coffey geotechnics
SPECIALISTS MANAGING THE EARTH

REVIEW OF MANAGEMENT OPTIONS FOR FOUR RIVER BANK COLLAPSE HIGH RISK SITES

Department of Environment, Water and Natural Resources

07093AA-AD Final report
12 December 2012

Coffey Geotechnics Pty Ltd ABN 93 056 929 483
Level 1, 33 Richmond Road Keswick SA 5035 Australia
T (+61) (8) 8375 4400 F (+61) (8) 8375 4499 www.coffey.com.au

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

12 December 2012

Project Manager, Hazards Team, Operations and Major Programs
Department of Environment, Water and Natural Resources
GPO Box 2834
ADELAIDE SA 5001

Attention: Mr Jai O'Toole

Dear Sir

RE: REVIEW OF MANAGEMENT OPTIONS FOR FOUR RIVER BANK COLLAPSE HIGH RISK SITES
FINAL REPORT

Please find attached our final report on the above project.

If you have any questions please contact Richard McKenna or the undersigned.

For and on behalf of Coffey Geotechnics Pty Ltd

Alan Moon

Alan Moon

Senior Principal

Distribution: Original held by Coffey Geotechnics Pty Ltd

1 electronic copy to Department of Environment, Water and Natural Resources

SUMMARY

At the request of the former Department for Water (DFW) now the Department of Environment and Natural Resources (DEWNR), Coffey Geotechnics Pty Ltd (Coffey) has a carried out a review of management options for four sites on the banks of the River Murray which have been judged to be at high risk because of the potential for riverbank collapse. This report describes the results of the review and outlines recommended long-term management options for the four sites.

Relatively large collapses of the banks of the River Murray started in February 2008 as a result of very low water levels in the lower reaches of the river. The location and timing of the collapses are difficult to predict and the collapses can occur quickly and with little warning. In response to the collapses DEWNR sought geotechnical advice and restricted public access to some of the affected sites. Basin inflows have restored river levels to the normal operating range and there are now fewer reported riverbank collapses. However some sites continue to show instability and are likely to require on-going management or significant intervention to reduce the likelihood of failure. The four sites reviewed in this study are; East Front Road, Mannum; Caloote Southern Residential Area; Woodlane Reserve and River Front Road, Murray Bridge.

While there has been a great deal of previous work on the hazards our approach to this review has been to independently develop our understanding of the hazards rather than rely on the interpretations and opinions of others. This has involved using information from previous work but also assembling and interpreting other information (such as bathymetry, topography and aerial photographs). We have also carried out a field overview of other sites where collapse or cracking has occurred to help understand the history of collapses on the river.

The main collapse failure mechanism is slope failure in the soft and very soft clays of Holocene age (referred to as Soft Clay in this report). The Soft Clay was deposited when the sea level returned to its present level after the last glaciation. The Soft Cay is underlain by stiffer soil and rock. On sites near the edge of the flood plain the Soft Clay is overlain by colluvium and in many areas there is fill associated with house, roads and other infrastructure.

The slope failures that have occurred can be classified into four categories (ranging from very rapid large collapses to relatively minor cracking) depending on the magnitude and speed of the slope movement. Sites can also be classified into three land use categories (high intensity infrastructure, open public access and restricted use). The slope failure and land use categories have been combined to form a risk matrix which helps understanding of how the risk to life, assets and navigation vary with the different kinds of slope failure and land use. With respect to response to risk we understand that DEWNR is seeking advice on reasonable practicable precautions to reduce or avoid risk.

Back analyses of some of the larger past failures have been carried out in order to better understand conditions which lead to instability, how the slopes fail and why large riverbank regressions can occur. The analyses have shown that unusually low river levels cause a large reduction in stability and appear to be the major precursor of the riverbank collapses. Small variations in strength of the Soft Clay also have a big effect on stability. Fill on the bank reduces the stability but recent experience has shown that riverbank collapse can occur where there is no fill. The collapses that cause large regressions are probably the result of progressive failure (i.e. a rapid succession of collapses).

In our opinion, on present knowledge, it should be assumed that during periods of low river level riverbank collapse could occur wherever the bank is underlain by Soft Clay. Sites which have already been affected by significant cracking may be particularly vulnerable to collapse. Elsewhere, where there has only been relatively minor cracking (or no reported cracking), collapse may be more likely where there is fill but can also occur where there is little or no fill.

It is usual practice to take a staged approach to the investigation, design and construction of major engineering works. In our experience this is a particularly cost effective approach for remedial works in variable ground conditions where a variety of options need to be considered. A staged approach allows time for informed decisions to be made by the stakeholders and the optimum remedial option to be adopted. Selecting options, or proceeding to construction, without adequate knowledge of ground conditions and other constraints (such as safety and environmental) often results in significantly higher costs and less effective solutions. With respect to the remediation options, this project is at the prefeasibility/concept stage. The proposed geotechnical investigations discussed in this report represent the start of the preliminary design stage.

The hazards and the risks are discussed for the four sites based on current knowledge. At three sites we have recommended further geotechnical investigations to better understand the distribution (presence and depth) of Soft Clay and we have discussed remediation options. In our opinion, at the fourth site remediation is impracticable. We have estimated the costs of the further geotechnical investigations and provided some indication of the range of costs for remediation options. The remediation costs are indicative only and they will need to be reviewed and revised after the further investigations.

East Front Road, Mannum is located on fill overlying a wedge of soft clay overlying granite. A section of road has been closed since April 2010 due to tension cracks in several places. In our opinion the road close to the river is vulnerable to collapse when the river level is low and this could result in injury or death to a road user. Various risk management options have been previously considered. We recommend that further consideration should be given to lowering the road to improve the stability. Further geotechnical investigations are required and depending on the results of the investigation other options could be considered. We estimate that the cost of the investigations is likely to be in the range of $60,000 to $90,000. At this stage we estimate that the indicative design and construction costs for lowering the road is likely to be in the range of $800,000 to $2.2 million depending on the type of road (width, sealed or unsealed) required.

The Caloote Southern Residential Area consists of six houses between a limestone ridge and the river. The area is underlain by a wedge of Soft Clay which may extend inland as far as the front of the houses. The limestone cliff overhangs by at least 3 m behind two of the houses. Collapse of the cliff could severely damage the houses and injure or kill any occupants. In our opinion the residents should be warned of the rock fall risk and advised to seek their own independent advice. In our opinion the area in front of the houses is vulnerable to riverbank collapse when the river level is low. On present knowledge it appears to be feasible to carry out engineering works (e.g. piling or other retaining structures) at the front of the houses to reduce or eliminate the effects of riverbank collapse. The cost of further geotechnical investigations is likely to be in the range of $50,000 to $80,000. At this stage we estimate that indicative design and construction costs for a retaining structure located 5 to 6 m in front of the houses are likely to be in the range of $600,000 to $1 million.

There are two areas of concern at Woodlane Reserve. One is a grassed area underlain by a wedge of Soft Clay in front of houses upstream of a shallow lagoon where small riverbank failures have occurred. In our opinion, the area in front of the houses is vulnerable to collapse when river levels are low. Further geotechnical investigation is required to assess whether remedial measures are required to support parts of the access road close to the houses. The second area of concern is the access road which crosses the lagoon close to where two large collapses destroyed the pumpstation. This area is also underlain by a wedge of Soft Clay. In our opinion further regression could result in local failure of the access road and consideration should be given to moving the road inland. The total cost of both geotechnical investigations is likely to be in the range of $70,000 to $100,000. At this stage we estimate that the indicative cost of realigning the access road crossing the lagoon is $400,000 to $600,000.

River Front Road in Murray Bridge is downstream of Sturt Reserve and about 1.5 km upstream of Long Island Marina where four large riverbank collapses occurred in 2008 and 2009. The area is underlain by Soft Clay to depths of up to at least 20 m. Cracks have been observed parallel to the river. In our opinion, the riverbank (including the houses) is vulnerable to collapse when river levels are low and the lives of residents are at risk. Engineering solutions to reduce the likelihood of riverbank collapse and the risk to life of the occupants of the houses are likely to be prohibitively expensive. Land based sheet piling or other structural solutions would be extremely expensive (indicative costs in the range of $8 to $12 million) and very difficult to construct. Some of the existing structures would probably have to be demolished to allow access for plant and equipment. Works undertaken from the water would be very much more expensive.

At many places along the River Murray there are open public access areas that are vulnerable to collapse when the river level is low. In our opinion it would be useful to develop a long term overall approach to managing the risks associated with these areas. An important component of long term risk management in areas of open public access will be development controls including limiting construction of new infrastructure, avoiding placing more fill, and if possible managing the river so that very low river levels do not occur.

Another important component of long term management will be monitoring. For such monitoring to be effective it will need to be unobtrusive so that it is not vulnerable to damage or vandalism. It will also need to be robust so that it will last for many years and sites can be resurveyed in the future even if there has been a break in regular monitoring. More sophisticated monitoring (including real time monitoring linked to warning systems) can be useful in the shorter term for high risk sites. However, such systems are often not well maintained in the longer term. In our opinion, for higher risk sites, remediation or other risk management options that eliminate or greatly reduce the risk are usually preferable.

CONTENTS

CONTENTS

1 INTRODUCTION

1.1 Purpose

At the request of the former Department for Water (DFW) now the Department of Environment, Water and Natural Resources (DEWNR), Coffey Geotechnics Pty Ltd (Coffey) has carried out a review of management options for four sites on the banks of the River Murray which have been judged to be at high risk because of the potential for riverbank collapse (Contract reference, DFW0892/11, dated 28 June 2012). This report describes the results of the review and outlines recommended long-term management options for the four sites. The report also includes recommendations for further work which will help understand the hazard and manage the risk.

1.2 Background

Relatively large collapses of the banks of the River Murray started in February 2008 as a result of very low water levels in the lower reaches of the river (from Lock 1 at Blanchetown downstream to Wellington). The location and timing of the collapses are difficult to predict and the collapses can occur quickly and with little warning. At some sites (e.g. Long Island Marina in Murray Bridge) riverbank collapse has resulted in bank retreats of up to about 30 m.

The riverbank collapse hazard poses a direct risk to people, property, public utilities including roads on the banks of the river and also can seriously affect navigation (particularly where trees are involved). In response to the collapses and the potential for further collapses, DEWNR sought geotechnical advice and has cordoned off and restricted public access to some of the affected sites. The site closures are an interim measure to reduce risk and do not address remaining site instability and long-term management. Until a management or permanent mitigation solution is found some sites remain closed.

Basin inflows in late 2010 and early 2011 have restored River Murray water levels to within a normal operating range (approximately +0.75 m AHD) and there are now fewer reported riverbank collapses. However some sites continue to show instability and may require on-going management or significant intervention to reduce the likelihood of failure.

In this report recommendations for long-term management are given for the following four sites previously judged to be at high risk because of the potential for riverbank collapse:

- East Front Road, Mannum
- Caloote Southern Residential Area
- Woodlane Reserve
- River Front Road, Murray Bridge

Brief descriptions of the history of these sites and why they were judged to be of concern are given in Section 5 of this report.

1.3 Previous work

There has been a great deal of previous work on the riverbank collapse hazards by Sinclair Knight Merz (SKM), Golder Associates (Golder), other consultants, government agencies, universities and others. A list of the key references used during this study is given in Appendix A.

Most of the previous geotechnical work at the four sites in question has been carried out by SKM. Golder has largely had a review role but they have also provided advice on particular issues at some of the sites. The February 2010 SKM Geotechnical investigation report (Study into riverbank collapsing for Lower River Murray, February 2010) is the major source of subsurface geotechnical information.

1.4 Our approach to this review

The overall purpose of this review is to provide informed advice on managing the risks associated with riverbank slopes. In our experience understanding the hazards (e.g. what has happened, why it has happened and what else might happen) is the most important (and usually the most time consuming) activity in assessing and managing slope risks. Without a thorough understanding of the hazards and their consequences it is not possible to understand the risks and attempts at management and remediation may be ineffective or unnecessary or even counterproductive. As discussed in Reference 2, assessments of landslide (e.g. riverbank collapse) likelihood are evidence-based judgements dependent on the knowledge and skills of the assessor. There is no right answer (unique probability) that can be found by analysis and different assessors may come up with different judgements of the probability and consequences of slope failure.

Our approach to this review has been to independently develop our understanding of the hazards rather than rely on the interpretations and opinions of others. This has involved obtaining primary information from previous work (such as test results and borehole logs) but also assembling and interpreting other important information (such as bathymetry, topography and aerial photographs) from DEWNR and other sources. Another important component of our approach has been a field overview of other sites where collapse or cracking has occurred (with the help of DEWNR). This field overview has helped us to understand the history of collapses on the river, the range of different situations where collapses have occurred and the history of cracking and other slope movements observed by DEWNR. We have also made our own site observations at the four sites of particular concern.

Our emphasis on independently developing our own understanding of the hazards has meant that we have overviewed the previous work for relevant evidence rather that attempting to review all the previous work including all the previous interpretations and judgements made.

1.5 Scope of work

This review has included the following activities:

- Review of published geological reports and maps.
- Review of available topographical maps, river bathymetry and recent and historic aerial photographs and site photographs (provided by DEWNR and obtained from other sources).
- Selective overview of existing geotechnical reports (provided by DEWNR).
- Overview of information on monitoring (provided by DEWNR).
- Field review of 18 riverbank areas (16 with Mr Jai O'Toole of DEWNR) where collapses have occurred in the past or cracking has been observed.
- Further site observations (of topography, cracking, other site, slope and riverbank features) at the four sites in question and initial field assessment of potential risk management and remedial options.
- Development of preliminary ground models, working maps and representative sections for the four sites in question.
- Assembling and review of available bathymetry for sites where failures have occurred and preparation of profiles showing pre-failure and post failure geometry.
- Development of an approach to risk assessment for riverbank collapse including a classification of types of riverbank failure and land use within the failure footprint.
- Back analyses of some of the large rapid collapses that have occurred. This involved comparing pre-failure and post failure geometries and developing the ground model including making judgements about the shear strength of the materials (particularly the soft clay).
- Carrying out sensitivity analyses of the ground model to help assess the effects of water level changes, tension cracks, surcharge, changes in the strength of the soft clay and flattening the slope. Comparisons were also made between circular and non-circular analyses.
- Assessment of the stability of the four sites in question. This included assessing the history, geometry and geology of the slopes and stability analyses of selected sections.
- Review of potential risk management and remedial options at the four sites based on current knowledge.
- Assessing the value of cost further geotechnical investigations.
- Indicative costing of some of the remedial options considered.
- Preparation of draft reports.
- Preparation of a final report following receipt of review comments.

2 GEOLOGICAL SETTING AND SUBSURFACE MATERIALS

Understanding the geological history helps understand the distribution, engineering properties and behaviour (including stability) of the materials in the ground. Diagrammatic sketches of the relevant geological history of the River Murray Valley in the project area are given in Figure 1. A summary (geological origin, brief description and occurrence) of the observed and inferred subsurface materials shown on Figure 1 is given in Table 1. All of the sketches on Figure 1 are diagrammatic and are not to scale. The relative thickness of some of the thinner units (e.g. Holocene colluvium and Fill) has been exaggerated for clarity but the sketches do show the relative distribution of the subsurface materials.

The first four sketches on Figure 1 summarise what has happened in the past 120,000 years (until European settlement). The bottom sketch summarises the present situation where the landscape has been modified by cut and fill and there is infrastructure close to the river. From the riverbank collapse point of view the most significant subsurface material is the Soft Clay which underlies the banks of the river. As discussed in Section 3.1 and Appendix C, most if not all of the riverbank collapses in the past few years have been because the Soft Clay next to the river has failed (given way).

The left hand side of the bottom sketch on Figure 1 shows the typical distribution of subsurface materials near the sides of the river valley. In this situation the Soft Cay overlies stiffer soil (in places) and a sloping surface of rock. Overlying the wedge of Soft Clay there may be some younger slope deposits (Holocene Colluvium) and Fill. Three of the four sites reviewed in this study (East Front Road near Mannum, Caloote Southern Residential Area and Woodlane Reserve) are near the valley sides and have subsurface conditions broadly similar to those shown on the sketch.

The right hand side of the bottom sketch on Figure 1 shows the typical subsurface conditions away from the sloping sides of the river valley. In this situation the rock is deeper and the Soft Clay is much thicker. The subsurface conditions at the fourth site reviewed in this study (River Front Road, Murray Bridge) are broadly similar to this part of the sketch.

Table 1: Summary of observed and inferred subsurface materials

Material type	Geological origin / brief description	Occurrence / comments
FILL	Man-made deposits. Various mixtures of gravel, sand silt and clay and rock fragments. Most fill observed appears to be uncontrolled (non-engineered).	Up to at least 3 m high associated with houses, shacks, roads and other infrastructure.
HOLOCENE COLLUVIUM	Slope deposits of Holocene age. Various mixtures of soil and rock fragments derived from upslope.	Near the sides of the river valley Holocene Colluvium formed in the past 7500 years overlies Soft Clay.
SOFT CLAY	River and floodplain deposits mostly deposited as the sea returned to its present level after the Last Glaciation (reached present level about 7500 years ago). Mainly consists of very soft to soft clay and sandy clay with lenses and layers of sandier material.	Soft Clay extends to at least 30 m below sea level in the Murray Bridge area and at least 20 m below sea level in the Mannum area. In many places, the upper part of the Soft Clay (e.g. the top metre or two) below the riverbank is stiffer because of the effects of desiccation).
LOOSE SAND	River deposits mostly deposited towards the end of the Last Glaciation (of late Pleistocene age) and during the post glacial rise in sea level (of early Holocene age). Mainly consists of sand with fine gravel likely to occur at the base. Likely to be mainly very loose to loose.	Underlies the Soft Clay in most places. May extend to at least 60 m below sea level in the Murray Bridge area and at least 40 m below sea level in the Mannum area.
STIFFER SOIL	Stiffer Soil is the term we have used for those stiffer materials (of various geological origins) which underlie the soft and loose soils and overlie rock in the Murray Valley close to the river. Geological origins of the Stiffer Soil include older (of Pleistocene age) colluvium or colluvium (strengthened by desiccation) and residual soil or extremely weathered (soil strength) materials derived from the underlying Rock. In the project area Stiffer Soil encountered in the investigations includes firm to very stiff cohesive (clayey) soil and medium dense to very dense granular (sandy or gravelly) soils.	Stiffer soil underlies the Soft Clay and Loose Sand in places and overlies Rock. Without seeing the materials the geological origin of the Stiffer Soil is difficult to assess. Stiffer Soil is an important material type from the engineering point of view because where it exists it may be a potential founding material for sheet piles or other retaining structures.
ROCK	Mainly LIMESTONE (shallow marine deposit of Tertiary age) which is mainly of low to medium strength. GRANITE (and igneous rock of Devonian age) occurs in some places (e.g. at East Front Road, Mannum). The granite is mainly of high to very high strength.	Rock underlies all the other material types.

3 APPROACH TO RISK ASSESSMENT

3.1 What has happened

As discussed previously there have been a large number of investigations and reports into a number of river bank failures and potential river bank failures along the banks of the River Murray. The general conclusions from a number of these reports is that the principal failure mechanism is slope failures in the Soft Clay due to the increased weight (effective stress) of the river bank resulting from lower pond levels in the Murray.

In very general terms the sites that have been investigated can be classified into four categories of failure **(based on their present condition)** with respect to the magnitude and speed of river bank instability that has occurred to date:

A. Very rapid failures that occurred with little or no warning (occurring in seconds to minutes) and resulted in large regression of the river bank (greater than 15m). The two best documented failures that we are aware of in this category are Long Island Marina and Woodlane Pumpstation.

B. Failures that occurred very rapidly or rapidly (minutes to hours) resulting in some loss of the river bank (up to 15m). Failures falling into this category include Freds Landing, White Sands, Murrayview Estates and Thiele Reserve (discussed in Appendix B).

C. Locations where some larger cracking, and horizontal and vertical movements (up to at least 1 m) have been detected that are consistent with bank instability but there was no significant riverbank regression or rapid collapse other than may have been caused by erosion. McRae Road and Bells Landing Reserve are examples of this category.

D. Locations with where some relatively minor cracking (mainly less than 20mm), and horizontal and vertical movements (less than 100mm) has occurred that may be bank instability, but may also be due to consolidation settlement of the Soft Clay.

As discussed in Section 3.2, Category C (in particular) but also Category D or even sites with no reported cracking have the potential to be affected by Category A and Category B failures during future periods of low river level.

Sites can also be assessed with respect to the consequences of a failure with respect to possible loss of life or extensive property damage by categorising the land use within the footprint of potential failures as follows:

1. High intensity infrastructure: Buildings, roads, car parks caravan parks, marinas or long jetties within the footprint of the collapse.

2. Open public access: Frequently used open public space, playgrounds, boat launching or short jetties within the footprint of the collapse.

3. Restricted use: Inaccessible or rarely accessed land with no development.

The four sites of concern assessed in this review all include some Category 1 land use (high intensity infrastructure). At Caloote Southern Residential Area and Woodlane reserve there is also some Category 2 land use (open public access).

Using the above categories a "present condition" risk matrix of relative risk level with respect to potential loss of life or serious injury, financial loss of assets and navigation hazard could be considered as shown in Table 2.

Table 2: Risk Matrix

Type of Failure	Land use within footprint of failure		
	1 - High intensity infrastructure	2- Open public access	3 – restricted use
Type A (Very rapid, more than 15 m regression)	Life: High Assets: High Navigation: High	Life: Low Assets: Med Navigation: High	Life: Very Low Assets: Low Navigation: High
Type B (Very rapid or rapid, less than 15 m regression)	Life: High Assets: High Navigation: Medium	Life: Low Assets: Low Navigation: Medium	Life: Very Low Assets: Very Low Navigation: Medium
Type C (Severe cracking consistent with instability)	Life: Very Low Assets: High Navigation: Very Low	Life: Very Low Assets: Low Navigation: Very Low	Life: Very Low Assets: Very Low Navigation: Very Low
Type D (Relatively minor cracking)	Life: Very Low Assets: Medium Navigation: Very Low	Life: Very Low Assets: Very Low Navigation: Very Low	Life: Very Low Assets: Very Low Navigation: Very Low

The basis for these risk ratings is summarised below:

A1,B1 A very rapid or rapid failure resulting in riverbank regression has a high likelihood of causing loss of life if it results in collapse of an occupied building or long jetty, a parked car or caravan with occupants going into the river, or loss of a section of a road resulting in a car crash. Such collapses can also result in significant financial loss of assets within the footprint of the collapse and navigation hazards post-collapse from the debris, trees etc. are high.

A2,B2 A very rapid or rapid failure resulting in riverbank regression has a much lower likelihood of causing loss of life if the area affects open space for parks or short jetties, since generally people in the area would be able to move away from the area of collapse. There is nevertheless a risk that someone might be injured or killed from a falling tree or drown if unable to evacuate the area in time. Financial loss of assets within the footprint of the collapse is less severe than Case 1 due to the lower value of assets, but navigation hazards post-collapse from the debris, trees etc. is still a high risk.

C1,D1, C2 A slow moving failure that does not result in riverbank regression but might result in movements of up to 1 m does not pose a risk to life as evacuation can occur, and there is no navigation hazard introduced, but deformations are sufficiently large that buildings and other rigid structures within the footprint could undergo significant damage. More flexible and lower cost structures in open space would likely sustain less damage particularly at smaller movements.

A3,B3 A very rapid or rapid failure resulting in riverbank regression has a very low likelihood of causing loss of life if the area is not accessible to the public and there is a low likelihood that anyone would be at the location of the failure when it occurred. Assets damaged by a collapse are likely to be of low value, but navigation hazards post-collapse from the debris, trees etc. is still a high risk.

D2,C3,D3 A slow moving failure that does not result in riverbank regression does not pose a risk to life if people are not there, or can evacuate, and there is no navigation hazard created. Assets are likely to be of low value or where present deformations are relatively low hence risk to assets is also low to very low.

3.2 What might happen

From the above discussion of risks it is clear that a Type A failure has the greatest potential to cause loss of life or damage. It is therefore important to understand what conditions lead to a rapid failure with a large river bank regression, and if riverbank regression can occur, the likely extent of regression that can occur in one event. Previous work has established that the conditions leading to these types of failures require the presence of deep deposits of Soft Clay in combination with an increase in load (stress) due to lowering of river levels.

As part of the current study, additional bathymetry, survey and site mapping information of the extent of two of the most dramatic Type A riverbank failures (Long Island Marina and Woodlane Pumpstation) has been collected. In Appendix C we have reassessed the failures at these two sites (by back-analysis) in order to better understand the conditions which lead to instability, how the slopes fail and why large riverbank regressions can occur.

We have carried out back analyses of four slope profiles (two at each site). The failure geometries, soil properties and the results of the sensitivity analyses to investigate the effects of the various parameters affecting stability are discussed in Appendix C. We have also carried out some analyses to investigate the role of progressive failure (a rapid succession of several failures at the one place) to help understand the amount of riverbank regression that occurred in some of the larger failures.

Three of the four slope profiles reviewed had similar initial (pre-failure) and final (post-failure) riverbank slopes (about 26^0 and 15^0 respectively). However as discussed in Appendix B riverbank collapses have occurred on a wide range of initial riverbank slopes. In our analyses we used similar soil strength and other parameters to those used in previous studies.

The results of the analyses are given in Appendix C. In summary the reassessment of past riverbank collapses has shown that:

- The drop in river level causes a large reduction in stability and appears to be the major cause of the riverbank collapses.
- A Fill surcharge on the bank also reduces the stability but recent experience has shown that riverbank collapse can occur where there is no Fill.
- The collapses that cause large regressions (e.g. Type A failures where more than say 15 m of bank are lost) are probably the result of progressive failure (i.e. a rapid succession of collapses).

- Small variations in strength of the Soft Clay have a big effect on stability. However, it is very difficult to predict where such strength variations might occur and how extensive they might be without very intensive (closely spaced) subsurface investigations.

In our opinion, on present knowledge, it should be assumed that during periods of low river level riverbank collapse (Type A or B failures) could occur wherever the bank is underlain by Soft Clay. Sites which have already been affected by Type C failures (i.e. already have experienced significant cracking) may be particularly vulnerable to collapse. Elsewhere, where there has only been relatively minor (Type D) cracking (or no reported cracking), collapse may be more likely where there is Fill but can also occur where there is little or no Fill.

3.3 Response to risk

As far as we are aware there are no regulatory requirements in South Australia relating to judged risk to life associated with landslides. In general, it is the responsibility of the client and/or owner and/or regulatory authority and/or others who may be affected to decide whether to accept or treat risk.

There are some published guidelines in Australia which discuss tolerable risk of loss of life (to the *"person most at risk"*) for some slopes but we are not aware of any guidelines which relate to societal (or total) risk to life associated with landslides. We are also aware of a trend (consistent with the new Work Health and Safety Acts introduced recently in some states) to move away from setting target or acceptable levels of risk to a due diligence approach which focusses on ensuring reasonable practicable precautions are in place.

For this project we have not nominated target levels of risk and understand that DEWNR is seeking advice on reasonable practicable precautions to reduce or avoid risk.

4 STAGED APPROACH TO INVESTIGATION, DESIGN AND CONSTRUCTION OF REMEDIAL OPTIONS

It is usual practice to take a staged approach to the investigation, design and construction of major engineering works. In our experience this is a particularly cost effective approach for remedial works in variable ground conditions where a variety of options need to be considered. A staged approach allows time for informed decisions to be made by the stakeholders and the optimum remedial option to be adopted. It also allows time for consultation and consideration of other issues (such as environmental and planning). Selecting options, or proceeding to construction, without adequate knowledge of ground conditions and other constraints (such as safety and environmental) often results in significantly higher costs and less effective solutions.

The typical project stages with brief comments on the engineering activities are as follows:

1. **Prefeasibilty/concept.** Initial consideration of options based on available information. Only indicative costing is possible at this stage.
2. **Preliminary design.** Preferred concepts are developed and geotechnical investigations undertaken, selection of the preferred option and preliminary design. Better cost estimates can be made at this stage.
3. **Detailed design.** Design progressed and developed further for preferred option. Depending on the options selected and the information collected at the preliminary stage there may be the need for further geotechnical investigations. Detailed design and costing.

4. **Construction.** Construction works are undertaken by a preferred contractor. Geotechnical input is essential during construction to ensure ground conditions are as assumed in design. Further design development can be undertaken and construction changes are made if required to ensure optimal design.

5. **Monitoring and maintenance.** The extent of monitoring and maintenance requirements depends on the type of project and decisions made by the client and other stakeholders at the option selection and design stages on the balance required between capital cost, project life and maintenance costs.

With respect to the remediation options, this project is at the prefeasibility/concept stage (Stage 1). There is some geotechnical information available but geotechnical investigations to date have largely been aimed at helping understand the hazards rather than for consideration of remediation options. The proposed geotechnical investigations discussed in Section 5 represent the start of the preliminary design stage (Stage 2).

5 REVIEW OF HAZARDS, RISKS AND MANANAGEMENT OPTIONS AT THE FOUR SITES BASED ON CURRENT KNOWLEDGE

5.1 Comments on uncertainties in the indicative cost estimates for remediation and the need for further geotechnical investigations

For each of the four sites in question we have summarised the site history and the hazards and discussed the risk based on current knowledge. At three of the sites we have recommended further geotechnical investigations and discussed remediation options. In our opinion, at the fourth site remediation is impracticable. We have provided an estimate of the costs of the further geotechnical investigations and provided some indication of the range of costs for remediation options. At this prefeasibility/concept stage the construction costs are indicative only and they will need to be reviewed and revised after the further investigations. It must be understood that, even after further investigations, cost and time overruns are commonly associated with the design and construction of engineering works in variable and difficult ground conditions.

The three sites where we have recommended that remediation options should be considered are East Front Road near Mannum, Caloote Southern Residential Area and Woodlane Reserve. As discussed in Section 2, these sites are near the sides of the River Murray Valley and have subsurface conditions broadly similar to those shown on the left hand side of the bottom sketch on Figure 1 (i.e. there is a wedge of Soft Clay overlain in places by Holocene Colluvium and Fill and underlain by a sloping surface of Stiffer Soil and Rock). In the areas where remediation is being considered there has been relatively little subsurface investigation to date. This means that while we can anticipate the presence of the wedge of Soft Clay we are unsure of just how far inland it extends, how steeply the base of the Soft Clay slopes towards the river, what underlies the Soft Clay (Stiffer Soil or Rock) and the depth of desiccation. All of these factors influence the potential size, depth and location (particularly inland extent) of any riverbank collapse that might occur and the type of remediation that is most appropriate for the site.

The main reason that we have recommended further geotechnical investigation at the three sites where remediation may be considered is to get a better understanding of the distribution (presence and depth) of Soft Clay. This will allow the extent of the potential collapses to be better understood and remediation options to be reviewed (including, if appropriate, alternatives to those presented in this report). Selected remediation options can then be designed and costed with more certainty. In our estimates of the costs of the further geotechnical investigations we also have allowed for drilling and sampling (including coring of rock where necessary) below the Soft Clay so that foundation conditions for any subsurface works are understood.

General comments on the value of a staged approach to the design and construction of remedial works are given in Section 4.

5.2 East Front Road, Mannum

5.2.1 Site description and background

East Front Road is on the left bank of the River Murray about 4 km upstream (east north east) of Mannum. The site is the edge of the flood plain (next to sloping ground) on the outside of a gentle bend in the river. A review of historical aerial photographs (dating back to 1956) indicates that there has been no significant change to the riverbank in the area in the past 56 years. We understand from local residents that the road was raised by about 2 m in the 1970's or 1980's and we observed Fill up against large pre-existing trees growing on the narrow flood plain. At the same time, the road was straightened by construction of an embankment across a shallow lagoon.

Figures 2 and 3 are working maps and sections of the area. Both the bathymetry and land contours are at 0.25 m intervals. The outcrops uphill of the road are granite and there is a large disused granite quarry to the east of the area of concern. The irregular bathymetry near the quarry is probably the result of Fill and broken piles associated with the old wharf next to the quarry.

A section of East Front Road has been closed since April 2010 due to tension cracks in three different locations. The 2010 SKM report (Reference 1) judged that there was a high likelihood of riverbank failure at the site and that the site had one of the lowest factors of safety of all the sites modelled.

Undulations in the road present a traffic hazard and in March 2011 two cavities formed in the road. Permanent gates have been installed at both ends of the road. Local residents have been advised of the risks in continuing to access their properties using the closed section of the road. Visual monitoring, including photographs has identified continued deterioration of the road. Survey monitoring has been undertaken since 6 October 2011. Recorded movements at the monitoring locations since then have been small (mostly less than 3 mm) and are probably mostly within the range of survey error.

The approximate extent of cracked road observed by Coffey (now four areas) is shown on Figure 2. At the sites close to the river (shown on Sections E1, E2 and E3 on Figure 3), the cracks, shallow depressions and uneven sections of road form broadly arcuate patterns which are concave towards the river. At the time of our site review most depressions were less than 50 mm deep. At the site close to the shallow lagoon (represented by Section E4 on Figure 3), the cracks and depressions tend to be parallel to the road.

5.2.2 Hazards and risks

As shown on Section E3 (Figure 3) there is a wedge of Soft Clay below the road. The wedge of Soft Clay is overlain by Fill. In places there may also be some Holocene Colluvium overlying the Soft Clay as shown on the diagrammatic sketches on Figure 1.

There is also likely to be similar wedges of Soft Clay below the road at Sections E2 and E4. At Section E1 the nearest borehole indicated that Fill occurred to the top of rock at a depth of RL -2.5 m. At Section 1 it is likely that the Soft Clay was displaced (as a "mud wave") by the Fill when the road level was raised. The existing houses appear to have been founded on granite (or overlying residual soil or colluvium) and, in our opinion, it is unlikely that any the houses are underlain by Soft Clay.

The arcuate pattern of cracks at Sections E1, E2 and E3 is consistent with incipient slope instability and it is prudent to assume that the factor of safety (FOS) was close to 1 during the recent period of lower river levels in 2009. However, it is important to understand that differential settlement associated with the underlying wedge of soft soil will also result in cracking and (possibly some outward movement) and not all the cracks observed will be associated with slope instability.

The results of the stability analyses at East Front Road (given on Figures D1 to D4 in Appendix D) are summarised below. All the analyses were carried out assuming a river level of RL -0.9 m (close to the average level during 2009).

- Figure D1 shows the results of the stability analysis at Section E1 which probably had a FOS of about 1 when the river level was low. The calculated FOS for the model analysed of 0.69 indicates that the model analysed is very conservative. It is possible that the road Fill has a higher shear strength than assumed and that the Fill extends deeper and further towards the river than assumed in the model analysed.
- Figure D2 shows the same section where the road level has been lowered to RL 2 m (from about RL -3.8 m). The lowering of the road by about 1.8 m reduces the disturbing forces and significantly increases the FOS (from 0.69 to 1.08).
- Figure D3 shows the results of the stability analysis for Section E3 and gives a FOS of 1. The head scarp of the critical failure surface in the slope model is close to arcuate line of cracks and depressions on the road.
- Figure D4 shows the results of the stability analyses for Section E4 (where the road embankment crosses the shallow lagoon). The FOS for this model is 1.28. It is likely that the deformations in this area are largely the result of settlement rather than slope instability.

In our opinion, in its present condition, the road close to the river (Sections E1, E2 and E3), is vulnerable to slope instability during periods of low river level. This could involve relatively rapid movements of a metre or more resulting in a drop or void in the road and movement of the landslide debris towards the river (i.e. Type B or even Type A failures could occur). This presents a risk of injury, or death, to a road user if they drive into the void or the ground moves while they are on the road.

In our opinion the existing houses in the area are not likely to be affected by slope instability as they do not appear to be underlain by soft soils.

5.2.3 Risk management options previously considered

In 2010 Golder (Reference 3) advised the then DFW (now DEWNR) on options for reducing the probability of landslides at East Front Road. In that letter Golder discussed various options and advised that:

> *"...buttressing/shear keys or soil nailing/stone columns/sheet piles/driven piles are likely to be the most viable to options to reduce the probability of landsliding on East Front Road..."*

Golder pointed out that there would be significant difficulties associated with construction and that there would need to be a:

> *"...significant commitment to risk management to allow construction to proceed within acceptable OH, S and E considerations."*

We agree with Golder that an approach using driven (or perhaps bored) piles for example could potentially be effective and we also agree with their concerns about the construction and risk management difficulties. It is not clear from this letter if Golder was just referring solely to the cracked areas close to the river or the cracked areas alone but even treating the cracked areas alone would involve about 200 m of road. On the basis that similar ground conditions occur between the already cracked areas it is possible that other, currently uncracked sections of road could be affected by slope instability during future periods of low river level (Figure 2).

While we agree that the approach suggested by Golder could significantly reduce the probability of landslides in the sections treated, it is likely to be relatively very expensive (in the region of several million dollars) particularly considering that the road only provides access to four properties and an alternative route exists for through traffic.

In the following section we consider relatively less expensive methods of reducing or managing the risk of riverbank collapse at this site.

5.2.4 Other methods of reducing the risk

Option A: Lowering the level of the road

As shown by the stability analyses discussed above, lowering the level of the road could significant improve the stability. At Section E1 a lowering of 1.8 m improved the FOS by 0.4. In the 300 m close to the river the present road level varies from about RL 4 m to RL 3 m. The lowest point on the road in the area is about RL 2 m where the road crosses the shallow lagoon. If it was possible to lower the road level to, say, RL 2 m (or even a level of RL 2.5 m) the stability of the road would be significantly improved and the likelihood of collapse would be significantly reduced.

When discussing this option Golder correctly pointed out that a lower road would be more prone to flooding (although the road is already at RL 2 m near the shallow lagoon). Golder also pointed out that access drives to the resident's houses would need to be modified, the effect on the subgrade would need to be considered, services may need to be relocated and sight lines considered.

If this option is considered, minor changes to the alignment (away from the river) could be considered where practicable. The damaged section of road crossing the shallow lagoon (Section E4) could also be repaired. The use of a geofabric will also help to reduce the likelihood of future cracking during periods of low river level.

Consideration could also be given to making the lower road unsealed and/or one lane with passing places in order to reduce costs.

