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MATAKANUI GOLD LIMITED
BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS SILT POND
TECHNICAL REPORT

Prepared for: 08 August 2025

Matakanui Gold Limited

EXECUTIVE SUMMARY

This technical report presents the proposed design of the Shepherds Silt Pond. The silt pond has been sized to provide 30,000m^3 of back up process water and sediment retention control for a 179 ha area of the Shepherd Engineered Landform and adjacent disturbed area with haul roads and topsoil stockpiles. Under normal operating conditions the water level within the impoundment is 538.5 mRL The silt pond allows passage of runoff associated with a 1 in 1,000 year Annual Exceedance Probability (AEP) rainfall event.

Shepherds Silt Pond will be formed by an earth embankment dam with a proposed crest of approximately 543 mRL. Downstream the embankment will be buttressed by the infill of the valley floor. The top level of the buttress fill is approximately 538 mRL. The effective height of the embankment dam will be 5 m. The depth of the Valley Infill at the downstream toe of the embankment is approximately 16 m. Within the reservoir the floor is approximately 527 mRL. The upstream slope height is approximately 16 m.

The design incorporates two spillways. The primary spillway consists of decant tower and outlet pipe. The top of the decant tower extends to 540.5 mRL. The primary spillway (decant tower) is designed to retain sediment and pass a 1 in 10 year AEP rainfall event. Perforations in the decant tower between 538.5 and 540.5 mRL allow the water level in the impoundment to return to normal operating conditions within 3 days. The auxiliary spillway consists of a broad crested weir and spillway channel located at 541 mRL in the rock making up the true right hand abutment of the silt pond. The auxiliary spillway allows passage of a 1 in 1,000 year AEP rainfall event, and allows for the situation where the primary spillway is blocked and the northern diversion drain has failed, increasing the catchment area from 179 ha to 634 ha. At the maximum inflow design flood the dam will impound approximately 75,000 m^3 and water level reach 542.1 mRL

Dam breach assessment has been undertaken and Shepherds Silt Pond is assessed as a Low Potential Impact Classification dam. The design, construction, operation, maintenance, surveillance and closure will be undertaken in accordance with the NZDSG (Ref. 5), the Building Act (Ref. 2), and the Building (Dam Safety) Regulations (Ref. 20, 21). Further foundation investigation and detailed design is required as part of the Building Consent, which is required from Environment Canterbury (the relevant Building Consent Authority for dams) prior to construction.

Measures for mitigating potential adverse effects associated with the proposed Shepherds Silt Pond are summarised below:

1. The design, construction, operation, maintenance, surveillance, and closure of the Shepherds Silt Pond will be in general accordance with the NZDSG (Ref. 5).
2. The detailed design will be undertaken by a suitably qualified engineer with experience in dam design and construction.
3. Detailed geotechnical investigation and design will be undertaken, and a Building Consent will be applied for from Environment Canterbury.
4. The dam construction will be managed by personnel experienced in dam construction.
5. Shepherds Silt Pond will be operated under the site Pond and Water Reservoir Management Plan and have a specific Operation, Maintenance, and Surveillance Manual and Emergency Action Plan.
6. Erosion and sediment controls will be detailed in the Erosion and Sediment Control Plan for the construction of silt pond area.

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Document applicability and disclaimers

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This report shall only be read in its entirety.

Where this report is issued in draft the contents shall be for initial information and review only and are subject to change and shall not be relied upon.

The author of this report acknowledges that this report will be relied on by a Panel appointed under the Fast Track Approvals Act 2024 and these disclaimers do not prevent that reliance.

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MATAKANUI GOLD LIMITED
BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS SILT POND
TECHNICAL REPORT

1.0 INTRODUCTION

Engineering Geology Limited (EGL) was engaged by Matakanui Gold Limited (MGL) to provide this technical report on the Shepherds Silt Pond for the Bendigo-Ophir Gold Project (BOGP). MGL are proposing to establish the BOGP, which comprises a new gold mine, ancillary facilities and environmental mitigation measures on Bendigo and Ardgour Stations in the Dunstan Mountains of Central Otago.

The BOGP involves mining the identified gold deposits at Rise and Shine (RAS), Come in Time (CIT), Srek (SRX) and Srek East (SRE). Both open pit and underground mining methods will be utilised within the project site to access the gold deposits. Infrastructure to support the project will be constructed in the lower Shepherds Valley.

The purpose of this report is to outline the proposed design, construction, operation, maintenance, surveillance, and the risks and mitigations for the Shepherds Silt Pond.

This technical report has been prepared for Fast Track Approval (Ref. 1). Detailed design will be required under the Building Act (Ref. 2).

2.0 LOCATION AND SITE DESCRIPTION

The project site is located approximately 20 km northeast of Cromwell. The RAS and CIT gold deposit are located within a ridge between Shepherds Creek to the northeast and RAS Creek to southwest. Shepherds Creek has a single named tributary know as Jean Creek. The Srex gold deposit is located on the southern slopes of Rise and Shine Valley. Watercourses in both valleys flow from a divide in the southeast to outlets in the northwest.

The general location of the proposed site is shown in Figure 1. The location of the Shepherds Silt Pond relative to the other mine site features is shown in Figure 2. It is required for erosion and sediment control for Shepherd ELF (Ref. 3). It is located in the Shepherd Valley floor at the toe of the proposed Shepherds ELF, across Shepherds Creek. Between the toe of Shepherds ELF and the Shepherds Silt Pond reservoir i.e. pond, is the Shepherds Seepage Collection Sump (Ref. 4). This sump is a HDPE lined pond and is for the management of seepage collected in the Shepherds TSF and Shepherds ELF underdrains. The details of this sump can be found in the Shepherds ELF Report (Ref. 4).

3.0 DESIGN CRITERIA

The design has been based on the following criteria:

- A silt pond is required for the management of sediment laden water off the Shepherds ELF (Ref. 4). The catchment area allowed for is 179 ha. This includes Shepherd ELF footprint and the catchments disturbed by haul roads and stockpiles on the southern side of the ELF up to the ridgeline, as shown on Figure 13.

- The dam has been assessed as having a potential to be a Low Potential Impact Classification (PIC) following the NZDSG (2024) (Ref. 5) criteria. See Section 5.0 and Appendix A.

- Provide 30,000 m^3 of water storage for back up process water.

- Provide 5,500 m^3 of dead storage for sediment retention.

- Provide sediment retention control for up to a 1 in 10 year storm event with inflows from the Shepherds ELF (Ref. 3).

- Decant tower outlet pipe (primary spillway) sized to pass 1 in 10 year AEP rainfall events.

- Silt pond spillway (auxiliary spillway) sized to pass Inflow Design Flood (IDF) with 1 in 1,000 year AEP.

- Buttressing with the Valley Infill downstream.

- Dam, spillway and associated structures shall be designed, constructed, operated and maintained for the life of the dam in accordance with the general principles of NZDSG (2024) (Ref. 3).

4.0 SITE GEOTECHNICAL INVESTIGATION AND GEOLOGY

The silt pond is located in the Shepherds Valley across Shepherds Creek. Figure 03 shows the Shepherds Silt Pond site.

Geological and geomorphology maps of the wider Shepherds Silt Pond area are shown in Figure 11 and 12.

The Shepherds Silt Pond area is in Textual Zone III (TZ3) schist. TZ3 schist is undifferentiated pelitic and psammitic schist and greenschist sequences and is locally weathered to a silty gravel for a shallow depth at the surface. Shear zones and faults are interpreted to be present within the TZ3 schist (Figure 11). See the site geotechnical factual report for further information on geology of the site (Ref. 6).

High level geological mapping indicates that the side slopes of the Shepherds Valley have areas of displaced schist rock masses. High-level mapping on Figure 12 indicates potential for displaced masses on both the north and south slopes. For the northern slope there is potentially an intact ridge aligned north to south approximately 100 m downstream of the toe of the Silt Pond Embankment. At the Silt Pond Embankment position the topography of the north slope protrudes into the valley causing a bend in the creek. This could be an intact ridge or be part of a displaced block of schist. In summary there is potential for open defects in the schist which may require an engineering solution. Further investigation will be required for detailed design to confirm the nature of the rock abutments at embankment and within the impoundment. This could include boreholes and permeability testing of the abutments and valley floor. See Section 6.0 for further details on proposed investigation and design aspects.

Four test pits (MTP029, MTP030, MTP031, MTP032) were undertaken in alluvial materials in the upper part of the pond area and beneath the Shepherds Seepage Collection Sump (Figure 11 and 12). The area of test pitting is at the base of a gully on the north slopes which is within an ancient, displaced rock mass. The thickness of alluvial material in this area is likely to be greater than the embankment location which is narrowly constrained. In this area the test pits indicate 2.1 m (MTP029), 3.7 m (MTP030), 2.2 m (MTP031) to TZ3 schist. The fourth test pit (MTP032) was only 1 m deep. However, it is unclear if this terminated on the top of the schist. See the site geotechnical factual report for test pit logs and photos (Ref. 6).

The typical soil profile on the valley side slopes at the silt pond is expected to comprise a thin layer of topsoil overlying the schist bedrock. It is possibly that in locations there is localised pockets of sandy silt (loess). Colluvium and/or alluvial deposits are present in the base of the valley floor. Deposits are generally boulders, cobbles, gravels, sands and silts derived from the weathering and erosion of the schist rock slopes. The depth to TZ3 schist at the embankment is expected be in the order of 2.0 to 5.0 m in the middle of the valley floor. Specific investigations at detailed design stage will confirm depth to TZ3 bedrock.

5.0 POTENTIAL IMPACT CLASSIFICATION

The NZDSG (Ref. 1) provides recommendations for the design, construction, and operation of dams. They are dependent on the PIC of the dam. The PIC of a dam reflects the potential consequences of a hypothetical scenario failure on people, property, infrastructure, and the environment. Assessment of the PIC considers various factors including Population at Risk (PAR), Potential Loss of Life, damage to houses, infrastructure, environment, and community recovery time.

A comprehensive dam breach analysis has been conducted to determine the PIC in accordance with the NZDSG (Ref. 1). This is used to set appropriate design, construction, operation, maintenance and surveillance criteria for the dam.

The PIC of the proposed Shepherds Silt Pond is assessed to be **Low** based the NZDSG (Ref. 5). A summary of the dam breach analysis, assessed damage levels, and PIC assessment is summarised in Appendix A.

6.0 DESIGN

The proposed design for the Shepherds silt pond is shown on Figures 03 to 05 and 07 to 10.

Description of the proposed design are provided in the following sections.

6.1. Pond design for water storage and sediment retention

Shepherd Silt Pond is required to provide sediment retention control for the management of sediment-ladened surface water runoff from the Shepherds ELF and back up process water storage.

6.1.1. Sediment retention control

The sediment retention control function is designed following the general principles in the ICEA (Ref. 7) Best Practice Erosion and Sediment Control for Type C sediment basins. See the BOGP Erosion and Sediment Control Report (Ref. 3) for more details.

The requirement for Type C sediment basins is less than 33% of soil is finer than 0.02mm and no more than 10% of soil is dispersive. The site soils and overburden rock placed in the ELF meet the criteria (Ref. 3).

The calculation approach for Type C sediment basins (Ref. 7) allows sufficient settling time for the sediment to drop out of suspension over the depth of the settling zone. The method is depth independent and is governed by the area of the sediment settling zone. However, the settling zone is required to be limited to 0.6 to 2.0 m depth to achieve laminar flow across the pond.

Using the ICEA Type C sediment basins (Ref. 7) calculation the required surface area for the sediment settling zone is 11,500 m^2. This calculation assumes a 179 ha catchment, with a peak inflow of 5.6 m^3/s, a minimum length to width ratio of 6 to 1 (which is naturally achieved in the valley), and concentrated overland inflow into the pond. See Table 10 for a summary of the design parameters for sediment retention. At the decant top level of 540.5 mRL the area is 11,300 m^2 (See Table 10), which is close to the minimum requirement (difference of 2%). Final refinement of levels will be confirmed in detailed design.

The decant tower is designed such that the water level will build up during a storm event and overtop and flow into the decant tower at 540.5 mRL. The concept being that water decanted off the surface has sufficient time in the pond that sediment has fallen to the bottom of the pond. Over the depth of the top 2 m of the decant tower, termed the sediment settling zone, decant holes are included to allow water post event to slowly be released. With the limited decant hole depth and the notable water storage volume there is little potential for short cutting of flow that can sometimes be a concern for silt ponds treating more clayey soils. The risk of short cutting is low due to the nature of the soils at this site which settle readily. The decanting holes allowed for achieve draw down of the water in this zone over 3 days. EGL consider 1 to 5 days to be an appropriate range for this draw down period. Note this 2 m settling zone depth assists with attenuating peaks for short duration high flow events.

Allowance for 5,500 m^3 of sediment retention is included. EGLs experience at the Macraes Gold Mine Operation in Otago, in similar schist terrain is there is very little sediment reaching the silt ponds i.e. sediment drops out before it reaches the pond. The allowance of 5,500 m^3 is 8% of the total runoff volume from a 24 hour storm. EGLs experience is this is well sufficient. The water storage volume provides notable redundancy if required.

6.1.2. Back up process water storage

The proposed backup process water storage volume is 30,000 m^3. This volume is allowed for below the level of the sediment settling zone and above the dead storage volume for sediment retention. See Figure 04 and 07.

The backup process water volume will be drawn by a separate gravity outlet or decant pumps. This detail will be confirmed as part of detailed design. The top of the water storage volume is set at 538.5 mRL and is control by the lowest decant hole in the sediment settlement zone depth.

6.1.3. Storage volume

The elevation storage curve data is shown in Table 3. At the maximum inflow design flood level the volume of water impounded is approximately 75,000 m^3. Crest full the maximum volume is approximately 90,000 m^3. The potential volume impounded by the embankment section above the Valley Infill level is at a maximum 72,000 m^3 (to crest full). This is the volume assumed for the Rainy Day dam breach case.

6.2. Embankment

The embankment in plan and cross section is shown in Figures 03 and 04. The embankment has a crest length of approximately 50 m and a maximum downstream height of 5 m. It can impound water up to a depth of 16 m to the crest. However, under inflow design flood levels the maximum depth of the water is 15 m.

The embankment has a low permeability central core (Zone A1) which also extends as a blanket beneath the upstream shoulder. The upstream and downstream shoulders are constructed from rockfill (Zone B and B1). Zone B1 is positioned immediately downstream of Zone A1 core. Further downstream shoulder the downstream slope is constructed from rockfill (Valley Infill).

The embankment profile includes provision for potential difficult ground conditions which may require engineered solutions. This currently includes a wide crest and buttressing with the Valley Infill. If the foundation rock is not of sufficiently low permeability, protection of the core with slush grouting and/or a grout curtain may be required.

The embankment has upstream and downstream shoulder slopes of no steeper than 2H:1V.

Construction of the embankment will require approximately 38,000 to 47,000 m^3 of fill. The volume to the proposed surface from the existing ground is 38,000 m^3. The volume depends on the depth of undercut. The value of 47,000 m^3 assumes approximately 3 m undercut on average under Zone A1 and B and 1m over the rest of the downstream part of the embankment.

The zoning is shown in Figure 04 and is summarised below:

Zone A1

The primary function of this zone is to limit seepage. It also provides sufficient strength to prevent the likelihood of instability, particularly when subject to the design seismic loads. The low permeability Zone A1 can be sourced from mining waste or locally borrowed weathered schist supplemented with loess and colluvium as necessary. Additional weathered schist can be obtained from mining operations if necessary. Zone A1 will be placed in 0.35 m loose layers and subjected to compaction.

Zone A1 requires heavy compaction and conditioning (grubbing with tines) to achieve the specified permeability ($<10^{-7}$ m/s).

Zones B

Zone B is a structural fill zone placed in 0.6 m loose layers and subjected to compaction.

Zone B1

Zone B1 is structural fill placed between Zone A1 and Zone B to provide a transition zone. It has an intermediate particle size distribution, more suitable for filter compatibility, between the two fill types. Zone B1 is specifically selected, or reworked, to include a higher proportion of fines, sand and gravel, and a smaller maximum rock size than Zone B.

Valley Infill

2.5 m lift heights where in the vicinity of the embankment and spillway.

6.3. Embankment stability

6.3.1. Recommended design standards

NZDSG (Ref. 5) criteria for stability have been adopted. For static stability a FOS of 1.5 shall be achieved. An unbuttressed profile has been analysed. This is conservative.

For assessing stability under earthquake loads, two levels of shaking have been considered. The lower level known as the Operating Basis Earthquake (OBE) is based on the 1 in 150 AEP level of ground motion. Embankments should withstand this loading without significant damage (i.e. minor repairable damage permitted). The higher level known as Safety Evaluation Earthquake (SEE) has been taken equal to the ground motion associated with a 1 in 1,000 AEP. Some deformations and damage to the embankment are permitted but the contents retained by the embankment should not be released.

The OBE and SEE design earthquake ground motions are based on the Site-Specific Seismic Hazard Study Report undertaken by EGL for the Bendigo-Ophir Gold Project (Ref. 8).

6.3.2. Potential failure modes

As part of detailed design potential failure modes will be reviewed. Possible modes of failure include:

1. Instability of the embankment slopes
2. Bearing capacity type failure of the foundations
3. Instability or deformation associated due to earthquake shaking
4. Piping (internal erosion)
5. Loss of freeboard or overtopping

6.3.3. Analysis Procedures and Input Parameters

6.3.3.1. Analysis procedure

Stability of the embankment has been analysed using the two-dimensional SLOPE/W (Ref. 9) computer program applying a limit equilibrium calculation using the Spencer solution method (Ref. 10).

6.3.3.2. Phreatic surface

The phreatic surface through the upstream Zones B and A1 has been taken equal to the impounded pond water level, other than for the rapid drawdown case. The phreatic surface in the downstream Zones B1 and Valley Infill (noted as Zone C on stability sections) has been taken equal to the original ground level because the rockfill has a high permeability and will drain easily.

6.3.3.3. Shear strength parameters

The design shear strength parameters for the embankment fill and in-situ ground are summarised in Table 4. The seismic soil strengths have been taken equal to the static soil strengths on the basis that the materials forming the various embankment zones are unlikely to be susceptible to strength loss during earthquake shaking.

6.3.4. Stability Analyses

Stability of the silt pond embankment has been analysed for both static and seismic loading cases. The results of the analyses are summarised in Table 5. Results for the individual analyses are also included in Figures B1 to B8 in Appendix B.

Under static loading conditions, the stability analyses show that the Factor of Safety (FoS) is greater than the required minimum value of 1.5.

Seismic stability has been analysed for the OBE and SEE levels of earthquake shaking. Stability has been assessed for potential failure surfaces located H, 2/3H and 1/3H below the embankment crest, where H is the height of the embankment. Where the factor of safety is less than 1.0, yielding occurs and predicted permanent deformations were estimated using the Bray and Macedo method (Ref. 11). This method allows for the dynamic response of the potential failure sliding mass to be considered.

The Bray and Macedo method (Ref. 11) requires a spectral acceleration and mean magnitude for the fundamental period T of each slide mass and level of shaking (i.e. OBE and SEE). Selection of parameters are summarised in Table 6. The fundamental period was estimated by using $T = 2.6H/V_s$, in which H is the height of the slide mass and the average shear wave velocity Vs of the embankment. The Vs of the embankment was determined as the lower bound recommendation of Sawada and Takahashi (Ref. 12), i.e., Vs = 210 m/s for depth of 0 to 5 m and $140Z^{0.34}$ for depth greater than 5 m. The mean magnitude is based on the site-specific seismic hazard assessment (SSSHA) (Ref. 8). Allowance for site and topographic amplification from the base to the top of the embankment has been allowed considering by Park and Kishida (Ref. 13).

Under the OBE loadings, the FoS is greater than 1.0 indicating no or minor deformations, meeting design criteria.

Under the SEE loading, some minor permanent displacements of up to a maximum of 18 cm are estimated. The estimated deformations are minor and will not compromise the integrity of the embankment or result in a loss of freeboard leading to overtopping of the embankment, meeting design criteria.

6.4. Potential for Earthquake Induced Settlement and Cracking

Earthfill embankments subjected to very high levels of earthquake shaking can experience settlement, permanent deformations and cracking. We have assessed the potential settlement and cracking associated with earthquake loading using empirical methods.

Estimates of crack sizes (width and depth) for the OBE and SEE load levels have been based on Pells and Fell (Ref. 14) for longitudinal cracks and Fong and Bennett (Ref. 15) for transverse cracks. The Pells and Fell method require assessment of damage class which is dependent on the foundation PGA and earthquake magnitude. The damage class for OBE and SEE ground motion are characterised as Class 0 (no or slight damage) and Class 1 (minor damage), according to the Pells and Fell (Ref. 14) criteria for earth fill embankment dams. On this basis, the estimated crack widths and depths under OBE and SEE design ground motions are summarised in Table 7.

Transverse cracking is a notable risk to water retaining embankments because cracks provide an open pathway for water to seep through. Pells and Fell (Ref. 14) summarise the factors that affect the likelihood of cracking or hydraulic fracturing.

Factors that give rise to the greatest potential for the development of transverse cracking include:

- Overall abutment profile
- Small scale irregularities in abutment profile
- Differential foundation settlement

- Core characteristics
- Closure sections (during construction)

The lower permeability Zone A1 core and Zone B1 with downstream buttressing will serve to mitigate the potential of water seeping through the cracks risking the integrity of the embankment and a breach.

Following any significant earthquake event, it will be necessary to carry put a visual inspection of the embankment and repair any damage if evident.

6.5. Spillways

There are two spillways. The primary spillway is the decant tower and outlet pipe and the auxiliary spillway is the spillway (open channel) over the true right abutment.

The decant tower and the outlet pipe are shown in Figures 03, 04, 05, 07.

The spillway (open channel) is shown in Figures 03, 08, 09, and 10.

6.5.1. Primary Spillway

The proposed decant outlet (primary spillway) from the pond is a concrete manhole structure with an HDPE outlet pipe. The tower is perforated over the top 2 m to allow detention of inflows, throttling back outflows through the outlet pipe.

The top of the manhole is 2.5 m below the crest of the embankment. There are ten rows of holes uniformly spaced around the perimeter of the manhole. Armour rock (riprap) is required around the manhole structure to provide support over the lower portion of the manhole and counteract buoyancy forces. The decant manhole consists of 1.05 m diameter manhole risers bolted together and connects an outlet pipe beneath the upstream shoulder of the embankment. An access way provides restraint and access to the top of the decant tower. The proposed outlet pipe consists of an 800 OD SDR17 PE100 pipes.

The pipe is in rock on the true left abutment slope through the Zone A1. It then transitions to going through the embankment in the Zone B1, B and Valley Infill. Through the Zone A1 the pipe is bedded on reinforced concrete. Downstream through the Zone B1, B and Valley Infill the outlet pipe is bedded on and surrounded by drainage material.

6.5.2. Auxiliary Spillway

The auxiliary spillway is located through the rock making up the true right-hand abutment as shown in Figures 03 and 08. It consists of a broad crested weir and channel excavated into the schist rock that underlies this area.

The weir and channel have a base width of 12.0 m designed with side slopes of 1.0H:1.0V. The spillway weir inlet level is 2.0 m below the crest of the embankment. The inlet weir control is reinforced concrete. The rock may be sufficiently hard and durable to not require protection. If not spray concrete lining with hold down anchors and underdrains will be required on the chute. Armour rock will be placed at the end of the auxiliary spillway in a stilling basin arrangement to ensure dissipation of energy associated as water transitions from the spillway chute to the valley floor filled with natural sediments.

6.5.3. Hydraulic Routing

In a 1 in 10 year flood event the pond will rise from the water storage level (538.5 mRL), then spill over the top of the tower (at 540.5 mRL) staying below the auxiliary spillway (541.0 mRL), and then draw down over 3 days returning to its start pre-storm level.

In a large storm event greater than a 1 in 10 AEP water will flow over the auxiliary spillway (541.0 mRL). Hydraulic Routing has been undertaken using the program HEC-HMS (Hydrological Modelling System from Hydrologic Engineering Centre, Ref. 16) to design and assess the performance of the spillways. Figures showing the flows and water levels routed through the decant tower and over the spillway, are summarised in Appendix C.

The silt pond decant tower holes and primary spillway are capable of the passing the flow from 10-year event on its own, as shown in Figures C3 to C9 and Table C3 in Appendix C.

The auxiliary spillway is able to pass the 1,000 year rainfall event with the primary spillway blocked and the Northern Diversion Drain failure into the silt pond. In this situation the total catchment reporting over the spillway is 634 ha. This is made up of 179 ha from the Shepherds ELF area and 455 ha from the slopes above the North Diversion Channel (Figure 13). The routing plots are shown in Figures C11 to C17 and Table C4 in Appendix C.

6.6. Wind and Wave Evaluation

6.6.1. Wind and Waves

Estimates of extreme wind speed have been obtained from NZS1170:2004 (New Zealand Structural Design Actions for Wind Loads) (Ref. 17).

The significant wave height (Hs) has been calculated using the procedures in ICOLD Bulletin 91 (Ref. 18). Wave set-up is insignificant for the size of the proposed reservoir. Wave set-up has been estimated for the highest 10% of the waves using the procedures recommended in ICOLD Bulletin 91 (Ref. 18). The results are summarised in Table 8.

6.6.2. Freeboard

The main factor controlling freeboard for the dam is a rise in water during flood conditions together with waves. The results of the freeboard under normal conditions and at a maximum level during an IDF event are summarised in Table 9.

Under normal operation reservoir level of 540.81 mRL (in a 1 in 10 year storm) the freeboard without wave run-up, setup and settlement are 2.19 m. Allowing for wave run-up and setup of 0.161 m (1 in 100 AEP wind speed) combined with the long-term settlement of the embankment of up to 0.085 m, the freeboard would be 1.944 m.

At maximum reservoir level of 542.027 mRL during a 1 in 1,000 AEP IDF, the freeboard without wave run-up, setup and settlement are 0.973 m. Allowing for wave run-up and setup of 0.142 m (1 in 10 AEP wind speed) combined with the long-term settlement of the embankment of up to 0.085 m, the freeboard would be 0.746 m.

Under both conditions sufficient freeboard is achieved.

7.0 DETAILED DESIGN AND BUILDING CONSENT

The design of the dam will be in accordance with the NZDSG (Ref. 5). The design will be undertaken by a suitably qualified engineer with experience in dam design and construction.

This report outlines a design for an assessment of effects. Detailed design is required for construction. Detailed geotechnical investigation of the foundations is likely required to confirm final design details.

Shepherds Silt Pond classifies as a large dam (greater than 4m high and volume over 20,000 m^3) under the Building Act (Ref. 2). It will therefore require a Building Consent, applied for through Environment Canterbury as the Building Consent Authority (BCA) for large dams for Otago.

As a Low PIC dam, peer review of the design is not required.

A detailed design report, specification and drawings will be required.

8.0 CONSTRUCTION

The construction of Shepherd Silt Pond will be in accordance with the NZDSG (Ref. 5).

Embankment construction will be undertaken by a contractor or MGL employees with experience in dam construction.

Material will be sourced from either the initial excavation of the Rise and Shine Pit, North Diversion Channel, stripping of the ELF and TSF footprints and local borrows within the silt pond impoundment.

8.1. Erosion and Sediment Control

A site specific erosion and sediment control plan (covering the Shepards Silt Pond area) prepared in accordance with the site Erosion and Sediment Control Management Plan (Ref. 23) will be required. This will be developed to fit the contractors or MGL proposed construction sequence.

Dust will be controlled with water trucks.

9.0 DAM SAFETY MANAGEMENT

Dam safety management on the BOGP will be in accordance with the NZDSG (Ref. 5), the Building Act (Ref. 2) and the Building (Dam Safety) Regulations (Ref. 20, 21).

9.1. Potential Impact Classification Certification

The Building Act (Ref. 2) and Building (Dam Safety) Regulations (Ref. 20, 21) set out the legally requirement that all classifiable dams (greater than 4m high and volume over 20,000 m^3) have a PIC certified by a Recognised Engineer (PIC) and submitted to the relevant Regional Council. For the BOGP this is the Otago Regional Council. The certification is required to be submitted by to Otago Regional Council 3 months after the dam is commissioned (Ref. 2).

The owner is required to review the dam’s classification every 5 years or at any time any building work is carried out that requires a building consent, or works could result in a change to the PIC (Ref. 2).

9.2. Dam Safety Management

The responsibility for the safety of the dam rests with the dam owner (Ref. 5).

A Dam Safety Assurance Programme (DSAP) or Dam Safety Management System (DSMS) provides dam owners with a structured framework of plans and procedures to plan and complete the activities required for the safe operation and management of their dams (Ref. 22). The term DSAP refers to the Building Act (Ref. 2) and Building (Dam Safety) Regulations (Ref. 20, 21), covers only seven of the 12 parts of a DSMS outlined in the NZDSG (Ref. 5). EGL recommend that all ponds and water reservoirs onsite are managed the Pond and Water Reservoir Management Plan (Ref. 24) which will function as a DSMS. Ponds and water storage facilities greater than 4m in height and 20,000 m^3 will have higher dam safety requirements than those of lesser volume and height.

As the Shepherds Silt Pond is a Low PIC dam there is no legislative requirement for a DSMS (Ref. 5) or a DSAP (Ref. 2) to be submitted to the Otago Regional Council.

EGL recommends Shepherds Silt Pond has a specific Operation, Maintenance and Surveillance manual and an Emergency Action Plan in addition to the Pond and Water Reservoir Management Plan. This is recommended as while it is a Low PIC dam it can impound a notable volume of water and has specific items of operation that need further detail. Dam Breach modelling has been undertaken which will inform the Emergency Action Plan.

If the PIC was to change to Medium or High PIC, most likely due to a change in downstream land use, a DSAP is required to be certified by a Recognised Engineer (DSAP) and submitted to the Otago Regional Council. Further annual certification reviewing compliance with the DSAP would be required from a Recognised Engineer (DSAP).

Visual inspections, dam safety reviews, deformation survey and seepage monitoring shall be undertaken at intervals recommended in the NZDSG (Ref. 5). Deformation monitoring points are recommended on the crest of the dam and abutments. Deformation monitoring points may also be recommended within the impoundment in detailed design depending on the detailed geotechnical investigation and construction.

10.0 COMMISSIONING

Prior to commissioning appropriate procedures will be prepared. They will include guidelines on the rate of filling, and recommendations for monitoring and surveillance. It will be necessary to undertake baseline measurements of all instrumentation (settlement markers and drains flow). The commissioning procedures will also provide actions to be taken in the event of a developing dam safety emergency.

11.0 CLOSURE

At closure the Shepherds Silt Pond will be decommissioned or repurposed following the NZDSG (Ref. 5).

