16 August 2012 Document Ref. 2013-0007AC

Instant Waste
PO Box 419
Morley Business Centre
Morley, WA, 6943

Attention: Mr Sam Mangione

RE: GEOTECHNICAL STABILITY REVIEW, OPAL VALE LANDFILL, CHITTY ROAD, TOODYAY, WA

1 INTRODUCTION AND BACKGROUND INFORMATION

CMW Geosciences Pty Ltd (CMW) was authorised by Instant Waste (Sam Mangione) to undertake embankment stability assessments of a proposed landfill under pseudo static seismic loading conditions at the Opal Vale Landfill, located at Chitty Road, Toodyay, WA. We understand that the Department of Environment and Conservation (DEC) require this additional information before they can accept the overall landfill design.

The proposed landfill is located in a disused site previously used to mine clay materials for brick making. The existing slopes of the clay pit excavation are up to approximately 12 metres high and have been cut back to an angle of approximately 1V:0.36H (70 degrees). We have been advised that the batters have not been the subject of instability and have remained stable for approximately the last 10 years despite exposure to the elements i.e. rainfall events. We understand that the landfill will comprise Class II waste, defined as a mixture of:

- Clean Fill
- Type 1 Inert Waste
- Putrescible Wastes
- Contaminated solid waste materials that meets the acceptance criteria specified for Class II landfills (possibly with specific licence conditions)
- Type 2 Inert Wastes (with specific licence conditions)
- Type 1 and Type 2 Special Wastes (for registered sites as approved under the Controlled Waste Regulations).

We have liaised with I W Projects (Ian Watkins) to determine the staging associated with the construction of the landfill cells, the methodology associated with the installation of the liner and the backfilling of the Class II waste.

We understand that the construction of Cell One will include a cut to fill (actual quantum of earthworks is unknown) to form 1V:3H (18 degree) batter slopes. Cell One will be contained by a temporary clay bund near the Cell One boundary but it is our understanding that this will not be designed to support any loads from the waste materials. We also understand that the base of the landfill will be graded back slightly into the pit wall to assist with global stability and the control of leachate. Once this cell has been backfilled with Class II waste the remaining 6 cells will be constructed in succession, over a number of years, to complete the landfill. This sequential development will require that the existing slope heights and angles remain ‘as is’ until the construction occurs at each future cell location.

The scope of work and associated terms and conditions of our engagement were detailed in our proposal letter referenced 2013-0007AB dated 26 July 2012.

2 SUMMARY OF DATA SUPPLIED

We have been supplied with SGS laboratory test results, dated December 2010, which included 6 Atterberg Limit and permeability tests on the clay samples for liner design purposes. We have also been supplied with a copy of the Stass Environmental (Stass) Groundwater Review report dated June 2011.

We have not currently had the opportunity to complete a site investigation to quantify a ground model or obtain specific strength properties of the materials present. We have used the Atterberg Limits obtained during laboratory testing (Table 1 below) and published correlations between the Liquidity Index (LI) and undrained shear strength (Su) to estimate the likely strength of the materials present on site. Based on the laboratory tests provided we would expect the clay undrained shear strength (Su) to be in excess of 200kPa.

Table 1: Laboratory Test Results – SGS Australia Pty Ltd dated December 2010

Sample	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Linear Shrinkage	Bulk Density (t/M^3)	Moisture Content (%)
OPAL 1	38	24	14	4.0	1.75	15.8
OPAL 2	35	22	13	5.5	1.63	20.2
OPAL 3	36	23	13	5.0	1.81	14.5
OPAL 4	39	24	15	2.5	1.67	18.5
OPAL 5	35	24	11	2.5	1.76	15.0
OPAL 6	39	23	16	4.5	1.67	18.5

Where the insitu moisture content is less than the PL as is the case with the samples tested, the soil type is likely to be desiccated and pseudo over consolidated (due to drying). Based on this model we would expect the type of failure to be brittle if sheared. This has an implication on safe working distances from the existing slope which are discussed later in this report.

3 GEOLOGICAL MODEL

The 1:250,000 (Perth) Sheet produced by the Geological Survey of Western Australia (GSWA) indicates that the site area is located within the geological units outlined in Table 2 below.

Table 2: Geological Units (1:250,000 Perth GSWA)

Unit	Description
Czl	Laterite chiefly massive, but includes overlying pisolithic gravel and laterised sand.

Alm	Muscovite – chlorite phyllitic schist.
Qrc	Colluvium including valley filled deposits variably laterised and podsolised.
Note: The map also depicts the presence of nearby quarries and abandoned quarries for pisolithic laterite gravel, clay, building stone and iron. It also suggests areas where bedrock is obscured by both residual and colluvium deposits.	

The Stass report described the area to be filled is a void cut into deep micaceous clays formed from the weathering of schists of the Jimperding Metamorphic Belt. These schists have been subjected to a long period of weathering, in the Mesozoic - Cainozoic, to produce the laterite erosion surface, of which a remnant caps the nearby hills. The groundwater level was measured at 4 locations by Stass during their groundwater study which indicated depths ranging from 7.41m (266.49 mAHD) to 18.21m (281.15 mAHD) below ground levels. These monitoring wells were located around the southern boundary of the proposed landfill area.

We have reviewed photographs of the cuts provided in the Stass Report which show slopes with no signs of instability despite being exposed to the elements for approximately 10 years, other than signs of surficial weathering.

4 STABILITY ASSESSMENT

The degree of stability of a slope is expressed as the factor of safety, which is the ratio of the forces resisting failure to the driving forces causing instability. Theoretical failure of a slope is possible when the factor of safety is ≤1.0, while increasing values above 1.0 indicate improving stability. Conventional slope stability analyses usually result in a minimum value of 1.5 being adopted for permanent slopes under static conditions but other considerations such as the geology, slope geometry, groundwater and history of the site, site use etc are taken into account in assessing the acceptable degree of risk.

Cross sections were drawn through strategic areas of the project where shown on the appended site plan. These sections were selected as being the most appropriate for computer stability analyses because the slopes were the steepest and the before and after earthworks profiles are significantly different. The cross-sections were analysed for deep seated circular slips. The slope stability software program used was SLIDE version 6.0.

Strength values for overconsolidated clays and clay shales range from peak undrained shear strengths down to as low as residual shear strength after displacement has occurred. The decision process regarding the selection of the design strength of these materials includes both technical and non-technical issues such as:

- Structural and groundwater conditions of the material
- Presence and inclination of bedding planes
- Presence of relict landslides in the area
- Other discontinuities in the mass
- Design life of the project.

