{ms}$	=	8		
R_{hp}	=	139	R_{ha}	=	1610	R_r	=	1240	R_s =	1550

f_{ms}, f_{fs} and f_q are partial factors in accordance with Table 22 of BS8006:2010

R_{hp}, R_{hp}, R_r and R_s calculated in accordance with the formula given in Section 8 of BS 8006:2010

R_{hp} + Rr + Rs = 2928.8 **R_{ha} = 1609.95**

Rha < Rhp + Rr + Rs Therefore OK

Stability OK also this is conservative assessment as the very soft alluvium will gain strength

A sensitivity analysis has been undertaken to assess the maximum thickness z_c that is possible before extrusion is an issue this is presented on the following sheet.

Calculations

Project Title	Docksway Disposal Site: Area 2							
Project No	14739	156	By	LJT	Checked	AD	Date	09 04 2015

Test for base extrusion — See Sheet Extrusion 1 for explanation of analysis

Using undrained shear strength of 10 kN/m^2

Variables

z_c	=	5.7			m					
L_s	=	155			m					
c_u	=	10			kN/m^2					
f_{ms}	=	1								
f_{fs}	=	1.3								
f_q	=	1.3								
γ_1	=	12			kN/m^3					
γ_2	=	17.5			kN/m^3					
H	=	34			m					
W_s	=	0			kN/m^2					
α_{bc}	=	0.8								
σ_{hp1}	=	20	σ_{ha1}	=	510	c_u/f_{ms}	=	10		
σ_{hp2}	=	120	σ_{ha2}	=	610	$\alpha_{bc}\ c_u/f_{ms}$	=	8		
R_{hp}	=	398	R_{ha}	=	3193.6	R_r	=	1240	R_s =	1550

f_{ms}, f_{fs} and f_q are partial factors in accordance with Table 22 of BS8006:2010

R_{hp}, R_{hp}, R_r and R_s calculated in accordance with the formula given in Section 8 of BS 8006:2010

R_{hp} + Rr + Rs = **3188.3** **R_{ha}** = **3193.6**

Rha > Rhp + Rr + Rs Therefore potential for instability

It is found that for a thickness z_c of 5.7m that extrusion stability may be an issue it is however unlikley that at the locations of the bunds that the thickness of soft Alluvium will be anywhere near this so there is not considered to be an issue with extrusion.

Appendix 9 Asbestos Sub Cell Stability Assessment

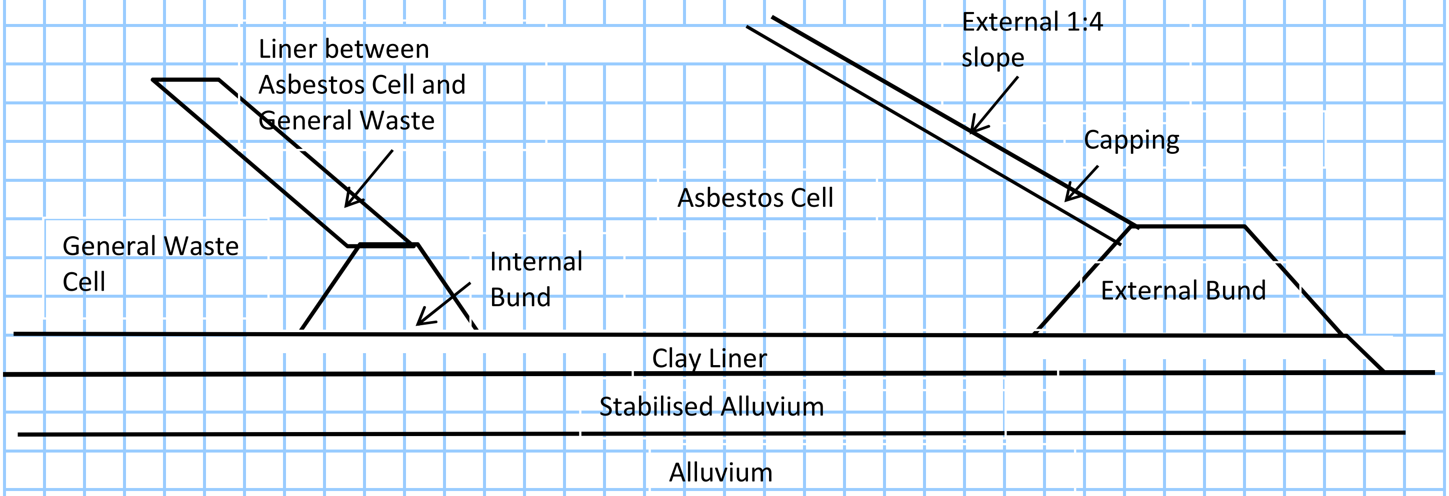
Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015										
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015	

It is proposed to form an Asbestos Sub-Cell within part of the Cell 3 development and the assessment presented in this Appendix considers the stability of the slope between the Asbestos Sub-Cell and the general waste within the remainder of Cell 3.

To form an asbestos cell it is a requirement that the asbestos containing waste is separated from the the general waste with a liner. The intention is that this liner will be formed of Clay which will be placed as landfilling progresses.

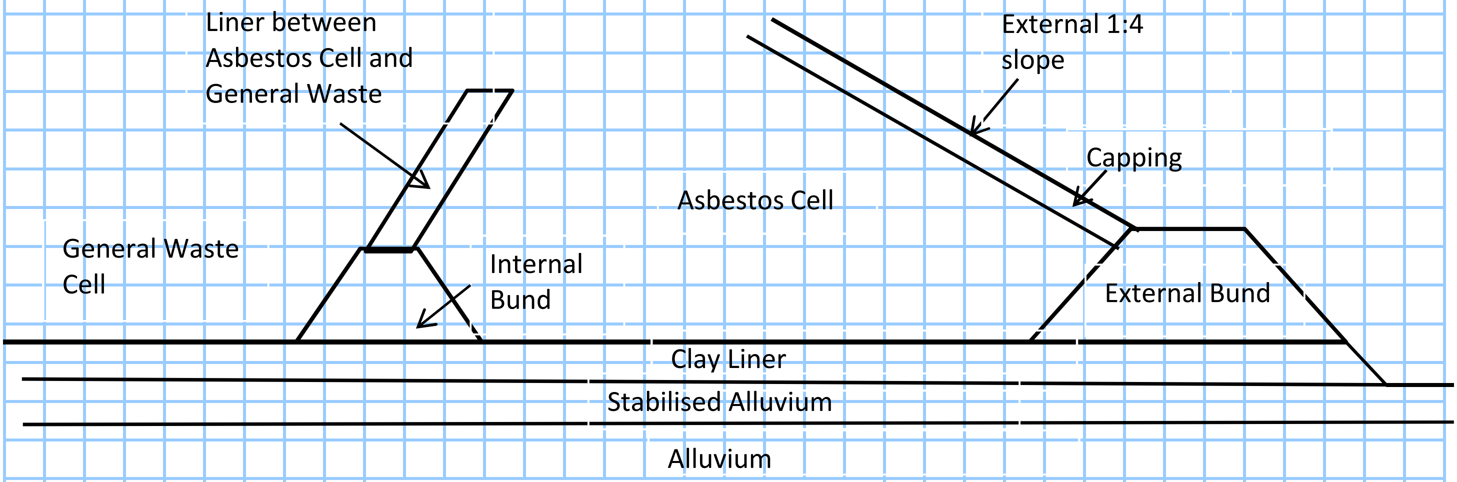
Currently the likely rate of filling of the Asbestos Sub-Cell is unknown and in this respect there could be three potential scenarios of filling that may be progressed as indicated on the following schematic sections.