Option B: Permanent closing of the road to the public (but allowing existing residents access)

At present the road is closed to the public and only the residents of the four houses have access. One option would be to permanently close the road to the public. This option could involve converting parts of the road where practicable into a one lane access road for the residents only. Close to the areas of concern the access road could be moved away from the river and lowered to reduce the likelihood of being affected by slope failure.

A variation of this option could be to completely abandon the section of road between Sections E1 and E2. The residents of the three houses near Section E2 would then only have access downstream (to the west) and the residents at Section E3 would only have access upstream (to the east).

The use of unsealed roads could also be considered for Option B.

Option C: Setting up an early warning system

This option could involve laying a cable in a shallow trench along the road and continuously monitoring for movement. The monitoring system could be linked to lights to provide a warning if the road collapses so that an accident is avoided.

5.2.5 Recommendations, comments on costs and further work

We recommend that further consideration be given to Options A and B above. Option C could help manage risk in the short term but does not reduce the likelihood of collapse. For an early warning system to be permanently effective in reducing the likelihood of an accident it would have to regularly checked and well maintained. For infrequent hazards, experience has shown that people become complacent, and unless managed very well warning systems are often neglected in the longer term.

As discussed in Section 4.1 further investigation is required to get a better understanding of the distribution (presence and depth) of Soft Clay. This could be achieved by geotechnical drilling, cone penetration tests and test pits. Depending on the results of the further investigations the cost benefits of other options or combinations of options should be reviewed.

Indicative costs

For indicative costing purposes we have assumed that geotechnical investigations will involve 6 to 10 boreholes, 4 to 8 cone penetrometer tests and 4 to 6 test pits. The total cost of the geotechnical investigation including planning, assessment and reporting is likely to be in the range of $60,000 to $80,000.

The design and construction costs of remediation options depend on which option is adopted and the design life required. At this stage we estimate that the preliminary budget allowance for the design and construction costs of a lower level road should be in the range of $800,000 to $2.2 million for options ranging from a single lane unsealed road with passing places to a two lane sealed road.

General comments on the value of a staged approach to the design and construction of remedial options are given in Section 4.

5.3 Caloote Southern Residential Area

5.3.1 Site description and background

Caloote Landing is on the right bank of the River Murray about 7 km downstream (south west) of Mannum. The site is on the edge of the flood plain (next to sloping ground) on the outside of a sharp bend in the river. The actual landing area is at the mouth of a small buried valley where Soft Clay is likely to be deeper (further away from sloping ground). The Caloote Southern Residential Area is immediately south of the landing next to a ridge of limestone. There are six houses in this area (Nos. 9 to 14). A review of historical aerial photographs (dating back to 1956) indicates that there has been no significant change to the riverbank in the area in the past 56 years.

Figures 4 and 5 are working maps and sections of the Caloote Southern Residential Area. Both the bathymetry and land contours are at 0.25 m intervals. The limestone cliff behind the houses is very steep and as shown on Section C2 overhangs behind No. 14 (and also No. 13). We were unable to enter the private land (which includes the cliffs and the top of the limestone ridge) behind the houses. South of the houses we observed many boulders below the cliff including a single boulder several metres across.

Caloote Landing was closed in September 2009 because of concerns about the high likelihood of riverbank collapse at the site. Subsequent risk assessments and site monitoring have allowed parts of Caloote Landing to be re-opened, however the Southern Residential Area remains closed and cordoned off to prevent public access due to concerns over continued site instability both from the riverbank and the limestone cliffs behind the houses. In 2009, cracking of the ground at and close to the front of the houses was observed. Local property owners within the closed area continue to access their properties despite the information and advice provided by geotechnical consultants.

Monitoring is continuing in the Southern Residential Area, but reported movements are small (mostly less than 3 mm and are probably mostly within the range of survey error.

5.3.2 Hazards and risks

As shown on Section C1 (Figure 5) there is a wedge of Soft Clay below the river bank. In places the Soft Clay is overlain by Holocene Colluvium and Fill (as shown on the diagrammatic sketches on Figure 1). The Soft Clay may have extended inland on Section C1 as far as the verandah of No. 11 (house). Above about RL 0 m the Soft Caly is probably desiccated and therefore has a higher undrained shear strength. The Soft Clay is likely to be underlain by Stiffer Soils and Limestone.

Ground conditions are likely to be broadly similar under and in front of all the houses of concern (Nos. 9 to 15 on Figure 4) because they have all been built at the base of the limestone ridge. There is relatively little subsurface geotechnical information close to the houses but, on present knowledge, it appears likely that there is probably no Soft Clay (or very little now desiccated) under any of them. In front of Nos 9 and 10 the wedge of Soft Clay deepens to the north (towards the buried valley) as well as to the east (towards the river).

The cracking and minor tilting close to the front of the houses observed in 2009 may be the result of shrinkage associated with the clayey soil and some differential settlement associated with the wedge of Soft Clay in front of the houses.

The results of the stability analyses at Caloote Southern Residential Area (given on Figures D5 to D7 in Appendix D) are summarised below. All the analyses were carried out assuming a river level of RL -0.9 m (close to the average level during 2009).

- Figure D5 shows the results of the stability analyses at Section C1. The calculated factor of safety (FOS) for the model analysed is 0.95 and the head scarp of the critical failure surface is about 13 m from the river bank and about 9 m from the front of No. 11 (about 7 m from the front of the verandah). The analysis indicates that the riverbank was only marginally stable when the river level was low.
- Figure D6 shows the results of the stability analyses of Section C1 where the failure surface has been forced to extend back to the verandah of No. 14. For this model the FOS is 1.02 which is 7 % higher than the FOS for the critical failure surface closer to the river. These two analyses indicate that it may be possible for the failure surface to extend back to the verandah but it is more likely to be 5 to 10 m in front of the house.

- Figure D7 shows the results of the stability analyses at Section C2 where a similar wedge of Soft Clay in front of the house (No.14) has been assumed. The FOS of 0.86 again suggests that the riverbank in front of the house may have been only marginally stable during the periods of low river level.

Section C2 (Figure 5) shows the overhanging limestone cliff behind No. 14 (and No. 13). The cliff overhangs by up to at least 3 m. During our site visit we observed a near vertical open joint close to the overhang. The potential failure at the overhang has a volume of several tens of m^3. If the slope failed, very large boulders could severely damage the houses downslope and have the potential to injure or kill any occupants. The rock fall risk appears to be highest for Nos. 14 and 13. Without access to private land it was difficult to assess the hazard for the other four properties. From a distance, and from aerial photographs, the rock fall risk appears to be less for Nos. 11 and 12 and may be low or very low for Nos. 9 and 10.

In our opinion, the riverbank in front of the six houses is vulnerable to collapse during periods of low river level (i.e. Type A or Type B failures could occur). However, most collapses are not likely to extend back as far as the houses. It is possible that a very large riverbank collapse could fail progressively and extend back as far as the front verandah of some of the houses. However, on present knowledge, in our opinion it is very unlikely that a riverbank failure could cause one of the houses to collapse and injure or kill an occupant.

5.3.3 Risk management options and further work

Risk associated with the limestone cliff

In our opinion the highest risk to the properties and their occupants (particularly Nos. 13 and 14) is associated with the limestone cliffs. As a matter of priority the residents should again be warned of the rock fall risk and advised to seek their own independent advice. Further work (including access to the private properties) may indicate that the risk associated to some of the houses (particular Nos. 9 and 10) is low or can easily be managed but we are unable to make that judgement on present knowledge.

As far as the most at risk properties are concerned our initial assessment is that it would be difficult to stabilise the overhanging cliff while the houses are in place. One possible course of action might be to move or demolish the houses below the overhang and then remove the overhang and stabilise the cliff before rebuilding.

Risk associated with riverbank collapse

On present knowledge it appears that it may be feasible to carry out engineering works at the front of the six properties to reduce or eliminate the risks associated with riverbank collapse affecting the properties. These could take the form of sheet, driven or bored piles or some other form of retaining structures such as construction of a shear key (trench backfilled with stronger materials parallel to the riverbank) founded in stiffer material below the Soft Clay. Such a retaining structure would prevent riverbank collapse damaging the houses. During the design and construction of stabilising measures, care will need to be taken to reduce any adverse effects on the stability of the open public access area on the river side of the works.

As discussed in Section 5.1 further investigation is required to get a better understanding of the distribution (presence and depth) of Soft Clay. This could be achieved by geotechnical drilling, cone penetration tests and test pits.

The area in front of the six houses that could be affected by the riverbank collapse is a Category 2 open public access area as defined in Section 3.1 of this report. On present knowledge it is also possible that a large collapse or a progressive failure could affect the access road in front of the houses. Developing an overall approach to long-term risk management of public access areas vulnerable to riverbank collapse is discussed in Section 6.

Indicative costs

For indicative costing purposes we have assumed that geotechnical investigations will involve 8 to 12 cone penetrometer tests, 4 to 6 boreholes and 4 to 6 test pits. The total cost of the geotechnical investigation including planning, assessment and reporting is likely to be in the range of $50,000 to $70,000. At this stage we estimate that the preliminary budget allowance for the design and construction costs of a retaining structure located about 5 to 6 m in front of the houses (on our near the access road) should be in the range of $600,000 to $1 million.

A retaining structure closer to the river (say 10 to 15 m in front of the houses) where the Soft Clay is deeper would have to be substantially deeper and would be substantially more expensive (perhaps in the range of $1.5 to $2.5 million). There would also be more significant work place, health and safety and environmental issues associated with working on the river bank.

General comments on the value of a staged approach to the design and construction of remedial options are given in Section 4.

5.4 Woodlane Reserve

5.4.1 Site description and background

Woodlane Reserve is on the right bank of the Murray River about 9 km (as the crow flies) downstream of Mannum. The site is on the edge of the flood plain (next to sloping ground) on a relatively straight section of the river immediately downstream from the outside of a bend. A review of historical aerial photographs (dating back to 1956) indicates that there has been no significant change to the riverbank in the area in the past 56 years.

Figures 6 and 7 are working maps and sections of the Woodlane Reserve area. Both the bathymetry and land contours are at 0.25 m intervals.

There are two areas of concern at Woodlane Reserve upstream of the shallow lagoon. One is the grassed area in front of the houses upstream of the access road where cracks have been observed close to the river bank. This area has been cordoned off in the past. From discussions with local residents, review of the bathymetry and old photographs it appears that relatively small shallow bank failures may have occurred in the past. Some of these may have involved the failure of small sandy beaches constructed by the residents (e.g. Section W1 on Figure 7).

The other area of concern is the access road close to where two riverbank collapse events occurred at this site next to the reserve car park in early 2009 (late February and 7 March). Pumping infrastructure was damaged and collapsed into the river channel following the collapse events. Cracks were observed in the access road close to the collapse sites in 2009. By reviewing old photographs we have been able to find out the approximate locations of the cracks (shown on Figure 6). We understand that the cracks were only a few mm wide. The access road crosses the mouth of a small buried valley where Soft Clay is likely to occur further inland than where the ground slopes. Use of the road is largely restricted to local residents.

Recent monitoring (since October 2011) has indicated relatively little movement in the area.

5.4.2 Hazards and risks

As shown on Section W1 (Figure 7) there is a wedge of Soft Clay below the river bank in the area in front of the houses. On the riverbank the Soft Clay is overlain by Holocene Colluvium and Fill (as shown on the diagrammatic sketches on Figure 1). The Soft Clay is underlain by Stiffer Soil (including older alluvium) and limestone which can be observed in cuttings near the houses.

The houses are likely to be underlain by Stiffer Soil or Rock.

In the area where the access road crosses the buried valley and is close to the Pumpstation collapses (Sections W2 and W3 on Figure 7) there is also a wedge of Soft Soil overlying stiffer soils. The access road is on a fill embankment.

The cracking on the access road observed in 2009 may have been associated with settlement of the Fill embankment or may have been the result of very small movements towards the river associated with stress relief following the Pumpstation riverbank collapses.

Figure D8 (In Appendix D) shows the results of stability analyses of Section W1. The analyses were carried out assuming a river level of RL -0.9 m (close to the average level during 2009). The calculated factor of safety (FOS) for the model analysed is 0.75 and the head scarp for the critical failure surface is about 14 m from the bank of the river. While the low FOS of the unfailed slope indicates that the model analysed is conservative with respect to the distribution of materials or the parameters chosen, the analyses do indicate that the bank was probably only marginally stable when the river level was low.

In our opinion, the riverbank in front of the houses is vulnerable to collapse during periods of low river level. On present knowledge it is also possible that a very large collapse or a progressive failure could affect parts of the access road in front of the houses. However, the collapses are not likely to extend back to the houses.

As shown on Section W2 (Figure 7) the access road is only about 4 m from the present riverbank at the site of the Pumpstation collapses. In our opinion further regression of the riverbank could occur which could cause local failure of the access road.

5.4.3 Risk management options and further work

Open public access area

The area in front of the houses that could be affected by riverbank collapse is a Category 2 open public access area as defined in Section 3.1 of this report. Developing an overall approach to long term risk management of public access areas vulnerable to riverbank collapse is discussed in Section 6.

There is has been relatively little subsurface investigation in front of the houses. Further investigations will be required to assess the likelihood of all or parts of the road being affected by riverbank collapse and potential remedial options. Geotechnical drilling and cone penetrometer tests will help confirm the extent of Soft Clay and the nature of underling material. Test pits would also be helpful. The type and extent of engineering works, if required, will depend on the location and depth of Soft Clay underlying the road in front of the houses. For example, if there are only isolated sections of road underlain by shallow Soft Clay localised excavation and replacement may be all that is required. However, if the Soft Clay under the road is more extensive, options such as piling, a shear key, or buttressing may be more appropriate.

Access road near pumpstation collapse

In our opinion, consideration should be given to moving the access road further away from the riverbank. Realignment would have the effect of significantly reducing the likelihood of the access road being affected by riverbank collapse. Before choosing the alignment it would be necessary to obtain more information on the depth of soft soils across the buried valley below the access road lagoon. As shown on Figure 6 the deepest part of the buried valley may be closer to the north west side of the lagoon (because of the steeper slope on that side). Cone penetrometer tests (mostly on or close to the access road) could be used for the investigation.

The realigned access road would have to cross the lagoon. Using geofabrics and keeping the embankment low would reduce the potential effects of settlement on the unsealed road. If an alternative to an embankment is required (e.g. for environmental reasons) a flexible structure such as a low bailey bridge could be considered.

Indicative costs

For indicative costing purposes we have assumed that geotechnical investigations for both the open access area and the access road near the pumpstation collapse will involve 15 to 20 cone penetrometer tests, 5 to 6 boreholes and 4 to 6 test pits. The total cost of both geotechnical investigations, including planning, assessment and reporting is likely to be in the range of $70,000 to $100,000. At this stage we estimate that the preliminary budget allowance for the design and construction costs of realigning the access road near the pumpstation collapse should be in the range of $400,000 to $600,000.

The cost of any remedial works to support the access road in front of the houses will depend on what if anything is required. This will be assessed after the geotechnical investigations.

General comments on the value of a staged approach to the design and construction of remedial options are given in Section 4.

5.5 River Front Road, Murray Bridge

5.5.1 Site description and background

River Front Road is on the right bank of the River Murray in Murray Bridge immediately downstream of Sturt Reserve. The site is within the flood plain on a straight section of the river. River Front Road is about 1.5 km upstream of Long Island Marina where four large river bank collapses occurred in 2008 and 2009. A review of historical aerial photographs (dating back to 1956) indicates that there has been no significant change to the riverbank in the area for the past 56 years.

Figures 8 and 9 are working maps and sections of the area. Both the bathymetry and land contours are at 0.25 m intervals.

River Front Road was gazetted closed by the Rural City of Murray Bridge in 2010. Warning signs have been installed along River Front Road and residents are advised not to access their properties. Cracks (parallel to the river bank) have been observed in the area in the past but no monitoring results were available for this review. Some of the fronts of the houses appear to be slightly tilted towards the river.

5.5.2 Hazards and risks

As shown on Sections R1 and R2 (on Figure 9) Soft Clay extends to below the depth of the river. In places the Soft Clay is overlain by Fill (as shown on right hand side of the bottom diagrammatic sketch on Figure 1).

The crack parallel to the riverbank observed in 2009 and apparent minor tilting of some of the houses may be associated with desiccation of the near surface clayey soils and some differential settlement associated with the deep wedge of Soft Clay.

Figure D9 (In Appendix D) shows the results of stability analyses of Section R2. The analyses were carried out assuming a river level of RL -0.9 m (close to the average level during 2009). The calculated factor of safety (FOS) for the model analysed is 0.97 and the head scarp for the critical failure surface is about 9 m from the present bank of the river. The analyses indicate that the bank was probably only marginally stable when the river level was low.

In our opinion, the riverbank (including) the houses are vulnerable to collapse during periods of low river level. Although the riverbank is lower than at Long Island Marina, the depth of river is greater and buildings are right on the edge of the bank providing some surcharge. If large scale rapid collapses occur houses are likely to be destroyed and occupants could be killed.

5.5.3 Risk management options and further work

Engineering solutions to reduce the likelihood of riverbank collapse and the risk to life of the occupants of the houses are likely to be prohibitively expensive. As shown by sensitivity analysis (Appendix C) flattening the riverbank slope only marginally improves the stability and the bank is already flood prone so any existing surcharge cannot be removed. Land based sheet piling, contiguous piled walls or other structural solutions would be extremely expensive because of the depth of Soft Clay (indicative design and construction costs are likely to be in the range of $8 to $12 million). Structural solutions would be very difficult to construct because the existing buildings and jetties extend over the water (some of the structures would probably have to be demolished to allow access for plant and equipment). Works could be undertaken from the water but this would greatly increase the costs of the works (perhaps by a factor of 4 to 8 on the figures given above). There would also be significant work place health and safety and environmental considerations associated with working at the edge of, and in, the river.

In our opinion there is a high risk to life at River Front Road. We recommend that the residents again be warned of the risk. We are not familiar with the previous advice given or the legal and regulatory powers of local and state government with respect to taking action to reduce the risk to life but we recommend that, in the interest of public safety, all options are considered.

6 COMMENTS ON LONG TERM RISK MANAGEMENT OF OPEN PUBLIC ACCESS AREAS CLOSE TO THE RIVER

At many places along the banks of the River Murray there are open public access areas (defined as Category 2 land use in Section 3.1). These open public access areas include parts of the four sites assessed in this report (e.g. Caloote, Woodlane Reserve and Sturt Reserve north of River Front Road at Murray Bridge). Many of these areas are vulnerable to large scale rapid collapse during periods of low river level. As discussed in Section 3.2, collapse may be more likely where there is Fill but can also occur where there is little or no Fill.

DEWNR, their consultants and other parties have carried out a lot of useful work on understanding and managing the risks associated with riverbank collapse in recent years, particularly in areas where infrastructure is at risk (Category 1 land use as defined in Section 3.1). In our opinion, it would be useful to review the risk management approach taken to date and develop a long term overall approach to managing the risk associated with open public access areas (Category 2 land use).

Long term risk management of these areas is a difficult challenge because people become complacent about infrequent hazards. Over time there is a natural tendency to discontinue monitoring and to take less notice of potential warning signs. Another challenge is that future collapse during periods of low river level could occur anywhere where deep Soft Clay occurs and is not likely to be restricted to previous areas of concern. It is also important that the collapse hazard risks are understood in the context of other risks associated with the riverbank and river (such as drowning). While every fatality is unacceptable it can be illogical to put disproportionate resources into managing one risk when other, perhaps greater risks are neglected.

An important component of long term risk management in areas of open public access will be development controls. Clearly it will be desirable to limit the construction of new infrastructure close to the river and avoid, as much as possible, placing more fill. Managing the river so that very low river levels do not occur will also reduce the risks associated with collapse.

Another important component of long term risk management will be monitoring. For such monitoring to be effective it will need to be unobtrusive so that it is not vulnerable to damage or vandalism. It will also need to be robust so that it will last for many years and sites can be resurveyed in the future even if there has been a break in regular monitoring. In our opinion some riverbank sites which do not currently show signs of instability or failure should be included in the monitoring program. These sites can be used as reference or control locations.

Monitoring data is an extremely valuable tool in the assessment of site stability and understanding associated risks in the interest of public safety and the management of infrastructure. Implementation of a robust monitoring program is a critically important activity in understanding and managing the riverbank collapse hazard and reducing the risk to the public.

More sophisticated monitoring (including real time monitoring linked to warning systems) can be useful in the shorter term for high risk sites (Category A in Section 3.1). However, as discussed in Section 4.2.5, such systems are often not well maintained in the longer term. In our opinion, for higher risk sites, remediation or other risk management options that eliminate or greatly reduce the risk are usually preferable.

We are also aware that several research projects have been carried out on the collapse hazards. We are unaware of the details of the work to date but would be happy to contribute ideas on how research can be used to help understand and manage the collapse risks in the future. For example, for this project we carried out a brief overview of 18 sites where collapses had occurred or cracking had been observed. If it has not already been done, a great deal more could be learned from a thorough study and back analyses of all collapse failures that have occurred along the lower River Murray.

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Important information about your **Coffey** Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Important information about your **Coffey** Report

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment.
Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

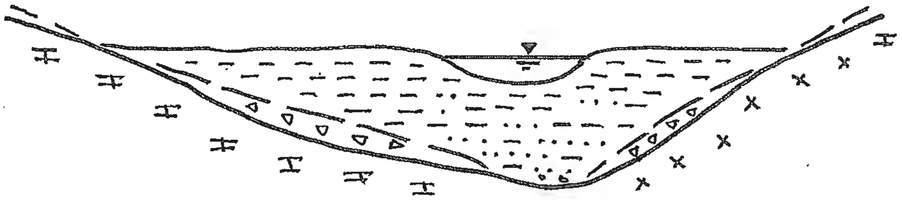
Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.

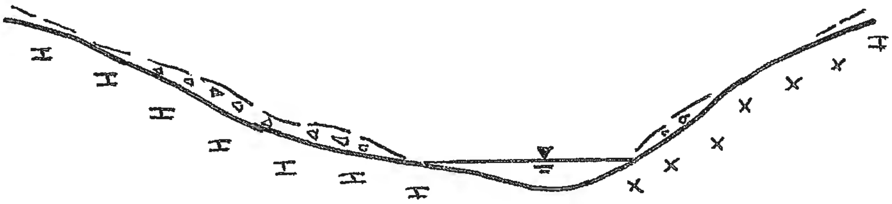
Figures

DIAGRAMMATIC SKETCHES - WHOLE VALLEY

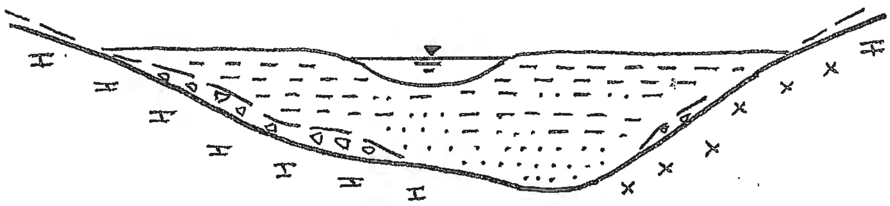
TIME/EVENT



120,000 YEARS AGO
Last Interglacial sea level slightly higher than present level

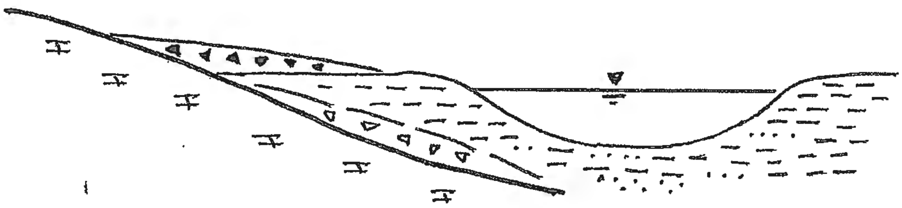


18,000 YEARS AGO
Coldest part of last glacial. River lower because sea level 120m lower than present

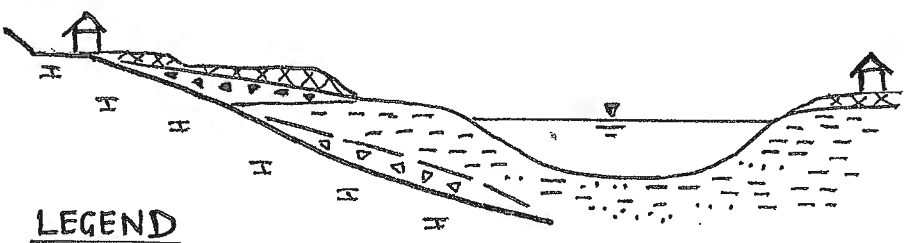


7500 YEARS AGO
Sea level reaches present level. Valley full of soft clay and loose sand

DIAGRAMMATIC SKETCHES - VALLEY SIDE



200 YEARS AGO
(PRE-EUROPEAN SETTLEMENT)
Holocene colluvium overlies soft clay near valley side



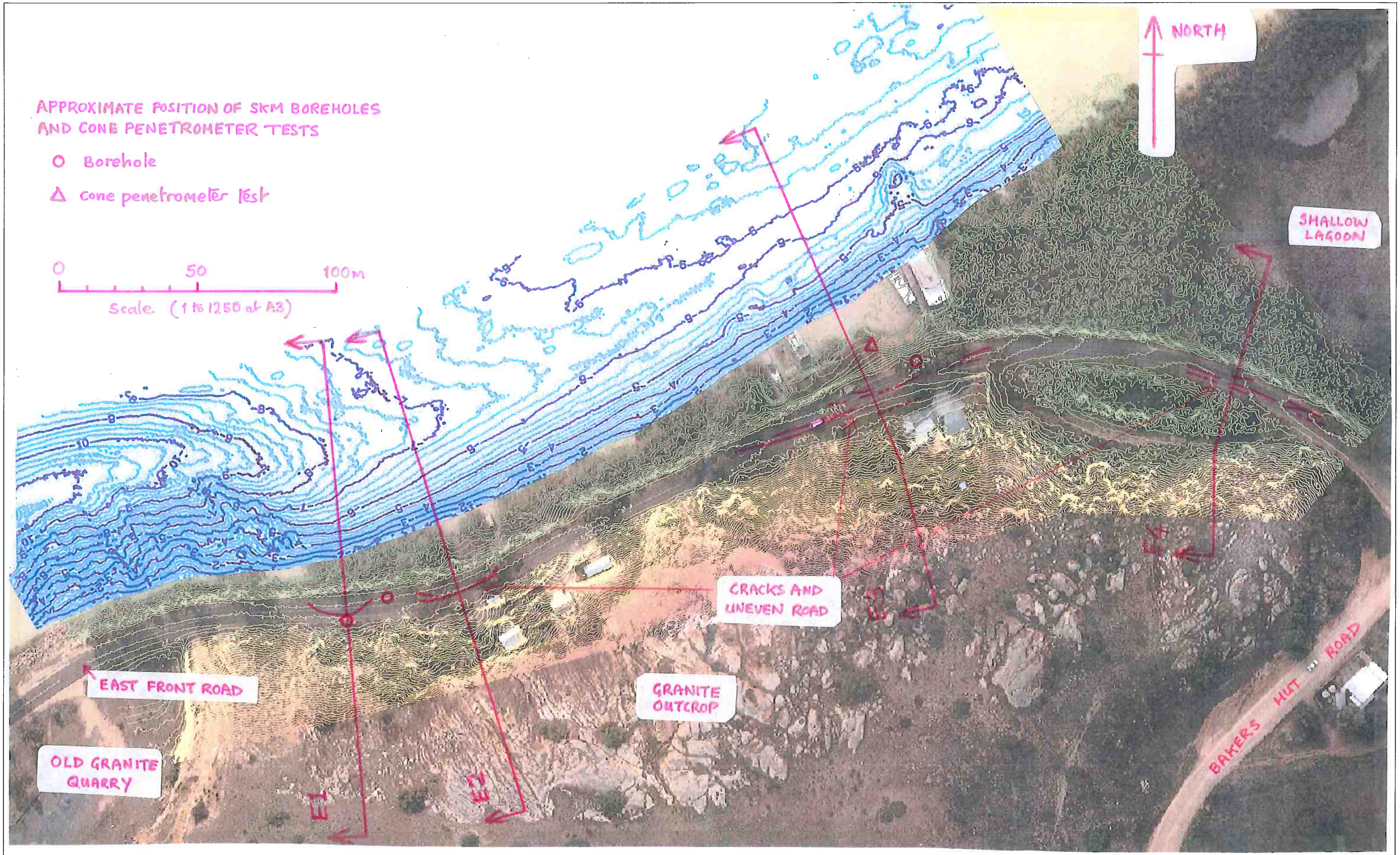
PRESENT
Landscape modified by cut and fill. Houses roads and other infrastructure near river

LEGEND

- XXX Fill
- Holocene colluvium
- Soft clay and loose sand
- Stiffer soil
- Limestone rock
- Granite rock

1 Top of rock shown by thicker line
2 Top of stiffer soil shown by dashed line
3 See text for more information on geology and materials
4 All sketches are diagrammatic and not to scale. The relative thickness of some materials is exaggerated

drawn	ATM	coffey geotechnics SPECIALISTS MANAGING THE EARTH	DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES
approved	ATM		REVIEW OF RIVER BANK RISK MANAGEMENT OPTIONS
date	12 Dec 2012		FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA
scale	Not to scale		DIAGRAMMATIC SKETCHES OF GEOLOGICAL HISTORY
original size	A4		project no: GEOTMEND07093AA — figure no: FIGURE 1



revision	description	drawn	approved	date

drawn	ATM
approved	ATM
date	12 Dec 2012
scale	As shown
original size	A3

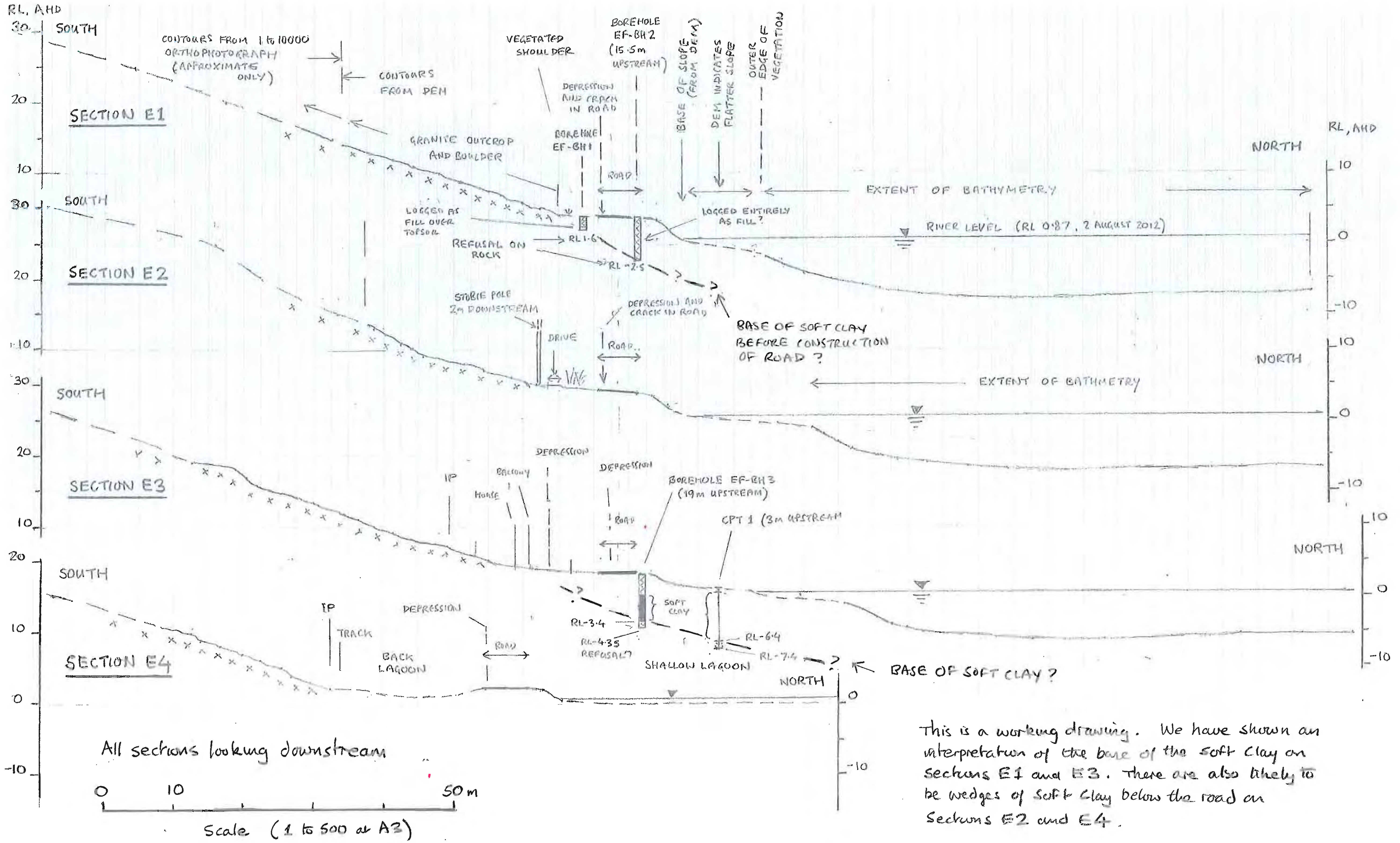
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES

REVIEW OF RIVERBANK RISK MANAGEMENT OPTIONS. FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA

EAST FRONT ROAD, MANNUM: LOCATION MAP

project no: GEOTMEND07093AA | figure no: FIGURE 2



This is a working drawing. We have shown an interpretation of the base of the soft clay on sections E1 and E3. There are also likely to be wedges of soft clay below the road on sections E2 and E4.

revision	description	drawn	approved	date

drawn	ATM
approved	ATM
date	12 Dec 2012
scale	As shown
original size	A3

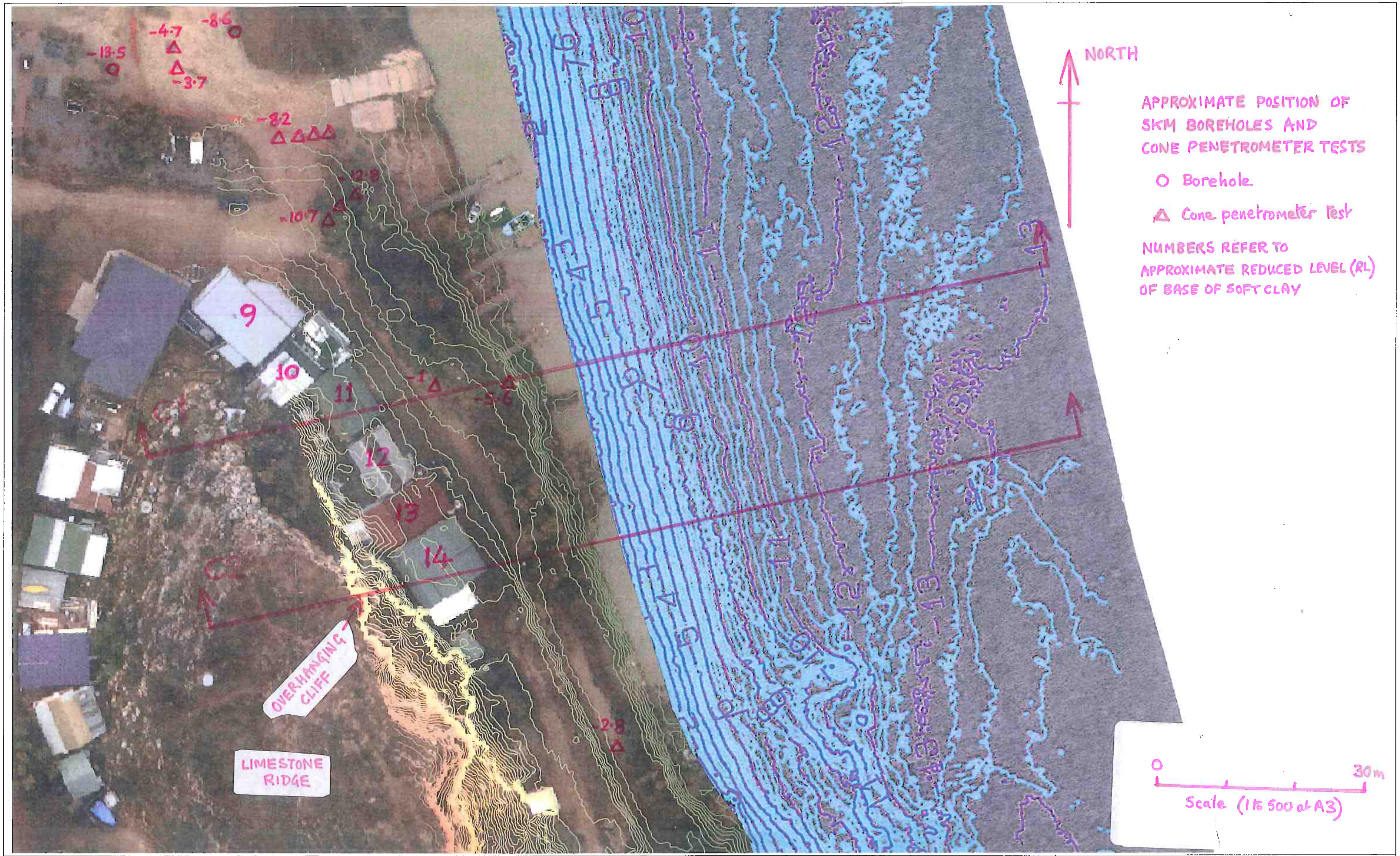
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES

REVIEW OF RIVERBANK RISK MANAGEMENT OPTIONS. FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA

EAST FRONT ROAD, MANNUM: SECTIONS

project no: GEOTMEND07093AA | figure no: FIGURE 3



revision	description	drawn	approved	date

drawn	ATM
approved	ATM
date	12 Dec 2012
scale	As shown
original size	A3

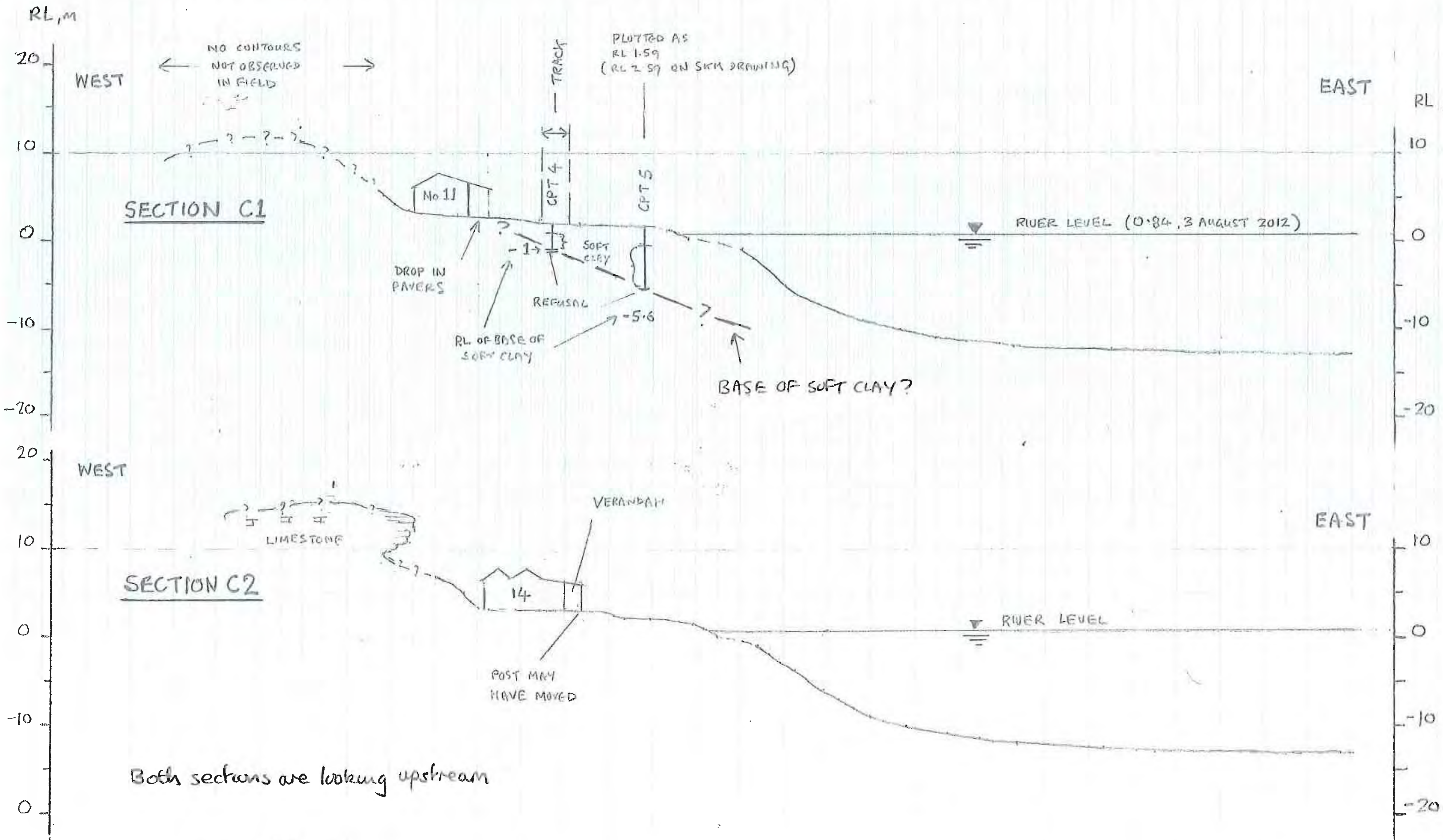
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES

REVIEW OF RIVERBANK RISK MANAGEMENT OPTIONS. FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA

CALOOTE SOUTHERN RESIDENTIAL AREA: LOCATION MAP

project no: GEOTMEND07093AA | figure no: FIGURE 4



NOTE SOFT CLAY DOWN TO RL -13.45 AND LOOSE SAND TO RL -15.95 IN BOREHOLE BH3 (55m UPSTREAM OF TOP SECTION)

0 10 50M

Scale (1 to 500 at A3)

This is a working drawing. We have shown an interpretation of the base of Soft Clay on Section C1. There is also likely to be a wedge of Soft Clay on Section C2

revision	description	drawn	approved	date

drawn	ATM
approved	ATM
date	12 Dec 2012
scale	As shown
original size	A3

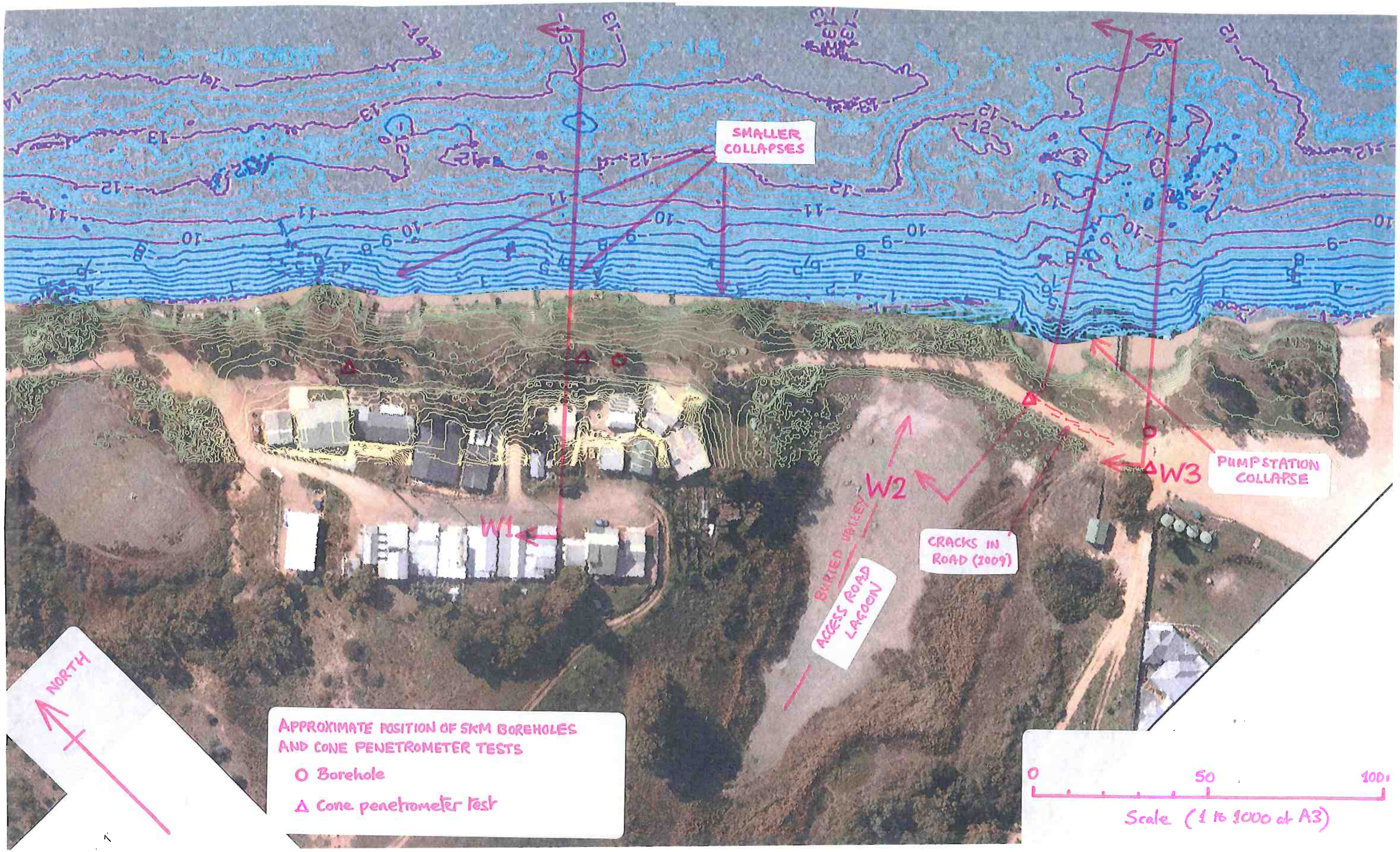
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES

REVIEW OF RIVERBANK RISK MANAGEMENT OPTIONS. FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA

CALOOTE SOUTHERN RESIDENTIAL AREA: SECTIONS

project no: GEOTMEND07093AA | figure no: FIGURE 5



revision	description	drawn	approved	date

drawn	ATM
approved	ATM
date	17 Dec 2012
scale	As shown
original size	A3

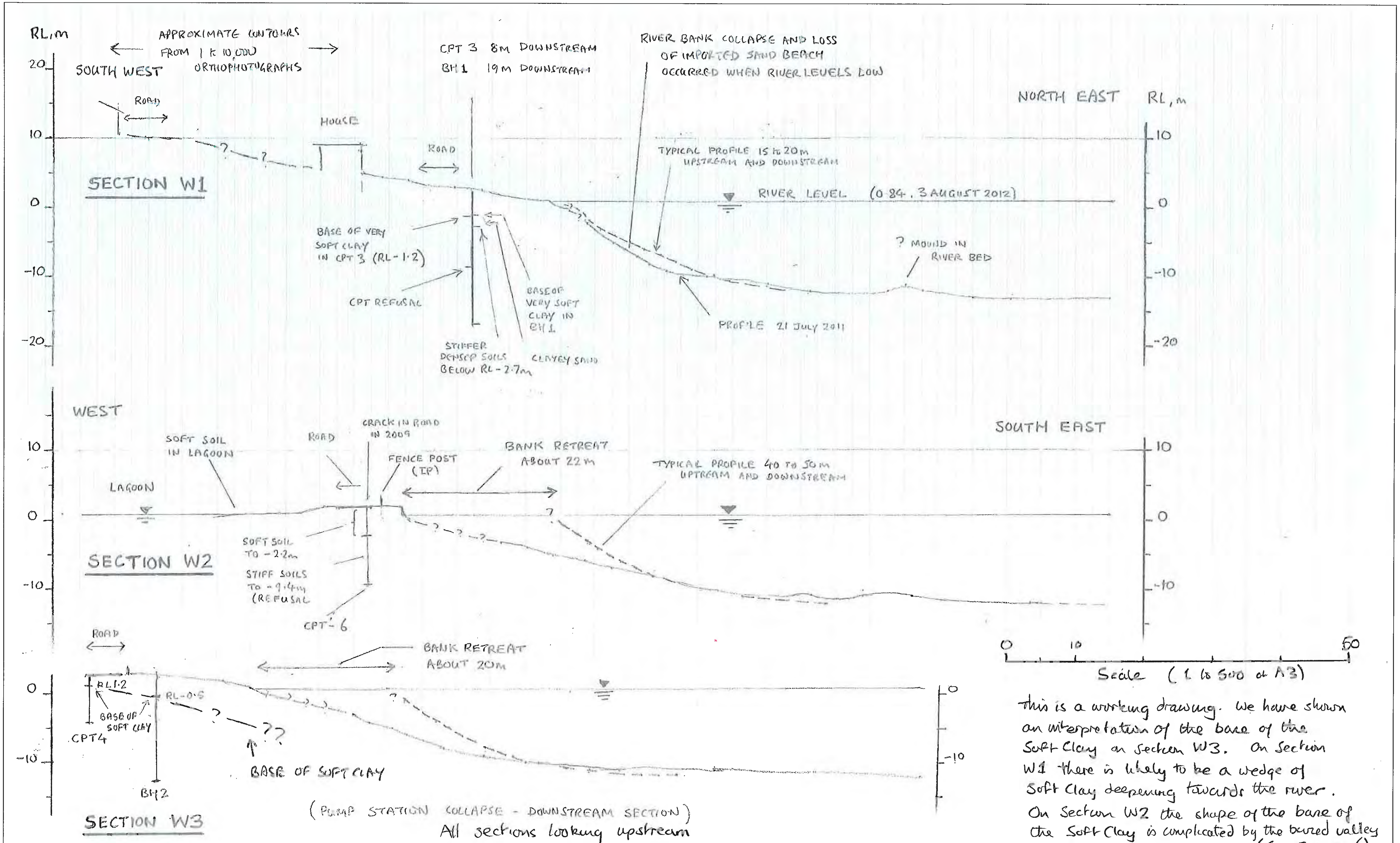
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES

REVIEW OF RIVERBANK RISK MANAGEMENT OPTIONS. FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA

WOODLANE RESERVE: LOCATION MAP

project no: GEOTMEND07093AA | figure no: FIGURE 6



this is a working drawing. We have shown an interpretation of the base of the Soft Clay on Section W3. On Section W1 there is likely to be a wedge of Soft Clay deepening towards the river. On Section W2 the shape of the base of the Soft Clay is complicated by the buried valley (See Figure 6)

revision	description	drawn	approved	date

drawn	ATM
approved	ATM
date	12 Dec 2012
scale	As shown
original size	A3

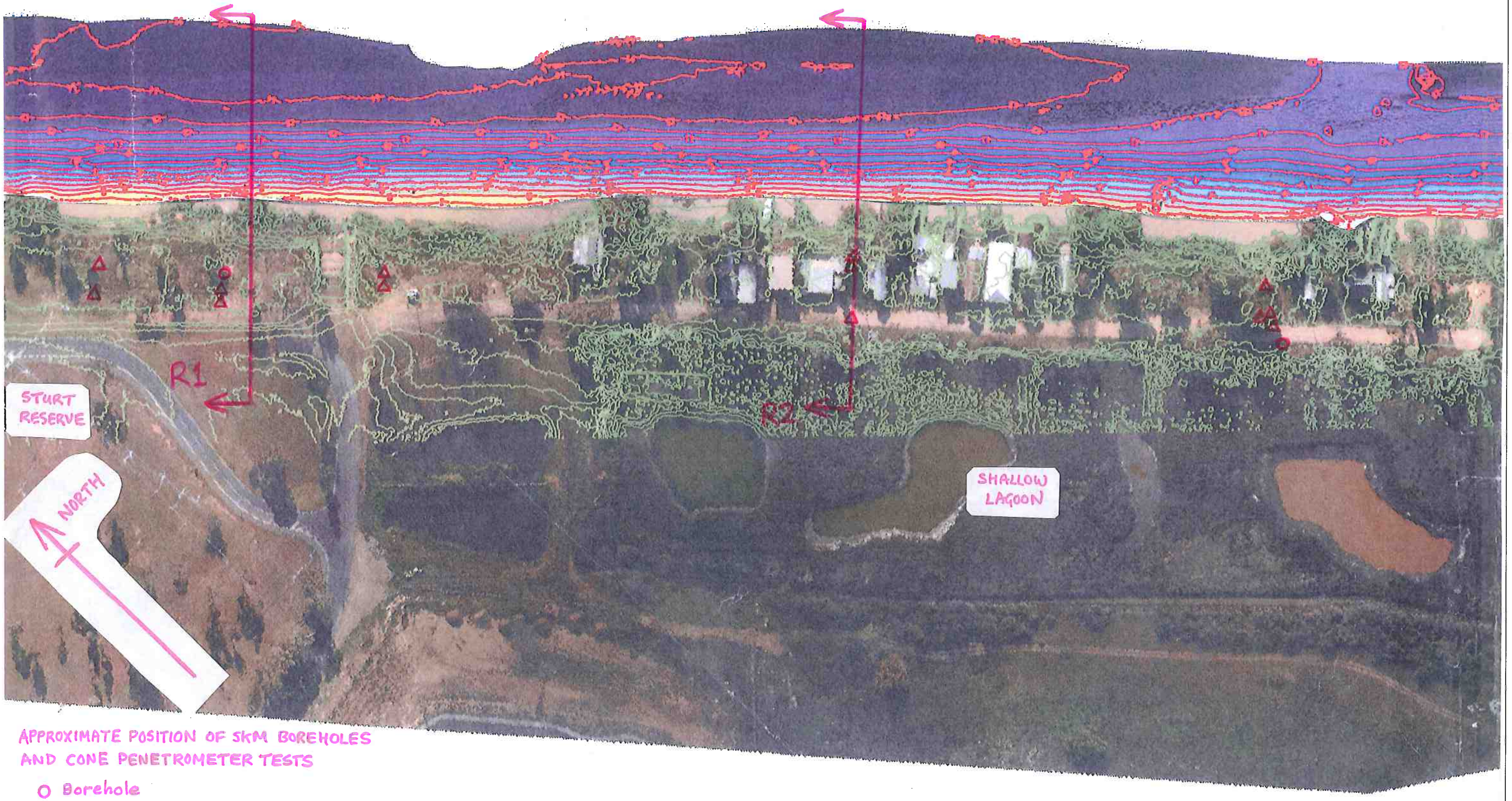
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES

REVIEW OF RIVERBANK RISK MANAGEMENT OPTIONS. FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA

WOODLANE RESERVE: SECTIONS

project no: GEOTMEND07093AA | figure no: FIGURE 7



APPROXIMATE POSITION OF SKM BOREHOLES
AND CONE PENETROMETER TESTS

○ Borehole

△ Cone penetrometer test

0 50 100m

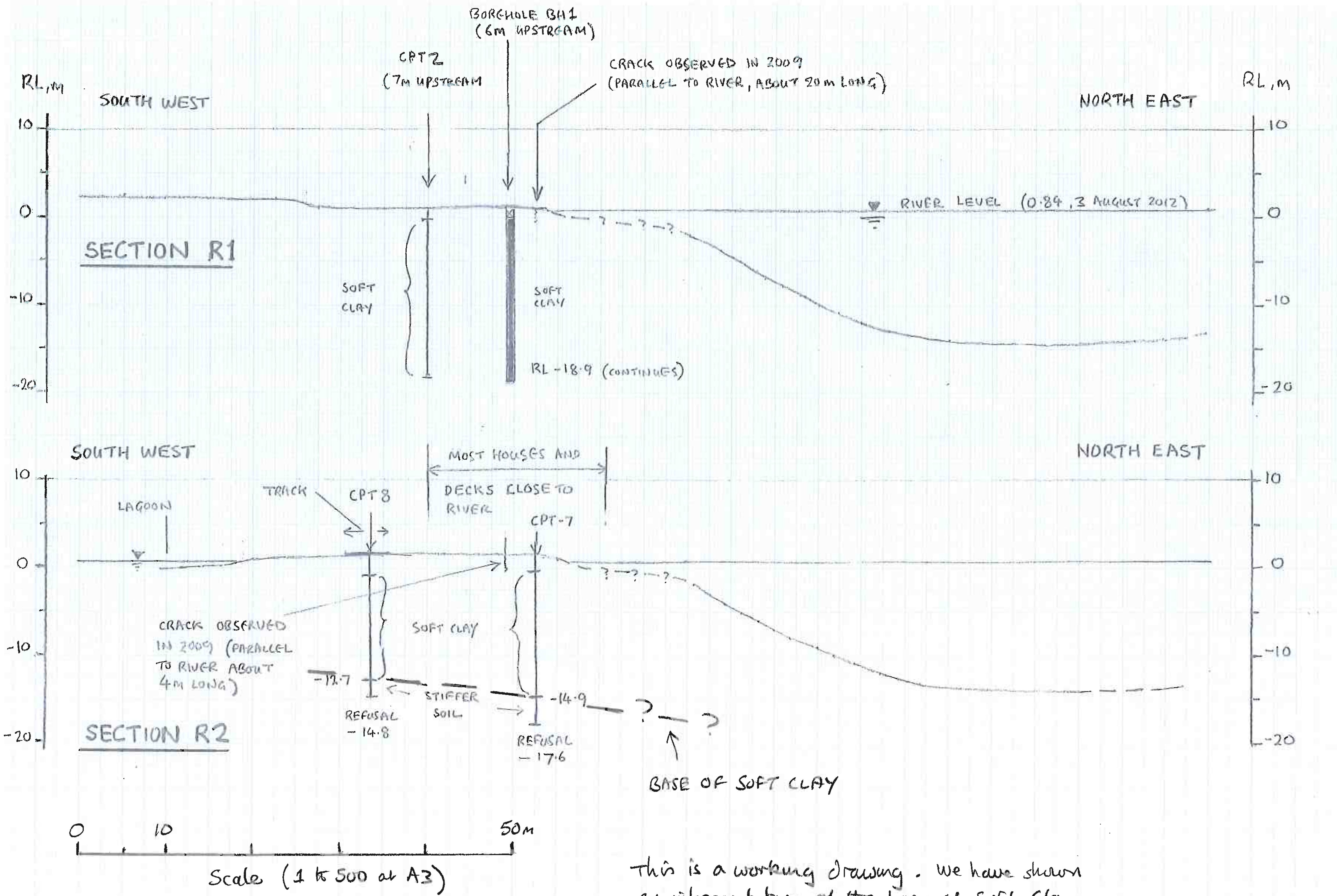
Scale (1 to 1250 at A3)

revision	description	drawn	approved	date

drawn	ATM
approved	ATM
date	12 Dec 2012
scale	As shown
original size	A3

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES	
REVIEW OF RIVERBANK RISK MANAGEMENT OPTIONS. FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA	
RIVER FRONT ROAD, MURRAY BRIDGE: LOCATION MAP	
project no: GEOTMEND07093AA	figure no: FIGURE 8



This is a working drawing. We have shown an interpretation of the base of Soft Clay on Section R2. On Section R1 the base of the Soft Clay is at least 20m below the ground surface.

revision	description	drawn	approved	date						
						drawn	ATM	coffey geotechnics SPECIALISTS MANAGING THE EARTH	DEPARTMENT OF ENVIRONMENT WATER AND NATURAL RESOURCES	
						approved	ATM		REVIEW OF RIVERBANK RISK MANAGEMENT OPTIONS. FOUR SITES ON RIVER MURRAY, SOUTH AUSTRALIA	
						date	12 Dec 2012			
						scale	As shown		RIVER FRONT ROAD, MURRAY BRIDGE: SECTIONS	
						original size	A3		project no: GEOTMEND07093AA	figure no: FIGURE 9

Appendix A

References

APPENDIX A

REFERENCES

1. Sinclair Knight Merz (2010). *Study into riverbank collapsing for Lower River Murray.* Geotechnical investigation report, February 2010.

2. Moon, AT and Wilson, RA (2004). *Will it happen? – Quantitative judgements of landslide likelihood.* Proc. of the 9th Australia New Zealand conf. on Geomechanics, Uni. of Auckland, Vol 2, 754-760.

3. Golder Associates (2010). *East Front Road, Mannum, South Australia; options for reducing the probability of landsliding.* Report reference No. 107662007-009-L-Rev0, 28 April 2010.

4. Liang, C, Jaksa, MB and Ostendorf B (2012). *GIS back analysis of riverbank instability in the Lower River Murray.* Proc. of the 11th Australia New Zealand conf. on Geomechanics, Melbourne.

5. Sinclair Knight Merz (2010). *River bank collapse hazard – Lower River Murray; stability risk assessment for Caloote Landing.* 11 November 2010.

Appendix B

Brief overview of other riverbank sites

APPENDIX B

BRIEF OVERVIEW OF OTHER RIVERBANK SITES

B1 Sites assessed and excel slope profiles

Most of our field review focussed at the four sites in question. However, as discussed in Section 1.4 we have carried out a brief overview of other sites where collapse or cracking have been observed to help understand the hazards.

For the four sites assessed in this report we have prepared working cross sections based on bathymetry, land contours based on digital elevation models (DEMs) provided by the DEWNR, and available subsurface information (Figures 3, 5, 7 and 9). For those four sites we have also prepared slightly simplified Excel slope profiles. For the following other sites we have prepared Excel slope profiles from the bathymetry.

> Long Island Marina (2 profiles); Freds Landing (2); White Sands (2); McRae Road, Bells Landing Reserve; Murrayview Estates; Thiele Reserve.

Where collapse has occurred we show both the collapse profile and an interpretation of the pre-failure profile based on the riverbank profile upstream and downstream of the collapse. For the other sites, the Excel profiles show the approximate position of the present river bank. The slope profiles are given in this appendix.

We also visited the following sites where cracking has been observed but collapse has not occurred.

> Mannum Caravan Park; East Front Road (Younghusband); Murrawong; Sunnyside; Dixon Reserve; Rustic Cottages; Placid Estate; Walker Flat.

We have not prepared slope profiles of the above sites (where there are cracks but no collapse) for this review.

B2 Initial comments on sites where riverbank collapse has occurred

In Section 3.1 we have subdivided riverbank collapse on the basis of the amount of riverbank lost (regression). In Type A failures more than 15 m of riverbank is lost and in Type B failures less than 15 m of bank is lost. Clearly Type A failures have the greatest potential to cause loss of life or damage. As discussed in Section 3.2 and Appendix C we have reassessed Type A failures at Long Island Marina and Woodlane Pumpstation in order to better understand the conditions that lead to instability, how the slopes fail and why large riverbank regressions can occur.

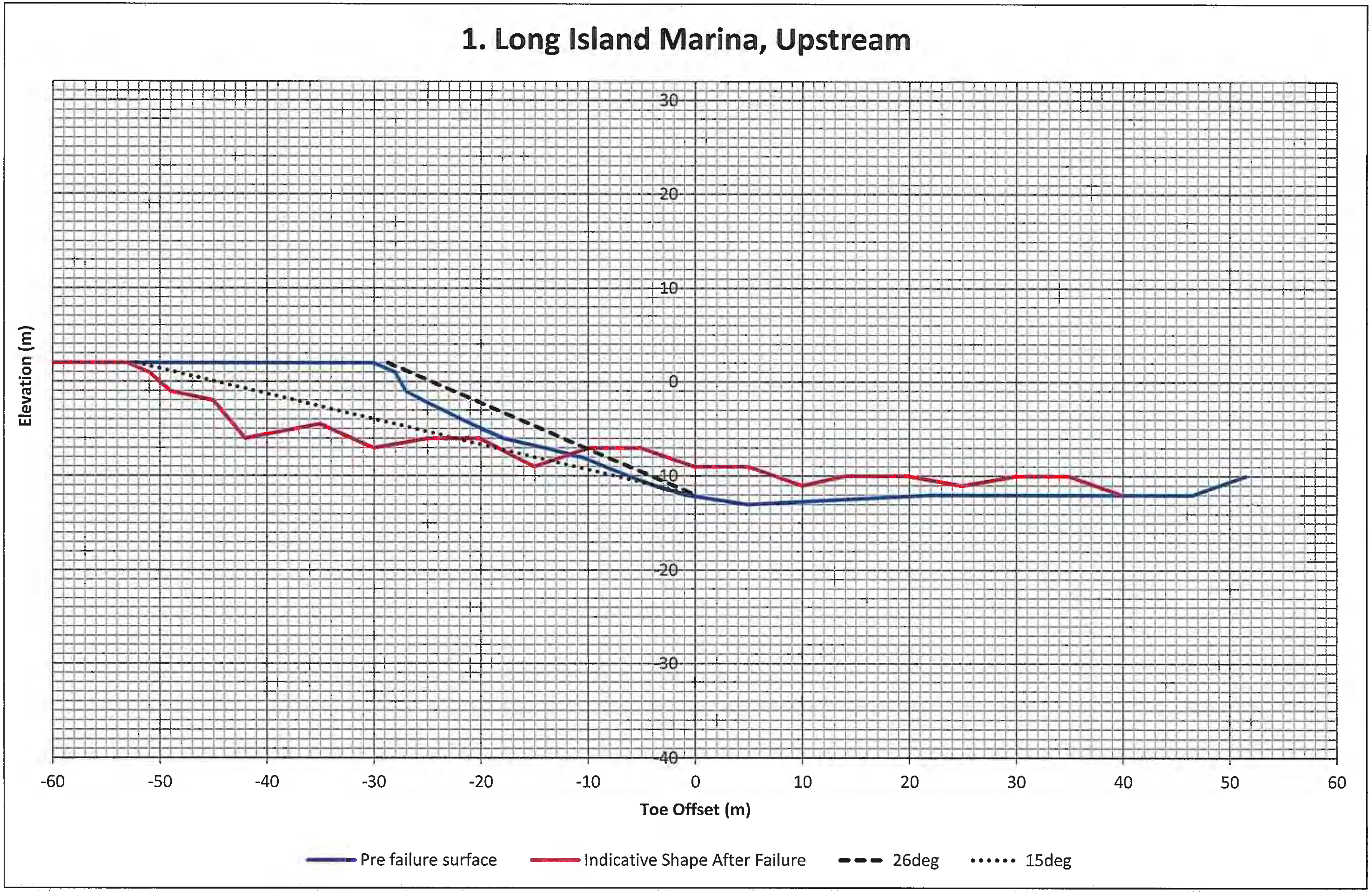
We have also carried out a brief overview of some of the Type B collapses that have occurred. These are represented by Excel Slope Profiles 1 to 6, 9 and 10. We have adjusted the profiles so that zero corresponds approximately to the toe of the original riverbank. In Profiles 24 and 25 we have overlaid the pre-failure and post-failure geometries of the Type B failures we briefly overviewed. Even allowing for some uncertainty in slope geometries it appears that riverbank collapse can occur on a wide range of initial riverbank slopes (e.g. ranging from between about 18^0 and 30^0). We understand is that some of these collapses appear to have occurred where there appears to be little or no fill.

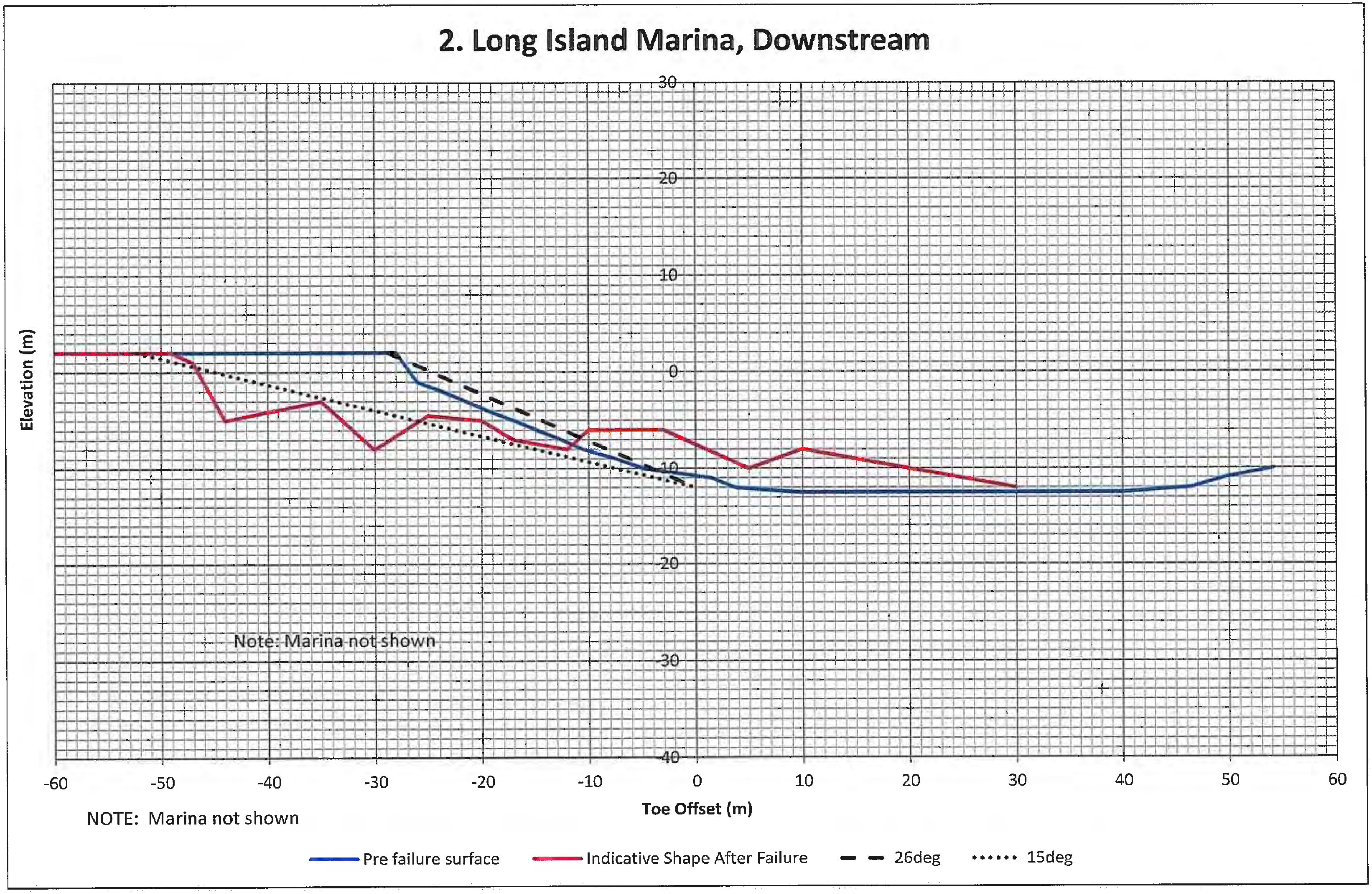
To date we have only carried out a brief overview of some of the sites. As discussed in Section 6 of this report a great deal more could be learned from a thorough study and back analysis of all the collapse failures that have occurred along the lower River Murray.

Excel slope profiles used for slope comparisons

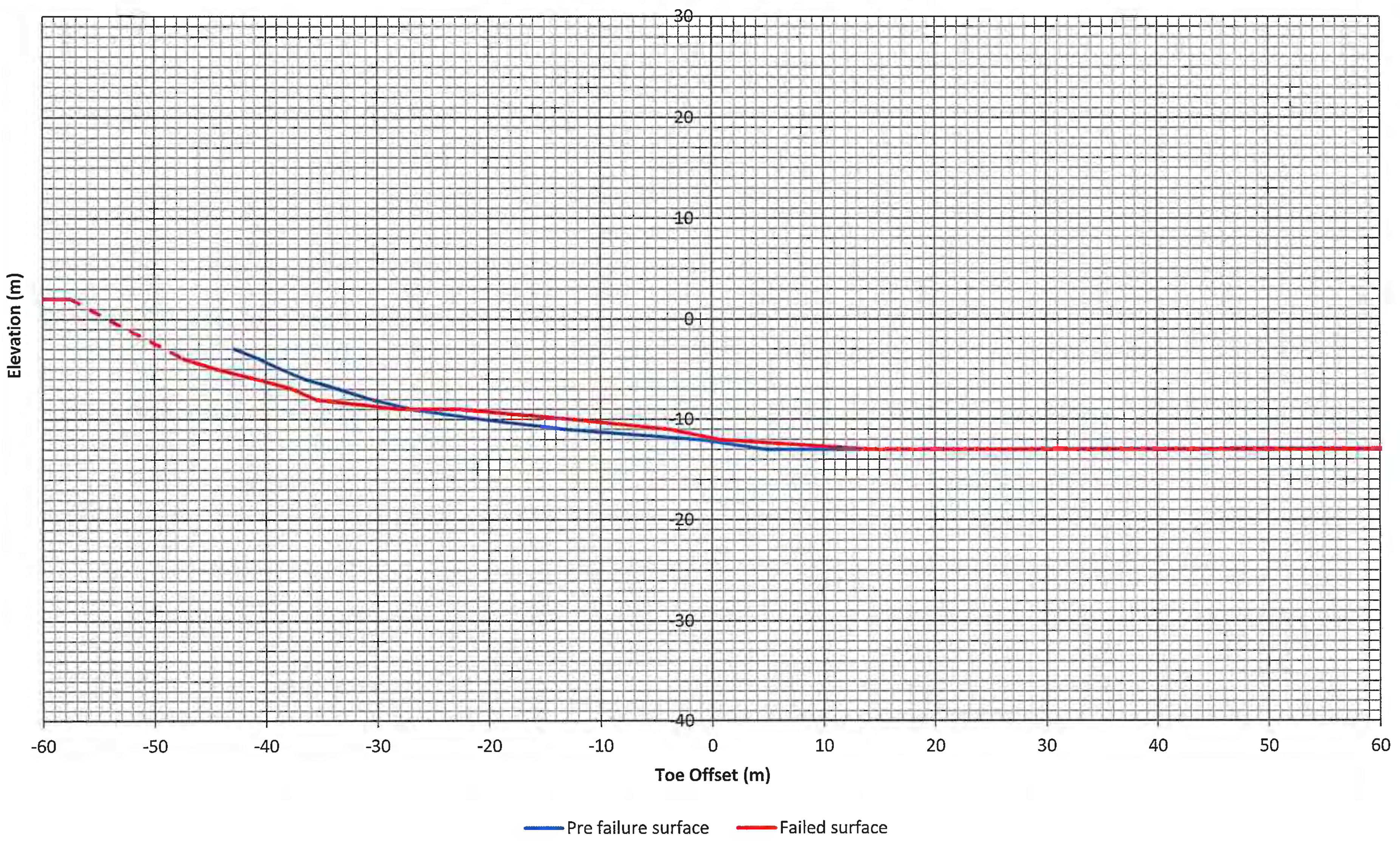
NOTES:

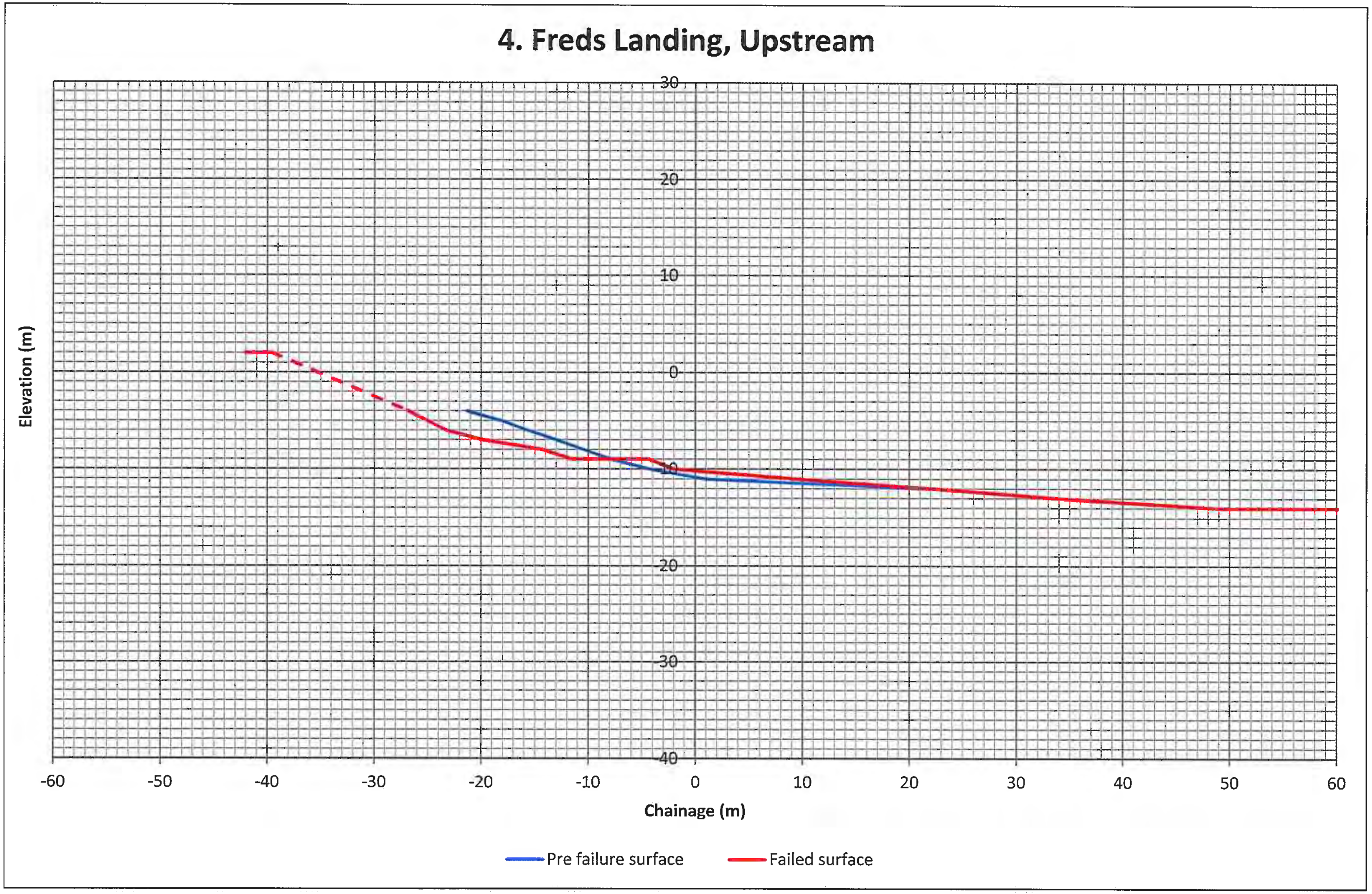
1. Profiles 1 and 2 are based on bathymetry and maps provided by DEWNR.
2. Profiles 3 to 6, 9 and 10 are based on bathymetry (provided by DEWNR) only. The position of the riverbank noted on the profiles is approximate only. It is based on the approximate position of the riverbank upstream and downstream of the line of the profile where visible on aerial photographs provided with the bathymetry.
3. Profiles 11 to 21 are based on bathymetry and DEM provided by DEWNR. The profiles are slightly simplified versions of the cross sections given on Figures 3, 5, 7 and 9 in the main text.
4. The original slope profiles have been adjusted so that zero corresponds approximately to the toe of the original riverbank.
5. For slope profiles 3 to 6, 9 and 10 the approximate post failure profile above the available bathymetry is shown as a dashed line. It is approximate only.
6. The pre-failure and post-failure geometry of slope profiles 1, 2, 12 and 13 have been overlaid on single plots on profiles 22 and 23 (as discussed in Section C2 in Appendix C). These are Type A failures as discussed in Section 3.1.
7. The pre-failure and post-failure geometry of slope profiles 3 to 6, 9 and 10 have been overlaid on single plots on profiles 24 and 25. These are Type B failures as discussed in Section 3.1 and in Section B2 of this appendix.

