NSW Department of Commerce
NSW Water Solutions

Hunter Water Corporation

TILLEGRA DAM DESIGN - CONSULTANCY 361802

Storage Rim Stability and Seepage Potential
Engineering Geotechnical Report

VOLUME IV

Report No. 08–GN31A–R2, Final Report V 4.1
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VOLUME IV

Pells Sullivan Meynink Pty Ltd (PSM 2009), Tillegra Dam – Storage Rim Landslide Geotechnical Assessment, Report PSM1271.R1, Final Report, February 2009.

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Date: 3 February 2009

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ATTENTION: DENE JAMIESON

Dear Dene,

RE: <u>TILLEGRA DAM – STORAGE RIM LANDSLIDE GEOTECHNICAL ASSESSMENT</u>

Please find attached our final report presenting the storage rim landslide assessment for Tillegra Dam. We trust this report is in keeping with your requirements and would be pleased to discuss any element.

For and on behalf of
<u>PELLS SULLIVAN MEYNINK PTY LTD</u>

<u>T.D. SULLIVAN</u>

Distribution: 1 email copy NSW Department of Commerce
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NSW Department of Commerce

TILLEGRA DAM

STORAGE RIM LANDSLIDE GEOTECHNICAL ASSESSMENT

Report PSM1271.R1 **February 2009**

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1. INTRODUCTION

This report presents the results of geotechnical studies undertaken by this firm as part of overall storage area geotechnical investigations for the Tillegra Dam. The Tillegra Dam is a planned 80m high concrete faced rockfill dam on the Williams River approximately 3.5 kilometres upstream of the confluence with the Chichester River.

The study provides information to facilitate a landslide risk assessment undertaken at the request of NSW Department of Commerce (Commerce). This study is based in large part on the extensive investigations onsite by Mr Young (Commerce), which included test pitting, drilling, seismic refraction surveys, geological mapping and logging.

The involvement of this firm has a narrow focus on landslides and reservoir rim stability.

The information contained in this document has been used as the basis for the risk assessment workshop (Reference 5) and the general reporting by Commerce. Hence while this document is largely a stand-alone report, many of the ultimate study conclusions arising from this work are contained in the documents referred to above.

2. OBJECTIVES OF INVESTIGATIONS

Investigations at the Tillegra Dam by Commerce have been ongoing since October 2007. Arising from these investigations, the following issues were identified as requiring study or assessment:

- Potential for large scale failure of eastern ridge system and loss of storage,
- Identification of pre-existing landslides that could affect the reservoir integrity and/or dam safety,
- Potential for remobilisation of pre-existing landslides and assessment of impact on storage operation and
- Consideration of viable first time slides.

3. EXISTING STUDIES AND AVAILABLE INFORMATION

Geotechnical Investigations by Commerce were still in progress when PSM were engaged to assist. The information provided by Commerce has comprised:

1. NSW Department of Commerce
   Tillegra Dam Storage Rim Stability Interim Engineering Geotechnical Draft Report – Volumes I and II
   Report 08-GN21A-R1, July 2008
2. NSW Department of Commerce
   Risk Management Workshops 22nd/23rd September 2008 –
   Information Package
   Report No. GN31A, September 2008

3. NSW Department of Commerce
Tillegra Dam Design Meeting Agenda
Storage Rim Landslide Risk Assessment Workshop Report, 20th September 2008

4. Bureau of Meteorology
Rainfall Data for Chichester Dam and Dungog

5. URS
Tillegra Dam Site Visit, November 2007
Report No. 43167549.00002, 10th December 2007

6. UNSW
Mineralogy of Clay Sample Geological Analysis Report, 10th October 2008

In addition to this information, the Tillegra Dam Storage Rim Stability and Seepage Potential Engineering Geotechnical Report (Reference 12) was in preparation during the course of the PSM Study. The Engineering Geotechnical Report includes the investigations and testing undertaken in conjunction with the landslide studies by PSM.

The information from these studies have been taken into account for this assessment.

4. <u>BRIEF AND SCOPE OF WORK</u>

The Brief for the studies undertaken by this firm was provided by Commerce (Reference 1). The specific tasks identified included:

- "*Review available geotechnical information provided by Commerce.*
- *Attend and advise on field investigations (geomorphologic mapping, test pitting and trenching, seismic and drilling) associated with possible landslides 2A and 8A.*
- *Review overall geotechnical model of the study area and interpreted "geological domains".*
- *Review geomorphologic mapping of slides 2A and 8A.*
- *Assess the depth and extent of slide colluvium and the extent, if any, of stress relief effects up-slope and adjacent to the existing slide 2A.*
- *Preparation of a geotechnical model of slide 2A in consultation with the project geologist.*
- *Assess whether the mechanism allowing sliding in slide 2A may persist up-slope and allow further significant regression of the slide through the top of the storage perimeter.*
- *Provide a description of the various landslides as part of the preparatory work for the risk workshop: classes, locations, volumes and potential velocity. This will be based on the mapping of existing slides and assessment of slide mechanics and activity. In addition, provide consideration of viable mechanisms of potential large first time slides taking account of the bedding, faulting and jointing in each domain.*

These should be partitioned into existing and first time slides into and out of the reservoir

- *Assess the likely landslide velocity using for example Fell, Glastonbury and Hunter (2007).*
- *For landslides which may travel into the reservoir, assess the likely size of waves that may be generated using methods such as Huber (1997). Subsequently assess the height of waves which could be generated at the dam and determine whether the dam could be overtopped, and if so whether the dam could be breached.*
- *Carry out indicative calculations of stability of existing and viable first time slide mechanisms using estimated strengths on bedding surfaces, joints and shears, allowing for likely groundwater levels, to determine whether an apparently viable mechanism is really a potential slide. Stability calculations shall take account of rainfall induced slides, the influence of the reservoir and earthquakes.*
- *Preparation of summary report on investigations and studies carried out.*
- *Attendance and input to risk workshop.*
- *Amend report as necessary to reflect follow up review comments by Commerce".*

The results of the risk assessment workshop are reported by URS (Reference 5). A site visit was undertaken in the company of Messrs Jamieson and Young (Commerce). The first eight points in the Brief were assessed in conjunction with Commerce, initially onsite during the site visit.

The assessment of potential for overtopping and possible breaching of the dam was carried out in the Risk Workshop and is reported by URS (Reference 5).

5. GEOTECHNICAL SETTING OF TILLEGRA DAM

5.1. Introduction

As a background to the assessment set out in the following sections and to understand the level of the wider stability risk issues at Tillegra Dam, it is important to compare the geotechnical conditions at Tillegra with those commonly associated with Sedimentary Rocks.

5.2. Common Stability Issues With Sedimentary Rocks

Bedded sedimentary rocks, particularly of younger geological age, are very well known as sources of large scale landslides and landslides at all scales.

Key geological and geotechnical features associated with this characteristic include:

- Gentle folding of the strata, resulting in inclined bedding and fairly planar continuous bedding surfaces,

- Low strength bedding parallel shears throughout the profile including in fresh rock,
- Very low shear strength along these shears,
- Generally lower strength rocks, particularly shale and claystone beds,
- Interbedded aquifers and aquicludes,
- Particular sensitivity to rainfall induced instability because of the combination of factors described above,
- Slaking rocks, particularly the claystones, siltstones and shales; and
- Deep weathering profiles.

5.3. Geotechnical Characteristics of Tillegra Dam Rocks Compared to these Typical "Performance" Factors

Compared to these factors the geotechnical characteristics of the rocks at Tillegra Dam as revealed by the geotechnical investigations are:

- The rocks are quite old, Carboniferous age, (400 My),
- The rocks are of very high strength,
- There are minor finer grained beds in the profile (termed a meta-shale by Commerce),
- The weathering profile is very shallow, with soil around 0.5m and fresh rock occurring at less than 5 to 6m depth in the boreholes drilled for this study,
- Rocks are generally massive to very widely jointed,
- Non-slaking rocks, particularly the meta-shale,
- Limited evidence of low strength layers parallel to bedding in fresh rock,
- Simple hydrogeological regime, that is a single water table,
- With one exception (Slide 2A) the rocks are quite folded and block faulted; and
- In general bedding dips are either intermediate (30º to 60º) or flat, with minor exceptions.

5.4. Summary

In summary, many of the geotechnical characteristics normally associated with widespread, large scale and deep seated landsliding or creep movements in Sedimentary Rocks are not present at Tillegra Dam.

6. DISCUSSION OF GEOTECHNICAL CONDITIONS

6.1. Geomorphology

Because of the geotechnical characteristics of the rocks at Tillegra Dam (refer Section 5.0), the geomorphology is quite an accurate indicator of the underlying bedrock geology. Figure 1 shows the topography around the dam site, focussed on the potential landslide areas identified to date and the Eastern Rim of the storage.

Broadly the ridge lines may be divided into four geomorphological units. The subdivision into these units was made initially on the basis of topographic form alone, then calibrated with the results of the geotechnical investigations; trenching, mapping and drilling; and finally "ground truthed" in the field by inspection.

These units are not the classic geomorphological terrain units, but focussed on shedding light on the main objectives of the overall investigations (refer Section 2.0).

In summary the geomorphological units, Figure 1, are:

1. Unit A

   Steep sided ridges formed in bedrock which mainly dips at intermediate angles (30º to 60º) to the west, Figure 2. The eastern sides of these ridges are formed subparallel to cross cutting joints while the western sides of the ridges are in part dip slopes, at least in the very upper parts of the ridges. Elwari Mountain is a slight exception with bedding dipping into the slopes on both sides of the ridge.

2. Unit B

   A unit comprising a series of comparatively steeply incised, curved (arcuate) subparallel creek lines, Figure 3. The creek lines follow bedding, which is folded and dips at intermediate angles to the south southwest.

3. Unit C

   A narrow, flat topped, plateau ridge line with a moderate to steep sided eastern flank and a variable, steep to flat sided western flank. The "plateau ridge" is a dip slope with very flat dips to the west into the storage, Figures 4 and 6.

4. Unit D

   A narrow, flat topped, plateau ridge with a moderate to flat eastern flank and a steeper, but still moderate to flat, western flank. The "plateau ridge" is a dip slope with very flat dips to the east out of the storage, Figures 5 and 6.

6.2. Geotechnical Characteristics

Two geotechnical boreholes were drilled above Slide 2A in Geomorphological Unit C, DDH29 and DDH30, Figure 18. Both boreholes show the same geotechnical conditions and these conditions are also in agreement, with some exceptions noted and explainable, with the geotechnical drilling at the dam site.

Figure 7 shows the core from DDH29 in the interval from 4 to 14m. The upper part of the hole is shown in Figure 8. Figure 9 shows the upper part of the core from DDH30 in the interval from 1.5m to 16m. These figures show:

1. A shallow depth of soil overlying high strength rock at shallow depth (1.5 to 2m), Figures 8 and 9.
2. A very shallow depth of weathering, less than 4m deep in DDH29, Figure 8, and 5m deep in DDH30, Figure 9.
3. The meta-shale beds are thin, poorly bedded and of high strength. These beds are not prone to loss of strength on weathering, see 3m, 6m and 12m depths in Figures 7 and 8; and
4. The sandstones are lithic, massive to very widely jointed.

The test pits and the natural exposures in farm dams around the site, confirm the upper part of this geotechnical model, Figure 10.

6.3. Structural Data

Figures 11 and 12 present all the mapped data for bedding and jointing respectively. This data shows:

- There are three principal bedding orientations, as summarised by the following mean orientations:
  - 40º towards 245º,
  - 10º towards 230º and
  - 30º towards 070º.
- There are four main joint orientations, as summarised by the following mean orientations:
  - 70º towards 345º,
  - 45º towards 060º,
  - 85º towards 170º and
  - 85º towards 130º.

6.4. Hydrogeology

Water level monitoring in boreholes DDH29 and DDH30 gave the following standing water levels in open holes:

- DDH29 – 13.88m (depth from natural surface); and
- DDH30 – 5.84m (depth from natural surface).

The packer permeability testing in these holes gave low permeabilities, in the range from 0 to 1 Lugeon Unit. These results confirm the presence of a “tight” low permeability rock mass and are also in keeping with the rock character shown in the cored holes, Figures 8 and 9.

One unusual feature of the site is the presence of farm dams, filled with water, along the tops of the eastern ridge line, Figure 10, particularly in Geomorphological Units C and D. The presence of these dams in these locations confirms the low permeability for the rocks and the other geotechnical and hydrogeological characteristics described above.

So in summary the Eastern Ridge comprises a “tight” (low permeability) rock mass, with shallow depths of weathering and shallow groundwater levels.

Because the ridge line is a flat topped plateau formed on shallow dip slopes, the ground water and surface water divides to form a catchment that in Domain 2, above Landslide 2A, directs water towards the landslide. These conditions result in the unusual growth of “Paper Bark” trees above Landslide 2A, Figure 24. Paper Bark trees normally thrive in areas of high groundwater level, that is, in valley floors.

7. LANDSLIDE AND RESERVOIR RIM DOMAINS

7.1. Domain Selection Process

In consultation with this firm, Commerce have defined a series of geotechnical domains focussed on the eastern ridge, Storage Rim Stability and Seepage Potential Engineering Geotechnical Report, Volume 1. The basis for the domain selection is:

1. Topographic form.
2. Test pitting and mapping by Commerce.
3. Geotechnical investigations.
4. The consistency of and the dip direction of bedding.
5. Faulting.

The domains are in general accord with the Geomorphological Units. There is a small difference between the northern boundary of Domain 2 as defined in Volume 1 and reported here. In this report, the northern boundary is further north and was determined based on consistency of bedding dip and geomorphology. This small difference does not make any material difference to the outcomes of either report.

7.2. Structural Data by Domain

For the purposes of this report, the structural mapping by Commerce has been collated by Domain and divided into joints and bedding. The data is presented at Appendix B in the form of geological stereoplots. Figure 13 summarises the defect data by geotechnical domain. The differences between the domains are readily evident from this figure.

8. LANDSLIDE ASSESSMENT

8.1. Summary

Overall around the Tillegra Dam area landslides are fairly uncommon. However three types of slides have been identified:

1. Very shallow soil slumps, mainly developed on steeper parts of slopes that cross-cut the bedding.
2. Small slumps in weathered soil/rock in the heads of gullies and
3. A medium to large scale dip slope slide that shows a history of sliding at two different elevations.

The majority of the landslides are very shallow, very small, soil slumps located on the eastern flank of the eastern ridge, mainly located in Domains 4 and 5 but also in localised areas in Domains 1 and 2. Most of these small soil slumps are located above Full Supply Level (FSL) and hence will be unaffected by any filling of the reservoir and will not affect storage rim stability.

Within the storage valley area that was the focus for this study only two landslides could be identified:

- Slide 1A; and
- Slide 2A.

However in addition to these, Slide 8A, is also discussed because it was identified in earlier studies (Reference 11) as a possible landslide. The locations of these three slides are presented in Figure 14.

8.2. Slide 1A

Slide 1A is located in Domain 1, where the bedding dips at a shallow angle out of the reservoir, Figure 6. The slide is small and estimated at 4,000m^3. This slide is a slump of weathered soil/rock in conglomerate.

The slide is located at the head of a steeply incised creek, Figure 15. The slump has only travelled a short distance, with the only significant travel shown by some boulders in the outwash fan, Figure 16.

This area shows evidence of an older smaller slide to the south and adjacent to Slide 1A. The old outwash fan, Figure 16, is assumed to have been formed over a long period from landslide debris and erosion of earlier slides.

It should be noted that below, Slide 2A, there is a similar outwash fan.

8.3. Slide 8A

"Slide 8A" was identified as part of earlier studies as a possible landslide (Reference 11). The area has now been mapped by Commerce and that investigation has shown there is significant rock outcrop across the area, Figure 2.

Kinematic analysis of the structural data from Elwari Mountain shows the slope is formed by cross cutting joints, Figure 17. Kinematic Analysis is a geometric technique for modelling rock mass structure that examines which modes of slope failure are possible in a rock mass.

8.4. <u>Slide 2A</u>

8.4.1. Investigations

A major focus for this study is Slide 2A. The investigations of this slide were also very helpful and assisted in explaining the geotechnical conditions around Tillegra Dam and both the potential for and absence of other landslides elsewhere.

The geotechnical model formulated for this landslide is based on:

- Investigations by Commerce up to the time of the site visit in August 2008,
- Borehole DDH29, which was in progress at the time of the visit,
- Test pits TP138 to TP151,
- Borehole DDH30, which was drilled after the site visit,
- Topographic plan,
- Aerial photographs,
- Geomorphological map by Commerce, which was prepared after the site visit,
- Seismic traverse Line 9 through the landslide area, which was undertaken after the site visit; and
- Engineering index tests and clay mineralogy of clay samples retained from slide plane(s).

The locations of the investigations are shown in plan in Figure 18.

The test pitting, which was undertaken during the site visit was focussed on:

1. Investigating the landslide scarp area.
2. Investigating the toe region.
3. Investigating the up slope and up dip geotechnical conditions.
4. Investigating the potential for deeper lower strength bedding planes above the landslide that could result in either regression of the landslide back up the slope or development of a deeper seated landslide.

Test pitting was not carried out within the landslide itself as it was adjudged from the initial test pitting that the conditions were too deep for the excavator.

8.4.2. Geomorphology

A geomorphological plan of the landslide has been prepared by Commerce, Figure 19. This plan accurately reflects the main geomorphological features at the site. The plan is significantly different to the initial draft plan of the landslide prepared for the Interim Engineering Geotechnical Draft Report (Reference 6).

The significant differences are:

1. The colluvial tongue is not related to the current slide but is probably related to historical landsliding and/or erosion in this region.
2. The Slide 2A is significantly larger.
3. The landslide actually comprises two components, the active landslide itself and a creep zone up slope and up dip, which shows small scale and inferred long term evidence of creep movements, but no active landsliding.
4. There is an active scarp across the northeastern edge of the landslide and extending along the northern side.

8.4.3. Re-Activation

The landslide shows signs of recent re-activation and from the condition of the active scarp this is interpreted to have occurred in 2007. Rainfall records for Chichester Dam and Dungog show a peak rainfall event in June 2007. Figure 20 shows the daily rainfall for 2007 and the historic monthly rainfall since 1997. As shown there have been many large rainfall periods in the past decades, some of which appear of greater magnitude than the 2007 rainfall. The June 2007 rainfall does not appear to be excessive based on the historic records.

Particular features of the recent scarps include their linearity and the fact the scarps are linear in a number of directions.

Figure 21 shows structural mapping data for the landslide shown as a lower hemisphere equal area stereographic projection of the poles to all planes. This mapping is from the test pits. As shown, the alignment of the landslide scarps approximately matches the joint set data. This infers a bedrock controlled sliding event. Figure 22 illustrates this linear control on the recently reactivated landslide scarps.

8.4.4. Surface Features

Figure 23 is a view to the northeast, up Landslide 2A. This figure, in conjunction with the figures referred to below provide the basis for a description of the surface features of the landslide:

1. The landslide comprises three parallel lobes, Figure 23.
2. The active landslide is surrounded in the east northeast by a zone exhibiting small, probably intermittent creep, Figure 24.
3. The landslide is largely covered with tussock grass and the lobes show evidence of old small scale slumping reactivation of the toe, Figures 25 and 26.
4. The overall ground surface slope through the main part of the slide is only about 11½º.

5. At the toe of the main landslide lobe, there is bedrock outcrop in a farm dam. The bedding in this outcrop dips 15º toward the southwest, Figures 23 and 27.

6. The colluvial fan is partly separated from the landslide by a farm dam with bedrock outcrop, Figures 27 and 28.

7. There are a number of very mature trees within the landslide itself that show minimal disturbance even though some are adjacent to the recently active scarp, Figure 29.

8.4.5. Subsurface Description

The subsurface conditions of the slide may be described as follows:

1. Two slide planes, at 2.1 and 5.5m depth, respectively, in Test Pit TP142 immediately above the recently active scarp in the main lobe of the landslide, Figure 30.

2. The lower slide plane is an extremely thin (<1mm thick) pale green coloured irregular clay layer, Figure 31.

3. The upper slide plane is a meta-shale layer that is still rock like, but shows significant aperture on joints, Figure 32.

4. The upper slide plane has been folded by the recent reactivation, Test Pit TP143, Figure 32.

5. Test Pits at the toe of the landslide did not intersect any slide plane; Test Pits TP149, TP150 and TP151.

6. The test pit at the toe of the main landslide lobe and located immediately above the outcrop in the farm dam, Test Pit TP149, showed brecciated rock with no distinct bedding or layering, Figure 33.

8.4.6. Kinematic Analysis

Kinematic analysis of Slide 2A using the mapped bedding and slide planes from within the landslide itself, Figure 34, shows:

1. The average dip of the lower slide plane is 14º towards 216º (southwest).

2. The average dip of the ground surface is 11½ º.

3. Based on this data, the outcrop in the farm dam and the test pitting at the toe, it is clear the landslide failure plane does not daylight.

8.4.7. Creep Zone

Surrounding the landslide in the north, northeast and east is a zone termed a creep zone. This zone has ground surface slopes in the range of about 6º to locally 12º. The ground surface is fairly smooth with some scattered boulders, Figure 35.

Based on ground surface inspection the zone gave no significant indications of any landsliding. However, on the aerial photographs there was a small irregular “signature”. Based on this anomaly, additional test pits were excavated and an additional borehole was drilled, DDH30. This borehole was drilled up dip from the landslide and in a location that would test the potential for landslide regression to affect the reservoir rim.

The test pits and the borehole showed:

1. Evidence of small movement on the upper creep plane at the base of the meta-shale layer, Figure 36.
2. This movement was shallow, at a depth of 1.5 to 2.0m, TP147 and TP148.
3. The mass above the meta-shale layer was often high strength, blocky rock, with clay infill, and locally extremely to highly weathered sandstone, Figure 36.
4. Some test pits refused on rock at shallow depths and did not reach the creep plane.
5. Borehole DDH30 confirmed there was not a deeper slide plane present, Figure 9.
6. Borehole DDH30 also confirmed that regression of Slide 2A could not affect the storage rim, Figure 39.

8.4.8. Seismic Refraction Survey

The result of the seismic refraction survey was provided by Douglas Partners Pty Ltd, Figure 37, incorporating all available subsurface information. The seismic velocities are grouped into three colour coded groupings in Figure 38 for ease of interpretation.

The survey shows:

1. Four layers are present.
2. A new surface layer, less than 0.5m thick, with velocities in the range of about 250 to 700 m/sec.
3. A second layer, ranging from about 4 to 8m deep and locally up to 10m, with seismic velocity in the range of 500 to 1400m/sec. This is interpreted to be the actual landslide.
4. A third layer with seismic velocities in the range from 1300 to 3300m/sec. This is a zone of weathered and de-stressed rock of very good quality, see DDH30, but underlying the landslide.
5. A layer of fresh intact rock with high seismic velocities.
6. Bedrock outcrop present at the small farm dam at the toe of the landslide.

The seismic survey indicates that high strength rock (velocities of up to 5000m/s) is present at relatively shallow depth; and in combination with borehole DDH30 shows deep seated sliding is not present.

8.4.9. Summary Geotechnical Model

Figure 39 is a typical section (Section 1) through the main lobe of the landslide, along the seismic line and through borehole DDH30. Figure 40 is another section (Section 1A) through the main lobe of the landslide, borehole DDH30 and extended through East Ridge. These figures show:

- The main slide,
- The creep zone,
- The slide scarp,
- The inferred basal failure plane; and
- Full supply level.

This information shows there is no deep seated slide plane in Slide 2A. Based on the presence of the high velocity rock on the seismic survey, the absence of any deeper slide surface in borehole DDH30 and the topography of the East Ridge, it is clear that there is no potential for major deep seated regression of Landslide 2A that could impact on the reservoir rim.

The summary geotechnical model for Landslide 2A is:

1. This is a very old slide or series of slides.
2. The colluvium lobe at the toe is inferred to indicate a long history of landsliding and/or erosion in this location.
3. The slide appears to be very slow moving.
4. The average dip of the slide plane is 14º towards 216º, which is steeper than the ground surface slope of 11½º.
5. The landslide is non-daylighting with a thick “shove zone” of brecciated rock at the toe.
6. The toe of the slide is also partly constrained because the dip direction of sliding is towards 216º, that is south west, towards Domain 4. The overall ground surface slopes on the western side of the eastern ridge are towards about 260º. The two eastern lobes of landslide material abut intact material in the creek forming the boundary between Domains 2 and 4.
7. The landslide shows a history of sliding at two different elevations.
8. The landslide is relatively shallow, about 5 to 8m deep.
9. The sliding planes are very thin, and very irregular. They appear to have formed by weathering and/or alteration of the upper boundary of the lithic sandstones immediately below meta-shale layers. Figure 8 shows a core loss and weathering reversal below a meta-shale layer at 3.3m depth in DDH29. Similar features are also evident in DDH30, Figure 9. This is not a “classic” sedimentary landslide plane.
10. Surrounding the landslide in the north, northeast and east is a creep zone. This zone exhibits shallow, 1.5 to 2.0m, very small scale and inferred periodic creep movement, probably under extreme rainfall conditions.
11. The main landslide showed a small scale re-mobilisation around the scarp area, probably due to the 200mm three day rainfall event in June 2007.
12. The main landslide is about 370,000m^3.
13. The upper creep zone is about 200,000m^3.

In summary, Landslide 2A is located in a particular area with a unique set of geological, geotechnical, geomorphological and environmental circumstances. These same circumstances are not found elsewhere within the study area.

9. ENGINEERING ANALYSIS SLIDE 2A

9.1. Strength of Slide Planes

Back analysis for plane sliding of an infinite slope to represent the creep zone and assuming full saturation gave an angle of friction (ϕ) around 20º. This seems slightly low based on experience and the character of the planes as observed. Notwithstanding this, because no actual strength testing is available a conservative approach was used in the analyses and strengths of 16º, 18º and 20º were assumed.

A limited sample of the lower slide plane was collected. Engineering index testing and X-ray Diffraction Analysis (XRD) to determine the clay mineralogy of the lower slide plane material was used to estimate the strength of this plane based on a number of published empirical correlations. The results are included in Appendix C. These correlations gave a very wide range of values, from about 10º to 28º, with a mean of around 15º to 16º. On the basis of judgment, experience and the observed character of those planes it is estimated the shear strength of the lower plane is also greater than 20º. However, because no actual strength testing is available, a conservative approach has been used and for the purposes of the assessment, the plane has been assigned shear strengths of 16º, 18º and 20º.

The strength of the toe breakout plane, which is through the brecciated rock mass has been assigned a strength of:

- Cohesion (c) = 5 kPa and
- Angle of Friction (ϕ) = 28º.

9.2. Stability Analysis Results

The analysis results are presented in Appendix D, summarised in Table 9.1 and Figure 41. The stability was analysed for Section 1, Figure 39, and comprised the Slide 2A, the creep zone and both Slide 2A and creep zone together.

The results show that with a fully saturated slope, the Factor of Safety for Slide 2A and the creep zone falls below 1.0, with or without the presence of the dam, but that is for a conservative angle of friction (16 º). Where a more realistic angle of friction (20º) is used the Factor of Safety approaches 1.0, reflecting in reality the probable small re-activations of the slide during periods of heavy rain. The use of 20º is consistent with back analysis, however in the absence of actual strength testing, 16º is used as a design value. The shear strength of the slide plane in the creep zone is estimated to be considerably greater than in Slide 2A.

All these analysis results are in accord with the site observations on the character and nature of Slide 2A and the creep zone.

TABLE 9.1
SUMMARY OF STABILITY ANALYSIS RESULTS

DESIGN CASE	DENSITY OF SLIDE MATERIALS (kN/m^3)	BASAL PLANE	TOE BREAKOUT PLANE		FACTORS OF SAFETY		
		ϕ	c'	ϕ	UPPER ZONE	SLIDE 2A	BOTH
Dry	18	16	5	28	1.18	1.79	1.74
No FSL, Saturated	18	16	5	28		0.80	0.86
	18	18	5	28		0.89	
	18	20	5	28		0.98	
Full Supply Level & Dry	18	16	5	28	1.18	1.49	1.44
FSL & Saturated	18	16	5	28	0.54	0.95	1.00
	18	18	5	28		1.20	
	18	20	5	28		1.29	
FSL & Saturated Minus 2m	18	16	5	28	0.92	1.41	1.07
FSL & Saturated minus 4m	18	16	5	28	1.15	1.26	1.23

9.3. Summary Engineering Analysis

The stability analyses show that with a fully saturated slope, the Factor of Safety for Slide 2A approaches or falls below 1.0. This indicates that Slide 2A is likely to be re-activated when the slide is saturated. With the dam present and FSL with a fully saturated slope, the Factor of Safety approaches 1.0, indicating likely re-activation.

10. ASSESSMENT OF FIRST TIME SLIDES

10.1. General Assessment

This assessment has shown that the potential for medium to large scale sliding in the Tillegra Dam area firstly requires areas with consistent flat to moderate dipping bedding. Figure 42 summarises the average dip and dip direction of bedding in each Domain. The second element required is suitable landform, with the bedding dipping either flatter than or approximately subparallel to the ground surface. For this to impact on the stability of the storage rim these two factors have to combine over a sufficiently large area to allow landsliding on a scale such that storage rim impacts are feasible. The potential for first time sliding has been assessed based on the geological mapping by

Commerce, the structural data, kinematics and geomorphology. The results are presented graphically in Appendix E for each domain.

Some faults have been mapped and inferred by Commerce. There is no evidence to indicate the faulting would affect storage rim stability.

The assessment of first time slide potential for each domain is summarised as follows:

Domain 1

- No visible signs of any creep or large scale landslide activity.
- Ridge line consists of a narrow flat topped ridge with extensive subparallel ridges running off normal to the main ridge. These ridges would act to buttress the main ridge.
- There is a 47° difference between the ground surface slope and the dip direction of bedding. This difference is normally sufficient as to preclude sliding as feasible.
- Bedding has an average dip to the east southeast of about 7°. Given the shear strengths calculated for Slide 2A, this dip would normally be taken to be too flat to allow sliding to occur.
- Although there is no evidence of sliding in the past and empirically the structural data says that sliding should not occur in the future, this is the one domain where bedding dips out of the reservoir and hence some analysis has been carried out, as detailed below.

Domain 2

- The geotechnical investigations have shown that other first time slides in this domain and/or deep seated regression of the existing slide is not feasible.

Domain 3

- Bedding dips into the slope and there are no feasible mechanisms for first time slides.

Domain 4

- There is no evidence of creep movements or landsliding in this domain.
- The bedding dips to the south southwest and is quite folded.
- First time sliding could only occur as a wedge with bedding and a continuous joint intersecting to form a long narrow wedge. The joints would also have to be curved otherwise the potential wedge size would be limited by geometry.
- There is no evidence of joints at the site that are either continuous for great distances or curved.
- No feasible mechanisms could be identified for first time slides in this domain.

Domain 5

- A mechanism has been identified for possible first time sliding in this Domain and this is assessed below.

Domain 8

- Bedding dips into the ridge on both sides of the Elwari Mountain and no feasible mechanism could be identified for first time slides in this domain.

Saddles A and B

- Saddles A and B are both areas where a small section of a relatively narrow ridge forms part of the reservoir rim.
- In both saddles the length of saddle is very small compared to the adjacent ridges, which are quite broad.
- In both saddle areas the bedding is generally steeply dipping, greater than 45º, and/or is quite folded. The bedding dip changes occur over distances that are much less than the length of saddle.
- Based on the structural data and geomorphology there is no feasible mechanism for first time sliding at either saddle.

Based on this assessment the only areas with potential for first time sliding that could possibly impact on the storage rim or reservoir are Domains 1 and 5.

In all domains within the study area, based on geology, geomorphology and geotechnical conditions, no credible mechanisms were identified where first time slides could affect the storage rim integrity and allow release of the storage.

10.2. Stability Analysis, Domain 1

In Domain 1 there is an area where the flood plain on the eastern side of the Chichester Range combines with the topography on the western side of the Chichester Range to form a very small length of narrow ridge. As shown in Figure 42, this is the location of Section AA' in the Storage Rim Stability and Seepage Potential Engineering Geotechnical Report Volume 1.

It is also readily evident in Figure 42, that the topography is such that even though Section AA' indicates a potentially feasible mechanism for sliding in two dimensions, any such sliding is severely constrained by the topography which buttresses the slope.

Nevertheless, a stability analysis of Section AA' through Chichester Range was undertaken, assuming a tension crack at the ridge crest and plane sliding failure along bedding. The results are presented in Appendix F.

The analysis assumed:

- A full depth tension crack;
- Failure along a bedding plane, with shear strength as per Section 9.1:
  - Cohesion (c) = 0 kPa and
  - Angle of Friction (ϕ) = 16º;
- Mobilisation of sliding mass through failure of buttressing intact rock mass, with shear strength as per Section 10.3:
  - Cohesion (c) = 3.5 MPa and
  - Angle of Friction (ϕ) = 62º;
- Full water pressure in the tension crack below a depth of 10m; and
- The piezometric surface 10m beneath the surface level.

The analysis methodology was approximate and entailed determining an equivalent rock mass strength based on the proposition of the 3-d failure plane comprising bedding and rock mass respectively. The factored parameters were then applied to a 2-d analysis as follows:

- Cohesion (c_{eq}) = 420 kPa and
- Angle of Friction (ϕ_{eq}) = 25.5º.

The Factor of Safety for this approximate analysis was 3.1.

Based on the geotechnical conditions, the geomorphology and the results of the approximate stability analyses, first time sliding leading to partial and/or complete failure of the reservoir rim in Domain 1 is not a credible mechanism.

10.3. <u>Stability Analysis, Domain 5</u>

In this domain, the upper slopes above FSL are dip slopes that dip to the west, into the reservoir. There is potential for a first time slide to occur in this domain above FSL. However as set out below, it was very difficult to conceptualise a viable breakout mechanism that would allow sliding to occur.

First time Slide 5A would involve sliding on a bedding plane with breakout across bedding and through the rock mass at the toe. The potential slide details are:

- Top of ridge 225m RL,

- Location of potential toe breakout, the change in slope at 180 to 185m RL,
- Bedding dip 32º to 36º,
- Outcrops show massive sandstone in the toe region with continuous exposures of 20m by 15m showing no defects;
- There is no evidence of meta-shale layers,
- There is no evidence of any creep, incipient instability or previous sliding; and
- The overall potential mass is assumed to be 290,000m^3, assuming a bedding plane slide 20m deep.

Estimates of the mass strength for failure across bedding were carried out using Roclab (Appendix I) and gave:

- Cohesion 3.5 MPa and
- Angle of Friction 62º.

Using these parameters and an angle of friction of 20º along bedding gives very high Factors of Safety. Even if the cohesion is reduced to zero the Factor of Safety is still 1.5. These analyses assumed a dry slope but nonetheless even with some water pressures the Factors of Safety would still be high.

Pseudo-static analyses of earthquake loads under a Maximum Design Earthquake (MDE) and Maximum Credible Earthquake (MCE) were also carried out. Peak ground accelerations (pga) of 0.24g for MDE (1 in 10,000 event) and 0.5g for MCE (1 in 100,000 event) were given. Appendix I presents the details and results of our analyses.

Pseudo-static analysis involves the conversion of a dynamic acceleration (*pga*) into a static force, usually by a factor of 0.5. The design horizontal seismic inertia force, F_H, can be assessed using (Eurocode 8 Part 5):

$$F_H = 0.5\ \alpha\ S_T\ S\ W$$

where α = ratio of *pga* on type A ground to the acceleration of gravity, *g*
S_T = topographic amplification factor
S = soil parameter
W = weight of sliding mass

S_T of 1.4 and S of 1.0 were assigned for Slide 5A based on the draft Eurocode 8 Part 5.

The horizontal seismic inertia forces of 0.168W (MDE) and 0.35W (MCE) gave Factors of Safety of 4.8 and 3.9 respectively.

The results are in accord with the site observations and the long term performance of this slope.

11. IMPULSE WAVE EFFECTS FROM LANDSLIDES

11.1. Introduction

Impulse waves occur when there is a rapid failure into the reservoir that is large enough to cause a wave. To pose a problem at the dam, such a wave must also firstly be in a location such that it can travel directly to the dam without major attenuation, secondly be large enough to overtop the dam and thirdly the overtopping needs to be of sufficient duration to erode the dam crest.

Based on the analysis and assessment above there is potential for impulse wave effects to be generated by landslides from the following sources:

- Slide 1A,
- Slide 2A,
- Slide 2A upper creep zone; and
- First time Slide 5A.

The detailed analysis results for these four potential slides are included in Appendix G and summarised in the following sections. The analyses have been carried out using the methodologies of Huber; Huber and Hager; and Glastonbury and Fell (References 2 to 4).

The important point to note is that for Slide 1A, there is no direct line of sight to the dam. For Slide 2A (including upper creep zone) and 5A, the lateral angles for such impulse wave are large, therefore the 3-d calculations of wave height would be more appropriate.

The results of the impulse wave generation assessments are summarised in Table 11.1.

TABLE 11.1
SUMMARY OF IMPULSE WAVE GENERATION

SLIDE	POINT	DISTANCE (m)	WAVE PROPAGATION VELOCITY (m/s)	ESTIMATED WAVE HEIGHT		ESTIMATED RUN UP HEIGHT 2D (m)
				2D (m)	3D (m)	
1A	Elwari Mountain T	1550	22.1	0.9	0.01	1.0
	Elwari Mountain S	2500	14.0	0.6	0.01	0.7
2A	Dam A	1700	22.2	5.2	0.04	7.6
	Elwari Mountain B	1900	22.2	5.0	0.04	7.4
	Elwari Mountain C	1250	17.2	4.9	0.07	7.4
2A Creep zone	Dam A	1700	22.2	2.2	0.02	3.0
	Elwari Mountain X	1850	22.2	2.2	0.02	3.0
	Elwari Mountain Y	1200	22.2	2.4	0.02	2.8
5A	Elwari Mountain E	800	17.2	5.9	0.1	6.6
	Dam D1	700	22.2	7.0	0.05	10.6

11.2. Discussion

It is known from experience that the reservoir characteristics influence the impulse wave generation and propagation and that the highest waves occur in the direction of momentum of the slide (Reference 2). Tillegra Reservoir in the study area is slightly elongated, but more importantly the main body of the dam itself is oriented at large angles to the direction of sliding for all the actual and potential landslides, thus the angle between the direction of sliding and the point where the wave would impact the dam is large.

The analysis method used here also assumes that the dam is visible from the landslide impact site. If this is not the case then reflection and diffraction of the waves will occur, and the heights of indirect waves are small compared to the direct waves (Reference 3).

The ground surface slope angle at the point of impact for the landslide into the reservoir is also important, where the ground surface slope angles are less than 25º, friction inhibits the sliding mass (Reference 3). Reference 3 quotes a range of valid ground surface slope angles from 28º to 60º. For Tillegra Reservoir, the maximum ground surface slope at the potential point of impact for all slides analysed is 18º, hence this would infer this empirical analysis is conservative.

Thus for a number of reasons the 2-d wave height estimate is considered to be a very conservative upper bound. This does not include the additional conservative assumptions built into the stability analyses themselves.

The 3-d wave height prediction model probably represents a lower bound of the actual wave heights (Reference 13) but one which in this case is considered a more realistic estimate.

11.3. Slide 1A

This is a very small slide located above FSL. The estimated volume is of the order of 4,000m^3. This slide cannot actually "see" the dam and hence in order to assess possible impacts, the assessment was for a wave impacting on two locations on Elwari Mountain. Any waves that impacted the dam would be substantially smaller.

11.4. Slide 2A and Creep Zone

Slide 2A and the Creep Zone are described in Sections 8.0 and 9.0. Post failure slide velocities have been calculated for both the Upper creep zone and Slide 2A itself and these calculations are presented in Appendix H. The result shows a 85% probability that Slide 2A will be extremely slow to slow and a 45% probability that the Upper creep zone will be a very rapid slide. Despite this assessment, for the purposes of wave generation, rapid sliding has been assumed.

The estimations for Slide 2A and the Upper creep zone are for a point on the left abutment ridge near the dam, Point A in Table 11.1. The other estimates are for points on Elwari Mountain. Any waves that impacted the dam would be substantially smaller than those in Table 11.1.

11.5. Slide 5A

For the purposes of estimating the impulse waves, rapid failure of the whole mass has been assumed.

11.6. Conclusions

Because of the factors described in the sections above it is very difficult to accurately predict the actual wave heights and wave run-up heights at the dam, due to the known and inferred potential first time slide, Slides 1A, 2A and 5A.

The key technical factors limiting impulse wave effects at Tillegra Dam are:

- The maximum wave height is in the direction of sliding;
- The dam is located lateral to the direction of sliding, thus the propagation angle is very wide;
- The ground surface slopes at the point of impact of the landslides are less than the valid range allowed in the analyses.

Notwithstanding these limitations conservative assumptions have also been made for each landslide, including:

- The total mass occurs as a single slide,
- The landslides are very rigid; and
- The shear strengths of the sliding planes have been assumed to be low.

The analyses include estimates of both 2-d and 3-d wave heights. Because of the factors described above the 2-d wave heights are considered an upper bound. It is assessed the actual wave heights that could impact the dam from these slides would be substantially less than the 2-d estimates and much closer to the 3-d wave heights.

12. CONCLUSIONS

The Tillegra Dam Peer Review Panel identified four main issues to be studied and assessed. These issues and the conclusions derived from this study are set out below:

1. *"Potential for large scale failure of eastern ridge system and loss of storage."*

   No credible mechanism has been identified that could result in large scale failure of the eastern ridge system and/or loss of storage.

2. *"Identification of pre-existing landslides that could affect the reservoir integrity and/or dam safety."*

   Landslides within the Tillegra Dam Storage Area are rare and only two significant landslides could be identified; Landslide 1A and Landslide 2A.

   This is in keeping with the geotechnical characteristics of the rocks at Tillegra Dam, which are significantly different to those normally associated with large scale and deep seated landsliding or creep movements in Sedimentary Rock Terrains.

   Landslide 1A is a small scale, only 4,000m^3, slump in weathered soil rock. Assessment of this slide showed that it has no effect on reservoir rim integrity and that it has no dam safety implications.

   Landslide 2A comprises the landslide itself and a smaller area up dip and up slope from the landslide that shows signs of long term creep movements. The summary geotechnical model for this landslide is:

   - This is a very old slide or series of slides,
   - The slide appears to be very slow moving,
   - The average dip of the slide planes is 14º towards 216º, which is steeper than the ground surface slope of 11½º,
   - The landslide is non-daylighting with a thick "shove zone" of brecciated rock at the toe,

- The toe of the slide is also partly constrained because the dip direction of sliding is south west, and the two eastern lobes of landslide material abut intact material,
- The landslide shows a history of sliding at two different elevations,
- The landslide is relatively shallow, about 5 to 8m deep,
- Surrounding the landslide in the north, northeast and east is a creep zone; this zone exhibits shallow, 1.5 to 2.0m, very small scale and inferred periodic creep movement, probably under extreme rainfall conditions,
- The main landslide showed a small scale re-mobilisation around the scarp area, probably due to the 200mm three day rainfall event in June 2007,
- The main landslide is about 370,000m^3; and
- The upper creep zone is about 200,000m^3.

Assessment of this slide showed that it poses no threat to reservoir rim integrity and that it has no dam safety implications.

3. *“Potential for remobilisation of pre-existing landslides and assessment of impact on storage operatio*n.”

   Both Landslides 1A and 2A could remobilise. However Slide 1A is above FSL and any remobilisation will be unaffected by the reservoir. The stability of Landslide 2A could be impacted by the reservoir and the Factor of Safety does reduce for FSL. However, the analyses show that both FSL and full saturation of the whole slope is required for the Factor of Safety to fall below 1.0, implying remobilisation. The analyses also show that even without the dam being present (no FSL) where the slope is fully saturated the Factor of Safety approaches 1.0, implying remobilisation. This is evident in creep movements during periods of heavy rainfall.

   Notwithstanding these results, the likelihood of both slides remobilising and being rapid has been allowed for in the assessment.

   Remobilisation of Slide 1A will have no impact on storage operation. Under these assumed adverse stability conditions Slide 2A would have minimal impact on dam operations.

4. *“Consideration of viable first time slides.”*

   Only one location with the potential for first time sliding was identified, Slide 5A. It is located approximately 500m upstream of the dam and entails a section of a dip slope that is well above the FSL. The area is currently stable with no evidence of any creep or landslide movement. Notwithstanding this a very conservative assumption has been made that a slide of about 290,000m^3 (20m deep) could occur and fails rapidly into the reservoir. The impacts on the dam of this failure have been assessed with a two dimensional wave run-up height of 10.4m.

   However, this is a very conservative upper bound estimate. Allowing for 3-d effects the actual wave heights generated at the dam will be substantially less. The estimated impact on the dam even under this series of conservative assumptions is minimal.

For and on behalf of
PELLS SULLIVAN MEYNINK PTY LTD

T.D. SULLIVAN

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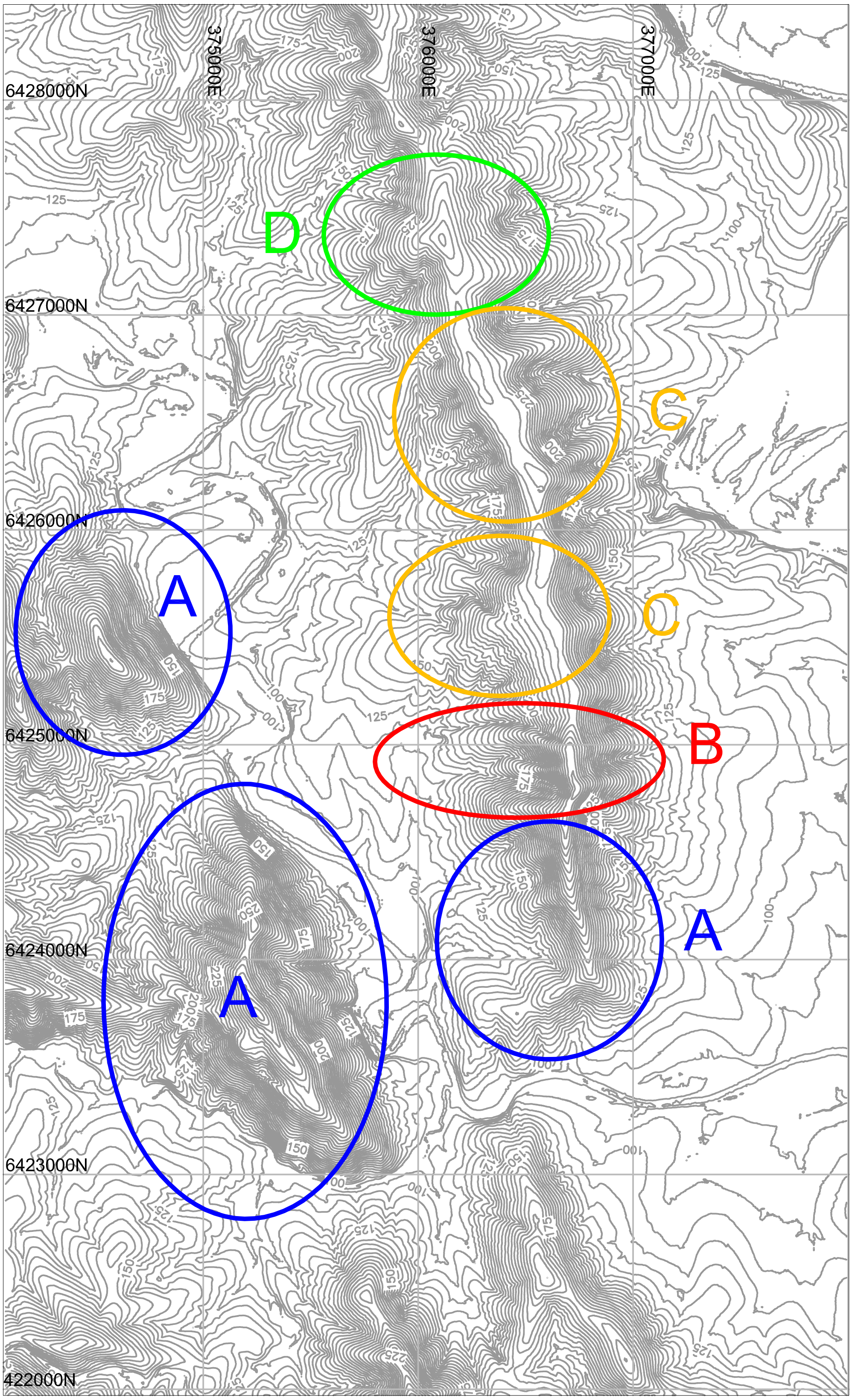

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Tillegra Dam
Storage Rim Landslide Assessment

GEOMORPHOLOGICAL UNITS

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PSM 1271.R1	Figure 1

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VIEW OF EASTERN FLANK OF
GEOMORPHOLOGICAL UNIT A

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PSM 1271.R1	Figure 2

Note: The curved incised creek lines in the middle distance of Domain 4. Foreground is Landslide 2A

PSM

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NSW Department of Commerce Tillegra Dam Storage Rim Landslide Assessment WESTERN FLANK OF GEOMORPHOLOGICAL UNIT B (View to South East)	
PSM 1271.R1	Figure 3

Note: Bedding dip to the West
into the storage.

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GEOMORPHOLOGICAL UNIT C
(View to South)

PSM 1271.R1	Figure 4

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GEOMORPHOLOGICAL UNIT D
(View to North)

PSM

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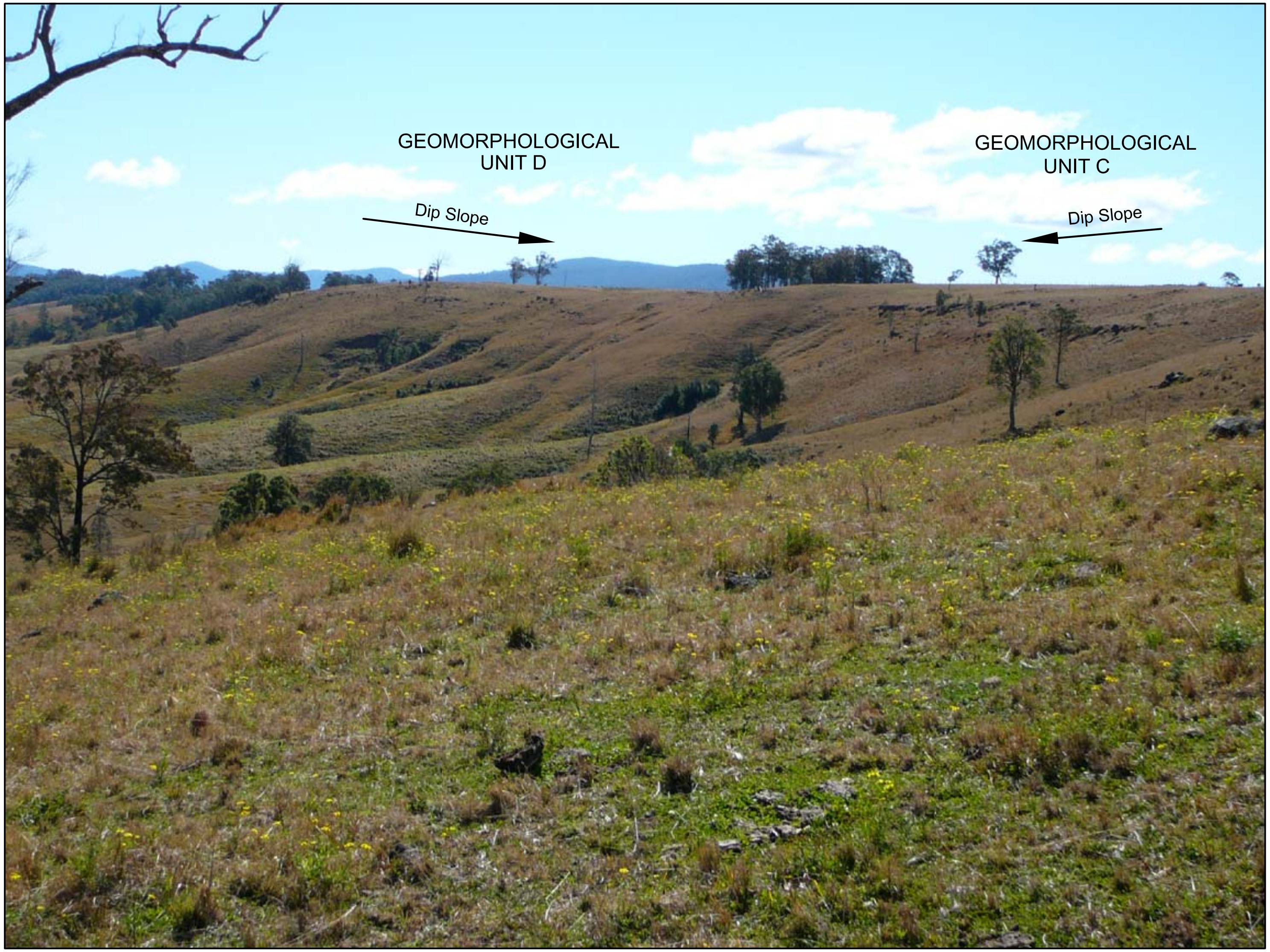
PSM 1271.R1	Figure 5



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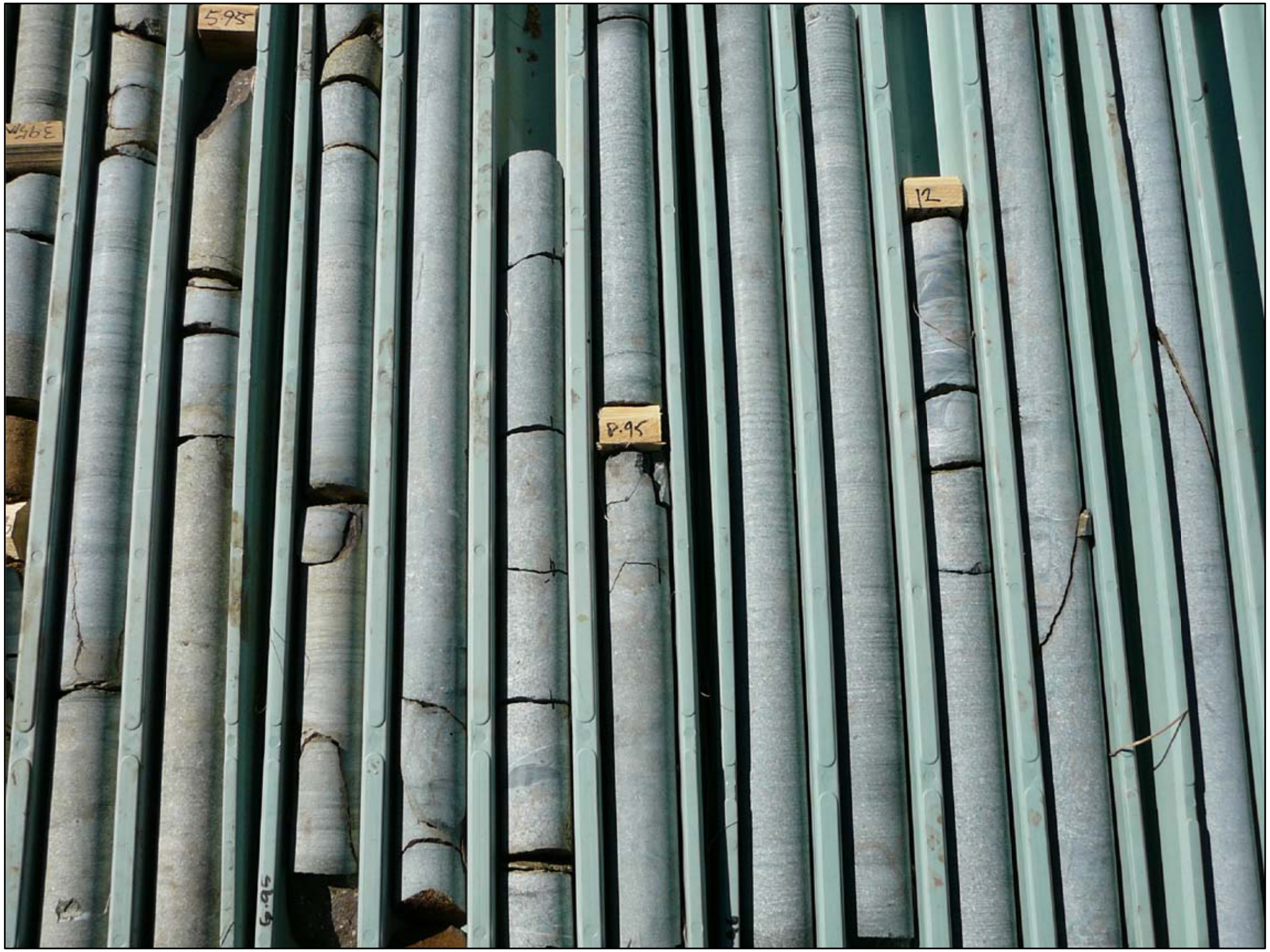
COMPARISON OF
GEOMORPHOLOGICAL UNITS C & D
(View to North)

PSM 1271.R1

Figure 6



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DDH29
FROM 4 to 14m DEPTH

PSM

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PSM 1271.R1	Figure 7

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DDH29
FROM 2 to 9m DEPTH

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PSM 1271.R1 | Figure 8

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DDH30
FROM 1.5 to 16m DEPTH

PSM

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PSM 1271.R1	Figure 9

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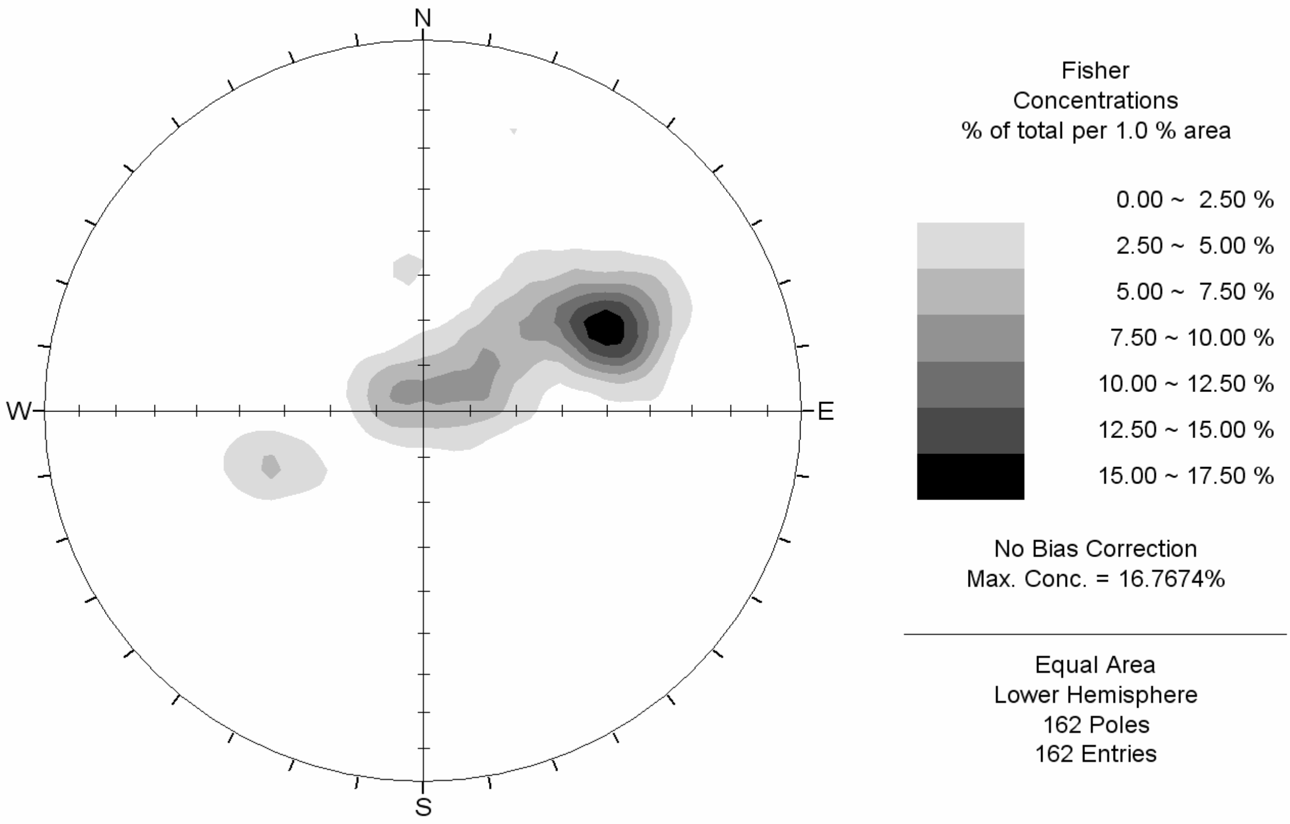
SMALL DAM ON THE WAY TO SLIDE 1A
BEDDING DIPS EAST TO SOUTH EAST
AT 5 to 7°

