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VE23346.10

Dear Richard

River Bank Collapse Hazard - Lower River Murray
Stability Risk Assessment for Caloote Landing

1. General

DWLBC (now the Department for Water (DFW)) commissioned SKM in late 2009 to investigate the stability of river banks at seven sites along the lower pool of the River Murray. This investigation was followed by additional studies in 2010 relating to the effects of pool water level change on stability of the riverbanks and ongoing validation of the conditions found in the initial geotechnical investigations undertaken by SKM.

Our responses included advice to local government and the communities along the lower pool (below Lock 1) via DWLBC, with regard to potential future river bank collapse and options for monitoring and exclusion measures such as signage and temporary fencing of river bank areas considered at the time to be susceptible to collapse or with factors of safety less than the industry accepted value of 1.50.

Recent inspection of the investigated sites by DFW show temporary fencing and bunting cut, damaged and removed by users of the facilities. This is clearly demonstrating community concern and frustration over the restrictions to riverbank access and use posed by the temporary fencing, especially at sites where no cracking or collapse has occurred and the perception that rising water levels signal the riverbank collapse risk is passed.

This is especially so at sites such as Caloote where there is a community boat ramp with associated riverbank amenities and no cracking is evident in the boat ramp area and carpark/picnic area landside of the local residents “jetties” (**Figures 5 and 6 attached**). The use of bunting and collapse warning signs as public safety mitigation, especially where it is visibly damaged, may be seen as sending a mixed message that the site is not a serious risk to safety.

This letter details our risk based study of the riverbank stability on public safety at Caloote Landing in order to develop a risk based management tool for decision making for this site, as commissioned on 9 Sep 2010. The previous version of the letter has been peer reviewed and clarifications responding to the peer review included in this version of the report.

2. Risk Based Stability Assessment

2.1 Risk Assessment

According to "*Guideline for Landslide Susceptibility, Hazard and Risk Zoning for Land Use Planning*" AGS (2007a), *Hazard* is a condition with the potential for causing an undesirable consequence. The description of hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material and the probability of their occurrence within a given period of time.

Risk is a measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability and consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form. Two major risks are:

- **Loss of Life**, the annual probability that the person most at risk will lose his or her life taking account of the landslide hazard and the temporal spatial probability and vulnerability of the person.
- **Loss of Property**, the annual probability of the consequence or the annualised loss taking account of the elements at risk, their temporal spatial probability and vulnerability.

For the purpose of this study, which relates largely to recreational use of the Caloote landing site, only loss of life was considered.

The Australian Geomechanics Society (AGS 2007) provides guidance on the estimation of landslide risk to life. Section 7 of the "*Practice Note Guidelines for Landslide Risk Management 2007*" provides an equation for calculating the individual risk for loss of life, which is reproduced below:

$$\mathbf{R}_{(LoL)} = \mathbf{P}_{(H)} \times \mathbf{P}_{(S:H)} \times \mathbf{P}_{(T:S)} \times \mathbf{V}_{(D:T)}$$

where:

$\mathbf{R}_{(LoL)}$ is the risk (annual probability of Loss of Life of an individual)

$\mathbf{P}_{(H)}$ is the annual probability of the hazardous event (river bank collapse due to river water level change)

$\mathbf{P}_{(S:H)}$ is the probability of spatial impact by the hazard given the event

$\mathbf{P}_{(T:S)}$ is the temporal probability of affecting a person given the spatial impact

$\mathbf{V}_{(D:T)}$ is the vulnerability of the individual (probability of loss of life of the individual given the impact)

SKM has used a similar approach to AGS, where:

$\mathbf{P}_{(H)}$ is covered in Section 4.7 (**Table 3**) and Section 5.2. $\mathbf{P}_{(S:H)}$ is explained in Section 5.3. $\mathbf{P}_{(T:S)}$ and $\mathbf{V}_{(D:T)}$ are discussed in Section 5.4.

According to AGS 2007c guideline, it is important to distinguish between "acceptable risks" and "tolerable risks".

- **Tolerable Risks** are risks within a range that society can live with so as to secure certain benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if practicable; and
- **Acceptable Risks** are risks which everyone affected is prepared to accept. Action to further reduce such risk is usually not required unless reasonably practicable measures are available at low cost in terms of money, time and effort.

2.2 Probabilistic Analyses

Traditional (deterministic) slope stability analysis, as applied to river bank collapse assessments by SKM, uses single value estimates for each variable in the slope stability equations. The variables used for slope stability analysis are the physical characteristics of the soil and the slope geometry. The output of a traditional stability analysis is a single-value deterministic estimate of whether the slope will stand or collapse.

The ability of geotechnical engineers to accurately model slope performance is compromised by a variety of factors. These may be broadly classified as theoretical and practical considerations. Theoretical considerations include approximations and assumptions made for model development. Practical considerations are the natural variation of the soil layers such as slope, thickness, soil strength and groundwater level. Furthermore, it is not practical to have a large number of undisturbed samples and laboratory and in-situ tests for each site to statistically determine the parameters to be used in slope stability analyses.