12.0 POTENTIAL RISKS AND MITIGATIONS MEASURES

Measures for mitigating potential adverse effects associated with the proposed Shepherds Silt Pond are summarised below:

1. Shepherds Silt Pond has been sized to provide 30,000 m^3 of back up process water storage and sediment retention control for runoff from up to 179 ha of Shepherd Engineered Landform.
2. The design, construction, operation, maintenance, surveillance, and closure of the Shepherds Silt Pond will be in general accordance with the NZDSG (Ref. 5).
3. The detailed design will be undertaken by a suitably qualified engineer with experience in dam design and construction.
4. Detailed geotechnical investigation and design will be undertaken, and a Building Consent will be applied for from Environment Canterbury.
5. The dam construction will be managed by personnel experienced in dam construction.
6. Shepherds Silt Pond will be operated under the site Pond and Water Reservoir Management Plan and have a specific Operation, Maintenance, and Surveillance Manual and Emergency Action Plan.
7. Erosion and sediment controls will be detailed in the Erosion and Sediment Control Plan for the construction of silt pond area. Dust will be control with water trucks.

13.0 CONCLUSION

This technical report presents the proposed design of the Shepherds Silt Pond for the assessment of environmental effects. Shepherds Silt Pond will be formed by an earth embankment dam with a proposed crest of 543 mRL. Downstream the embankment will be buttressed by the infill of the valley floor downstream. The top level of the Valley Infill is estimated to be approximately 538 mRL. The effective height of the embankment dam will be approximately 5 m. The depth of the Valley Infill at the downstream toe of the embankment is approximately 16 m. Within the reservoir the floor is approximately 527 mRL. The upstream slope height is approximately 16 m.

Shepherds Silt Pond has been sized to provide sediment retention control for 179 ha area of the Shepherd Engineered Landform and 30,000 m^3 of backup process water storage. At the maximum inflow design flood the dam will impound approximately 75,000 m^3. Dam breach assessment has been undertaken and Shepherds Silt Pond is assessed as a Low Potential Impact Classification dam.

The design, construction, operation, maintenance, surveillance and closure will be undertaken in accordance with the NZDSG (Ref. 5), the Building Act (Ref. 2), and the Building (Dam Safety) Regulations (Ref. 20, 21). Detailed investigation and design required as part of the Building Consent, which is required from Environment Canterbury (the relevant Building Consent Authority for dams) prior to construction.

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TABLES

Table 1: Design Criteria

Potential Impact Classification	Low PIC
Sediment retention design basis	Size for 10 year ARI event using ICEA (Australasia) Type C Sediment Basin for (storm duration with) maximum peak flow.
Water storage	30,000 m^3
Dead storage	5,500 m^3
Spillway	1 in 1,000 AEP IDF
Earthquake Operating Basis Earthquake (OBE) Safety Evaluation Earthquake (SEE)	 1 in 150 AEP 1 in 1,000 AEP
Stability ***Static*** ***Seismic*** OBE SEE	 Limit Equilibrium FoS ≥ 1.5 Minor deformations are acceptable, and the resulting damage is easily repairable. Some deformations and damages are permitted as long as the stability is ensured.

Notes:
AEP = Annual Exceedance Probability

Table 2: High Intensity Rainfall Database – Rainfall Depths

AEP	Rainfall (mm) for Duration (hr)									
	0.17	**0.33**	**0.5**	**1**	**2**	**6**	**12**	**24**	**48**	**72**
1 in 2	3.9	5.7	7.1	10.4	14.9	25.6	34.9	46.3	59.2	67.1
1 in 10	7.4	10.5	12.8	18.2	25.4	41.9	55.6	71.6	89.2	99.6
1 in 100	14.9	20.4	24.5	33.5	45.4	71.0	91.1	113.0	137.0	149.0
1 in 1000	26.6	35.5	42.1	56.2	74.2	110.4	137.1	156.3	214.3	248.8

Table 3: Elevation Storage Volumes

Elevation (mRL)	Volume to elevation (m^3)	Delta volume (m^3)	Delta area (m^2)
543.0	89,655	6784	13,830
542.5	82,870	6526	13,309
542.0	76,345	6270	12,795
541.5	70,074	6017	12,285
541.0	64,058	5766	11,782
540.5	58,292	5514	11,280
540.0	52,778	5261	10,776
539.5	47,517	5009	10,268
539.0	42,508	4758	9768
538.5	37,750	4503	9264
538.0	33,247	4243	8747
537.5	29,005	3979	8224
537.0	25,024	3693	7689
536.5	21,332	3339	7053
536.0	17,992	2947	6300
535.5	15,045	2547	5487
535.0	12,498	2179	4709
534.5	10,319	1861	4022
534.0	8,458	1576	3430
533.5	6,882	1327	2882
533.0	5,556	1122	2443
532.5	4,434	942	2050
532.0	3,493	789	1720
531.5	2,704	662	1444
531.0	2,042	549	1205
530.5	1,493	441	989
530.0	1,052	343	781
529.5	709	259	591
529.0	451	190	450
528.5	261	132	313
528.0	129	82	214
527.5	47	38	116
527.0	8	8	44
526.5	0	0	0

Table 4: Material Properties for Stability Analyses

Material	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	Uniaxial Compressive Strength, UCS (kPa)	Geological Strength Index, GSI	Material Constant, mi	Disturbance Factor, D	Unit Weight ($kN/^3$)
Foundation Material	Hoek-Brown	-	-	35,000	45	12	0	21.5
Zone A	Mohr-Coulomb	0	34	-	-	-	-	-
Zone B	$\tau = 2.133\sigma'^{0.87}$	-	-	-	-	-	-	-
Valley Infill (Zone C)	$\tau = 1.29\sigma'^{0.91}$	-	-	-	-	-	-	-

Table 5: Summary of Stability Analyses Results

Section	Potential Failure Surface	FoS	K_y (g)	Figure
A-A'	Critical Failure Surface at Crest to Toe - Downstream	2.63	-	B1
	Critical Failure Surface at Crest to Toe - Upstream	5.10	-	B2
	Critical Failure Surface OBE - Full Depth	1.86	-	B3
	Critical Failure Surface OBE - 2/3H Below Crest	1.75	-	B4
	Critical Failure Surface OBE - 1/3H Below Crest	1.65	-	B5
	Critical Failure Surface - Yield Acceleration Full Depth	1.00	0.850	B6
	Critical Failure Surface - Yield Acceleration 2/3H Below Crest	1.00	0.559	B7
	Critical Failure Surface - Yield Acceleration 1/3H Below Crest	1.00	0.664	B8

Table 6: Summary of Co-Seismic Displacement Results

Section	Loading Condition	Toe of Failure Surface below Crest	Slide Mass Spectral Period 1.3T (s)	Amplification Factor AMP	Spectral Acceleration SA(1.3Ts)	Mean Magnitude (M_w)	K_h (g)[1]	K_y (g)[2]	Pseudostatic FoS	Seismic Displacement (cm)[3] Bray and Macedo 2019 (84th and 16th Percentile)	Figure
A-A'	OBE	H	0.18	3.58	0.27	6.3	0.12	–	1.86	–	B3
	OBE	2/3H	0.11	3.58	0.27	6.3	0.23	–	1.75	–	B4
	OBE	1/3H	0.08	3.58	0.25	6.3	0.33	–	1.65	–	B5
	SEE[4]	H	0.18	2.12	0.73	7.1	–	0.850	–	<0.5 – <0.5	B6
	SEE[4]	2/3H	0.11	1.95	0.73	7.1	–	0.559	–	<0.5 – 2.9	B7
	SEE[4]	1/3H	0.08	1.91	0.67	7.1	–	0.664	–	<0.5 – 1.3	B8

(1) K_h (g) - average acceleration within the potential failure mass for various return period earthquakes (for pseudostatic analysis only).

(2) K_y (g) - yield acceleration within the potential failure mass for an FoS = 1.0, determined using pseudostatic approach.

(3) Estimated seismically induced permanent displacement during an earthquake. The range given here represents the lower and upper estimates. For the B&M 2019 method, it is the 84% and 16% probability of exceedance.
(4) SEE is 1,000 AEP.

Table 7: Embankment Crack Estimates

Design Ground Motion	EGL	Fong and Bennett (Ref. 15)		Pells and Fell (Ref. 14)	
	Estimated Crest Settlement (mm)	Estimated Transverse Crack Depth (m)	Estimated Transverse Crack Width (mm)	Estimated Longitudinal Crack Width (mm)	Estimated Crest Settlement (mm)
OBE	0	< 0.1	< 5	0	0
SEE	0 – 13	0.0 – 0.6	0 – 14	0 – 19	0 – 13

Table 8: Estimates of Wave Run-up and Wind Setup

Wind Speed AEP	Water Level	Wave Run-up Slope[3]	Fetch (km)	Significant Wave Heights, Hs (m)	Wave Length (m)	Wave Run-up, R10% (m)	Wind Set-up (m)	Wave Run-up and Wind Setup (m)
1 in 10	MRL[1]	1V:2.5H	0.136	0.073	0.651	0.092	0.00077	0.142
1 in 100	WSL[2]	1V:2.5H	0.136	0.084	0.721	0.107	0.00059	0.161

Notes:
(1) Water level under the Maximum Reservoir Level (MRL) during the Inflow Design Flood (IDF) of RL541.908.
(2) Water level under the Water Storage Level (MNRL) of RL538.5.
(3) Slope of upstream embankment surface 2.5H:1V.

Table 9: Summary of Freeboard Table

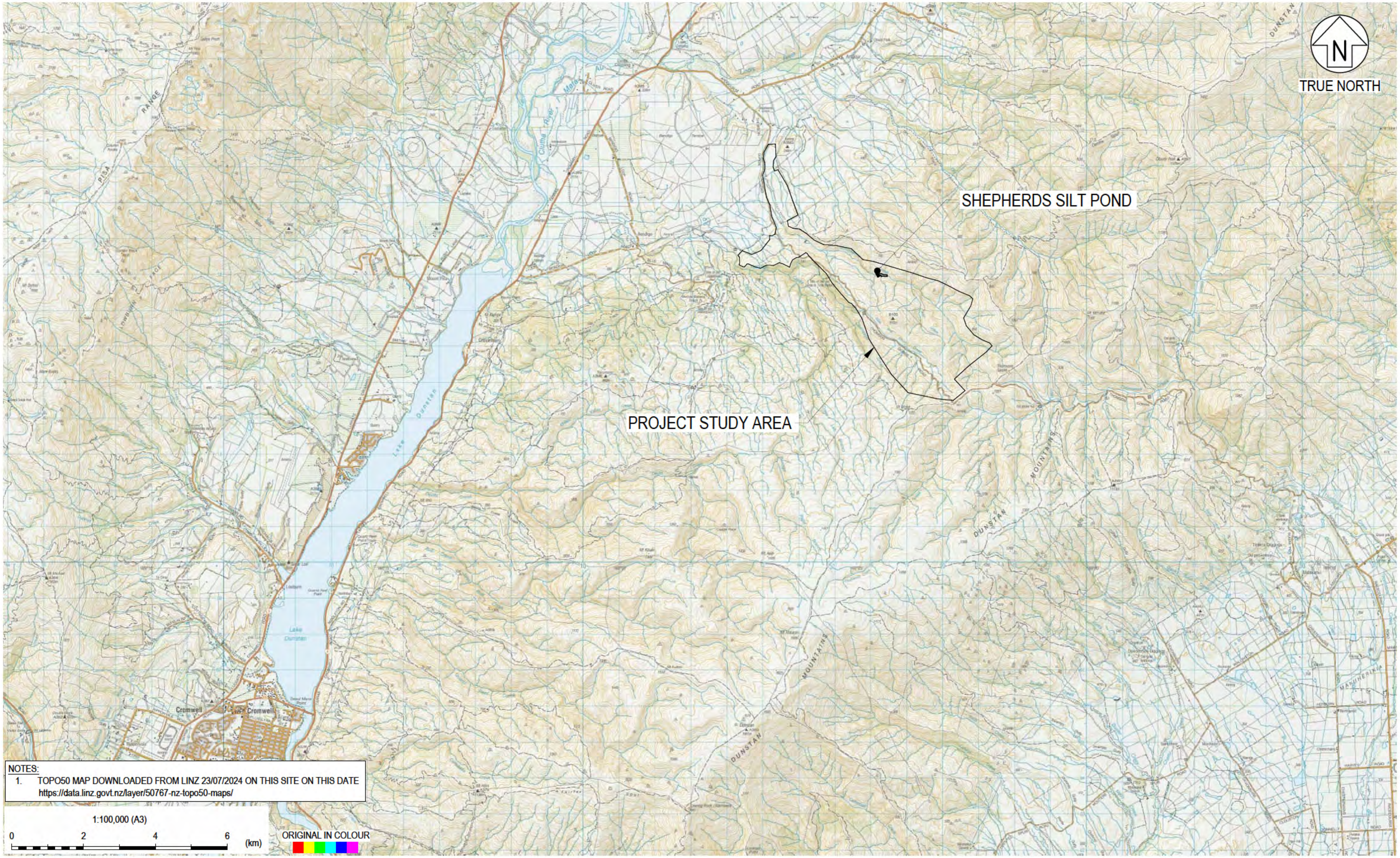
Parameters	Normal Conditions(1)	IDF(2)
Embankment Crest Level (RL)	543	543
Reservoir Water Level (RL)	540.935	542.027
Freeboard without wind, wave or settlement (m)	2.065	0.973
Design Wind Event	1 in 100 AEP	1 in 10 AEP
Wave run-up (R10%) + Setup (m) (3)	0.161	0.142
Freeboard allowing for wave run-up + Setup (m)	1.904	0.831
Static embankment settlement(m) (4)	0.085	0.085
Freeboard allowing static settlement + run-up/setup (m)	**1.819**	**0.746**

Notes:
(1) Normal Condition is for 1 in 10 AEP rainfall.
(2) IDF is for 1 in 1,000 AEP rainfall.
(3) Refer to Table 4.
(4) Fell et al. (Ref. 19) state a conservative approach for an earthfill embankment is to allow for settlement of about 0.5% of embankment height.

Table 10: IECA Type C Basin Design Parameters

Parameter	**Value**
Inflow catchment area	179 ha
Design run off coefficient	0.55
Shepherds Siltpond ESC Design Rainfall AEP	1 in 10 year
IECA method design peak inflow	5.6 m^3/s
IECA method 0.5 design peak inflow	2.8 m^3/s
Settling zone depth (assumed for routing)	2 m
Flow routing - Peak Outflow Case	1 in 10 year 6 hour duration
Flow routing - Peak Outflow	2.8 m^3/s
Pond length to width ratio at decant level	6 length to 1 width
Minimum surface area at decant level	11,500 m^2

FIGURES



NOTES:
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https://data.linz.govt.nz/layer/50767-nz-topo50-maps/

1:100,000 (A3)
0 2 4 6 (km)

ORIGINAL IN COLOUR

FIGURE 01

EGL Geotechnical, Earthquake and Dam Engineers
Engineering Geology Ltd www.egl.co.nz

MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK & RISE & SHINE CREEK AREA
SHEPHERDS SILT POND
LOCALITY PLAN

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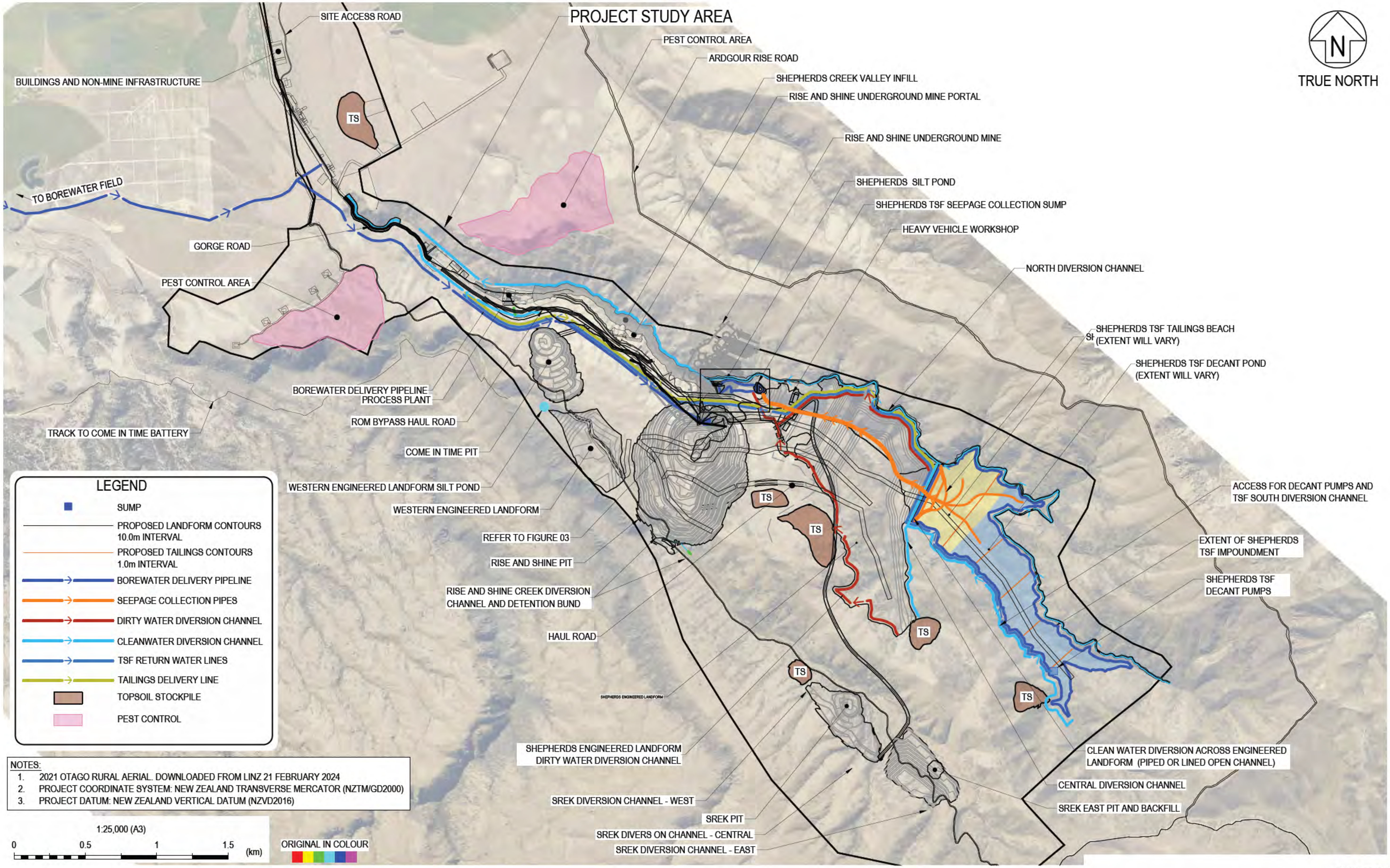

FIGURE 02

EGL Geotechnical, Earthquake and Dam Engineers
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MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK & RISE & SHINE CREEK AREA
SHEPHERDS SILT POND
SITE PLAN

DRAWN	R.M.	DATE	JOB No	SCALE (A3)	REV.
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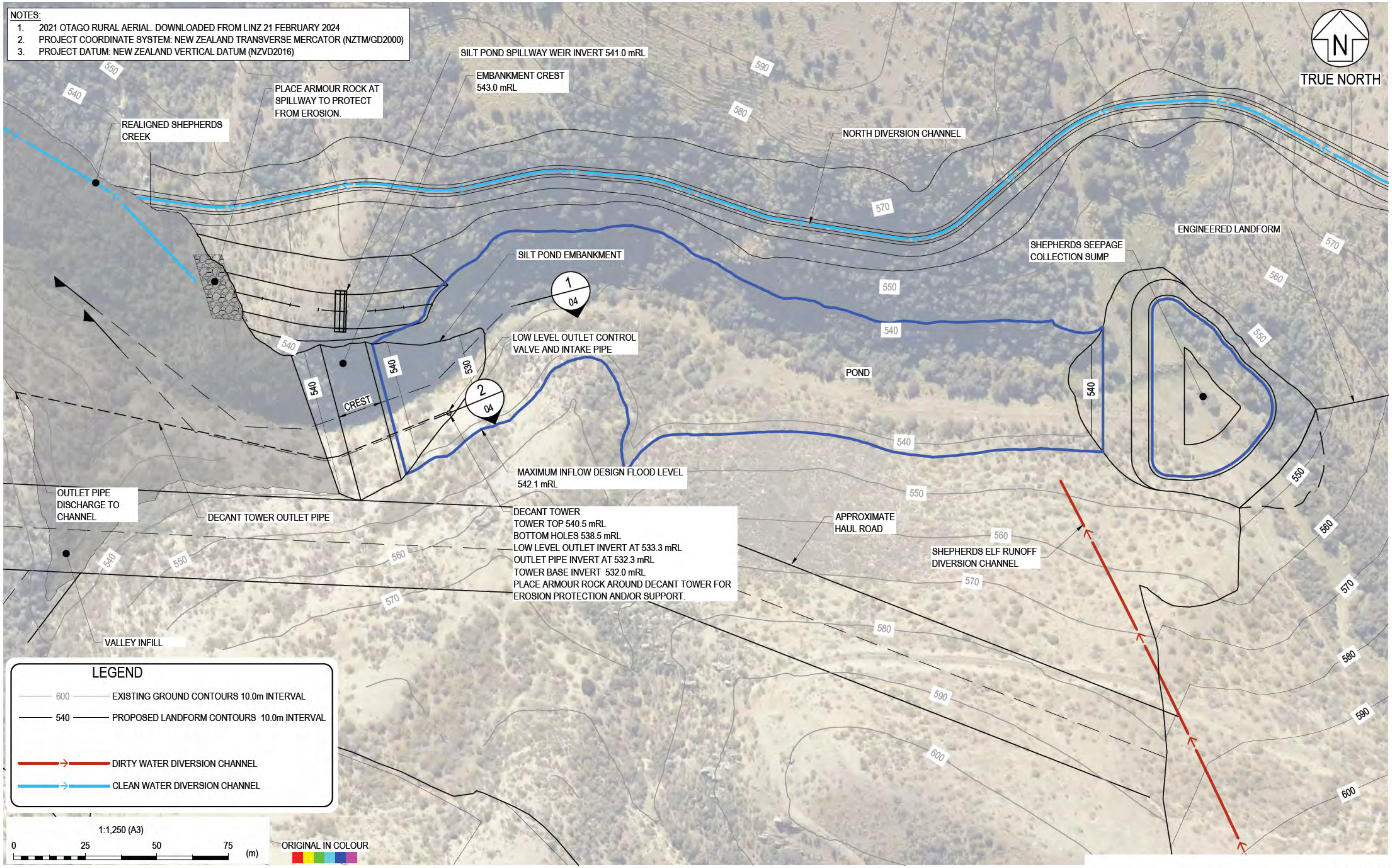

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FIGURE 03

EGL Geotechnical, Earthquake and Dam Engineers
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MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK & RISE & SHINE CREEK AREA
SHEPHERDS SILT POND
LAYOUT PLAN

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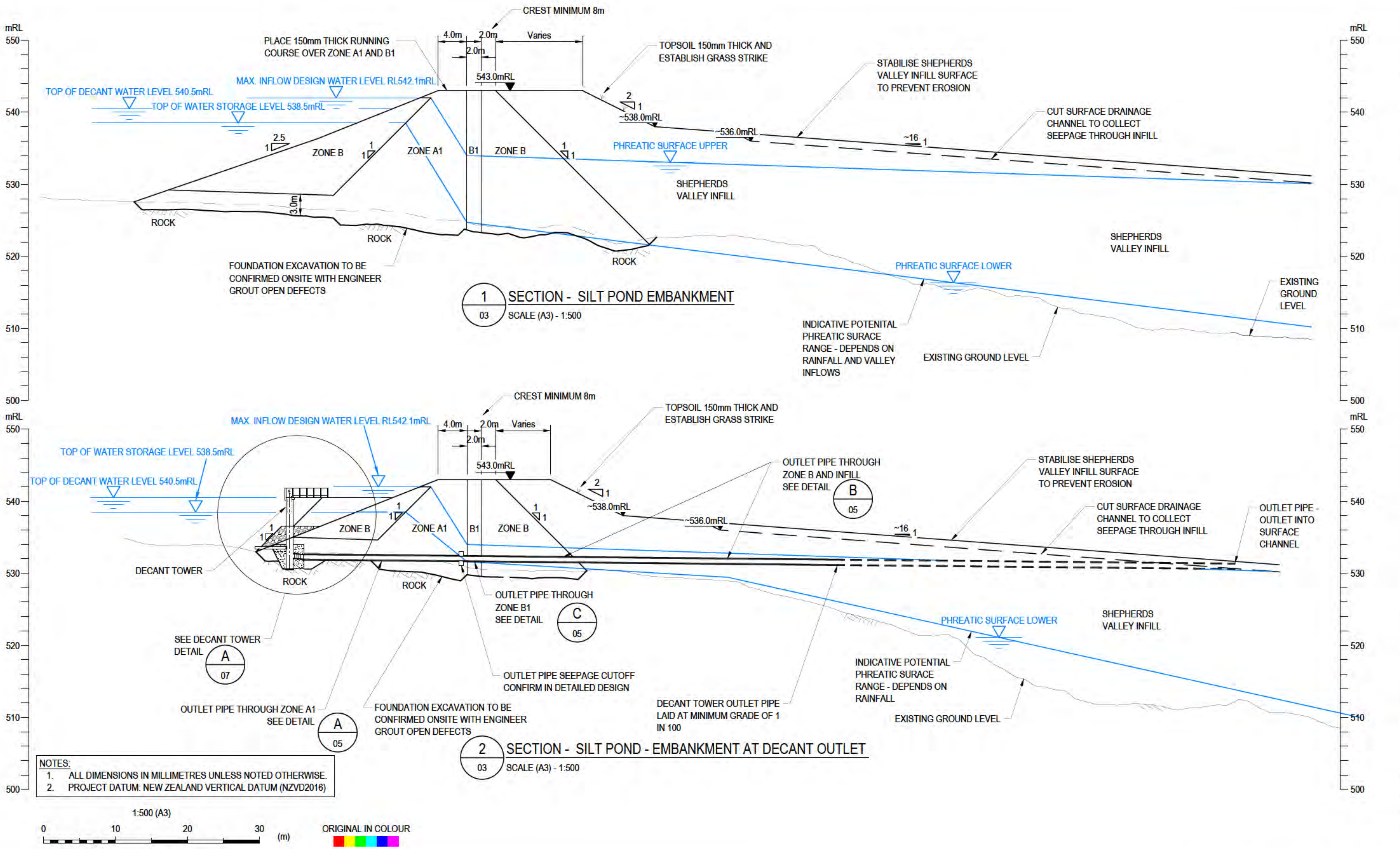

FIGURE 04

EGL Geotechnical, Earthquake and Dam Engineers
Engineering Geology Ltd www.egl.co.nz

MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK & RISE & SHINE CREEK AREA
SHEPHERDS SILT POND
CROSS SECTIONS

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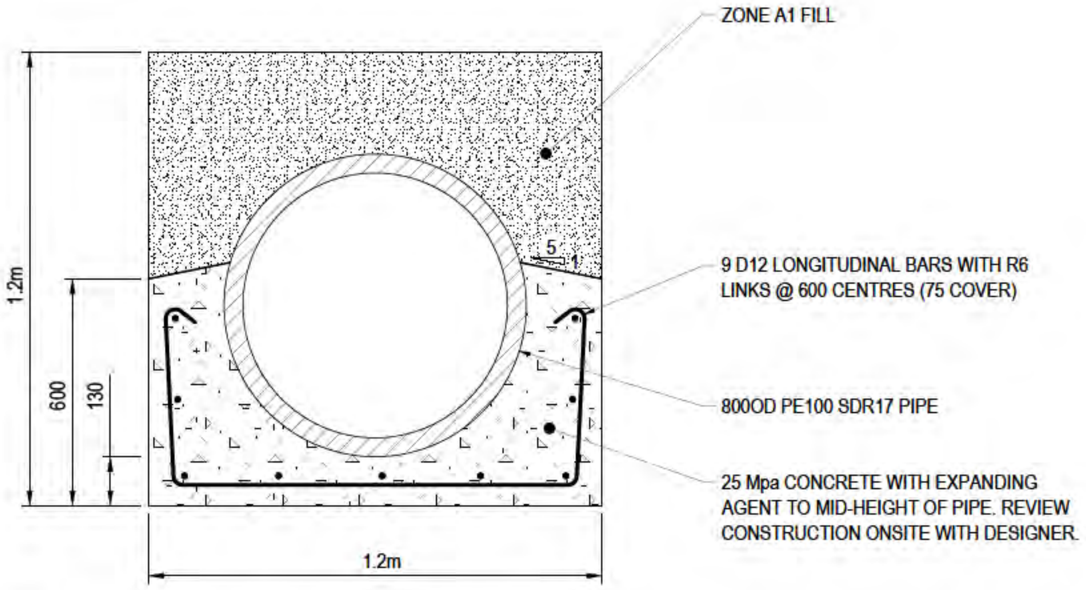
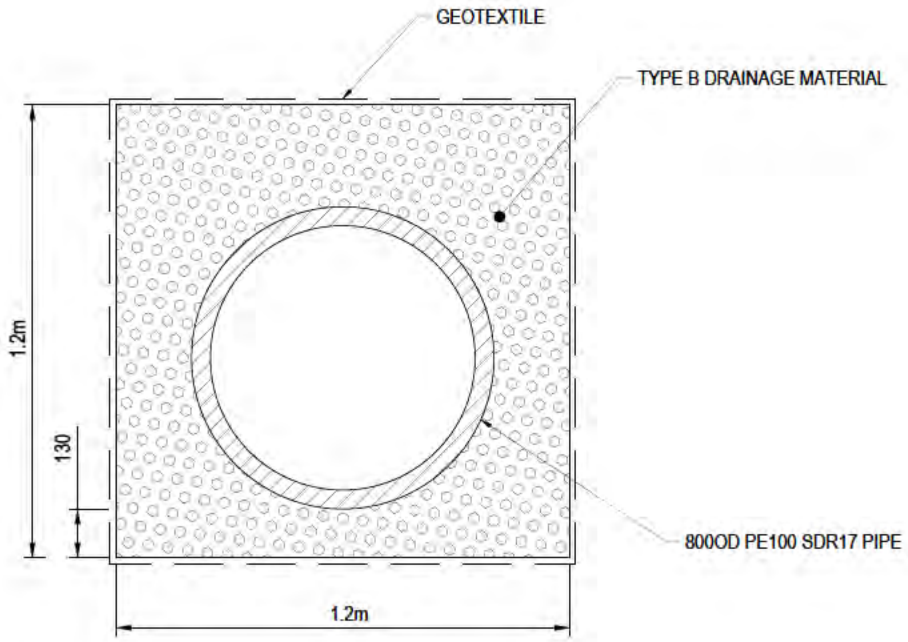


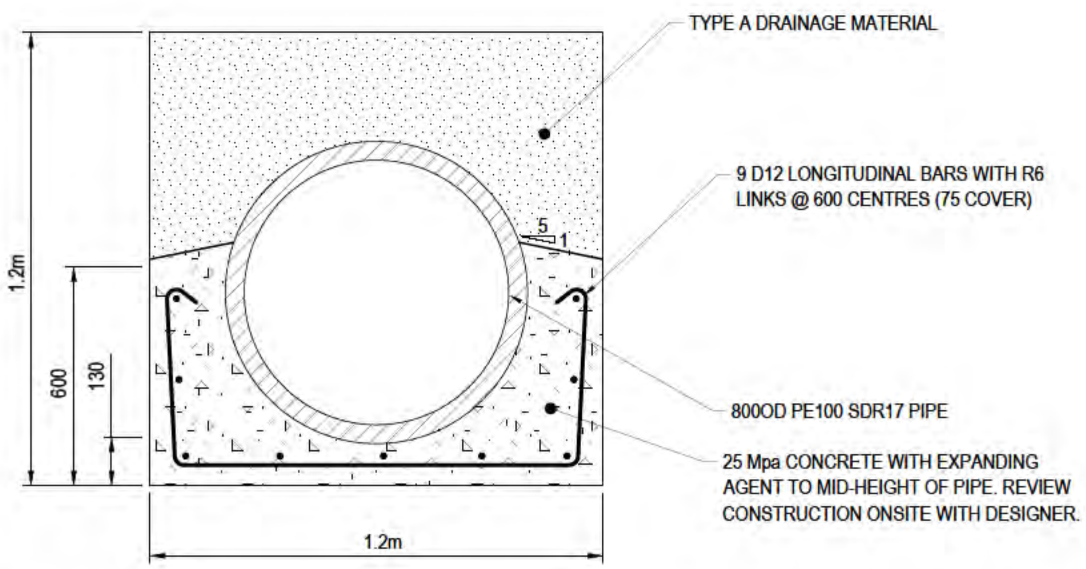
A / 04 DETAIL - OUTLET PIPE - THROUGH ZONE A1

SCALE (A3) - 1:20



B / 04 DETAIL - OUTLET PIPE - THROUGH ZONE B AND INFILL

SCALE (A3) - 1:20



C / 04 DETAIL - OUTLET PIPE - THROUGH ZONE B1

SCALE (A3) - 1:20

NOTES:
1. ALL DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.