PSM

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1 | Figure 10



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DOMAINS 1 to 9
ALL BEDDING

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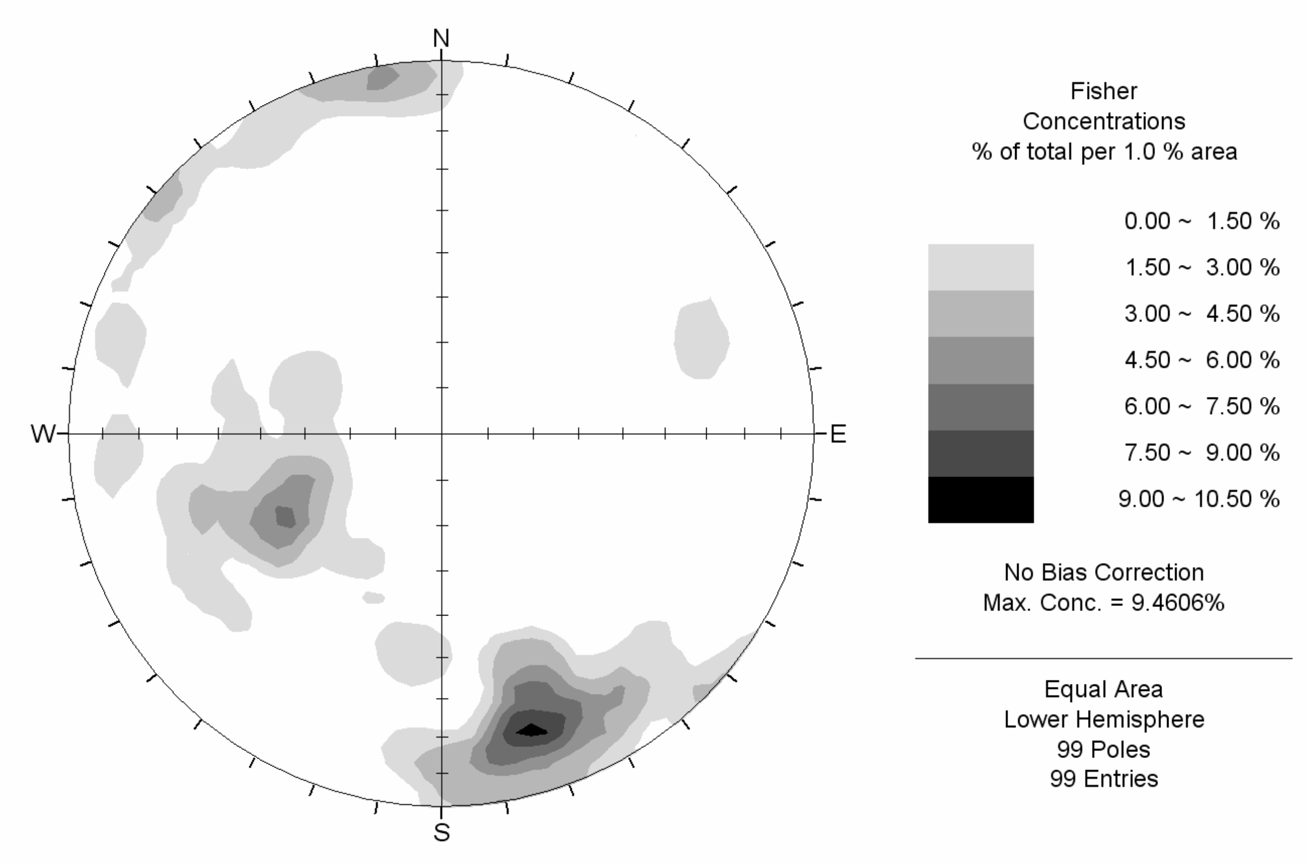
PSM 1271.R1

Figure 11



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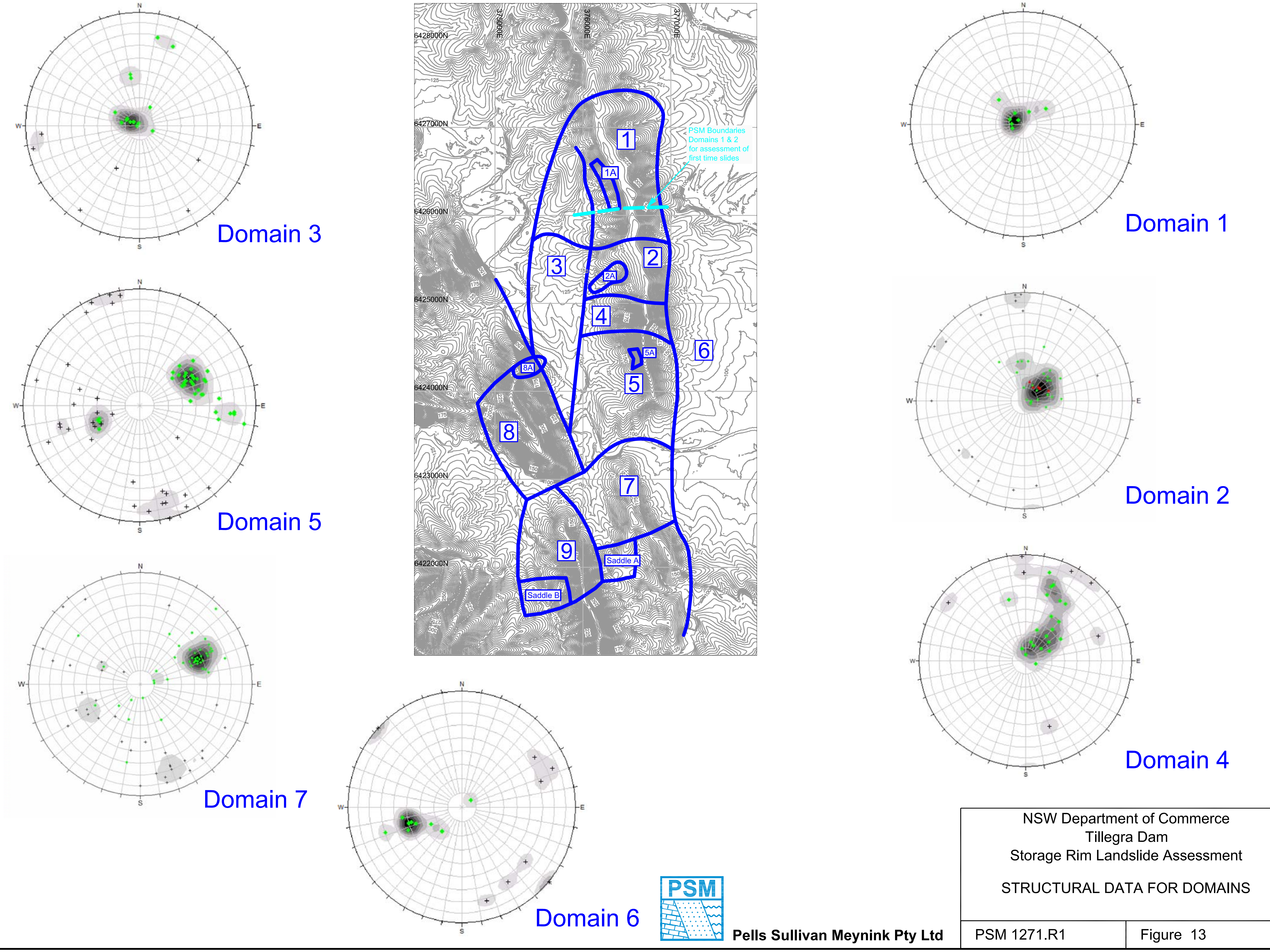
DOMAINS 1 to 9
ALL JOINTS

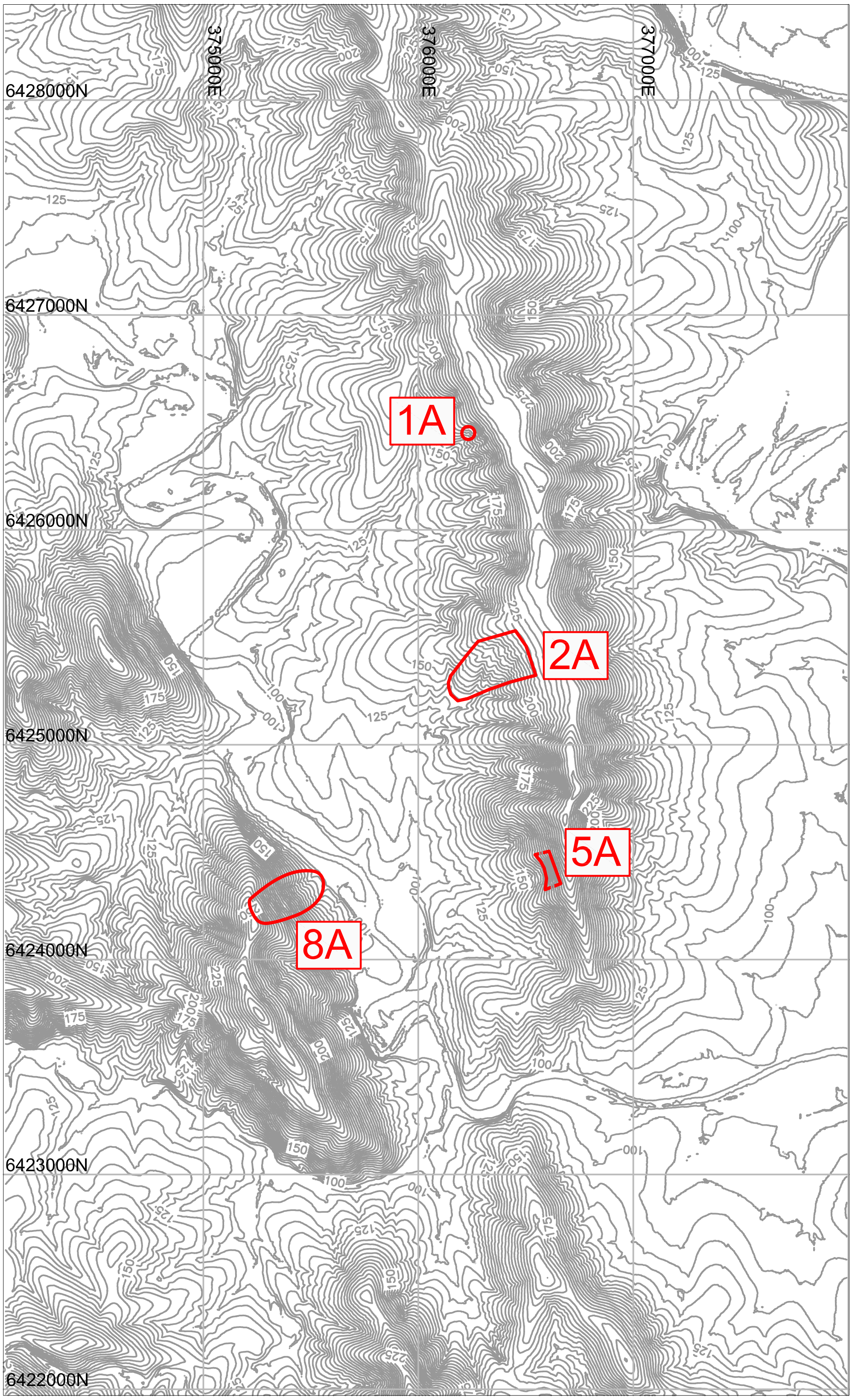
Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 12



PSM
Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce Tillegra Dam Storage Rim Landslide Assessment STRUCTURAL DATA FOR DOMAINS	
PSM 1271.R1	Figure 13



NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

LOCATION OF LANDSLIDES
UNDER CONSIDERATION

PSM

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1

Figure 14

Pells Sullivan Meynink Pty Ltd

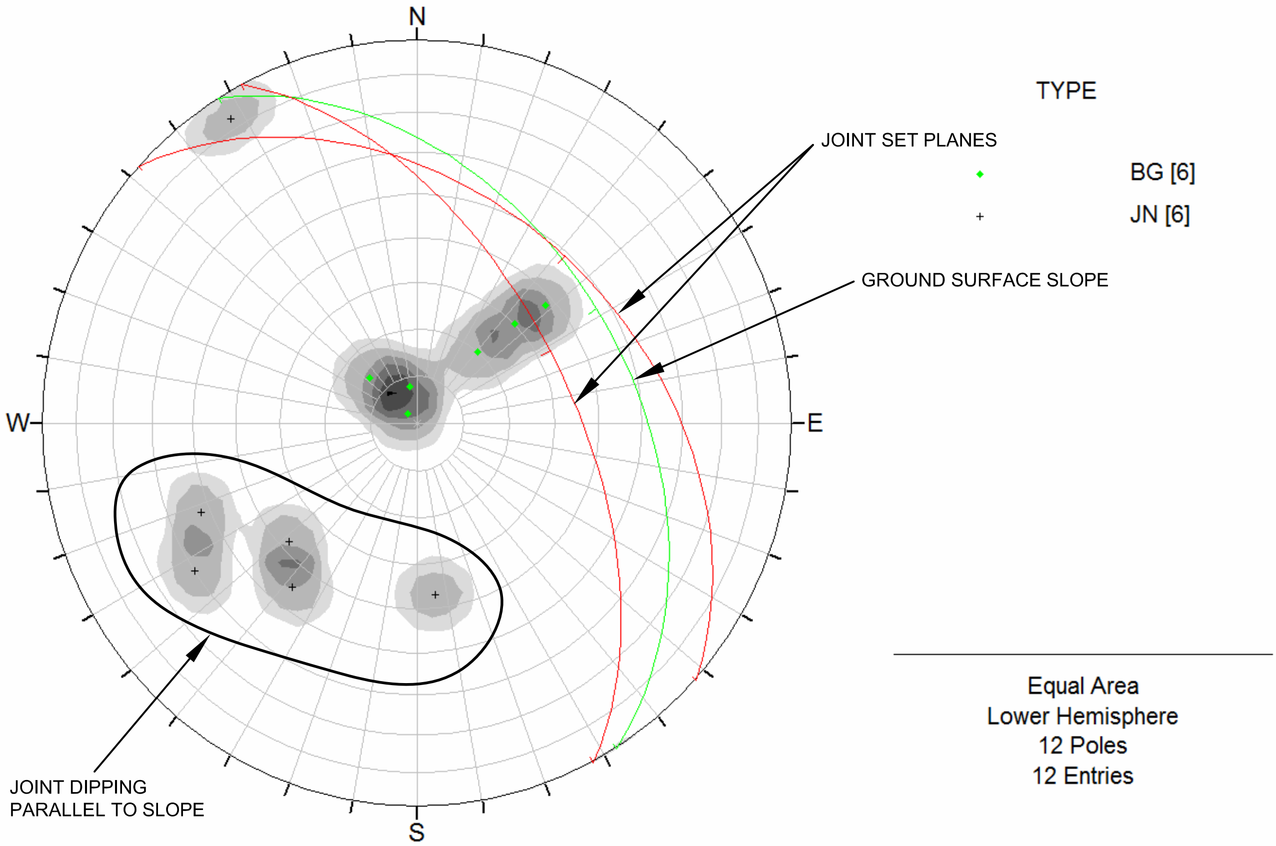
NSW Department of Commerce Tillegra Dam Storage Rim Landslide Assessment DOMAIN 1 & SLIDE 1A	
PSM 1271.R1	Figure 15

Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

VIEW LOOKING DOWNSTREAM
SLIDE 1A

PSM 1271.R1	Figure 16

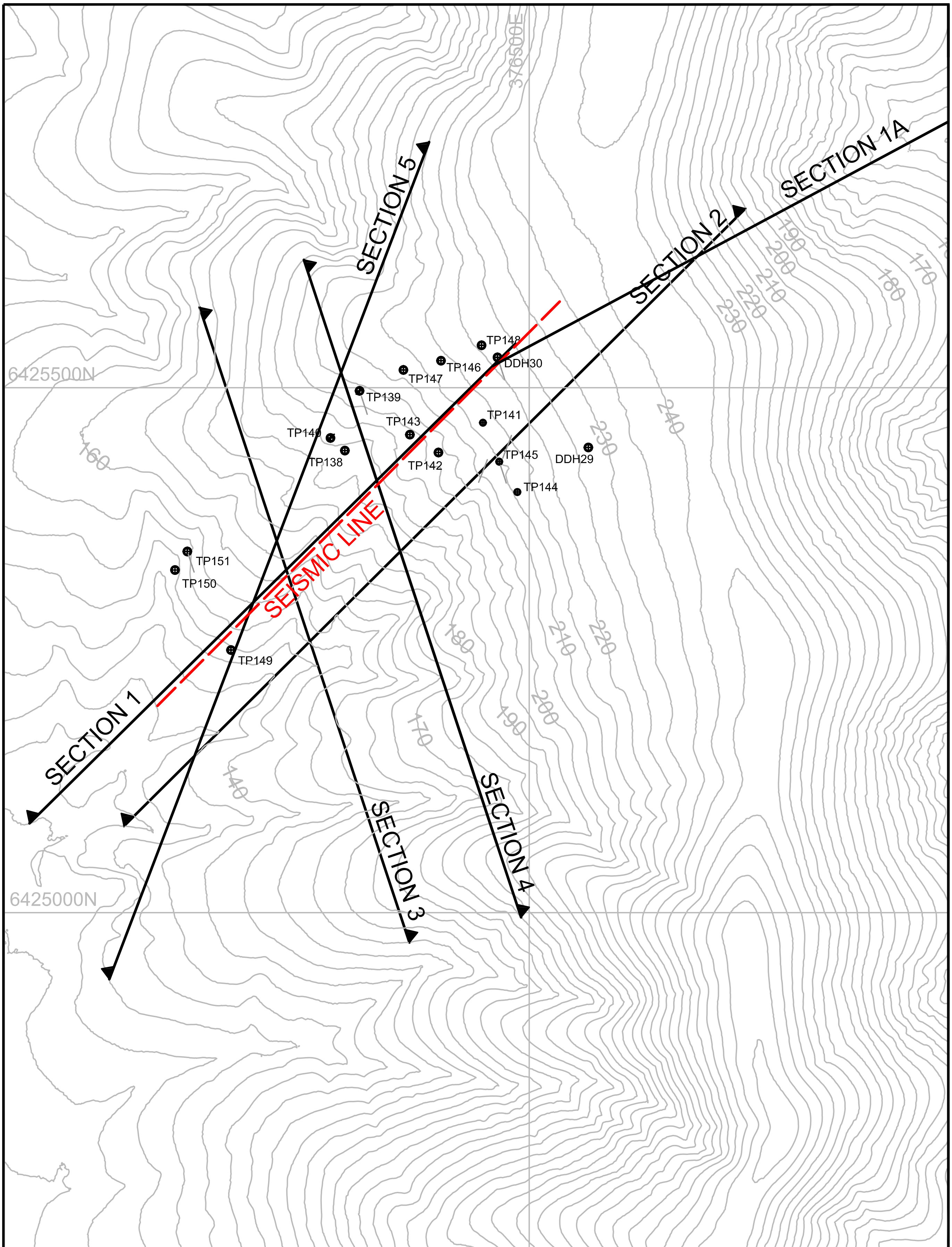


NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

KINEMATIC ANALYSIS
POSSIBLE SLIDE 8A

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 17

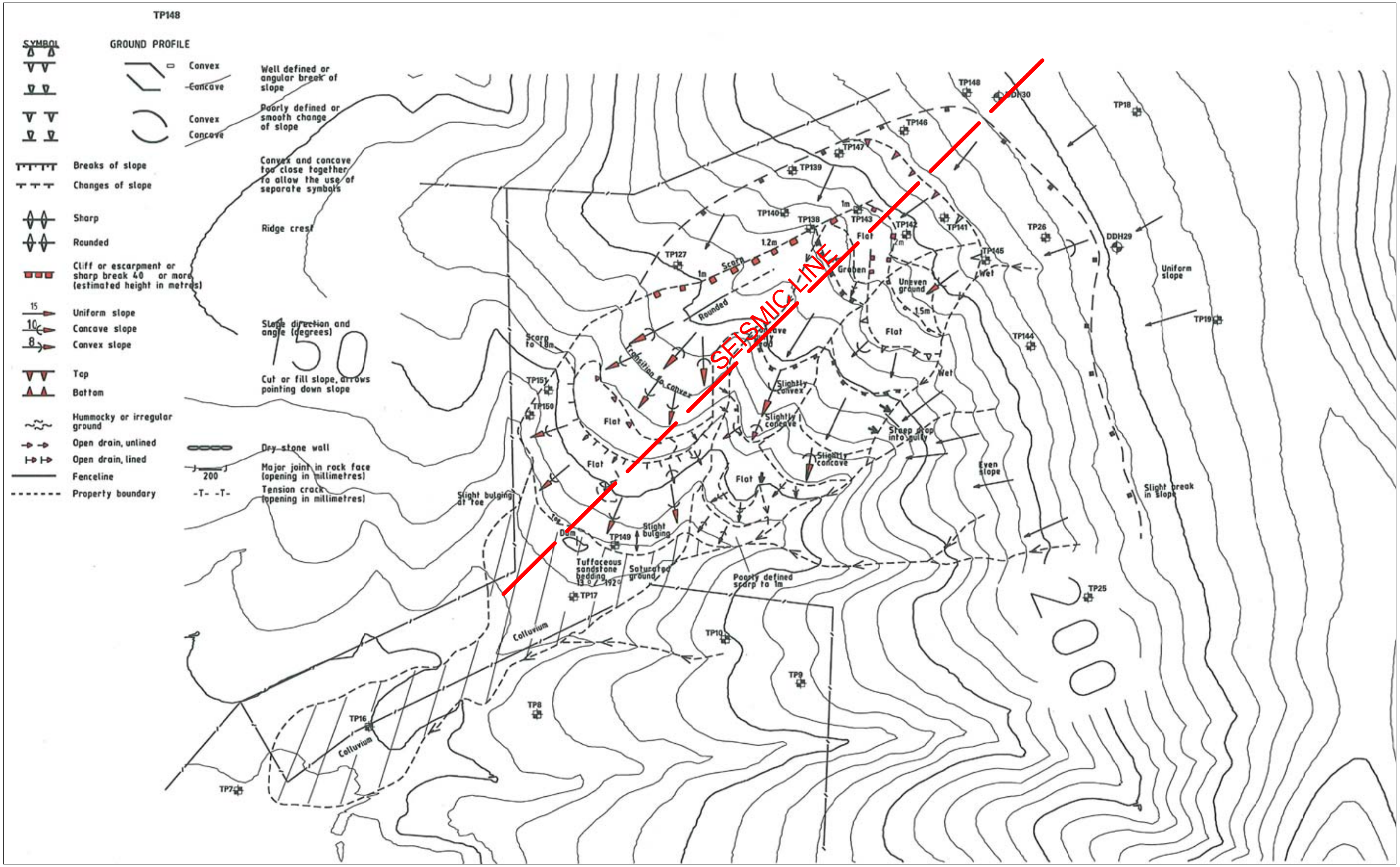


Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

SLIDE 2A INVESTIGATIONS

PSM 1271.R1	Figure 18

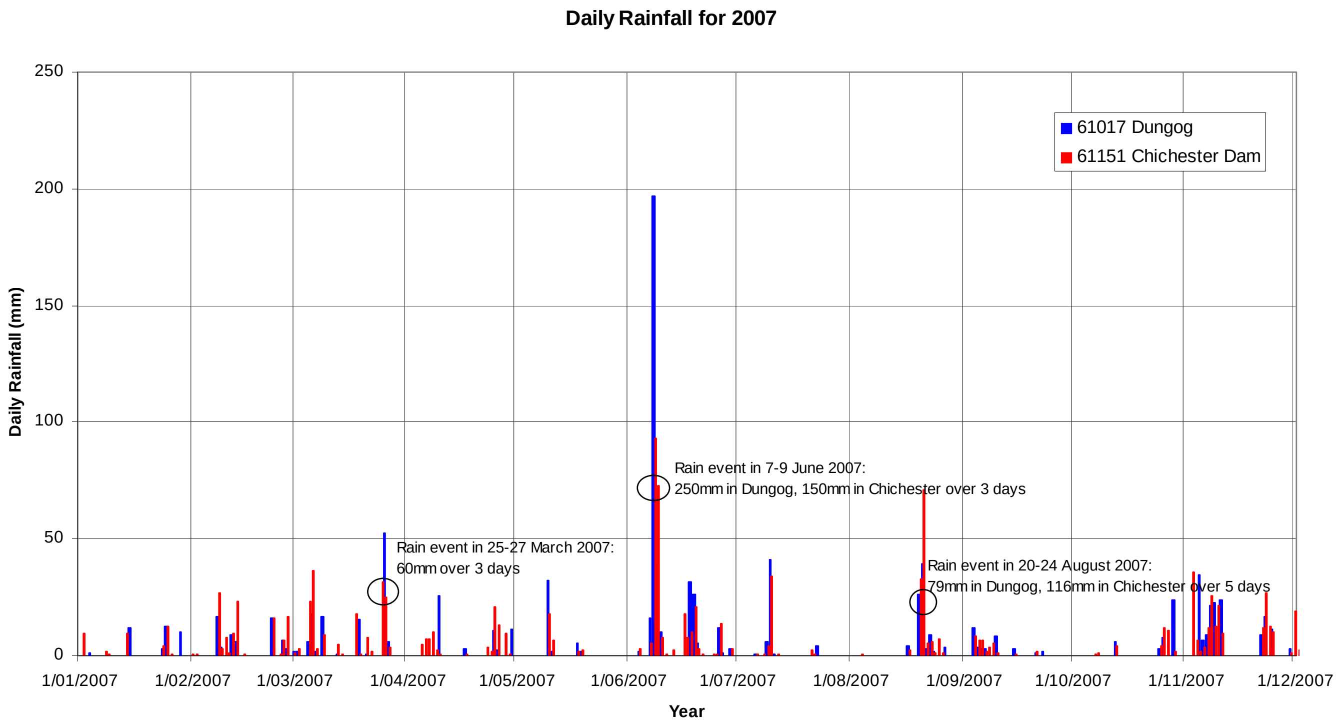


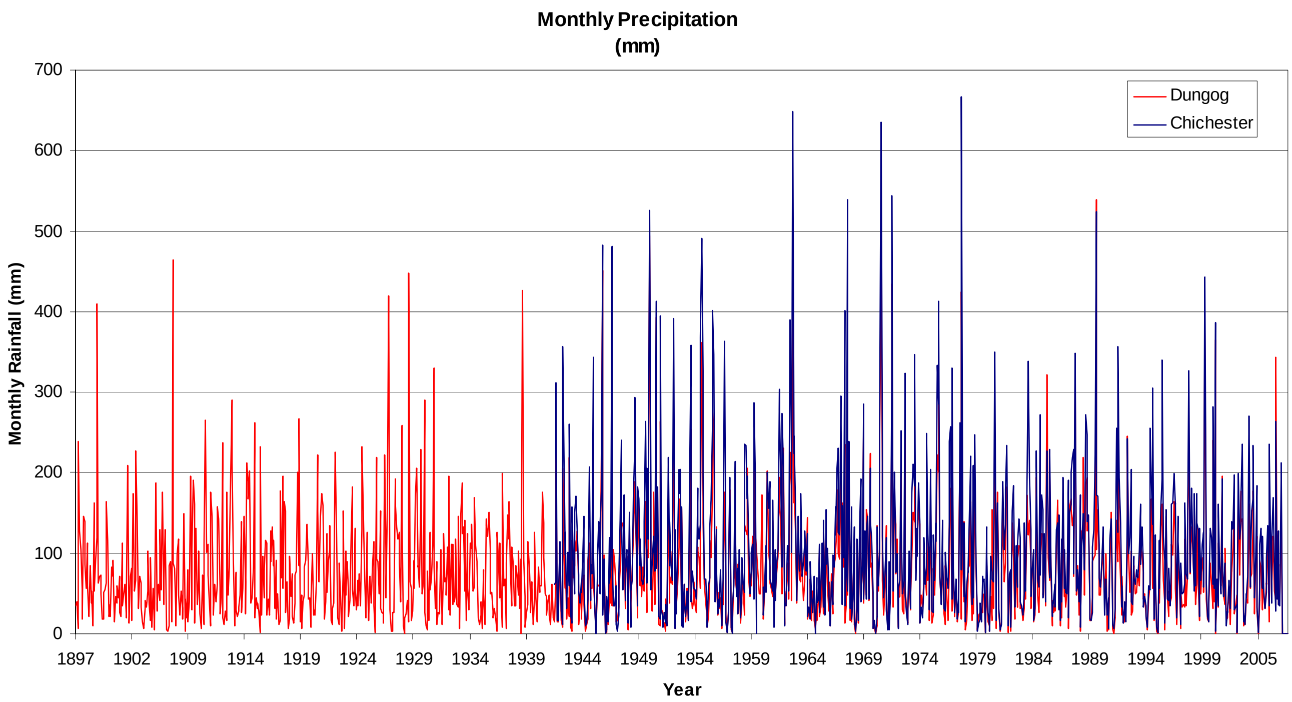
NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

GEOMORPHOLOGICAL MAP
(Prepared by Commerce)

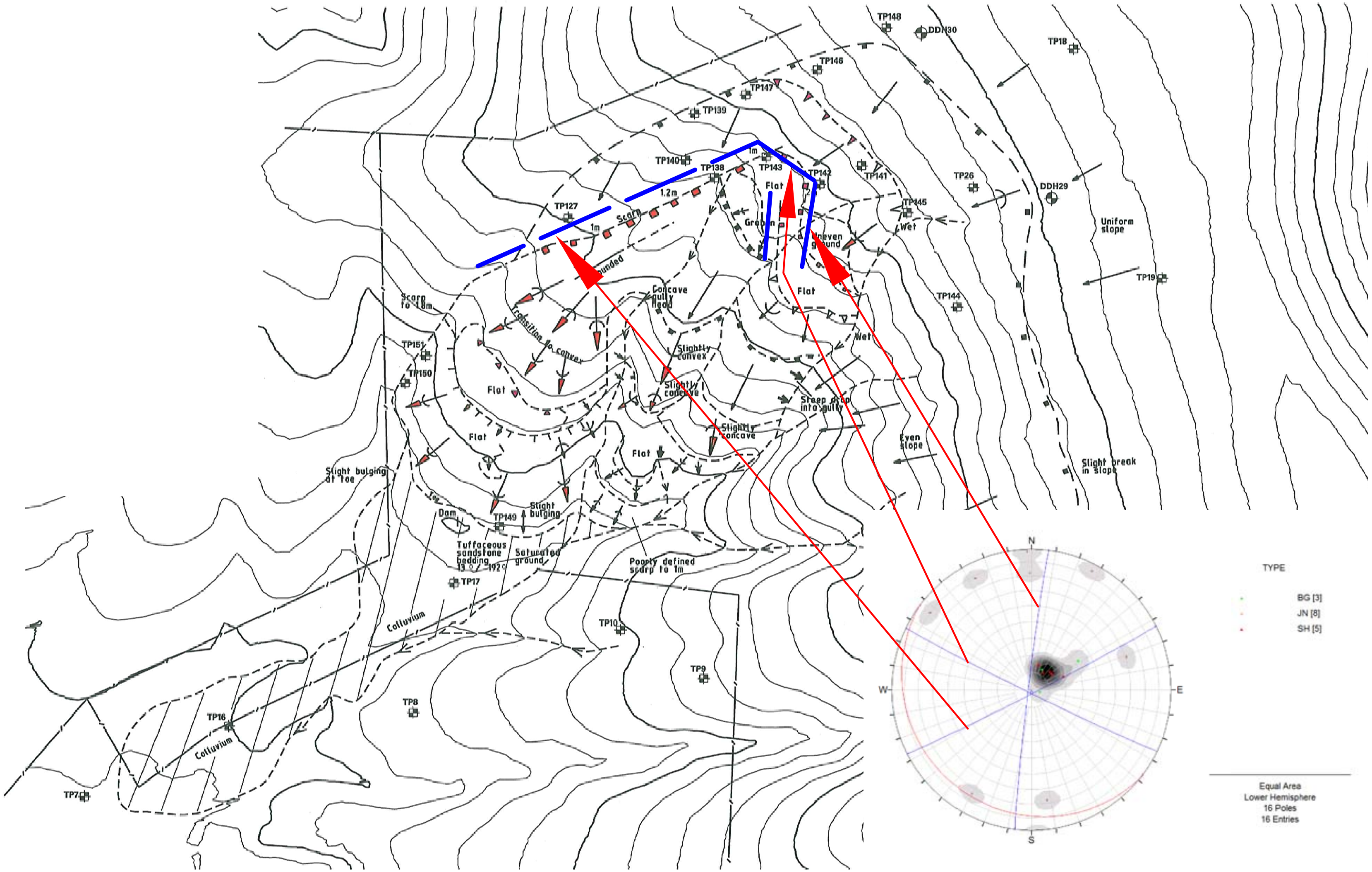
Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 19





Pells Sullivan Meynink Pty Ltd	NSW Department of Commerce Tillegra Dam Storage Rim Landslide Assessment RAINFALL CHICHESTER DAM & DUNGOG
PSM 1271.R1	Figure 20

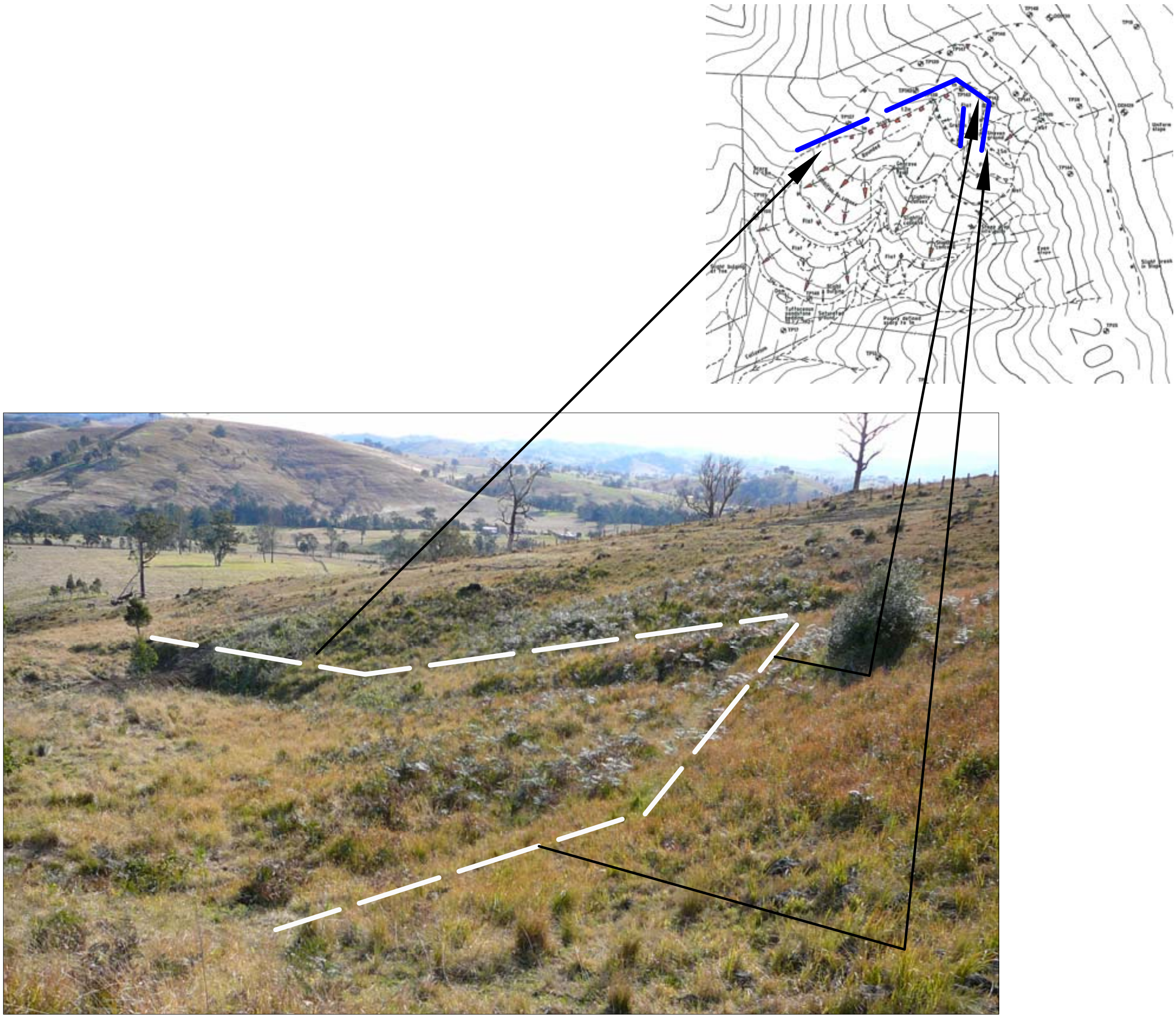


NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

GEOMORPHOLOGICAL PLAN

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 21



NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

LANDSLIDE CONTROLS

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 22

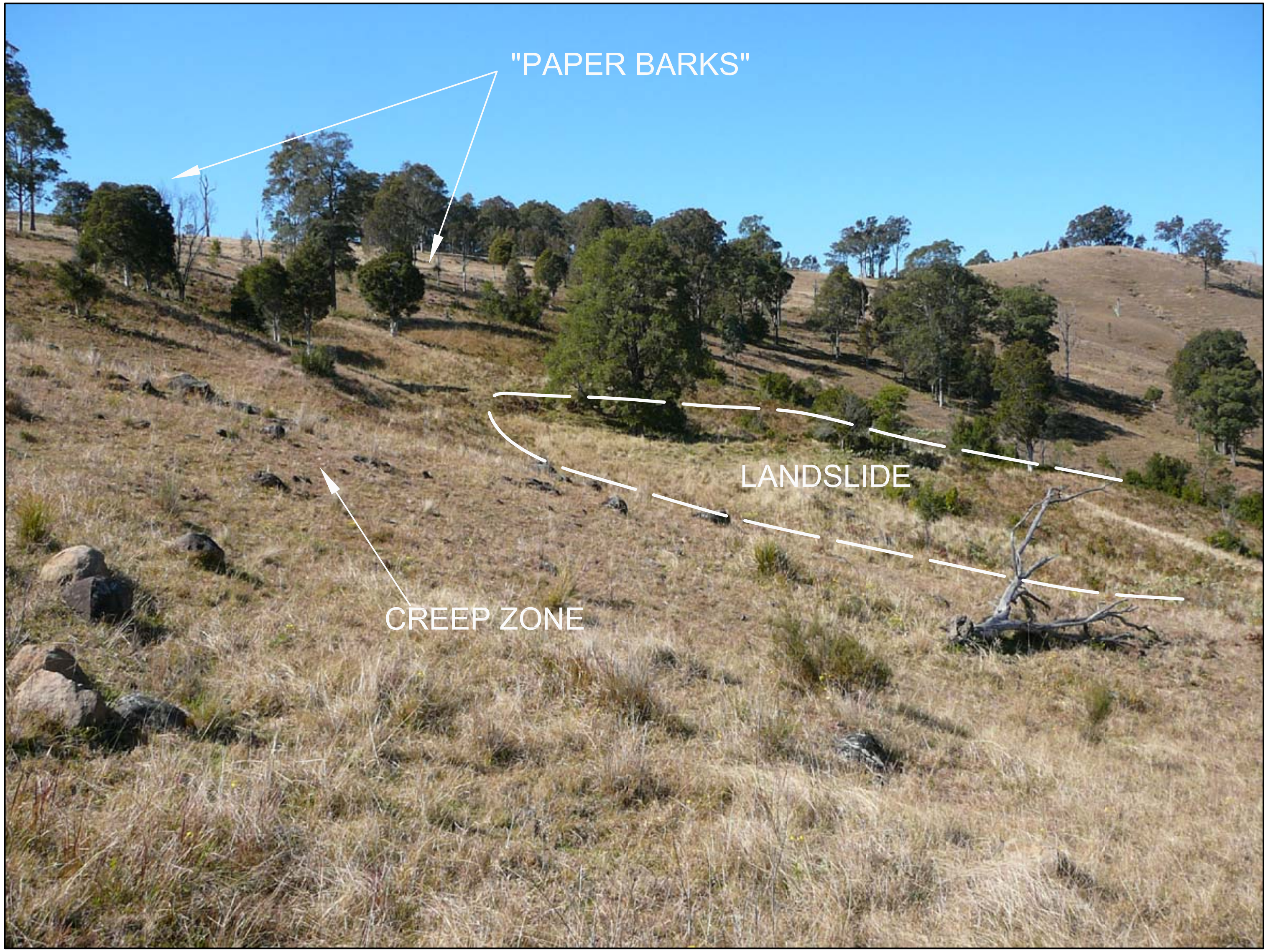
NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

VIEW TO NORTHEAST
LANDSLIDE 2A

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1

Figure 23



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NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

LANDSLIDE 2A
UPPER SECTION

PSM 1271.R1

Figure 24

Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

VIEW TO SOUTHEAST
ACROSS LANDSLIDE TO DOMAIN 4

PSM 1271.R1

Figure 25

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

TOE OF CENTRAL
LANDSLIDE LOBE

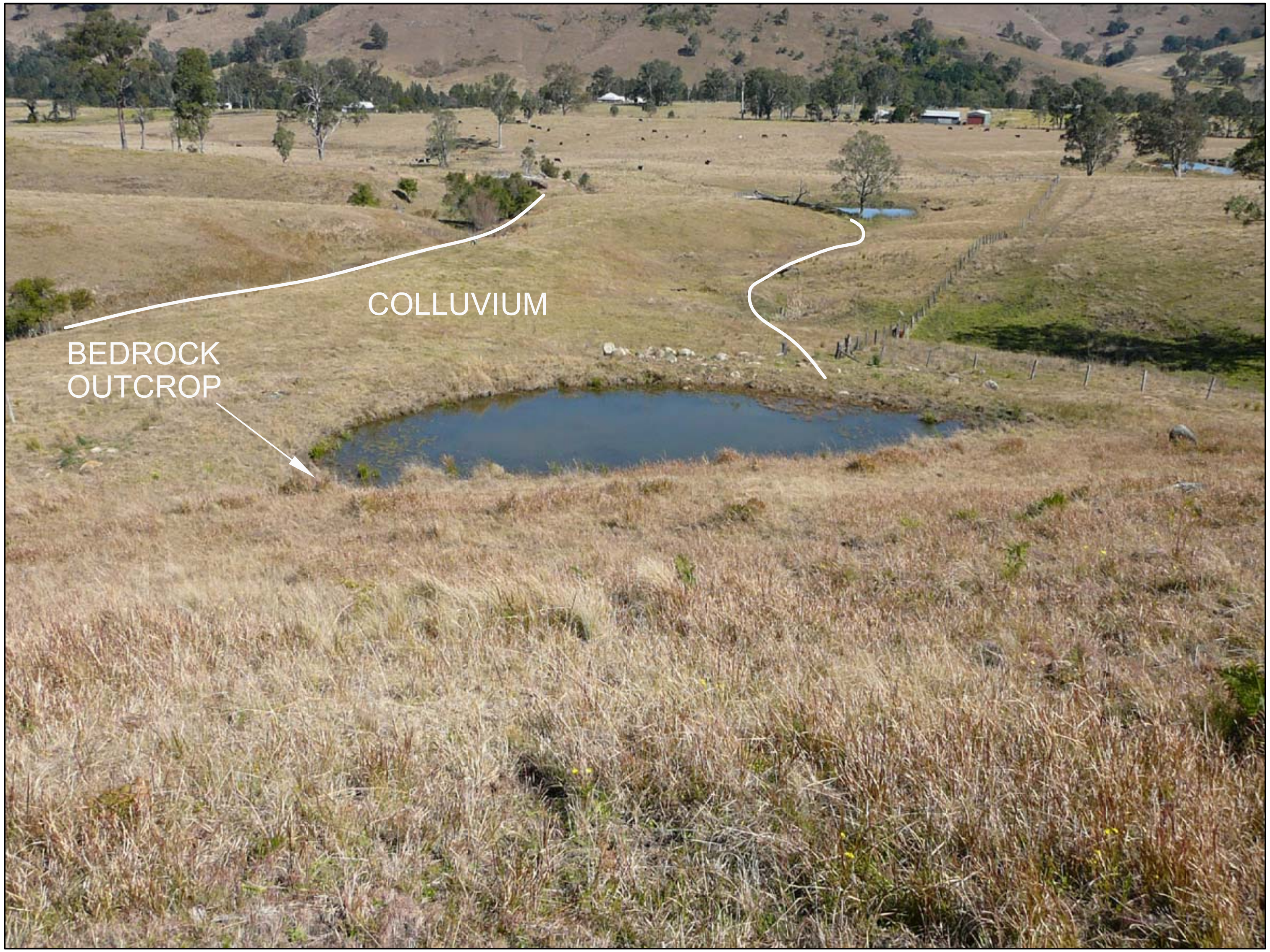
Pells Sullivan Meynink Pty Ltd

PSM 1271.R1

Figure 26

Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce Tillegra Dam Storage Rim Landslide Assessment BEDDING DIP 15° IN DAM AT TOE OF SLIDE 2A	
PSM 1271.R1	Figure 27



Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

TOE OF LANDSLIDE 2A

PSM 1271.R1

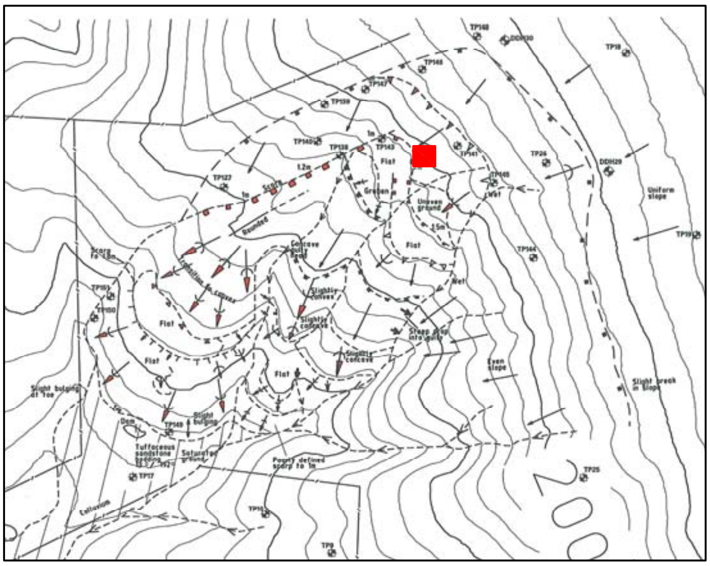
Figure 28

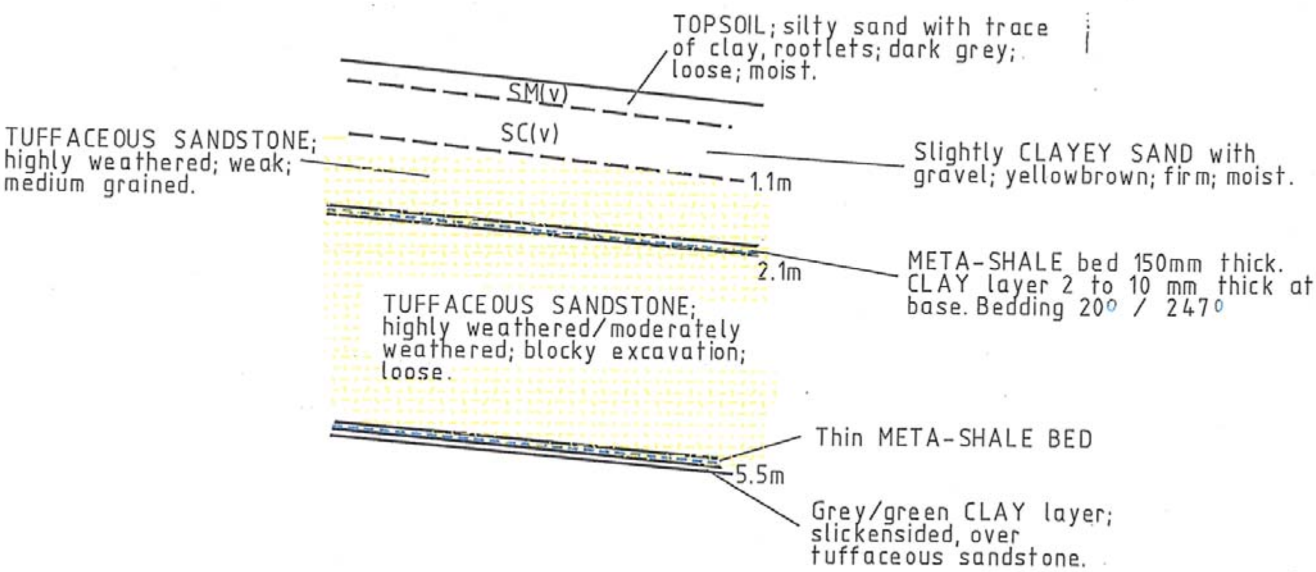
PSM
Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

SLIDE 2A - MATURE TREE
BELOW SLIDE SCARP

PSM 1271.R1 | Figure 29



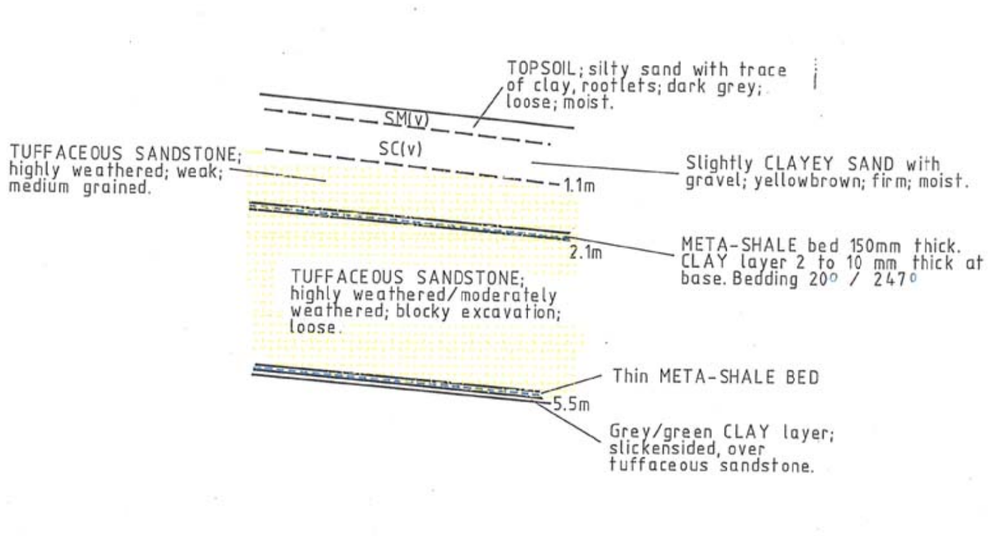


NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

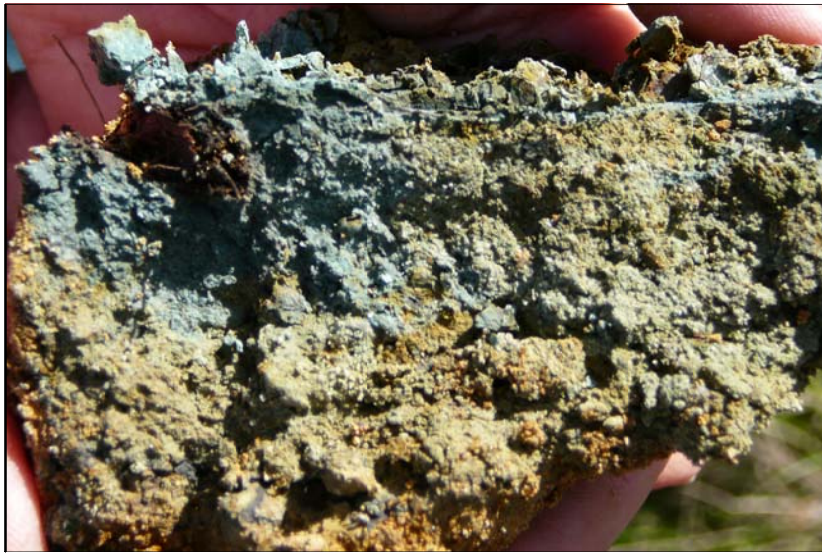
TP142 - TWO SLIDE PLANES
MULTI STOREY

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 30



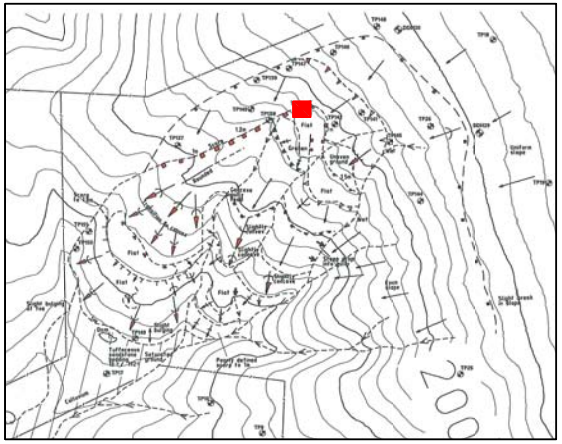
SURFACE OF CLAY LAYER

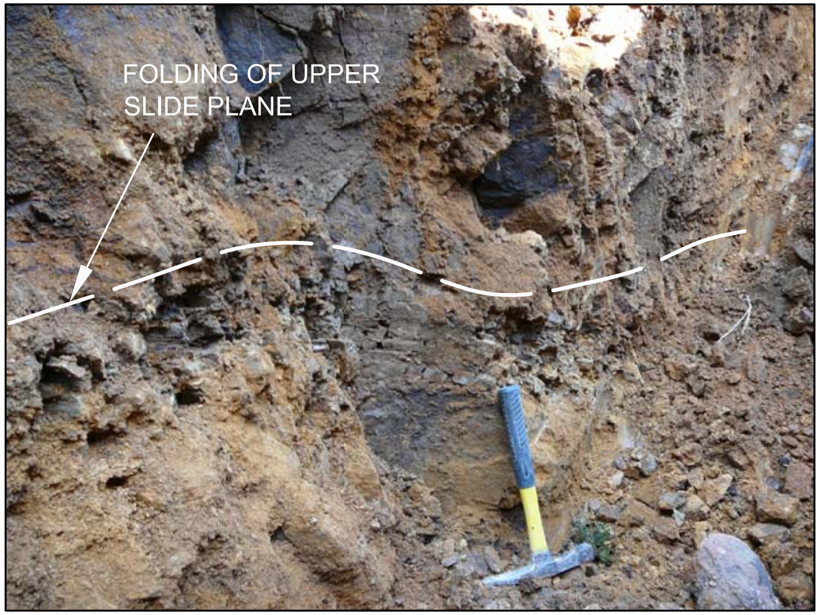


CLAY LAYER

NSW Department of Commerce Tillegra Dam Storage Rim Landslide Assessment BASAL SLIDE PLANE TP142	
PSM 1271.R1	Figure 31

Pells Sullivan Meynink Pty Ltd





NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

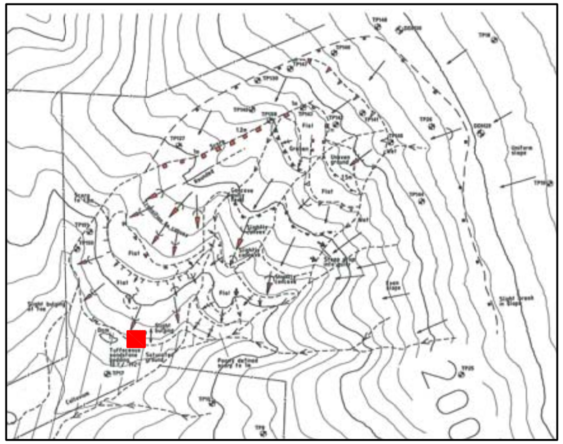
TWO SLIDE PLANES MULTI STOREY
PLUS REMOBILISATION
TP143

PSM

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1

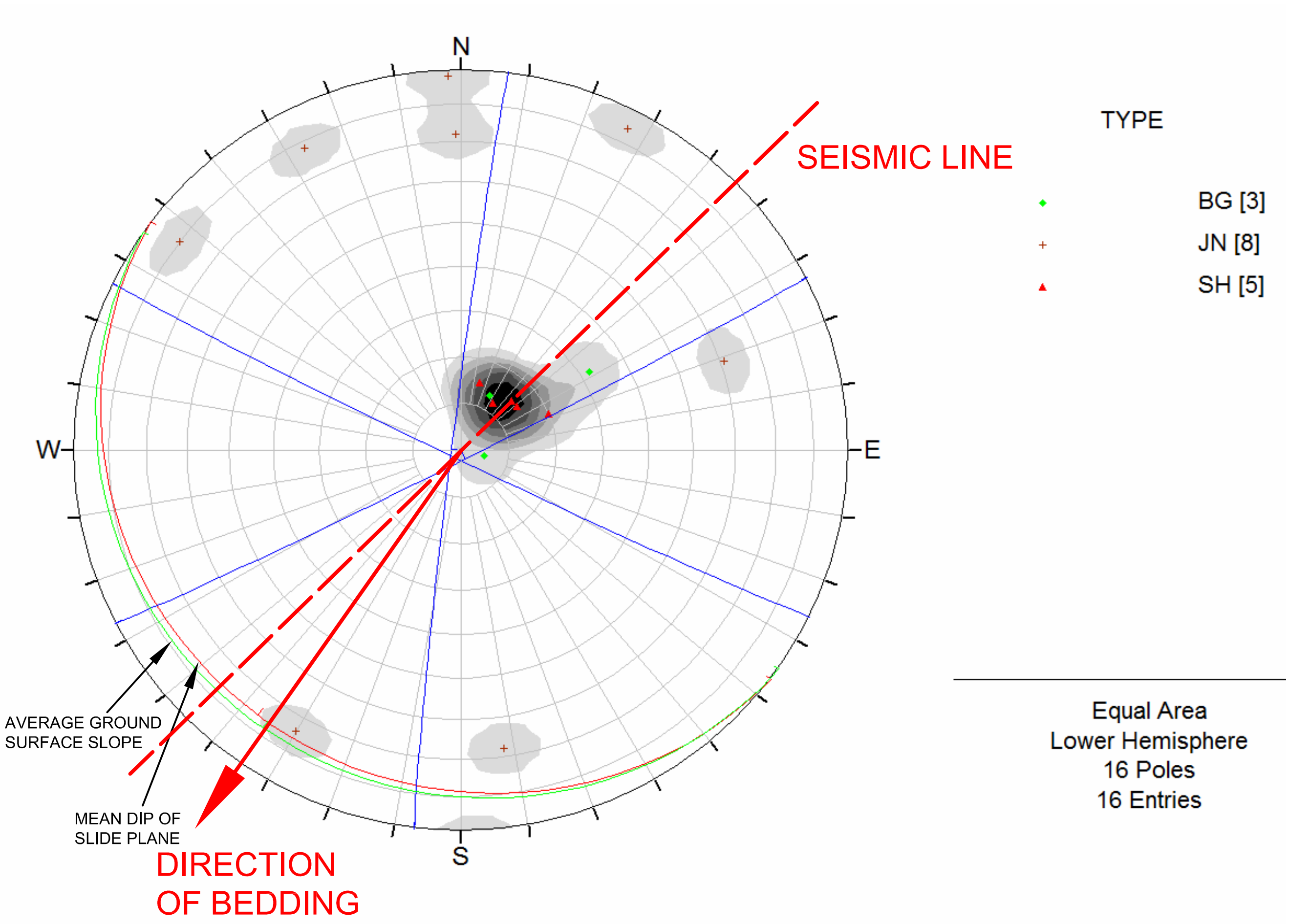
Figure 32



NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment
NON DAYLIGHTING TOE
"SHOVE ZONE"
TP149