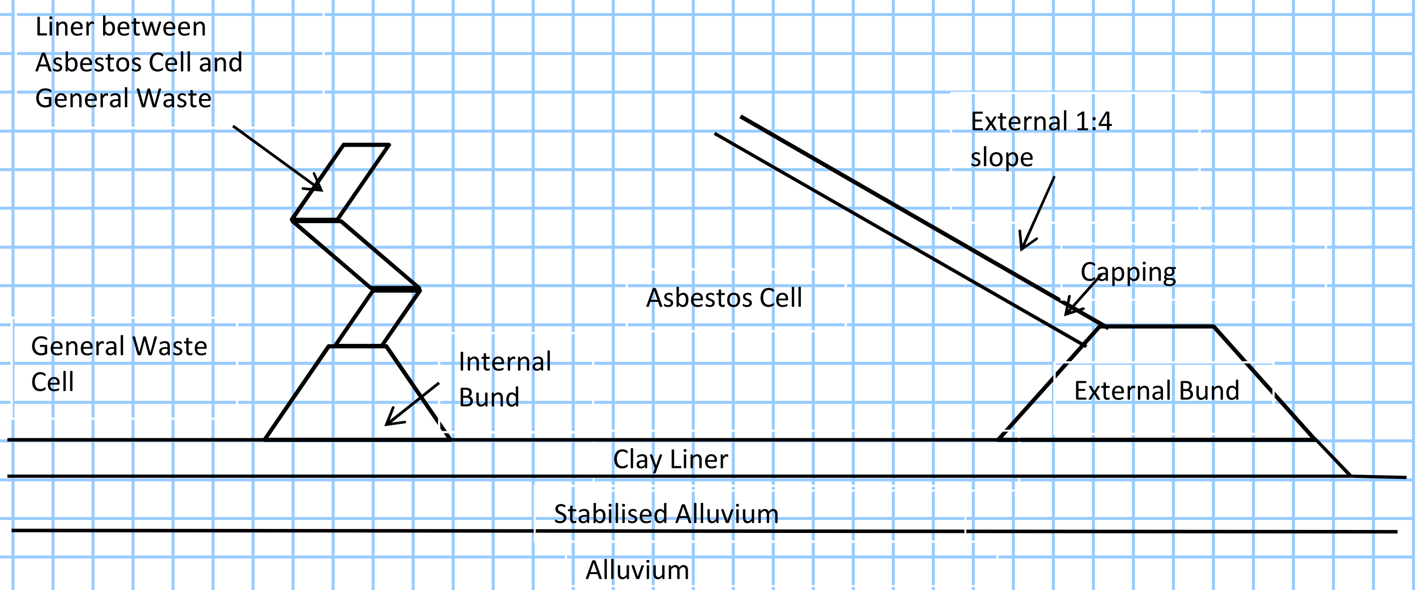
Scenario 1 Asbestos Cell Formed Over General Waste Cell

Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015									
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015



Scenario 2 Asbestos Cell Formed Below General Waste Cell



Scenario 3 Asbestos Cell and General Waste Cell formed as series of steps overlapping each other to suit rate of filling

Calculations

peterbrett

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015									
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

With any of the scenarios indicated on the preceding sheets the most critical factors will be the angle of the liner between the asbestos waste and general waste and the difference in height between the asbestos cell and and the general waste cell. In consideration of this the following assessment is undertaken to assess the most appropriate angle at which a separating liner can be formed and what the greatest difference in height can be between the differing types of waste. From this a recommendation can be made to cover all three scenarios.

The waste materials and in particular the clay forming the day cover in the asbestos cell and liner between the asbestos cell and general waste cell will be placed at a moisture content most probably some way below fully saturated and it will take a significant amount of time for these engineered clay mateials to reach fully saturated conditions.Ttherefore fully drained effective stress condtions are not considered appropriate for this assessment. Therefore the assessment presented below considers only the short term undrained stability.

Asbestos containing waste will not be compacted as it is placed in the cell and in this respect the density of asbestos containing waste will be lower than general waste. In addtion it may be that asbestos waste could be expected to include more fibrous material which would potentially enhance the shear strength en-masse, however for the asbestos cell there will be a requirement for daily soil cover and it would be better to consider the waste in the asbestos cell as having a shear strength of the cover soil without any benefit to the shear strength due to fibers.

The cover soil could comprise low strength clays and in consideration of this an undrained shear strength c_u = 25kN/m^2 is adopted this is a very conservative value.

The intention is that the asbestos cell is divided into much smaller sub-cells that will only be filled on a weekly basis such that the thickness of asbestos containing waste will be large compared to the cover soils with the ratio of asbestos waste thickness to cover soil thickness of about 2.5 : 1.

Based on Department of Environment Industrial profile for Asbestos the density of asbestos materials can range from 7kN/m^3 for insulating board to 16kN/m^3 for fully compressed flat sheets asbestos lagging would be much lower than7kN/m^3. In consideration of this and the fact that the asbestos waste will not be compacted then for the asbestos waste a conservative value for the bulk density of 13.5kN/m^3 is adoped for the asbestos waste. However for the soil cover a density of 18kN/m^3 is considered approriate. Based on the height ratio of 2.5:1 Asbestos to Soil then an enmass density for the material in the asbestos waste cell of would be 14.8kN/m^3 therefore a value oif 15kN/m^3 is adopted in this assessment.

Calculations

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There is a potential that during the filling that there may be perched leachate in the wastes and this is modelled as a phreatic surface equivalent to one third the slope height considered in each case analysed this is considered to be a conservative approach as leachate levels should be controlled within the landfill mass to a level below the upper level of the internal bund.

The following table indicates the geotechnical strength parameters used in this assessment

		Undrained	
Material	γ_b	ϕ_u	c_u
	kN/m^3	Degrees	kN/m^2
Landfill	12	25	5
Asbestos Waste	15	0	25
Clay Liner	20	0	40
Alluvium	17.5	0	10
Stabilised Alluvium	20	0	65
Gravel	19	35	0
Mudstone	20	0	75

Note no account is taken of any strength gain in the underlying alluvium in this assessment and therefore this assessment will be conservative.

Calculations

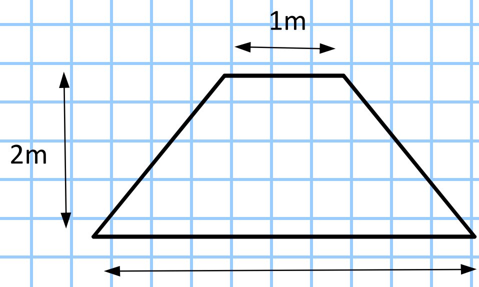
Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015									
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

The assessment considers the following cases to determine the most appropriate angle for the liner and critical height difference.

