# Witcham Meadlands Landfill Site

## 784-B070447

# Stability Risk Assessment

### Environmental Permit Variation

**Mick George Limited**

**September 2025**

**Document prepared on behalf of Tetra Tech Limited. Registered in England number: 01959704**

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# Table of contents

## Appendices

# 1.0 Introduction

## 1.1 Report context

Mick George Limited (MGL) has commissioned Tetra Tech Limited, to undertake a Stability Risk Assessment (SRA) for Phase 9 at Mepal Landfill Site. This report forms part of the Environmental Permit Variation Application document for the site.

This report has been completed in conjunction with all relevant documentation, as required by The Environmental Permitting (England and Wales) Regulations. It is not a standalone document and factual data related to the site, its setting and the receiving environment are located in the Application Documents and referred to in this document. All drawings referred to in this SRA are to be found in the Application Document.

Mick George are now seeking to vary the environmental permit to apply a number of changes which are summarised in the Non-Technical Summary.

### Site location

Mepal Landfill Site is located approximately 5km to the south east of Chatteris in Cambridgeshire. The site is located in the Block Fen area of Mepal, at National Grid Reference TL 4400 8396. The landfill facility is operated by Mick George Limited in accordance with an Environment Agency Waste Management Environmental Permit.

The site is irregular in shape and covers an area of approximately 55 hectares. The current installation boundary is shown on MGL/A075924/PER/01A.

Mineral extraction operations have been carried out at the site since 2000 and these werecompleted by 2015.

The site has been accepting inert waste and non-hazardous waste imports since 2001 and 2008 respectively and these have been deposited within the completed mineral extraction areas in order to backfill the quarry and restore the land.

## 1.2 Conceptual Stability Site Model

### 1.2.1 Historical information

Tetra Tech has consulted the following documents during the preparation of the stability risk assessment:

- CQA Plan for Phase 9 Mepal Landfill Site, Witcham Meadlands Quarry, November 2010; ASL Report No. 053-09-081-14A Rev 2.

- Email dated 1 December 2012, detailing the Witcham Meadlands Additional Stability Assessment, 053-09-081.
- Preliminary Design Report for the Proposed Landfill Cell at Mepal, Cambridgeshire, April 2006; LS3I2003/DAF/DESIGN/APR06/V1.
- Stability Risk Assessment Report Phase 9, Mepal Landfill Site, ASL Report No. 053-09-081-13A Rev 2, July 2010.

Copies of the following drawings have also been consulted:

- 053-09-081-15A/02 Typical Liner System Sections, June 2010.
- MGL/A075924/PER/01A - Site Boundary Plan
- MGL/A075924/REC/01A - Potential Receptors
- MGL/A075924/ENG/04 – Engineering Details
- M34-7-24-03 – Restoration Surface
- MGL/B070447/LOC/01 - Location of Monitoring Boreholes

### 1.2.2 Geology

The geology of the site is covered by 1:50,000 British Geological (BGS) Sheet No. 172 Ramsey. The BGS sheet indicates the site geology to comprise; River Terrace Deposits underlain by Ampthill Clay Formation.

The River Terrace Deposits at the site are encountered directly beneath a layer of Tospoil and are described as dense orange brown sand and gravel with subordinate amounts of yellow-brown slightly sandy clay. The Ampthill Clay Formation is described as dark grey clays and pale grey calcareous clays with bands and lenses of argillaceous limestone (cementstone) at several levels.

Based on previous site investigation carried out at the site it is determined that the cement stone dips in a north east direction.

### 1.2.3 Groundwater

The Environment Agency class the River Terrace Deposits as a Secondary-A Aquifer (formerly classified as a minor aquifer). Groundwater levels in the River Terrace Deposits have previously been recorded at between 1.0m and 3.0m below ground level (-1.0m AOD to -2m AOD).

Historic ground investigation at the site encountered a continuous band of Cementstone at around 10mbgl. Trial pits previously excavated into the Cementstone proved no groundwater was encountered.

### 1.2.4 Basal sub-grade model

In the location of Phase 9 the River Terrace Deposits have been excavated to the top of the underlying Ampthill Clay Formation during the mineral extraction operations.

The basal sub-grade is comprised of in-situ Ampthill Clay Formation and comprises firm to very stiff occasionally hard, light to medium grey clay. Cementstone has been encountered during previous site investigation at the site at approximately -10m AOD at the south west boundary of the extension area. During installation of monitoring boreholes for the extension area cement stone was encountered at approximately -11.4m AOD on the north eastern boundary.

The Ampthill Clay shall be excavated from the footprint of Phase 9 to form the side slope liner. If the in situ clay does not provide a suitable basal liner i.e. pre-requisite permeability; then the top 1.0m shall be re-worked and engineered to achieve the necessary permeability.

Furthermore; it is proposed to maintain a minimum 2.5m vertical stand-off between the clay and underlying cementstone bands to ensure continuity of the geological barrier.

The basal subgrade is considered to be relatively non-compressible.

### 1.2.5 Side slopes sub-grade model

The side slope sub-grade comprises in situ River Terrace Deposits formed during the mineral extraction operations. The sub-grade shall be trimmed to achieve a gradient of 1(v):1(h) prior to placement of the engineered clay side slope liner. The side slope sub-grade slope will be in the order of 8.0m high.

The maximum recorded groundwater level in the River Terrace Deposits is 1.0m below ground level (-1m AOD).

### 1.2.6 Basal lining system model

It is proposed to form the basal liner with the in situ Ampthill Clay Formation (See Section 1.2.4).

Where the pre-requisite permeability cannot be demonstrated within the in situ clay through in situ testing and laboratory analysis then the top 1.0m of the basal sub-grade shall be re-worked and engineered to achieve the necessary permeability.

The basal liner shall provide a 1 in 100 fall to one leachate sump.

The basal liner shall be constructed to maintain a minimum 2.5m vertical stand-off from the underlying main cementstone band to ensure continuity of the geological barrier.

A drainage blanket comprising a geotextile protection layer and 750mm thick shredded tyre layer (allows for compaction to 500mm) shall be installed in direct contact with the upper surface of the engineered clay basal liner.

It is proposed to construct an inert separation bund within the Phase 9 during the operational waste filling phase.  The internal separation bund shall segregate placement of the asbestos and SNRHW.

### 1.2.7 Side slope liner model

The side slope liner shall be constructed against the 1(v):1(h) side slope sub-grade.  The low permeability side slope liner shall be constructed along the boundary of Phase 9 and formed using re-worked and engineered Ampthill Clay Formation with a horizontal crest width of 3.0m, height of approximately 8.0m and a maximum gradient of 1 (v) in 2 (h) (25.5°).  This will provide a minimum side slope liner thickness of 2m perpendicular to the slope.  The side slope liner shall be engineered in layers to achieve permeability no greater than 1x10-9 m/s.

### 1.2.8 Waste mass model

It is proposed that Phase 9 will be used for the containment of non-hazardous waste, stable non reactive hazardous wastes (SNRHW) and asbestos wastes.  The asbestos and SNRW shall be separated by a vertical inert separation bund to be constructed in incremental lifts during waste placement.

The maximum temporary waste slope during placement operations will be restricted to 1 in 3 (18.4°).

The waste will be compacted in horizontal layers across the base of the Phase to the pre-settlement restoration level.  The final waste height shall be approximately 4.0m above ground level and permanent waste slopes formed to a maximum gradient of 1(v):6(h).

### 1.2.9 Capping system model

The proposed capping system as the site shall comprise a 500mm thick engineered clay liner overlain with 1000mm thick restoration soils.

The proposed final restoration profile indicates that the maximum permanent waste slopes shall be in the order of 1(v):6(h) with a height of 4m.

Due to the characteristics of the proposed waste; landfill gas monitoring and control systems are not required.

# 2.0 Stability risk assessment

## 2.1 Risk screening

### 2.1.1 Basal sub-grade screening

The basal sub-grade shall be formed within the Ampthill Clay Formation and shall be excavated to achieve a minimum fall of 1 in 100 to a sump area.

The in situ Ampthill Clay Formation encountered in the development area is stiff and occasionally hard and is not anticipated to be subject to excessive loading by the waste. Settlement as a result of compressibility and loading is considered insignificant and further investigation is not required.

Cementstone bands have previously been encountered in the Ampthill Clay Formation at approximately -10m AOD and -11.4m AOD and are typically thin and/or latterly discontinuous. Groundwater has not been encountered in the Ampthill Clay Formation and/or the cementstone bands and therefore the risk of basal heave is considered low and further investigation is not required.

Cementstone bands encountered during basal sub-grade formation shall be removed and backfilled with engineered clay. It is proposed to excavate the basal sub-grade to a minimum 2.5m above the cementstone to ensure continuity of the geological barrier.

The Ampthill Clay Formation is not known for dissolution features and cavities have not been encountered during previous site investigation at the site; therefore further investigation is not required.

### 2.1.2 Side slopes sub-grade screening

The side slope sub-grade has been formed as a result of mineral extraction operations in the River Terrace Deposits.

The side slope sub-grade consist granular soils of the River Terrace Deposits and cut to maximum gradient of 1(v):1(h). Groundwater has previously been recorded to a maximum 1.0m below ground level in the River Terrace Deposits.

The stability of the side slope sub-grade together with the subsequent placement of engineered clay side seal should be considered further.

### 2.1.3 Basal lining system screening

The Phase 9 basal liner shall comprise 1.0m thick engineered clay (reworked Ampthill Clay Formation).

Compressible sub-grade, cavities and basal heave have previously been considered in Section 2.1.1.

The stability of the basal lining system shall be considered in conjunction with the side slope lining system.

### 2.1.4 Side slope lining system screening

The side slope lining system shall comprise engineered clay (re-worked Ampthill Clay Formation) placed against the side slope sub-grade. The side slope lining system shall be engineered to approximately 8.0m height and shall have a maximum gradient of 1(v):2(h) (25.5°).