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 33



NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

KINEMATIC ANALYSIS SLIDE 2A

Pells Sullivan Meynink Pty Ltd

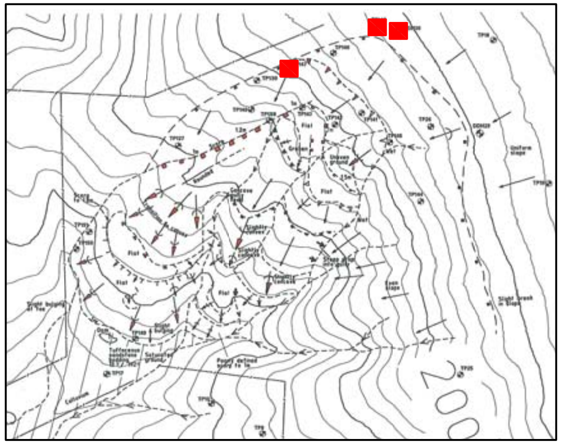
PSM 1271.R1	Figure 34

Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

CREEP ZONE ABOVE
LANDSLIDE 2A

PSM 1271.R1	Figure 35



CREEP ZONE

META SHALE

CLAY INFILL

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

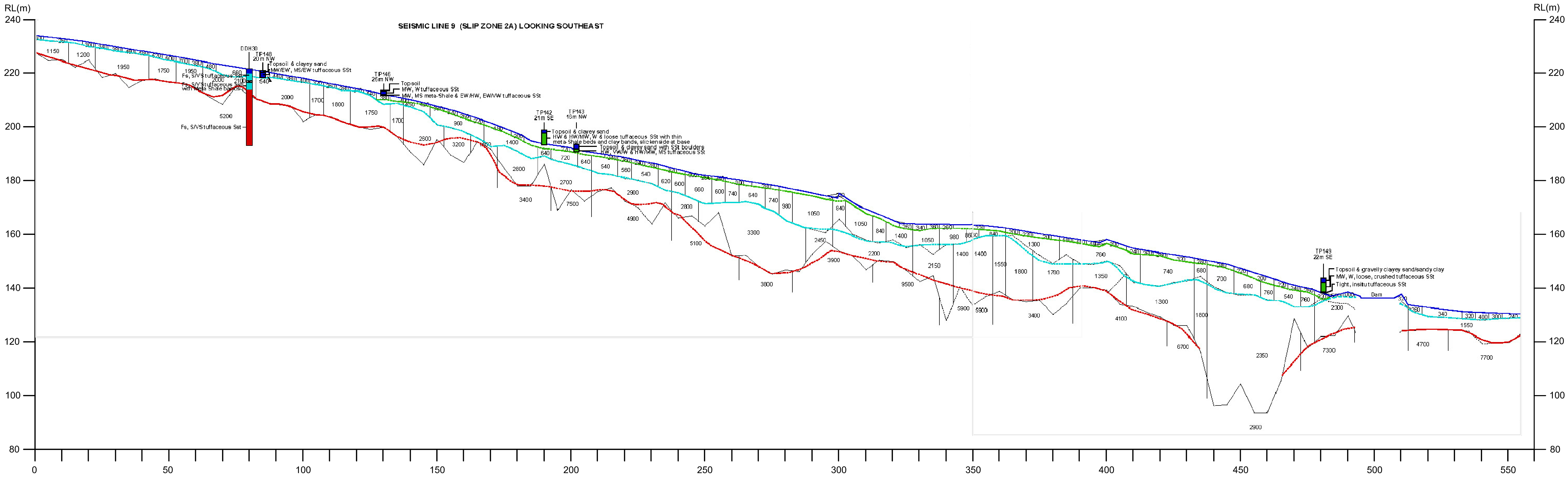
CHARACTER OF CREEP ZONE ABOVE LANDSLIDE 2A TP147 & TP148

PSM

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1

Figure 36

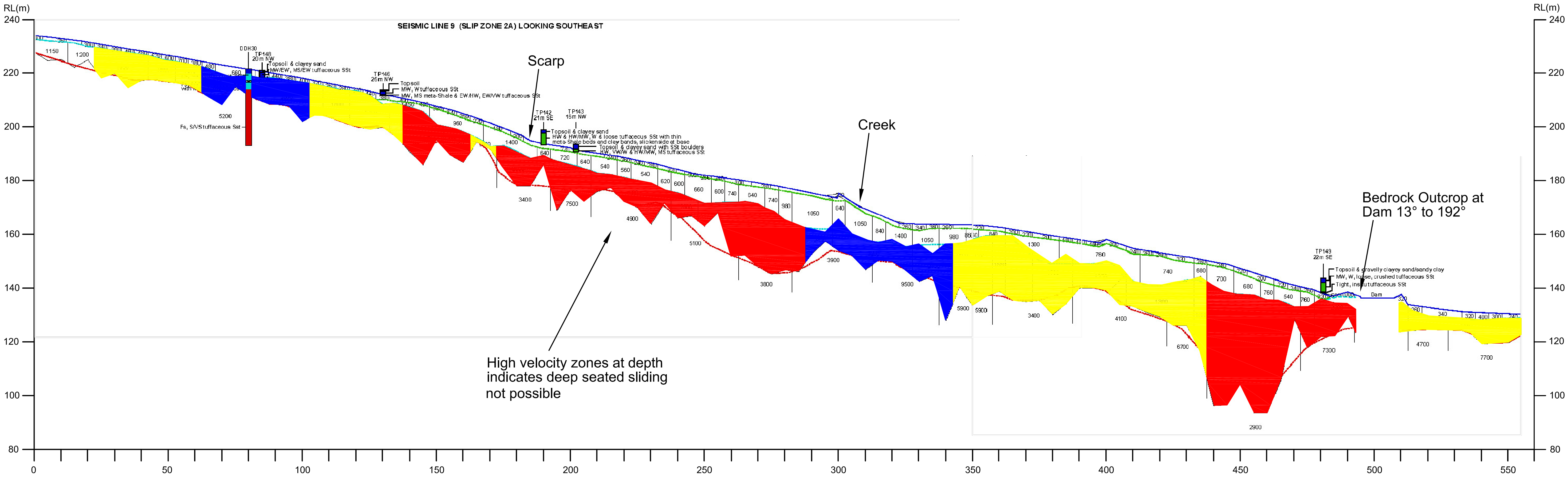


Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

SEISMIC REFRACTION
SURVEY RESULTS

PSM 1271.R1	Figure 37



LEGEND

Velocity Boundaries (m/s)

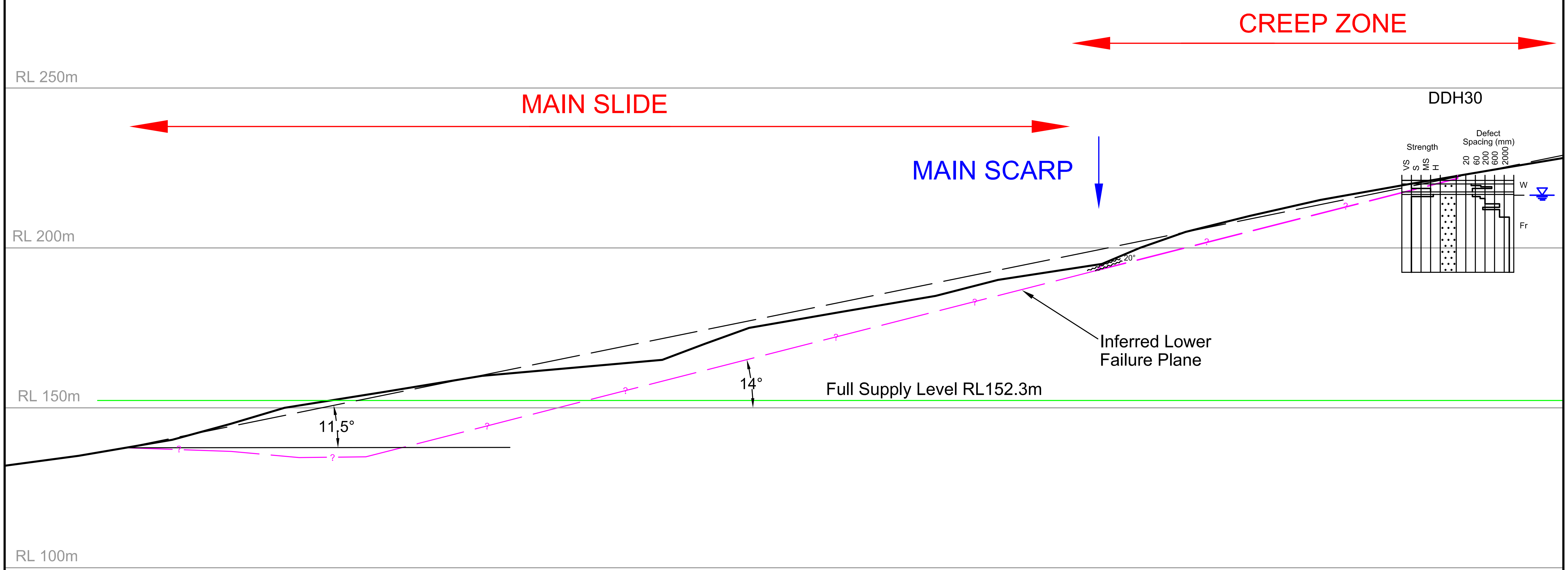
- < 2000
- > 2000 - 2500
- > 2500

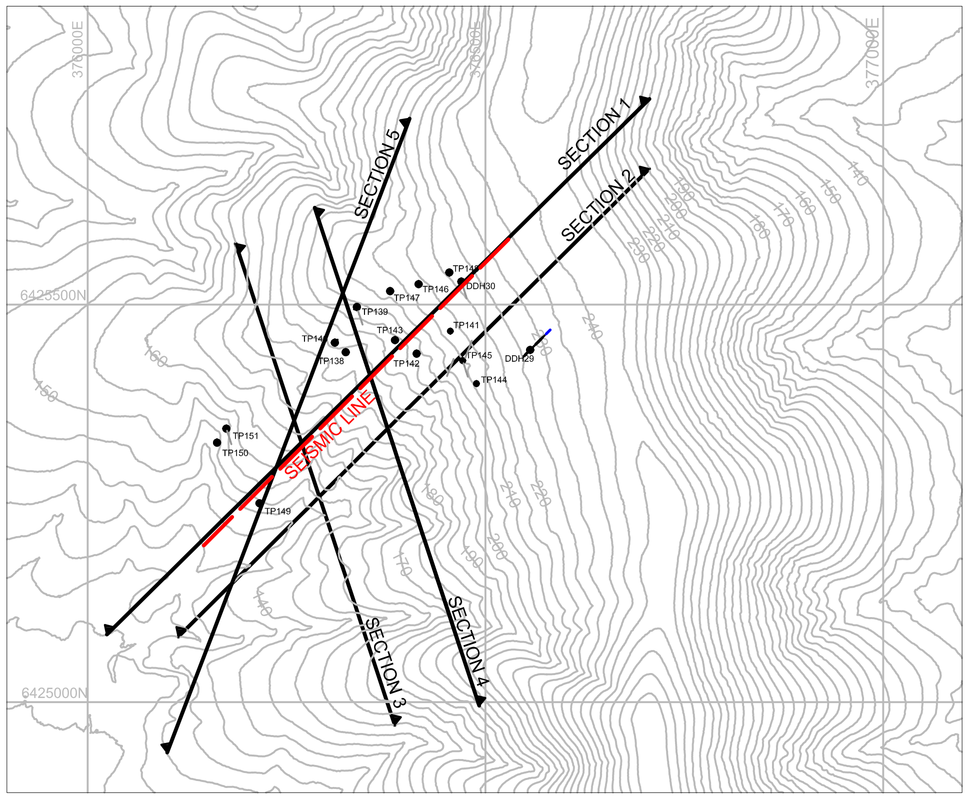
Pells Sullivan Meynink Pty Ltd

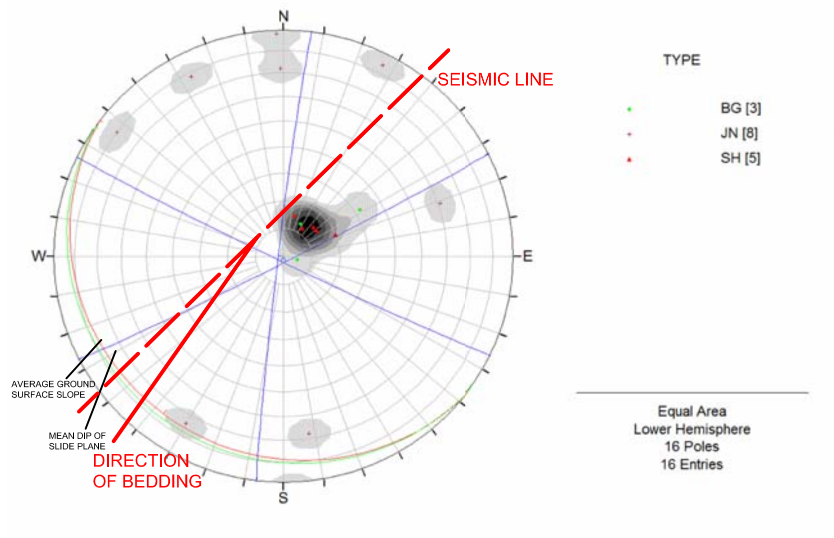
NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

SEISMIC REFRACTION
SURVEY INTERPRETATION

PSM 1271.R1	Figure 38





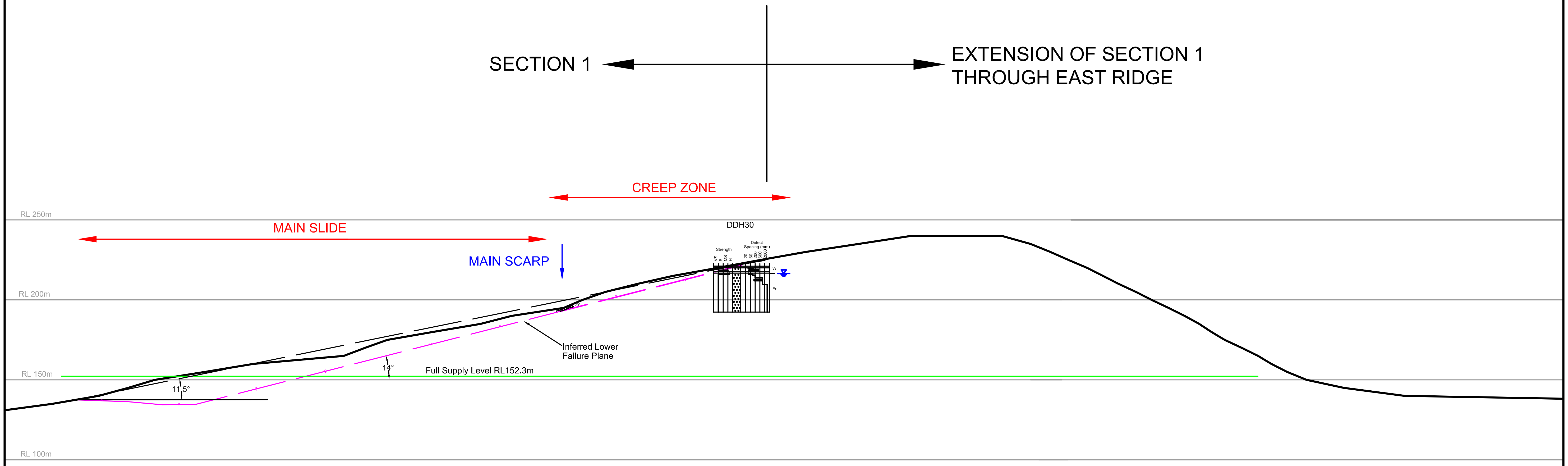


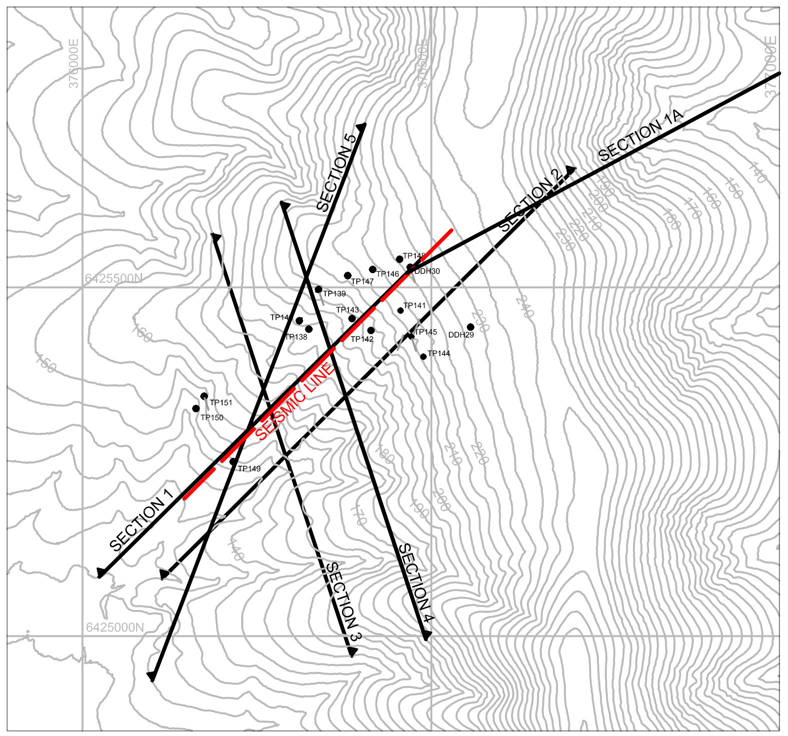
Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

TYPICAL SECTION (1)

PSM 1271.R1	Figure 39



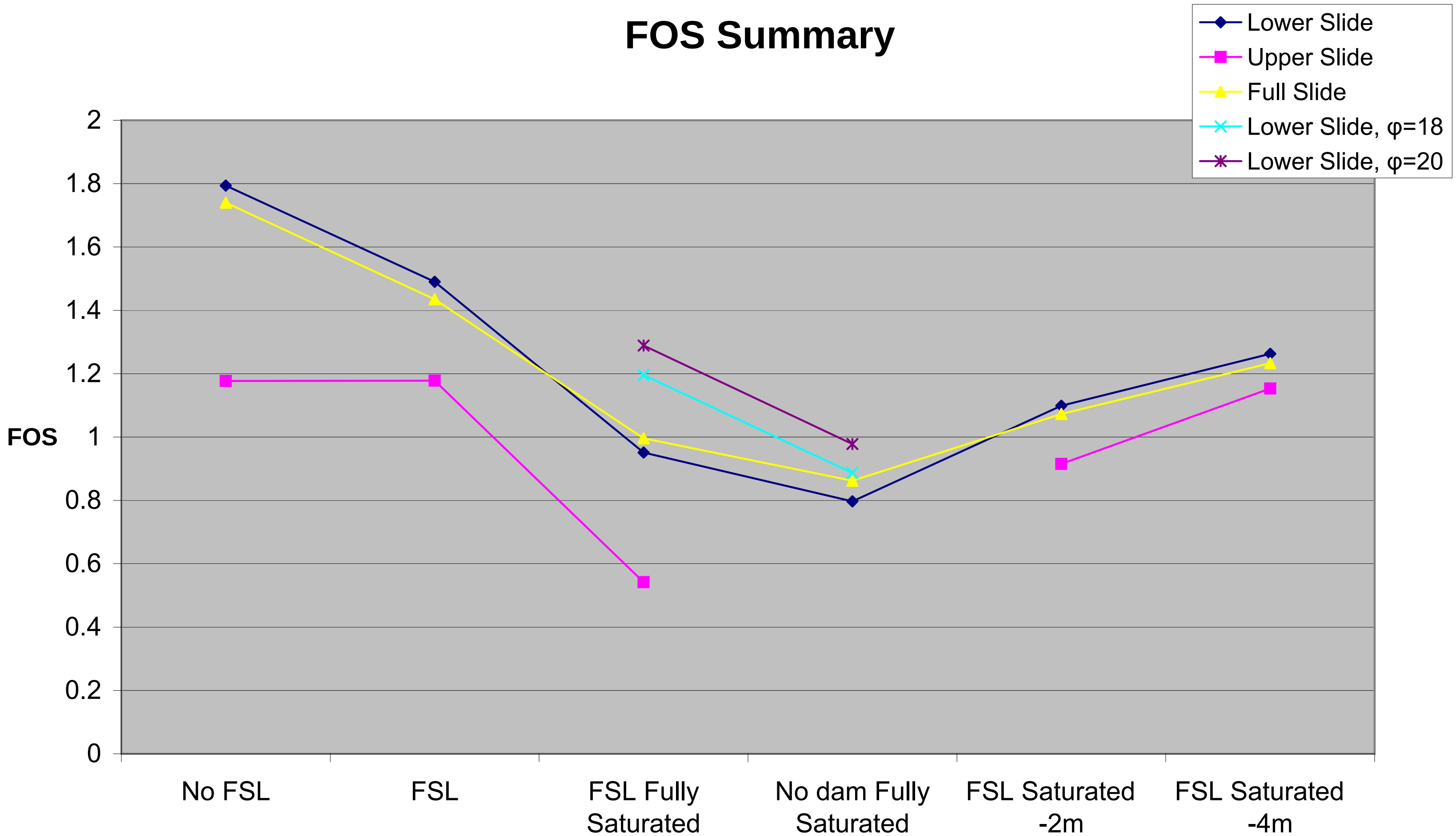


NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

TYPICAL SECTION 1A

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 40



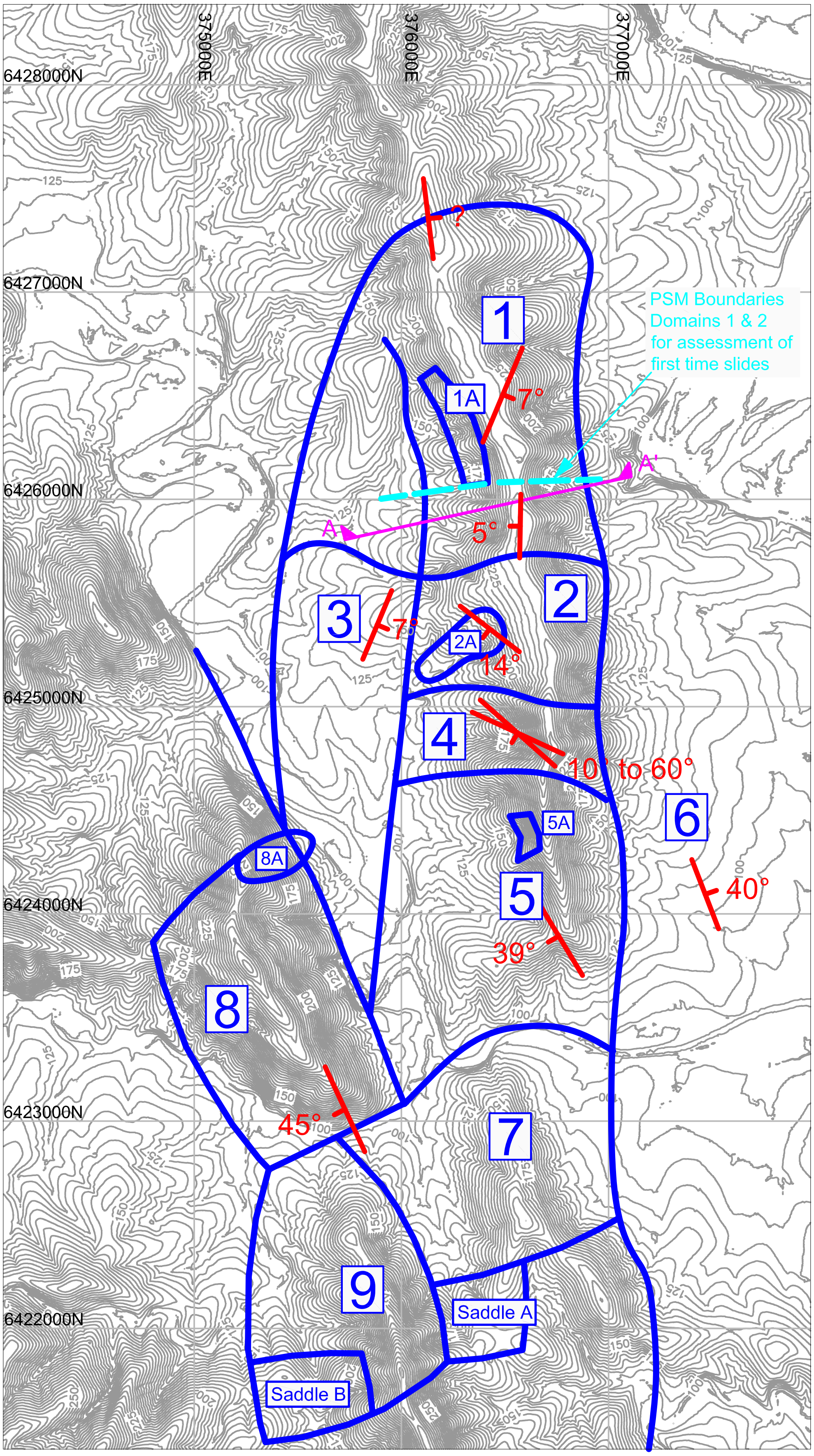
Pells Sullivan Meynink Pty Ltd

NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

STABILITY ANALYSIS RESULTS, SLIDE 2A

PSM1271.R1	**Figure 41**

U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\[Slide Summary.xls]Graph 1



NSW Department of Commerce
Tillegra Dam
Storage Rim Landslide Assessment

SUMMARY MODEL FOR FIRST TIME SLIDE & RIM STABILITY

PSM

Pells Sullivan Meynink Pty Ltd

PSM 1271.R1	Figure 42

APPENDIX A

ADDITIONAL PHOTOGRAPHS

Photo 1: Trig Hill

Photo 2: Views of the slide 2A

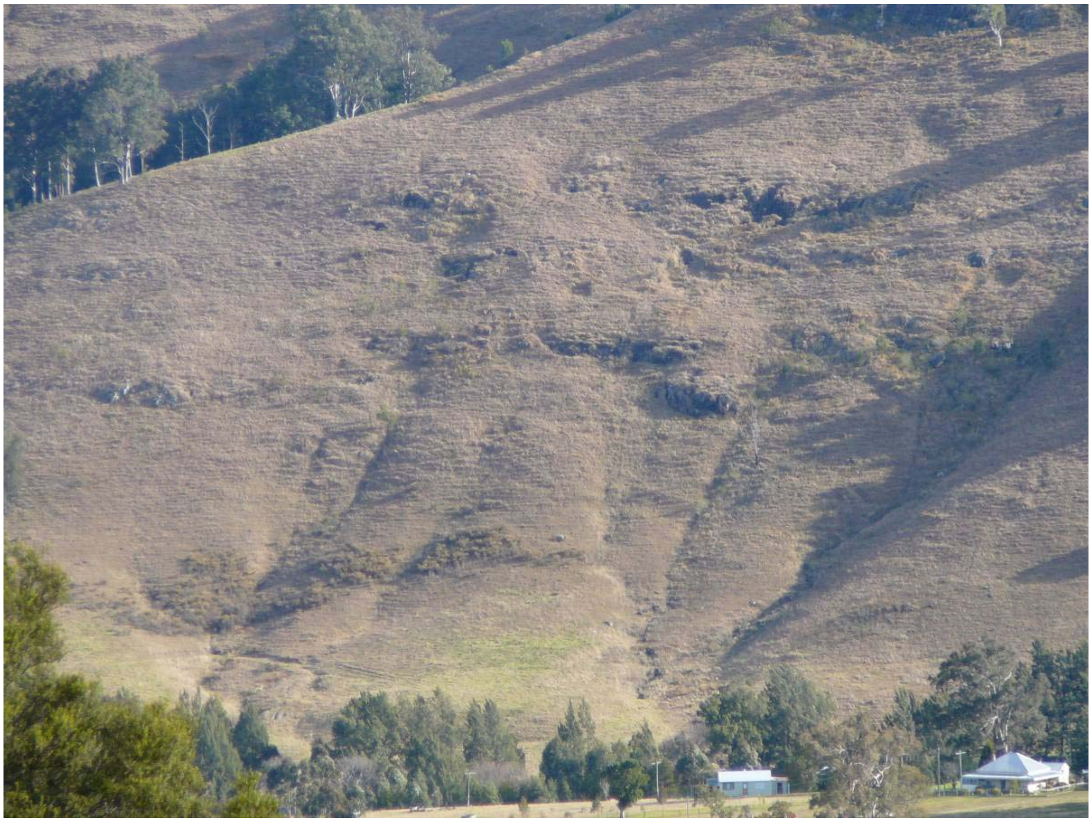


Photo 3: Possible Slide 8A

Photo 4: Outwash fan below Slide 1A note small boulders only

Photo 5: View to South east in Domain 2 above Head scarp Slide 2A
Note Melaleucas “paper bark"

Photo 6: Slope above Slide 2A note melaleucas and change in slope

Photo 7: Slide 2A

Photo 8: TP138 Scarp and topsoil “infill” tension zone


Photo 9: TP138 and TP139


Photo 10: TP140 Note loosened rock mass in creep zone

Photo 11: TP143 upper slide plane

Photo 12: TP142 Lower slide plane, thin clay seam in extremely weathered rock

Photo 13: View of TP147 and TP148

Photo 14: TP146, 147, 148

Photo 15: TP149 in toe of main slide

Photo 16: TP150, TP 151 setting and photos

APPENDIX B

STRUCTURAL DATA FOR DOMAINS

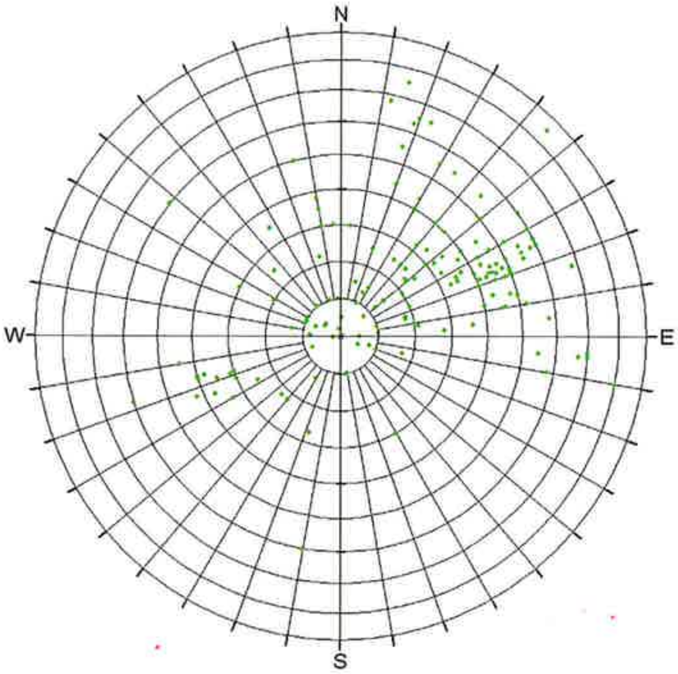
Domains 1 to 7 Combined Bedding
Domains 1 to 7 Combined Joints
Domains 1 to 7 Bedding
Domains 1 to 7 Joints

PSM **Pells Sullivan Meynink Pty Ltd**

Stereo Plot

Job no: 1271

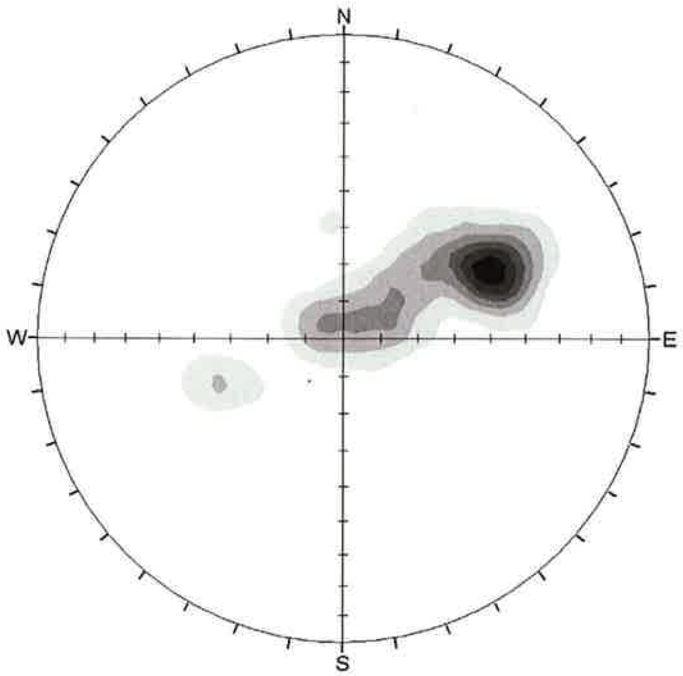
Project: Tillegra Dam	**North datum:** Grid	**Processed by:** James Bevis
		Date: 26/08/2008
Domain: Domains 1-7	**Traverse:** N/A	**Defect type:** Bedding
Lithology: N/A	**Borehole:** N/A	**Length:** N/A



TYPE

BG [162]

Equal Area
Lower Hemisphere
162 Poles
162 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 2.50 %
2.50 ~ 5.00 %
5.00 ~ 7.50 %
7.50 ~ 10.00 %
10.00 ~ 12.50 %
12.50 ~ 15.00 %
15.00 ~ 17.50 %

No Bias Correction
Max. Conc. = 16.7674%

Equal Area
Lower Hemisphere
162 Poles
162 Entries

1271_Domains_1-7_Bedding\10/20/2008

PSM **Pells Sullivan Meynink Pty Ltd**

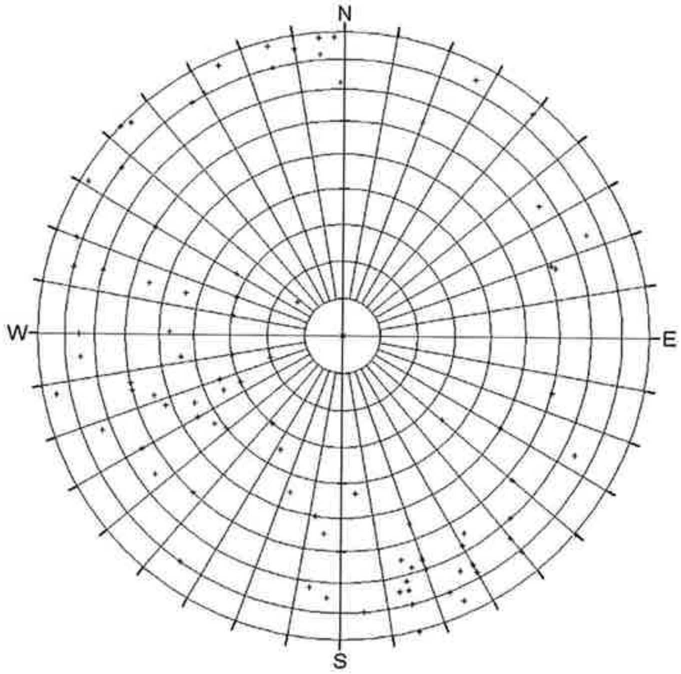
Stereo Plot

Job no: 1271

Project: Tillegra Dam
Domain: 1-7
Lithology: N/A

North datum: Grid
Traverse: N/A
Borehole: N/A

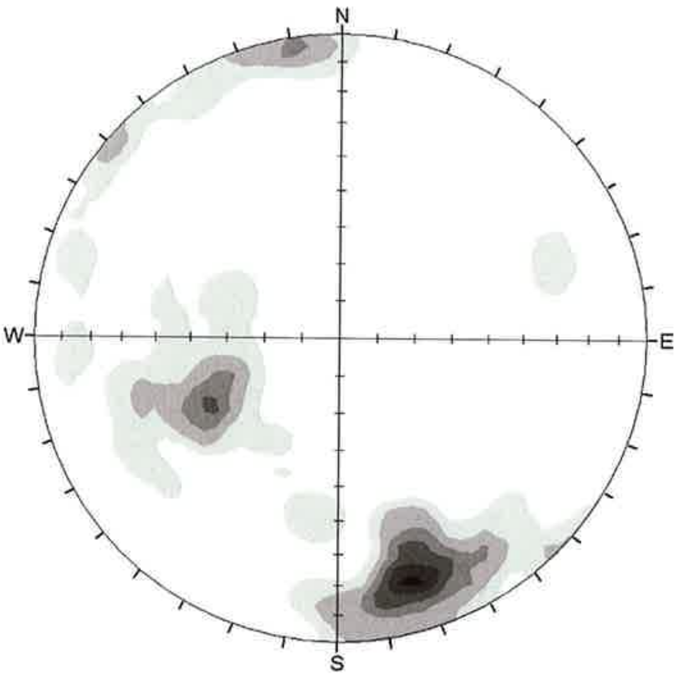
Processed by: James Bevis
Date: 26/08/2008
Defect type: Joints
Length: N/A



TYPE

JN [99]

Equal Area
Lower Hemisphere
99 Poles
99 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 1.50 %
1.50 ~ 3.00 %
3.00 ~ 4.50 %
4.50 ~ 6.00 %
6.00 ~ 7.50 %
7.50 ~ 9.00 %
9.00 ~ 10.50 %

No Bias Correction
Max. Conc. = 9.4606%

Equal Area
Lower Hemisphere
99 Poles
99 Entries

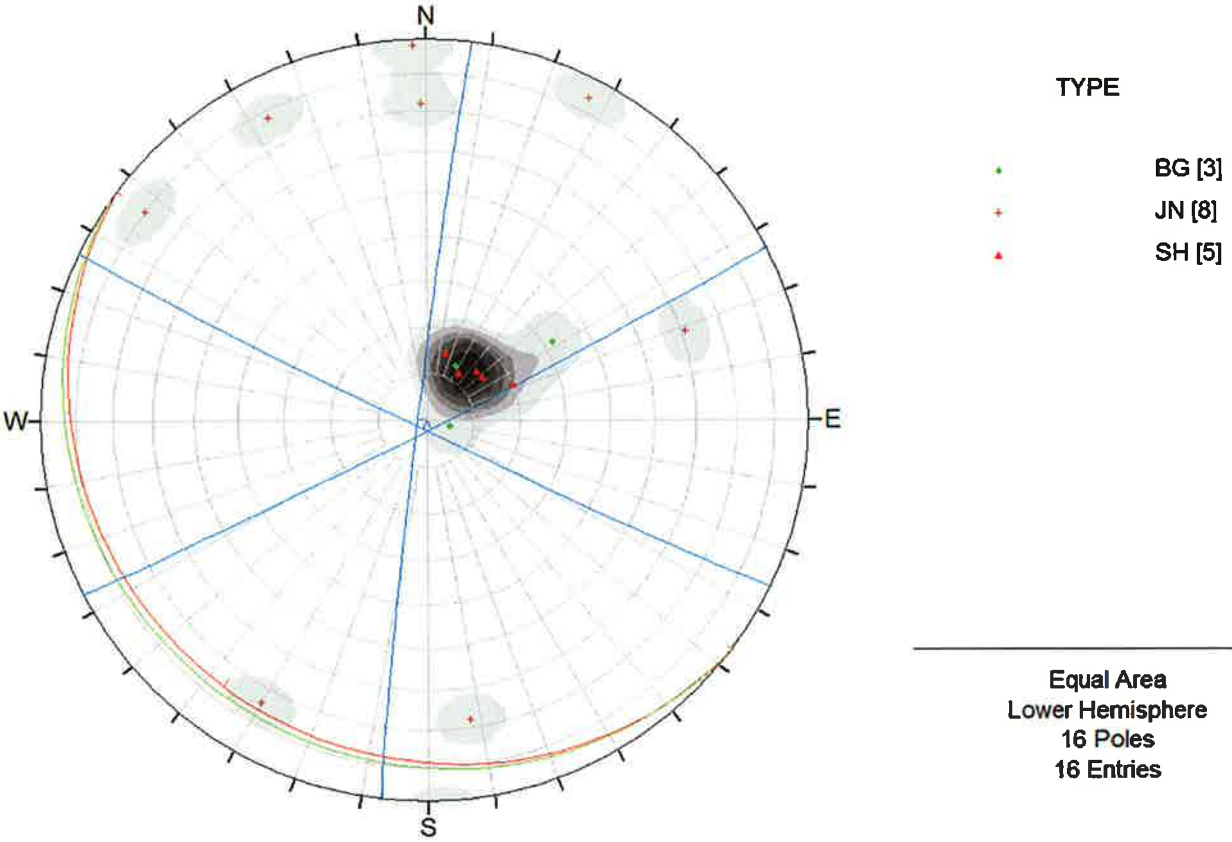
1271_Domains_1-7_Joints\10/20/2008

PSM **Pells Sullivan Meynink Pty Ltd**

Stereo Plot

Job no: 1271

Project:	TILLEGRA DAM	**North datum:**		**Processed by:**	TS
		Date:	18-9-08		
Domain:	SLIDE 2A	**Traverse:**		**Defect type:**	
Lithology:		**Borehole:**		**Length:**	



Equal Area
Lower Hemisphere
16 Poles
16 Entries

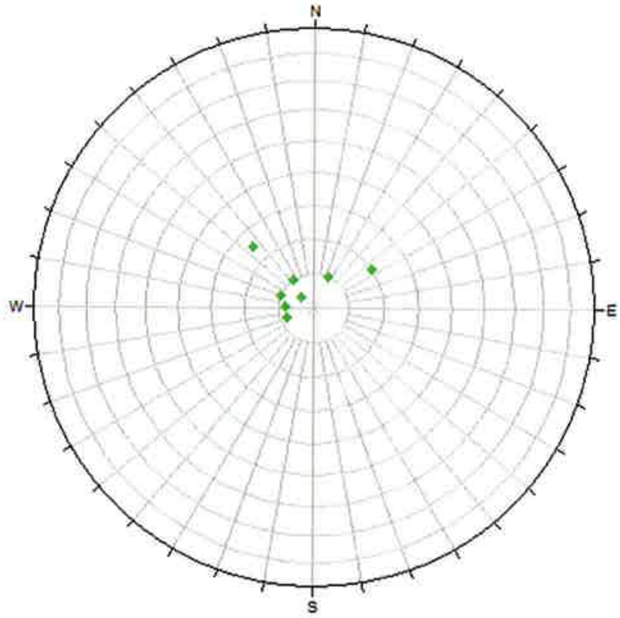
SLIDE 2A\10/20/2008

Pells Sullivan Meynink Pty Ltd

Stereo Plot

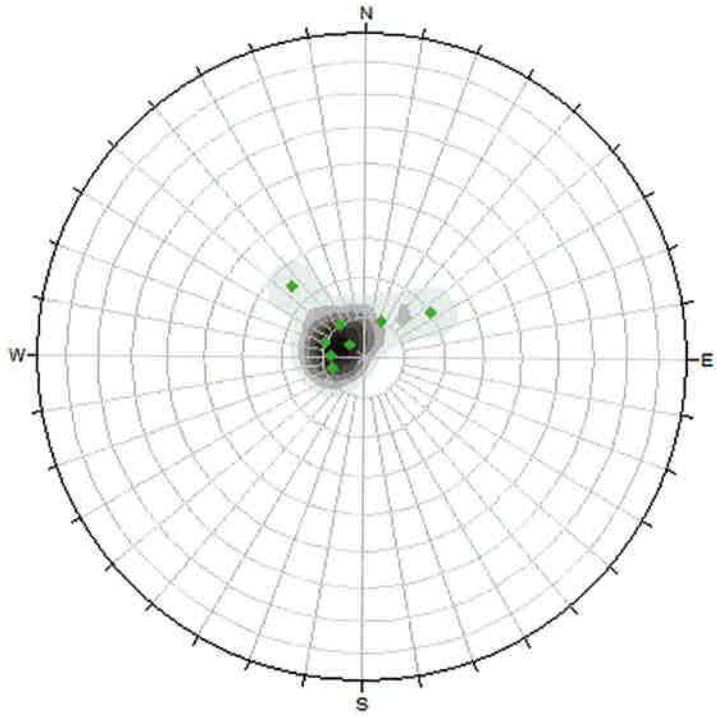
Job no: 1271

Project: Tillegra Dam	**North datum:** Grid	**Processed by:** James Bevis
Domain: 1	**Traverse:** N/A	**Date:** 26/08/2008
Lithology: N/A	**Borehole:** N/A	**Defect type:** Bedding
		Length: N/A



Poles

Equal Area
Lower Hemisphere
8 Poles
8 Entries



Poles

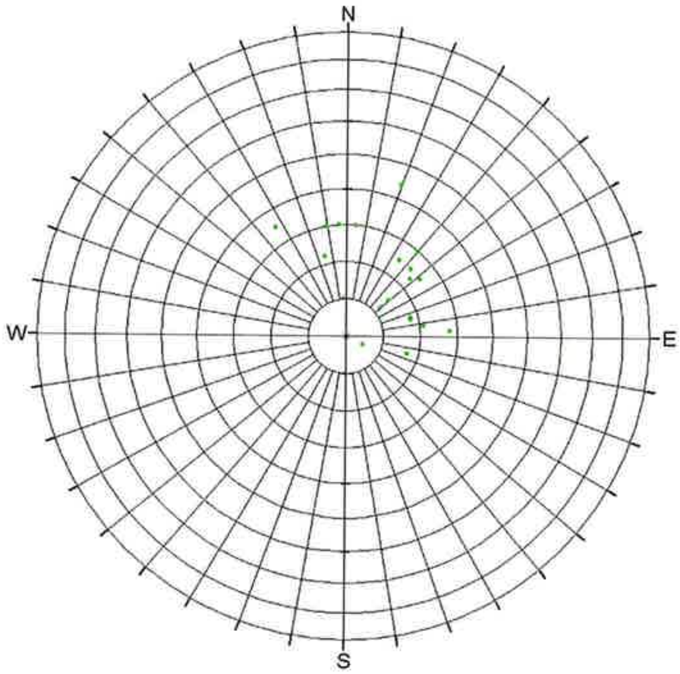
Equal Area
Lower Hemisphere
8 Poles
8 Entries

PSM **Pells Sullivan Meynink Pty Ltd**

Stereo Plot

Job no: 1271

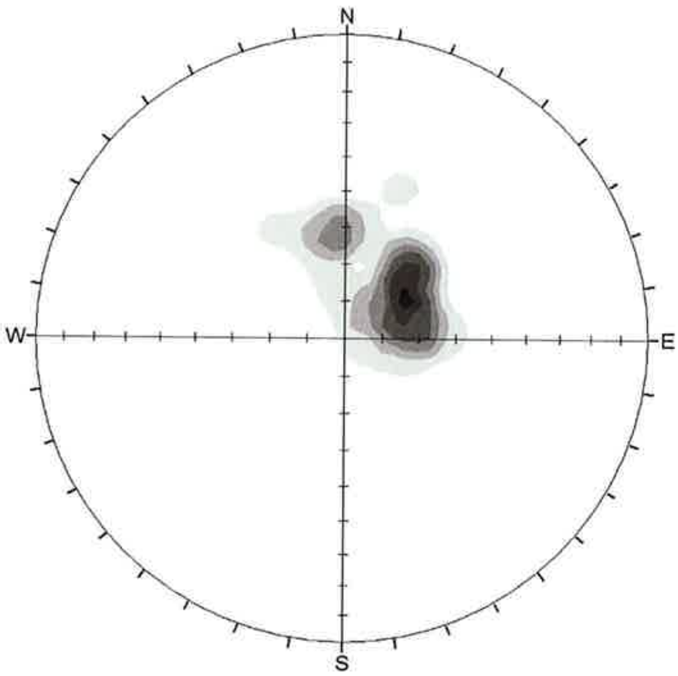
Project: Tillegra Dam	**North datum:** Grid	**Processed by:** James Bevis
Domain: 2	**Traverse:** N/A	**Date:** 26/08/2008
Lithology: N/A	**Borehole:** N/A	**Defect type:** Bedding
		Length: N/A



TYPE

BG [20]

Equal Area
Lower Hemisphere
20 Poles
20 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 4.50 %
4.50 ~ 9.00 %
9.00 ~ 13.50 %
13.50 ~ 18.00 %
18.00 ~ 22.50 %
22.50 ~ 27.00 %
27.00 ~ 31.50 %

No Bias Correction
Max. Conc. = 29.5086%

Equal Area
Lower Hemisphere
20 Poles
20 Entries

Pells Sullivan Meynink Pty Ltd

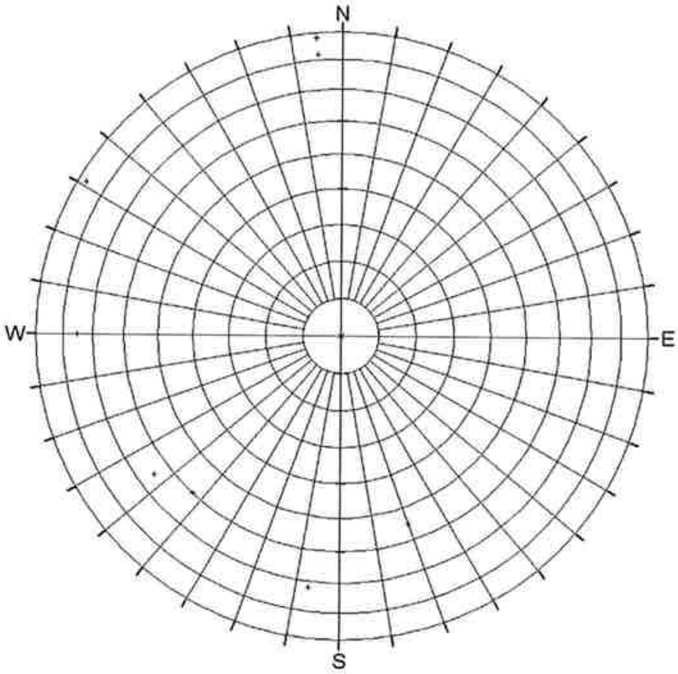
Stereo Plot

Job no: 1271

Project: Tillegra Dam
Domain: 2
Lithology: N/A

North datum: Grid
Traverse: N/A
Borehole: N/A

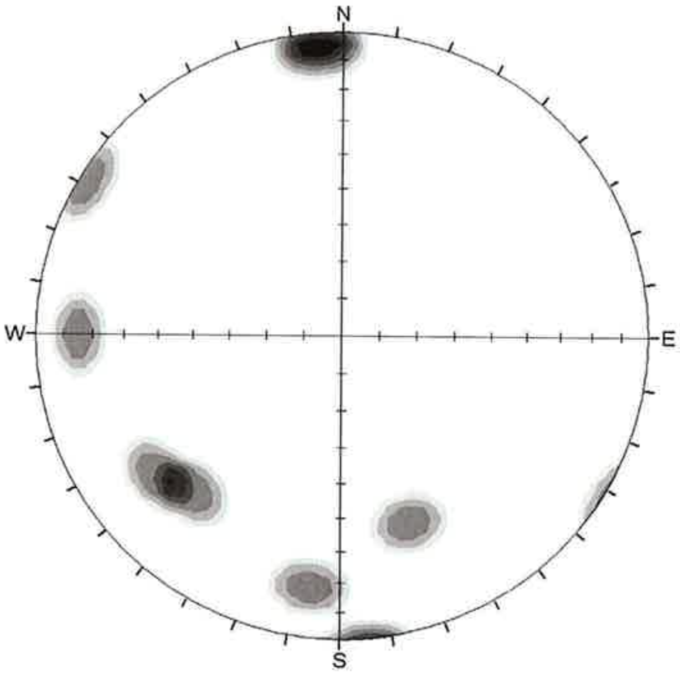
Processed by: James Bevis
Date: 26/08/2008
Defect type: Joints
Length: N/A



TYPE

JN [8]

Equal Area
Lower Hemisphere
8 Poles
8 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 3.50 %
3.50 ~ 7.00 %
7.00 ~ 10.50 %
10.50 ~ 14.00 %
14.00 ~ 17.50 %
17.50 ~ 21.00 %
21.00 ~ 24.50 %

No Bias Correction
Max. Conc. = 24.1124%

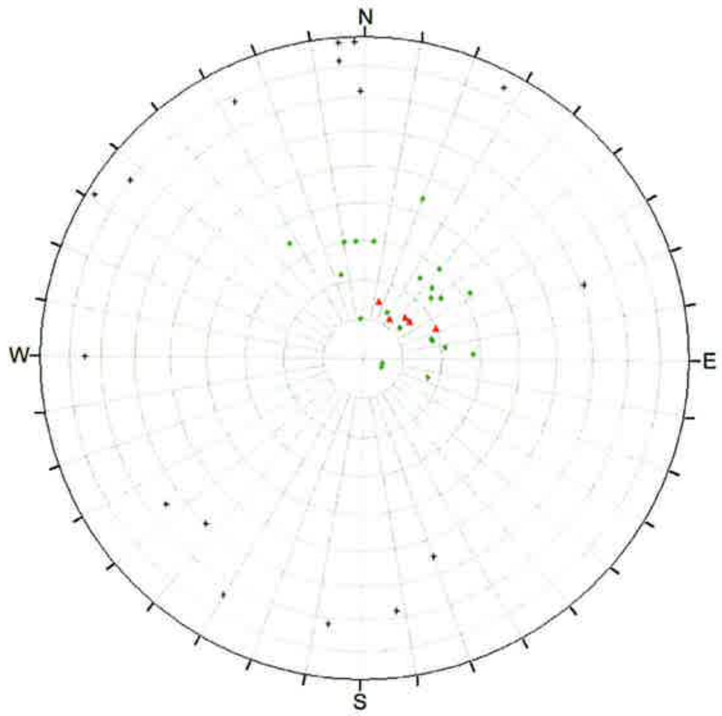
Equal Area
Lower Hemisphere
8 Poles
8 Entries

PSM **Pells Sullivan Meynink Pty Ltd**

Stereo Plot

Job no: 1271

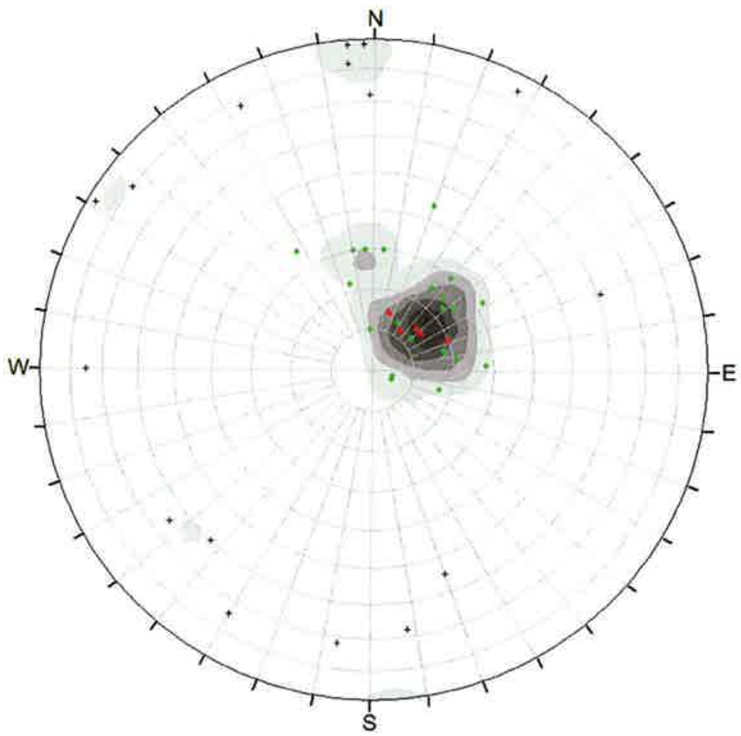
Project:	TILLEGRA	**North datum:**		**Processed by:**	TS
		Date:	17-9-08		
Domain:	2	**Traverse:**		**Defect type:**	ALL
Lithology:		**Borehole:**		**Length:**	



TYPE

BG [23]
JN [16]
SH [5]

Equal Area
Lower Hemisphere
44 Poles
44 Entries



TYPE

BG [23]
JN [16]
SH [5]

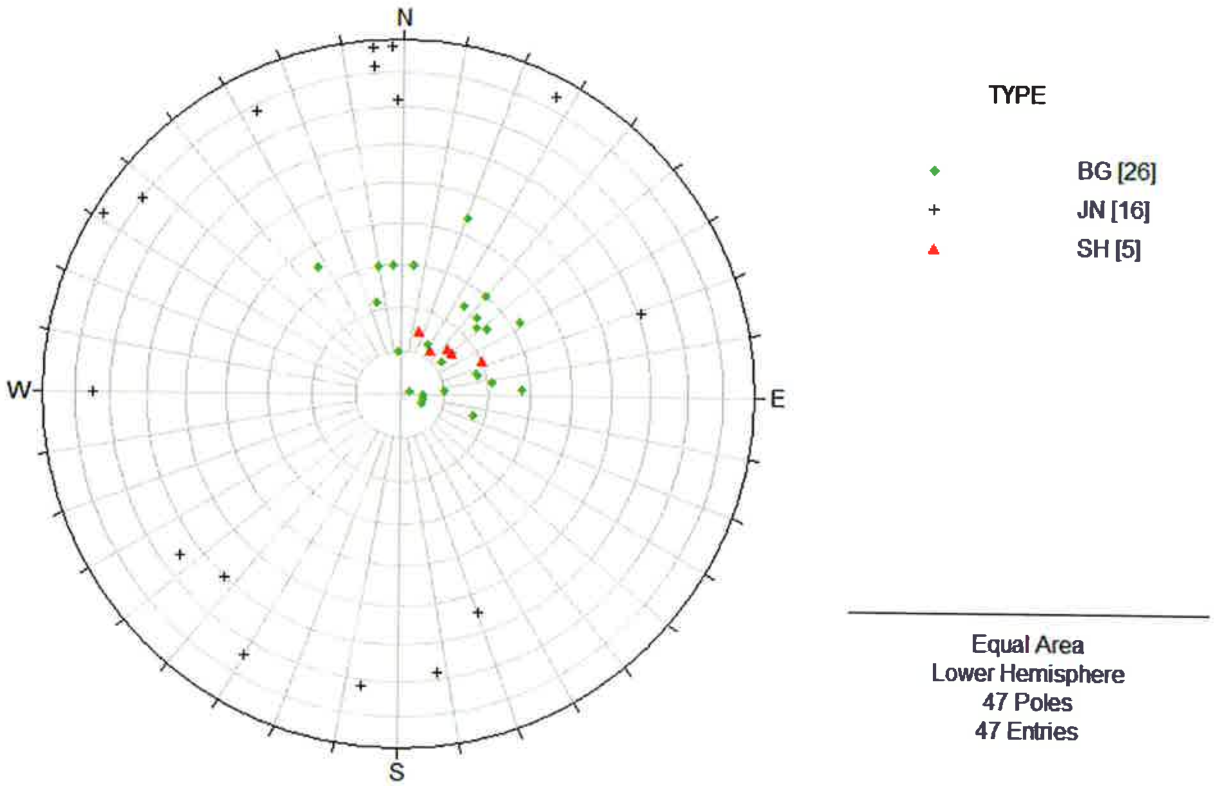
Equal Area
Lower Hemisphere
44 Poles
44 Entries