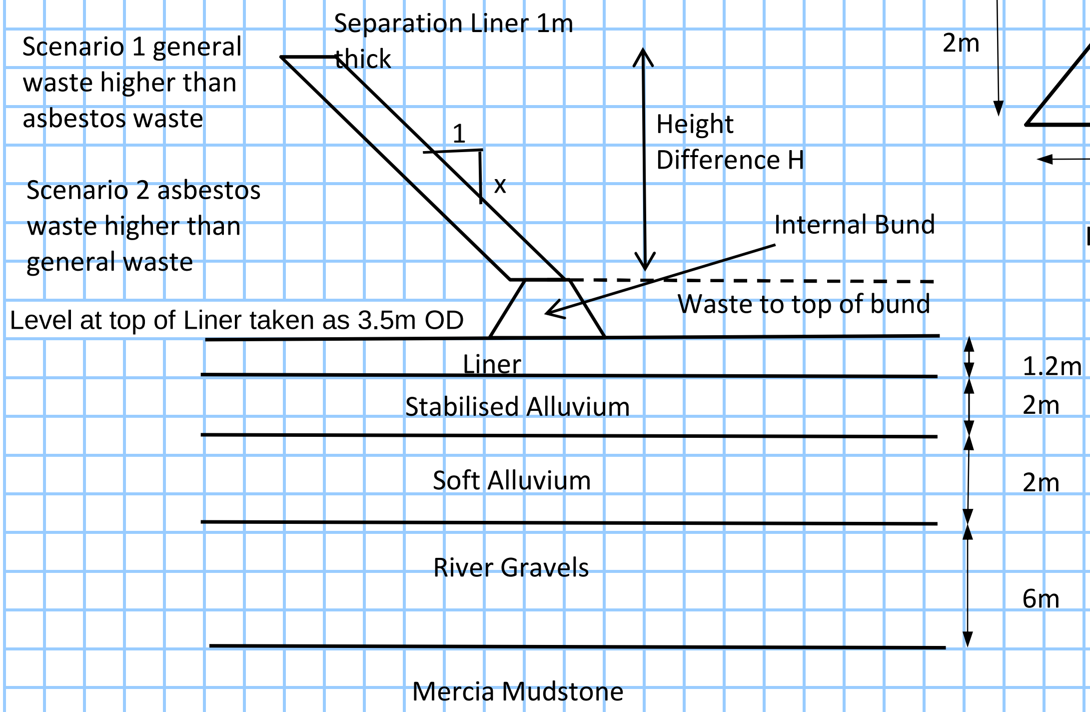
	Slope		
Difference in Height H	1v : 2h	1v : 1.5h	1v :1h
5m	Case 1	Case 2	Case 3
7.5m	Case 4	Case 5	Case 6
10m	Case 7	Case 8	Case 9

Note these cases are considered for two scanarios
Scenario 1 general waste higher than Asbestos waste
Scenario 2 Asbestos waste higher than General waste
From this a limiting Height and maximum angle of the interface will be detemined and can be applied to Scenario 3.

In this assessment it is assumed that the internal bund is formed of liner clay and that it has dimensions as indicated below on the generalised ground model used in the assessment. Also that there is asbestos waste and general waste up to the height of the top of the bund



Internal Bund dimensions



Note artesian water pressure is included in the River Gravels with a head equivalent to 4.4m OD

Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015

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Based on a circular analysis using GGU Stability the following utilisation factors were assessed for Scenario 1

	Slope		
Difference in Height H	1v : 2h	1v : 1.5h	1v :1h
5m	0.72	0.73	0.85
7.5m	0.91	0.92	1.01
10m	1.12	N/A	N/A

The GGU outputs are presented on the following sheets

N/A = Not Analysed

A check on the stability of a 7.5m high slope at 1:1.5 using a wedge analysis indicates a lower utilisation factor therefore it can be assumed from this that the circular method of analysis gives the worst case.

Based on a circular analysis using GGU Stability the following utilisation factors were assessed for Scenario 2

	Slope		
Difference in Height H	1v : 2h	1v : 1.5h	1v :1h
5m	0.58	0.61	0.73
7.5m	0.89	0.93	1.02
10m	1.17	N/A	N/A

The GGU outputs are presented on the following sheets

N/A = Not Analysed

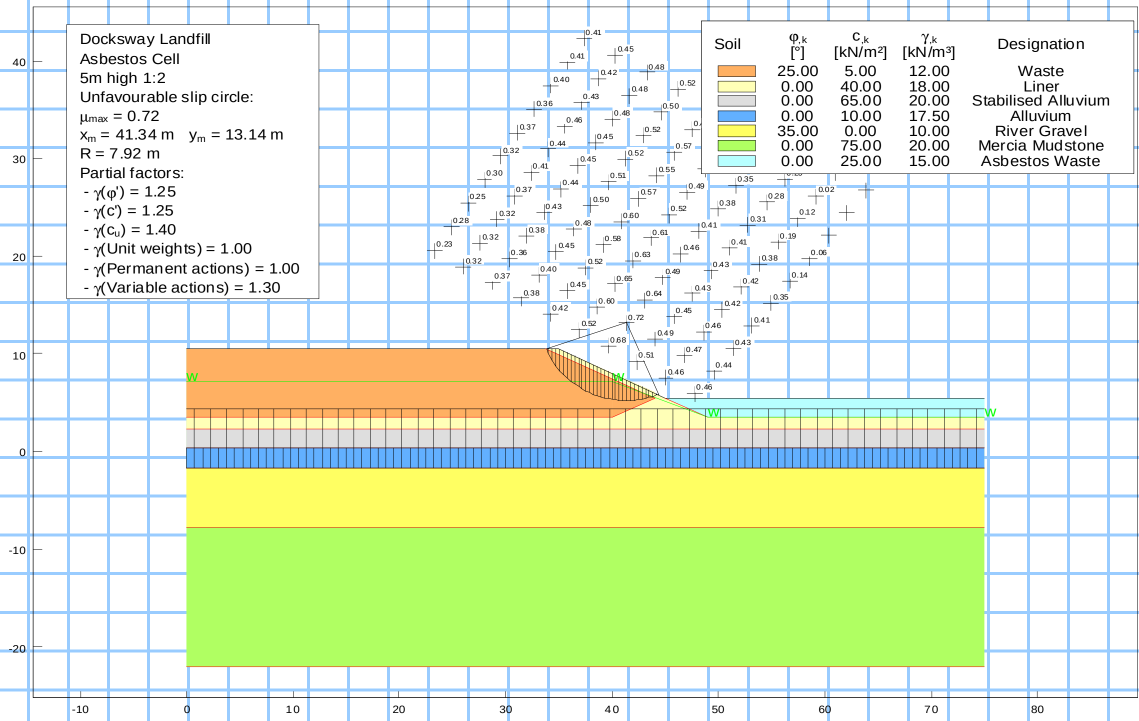
Based on this assessment the following limits are recommended for slope gradients and heights for the different scenarios:

