Riverbank Collapse Along the Lower River Murray

- Literature Review

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1 Executive Summary

This report aims to present a critical and systematic review of the research literature associated with riverbank collapse. Particular attention is given to the possible causes of such collapse along the Lower River Murray between 2008 and 2010, during the peak of the *Millennium Drought*. The review examines the broad geotechnical, and geological and geomorphic, contexts of riverbank collapse along the Lower River Murray and these are treated separately in the report.

The first part focuses on existing methodologies applied to slope susceptibility assessment, the use of factors of safety and probabilities of failure in slope stability assessment, the influence of groundwater flow and vegetation on riverbank stability, the known and plausible slope failure process and triggering mechanisms, and finally, the proposed geographic information system (GIS) approach to be adopted in the development of riverbank collapse hazard mapping is discussed.

The second part of the report summarises the key findings from previous riverbank failure reports, related geotechnical investigation information and bathymetric survey data. The occurrence of bank failure is placed into the geological and geomorphic contexts through the examination of literature concerning the channel and floodplain sediments and the regional and recent geological evolution (late Quaternary and Holocene events) of the South Australian landscape.

This review highlights the multifaceted aspects of the riverbank collapse problem and how these are influenced by the dynamic nature and evolution of the river, as well as climatic factors such as rainfall and evaporation. It is identified that loading of the channel margin due to the placement of fill, or the construction of levees, is likely to increase the probability of riverbank collapse, particularly during periods of lowered pool-level and/or lowering of the river level. In addition, the ubiquity of shallow failures that almost certainly predate the large deep-seated 2009 – 2011 failures, indicates that the channel is naturally widening by mass failure of the channel margins, that the channel margins are probably inclined at angles that are near the natural limit of their stability, and that both this widening and shallow sliding is probably a response of the channel to its geomorphic evolution and geologic setting. Finally, informed by the review of the available literature, the report identifies several fundamental knowledge gaps and key research questions which will be investigated in the succeeding stages of this project.

2 Background

The stability of riverbanks is a multifaceted issue. To appreciate the processes affecting riverbank collapse and to understand the mechanics driving these collapse events, advanced modelling techniques, sophisticated engineering analysis and a large amount of site- or regional-specific data (e.g. river geometry, soil properties and their variability, and possibly the groundwater regime) are needed. In the case of the River Murray, there is limited recorded evidence of previous riverbank collapse incidents and the understanding of the processes driving these riverbank collapse events is not well understood. The River Murray is one of the only river systems in the world that can fall below sea level due to the barrages preventing the inflow of sea water during periods of low river flows. Other riverbank collapse events globally, typically result from lower bank scour erosion and rapid draw down of river levels during and after flood events or periods of high flow.

A systematic process of risk management to date has identified a number of critical knowledge gaps in understanding hazard dynamics. This research project focuses on addressing fundamental knowledge gaps of collapse processes which is affecting the ability of the South Australian Department of Environment, Water and Natural Resources (DEWNR) to assess accurately and reliably the likelihood of failure events and riverbank collapse risk. The outcomes of this project will enable DEWNR to undertake comprehensive risk assessments now and in the future, and at a variety of scales, to develop and implement long-term hazard management and site specific management plans.

As part of the risk assessment process (and in an attempt to develop a predictive capability), spatial analysis to correlate the distribution of incidents with potential driving factors has been undertaken. To date, the results of this work are inconsistent and require access to data and knowledge including floodplain processes, river bathymetry and sub-channel sediment composition.

Advice received from the Riverbank Collapse Hazard (RbCH) Expert Panel indicates that further research is required to identify the mechanisms prior to riverbank collapse events occurring and identify the soil and riverbank characteristics that are causing some areas of the riverbank to be more susceptible to collapse than others. While an extensive geotechnical investigation was undertaken at seven key sites in September 2009, these sites only represented a small snapshot of areas affected by riverbank collapse. Additional investigations are required to analyse a representative range of affected sites whilst also increasing the suite of analyses undertaken to develop robust stability models and complete risk assessments. Furthermore, there is clearly a need for sections of the river with different land uses, geometries and geologies to be identified, and for all failures associated with the recent river level lowering to be identified.

The project will acquire data for developing a range of failure models, stability assessment models and predictive tools. This will be achieved by building upon existing knowledge of riverbank collapse gained in recent years via direct involvement with local councils, SA Water, Golder Associates, Sinclair Knight Merz and DEWNR through investigations conducted at sites known to be affected by riverbank collapse.

3 Objectives

The project will address key knowledge gaps aimed at obtaining the necessary information to answer key decisions in the future management of the Riverbank Collapse Hazard. These include:

1. What are the failure mechanisms driving riverbank collapse events;
2. What are the potential triggers for future riverbank collapse events that should be monitored and managed;
3. What is the safe operating range of the river to minimise the impacts of river level fluctuations on riverbank stability;
4. What are potential long-term sustainable management options for key high risk riverbank collapse affected sites; and
5. What areas along the Lower River Murray are likely to be more susceptible to riverbank collapse events.

From the research outcomes DEWNR will develop a long-term management strategy for riverbank collapse and identify changes that are required to development planning guidelines and legislation to reduce the likelihood of future risks associated with riverbank collapse events.

4 Projects

This is a collaborative research between the University of Adelaide and the University of Sydney, and this project consists of two parallel investigations as outlined in the following:

1. **Investigators: Prof. Mark Jaksa, Assoc. Prof. Bertram Ostendorf, Dr. Yien Lik Kuo, Simon Chen Liang (University of Adelaide)**

   The first part of the project will explore occurrences of riverbank instability along the River Murray downstream of Lock 1 at Blanchetown (Figure 1.1). It will use geotechnical data obtained from investigations performed by consultants at the sites of bank collapse, as well as additional data obtained during the project by the research team. Modelling will be performed using state-of-the-art geotechnical slope stability analysis software in conjunction with Geographic information system (GIS). Climatic conditions, geotechnical characteristics and river levels will be varied systematically in order to develop an enhanced understanding of the mechanisms which influence riverbank instability. Finally, the first part of the project will develop a risk-based model to express the probability of occurrence of riverbank instability that will facilitate future planning and development.

2. **Investigators: Assoc. Prof. Tom Hubble, Elyssa De Carli (University of Sydney)**

   The second part of the project will investigate known occurrences of riverbank instability along the River Murray downstream of Lock 1. It will use previously acquired multibeam bathymetry and sources of archival aerial photographs and other historical information, such as early surveys and maps, to assess whether or not bank instability on the Lower Murray is a new and recent phenomenon or if the recent bank failures are examples of a longer-term geomorphic process. The project will also undertake geophysical profiling and collect geological samples that provide relatively undisturbed material for geotechnical testing to determine the spatial distribution of the sediments that form the banks. This information will be used to develop an explanatory geological model for the occurrence of bank instability on the Lower Murray that will contextualise and enable robust geotechnical modelling of slope failure. Multibeam bathymetric data will be examined using state-of-the-art, three-dimensional representation software and these data, as well as geophysical profile data, sediment distribution data and archival airphoto information, will be managed using GIS software. This information will be integrated with slope stability models to develop an enhanced understanding of the bank failure process on the Lower Murray, its geological context, and the potential for reoccurrence which will facilitate the management of this hazard.

5 Literature Review and Knowledge Gap Analysis

In the first half of 2013, the joint research team, from the Universities of Adelaide and Sydney has undertaken a literature review and knowledge gap analysis which involved the following:

- Literature review of existing reports and investigations;
- Evaluation of existing data;
- Diagnostic assessment of past failure events; and
- Analysis of spatial and temporal distributions.

These are detailed in two separate parts of the report, which together provide key background knowledge, summarise past bank failure reports and related geotechnical investigation information, place the occurrence of bank failure into geotechnical, geological and geomorphic contexts, and identify the key questions and knowledge gaps for investigation in the next stage of this Goyder Institute Research Project: E.1.8 Riverbank Collapse in the Lower River Murray.

Part 1

Geotechnical

Professor Mark Jaksa, Dr. Yien Lik Kuo and Mr. Simon Chen Liang

6 Project Introduction

Slope instability is one of the major problems in geotechnical engineering where loss of life and property can and do occur (Vanmarcke, 1977). Slope instability can arise both in human-made or natural slopes (e.g. mountainous regions, embankments, road cuts, open-pit mining, excavations, riverbanks and landfills) around the world. Consequently, not only are considerable financial costs incurred, but also major ecological and environmental problems occur over large geographical areas (Li, 1994; Larsen and Torres-Sanchez, 1998). Mapping or delineating areas susceptible to slope failure is essential for land-use planning in mountainous or riverbank areas.

Slope stability failures that have occurred along the Lower River Murray downstream of Lock 1 and riverbank instability is the focus of the current study. The River Murray is the longest river in Australia at 2,375 kilometres in length. It begins in the Australian Alps and terminates in Goolwa, South Australia at the Murray mouth in the Southern Ocean, with an annual average discharge of 767 m^3/s and a history spanning more than 60 million years (Figure 1.1**Error! Reference source not found.**). The River Murray is one part of the Murray-Darling Basin catchment area which covers around one million square kilometres. This is about one-seventh of Australia's land mass, and extends across parts of South Australia, Victoria, New South Wales and Queensland.

The Lower River Murray (which is roughly a 210 km stretch from Lock 1 at Blanchetown to Wellington, as shown in Figure 6.2) was adversely affected by unprecedented low water levels during the millennium drought, which resulted in several bank-slope failures. The instability of the riverbank caused by the low water level in the river during the drought now poses a number of economic and ecological problems for the residents and the environment adjacent to the failure location (approximately 40 metres from the centre of the river bed) in the form of land loss, tension cracks, riparian tree collapse, destabilized structures, impairment of water quality and downstream aggradations with excessive sediment delivery. As recorded in state government inventories, there were several major and 50 smaller riverbank collapses between Blanchetown and Wellington in 2009. Some of the collapses occurred catastrophically (i.e. soil mass, vegetation and infrastructure rapidly collapsing into the river without warning); while at other locations, collapse occurred less rapidly (Miller and Sias, 1998).

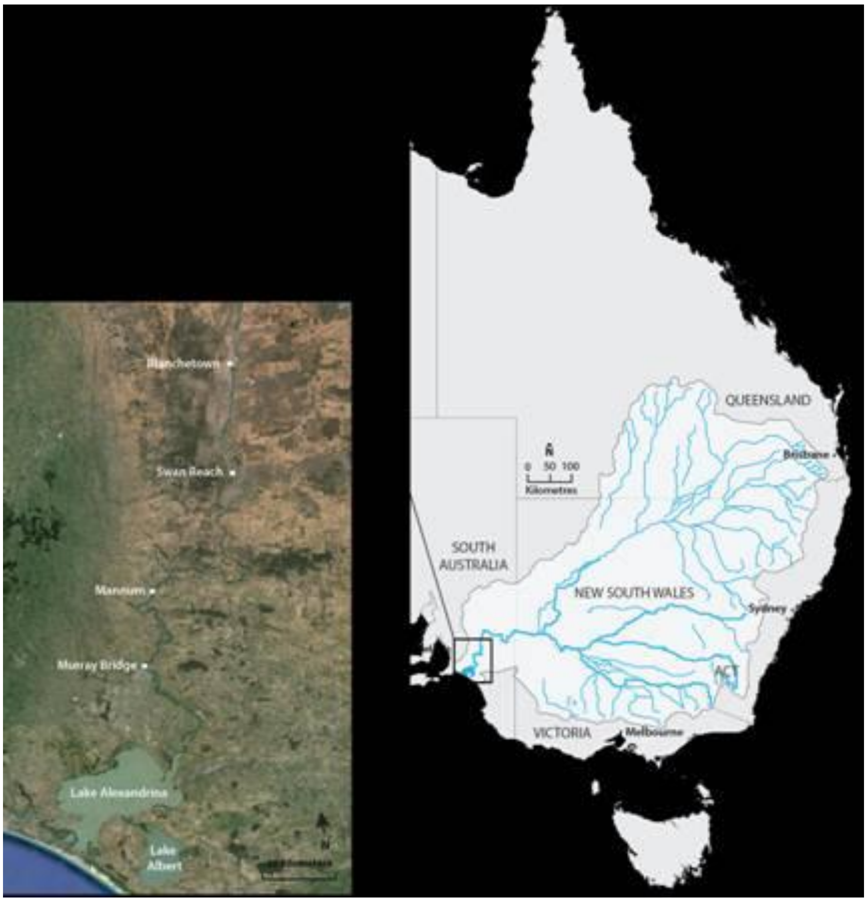


Figure 6.1 Overview of the River Murray and the study area.

Riverbank slope failures, as shown in Figure 6.3, are influenced by several factors:

- climatic, such as precipitation and evaporation;
- river level fluctuation;
- geological factors, including soil and rock properties;
- topographical factors, including slope gradient, aspect and angle;
- riparian vegetation (grasses/woody species); and
- land use/cover factors, including infrastructure.

In practice, probabilistic, deterministic, statistical, empirical and monitoring are five major approaches that are often used for slope instability assessment (Hartle'n and Viberg, 1988). The stability of a slope is usually assessed using conventional limit equilibrium methods (LEM) and finite element methods (FEM) in a deterministic or probabilistic framework, accompanied by site exploration to acquire the geotechnical data to support calculations.

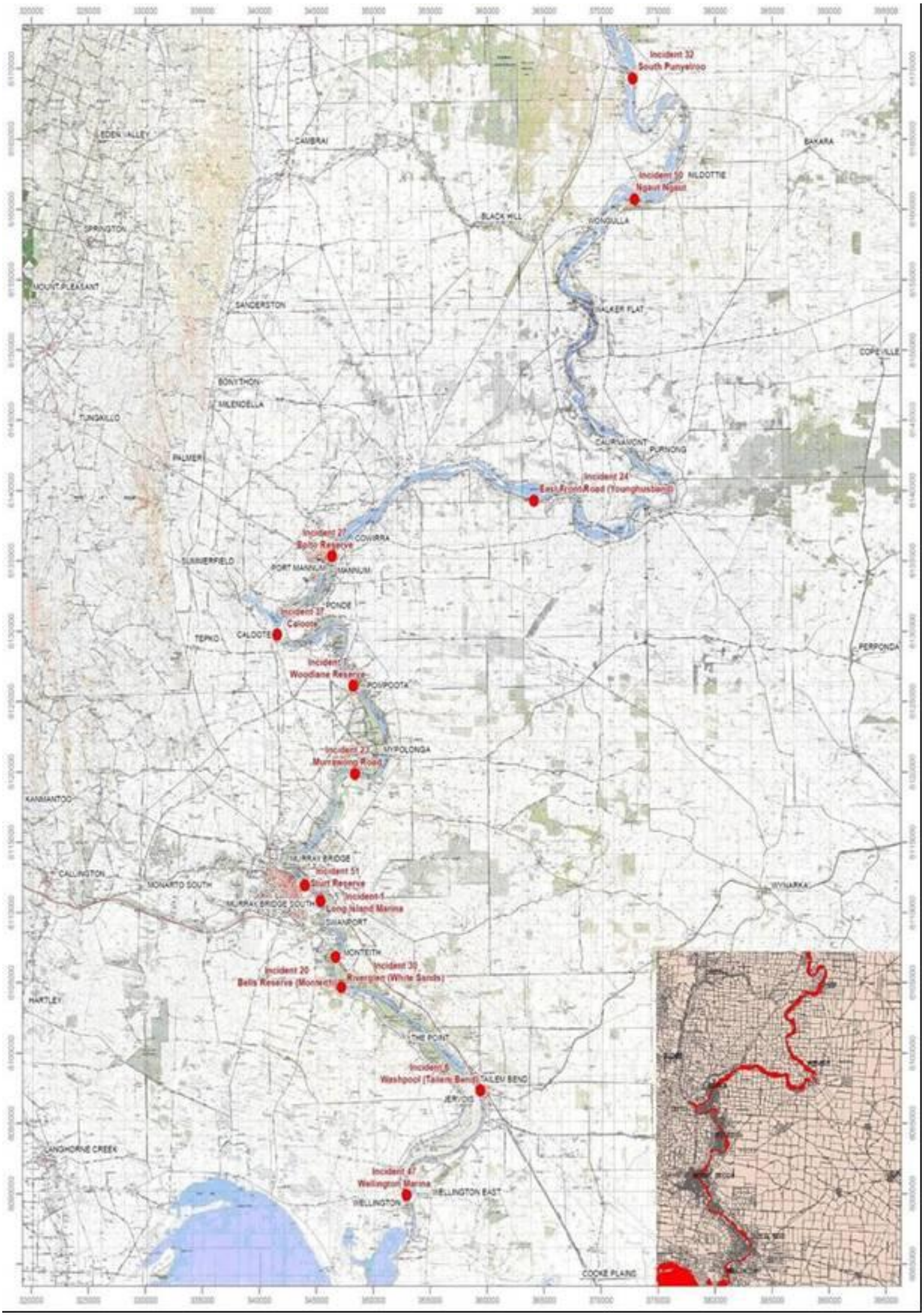
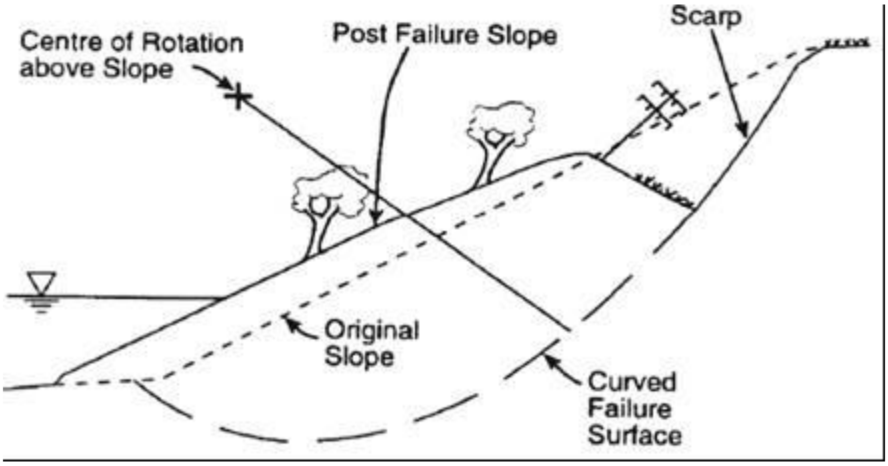
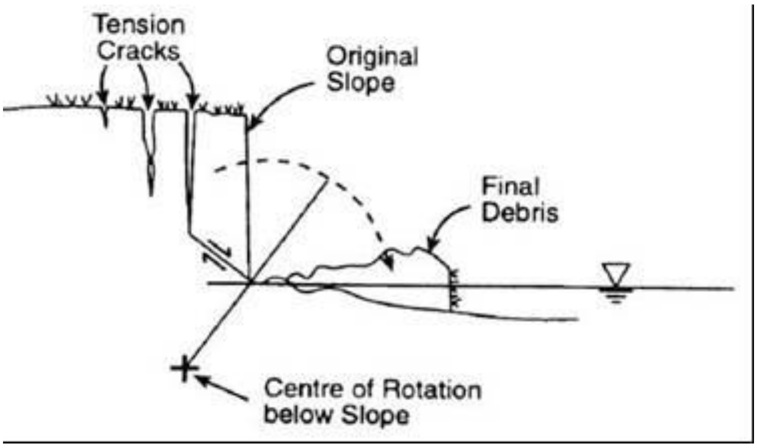


Figure 6.2 Overview of Lower River Murray (Source: SKM, 2010).



(a)



(b)

Figure 6.3 Slope failure on riverbanks (a) rotational slip on over-steepened riverbanks, (b) slab failure on over-heightened riverbanks) (Source: Thorne, 1999).

However, when the areas of research expand to a regional scale, hazard assessment and mapping become complex and difficult due to the time and effort required for the manual handling and processing of the data (Dhakal et al., 1999). Furthermore, the results become inaccurate because the subsoil profiles and the land-use distributions and topographies often vary significantly due to the increasing scale.

To account for these spatial variabilities and to facilitate the analysis and mapping of slope instability, a geographical information system (GIS) based model can be adopted which provides:

- spatial data pre-treatment;
- spatial visual interpretation;
- spatial item vectorisation; and
- database construction incorporating both probabilistic and deterministic methods.

The GIS can assist researchers obtain better information in two and three-dimensional space, which leads to improved decision making. In a GIS, data about real-world objects are linked to a map. Geographical features are accessed and displayed quickly and can be presented using different information in the database.

Recently, GIS technology has greatly facilitated the handling, processing, analysing and reporting of data (Burrough, 1986; Aronoff, 1989; Marble, 1990). In addition, with the development of remote sensor (RS) technology, more and more data have become available in a high resolution digitised format (such as LiDAR based topographical maps; soil maps; digital elevation models; and land use/cover maps). These high resolution data provide high-precision, more reliable, comprehensive and multifunctional treatment options for spatial and temporal analysis in GIS.

This report on the current study provides a brief review of the research into slope stability and the riverbank failure process, including the influence of vegetation root effect. It introduces the GIS approach to landslide hazard mapping and then identifies the key research areas and knowledge gaps related to the project.

7 Literature Review

7.1 Methods of approaches to slope susceptibility assessment

Generally speaking, the methods of approaching slope susceptibility assessment can be either direct or indirect, and can be divided into two main classifications: qualitative and quantitative. There are many approaches that can be employed to assess slope stability and landslide hazards with different requirements to be emphasised in different situations (Sidle et al., 1985; Dietrich et al., 1986; Montgomery and Dietrich, 1988; Montgomery and Dietrich, 1989; Carrera et al., 1991; Dietrich et al., 1992; Sidle, 1992; Dietrich et al., 1993; Montgomery and Dietrich, 1994; Wu and Sidle, 1995; Pack, 1995).

There are four widely used methods by which slope stability is usually assessed. The choice of method depends on various characteristics of the slope, including situations where the stability is probably controlled by surface topography through shallow subsurface flow convergence; or partly controlled by soil saturation index fluctuations; sometimes by porewater pressure fluctuations; or by soil shear strength changes (Montgomery and Dietrich, 1994). Currently, there are 4 main categories of assessment methodologies by which to assess slope stability, namely: (1) field inspection with empirical experiences to help identify sites susceptible to landslides, accompanied by the predictions made from analysis of landslide inventories; (2) multivariate analysis of physical influencing factors; (3) stability ranking with statistical analysis based on criteria such as slope, lithology, land use form, or geology structure and (4) probability analysis of slope failure based on slope stability models with hydrologic simulations. The more detailed categories of assessment methodologies are shown in Figure 7.1.

7.2 Methods for calculating the factor of safety

7.2.1 Introduction

Once the geometry and the subsoil conditions, including the groundwater level beneath a slope, have been determined, the stability can be assessed using either published chart solutions or numerical modelling (Abramson et al., 2002). The methods include the use of the limit equilibrium method (LEM) to analyse two-, and sometimes three-dimensional slope

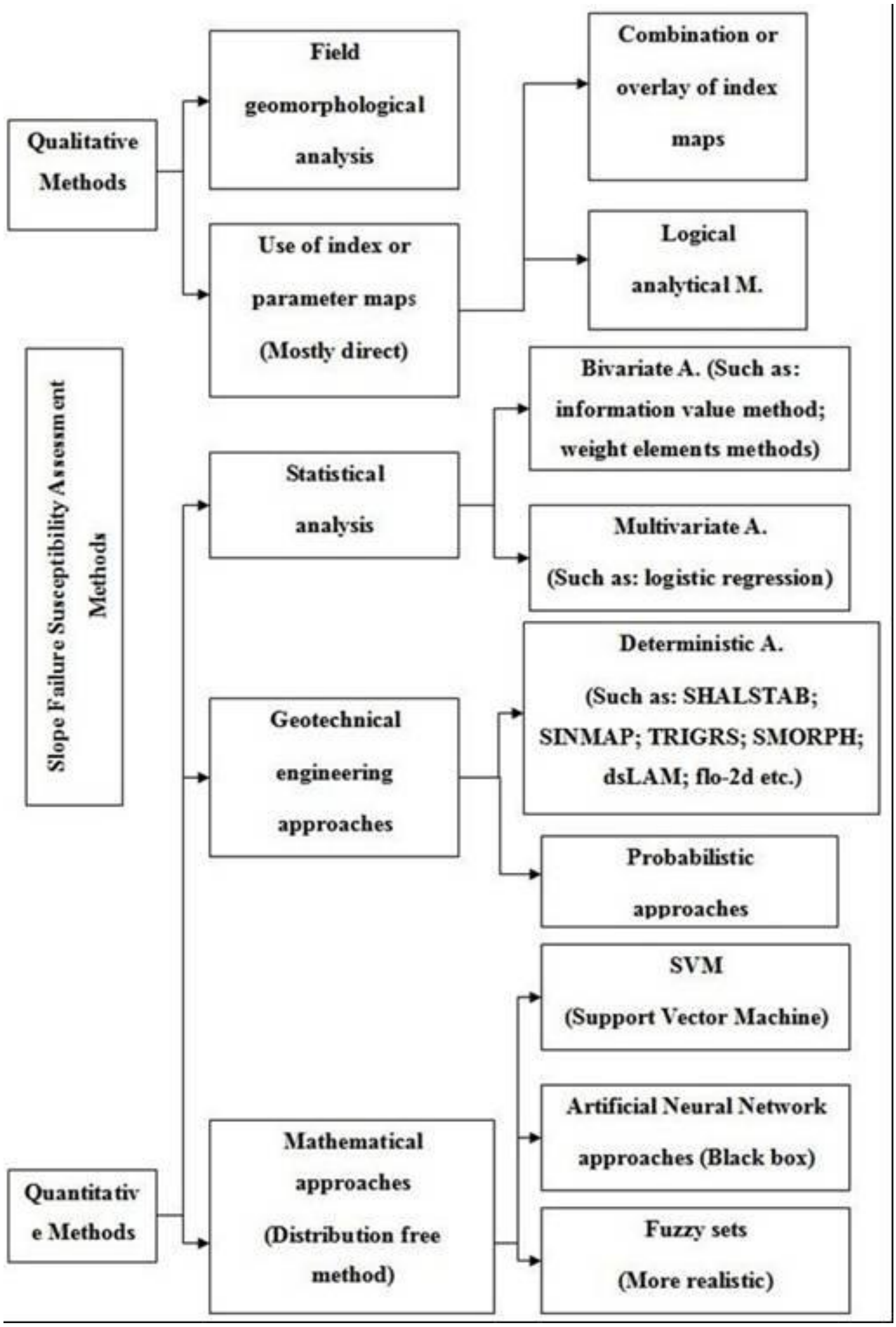


Figure 7.1 Proposed classification of slope failure susceptibility assessment methods (Source: Aleotti and Chowdhury, 1999)

models; some complex numerical methods that employ finite element methods or boundary element methods, especially, the probabilistic slope stability analysis models such as: the first order second moment (FOSM); Rosenblueth's (1975) point estimate method; Monte Carlo simulation; and considerations for incorporating spatial variability into the finite element method (Abramson et al., 2002).

