Local People. Global Experience.

Eurobodalla Southern Storage

Concept Design Volume 2: Storage

Prepared for: Eurobodalla Shire Council
Reference No: 30012127_R05_V04
25/07/2017

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

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EXECUTIVE SUMMARY

Background

A new water storage was recommended for Eurobodalla Shire in a 2003 Integrated Water Cycle Management Strategy (IWCMS) to increase water supply security for the region. The preferred solution was an off-stream storage supplied by high flow water from the Tuross River, to increase supply capacity to the southern parts of the system in peak summer months.

A concept design of the storage was subsequently undertaken in 2006 by the NSW Department of Commerce. At the time and due to the impact of water conservation measures, slower than anticipated growth and uncertainty associated with the proposed water sharing plans, Eurobodalla Shire Council resolved not to proceed with the project beyond preparation of the draft concept and environmental assessment documents.

A review of the water sharing plans in 2012 concluded that additional water storage would be required by 2023. A review of the IWCMS in 2016; confirmed the Eurobodalla Southern Storage as the preferred option.

Council engaged SMEC in 2016 to undertake an environmental assessment and design of the Eurobodalla Southern Storage (ESS) and the ancillary works required to integrate the ESS with existing infrastructure to service local communities.

Scope

This report forms Volume 2 of the concept design of the Eurobodalla Southern Storage. The scope of work of the concept design phase of works was to review and revise where required the previous concept design for the storage undertaken by the NSW Department of Commerce (2006b). The concept design covers structures associated with the storage, including the embankment, the spillway, inlet structure, outlet structure and temporary works.

Findings

The findings presented within this report are summarised below:

Infrastructure Staging

The upgrade of the existing treatment plant, and construction of the new storage and associated ancillary infrastructure is to be constructed in a staged process.

Stage 1 proposed to be commissioned by 2023. The works will involve:

- Construction of a new 3,000 ML off-stream storage (Eurobodalla Southern Storage).
- Construction of ancillary works infrastructure including a new river intake pump station and pipelines (these are detailed in the Volume 1 report) but not upgrades to treatment facilities.
- Other components of the project to be constructed during stage 1 include:
  - ESS access road to provide access to the storage during construction and operation;
  - Provision of power supply to the storage and ancillary works infrastructure; and
  - Fencing of the catchment boundary of the storage.

Stage 2 works scheduled for construction by 2030 (or later) will include:

- Construction of a new water treatment plant with a capacity of 25 ML/d;
- New pump station and pipeline to enable transfer of water from the ESS to the future WTP;

- New pump station and pipeline from the future WTP to Big Rock Reservoir; and
- Decommissioning of the existing WTP.

Stage 3 works will involve upgrading the ESS to increase the capacity of the storage to 8,000 ML. The timing of Stage 3 works is estimated to be in the order of 50 years after construction of Stage 1. The key components of the Stage 3 works are anticipated to include:

- Raising of the embankment to increase the storage capacity to 8,000 ML;
- Raising of the outlet tower including construction of new access bridge; and
- Construction of a new spillway.

Geology and Geotechnical Conditions

The storage site is situated within the Narooma Accretionary Complex (Terrane) of the Lachlan Fold Belt and comprises generally Ordovician to early Silurian age rocks.

The past investigations at the storage site indicate that the site is suitable for an off stream storage. Based on existing documents, and a small geotechnical investigation conducted by SMEC at Eurobodalla Quarry it is currently concluded that:

- No materials suitable for construction of the earthfill core were identified on-site. This material will need to be imported.
- No materials suitable for construction of the embankment filters were identified on-site. This material will need to be imported.
- Weathered rock material suitable for use in the outer zone of the embankment is likely available from within the proposed reservoir, however is expected to exhibit soil like properties following excavation and placement.
- On site materials are not suitable for supply of Rip Rap, imported materials will be required.
- The rock type expected on site is not suitable as concrete aggregate.
- The depth of cutoff will need to be assessed during the proposed investigations. A variable and undulating weathering profile is expected as a result of the geological structure meaning that a cut-off structure may exist from 4m depth to potentially 12 m depth on the left abutment.

Significant further geotechnical investigations and geological mapping is proposed. These further investigations are nominated to be conducted as a component of the detailed design.

Embankment

An options assessment was conducted in a previous phase of this project. The options assessment considered four dam types:

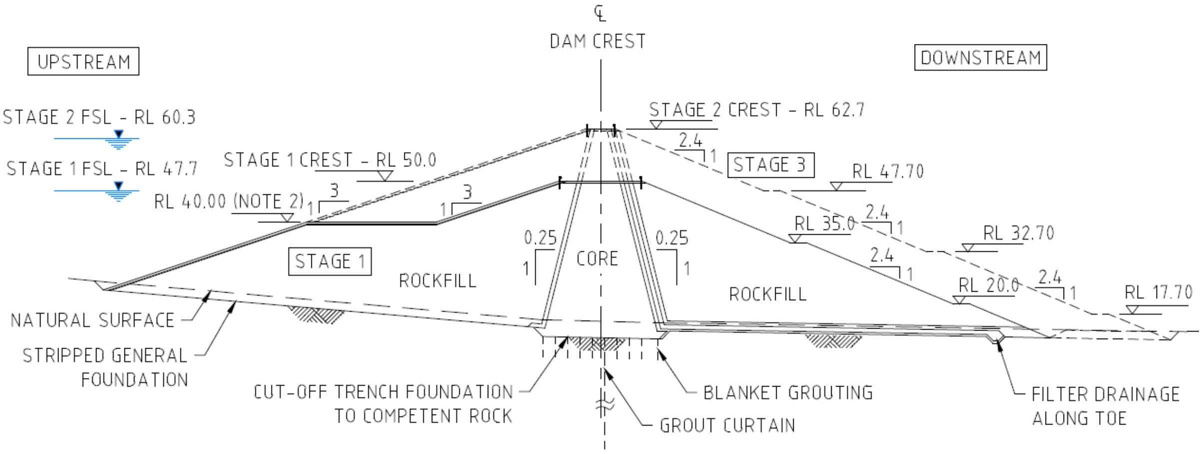
1. Roller compacted concrete dam
2. Zoned earth and rockfill embankment with central core
3. Zoned earth and rockfill embankment with sloping core
4. Concrete face rockfill dam

The results of the options evaluation concluded that the zoned earth and rockfill embankment with a central core is the preferred option to satisfy Council's Southern Storage objectives. Accordingly, this concept design report has been prepared based on the central core option.

The preferred alignment of the embankment was assessed to be positioned approximately along the lateral ridgeline on the left and right abutments, a similar position to the DoC concept design (2006b) but moved slightly to minimise earthworks for the Stage 1 embankment and the proposed cofferdam.

The key components of the embankment are described below and shown in the figure.

- Central clay core with symmetric side slopes at 0.25H:1V.
- Three stage filter/transition zone downstream of the core.
- Single stage transition zone upstream of the core.
- Rockfill shoulders:
  - Upstream shoulder to be constructed to full raised embankment profile to a berm at RL 40.0 m (level to be confirmed subject to allowable drawdown level during Stage 3 works).
  - Downstream should be designed to provide minimum width to achieve stability criteria and provide adequate support to the clay core, allowing for future raising of the core.
- Rip rap and bedding on the upstream face to provide erosion protection to the shoulder.
- Crest capping.



Item	Stage 1	Raised Embankment
Crest Elevation	RL 50.0 m	RL 62.7 m
Maximum Embankment Height (measured at downstream toe)	36m	50m
Full Supply Level (FSL)	RL 47.7 m	RL 60.3 m
Minimum Operation Level (MOL)	RL 27.4 m	RL 27.4 m
Crest Width	20 m	7 m
Upstream Slope	3H:1V	3H:1V
Downstream Slope	2.4H:1V (with 4m wide benches at 15m vertical spacing)	2.4H:1V (with 4m wide benches at 15m vertical spacing)
Upstream Core Slope	0.25H:1V	0.25H:1V
Downstream Core Slope	0.25H:1V	0.25H:1V
Total Storage Volume to FSL	3,120 ML	8,071 ML
Active Storage Volume to FSL (above MOL)	2,980 ML	7,931 ML
Embankment Material Volume	810,000 m^3	534,000 m^3 (Raised section only)

Spillway

The spillway for Stage 1 is proposed to be located through the right abutment and comprise:

- A trapezoidal shaped concrete crest
- A concrete lined chute
- A concrete dissipator structure at the downstream end
- Erosion control structures across the gully downstream of the dissipator to control velocities in the gully

Some discussion is provided within this document in regards to alternatives to alter the design of the spillway to reduce costs which will not impact dam safety but may require additional maintenance.

Hydrology and Consequence Assessment

The study area is located on an unnamed tributary of the Tuross River with a catchment area of 4.6 km^2 including the area downstream of the proposed storage to the confluence with the Tuross River. The overall catchment ranges in elevation from approximately RL 1.0 m (at the confluence with the Tuross River) to RL 153 m, and consists predominately of forested area. Of that catchment, the proposed storage will command a smaller portion of that tributary with a catchment area of 1.6 km^2.

From the flood frequency analysis, the results indicate the peak PMF inflow to the storage is estimated to be 390 m^3/s. The design peak PMF outflow from the storage occurs in the 3 hour storm event with a peak outflow estimated at 76.1 m^3/s. The PMF 3 hour event brings the storage to the design peak PMF water level of RL 49.4 m.

The consequence category for the Stage 1 embankment in accordance with ANCOLD (2012) has been rated as High C for both sunny day and incremental flood failure scenarios.

Cost Estimate

A cost estimate has been developed for the Stage 1 Storage construction costs. Costs were also developed separately for the clearing and fencing component of the works.

The current combined (base) estimate allowing for the storage and clearing and fencing components of work is approximately $75M. This estimate represents the base estimate and does not include allowance for inherent risks. Inherent risks are associated with the potential variability in the estimated unit rates and quantities.

The risk based estimate for the total project will be updated following completion of the concept design process and confirmation of the components of work to be taken forward to detailed design. The risk based estimate will include consideration of the inherent risks associated with each item.

Consistent with the previous risk based estimate undertaken, a contingency allowance of 15% has been allowed for. The contingency allows for contingent risks i.e. those risks that are dependent on an event occurring to be realised such as additional scope, industrial action etc. A 15% contingency is low for this stage of design, however a large variability in the contingency range will be adopted in the risk based estimate to reflect the uncertainty associated with this value.

Cost estimates associated with the Ancillary Works components have been provided in the Volume 1 concept report and have been based on the NSW Reference Rates Manual.

GLOSSARY

A glossary of terms is provided in Table 1.1.

Table 1.1: Glossary of Terms

Term	Abbreviation	Definition
Ancillary Works		Components of the project excluding works directly associated with the storage namely river intake pump station, pipelines, water treatment plant.
Annual Exceedance Probability	AEP	Probability at which an event of specified magnitude will be equalled or exceeded in any year, normally used in relation to floods and earthquakes.
Appurtenant Structures		Components of the Storage excluding the embankment namely Spillway, inlet and outlet works
Australian National Committee on Large Dams	ANCOLD	The Australian National Committee on Large Dams Incorporated is a non-government voluntary association of organisations and individual professionals with an interest in dams in Australia. ANCOLD is the industry body in Australia for disseminating knowledge, developing capability and providing guidance in all aspects of dam engineering, management and associated issues.
Consequence Category		Classification to categorise a dam for the potential consequences associated with failure. It is used to determine aspects such as the level and frequency of surveillance of a dam, and magnitudes of load cases to be used in the design and analysis of a dam.
Dam		An artificial barrier constructed for storage or control of water (ANCOLD 2003).
Dam Crest Flood	DCF	The flood which can be passed through the spillway with the reservoir level at the dam crest.
Dam Crest Level	DCL	Design elevation of the dam crest.
NSW Dam Safety Committee	DSC	NSW government statutory authority created under the Dams Safety Act 1978.
Design Flood	DF	The flood for which the dam is designed to safely operate with appropriate freeboard.
Environmental Impact Statement	EIS	Report documenting the environmental assessment of a project.
Eurobodalla Shire Council	Council	Municipal Council for the Eurobodalla Shire.
Eurobodalla Southern Storage	ESS/ Storage	The storage structure including the dam and associated appurtenant works (inlet, outlet, spillway).
Factor of Safety	FOS	Ratio of resisting to acting loads - indicative of the level of safety
Freeboard		The vertical distance between a stated water level and the top of the embankment.
Foundation		The material of the valley floor and abutments on which the dam is constructed.

Term	Abbreviation	Definition
Full Supply Level	FSL	The level of the water surface when the reservoir is at maximum operating level, excluding periods of flood discharge.
Left and Right hand direction		The left and right hand directions when looking downstream at the dam site.
Maximum Credible Earthquake	MCE	A Maximum Credible Earthquake (MCE) is the largest reasonably conceivable earthquake magnitude that is considered possible along a recognized fault or within a geographically defined tectonic province, under the presently known or presumed tectonic framework
Maximum Design Earthquake	MDE	The earthquake which just threatens the dam. Some damage to the structure is to be expected but the dam must not fail.
Megalitre	ML	One million litres
Minimum Operating Level	MOL	The level in the reservoir where extraction of water will cease – typically the lowest level at which the pumps can operate
NSW Department of Commerce	DoC	
Operation Basis Earthquake	OBE	The earthquake which has a 10% chance of being exceeded in a 50 year period, or an AEP of 1 in 475. Selection of the OBE is considered an economic criterion and is negotiable in consultation with the owner.
Population at Risk	PAR	The Population at Risk includes all those persons who would be directly exposed to flood waters within the dam break affected zone if they took no action to evacuate.
Potential Loss of Life	PLL	Estimated loss of life from a flood event.
Peak Ground Acceleration	PGA	Maximum ground acceleration predicted to occur during the design earthquake
Probable Maximum Flood	PMF	The flood hydrograph resulting from the probable maximum precipitation and, where applicable, snowmelt, coupled with the worst flood-producing catchment conditions that can be realistically expected in the prevailing meteorological conditions.
Probable Maximum Precipitation	PMP	The theoretical greatest depth of precipitation for a given duration that is physically possible over a particular drainage basin.
Reduced Level	RL	Elevation to Australian Height Datum.
Reservoir		An artificial lake, pond or basin for storage, regulation and control of water (ANCOLD 2003).
Safety Evaluation Earthquake	SEE	The maximum level of ground motion for which the dam is to be designed for. Damage can be tolerated but it is required that there be no uncontrolled release of water when the dam has been subjected to the seismic imposed loading from the SEE.
Shear Wave Velocity	SWV	Shear waves are characterised by particle motion which may lie anywhere in a plane perpendicular to the path of the seismic ray (i.e. grain to grain contact) which may be modified when a shear wave encounters an interface (Whiteley et al, 1990).

Term	Abbreviation	Definition
United States Army Corp of Engineers	USACE	United States Army Engineering corporation which operates and manages dams and hydropower stations in the western states of the USA.
United States Bureau of Reclamation	USBR	One of the world's leading dam authorities, who are responsible for water supply dams in western USA.
Work Health and Safety	WHS	Relates to the health and safety of workers as defined in the Work Health and Safety Act 2011.
Water Treatment Plant	WTP	A plant used to treat raw water prior to distribution.

1 INTRODUCTION

1.1 General

SMEC has been engaged by Eurobodalla Shire Council (Council) to complete an environmental assessment and design of an off-stream storage located on an unnamed tributary to the Tuross River. The scope of works of the project includes:

- Preparation of an Environmental Impact Statement (EIS)
- Storage design (referred to as the Eurobodalla Southern Storage or ESS)
- Ancillary works design
- Preparation of tender documentation

This report documents the outcomes of the concept design for the new storage. Concept design of the system is documented in two volumes as described below.

- Volume 1 (30012127_R04_V02 dated 16/6/17): relates to the proposed ancillary works infrastructure including the new river intake pump station, pipelines to and from the new storage, siting of a future Water Treatment Plan (WTP) and power supply.
- Volume 2 (this report): documents the outcomes of the concept design for the storage including the embankment, spillway, inlet and outlet works, access roads and catchment management. The report also documents the findings from the consequence assessment undertaken for the new storage.

1.2 Background

A new water storage was recommended for Eurobodalla Shire in a 2003 Integrated Water Cycle Management Strategy (IWCMS) to increase water supply security for the region. The preferred solution was an off-stream storage supplied by high flow water from the Tuross River, to increase supply capacity to the southern parts of the system in peak summer months.

A concept design of the storage was subsequently undertaken in 2006 by the NSW Department of Commerce. At the time and due to the impact of water conservation measures, slower than anticipated growth and uncertainty associated with the proposed water sharing plans, Eurobodalla Shire Council resolved not to proceed with the project beyond preparation of the draft concept and environmental assessment documents.

A review of the water sharing plans in 2012 concluded that additional water storage would be required by 2023. A review of the IWCMS in 2016; confirmed the Eurobodalla Southern Storage as the preferred option.

Council engaged SMEC in 2016 to undertake an environmental assessment and design of the Eurobodalla Southern Storage (ESS) and the ancillary works required to integrate the ESS with existing infrastructure to service local communities.

1.3 Scope of Report

This report forms Volume 2 of the concept design of the Eurobodalla Southern Storage. The scope of work of the concept design phase of works was to review and revise where required the previous concept design for the storage undertaken by DoC (2006b). This Volume 2 concept design covers structures associated with the storage, including the embankment, the spillway, inlet structure, outlet structure and temporary works.

The components presented in this report include the following:

- Staging of Works
- Design criteria
- Geology and geotechnical considerations
- Embankment design
- Foundations
- Spillway design
- Hydrology and hydraulics
- Inlet and outlet structures
- Earthquake parameters
- Instrumentation
- Access road and boat ramp
- Water quality and catchment management including clearing and fencing
- Filling strategy
- Cost estimate

2 INFRASTRUCTURE STAGING

The Eurobodalla Southern Storage Project is proposed to be constructed in three stages as described in the following sections.

2.1 Stage 1

Stage 1 proposed to be commissioned by 2023. The works will involve:

- Construction of a new 3,000 ML off-stream storage (Eurobodalla Southern Storage), the key components of which include:
  - Embankment constructed on an unnamed tributary to the Tuross River;
  - Inlet works to dissipate energy from pumped flows to the storage;
  - Outlet works to allow transfer of water from the storage to the existing and future WTP, release of environmental flows (if required) and emergency dewatering of the reservoir. The outlet works include an outlet tower, conduit, outlet valve house and access bridge; and
  - Spillway to allow the storage to safely pass flood events within the storage catchment.
- Construction of ancillary works infrastructure including (these are detailed in the Volume 1 report):
  - New river intake pump station located on the Tuross River and associated access road;
  - Connection to existing pipework between the existing borefield and the new river intake pump station to enable transfer of borefield water to the ESS;
  - New Pipeline (Segment A) between the new river intake pump station and the ESS inlet; and
  - New Pipeline (Segment B) cross connection between Segment A pipeline and the balance tank at the existing Southern WTP.
  - New pipeline cross connection between the storage outlet works and Segment A pipeline to enable transfer of storage water to the existing Southern WTP.
- Other components of the project to be constructed during stage 1 include:
  - ESS access road to provide access to the storage during construction and operation;
  - Provision of power supply to the storage and ancillary works infrastructure; and
  - Fencing of the catchment boundary of the storage.

2.2 Stage 2

Stage 2 works scheduled for construction by 2030 (or later) will include:

- Construction of a new water treatment plant with a capacity of 25 ML/d;
- New pump station and pipeline to enable transfer of water from the ESS to the future WTP;
- New pump station and pipeline from the future WTP to Big Rock Reservoir; and
- Decommissioning of the existing WTP.

2.3 Stage 3

Stage 3 works will involve upgrading the ESS to increase the capacity of the storage to 8,000 ML. The timing of Stage 3 works is estimated to be in the order of 50 years after construction of Stage 1.

The key components of the Stage 3 works are anticipated to include:

- Raising of the embankment to increase the storage capacity to 8,000 ML;
- Raising of the outlet tower including construction of new access bridge; and
- Construction of a new spillway.