Groundwater has been encountered in the River Terrace Deposits and the effect of groundwater on the stability of the side slope lining system will need to be considered further.

The side slope lining system will be exposed in the short term during landfilling operations and will ultimately be confined on completion of the waste filling to the pre-settlement restoration level.

The stability of the side slope lining system in the short to medium term is considered to require further investigation.

### 2.1.5 Waste mass screening

Waste is likely to be placed in phases within Phase 9 resulting in temporary waste slopes. The stability of temporary waste slopes is considered to require further investigation.

Restored waste mass slopes will be relatively shallow i.e. maximum 1(V):6(H) and therefore further investigation is not required.

### 2.1.6 Internal separation bund

The integrity and stability of the internal separation bund is based on the bund being fully confined by waste during construction.

The internal separation bund shall be constructed in incremental lifts no greater than 1.0m high and shall be encapsulated by waste before proceeding to the next lift. It is therefore considered that the stability of the internal separation does not require further investigation.

### 2.1.7 Capping system screening

The capping system shall comprise a 500mm thick engineered clay layer (re-worked Ampthill Clay Formation) overlain with 1000mm thick restoration soils.

It is proposed to alter the final restoration surface of Phase 9 to accelerate water run-off. The pre-settlement restoration contours shall comprise a maximum gradient of 1(v):6(h). The risk of restoration soil slipping on the clay capping system and affect of porewater pressure requires further investigation.

Due to the relatively shallow maximum gradient of the pre-settlement restoration contours and typical loading conditions; further investigation of the mineral capping liner and restoration soil is not required.

Loading other than from the restoration soils and construction plant is not anticipated and therefore further investigation of settlement is not considered further.

## 2.2 Data summary

The geotechnical parameters for the River Terrace Deposits, Ampthill Clay and waste within the stability analyses have been adopted following review of information presented in previous stability reports and review of laboratory testing results (see Section 1.2.1).

In the absence of site specific testing, the likely strength parameters of the in situ materials at this site have been derived from published literature.

## 2.3 Justification for modelling approach and software

The method of analysis was based on limit equilibrium principles and included checks on global slope stability, predominantly passing through the toe and shallow circular failures affecting only the surface.

For analysis the programme Slide (Rocscience Inc., Version 5.0040) was used which incorporates a search routine for identifying the critical surface with the lowest factors of safety against instability. Pore pressures can be defined by using a wide range of options including grid of user defined pore pressure points, phreatic surfaces and ru values. The design approach adopted was to use water table boundary with Hu values to define the groundwater and pore pressure regime.

There are several analysis methods available within the Slide program, including “rigorous” methods such as Morgenstern and Price and simplified methods such as Bishops or Janbu methods. For the Phase 9 stability review it was considered that Morgenstern’s method and Janbu’s method was appropriate for the circular and non-circular analysis respectively.

In order to assess slope stability sensitivity analyses were undertaken for assessment of safe slopes in both un-drained (total stress) and long term (effective stress or drained) conditions.

The capping stability analysis has been carried out following the procedure detailed by Jones and Dixon (1998).

In all cases the worst case scenario has been modelled. This includes the steepest and/or longest waste slopes and the steepest and/or longest capping system slopes.

### 2.3.1 Sensitivity analysis

The sensitivity analyses comprised reviewing the effect of varying the total and effective stress soil parameters for the engineered side slope liner and circular and non-circular slip surfaces.

The computer code Slide 5.040 was used to calculate the factor of safety of the full height of the proposed engineered side slope for a range of unconfined circular failures at gradients of 1(v) in 2(h) for the engineered fill and 1(v) in 3(h) for the temporary waste slopes.

The peak and residual soil conditions (short and long term condition) have been considered together with Parallel Submergence Ratio (PSR) of up to 1 during the assessment of stability on the restoration soil veneer.

### 2.3.2 Geological sections

The geological section used in the analyses is based on the typical section shown on drawing: MGL/A075924/ENG/04.

It has previously been proposed that a backwall drain shall be installed behind the engineered fill to intercept any groundwater seepages and prevent build-up of pore pressures behind the side wall lining system.  Analysis has been carried out considering the side slope liner with and without backwall drainage.

A maximum slope gradient of 1(v) in 2(h) has been used to assess the stability of the proposed side slope liner and a slope gradient of 1(v) in 3(h) was modelled for the proposed temporary waste slopes.

## 2.4 Justification of geotechnical parameters selected for analyses

The following geotechnical parameters have been adopted for the River Terrace Deposits, Ampthill Clay and waste within the stability analyses.

### 2.4.1 Parameters selected for basal sub-grade analyses

**Table 1: Geotechnical parameters basal sub-grade**

| Material | | Bulk Density (kN/m$^3$) | Total Stress | | Effective Stress | | Source |
|---|---|---|---|---|---|---|---|
| | | | C (kN/m$^2$) | $\phi$ (°) | c' (kN/m$^2$) | $\phi$'(°) | |
| Ampthill Clay Formation | Clay[1] | 20 | 60 | 0 | 35 | 25 | SRA Report; 053-09-081 |

### 2.4.2 Parameter selected for side slopes sub-grade analyses

**Table 2: Geotechnical parameters side slope sub-grade**

| Material | | Bulk Density (kN/m$^3$) | Total Stress | | Effective Stress | | Source |
|---|---|---|---|---|---|---|---|
| | | | C (kN/m$^2$) | $\phi$(°) | c' (kN/m$^2$) | $\phi$'(°) | |
| Superficial Deposits | Overburden | 16 | 3 | 20 | 3 | 20 | SRA Report; 053-09-081 |
| River Terrace Deposits | Clay | 19 | 3 | 38 | 3 | 38 | SRA Report; 053-09-081 |

### 2.4.3 Parameters selected for basal line analyses

The engineered clay is to be placed under controlled earthworks specifications and a compactive effort determined through HA DMRB to achieve less than 5% air voids in the field during placement.

The clays will initially attain undrained conditions appropriate to a minimum placement value. For the purposes of the stability analyses a conservative shear strength value of 60kN/m2 has been used. In the medium and longer term the effective stress or drained conditions will become predominant. The angle of internal shearing resistance has been based on the plasticity of the clay after Gibson, 1953.

Limited laboratory testing of the proposed engineered clay fill indicates the clay is of high plasticity and therefore prone to the generation of soil suction / negative pore pressure. Initially the apparent drained cohesion will be quite large but will reduce with time. For the purposes of the stability modelling a value of 0kN/m2 for the effective cohesion has been used for the proposed side slope in the long term (effective stress) analyses. Using the empirical relationship between plasticity index and angle of shear resistance and based on an average plasticity index of 40; a friction angle of 25° has been assessed.

**Table 3: Geotechnical parameters basal liner**

| Material | | Bulk Density (kN/m³) | Total Stress | | Effective Stress | | Source |
|---|---|---|---|---|---|---|---|
| | | | C (kN/m²) | ϕ (°) | c' (kN/m²) | ϕ'(°) | |
| Engineered Side Seal | Clay | 19 | 60[1] | 0 | 0 | 25[2] | 1. Assumed<br>2. Based on PI after Gibson 1953 |

### 2.4.4 Parameters selected for side slope liner analyses

See section 2.4.3.

### 2.4.5 Parameters selected for waste analyses

In the absence of site specific data relating to the waste properties at the site, the design values of suggested by Jones et al (1997) in the Environment Agency R&D Technical Report P1-385/TR1 have been adopted. It is understood that the waste shall be classified as stable non-reactive hazardous waste comprising heterogeneous soils and subordinate inert inclusions including fine grained and coarse grained soils, concrete, brick and stabilised quarry washing with parts of the waste mass put over to asbestos containing waste. On the basis that the values suggested by Jones et al are derived from putrescible materials adopting these values are considered conservative.

**Table 4: Geotechnical parameters waste**

| Material | Bulk Density (kN/m³) | Total Stress | | Effective Stress | | Source |
|---|---|---|---|---|---|---|
| | | C (kN/m²) | ϕ (°) | c' (kN/m²) | ϕ'(°) | |
| Waste Soils (SNRHW) | 13 | 5 | 25 | 5 | 25 | Jones et al |

### 2.4.6 Parameters selected for capping analyses

**Table 5: Geotechnical parameters capping system**

| Material | | Bulk Density ($kN/m^3$) | Total Stress | | Effective Stress | | Source |
|---|---|---|---|---|---|---|---|
| | | | C ($kN/m^2$) | $\phi$ (°) | c' ($kN/m^2$) | $\phi$'(°) | |
| Engineered clay cap | Clay | 19 | 60[1] | 0 | 0 | 25[2] | 1. Assumed<br>2. Based on PI after Gibson 1953 |

It is anticipated that the restoration soils will typically comprise >35% fines and therefore geotechnical parameters characteristic of fine grained soils with low plasticity have been adopted for the restoration soils within the stability analyses.

**Table 6: Restoration soil parameters**

| Restoration Soil | |
|---|---|
| Cover soil unit weight (dry) $\gamma_{dry}$ | $18kN/m^3$ |
| Cover soil unit weight (saturated) $\gamma_{sat}$ | $21kN/m^3$ |
| Cover soil internal shear strength $\phi$ | 28 |
| Cover soil cohesion c | 0 |
| Parallel Submergence Ration (PSR) | 0 to 1 |

**Table 7: Interface shear strength parameters**

| Interface | Peak | | Residual | | Source |
|---|---|---|---|---|---|
| | $\alpha$ ($kN/m^2$) | $\delta$(°) | $\alpha$' ($kN/m^2$) | $\delta$'(°) | |
| Restoration soil / Mineral cap | 0 | 24 | 0 | 17 | Estimated (conservative) |
| Mineral cap / regulating layer | 5 | 25 | 5 | 25 | Estimated (Conservative) |

## 2.5 Selection of appropriate factors of safety

The factor of safety is the numerical expression of the degree of confidence that exists, for a given set of conditions, against a particular failure mechanism occurring. It is commonly expressed as the ratio of the load or action which would cause failure against the actual load or actions likely to be applied during service. This is readily determined for some types of analysis (e.g. limit equilibrium slope stability analyses). It must be emphasised that the factor of safety does not indicate the value of the probability of failure.

