



Melbourne Water Corporation

Beaconsfield Reservoir Concept Design Report – FINAL

December 2019

Executive Summary

This report presents the concept design to upgrade Beaconsfield Reservoir. The purpose of the upgrade is to reduce the Consequence Category from High A to Low and although not formally assessed, it is expected that this upgrade would largely satisfy ALARP.

Beaconsfield Reservoir is now disconnected from the water supply network. The proposed concept design focuses on reducing the risk profile of the dam as well as reducing any future maintenance and operation requirements for Melbourne Water Corporation.

A previous risk assessment by URS in 2010 identified that Beaconsfield Reservoir lies within an order of magnitude of the ANCOLD (2003) Limit of Tolerability. A dam safety upgrade concept design, which assumed no reduction in reservoir level, was developed by GHD in 2012.

The dam safety upgrade was assessed against a partial decommission upgrade; full decommission upgrade and a Do Nothing approach, to determine the preferred way forward. Based on a multi-criteria analysis it was identified that a partial decommissioning option would successfully reduce the Consequence Category to Low whilst still maintaining a permanent water body, and therefore providing a long-term amenity for the public.

Three partial decommissioning concept options were originally considered (labelled 1A to 1C), with different crest and spillway arrangements. The designs were developed by adopting a FSL of RL 94.0 mAHD, which was required to achieve a Low sunny day Consequence Category. However, none of these concept options resulted in a Low Consequence Category for wet day failure. Therefore, an iterative approach was undertaken, in which a fourth concept option (1D) was identified. This option resulted in a Low Consequence Category under both sunny day and wet day failure scenarios. The concept design of Option 1D includes the following key components:

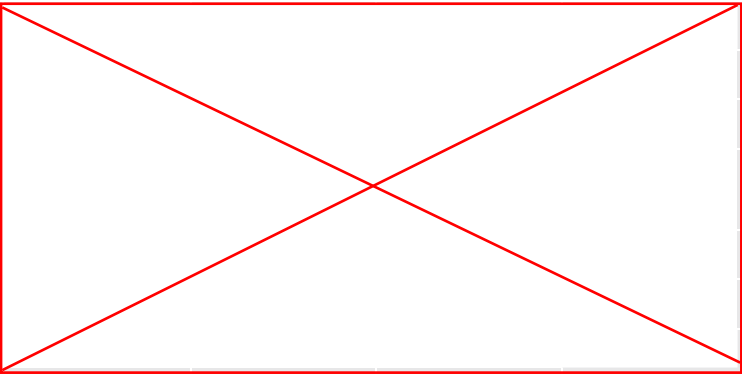
- Crest at RL 96.10 mAHD, which is 8 m below the current crest level of RL 104.05 mAHD
- A downstream embankment slope of 5H:1V
- FSL at RL 94.0 mAHD, 4.5 m lower than current restricted FSL of RL 98.5 mAHD
- Retrofitting the existing low-level outlet to be utilised as the primary spillway
- A secondary spillway at RL 95.5 mAHD located on the left abutment
- A rock-lined spillway chute and energy dissipator

In addition, the recommended concept design (Option 1D) also includes the landscape design of the site, namely:

- A re-designed smaller water body including smaller pools extending the visual appearance of the water body
- Circuit walking trails including tracks around the water body and along the existing spillway channel
- A picnic and passive recreation area located at the downstream toe of the upgraded embankment

A RANE analysis was completed for the four concept designs, with a summary of outputs shown in the table below.

Table E-1 RANE analysis output

	RANE Output (\$M)			
	Option 1A	Option 1B	Option 1C	Option 1D
Base Project Cost				
Low Expected Project Cost, P5				
Expected Project Cost, P50				
High Expected Project Cost, P95				
Contingency (P95 – P50)				
(P95-P50)/P50				
(P95 – Base Cost) / Base Cost				

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1. Introduction

1.1 Purpose of this report

GHD was engaged by Melbourne Water Corporation (MWC) on 24 July 2018 to undertake a Comprehensive Inspection and Concept Design for Beaconsfield Reservoir. This Report describes the Concept Design developed for the dam.

The key purpose of the concept design is to provide guidance to MWC on options to achieve a Consequence Category of Low and reduce risks associated with the storage, which were identified through a risk assessment undertaken by URS (now AECOM) in 2010. The main failure modes identified as key contributors to the existing risk are:

- Piping along the spillway interface (47%)
- Downstream instability (42%)

The results of the risk assessment indicated Beaconsfield Dam plots below the ANCOLD Limit of Tolerability (LoT) under the 50% and 80% confidence levels and above the LoT at the 95% confidence level. The risks at Beaconsfield Dam were assessed as not satisfying the As Low As Reasonably Practicable (ALARP) principle, predominantly on the basis that the "...dam has inadequate factor of safety for embankment stability." and the "...dam has no filters." (MWC, 2015e). Potential failure modes are further discussed in Sections 1.2.2 and 6.4.

This report, Beaconsfield Reservoir Concept Design Report, follows the Preliminary Concept Design undertaken by Jacobs (2018) and utilises current industry practice for updating the hydrology and dambreak assessment using ARR (2016/2019) and RCEM (2014) guidance. This report details the process undertaken to achieve the following criteria:

- Reduction of the Consequence Category from High to Low
- Minimise ongoing operation and maintenance requirements for MWC, and
- Maintain or improve amenity for key stakeholders

This report provides recommendations on the preferred option to progress to the detailed design phase, in line with the design criteria identified in this report. This report summarises the following:

- Background information on the storage
- Design criteria used in the development of concept upgrade options
- Key input information, including hydrology, dambreak and consequence assessments
- Preliminary option assessment, including the multi-criteria assessment for high level options
- Initial concept design development - based on sunny day failure, including landscape design
- Final concept design development - based on wet day failure, including landscape design
- Cost estimates and RANE estimates, and
- Conclusions and recommendations

Supporting information is provided in the Appendices to this report.

1.2 Background Information

1.2.1 Safety Review (GHD, 1999)

MWC engaged GHD to undertake a Safety Review of Beaconsfield Reservoir in 1999. During the review, GHD arrived at a number of key conclusions including:

- The left side of the embankment is unstable and requires remedial works
- The existing spillway capacity is adequate when the reservoir is operated 4 m below the spillway crest level, but is insufficient if the FSL is maintained at the spillway crest level
- The existing outlet works should be modified to operate as a primary spillway *
- The scour control valve should be repaired and operated at regular intervals to ensure its continuing serviceability *
- Erosion control works are required in the Haunted Gully Creek near the outlet of the scour and the return channel from the (proposed) primary spillway *
- Rainfall runoff from the catchment is needed to sustain the reservoir level, and the diversion drain should be breached with provision made to discharge into the reservoir *
- The indicative costs of the above remedial works were in the order of \$500,000

** Items have been actioned by MWC and DELWP/CEC*

1.2.2 Detailed Risk Assessment (URS, 2010)

Following the Safety Review in 1999, MWC engaged URS (now AECOM) to undertake a detailed risk assessment of Beaconsfield Reservoir. The report involved the identification, screening and quantification of the risks associated with Beaconsfield Reservoir. A Monte Carlo simulation was undertaken to assess confidence intervals to further understand the sensitivity around the estimates of some of the risks. The outcomes of the risk assessment were:

- The main contributors to risk were piping along the spillway interface (47%) and downstream instability (42%).
- The three highest ranked failure modes in terms of annual failure probability were piping along the spillway interface (5.5×10^{-4}), downstream instability (6.5×10^{-5}) and flood overtopping (4.1×10^{-5}).
- Slope stability analyses should be undertaken using undrained strengths.
- Simplified deformation analyses, such as the method described by Khalili, should be undertaken.
- Remedial works at the embankment spillway interface, as recommended in GHD's 1999 Safety Review report, should be undertaken.
- The spillway wall low point (RL 104.05 mAHD) should be raised.
- Lowering of the spillway crest level should be considered to ensure the water level restriction (i.e. 4 m below FSL) is achieved.
- Grouting up of the annulus of the original outlet works should be considered. Works should also include removal of the upstream gate and installation of an additional downstream gate valve.
- Hydrology and dambreak modelling should be updated to current best practice.

1.2.3 Remedial Works Design and Revised Hydrology and Dambreak Assessment (GHD, 2012)

Following the risk assessment, GHD was engaged in 2012 to undertake an upgrade options assessment. The preferred option, which was progressed to concept design included:

- Full height filter buttress with a stabilising berm
- Outlet works modified to act as a primary spillway

The scope of works was established from recommendations arising from the URS (2010) risk assessment and included the following criteria:

Scope item	Conclusion / Outcome
Stability assessment using undrained shear strengths (and additional geotech investigations)	FOS _{undrained} = 1.89 FOS _{drained} = 1.45
Simplified deformation analysis	8.8-11.0 m along circular failure surface equivalent to 2.5-5.4 m vertical displacement. The proposed freeboard (at the time) was 5.77 m.
Proposed remedial works for the preferred option	<ul style="list-style-type: none"> • Full height filters • DCF varies – RL 104.00-104.62 mAHD • Berm at RL 95 mAHD • Spillway interface works • Energy dissipator and rock-lined channel • Modification to valves, pipework, valve house and valve pit • Replacing outlet tower screens
Revised hydrology and hydraulics	FSL = RL 98.95 mAHD DCF = RL 104.02 mAHD <ul style="list-style-type: none"> • Spillway only – 1 in 280,000 AEP • Spillway and 1 outlet pipe – 1 in 500,000 AEP • Spillway and 2 outlet pipes – 1 in 700,000 AEP DCF = RL 104.62 mAHD <ul style="list-style-type: none"> • Spillway only – 1 in 200,000 AEP • Spillway and 1 outlet pipe – 1 in 1,600,000 AEP • Spillway and 2 outlet pipes – 1 in 2,000,000 AEP
Dambreak and consequence assessment	PAR _{day} = 1451 and PAR _{night} = 235 (incremental) PLL _{day} = 4 and PLL _{night} = 4 (incremental) Sunny Day Failure = High C Consequence Category Wet Day Failure = High C Consequence Category

1.2.4 Modelling for Removal of Beaconsfield Reservoir Part 1 (MWC, 2015a)

In 2015, modelling of the proposed decommissioning options was undertaken to better understand the full range of options (from decommissioning to full upgrade) available to MWC. The purpose of the study was to assess the impact to downstream properties due to full decommissioning of the dam. The conclusions of the study were:

- Flows increased (as expected) – at the Urban Growth Boundary (Brown Road) the peak flows increased in Haunted Gully Creek by 13 m³/s.

- Depth of flooding was estimated to increase on all properties currently subjected to flooding.
- The removal of the dam led to three (3) additional properties within the inundation zone for the 1 in 100 AEP flood.
- The average increase of flood depths was 0.42 m with two (2) properties increasing by over 1 m.
- If a more accurate flow path and/or depth was required, it was recommended that a 2D model of the floods be undertaken.

1.2.5 Modelling for Removal of Beaconsfield Reservoir Part 2 (MWC, 2015b)

Part 2 of the decommissioning modelling noted that "...complete removal of Beaconsfield Reservoir will lead to unacceptable increases in flood levels along Haunted Gully Creek...". In response, MWC modelled further scenarios, including operating the reservoir via the lower outlet pipe, to reduce the water level in Beaconsfield Reservoir while limiting any increases in water levels downstream, during a storm with an AEP of 1 in 100. The modelled scenarios included:

- Lower outlet pipe current set up
- Lower outlet pipe size increased to 1050 mm diameter pipe
- Lower outlet pipe size increased to 1200 mm diameter pipe
- Installation of a 1050 mm diameter pipe at the base of the reservoir
- Installation of a 1200 mm diameter pipe at the base of the reservoir

The results of the study found were:

- Peak discharges up to 2.8 m³/s were "acceptable" for changes in flood levels along Haunted Gully Creek.
- All modelled scenarios were deemed to be "acceptable" based on the 2.8 m³/s threshold, except for full decommissioning.
- The "recommended solution" was determined to be operating Beaconsfield Reservoir via the Lower Outlet given it would reduce flood levels at each parcel.

1.2.6 Beaconsfield Reservoir Consequence Assessment (GHD, 2016)

GHD was engaged to reassess the Consequence Category of the existing dam and investigate two possible future scenarios, which could reduce the Consequence Category. The scenarios included:

- Current conditions
- Reduce FSL to half of existing FSL
- Reduce FSL to one quarter of existing FSL.

The results of the study concluded that:

- Current conditions had a High C Consequence Category for Sunny Day Failure (SDF), and a Significant Consequence Category for Wet Day Failure (for the DCF), respectively.
- At half FSL, failures for the SDF and DCF both had a Consequence Category of Significant.
- At quarter FSL, failures for the SDF and DCF both had a Very Low Consequence Category.

1.2.7 Dam Consequence Assessment Review – Stage 2 (HARC, 2016)

MWC engaged Jacobs and HARC in 2015 to “address the perceived inconsistencies in PLL estimates across MWC’s portfolio of water supply dams and retarding basins (RBs)”. To rectify this, MWC commissioned a project (Stage 1 – Jacobs and HARC) to review the data, methods and assumptions used to estimate PLL, and to consider recently emerged methods for estimating PLL. Stage 2 (HARC, 2016) involved using the Reclamation Consequence Estimating Methodology (RCEM) to estimate the PLL from failure of five dams - Beaconsfield, Frankston, Tarago, Thomson and Yan Yean. The results for Beaconsfield Reservoir were:

- SDF: PLL of 3.2
- DCF: PLL of 0.4
- Medium Severity of Damage and Loss
- The DCF had a ‘Significant’ Consequence Category
- The SDF was assessed as ‘High A’.

1.2.8 Beaconsfield Dam Decommissioning Basis of Design Report (Jacobs, 2018)

Jacobs was engaged to develop a Basis of Design Report to further progress the state of the partial decommissioning:

- Preliminary concept design
- Discussion of the basis of design for each parameter and design references
- Stakeholder engagement
- Identification, assessment and proposed mitigation of data gaps and associated risks

At the commencement of the project, MWC indicated that they wished to decommission Beaconsfield Dam. In consultation with stakeholders as part of the project, it was found that maintaining a permanent water body was an important requirement. Therefore, the approach was changed to that of reducing the Consequence Category of the dam to a level where MWC would be able to safely handover the dam to the local council. It was determined that, under the Water Act 1989, this would require reducing the current Consequence Category of High C to Low or Very Low.

The Jacobs preliminary concept design involved the following design criteria:

- Lowering FSL to RL 92.0 mAHD by converting the existing scour to the primary spillway.
- Installing a hardened earth secondary spillway on the embankment, approximately 20 m wide down the centre of the crest at RL 97.0 mAHD.
- Lowering the dam crest level to RL 98.0 mAHD.
- Wetlands rehabilitation and stabilisation.

1.2.9 Beaconsfield Reservoir – Additional Hydrology, Dambreak and Consequence Assessment (GHD, 2018b)

GHD was engaged to develop Sunny Day Failure (SDF) and Wet Day Failure (WDF) consequence assessments of each of the three (3) options (A, B and C) proposed in the Jacobs concept design. GHD (2018) followed the general method undertaken in GHD (2016). The key difference was GHD (2018) used the revised stage-storage relationship determined from updated bathymetric survey undertaken by Taylors in 2017, which reduced the storage volume at all levels.

Using the updated stage-storage relationship, GHD arrived at the following:

- Option A, FSL RL 92.0 mAHD – Low Consequence Category
- Option B, FSL RL 93.0 mAHD – Low Consequence Category
- Option C, FSL RL 94.0 mAHD – Significant Consequence Category.

2. Background

2.1 Description of storage

Beaconsfield Reservoir is located on Haunted Gully Creek, approximately 45 km southeast of Melbourne in the suburb of Officer. The reservoir is an on-stream storage, with a local catchment area of approximately 334 ha. It was constructed by the State Rivers and Water Supply Commission in 1918 as part of a new water supply scheme for the Mornington Peninsula. Water was harvested from the Bunyip River and conveyed to Beaconsfield Reservoir by the Bunyip Main Race (BMR), which was later supplemented by the construction of the Tarago Main Race (TMR). The reservoir was permanently disconnected from Melbourne's water supply and distribution network in 1988 and now serves as an ornamental lake.

The site consists of a 24 m high earthfill embankment, a spillway on the left abutment, a low-level outlet passing through the foundation beneath the embankment, and a high-level outlet. During an upgrade in 1970, the original high-level outlet was decommissioned and grouted up, and a new high-level outlet was installed.

At the original Full Supply Level (FSL) of RL 103.08 mAHD, Beaconsfield Reservoir has a total storage capacity of 912 ML and a surface area of 14.6 ha. The reservoir is currently operated at a restricted level of 4.23 m below FSL at RL 98.85 mAHD, and has a capacity of approximately 410 ML.

Key information is summarised in Table 2-1 below with a comprehensive table shown in Appendix A.

Beaconsfield Reservoir is operated by Melbourne Water, but is located on Crown Land managed by the Department of Environment, Land, Water and Planning (DELWP). The Cardinia Environment Coalition (CEC) manage the surrounding Beaconsfield Nature Conservation Reserve under an agreement with the Minister for Water.

Table 2-1 Key information

Component	Description
Name	Beaconsfield Reservoir
Watercourse	Haunted Gully Creek is directly downstream of the reservoir
Location	Access from O'Neil Rd, Officer
Current Purpose	Ornamental lake
Upgrades	1970, 1988, 2014 (See Appendix A for details)
Population at Risk (PAR)	Sunny Day Failure: 334-408 Incremental Wet Day Failure: 1372-1676
Potential Loss of Life (PLL)	Sunny Day Failure: 2.2-2.8 Incremental Wet Day Failure: 8.9-11.6
Severity of Damage and Loss	Sunny Day Failure: Medium Incremental Wet Day Failure: Medium
Sunny Day Failure Consequence Category (ANCOLD 2012)	High C (based on PLL)
Incremental Wet Day Failure Consequence Category (ANCOLD 2012)	High A (based on PLL)

Component	Description
Type	On-stream storage
Full Supply Level (FSL)	RL 103.08 mAHD
Reduced Maximum Operating Level (MOL)	RL 98.85 mAHD
Minimum Operating Level	RL 90.86 mAHD
Total Storage Capacity at FSL	912 ML
Storage Capacity at restricted MOL	320 ML (revised by Taylors 2017)
Catchment area	334 ha
Reservoir surface area at FSL	14.6 ha
Type	Earthfill with (puddle) clay core and partial concrete cut-off
Crest level	RL 104.62 mAHD (nominal crest level) Sags by up to 0.6 m (to RL 104.02 mAHD)
Crest length	174 m
Crest width	1.8 m
Normal freeboard	1.54 m (FSL to nominal crest level) 5.77 m / 5.17 m (restricted MOL to nominal crest level / restricted MOL to lowest crest level)
Embankment Height	24.0 m
Upstream slope	2H:1V (above FSL) 3H:1V (below FSL)
Downstream slope	2H:1V Berm at RL 93.0 mAHD (approx.)
Type	Ogee crest with concrete channel at left abutment (Note: High Level Outlet now acts as the primary spillway, and the left abutment spillway as a secondary spillway)
Invert level	RL 103.08 mAHD
Length	17.8 m
Capacity	See Appendix A for further details.
Dam Crest Flood AEP	See Appendix A for further details.
<u>High Level outlet works (current Primary Spillway)</u>	Twin 1050 mm dia. nominal (42") MSCL pipes. See Appendix A for further details.
<u>Low Level outlet works & tunnel</u>	Circular tower with intake at RL 90.87 mAHD and 450 mm or 500 mm (18") nominal diameter pipeline (varies depending on source) through embankment within a tear drop culvert. Upstream and downstream valve control. See Appendix A for further details.
<u>Abandoned structures</u>	See Appendix A for further details.
<u>Valves</u>	See Appendix A for further details.
Outlet works capacity	See Appendix A for further details.
Inlet works	See Appendix A for further details.

Component	Description
Instrumentation	<ul style="list-style-type: none"> • 2 gauge boards • Electronic reservoir and rainfall monitoring • 9 standpipe piezometers • 11 movement markers

2.2 Identified risks

In 2010, URS (now AECOM) undertook a risk assessment of Beaconsfield Reservoir. The risk assessment identified a number of credible failure modes, which are presented below in Table 2-2, together with the annual probability of failure for each of the failure modes.

Table 2-2 Annual probabilities of failure (URS, 2010)

Failure mode	Condition	Annual Pr(f)
Downstream instability	Sunny Day	6.45E-05
Piping along spillway interface	Flood	5.50E-05
Piping through embankment	Sunny Day	8.18E-06
Flood overtopping	Flood	3.86E-06
Overtopping of spillway channel	Flood	2.18E-06
Downstream instability	Flood	8.10E-07
Piping through embankment	Flood	2.75E-07
Piping along outlet works	Flood	2.48E-09

The Beaconsfield Reservoir site presents a series of concerns as identified in the risk assessment, which have been actively managed by Melbourne Water. These include:

- Historical seepage: Most recently observed in August 2018 on the downstream right abutment groin. A reduced operating level of RL 98.85 mAHD has continued to assist limiting risk associated with this deficiency.
- Structural instability: Beaconsfield Dam has a factor of safety (FoS) of 1.36, which is below the minimum required FoS of 1.5 for long-term steady state loading.
- All other identified deficiencies are related to minor capital works, or operation and maintenance of Beaconsfield Dam.

2.3 Unit conversion

There are three different level datums referred to in the available drawings and documents relating to Beaconsfield Reservoir. The crest level of the spillway (the FSL) has been used as a reference point for unit conversions on the level datum. Equation 2-1 has been used for imperial to metric unit conversion.

Equation 2-1 Conversion

$$RL \text{ mAHD} = RL \text{ (imperial)} \times 0.3048 - 0.552$$

2.4 Reference drawings

A list of reference drawings for Beaconsfield Reservoir is provided in Appendix B.

3. Design criteria

3.1 General

The design criteria for this project have been developed to guide the concept design. It is noted that the design criteria for this project are focused on partial decommissioning of the dam. Additional design criteria will need to be developed if it is decided to completely decommission the dam, or retain the current FSL and fully upgrade the dam.

The key inputs used in the development of the design options include:

- Original design drawings – A list of reference drawings is provided in Appendix B
- Storage-elevation curve – The storage-elevation curve used in the analysis is provided in Appendix G
- Hydrology – The inflow hydrology used in the development of concept design is discussed in Appendix C
- Community and stakeholder input

3.2 Key design criteria

The key design criteria for the partially decommissioned storage include:

- The Consequence Category is reduced to Low. Based on the preliminary dambreak and consequence assessment, the following is noted:
 - Dambreak modelling described in Section 4 showed that an FSL of RL 94.0 mAHD was required to achieve a Low sunny day failure Consequence Category
- The key hydraulic criteria:
 - The partially decommissioned dam should not exceed the existing peak outflow for up to the 1 in 100 AEP flood event. The existing peak outflow is approximately 3.4 m³/s for the 1 in 100 AEP flood.
 - The ANCOLD fallback criteria for flood capacity for a 'Low' Consequence Category dam is between 1 in 100 to 1 in 1000 AEP (ANCOLD Draft, 2016). For the purposes of this concept design, it has been assumed that the upgraded dam will be required to safely pass the 1 in 1000 AEP.
- Any upgrade works should be accordance with current industry practice.

Other design criteria, specific to each option, are discussed as required in the relevant sections of this report.

4. Full Supply Levels for Sunny Day Low Consequence Category

Sunny Day Failure breach modelling was undertaken with three different FSL scenarios, RL 95.0 mAHD, RL 94.0 mAHD and RL 93.0 mAHD, to confirm the design FSL that would result in a 'Low' Consequence Category. For all three scenarios, an embankment crest level of RL 97.0 mAHD was assumed for Sunny Day Failure modelling.

The details of the breach parameter estimation and modelling are provided in Appendix D.

The breach parameters adopted and resulting peak breach flows are summarised in Table 4-1.

Table 4-1 Adopted breach parameters

Scenario	Basis	Breach base width (m)	Breach development time (min)	Peak flow (m ³ /s)
Sunny Day FSL = RL 93.0 mAHD (upper bound sensitivity)	Bureau of Reclamation	16	12	196
Sunny Day FSL = RL 93.0 mAHD (most likely)	Froehlich (2008)	3.2	8	120
Sunny Day FSL = RL 93.0 mAHD (lower bound sensitivity)	Min. Singh and Scarlatos breach base width with adjusted MacDonald Langridge Monopolis (after Wahl) breach time	5.4	22	84
Sunny Day FSL = RL 94.0 mAHD (upper bound sensitivity)	Bureau of Reclamation	19.2	14	273
Sunny Day FSL = RL 94.0 mAHD (most likely)	Froehlich (2008)	4.5	10	184
Sunny Day FSL = RL 94.0 mAHD (lower bound sensitivity)	Min. Singh and Scarlatos breach base width with adjusted MacDonald Langridge Monopolis (after Wahl) breach time	5.4	22	114
Sunny Day FSL = RL 95.0 mAHD (upper bound sensitivity)	Bureau of Reclamation	22	16	368
Sunny Day FSL = RL 95.0 mAHD (most likely)	Froehlich (2008)	5.9	12	256
Sunny Day FSL = RL 95.0 AHD (lower bound sensitivity)	Min. Singh and Scarlatos breach base width with adjusted MacDonald Langridge Monopolis (after Wahl) breach time	5.4	22	147

4.1 Downstream hydraulic modelling

A two dimensional model (TUFLOW) was used to model the floodplain flows below Beaconsfield Reservoir. The model extent is shown in Figure 4-1.

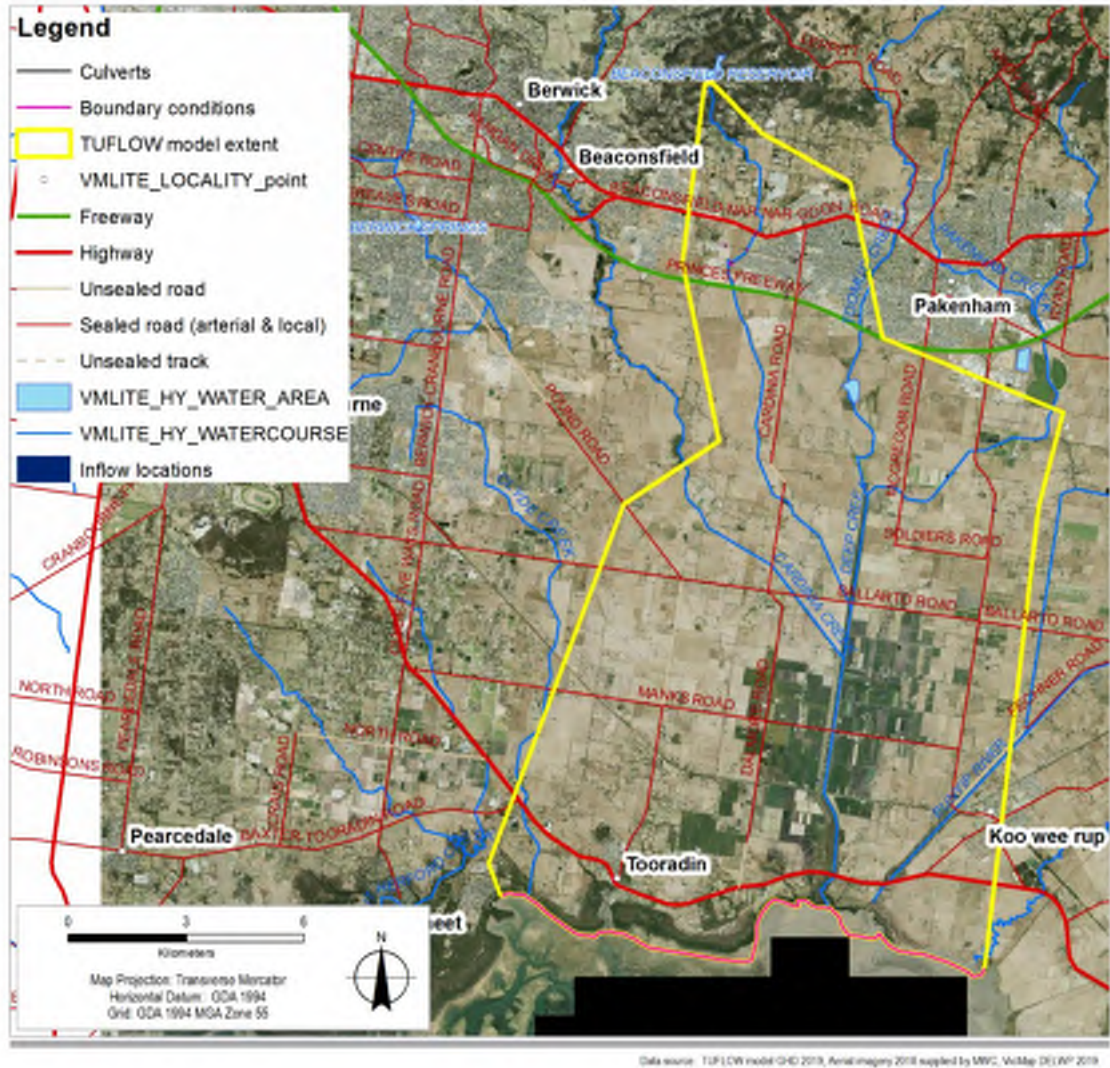


Figure 4-1 TUFLOW model extent

The reservoir outflow hydrographs generated using FLDWAV were used as input flows to the TUFLOW model.

The flood level and hazard (velocity depth product) results from the TUFLOW model were subsequently used to determine the Population at Risk (PAR) and the Potential Loss of Life (PLL).

Further details on the TUFLOW model may be found in the “Beaconsfield Comprehensive Inspection” and “Design-Hydrology and Consequence Category update” reports (GHD, 2019).

4.2 Population at Risk (PAR)

In accordance with the “Guidelines on the Consequence Categories for Dams” (ANCOLD, 2012), the Consequence Category is determined by assessing either the PAR or PLL, together with the Severity of Damage and Loss arising from downstream inundation caused by a dambreak.

According to ANCOLD (2012), the definition of PAR includes all persons who would be directly exposed to flood waters within a dambreak-affected zone at the onset of the dambreak if they took no action to evacuate. The dambreak-affected zone is defined as the zone of flooding where the changes in depth and velocity of flooding due to dambreak are such that there is potential for undesirable consequences. Although not specified in ANCOLD (2012), industry practice is to generally limit the count to those areas where the dambreak causes a rise in level of floodwaters greater than 300 mm.

The PAR downstream of Beaconsfield Reservoir may originate from a number of sources:

- Scout camp (Camping ground, nature walk, canoe area)
- Boon Roses
- Dwellings
- Princes Highway

GIS procedures were used to calculate the number of buildings and length of major roads inundated. Two distinct time periods were assumed for estimating the PAR:

- Day time (10 hours)
- Night time (14 hours)

4.3 PLL summary for Sunny Day Failure

The Sunny Day Failure PLL from each main source is summarised in Table 4-2.

Table 4-2 Summary of Sunny Day PLL estimates

Scenario	PLL buildings floor level – night UK RARS	PLL buildings floor level – night USBR suggested median	Scout Park – USBR overall upper	Boon Roses – USBR suggested upper (day only)	Princes Highway	Total day	Total night (using USBR suggested median for dwellings)
FSL=93 mAHD (2-3 buildings affected above floor)	0.007 – 0.02	0.03-0.04	0.01 (day)	0.04	0	0.06 - 0.07	0.03-0.04
FSL=94 mAHD (3-4 buildings affected above floor)	0.02-0.05	0.04-0.06	0.011-0.02(day) 0-0.02 (night)	0.04	0	0.07 - 0.09	0.04-0.08
FSL=95 mAHD (3-4 buildings affected above floor)	0.03-0.11	0.04-0.06	0.03-0.06 (day) 0.02- (night)	0.04	0.01 (day and night)	0.10 - 0.15	0.05-0.09

The ANCOLD (2012) Guidelines for Consequence Categories states that a PLL of <0.1 is required to achieve a Low consequence. The estimated PLL during a Sunny Day Failure for the FSL of either RL 93.0 mAHD or RL 94.0 mAHD achieves a Low Consequence Category in accordance with ANCOLD (2012). A FSL of RL 95.0 mAHD shows an estimated PLL of >0.1. The Severity of Damage and Loss is estimated to be Medium, the basis for which is included in Appendix E . Due to the community desire to keep the water level in the reservoir as high as possible, the RL 94.0 mAHD FSL option has been progressed.

5. Options assessment

5.1 General

As part of the assignment, a wide range of upgrade options were considered. The reason for this initial phase was to demonstrate that partial decommissioning was the most appropriate for dam safety, stakeholders and MWC.

The options considered for Beaconsfield Reservoir included three partially decommissioned options (refer Options 1A to 1C), and one fully decommissioned option (refer Option 2). These were compared with a full dam safety upgrade as per GHD's 2012 concept design (refer Option 3).

Table 5-1 below summarises the key changes to Beaconsfield Reservoir based on the upgrade options initially considered.

Table 5-1 Summary of concept options as initially considered

Component	Option	Partial decommissioning			Full decom'ing	Full dam safety upgrade
		1A	1B	1C	2	3
N/A	Retain Current FSL (Crest raising, increase spillway, full-height filters)					✓
Low Level Outlet Modifications	Modify Low Level Outlet to Primary Spillway	✓	✓			
	Decommission Low level Outlet			✓		
High Level Outlet Modifications	Decommissioning High Level Outlet	✓	✓	✓		
Embankment Modifications	Lowered embankment crest – Partial section overtops (secondary spillway) – Concrete with rock-lined chute	✓				
	Lowered embankment crest – Full length overtops (secondary spillway)		✓			
	Lowered embankment crest – Partial section overtops (secondary spillway) – Culverts			✓		
	Complete removal of embankment and structures				✓	

Preliminary details of the scope of work of these options, and preliminary hydraulics, are provided in the following sub-sections.

5.2 Option 1 – Partial decommissioning

Partial Decommissioning involves a reduction in the Consequence Category to Low or Very Low without an increase in the peak outflow, up to the 1 in 100 AEP, when compared with the existing arrangement. A partial decommissioning upgrade offers the benefit of retaining the ornamental lake for community benefit while minimising risk and cost to Melbourne Water. Reducing Beaconsfield Reservoir from a High A to Low Consequence Category reduces the ANCOLD (2003a) recommended frequencies of inspections. Comprehensive Dam Safety Inspections are reduced from 5-yearly to 'not required'. Intermediate Dam Safety Inspections are reduced from annual to 5-yearly and routine visual inspections are reduced from daily-tri-weekly to monthly. Three partial decommissioning options were initially investigated.

The three partial decommissioning options assessed include:

- Decommissioning the High Level Outlet including demolishing the outlet tower and valve pit, grouting the pipework with valves to be 'locked out', whilst the Valve House would be retained for storage.
- A FSL at RL 94.0 mAHD.
- A primary spillway as either a new or retrofitted pipe and inlet structure to pass the 1 in 100 AEP event without changing the current peak outflow.
- A secondary spillway to pass the 1 in 1000 AEP event.
- The Low Level Outlet tower superstructure including bridge and hoist house removed and the substructure cut flush with the embankment.
- Concrete grouting of the annulus between the Low Level Outlet cast iron pipe and concrete tunnel.
- Erosion protection required at toe, and to be considered for the embankment based on estimated velocities during detailed design.
- 5H:1V downstream slope tied into the natural surface.
- Landscaping of the site and wetlands to maximise the quality of community space.

5.3 Option 2 – Full decommissioning

Full Decommissioning eliminates all dam safety risks associated with Beaconsfield Reservoir by removing the water retaining structure, and has no ongoing dam maintenance costs.

However, there would be no permanent water body, a large construction period, impacts to the flora and fauna within a Nature Conservation Reserve and risks associated with the removal of potentially hazardous silt.

Full decommissioning includes:

- Removal of the embankment.
- Removal of all appurtenant works including the current outlet works (including Valve House), original outlet works (including those previously abandoned through grouting), Low Level Outlet and spillway.
- Removal and disposal of deposited silt.
- Stream bed and bank rehabilitation.
- Return stream to pre-dam flows.

5.4 Option 3 – Full Dam Safety Upgrade

A Full Dam Safety Upgrade will address the risks identified by URS (2010) and although not formally assessed, it is expected that this upgrade would largely satisfy ALARP principles. For the purpose of this report, the concept design (GHD, 2012) was considered appropriate.

The Full Dam Safety Upgrade would retain the restricted FSL (or higher depending on Melbourne Water's appetite for risk) thereby retaining maximum functionality of the reservoir for community use.

The upgrade is considered to undergo a longer and more costly construction phase than Options 1 and 2, causing disruptions to the community's accessibility to the reservoir. Ongoing dam safety surveillance and maintenance would be required due to an either High C or High B Consequence Category, and therefore it is considered prudent to have the site closed to the public due to public safety issues such as the high embankment and exposed rock faces.

The Full Dam Safety Upgrade (GHD, 2012) includes:

- Full-height filter buttress placed on the downstream batter with weighting fill placed over the top. The filters are designed to reduce the risk of piping which was a key contributing risk (URS, 2010).
- Restricted FSL becomes the permanent FSL at RL 98.85 mAHD.
- Convert the High Level Outlet to the primary spillway including the removal of all valves, replacement of the intake screens from a fine screen to a coarse screen. The pipe would be altered to combine flows (as opposed to running parallel the entire length) and plugging the unused section of the pipe downstream. A USBR Impact Basin energy dissipator to retard flows into Haunted Gully Creek would be constructed.
- Concrete grouting of the annulus between the cast iron pipe and concrete tunnel in the Low Level Outlet. This will reduce the risk of piping along and within the outlet and is considered a prudent measure.
- Re-profile the embankment crest where low points exist. Removing any low points will reduce the risk of overtopping.
- Minor capital works as noted in previous Annual Inspections, including works to the access roads, Valve House and operations and maintenance improvements such as pit lids, railing and platforms.

5.5 Option 4 – 'Do Nothing'

A 'Do Nothing' option is a control option and used to provide a base case for the options. By doing nothing, the Consequence Category and risk profile remain unchanged. Beaconsfield Reservoir is considered to not currently meet ALARP, plotting within an order of magnitude of the ANCOLD Limit of Tolerability at the 50% and 80% confidence intervals, and plotting above the ANCOLD Limit of Tolerability for the 95% confidence interval. Therefore, 'Do Nothing' is not in accordance with ANCOLD guidelines, good practice and precedent or the Strategic Framework for Dam Safety Regulation (DELWP, 2014).

5.6 Multi-criteria analysis on options

Following discussions with stakeholders in September 2018, a multi-criteria analysis was requested by a select group of community representatives to explain the more technical aspects of the project be undertaken on the four options provided above. The purpose of a multi-criteria analysis is to undertake a complimentary approach, in order to identify the best method to achieving a series of goals set out by stakeholders directly and indirectly involved with this reservoir.

The analysis was undertaken initially by determining a set of categories and weightings. This was then presented to MWC where updates were made as a group. Finally, the analysis was presented to the public where additional opinions and suggestions were received and updated where appropriate.

The analysis considered cost and dam safety as equally weighted categories and contributing 60% to the total score. Community impacts and environment and conservation impacts were equally weighted and made up the remaining score.

Each of the four categories contained a number of sub-categories each with weighting as well.

The MCA results from Table 5-2 illustrate that a Partial Decommissioning option is the most appropriate strategy for Melbourne Water. Partial Decommissioning addresses community interest in the reservoir while minimising construction cost and lowering ongoing maintenance costs. Commentary on the Category, Sub-category and Options weightings are provided in Appendix F.

Table 5-2 Multi-criteria analysis

MCA categories & sub-categories		Category weighting	Sub-category weighting (out of 4)	OPTIONS			
				1	2	3	4
				Partial decom'ing / partial height dam	Full decom'ing / removal of dam	Safety upgrade (full upgrade)	Do nothing / current arrangement
1	Cost	30		21.3	15.6	17.3	23.1
1.1	Construction cost		4	3	1	2	4
1.2	Ongoing maintenance cost		4	3	4	2	1
1.3	Cost of public amenity operations and maintenance		1	3	3	4	4
1.4	Approvals, public engagement costs		2	2	1	3	4
1.5	Design, engineering costs		2	3	1	2	4
2	Satisfying ALARP	30		25.7	30.0	21.4	7.5
2.1	F-N Position / Life safety risk		4	3	4	2	1

MCA categories & sub-categories		Category weighting	Sub-category weighting (out of 4)	OPTIONS			
				1	2	3	4
				Partial decom'ing / partial height dam	Full decom'ing / removal of dam	Safety upgrade (full upgrade)	Do nothing / current arrangement
3.6	Fire		3	3	1	4	4
3.7	Flood mitigation		3	3	1	4	4
4	Environmental and conservation impacts	20		15.0	5.0	10.0	17.5
4.1	Construction and rehabilitation period		3	2	1	2	4
4.2	Long-term impacts on flora & fauna communities		3	4	1	2	3
	TOTAL SCORE	100		78.3	61.1	65.2	64.6

MCA categories & sub-categories		Category weighting	Sub-category weighting (out of 4)	OPTIONS			
				1	2	3	4
				Partial decom'ing / partial height dam	Full decom'ing / removal of dam	Safety upgrade (full upgrade)	Do nothing / current arrangement
2.2	Compliance with good practice		3	4	4	4	1
3	Community impacts	20		16.3	10.5	16.5	16.5
3.1	Provision of public amenities and safe access		3	4	4	3	2
3.2	Visual appearance of landscape		4	4	4	3	2
3.3	Visual appearance of lake/retained water		3	2	1	4	4
3.4	Retention/incorporation of heritage & 'past infrastructure' elements		1	4	2	3	4
3.5	Impact on community by construction activity, vehicle movements, etc		3	3	1	2	4

6. Partial decommissioning options assessment

6.1 General

Following the MCA, three partial decommissioning options were further developed. The key design criteria for the partial decommissioning options were:

- The dam must have a Low Consequence Category as per the ANCOLD Guidelines on the Consequences Categories for Dams (2012).
- No increase in flows up to the 1 in 100 AEP outflow. The current peak outflow at Beaconsfield Reservoir is 3.4 m³/s and should not increase as part of the upgrade, so as to not increase downstream flooding during frequent flood events.
- Safely pass the 1 in 1000 AEP flood event. The ANCOLD Guidelines on Selection of Acceptable Flood Capacity for Dams (Draft, 2016) recommends that dams with a Consequence Category of Low or Very Low should safely pass floods between 1 in 100 AEP (for Low and Very Low Consequence Category) and 1 in 1 000 AEP (Low Consequence Category).