Considering the variability of input parameters, Factor of Safety is a number indicating the relative stability of a slope and does not imply the actual risk level of the slope. The probability of failure and the reliability index can quantify the stability or the risk level of a slope.

The probability of failure is the probability of obtaining a factor of safety less than 1.0 in stability analyses using Monte Carlo trials method. There is no direct relationship between the deterministic factor of safety and probability of failure and a slope with a higher factor of safety may not be more stable than a slope with a lower factor of safety.

The reliability index describes stability in terms of the number of standard deviations between the mean factor of safety and the defined failure value of 1.0 (normalizing the factor of safety with respect to its uncertainty). The relation between reliability index and the probability of failure is known for a normal distribution.

3. Caloote Landing

3.1 Site Description

Caloote is located on the south bank of a tight meander bend of the River Murray. A boat ramp, foreshore and car parking area is located at this site, known as Caloote Landing. The area around Caloote Landing was formerly a swamp (lagoon), which was partly reclaimed in the 1930's/1940's. The quality of the materials and construction is unknown; however, it is expected that the local materials from the surrounding area have been used to provide uncontrolled fill to reclaim the lagoon area.

The first reports of cracking and movement in the area are as early as February 2008. Domenic Priolo, who owns one of the residences downstream of the Landing area, first noted movement around his doorways and cracking in his residence. Cracking associated with settlement of underlying soils continued through 2009.

Tension and settlement cracks run through properties downstream of the landing area and there is evidence of slumping at the front of the houses. Furthermore, long tension cracks are evident in the northern area of the site which resulted in change in the alignment of an overhead power line. The alignment of the cracks and slumping appear to approximate the western bank of the part infilled lagoon.

3.2 Site Geology and Subsurface Conditions

In SKM's geotechnical report (2010), the Caloote site is mapped as Quaternary grey fluvial silts, sands and gravels. The site is also mapped close to the boundary of the Quaternary Hindmarsh clays and the Tertiary Mannum Formation. The Caloote landing site straddles the west bank of a lagoon or wetland that has been partly filled to form carpark area and access track to the boat ramp and riverbank, where the underwater profile drops steeply into the river channel.

The subsurface profile comprised of Silty/Gravelly SAND underlain by Silty CLAY. The layer of Silty CLAY was typically encountered at 1m to 1.5m below ground level, and was generally very soft and wet. The thickness of the very soft Silty CLAY layer varies from 3m to 15m. There are some sandy clay/clayey sand layers/lenses in very soft silty clay layer from 6 to 15m depth. Beneath this layer, the sand content generally increased, transitioning to a Clayey SAND and Gravelly SAND. One borehole terminated at 4m, due to refusal on SANDSTONE, confirming presence of the Mannum Formation.

3.3 Discussion of Riverbank Issues at Caloote

The Murray River levels have dropped approximately 2m since 2005 along the 200km stretch of the lower pool of the river south of Blanchetown (Lock 1). Pool level is normally at a high of ~+0.75mAHD and has dropped to as low as -1.2m AHD during 2009. (It is noted that wind seiche in Lake Alexandrina and along the lower pool can raise water level by as much as 0.65m and lower it by about 0.15m over short periods of time – typically less than a day.) Pool level has returned to normal at the time of writing this report.

The general lowering of pool level has led to groundwater levels lowering adjacent to the river bank which has the following adverse affects on the stability of the banks:

- Surcharging of the riverbanks by about 20kPa due to loss of buoyancy associated with lowered pool level of about 2m;
- Subsidence due to the lowering of the groundwater level has led to surface settlement over the last 4 years and into the future, with some time delay effects due to the rate of dissipation of the pore water pressures after drawdown;
- Tilting of the soil blocks towards the river due to differential subsidence;
- Tension at the crest of the slope due to tilting;
- Shrinkage cracks increasing to the depth of the water table due to drying of the reactive clays that have been exposed to surface temperatures and evaporation to a greater depth.

The seasonal moisture zone in the river Murray area is approximately 4m deep so, as the groundwater table drops, so the shrinkage cracks follow.

- Shrinkage cracks become tension cracks that fill with surface water and drive failure from the crest when it rains. Tilting increases the width of shrinkage cracks in the direction of the tilting.
- Tree roots exaggerate the shrinkage effects in reactive soils and if the root system cannot follow the falling groundwater level then the tree will become brittle and fall. A row of trees may cause instability at the edge of the river if there is a vertical bank along which a row of trees is subject to high winds leading to an over-turning moment. This may also apply to trees already leaning over the bank, especially Willow trees which characteristically have large volume of leaves, large proportion of which float on the water surface. When water level drops, these leaves exert large toppling force on the tree and the adjoining riverbank. A toppling failure can occur immediately behind the row of trees with the minimum volume of soil in the slide mass to resist the failure.