7.2.2 Conventional calculation

Conventionally, in slope stability analysis, the failure surface along which sliding occurs is speculated and an analysis is then performed to determine the shear forces acting on the failure surface and the shear resistance that the soil can mobilise against sliding (Craig, 2004). A factor of safety, *FS*, against failure is then calculated as the ratio of forces opposing motion to the forces causing motion, that is:

$$FS = \frac{\text{Forces opposing motion}}{\text{Forces causing motion}} = \frac{\text{Shear resistance against sliding}}{\text{shear force acting on the failure surface}}$$ **Equation 7.1**

FS is calculated for a number of speculated or known potential sliding surfaces, and the minimum value is taken as the factor of safety against slope failure. A $FS < 1$ is indicative of instability. When $FS = 1$, these forces are exactly balanced, and any slight increase to the forces causing motion, or slight reduction to the forces opposing the motion can result in instability (DFW, 2010). As *FS* increases beyond unity, the slope becomes more stable.

In practice, to suit different conditions and the requirements of the research, the *FS* criteria can be modified because the various methods differ in their assumptions and the manner in which equilibrium conditions are satisfied. Based on Equation 2.1, a wide variety of methods have been developed for slope stability analysis with different kinematics associated with each (e.g. Sowers, 1979; Whitlow, 1990; Fang, 1991; Montgomery and Dietrich, 1994; van Westen and Terlien, 1996; Centre for Geotechnical Research, 1998; Burton and Bathurst, 1998; Pack et al., 1998, 2001; Borga et al., 2002; Saha et al., 2002; Dhakal and Sidle, 2004; Craig, 2004).

In most common use in geotechnical engineering is the limit equilibrium method (LEM) and several commercial software packages are available which utilise this approach. The majority of stability analyses are carried out in terms of effective stresses in problems where changes in porewater pressures take place. Because of the variations in these stresses along a trial slip surface, the slip mass is considered as a series of slices, as shown in Figure 7.2, where *W* = the body weight of the slice; N' = the effective normal reacting force at the base of the slice; T = the shearing force induced along the base = $W\sin\alpha$; R_1 and R_2 = forces imposed on the sides from adjacent slices, which may be resolved into: E_1 and E_2 = normal interslice forces; and X_1 and X_2 = tangential interslice forces (Whitlow, 1990). A trial slip circle is selected having a centre, O, and a radius, R, and the horizontal distance between the two ends A and B divided into slices of equal breadth, b (Whitlow, 1990).

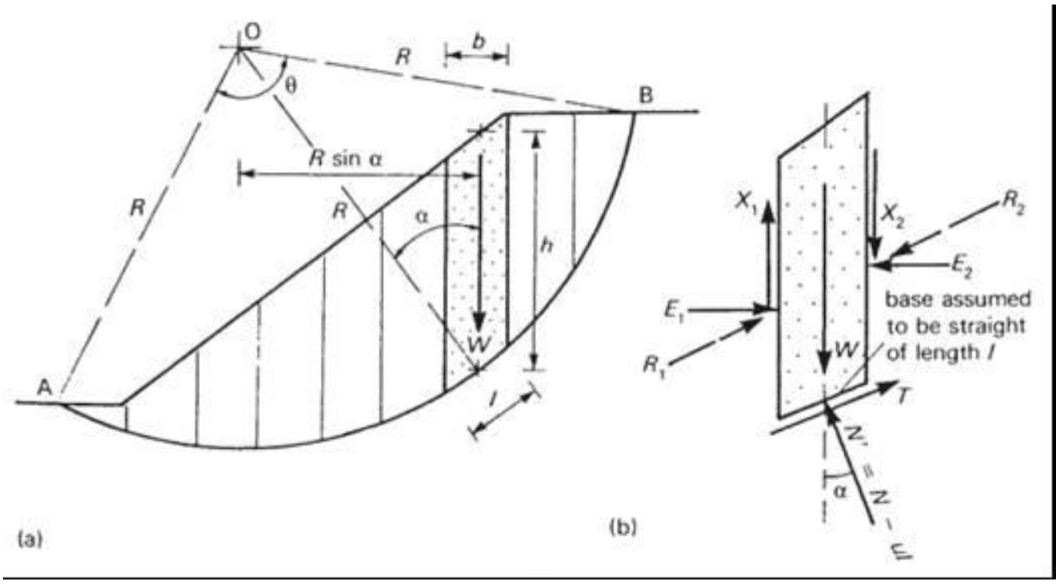


Figure 7.2 Method of slices: (a) division of slip mass; (b) forces on a slice (Source: Whitlow, 1990).

The simplified LEM is based on two assumptions:

1. The soil mass is discretised into several vertical slices in the direction normal to the plane of the section. The forces at the ends of each slice are negligible; that is, a purely two-dimensional approach is adopted.

2. The Coulomb failure criterion applies. The factor of safety, *FS*, is defined such that, when c' and $\tan\phi'$ are replaced by c'/FS and $\tan\phi'/FS$, the conditions become those of limiting equilibrium. It is also assumed that all slices have the same *FS*.

The effects of any surcharge loading on the surface must be included in the computation of the body weight and other forces. If a number of *K* slices is assumed, then:

$$\text{Total disturbing force} = \sum_{i=1}^{k} w_i \sin\alpha_i \quad \textbf{Equation 7.2}$$

$$\text{Total resisting shear force} = \sum_{i=1}^{k} \tau_i l \quad \textbf{Equation 7.3}$$

In terms of effective stresses:

$$\tau = c' + \tan\phi' \qquad \textbf{Equation 7.4}$$

and

$$\tau l = c'l + N'\tan\phi' \qquad \textbf{Equation 7.5}$$

Therefore:

$$\mathrm{FS} = \frac{\sum_{i=1}^{k} c_i' l + \sum_{i=1}^{k} \mathrm{N}'\tan\phi'}{\sum_{i=1}^{k} w_i \sin\alpha_i} \qquad \textbf{Equation 7.6}$$

Note that the *FS* being calculated by Equation 7.6 depends on the manner in which the values of *N*' are obtained**.** From Equation 2.6, it is apparent that long-term (drained) shear parameters, c′ and ϕ′, are used to determine the *FS*. However, the analysis of the riverbank failures, undertaken by several external consultants, has shown that collapses along the Lower Murray can be explained by short term (undrained) instability resulting from lowered water levels. The basis of this is an assumption that, even though the time scale involved is 2 to 3 years, the low permeability of the clayey soils suggests that undrained conditions are likely to be relevant. This research will examine this possibility in greater detail in Phases 3 and 4.

The role of effective stress on riverbank stability can be explained by Equation 2.6. A rise in groundwater level, and hence the porewater pressure, will reduce the effective stress, *N'* (refer to Figure 2.2), which in turn reduces the shear resistance against sliding and the factor of safety. On the other hand, a drop in groundwater level, and hence the porewater pressure, will increase the effective stress, *N'*, and hence increase the shear resistance against sliding and the factor of safety. Fluctuations in river level directly affect groundwater levels.

The shear strength of soils increases with consolidation under load. The soils below the water level are generally normal-consolidated. However, the clays at levels well above the water level are usually over-consolidated as a result of desiccation. As the clay dries out, the capillary tension in the porewater rises (matric suction) and can become quite large and cause the soil to shrink. Desiccation is a common factor in over-consolidation and tension cracking.

7.2.3 Infinite slope stability calculation

The infinite slope method is used to calculate the slope stability factor (Skempton and DeLory, 1957), which relates to a slope that extends for a relatively long distance with a consistent soil and groundwater profile. The method assumes an infinite slope and a failure plane parallel to the slope surface.

For cohesive-frictional (c - ϕ) soil in a fully saturated condition, the same limit equilibrium concept can also be applied to determine *FS*, as shown in Figure 7.3. As depicted in Figure 7.3, *U* represents the porewater force; and *S* is the effective normal force, determined as follows:

$$S = c'b\sec\beta + (N - U)\tan\phi'$$ **Equation 7.7**

and $W = \gamma_{sat}bh$. Therefore, *FS* can be obtained as follows:

$$FS = \frac{c' + h(\gamma_{sat} - \gamma_w)\cos^2(\beta)\tan\phi'}{\gamma_{sat}h\sin\beta\cos\beta}$$ **Equation 7.8**

where $\gamma' = \gamma_{sat} - \gamma_w$. For $c' = 0$ soil, the above expression may be simplified to:

$$F = \frac{\gamma'}{\gamma_{sat}} \cdot \frac{\tan\phi'}{\tan\beta}$$ **Equation 7.9**

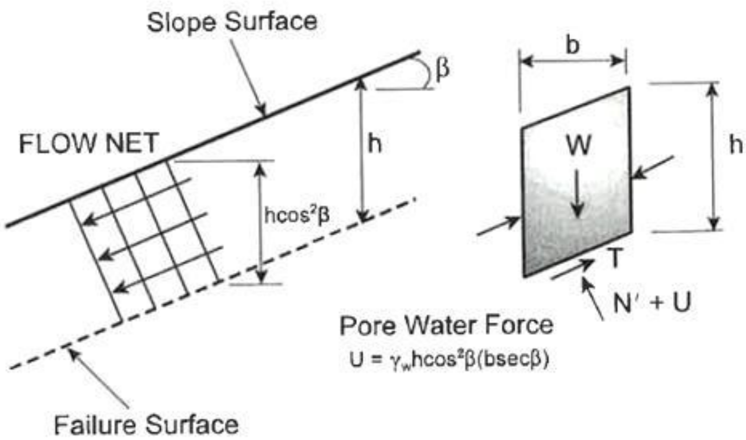


Figure 7.3 Infinite slope failure in *c*- ϕ soil with parallel seepage (Source: Abramson et al., 2002)

From Equation 7.9, it is clear that *FS* is independent of the slope height and depth, *h*, but is reduced by the parameter $\frac{\gamma'}{\gamma_{sat}}$. For typical soils, this reduction will b**e** 50% for fully saturated conditions when compared to dry conditions (Abramson et al., 2002).

7.2.4 Finite slope stability calculation

The finite element method (FEM) is a relatively new and more powerful method for slope stability calculation, which was first introduced to geotechnical engineering by Clough and Woodward (1967). Compared with the conventional simple LEM, the FEM cannot only resolve problems, such as newly constructed embankments, recent excavations or an existing natural slope like the conventional method, but can also account for K_0 (the ratio of lateral to vertical normal effective stresses), which is ignored in conventional limit equilibrium procedures (Chowdhury, 1981).

Compared with the conventional method, the use of the FEM has been limited to the analysis of complex earth structures, such as large earth dams (Duncan, 1996). This is because the quality of the FEM is directly dependent on the ability of the selected constitutive model to simulate realistically the nonlinear behaviour of the soil within the slope (Abramson et al., 2002). The FEM therefore refers to more sophisticated concepts and typically requires more work in determining model parameters, performing the computer analyses and evaluating the results (Duncan, 1996).

As shown in Figure 7.4, the FEM essentially divides the slope surface into discrete units called elements. Each node and predefined boundaries of the continuum, as shown in Figure 7.4, connects the neighbouring elements. The displacement method formulation of the FEM is typically used for geotechnical applications and presents results in the form of displacement, stresses and strains at node points (Abramson et al., 2002). In the FEM, the soil on the failure surface is modelled as numerous discrete elements, and the failure mechanism of these discrete elements is considered as a progressive phenomenon because not all elements fail simultaneously. The failure range can therefore extend from the point where yield first occurs to the final failure state where all elements have totally failed (Wong, 1984).

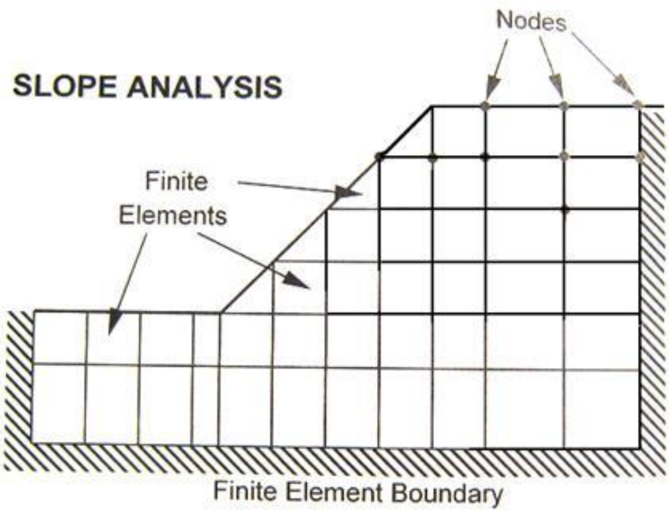


Figure 7.4 Definitions of terms used for finite element method (FEM) (Source: Abramson et al., 2002)

The FEM was first applied to slope stability analysis by Duncan and Dunlop (1969), who referred to it as the *limit shear failure criterion*. These researchers directly used the computed FEM stresses along a potential failure surface in order to estimate the *FS* value which would correspond to the ratio of available strength along the failure plane compared to imposed stresses (Duncan and Dunlop, 1969). Zienkiewicz (1971) later defined another failure criterion for the FEM referred to as non-convergence of the solution, as the shear strength parameters are reduced until non-convergence or a wide range of failures occur; and the *FS* can be reported as the ratio of the actual available strength to the lowest strength value. Zienkiewicz's (1971) approach has been used in more recent research by Dawson et al. (1999) and Griffiths and Lane (1999).

Based on the elasto-plastic soil model, Smith and Hobbs (1974) used the FEM for slope stability analysis on $\phi_u = 0$ slopes. After that, Zienkiewicz et al. (1975) and Griffiths (1980) introduced the FEM into $c' - \phi'$ slope stability analysis, and proved that the method was in good agreement with the results calculated by the conventional LEM. Snitbhan and Chen (1976) specified a maximum tolerable limit for the horizontal displacements of the surface of the slope, and named this new criterion *bulging of the slope line*. Since then, an increasing number of slope stability studies have focussed on the use of the FEM (e.g. Potts et al., 1990; Matsui and Sun, 1992; Jeremic, 2000; Lane and Griffiths, 2000; Lechman and Griffiths, 2000; Sainak, 2004; Zheng et al., 2006; Griffiths and Marquez, 2007; Li, 2007).

7.2.5 Slope Stability Classification

As discussed above, the value of the *FS* is that it is used to determine whether a slope is stable or not, and identify the stability class. Previous research has indicated that several considerations influence the selection of *FS* with respect to the slope stability class, such as the:

- uncertainties associated with and the nature of the loading;
- uncertainties associated with and variability of thickness and orientation of the soil layers;
- uncertainties in the measurement and the nature of soil strength in short term and long term loading situations;
- adoption of a reasonable lower quartile strength envelope for data;
- uncertainties in the failure mode;
- climatic influences which may affect soil strength;
- redundancy in the failure mode; and
- consequence of slope failure and the cost of over estimating the *FS*.

The selection of an appropriate *FS* depends on the levels of these uncertainties. For example, if the problem is well understood and the ground exhibits limited variability, a *FS* as low as 1.05 may be acceptable. Usually, in geotechnical engineering, however, only a small volume of the ground is tested and the problem is complex, so higher factors of safety are often adopted. On the other hand, it is because of the uncertainties described above, that the *FS* has limited value. For example, Figure 2.5(a) shows a situation where the applied load and the strength (resistance) of the soil exhibit large variability, as evidenced by the wide probability distribution functions. The overlapping area represents where the load exceeds the resistance and, hence, is the *probability of failure*. In contrast, Figure 2.5(b) shows the situation where the applied load and the strength exhibit less variability, perhaps due to a more detailed site investigation or a more homogeneous soil, and the resulting probability of failure is smaller. The *FS* in both cases, however, is identical and is not affected by uncertainties (Lee et al. 1983).

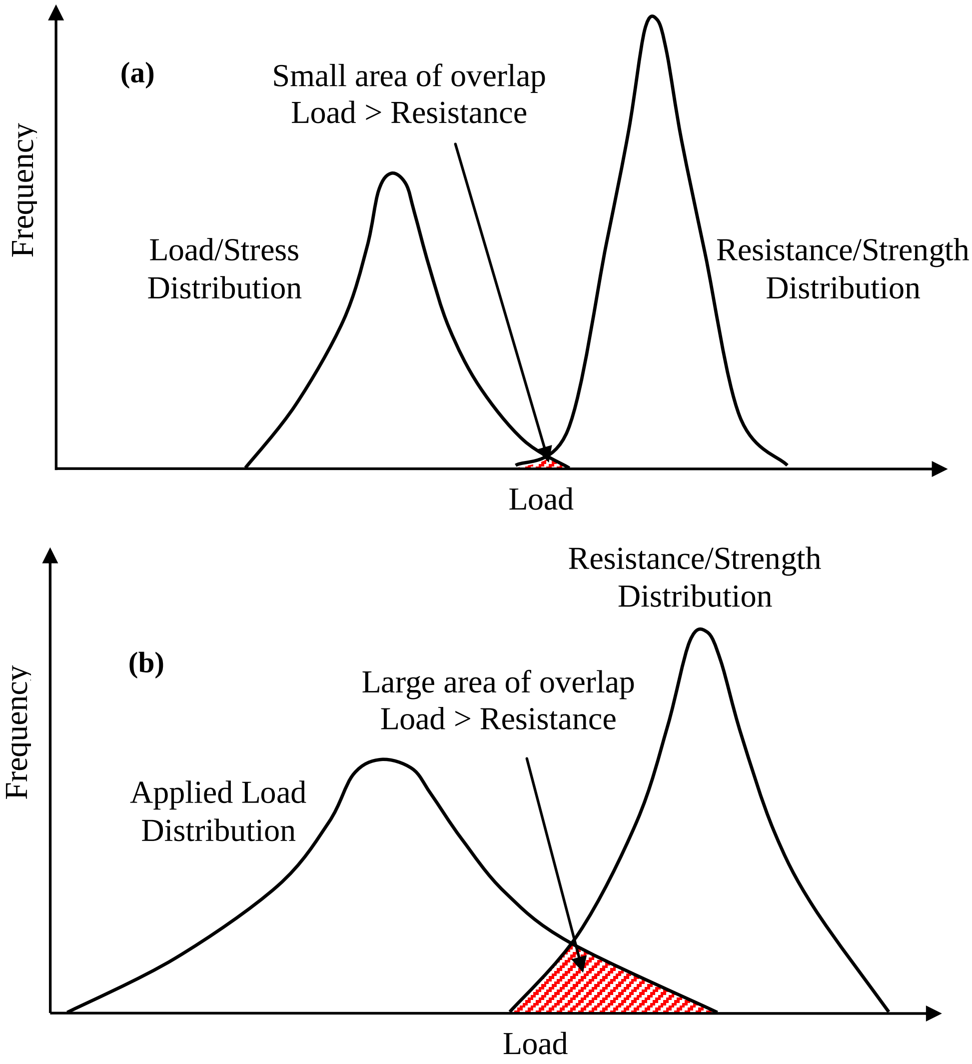


Figure 7.5 Limitation of *FS* compared with probability of failure

In order to calculate the probability of failure, a large amount of reliable data is needed. In the majority of cases in the assessment of the stability of slopes and riverbanks, such data sets are not available. Consequently, the factor of safety remains the dominant measure of safety, or failure.

The literature recommends a long term *FS* equal to 1.5 as a minimum for slopes. Based on the work of Ray and de Smedt (2009) suggested stability classes are given in Table 1. Slopes are denoted as unstable for cases in which *FS* is less than 1, quasi stable if *FS* is between 1 and 1.25, moderately stable if *FS* is between 1.25 and 1.5, and stable if *FS* is larger than 1.5. This study will examine the geotechnical data gathered from site investigation and the analyses, and recommend factors of safety to be adopted for classification of riverbank stability. Where possible, the probability of failure will also be evaluated.

Table 2.1 Slope stability classes (modified from Ray and de Smedt, 2009)

Safety factor	Slope stability class	Remarks
$FS > 1.5$	Theoretically stable	Only major destabilizing factors cause instability
$1.25 < FS < 1.5$	Moderately stable	Moderate destabilizing factors cause instability
$1 < FS < 1.25$	Quasi stable	Minor destabilizing factors can cause instability
$FS < 1$	Unstable	Stabilizing factors are needed for stability

7.3 Groundwater and subsurface flow

A rise in the groundwater table is generally considered to be a trigger mechanism for slope instability, because it raises soil saturation levels and increases porewater pressure, leading to the reduction of normal effective stresses and also the shear strength along potential failure surfaces (Ray and de Smedt, 2009). On the other hand, lowering of the river level in the Lower Murray has been found to cause slope instability; one of the reasons being the lower river level increases seepage flow pressure towards the river due to an increase in head difference.

Various slope stability models, which incorporate water seepage, have been proposed by a number of researchers. An example has been discussed earlier in §2.2.3. However this method, which is based on the assumption of an infinite slope and is used widely in hill- and mountain-slope stability analysis, may not suitable for riverbank stability analysis. Hubble et al. (2010) highlighted two ways in which they are different. Firstly, the scale of a riverbank slide feature is generally less than, but similar to, the size of the entire slope – riverbank slump commonly occupies more than 60% of the slope length (Abernethy and Rutherfurd, 2000). Secondly the hydrological conditions which contribute to failure are usually different, with overbank flooding saturating the soil mass and sometimes followed by rapid drawdown (e.g. Hubble and Hull, 1996, 2004), rather than direct infiltration of rainfall followed by groundwater flow. The typical riverbank failure mechanisms observed in the SKM (2009) report are deep rotational failure followed by slab failures. These types of failures also suggest that the infinite slope method is inadequate for the analysis of riverbank stability. For these reasons, it is recommended that the modelling of groundwater flow in the riverbank stability analysis in Phases 3 and 4 of this study will be undertaken using more sophisticated techniques, such as the finite element (FE) method.

There are a number of commercial FE packages available which model seepage in soil and groundwater flow. For example, *SVFlux* is a versatile 1D, 2D, axisymmetric and 3D FE program for modelling saturated and unsaturated groundwater flow with climatic coupling. The historical rainfall and river level data can be employed to study the change soil moisture, groundwater level, seepage flow due to seasonal fluctuations, as well as the suction (negative porewater pressure) change due to changes in soil moisture. The seepage porewater pressures obtained from *SVFlux* can subsequently be incorporated into *SVSlope* – a 2D/3D slope stability program based on limit equilibrium analysis and the FE method. Data from boreholes, CPTs and piezometers may also be used to build a sophisticated groundwater models.

As mentioned above, lowering of the river level in the Lower Murray increases seepage flow pressures towards the river due to an increase in head difference, particularly where there are lagoons adjacent to the sites. The amount of increased seepage flow also depends on the soil's permeability. Field investigation data obtained by SKM (2010) showed the riverbanks along the Lower Murray are comprised of Silty Clays, Silty/Clayey Sands, Silty/Clayey Gravels and Fills. Typical soil profiles (at sites such as Riverfont Road, Murray Bridge, Caloote and Woodlane Reserve) are comprised of a layer of Silty Clay that is underlain by a layer of Silty/Clayey Sands or Clayey Sand/Sandy Clay. A layer of engineering fill (comprising mainly Silty/Clayey Sands) can be found at the ground surface of reclaimed sites. The Silty Clay layer was typically encountered at depths of 1 m to 2.5 m below ground level, and is described as very soft and wet with a moisture content close to the liquid limit. This normally consolidated Silty Clay layer is highly impermeable but does contain some permeable sand lenses and is highly expansive.

Fluctuations in groundwater level can have significant effects on expansive clays. A highly expansive, or reactive, soil means that shrinkage cracks can develop at the surface and extend to the depth of the water table due to drying as the result of increased surface temperatures and evaporation at depth. The seasonal moisture zone in the River Murray area is approximately 4 m in depth, so when the groundwater table drops, shrinkage cracks follow. These shrinkage cracks can subsequently fill with surface water which can initiate failure from the crest as a result of rainfall. Tree roots also exacerbate shrinkage in reactive soils. The FE method can be used to model the variation of saturated and unsaturated groundwater flow due to rainfall, groundwater, river level fluctuation, vegetation, suction and other factors. Furthermore, the role of progressive failure and possible effects due to changes in geochemistry (pH, salinity) of porewater, which can be significant in clayey soils, can also be examined using this method.

7.4 Failure processes

7.4.1 Introduction

There are different types of slope failure processes, which are controlled and influenced by a variety of factors. For instance, shallow landslides of soil slopes and deep landslides of rock slopes are controlled by different physical subsurface materials, just as planar failure and rotational failure are controlled by different slide mechanisms. For this reason, categorising failure processes is essential prior to landslide susceptibility assessment and hazard mapping. Among numerous criteria for categorising failure processes suggested by various researchers, the categories proposed by Varnes et al. (1984), Hutchinson (1988) and the Working Party commissioned by the International Consortium on Landslides (ICL) for World Landslide Inventory (Sassa, 2004) are the most relevant and internationally recognised.

Bank erosion problems are rarely the result of a single process or mechanism, but rather are usually the result of complex interactions between a number of processes and mechanisms that may operate on the bank either simultaneously or sequentially (Thorne et al. 1996). These can be grouped into three broad categories:

1) **Erosion processes** which detach, entrain and transport individual particles or assemblages of particles away from the toe of the face of the retreating bank;
2) **Failure mechanisms** which lead to collapse of all or part of the bank; and
3) **Weakening processes** which operate on and within the bank to increase its erodability and, hence, to reduce its geotechnical stability.

The following sections deal in turn with erosion processes, failure mechanisms and processes of weakening, and consider the role that each play in accounting for the problems of bank erosion along the Lower River Murray (SKM, 2009).

7.4.2 Erosion processes

Seven categories of bank erosion are recognised in the literature (Thorne et al. 1996):

1) **Parallel flow (fluvial entrainment)** – Sediment is detached and carried away by flow parallel to the bank;

2) **Impinging flow (fluvial entrainment)** – Sediment is carried by flow striking the bank at an angle to the long-stream direction;

3) **Boatwash** – Sediment is carried away by waves and currents generated by passing boats;

4) **Wind-waves** – Sediment is carried away by waves and currents generated by the wind;

5) **Rills and gullies** – Banks are eroded by concentrated surface runoff draining across the bank line;

6) **Piping** – Subsurface erosion occurs by water draining through the bank; and

7) **Freeze/thaw** – Particles and aggregates are loosened by freezing and fall off the bank face during flow or boat wash.

SKM (2009) suggested that effects of flow on the erosion of the bankline (fluvial entrainment) is low as a result of the regulated nature of the Lower Murray. The channel is generally characterised by a low energy flow regime with low shear stresses and cohesive clay banks. Under these circumstances, the potential for fluvial entrainment is limited. SKM (2009) also concluded that there was no field evidence for scouring and undercutting. However, SKM (2009) identified the fluctuations in water level as a result of weir operations, boat wash and wind-waves do appear to be effective in washing away imported sand material from the channel margins. The removal of sand material at artificial beaches and exposure of the underlying clays was noted at a number of locations. SKM (2009) concluded that bankline retreat at sites inspected along the Lower Murray is due mainly to bank slumping into the river. Nevertheless, the possibility of slopes destabilised due to slope undercutting caused by wind-waves and boat wash during low water level periods, as well as the erodibility of silt clays and clayey sands, will be examined in this project.