Three options were initially considered in terms of partial decommissioning, namely:

- Option 1A - Allowing discharges greater than the 1 in 100 AEP to pass through a wide channel through part of the embankment section on the abutment.
- Option 1B - Allowing discharges greater the 1 in 100 AEP to overtop the full length of the embankment.
- Option 1C - Allowing discharges greater than the 1 in 100 AEP to pass through box culverts, which are narrower and taller than the channel in Option 1A.

Appendix I contains drawings showing key details of concept options 1A to 1C.

6.2 Concept Design Options 1A to 1C

Table 6-1 Detailed of Concept Options 1A to 1C

Component	Option 1A	Option 1B	Option 1C
Description	Modify low level outlet to primary spillway and broad crested weir secondary spillway	Modify low level outlet to primary spillway and overtoppable embankment as secondary spillway	Decommission low level outlet and install new pipe as primary spillway and concrete culverts as secondary spillway
FSL	RL 94.0 mAHD	RL 94.0 mAHD	RL 94.0 mAHD
Embankment	<ul style="list-style-type: none"> • Dam crest at RL 96.80 mAHD • 37 m (approx.) crest width – to improve stability and reduce piping risk • 5H:1V downstream batter grade – to improve downstream slope stability and reduce piping risk • Fill from crest lowering to be placed on downstream embankment • Topsoil and plants per landscape design – to improve public amenity 	<ul style="list-style-type: none"> • Overtoppable dam crest at RL 96.30 mAHD • 110 m (approx.) overtoppable embankment length • Hardened surface of either concrete or hardened earthfill to control dam crest level • 39 m (approx.) crest width – to improve stability and reduce piping risk • Reno mattress approx. 250 mm thick where overtoppable with topsoil and plants per landscape design – to provide erosion protection • 5H:1V downstream batter grade – to improve downstream slope stability and reduce piping risk • Fill from crest lowering to be placed on downstream embankment • Swale drain at toe to prevent pooling of water 	<ul style="list-style-type: none"> • Dam crest at RL 97.00 mAHD • 36 m (approx.) crest width – to improve stability and reduce piping risk • 5H:1V downstream batter grade and tied into natural surface – to improve downstream slope stability and reduce piping risk • Fill from crest lowering to be placed on downstream embankment • Topsoil and plants per landscape design – to improve public amenity

Component	Option 1A	Option 1B	Option 1C
Secondary Spillway	<ul style="list-style-type: none"> • Spillway crest at RL 96.0 m – to not increase the peak flows for the 1 in 100 AEP event • 40 m (approx.) spillway width through crest • An earthen approach channel leading to the concrete structure – to improve the efficiency of the spillway, minimising the required size to safely pass the 1 in 1000 AEP event • Spillway founded on good quality rock – to minimise scour • Cut-offs at the upstream and downstream ends of the structure – to minimise piping along the spillway interface • Downstream chute; rock-lined channel and able to be covered by topsoil and grass – to maximise public amenity 	See <i>Embankment</i> for details	<ul style="list-style-type: none"> • Twin concrete culverts at RL 95.8 mAHD – to not increase the peak flows for the 1 in 100 AEP event • Twin concrete box culverts approximately 1.7 m in width with internal dimensions of 1.5 m wide and 1.2 m high • 40 m (approx.) spillway width through crest • An earthen approach channel leading to the concrete structure – to improve the efficiency of the spillway, minimising the required size to safely pass the 1 in 1000 AEP event • Spillway founded on good quality rock – to minimise scour • Cut-offs at the upstream and downstream ends of the structure – to minimise potential for piping along the spillway interface • Downstream chute -rock-lined channel
Primary Spillway	See <i>Low Level Outlet</i> for details		<ul style="list-style-type: none"> • New DN 600 pipe installed at RL 94.0 mAHD • Concrete encased under the secondary spillway to prevent piping • Discharge into the secondary spillway downstream chute

Component	Option 1A	Option 1B	Option 1C
Low Level Outlet	<ul style="list-style-type: none"> • Install concrete riser to raise low level outlet to RL 94.0 mAHD at upstream toe of dam with coarse trashrack on top – to maximise water level for community while maintaining Low or Very Low Consequence Category • Low level outlet Tower Bridge flush with the embankment crest. Removal of hoist house and top section of the tower. Bulkhead facility to be retained in case isolation is required – for public safety • Downstream valve pit to be decommissioned by removing valves and installing a manhole cover for public safety while retaining accessibility • Concrete backfill the annulus between the cast iron pipe and the concrete tunnel to reduce the risk of bursting of the cast iron pipe – to reduce the risk of piping • Consideration should be given to the installation of a filter collar around the downstream end of the modified Low Level Outlet – to reduce the risk of piping • Consideration should be given to sleeving the section of pipe between the upstream intake and the tower with a DN 450 pipe and grouting the annulus given the potential risk of piping (currently unknown condition of pipe and joints) – to reduce the risk of piping 		<p>Decommissioning of the Low Level Outlet. Works include:</p> <ul style="list-style-type: none"> • Pipework – Concrete backfilling or grouting of the complete section of pipe from the concrete core wall to the upstream intake shaft • Tower – The existing Low Level Outlet tower bridge to be removed and the top of the tower flush with the revised embankment crest level. The remaining 'stub' section of the tower would be backfilled with concrete • Consideration should be given to concrete backfilling the annulus between the cast iron pipe and the concrete tunnel to reduce the risk of bursting of the cast iron pipe – to reduce the risk of piping • Consideration should be given to the installation of a filter collar around the downstream end of the modified Low Level Outlet – to reduce the risk of piping
High Level Outlet	<ul style="list-style-type: none"> • Grouting – The full length of the twin pipes would be grouted • Consideration of a downstream filter collar • Consideration of public safety and the demolition of the outlet tower – it is noted that demolition of the tower is not a dam safety requirement • Downstream valve house – The structure would be retained with valves locked out and ladder removed <p>It is noted that the extent of decommissioning on many of the High Level Outlet features is likely to be a risk-based decision in terms of both dam and public safety aspects.</p>		

6.3 Check of dambreak results for Option 1A

6.3.1 Determination of the AEP of the DCF

The preferred concept design Option 1A was configured in a RORB model. A range of very rare to extreme floods were routed through the reservoir. The AEP of the DCF estimated to be 1 in 1000 AEP, with the 12 hour GSAM duration being critical. The flood frequency (assuming glass walling) is shown in Figure 6-1 below.

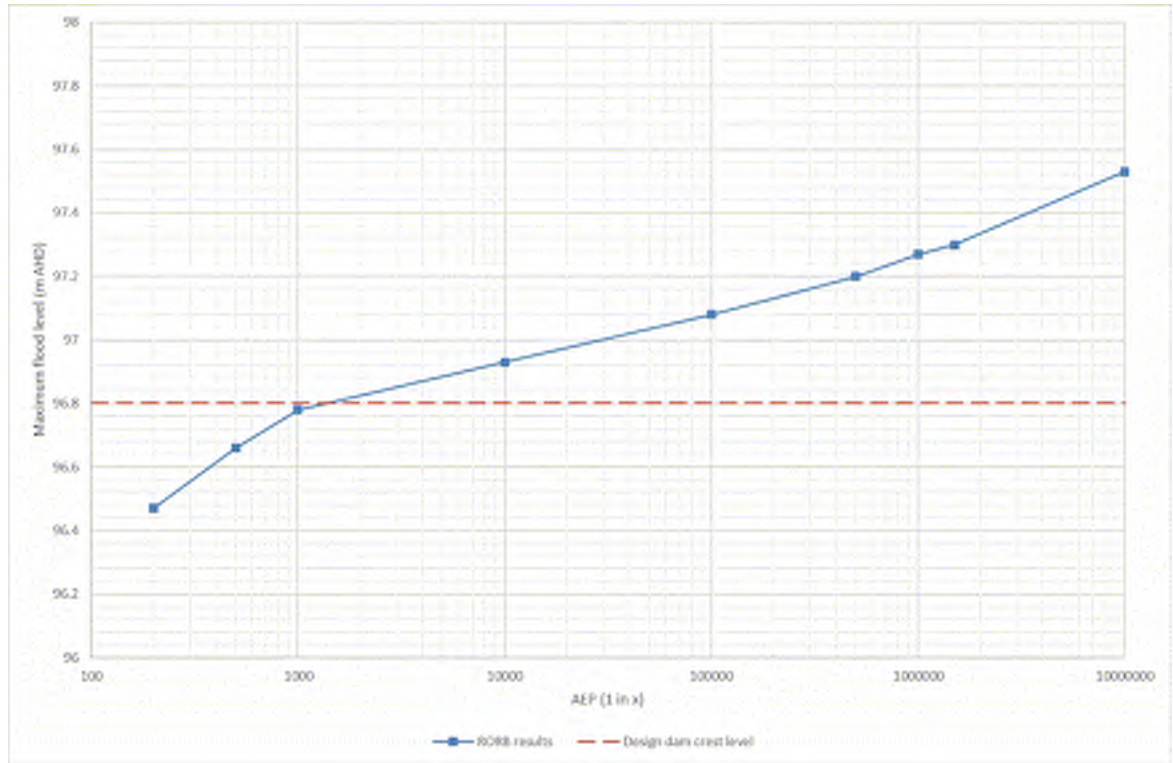


Figure 6-1 Concept Option 1A flood frequency (glass-walled)

6.3.2 Breach parameters

The range of breach parameters estimated from empirical equations is given in Table 6-2.

Table 6-2 Breach parameters from empirical equations for Option 1A DCF

Empirical equation	Side slope	Option 1A DCF breach base width (m)	Option 1A DCF breach development time (min)
MacDonald Langridge Monopolis (Wahl) ¹	0.2	1.2	17
Bureau of Reclamation	0.2	27.4	19
Froehlich (2008) ²	0.7	8.2	16
Von Thun Gillette (1990)	1	20.8	47
Singh and Scarlatos minimum breach base width	0.2	5.4	22 (applying Wahl earthfill equation to volume of embankment eroded)

These breaches were simulated in FLDWAV, with the resulting breach hydrographs shown in Figure 6-2 following.

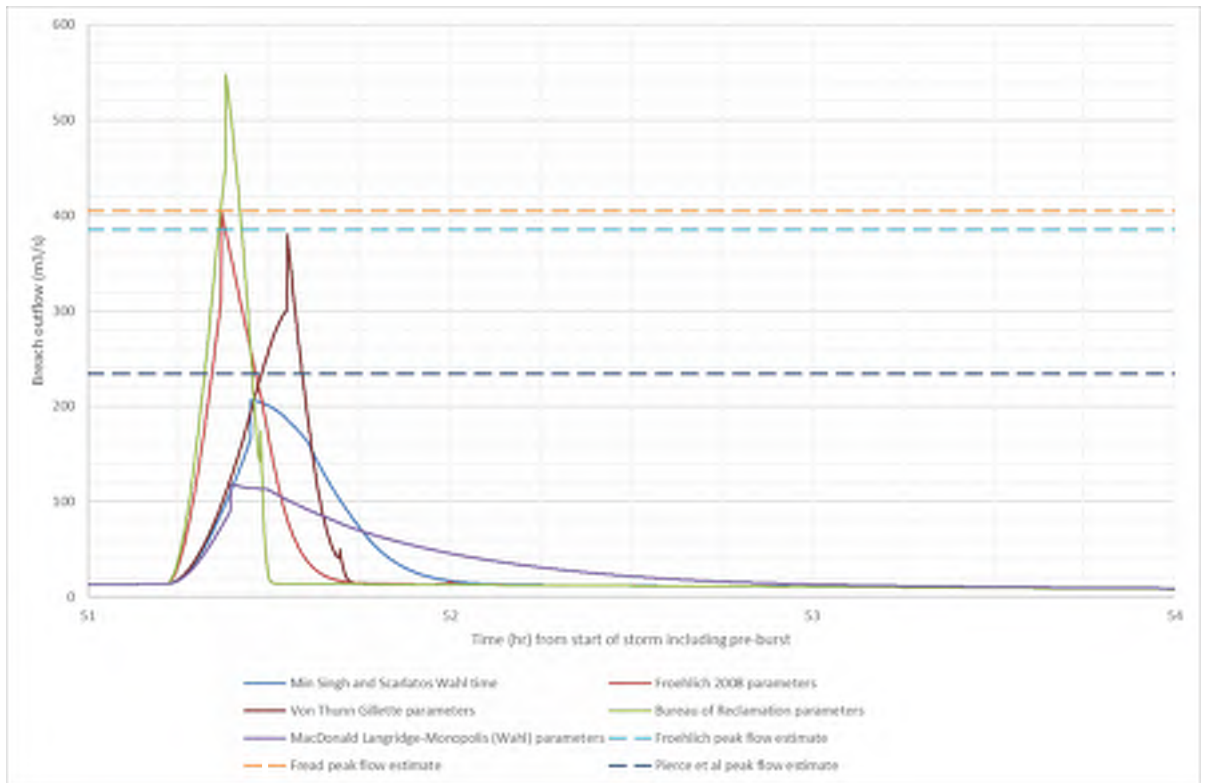


Figure 6-2 Concept design Option 1A – DCF breach hydrographs

Following inspection of the breach hydrographs the breach parameters shown in Table 6-3 were selected, with the respective hydrographs simulated in the TUFLOW model to inform the Consequence Category Assessment.

Table 6-3 Adopted breach parameters for Option 1A

Scenario	Basis	Breach base width (m)	Breach development time (min)	Peak flow (m ³ /s)
No breach		NA	NA	14
Upper bound sensitivity	Bureau of Reclamation	27.4	19	547
Most likely	Froehlich (2008)	8.2	16	403
Lower bound sensitivity	Min Singh and Scarlatos breach base width with adjusted MacDonald Langridge Monopolis (after Wahl) breach time	5.3	23	207

6.3.3 PAR and PLL estimation for Option 1A

The area of 300 mm incremental depth extends to the Princes Freeway, as shown in Figure 6-3 below. The flood maps for a dambreak following upgrade in accordance with Option 1A can be found in Appendix D.

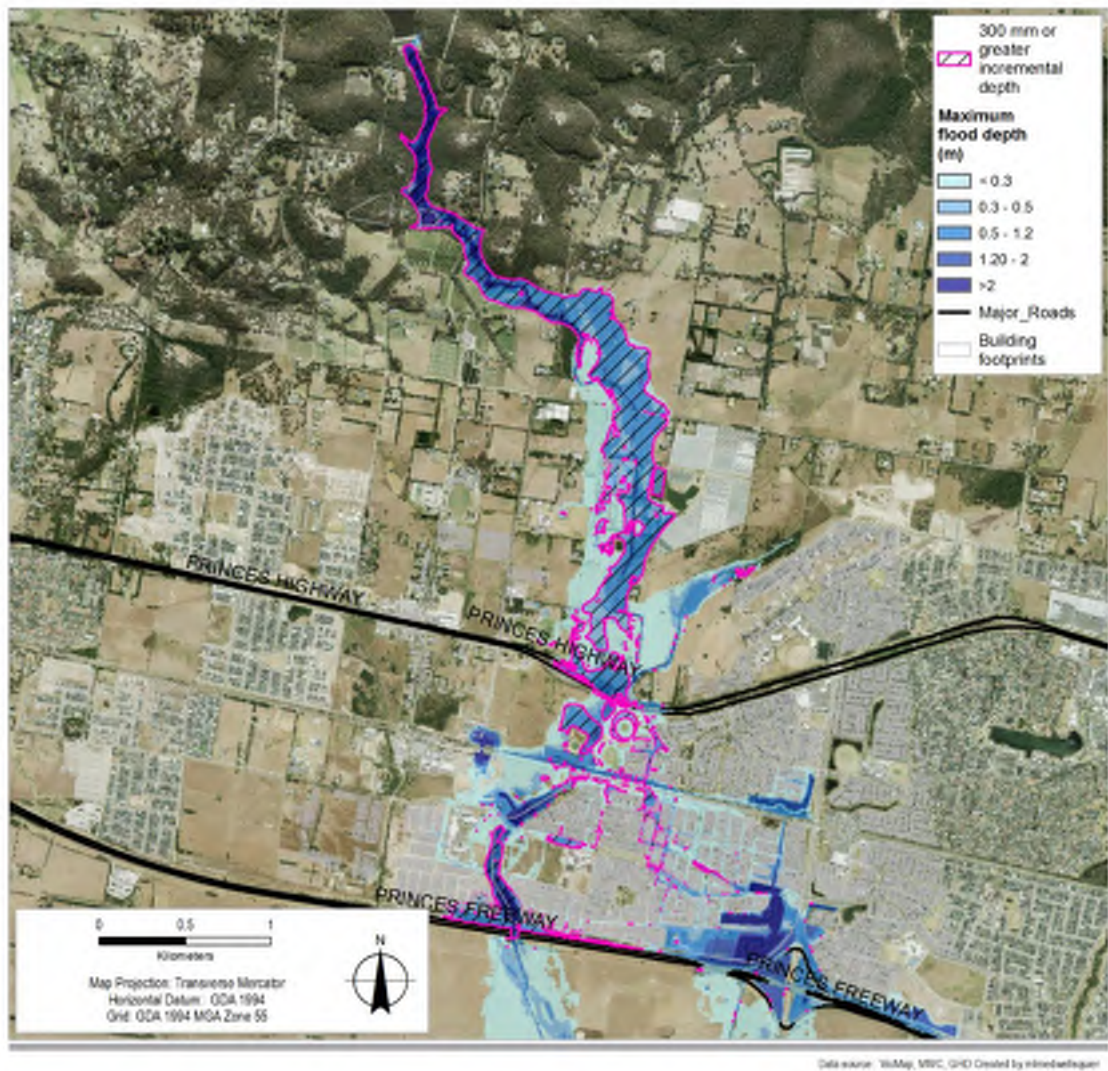


Figure 6-3 Area of greater than 300 mm incremental depth for Concept Design Option 1A (DCF breach)

6.3.4 No breach fatality rates for Option 1A

The fatality rates previously discussed for dambreaks in relation to the Sunny Day Failure are not directly applicable to the estimation of loss of life in non-dam failure flood situations. Hill et al. (2007) where the Graham fatality factors were compared to large historical floods undertook an investigation. In the 2007 investigation, it was recommended that for a low flood severity a fatality rate of 0.0002 be adopted. For a medium flood severity with a warning time greater than 60 minutes, a fatality rate of 0.005 (vague) to 0.002 (precise) was recommended.

6.3.5 Dwelling summary for Option 1A

The PAR and PLL from dwellings relating to a DCF breach, assuming upgrade in accordance with Option 1A, are shown in Table 6-4. The Scout Camp is assumed not to be occupied during extreme weather.

Table 6-4 Option 1A concept PAR and PLL for DCF failure

Parameter	Day	Night	Day/Night	Day	Night	Day/Night
Scenario	No Breach			DCF piping breach		
Number affected buildings - Floor Level	3			103-128		
PAR Buildings (USBR, 2014) - Floor Level	4	8	7	144-179	288-358	228-284
PLL Buildings (USBR, 2014) - Floor Level	0.000	0.001	0.001	0.728-0.905	1.456-1.810	1.153-1.433
PLL Buildings (UK RARS, 2013) - Floor Level	0.000	0.001	0.001	0.262-0.411	0.524-0.822	0.414-0.650

As they are greater than 0.1, these PLL estimates do not satisfy the requirements of a “Low” or “Very Low” Consequence Category. Therefore, the design was iterated as discussed in Section 6.4 in order to arrive at an upgrade solution that results in a “Low” Consequence Category for both sunny day and wet day failure scenarios.

6.4 Additional Concept Design (Option 1D)

6.4.1 Development of Option 1D

The initial concept options (Options 1A to 1C) were developed on the assumption that the sunny day failure was the critical scenario with a wet day scenario to only be considered as a ‘final check’. However, based on the revised hydrology and consequence assessment (refer to Section 6.3), it was apparent that the wet day failure scenario is more critical. The concept design was further developed (Option 1D) to achieve a Low Consequence Category under both sunny day failure and wet day failure scenarios.

The recommended concept design (Option 1D) maintains modifying the Low Level Outlet (LLO) to act as the primary spillway and installs a secondary spillway through the embankment. However, the embankment crest and spillway invert levels were both lowered. This was to reduce the volume of water stored under DCF loading and ultimately reduce the PLL to below 0.1 for DCF dambreak. This arrangement achieved a Low Consequence Category under both wet and sunny day scenarios.

Further details based on the recommended concept design (Option 1D) are discussed in Sections 6.4.2 to 6.4.5, with a comparison between Option 1A and 1D key design features presented below in Table 6-5.

Table 6-5 Comparison of key design features - Options 1A and 1D

Key Component	Option 1A	Option 1D
Primary Spillway	Existing LLO converted to primary spillway at RL 94.0 mAHD	
Dam Crest Level	RL 96.8 mAHD	RL 96.1 mAHD
Secondary Spillway type	Constructed on good quality natural rock through crest with concrete sill discharging into downstream rock beaching-lined channel	
Secondary Spillway level	RL 96.00 mAHD	RL 95.50 mAHD
Secondary Spillway length	10 m	
Existing HLO	Decommissioned	

6.4.2 Embankment details for Option 1D

The proposed concept design (Option 1D) involves lowering the embankment crest level from RL 104.03 mAHD (existing) to RL 96.1 mAHD. The excavated material would be used to construct the downstream batter at a slope of 5H:1V. This would assist in addressing key failure modes identified in the risk assessment by URS in 2010:

- Improving downstream stability through re-profiling the embankment.
- Reducing piping risk via an increased flow path and reducing the maximum hydraulic gradient.

The reduced dam crest level reduces the dam crest flood (DCF) given a smaller storage volume, reducing the consequences of failure. The embankment crest will be graded towards the downstream side at 3%, for approximately 37 m. The new downstream batter will have a 5H:1V slope and will be tied into the natural surface at the downstream toe with the top 150 mm (confirmed in future designs) stripped to provide good connection between the existing embankment and new weighting fill.

A number of erosion protection products have been discussed for the embankment downstream batter during the concept design including rock, reno mattress, GeoWeb and grass. For the purposes of this concept design and associated cost estimates, the crest and downstream batter of the embankment were assumed to be grassed (potentially planted with native grasses) to provide some degree erosion protection during overtopping of the embankment. The requirements for erosion protection of the downstream batter should be confirmed in the detailed design, when velocities are further assessed. The toe of the downstream batter will have a spoon drain to provide drainage, with specific reno mattress or rock erosion protection.

A typical cross section of the lowered embankment can be seen in Figure 6-4.

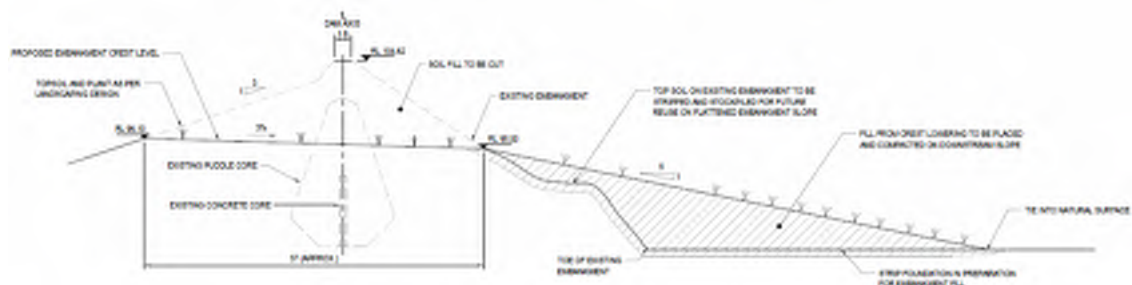


Figure 6-4 typical cross section of lowered embankment – Option 1D

6.4.3 Secondary spillway design for Option 1D

A new secondary spillway would be constructed at RL 95.50 m and be 10 m long. It is preferable to found the structure on rock wherever possible, to minimise differential settlement and foundation related issues. This means that the structure would be located on one of the abutments as opposed to the centre of the embankment section, as seen in Figure 6-5 following. The spillway would extend the width of the crest (approximately 40 m) directing flows into a rock-lined channel to discharge flows into the downstream creek. The spillway chute would need to be modelled as part of the detailed design. The key components of the new secondary spillway include:

- Approach channel – an earthen approach channel leading to the concrete sill.
- Excavation into rock – the structure would be founded on quality rock with local demolition of the core wall to accommodate the concrete sill structure being founded on rock (refer to Figure 6-6).

- It is also likely that cut-offs will be required at the upstream and downstream ends of the structure to minimise seepage and erosion beneath the spillway floor.
- Downstream chute – the chute would extend from the downstream shoulder of the crest to approximately the toe of the dam as seen in Figure 6-7 following. At the toe, the chute would direct flows into a rock-lined stilling basin, discharging into the downstream creek.

The secondary spillway design is set at RL 95.50 mAHD, which together with the primary spillway (refer to Section 6.4.4) does not increase peak flows downstream during frequent floods, up to and including the 1 in 100 AEP event. The corresponding peak flow was found to be approximately 5.5 m³/s. The relatively wide spillway also provides greater discharge capacity, thereby increasing the discharge capacity at all levels above the spillway crest, enabling the 1 in 1000 AEP event to be safely passed.

The decommissioning of the previous broad crested spillway reduces all probability associated with piping along the spillway interface which represented 47% of total risk at Beaconsfield Reservoir. The setting of the new secondary spillway in good quality rock will help to mitigate any potential for piping risks along the spillway interface.

A key requirement of the scope was to maintain and improve public amenity of the space. Landscaping the rock-lined channel, stilling basin and decommissioned broad crested spillway, as discussed in Section 8, will improve public space and contribute towards stakeholder satisfaction.

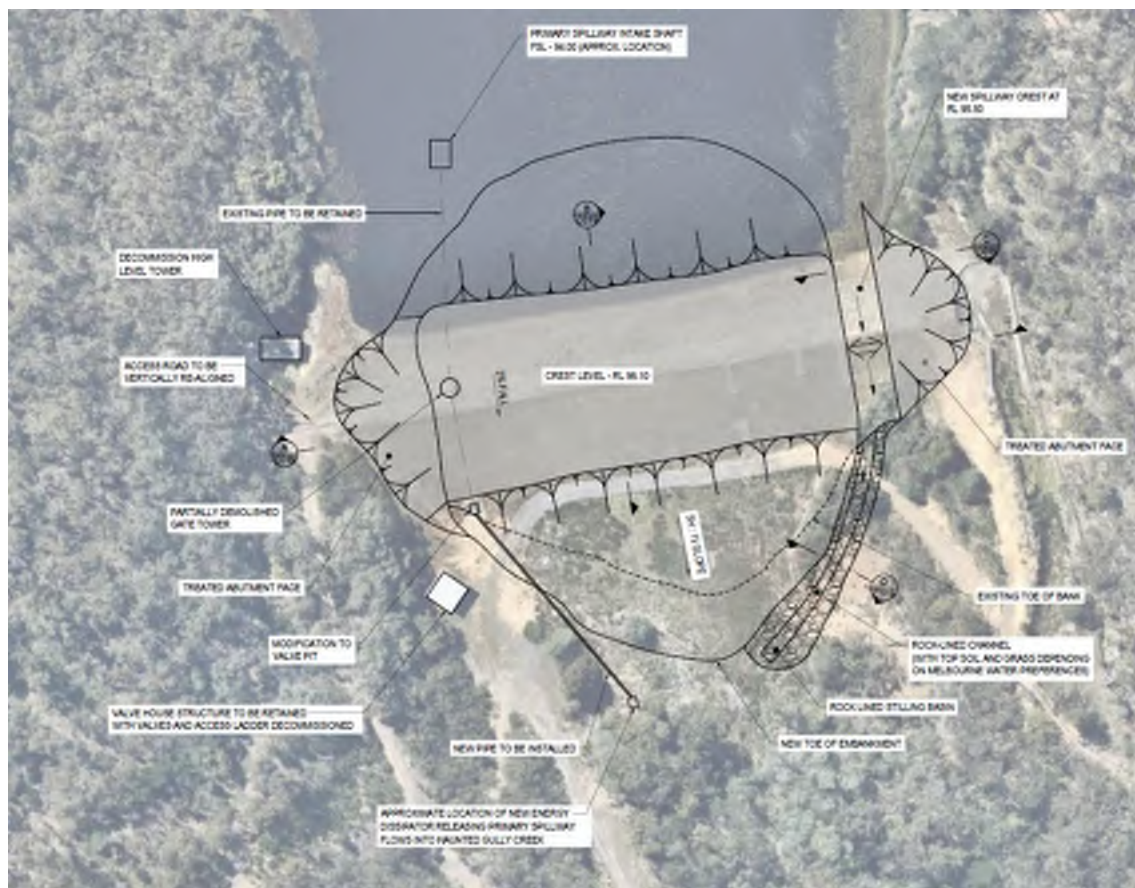


Figure 6-5 Arrangement of spillway and downstream chute – Option 1D

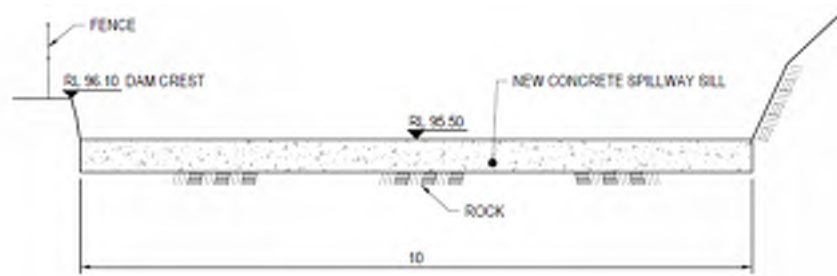


Figure 6-6 Cross section of concrete spillway crest – Option 1D

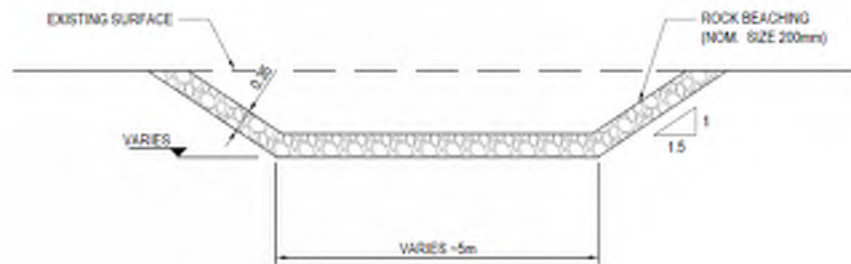


Figure 6-7 Typical section of rock-lined channel – Option 1D

6.4.4 Low Level Outlet converted to primary spillway for Option 1D

Under the proposed concept design, the current Low Level Outlet would be converted to the primary spillway with an invert level of RL 94.0 m. Included in this package of work are:

- Intake shaft – The concrete intake shaft at the upstream toe of the embankment would be raised to the revised Full Supply Level (RL 94.0 m) by breaking back the existing concrete to expose the existing reinforcement, splicing new reinforcement and forming a new vertical shaft. A coarse trashrack (birdcage) would be constructed over the intake shaft. Refer to Figure 6-8 and Figure 6-9 for details.



Figure 6-8 'Birdcage' trashrack (1)



Figure 6-9 'Birdcage' trashrack (2)

- Tower – The existing low-level outlet tower bridge would be removed, and the hoist house and top section of the tower would be demolished down to the revised embankment crest level. Minor works would be required at the top of the partially demolished tower to prevent public access to the tower. It is proposed that the bulkhead facility be retained in case isolation of the outlet downstream of the tower is required (for maintenance and inspection purposes). Refer to Figure 6-10 following for details.
- Cast iron pipework – Concrete backfilling the annulus between the cast iron pipe and the concrete tunnel should be undertaken to reduce the risk of bursting of the cast iron pipe
- Valve pit and outlet – The downstream valve pit would be partially removed to eliminate the confined space. The downstream valve would be removed and replaced with a manhole cover in case access is required for CCTV inspection of the pipe. Minor modifications would be made to the downstream pipework to enable uncontrolled discharges to the downstream creek including installing new pipework downstream of the valve pit and excavating (if required) the abandoned pipework. Refer to Figure 6-11 following for details.
- New energy dissipator – At the downstream end of the primary spillway pipework, the pipe would discharge into an Impact Energy Dissipating Basin. This basin has been successfully implemented on other projects with similar flow velocities. However, depending on MWC's preferences, there is flexibility in design. Downstream of the impact basin is rock beaching, which allows discharges to safely enter into Haunted Gully Creek. Refer to Figure 6-11 and Figure 6-12 following for details.
- Filter collar – Consideration should be given to installing a filter collar around the downstream end of the modified Low Level Outlet (i.e. Figure 6-11)
- Upstream concrete pipework – The section of waterway pipe between the upstream intake and the tower is currently a concrete section (refer Figure 6-10). The condition of this pipe is not known. If the structural condition of the pipe or the joints has deteriorated, there is a risk of collapse or piping failure through the joints. While it is considered unlikely that piping issues associated with this section of pipe could lead to complete breach of the dam due to the presence of the core wall, the design should consider sleeving this section of the pipe with a 450 mm pipe and grouting the annulus.

Retaining the Low Level Outlet and converting it to the primary spillway provides the most cost effective solution for not increasing peak flows up to the 1 in 100 AEP event (in combination with the secondary spillway for events less frequent than approx. 1 in 200 to 1 in 500 AEP events).

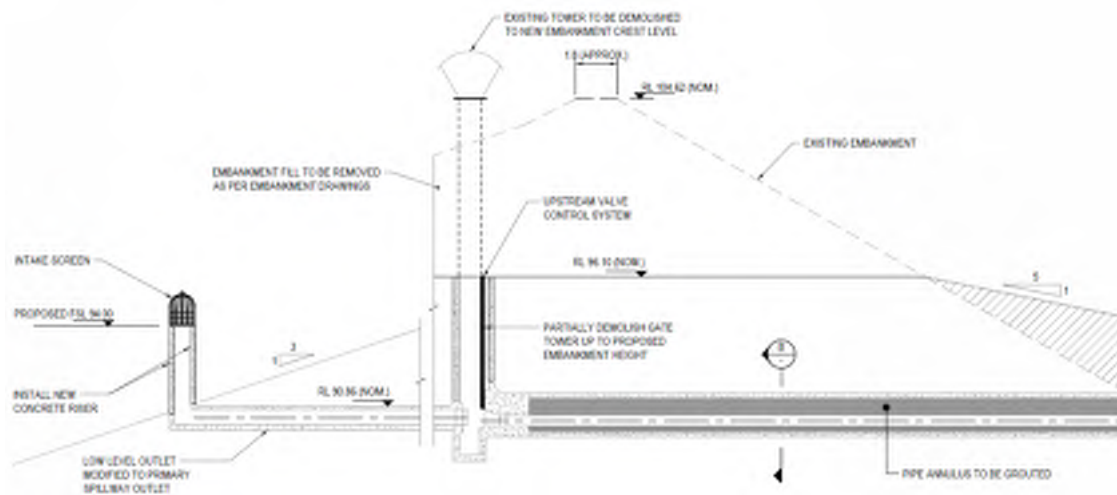


Figure 6-10 Typical section for Low Level Outlet and Tower modifications – Option 1D

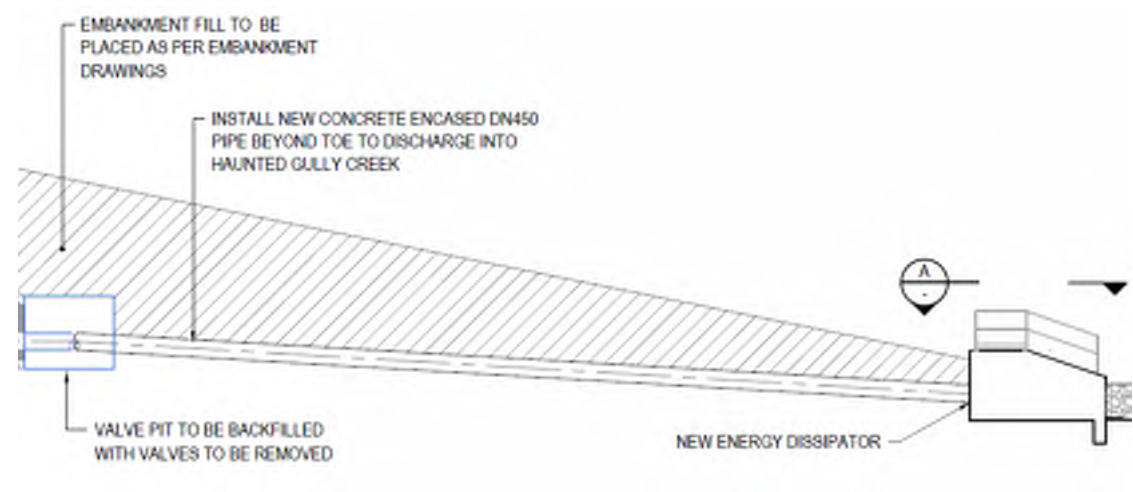


Figure 6-11 Section through Low Level Outlet – Option 1D

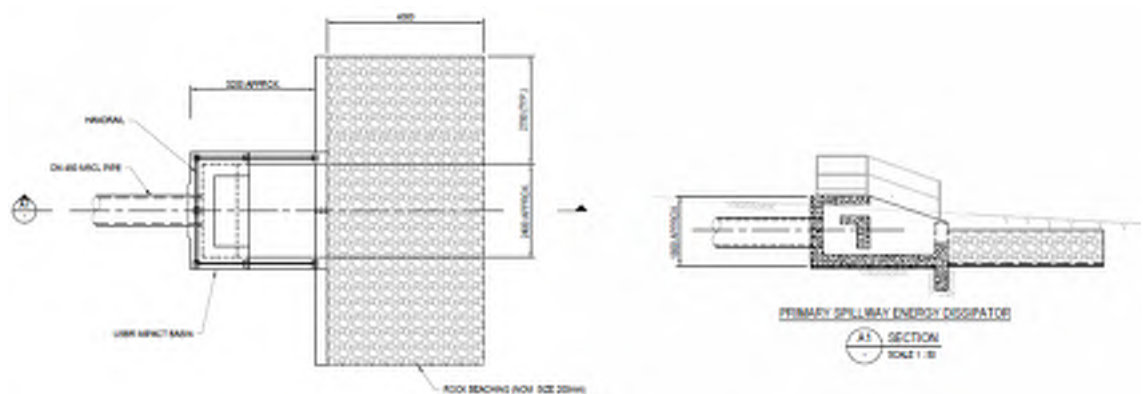


Figure 6-12 Plan (left) and section (right) of Energy Dissipator

6.4.5 Decommissioning of High Level Outlet – Option 1D

The invert of the High Level Outlet is currently set at approximately RL 97.9 mAHD, which is higher than the proposed reduced water level, although potentially lower than the expected design flood level. As such, concept design Option 1D requires that the High Level Outlet be fully decommissioned. The following works would be required:

- Grouting – the full length of the twin pipes would be grouted (or backfilled with concrete) from the upstream outlet tower to the downstream valve house.
- Filter collar – consideration should be given to installing a filter collar around the downstream end of the decommissioned outlet pipes.
- Demolition of outlet tower – consideration should be given to long-term public safety around the high-level outlet tower, including removal or treatment of the four asbestos cement columns (refer to Rec 2015/03 from 2015 Annual Dam Safety Inspection). Where considered to present a safety risk, partial demolition of this structure could be undertaken. It is noted that demolition of the tower is not a dam safety requirement. For details refer to Figure 6-13.
- Downstream valve house – The structure is proposed to be retained for use by Cardinia Environmental Coalition (CEC). Valves contained within the valve house will be locked out and the access ladder removed. A solid lid is proposed to be placed over the floor to hide the locked out valves beneath to prevent any untoward actions. It is noted that the proposed minor works to the valve house are subject to change following discussions with CEC.

It is noted that the extent of decommissioning on many of the High Level Outlet features is likely to be a risk-based decision, in terms of both dam safety and public safety. The design focuses on maintaining infrastructure for continued use for stakeholders, while safely passing peak flows up to and including the 1 in 100 AEP event, solely by the two spillways proposed for option 1D. Rarer floods pass over the overtoppable embankment to safely pass events rarer than the standards based 1 in 1000 AEP requirement. This minimises ongoing costs associated with inspection of the pipe and addresses piping risks along the interface of the conduits with the embankment.