Results of geotechnical investigations at the Caloote site indicate that the factors of safety against slope failure at adopted typical pool levels of 0.0m and +0.7m AHD in the carparking/foreshore area, around the boat ramp and the residential area immediately to the south, are about 1.25 and 1.35 respectively (Ref SKM Letter report VE23346.08 dated 28 Jun 2010), i.e. below the FoS that would normally be considered acceptable for permanent areas used by the public (FoS = 1.5).

Furthermore, stability analysis results indicate that reduction in pool to -1.5mAHD level could trigger failure (FoS = 1.06).

The houses downstream of the landing area lie beneath a cliff overhang. Tension cracks developed in the cliffs and the nature of the overhangs means that toppling/cantilever failure can result in blocks landing on the houses.

3.4 Previous Recommendations

It had been recommended (SKM 2010) that the residents in the south eastern area should be advised that there was a high risk of bank failure and rock fall.

No immediate damage to the structure of the riverside properties in the north western area was expected; however, the road and car park areas were assessed to be high risk.

It had been recommended that access to the boat ramp, parking area adjacent to river and residential area in the south eastern areas and the access road after the residential area in the north western side should be “roped or fenced” off to stop public and vehicle traffic from approaching near the riverbank. River traffic was warned of the very high probability of failure and advised not to moor in the area.

These actions were implemented in the summer of 2009/10 and have been in place since with public regard for the restrictions falling away as water levels rose to the current normal pool level as described in the introduction to this report.

4. Stability Assessment

The stability analysis was undertaken using the limit equilibrium software GeoStudio Slope/W (version 7.17). A representative cross section was generated for the Caloote site from survey data obtained by SKM as part of the 2009 investigation into riverbank instability on the Lower River Murray.

The stability of the site was analysed using the probabilistic method of analysis described in Section 2.2 above. Variations in the following site conditions were considered:

- Soil strength parameters;
- River water level;
- Soil layer thickness and alignment;
- Geometry (riverbank slope);
- Loading; and
- Pore water pressure.

Each of these components is discussed in the following sections.

4.1 Soil Parameters

4.1.1 Selection of N_{kt} Values

The parameter N_{kt} is a factor relating cone resistance from a CPT test to undrained shear strength. N_{kt} values were calculated based on vane shear test results in boreholes adjacent to CPTs.

- Vane shear results where the ratio of undisturbed to remoulded strength (sensitivity) was outside of a reasonable range (taken as 0.18 to 0.32) were not considered.
- The remaining undisturbed vane shear strengths (s_{uv}) were adjusted for plasticity (average PI of 54) by applying a Bjerrum correction factor of 0.76.
- For each vane shear test, an average value of cone resistance (q_t) over a range of 0.2m was obtained from adjacent CPT(s).

- N_{kt} was calculated for each vane shear test by dividing the average q_t by the corrected undisturbed vane shear strength.
- One anomalously high N_{kt} value was not considered further as it is considered that sample disturbance may have resulted in a non-representative vane shear strength reading.
- The 95% confidence interval on N_{kt} was calculated from the remaining data assuming a Student t-distribution.
- As a result, an N_{kt} of 17.5 ± 4 was adopted (95% confidence range of 13.5 to 21.5).

4.1.2 Selection of s_u Values for Type B Clay Layer

The previous study (SKM, 2010) suggests that the overall stability of the slopes is dependent on the undrained shear strength (s_u) of the soft clayey layer. The procedure for adopting the soil strengths for the probabilistic analyses in this risk assessment is as follows:

- q_t data were obtained from the four CPTu performed at Caloote by BIT.
- The data were filtered to exclude the overlying Type A fill layer and underlying Type C sandy material. Data near the top and bottom of the Type B layer which may have been affected by the adjacent material were excluded.
- The data were filtered to remove any local spikes in the profiles which may have represented local lenses which would not contribute to the global strength of the layer.
- The filtered q_t data from each CPT were plotted against depth. A linear regression model was applied, and the coefficient of determination (R^2) was calculated for each CPT.
- The R^2 values ranged from 0.89 to 0.98 indicating a good fit to a linearly increasing strength with depth model.
- The q_t data from the CPTs were merged to create a representative profile for the site. An R^2 value of 0.97 was obtained.
- N_{kt} value of 13.5 was applied to obtain the statistical upper bound.
- **Figure 4** attached presents the undrained shear strengths versus depth at the Caloote site from CPTs, Dilatometers, Unconsolidated Undrained Triaxial tests and Vane Shear tests. This figure was adapted from SKM (2009), with the CPT strength profiles updated to incorporate the N_{kt} values of 13.5 discussed above (The figure was previously presented with typical N_{kt} values of 15 to 20).
- A lower bound Normally Consolidated (NC) strength profile was plotted, assuming a typical s_u/σ'_v ratio of 0.22 and a soil unit weight of 16kN/m^3. This was seen to approximate a good lower bound representation of the data.
- The average profile was plotted between the lower and upper bound strength profiles. This line appears to be a good representation of the lower quartile strength from CA-CPT3s.
- The following profiles for expected soil shear strength were obtained and shown on **Figure 4** according to the following formulae, where z is depth below ground level (noting that the Type B layer starts from a depth of approximately z = 1.8m in the slope stability model.):

Parameter Range	Undrained Shear Strength *(kPa)*
Lowest Expected shear strength	s_u = 1.3 z *(chain dotted red line)*
Average shear strength	s_u = 1.3 z + 5.5 kPa *(solid red line)*
Highest Expected shear strength	s_u = 1.3 z + 11 kPa *(dotted red line)*

- These shear strength profiles were assumed to extend beyond the termination depths of the CPTs to the bottom of the Type B layer in the slope stability models.