Prior to determining appropriate factors of safety for the various components of the model, it is necessary to identify key 'receptors' and evaluate the consequences in the event of a failure (relating to both stability and integrity). Consideration of the following receptors is required:

- Property - relating to site infrastructure, third party property;
- Site operatives (i.e. direct risk).

The factor of safety adopted for each component of the model should be related to the consequences of a failure.

Based on the above considerations and using "worst case" scenarios e.g. minimum assessed (conservative) strength values, a factor of safety of 1.30 is considered appropriate for this assessment.

## 2.6 Analyses

### 2.6.1 Basal sub-grade analyses

Not applicable.

### 2.6.2 Side slopes sub-grade analyses

Not applicable.

### 2.6.3 Basal liner analyses

Not applicable.

### 2.6.4 Side slope liner analyses

The results of the stability analyses are presented in Appendix A and can be summarised as follows:

**Table 8: Summary of stability analysis for Phase 9 side slope liner – 1 (v):2(h) gradient with back wall drain.**

| File ID | Method | Slope | F of S | Comments |
|---|---|---|---|---|
| 00 | Total Stress | 1 in 2 | 2.44 | Undrained (60kpa) failure RTD/Ampt Clay and toe of fill. |
| 00A | Effective Stress | 1 in 2 | 1.02 | Failure in clay fill |

**Table 9: Summary of stability analysis for Phase 9 side slope liner – 1 (v):2(h) gradient without back wall drain**

| File ID | Method | Slope | F of S | Comments |
|---|---|---|---|---|
| 01 | Total Stress | 1 in 2 | 1.99 | Undrained (60kpa) failure in RTD and Ampthill Clay. |
| 01A | Effective Stress | 1 in 2 | 0.60 | Failure in clay fill |
| 01B | Effective Stress with waste buttress | 1 in 2 | 1.35 | Waste buttress 4.5m (h) x 10 x (w) |
| | Effective stress no waste | 1 in 3 | 1.45 | |

### 2.6.5 Waste analyses

The results of the temporary waste mass stability analyses are present in Appendix B and summarised in the table below.

**Table 10: Summary of stability analysis for Phase 9 temporary waste slope – 1 (v):3(h) gradient**

| File ID | Method | Slope | F of S | Comments |
|---|---|---|---|---|
| 02 | Effective Stress | 1 in 3 | 2.23 | Circular slip surface |
| 03 | Effective Stress | 1 in 3 | 2.33 | Non-circular slip surface |

### 2.6.6 Capping analyses

The results of the capping veneer (mineral liner and cover soil) analyses are present in Appendix C and summarised in the table below.

**Table 11: Summary of veneer stability analysis for the capping system – 1 (v):6(h) gradient**

| File ID | Condition | PSR | Factor of Safety Against Sliding | |
|---|---|---|---|---|
| | | | Restoration Soil | Mineral Liner |
| C1 | Peak | 0.00 | 3.16 | 5.22 |
| C2 | Peak | 0.25 | 2.74 | 4.69 |
| C3 | Peak | 0.50 | 2.35 | 4.21 |
| C4 | Peak | 1.0 | 1.66 | 3.38 |
| C5 | Residual | 0.0 | 2.33 | 3.29 |
| C6 | Residual | 0.25 | 2.03 | 2.84 |
| C7 | Residual | 0.50 | 1.75 | 2.44 |
| C8 | Residual | 1.0 | 1.23 | 1.73 |

## 2.7 Assessment

### 2.7.1 Basal sub-grade assessment

Not applicable.

### 2.7.2 Side slopes sub-grade assessment

See 2.7.4.

### 2.7.3 Basal liner assessment

Not applicable.

### 2.7.4 Side slope liner assessment

The analysis has shown that the stability of the side slope lining system with a maximum gradient of 1(v) in 2(h) has an acceptable factor of safety greater than 1.3, under total stress conditions using a minimum undrained shear strength of 60kPA. If a backwall drain is installed or no groundwater is encountered in the River Terrace Deposits the factor of safety for instability of the side wall lining system is 2.44. However; the factor of safety is reduced to 1.99 if groundwater is allowed to build-up behind the side wall liner.

The results show that for the side slope liner the critical condition will occur sometime after construction (circa >3 years) when all the excess pore water pressure has dissipated and effective stress conditions occur within the side slope liner. Under the effective stress condition the factor of safety for the side slope lining system with a back wall drain is at unity (1.02).

The factor of safety against instability for the clay side slope liner with a slope gradient of 1 (v) in 2(h) are greater than 1.3, for the values of undrained shear strength greater than 60kN/m2 and is considered to be satisfactory.

During and soon after construction the clay side slope liner will have soil suctions (negative pore pressure) and factor of safety greater than 1.3. However, after a number of years the pore pressure will equilibrate and the factor of safety will reduce to unity if the liner is left unconfined. It is therefore necessary to place waste (buttress) against the side slope liner 4.5m (h) x 10 (w) in the first couple of years following construction in order to maintain an adequate factor of safety in the medium term.

### 2.7.5 Waste assessment

The stability of the temporary waste slope with a gradient of 1(v) in 3(h) is considered satisfactory with the factor of safety being greater than the 1.3 in the short term.

The stability of the waste mass has been considered and is deemed satisfactory when placed with temporary waste slopes no greater than 1(v) in 3(h). The waste mass will ultimately be confined on completion of the Phase 9 landfill operations and therefore the long term stability of the waste mass including build-up of pore water pressure has not been considered.

### 2.7.6 Capping analyses

The factor of safety under normal conditions (PSR < 0.25) is > 2.74 for peak strength parameters and > 2.039 for residual strength parameters. The factor of safety for peak strength parameters is > 1.66 and for residual parameters > 1.23, for all the conditions considered. Therefore the proposed capping system is considered to be satisfactory.

## 2.8 Monitoring

### 2.8.1 Monitoring

Where appropriate each element of construction: subgrade, lining system and capping will be subject of an independent Construction Quality Assurance (CQA) regime during construction. This will ensure that all barriers are stable and to specification at the time of construction.

### 2.8.2 Basal sub-grade monitoring

Cementstone bands encountered during basal sub-grade formation shall be removed and backfilled with engineered clay. The basal sub-grade shall be excavated to a minimum 2.5m above the cementstone to ensure continuity of the geological barrier.

The CQA Engineer shall record depth and location of cementstone during excavation and confirm that any remedial works are carried out pursuant to the CQA Plan and Specification.

### 2.8.3 Side slopes sub-grade monitoring

It is understood that groundwater has previously been encountered as high as 1.0m below ground level at the site. A high groundwater table has a significant bearing on the factor of safety particularly on the unconfined slope under effective stress conditions. A back wall drain should therefore be installed behind the side wall liner if groundwater is anticipated or encountered during construction.

If the back wall drain is omitted then the absence of groundwater should be ascertained by long term monitoring and during the construction phase.

### 2.8.4 Basal liner monitoring

See 2.8.3.

### 2.8.5 Side slope liner monitoring

The risk of failure to the side slope liner is considered low. However; the CQA Engineer and contractor should monitor the construction works for incipient signs of failure. These will include cracking, bulging or seepage of water. In the event of a slope failure the Environment Agency should be informed and remedial action agreed.

If the rate of waste infill is likely to be intermittent; then it is recommended that waste placement is initially focussed to the toe of the side slopes to improve the long term factor of safety.

### 2.8.6 Waste monitoring

The risk of failure to temporary slopes within the waste mass is considered low. However; the site operator should ensure that waste is suitably compacted and temporary waste slope doesn't exceed 1 in 2. In the event of a slope failure the Environment Agency should be informed and remedial action agreed.

The operator should ensure that the internal separation bund is placed in 1m high lifts and following the complete confinement of the lower lift by waste.

### 2.8.7 Capping monitoring

The gradient of the final surface is such that slope failure within the waste mass and cover soil is considered low. The overall stability of the landfill capping system should be monitored by the operator following completion.

Features on the surface of the restoration soils to be monitored/recorded include:-

- Formation of cracks

- Depressions
- Ponding
- Seepages

Any remediation of the clay liner in the event of damage should be carried out in accordance with an approved CQA Plan and Specification.

Due to the characteristics of the SNRHW differential settlement of the cap is unlikely.  However where significant differential settlement of the cap is observed; this should be remediated by filling low areas with sufficient soils to allow surface run-off from the low areas, rather than allowing ponding within them.

# 3.0 References

British Geological Survey: 1:50000 Series map, Sheet 172 Ramsey.

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

Environment Agency (2002), Stability of Landfill Lining Systems: Report No. 2 Guidance. R&D Technical report P1-385/TR2, Environment Agency, Bristol.