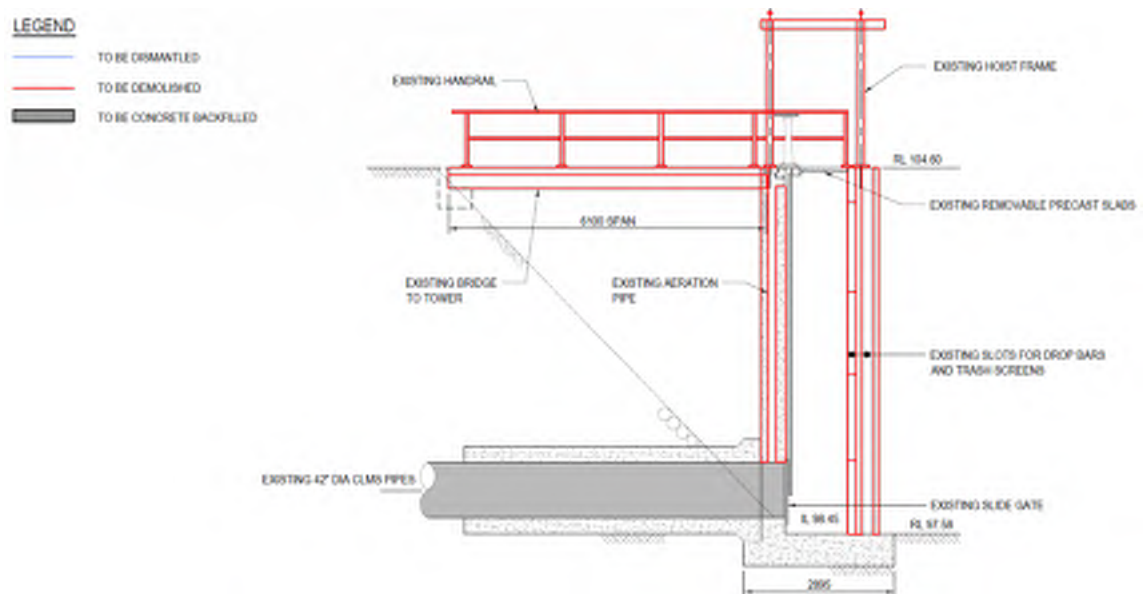


Figure 6-13 Typical section through High Level Outlet Tower – Option 1D

LEGEND

- TO BE DISMANTLED
- TO BE DEMOLISHED
- TO BE CONCRETE BACKFILLED

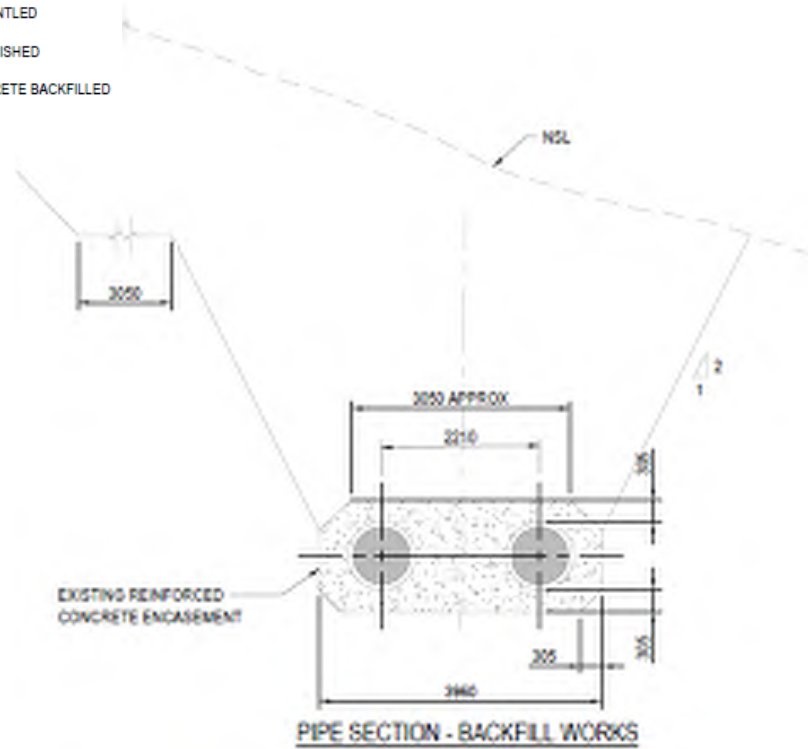


Figure 6-14 Typical cross section of High Level Outlet Pipes – Option 1D

LEGEND

- TO BE DISMANTLED
- TO BE DEMOLISHED
- TO BE CONCRETE BACKFILLED

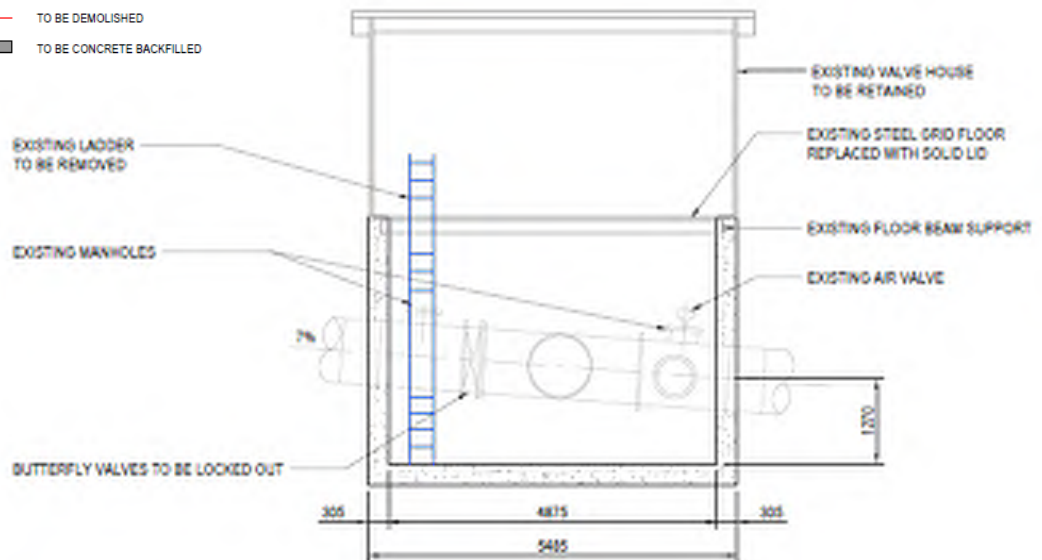


Figure 6-15 Typical cross section of High Level Outlet Pipes – Option 1D

7. Partial decommissioning concept design

7.1 Concept design Option 1D spillway configuration

The design was iterated with various secondary spillway configurations to approach but not exceed the peak outflows from existing conditions, over a range of events up to and including the 1 in 100 AEP.

7.1.1 Baseline “existing” conditions

A key consideration was how ‘existing’ conditions should be defined – either the original design, or the Restricted FSL case. The Developer Services group at MWC was consulted with regards to how the planning levels and design criteria were being set for new developments downstream of Beaconsfield Reservoir. The flows being used for planning purposes were compared to both the original FSL and Restricted FSL cases, as summarised in Table 7-1.

It should be noted that the Developer Services RORB model assumes significant development between Browns Road and the Princes Highway, and accordingly has increased impervious fractions in this area.

The Developer Services model includes three subareas upstream of the dam, and an approximation of the reservoir, which has a different storage discharge relationship to either the original FSL or the RFSL configurations. It represents a 150 m long spillway at RL 104.25 mAHD, which results in much higher peak flows from the reservoir. The impact of including a more accurate representation of the reservoir is shown in Line References 5 and 6 of Table 7-1 below.

With reference to Table 7-1 below it can be seen that:

- The flows being used for planning purposes are slightly lower (within 1.0 m³/s) compared to those generated by adopting ARR 2016 data and an ensemble approach. The same catchment file and k_c value were adopted.
- Representing the correct spillway level and length (original FSL) in the Developer Services model produces equivalent flows at Browns Road, and a reduction in peak flows at the Princes Highway, when adopting ARR 2016 data and an ensemble approach.

The FSL scenario was adopted as the existing case on this basis – refer to Line Reference 4 in Table 7-1 below.

Table 7-1 RORB model to Princes Highway – comparison with flows used for development planning/approvals

Line reference	Scenario	Kc	Number of subareas upstream of dam	Rainfall and losses	Cease to flow elevation (RL mAHD)	1% Beaconsfield Reservoir peak outflow (m ³ /s)	1% Browns Rd peak flow	1% Princes Hwy peak flow
1	MWC Developer Services planning flows (using ARR 1987 methodologies/parameters). $k_c=5.6$	5.6	3	ARR 1987	104.25	7.5	17.7	23.7
2	MWC Developer Services RORB model using ARR 2016 data and methodologies (ensemble approach). $k_c=5.6$	5.6	3	ARR 2016 (ensemble approach)	104.25	8.0	18.2	24.4
3	Extended Beaconsfield Reservoir RORB model for RFSL storage representation with k_c/d_{av} ratio of Reservoir model maintained	10.05	9	ARR 2016 (ensemble approach)	98.85	3.2	no print out	21.0
4	Extended Beaconsfield Reservoir RORB model for original FSL storage representation with k_c/d_{av} ratio of Reservoir model maintained	10.05	9	ARR 2016 (ensemble approach)	103.08	5.0	no print out	21.3
5	MWC Developer Services RORB model using ARR 2016 data and methodologies (ensemble approach). $k_c=5.6$. Storage representing Restricted FSL	5.6	3	ARR 2016 (ensemble approach)	98.85	3.6	17.4	21.5
6	MWC Developer Services RORB model using ARR 2016 data and methodologies (ensemble approach). $K_c=5.6$. Storage representing original FSL	5.6	3	ARR 2016 (ensemble approach)	103.08	5.6	18.2	23.1

7.1.2 Concept design flows for events as frequent as 1% AEP (up to 100-year ARI)

The concept design was configured in the catchment file, with the discharge curve combining:

- Primary spillway at RL 94.0 mAHD (Low Level Outlet)
- Secondary spillway at RL 95.5 mAHD (10 m wide broad-crested weir, $C_d=2$)

Both ensemble and Monte Carlo approaches have been used to check outflows are not increased between the existing (original FSL) and the concept design conditions. The ensemble approach uses median burst losses and assumes AEP neutrality (that 1% AEP rainfall produces a 1% AEP flood), whereas the Monte Carlo approach samples combinations of initial loss and rainfall depth. This is in recognition of the fact that a smaller rainfall event on a wet catchment may produce a greater flood peak than a larger rainfall event on a dry catchment.

The outflows are summarised in Table 7-2 and Table 7-3 below for the ensemble and Monte Carlo approaches respectively. The two approaches yield results, which are between 0-0.4 m³/s different, with the Monte Carlo results generally slightly smaller.

Table 7-2 Ensemble outflows (data hub losses and median pre-burst depths)

AEP	Existing (original FSL) peak outflow	Critical duration	Concept design peak outflow	Critical duration	Comment
1 in 10	2.3	9 hour	1.1	9 hour	Design secondary spillway not activated
1 in 20	3.3	9 hour	1.1	9 hour	Design secondary spillway not activated
1 in 50	4.5	9 hour	3.9	12 hour	
1 in 100	5.8	12 hour	5.5	9 hour	Developer Services assumed 7.5 m ³ /s for planning/design purposes

Table 7-3 Monte Carlo outflows (no pre-burst adjustment)

AEP	Existing (original FSL) peak outflow	Critical duration	Concept design peak outflow	Critical duration	Comment
1 in 10	2.2	9 hour	1.1	9 hour	Design secondary spillway not activated
1 in 20	3.1	9 hour	1.1	12 hour	Design secondary spillway not activated
1 in 50	4.4	12 hour	3.5	12 hour	
1 in 100	6.1	12 hour	5.5	12 hour	Developer Services assumed 7.5 m ³ /s for planning/design purposes

A range of continuing loss values were also tested for sensitivity. As shown in Figure 7-1 and Figure 7-2 following, the ensemble approach outflows are sensitive to the continuing loss assumption. For the 1% AEP, if the continuing loss value was reduced to 2.6 mm/hr or less, a slight adjustment would need to be made to the design for Option 1D to match the original FSL outflows for the 1% AEP.

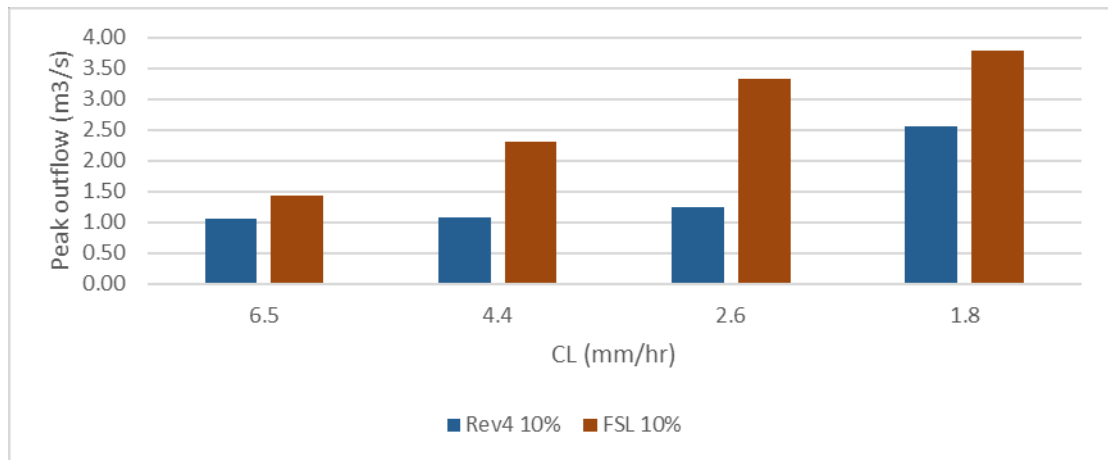


Figure 7-1 FSL and concept design Option 1D outflow comparison (10% AEP)

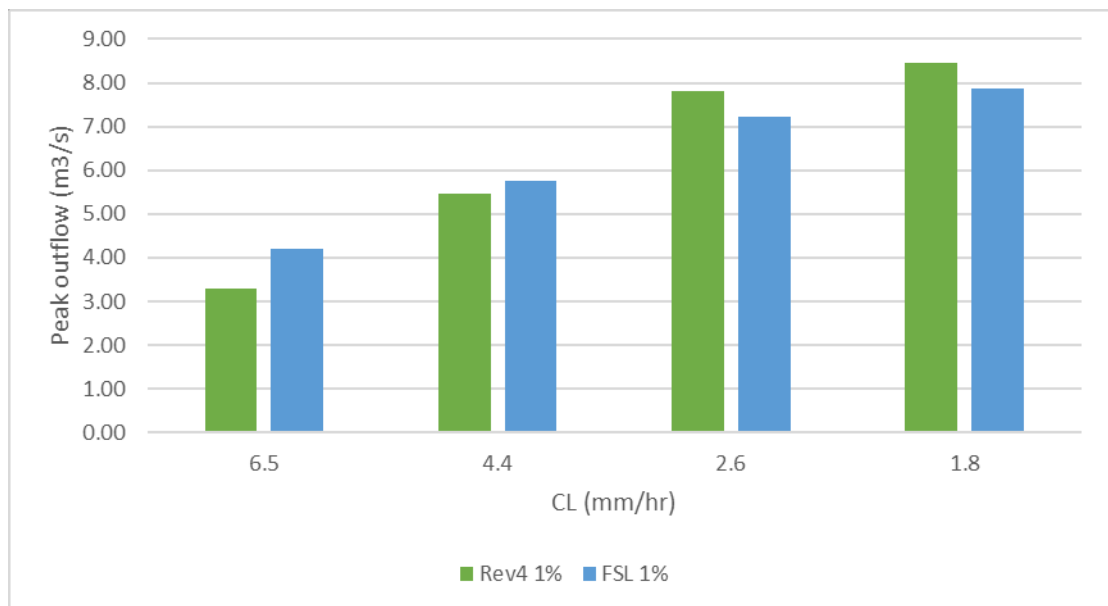


Figure 7-2 FSL and concept design Option 1D outflow comparison (1% AEP)

As a final check, the Developer Services RORB model was updated to reflect the Concept Design for Option 1D and rerun. As can be seen in Table 7-4 below, the design flows at Browns Road and the Princes Highway were not increased by this change.

Table 7-4 Comparison with flows used for development planning/approvals

Line reference	Scenario	1% Beaconsfield Reservoir peak outflow (m ³ /s)	1% Browns Rd peak flow	1% Princes Hwy peak flow
1	MWC Developer Services planning flows (using ARR 1987 methodologies/ parameters). $k_c=5.6$	7.5	17.7	23.7
2	MWC Developer Services RORB model using ARR 2016 data and methodologies (ensemble approach). $k_c=5.6$. Storage representing restricted FSL RL 98.85 mAHD.	8.0	18.2	24.4
3	MWC Developer Services RORB model using ARR 2016 data and methodologies (ensemble approach). $k_c=5.6$. Storage representing original FSL RL 103.08 mAHD.	5.6	18.2	23.1
4	MWC Developer Services RORB model including Concept Design for Option 1D using ARR 2016 data and methodologies (ensemble approach). $k_c=5.6$	5.4	17.4	21.4

Scenario 2 and 3 are similar, however scenario 3 uses a storage representing the original FSL of RL 103.08 mAHD, whereas scenario 2 adopts the restricted FSL of RL 98.85 mAHD.

7.2 Concept design Option 1D dam crest level

Following discussions with MWC, the embankment crest level was set at approximately the 1 in 200 AEP level, with some allowance for changes in continuing loss and/or Low Level Outlet blockage. The embankment crest level has been set at RL 96.1 mAHD, which allows approximately 10.4 m³/s through the primary and secondary spillways combined. This makes the DCF between the 1 in 200 AEP and 1 in 500 AEP events for various modelling sensitivity analyses undertaken. Critical duration outflows for different scenarios are given for comparison in Table 7-5 following. Given that a combination of data hub and GSDM/GSAM temporal patterns may be required to smooth the transition between the 1% AEP and the very rare events, both have been simulated to assess the impact of temporal patterns and pre-burst depth assumptions. Due to the storage, the critical outflow duration is 9-12 hours, however, the critical inflow duration is much shorter.

Table 7-5 Ensemble and Monte Carlo flow comparisons for 1 in 200 and 1 in 500 AEP events (assuming glass wall)

Scenario	Critical duration 1 in 200 AEP design outflow (m ³ /s)	Critical duration 1 in 500 AEP design outflow (m ³ /s)
Ensemble (base k_c and losses) data hub temporal patterns	8.6 (9 hour)	
Ensemble (base k_c and losses) GSDM/GSAM temporal patterns	8.2 (12 hour GSAM)	11.5 (12 hour GSAM)
Monte Carlo (base k_c , no pre-burst adjustment) data hub patterns	8.1 (12 hour)	12.8 (12 hour)
Ensemble (base k_c , 2.6 mm/hr CL)	10.1 (12 hour)	
Ensemble (base k_c , 2.6 mm/hr CL) GSDM/ GSAM temporal patterns	10.4 (12 hour GSAM)	13.3 (12 hour GSAM)

7.3 Concept design Option 1D – design overtopping event

It was agreed that given the widened crest and reduced embankment height a credible wet day failure is likely to occur only after a continuous overtopping of in excess of 300 mm for at least 6 hours occurred.

A range of events were simulated to estimate the AEP at which approximately overtopping failure of the embankment would occur for a period of approximately 6 hours or more. GSDM and GSAM patterns (with pre-burst) were used from the 1 in 200 AEP event.

As per Figure 7-3 the estimated AEP for an overtopping depth of 300 mm (RL 96.4 mAHD) is approximately 1 in 1,000,000 AEP (12 hour GSDM).

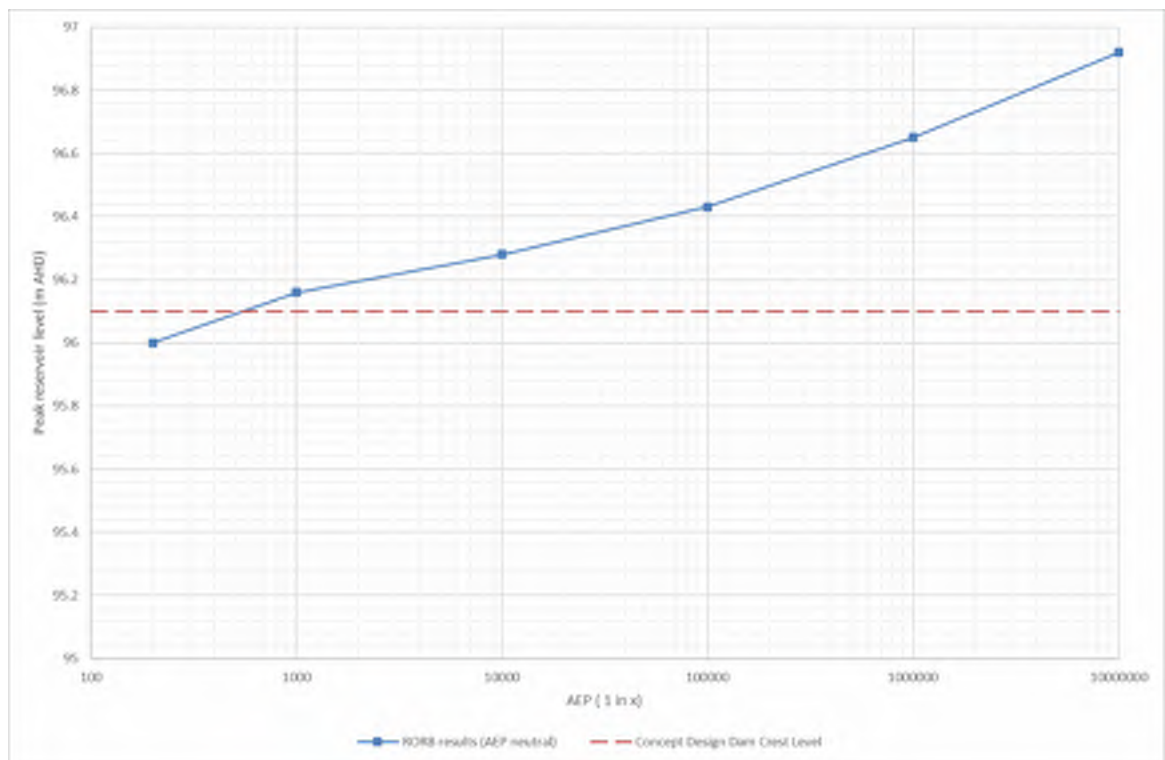


Figure 7-3 Beaconsfield Reservoir Concept Design Flood Frequency – Option 1D

7.4 Concept design Option 1D – wet day overtopping dambreak and consequences

7.4.1 Breach parameters

The range of empirical breach parameters for the 1 in 1,000,000 AEP overtopping breach are shown in Table 7-6 below.

Table 7-6 Empirical breach parameters for 1 in 1,000,000 overtopping breach (Option 1D)

Empirical equation	Side slope	Concept design overtopping breach base width (m)	Concept design overtopping breach development time (min)
MacDonald Langridge Monopolis (Wahl) ¹	0.2	1.2	16
Bureau of Reclamation	0.2	26.3	19
Froehlich (2008)	1	9.8	16
Von Thun Gillette (assuming erosion resistant)	1	20.5	47
Singh and Scarlatos minimum breach base width	0.2	4.9	22 (applying Wahl earthfill equation to volume of embankment eroded)
Singh and Scarlatos minimum breach base width	0.2	4.9	45 (applying Wahl earthfill equation to volume of embankment eroded)

The FLDWAV model was adapted to represent a breach triggering at the time of approximately 6 hours of overtopping by approximately 300 mm, and the different sets of breach parameters simulated. The resulting breach hydrographs are shown in Figure 7-4 following. Both the Bureau of Reclamation (1988) and Froehlich (2008) parameters produce very similar breach hydrographs. In this instance, it was not expected that these would show appreciable differences in flood behaviour at the location of the first dwelling (the Scout Park caretaker's residence). For the third breach simulated, the lower bound sensitivity was taken to be the minimum Singh and Scarlatos breach base width, with the time derived from the volume of embankment eroded, using the MacDonald Langridge-Monopolis development time equation from DERM (relationship shown in ANCOLD Bulletin 97).

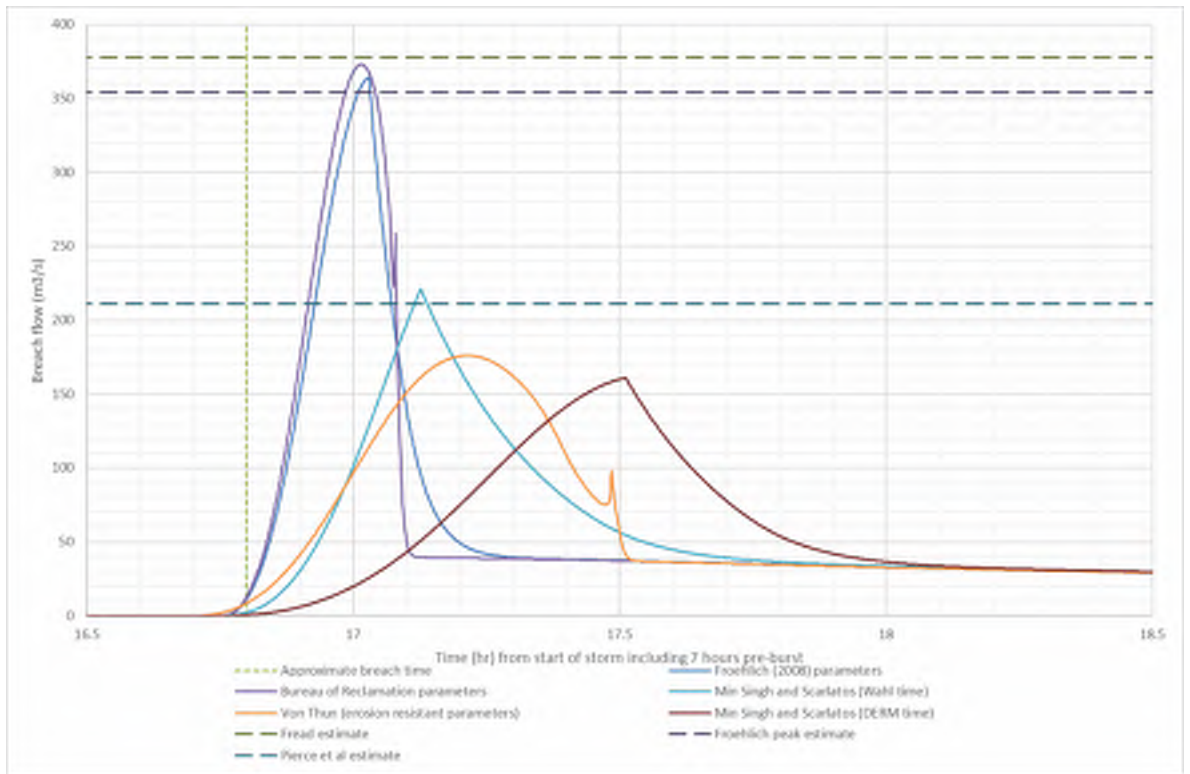


Figure 7-4 Wet day (overtopping failure) breach hydrographs – Option 1D

The adopted breach parameters are given in Table 7-7 below.

Table 7-7 Adopted overtopping breach parameters – Option 1D

Scenario	Basis	Breach base width (m)	Breach development time (min)	Peak flow (m³/s)
No breach		NA	NA	41
Upper bound sensitivity	Bureau of Reclamation	26.3	19	373
Most likely	Min Singh and Scarlatos breach base width with adjusted MacDonald Langridge Monopolis (after Wahl) breach time	4.9	22	222
Lower bound sensitivity	Min Singh and Scarlatos breach base width with adjusted MacDonald Langridge Monopolis (after DERM) breach time	4.9	45	161

7.4.2 PAR and PLL estimates – Option 1D

The area impacted by the breach as defined by 300 mm or greater incremental depth extends to the Princes Highway, as shown in Figure 7-5 below.

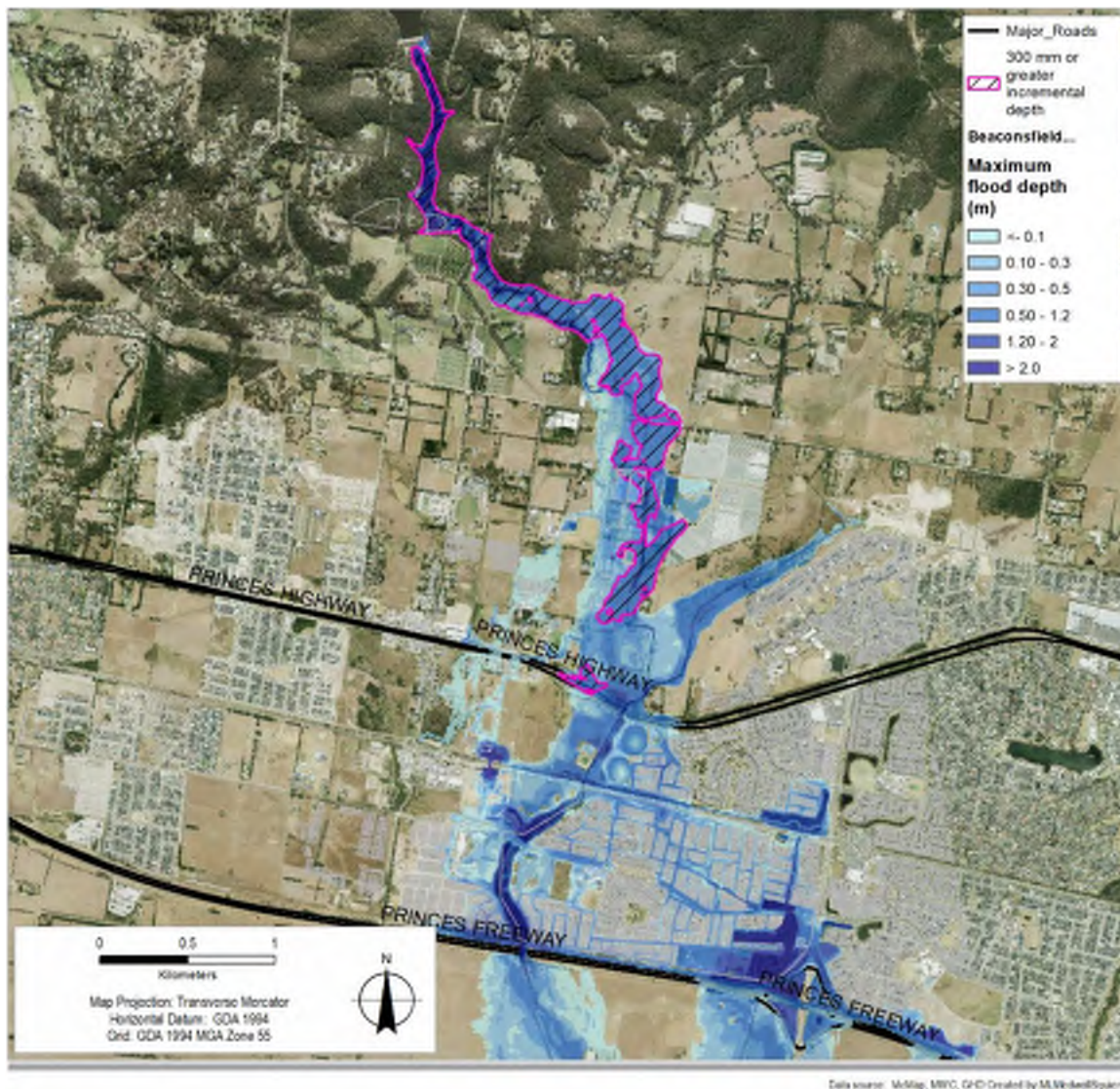


Figure 7-5 Overtopping breach affected zone – concept design Option 1D

Existing development in the breach-affected zone includes:

- Four dwellings, including the Scout Park caretaker’s residence
- Glass houses and packing sheds at Boon Roses (assume four staff for eight hours during the day and none at night)

Apart from the caretaker’s residence, it has been assumed that no one would be present at the Scout Park during such an extreme flood event.

It has been assumed that no traffic would be attempting to pass the Princes Highway in such a rare flooding event. As demonstrated by Figure 7-6 following, the road will have been inundated by more than 300 mm for an extended period of time prior to the arrival of the breach flood wave.

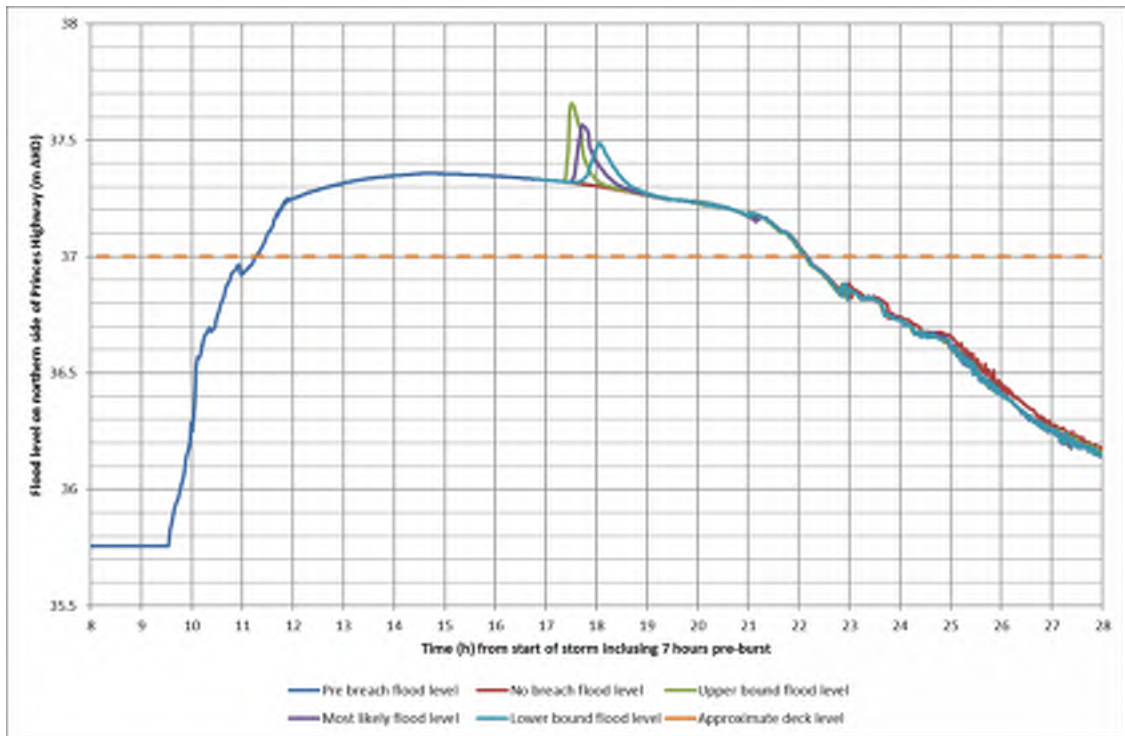


Figure 7-6 Flood level hydrographs at the Princes Highway during 1 in 1,000,000 AEP overtopping design event

Inspection of the pre-breach flood results shows that prior to the breach occurring, the dwellings would not have been inundated significantly, however the glasshouses and sheds at Boon Roses would have been inundated by between 0.1-0.5 m. The PAR at Boon Roses has still been considered, on the grounds that staff may still attempt to salvage plants and equipment under these conditions. This is considered a conservative assumption, as it is likely that workers would not have travelled to work in flooded conditions.

The resulting PAR and PLL estimates for the overtopping failure associated with concept design Option 1D are summarised in Table 7-8 following.

Table 7-8 PAR and PLL summary for Option 1D overtopping failure

Parameter	Day	Night	Day/ Night weighted average	Day	Night	Day/ Night weighted average	Day	Night	Day/ Night weighted average
	Overtopping failure			No breach (within 300 mm incremental depth boundary for respective breach)			Incremental		
Number of buildings flooded above floor	2-5			1-2			1-3		
PAR at floor level	3-9	6-11	4-10	1-5	3-6	2-4	2-4	3-5	2-6
PLL at floor level (USBR suggested median)	0.01-0.06	0.03-0.08	0.02-0.07	0.000-0.001	0.000-0.001	0.000-0.001	0.01-0.06	0.03-0.08	0.02-0.07

The application of the 300 mm incremental depth threshold means that Boon Roses is included in the no-breach numbers for the upper bound breach, but not for the lower bound and most likely breach (as it does not experience a 300 mm increase in flood levels over no breach conditions for these scenarios).

The weighted PLL under a Wet Day Failure is less than 0.1 and therefore achieves a Consequence Category of Low as per the design criteria.

8. Landscape design

8.1 General

A critical part of the concept design is to provide an indication of possible treatments of the upgraded site including:

- Improved access and connectivity
- Equitable use of site
- Cultural connections
- Quality open space
- Passive and active recreation space balance
- Enhancing connections to nature
- Access and safety
- Shade structures

The design is focused on providing details on proposed infrastructure such as walking trails, site furniture, shelters, boardwalks, lookouts, and potential bunding to the upstream area of the reservoir for improving biodiversity and habitat for the upstream area.

Landscape architecture concept designs have been developed for the three initial concept options (Options 1A to 1C) and the final concept design (Option 1D). Each design includes a masterplan and detailed section of the dam. Refer to Appendix I for drawings.

The overall landscape masterplan outcome for the partial decommissioning of Beaconsfield Reservoir can be broken down into three key elements. These include:

- The re-designed smaller water body
- Circuit walking trails
- The picnic and passive recreation area

8.2 Landscape elements

8.2.1 Smaller water body

The revised and smaller open water body provides opportunity to convert the remaining extent into a functioning wetland, to ensure the original footprint of the dam is utilised to its full potential. Characteristics of this wetland shall include and consider:

- A planting palette and profile of indigenous species that is consistent with MWC's preferred planting zones highlighted in their Constructed Wetlands Design Manual (Ephemeral Planting Zone, Shallow Marsh Zone, and Deep Marsh Zone, etc.).
- Species selected will be indigenous to the local area.
- Rock, earth bunding, and trees removed because of future works will be used to create pool and riffle elements that will aerate water within the wetland, as well as create opportunity for native flora and fauna habitat, helping biodiversity.
- Facilitate the migration and population of indigenous flora and fauna species through the reintroduction of conditions best suited to their cultivation and prosperity.

8.2.2 Circuit walking trails

Walking trails within the site extent are separated into a series of concentric circuit paths, providing options for users of varying walking abilities to navigate the site as well as several different experiential qualities. These will be clearly denoted by trail signage at regular intervals. The path networks include:

- Opening an existing walking trail to the public that meanders through the existing bushland immediately surrounding the wetland. The circuit path will be punctuated with picnic and seating opportunities to serve as respite areas for walkers, as well as capture key vistas and views of the waterbody. The path also extends to use parts of the spillway channel and decommissioned Bunyip Main Race.
- A new internal track around the current water level will service users who wish to experience the site at a more leisurely pace. New low profile steel bridges crossing the smaller tributaries to the north will allow for uninhibited views of the open water body and the tributaries upstream. This internal track and its structures will be robust enough to allow for it be utilised as periodical maintenance access.
- A new compacted gravel all access loop path that allows for pedestrian mobility in and around the old dam wall to the south and the amenities that service the passive recreation area. This will also serve as periodical maintenance access due to the location of the existing shed.

8.2.3 Picnic and passive recreation area

The picnic and passive recreation area will provide amenity for respite and end of trip facilities, as well as brief experiential landscape interventions. Elements of this area include:

- A new-cantilevered steel mesh viewing deck to the north to take advantage of the long views over the water.
- A low profile steel boardwalk along the base of the old spillway to the east to connect with the walking loop trail that takes in the old spillway and Haunted Gully Creek.
- Newly graded grass dam embankment to provide informal seating and passive recreation opportunities.
- Utilising this new open lawn area further to the south to include picnic tables, barbecue and a shelter, facilitated by a small pedestrian access path.

9. Cost estimates

9.1 General

Estimates of the project costs has been developed for the concept design (Options 1A to 1D). This section describes the assumptions, limitations and accuracy of these costs, selection of unit rates, and percentages assigned to overheads.

9.2 Key assumptions, limitations and accuracy

The level of project definition and the accuracy of the inputs influences the accuracy of the cost estimates. In this regard, the following should be considered for the cost estimate:

- The design has been developed at a 'concept' level of design, and the accuracy of the cost estimates, and in particular the adopted Very High and Very Low Variances, are based on this level of project definition.
- The cost estimates covers the extent of works as detailed in Section 6 for each of the initial Concept Options 1A, 1B, 1C and adopted Concept Option 1D, including Landscape Works as detailed in Section 8.
- A nominal allowance is included in the cost estimates for a number of items, which are unique to this project. These estimates were developed through engineering judgement.
- The cost estimates do not include any costs associated downstream of the Landscape Works (i.e. of the stilling basin and/or headwall discharging into Haunted Gully Creek).
- Estimates of foundation conditions have been based on interpreting the existing geotechnical information.
- It is assumed that the contractor will have relatively open access to the work site, and will not be restricted by limited hours of operation.
- Optimisation of the embankment width and slope should be investigated in future stages of design. The current cost estimates are based on a 5H:1V profile with disposal of spoil onsite. If this is not feasible, a revised downstream batter slope should be considered and costs should be re-calculated based on the new design. Likewise, if excess embankment material is required to be disposed offsite, costs should be re-evaluated.
- It is assumed that no extensive environmental permits will be required.
- It is assumed that drawdown of the reservoir can be achieved during the construction phase to allow for dry conditions when undertaking embankment works.

It is considered that the cost estimates have a -30% / +100% level of accuracy. This is comparable with:

- The Association for the Advancement of Cost Engineering International (AACEI) recommended ranges for 'Class 4' – Order of Magnitude/Concept Study (i.e. -30%/+100%), which was considered appropriate in this case where a large risk workshop involving experienced GHD and MWC personnel was not undertaken.

9.3 Unit rates and percentage items

9.3.1 General

Rates have been selected for items where the quantity of work is measurable in terms of units or lump sums. For other items, such as establishment/disestablishment costs, minor items and contingencies, where the quantity of work is more difficult to measure, allowances have been

included in the cost estimates as percentage-based items. The selection of these unit rates and percentages is discussed in the following sections.

9.3.2 Unit rates

Unit rates for various components of the work were selected with reference to rates used on recent previous construction projects. Reference was also made to the following documents:

- Rawlinsons, Australian Construction Handbook, 2013
- Information provided by Alan Rae Consulting as part of the 2012 Remedial Works Design for Beaconsfield Reservoir
- Estimates of MWC internal time and resources (pro rata where necessary) from Maroondah Reservoir New Outlet Concept Design (GHD, 2018a).

Unit cost rates for each item are all inclusive. For instance, unit rates for reinforced concrete includes labour, preparation, formwork, jointing, sealant, compaction, admixtures, reinforcing steel, bar bending and fixing, and surface treatment. Unit rates adopted for earthfill includes supply, placement and compaction with testing separately priced.

9.3.3 Percentage items

General costs for items including establishment, miscellaneous items, contingencies, and design and construction management have been taken as a percentage of the total construction costs, and are based on an assessment of recent project percentages as well as typically accepted values. In terms of contingency percentages, it is acknowledged that uncertainties exist in some of the quantities and unit rates, including material sources and foundation conditions.