1:20 (A3)

0 0.4 0.8 1.2 (m)

FIGURE 05

EGL Geotechnical, Earthquake and Dam Engineers
Engineering Geology Ltd www.egl.co.nz

MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK & RISE & SHINE CREEK AREA
SHEPHERDS SILT POND
OUTLET PIPE DETAILS

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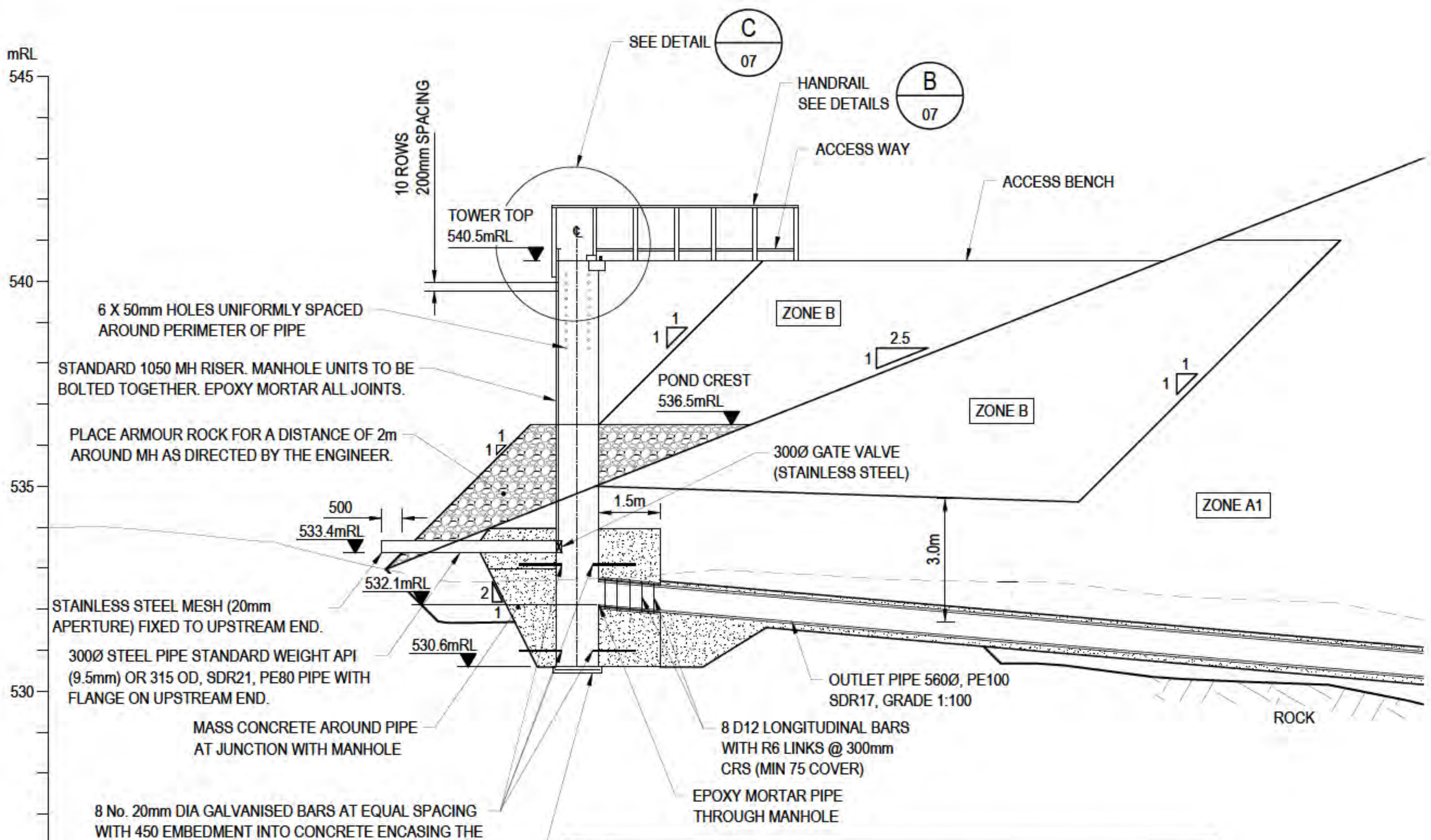
FIGURE 06 REMOVED

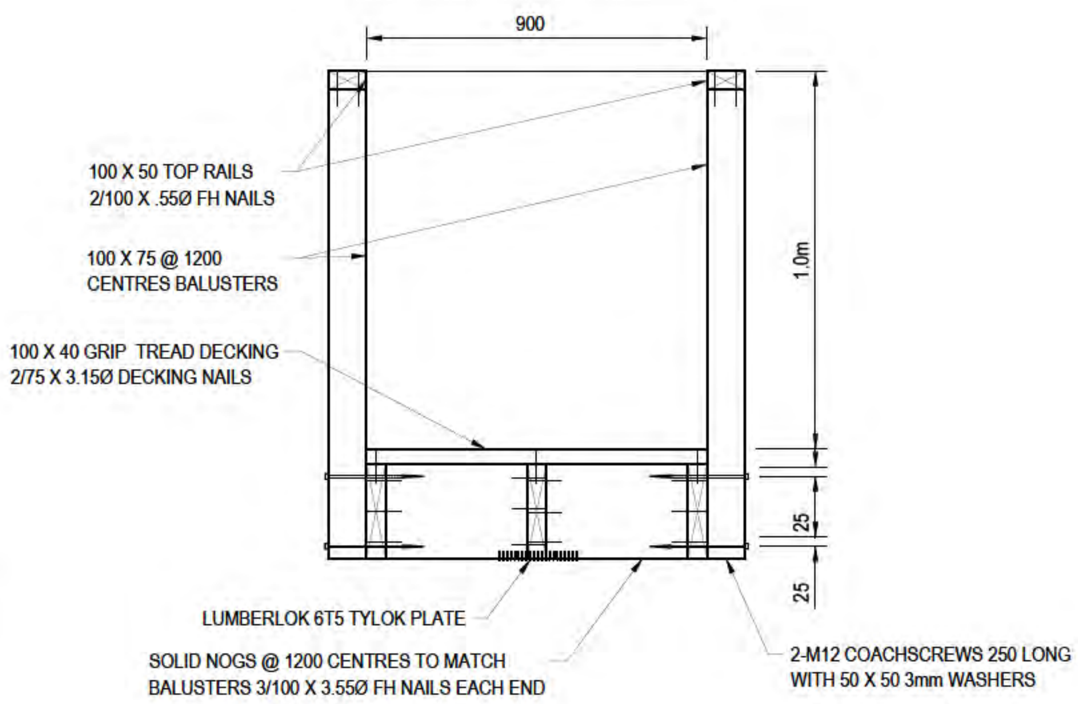


NOTES:

1. SPINDLE SHAFT TO GATE VALVE TO BE EXTENDED TO TOP OF MANHOLE. TO BE SUPPORTED BY PLUMMER BLOCKS ATTCHED TO SIDE OF MANHOLE. WHEEL TO BE ATTACHED TO TOP OF SHAFT TO OPEN AND CLOSE VALVE. DETAILS SUBJECT TO APPROVAL BY ENGINEER.
2. HEAVY DUTY GALVANISED STEEL GRATING TO BE FIXED TO TOP OF MANHOLE. DETAILS SUBJECT TO APPROVAL BY ENGINEER.
3. HANDRAIL TO BE FIXED AROUND TOP OF MANHOLE.
4. THERE SHALL BE NO JOINS IN THE MANHOLE UNITS WHERE THE CULVERT INTERSECTS THE MANHOLE.
5. BUTT WELD PUDDLE FLANGE OR EF COUPLER WITH RX WATERSTOP TO BE USED WHERE PE PIPE INTERSECTS MANHOLE.

A 04 DETAIL - DECANT INLET STRUCTURE
SCALE (A3) - 1:150

NOTE:
ALL FIXING TO BE GALVANISED AND ALL TIMBER ARE H4 TANALISED.



B 07 DETAIL - HANDRAIL
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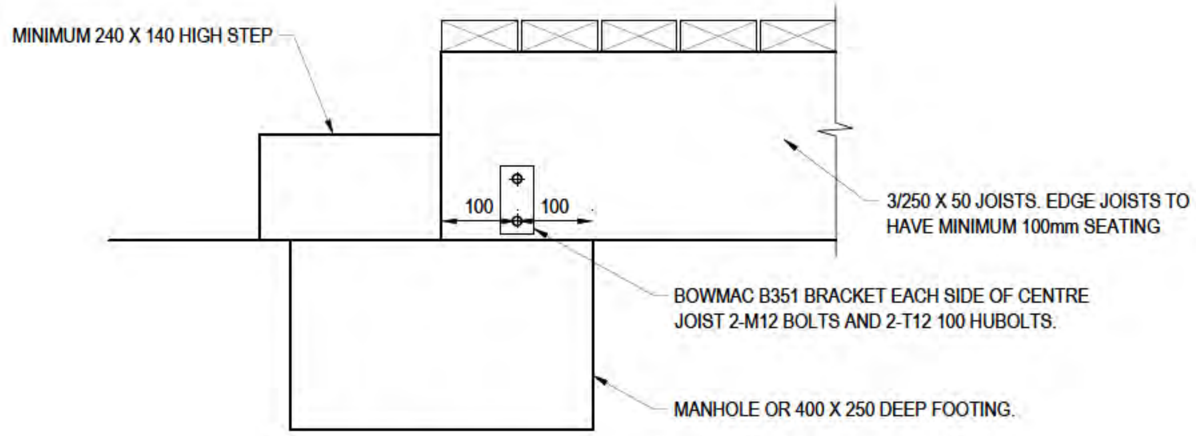

C 07 DETAIL - HANDRAIL
SCALE (A3) - 1:10

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.
2. PROJECT DATUM: NEW ZEALAND VERTICAL DATUM (NZVD2016)

1:150 (A3)
0 3 6 9 (m)

FIGURE 07

EGL Geotechnical, Earthquake and Dam Engineers
Engineering Geology Ltd www.egl.co.nz

MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK
SHEPHERDS SILT POND
DECANT DETAILS

DRAWN	R.M.	DATE	JOB No	SCALE (A3)	REV.
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Z:\Mining Jobs\0-Bendigo\9702 - Matakanui Gold Ltd\05 CAD\FIG\20240924 Shepherds Crk-Silt Pond-Tech Report\9702-FIG-07.dwg

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.
1. 2021 OTAGO RURAL AERIAL. DOWNLOADED FROM LINZ 21 FEBRUARY 2024
2. PROJECT COORDINATE SYSTEM: NEW ZEALAND TRANSVERSE MERCATOR (NZTM/GD2000)
3. PROJECT DATUM: NEW ZEALAND VERTICAL DATUM (NZVD2016)

PRINTED: 2025-07-08 16:04:30

FIGURE 08

EGL
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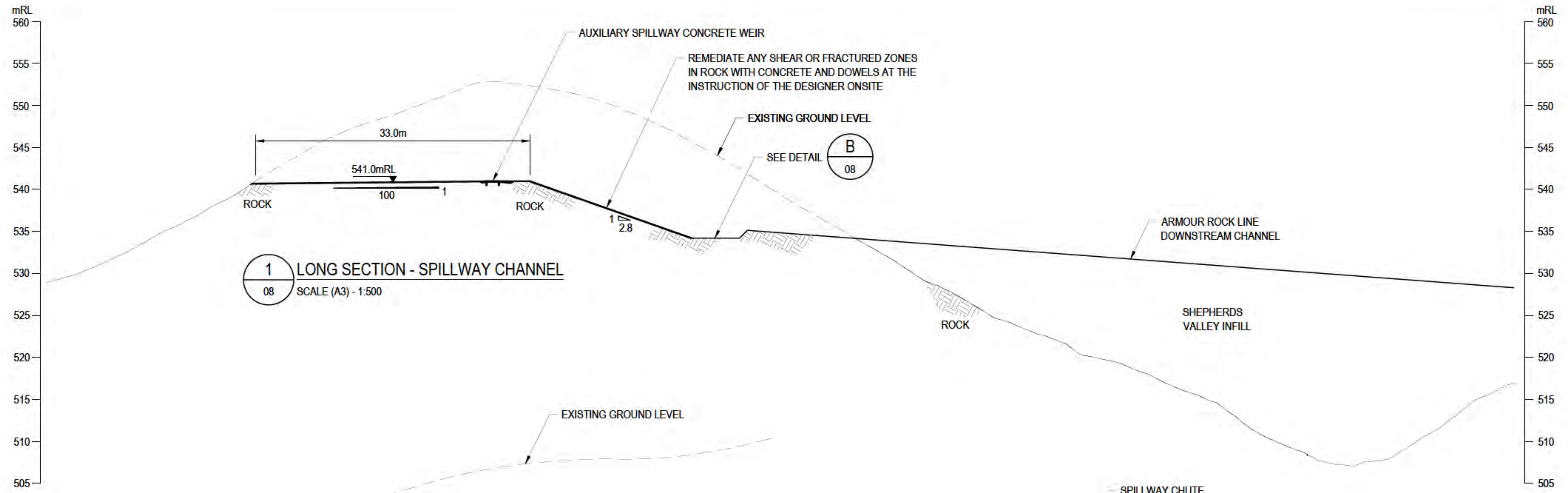
MATAKANUI GOLD LIMITED

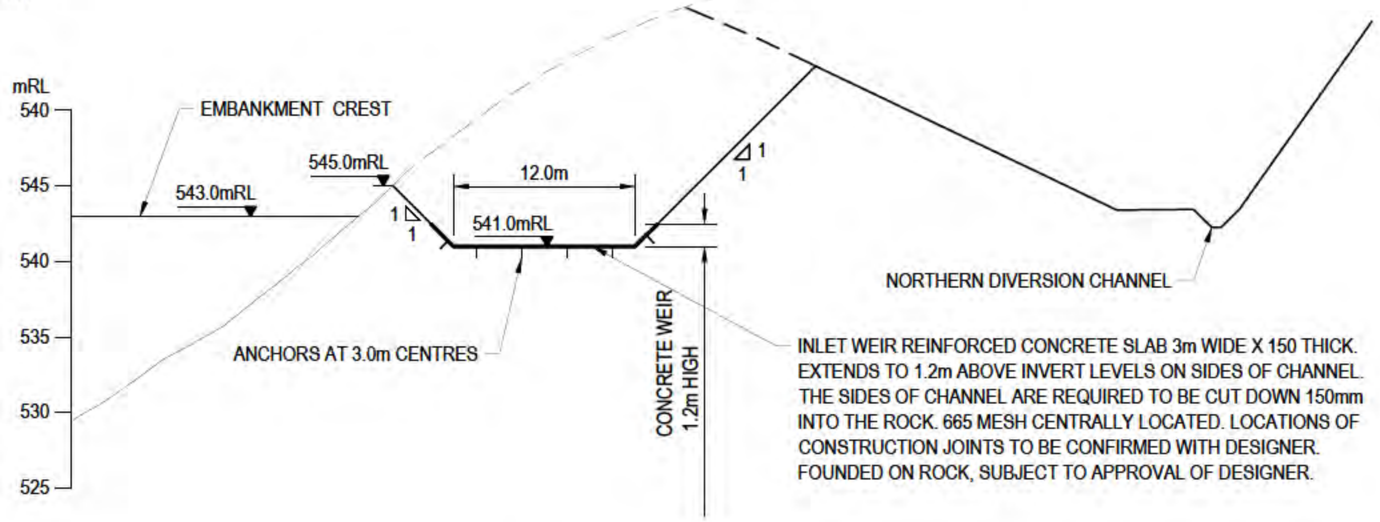
BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK & RISE & SHINE CREEK AREA
SHEPHERDS SILT POND
SPILLWAY LAYOUT PLAN

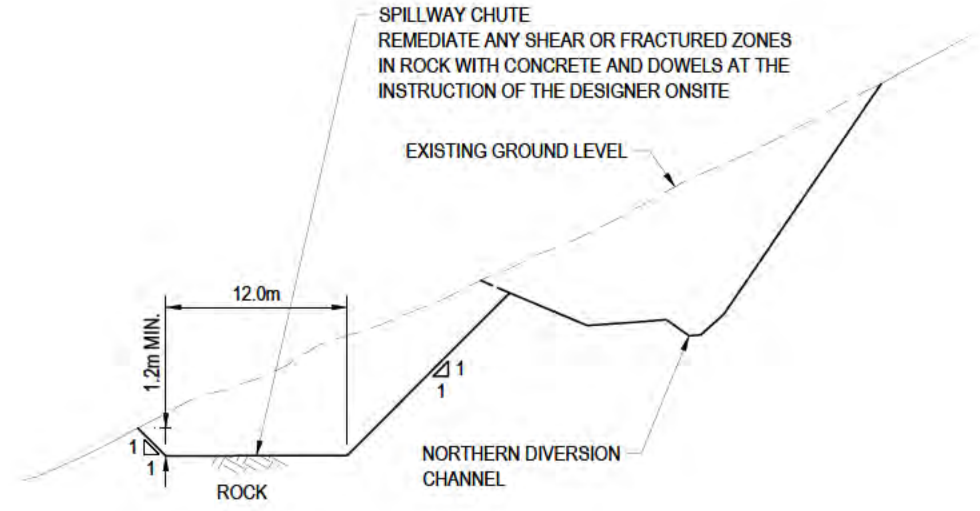
DRAWN	R.M.	DATE	JOB No	SCALE (A3)	REV.
CHECKED	E.T.	08/07/2025	9702	1:500	B

Z:\Mining Jobs\0-Bendigo\9702 - Matakanui Gold Ltd\05 CAD\FIG\20240924 Shepherds Crk-Silt Pond-Tech Report\9702-FIG-08.dwg



1 / 08 LONG SECTION - SPILLWAY CHANNEL
SCALE (A3) - 1:500





2 / 08 SECTION - SPILLWAY INLET WEIR
SCALE (A3) - 1:500

3 / 08 DETAIL - SPILLWAY CHUTE
SCALE (A3) - 1:500

NOTES:
1. ALL DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.
2. PROJECT DATUM: NEW ZEALAND VERTICAL DATUM (NZVD2016)

1:500 (A3)
0 10 20 30 (m)

FIGURE 09

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MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK & RISE & SHINE CREEK AREA
SHEPHERDS SILT POND
SPILLWAY SECTIONS

DRAWN	R.M.	DATE	JOB No	SCALE (A3)	REV.
CHECKED	E.T.	08/07/2025	9702	1:500	B

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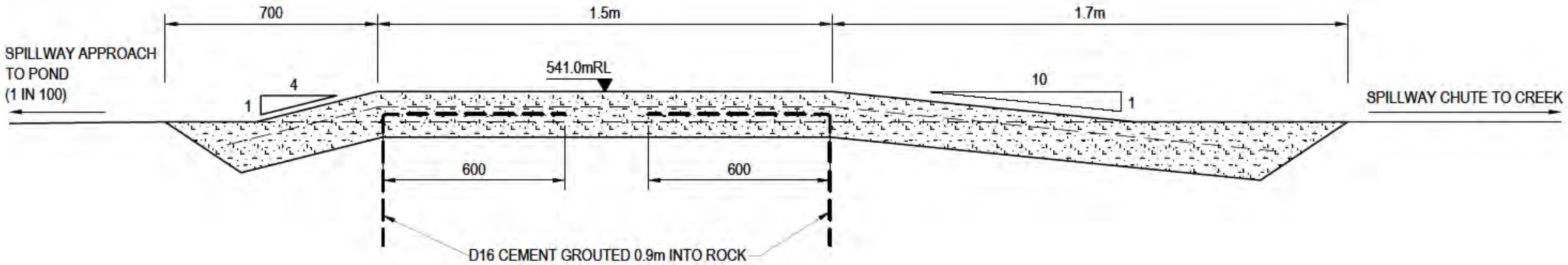
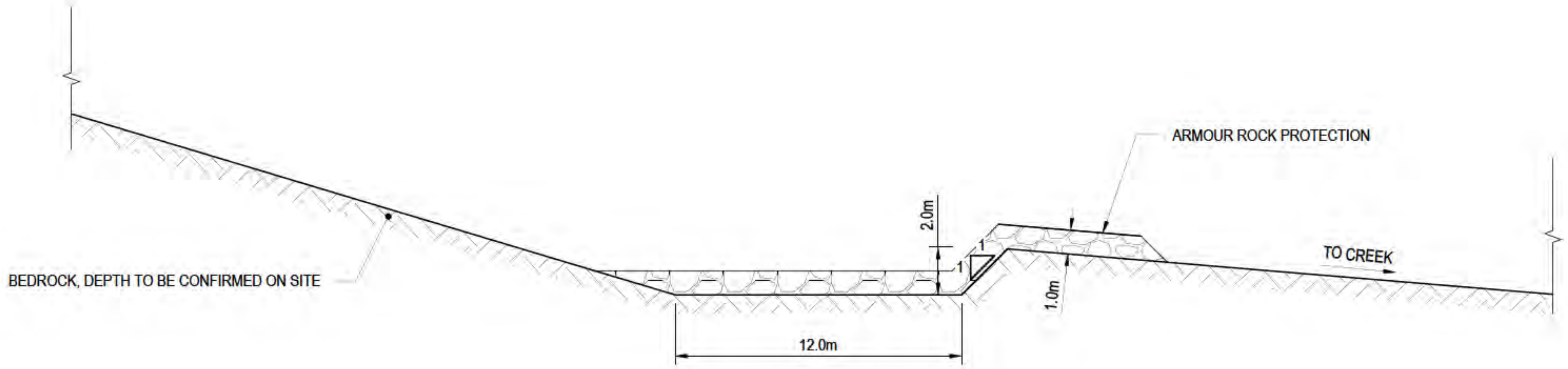
Z:\Mining Jobs\0-Bendigo\9702 - Matakanui Gold Ltd\05 CAD\FIG\2024\0924 Shepherds Crk-Silt Pond-Tech Report\9702-FIG-09.dwg



A / 08 DETAIL - SPILLWAY INLET WEIR

SCALE (A3) - 1:20



B / 08 DETAIL - SPILLWAY STILING BASIN

SCALE (A3) - 1:250

NOTES:
1. ALL DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.
2. PROJECT DATUM: NEW ZEALAND VERTICAL DATUM (NZVD2016)

1:20 (A3)

0 0.4 0.8 1.2 (m)

FIGURE 10

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MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK
SHEPHERDS SILT POND
SPILLWAY DETAILS

DRAWN	R.M.	DATE	JOB No	SCALE (A3)	REV.
CHECKED	E.T.	08/07/2025	9702	AS SHOWN	B

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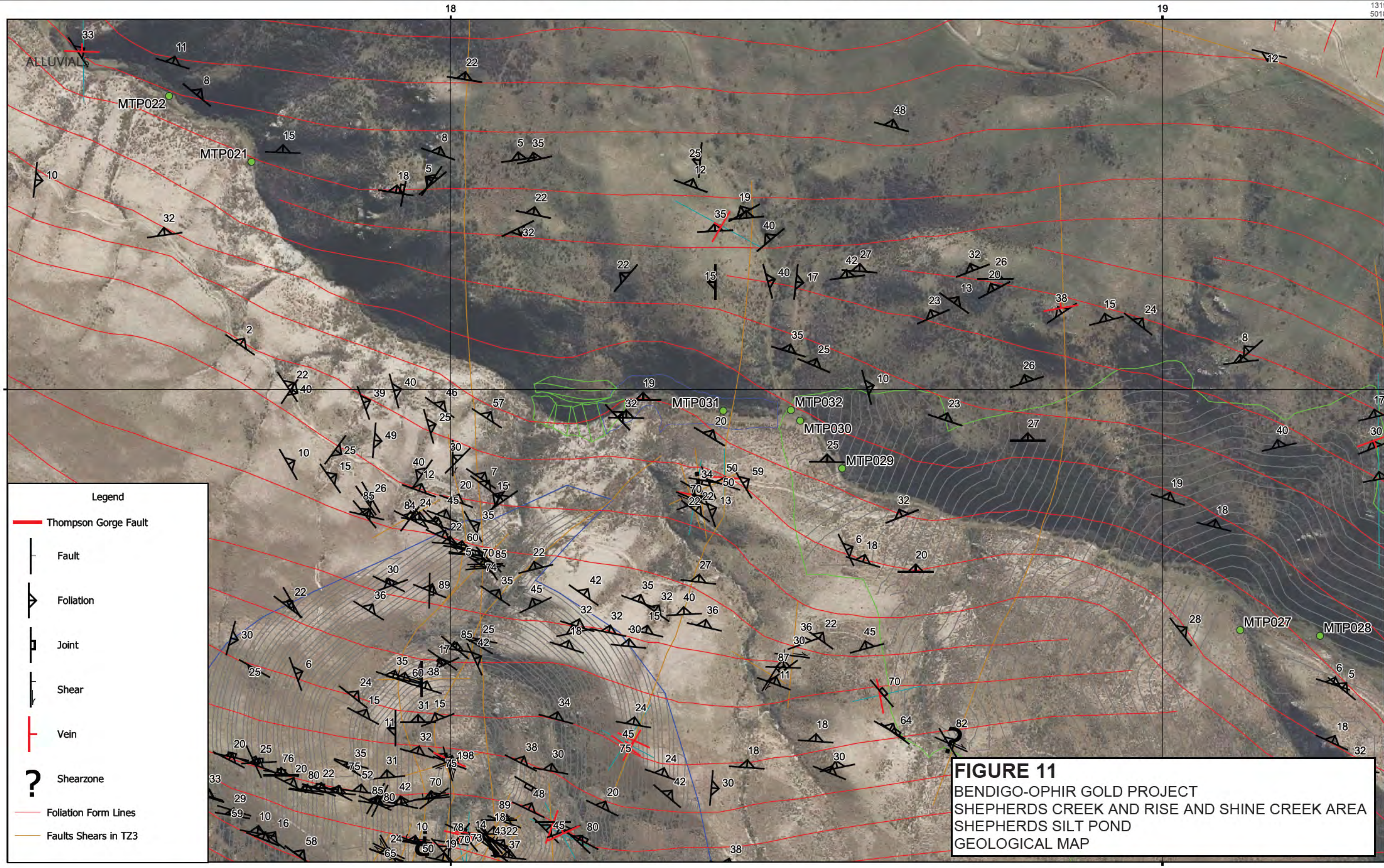
Z:\Mining Jobs\0-Bendigo\9702 - Matakanui Gold Ltd\05 CAD\FIG\20240924 Shepherds Crk-Silt Pond-Tech Report\9702-FIG-10.dwg

Scale 1:5,000

BENDIGO-OPHIR GOLD PROJECT



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Coordinate Units: Meters
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Project File: DATABASE.qgz
Page Size: 420 x 297 mm
Made with: QGIS 3.28 on Windows

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PO Box 301054, Albany, Auckland 0752
www.egl.co.nz

0 0.09 0.18 0.27 0.36 0.45
kilometers
Scale 1:5,000

SHP SILT POND AREA - 2025-04-03 - John Frengley

MATAKANUI GOLD LIMITED

Mapping information provided by Santana Minerals Limited.
Aerials and Topography from LINZ.
Projected in NZTM.