There are also a multitude of variable properties relating to the landfill waste including grain size distribution, porosity, moisture content, hydraulic conductivity, changes in ground / surface water conditions, unit weight, strength, compressibility etc. However, the properties most germane to slope stability analysis are unit weight and shear strength which we have estimated based on our research into typical engineering properties of landfill waste.

We have reviewed the consistency terms provided in AS1726-1993 for cohesive soils which depict stiff to very stiff clays with undrained shear strengths ranging from >50kPa to 200kPa. These published values correlate to the LI / Su correlation provided above. Further anecdotal evidence provided by I W Projects suggests that the exposed slopes have not been the subject of any slope instability and there are no signs of instability or tension cracks. On this basis we have analysed the worst case (steepest and highest) existing slopes using soil strength parameters as presented in Table 3 below.

Table 3: Soil Strength Parameters

Description	Undrained	Drained	
	Su (kPa)	C' (kPa)	Ø' (degrees)
Very Stiff clayey silts and silty clays	100	8 to 12	28 to 32
Very Stiff Engineered Fill	150	10	32
Class II Landfill Materials	40	3 to 5	20 to 25

The earthquake ground motion used for pseudo static analysis was determined using AS1170.4-2007, part 4 Earthquake Actions in Australia. We assigned a Level 4 for the structural importance of the site and used a class of Ce to depict a shallow soil site. The design working life of the landfill provided for an annual probability of exceedance (P) 1/2500 with an earthquake design category (EDC) of II. Following our Dynamic analysis calculation we determined that the horizontal design response spectrum was 0.23. The minimum factors if safety obtained for each scenario analysed in provided in Table 4 below.

Table 4: Minimum Factors of Safety

Soil Parameters	Conditions of analysis	Type of Failure	Factor of Safety
Drained (Long Term)	Existing slope height and angle (70 degrees) with highly saturated ground conditions - drained soil shear strength parameters	Circular	0.9
Drained (Long Term)	Existing slope height and angle (70 degrees) with no groundwater - drained soil shear strength parameters	Circular	1.1
Undrained (Short Term)	Existing slope height and angle (70 degrees) (Su ≥ 100kPa)	Circular	2.9
Undrained (Short Term)	Existing slope height and angle with Seismic Load - horizontal ground acceleration 0.23 (70 degrees) (Su ≥ 100kPa)	Circular	2.3
Drained (Long Term)	Proposed slope angle (1V:3H) with highly saturated ground conditions	Circular	2.1

Undrained (Short Term)	Proposed slope angle (1V:3H) with Seismic Load - horizontal ground acceleration 0.23	Circular	1.5
Drained (Long Term)	Cell One Completed without Seismic Load	Circular	1.3
Undrained (Short Term)	Cell One Completed with Seismic Load - horizontal ground acceleration 0.23	Circular	1.1

As can be seen from the above results, with drained soil shear strength parameters, the cross-section was found to have a minimum factor of safety of 0.9 for an overburden slip extending approximately 3 metres back from the crest of the slope. This factor is for 'worst case' highly saturated ground conditions, which should not occur on the site other than during temporary extreme storm conditions and accordingly the result is considered to be satisfactory. The analysis of a dry slope with drained soil strength parameters produced a factor of safety of 1.1.

Using undrained soil shear strength parameters the factor of safety was 2.9. Then using conservative undrained soil strength parameters under pseudo-static loading produced a factor of safety in excess of 2. The slope was then analysed at proposed angles of 1V:3H (18 degrees) with minor cuts at the crest and bulk filling placed and compacted at the toe of the slope. This produced a factor of safety in excess of 2 for a high phreatic surface while a factor of safety of 1.5 using undrained soil shear strength parameters under pseudo-static loading was determined.

Cell One was then analysed under seismic loading to access approximate safe batter angles of the waste materials. This produced a factor of safety of 1.1 for slope angles not exceeding 1V:2H (approximately 26 degrees). A design factor of safety >1.0 is satisfactory under seismic loading.

5 COMMENTS

We have reviewed and relied upon laboratory testing, a site specific groundwater report, geological maps and Australian Standards where appropriate. There are still a number of variables that affect the stability of the cut slopes and landfill. Despite these limitations we consider that once the batter slopes have been earthworked to form 1V:3H batter slope angles, the stability of the site should improve even under pseudo static loading. The following comments and qualifications must be noted:

- The lowest factors of safety were generated from the natural slopes during drained shear strength parameters. This analysis leads to slope failure when the land profile analysed was highly saturated. We therefore consider that the proposed landfill will ease the land contours and improve stability of the site. As suggested, the construction of each cell will happen sequentially so all existing slopes that are not affected by earthworks will need to be monitored for signs of instability and we should be contacted for further advice should slope movements occur.
- Based on the slip circle stability assessment, setback distances from the top and bottom of exposed natural slopes should be imposed as elevated ground conditions or high surcharge loads are likely to cause slope instability. We therefore suggest a setback / exclusion distance of approximately ≥10 metres be adopted in the absence of site specific shear strength parameters.
- We have analysed the soil fill materials to reflect a level of compaction suitable for Engineer certification. We therefore require that site won materials from excavations (excluding topsoil) should be compacted in layers not exceeding 300 mm in loose thickness compacted with a suitable roller at ±3% of the optimum moisture content. We understand that the specification

for this project includes compaction of materials to not less than 95% of the maximum (standard) Dry Density Ratio in accordance with the Main Road Specification 302 - Earthworks.

- During earthworks, site visits must be made by a suitably experienced Geotechnical Engineer or Engineering Geologist, who is familiar with the contents of this report, to ensure that topsoil stripping is carried out adequately (where appropriate), that the cut to fill earthworks are conducted in accordance with the specification and to audit compaction of earthworks. CMW would be pleased to perform this function if required.
- We have not undertaken settlement analysis and suggest that the likely depth of filling be determined so that the quantum of differential and total settlements can be established.
- The factor of safety for the completion of Cell One suggested finished slope angles of 1V:2H (approximately 26 degrees) should be appropriate for the interim exposed face of the landfill materials. This angle should not be exceeded unless consistent landfill shear strengths parameters can be confirmed and provided to us for use in additional stability analysis or onsite trails can be conducted to assess appropriate batter angles. The finished slopes of each cell could be benched to increase the overall stability of the slope but this will reduce landfill volume until the new cell is ready for filling.
- Site specific geotechnical investigations should be undertaken to confirm our findings with consideration given to relevant laboratory testing. As discussed above, we have adopted assumed shear strength parameters for the natural soils, filled ground and the Class II landfill materials. There are a number of variables that influence these parameters and our research into these correlations must be validated.