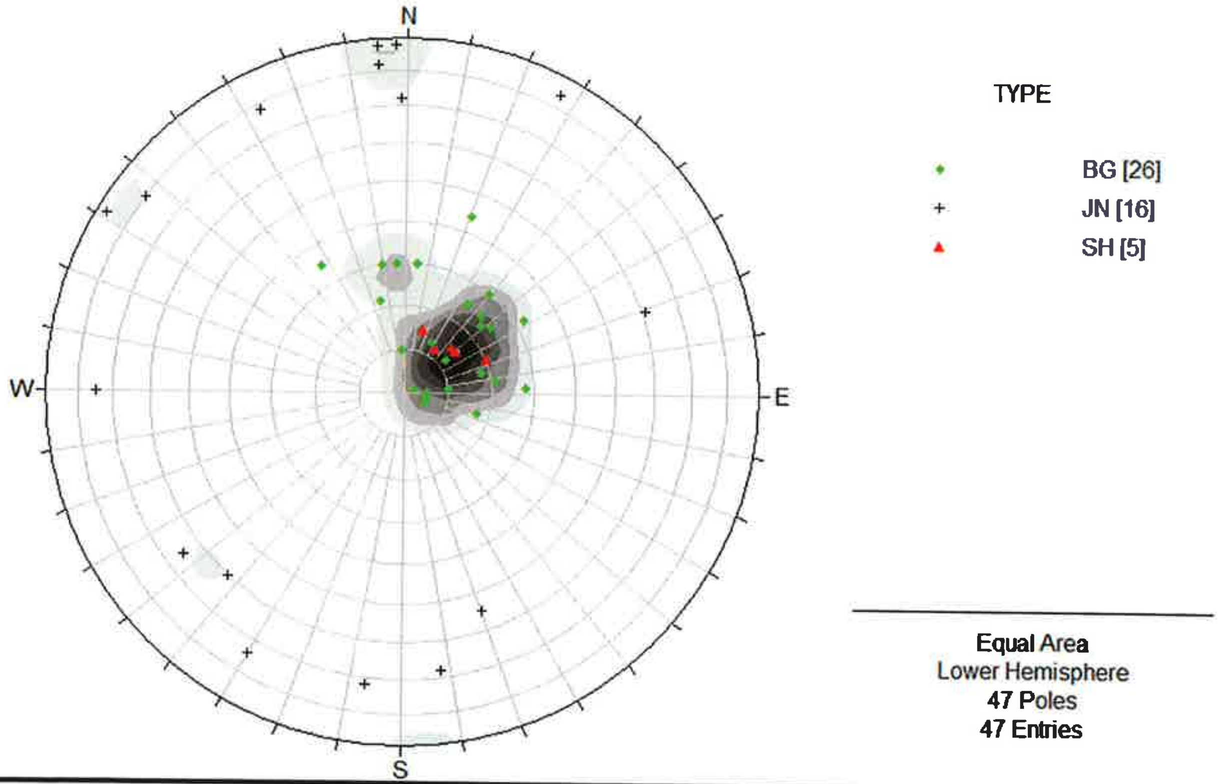
PSM **Pells Sullivan Meynink Pty Ltd**

Stereo Plot

Job no: 1271

Project:	TILLEGRA DAM	**North datum:**		**Processed by:**	TS
Domain:	2	**Date:**	18-9-08	**Defect type:**	ALL
Lithology:		**Traverse:**		**Length:**	
		Borehole:			





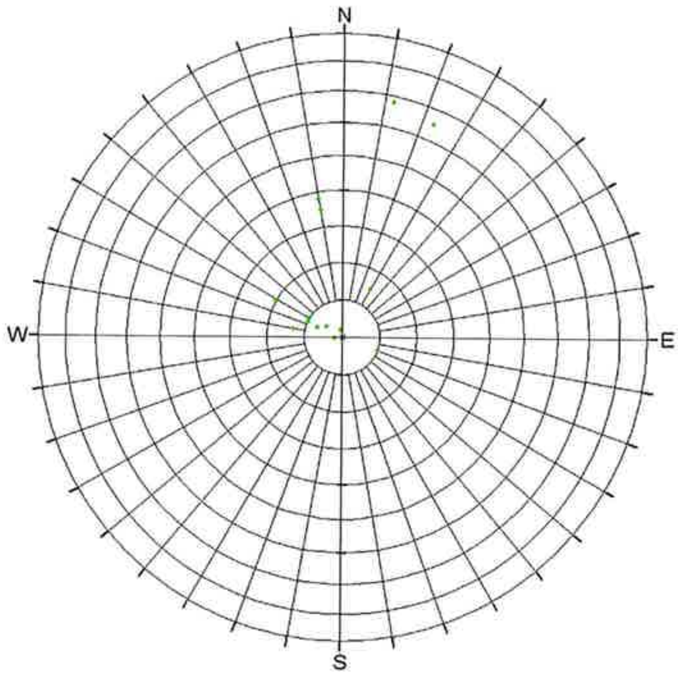
DOMAIN 2 ALL DATA\10/20/2008

Pells Sullivan Meynink Pty Ltd

Stereo Plot

Job no: 1271

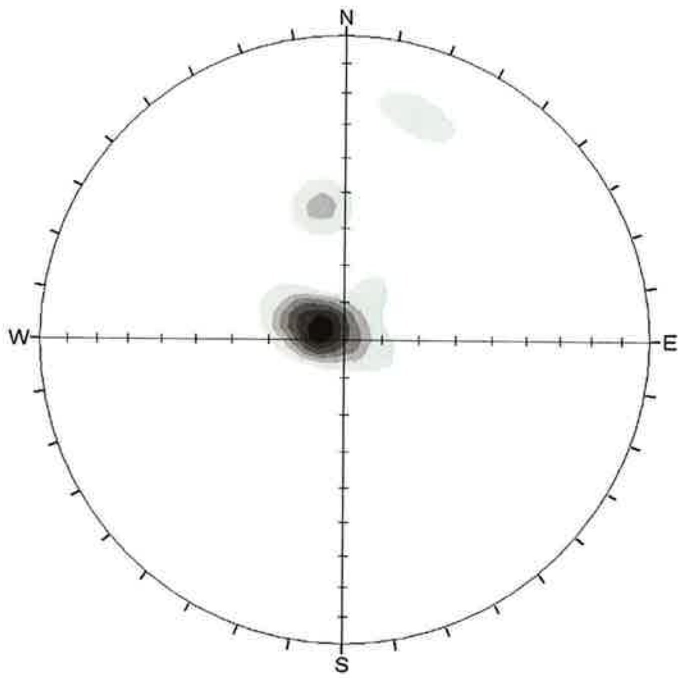
Project: Tillegra Dam	**North datum:** Grid	**Processed by:** James Bevis
Domain: 3	**Traverse:** N/A	**Date:** 26/08/2008
Lithology: N/A	**Borehole:** N/A	**Defect type:** Bedding
		Length: N/A



TYPE

BG [14]

Equal Area
Lower Hemisphere
14 Poles
14 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 6.50 %
6.50 ~ 13.00 %
13.00 ~ 19.50 %
19.50 ~ 26.00 %
26.00 ~ 32.50 %
32.50 ~ 39.00 %
39.00 ~ 45.50 %

No Bias Correction
Max. Conc. = 45.2252%

Equal Area
Lower Hemisphere
14 Poles
14 Entries

Pells Sullivan Meynink Pty Ltd

Stereo Plot

Job no: 1271

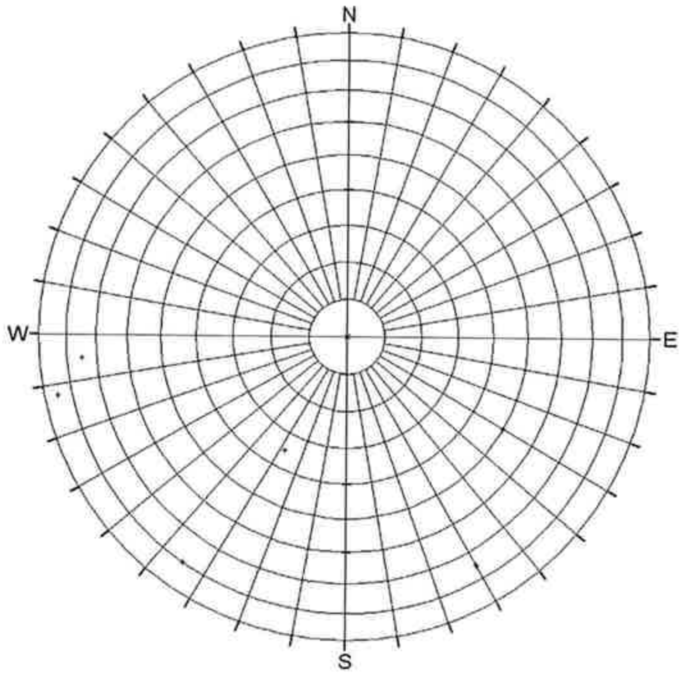
Project: Tillegra Dam

Domain: 3
Lithology: N/A

North datum: Grid

Traverse: N/A
Borehole: N/A

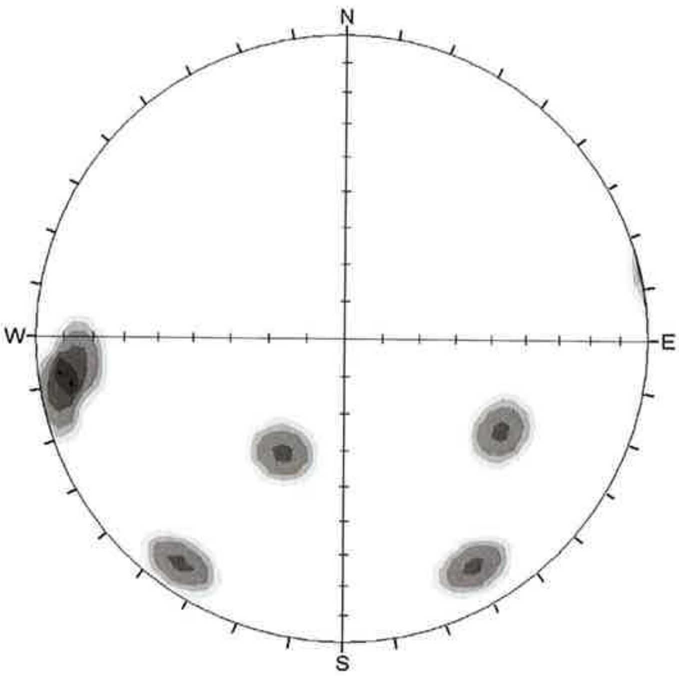
Processed by: James Bevis
Date: 26/08/2008
Defect type: Joints
Length: N/A



TYPE

JN [6]

Equal Area
Lower Hemisphere
6 Poles
6 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 4.00 %
4.00 ~ 8.00 %
8.00 ~ 12.00 %
12.00 ~ 16.00 %
16.00 ~ 20.00 %
20.00 ~ 24.00 %
24.00 ~ 28.00 %

No Bias Correction
Max. Conc. = 25.2780%

Equal Area
Lower Hemisphere
6 Poles
6 Entries

1271_Domain_3_Joints\10/20/2008

Pells Sullivan Meynink Pty Ltd

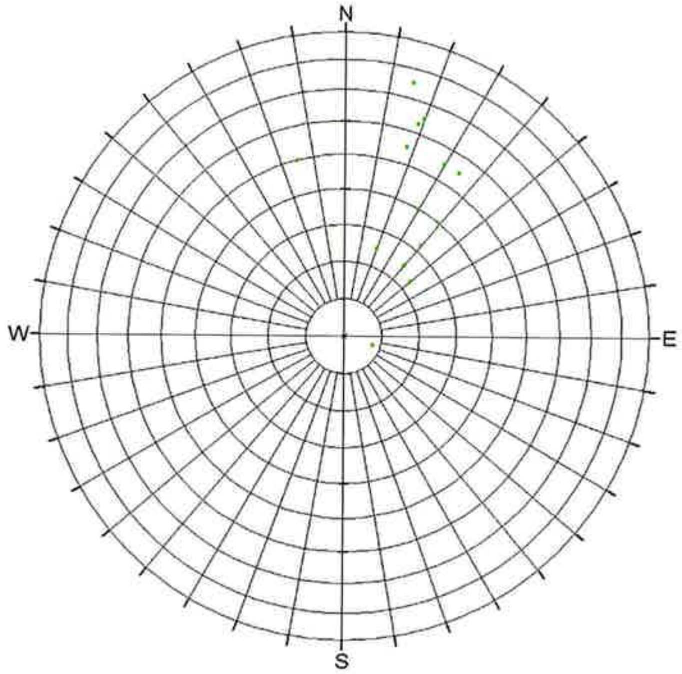
Stereo Plot

Job no: 1271

Project: Tillegra Dam
Domain: 4
Lithology: N/A

North datum: Grid
Traverse: N/A
Borehole: N/A

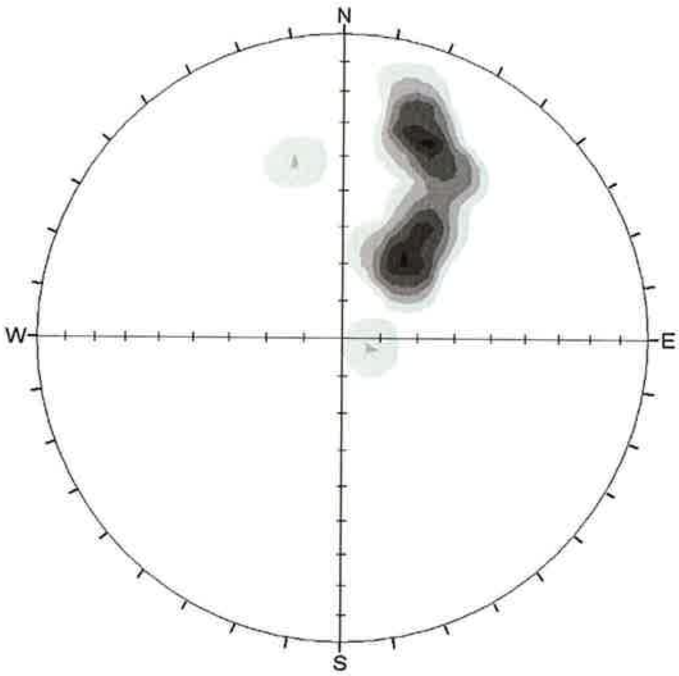
Processed by: James Bevis
Date: 26/08/2008
Defect type: Bedding
Length: N/A



TYPE

BG [14]

Equal Area
Lower Hemisphere
14 Poles
14 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 3.50 %
3.50 ~ 7.00 %
7.00 ~ 10.50 %
10.50 ~ 14.00 %
14.00 ~ 17.50 %
17.50 ~ 21.00 %
21.00 ~ 24.50 %

No Bias Correction
Max. Conc. = 22.8366%

Equal Area
Lower Hemisphere
14 Poles
14 Entries

Pells Sullivan Meynink Pty Ltd

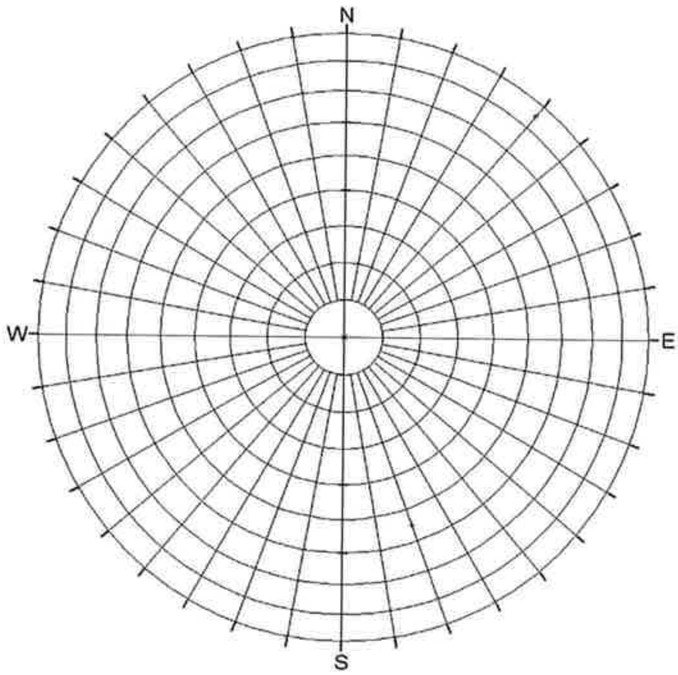
Stereo Plot

Job no: 1271

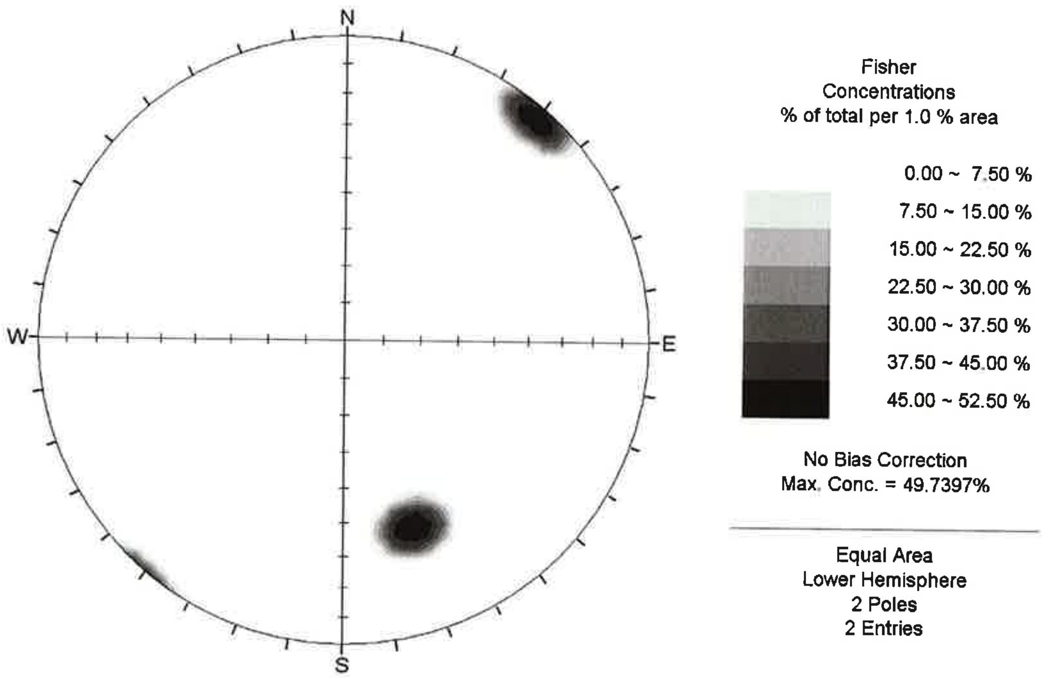
Project: Tillegra Dam
Domain: 4
Lithology: N/A

North datum: Grid
Traverse: N/A
Borehole: N/A

Processed by: James Bevis
Date: 26/08/2008
Defect type: Joints
Length: N/A



Equal Area
Lower Hemisphere
2 Poles
2 Entries



No Bias Correction
Max. Conc. = 49.7397%

Equal Area
Lower Hemisphere
2 Poles
2 Entries

PSM **Pells Sullivan Meynink Pty Ltd**

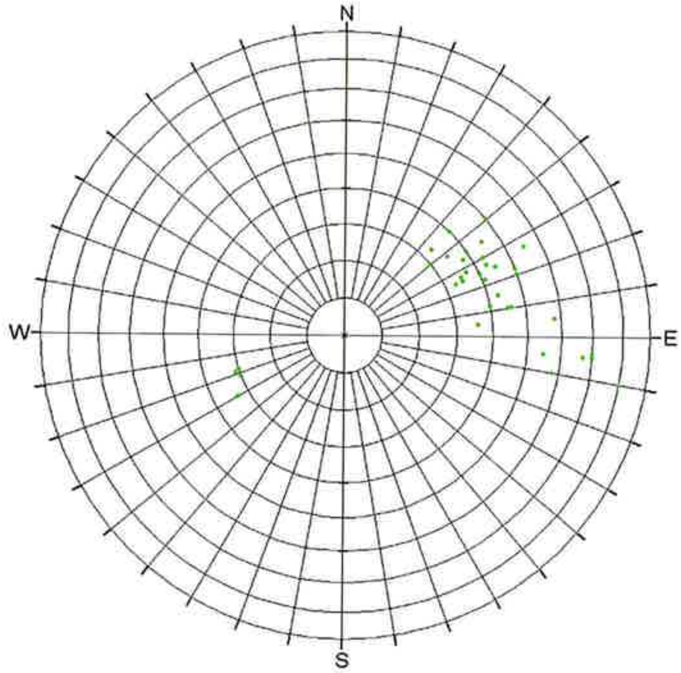
Stereo Plot

Job no: 1271

Project: Tillegra Dam
Domain: 5
Lithology: N/A

North datum: Grid
Traverse: N/A
Borehole: N/A

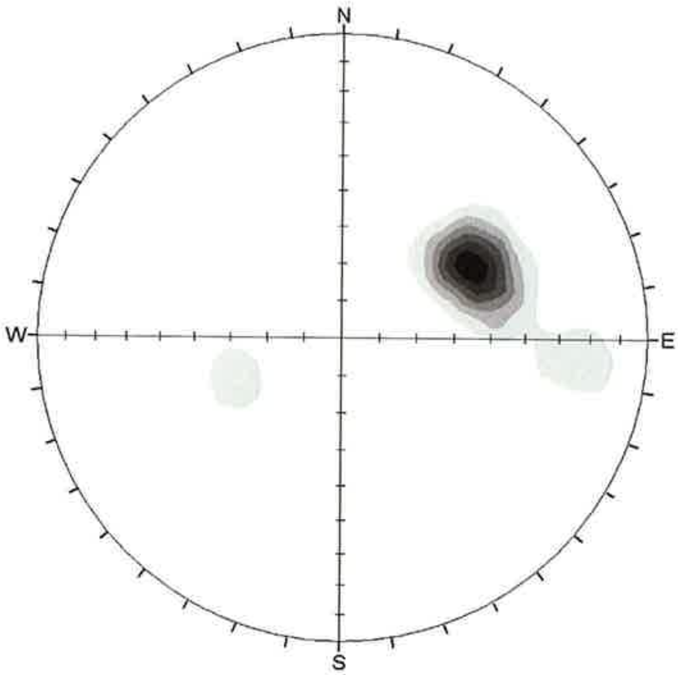
Processed by: James Bevis
Date: 26/08/2008
Defect type: Bedding
Length: N/A



TYPE

BG [39]

Equal Area
Lower Hemisphere
39 Poles
39 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 5.00 %
5.00 ~ 10.00 %
10.00 ~ 15.00 %
15.00 ~ 20.00 %
20.00 ~ 25.00 %
25.00 ~ 30.00 %
30.00 ~ 35.00 %

No Bias Correction
Max. Conc. = 34.6245%

Equal Area
Lower Hemisphere
39 Poles
39 Entries

Pells Sullivan Meynink Pty Ltd

Stereo Plot

Job no: 1271

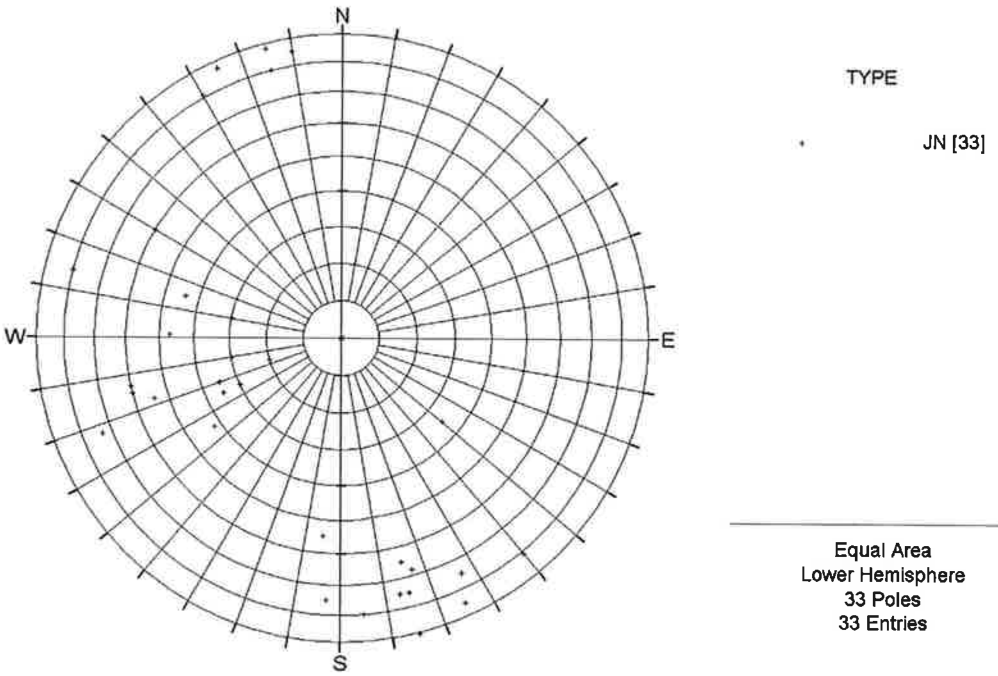
Project: Tillegra Dam

Domain: 5
Lithology: N/A

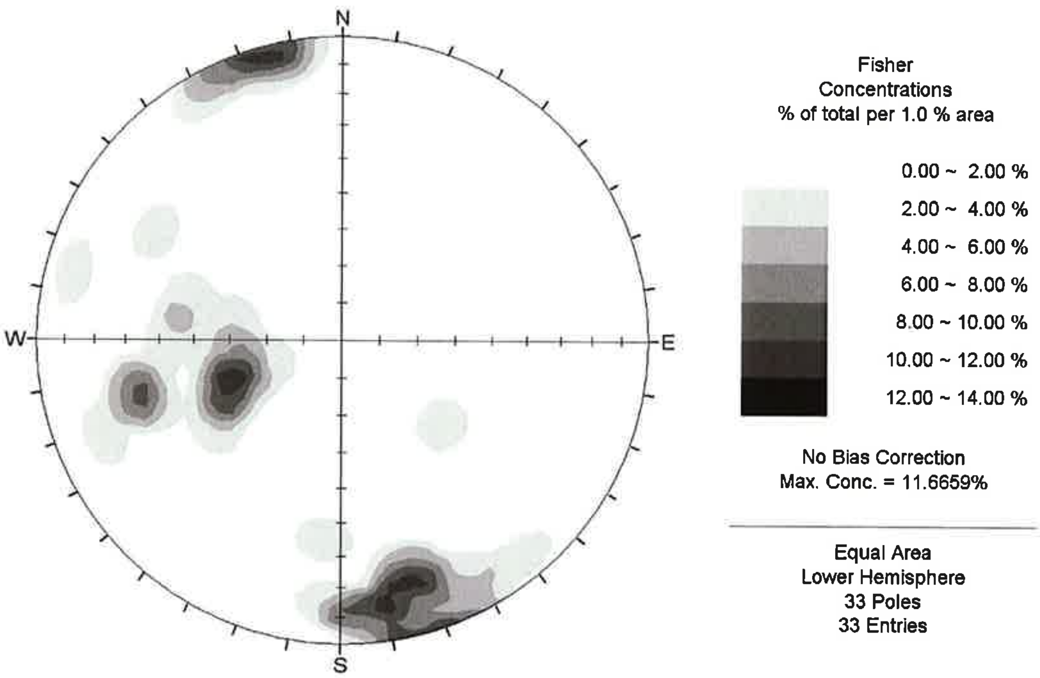
North datum: Grid

Traverse: N/A
Borehole: N/A

Processed by: James Bevis
Date: 26/08/2008
Defect type: Joints
Length: N/A



Equal Area
Lower Hemisphere
33 Poles
33 Entries



No Bias Correction
Max. Conc. = 11.6659%

Equal Area
Lower Hemisphere
33 Poles
33 Entries

Pells Sullivan Meynink Pty Ltd

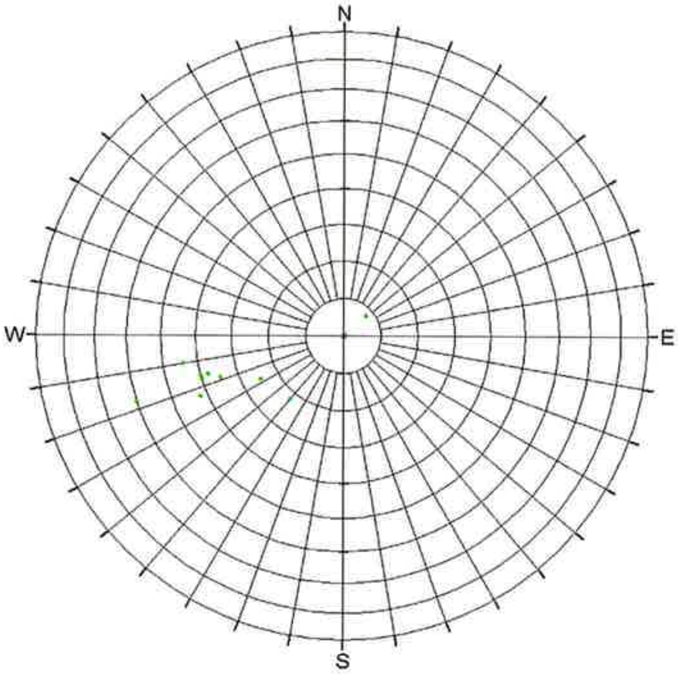
Stereo Plot

Job no: 1271

Project: Tillegra Dam
Domain: 6
Lithology: N/A

North datum: Grid
Traverse: N/A
Borehole: N/A

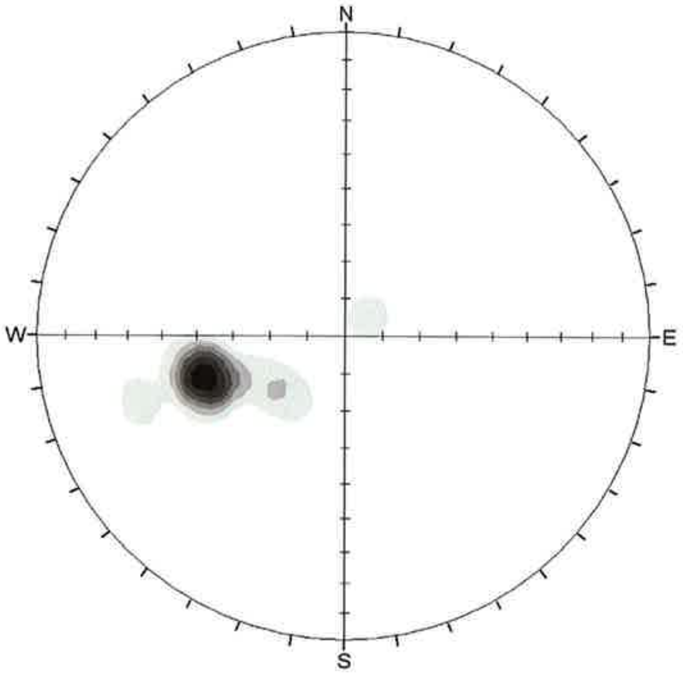
Processed by: James Bevis
Date: 26/08/2008
Defect type: Bedding
Length: N/A



TYPE

BG [10]

Equal Area
Lower Hemisphere
10 Poles
10 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 8.00 %
8.00 ~ 16.00 %
16.00 ~ 24.00 %
24.00 ~ 32.00 %
32.00 ~ 40.00 %
40.00 ~ 48.00 %
48.00 ~ 56.00 %

No Bias Correction
Max. Conc. = 54.0634%

Equal Area
Lower Hemisphere
10 Poles
10 Entries

Pells Sullivan Meynink Pty Ltd

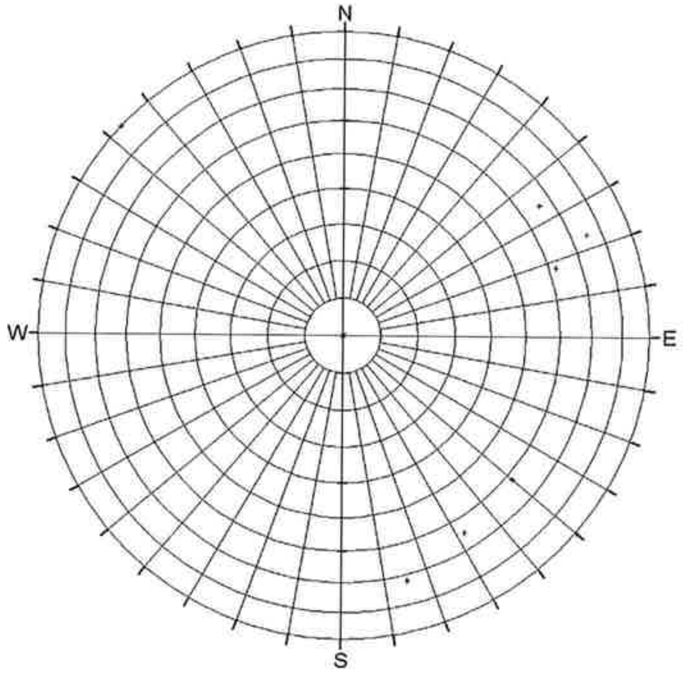
Stereo Plot

Job no: 1271

Project: Tillegra Dam
Domain: 6
Lithology: N/A

North datum: Grid
Traverse: N/A
Borehole: N/A

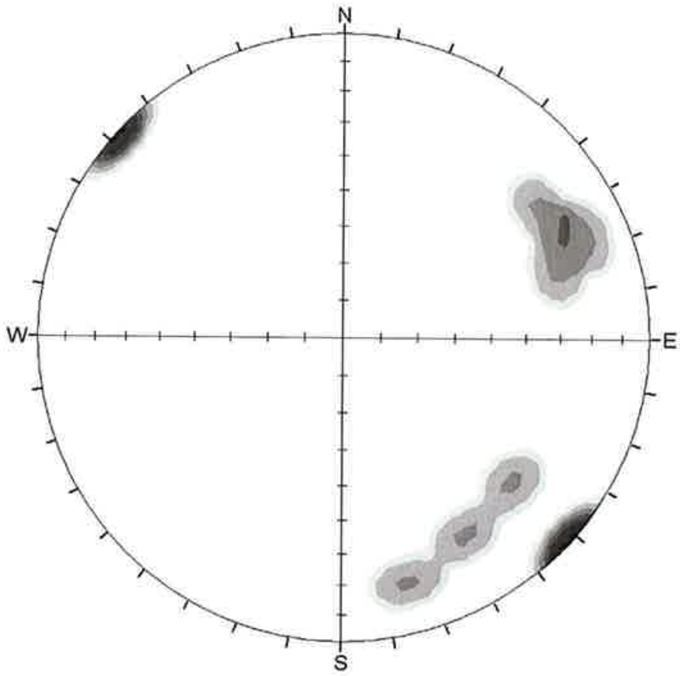
Processed by: James Bevis
Date: 26/08/2008
Defect type: Joints
Length: N/A



TYPE

JN [8]

Equal Area
Lower Hemisphere
8 Poles
8 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 4.00 %
4.00 ~ 8.00 %
8.00 ~ 12.00 %
12.00 ~ 16.00 %
16.00 ~ 20.00 %
20.00 ~ 24.00 %
24.00 ~ 28.00 %

No Bias Correction
Max. Conc. = 24.5290%

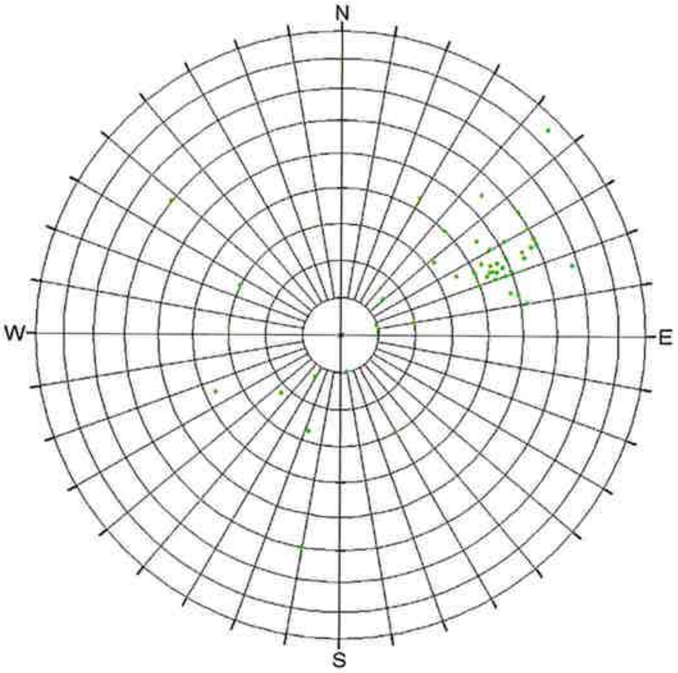
Equal Area
Lower Hemisphere
8 Poles
8 Entries

Pells Sullivan Meynink Pty Ltd

Stereo Plot

Job no: 1271

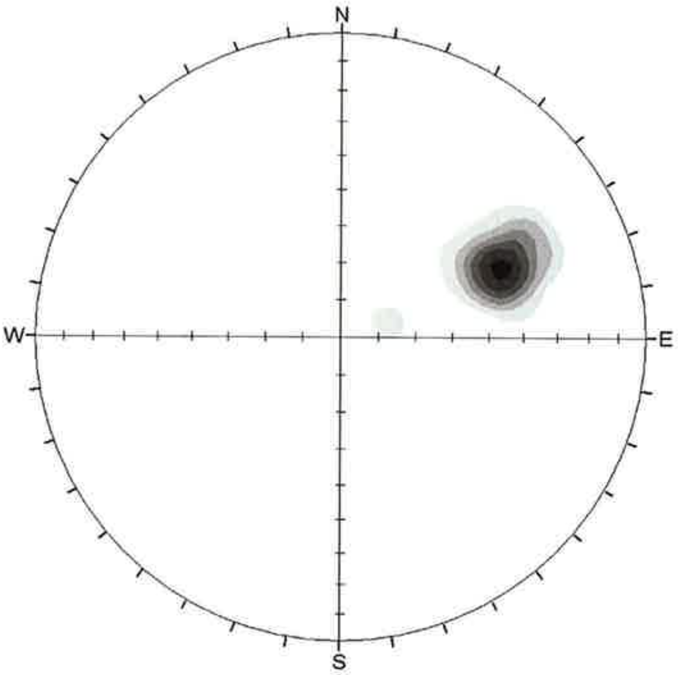
Project: Tillegra Dam	**North datum:** Grid	**Processed by:** James Bevis
Domain: 7	**Traverse:** N/A	**Date:** 26/08/2008
Lithology: N/A	**Borehole:** N/A	**Defect type:** Bedding
		Length: N/A



TYPE

BG [48]

Equal Area
Lower Hemisphere
48 Poles
48 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 5.50 %
5.50 ~ 11.00 %
11.00 ~ 16.50 %
16.50 ~ 22.00 %
22.00 ~ 27.50 %
27.50 ~ 33.00 %
33.00 ~ 38.50 %

No Bias Correction
Max. Conc. = 35.1342%

Equal Area
Lower Hemisphere
48 Poles
48 Entries

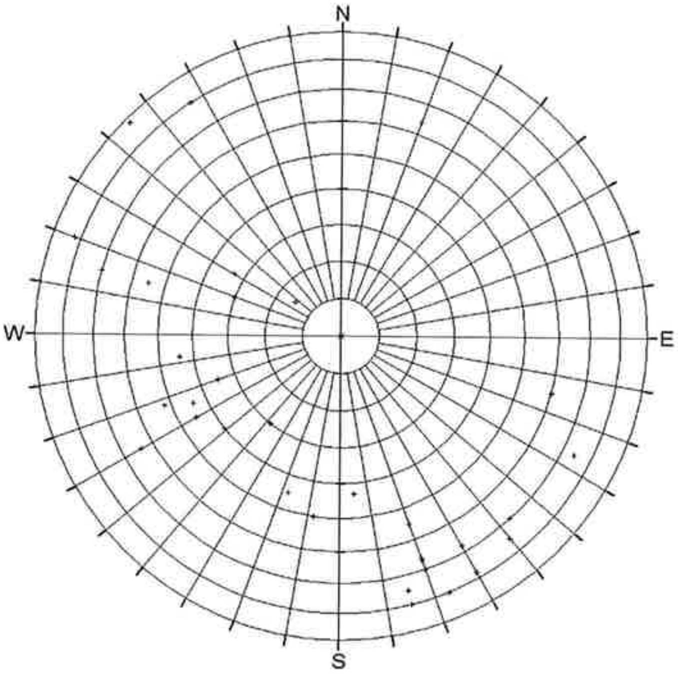
1271_Domain_7_Bedding\10/20/2008

PSM **Pells Sullivan Meynink Pty Ltd**

Stereo Plot

Job no: 1271

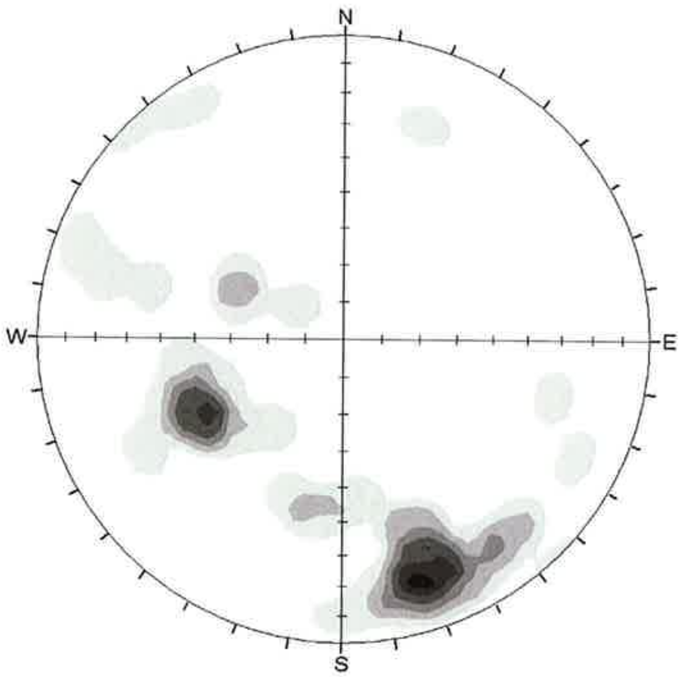
Project: Tillegra Dam	**North datum:** Grid	**Processed by:** James Bevis
Domain: 7	**Traverse:** N/A	**Date:** 26/08/2008
Lithology: N/A	**Borehole:** N/A	**Defect type:** Joints
		Length: N/A



TYPE

JN [37]

Equal Area
Lower Hemisphere
37 Poles
37 Entries



Fisher
Concentrations
% of total per 1.0 % area

0.00 ~ 2.00 %
2.00 ~ 4.00 %
4.00 ~ 6.00 %
6.00 ~ 8.00 %
8.00 ~ 10.00 %
10.00 ~ 12.00 %
12.00 ~ 14.00 %

No Bias Correction
Max. Conc. = 13.0357%

Equal Area
Lower Hemisphere
37 Poles
37 Entries

1271_Domain_7_Joints\10/20/2008

APPENDIX C

SHEAR STRENGTH ESTIMATES LOWER SLIDE PLANE

PSM1271
Clay Friction Angle ↔Mineralogy

Montmorilonite	40%		(smectite group)
Feldspar	31%	]	sand (noncohesive) portion 53%
Quartz	22%	]	
Kaolinite	7%		

No effect ? No information on how strength affected using the graphs available.

Binod Tiwari, 2005 – use smectite – quartz group

LL ø' 24 → 28
18 → 25
18 → 25

Binod Tiwari, 2005 –smectite – quartz

PI ø 24 → 26
16 → 23
15 → 22

Voight, 1973 – montinillontic clay-shales

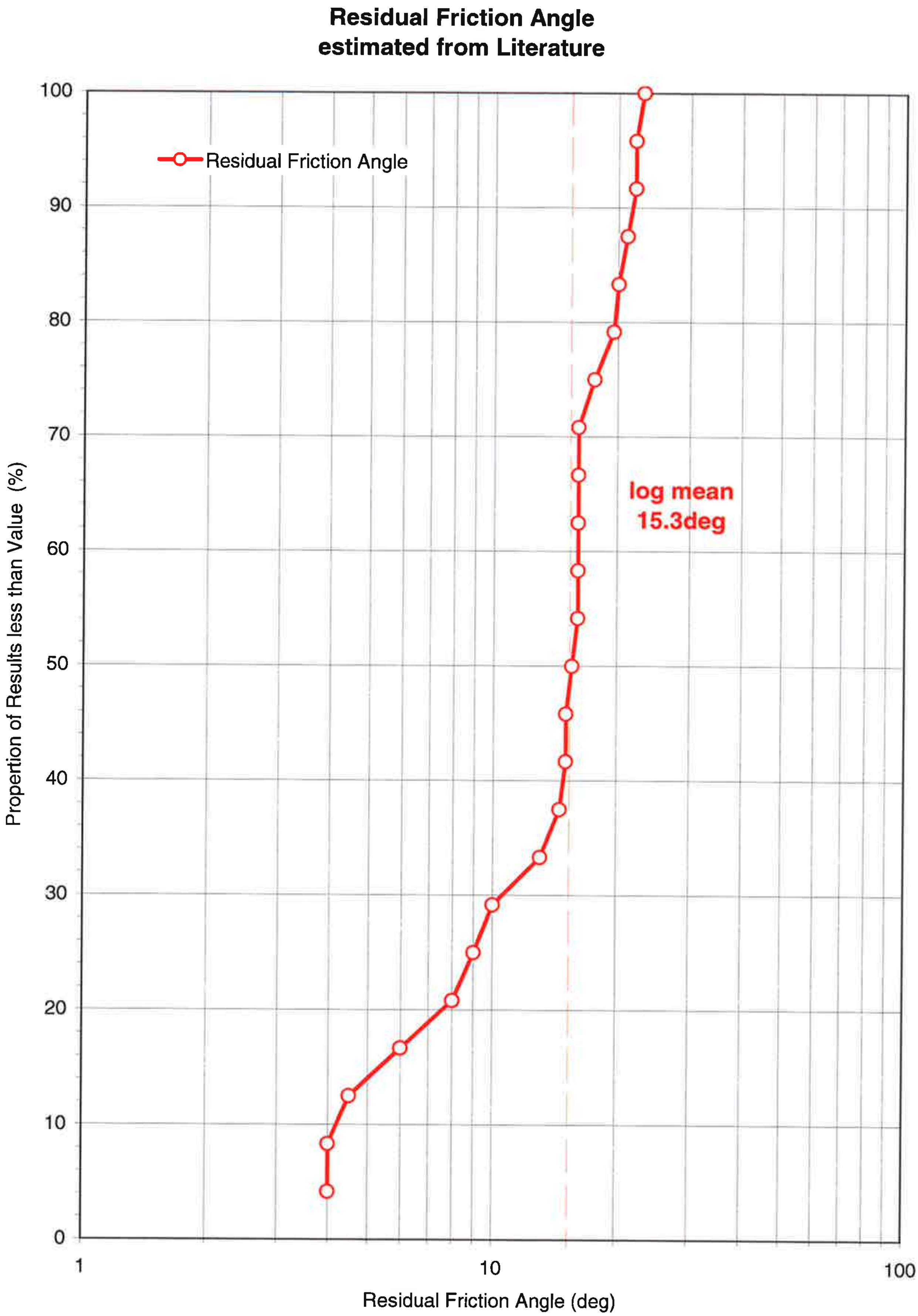
ø same

Binod Tiwari, 2005 –smectite – quartz

CF ø: 12 → 6
11 → 4
11 → 4

End Result → ø' (log mean) = 15.3º

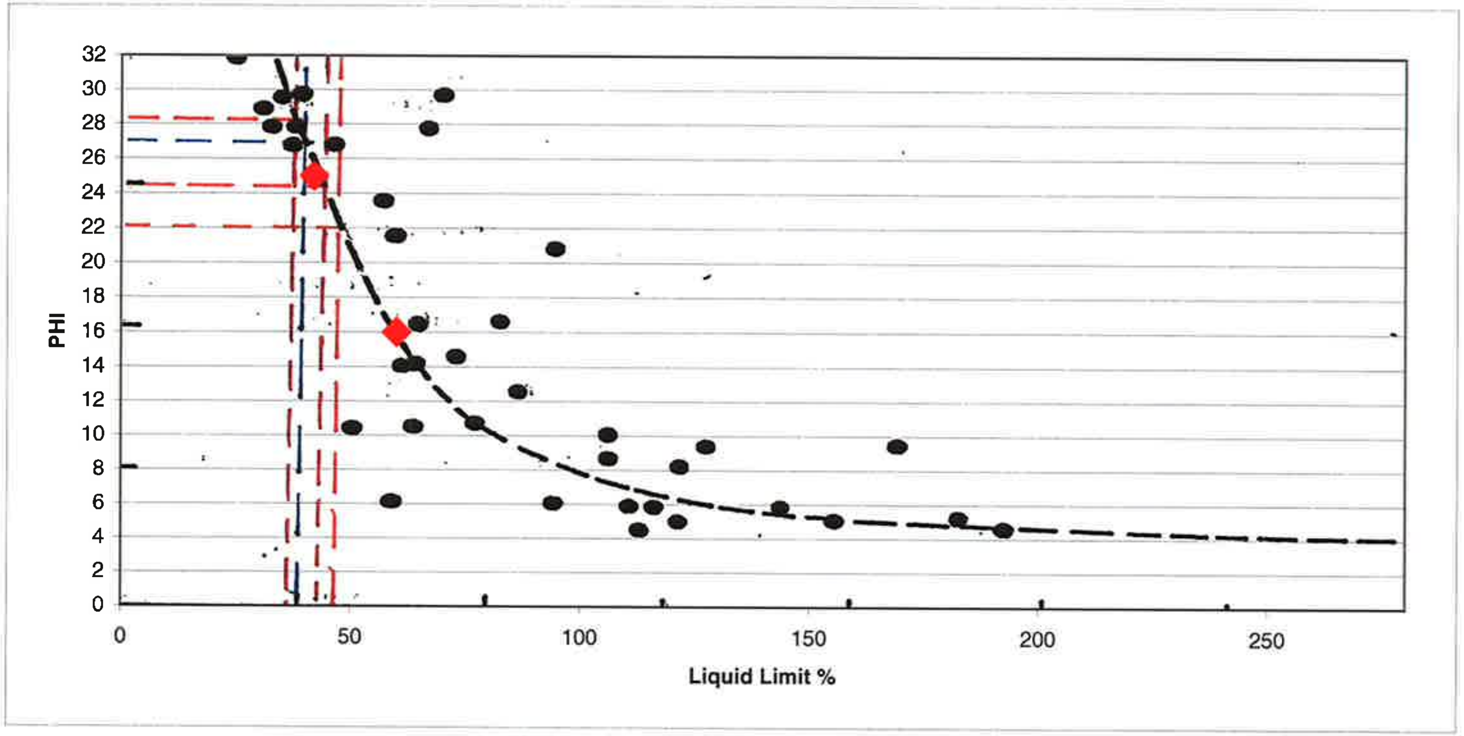
23/10/08



U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\Residual Strength - LAB\Stat new Chart 4

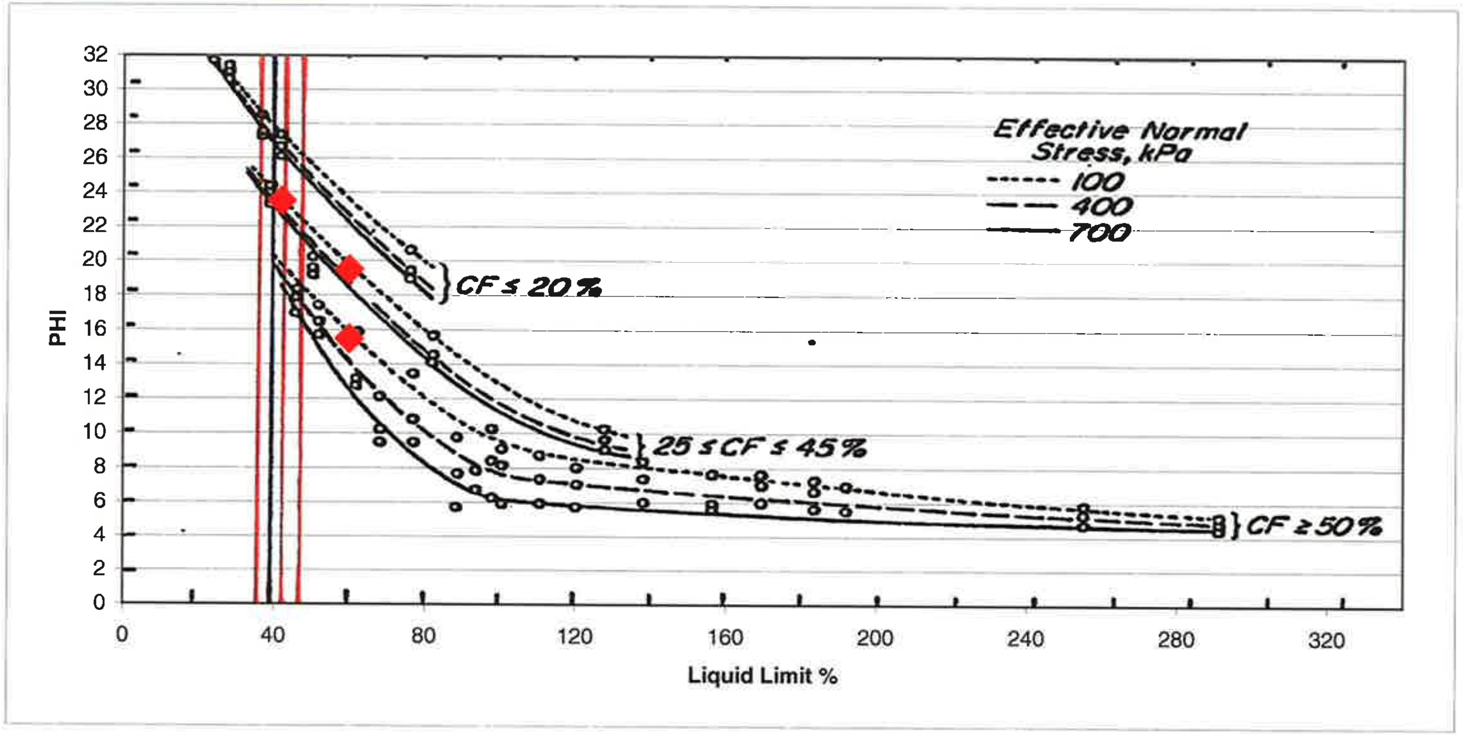
LIQUID LIMIT VS PHI

Mesri & Cefeda Diaz, 1986



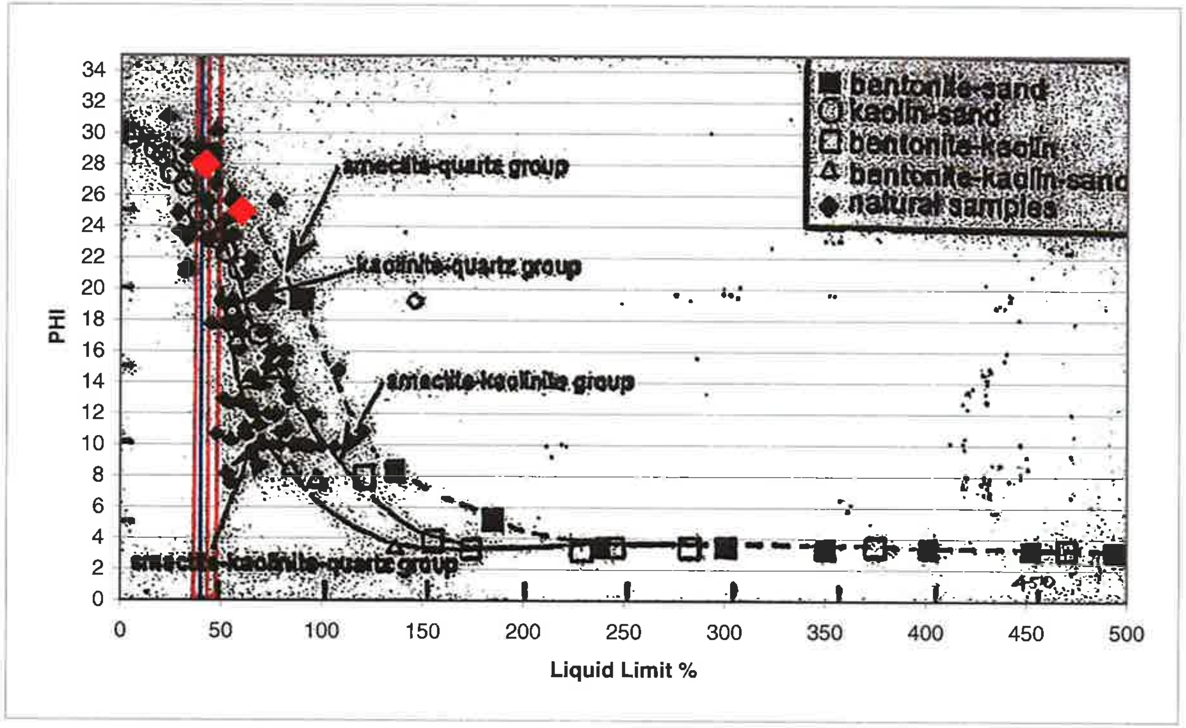
U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\Residual Strength - LAB