3 DESIGN CRITERIA

3.1 General

The components which form the concept design of the embankment and appurtenant structures will be developed based on the design criteria as summarised in the sections below. In some cases the design criteria are subject to the analyses documented in this report e.g. seismic loading, and in these cases the method for determining the design criteria has been provided. Where applicable, the design criteria for components of works will be explicitly assigned during detailed design.

In general, the embankment and appurtenant structures will be designed for Stage 1 for the concept design (and subsequent detailed design) with each component having provision to be upgraded as part of the proposed future raising of the embankment. Some components, in particular the outlet tower base and access bridge piers will be designed considering the ultimate loads following completion of Stage 3 works.

3.2 Storage Volume

To meet the future water demand for the region the required volume of active storage is provided below. Active storage refers to the volume of water accessible in the storage through the outlet works i.e. volume of water stored above the Minimum Operating Level (MOL) of the storage.

- Stage 1: 3,000ML
- Stage 3: Future raising of the embankment: 8,000ML

3.3 Embankment

The embankment design criteria is provided in Table 3.1. The concept design presented in this report is the zoned earth and rockfill embankment with a central clay core. This embankment type was identified as the preferred option to satisfy Council's requirements as documented in the alternative embankment options report (30012127_R02_V02).

The following table summarises the acceptance criteria and the adopted methodology for assessment of the embankment design. It should be noted that the freeboard assessment has been carried out as part of the concept design stage, however the stability assessments will be undertaken during the detailed design phase of the project based on the results of the geotechnical investigations and other inputs including the seismic study.

Table 3.1: Summary of Embankment Design Criteria for Stage 1 and 3

Design Criteria	Acceptance Criteria	Reference and Comment
Slope Stability		
Steady State with the storage at Full Supply Level (FSL)	FOS>1.5	Fell et al (2015) USACE EM 110-2-1902 (2003) Assessment undertaken for D/S slopes.
Rapid Drawdown	FOS>1.3	Fell et al (2015) Assessment undertaken for U/S slope.
Construction	FOS>1.3	Fell et al (2015) Assessment undertaken for U/S and D/S slopes.
Flood	FOS>1.4	Fell et al (2015) Assessment undertaken for D/S slopes with reservoir at PMF level and consideration of potential tail water level.
Seismic loading	FOS > 1 (Screening method) Acceptable deformation of the embankment (if required)	DSC3C – Acceptable Earthquake Capacity for Dams (2010). ANCOLD 1998 – Guidelines for Design of Dams for Earthquakes. Methodology as outlined in USACE (1984) adopted for screening assessment. Methodology by Bray & Travasarou (2007) for deformation assessment. Assessment undertaken for U/S and D/S slopes assuming reservoir at FSL. Seismic loading determined based on consequence category in accordance with DSC3C.
Filter Design		
Downstream filters (chimney and blanket)	Critical no erosion filter to the adjacent base soil	Fell et al (2015)
Upstream filter	Designed to be finer than excessive erosion boundary	Fell et al (2015)

3.4 Flood Capacity

3.4.1 Eurobodalla Southern Storage

ANCOLD Acceptable Flood Capacity Guidelines (2000) and the NSW Dam Safety Committee (DSC) Guidance Sheet DSC3B (2010) provide recommendations on the flood capacity based on the assessed consequence category of the storage. The flood capacity of the storage is required to meet the flood capacity recommended in these guidelines, at a minimum.

Regardless of the assigned consequence category of the ESS, the flood capacity of the storage will be the Probable Maximum Flood (PMF). The basis for this is:

- To account for potential future development downstream and resulting change in consequence category.
- The small catchment size and resulting relatively minor difference in peak outflows associated with a PMF compared with a less extreme flood event. This is in line with the *As Low As Reasonably Practicable* (ALARP) principle.
- To account for potential future changes in flood estimation procedures during the life of the storage.

3.4.2 Cofferdam

No guidance is provided by ANCOLD or the NSW Dam Safety Committee for the design flood for a cofferdam. Considering the likely very low consequences of cofferdam failure and the ability to undertake emergency measures in the event of a flood threatening the cofferdam, a design flood of 1 in 10 AEP, as recommended by DoC (2006b), is considered appropriate for concept design.

3.5 Inlet Works

The design criteria for the inlet works are as shown below in Table 3.2.

Table 3.2: Inlet Works Design Criteria

Design Criteria	Acceptance Criteria	Reference and Comment
Operational flow (all stages)	26 ML/d	Peak flow rate from river intake pump station.
Erosion protection		Erosion protection provided to MOL.
Seismic loading		No requirement to remain operational following seismic event.

3.6 Outlet Works

The design criteria for the outlet works including the outlet tower, conduit and valve house, are as shown below in Table 3.3.

Table 3.3: Outlet Works Design Criteria

Design Criteria	Acceptance Criteria	Reference and Comment
Diversion flow	1 in 10 AEP flood	Applicable to outlet conduit during construction. Storm event relates to catchment upstream of cofferdam.
Operational flow (all stages)	~320 L/s	Peak demand to future WTP (constructed in 2030 or later) during Stage 2. Peak demand is 25ML/d over 23hrs (302 L/s) and is to allow for losses during treatment. Peak demand to be met at MOL.
Emergency drawdown		In accordance with USBR Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low-Level Outlet Works (1990).
Seismic loading	Maintain normal operating basis following OBE Remain operational following SEE	DSC3C – Acceptable Earthquake Capacity for Dams (2010). ANCOLD 1998 – Guidelines for Design of Dams for Earthquakes. Chopra and Goyal (1991 and 1989)

3.7 Transparent Storage

Transparent storage refers to the release (from the storage) of rainfall runoff from within the catchment. The storage would, therefore, be considered 'transparent' from the perspective of catchment inflows, in that rainfall runoff into the storage is equivalent to water released through the outlet works.

The reservoir may be required to act as a transparent storage up to the 1 in 10 AEP flood events. This requirement will be confirmed during the environmental assessment phase of the project.

Attempting to match the peak flow rates for storms up to the 1 in 10 AEP event was considered, however is assessed as not being feasible for the following reasons:

- Requirement to gauge inflows into the storage which would be problematic given the several minor gullies flowing into the storage;
- The peak inflow for the 1 in 10 AEP storm event is estimated to be approximately 29 m^3/s and corresponds to a 15 minute storm event. Attempting to match this outflow would result in a substantially larger outlet conduit;
- Should peak inflows be attempted to be matched this could only be achieved for certain reservoir levels:
  - At low reservoir levels there would not be sufficient head above the invert of the lowest port in the outlet tower to match the peak inflow; and
  - When the storage was at the spillway crest, matching the peak inflow would not be possible as routing through the storage would result in a reduced peak outflow.

3.8 Access Roads

The following access roads are required as part of the works:

- Access to the storage area for construction;
- All weather access during operations to the storage including valve house and embankment crest. This is referred to as the ESS access road;
- Around the catchment boundary; and
- Access to the river intake structure to allow for operations, maintenance and repairs (covered in Volume 1 report).

The design criteria relating to the ESS access road are:

- Design speed of 50 km/hr;
- Road to be designed as Minor Access Road in accordance with Council’s Infrastructure Design Standards;
- Road designed for two-way construction vehicle access; and
- Design flood for road (where access is restricted) to be less frequent than for Eurobodalla Road.

4 GEOLOGY AND GEOTECHNICAL CONSIDERATIONS

4.1 General

Geotechnical investigations were undertaken previously by DoC both at the storage site and at several quarries (2005 and 2006). The reports which present the findings from these investigations include the following:

- NSW Department of Commerce, May 2005, Stony Creek Dam Site 2 – DRAFT Feasibility Design Stage Geotechnical Investigation.
- NSW Department of Commerce, January 2006 (DoC, 2006a), Stony Creek Dam Site 2 - Concept Design Stage Geotechnical Investigation.
- NSW Department of Commerce, March 2006 (DoC, 2006b), Stony Creek Storage – DRAFT Concept Design Report.
- NSW Department of Commerce, 28 September 2006 (DoC, 2006c), Memorandum: Stony Creek Dam Trial Embankment.

At the time of writing this document, additional geotechnical investigations at the proposed storage site are yet to be undertaken, and are scheduled to be carried out as part of the Detailed Design.

Investigations have been undertaken at Eurobodalla Quarry as part of the concept design stage of works and the findings from this investigation is documented in Appendix A. The observations and results of the DoC and SMEC investigations are summarised in the following sections.

4.2 Regional Geology

The storage site is situated within the Narooma Accretionary Complex (Terrane) of the Lachlan Fold Belt and comprises generally Ordovician to early Silurian age rocks.

The foundation geology consists of a sequence of metasandstones (generally minor component) and metamudstones (generally dominant component) which were originally deposited as marine sediments (turbidites). The sediments have subsequently been altered by low grade metamorphism. The original sedimentary units have been compressed, deformed (folded) and tilted and now comprise a steeply dipping rock sequence with a generally north to south strike. Some of the broader geological structure is potentially visible on aerial images of the local terrain. Folding and metamorphism may have also developed a secondary foliation / cleavage structure.

Tertiary age sediments and Quaternary age alluvial and colluvial sediments have been deposited in the lower parts of the local river valleys and are also present at the site. Weathering processes over geological time have affected the rock, reducing the strength and the geotechnical properties of the foundation. Geological movement of the foundation by shearing (for example) has over time, developed weaknesses along the surfaces which would also have been affected by subsequent weathering.

To the west of the site there are a series of faults which are collectively known as the Budawang Thrust System, comprising north trending structures which dip to the east and west. A network of unnamed, northwest trending faults are located several kilometres to the southeast of the site. Historical seismicity suggests that one or more of these faults may be active, however this aspect has not yet been investigated or documented.

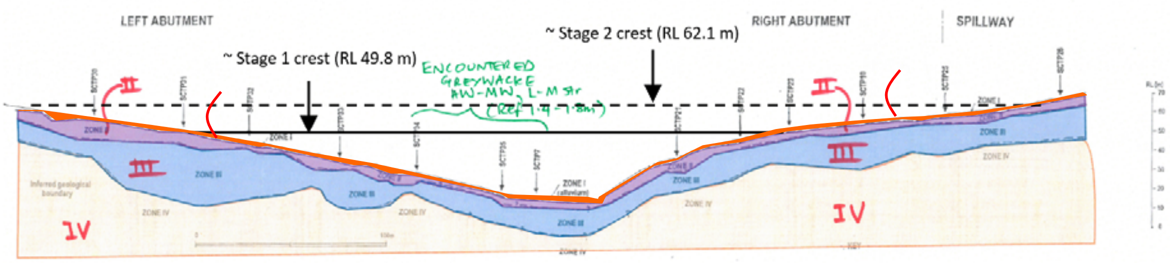
4.3 Previous Investigations

4.3.1 Storage Site and Site Quarry Areas

DoC undertook test pits (2005, 2006a), hand augers and seismic refraction testing at the storage site and adjacent quarry areas. The investigations included excavation of 15 test pits along the centreline of the proposed embankment and spillway channel; and a shallow seismic refraction survey along the proposed alignment of the embankment and spillway channel. Key findings are documented in Section 4.3.1.1 to Section 4.3.1.3.

4.3.1.1 Rock Type and Weathering

- Two rock types were identified from outcrops and from the test pit excavations. Argillite was the predominant rock type with lesser extent of fine to medium grained Greywacke also identified in the lower left abutment (as shown in Figure 4.1).
- Seismic refraction testing undertaken by Douglas Partners infer the following weathering zones:
  - Zone I Soil (often comprising mixtures of gravel, sand, silt and clay) and Extremely Weathered (XW) / Highly Weathered (HW) (refer to Appendix A for explanatory notes of geotechnical terminology) rock to about 1.5 m depth.
  - Zone II, HW to Moderately Weathered (MW) rock to about 6 m depth (12 m depth in left abutment).
  - Zone III, MW to Slightly Weathered (SW) rock to about 20 m depth.
  - Zone IV, Fresh rock from about 20 m depth.



I – Surficial zone (typ 0.5m-1.5m and up to 3.5m depth) comprising shallow soil profile (typ<0.5m), XW-HW rock (typically argillite)

II – Typically persist to 6m depth and up to 12m on left abutment. Comprises HW-MW rock (typ argillite)

III – Typically extends to 18m-20m and up to 35m on upper left abutment. Comprises MW-SW rock (typ argillite) with more weathered beds (HW) in upper portion and expected fresh rock in lower portion

IV – Expected to be fresh rock

Figure 4.1: Interpreted Geological Section adapted from NSW DoC 2005

4.3.1.2 Construction Material

- No materials suitable for construction of the earthfill core were identified on-site.
- Weathered rock material suitable for use in the outer zone of the embankment is available from within the proposed reservoir. It was expected that this material would be rippable to depths in the order of approximately 6m.
- Stripping was estimated to be required to a depth of approximately 0.5 m to 0.75 m beneath the shoulders of the embankment to expose typically highly weathered rock. Beneath the core an additional 2 m of excavation was estimated to expose typically moderately weathered rock and eliminating the majority of clay filled defects.

4.3.1.3 Rock Structure

- Original bedding of the rock after folding was identified to be near vertical and oriented in an approximately upstream-downstream direction.
- A major defect set was identified within the test pits normal to the bedding strike (east-west) dipping steeply either side of vertical.
- Two conjugate defect sets, north east / south west striking
- Two conjugate defect sets, north west / south west striking
- Shear zones were reported to have been observed along bedding surfaces and associated with the major defect (i.e. across the bedding) and were estimated to range in width from 350 mm to 1 m. The continuity of the shears are not known.
- No groundwater was encountered at the storage site during investigations.

Further investigations were undertaken (DoC, 2006a) at the site comprising hand augers of the materials within the storage site to test for dispersion. The key findings include the following:

- Based on the results of six Emerson Class tests, material sampled from around the reservoir rim was found to be non-dispersive with five tests returning results of ECN 5 and one with ECN 3. The report concluded that soils at the proposed storage site are not expected to wash into the reservoir. Soils were also observed to be very thinly developed over the Ordovician meta-sedimentary rocks.
- The anticipated highly to moderately weathered rock exposed in the spillway channel excavation is expected to have excessive to moderate erosion potential. Accordingly, the spillway was recommended to be concrete lined.

4.3.2 Commercial Quarry Sites

Two commercial quarries were investigated by DoC for potential clay sources as part of the Concept Design investigations (2006a). These quarries included Eurobodalla Quarry and Springwater Quarry, located approximately 5km and 42 kms from the site respectively.

Recognising that these investigations were undertaken approximately 10 years ago, SMEC also undertook investigations at Eurobodalla Quarry between the 18th and 20th January 2017 (results of this investigation is presented in Appendix A). The objectives of the 2017 investigation at Eurobodalla Quarry were to:

- Understand the variability of the material.
- Confirm findings and outcomes from previous investigations by conducting similar test pitting.
- Estimate quantity of potential earthfill material.

The key findings and outcomes from the DoC and SMEC investigations are summarised in the following sections.

4.3.2.1 Eurobodalla Quarry

The DoC investigations conclude the following:

- A mixture of clay soils and extremely to highly weathered Dolerite was considered suitable as core material.
- A total reserve volume of 140,000 m^3 of blended earthfill suitable for use in the embankment core was estimated to be present at the Eurobodalla Quarry and the area immediately to the north of the existing quarry workings.

- Potential difficulties in the mixing of the two materials were identified and it was highlighted that conditioning of materials may be required prior to mixing.

As outlined in the SMEC quarry investigation memo (refer Appendix A) the ground conditions observed during SMEC investigation comprised predominantly of the following units:

- Residual Soil: typically classified as high plasticity Clay (CH) (refer to Appendix A for explanatory notes of geotechnical terminology), red-brown, commonly with trace sand and gravel, typically moist at the time of investigations MC<PL, very stiff.
- Extremely Weathered Dolerite: material classified as CH Clay/ Sandy Clay, CL Clay/ Sandy Clay and SC Clayey Sand, mottled brown, orange and grey, with minor constituents of less weathered Dolerite gravels and cobbles. Material was typically found to be moist at the time of the investigations at close to the plastic limit and of stiff to very stiff consistency.
- Highly Weathered Dolerite: material was typically recovered as fractured rock consisting of gravels and cobbles within a clayey sand matrix. The strength of the intact rock was estimated to be typically high strength but was estimated to range between medium to very high strength.

The volume of suitable fill for use in the clay core was calculated by estimating the thicknesses of the respective material zones for the residual and/or extremely weathered material observed within the test pits and applying these thicknesses over a plan area.

The estimated total volume of potentially suitable material for use in earthfill core of the embankment is estimated at 175,000m^3. An area to the south (designated as areas B1 and B2 – refer Appendix A) is currently outside of the existing and proposed quarry license and was observed to provide minimal earthfill relative to its area. If this area is excluded from the estimation, the total volume reduces to 150,000m^3.

4.3.2.2 Springwater Quarry

The DoC investigations conclude the following:

- A mixture of the highly plastic soils with the clayey sands would produce a suitable material for use as core.
- Mixing of the two materials could be problematic.
- Potential source for fine filter.

The quarry is not currently in operation and has not been for several years, however SMEC was advised that plans were in place to upgrade the site to include washing and processing of sand using a cyclone with capacity to produce processed sand with 2-3% fines.

4.3.2.3 Other Sources

Various other commercial sources for fine filters were investigated by DoC including Cadgee Quarry and Congo Pit. These were identified as potential sources for suitable filter material though some samples collected by DoC indicated poor gradings.

SMEC observations from inspections of these quarries are summarised as follows:

- Congo Sand Pit: Quarry not investigated but owner was contacted by SMEC. The quarry is understood to supply poorly graded sand, unlikely to be suitable for use as fine filter within the embankment.
- Cadgee Quarry Concrete: Alluvial sands and cobbles are supplied. The sands were observed to comprise medium to fine sands likely to have the coarse fraction missing. As such, the sand is likely to not be suitable for use within the embankment.

- Bay Sand and Gravel: The material inspected at this quarry supplied crushed rock potentially suitable for use as a coarse filter (Zone 2B), subject to results from laboratory testing.

4.3.3 Trial Embankment

A trial embankment was constructed by DoC (2006c) using material sourced from the Eurobodalla Quarry overburden soils. The trial embankment was constructed from approximately equal proportions of residual soil (classifying as CH with a liquid limit of 81%) and extremely weathered to highly weathered Dolerite (classifying as GC with a liquid limit of 56%). The aim of the trial was to assess the suitability of the combined material for use in the core of the embankment.

The earthfill was compacted in approximately 150 mm thick (compacted) lifts using a 3 tonne twin (smooth) drum roller. It is understood that minimal moisture conditioning of the material was undertaken, with only light spraying of the lift prior to placement of the subsequent lift. Despite the high plasticity of the individual materials, they were observed to be reasonably workable during placement.

Field density and permeability testing was undertaken on the completed embankment. Hilf density ratios of between 94.2% to 99.3% with a moisture range of 0.1% wet to 0.4% dry of optimum based on four test results, however the trial is understood to have followed a period of consistently wet weather. In-situ permeability testing undertaken on the compacted trial embankment resulted in a permeability range of between 1.0×10^{-9} m/s to 3.0×10^{-9} m/s.

Four bulk samples were taken of the material, one from each of the different sources (residual soil and weathered dolerite) and two from the combined samples following compaction. The combined samples classified as CH sandy, gravelly clay with fines contents of 62% and 63% and both tested as ECN 1 in distilled water. The results of the testing are summarised in Table 4.1.

Table 4.1: Summary of Trial Embankment Test Results

	Residual Soil Stockpile (2721)	**Weathered Rock Stockpile (2722)**	**Combined Sample (2723)**	**Combined Sample (2724)**
Description	High plasticity clay	Clayey sandy gravel	High plasticity clay with sand and gravel	High plasticity clay with sand and gravel
Classification	CH	GC	CH	CH
Cobble size (%)	0	3	3	0
Gravel size (%)	8	33	25	14
Sand size (%)	11	30	19	22
Silt size (%)	25	14	29	21
Clay size (%)	56	20	33	43
Liquid Limit (%)	81	56	65	70
Plastic Limit (%)	28	27	27	27
Plasticity Index (%)	53	29	38	43
Emerson Class No.	2	2	1	1

4.4 Geotechnical Considerations

Geotechnical considerations associated with the storage site foundation/ geology are presented below and will inform the development of the next stage of geotechnical investigation and assessment.

