REPORT

Engineering Design Report: Bauxite Residue Disposal Area (BRDA) Raise Development

Aughinish Alumina Limited

Submitted to:

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Executive Summary

This report presents a review and an interpretation of site investigation and laboratory testing for BRDA foundation soils and the deposited bauxite residue tailings and the engineering design of the following BRDA Raise Development components:

- **Raise of the BRDA:** The current BRDA is permitted to be constructed to Stage 10, which has a perimeter crest elevation of 24 mOD and a maximum dome crown elevation of 32 mOD. It is proposed that the permitted height of the overall BRDA be increased to accommodate the further storage of bauxite residue within the footprint (circa 8 million m^3 of storage) and to extend the life of facility by approximately 9 years. The proposed increased in height is 12m which will comprise 6 x 2m high stages raises (Stages 11 to 16), to provide a new perimeter crest elevation of 36 mOD and a maximum dome crown elevation of 44 mOD.
- **Raise of the SCDC:** The current SCDC is located within the BRDA, comprises a footprint of approx. 1 ha. and is constructed to a perimeter crest elevation of 29 mOD. It is proposed the SCDC be vertically extended to accommodate further storage of salt cake within its current footprint (circa 22,500 m^3 of storage) and to provide the equivalent of 3 years storage capacity. The proposed increase in height is 2.25m which will comprise a single raise to provide a new perimeter crest elevation of 31.25 mOD and a storage footprint of 1.45 ha.
- **Extension of the Borrow Pit:** The current permitted Borrow Pit is located to the east of the BRDA and is scheduled to provide circa 374,000 m^3 of rock fill material to construct the BRDA to Stage 10. It is proposed to extend the footprint of the Borrow Pit from circa 4.5 hectares to circa 8.4 hectares (an additional footprint of 3.9 ha.) to provide an additional circa 380,000 m^3 of rock fill material.

The following assessments were undertaken to support the design of the BRDA Raise.

- Geotechnical Analyses
  - Seismic Liquefaction Assessment (foundation soils and bauxite residue);
  - Stability Assessment;
  - Blast Assessment;
  - Consolidation Assessment; and
  - Breach Assessment.
- Seepage and Water Quality at Closure Assessment
- Water Balance and Hydrological Assessment
- BRDA Closure Engineering Design
- BRDA Raise Engineering Design, Operating Philosophy / Requirements
- BRDA Instrumentation, Monitoring and Surveillance
- Borrow Pit Extension Engineering Design
- Salt Cake Disposal Cell Engineering Design

Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 Aughinish Alumina Limited

Aughinish Alumina Limited (AAL) is wholly owned by United Company RUSAL (UC Rusal) and operates the alumina refinery situated on Aughinish Island on the south side of the Shannon Estuary, in Co. Limerick, in accordance with the Conditions of the Industrial Emissions Licence (IEL) P0035-07 issued by the Environmental Protection Agency (EPA).

The AAL plant is the largest alumina extraction plant in Europe and represents circa 33% of the total alumina production in Western Europe. The importation of bauxite ore, primarily from West Africa, and the exportation of alumina (aluminium oxide) is undertaken by ship, via the dedicated AAL jetty located in the Shannon Estuary. The alumina extracted is exported to smelters in other countries for processing into aluminium. The tailings (bauxite residue) are stored on site in a designated facility termed the Bauxite Reside Disposal Area (BRDA).

The plant and ancillary structures were constructed between 1978 and 1983. Plant production has been continually increased since the commissioning of the plant in 1983 up to its maximum production of approximately 1.95 million tonnes of alumina per annum.

1.2 Objective / Scope of Report

Golder Associates Ireland Limited (Golder) was appointed by AAL to undertake the engineering design of the BRDA Raise Development comprising a raise of the BRDA from Stage 10 to Stage 16, and the engineering design of ancillary works including an extension of the existing Salt Cake Disposal Cell (SCDC), which is located within the BRDA footprint and the extension of a permitted Borrow Pit to provide construction materials for the development.

Golder has served as the Design Engineer for the BRDA since 2003 and is the AAL appointed Engineer of Record (EoR) for the BRDA, following the AAL adoption of the Canadian Dam Association (CDA) Dam Safety Guidelines (CDA 2013, 2014) for the BRDA in 2018.

Note: *The appointment of an EoR and the scope of the role and the responsibilities for both the EoR and the Client are industry recognised best practice recommended by the both the Dam Safety Guidelines published by the CDA (CDA 2013, 2014) and the recent Global Industry Standard on Tailings Management (GISTM 2020).*

This report presents a review and an interpretation of site investigation and laboratory testing for BRDA foundation soils and the deposited bauxite residue tailings and the engineering design of the following BRDA Raise Development components:

- **Raise of the BRDA:** The current BRDA is permitted to be constructed to Stage 10, which has a perimeter crest elevation of 24 mOD and a maximum dome crown elevation of 32 mOD. It is proposed that the permitted height of the overall BRDA be increased to accommodate the further storage of bauxite residue within the footprint (circa 8 million m^3 of storage) and to extend the life of facility by approximately 9 years. The proposed increased in height is 12m which will comprise 6 x 2m high stages raises (Stages 11 to 16), to provide a new perimeter crest elevation of 36 mOD and a maximum dome crown elevation of 44 mOD.
- **Raise of the SCDC:** The current SCDC is located within the BRDA, comprises a footprint of approx. 1 ha. and is constructed to a perimeter crest elevation of 29 mOD. It is proposed the SCDC be vertically extended to accommodate further storage of salt cake within its current footprint (circa 22,500 m^3 of storage) and to provide the equivalent of 3 years storage capacity. The proposed increase in height is 2.25m which will comprise a single raise to provide a new perimeter crest elevation of 31.25 mOD and a storage footprint of 1.45 ha.

- **Extension of the Borrow Pit:** The current permitted Borrow Pit is located to the east of the BRDA and is scheduled to provide circa 374,000 m^3 of rock fill material to construct the BRDA to Stage 10. It is proposed to extend the footprint of the Borrow Pit from circa 4.5 hectares to circa 8.4 hectares (an additional footprint of 3.9 ha.) to provide an additional circa 380,000 m^3 of rock fill material.

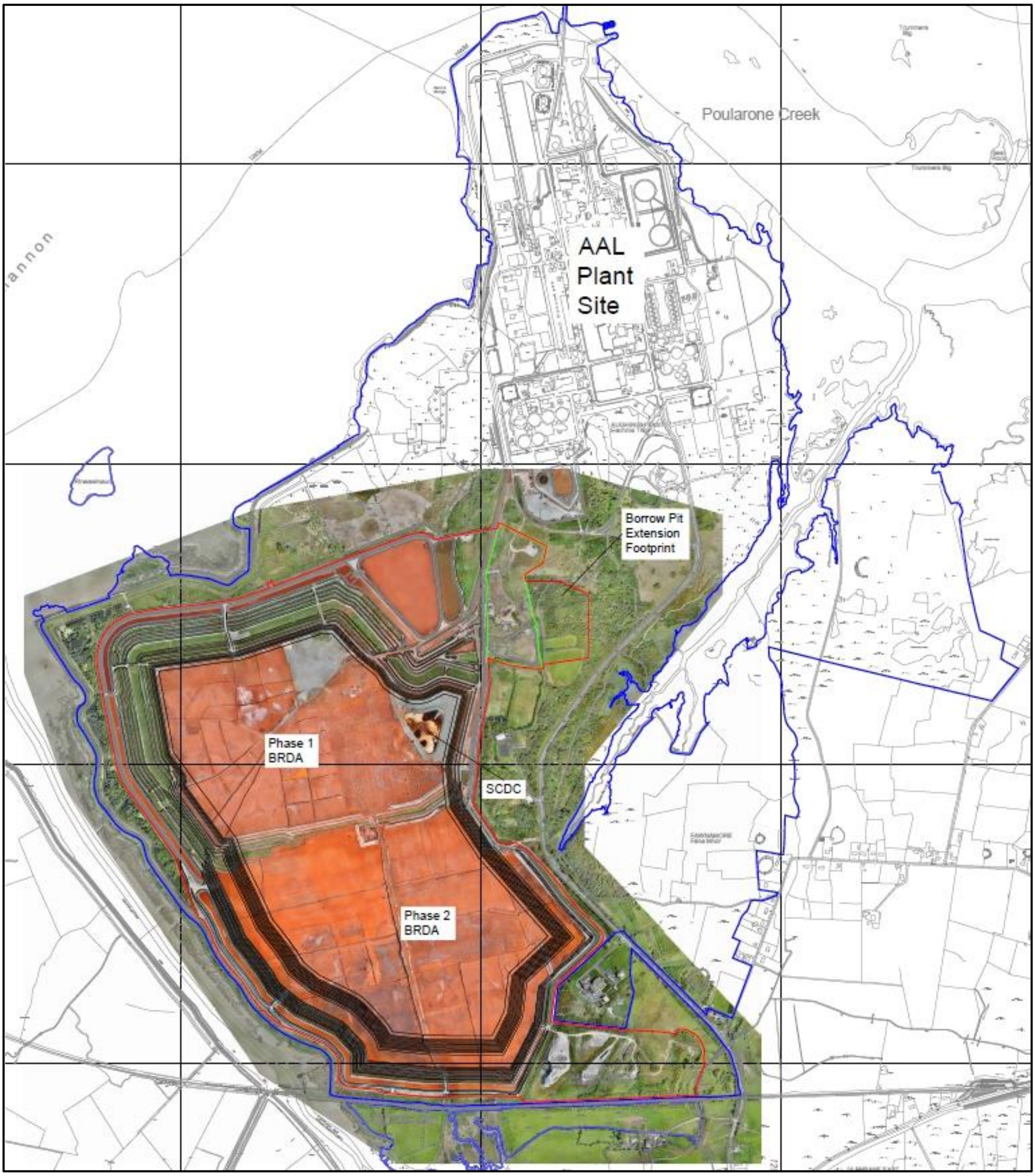

Figure 1: Site Location Map - Blue Line is the AAL Ownership Boundary, Red Line is the Application Boundary and Green Line is the permitted Borrow Pit Footprint

1.3 Project Background

Bauxite residue from the alumina production process is deposited within the BRDA which is located to the south-west of the plant. The BRDA was constructed in three phases and comprises two distinct disposal areas which are currently merging:

- The Phase 1 BRDA is formed from two facilities; the original Phase 1 BRDA constructed in the early 1980s, covering an area of 72 ha, and the Phase 1 BRDA Extension, constructed in the mid-to-late 1990s, covering an area of 32 ha. The initial design for the Phase 1 BRDA was to provide a disposal area to the year 2009 based on the facility constructed to Stage 7 (elevation 18 mOD), which equates to a central dome crown elevation of 27.5 mOD or 26m above the original ground level.

  Permission was granted in 2007 (Limerick County Council (LCC) Reg. Ref. 05/1836; An Bord Pleanála (ABP) Ref. PL13.217976) to raise both the Phase 1 BRDA and the Phase 1 BRDA Extension to Stage 10 as part of the overall BRDA development, which included the Phase 2 BRDA.

- The Phase 2 BRDA is a southern extension of the Phase 1 BRDA that was permitted in 2007 (Limerick County Council (LCC) Reg. Ref. 05/1836; An Bord Pleanála (ABP) Ref. PL13.217976) to be constructed to Stage 10, with a maximum perimeter elevation of 24 mOD and a maximum dome crown elevation of 32 mOD.

  The Phase 2 BRDA merges with the southern extent of the Phase 1 BRDA. The Phase 2 BRDA covers an area of approximately 80 ha. and was commissioned in 2011.

- The permitted BRDA's have capacity to provide a disposal area for bauxite residue until circa 2030, for the current rate of production. The current elevation of the BRDA varies, from 21 mOD to 31mOD in Phase 1 to 10mOD to 21mOD in Phase 2 (see Drawing 01 for aerial survey contours from April 2021).

  The bulk of the annual bauxite residue produced (≈ 83%) has been deposited in the Phase 2 BRDA for 2019 and 2020, with 100% being deposited in the Phase 2 BRDA for 2021, to date.

As part of the overall permitted Phase 2 BRDA development, a Salt Cake Disposal Cell (SCDC) was permitted to be constructed within the Phase 1 BRDA Extension area. Organic impurities enter the refinery caustic soda liquor stream when the bauxite is dissolved. These impurities must be removed from the process to ensure optimum product quality and efficiency. The impurities are precipitated out as a crystalline salt material of various sodium compounds. Salt Cake is a hazardous waste which is disposed of in the SCDC, which is a dedicated and compositely lined cell within the BRDA.

To the east of the BRDA, permission was granted in 2018 for the development of a Borrow Pit with an extraction area of c. 4.5 hectares to produce 374,000 m³ of rock, which will be used in the ongoing development of the BRDA to Stage 10 (LCC Reg. Ref. 17/714; ABP Ref. PL91.301011). The operation of the Borrow Pit has not yet commenced but is scheduled for Q2 of 2022.

2.0 BASIS OF DESIGN

A basis of design and design criteria / parameters document was prepared for the engineering design and is included in Appendix A. A summary of the key principles is provided in the Sections below.

2.1 BRDA Characteristics

Table 1: AAL BRDA Characteristics

Design Criteria / Parameters		Value	Source / Comment
General			
Level of Study		Engineering Design for Planning and Approval Processes	Request for Proposal / Scope of Work
Focus of Study		Residue Surface Disposal. Upstream raise of current facility from Stage 11 to Stage 16.	Request for Proposal / Scope of Work
Mineral(s)		Alumina (aluminum oxide) produced from bauxite material and producing waste bauxite residue for disposal in the BRDA.	Request for Proposal / Scope of Work
Operations and BRDA Configuration			
BRDA Information	Storage Geometry	Storage Footprint at base ≈ 168.5 ha (94.5 ha for Phase 1 and 74 ha for Phase 2) Storage Footprint at Stage 11 ≈ 96 ha Base Elevation varies, typically between 0 to 2 mOD. Base of raise (Stage 11 to 16) is at crest of Stage 10 = 24 mOD Stage 16 = 36 mOD, Dome (top) = 44 mOD	Inner crest of basin Inner crest of Stage 11 Residue is deposited in a dome shape that results in elevations above 24 mOD Dome Crown is 8m above perimeter elevation
	Life of BRDA	Total storage of circa 53.1 million tonnes. Circa 9.12 million m^3 of void remaining from April 2021, representing 14.9 million tonnes or 9.6 years of storage at the current rate of production.	AAL 2030
	Life of BRDA with proposed BRDA Raise Development	Total storage of circa 66.2 million tonnes. Circa 17.16 million m^3 of void remaining from April 2021, representing 28.0 million tonnes or 18.5 years of storage at the current rate of production.	AAL / Golder Additional 9 years 2039
	Permitted Alumina Production (annual)	≈ 1.95 million tonnes/year	AAL
	Residue Production (annual)	≈ 1.57 million tonnes/year Residues by weight (AER 2020) are: • 90.6 % bauxite residue • 6.9 % process sand • 1.0 % salt cake • 1.5 % scales and sludges	AAL
	Rate of Rise (annual)	0.86m to 1.00m / year for Phase 1 BRDA and 1.25m to 1.75m / year for Phase 2 BRDA. Rate of rise dependent on zonal deposition prioritization	AAL / Golder Aerial survey data for Phase 1 and 2 BRDA from 2005 to April 2021
	Total Stored Residue	≈ 36.0 million tonnes, 1983 to December 2020. Phase 1 BRDA commissioned in 1983 and Phase 2 BRDA commissioned in 2011.	AAL

Design Criteria / Parameters		Value	Source / Comment
	Deposition Method	Hydraulic deposition discharge of bauxite residue paste from 'Mud Points' located centrally within the BRDA. Bauxite residue paste migrates by gravity to perimeter stage raises and/or cell bunds at between 2% and 4% grade. Layered deposition to dewater paste and facilitate mud farming. Trucking for all other residues (process sand, salt cake and scales). Designated tipping area for salt cake is the SCDC. Scales are deposited in the interior of the BRDA. Process sand utilized for internal haul roads and bunds.	AAL / Golder Positive displacement high pressure pumping of paste at approx. 75% moisture content (≈ 58% solids) via distribution network to controlled cells within the BRDA. Mud farming reduces pH < 11.5, reduces moisture content, increases density and increases strength.
BRDA Construction	Embankment raise methodology	Upstream in 2m high lifts of rock fill (stage raises) with a 4m wide crest and side-slopes of 1.5(H):1(V). Next stage raise is offset upstream by a bench. Bench widths are 4m typically. Wider bench at Stage 5 (≈ 28m) and at Stage 10 (12.5m)	AAL / Golder Stage raises constructed on farmed and prepared bauxite residue footprint in accordance with AAL SWM for staged construction (1m lifts).
	Downstream Slope	Overall slope to Stage 10 is 6.3(H):1(V). Overall slope to Stage 16 will be 7.2(H):1(V)	AAL / Golder
	Dome gradient	Maximum of 4% at closure or 1(V):25(H)	AAL / Golder
SCDC Construction	Embankment raise methodology	Downstream and centre-line in lifts constructed of rock fill material. Proposed raise will be 4th vertical extension of the SCDC and is a 2.25m high raise (to 31.25 mOD). Cell walls are buttressed by deposited bauxite residue / process sand as BRDA increases in elevation.	Golder Triangular shaped cell with original cell base at circa 19 mOD.
	Side-Slopes and Crest	**Downstream Raises:** North and East dam walls. 2.0(H):1(V) downstream slope and 2.5(H):1(V) upstream slope. 8m width crest **Centre-Line Raise:** West dam wall (tipping wall) ≈ vertical slope with 1.5m offset bench on upstream slope. 22m width crest (tipping wall)	Golder Downstream raises constructed of rock fill. Centre-line raise constructed of gabion terramesh basket retaining walls with a rock fill core.
Tailings Characteristics			
Bauxite Residue Physical Characteristics	Gradation	90% by weight < 40 microns, D_{50} between 2 and 5 microns (0.002 to 0.005 mm)	Golder, lab testing
	Classification	SILT of intermediate plasticity	Golder, CPT & lab testing
	<u>Characteristic Values</u> Unfarmed Farmed	<u>Dry / Bulk Density and Moisture Content</u> 1.58 Mg / m^3 / 2.19 Mg / m^3 (38%) 1.63 Mg / m^3 / 2.19 Mg / m^3 (34%)	Golder In-situ and lab testing programmes since 2003
	<u>Characteristic Values</u> Unfarmed Farmed Farmed and Amended	<u>Hydraulic Conductivity</u> k_v = 5.0 x 10^{-9} m/s (k_h ≈ 10 times greater) k_v = 1.9 x 10^{-8} m/s (k_h ≈ 10 times greater) k_v = 1.0 x 10^{-6} m/s (k_h ≈ 10 times greater) (addition of compost, sand and gypsum)	Golder In-situ and lab testing programmes since 2003
	N_{kt}	14	Golder

Design Criteria / Parameters		Value	Source / Comment
	Particle Density	3.40	Golder, various lab testing
	Effective Strength (ϕ) Unfarmed Farmed	Effective Friction Angle and Cohesion 32°, c = 0 kPa 32° to 35°, c = 0 kPa	Golder In-situ and lab testing programmes since 2003
	Undrained Strength (s_u/σ'_{v0}) Unfarmed Farmed	Undrained Shear Strength Ratio 0.20 to 0.30 (simple shear) 0.5 to 0.7 (compression) 0.60 (shear and compression)	Golder, In-situ and lab testing programmes since 2003. TBD at each section
	Critical State Parameters Unfarmed Farmed	Γ = 1.35 and λ_{10} = 0.129 Ψ between - 0.05 and 0.05 Ψ between - 0.06 and - 0.05	Golder Porto 2021 and in-situ and lab testing programmes since 2003
	Range of Values Unfarmed Farmed	Consolidation Parameters m_v = 0.3 to 3.0 m^2 / MN and c_v = 3 to 32 m^2/year (90% consolidation) m_v = 0.020 to 0.081 m^2 / MN and c_v = 34 to 100 m^2/year (90% consolidation)	Golder In-situ and lab testing programmes since 2003
Lining System			
Side-Slope of Basin and Basal Lining Systems	Equivalent to min. 0.5m depth of material with a hydraulic conductivity of 1 x 10^{-9} m/s (MWEI BREF 2018)	Natural soils constructed soils and geosynthetic layers. Phase 1 BRDA is founded on estuarine deposit. Phase 1 BRDA Extension and Phase 2 BRDA are composite lined: HDPE Geomembrane overlying GCL or min. 1m depth of screened till.	AAL / Golder
Foundation Soils			
Estuarine Deposit	Gradation	Generally, two layers of estuarine soils are present: Clay content of 0% to 34%, Silt content of 44% to 89% and sand content of 4% to 40%.	Golder, Various lab testing
	Classification	Generally, two layers of estuarine soils are present: SILT of intermediate plasticity (PI ≈ 9) and a CLAY of high plasticity (PI ≈ 17)	Golder, CPT and lab testing
	Characteristic Values Sandy Silt Silty Clay	Dry / Bulk Density and Moisture Content 1.63 Mg / m^3 / 1.94 Mg/m^3 (30%) 1.31 Mg / m^3 / 1.82 Mg/m^3 (38%)	Golder In-situ and lab testing programmes since 2003
	Characteristic Values Sandy Silt Silty Clay	Hydraulic Conductivity and Void Ratio 1.0 x 10^{-8} m/s and e ≈ 0.80 1.0 x 10^{-9} m/s and e ≈ 1.05	Golder In-situ and lab testing programmes since 2003
	N_{kt}	Calculated from B_q value measured in CPT soundings. Varies between 11 and 17	Golder, CPT and lab testing
	Particle Density	2.7	Golder, lab testing
	Effective Strength (ϕ)	Effective Friction Angle and Cohesion = 30° , c = 0 kPa	Golder, In-situ and lab testing programmes since 2003
	Undrained Strength (s_u/σ'_{v0}) Sandy Silt Silty Clay	Undrained Shear Strength Ratio 0.25 to 0.50 (shear) 0.20 to 0.30 (shear)	Golder, In-situ and lab testing programmes since 2003. TBD at each section

2.2 Consequence Classification of BRDA and Ancillary infrastructure

In accordance with Section 4.2.1.3.4.3 of the 2018 Best Available Techniques (BAT) Reference Document for the Management of Waste from the Extractive Industries, with Directive 2006/21/EC, EUR 28963 EN, (MWEI BREF 2018), and in the absence of a National or EN Standard, AAL have selected to undertake the classification of the BRDA and ancillary infrastructure in accordance with the CDA Guidelines (CDA 2014) and to adopt the target level standard-based criteria for design parameters (inflow design flood, seismic event and factors of safety for static, pseudo-static and post-seismic stability), which are dependent on the consequence of failure.

Tailings dams are classified according to the consequence in the event of failure and takes into account the incremental loss of life, environmental impact and economic impact that a failure of the dam may inflict on downstream or upstream areas, or at the dam location itself. The CDA classification assigned to a dam is the highest rank determined among the incremental loss categories and the dam class range has five (5) categories of consequence: Low, Significant, High, Very High and Extreme.

- Golder has classified the BRDA as a facility with a '**High**' hazard potential classification (HPC) rating, whilst the BRDA water management infrastructure has been classified as dams having a "**Low**" HPC rating.

2.3 Target Levels for Standard-Based Design Criteria

The CDA Guidelines promote a risk-informed approach to dam safety analysis and assessment as it includes deterministic standards-based analysis among many considerations. The consequence classification does not address all of the potential risks presented by a dam and the risk-informed approach is continuing to develop. Hence, a standards-based approach is considered appropriate for elements of dam design and assessment. The target levels for standards-based design criteria for tailings dams during Construction, Operation and Transition Phases are presented in the CDA Guidelines 2014 (CDA 2014) and are replicated in Table 2 below.

Table 2: Target Levels for Standards-Based Design Criteria for Tailings Dam. Source: CDA 2014

Dam Class	AEP [a] - Floods	AEP [a] - Earthquakes	Minimum Static Factor of Safety (FoS)		
			During / at End of Construction	Long Term	Pseudo-static and Post-seismic
Low	1/100	1/100	1.3 (Downstream Slope) depending on risk assessment during construction)	1.5 (Downstream Slope)	1.0 (pseudo-static) 1.2 (post seismic)
Significant	Between 1/100 and 1/1,000	Between 1/100 and 1/1000			
High	1/3 between 1/1,000 and PMF [b]	1/2,475			
Very High	2/3 between 1/1,000 and PMF [b]	1/2 between 1/2,475 and 1/10,000 or MCE [c]			
Extreme	PMF [b]	1/10,000 or MCE [c]			

__Notes:__

a) *AEP is the Annual Exceedance Probability*

b) *PMF is the Probably Maximum Flood. The Probable Maximum Precipitation (PMP) is the greatest depth of precipitation (mm of rainfall) for a given duration that is physically possible over the catchment or drainage area. The PMF is the most severe possible flood (m3/s), which has been calculated for a rainfall equal to the PMP on the catchment or drainage area.*

c) *MCE is the Maximum Credible Earthquake. The MCE is the largest hypothetical earthquake that may be reasonably expected to occur along a given fault or other seismic source could produce under the current tectonic setting. The 1 in 10,000-year event is adopted for the MCE.*

2.4 Geotechnical Parameters

The geotechnical parameters selected for the bauxite residue, process sand and estuarine soils have been assessed following field investigation comprising in-situ testing, sampling, laboratory testing and interpretation by others prior to 2004 and by Golder after 2004 (see Section 5.0).

Geotechnical strength parameters can have a wide range, have a high likelihood of outliers, and are typically dependent on the selection of other parameters for their interpretation. The design or characteristic values for the geotechnical strength parameters for use in the deterministic stability calculations are selected to provide a high level of confidence that the measured values will be greater than the characteristic value i.e., 99% exceedance probability.

Geotechnical index properties typically have a narrower range and a lower likelihood of outliers for a particular soil type or tailings stream and the mean value is typically selected for the characteristic value.

2.5 Critical Stability Aspects

CDA serves as the Canadian National Committee of the International Commission on Large Dams (ICOLD). Bulletin 139 'Improving Tailings Dam Safety' (ICOLD 2011) published by ICOLD Tailings Dam Committee deals with critical aspects of management, design, operation and closure of tailings dams. The following aspects are considered relevant for the BRDA, and the assessment of these aspects is detailed in Section 7.0.

- Seismic Liquefaction;
- Static Liquefaction; and
- Dynamic Liquefaction.

2.6 Inflow Design Flood

The BRDA has been classified to have a "**High**" HPC rating under the CDA Guidelines and hence the Inflow Design Flood (IDF) will be 1/3 between the 1,000-year and the PMF event. A storm duration of 24 hours has been selected for the design criteria; this duration is commonly used for hydrological analysis when the intent is to maximise the volume of water to be stored in the water management system facilities and is considered appropriate for a temperate climatic zone i.e., the south-west of Ireland is classified as Climatic Zone 9.

Note: *The PMF is the most extreme meteorological event, among extreme events, corresponding to a theoretical maximum flood with an undefined return period (i.e., greater than 1 in 10,000 years). The methods for estimating the PMF include accounting for climate change (WMO 2009) and no additional factors are required to be applied to the PMF or the IDF (which is derived from the PMF).*

2.7 Closure Design

The hydraulic design for the Passive Care phase for the BRDA is required to transfer the IDF from the dome and side-slopes to the PIC, mitigating erosion, flood routing and overtopping issues, and subsequently to discharge via the breach locations in the PIC (see Section 8.5.4). Golder considers that the BRDA classification would reduce from a '**High**' HPC to a "**Significant**" HPC following the transition from Active Care Phase to Passive Care and the BRDA enters a long-term equilibrium condition (CDA 2014). The reduction in HPC will require validation of the closure design works, agreement for PIC breaching and discharge, and demonstration of satisfactory performance of equilibrium or reduced phreatic surface and seepage conditions.

A "**Significant**" HPC for Passive Care corresponds to an IDF 1/3 between the 1,000-year and the PMF (CDA 2014). The potential consequences resulting from a flood event of greater magnitude to the IDF i.e., very low probability and high magnitude, and the potential for long-term clogging of the system (locally or globally), have been considered qualitatively in the closure design.

3.0 SITE CONDITIONS

The site conditions are described in detailed in the pertinent chapters of the Environmental Impact Assessment Report (EIAR) and a summary is provided in the Sections below.

3.1 Site Location / Coastal Setting

AAL own a circa 601.22 ha. landholding (the Site) on Aughinish Island which is shown by the blue line on Drawing 01. The Island is predominantly rural in character with the remaining land usage comprising agriculture, single low density residential housing and protected habitats (wetlands and grasslands).

Aughinish Island is located on the south banks of the Shannon Estuary, at approximately 50km from the outlet to the North Atlantic, in the south-west of Ireland, and is bounded by the River Shannon to the north, the Robertstown River to the west and southwest and the Poulaweala creek to the east and southeast. The nearest towns are Askeaton (ca. 6.0 km to the east) and Foynes (ca. 3.5 km to the west) and the Site is located circa 30 km west of Limerick City.

The Limerick – Foynes railway line (now disused) runs to the south of the Island, as does the N69 National Secondary Route between Limerick and Tarbert. Aughinish Island is accessed via the L1234 Aughinish Road, which is a two-way local road which connects with the N69.