SCENARIO	MAX SLOPE HEIGHT	MAX SLOPE GRADIENT
1	7.5m	1:1.5
2	7.5m	1:1.5
3	7.5m	1:1.5

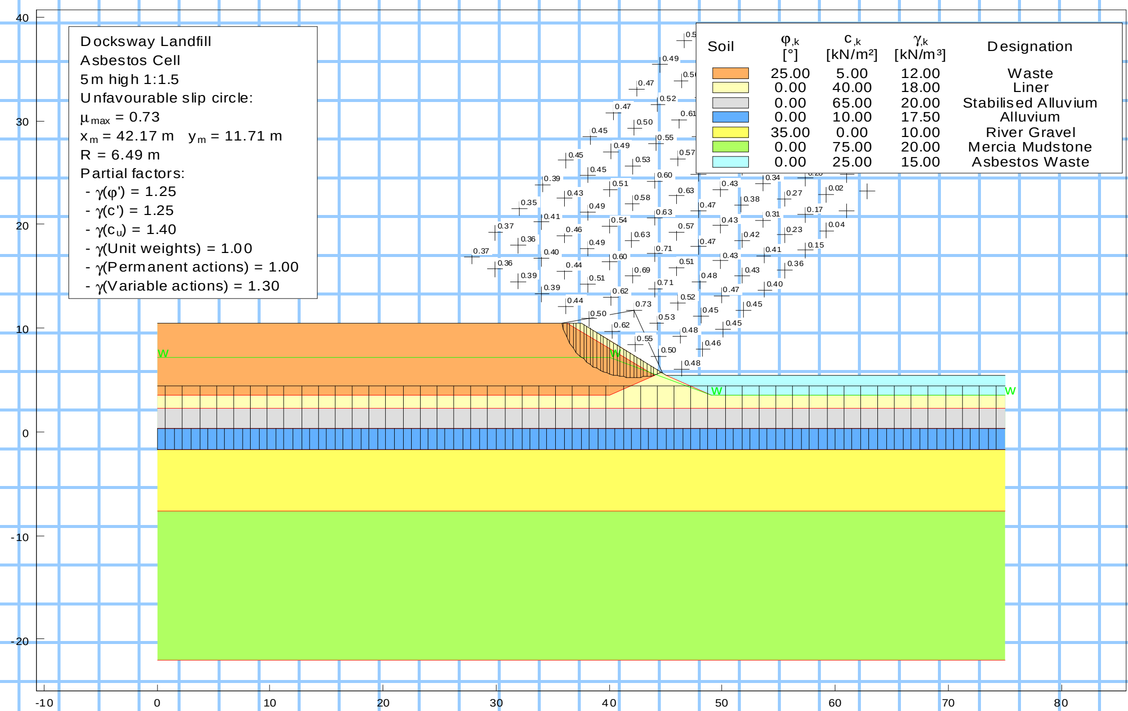
Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015								
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

5m high 1:2 slope circular max utilisation factor 0.72 Scenario 1



5m high 1:1.5 slope circular max utilisation factor 0.73 Scenario 1

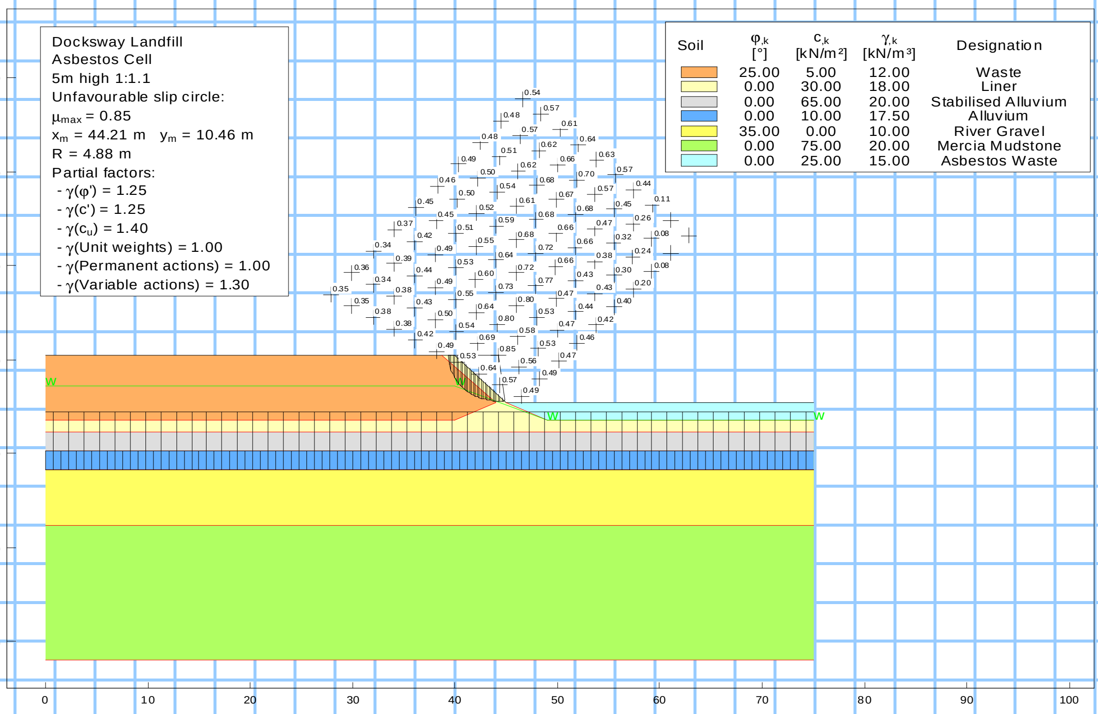


Calculations

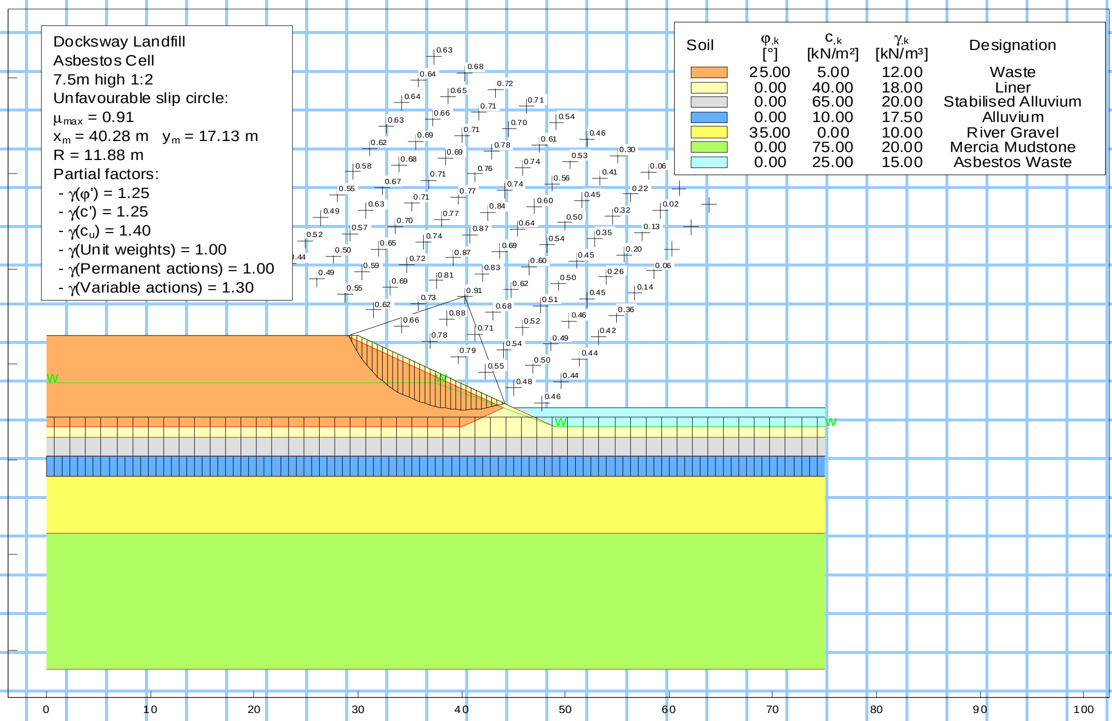
Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015

Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

5m high 1:1 slope circular max utilisation factor 0.85 — Scenario 1



7.5m high 1:2 slope circular max utilisation factor 0.91 — Scenario 1



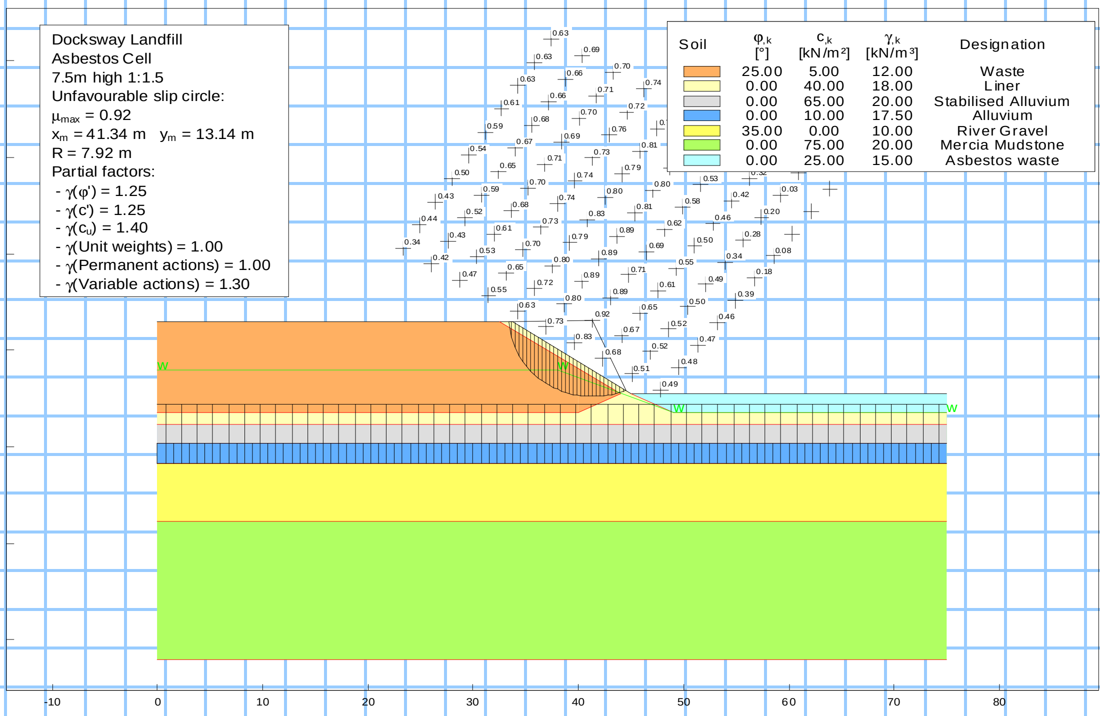
Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015

Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

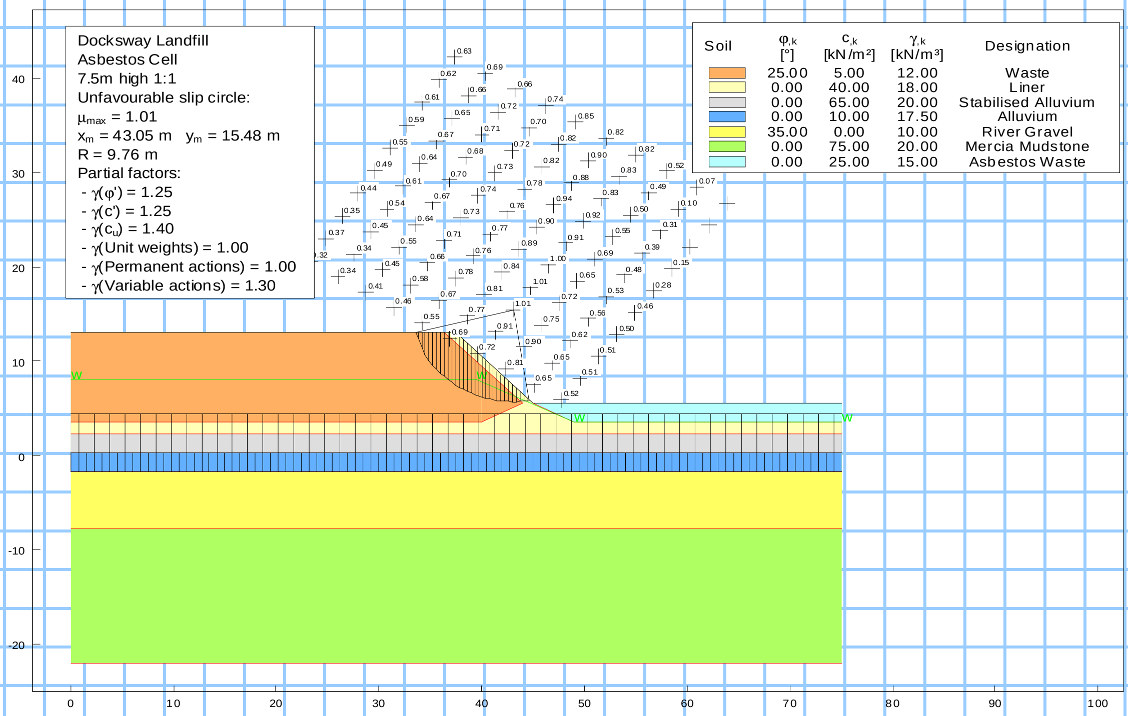
7.5m high 1:1.5 slope circular max utilisation factor 0.92