6 CONCLUSION

In the short term, the existing 70 degree slope during static conditions has an adequate factor of safety. However, the lowest factors of safety were obtained in the long term for the existing steep slopes when the phreatic surface is highly elevated. Unfortunately we are unable to determine what time period long term could be. Once the slopes are recontoured to 18 degrees, then they are stable even under seismic loading with the parameters used.

7 CLOSURE

Should you require any further information or clarification regarding our proposal, please do not hesitate to contact the undersigned.

For and on behalf of
CMW Geosciences Pty Ltd

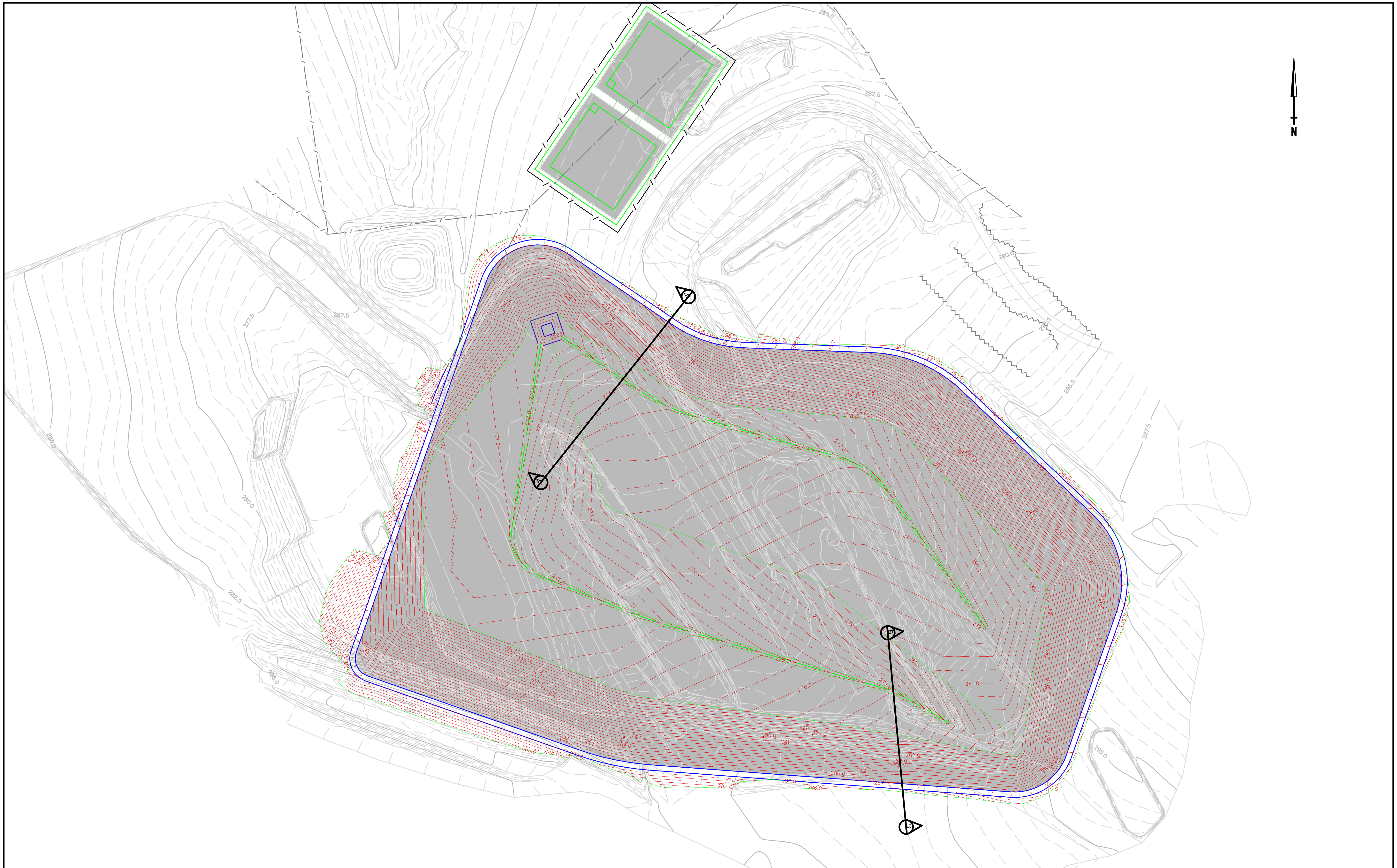
Phil Chapman

Managing Director

Distribution: 1 copy to Opal Vale Landfill (electronic) Original held by CMW Geosciences Pty Ltd