Stark and Eid, 1994



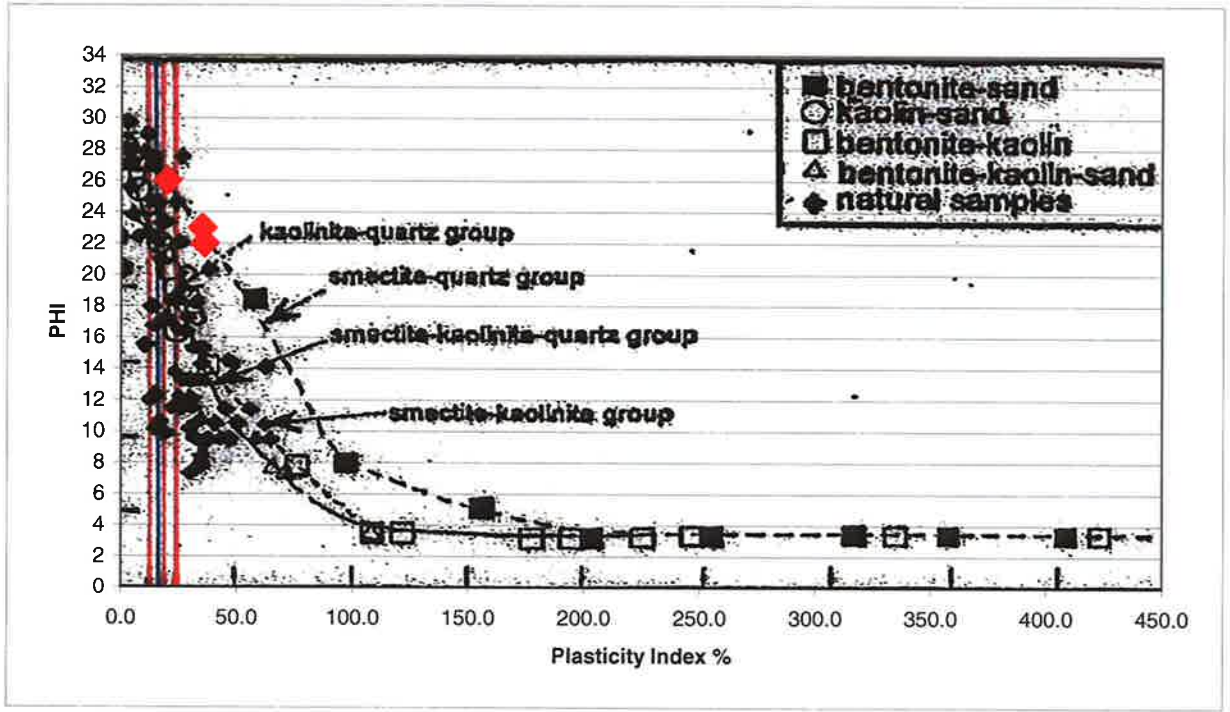
U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\Residual Strength - LAB

Binod Tiwari, 2005

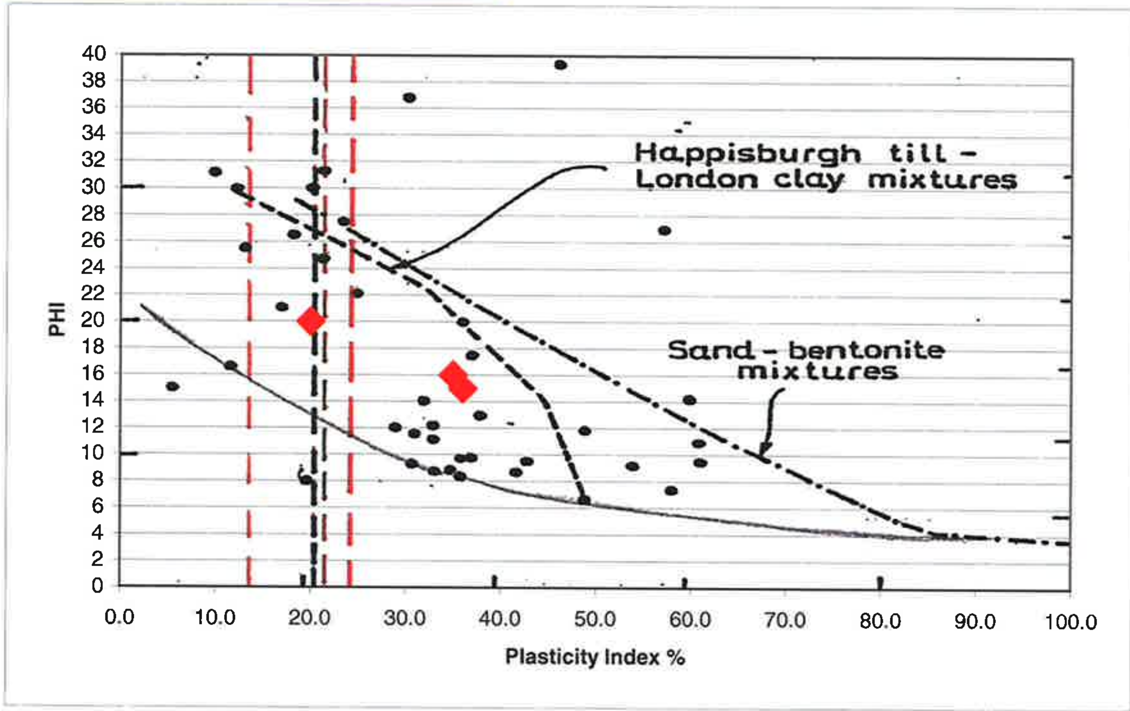


PLASTICITY INDEX VS PHI

Binod Tiwari, 2005

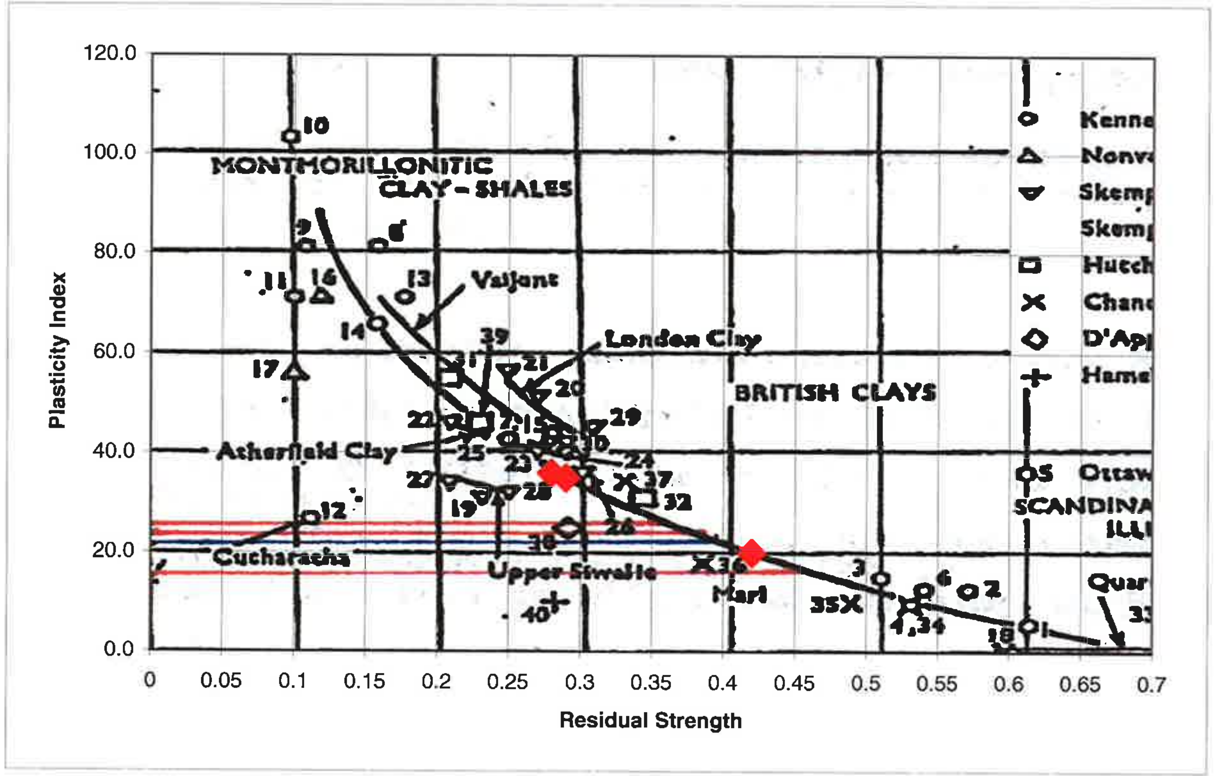


Lupini et al, 1981



U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\Residual Strength - LAB

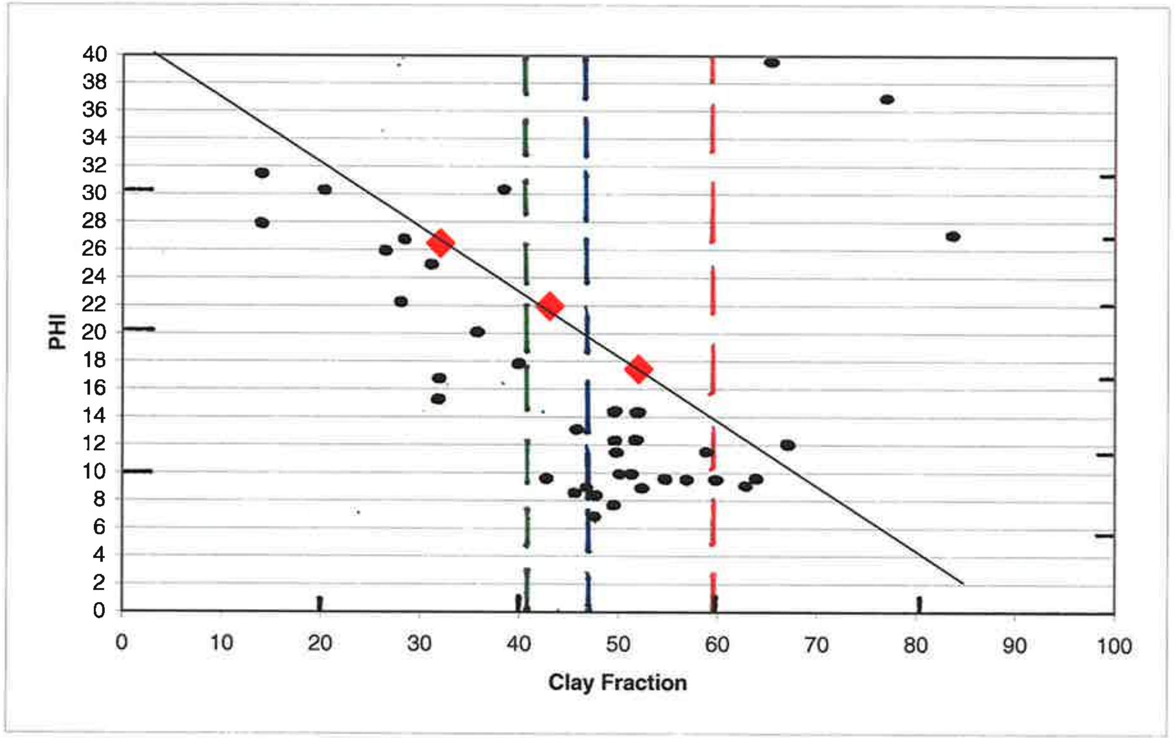
Voight, 1973



Plasticity Index
Residual Strength
MONTMORILLONITIC CLAY - SHALES
Vaijont
London Clay
BRITISH CLAYS
Atherfield Clay
Cucharacha
Upper Siwalic
Marl

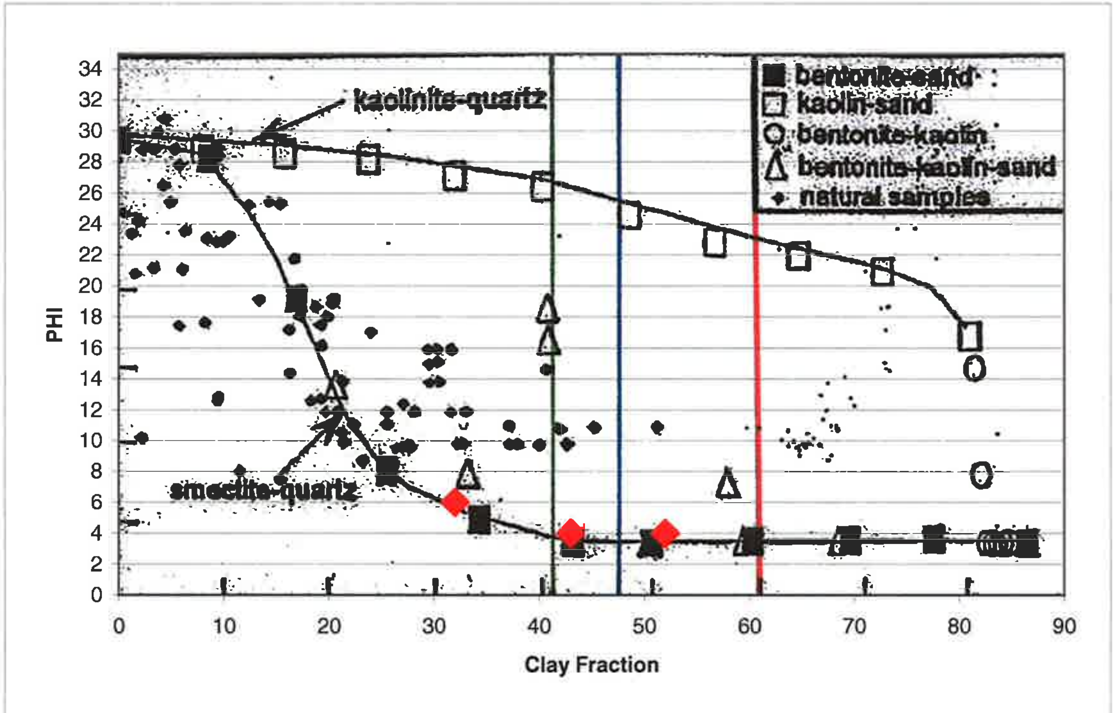
CLAY FRACTION VS PHI

Upper Bound Lupini, 1981



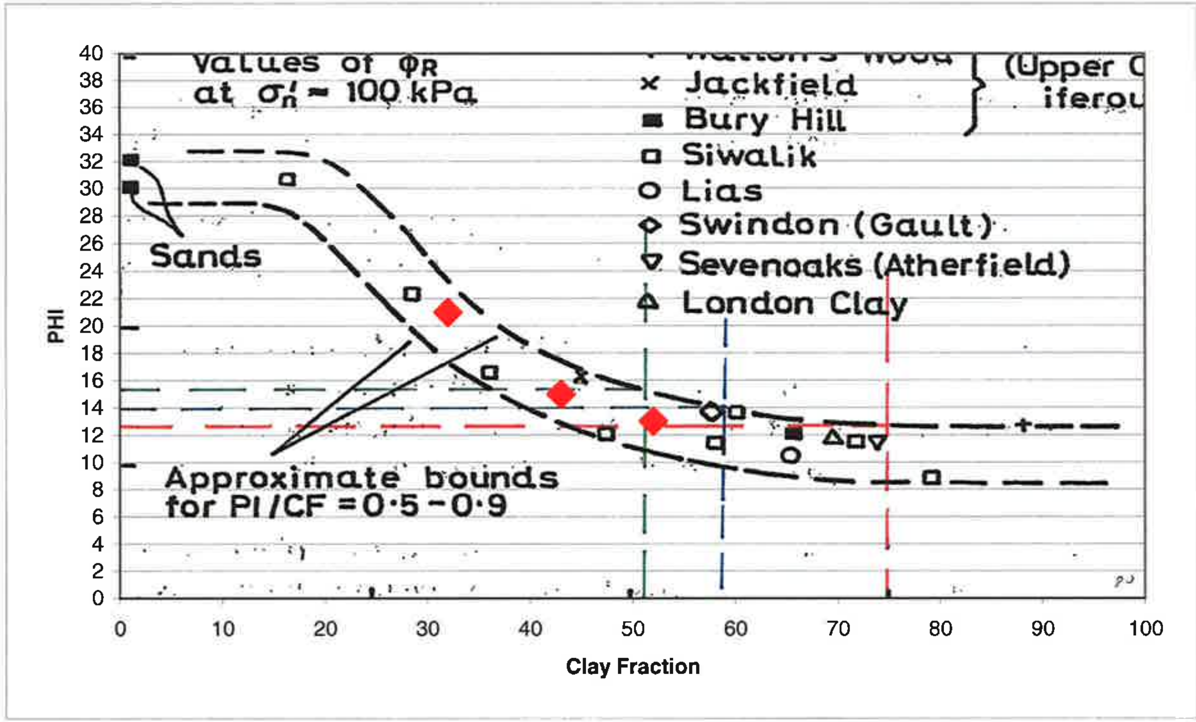
U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\Residual Strength - LAB

Binod Tiwari, 2005



U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\Residual Strength - LAB

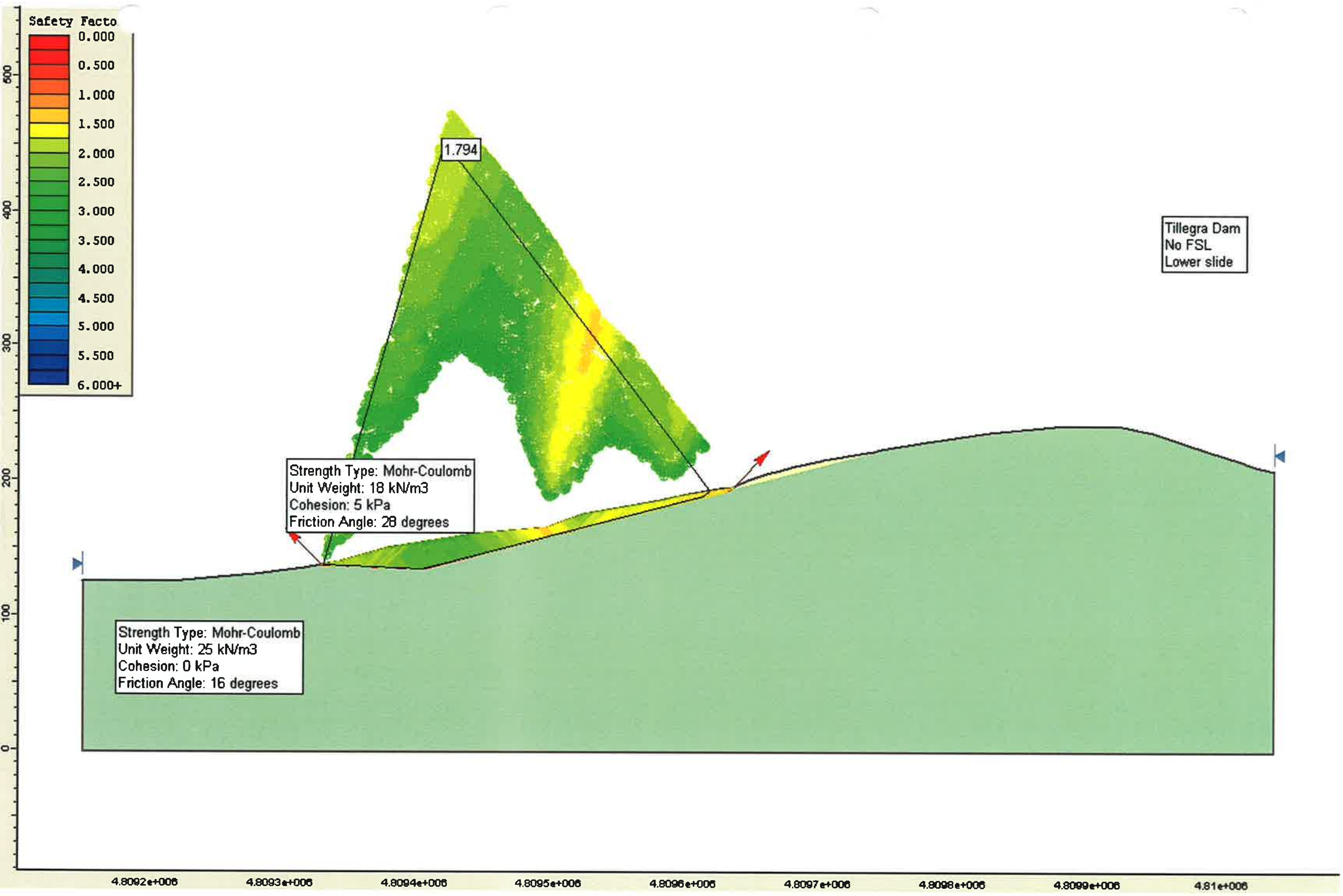
Skempton, 1985



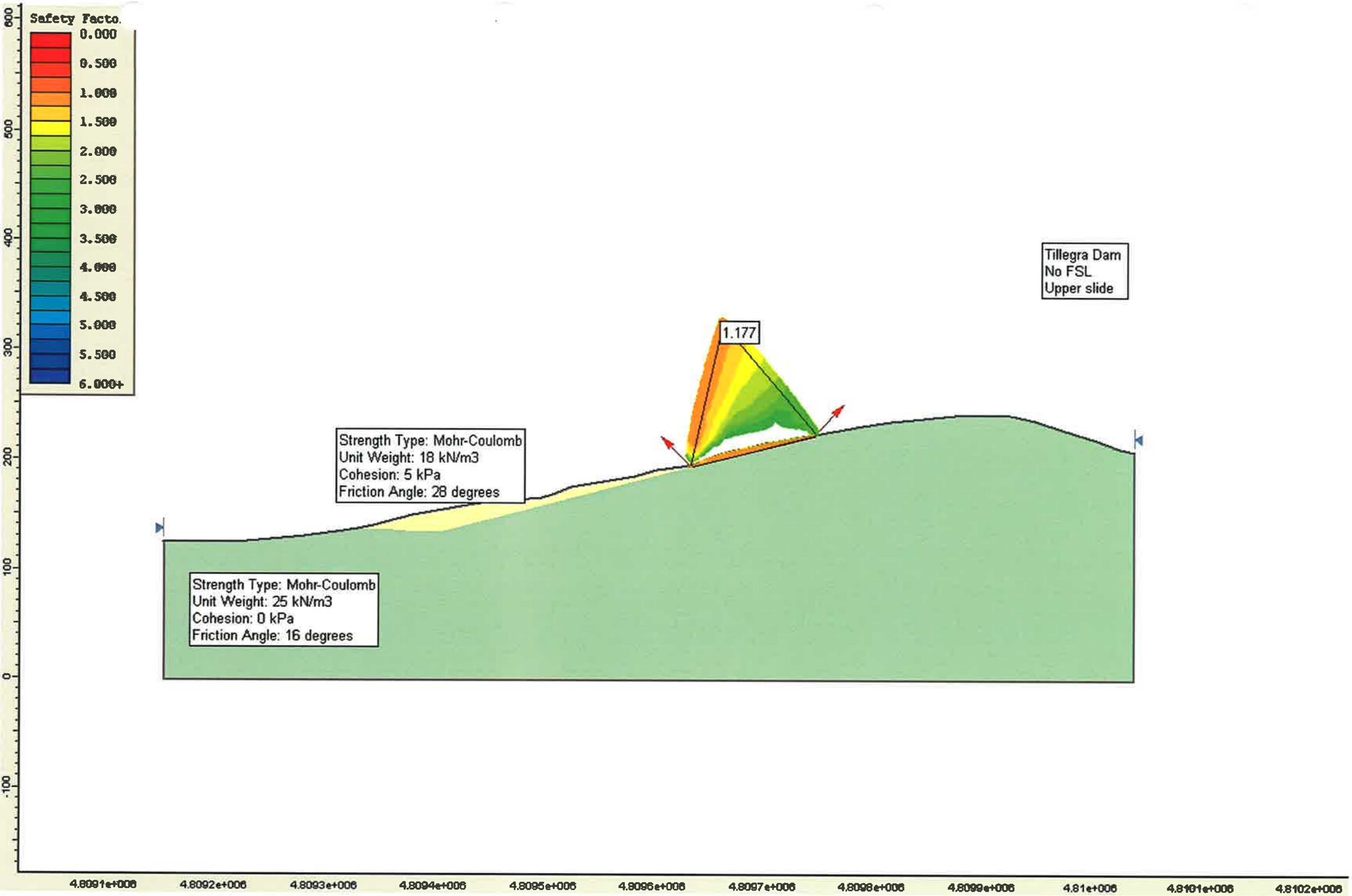
U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\Residual Strength - LAB

APPENDIX D

STABILITY ANALYSIS RESULTS, SLIDE 2A



Safety Facto
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.794
Tillegra Dam
No FSL
Lower slide
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006



Safety Facto.
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Tillegra Dam
No FSL
Upper slide
1.177
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
600
500
400
300
200
100
0
-100
4.8091e+006
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
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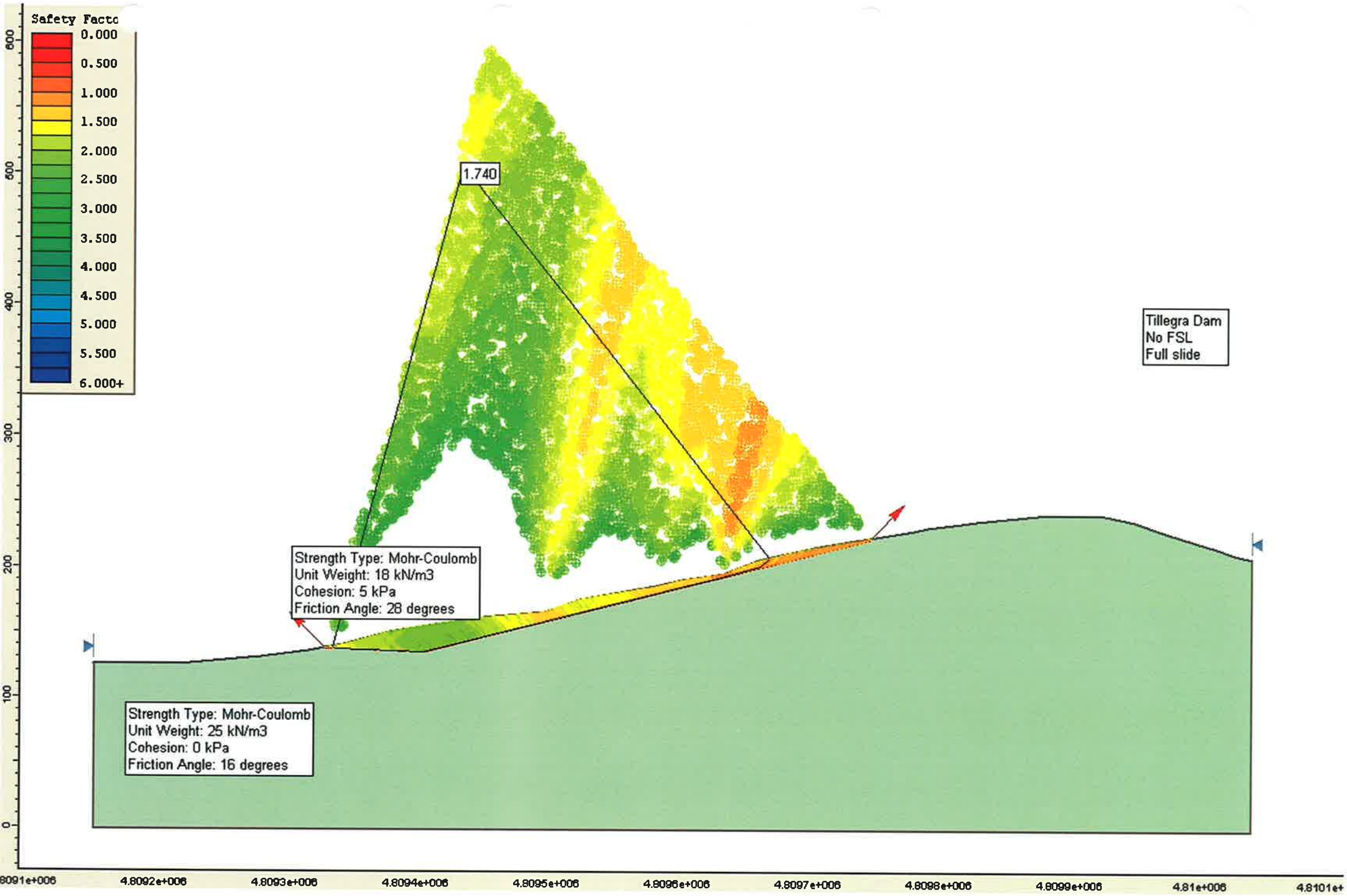
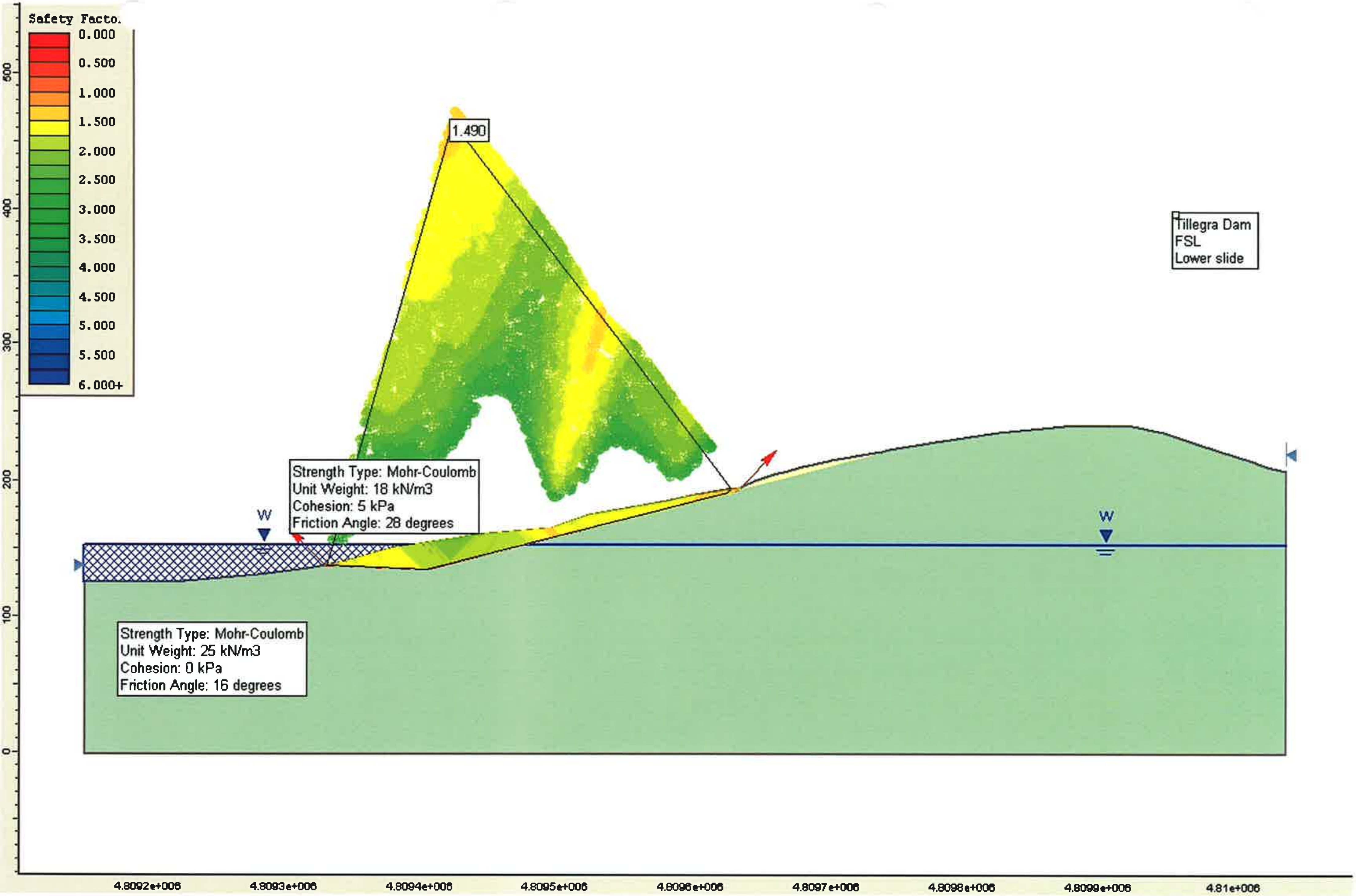
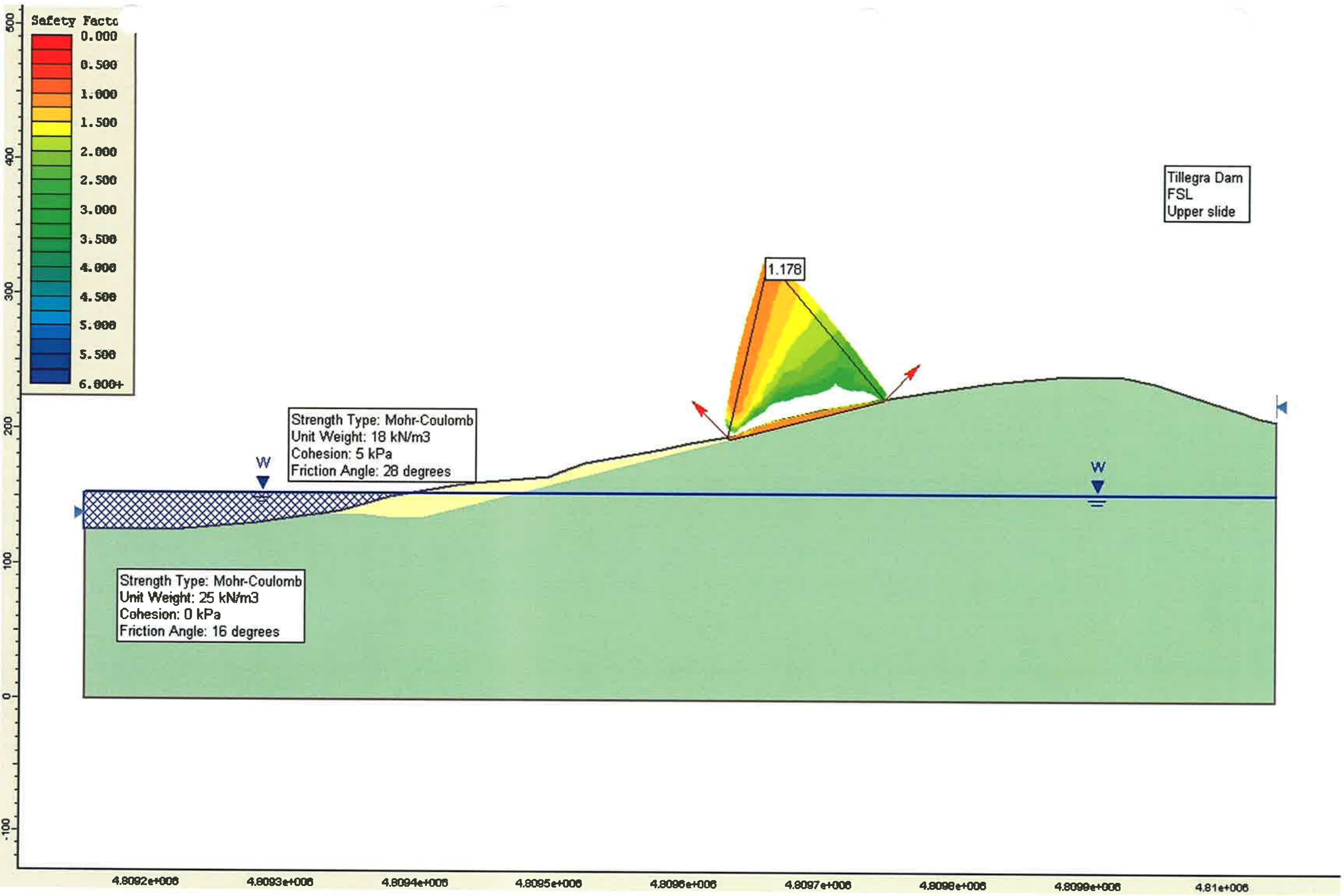
4.81e+006
4.8101e+006
4.8102e+006



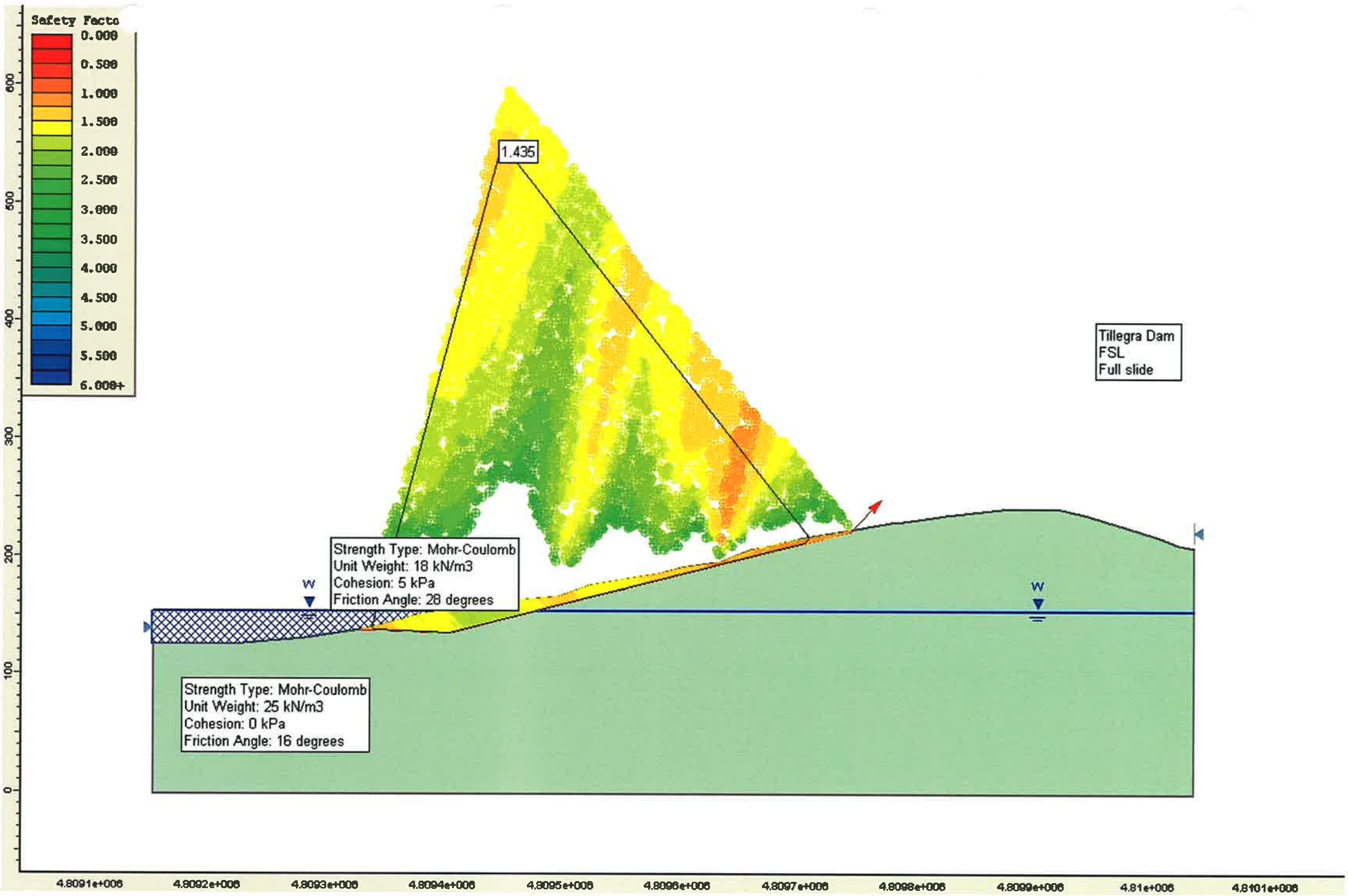
Safety Facto
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.740
Tillegra Dam
No FSL
Full slide
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
600
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006



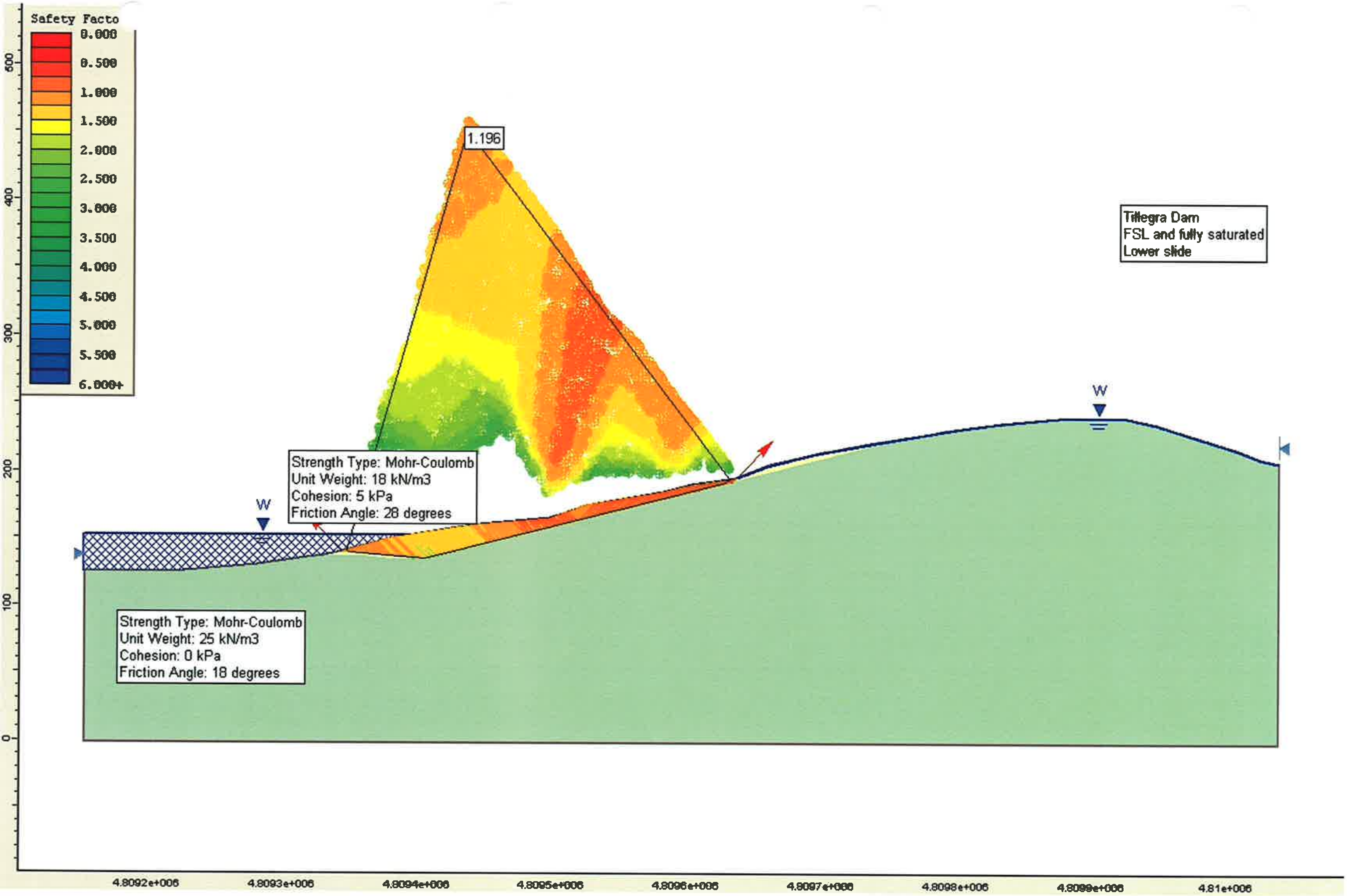
Safety Facto.
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.490
Tillegra Dam
FSL
Lower slide
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006



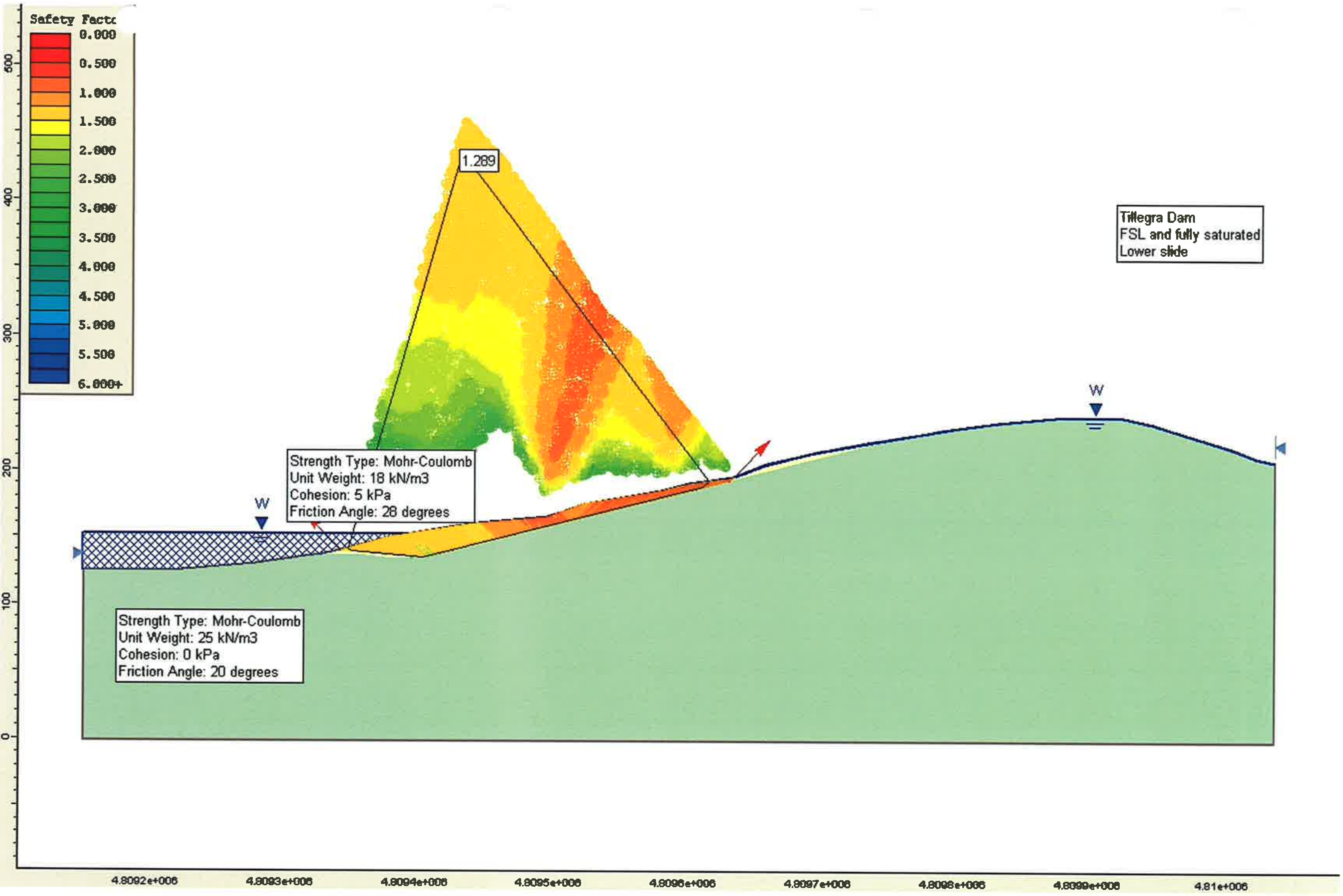
Safety Facto
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Tillegra Dam
FSL
Upper slide
1.178
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
-100
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006



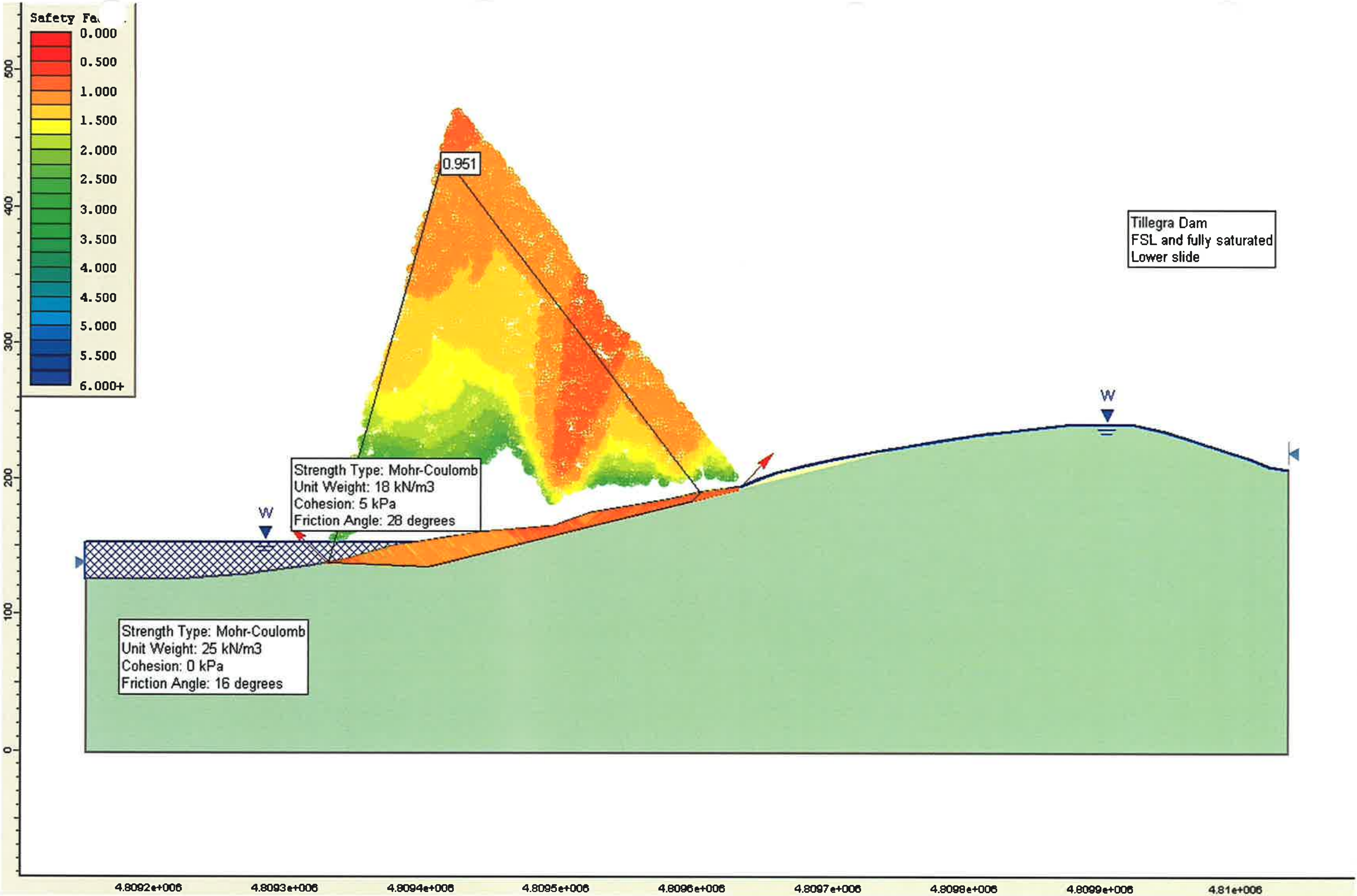
Safety Facto
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.435
Tillegra Dam
FSL
Full slide
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
W
W
600
500
400
300
200
100
0
4.8091e+006
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006
4.8101e+006

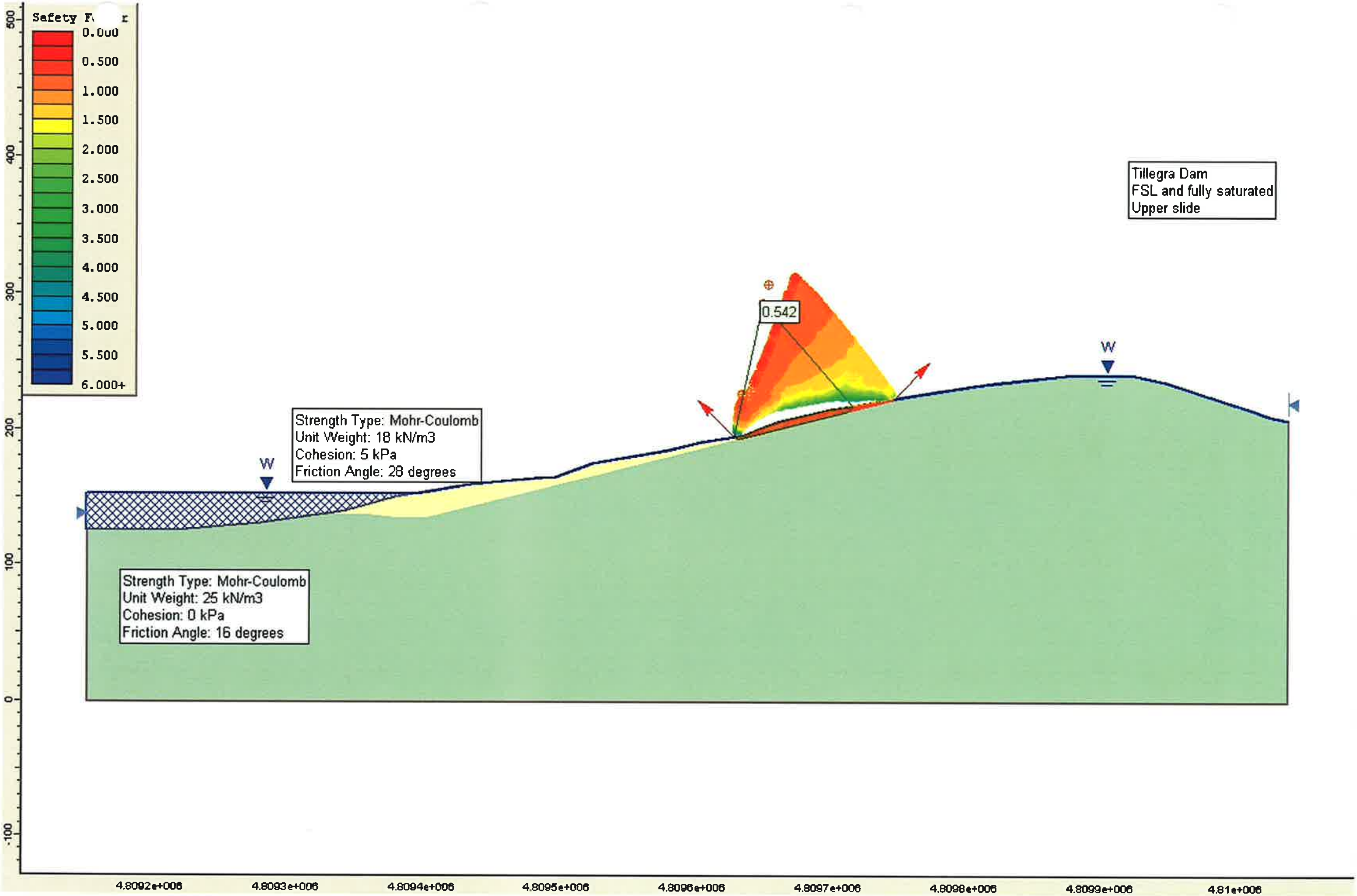


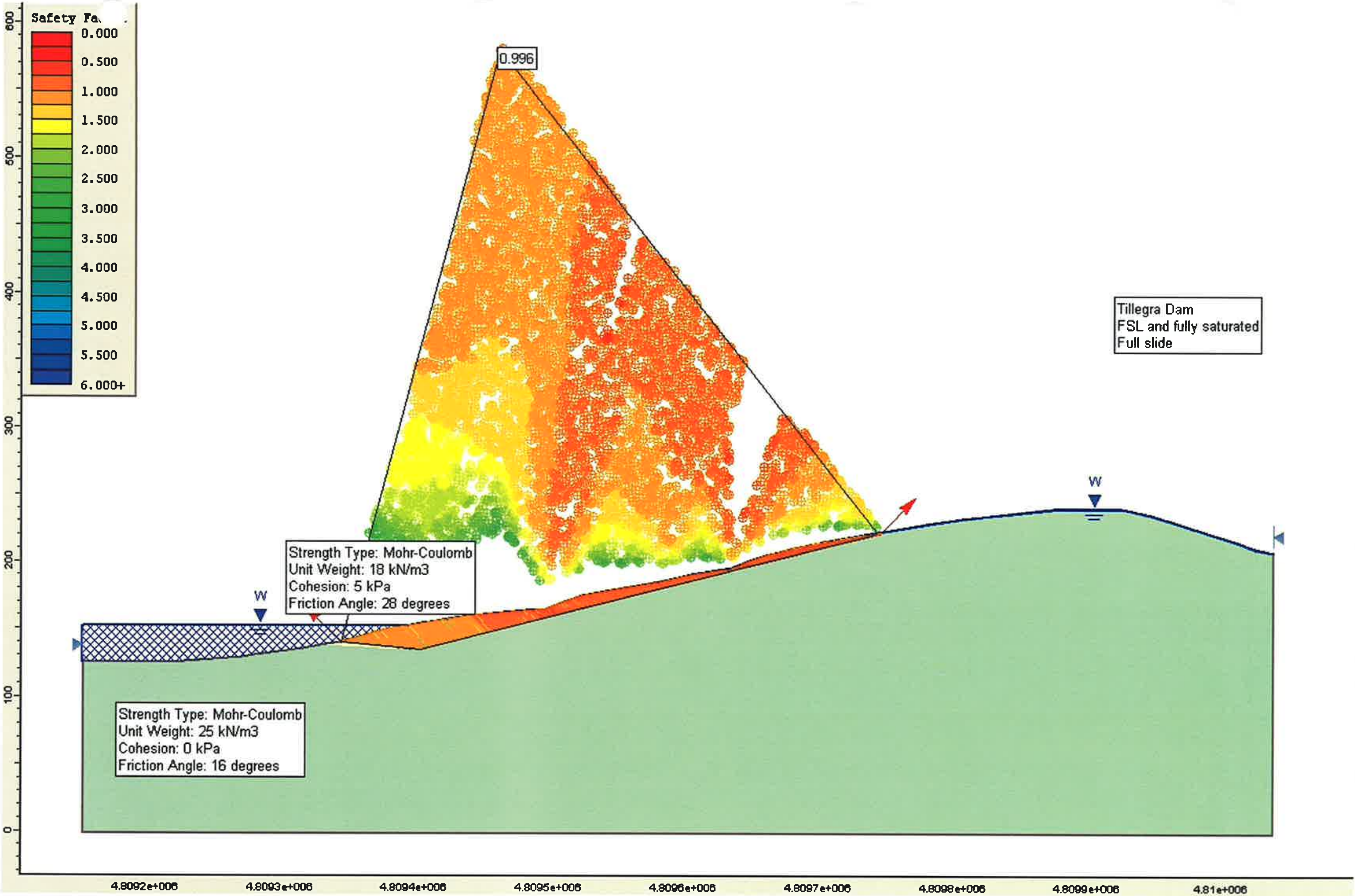
Safety Facto
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.196
Tillegra Dam
FSL and fully saturated
Lower slide
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 18 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