Scale 1:5,000

BENDIGO-OPHIR GOLD PROJECT

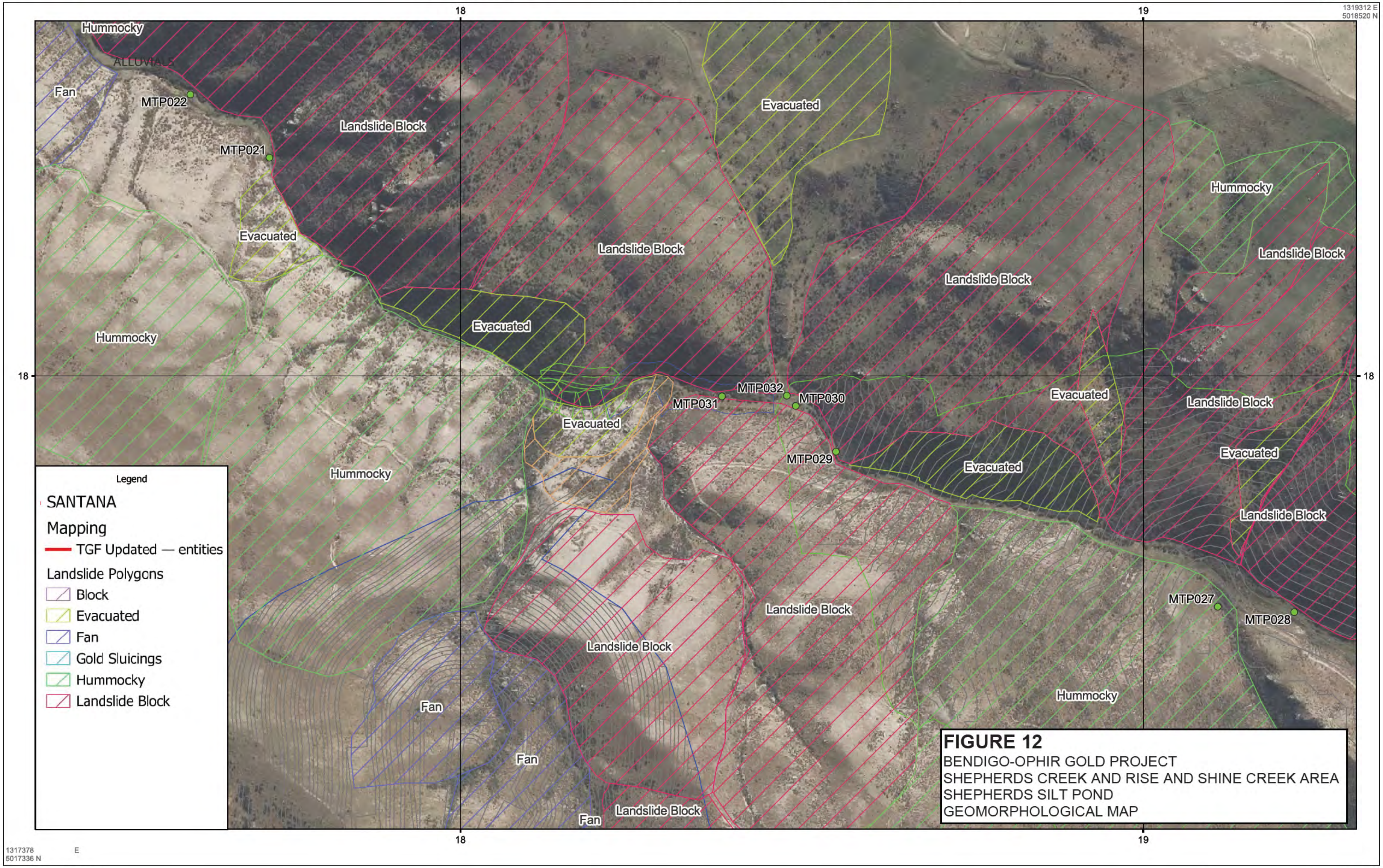

FIGURE 12
BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK AND RISE AND SHINE CREEK AREA
SHEPHERDS SILT POND
GEOMORPHOLOGICAL MAP

Map CRS: EPSG:2193
Coordinate Units: Meters
Map Scale: 1:5,000
Project File: DATABASE.qgz
Page Size: 420 x 297 mm
Made with: QGIS 3.28 on Windows

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0 0.09 0.18 0.27 0.36 0.45
kilometers
Scale 1:5,000

SHP SILT POND AREA - 2025-04-03 - John Frengley

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Mapping information provided by Santana Minerals Limited.
Aerials and Topography from LINZ.
Projected in NZTM.

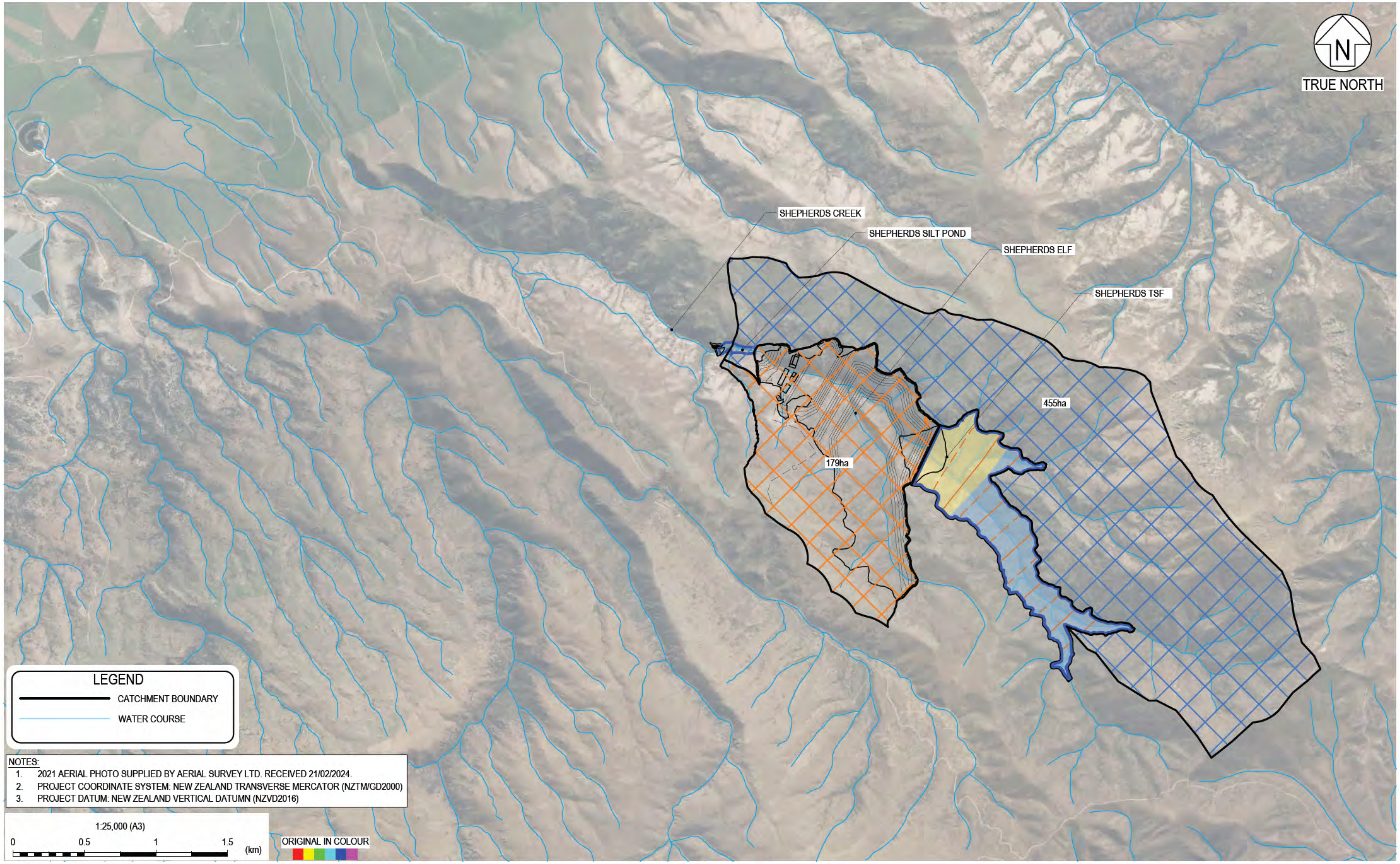

FIGURE 13

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MATAKANUI GOLD LIMITED

BENDIGO-OPHIR GOLD PROJECT
SHEPHERDS CREEK
HYDRAULIC ROUTING CATCHMENT AREA
CATCHMENT PLAN

DRAWN	P.Q.	DATE	JOB No	SCALE (A3)	REV.
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Z:\Mining Jobs\0-Bendigo\9702 - Matakanui Gold Ltd\05 CAD\FIG\20240924 Shepherds Crk-Silt Pond-Tech Report\9702-FIG-13.dwg

APPENDIX A

DAM BREACH AND POTENTIAL IMPACT CLASSIFICATION ASSESSMENT

A1.0 INTRODUCTION

Matakanui Gold Limited (MGL) is developing the Bendigo-Ophir Gold Project in Central Otago, New Zealand. As part of this development, a silt ponds to be called the Shepherds Silt Pond is required to control sediment associated with runoff from the Jean and Shepherds Creek ELF. This summary provides a high-level overview of the dam breach and consequence assessment undertaken for the Shepherds Silt Pond. The assessment is structured to align with the New Zealand Dam Safety Guidelines (NZDSG) (Ref. 1).

A2.0 PURPOSE AND OBJECTIVES

The purpose of this assessment is to evaluate the potential consequences of hypothetical dam breach scenarios for the Shepherds Silt Pond. The objectives are to:

- Identify and quantify the Population at Risk (PAR), Potential Loss of Life, and infrastructure impacts
- Determine the Potential Impact Classification (PIC) in accordance with the New Zealand Dam Safety Guidelines
- Inform the design, construction, and operational requirements for the Silt Pond.
- Support emergency planning and response measures

The assessment has been undertaken to support the application for resource consent. The assessment should be updated as part of detailed design.

A3.0 DESCRIPTION OF FACILITY AND LOCATION

The Shepherds Silt Pond will have a crest elevation of 543 mRL, with Shepherds Valley infilled to an embankment toe level of 536 mRL.

The Shepherds Silt Pond is located in a gully at the toe of the proposed Shepherds ELF. The Shepherds Silt Pond is designed using a downstream-constructed embankment. Figure 1 presents the potential path of the breach flood. A breach of the silt pond embankment dam could result in discharges of the silt pond stored water into the Shepherds Creek, and over the flood plain further downstream, across Thomson Gorge Road and Ardgour Road. A breach would join the Lindis River and eventually the Clutha River which is approximately 11 km downstream of the Shepherds Silt Pond.

The locations of downstream items of interest, including community buildings, roads, and bridges, are shown in Figure 1. The Mine Process Plant will be constructed on an engineered fill platform. Final platform levels are still to be determined. The Underground Mine Portal will be upstream of the Process Plant. The portal level is assumed to be set above the maximum dam breach flood level. The number of people expected to be at the facilities within the BOGP are summarised in Table A1.

Figure 1: Potential Path of Breach Flood and Locations of Downstream Items of Interest (Source: Google Hybrid)

A4.0 BREACH SCENARIO

This section describes the development of dam breach scenarios used to assess the potential consequences of the failure of the Shepherds Silt Pond. The scenarios are structured to reflect credible failure events under both operational and extreme weather conditions. This forms the basis for consequence classification in accordance with the NZDSG (Ref. 1).

A4.1. Dam Breach Assessment Approach

The breach assessment methodology was designed to quantify the timing, volume, and intensity of potential outflows from the silt pond in the event of failure. This involved:

- Identifying plausible failure mechanisms,
- Defining breach geometry and failure dynamics using empirical methods,
- Developing breach hydrographs using industry-standard approaches, and
- Modelling inundation extents using two-dimensional hydraulic routing software (HEC-RAS 2D).

A4.2. Failure Modes and Breach Scenarios

The assessment considers two representative breach scenarios based on plausible failure modes that could arise during the silt pond's operational life.

- **Sunny Day breach scenario –** a sudden fam failure occurring during normal hydrologic conditions (i.e., not due to a storm event). There is no triggering storm or flood event and potentially no advanced warning of failure to the downstream areas in this scenario. During normal operations, the breach of the embankment dam could develop from overtopping failure modes due to instability of the embankment, which can be caused by strong earthquake shaking, elevated pore pressure in the embankment fill, piping, and/or foundation instability.

- **Rainy Day breach scenario -** a failure triggered by extreme rainfall events. Under this scenario, the dam could be overtopped, and the downstream face of the dam could be eroded, resulting in uncontrolled release of stored water. The pre-breach flood condition in the downstream environment could cause the relocation of people or damage to infrastructure (e.g., culverts, bridges, etc.). The downstream incremental impacts can be analysed for a range of flood events by comparing the consequences with and without the failure of the dam.

A5.0 MODELLING METHODOLOGY

This section outlines the hydrodynamic modelling approach adopted for the Shepherds Silt Pond dam breach assessment. The modelling was undertaken to simulate the flood wave and inundation extents resulting from hypothetical breach scenarios under both fair weather (Sunny-Day) and extreme weather (Rainy-Day) conditions. The methodology integrates breach hydrograph generation, terrain-based flow routing, and estimation of downstream flood impacts.

The modelling outputs inform the consequence assessment by identifying:

- Peak flood depths and velocities at key downstream receptors;
- Arrival times for emergency response consideration;
- Estimated areas of inundation;
- Hazard categories used for Potential Impact Classification (PIC) under NZDSG (Ref. 1).

A5.1. Modelling Software

The two-dimensional hydraulic model was developed using HEC-RAS 2D Version 6.0 (Ref. 2), developed by the U.S. Army Corps of Engineers.

A5.2. Terrain Model

The digital terrain model used for flood routing was constructed by combining two sources:

- 1 m LiDAR (2021): Site survey data provided by the client, covering the Shepherds Silt Pond area, valley floor, and mine infrastructure corridor down to the Process Plant and the Administration Buildings (~4 km downstream).
- 8 m National LiDAR (2012): Sourced from the LINZ database and used for the wider floodplain downstream of the site, extending to the Lindis River.

The combined terrain data provided sufficient resolution to capture both critical infrastructure (roads, buildings) and natural drainage features over a ~13 km downstream model domain.

A5.3. Roughness and Boundary Conditions

Manning's n values were selected based on land use and guidance from the HEC-RAS manual:

- 0.05 – Site Access Road corridor (lined or constructed surfaces)
- 0.04 – Rural floodplain (pastoral and open land)
- 0.045 – Channelised or meandering creek systems

Boundary conditions:

- Upstream: Inflow hydrographs generated from breach scenario modelling
- Downstream: Normal depth based on topography

Rainy-Day scenarios also incorporated baseflow hydrographs from the 1-hour PMP event for incremental flood assessments.

A5.4. Breach Hydrograph Development

Breach hydrographs were generated for each scenario using a Soil Conservation Service (SCS) unit hydrograph shape (Ref. 3) and three empirical peak flow estimation methods:

- Froehlich, 1995 (Ref. 4)
- Langridge-Monopolis, 1984 (Ref. 5)
- Hagen, 1982 (Ref. 6)

The average peak outflow from the three methods was adopted.

A5.5. Modelling Extents and Termination Criteria

Modelling extents were determined using FEMA (Ref. 7) guidance:

- **Sunny Day scenarios**: Extended until floodwaters re-entered natural channels or dissipated to within-bank conditions.

- **Rainy Day scenarios**: Simulations extended to where the incremental flood depth reduced below 0.3 m or the wave peak travelled beyond 24 hours downstream.

The downstream model extent terminated at the Lindis River floodplain. Cross sectional channel flow calculations were undertaken at the SH8 bridge and Ardgour Road crossings.

A5.6. Hydrologic Analysis

The NZDSG (Ref. 1) requires that reservoir inflows and levels, and downstream watercourse flows, be those most likely to occur coincident with the potential dam failure mode, for both the Sunny-Day and Rainy-Day breach scenarios.

For the Sunny-Day breach scenario, the pond volume for the breach is assumed to be the maximum normal operation pond volume.

For the Rainy-Day it is assumed to at the crest level.

For the Rainy-Day breach scenario, base flood flows for a PMP event with the 1-hr, 2-hr, and 6-hr duration were modelled. The PMP depths and temporal patterns were calculated by using the method of Thompson and Tomlinson 1993 and 1995 (Refs. 8, 9). The runoff coefficient of 0.8 was adopted for the applied PMP rainfall in the analysis. The most significant incremental impacts were observed with the 1hr PMP base flood event. This was selected as the critical Rainy-Day breach scenario base flood flows for the PIC assessment.

A5.7. Breach Parameters

The breach geometry and hydrograph parameters were developed using multiple empirical methods (Ref. 4, 5, and 6). Parameters were selected and breach flow rates averaged across methods to reflect typical embankment erosion processes. The breach parameters are summarised in Table A2.

A6.0 INUNDATION MODELLING RESULTS

A6.1. Inundation Extents

The predicted flood extents vary with breach size and hydrologic condition. Maps of inundation depth and hazard category for all scenarios are provided in Figures A1 to A6. A summary of downstream distances, arrival times, and affected infrastructure is provided in Tables A3-A5.

A6.2. Flood Hazard Categorisation

Flood hazard levels were assigned using the Smith et al. 2014 (Ref. 10) flood hazard classification method, endorsed in NZDSG. These categories incorporate both flood depth and velocity and are used to assess the risk to life and building functionality.

Hazard mapping is provided in Figures A2, A4, A6 and informs the assessment of Population at Risk (PAR), Potential Loss of Life, and infrastructure damage categories.

A6.3. Arrival Times and Warning Implications

The arrival times of the breach flood at key locations are critical for emergency response and warning system design. These vary depending on breach size and condition. A summary of the breach times for each scenario are summarised in Tables A4 to A7.

The results highlight the warning time available to personnel on-site and the need for robust early warning and evacuation protocols as part of the silt pond' Emergency Action Plan (EAP).

A7.0 CONSEQUENCE ASSESSMENT

This section presents the consequence assessment for breach scenarios of the Shepherds Silt Pond. It follows the framework established in NZDSG Module 2 for assessing the Potential Impact Classification (PIC) of dams in New Zealand.

The assessment incorporates:

- The number of people potentially at risk (PAR),
- Estimated Potential Loss of Life,
- Damage to community, cultural, and critical infrastructure,
- Environmental consequences and restoration practicability.

The PIC has been assessed for both Sunny-Day and Rainy-Day breach conditions for the Shepherds Silt Pond.

A7.1. Methodology

The consequence assessment methodology includes:

- Flood hazard assessment using the Smith et al. (Ref. 10) classification for flood hazard categories (H1 to H6);
- PAR and Potential Loss of Life estimation, using guidance from the U.S. Bureau of Reclamation's Reclamation Consequence Estimating Methodology (RCEM, Ref. 11) and adjusted for context-specific occupancy and warning times;
- Assessment of infrastructure and environmental damage, based on flood hazard thresholds and site-specific design levels;
- Assignment of PIC per Table 2.1 of NZDSG based on the worst-case consequence outcome across all breach scenarios.

According to the NZDSG, individuals inside buildings or occupied areas are considered part of the PAR if flood hazard levels exceed the H2 category, while road users are included if flood hazard levels exceed the H1 category. Building damage was assessed under the assumption that floor levels are at least 150 mm above ground level. This could be conservative. However, the assessment of PIC in Section 10.0 is mainly based on the combination of PAR and Potential Loss of Life, which are not affected by the assumption of the floor level. As per NZDSG, buildings are considered uninhabitable or inoperable if flood hazard exceeds the H4 category. Potential Loss of Life was evaluated using fatality rates recommended by RCEM (Ref. 11). The upper limit of the recommended threshold for the little to no warning category was applied, per NZDSG (Ref. 1).

The PAR and Potential Loss of Life for road users were estimated using the Campbell method (Ref. 12). The AADT for roads further downstream were obtained from NZ Transport Agency (Ref. 13).

Tables A2 summarises the number of workers typically present on site for different breach scenarios.

Potential Loss of Life is calculated based on the fatality rate and a time reduction factor, which accounts for the likelihood of a worker being present on site. The Potential Loss of Life is calculated using the fatality rate and an area reduction factor which is based on the ratio of the affected area to the total building area.

A7.1. Population at Risk and Potential Loss of Life

The estimated PAR and Potential Loss of Life under each breach scenario are summarised in Tables A3 to A7.

Sunny-Day PAR is 201 and Potential Loss of Life is 0.02 (no persons).
Rainy-Day incremental PAR is 30 and incremental Potential Loss of Life is zero (no persons).

A7.2. Damage Assessment

Community Infrastructure

- Sunny-Day and Rainy-Day breach scenarios result in potential flood hazard categories H5 to H6 at the Site Access Road and the Process Plant. The Process Plant hazard depends on final platform levels.
- The Administration Building is not affected in both scenarios.
- Multiple downstream dwellings and farm buildings are incrementally inundated in H1 only.
- The Site Access Road is rendered inoperable under Sunny-Day and Rainy-Day breach scenarios.
- Modelling indicates that breach flow will pass the majority of the Process Plant area. Small area of Process Plant maybe inundated and this depends on the final layout of the Process Plant. It is assumed for this assessment that the Process Plant remain operable.

Cultural and Historical Sites

- No known cultural heritage sites or wāhi tapu areas are located within the inundation footprint. The damage in this category is therefore assessed as Minimal.

Critical or Major Infrastructure

- The SH8 bridge, the breach flood is expected to travel beneath the bridge. Thus, no damage to the bridge and road structure.

Natural Environment

- Loss of vegetation and localised erosion at the Shepherds valley, particularly along Shepherds Creek and farmland adjacent to the Lindis River.

A7.3. Potential Impact Classification (PIC)

The consequence assessment confirms that both the Sunny-Day and Rainy-Day configurations of the Shepherds Silt Pond warrant classification as Low Potential Impact Classification (PIC) structures under NZDSG. This classification reflects:

- Minimal to community infrastructure and key transport routes depending on the scenarios

Our assessments of the PAR, Potential Loss of Life, damage levels, and PIC for both Sunny-Day and Rainy-Day breach scenarios are summarised in Tables 1 to 7.

This PIC designation has implications for the Shepherds Silt Pond's design criteria, operational monitoring.

A8.0 CONCLUSIONS

A dam breach and PIC assessment have been undertaken for the Shepherds Silt Pond, the consequences of the breach assessed and the Potential Impact Classification (PIC) of the dam determined in accordance with the guidance in Module 2 of the New Zealand Dam Safety Guidelines 2024 (Ref.1). Both Sunny-Day and Rainy-Day breach scenarios were assessed.

Under the Sunny-Day breach of the Shepherds Silt Pond, a Low PIC is assessed , PAR of 201 and Potential Loss of Life is zero, and an overall Minimal damage level. Under the Rainy-Day breach of the Shepherds Silt Pond , a Low PIC is assessed, the incremental PAR of 30, zero Potential Loss of Life and an overall a Minimal damage level.

The breach study is based on scenarios that are not connected to the probability of occurrence. The dam breach study and assessment of consequences do not assess the likelihood of the failure and the associated risks.

A9.0 REFERENCES

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List of Tables in Appendix A

Table A1: Summary of Worker Numbers for Site Buildings

Area	Process Plant
Typical Average Number of Workers in Area During the Day (12 hr per day) (applies to Sunny and Rainy Day conditions)	30
Minimum number of workers possible in area (applies to Sunny and Rainy Day conditions)	10 (Nightshift and weekends)
Maximum number of workers in area for special situations (applies to Sunny Day condition only)	Up to 200 during scheduled shutdown/maintenance (approx. two weeks a year)

Table A2: Summary of Breach Parameters

Parameters	**Breach Scenarios**	
	Sunny Day	**Rainy Day**
Breach Embankment Height (m)	7.0	7.0
Breach Bottom (mRL)	536.0	536.0
Water Elevation at Breach (mRL)	538.5	543.0
Depth of Release Water Volume (m)	2.5	7.0
Volume of Discharge (m^3)	19,760	71,670
Peak Discharge (m^3/s)	112	272

Table A3. Summary of Consequences for Rainy Day Base Flood (PMP 1 hr) Scenario

Items	Distance Downstream (km)	No. of Buildings Affected	Time of Arrival	Hazard Category (No. of Buildings)	Inundated Length (km)	PAR	Potential Loss of Life
Site Access Roads in Valley (AADT = 200)	0.0	-	-	H6	3.0	0.67	0.4715
Process Plant	2.0	-	-	-	-	-	-
Administration Building	4.0	-	-	-	-	-	-
Thomson Gorge Rd (AADT = 259)	5.5	-	-	-	-	-	-
Farmland (~70ha)	1.7	-	-	H6	-	0.08	0.0000
Buildings Downstream	6.5	1	-	H3 (1 dwelling) H4 (1 dwelling) H5 (2 commercial)	-	8.10	0.0008
Ardgour Road (AADT = 400)	7.0	-	-	H5	0.7	1.08	0.0357
SH8 Bridge (AADT = 1945)	10.5	-	-	-	-	-	-
Summary						9.94	**1** (0.51)

Table A4. Summary of Consequences for Sunny Day Breach Scenario

Items	Distance Downstream (km)	No. of Buildings Affected	Time of Arrival	Hazard Category (No. of Buildings)	Inundated Length (km)	PAR	Potential Loss of Life
Site Access Roads in Valley (AADT = 200)	0.0	-	<1 min	H6	3.2	0.67	0.0135
Process Plant	2.0	1	5 min	H6	-	200	0.0091
Administration Building	4.0	-	-	-	-	-	-
Thomson Gorge Road (AADT = 259)	5.5	-	-	-	-	-	-
Farmland	1.7	-	-	-	-	-	-
Buildings Downstream	6.5	5	-	-	-	-	-
Ardgour Road (AADT = 400)	7.0	-	-	-	-	-	-
SH8 Bridge (AADT = 1945)	10.5	-	-	-	-	-	-
Summary						201	**0** (0.0225)

Table A5. Summary of Consequences for Rainy Day Breach Scenario

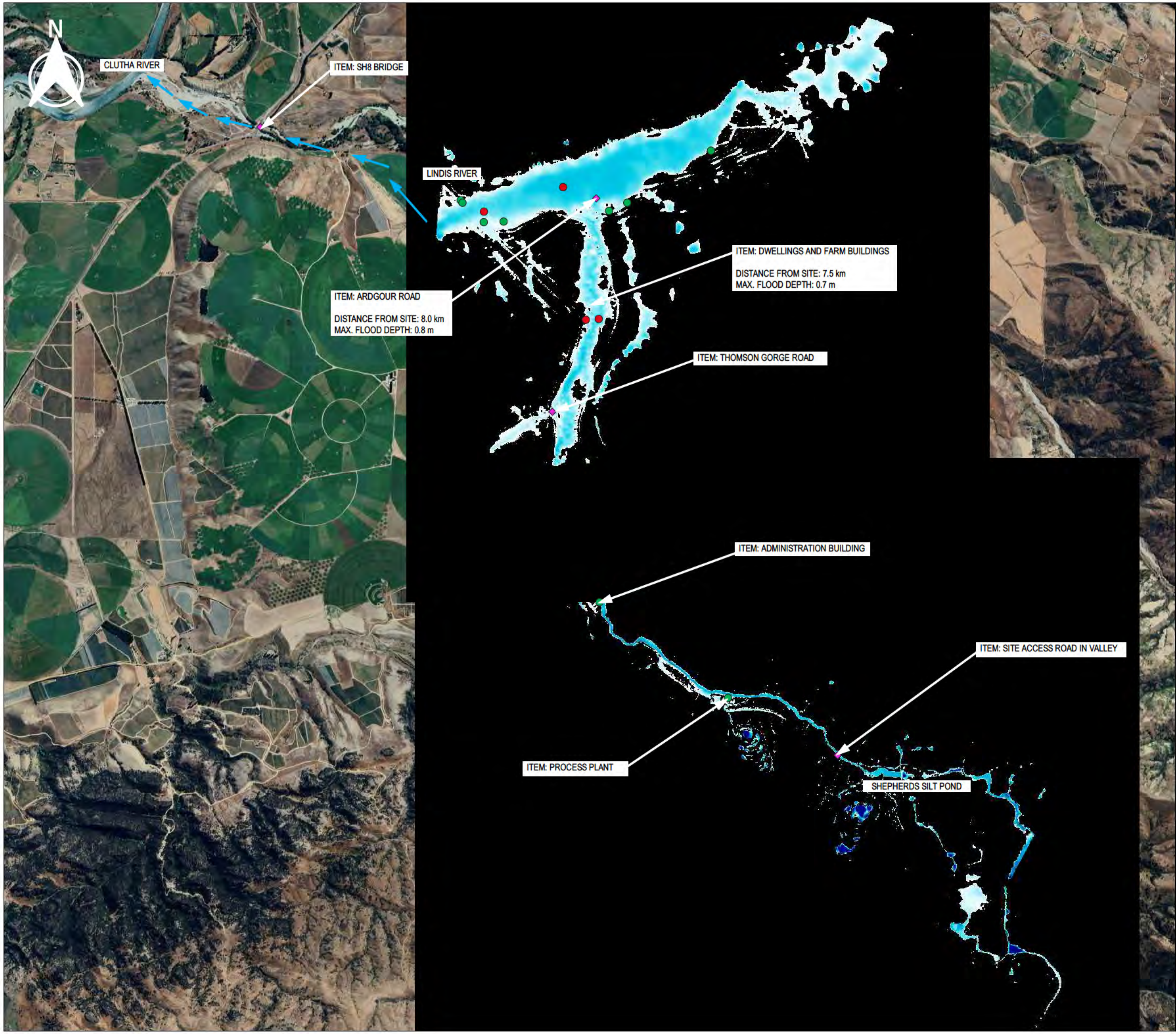
Items	Distance Downstream (km)	No. of Buildings Incrementally Affected	Time of Arrival	Hazard Category (No. of Buildings)	Inundated Length (km)	PAR	Potential Loss of Life
Site Access Roads in Valley (AADT = 200)	0.0	-	<1 min	H6	3.2	0.67	0.4715
Process Plant	2.0	1	<5 min	H6	-	30	0.0048
Administration Building	4.0	-	-	-	-	-	-
Thomson Gorge Road (AADT = 259)	5.5	-	35 min	H5	0.15	0.09	0.0014
Farmland (~100ha)	1.7	-	40 min	H5	-	0.11	0.0001
Buildings Downstream	6.5	4	40 min	H4 (1 dwelling) H5 (1 dwelling and 1 commercial) H6 (1 commercial)	-	8.10	0.0008
Ardgour Road (AADT = 400)	7.0	-	50 min	H5		1.08	0.0357
SH8 Bridge (AADT = 1945)	10.5	-	-	-	-	-	-
Summary						40	**1** (0.51)
Incremental Summary						30	**0** (0)

Table A6. Summary of PAR and Potential Loss of Life for Sunny Day Breach Scenario

Breach Scenario	**Item**	**PAR**	**Potential Loss of Life**	**Damage**				**Potential Impact Classification (PIC)**
				Community	**Cultural**	**Critical and Major Infrastructure**	**Natural Environment**	
Sunny Day Breach	Buildings	200.67	0.0091	Minimal	Minimal	Minimal	Minimal	**Low** PIC
	Roads & Bridges	0.67	0.0135					
	Farmlands	0.00	0.0000					
	Summary	**200.67**	**0** (0.0225)					

Table A7. Summary of PAR and Potential Loss of Life for Rainy Day Breach Scenario

Breach Scenario	Item	PAR	Potential Loss of Life	Damage				Potential Impact Classification (PIC)
				Community	Cultural	Critical and Major Infrastructure	Natural Environment	
Rainy Day Base Flood (PMP – 1 hr)	Buildings	8.10	0.0008	-	-	-	-	-
	Roads & Bridges	1.75	0.5072					
	Farmlands	0.08	0.0000					
	Summary	9.94	1 (0.5080)					
Rainy Day Breach with Base Flood (PMP – 1 hr)	Buildings	38.10	0.0056	Minimal	Minimal	Minimal	Minimal	**Low** PIC
	Roads & Bridges	1.85	0.5086					
	Farmlands	0.11	0.0001					
	Summary	40.06	1 (0.5142)					
	Incremental Summary	**30**	**0** (0.01)					



FLOOD DEPTH (m)