# Appendices

Appendix A: Results of the side slope stability analyses

Appendix B: Results of the waste mass slope stability analyses

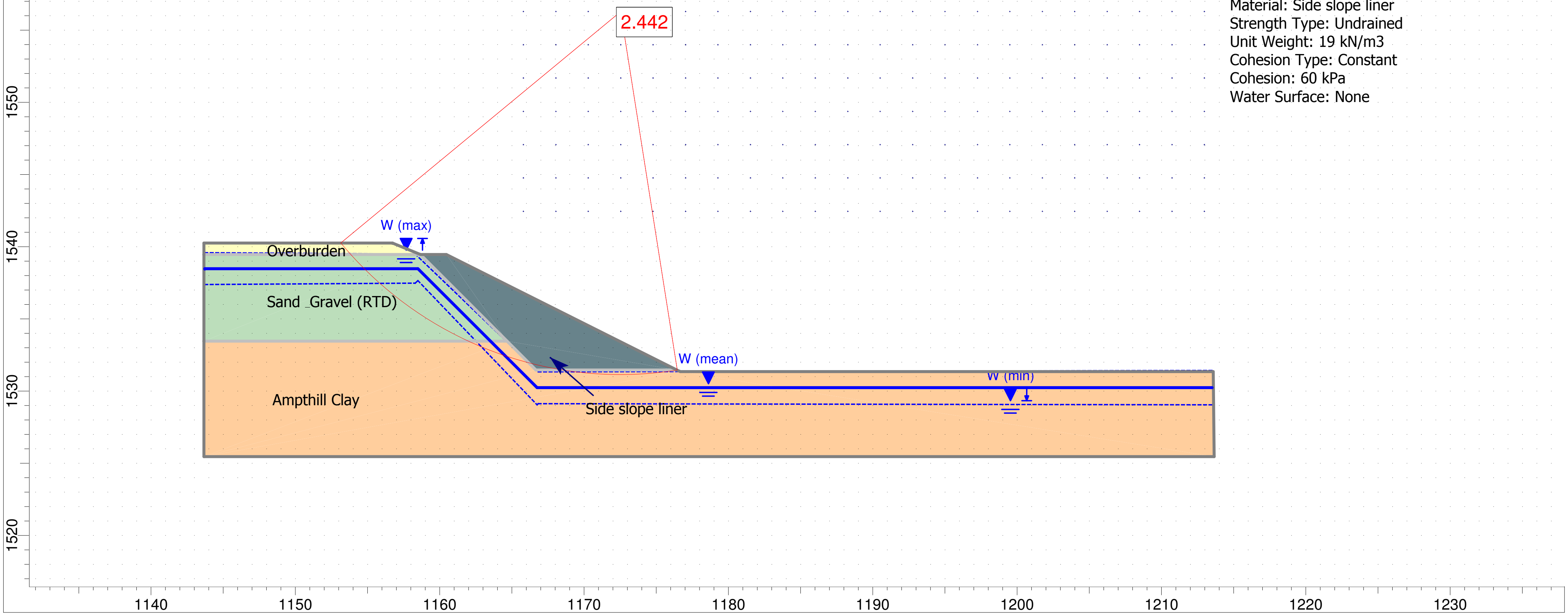
Appendix C: Results of the capping stability analyses

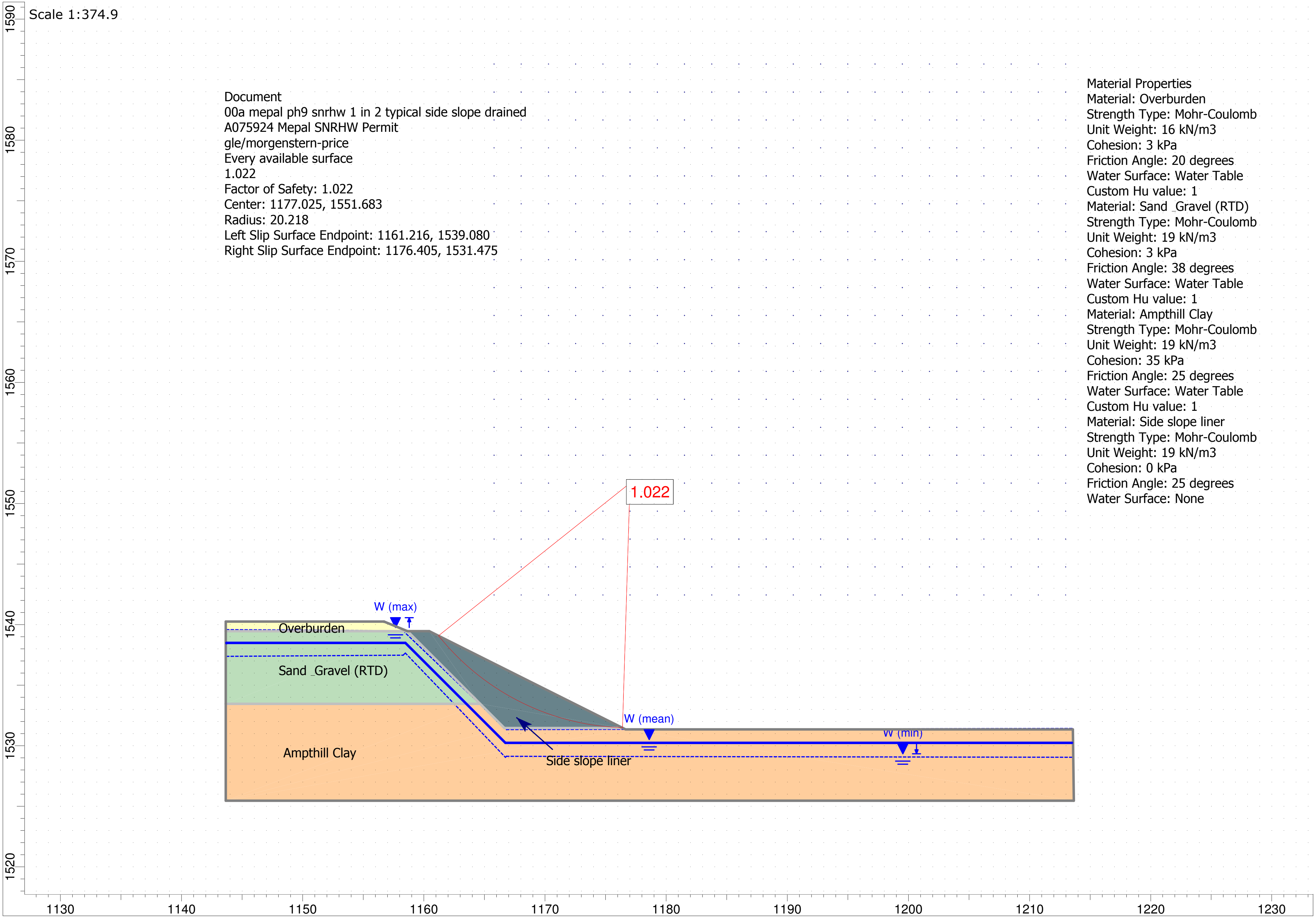
# Appendix A: Results of the side slope stability analyses

Scale 1:374.9

Document
00 Mepal Ph9 SNRHW 1 in 2 typical side slope undrained
A075924 Mepal SNRHW Permit
gle/morgenstern-price
Every available surface
2.442
Factor of Safety: 2.442
Center: 1172.531, 1556.298
Radius: 25.159
Left Slip Surface Endpoint: 1153.142, 1540.264
Right Slip Surface Endpoint: 1176.461, 1531.447

Material Properties
Material: Overburden
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 3 kPa
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Sand _Gravel (RTD)
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 3 kPa
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Ampthill Clay
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 35 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Side slope liner
Strength Type: Undrained
Unit Weight: 19 kN/m3
Cohesion Type: Constant
Cohesion: 60 kPa
Water Surface: None