Appendix A

Site Plan

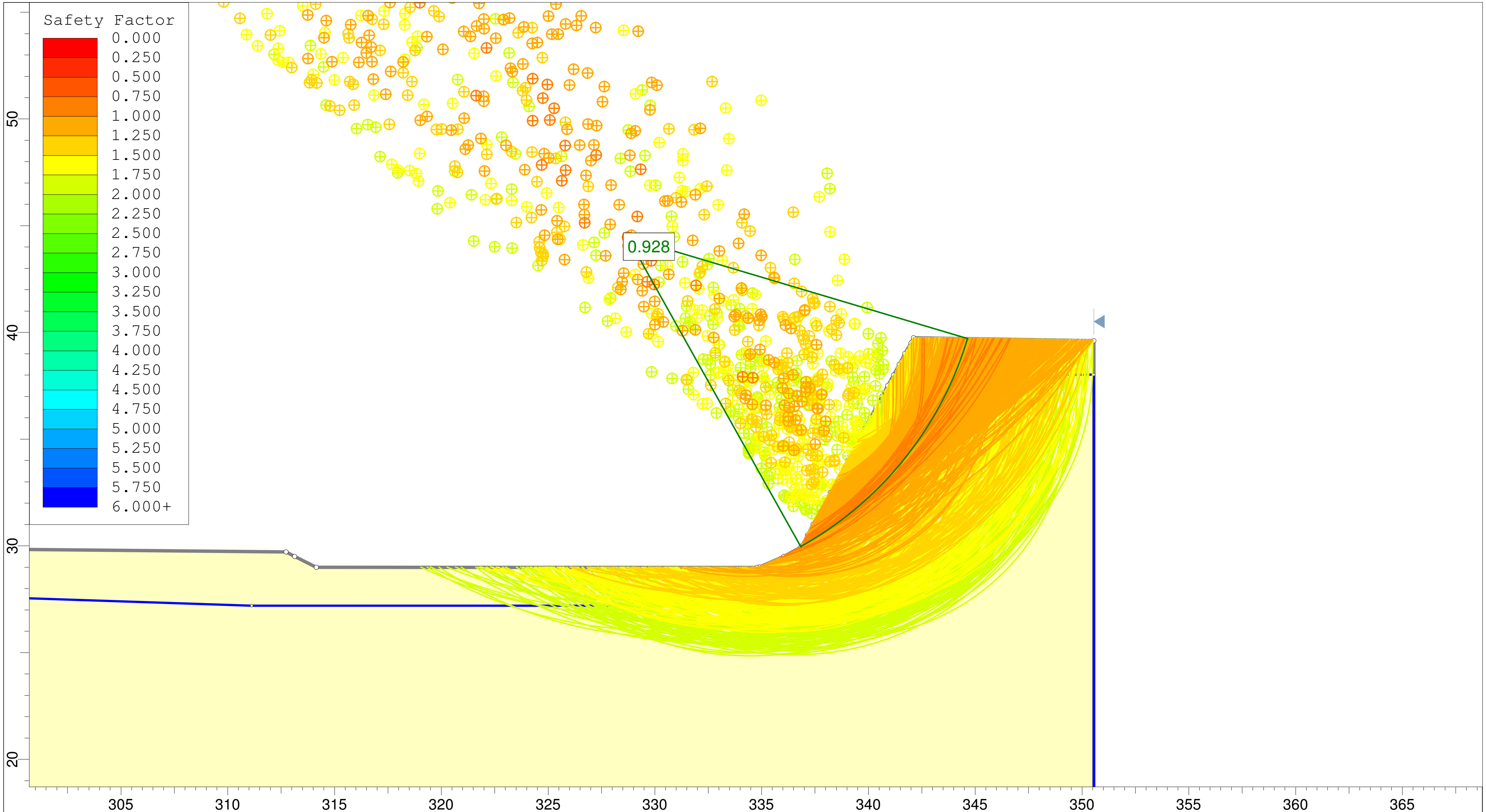


CMW Geosciences Pty Ltd
Chapman Morton Woodward

CLIENT: OPAL VALE PTY LTD			
PROJECT: PROPOSED CLASS II LANDFILL LOT 11 CHITTY ROAD, TOODYAY			
TITLE: SITE PLAN			
DRAWN:	LJT	DATE:	20/08/12
CHECKED:	PDC	DATE:	20/08/12
PROJECT No.	2013-0007	FIGURE No.	01
SCALE:	1:2000	0 20 40 60 80 100 Metres	
REVISION:	A	DATE:	20/08/12
			A3

Appendix B

Stability Analysis



CMW Geosciences Pty Ltd Chapman Morton Woodward	*Project* OPAL VALE LANDFILL		
	Slope stability analysis condtion Existing Slope Height and Angle (70 Deg) with highly saturated ground conditions - drained soil shear strength parameters		
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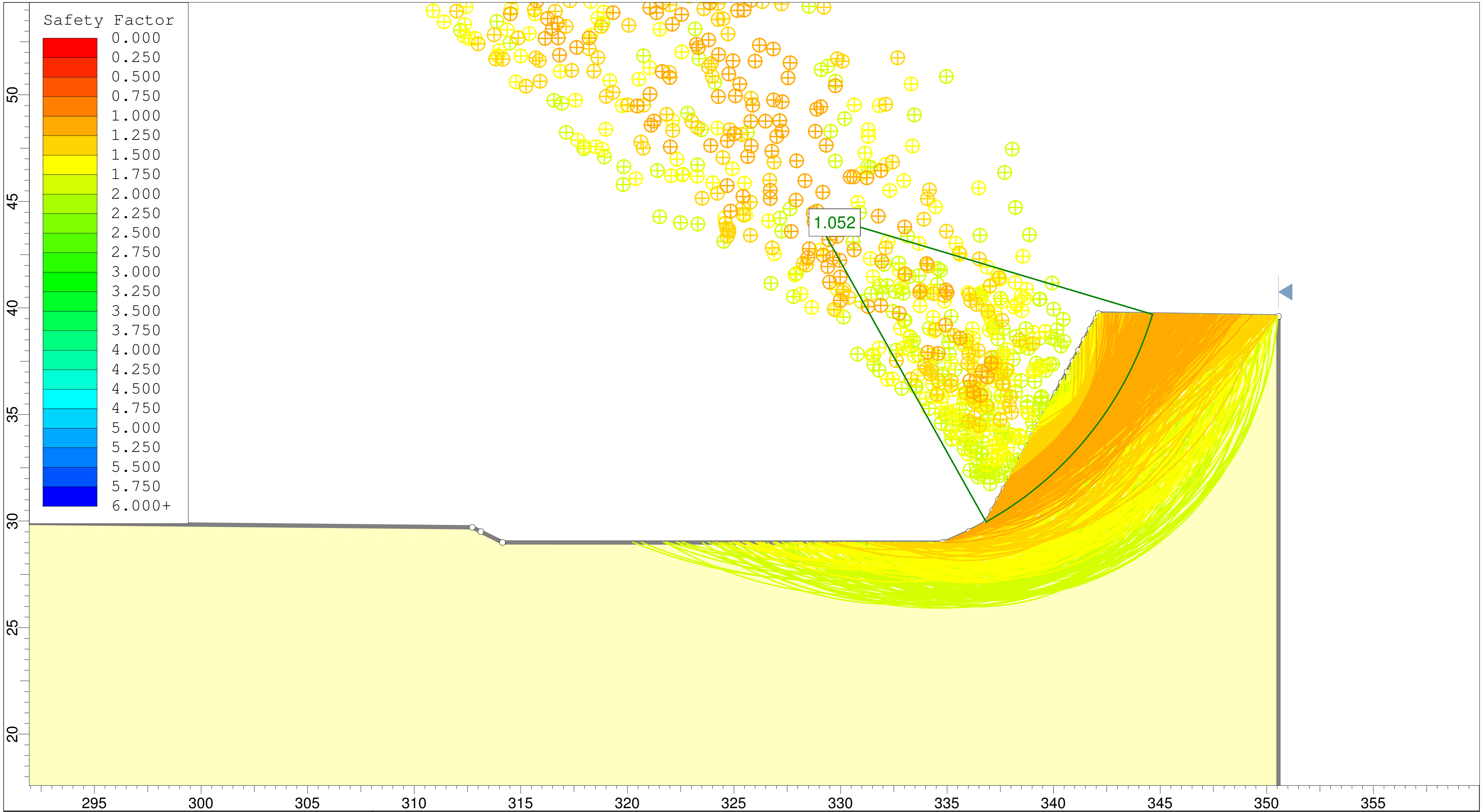