4.2 River Water Level

Three water levels which we understand represent the range which might reasonably be expected in the lower Murray pool were considered.

- **+ 0.7m AHD**: represents the typical or normal pool water level of around +0.75mAHD;
- **0.0m AHD**: represents, historically, about the low river level in drought conditions prior to construction of barrages in the lower lakes;.
- **-0.8m AHD**: represents the low water level measured by SKM during the 2009 site investigations, following a prolonged period of low river inflows.

The current lower pool water level (November 2010) is just above normal pool level.

4.3 Soil Layer Thickness and Alignment

The site investigation locations were restricted to areas accessible safely and non-destructively by the drilling and CPT rigs. As a result, the closest investigation locations vary along the site from between approximately 10m and 55m inland of the top of the riverbank.

Therefore, while the sub-surface profile in from the riverbank is generally well defined at the section locations, the profile towards and into the river is unknown. Geotechnical judgement must be exercised in inferring the most probable and critical profile.

In the case of this site, the extent of the Type B very soft clay layer is critical to the slope stability. Three sub-surface profiles were considered.

- Clay layer extending horizontally from the onshore investigation locations to the riverbank and below the river;
- Uniform clay layer thickness across the model. That is, the bottom of the clay layer follows approximately the same shape as the land and riverbank profile at depth;
- Clay layer steeply increasing in thickness towards the river.

Each of these profiles was initially assessed with all other variables held constant. It was found that the critical slip surface depth did not increase in depth when the thickness and slope of the clay layer was increased, due to its already relatively high thickness.

As such, the probability of failure was shown to be insensitive to the range of soil layer thickness and alignment considered in this study.

4.4 Geometry (Riverbank Slope)

The riverbank profile was surveyed by SKM in 2009 along three sections (CA1, CA2 and CA3) and presented on **Figure 5** attached. The sections showed variations in slope angle and maximum river bed depth, as summarised in **Table 1**.

- **Table 1 - Summary of Riverbank Section Geometry**

Survey Section (Land Use Zone)	Riverbank Slope	Maximum River Bed Depth at Section
CA1 (Boat Ramp)	31 degrees	-15.8m AHD
CA2 (Shelter Area)	29 degrees	-14.3m AHD
CA3 (Northern Residential Area)	24 degrees	-14.0m AHD

The site was considered in four zones representing different land use and associated activities. The above survey sections as established in initial site investigations in 2009 were given a weighting as to the probability of being representative of each zone, with the likelihood of influence of each section type for each zone adding to unity.

The general extent of the land use zones is indicated on **Figure 5** and the weighting of each section to each zone is summarised in **Table 2**. The spatial probabilities of influence of the sections at each zone are presented in this table and the effect of frontage has been considered in the likelihood values.

- **Table 2 - Likelihood of Survey Sections for Geometry of each Land Use Zone**

Land Use Zone	Likelihood of CA1	Likelihood of CA2	Likelihood of CA3	Sum
Northern Residential Area	0.00	0.50	0.50	1.00
Car Park and Picnic Shelter Area	0.01	0.70	0.29	1.00
Boat Ramp Area	0.20	0.70	0.10	1.00
Southern Residential Area	0.50	0.50	0.00	1.00

4.5 Loading

A general uniform load of 2.5kPa was previously adopted to represent the permanent loads from structures and temporary loads from vehicles. This is equivalent to the weight of approximately 0.15m of soil (above groundwater) which is less than the effect of variation in the groundwater level which exists across the zones.

4.6 Pore Water Pressure

No evidence of non-hydrostatic pore water pressure was evident from the CPTu data. As such, pore-water pressures are defined by the piezometric lines representing the river water levels, and no additional analysis is required.

4.7 Probability of Failure

The schematic event tree for the probability of failure has been presented in **Figure 1** below.

The slope stability models were run for each of the three riverbank slope geometries. The probability of failure of each section was then weighted by the likelihood factors presented in **Table 2** and summed to obtain the probability of failure in each zone (with each water level considered separately).

The results of the analysis are summarised in **Table 3** in terms of probability of failure in the event of the river water levels considered.