7.4.3 Failure mechanisms

Seven categories of mechanism responsible for bank collapse can be identified (Thorne et al. 1996):

1) **Shallow slide** – Shallow seated failure along a shear plane parallel to and just below the bank surface, typically occurs in weakly cohesive soils, as depicted in Figure 7.6(a) is a shallow failure and (b) a planar failure;

2) **Slab failure** – Blocks or columns of soil topple forward into the channel, often with deep tension cracks separating the failure blocks from the intact bank. This represents a severe type of failure involving the movement of large volumes of material and serious bank line retreat, as shown in Figure 7.6(c);

3) **Rotational slip** – This is a deep-seated movement of all or part of the bank profile in which a block of soil slips along a curved surface. Similar to slab failures, this is a severe type of failure that involves the movement of a large volume of material and generates serious bank line retreat. Depicted in Figure 7.6(d) is a rotational failure in homogenous material, (e) a rotational failure with a weak zone, and (f) a massive rotational failure/landslide;

4) **Cantilever failure** – Overhanging blocks of soil collapse into the channel by shear, beam or tensile failure. Overhangs are found in layered banks where a resistant, cohesive or root bound layer overlies an erodible, non-cohesive layer. Shown in Figure 7.6(g) is a failure of a composite bank (in tension) and (h) failure of a composite bank (as beam);

5) **Soil fall** – Soil falls directly into the channel from a near-vertical or undermined, cohesive bank face. This frequently follows weakening by desiccation, saturation or frost action on a non-vegetated surface;

6) **Dry granular flow** – Avalanching of dry, granular bank material down the upper part of a non cohesive bank. When it occurs in a lower bank, this can cause instability of the upper bank resulting in bank line retreat; and

7) **Wet earth flow** – Liquefaction and flow of a section of bank due to saturation and high porewater pressures. This can result in rapid bank line retreat in zones of strong seepage and poor drainage.

Of these bank failure modes, deep rotational slips and slab failures appeared to be the main mechanisms causing the large failures and retreat of bank lines at sites inspected along the Lower Murray. These two modes of failure can also occur together and sequentially, with a large rotational slip forming a steep face which then continues to retreat through slab failures. These types of bank failure modes appear to have occurred at Long Island Marina and Woodlane Reserve, and they represent the most severe form of bank failure. They represent a serious form of instability, which is deep and below the riparian vegetation root zone. Significant engineering intervention through re-profiling and improved drainage to increase bank stability will be necessary to mitigate bank line retreat (SKM, 2009).

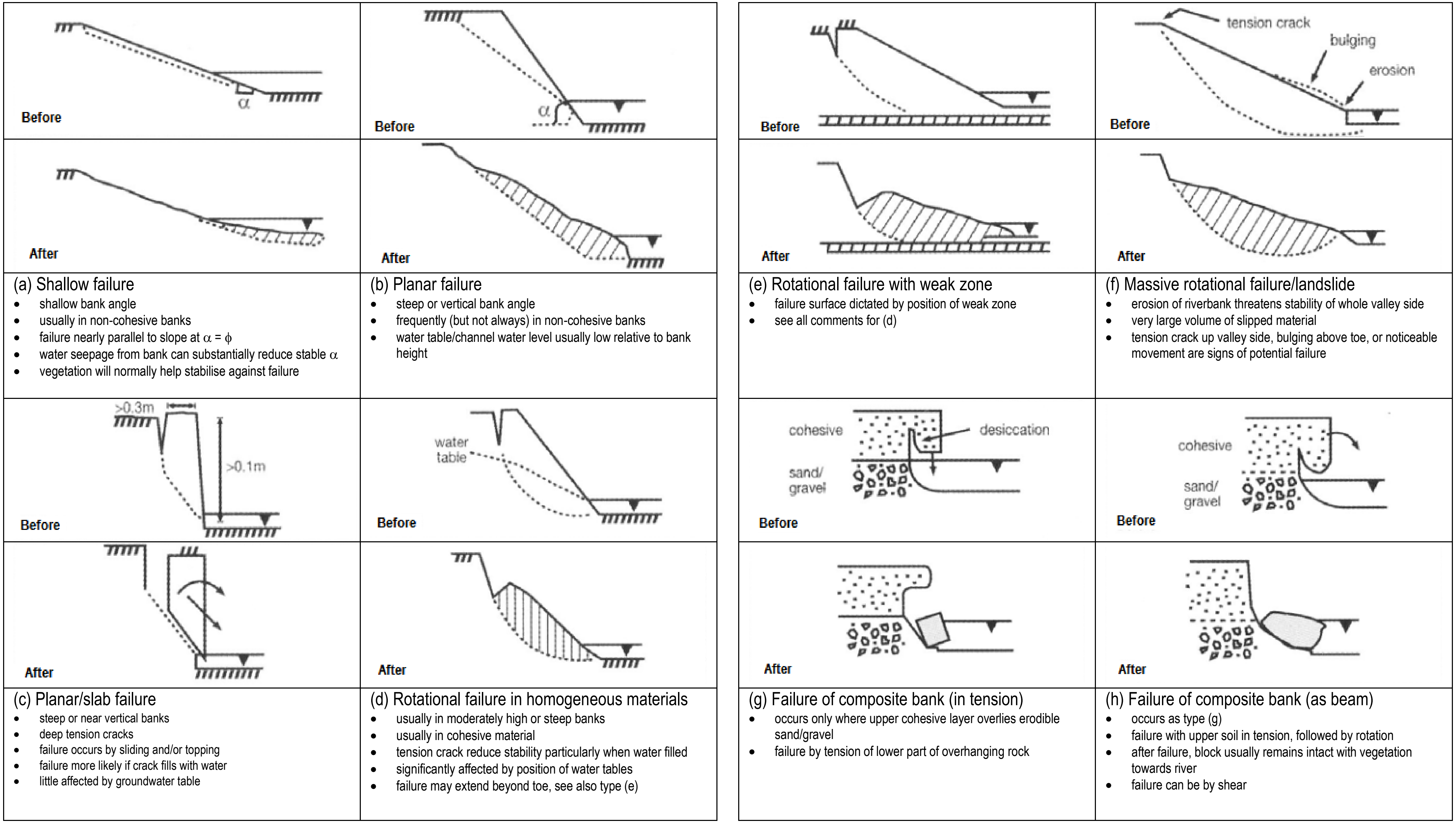


Figure 7.6 Bank failure modes (Source: Hey et al., 1991)

7.4.4 Weakening factors

Six categories of factors responsible for decreasing the erosion resistance and mechanical stability of a riverbank have been broadly identified in the literature (Thorne et al. 1996):

1) **Leaching** – Leads to a weakening of the bank through a reduction in cohesion that occurs when minerals are removed by groundwater percolating through the bank. The removal of minerals and change in pH level may induce progressive failure at the micro-level in clayey platelets, forming ‘face to face’ bonds rather than ‘edge to face’. This slow and long process changes the geochemistry (pH, salinity) of the porewater, which can be significant in clayey soils, will be explored in this project.
2) **Trampling** – Destruction of soil fabric by crushing under the weight of pedestrians or grazing animals.
3) **Destruction of riparian vegetation** – Damage or destruction of riparian vegetation by a variety of natural processes and human actions.
4) **Mechanical damage** – Damage of banks by boat mooring, stock access or angling.
5) **Positive porewater pressures** – Occurs when drainage of water through the bank is restricted resulting in a build-up of porewater pressures. This reduces the effective strength of the bank material, weakens the bank and increases the probability of block failure or, in extreme cases, leads to liquefaction and wet earth flow.
6) **Desiccation** – Cracking and crumbling of soil due to intense drying that breaks electrochemical bonds and loosens soil peds on the exposed bank surface during hot and dry summers.

In the case of the Lower River Murray, a number of additional factors are noted which are considered significant in decreasing the erosion resistance and mechanical stability of the riverbanks:

1) **Construction adjacent to banks** – Construction of infrastructure adjacent to the riverbank, such as jetties, roads and dwellings, increases the imposed load, which increases the likelihood of bank collapse and settlement.
2) **Fluctuation of water levels** – Changes in moisture status associated with fluctuating water levels can cause expansion and shrinking of clays and affects the porewater pressures within the banks.

This discussion has been limited by the available information and visual site inspection of a number of sites. Further geotechnical investigations are needed to quantify the role of relevant weakening factors, material properties and bank parameters. These additional investigations and testing will be undertaken as part of Task 3 of this Goyder Institute project.

7.5 Effects of vegetation root reinforcement

7.5.1 Background

Riparian vegetation significantly affects the hydrological and mechanical properties of riverbanks (Schwarz et al., 2010), and its presence has both beneficial and detrimental effects on bank stability. Traditionally, however, vegetation has been assumed to have only a minor effect on slope stability. For this reason, it is sometimes ignored by scholars and engineers in conventional *FS* analysis.

Incorporating the effects of vegetation in slope stability analysis was first attempted in the 1960s, although grass, shrubs and trees have been used to stabilise slopes for many years. Terzaghi (1950) treated deforestation as a highly plausible cause of a landslide that occurred in 1915 at Hudson, New York. Following his lead, a few researchers focussed on the effects of vegetation removal on stream bank stability using quantitative analysis, noting that after deforestation there was a significant increase in the frequency of landslides (Bethlahmy, 1962, Bishop and Stevens, 1964). These pioneering studies, among others, raised awareness of the importance of riparian vegetation on riverbanks, demonstrating that it not only provides ecological benefits, but also offers stabilisation of the riverbank slope, as shown in Figure 7.7 (Abernethy and Rutherfurd, 1998; Simon and Collison, 2002).

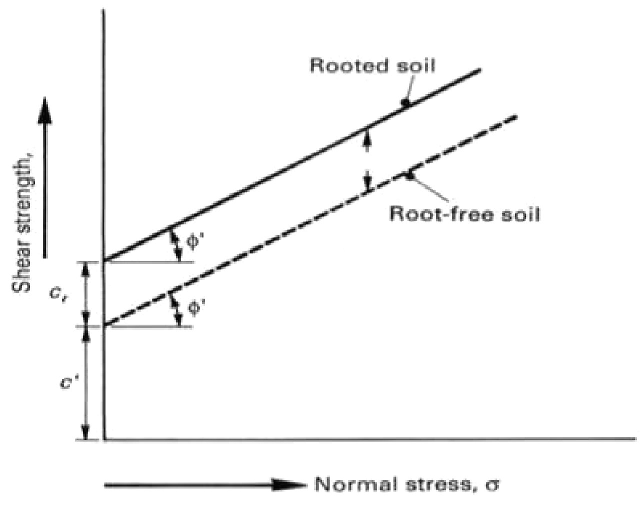


Figure 7.7 Effect of root reinforcement on shear strength of soil
(Source: Coppin and Richards, 1990)

7.5.2 Hydrological effects

The hydrological effects provided by vegetation refer to the change or modification of soil moisture content and porewater pressure caused by the hydrological cycle when woody or grassy species are present. The hydrological effects are beneficial. Firstly, the vegetation canopy intercepts precipitation thereby reducing the amount of rainfall, which infiltrates the slope. Secondly, the plant roots extract moisture from the soil, as shown in Figure 2.8, by means of transpiration. Both processes enhance soil shear strength due to a decrease in porewater pressure in saturated and semi-saturated soils or an increase in matric suction in unsaturated soils (Selby, 1993). Both the decrease of porewater pressure and increase of the matric suction raise the factor of safety (*FS*).

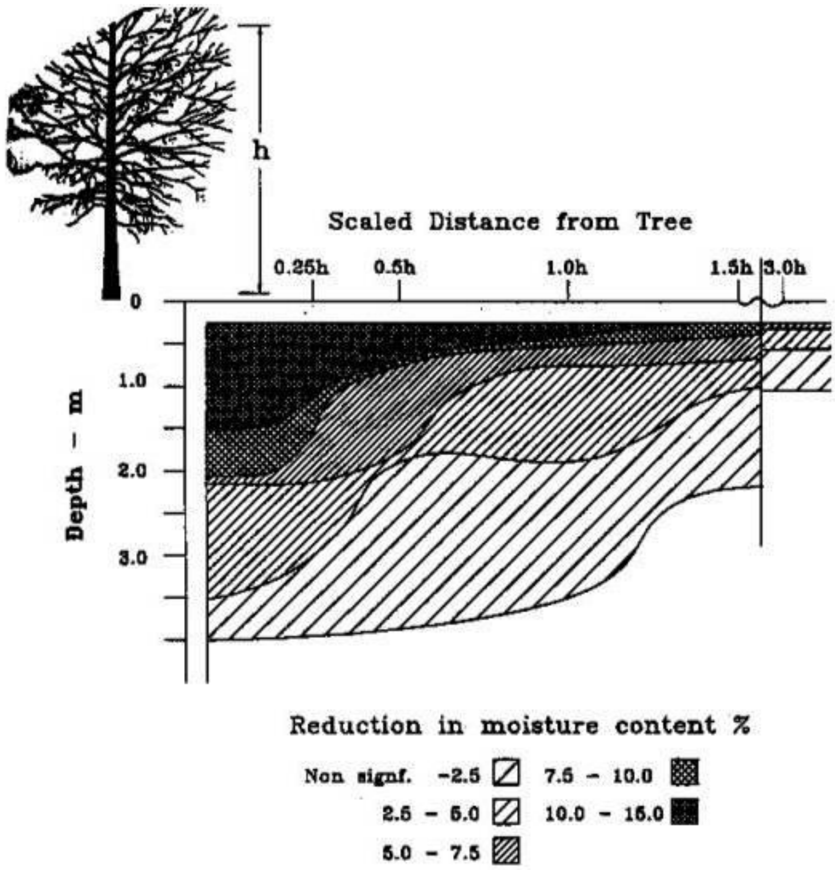


Figure 7.8 Reduction in soil moisture content near a Poplar tree growing in boulder clay (Source: Biddle, 1983)

On the other hand, vegetation can be detrimental to slope stability, due to certain soil infiltration characteristics and biological activities, which act not only on the soil surface, but also at depth. Canopy interception and stem flow tends to concentrate infiltration locally around the stem of the plants, causing higher local porewater pressures at the surface (Durocher, 1990). In addition, the increase of the infiltration rate capacity caused by deep-rooted systems and associated biological activity can accelerate the delivery of water at depth by creating preferential flow paths (Simon and Collison, 2002).

Recent research has shown that the impact of hydrological effects on riverbank stability depends on the types of vegetation and characteristics of the local rainfall. Compared to grassy species, woody species are more efficient in removing soil moisture and preventing rainfall from infiltrating into the soil (Simon and Collison, 2002). The hydrological effects are more significant in wet periods or areas.

7.5.3 Mechanical effects

The mechanical effects are caused by the physical interaction between the vegetation and the soil mass on or under the slope surface. Closely spaced root matrix systems, as shown in Figure 7.9, are able to increase the confining stress in the soil mass and provide reinforcement by transferring the shear stress in the soil to tensile resistance in the root system, as shown in Figure 7.10. Typically, mechanical effects are mostly beneficial because roots anchor themselves into the soil. As a result, the soil mass is bound together by the plant roots and the soil shear strength is increased because of the additional apparent cohesion (Coppin and Richards, 1990).

The detrimental impact of vegetation on bank slope stability is caused by the weight of the vegetation. The weight of large trees applies an additional surcharge to the slope, increasing both the down-slope forces and the confining stress of the soil at the potential slip surface. The locations of trees on the slope surface can have either adverse or beneficial effects on slope stability (Coppin and Richards, 1990). Generally speaking, trees which are located at the toe of a slope benefit slope stability by adding resistance and increasing the frictional component of soil shear strength. On the other hand, if trees are located at the top of slope, the additional load will increase the down-slope forces, thus destabilising the slope.

Furthermore, wind loads imposed on large tress can causes an increase in the driving force acting on the slope. The wind load is transmitted to the soil, becoming a driving force that ultimately reduces the factor of safety (Hsi and Nath, 1970; Brown and Sheu, 1975).

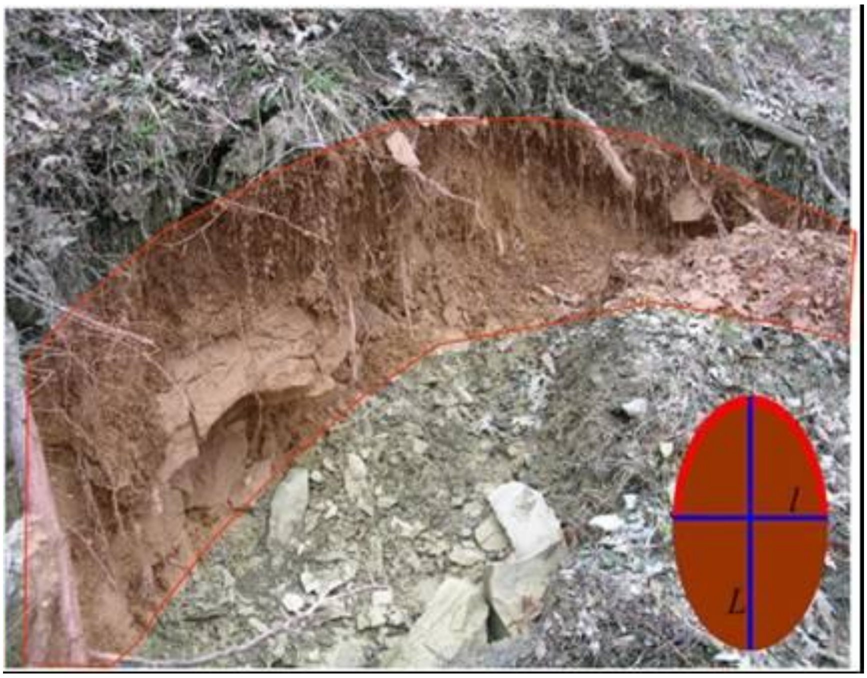


Figure 7.9 Illustration of the root matrix system of vegetation on riverbank (Source: Schwarz et al., 2010)

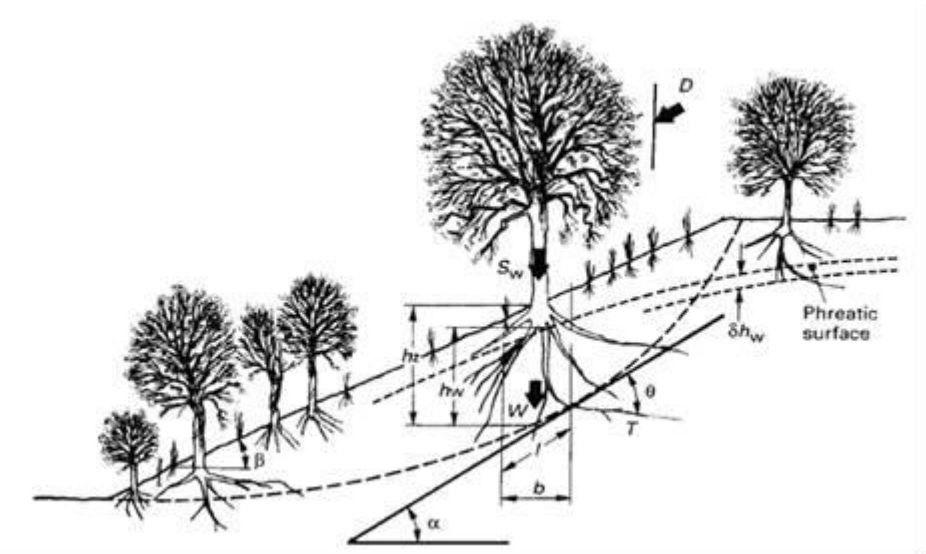


Figure 7.10 Influence of vegetation on riverbank (Source: Coppin and Richards, 1990)

Hubble et al. (2010) conducted an integrated review of field and experimental studies in eastern Australia to examine the role of native vegetation in the mass failure of riverbanks. They found that the presence of riparian forests on riverbanks of the upper Nepean River significantly reduces the likelihood of erosion by mass failure due to root reinforcement. It was also found that a number of Australian tree species have apparently evolved roots that seek the permanent, summer water table for survival in prolonged periods of drought, and these root systems are particularly effective in mass mitigation due to rooting depths that are greater than 5 m and are sometimes well in excess of 20 m. For the Lower Murray, however, the permanent water level is much shallower. The root system might not extend deep enough beyond the slip surface to mitigate riverbank failures, but act as an additional surcharge to the slope, thus destabilising the riverbank. Furthermore, wind loads imposed on large trees increase the likelihood of riverbank collapse, as explained earlier.

7.5.4 Reinforcement calculation

Several researchers have introduced and applied root reinforcement in their *FS* estimations. A growing number of models have therefore been developed for quantifying root reinforcement. The models often include the effects of root system density and the root branching, Young's modulus and tensile strength of the roots (Greenway, 1987).

Simple perpendicular root models were proposed to calculate the root reinforcement, mostly based on the Mohr-Coulomb equation (Endo and Tsuruta, 1969; Waldron, 1977; Wu et al., 1979). In the following equation, soil shear strength is calculated from both the cohesive and frictional stresses:

$$s = c + \Delta s + (\sigma_n - u)\tan\phi$$ **Equation 7.10**

where s represents the shear strength of the soil (kPa); c represents soil cohesion (kPa); σ_n represents normal stress (kPa); u represents porewater pressure (kPa); and ϕ represents the soil's internal angle of friction (°). Equation 7.10 was established based on the assumption that all roots extend vertically across a horizontal shearing zone, and act like laterally loaded piles; so tension is transferred to them as the soil is sheared.

In this model, a tangential component resisting shear and a normal component increasing the confining pressure on the shear plane was proposed by Waldron (1977). The change in shear strength, Δs, is expressed as:

$$\Delta s = T_r(\sin\theta + \cos\theta\tan\Phi)(A_R / A)$$ **Equation 7.11**

where T_r is average tensile strength of roots per unit area of soil (kPa); A_R / A is the root area ratio or the cross sectional area of roots crossing a plane within soil. The parameter θ represents the angle of shear distortion in the shear zone (°), as shown in Figure 2.11.

Previous field- and laboratory-based research has shown that the angle of shear distortion, θ, is generally within the range from 40° to 70° (Gray and Leiser, 1982). Sensitivity analyses undertaken by Wu et al. (1979) showed that the term in Equation 7.11, $\sin\theta + \cos\theta \tan\Phi$, is somewhat insensitive to normal fluctuations in θ and Φ, as it varies from 40° to 90° and 25° to 40°, respectively. The values for the first term varies from 1.0 to 1.3, therefore, Wu et al. (1979) proposed a coefficient of 1.2 as a replacement for $\sin\theta + \cos\theta \tan\Phi$ term, and the equation is then simplified as:

$$\Delta s = 1.2 T_r (A_R / A)$$ **Equation 7.12**

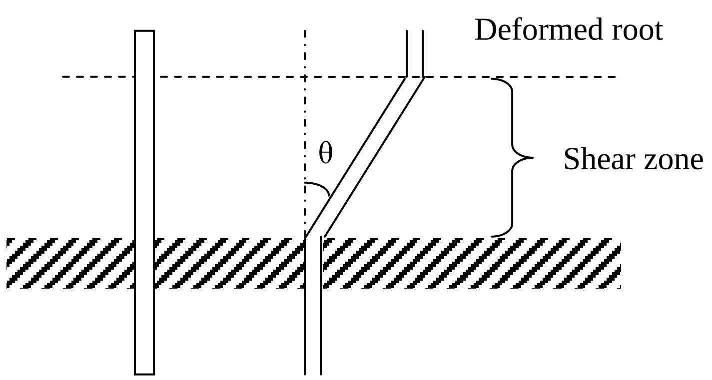


Figure 2.11 Angle of angle of shear distortion in the shear zone.

Recent research (e.g., Thomas and Bankhead, 2010) suggested that Wu et al.'s (1979) coefficient of 1.2 is inaccurate because, if root orientation is allowed to vary between 0° and 180° and both θ and Φ remain in the same ranges, the results of $\sin\theta + \cos\theta \tan\Phi$ vary from 0.69 to 1.22, and 0.97 to 1.39, respectively (Robert and Natasha, 2010). The field testing carried out by Docker and Hubble (2008) to study the increased shear resistance of soil due to root-reinforcement by four common Australian riparian trees, *Casuarina glauca*, *Eucalyptus ampliforia*, *Eucalyptus elata* and *Acacia floribunda* with a large-scale shear box, suggested that the tree root failed progressively rather than simultaneously, which is proposed by Wu et al. (1979). Docker and Hubble (2008) also showed that the calculated shear strength of the root-reinforced soil, assuming simultaneous root failure, yielded values 50% and 215% higher than directly measured shear strengths. The shear stress versus displacement plots for the four aforementioned tree species and soil only tests by Docker and Hubble (2008) is presented in Figure 2.12.

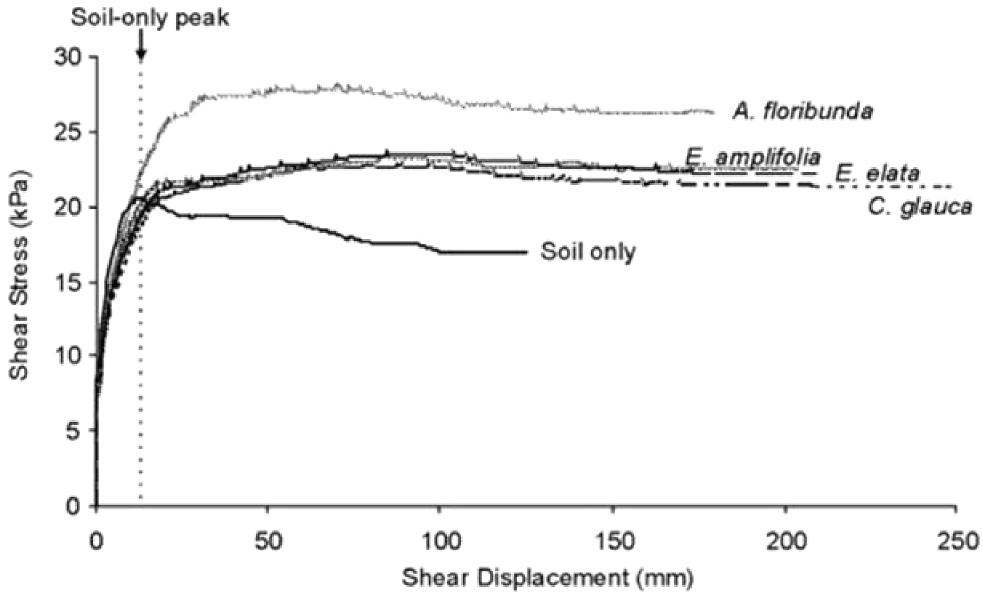


Figure 2.12 Average shear stress versus displacement plots for the four tree species and the soil-only tests. (Source: Docker and Hubble, 2008)

To estimate better the increase of shear resistance of soil due to root-reinforcement, a method known as fibre-bundle models (FBMs) were introduced to overcome the overestimation introduced by Wu et al.'s (1979) equation. FBMs aid in the understanding of composite materials (Daniels, 1945). The models use a dynamic approach to remove the assumption made in the Wu et al. (1979) model that all of the roots in the soil matrix rupture simultaneously. When a load is applied to the bundle of fibres it is apportioned equally between all intact fibres (Daniels, 1945). The maximum load that can be supported by the bundle corresponds not to the weakest or strongest fibre, but to one of the fibres in the centre of the bundle (Robert and Natasha, 2010). FBMs conform to the following rules:

- An initial load is added to the bundle which contains a number, *n*, of fibres and the fibres are assumed to be parallel to one another).
- Although at first the load is distributed equally among the *n* fibres, and hence divided into *n* parts, once the load is increased sufficiently for a fibre to break (conceptual research assumes one, but there may be more in practice), the load that was previously carried by the broken fibre is redistributed to the remaining (*n*–1) intact roots.
- Each of the remaining fibres then bears a larger share of the load than before and is hence more likely to rupture.