4.4.1 Foundation

- There is a strong upstream to downstream oriented geological fabric which contains shears. The shears may range up to 1 m wide and potentially contain material with strength typical of soil rather than rock.
- The geotechnical condition of the shears will need specific assessment from a foundation piping/erosion viewpoint.
- The depth of cutoff will need to be assessed during the investigations. A variable and undulating weathering profile is expected as a result of the geological structure meaning that a cut-off structure may exist from 4m depth to potentially 12 m depth on the left abutment.
- The permeability will need to be further investigated, particularly in recognition of the presence of N-S striking, steeply dipping bedding and defects.

4.4.2 Spillway

- Previous work by DoC have assessed that the spillway cut will expose HW to MW rock and that rock erosion may be a key issue.
- Assessment of erodibility is to be investigated as part of the mapping, test pitting and drilling in this area.

4.5 Proposed Geotechnical Investigations

Geotechnical investigations are planned at the proposed storage site and the location of ancillary works infrastructure as part of the detailed design phase of works. The investigations are required to gain a better understanding of the subsurface geology to inform the detailed design of the embankment and related structures.

The following components of the project require design input from the geotechnical investigations:

- Foundations for embankment and spillway.
- Foundation conditions for the proposed inlet and outlet structures.
- Foundation conditions for proposed access road to the storage.
- Quarry area within the proposed water body of the storage to understand the characteristics of the potential shoulder fill.
- Ancillary structures including the river intake pump station, pipelines between the existing Southern WTP and the storage and the pipeline from future the WTP to Big Rock Reservoir.

The proposed geotechnical investigations to address the knowledge gaps at these areas are summarised in the below Table 4.2. The results of these investigations will be detailed in subsequent design and investigations reports.

Table 4.2: Summary of geotechnical investigations to be undertaken

Structure	Geotechnical Investigation Undertaken in Detailed Design	Information to be Obtained
Embankment		
Foundation	Core drilling along centreline	Rock/soil structure, strength, composition.
Abutments	Installation of piezometers Water pressure testing Trench along centreline and test pits Seismic Refraction Rock Mapping	Foundation seepage behaviour and permeability. Degree of weathering and suitability for usage within embankment. Testing to include characterisation testing for soils, Unconfined Compressive Strength (UCS) to be undertaken on rock samples and potentially Abrasion Resistance, Sodium Sulphate Soundness, cyclic wet/ drying and compaction to understand potential material breakdown. Understanding slope stability, excavation batters. Excavatability and depth for foundation and core trench
Spillway		
Foundation	Test Pitting Core drilling Seismic Refraction Rock Mapping	Rock/soil structure, strength, composition. Rock erodibility. Degree of weathering and suitability for usage within embankment. Testing to be consistent with embankment. Understanding slope stability, excavation batters. Excavatability
Storage		
Inundation Zone	Rock Mapping Test Pitting Seismic Refraction	Rock quarry depths and potential extents. Understanding slope stability, excavation batters. Testing to be consistent with embankment. Indicative soil/rock strengths. Material unit depths.
Ancillary Structures		
River intake structure	Core drilling	Excavatability. Understanding slope stability, excavation batters. UCS testing of rock core. Soils tested for acid sulphate potential. Foundation conditions.

Structure	Geotechnical Investigation Undertaken in Detailed Design	Information to be Obtained
Pipelines and access road	Test Pitting	Excavatability. Foundation conditions. Soils tested for acid sulphate potential and dispersion.

5 EMBANKMENT

5.1 General

As part of the initial phase of concept design an Alternative Dam Options Report was prepared by SMEC and is documented in 30012127_R02_V02. The objective of this report was to review the concept design of the dam developed by DoC (2006b), and to evaluate potential alternative dam options in order to identify a preferred option to progress to concept design stage.

The four dam types considered were:

1. Roller Compacted Concrete Dam
2. Zoned Earth and Rockfill Embankment with Central Core
3. Zoned Earth and Rockfill Embankment with Sloping Core
4. Concrete Face Rockfill Dam

The dam options were evaluated against six criteria which included:

- Technical Merit
- Dam and Public Safety
- Duration, Risk and Constructability
- Environmental Risk
- Operation and Maintenance
- Cost

The results of the options evaluation concluded that the Zoned Earth and Rockfill Embankment with Central Core is the preferred option to satisfy Council's Southern Storage Objectives. Accordingly, this concept design report has been prepared based on the Central Core option.

5.2 Location

Four potential sites for the storage were investigated by the NSW Department of Public Works and Services in 2002. Following environmental investigations of the sites in 2005 (Department of Public Works and Services) the preferred location for the storage was selected to be within the Bodalla State Forest at a site closest to existing water supply infrastructure. The location of the site was referred to as 'Stony Creek No. 2' as shown in Figure 5.1.

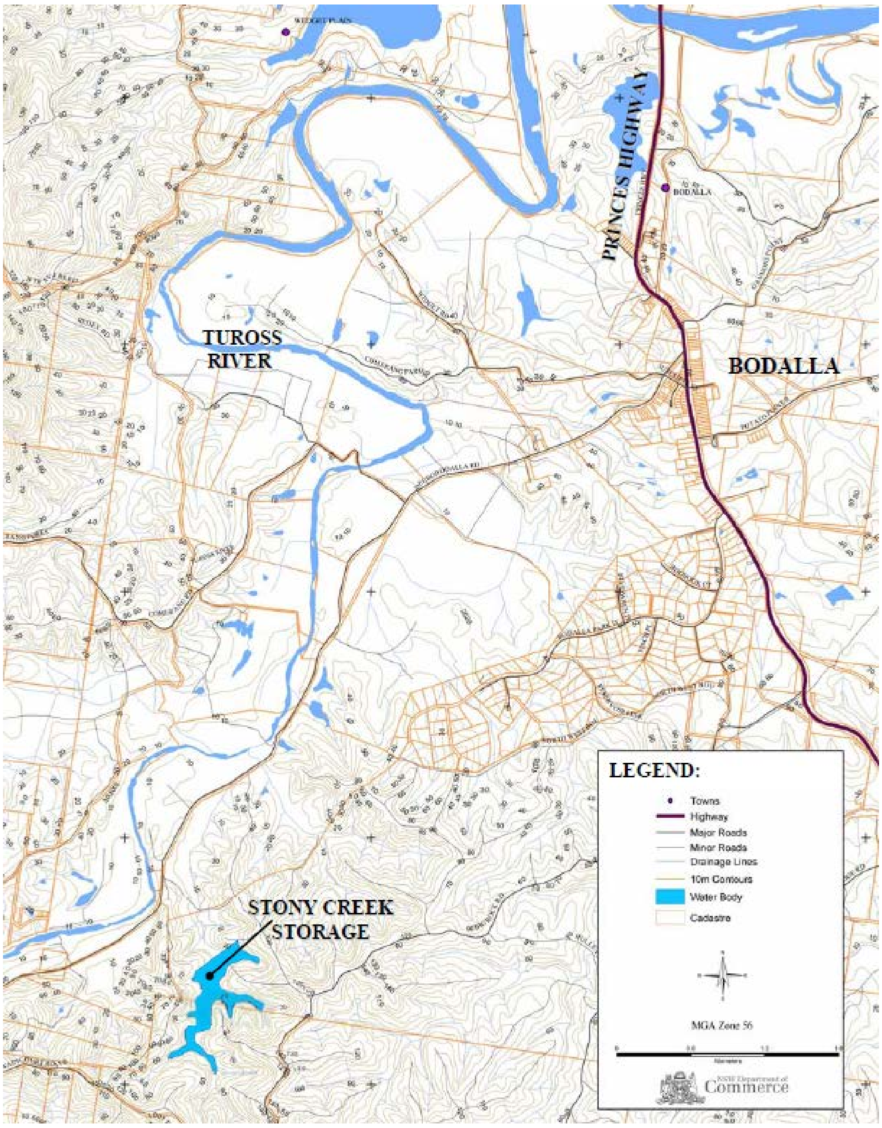


Figure 5.1: Proposed Location of the Eurobodalla Southern Storage Site – referred to as 'Stony Creek Storage' (DoC 2006)

5.3 Embankment Alignment

The preferred alignment of the embankment was assessed to be positioned approximately along the lateral ridgelines on the left and right abutments, a similar position to the DoC concept design (2006b). The narrow widths of the ridgelines, and the steep drop into adjacent gullies will result in the embankment shoulders chasing the cross-slope. Locating the core either downstream or upstream of the ridgelines would reduce either the upstream or downstream slopes into progressively narrower sections which is not preferred. Hence the core and filters were preferentially positioned to remain on the ridgelines, with acceptance that the shoulders of the potential future raised embankment would drop off the ridge. The proposed footprints of the Stage 1 and future raised embankments are shown in Figure 5.2.

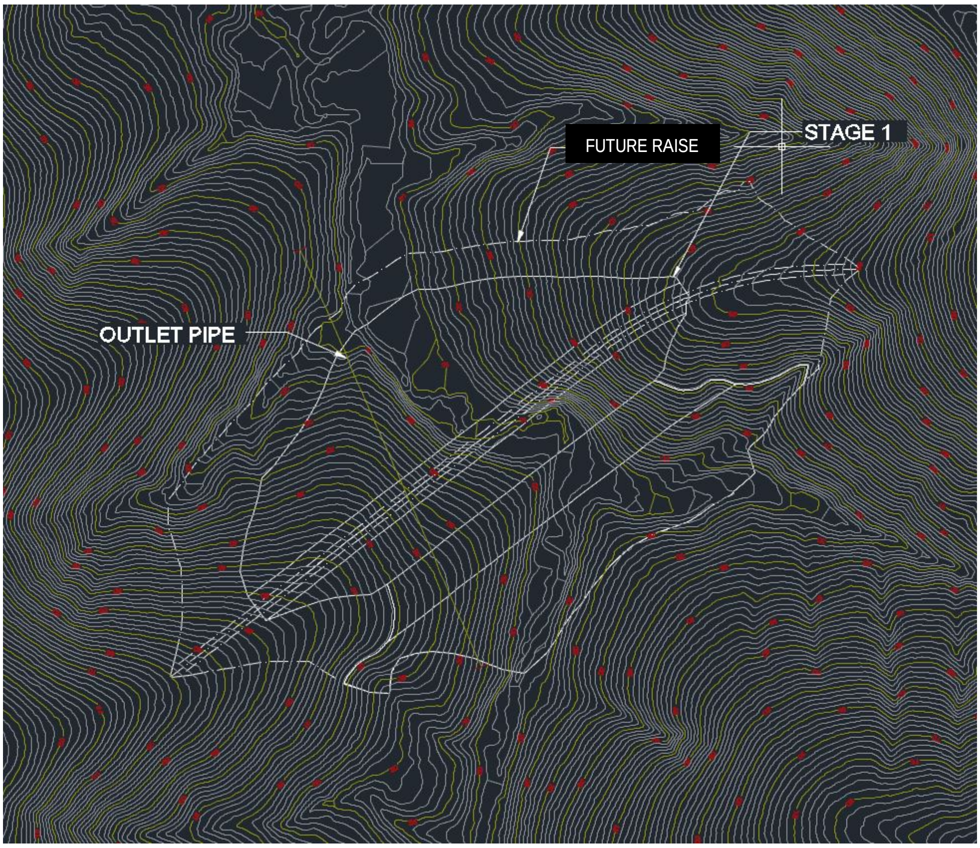


Figure 5.2: Plan view of embankment showing Stage 1 and potential future raised footprints

5.4 Embankment Configuration and Geometry

The key components of the embankment are described below and shown in Figure 5.3. The full set of concept design drawings are provided in Appendix B.

- Central clay core with symmetric side slopes at 0.25H:1V. At this stage, the core has been assumed to extend approximately 2m below the general founding depth of the shoulders. The depth of the proposed cutoff will be refined following the proposed geotechnical investigations.
- Three stage filter/transition zone downstream of the core.
- Single stage transition zone upstream of the core.
- Rockfill shoulders:
  - Upstream shoulder to be constructed to full raised embankment profile to a berm at RL 40.0 m.
  - Downstream should be designed to provide minimum width to achieve stability criteria and provide adequate support to the clay core, allowing for future raising of the core.
- Rip rap and bedding on upstream face to provide erosion protection to the shoulder.
- Crest capping.

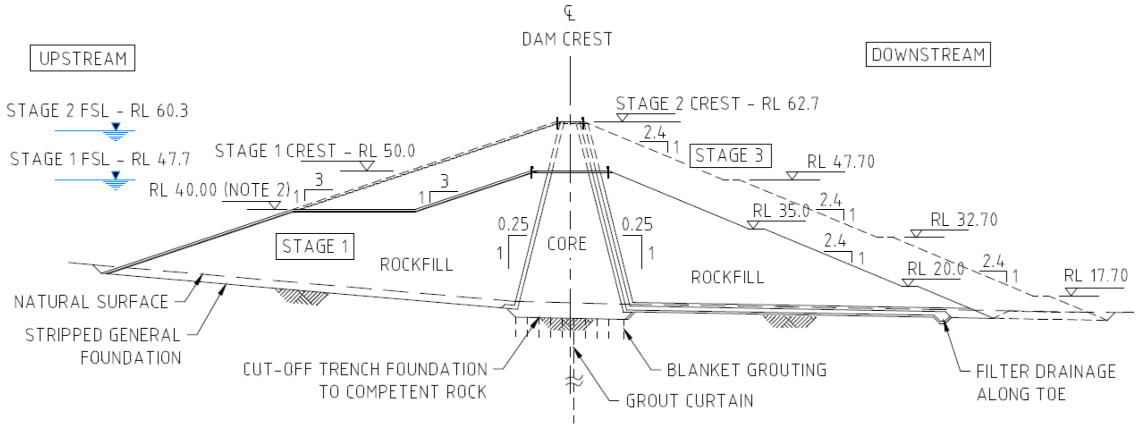


Figure 5.3: Cross Section through Embankment (extract from SMEC Concept Drawing 3101)

The ratio of core width at the cut-off base to the height is 0.5. This has been increased from 0.3 adopted by DoC (2006b). This ratio is assessed to be the minimum width of core required to provide adequate seepage control to the structure, considering the ultimate raised embankment height, embankment use (water supply potentially subject to considerable fluctuation), geology of the site, and in recognition that stored water must be pumped from the Tuross River. It also results in a lower hydraulic gradient across the core and hence a reduced probability of initiating internal erosion.

The central core also provides a uniform projection of Stage 1 geometry to the final raised profile as shown in Figure 5.3, with the Stage 1 embankment forming a truncated profile of the future raised embankment. Hence there are no discontinuities in profile between stages, as was proposed by DoC (2006b).

To allow two-way access on the final raised embankment crest, the width has been designed at 7m (excluding rip rap). This will also provide sufficient width to contain the various zones with constructible widths, and for trafficking during construction. Accordingly, the Stage 1 embankment will have a significantly wider crest to accommodate the future raising to the final design crest geometry. As shown in Figure 5.3, the Stage 1 crest width is approximately 20m, which includes 4m of rockfill either side of the filter/transition zones. The basis of this geometry includes:

- Truncated core geometry with a width of approximately 9.5m during Stage 1 and narrowing to 3m following Stage 3 raising. This arrangement avoids interim changes in core geometry, which would require removal of shoulder and filter materials for core placement below Stage 1 crest level for future raising;
- Provide adequate support to the broad interim core to limit moisture variations and/or core deformation, hence reduce the potential for having to re-work the upper portion of the core prior to raising.

The upstream batter comprises a lower level berm, 10m below the Stage 1 embankment crest at RL40m. The purpose of this arrangement is to reduce the fill required for construction of Stage 1 i.e. rather than constructing the full raised embankment profile to the Stage 1 dam crest. The elevation of the upstream berm is flexible and is dependent on the acceptable reservoir level during the Stage 3 embankment raising works. Lowering of the water level in the storage is only required during the construction of the upstream portion of raised embankment or as required to manage the dam safety risk during construction.

By lowering the storage to say 1m below the proposed upstream berm level (water level at RL 39 m) during construction of the raised embankment (as part of Stage 3 works) the volume of active storage (above MOL) would be reduced to 1,056ML. This equates to the reservoir being approximately 35% full. Should a reduced volume of water within the storage during future raising of the embankment be acceptable to Council, the height of the upstream berm could be lowered further with a resulting reduction in earthworks volume required for Stage 1.

The upstream slope for the embankment is set at 3H:1V, and with the downstream slope set at 2.4H:1V with 4m wide berms at 15m vertical intervals (this equates to an average slope of approximately 2.5H:1V). These slopes are considered reasonable, but it is noted that the properties of the on-site quarry run rockfill for the shoulders have not been established at this stage. There is a possibility that steeper slopes may be suitable, however given that the material for the shoulders is likely to breakdown during placement (producing fines), these slopes are assessed to be appropriate for the concept design phase.

The height of the embankment has increased slightly compared to the DOC concept design to allow for:

- The adjustment of the embankment alignment;
- Change in the design flood (refer to Section 8.5);
- Change in freeboard allowance (refer to Section 8.6)
- Change in MOL (refer to Section 9.3.1) while retaining storage capacities of approximately 3,000 ML for Stage 1 and 8,000 ML for the future raised embankment (during Stage 3 works). Figure 5.3 shows the typical section of the zoned earth and rockfill with central core.

The key features of the embankment and reservoir are summarised in Table 5.1 below.

Table 5.1: Zoned Earth and Rockfill Embankment (Central Core)

Item	**Stage 1**	**Stage 3 (raised embankment)**
Crest Elevation	RL 50.0 m	RL 62.7 m
Maximum Embankment Height (measured at downstream toe)	36m	50m
Full Supply Level	RL 47.7 m	RL 60.3 m
Minimum Operating Level	RL 27.4 m	RL 27.4 m
Crest Width	20 m	7 m
Upstream Slope	3H:1V	3H:1V
Downstream Slope	2.4H:1V (with 4m wide benches at 15m vertical spacing)	2.4H:1V (with 4m wide benches at 15m vertical spacing)
Upstream Core Slope	0.25H:1V	0.25H:1V
Downstream Core Slope	0.25H:1V	0.25H:1V
Total Storage Volume to FSL	3,120 ML	8,071 ML
Active Storage Volume to FSL (above MOL)	2,980 ML	7,931 ML
Embankment Material Volume	810,000 m^3	534,000 m^3 (Raised section only)

It is noted that the active volume is marginally below the target volumes of 3,000 ML for Stage 1 and 8,000 ML following raising of the embankment during Stage 3. These volume estimates do not account for the additional storage gained from the excavation within the storage for the rockfill quarry. Accounting for these volumes would readily increase the active storage to greater than the target volumes.

5.5 Stability

SMEC has not undertaken stability assessment for the embankment at this stage noting that the same slopes for the embankment have been adopted as per the previous DoC Concept Design (2006), albeit SMEC has increased the core width, and included additional filter zones. The minimum factor of safety assessed by DoC for the critical failure surface was 1.92. No further testing or geotechnical investigations have been carried out since 2006, and as such, material parameters for model input are no better informed. Stability analyses will be undertaken as part of the detailed design phase of works, following completion of the proposed geotechnical investigations.

5.6 Construction Materials

5.6.1 Volume Estimates

The use of on-site materials for construction of the embankment has been maximised where practicable. Geotechnical information to date suggests that the on-site materials are not suitable for the core nor the critical filter zones. As such, a combination of quarrying and processing of on-site material and importation of materials, are required for placement within the embankment. The estimated volumes required for each zone and an indication of their source (on-site or imported) is presented in Table 5.2 below.