CMW Geosciences Pty Ltd Chapman Morton Woodward	*Project* OPAL VALE LANDFILL		
	Slope stability analysis condtion Existing Slope Height and Angle (70 Deg) with no ground conditions - drained soil shear strength parameters		
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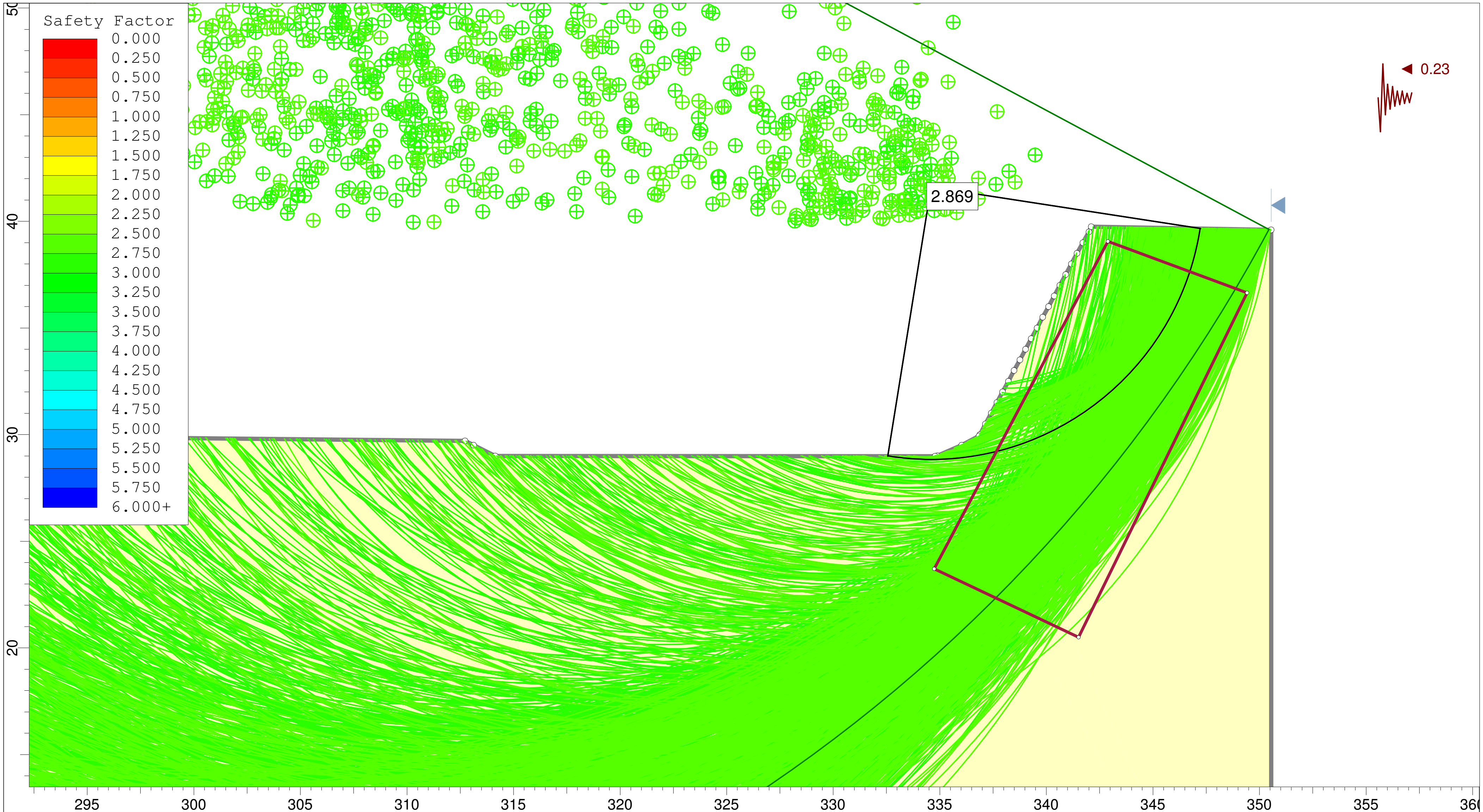