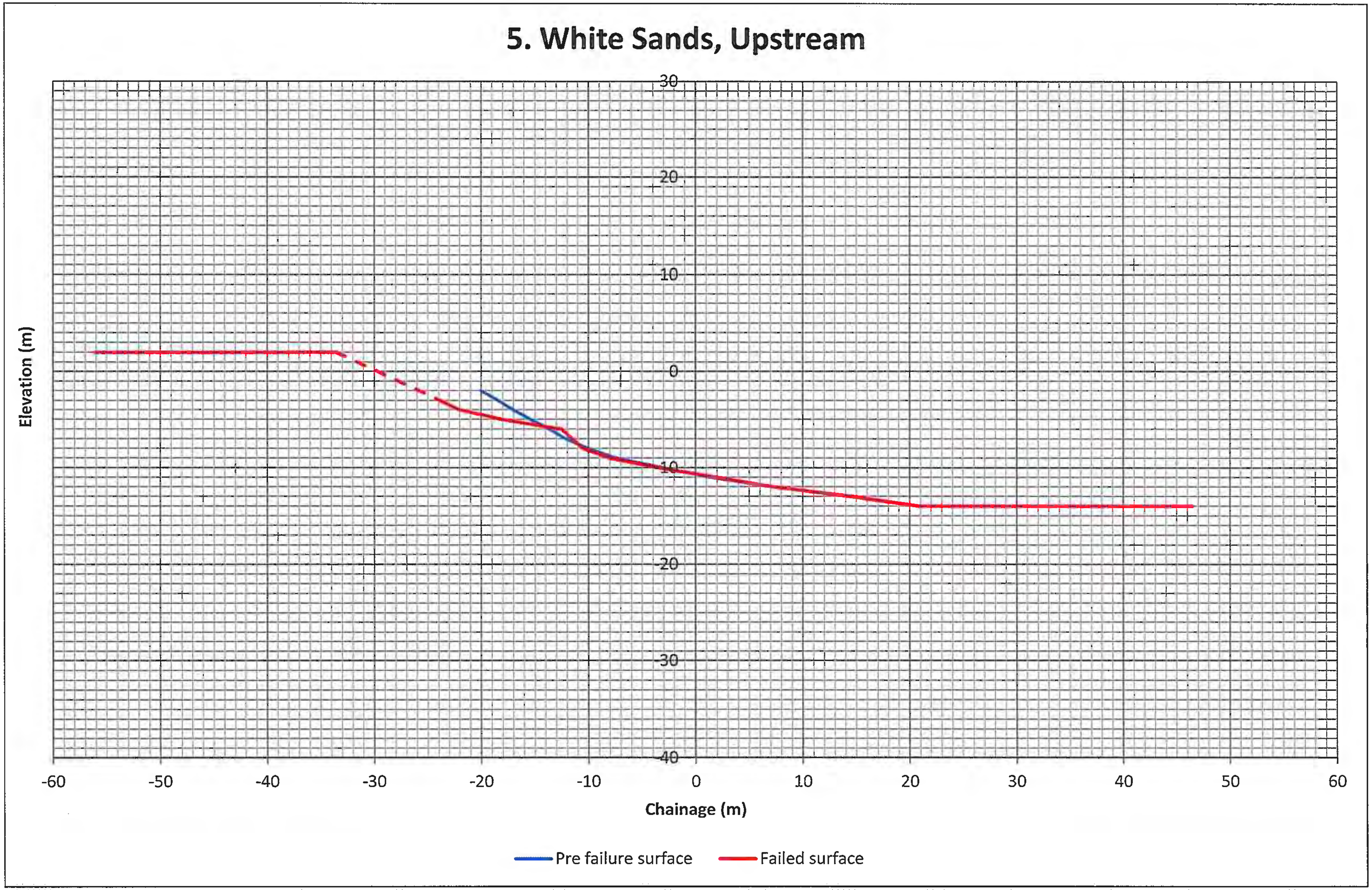

1. Long Island Marina, Upstream
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Toe Offset (m)
Pre failure surface
Indicative Shape After Failure
26deg
15deg


2. Long Island Marina, Downstream
Elevation (m)
30
20
10
0
-10
-20
-30
-40
Note: Marina not shown
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Toe Offset (m)
NOTE: Marina not shown
Pre failure surface
Indicative Shape After Failure
26deg
15deg

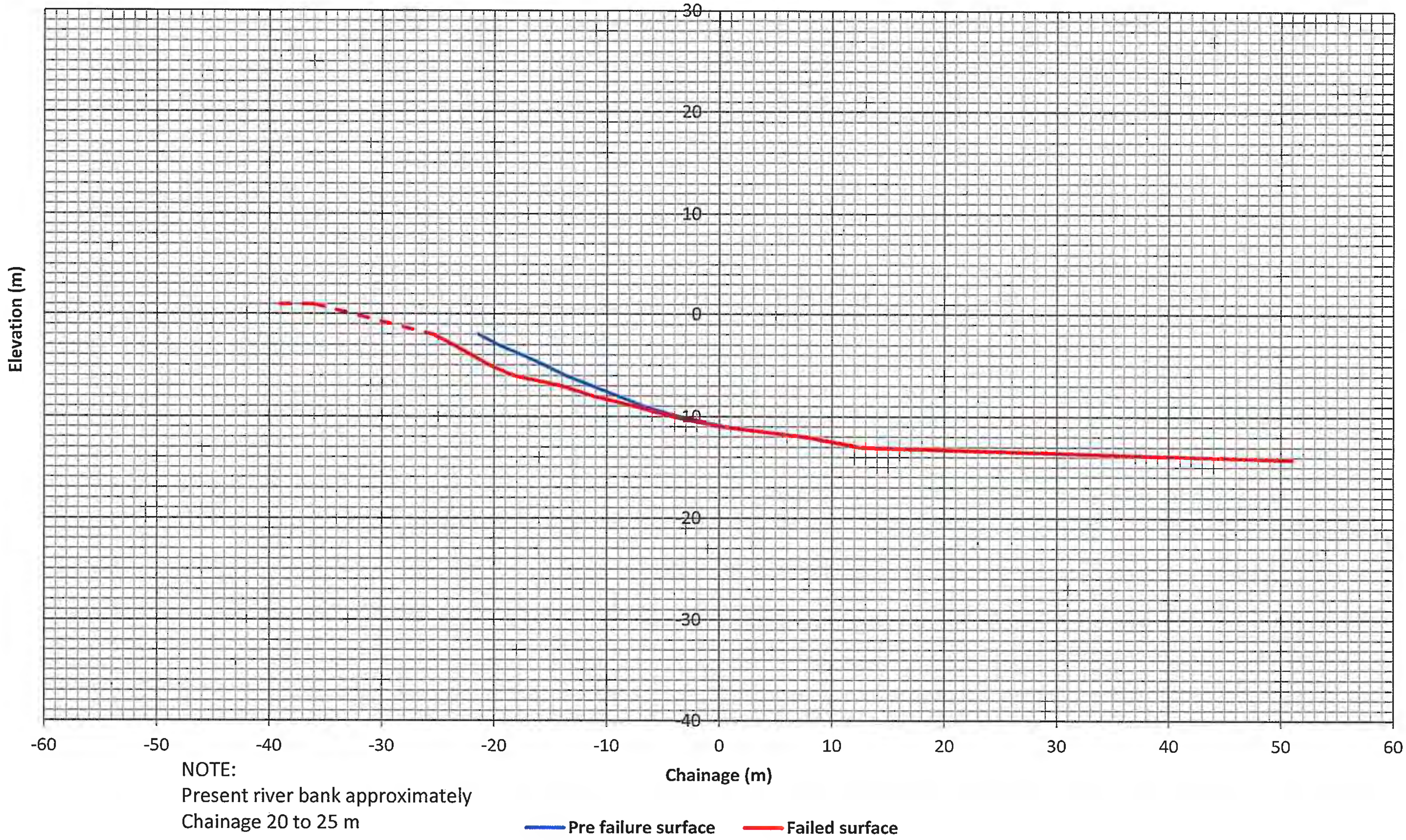
3. Freds Landing, Downstream



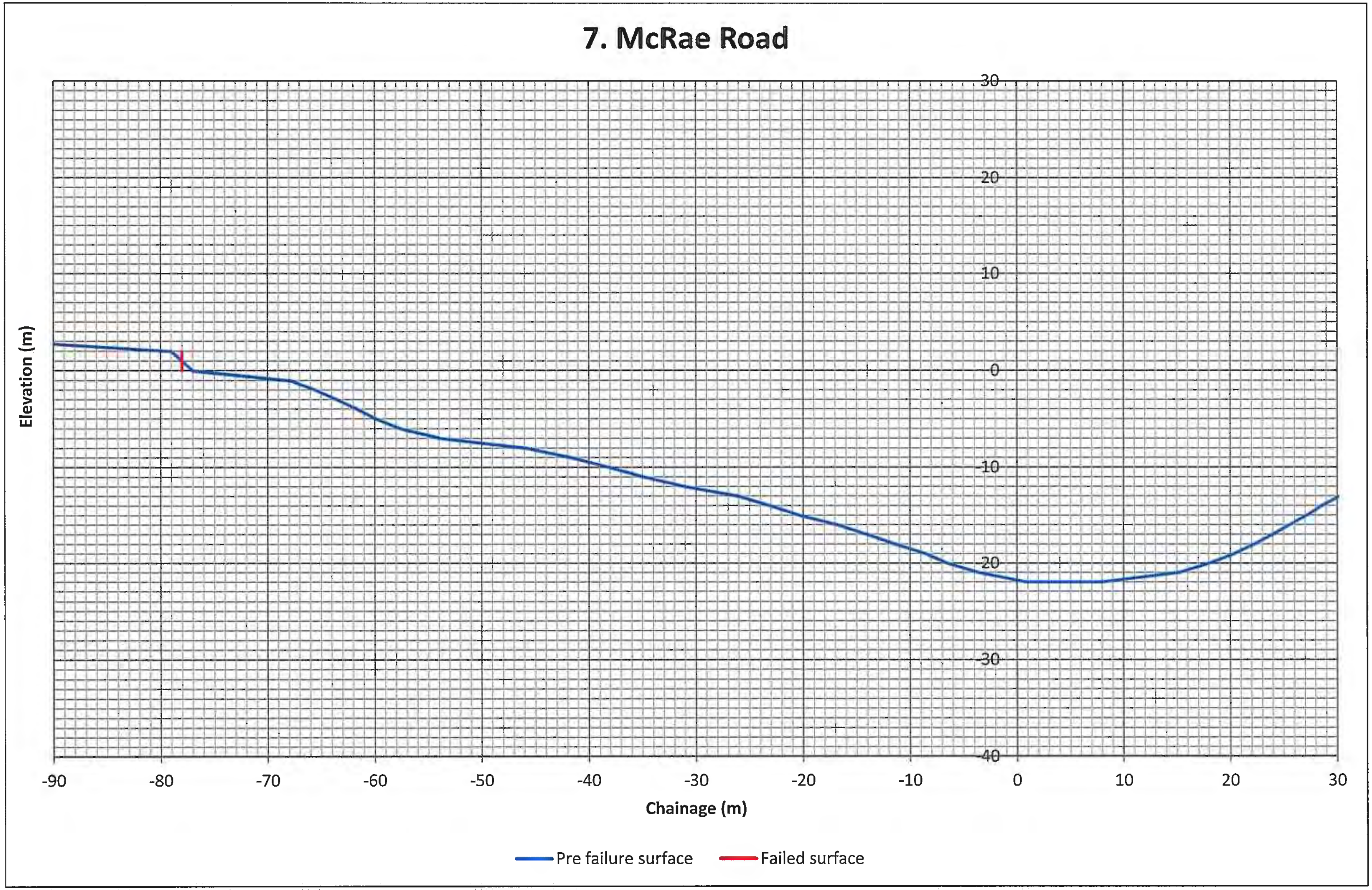

4. Freds Landing, Upstream
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Chainage (m)
Pre failure surface
Failed surface


5. White Sands, Upstream
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Chainage (m)
Pre failure surface
Failed surface

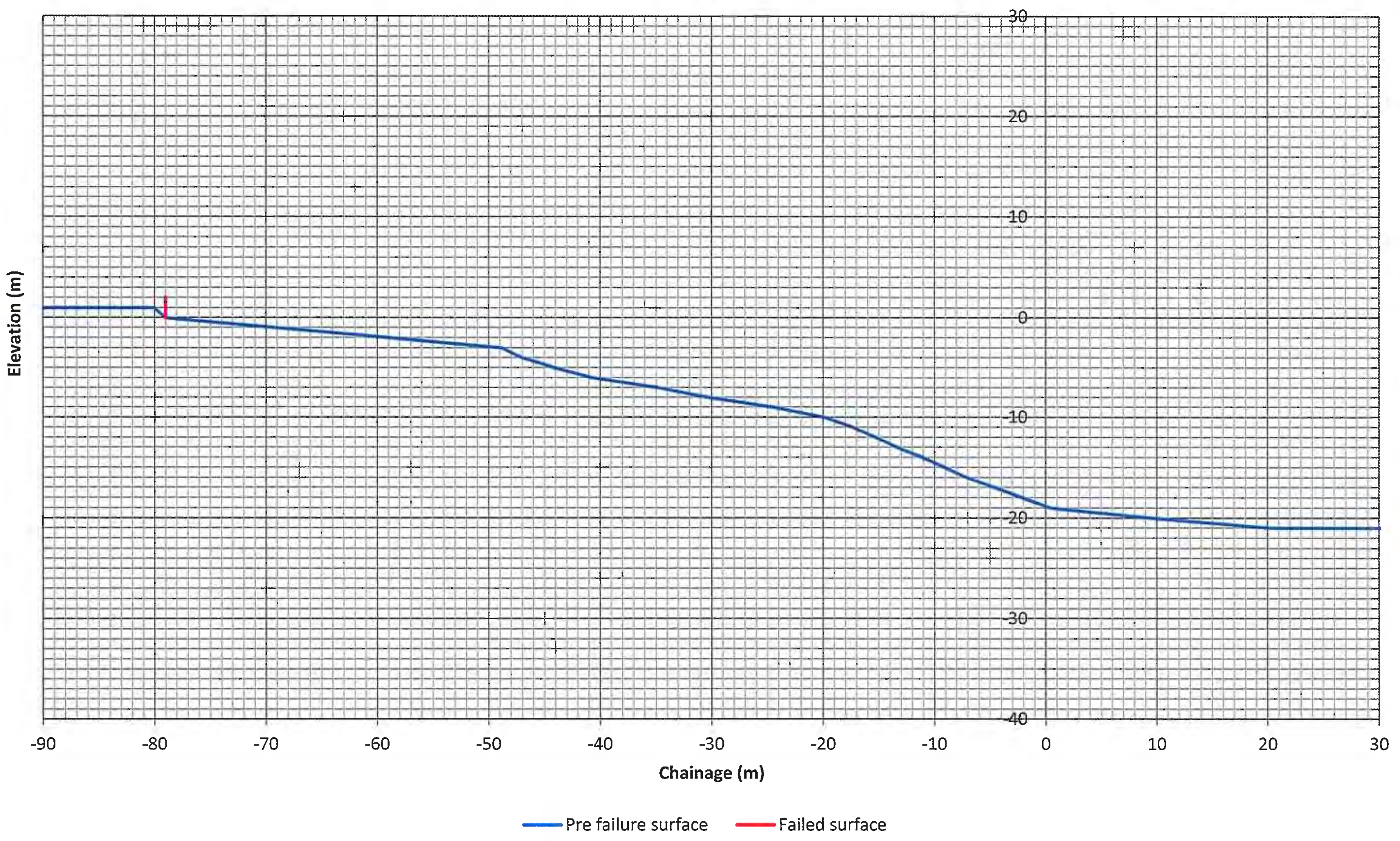
6. White Sands, Downstream

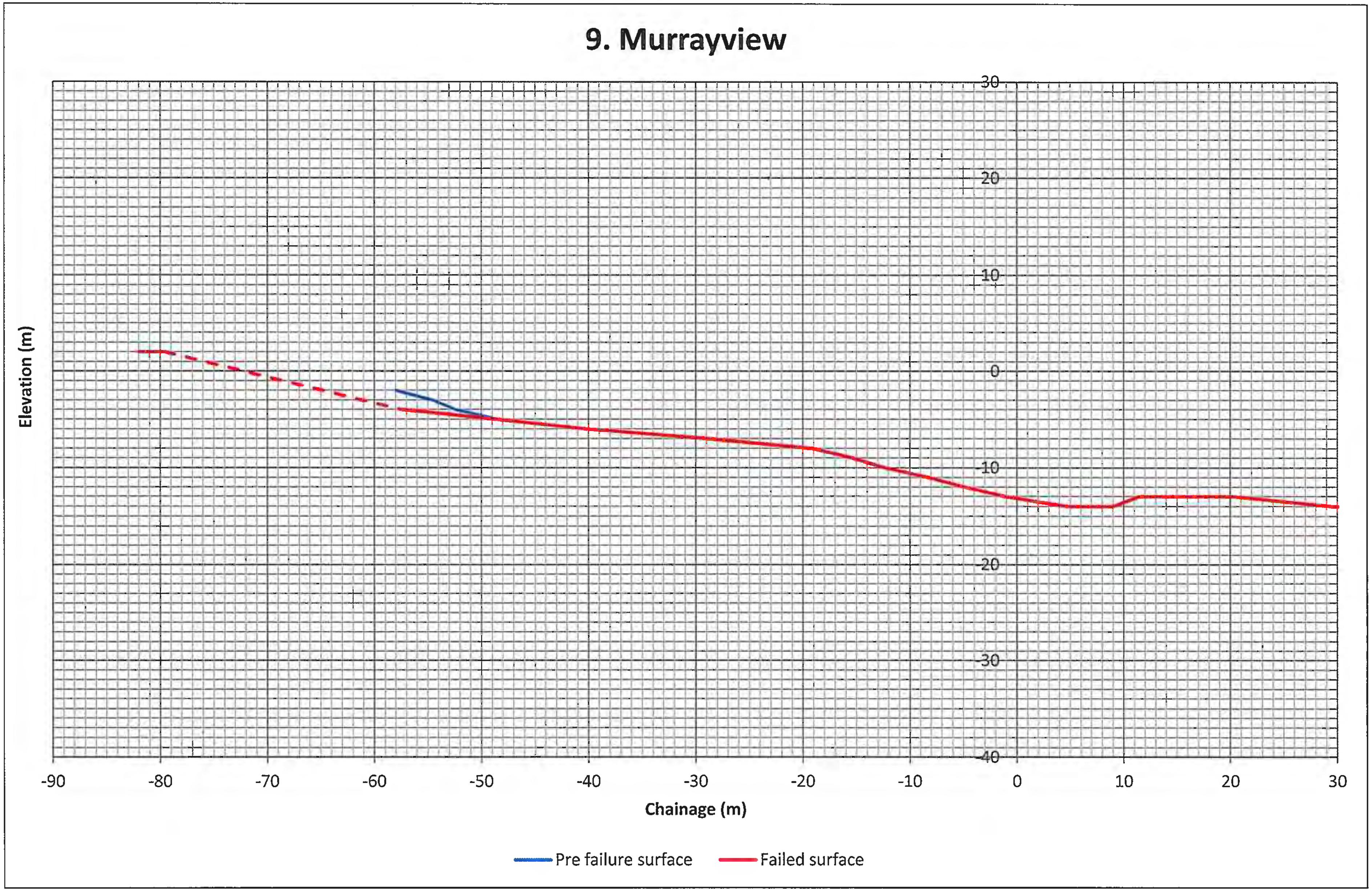


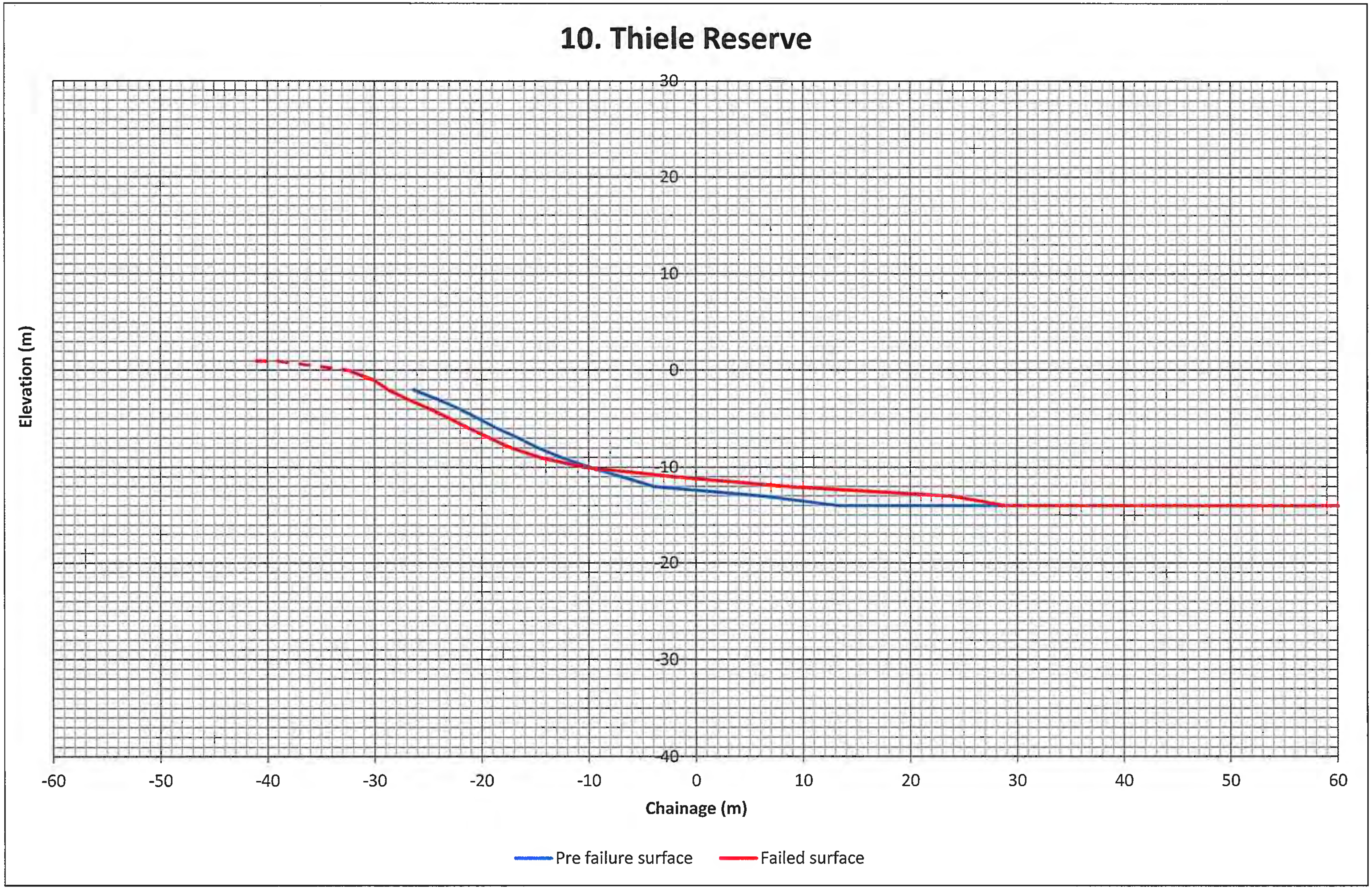
NOTE:
Present river bank approximately
Chainage 20 to 25 m


7. McRae Road
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Chainage (m)
Pre failure surface
Failed surface

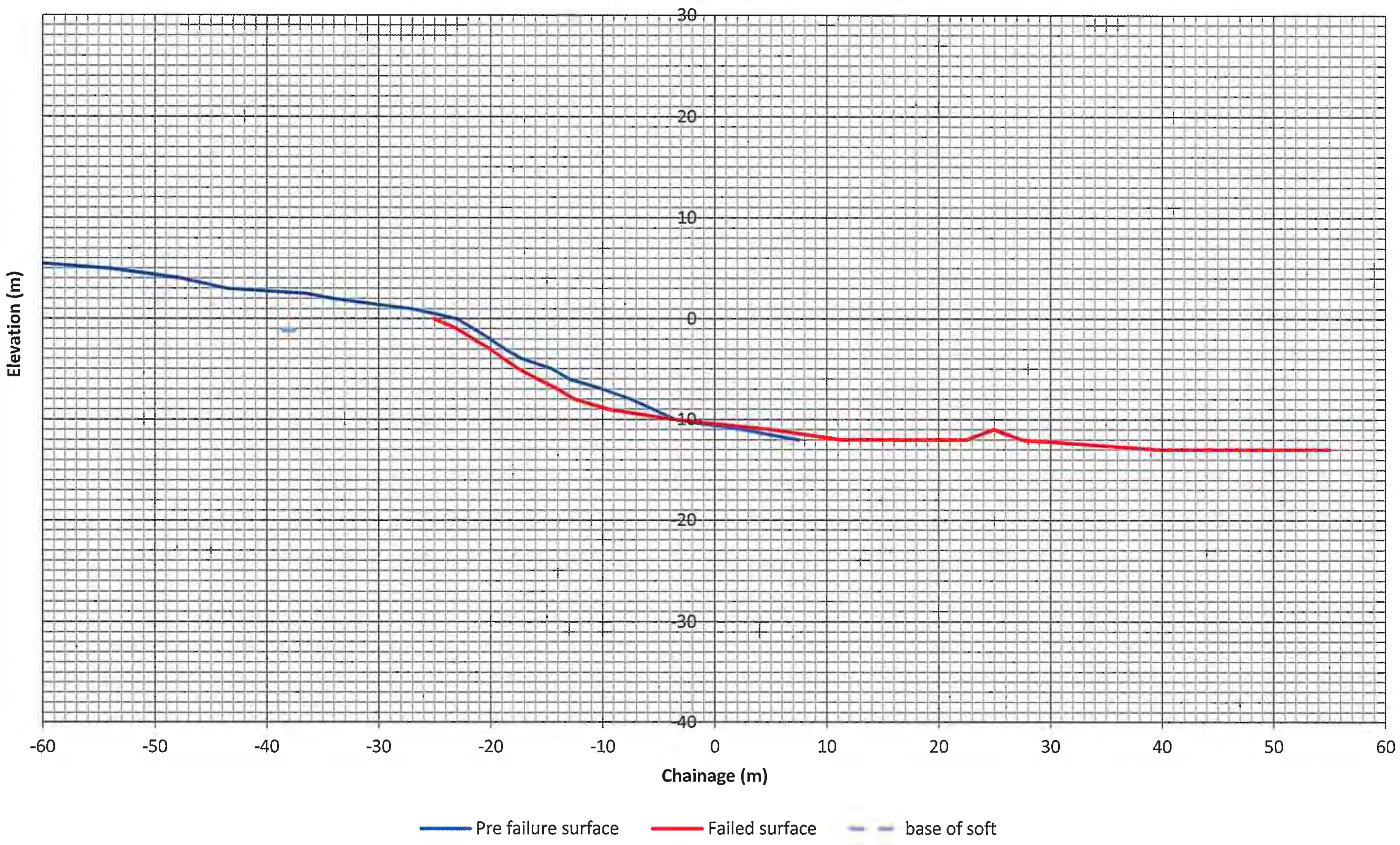
8. Bells Reserve



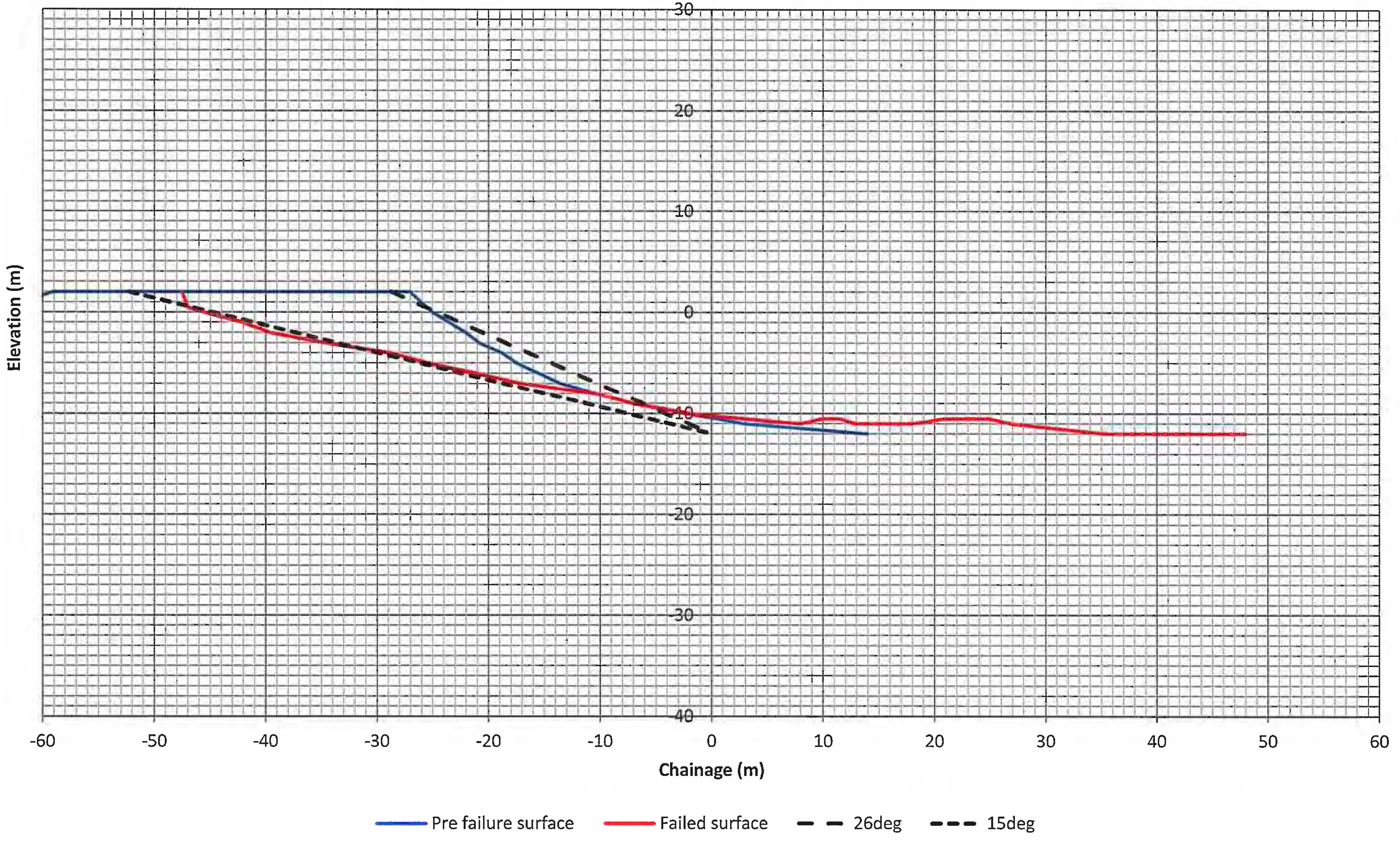

9. Murrayview
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Chainage (m)
Pre failure surface
Failed surface


10. Thiele Reserve
Elevation (m)
Chainage (m)
30
20
10
0
-10
-20
-30
-40
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Pre failure surface
Failed surface

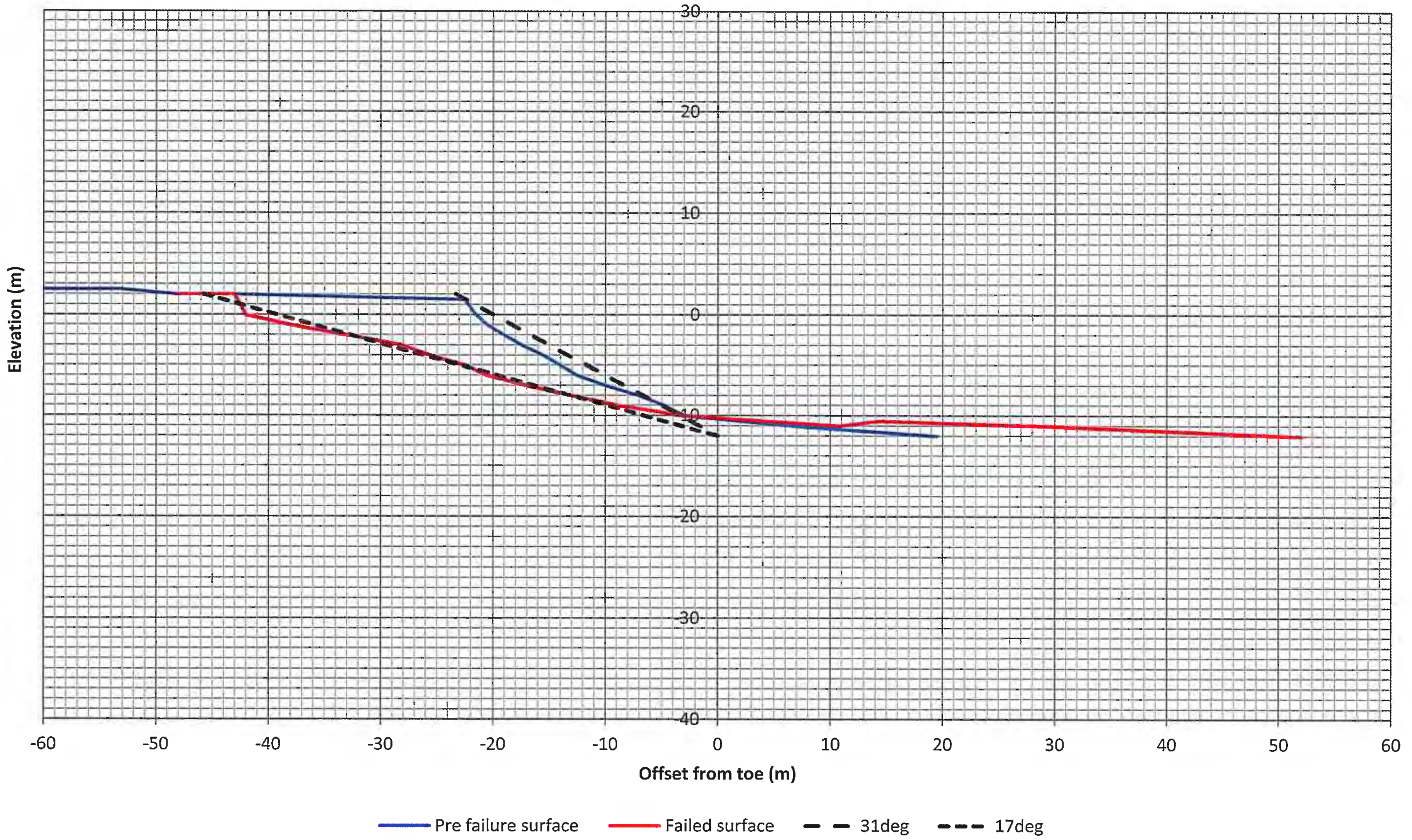
11. Woodlane Reserve (houses)