The percentages adopted in the cost estimates are:

- SP Project Management– 15% of Direct Cost
- SP Detailed Design – 14% of Direct Cost
- SP Risk – 12% of Total Cost
- SP Margin – 15% of Total Cost
- Program Management Allocation – 7.5%

9.5 RANE analysis

RANE analysis was undertaken using the base cost (CAPEX) inputs and the risk register inputs. The RANE inputs (costs and risks) for are included in Appendix K and in Appendix L respectively.

A summary of the RANE outputs is provided in Table 9-1 below.

Table 9-1 Summary of RANE outputs

	RANE output (\$M)			
	Option 1A	Option 1B	Option 1C	Option 1D
Base Project Cost				
Low Expected Project Cost, P5				
Expected Project Cost, P50				
High Expected Project Cost, P95				
Contingency (P95 – P50)				
$(P95-P50)/P50$				
$(P95 - \text{Base Cost}) / \text{Base Cost}$				

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10. Next steps

10.1 Overall Beaconsfield Reservoir concept design strategy

The next steps for the project are largely associated with MWC's assessment of the options for Beaconsfield Reservoir Concept Design. Based on earlier strategy work (MCA), MWC identified that the focus of this Concept Design Report should be on partial decommissioning options. As such, the three main options to review as part of the overall strategy were as follows:

1. **Option 1A:** Modify the Low Level Outlet to act as a Primary Spillway and install a wide concrete structure as the Secondary Spillway.
2. **Option 1B:** Modify the Low Level Outlet to act as a Primary Spillway and make the embankment overtoppable to act as the Secondary Spillway.
3. **Option 1C:** Decommission the Low Level Outlet and install a new pipe to function as the Primary Spillway and concrete culverts as the Secondary Spillway.

The key differences between Options 1A and 1C are the outflow channel in Option 1C would be narrower and taller and concrete would be visible, while Option 1A has the potential to have topsoil and grasses covering the rock-lined channel. As described in Section 8 within the Landscape Drawings, there exists the potential to improve the aesthetics of the rock-lined channel to cascade, thereby creating a "natural" rock stream with pools.

Option 1B would be able to be overtopped for the entire length of the crest. This would engage at floods larger than 1 in 100 AEP, which would erode the topsoil and grass in the event of significant overtopping. The Primary Spillway would remain the same as in Option 1A, with the current Low Level Outlet modified to act as the Primary Spillway. Options 1A through to 1C all satisfied the design criteria for a Sunny Day Failure ($PLL < 0.1$) but did not satisfy the criteria during a Wet Day Failure.

Option 1D was developed to meet the design criteria under both Sunny Day Failure and Wet Day Failure scenarios to have an estimated PLL of less 0.1. The key design components for Option 1D are outlined in Table 10-1.

Table 10-1 Key design features of Option 1D

Key Component	Detail
FSL	RL 94.0 mAHD
Dam crest level	RL 96.1 mAHD
Secondary Spillway Type	Constructed on good quality natural rock through crest with concrete sill discharging into downstream rock beaching-lined channel
Secondary Spillway Level	RL 95.50 mAHD
Secondary Spillway Length	10 m

10.2 Requirements for future stages

10.2.1 General

A Detailed Design (and a Functional Design prior if deemed necessary) would be undertaken in future stage(s) before the construction phase to confirm reduction of the Consequence Category to 'Low'. This section, "Requirements for Future Stages", outlines aspects which future designers, MWC and eventually construction contractors should consider, which are a function of the assessments made and designs undertaken as reported herein, subject to the assumptions and limitations associated with this Report.

10.2.2 Future stages

Requirements during future stages relate to scope and information limitations and assumptions made in this Report, which must be considered as part of any future stage scopes. These include:

- The scope of the current study did not include any supporting studies to identify environmental, cultural or heritage issues that might affect the identified options. All information reviewed in this regard was provided by MWC. Consideration should be given to the investigation regarding environmental, cultural and heritage issues if deemed necessary by MWC or future designers.
- No seismic assessment was undertaken and no inherent defects were assumed at the site, apart from the already known issues with seepage originating on the downstream right abutment groin and dam instability.
- It is anticipated that during Detailed Design (and prior Functional Design if undertaken) that further site investigations will be undertaken as deemed required, such as boreholes, UCS testing of rock (pending confirmation of option selection) and other tests deemed necessary by future designers.
- The cost estimates are based on simplified estimates of quantities and rates and should only be relied on for the purposes as set out in the Project Brief. The conditions required high-level concept designs and cost estimates of the upgrade options for planning purposes. A more detailed cost estimate should be undertaken following future stages to confirm initial cost estimates.
- The current study was undertaken at a feasibility level, which is a preliminary study typically undertaken to determine, analyse, and select the best business scenarios. This level of study is required where there is more than one business scenario, and it is necessary to determine which one is the best, both technically and financially.
- The LiDAR survey undertaken in 2017 terminates marginally downstream of the berm on the downstream batter of the embankment. This is short of the proposed new toe of embankment and hence there exists some uncertainty in the levels and earthworks quantities.
- The dambreak modelling and Consequence Category Assessment has been undertaken based on existing conditions. Development occurring downstream (between Browns Road and the Princes Highway in particular) will alter the terrain, drainage features (and in turn the flood behaviour), and locations and types of PAR in some areas. These changes have the potential to affect the outcomes of the Consequence Category Assessment. Updated survey and building footprints will need to be incorporated into the assessment at Detailed Design phase.

- Construction support and supervision is anticipated to be required by the designers engaged for the Detailed Design. This will include but not be limited to the construction of critical dam safety structures including any modification, decommissioning or installation of existing or a new Primary Spillway. Similarly, Secondary Spillway structures will require supervision during the construction of critical components.
- The Concept Design Drawings as presented as part of this report in Appendix I are not suitable for construction tendering purposes and should not be relied on. A suitable designer should be engaged to further examine site conditions and obtain more accurate estimates and information to develop Detailed Design Drawings before the construction tender process.

Future requirements can also relate to key risks identified in this Report, which should be considered as part of following stages. These include:

- There is strong community interest in Beaconsfield Reservoir. It is critical to manage risks associated with fauna, flora and water quality (upstream, downstream and bore) to prevent project delays and poor public relations

11. Conclusions and recommendations

11.1 Conclusions

The multi-criteria analysis undertaken suggests that the most appropriate option is to partially decommission Beaconsfield Dam. The analysis was weighted to reflect the relative importance of each criterion. The criteria included:

- Cost
- Satisfying ALARP
- Community impacts
- Environmental and conservation impacts

Hydrological analysis of Beaconsfield Reservoir found that a Low Consequence Category is achieved for a Sunny Day Failure scenario at a lowered FSL of RL 94 mAHD (primary spillway level). The secondary spillway and crest design was then further iterated to also achieve a Low Consequence Category for the Wet Day Failure scenario. To achieve a Very Low Consequence Category, the reservoir level may need to be further lowered so that the Severity of Damage and Loss becomes 'Minor'.

With a target Low Consequence Category, Beaconsfield Reservoir required:

- No increase in the peak outflows for the 1 in 100 AEP flood event (in the order of 5 m³/s)
- To safely pass the 1 in 1000 AEP flood event

There are four Partial Decommissioning Options presented as part of this Report. It is noted there exists the potential for variations to design based on community feedback, an internal review, and in compliance with dam safety regulations and ANCOLD. The four options include:

1. **Option 1A:** Modify the Low Level Outlet to act as the Primary Spillway and construct a Secondary Spillway by excavating into good quality rock, with invert controlled by a concrete sill.
2. **Option 1B:** Modify the Low Level Outlet to act as the Primary Spillway and make the embankment overtoppable to act as the Secondary Spillway.
3. **Option 1C:** Decommission the Low Level Outlet and install a new pipe to function as the Primary Spillway and concrete culverts as the Secondary Spillway.
4. **Option 1D:** Modify the Low Level Outlet to act as the Primary Spillway and construct a Secondary Spillway by excavating into good quality rock, with invert controlled by a concrete sill. Same as Option 1A but crest and spillway at lower relative levels to achieve a Low Consequence Category for both Sunny Day and Wet Day failures.

11.1.1 Comparison of Options 1A to 1C

Options 1A and 1B would modify the current Low Level Outlet to act as the new Primary Spillway. Option 1A would construct a Secondary Spillway on one of the abutments comprised of a wide concrete structure for the control sill and good quality rock for the chute, whereas Option 1B would have the embankment crest as overtoppable for the full length of the crest. Option 1C would decommission the current Low Level Outlet and construct a new Primary Spillway and Secondary Spillway on the abutment, with new pipe and concrete culverts. Option 1C would have the Primary Spillway discharging into the Secondary Spillway chute and therefore would require a rock-lined channel without topsoil, whereas the Secondary Spillway

chute for Option 1A could be topsoiled and grassed (over the channel), noting this would erode away during sizeable floods above the 1 in 100 AEP event.

Options 1A through 1C satisfied the design criteria under a Sunny Day Failure where the PLL is less than 0.1. However, under a Wet Day Failure all three options did not achieve this target. An iterative approach was undertaken to determine the option that would not only achieve a $PLL < 0.1$ during a Sunny Day Failure but also under a Wet Day Failure (Option 1D).

11.1.2 Option 1D

Option 1D was developed via an iterative approach, to achieve a Low Consequence Category for both Sunny Day and Wet failure scenarios. Although it has similarities to the initial options proposed, the modified geometry and elevations in Option 1D have resulted in a PLL of less than 0.1 under both Sunny Day and Wet Day failure scenarios.

Option 1D would involve modifying the Low Level Outlet to act as the new Primary Spillway, with a FSL at RL 94.0 mAHD. A Secondary Spillway, 10 m long, would be excavated into good quality rock on one of the abutments, with the unlined rock channel forming the spillway chute. A concrete sill would fix the invert level of the spillway, designed at RL 95.5 mAHD, with an earthen approach channel. The natural rock chute through the crest would be tied into a fabricated rock beaching-lined channel. The dam crest is RL 96.1 mAHD, with overtopping commencing in events rarer than 1 in 200 AEP. The rock beaching-lined channel and/or spillway chute could be topsoiled and grassed, as landscape drawings depict.

Landscaping Designs have been prepared with a focus on promoting to the picnic area and wetlands. These designs have the flexibility to be altered with other suggested options of design possible, as noted in Section 8, depending on MWC’s preferences following internal and stakeholder reviews.

Table 11-1 RANE estimates

	RANE Output (\$M)			
	Option 1A	Option 1B	Option 1C	Option 1D
Base Project Cost				
Low Expected Project Cost, P5				
Expected Project Cost, P50				
High Expected Project Cost, P95				
Contingency (P95 – P50)				
$(P95-P50)/P50$				
$(P95 - \text{Base Cost}) / \text{Base Cost}$				

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11.2 Recommendations

If the works to reduce Beaconsfield Reservoir to a Low Consequence Category are not intended to be undertaken soon, interim actions should include:

- Visually monitor seepage – especially that originating from the downstream right abutment groin.
- Visually monitor the dam for signs of instability.

- Plan and undertake recommended actions developed in 2018 Beaconsfield Comprehensive Inspection (GHD, 2019). It is noted that those actions are independent of any major capital works.

There exists some uncertainty around cost estimates for the construction of the partial decommissioning based on the simplified geometries assumed given the stage of the assessment and finite amount of information. To better understand the costs of each option, further site works should be undertaken to reduce uncertainty. Future investigations could include:

- Boreholes to assess foundation level.
- Further investigation of the Low Level Outlet condition, including full-length external tunnel inspection and an internal pipe inspection to reduce uncertainty around the condition and cost estimate for works associated with the pipe.
- Obtain/request LiDAR data from the Capacity Survey (Taylors, 2017) extending beyond the berm on the downstream embankment to reduce uncertainty in quantity estimates. If the data does not exist, consider commissioning new survey, which captures the downstream toe area of the new embankment.
- If an option that includes a Secondary Spillway on the abutments is the preferred option, consider laboratory testing to confirm rock strength.

12. References

Various Drawings (refer to Appendix B).

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Appendices

Appendix A – Dam information

Component	Description	Reference
General		
Name	Beaconsfield Reservoir	
Watercourse	Haunted Gully Creek is directly downstream of the reservoir	MWC 2015
Location	Access from O'Neil Rd, Officer	SMEC 2012
Purpose	<p><i>Original purpose</i></p> <p>Water supply (no longer connected to supply) by Tarago Reservoir via the Tarago Main Race and a series of small diversion weirs via the Bunyip Main Race.</p> <p><i>Since 1988</i></p> <p>Ornamental lake</p>	MWC 2015
Coordinates (Map Grid of Australia)	Zone 55, 5789728 N 360403 E	MWC 2015
Map Reference	Melway Map 212 H 7	MWC 2016 sketch plan
Year of original construction	1918	SMEC 2012
Original designer and constructor	State Rivers & Water Supply Commission	SMEC 2012
Operator	Melbourne Water Corporation	MWC 2015
Imperial datum conversion (for old drawings)	RL (m AHD) = { RL (ft.) * 0.3048 m/ft. } – 0.552 m	GHD 2012
Upgrades and remedial works		
1970	<ul style="list-style-type: none"> • New Outlet constructed • Old high level outlet “abandoned” by c. 1972 	SMEC 2012 Dwg 113610C
1988	<ul style="list-style-type: none"> • Old Outlet converted to scour outlet • Old scour pipe and base of old tower grouted • Old Tower cut down flush with upstream face of embankment • Toe area cleared, and stabilising fill placed to construct stabilising berm. 	SMEC 2012
2014	<ul style="list-style-type: none"> • Obstructions removed from spillway including the outlet pipe • Training wall constructed on RHS of spillway • DN 225 Scour replaced by DN 450 scour with baffle, rock chute and a DN 100 bypass for low flows. • New reservoir level sensor and rain gauge connected to SCADA 	MWC 2015
2016 (unconfirmed but planned)	<ul style="list-style-type: none"> • Works to seal around perimeter of new penstock gate. 	MWC 2015
Consequences of dam failure		
Population at Risk (PAR)	Sunny Day Failure: 334-408 Incremental Wet Day Failure: 1372-1676	GHD 2019

Component	Description	Reference
Potential Loss of Life (PLL)	Sunny Day Failure: 2.2-2.8 Incremental Wet Day Failure: 8.9-11.6	GHD 2019
Severity of Damage and Loss	Sunny Day Failure: Medium Incremental Wet Day Failure: Medium	GHD 2019
Sunny Day Failure Consequence Category (ANCOLD 2012)	High C (based on PLL)	GHD 2019
Incremental Wet Day Failure Consequence Category (ANCOLD 2012)	High A (based on PLL)	GHD 2019
Reservoir		
Type	On-stream storage	SMEC 2012
Full Supply Level (FSL)	RL 103.08 mAHD	SMEC 2012
Reduced Maximum Operating Level (MOL)	RL 98.85 mAHD (4.23 m below FSL, sill level of high level inlet) Restriction due to possible core leakage near FSL, and inadequate stability and flood handling capacity	MWC 2015 SMEC 2012
Minimum Operating Level	RL 90.86 mAHD	SMEC 2012 & MWC 2016 sketch plan
Minimum RL upstream of dam	RL 87 mAHD	GHD 2016
Total Storage Capacity at FSL	912 ML	SMEC 2012
Storage Capacity at restricted MOL	320 ML (revised by Jacobs, 2018)	Jacobs 2018
Catchment area	334 ha	SMEC 2012
Reservoir surface area at FSL	14.6 ha	SMEC 2012
Dam wall		
Type	Earthfill with (puddle) clay core and partial concrete cut-off	SMEC 2012
Crest level	RL 104.62 mAHD (nominal crest level) Sags by up to 0.6 m (to RL 104.02 mAHD)	MWC 2015
Crest length	174 m	SMEC 2012
Crest width	1.8 m	SMEC 2012
Normal freeboard	1.54 m (FSL to nominal crest level) 5.77 m / 5.17 m (restricted MOL to nominal crest level / restricted MOL to lowest crest level)	Re-calculated from MWC 2015
Embankment Height	24.0 m	SMEC 2012
Upstream slope	2H:1V (above FSL) 3H:1V (below FSL)	SMEC 2012
Downstream slope	2H:1V Berm at RL 93.0 mAHD (approx.)	SMEC 2012

Component	Description	Reference
Spillway		
Type	Ogee crest with concrete channel at left abutment (Note: High Level Outlet now acts as the primary spillway, and the left abutment spillway as a secondary spillway)	SMEC 2012
Crest level	RL 103.08 mAHD	SMEC 2012
Crest length	17.8 m	SMEC 2012
Capacity	25 m ³ /s (left abutment spillway only at lowest DCF) Approx. 30 m ³ /s (left abutment spillway & one 1050 mm outlet pipe at lowest DCF) Approx. 33 m ³ /s (left abutment spillway & two 1050 mm outlet pipes at lowest DCF)	GHD 2012
Dam Crest Flood AEP	Spillway only: 1 in 280,000 (to crest low) / 1 in 1,200,000 (to nominal crest level) Spillway & twin 1050 mm diameter outlet pipes: 1 in 700,000 (to crest low) / 1 in 2,000,000 (nominal crest level)	GHD 2012
Outlet works		
<u>High Level outlet works</u>		
Concrete tower & access deck	Constructed c. 1970 13.7 m (45 ft.) high, 4.6 m (15 ft.) wide, 2.3 m (7 ft. 5") deep concrete tower (inc. 0.75 m / 2.5 ft. thick base/foundation as per original design drawings). Foundation level on drawings approx. RL 97.0 mAHD (RL 320 ft.). Tower top grated platform at RL 104.6-104.8 mAHD (from survey). Walkway supported by four (4) asbestos cement pipe columns. Concrete sill at tower entrance controlling lowest drawdown level to RL 98.85 mAHD (RL 323 ft.). Invert level of twin 1050 mm (42") outlet pipes approx. RL 97.9 mAHD. Twin 1750 mm (69") wide inlet trash screens and opening into tower for full-face height of tower.	Drawings, photographs, survey
Concrete tower slide gates	Twin 1050 mm (42") slide gates (crank operated, left open for reduced FSL).	MWC 2015
High level outlet pipes	Twin 1050 mm dia. nominal (42") MSCL pipes from High Level Outlet Tower to Valve House. Pipe capacity nominally 4.2 m ³ /s for DCF head (GHD 2012)	MWC 2015 GHD 2012
Valve house	Twin 1050 mm diameter nominal (42") butterfly valves on twin outlet pipelines, with third 1050 mm diameter nominal (42") butterfly valve on cross connection within valve house. Eastern outlet pipeline blanked off downstream of butterfly valve. 1050 mm diameter RC pipeline downstream abandoned. Western outlet pipeline (1050 mm dia. MSCL) continues to riparian scour works; blanked off and abandoned beyond.	MWC 2015 MWC 2016 sketch plan

Component	Description	Reference
<u>Low Level outlet works & tunnel</u>	Original construction c. 1918 Converted to scour works c. 1988	
Upstream intake	Intake approx. RL 90.87 mAHD (RL 299.92 ft.)	
Concrete tower/shaft & access deck	Circular tower, 1700 mm outer diameter, 1250 mm inner diameter, 12.5 m internal depth (tower extending to embankment foundation). Located on upstream face of embankment, upstream of concrete core wall. Steel superstructure (installed c. 2012) incorporating access hatch and walkway deck.	Drawings MWC 2015 1994 survey
Penstock gate	500 mm nominal penstock gate at base of low-level outlet tower (installed c. 2012, replaced previous slide gate). Reportedly commissioned c. 2016.	MWC 2015
Low level outlet pipeline	18" nominal diameter (given as 450 mm or 500 mm nominal diameter in various sources) CI pipe within low-level culvert. (Original pipeline c. 1918)	MWC 2015
Low level outlet tunnel/ culvert	'Upside down' teardrop-shaped 'ovoid' culvert at foundation level through base of concrete core wall. Culvert dimensions 1.37 m (4 ft. 6") tall by 1.09 m (3 ft. 7") wide.	MWC 2015
Low level outlet isolation valve & valve pit	Outlet valve pit at downstream end of tunnel/culvert. 18" CI gate valve within downstream isolation valve pit. Downstream connection to 1050 mm RC old outlet pipeline abandoned. Downstream connection to 1050 mm MSCL outlet pipeline.	MWC 2015 MWC 2016 sketch plan
<u>Abandoned old high level outlet</u>	Decommissioned c. 1972	
Old high level intake pit	Decommissioned high-level outlet pit within reservoir near right abutment, intake RL 99.42 mAHD (RL 328 ft.), plugged with concrete.	Drawings (Dwg 113610C)
Old high level pipeline	Decommissioned 600 mm nominal diameter (24") concrete pipeline passing through right abutment foundation at RL 99.42 mAHD (RL 328 ft.), with concrete cut-off wall at embankment centreline. Current presence/demolition unknown.	Drawings (Dwg 113610C, Dwg 26210)
<u>Abandoned scour works</u>	Original construction c. 1918 Decommissioned c. 1988	
Original scour tower and conduit through embankment	Tower cut down flush with embankment, outlet conduit and base of old tower grouted c. 1988. Original 18" (450 mm) nominal diameter CI scour pipe upstream invert RL 85.94 mAHD (RL 283.75 ft.). Control point RL 86.0 mAHD. Pipe invert at concrete core wall RL 85.94 mAHD (RL 283.75 ft.). Concrete encased through embankment.	SMEC 2012 Drawings (Dwg 26206)
Downstream scour works	Original 225 mm nominal dia. scour works downstream of embankment decommissioned c. 2014.	MWC 2015

Component	Description	Reference
<u>Scour & riparian release</u>	Constructed c. 2014	
Outlet pipeline	450 mm nominal diameter MSCL pipeline tee off 1050 mm MSCL pipeline downstream of High Level Outlet Valve House. 1050 mm pipeline blanked off beyond tee, abandoned downstream.	MWC 2015
New DN450 scour valve	450 mm nominal diameter butterfly valve on scour through line. Valve left closed except during scour operation.	MWC 2015
New DN100 bypass valve	100 mm nominal diameter butterfly valve on 100 mm bypass line for environmental flows. Valve left open.	MWC 2015
Scour pipeline & riparian discharge	450 mm nominal diameter MSCL pipe from scour valves to riparian discharge flow diffuser and baffle, discharging into rock-lined channel into Haunted Gully Creek approx. 100 m downstream of embankment.	MWC 2015
Outlet works capacity	Reported capacity of 450 mm scour: 80 ML/day (not verified) Reported capacity of 100 mm environmental bypass: 8 ML/day (not verified)	MWC 2015c (DSEP data sheet)
<i>Inlet works</i>		
Bunyip Main Race channel	Abandoned inlet channel on east side of reservoir (adjacent to spillway). Channel penstock gates still in position.	MWC 2015
Cardinia 'Siphon' transfer pipeline	Inlet structure on west side of reservoir upstream no longer in use. Structure reportedly in 'satisfactory' condition c. 2012.	MWC 2011
Catch drain	Catch drain around reservoir (for previous water quality management purposes) broken out, abandoned.	MWC 2011
<i>Monitoring instrumentation</i>		
Reservoir water level gauge boards	Two (2) gauge boards: One mounted to High Level Outlet Tower eastern wall. Another on upstream batter between the High Level and Low Level outlet towers.	
Reservoir water level sensor	Commissioned c. 2015. Automatic electronic sensor with telemetry. Mounted on High Level Outlet Tower.	MWC 2015
Local rain gauge sensor	Commissioned c. 2015. Automatic electronic sensor with telemetry.	MWC 2015
Piezometers	Nine (9) standpipe piezometers on downstream batter and berm. Instruments WH090PIBA-P1 to P9. Six (6) installed c. 1988, three (3) installed c. 1999.	SMEC 2012

Component	Description	Reference
Survey markers	<p>Eleven (11) survey markers measuring settlement and movement offset.</p> <p>Five (5) located on crest, instruments WH090SUR-CS01 to CS05.</p> <p>Three (3) located on upper slope of downstream batter, instruments WH090SUR-ES06 to ES08.</p> <p>Three (3) located on downstream berm, instruments WH090SUR-ES09-ES11.</p>	SMEC 2012
Gauge board	Gauge Board (in poor condition) mounted on eastern (embankment) side of High Level Outlet Tower	

Appendix B – Drawing list

No.	Title/ Description	Drawing No.	Year	Authority
1	Beaconsfield Reservoir – General Layout	5016_20.1	1972	SR&WSC
2	Beaconsfield Reservoir – Capacity Chart	5016_20.2	-	SR&WSC
3	Beaconsfield Reservoir – Locality Plan	5016_20.3	1988	SR&WSC
4	Beaconsfield Reservoir – Capacity Chart	5016_20.4	-	SR&WSC
5	Beaconsfield Reservoir – Contour and Capacity Plan	5016_20.5	-	SR&WSC
6	Beaconsfield Reservoir – Contour Plan of Site of Dam	5016_20.6	-	SR&WSC
7	Beaconsfield Reservoir – Contour Plan	5016_20.7	-	SR&WSC
8	Beaconsfield Reservoir – Contour and Capacity Plan	5016_20.8	-	SR&WSC
9	Beaconsfield Reservoir – Clay Core Wall Depth Below Natural Surface	5016_20.9	1987	SR&WSC
10	Beaconsfield Reservoir – Sections of Dam	5016_21.1	-	SR&WSC
11	Beaconsfield Reservoir – Section Showing Fill Required to Raise Crest to RL 347.00	5016_21.2	1956	SR&WSC
12	Beaconsfield Reservoir – Locality and Cross Sections Outer Batter of Bank	5016_21.3	1986	SR&WSC
13	Beaconsfield Reservoir – Locality & Cross Sections	5016_21.4	-	SR&WSC
14	Beaconsfield Reservoir – Waste Weir & By Wash	5016_21.5	-	SR&WSC
15	Beaconsfield Reservoir – Contour and Inlet Channel	5016_24.1	-	SR&WSC
16	Beaconsfield Reservoir – Remodelled Outlet Works - Arrangement	5016_25.1	1969	SR&WSC
17	Beaconsfield Reservoir – Outlet	5016_25.2	-	SR&WSC
18	Beaconsfield Reservoir – Outlet & Scour Pipe	5016_25.3	-	SR&WSC
19	Beaconsfield Reservoir – Outlet Tunnel Lining	5016_25.4	-	SR&WSC
20	Beaconsfield Reservoir – Outlet Tunnel Lining	5016_25.5	-	SR&WSC
21	Beaconsfield Reservoir – Outlet Pit to Tunnel	5016_25.6	-	SR&WSC
22	Beaconsfield Reservoir – Longitudinal Section of Scour Pipe	5016_25.7	1954	SR&WSC
23	Beaconsfield Reservoir – Groundwater Boreholes	5016_3.1	1994	SR&WSC
24	Beaconsfield Reservoir – Outlet Tower – Critical Path Network	112923	1970	SR&WSC
25	Beaconsfield Reservoir – Fencing Around Outlet Tower	114252	1972	SR&WSC
26	Beaconsfield Reservoir – Fencing Around Outlet Tower	27750	1972	SR&WSC
27	Beaconsfield Reservoir – Outlet Tower Grill Floor	91296A	1973	SR&WSC
28	Beaconsfield Reservoir – 1.5 Ton Crane Class 2	91439	1978	SR&WSC

No.	Title/ Description	Drawing No.	Year	Authority
29	Beaconsfield Reservoir – Outlet Tower – Reinforcement Details Sheet 1	97543	1970	SR&WSC
30	Beaconsfield Reservoir – Outlet Tower – Reinforcement Details Sheet 2	97544C	1970	SR&WSC
31	Beaconsfield Reservoir – Outlet Works Remodelling Outlet Tower – Hoist Frame Details	97620A	1970	SR&WSC
32	Beaconsfield Reservoir – Outlet Tower Drop Bar Arrangement & Details	97628	1970	SR&WSC
33	Beaconsfield Reservoir – Construction Program	112922	1970	SR&WSC
34	Beaconsfield Reservoir – Outlet Works - Feature Survey	94945	1968	SR&WSC
35	Beaconsfield Reservoir – Outlet Works Long Section of 42_Dia.	97441	1970	SR&WSC
36	Beaconsfield Reservoir – Outlet Works	97443	1970	SR&WSC
37	Beaconsfield Reservoir – Outlet Works 42_dia. Pipeline – 24 Dia. Offtake	97444	1970	SR&WSC
38	Beaconsfield Reservoir – Outlet Works - 42_ Dia. Conduit 3_ Dia. And 6_ Dia. Offtakes	97445	1970	SR&WSC
39	Beaconsfield Reservoir – Outlet Works 42_ Dia. Pipeline Dismantling Joint	97446	1970	SR&WSC
40	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline Arrangement	97520	1970	SR&WSC
41	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97521	1970	SR&WSC
42	Beaconsfield Reservoir – Outlet Works 42_dia. Pipeline-Barrel Mk No.2	97522	1970	SR&WSC
43	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97523	1970	SR&WSC
44	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97524	1970	SR&WSC
45	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97525	1970	SR&WSC
46	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97527	1970	SR&WSC
47	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97529	1970	SR&WSC
48	Beaconsfield Reservoir – Outlet Works Remodelling Valve House	97530	1970	SR&WSC
49	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97531	1970	SR&WSC
50	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97532	1970	SR&WSC
51	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97533	1970	SR&WSC

No.	Title/ Description	Drawing No.	Year	Authority
52	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97534	1970	SR&WSC
53	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97535	1970	SR&WSC
54	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97536	1970	SR&WSC
55	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97537	1970	SR&WSC
56	Beaconsfield Reservoir – Outlet Works Remodelling Faucet - Lead To Existing Pipe	97538	1970	SR&WSC
57	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97539	1970	SR&WSC
58	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline	97540	1970	SR&WSC
59	Beaconsfield Reservoir – Outlet Works Remodelling Connection To Existing Pipeline	97541	1970	SR&WSC
60	Beaconsfield Reservoir – Outlet Works Remodelling 42_Dia. Pipeline Thrust Block, Cut-off Wall	97542	1970	SR&WSC
61	Beaconsfield Reservoir – Outlet Works Remodelling - Valve House	97548	1970	SR&WSC
62	Beaconsfield Reservoir – Outlet Works Valve House Handrail Details	97624	1970	SR&WSC
63	Beaconsfield Reservoir – Outlet Works Trash Screens - Arrangement & Details	97625	1970	SR&WSC
64	Beaconsfield Reservoir – Outlet Works Removable PC Planks	97626	1970	SR&WSC
65	Beaconsfield Reservoir – Outlet Works Remodelling Footbridge Deck Planks	97627	1970	SR&WSC
66	Beaconsfield Reservoir – Outlet Works Remodelling Valve House Removable Roof Details	97629	1970	SR&WSC
67	Beaconsfield Reservoir – Outlet Works - Valve House – A-Frame Arrangement 2 Ton Capacity	98120	1971	SR&WSC
68	Beaconsfield Reservoir – Outlet Works - Valve House Grid Flooring & Access Ladder RI 305-5	98121	1971	SR&WSC
69	Beaconsfield Reservoir – Outlet Works - Valve House Concrete Apron	98122	1971	SR&WSC
70	Beaconsfield Reservoir – Spillway – Erosion Control	113559	1971	SR&WSC
71	Beaconsfield Reservoir –Showing Capacity	110132	-	SR&WSC
72	Beaconsfield Reservoir – Link Up At High Level Outlet At Beaconsfield Reservoir	110543	1957	SR&WSC
73	Beaconsfield Reservoir – Outlet	110575	-	SR&WSC

No.	Title/ Description	Drawing No.	Year	Authority
74	Beaconsfield Reservoir – Locality Plan	111851	1965	SR&WSC
75	Beaconsfield Reservoir – Inlet Pipe Conc. Pipe Supports	112438	1968	SR&WSC
76	Beaconsfield Reservoir – Longitudinal Section	112439	1968	SR&WSC
77	Beaconsfield Reservoir – Longitudinal Section	112440	1968	SR&WSC
78	Beaconsfield Reservoir – Locality Plan	112441	1968	SR&WSC
79	Beaconsfield Reservoir – Graph Of Flow Upstream Of Beaconsfield Reservoir	112533	1968	SR&WSC
80	Beaconsfield Reservoir – Graph Of Flow Upstream Of Beaconsfield Reservoir	112534	1968	SR&WSC
81	Beaconsfield Reservoir – Drainage Easement CA 142	112677	1969	SR&WSC
82	Beaconsfield Reservoir – Feature Survey - West End Of Bank Street	112895	1969	SR&WSC
83	Beaconsfield Reservoir – Feature Survey - West End Of Bank Street	112896	1969	SR&WSC
84	Beaconsfield Reservoir – Western Access Road - Locality Plan	112973	1970	SR&WSC
85	Beaconsfield Reservoir – Western Access Road - Longitudinal & Cross Sections	112974	1970	SR&WSC
86	Beaconsfield Reservoir – Capacity Table	113035	-	SR&WSC
87	Beaconsfield Reservoir – Water Levels Gauge	113129	1970	SR&WSC
88	Beaconsfield Reservoir – Valve House Pit - Beaconsfield Reservoir	113148	1971	SR&WSC
89	Beaconsfield Reservoir – General Layout	113610	1971	SR&WSC
90	Beaconsfield Reservoir – Cardinia Syphon Offtake	113701	-	SR&WSC
91	Beaconsfield Reservoir – Contents Chart	114087	1972	SR&WSC
92	Beaconsfield Reservoir – Contents Chart	114672	1977	SR&WSC
93	Beaconsfield Reservoir – Western Outlet - Steel Grating Pit Covers	115289	1976	SR&WSC
94	Beaconsfield Reservoir – Contour & Capacity Plan	115914	-	SR&WSC
95	Beaconsfield Reservoir – Western Outlet, Screen Cleaning Arrangement	116037	1980	SR&WSC
96	Beaconsfield Reservoir – Screen Cleansing Arrangement MS Grate	116086	1980	SR&WSC
97	Beaconsfield Reservoir – Locality Sections (Outer Batter)	117172	1986	SR&WSC
98	Beaconsfield Reservoir – (Long Section) Clay Core Wall Depth Below Natural Surface	117627	1987	SR&WSC
99	Beaconsfield Reservoir – Range Road Chlorinator Building - Chain Hoist	117977	1988	SR&WSC

No.	Title/ Description	Drawing No.	Year	Authority
100	Beaconsfield Reservoir – Feature Record	118989	1991	SR&WSC
101	Beaconsfield Reservoir – Widening Of Bridge (Access)	136099	1983	SR&WSC
102	Beaconsfield Reservoir – Contour Plan	19633	-	SR&WSC
103	Beaconsfield Reservoir – Locality Plan Showing FSL	19634	-	SR&WSC
104	Beaconsfield Reservoir – Contours	25059	1942	SR&WSC
105	Beaconsfield Reservoir – Sections Of Dam	26010	1917	SR&WSC
106	Beaconsfield Reservoir – Outlet & Scour Pipes	26201	-	SR&WSC
107	Beaconsfield Reservoir – Outlet Tunnel Lining	26202	1954	SR&WSC
108	Beaconsfield Reservoir – Outlet Pit To Tunnel	26203	-	SR&WSC
109	Beaconsfield Reservoir – Section Showing Fill Required To Raise Crest To RL347	26205	1954	SR&WSC
110	Beaconsfield Reservoir – Long. Section Of Scour Pipe	26206	1954	SR&WSC
111	Beaconsfield Reservoir – Lifting Gear - Outlet	26207	-	SR&WSC
112	Beaconsfield Reservoir – Lifting Gear - Scour Gates	26208	1954	SR&WSC
113	Beaconsfield Reservoir – Contour Plan of Site Of Dam	26209	-	SR&WSC
114	Beaconsfield Reservoir – Offtake At RL 328	26210	-	SR&WSC
115	Beaconsfield Reservoir – Scour & Offtake	26211	1917	SR&WSC
116	Beaconsfield Reservoir – Waste Weir & By wash	26212	-	SR&WSC
117	Beaconsfield Reservoir – Contour Inlet Channel	26213	-	SR&WSC
118	Beaconsfield Reservoir – Additional Grids	26228	1954	SR&WSC
119	Beaconsfield Reservoir – Discharge Curve - Main Race	26232	1923	SR&WSC
120	Beaconsfield Reservoir – Discharge Of Main Race	26233	1923	SR&WSC
121	Beaconsfield Reservoir – Contours Around Haunted Gully	26234	1954	SR&WSC
122	Beaconsfield Reservoir – Embankment Site - Haunted Gully	26238	1954	SR&WSC
123	Beaconsfield Reservoir – Details Of Trash And Crest Angle For Inlet Measuring Weir	27334	1968	SR&WSC
124	Beaconsfield Reservoir – 42_ CLMS Field Welding Details	27510	-	SR&WSC
125	Beaconsfield Reservoir – Proposed Enlargement Of Inlet Works	27624	-	SR&WSC
126	Beaconsfield Reservoir – Inlet Control Structure & Measuring Weir	27630	1968	SR&WSC

No.	Title/ Description	Drawing No.	Year	Authority
127	Beaconsfield Reservoir – Inlet Control Structure Gate Support Beams	27631	1968	SR&WSC
128	Beaconsfield Reservoir – Outlet Connections To New Supply Main (24_Dia.)	29044	1965	SR&WSC
129	Beaconsfield Reservoir – Connection To New 42_Dia. RC Pipe Details Of 45dg Branch	29051	-	SR&WSC
130	Beaconsfield Reservoir – Beaconsfield-Langwarrin Pipe Line	29061	-	SR&WSC
131	Beaconsfield Reservoir – Commission Occupied Land	77404	1966	SR&WSC
132	Beaconsfield Reservoir – West Outlet Temporary Pumps Wiring Diagram	91387	1973	SR&WSC
133	Beaconsfield Reservoir – West Outlet Temporary Panel & Elec. Layout	91388	1973	SR&WSC
134	Beaconsfield Reservoir – Cranbourne Pipelines No. 2 & No. 3 Automatic Chlorinator Building	92715	1973	SR&WSC
135	Beaconsfield Reservoir – Cranbourne Pipelines No. 2 & No.3 Automatic Chlorinator Monorail Steel	92716	1979	SR&WSC
136	Beaconsfield Reservoir – Cranbourne Pipelines No. 2 & No. 3 Automatic Chlorinator Building	92717	1979	SR&WSC
137	Beaconsfield Reservoir – Feature Survey At Exit Tunnel - July 1968	94948	1968	SR&WSC
138	Beaconsfield Reservoir – Feature Survey West End Of Bank	95293	1969	SR&WSC
139	Beaconsfield Reservoir – Tower Access Bridge	97545	1970	SR&WSC
140	Beaconsfield Reservoir – 42_Dia. Pipeline - Blank Flange For 24_Dia. Offtake	97546	1970	SR&WSC
141	Beaconsfield Reservoir – 42_Dia. Pipeline - Blank Flange	97547	1970	SR&WSC
142	Beaconsfield Reservoir – Valve House Arrangement Of Grid Flooring At RL 306.0ft	97621	1970	SR&WSC
143	Beaconsfield Reservoir – Valve House - Grid Flooring	97622	1970	SR&WSC
144	Beaconsfield Reservoir – Valve House - Access Ladder	97623	1970	SR&WSC
145	Beaconsfield Reservoir – Brick Valve House	99291	1971	SR&WSC
146	Beaconsfield Reservoir – Mits Telemetry Terminal Connections Wiring Diagram	WD016_002	1999	SR&WSC
147	Beaconsfield Reservoir – Detail Survey	WD016_031	1994	SR&WSC
148	Beaconsfield Reservoir – Beaconsfield Reservoir, Detail Survey Enlargement Plan	WD016_032	1995	SR&WSC
149	Beaconsfield Reservoir – Detail Survey	WD016_033	1995	SR&WSC

Appendix C – Hydrology

C.1 Existing hydrology

C.1.1 ARR 2016 revision of existing hydrology

The hydrology for Beaconsfield Reservoir was updated to be in line with the revised Australian Rainfall and Runoff guidelines (ARR, 2016) using an ensemble approach (taking the representative result of ten temporal patterns for each duration). The stage storage relationship was also updated based on bathymetric survey undertaken in 2017. The RORB layout for the dam catchment is shown in Figure C-1 below.