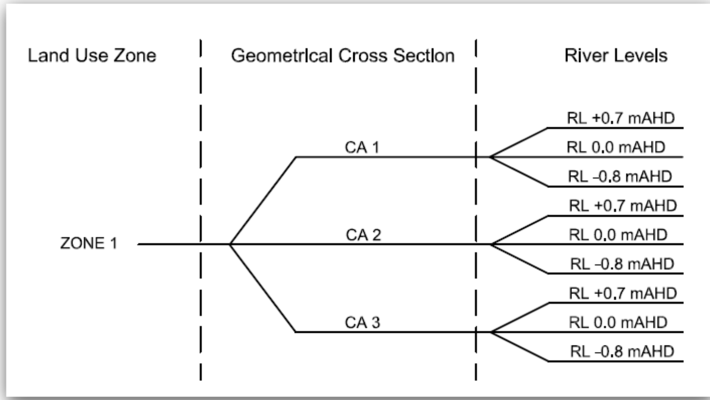


Figure 1 - Schematic Event Tree for Probability of Failure

- **Table 3 - Probability of Failure in each Zone for Given Water Levels**

Zone	River Water Level *(mAHD)*	Representative Factor of Safety	Probability of Failure
Northern Residential Area	+ 0.7	1.48	$< 1 \times 10^{-5}$
	+0.0	1.38	6×10^{-4}
	- 0.8	1.25	3×10^{-3}
Car Park and Picnic Shelter Area	+ 0.7	1.48	8×10^{-5}
	+0.0	1.37	1×10^{-3}
	- 0.8	1.23	7×10^{-3}
Boat Ramp Area	+ 0.7	1.42	2×10^{-3}
	+0.0	1.32	9×10^{-3}
	- 0.8	1.19	7×10^{-2}
Southern Residential Area	+ 0.7	1.33	4×10^{-3}
	+0.0	1.24	2×10^{-2}
	- 0.8	1.12	2×10^{-1}

5. Risk Assessment

5.1 Hazard

Failure of the river bank due to change in the river water level has been considered as the main hazard, given the underlying low soil strengths that have existed at this site since the river was laid down. Other hazards such as earthquake, flood or rock fall have not been included in this study.

5.2 Annual Probability of River Level Change

The water level at Caloote Landing and lower pool sites is highly dependent on human activities and the management decisions of the river regulators for flows from pools upstream of Lock 1 and water levels in Lake Alexandrina. Therefore, the lower pool water level may not follow expected seasonal natural patterns of a river.

Due to the water level conditions above, the annual probability of water level change has not been considered for the risk assessment ($\mathbf{P}_{(H)}$). Risk at each zone estimated at three water levels of RL+0.70m AHD, RL+0.00m AHD and RL-0.80m AHD and Probabilities of Failure from the previous section have been used as the Annual Probability of Failure ($\mathbf{P}_{(H)}$).

It is understood that DFW propose to manage water levels in the lower pool such that level does not go below 0.00m AHD in future. This may not be within the ability of anybody to achieve in fact so, in the event pool level goes below 0.00m AHD, the Caloote Landing site will need to be closed pending reassessment.

5.3 Spatial Probabilities

It is assumed that the entire area may fail in case of any landslide ($\mathbf{P}_{(S:H)}$=1). The spatial variability may reduce the probability of the failure at each area by factor of 1 to 5 ($\mathbf{P}_{(S:H)}$=0.20 to 1.00) which does not change the results of the risk assessment i.e. the risks are still not acceptable. For more accurate analyses, additional geotechnical information at each zone and potentially 3D analyses are required.

For $\mathbf{P}_{(S:H)}$, as there is no information on the structural stability of the houses in the residential areas, it is assumed that if a house is within the failure zone when slope failure occurs, it will collapse. We note existing settlements in the front veranda section of the residences in the southern residential area have resulted in outward movements of the bases of the posts and cracking of paving but without veranda collapse to date.

5.4 Population at Risk

No information on the recreational population of the site which may use the Caloote Landing area is available. The following assumptions have been made for the Population At Risk (PAR) for each area ($\mathbf{P}_{(T:S)}$).

Table 4 Population at Risk for Caloote Landing - $P_{(T:S)}$

LAND USE AREA	NUMBER OF PEOPLE	DURATION PER PRESENCE - h	FREQUENCY PER YEAR -d	POPULATION AT RISK
Northern Residential	2 + 4	24 + 24	365 + 129	2.0 + # 4 1.4
Southern Residential	2 + 6	24 + 24	365 + 129	2.0 + # 3 1.1
Car park and Shelter	16	6	129	1.4 # 2
Boat Ramp	4	1	365	0.2 # 1

It is assumed that boat ramp use can be restricted such that only one vehicle/trailer uses the boat ramp at a time with typically up to 4 people associated with boat launching, or as otherwise appropriate for safe launching. The PAR numbers are acknowledged to be low, and deliberately so. SKM has used the minimum population figures to show that even with the minimum figures, the site is high risk.

The PAR assumptions above have been confirmed as reasonable by Mid Murray Council.

The likelihood of Loss of Life during slope failure ($\mathbf{V}_{(D:T)}$) has been considered as 1.00 for this study. If a house is within the failure zone in the event of a slope failure, it will collapse and occupants are vulnerable.