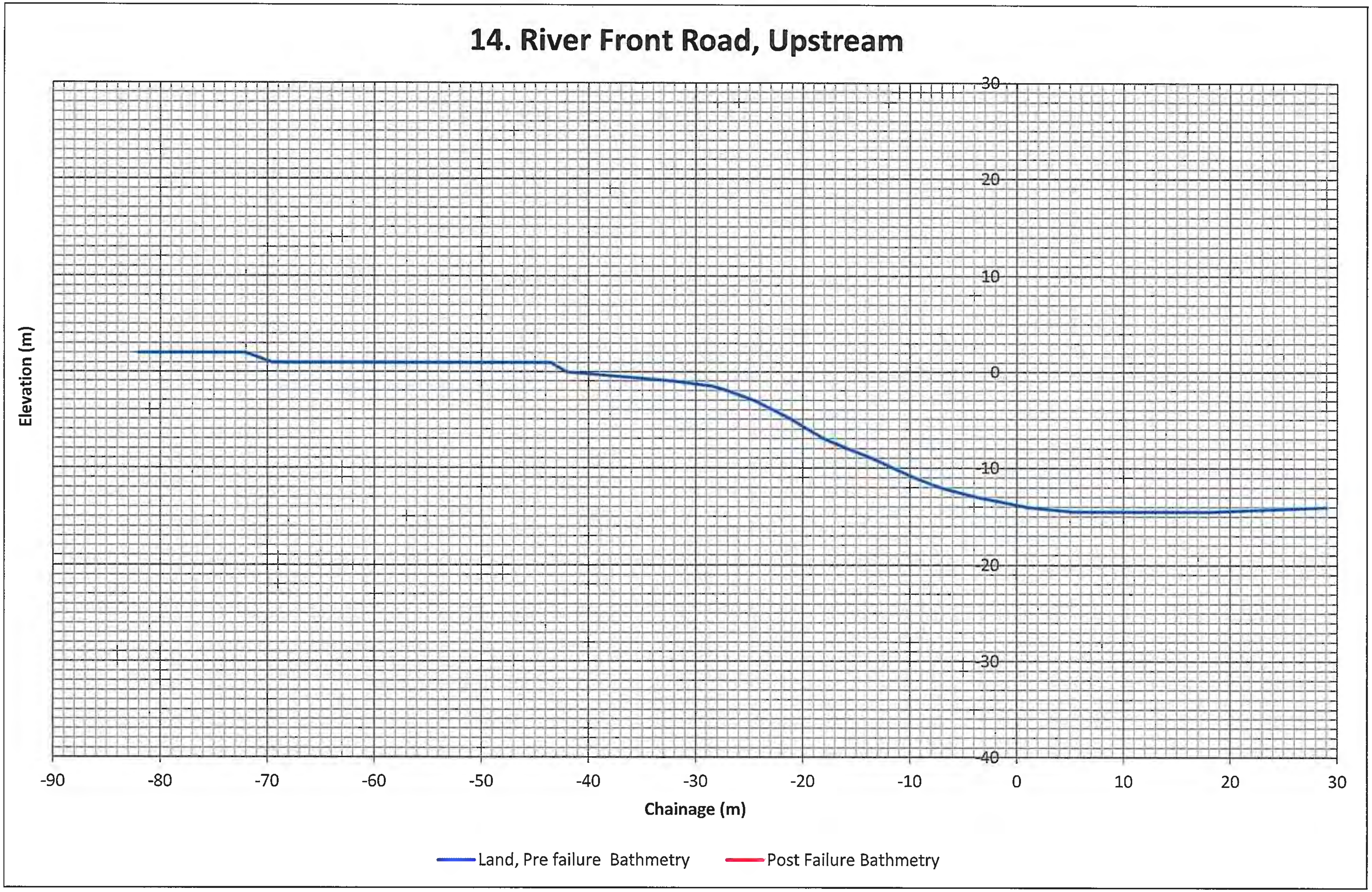


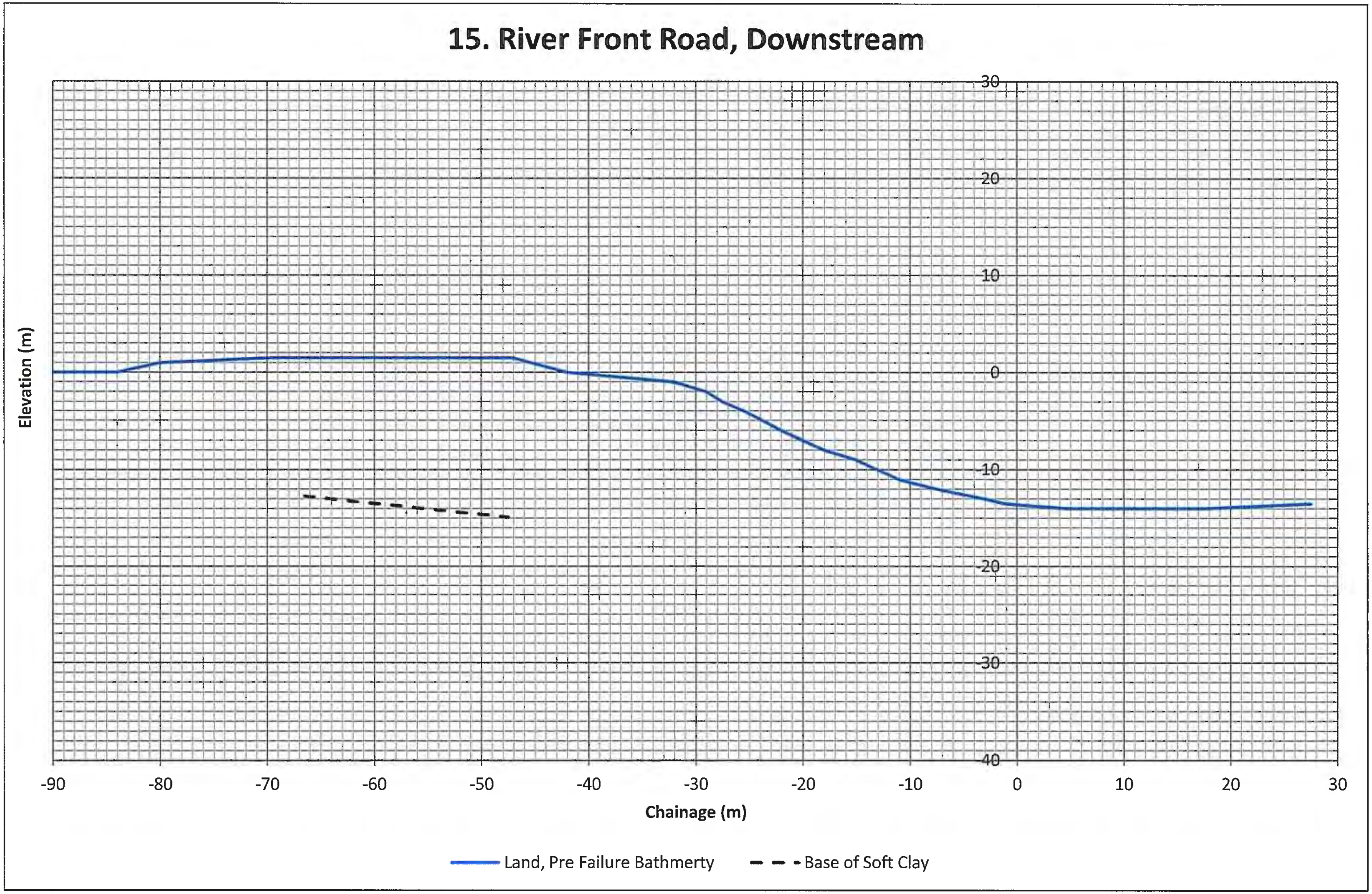
12. Woodlane Reserve (pump station upstream)

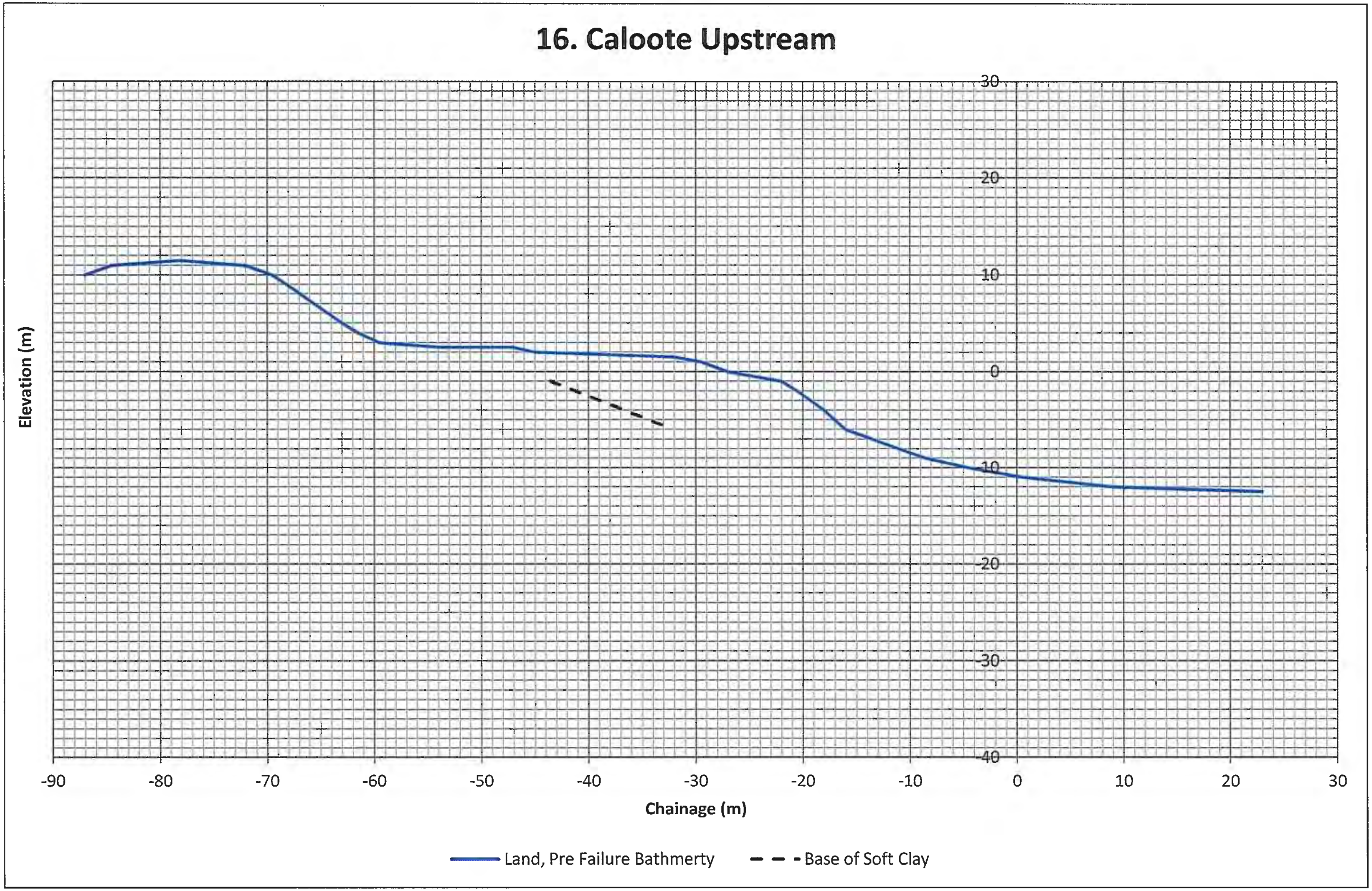


13. Woodlane Reserve (pump station downstream)

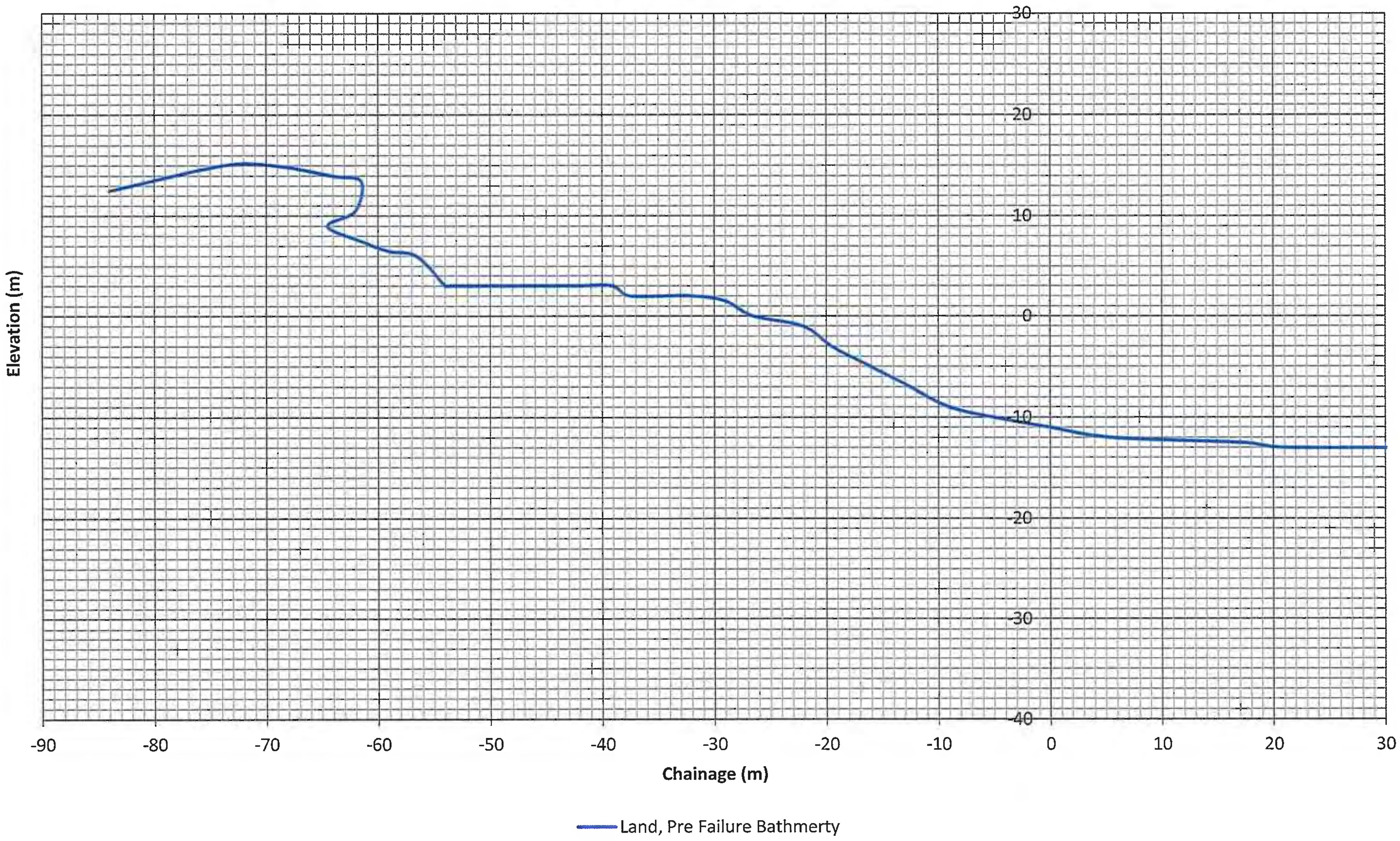



14. River Front Road, Upstream
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Chainage (m)
Land, Pre failure Bathmetry
Post Failure Bathmetry

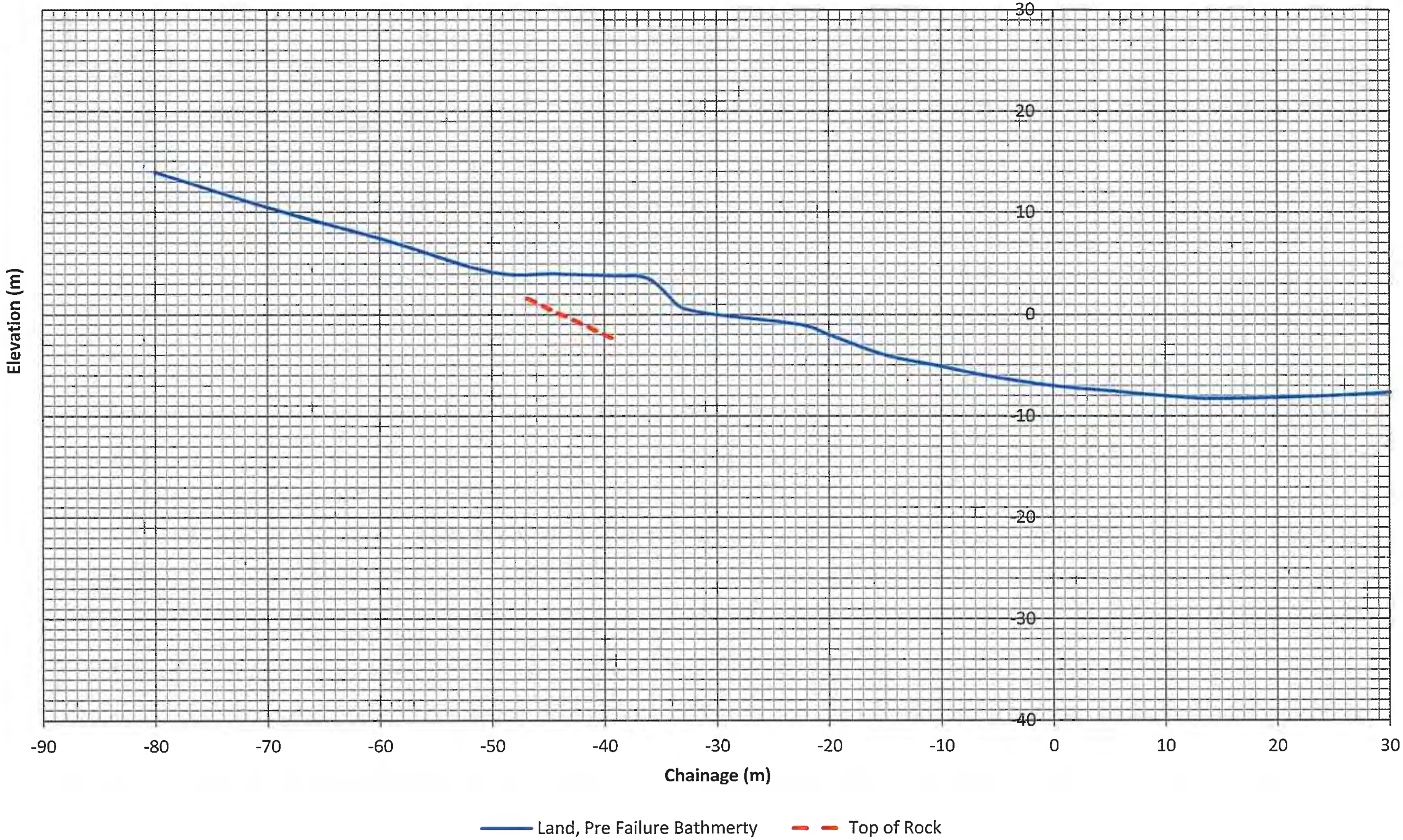

15. River Front Road, Downstream
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Chainage (m)
Land, Pre Failure Bathmerty
Base of Soft Clay

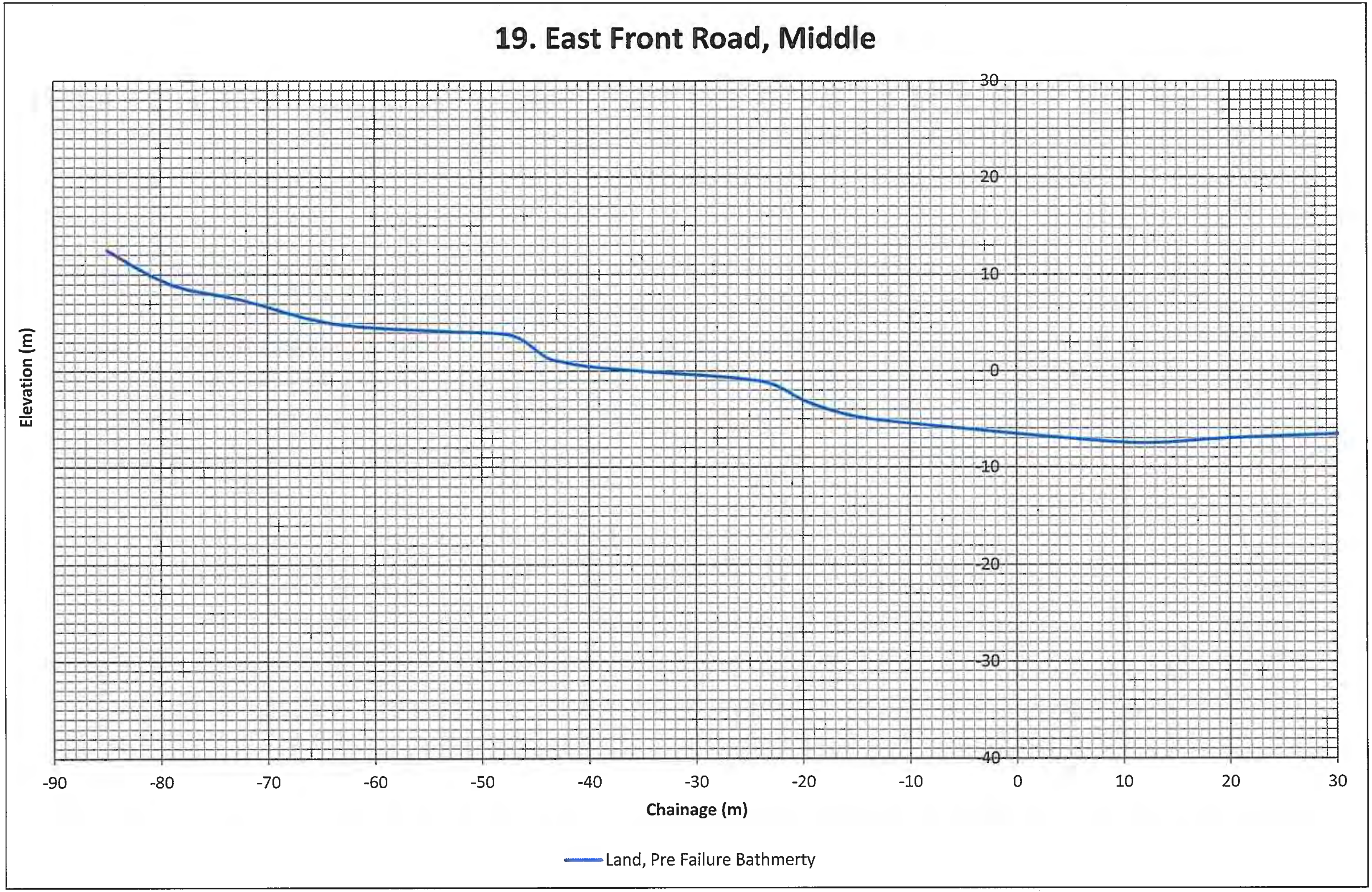

16. Caloote Upstream
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Chainage (m)
Land, Pre Failure Bathmerty
Base of Soft Clay

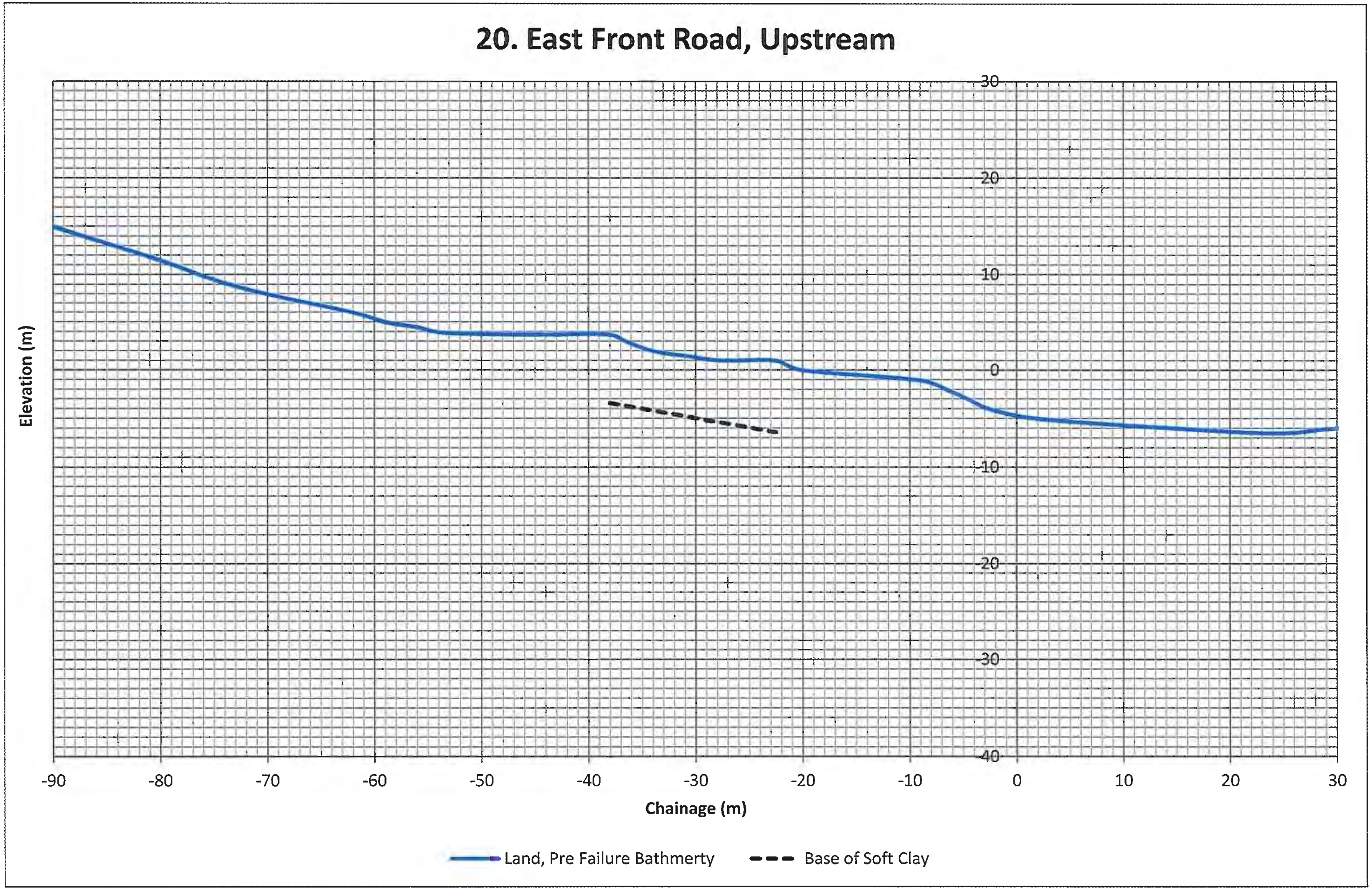
17. Caloote Downstream

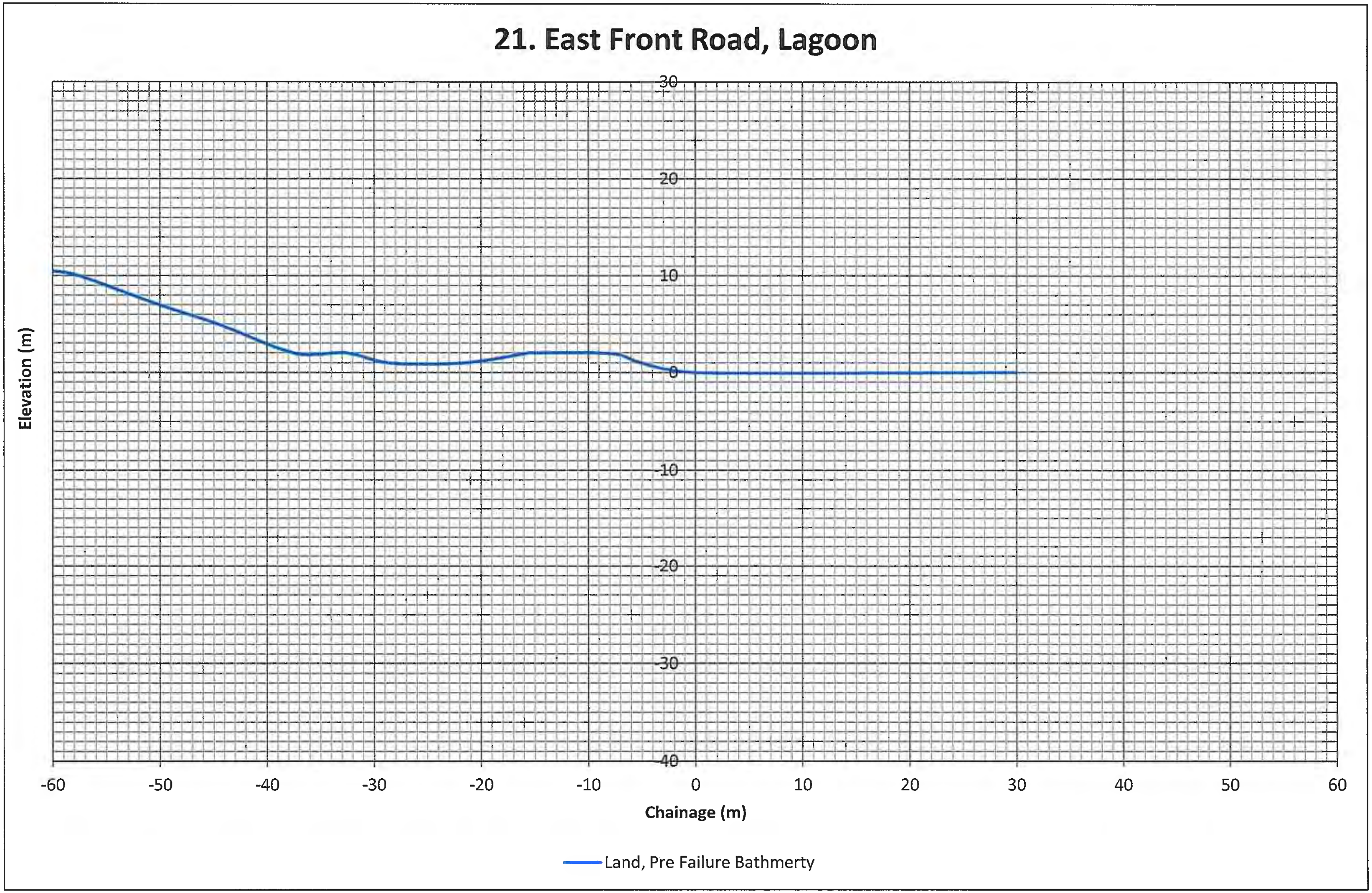


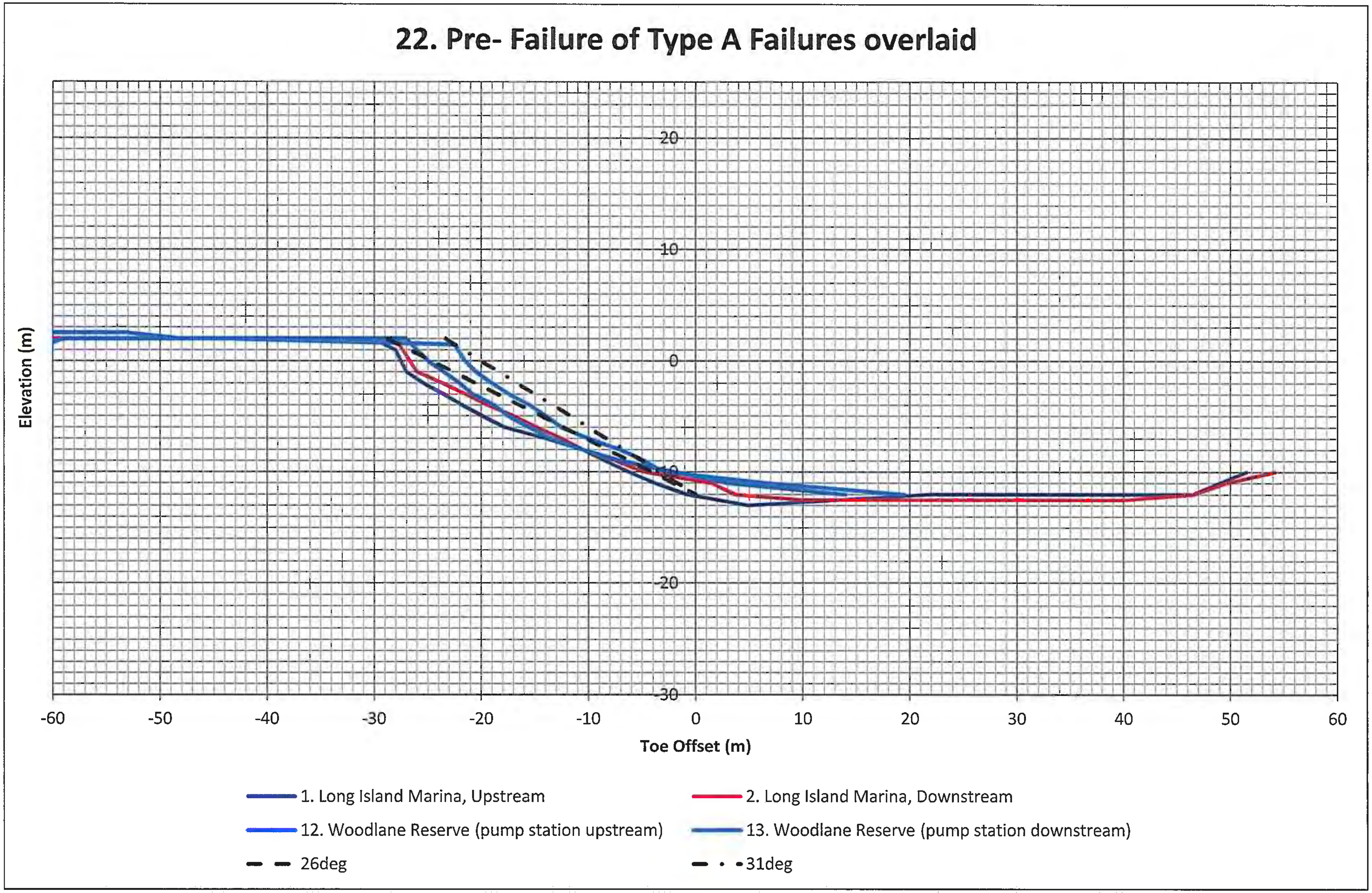
18. East Front Road, Downstream

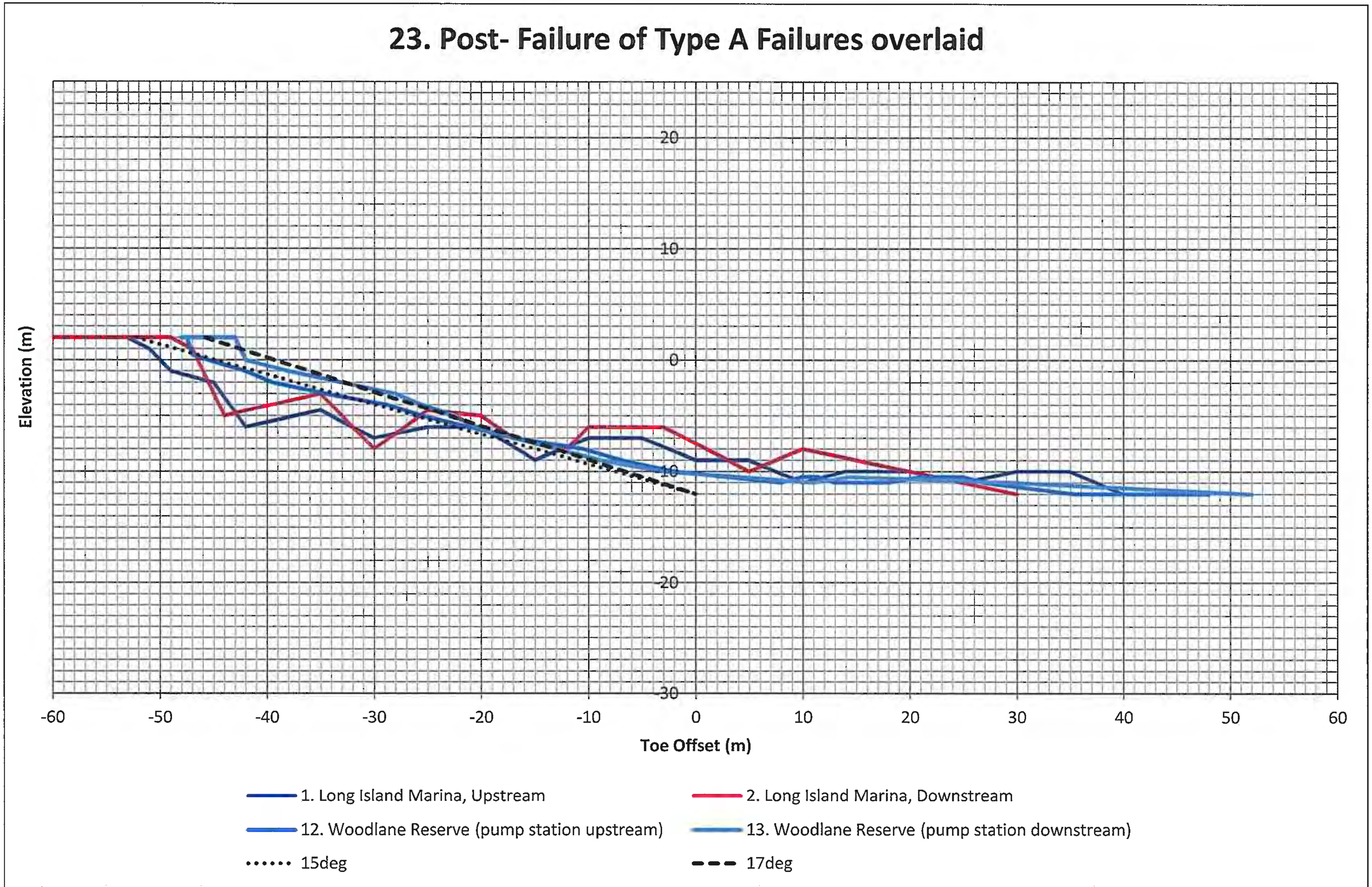


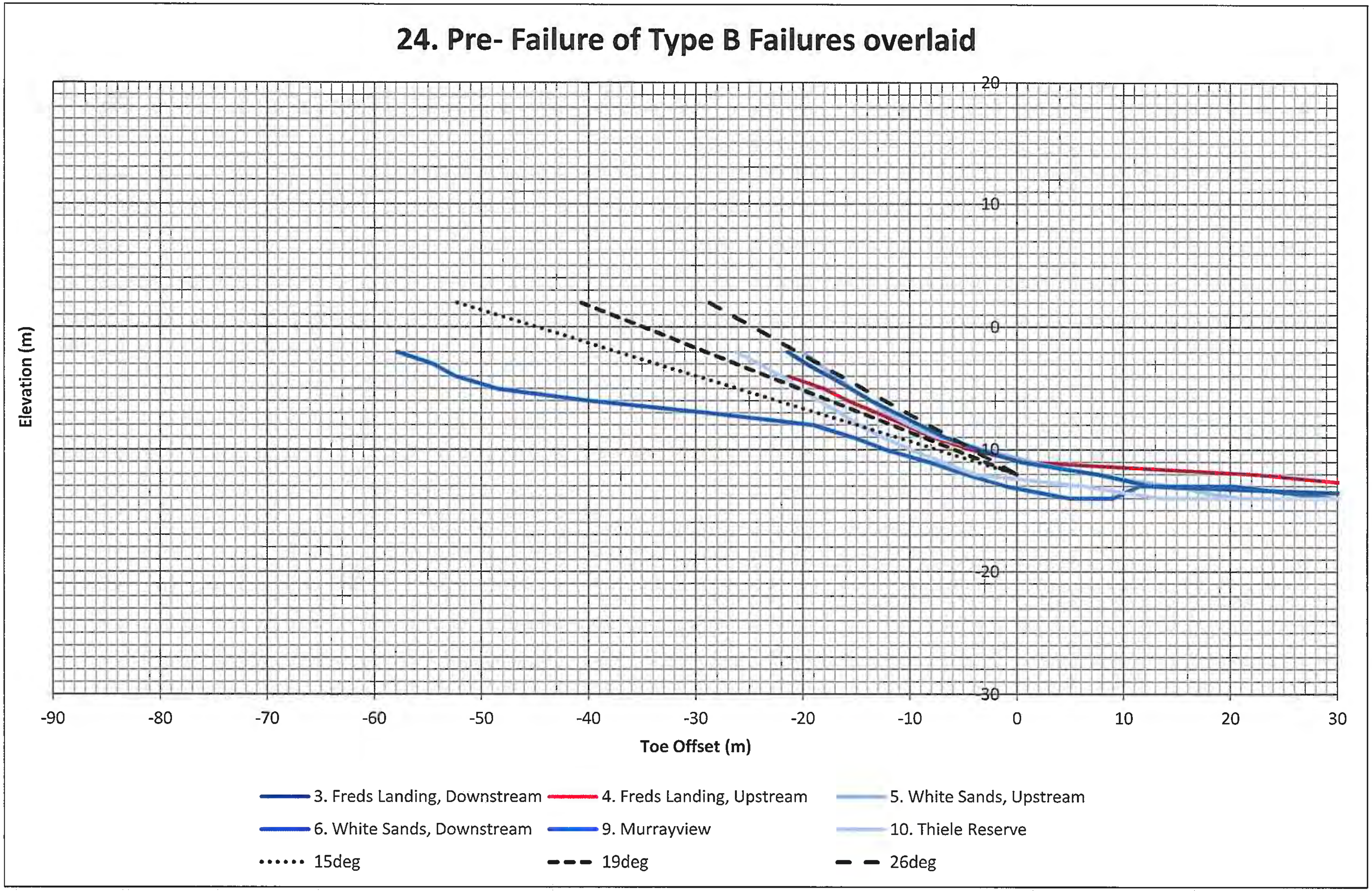

19. East Front Road, Middle
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Chainage (m)
Land, Pre Failure Bathmerty