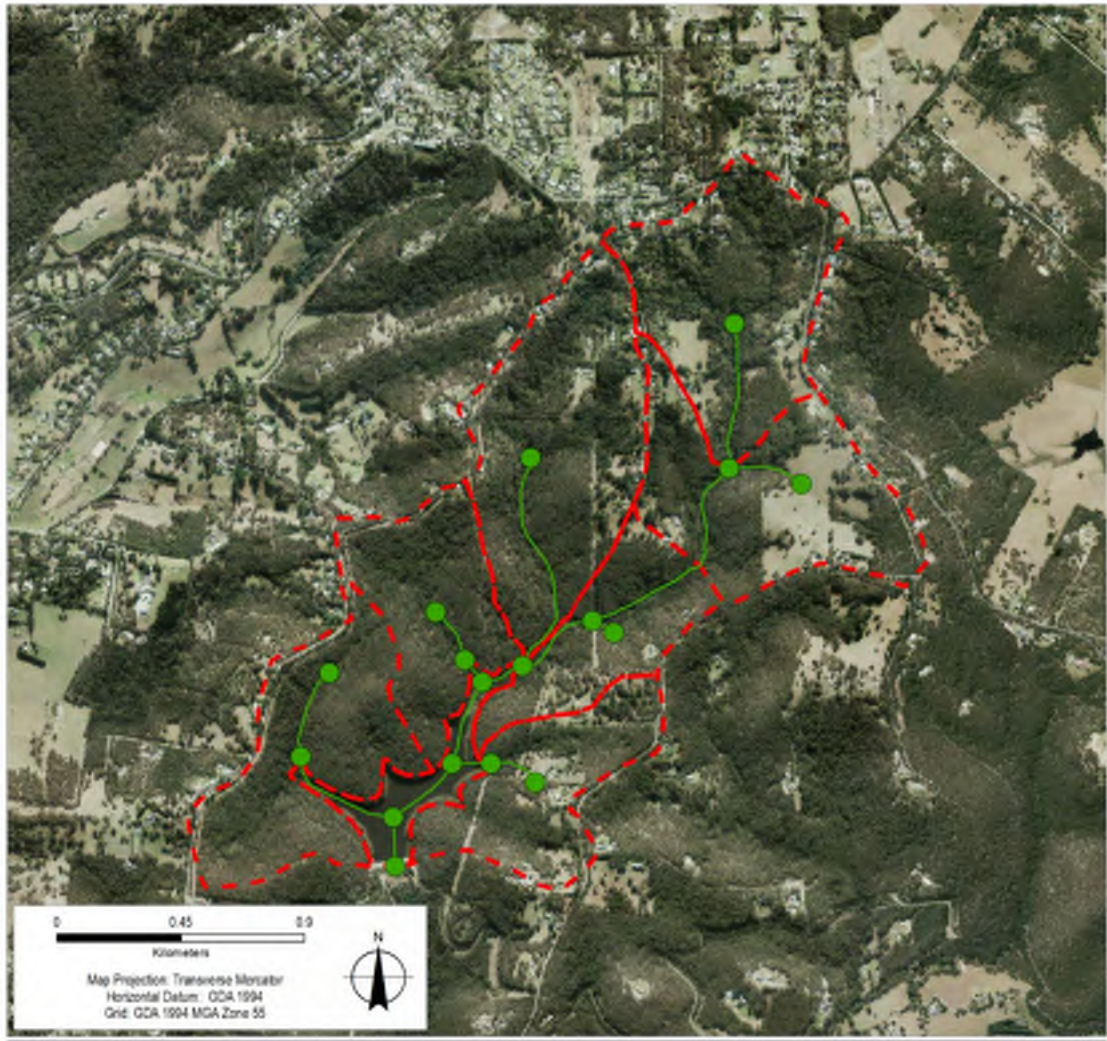


Figure C-1 Dam catchment RORB layout

C.1.2 Regional kc estimates

A number of different empirical equations exist for estimating an appropriate storage routing parameter (which affects the amount of attenuation when routing through a reach). Some of the equations commonly used in the Melbourne area are given in Table C-1.

The Beaconsfield Reservoir catchment area is 3.3 km², and the d_{av} (average flow distance in the channel network of all the sub-area inflows) for the reservoir catchment is 1.62 km. The Mean Annual Rainfall (MAR) for the Beaconsfield Reservoir catchment is in excess of 800 mm, according to the Bureau of Meteorology gridded mean annual rainfall data.

Table C-1 kc equations and values for Beaconsfield Reservoir

Equation	Source	kc value for Beaconsfield Reservoir
$k_c = 0.49 \times A^{0.65}$ (MAR < 800 mm)	Australian Rainfall and Runoff	1.06
$k_c = 2.57 \times A^{0.45}$ (MAR > 800 mm)	Australian Rainfall and Runoff	4.40
$k_c = 2.2 \times A^{0.5}$	RORB Manual	4.00
$k_c = 1.25 \times d_{av}$	Pierce et al. (2010)	2.03
$k_c = 1.53 \times A^{0.55}$ (South East ("DVA") area)	MWC	2.95
$k_c = 1.19 \times A^{0.56}$ (Melbourne Metropolitan Board of Works area)	MWC	2.32

For Beaconsfield Reservoir, the applicable regional equation given in ARR (2016) is the second equation, which yields a suggested k_c value of 4.4. Sensitivity analysis on the k_c value is shown as part of the validation and verification process in Section C.1.4.

C.1.3 Losses

Rainfall losses were downloaded from the ARR Data Hub. These are shown in Table C-2 below, along with the previously adopted values for comparison.

Table C-2 Data Hub losses for Beaconsfield Reservoir

Loss type	Data Hub Value	Previous Cardinia Reservoir calibration values (using ARR, 1987)
Storm initial loss (IL _s)	25	26.3
Continuing loss (CL)	4.4	2.6

The storm initial loss is slightly lower than previously adopted. The way in which the burst loss is calculated has changed, with median pre-burst depths now available from the data hub.

Prior to ARR (2016), losses were derived in accordance with the equations from the Cooperative Research Centre for Catchment Hydrology (CRCCH, 1996), using a base flow index (BFI) as shown below:

$$IL_s = -25.8 \text{ BFI} + 33.8$$

$$CL = 7.97 \text{ BFI} + 0.00659 \text{ PET} - 6.0$$

The data hub pre-burst depth is now subtracted from the storm loss to calculate the burst loss applicable to the AEP and duration for ensemble modelling. The data hub continuing loss value is larger than obtained in the Cardinia Reservoir calibration previously, using ARR (1987) methodologies.

C.1.4 Verification

For a given catchment within RORB, the hydrograph peak, shape and volume are influenced by both the k_c value and rainfall losses. The k_c value is a non-linear storage routing factor applied to reaches. The larger the k_c value the higher the attenuation. Losses affect the peak of the hydrograph, and the volume of the hydrograph, with larger losses reducing the volume.

Where there is simultaneous rainfall and streamflow gauging, both the k_c value and the losses may be adjusted to provide a good match between the modelled and observed hydrographs.

Catchment files

A number of different catchment files were run using the ARR (2016) IFD, the data hub losses, median pre-burst depths and ensemble temporal patterns. The catchment files and purpose are summarised in Table C-3 below.

Table C-3 Summary of catchment files and purpose

Catchment file	Purpose	Comment
Natural (i.e. no reservoir)	For comparison to Regional Flood Frequency Estimation	Validation for ungauged catchment. There is no rainfall or streamflow gauge within catchment.
Reservoir catchment model (at original FSL)	Estimating existing flows	N/A
Reservoir at original FSL (model extended to Princes Highway)	Verifying against flood frequency at Officer gauge	Low impervious fractions, representative of conditions for most of the gauge record. No rainfall gauge within catchment. Rainfall gauge is located at same location as streamflow gauge.
MWC Developer Services planning model	Comparison of flows used for planning purposes, including verifying against flood frequency at Officer gauge	Represents almost full development of area between Browns Road and Princes Highway. Stage storage and storage discharge curve for reservoir updated (to correctly represent original FSL).

Gauge data

The nearest gauge is located on Gum Scrub Creek at the Princes Highway in Officer (228365A). Beaconsfield Reservoir is located within the catchment of the Officer gauge. Key information on the gauge is summarised in Table C-4 following.

Table C-4 – Officer Gauge – key information

Parameter	Value	Comment
Period of record	26 Jun 1980 - present	Significant development has occurred in parts of the catchment during the gauging period. The creek profile downstream of the gauge has also changed recently.
Catchment size	41 km ²	At 3.3 km ² , the Beaconsfield Reservoir catchment is a small proportion of this catchment.
Largest actual gauging	6.2 m ³ /s	Occurred 29/07/1987 (prior to restricted FSL). Catchment would have been almost entirely rural in those days.
Largest estimated flow		Occurred Converted from level to flow using rating curve.
BoM Flood frequency 1% AEP estimate	18-23	LP3/ GEV-L moments

Flood frequency analysis was undertaken on the annual maximum series using the FLIKE analysis package. FLIKE offers fitting using a number of different distributions. Of these, the LP3 (no prior information) and Gumbel distributions appeared to have the best “goodness of fit” as data points were not outside of the confidence limits, and the confidence limits were narrowest. The FLIKE plots are included below.

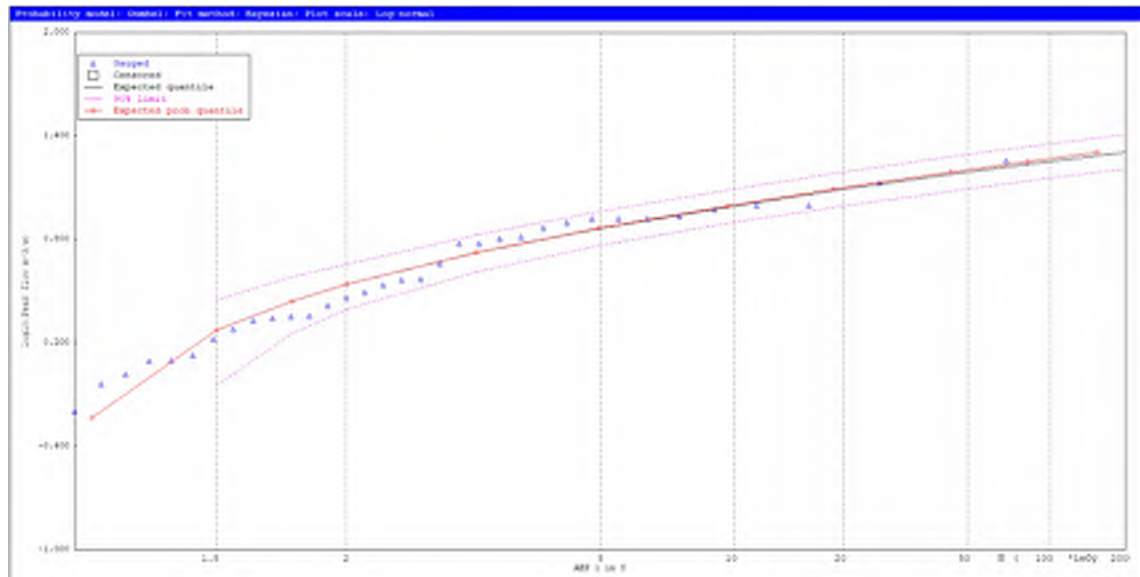


Figure C-2 Gumbel probability model on Log normal scale

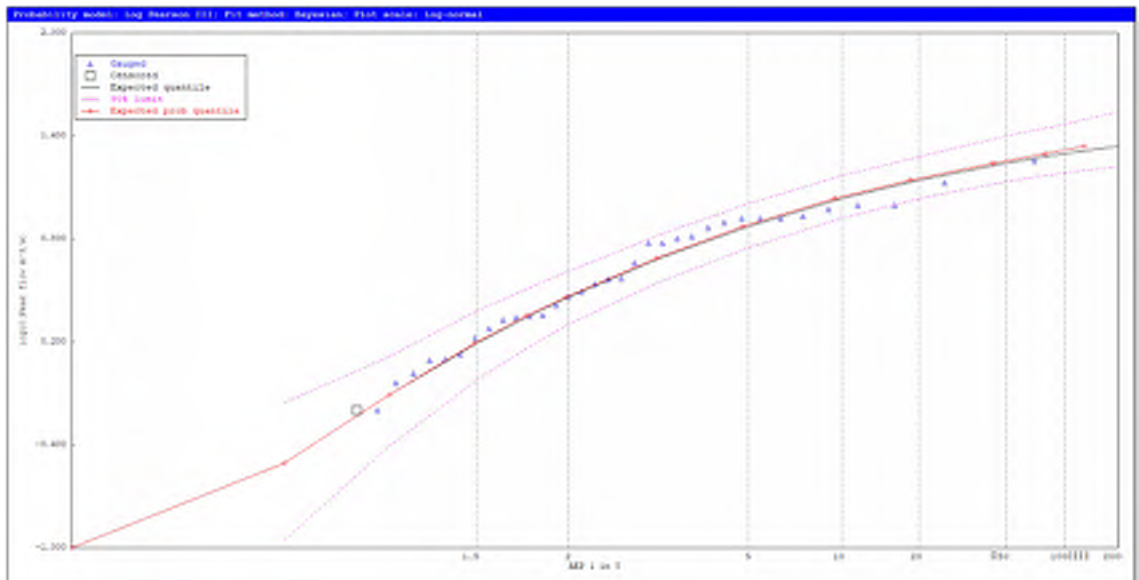


Figure C-3 LP3 no prior info probability model on Log normal scale

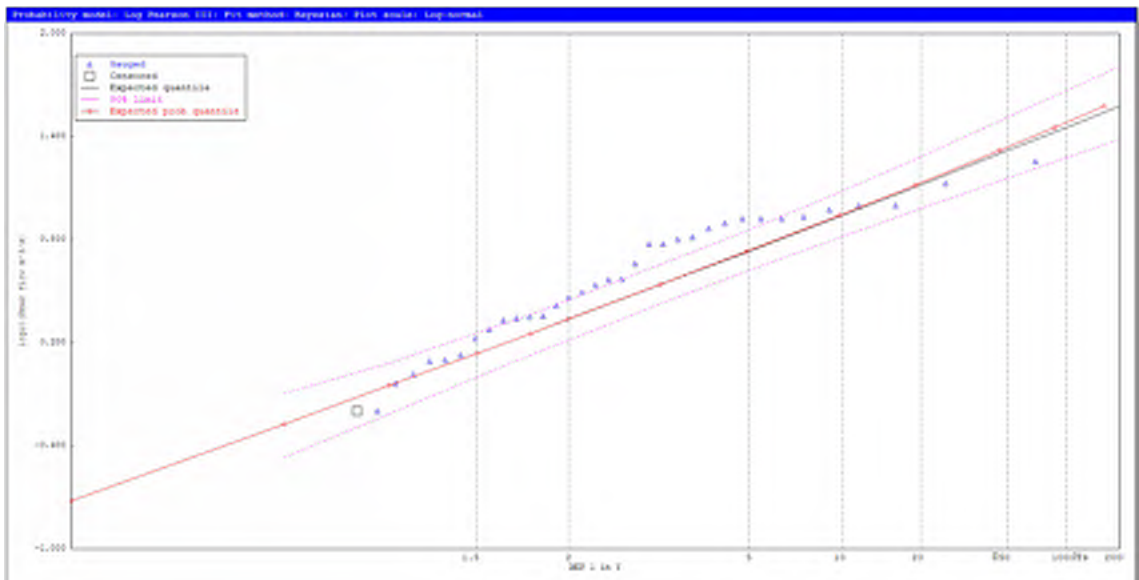


Figure C-4 LP3 regional probability model on Log normal scale

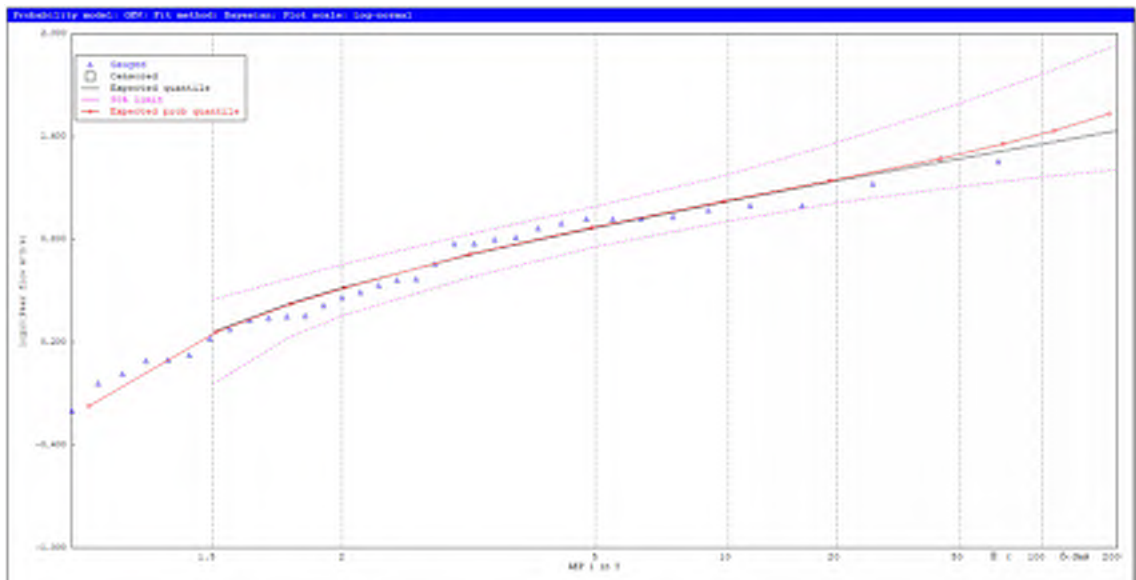


Figure C-5 GEV probability model on log normal scale

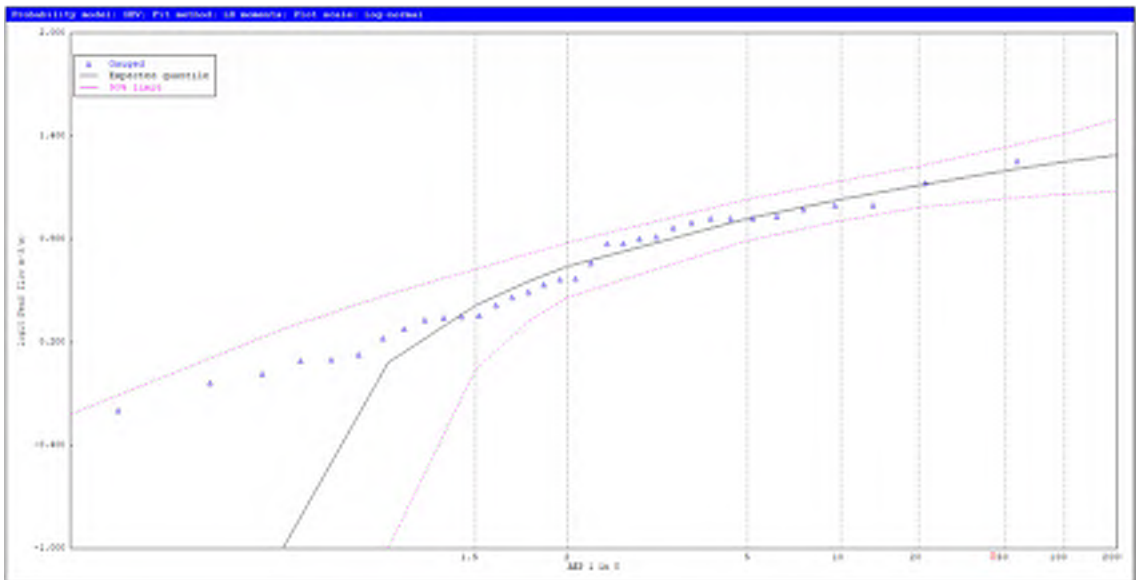


Figure C-6 GEV probability model with optimised LH moments on log normal scale

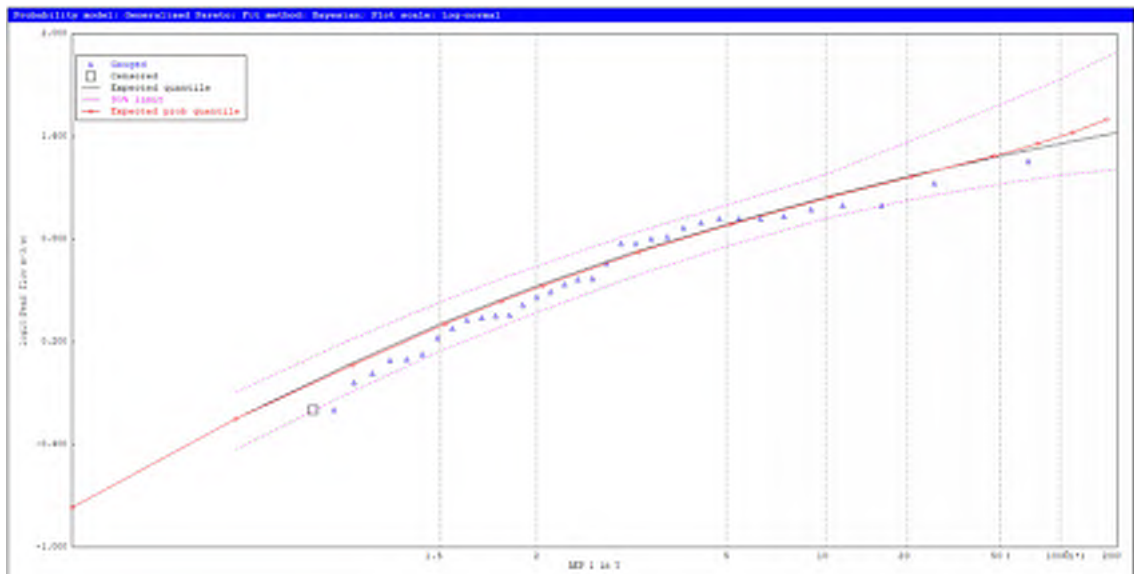


Figure C-7 Generalised Pareto probability model on Log normal scale

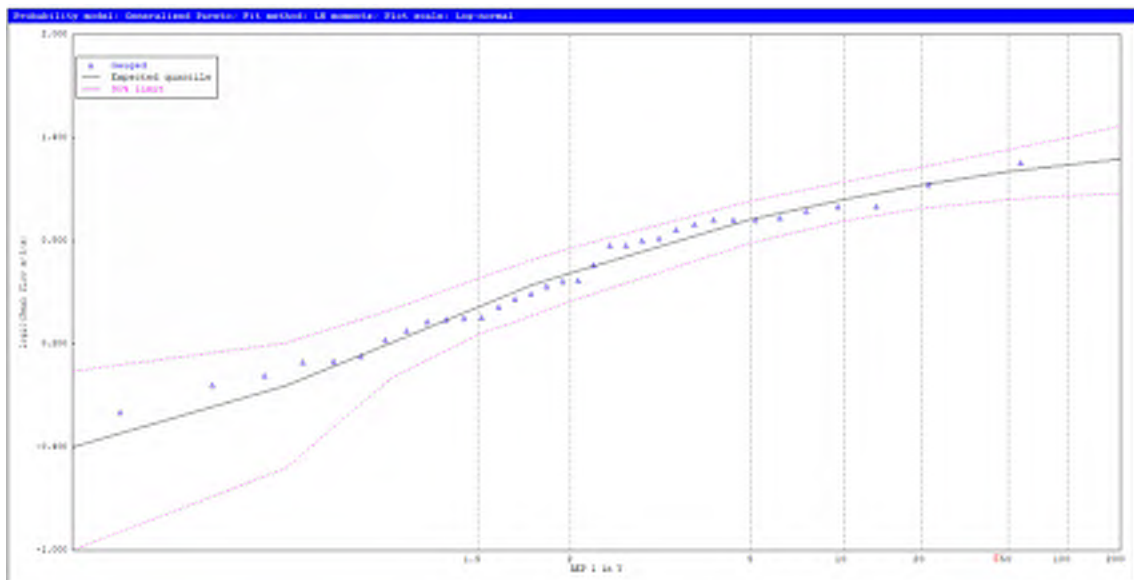


Figure C-8 Generalised Pareto probability model with optimised LH moments on Log normal scale

An extended RORB model for the larger catchment area to the Princes Highway was compiled and undeveloped fraction impervious values were applied (accounting for conditions throughout most of the gauge record). The Areal Reduction Factor (ARF) applied was for the catchment area upstream of the Princes Highway, and a k_c value, which maintains the k_c/d_{av} ratio of the reservoir catchment only model, was adopted.

With reference to Figure C-9 following, the 1 in 100 AEP peak design flows derived from ARR (2016) ensembles, and the data hub losses accounting for median pre-burst, are within the ranges of the gauge Flood Frequency Analysis, being slightly above the expected quantile for LP3 (no prior information) and Gumbel fits.

The flood frequency analysis results support both the k_c and loss values adopted for the reservoir catchment.

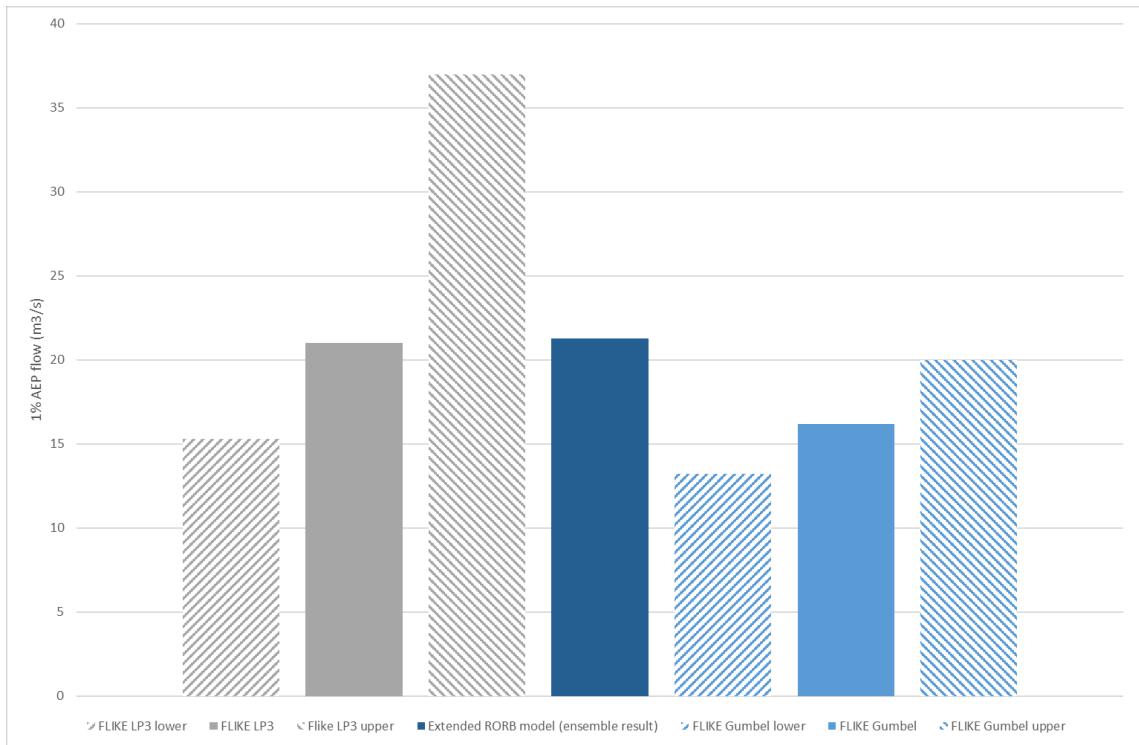


Figure C-9 FLIKE and RORB model results for Gum Scrub Creek at Officer

Regional Flood Frequency Estimation (RFFE)

As part of the ARR revision, a Regional Flood Frequency Estimate software tool has been developed to estimate flows for ungauged catchments. The software to infer flow estimates from nearby gauged catchments uses the location of the catchment centroid, catchment outlet location and catchment size. The Rational Method is no longer recommended for estimating peak flows from ungauged catchments.

The natural catchment RORB model was simulated for the 1 in 100 AEP using an ensemble simulation with data hub losses and median pre-burst depths; the resulting peak flows for the various k_c equations are compared in Figure C-10 following. Both the RORB equation and $MAR > 800$ mm k_c values provide a good match to the RFFE expected quantile using the data hub losses in an ensemble approach.

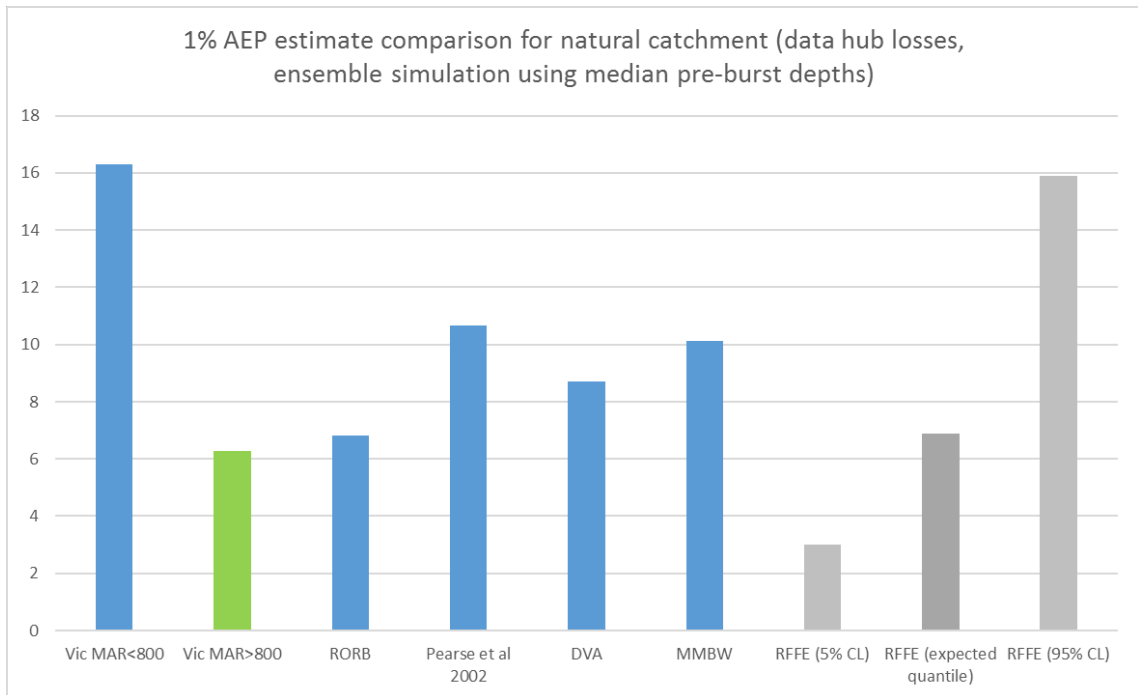


Figure C-10 Peak flow comparison using various equations for k_c

A sensitivity analysis was also undertaken by varying the continuing loss value for a k_c value of 4.4. The results of the sensitivity analysis are presented in Table C-5

Table C-5 RFFE and ensemble 1% AEP design flow comparison

Estimate	1% AEP flow (m³/s)
RFFE 5% confidence limit	3.0
RFFE expected quantile	6.9
Natural RORB model ensemble result (no drowned reaches) with temporal pattern filtering using $k_c=4.4$	6.3
Natural RORB model ensemble result (no drowned reaches) CL=2.6 mm/hr with temporal pattern filtering using $k_c=4.4$	7.7
Natural RORB model ensemble result (no drowned reaches) CL =1.8 mm/hr with temporal pattern filtering using $k_c=4.4$	8.4
RFFE 95% confidence limit	15.9

For Beaconsfield Reservoir, there is not sufficient direct gauge data for calibration, however, validation against both the RFFE for the reservoir catchment without a dam, and the Gum Scrub Creek catchment to the Princes Highway at Officer, suggests that the k_c value from the > 800 mm MAR regional equation, and the data hub losses and pre-burst are appropriate.

No adjustment has been made to the recommended k_c (from the applicable regional equation in ARR) or data hub loss values on the basis that:

1. The reliability of the RFFE is not always high.
2. Peak flows can be adjusted in a variety of ways which affect the volume of the hydrograph and bias the routing results without sound justification.
3. Adjusting loss values to match the RFFE may be contrary to concern that (in some areas at least) data hub loss values are already creating an underestimation bias.

Published on 12/02/2019, “Review of ARR Design Inputs for NSW” prepared by WMAWater for the NSW Office of Environment and Heritage, suggests that the losses are too high for NSW, and make a number of recommendations. Whilst based on NSW data, investigation of Victorian data would be required before disregarding the findings, and many of the fundamentals may still be of relevance. Summarised from the Executive Summary of ‘Review of ARR Design Inputs for NSW’, the recommended hierarchical approach to loss selection is (pending further research and advice that is more definitive):

1. Use the average of calibration losses from the actual study if available
2. Use the average calibration losses from other studies in the catchment if available and appropriate for the study
3. Use the average calibration losses from other studies in the similar adjacent catchments if available and appropriate for the study
4. Use FFA-Reconciled Losses for nearby similar sites (data is provided for NSW). Additional scrutiny should be applied to initial loss values for catchments of 100 km² or less, and
5. Until revised losses are generated using a better predictor equation, based on NSW data, WMAWater suggests applying a multiplication factor of 0.4 to the raw data hub values. Use the unmodified ARR data hub initial losses, and apply additional scrutiny to them for catchment areas of 100 km² or less, to ensure they are representative for the catchment

The WMAWater work calculated probability neutral burst initial loss values to be used in all instances where good local initial loss data is not available (Cases 4 and 5), unless a detailed Monte Carlo assessment of pre-burst and losses has been carried out.

The final RORB parameters adopted are summarised in Table C-6 below.

Table C-6 Adopted RORB parameters

Parameter	Value
m	0.8
k _c	4.4
Storm initial loss (mm)	25
Continuing loss (mm/hr)	4.4

Appendix D – Dambreak

D.1 Breach modelling

D.1.1 General

Breach parameters were estimated using a variety of empirical equations and simulated in 'FLDWAV' hydraulic modelling software to generate breach hydrographs.

Parameters which can be varied and that will affect the breach outflows include:

- Time for breach development
- Height of breach
- Width of breach
- Breach side slopes
- Mode of breaching (piping or overtopping)

Characteristics such as the volume and profile of the storage and the head (height of water in the storage above the downstream level or whether the breach is submerged) also affect the outflow.

Piping breaches along the foundation were assumed (centreline of piping breach and minimum breach level RL 87 mAHD). Consistent with previous assumptions based on poor compaction, a breach side slope of 1.0V:0.2H was adopted, unless a specific side slope is given for an estimation method.

D.1.2 Estimation of breach parameters

Various empirical equations have been derived using regression analysis on data from historical failures. Given the fact that there is significant scatter observed in the historical breaches, these equations are all different approximations of the most likely breach parameters and flows. An actual breach may fall either side of these estimates (larger or smaller), although some methods have better prediction accuracy than others (Wahl, 2004, Pierce et al., 2010).

"Dam Break Mechanisms" (Allen, 1994) recommends that the breach time and breach size be estimated in accordance with MacDonald Langridge-Monopolis relationships, which consider the volume of material in the embankment eroded to form a breach. A sensitivity analysis is also recommended.

As suggested in ANCOLD Bulletin 97 (Allen, 1994), minimum breach base widths conforming to the Singh and Scarlatos (1988) relationships were also considered.

The breach parameters predicted by the various empirical equations described below are summarised for each scenario in Table D-1 to Table D-3 following. Notes related to all three tables are provided at the end of Table D-6.

Wahl (2004) provides separate equations for earthfill and rockfill embankments based on MacDonald Langridge-Monopolis relationships. The following equations show the relationship as cited in Wahl.

Equation D-1 MacDonald Langridge-Monopolis (Wahl, 2004)

$$V_{er} = 0.0261(V_w \times h_w)^{0.769}$$

$$tf = 0.0261V_{er}^{0.364}$$

Where V_{er} the volume of material eroded from the embankment in cubic metres
 V_w the volume of water in the storage in cubic metres
 h_w the head of water in metres

Side slopes of 1H:2V could be assumed in most cases according to Wahl (2004).

There are also a number of other empirical equations for estimating breach time and size, which have been used for sensitivity analysis, as shown in Table D-1.

Table D-1 Empirical equations for breach parameters

Method	Equation for breach size	Equation for breach development time (time to fully form)	Side slope recommendations
Bureau of Reclamation	$B_{ave} = 3 \times h_w$	$t_r = 0.011 \times B_{ave}$	1H:1V
Von Thun and Gillette (erosion resistant)	$B_{ave} = 2.5 \times h_w + C_b$	$t_r = \frac{B_{ave}}{4.0 \times H_w}$	1H:1V (unless with cohesive shell or very wide cohesive core where 1H:2V or 1H:3V could be more appropriate)
Froehlich (2008)	$B_{av}=0.27K_0V_w^{0.32}H_b^{0.04}$	$t_r = 0.01756 \sqrt{\frac{V_w}{gh_b^2}}$	1H:1V overtopping 0.7H: 1V otherwise

Table D-2 contains bounds for breach development time and width, developed by Singh and Scarlatos (1988) using historical dam failure data.

Table D-2 – Singh and Scarlatos recommended bounds

Parameter	Suggested minimum	Suggested maximum
Breach top width/breach base width: B/d	1.06	1.74
Breach top width/breach depth: B/d	0.84	10.93
Breach angle:	10	50

The resulting minimum and maximum breach sizes, which conform to the Singh and Scarlatos geometry bounds, are summarised below in Table D-3.

Table D-3 Minimum and maximum base widths conforming to Singh and Scarlatos geometry bounds

Crest level (m AHD)	Breach height 'd' (m)	Assumed side slope	Minimum base width based on Singh and Scarlatos (m) based on B/b=1.74	Maximum base width based on Singh and Scarlatos (m) based on top width 10.93 x d
97	10	0.2	5.4	105.3
96.1	9.1	0.2	4.9	95.8

The author of FLDWAV, Fread recommends breaches be calibrated to the following equation:

Equation D-2 Fread (1981) peak flow equation

$$Q = B_r \times \left(\frac{C}{Tf + \frac{C}{H^{0.5}}} \right)^3$$

- Where Q is the breach flow in cubic feet per second (ft³/s)
B_r is the average final breach width in feet (1H to 5H)
- And C equals 23.4 * (A_s / B_r)
- Where A is the reservoir surface area at failure elevation in acres
H is the failure depth above breach elevation in feet
Tf is the time of failure (H/120) hours (minimum 10 min (0.17 hours))

Froehlich (1995) also produced an empirical equation for peak breach outflow, which produces a good fit to many of the available case studies (Pierce et al., 2010).

Equation D-3 Froehlich (1995) peak flow equation

$$Q=0.607*(V_w^{0.295*} \times H_w^{1.24})$$

- Where Q is the breach flow (m³/s)
V_w is the volume of water in m³
- And H_w is the height of the water in m

The work of Pierce et al. (2010) for the US National Dam Safety Review Board Steering Committee on Dam Breach Equations, suggests an empirical equation based on volume, head, and crest length may provide a very high correlation of predicted to measured peak outflow. This was derived by multi-variable regression analysis of historical failure data and has a quoted R² of 0.919 – 0.99 depending on the dataset used.

This is shown in Equation D-4 below.

Equation D-4 Pierce et al. (2010)

$$Q_p = 0.012 \times V_w^{0.493} \times H_w^{1.205} \times L^{0.226}$$

- Where V_w is volume in cubic metres
H_w is the height of water behind the dam in metres
L is the embankment length in metres

Table D-4 Breach parameters from empirical equations for FSL = RL 93.0 mAHD

Empirical equation	Side slope	Sunny day base width (m)	Sunny day breach development time (min)
MacDonald Langridge Monopolis (Wahl) ¹	0.2	-0.6	10
Bureau of Reclamation	0.2	16	12
Froehlich (2008) ²	0.7	3.2	8
Von Thun Gillette (1990)	1	11.1	53
Singh and Scarlatos minimum breach base width	0.2	5.4	22 (applying Wahl earthfill equation to volume of embankment eroded)

Table D-5 – Breach parameters from empirical equations for FSL = RL 94.0 mAHD

Empirical equation	Side slope	Sunny day base width (m)	Sunny day breach development time (min)
MacDonald Langridge Monopolis (Wahl) ¹	0.2	-0.1	12
Bureau of Reclamation	0.2	19.2	14
Froehlich (2008) ²	0.7	5.1	11
Von Thun Gillette (1990)	1	14.5	51
Singh and Scarlatos minimum breach base width	0.2	5.4	22 (applying Wahl earthfill equation to volume of embankment eroded)

Table D-6 – Breach parameters from empirical equations for FSL = RL 95.0 mAHD

Empirical equation	Side slope	Sunny day base width (m)	Sunny day breach development time (min)
MacDonald Langridge Monopolis (Wahl) ¹	0.2	0.5	14
Bureau of Reclamation (2014)	0.2	22	16
Froehlich (2008) ²	0.7	5.9	12
Von Thun Gillette (1990)	1	16.1	49
Singh and Scarlatos minimum breach base width	0.2	5.4	22 (applying Wahl earthfill equation to volume of embankment eroded)

Notes:

- 1) Breaches predicted do not conform to the geometry bounds suggested by Singh and Scarlatos. The breach base widths were increased to the minimum base width suggested by Singh and Scarlatos, and the breach time adjusted for the larger amount of embankment material removed, as shown in the last row of the table.
- 2) Breaches predicted do not conform to the geometry bounds suggested by Singh and Scarlatos. As there is no relationship between the predicted size of the breach and the predicted breach time these have not been adjusted.

D.1.3 Breach results and validation

The resulting breach hydrographs for each scenario are shown in Figure D-1 to Figure D-3, along with peak breach estimates from three empirical equations for validation described previously. A consistent scale has been used on all three figures for comparison purposes.