Figure 2: Site Location (source Google Maps, annotated by Tom Phillips + Associates, November 2020)

Environmental receptors within the vicinity of the Site include the Lower River Shannon SAC (ca. 500 m north and ca. 600 m south) and River Shannon and River Fergus Estuaries SPA (ca. 500 m north and ca. 650 m south).

The coastal location introduces several risks to the facility i.e., the Shannon Estuary is tidal, and the water elevation may be influenced by tidal events, wave development and storm surge events, climate change and sea level rise. The tsunami hazard for Ireland is considered Highly Unlikely (Ireland Natural Risk Matrix for Natural Hazards 2017) and the risk to the Aughinish Site is further mitigated by its location in the Shannon Estuary and not directly on the coastline. These coastal risks have been assessed as part of the Risk Assessment and Break-out Study for the BRDA (Golder 2019A).

3.2 Geology and Foundation Characterization

The GSI bedrock and groundwater vulnerability 1:100,000 maps (see Figure 3 and Figure 4 below) show that the regional geology of the Site is split between two bedrock formations and there is a significant difference in elevation and soil covering between the two formations.

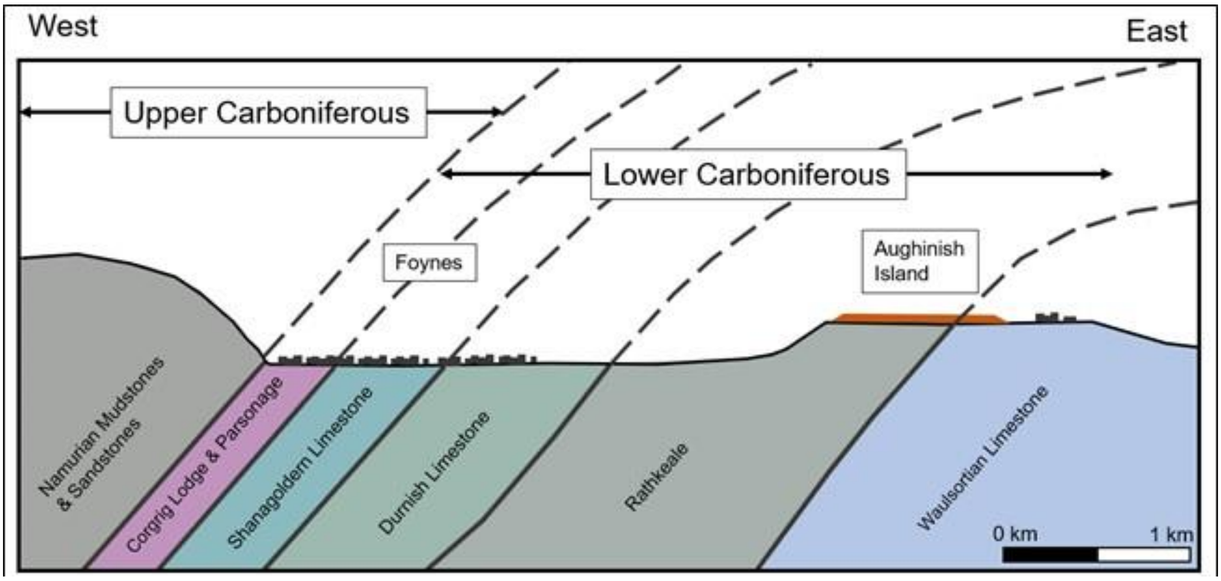

Figure 3: Schematic section showing geological stratigraphy between Foynes and Aughinish Island (Clark et al. 1981)

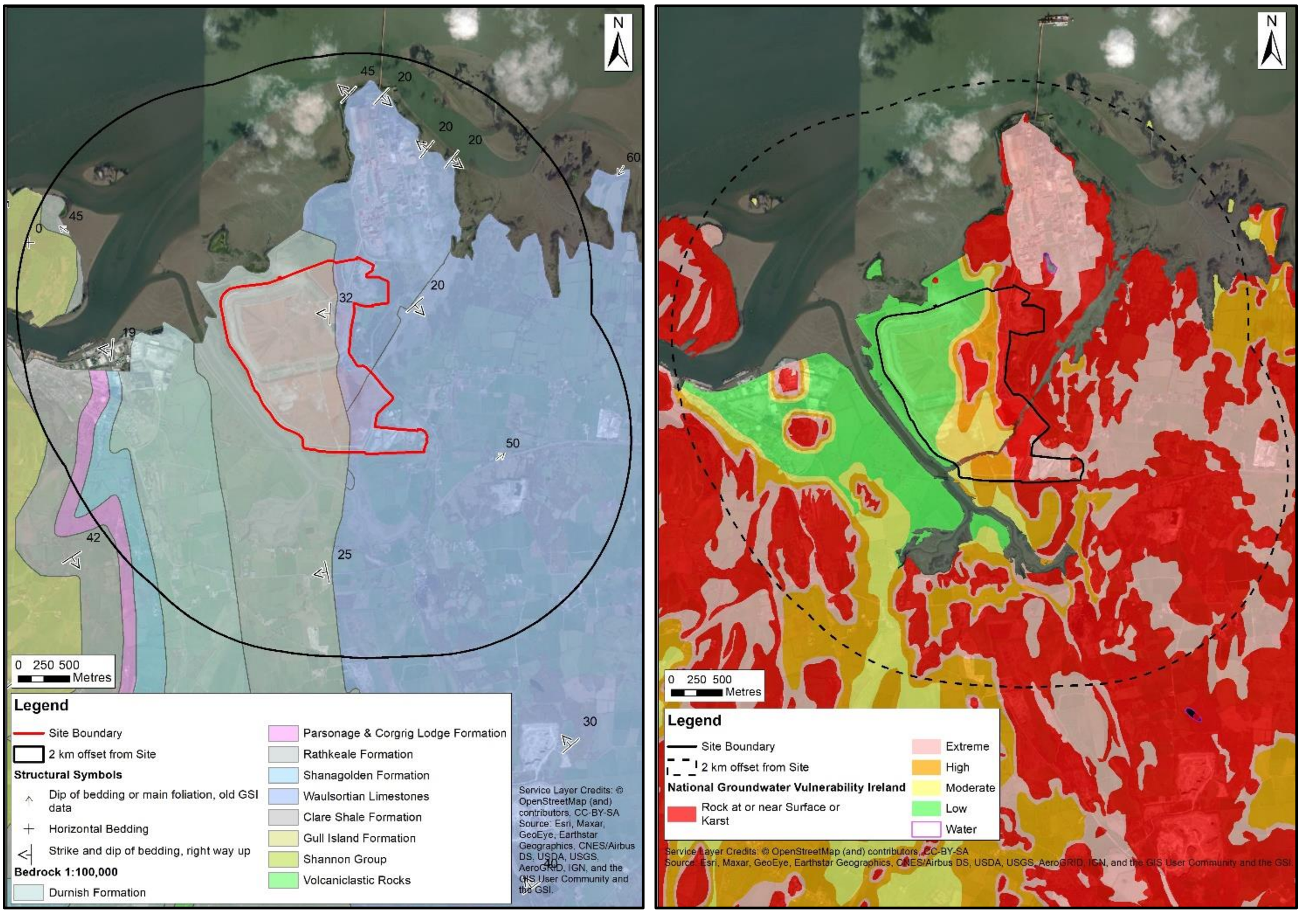

Figure 4: Site Bedrock Geology and Exposed Rock at Surface

- The AAL Plant Site and the Borrow Pit Extension footprint is underlain by marine shelf facies, comprising limestone and calcareous shale and Waulsortian mudbank comprising pale-grey massive, unbedded limestone and is part of the Lower Carboniferous Limestone Group. This Waulsortian limestone outcrops along the eastern extent of the BRDA at a crest elevation of between 14 mOD and 18 mOD and has soil covering of rendzinas - lithosols, which have generally originated from limestone glacial till. Rendzinas are shallow soils, usually not more than 0.5m deep, whose patent material contains over 40% carbonates. Lithosols are shallow, stony soils, usually overlying solid or shattered bedrock (in this case limestone).

- The BRDA is predominately underlain by the Rathkeale Formation, which is described as dark grey, argillaceous (muddy) limestone and shaley mudstone. The Rathkeale Formation dips between 25° and 32° to the west, with no additional structural features noted in the vicinity of the Site (GSI). No karst features have been identified in the footprint of the BRDA. The limestone bedrock has undergone broad folding and faulting with the main fault zones trending NNE-SSW and WNW-ESE which are visible in the outcrops along the shoreline.

  Historical mapping by Ordnance Survey Ireland (OSI) indicates that the bulk of the Phase 1 BRDA and the western sector of the Phase 2 BRDA is constructed over relatively flat, low-lying and poorly drained farmland (elevations between 0 mOD and 2 mOD) with the underlying soils comprising comprise estuarine silts and clays with intermittent overlying thin till layers (sandy gravelly CLAY to silty sandy gravelly CLAY of low plasticity, typically 8% to 10%). The estuarine silts and clays vary in depth from 10m to 30m along the northern perimeter of the Phase 1 BRDA (greatest depth at the north-east and north-west sectors), from 4m to 10m along the western perimeter of the Phase 1 BRDA, from 0m to 8m along the north-western perimeter of the Phase 2 BRDA and are largely absent under the centre of the Phase 1 BRDA, under the Phase 1 BRDA Extension and under the bulk of the Phase 2 BRDA.

  The eastern sector of the Phase 1 BRDA (the Phase 1 BRDA Extension) and the eastern sector of the Phase 2 BRDA are constructed over a ridge of outcropping rock, sloping upwards from west to east, with intermittent thin layers of till material. This ridge was excavated, shaped and surface dressed with a layer of till to permit the installation of the composite lining system for these sectors of the BRDA.

- The SCDC overlies made ground (deposited bauxite residue) with a base elevation of approx. 19 mOD corresponding to an approx. 18m depth of bauxite residue. The new dam walls will overlap the current dam walls (downstream and centre-line raises) which are constructed to elevations between 24 mOD and 29 mOD.

3.2.1 Estuarine Deposits

Generally, two layers of estuarine soils are present, comparable to the findings from the investigation at the adjacent Foynes Harbour (Long 2018).

- **Sandy Silt Layer** – Generally occurs as the surface layer and some underlying layers. Characterised by a higher tip resistance (qt), in the form of spikes and higher undrained shear strength.

- **Silty Clay Layer** – Generally occurs underlying the Sandy Silt layers. Characterised by lower, more uniform tip resistance (qt), and lower undrained shear strength.

Geotechnical properties of the estuarine deposits have been determined from work carried out prior to 2018 (Golder 2005A to 2005D) and as part of the 2018 and 2019 Site Investigations (Golder 2018 and Golder 2020).

3.3 Topography

The Waulsortian bedrock rises a little further to the east into the footprint of the Borrow Pit Extension (to approx. 21 mOD) before dipping to the east towards the Access Road to the AAL Plant (to approx. 12 mOD), to the north towards the southern portion of the AAL Plant (to approx. 15 mOD) and to the south towards the AAL Sports Grounds (to approx. 15 mOD). The average bedrock elevation in the Borrow Pit Extension Footprint is approx. 20 mOD.

The base of the bulk of the BRDA is in the 0 mOD to 2 mOD elevation range with the eastern extent rising in elevation, corresponding to the ridge of outcropping Waulsortian bedrock, to elevations ranging from 14 mOD to 18 mOD. The ground elevation dips sharply along the eastern side of the Phase 2 BRDA, dropping from approx. 16 mOD at the merging with the Phase 1 BRDA Extension to approx. 6 mOD at the south-east sector. The ground elevation then grades to the west and returns to a ground elevation of approx. 2 mOD at the south-west sector.

The current Phase 1 BRDA residue surface varies in elevation from approx. 22 mOD at the perimeter to approx. 32 mOD centrally. The current Phase 2 BRDA residue surface varies in elevation from approx. 11 mOD at the perimeter to approx. 20 mOD centrally. The Phase 2 BRDA is overlapping the southern side-slopes of the Phase 1 BRDA.

The current SCDC has a crest elevation of 29 mOD with the north and east dam wall side-slopes grading down to the perimeter Stage 10 crest at 24 mOD and with bauxite residue deposited to approx. 26 mOD alongside the west dam wall.

There is no external high ground in the vicinity to the Site nor external catchments contributing surface water to the BRDA.

3.4 Climate

The Gulf Stream and the North Atlantic drift, have a major influence on the climate in Ireland. This warm current and the predominantly south westerly winds that blow over it give Ireland an essentially maritime climate of mild winters, cool summers and all-the-year-round rainfall. The west, south and much of the east coastal areas of Ireland are classified as Climatic Zone 9, the remainder of Ireland is Climatic Zone 8.

Summer temperatures are comparatively low. Prolonged summer heat and extreme winter cold are uncommon. In the south-west of the country there is only 8° C difference in temperature between the means of the coldest and warmest months.

Rainfall varies from 750 to 1000 mm in the drier east and midlands and from 1,000 to 1,250 in the south and west. Rain falls in every month of the year, although there is a tendency for the period from May to July to be the driest and for December to February to be the wettest.

3.4.1 Temperature

The Met Eireann website (www.met.ie) provides data for the nearest weather station to the Aughinish Site (Shannon Airport). 30-year mean and extreme values from 1981 to 2010 show an annual mean of 10.7°C with mean daily minimums and maximums between 3.2 and 19.8°C. The absolute minimum and maximum temperatures recorded for the period are -11.4°C and 30.6°C.

The coldest months are December to February and the warmest months are June to August.

3.4.2 Precipitation

Rainfall frequency analysis for the BRDA was previously undertaken by Golder (Golder 2021A) and Table 3 below presents the results of this analysis for a range of design events and durations, including the Probably Maximum Precipitation (PMP) depth, which would result in the Probable Maximum Flood (PMF).

Table 3: Design Rainfall Depths (BRDA)

Rainfall Duration (hrs)	Rainfall Depths (mm)			
	200-year	1,000-year	1/3 between 1,000-year and PMP	PMP
6	58.9	80.9	102.2	144.7
12	70.2	93.6	120.6	174.6
24	83.7	107.6	141.0	208.0
48	103.3	131.3	172.8	255.9

A storm duration of 24 hours is selected as being appropriate for the Irish climate. The "**High**" HPC for the BRDA determines the appropriate Inflow Design Flood (IDF) to be the 1/3 between the 1,000-year and the PMF. Therefore, the design rainfall depth considered for the purposes of flood routing is 141.0 mm.

Golder considers that the BRDA classification would reduce from a '**High**' HPC to a "**Significant**" HPC following the transition from Active Care Phase to Passive Care and the BRDA enters a long-term equilibrium condition (see Section 2.7). A "**Significant**" HPC which also corresponds to an IDF 1/3 between the 1,000-year flood and the PMF and will have a similar design rainfall depth of 141.0 mm.

<u>Note:</u> *The methods for estimating the PMF include accounting for climate change (WMO 2009) and no additional factors are required to be applied to the PMF or the IDF (which is derived from the PMF).*

3.4.3 Wind

The Met Eireann website (www.met.ie) provides data for the nearest weather station to the Aughinish Site (Shannon Airport). The major wind directions range from approximately 110º to approximately 285º (north is 0º), which correspond to range of wind directions from south-east to south-west.

30-year mean and extreme values from 1981 to 2010 show mean monthly speeds ranging from 4.2 to 5.2 m/s with maximum gusts ranging from 26.2 to 42.7 m/s and maximum 10-minute wind speeds ranging from 18.0 to 29.3 m/s, which corresponds to wind force 10 (storm). BS6399 Part II which provides a plot of the fifty-year maximum hourly windspeed for Great Britain and Ireland and shows the Shannon Estuary area to have a maximum hourly windspeed of approximately 25 m/s.

3.5 Hydrology

The Site is bounded to the north and west by the Shannon Estuary, to the northeast by Poularone Creek, to the southeast by Poulaweala Creek and to the southwest by the Robertstown River.

There are no surface water features in the footprint or in the near vicinity of the footprint of the Borrow Pit Extension. The Poulaweala Creek flows from north-east to south-west at approx. 150m distance to the south-east from the closest boundary to the footprint.

Historical mapping suggests that a small groundwater water drain fed by multiple groundwater springs originally flowed southwards along the base of the ridge of limestone / glacial till which previously existed at the BRDA site. This groundwater stream most likely delineated the contact between the outcropping rock and the lower lying, poorly drained estuarine soils. This groundwater stream and other surface water land-drains were culverted during the development of the Phase 1 BRDA and the Phase 1 BRDA Extension and further culverted during the development of the Phase 2 BRDA. The Poulaweala Creek, a former estuarine channel, which originally divided Aughinish Island from the 'Mainland' to the south at Island Macteige and Glenbane West, was also partially culverted and infilled with coarse rock fill during the development of the Phase 2 BRDA. All of the culverted streams beneath the BRDA discharge into the Perimeter Drain at the west side of the BRDA, which leads to the Robertstown River.

Seepage, surface water runoff, sprinkler water runoff and bauxite residue bleed water from the BRDA is collected in the encompassing PIC. Any seepage that bypasses the PIC is collected in the Toe Drains, located downstream of the PIC, from where it is pumped back into the PIC. The PIC waters are conveyed via pumps to the SWP and subsequently to the Effluent Clarification System (ECS) and/or directly to the ECS, which is located in the Plant Site. Treated waters are then pumped back to the LWP for cooling and settling prior to being discharged to the River Shannon. The SWP and LWP are located in the north-east sector of the Phase 1 BRDA. The PICs are separated into PIC segments (PIC-A to PIC-M) that are separated by culverted 'choke points'; these culverted sections provide vehicular access to the BRDA across the PICs.

Surface water in the Perimeter Drain, sourced from the catchment beyond the Toe Drain (and enclosed by the flood tidal defence berm, see Section 3.5.1 below) and from culverted streams beneath the BRDA, is allowed to discharge into the Robertstown River only through a Penstock, located to the west of the Phase 1 BRDA, and via a Flap Valve during periods of low tide. This Penstock can be closed via a manual penstock valve should contamination be identified in the Perimeter Drain or should a significant event occur, that may potentially impact on the water quality in the Perimeter Drain.

Figure 5 below presents the surface water (dark blue) and the sub-BRDA groundwater drains (light blue) drainage patterns associated with the BRDA site overlain on a recent aerial photograph.

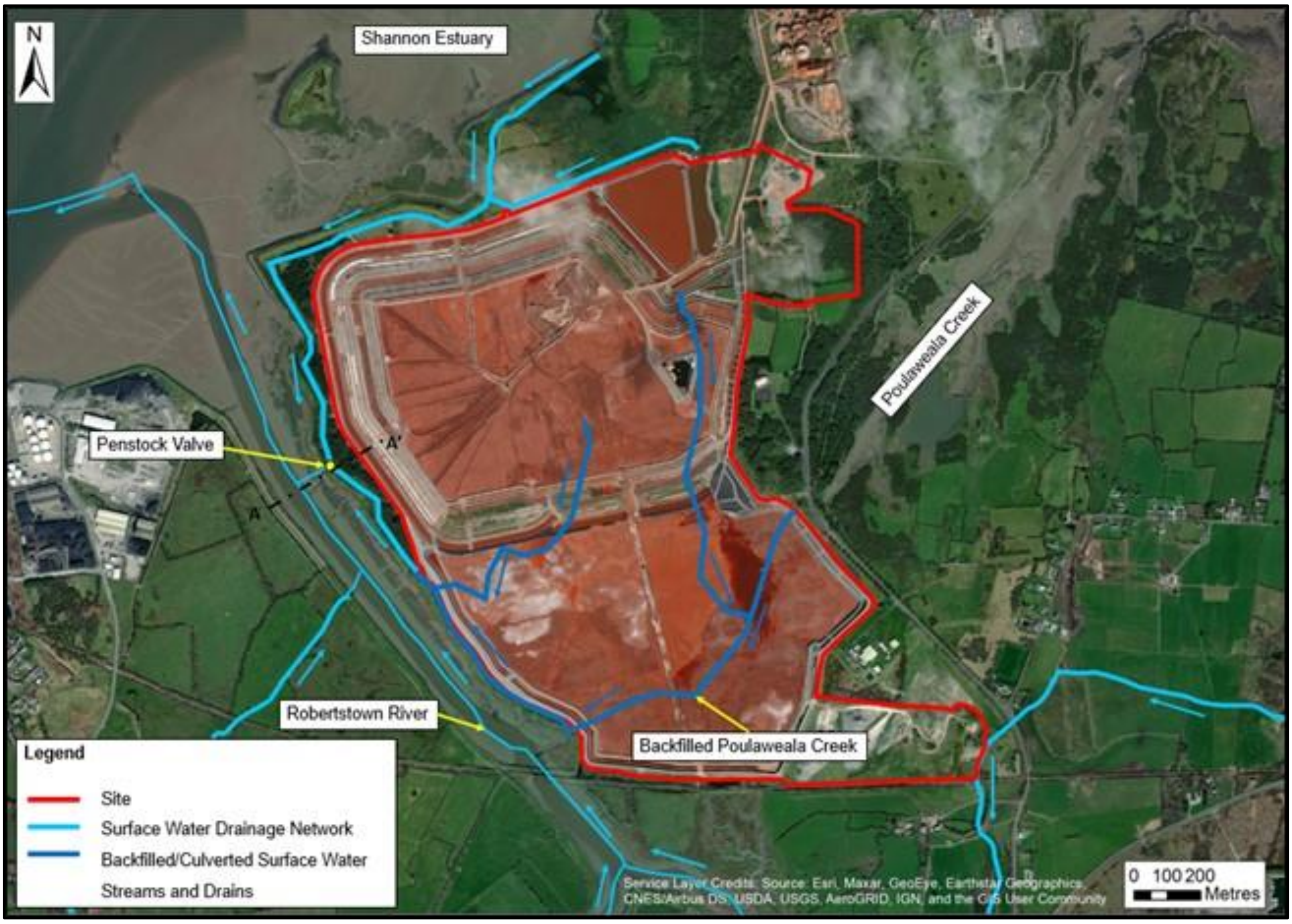

Figure 5: BRDA Surface Water Drainage and Sub-BRDA Drainage (backfilled / culverted)

Figure 6 below presents a schematic west-to-east cross section (A-A' on Figure 5) showing the surface water drainage arrangement from the area surrounding the BRDA to the Robertstown River.

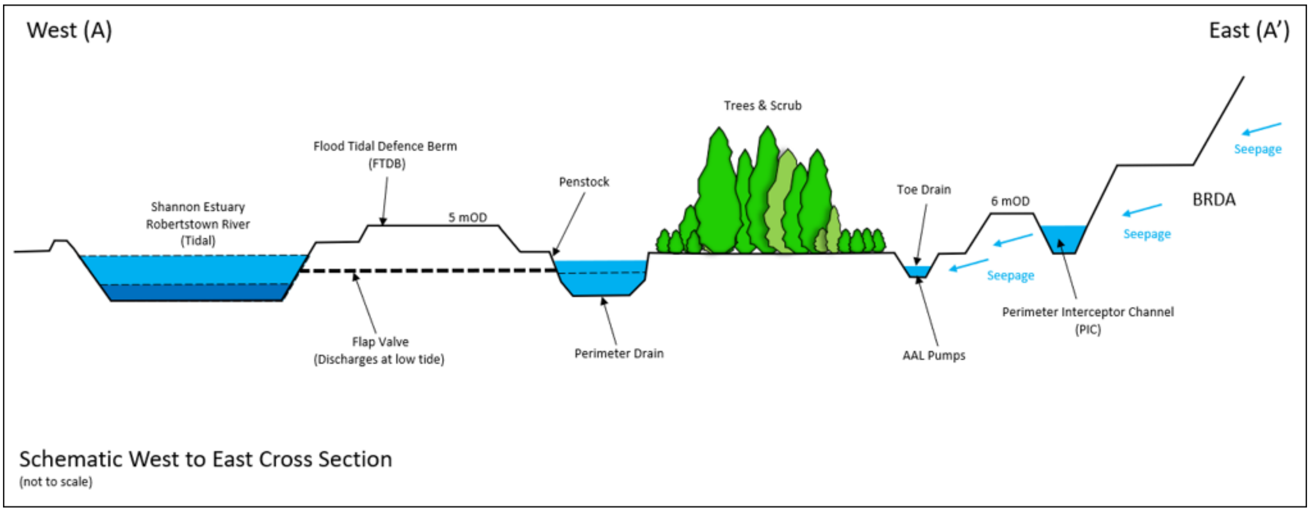

Figure 6: Schematic Cross-Section A-A' showing surface water drainage to Robertstown River

3.5.1 Flooding

Flooding events have been recorded by the Office of Public Works (OPW) to the east and west of the Aughinish Site, and are reoccurring flood events, but no flood events have been recorded at the AAL Plant Site or at the BRDA footprint (OPW www.floodmaps.ie). The BRDA footprint and surrounding catchment is defended by the OPW constructed flood protection works on the north and western banks of the Island, where a flood tidal defence berm (FTDB) has been constructed to a crest elevation of approx. 5 mOD.

3.6 Hydrogeology

The interpretation of the hydrogeological conceptual model presented in Golder 2015 identified that the groundwater present beneath Aughinish Island comprises a freshwater lens isolated laterally from the mainland by being laterally hydraulically isolated by Poulaweala Creek, Poularone Creek and the Robertstown River and the underlying saline groundwater.

The groundwater present in the Waulsortian Limestone underlying the Plant Site, the Borrow Pit Extension and the eastern sector of the BRDA is classified as a Regionally Important Aquifer for the water resources of County Limerick as a consequence of enhanced secondary permeability from faulting and fracturing and enhanced primary permeability from dolomitization. The Waulsortian Limestone bedrock has a low primary permeability (≈ 3×10^{-6} to 8×10^{-8} m/s in the Plant Site and ≈ 2×10^{-5} to 5×10^{-6} m/s beneath the BRDA). As a consequence, flow of groundwater is dominated by the location of karstified fracture zones and valley infill. The depth at which groundwater is encountered across this unit is typically within 1.5m to 10m depth below ground level which implies that the fracture zones start from a relatively shallow depth, and that, in the centre of the unit, groundwater flows preferentially through the limestone rock fill used to level the valleys during the initial construction phase of the Plant Site. In the footprint of the Borrow Pit Extension, the groundwater contour maps indicate a variation in groundwater elevation of between 2 mOD (south) and 7 mOD (north).

Aquifer classification mapping by the GSI indicates that the Rathkeale Formation beneath the BRDA can be classified as a Locally Important Aquifer which is moderately productive in local zones only (LI). This reflects the presence of brackish or saline water bearing bands of marine argillaceous limestones within the mudstone. The Rathkeale Formation also has a very low permeability range (1×10^{-5} to 1×10^{-7} m/s).

An extensive groundwater monitoring network comprising coupled pairs of observation wells (OWs) installed offset from the downstream toe of the outer perimeter wall of the PIC encompasses the BRDA (see Figure 7). The OWs are generally coupled with one well drilled into the overburden and its partner driller drilled into the limestone bedrock. Originally, there were 12 OW locations installed for the Phase 1 BRDA, but some of these have been lost with the margining of the BRDAs, and 18 OW locations were installed for the Phase 2 BRDA.

The principal contaminant indicator is elevated pH and an assessment of the OW monitoring data from 2008 to 2020 is provided in Chapter 10: Hydrology and Hydrogeology of the EIAR; a summary is provided below:

- The annual average pH levels in the majority of the Phase 1 BRDA OWs have remained consistently between pH 8.0 and pH 6.9 between 2008 and 2020. The exceptions are OW1 and OW2 which showed elevated pH levels (between pH 9.0 and pH 10.0) between 2008 and 2010. Since 2010, pH levels in these two wells have steadily decreased and all the Phase BRDA 1 observation wells currently have an annual average pH of below 8.2
- The annual average pH levels for the Phase 2 BRDA OWs (all of which were commissioned in 2011) have remained consistent at a range of pH 6.6 to pH 7.8 between 2011 and 2020.