20. East Front Road, Upstream
Elevation (m)
30
20
10
0
-10
-20
-30
-40
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Chainage (m)
Land, Pre Failure Bathmerty
Base of Soft Clay


21. East Front Road, Lagoon
30
20
10
0
-10
-20
-30
-40
Elevation (m)
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Chainage (m)
Land, Pre Failure Bathmerty

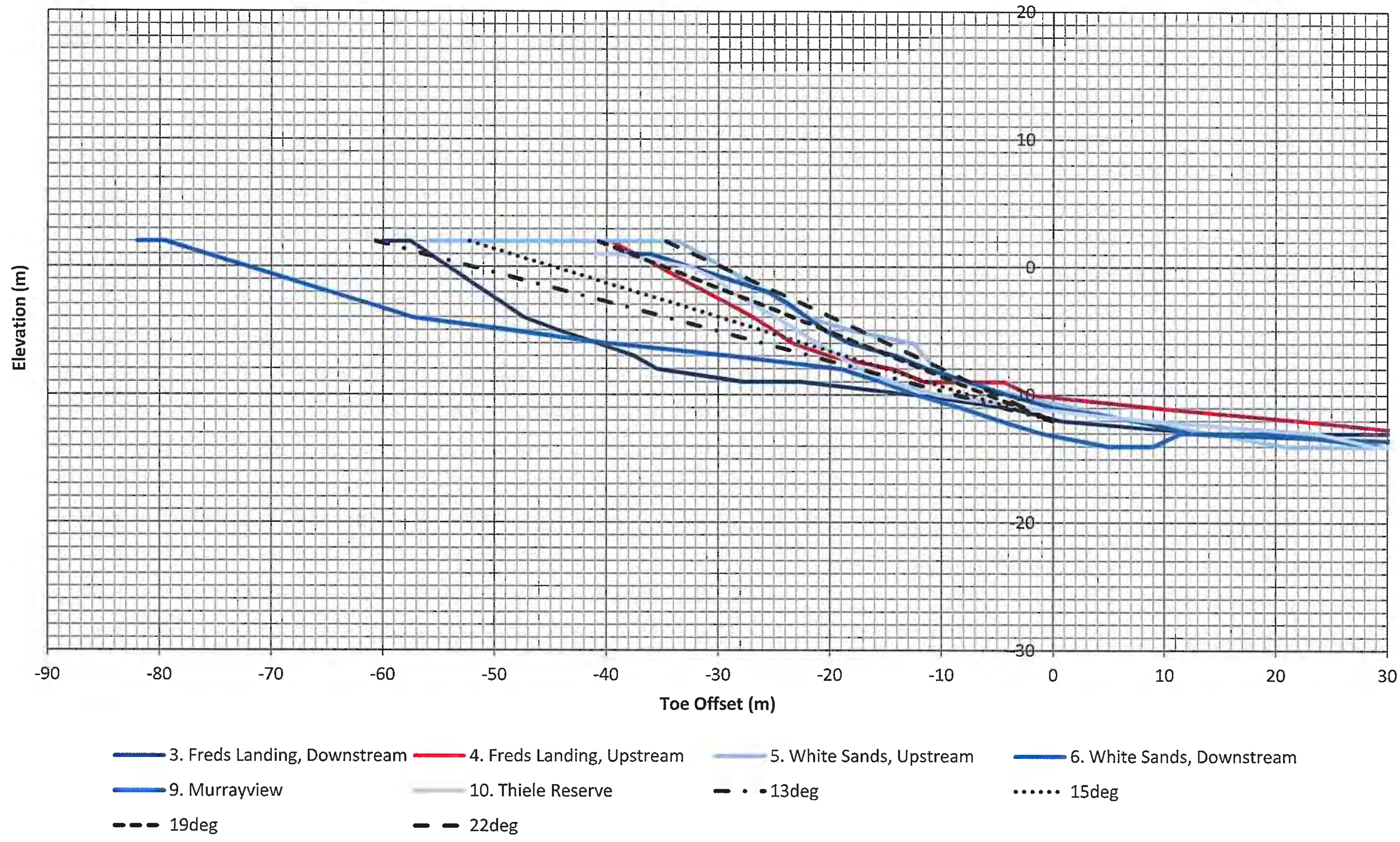

22. Pre- Failure of Type A Failures overlaid
Elevation (m)
20
10
0
-10
-20
-30
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Toe Offset (m)
1. Long Island Marina, Upstream
2. Long Island Marina, Downstream
12. Woodlane Reserve (pump station upstream)
13. Woodlane Reserve (pump station downstream)
26deg
31deg


23. Post- Failure of Type A Failures overlaid
Elevation (m)
20
10
0
-10
-20
-30
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Toe Offset (m)
1. Long Island Marina, Upstream
2. Long Island Marina, Downstream
12. Woodlane Reserve (pump station upstream)
13. Woodlane Reserve (pump station downstream)
15deg
17deg



24. Pre- Failure of Type B Failures overlaid
Elevation (m)
20
10
0
-10
-20
-30
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Toe Offset (m)
3. Freds Landing, Downstream
4. Freds Landing, Upstream
5. White Sands, Upstream
6. White Sands, Downstream
9. Murrayview
10. Thiele Reserve
15deg
19deg
26deg

25. Post- Failure of Type B Failures overlaid



Appendix C

Back analyses of large very rapid failures

APPENDIX C

BACK-ANALYSIS OF LARGE RAPID FAILURES

C1 Previous work and approach

As discussed in the text (Section 3) it is clear that large (particularly Type A) riverbank failures have the greatest potential to cause loss of life or damage. It is therefore important to understand what conditions lead to a rapid failure with a large river bank regression, and if riverbank regression can occur, the likely extent of regression that can occur in one event.

From the numerous reports and assessments it has been established that the conditions leading to these types of failures require the presence of deep deposits of Soft Clay in combination with an increase in load (stress) due to lowering of river levels. Previous authors have already made assessments and back analysis of the river bank stability under these conditions, most notably References 4 and 5 (Appendix A).

As part of the current study, additional bathymetry, survey and site mapping information of the extent of two of the most dramatic Type A riverbank failures (Long Island Marina and Woodlane Pumpstation) has been collected. In Appendix C we have reassessed the failures at these two sites (by back-analysis) in order to better understand the conditions which lead to instability, how the slopes fail and why large riverbank regressions can occur. We have summarised the results of the reassessment at the end of the appendix (Section C5).

The 2-D limit equilibrium stability analyses used in Appendices C and D have been carried out using the commercial program SLIDE and the Morgenstern and Price method.

C2 Failure geometry

The Long Island Marina slope pre-failure profiles (Nos. 1 and 2 in Appendix B) are from the late February 2009 bathymetry. The upstream profile (No. 1) is located about 10 m upstream of the large February 2009 failures through a section of riverbank that subsequently failed. The downstream pre-failure profile (No. 2) is located about 10 m downstream of the large February 2009 failures. The post failure profiles shown are based on the approximate shape of the nearby slope failure. They are indicative only, but do show how the failed mass broke up and spread into the river.

The location of the two Woodlane Pumpstation slope profiles (Nos. 11 and 12 in Appendix B) are shown on Figure 7. Profiles 11 and 12 are slightly simplified versions of Sections W2 and W3 given on Figure 7.

The following is a brief summary of the key features and observations of the pre- and post-failure geometry of these two sites:

- The offset distances on the four profiles from Long Island Marina and Woodlane Pumpstation have been set so that zero corresponds approximately to the toe of the original batter slope. This is somewhat subjectively adopted as the intersection between the slope of the main part of the bank and the slope of the main part of the bed.
- The pre-failure and post-failure geometry of the four profiles have been overlaid on single plots as Profile 22 and Profile 23 respectively in Appendix B (profiles used for slope comparisons).

- At all four profiles the river bed level was approximately RL -12 m and the top of bank level was approximately RL 2 m, with a near horizontal riverbank.
- At three of the four profiles the pre-failure average batter slope was approximately 26^0. The exception was the downstream section at Woodlane Pumpstation which was slightly steeper at 31^0.
- At both sites the initial failure was reported to have been rapid with at least 10 to 15 m of riverbank regression followed by further failures to result in the current slopes.
- At Woodlane Pumpstation, the back scarp was intentionally flattened to prevent further failures.
- At Long Island Marina, the initial failure probably occurred in seconds to a minute or two and immediately after the initial failure there was a back scarp extending to about 5 m below water level.
- The angle from the original toe of the slope to the top of the back scarp once the slope stabilised was approximately 15^0 at three of the four sections and slightly steeper at 17^0 at Woodside downstream where the initial batter was steeper.
- The average river pond level at the time of the failures was between approximately RL -0.6 m and RL -0.9 m.
- The post-failure bathymetry of both slides suggests the river bank slid into the river on a failure surface predominantly shallower than the base of the river and was deposited as debris on the river bed as opposed to a deep seated rotational failure extending significantly deeper than the river bed with heave occurring in the river bed beyond the toe.

C3 Soil stratigraphy and properties

At the locations where Coffey have borehole data, the top of the Soft Clay is consistently at around RL 0 m. This elevation is consistent with the geomorphology of the river as discussed in Section 2 and shown on Figure 1. The depth of the River Murray channel at the start of the Holocene Period would also indicate that the base of the Soft Clay probably extends to well below RL -30 m.

For the purposes of this back-analysis the average strength and density properties were used for the materials and then the sensitivity of the stability to the parameters was assessed by varying parameters individually.

The average undrained strength and density properties, measured at several sites with deep Soft Clay along the river, is:

Undrained shear strength at top of Soft Clay (c_{uo}) = 5.5 kPa

Rate of increase in shear strength with depth (dc_u/dz) = 1.25 kPa/m

Saturated unit weight (γ) = 16 kN/m^3

Previous reports have suggested that the likely range of undrained shear strength is within +/- 5 kPa of these average values. The existing measurements of undrained strength have all been taken from land based boreholes and so it has not been established whether the undrained strength is a function of reduced level or depth below surface. Given the geomorphology of the river, where the river channel would have meandered with time, Coffey consider that, on present knowledge, adopting an undrained strength as a function of reduced level with c_{uo} set at RL 0 m is a reasonable assumption.

It should be noted that for near-normally consolidated clays subjected to a lowering of the water-table the undrained shear strength should be the critical strength parameter, since with time consolidation of the clays with a lower water level would be expected to result in a higher drained effective strength.

By contrast, the Fill and desiccated crust above RL 0 m is overconsolidated and hence drained effective strength parameters would be expected to result in lower shear strength than undrained shear strength properties. However it should be noted that in materials with high undrained shear strength a tension crack can develop to a considerable depth. In the rapid failures a near-vertical back scarp several metres deep was observed post failure which is consistent with development of a tension crack under undrained conditions.

The average parameters adopted in previous reports for the Fill and desiccated crust were:

Undrained shear strength (c_u) = 50 kPa

Effective cohesion (c') = 2 kPa

Effective angle of friction (ϕ') = 28°

Moist unit weight (γ_m) = 18 kN/m^3

C4 Sensitivity analyses

The results of sensitivity analyses to investigate the effects of various parameters on the stability of the slopes are summarised in Table C1. Plots (Figures C1 to C14) showing the outputs of the stability analyses are given at the end of this appendix.

A brief summary of the results of the sensitivity analyses is given below:

- The base case has a factor of safety (FOS) of well below 1 using typical strength parameters, low river level and fully drained conditions in the fill or desiccated crust.
- Non circular failures surfaces do not significantly alter the stability assessments.
- A tension crack in the crust/fill gives almost the same FOS as a drained crust.
- If undrained parameters without a tension crack are used the FOS is about 1.1. The development of a tension crack (relatively quick loss of strength) would give a good explanation for the onset of a sudden failure and large travel distances (which is consistent with the observations).
- The river level has a big effect on stability due to the effect of its restoring force on the riverbank slope. Lowering the water level from RL 0.5 m to -0.9 m reduces the FOS by about 0.2. (i.e. about 0.15 per metre drop).
- Surcharge on the crest has a significant but smaller effect than an equivalent water drop. A 10 kPa surcharge is equivalent to a 1 m water level drop in load terms but only reduces the FOS by 0.08 (i.e. about half the amount or reduction caused by a 1 m drop in river level).
- Changes in the undrained shear strength of the Soft Clay have a significant effect on stability. Each 1 kPa change in strength changes the FOS by about 0.07.

Table C1: Summary of slope stability parametric analysis

Stability run	Analysis type	Batter	Fill / crust properties	Soft Clay Cu0 (KPa)	Surcharge (KPa)	Water level (RLm)	FOS	Change in FOS from BASE CASE	Head scarp regression (M)	Toe of failure (RLm)
slope26_water-09	circular	26^0 (BASE CASE)	drained	5.5	0	-0.9	0.88		10.6	-6.2
slope26_water-09_nc	non-circular	26^0	drained	5.5	0	-0.9	0.87	-0.01	10.0	-5.9
slope26_water05	circular	26^0	drained	5.5	0	0.5	1.09	0.21	10.7	-6.1
slope26_water-09_sur	circular	26^0	drained	5.5	10	-0.9	0.80	-0.08	10.8	-6.1
slope26_water-09_co9	circular	26^0	drained	9	0	-0.9	1.11	0.23	12.2	-6.5
slope26_water-09_co2	circular	26^0	drained	2	0	-0.9	0.64	-0.24	8.7	-6.2
slope26_water-09_TC	circular	26^0	tension crack	5.5	0	-0.9	0.86	-0.02	8.7	-6.1
slope26_water-09_uf	circular	26^0	undrained cu=50 kPa	5.5	0	-0.9	1.08	0.20	18.5	-12
slope 29water-09	circular	29^0	drained	5.5	0	-0.9	0.82	-0.06	8.4	-5.0
slope 23water-09	circular	23^0	drained	5.5	0	-0.9	0.95	0.07	14.3	-6.7
slope20_water-09	circular	20^0	drained	5.5	0	-0.9	1.02	0.14	13.4	-7.5
slope17_water-09	circular	17^0	drained	5.5	0	-0.9	1.11	0.23	19.7	-8.2
failed_water-09	non-circular	initial failed slope	drained	5.5	0	-0.9	0.92	-	22	-
failed2_water-09	non-circular	secondary failed slope	drained	5.5	0	-0.9	0.61	-	28	-

- Flattening or steepening the average slope also affects stability but perhaps to a lesser extent than might be expected. A 3^0 change in slope changes the FOS by about 0.07. It is also important to note that the toe of most of the critical failure surfaces only extended to about RL - 6 m. Hence the slope in the upper part of the batter is the most important factor in the geometry unless the batter steepens in the lower part of the slope.

We did not assess the effect of seismicity in this review. Earthquakes have the potential to trigger riverbank collapse if they are already close to failure (e.g. when river levels are low).

C5 Staged progressive failure model

Table C1 also shows the head scarp regression associated with the critical failure surface. The first 12 analyses in Table C2 are for the initial failure only. For most of these analyses the head scarp regression is much less than the maximum regression observed at Long Island Marina and Woodlane Pumpstation.

For the large collapses what almost certainly happens is a progressive failure of the riverbank. The initial failure creates a back scarp steeper than the original slope. After the initial failure (in some cases apparently only seconds or minutes after) the new back scarp fails and pushes the debris associated with the initial failure across the river. In the larger failures this process might be repeated several times.

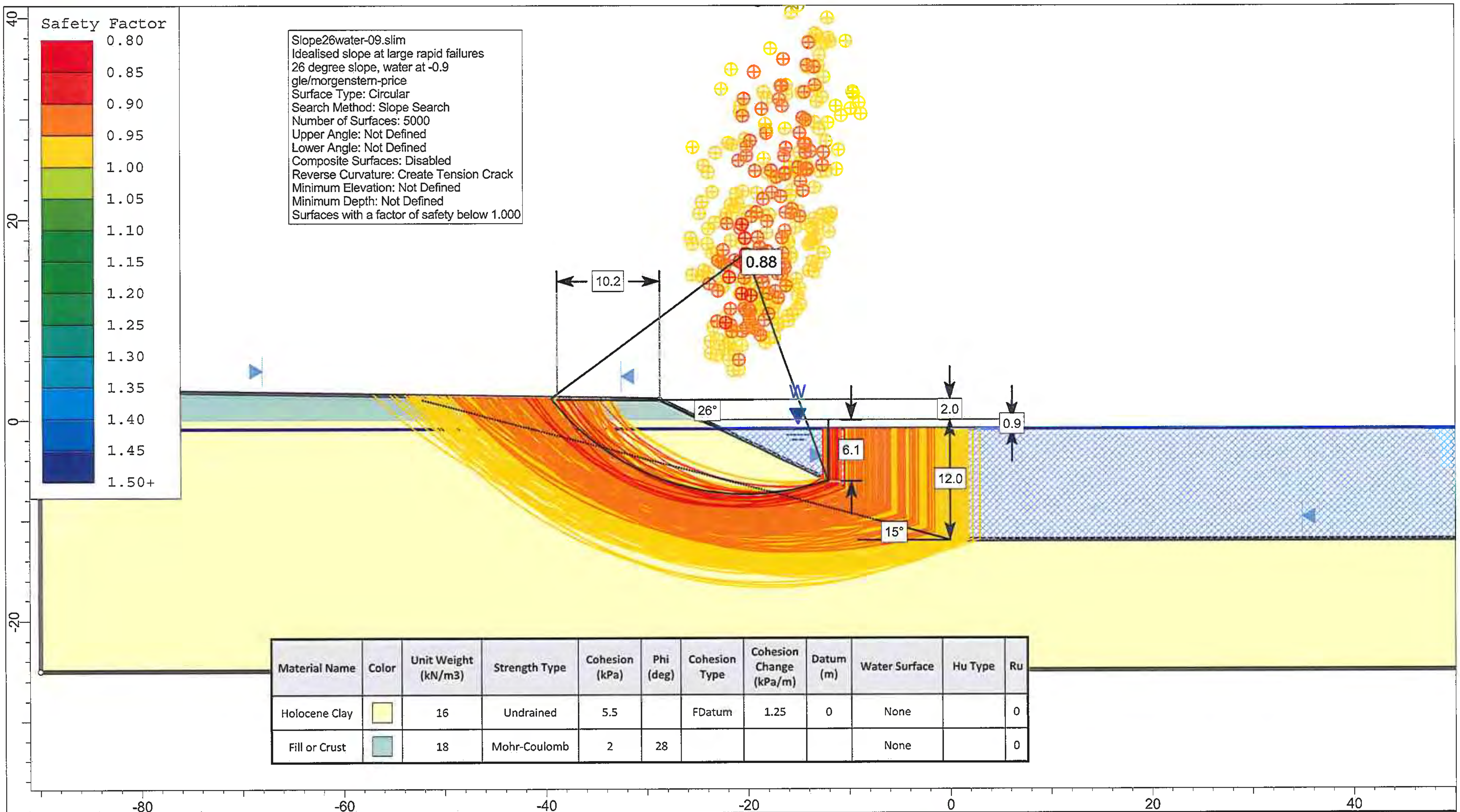
Figures C13 and C14 (Appendix C) show two stages in the progressive failure process.

C6 Summary

In summary the reassessment of the past riverbank collapse has shown that:

- The drop in river level causes a large reduction in stability and appears to be the major cause of the riverbank collapses.
- A Fill surcharge on the bank also reduces the stability but recent experience has shown that riverbank collapse can occur (and has occurred) where there is no Fill.
- The collapses that cause large regressions (e.g. Type A failures where more than 15 m of bank are lost) are probably the result of progressive failure (i.e. a rapid succession of collapses).
- Small variations in strength of the Soft Clay have a big effect on stability. However, it is very difficult to predict where such strength variations might occur and how extensive they might be without very extensive subsurface investigations.

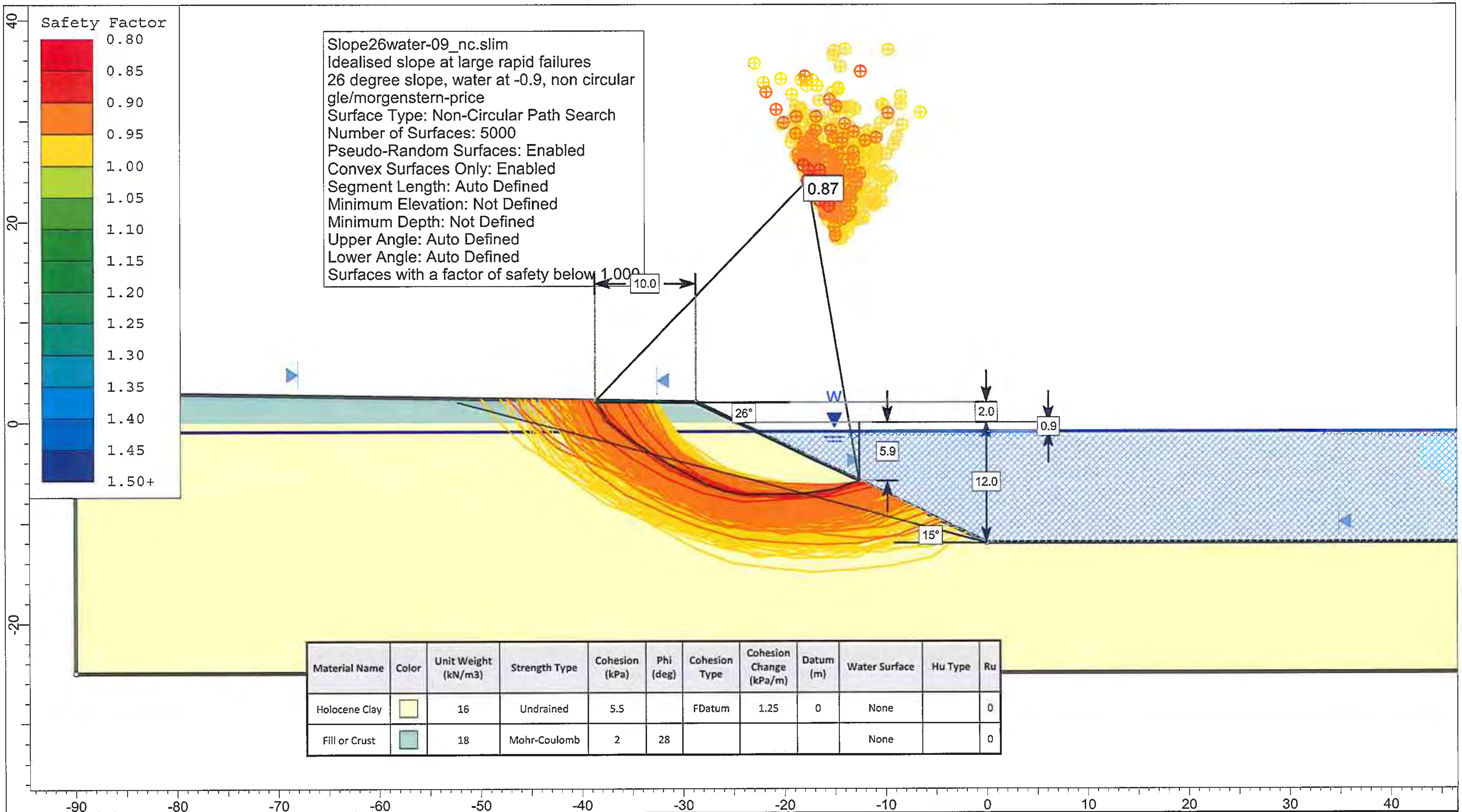
In our opinion, on present knowledge, it should be assumed that during periods of low river level riverbank collapse (Type A or B failures) could occur wherever the bank is underlain by Soft Clay. Sites which have already been affected by Type C failures (i.e. already have experienced significant cracking) may be particularly vulnerable to collapse. Elsewhere, where there has only been relatively minor (Type D) cracking (or no reported cracking), collapse may be more likely where there is Fill but can also occur where there is little or no Fill.



Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12/12/2012	Project:	Idealised slope at large rapid failures
Date	12/09/2012,	Title:	Idealised slope at large rapid failures 26 degree slope, water at -0.9
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	C1

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope26water-09.slim



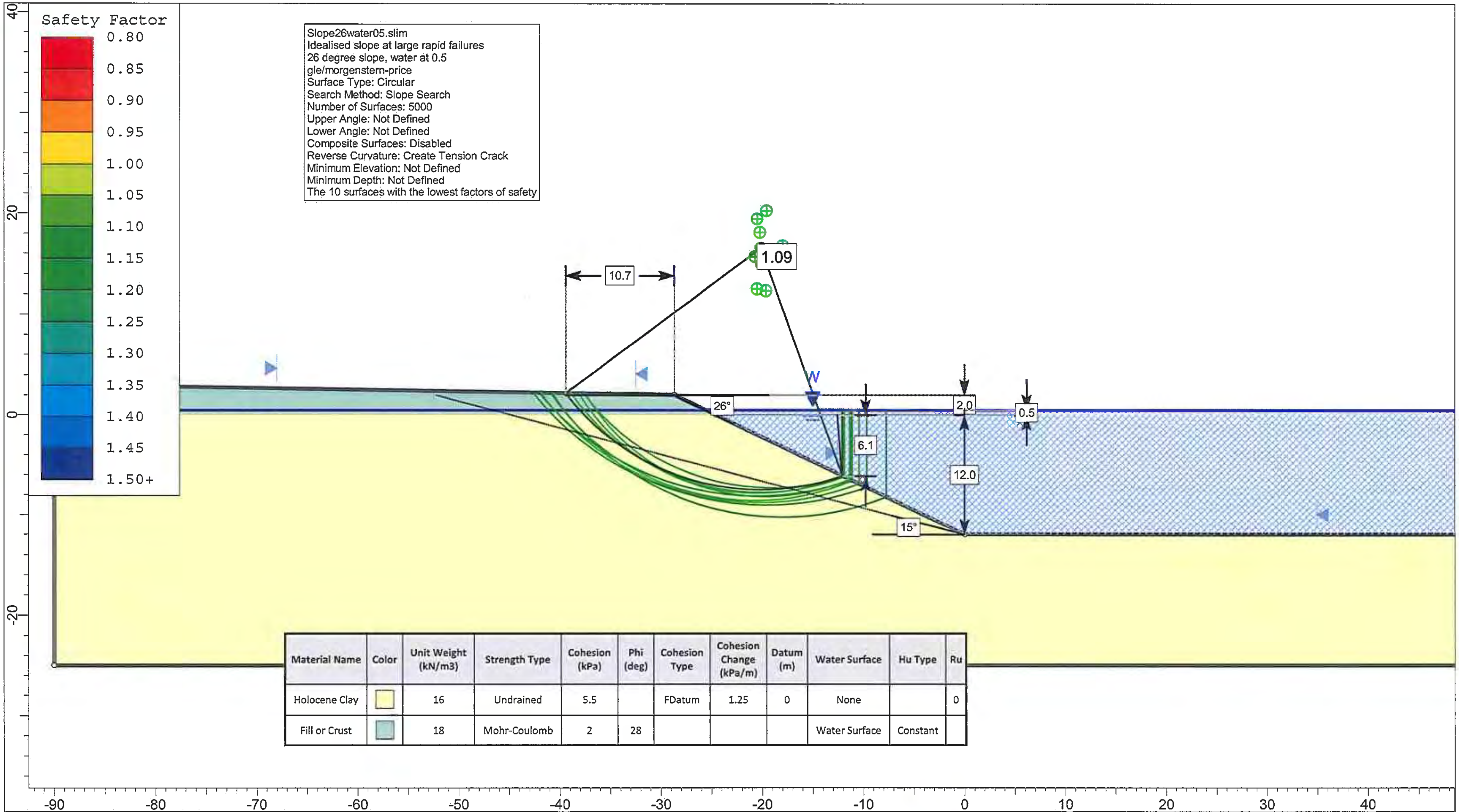
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				None		0

Author	RAD
Approved	ATM 12/12/2012
Date	12/09/2012, (
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Client:	Coffey Geotechnics Pty Ltd
Project:	Idealised slope at large rapid failures
Title:	Idealised slope at large rapid failures 26 degree slope, water at -0.9, non circular
doc no: GEOTMEND07093AA-AB	fig no: C2

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope26water-09_nc.slim



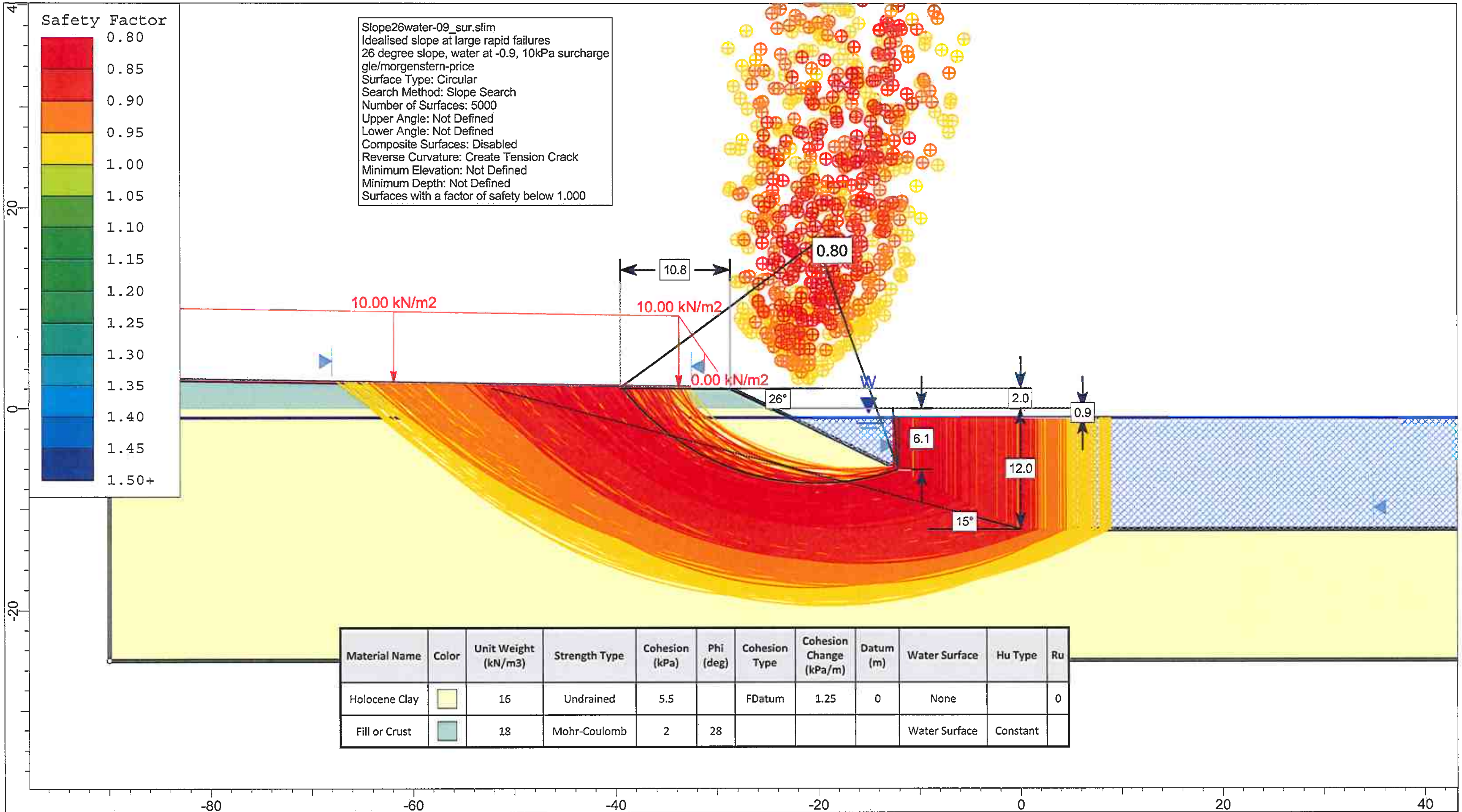
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	

Author	RAD
Approved	ATM 12/12/2012
Date	12/09/2012,
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Client:	Coffey Geotechnics Pty Ltd
Project:	Idealised slope at large rapid failures
Title:	Idealised slope at large rapid failures 26 degree slope, water at 0.5
doc no:	GEOTMEND07093AA-AB
fig no:	C3