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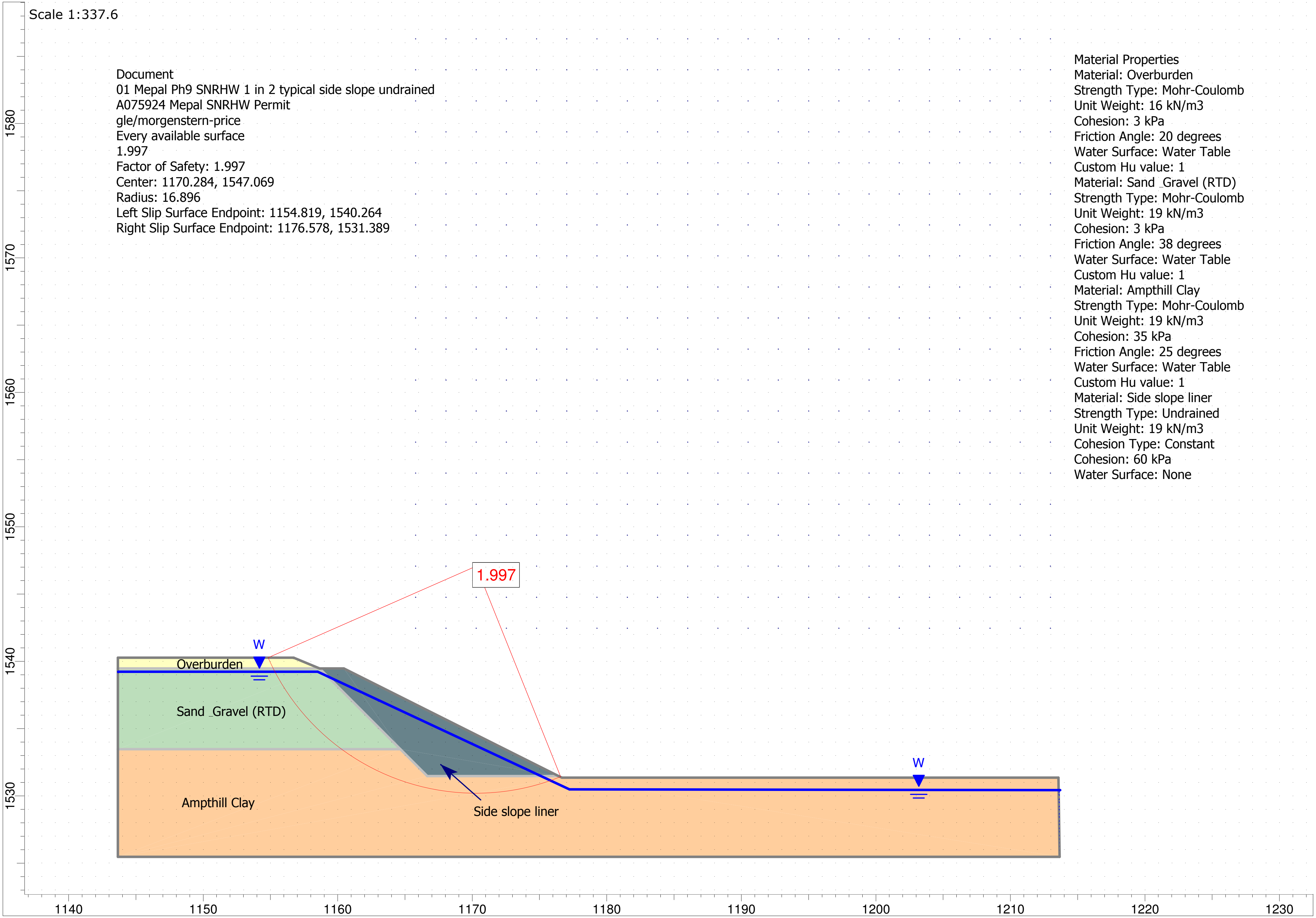
Document
00a mepal ph9 snrhw 1 in 2 typical side slope drained
A075924 Mepal SNRHW Permit
gle/morgenstern-price
Every available surface
1.022
Factor of Safety: 1.022
Center: 1177.025, 1551.683
Radius: 20.218
Left Slip Surface Endpoint: 1161.216, 1539.080
Right Slip Surface Endpoint: 1176.405, 1531.475
Material Properties
Material: Overburden
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 3 kPa
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Sand _Gravel (RTD)
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 3 kPa
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Ampthill Clay
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 35 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Side slope liner
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 25 degrees
Water Surface: None
1.022
W (max)
Overburden
Sand _Gravel (RTD)
Ampthill Clay
W (mean)
W (min)
Side slope liner
1590
1580
1570
1560
1550
1540
1530
1520
1130
1140
1150
1160
1170
1180
1190
1200
1210
1220
1230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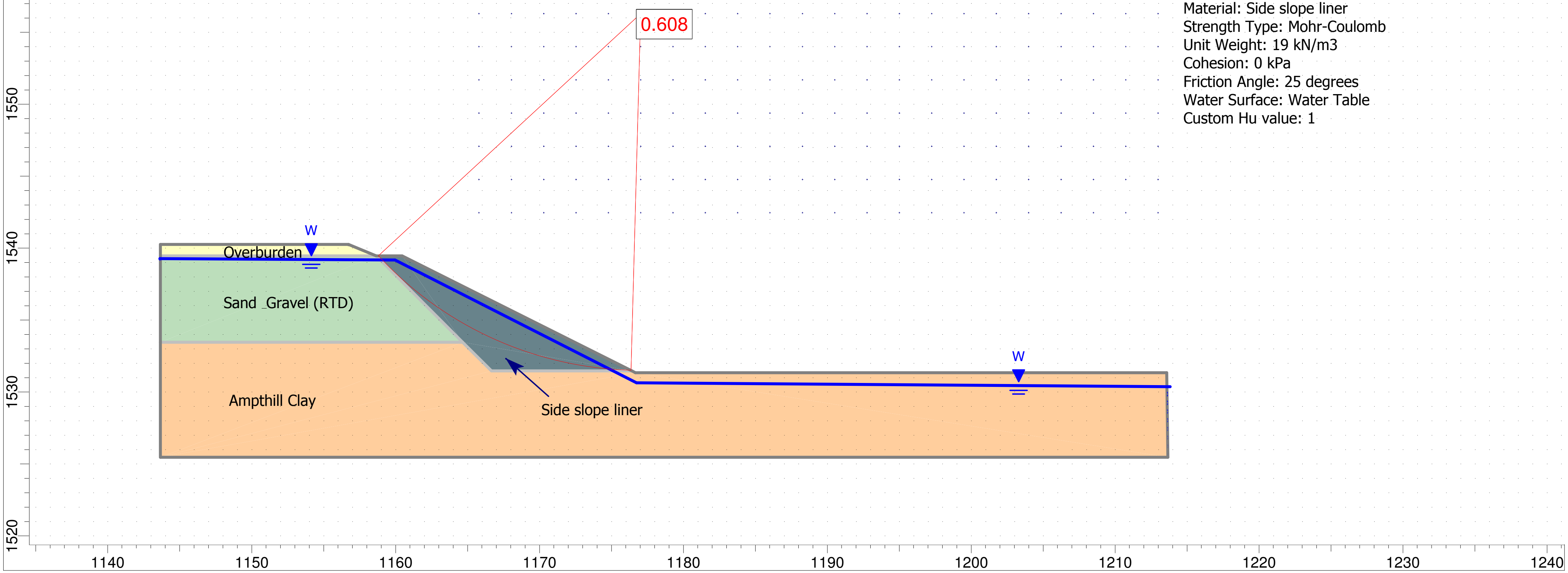
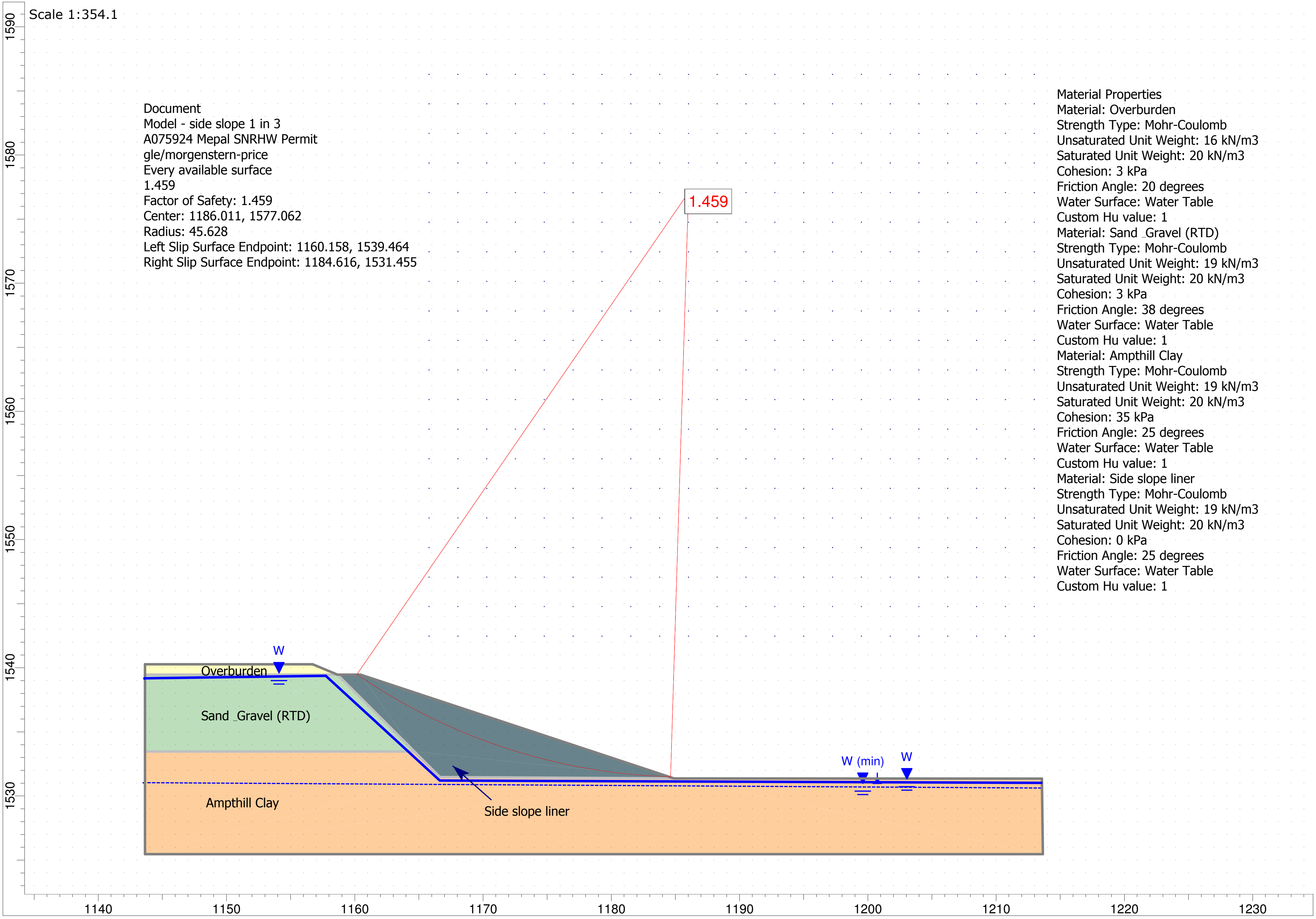
Document
01 Mepal Ph9 SNRHW 1 in 2 typical side slope undrained
A075924 Mepal SNRHW Permit
gle/morgenstern-price
Every available surface
1.997
Factor of Safety: 1.997
Center: 1170.284, 1547.069
Radius: 16.896
Left Slip Surface Endpoint: 1154.819, 1540.264
Right Slip Surface Endpoint: 1176.578, 1531.389
Material Properties
Material: Overburden
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 3 kPa
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Sand _Gravel (RTD)
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 3 kPa
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Ampthill Clay
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 35 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Side slope liner
Strength Type: Undrained
Unit Weight: 19 kN/m3
Cohesion Type: Constant
Cohesion: 60 kPa
Water Surface: None
1.997
W
Overburden
Sand _Gravel (RTD)
Ampthill Clay
Side slope liner
W
1580
1570
1560
1550
1540
1530
1140
1150
1160
1170
1180
1190
1200
1210
1220
1230

Scale 1:376.0

Document
01A Mepal Ph9 SNRHW 1 in 2 typical side slope undrained
A075924 Mepal SNRHW Permit
gle/morgenstern-price
Every available surface
0.608
Factor of Safety: 0.608
Center: 1177.025, 1556.298
Radius: 24.807
Left Slip Surface Endpoint: 1158.803, 1539.464
Right Slip Surface Endpoint: 1176.357, 1531.500