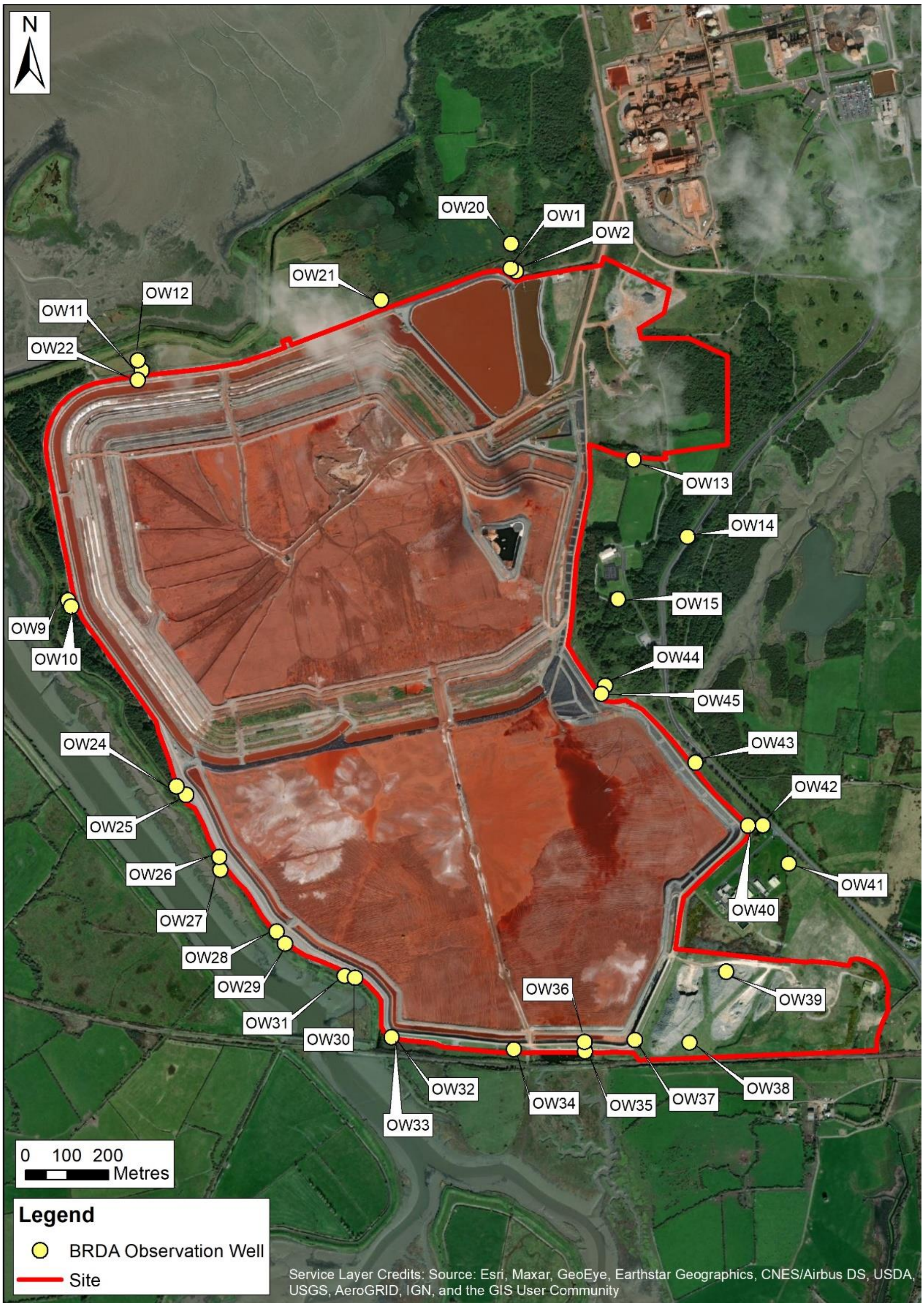

Figure 7: Location of Observation Wells (OWs) encompassing the BRDA

3.6.1 Groundwater Basin

Groundwater basins have been defined by the EPA/GSI to determine the catchment areas and divides within areas, in a similar fashion to the river basins defined for surface water features. The Site occurs within a sub-basin, 'Industrial Facility' (IE_SH_G_252) within the Askeaton Groundwater Basin (GWB) (IE_S_G_010), which is characterized as having a status of 'poor', with the overall Groundwater Body being classified as 'good'.

3.6.2 Groundwater Flow Direction

Groundwater levels measured in monitoring boreholes across Aughinish Island indicate that groundwater flow is outwards from the central part of the Plant Site towards the coastline via springs (the Estuarine Streams) to the Shannon Estuary and the Poularone Creek and is outwards from the central part of the BRDA to the Shannon Estuary, the Robertstown River and the Poularone Creek, as shown in Figure 8 below.

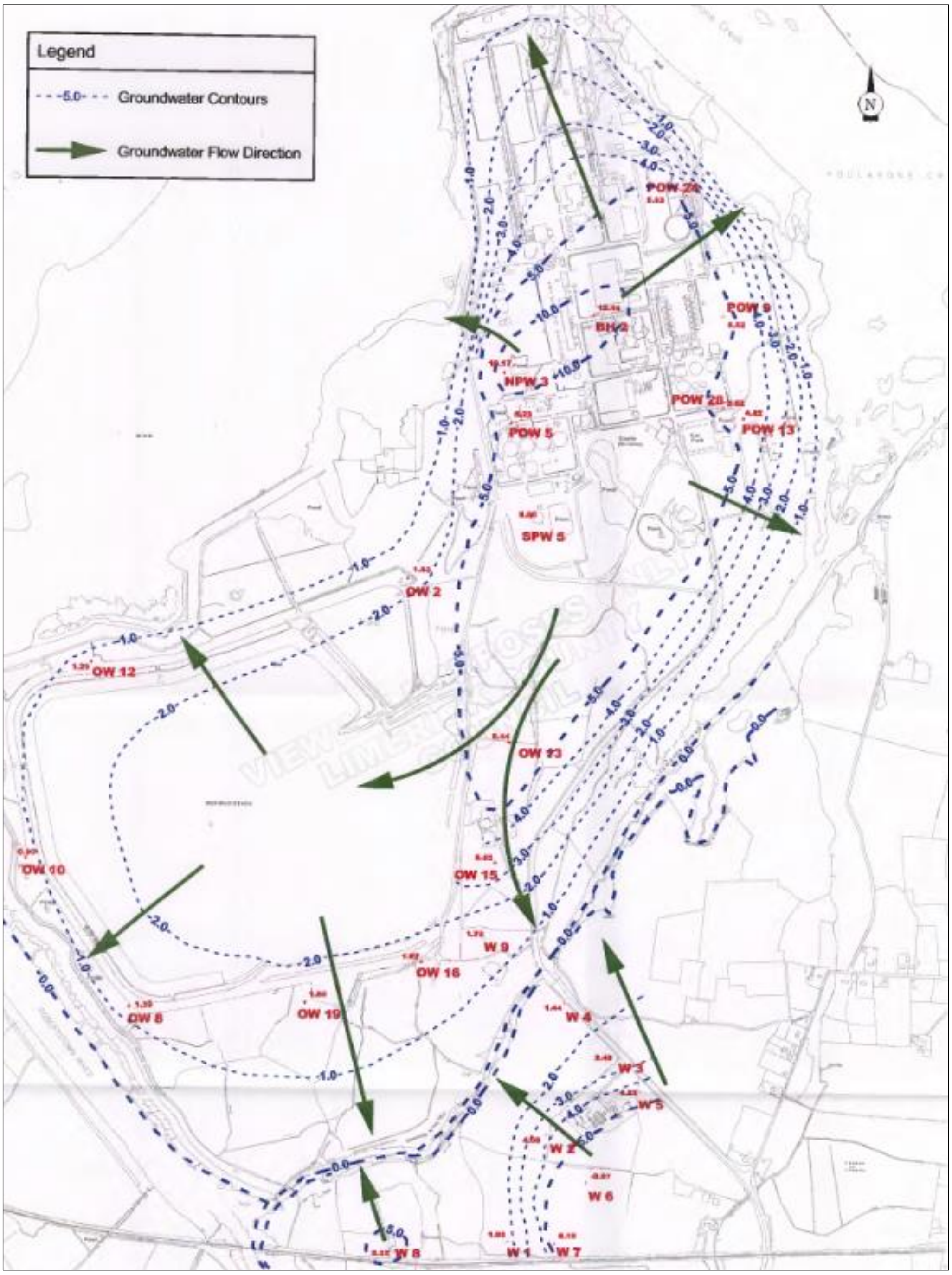

Figure 8: Groundwater Contours mOD (RPS 2005)

Groundwater flow to the west and south of the BRDA is likely to be towards the Robertstown River through flow and run-off from estuarine deposits. Much of the shallow groundwater to the south and west of the BRDA is collected in the Perimeter Drain and discharged via the penstock to the Robertstown River, located to the west of the Phase 1 BRDA.

3.7 Seismicity

The seismic assessment for the Site is described in detail in the Risk Assessment and Break-out Study for the BRDA (Golder 2019) and a summary is provided below.

Ireland lies at the north-west margin of Europe, adjacent to the continental shelf and is characterised by very low levels of seismic activity. This lack of seismic activity in Ireland has been demonstrated by the low number of historical observations, regional seismic assessments and modern instrumental readings. Seismic activity in Ireland is significantly lower than in Britain, despite a similar geology.

The SHARE (Seismic Hazard Harmonization in Europe) Project (SHARE 2013) used the seismicity present in Ireland's source zone to estimate the earthquake activity rate for this region and predicts that there an earthquake with a magnitude of 4.0 or greater approximately can be expected every 476 years,

The HSE document, Seismic Hazard: UK Continental Shelf (HSE 2002) provides contour maps for UK and Ireland and a zonation model which lists the south-west coast of Ireland (zone A13) as an area with an earthquake magnitude observation threshold of 5.0.

Table 4 below provides the peak ground accelerations (PGAs) for various annual exceedance probabilities. The PGA value for 1/2,475 has been extrapolated from the lower return periods and is 0.05 g.

An earthquake of M = 5.0 would be required within 1 km epicentre of the Site for the return period of 2,475 years in order to achieve a PGA of 0.05 g and this is considered the Maximum Design Earthquake (MDE).

Table 4: Aughinish PGAs for various Annual Exceedance Probabilities

Annual Exceedance Probability	PGA (m/s^2)	PGA (g)
1/100	0.09	0.009
1/200	0.12	0.012
1/500	0.2	0.02
1/1,000	0.3	0.03
1/2,475	0.5	0.05
1/10,000 (MCE)	0.9	0.09

A Maximum Credible Earthquake (MCE) is the largest conceivable earthquake magnitude along a recognised fault or within a geographically defined tectonic province.

In areas of low seismicity and areas which lack direct seismic correlation with well-defined or active faults, the MCE ground motions are determined from a probabilistic approach and are typically linked to a long return period e.g., 10,000 years, and is interpreted to be 0.09 g.

However, this greater PGA value would require a larger earthquake than a Magnitude 5.0, which is greater than the estimated magnitude observation threshold.

4.0 TAILINGS CHARACTERISATION

AAL produce four waste streams derived from the extraction process that are deposited in the BRDA (see Table 1). The two waste streams that comprise the bulk of the material deposited (≈ 90.6% bauxite residue and ≈ 6.9% process sand) and that contribute to the stability of the BRDA are discussed below.

The salt cake (≈ 1.0%) is stored in an independent compositely lined cell, within the BRDA, and is discussed further in Section 13.0. The scales and sludges (≈ 1.5%) are hauled and tipped at internal designated areas within the BRDA.

4.1 Bauxite Residue Characterization

The AAL bauxite ore originates from South America (primarily Brazil) and West Africa (primarily Guinea).The bauxite residue results from the production of alumina via the Bayer process. The bauxite ore is crushed, ground and mixed with caustic soda solution and then pumped into digester pressure vessels. Under high pressure and heat, the alumina is dissolved by and combines with the caustic soda to produce sodium aluminate (Digestion Process). The solid residues (bauxite residue and process sand) in the digested bauxite slurry are separated by settling out from the sodium aluminate solution (Clarification Process). The residues are then washed, and the bauxite residue is thickened by both deep thickening and vacuum filtration and pumped as a paste to the BRDA.

Delft Geotechnics undertook a mineralogical study of the AAL bauxite residue (Delft 1988). They concluded that the unfarmed bauxite residue consisted of porous agglomerated particles containing some 70% to 80% of amorphous material (oxides, hydrated oxides and oxi-hydroxides such as boehmite, goethite and gibbsite) with fine crystals of quartz, haematite, rutile and other opaque minerals. A limited number of very coarse haematite and ilmenite crystals of 10 to 70 microns were observed whilst the remainder were less than 4 microns. Little or no clay minerals are present, and the quartz (silica) content is less than 1%. Other silicates include natrodavyne, zeolites, cancrinite and sodalite.

AAL conduct full chemical analyses of the farmed bauxite residue composition on a quarterly basis and Table 5 below provides a summary of the data from 2018 to 2020.

Table 5: AAL BRDA Farmed Bauxite Residue Composition (2018-2020)

Compound	Formula	Wet Basis (w/w%) Range and Average (2018-2020)	
Moisture	Free H_2O	21.64 - 27.52	23.98
Hematite	Fe_2O_3	16.65 - 20.7	17.96
Aluminium Goethite	$(Fe,Al)2O_3.H_2O$	20.79 - 25.33	23.17
Calcium Cancrinite	$3(Na_2O.Al_2O_3.2SiO_2)2CaCO_3$	6.83 - 13.41	10.38
Gibbsite	$Al_2O_3.3H_2O$	3.99 - 4.91	4.55
Bayer Sodalite	$3(Na_2O.Al_2O_3.2SiO_2.2H_2O)0.8Na_2CO_3.0.2Na_2SO_4$	3.10 - 7.37	5.56
Perovskite	$CaTiO_3$	3.10 - 4.29	3.89
Anatase and Rutile	TiO_2	2.67 - 3.7	3.17
Hydrogarnet	$3CaO.Al_2O_3.SiO_2.4H_2O$	1.20 - 4.40	2.34
Boehmite	$Al_2O_3.H_2O$	0.72 - 2.02	1.57
Quartz	SiO_2	0.57 - 1.17	0.90
Sodium Carbonate	Na_2CO_3	0.06 - 0.86	0.46
Zircon	$ZrSiO_4$	0.22 - 0.28	0.25
Gypsum	$CaSO_4.2H_2O$	0.04 - 0.19	0.11

Carbonate Apatite	$5.2CaO.0.8Na_2O.2.5CO_2.P_2O_5$	0.28 - 0.38	0.32
Sodium Sulphate	Na_2SO_4	0.00 - 0.28	0.06
Sodium BiCarbonate	$NaHCO_3$	0.00 - 0.45	0.08
Sodium Fluoride	NaF	0.00 - 0.02	0.01
Sodium Aluminate	$NaAl(OH)_4$	0.02 - 0.11	0.06
Sodium Hydroxide	NaOH	0.00 - 0.05	0.00
Trace Metals: Semi-Quantitative XRF			
Chromium TriOxide	Cr_2O_3	0.12 - 0.16	0.14
Vanadium Pentoxide	V_2O_5	0.00 – 0.00	0.00
Magnesium Oxide	MgO	0.07 - 0.12	0.10
Cerium Oxide	CeO	0 .00- 0.00	0.00
Potassium Carbonate	K_2CO_3	0.02 - 0.06	0.04
Manganese Oxide	MnO	0.02 - 0.04	0.03
Gallium TriOxide	Ga_2O_3	0.00 - 0.01	0.01
Arsenic TriOxide	As_2O_3	0.00 - 0.01	0.00
Niobium PentOxide	Nb_2O_5	0.01 - 0.01	0.01
Zinc Oxide	ZnO	0.00 - 0.01	0.00
Lead oxide	PbO	0.00 - 0.01	0.01
Yttrium TriOxide	Y_2O_3	0.01 - 0.01	0.01
Strontium Oxide	SrO	0.00 - 0.01	0.01
Copper Oxide	CuO	0.00 - 0.05	0.01
Cobalt Oxide	CO_3O_4	0.00 – 0.00	0.00
Thorium Oxide	ThO	0.00 – 0.00	0.00

The five (5) principal compounds of the farmed bauxite residue, which account for ≈ 75% of the composition, are Moisture, Aluminium Goethite, Hematite or Ferric Oxide (which accounts for the characteristic colour), Calcium Cancrinite and Bayer Sodalite. These five (5) compounds have no associated hazardous classification. Gibbsite, Perovskite and Antase & Rutile make up the next (3) largest compounds, which account for ≈ 13% of the composition. Antase & Rutile have hazardous classifications in their pure form but at the 4 to 5% range present here, their concentrations are not considered to confer hazardous properties. The overall classification for the AAL farmed bauxite residue is non-hazardous (see Chapter 6: Human Health of the EIAR).

Hydraulic deposition discharge of bauxite residue paste is from 'Mud Points' located centrally within the BRDA into purpose-built cells. The bauxite residue paste then migrates by gravity to perimeter stage raises and/or cell bunds at between 2% and 4% grade, and dewatering occurs through the rock fill of the stage raises. Layered deposition to aid dewatering of the paste has been implemented since start-up and AAL have engaged intensive mud-farming techniques since 2009. AAL pump out their bauxite residue paste at approx. 58% solids content and mud farming increases the solids content to approx. 74%. The bauxite residue is farmed, carbonated and compacted using specially adapted plant and subsequently allowed to mature for as long as possible (process typically takes 19 to 20 weeks) prior to placing the next layer. The moisture content and pH are thus reduced (via carbonation to pH < 11.5 in accordance with the IEL Conditions) and the density and strength parameters are increased. The early introduction of the amphirol on the bauxite residue surface is key to the mud-farming timeline as it compresses the surface of the bauxite residue and enhances the drying and carbonation processes by increasing the surface area of the bauxite residue exposed, but there is a threshold solids content for access (typically > 66%).

4.2 Process Sand Characterisation

Process sand is a bi-product of the Bayer process removed at the clarification stage by sand traps (see Section 4.1) and contains a level of caustic soda. Process sand is hauled from the AAL Plant and tipped at designated locations in the BRDA. It is typically used in the construction of internal haul roads and berms in the BRDA and has also been utilized in the 'amended mud' layer when washed (see Section 8.2.1).

Delft Geotechnics undertook a mineralogical study of the AAL process sand (Delft 1988). The process sand consists of an agglomerate of particles of less than 1,000 microns (1 mm). The agglomerates comprise clusters of mineral grains which are generally less than 4 microns (0.004 mm). The mineral grains are amorphous or very poorly crystalline and comprise red brown friable particles of oxides, hydrated oxides and oxi-hydroxides such as boehmite, goethite and gibbsite which are sub rounded and readily crushed between the fingers.

4.3 Geotechnical Characterisation

The geotechnical parameters selected for the bauxite residue, process sand and estuarine soils have been assessed following field investigation comprising in-situ testing, sampling, laboratory testing and interpretation by others prior to 2004 and by Golder after 2004 (see Section 5.0).

Geotechnical strength parameters can have a wide range, have a high likelihood of outliers, and are typically dependent on the selection of other parameters for their interpretation. The design or characteristic values for the geotechnical strength parameters for use in the deterministic stability calculations is recommended be selected to provide a high level of confidence that the measured values will be greater than the characteristic value. The confidence % (or equivalent percentile / fractile) of the characteristic value is then combined with the Factor of Safety (FoS) to determine the 99% exceedance probability (Been and Jefferies, 2016), e.g., a 70% confidence value (or 30th percentile) combined with a FoS = 1.45 would provide the desired 99% exceedance probability. A range has been selected for the characteristic strength parameters in Table 1 as the characteristic value is determined for each stability section based on the interpreted CPTu strength, which is validated by laboratory testing of samples taken at the section, where available. The geotechnical strength parameters and the seismic parameters are discussed further in Section 7.0, as part of the stability and liquefaction assessments.

Geotechnical index properties typically have a narrower range and a lower likelihood of outliers for a particular soil type or tailings stream and the mean value is typically selected for the characteristic value, which is ≈ 50th percentile. Combined with a FoS = 1.45, would provide a 72.5% exceedance probability for measured values.

4.3.1 Bauxite Residue

Bauxite residue is generally regarded as a thixotropic clayey silt and there is an indication that bauxite residues may be cemented or aggregated. The bauxite residue particles are sub-rounded, friable with a low crushing strength. Based on the mineralogy, it can be expected that the bauxite residue would not behave as a clay but would exhibit properties similar to those of a granular silt. However, unlike conventional soils, the amorphous particles could retain water which could have none or a limited effect on the overall geotechnical properties of the material (Golder 2014). Typical index properties for the AAL bauxite residue are discussed below:

- **Gradation:** The majority of the material is clay and silt size. About 90% by weight of the bauxite residue is finer than 40 microns and the D_{50} is between 2 and 5 microns (0.002 to 0.005 mm)

- **Moisture Content:** Moisture content values typically range between 32% and 45% for unfarmed bauxite residue and typical range between 29% and 36% for farmed bauxite residue (Golder testing from 2004 to 2019). Characteristic values of 39% and 33% are selected for unfarmed and farmed bauxite residue, respectively. Golder 2014 assessed the amorphous water content and determined that the AAL bauxite residue contains a mean of 1.3% and a maximum of 3.0% of moisture contained within the cryptocrystalline structure. This amorphous moisture does not form part of the “free water” and as such cannot be lost during normal consolidation. As such this water should not be considered when deriving geotechnical properties.

- **Atterberg Limits:** Liquid limits (LL) ranging between 41% and 47% and plastic limits (PL) generally between 29% and 36% with reasonably consistent results and on the Casagrande chart plots as a silt of intermediate plasticity (PI in the 10 to 15 range).

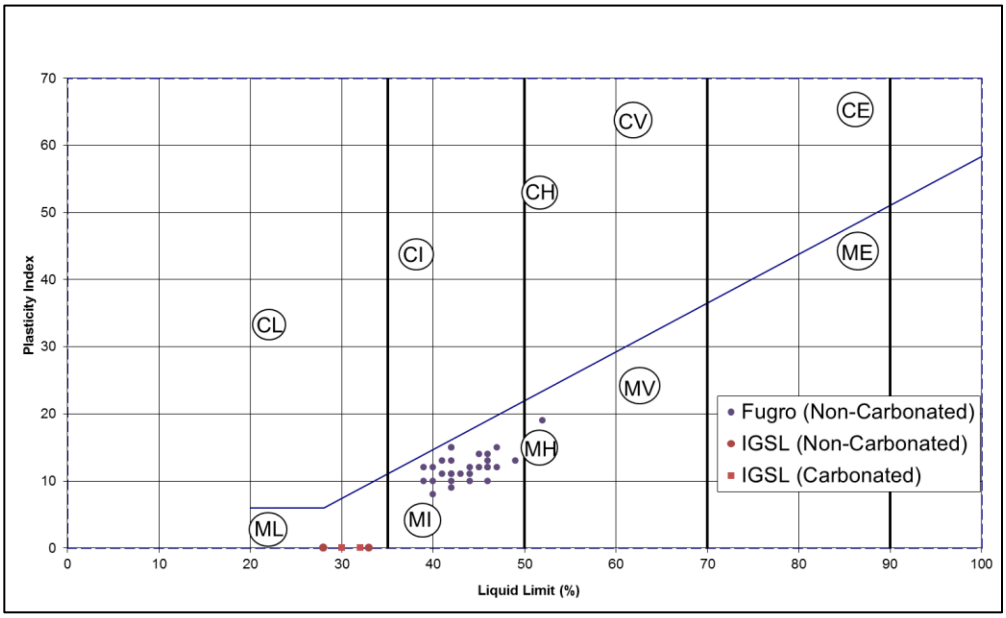

Figure 9: Atterberg Limit Test Results for Unfarmed and Farmed Bauxite Residue (Golder 2014)

- **Density:** Bulk and dry density values from Golder testing between 2004 and 2019.

  Bulk density values for unfarmed and farmed bauxite residue ranges between 2.02 and 2.32 Mg/m^3 and a characteristic value of 2.19 Mg/m3 is selected for both, which corresponds to a unit weight of 21.48 kN/m^3.

  Dry density for unfarmed bauxite residue ranges between 1.48 and 1.66 Mg/m^3 and a characteristic value of 1.58 Mg/m3 is selected, which corresponds to a unit weight of 15.49 kN/m^3.

  Dry density for farmed bauxite residue ranges between 1.51 and 1.71 Mg/m^3 and a characteristic value of 1.63 Mg/m3 is selected, which corresponds to a unit weight of 15.98 kN/m^3. Dry Density – Moisture Content (DD-MC) relationship testing (Golder 2014 and 2020) using the 2.5 kg Standard Proctor suggests an optimum moisture content (OMC) of 26% to achieve a maximum dry density (MDD) of 1.70 Mg/m^3.

- **Particle Density:** The particle density of the AAL bauxite residue has been measured by both the Alcan Method and the British Standard Method (BS 1377, part 2, 1990) by Golder between 2004 and 2019. The measured values have varied between 3.2 and 3.7 and a characteristic value of 3.4 is selected. Typically, the Alcan Method had returned slightly lower values than the BS Method.

- **Void Ratio:** Void ratio values from Golder testing between 2004 and 2019. The established void ratio range for the AAL bauxite residue is from approx. 0.85 to 1.50. In-situ unfarmed bauxite residue void ratios typically range from 1.05 to 1.25, with a characteristic value of 1.17 selected. In-situ farmed bauxite residue void ratios typically range from 0.99 to 1.13, with a characteristic value of 1.09 selected.

- **Consolidation Parameters:** Vales returned from Golder testing in 2011 and 2019.

  Golder 2011 conducted consolidation testing on unfarmed bauxite residue samples and returned values for the coefficient of volume compressibility, m_v in the range of 0.3 m^2 / MN to 3.0 m^2 / MN and for the coefficient of consolidation, c_v in the range of 1 to 8 m^2 / year for 50% consolidation and 3 to 32 m^2 / year for 90% consolidation for depths up to 5 m, which are indicative of material with high to very high compressibility and is still undergoing consolidation.

Golder 2019 conducted consolidation testing on farmed bauxite residue samples and returned values for m_v ranging from 0.020 to 0.081 m^2 / MN and for c_v ranging from 33 to 100 m^2 / yr, which indicate a material with very low to low compressibility and that has already undergone the bulk of its potential consolidation.

Note: *No significant variation in density or void ratio of the unfarmed bauxite residue is noticeable from 2004 to 2018. The thixotropic nature of the bauxite residue, which leads to a build up of strength over time by forming a structure, may be partly restricting consolidation (Poulus et. al. 1985) and (Newson et al. 2006).*

- **Hydraulic Conductivity:** The AAL bauxite residue is consider a material with very low permeability.

  Hydraulic conductivity testing on unfarmed bauxite residue by Golder and others has returned values ranging from 1 x 10^{-10} m/s to 5.6 x 10^{-9} m/s, with a characteristic value of 5.0 x 10^{-9} m/s is selected for k_v.

  Golder 2016 conducted hydraulic conductivity testing on farmed bauxite residue and returned values ranging from 8.5 x 10^{-9} m/s to 3.7 x 10^{-8} m/s, with a characteristic value of 1.9 x 10^{-8} m/s is selected for k_v. It is considered that the layered deposition and mud farming results in preferential paths that provide a greater horizontal conductivity value i.e., $k_h \approx 10$ times greater than k_v.

  Golder 2021B conducted hydraulic conductivity testing on amended layer bauxite residue and returned values ≈ 1 x 10^{-10} m/s. This was unexpected as higher conductivity of the amended layer material was expected due to the addition of sand to create pore space. It is considered that the presence of compost in the core samples may have adversely impacted the laboratory testing. A bulk hydraulic conductivity of 1.0 ×10^{-6} m/s was assigned for the amended bauxite residue (no hydraulic conductivity anisotropy).

4.3.2 Process Sand

Process sand is a poorly graded, medium sand bi-product of the Bayer process, primarily resulting from the addition of limestone in the early stages. Typical properties of the AAL process sand following from testing in 2014 and 2019 (Golder 2016, Golder 2019B) are discussed below:

- **Gradation:** 100% of the particles less than 2mm in diameter, ≈ 50% of particles between 2mm and 0.425mm in diameter and ≈ 96% of particles greater than 0.063mm in diameter. Circa 25% of the particles are in the coarse sand range, ≈ 60% in the medium sand range and ≈ 15% in the fine sand or smaller particle range. The coefficient of uniformity, C_u = 2 and the coefficient of curvature, Cc = 0.72, both which correspond to a poorly graded sand.

- **Density:** The average in-situ bulk density of the placed process sand in the BRDA was assessed to be 1.71 Mg/m^3 which corresponds to a dry density of 1.45 Mg/m^3 at 18% moisture content, corresponding to a unit with of 19.0 kN/m^3. Dry Density – Moisture Content (DD-MC) relationship testing using the 2.5 kg Standard Proctor suggests an optimum moisture content (OMC) of 19.5% to achieve a maximum dry density (MDD) of 1.61 Mg/m^3.

- **Friction Angle:** The angle of shearing resistance (friction angle) was determined to be 33.5 degrees and was assessed by Quick Shearbox Tests on samples compacted to 90% of the 2.5 kg Standard Proctor and prepared to the average in-situ state.

- **Moisture Content:** Moisture content values typically range between 13% and 23% and a characteristic value of 18% has been selected.

4.4 Geochemical Characterisation

There are no Metal Leaching (ML) or Acid Rock Drainage (ARD) issues in regard to the AAL waste residues.

Both Thorium 232 and Uranium 238 are present in measurable but low amounts in the bauxite residue and have been assessed by the Radiological Protection Institute of Ireland (RPII) as naturally occurring radioactive material (NORM) which are at levels below the threshold of the regulations and do not present a hazard.

The pH for the unfarmed bauxite residue is typically between 12.0 and 13.0 and farmed bauxite is < 11.5.

Process sand initially has a similar pH level to the unfarmed bauxite residue (12 to 13 range) but is quickly reduced to < 11.5 following deposition due to weathering effects. It is further washed to reduce pH for use in the Amended Mud composition (see Section 8.2.1).

Salt cake is a bi-product of the process of purification of the caustic soda liquor used in the alumina extraction process from the bauxite ore. Salt cake is classified as a hazardous waste and is required to be segregated from the bauxite residue within the BRDA i.e., within the composite lined, independent SCDC. The salt cake has a high concentration of caustic soda (≈ 40%), Oxalate (≈ 26%), Alumina (≈ 16%) and Organic Carbon (≈ 11%). The caustic liquor is decanted from the cell via a caustic recovery system (decant tower, recovery pipeline and recovery tank) and is recycled in the Plant.

During operation, all residue influenced water (IW) comprising seepage, surface water runoff, sprinkler water runoff and bauxite residue bleed water from the BRDA is collected in the encompassing perimeter interceptor channel (PIC) with any seepage bypassing being collected in the Toe Drain and pumped back to the PIC. The PIC waters are conveyed via pumps to the SWP and subsequently to the Effluent Clarification System (ECS) and/or directly to the ECS, which is located in the Plant Site. Treated waters are then pumped back to the LWP for cooling and settling prior to being discharged to the River Shannon.