5.5 Tolerable and Acceptable Risks

According to "*Guideline for Landslide Susceptibility, Hazard and Risk Zoning for Land Use Planning*" AGS (2007a), **Tolerable Risk** for Loss of Life for the person most at risk for the existing slopes and developments is **10^{-4} per annum** (or 1.E-04) while **Acceptable Risks** are usually considered to be one order of magnitude lower than the Tolerable Risks (**10^{-5} per annum** (or 1.E-05) in this study).

"*Guidelines on Risk Assessment*" of Australian National Committee on Large Dams, uses the Population At Risk approach. Refer to the revised ANCOLD societal risk guideline for the existing dams; probability of failure below certain level is tolerable only if they can satisfy the ALARP (As Low As Reasonably Practicable) principles.

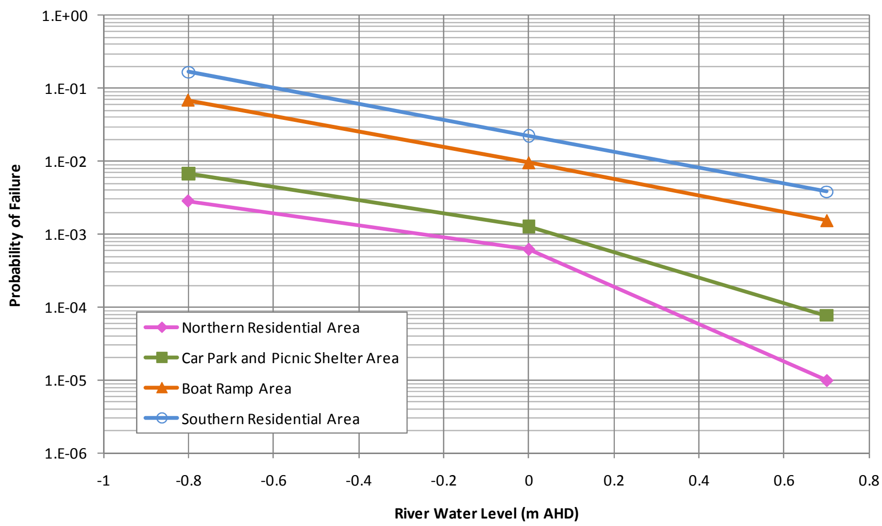
For the purpose of this study, the ANCOLD approach was considered more appropriate as it takes into account the number of people at risk and duration of the presentation at site for each zone.

Three Risk Zones have been considered applicable for consideration across the land use areas of the site, as shown in **Figure 3** below:

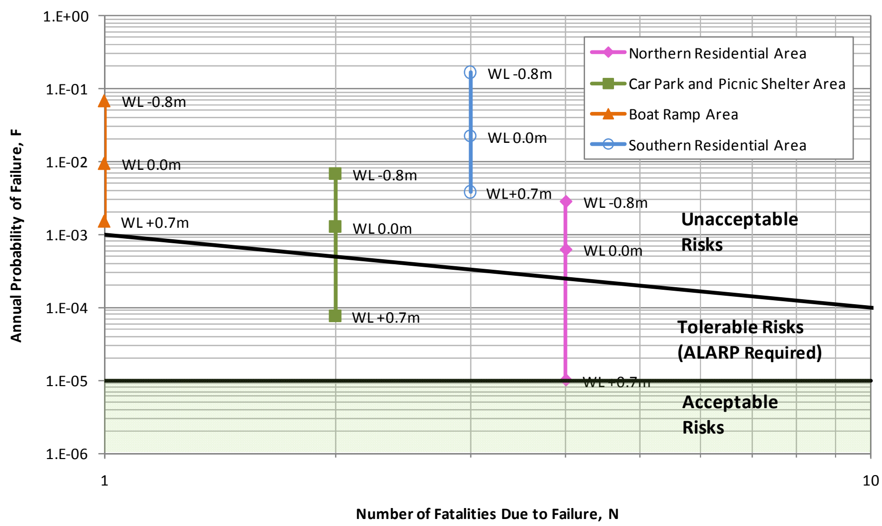
- **Risk Zone 1:** with Probability of Failure above the Tolerable Risk line for which the risks are considered not tolerable;
- **Risk Zone 2**: with Probability of Failure below the Tolerable Risk and above the Acceptable Risk line for which ALARP study is required; and
- **Risk Zone 3:** with Probability of Failure below the Acceptable Risk line for which the risks are acceptable.

5.6 Risk Assessment Results

The results of the study have been summarised in **Figures 2 and 3** below**,** where it must be appreciated that the boundary lines between Risk Zones are indicative only.



■ **Figure 2 - Probability of Failure and River Water Levels**



■ **Figure 3 - Comparison of Probability of Failure for Different River Water Levels at each Area with Tolerable and Acceptable Risks**

From **Figure 3**, it can be seen that for pool level +0.7m AHD, the car park, picnic shelter and northern residential area all fall within the tolerable risk zone and the boat ramp area is just above the tolerable risk boundary.