Material Properties
Material: Overburden
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 3 kPa
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Sand _Gravel (RTD)
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 3 kPa
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Ampthill Clay
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 35 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Side slope liner
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1

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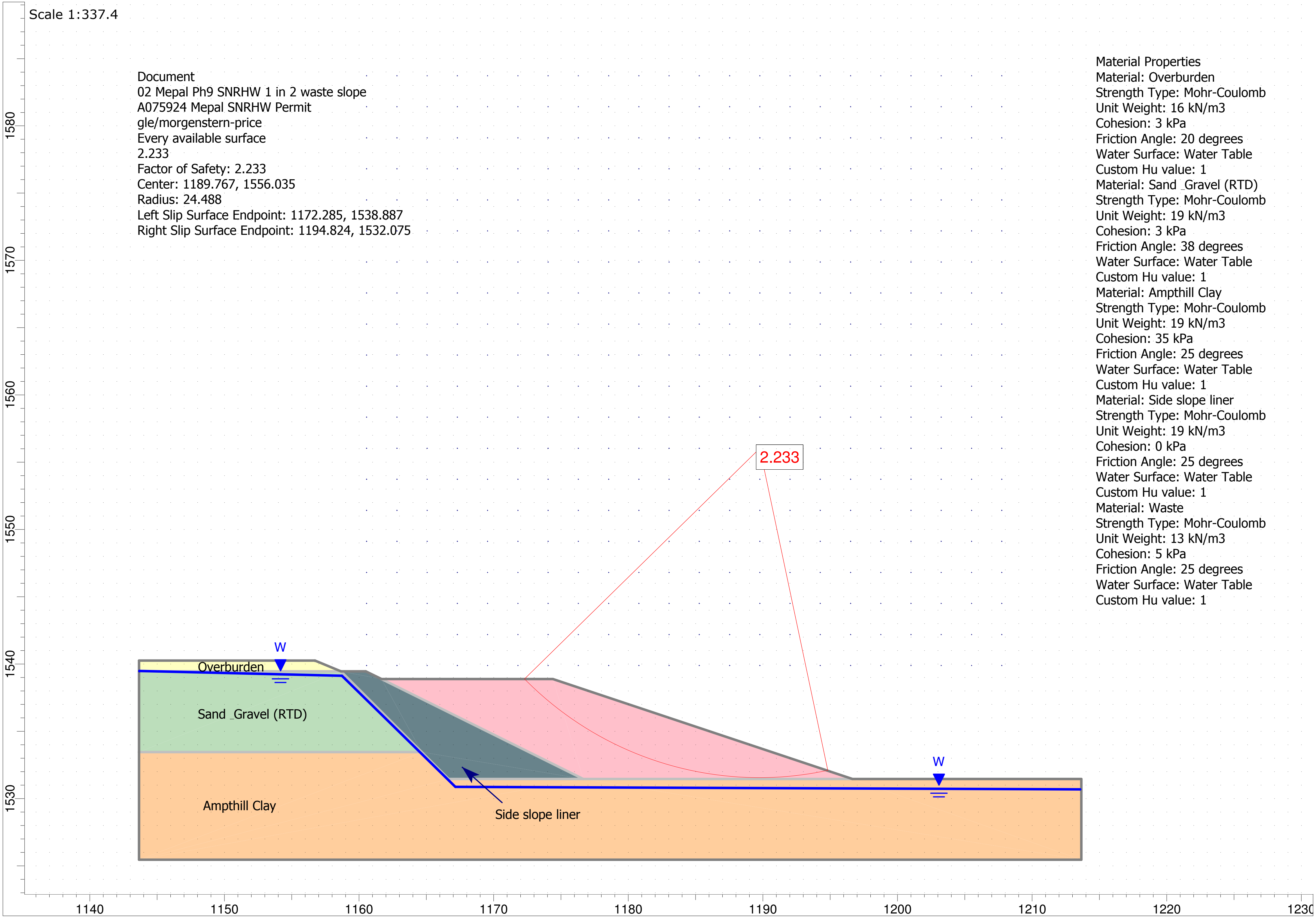
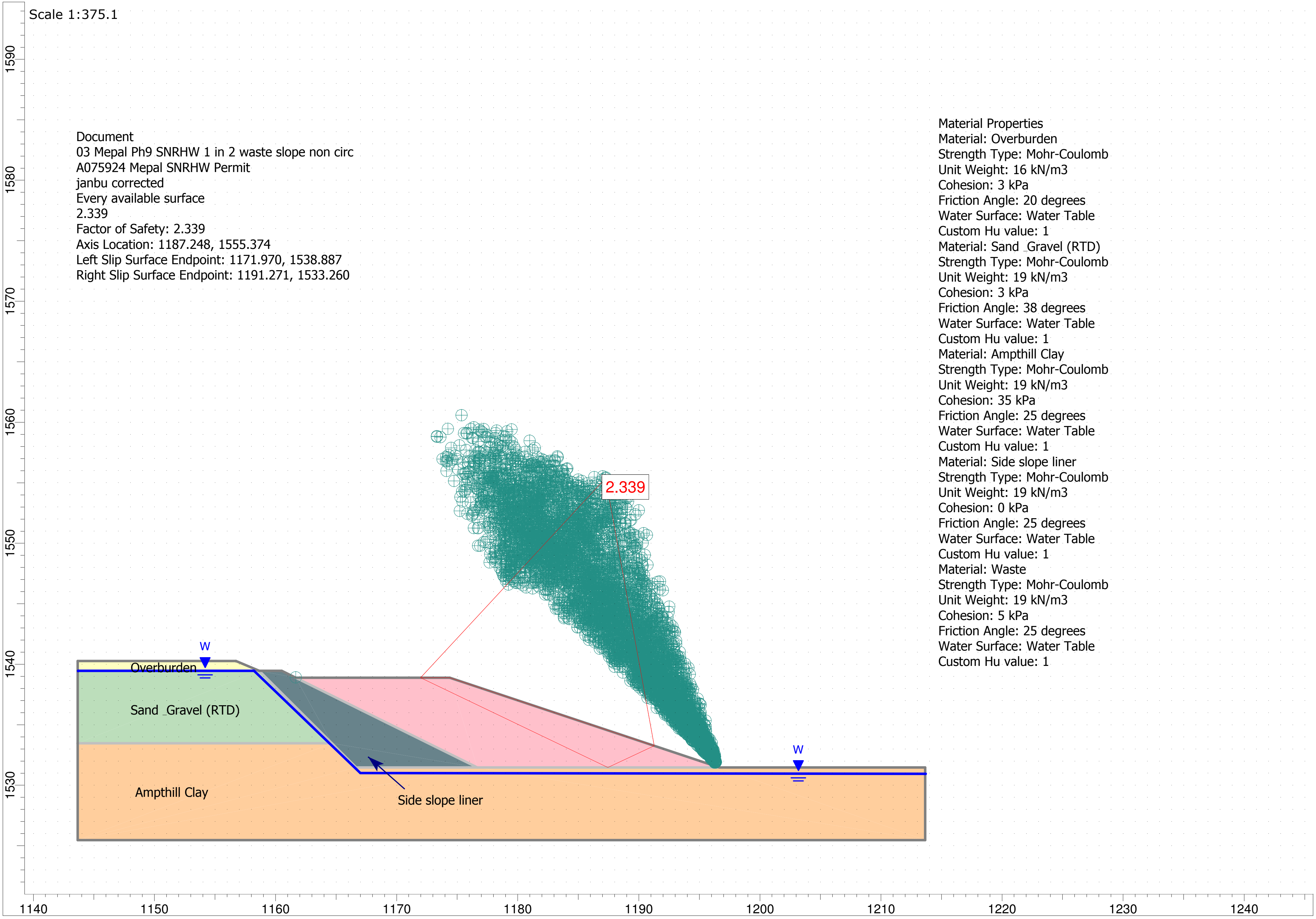
Model - side slope 1 in 3
A075924 Mepal SNRHW Permit
gle/morgenstern-price
Every available surface
1.459
Factor of Safety: 1.459
Center: 1186.011, 1577.062
Radius: 45.628
Left Slip Surface Endpoint: 1160.158, 1539.464
Right Slip Surface Endpoint: 1184.616, 1531.455
1.459
Material Properties
Material: Overburden
Strength Type: Mohr-Coulomb
Unsaturated Unit Weight: 16 kN/m3
Saturated Unit Weight: 20 kN/m3
Cohesion: 3 kPa
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Sand _Gravel (RTD)
Strength Type: Mohr-Coulomb
Unsaturated Unit Weight: 19 kN/m3
Saturated Unit Weight: 20 kN/m3
Cohesion: 3 kPa
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Ampthill Clay
Strength Type: Mohr-Coulomb
Unsaturated Unit Weight: 19 kN/m3
Saturated Unit Weight: 20 kN/m3
Cohesion: 35 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Side slope liner
Strength Type: Mohr-Coulomb
Unsaturated Unit Weight: 19 kN/m3
Saturated Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
W
Overburden
Sand _Gravel (RTD)
Ampthill Clay
Side slope liner
W (min)
W
1590
1580
1570
1560
1550
1540
1530
1140
1150
1160
1170
1180
1190
1200
1210
1220
1230

# Appendix C: Results of the capping stability analyses


Scale 1:337.4
Document
02 Mepal Ph9 SNRHW 1 in 2 waste slope
A075924 Mepal SNRHW Permit
gle/morgenstern-price
Every available surface
2.233
Factor of Safety: 2.233
Center: 1189.767, 1556.035
Radius: 24.488
Left Slip Surface Endpoint: 1172.285, 1538.887
Right Slip Surface Endpoint: 1194.824, 1532.075
Material Properties
Material: Overburden
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 3 kPa
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Sand _Gravel (RTD)
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 3 kPa
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Ampthill Clay
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 35 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Side slope liner
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 13 kN/m3
Cohesion: 5 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
2.233
W
W
Overburden
Sand _Gravel (RTD)
Ampthill Clay
Side slope liner
1580
1570
1560
1550
1540
1530
1140
1150
1160
1170
1180
1190
1200
1210
1220
1230