- If this redistribution of load causes further roots to rupture, further redistribution of the load occurs (this is known as an avalanche effect in this type of model) until no further breakages occur (if no bundle of fibres is left, the analysis terminates).
- The load on the system is then increased, and the process is repeated until either all of the fibres have been broken, or the maximum driving force acting on the matrix is supported by the fibres contained within it (Robert and Natasha, 2010).

FBMs provide a means by which to identify the root reinforcement characteristics and quantify the restraining shear strength. Recent research also indicates that both of the different mechanisms of load redistribution – global load sharing (GLS) and local load sharing (LLS) (Hidalgo et al., 2002) – as well as different species of roots, different root diameters and soil saturation indices, can also affect the root reinforcement process.

7.6 GIS approach on landslide hazard mapping

Landslide hazard mapping is an essential part of landslide susceptibility evaluation. The landslide hazard map includes the predicted landslide locations, dimensions and failure types, and depicts the levels of potential slope failures with its spatial distribution. In its early stages of development, landslide hazard mapping was typically based on topographic relief maps. However, with the advent of remote sensing (RS) and global positioning systems (GPS), and combined with geographical information systems (GIS), such approaches have become mainstream in landslide hazard mapping (Varnes and The International Association of Engineering Geology Commission on Landslides and other Mass Movements, 1984; Hansen, 1984; van Westen, 1994; Bonham-Carter, 1994; Carrara et al., 1995; Hutchinson, 1995; Soeters and van Westen, 1996; van Westen et al., 1997; Aleotti and Chowdhury, 1999; Guzzetti et al., 1999; Gorsevski et al., 2003). However, such approaches are restricted by a number of limitations, such as the different types of failure, limited input data resolution and different methods of GIS interpretation.

Generally, GIS-based landslide susceptibility evaluation and hazard mapping can be classified into two major categories: qualitative and quantitative. Qualitative evaluation is typically based on the evaluation scores or ranks given by geologists and geomorphologists. This method was popular and widely used in 1970s. It uses several maps representing the spatial distribution of those physical parameters which may influence the occurrence of landslides.

The spatial distribution of the factors indentified to be important in assessing slope instability are combined into a hazard map using subjective decision-making rules based on the experience of geoscientists (Anbalagan, 1992; Pachauri and Pant, 1992; Sarkar et al., 1995; Anbalagan and Singh, 1996).

More recently, with the development of GIS technology and improved computing performance, quantitative methods have become more widely used. Generally, deterministic, statistical and mathematical methods, which are also called distribution free methods, are the three main types of the quantitative methods.

A GIS-based statistical model can account for landslide susceptibility evaluation on a wide range of scales (i.e. from the small scale [<1:200,000]; medium scale [1:25,000 – 1:200,000]; to the large scale [>1:25,000]). Those methods can be divided into bivariate methods, such as the information value method and weight elements method, and multivariate methods, such as logistic regression (Carrara et al., 1991; Mark and Ellen, 1995; Rowbotham and Dudycha, 1998). These techniques involve the statistical determination of the combinations of physical parameters that have led to past landslides.

Quantitative or semi-quantitative estimates were made for areas along the river currently with no recorded landslides, but conditions exist similar to those with recorded land movement. Both multiple regression and discriminant analyses have been undertaken to explore relationships between landslide occurrence and terrain variables (e.g. Yin and Yan, 1988; Carrara et al., 1991, 1995; Brunori et al., 1996). Statistically-based research has also been carried out, notably by Skirikar et al. (1998), Dhakal et al. (1999) and Pathak and Nilsen (2004). The disadvantage is the statistical results are quite sensitive to the quality of the input data.

More recently GIS-based deterministic models have been developed and have become increasingly popular. These new models concentrate mainly on the development of software routines within GIS applications that are able to perform slope stability analysis. Among these computer programs, the combination of physical slope stability models and hydrological distribution models are the most popular and widely used. The most common are:

- *SHALSTAB* (Montgomery and Dietrich, 1994);
- *TRIGRS* provided by United States Geological Survey, USGS (Baum et al. 2008);
- *SINMAP* (Pack et al., 2005);

- *SMORPH* [an acronym for a semi-empirical method: Slope MORPHology from Shaw and Vaugeois (1999)]
- *dSLAM* [an acronym for distributed Shallow Landslide Analysis Model from Sidle and Wu (1999)]
- *FLO-2D* [an acronym for 2-dimensional Flood Routing Model (FLO-2D Software Inc., 2009)].

Mathematical methods consist of GIS-based artificial intelligence methods, such as artificial neural networks (ANNs); support vector machines (SVMs) and fuzzy sets. These methods require numerous data transformations and calculations and develop predictions based on the learning of patterns from data sets (Thapa and Dhital, 2000; Dhital, 2000; Saha et al., 2002; Sarkar and Kanungo, 2004). These methods are realistic and objective but care must be exercised when using these methods, as it is difficult for the user to appreciate the nature of the internal representations generated by these methods when the number of variables is large.

8 Knowledge Gap and Research Aims

8.1 Knowledge gaps

- Idealised bank profile geometries are typically used as a hypothesis in conventional quantitative, 2D bank stability studies. More accurate geometries are needed, which will lead to more accurate results.
- Compared with the classical 2D slope stability research, hazard mapping is undertaken on a regional scale and incorporates detailed examination of specific sites.
- Riverbank slope stability is a multifaceted issue. Compared with the slopes in mountainous regions, riverbanks are greatly influenced by river water level fluctuation, climatic factors, river flow and surface waves. In particular, more focused research is needed at a regional scale examining riverbank instability triggered by water level fluctuation and climatic influences.
- It has been demonstrated in recent research that riparian vegetation and human infrastructure can increase soil strength and vertical loads, respectively. The interaction between these factors and those listed above requires further investigation, both at a local and a global level.
- The lack of site specific data, such as river geometry, soil properties and their variability, land use, geology and the groundwater regime, are the major challenges in this investigation. In particular, additional geotechnical data are needed to provide more reliable riverbank stability assessment at the regional scale.

8.2 Research aims

The project will address key knowledge gaps aimed at obtaining the necessary information to answer key decisions in the future management of the riverbank collapse hazard along the Lower River Murray. These include:

1. What are the failure mechanisms driving riverbank collapse events;
2. What are the potential triggers for future riverbank collapse events that should be monitored and managed;
3. What is the safe operating range of the river to minimise the impacts of river level fluctuations on riverbank stability;

4. What are potential long-term sustainable management options for key high risk riverbank collapse affected sites; and
5. What areas along the Lower River Murray are likely to be more susceptible to riverbank collapse events.

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Part 2

Geological and Geomorphic

Professor Tom Hubble and Ms. Elyssa De Carli

10 Introduction – Geological and Geomorpthic

Riverbank failures are a common, expected and well-studied phenomenon on a world scale and within Australia's river systems (Brizga & Finlayson, 1999; Schumm, 2005; Hubble et al., 2010). Rivers are complex and dynamic systems which transport water and sediment from their upland source catchments, through a transfer zone to a downstream sink. River behaviour changes through time, with the morphology and character of a river at a particular time and place being a consequence of complex interactions between hydrology, sediment load, geologic setting and geomorphic history. The onset of riverbank collapses on the lower Murray River towards the end of the Millennium Drought (1997-2009), while unusual, can be regarded to be an example of the variety of complex river behaviours.

The widespread and frequent occurrence of bank collapses of the Lower Murray River's banks between 2009 and 2011 presented an apparently new problem for, and significant risk to, private landholders, public infrastructure and river users. The aim of this brief report is to summarize the several specific investigations of riverbank failure that occurred during this time; to place these investigations in their broader geological and geomorphic context and to identify the knowledge gaps in the available information. This process will focus subsequent future work on the riverbank collapse problem and identify key questions required to enable a better understanding of why these bank failures developed.

Examination and direct investigations into the phenomenon of riverbank collapse on the Lower Murray River can be divided into two main types: a) geotechnical investigations by civil engineers and engineering geologists and b) site mapping studies by professional surveyors using traditional instrumental methods and multibeam echo-sounders. This information is contextualized by a third set of data: c) regional geological and geomorphological studies which detail the landscape history that determined the present-day nature of the Lower Murray's channel and floodplain materials.

It is important to note, that despite the Murray being the most studied river system in Australia the number of studies that address the geologic and geomorphological history of the Lower Murray River are relatively few in number and the majority of those that are available deal with either the development of Riverine Plain upstream of Cadell; or the coastal beach and dune systems downstream of the Murray's present-day terminal lakes of Lake Alexandrina and Lake

Albert. Geotechnical reports have been generated since 2009 in response to the recent riverbank collapse problem.

This report will summarize this information, place the occurrence of bank failure into a geological and geomorphic context, and identify the key questions or knowledge gaps for investigation in the next stage of this Goyder Institute Research Project: E.1.8 Riverbank Collapse in the Lower River Murray.

11 Summary of Geotechnical Investigations

Towards the end of the Millennium Drought (1997-2009), Murray-Darling Basin inflows dropped significantly from a normal pool level of around 0.75 AHD to a 2008-09 level of around –1.05m AHD, which was the lowest recorded river level since the keeping of instrumental records began soon after European settlement of the area. In November 2008 The Department of Water, Land & Biodiversity Conservation (DWLBC) commissioned Arup to assess soil cracking and slumping on riverbanks, levees and control structures on the Lower River Murray as a reactive response to reports of instability (Arup, 2008a, b).

Sites assessed in these investigations were found to show signs of cracking, erosion, and minor slumping, and it was concluded at this time that any further lowering of the river level would be likely to trigger further slumping, erosion and cracking due to the rapid drawdown effect (*cf* Morgenstern & Price, 1965). River levels continued to fall, and in early February 2009 major collapses occurred at numerous sites in the vicinity of Murray Bridge; with the most severe occurring at Long Island Marina when a 20 metre by 6 metre section of riverbank slid into the river without warning about 3 pm on a Wednesday afternoon. Three cars and several river red gums were transported and submerged in the middle of the river by this event, and at least one car is still to be located and recovered from the river. The condition of the site continued to deteriorate, with two more major collapse events occurring in early March 2009.

Due to the progressive deterioration of riverbanks and an increasing prevalence of similar riverbank collapse events during the lowering of the river pool levels between 2009 to 2011 (Figures 2.1 and 2.2), a program of further works investigating this phenomenon was commissioned by Department of Environment, Water and Natural Resources (DEWNR) and in February 2010 Sinclair Knight and Merz (SKM) released an inspection (SKM, 2010a) and geotechnical report (SKM, 2010b). These reports provided an assessment of the lower 209 km of the Murray between Blanchetown (Lock 1) and Lake Alexandrina. SKM examined numerous reported failures, and focused on several key sites where detailed site investigations were undertaken, collecting cone penetrometer test data (CPT), piezometer and borehole sample data.

Slope stability modelling was undertaken using this data by SKM in 2010 and in a subsequent work by Coffey Geotechnics Pty Ltd (Coffey) in 2012 for numerous failure sites on the Lower Murray. Similar results and interpretations were developed for all the sites investigated in this manner and results for Woodlane Reserve are presented and discussed in this synopsis as it is considered to be a representative example of a typical Millennium Drought Lower Murray River Bank Failure (SKM, 2010b; Coffey, 2012: Examples for Woodlane Reserve given in Figure 2.3).

The SKM and Coffey analysis works indicate that their slope stability modelling results return factors of safety (*FS*) values consistent with the actual failures that occurred, and in the case of the Coffey (2012) report the modelling results are unequivocally indicative of slope failure during lowered river pool-levels. SKM’s (2010b) results of analysis returned *FS* below 1.5 for a river pool-level of –0.8 m AHD, and below 1.2 for a river pool-level of –1.5 m AHD. Coffey (2012) results for similar cases returned values well below 1.0 when lower Murray River pool levels were between –0.5 to –0.8 AHD. The differences in *FS* values for these similar cases can be ascribed to the lower values of cohesion assigned to the soft clays in the Coffey (2012) analysis. The interpretations and findings of the two reports are entirely consistent with each other.

The Factor of Safety values determined in the modelling undertaken by SKM (2010b) were interpreted as indicative of the actual failures that occurred at these sites. Both the SKM (2010b) and Coffey (2012) reports inferred that the historically low river levels brought about by severe drought conditions were a probable precursor to riverbank instability, and were a contributing factor to the reported riverbank collapses. A common characteristic of the vast majority of the modelling undertaken for both these reports is that the minimum slip circles, i.e. the most likely failure trajectories, are relatively deep seated failures that crest or daylight approximately 10 m inland of the normal pool-level water line. These failure masses tend to consist of a relatively large block of material which generally includes the entire channel margin slope and is commonly 5 – 10 m thick.

The following quote, taken from the executive summary of Coffey (2012), captures the considered opinion on this issue:

“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."

Subsequent investigations have included ongoing investigation at specific high-risk sites by the Universities of Adelaide and Sydney, as well as ground-motion monitoring by Alexander & Symonds in the form of repeat surveys. The site-survey monitoring has found evidence of movement and subsidence of particular sites, despite the return of normal river flows from the catchment and the re-establishment of the river pool-level to its long-term norm. For example, the March 2013 surveys at East Front Rd., Younghusband and Murray View Estates, Tailem Bend have demonstrated motion and maintenance of fresh bank-cracks on the channel margins (Alexander & Symmonds, 2013).

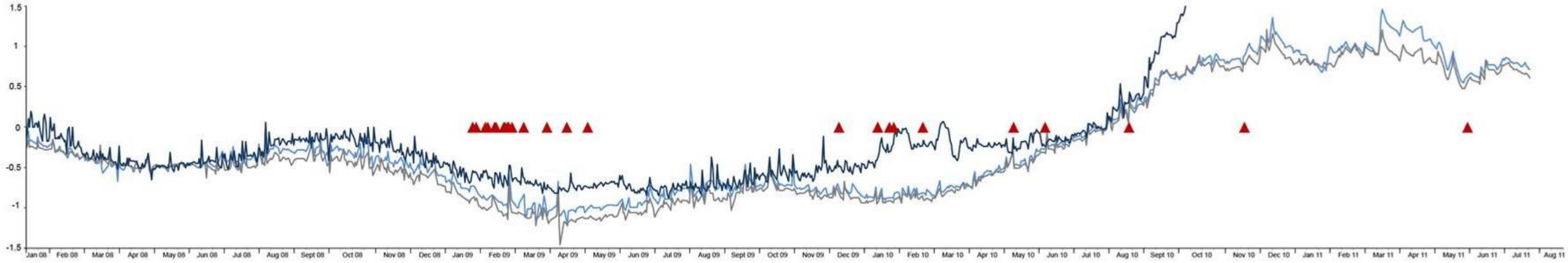

Figure 11.1 Lower Murray River Pool Levels (AHD) at Murray Bridge (grey), Mannum (light blue) and Blanchetown Lock 1 (dark blue) between December 2007 and December 2011 in relation to reported riverbank collapse incidents (red triangle) in the DEWNR Incident Register.

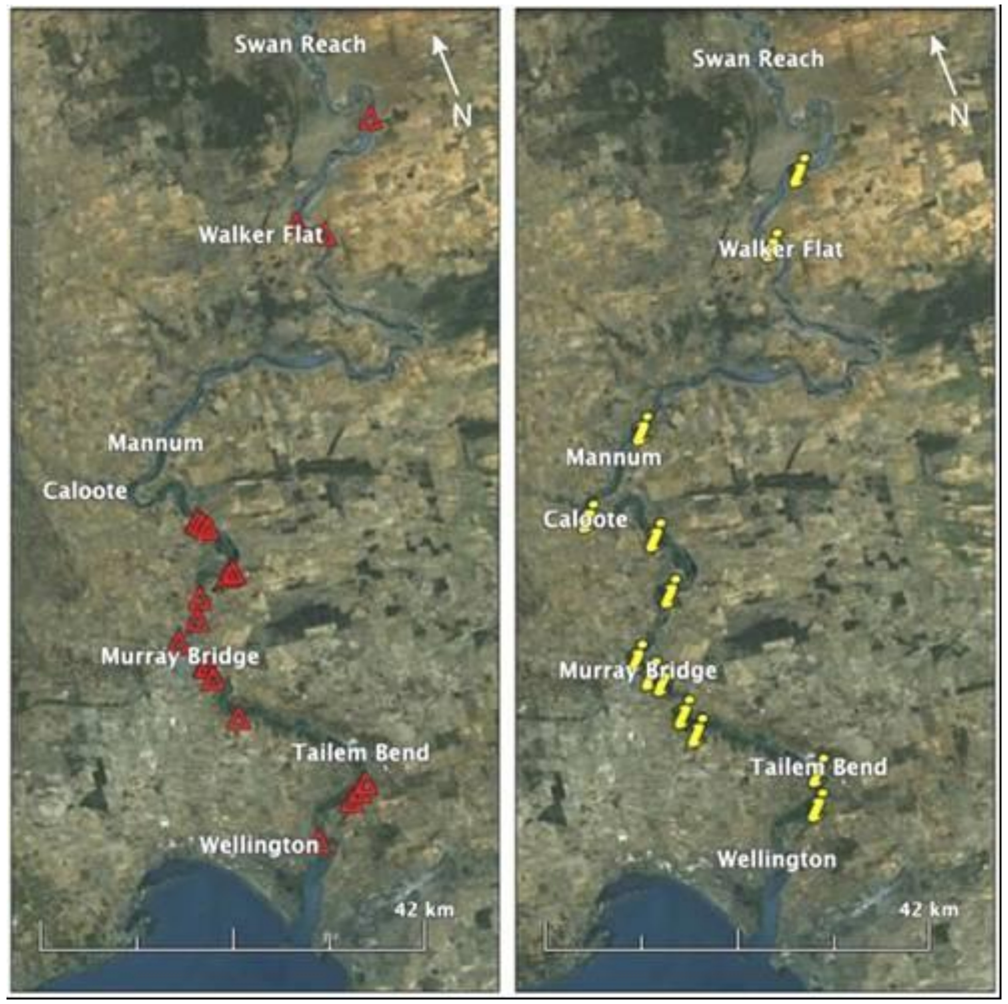

Figure 11.2 Reported bank failure sites 2009-2011 (red triangles) and location of Multibeam Bathymetry Surveys (bold yellow i's) undertaken by Gareth Carpenter for DEWNR on the Lower Murray River.

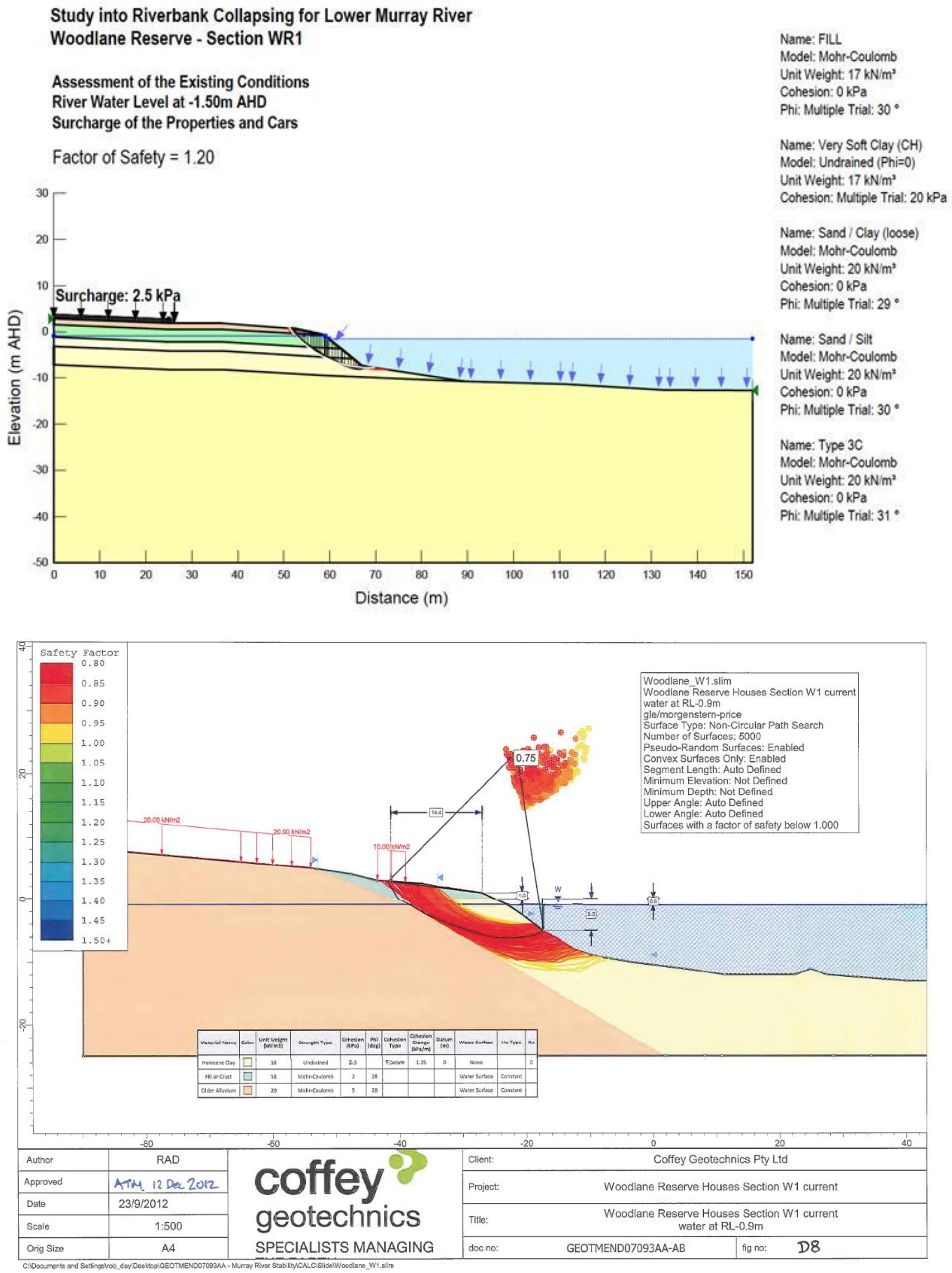

Figure 11.3 Examples of slope stability models for Woodlane Reserve presented in SKM (2010b) (*upper diagram*) and Coffey (2012) (*lower diagram*).

12 Geological and Geomorphic Context

Several statements presented in a number of major standard reference works on riverine processes have become guiding principles of river management manuals. These indicate that there is a fundamental need to characterise the geologic and geomorphic history of a particular river in order to fully understand the particular changes that may be occurring within that river's channel, banks or floodplain (*cf* Brierley & Fryirs, 2005). It is apparent that two verities from this literature are particularly pertinent to understanding the riverbank stability problem on the Lower River Murray. They are:

- Rivers change naturally through time as a result of hydrologic, climatic and geological change (Schumm, 2005); and
- Channel morphology on a particular river can vary considerably in response to geologic and geomorphic controls (Schumm et al., 1972).

Rivers are complex, non-stationary systems and changes in a river's planform and cross-sectional geometry are normal, thus river managers should expect events such as bank failure or erosion to occur on an ongoing basis rather than be surprised by individual instances of their occurrence. Understanding changes in a river's behaviour and managing them effectively requires that the changes of current concern be contextualized within their geomorphic and geologic context.

Examination of the available literature concerning the channel and floodplain sediments, and the regional geology and geologically recent evolution of the South Australian landscape indicates that both the bedrock gorges which the present-day river has excavated, and the sediments deposited within these gorges are surprisingly young features – surprising in that much of the Australian Landscape is perceived to be relatively old, and even ancient when compared to the landscapes of Europe, the Americas and Asia.

The geological information (Appendix B) indicates that the area of South Australia around the Lower Murray is being uplifted at a rate of 70 m per million years (Figure 3.1). This means that the layers of limestone rock exposed in the walls of the Murray's gorges and which underlie the low plateau into which the Murray River's Gorges are incised, are gradually rising up due to tilting of the entire Australian continent as the Australian Plate slowly collides with the Asian tectonic plate (Sandiford et al., 2009; Quigley et al., 2010 and related works).

The rate of continental uplift in South Australia while significant is still slow enough for the Murray River's waters to erode through and erode away the rock as it is uplifted. The erosion is enabled by the trapping or concentration of the entire downstream flow derived from the Murray-Darling catchment within a relatively confined space. A consequence of this process is the stripping and probable complete removal of consolidated sediment and some fresh rock during deepening and renewed gorge incision during the lowstands of sea-level that occur when the northern and southern hemisphere ice-sheets expand to their largest extents. These events are known as glacial maxima and they have occurred regularly during the last several million years or so and have presented in a well-understood regular 100,000 year cycle for approximately the last 600,000 (*cf* Imbrie, 1978; Imbrie et al., 1993; Ruddiman, 2003). Back-filling of the freshly incised gorges with river-delivered sediment occurs during sea-level rise when the global ice-pack contracts. These are known as highstand events, the most recent of which occurred between ~15,000 and ~7,000 years ago and is ultimately responsible for the deposition of the sediment deposits that fill the Murray Gorges about 60 m above bedrock at Murray Bridge, about 20 m about bedrock at Swan Reach and about 40 m above the bedrock gorge floor at Renmark. The top surface of these sediments forms the present-day floodplain surface or lagoon floors adjacent to the Murray's channel.

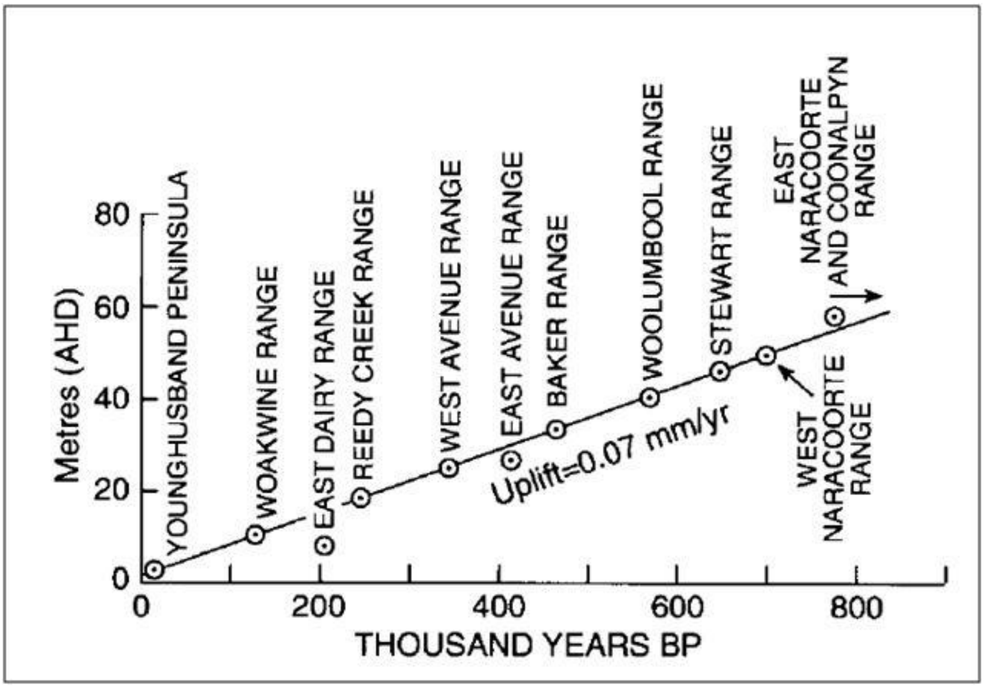

Figure 12.1 Uplift of the South Australian coastal zone determined from present elevation of former coastal barrier and dune systems located between Robe to Naracoorte. (From: Bourman et al., 2000) (Original Source: Belperio et al., 1995).