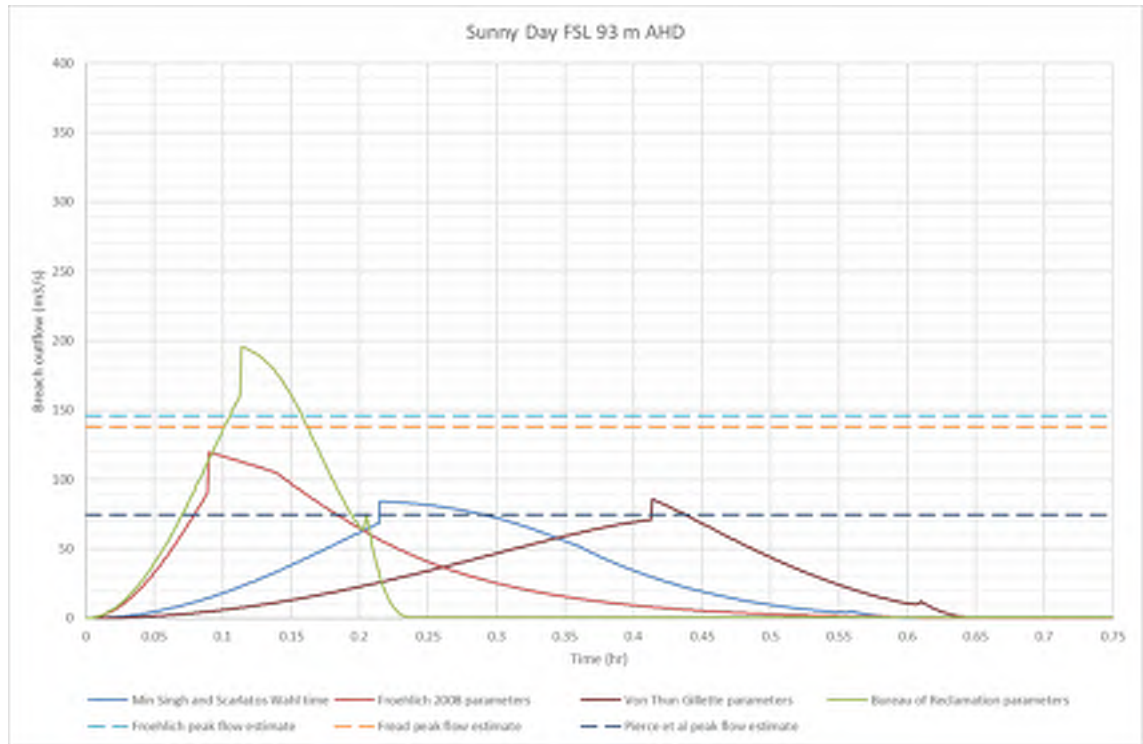


Figure D-1 SDF for FSL = RL 93.0 mAHD breach hydrographs

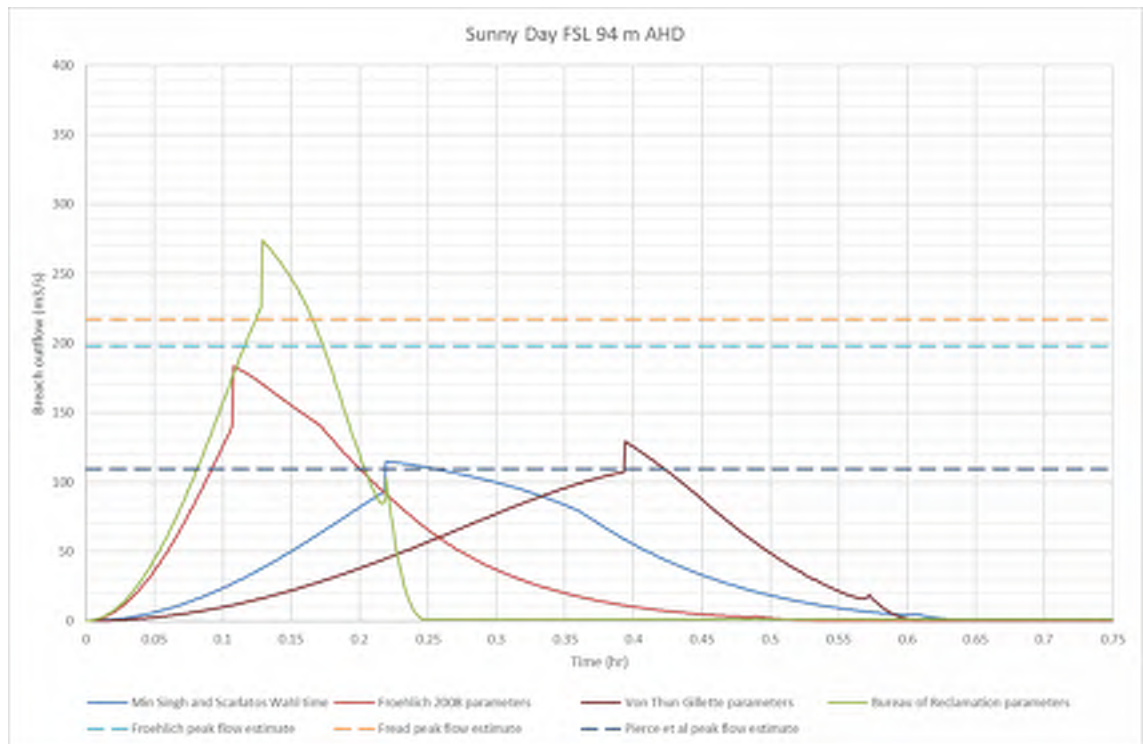


Figure D-2 SDF for FSL = RL 94.0 mAHD breach hydrographs

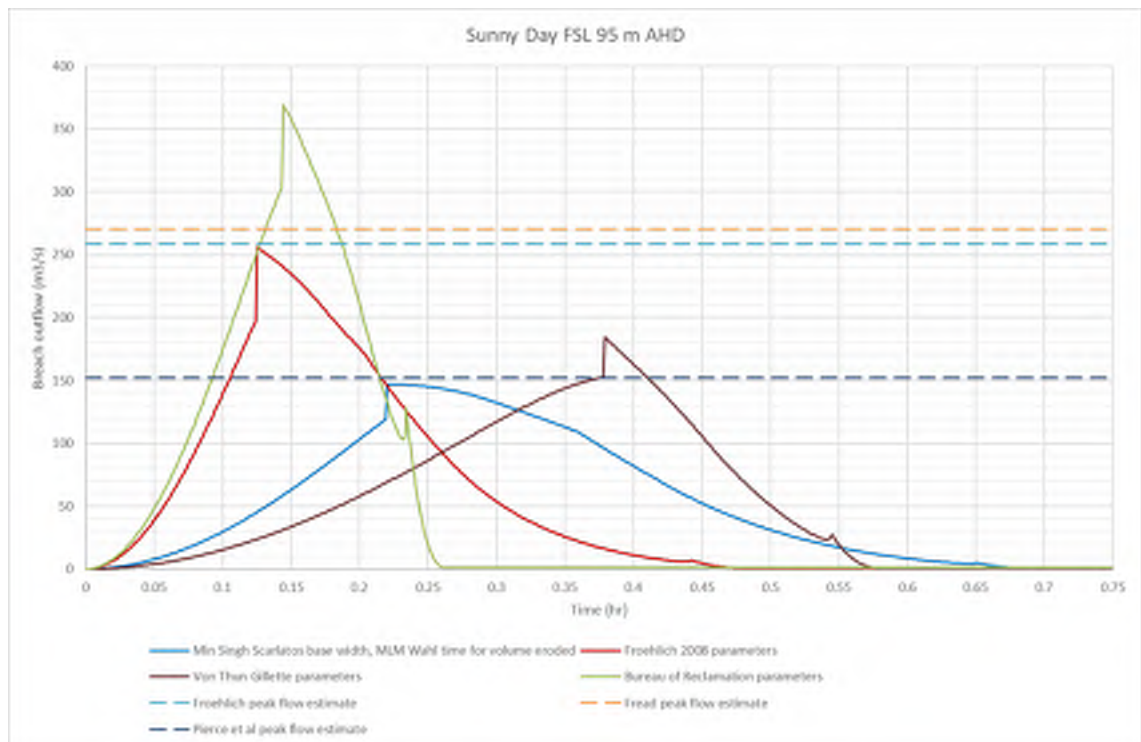


Figure D-3 SDF for FSL = RL 95.0 m AHD breach hydrographs

D.1.4 Adopted breach parameters

For each scenario, a most likely, upper bound and lower bound breach were selected after reviewing all the breach hydrographs collectively along with the empirical peak flow estimates (refer Section 4.1 and D.1.1. to D.1.3). There are a number of reasons for selecting multiple breach hydrographs rather than a single value:

1. To provide a sensitivity analysis on the breach assumptions (not just the most conservative);
2. To test the impact of timing on downstream flooding, acknowledging that a latter but smaller breach peak may coincide with downstream flood peaks to produce worse flooding in some areas; and
3. Depending on the floodplain characteristics, features such as constrictions or diversions may allow greater volume to reach downstream PAR during a more gradual release, resulting in higher consequences even though the peak at the dam was lower.

D.1.5 Dwellings

PAR for dwellings was derived by applying average occupancy rates of 2.8 from the 2016 census to the number of residential properties that were within the estimated inundation area.

During day time, dwelling occupancy was assumed half the night time rate recorded in the census. Floor levels were assumed to be 300 mm above the ground surface where no surveyed floor level was specified.

D.1.6 Boon Roses

Located on McMullen Road in Officer, the Boon Roses sheds and glasshouses are inundated by breach flows. It has been assumed that on average there are four (4) people working here during the day only.

D.1.7 Scout Park

The GWS Anderson Scout Park (<http://www.gwsandersonscoutcamp.org.au/>) is located on Haunted Gully Creek approximately one kilometre downstream of Beaconsfield Reservoir. In addition to buildings, the Scout Park includes a number of areas where numbers of people gather outside on a regular basis.

Occupancy assumptions for the Scout Camp were based on discussions with the caretaker in 2016, as outlined in Table D-7. Locations are shown in Figure D-4. Whilst it is noted that the maximum occupancy will not always be realised, there was not sufficient information available to reliably assign a “typical” occupancy.

Table D-7 Scout Park usage/occupancy assumptions

Area	ID	Frequency	Max. day time PAR	Max. night time PAR	Day time PAR (exposure factors applied)	Night time PAR (exposure factors applied)
Family Camp	UNI1	Four times a year	140	140	1.53	1.53
Scouts Camp	UNI2	Two days a fortnight	30	30	4.29	4.29
Nature Walk	UNI3	Two hours, once a month	30	0	0.20	0.00
Loch Lowe Canoe Area	UNI4	Two hours once a fortnight	30	0	0.43	0.00



Figure D-4 Locations of PAR considered at GWS Anderson Scout Park

D.1.8 Major roads

The Princes Highway is affected by some of the SDF breaches, and was considered separately to the PAR originating from buildings. The approximate Average Annual Daily Traffic volume (AADT) is 13,000 vehicles in each direction.

An average vehicle occupancy rate of 1.5 was applied. The number of vehicles assumed to be on a length of road at any given time was estimated based on the speed limit, length of road inundated and assumption that 80% of traffic occurs during the 10 hour day time period. It was assumed that 90% of the traffic were itinerants (from outside the area and not already accounted for).

If the water level rises almost instantaneously to the peak level on the road, it can be assumed that the PAR equals the number of people expected on average to be on the length of road that becomes inundated:

$$\frac{\text{Length inundated (km)}}{S \text{ (km/hr)}} \times \text{average vehicle occupancy} \times \text{average number of vehicles per hour}$$

Average day time vehicles per hour = 0.8 x AADT/10

Average night time vehicles per hour = 0.2 x AADT/14

Table D-8 Major road PAR assuming instantaneous inundation

Scenario	Length of Princes Hwy inundated (km)	Length of Princes Fwy inundated (km)	Day time PAR	Night time PAR	Average weighted PAR
SDF for FSL= RL 93.0 mAHD	0	0	0	0	0
SDF for FSL= RL 94.0 mAHD	0	0	0	0	0
SDF for FSL= RL 95.0 mAHD	0.09-0.11	0	1.61-2.08	0.29-0.37	0.84-1.08

D.1.9 Total Population at Risk

The total PAR for each scenario is summarised in Table D-9 to D-11 below.

Table D-9 – Total PAR-SDF for FSL = RL 93.0 mAHD

Location	Day (10 hours)	Night (14 hours)	Weighted average
Dwellings (2-3 flooded above assumed floor level)	3-4	6-8	4-7
Boon Roses	4	0	2
Scout Camp (UNI3, UNI4)	0.6	0	0.4
Princes Hwy	0	0	0
Total	8-9	6-8	6-9

Table D-10 – Total PAR-SDF for FSL = RL 94.0 mAHD

Location	Day (10 hours)	Night (14 hours)	Weighted average
Dwellings (3-4 flooded above assumed floor)	4-6	8-11	7-9
Boon Roses	4	0	2
Scout Camp (UNI1, UNI3, and UNI4)	0.6-2	0-2	0.4-2
Princes Hwy	0	0	0
Total	9-13	8-13	9-13

Table D-11 – Total PAR-SDF for FSL = RL 95.0 mAHD

Location	Day (10 hours)	Night (14 hours)	Weighted average
Dwellings (3-4 flooded above assumed floor level)	4-6	8-11	7-9
Boon Roses	4	0	2
Scout Camp (UNI1, UNI3 and UNI4)	2	2	2
Princes Hwy	1.61-2.08	0.29-0.37	0.84-1.08
Total	12-14	10-13	12-14

D.2 Probable Loss of Life (PLL)

D.2.1 Breach fatality rates

A fatality rate is applied to each location of PAR, based on the amount of warning and how hazardous the flood is (as defined by the velocity depth product). In 2014, the U.S. Bureau of Reclamation released an interim report titled “RCEM – Reclamation Consequence Estimating Methodology: Guidelines for Estimating Life Loss for Dam Safety Risk Analysis” (USBR, 2014). The new method has largely replaced Graham (USBR, 1999), which was released by the Bureau of Reclamation in 1999.

RCEM is very similar to the 1999 procedure in that it continues to rely on case history data to guide the selection of fatality rates; however, it now relies on a graphical representation of fatality rate as a function of flood severity and warning time. Flood severity is now defined quantitatively in terms of DV (product of flood depth and velocity).

The method uses two sets of curves, one representing where little or no warning is given and another for adequate warning. As it has been judged that there is little or no warning, the set of curves shown in Figure D-5 was used in the estimation of PLL.

The four curves represent the upper and lower bounds of overall and suggested limits. The final estimate for fatality rate is generally the median of the two suggested curves. In circumstances where the population is more vulnerable due to the nature of shelter available (such as at the Scout Camp), the overall upper curve may be appropriate.

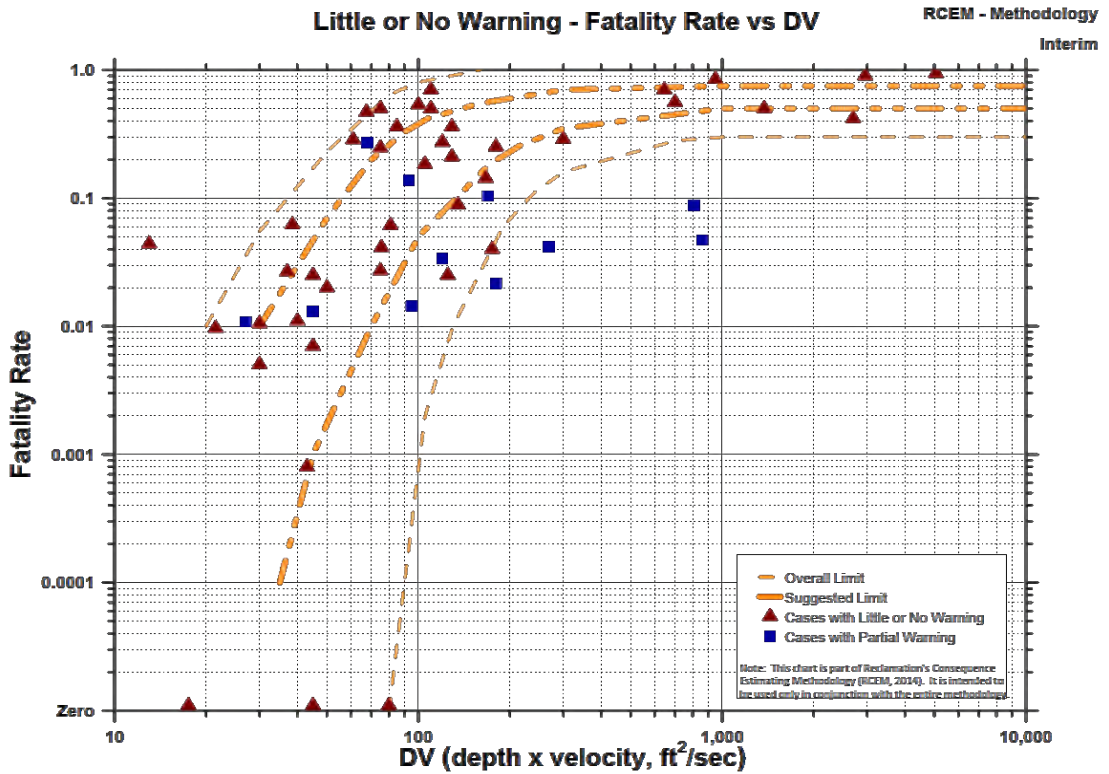


Figure 3 - Fatality Rate vs. DV for Little or No Warning 34

Figure D-5 Little or no warning - fatality rate vs DV (reproduced from USBR, 2014)

UK RARS (EA, 2013) offers an alternative fatality rate curve, which is derived from the Graham data, and intended to apply to small storages in populated areas. The no warning UK RARS, USBR suggested upper and USBR suggested lower curves are plotted together for comparison in Figure D-6 following.

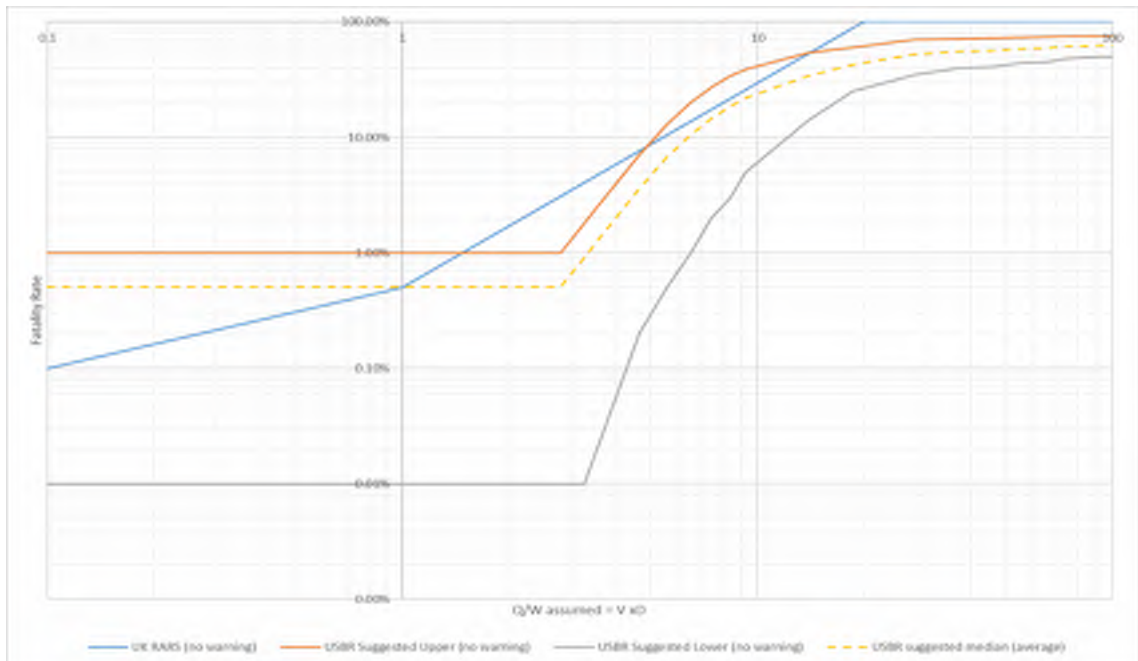


Figure D-6 Comparison of UK RARS and USBR no warning fatality rate curves

D.2.2 Potential Loss of Life on roads

Modelling of the SDF for an FSL of RL 95.0 mAHD predicts inundation of the northern (eastbound) carriageway of the Princes Highway.

The principles of the Campbell et al. (2013) 'Flooded Cars' method were applied to estimate PLL on the Princes Highway, considering the likelihood of vehicles being on the road when the flood wave arrives, or driving into flood waters when the road is already inundated.

Appendix E – PLL estimation and severity of damage and loss

Consequence Assessment of Sunny Day Upper bound Failure Estimate of Severity of Damage and Loss

Type	Explanatory Notes	Estimate	Category
1. Total Infrastructure Costs			
Residential	Total number of houses affected, some destroyed and some damaged.	8,443,260	
Commercial	Including business and agriculture, eg retail, manufacturing, resources, agriculture. These services should be assessed in terms of average annual wage.		
Infrastructure	Such as roads, railways, power, communications, gas, water supply, sewerage, irrigation, drainage, schools, hospitals, community facilities and public buildings. May be expressed in terms of annual cash flow or turnover.		
Dam repair and replacement cost	Repairs to the embankment or wall and appurtenant works which will return the dam to its previous level of service.		
Total (including indirect damages)		8,443,260	1
Assessment:			Minor
2. Impact on dam Owner's Business			
Importance to the business	Loss of storage is likely to affect the service provided to some degree. It may be appropriate, on one hand, to increase the severity level because of the importance of the reservoir. On the other hand, a less vital water resource may lead to a reduction in the severity of the cost of replacement or repair.	Restrictions needed during dry periods	Minor
Effect on services provided by the owner	Water supply, power or recreational facility is no longer available or disrupted to a proportion of the community supplied by the agency.	Minor difficulties in replacing services	Minor
Effect on continuing credibility	Standing or reputation of the organisation in the community	Severe widespread reaction	Medium
Community reaction and political implications	There may be community objection to replacement of the dam. Also, the relationship between the dam owner and local, state and federal legislature.	Severe widespread reaction	Medium
Impact on financial viability	Economic and legal liability; ability to meet the costs of repairs and damage; and ability to meet claims from others.	Able to absorb in one financial year	Minor
Value of water in the storage	Loss of income from loss of the stored water.	Can be absorbed in one financial year	Minor
Assessment:			Medium
3. Health and Social Impacts			
Public Health	Human health could be affected by: Contamination of drinking water * * Failure of lack of water supplies, sewage treatment works, power Contamination of services such as food, health, recreation areas and facilities caused by the uncontrolled release of sewage, industrial or toxic waste as a result of a dam break	<100 people affected	Minor
Loss of Services to the community	Loss of gas/power/communications and transport. Distribution of medical supplies, food, especially perishable food item	<100 people affected for one month	Minor
Cost of emergency management	Police, Emergency Services and volunteers will incur a cost both direct and indirect	<1,000 person days	Minor
Dislocation of people	People whose homes are destroyed or damaged will need to be housed or billeted for various times.	<100 person months	Minor
Dislocation of businesses	Business will be prevented from trading in the short term and may be affected in the long term.	<20 business months	Minor
Employment affected	Loss of employment.	<100 jobs lost	Minor
Loss of heritage	Historic sites, both pre and post European settlement.	Local facility	Minor
Loss of recreational facility	Many communities rely, to various degrees, on bodies of water for boating, fishing and other recreational aspects, including visual relief. Other recreational facilities may be located downstream of the reservoir, eg golf course, sports grounds.	Local facility	Minor
Assessment:			Minor
4. Environmental Impacts			
Area of impact	Land damaged by dam failure exclusive of land prone to natural flooding. For tailings dams, the damage will relate to the toxicity of the material in relation to both area of impact and the depth of penetration of the toxic materials	<1km ²	Minor
Duration of impact	Habitats may take a long time to recover. (e.g. Substantial erosion, deposition of flood borne materials). The duration of the impact will also relate to the toxicity of discharged material (e.g. saline, tailings, sewerage, cold water, deoxygenated water)	< 1year	Minor
Stock and Fauna	Stock and fauna may ingest contaminated water/fodder. Stock may need to be removed from the area or destroyed. Contaminants may cause damage in relation to reproduction cycle.	Discharge from dam break would not contaminate water supplies used by stock and fauna	Minor
Ecosystems	Includes organisms and non-living components which interact to form a stable system. Consideration should be given to their environment, habitat, breeding grounds and food chain.	Discharge from dam break is not expected to impact on ecosystems. Remediation possible.	Minor
Rare and Endangered Species	Information can be gained from state and federal agencies in relation to areas known to contain rare and endangered flora and fauna.	Species exist but minimal damage expected. Recovery within one year	Minor
Assessment:			Minor
OVERALL ASSESSMENT			Medium

Consequence Assessment of Wet Day Overtopping Failure

Estimate of Severity of Damage and Loss

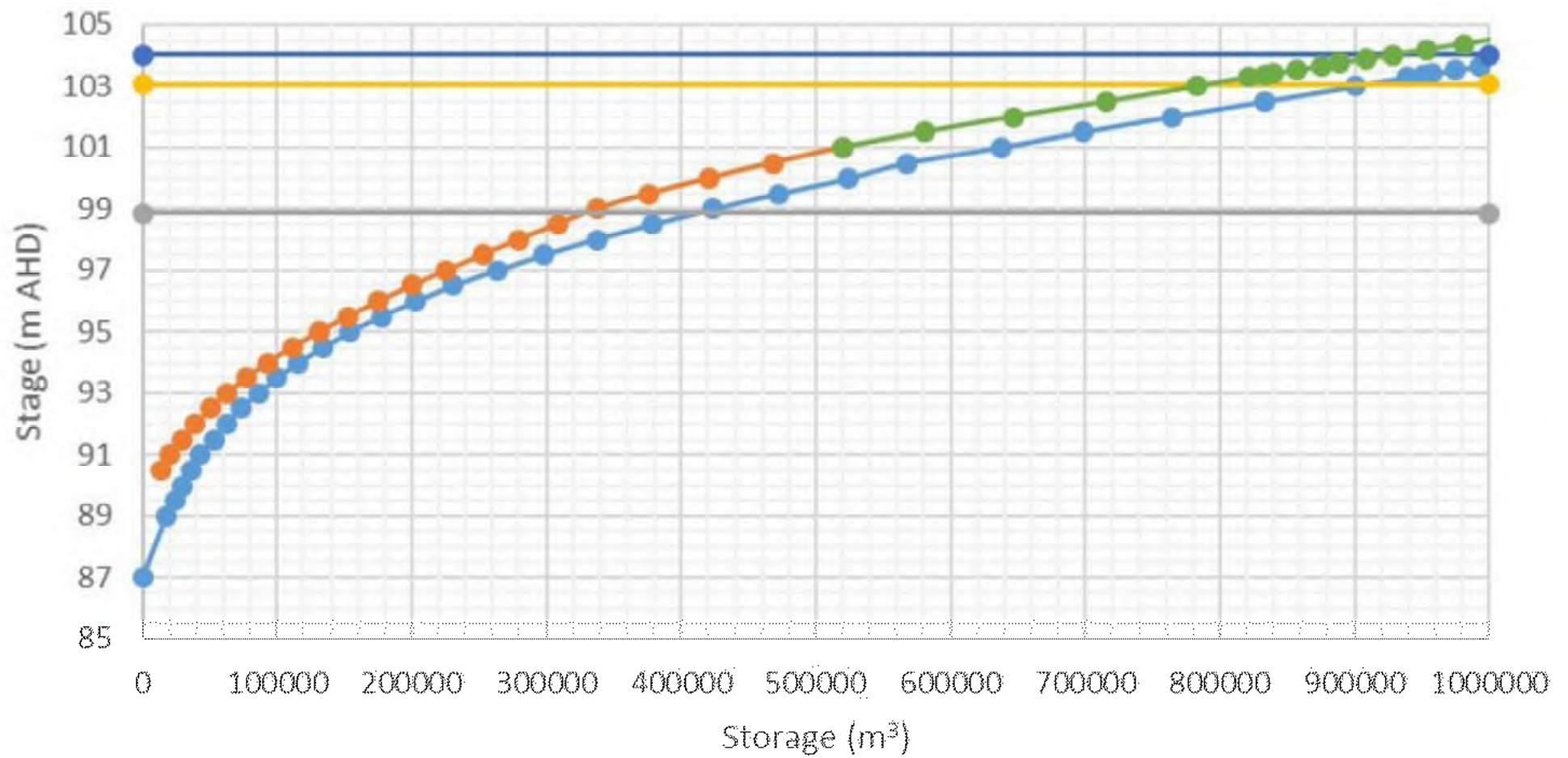
Type	Explanatory Notes	Estimate	Category
1. Total Infrastructure Costs			
Residential	Total number of houses affected, some destroyed and some damaged.	20,238,552	
Commercial	Including business and agriculture, eg retail, manufacturing, resources, agriculture. These services should be assessed in terms of average annual wage.		
Infrastructure	Such as roads, railways, power, communications, gas, water supply, sewerage, irrigation, drainage, schools, hospitals, community facilities and public buildings. May be expressed in terms of annual cash flow or turnover.		
Dam repair and replacement cost	Repairs to the embankment or wall and appurtenant works which will return the dam to its previous level of service.		
Total (including indirect damages)		20,238,552	2
Assessment:			Medium
2. Impact on dam Owner's Business			
Importance to the business	Loss of storage is likely to affect the service provided to some degree. It may be appropriate, on one hand, to increase the severity level because of the importance of the reservoir. On the other hand, a less vital water resource may lead to a reduction in the severity of the cost of replacement or repair.	Restrictions needed during dry periods	Minor
Effect on services provided by the owner	Water supply, power or recreational facility is no longer available or disrupted to a proportion of the community supplied by the agency.	Minor difficulties in replacing services	Minor
Effect on continuing credibility	Standing or reputation of the organisation in the community	Severe widespread reaction	Medium
Community reaction and political implications	There may be community objection to replacement of the dam. Also, the relationship between the dam owner and local, state and federal legislature.	Severe widespread reaction	Medium
Impact on financial viability	Economic and legal liability; ability to meet the costs of repairs and damage; and ability to meet claims from others.	Able to absorb in one financial year	Minor
Value of water in the storage	Loss of income from loss of the stored water.	Can be absorbed in one financial year	Minor
Assessment:			Medium
3. Health and Social Impacts			
Public Health	Human health could be affected by: Contamination of drinking water * * Failure of lack of water supplies, sewage treatment works, power Contamination of services such as food, health, recreation areas and facilities caused by the uncontrolled release of sewage, industrial or toxic waste as a result of a dambreak	<100 people affected	Minor
Loss of Services to the community	Loss of gas/power/communications and transport. Distribution of medical supplies, food, especially perishable food item	<100 people affected for one month	Minor
Cost of emergency management	Police, Emergency Services and volunteers will incur a cost both direct and indirect	<1,000 person days	Minor
Dislocation of people	People whose homes are destroyed or damaged will need to be housed or billeted for various times.	<100 person months	Minor
Dislocation of businesses	Business will be prevented from trading in the short term and may be affected in the long term.	<20 business months	Minor
Employment affected	Loss of employment.	<100 jobs lost	Minor
Loss of heritage	Historic sites, both pre and post European settlement.	Local facility	Minor
Loss of recreational facility	Many communities rely, to various degrees, on bodies of water for boating, fishing and other recreational aspects, including visual relief. Other recreational facilities may be located downstream of the reservoir, eg golf course, sports grounds.	Local facility	Minor
Assessment:			Minor
4. Environmental Impacts			
Area of impact	Land damaged by dam failure exclusive of land prone to natural flooding. For tailings dams, the damage will relate to the toxicity of the material in relation to both area of impact and the depth of penetration of the toxic materials	<1km2	Minor
Duration of impact	Habitats may take a long time to recover. (e.g. Substantial erosion, deposition of flood borne materials). The duration of the impact will also relate to the toxicity of discharged material (e.g. saline, tailings, sewerage, cold water, deoxygenated water)	< 1year	Minor
Stock and Fauna	Stock and fauna may ingest contaminated water/fodder. Stock may need to be removed from the area or destroyed. Contaminants may cause damage in relation to reproduction cycle.	Discharge from dambreak would not contaminate water supplies used by stock and fauna	Minor
Ecosystems	Includes organisms and non-living components which interact to form a stable system. Consideration should be given to their environment, habitat, breeding grounds and food chain.	Discharge from dambreak is not expected to impact on ecosystems. Remediation possible.	Minor
Rare and Endangered Species	Information can be gained from state and federal agencies in relation to areas known to contain rare and endangered flora and fauna.	Species exist but minimal damage expected. Recovery within one year	Minor
Assessment:			Minor
OVERALL ASSESSMENT			Medium

Appendix F – Multi-Criteria analysis

MCA categories & sub-categories	Category weighting	Sub-category weighting (out of 4)	OPTIONS				Category & sub-category comments	Options comments	
			1 Partial Decom'ing / partial height dam	2 Full Decom'ing / removal of dam	3 Safety upgrade (full upgrade)	4 Do nothing / current arrangement			
1	Cost	30		21.3	15.6	17.3	23.1	High category weighting for costs due to importance of cost to MWC and community for upgrade/construction works and ongoing maintenance.	
1.1	Construction cost		4	3	1	2	4	All construction costs for each option, including removal/treatment of reservoir silt for full decommissioning option. High sub-category weighting of 5 for construction cost due to magnitude of costs (millions of dollars).	No cost for "Do nothing" (hence best score), lower cost for "Safety Upgrade", lowest costs for "Full decommissioning" due to full removal of dam.
1.2	Ongoing maintenance cost		4	3	4	2	1	Maintenance costs include ongoing dam safety operations, including routine and periodical dam safety inspections, and regular dam maintenance. High sub-category weighting of 4 for dam maintenance/safety costs due to ongoing commitment of these costs after any upgrade/construction works.	Highest maintenance costs for "Do nothing" (as significant maintenance work still needs to be performed, hence worst score), no dam costs for "Full decommissioning" (hence best score). Some costs for partial to allow for asset inspection and blockage removal.
1.3	Cost of public amenity operations and maintenance		1	3	3	4	4	Public amenities maintenance costs include items such as toilets, roads, carparks, boardwalks, tracks, benches/shelters/picnic areas, playgrounds, etc, along with environmental maintenance such as treating erosion over time and managing plants/trees/weeds/grass/etc. Lower weighting of 2 for amenities maintenance costs due to lower costs associated with maintaining public park amenities, specifically lower cost to MWC.	Higher costs expected for ongoing environmental maintenance for "Partial decommissioning" and particularly for "Full decommissioning" due to potential ongoing environmental management and erosion treatment required in later years (hence lower scores). No additional public assets created for do nothing or safety upgrade, so lowest costs.
1.4	Approvals, public engagement costs		2	2	1	3	4	Approvals (inc environmental/EBPC) and public consultation costs, etc. Lower weighting of 2 for approvals and public engagement costs due to lower cost compared with total construction cost.	Higher costs expected for "Full decommissioning" due to larger environmental impacts (may require more approvals) and public consultations (hence lower score).
1.5	Design, engineering costs		2	3	1	2	4	Costs for engineering design, consultants and MWC for design tasks. Lower weighting of 2 for design and engineering costs due to lower cost compared with total construction cost.	Larger costs expected for "Safety Upgrade" and "Full decommissioning" (due more design tasks required, hence lower score). No cost for "Do nothing" (hence best score).
2	Satisfying ALARP	30		25.7	30.0	21.4	7.5	High weighted category due to critical aspect of reducing risks to life and community safety from dam operations, and key driver of upgrade/construction works for Beaconsfield Reservoir.	
2.1	F-N Position / Life safety risk		4	3	4	2	1	Highly weighted sub-category due to importance of achieving "As Low As Reasonably Practicable" risk management for life and community safety for Beaconsfield Res.	"Partial decommissioning" greatly reduces risks associated with the dam (high score). Full decommissioning removes the risk entirely, thus highest score. "Safety Upgrade" reduces risks associated with current dam (residual risks larger than decommissioning options remain, hence medium score). No improvement from current inadequate risk profile for "Do nothing" (hence lowest score).
2.2	Compliance with good practice		3	4	4	4	1	Moderately highly weighted sub-category due to importance of sufficient flood handling capacity for dam safety, and reducing likelihood of dangerous flash flooding from dam failure.	"Partial/Full decommissioning" and "Safety upgrade" greatly reduces risks associated with the dam and meet required spillway capacity (hence highest score). No improvement from current inadequate spillway capacity for "Do nothing" (hence lowest score).
3	Community impacts	20		16.3	10.5	16.5	16.5	Lower category weighting due to lower impact visual appearance and amenity compared with life safety risks.	
3.1	Provision of public amenities and safe access		3	4	4	3	2	Potential for provision of amenities such as toilets, roads, carparks, boardwalks, tracks, benches/shelters/picnic areas, playgrounds, etc. Medium weighting for sub-category due to moderate importance.	"Partial/Full decommissioning" options provide greatest opportunity for adding public amenities to site (hence highest score). "Safety Upgrade" offers a good opportunity to add public amenities (hence medium score). "Do nothing" offers little/no opportunity for amenities (hence lowest score).
3.2	Visual appearance of landscape		4	4	4	3	2	Judgement about aesthetics of landscape and environment (after environment and plantings established/recovered from upgrade/construction work). Moderately-high weighting for sub-category due to higher importance to community.	"Partial decommissioning" allows for greatest range of vegetation, views and environments to provide visual appearance to site (hence highest score). "Full decommissioning" allows for re-planting to 'natural creek' environment but does not have any lake/retained water (hence moderately high score). "Do nothing" maintains current visual appearance with water and bushland (low score). "Safety upgrade" requires expanding dam and spillway footprints which would reduce visual amenity (hence lowest score).
3.3	Visual appearance of lake/retained water		3	2	1	4	4	Judgement about aesthetics of retained lake water (after environment and plantings established/recovered from upgrade/construction work). Moderately-high weighting for sub-category due to higher importance to community.	Visual appearance of lake/retained water high for "Safety upgrade" and "Do nothing" (hence moderately-high score, note reduced Full Supply Level to be maintained, so water level in reservoir not maximised). Partial area of lake maintained for "Partial decommissioning" (hence medium score). No water retained for "Full decommissioning" (hence lowest score).
3.4	Retention/incorporation of heritage & 'past infrastructure' elements		1	4	2	3	4	Potential educational & public interest benefits from retaining old elements of dam. Low weighting for sub-category due to low importance.	"Partial decommissioning" offers potential to preserve/relocate dam infrastructure through out the park for visitors (hence highest score). "Full Decommissioning" - after further community consultation, little heritage benefit is retained if the dam is fully decommissioned. "Safety upgrade" offers potential for some old infrastructure to be preserved/relocated for visitors, but others may need to be retained for use or replaced (hence medium score). "Do nothing" keeps all existing infrastructure in place (hence highest score as well).
3.5	Impact on community by construction activity, vehicle movements, etc		3	3	1	2	4	Impacts to local residents and surrounding community by construction activity. Moderately-highly weighted due to being often a key issue for local communities.	Lowest impacts for "Do nothing" (hence highest score). Moderate impacts for "Partial decommissioning" due to lowest requirement for importing of materials to and from site (hence medium score). Large impacts for "Full decommissioning" due to potential to have to import larger quantities of erosion protection and remove silts/spoil from site (hence lower score). Major impacts for "Safety upgrade" due to volume of material required to be imported (hence lowest score).
3.6	Fire		3	3	1	4	4	In order for an air crane access a water body it must be min. 2 m deep and be clear of trees and other obstruction at a 35 m radius from its center. Beaconsfield Reservoir is not in the fire fighting handbook provided to pilots, Cardinia Reservoir and Lake Aura Vale (6km north) are. Saying that, in the event of an emergency any water source can be used for fire fighting.	"Do nothing" and "Safety upgrade" have highest scores at WL 98.85. "Partial decommissioning" has a moderate score as significant (for firefighting purposes) volume of water retained. Low score for "Full decommissioning" as no water retained.
3.7	Flood mitigation		3	3	1	4	4	Flood mitigation potential for dam options. Note that Beaconsfield Reservoir does not primarily function as a flood detention reservoir, hence medium weighting for this sub-category. However, this is a requirement for existing catchments not to increase flooding up to 1% AEP event.	Higher level of (not-extreme storm) flood mitigation achieved with "Do nothing" and "Safety upgrade" conditions (hence moderately-high score). Some flood mitigation possible with "Partial decommissioning" (hence medium score). No flood mitigation possible with "Full decommissioning" (hence lowest score).
4	Environmental and conservation impacts	20		15.0	5.0	10.0	17.5	Lower category weighting due to lower impact of environment and conservation of nature compared with life safety risks.	
4.1	Construction and rehabilitation period		3	2	1	2	4	Likely requirements and difficulties for permitting and approvals for upgrade/construction designs. Lower weighting for sub-category due to lower importance.	Full decommissioning will have impacts on aquatic species in the reservoir + weeds may invade water body area during rehabilitation and impacts of construction vehicles (hence lowest score). The partial upgrade will also have a reduced potential for weed invasion during the rehabilitation and construction vehicle impact. The full dam safety upgrade will have impacts due to heavy machinery. Do Nothing has no impact.
4.2	Long-term impacts on flora & fauna communities		3	4	1	2	3	Potential long-term impacts on environment due to rehabilitation or habitat availability. Current conditions are the 'baseline' (with potential for positive improvements increasing scores). Medium weighting for sub-category due to moderate importance.	Greater opportunity for beneficial environmental conditions when environment recovers post-construction, with planting/colonisation of previously-inundated areas for different flora environments (wetland, treed/bushland) for "Partial/Full decommissioning" (hence moderately-high score). Current environmental conditions maintained for "Safety upgrade" and "Do nothing" (hence medium score as 'baseline' conditions).
TOTAL SCORE				78.3	61.1	65.2	64.6		

Rating	4	Excellent / best performance, best for purpose
	3	Good / performs well
	2	Poor / low performance
	1	Very Poor / worst performance, worst for purpose