Scale 1:375.1
Document
03 Mepal Ph9 SNRHW 1 in 2 waste slope non circ
A075924 Mepal SNRHW Permit
janbu corrected
Every available surface
2.339
Factor of Safety: 2.339
Axis Location: 1187.248, 1555.374
Left Slip Surface Endpoint: 1171.970, 1538.887
Right Slip Surface Endpoint: 1191.271, 1533.260
Material Properties
Material: Overburden
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 3 kPa
Friction Angle: 20 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Sand _Gravel (RTD)
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 3 kPa
Friction Angle: 38 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Ampthill Clay
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 35 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Side slope liner
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 19 kN/m3
Cohesion: 5 kPa
Friction Angle: 25 degrees
Water Surface: Water Table
Custom Hu value: 1
2.339
W
Overburden
Sand _Gravel (RTD)
Ampthill Clay
Side slope liner
W
1590
1580
1570
1560
1550
1540
1530
1140
1150
1160
1170
1180
1190
1200
1210
1220
1230
1240

# Appendix B: Results of the waste mass slope stability analyses

# WYG Environment

Mepal Ph.9 SNRHW
Venner Capping Stability Assessment

**Notes:**
1.Method of Analysis: Limit equilibrium method after Jones and Dixon 1998.

**Slope Geometry**

| | | | |
|---|---|---|---|
| Height of slope | H | | 4.00 m |
| Slope gradient | | 1 : | 6.00 |
| Slope angle | $\beta$ | | 9.5 ° |

**Cover Layer Properties**

| | | |
|---|---|---|
| Apparent cohesion | c' | 0 $kN/m^2$ |
| Internal soil friction | $\phi$' | 28 ° |
| Unit weight (dry) | $\gamma_{dry}$ | 18 $kN/m^3$ |
| Unit weight (saturated) | $\gamma_{sat}$ | 21 $kN/m^3$ |
| Thickness | h | 1.00 m |

**Geosynthetic Interface Shear Strengths**

**Interface 1**

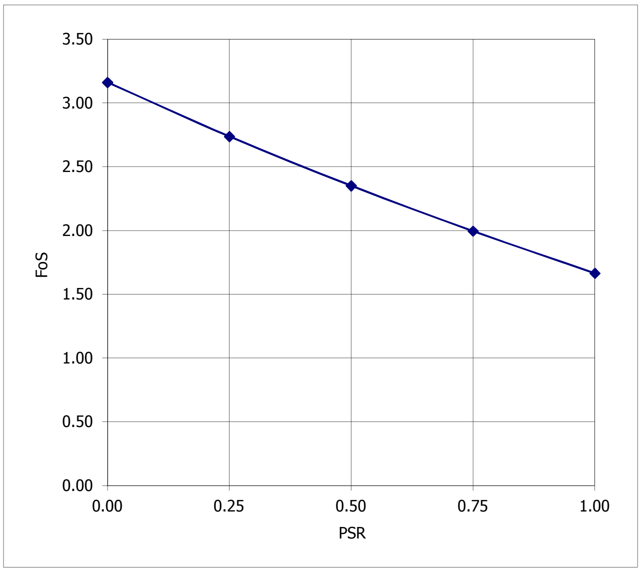
| | | | |
|---|---|---|---|
| *Cover soil - Mineral cap* | *Peak* | | |
| Upper interface friction angle | | $\delta_u$ | 24 ° |
| Upper interface cohesion intercept | | $\alpha_u$ | 0 $kN/m^2$ |
| *Mineral cap - Regulating layer* | *Peak* | | |
| Lower interface friction angle | | $\delta_l$ | 25 ° |
| Lower interface cohesion intercept | | $\alpha_l$ | 5 $kN/m^2$ |

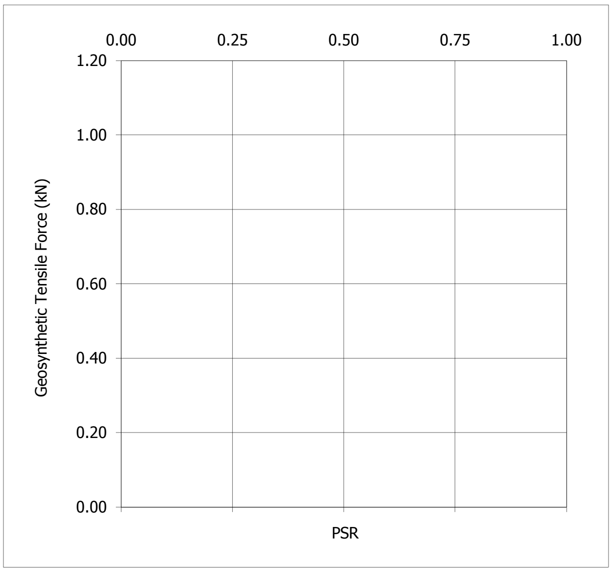
**Calculated Parameters**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Parallel submergence ratio | PSR | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | |
| Length of slope | L | 24.331 | 24.331 | 24.331 | 24.331 | 24.331 | m |
| Thickness of water | $h_w$ | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | m |
| Weight of active wedge | $W_A$ | 382.459 | 400.129 | 416.643 | 432.001 | 446.202 | kN |
| Weight of passive wedge | $W_P = N_P$ | 55.500 | 56.078 | 57.813 | 60.703 | 64.750 | kN |
| Pore pressure perpendicular to slope | $U_n$ | 0.00 | 58.10 | 112.40 | 162.89 | 209.59 | kN |
| Pore pressure in interwedge surface | $U_h$ | 0.00 | 0.31 | 1.25 | 2.81 | 5.00 | kN |
| Vertical pore pressure on passive wedge | $U_v$ | 0.00 | 1.88 | 7.50 | 16.88 | 30.00 | kN |
| Force normal to active wedge | $N_A$ | 377.26 | 336.64 | 298.78 | 263.69 | 231.37 | kN |

**Factor of Safety**

| | | | | | | |
|---|---|---|---|---|---|---|
| | a | 62.02 | 64.89 | 67.60 | 70.13 | 72.49 |
| Quadratic constants | b | -200.69 | -182.38 | -163.85 | -145.08 | -126.07 |
| | c | 14.68 | 13.10 | 11.63 | 10.26 | 9.00 |
| Factor of safety against cover soil sliding | FoS | 3.16 | 2.74 | 2.35 | 2.00 | 1.66 |



# WYG Environment

Mepal Ph.9 SNRHW
Venner Capping Stability Assessment

**Notes:**

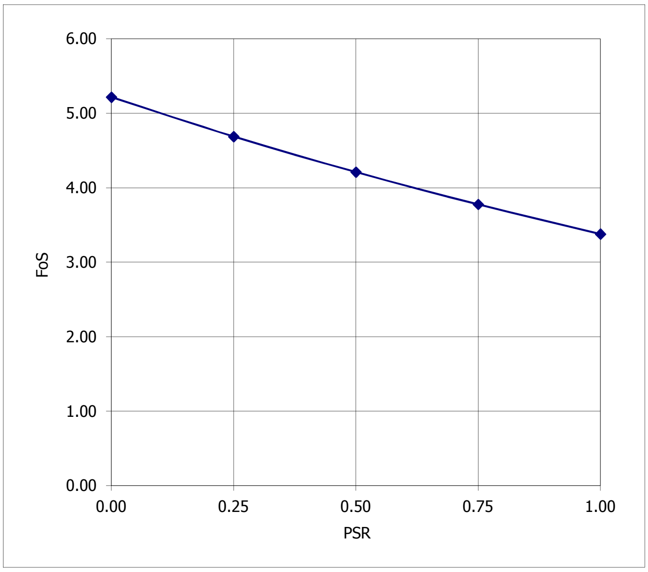
| | | | |
|---|---|---|---|
| **Slope Geometry** | | | |
| Height of slope | H | 4.00 | m |
| Slope gradient | 1 : | 6.00 | |
| Slope angle | $\beta$ | 9.5 | ° |
| **Cover Layer Properties** | | | |
| Apparent cohesion | c' | 0 | $kN/m^2$ |
| Internal soil friction | $\phi'$ | 28 | ° |
| Unit weight (dry) | $\gamma_{dry}$ | 18 | $kN/m^3$ |
| Unit weight (saturated) | $\gamma_{sat}$ | 21 | $kN/m^3$ |
| Thickness | h | 1.00 | m |

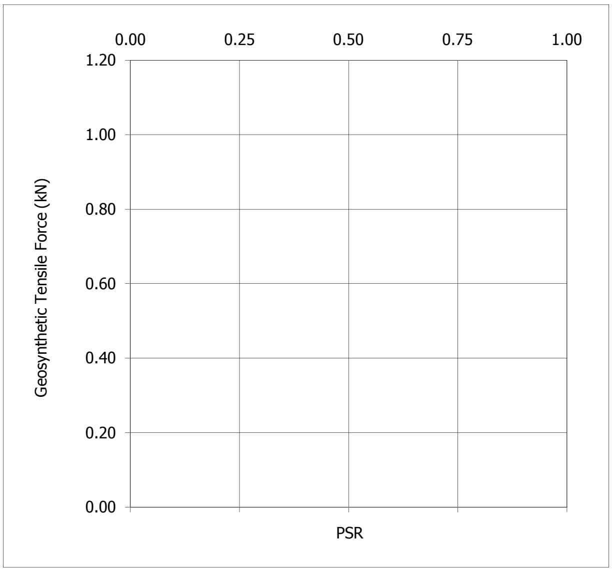
**Geosynthetic Interface Shear Strengths**