CMW Geosciences Pty Ltd Chapman Morton Woodward	*Project* OPAL VALE LANDFILL		
	Slope stability analysis condtion Existing slope height and angle (70 deg) (Su>100kPa)		
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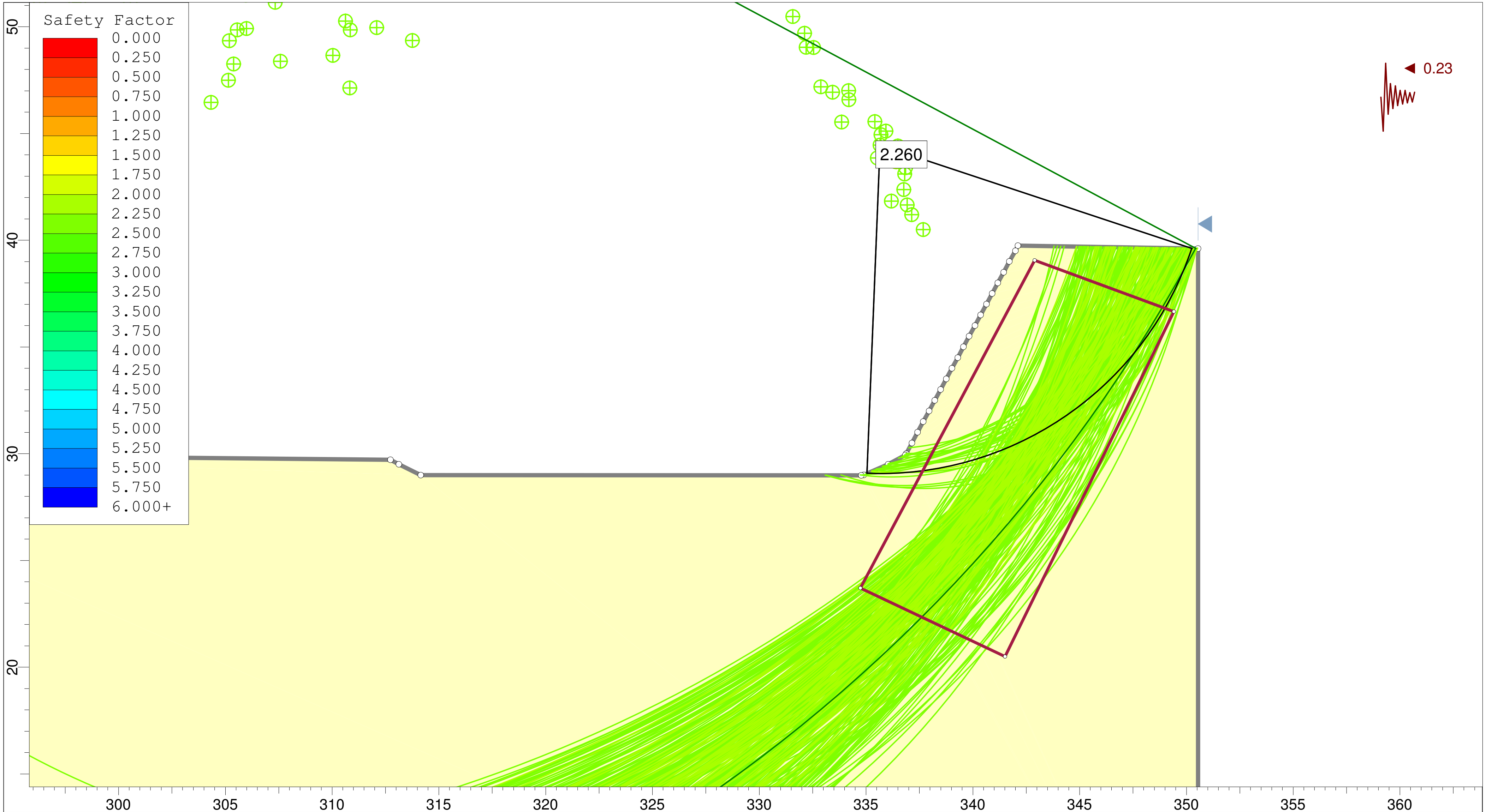

Chapman Morton Woodward

Project	OPAL VALE LANDFILL		
Slope stability analysis condtion	Existing slope height and angle with Seismic Load - horizontal ground acceleration 0.23 (70 deg) (Su>100kPa)		
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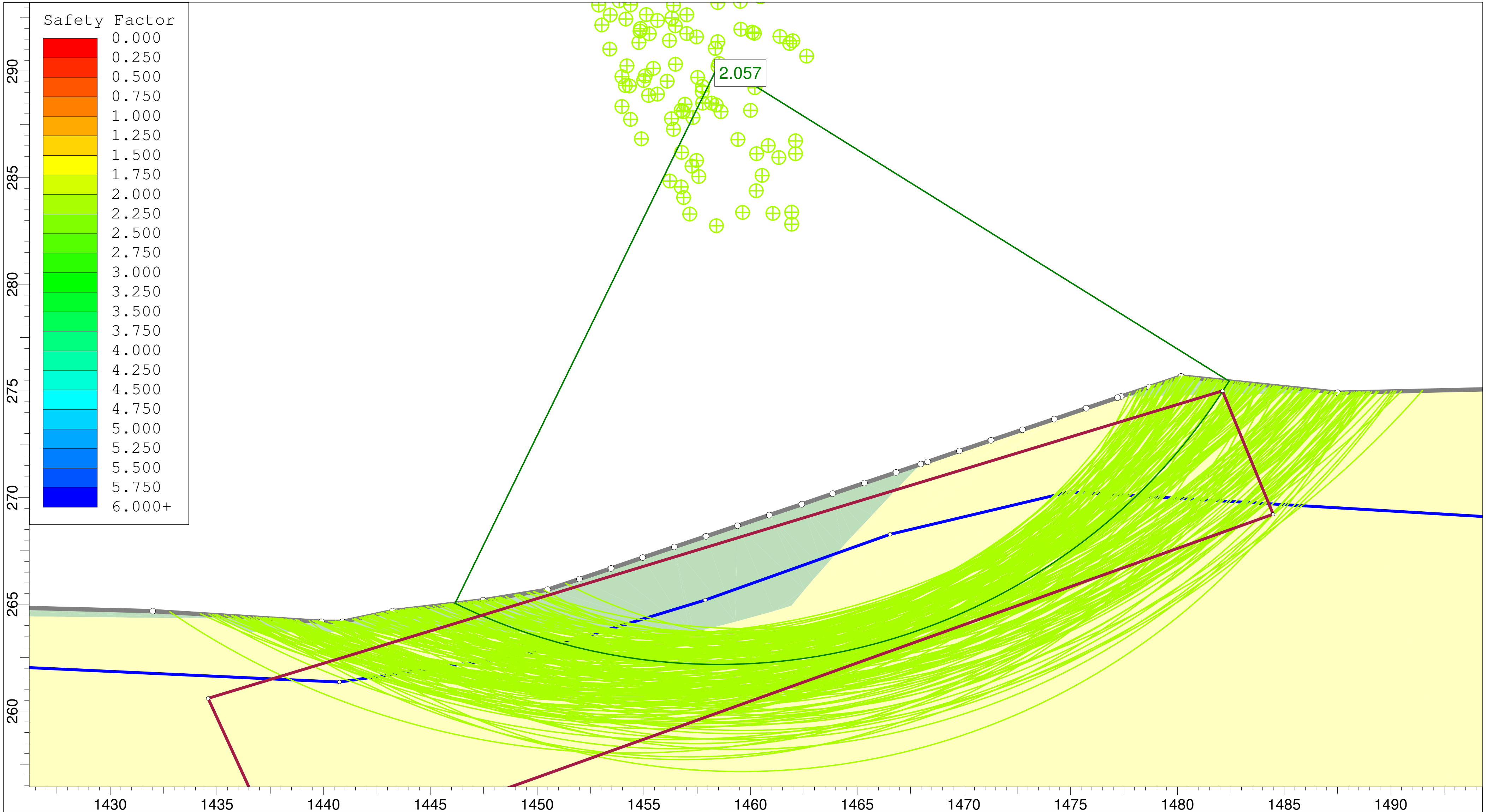