At closure, the dome of the BRDA will have a minimum 1m depth layer of Amended Mud and vegetative cover (see Section 8.3). The Amended Mud typically has a pH in the 8.0 to 9.0 range. The side-slopes of the BRDA shall be capped with rock fill to provide a drainage blanket and will have an overlying vegetative cover (see Section 8.4), thus alleviating direct surface water flow contact with the bauxite residue.

At closure, the SCDC will have a composite geosynthetic lined cap and will also be overlain with a minimum 1m depth layer of Amended Mud and vegetative cover (see Section 13.0).

Golder 2021D provides an assessment for the potential water quality following closure of the BRDA at Stage 16. In the post-closure conditions for the BRDA, the water quality in the perimeter interceptor channel (PIC) is expected to be comprised predominantly of runoff (dilute contact water over the surface of the BRDA) and a minor amount of seepage (highly alkaline liquid held in the pore space of the bauxite residue, expressed slowly as seepage due to overlying pressure). After mixing, equilibration with atmospheric carbon dioxide and oxygen, precipitation of pertinent secondary mineral phases and sorption of trace metals onto precipitated iron hydroxides, the water quality has the potential to be at a pH < 9 at discharge, although there are several slightly elevated dissolved metals concentrations (arsenic, copper and zinc).

Note: *The regulatory thresholds for discharge of waters from the future closed BRDA have not yet been established.*

4.5 Rheological Characterisation

AAL have conducted an assessment of the bauxite residue rheology; the following data has been extracted from the progress reports (AAL 2020).

- The average mud-line pressure is typically in the 6,300 to 6,500 kPa range and the average flow rate is ≈ 207 m^3/hr.
- The viscosity of the bauxite residue paste at discharge at ≈ 58% solids content is approx. 900 Pa.s.
- The viscosity of the bauxite residue paste following maturation for 8 to 9 weeks (and suitable for amphirol access) is approx. 5,000 Pa.s and the solids content has improved to > 66.0%.
- Mud-farming comprising amphiroling and aging continues until a pH of < 11.5 is achieved and the final surface is compacted. The typical solids content at completion is approximately 74%.

5.0 FIELD INVESTIGATIONS

A considerable amount of site investigation work has been carried out beneath and within the footprint of the BRDA during its existence. Table 6 below provides a timeline and summary of the site investigation scopes.

Table 6: Timeline and Scope of Site Investigations for the BRDA and Borrow Pit Extension

Year	Consultant	Scope of Investigation
1971 to 1973	Soil Mechanics Limited (SML)	Preliminary Study for the Plant and the BRDA, Boreholes and Trial Pits, 7 No. Boreholes in Phase 1 and 4 No. Boreholes in Phase 2, (SML 1971, 1973)
1974 to 1975	Soil Mechanics Limited (SML)	Further Site Investigation for the Plant and the BRDA, Boreholes and Trial Pits, (SML 1974, 1975)
1974	Ercon	Hydrological Study of the Plant Site and the BRDA, (Ercon 1974)
1979 to 1982	Ercon	Interim Site Investigation Reports for the Plant Site and BRDA, Boreholes and Monitoring Wells in Plant Site, Ercon (1979 – 1982)
1987 to 1988	Site Investigations Limited (SIL)	Site Investigations for the Phase 1 BRDA Extension and Ponds, 18 No. Boreholes, (SIL 1987, 1988)
1988	Delft Geotechnics	Site Investigation and Laboratory Testing of Bauxite Residue, Estuarine Soils and Till, Capacity Optimization Assessment for the BRDA 16 No. Boreholes for Phase 1 BRDA, 5 No. Boreholes for Phase 2 BRDA, 202 No. DCP Tests for the Phase 1 BRDA and 127 No. for the Phase 2 BRDA and Shear Vane Testing, (Delft 1988)
1989	University College Galway (UCG) and Irish Geotechnical Services Limited (IGSL)	CPT Soundings, Estuarine Thickness Probes, Shear Vane Testing and Laboratory Testing for Bauxite Residue, Estuarine Soils and Till for the BRDA 32 No. Boreholes and 6 No. CPT Soundings, (UCG 1989 and IGSL 1989)
1992 to 1993	University College Galway (UCG)	Tests Pits, Shelby Tube Sampling, Laboratory Testing and Monitoring Wells Installations for the BRDA, (UCG 1993)
1993	Geocon	Field Tests for the Phase 1 BRDA Extension, Review of pre-1989 data and Site Investigation Data from 1989 and 1992-1993, (Geocon 2003)
1993	Ove Arup	EIS for the Phase 1 BRDA Extension, (Ove Arup 1993)
1999 to 2002	Glover, Lankelma, Dames and Moore and URS	Boreholes, Piezometer Installation, Shear Vane Testing, Window Sampling, CPTu, and Laboratory Testing of Bauxite Residue, Hydrogeological Study for the Plant and Phase 1 BRDA, 12 No. Boreholes, 12 No. Piezometer Installations and 15 No. CPTu, (Glover 2002, Lankelma 2003, URS - Dames and Moore 1999 - 2002)

Year	Consultant	Scope of Investigation
2004	Golder and Fugro	Site Investigation for the Phase 1 BRDA Raise: Stage 7 to Stage 10, Review of previous Site Investigation Data from 1971 to 2003, Sampling, Shear Vane Testing, CPTu and Laboratory Testing of Bauxite Residue, 20 No. CPTu, 99 No. Shear Vane Tests and 12 No. Vibrating Wire Piezometers Installations, (Golder 2005A)
2004	Golder and Fugro	Site Investigation for the proposed Phase 2 BRDA to Stage 10. Review of previous data (1971 to 2003), Trial Pitting, Sampling, Shear Vane Testing, Boreholes, SPT, and Piezometer Installation, 43 No. Trial Pits, 11 No. Boreholes with SPT and 5 No. Piezometers Installations, (Golder 2005B)
2005	RPS Group	EIS for Expansion of the BRDA, Phase 1 BRDA Raise to Stage 10 and Phase 2 BRDA, (RPS 2005)
2005 to 2007	Golder and Fugro	Site Investigation in the Phase 1 BRDA. New CPTu testing, Shear Vane Testing, Sampling, Monitoring Instrument Installation (Piezometer and Inclinometers) and Laboratory Testing of Bauxite Residue (Golder 2005 – 2007)
2010	Golder and Fugro	Site Investigation: Ongoing Monitoring and Assessment of the performance of the operational Phase 1 BRDA. Review of previous CPTu testing (2004 to 2010), New CPTu testing, Shear Vane Testing, Sampling, Monitoring Instrument Installation and Laboratory Testing of Bauxite Residue. 23 No. CPTu in the Phase 1 BRDA and 10 No. Piezometer Installations, (Golder 2011)
2014	Golder and Fugro	Site Investigation: Ongoing Monitoring and Assessment of the performance of the operational Phase 1 BRDA. New CPTu testing, Shear Vane Testing, Sampling, Monitoring Instrument Installation and Laboratory Testing of Bauxite Residue. 38 No. CPTu in the Phase 1 BRDA and 16 No. Piezometer Installations, (Golder 2014)
2016 to 2021	Golder	Site Investigation for the Borrow Pit and Borrow Pit Extension. Desktop Review, Geophysical Surveys, Boreholes, Monitoring Well Installations, 5 No. Boreholes and 3 No. Monitoring Wells, (Golder 2017A) and 7 No. Monitoring Wells, (Golder 2021F)
2018	Golder and Fugro	Site Investigation: Ongoing Monitoring and Assessment of the performance of the operational Phase 1 and 2 BRDA. Review of previous CPTu testing, New CPTu testing, Seismic CPTu, Sampling, Monitoring Instrument Installation and Laboratory Testing of Bauxite Residue and Estuarine Soils. 39 No. CPTu in the Phase 1 BRDA and 3 No. CPTu in the Phase 2 BRDA, 9 No. Piezometer Installations, 6 No. Inclinometer with Extensometers Installations (Golder 2018)
2019	Golder and Fugro	Site Investigation for specific sectors of the Phase 1 and Phase 2 BRDA. CPTu testing, Shear Vane Testing, Sampling and Laboratory Testing of Bauxite Residue and Estuarine Soils. 21 No. CPTu in the Phase 1 BRDA, 4 No. CPTu in the Phase 2 BRDA and 4 No. at the downstream toe of the north-west sector of the Phase 2 BRDA, (Golder 2020)
2021	Golder	Laboratory testing and constitutive modelling calibration of AAL bauxite residue to determine NorSand parameters (Golder 2021E)

5.1 Geotechnical Investigation

Table 6 provides a summary of the major site investigations undertaken at the Site. A variety of near surface and deeper investigation techniques have been undertaken for the foundation soils and underlying bedrock for the BRDA during the period 1971 to 2004.

The engineering properties of the bauxite residue were investigated in 1988 and geotechnical investigation since 2004 has primarily focused on the bauxite residue that has been deposited and the underlying estuarine soils for the Phase 1 BRDA. In recent years, the Phase 2 BRDA has achieved sufficient depth of bauxite residue such that CPTu investigation has been undertaken and the initial monitoring instruments have been installed.

The Borrow Pit footprint was investigated in 2016 / 2017 and its findings are applicable to the proposed Borrow Pit Extension footprint. Additional monitoring wells were installed for the Borrow Pit Extension during 2020 and 2021 (Golder 2021F).

The references listed in Table 6 provide the detailed logs and interpretations of their respective investigations. Reviews of previous data have been carried out at various intervals, in particular for Geocon 1993, Golder 2005A, Golder 2005B, Golder 2011, Golder 2018 and Golder 2020.

5.1.1 Near Surface Investigation Methods

The following near surface investigation methods were undertaken to assess the estuarine soils and tills in the footprint of the BRDA. The bulk of the tills in the footprint were subsequently excavated and reutilized as the lower component of composite lining system for the Phase 1 BRDA Extension, the Phase 2 BRDA and the Ponds, for surfacing dressing of exposed bedrock and/or for the construction of the perimeter dam walls.

- Trial Pits (Estuarine Soils and Tills)
- Boreholes (≈ 90 No.), SPTs, and Piezometer Installations (Estuarine Soils and Tills)
- Shear Vane Testing (Estuarine Soils and Bauxite Residue)
- Sampling via Shelby Tube, Split Spoon and Window Sampling (Estuarine Soils and Tills)
- Dynamic Cone Penetrometer (DCP) Tests (319 No.)
- 'Pionjar' Probe (Estuarine Soils thickness) and Sampling
- Geophysics Resistivity Survey (Borrow Pit Footprint)

5.1.2 Deep Investigation Methods

The following deep investigation methods were undertaken to assess the underlying bedrock in the footprint of the BRDA.

- Boreholes (≈ 10 No.) and monitoring well installations.
- Sampling via rock coring

5.1.3 Data Interpretation

The BRDA has been divided into sectors which have similar foundation conditions, bauxite residue deposition characteristics and side-slope profile. These sectors are represented by stability Sections i.e. A-A to X-X.

The following Sections (5.2 and 5.3) provide a representation of the data obtained and demonstrate the methodology utilized for the determination of the select geotechnical parameters that feed into the stability analysis for each stability Section (see Appendix D-1 and Section 7.3) and the other assessments for the BRDA (see Appendices and Section 7.0).

5.1.4 Critical State Soil Mechanics

The behaviour of tailings under loading is assessed using the critical state soil mechanics conceptual model which is suitable for saturated remoulded soils, such as bauxite residue.

Materials when sheared reach a critical state defined as the state where the soil continues to deform at constant stress and constant void ratio. The critical state void ratio varies with mean effective stress and can be referred to as the Critical State Locus (CSL). This CSL is specific to the soil or tailings material.

The CSL is a semi-logarithmic curve represented by the following equation: $e_c = \Gamma - \lambda_{10} \log(p')$ where:

- e_c is the critical void ratio;
- p' is the mean effective stress; and
- Γ and λ_{10} are intrinsic soil properties, which are independent of the soil density, which represent the y-intercept and slope, respectively.

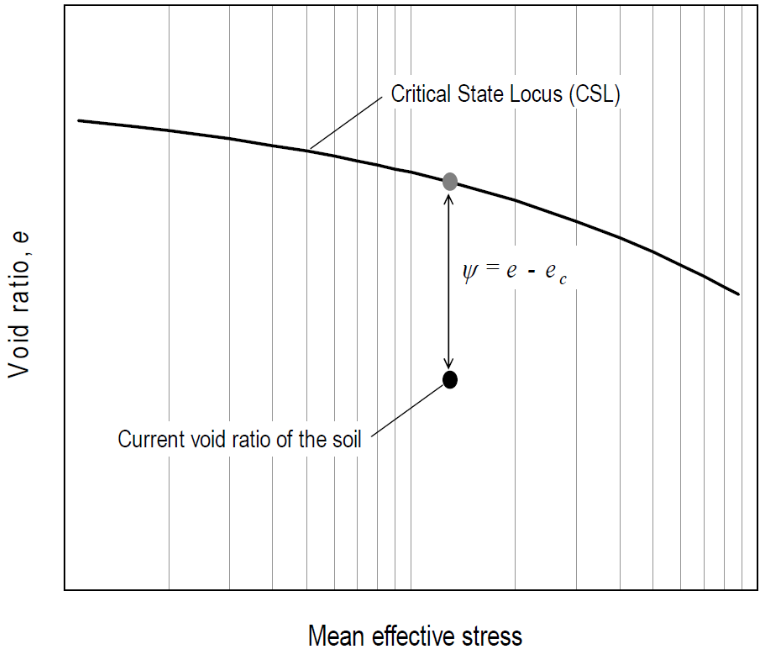

Figure 10: Definition of Critical State Locus and State Parameter (dense dilatant soil)

Critical State Approach: dense soils are strong and dilatant and loose soils are weak and compressive. Soil behaviour is related to its density (or void ratio). The state parameter (Ψ) is density dependent and is the void ratio difference between the current state of the soil and the critical state at the same mean effective stress (p'). Figure 10 below shows $\Psi = e - e_c$, where e = the current void ratio of the soil (in-situ or consolidated void ratio) and e_c is the void ratio on the CSL directly vertically above or below the e value.

Soil constitutive behaviour is related to Ψ:

- Dense dilatant soils have negative Ψ values i.e., e is less than e_c and is below the CSL. These soils can still have potential for liquefaction even though they plot below CSL. Typically require $\Psi < -0.06$ to ensure undrained strength is greater than drained strength.
- Loose contractive soils have positive Ψ i.e., e is greater than e_c and is above the CSL. These soils can have brittle stress-strain behaviour and are susceptible to liquefaction.
- The further the current state is from the CSL the greater the rate of dilation or contraction.

5.2 CPTu Investigation

Cone penetration testing with pore pressure measurement (CPTu) is used to characterize soft soil materials and is particularly suitable for tailings. The results can be used to identify stratigraphy, provide parameters to interpret strength and to assess liquefaction potential.

5.2.1 Stratigraphy

Both the estuarine soils layers and the bauxite residue are suited to CPTu testing, and the first soundings were conducted at the Site in 1989. It has become the principal investigation method for the BRDA since 1999. Samples are taken via the Monster Steek Apparaat (MOSTAP) sampler within holes pushed adjacent to the CPT soundings following an initial assessment of the sounding.

Generally, two layers of estuarine soils are present in the foundation soils, comparable to the findings from the investigation at the adjacent Foynes Harbour (Long 2018).

- **Sandy Silt Layer** – Generally occurs as the surface layer and some underlying layers. Characterised by a higher tip resistance (qt), in the form of spikes, lower pore pressure and higher undrained shear strength as confirmed from DSS testing on Sample from GA18-3A.
- **Silty Clay Layer** – Generally occurs underlying the Sandy Silt layers. Characterised by lower, more uniform tip resistance (qt), higher pore pressure and lower undrained shear strength as confirmed from Direct Simple Shear (DSS) testing on samples from GA18-1A, GA18-5A, GA19 -5A-E2 and GA19-5C-E4.

Two layers of bauxite residue are present:

- **Unfarmed Bauxite Residue** – Present in the Phase 1 BRDA from the base to circa 14 mOD (Stage 7). Characterized by lower and more uniform tip resistance (qt), and lower undrained shear strength as confirmed from triaxial and DSS testing on samples and comparison with shear vane tests.
- **Farmed Bauxite Residue** – Present in the Phase 1 BRDA from circa 14 mOD (Stage 7) upwards and in the Phase 2 BRDA. Characterized by a higher tip resistance (qt), in the form of spikes, as a result of the mud-farming layering effects, and higher undrained shear strength as confirmed from triaxial testing.

5.2.2 Undrained Shear Strength

The undrained strength, s_u ,depends on the effective confining stress (σ'_{v0}) prior to shearing. The CPTu data is used to interpret the undrained shear strength (s_u) using the undrained strength factor, N_{kt}, and the following relationship: **s_u = (Net cone resistance) / N_{kt}**

A **N_{kt} value of 14** has been selected for the bauxite residue following correlation with laboratory testing and review of shear vane testing data.

An **N_{kt} value of 15** is generally used for the estuarine soils, which provides a reasonable estimate of the undrained shear strength ratio profile interpreted from the CPTu when compared to the DSS testing at the equivalent location and from the trend line developed for Irish Clays based on, $\boldsymbol{B_q}$ (Long 2018).

$$\boldsymbol{N_{kt}} = \mathbf{7.82}\ \boldsymbol{B_q}^{\mathbf{-0.65}} \quad \textbf{for Irish Clays.}$$

The B_q value in the silty clay layer varies from 0.3 to 0.45 and returns an N_{kt} value between 13.1 and 17.1. Both the unfarmed and farmed bauxite reside typically demonstrate increased undrained shear strength **s_u** with vertical effective stress **σ'_{v0}**. The estuarine soils contain alternate layers of stronger and weaker material, but a similar profile is evident within the body. The interpreted undrained shear strength profiles for the CPTu soundings are compared to an undrained shear strength ratio (s_u/ σ'_{v0}), which assumes a uniform increase in s_u with vertical effective stress (σ'_{v0}).

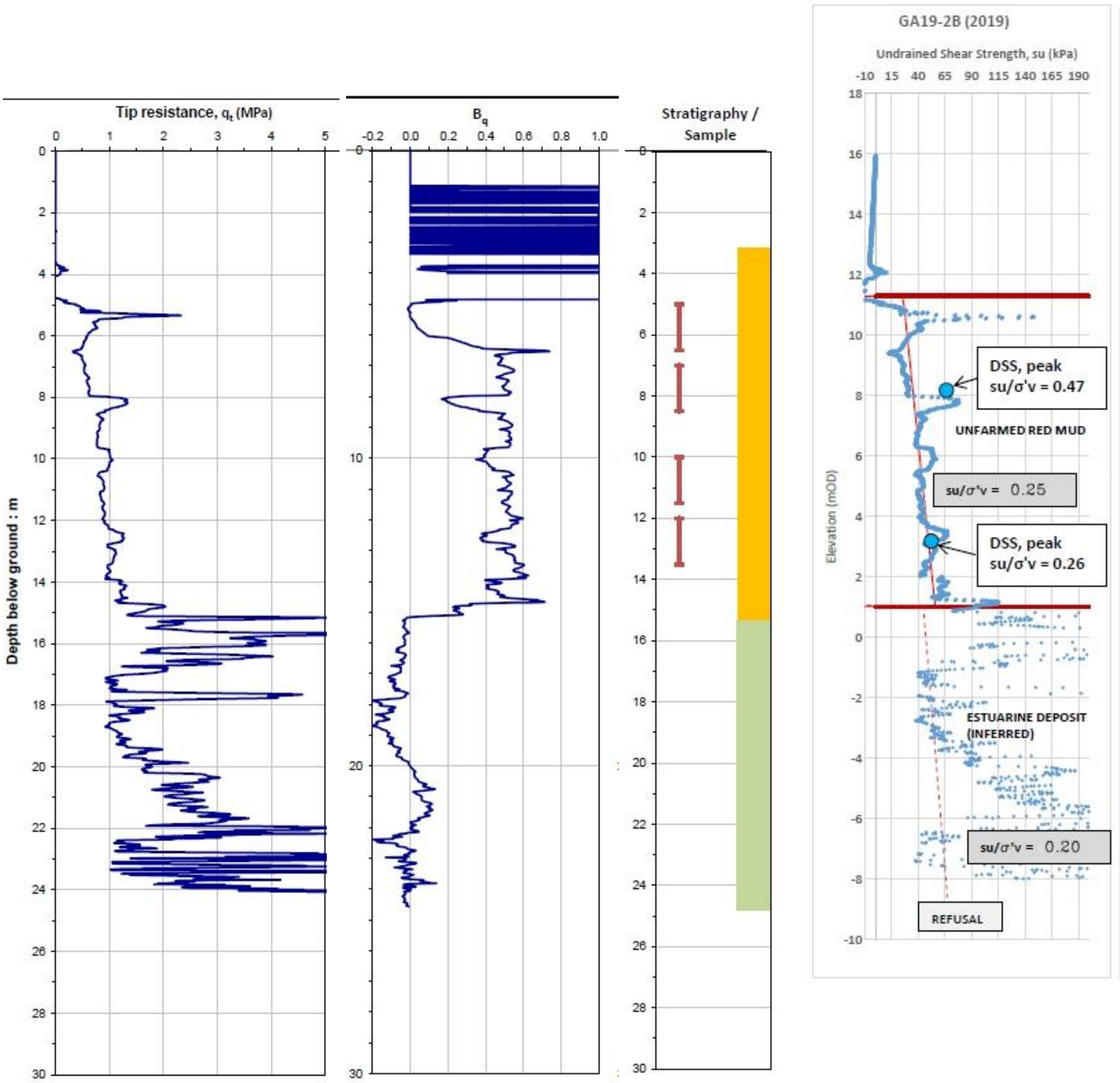

Figure 11: CPTu Interpretation of Undrained Shear Strength: Section F-F, Stage 6

The average $s_u/\ \sigma'_{v0}$ ratios chosen for the unfarmed and farmed bauxite residue and estuarine soils are as follows (see Sections 5.3.5.1 and 5.3.5.3) :

- **Unfarmed Bauxite Residue, $s_u/\ \sigma'_{v0}$ = 0.25**, selected characteristic value based on comparison of interpreted CPTu data with triaxial CIU and DSS laboratory testing and further comparison with shear vane data;
- **Unfarmed Bauxite Residue, $s_u/\ \sigma'_{v0}$ = 0.50 to 0.70 for undrained compression strength ratio,** selected characteristic value based on comparison with 2018 and 2019 triaxial CIU laboratory testing;
- **Farmed Bauxite Residue, $s_u/\ \sigma'_{v0}$ = 0.60**, selected characteristic value based on comparison of interpreted CPTu data with triaxial CIU laboratory testing. Relatively higher tip resistance when compared to the unfarmed bauxite residue; and
- **Estuarine Soils, $s_u/\ \sigma'_{v0}$ = 0.20 to 0.50** - selected characteristic value based on comparison of interpreted CPTu data with triaxial CIU and DSS laboratory testing and further comparison with shear vane data.

Figure 11 above shows a typical CPTu profile sounding and interpretation conducted through Stage 6 at Section F-F. The sounding required the drilling and casing of a borehole through the Stage 6 rock fill to a depth of approx. 4.5m before the piezocone was pushed (from approx. 16 mOD to approx. 11.5 mOD).

The plots from left to right show: tip resistance (q_t) versus depth, the normalized excess pore pressure parameter (B_q) versus depth, the interpreted soil stratigraphy (unfarmed bauxite residue above estuarine soils) and the location of the 4 No. MOSTAP samples taken (vertical red bars).

The image on the right shows the interpreted undrained shear strength, along with selected undrained shear strength ratio (s_u/ σ'_{v0}) profiles for unfarmed bauxite residue and estuarine soils and the results of the DSS tests conducted on samples taken from the MOSTAPs.

5.2.3 Undrained Shear Strength (Shear Vane)

A Geonor Shear Vane was fitted to the CPT rods following pushing in the unfarmed bauxite residue and estuarine soils during various site investigation campaigns from 1989 onwards and rotated to estimated peak and residual shear strengths. Attempts to conducted similar tests in the farmed bauxite residue during the 2018 and 2019 site investigations proved unsuccessful as the farmed bauxite residue material was too stiff.

Note: *A review of previous shear vane testing in bauxite residue has generally shown a relatively large variation in comparison to other methods and is not considered to be sufficient reliable to be used as a calibration assessment. There is also uncertainty in regard to the suitability for use of the shear vane in bauxite residue.*

5.2.3.1 Bauxite Residue

- 48 no. shear vane tests in the unfarmed bauxite residue were carried out during the 2002 site investigation (Glover 2002) and returned peak values ranging from 1 to 26 kPa and residual values ranging from 0 to 3 kPa, for depths ranging from 2m to 12m, and a trend of increasing strength with depth.

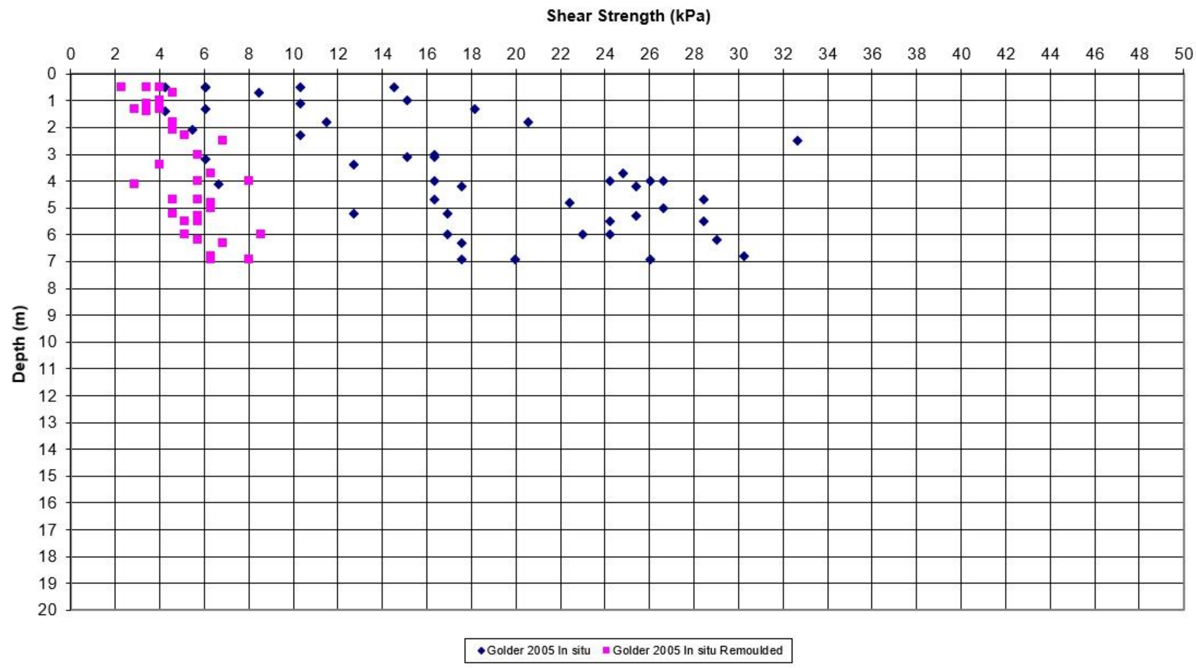

Figure 12: Geonor Shear Vane Testing in Unfarmed Bauxite Residue during 2005 (Golder 2005)

- Figure 12 above shows the 47 no. shear vane test results conducted in the unfarmed bauxite residue during 2005 site investigation (Golder 2011) and returned peak values ranging from 4 kPa to 32 kPa and residual values ranging from 2.0 to 8.0 kPa, for depths ranging from 0m to 7m, and a trend of increasing strength with depth.

- 33 no. shear vane tests in the unfarmed bauxite residue were carried out during the 2007 site investigation (Golder 2007) and returned peak values ranging from 9 kPa to 44 kPa, for depths ranging from 3m to 13.5m, and a trend of increasing strength with depth.
- 4 no. shear vane tests in the unfarmed bauxite residue were carried out at Section K-K (GA18-10A) during the 2018 site investigation (Golder 2018) and returned peak values ranging from 32.5 to 50.5 kPa and residual values ranging from 10 to 11 kPa, for depths ranging from 2m to 5m, and a trend of increasing strength with depth.

5.2.3.2 Estuarine Soils

Figure 13 below shows the results from historic in-situ shear vane and laboratory unconsolidated undrained (UU) triaxial testing. Values range from 15 kPa to 40 kPa below 3m depth i.e., below desiccation. The undrained shear strength ratio shown in the graph is based on a bulk unit weight of 18 kN/m^3 and the ground water level at the surface.