Scenario 1



7.5m high 1:1 slope circular max utilisation factor 1.01

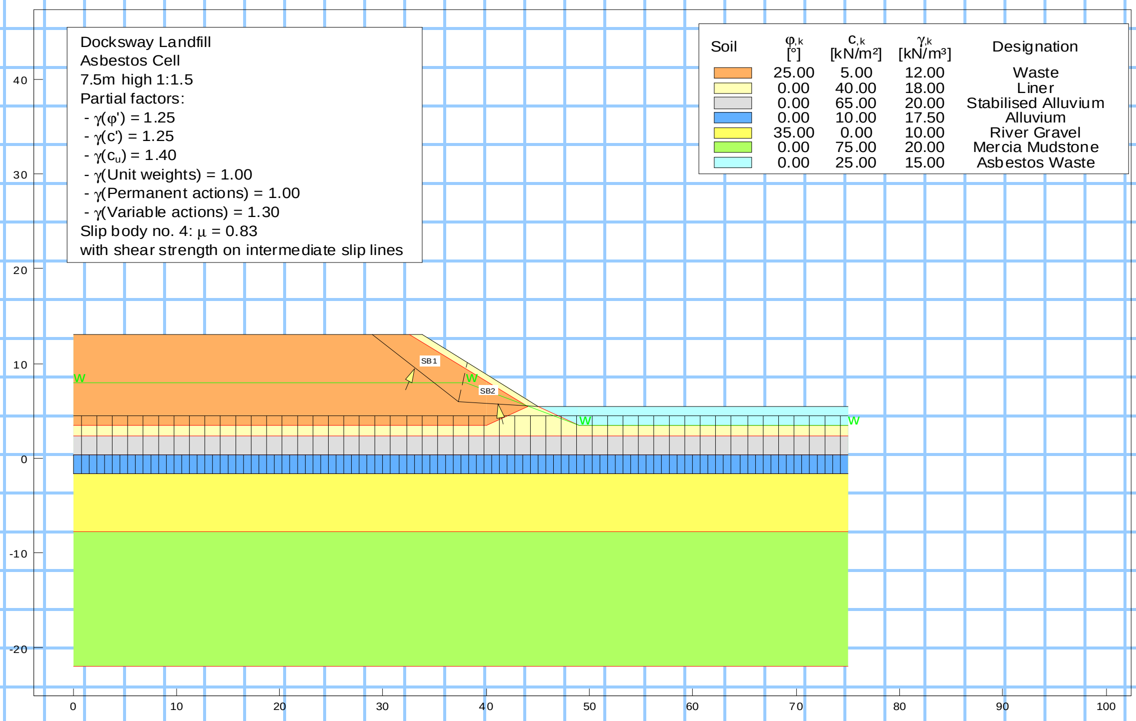
Scenario 1



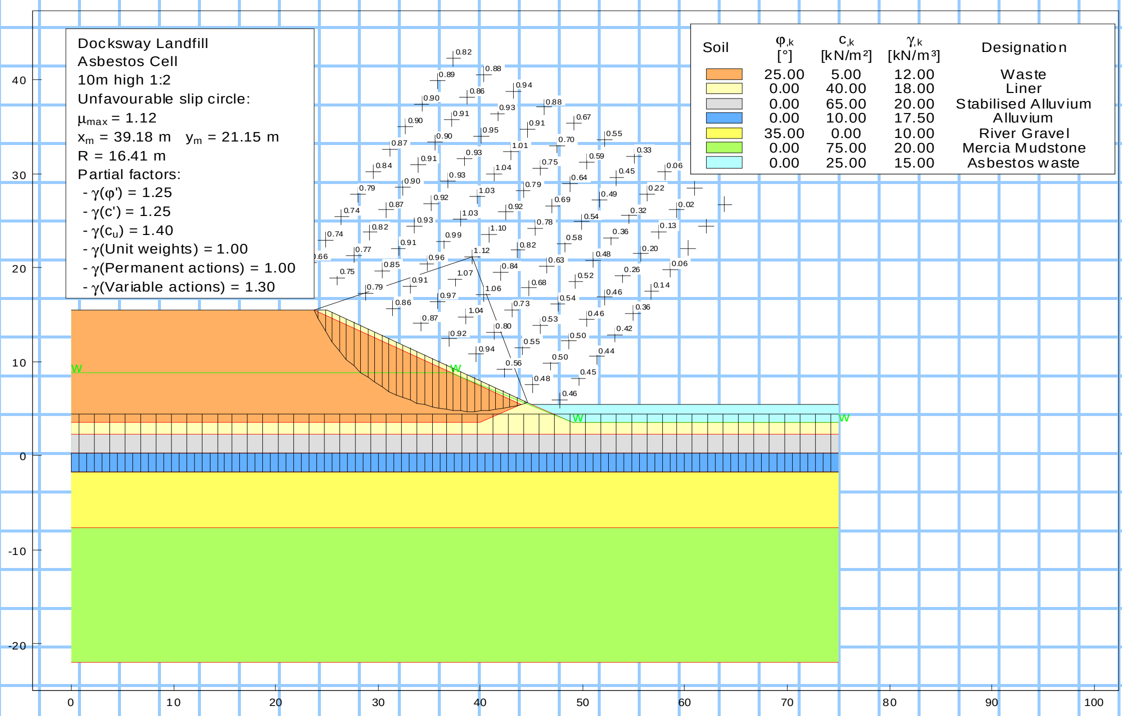
Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015								
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

7.5m high 1:1.5 slope wedge max utilisation factor 0.83 — Scenario 1



10m high 1:2 slope circular max utilisation factor 1.12 — Scenario 1



Sheet	Asbestos Cell 10	of	14

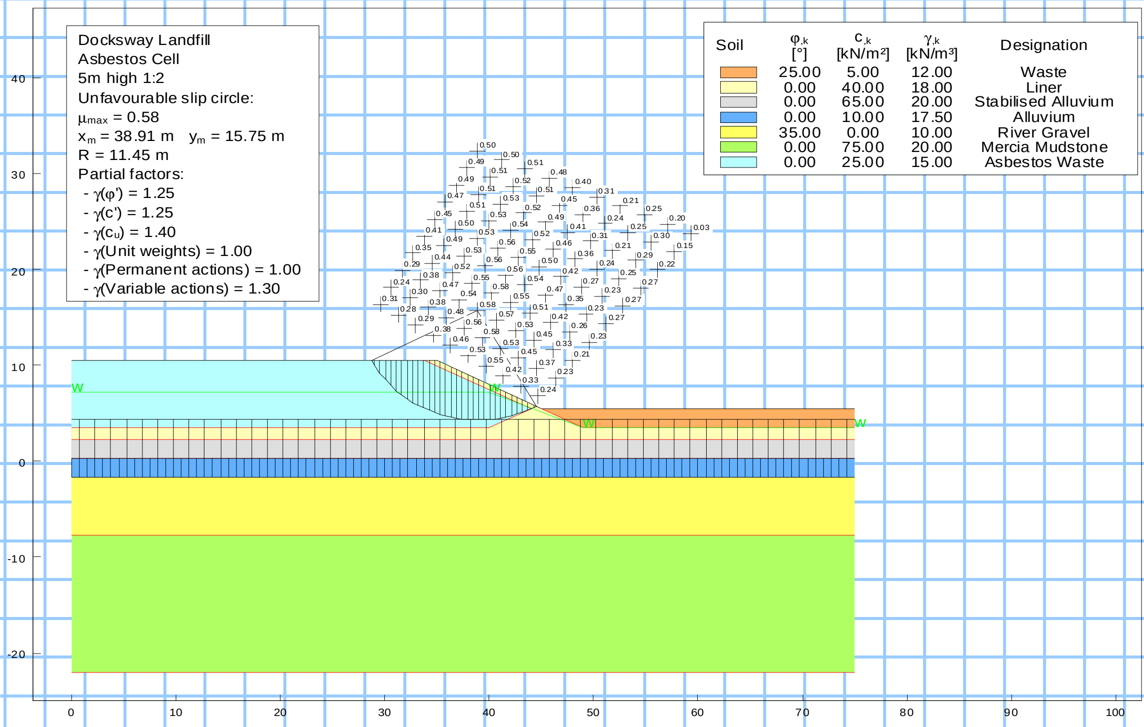
Calculations

peterbrett

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015									
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

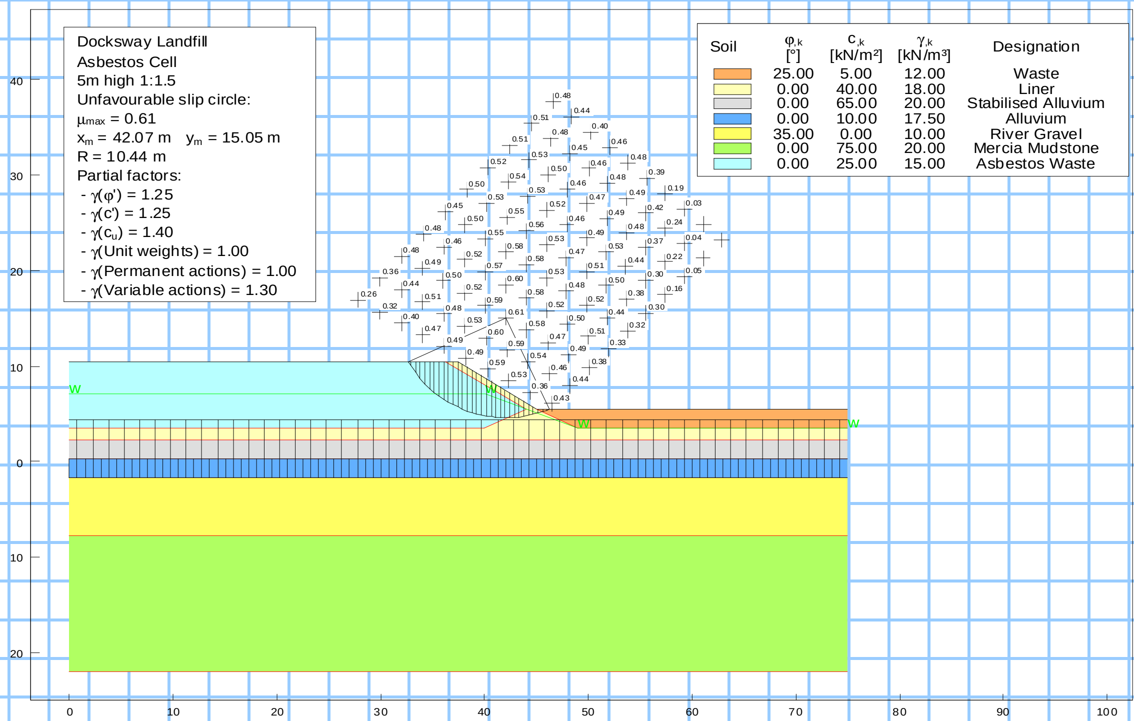
5m high 1:2 slope Circular max utilisation factor 0.58