Table 5.2: Summary of Embankment Zone Volumes

Material Zone	**Estimated Volume**
	Stage 1
Imported Materials	
Core	107,000 m^3
Zone 2A	18,500 m^3
Zone 2B	24,500 m^3
Zone 2C	6,000 m^3
Bedding	10,500 m^3
Rip Rap	35,000 m^3
Total Imported Volume	**201,500 m^3**
Sourced On-Site	
Rockfill Shoulders	586,000 m^3
Zone 3A (Transition)	22,500 m^3
Total Material Sourced On-Site	**608,500 m^3**

5.6.2 Shoulders

The shoulder fill material represents the bulk of the material forming the embankment. It is expected that this material will be quarried rockfill material won from on-site excavation comprising a mixture of XW/HW rock to variably weathered HW/MW (Weathering Zone I and Zone II identified in DoC, 2005). As such the material is notionally described as random rockfill material, potentially with 'soil-like' strength properties, and variable grading.

The rockfill is expected to be placed into the embankment directly from excavation with minimal processing, though the specification for this zone will likely have a maximum size limit.

The moisture content for placement will be specified following geotechnical investigations, but is likely to be drier than the core zone material.

5.6.3 Fine and Coarse Filter

The random nature of the shoulder materials could result in higher permeability layers being present within the embankment shoulders, with poor compatibility with the clay core material. As such, filter protection on both sides of the core is recommended at this stage. A three stage filter has been designed downstream of the core, and a single stage 'transition' filter upstream of the core between the interface of the core and shoulder materials.

The three-stage downstream filter is to comprise the following layers:

- Zone 2A fine filter: a processed alluvial material (gravelly sand), imported from off-site;
- Zone 2B coarse filter: processed quarried rock (gravel), imported from off-site; and
- Zone 3A: processed quarry run material (sandy gravel), quarried and processed (coarse and fine screened only) from the on-site quarry.

The downstream filters provide erosion protection to the core material as it transitions to the random rockfill shoulder material. The filters also provide the drainage of pore pressures at the clay/filter interface to limit the potential for saturation of the anticipated 'soil-like' downstream shoulder.

This three-stage filter system continues as a blanket filter along the base of the downstream shoulder at the interface with the general foundation. The purpose of this blanket filter system is to protect the embankment from foundation piping and to provide a drainage outlet to the chimney filter. A filter blanket is considered warranted for the embankment as the foundation materials are noted to comprise variably weathered rock with potential for preferred seepage paths due to the sub-vertical bedding striking sub-perpendicular to the embankment axis.

The fine filter zone is proposed to be construction on the downstream side of the cut-off trench. Depending on the results of the geotechnical investigations, a filter trench may also be required to be excavated into the foundation to intercept potential piping pathways in the foundation.

The upstream single stage filter system comprises a single layer of Zone 2C filter sand/gravel material. The purpose of this upstream filter is to protect the embankment from backward erosion of core into the shoulder fill under drawdown conditions. It would also provide flow limiting and crack filling materials for the core, should defects develop in the core.

The width of the filter zones are considered to be the minimum thickness to address filtration and drainage requirements, allowing for construction methodology and the anticipated variability of the on-site sourced rockfill zone.

A potential source for Zone 2A and 2C is Springwater Quarry subject to laboratory testing and capability of the quarry to meet specified material gradings and production requirements of the Contractor following the proposed upgrade of the quarry processing equipment.

Eurobodalla Quarry is identified as the most likely site for sourcing the Zone 2B (and the rip rap layer) due to the proximity to the storage site. Eurobodalla Quarry produce aggregate (and rockfill) products from crushed from Doleritic rock. Other sources of aggregate products are available in the region (e.g. Bay Sand and Gravel).

5.6.4 Filter Specification

The filter specification for the Zone 2A fine filter will be designed as a critical filter to the core, with Zone 2C fine filter designed to avoid excessive erosion of the core based on the methodology described in Fell et all (2015). Samples were retrieved as part of the investigations at the Eurobodalla Quarry (as detailed in Appendix A) for the purpose of designing the filter zones.

The Zone 2A will be designed to accommodate the available filter sources where practicable within the design criteria. At this stage (prior to assessment of all laboratory sample results), the DoC designed Zone 2A filter envelope is considered suitable.

The filter specification for the coarse filter (Zone 2B) will be designed to comply with the Zone 2A and may be further refined based on the washed Zone 2A grading results during construction.

5.6.5 Core

The core material will likely be sourced from the Eurobodalla Quarry some 5km from the storage site.

DoC report (2006c) presented testing of the samples taken from the Eurobodalla Quarry. The testing was carried out on representative samples of the two distinct materials within the quarry being the residual CH clay and the XW Dolerite classifying as GC, as well as on a sample comprised of a mixture of the two (50/50 by weight).

The results of these testing demonstrated that for use within the embankment core:

- The highly plastic residual soils were suitable;
- The XW material alone was not suitable ; and
- The mixture (50/50) of the two materials was suitable.

Laboratory testing from the SMEC geotechnical investigation at the quarry has not been returned at the time of writing this report, however field classification of the materials generally confirmed the properties reported by DoC.

One significant outcome of the SMEC investigation at Eurobodalla Quarry was the expected availability of materials required to meet the demands for the Stage 1 embankment. SMEC estimates that approximately 150,000m^3 of suitable material is presently available at the quarry. This is increased to 175,000 m^3 if areas outside of the proposed lease extension area are included. While this is more than that estimated by DoC, it is still unlikely to be adequate to cater for the potential future raising of the embankment.

The estimated volume of earthfill core required for construction of the Stage 1 embankment, is approximately 107,000m^3. This is a placed volume and does not account for over placement or losses due to wastage, contamination or moisture content. This equates to approximately 40% excess volume in-situ at Eurobodalla Quarry compared to the material demand. Preference is to typically allow for greater than 50% excess to account for losses in handling, compaction and wastage. Close management of the excavation and blending process will be required during construction to manage the risk of a material shortfall for the earthfill source.

6 FOUNDATIONS

6.1 General

The foundation in general was observed by DoC (2005 and 2006a) to be variable in weathering and strength. The rock structure indicates potential for seepage along defects within the foundation particularly along the bedding striking north-south (upstream to downstream).

These key issues that will impact on the construction and design of the embankment and appurtenant works, particularly with respect to permeability, excavation and erodibility are discussed in the following sections.

6.2 Excavation

The DoC (2005) classified the subsurface material into four categories, based on the Shear Wave Velocities (SWV) and confirmed by test pitting via 20 tonne excavator undertaken along the proposed embankment location and foundation. These included:

- Zone I: Surficial zone comprising soils, grading to XW/HW rock. The zone generally varies from 0.5m to 1.5m, ranging to 3.5m in places. Generally corresponds to SWV between 220 to 600m/sec.
- Zone II: Variably weathered, ranging from HW to MW with depth. Generally persists to depths of 6m, ranging to 12m on the upper left abutment. Generally corresponds to SWV of 900 to 1,000m/sec.
- Zone III: MW/SW rock generally extending to depths in the order of 18m to 20m (up to 35m on the left abutment). Fresh rock is expected towards the base. Generally corresponds to a SWV between 1,600 to 2,000m/sec.
- Zone IV: Fresh rock corresponding to SWV of 2,700 to 5,000m/sec.

Excavation of the foundation for the embankment will predominantly be undertaken within Zone I and Zone II. The general foundation is expected to found on top of, or within the Zone II layer. The cut-off foundation will found at the base of the Zone II layer (top of the Zone III layer).

As such, excavation will predominantly be undertaken of the Zone I and Zone II materials. Excavation of rock using CAT Dozers D8 to D11 can usually be undertaken for rocks with SWV of up to 1700m/sec to 2600m/sec respectively (Look, 2014). This indicates that excavation within the foundation will likely be achievable using conventional machinery.

It should be noted however that these excavatability correlations are indicative only. It is difficult to estimate the excavatability accurately at this stage without further geotechnical investigations. Defect spacing, persistence, structure, intact strength and weathering profiles all influence excavatability and cannot be confirmed until completion of the proposed geotechnical investigations.

The spillway is proposed to be positioned on the upper right abutment. The spillway is most likely to be founded on Zone III rock where practicable, however more weathered materials will be exposed in the approach and discharge ends of the spillway as excavations are shallower. The DoC seismic results indicate that this area has SWV of up to 1800m/sec, which is expected to be readily excavated by a CAT D11 Dozer.

6.3 Foundation Grouting

The embankment foundation is expected to require grouting. The foundation treatment will likely include:

- Consolidation grouting;

- Installation of grout cut-off curtain; and
- Shotcreting foundation beneath the core.

Consolidation grouting along the base of the cut-off is likely to be required given the type of rock encountered (Argillaceous and sedimentary Mudstones) and the seepage pressures with which the reservoir will be imposing upon the cut-off/foundation interface. The consolidation grouting is intended to fill defects within the upper portion of the foundation and additionally strengthen the foundation to reduce the potential for differential settlement/movement.

It is anticipated that a grout curtain will be required beneath the embankment cut-off trench. This grout curtain is expected to extend into the foundation to lengthen the seepage path and therefore further reduce the seepage potential within the foundation.

Shotcreting of the foundation within the cut-off trench is expected to be required to reduce the potential for piping related failure modes at the core-foundation interface.

The foundation grouting design will be undertaken during detailed design, following completion of the geotechnical investigations. It is currently expected that the grout curtain will be designed to be equivalent in depth to the final raised embankment height (constructed during Stage 3), however this will be dependent on the discontinuities within the foundation rock and the results of the packer testing.

7 SPILLWAY

7.1 General

The spillway is proposed to be located through the right abutment and comprise:

- A trapezoidal shaped concrete crest
- A concrete lined chute
- A concrete dissipator structure at the downstream end
- Erosion control structures across the gully downstream of the dissipator to control velocities in the gully

The spillway services the first stage of storage only, with the future raised section (constructed during Stage 3 works) requiring a new spillway to be constructed. During raising of the embankment, the Stage 1 spillway will be backfilled and the raised embankment constructed over the top.

Most of the material excavated from the spillway during both Stage 1 and 3 works is expected to be used for placement within the embankment for the respective stages of construction.

7.2 Description

The spillway is a 15m wide concrete chute through the right abutment (see Figure 7.1).

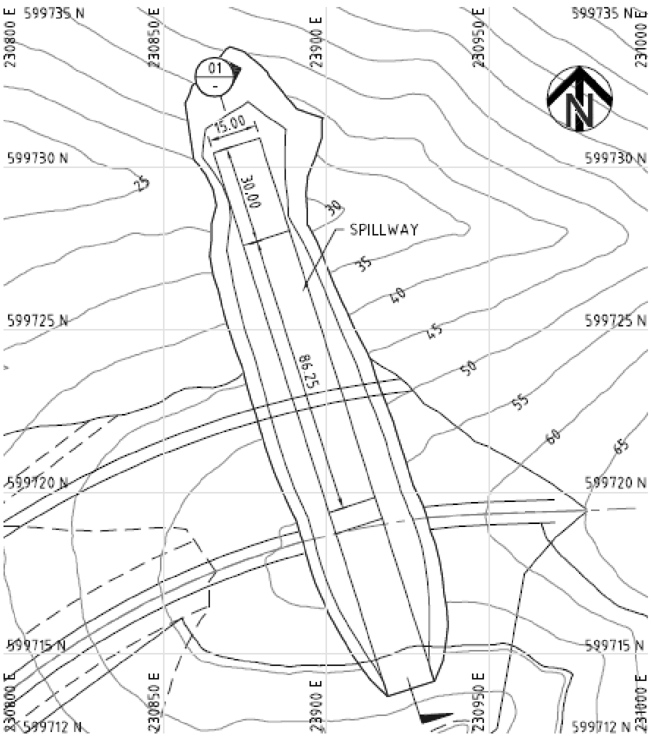


Figure 7.1: Plan view showing spillway alignment (Extract from Drg ST-3300)

The approach to the crest is a 15m wide unlined channel excavated into rock, leading to an 800mm high trapezoidal shaped concrete crest. The need for lining this section upstream of the control structure is dependent on the erodibility of the rock, and this will be confirmed during detailed design and following completion of the geotechnical investigations.

Below the crest is a prismatic trapezoidal concrete chute, 15m bed width, through the abutment to a hydraulic jump dissipator. The chute is straight and at right angles to the spillway crest to reduce the potential for transverse pressure waves to rebound off the side walls which will ensure even distribution of flow entering the dissipator achieving maximum efficiency of dissipation of energy.

The chute is proposed to be concrete lined and trapezoidal in cross section with the sloping side cut concrete faced to 600mm above the estimated PMF flow level, including an allowance for aeration of the flow. The design of the concrete-lined wall will be reviewed following completion of the geotechnical investigations. Should foundation conditions not be suitable for construction of a lined concrete batter e.g. poor rock quality and elevated groundwater levels, freestanding cantilever walls may be required.

The floor and wall is proposed to be anchored into the rock to reduce the potential for uplift of the slabs. Floor slab thickness is proposed to be typically 250mm with two layers of reinforcement. The anchor pattern is typically 3m x 3m grid and 2.4m into sound rock. The anchor hole size will depend on the rock conditions and the assessed bond strength with the grout.

Slab and wall lengths are proposed to be 20m long, with the transverse joints profiled to provide cut-offs to anchor the slab and waterstops provided between joints to limit longitudinal under-seepage. Given the relatively small width of the spillway (15m), longitudinal joints are unlikely to be required. This will be confirmed during detailed design.

The 800mm high concrete crest can be constructed as a series of flat surfaces to reduce construction complexity, but a rounded upstream edge will be retained. This arrangement is shown in Figure 7.2.

As the spillway is directed into the same watercourse downstream of the embankment, there is the potential that storage outflows could cause erosion and inundation issues for the embankment. Design features such as a coarse rockfill toe will be considered in detailed design to mitigate this.

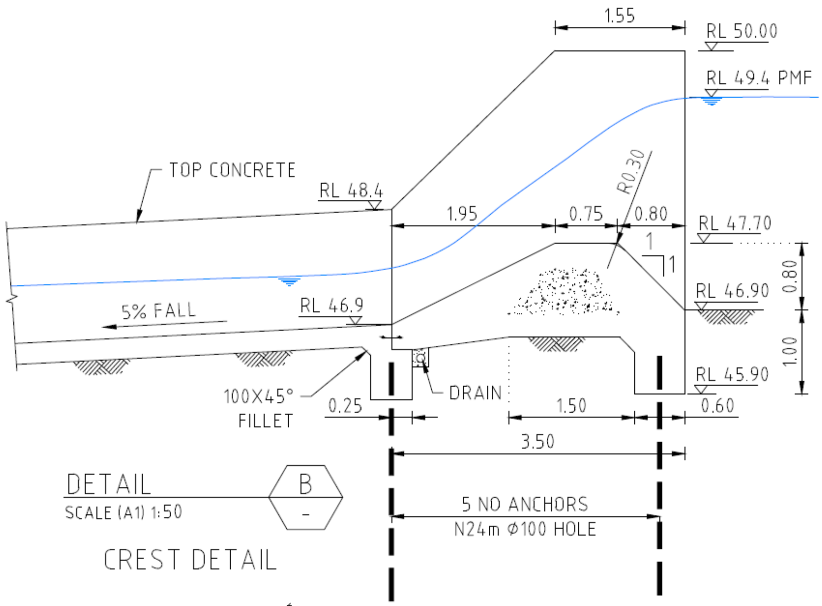


Figure 7.2: Detail of spillway crest (Extract from Drg ST-3300)

7.3 Capacity and Design

The design flood for the spillway is proposed to be PMF with a corresponding peak outflow of approximately 76.1 m^3/s. This analysis is described in Section 8 and Appendix C.

7.3.1 Head – Discharge for Crest

For calculation of the head discharge curve, the crest is divided into a horizontal crest and a V-notch weir with included angle of 90^o to suit the 1H to 1V side slopes.

The head discharge curve for the horizontal portion of the crest is developed using the expression:

$Q = Cd\ L\ H^{1.5}$,

Using crest coefficients in the publication 'Geological Survey Circular 397' (Tracy, 1957) plus an allowance for a rounded upstream edge to the crest. The variation of coefficient with head is shown in Figure 7.3.

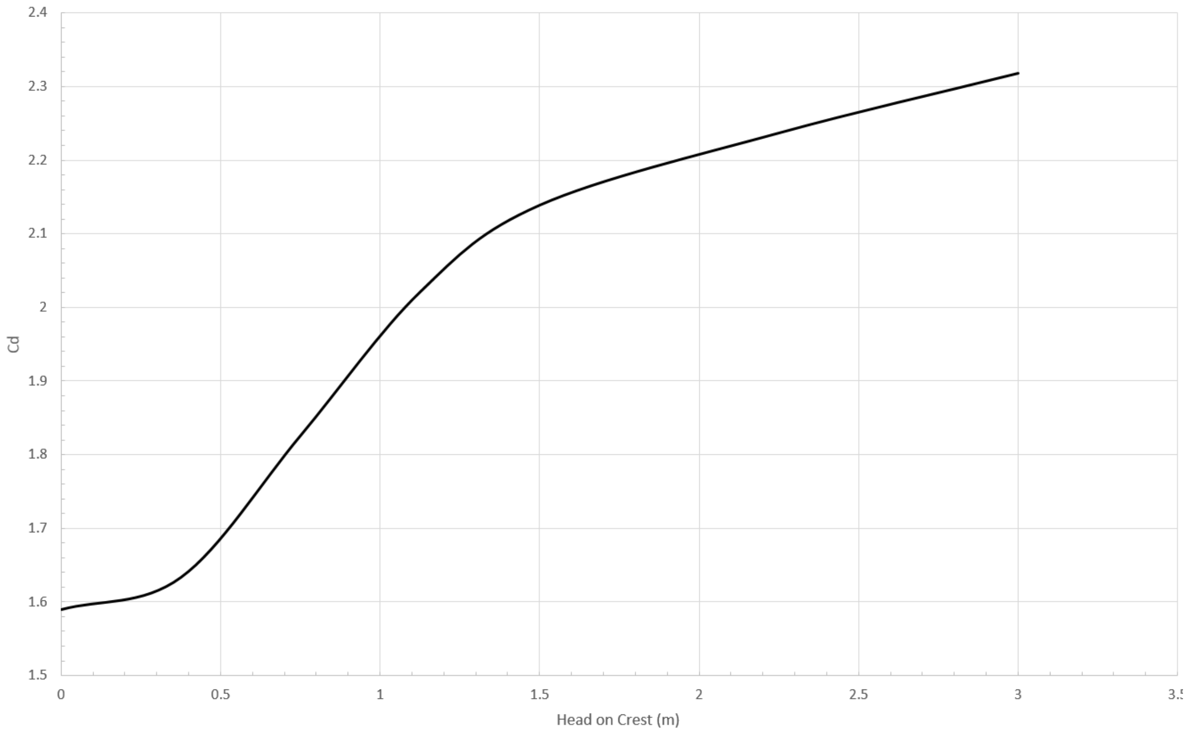


Figure 7.3: Variation of Coefficient Cd with Head

For the sloping sections, the equation of a broad crested triangular weir (Ghodsian, 2009) was adopted;

$Q = 16/(25\surd 5).Cd.\surd(2g).\ h^{5/2}.\tan(\Theta/2)$

With $Cd\ = 0.984\ (h/H)^{0.025}$

Losses in the approach channel, although minor, were calculated using Manning's Formula with a friction coefficient of n = 0.035.

The resulting head discharge curve for the 15m spillway crest including the sloping sides is presented in Figure 7.4 along with the proposed Dam Crest Level (DCL) and PMF reservoir level.