- 0
- 1
- 2
- 3
- 4
- 5

- ROADS AND BRIDGES
- ITEMS OF INTEREST NOT AFFECTED
- ITEMS OF INTEREST INUNDATED

NOTES:
1. SATELLITE IMAGE: GOOGLE SATELLITE IMAGE

0.25 0 0.25 0.5 km

REFERENCE SCALE: 1:50,000 @A3
PROJECTION: NZGD 2000 NEW ZEALAND TRANSVERSE MERCATOR

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TITLE
SHEPHERDS SILT POND DAM BREACH INUNDATION MAP

BASE FLOOD PMP 1 hr
FLOOD DEPTHS MAP

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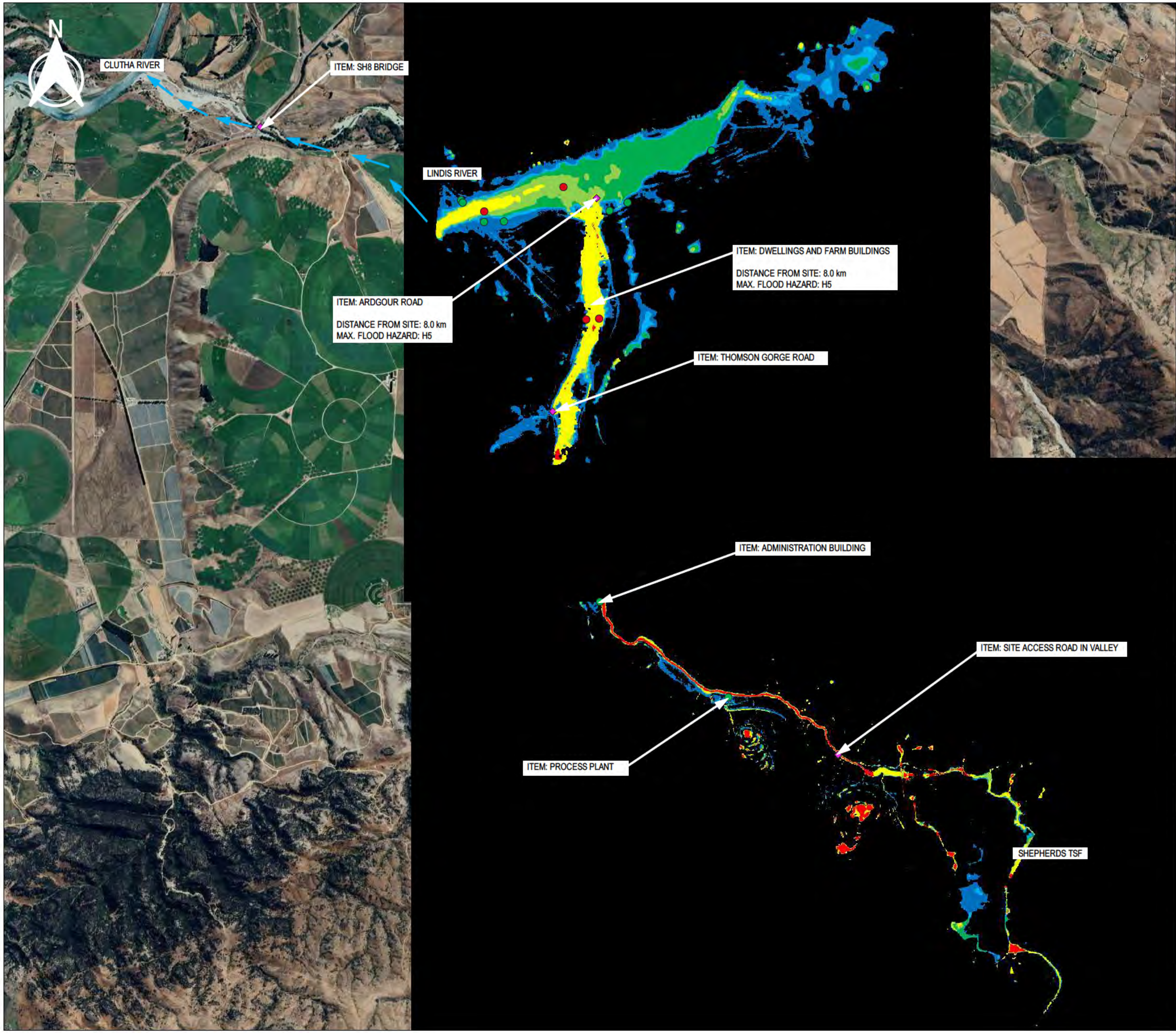

FLOOD HAZARD CATEGORY

- H1
- H2
- H3
- H4
- H5
- H6

- ROADS AND BRIDGES
- ITEMS OF INTEREST NOT AFFECTED
- ITEMS OF INTEREST INUNDATED

NOTES:
1. SATELLITE IMAGE: GOOGLE SATELLITE IMAGE

0.25 0 0.25 0.5 km

REFERENCE SCALE: 1:50,000 @A3
PROJECTION: NZGD 2000 NEW ZEALAND TRANSVERSE MERCATOR

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BASE FLOOD PMP 1 hr
FLOOD HAZARD MAP

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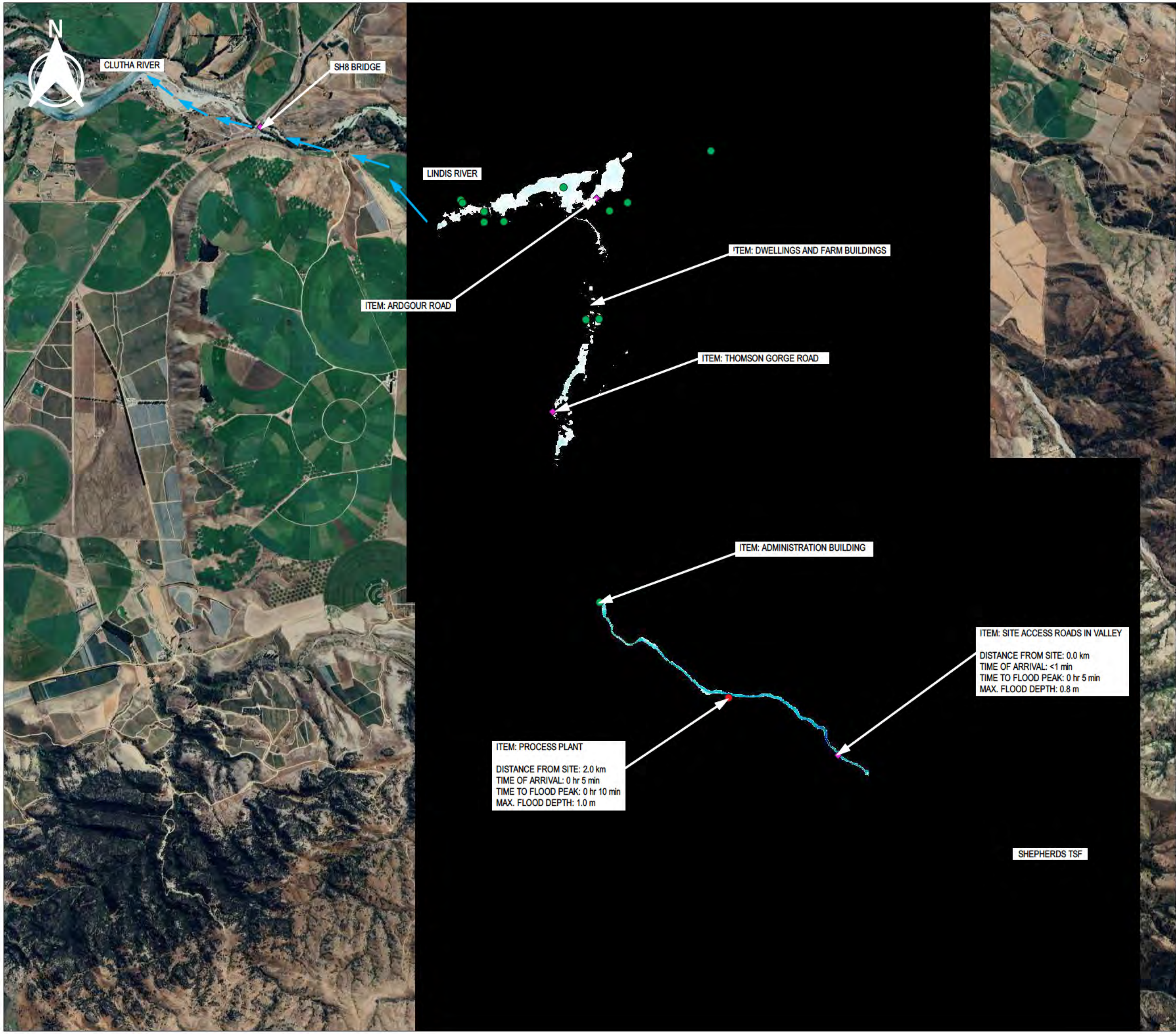

FLOOD DEPTH (m)

- 0
- 1
- 2
- 3
- 4
- 5

- ROADS AND BRIDGES
- ITEMS OF INTEREST NOT AFFECTED
- ITEMS OF INTEREST INUNDATED

NOTES:
1. SATELLITE IMAGE: GOOGLE SATELLITE IMAGE

0.25 0 0.25 0.5 km

REFERENCE SCALE: 1:50,000 @A3
PROJECTION: NZGD 2000 NEW ZEALAND TRANSVERSE MERCATOR

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TITLE
SHEPHERDS SILT POND DAM BREACH INUNDATION MAP

SUNNY DAY BREACH
FLOOD DEPTHS MAP

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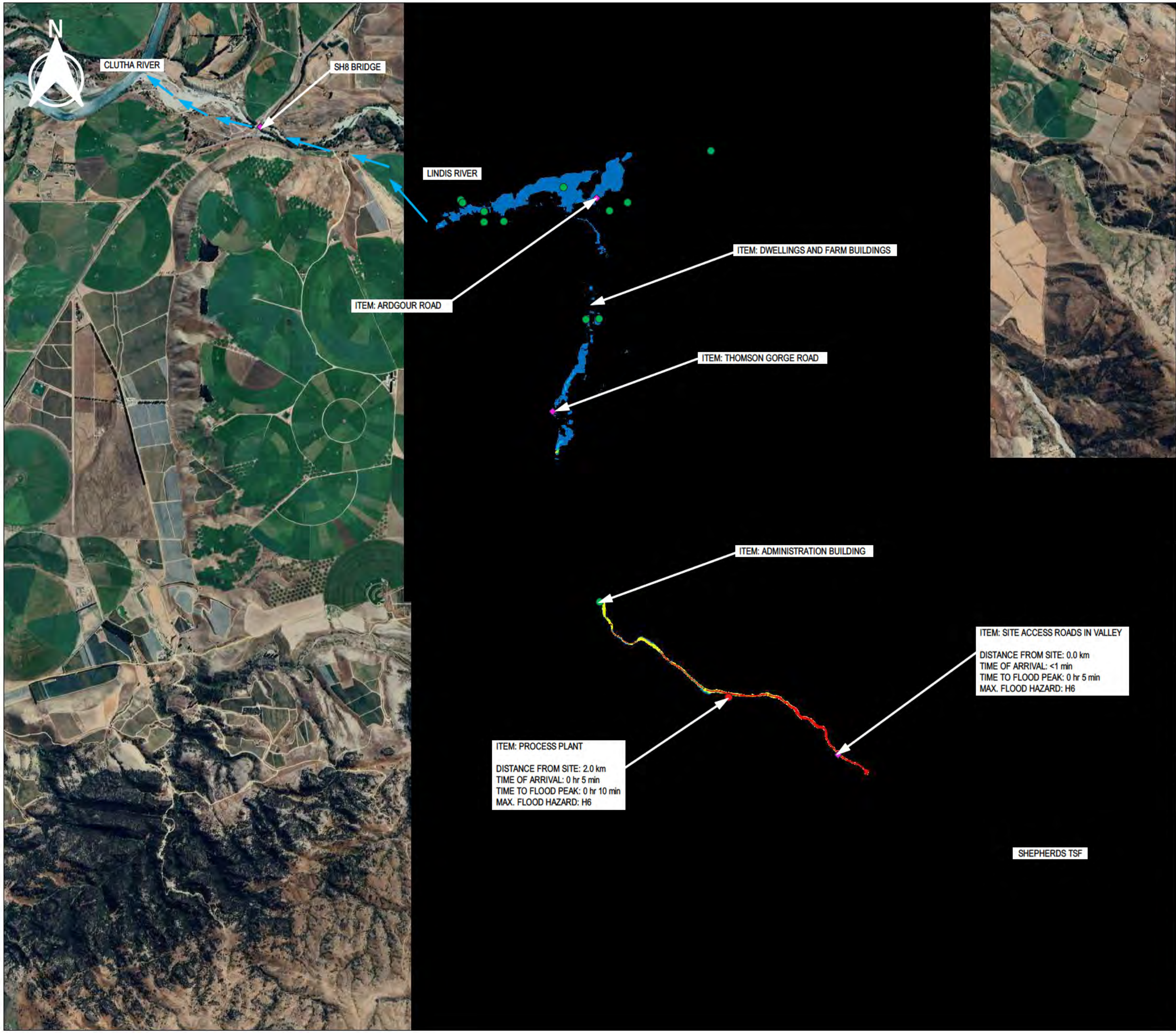

FLOOD HAZARD CATEGORY

- H1
- H2
- H3
- H4
- H5
- H6

- ROADS AND BRIDGES
- ITEMS OF INTEREST NOT AFFECTED
- ITEMS OF INTEREST INUNDATED

NOTES:
1. SATELLITE IMAGE: GOOGLE SATELLITE IMAGE

0.25 0 0.25 0.5 km

REFERENCE SCALE: 1:50,000 @A3
PROJECTION: NZGD 2000 NEW ZEALAND TRANSVERSE MERCATOR

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TITLE
SHEPHERDS SILT POND DAM BREACH
INUNDATION MAP

SUNNY DAY BREACH
FLOOD HAZARD MAP

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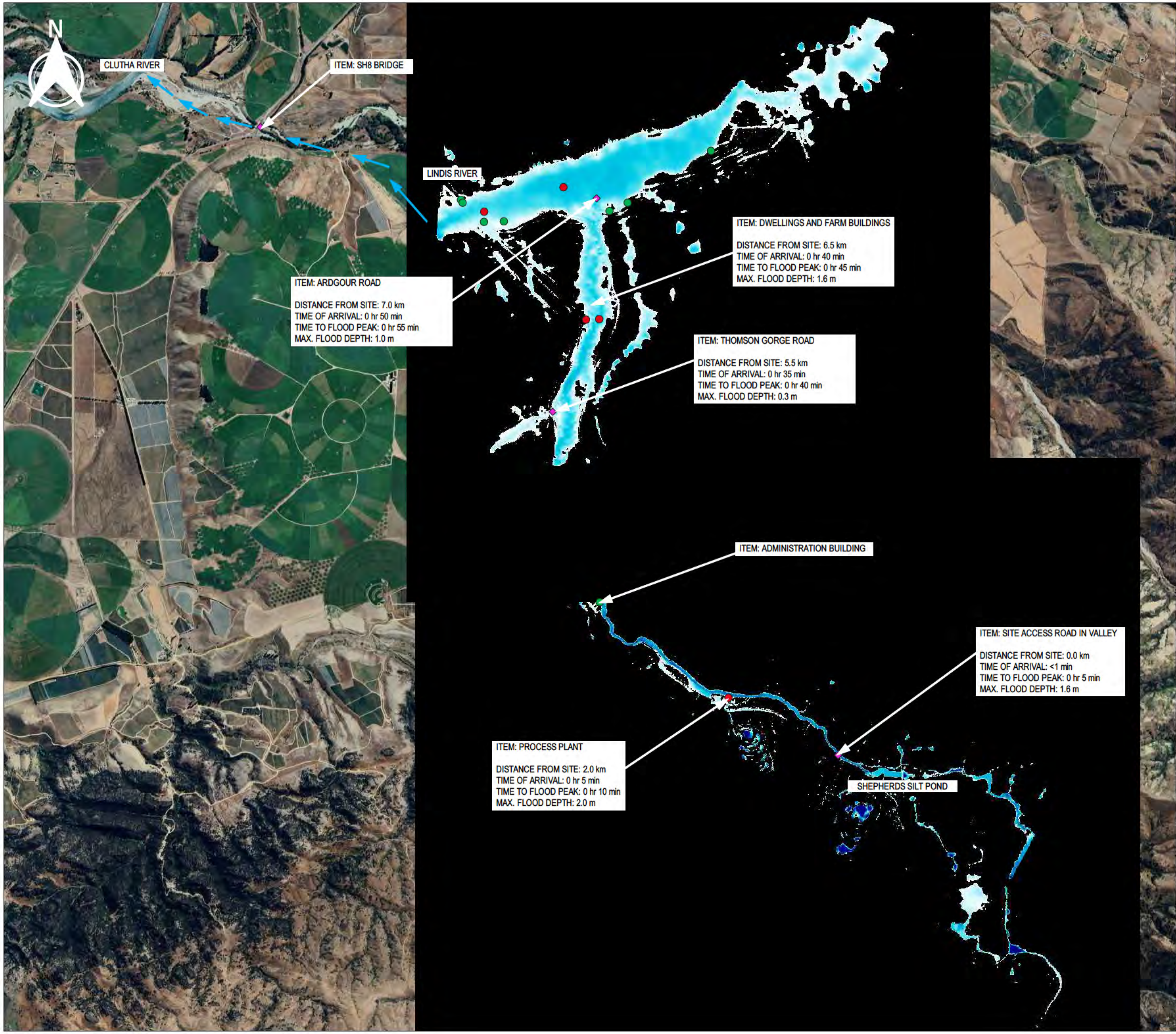

FLOOD DEPTH (m)

- 0
- 1
- 2
- 3
- 4
- 5

- ROADS AND BRIDGES
- ITEMS OF INTEREST NOT AFFECTED
- ITEMS OF INTEREST INUNDATED

NOTES:
1. SATELLITE IMAGE: GOOGLE SATELLITE IMAGE

0.25 0 0.25 0.5 km

REFERENCE SCALE: 1:50,000 @A3
PROJECTION: NZGD 2000 NEW ZEALAND TRANSVERSE MERCATOR

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TITLE
SHEPHERDS SILT POND DAM BREACH
INUNDATION MAP

RAINY DAY BREACH
FLOOD DEPTHS MAP

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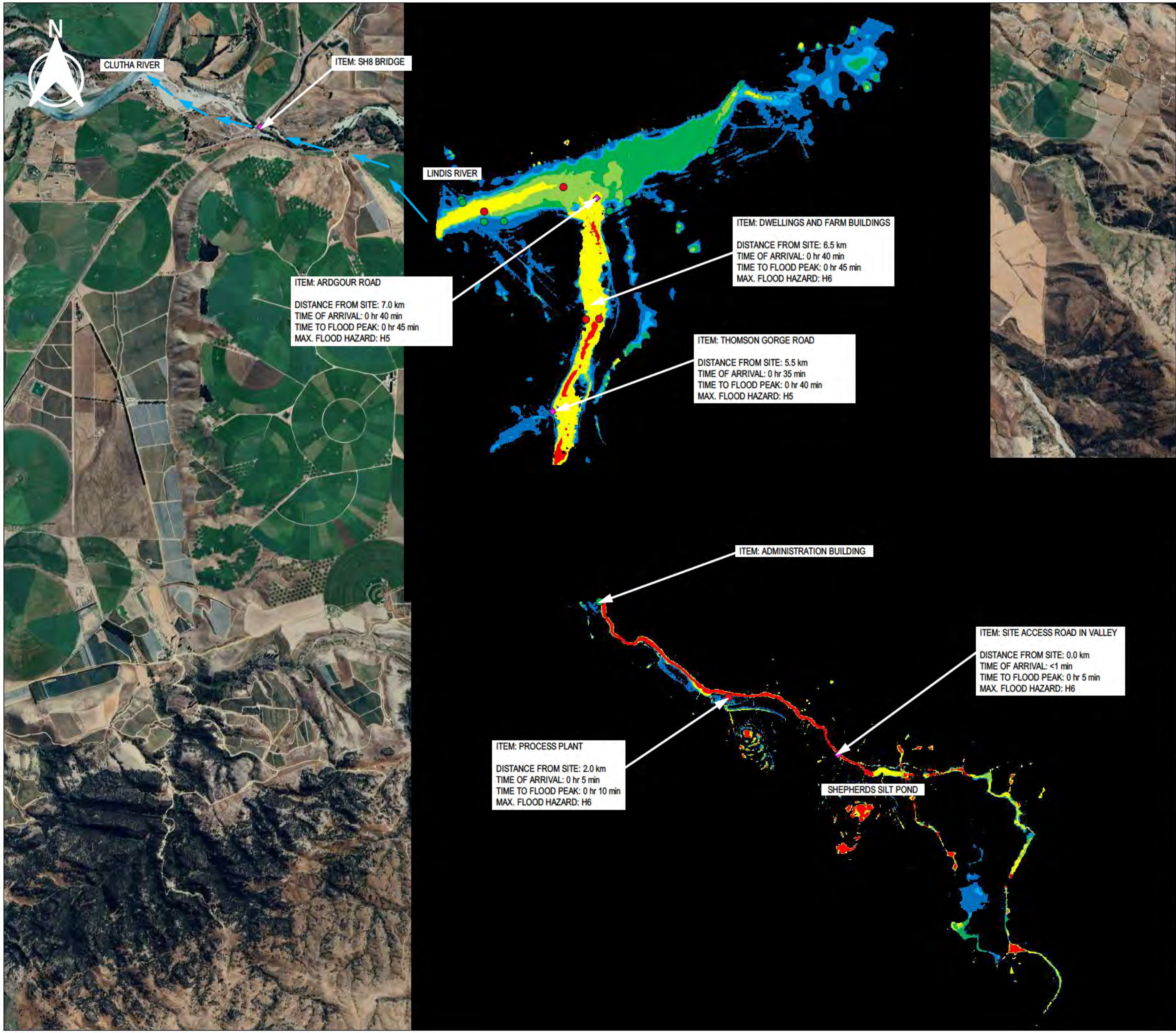

FLOOD HAZARD CATEGORY

- H1
- H2
- H3
- H4
- H5
- H6

- ROADS AND BRIDGES
- ITEMS OF INTEREST NOT AFFECTED
- ITEMS OF INTEREST INUNDATED

NOTES:
1. SATELLITE IMAGE: GOOGLE SATELLITE IMAGE

0.25 0 0.25 0.5 km

REFERENCE SCALE: 1:50,000 @A3
PROJECTION: NZGD 2000 NEW ZEALAND TRANSVERSE MERCATOR

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TITLE
SHEPHERDS SILT POND DAM BREACH
INUNDATION MAP

RAINY DAY BREACH
FLOOD HAZARD MAP

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DATE: JULY 2025 DRAWN BY: PQ FIGURE A6

APPENDIX B
STABILITY ANALYSES

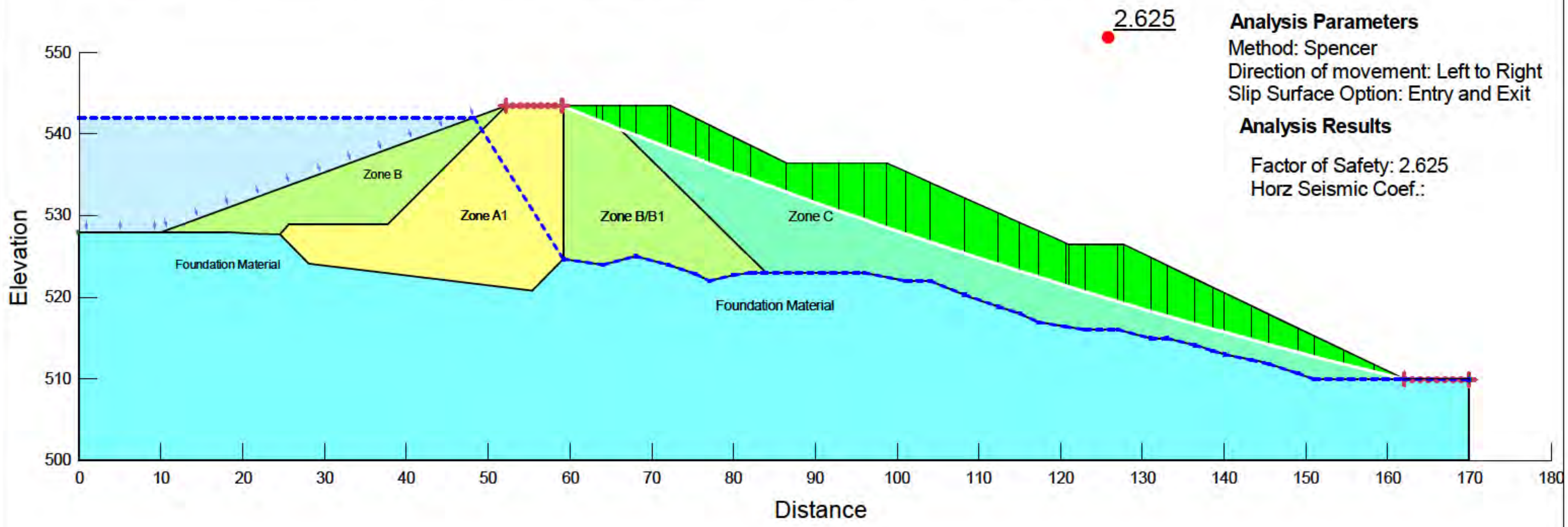
Table B1. Design Earthquake Response Spectra, NSHM 2024 (Vs30 = 1,000 m/s)

Bendigo (Vs30 = 1,000 m/s)		
Period (s)	**OBE** **1 in 150 AEP**	**SEE** **1 in 1,000 AEP**
PGA	0.120	0.313
0.02	0.127	0.333
0.03	0.144	0.379
0.04	0.166	0.441
0.075	0.243	0.655
0.1	0.268	0.725
0.15	0.280	0.758
0.2	0.262	0.706
0.25	0.237	0.637
0.3	0.214	0.574
0.4	0.178	0.478
0.5	0.150	0.407
0.75	0.109	0.298
1	0.084	0.231
1.5	0.055	0.150
2	0.041	0.114
3	0.024	0.072
4	0.018	0.054
5	0.013	0.042
6	0.010	0.032
7.5	0.008	0.024
10	0.005	0.016

Design Earthquake	**OBE** **1 in 150 AEP**	**SEE** **1 in 1,000 AEP**
PGA (g)	0.120	0.313
M_w[1]	6.3	7.1

Note:
1. Base on the disaggregation results for SA(0.15)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	UCS Intact (kPa)	Parameter mb	Parameter s	Parameter a	Calculated from	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Max Confining Stress Sigma 3 (kPa)
	Foundation Material	Hoek-Brown	21.5				35,000	1.6830724	0.0022180849	0.50808574	Yes	12	45	0	5,000
	Zone A	Mohr-Coulomb	22		0	34									
	Zone B	Shear/Normal Fn.	22	Function B - 2.133 x Sigma^0.87											
	Zone C	Shear/Normal Fn.	21.5	FunctionC - 1.29xSigma^0.91											



Crest To Toe Downstream

Figure B1

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Stability Analysis

Date: 25/03/2025
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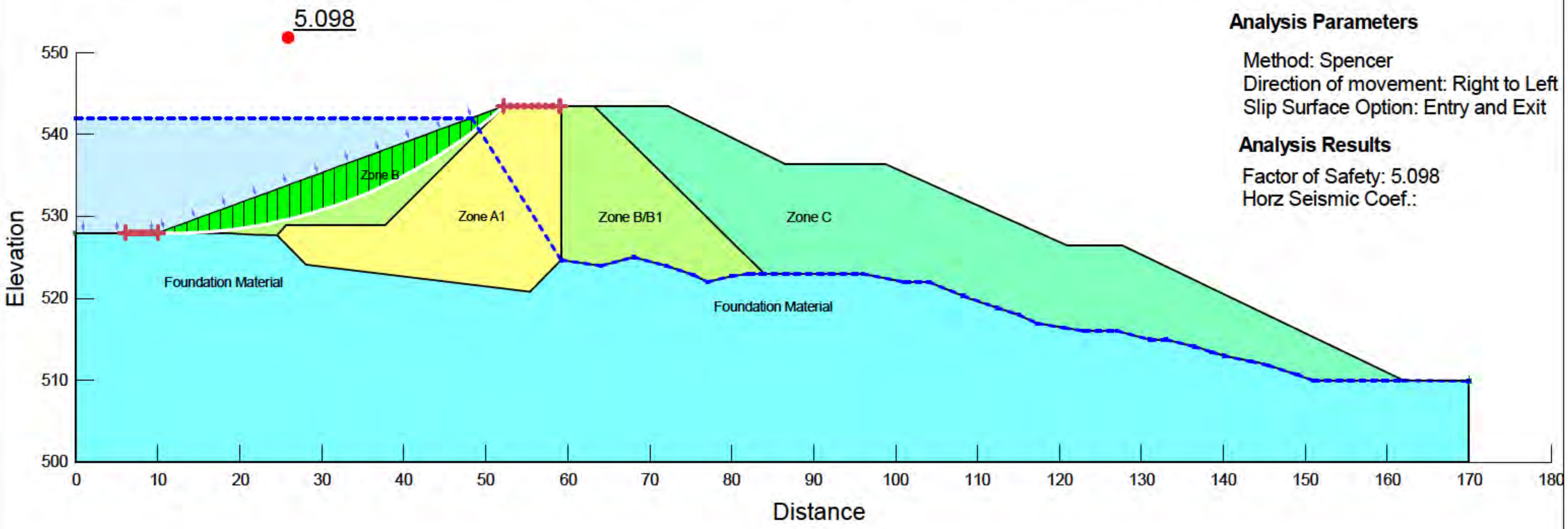
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	UCS Intact (kPa)	Parameter mb	Parameter s	Parameter a	Calculated from	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Max Confining Stress Sigma 3 (kPa)
	Foundation Material	Hoek-Brown	21.5				35,000	1.6830724	0.0022180849	0.50808574	Yes	12	45	0	5,000
	Zone A	Mohr-Coulomb	22		0	34									
	Zone B	Shear/Normal Fn.	22	Function B - 2.133 x Sigma^0.87											
	Zone C	Shear/Normal Fn.	21.5	FunctionC - 1.29xSigma^0.91											



Analysis Parameters

Method: Spencer
Direction of movement: Right to Left
Slip Surface Option: Entry and Exit

Analysis Results

Factor of Safety: 5.098
Horz Seismic Coef.:

Crest To Toe Upstream

Figure B2

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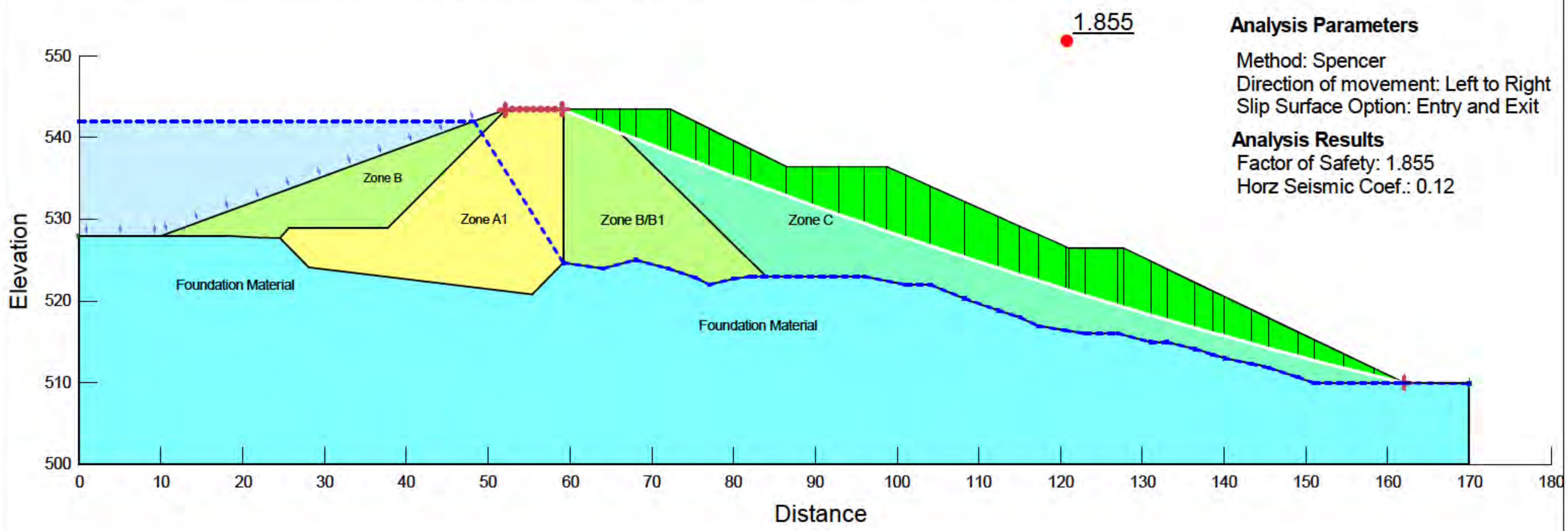
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	UCS Intact (kPa)	Parameter mb	Parameter s	Parameter a	Calculated from	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Max Confining Stress Sigma 3 (kPa)
	Foundation Material	Hoek-Brown	21.5				35,000	1.6830724	0.0022180849	0.50808574	Yes	12	45	0	5,000
	Zone A	Mohr-Coulomb	22		0	34									
	Zone B	Shear/Normal Fn.	22	Function B - 2.133 x Sigma^0.87											
	Zone C	Shear/Normal Fn.	21.5	FunctionC - 1.29xSigma^0.91											



OBE Full Depth

Figure B3

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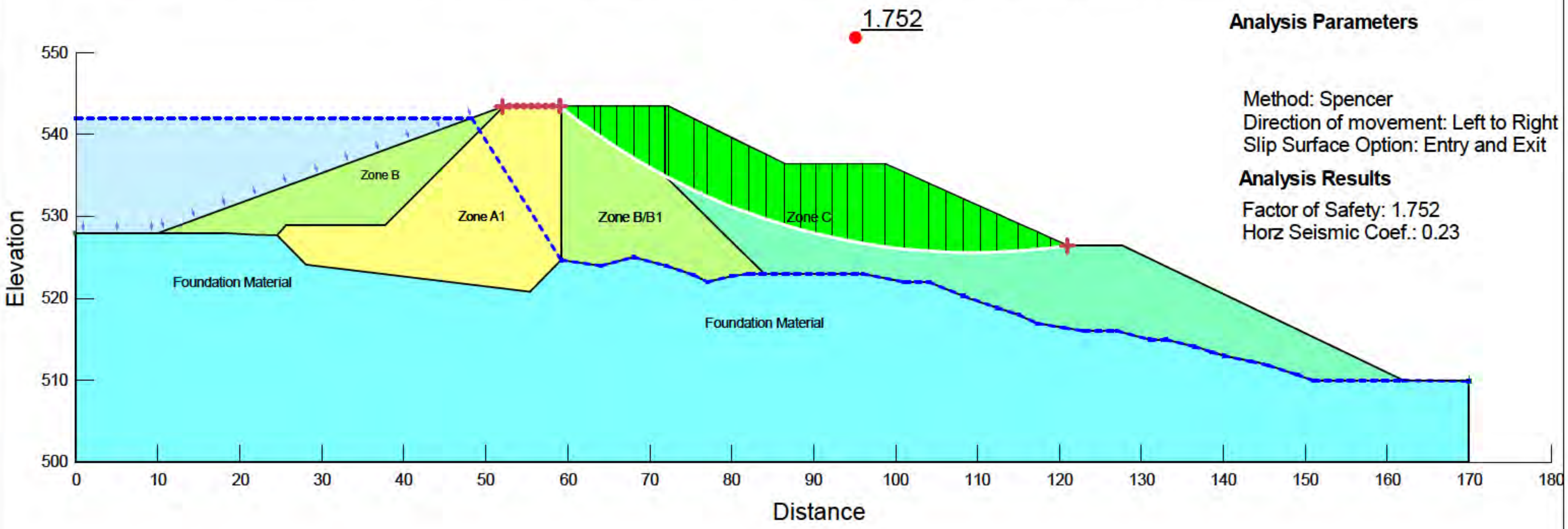
Ref: 9702
Drawn: PQ

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	UCS Intact (kPa)	Parameter mb	Parameter s	Parameter a	Calculated from	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Max Confining Stress Sigma 3 (kPa)
	Foundation Material	Hoek-Brown	21.5				35,000	1.6830724	0.0022180849	0.50808574	Yes	12	45	0	5,000
	Zone A	Mohr-Coulomb	22		0	34									
	Zone B	Shear/Normal Fn.	22	Function B - 2.133 x Sigma^0.87											
	Zone C	Shear/Normal Fn.	21.5	FunctionC - 1.29xSigma^0.91											



OBE - Failure 2/3H below crest

Figure B4

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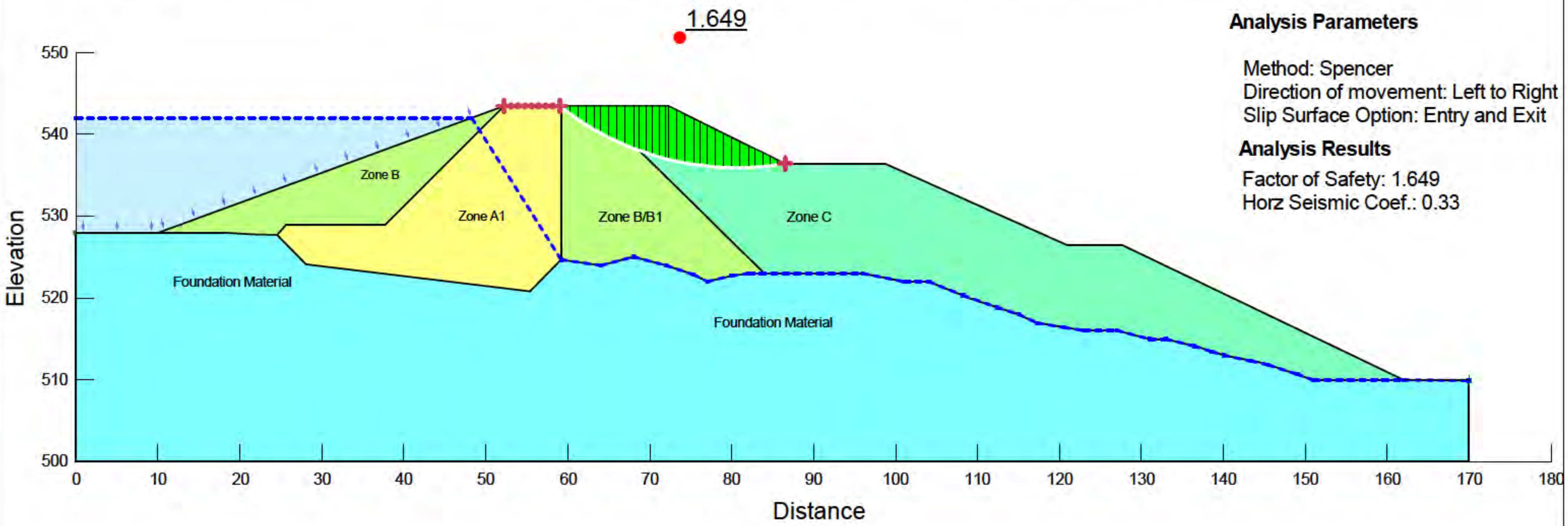
Stability Analysis

Date: 25/03/2025
Ref: 9702
Drawn: PQ

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	UCS Intact (kPa)	Parameter mb	Parameter s	Parameter a	Calculated from	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Max Confining Stress Sigma 3 (kPa)
	Foundation Material	Hoek-Brown	21.5				35,000	1.6830724	0.0022180849	0.50808574	Yes	12	45	0	5,000
	Zone A	Mohr-Coulomb	22		0	34									
	Zone B	Shear/Normal Fn.	22	Function B - 2.133 x Sigma^0.87											
	Zone C	Shear/Normal Fn.	21.5	FunctionC - 1.29xSigma^0.91											



OBE - Failure 1/3H below Crest

Figure B5

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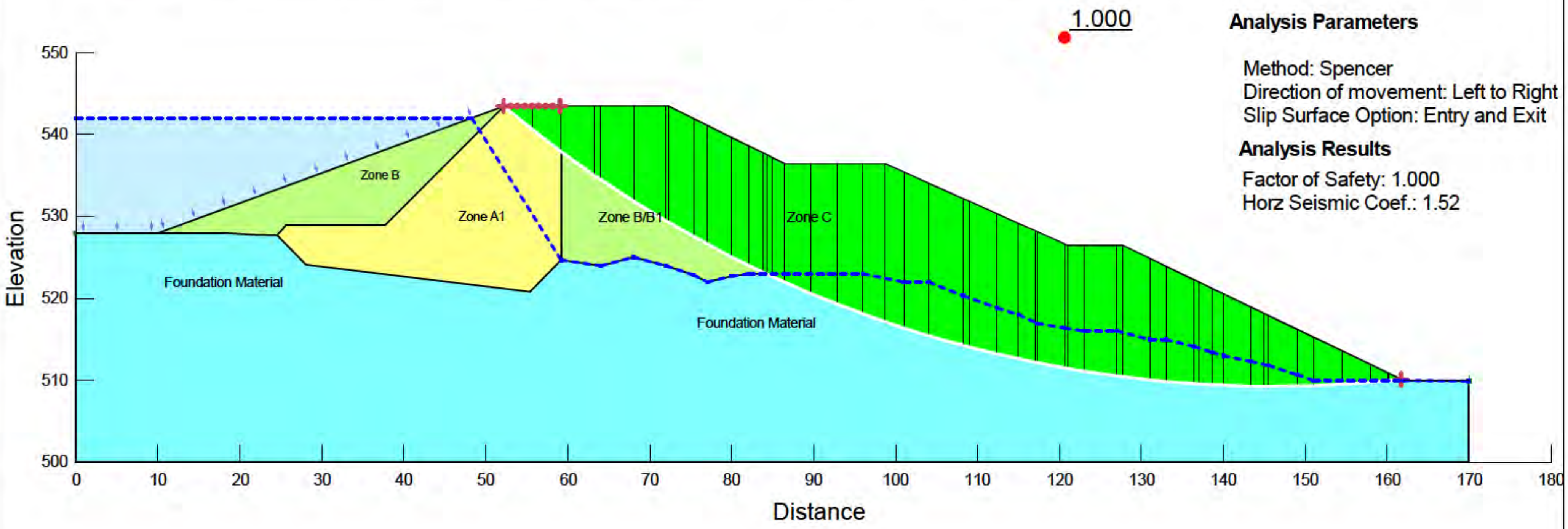
Stability Analysis

Date: 25/03/2025
Ref: 9702
Drawn: PQ

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	UCS Intact (kPa)	Parameter mb	Parameter s	Parameter a	Calculated from	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Max Confining Stress Sigma 3 (kPa)
	Foundation Material	Hoek-Brown	21.5				35,000	1.6830724	0.0022180849	0.50808574	Yes	12	45	0	5,000
	Zone A	Mohr-Coulomb	22		0	34									
	Zone B	Shear/Normal Fn.	22	Function B - 2.133 x Sigma^0.87											
	Zone C	Shear/Normal Fn.	21.5	FunctionC - 1.29xSigma^0.91											



Analysis Parameters

Method: Spencer
Direction of movement: Left to Right
Slip Surface Option: Entry and Exit

Analysis Results

Factor of Safety: 1.000
Horz Seismic Coef.: 1.52

Yield Acceleration Full Depth

Figure B6

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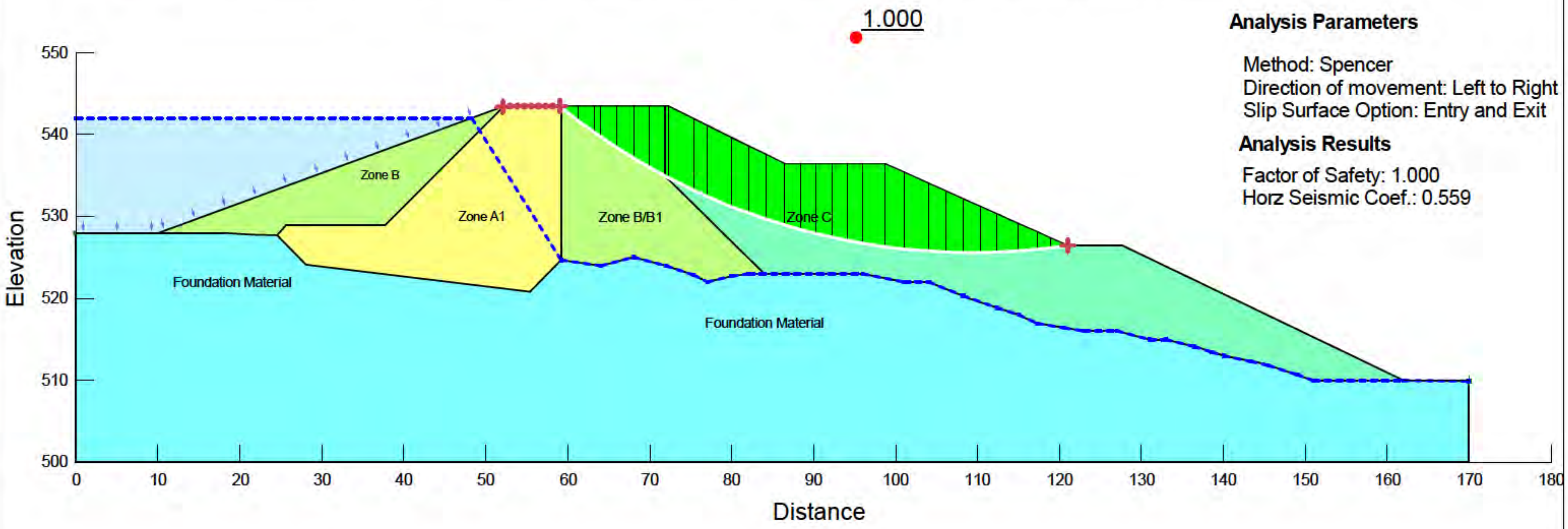
Stability Analysis

Date: 25/03/2025
Ref: 9702
Drawn: PQ

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	UCS Intact (kPa)	Parameter mb	Parameter s	Parameter a	Calculated from	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Max Confining Stress Sigma 3 (kPa)
	Foundation Material	Hoek-Brown	21.5				35,000	1.6830724	0.0022180849	0.50808574	Yes	12	45	0	5,000
	Zone A	Mohr-Coulomb	22		0	34									
	Zone B	Shear/Normal Fn.	22	Function B - 2.133 x Sigma^0.87											
	Zone C	Shear/Normal Fn.	21.5	FunctionC - 1.29xSigma^0.91											



Analysis Parameters

Method: Spencer
Direction of movement: Left to Right
Slip Surface Option: Entry and Exit

Analysis Results

Factor of Safety: 1.000
Horz Seismic Coef.: 0.559

Yield Acceleartion - Failure 2/3H below Crest

Figure B7

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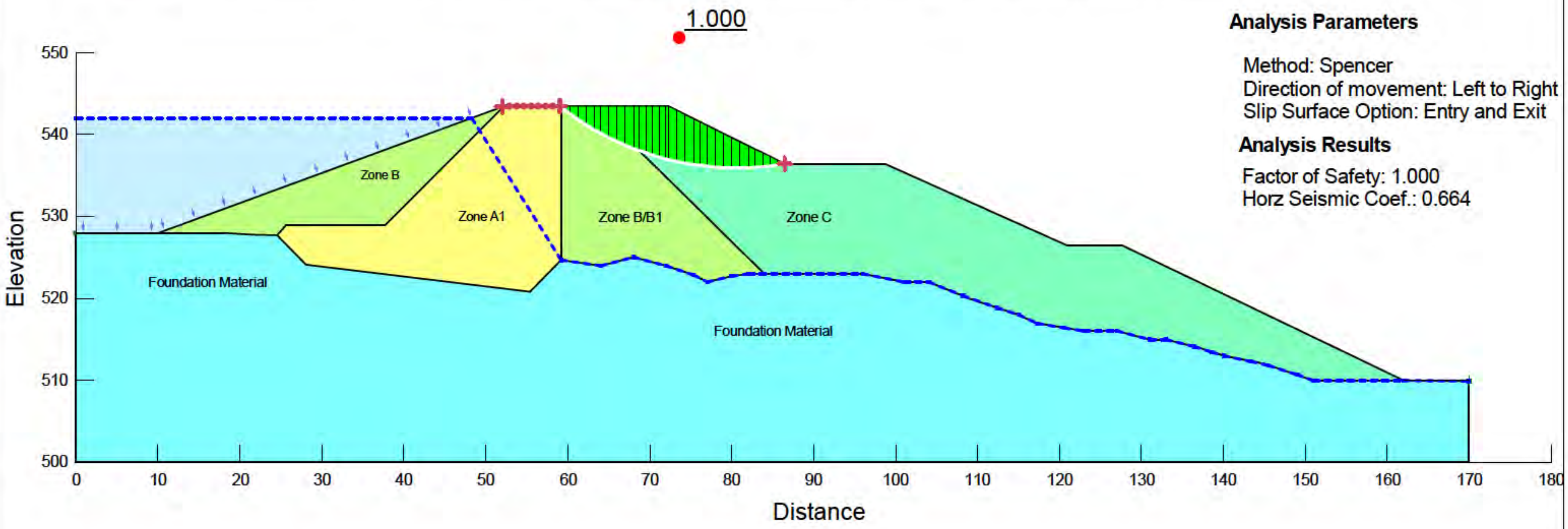
Stability Analysis

Date: 25/03/2025
Ref: 9702
Drawn: PQ

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Strength Function	Effective Cohesion (kPa)	Effective Friction Angle (°)	UCS Intact (kPa)	Parameter mb	Parameter s	Parameter a	Calculated from	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Max Confining Stress Sigma 3 (kPa)
	Foundation Material	Hoek-Brown	21.5				35,000	1.6830724	0.0022180849	0.50808574	Yes	12	45	0	5,000
	Zone A	Mohr-Coulomb	22		0	34									
	Zone B	Shear/Normal Fn.	22	Function B - 2.133 x Sigma^0.87											
	Zone C	Shear/Normal Fn.	21.5	FunctionC - 1.29xSigma^0.91											



Analysis Parameters

Method: Spencer
Direction of movement: Left to Right
Slip Surface Option: Entry and Exit

Analysis Results

Factor of Safety: 1.000
Horz Seismic Coef.: 0.664

Yield Acceleration - Failure located 1/3H below crest

Figure B8

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Stability Analysis

Date: 25/03/2025
Ref: 9702
Drawn: PQ

APPENDIX C
HYDRAULIC DESIGN CALCULATIONS

List of Table in Appendix C

Table C1. High Intensity Rainfall Database (HIRDS V4) Historical and 2031 to 2050

Coordinate system: WGS84

Longitude: 169.457; Latitude: -44.9474

Rainfall depths (mm): Historical Data

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	3.14	4.57	5.72	8.36	12.1	21.3	29.6	39.8	51.6	59	64.2	68.2
2	0.5	3.57	5.18	6.45	9.38	13.6	23.6	32.6	43.6	56.3	64.1	69.6	73.7
5	0.2	5.23	7.47	9.22	13.2	18.8	31.8	43.2	56.9	72.2	81.4	87.8	92.6
10	0.1	6.67	9.41	11.5	16.3	23	38.3	51.5	67	84.2	94.4	101	106
20	0.05	8.33	11.6	14.2	19.9	27.7	45.4	60.3	77.8	96.7	108	115	121
30	0.033	9.44	13.1	15.9	22.2	30.7	49.9	65.9	84.4	104	116	124	129
40	0.025	10.3	14.2	17.2	23.9	33	53.2	70	89.3	110	122	130	135
50	0.02	11	15.1	18.3	25.3	34.8	55.9	73.2	93.2	114	126	134	140
60	0.017	11.6	15.9	19.2	26.5	36.3	58.1	76	96.4	118	130	138	144
80	0.013	12.6	17.2	20.7	28.5	38.9	61.7	80.4	102	124	136	144	150
100	0.01	13.4	18.3	22	30.1	40.9	64.7	83.9	106	128	141	149	155
250	0.004	17.2	23.1	27.6	37.2	50	77.4	99.1	123	148	161	170	175

Rainfall depths (mm): Climate Change Scenario Model RCP8.5 for the period 2031-2050

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	3.45	5.04	6.29	9.21	13.3	23	31.6	42.2	54.2	61.5	66.8	70.8
2	0.5	3.94	5.71	7.12	10.4	14.9	25.6	34.9	46.3	59.2	67.1	72.6	76.7
5	0.2	5.8	8.28	10.2	14.6	20.7	34.7	46.6	60.6	76.3	85.7	92.1	96.9
10	0.1	7.41	10.5	12.8	18.2	25.4	41.9	55.6	71.6	89.2	99.6	106	112
20	0.05	9.28	13	15.8	22.1	30.7	49.7	65.3	83.2	103	114	121	127
30	0.033	10.5	14.6	17.7	24.7	34.1	54.6	71.4	90.4	111	122	130	136
40	0.025	11.5	15.8	19.2	26.6	36.6	58.3	75.9	95.7	117	129	137	142
50	0.02	12.2	16.9	20.4	28.2	38.6	61.2	79.4	99.8	121	134	142	147
60	0.017	12.9	17.8	21.4	29.6	40.3	63.7	82.4	103	125	138	146	151
80	0.013	14	19.2	23.1	31.8	43.2	67.7	87.2	109	132	144	152	158
100	0.01	14.9	20.4	24.5	33.5	45.4	71	91.1	113	137	149	158	163
250	0.004	19.1	25.8	30.8	41.6	55.6	85	108	132	157	170	179	185

Ref. https://hirds.niwa.co.nz/

Table C2. High Intensity Rainfall Database (HIRDS V4) 2081 to 2100

Coordinate system: WGS84

Longitude: 169.457; Latitude: -44.9474

Rainfall depths (mm): RCP8.5 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	4.1	5.98	7.47	10.9	15.7	26.5	35.7	47	59.4	66.7	72	76.1
2	0.5	4.69	6.81	8.48	12.3	17.7	29.6	39.7	51.7	65.1	73.2	78.7	82.9
5	0.2	6.96	9.93	12.3	17.5	24.7	40.4	53.4	68.3	84.7	94.5	101	106
10	0.1	8.92	12.6	15.4	21.9	30.5	49	64.1	81	99.4	110	117	122
20	0.05	11.2	15.6	19.1	26.7	36.8	58.4	75.4	94.3	115	126	134	139
30	0.033	12.7	17.6	21.4	29.8	40.9	64.3	82.5	103	124	136	144	149
40	0.025	13.8	19.1	23.2	32.2	43.9	68.7	87.9	109	131	143	151	156
50	0.02	14.8	20.4	24.7	34.1	46.5	72.2	92	113	136	148	157	162
60	0.017	15.6	21.5	25.9	35.7	48.5	75.2	95.6	118	140	153	161	166
80	0.013	17	23.3	28	38.5	52	79.9	101	124	148	160	168	174
100	0.01	18.1	24.7	29.7	40.6	54.7	83.8	106	129	153	166	174	180
250	0.004	23.2	31.2	37.2	50.3	66.9	100	125	151	176	190	198	203

Table C3. Summary of Silt Pond Hydraulic Routing (1 in 10 Year)

Duration (hr)	Inflow to Silt Pond (m^3/s)	Storage (m^3)	Elevation (mRL)	Primary Spillway Outflow (m^3/s)	Freeboard to crest (m)
1	5.9	56,300	540.31	0.3	2.69
2	5.8	61,000	540.74	1.7	2.26
6	3.7	63,400	540.95	2.7	2.05
12	2.8	62,500	540.87	2.6	2.13
24	2.0	61,300	540.76	2.0	2.24
48	1.8	61,000	540.74	1.8	2.26
72	1.6	60,900	540.72	1.6	2.28

Table C4. Summary of Silt Pond Hydraulic Routing (1 in 1,000 Year)

Duration (hr)	Inflow to Silt Pond (m^3/s)	Storage (m^3)	Elevation (mRL)	Auxiliary Spillway Outflow (m^3/s)	Freeboard to crest (m)
1	25.5	76,400	542.01	22.4	1.00
2	24.0	76,700	542.03	23.2	0.97
6	21.8	76,200	541.99	21.7	1.01
12	12.7	72,600	541.70	12.7	1.30
24	9.6	71,100	541.58	9.5	1.42
48	10.9	71,700	541.63	10.9	1.37
72	9.9	71,300	541.60	9.9	1.40

List of Figures in Appendix C

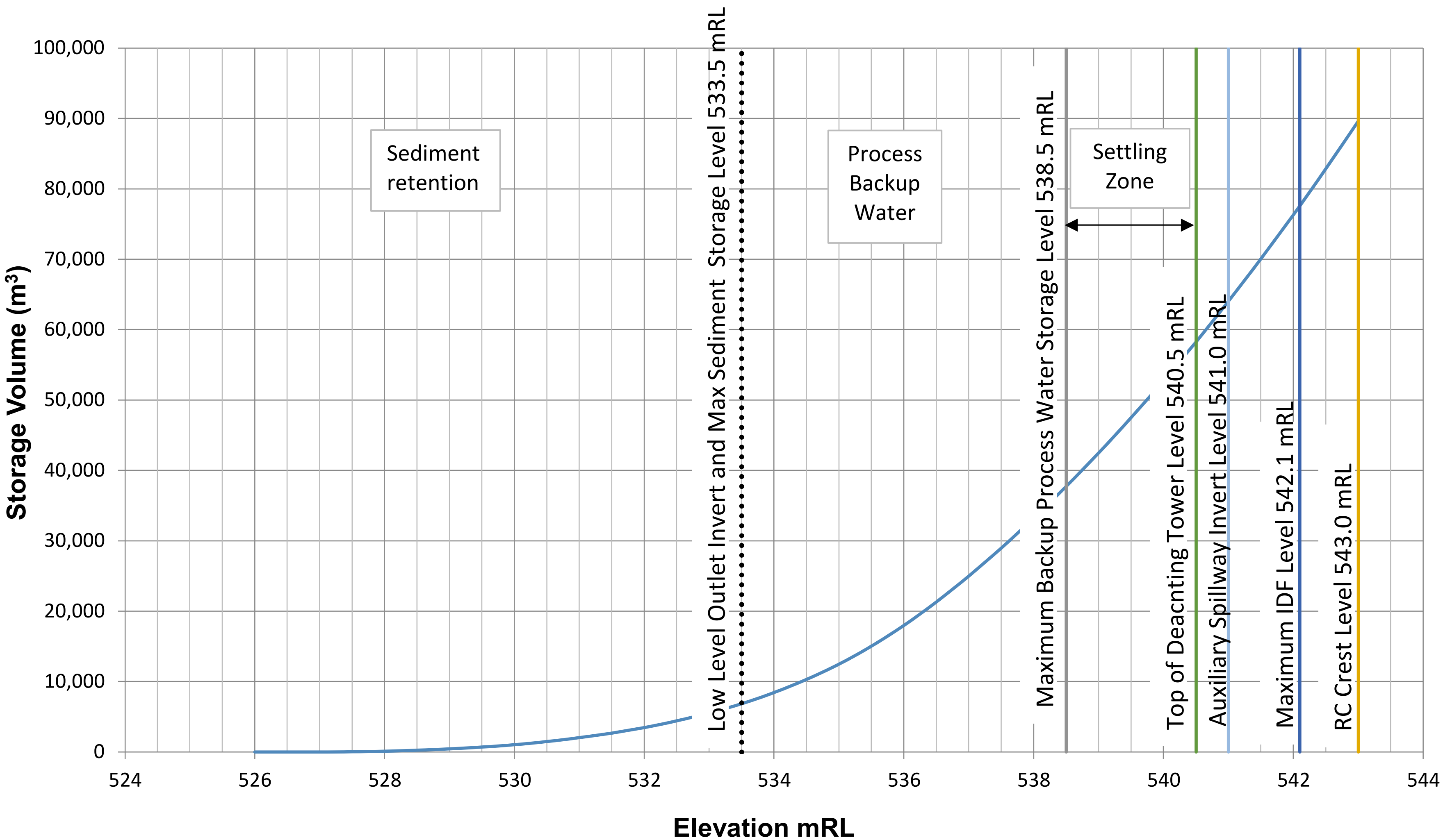

Figure C1

Note: Decanting Tower act as Primary Spillway

EGL Geotechnical, Earthquake and Dam Engineers Engineering Geology Ltd +64 9 486 2546 info@egl.co.nz Unit 7C, 331 Rosedale Road, Albany, Auckland PO Box 301054, Albany, Auckland 0752 www.egl.co.nz	MATAKANUI GOLD LIMITED BENDIGO OPHIR GOLD PROJECT SHEPHERDS SILT POND *Elevation-Storage Curve*	Ref. No: 9702 Date: Mar 2025 Drawn: ET File: 9702