Appendix G – Storage-elevation curve



2012 modelled

Bathmetry survey

Drawdown level

FSL

Embankment low-point

Bathmetry extrapolated

Appendix H – Landscape drawings

Option 1A

Option 1B

Option 1C

Option 1D

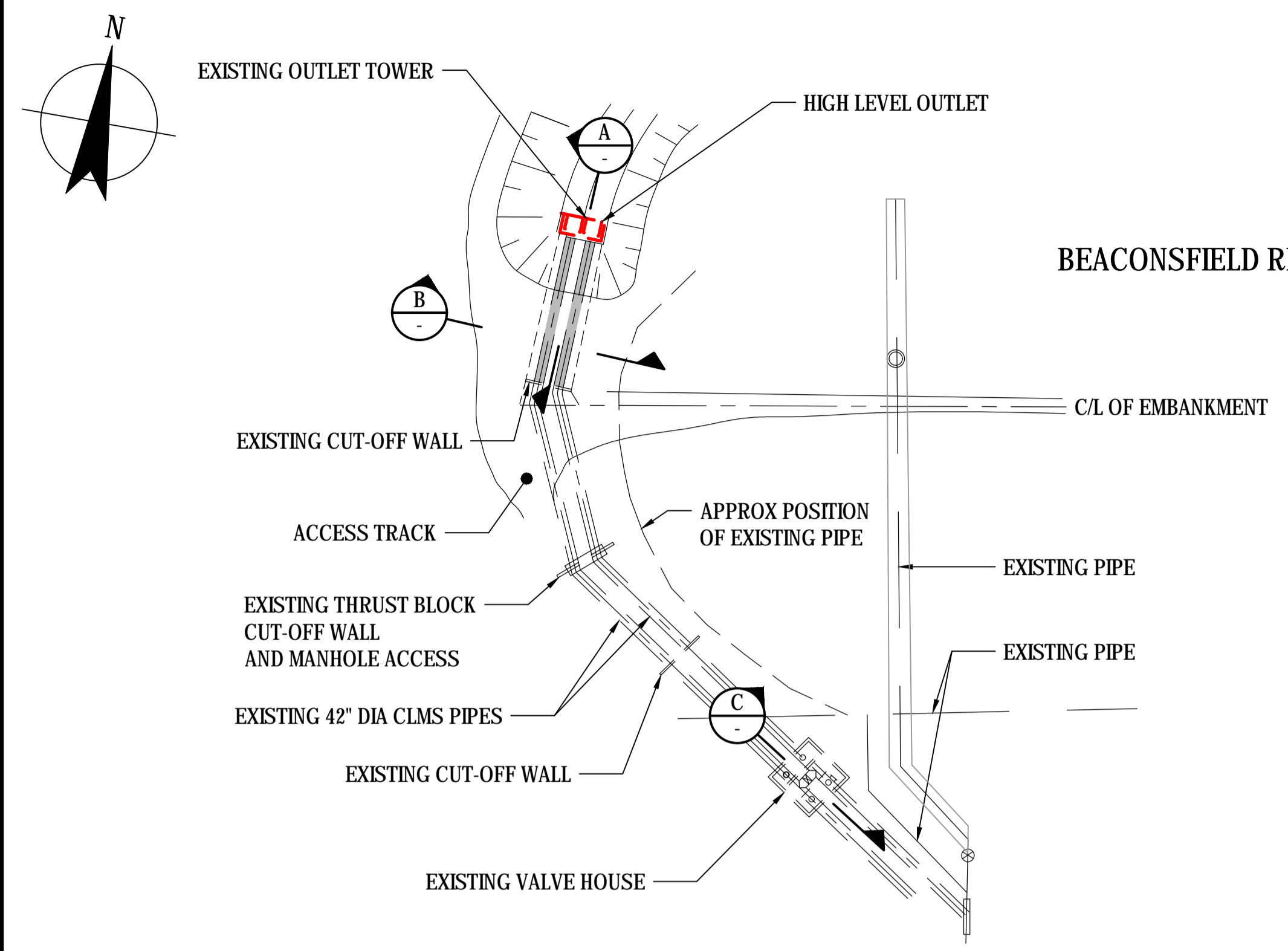
Appendix I – Concept Options drawings

Option 1A

Option 1B

Option 1C

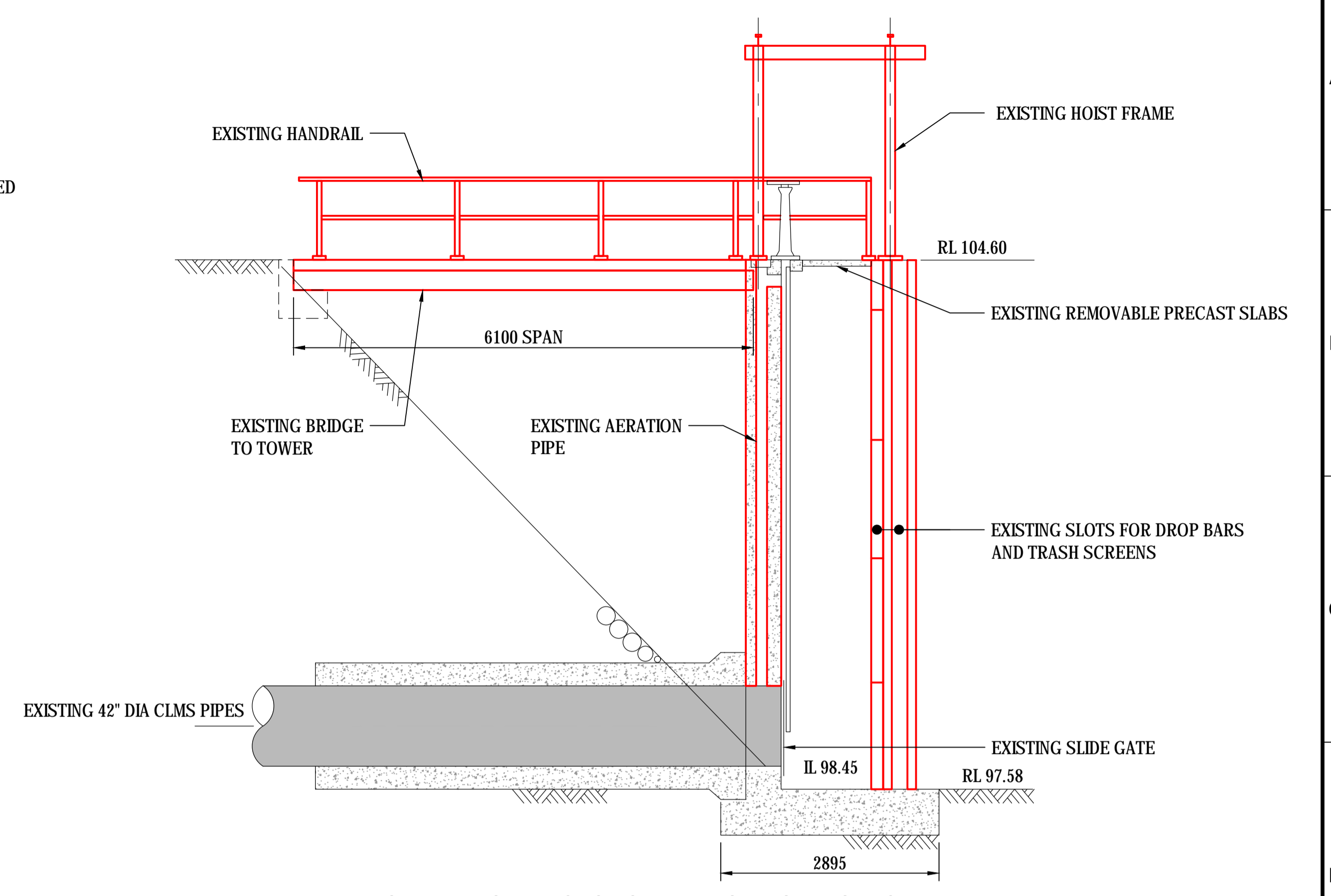
Option 1D



PLAN OF HIGH LEVEL OUTLET
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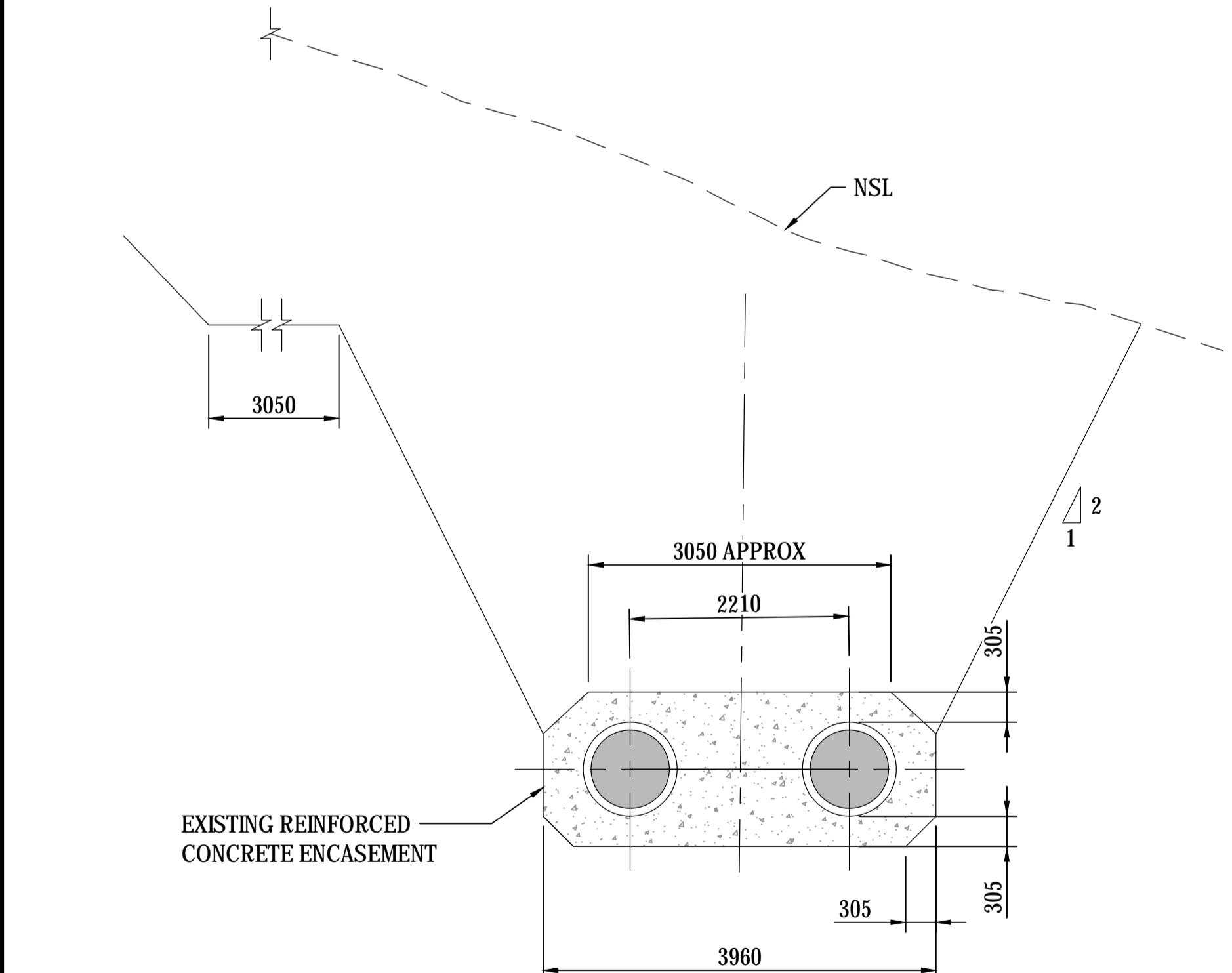
LEGEND

- TO BE DISMANTLED
- TO BE DEMOLISHED
- TO BE CONCRETE BACKFILLED



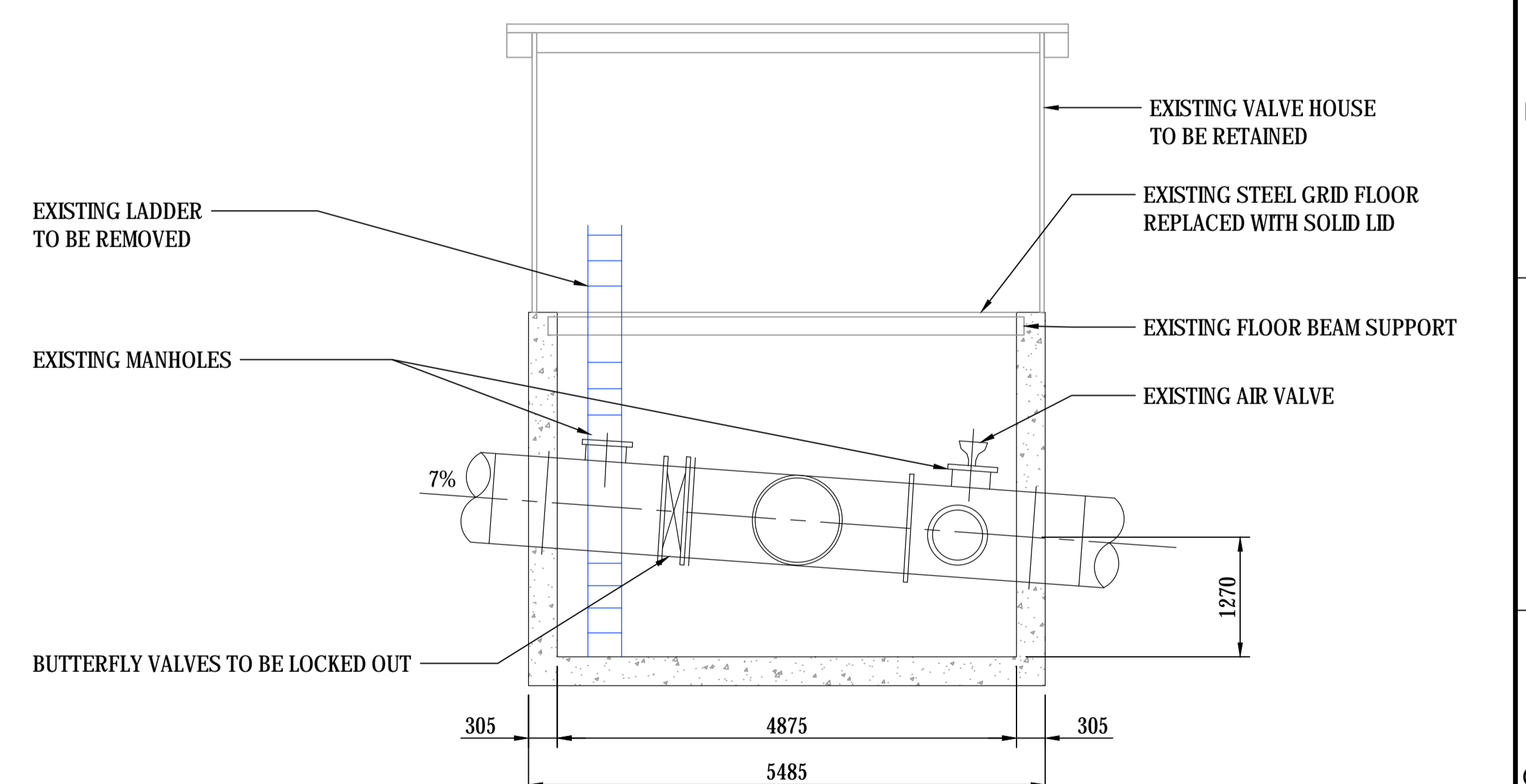
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PIPE SECTION - BACKFILL WORKS

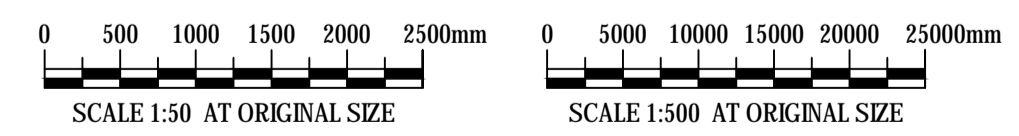
B SECTION
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VALVE HOUSE - MINOR WORKS

C SECTION
SCALE 1: 50

PRELIMINARY



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B	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	31.07.2019
A	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	25.02.2019

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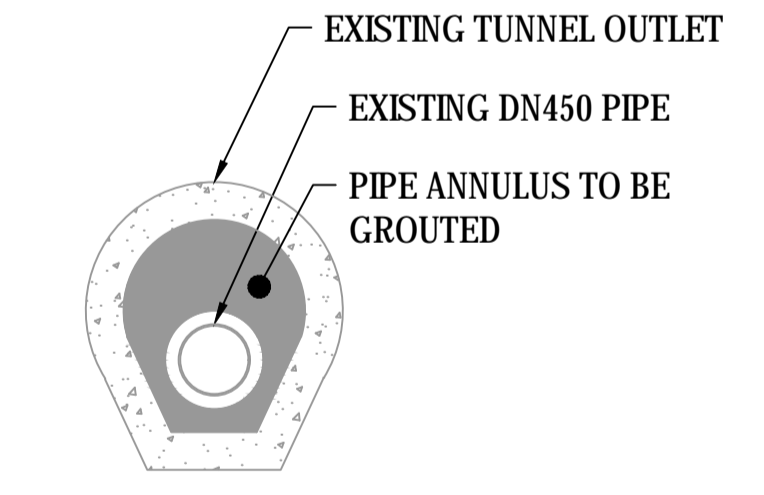
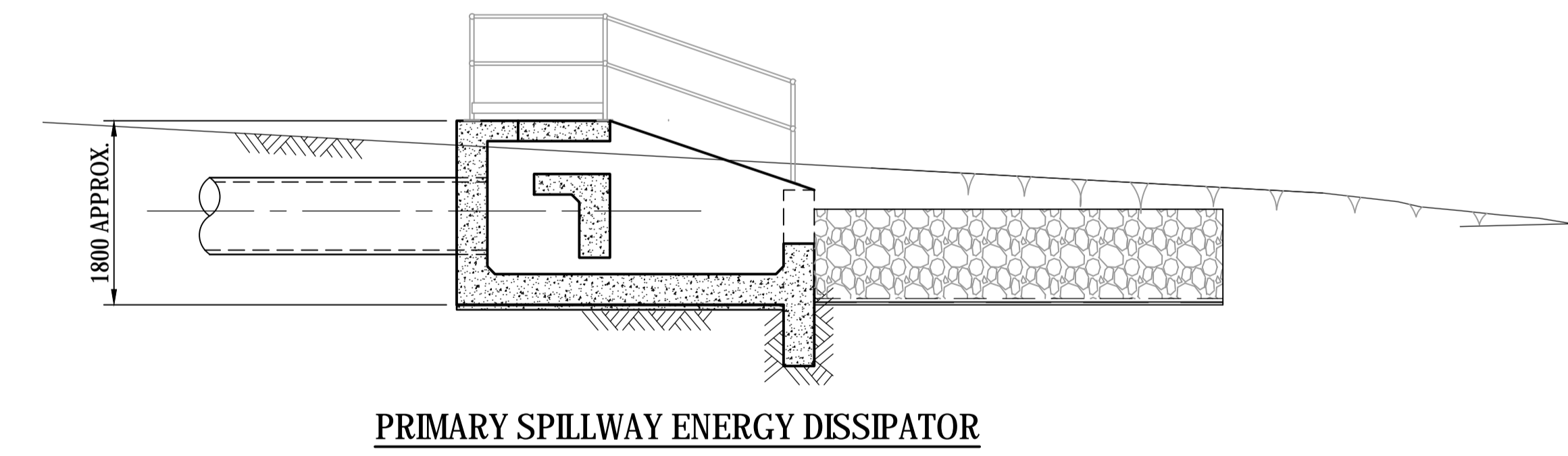
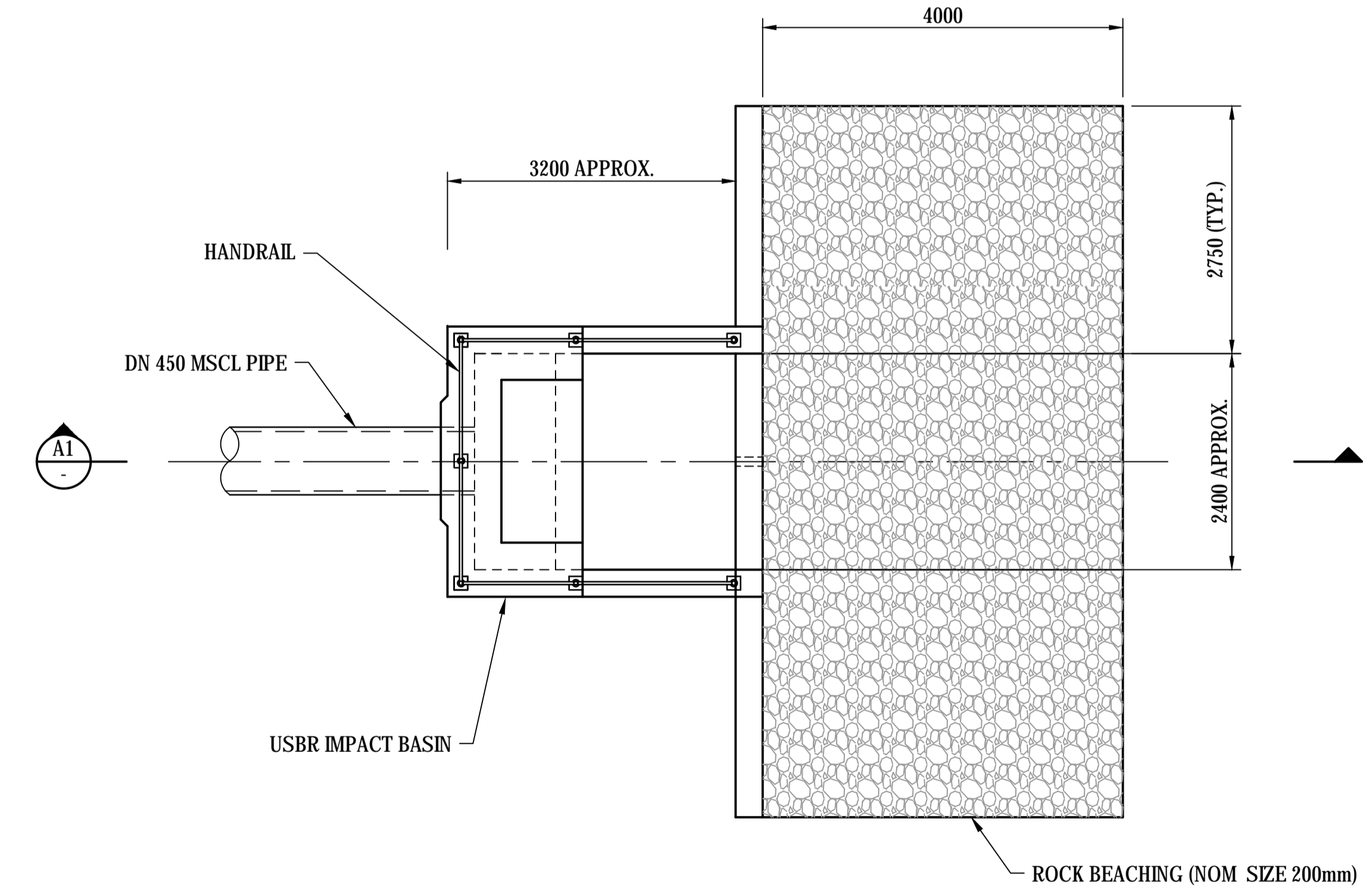
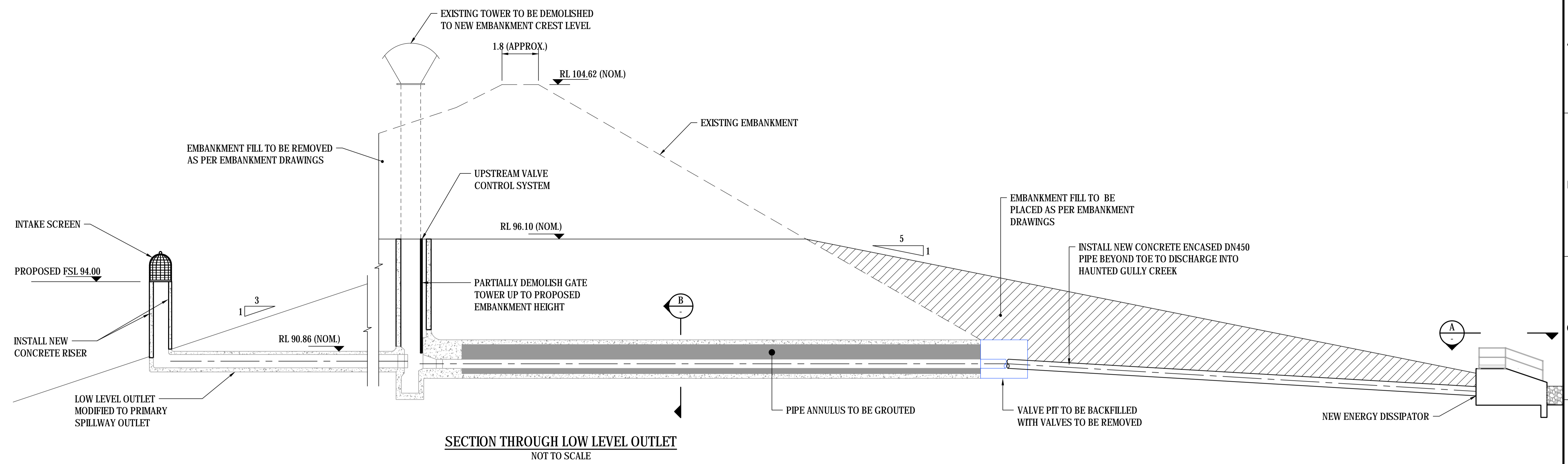
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BEACONSFIELD RESERVOIR

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DRAFTING CHECK	ENGINEERING REVIEW		

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CONCEPT DESIGN
HIGH LEVEL OUTLET - DECOMMISSION WORKS

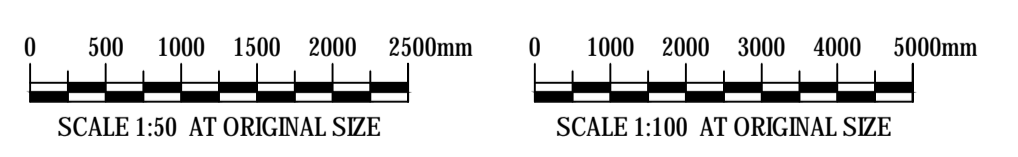
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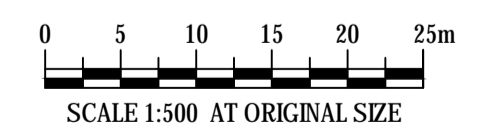
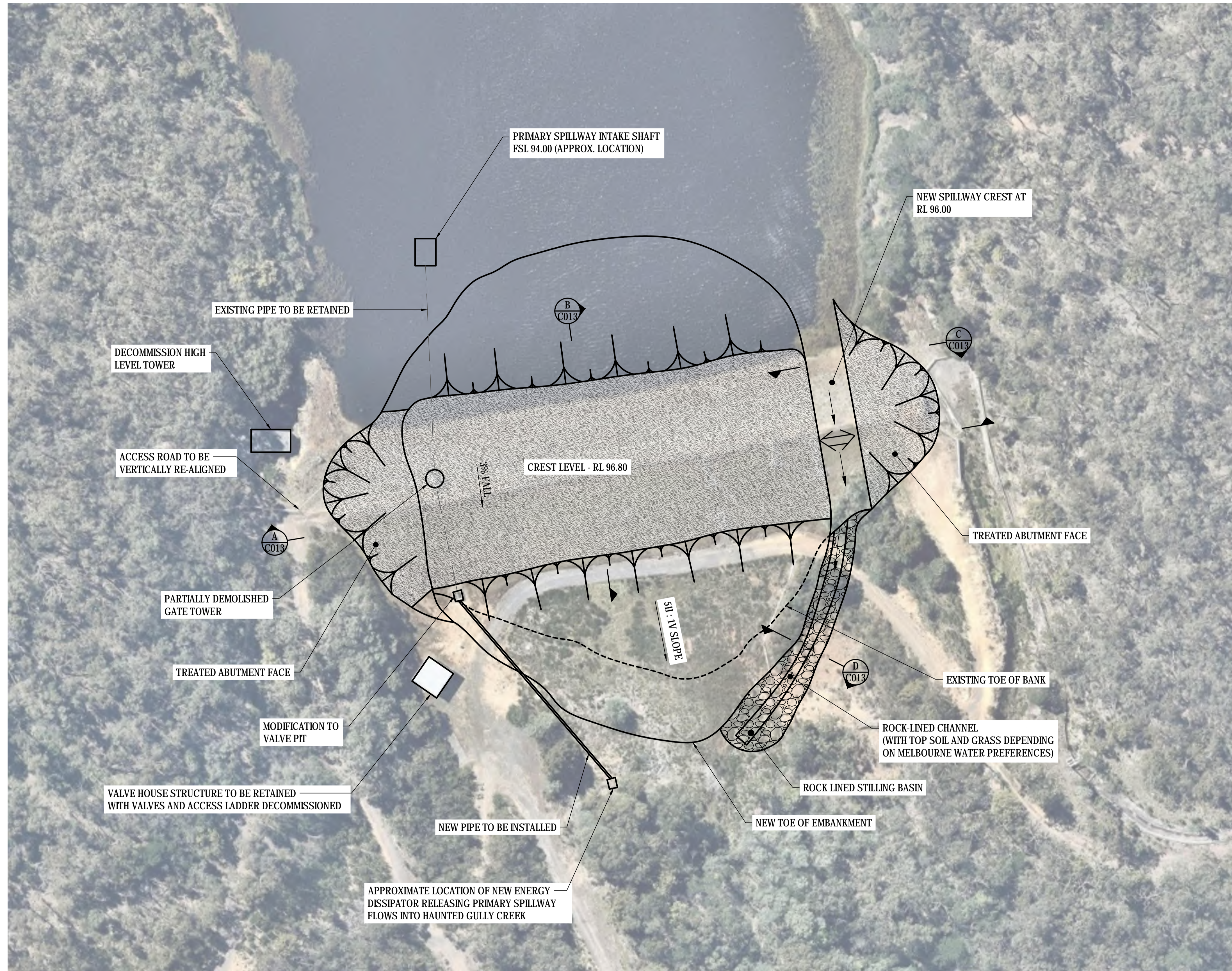
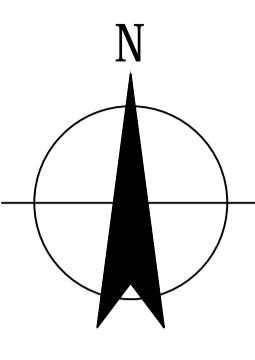
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B SECTION
SCALE 1 : 50



PRELIMINARY

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Level 8, 180 Lonsdale Street, Melbourne VIC 3000 Australia T 61 3 8687 8000 F 61 3 8687 8111 E melmail@ghd.com.au W www.ghd.com					GHD DRAWING NUMBER 31-36304-C011		PROJECT DATUM SCALE AS SHOWN		Original Size A1		MELBOURNE WATER CORPORATION MW Drawing No MWC DRAWING NUMBER																													
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PLAN
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PRELIMINARY

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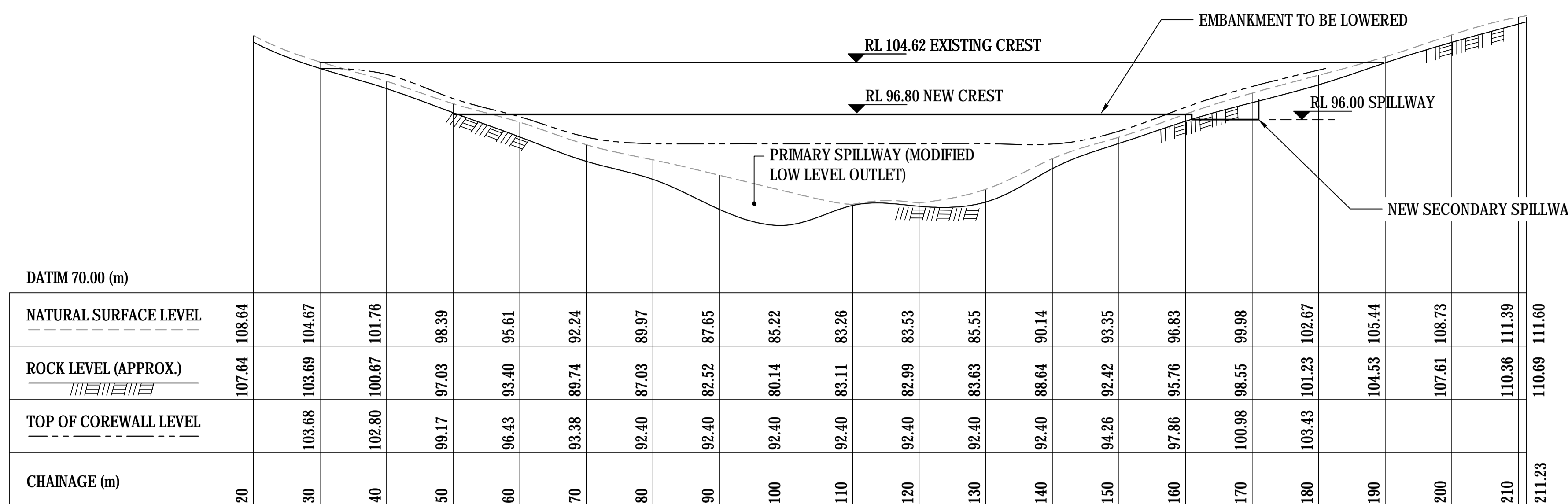
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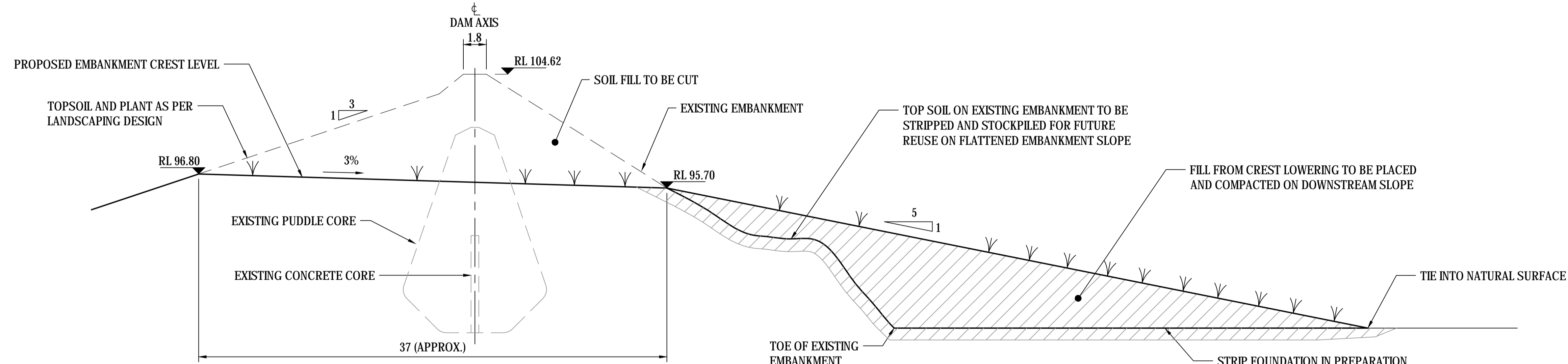
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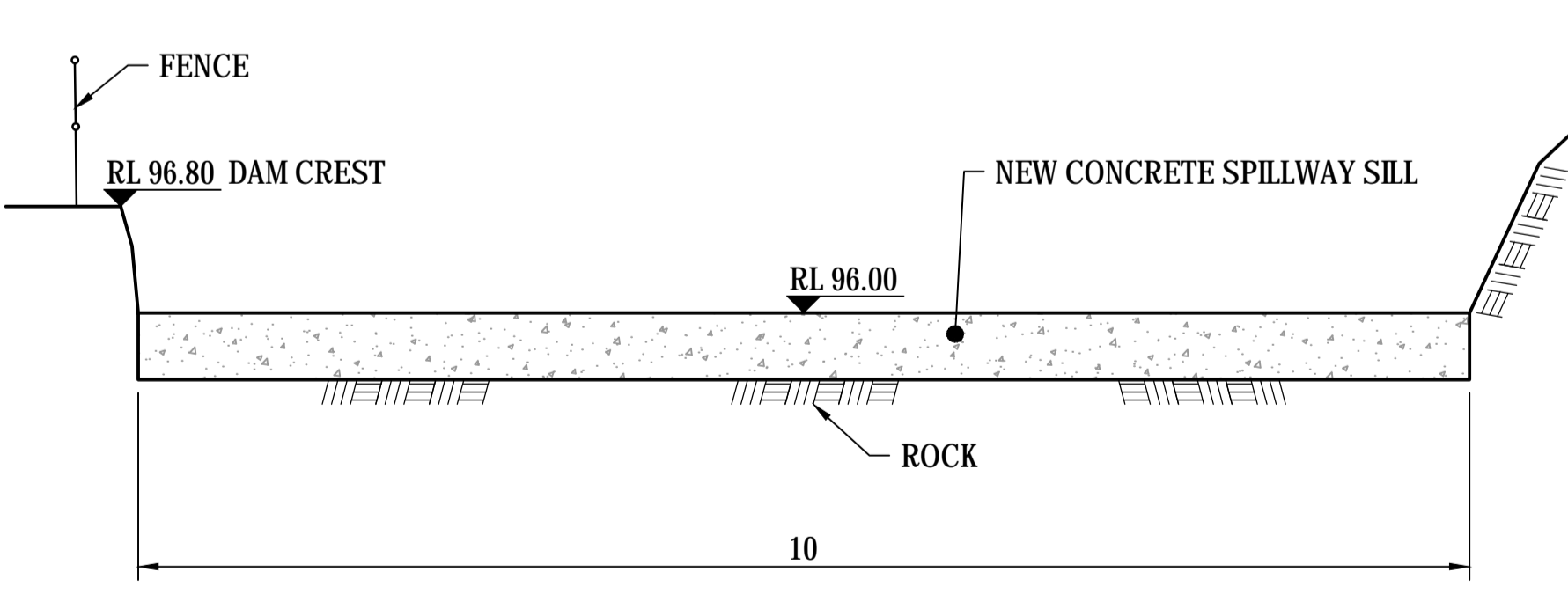
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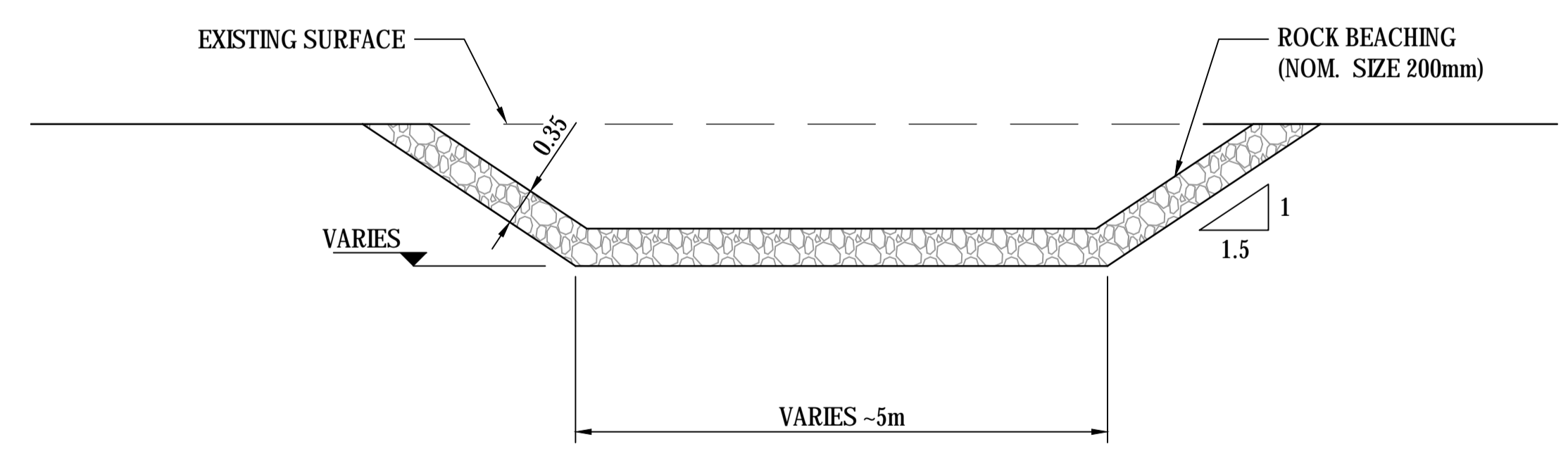
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B SECTION
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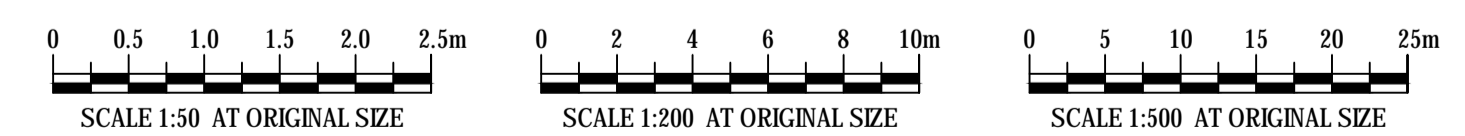
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ROCK LINED CHANNEL TYPICAL DETAIL

D SECTION
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PRELIMINARY



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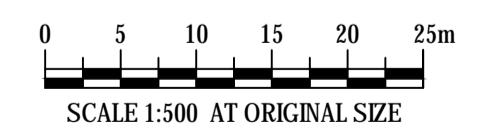
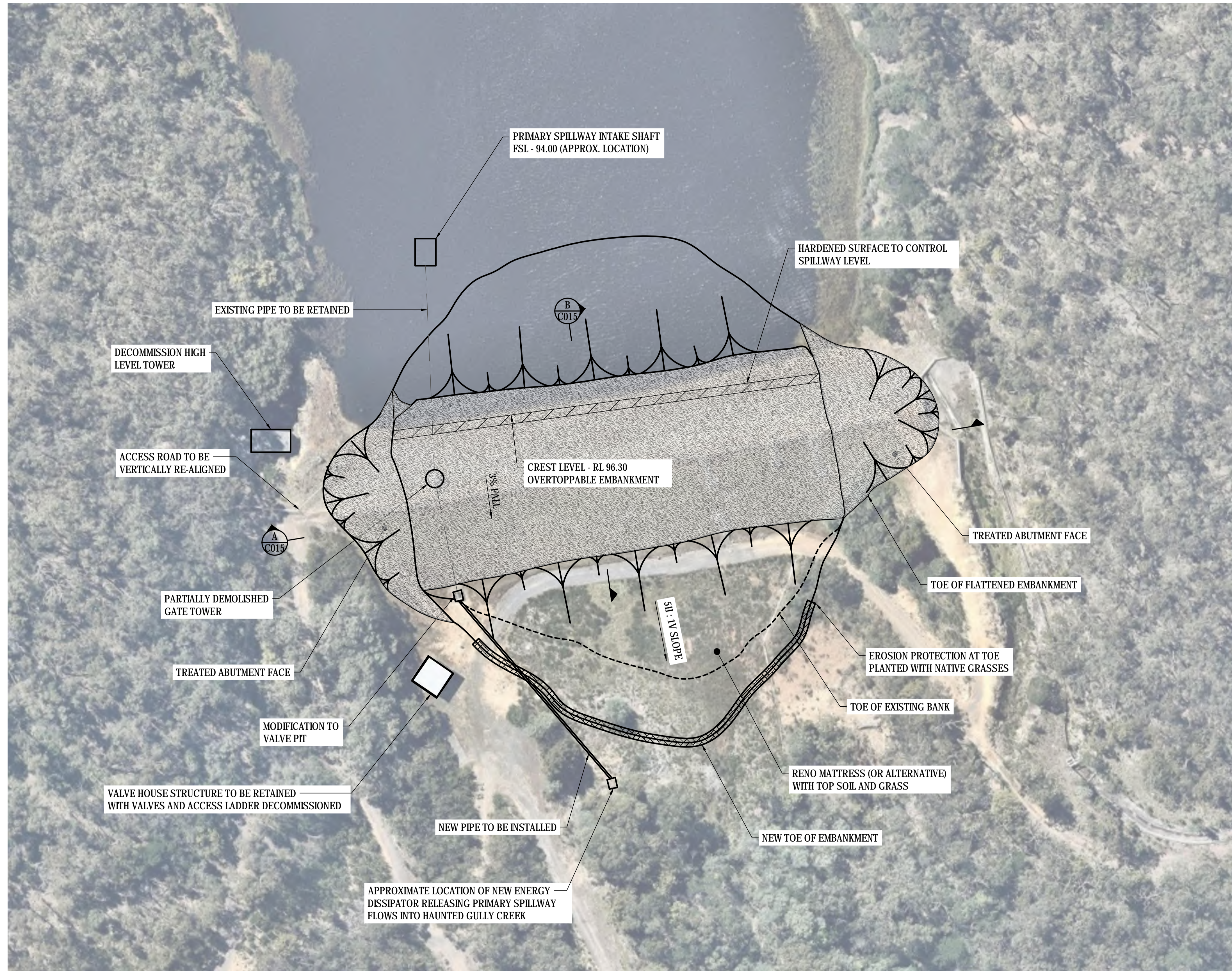
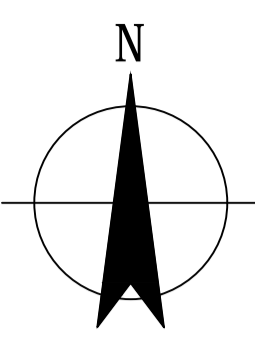
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BEACONSFIELD RESERVOIR