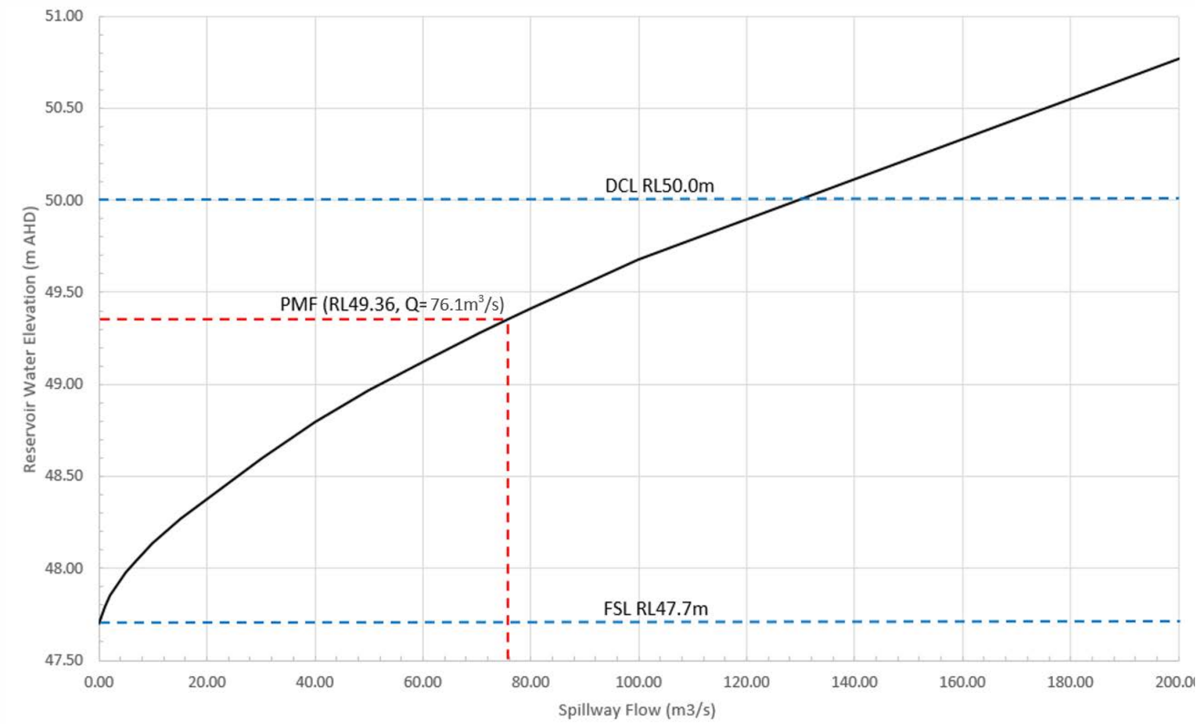


Figure 7.4: Head Discharge Curve for 15m spillway crest

The reservoir level at a discharge of 76.1m^3/s is RL 49.4m AHD.

7.3.2 Flow Depths in Chute

The depth of flow in the discharge channel was calculated using a friction coefficient of 'n' = 0.018. This is considered high for a concrete lined chute however will give a conservative (greater) estimate of flow depth.

The estimated depth of flow in the chute downstream of the crest block during the PMF outflow is presented in Figure 7.5, with the indicative target wall height varying from approximately 1.5m at the crest to 1.0m at the downstream end. This provides a 600m freeboard buffer to the top of the wall.

The flow profile shown in Figure 7.5 shows a reduction in flow depth at approximately 6.8m downstream of the crest block. This is representative of the increase in design grade of the chute from 5% to 25%.

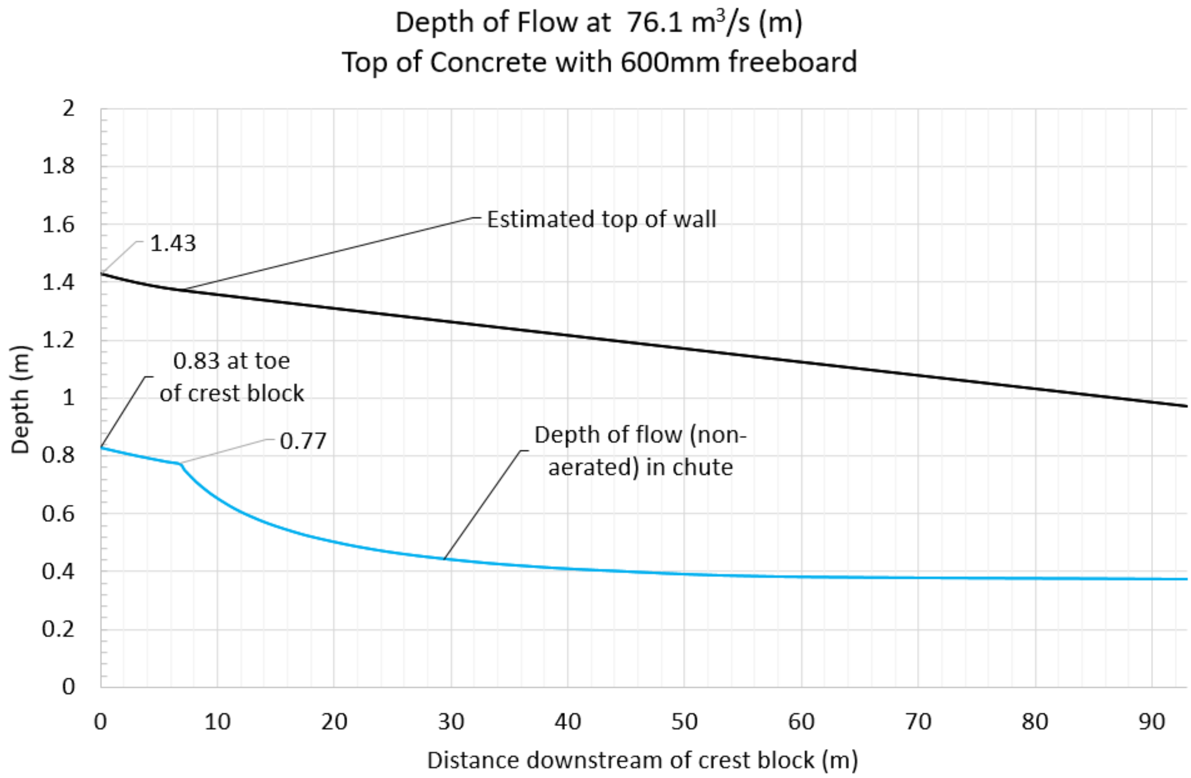


Figure 7.5: Depth of flow within chute for PMF outflow

7.3.3 Dissipator

The chute discharges into a dissipator (with 1H:1V sloping side walls) which is estimated to dissipate up to 80% of the flow energy before directing the flow over a side sill at right angles to the chute, into a natural gully, then to the waterway downstream of the storage.

The depth of water in the dissipator is controlled by the length and level of the side sill. The design levels proposed by DoC (2006) were checked against the criteria developed by Bradley and Perterka and presented in the USBR publication Design of Small Dams (1987). For a design flow of 76.1m3/s, the calculations show that an invert level of RL 25.0 m is appropriate, but there is room to raise this by 0.5m if needed. The use of smaller flows for the design of the dissipator is discussed below.

As for the spillway chute walls, the dissipator walls are proposed to be lined concrete walls, anchored into the foundation. Depending on the rock quality and groundwater levels, freestanding cantilever walls may be required.

It is noted that the flood level within the Tuross River at the Eurobodalla Gauge (close to the confluence with the unnamed tributory of the ESS catchment) for the 1 in 100 AEP flood event is estimated to be approximatley RL14.2m AHD (as reported in Volume 1 of this report 30012127_R04_V02). This compares to the design elevation of the dissipator wall of RL30.5m indicating that tail water levels are not expected to influence the spillway capacity.

7.3.4 Design Basis

The erodibility of the spillway was highlighted as a key issue by DoC (2006a). The spillway channel as such has been designed assuming that erosion of an unlined rock excavation channel will occur. This is in recognition of the fact that the spillway is expected to accommodate flows of approximately 75m^3/s during the PMF with flow depths up to 1.5m (refer Section 7).

As such the excavated spillway is proposed to be concrete lined along the floor and against the excavated batters.

The approach channel is short with little losses giving full head available at the crest to drive flows.

The approach depth of 800mm is the minimum to limit weed growth and this has been adopted for the revised design.

The need for underdrains will be assessed in detail when the geology of the foundation is known. With a narrow chute width the potential for development of significant uplift pressures is limited and anchorage is all that may be required.

The fully lined chute was required as part of the original concept to direct flow into the dissipator, however, from a risk/hazard viewpoint, the potential for the spillway to break out and erode the embankment is limited by the ridge of land between the bank and chute. Hence there is potential to save cost by removing the lining downstream of the change in grade should rock quality permit. Anchors are currently proposed to be concentrated in the upstream cutoff of each slab, and the upper third of the slab length (subject to outcomes of the geotechnical investigations and the rock quality in this area). The degree of concrete lining and slab anchoring will be assessed following completion of the geotechnical investigations.

Additionally, the width of the chute can be reduced at the bottom end to reduce the excavation, concrete quantities and anchors in the chute and dissipator. This would increase the depth of flow, but with velocities around 12 m/s, there is potential for some contraction of the width. This has the advantage of reducing the Froude Number (the velocity remains relatively constant because the total head is constant but the depth increases) with a consequent reduction in dissipation requirements.

The length of the dissipator basin could be further reduced if the design flow is lowered potentially reducing costs related to the dissipator (the crest remains the same). This is based on the presence of a gully located in the lower half of the spillway positioned remote from the embankment where failure or erosion in the area of the dissipator is unlikely to contribute to failure of the embankment. As such, a lower design flow for the dissipator could be tolerated (this approach at Eppalock Dam, Victoria). For example, designing the dissipator for the 1 in 100 AEP outflow would reduce costs in the dissipator without impinging on the safety of the dam. This may lead to ongoing increased maintenance for the dissipator, and as such is not the preferred approach.

The rock control weirs to be located downstream of the dissipator structure would be designed based on landscape design criteria for through-and-overflow rock weirs. As with the dissipator design, these could target a smaller but more frequent flow and still function well at higher flows.

8 HYDROLOGY AND HYDRAULICS

8.1 General

A consequence assessment for the ESS has been undertaken in accordance with ANCOLD (2012) and is provided in Appendix C. The assessment procedure expresses the consequences of dam failure in terms of the damages and loss to Council and third parties, the Population At Risk (PAR) and Potential Loss of Life (PLL). The failure consequences depend upon the number and type of properties within the inundated area of a potential dam breach outflow.

8.2 Flood Frequency

An analysis of the catchment hydrology was undertaken for the purposes of developing a flood frequency curve. The flood frequency curve has in turn been used to assist in developing the spillway design flood, the Probable Maximum Flood (PMF), the required freeboard, and the cofferdam design.

The study area is located on an unnamed tributary of the Tuross River with a catchment area of 4.6 km^2. The overall catchment ranges in elevation from approximately RL 1.0 m (at the confluence with the Tuross River) to RL 153 m, and consists predominately of forested area. Of that catchment, the proposed storage will command a smaller portion of that tributary with a catchment area of 1.6 km^2.

A RORB runoff routing model has been used to simulate the hydrologic performance of the ESS catchment and develop a flood frequency curve. The RORB modelling results for the catchment upstream of the ESS are presented in Figure 8.1 and Figure 8.2 with full details on model development and calibration included in Appendix C. It is noted that the peak inflow values reported in Figure 8.1 are for very short duration flood events, 15 or 20 minute storms. The peak outflow events are longer duration events with the critical duration storms events typically being 9 hour storms and 3 hour storms for the very extreme events. These results are provided in Table 2-12 of Appendix C.

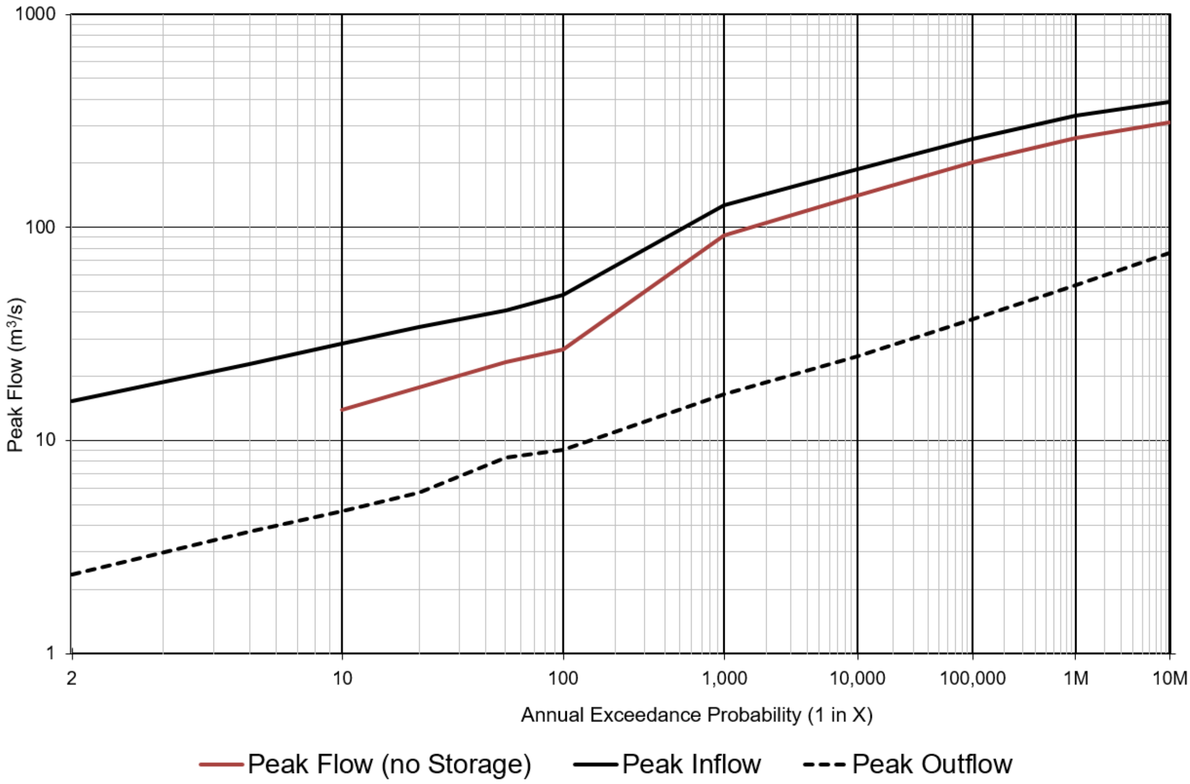


Figure 8.1: Flood Frequency Curve

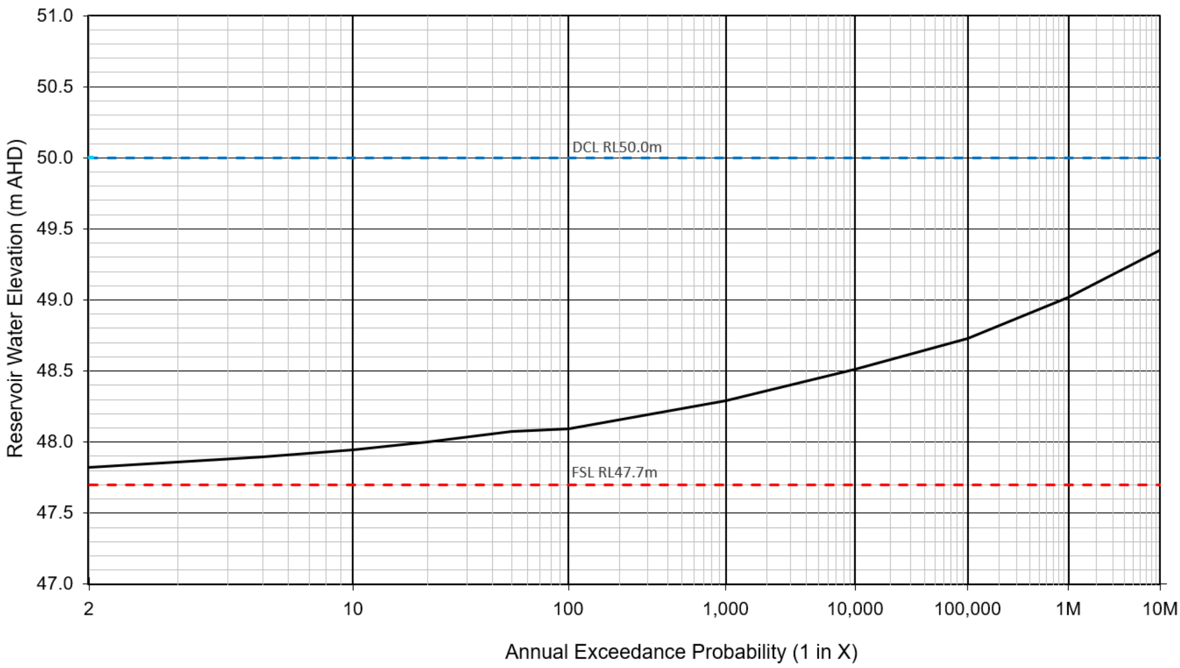


Figure 8.2: Elevation Frequency Curve

8.3 Probable Maximum Flood

From the flood frequency analysis, the RORB modelling results indicate the peak PMF inflow to the storage is estimated to be 390 m^3/s corresponding to a 15 minute duration storm event. The design peak PMF outflow from the storage occurs in the 3 hour storm event with a peak outflow estimated at 76.1 m^3/s. The PMF 3 hour event brings the storage to the design peak PMF water level of RL 49.4 m. Full details on the flood frequency analysis and RORB model development and calibration are included in Appendix C.

8.4 Consequence Assessment

The consequences of failure have been assessed for the ESS in accordance with ANCOLD (2012).

The scenarios considered in assessing the consequence category of the storage are listed below with discussion of the coincident flooding within the Tuross River and sensitivity analyses for these scenarios discussed in Appendix C:

- Sunny Day failure of the dam commencing at FSL
- 1 in 100 AEP flood without failure
- 1 in 100 AEP flood with failure
- PMF without failure
- PMF with failure
- 1 in 10 AEP flood with cofferdam failure

Aerial photographs were used to assess the number of properties and type within the inundated area. Flood affected properties were identified using a GIS software package (QGIS). Maximum water depth and Maximum Depth x Velocity (DV) outputs from dambreak modelling were used in assessing the PAR and PLL for the breach scenarios.

PAR was identified based upon the information available from the most recent census undertaken by the Australian Bureau of Statistics as available at www.abs.gov.au. PLL was computed using the approach described in USBR (2014b) but also giving consideration to the method described in Graham (1999).

The consequence category is computed in part utilising the severity of damage and loss (ANCOLD 2012). The severity is broadly classified in terms of damage costs, business, health and social and environmental impacts. In most scenarios considered, the magnitude of damage cost was medium, predominantly due to the expected cost of embankment repair works. “Impacts on the dam owners business” are estimated to be either medium or major. In particular, the major aspects are related to the effect on the owner’s credibility and political implications.

The estimated PAR and PLL for the various scenarios considered are provided in Table 8.1 with the assessed consequence categories presented in Table 8.2. Further detail on the procedure and results are provided in Appendix C.

Table 8.1: Assessed PAR and PLL

Consequence	Modelled Scenario		
	PMF	Sunny Day	Cofferdam (DCF)
Breach PAR	55	0	55
No Breach PAR	55	0	55
PAR Incremental	**0**	**0**	**0**
Breach PLL	0.45	0.45	0.65
No Breach PLL	0	0	0.56
PLL Incremental	**0.45**	**0.45**	**0.08**

Table 8.2: Assessed Dam Consequence Category

Event	Consequence Category
PMF	High C
Sunny Day	High C
Cofferdam (at Cofferdam DCF)	Very Low

8.5 Design Flood

8.5.1 Eurobodalla Southern Storage

ANCOLD (2000) and Dam Safety Committee Guidance Sheet DSC3B (2010) provide recommendations on the deterministic fallback flood capacity. The ESS is assessed to have a consequence category of High C for flood induced failure of the embankment. Based on the DSC3B, the recommended fallback flood capacity (or design flood) for a High C Consequence dam is a flood with an AEP of 1 in 100,000. From the flood frequency analysis, for a flood event with an AEP of 1 in 100,000 the peak inflow is estimated to be 261 m^3/s; peak outflow of 37 m^3/s and peak water level of RL 48.7 m.