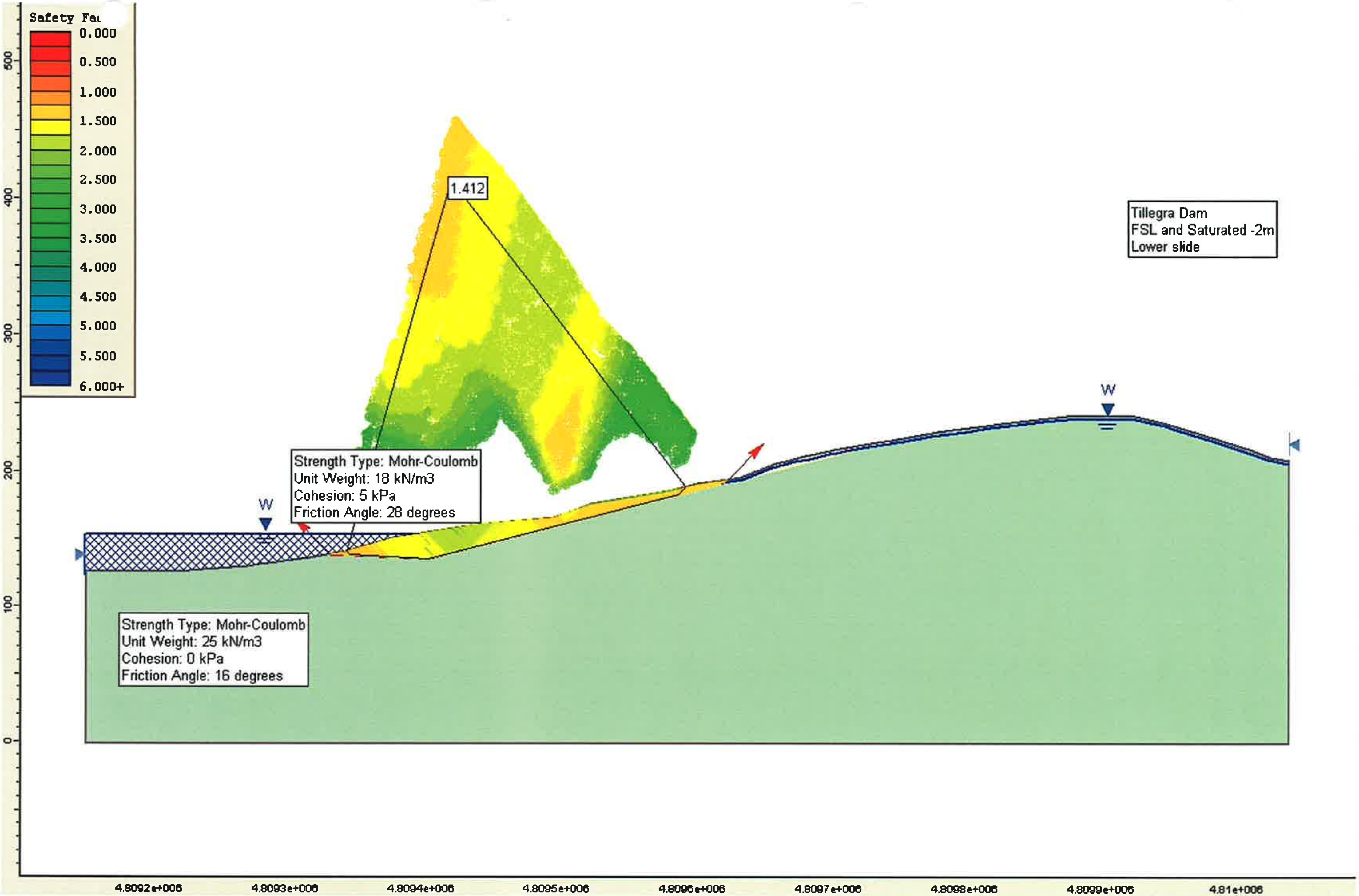


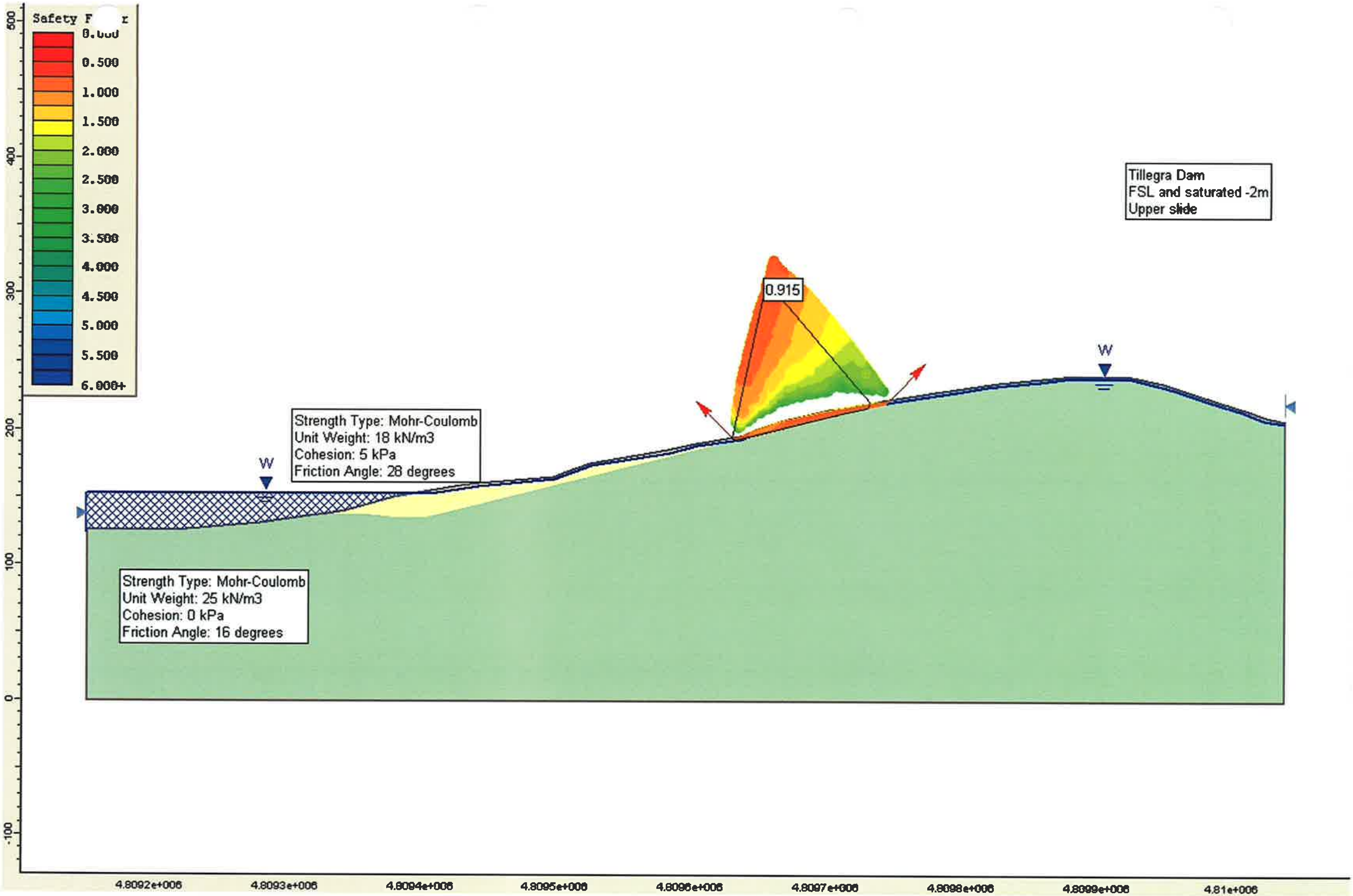
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.289
Tillegra Dam
FSL and fully saturated
Lower slide
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 20 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

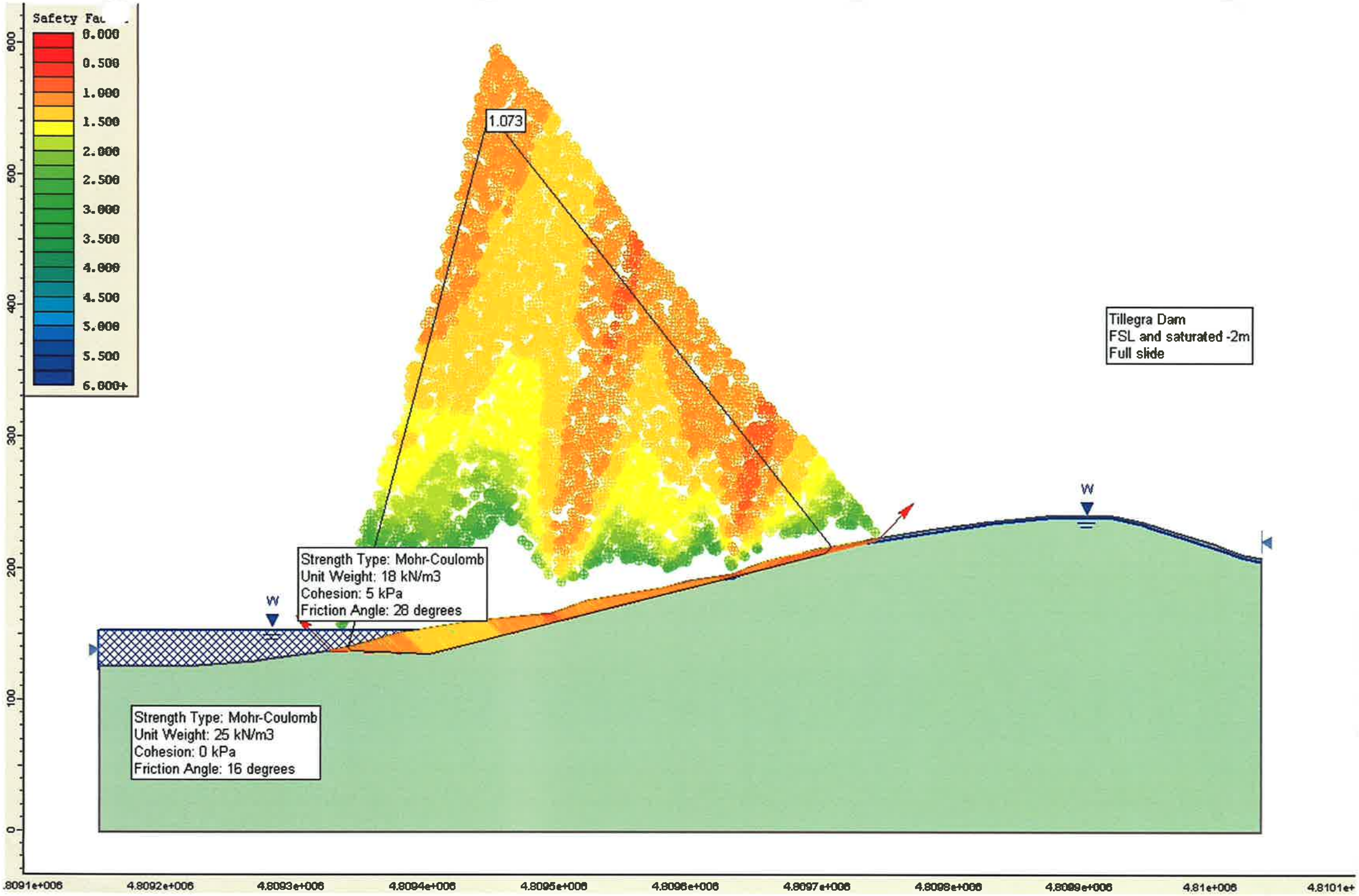

Safety Fa
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.951
Tillegra Dam
FSL and fully saturated
Lower slide
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
W
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Tillegra Dam
FSL and fully saturated
Upper slide
0.542
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
-100
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

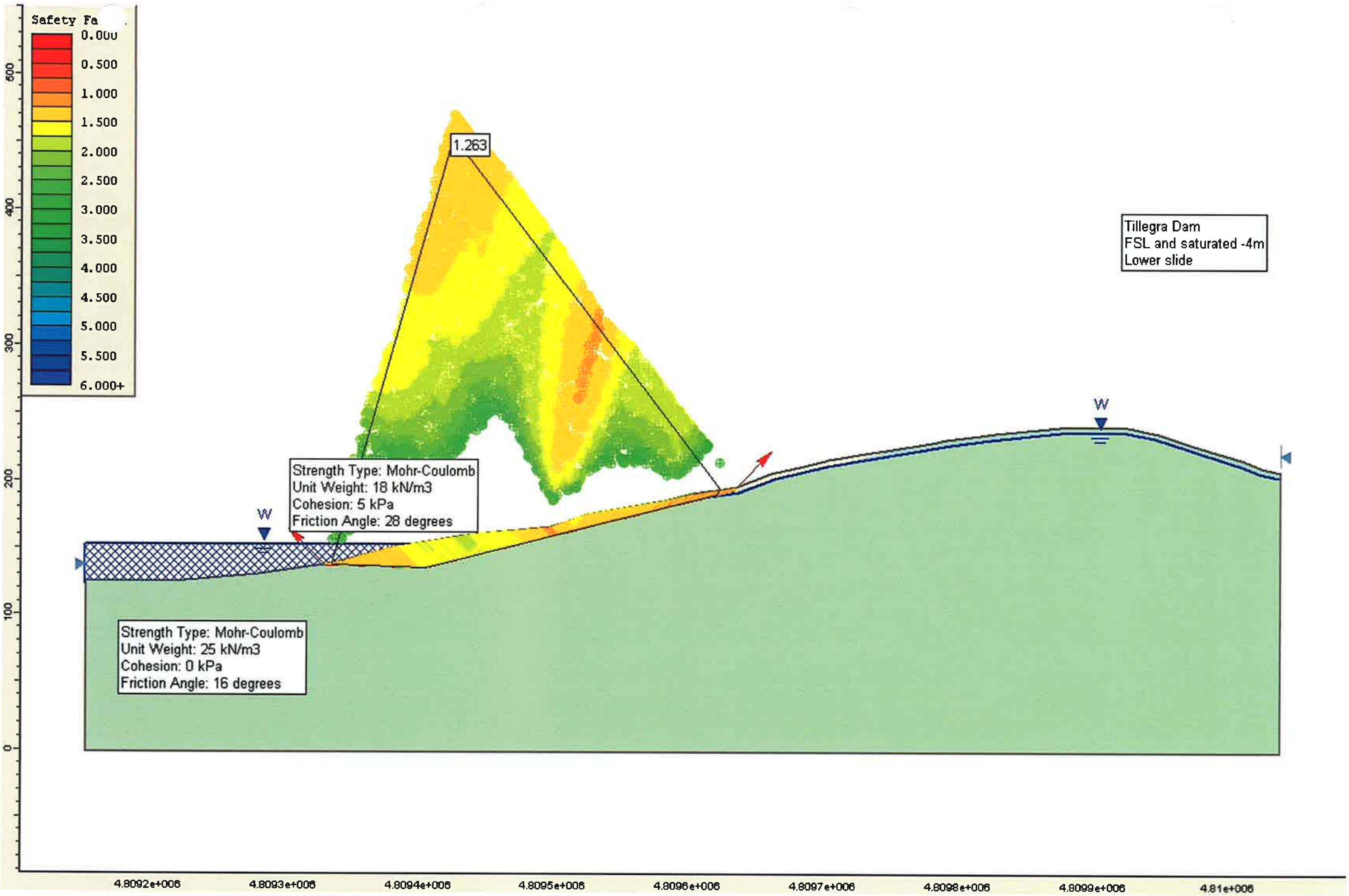

Safety Fa
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.996
Tillegra Dam
FSL and fully saturated
Full slide
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
600
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006


Safety Fac
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.412
Tillegra Dam
FSL and Saturated -2m
Lower slide
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

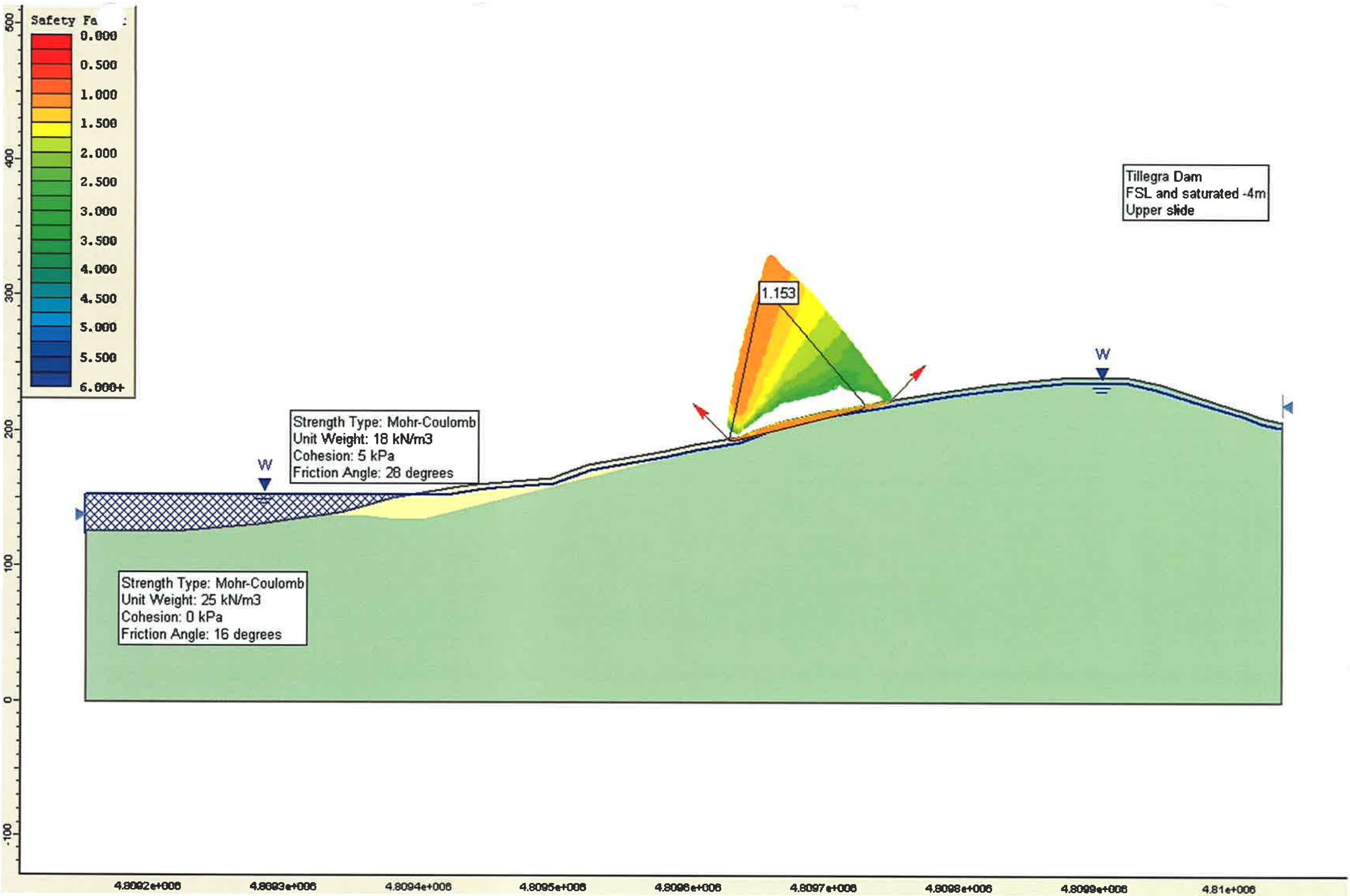

Safety F r
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Tillegra Dam
FSL and saturated -2m
Upper slide
0.915
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
-100
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

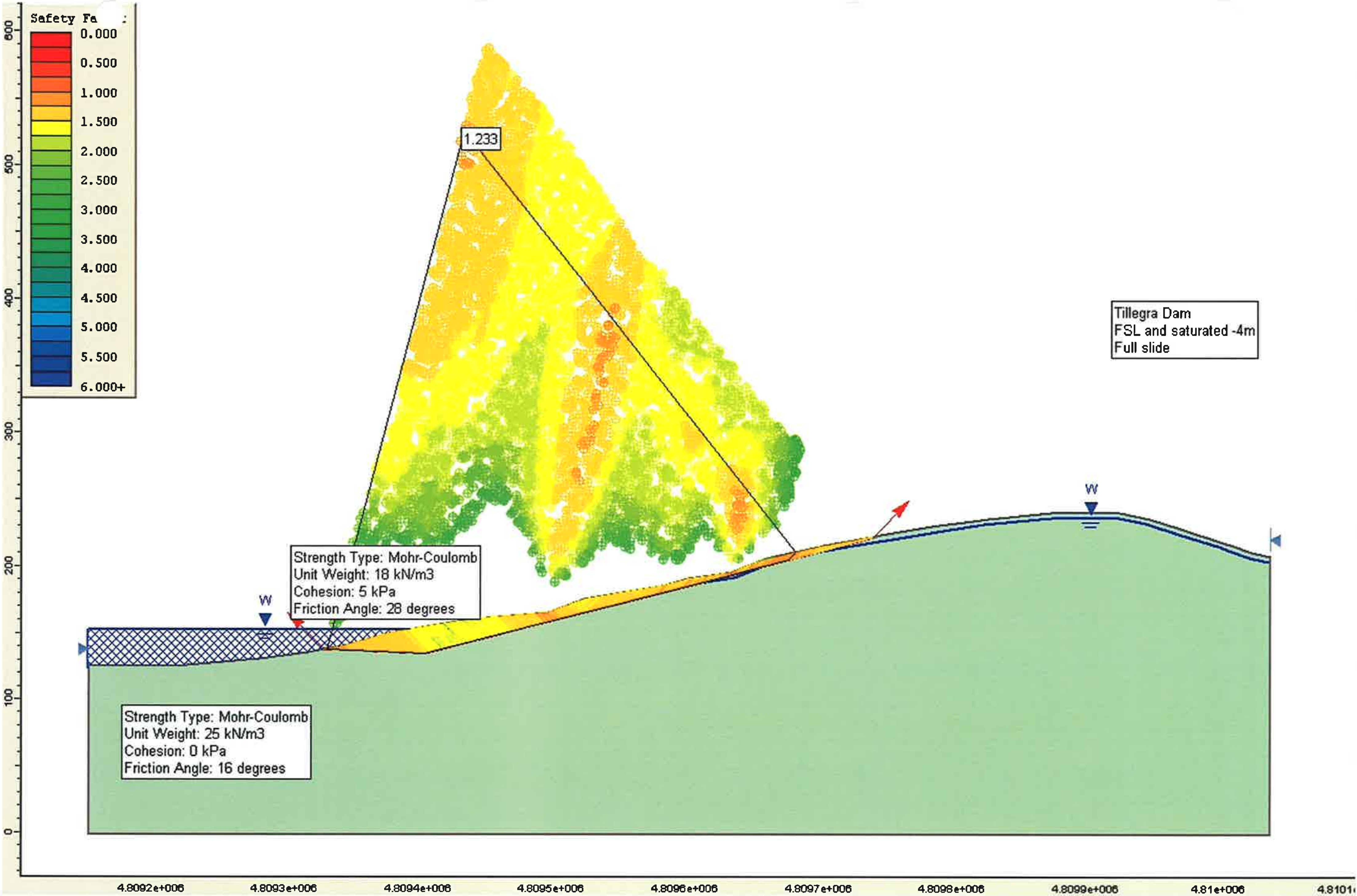


Safety Fa
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.073
Tillegra Dam
FSL and saturated -2m
Full slide
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
W
W
600
500
400
300
200
100
0
.8091e+006
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006
4.8101e+

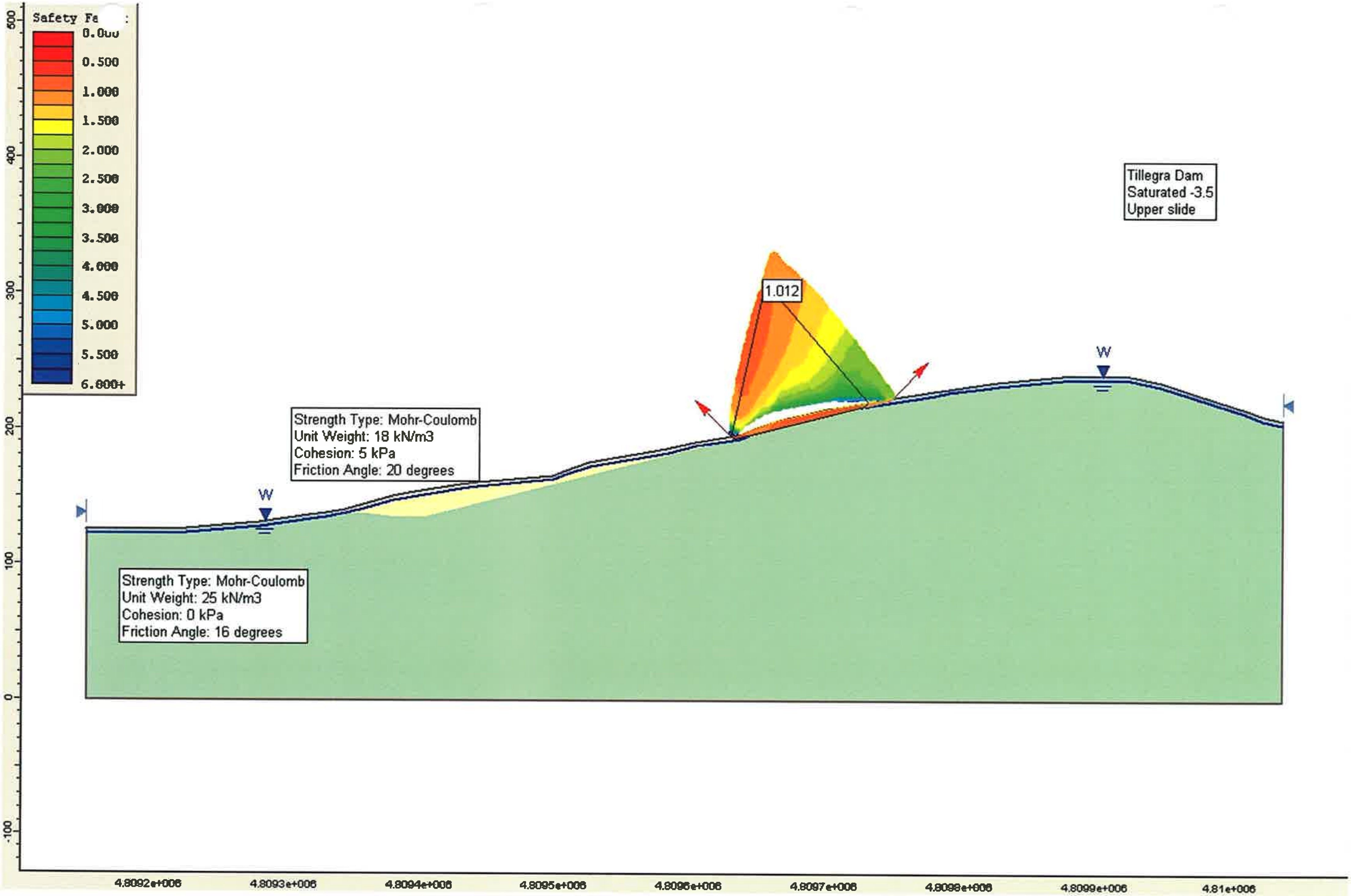


Safety Fa
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.263
Tillegra Dam
FSL and saturated -4m
Lower slide
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

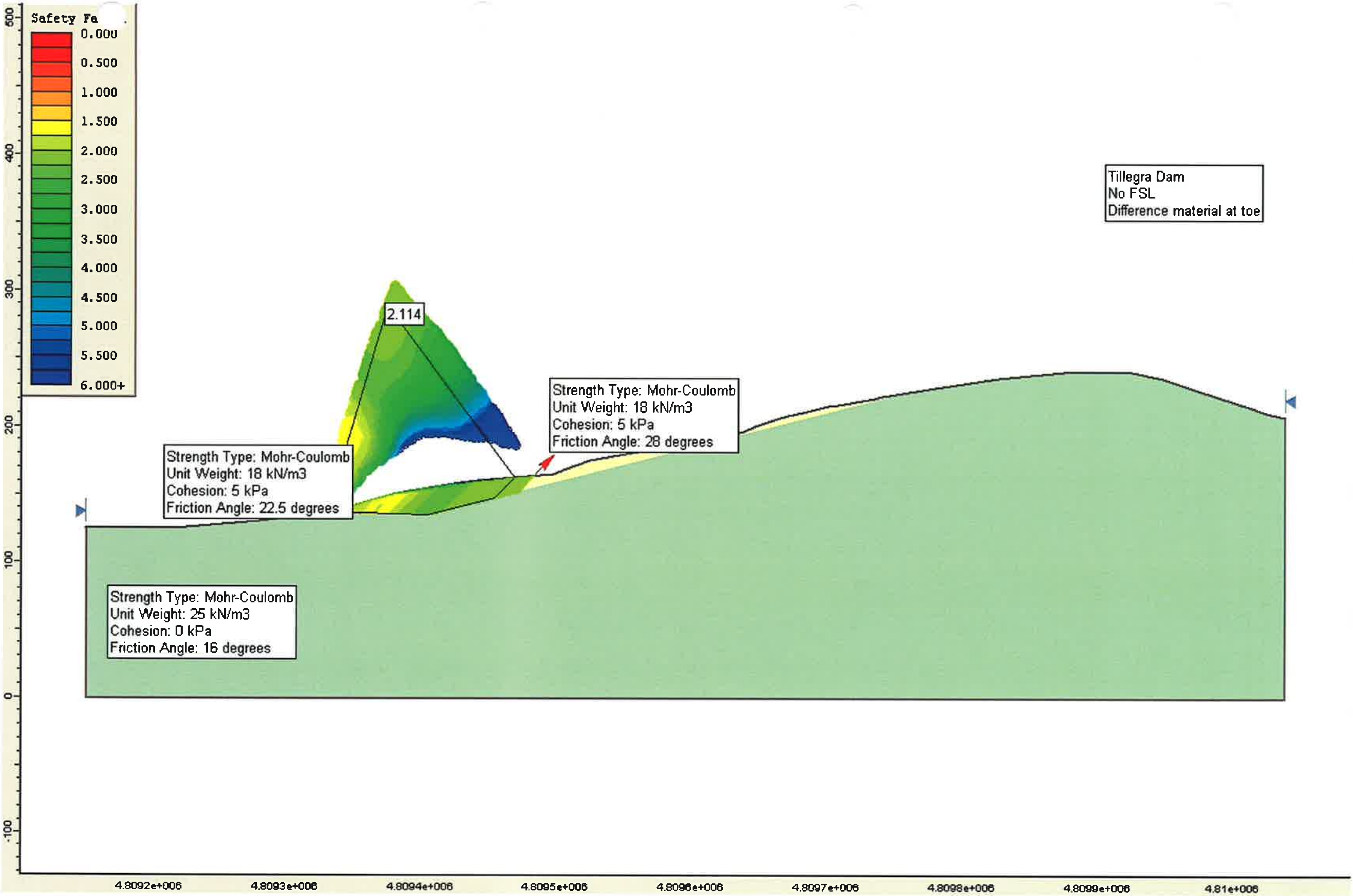

Safety Fa
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Tillegra Dam
FSL and saturated -4m
Upper slide
1.153
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
W
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
-100
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006



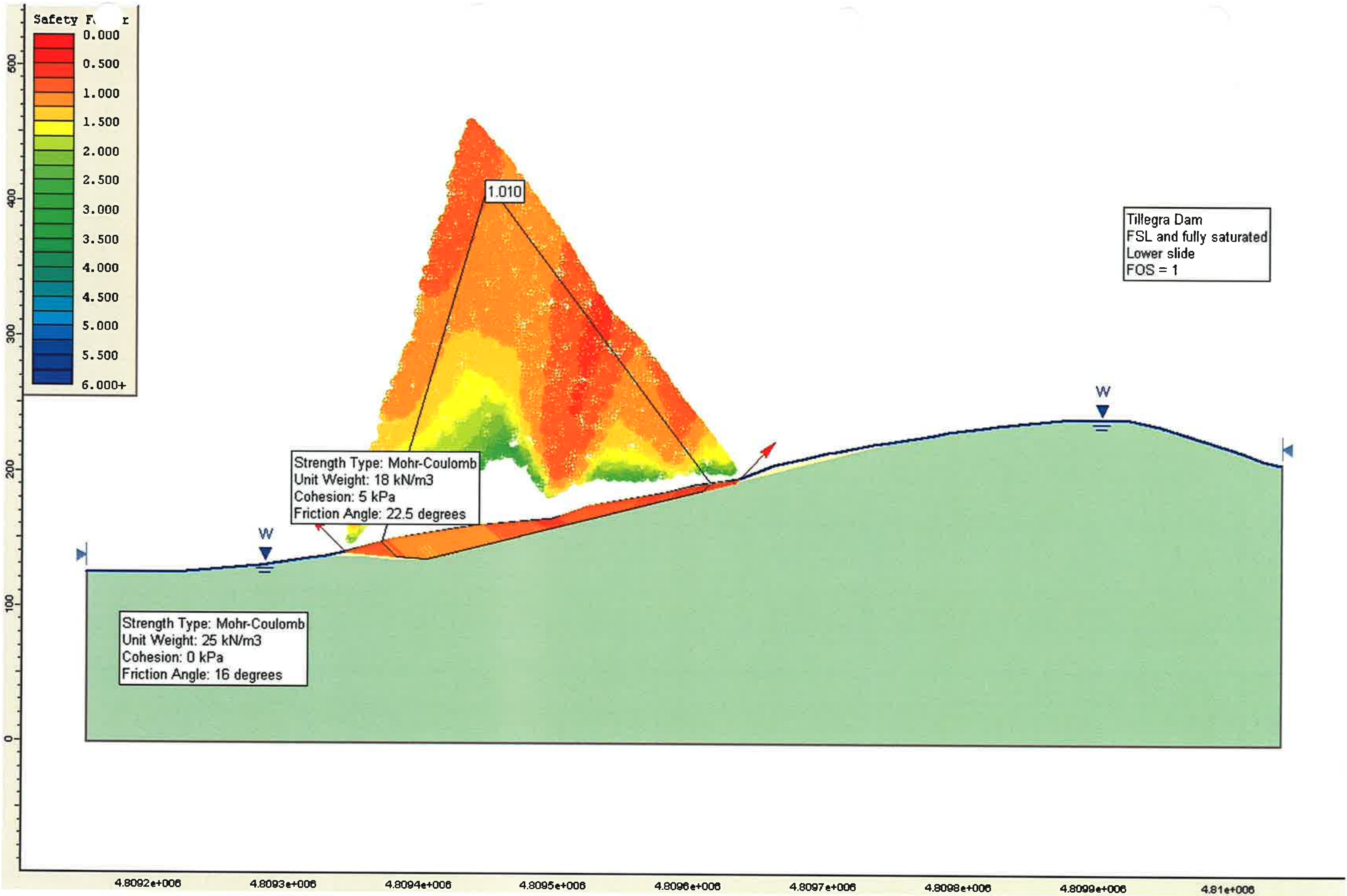
Safety Fa :
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.233
Tillegra Dam
FSL and saturated -4m
Full slide
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
600
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006



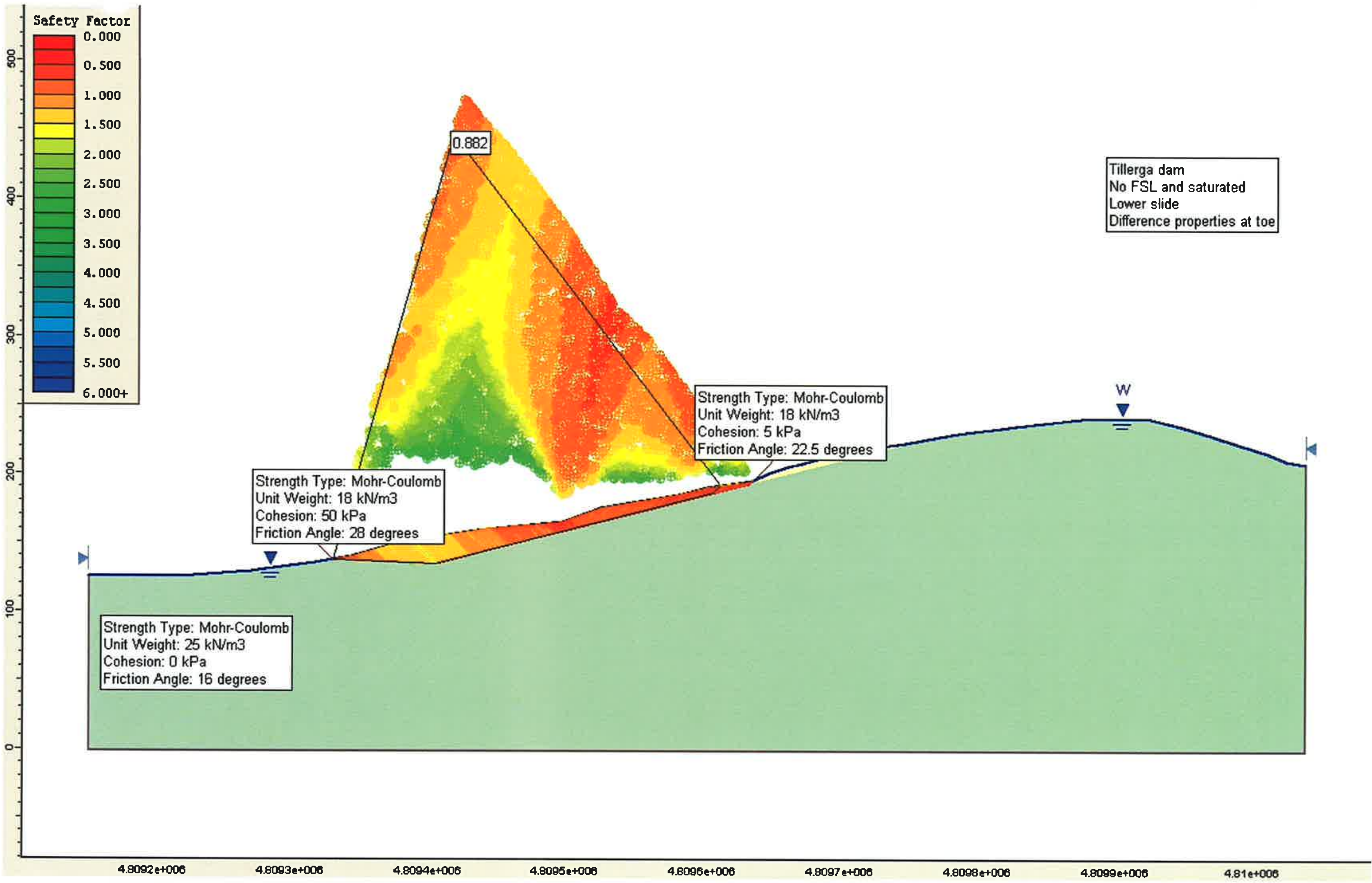
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Tillegra Dam
Saturated -3.5
Upper slide
1.012
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 20 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
-100
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006



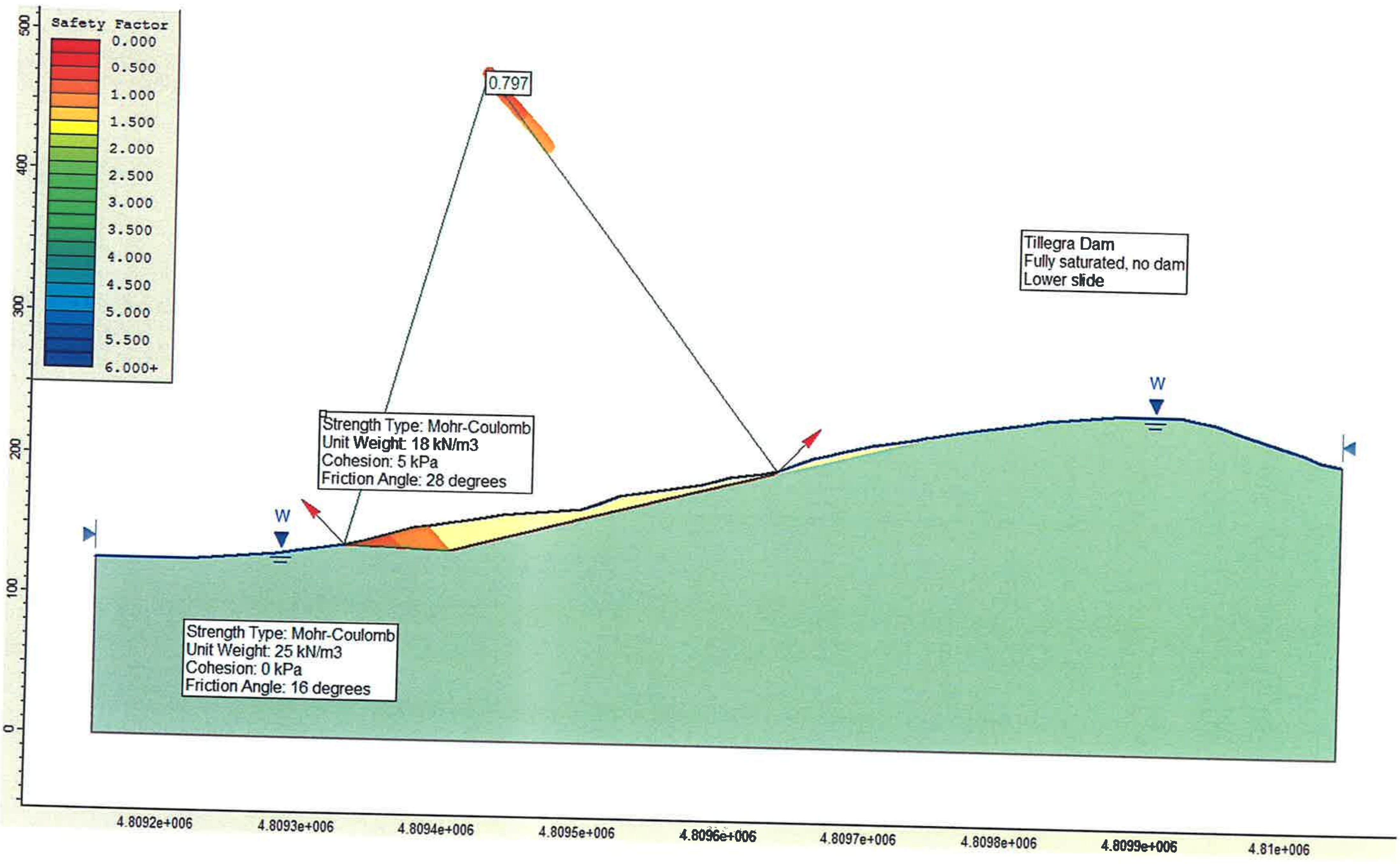
Safety Fa
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Tillegra Dam
No FSL
Difference material at toe
2.114
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 22.5 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
-100
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

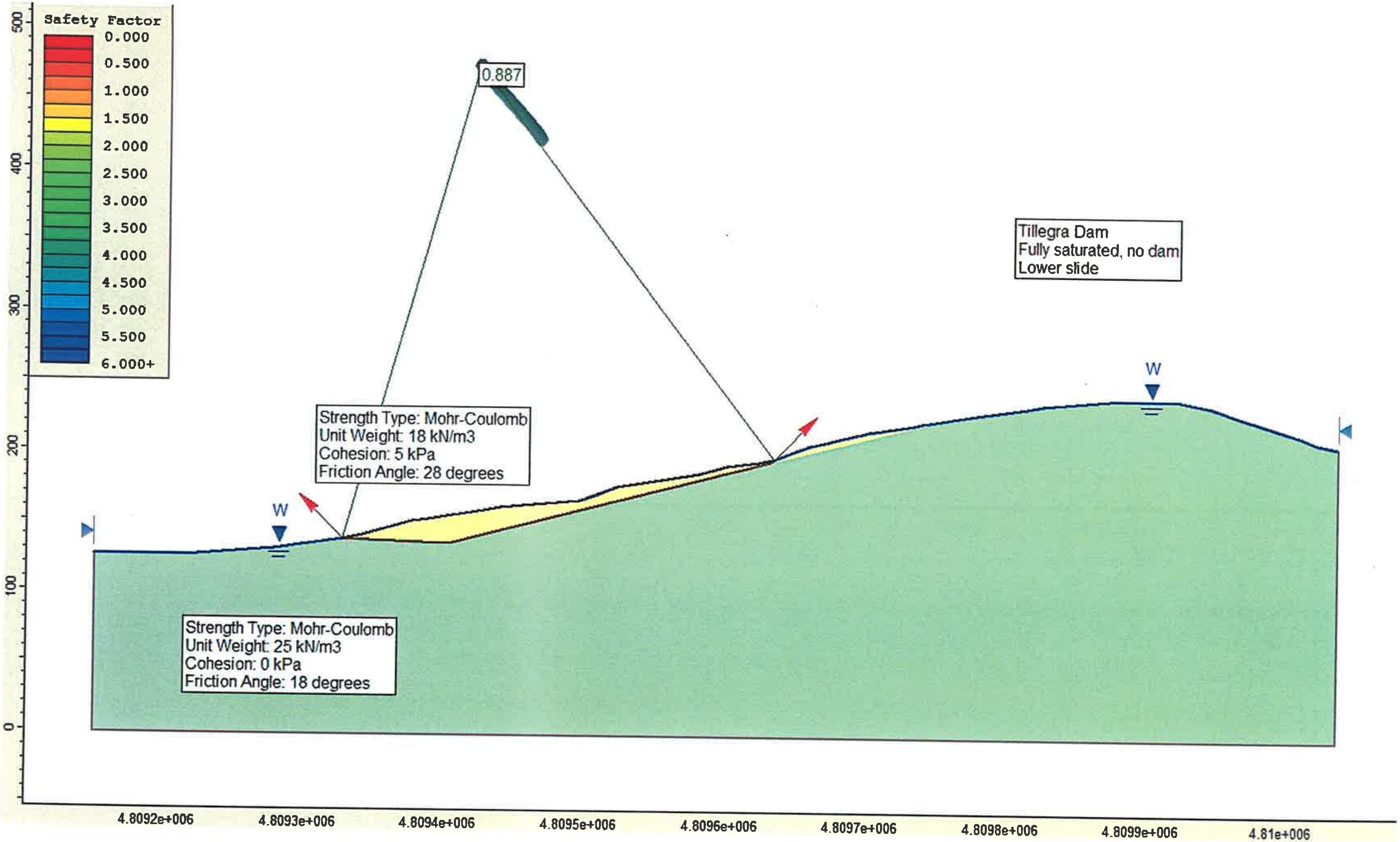


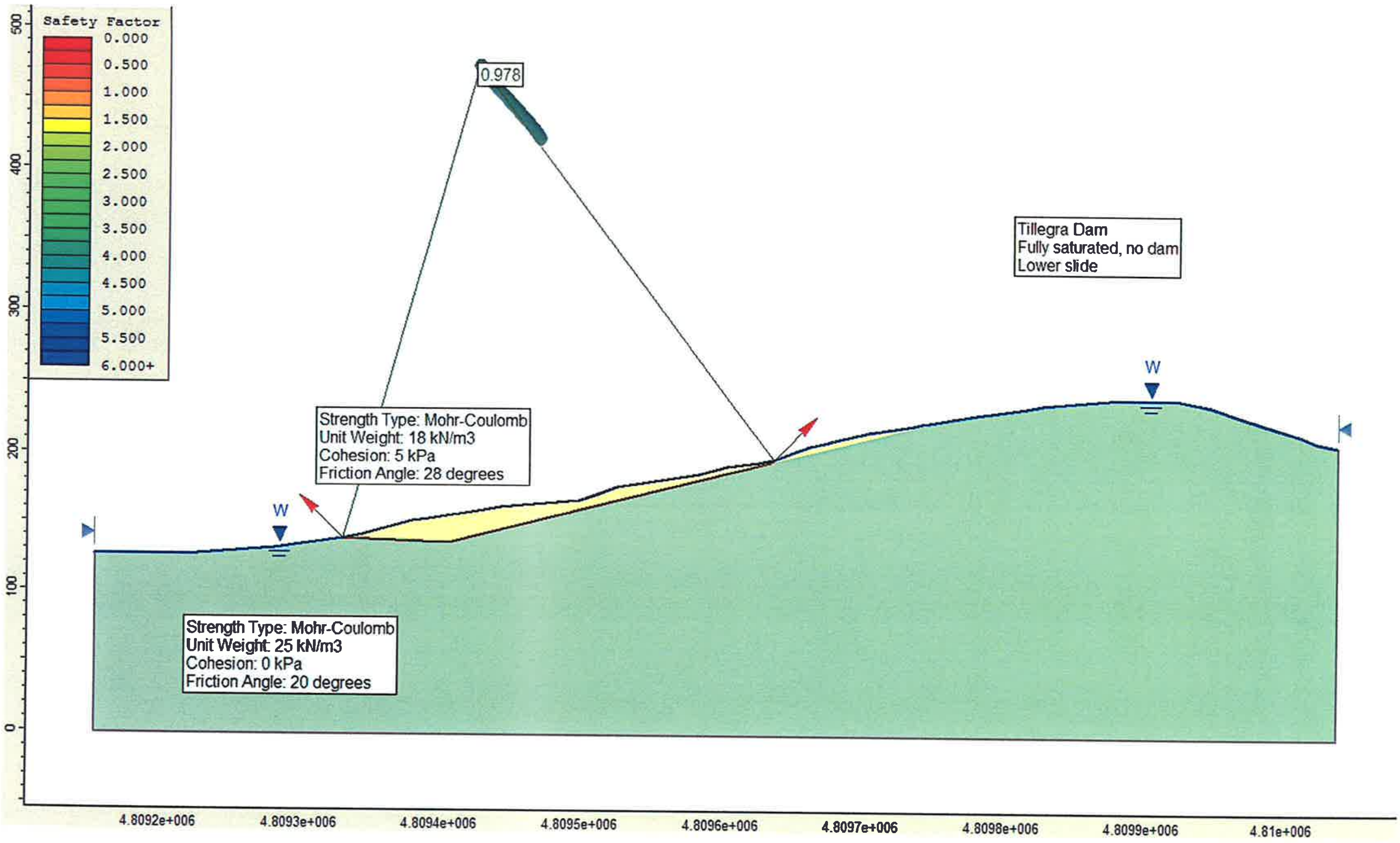
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
500
400
300
200
100
0
1.010
Tillegra Dam
FSL and fully saturated
Lower slide
FOS = 1
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 22.5 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

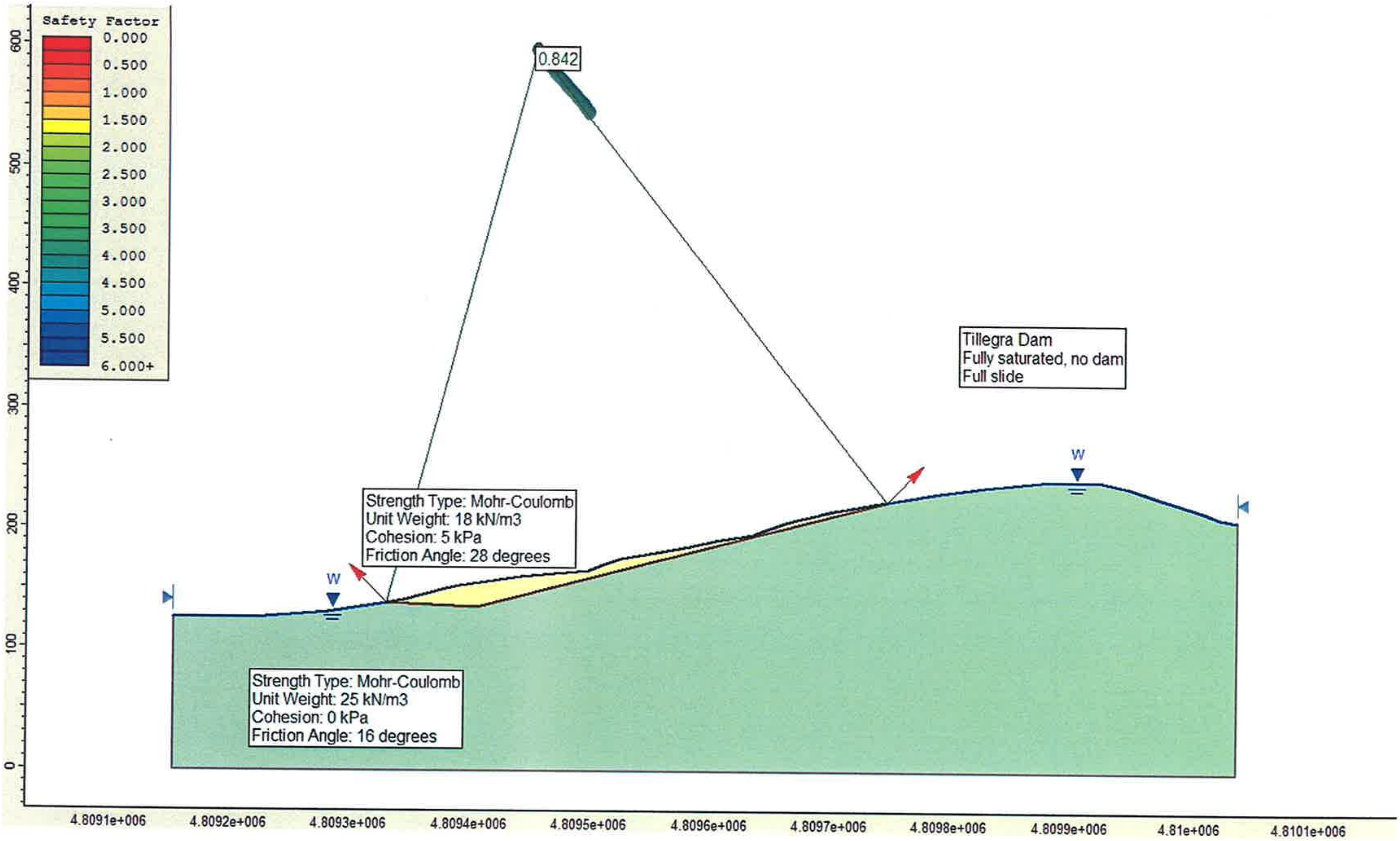


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.882
Tillerga dam
No FSL and saturated
Lower slide
Difference properties at toe
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 22.5 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 50 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
W
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.797
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Tillegra Dam
Fully saturated, no dam
Lower slide
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.887
Tillegra Dam
Fully saturated, no dam
Lower slide
W
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 18 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.978
Tillegra Dam
Fully saturated, no dam
Lower slide
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
W
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 20 degrees
500
400
300
200
100
0
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006

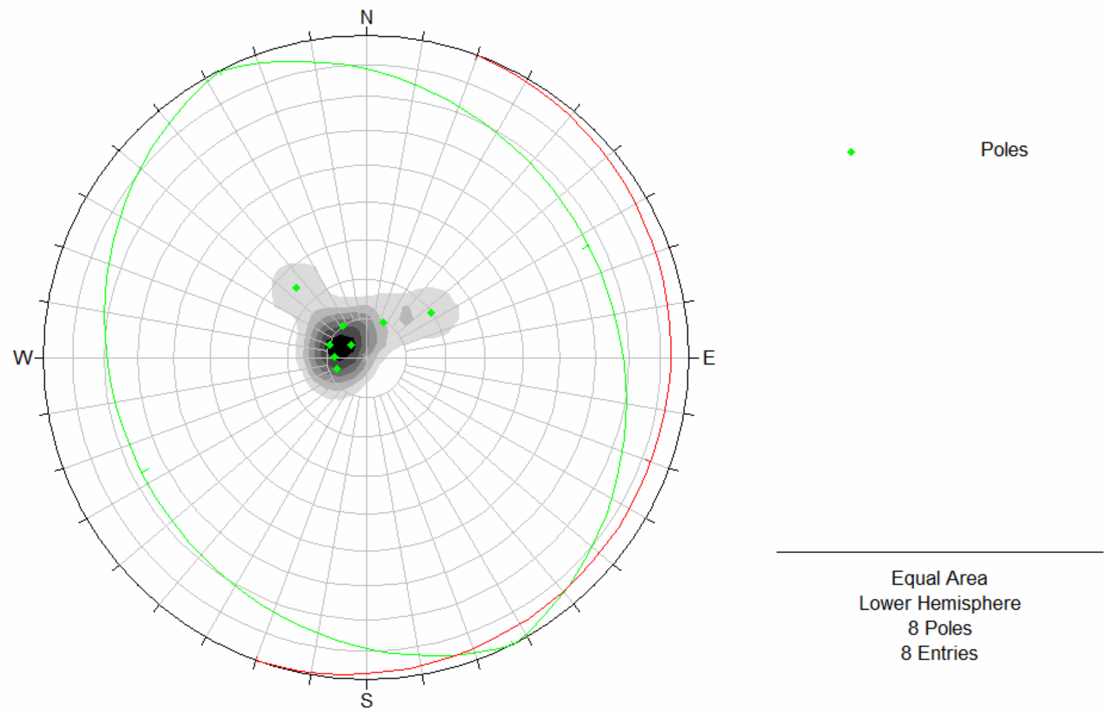


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.842
Tillegra Dam
Fully saturated, no dam
Full slide
W
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 degrees
W
Strength Type: Mohr-Coulomb
Unit Weight: 25 kN/m3
Cohesion: 0 kPa
Friction Angle: 16 degrees
600
500
400
300
200
100
0
4.8091e+006
4.8092e+006
4.8093e+006
4.8094e+006
4.8095e+006
4.8096e+006
4.8097e+006
4.8098e+006
4.8099e+006
4.81e+006
4.8101e+006