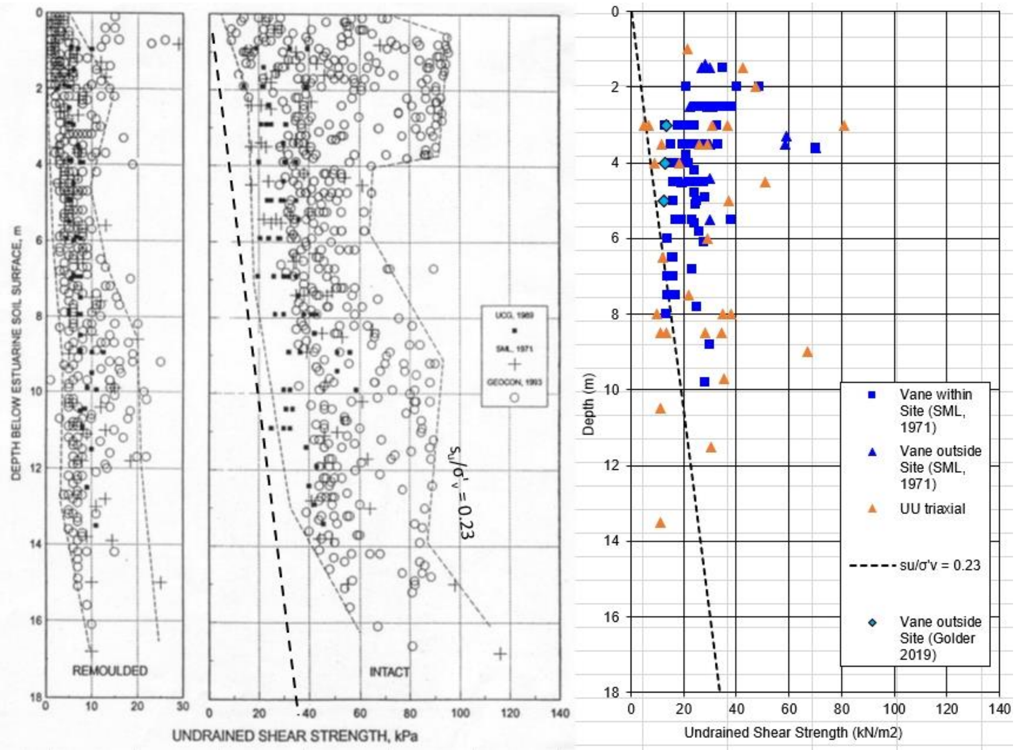

Figure 13: Undrained Shear Strength in Estuarine Soils (Shear Vane and UU Triaxial from 1971 to 2019)

Three (3 no.) Geonor Shear Vane readings were taken in the estuarine soils at GA19-5D during the 2019 CPTu investigation (Golder 2020). The in-situ test locations were downstream toe of the OPW in the north-west sector of the Phase 2 BRDA at 3m, 4m and 5m depths. Natural shear strengths of 13.5, 13.0 and 12.5 kPa were recorded which correspond to interpreted undrained shear strength ratios of 0.27, 0.22 and 0.19, respectively. Based on the depth and the CPTu interpretation, it is considered that these readings were taken in silty CLAY layer. These results are in agreement with previous in-situ shear vane testing conducted within the estuarine soils, and with the results of the investigation at Foynes harbour, which showed a variation in s_u/σ'_{v0} of approximately 0.30 to 0.50 (Long 2018), representing the silty CLAY and the clayey SILT layers, respectively.

5.2.4 Dissipation Testing

Dissipation testing with CPTu profiles measure the decay in pore pressure over a specified amount of time and are used to provide a profile of equilibrium piezometric pressures and help identify flow regimes i.e., no vertical flow is hydrostatic conditions. The results can be used to give an estimate of the horizontal and vertical coefficient of consolidation (c_h and c_v). Dissipation testing has been carried out as part of CPTu site investigations since 1989 and have generally determined hydrostatic conditions in the bauxite residue.

Five (5 No.) dissipation testing were carried out during the 2018 Site Investigation (Golder 2018), 3 No. in the bauxite residue and 2 No. in the estuarine soils, at depths varying between 8.2m and 31.8m from surface, and at three locations, GA18-1C at Section A-A, GA18-3B at Section C-C and GA18-3D at Section C-C.

Table 7: Dissipation Testing for 2018 Site Investigation

Location	Material	Depth (m)	t_{50} (secs)	Findings
GA18-1C (24.87 mOD)	Unfarmed Bauxite Residue	8.19	280	Hydrostatic: c_h = 11.6 m^2/year, c_v = 8.9 m^2/year
GA18-1C (24.87 mOD)	Estuarine	31.84	290	Downwards: c_h = 8.7 m^2/year, c_v = 6.7 m^2/year
GA18-3B (24.2 mOD)	Unfarmed Bauxite Residue	9.99	255	Hydrostatic: c_h = 19.7 m^2/year, c_v = 15.1 m^2/year
GA18-3B (24.2 mOD)	Unfarmed Bauxite Residue	19.96	97	Hydrostatic / Downwards: c_h = 37.7 m^2/year, c_v = 29.0 m^2/year
GA18-3D (23.6 mOD)	Estuarine	21.86	40	Downwards: c_h = 109 m^2/year, c_v = 84 m^2/year

The dissipation tests conducted at in the unfarmed bauxite residue at the two locations and at various depths indicate hydrostatic conditions. The c_v values returned for the dissipation tests correspond to the range of laboratory test results for unfarmed bauxite residue (Golder 2011).

The dissipation tests in the estuarine at both locations indicate downward flow but is more pronounced at Section C-C (GA18-3D) where the depth of estuarine below the test depth is less. The CPT profiles indicate only a nominal depth of estuarine soils at GA18-3D (≈ 2m) and the presence of an internal process sand road, whilst there is ≈ 12m depth of estuarine soils present at GA18-1C on Section A-A.

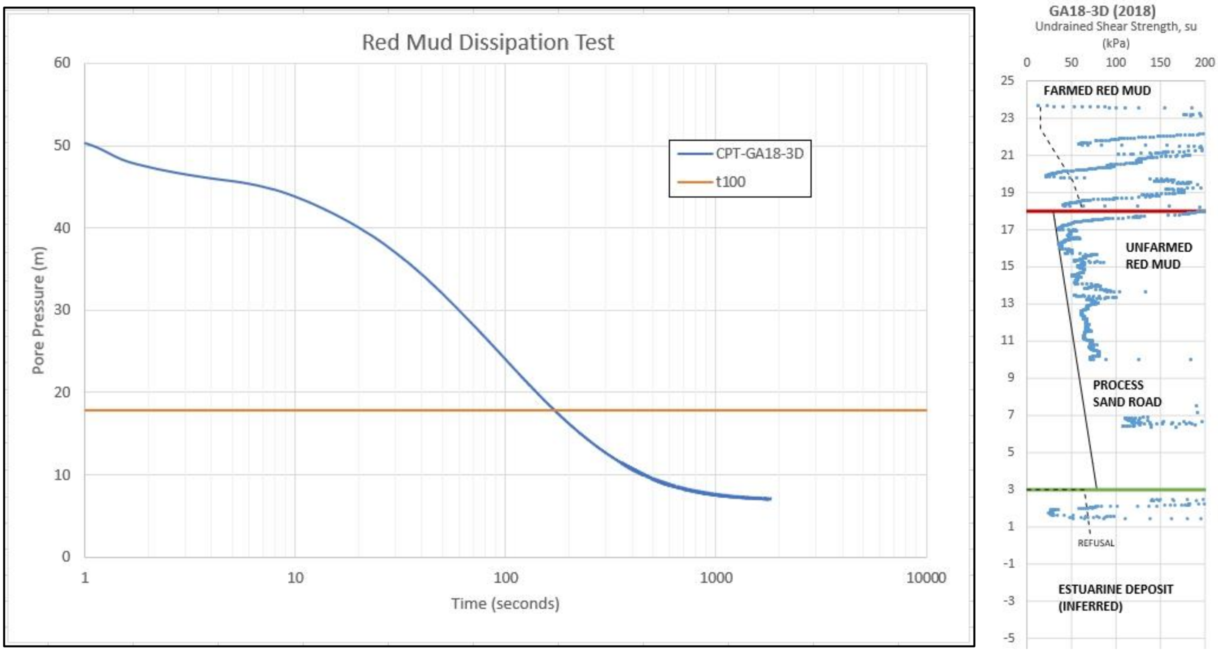

Figure 14: Dissipation Test at GA18-3D at 21.86m depth and Interpreted CPTu Sounding from Stage 10 (≈ 24 mOD)

5.2.5 Liquefaction Potential

The interpreted CPTu data can provide a potential for liquefaction assessment based on the relationship between the Friction Ratio (F), the normalized tip resistance (Q) and the excess pore pressure parameter (B_q), (Shuttle & Cunning 2008). A typical CPTu profile for the Phase 1 BRDA from Stage 10 is shown in Figure 15 below and shows that the bulk of the material is below the threshold line, indicating post-liquefaction strain hardening behaviour.

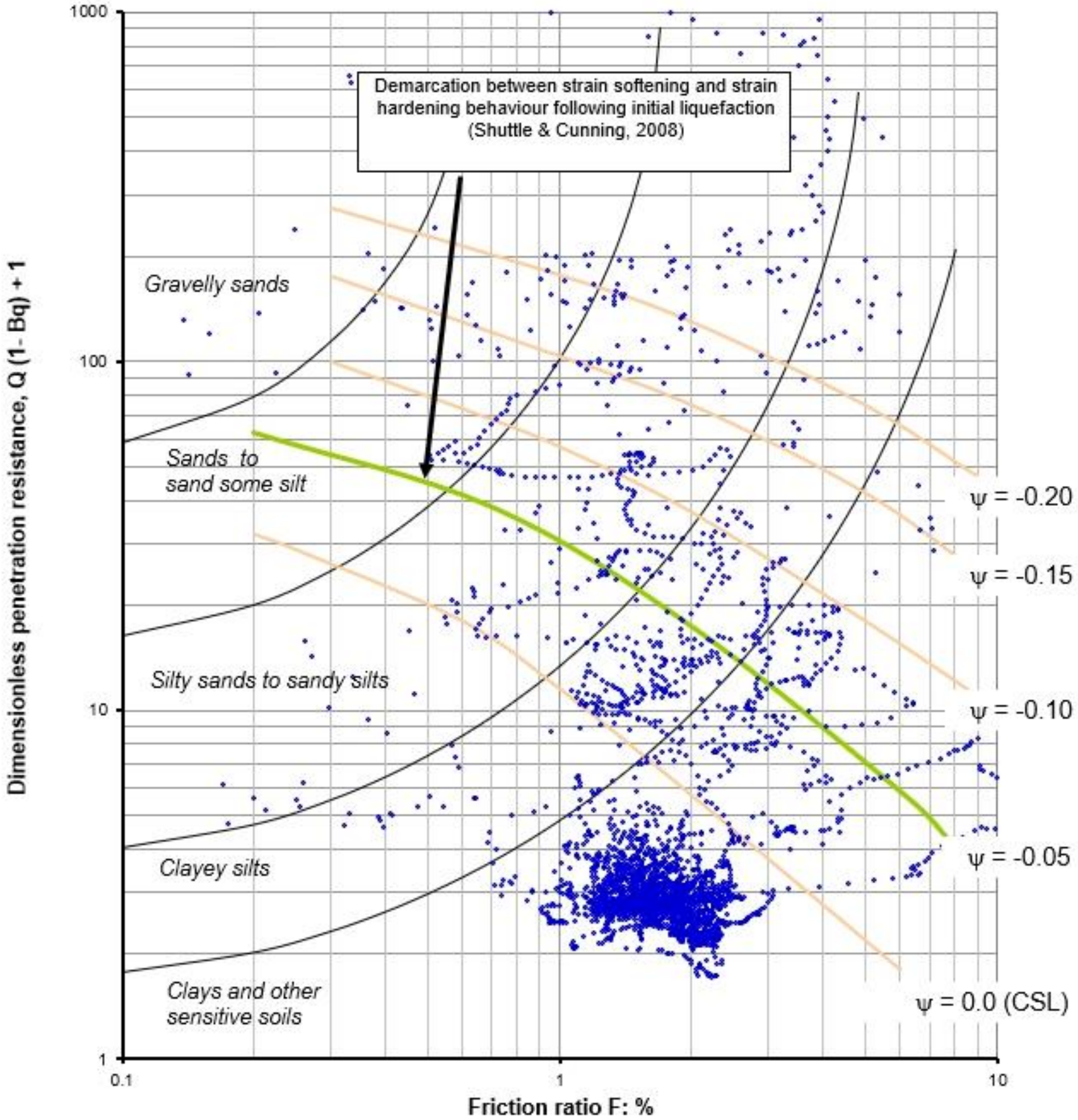

Figure 15: Interpreted CPTu data for Liquefaction Potential at GA18-2B (Stage 10 at Section B-B)

Tailings deposited by hydraulic methods are typically assessed for liquefaction potential by determination of the state parameter (**Ψ**) for the material. Soil constitutive behaviour is related to Ψ:

- Dense dilatant soils have negative Ψ values and strain hardening behaviour. Typically require Ψ < -0.06 to ensure undrained strength is greater than drained strength and soils are not susceptible to liquefaction.
- Loose contractive soils have positive Ψ and strain softening behaviour. These soils can have brittle stress-strain behaviour (residual strength < 70% of peak strength) and are susceptible to liquefaction.

The state parameter can be determined in two ways:

- Triaxial testing of samples that are consolidated to their in-situ conditions and comparison of the consolidated void ratio with the determined Critical Stage Locus (CSL), see Section 5.3.5.2 ; and
- The in-situ state parameter (Ψ) can be interpreted from CPT data, for undrained cone penetration, based on the following relationship: $\boldsymbol{Q_p(1-B_q)+1 = k\,\mathbf{exp}(-m\Psi)}$ where:
  - Q_p = Tip resistance normalized by vertical effect stress = $\frac{(q_t - \sigma_{vo})}{\sigma'_{vo}}$, Q_p = Q for K_o = 1
  - B_q = Normalized excess pore pressure = $\frac{(u-u_o)}{(q_t-\sigma_{vo})}$
  - *k* and *m* were selected based on the interpreted relationship from triaxial testing on farmed and unfarmed bauxite residue is shown in Figure 16 below, (k = 2.2 and m – 5.5). (Golder 2020)

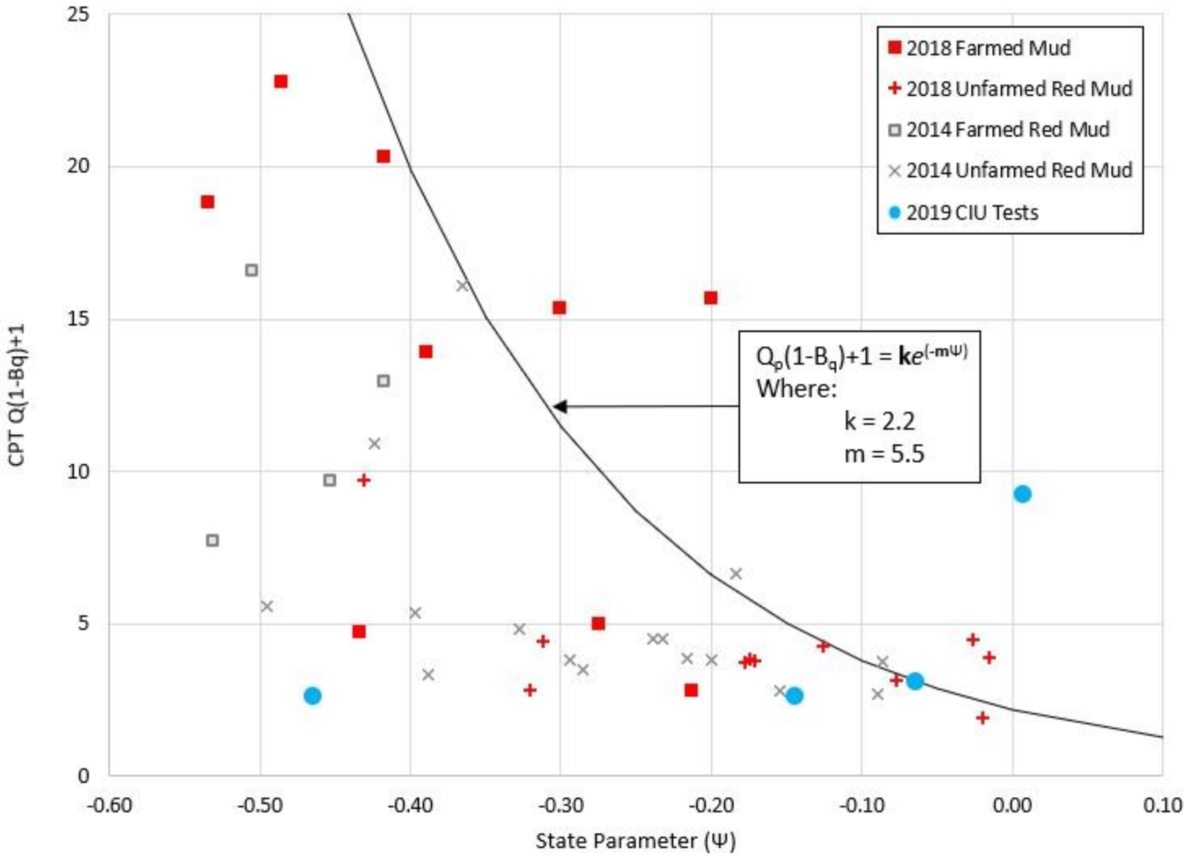

Figure 16: Relationship between CPTu data and State Parameter

The variation in state parameter with depth for various CPTu soundings conducted in 2018 and 2019, and based on the above relationship, is shown in Figure 17 below.

A state parameter of -0.05 (shown as red dashed line in Figure 17) is the typical demarcation between strain softening and strain hardening following initial liquefaction.

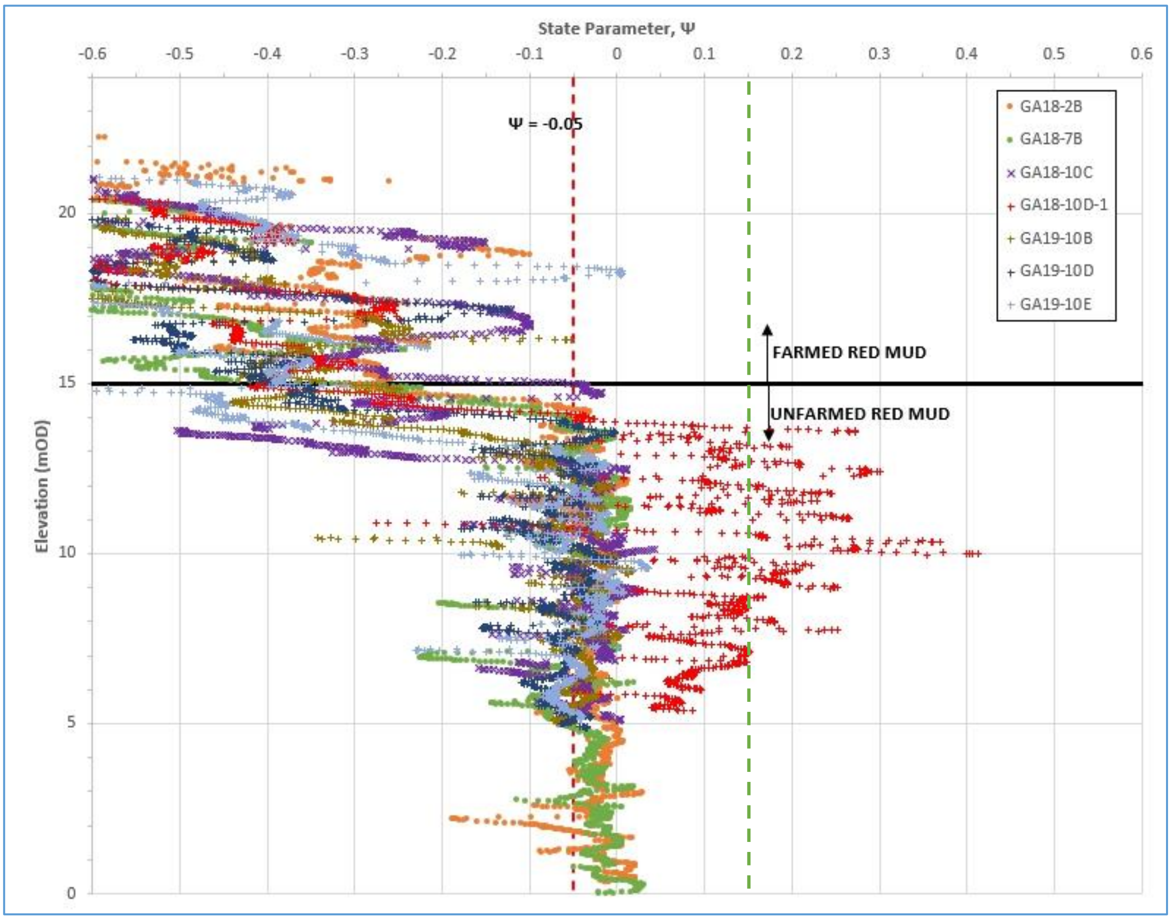

Figure 17: Variation in State Parameter with Depth for AAL Bauxite Residue

The farmed bauxite residue generally has a state parameter less than -0.05, indicating strain hardening.

The unfarmed bauxite residue generally has a state parameter between -0.05 and 0.05 (shown as green dashed line in Figure 17), indicating potential strain softening. If potential strain softening occurs by a disturbance in the unfarmed bauxite residue allowing dilation, the state parameter shows that the unfarmed bauxite residue has potential for flow liquefaction.

The screening method for the liquefaction susceptibility of soils (Bray and Sancio 2006) based on moisture content (Mc), Liquid Limit (LL) and Plasticity Index (PI), combined with the low level of seismicity in Ireland indicates that the estuarine soils are <u>not</u> susceptible to liquefaction (Golder 2018).

5.2.6 Seismic Parameters

In-situ seismic CPTu were also conducted at two locations during the 2018 site investigation (Golder 2018). Shear wave velocity (V_s) was measured at approximately 1m depth intervals and was used to interpret the small strain shear modulus (G_{max}) and was plotted against vertical effective confining stress, $\boldsymbol{\sigma'_v}$.

The CPTu tip resistance (q_t) has been calibrated to the G_{max} calculated from the shear wave velocity measurements, to allow determined of the G_{max} profile with depth at the other CPT locations where no shear wave velocity measurements were undertaken. Unfarmed bauxite residue provided a better correlation than farmed bauxite residue due to its smooth tip resistance profile.

Following comparison with bender element testing at various confining stresses, the relationship between G_{max} and vertical effective confining stress, σ'_v, was established to be $Gmax = 80(\sigma'_v / 100)^{0.5}$, with G_{max} varying between 35 MPa and 190 MPa (see Figure 28 in Section 5.3.7).

5.2.7 Drained Strength (Effective Friction Angle)

The variation in effective friction angle with depth for various CPTu soundings conducted during 2018 and 2019 and based on the relationship determined for the variation in stress ratio (**η_{max}**) with the critical stress ratio or critical friction ratio (**M_{tc}**) and the state parameter (**Ψ**) (Golder 2018).

$$\boldsymbol{\eta_{max} = M_{tc} - 0.2\Psi}$$

The corresponding effective friction angle for the average **η_{max}** value of 1.50 is approximately 37 degrees and for the lower bound **η_{max}** value of 1.29 is approximately 32 degrees.

An effective friction angle of 32 degrees (shown as red dashed line in Figure 18) was the value used in previous stability analyses for both the farmed and unfarmed bauxite residue which resulted from triaxial testing conducted on unfarmed bauxite residue in 2004 and 2005.

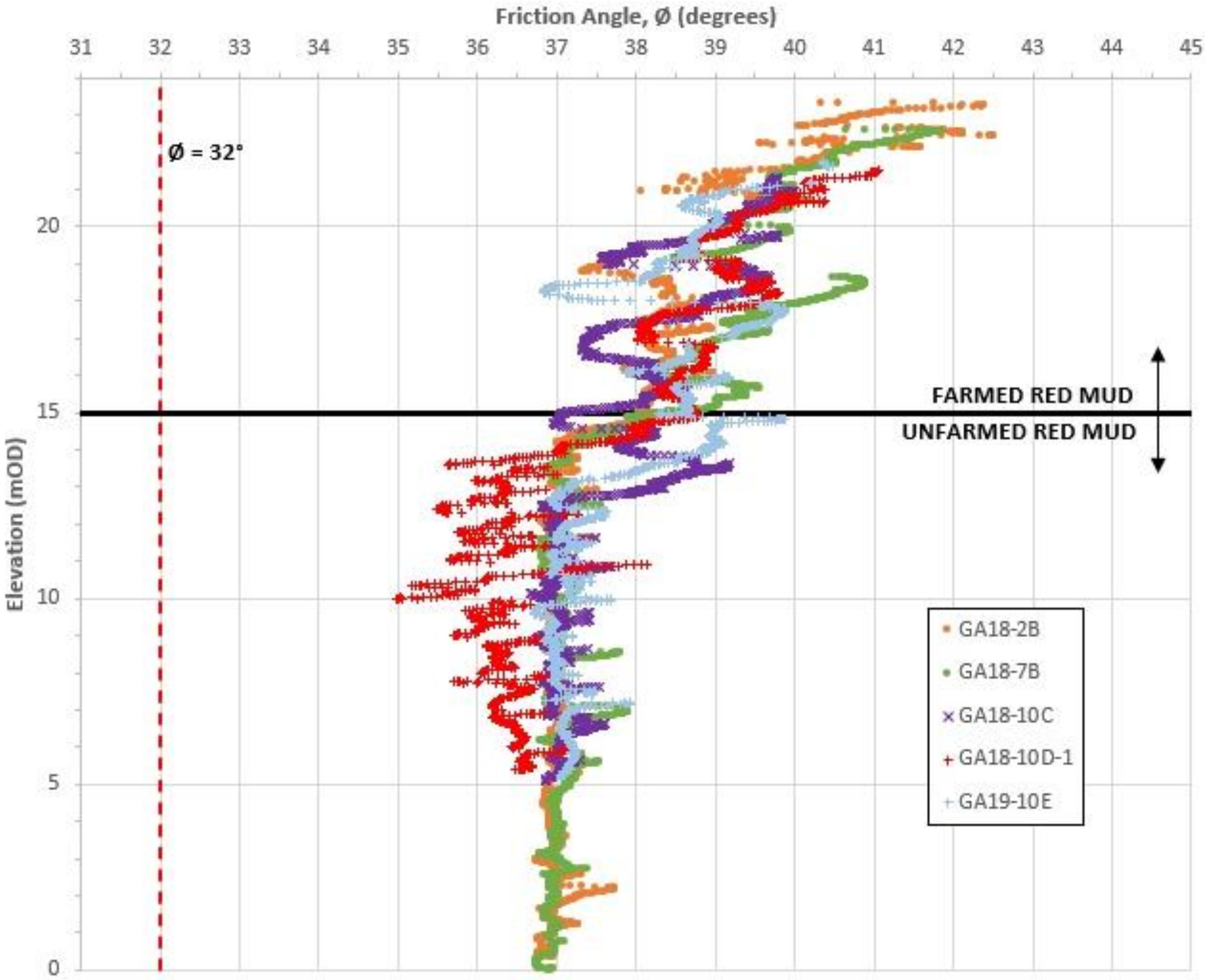

Figure 18: Variation in Effective Friction Angle (Ø') with depth for Bauxite Residue and Estuarine Soils

The CPTu interpretation of the effective friction angle shows that this 32-degree value is exceeded in all instances. The average effective friction angle of the unfarmed bauxite residue is estimated to be 37 degrees and the average friction angle of the farmed bauxite residue is estimated to be 39 degrees, based on the 2018 and 2019 CPTu data.

The average effective friction angle of the estuarine soils is estimated to be 37 degrees (lower portion of graph).

5.3 Laboratory Testing

Samples were extracted from test pits, augered holes, boreholes, MOSTAPs and/or bulk material in the footprint of the BRDA for the purpose of laboratory testing during the various site investigation campaigns listed in Table 6 from 1971 to 2019.

Laboratory testing methods were selected to determine parameters for use in analysis and design. The prior data has been reviewed at various intervals, typically on the lead-up to a change of the development and following significant site investigations, and the characteristic values for the various parameters are fine-tuned.

The following laboratory test methods were utilized to assess the engineering properties of the estuarine soils, tills and the bauxite residue in the footprint of the BRDA.

5.3.1 Classification Testing

5.3.1.1 Bauxite Residue and Estuarine Soils

Classification or Index Testing methods provide the fundamental parameters to identify soils and to assess soils in their current state and potential for engineering purposes and included: Particle Size Distribution, Moisture Content, Amorphous Water Content, Atterberg Limits, Bulk Density and Dry Density and Particle Density, which were carried out to the relevant British Standard Methods at the time.

Table 1 provides a summary of the characteristic values for the various geotechnical parameters determined for bauxite residue and the estuarine soils.

5.3.1.2 Till

Similar tests were undertaken for the till materials in the footprint of the BRDA and from local borrow sources to assess their suitability as construction materials. The properties of the in-situ till material are summarized below:

- Variable material, generally classified as a gravelly, clayey sandy SILT / silty SAND.
- Clay content between 8% and 10%, silt content between 20% and 25%, sand content between 25% and 30% and gravels and cobbles between 30% and 50%.
- Liquid Limits between 10% and 33%. Plastic Limits between 10% and 20%.
- Moisture Content between 7% and 13%.
- Stiff to Very Stiff (SPT N values of 30 to 40)

5.3.2 Mineralogical Testing

Mineralogical analysis is used to determine the mineral composition and mineral structure to aid the understanding of the material characteristics and geotechnical parameters.