Scenario 2



5m high 1:1.5 slope Circular max utilisation factor 0.61

Scenario 2

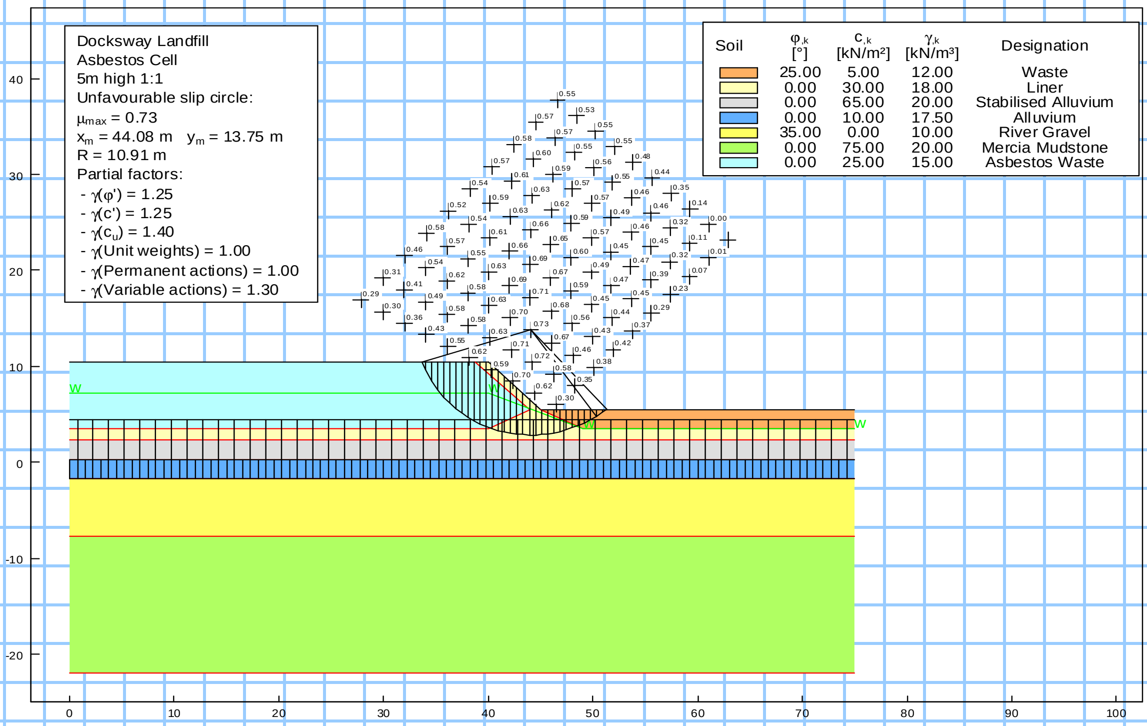


Sheet	Asbestos Cell 11	of	14

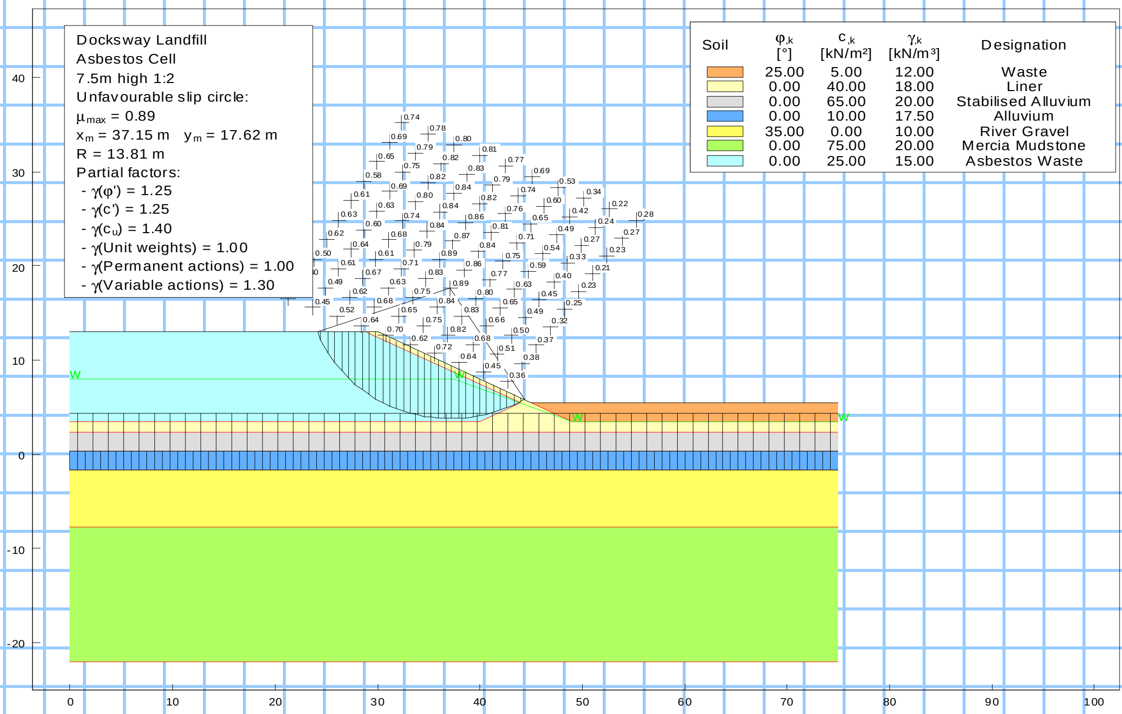
Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015									
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