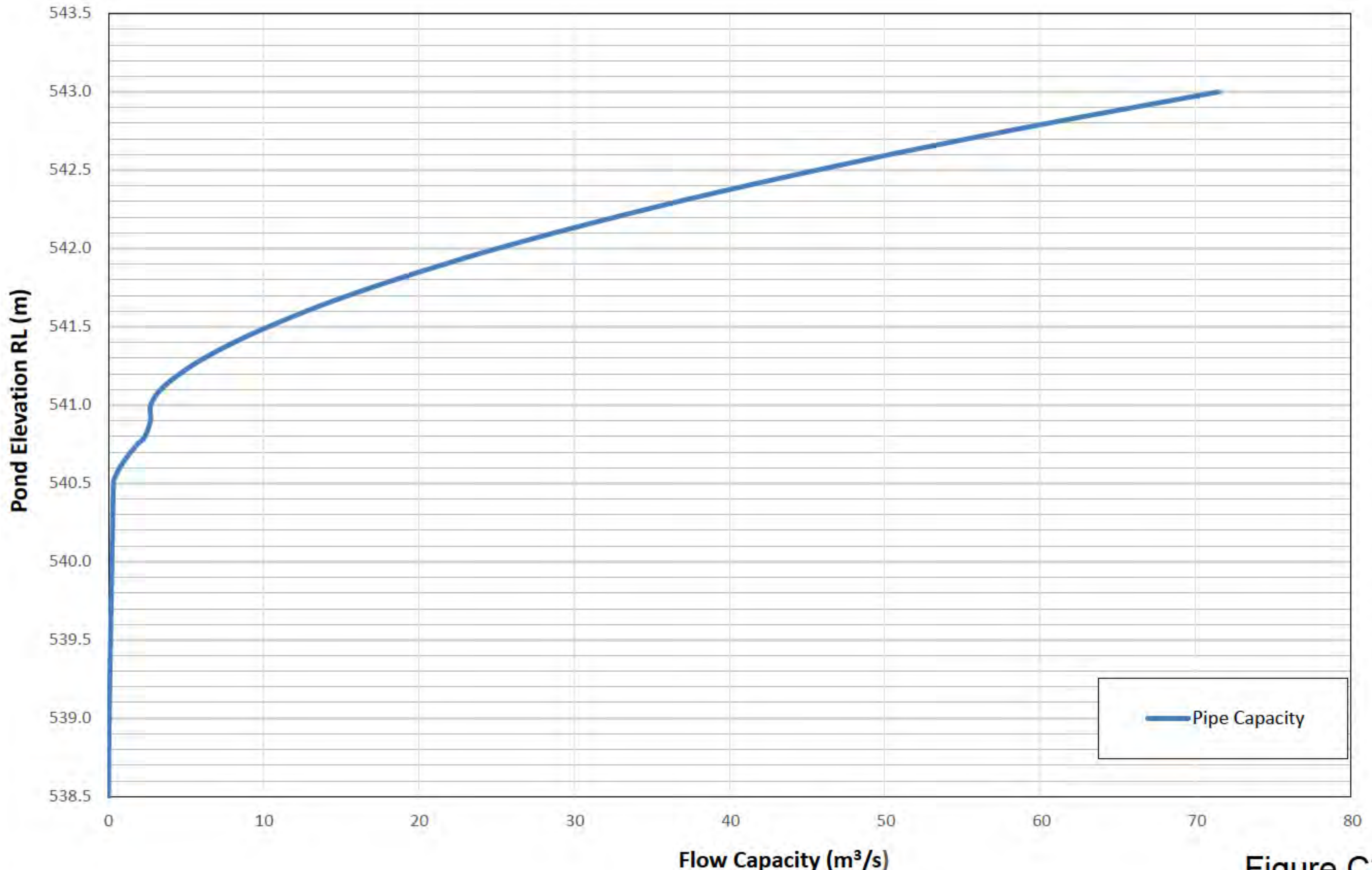

Figure C2

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MATAKANUI GOLD LIMITED
BENDIGO-OPHIR GOLD PROJECT
Silt Pond Outlet Pipe for 1/10 AEP
Stage Discharge Curve of the Pipe 800OD Combined with Auxiliary Spillway

Ref. No: 9702
Date: July 2025
Drawn: PQ

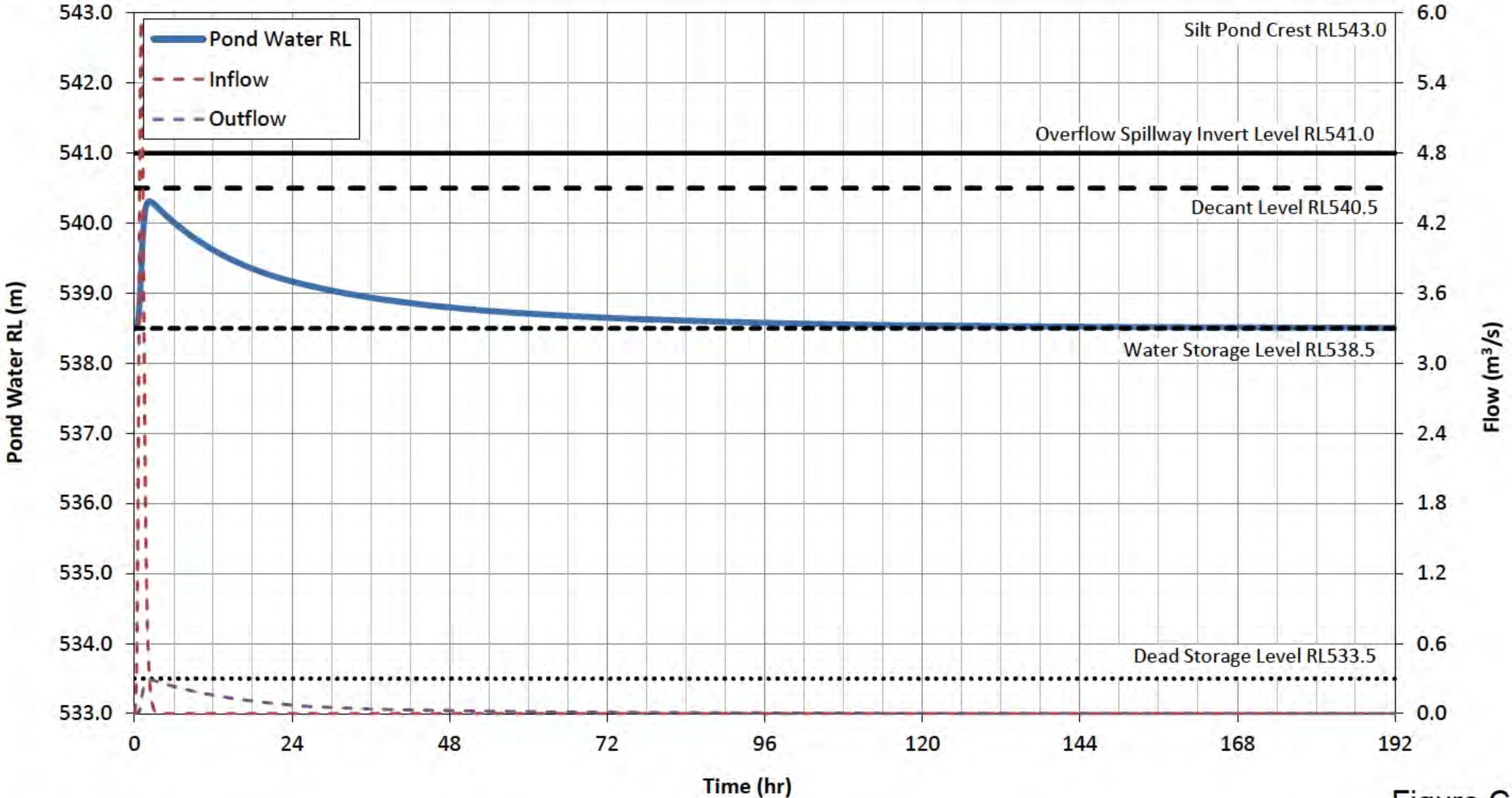

Figure C3

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MATAKANUI GOLD LIMITED
BENDIGO OPHIR GOLD PROJECT
SHEPHERDS SILT POND DESIGN
Hydraulic Routing Curves - 1 hr - 10 Year Design Rainfall Event

Ref. No: 9702
Date: July 2025
Drawn: PQ

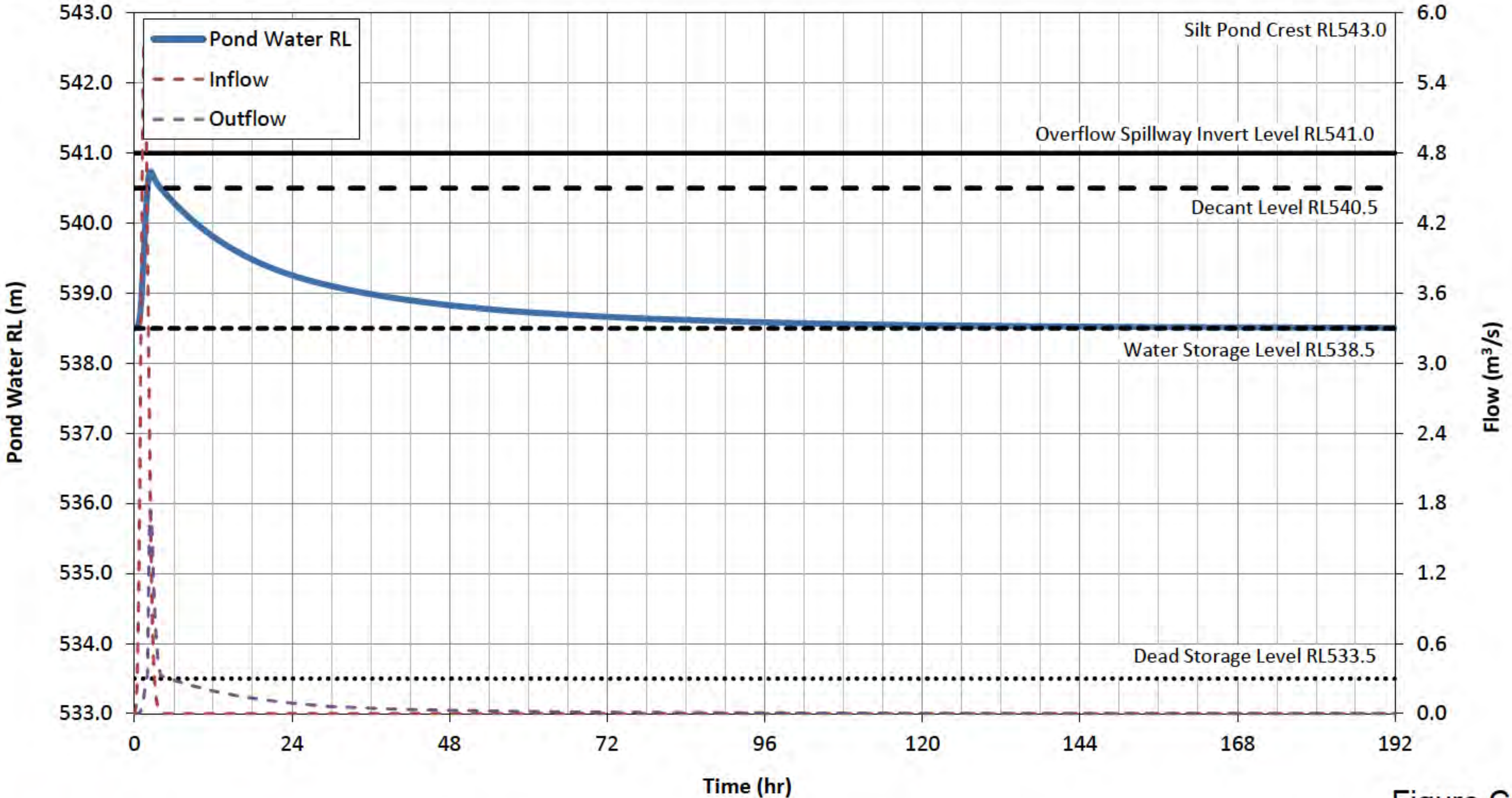

Figure C4

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SHEPHERDS SILT POND DESIGN
Hydraulic Routing Curves - 2 hr - 10 Year Design Rainfall Event

Ref. No: 9702
Date: July 2025
Drawn: PQ

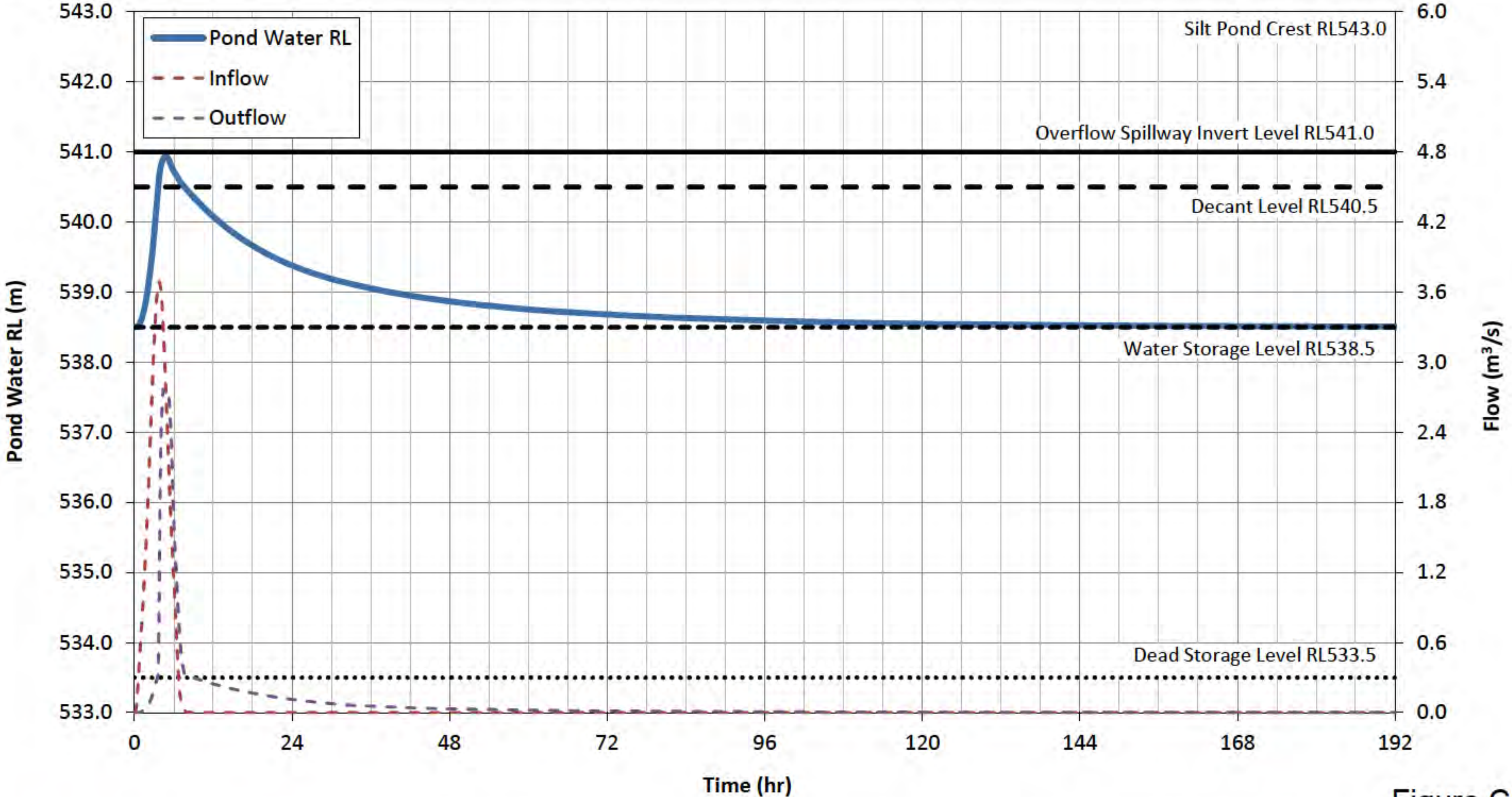

Figure C5

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MATAKANUI GOLD LIMITED
BENDIGO OPHIR GOLD PROJECT
SHEPHERDS SILT POND DESIGN
Hydraulic Routing Curves - 6 hr - 10 Year Design Rainfall Event

Ref. No: 9702
Date: July 2025
Drawn: PQ

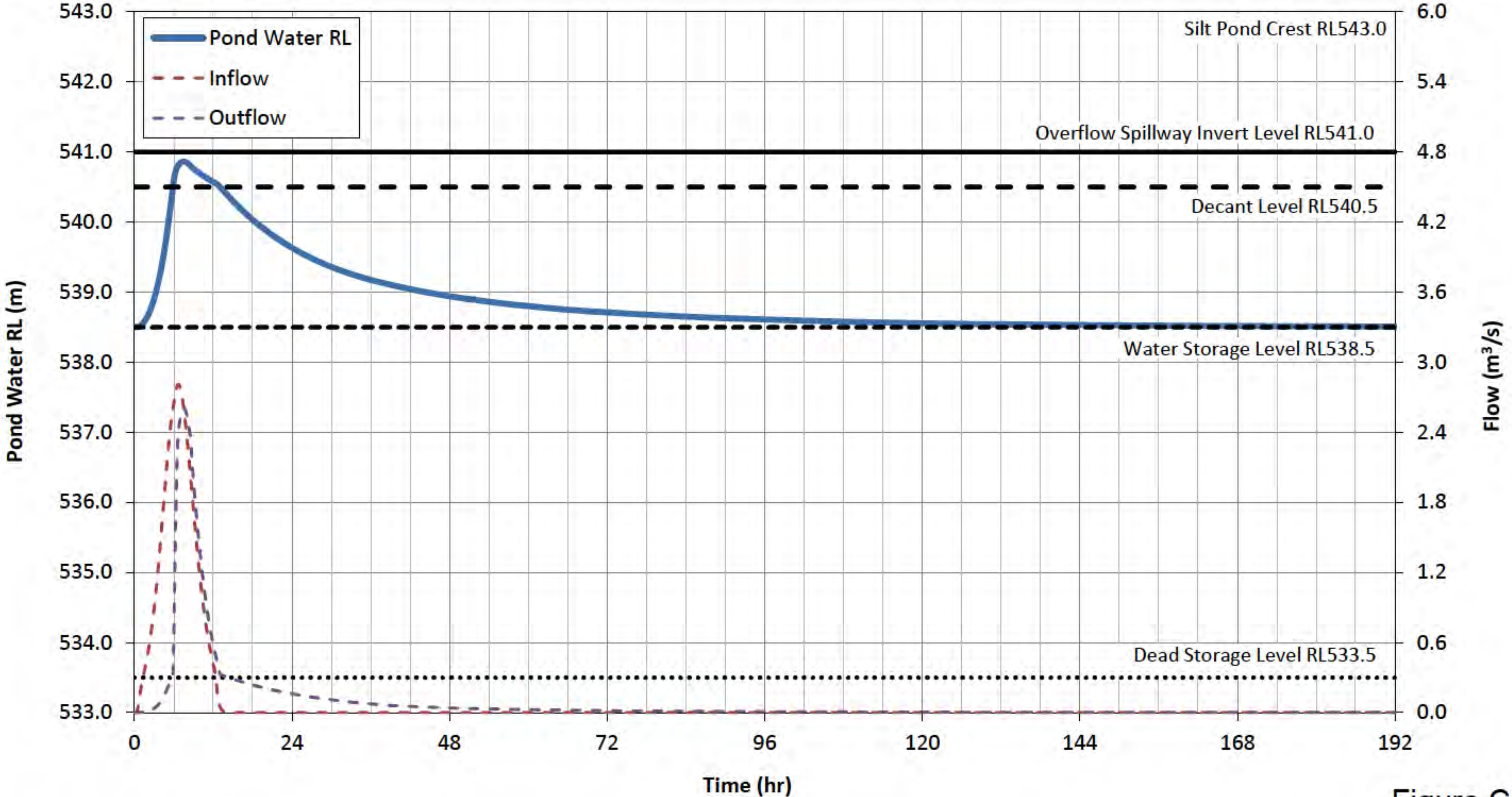

Figure C6

EGL Geotechnical, Earthquake and Dam Engineers Engineering Geology Ltd +64 9 486 2546 info@egl.co.nz Unit 7C, 331 Rosedale Road, Albany, Auckland PO Box 301054, Albany, Auckland 0752 www.egl.co.nz	**MATAKANUI GOLD LIMITED** **BENDIGO OPHIR GOLD PROJECT** SHEPHERDS SILT POND DESIGN *Hydraulic Routing Curves - 12 hr - 10 Year Design Rainfall Event*	Ref. No: 9702 Date: July 2025 Drawn: PQ

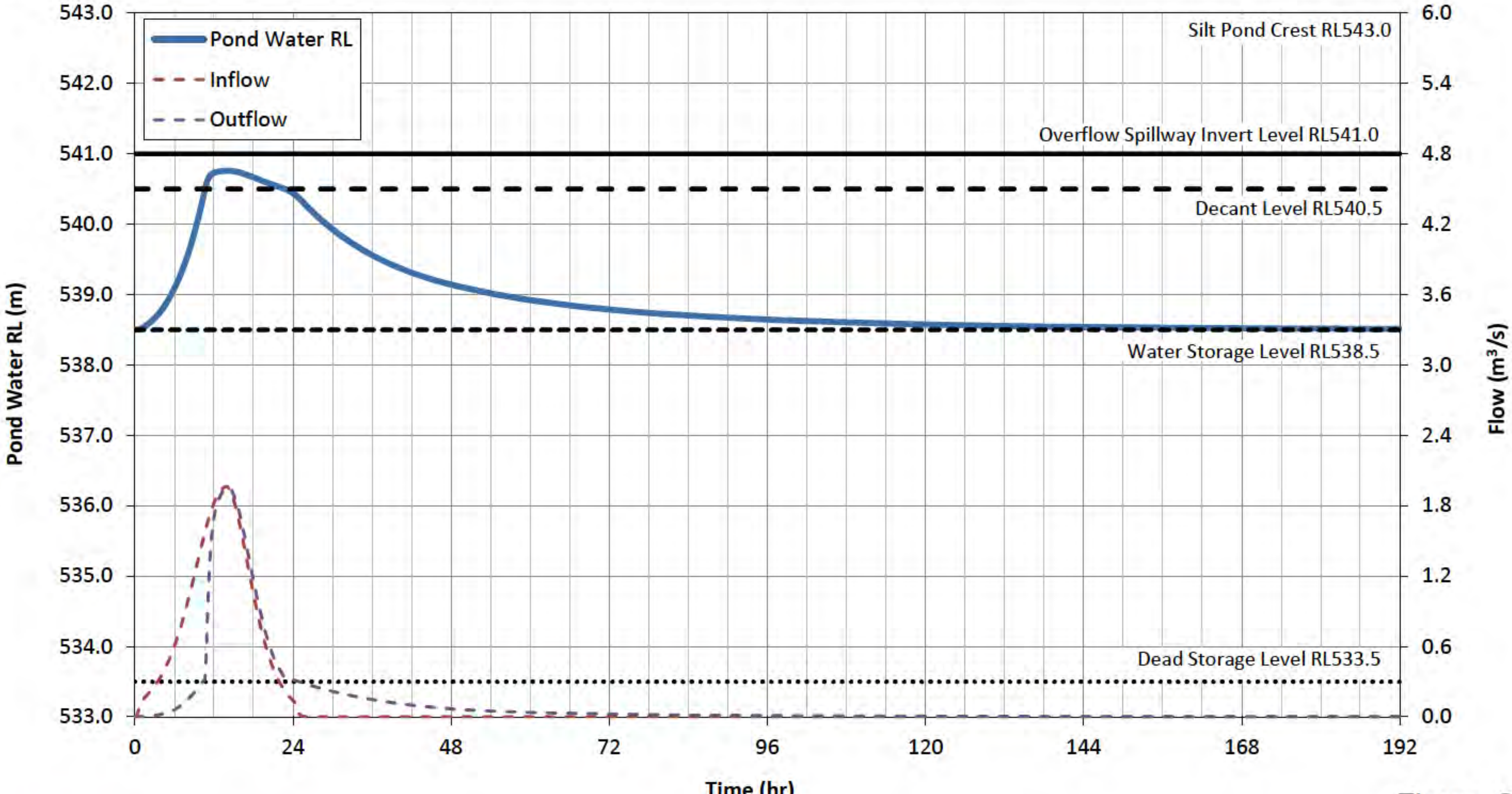

Figure C7

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BENDIGO OPHIR GOLD PROJECT
SHEPHERDS SILT POND DESIGN
Hydraulic Routing Curves - 24 hr - 10 Year Design Rainfall Event

Ref. No: 9702
Date: July 2025
Drawn: PQ

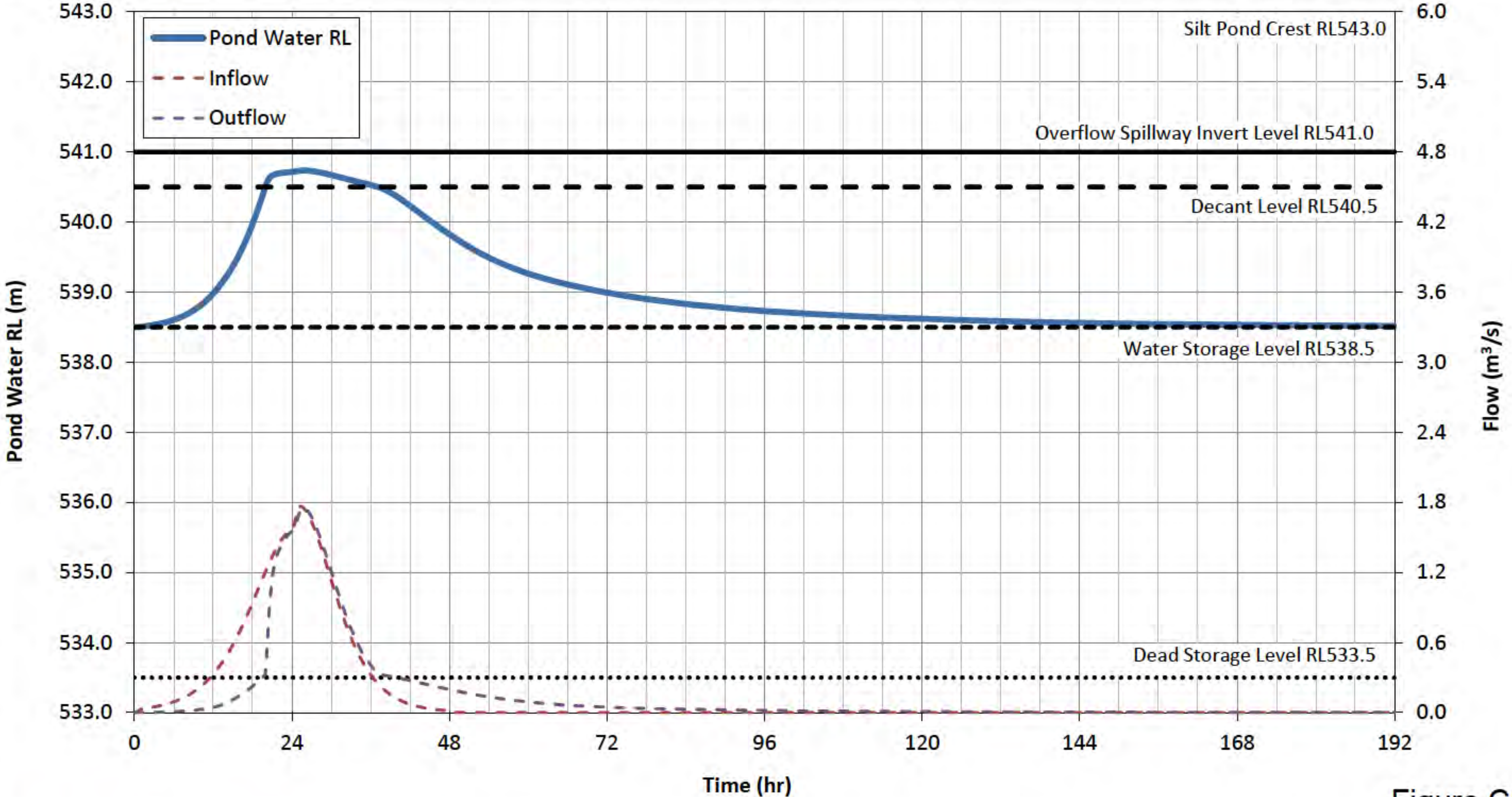

Figure C8

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Hydraulic Routing Curves - 48 hr - 10 Year Design Rainfall Event

Ref. No: 9702
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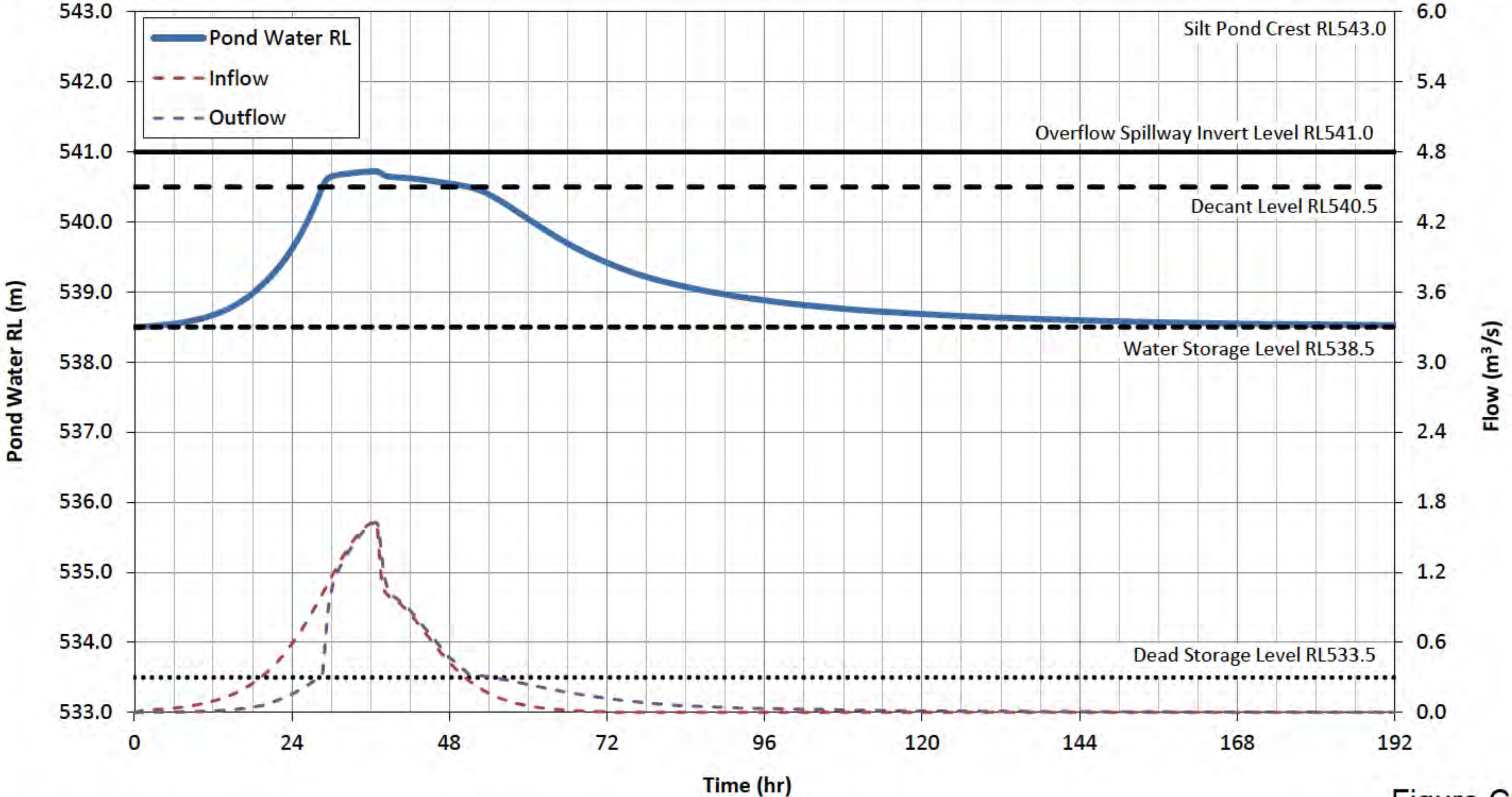

Figure C9

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SHEPHERDS SILT POND DESIGN
Hydraulic Routing Curves - 72 hr - 10 Year Design Rainfall Event