**Interface 2**

| | | | | |
|---|---|---|---|---|
| *Mineral cap - Regulating layer* | *Peak* | | | |
| Upper interface friction angle | | $\delta_u$ | 25 | ° |
| Upper interface cohesion intercept | | $\alpha_u$ | 5 | $kN/m^2$ |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Calculated Parameters** | | | | | | | |
| Parallel submergence ratio | PSR | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | |
| Length of slope | L | 24.331 | 24.331 | 24.331 | 24.331 | 24.331 | m |
| Thickness of water | $h_w$ | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | m |
| Weight of active wedge | $W_A$ | 382.459 | 400.129 | 416.643 | 432.001 | 446.202 | kN |
| Weight of passive wedge | $W_P = N_P$ | 55.500 | 56.078 | 57.813 | 60.703 | 64.750 | kN |
| Pore pressure perpendicular to slope | $U_n$ | 0.00 | 58.10 | 112.40 | 162.89 | 209.59 | kN |
| Pore pressure in interwedge surface | $U_h$ | 0.00 | 0.31 | 1.25 | 2.81 | 5.00 | kN |
| Vertical pore pressure on passive wedge | $U_v$ | 0.00 | 1.88 | 7.50 | 16.88 | 30.00 | kN |
| Force normal to active wedge | $N_A$ | 377.26 | 336.64 | 298.78 | 263.69 | 231.37 | kN |
| ***Factor of Safety*** | | | | | | | |
| | a | 62.02 | 64.89 | 67.60 | 70.13 | 72.49 | |
| Quadratic constants | b | -328.53 | -309.38 | -290.06 | -270.56 | -250.88 | |
| | c | 26.01 | 24.36 | 22.81 | 21.38 | 20.06 | |
| Factor of safety against mineral cap sliding | FoS | 5.22 | 4.69 | 4.21 | 3.78 | 3.38 | |



# WYG Environment

Mepal Ph.9 SNRHW
Venner Capping Stability Assessment

**Notes:**
1.Method of Analysis: Limit equilibrium method after Jones and Dixon 1998.

## Slope Geometry

| | | | |
|---|---|---|---|
| Height of slope | H | 4.00 | m |
| Slope gradient | 1 : | 6.00 | |
| Slope angle | $\beta$ | 9.5 | ° |

## Cover Layer Properties

| | | | |
|---|---|---|---|
| Apparent cohesion | c' | 0 | $kN/m^2$ |
| Internal soil friction | $\phi$' | 28 | ° |
| Unit weight (dry) | $\gamma_{dry}$ | 18 | $kN/m^3$ |
| Unit weight (saturated) | $\gamma_{sat}$ | 21 | $kN/m^3$ |
| Thickness | h | 1.00 | m |

## Geosynthetic Interface Shear Strengths

### Interface 1

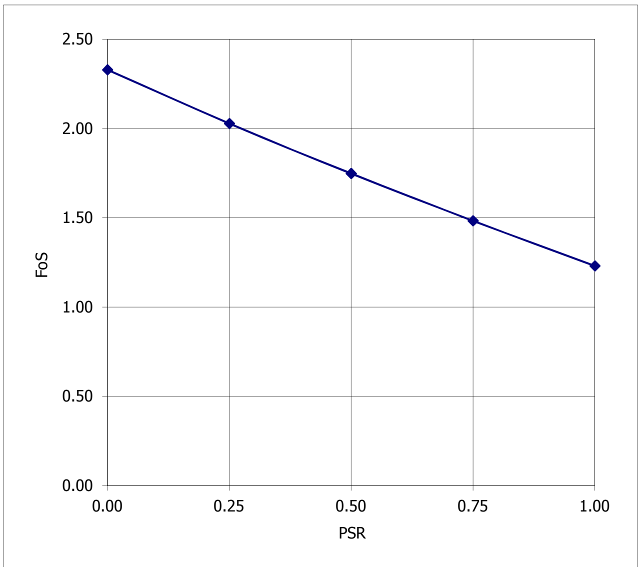
| | | | |
|---|---|---|---|
| *Cover soil - Mineral cap* | *Residual* | | |
| Upper interface friction angle | $\delta_u$ | 17 | ° |
| Upper interface cohesion intercept | $\alpha_u$ | 0 | $kN/m^2$ |
| *Mineral cap - Regulating layer* | *Residual* | | |
| Lower interface friction angle | $\delta_l$ | 25 | ° |
| Lower interface cohesion intercept | $\alpha_l$ | 0 | $kN/m^2$ |

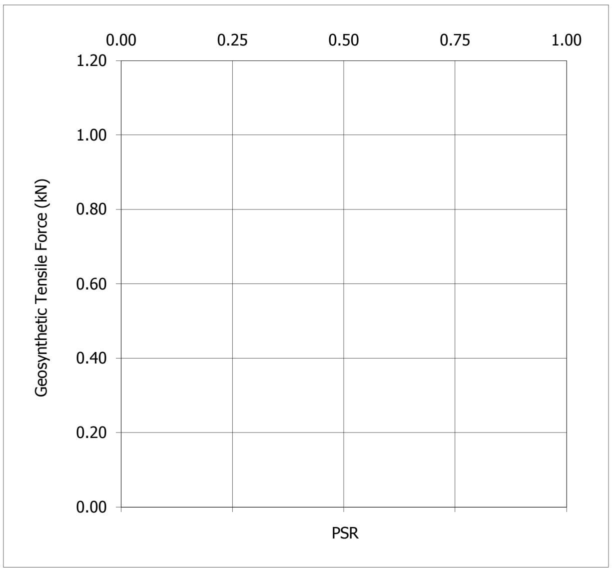
## Calculated Parameters

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Parallel submergence ratio | PSR | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | |
| Length of slope | L | 24.331 | 24.331 | 24.331 | 24.331 | 24.331 | m |
| Thickness of water | $h_w$ | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | m |
| Weight of active wedge | $W_A$ | 382.459 | 400.129 | 416.643 | 432.001 | 446.202 | kN |
| Weight of passive wedge | $W_P = N_P$ | 55.500 | 56.078 | 57.813 | 60.703 | 64.750 | kN |
| Pore pressure perpendicular to slope | $U_n$ | 0.00 | 58.10 | 112.40 | 162.89 | 209.59 | kN |
| Pore pressure in interwedge surface | $U_h$ | 0.00 | 0.31 | 1.25 | 2.81 | 5.00 | kN |
| Vertical pore pressure on passive wedge | $U_v$ | 0.00 | 1.88 | 7.50 | 16.88 | 30.00 | kN |
| Force normal to active wedge | $N_A$ | 377.26 | 336.64 | 298.78 | 263.69 | 231.37 | kN |

## Factor of Safety

| | | | | | | |
|---|---|---|---|---|---|---|
| | a | 62.02 | 64.89 | 67.60 | 70.13 | 72.49 |
| Quadratic constants | b | -148.78 | -136.06 | -122.74 | -108.79 | -94.23 |
| | c | 10.08 | 9.00 | 7.98 | 7.05 | 6.18 |
| Factor of safety against cover soil sliding | FoS | 2.33 | 2.03 | 1.75 | 1.48 | 1.23 |



# WYG Environment

Mepal Ph.9 SNRHW
Venner Capping Stability Assessment

**Notes:**

| | | | | |
|---|---|---|---|---|
| **Slope Geometry** | | | | |
| Height of slope | H | | 4.00 | m |
| Slope gradient | | 1 : | 6.00 | |
| Slope angle | $\beta$ | | 9.5 | ° |
| **Cover Layer Properties** | | | | |
| Apparent cohesion | c' | | 0 | $kN/m^2$ |
| Internal soil friction | $\phi$' | | 28 | ° |
| Unit weight (dry) | $\gamma_{dry}$ | | 18 | $kN/m^3$ |
| Unit weight (saturated) | $\gamma_{sat}$ | | 21 | $kN/m^3$ |
| Thickness | h | | 1.00 | m |

**Geosynthetic Interface Shear Strengths**

**Interface 2**

| | | | |
|---|---|---|---|
| *Mineral cap - Regulating layer* | *Residual* | | |
| Upper interface friction angle | $\delta_u$ | 25 | ° |
| Upper interface cohesion intercept | $\alpha_u$ | 0 | $kN/m^2$ |

| **Calculated Parameters** | | | | | | | |
|---|---|---|---|---|---|---|---|
| Parallel submergence ratio | PSR | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | |
| Length of slope | L | 24.331 | 24.331 | 24.331 | 24.331 | 24.331 | m |
| Thickness of water | $h_w$ | 0.00 | 0.25 | 0.50 | 0.75 | 1.00 | m |
| Weight of active wedge | $W_A$ | 382.459 | 400.129 | 416.643 | 432.001 | 446.202 | kN |
| Weight of passive wedge | $W_P = N_P$ | 55.500 | 56.078 | 57.813 | 60.703 | 64.750 | kN |
| Pore pressure perpendicular to slope | $U_n$ | 0.00 | 58.10 | 112.40 | 162.89 | 209.59 | kN |
| Pore pressure in interwedge surface | $U_h$ | 0.00 | 0.31 | 1.25 | 2.81 | 5.00 | kN |
| Vertical pore pressure on passive wedge | $U_v$ | 0.00 | 1.88 | 7.50 | 16.88 | 30.00 | kN |
| Force normal to active wedge | $N_A$ | 377.26 | 336.64 | 298.78 | 263.69 | 231.37 | kN |
| ***Factor of Safety*** | | | | | | | |
| | a | 62.02 | 64.89 | 67.60 | 70.13 | 72.49 | |
| Quadratic constants | b | -208.53 | -189.38 | -170.06 | -150.56 | -130.88 | |
| | c | 15.38 | 13.72 | 12.18 | 10.75 | 9.43 | |
| Factor of safety against mineral cap sliding | FoS | 3.29 | 2.84 | 2.44 | 2.07 | 1.73 | |