The design criteria for the storage is to pass the PMF. As discussed in Section 8.3. This corresponds to a peak inflow of 390 m^3/s; peak outflow of 76.1 m^3/s and peak water level of RL 49.4 m.

8.5.2 Cofferdam

The cofferdam consequence category is assessed to be Very Low for a flood induced failure. NSW DSC does not have any specific requirements for a Very Low consequence storage, however advises that a flood capacity of 1 in 100 AEP will normally be in the interests of the owner for asset protection. Considering the ability to undertake emergency measures in the event of a flood threatening the cofferdam, a design flood of 1 in 10 AEP, as recommended by DoC (2006b), is considered appropriate for concept design. This will be reviewed during detailed design.

8.6 Freeboard

The freeboard for embankments has been assessed based on the procedure outlined by USBR Design Standards 13, Chapter 6 (2012) and the requirements set out in DSC3B guidelines.

The criteria for setting the crest elevation is presented below, and incorporates the requirements from both the USBR and the DSC3B guidelines:

- Minimum Freeboard to be the greater of:
  - Design Flood + 0.6m; and
  - Design Flood + runup and setup from wind velocity exceeded 10% of the time.
- Normal: FSL + runup and setup from 160kph wind velocity

The Design Flood that the embankment will be designed for is the PMF. The 0.6m above the Design Flood represents the minimum fallback freeboard requirement from the DSC3B guidelines for a High A or Extreme Consequence (i.e. for dams with a flood capacity of PMP DF/ PMF) category dam, noting that the assessed consequence category of the ESS is High C as discussed in Section 8.4 and Appendix C.

The effective fetch was calculated to be approximately 0.7km. The results of the freeboard assessment are summarised in Table 8.3.

Table 8.3: Freeboard Assessment Results

Calculation	Freeboard	Required Crest Elevation
Minimum Freeboard		
	PMF + 0.6 (DSC3B)	50.0m AHD
	PMF + Wind	49.4m AHD
Normal Freeboard		
	FSL + 160kph Wind	48.08m AHD

The assessment concluded that wave runup and wind setup resulted in marginal increases over the water level. This resulted from the fetch not having adequate length (Fetch <1km) to sustain and build up a significant wave. Additionally, the significant wave was further reduced based on the orientation of the longest fetch which strikes the embankment from the south west (approximately at 45° angle).

8.7 Embankment Crest Level

The highest crest elevation required based on the freeboard assessment therefore is RL 50.0 m, as a result of the freeboard requirement of PMF + 0.6m from DSC3B (for a High A or Extreme Consequence storage). This has been adopted as a conservative dam crest level for the purpose of concept design and includes the 0.4m thick crushed rock pavement below this level.

In recognition that the Consequence Category of the embankment is High C, consideration will be given to lowering the embankment crest level during detailed design. Table 8.4 below presents the potential design levels for different basis of design.

Table 8.4: Embankment Crest Level variations

Flood Capacity	Flood Level (AHD)	Freeboard (m)	Design Level (AHD)	Comment
PMF	RL49.4m	0.0	RL49.4m	Flood capacity to meet design criterion of PMF. Freeboard based on estimated wind runup and setup.
1 in 100,000 year	RL48.7m	0.4	RL49.1m	Design flood and freeboard in accordance with DSC3B for High C Consequence Storage.

To meet the design criteria of passing the PMF, the design flood level of RL49.4m is proposed as this exceeds the design level based on meeting the fallback flood capacity with additional freeboard.

The design level of RL49.4m has been adopted to be representative of the top of the core zone. Pavement (of low permeability) of 0.4m thickness is proposed on top of this, which provides an additional level of freeboard. This is considered to be a conservative basis for design, noting the estimated negligible impact of wind runup and setup and the design flood exceeding the recommended fallback flood capacity.

Therefore, the proposed flood capacity for the ESS is to pass the PMF to the top of the core, with the dam crest level 0.4m above this required for crest pavement. The options to meet this requirement will be considered during detailed design and will be either:

- Reduce DCL from RL50.0m adopted for concept design to RL49.8m (top of pavement); or
- Keep DCL at RL50.0m and reduce width of spillway to result in a peak reservoir level during the PMF of RL49.6m.

8.8 Cofferdam

The cofferdam diversion is to be designed to be integral with the low level outlet, which is proposed to comprise a 1200 mm diameter conduit. The diversion will act as an uncontrolled crest (culvert) structure. The capacity of the culvert has been computed as an open channel flow structure up to the height of the culvert and from there as an inlet control culvert.

The cofferdam storage and discharge characteristics have been incorporated into the RORB model with storm events up to the 1 in 10 AEP modelled to establish the required height of the cofferdam embankment.

The modelling indicated that the characteristics detailed in Table 8.5 would be required for the cofferdam. Further details are included in Appendix C.

The invert level of the cofferdam diversion (1200mm diameter conduit) has been designed at RL 25.0 m. This provides approximately 32 ML of storage available to the Contractor for use during construction (e.g. for dust suppression and material conditioning). Should the Contractor not require this volume of water, there will be potential to lower the invert level (and hence cofferdam height) of the conduit.

Table 8.5: Cofferdam Design Characteristics

Characteristic	Value
Outlet pipe invert	RL25.0m
1 in 10 AEP peak discharge	3.9 m^3/s
Peak 1 in 10 AEP water level (m)	RL28.6m
Cofferdam design crest level (m)	RL30.0m

9 INLET AND OUTLET WORKS

9.1 General

The inlet and outlet works form the means in which the reservoir is filled and emptied. The key components comprise:

- Inlet Chute: Structure conveying and dissipating water transferred from the river intake pump station via a pipeline constructed along the left abutment.
- Outlet Tower: The tower forms the structure which provides the ability to selectively draw from different elevations within the reservoir to be discharged either downstream or to the existing and future WTPs.
- Outlet Conduit: Conveys water from the reservoir under the embankment.
- Outlet Valve House: Provides the ability to control the outflow of the reservoir water and direct it to the relevant discharge point (i.e. downstream or to the existing or future WTP).

Most of the arrangements developed in the concept design by the DoC (2006b) have been maintained with minor amendments. A description of the components and the design basis for each component is provided in the following sections.

9.2 Inlet Works

The inlet comprises a riser box and baffled chute. The inlet is located at the end of the rising main from the river intake structure and comprises a standard 1m wide baffled chute, approximately 50m long, located on the left abutment upstream of the embankment.

The baffled chute is a standard arrangement for discharging down slopes to aerate flow and dissipate energy to avoid excessive erosion. This type of chute is well suited to this application as the energy is dissipated continuously along the chute and is independent of the water elevation, allowing the chute to work at all levels of the storage. The lower end has a horizontal apron surrounded by rock beaching to distribute the flow on first filling.

The design methodology is well covered in the USBR publication ‘Design of Small Canal Structures’ and the concept design follows closely the outcomes of this methodology.

The proposed arrangement has changed from the previous concept design (DoC 2006b) slightly which had an upturned pipe end in a closed pit discharging over a sill into the chute.

At the top end, the upturned pipe is terminated at the pit wall and the water energy is allowed to partially dissipate in the pit, with the water then flowing vertically up the pit in a uniform manner over the sill and onto the chute.

The 1m wide rectangular chute has 160mm high baffles, 250mm square, in alternating rows spaced down the chute to create turbulence and dissipate energy. The first baffle has been located at least 300mm below the level of the top sill. The spacing of the baffles is nominally 600mm and this is considered appropriate to aerate and dissipate the flows.

No underdrains or anchors are required, although shear keys/cutoffs are included at 5m intervals under the chute to prevent downstream creep. The exposed soil in the batters at the side of the chute are proposed to be covered with light beaching to prevent erosion.

9.3 Outlet Works

9.3.1 Outlet Tower

9.3.1.1 General

The outlet comprises an Outlet Tower with multiple intakes, a concrete encased 1200mm outlet pipe and a valve house with two control valves and tee connections for the existing Southern WTP and future WTP. The top of the tower has an access bridge to operate and change the level of the intakes via baulks and trashracks.

9.3.1.2 Outlet Tower and Pipe Location

The location of the diversion/outlet pipe and Outlet Tower has been relocated to the left abutment (originally proposed to be located on the right abutment by DoC, 2006b). This location better suits the diversion pipe which is installed through the cofferdam. The final plan position will be determined during detailed design and will consider such issues as the final stripping level along the outlet alignment, geology, impacts on the embankment above, depth of cut, location of the tower, length of access bridge and may include a horizontal bend to optimise the location of the tower.

This left abutment alignment is a departure from the original location and is largely due to:

- The cofferdam being located upstream of the embankment and not incorporated within the toe of the main embankment. This results in the cofferdam shifting to the left of the embankment, along with the diversion pipe.
- If retained on the right abutment, the diversion would be along the upstream toe of the embankment, which is undesirable. Relocating to the left removes this and improves the outlet alignment with the diversion pipe through the cofferdam.
- The longitudinal profiles will be similar to the original location, with similar depths of cut. The original depth of cut is 15m and relocation to the left side shows a marginally reduced cut depth.

The relocation to the left side has no adverse impacts on the overall length of the outlet, excavation quantities, or on the length of the access bridge.

9.3.1.3 Outlet Tower Description

The current concept design comprises the following components (refer Appendix B for Concept Design Drawings):

- Seven openings each 1.5m X 0.9m
- A submerged penstock guard gate in the tower base across the outlet pipe
- Trashracks and baulks as intended in the original concept design (DoC, 2006b).
- Provision for attachment of the diversion pipe.

The Outlet Tower is a standard configuration reinforced concrete, 30m high and 6m diameter vertical shaft.

The 30m height covers the Stage 1 development only. The tower would be required to be extended during future raising of the embankment (during Stage 3 works) to add approximately 12.7m to the tower height to meet the anticipated future embankment level of RL62.70m. Consideration was given to constructing the outlet tower to the full Stage 3 height during the Stage 1 works, however given the very long time period between the stages of works (approximately 50 years), delaying unnecessary expenditure to Stage 3 is recommended for financial reasons.

The tower has seven intakes to allow extraction of best quality water from the storage regardless of the level of the storage. Intakes are normally baulked but the selected outlets would have trashracks in lieu of the baulks to access better quality water. To limit the velocity through the trashracks, it is intended that three openings will be in use at any time. A monorail gantry crane is provided to manipulate the baulks and trashracks.

An additional environmental requirement is to limit trashracks velocity further to avoid drawing in fish species to the outlet. However, given that this is an off-river storage, filled largely by pumping, with no native fish permanently resident at the site, this seems to be of lessor importance, even if the storage is later stocked with fish.

To meet the design criteria of an operational flow of approximately 300L/s during Stage 2 operation, approximately 0.4m depth of water is required over a single port. Accordingly, the MOL for the storage has been adopted to be RL27.4m i.e. 0.4m above the invert of the lowest port. The elevation of the bottom port was designed at RL27m for the following reasons:

- Allowing for some siltation in the lower part of the reservoir; and
- Within an elevation close to the cofferdam crest level (RL30m) to allow access to the storage upstream of the cofferdam without requiring complete removal of the cofferdam.

The outlet pipe at the base of the tower would have a bulkhead gate installed in front of the pipe, allowing isolation for conduit or valve maintenance. The bulkhead gate would be selected for closure into (during) flow capability for emergency isolation of the conduit and valve pit.

A refilling bypass is included at the base of the tower. This will allow the tower to be fully emptied for inspection of the tower structure or the outlet pipe. This same pipe can be used to refill the outlet following inspection/maintenance. An outlet pipe air vent has been provided in the design, as a vent cast within the concrete.

The outlet pipe has been designed as a concrete encased 1200mm diameter MSCL pipe, with both the steel thickness and concrete encasement designed to resist hydraulic and external loads independently.

Providing a design for the outlet tower that has proved effective in other similar storages and which comprises relatively low-tech, high reliability, as was proposed by DoC (2006b) is considered an appropriate design for the ESS. Despite the proposed arrangement being relatively labour intensive to operate, based on the reservoir modelling discussed in Section 14.2, it is expected that the reservoir level will be able to remain relatively consistent and hence operation of the baulks and trashracks to access lower storage levels will be a relatively infrequent event.

However, during low flow in the Tuross River with no inflow into the storage and the outlet operating at the maximum demand (25ML/d plus treatment losses), the drawdown could exceed the height of an inlet in approximately 10 days, requiring that the baulks and trashracks be manipulated to a lower level every 10 days.

For the future raising of approximately 12.7m, an alternative to raising the outlet tower could be to attach a floating arm via a trunnion at the top intake to command the top portion of the reservoir. This could be retro-fitted to the top intake of this tower.

The base of the tower will be sized for stability under earthquake, but the shape and fittings are sized to suit the diversion and operational requirements. The 90 degree bend and plug bulkhead, as proposed by DoC (2006b), have been replaced with a submerged penstock gate and a flanged pipe section cast into the upstream wall for attachment of the diversion pipe. This pipe will be sealed and grouted once diversion capability is no longer required.

9.3.2 Outlet Pipe

The outlet pipe is proposed to be mild steel cement lined (MSCL) and 1200mm diameter, to meet the 1 in 10 AEP diversion flow during construction. This is the critical design flow for the outlet pipe with an estimate peak flow of 3.9 m^3/s as provided in Table 8.5. This corresponds to a peak velocity of approximately 3.4 m/s which is acceptable for a MSCL pipeline. The original design (DoC 2006b) also assessed the outlet sizing with respect to diversion, emergency dewatering, current and future demand and 'transparency' flows.

The diversion capacity will be the peak flow required through the outlet pipe and the ϕ1200mm is adequate to meet the diversion capacity for a 1 in 10 AEP flood event (as discussed in Section 8.5.2).

For its function as an outlet under the embankment, the pipeshell steel thickness will be based on the maximum of internal pressure or handling stiffness. The DoC design (2006b) shows 12mm thickness which should be adequate to carry all internal pressures including an allowance for transients. To ensure long-term safety, the vertical reinforcement of the concrete surround will be designed to be sufficient to carry the peak internal pressure (without external loads) at the yield strength of the bars, assuming the concrete is cracked through.

The concrete encasement will also be designed to carry external construction and embankment loads with minimal deflection.

A drainpipe and service conduit are proposed to be laid with the outlet pipe. The drainpipe will allow draining of the outlet tower if required, while the service conduit will convey hydraulic lines and other cabling required for the outlet tower.

9.3.3 Valve House

The original outlet valve house design proposed by DoC (2006b) comprised two fixed dispersion cone (FDC) valves (DN600 and DN300), two electromagnetic flow meters (DN600 and DN450), four butterfly isolating valves (2xDN750 and 2xDN450), one manhole and DN450 offtake for the water treatment plant. Individual concrete dissipator boxes were required to dissipate the energy from the cone valve discharge.

In order to minimise the potential for water discharging from the outlet pipe and eroding the downstream waterway, a Vertical Discharge Regulating (VDR) valve has been proposed. The VDR valve would discharge water into a stilling pit to dissipate the energy. The stilling pit comprises an overflow where the dissipated water flows back into the waterway.

The second/smaller cone valve proposed by DoC (2006b) was provided to accommodate smaller flows as the large cone valve could not operate effectively in the low discharge range (5-10% capacity). VDR valves do not have this problem, as they are able to operate effectively from near closed to fully open. As such, the second/smaller valve would not be required.

The main DN1200 conduit from the storage would enter the valve pit at the upstream end. A DN900 manhole has been incorporated immediately downstream of the pipe entry into the pit, and upstream of the DN1200 x DN600 reducer. The DN600 conduit leads to an isolating valve, flow meter and the discharge valve.

The revised arrangement uses only one VDR valve to accommodate the entire range of discharge. This type of valve is typically operated by hydraulic cylinder.

The size of the stilling/dissipating pit as shown in the current concept drawings (refer to Appendix B) may be refined and reduced in size if smaller discharge flows can be more accurately defined during the detailed design.

An additional scour line and a valve are also included as part of the design. A DN150 gate valve would be suitable. The purpose of this scour line is to discharge sediment out from the main outlet pipe.

A DN600 offtake and isolating valve is provided for cross connection to the storage inlet pipeline to allow transfer of water to the existing Southern WTP during Stage 1 operation. Branching off from this DN600 offtake is a blank flange for connection to the future WTP when commissioned (Stage 2 works).

All isolation valves require the ability to close into (during) flow. Butterfly valves with hydraulic cylinders are proposed for the valves inside the pit.

A single hydraulic power pack could be utilised for the butterfly valves and the VDR valve. Electro-mechanical actuators are also widely used and could be incorporated into the design. Main power would be from the grid with optional use of generators. Tertiary power supply may be via use of hand pumps (or handwheel if electro-mechanical actuators are used).

The manual DN600 isolating valve for the existing Southern WTP is to be used temporarily until the plant is decommissioned. As it is located outside the valve pit and buried, it is proposed that a gate type is more suitable.

The flow meter is to be the electromagnetic type.

Three stairways are provided for inspection and minor maintenance. For installation and major maintenance, a mobile crane or hiab could be used. Preference is for an open-air valve pit to allow use of mobile cranes when required, rather than constructing the valve pit within a building, requiring installation of a permanent gantry for maintenance purposes. Drainage will be provided in the pit to manage rainfall inflows.

9.3.4 Transparent Storage

Should the reservoir be required to act as a transparent storage for up to the 1 in 10 AEP flood events (subject to confirmation during the environmental assessment), this is proposed to be undertaken by releasing an equivalent volume of water to the rainfall runoff entering the storage on a daily basis. However, it is noted that the size of the catchment upstream of the ESS is approximately 1.6 km^2 and forms approximately 0.1% of the Tuross River catchment with an approximate area of 1,586 km^2. The catchment downstream of the ESS is predominantly agricultural land with flow directed through a constructed channel.

The reservoir level would be monitored during a rainfall event with an equivalent volume of water then released through the outlet works to the downstream creek. Should pumping into the storage from the river intake pump station be undertaken concurrently with a rainfall event, this will be allowed for within the Programmable Logic Controller (PLC) and the volume of water released adjusted to account for this.

Based on the hydrological assessment (refer to Appendix C), the estimated peak inflow rates and total inflow volumes for flood events up to the 1 in 10 AEP flood are presented in Table 9.1. The total inflow volume represents the maximum volume of water required to be released from the storage following a storm event to meet the transparent storage criteria i.e. should this be a requirement of the storage.

Table 9.1: Estimated Storage Inflow Rates and Volume

AEP (1 in X)	Peak Inflow Rate m^3/s (Storm Duration)	Total Inflow Volume ML (72hr Storm Duration)
2	15.4 (20 mins)	192
5	23.0 (20 mins)	321
10	28.5 (15 mins)	410

10 EARTHQUAKE LOADING

10.1 Design Earthquake

10.1.1 Earthquake Loadings

A seismic assessment was previously undertaken by ES&S (now called SRC) on behalf of DoC (2006b). An updated seismic assessment is currently underway. It should be noted that the term Maximum Design Earthquake or MDE (used by DoC previously) has been replaced with Safety Evaluation Earthquake (SEE) and this will be used for this project.

The recommended PGA from the updated seismic assessment, will be adopted for use in the seismic design of the storage. It is expected that PGA values corresponding to earthquake magnitudes of Mw=4 or greater will be adopted corresponding to the return periods for the OBE and the SEE. Magnitudes of 4 or less are considered to rarely cause damage to dam structures.

The return period to be adopted for the OBE and the SEE are provided in the subsequent sections of the report.

10.1.2 Seismic Design

Embankment Design

The seismic assessment of the embankment will be undertaken using the methodologies presented in the *ANCOLD Guidelines for Design of Dams for Earthquake* (1998). SMEC will also review the assessment in conjunction with the updated ANCOLD guidelines for the *Design of Dams For Earthquake*, which is still in draft form. SMEC is part of the team preparing this ANCOLD update and are cognisant of the changes being proposed. SMEC will apply updated procedures outlined in these guidelines should the guidelines be released during the detailed design stage.