12.1 Latest Quaternary and Holocene events

The work of Thomson (1975), Twidale et al. (1978) and the results of several engineering site investigations (Fryar & Rowan, 1968; Steel, 1968) strongly suggest that the Lower Murray River Gorge/Valley was completely stripped of sediment infill during the last Glacial Maximum (~20,000 ybp) and subsequently backfilled firstly with the sands of the Monoman Formation, followed by the clays and muds of the Coonambidgal Formation (Figure 3.2). The Monoman Formation is commonly referred to as the 'lower valley-fill' and is comprised of coarse-grained, high-energy fluvial deposits and was laid down during a final post-glacial transgression. By ~7,000 years BP, as the sea approached its modern level around the South Australian coast (Belperio et al., 1983), depositional surfaces in the Murray River valley upstream of Renmark became more stable and forests similar to those present when European Settlers arrived were developed. The top of the Monoman Formation is marked by a buried forest and palaeosol (Gill, 1973) upstream of Renmark which marks a major change in

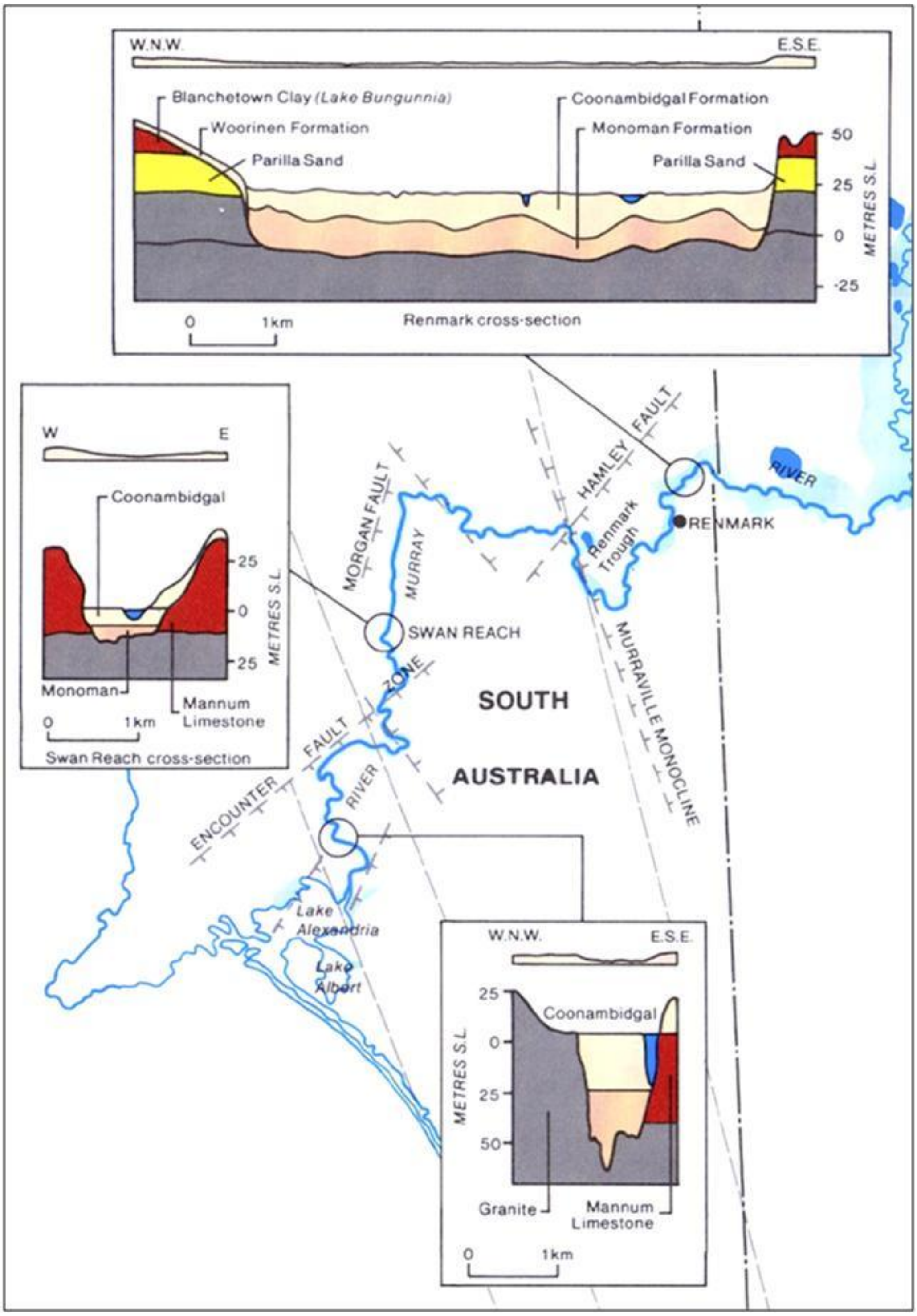

Figure 12.2 Geological cross-sections of the Lower Murray River Gorges at Renmark Swan Reach and Murray Bridge. From: Rutherfurd (1990) modified from Twidale et al. (1978). Note the characteristic backfilling of the excavated gorge firstly by the sands of the Monoman Formation which probably occurred between 12,000 and ~7,000 years ago; and then secondly by the laminated muds of the Coonambidgal Formation from about ~7,000 years ago to the present day.

depositional style, typically considered to be a transition from high-energy deposition of sands typical of a braided river to lower-energy deposition of fine-grained material, i.e. deposition of floodplain muds typical of overbank deposits of meandering rivers such as the lower Mississippi. This interpretation was developed from, and is somewhat consistent with, the present-day meandering-river planform geometry that characterises the modern river.

Deposition of low-energy flood-plain sediments of the Coonambidgal Formation commenced at ~7,000 years BP upstream of Renmark, well after sea-level stabilised near its present-day elevation, and continues to the present day (Firman, 1966; Lawrence, 1966; Firman, 1971; 1973; Twidale et al., 1978). The transition from high-energy sand deposition to mud and soft clay deposition in the Lower Murray is more likely to date from the time when sea-level stabilised near its present day level approximately 12,000 years ago. Geological cross-sections across the Murray River Gorge indicate that sediments of the Coonambidgal Formation are 15 – 25 m thick and sediments of the Monoman Formation are a further 20 – 25 m thick (Ludbrook, 1960; Steel, 1967; Firman, 1973; Twidale et al., 1978), with the greater thicknesses present downstream in the vicinity of Murray Bridge, with the transition between sands and muds located about −20 m AHD.

Further downstream, where the River Murray debouches into the terminal lakes complex, the Murray's mouth has been described as 'a failed delta' due to the absence of a substantial deltaic complex that might be expected from the deposition of material derived from such a large catchment. Murray-Wallace et al. (2010) have demonstrated that sediment delivered to the mouth of the Murray has likely been incorporated into aeolian deposits such as the sand ridges developed at Narracorte during sea-level highstands, or transported offshore beyond the edge of the Lacepede Continental Shelf during glacial maxima. These authors assert that "*The Holocene and modern River Murray have not established a marine delta, but deposit its load in the settling basins of the terminal lakes, Alexandrina and Albert*". They indicate that a small digitate delta has formed where the river enters Lake Alexandrina, indicating that the majority of the sediment derived from the catchment during the current sea-level highstand is deposited upstream of the terminal lakes.

Paleoclimates during the late Quaternary indicate increasing seasonal aridity (Figure 3.3). A general change from the deposition of fluvial matter to aeolian dust recorded in offshore cores adjacent to the southeastern Australian coast, have been interpreted as indicating a trend towards more arid conditions during the Holocene from 13.5 ka to present. This trend was interrupted by 2 periods of influx of fluvial material from the Murray Catchment between 13.5 – 11.5 ka and 9.5 – 7.5 ka, representing more humid conditions in the southern part of the Murray-Darling Basin (MDB) (Gingele et al., 2007). Although the climate in the region was generally dry between 4 ka and 2 ka (Stanley & Deckker, 2002), there is an indication of varying climate conditions with a humid event recorded in sediments at 2.8 ka BP (Gingele et al., 2007).

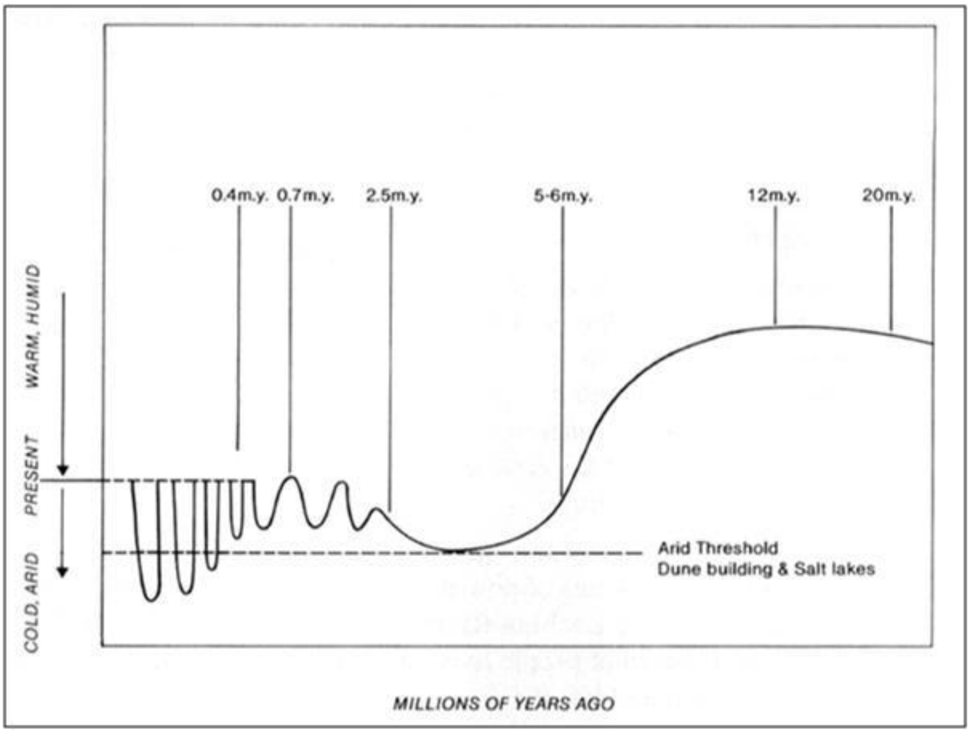

Figure 12.3 Regional climatic variation for the South-Eastern Australia landmass during the last 20 million years (Neogene). Note the cycling between present-day conditions and cold and arid conditions during the last 700,000 years in response to the climatic effects of the expanding and contracting global ice-sheets (From: Bowler, 1990).

Increasing regional climatic aridity would have had hydrologic and geomorphic implications for the ancestral Murray River. Reduced precipitation in the Murray Catchment would have reduced freshwater outflow and constricted the Murray Mouth, possibly closing it to the Southern Ocean. This coincides with an onset of estuarine-lagoonal sedimentation at about 3.5 ka BP recorded in the lower estuary of the River Murray (Cann et al., 2000). During this period it is plausible to assume that the Lower Murray River would have been completely isolated from oceanic sediment input, with river drainage constrained, creating a static environment resembling lagoonal and lake depositional conditions.

An interpretation of the recent geomorphic and geologic development of channel evolution and sedimentation in the Lower Murray River gorges outlined above was developed by Mr. Alan Moon and was been presented in diagrammatic form (Figure 3.4) in the Coffey (2012) “Review of Riverbank Collapse.”

An alternative interpretation of the same data has been developed (Hubble and De Carli in prep.) after examination of riverbank cores collected in May 2013 as part of this project. This model posits that the sediment delivered to the Lower Murray since the stabilisation of sea-level near its present day level has been trapped upstream of Lake Alexandrina. Hence, the transition of the high-energy sands of the Monoman formation to the low-energy sediments of the Coonambidgal Formation when sea-level reached its present level approximately 12,000 years ago marks a change in nature of the Murray. At this point in time sufficient sand had been delivered to the mouth to contribute to the Coorong barrier complex which effectively closed the Murray River mouth, forming a series of lakes including the Coorong Lake Albert and Alexandrina and a suspected fourth lake, “Lake Mannum” which would have occupied the present-day river reaches between Lake Alexandrina and Renmark or the possibly the Riverine Plains upstream.

The development of a lake in this location, rather than a prolonged period of aridity, better explains the suppression or cessation of the delivery of fine fluvial sediment to the shelf

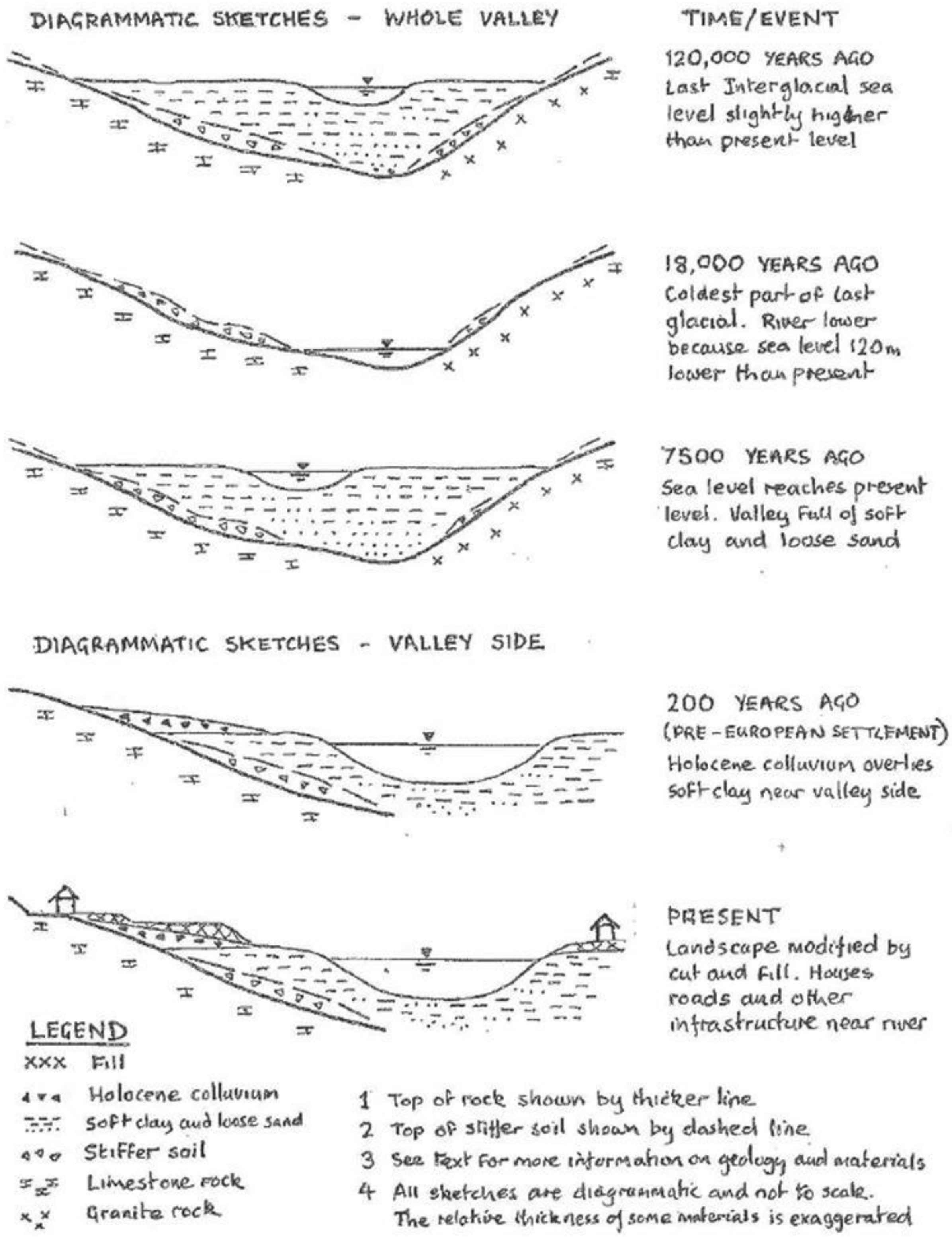

Figure 12.4 Geological and Geomorphic development of the Lower Murray River floodplains during the Late Quaternary and Holocene (From: Coffey 2012).

identified by Gingele et al. (2004). Instead, it is strongly suspected that the fine-grained sediment delivered by the Murray and Darling Rivers to the Lower Murray River during the second half of the Holocene has been trapped and deposited in the posited lake. This would explain why there has been little discharge of sediment to the ocean through the river mouth at Goolwa during much of the Holocene. In this scenario it is envisioned that this hypothesised lake, developed behind the dune and strand-plain complex of the Coorong Lakes. If the Lower Murray behaved as a lacustrine system rather than a freely discharging river during much of last 12,000 years, then it follows that the sediments of the Coonambidgal Formation would present sedimentary features typical of lakes and submerged plants, rather than sedimentary structures typical of overbank floodplain deposits and emergent terrestrial plants.

This alternate interpretation of the recent geomorphic development of the Lower Murray's channel and sediments is presented in diagrammatic form in Figure 3.5 and requires verification by dating of the sediments at different depths in the sequence.

There is little practical difference between the Moon and Hubble interpretations in respect of the bank failure problem. The two models explain the presence of the thick layer of soft clay in the floor of the Lower Murray Gorges in slightly different ways. Both models posit incision and widening of the Murray's present-day channel after the point in time when the volume of soft clay deposited in the Lower Murray Gorges raised the level of this valley fill deposit to about present-day sea-level. When this event occurred, direct flow of the Murray to the ocean at Goolwa was re-established and the Murray began to incise its channel down into the clays in turn has resulted in ongoing widening of the channel margins. This sequence of events is similar to the recent geomorphic development of a number of channels in Australia and overseas, and the recent channel behaviour presents a typical example of a channel's response to an incision event (Figure 3.6). There are strong general similarities between the Murray's Holocene history and the Holocene development of coastal rivers in New South

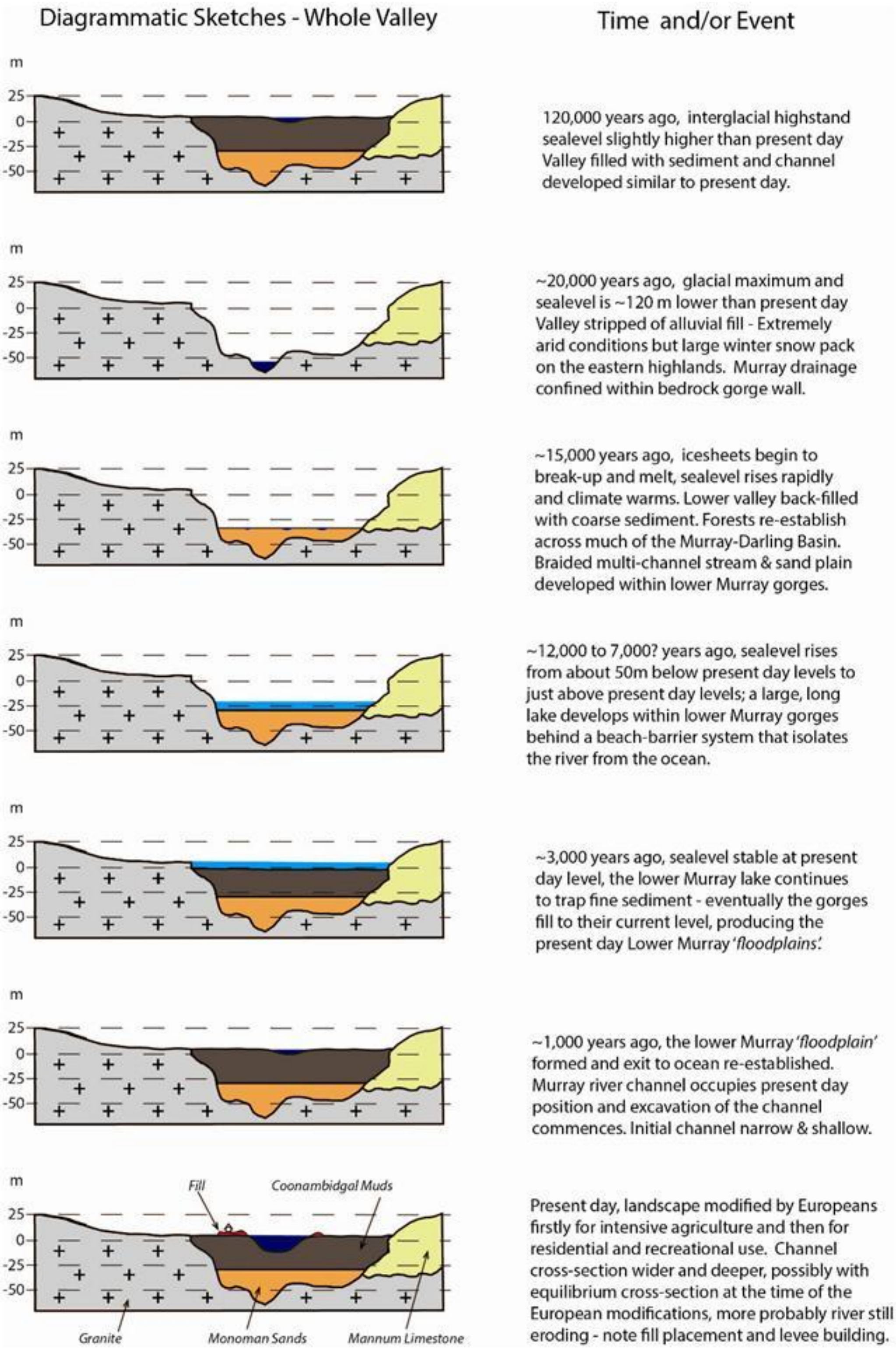

Figure 12.5 Geological and Geomorphic development of the Lower Murray River floodplains during the Late Quaternary and Holocene (Hubble and De Carli in prep). Generalised whole valley cross-section in the vicinity of Mannum or Murray Bridge – valley width approximately 3 km.

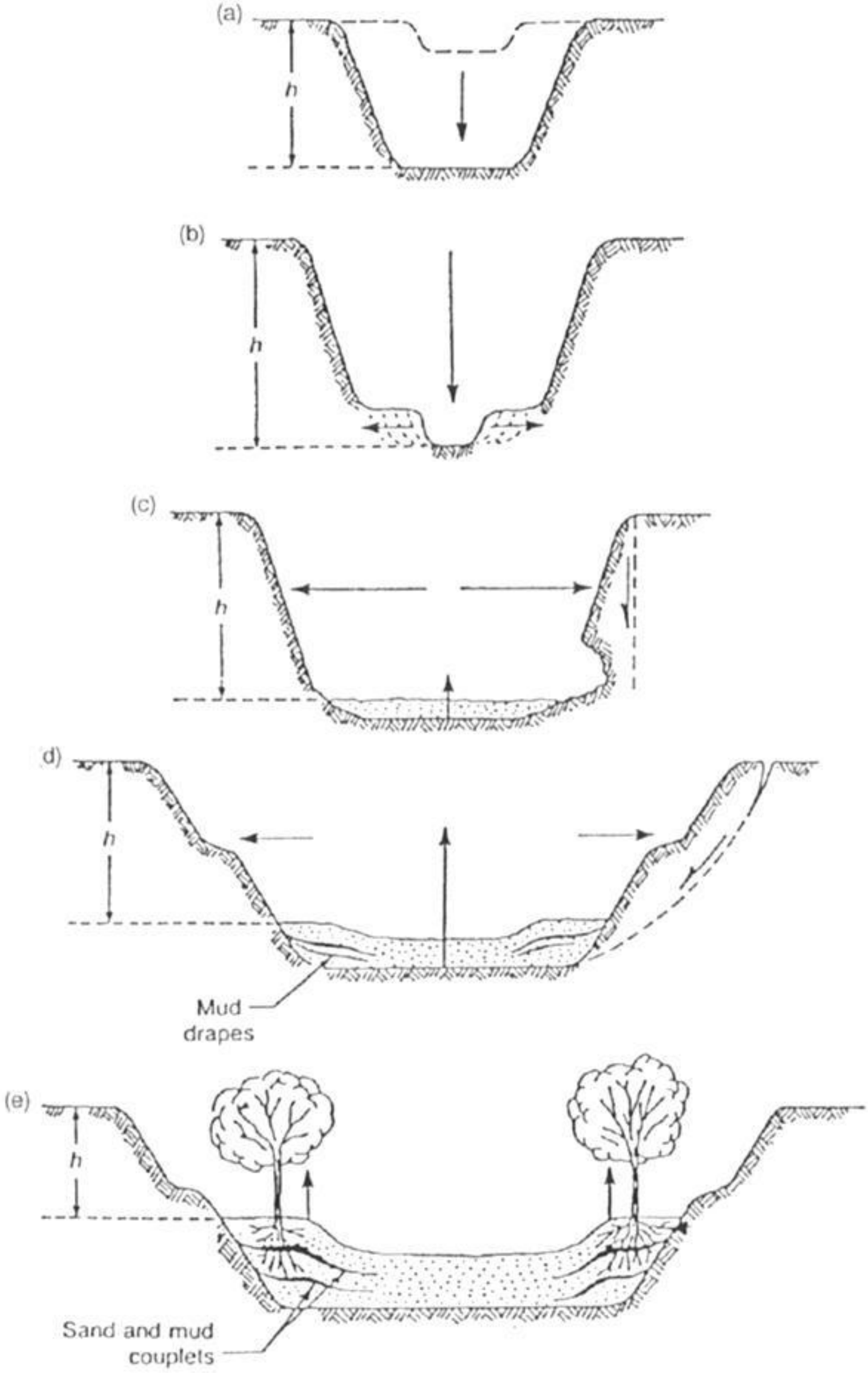

Figure 12.6 Expected sequence of events following channel incision (From: Schumm, 2005). Stages (a) and (b) initial incision; Stage (c) widening; Stage (d) widening and aggradation; and Stage (e) equilibrium and stability.

Wales. The lower Mississippi River's response to the retreat of the Northern hemisphere's ice sheets and the rise of sea-level has also produced a similar stratigraphy of early Holocene sands and Late Holocene muds deposited above a pre- Holocene erosional surface (*cf* Schumm, 2005).

13 Summary of Bathymetric Investigations

High-resolution multibeam bathymetry data were collected at particular sites of reported riverbank instability at the request of DEWNR by Gareth Carpenter in 2011. Table A.3 (Appendix A) and Figure 2.2 list the sites where this information was collected.