- The primary test methods utilized to assess the bauxite residue, process sand and the estuarine soils included X-Ray Diffraction (XRD), Scanning Electron Microscopy (SEM) with associated energy dispersive micro analysis (EDA), Optical Microscopy and Petrographic Analysis.

  The findings for the bauxite reside and process sand are discussed in Section 4.0.

- Testing on three samples of estuarine soils with a plasticity index (PI) range of 6% to 35%, indicated quartz content between 38% and 55%, calcite between 18% and 20%, kaolinite between 5% and 10% and illite between 18% and 33% (Golder 2005A, Golder 2005B). The lower PI samples returned higher quartz content and lower illite content.

5.3.3 Consolidation Testing

5.3.3.1 Bauxite Residue

Section 4.3.1 discuss the results of the consolidation testing for farmed and unfarmed bauxite residue.

Two (2) No. one-dimensional consolidation (oedometer) tests were undertaken on selected samples, taken from the MOSTAP tubes and assessed to be intact, homogenous and visually undisturbed portions, of farmed bauxite residue samples taken from the Phase 2 BRDA (Golder 2020). The 6-stage oedometer loading increment was selected based on the potential height of the BRDA to Stage 16 and the samples were tested to a maximum final stress of 1,600 kPa.

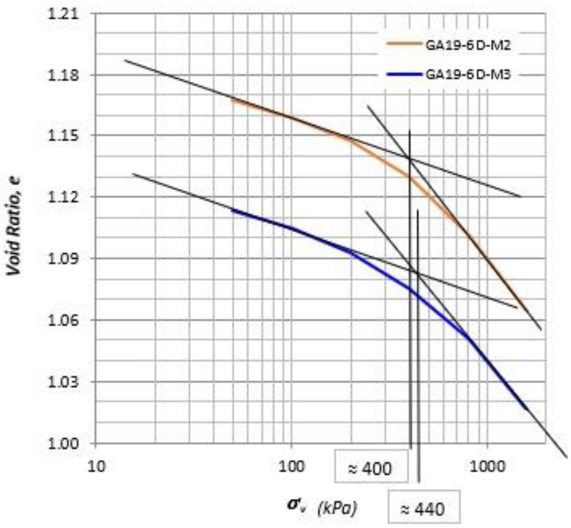

Figure 19: 1-D Consolidation Testing on Farmed Bauxite Residue Samples (Golder 2020)

The estimated existing overburden pressures (σ'_{vo}) for both samples are significantly less than estimated pre-consolidation pressure; 60 kPa and 400 kPa for GA19-6D-M2 and 145 kPa and 440 kPa for GA19-6D-M3. This apparent over consolidation is the result of the bauxite residue farming.

One-dimensional consolidation (oedometer) testing of the unfarmed bauxite residue has been undertaken on a number of site investigation campaigns (Delft 1988, Golder 2005A, Golder 2011)

5.3.3.2 Estuarine Soils

Three (3) No. one-dimensional consolidation (oedometer) tests were undertaken on selected samples, taken from the MOSTAP tubes and assessed to be intact, homogenous and visually undisturbed portions, of estuarine soils on taken from the downstream toe of the outer perimeter wall (OPW) of the PIC at the north-west sector of the Phase 2 BRDA (Golder 2019).

The 6-stage oedometer loading increment was selected based on the potential height of the BRDA to Stage 16 and the samples were tested to a maximum final stress of 1,600 kPa. Two of the samples were assessed to be from the Silty CLAY layer and one of the samples was assessed to be from the Sandy SILT layer.

The OCR was determined to be 2.3 and 3.3 for the silty CLAY layer and 4.4 for the upper clayey SILT layer, which may reflect a level of desiccation in this upper layer. Testing by others conducted on the estuarine soils at the nearby Foynes Harbour measured OCRs of up to 6 near surface and reducing to normally consolidated at 14m depth (Long 2018).

The initial settlement in the estuarine soils beneath the BRDA can be expected to be low but, as the BRDA increases in height and beyond the pre-consolidation pressure, depending on the rate of rise, greater settlements may be experienced, corresponding to the value of the Compression Index (C_c). This increase in settlement and transition to a normally consolidation material can be expected to occur once the depth of bauxite residue exceeds 12m depth i.e., above Stage 4.

- The interpreted m_v values indicate that the clayey SILT layer has low to medium compressibility (m_v in the range of 0.025 m^2/MN to 0.19 m^2/MN) and that the silty CLAY layer has medium to high compressibility (m_v in the range of 0.045 m^2/MN to 0.47 m^2/MN).
- The interpreted c_v values indicate that the clayey SILT layer has medium duration for consolidation (10 years to 30 years with c_v values in the range of 11.0 to 30.0 m^2/year) while the silty CLAY layer has medium to high duration for consolidation (10 to 100+ years with c_v values in the range of 2.5 to 19.0 m^2/year)

Previous consolidation testing on the estuarine soils returned values for m_v in the range of 0.06 m^2/MN to 1.09 m^2/MN and values for c_v in the range of 8.7 to 12.5 m^2/year under loads of 100 to 300 kPa (Golder 2005B).

5.3.4 Hydraulic Conductivity Testing

5.3.4.1 Bauxite Residue

Section 4.3.1 discuss the results of the hydraulic conductivity testing for unfarmed and farmed bauxite residue.

- Testing has been undertaken both in-situ and in the laboratory for the unfarmed bauxite residue. The laboratory testing (Falling Head Method) for 18 No. tests returned values ranging between 1.5 x 10^{-10} m/s and 8 x 10^{-9} m/s (average of circa 1.5 x 10^{-9} m/s) (Delft 1988).
- In-situ testing (URS 2002) indicated values between 4.7 x 10^{-9} m/s and 5.6 x 10^{-8} m/s, at generally below a depth of 3 m.
- Laboratory tests conducted (Falling Head Method) for 11 No. test returned values ranging between 5.0 x 10^{-5} m/s and 1 x 10^{-9} m/s (average of circa 5.2 x 10^{-6} m/s) (URS 2002).
- Golder 2016 conducted testing on three farmed bauxite residue core samples, using a Flexible Wall Triaxial Permeameter Test conducted to ASTM D 5084, Method C - Falling Head with Increasing Tail Water Pressure, and returned values ranging from 8.5 x 10^{-9} m/s to 3.7 x 10^{-8} m/s, with a characteristic value of 1.9 x 10^{-8} m/s is selected for k_v.
- Golder 2021B conducted testing on two amended bauxite core samples, using the Permeability in a Triaxial Cell Method BS 1377:Part 6:1990 Clause 6 – Constant Head, and returned values ranging from 1.04 x 10^{-10} m/s to 9.77 x 10^{-11} m/s.

5.3.4.2 Estuarine Soils

There has been no recent permeability testing of the estuarine soils and the results available date from pre-2004 and are indicative of the in-situ conditions prior to the Phase 1 BRDA development.

Laboratory testing on undisturbed samples returned permeabilities ranging from 1.0 x 10^{-7} to 1.0 x 10^{-10} m/s whilst in-situ testing returned permeabilities ranging from 5.0 x 10^{-5} to 5.0 x 10^{-9} m/s. Recompacted samples on estuarine returned permeabilities ranging from 2.1 x 10^{-9} to 2.5 x 10^{-10} m/s.

5.3.4.3 Till

Golder 2005B conducted permeability testing on screened and recompacted till material to 95% of the Modified Compaction Proctor and returned permeabilities ranging from 2.2 x 10^{-9} to 4.7 x 10^{-10} m/s.

Previous in-situ testing at 12 locations returned permeabilities ranging from 5.5 x 10^{-9} to 3.2 x 10^{-11} m/s. Results at 2 locations returned values ≈ 5 x 10^{-6} m/s.

5.3.5 Strength Testing

The strength properties of the bauxite residue and estuarine soils have been interpreted as either drained or undrained:

- The drained (effective stress) parameter, expressed as a friction angle (∅'), represents the response to shearing under drained conditions with no excess pore pressure build-up.
- The undrained (total stress) parameter, expressed as an undrained shear strength ratio (s_u/σ'_{v0}), represents a condition where excess pore pressure has built up within the material. This condition may result due to a number of conditions, which includes rate of rise leading to pore pressure build-up that exceeds the rate of pore pressure dissipation.

5.3.5.1 Bauxite Residue

Farmed and unfarmed bauxite residue samples, assessed to be intact, homogenous and visually undisturbed, and remoulded samples to target specific void ratios were tested by:

- Triaxial testing:
  - Isotropically Consolidated Undrained (CIU)
  - Isotropically Consolidated Drained (CID)
  - Quick CIU
  - Undrained Unconsolidated (UU)
- Consolidated Undrained Direct Simple Shear (DSS)

typically to a maximum strain of 20% to assess the strength at higher strains, the reduction in the shear strength ratio and the behaviour of the bauxite residue after peak failure, which typically occurs at <10% strain.

Notes:

1. *The tests were conducted with triaxial cells which had steel platens; bauxite residue interaction with aluminium platens can result in gas generation during testing which can make the test data unreliable.*
2. *Undrained unconsolidated (UU) triaxial tests have not been included in recent site investigations and have been included in correlations and interpretations as the results have demonstrated large variations and it is considered that the effects of sample disturbance generally produce lower shear strength results.*
3. *Quick CIU Tests are only used for a comparison of undrained shear strength. Due to the rapid rate of loading, the pore pressure readings may not represent the pressure throughout the sample.*

Figure 20 below shows the plots for Deviator Stress versus Axial Strain of CIU and CID tests conducted in the 2018 Site Investigation (Golder 2018). The samples are mixture of undisturbed and remoulded samples at initial void ratios ranging between 0.85 and 1.29 and at consolidation pressures ranging from 200 kPa to 1,700 kPa.

- The tests generally show a defined peak deviator stress followed by a reduction, sometimes but not always to a steady state.
- Peak strengths varied between 150 kPa and 1,900 kPa, undrained strength ratios varied between 0.36 and 0.87 and effective friction angles varied between 35.6 and 49.4 degrees.
- The CID tests showed compression during shearing where the consolidation stress exceeded 250 kPa, except for samples which were remoulded to an initial relatively loose state.

- The CIU tests all showed an increase in pore pressure during shearing with the pore pressure remaining relatively constant once the peak deviator stress was reached.
- There is also no significant reduction in deviator stress once the peak is reached, which indicates that no significant strain softening is occurring, and hence no indication of brittle behaviour (residual strength < 70% of peak strength). Test CIU_GA18-2B conducted at a high confining stress (1,500 kPa), and prepared to an initial void ratio of 0.98, showed a drop in deviator stress following the peak. The undrained shear strength ratio (s_u/σ'_{v0}) varied from a peak of 0.63 at 7% strain to 0.39 at 18% strain.
- The end points of the test stress paths corresponded to a stress ratio η (=q/p′) that lies in the range 1.48 < M < 2.03 and where M is called the critical stress ratio and denotes the end condition.

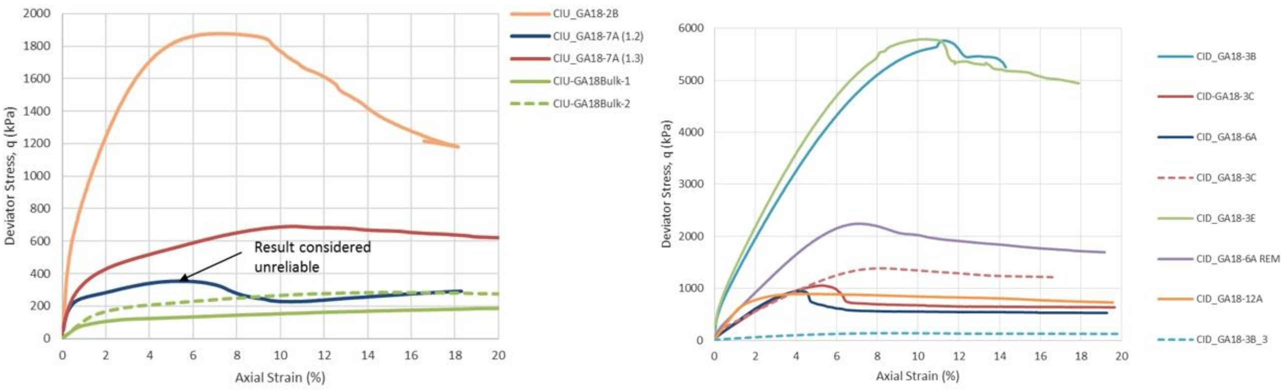

Figure 20: Triaxial Testing (CIU and CID) on Bauxite Residue Samples (Golder 2018)

Figure 21 below shows the plots for Shear Stress versus Shear Strain and Undrained Strength Ratio versus Shear Strain for DSS tests conducted in the 2019 Investigation (Golder 2020). The samples are undisturbed unfarmed bauxite residue samples taken at depths ranging from 7m to 13.5m from the crest of Stage 6 at Section F-F. Initial void ratios ranged between 1.14 and 1.49 and consolidation pressures ranged from 300 kPa to 430 kPa (to replicate in-situ conditions).

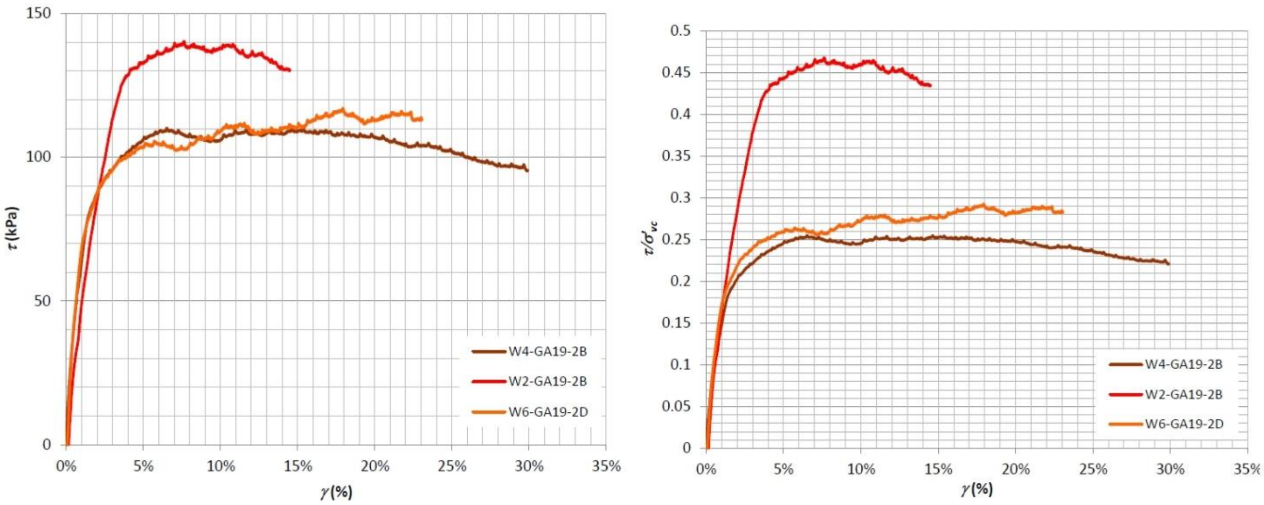

Figure 21: DSS Testing on Unfarmed Bauxite Residue Samples (Golder 2020)

- Peak strengths varied between 105 and 140 kPa.
- Undrained strength ratios varied between 0.26 and 0.47 at peak and 0.22 and 0.43 at residual.

- The DSS tests generally show a defined peak shear stress followed by little change in stress as the strain increases to > 20% and hence no indication of brittle behaviour (residual strength < 70% of peak strength).
- Two tests conducted on remoulded samples during the 2018 Site Investigation (Golder 2018) returned similar values for peak and residual undrained strength ratios, 0.26 to 0.26 and 0.28 and 0.31, respectively, with both tests showing strain hardening characteristics. Void ratios ranged from 1.11 to 1.15 and consolidation pressures ranged from 150 kPa to 250 kPa.

The undrained shear strength ratios (s_u/ σ'_{v0}) determined from the CIU Triaxial tests DSS Tests conducted in 2018 and 2019 are shown in Figure 22, plotted against their interpreted state parameter (**Ψ**) value based on the current CSL parameters.

The minimum undrained strength ratio line is primarily developed for compression strength obtained from the CIU triaxial tests. The trend of decreasing undrained shear strength ratio with increasing state parameter is evident up to a state parameter **(Ψ)** of approximately 0.05. Where **Ψ** > 0.05, the undrained strength ratio for both compression and shear are the same. The Quick Triaxial CIU tests show a higher undrained shear strength that is believed to be due the quicker rate of shearing, as has been reported from testing on cohesionless soil (Yamauro et. al. 2011).

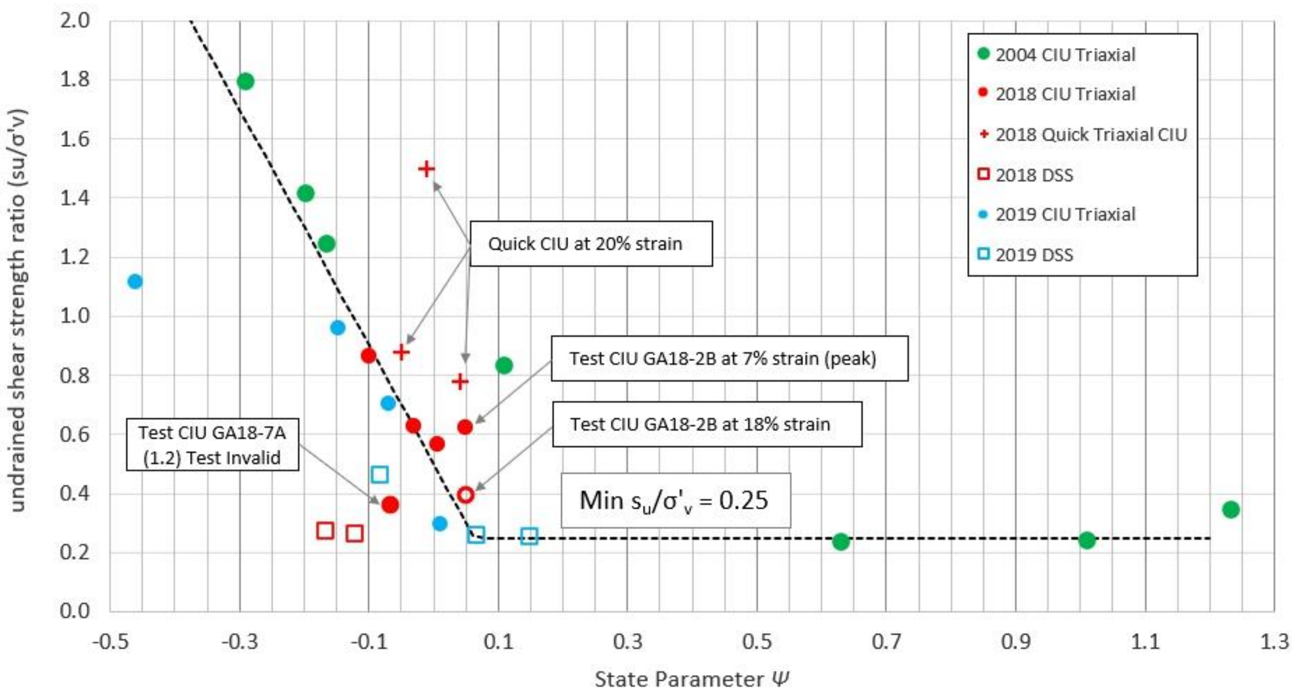

Figure 22: Bauxite Residue Undrained Shear Strength (su/σ′$_{vo}$) vs State Parameter (Ψ)

Notes:

1. *Mayne 2016 suggests that the DSS test is the most appropriate test to use when correlating the interpreted undrained strength from CPTu data as it presents s_u results that fall more-or-less mid-way between the other test modes (compression and extension) and thus provides an 'average' result.*
2. *The undrained shear strength, s_u, is not a unique soil property. It varies depending on the mode of failure, the stress state of the soil, anisotropic effects and rate of failure. Furthermore, in situ tests may not necessarily be fully drained, as may be the case for silty soils, and so not truly represent undrained shear strength.*

The 2018 and 2019 Triaxial CIU show variability with state parameter but have minimum peaks values for s_u/σ'_{vo} value of 0.57 and 0.71, respectively. A s_u/σ'_{vo} range of 0.50 to 0.70 has been selected for the undrained compression strength ratio of unfarmed bauxite residue.

The results of the CIU triaxial and DSS testing indicate that undrained strength anisotropy (variation in undrained strength) with direction of shearing exists for the bauxite residue. DSS testing conducted on unfarmed bauxite residue samples in 2018 and 2019 has returned s_u/σ'_{v0} values ranging from 0.26 to 0.28. A s_u/σ'_{vo} range of 0.20 to 0.25 has been selected for the undrained shear strength ratio of unfarmed bauxite residue. This equates to a total stress frictional angle (∅) of 14 degrees, which is approximately 40% of the previously determined unfarmed bauxite residue effective frictional angle (32 degrees) and represents excess pore pressure generation in the bauxite residue due to its contractive state. At very high state parameter (void ratio) values i.e., Ψ = 1.0 , the undrained compression strength ratio reduces to 0.25. There is no area within the BRDA where the bauxite residue is present at this high a void ratio or state parameter

A range of minimum undrained shear strengths of between 10 kPa and 25 kPa are selected based on the relevant CPTu data for each stability section which represents the 20th percentile observed. Typically, 15 kPa is used.

The characteristic s_u/σ'_{v0} of 0.60 for farmed bauxite residue equates to a total stress frictional angle (∅) of 31 degrees which is approximately equal to the previously determined unfarmed bauxite residue effective frictional angle (32 degrees). The farmed bauxite residue generally plots above the s_u/σ'_{v0} = 0.6 line, indicating a greater undrained shear strength that its effective shear strength and is reflective of its dilatant behaviour. A minimum undrained shear strength of 25 kPa is selected and is based on the 20th percentile of interpreted values.

5.3.5.2 Critical State Locus

An AAL bauxite residue bulk sample and corresponding process water was sent to the Golder Portugal Geotechnical Laboratory in Porto in order to be characterized through advanced geotechnical testes basing on the Critical State Soil Mechanics framework, namely the determination of the CSL (Golder 2021E).

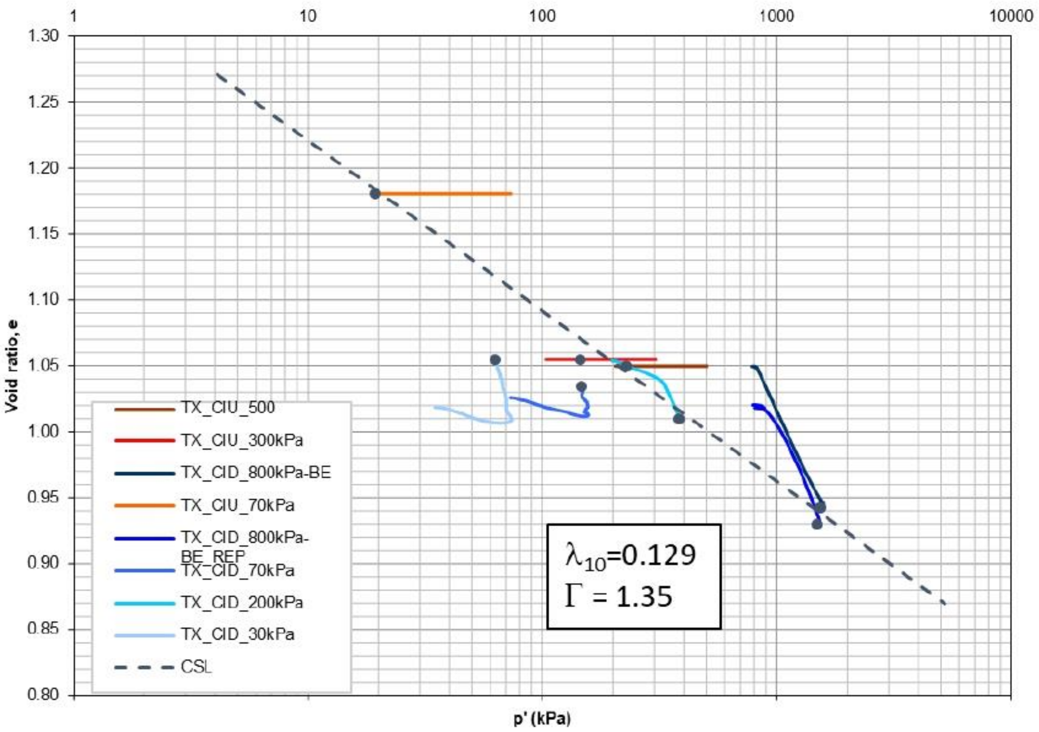

Figure 23: CSL for AAL BRDA Bauxite Residue (Golder 2021E)

Figure 23 above shows that the CSL is well defined by the final points of the tests for the range of interest of the mean effective stress i.e., 100 kPa to 800 kPa, and for the typical in-situ void ratio range of 1.05 to 1.25 for unfarmed bauxite residue and 0.99 to 1.13 for farmed bauxite residue.

The specimens were prepared to target loose and dense void ratios, were moulded using the moist tamped under-compaction technique (Ladd 1978) and tested in a triaxial chamber.

After saturation, the specimens were consolidated isotropically and sheared under drained and undrained conditions i.e., CID and CIU triaxial tests, at varying effective confining stress, to a minimum of 20% strain.

The sample freezing technique (Sladen and Handford 1987) was applied to the specimens at the end of each test in order to accurately determine the final volumes and void ratios.

5.3.5.3 Estuarine Soils

Select estuarine samples, assessed to be intact, homogenous and visually undisturbed were tested by:

- Triaxial testing:
  - Isotropically Consolidated Undrained (CIU)
  - Quick CIU
- Consolidated Undrained Direct Simple Shear (DSS)

typically to a maximum strain of 20% to assess the strength at higher strains, the reduction in the shear strength ratio and the behaviour of the bauxite residue after peak failure, which typically occurs at <15% strain.

Figure 24 below shows the plots for Deviator Stress versus Axial Strain and Pore Water Pressure versus Axial Strain for two CIU tests conducted in the 2019 Site Investigation (Golder 2020). The undisturbed samples were taken from the downstream toe of the OPW at the north-west sector of the Phase 2 BRDA, at depths ranging from 3.6m to 6.0m and selected to target the silty CLAY layer. Consolidation pressures were selected based on their assessed in-situ conditions.

The samples had initial void ratios ranging between 0.82 (GA19-5A-E2) and 0.99 (GA19-5C-E4), moisture contents of 34% and 40% respectively, and were consolidated to pressures reflecting their assessed in-situ conditions, i.e., 250 kPa and 150 kPa, respectively.

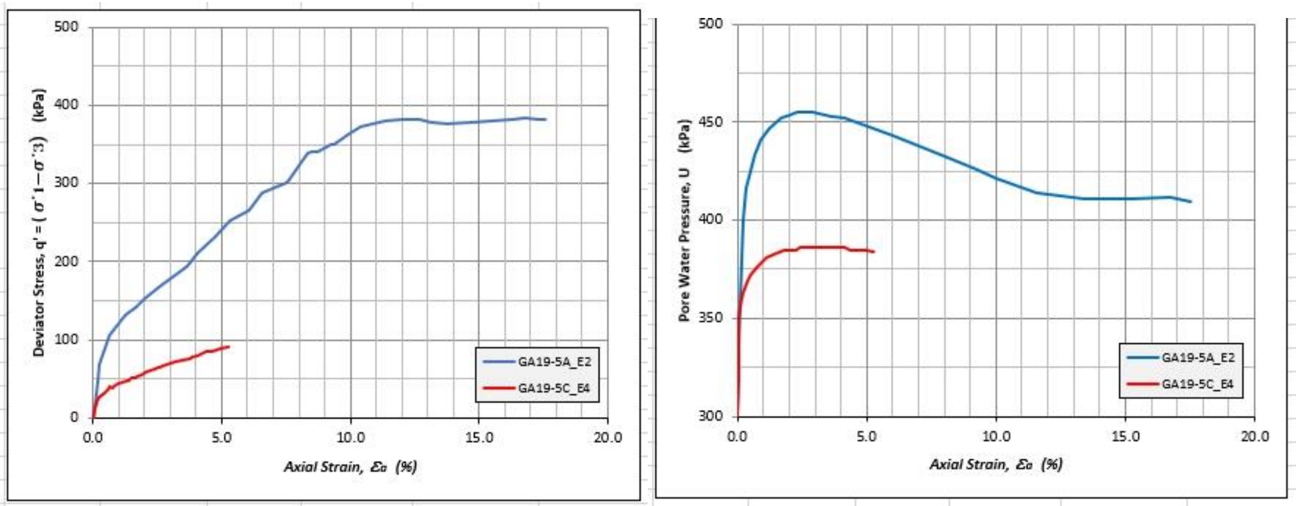

Figure 24: Triaxial Testing (CIU) on Estuarine Soil Samples (Golder 2020

- The test for GA19-5A-E2 shows a defined peak deviator stress (383 kPa at 12% strain) followed by a slight reduction to a steady state to a strain of 17.5%. There is no indication of brittleness (residual strength < 70% of peak strength). The test for GA19-5A-E4 appears to indicate that sample deformation may have prevented the test from progressing its peak at higher strains. The peak deviator stress recorded was 91 kPa and occurs at 5.2% strain.
- The interpreted undrained shear strength ratio (s_u/σ'_{v0}) varies from a peak of 0.77 at 12.0 % strain to 0.31 at 5.2% strain. The corresponding effective strength friction angles ranged from 35.5˚ to 24.4˚ and the undrained strength ratios ranged from 0.77 to 0.31.