Should the embankment materials be assessed to not be susceptible to liquefaction, the embankment performance under seismic loading would initially be considered using a screening level assessment (e.g. Perlea and Beaty, 2010) and if warranted (FoS below 1), a simplified method for estimating post-earthquake deformations will be undertaken (Bray & Travasarou, 2007). The tolerability of post-earthquake deformation estimated from the above method would be assessed by considering the available freeboard to the full supply level.

Appurtenant Structures

The appurtenant structures considered as part of the seismic loading include:

- Outlet tower including access bridge
- Outlet conduit
- Outlet works

The appurtenant structures will be assessed to meet the following performance criteria as outlined in the ANCOLD Earthquake guidelines (1998) which has been endorsed as a suitable criteria by DSC3C (Section 5.8).

Table 10.1: Performance Criteria for Appurtenant Structures

	OBE	MDE
Outlet Tower (incl access bridge)	Stresses induced satisfying AS3600 requirements.	Emergency closure and regulation of flow to maintain operability.
Outlet Conduit	Stresses induced satisfying AS3600 requirements.	Conduit not to collapse or rupture.
Outlet Works	Maintain normal operating capabilities.	Emergency closure and regulating valves to maintain operability.
Access Roads	Must be able to be passable.	Can become impassable, but must have ability to be cleared easily.

The outlet tower is expected to be the most critical appurtenant structure as it comprises a free standing concrete tower over 40m in height when raised as part of the Stage 3 works, a diameter of 6m and a 9m square base (refer Concept Drg ST-3200). The SSE will be adopted for assessing the stability and structural capacity of the tower at the lower intake, the tower/base connection and tower/conduit connection. The analysis will not cover the assessment of the earthquake loads on internal mechanical components of the tower such as the gates, lifting winch, and access ladders and platforms. The effect of these components on the concrete Outlet Tower will be included in the analysis on the tower as added mass at the appropriate nodes in the model which will influence the natural frequency of the tower.

The access bridge piers will be assessed in the same manner as the outlet tower, with the bridge mass being included as loads on the piers. The SSE will be adopted for use in assessing the access bridge and piers.

The conduit is not expected to have additional loading as the design is to be fully encased in concrete within a moderately weathered rock trench. The conduit/valve pit connection, however will have the embankment SSE loading applied as it is required to be functional following an earthquake event.

For other components of the outlet works, the seismic assessment will include the identification of the failure modes and whether or not these failure modes could lead to failure of the embankment. Should these failure modes lead to embankment failure, the same SSE will be adopted for the outlet works as the embankment.

10.1.3 Operating Basis Earthquake (OBE)

The DSC3A guidelines state that:

"The DSC has no requirements for earthquake stability of new or existing outlets towers, bridges and ancillary works unless their failure would result in uncontrolled loss of storage or would threaten dam failure".

ANCOLD (1998) guidelines on earthquake design suggest an OBE of 1 in 200 AEP. This will be adopted with no damage sustained generally, however for different components of the storage (particularly the outlet works), minimum design loads will apply based on requirements in AS1170.4 (Minimum Design Loads on Structures, Part 4, Earthquake Loads).

10.1.4 Safety Evaluation Earthquake (SEE)

The DSC3C guidelines recommend that all high and extreme category dams be designed to withstand earthquake shaking without an uncontrolled loss of storage due to partial or complete failure of the dam as part of the MDE. The suggested AEP of the SSE recommended by DSC are presented below in Table 10.2

Table 10.2: DSC3C Recommended AEP of Maximum Design Earthquake

Consequence Category	Earthquake Annual Exceedance Probability (AEP)
Extreme	<1 in 10,000
High A	1 in 10,000
High B	1 in 5,000
High C	1 in 1,000
Significant	1 in 500

Given that the Sunny Day consequence category for Stage 1 embankment is High C, the AEP required for event, is the 1 in 1,000 AEP.

No consequence assessment has currently been undertaken for the raised embankment (as part of Stage 3 works). In recognition that this upgrade may be required several decades into the future, undertaking a consequence assessment for the raised embankment now may not accurately reflect the true consequences at the time it is constructed. Furthermore, should the consequence category for the raised embankment be higher, with a corresponding lower AEP (i.e. more extreme) design earthquake, the raised embankment could be designed to accommodate the higher earthquake loads. Additional measures incorporated into the future raised embankment to address potential higher seismic loading could include a wider crest with additional freeboard to allow for larger deformations.

Conversely, the outlet tower and outlet conduit must be designed to accommodate the design earthquake for the raised structure. This is due to the fact that any future upgrades to the tower (i.e. widening the base and shaft etc) or outlet conduit would not be easily undertaken without significant excavation and complete draining of the reservoir.

Therefore, in recognition of the uncertainty of the future conditions downstream and the resulting future consequence category for the raised embankment, it is assessed that the outlet should be designed for the 1 in 10,000 AEP earthquake (corresponding to the High A and Extreme Category). The sensitivity of this design standard on the requirements for the outlet tower and conduit will be reviewed following completion of the seismic study.

11 INSTRUMENTATION

11.1 General

Instrumentation is proposed to be installed both within the embankment and the outlet tower. The monitoring that is proposed will be determined based on an assessment of the potential failure modes and is likely to include:

Embankment:

- Seepage measurement structure to measure flow and potentially chemical analysis
- Pore pressure
- Surface movement points

Outlet Tower:

- Reservoir level measurement
- Temperature
- Water quality parameters (these are detailed in Section 13.2)

11.2 Instrumentation

The following instrumentation and a brief description of their purpose and expected location is described below.

11.2.1 Embankment

Seepage Measurement Structures

A seepage monitoring weir is proposed to be installed at the downstream toe of the embankment to measure the seepage flow being collected and conveyed from the filter drainage system within the embankment via the downstream toe drain.

The seepage monitoring structure will likely comprise a V-Notch seepage weir for measurement of flows.

Vibrating Wire Piezometers (VWPs)

VWPs provide the most efficient means of obtaining data on pore pressures within the bank. When installed together with telemetry, the VWPs will provide Council with the ability to monitor pore pressures within the bank in real time. Alternatively the VWPs can be fitted with data loggers that are periodically downloaded.

A series of VWPs will likely be installed within the embankment at the following locations:

- At various elevations within the core
- Within the downstream shoulder
- Adjacent to the downstream blanket filter layers

As part of the future embankment raise, the VWP cables could be extended further up the embankment to maintain continuity in readings at the same elevation. These VWPs will also provide the ability to monitor the embankment pore pressures during construction of the embankment raise.

Surface Movement Points

Surface movement points are proposed to be installed to monitor the movement of the structure over time. These points will likely be installed along the upstream and downstream edges of the crest and along the downstream berms.

11.2.2 Outlet Tower

Reservoir level measurement is proposed to be undertaken within the outlet tower. The water level can either be achieved by installing a measurements staff against the outlet tower with graduated intervals, or a dip meter lowered into a pre-formed wet hole within the outlet tower or just within the outlet tower itself.

12 ACCESS ROAD AND BOAT RAMP

12.1 ESS access road

The ESS access road is required to provide access to the storage during construction and permanent access during operation.

The ESS access road is proposed to connect to Eurobodalla Road opposite to the existing Southern WTP at the location of the existing driveway to 530 Eurobodalla Rd. The alignment of the road is proposed to generally be between RL 20 m and RL 25 m along the valley up to the proposed outlet valve house at approximately RL 21 m. A hardstand area will be provided at the outlet valve house to allow parking for maintenance vehicles.

The access road will then be constructed up the left abutment connecting to the crest of the embankment at RL 50.0 m.

The road elevation is required to be designed above the elevation of Eurobodalla Road, to ensure that access to the storage is not limited (by the new road) during a potential dam safety event. To the North and South of the proposed intersection of the ESS access road with Eurobodalla Road, the elevation of Eurobodalla Road reduces to approximately RL10m. Given the design elevation of the ESS access road is typically above RL20m, in the event of a severe flood, access to the storage is expected to restricted by flooding of Eurobodalla Rd. Alterative access can be gained to the site from Big Rock Rd, via the Princes Hwy.

The ESS access road will be designed in accordance with Austroads Guide to Road Design and Eurobodalla Shire Council's Infrastructure Design Standard. In-line with Council's infrastructure design standards the road would be classified as a 'Minor' road with less than 250 vehicles per day expected.

Council's requirements with respect to the ESS access road include:

- Road is required to be sealed for the full length between Eurobodalla Rd and the storage; and
- Pavement width is required to be 6.5 m in accordance with a two-lane 'Minor Road' from Council's Infrastructure Design Standards.

Figure 12.1 shows the typical cross section for the ESS access road. The width of the road is proposed to be typically 10 m wide, which includes 6.5 m for the sealed carriageway, 2.0 m wide shoulder on the cut side to allow for services including water mains and potential electrical conduit, and 1.5 m shoulder on the fill side to allow for guardrail construction. During detailed design the width of the road will be refined to include localised widening around bends.

Batter slopes are currently proposed to be 2H:1V for fill batters and 1.5H:1V for cut batters, however these will be subject to confirmation following the geotechnical investigations.

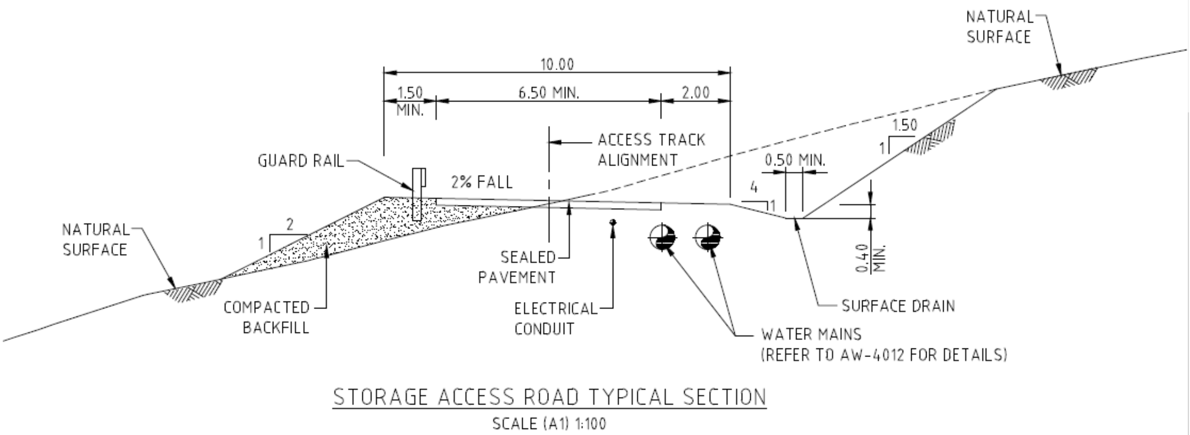


Figure 12.1: ESS access road Typical Section

A future extension of the access road will be required to be constructed up to approximately RL 80 m to the ridgeline on the left abutment to access the proposed future WTP to be constructed as part of the Stage 2 works.

12.2 Boat Ramp

The boat ramp will be designed in accordance with the NSW Boat Ramp Facility Guidelines (NSW Roads and Maritime Services, 2015). The boat ramp is proposed to be constructed on the left abutment and connect to the outlet tower bridge. Details of the boat ramp include:

- Maximum grade of 7H:1V;
- Minimum width of 4.5 m;
- Hardstand at top of ramp at RL 50 m to allow for vehicular turnaround;
- Base of ramp at RL 33 m allowing for a minimum water level of RL 34 m to utilise the boat ramp

13 WATER QUALITY AND CATCHMENT MANAGEMENT

13.1 Destratification Design

13.1.1 Thermal Stratification

Thermal stratification is a seasonally variable natural occurrence observed in many lakes and water bodies. Generally in the warmer months the surface waters increase in temperature which begins the process of thermal stratification.

Thermal stratification occurs when a separation of surface layers and deeper cooler layers occurs, largely driven by the effects temperature change on the density of the water within the water column. The potential for a water body to stratify is not necessarily based on the peak maximum temperatures but more the extent of the seasonal variability in temperatures. This separation can be evidenced by the appearance of a temperature gradient through the water column. The “separation”, during warmer seasons maintains a low oxygen environment in the deeper water body zones and particularly at the interface with the sedimentation layer. Conversely when the surface waters cool the phenomenon known as “lake turnover” can occur where cooler and more dense surface waters (due to prolonged cool weather after a warmer period) sink, resulting in poor water quality, and often nutrient spikes, distributed through the water body. This occurrence has a number of physical and chemical impacts on the water body which can lead to a range of water quality issues, including, but not limited to the following

- Nuisance algae bloom spikes (including Blue Green Algae) supported by the release of nutrients bound in the sediments.
- Release of iron and manganese, which are generally bound in the anoxic sediment, due to the sudden exposure of waters with a higher dissolved oxygen content leading to taste and odour problems.
- Significant environmental impacts i.e. fish kills.

These water quality impacts have a significant impact on the ongoing costs of water quality management, particularly when the water is relied upon for potable purposes as will be the case for the ESS. In such cases treatment to remove tastes, toxins and design considerations to manage physical impediments from filters and screens must be considered along with additional labour resources. To minimise the water quality impacts a management approach involving the maintenance of a destratified water body is required. This involves minimising the variance in temperature gradient through the depth profile of the ESS

Increasingly, destratification systems for water bodies of the capacity, surface area and depth of the ESS are included in the design and overall management of the storage to maintain a consistent temperature profile over the vertical water column. Such installations have been observed to reduce the impacts of water quality issues created by the seasonal thermal stratification, reducing the risks associated with fluctuations in water quality and ultimately reliability of supply.

Both stages of the proposed ESS, with an increase in volume from approximately 3,000 ML for stage 1 to 8,000 ML in the future, will be subject to the impacts of thermal stratification. The bathymetry which will range from its deepest section of approximately 32 m during stage 1 to 45 m following raising of the embankment, provides a typical set of physical conditions which are favourable to thermal stratification developing. Regional temperature fluctuations also demonstrate considerable seasonal variability further increasing the likelihood that the proposed storage will be impacted by thermal stratification without measures to thermally destratify the water profile.

13.1.2 Option Assessment

A range of options are available for destratification which include both custom designed solutions and suitable propriety products. The two common accepted methods are through the direct injection of air, or bubble plumes, through a network of pipes laid on the bottom of the reservoir and supplied by a shore based compressor or the installation of floating or fixed large impellers. The DoC (2006) design incorporated a combination of bubble plume system and fixed impeller. Both options have a range of considerations and have had success in a range of installations across Australia. Key considerations in assessing which system to use are as follows

- Performance – will the water quality objectives be satisfied?
- Capital cost – total cost for supply and installation
- Operation costs – critical consideration specifically for power costs and maintenance requirements
- Reliability and equipment backup
- Safety and accessibility

In assessing a system which will provide a suitable output to avoid stratification, a comparison of three systems was undertaken. These are listed below and described in the following sections.

1. Bubble Plume System as proposed by DoC in the 2006 concept design.
2. SolarBee surface mounted units as installed at Deep Creek Reservoir.
3. ResMix3000 manufactured by WEARS Australia.

13.1.2.1 Bubble Plume System

Bubble plume systems provide a common means of providing destratification of a range of water bodies through the direct injection of compressed air, through a number of submerged ports creating a vertical flow profile across the water body. While this system is effective in maintaining suitable conditions it does have a number of negative considerations. The previous design involved the placement of a network of pipe anchored on the reservoir floor connected back to a compressor mounted within a purpose built structure at the downstream toe of the embankment. The capital costs of this system was estimated to be $345,000 (base estimate by Evans and Peck, 2006) which was escalated to approximately $450,000 (base estimate) in the revised cost estimate (SMEC 30012127_R01_V02 in October 2016). The operational costs of the system from both a power consumption and service requirements were estimated at approximately $20,000 per annum by DoC (2006) based on a average cost of electricity of $0.1/kWh. With a number of less capital and operational cost intensive solutions on the market this solution while likely to be effective is considered to be expensive.

13.1.2.2 SolarBee

Medora Corporation (distributor of SolarBee) proposed a solution to maintain suitable water quality through the installation of 3 SolarBee model SB10000LSv20 reservoir circulators at even distances along the longest fetch of the proposed reservoir. The units consist of a surface mounted impeller, a draw tube and floats anchored back to the reservoir floor. The unit is powered by a PV solar arrangement and does have battery standby. It is acknowledged that these units are installed in the Deep Creek Reservoir but data representing level of a performance was not available at the time of writing this document. Based on discussions with Council the SolarBees are reported to be generally performing well but they were being supplemented with a compressed air system from time to time.

The cost estimate provided by Medora Corporation was $124,050 US Dollars (approx. $161,000 AUD March 2017) and installation and freight was additional to this at an estimated $55,000. It is understood that the units require monthly servicing to clean solar panels and remove fouling from aquatic weed growth. The inherent Work Health and Safety (WHS) risks associated with the process of servicing the units on the water at such a frequency is a negative consideration. On the basis of the SolarBee cost of approximately $216,000 capital outlay plus ongoing maintenance and the ongoing HW&S liability it was determined a more suitable option should be explored for comparison.

13.1.2.3 ResMix3000

ResMix3000 provides an economical low maintenance option for surface mounted destratification and is supported by considerable performance data. The ResMix unit is surface mounted on floats which are anchored in the same manner as the SolarBee. An example of the ResMix3000 system is shown in Figure 13.1. The unit has a low scheduled maintenance frequency (annually) and is comparable in price to the SolarBee option with an installed cost of approximately $180,500 (ex GST). Product back-up and service is available on the Eastern Seaboard with manufacture undertaken in Brisbane. Importantly its serviceability and access eliminated many of the WHS issues that the SolarBee encountered.

Figure 13.1: Example installation of ResMix3000 (Photo supplied by WEARS Australia)

For periods, it is expected that the ResMix will be continuously operated throughout the day i.e. 24 hours of operation. Accordingly, the use of a photovoltaic system as the sole power supply to the ResMix is not expected to be economic due to the high costs of storage required to allow operation in non-daylight times.

The ResMix3000 has a 2.2kW electric motor with annual operating costs estimated at $4,000 assuming electricity costs of $0.3/kWh and seasonal operation of the system. The operational costs are summarised in Table 13.1.

Table 13.1: ResMix3000 Estimated Operational Costs

Item	Cost	Frequency
Power variable @ $0.30/kWh	$4,000	Annually
Service	$3,500	Annually
Motor	$2,000	5 yearly
Refit	$18,000	After 15 years

13.1.3 Recommended Destratification System

On assessment of the three options listed above, the compressor and diffuser network are considered an expensive option from both a capital outlay perspective and annual operating costs. The recommended system is therefore based on the outputs of a surface mounted impeller type system (namely SolarBee or ResMix) due to the financial advantage these types of units offer.

It is uncertain whether a SolarBee option will maintain suitable water quality conditions due to the lack of supporting data and the fact that the system installed in Deep Creek Reservoir is being supplemented by an air compressor and diffuser network.

The ResMix system is supported with detailed performance data that indicates the proposed depth of the ESS is within its optimum range of effectiveness.

In comparing surface mounted and in-situ type circulation impellers such as the SolarBee and ResMix, WHS implications must be considered during servicing and maintenance. The SolarBee places operators at an increased risk of harm due to the frequency of required servicing from a suitable water craft. Such a requirement should be reduced through design or alternate options which minimise these risks.

It is recommended that the ResMix300 system be adopted during operation of the Stage 1 ESS to mitigate the potential for thermal stratification of the water body. It is proposed to install the system within 100 m upstream of the embankment and above the deeper zones of the storage as shown in Figure 13.2. This will allow for the greatest mixing across the water body and gain maximum horizontal influence through the storage.