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DRAFTING CHECK	ENGINEERING REVIEW		

TITLE: BEACONSFIELD RESERVOIR
CONCEPT DESIGN
OPTION A - SECTIONS

PROJECT DATUM	Original Size A1	MELBOURNE WATER CORPORATION	
SCALE AS SHOWN	C	MW Drawing No	C
	CODE	MVC DRAWING NUMBER	REV



PLAN
SCALE 1:500

PRELIMINARY

REV	DESCRIPTION	COMPANY	PROJECT OR WO NUMBER	DRAWN	ENG. CHECK	PR. MAN. APPD.	DATE
B	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	28.10.2019
A	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	25.02.2019

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GHD DRAWING NUMBER
31-36304-C014

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BEACONSFIELD RESERVOIR

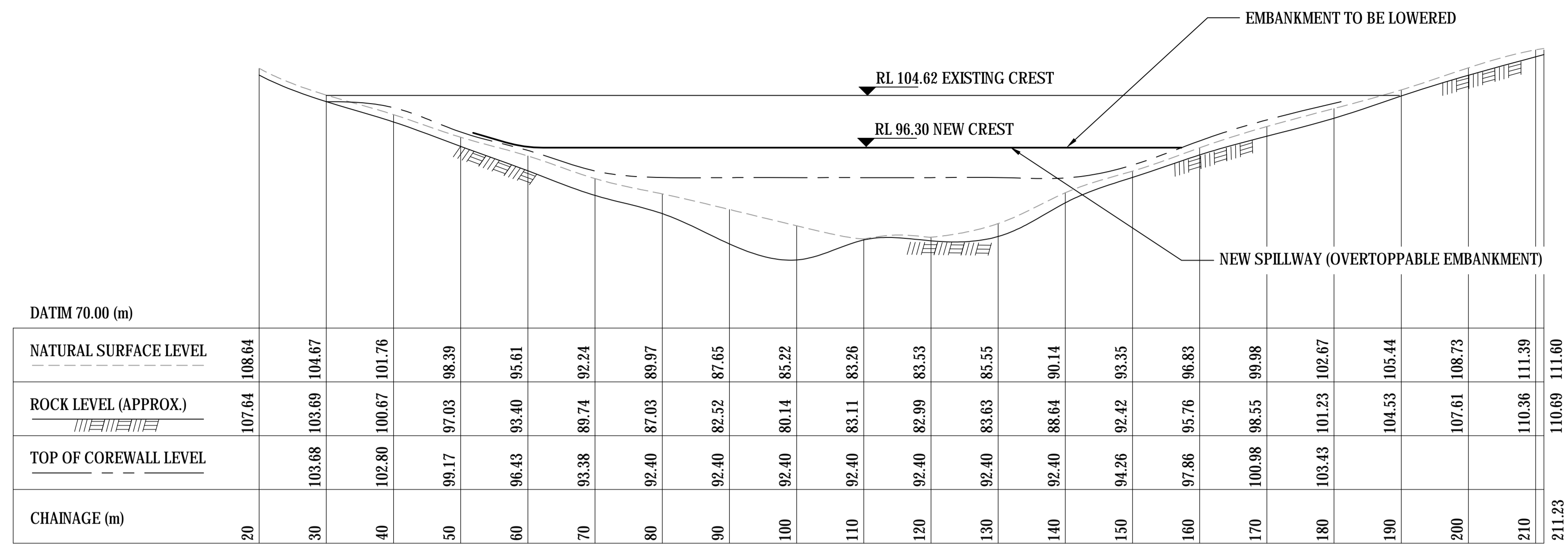
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DRAFTING CHECK	ENGINEERING REVIEW		

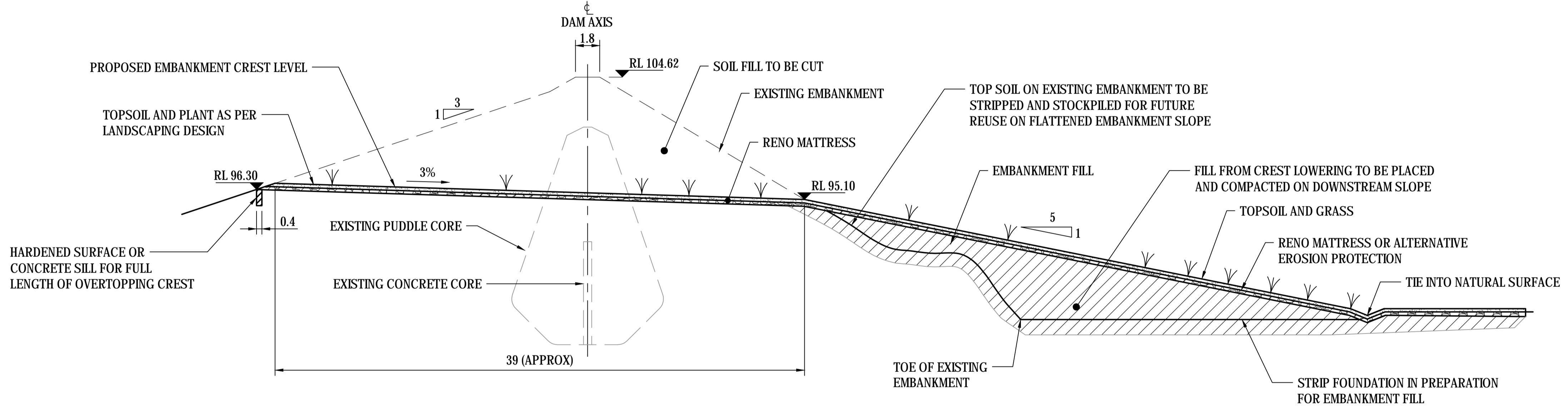
TITLE: BEACONSFIELD RESERVOIR
CONCEPT DESIGN
OPTION B - PLAN

PROJECT DATUM	Original Size A1	MELBOURNE WATER CORPORATION	
SCALE AS SHOWN	C	MW Drawing No	B
	CODE	MWC DRAWING NUMBER	REV

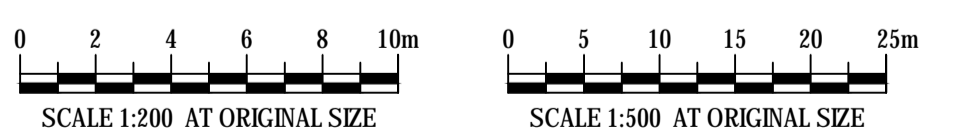
MW Drawing No
MVC DRAWING NUMBER
CODE



DAM AXIS LONG SECTION - LOOKING UPSTREAM
SECTION A
C014 SCALE 1 : 500



TYPICAL EMBANKMENT LOWERING SECTION
SECTION B
C014 SCALE 1 : 200



PRELIMINARY

REV	DESCRIPTION	COMPANY	PROJECT OR WO NUMBER	DRAWN	ENG. CHECK	PR. MAN. APPD.	DATE
B	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	28.10.2019
A	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	25.02.2019

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GHD DRAWING NUMBER
31-36304-C015

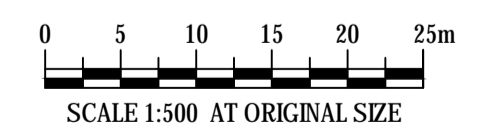
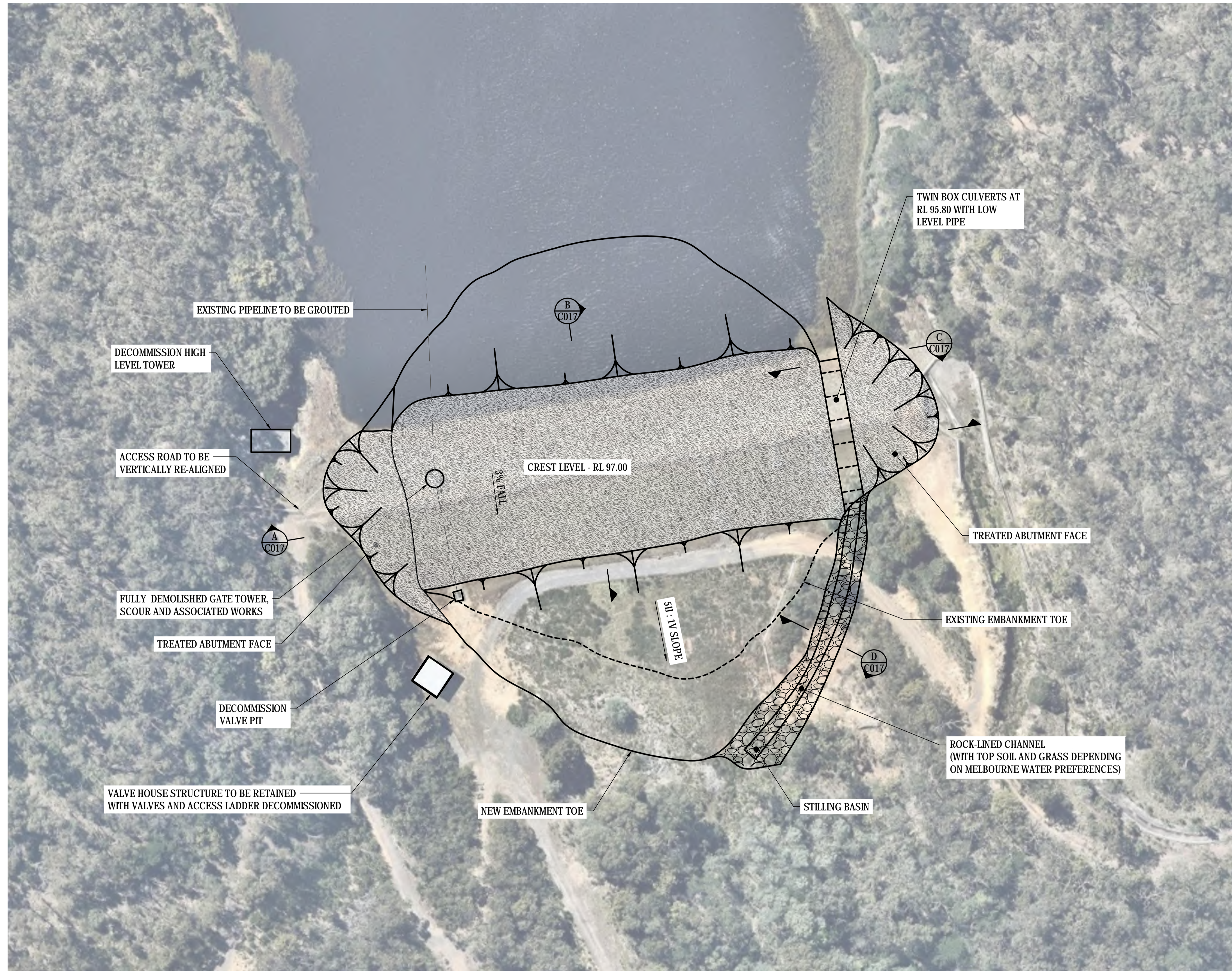
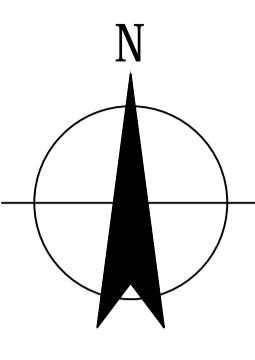
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DRAFTING CHECK	ENGINEERING REVIEW		

TITLE: BEACONSFIELD RESERVOIR
CONCEPT DESIGN
OPTION B - SECTIONS

PROJECT DATUM	Original Size A1	MELBOURNE WATER CORPORATION	
SCALE AS SHOWN	C	MW Drawing No	B
	CODE	MVC DRAWING NUMBER	REV



PLAN
SCALE 1:500

PRELIMINARY

REV	DESCRIPTION	COMPANY	PROJECT OR WO NUMBER	DRAWN	ENG. CHECK	PR. MAN. APPD.	DATE
B	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	28.10.2019
A	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	25.02.2019

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GHD DRAWING NUMBER
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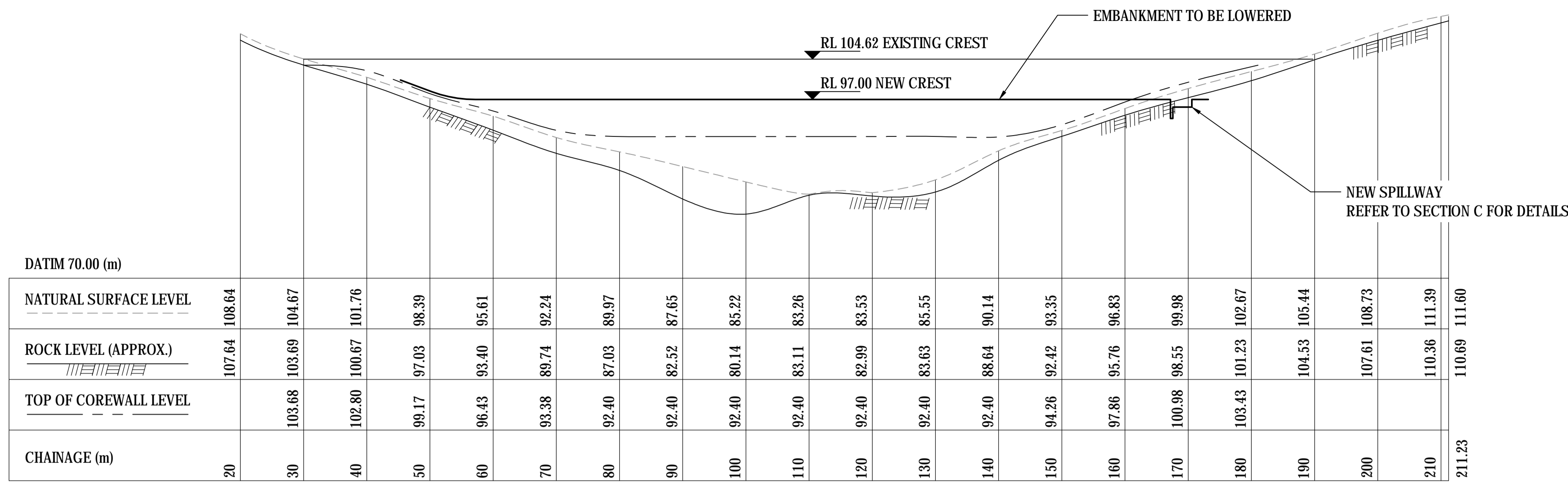
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DRAFTING CHECK	ENGINEERING REVIEW		

TITLE: BEACONSFIELD RESERVOIR
 CONCEPT DESIGN
 OPTION C - PLAN

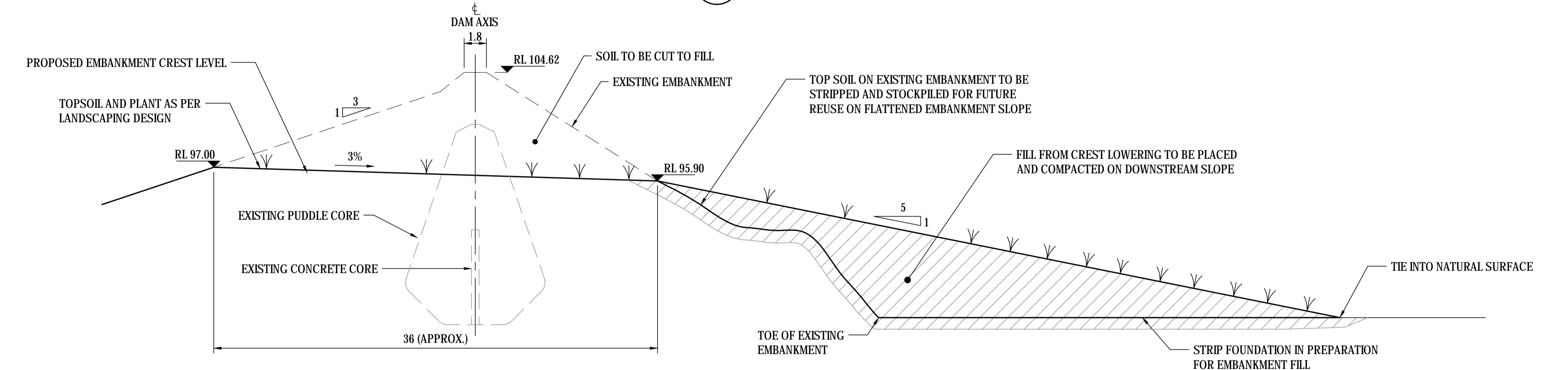
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SCALE AS SHOWN	C	MW Drawing No	B
	CODE	MWC DRAWING NUMBER	REV

Plot Date: 28 October 2019 - 1:45 PM



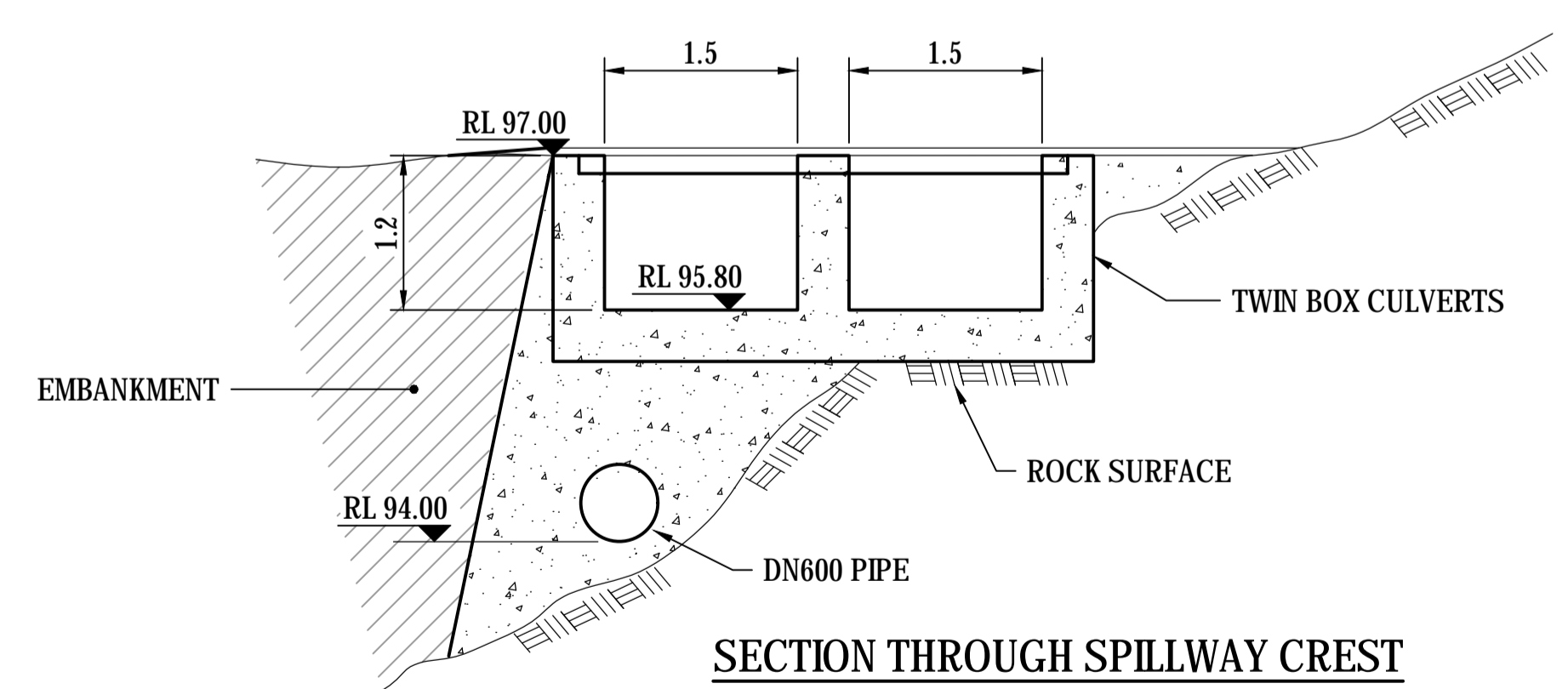
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A SECTION
C016 SCALE 1 : 500



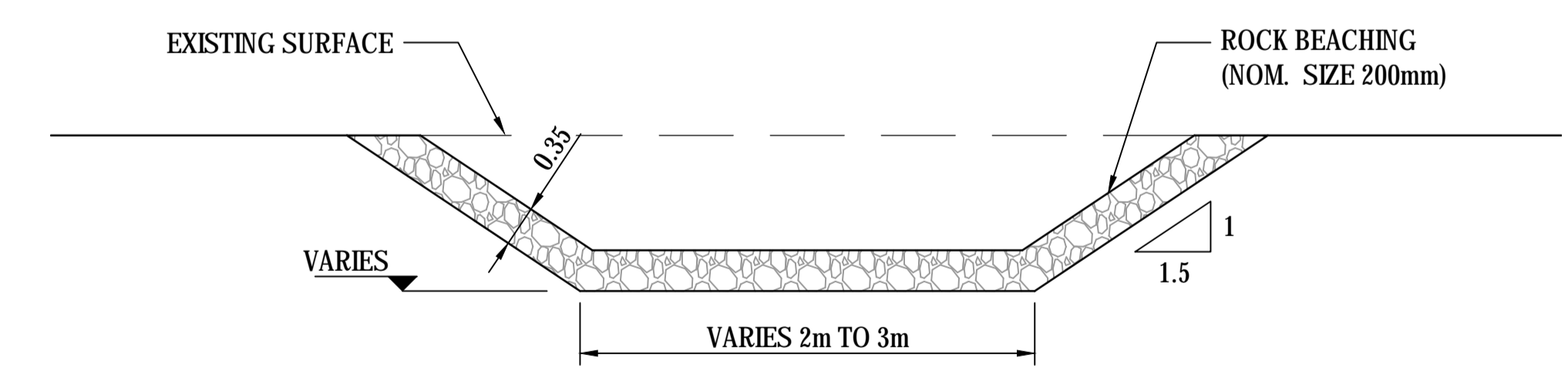
TYPICAL EMBANKMENT LOWERING SECTION

B SECTION
C016 SCALE 1 : 200



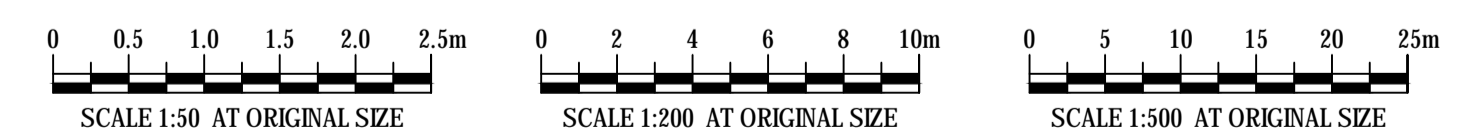
SECTION THROUGH SPILLWAY CREST

C SECTION
C016 SCALE 1 : 50



ROCK LINED CHANNEL TYPICAL DETAIL

D SECTION
C016 SCALE 1 : 50



PRELIMINARY

REV	DESCRIPTION	COMPANY	PROJECT OR WO NUMBER	DRAWN	ENG. CHECK	PR. MAN. APPD.	DATE
B	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	28.10.2019
A	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	25.02.2019

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DRAFTING CHECK	ENGINEERING REVIEW		

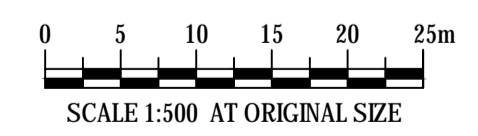
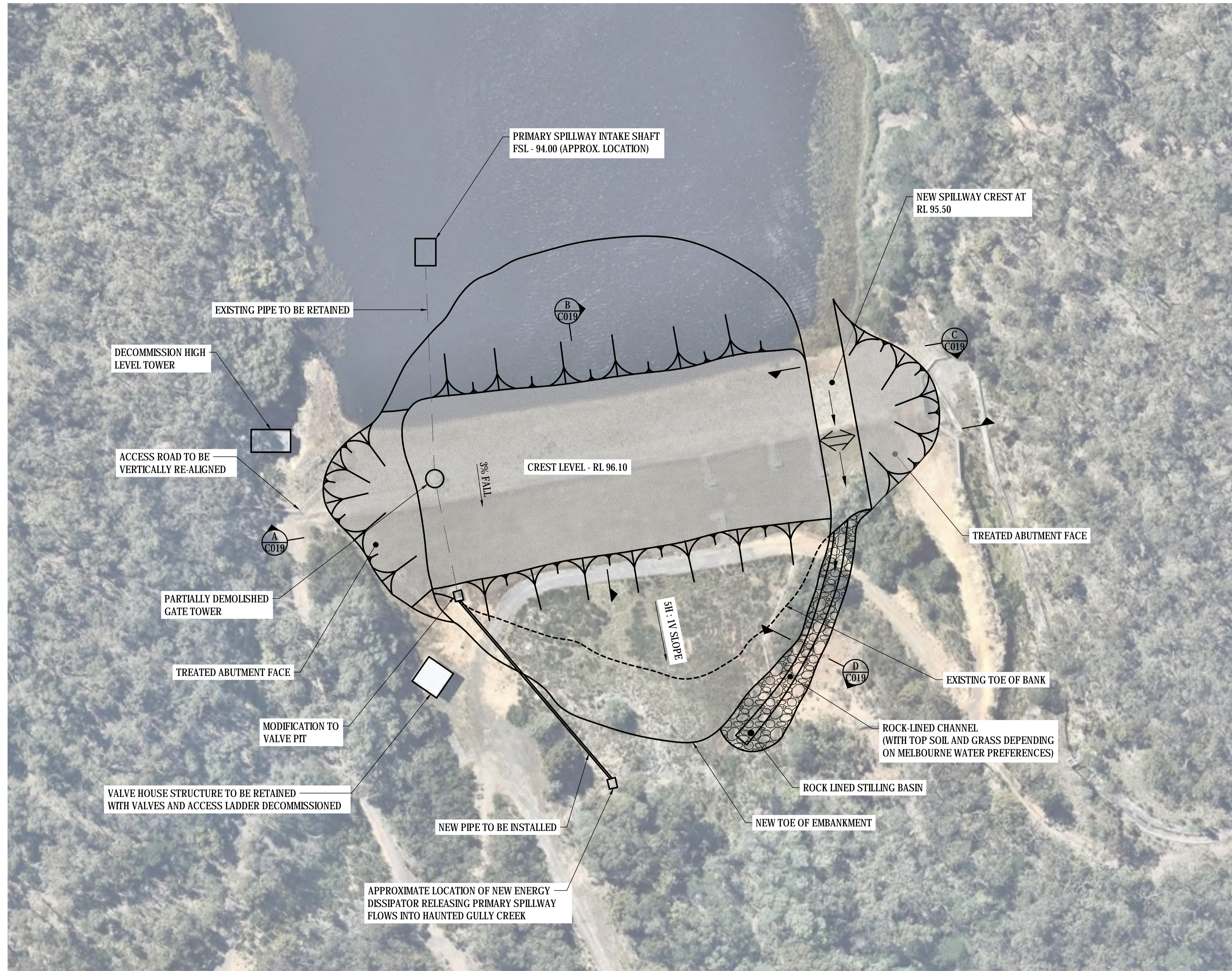
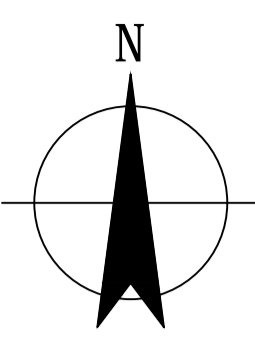
TITLE BEACONSFIELD RESERVOIR
CONCEPT DESIGN
OPTION C - SECTIONS

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SCALE AS SHOWN	C	MW Drawing No	B
	CODE	MWC DRAWING NUMBER	REV

REV	DESCRIPTION
A	DRAFT UPGRADE CONCEPT DESIGN

CODE	

1	2	3	4	5	6	7	8	9	10	11	12
A	B	C	D	E	F	G	H				



PLAN
SCALE 1:500

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REV	DESCRIPTION	COMPANY	PROJECT OR WO NUMBER	DRAWN	ENG. CHECK	PR. MAN. APPD.	DATE
A	DRAFT UPGRADE CONCEPT DESIGN	GHD		BW		CB	28.10.2019

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GHD DRAWING NUMBER
31-36304-C018

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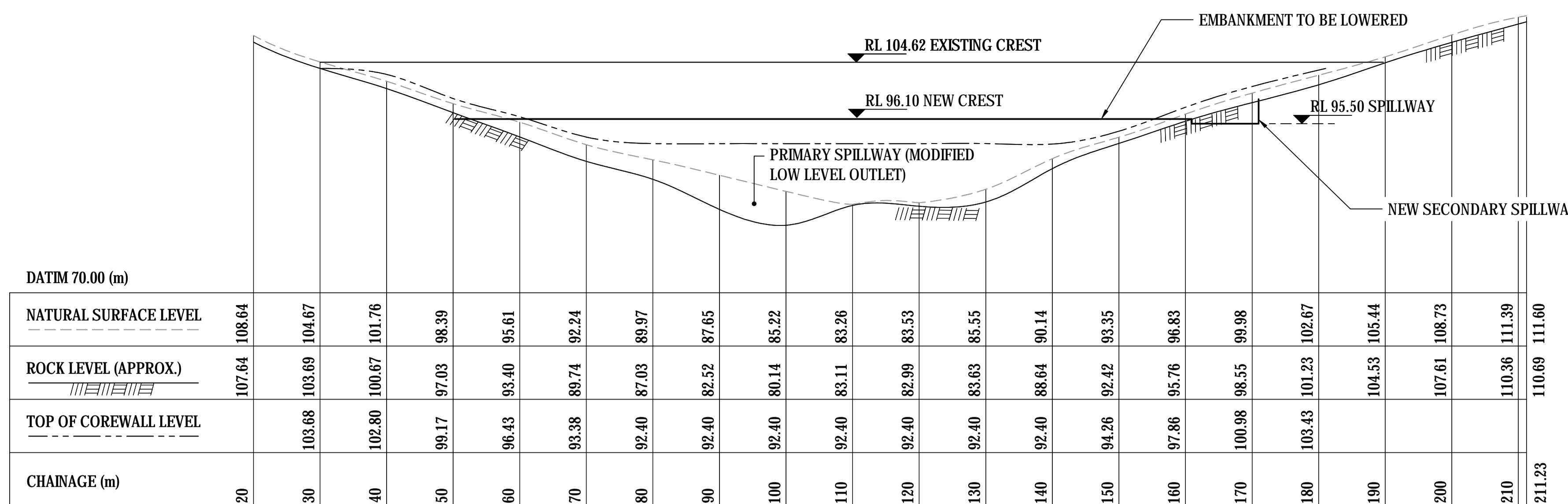
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DRAFTING CHECK	ENGINEERING REVIEW		

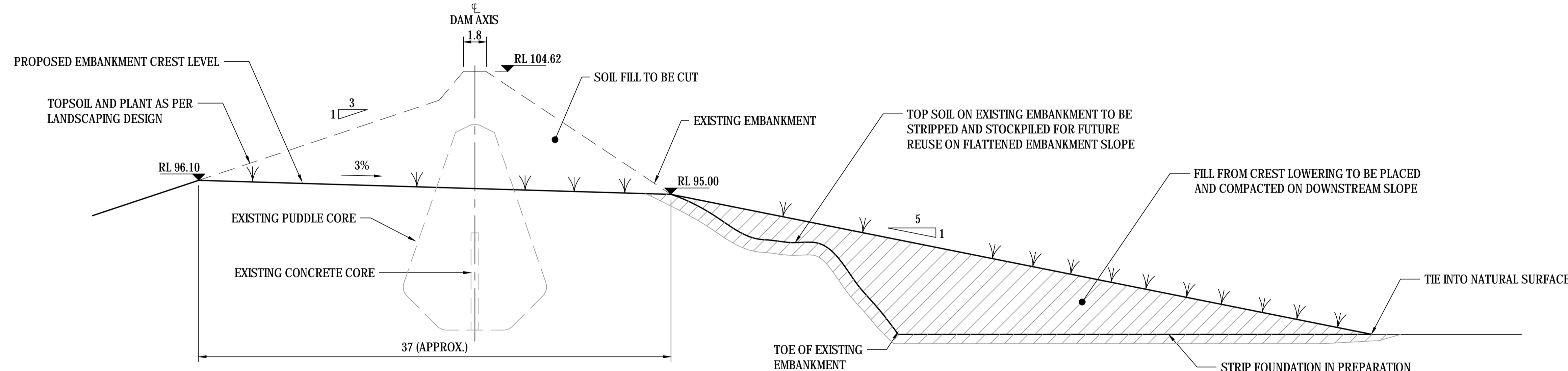
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Plot Date: 28 October 2019 - 1:46 PM



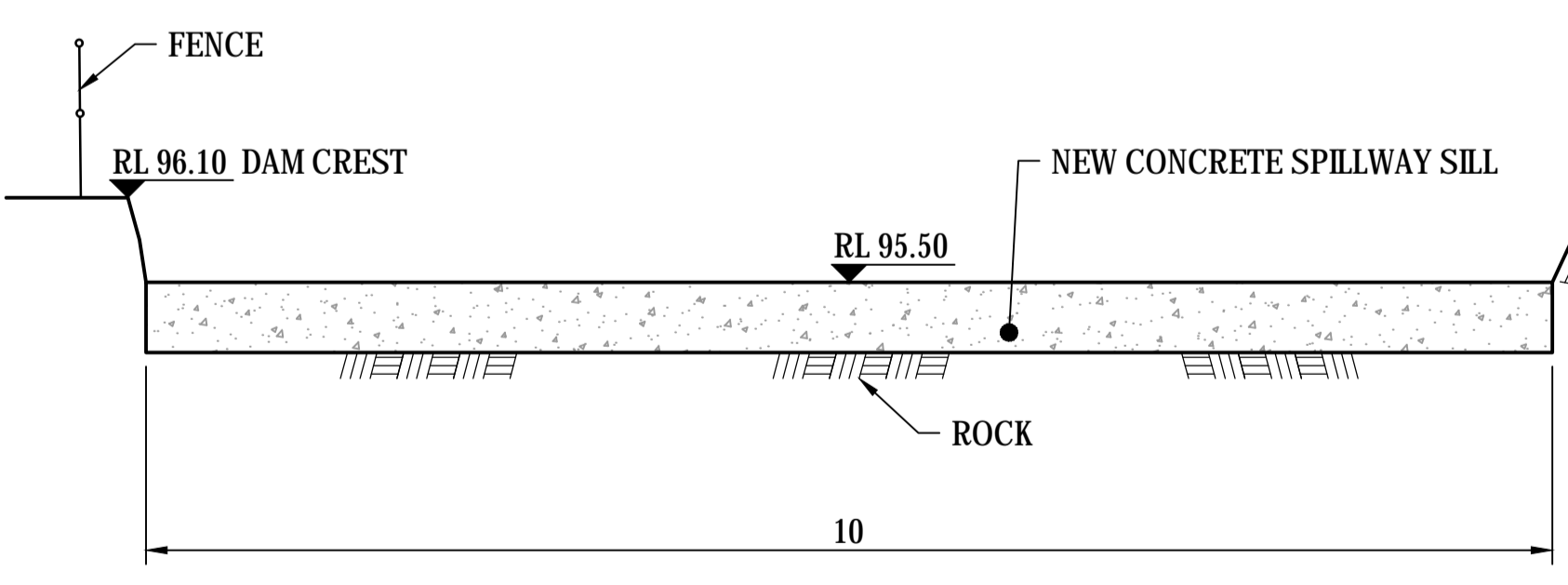
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A SECTION
C018 SCALE 1 : 500



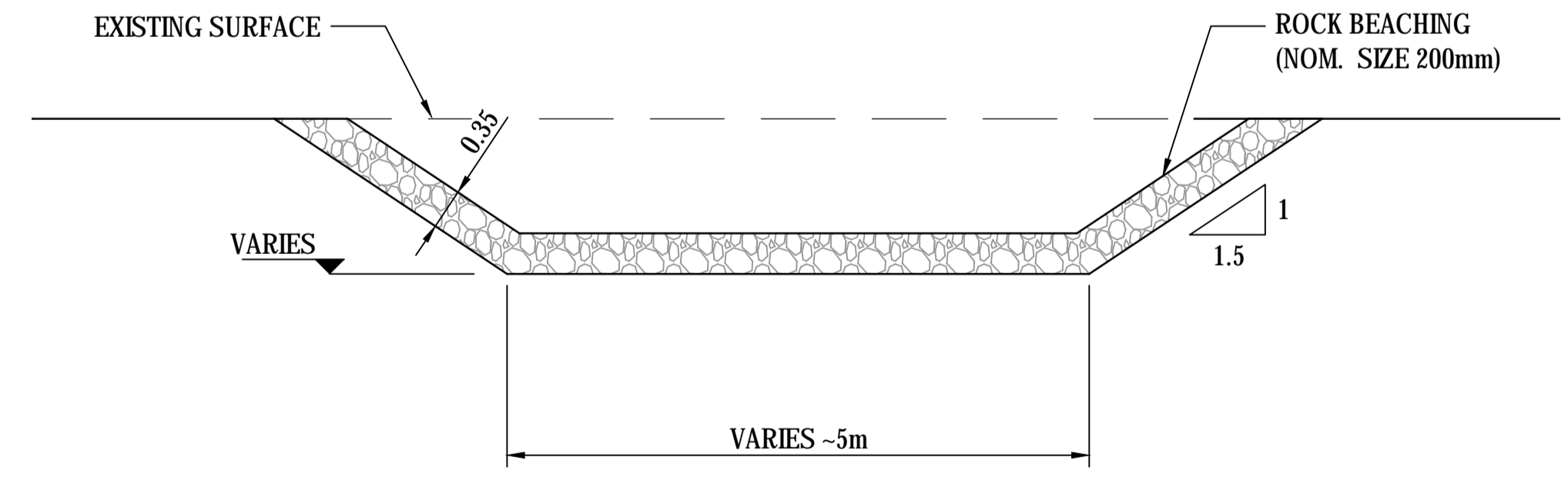
TYPICAL EMBANKMENT LOWERING SECTION

B SECTION
C018 SCALE 1 : 200



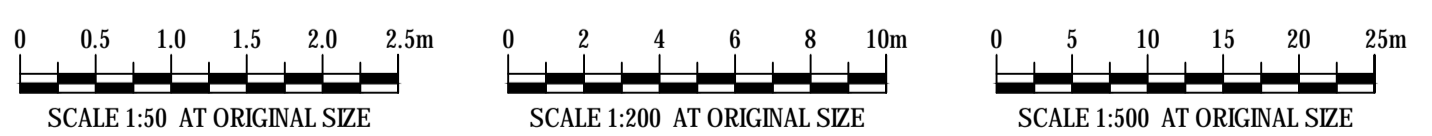
SECTION THROUGH SPILLWAY CREST

C SECTION
C018 SCALE 1 : 50



ROCK LINED CHANNEL TYPICAL DETAIL

D SECTION
C018 SCALE 1 : 50



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REV	DESCRIPTION	COMPANY	PROJECT OR WO NUMBER	DRAWN	ENG. CHECK	PR. MAN. APPD.	DATE					GHD DRAWING NUMBER	DRAFTER	DESIGNER	DESIGN MANAGER APPROVAL	PROJECT MANAGER APPROVAL	SCALE AS SHOWN
1								31-36304-C019	B. WAI	S. TAUBERT			AS SHOWN				

Appendix J – CAPEX cost estimates

Option 1A

Option 1B

Option 1C

Option 1D

Option 1A

Option 1B

Option 1C

Option 1D (recommended concept design)

Appendix K – RANE cost estimates

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RANE Template - Output

Beaconsfield Reservoir Concept Design

3136304



Appendix L – RANE risk estimates

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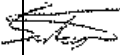

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<https://projects.ghd.com/oc/Victoria1/beaconsfielddaminspe/Delivery/Documents/3136304-REP-1-Beaconsfield Reservoir Concept Design Report.docx>

Document Status

Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
0	T Solomon M Medwell Squier	S Taubert		C Baker		20/11/2019
1	T Solomon M Medwell Squier C Young	S Taubert		C Baker		18/12/2019

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