6. Discussion and Conclusion

The conclusions drawn from this study are as follows, based on satisfying normal developmental levels of safety as required by Australian Standards and the Building Code of Australia:

- Probabilistic analyses for each area of the Caloote Landing Site have been carried out. Sensitivity of the factors of safety to following parameters has been established to be as follows:
  - Undrained Shear Strength of Soft Clay — Sensitive
  - Geometry of the Slope — Sensitive
  - River Water Level — Sensitive
  - Thickness and alignment of the Clay Layer — Not Sensitive
  - Loading — Not Sensitive
  - Pore Pressure in Soft Clay — Not Applicable
- The probability of failure at each land use zone increases with decrease in river water level.
- The risk of failure at the Southern Residential Area is rated on **Figure 3** as <u>unacceptable at all pool levels</u> and the area should remain closed to the public as well as residents.
- The risk of failure at the Boat Ramp borders tolerable on **Figure 3** at pool level 0.7m AHD and rates as unacceptable at lower water levels. It is considered prudent that a practical form of local load testing be undertaken, using river water to apply a surcharge of water at depth 1m (total 10kPa) to simulate occupation loads of say 2.5kPa at pool level 0.0m AHD, in advance of formally reopening the boat ramp. Surcharging does not change the risk assessment described herein.
- The risks of failure at Car Park, existing Picnic Shelter and Northern Residential Areas are rated not acceptable for pool levels below RL 0.0m AHD at Caloote.
- The risks of failure at Car Park, Picnic Shelter and Northern Residential Areas are tolerable for pool levels above RL+0.0m AHD at Caloote. Mitigation requires application of ALARP principles. A suggested site design for managing access in these areas is shown in **Figure 6** attached.
- It is understood that assumption of $\mathbf{P}_{(S:H)} = \mathbf{P}_{(T:S)} = \mathbf{V}_{(D:T)} = 1.00$ may be too conservative; however, **Figure 4** indicates that even by reducing risk by an order of 10, the recommendations of this report will not change.
- The pool level in the study area along the River stands about 50mm (0.05m) above the water level in Lake Alexandrina under "normal" flow conditions, so if water level in Lake

Alexandrina is used as the pool level, trigger levels at the lake should be 50mm lower than actual level at Caloote, neglecting wind effects.

- Short term (~daily) water level changes induced by wind are expected at lower pool sites to be less than +0.65/-0.15m from average water level. Therefore, for management considerations, pool level of RL+0.15m AHD and falling at Caloote theoretically triggers unacceptable risk at the Carpark and Northern Residential areas at this site. This level has been proposed as the trigger level for site closure in development of the site management plan, but for simplicity of communications with the public, based on the tolerable risk boundaries and soil parameters modelled being indicative, a pool level 0.0m AHD has been adopted.

Figure 5 attached is a plan of land use areas and tolerable risk boundaries drawn as lines across the Caloote Landing site for water levels at 0.0mAHD and -0.8m AHD, from the Northern Residential Area to the Southern Residential Area, as discussed above. We note the water level at present (Oct 2010) is near normal pool level of +0.7m AHD and for this water level the tolerable risk boundary is at the water line for all except the Southern Residential Area, where the boundary is just behind the residences, effectively at the foot of the cliff.

The above conclusions need to be considered by DFW in the context of the history of public use of the riverbank area at Caloote Landing, and accepted understanding of riverbank stability as discussed herein and in previous reports.

Historical information indicates that much of the carpark and riverbank at the boat ramp must be on fill placed over the years in the lagoon. The riverbank area at Caloote has been settled and in use for residential and recreational purposes since at least WWII (about 1946). In the intervening period, no riverbank collapse incidents have been reported to our knowledge until the recent bank cracking associated with the unprecedented low water levels of 2008-10.

Site history from imagery and publically available river maps gives locations of former lagoon banks as approximating the line of observed surface cracking in the northern residential area and the ground cracks and settlements seen among dwellings in the southern residential area. It is reasonable to take it that the influence of drying out and shrinkage of lagoon sediments during the period of low pool levels has contributed to the observed ground cracking and differential settlements.

No cracking has been observed in the boat ramp, car park, picnic shelter and related foreshores, so it is reasonable to conclude that these areas are no more hazardous to use now, in a strictly short term recreational sense, than they were prior to 2005. We have already noted a factor of safety of 1.25 at pool level 0.0m AHD applies in these areas, based on earlier stability assessments. Consequently we conclude that it may be possible to treat the boat ramp as within the tolerable risk zone for pool level at 0.0m, subject to confirmation that the riverbank safely sustains a load test of the area local to the boat ramp as suggested above (p15) to demonstrate that occupation loads are not, in themselves, a hazard driver.

Based on current water levels, which DFW advise are likely to persist for the next two years at above about +0.3m AHD, we have developed **Figure 6** attached as a proposed management plan, with permanent traffic control bollards, signage, pool water level gauge and relocated picnic shelter, to reflect the safety priorities shown on **Figure 5** and discussed above.