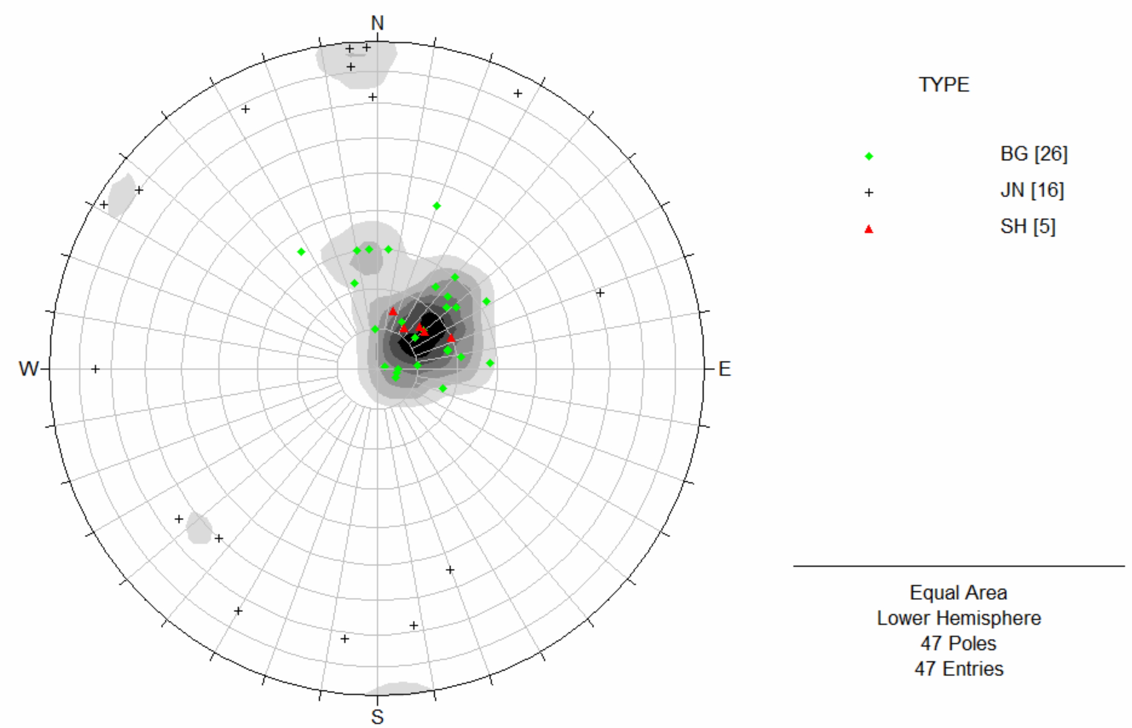
APPENDIX E

KINEMATIC ANALYSIS FOR FIRST TIME SLIDES BY DOMAIN

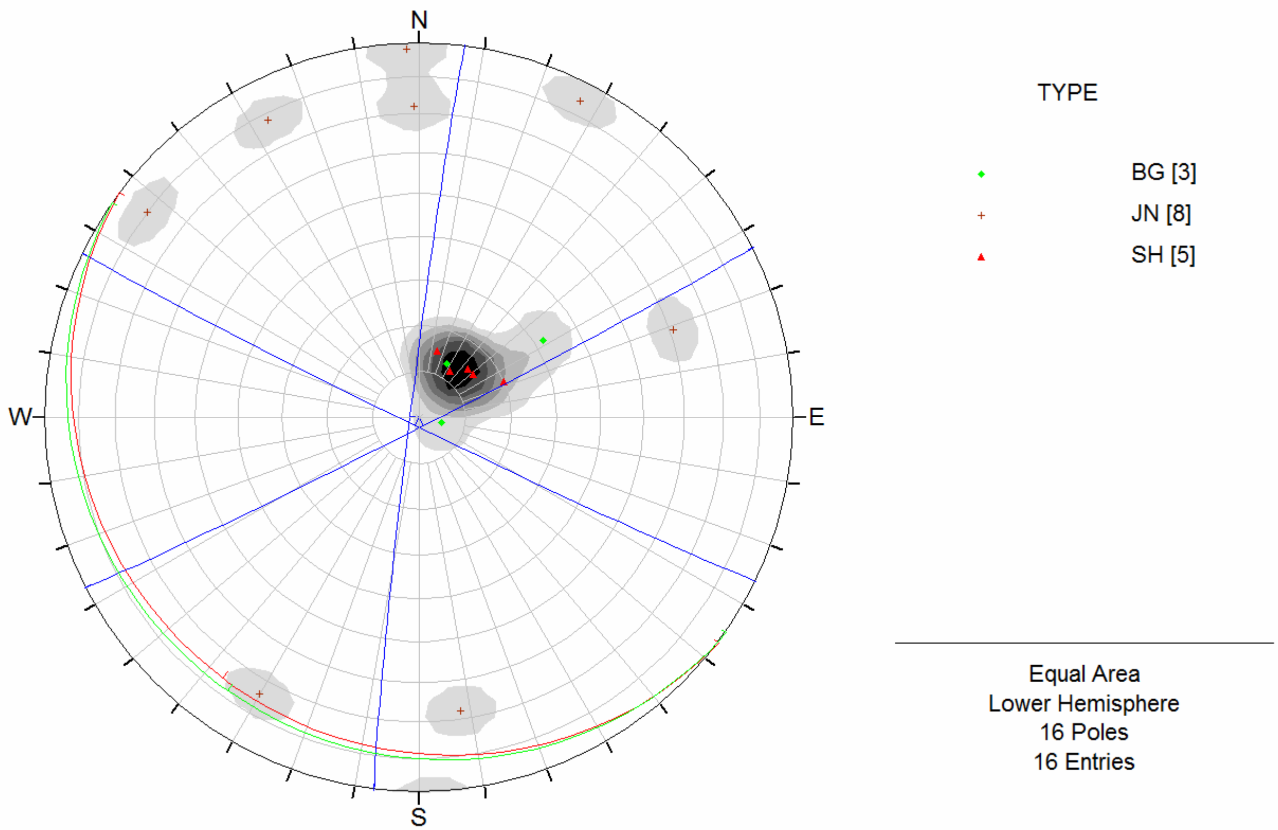
DOMAIN 1 SLIDING ANALYSIS



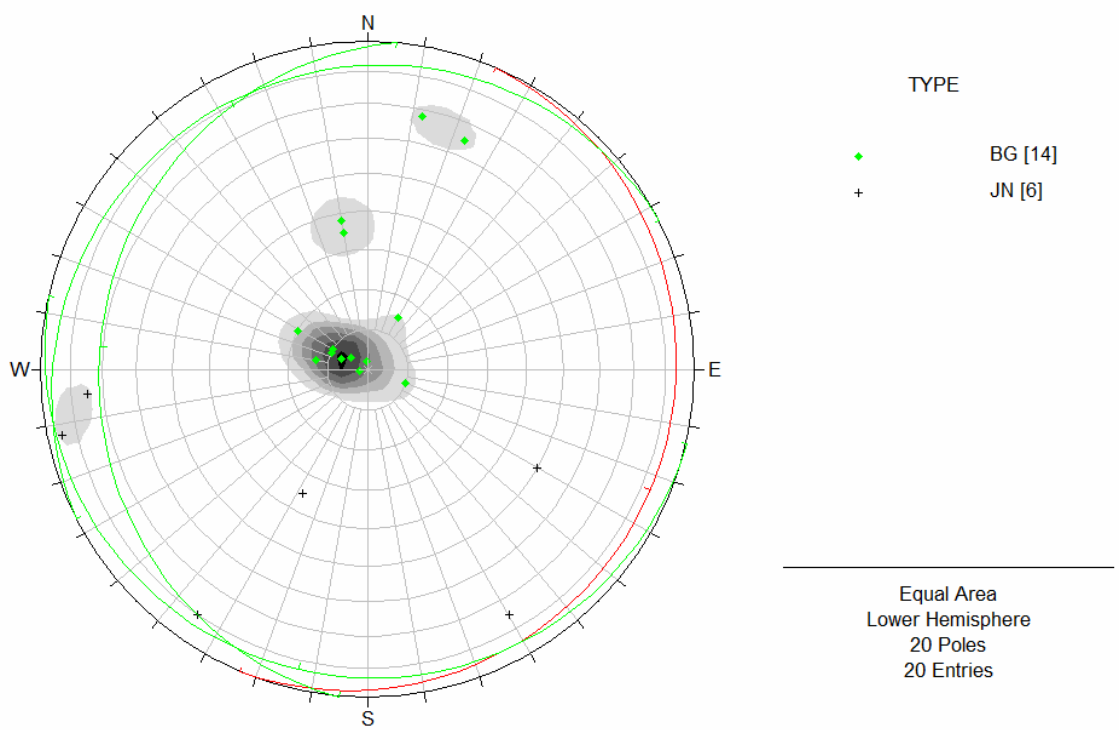
DOMAIN 2 SLIDING ANALYSIS



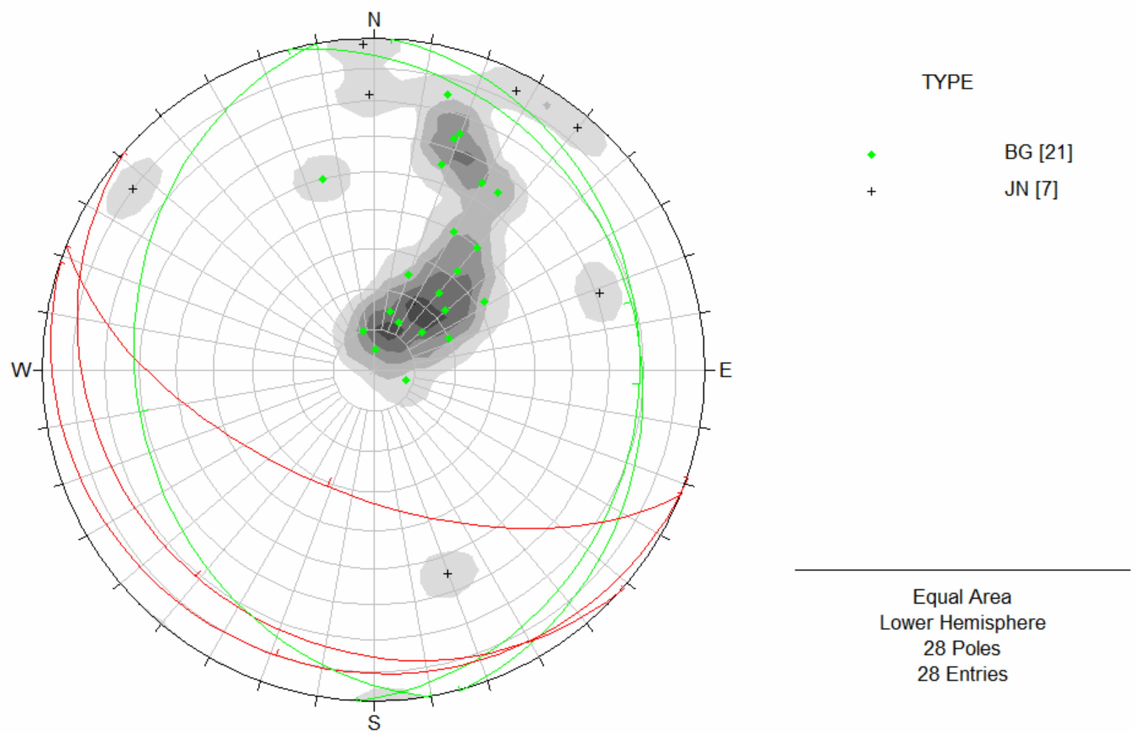
SLIDE 2A SLIDING ANALYSIS



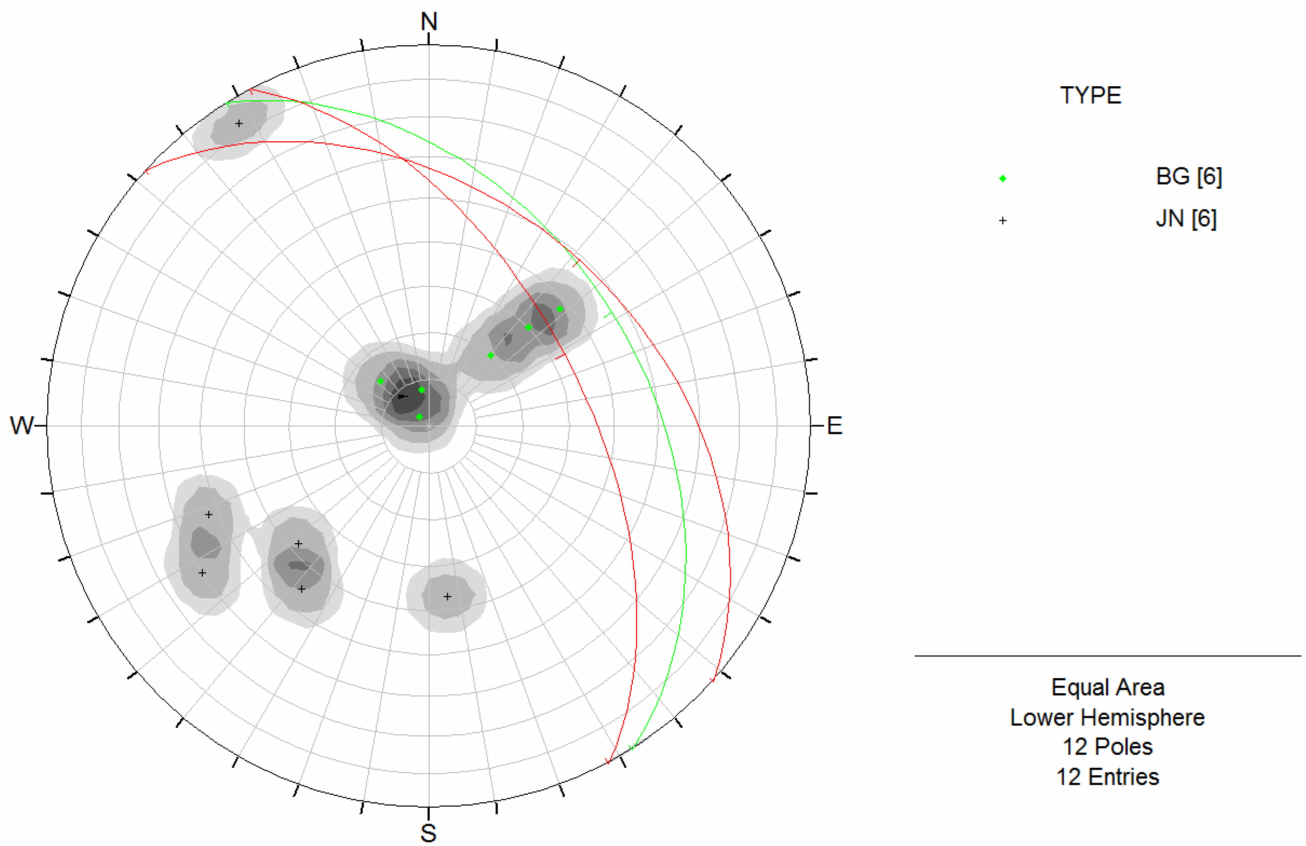
DOMAIN 3 SLIDING ANALYSIS



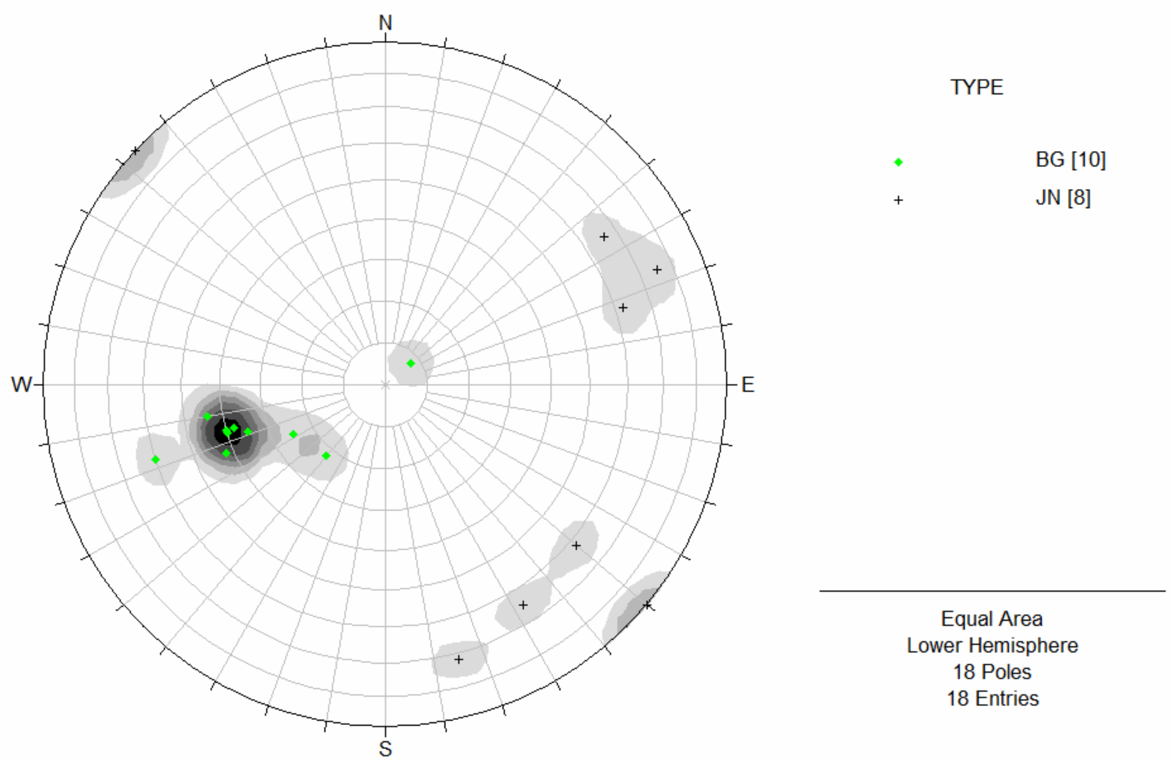
DOMAIN 4 SLIDING ANALYSIS



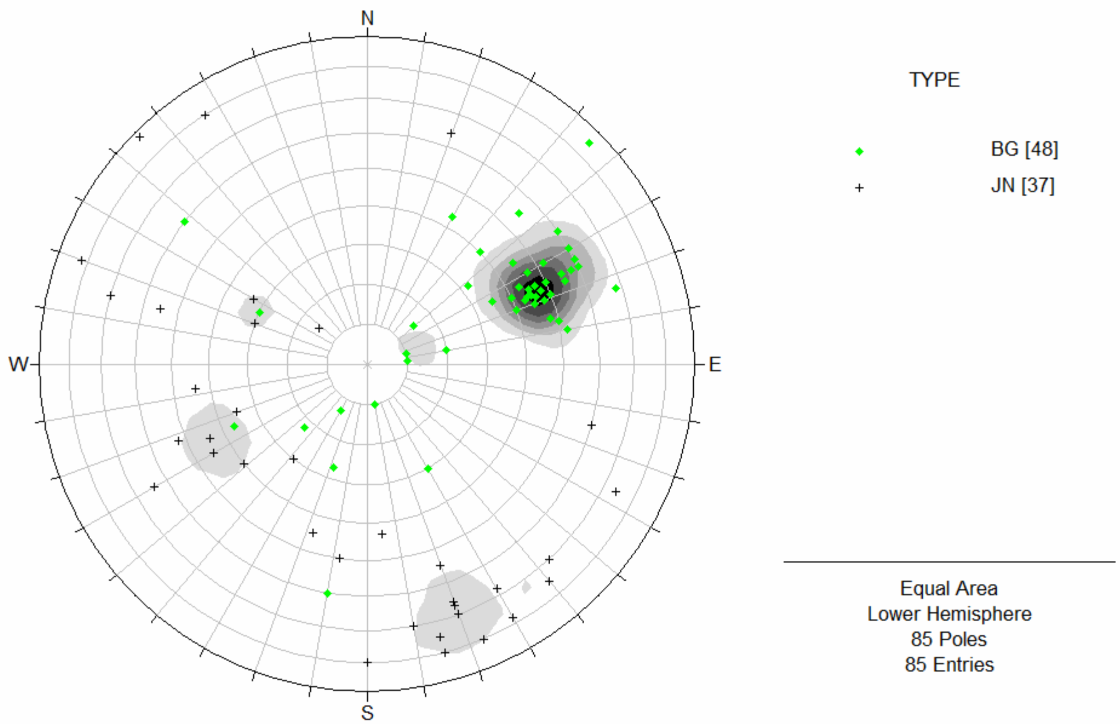
SLIDE 5A SLIDING ANALYSIS



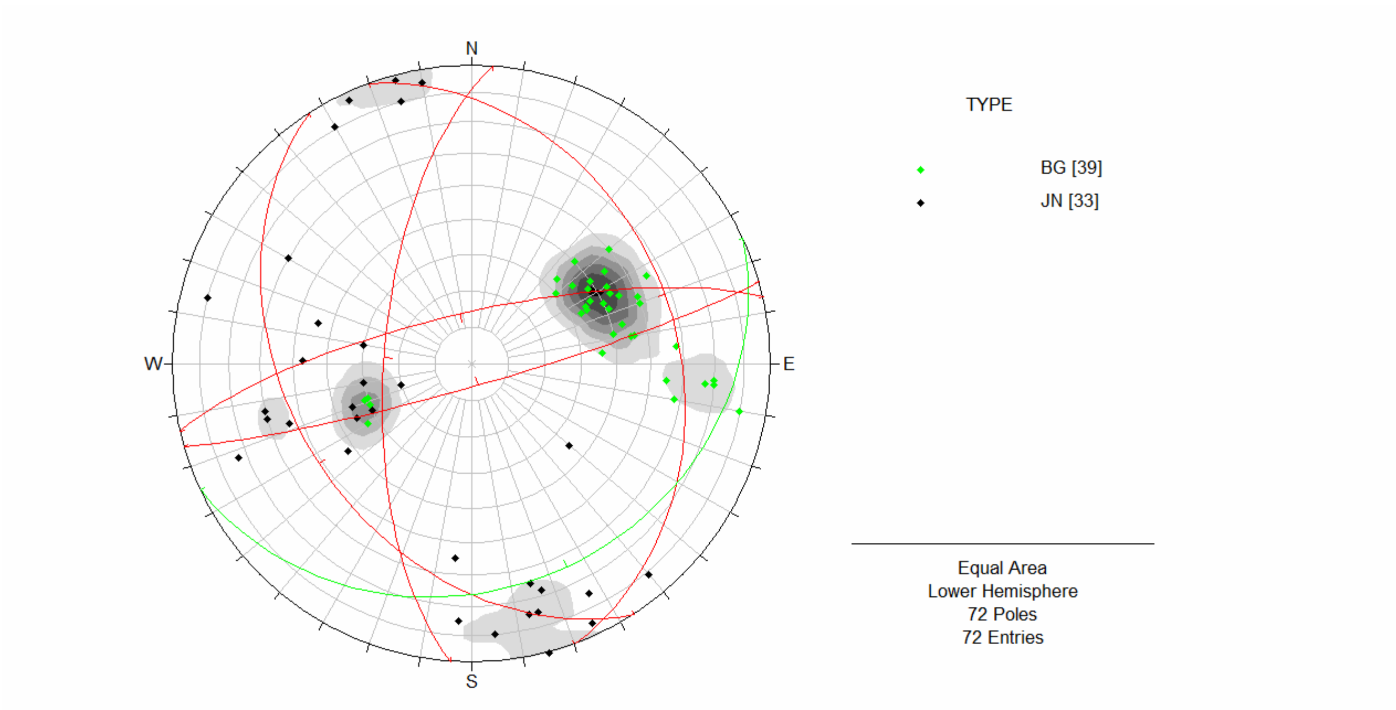
DOMAIN 6 SLIDING ANALYSIS



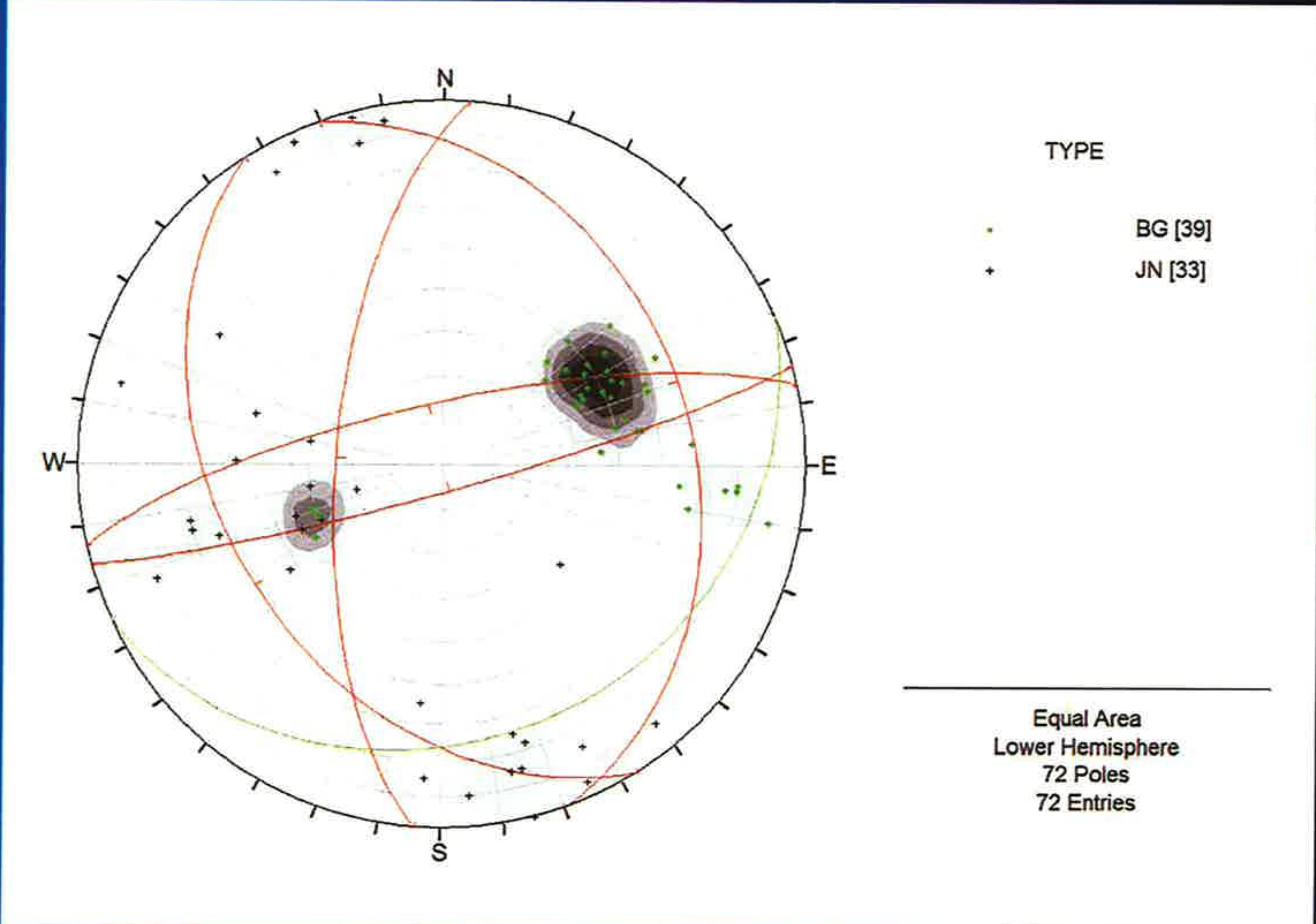
DOMAIN 7 SLIDING ANALYSIS



DOMAIN 8 SLIDING ANALYSIS



UPPER WEST ABUTMENT SLIDING ANALYSIS OUT OF RESERVOIR

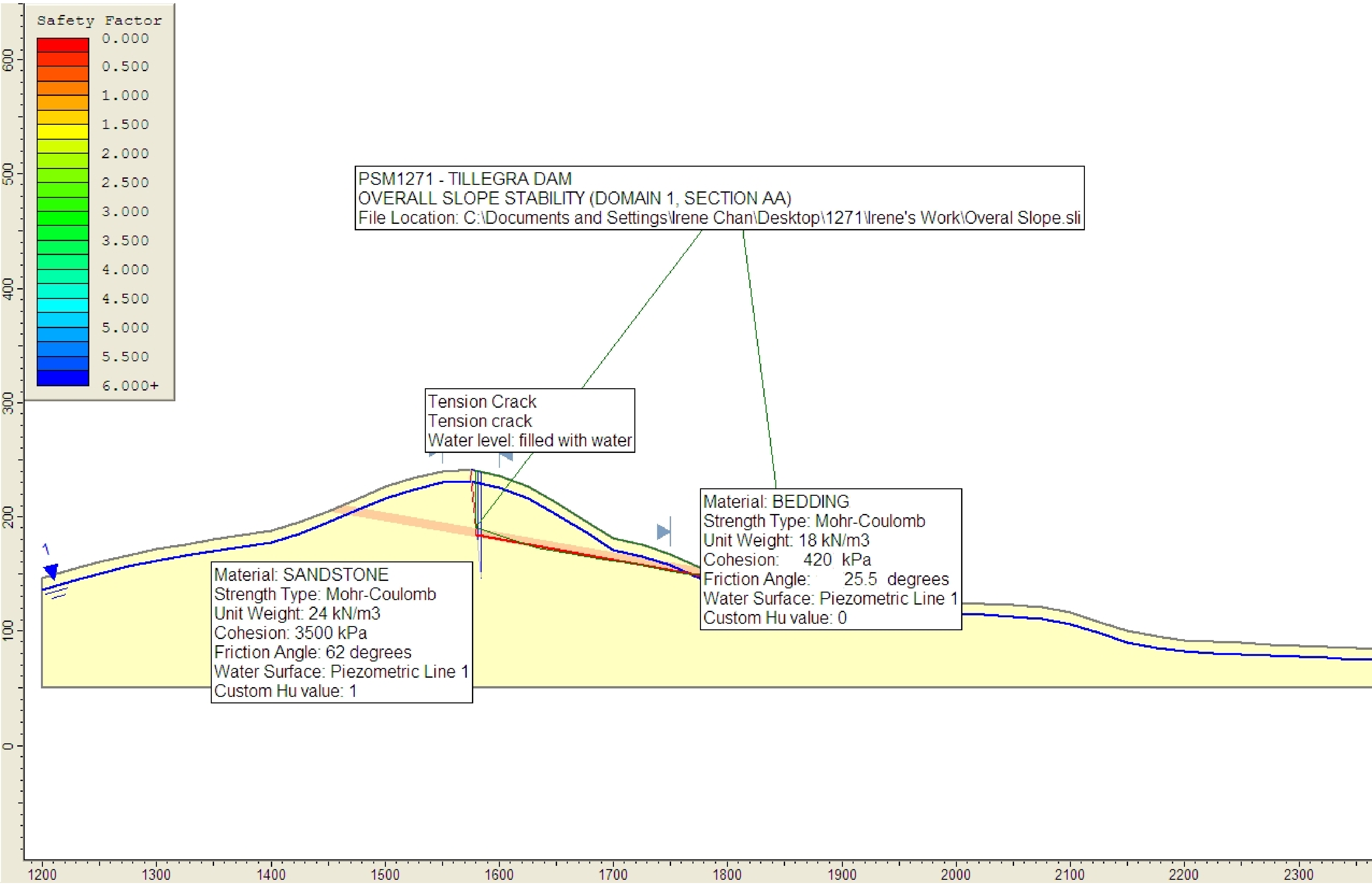


Pells Sullivan Meynink

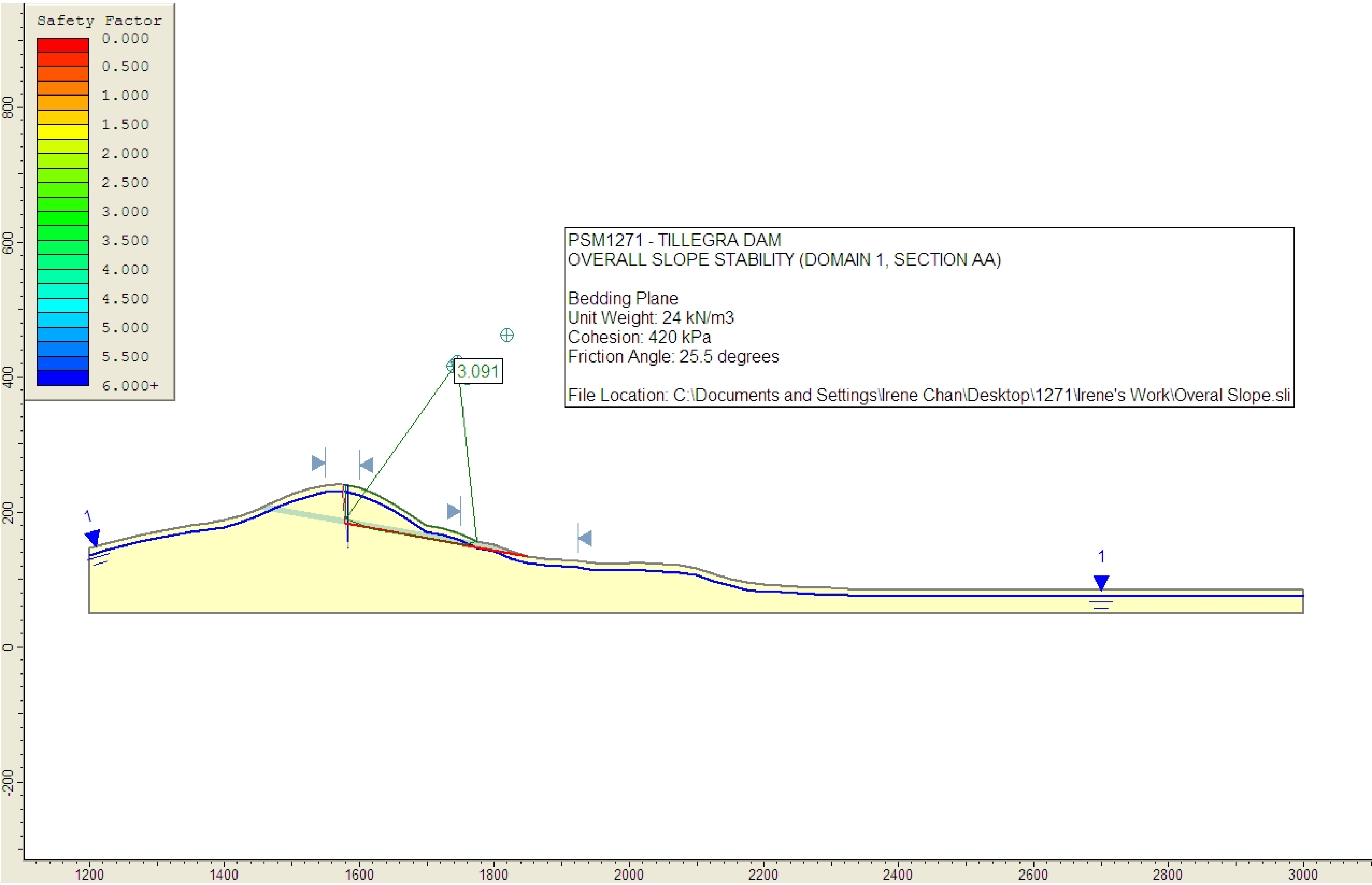
PSM

APPENDIX F

STABILITY ANALYSIS RESULTS, OVERALL SLOPE



Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
PSM1271 - TILLEGRA DAM
OVERALL SLOPE STABILITY (DOMAIN 1, SECTION AA)
File Location: C:\Documents and Settings\Irene Chan\Desktop\1271\Irene's Work\Overal Slope.sli
Tension Crack
Tension crack
Water level: filled with water
Material: BEDDING
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 420 kPa
Friction Angle: 25.5 degrees
Water Surface: Piezometric Line 1
Custom Hu value: 0
Material: SANDSTONE
Strength Type: Mohr-Coulomb
Unit Weight: 24 kN/m3
Cohesion: 3500 kPa
Friction Angle: 62 degrees
Water Surface: Piezometric Line 1
Custom Hu value: 1
600
500
400
300
200
100
0
1200
1300
1400
1500
1600
1700
1800
1900
2000
2100
2200
2300



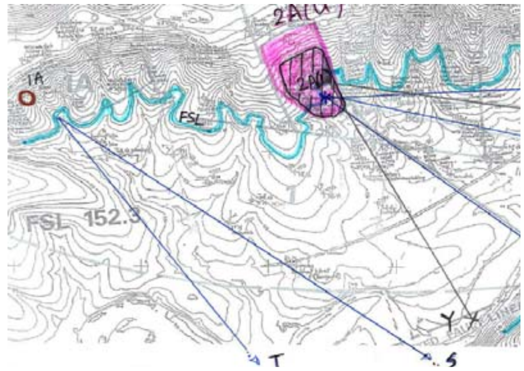
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
PSM1271 - TILLEGRA DAM
OVERALL SLOPE STABILITY (DOMAIN 1, SECTION AA)
Bedding Plane
Unit Weight: 24 kN/m3
Cohesion: 420 kPa
Friction Angle: 25.5 degrees
File Location: C:\Documents and Settings\Irene Chan\Desktop\1271\Irene's Work\Overal Slope.sli
3.091
1
1
800
600
400
200
0
-200
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000

APPENDIX G

ASSESSMENT OF IMPULSE WAVES

PSM1271 Tillegra Dam IC 18/09/08

Forecasting Impulse Wavers in Reservoirs - A Huber & W Hager (1997)



SLIDE 1A

SIMPLIFIED WAVE - 2D

(Eqn 3)

Variables

α	11	deg
ρs	2.65	t/m3
ρw	1	t/m3
Vs	3,927	m3
b	40	m
d_S	20	m
d_T	50	m

x_S	2500 m	γ_S	0 deg
x_T	1550 m	γ_T	17 deg

Height of Impulse Wave (2D)

H_S	*0.6*	m
H_T	*0.9*	m

NB. Not valid as α less than 28 deg

3D IMPULSE WAVE PROPAGATION

(Eqn 6)

M_S	*0.2*
M_T	*0.0*

Height of Impulse Wave (3D)

H_S	*0.01*	m
H_T	*0.01*	m

WAVE CELERITY ie, propagation velocity

(Eqn 8)

c_S	*14.0*	m/s
c_T	*22.1*	m/s

WAVE RUN-UP ON SHORE

(Eqn 9)

β_S	0.20 rad	R/d_S	*0.04*
β_T	0.53 rad	R/d_T	*0.02*
L	*500* m		

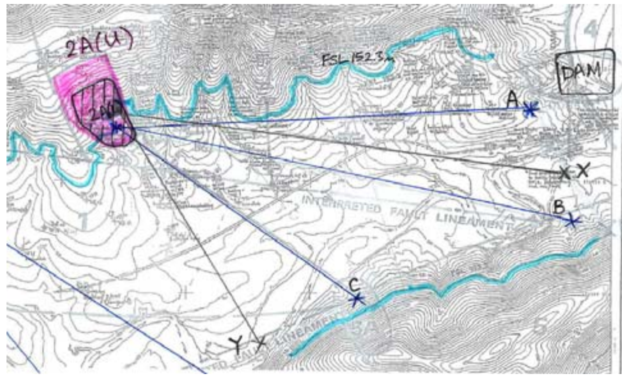
Run-up height

R_S	*0.7*	m
R_T	*1.0*	m

PSM1271 Tillegra Dam IC 18/12/08

Forecasting Impulse Wavers in Reservoirs - A Huber & W Hager (1997)

SLIDE 2A - UPPER



SIMPLIFIED WAVE - 2D

(Eqn 3)

Variables

α	9	deg			
ρs	2.65	t/m3			
ρw	1	t/m3			
Vs	200,000	m3			
b	210	m			
d_A	50	m			
d_X	50	m			
d_Y	50	m			
x_A	1700	m	γ_A	10	deg
x_X	1850	m	γ_X	0	deg
x_Y	1200	m	γ_Y	50	deg

Height of Impulse Wave (2D)

H_A	*2.2*	m
H_X	*2.2*	m
H_Y	*2.4*	m

NB. Not valid as α less than 28 deg

3D IMPULSE WAVE PROPAGATION

(Eqn 6)

M_A	*0.4*
M_X	*0.4*
M_Y	*0.4*

Height of Impulse Wave (3D)

H_A	*0.02*	m
H_X	*0.02*	m
H_Y	*0.02*	m

WAVE CELERITY ie, propagation velocity

(Eqn 8)

c_A	*22.2*	m/s
c_X	*22.2*	m/s
c_Y	*22.2*	m/s

0.37037

WAVE RUN-UP ON SHORE

(Eqn 9)

β_A	0.79	rad	R/d_A	*0.06*
β_X	0.79	rad	R/d_X	*0.06*
β_Y	0.35	rad	R/d_Y	*0.06*
L	*500*	m		

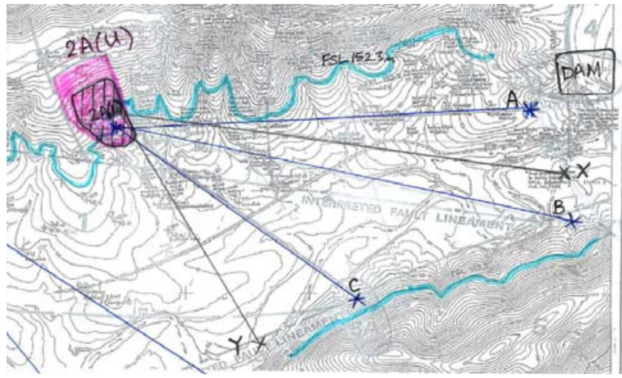
Run-up height

R_A	*3.0*	m
R_X	*3.0*	m
R_Y	*2.8*	m

PSM1271 Tillegra Dam IC 18/09/08

Forecasting Impulse Wavers in Reservoirs - A Huber & W Hager (1997)

SLIDE 2A - LOWER



SIMPLIFIED WAVE - 2D

(Eqn 3)

Variables

α	14 deg			
ρs	2.65 t/m3			
ρw	1 t/m3			
Vs	372,000 m3			
b	175 m			
d_A	50 m			
d_B	50 m			
d_C	30 m			
x_A	1700 m	γ_A	40 deg	
x_B	1900 m	γ_B	25 deg	
x_C	1250 m	γ_C	0 deg	

Height of Impulse Wave (2D)

H_A	*5.2* m
H_B	*5.0* m
H_C	*4.9* m

NB. Not valid as α less than 28 deg

3D IMPULSE WAVE PROPAGATION

(Eqn 6)

M_A	*0.9*
M_B	*0.9*
M_C	*2.4*

Height of Impulse Wave (3D)

H_A	*0.04* m
H_B	*0.04* m
H_C	*0.07* m

WAVE CELERITY ie, propagation velocity

(Eqn 8)

c_A	*22.2* m/s
c_B	*22.2* m/s
c_C	*17.2* m/s

WAVE RUN-UP ON SHORE

(Eqn 9)

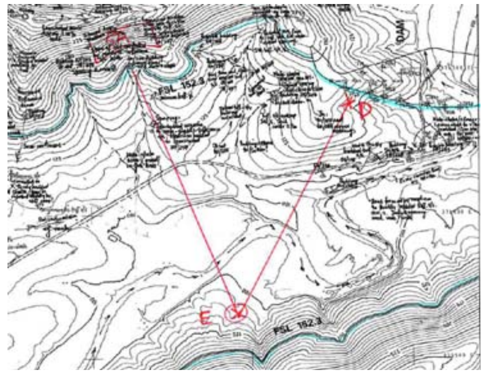
β_A	0.79 rad	R/d_A	*0.15*
β_B	0.79 rad	R/d_B	*0.15*
β_C	0.46 rad	R/d_C	*0.25*
L	*500* m		

Run-up height

R_A	*7.6* m
R_B	*7.4* m
R_C	*7.4* m

PSM1271 Tillegra Dam IC 18/12/08

Forecasting Impulse Wavers in Reservoirs - A Huber & W Hager (1997)



SLIDE 8A

SIMPLIFIED WAVE - 2D

(Eqn 3)

Variables

α	18.4 deg			
ρs	2.65 t/m3			
ρw	1 t/m3			
Vs	290,000 m3			
b	200 m			
d_D	50 m			
d_E	30 m			
d_{D1}	50 m			
x_D	1600 m	γ_D	0 deg	
x_E	800 m	γ_E	0 deg	
x_{D1}	700 m	γ_{D1}	60 deg	

Height of Impulse Wave (2D)	
H_D	*5.7* m
H_E	*5.9* m
H_{D1}	*7.0* m

NOTE: D1 - direct hit onto dam point D from slide; D - reflected off E

3D IMPULSE WAVE PROPAGATION

(Eqn 6)

M_D	*0.6*
M_E	*1.6*
M_{D1}	*0.6*

Height of Impulse Wave (3D)	
H_D	*0.05* m
H_E	*0.10* m
H_{D1}	*0.05* m

WAVE CELERITY ie, propagation velocity

(Eqn 8)

s_D	*22.2* m/s
s_E	*17.2* m/s
s_{D1}	*22.2* m/s

WAVE RUN-UP ON SHORE

(Eqn 9)

β_D	0.79 rad	R/d_D	*0.17*
β_E	0.10 rad	R/d_E	*0.22*
β_{D1}	0.79 rad	R/d_{D1}	*0.21*
L	500 m		

Run-up height	
R_D	*8.4* m
R_E	*6.6* m
R_{D1}	*10.6* m

NB. L is approx 10 times the water depth, d, in front of dam (P.1003)

APPENDIX H

POST FAILURE SLIDE VELOCITY CALCULATIONS

2. LIKELY LANDSLIDE VELOCITY

Using method in "*A decision analysis framework fro the assessment of likely post-failure velocity of translational; and compound natural rock slope landslides*", Glastonbury and Fell, Can Geot J Vol 45, pp.329-350 (2008)

Computed only for **LS 2A Upper** and **LS 2A Lower**

(see P5&6 of pdf files for decision tree results)

You have to answer several questions to progress through the decision tree. The probability value you get for each question is the sum of all the qualitative assessments you do in accordance with the paper.

Eg Q1. is failure surface at residual strength?
For LS 2A Upper

Indicator	weighting	Information?	Prob	Prob x weighting
Geomorphologic	1	Y	0.5	0.5
Subs displ. Mon	1	N	0.5	0.5
Lab test	0.5	N	0.5	0.25
Visual obs	0.5	Y	0.125	0.0625
Subs. Invest.	0.5	Y	0.125	0.0625
Surf. Mon.	0.5	N	0.5	0.25
Geolog.	0.33	Y	0.125	0.0413
SUM				**0.38**

NB. Where there is no information, the probability is 0.5 (default). Where there is information, the probability is a qualitative (and subjective) assessment.

Answer: 40% at residual, 60% not at residual. So you progress through the decision tree through "NO" (P=0.6), and so on.

END RESULT:
LS 2A Upper –

Velocity	**P**
Ext Slow – slow	0.15
moderate	0.2
Rapid	0.2
V rapid	0.45

LS 2A Lower –

Velocity	**P**
Ext Slow – slow	0.85 to 0.5
moderate	0.1 to 0.3
Rapid	0.05 to 0.15
V rapid	0 to 0.05

Fig. 4. Decision tree for assessment of the post-failure velocity of translational slides from natural rock slopes.

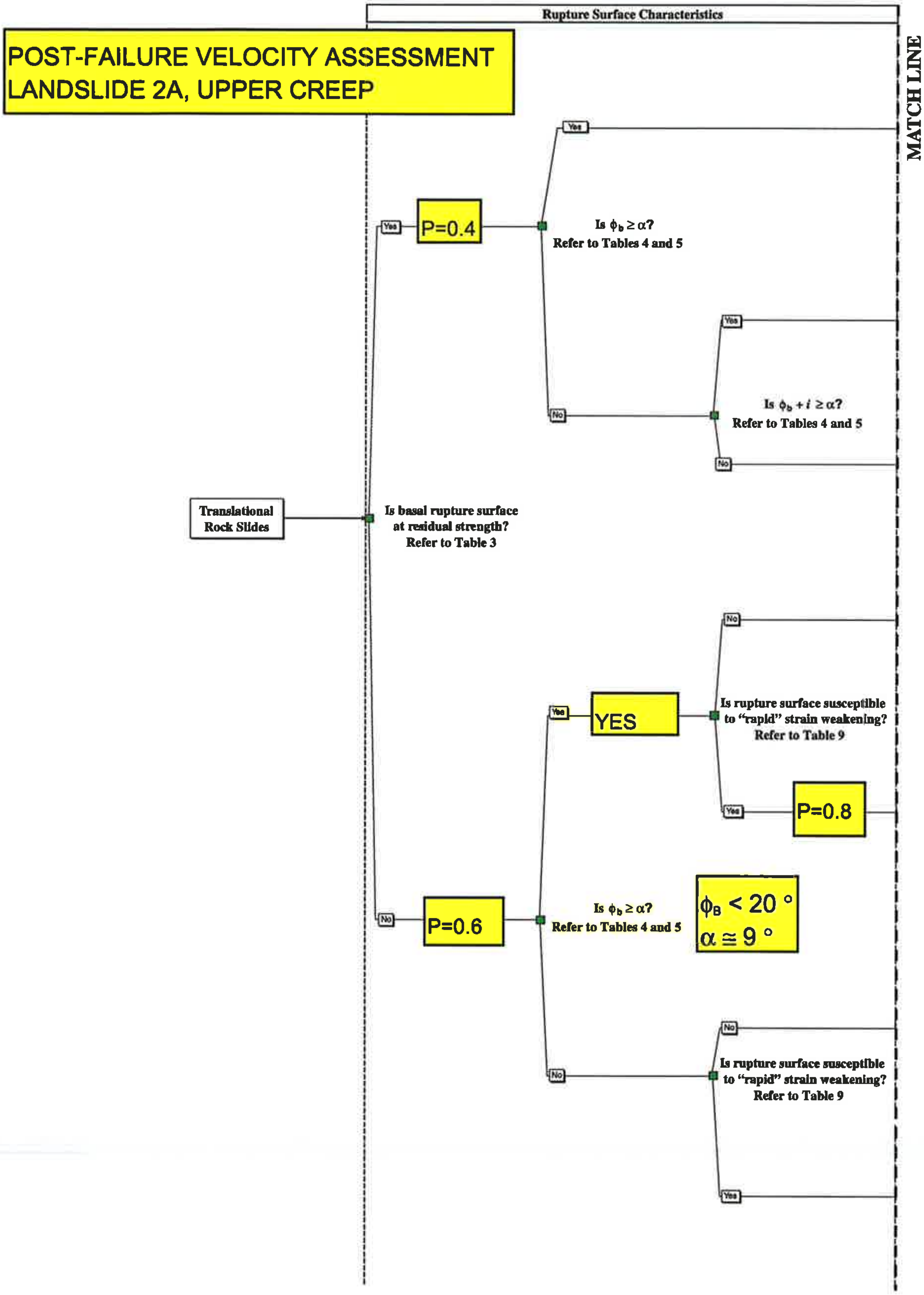


Fig. 4 (*concluded*).

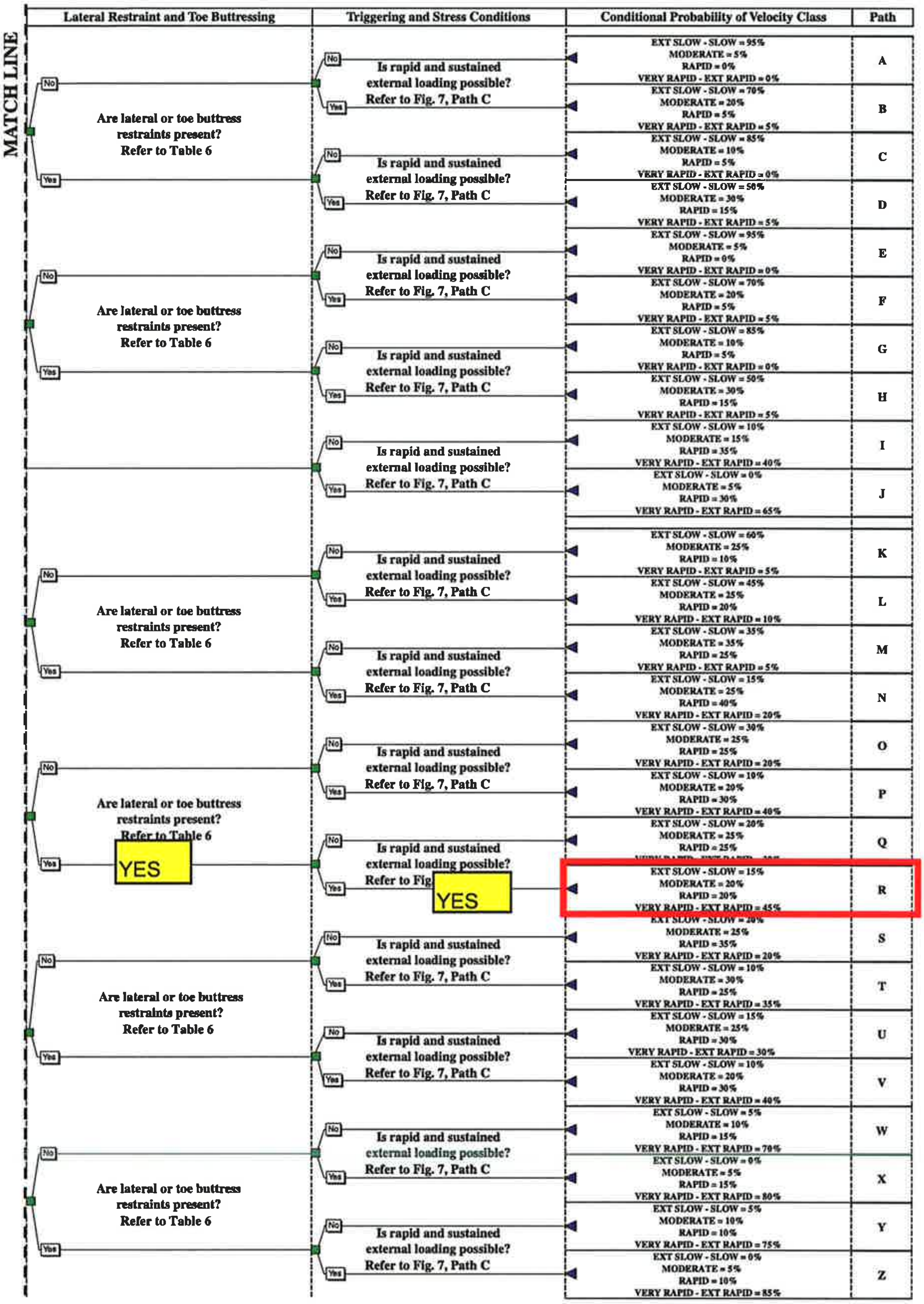


Fig. 4. Decision tree for assessment of the post-failure velocity of translational slides from natural rock slopes.

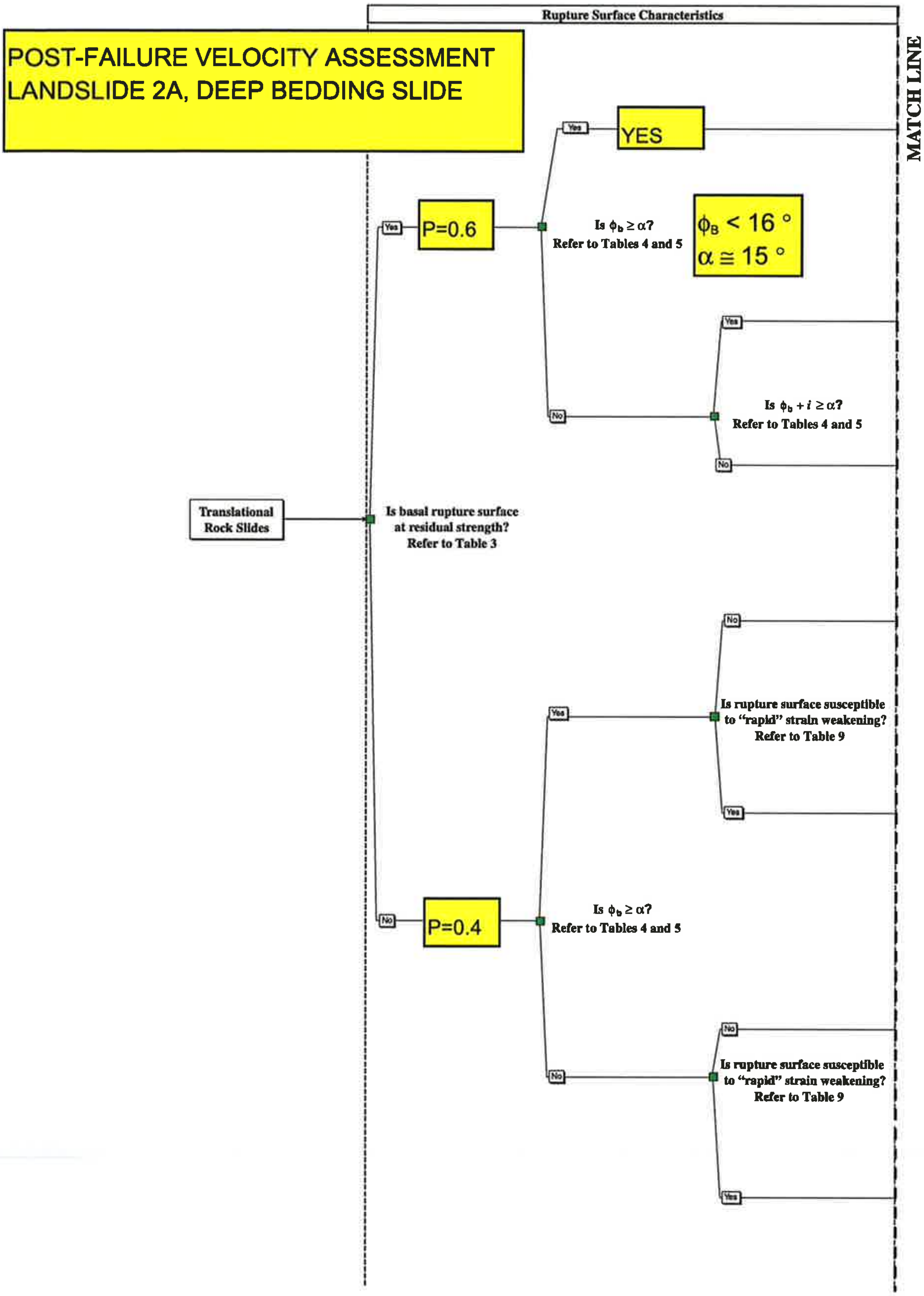
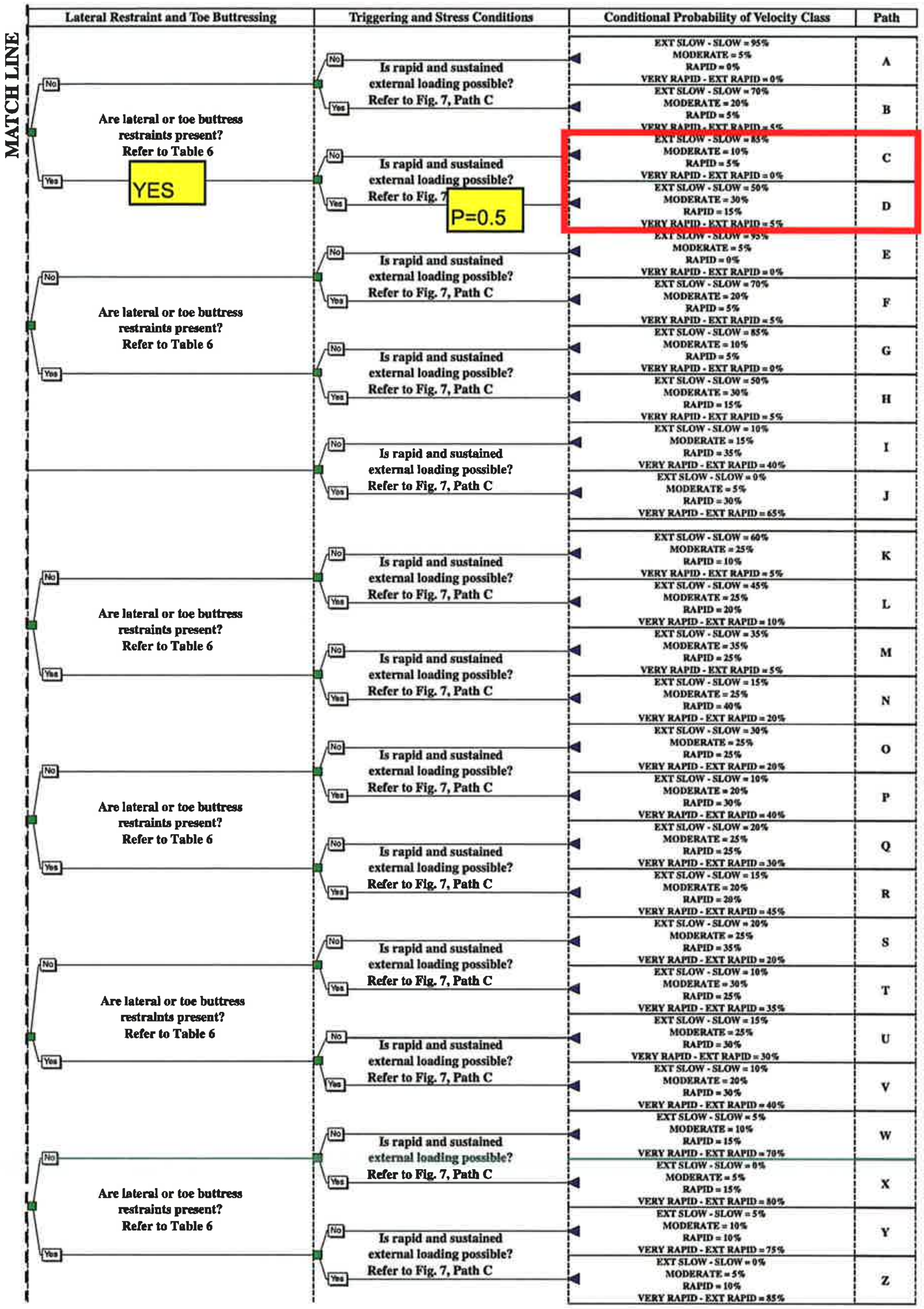


Fig. 4 (*concluded*).