Ref. No: 9702
Date: July 2025
Drawn: PQ

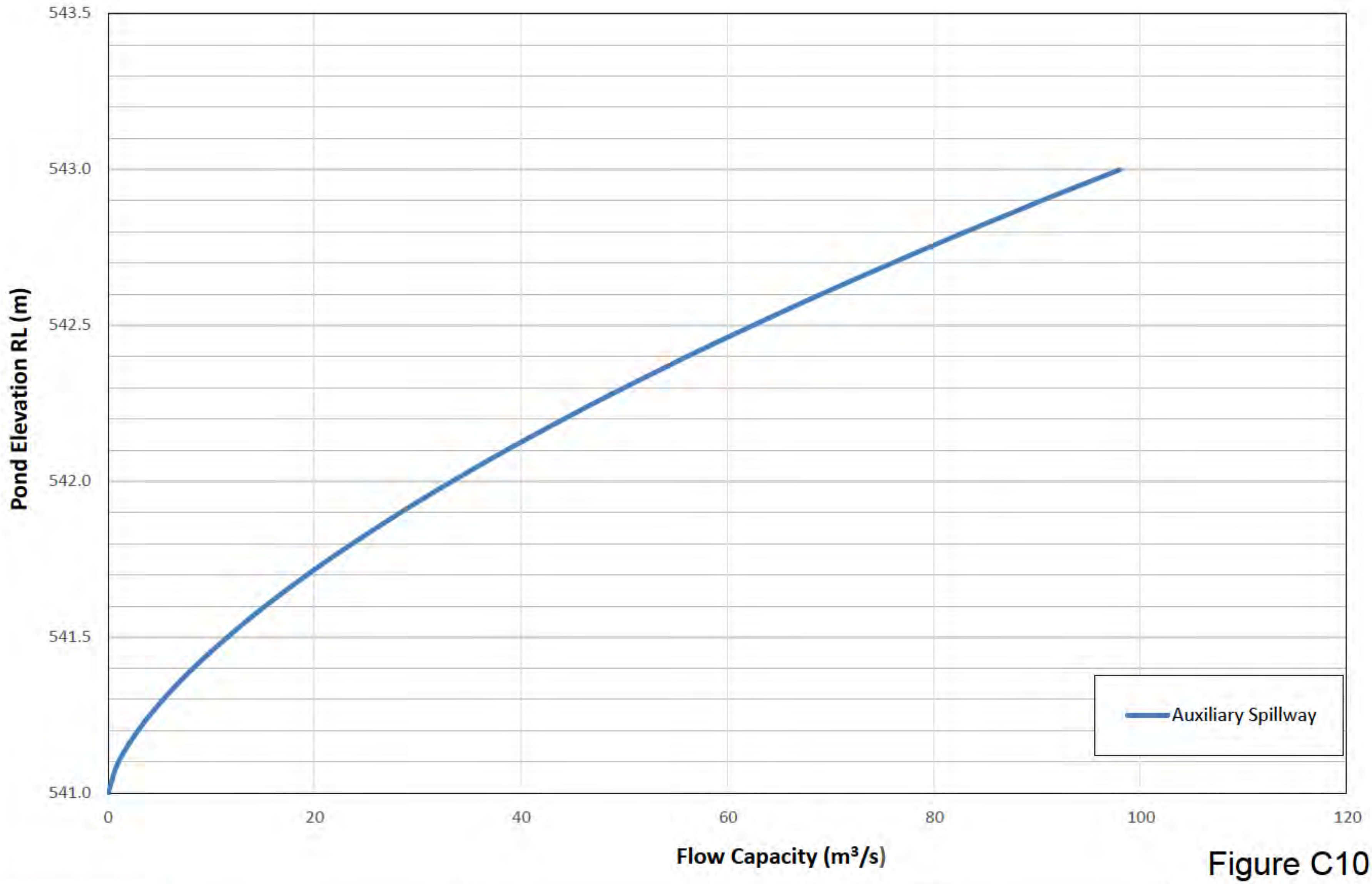

Figure C10

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BENDIGO-OPHIR GOLD PROJECT
Silt Pond Auxilliary Spillway
Stage Discharge Curve Auxilliary Spillway for 1/1,000 AEP

Ref. No: 9702
Date: July 2025
Drawn: PQ

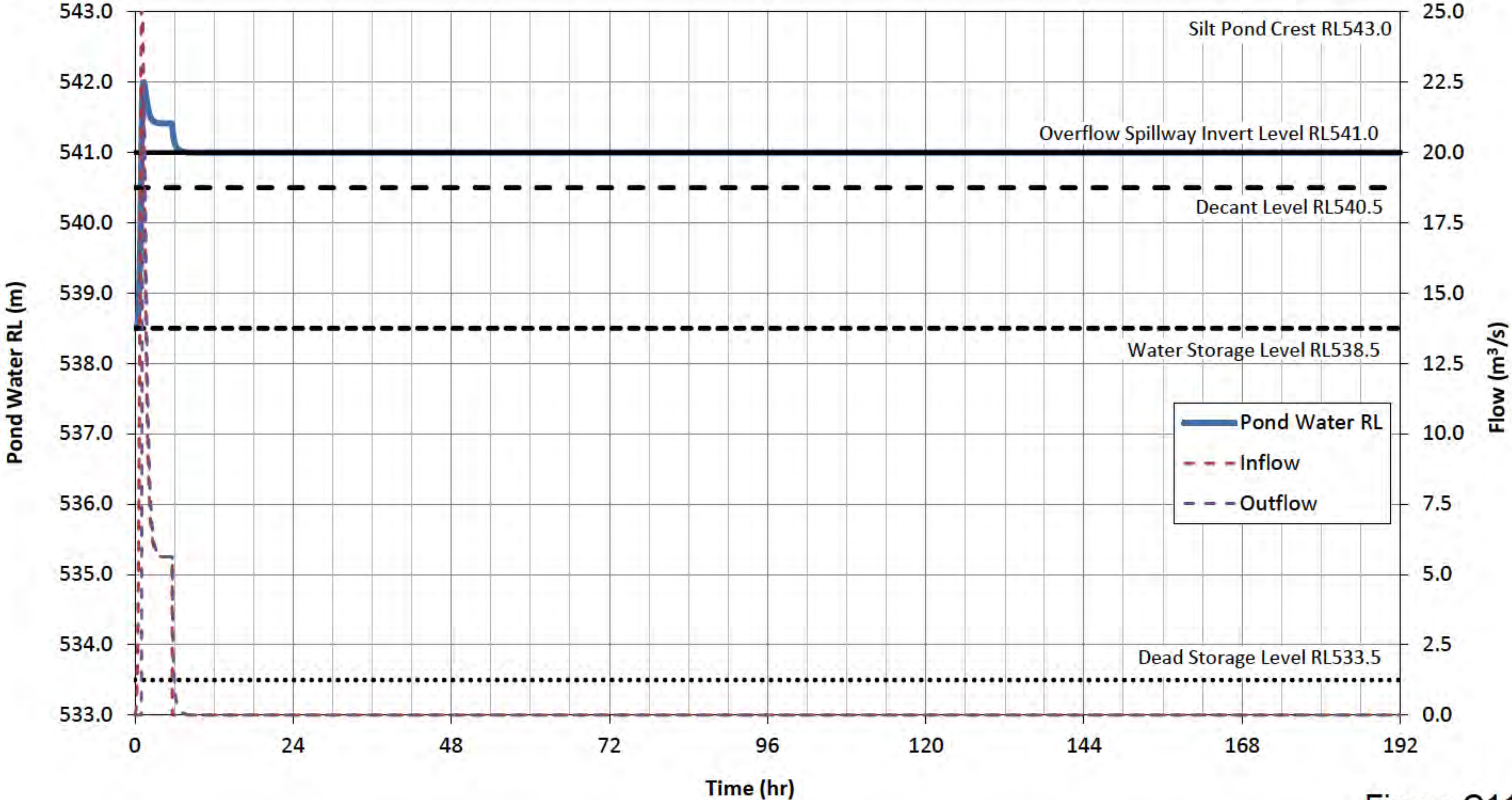

Figure C11

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Hydraulic Routing Curves - 1 hr - 1,000 Year Design Rainfall Event

Ref. No: 9702
Date: July 2025
Drawn: PQ

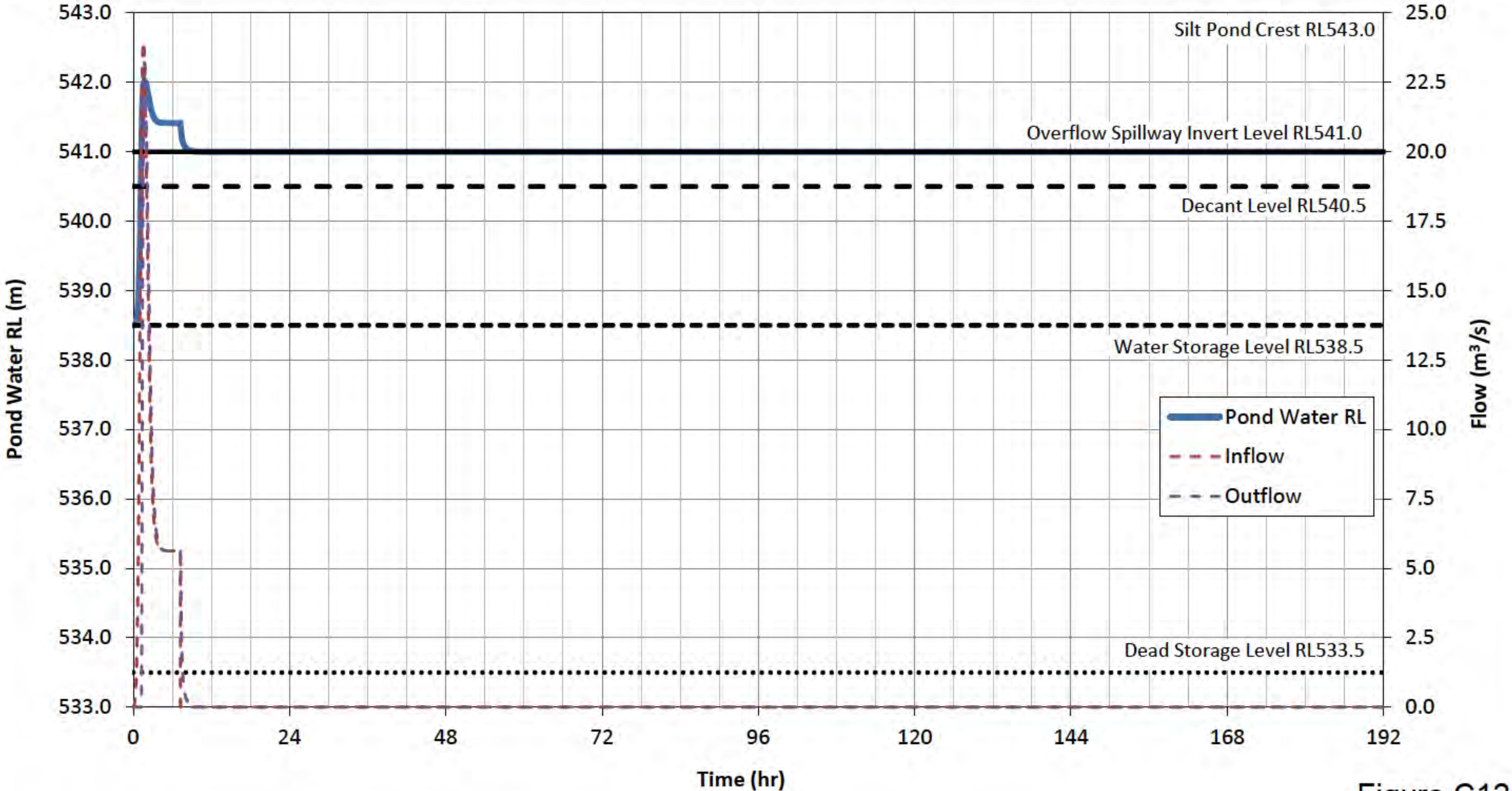

Figure C12

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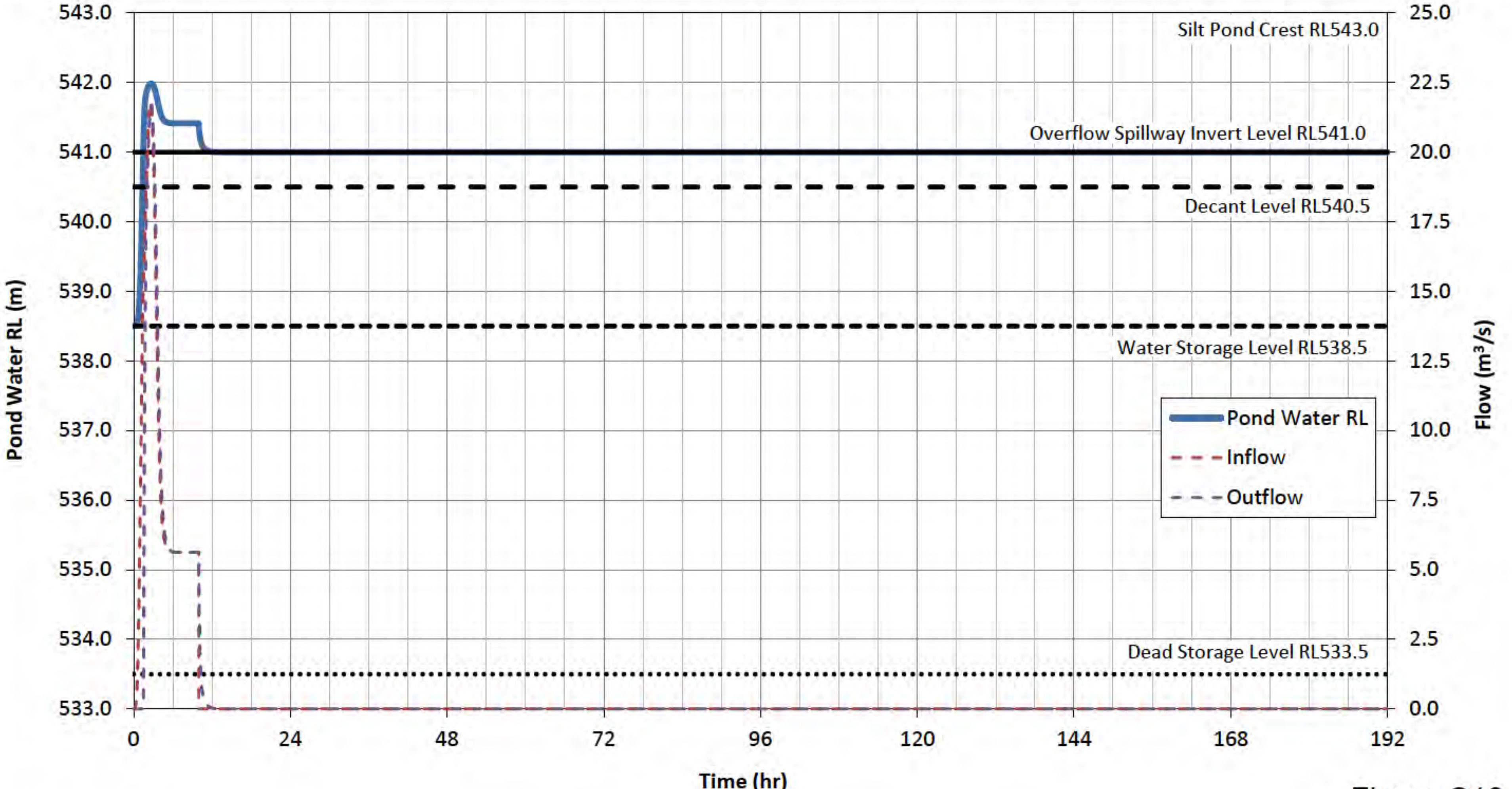

Figure C13

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SHEPHERDS SILT POND DESIGN
Hydraulic Routing Curves - 6 hr - 1,000 Year Design Rainfall Event

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Date: July 2025
Drawn: PQ

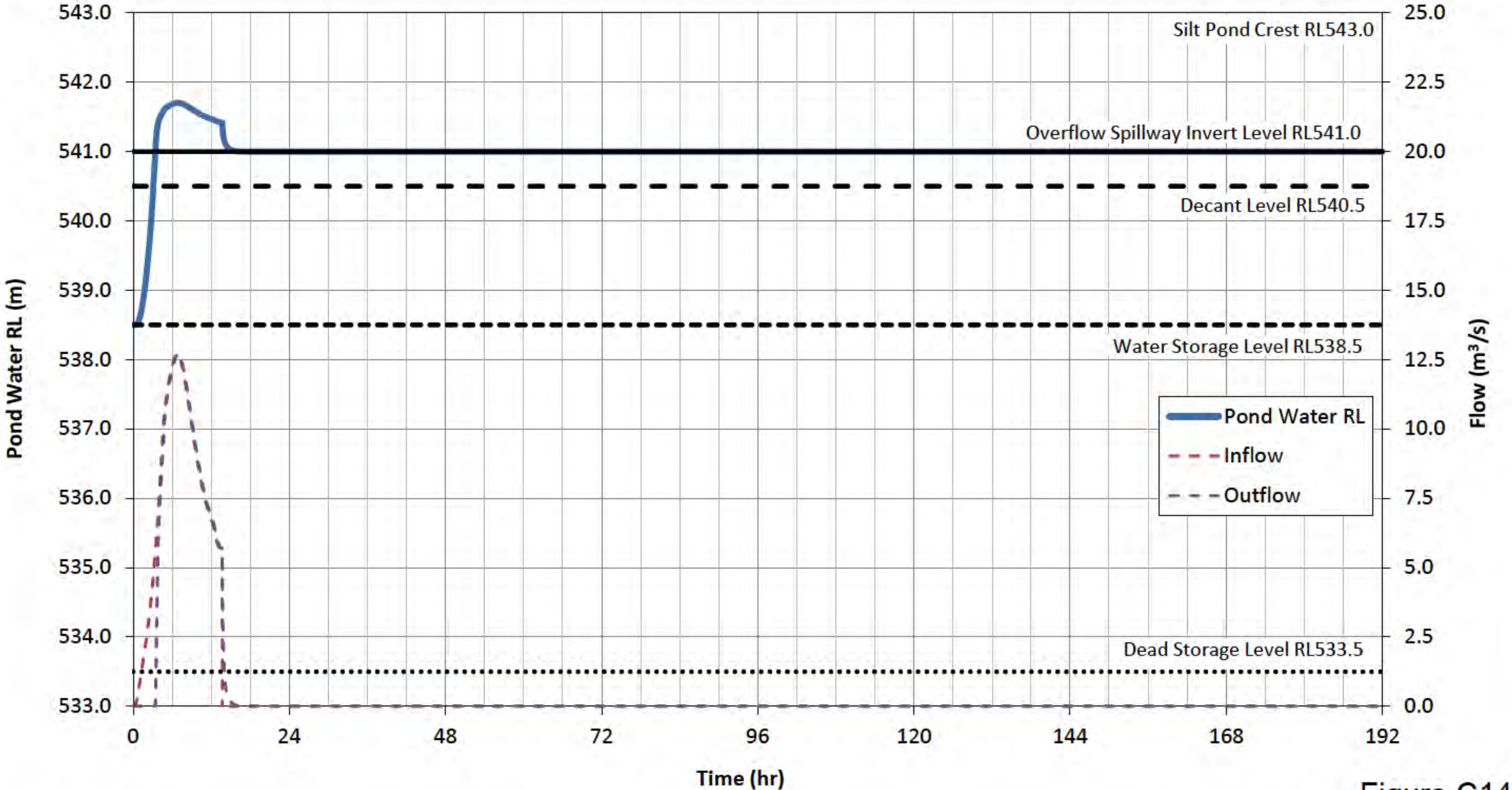

Figure C14

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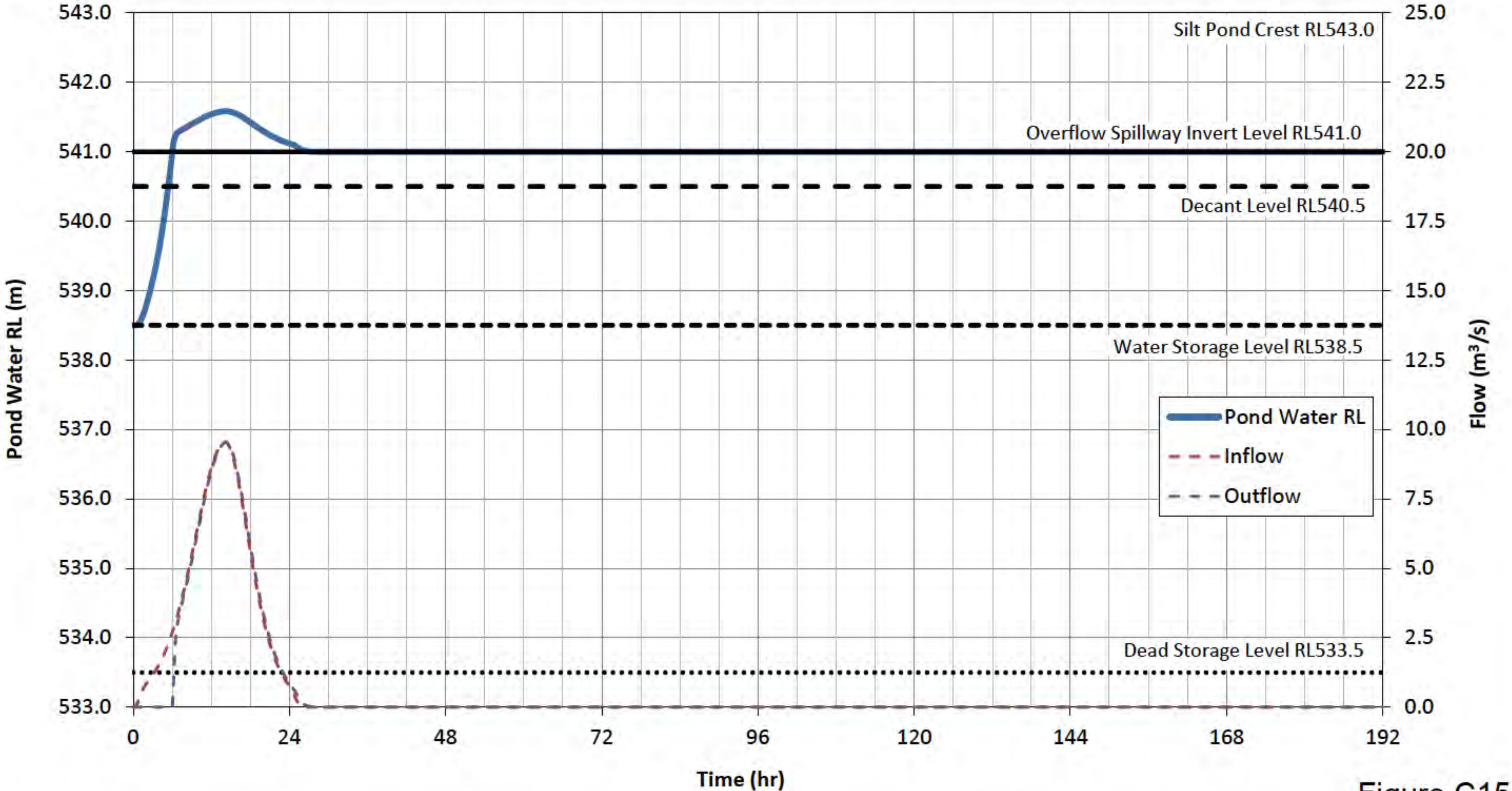

Figure C15

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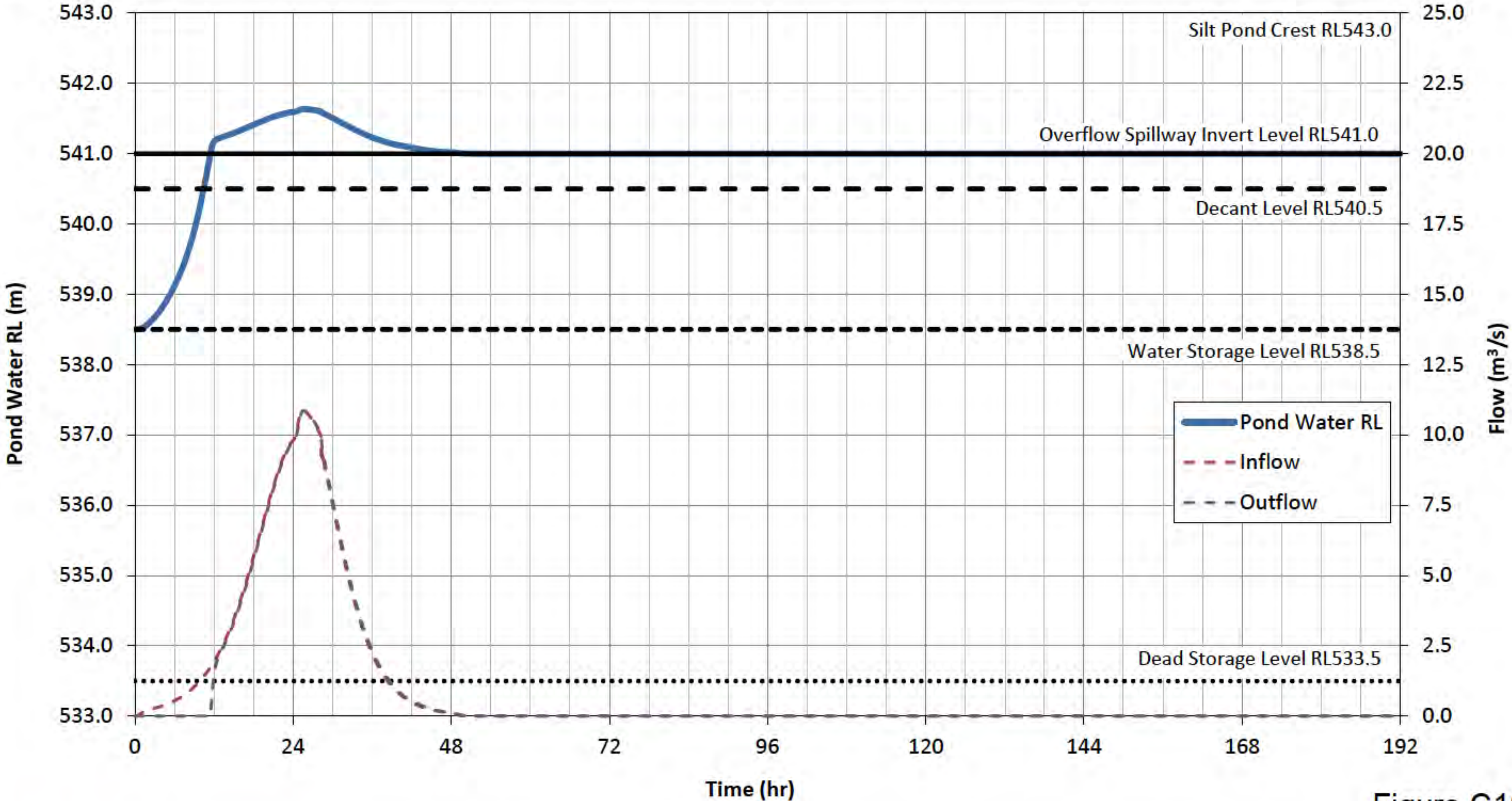

Figure C16

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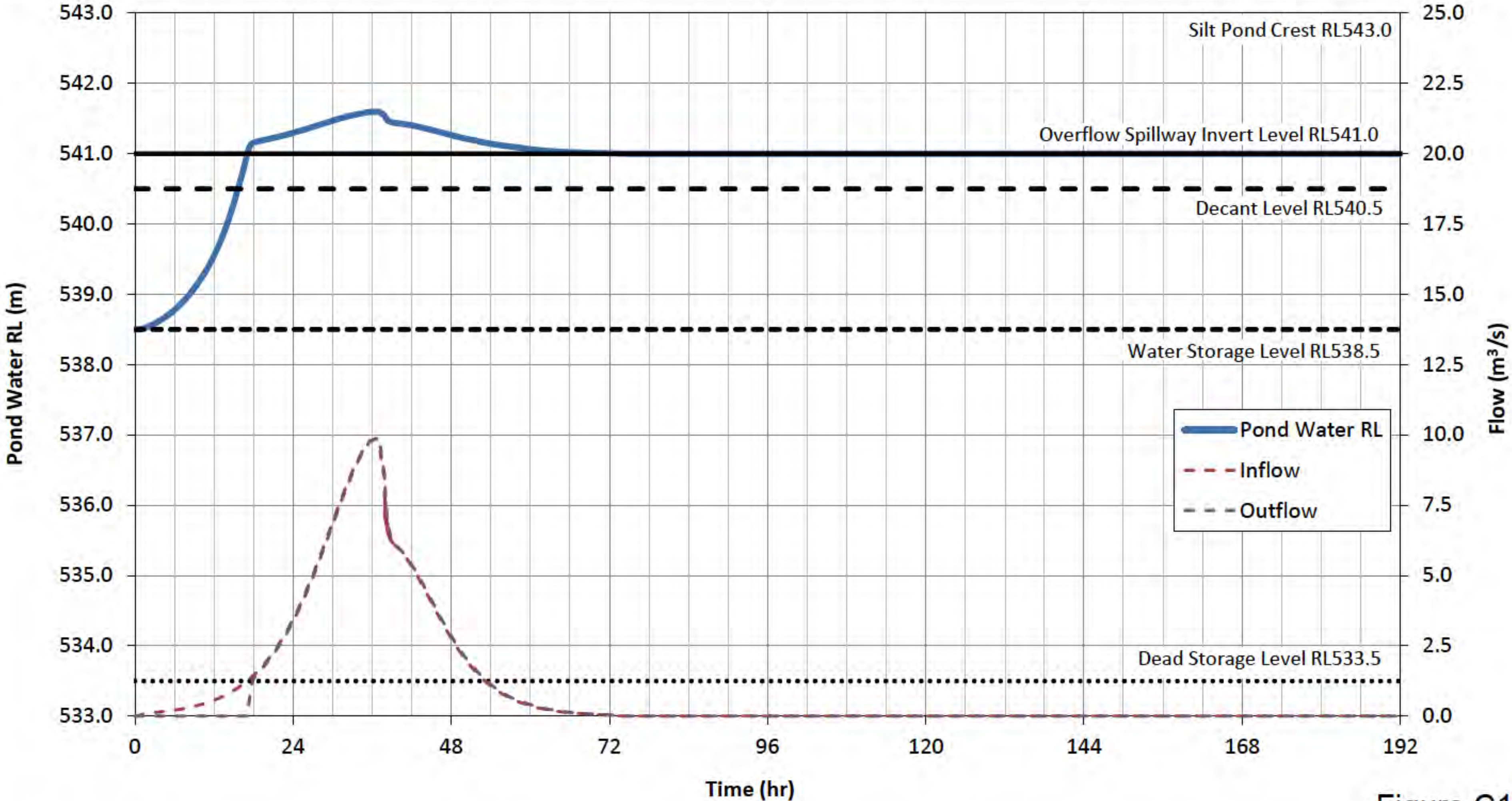

Figure C17

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