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope26water05.slim

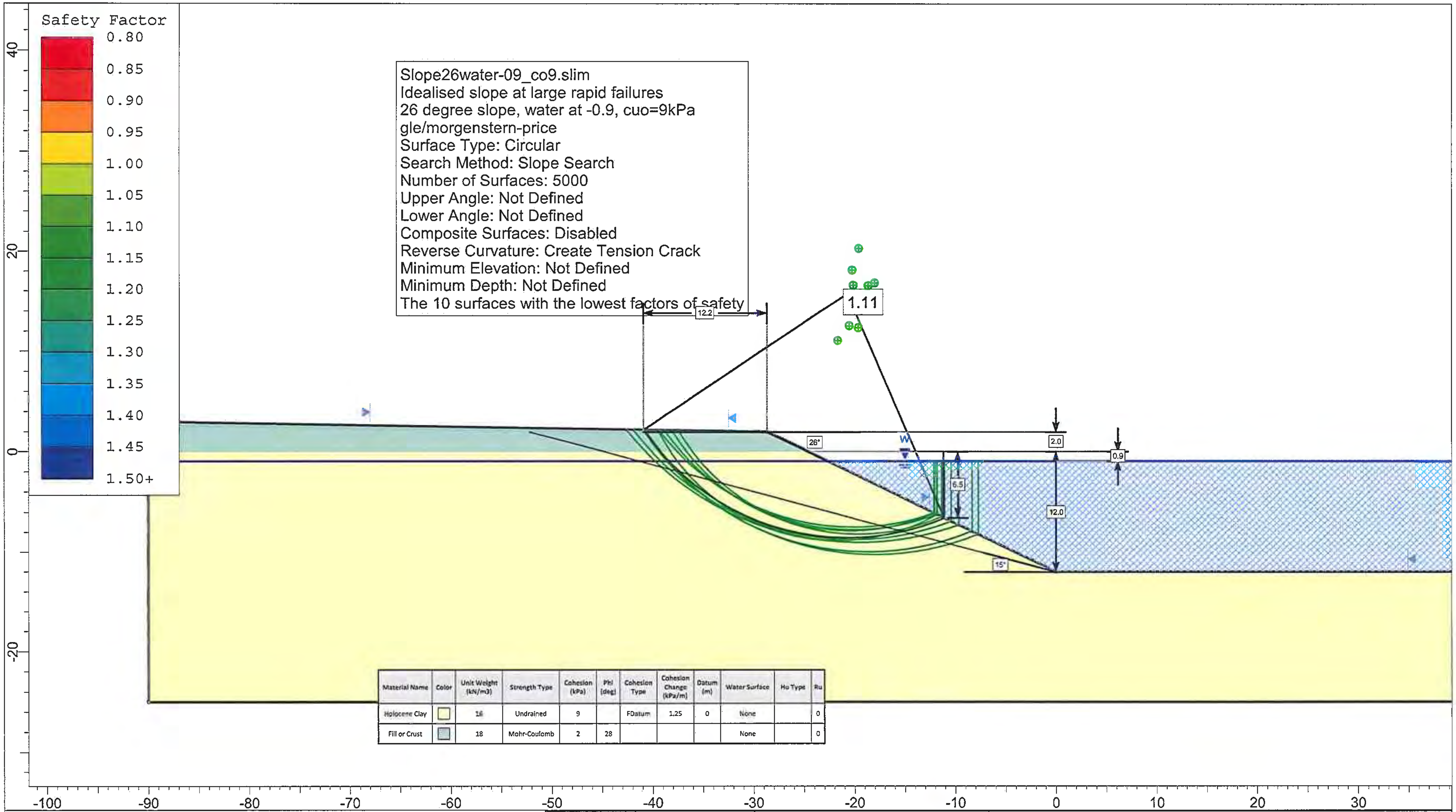


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	

Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12/12/2012	Project:	Idealised slope at large rapid failures
Date	12/09/2012, 9	Title:	Idealised slope at large rapid failures 26 degree slope, water at -0.9, 10kPa surcharge
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	C4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope26water-09_sur.slim

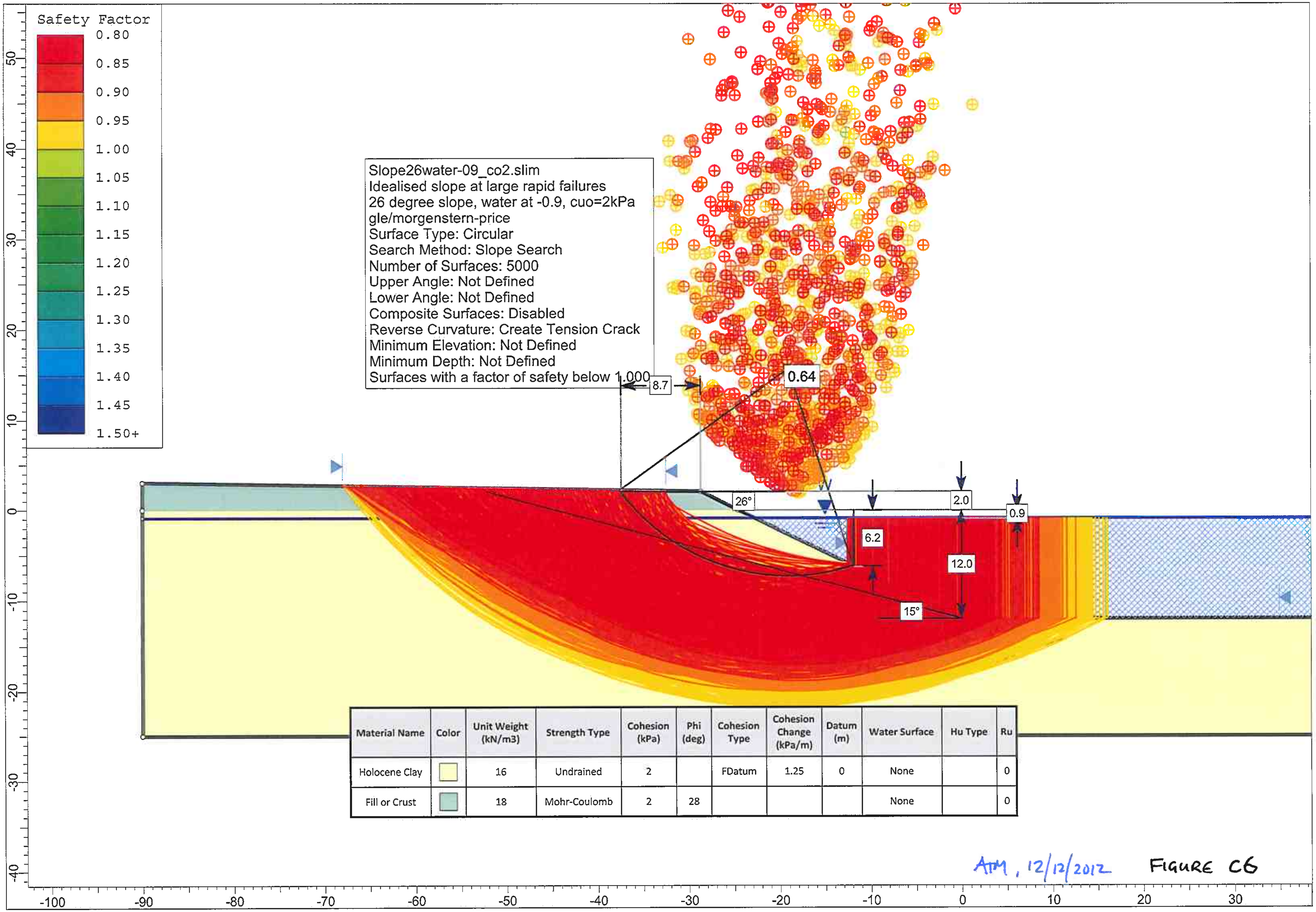


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	9		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				None		0

Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12/12/2012	Project:	Idealised slope at large rapid failures
Date	12/09/2012, 9	Title:	Idealised slope at large rapid failures 26 degree slope, water at -0.9, cuo=9kPa
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	C5

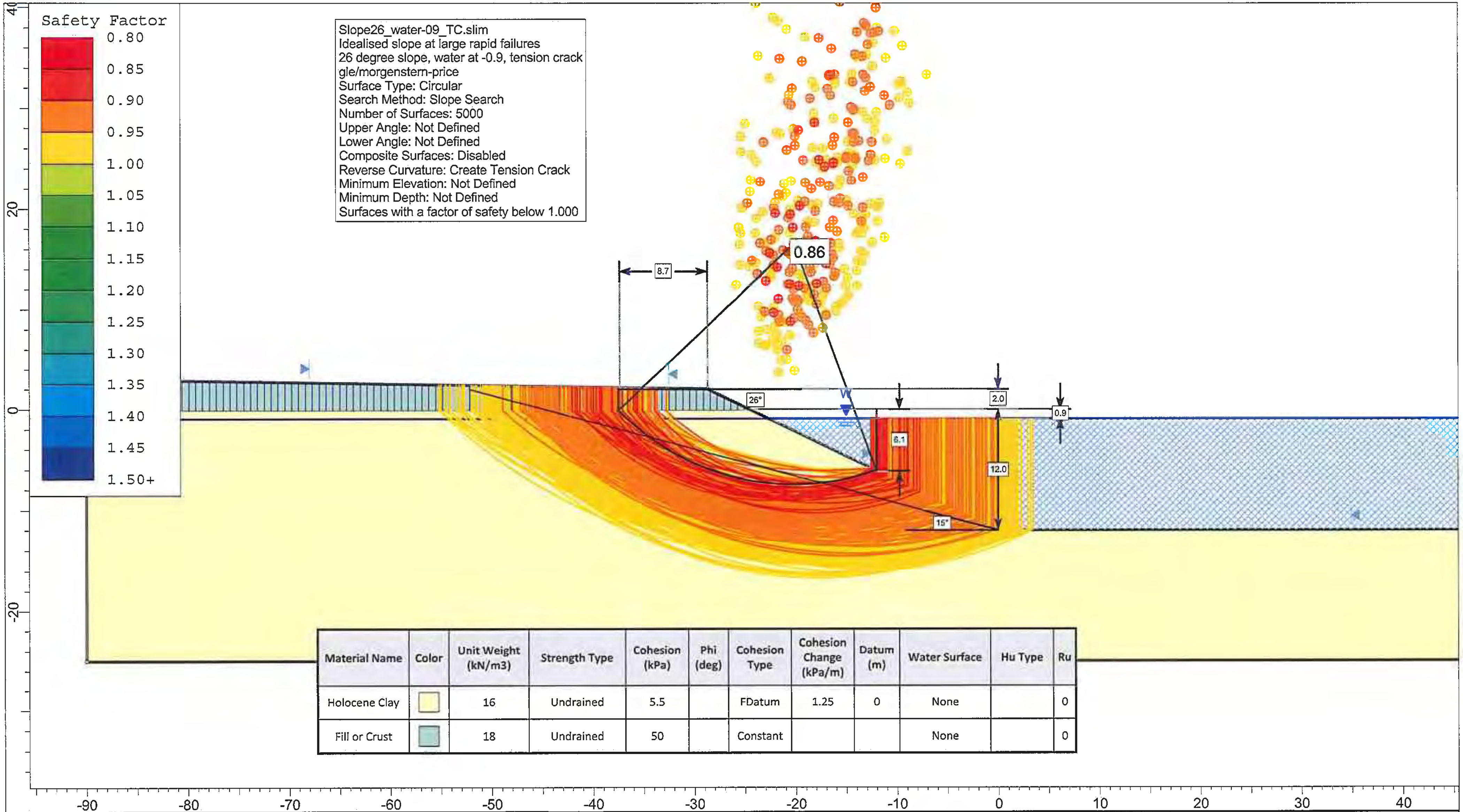
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope26water-09_co9.slim



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	2		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				None		0

ATM , 12/12/2012 FIGURE C6

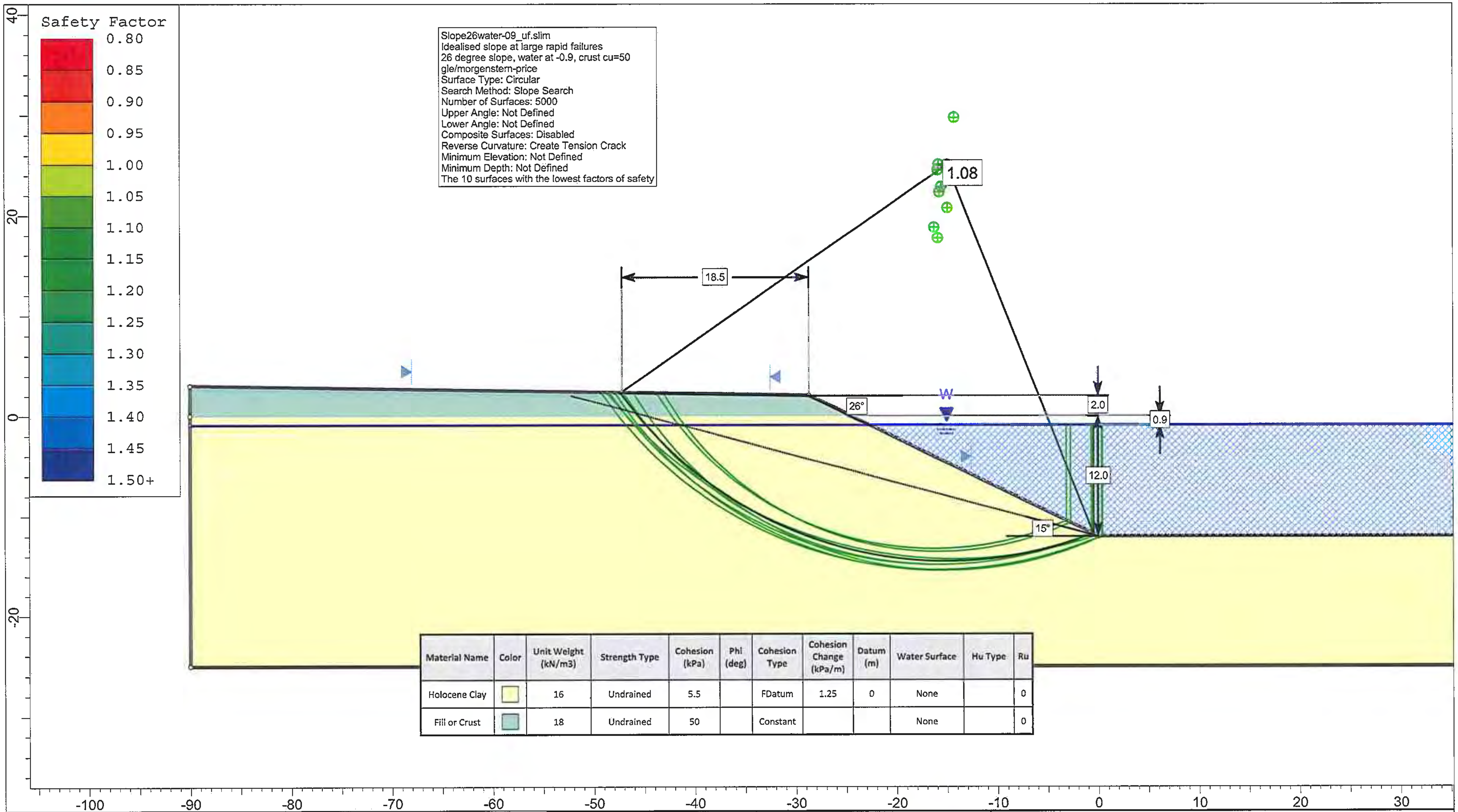


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Undrained	50		Constant			None		0

Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12/12/2012	Project:	Idealised slope at large rapid failures
Date	12/09/2012,	Title:	Idealised slope at large rapid failures 26 degree slope, water at -0.9, tension crack
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	C7

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

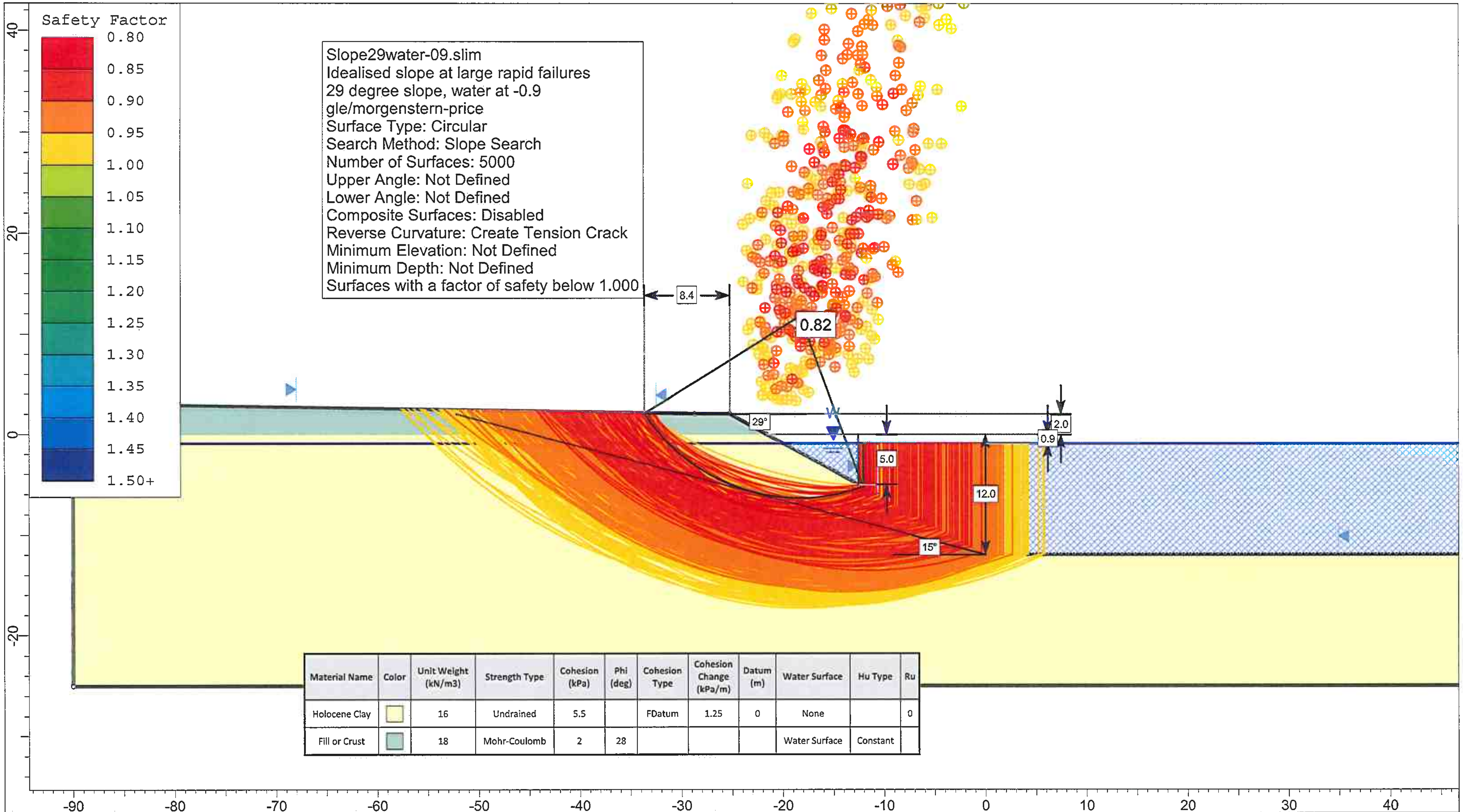
F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope26_water-09_TC.slim



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Undrained	50		Constant			None		0

Author	RAD	coffey geotechnics SPECIALISTS MANAGING THE EARTH	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12/12/2012		Project:	Idealised slope at large rapid failures
Date	12/09/2012,		Title:	Idealised slope at large rapid failures 26 degree slope, water at -0.9, crust cu=50
Scale	1:500		doc no:	GEOTMEND07093AA-AB
Orig Size	A4		fig no:	C8

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope26water-09_uf.slim

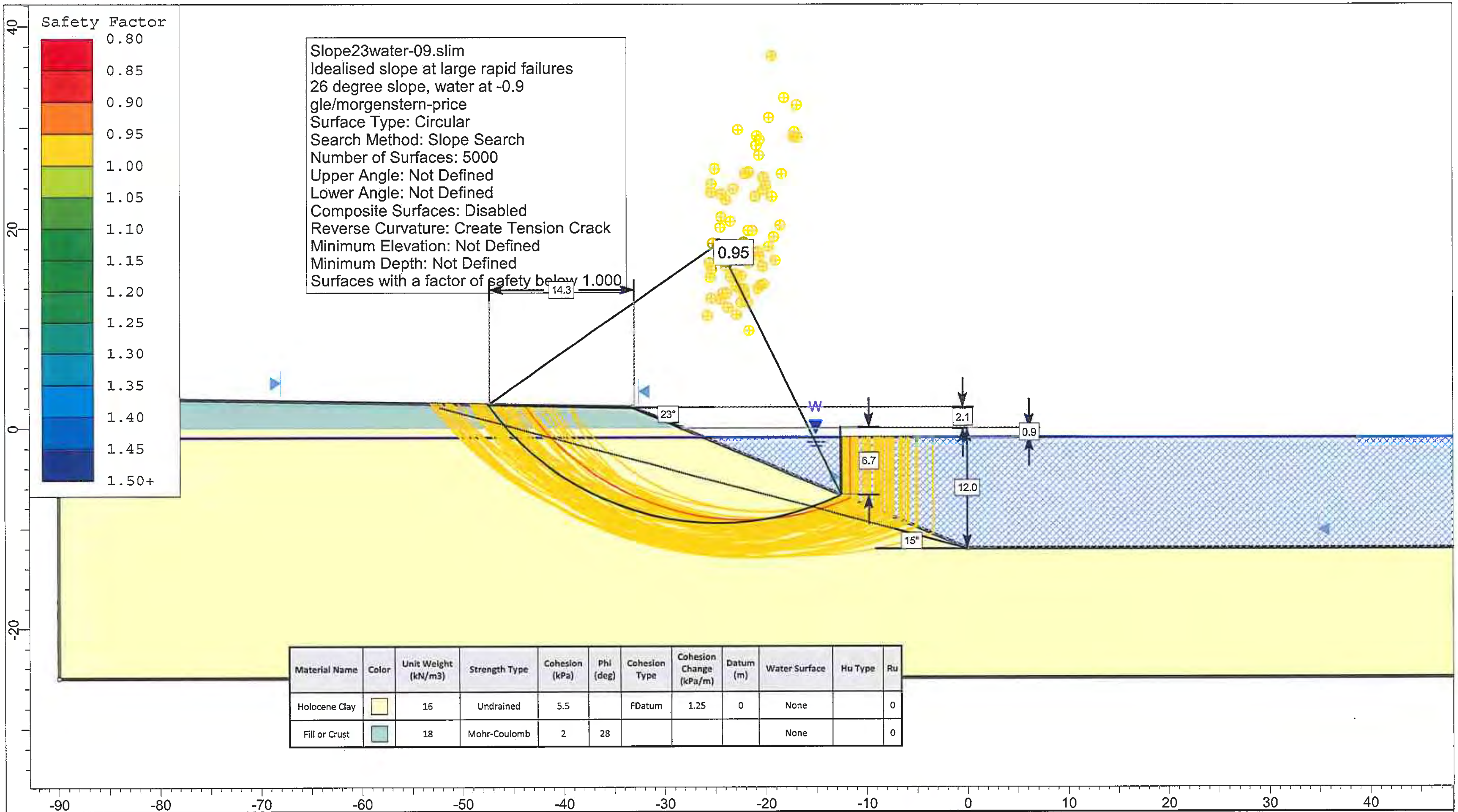


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	

Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12/12/2012	Project:	Idealised slope at large rapid failures
Date	12/09/2012, 9	Title:	Idealised slope at large rapid failures 29 degree slope, water at -0.9
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	C9

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope29water-09.slim



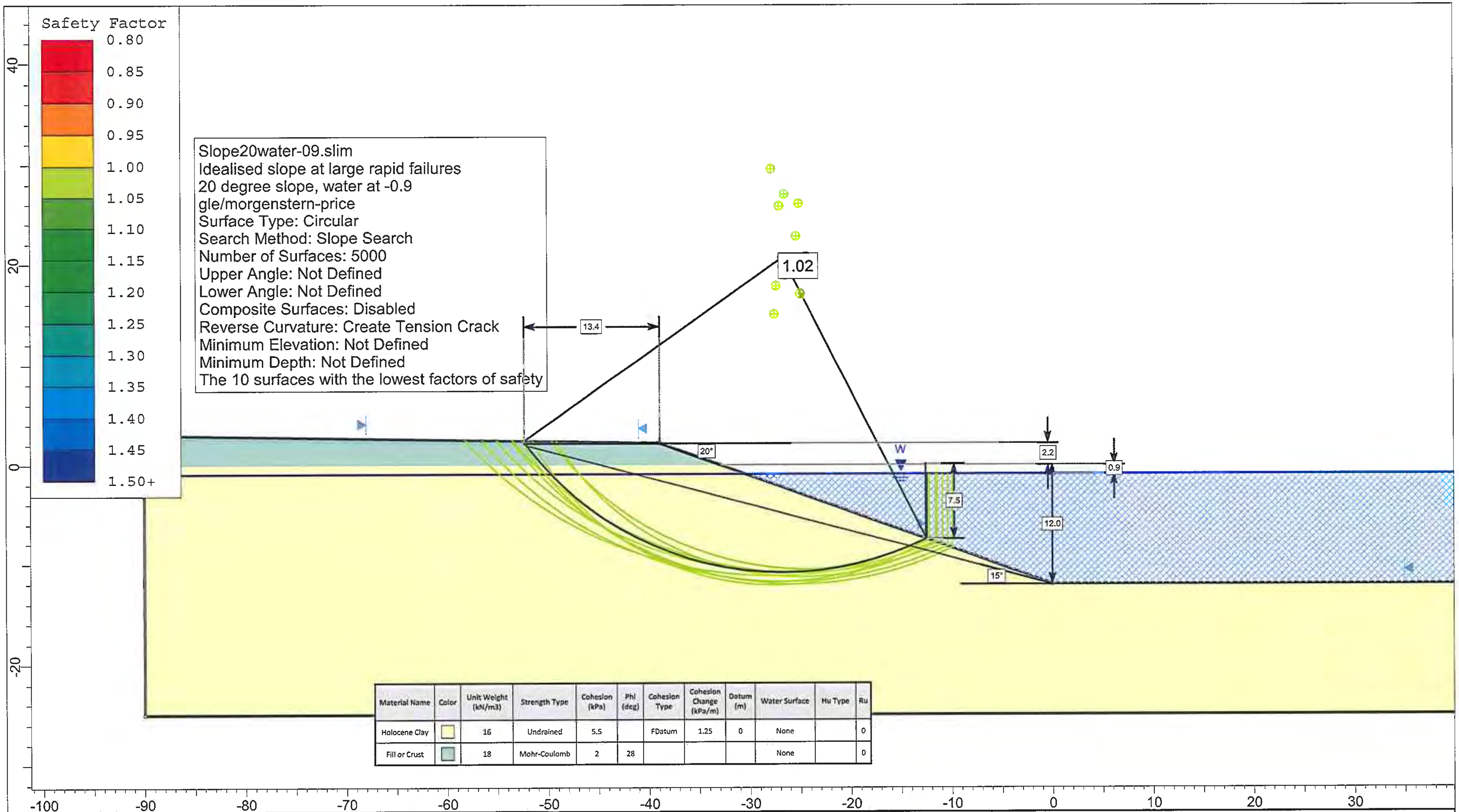
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				None		0

Author	RAD
Approved	ATM 12/12/2012
Date	12/09/2012,
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Client:	Coffey Geotechnics Pty Ltd
Project:	Idealised slope at large rapid failures
Title:	Idealised slope at large rapid failures 26 degree slope, water at -0.9
doc no:	GEOTMEND07093AA-AB
fig no:	C 10

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope23water-09.slim



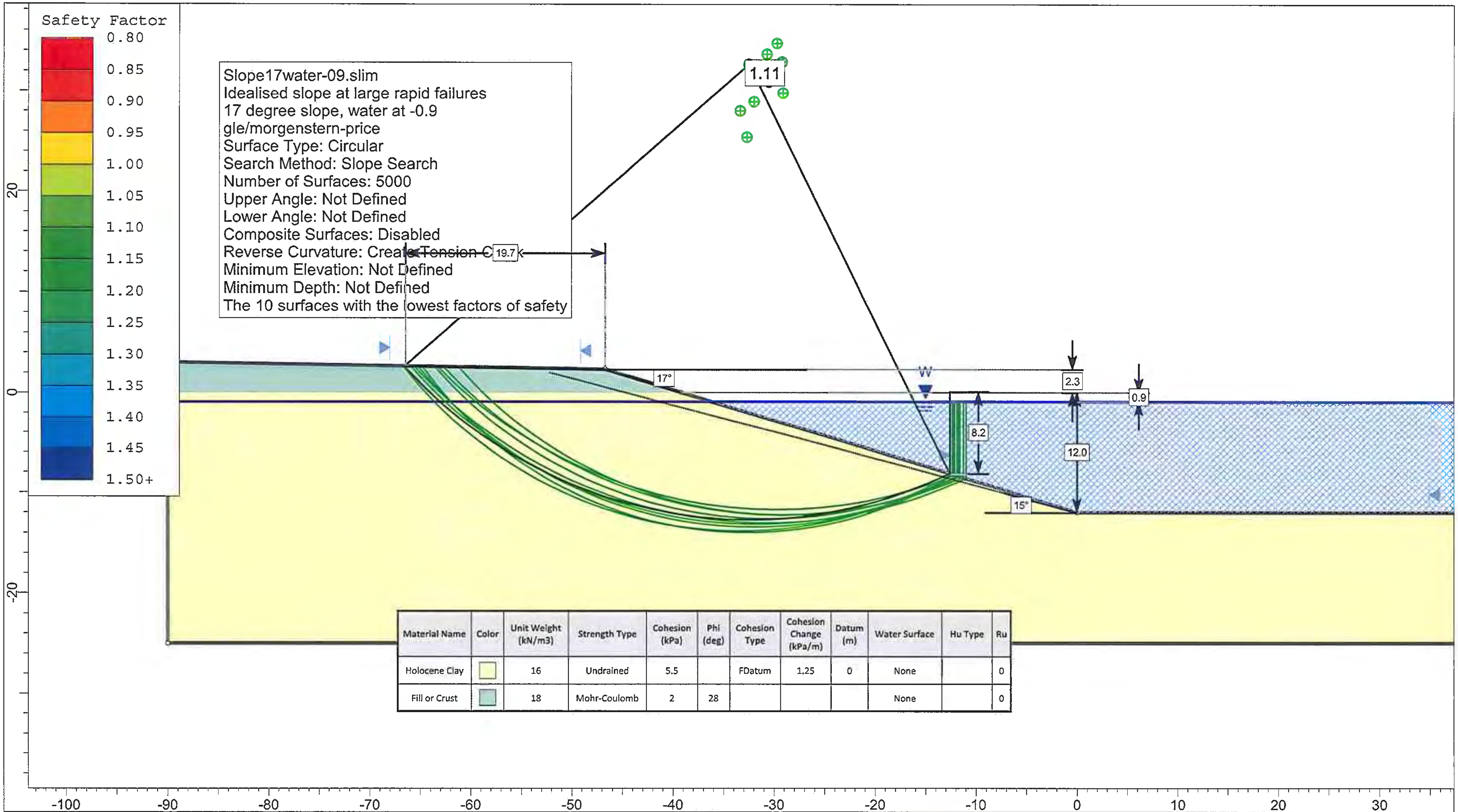
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				None		0

Author	RAD
Approved	ATM 12/12/2012
Date	12/09/2012, ¢
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Client:	Coffey Geotechnics Pty Ltd
Project:	Idealised slope at large rapid failures
Title:	Idealised slope at large rapid failures 20 degree slope, water at -0.9
doc no:	GEOTMEND07093AA-AB
fig no:	C 11

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope20water-09.slim



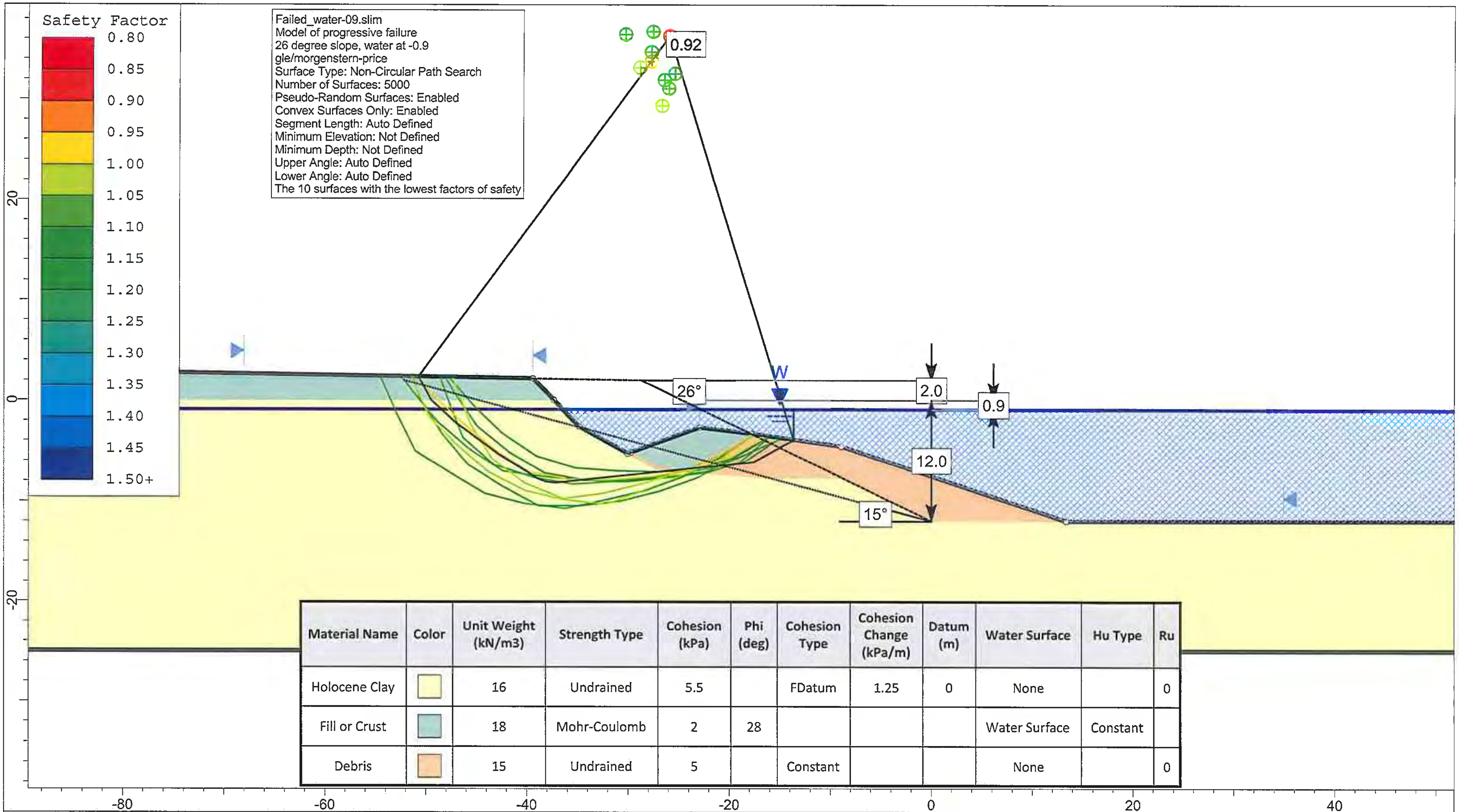
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				None		0

Author	RAD
Approved	ATM 12/12/2012
Date	12/09/2012, !
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Client:	Coffey Geotechnics Pty Ltd
Project:	Idealised slope at large rapid failures
Title:	Idealised slope at large rapid failures 17 degree slope, water at -0.9
doc no:	GEOTMEND07093AA-AB
fig no:	C12

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Slope17water-09.slim

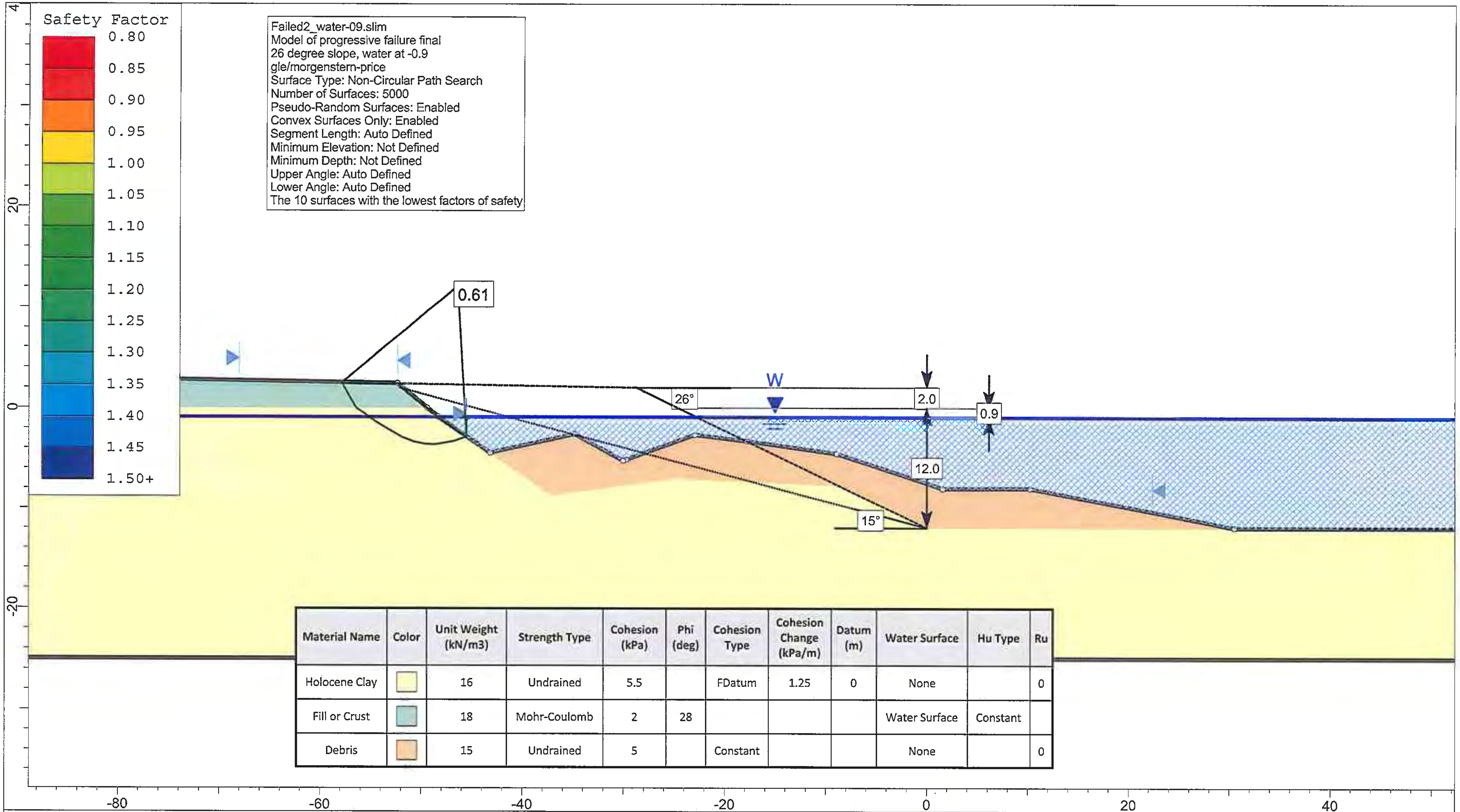


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Debris		15	Undrained	5		Constant			None		0

Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12/12/2012	Project:	Model of progressive failure
Date	12/09/2012,	Title:	Model of progressive failure 26 degree slope, water at -0.9
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	C13