APPENDIX I

SLIDE 5A ANALYSES

PSM

Pells Sullivan Meynink Pty Ltd

ABN 15 061 447 621

COMPUTATIONS

Job No. PSM1271	Sheet 2 of
Project Tillegra Dam	

Failure 5A.

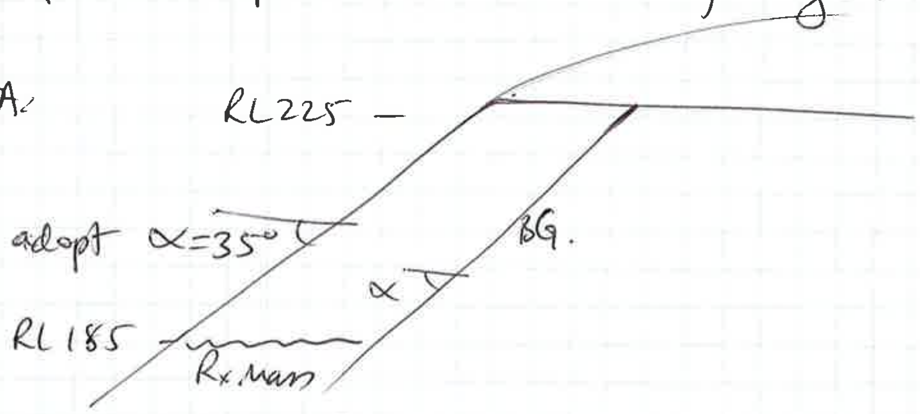
BG : 30-50°/230-250°M.
Mj Jt : 65-85°/320-345°M.

- Geot. Rept (July 08) p.16 + 17

Western side of Chichester Range:

- slopes flatter than bedding dip → stable slopes.
- Northern part of upper western slopes : // bd dip.
  → thickly bedded tuff SAST, massive sheets on steeper ~~slopes~~ slopes towards crest of ridge.

Failure 5A:



Extent : 200m wide, 55-90m down slope

check assumption $\boxed{\text{say up to 20m deep ?}}$

clay lab test literature

BG : c' = 0 φ' = 20° (backcalc. ~~worst case~~) / c'=0, φ'=15.3°

Rx Mass : c' = 3.5 MPa, φ' = 62° (RocLab)

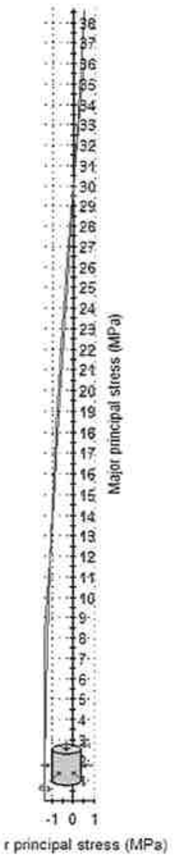
In "Slide" - assume RM c' = 0 → FoS = 1.5
c' = 3.5MPa → FoS = 6.2 } BG φ' = 20

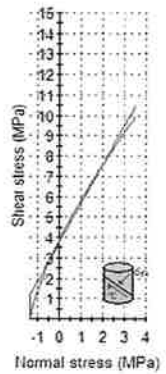
c' = 0 → FoS = 1.3
c' = 3.5MPa → FoS = 6.0 } BG φ' = 15.3°

By: IC | Date: 23/10/08

Copybook CB2039

Analysis of Rock Strength using RocLab





Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 70 MPa
GSI = 85 mi = 17 Disturbance factor (D) = 0
intact modulus (Ei) = 14000 MPa
modulus ratio (MR) = 200

Hoek-Brown Criterion
mb = 9.949 s = 0.1889 a = 0.500

Mohr-Coulomb Fit
cohesion = 3.712 MPa friction angle = 62.39 deg

Rock Mass Parameters
tensile strength = -1.329 MPa
uniaxial compressive strength = 30.403 MPa
global strength = 36.984 MPa
deformation modulus = 12972.30 MPa

PSM

Pells Sullivan Meynink Pty Ltd

By: IC

Date: 23/10/2008

Department of Commerce
TILLEGRA DAM

ROCK MASS ESTIMATE - FAILURE DOMAIN 5A

PSM1271	**Figure**

N

TRIG STN

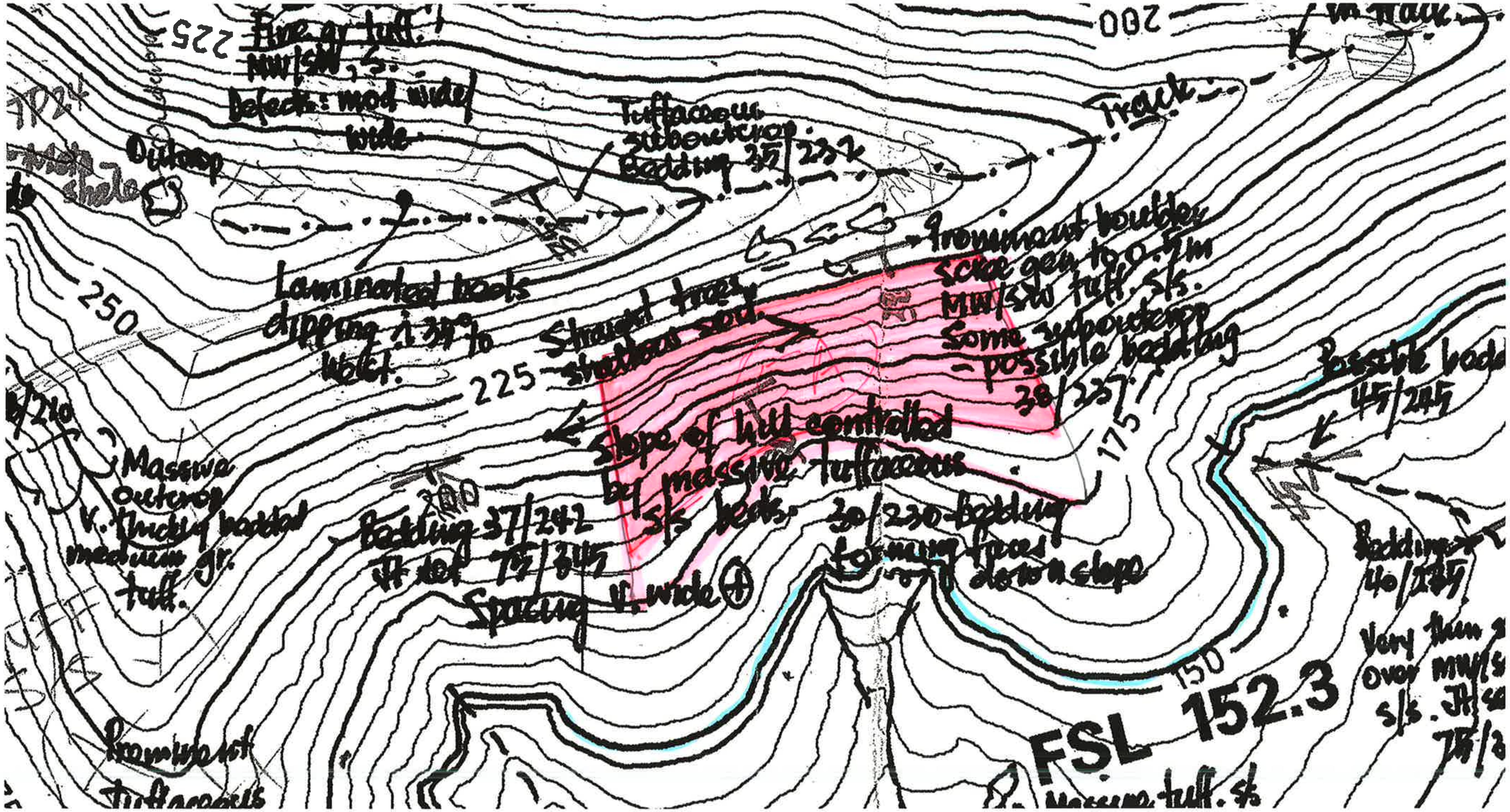
200

55

90

$100 \times (90+55) \times \text{Depth} = 500{,}000$

Depth = 35m.



75

18.4°

25

225

55m

36°

40m

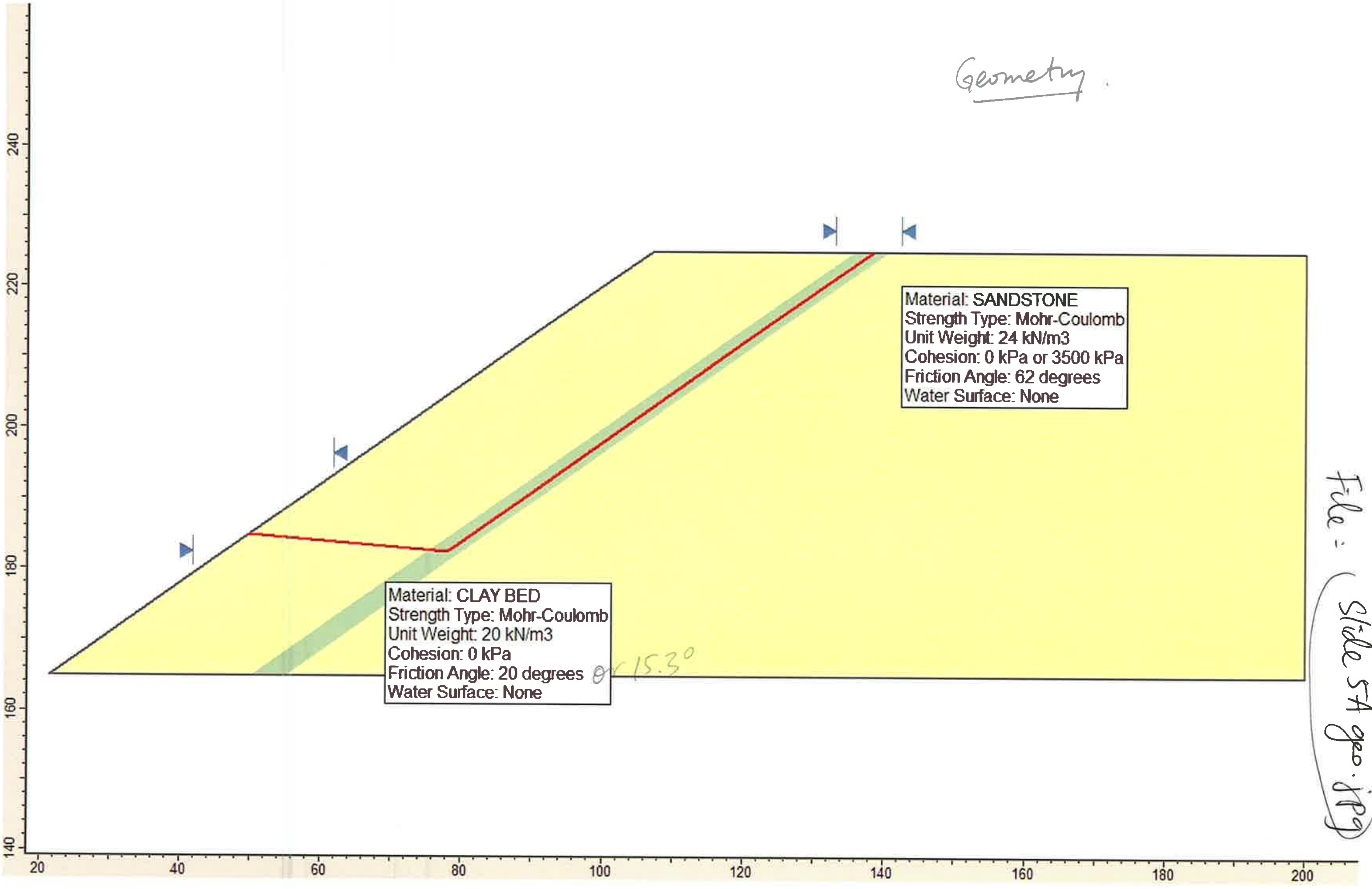
185

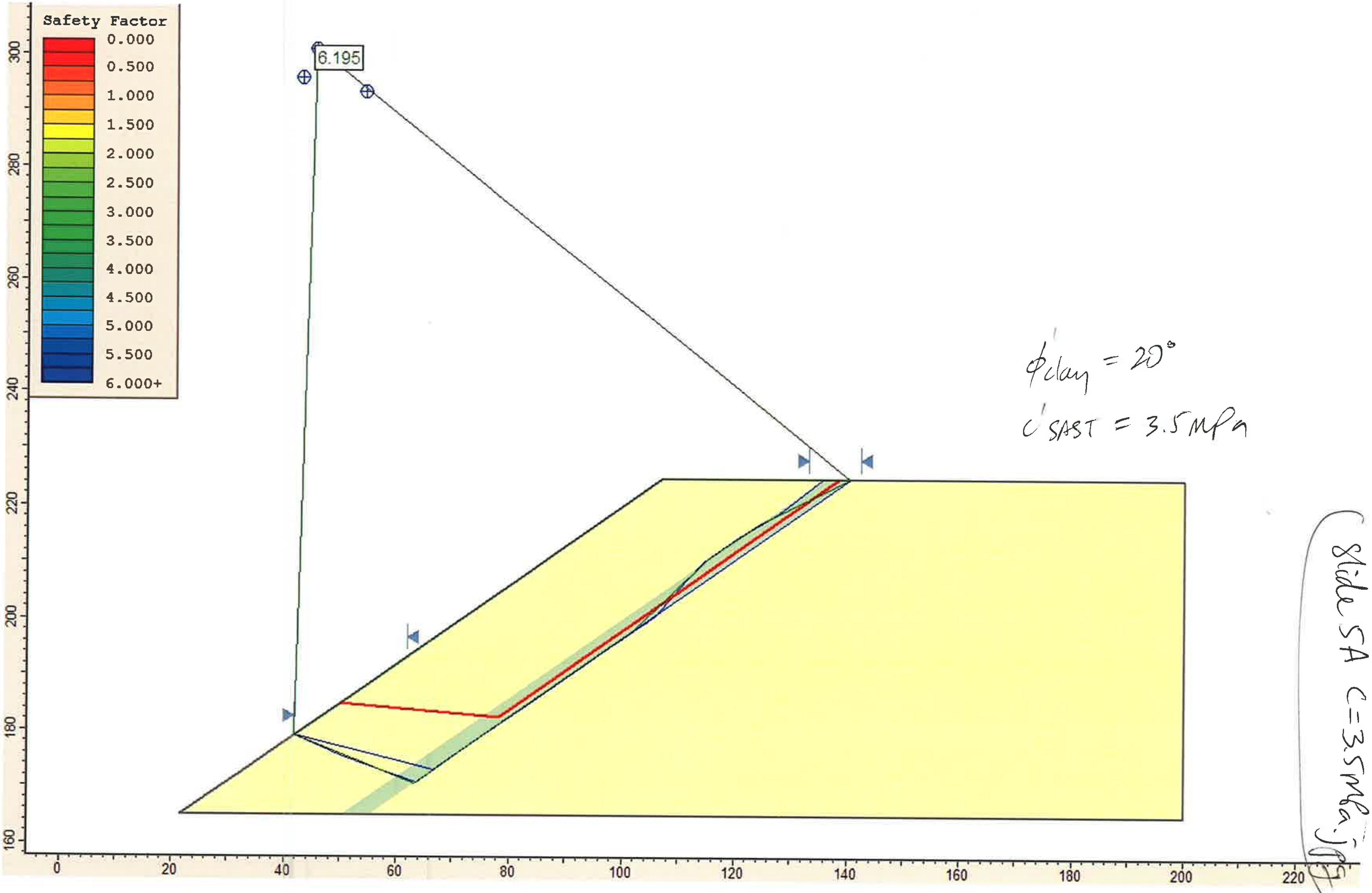
Scale 1:20,000

Vol ≤ 500,000 m^3

55

22 = 110 m.

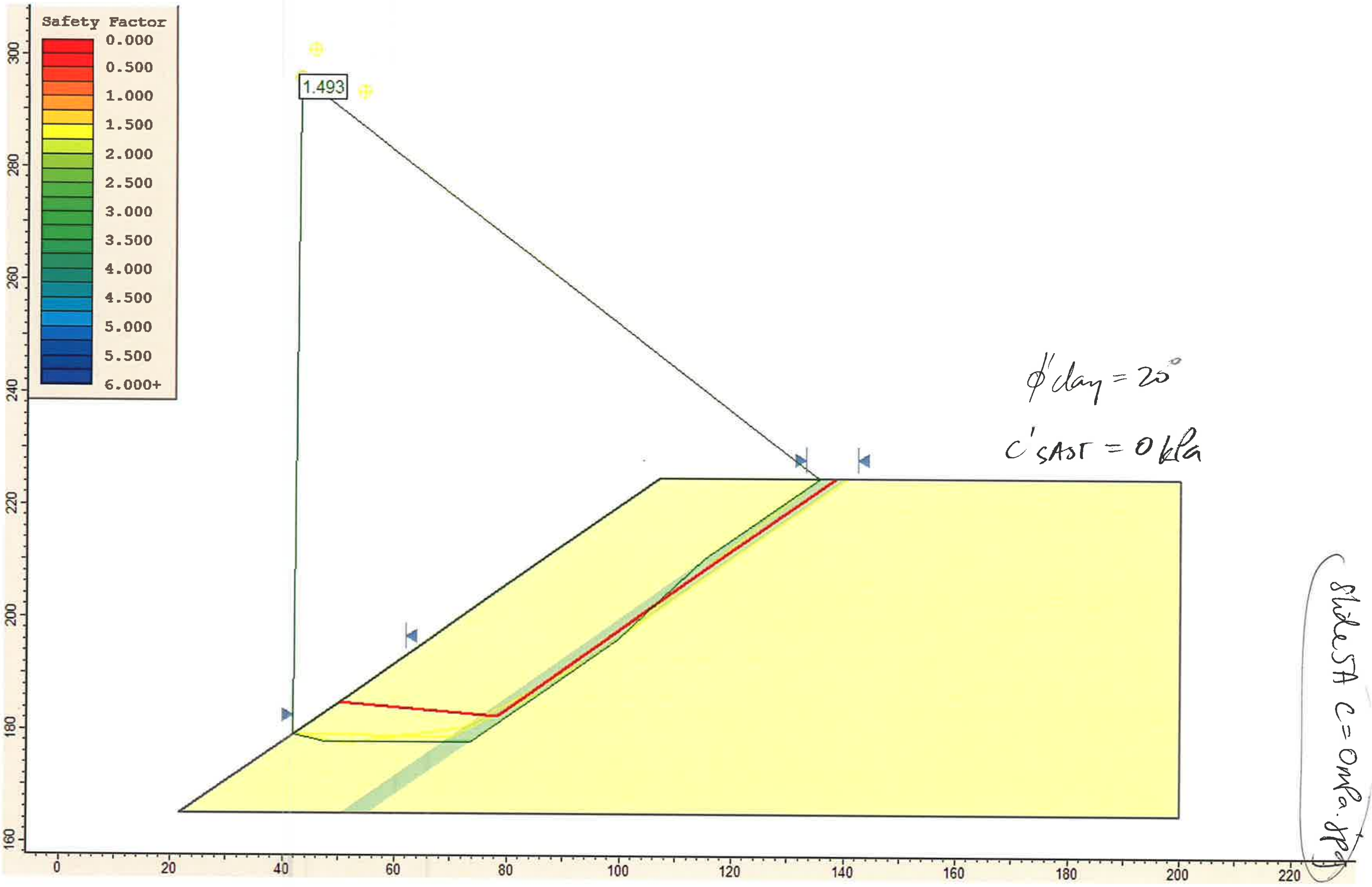

Geometry.
Material: SANDSTONE
Strength Type: Mohr-Coulomb
Unit Weight: 24 kN/m3
Cohesion: 0 kPa or 3500 kPa
Friction Angle: 62 degrees
Water Surface: None
Material: CLAY BED
Strength Type: Mohr-Coulomb
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 20 degrees or 15.3°
Water Surface: None
File: Slide 5A geo.jpg
240
220
200
180
160
140
20
40
60
80
100
120
140
160
180
200

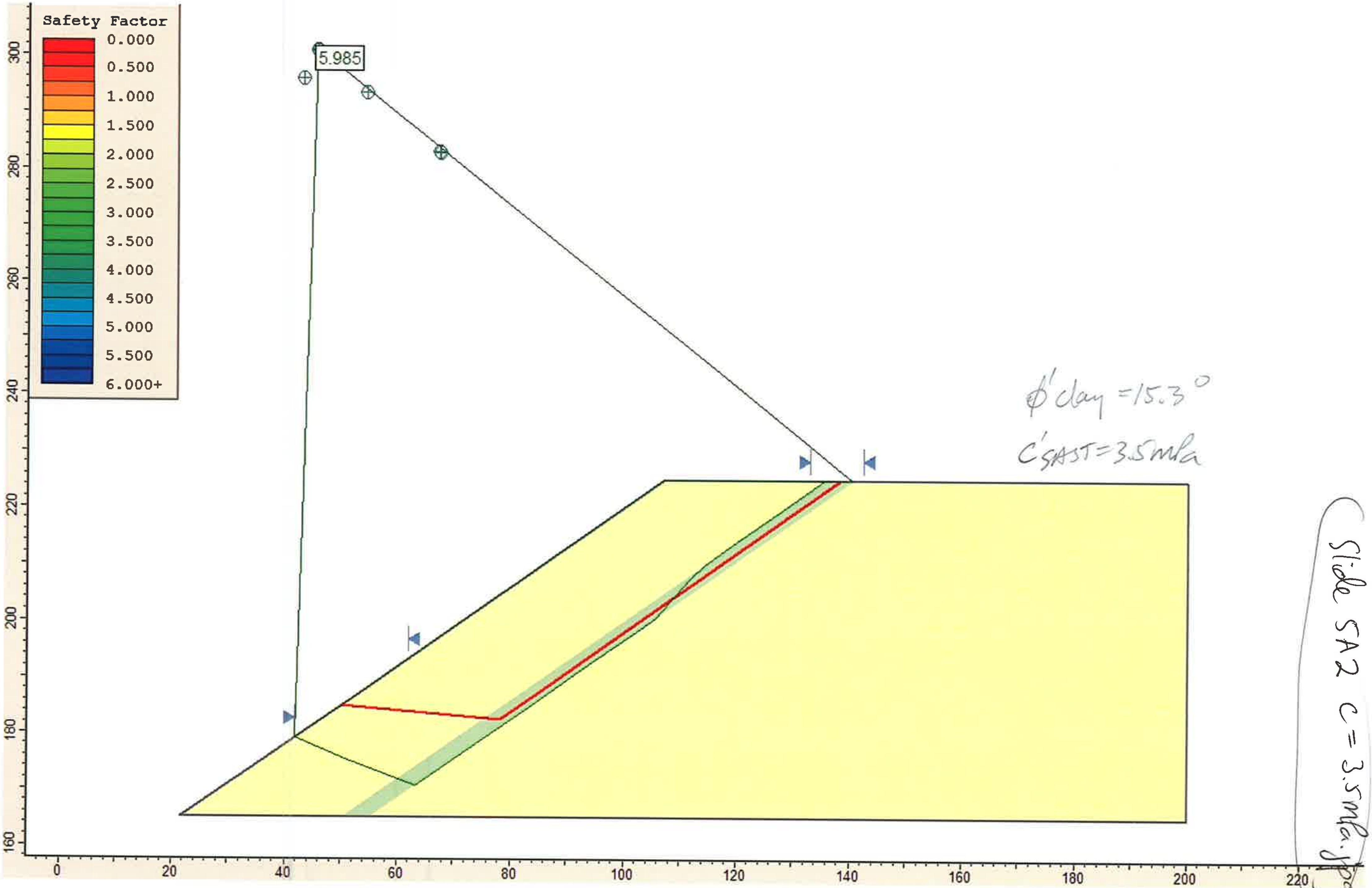


$\phi'_{clay} = 20°$

$c'_{SAST} = 3.5 MPa$

Slide SA c=3.5MPa.jpg

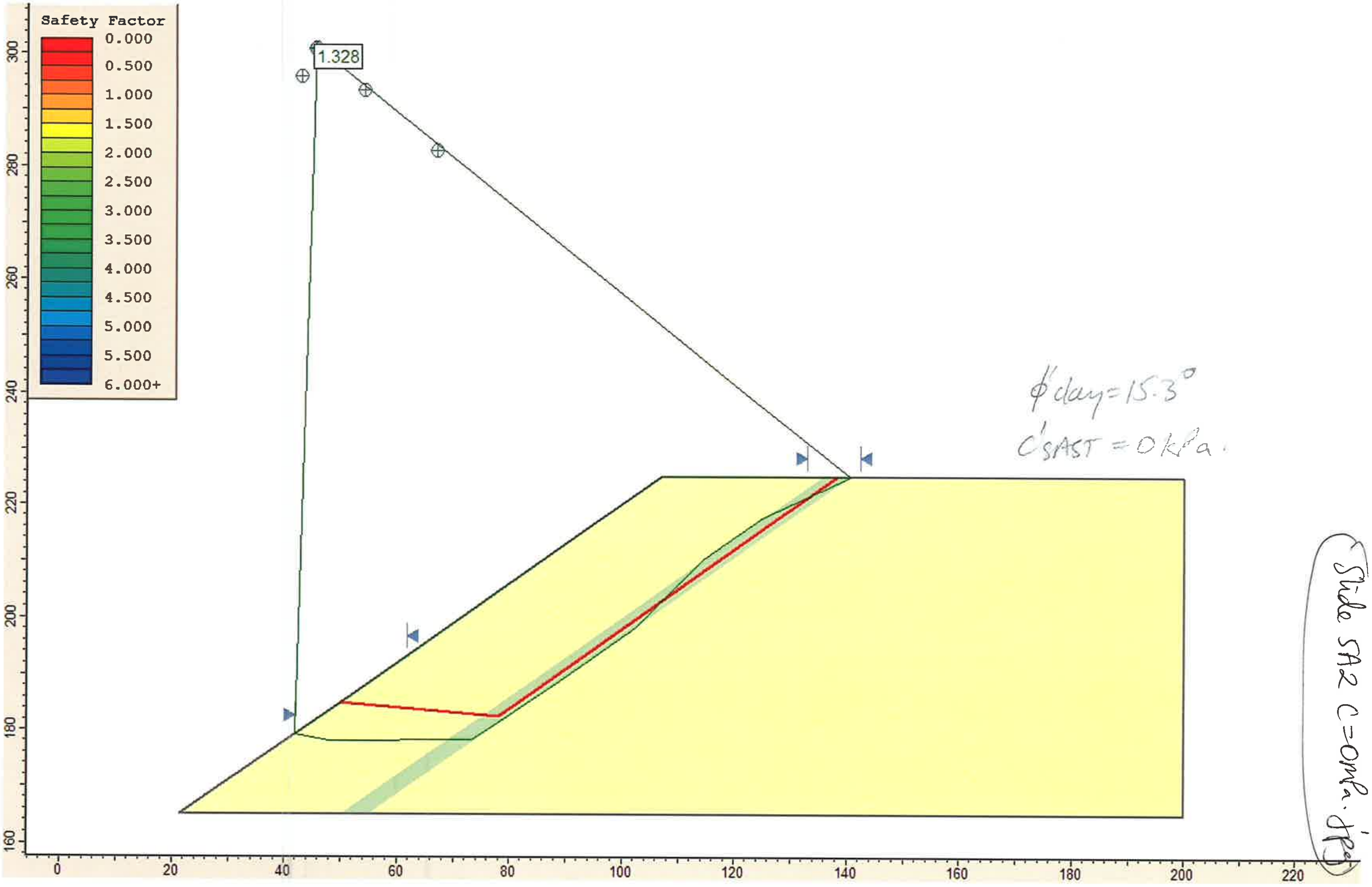

Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.493
φ'clay = 20°
c'SAST = 0 kPa
SlideSA c=0mPa.sp



ϕ' clay = 15.3°

c'_{SAST} = 3.5 mPa

Slide SA2 c = 3.5mPa.jpg


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.328
300
280
260
240
220
200
180
160
0
20
40
60
80
100
120
140
160
180
200
220
$\phi'_{clay} = 15.3°$
$c'_{SAST} = 0$ kPa.
Slide SA2 c=0kPa. jpeg

Earthquake Stability Assessment, Slide 5A –
Seismic Analysis. (Pseudo-static analysis)

Given – 1) Maximum Design EQ pga = 0.24g (1 in 10,000)
2) Maximum Credible EQ pga = 0.5g (1 in 100,000)

Pseudo-static analysis : put a horizontal seismic force to slope

→ what's the horizontal seismic coefficient?

Reference 1 : ANCOLD 'Interim Guidelines for Design of Dams for Earthquake' 1996

6.2 Pseudo-static analysis :

"US ACE ... recommend use of a seismic coefficient equal to one-half of peak ground acceleration ..." "They require a factor of safety greater than 1.0."

Reference 2 : European Standards Eurocode 8 : Design of Structures for Earthquake Resistance – Part 5 : Foundations, retaining structures and geotechnical aspects, (pr EN 1998-5), 2003

Ref. 3 & Part 1 : General Rules, seismic actions and rules for buildings (pr EN 1998-1), 2003

pr EN1998-1, 3.1.2 ground type – (Table 3.1)

A : rock or other rock-like geological formation, including at most 5m of weaker material at the surface. $V_{s,30}$ > 800 m/s.

pr EN1998-5, 4.1.3.3 (5)P

Design seismic inertia force F_H

$F_H = 0.5 \alpha S W$ (Eqn 4.1).

α = ratio of pga on type A ground (a_g) to ~~pga~~ acceleration of gravity, g.

S = soil parameter (pr EN 1998-1, 3.2.2.2)

W = weight of sliding mass

ie for MDE, $F_H = 0.5 \times \frac{0.24g}{g} \times 1.0 \times W$

for MCE, $F_H = 0.5 \times \frac{0.5g}{g} \times 1.0 \times W$.

<u>However</u> need to incorporate a topographic amplification factor for a_g → Annex A.

pr EN1998-5, Annex A Topographic amplification factor, S_T

A.2 Slope angle < 15° → $S_T = 1.0$

Slope angle 15-30° → $S_T \geq 1.2$ } at top of slope

Slope angle >30° → $S_T \geq 1.4$

In our case Slide 5A:

Slope angle ≈ 30°

Slide RL 185 - RL 225m

Top of slope (ridge) ~ RL 250m } 100m high slope

Bottom of slope (valley) ~ RL 150 m.

So slide in middle of slope.

COMPUTATIONS

Job No.	1271	Sheet	3 of 14
Project	Tillegra Dam (EQ Slide 5A)		

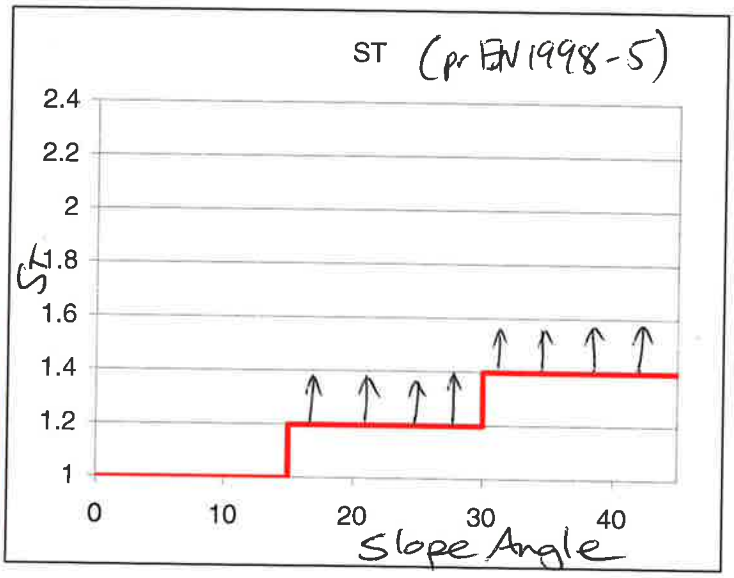
For conservatism – take Slide 5A as top of slope

What's S_T ? at angle = 30°, can be

$S_T \geq 1.2$

$S_T \geq 1.4$

at top of slope.



We take $S_T = 1.4$ for our slope.

Conclusion:

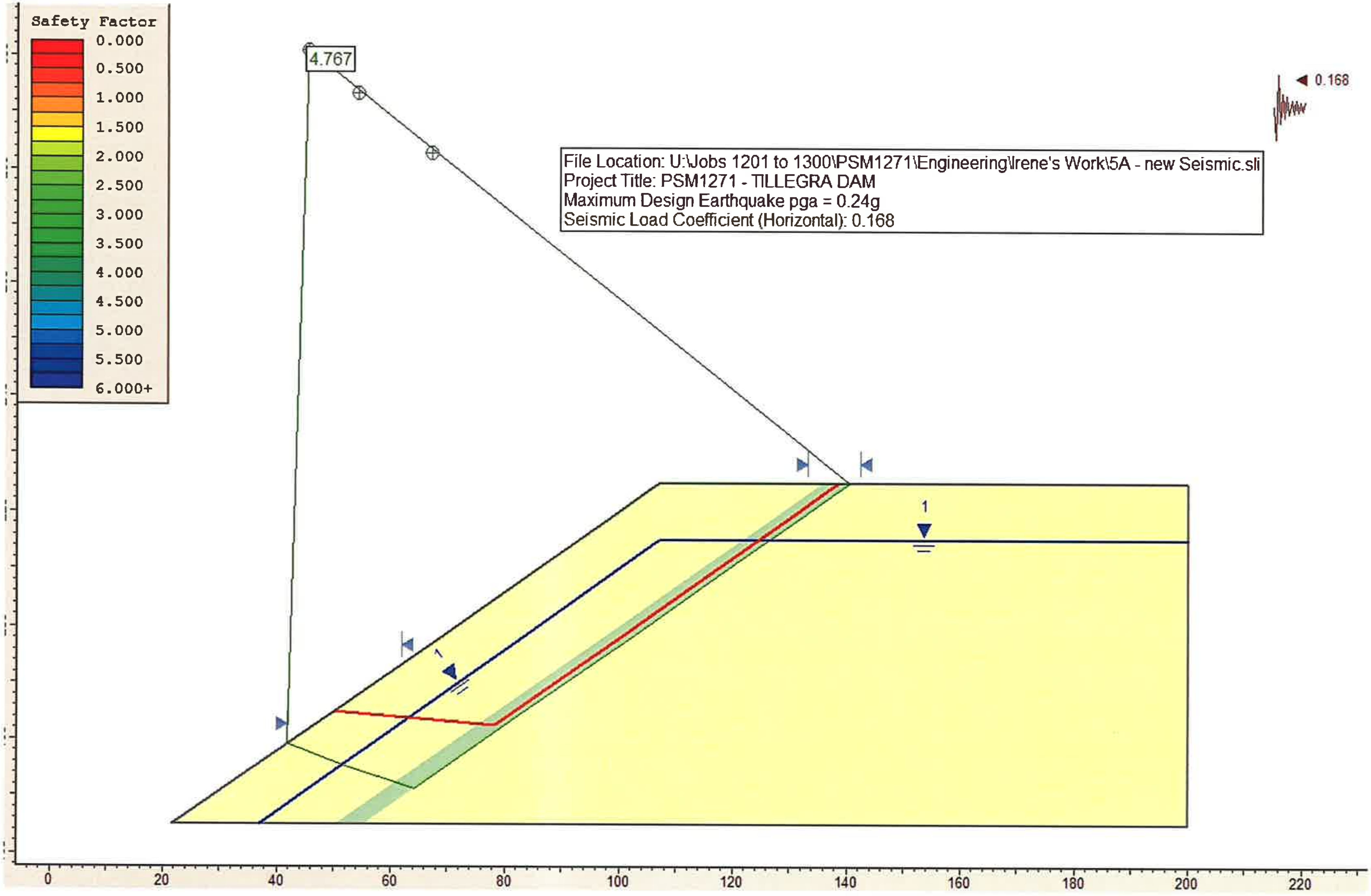
$F_H = 0.5 \; S_T \; \alpha \; S \; W$

For MDE $\quad F_H = 0.5 \times 1.4 \times 0.24 \times 1.0 \times W$

$= 0.168 \; W$.

For MCE $\quad F_H = 0.5 \times 1.4 \times 0.5 \times 1.0 \times W$

$= 0.35 \; W$.



Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
4.767
0.168
File Location: U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\5A - new Seismic.sli
Project Title: PSM1271 - TILLEGRA DAM
Maximum Design Earthquake pga = 0.24g
Seismic Load Coefficient (Horizontal): 0.168
1
1
0
20
40
60
80
100
120
140
160
180
200
220

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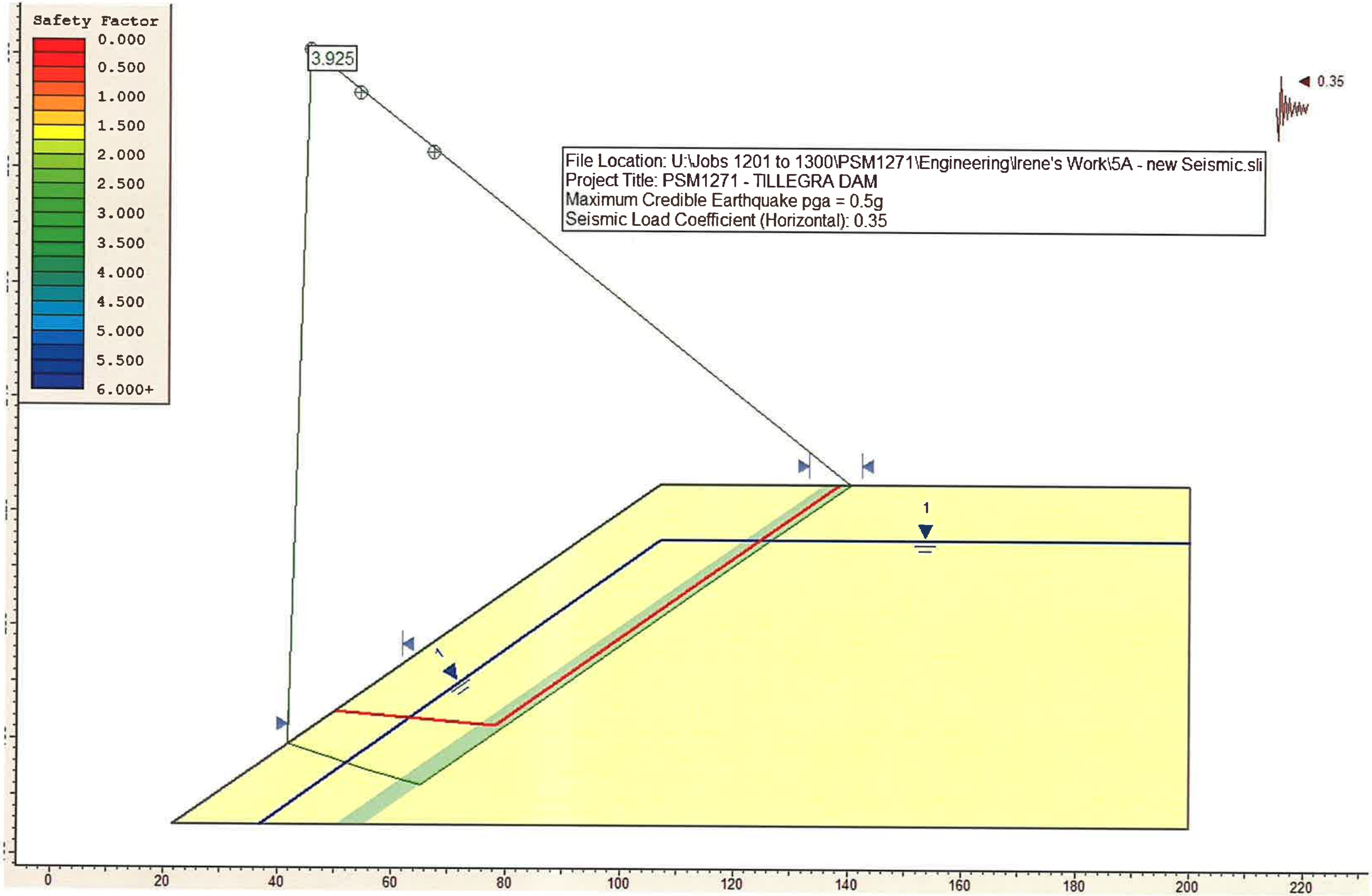

Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
3.925
0.35
File Location: U:\Jobs 1201 to 1300\PSM1271\Engineering\Irene's Work\5A - new Seismic.sli
Project Title: PSM1271 - TILLEGRA DAM
Maximum Credible Earthquake pga = 0.5g
Seismic Load Coefficient (Horizontal): 0.35
1
1
0
20
40
60
80
100
120
140
160
180
200
220

Reference 1 (ANCOLD, 1996) P.6/14

6.2 Pseudo-Static Analysis

Up until the 1970s, the pseudo-static analysis was the standard method of stability assessment for embankment dams under earthquake loading.The approach involved a conventional limit equilibrium stability analysis, incorporating a horizontal inertia force to represent the effects of earthquake loading. The inertia force was often expressed as a product of a seismic coefficient, k and the weight of the sliding mass W as shown in Figure 27. The larger the inertia force, the smaller the safety factor under the seismic conditions. In this approach, a factor of safety (FOS) of less than one implied failure, whereas FOS>1 represents seismically safe conditions, although as shown in Table 15, a higher factor of safety was often utilised to take into account material degradation under cyclic loading.

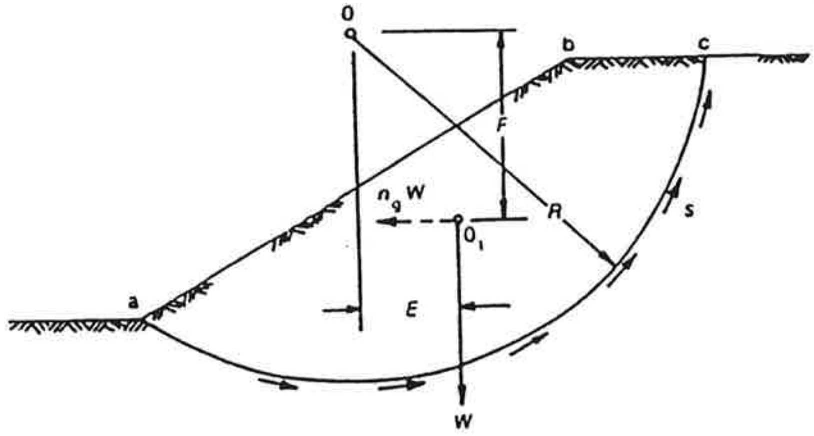

Figure 27. Pseudo-static method of assessing seismic stability of embankments.

The seismic coefficients used in this approach were typically less than 0.2 and were related to the relative seismic activity of the areas to which they apply. In the United States, for example, they ranged from 0.05 to 0.15. In Japan they have characteristically been less than about 0.2, and similar values have been used in other highly seismic regions throughout the world, as shown in Table 15.

Table 15. Seismic coefficients used in selected embankment dams (Seed, 1979).

Dam	Country	Horizontal seismic coefficient	Minimum factor of safety
Aviemore	New Zealand	0·1	1·5
Bersemisnoi	Canada	0·1	1·25
Digma	Chile	0·1	1·15
Globocica	Yugoslavia	0·1	1·0
Karamauri	Turkey	0·1	1·2
Kisenyama	Japan	0·12	1·15
Mica	Canada	0·1	1·25
Misakubo	Japan	0·12	—
Netzahualcoyote	Mexico	0·15	1·36
Oroville	USA	0·1	1·2
Paloma	Chile	0·12 to 0·2	1·25 to 1·1
Ramganga	India	0·12	1·2
Tercan	Turkey	0·15	1·2
Yeso	Chile	0·12	1·5

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For dams not susceptible to liquefaction, Seed (1979) suggested a seismic coefficient of 0.1g for magnitude 6.5 earthquakes and 0.15g for magnitude 8.25 earthquakes to obtain a safety factor of 1.15. However, he qualified this recommendation to point out that it applied to "most" earthquakes, and was "often adequate". The cases shown in Table 15 are all relatively old, and of interest only in giving the history of development of the methods of analysis.

The US Army Corps of Engineers (1984) have extended the basic pseudo-static method for use as a screening method for dams not susceptible to liquefaction. They recommend use of a seismic coefficient equal to one-half of peak ground acceleration and the use of undrained conditions for cohesive soils and drained conditions for free draining granular materials, with a 20 percent strength reduction to allow for strain weakening during the earthquake loading. They require a factor of safety greater than 1.0. If a dam fails to satisy this, more accurate and detailed analyses are required. Their approach has been calibrated against a large number of deformation analyses, and they state that up to 1m of deformation may occur.

The pseudo-static method of analysis, despite its earlier popularity, was based on a number of restrictive assumptions. For instance, it assumed that the seismic coefficient acting on the potential unstable mass is permanent and in one direction only. In reality, earthquake accelerations are cyclic, with direction reversals. Therefore, the concept it conveyed of earthquake effects on embankments was very inaccurate. Also, the concept of failure used in the approach was influenced by that used in static problems. It is clear that a factor of safety of less than one cannot be permitted under static conditions as the stresses producing this stage will exist until large deformations change the geometry of the structure. However, under seismic conditions, it may be possible to allow the FOS to drop below one, as this state exists only for a short time. During this time, earthquake induced inertia forces cause the potential unstable masses to move down the slope. However, before any significant movement takes place, the direction of the inertia forces is reversed and the movement of the soil masses stop and once again, the FOS rises above one. In fact, experience shows that a slope may remain stable despite having a calculated FOS less than one and it may fail at FOS>1, depending on the dynamic characteristics of the slope-forming material.

The deficiencies associated with the pseudo-static analysis were clearly demonstrated during the San Fernando Earthquake (M=6.6) in 1971. In this earthquake, the Lower San Fernando Dam experienced a massive slide in its upstream shell. This slide was very significant as the seismic stability of the Lower San Fernando Dam had been evaluated only five years before the earthquake and a number of reputable design agencies had concluded that the dam was safe against any earthquake that it might be subjected to. Clearly this was not the case and the dam failed despite having a pseudo-static FOS of around 1.3 because liquefaction occurred, with resultant large loss of shear strength. This near-disastrous event, more than any other single event, resulted in an extensive re-appraisal and gradual demise of the pseudo-static analysis.

Today, it is generally accepted that the pseudo-static analysis is not an accurate tool of seismic stability assessment of embankment dams, and that it may only be used as a screening tool (for dams not susceptible to liquefaction). It is recommended that the US Corps of Engineers (1984) method may be used as a screening method for well constructed earth and earth and rockfill dams, which are not susceptible to liquefaction or significant strain weakening in the dam or its foundation.

6.3 Simplified Methods of Deformation Analysis

6.3.1 Initial Screening

The US Bureau of Reclamation (1989) recommend that for dams not susceptible to liquefaction, dynamic deformations should not be a problem, and need not be analysed for such if the following conditions are satisfied.

Reference 3 R8/14

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

(2) The site should be classified according to the value of the average shear wave velocity, $v_{s,30}$, if this is available. Otherwise the value of N_{SPT} should be used.

(3) The average shear wave velocity $v_{s,30}$ should be computed in accordance with the following expression:

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \qquad (3.1)$$

Reference 2

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(2)P An increase in the design seismic action shall be introduced, through a topographic amplification factor, in the ground stability verifications for structures with importance factor γ_I greater than 1,0 on or near slopes.

NOTE Some guidelines for values of the topographic amplification factor are given in Informative Annex A.

(3) The seismic action may be simplified as specified in **4.1.3.3**.

4.1.3.3 Methods of analysis

(1)P The response of ground slopes to the design earthquake shall be calculated either by means of established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods subject to the limitations of (3) and (8) of this subclause.

(2)P In modelling the mechanical behaviour of the soil media, the softening of the response with increasing strain level, and the possible effects of pore pressure increase under cyclic loading shall be taken into account.

(3) The stability verification may be carried out by means of simplified pseudo-static methods where the surface topography and soil stratigraphy do not present very abrupt irregularities.

(4) The pseudo-static methods of stability analysis are similar to those indicated in EN 1997-1:2004, **11.5**, except for the inclusion of horizontal and vertical inertia forces applied to every portion of the soil mass and to any gravity loads acting on top of the slope.

(5)P The design seismic inertia forces F_H and F_V acting on the ground mass, for the horizontal and vertical directions respectively, in pseudo-static analyses shall be taken as:

$$F_H = 0{,}5\alpha \cdot S \cdot W \tag{4.1}$$

$$F_V = \pm 0{,}5 F_H \text{ if the ratio } a_{vg}/a_g \text{ is greater than 0,6} \tag{4.2}$$

$$F_V = \pm 0{,}33 F_H \text{ if the ratio } a_{vg}/a_g \text{ is not greater than 0,6} \tag{4.3}$$

where

α is the ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g;

a_{vg} is the design ground acceleration in the vertical direction;

a_g is the design ground acceleration for type A ground;

S is the soil parameter of EN 1998-1:2004, **3.2.2.2**;

W is the weight of the sliding mass.

A topographic amplification factor for a_g shall be taken into account according to **4.1.3.2** (2).

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(6)P A limit state condition shall then be checked for the least safe potential slip surface.

(7) The serviceability limit state condition may be checked by calculating the permanent displacement of the sliding mass by using a simplified dynamic model consisting of a rigid block sliding against a friction force on the slope. In this model the seismic action should be a time history representation in accordance with **2.2** and based on the design acceleration without reductions.

(8)P Simplified methods, such as the pseudo-static simplified methods mentioned in (3) to (6)P in this subclause, shall not be used for soils capable of developing high pore water pressures or significant degradation of stiffness under cyclic loading.

(9) The pore pressure increment should be evaluated using appropriate tests. In the absence of such tests, and for the purpose of preliminary design, it may be estimated through empirical correlations.

4.1.3.4 Safety verification for the pseudo-static method

(1)P For saturated soils in areas where $\alpha \cdot S > 0{,}15$, consideration shall be given to possible strength degradation and increases in pore pressure due to cyclic loading subject to the limitations stated in **4.1.3.3** (8).

(2) For quiescent slides where the chances of reactivation by earthquakes are higher, large strain values of the ground strength parameters should be used. In cohesionless materials susceptible to cyclic pore-pressure increase within the limits of **4.1.3.3**, the latter may be accounted for by decreasing the resisting frictional force through an appropriate pore pressure coefficient proportional to the maximum increment of pore pressure. Such an increment may be estimated as indicated in **4.1.3.3** (9).

(3) No reduction of the shear strength need be applied for strongly dilatant cohesionless soils, such as dense sands.

(4)P The safety verification of the ground slope shall be executed according to the principles of EN 1997-1:2004.

4.1.4 Potentially liquefiable soils

(1)P A decrease in the shear strength and/or stiffness caused by the increase in pore water pressures in saturated cohesionless materials during earthquake ground motion, such as to give rise to significant permanent deformations or even to a condition of near-zero effective stress in the soil, shall be hereinafter referred to as liquefaction.

(2)P An evaluation of the liquefaction susceptibility shall be made when the foundation soils include extended layers or thick lenses of loose sand, with or without silt/clay fines, beneath the water table level, and when the water table level is close to the ground surface. This evaluation shall be performed for the free-field site conditions (ground surface elevation, water table elevation) prevailing during the lifetime of the structure.

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3.2.2.2 Horizontal elastic response spectrum

(1)P For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions (see Figure. 3.1):

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \quad (3.2)$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \quad (3.3)$$

$$T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T}\right] \quad (3.4)$$

$$T_D \leq T \leq 4\text{s} : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C T_D}{T^2}\right] \quad (3.5)$$

where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_I . a_{gR}$);

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping, see **(3)** of this subclause.

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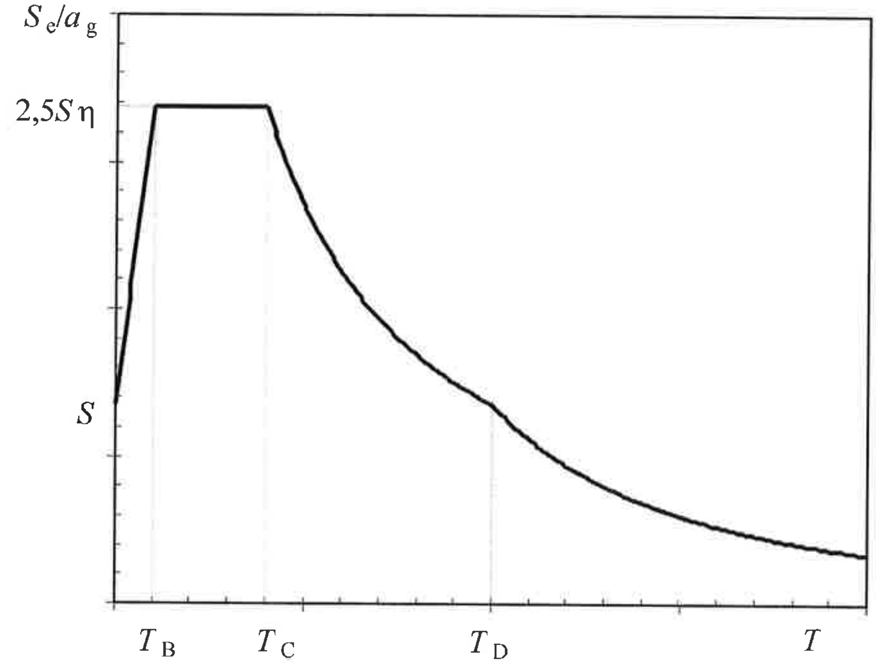

Figure 3.1: Shape of the elastic response spectrum

(2)P The values of the periods T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

NOTE 1 The values to be ascribed to T_B, T_C, T_D and S for each ground type and type (shape) of spectrum to be used in a country may be found in its National Annex. If deep geology is not accounted for (see **3.1.2(1)**), the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s, not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters S, T_B, T_C and T_D are given in Table 3.2 for the Type 1 Spectrum and in Table 3.3 for the Type 2 Spectrum. Figure 3.2 and Figure 3.3 show the shapes of the recommended Type 1 and Type 2 spectra, respectively, normalised by a_g, for 5% damping. Different spectra may be defined in the National Annex, if deep geology is accounted for.

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

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Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

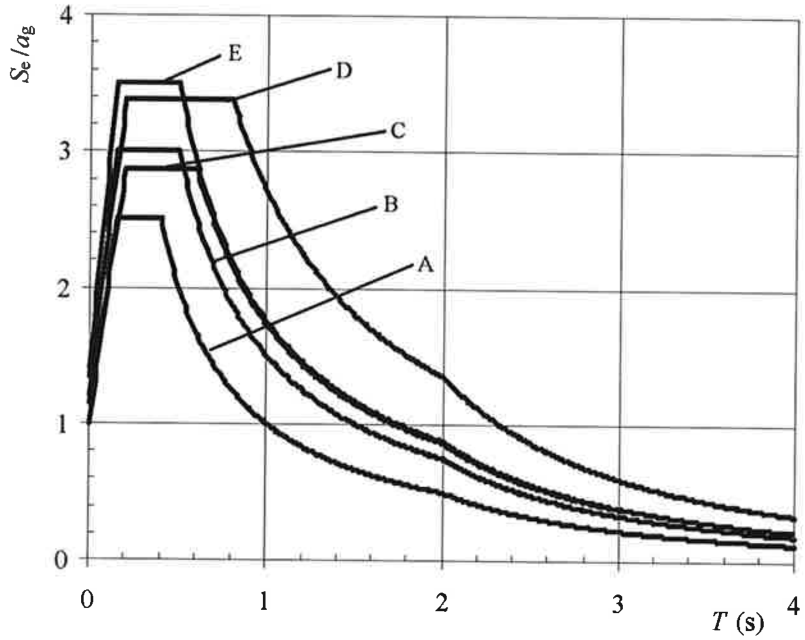

Figure 3.2: Recommended Type 1 elastic response spectra for ground types A to E (5% damping)

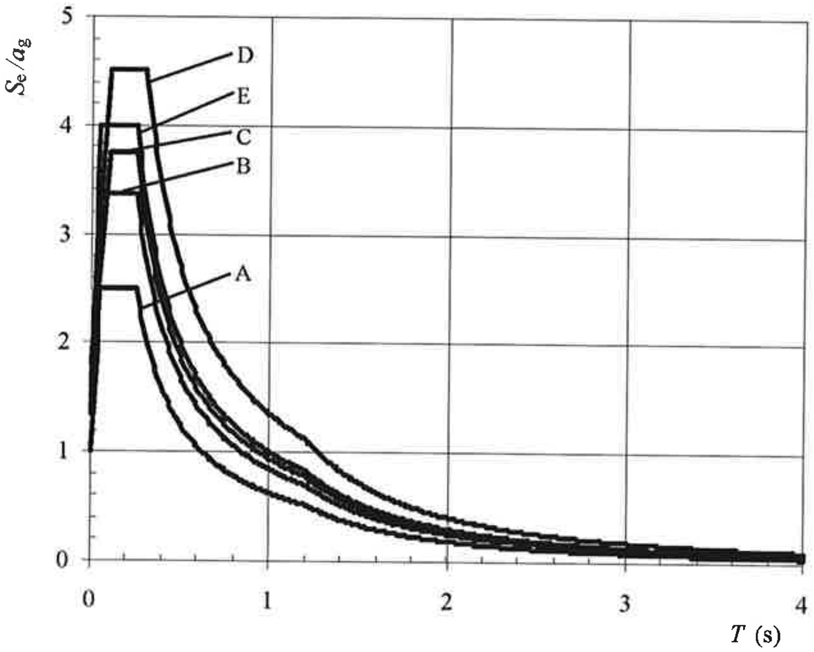

Figure 3.3: Recommended Type 2 elastic response spectra for ground types A to E (5% damping)

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Annex A (Informative)

Topographic amplification factors

A.1 This annex gives some simplified amplification factors for the seismic action used in the verification of the stability of ground slopes. Such factors, denoted S_T, are to a first approximation considered independent of the fundamental period of vibration and, hence, multiply as a constant scaling factor the ordinates of the elastic design response spectrum given in EN 1998-1:2004. These amplification factors should in preference be applied when the slopes belong to two-dimensional topographic irregularities, such as long ridges and cliffs of height greater than about 30 m.

A.2 For average slope angles of less than about 15° the topography effects may be neglected, while a specific study is recommended in the case of strongly irregular local topography. For greater angles the following guidelines are applicable.

a) *Isolated cliffs and slopes.* A value $S_T \geq 1{,}2$ should be used for sites near the top edge;

b) *Ridges with crest width significantly less than the base width.* A value $S_T \geq 1{,}4$ should be used near the top of the slopes for average slope angles greater then 30° and a value $S_T \geq 1{,}2$ should be used for smaller slope angles;

c) *Presence of a loose surface layer.* In the presence of a loose surface layer, the smallest S_T value given in a) and b) should be increased by at least 20%;

d) *Spatial variation of amplification factor.* The value of S_T may be assumed to decrease as a linear function of the height above the base of the cliff or ridge, and to be unity at the base.

A.3 In general, seismic amplification also decreases rapidly with depth within the ridge. Therefore, topographic effects to be reckoned with in stability analyses are largest and mostly superficial along ridge crests, and much smaller on deep seated landslides where the failure surface passes near to the base. In the latter case, if the pseudo-static method of analysis is used, the topographic effects may be neglected.