The suggested management plan for the Caloote Landing site is based on ALARP (As Low As Reasonably Practical) principles for pool levels nominally 0.0m AHD and above. The bollards restrict parking to an area above the tolerable risk level of 1/10,000 adopted in our report for this water level and access to the boat ramp is open but restricted to single lane access and no parking.

The picnic shelter has been relocated to a proposed position behind the tolerable risk line for pool level of -0.8m AHD so that it can remain in use, especially after gas BBQ's are installed, even at extreme low water levels should (or when) they return. The picnic area has been located in a vegetated area to the edge of the car park to reduce traffic risk to users of the shelter although any other locations landward of the grey/green line on **Figure 5** would be appropriate. We have located the bollard line by dimensions from the top of riverbank which we trust is clear enough for installation. The boat ramp entry should be based on the existing roadway.

7. Warning Signs

Suggested wording for general warning signs in the Caloote Landing area and potentially other areas along the lower pool is given below, for DFW and Council consideration:

"CAUTION

- **Take care in and on the River: water is cold and visibility poor. Snags, tree roots and branches generally impossible to see**
- **Tree limbs can fall: do not camp under large trees**
- **Riverbanks can be unstable: park at least 50m clear of bank in unregulated areas or in designated parking areas**
- **Watch for cracks along the riverbank and keep landside of any cracks. Contact DFW immediately on the Hotline 1800 751 970 when cracking is noticed.**
- **Do not use boat ramp if pool levels fall below 0.0m AHD. Refer water level gauge at boat ramp."**

8. Recommendations

Riverbanks are inherently unstable because of the way they are formed in hydraulically borne sediments, so caution in such areas is always warranted.

Specific recommendations arising from the above discussion and conclusions arising from this report on Caloote Landing are:

- Monitoring for cracking along the car park and boat ramp riverbank needs to be conducted regularly, say on Friday afternoons, on a weekly basis for the foreseeable future. Local residents could be encouraged to participate in monitoring the area for cracking.

- Risks of Failure are indicated by modelling to persist and not be tolerable at the Southern Residential Area (**Figure 5**) under existing and lower water levels. These areas should be signed as high risk of failure and the public warned for access to those sites. Residents should be advised that this part of the site remains unacceptably dangerous for human habitation and that, if occupants choose to remain, that they do so at their own risk. For more informed decision making in the long term in the Southern Residential Area, and for the sake of residents, it may be worthwhile considering further site investigations and potential remediation options for increasing bank stability in this area.

- As the changes in the river water level may trigger unacceptable risks of failure, the public should be warned by general site signage, especially for pool levels less than RL 0.0m AHD at Caloote Landing. Suggested wording has been provided above.

- The relevant Murray Darling catchment authorities should advise the Department for Water, Mid Murray Council and other relevant local authorities on future changes in the river water levels which may result in water levels less than RL 0.0m AHD at Caloote.

- To reduce risk to public safety, it is recommended that the picnic shelter to be relocated behind the grey/green line in **Figure 5,** as shown on **Figure 6**.

- Restrict parking to the area landside of the red line in **Figure 5** as shown by bollards on **Figure 6** in the immediate term, with the provision that when water level goes below 0.0 m AHD, more conservative precautions need to be put in place, reflecting the grey/green line on **Figure 5**;

- If no cracking is observed on the riverbank, for the river water levels above RL 0.0mAHD, access to the area riverside of the grey/green line is permissible. Warning signs regarding riverbank collapse should be visible to the public in the Caloote Landing area near the boat ramp and users need to be aware of potential cracks and respond as recommended on signage;

- Access to the Boat ramp, if permitted, must be based on one car and trailer at a time using the ramp and all cars parking in the car park as defined above and acceptance by potential users and managing authorities that the FoS is lower than normally acceptable and the risk level as assessed for this study remains intolerably high compared to currently accepted levels of risk as published in AGS, ANCOLD and the like. Use of the boat ramp could be based on results of local full scale load testing, using river water to simulate occupation loads of say 2.5kPa at pool level 0.0m AHD, in advance of formal reopening the boat ramp. **It should be noted that this load test will not reduce the risk to public safety.**

It is understood DFW are monitoring lower pool water levels and planning to maintain water levels at above 0.0m AHD as far as is practicable in future to mitigate river bank collapse potential. Prediction and maintenance of higher water levels is emphasised to be an important part of management plans at any public site along the lower pool. Independent of water level, the cliff toppling risk at the southern most residences is noted but not commented upon in our report. We recommend residents obtain independent advice for the rock fall hazard.

Please do not hesitate to contact SKM to clarify any of the above.

Yours sincerely,

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Attached

Figure 4: Undrained Shear Strength versus Depth at the Caloote Site
Figure 5: Caloote Site Plan with Land Use Areas and Risk Boundaries
Figure 6: Caloote Landing Site Management Plan