5m high 1:1 slope Circular max utilisation factor 0.73 — Scenario 2



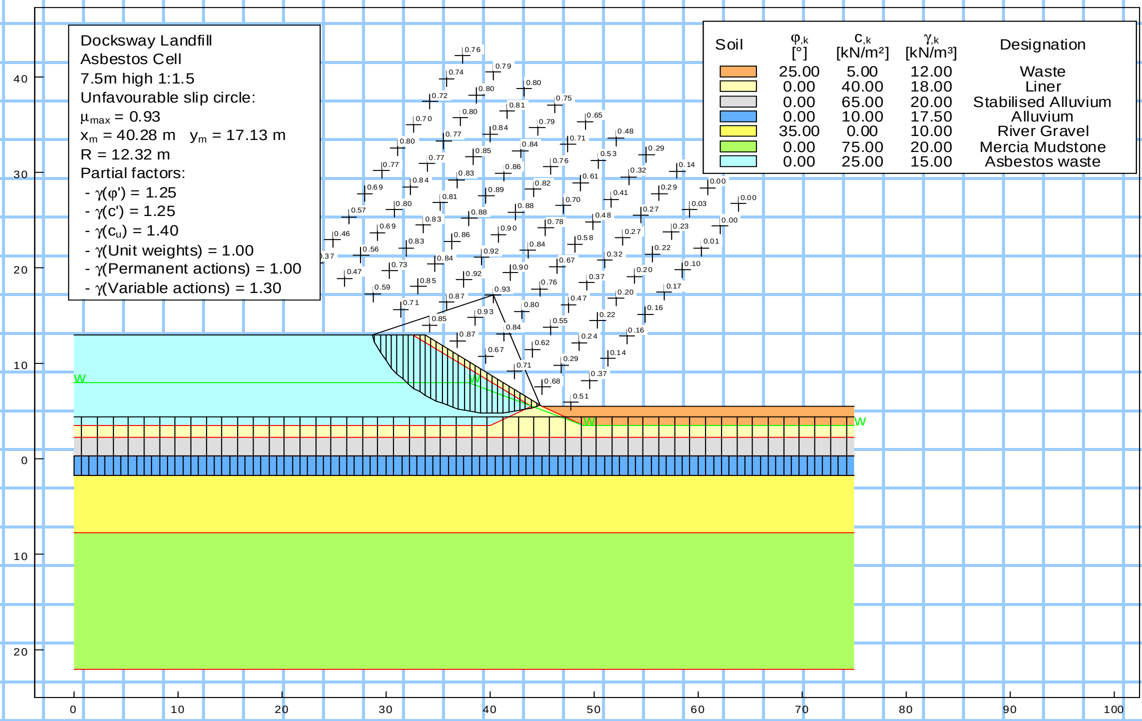
7.5m high 1:2 slope Circular max utilisation factor 0.89 — Scenario 2



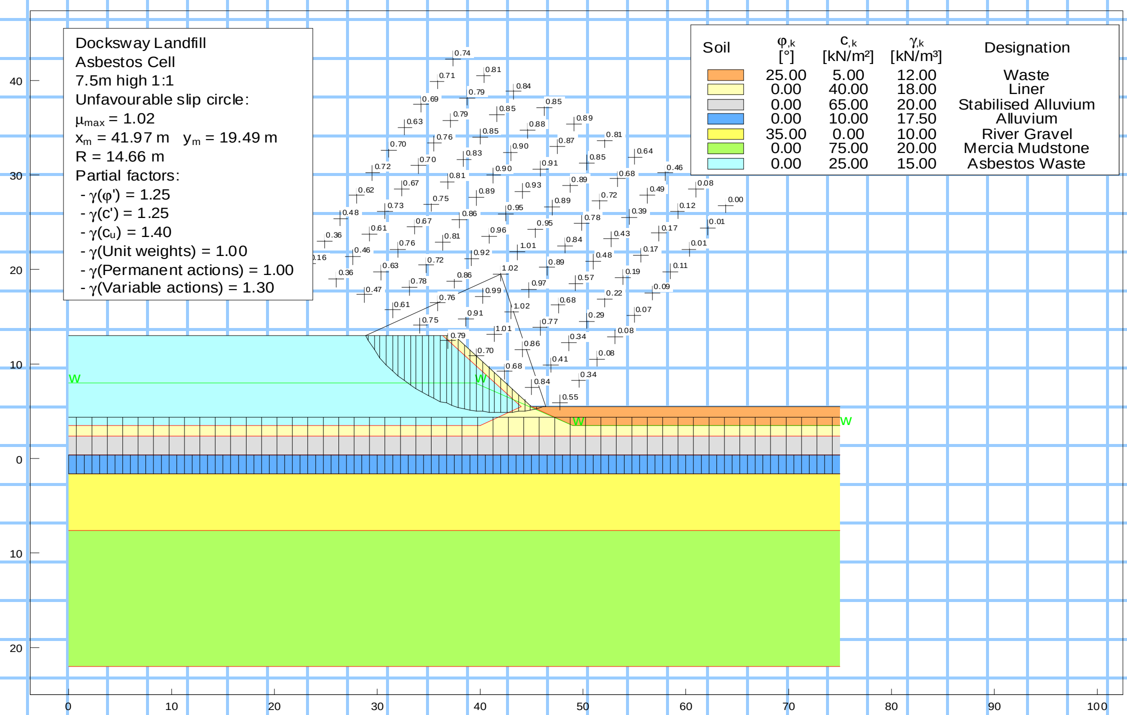
Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015									
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

7.5m high 1:1.5 slope Circular max utilisation factor 0.93 — Scenario 2



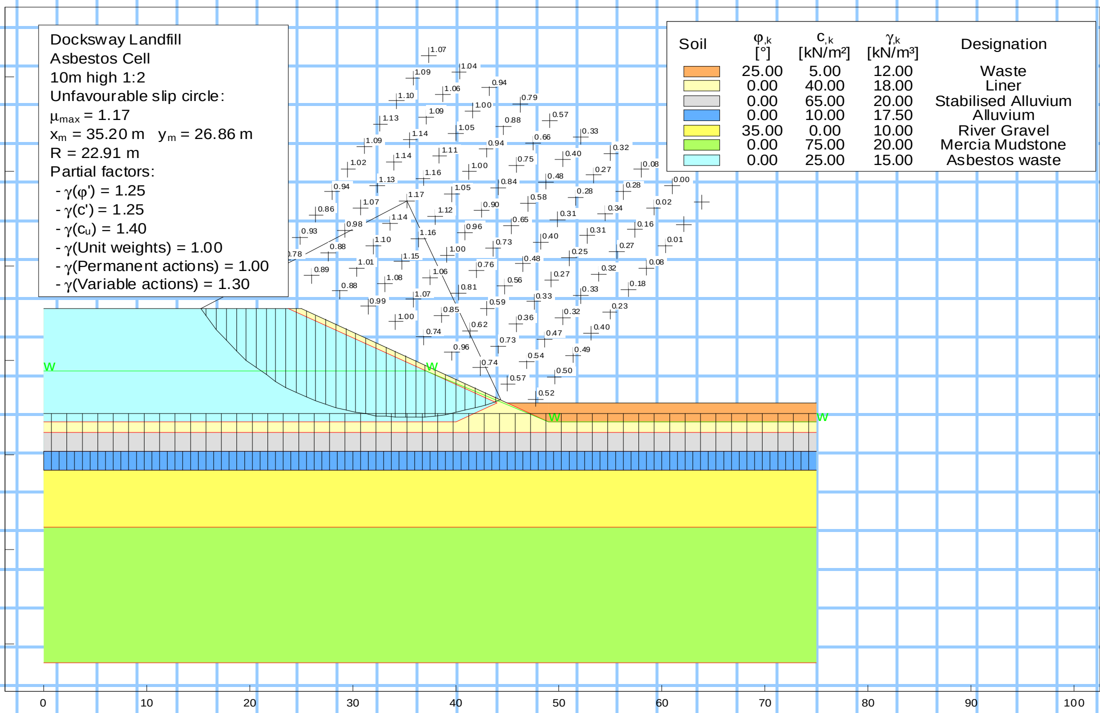
7.5m high 1:1 slope Circular max utilisation factor 1.02 — Scenario 2



Calculations

Project Title	Docksway Disposal Site: Area 2 Stability Assessment March 2015								
Project No	14739	156	By	LJT	Checked	ad	Date	24	04	2015

10m high 1:2 slope Circular max utilisation factor 1.17 Scenario 2



Sheet	Asbestos Cell 14	of	14