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Failed_water-09.slim



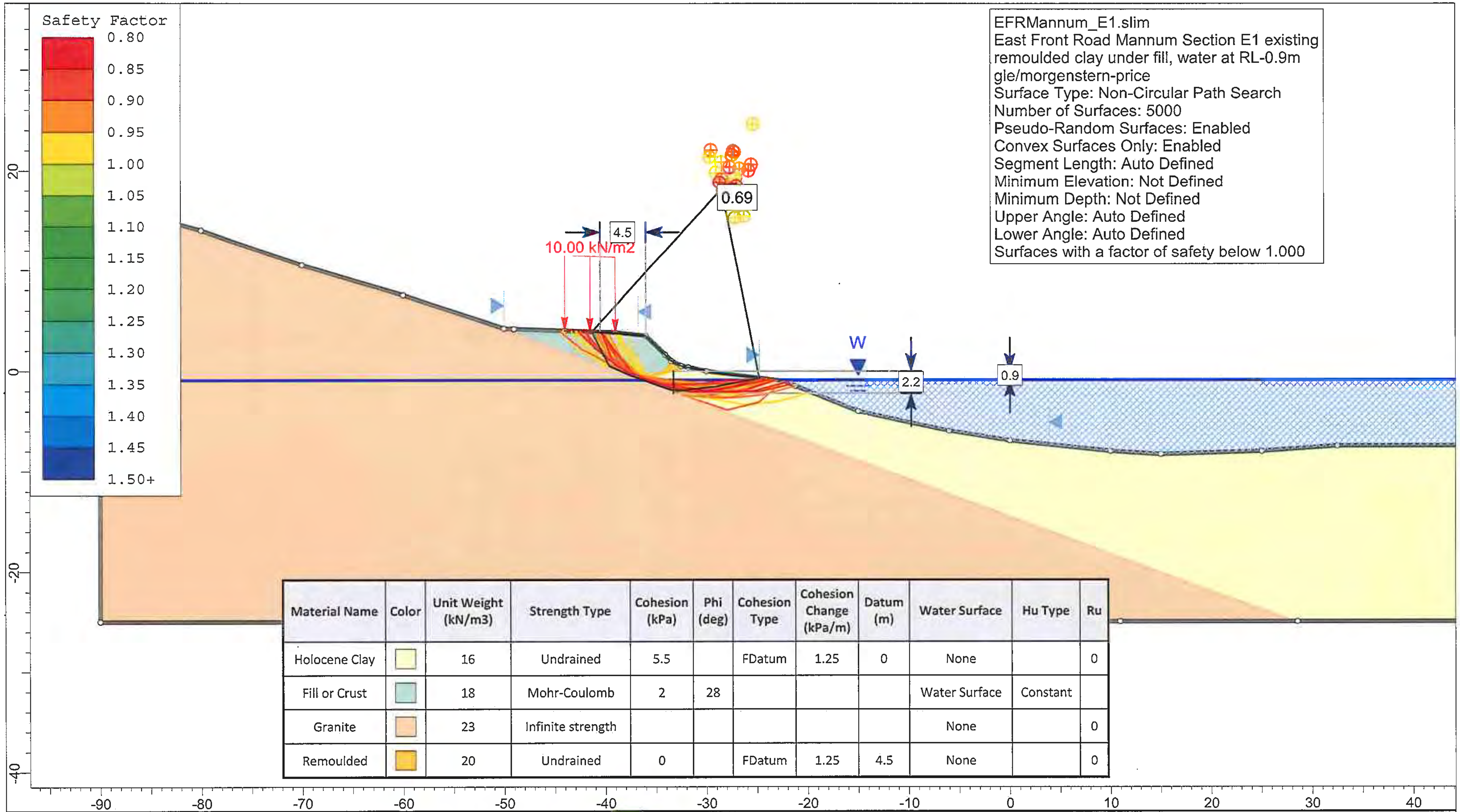
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Debris		15	Undrained	5		Constant			None		0

Author	RAD	coffey geotechnics SPECIALISTS MANAGING THE EARTH	Client:	Coffey Geotechnics Pty Ltd
Approved	AFM 12/12/2012		Project:	Model of progressive failure final
Date	12/09/2012, 9		Title:	Model of progressive failure final 26 degree slope, water at -0.9
Scale	1:500		doc no:	GEOTMEND07093AA-AB
Orig Size	A4		fig no:	C14

F:\GEOT\PROJ\OTHER\GEOT\GEOTMEND\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Failed2_water-09.slim

Appendix D

Stability analyses for the four sites assessed



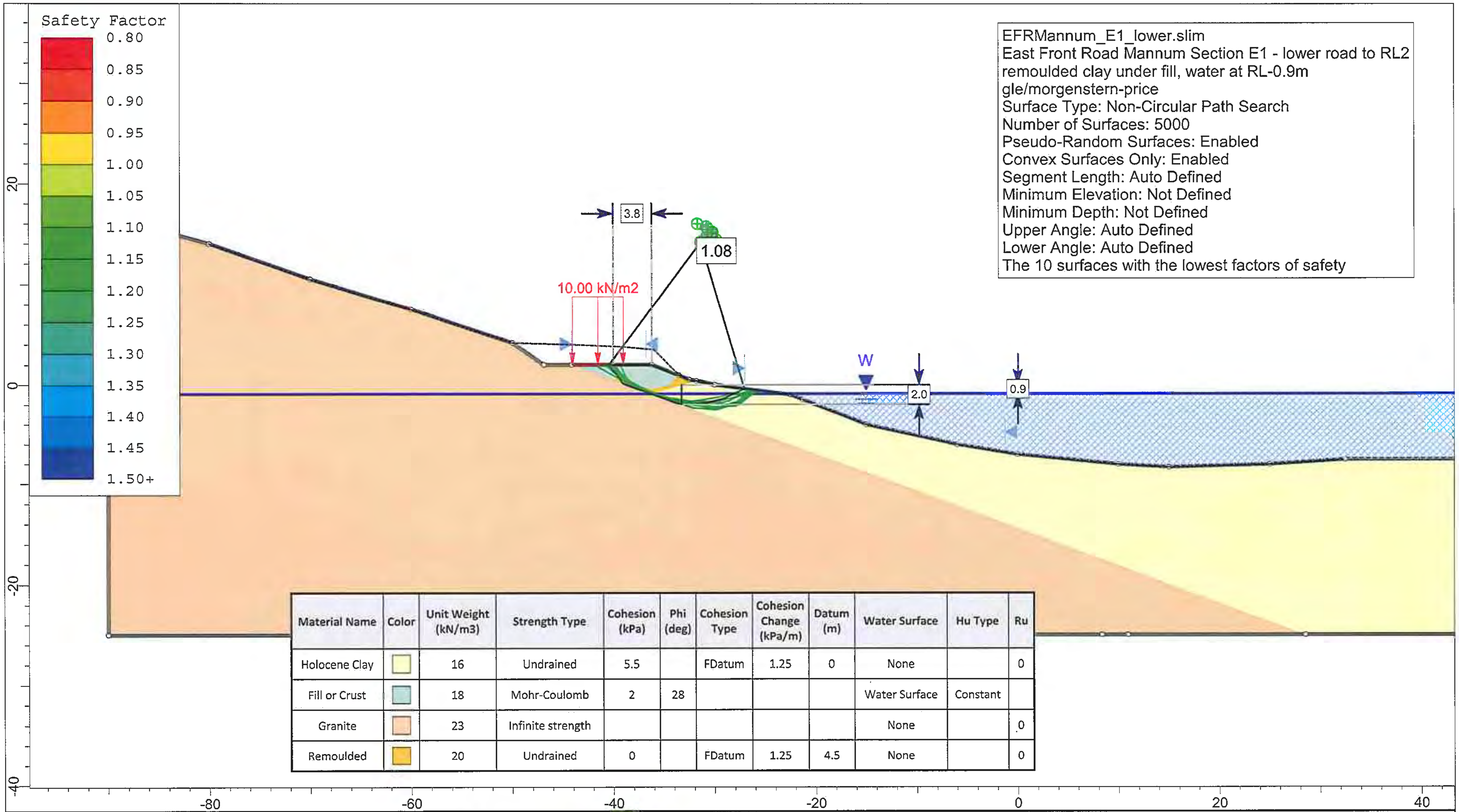
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Granite		23	Infinite strength						None		0
Remoulded		20	Undrained	0		FDatum	1.25	4.5	None		0

Author	RAD
Approved	ATM 12 Dec 2012
Date	23/9/2012
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING

Client:	Coffey Geotechnics Pty Ltd		
Project:	East Front Road Mannum Section E1 existing		
Title:	East Front Road Mannum Section E1 existing remoulded clay under fill, water at RL-0.9m		
doc no:	GEOTMEND07093AA-AB	fig no:	D1

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\EFRMannum_E1.slim



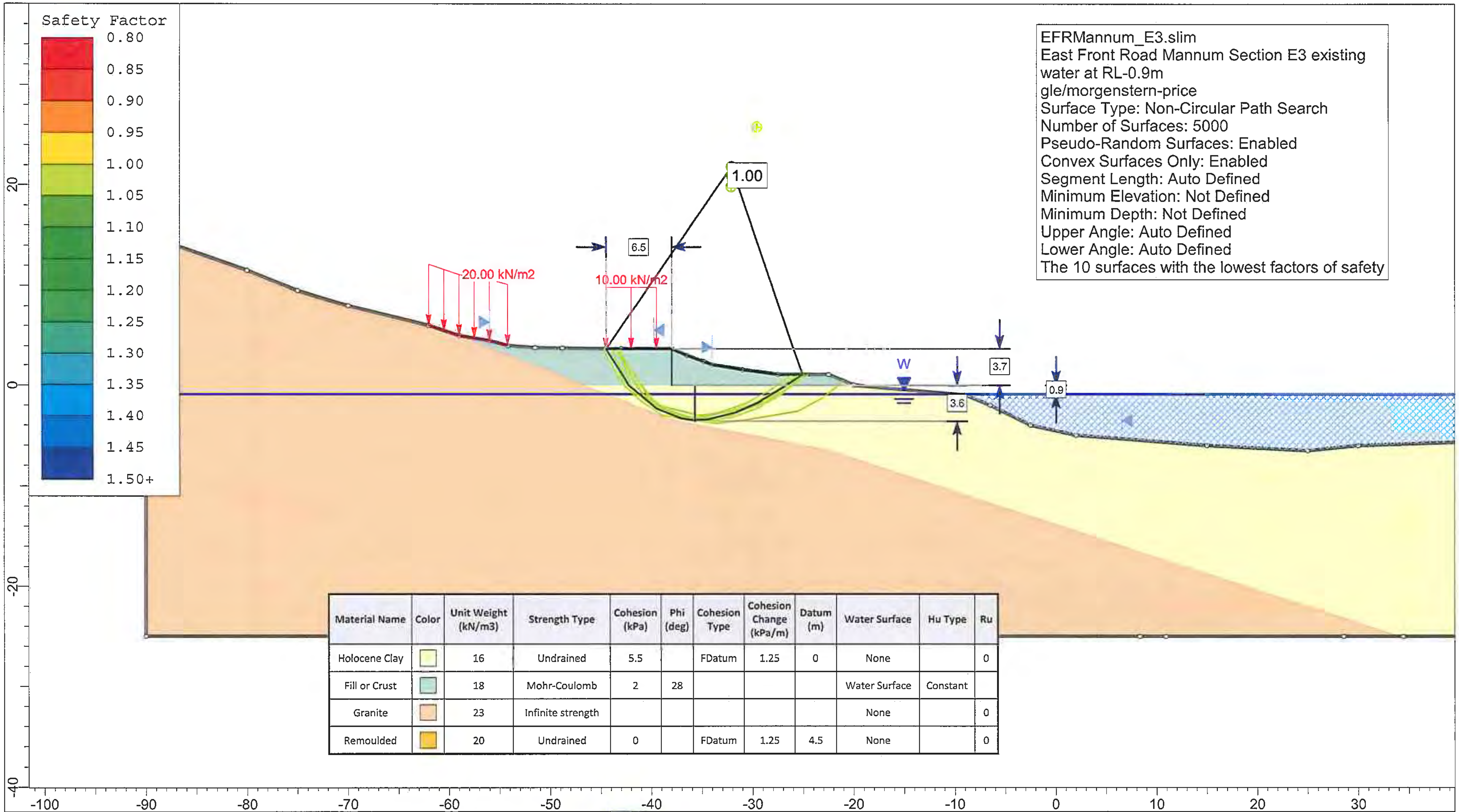
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Granite		23	Infinite strength						None		0
Remoulded		20	Undrained	0		FDatum	1.25	4.5	None		0

Author	RAD
Approved	ATM 12 Dec 2012
Date	23/9/2012
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Client:	Coffey Geotechnics Pty Ltd
Project:	East Front Road Mannum Section E1 - lower road to RL2
Title:	East Front Road Mannum Section E1 - lower road to RL2 remoulded clay under fill, water at RL-0.9m
doc no:	GEOTMEND07093AA-AB
fig no:	D2

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\EFRMannum_E1_lower.slim



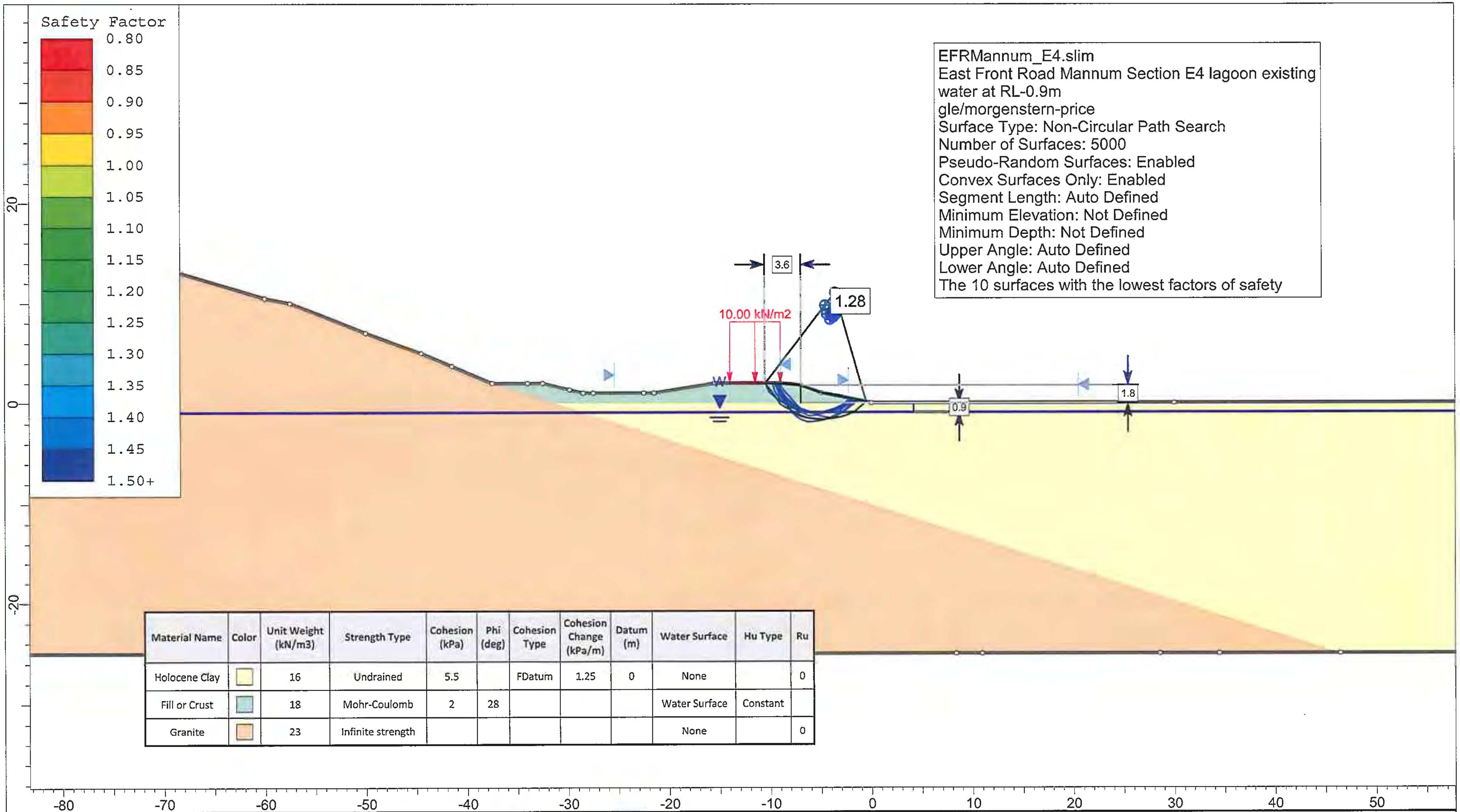
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Granite		23	Infinite strength						None		0
Remoulded		20	Undrained	0		FDatum	1.25	4.5	None		0

Author	RAD
Approved	ATM 12 Dec 2012
Date	23/9/2012
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING

Client:	Coffey Geotechnics Pty Ltd
Project:	East Front Road Mannum Section E3 existing
Title:	East Front Road Mannum Section E3 existing water at RL-0.9m
doc no:	GEOTMEND07093AA-AB
fig no:	D3

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\EFRMannum_E3.slim



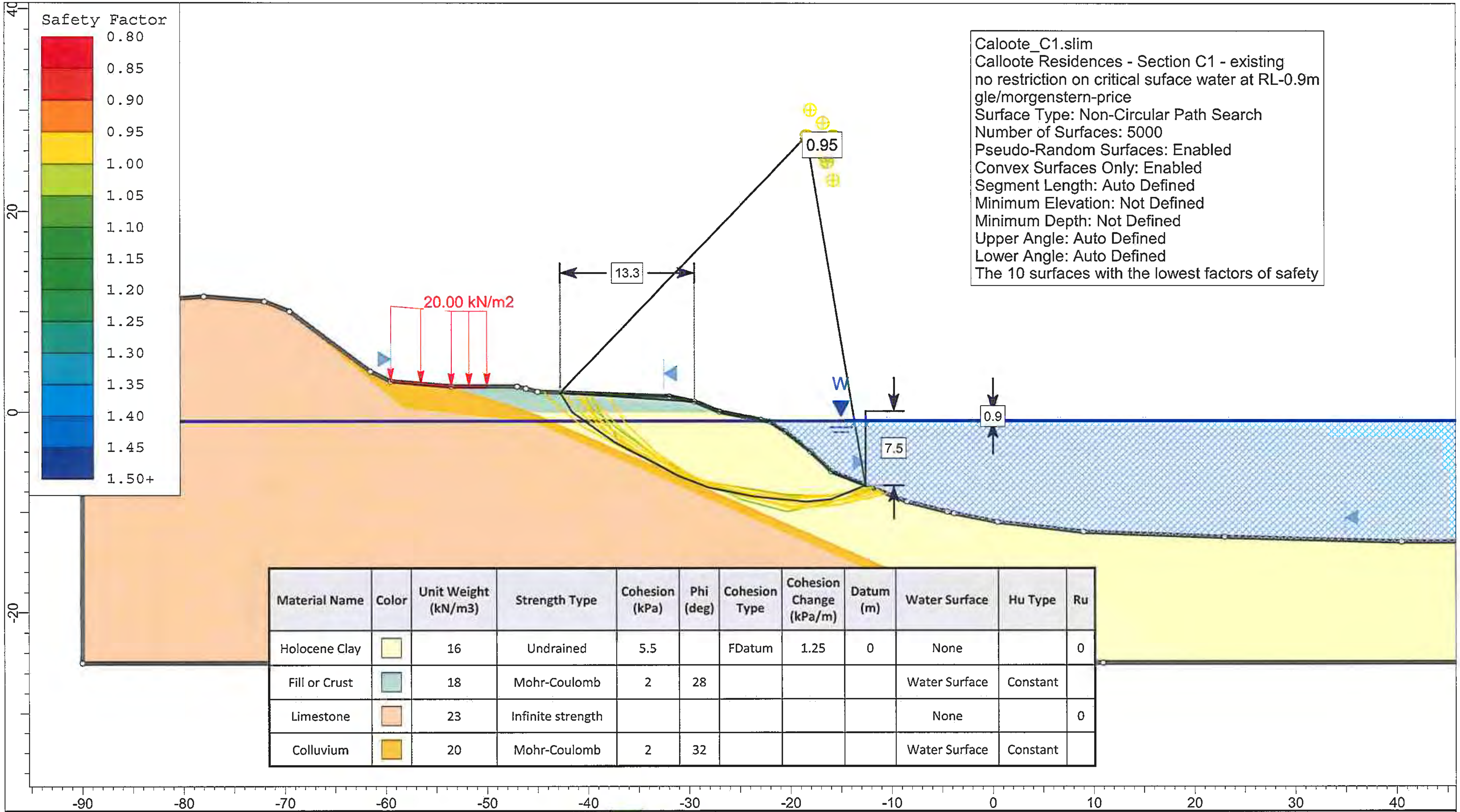
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Granite		23	Infinite strength						None		0

Author	RAD
Approved	ATM 12 Dec 2012
Date	23/9/2012
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Client:	Coffey Geotechnics Pty Ltd
Project:	East Front Road Mannum Section E4 lagoon existing
Title:	East Front Road Mannum Section E4 lagoon existing water at RL-0.9m
doc no:	GEOTMEND07093AA-AB
fig no:	D4

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\EFRMannum_E4.slim

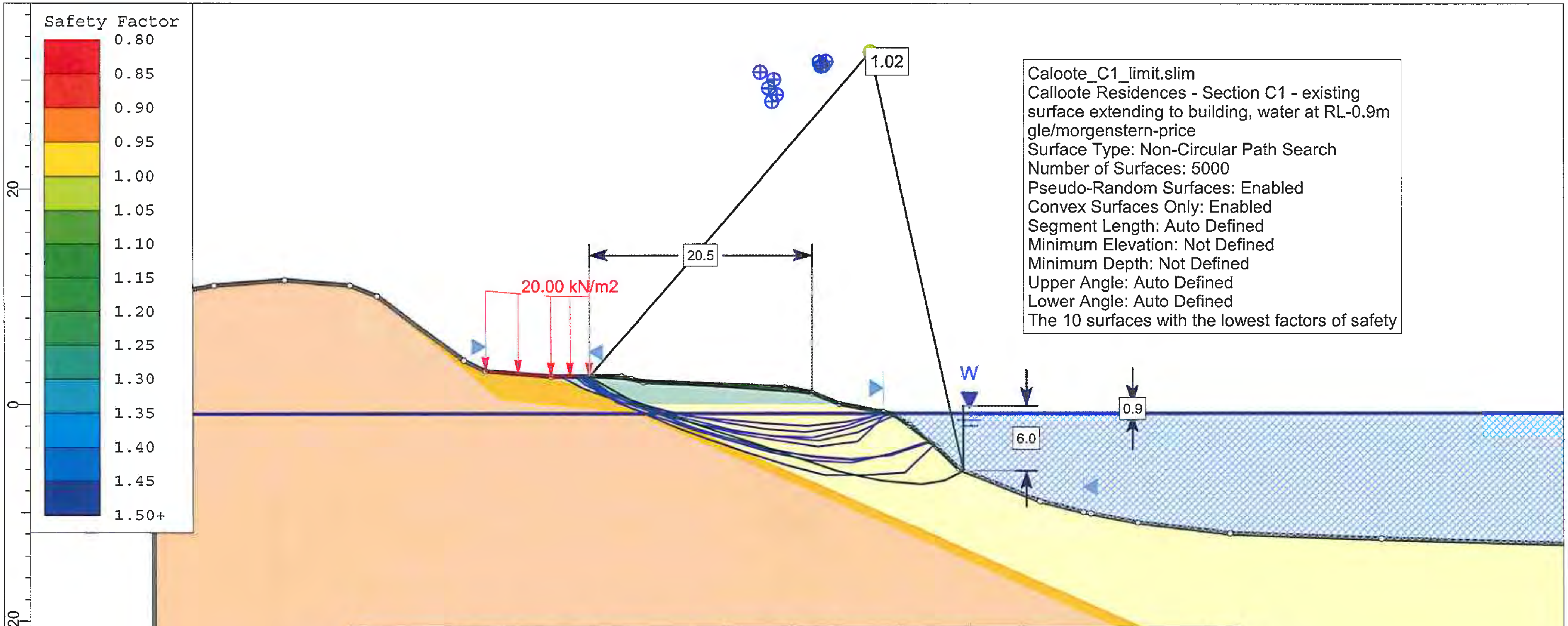


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Limestone		23	Infinite strength						None		0
Colluvium		20	Mohr-Coulomb	2	32				Water Surface	Constant	

Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12 Dec 2012	Project:	Caloote Residences - Section C1 - existing
Date	23/9/2012	Title:	Caloote Residences - Section C1 - existing no restriction on critical suface water at RL-0.9m
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	D5

coffey geotechnics
SPECIALISTS MANAGING

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Caloote_C1.slim

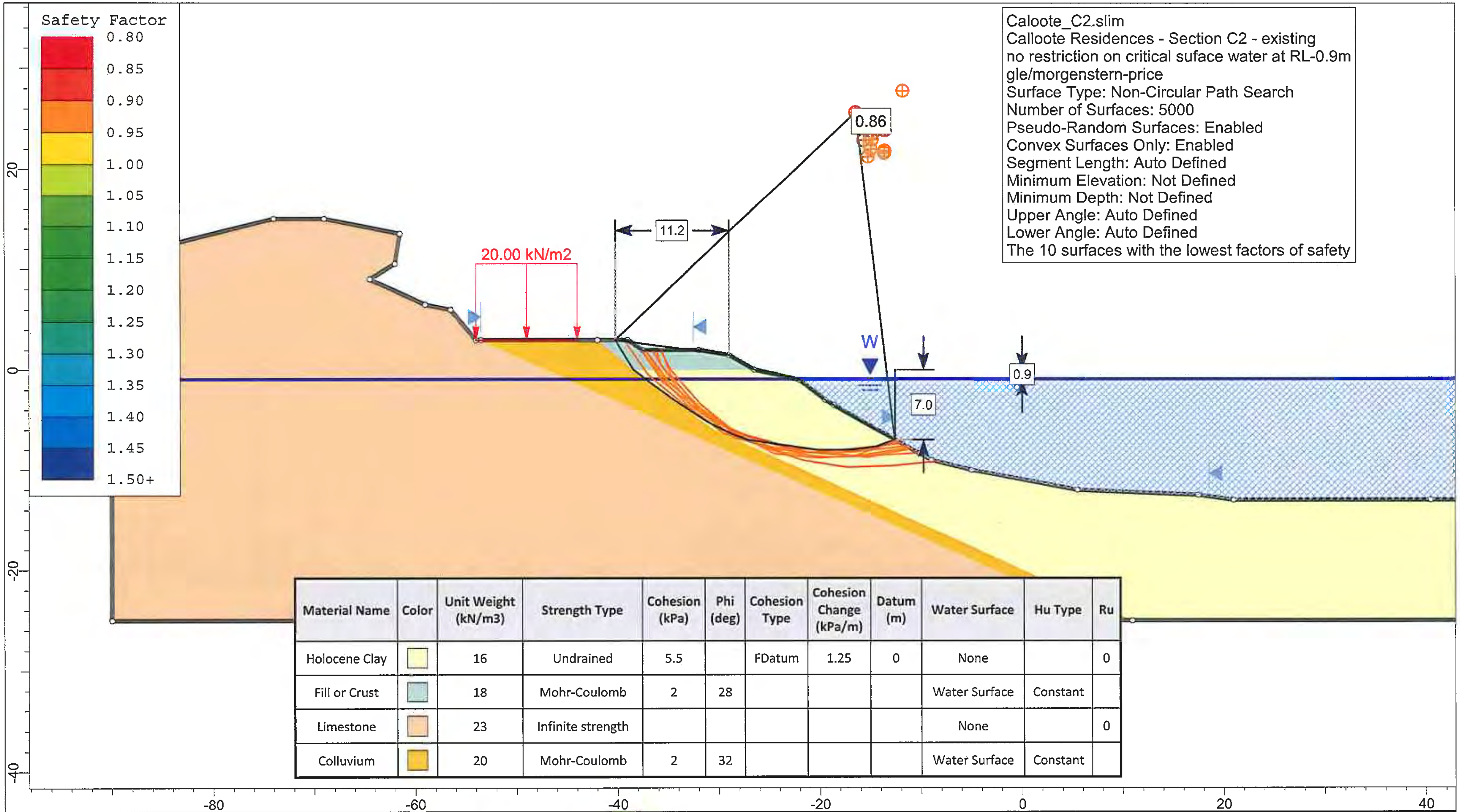


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Limestone		23	Infinite strength						None		0
Colluvium		20	Mohr-Coulomb	2	32				Water Surface	Constant	

Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12 Dec 2012	Project:	Caloote Residences - Section C1 - existing
Date	23/9/2012	Title:	surface extending to building surface extending to building, water at RL-0.9m
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	D6

coffey geotechnics
SPECIALISTS MANAGING

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Caloote_C1_limit.slim

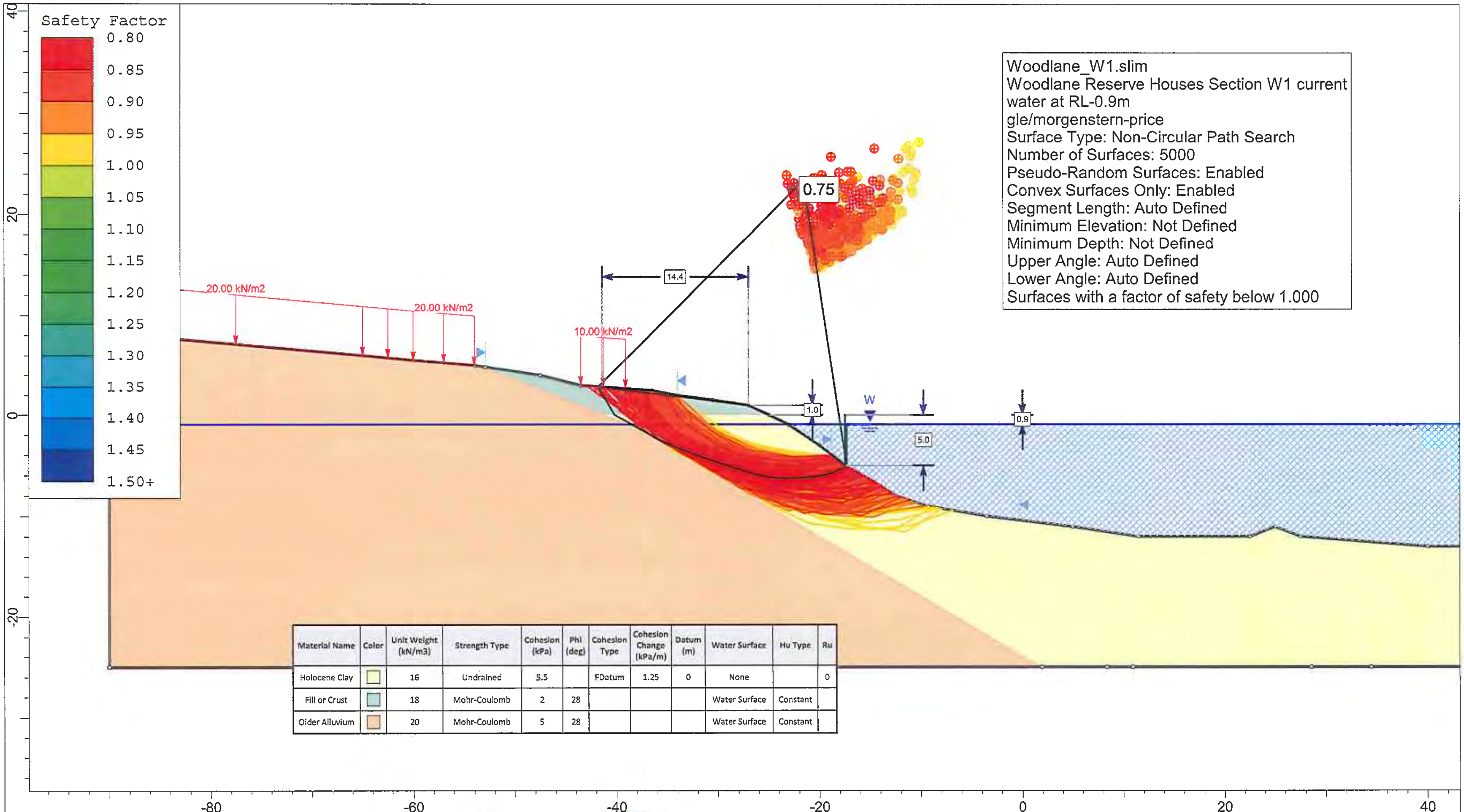


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Limestone		23	Infinite strength						None		0
Colluvium		20	Mohr-Coulomb	2	32				Water Surface	Constant	

Author	RAD	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12 Dec 2012	Project:	Caloote Residences - Section C2 - existing
Date	23/9/2012	Title:	Caloote Residences - Section C2 - existing no restriction on critical suface water at RL-0.9m
Scale	1:500	doc no:	GEOTMEND07093AA-AB
Orig Size	A4	fig no:	D7

coffey geotechnics
SPECIALISTS MANAGING

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Caloote_C2.slim



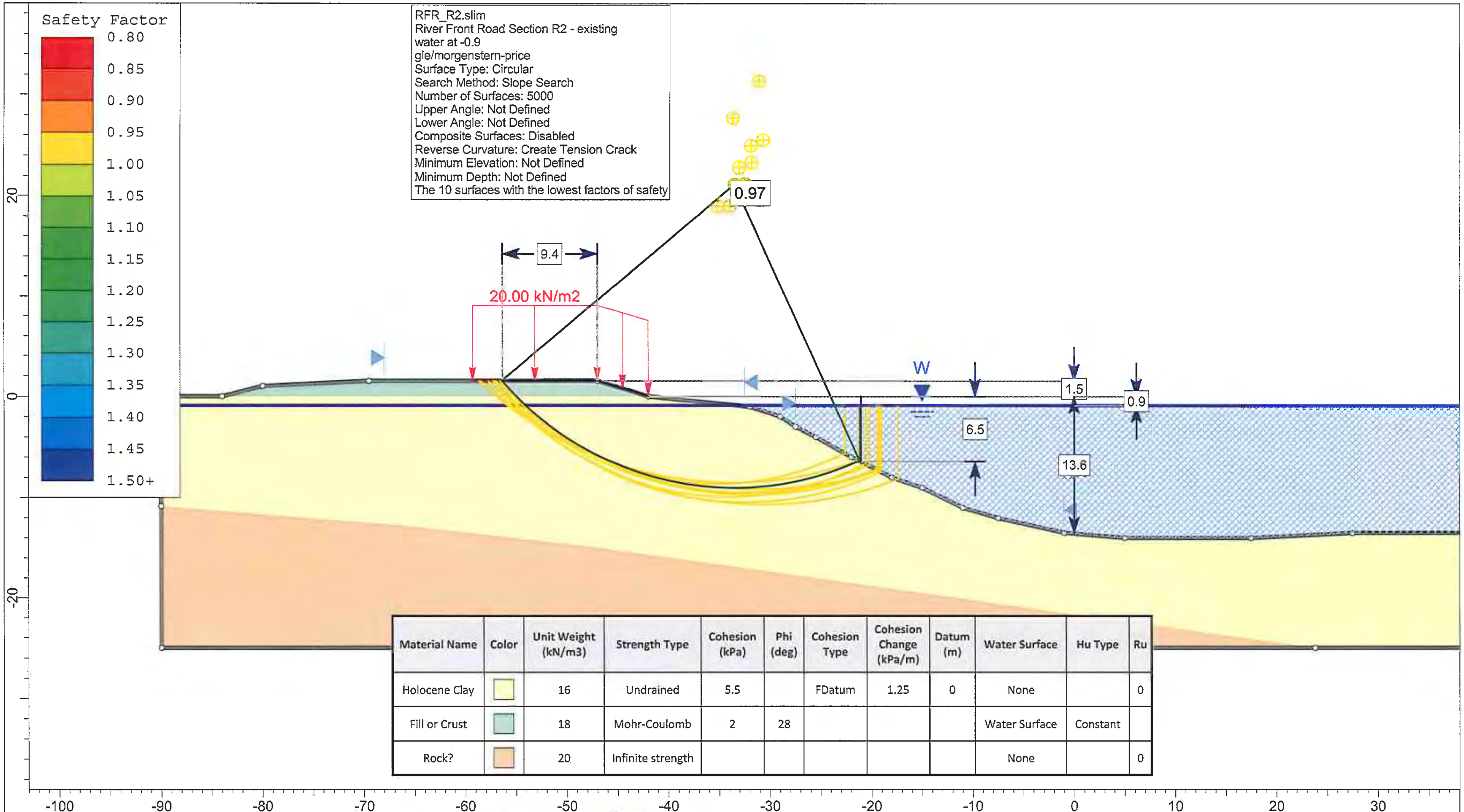
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Older Alluvium		20	Mohr-Coulomb	5	28				Water Surface	Constant	

Author	RAD
Approved	ATM 12 Dec 2012
Date	23/9/2012
Scale	1:500
Orig Size	A4

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

Client:	Coffey Geotechnics Pty Ltd		
Project:	Woodlane Reserve Houses Section W1 current		
Title:	Woodlane Reserve Houses Section W1 current water at RL-0.9m		
doc no:	GEOTMEND07093AA-AB	fig no:	D8

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\Woodlane_W1.slim



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Cohesion Change (kPa/m)	Datum (m)	Water Surface	Hu Type	Ru
Holocene Clay		16	Undrained	5.5		FDatum	1.25	0	None		0
Fill or Crust		18	Mohr-Coulomb	2	28				Water Surface	Constant	
Rock?		20	Infinite strength						None		0

Author	RAD	coffey geotechnics SPECIALISTS MANAGING	Client:	Coffey Geotechnics Pty Ltd
Approved	ATM 12 Dec 2012		Project:	River Front Road Section R2 - existing
Date	23/9/2012		Title:	River Front Road Section R2 - existing water at -0.9
Scale	1:500		doc no:	GEOTMEND07093AA-AB
Orig Size	A4		fig no:	D9

C:\Documents and Settings\rob_day\Desktop\GEOTMEND07093AA - Murray River Stability\CALC\Slide\RFR_R2.slim