Triaxial testing conducted during previous Site Investigations returned similar strength parameters and a similar testing response to GA19-5A-E2:

- 4 no. multi-stage CIUs were conducted on remoulded estuarine samples during the initial site investigations (SML 1971) at varying consolidation pressures (60 kPa to 450 kPa) and varying moisture contents (18% to 40%). The remoulded samples returned s_u/σ'_{v0} values ranging from 0.63 to 1.48 with corresponding effective strength friction angles ranging from 31˚ to 48˚. Strains at failure ranged from 9.4% to 18.3%. It is considered that these samples are a mixture of the sandy SILT and silty CLAY layers.
- 4 no. multi-stage CIUs were conducted on undisturbed estuarine samples in 1989 (IGSL 1989) at varying consolidation pressures (100 kPa to 300 kPa) and varying moisture contents (31% to 53%). The remoulded samples returned s_u/σ'_{v0} values ranging from 0.41 to 2.84 with corresponding effective strength friction angles ranging from 31˚ to 36.6˚. Strains at failure ranged from 9.8% to 20.2%. It is considered that these samples are mostly from the silty CLAY layer.
- 3 no. multistage CIUs were conducted on undisturbed estuarine samples for the 2004 site investigation (Golder 2005A) at varying consolidation pressures (300 kPa to 740 kPa) and varying moisture contents (33% to 47%). The remoulded samples returned s_u/σ'_{v0} values ranging from 0.22 to 0.40 with corresponding effective strength friction angles ranging from 37˚ to 39˚. Strains at failure ranged from 8.6% to 19.6%. It is considered that these samples are mostly from the silty CLAY layer.

Figure 25 below shows the plots for Shear Stress versus Shear Strain and Undrained Strength Ratio versus Shear Strain for DSS tests conducted in the 2019 Investigation (Golder 2020).

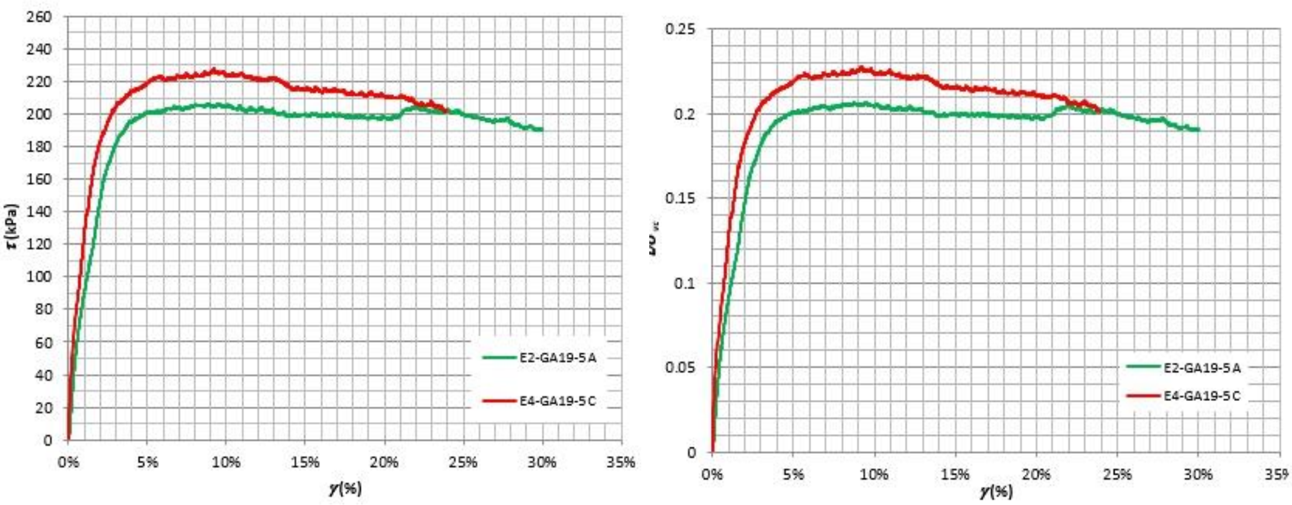

Figure 25: DSS Testing on Estuarine Soil Samples (Golder 2020)

The undisturbed samples were taken from the downstream toe of the OPW at the north-west sector of the Phase 2 BRDA, at depths ranging from 3.6m to 6.0m and selected to target the silty CLAY layer. Initial void ratios ranged between 0.92 and 1.38 and consolidation pressure was selected to be 1,000 kPa to replicate potential loading conditions from the BRDA to Stage 16.

- The tests generally show a defined peak shear stress followed by little change in stress as the strain increases to > 20% and hence no indication of brittle behaviour (residual strength < 70% of peak strength).
- Both tests returned similar peak strengths and residual strengths (206.6 and 190.7 kPa and 227.8 and 201.7 kPa, respectively) at similar peak strains (9.2% and 9.4% respectively) with both samples exhibiting marginal strain-softening behaviour after peak.
- Undrained strength ratios varied between 0.21 and 0.23 at peak and 0.19 and 0.20 at residual.

DSS tests conducted during the 2018 Site Investigation (Golder 2018) returned similar strength parameters and similar testing responses:

- 5 no. DSS tests were conducted on undisturbed estuarine soils samples consolidated at lower stresses (250 to 500 kPa) as they were sampled from beneath the existing Phase 1 BRDA, and pressures were selected to replicated in-situ conditions. The samples all displayed only marginal strain-softening behaviour after peak. They returned peak s_u/σ'_{v0} values ranging from 0.25 to 0.52, with the peak stresses ranging from 71.6 kPa to 207.3 and strains at failure ranging from 13.4% to 20.5%. Four of the five samples are considered to be from the silty CLAY layer with the s_u/σ'_{v0} values ranging from 0.25 to 0.29 for these 4 samples.

Figure 26 below shows a plot of undrained shear strength determined from triaxial and DSS testing versus vertical effective stress (Golder 2020).

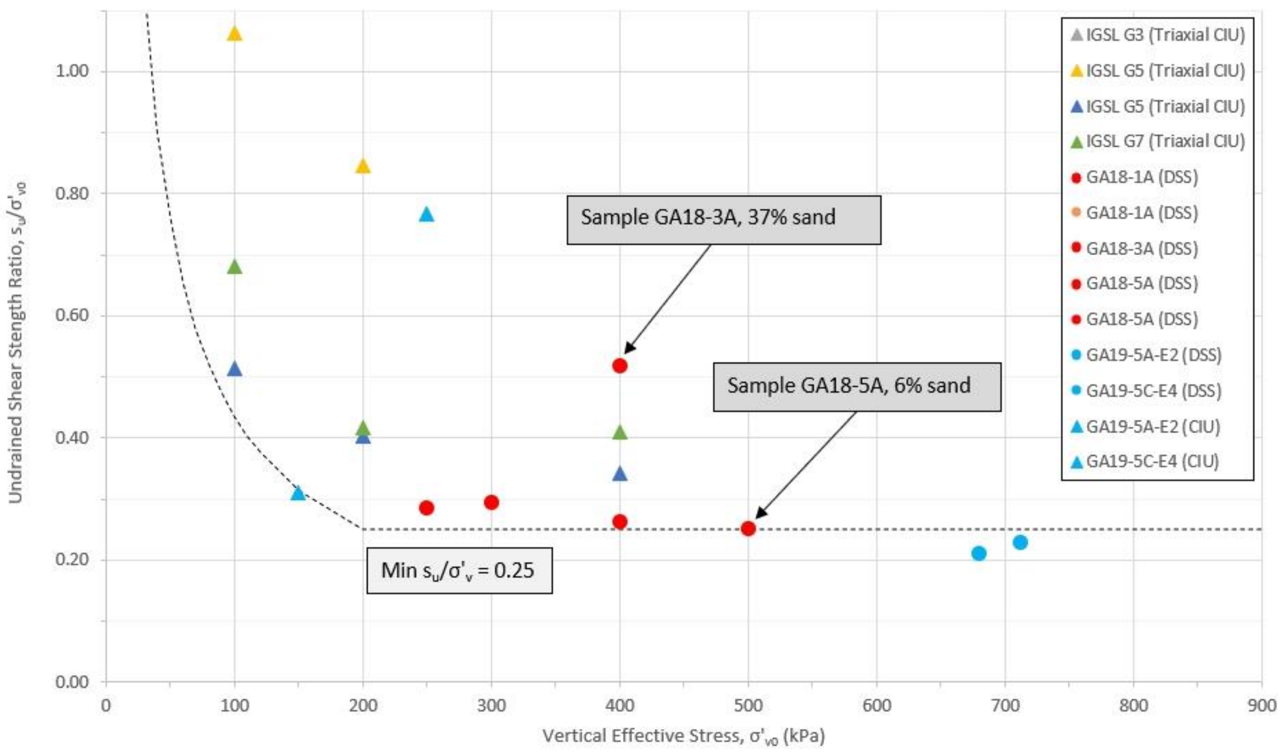

Figure 26: Estuarine Soils Undrained Shear Strength Ration versus Vertical Effective Stress

Notes:

1. *Mayne 2016 suggests that the DSS test is the most appropriate test to use when correlating the interpreted undrained strength from CPTu data as it presents s_u results that fall more-or-less mid-way between the other test modes (compression and extension) and thus provides an 'average' result.*
2. *The undrained shear strength, s_u, is not a unique soil property. It varies depending on the mode of failure, the stress state of the soil, anisotropic effects, and rate of failure. Furthermore, in situ tests may not necessarily be fully drained, as may be the case for silty soils, and so not truly represent undrained shear strength.*

5.3.5.4 Till

Till encountered in trial pits for the 2004 Site Investigation for the Phase 2 BRDA (Golder 2005B) varied from 1.2m to 5.35m depth in the western sector and was isolated in pockets of up 1.2m depth in the eastern sector but generally had only minimal (< 0.1m) depth.

SPT N-values were generally between 19 and 24, indicating a stiff to very stiff material. Lower values were recorded near surface at some location, with N-values of 7 to 8 indicating firm to material.

Undrained shear strength of 70 to 80 kPa has been estimated for the stiff till material based on correlations with the Index Testing and field testing (Golder 2005B).

5.3.6 Dry Density – Moisture Content Relationship Testing (Bauxite Residue)

Dry Density – Moisture Content (DD-MC) Relationship testing to the Standard and Modified Compaction Proctor was undertaken on farmed bauxite residue and compared with Nuclear Density Meter (NDM) readings taken in 2012 and 2014 on a selection of trial areas that were subject to different mud-farming methods (Golder 2014).

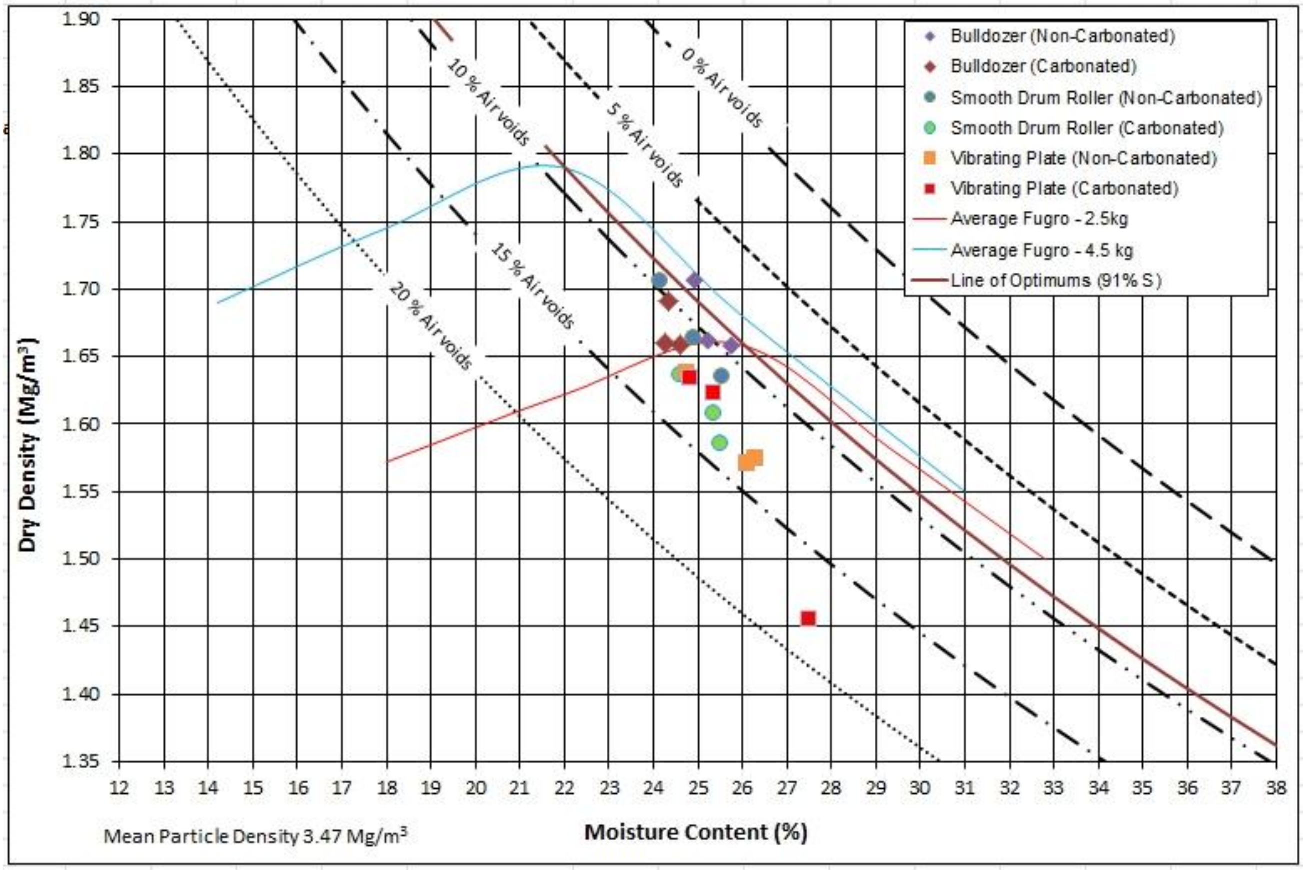

Figure 27: Standard & Modified Compaction Proctor Curves plotted with in-situ NDM tests

The Modified Compaction Proctor returned a Maximum Dry Density (MDD) ≈ 1.78 Mg/m^3 at an Optimum Moisture Content (OMC) of ≈ 21.5% and the Standard Compaction Proctor returned MDD ≈ 1.66 Mg/m^3 at OMC ≈ 25.5%. NDM readings and laboratory density testing returned in-situ values ranging from 1.57 to 1.71 Mg/m^3 for Dry Density and from 24 to 27 % for Moisture Content. Generally, mud farming achieves compaction ≈ 95% of the Standard Proctor and a characteristic value for farmed bauxite residue of 1.63 Mg/m^3 has been selected. Moisture Contents taken from in-situ samples below surface tend to have higher readings than those indicated by NDM readings and near surface core samples (≈ 33% versus ≈ 25%) and indicates that the farmed layers take back on moisture following covering with subsequent layers.

Golder 2020 conducted testing on the amended bauxite residue. DD-MC Relationship testing returned MDD ≈ 1.74 Mg/m^3 at OMC ≈ 21.0% for the Modified Compaction Proctor. NDM readings and laboratory density testing returned in-situ values ranging from 1.58 to 1.67 Mg/m^3 for Dry Density and from 21.9 to 26.8% for Moisture Content, which represent compactions at 91% to 96% of the Standard Proctor.

5.3.7 Seismic Parameters

Seismic parameters were determined for the bauxite residue to be able to undertake a liquefaction assessment (see Table 1 in Appendix C and Section 7.2)

The seismic parameters were determined from the in-situ seismic CPTu (see Section 5.2.6), laboratory Consolidated Undrained Cyclic DSS testing and Bender Element testing.

5.3.7.1 Cyclic Direct Simple Shear

4 No. Consolidated Undrained Cyclic Direct Simple Shear (Cyclic DSS) tests were conducted on remoulded bauxite residue samples. The point of liquefaction was evaluated by the following two methods:

- **At 3.75% single amplitude strain.** This is the definition for liquefaction previously suggested by the US national research council (NRC 1985) and allows comparison with test result values from literature as a consistent index for onset of liquefaction; and
- **Peak ratio of excess pore pressure to initial vertical confining stress ($\Delta Us/\sigma'_{vc}$).** No. of cycles where the excess pore pressure ratio reaches a peak at between 0.74 and 0.84, at ≈ 13% strain.

Table 8: Bauxite Residue Cyclic DSS Test Results (Golder 2018)

Vertical Consolidation Stress, σ'_{vc} (kPa)	Void Ratio	Cyclic Stress Ratio CSR (τ_{cy}/σ'_{vc})	≈ 3.75% Single Amplitude Strain		≈ 13% Single Amplitude Strain	
			NL	ΔUs / σ'_{vc}	NL	ΔUs / σ'_{vc}
130	1.15	0.14	15	0.740	25	0.84
	1.14	0.21	2	0.55	6	0.79
	1.22	0.29	1	0.40	3	0.74
200	1.13	0.25	1	0.36	3	0.78

5.3.7.2 Bender Element Testing

4 No. triaxial CIU tests and 3 No. triaxial CID tests with Bender Element were conducted on remoulded bauxite residue as part of 2004 Site Investigation (Golder 2005A).

Table 9: Bender Element Test Results (Golder 2005A)

Test ID	MC %	Consolidation Pressure (σ'_{vc})	G_{max} @ Strain (1% to 3%) MPa	G_{max} @ Strain (3% to 6%) MPa	G_{max} @ Strain (10% to 18%) MPa	Mean G_{max} MPa
FGT4a	39	100	37.5	37.0	42.2	38.9
FGT5a	40	800	191.0	192.9	199.5	194.5
FGT6c	48	100	45.3	46.4	55.1	48.9
FGT7a-1	34	100	72.0	77.1	99.1	82.7
FGT7a-2	40	800	120.1	145.4	178.6	148.0
FGT8d-1	32	300	144.8	156.0	224.5	175.0
FGT8d-2	41	300	50.9	58.2	68.6	59.2

The bender element tests were conducted at three confining stresses of 100 (3 No.), 300 (2 No.) and 800 kPa (2 No.) and the small strain shear modulus, G_{max}, was measured at 3 increasing strain points and the average value taken.

These results were plotted with the seismic CPTu parameters to determine a relationship between G_{max} and vertical effective confining stress, σ'_{v},(see Section 5.2.6) and utilized in the liquefaction assessment (see Section 7.2and Appendix C).

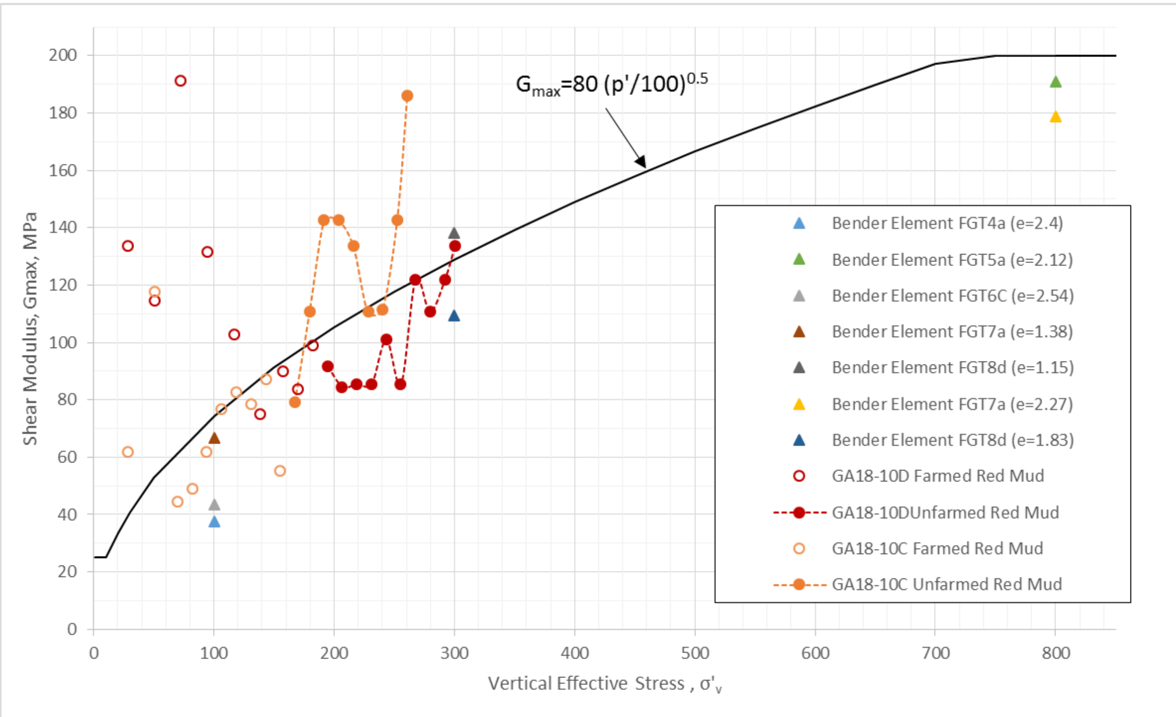

Figure 28: AAL Bauxite Residue Small Strain Modulus versus Vertical Effective Stress

5.3.8 Bedrock Testing

The Waulsortian Limestone beneath the Plant Site, the Borrow Pit and Borrow Pit Extension, and the eastern sector of the BRDA has undergone extensive testing in order to determine the design parameters for the foundations of the major structures in the Plant during the 1970s and 1980s (see Table 6).

Table 10: Typical Waulsortian Limestone Properties from Laboratory Testing (Clark et al. 1981)

Limestone Condition	Unconfined Compressive Strength (MPa)	Unconfined Modulus of Elasticity (MPa)	Modular Ratio
Faintly Weathered	100	70	700
Moderately Weathered	90	50	560
Penetratively Weathered	70	20	290
Highly Weathered	50	5	100
Altered Dolomitic	35	3	85

Samples obtained from borehole core drilling were tested to establish the compressive strength and the elastic modulus by the cyclic Unconfined Compression Test (UCT). Design characteristic values for compressive strength of 70 MN/m^2 (70 MPa) and for elastic modulus of 20,000 MN/m^2 (20 MPa) were selected for use.

The Waulsortian Limestone along the eastern flank of the BRDA (Boreholes 18 and 24) is described as being strong, massive, grey, coarsely crystalline and generally fresh or faintly weathered. It was assessed as being well jointed and sometimes fractured, particularly at near surface.

A geophysical survey was conducted in the footprint of the current Borrow Pit during Q4 of 2016 and comprised 8 electrical resistivity imaging (ERI) survey lines (see Figure 29 below)

The survey indicated that the footprint is underlain by competent limestone bedrock (over 2,000 ohm-meter) and associated discrete fracture zones/karst features. A possible karst feature was identified trending NE to SW and is most likely controlling the groundwater flow in the footprint (Golder 2017A).

Boreholes were targeted to intercept possible features identified by the geophysics survey and encountered zones of heavily fractured ground or cavities (karst features.

A number of the boreholes encountered groundwater and pump tests were attempted. The recharge was insufficient at any of the boreholes to complete a pump and it is considered likely that the features identified are not connected.

The Rathkeale Formation limestone beneath the bulk of the BRDA was investigated at various times during the development of the BRDA between 1971 and 2004 (see Table 6). The Rathkeale Formation is described as being a lower carboniferous limestone (Boreholes 16, 17, 22, 23, and 28), strong to very strong, medium to dark grey, medium to fine crystalline, fresh to faintly weathered, slightly argillaceous and thinly bedded, usually well jointed and sometimes fractured at the surface. Similar design characteristic values as selected for the Waulsortian Limestone for compressive strength of 70 MN/m^2 (70 MPa) and for elastic modulus of 20,000 MN/m^2 (20 MPa) are considered to be applicable to the Rathkeale Formation.

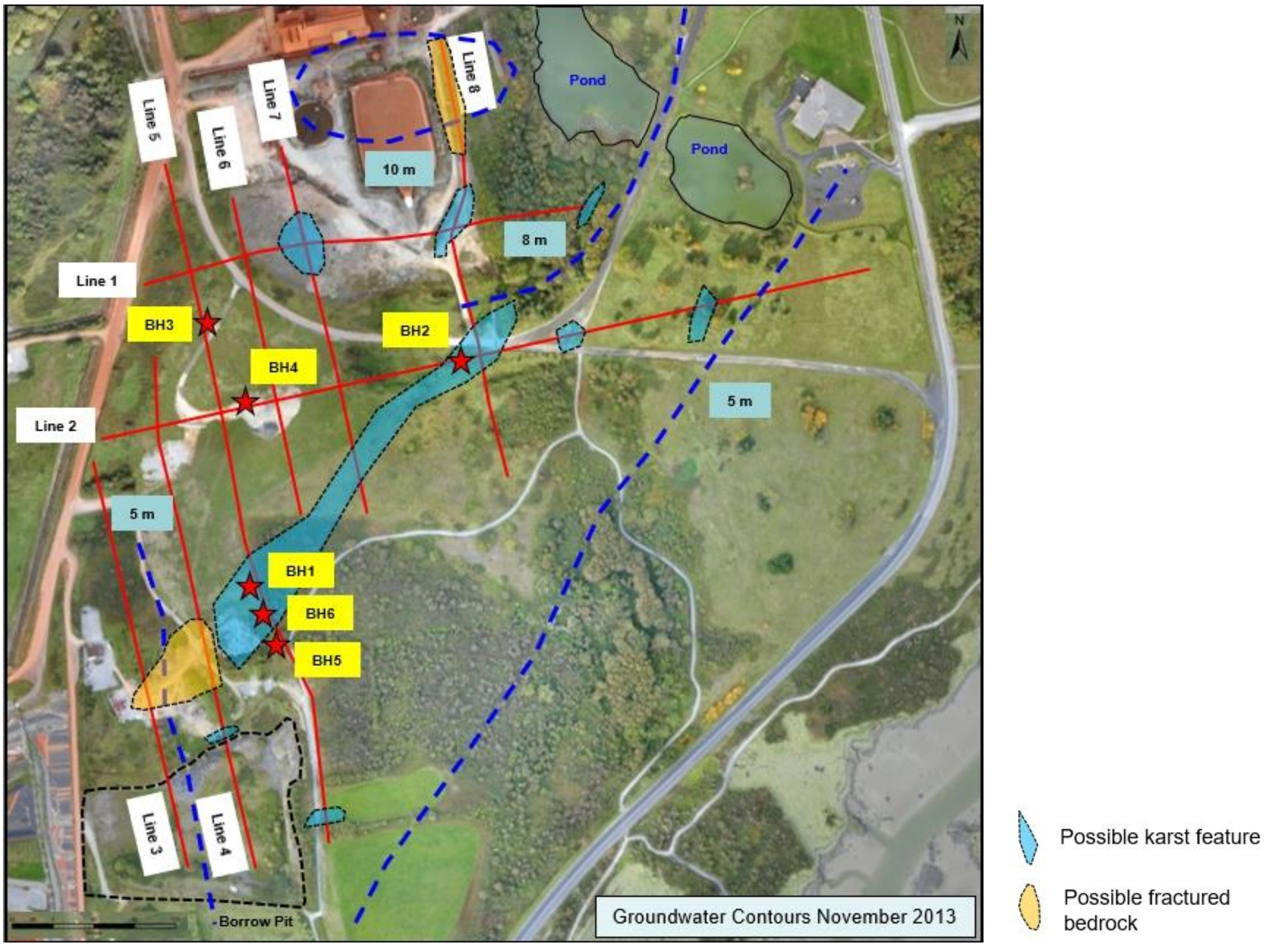

Figure 29: Geophysics Survey of Borrow Pit Footprint (Golder 2017A)

5.4 Other Investigations / Testing

5.4.1 Geosynthetic Interface with Bauxite Residue

Interface testing between the bauxite residue and HDPE geomembrane was undertaken to confirm the shear strength along this interface during the 2018 Site Investigation (Golder 2018). Interface shear strength testing along both textured and smooth HDPE geomembrane was conducted. The tests were conducted using a large shear box, according to ASTM D5321-12. The results of the testing are summarised in Table 11 below.

Table 11: Interface Shear Strength - Bauxite Residue and HDPE Geomembrane

Interface	Sample Initial Condition				Normal Stress (kPa)	Effective Stress (kPa)	Friction Angle (degrees)
	Bulk Density (Mg/m^3)	Dry Density (Mg/m^3)	MC (%)	Void Ratio			
Smooth HDPE	2.12	1.59	33.5	1.14	300	119.2	21.6
	2.10	1.58	32.9	1.15	600	237.9	21.6
Textured HDPE	2.15	1.59	35.3	1.15	600	400.5	33.7

5.4.2 Constitutive Modelling of the AAL Bauxite Residue

Section 5.3.5.2 discuss the determination of the CSL for the AAL bauxite residue (Golder 2021E).

The material behaviour was then modelled in accordance with the most recent update of NorSand (Jefferies and Bean 2015) using the laboratory data to determine the intrinsic AAL bauxite residue NorSand parameters.