Examination of the initial bathymetric images produced from these data has been undertaken at several sites and was used to determine the spatial dimensions and scale of reported collapse features during initial geotechnical investigations. Examination of this first generation of images indicated that there was evidence for unreported failures (slump features) at sites within the survey areas nearby or adjacent to the known collapse features, for example the reported and adjacent unidentified bank failure features along the Lower Murray River at Thiele Reserve, White Sands (Figures 4.1 and 4.2) and Woodlane Reserve (Figures 4.3 and 4.4).

Subsequent examination of the multibeam dataset using the 3-D imaging software package Fledermaus V7 has confirmed the widespread occurrence of slides and slumps from the channel margins, additional to the bank collapse features that were reported by the public during 2009-2011 and documented in the DEWNR Incident Register (See Figures 4.1 to 4.4, and Appendix C).

Many of these newly identified slump and slide features lack an obvious, related slide-debris field or related deposit of sediment. Note that these slide-debris deposits present as U-shaped mounds or ‘humps’ or a lobate deposit of blocks and sediment deposited ‘in front’ of the slump-scars. The curve of the ‘U’ is located in the floor of the channel and the axis of the ‘U’ is generally oriented perpendicular to the channel margin (i.e. waterline along the bank). Such mounds are commonly associated with, and characteristic of the 2009-2011 slide scars (e.g. Figure 4.1) whereas the additional identified slump scars either present: a rounded mound with subdued relief and protruding mounds rather than angular blocks; or no evidence of a related slide-debris deposit. In the latter instances it is also apparent that these slump scars’ morphology is more subdued or rounded and that their boundaries are not as sharply defined as the 2009-2011 slump features.

The presence of deposits of angular debris in-front of the 2009-2011 failure sites and the existence of both slumps related to smoothed debris fields; and subdued-relief slumps without debris fields, is interpreted to indicate that slumping and delivery of slide blocks into the channel is an ongoing characteristic of the Murray River's banks. These three slide types represent a transition from 'recent' through 'relatively-recent' to 'relatively-old' events. Over time successive flood flows will erode (smooth) and redistribute (remove) slide-debris deposits on the river channel floor. Similarly, flood flows will erode (smooth and subdue) the slump and slide scars.

Analysis of the Murray's floodplain and riverbanks has been entirely reactive, commissioned after the onset of recent bank destabilisation. Prior to recent geotechnical investigations (Table A.1, Appendix A), there exists little scientific literature (Table A.2, Appendix A). Whittingham (1987) identified features of prehistoric and modern signs of bank instability at SAB Aruma, Walker Flat, characterised by anomalous topography, hummocky ground, tension cracks and subsidence. The failure mechanism was described as a 'driving wedge translational slide', and Whittingham (1987) concluded that the surrounding river reaches between Wongulla and Lake Alexandrina showed no signs of instability at this time. The SAB Aruma site has since documented a small collapse in February 2010.

The presence of the 'unknown' failures and their apparently age-progressive morphologic character suggests that riverbank collapse on the Lower Murray may not just be a recent occurrence, but the result of a long-term geomorphic readjustment of the river channel. The identification of these features is both consistent with and expected from the recent geologic and geomorphic evolution models proposed in the Coffey assessment (2012; Figure 3.4) and by Hubble and De Carli (2013; Figure 3.5).

An additional, pertinent observation that has arisen during the examination of the multibeam bathymetry relates to the morphology of the slide scars. While some of the slide scars, particularly the large failures that occurred at Long Island Marina and Woodlane Reserve conform with and resemble the failure styles produced by the SKM (2010b) and Coffey (2012) geotechnical reports, a large number of the slide scars present a somewhat different morphology.

The Long Island Marina and Woodlane Reserve failures, for example, are substantial deep-seated failures with slip-circles located in such a way that they emerge near the break-in-slope at the toe of the riverbank and crest well (5 to 10 metres or so) back from the water-line within the levee or filled embankment. In contrast, many of the failures and particularly the newly identified failures present the shallow, planar-failure style with thin slabs that have slid down bank-parallel or bank-sub-parallel failure planes. While not entirely inconsistent with the geotechnical modelling presented to date, this apparent difference in failure geometry is worthy of further investigation.

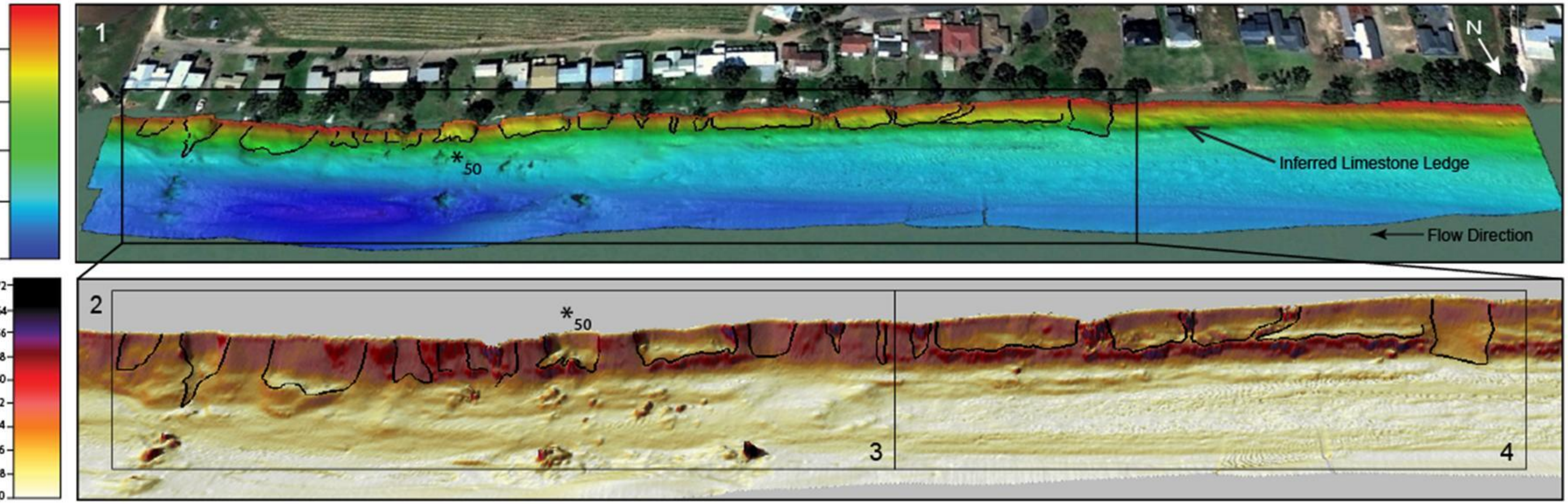

Figure 13.1 Upper Image 1 – Multibeam bathymetric image of the right-hand channel margin of the Murray River at White Sands, South Australia, showing eighteen identified slump features of which three are either reported in the formal register or informally (to the authors during a May 2013 site visit) by local residents. As far as can be ascertained it is the reported events that present related angular debris deposits – these are the site labelled *50 (Incident 50, Table C.1, Appendix C), the slump situated immediately upstream (Incident 297) and the second last downstream slump (reported informally) in Images 1 and 2. The multibeam bathymetry has been merged and geo-registered with an aerial view of the site in Google Earth. The scale bar associated with Image 1 represents decreasing depth in meters. Lower Image 2 – Enlarged views of the area outlined by the black box in Image 1. Enlargements of the areas labelled 3 and 4 are given in Figure 4.2. The scale bar associated with Image 2 represents increasing bank slope angle.

Notes:

1. The large number of undocumented or unreported slump failures and the general absence of associated debris lobes in majority of the upstream areas of the image (e.g. area 4 in Image 2). It is expected that the angular blocks and pinnacles presented in the slide debris deposit features will tend to smooth or become more subdued with greater age; Also note the presence of a suspected bedrock ledge (Murray Group Limestone), identified in the right hand side of Images 1 and 2.
2. The continuity of the subaqueous slide outlines with the curvate indentations presented at the waterline. Arcuate indentations at the waterline or “cuspate bites” may well be a tell-tale sign of failure in archival aerial photographs.
3. The generally restricted presentation of the crests of the majority of failure surfaces ‘behind’ the waterline, whereas the recent, debris-associated failures ‘eat-back’ into the floodplain/levee area by approximately 5 metres.
4. The majority of the slump masses are relatively thin, and are usually between one and two metres thick.

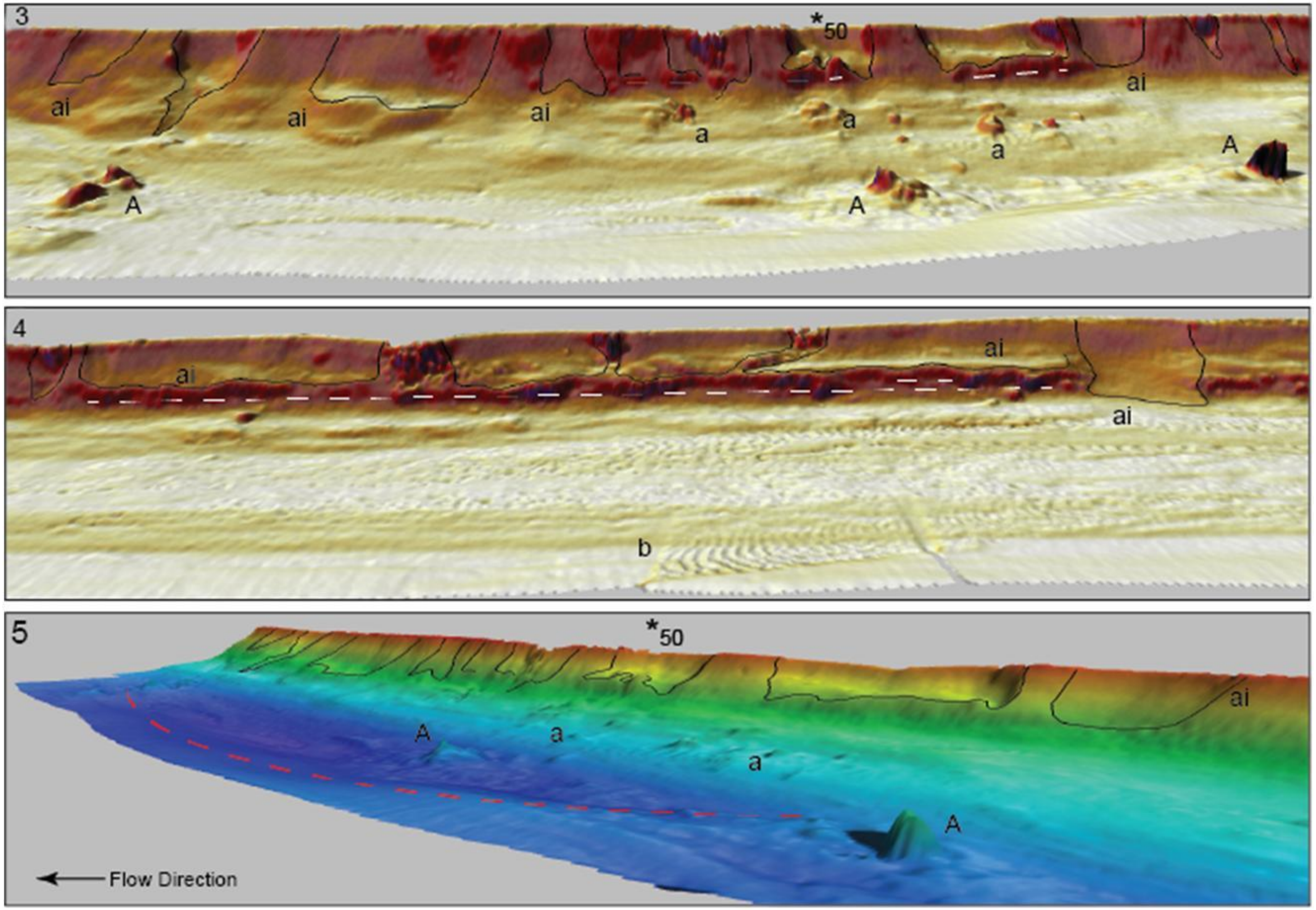

Figure 13.2 Multibeam bathymetric images of the right-hand channel margin of the Murray River at White Sands, South Australia. Images 3 and 4 provide enlarged views of areas 3 and 4 shown in Figure 4.1 (previous page) and an oblique view of area 3 is represented in the lower Image 5. Note again the almost ubiquitous prevalence of the slump and slide features on the channel margin; the general absence of identifiable associated slide-debris on the floor of the channel in Image 4; scour holes downstream of the larger recent angular pinnacles (A's); smoothed debris deposits down-slope of slide and slump features (a's); smoothed or subdued slide and slump features (ai's); and linear dunes or sedimentary bedforms (labelled b) in the centre of Image 4. Note also the widespread presentation of the suspected limestone ledge which is identified with a white dashed line in Images 3 and 4. The red-dashed line in Image 5 is the channel thalweg (deepest flow path).

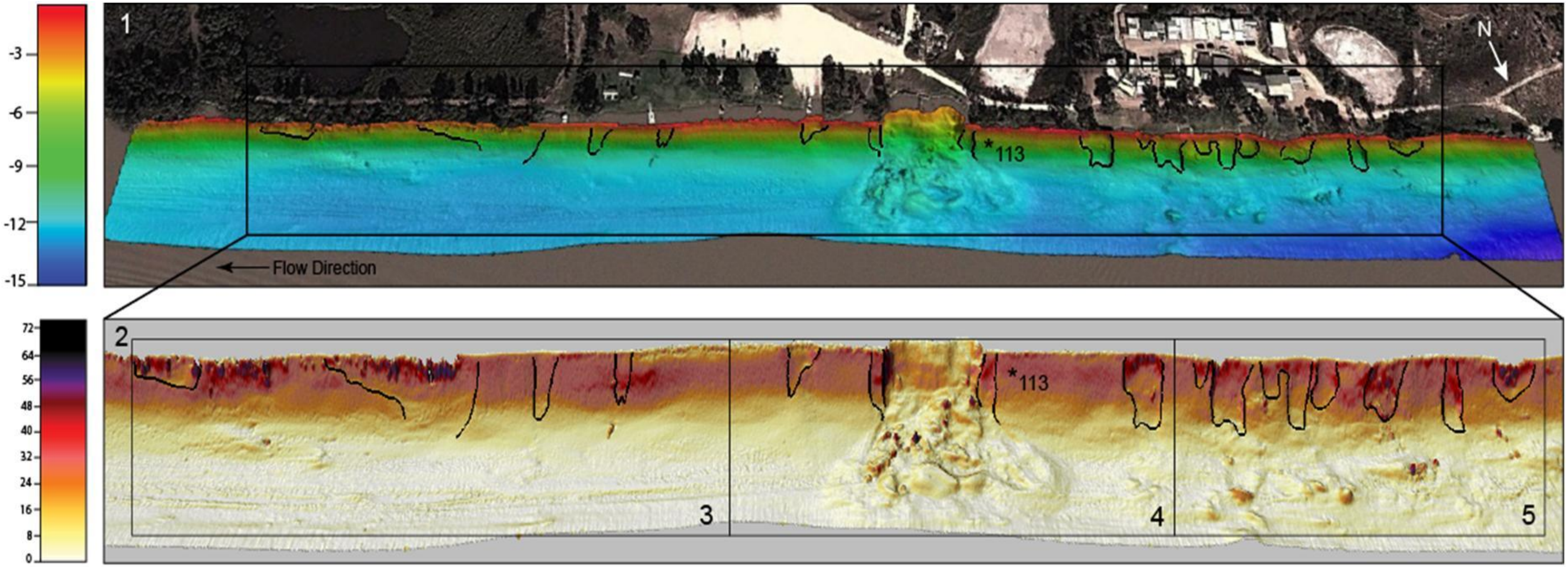

Figure 13.3 UPPER IMAGE 1 – Multibeam bathymetric image of the right-hand channel margin of the Murray River at Woodlane Reserve, Mypolonga South Australia showing seventeen identified slump features of which four are reported in the event register (Table C.1, Appendix C). As far as can be ascertained the four reported events are all related to the same large slide feature (represented here by Incident *113) which presents a particularly impressive debris deposit reaching the middle of the channel. Enlargements of the areas labelled 3, 4 and 5 are given in Figure 4.4. The multibeam bathymetry has been merged and geo-registered with an aerial view of the site in Google Earth. The scale bar associated with Image 1 represents decreasing depth in meters. LOWER IMAGE 2 – Enlarged views of the area outlined by the black box in Image 1. Enlargements of the areas labelled 3, 4 and 5 are given in Figure 4.4. The scale bar associated with Image 2 represents increasing bank slope angle.

Notes:

1. The large number of undocumented or unreported slump failures and the general absence of associated debris lobes in downstream half of the site image (i.e. area 3 in the lower image).
2. The generally restricted presentation of the crests of the majority of failure surfaces ‘behind’ the waterline. The recent, debris-associated, failures ‘eats-back’ into the floodplain/levee area by about 5 metres or so.
3. The majority of the slump masses are relatively thin, and are usually between one and two metres thick and tend not to present deep regression into the floodplain/levee immediately adjacent to the channel.

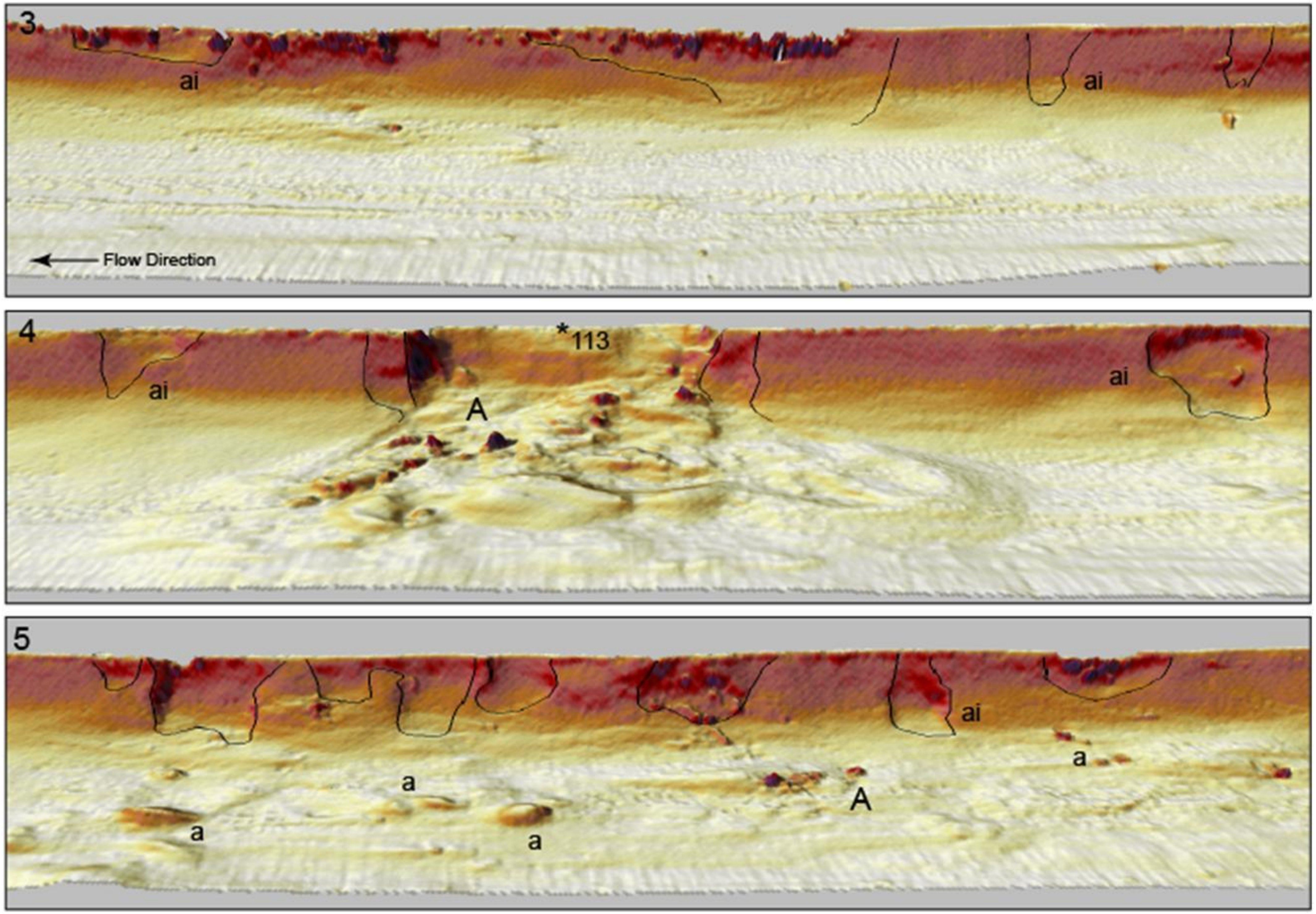

Figure 13.4 Multibeam bathymetric images of the right-hand channel margin of the Murray River at Woodlane Reserve, Mypolonga South Australia. These images provide enlarged views of areas 3, 4 and 5 shown in Figure 4.3 (previous page). Note again the almost ubiquitous prevalence of the slump and slide features on the channel margin; the general absence of identifiable associated slide-debris on the floor of the channel in Image 3; the larger, recent angular pinnacles (A's); smoothed debris deposits (a's in Image 5) and smoothed or subdued slide and slump features (ai's).

14 Key Questions and Knowledge Gaps

Re-examination of the geotechnical, geological and bathymetric information available has identified several previously unrecognised characteristics or aspects of the Lower Murray riverbank failure phenomenon that need to be considered in specifying the knowledge gaps and key questions that will frame and guide the program of further investigation.

These are:

- To date, investigations that have been conducted have been reactive and focused on sites where there is a perceived need due to the presentation of actual failure or precursor phenomena. A generalist or regional investigation strategy has not yet been applied to understanding the prevalence or otherwise of riverbank failure.

- Riverbank failure by slumping (deep-seated rotational failure) or sliding (shallow planar sliding) of the soft clay deposits which form the channel margins of the Lower Murray is likely an ongoing natural and normal response of these channel margin sediments to a phase of excavation (widening and incision) that the present-day Murray is undergoing in its lower reaches between the Lake Alexandrina and Blanchetown. In other words it is probable that the current normal behaviour of the channel in the Murray's lower reaches is erosional.

- The soft clays identified as the focus of the geotechnical investigations and the unit in which the failures actually occur are more likely to have been deposited in lacustrine (lake) conditions rather than as over-bank, floodplain deposits as might be expected from the present-day meandering river planform presented by the Murray River in its lower reaches.

- The 2009-2011 lowering of the water level in the channel is likely to have accelerated and amplified slumping on the channel margins, rather than initiated this phenomenon.

- There are probably many, many more slides and slumps present than have been identified. The slumps presently recorded in the incident register have been in high-use areas and/or have had infrastructure or trees located on them. Our examination of the available multibeam bathymetry indicates that slumping is more widespread.

14.1 The regional prevalence of failure

The geotechnical investigations thus far have reported on frequently visited and densely populated reaches of the river, and have been undertaken as a reactive response to particular collapse events. There is a significant lack of analysis on ‘reference’ reaches of the river, needed to be able to accurately quantify the extent of the riverbank collapse problem. The key questions that frame investigation of this issue are:

- How common are the failures? Are failures restricted to high-use areas of the river e.g. houseboat mooring sites; preferred wave-jumper and water-skiing sites; or ‘owner-occupied’ reaches of the channel e.g. holiday and permanently occupied housing adjacent to the channel, marina and popular houseboat mooring sites? Or do they occur as frequently in low-use, sparsely populated reaches of the river?

- What is the extent of unrecognised failures? Are shallow failures, such as those identified in §4 above, typical and widespread? Or are the deep-seated rotational failures such as the Woodlane Reserve and Long Island failures common but unrecognised in the less densely populated reaches of the river?

- How common is failure in general? That is, how many failures are recognisable per kilometre of channel? There is a need to examine occupied and unoccupied river reaches (i.e. reference reaches) with multibeam surveying (work of this type was not included in the original scope of this project).

14.2 Common geological and morphologic characteristics of failure occurrence

Collapses have occurred on opposite banks of the river channel, on the inside of river bends, on the outside of river bends and on straight sections of the channel. There is an apparent relationship evident in some of the sites presenting incipient and actual failures (East Front Road, Woodlane Reserve, Thiele Reserve, Murray Bridge, Monteith) where the failures and incipient failures are concentrated upstream and downstream of bedrock intrusions into the channel that constrict the channel by reducing its width.

- Are there any obvious links between channel morphology and the occurrence of failure; e.g. protrusion of bedrock in the channel such as the incipient and actual failures identified at East Front Road and Mannum sites?

- Is there are a relationship between bank angle and failure occurrence?

- Is failure more prevalent in areas where there are artificially constructed (or natural levees), and/or extensive reclamation of riverside land by filling in comparison to relatively unmodified riparian zones. Is the distance between the constructed levee crests and the waterline a critical factor?

- Is there are a relationship between fill presence, and failure occurrence?

14.3 Geological and geomorphic issues

The geological origin of the clays and geomorphic evolution of the channel is of interest as this will provide insight into the likely homogeneity and variation in the physical behaviour of the clays, that is determine the range of variation in their geotechnical properties and behaviour. Similarly the origin of the clays and the geomorphic evolution of the channel (i.e. the incision and widening models presented in §3), suggest that the recent 2009-2011 failures are amplifications or accelerations of the ongoing response of the channel to its geological and geomorphic setting.

Testing or determination of the sensitivity, mineralogy and salinity of the clays will assist in understanding if the materials in the banks and channel margins are likely to soften or lose strength over time. This could be due to loss of solute suction if the clays are estuarine in origin and were therefore relatively saline. Similarly, scour and erosion of the bank toe during high-flow events or floods may be inducing creep in the near-surface clays parallel to the bank slope, weakening the clays and contributing to the shallow, infinite-slope style slides evident in the bathymetric images.

- What are the geotechnical characteristics and probable origin of the soft clay? Are they fresh water deposits or were they originally saline deposits?

- What is the mineralogy of the clays? Are they clays inherently sensitive due to a dominantly swelling clay mineralogy (montmorillonites and smectites) or are the clays relatively stable kaolinites?

- Is the behaviour of the soft clay uniform within sites (laterally and vertically)?

14.4 Geotechnical data and modelling

Current modelling of the riverbanks has tended to assume homogeneity of material over large spatial areas, which may not be appropriate. Most if not all the material that has been sampled and tested, at this point in time, has been acquired from boreholes that have been drilled ‘onshore’, that is from drill-trucks located on the adjacent floodplain or levee banks. Direct sampling (and then testing) of materials from within the channel has not been within the scope of the investigations undertaken up to this point in time. Sampling is within the scope of this project and will be undertaken, but apart from the very preliminary findings of fieldwork undertaken in May 2013 the data are not available for consideration by review. Similarly, cone penetration testing (CPT) has been conducted onshore and the in-channel CPT testing conducted in May, 2013 will not be considered here.

To a certain extent the absence of this data may explain why the geotechnical models produced so far (see §2) indicate deep rotational failure which is the nature of the large recent failures rather than the apparently widespread shallow failure style observed in the bathymetric images, which raises the following questions:

- Is failure by shallow sliding a normal characteristic of channel widening with toe scour of bank sediments leading to failure of the whole bank?

- Is deep rotational failure a consequence of loading the soft clays with fill or an artificially constructed levee? Or will this present more generally?