CMW Geosciences Pty Ltd
Chapman Morton Woodward

Project	OPAL VALE LANDFILL		
Slope stability analysis condtion	Proposed slope angle (1V:3H) with highly saturated ground conditions		
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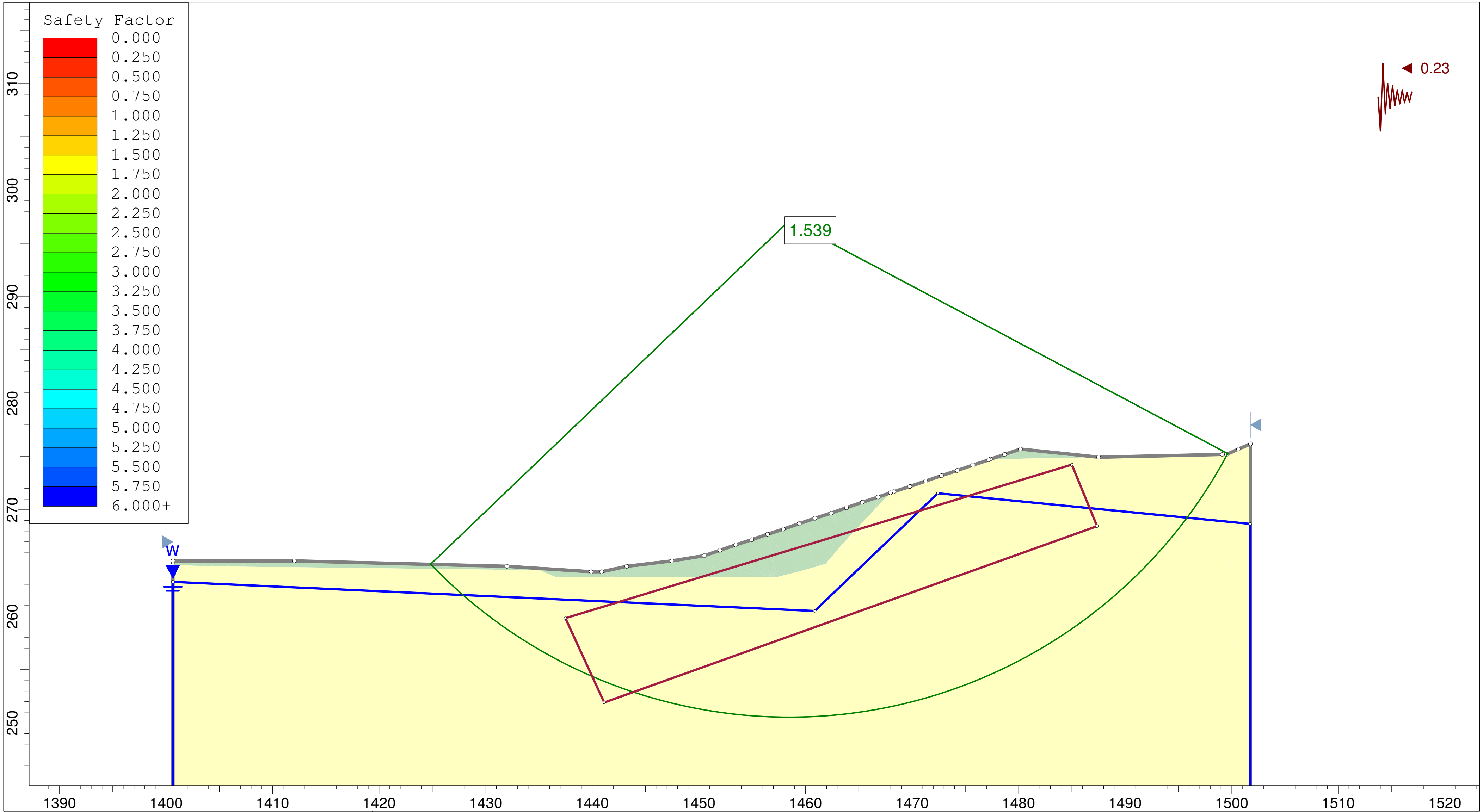