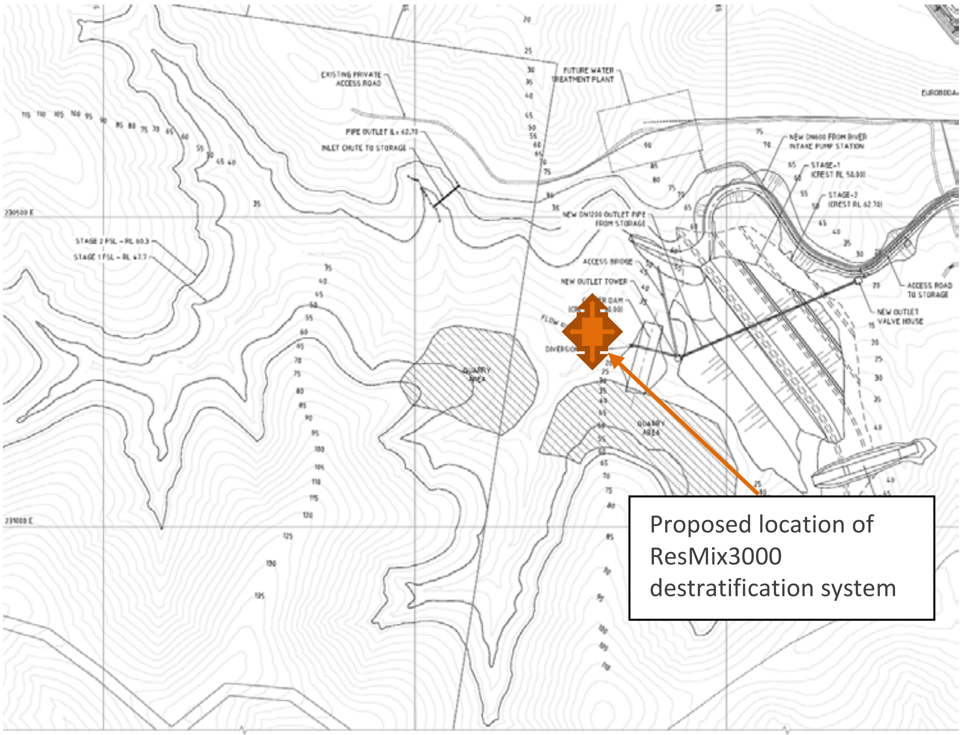


Figure 13.2: Proposed location of destratification system

13.2 Water Quality Monitoring

During operation of the storage, water quality monitoring will be undertaken to provide information to guide operation of the storage. Monitoring of the storage is required for two purposes:

1. Assessment of the effectiveness of the destratification process; and
2. Determination of any changes to parameters that are of significance to design of the future WTP.

Water quality monitoring of the raw water supply from the Tuross River is planned to be undertaken as part the EIS as outlined in Section 2.5 of the Volume 1 report. This water quality data will also be used to understand the water quality of the raw water source (prior to transfer to the storage) and assess the need for additional treatment measures including to address potential microbial risks.

With regard to management of the storage, the key issue is destratification. As described in Section 13.1.1, destratification is necessary, as the development of separate layers results in differing pH, oxygen and other conditions, which in turn results in different quality of water in the storage, at various depths.

The following parameters are proposed to be monitored to determine the efficacy of destratification:

- Radar measurement of the water level in the storage;
- Temperature measurement across the water column;
- Measurement of dissolved oxygen across the water column (typically, at or near the surface, and at around 2 m above floor level);
- pH and turbidity across the water column (typically, at or near the surface, and at a lower level); and
- Chlorophyll-*a* of the water at surface (an increase in the concentration of chlorophyll-*a* is an indication of active algae growth).

Effective destratification will generally provide good maintenance of water quality, but there are risks to be considered including:

- The possibility of algal blooms, which result in increased solids and other loads, and may require the use of Powder Activated Carbon (PAC) during treatment;
- Possible further contamination from runoff into the storage (unlikely to be an issue, as the catchment is relatively small, stock will be excluded and catchment access tracks will be graded to fall away from catchment); and
- The risks of chemical changes to the water in the storage.

As an example of the latter, evaporation can concentrate Total Dissolved Solids (TDS) when detention is very long, and iron species can change under some circumstances, resulting in more soluble iron and thus potential problems with staining in downstream uses. Effective destratification typically manages these risks.

To enable adequate understanding of these issues, it is recommended that samples be taken from the storage during operation at different water depths and tested for:

- Total alkalinity;
- Hardness;
- Soluble and total Mn;
- Soluble and total Fe;
- True colour;
- TOC;
- Temperature and pH; and
- Turbidity.

Initially, it is suggested that these samples be taken quarterly, with a review of the frequency and timing after a few years of initial data have been accumulated.

Furthermore, it is recommended that the following microbial testing be undertaken at both the river intake and storage to assess the treatment requirements:

- Escherichia coli;
- Protozoa - Cryptosporidium;
- Bacteria - Campylobacter; and
- Viruses - Norovirus.

13.3 Catchment Management

The catchment management measures proposed as part of construction of the ESS include:

- Grading of access tracks and/or drainage measures to ensure that rainfall runoff from access tracks drains away from the catchment. The intent of this is reduce potential contaminated runoff entering the storage.
- Fencing of the catchment boundary to reduce the risk of livestock entering the catchment.
- Maintaining vegetation within the catchment to reduce sediment-laden runoff.
- Provision of erosion control downstream of spillway chute and outlet works to reduce erosion and sediment transport.

The above measures would result in the ESS catchment being classified as a Category 1 Protected Catchment in accordance with the *Australian Drinking Water Guidelines: Draft Framework on Microbial Health Based Targets (National Health and Medical Research Council, 2016)*, noting that the raw water source from the Tuross River would classify as a Category 4 Unprotected Catchment.

14 FILLING STRATEGY

14.1 Water Sharing Plan

The Water Sharing Plan for the Tuross River Unregulated and Alluvial Water Sources 2016 (Water Sharing Plan) sets out the extraction limits relating to the Tuross River. The extraction limits as they relate to the Eurobodalla Southern Storage are outlined in Section 2.4 of the Volume 1 report.

14.2 Reservoir Level Modelling

Reservoir level modelling was undertaken to understand the potential time required to fill the storage and the rate of rise of the storage. The analysis was based on the following:

1. Historic stream flow data in the Tuross River at the Eurobodalla gauge (218008) over a 10 year period from 2007 to 2016, noting that South East of Australia was undergoing a period of prolonged drought between 2001 to 2009. This information was accessed from the NSW DPI website.
2. Assumed inflow rates into the storage at the maximum permissible extraction rates allowed for in the Water Sharing Plan.
3. No losses in the transfer (e.g. pipe leakage) and storage (through seepage and evaporation) of water has been allowed for in the analysis.
4. It has been assumed the storage is required to act as a transparent storage during filling i.e. no inflows into the reservoir from the storage catchment have been allowed for.
5. No restriction on the rate of rise of the storage.

Assumptions 2 and 3 above are considered conservative for the purpose of assessing the maximum possible rate of rise for the storage.

The modelling was undertaken based on the available historic data for individual years and is presented in Figure 14.1. Excluding year 2009, which was a particularly dry year, the time to fill the reservoir to FSL is estimated to take between approximately four to six months' time assuming filling were to commence on the 1st of January. Accounting for the likely seepage and evaporation within the storage and restrictions on the allowable rate of rise of the storage (as discussed in Section 14.3) would increase this timeframe. Based on the flow rates observed in the Tuross River in 2009, the storage would only have reached approximately RL42.7m (5m below FSL) by 31 December 2009.

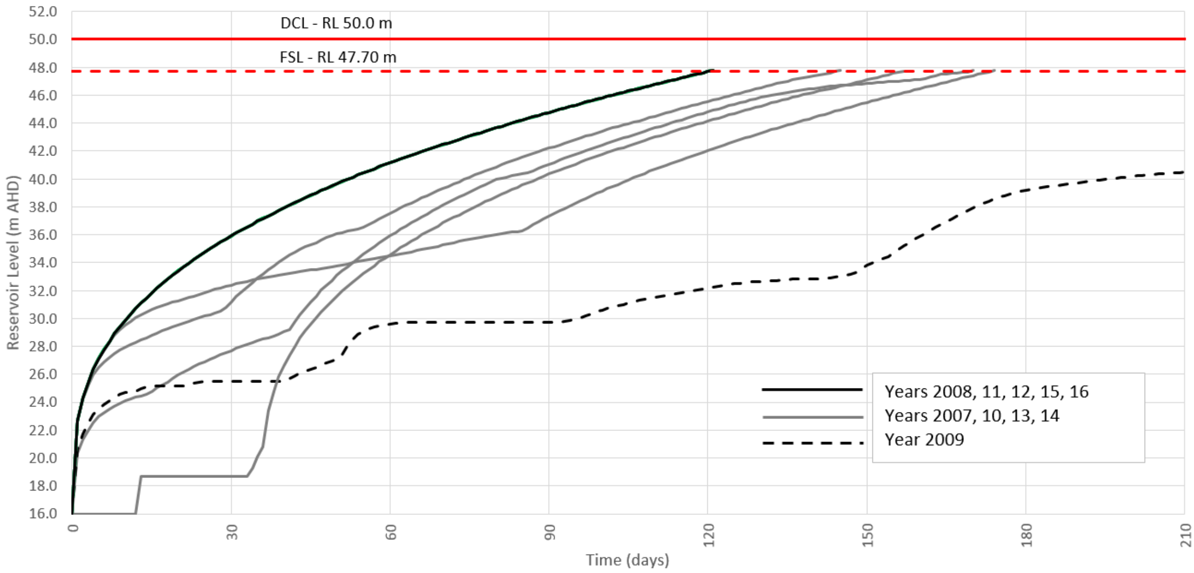


Figure 14.1: Reservoir level based on historic gauge data

14.3 Reservoir Level Rate of Rise

Of particular concern during first filling of the storage is the deformation of the embankment due to the applied water load and potential collapse compression on wetting of the upstream permeable rockfill zone. To enable monitoring of the deformation behaviour of the embankment during first filling, the maximum rate of rise for the storage is typically limited to 0.3 m/day.

Based on the maximum potential inflow into the storage of 26 ML/d (assuming the reservoir acts as a transparent storage) the maximum potential rate of rise for the ESS is shown in Figure 14.2. Based on the modelling, the rate of the rise is expected to be greater than 0.3 m/day over the first 20 days of filling. This occurs when the storage is at low reservoir levels, with relatively minor changes in volume corresponding to high changes in reservoir stage. This is not necessarily expected to be of a major concern given the relatively low loads imposed on the embankment and greater width of core available at the base of the embankment. Nevertheless, it is considered prudent to manage the rate of rise of the storage during these early filling stages to monitor the behaviour of the embankment.

The analysis shows that once the water level in the reservoir exceeds RL33.4m, the rate of rise cannot exceed 0.3 m/day based on the maximum permissible extraction rate from the Tuross River.

If the reservoir is restricted to a rate of rise of 0.3 m/day between RL20m (approximately 3m above upstream toe) and RL33.4m, the minimum duration to fill the reservoir to FSL at RL47.7m is estimated to be approximately 146 days.

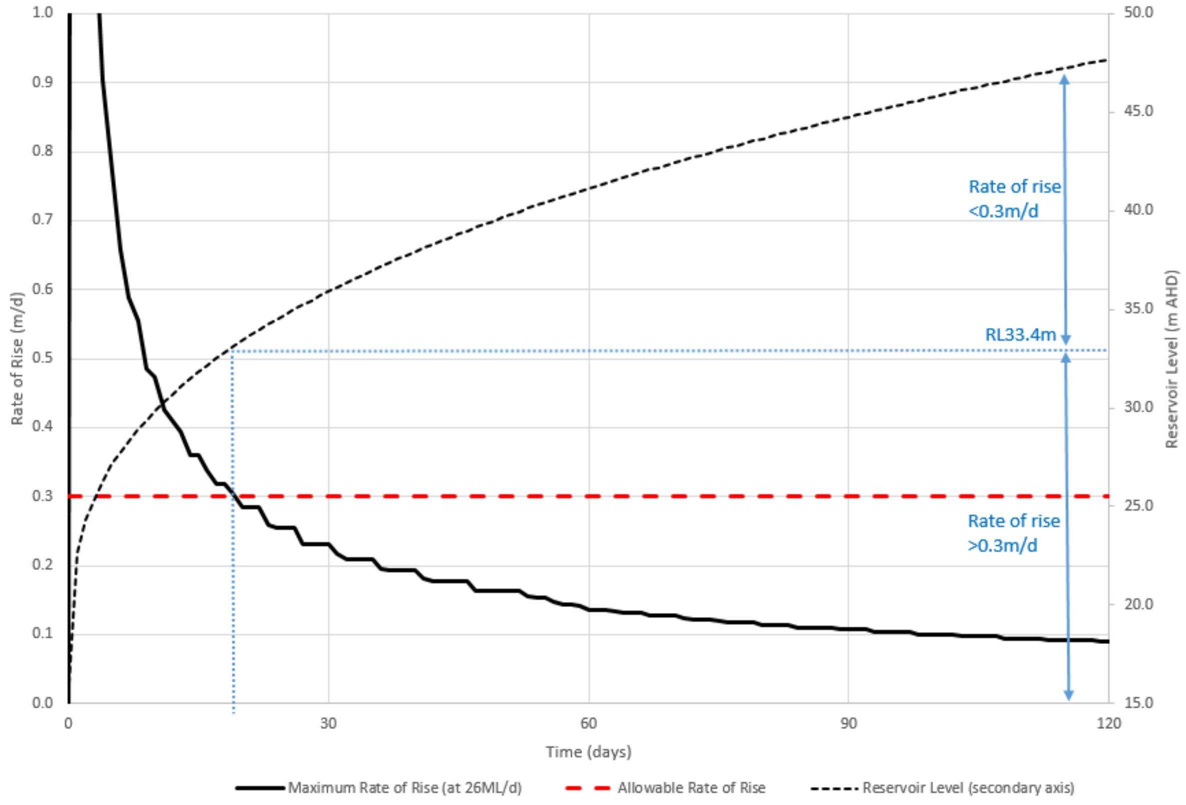


Figure 14.2: Maximum rate of rise of reservoir

14.4 Water Quality Monitoring

In addition to the ongoing water quality sampling during operation of the storage, as outlined in Section 13.2, a more intensive sampling regime is proposed during first filling. This testing will be undertaken at the river intake pump station, prior to transfer to the storage.

The increased testing regime during first filling will reduce the potential for low quality water within the storage by identifying any high concentrations of nutrients, microbes, chemicals, metals and sediments, particularly after rainfall events within the Tuross River (raw water) catchment, thus enabling preventative action to be undertaken.

In addition, the more intensive information will assist in establishing a baseline record of quality variations. This can be used to guide refinement of the long-term sampling program, and assist in any consideration of risks or matters associated with Health Based Targets.

The testing frequency is suggested to be weekly during first filling, with this frequency reviewed depending on the consistency of test results.

The water quality testing proposed to be undertaken as part of the EIS, refer to Section 2.5 of the Volume 1 report, will assist in assessing the water quality risk posed by the raw water source.

15 CONCEPT DESIGN COST ESTIMATE

A cost estimate has been developed for the Stage 1 Storage construction costs. Costs were also developed separately for the clearing and fencing component of the works. A summary of the estimated costs are presented in Table 15.1 and detailed in full in Appendix D.

The current combined (base) estimate allowing for the storage and clearing and fencing components of work is approximately $75M excluding GST (2016/17 terms). This estimate represents the base estimate and does not include allowance for inherent risks. Inherent risks are associated with the potential variability in the estimated unit rates and quantities.

The risk based estimate for the total project will be updated following completion of the concept design process and confirmation of the components of work to be taken forward to detailed design. The risk based estimate will include consideration of the inherent risks associated with each item.

Consistent with the previous risk based estimate undertaken, a contingency allowance of 15% has been allowed for. The contingency allows for contingent risks i.e. those risks that are dependent on an event occurring to be realised such as additional scope, industrial action etc. A 15% contingency is low for this stage of design, however a large variability in the contingency range will be adopted in the risk based estimate to reflect the uncertainty associated with this value.

Cost estimates associated with the Ancillary Works components have been provided in the Volume 1 concept report and have been based on the NSW Reference Rates Manual.

Unit rates for each item have typically been developed from bottom-up estimates by our cost-estimating sub-consultant Alan Rae Consulting.

The main embankment is the largest contributor to the total cost of the project, followed by the outlet works and diversion and watering items (items 6, 5 and 3 respectively). The supply and placement of materials in particular carry the highest costs, largely due to the requirement to either win the material on site (often with blasting and heavy excavation) or to import the material from commercial quarries.

The following assumptions have been made as part of developing the cost estimate for Stage 1:

- Costs for land matters including, land purchase for the storage area, compensatory habitat, easement/ land acquisition for access road have not been quantified at this point and are not included in the cost estimate.
- Core material, Zone 2B and Rip Rap to be sourced from the local Eurobodalla Quarry. Preliminary investigations generally confirm that the materials are suitable and that sufficient quantities of earthfill are available for Stage 1.
- Zone 2A, Zone 2C and bedding material supplied from local commercial quarry suppliers.
- For rockfill sourced from on-site quarries within the storage area, for use as the predominant embankment fill, it has been assumed that 50% of the material is rippable and 50% requires blasting. Similar assumptions have been made for the spillway, noting that the spillway excavation is a trenching excavation. A more detailed assessment of the excavatability of material will be made following completion of the geotechnical investigations.
- 2m depth of stripping has been assumed across the full extent of the embankment footprint.
- Placement of the rockfill shoulders are undertaken with minimal processing prior to placement.
- The spillway chute and dissipator walls have been assumed to be concrete lined consistent with the concept design, noting the potential for the freestanding walls to be required

should the geotechnical investigations indicate poor foundation conditions or elevated groundwater levels.

- Access bridge to tower to comprise 60m span with through steel I girders. The bridge span has been increased from 30m adopted in previous revisions to reduce the dominant cost contributor being the high concrete piers. With the increased spans length, the I girders are large (1.7m deep). To reduce potential maintenance costs associated with the coating the steel girders, ‘weathering steel’ can be considered during detailed design.
- The existing access tracks around the storage comprising portions of Bullockys Hut Rd, Waincourt Rd and Big Rock Rd has been assumed to be regraded to drain away from the catchment and reinstated with a crushed rock pavement. The access track has been adopted to be around the catchment boundary only and is estimated to be approximately 4.8 km long with the width assumed to be 4 m.
- Clearing has been allowed for below the Stage 1 full supply level; rock quarry areas upstream of the embankment; the Stage 2 embankment footprint; access road to storage; and a 5 m corridor along the catchment boundary fence.
- Given the storage is intended to be closed to the public and to manage water quality, security fencing has been assumed to be required around the catchment boundary. Previous estimates were based on stock-proof fencing.

Table 15.1: Summary of Storage Construction Costs

Item		Cost ($ Ex GST)
	Storage construction costs	
1	Access Roads	1.9M
2	Environmental Management	0.3M
3	Diversion & Dewatering	3.7M
4	Inlet Works	1.0M
5	Outlet Works	4.3M
6	Main Wall	28.4M
7	Spillway	2.4M
8	Water Quality	0.3M
9	Electrical	0.2M
	Misc Items (Investigations, Supervision, Contractors costs etc)	21.2M
	Contingency (15%)	9.6M
	TOTAL STORAGE CONSTRUCTION COSTS	**$73.3M**
	Clearing and Fencing	
1	Storage Clearing	0.45M
2	Fencing	0.44M
	Misc Items	0.38M
	Contingency (15%)	0.19M
	TOTAL CLEARING AND FENCING COSTS	**$1.47M**

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DOCUMENT / REPORT CONTROL FORM

File Location Name:	I:\Projects\30012127 - Eurobodalla Storage\07 Working\01 Storage Design\01 Concept Design\Concept Design Review
Project Name:	Eurobodalla Southern Storage
Project Number:	30012127
Revision Number:	4

Revision History

Revision #	Date	Prepared by	Reviewed by	Approved for Issue by
1	20/03/2017	C.Loh, C.Purss	R.Westmore	D.Evans
2	21/03/2017	C.Loh, C.Purss	R.Westmore	D.Evans
3	23/06/2017	C.Loh	C.Purss	D.Evans
4	25/07/2017	C.Purss	K.White	D.Evans