15 Concluding Remarks

The review of existing geotechnical investigations and geological literature presented above has provided several new and useful insights into the occurrence of bank failure on the Lower Murray River between 2009 and 2011. In particular the following findings presented in this document will focus the project's future work by providing an investigative focus, and clear indication of the data required to improve the understanding of why these failures occurred.

- Loading of the channel margin due to the placement of fill or the construction of levees probably increases the likelihood of failure, particularly during periods of lowered pool-level and/or lowering river level.

- The ubiquity of shallow, planar slide failures that almost certainly predate the large deep-seated 2009-2011 failures, indicates that the channel is naturally widening by mass failure of the channel margins; that the channel margins are probably inclined at angles that are near the natural limit of their stability; and that both this widening and shallow sliding is probably a response of the channel to its geomorphic evolution and geologic setting, essentially a phase of incision and widening as the Murray excavates a channel in the soft clays deposited during the Holocene.

- Additional slope stability modelling should focus on establishing the conditions that generate the shallow failures in the soft clays of the channel margins.

- Developing a better understanding of a) the origin of the soft clays which are the predominant material present in the Lower Murray's channel margins, and b) the natural variability in the range of physical properties presented by this 'soft clay' is a major need. In particular, sampling and testing the clays of the submerged, in-channel portions of the banks is required to maximize the validity of slope stability modelling.

- There is a definite need for a generalist regional study of the channel's morphology (i.e. an additional regional bathymetric mapping study) to contextualise the present study and establish the prevalence of failure at a regional scale rather than at a site specific scale.

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Appendix A
Relevant Literature

Table A.2 Riverbank collapse hazard investigation and geotechnical reports.

Date	Geotechnical Company	Author	Title
20/11/08	Arup	-	Assessment of Soil Cracking and River Bank Slumping in the Lower Murray Part 1 Report Rev C
20/11/08	Arup	-	Assessment of Soil Cracking and River Bank Slumping in the Lower Murray Part 2 Report Rev C
20/03/09	Sinclair Knight Merz	Daryll Pain	Lower River Murray - Riverbank Slumping - Site Inspection of 13 March 09
18/05/09	Robert C Frazer & Associates	Robert Frazer	Proposed Reopening of Navigation Channel (Long Island / Mainland Opp. Long Island Marina, Murray Bridge)
20/05/09	Robert C Frazer & Associates	Robert Frazer	Investigation into Slope Stability of Existing Riverbank /Roadway - Preliminary Report: Type 2 (Murrawong)
15/06/09	Robert C Frazer & Associates	Robert Frazer	Investigation into Slope Stability of Existing Riverbank - Preliminary Report: Type 2 (Sturt Reserve South - Shack Site)
22/06/09	Robert C Frazer & Associates	Robert Frazer	Investigation into Slope Stability of Existing Riverbank - Preliminary Report: Type 2 (Wellington Emergency Mooring Site)
1/02/10	Sinclair Knight Merz	-	Study in to Riverbank Collapsing for Lower Murray River - Geotechnical Investigation Report - Rev E - FINAL (phase 3 major investigation)
16/02/10	Golder Associates	Lyndon Sanders	Peer Review - SKM study of Riverbank Collapsing Lower Reaches - River Murray
17/02/10	Sinclair Knight Merz	-	Study in to Riverbank Collapsing for Lower Murray River - Inspection Report (phase 2 major Investigation)
19/02/10	Golder Associates	Lyndon Sanders	Assessment of Landslide Risk, Ngaut Ngaut Conservation Park, South Australia
11/03/10	Golder Associates	Lyndon Sanders	Landslide Risk Management, Ngaut Ngaut Conservation Park, Lower Reaches - River Murray
11/03/10	Sinclair Knight Merz	Daryll Pain	Part B: Influence of Pomanda Island Weir on the potential for Riverbank Collapse
24/03/10	Sinclair Knight Merz	Daryll Pain	Part A: Lower River Murray - Influence of NSW Floodwaters on the potential for a Riverbank collapse to occur

Table A.1 Riverbank Collapse Hazard Investigation and Geotechnical Reports. (Continued)

Date	Geotechnical Company	Author	Title
13/04/10	Golder Associates	Lyndon Sanders	Peer Review - SKM Study of influence on NSW floodwaters, Lower Reaches, River Murray
19/04/10	Golder Associates	Lyndon Sanders	Review of Stability Analysis Swan Reach Waste Disposal Station - River Murray
28/04/10	Golder Associates	Lyndon Sanders	East Front Road, Mannum, South Australia, Options for Reducing the Probability of Landsliding
6/07/10	Sinclair Knight Merz	Daryll Pain	Lower River Murray - Task 8.1 - Influence of Water level rises from +0.10mAHD to +0.75mAHD on the potential for a Riverbank Collapse to occur
7/07/10	Golder Associates	Lyndon Sanders	Landslide Risk Management, East Front Road, Mannum to Younghusband, River Murray, South Australia
22/07/10	Sinclair Knight Merz	Daryll Pain	Lower River Murray - Tasks 8.2, 8.3, and 8.4/8.5 - Validity Period of Investigations and Influence of variation of soil parameters and site conditions on the potential for a Riverbank Collapse to occur.
4/08/10	Golder Associates	Lyndon Sanders	Riverbank Stability Risk Management, Caloote Boat Ramp, South Australia
10/08/10	Golder Associates	Lyndon Sanders	Landslide Risk Management, Mannum Caravan Park, South Australia
14/09/10	Sinclair Knight Merz	Peter Sandercoc k	Riverbank Collapse Risk Management, Site Inspection Report
29/10/10	Golder Associates	Steve Barrow	Caloote Boat Ramp Load Test
3/11/10	Golder Associates	Lyndon Sanders	Peer Review - Riverbank Collapse Hazard - Lower Reaches River Murray, Peer Review, Stability Risk assessment, Caloote Landing
11/11/10	Sinclair Knight Merz	Daryll Pain	River bank Collapse Hazard - Lower River Murray Stability Risk Assessment for Caloote Landing
17/11/10	Golder Associates	Lyndon Sanders	Peer Review - Riverbank Collapse Hazard - Lower Reaches River Murray, Peer Review, Stability Risk assessment, Caloote Landing
29/07/11	Golder Associates	Lyndon Sanders	Peer Review - Riverbank Collapse Hazard, Lower Reaches River Murray, Stability Risk Management, Caloote Landing
4/08/11	Coffey Geotechnics	Dave Morton	Peer Review - Caloote Landing Boat Ramp, Lower River Murray Comments on Stability Risk Management Proposal

Table A.1 Riverbank Collapse Hazard Investigation and Geotechnical Reports. (Continued)

Date	Geotechnical Company	Author	Title
21/09/11	Golder Associates	Lyndon Sanders	Riverbank Collapse Hazard, Lower Reaches River Murray, Stability Risk Management, Caloote Landing
27/09/11	Golder Associates	Lyndon Sanders	Riverbank Collapse Hazard, Lower Reaches River Murray, Stability Risk Management, Caloote Landing
29/11/11	Coffey Geotechnics	Dave Morton	Peer Review - Caloote Landing Boat Ramp, Lower Murray, Comments on Stability Risk Management Report
6/03/12	Golder Associates	Lyndon Sanders	Stability Risk Management, Review of Monitoring Data, Mannum Caravan Park
13/07/12	Durhan University	David Petley	Peer Review - Walker Flat River Vessel Waste Disposal Station Report VE23686, Peer Review
13/07/12	Golder Associates	Lyndon Sanders	Peer Review - Walker Flat River Vessel Waste Disposal Station Peer Review, SKM Report *Investigation of Actions required to return to service*
14/09/12	Sinclair Knight Merz	Robert Scott	Riverbank Collapse Hazard, Lower Reaches River Murray Stability Risk Management, Caloote Landing, Monitoring Review - August 2011 to August 2012
18/09/12	Sinclair Knight Merz	Robert Scott	Walker Flat River Vessel Waste Disposal Station - Investigation of actions required to return to service
25/09/12	Golder Associates	Lyndon Sanders	Peer Review - Riverbank Collapse Hazard, Lower Reaches River Murray Stability Risk Management, Caloote Landing, Monitoring Review - August 2011 to August 2012
30/11/12	Golder Associates	Lyndon Sanders	Peer Review - Riverbank Collapse Hazard, Lower Reaches River Murray, Management Options for Four High Risk Sites, Peer review of Coffey Geotechnics Report, November 2012
12/12/12	Coffey Geotechnics	Allan Moon	Review of Management Options for Four Riverbank Collapse High Risk Sites
15/03/13	Sinclair Knight Merz	Robert Scott	Riverbank Collapse Hazard, Lower Reaches River Murray Stability Risk Management, Murray View Estates, Tailem Bend, Monitoring Review - Feb 2012 to Jan 2013
17/12/13	Sinclair Knight Merz	Robert Scott	Riverbank Collapse Hazard, Lower Reaches River Murray, Stability Risk Management, Bells Reserve Monteith, Monitoring Review April to December 2012

Table A.3 Scientific literature associated with riverbank instability on the Lower Murray River (Nb. The items listed are directly associated to riverbank collapse on the Lower Murray River, and do not represent a comprehensive literature review).

Year	Author	Title
1968	Dept of Mines SA (R.D. Steel)	South East Freeway - River Murray Crossing (Monteith Vs Swanport)
1975	R.M. Thomson	The geomorphology of the Murray River in South Australia.
1978	Twidale et al.	Proceedings of the Royal Society of Victoria: Age and Origin of the Murray River and Gorge in South Australia
1987	A. Whittingham	Stability of slopes of the Murray River between Wongulla and Lake Alexandrina.
1989	S.R. Barnett	The hydrogeology of the Murray Basin in South Australia with special reference to the alluvium of the River Murray floodplain
1991	Brown & Stephenson	Geology of the Murray Basin, Southeastern Australia
1991	I.D. Rutherfurd	Channel form and stability in the River Murray: a large, low energy river system in South Eastern Australia.
1993	M.C. Thoms & K.F. Walker	Channel changes associated with two adjacent weirs on a regulated lowland alluvial river
2008	D.S. Baldwin	Impacts of Recreational Boating in River Bank Stability: wake Characteristics of Powered Vessels: *Report for the Murray Catchment Management Authority.*
2010	M.Schiller & S.Wynne	The effect of declining water levels on the stability of riverbank slopes
2012	Liang et al.	GIS-based Back Analysis of Riverbank Instability in the Lower River Murray

Table A.4 Location of multibeam bathymetric surveys and date undertaken.

Location	Date Taken
Bells Reserve, Murray Bridge	8/6/2011
Caloote Landing	21/7/2011
Dickson Reserve, Tailem Bend	8/6/2011
East Front Rd, Younghusband	21/7/2011
Fred's Landing	2012
Long Island Marina, Murray Bridge	8/6/2011
Murrawong Road	21/7/2011
Ngaut Ngaut Conservation Park	20/7/2011
River Front Rd, Murray Bridge	8/6/2011
Thiele Reserve, Murray Bridge	2012
Walker Flat Waste Disposal Station	20/7/2011
White Sands	8/6/2011
Woodlane Reserve, Mypolonga	21/7/2011

Appendix B

Geological Setting (Deep Time)

The following is an overview of the geological and geomorphic history of the Murray Basin, with focus on the Cainozoic era and the Tertiary and Quaternary stratigraphic units present within the Lower Murray River. It is important to note that despite the Murray being the most studied floodplain in Australia, the number of studies that address the geomorphological history of the Murray River are relatively few in number and the majority of those that are available deal with either the development of Riverine Plain upstream of Cadell or the coastal beach and dune systems downstream of the Murray's present-day terminal lakes of Lake Alexandrina and Lake Albert.

The Murray Basin is an intra-cratonic basin of Cainozoic fluvial to shallow marine sediments covering an area of approximately 330,000 km^2 including parts of south-east SA, south-west NSW and western VIC (Stephenson, 1986). It overlies the Devonian to Permian Nadda Basin and Cretaceous Berri Basin. The South Australian share of the Murray Basin is bordered to the west by Adelaidean to Cambrian rocks of the Adelaide Geosyncline and Kanmantoo Trough, and Proterozoic rocks of the Curnamona Province to the north. It developed following the break up between Antarctica and the southern margin of Australia, beginning as a region of slow subsidence coupled with low rates of sediment supply (Brown & Stephenson, 1991).

A pictorial overview of the physiography, geological structure and stratigraphy of the major sedimentary units present in the basin is given in Figures B.1 and B.2 (From: Evans et al., 1990) and Figure B.3 (compiled from: Kennett, 1978; Exon et al., 2004).

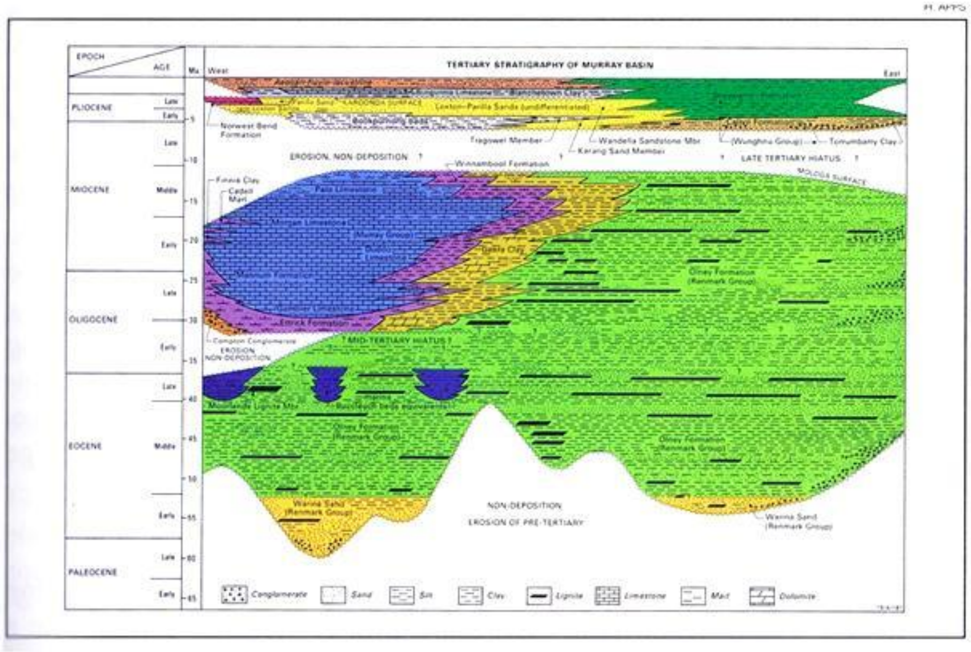

Figure B.1 East-West section showing geology of the Murray Basin (From: Evans et al., 1990).

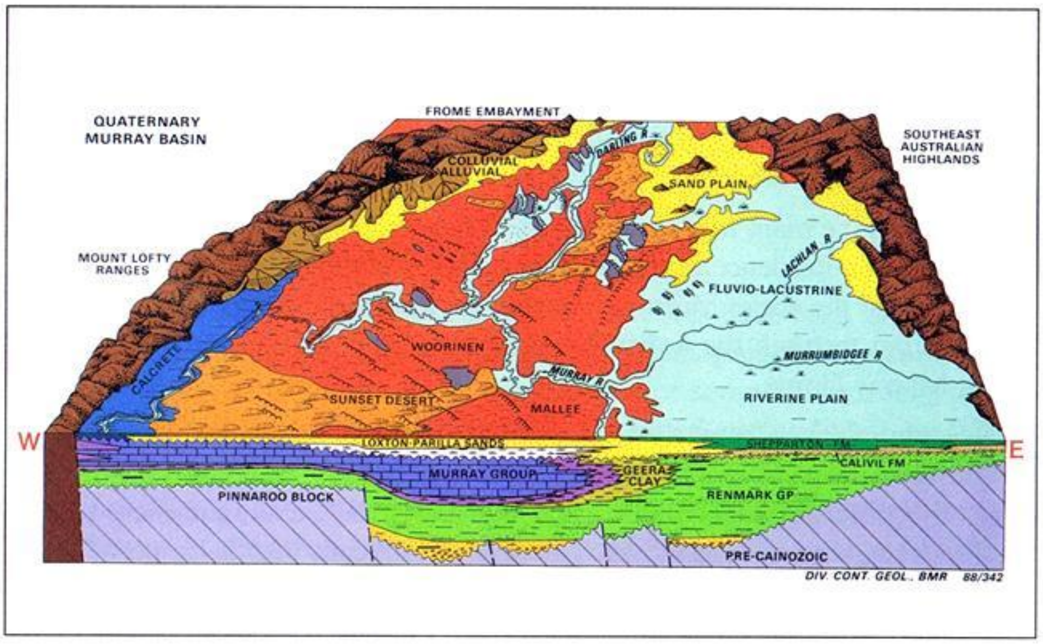

Figure B.2 Stratigraphic succession of sedimentary formations in the Murray Basin (From: Evans et al., 1990).

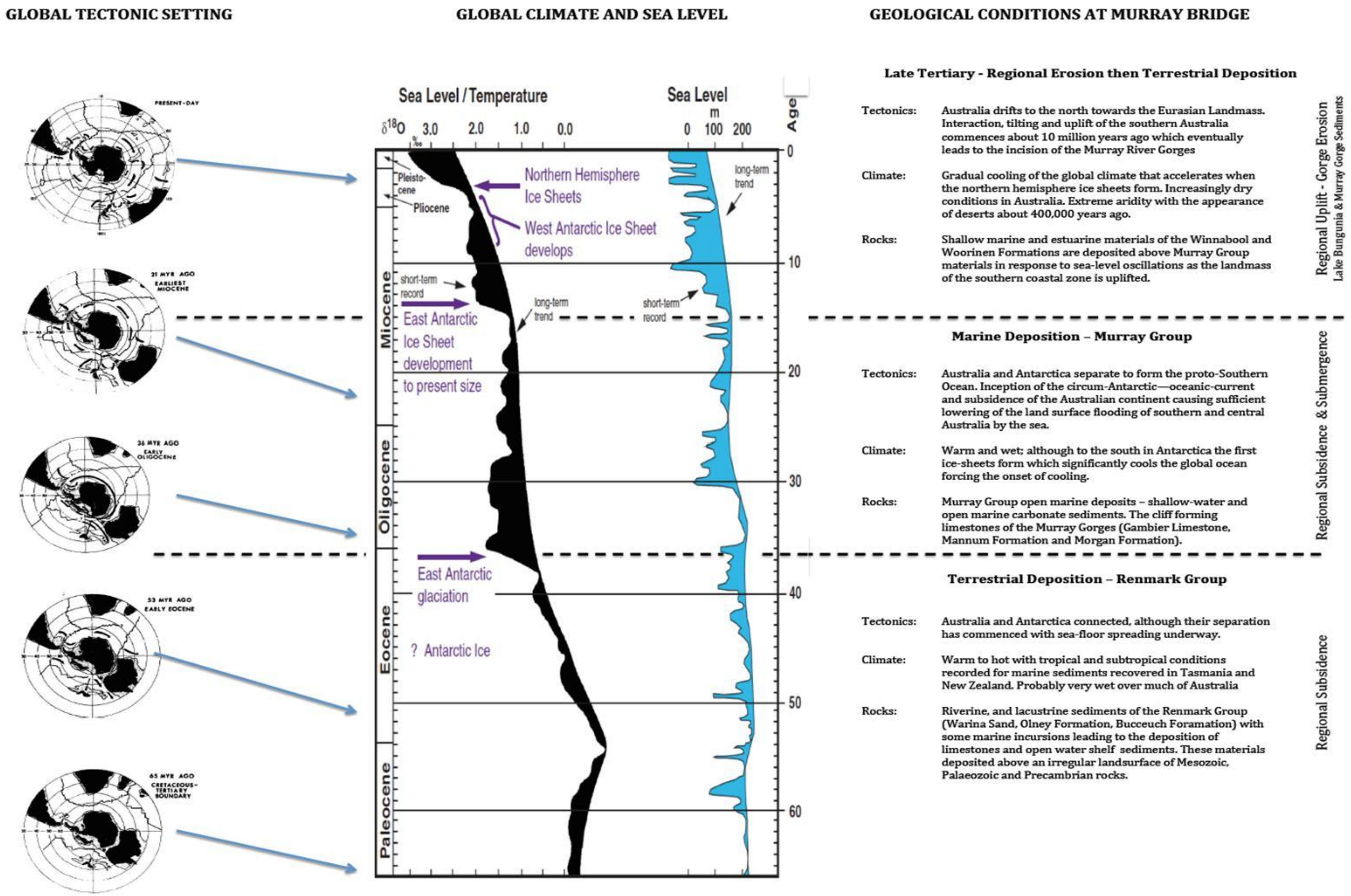

Figure B.3 Global tectonic setting, climate, sea-level and geological conditions of the Lower Murray River (Murray Bridge area) in the Tertiary Era (compiled from: Kennett, 1978; Exon et al., 2004).

Tertiary

Paleocene – Eocene

Sedimentation commenced in the Murray Basin 60 Ma ago during the Tertiary period. During the Paleocene to Eocene, floodplain and swamp fluvio-lacustrine environments dominated, depositing carbonaceous sand, clay and silt of the Renmark Group (Warina Sand, Olney Formation, Buccleuch Formation). Warina Sand consists mainly of stacked massive sand bodies that coalesce to form a sand sheet, thought to have been deposited in a mainly fluviatile, possibly braided-channel environment. Minor thin lenses of fine-grained sand, silt and clay were deposited in lacustrine and flood-plain environments, and are highly carbonaceous, with abundant pyristised wood. Deposition of the Olney Formation followed, with fluvio-lacustrine environments depositing an extensive, lithologically variable blanket of laterally discontinuous, unconsolidated to poorly-consolidated carbonaceous silt, sand and clay, with common interbeds of lignitic coal and peat (Brown & Stephenson, 1991). Occasional interbeds of shallow-marine glauconite calcareous clay, thin bryozoan limestone and minor carbonaceous sand of the Buccleuch Formation suggest minor marine incursions (Twidale et al., 1978; Brown & Stephenson, 1991; Fabris, 2002). Variations in the thickness of the Renmark Group reflect the unconformable onlap of the unit over irregularities in the pre-Cainozoic surface, as well as subtle and localized subsidence.

Ogliocene – Middle Miocene

A marine transgression in the early-Oligocene resulted in the deposition of the marine and marginal-marine formations of the Oligocene to mid-Miocene Murray Group (Figure B.4). The Murray Group sediments form the base of the cliff-forming rocks of the River Murray gorge section. Deep water glauconitic calcareous clay (marl) of the Ettrick Formation form the basal unit of this sequence, and were deposited during the initial marine transgression. As sea-level continued to rise, shallow marine platform limestones (Gambier Limestone, Mannum Formation, Morgan Limestone) were deposited, consisting of coarse-grained skeletal debris, calcareous clay, micrite and quartz sand. The Winnambool Formation was deposited in shallow to marginal-marine platform and lagoonal environments, and is richly fossiliferous (Brown & Stephenson, 1991).

The Winnambool Formation grades laterally into the Geera Clay, consisting of black, locally carbonaceous silt and mud with minor dolomite and sand. This unit was deposited in shallow to marginal-marine platform environments, including extensive interdistributary-bay and tidal-flat settings. Eustatic sea-level fall in mid-Miocene caused the Olney Formation, Geera Clay and Winnambool Formation to locally prograde back over the platform limestones of the Murray Group, followed by a relatively short period of weathering, erosion, mild warping and block-faulting (Twidale et al., 1978; Brown & Stephenson, 1991; Fabris, 2002).

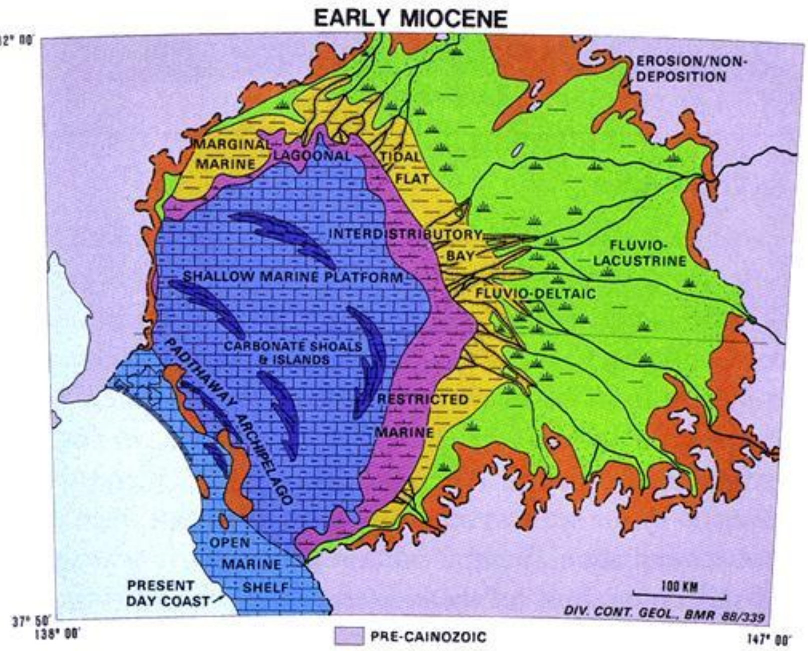

Figure B.4 The Early Miocene paleogeography of the Murray Basin 20 million years ago (From: Evans et al., 1990).

Upper Miocene – Pliocene

Deposition of the last major Tertiary sequence took place in the upper-Miocene to Pliocene during a short-lived marine transgression-regression. The initial transgression led to drowning of fluvial tracts, depositing highly fossiliferous clay and marl of the Bookpurnong Formation in shallow marine-environments. Loxton-Parilla sands were deposited during the final regression of the Pliocene, in prograding beach strandplain and barrier island environments, and underlie much of the western Murray Basin (Ludbrook, 1959, Roy et al., 2000; Fabris, 2002). During this time the Calivil Formation consisting of coarse-grained quartose sand and gravel was deposited in fluvial and fluvio-lacustrine environments. This unit forms an extensive sandsheet underlying much of the eastern and northern Murray Basin. In the late-Pliocene estuarine oyster banks and fossiliferous sands of the North Bend Formation were deposited during a minor subsequent highstand (Ludbrook, 1959, 1961, 1963). This late-Pliocene regression also deposited the fine-grained clastics and polymictic sand and gravel of the Shepparton Formation, by aggradation in the floodplain environment.

A regional marine regression began at the end of the Miocene due to regional uplift of the southern Australian continental margin. This uplift is a consequence of the long-term northerly motion of the Australian continent and the convergence of the Australian and Asian landmasses during the Tertiary. Constant and ongoing uplift of the entire South Australian coastal zone probably began in the late Miocene (about 10 million years ago) and continues to the present day. The configuration of the Australian land surface has gradually changed as a consequence of the convergence of the two plates and this has resulted in a gradual tilting of the entire continent landmass (Sandiford, 2007) with north-western coastal zone subsiding as the southern coastal margin rises. The regional uplift of the South Australian coastal zone occurred as the Australian and south-east Asian landmasses interacted with one another due to the subduction of the Australian tectonic plate beneath the Asian tectonic plate. This uplift enabled the formation of the spectacular Murray River gorges located between Renmark and Monteith, as the ancestral Murray incised into, and removed the bedrock. The confinement of the Murray River within this river-excavated bedrock valley is a direct consequence of this regional continental uplift event; as is the preservation of the spectacular Pleistocene strandplains and dune systems of the south-eastern corner of South Australia (Bourman et al., 2000).