Geosciences Pty Ltd
Chapman Morton Woodward

Project	OPAL VALE LANDFILL		
Slope stability analysis condtion	Cell One Completed with Seismic Load - Horizontal ground acceleration 0.23		
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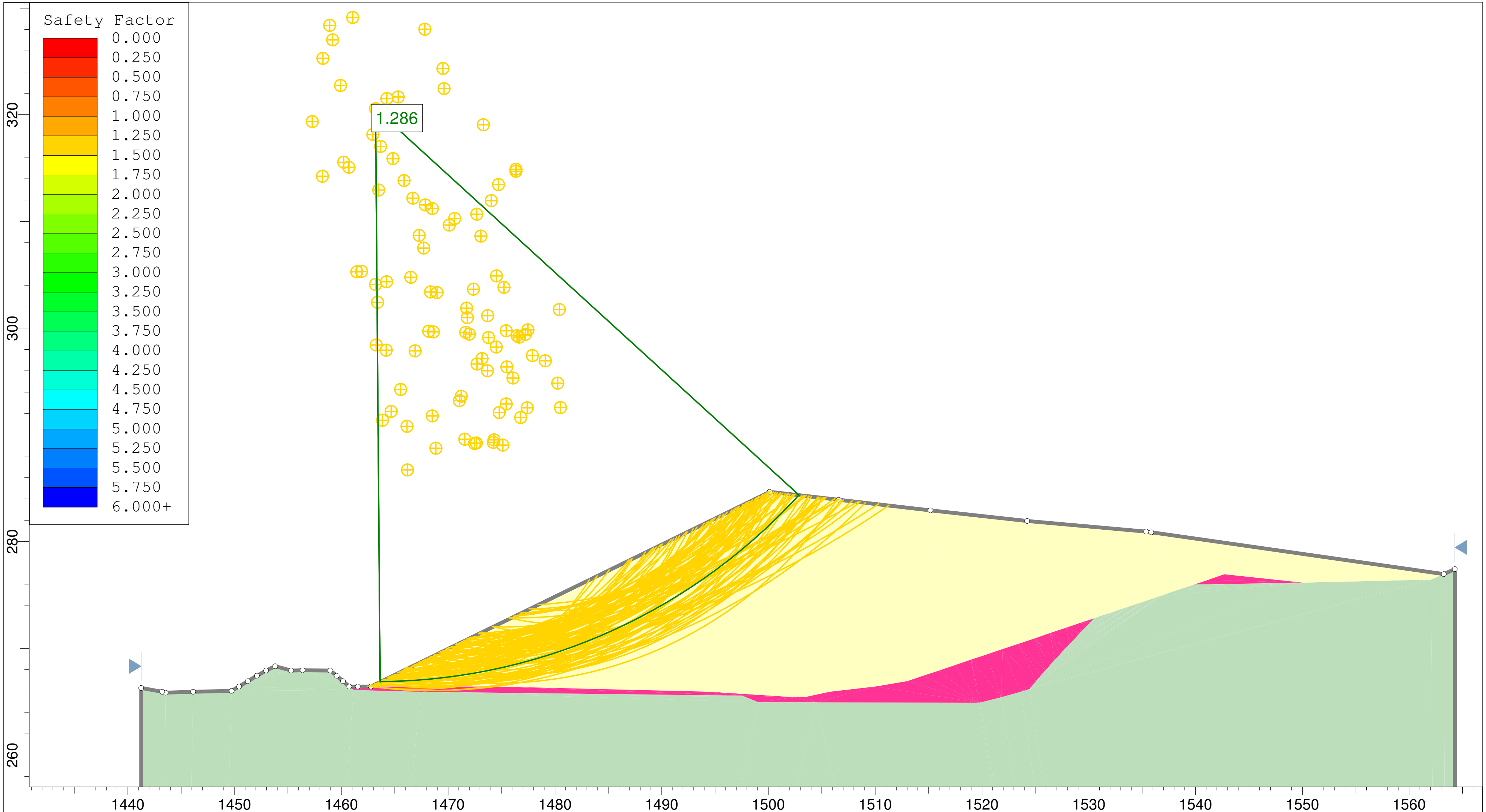

CMW Geosciences Pty Ltd
Chapman Morton Woodward

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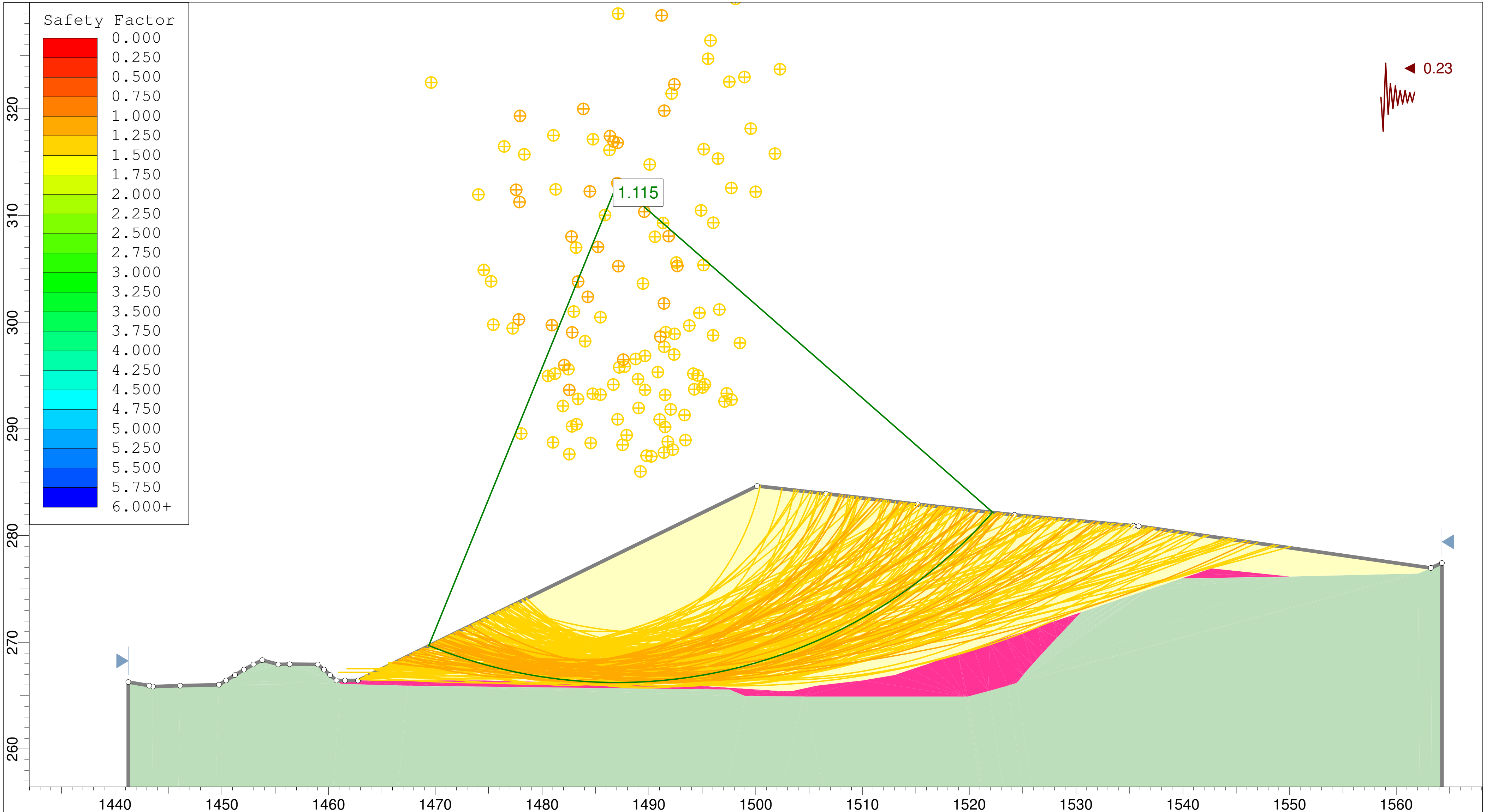

CMW Geosciences Pty Ltd
Chapman Morton Woodward

Project	OPAL VALE LANDFILL
Slope stability analysis condtion	Cell One Completed with Seismic Load - Horizontal ground acceleration 0.23
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