Fault motions on the prominent structural lineaments, such as the Cadell Fault, which were active during the latter stages of the Tertiary period controlled the course of the ancestral Murray River during this time (Steel, 1967), trapping the lower reaches of the river in the present-day course.

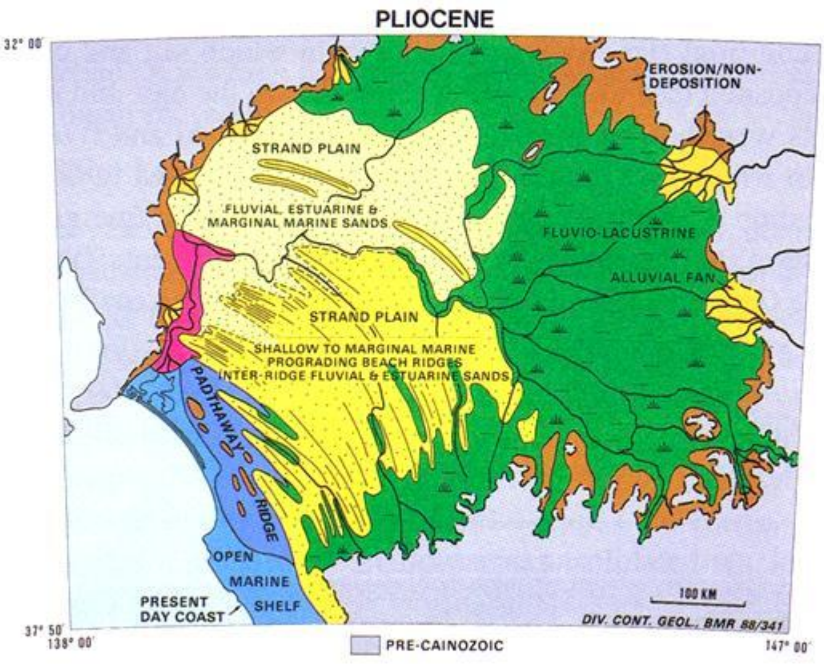

Figure B.5 The Pliocene paleogeography of the Murray Basin 3.5 million years ago (From: Evans et al., 1990).

Quaternary

The deposition of Quaternary sediments in the Lower Murray has been greatly affected by interactions between topography, tectonics, climatic fluctuations, vegetation, groundwater levels, and base level changes of both sea-level and internal drainages, such as Lake Bungunnia. Late Quaternary sedimentation styles and processes have been particularly sensitive to subtle palaeoclimatic and palaeohydrologic fluctuations (Brown & Stephenson, 1991) and the ongoing uplift of the continental mass which has proceeded at a rate of 70 m per million years (Belperio et al., 1995).

Early Pleistocene

During the early Pleistocene, uplift of the Pinnaroo Block along the western margin of the Murray Basin led to the tectonic damming of the Murray River, forming Lake Bungunnia (Gill, 1973, Twidale et al., 1978; Brown & Stephenson, 1991; Fabris, 2002). The fluvial Chowilla Sand occurs at the base of this lake, reworked from underlying Loxton-Parilla Sands (Firman, 1966). Blanchetown Clay was deposited within the shallow Bungunnia Lake, and further shallowing led to deposition of variably dolomitic Bungunnia Limestone (Fabris, 2002). Lake Bungunnia existed under conditions at least twice as wet as those of the Holocene, and climatic fluctuations caused major lake shore migrations, resulting in interfingering of lacustrine Blanchetown Clay and fluvial Chowilla Sand near lake margins (Stephenson, 1986; Rogers, 1995). Along the northwest margin, uplift also increased rates of erosion, depositing the colluvial Pooraka Formation. In the eastern and northern margins, fluvio-lacustrine sedimentation of the Shepparton Formation continued, resulting in the formation of flat Riverine Plain (Brown & Stephenson, 1991).

In coastal areas to the southwest of the basin the early-Pleistocene was a time of marine transgression. Stratigraphic units of the Murray Group were extensively eroded and reworked during this transgression, and the sandy limestone of the Coomandook Formation formed from the debris.

Late Pleistocene – Holocene

In the western Murray Basin, significant changes in sedimentation patterns took place following the demise of Lake Bungunnia, accompanying the onset of modern semi-arid climatic conditions at about 0.4 Ma (An et al., 1986). Lake Bungunnia drained during a period of low sea level when fluvial erosion and downcutting breached coastal barriers, allowing the release of the lake waters. The drainage of Lake Bungunnia fragmented the lake into several smaller lake basins, of which Lake Tyrrell is the largest of these surviving (Stephenson, 1986). Low-lying areas in the bed of the former Lake Bungunnia evolved into modern gypsiferous saline playas, fed partly by groundwater discharge and depositing the Yamba Formation (Firman, 1966, 1973). Glacial aridity activated processes of aeolian dunefields, groundwater discharge lakes, gypsum and clay lunettes and bed-load channel sands. Much of the Murray Basin in South Australia is composed of aeolian landforms dominated by linear and parabolic dunes of quartz sand comprising the Woorinen Formation, Molineaux Sand and Bunyip Sand.

The Woorinen Formation began to accumulate by at least 0.4 Ma, forming east-west trending longitudinal dunes in the northern part of the basin. The dunes were remobilised during a number of past periods of aridity, reflecting late-Quaternary oscillations between arid and semi-arid climatic conditions. Several such phases of remobilisation are recorded by the presence of calcareous palaeosols and calcrete horizons, such as the Ripon and Bakara Calcretes. Molineaux Sand developed but the deflation of the Loxton-Parilla Sands, forming extensive east-west parabolic dunes. Bunyip Sand was deposited as large tongues of irregular linear and parabolic dunes trending east-northeast from the Murray River valley between

Morgan and Swan Reach. The unit also consists source-bordering dunes within the Valley from sand blown out of the Murray River Gorge (Firman, 1966). Despite there being significant tectonic activity in the Pliocene-Pleistocene, the Quaternary has been relatively quiescent; significant movements only established for the Morgan Fault in the western margin of the basin. The Murray Basin subsequently has little relief and low gradient, the Murray River only falling by > 50 mm/km at Mildura and by 16 mm/km closer to the sea (Firman, 1966; Gill, 1973). Consequently, it is thought that drainage of the Murray Basin from the highlands has essentially followed the same path since the mid-Eocene (Macumber, 1978; Stephenson & Brown, 1989), with the form and load of the Murray River changing little over the Holocene (Pels, 1966; Bowler, 1978).

The deep incision of Murray River Gorge and formation of the present-day bedrock valley floor is probably related to both the draining of Lake Bungunnia and repeated oscillation of sea-level due to the cyclic expansion and contraction of the global ice-sheet. Maximum lowering of sea-level during this period was to base levels between 120 m and 150 m below the current sea surface level, and the Murray responded by eroding its base down to these levels which including cutting a channel across the present-day continental shelf offshore from Goolwa (Gingele et al., 2004). The bedrock base of the Murray Gorge is located about 15 m below present-day sea level at Blanchetown and about 65 m below present-day sea level near Murray Bridge (Twidale et al., 1978). During these deep incision events, deposits of river-channel sediment formed at the valley margins and have been identified as the remnant 'cliff-side channels' between Scrubby Flat and Pompoota – their preservation has been suggested to be a consequence of localized steepening of the river bed and the bedforms within them suggest that the river environment was of braided form rather than the meandering form of the present-day channel (Steel 1962, Frahn 1971, Thomson 1975, Twidale et al., 1978). This period of incision exhumed several granitic landforms, which had been buried by Oligo-Miocene marine sediments, for example the river bottom at Murray Bridge and the batholith exposed upstream of Mannum at Younghusband (Sprigg, 1952; Johns, 1960, 1961).

The work of Thomson (1975), Twidale et al (1978) and the results of several engineering site investigations (Fryar & Rowan 1968, Steel 1968) strongly suggest that the Lower Murray River Gorge/Valley was completely stripped of sediment infill during the last Glacial Maximum (~20,000 ybp) and subsequently backfilled firstly with the sands of the Monoman Formation and then the clays and muds of the Coonambidgal Formations. These works suggested that much of this sediment was derived from the Pliocene Loxton-Parilla Sands. The Monoman Formation is commonly referred to as the 'lower valley-fill' and is comprised of coarse-grained, high-energy fluvial deposits and was laid down during a final post-glacial transgression. By ~7,000 years BP, as the sea approached its modern level around the South Australian coast (Belperio et al., 1983), depositional surfaces in the Murray River valley became more stable and forests developed. The top of the Monoman Formation is marked by a buried forest and palaeosol (Gill 1973) which marks a major change in depositional style which is typically considered to be a transition from high-energy deposition of sands typical of a braided river to lower-energy deposition of fine-grained material, i.e. deposition of floodplain muds typical of overbank deposits of meandering rivers such as the lower Mississippi – this interpretation was developed from and is somewhat consistent with the present-day meandering-river planform geometry that characterises the modern river.

Deposition of low-energy flood-plain sediments of the Coonambidgal Formation commenced at ~7,000 years BP, when sea-level stabilized near its present-day elevation, and continues to this present day (Firman 1966, Lawrence, 1966, Firman 1971, 1973, Twidale et al., 1978). Geological cross sections across the Murray River Gorge indicate that sediments of the Coonambidgal Formation are 15 – 25 m thick and sediments of the Monoman Formation are a further 20 – 25 m thick (Ludbrook 1960; Steel 1967; Firman 1973; Twidale et al., 1978). An analysis of the Coonambidgal Formation of the Riverine Plain between Albury and Wentworth led to the assertion that Coonambidgal sediments were deposited by three stream phases over the last 50,000 years as a product of an ancestral tributary stream system of large meandering channels (Pels 1964, 1966) characterised by declining discharges and bedloads (Bowler, 1978; Bowler & Wasson, 1984; Rutherfurd, 1991).

Further downstream, the River Murray mouth region has been described as 'a failed delta' due to the absence of a substantial deltaic complex that might be expected from the deposition of material derived from such a large catchment. Murray-Wallace et al. (2010) have demonstrated that sediment delivered to the mouth of the Murray has probably been incorporated into aeolian deposits such as the sand ridges developed at Narracorte during sea-level highstands, or transported offshore beyond the edge of the Lacepede Continental Shelf during glacial maxima. These authors assert that "*The Holocene and modern River Murray has not established a marine delta, but deposits its load in the settling basins of the terminal lakes, Alexandrina and Albert*". They indicate that a small digitate delta has formed where the river enters Lake

Alexandrina which probably means that the majority of the sediment derived from the catchment during the current sea-level highstand is deposited upstream of the terminal lakes.

Paleoclimates during the late Quaternary indicate increasing seasonal aridity (Figure B.6). A general change from the deposition of fluvial matter to aeolian dust recorded in offshore cores adjacent to the southeastern Australian coast, have been interpreted as indicating a trend towards more arid conditions during the Holocene from 13.5 ka to present. This trend was interrupted by 2 periods of influx of fluvial material from the Murray Catchment between 13.5 – 11.5 ka and 9.5 – 7.5 ka, representing more humid conditions in the southern part of the MDB (Gingele et al., 2007). Although the climate in the region was generally dry between 4 ka and 2 ka (Stanley & Deckker, 2002), there is an indication of varying climate conditions with a humid event recorded in sediments at 2.8 ka BP (Gingele et al., 2007).

Increasing regional climatic aridity would have had hydrologic and geomorphic implications for the ancestral Murray River. Reduced precipitation in the Murray Catchment would have reduced freshwater outflow and constricted the Murray Mouth, possibly closing it to the Southern Ocean. This coincides with an onset of estuarine-lagoonal sedimentation at about 3.5 ka BP recorded in the lower estuary of the River Murray (Cann et al., 2000). During this period it is plausible to assume that the Lower Murray River would have been completely isolated from oceanic sediment input, with river drainage constrained, creating a static environment resembling lagoonal and lake depositional conditions on the Lower Murray River.

An alternative interpretation of the data presented by these authors (Gingele, Murray-Wallace, Cann etc) is the following one which has been developed (Hubble and De Carli in prep) after examination of river-bank cores collected in May 2013 as part of this project. This model posits that the sediment delivered to the Lower Murray since the stabilization of sea-level near its present day level has been trapped upstream of Lake Alexandrina. Hence, the transition of the high-energy sands of the Monoman formation to the low-energy sediments of the Coonambidgal Formation marks a change in nature of the Murray. At this point in time sufficient sand had been delivered to the mouth to contribute to the Coorong barrier complex which effectively closed the Murray River mouth, forming a lake. An event of this type, rather than a prolonged period of aridity, explains the suppression or cessation of the delivery of fine fluvial sediment to the shelf identified by Gingele et al. (2004). Instead, it is strongly suspected that the fine-grained sediment delivered by the Murray and Darling Rivers to the Lower Murray River during the second half of the Holocene has been trapped and deposited in the posited lake which would have been developed between Lake Alexandrina and Renmark. This would explain why there has been little discharge of sediment to the ocean through the river mouth at Goolwa during much of the Holocene. In this scenario it is envisioned that this hypothesized lake, developed behind the dune and strand plain complex of the Coorong Lakes. If the Lower Murray behaved as a lacustrine system rather than a freely discharging river during much of last 7,000 years, then it follows that the sediments of the Coonambidgal Formation would present sedimentary features typical of lakes as well as submerged plants rather than sedimentary structures typical of overbank floodplain deposits and emergent terrestrial plants.

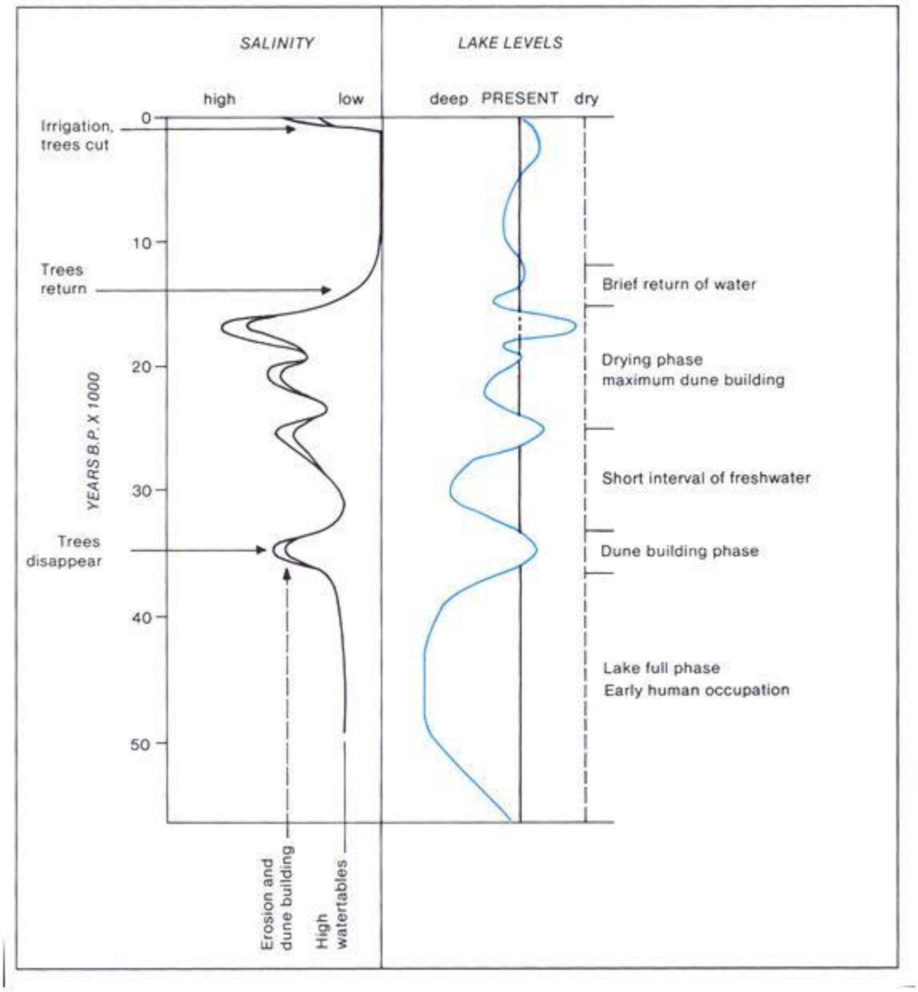

Figure B.6 Generalised record of lake level oscillations in south eastern Australia over the last 50,000 years (From: Bowler 1990).

Appendix C
Event Register

Table C.1 Lower Murray River Incident Register (Source: DEWNR, summarised by E. De Carli).

Location	Easting	Northing	Reported Date	Incident Types	Incident Number
Murray Bridge - Swan Port Bridge	345857	6109356	1/02/2009	Bank Collapse	36
Ngaut Ngaut Conservation Park - Canoe Landing Site	372669	6160837	1/02/2009	Bank Cracking	222
Tailem Bend - Dixon Reserve	359374	6097344	1/02/2009	Bank Cracking	221
Toora - Jaensch Rd	346492	6118095	1/02/2009	Bank Collapse & Cracking	85
Murray Bridge - Long Island	345254	6110931	4/02/2009	Bank Collapse	47
Tailem Bend - Murrayview Community Recreation Reserve	357371	6092978	12/02/2009	Bank Collapse	49
White Sands - Hann Road	347534	6104588	14/02/2009	Bank Collapse	50
Murray Bridge - Long Island	345488	6110748	20/02/2009	Bank Collapse	51
Washpool			21/02/2009	Bank Collapse	99
Mypolonga - Woodlane Reserve	348216	6126137	28/02/2009	Bank Collapse	113
Avoca Dell	345715	6115872	1/03/2009	Bank Collapse	1
Mypolonga - North Bokara Rd	348582	6125775	3/03/2009	Bank Collapse	58
Murray Bridge - Long Island	345427	6110800	4/03/2009	Bank Collapse	52
Mypolonga - Woodlane Reserve	348216	6126137	7/03/2009	Bank Collapse	114
Tailem Bend - Placid Estate	354436	6092376	8/03/2009	Bank Cracking	53
Wellington - Lot 653 Jervois Rd	352876	6089504	17/03/2009	Bank Collapse	100
Monteith - Bells Reserve	346689	6106848	23/03/2009	Bank Cracking	32
Monteith - Bells Reserve	346644	6106821	27/03/2009	Bank Slumping & Cracking	33
Pompoota - Burbidge Irrigation Area	348380	6126267	27/03/2009	Levee Collapse, Cracking, Slumping	65

Table C.1 Lower Murray River Incident Register (Source: DEWNR, summarised by E. De Carli). (Continued)

Location	Easting	Northing	Reported Date	Incident Types	Incident Number
Younghusband - East Front Rd	364273	6139298	27/03/2009	Bank Cracking	118
Ngaut Ngaut Conservation Park	372945	6160746	30/03/2009	Bank Cracking	63
Tailem Bend - Princes Highway	357153	6099301	30/03/2009	Bank Cracking	75
Mannum - Bolto Reserve	346368	6135353	31/03/2009	Bank Cracking	17
Mannum - Bolto Reserve	345818	6135054	31/03/2009	Bank Cracking	17
Murray Bridge - Thiele Reserve	343182	6113981	6/04/2009	Bank Collapse	55
Swan Reach - North of Pump Station	371654	6174658	9/04/2009	Bank Slumping & Cracking	73
Mannum - River Lane	345116	6133287	20/04/2009	Bank Slumping	23
Tailem Bend - Murrayview Estates	357585	6093107	22/04/2009	Bank Cracking	76
Mannum - Caravan Park	346383	6135932	23/04/2009	Bank Cracking	24
Sunnyside - Sunnyside Reserve	350402	6119754	23/04/2009	Bank Cracking	69
Tailem Bend - Freds Landing	358401	6093792	23/04/2009	Bank & Tree Collapse	77
Younghusband - East Front Road (EF5)	363972	6139316	27/04/2009	Bank Cracking	119
Wood's Point - Woods Point Reserve			7/05/2009	Bank Cracking	117
Tailem Bend - Heritage Trail	359267	6094477	11/05/2009	Bank Collapse & Cracking	79
Tailem Bend - Princes Highway, Jervois	357854	6098871	18/05/2009	Bank Cracking	78
Wellington - Jervois Road	352571	6090304	29/05/2009	Bank Cracking	102
Toora - Levee			3/06/2009	Levee Slumping & Cracking	84
Wellington			4/06/2009	Bank Cracking	103
Caloote Landing - North of Residential Area	341496	6129981	5/06/2009	Bank Cracking	276

Table C.1 Lower Murray River Incident Register (Source: DEWNR, summarised by E. De Carli). (Continued)

Location	Easting	Northing	Reported Date	Incident Types	Incident Number
Caloote Landing - Southern Residential Area	341569	6129775	5/06/2009	Bank Cracking	278
Blanchetown - McBean Pound			24/06/2009	Bank Slumping	2
Swan Reach - Upstream of Waste Disposal Station	371654	6174658	16/07/2009	Bank Slumping & Cracking	71
Murray Bridge - Riverfront Road	343974	6111940	18/08/2009	Bank Cracking	104
Walker Flat - Waste Disposal Station	367903	6153646	28/09/2009	Bank Cracking	88
Swan Reach - Mark's Landing	371189	6172082	23/10/2009	Bank Cracking	72
Morgan			15/11/2009	Bank Slumping & Wake Erosion	38
Monteith - Bells Reserve	346644	6106821	16/11/2009	Bank Cracking	34
Wellington	352978	6089776	16/11/2009	Bank Cracking	106
Tailem Bend - 46 Princes Highway	359501	6097014	19/11/2009	Bank Cracking	80
Lake Carlet	365807	6139966	3/12/2009	Bank Slumping & Cracking	13
Murrawong - Kettelty Landing No.16	348323	6119905	9/12/2009	Bank Cracking	46
Walker Flat			13/12/2009	Bank Collapse	89
Wall Flat	346704	6130276	21/12/2009	Bank Cracking, Tree Collapse	97
Mypolonga - Woodlane Reserve	348099	6126236	26/12/2009	Bank & Tree Collapse	116
Neeta			28/12/2009	Levee Cracking	16
Walker Flat - Rob Loxton Road	369139	6153009	5/01/2010	Bank Cracking	91
Mannum - Bolto Reserve -Shack 29	346324	6135097	7/01/2010	Bank Cracking	28
Murray Bridge - Longflat Rd	344320	6111981	14/01/2010	Bank Cracking	56
Mypolonga - North Levee			15/01/2010	Levee Cracking	252

Table 4: Lower Murray River Incident Register (Source: DEWNR, summarised by E. De Carli). (Continued)

Location	Easting	Northing	Reported Date	Incident Types	Incident Number
Mypolonga - Rivergum Drive	350044	6119751	15/01/2010	Bank Collapse & Cracking	59
Mypolonga - Woodlane Reserve	349132	6125247	15/01/2010	Bank Collapse	60
Murray Bridge - Mobilong Levee	342657	6113073	20/01/2010	Levee Cracking	251
South Punyelroo	372757	6168794	21/01/2010	Bank Cracking	67
Murray Bridge - Long Island	345445	6110780	25/01/2010	Bank Collapse	57
Morgan - Shack 153 Scotts Creek			29/01/2010	Bank Collapse	42
Walker Flat - SAB Aruma Caravan Park	366154	6154928	23/02/2010	Bank Collapse	94
Tailem Bend - Princes Highway	359501	6097014	5/03/2010	Bank Cracking	11
Teal Flat - Shack 49 - 50	367715	6138044	9/03/2010	Tree Collapse & Bank Cracking	86
Mypolonga - Mypolonga North	349051	6125306	7/04/2010	Bank Cracking	61
Sunnyside - Sunnyside Reserve	350679	6119950	12/05/2010	Bank Collapse, Cracking & Tree Leaning	213
Blanchetown - Upstream			19/05/2010	Bank Cracking	215
Walker Flat	369220	6158393	20/05/2010	Bank Cracking	214
Mannum - Noah No Landing	350278	6138656	25/05/2010	Bank Cracking	217
Mypolonga - Rivergum Drive	350049	6119761	8/06/2010	Bank Collapse	219
Bowhill - Providence Road	373502	6138937	3/08/2010	Bank Cracking	211
Sunnyside - Sunnyside Reserve	350679	6119950	19/08/2010	Bank Collapse & Erosion	244
Punyelroo - Roys Landing			26/08/2010	Bank Cracking	220
Murray Bridge - Long Island Reserve			13/09/2010	Tree Collapse on Levee Bank	226
Mypolonga - Jury Reserve			16/09/2010	Bank Slumping	239
Ponde - Levee			20/09/2010	Levee Slumping & Cracking	231

Table C.1 Lower Murray River Incident Register (Source: DEWNR, summarised by E. De Carli). (Continued)

Location	Easting	Northing	Reported Date	Incident Types	Incident Number
Jervois - Craton Lane Levee			22/09/2010	Levee Slumping	232
Murray Bridge - Riverglen Marina			29/09/2010	Bank Cracking	235
Wood's Point - Jervois Levee	354844	6100991	13/10/2010	Levee Breach & Cracking	257
Pompoota - Burbidge Irrigation Area	350268	6124338	14/10/2010	Levee Breach	262
Ponde - Neeta Levee			14/10/2010	Levee Cracking	253
Walker Flat	369093	6153005	22/10/2010	Bank Slumping	269
Walker Flat	369168	6155519	22/10/2010	Bank Erosion, Tree Leaning	268
Pompoota - Irrigation Area			1/11/2010	Levee Slumping	272
Pompoota - Levee			8/11/2010	Tree Collapse	274
Mypolonga - Woodlane Reserve	348135	6126227	17/11/2010	Bank Cracking	280
Swan Reach - near Pump Station	368716	6180863	17/11/2010	Bank Cracking	279
Morgan - Shack 153 Scotts Creek			22/11/2010	Bank Cracking	282
Wongulla - Devon Downs North Wetland	376725	6164154	26/11/2010	Bank & Tree Collapse	284
Murray Bridge - Long Island Reserve	344461	6111451	6/12/2010	Bank Cracking	285
Blanchetown - McPhee Road	373094	6197383	11/12/2010	Bank Cracking	289
Mannum - 144 Riverlane			24/12/2010	Bank Cracking	295
White Sands - Woods Point, Private Levee			6/01/2011	Levee Cracking & Erosion	297
Murray Bridge - Mobilong Levee			21/02/2011	Levee Breach	301
Jervois - Levee			24/02/2011	Levee Collapse/Erosion	304
Mannum - East Front Road (EF3)	349896	6137756	2/03/2011	Bank Cracking & Infrastructure damage	305

Table C.1 Lower Murray River Incident Register (Source: DEWNR, summarised by E. De Carli). (Continued)

Location	Easting	Northing	Reported Date	Incident Types	Incident Number
Swan Reach - Waste Disposal Station	371654	6174658	25/03/2011	Bank Cracking	306
Mannum - East Front Road (EF3)	349896	6137756	30/03/2011	Bank Cracking & Infrastructure damage	307
Morgan - Shacks 16-18, Brenda Park	377947	6229778	6/06/2011	Bank Collapse	309

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