Table 12: NorSand Parameters for the AAL bauxite residue (Golder 2021E)

NorSand Symbol	NorSand Parameter	Value
Γ	Critical State Large Deformation Parameter – y axis intercept	1.35
λ_{10}	Critical State Large Deformation Parameter – slope of CSL	0.129
M_{tc}	Critical Friction Ratio	1.4
N_{tc}	Dilatancy Parameter – Volumetric Coupling Coefficient	0.25
χ_{tc}	Dilatancy Parameter – State Dilatancy Coefficient	4
$G_{max\ @\ p\_atm}$ (MPa)	Elasticity Parameter – Shear Bulk Modulus	24.6 MPa
G_{exp}	Elasticity Parameter – Shear Bulk Modulus	0.6
ν	Elasticity Parameter - Poisson's Ratio	0.15
H_0	Plastic Hardening Modulus – Fixed Component	75
H_ψ	Plastic Hardening Modulus – State Dependent Component	500

The NorSand parameters were then utilized to produce a continuous spectrum of possible deposited residue responses (state parameter) in terms of peak and residual undrained shear strength ratios against the initial void ratio of the bauxite residue (see Figure 30 below).

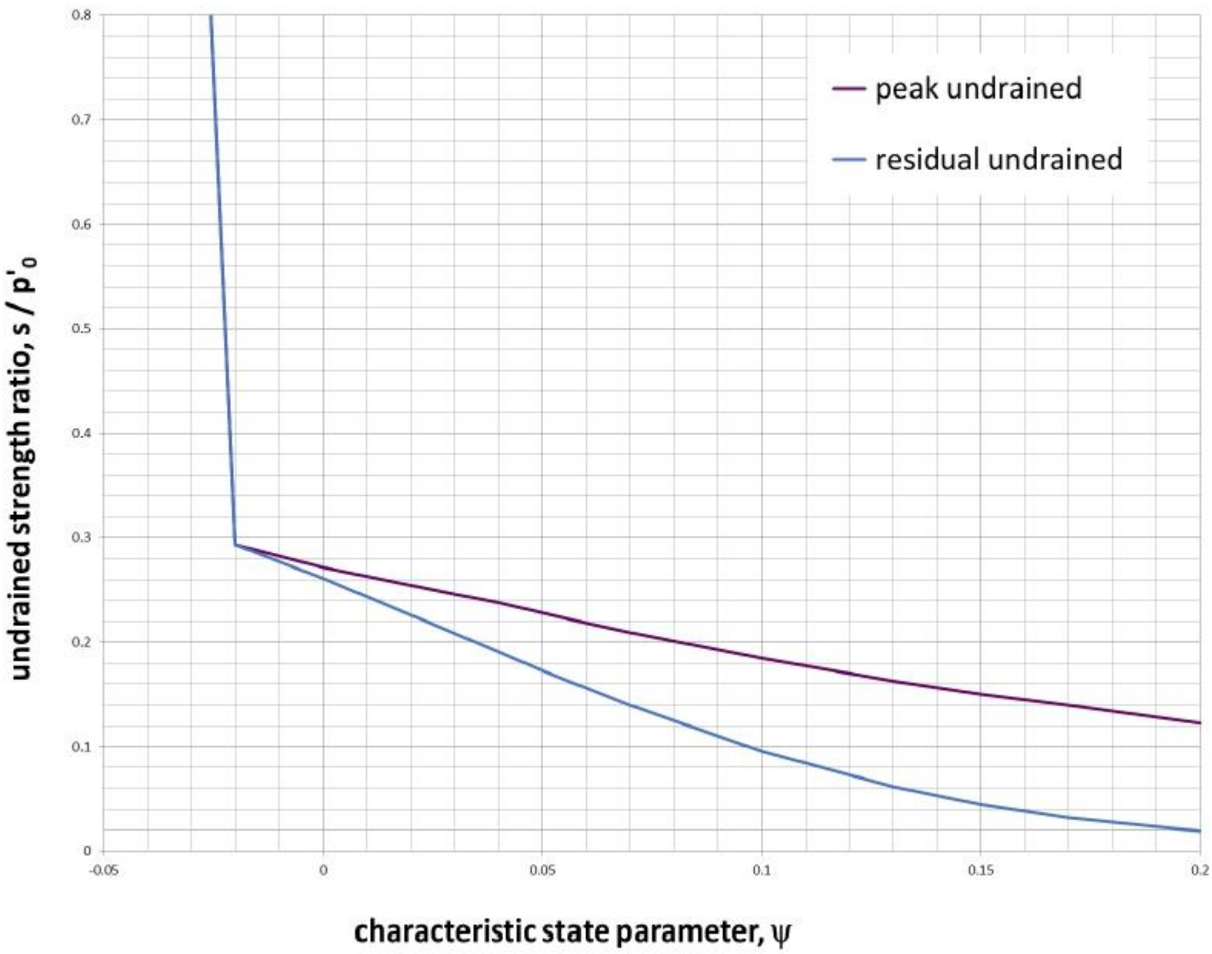

Figure 30: Spectrum of deposited residue responses (state parameter) versus undrained shear strength ratio

The characteristic undrained shear strength values selected (see Section 5.2.2) for the range of state parameters for the deposited farmed and unfarmed bauxite residue (see Section 5.2.5) are compatible with the spectrum provided in Figure 30.

Residual strengths are estimated to be in the 0.12 to 0.29 range for undrained shear strength ratio in the range of initial state parameter values of the deposited bauxite residue.

6.0 DESCRIPTION OF BRDA

6.1 Construction History and BRDA Footprint

The construction history of the principal components of the BRDA is summarized in Table 13 below.

Table 13: Construction History of the AAL BRDA

Date	Activity
1978 to 1983	Plant and Phase 1 BRDA Construction
1981 to 1982	SWP and LWP Construction
1983	Deposition of Bauxite Residue in the Phase 1 BRDA
1996 to 1998	Construction of the Phase 1 BRDA Extension (composite lined)
2007	Re-lining of the SWP (composite lined) and raising of the SWP west dam wall
2008 to 2011	Construction of the Phase 2 BRDA (composite lined)
2009	Commencement of intensive Mud-Farming activities
2010	Wick-Drain installation at Section F-F
2011	Deposition of Bauxite Residue in the Phase 2 BRDA
2012	Raise of the SWP and LWP to 6 mOD
2012 to 2013	Salt Cake Disposal Cell Construction (independent, composite lined cell) to 24 mOD
2013	BRDA Merger Works (Phase 1 and Phase 2 BRDA) & Phase 1 BRDA side-slope capping, and containment works to Stage 8
2016	Construction of Raise to the Salt Cake Disposal Cell (≈ 2.25m)
2018	Construction of Raise to the Salt Cake Disposal Cell (≈ 3.0m)

Bauxite residue from the production process is deposited in the BRDA located to the south-west of the Plant. The BRDA was constructed in 3 phases, and comprises 2 distinct disposal areas which are currently merging:

- The Phase 1 BRDA is formed from two facilities; the original Phase 1 BRDA constructed in the early 1980s, covering an area of 72 ha., and the Phase 1 BRDA Extension, constructed in the mid-to-late 1990s, covering an area of 32 ha. The design elevation at Stage 10 has a perimeter elevation of 24 mOD and maximum dome crown elevation of 32 mOD.

- The Phase 2 BRDA is a southern extension of the Phase 1 BRDA that was permitted in 2007 also to be constructed to Stage 10, elevations as above. Phase 2 BRDA covers an area of approximately 80 ha. and was commissioned in 2011.

- The permitted BRDA's have capacity to provide a disposal area for bauxite residue until circa 2030, for the current rate of production. The current elevation of the BRDA varies, from 22 mOD to 32mOD in Phase 1 BRDA, and from 11mOD to 20mOD in Phase 2 BRDA. The bulk of the annual bauxite residue produced (≈ 83%) has been deposited in the Phase 2 BRDA for 2019 and 2020, with 100% being deposited in the Phase 2 BRDA for 2021, to date.

6.2 BRDA Basin

The key design components of the BRDA basin are:

- Low Permeability Outer Perimeter Wall (OPW);
- Permeable Inner Perimeter Wall (IPW), which is the starter dam for the BRDA;
- Perimeter Interceptor Channel (PIC);
- Low permeability estuarine soils at the base of the Phase 1 BRDA;
- Composite lining system for the Phase 1 Extension BRDA and the Phase 2 BRDA;
- Permeable 2m high rock fill upstream Stage Raises, offset with 4m wide benches; and
- Upper-level wide bench (Stage 5) to reduce the overall side slopes gradient to 6.3(H):1(V).

The pre-construction ground elevations for the Phase 1 BRDA generally varied between 0 mOD to 14 mOD (west and central to east). Similarly, the pre-construction ground elevations for the Phase 2 BRDA varied from 0 mOD to 14 mOD (west and central to east). Hence, future raising and merging of the stack walls needed to accommodate these differences in elevation.

The Phase 1 and Phase 2 BRDAs are surrounded by a composite lined PIC which is formed by constructing the IPW and the OPW.

- The OPW has crest width of 5m and has a crest elevation of 4.7 mOD for the Phase 1 BRDA and 5.0 mOD for the Phase 2 BRDA (north, west and south sectors). The OPW for the eastern sector has crest elevations varying between 8.0 mOD and 16.0 mOD. The OPW is constructed of either till or rock fill and is composite lined on the upstream slope to form the PIC. The downstream slope has been overlain with a gabion mattress for the northern and western extents of the Phase 1 BRDA.
- The IPW is constructed of permeable rock fill, has a crest width of approx. 4.5m and generally to a crest elevation of 4.5m (north, west and south sectors) and provides the starter dam for the BRDA. The IPW for the eastern sector of the Phase 2 BRDA will be constructed on deposited bauxite residue to the design elevations corresponding to the OPW. The IPW for the Phase 1 BRDA is constructed above the estuarine soils, whilst the IPW for the Phase 1 BRDA Extension and the Phase 2 BRDA has the composite lining system passing beneath which subsequently forms the base of the PIC.
- Seepage, bleed water, sprinkler water and surface water runoff percolate through the rock fill stage raises and discharges into the encompassing PIC.

The current Phase 1 and Phase 2 BRDA PICs connect at the west sector of the facility where the Phase 1 and Phase 2 BRDAs' adjoin. The flow is in a clockwise direction to the Storm Water Pond (SWP), which is located in the north-east sector of the Phase 1 BRDA. The PIC for the eastern sector of the Phase 2 BRDA will be formed as the bauxite residue reaches the design elevation and the catchment is split with the bulk of flow going clockwise, whilst the remaining flow merges with the PIC segments of the Phase 1 BRDA Extension and flows anti-clockwise to the SWP.

Surface water collected in the PICs is pumped directly to the Effluent Clarification System (ECS) in the Plant and/or into the SWP, which is located in the north-east sector of the Phase 1 BRDA, and subsequently to the ECS. Treated water is pumped from the ECS and into the Liquid Waste Pond (LWP) to cool and settle prior to being pumped to the designated discharge point into the River Shannon.

6.3 BRDA Deposition

Bauxite residue resulting from the process is dewatered in the plant using vacuum drum filters and a deep thickener, which are aimed at reducing the caustic content. Water is then added, and the bauxite residue paste is pumped at a circa 58% solids content by positive displacement pumps to the discharge platforms in the BRDA via 1 No. 12” dia. steel pipe and 1 No. 14” dia. steel pipe, at an average pressure of 6,500 kPa.

There are two discharge platforms; one located in the Phase 1 and the other in the Phase 2 BRDA. These discharge platforms feed a network distribution of fixed spigot points called mud points (MPs) for layered deposition within cells, with decreasing slope on the bauxite residue surface further away from the discharge point. The cells have perimeter berms constructed from bulldozed farmed bauxite residue to a height of approx. 2m. Currently, there are 17 No. mud points (MP1 to MP17) in the BRDA with MP1 to MP9 located in the Phase 1 BRDA and MP10 to MP17 located in the Phase 2 BRDA. The distribution network for the discharge platforms and the MPs were installed at the base of the facility when the basin was constructed, and the MPs are raised vertically, in 1m to 2m intervals, corresponding to the increase in height of the BRDA.

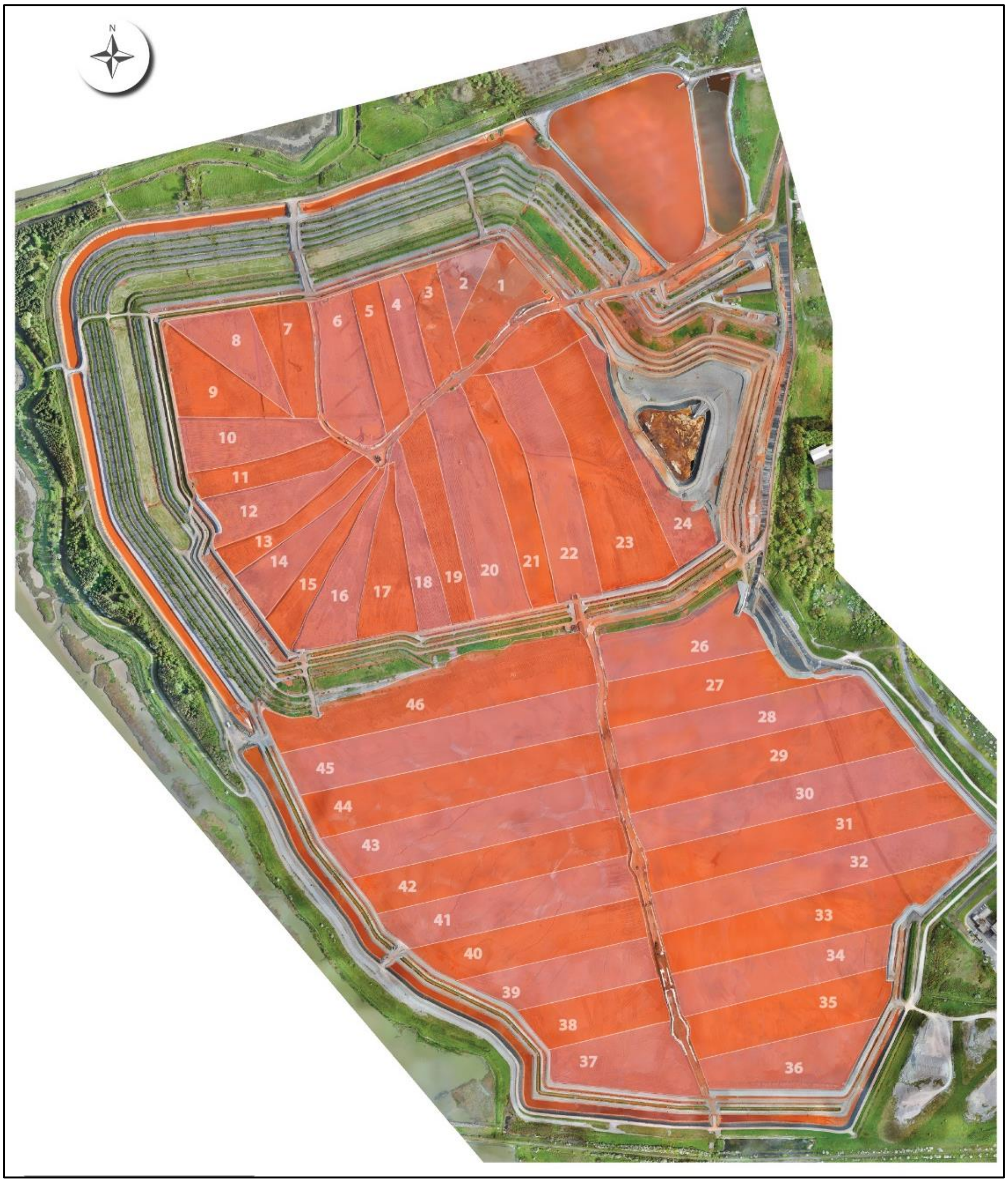

Figure 31: BRDA Layered Deposition Cell Layout (December 2020)

The deposited bauxite residue is mud-farmed (since 2009) to enhance drying of the residue, promote densification and to enhance exposure of the residue to the atmospheric carbon dioxide to reduce the liquid phase alkalinity. The farmed bauxite residue is tested to achieve a pH < 11.5 and is subsequently graded and compacted in preparation for the next deposition layer. Additional spading and harrowing are deployed if the pH is below target.

The current layout of layered deposition cells for the Phase 1 BRDA (Cells 1 to 25) and Phase 2 BRDA (Cells 26 to 46) is shown in Figure 31 above. The turn-around time for the cells allows for the reduction off the pH to < 11.5 and for the increase in the density and strength parameters of the deposited bauxite residue layer. Two layers are deposited in each cell annually, after which the cell bunds are then re-formed from locally sourced farmed bauxite residue using a dozer.

AAL implement twice annual aerial surveys (typically April and October) and the annual bauxite residue volume deposition is determined from an assessment of the aerial survey data minus the volume of rock fill placed for stage raise construction.

The approximate rate of rise was 12m in 14 years (0.86m / year from 2005 to 2019) for the Phase 1 BRDA and 14m in 14 years (1.00m / year from 2005 to 2019) for the Phase 1 BRDA Extension. This represents a reduction in the pre-2005 rate of the raising of the Phase 1 facility that can be attributed to the additional footprint provided by the Phase 2 BRDA since 2011.

The majority of bauxite residue is being placed within the Phase 2 BRDA in recent years (80% in 2018 and 82.5% in 2019), the rate of rise in the Phase 2 BRDA has been slightly greater than the Phase 1 facility with an average depth of 14m placed alongside the centre of the North-South Road during the 8 years of operation (≈ 1.75 m / year). An average depth of 10m has been placed at the perimeters (east, west and south) during the 8 years of operation (≈ 1.0m / year).

This rate of raising of the bauxite residue is considered slow, relative to other tailings facilities using upstream embankment construction, and thus allows any excess pore pressures that develop to dissipate.

Vick (1990) suggests that for rates of rise between 15ft and 30ft / year (4.5m to 9.0m / year), excess pore pressures are usually assumed to dissipate as rapidly as the load is applied and therefore a normally consolidated state (i.e., zero excess pore pressure) can be assumed. Mittal and Morgenstern (1975, 1977) also suggested this range as being sufficient to dissipate excess pore pressures in slimes.

6.4 BRDA Raising

The BRDA is progressively raised by the upstream method which involves constructing a permeable rock fill berm (stage raise) at the perimeter which is founded on the previously deposited and farmed bauxite residue. The stage raises are constructed in 2m vertical lifts (4m crest width, side-slopes of 1.5(H):1(V) and typically offset from inner crest to starting toe by a 4m wide bench), thus forming a supporting face to the overall structure, whilst also allowing drainage.

Unlike conventional tailings facilities or water retaining dams, the BRDA retains little to no surface water on the surface. The bauxite residue is discharged as a paste from several near central discharge points and migrates to the perimeter stage raises to form a dome which typically has the apex some 6m to 8m above the perimeter stage raise elevation. The slope produced averages grades between 2% and 4%. The final elevation of the perimeter stack wall will be 24 mOD at Stage 10, and the highest elevation of stacked residue for the dome will be 32 mOD, or some 30m above surrounding ground elevation.

In recent years, a collection drain has been formed in the bench of the uppermost stage raise to collect seepage and runoff and divert the waters towards a piped drainage system (450mm OD twin-walled HDPE pipes at max. 100m centres) leading directly to the PIC. This system allows for the progressive restoration of lower benches as the BRDA increases in height by eliminating the trickle down of the alkaline waters over vegetation.

Downstream side slope restoration, comprising side-slope drainage and planting berms, was completed during 2013 along the northern and western sectors of the Phase 1 BRDA from Stage 0 to Stage 8. Interim side-slope restoration, comprising drainage between toe drains of stage raises and hydroseeding of the upstream faces of the stage raises, is ongoing, and has been completed along the northern and western sectors of Phase 1 BRDA to Stage 10 and along the western flanks of the Phase 2 BRDA to Stage 3.

The facility is designed to operate with a high phreatic surface (design target is minimum of 3m below surface) because the stack wall slopes are relatively shallow. The stack wall has an overall slope of ≈ 6.3(H):1(V) consisting of a lower and upper slope formed at 6(H):1(V) separated by a 28m wide bench at Stage 5 (14 mOD). At Section F-F, the upper slope was steepened to ≈ 4(H):1(V) from Stage 5 to Stage 8 for a distance of approx. 300m to avoid constructing subsequent stage raises directly over a Sludge Disposal Area that was previously located in the south-west sector of the Phase 1 BRDA. Wick drains were installed into the Stage 5 bench for a distance of approx. 300m and a width of approx. 25m to help strengthen the bauxite residue for the foundations of Stage 6, 7 and Stage 8. The Stage 8 bench was subsequently adjusted to an approx. 20m width to realign Stage 9 and 10 with the rest of the BRDA.

Historically, the stability analysis of the BRDA was required to achieve a target factor of safety (FoS) of 1.3 for the undrained condition. The design of the Phase 1 and Phase 2 BRDA to Stage 10 was approved for this criterion in 2007 (Limerick County Council (LCC) Reg. Ref. 05/1836; An Bord Pleanála (ABP) Ref. PL13.217976).

AAL have since adopted the CDA Guidelines with the target FoS for the stability analyses of the BRDA now being 1.5 for the undrained condition. The implementation of mud-farming since 2009, when the Phase 1 BRDA was at circa 14 mOD elevation (Stage 7) and prior to development of the Phase 2 BRDA, has made this a viable FoS target. Mud-farming achieves dry densities far greater than would be expected from a standard thickened tailings facility and improves the strength parameters.

The underlying depth of unfarmed bauxite residue and estuarine soils for the Phase 1 BRDA are the central issues in achieving the target FoS but these are mitigated by the shallow gradient of the stack wall and the low rate of rise.

6.5 BRDA Current Status

AAL have raised the stack wall for the Phase 1 BRDA to Stage 10 along the east, north-east and north-west sectors and also have recently completed / currently constructing the south-west and south sectors to Stage 10. The elevation of bauxite residue deposited varies from approx. 32 mOD at the centre to approx. 22 mOD to 24 mOD at the perimeter stage raises.

For the Phase 2 BRDA, AAL have constructed to Stage 4 (12 mOD) along the east, west and south boundaries. Bauxite residue has been placed to approx. 10.5 mOD along the east perimeter wall, which will subsequently form the base of the internal perimeter interceptor channel (PIC) along this extent. The crest of east perimeter wall currently varies in elevation from Stage 6 (16 mOD) to Stage 4 (12 mOD) from its north-eastern extent to its eastern extent and transitions into the external PIC at the Observation Area located centrally on the east perimeter wall. The elevation of the bauxite residue deposited varies from approx. 20mOD centrally along the internal access road (north-south road), splitting the Phase 2 BRDA into east and west sectors. The elevation of bauxite residue at the east, south and west perimeter stages raises is at approx. 10.5 mOD, 9 mOD and 10 mOD, respectively.

The Phase 1 and Phase 2 BRDAs are being progressively merged, with the Phase 2 BRDA overlapping on the upstream raises on the south face of the Phase 1 BRDA to a current elevation of approx. 15 mOD.

The current rate of production of bauxite residue is circa 1.57 million tonnes / year (dependent on grade of ore) and is deposited at a characteristic bulk density of 2.19 tonnes / m^3, following mud-farming activities. The rate of void consumption is 0.9 to 1.0 million m^3 / year for bauxite residue and approx. 35,000 m^3 / year for rock fill. A degree of consolidation could be expected from the unfarmed bauxite residue in the Phase 1 BRDA and the underlying estuarine soils, but little settlement can be expected from the farmed bauxite residue (see Section 5.3.3.1). The greater depths of estuarine soils are located outer extents of the Phase 1 BRDA and will not be subjected to further loading as the BRDA progresses in an upstream fashion. The overall gain in BRDA capacity from consolidation of the bauxite residue and underlying estuarine soils is expected to be minimal.

An estimated 36.0 million tonnes of bauxite residue have been deposited in the BRDA from start-up in 1983 to December 2020. An estimated 9.12 million m^3 of bauxite residue void (discounted for volume of rock fill stage raises) is remaining following from the April 2021 aerial survey, which represents a total bauxite residue storage volume of 53.1 million tonnes for the permitted facility and a remaining life of 9.6 years, based on the current rate of production i.e., to 2030

6.6 BRDA Raise Design

The permitted BRDA has capacity to provide a disposal area for bauxite residue until circa 2030, for the current rate of production, which would have a final perimeter elevation of 24 mOD and a maximum dome crown elevation of 32 mOD, see Drawings 04, 05 and 06. It is proposed that the permitted height of the overall BRDA (Phase 1 and 2 BRDA) be increased to accommodate the further storage of bauxite residue at the facility to provide an additional 9-year capacity, until 2039. The raising of the BRDA does not require any extension of footprint and will only require minor modifications to the existing water management infrastructure.

It is proposed that the existing BRDA can facilitate an increase in height to Stage 16 (currently permitted to Stage 10) which would provide a perimeter elevation of 36mOD and a maximum dome crown elevation of 44 mOD. The proposed development will provide for the deposition of 0.9 to 1.0 million m^3 / year of bauxite residue and total of circa 8.0 million m^3 over the lifetime of the development until 2039.

The proposed method of raising the BRDA from Stage 10 to Stage 16 will be the upstream method, consistent with the construction methodology for the current BRDA and involves the construction of rock fill embankments (stage raises) offset internally and founded on the previously deposited and farmed bauxite residue, in 2m high vertical lifts. Figure 32 below shows a typical section of the proposed BRDA Raise from Stage 11 to Stage 16.

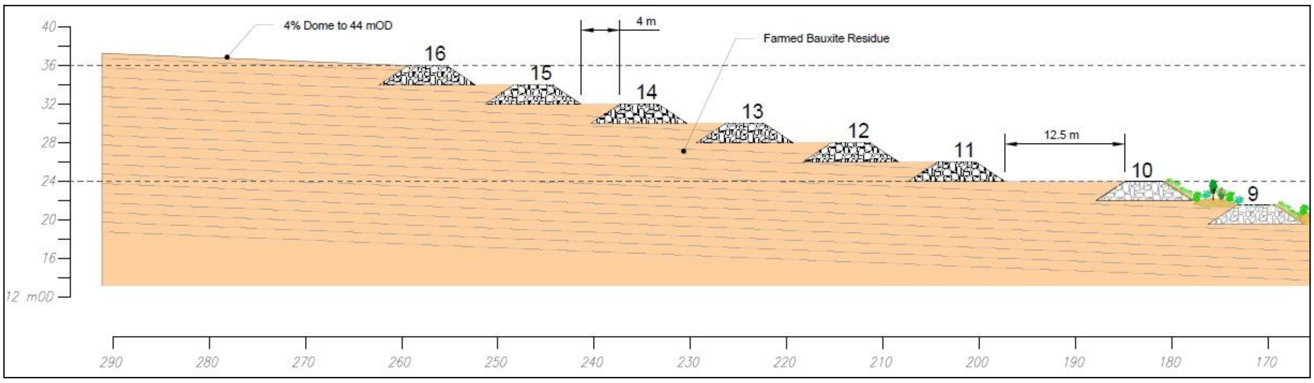

Figure 32: Typical Section of BRDA Raise from Stage 11 to Stage 16 (without side-slope closure capping)

The proposed increased in height is 12m which will comprise 6 x 2m high stages raises (Stages 11 to 16), to provide a new perimeter crest elevation of 36 mOD and a maximum dome crown elevation of 44 mOD. The area enclosed by the toe of the perimeter Stage 11 raise is 96.37 ha. The Stage 10 bench is 12.5m wide bench, and subsequent benches from Stage 11 to Stage 16 are the standard 4m width, to form a new upper gradient of 4.83(H):1(V) and an overall stack wall gradient of ≈ 6.8(H):1(V).

The proposed BRDA Raise Development will provide an additional estimated 8.04 million m^3 of void for bauxite residue storage (discounted for volume of rock fill stage raises) following from the April 2021 aerial survey, which represents an additional 13.1 million tonnes of bauxite residue and an additional 9 years of life. The estimated total remaining void for bauxite residue storage would be 17.16 million m^3 (discounted for volume of rock fill stage raises) following from the April 2021 aerial survey, which represents at total bauxite residue storage volume of 66.2 million tonnes and a remaining life of 18.5 years, based on the current rate of production.

The current BRDA water management infrastructure was designed to accommodate the BRDA development to Stage 10 and for an inflow design flood (IDF) with a return period of 1 in 200 years. It is proposed to modify the existing water management infrastructure (see Section 7.8) to accommodate the BRDA development to Stage 16 and for an IDF of a greater return period), in accordance with CDA guidelines, based on the classification of the BRDA (see Section 2.0).

6.7 Stage Raise Construction Materials and Methodology

The stage raises are constructed of hard, durable, well graded limestone rock fill, free of deleterious materials and with a maximum particle size of 300mm, that is termed Type B material. The Type B material is sufficiently permeable to permit the initial draining of the bauxite residue paste and surface water runoff but becomes less effective as the deposition elevation increases due to fines content of the bauxite residue.

This rock fill has been sourced either from:

- On-site stockpiles of blast rock remaining from the Phase 2 BRDA development that are crushed by AAL nominated Contractor's to the required specifications; or
- As the stockpiles have diminished, the rock fill has been imported from the nearby Roadstone Barrigone Quarry, located circa 2.5 km from the Plant Site, which has been crushed to specification.

The rate of consumption of rock fill for stage raise construction in recent years has been in the 30,000 to 40,000 m^3 / year range. AAL have received approval to develop their own Borrow Pit on Site for rock extraction and the initial blasts are expected to take place during Q2 2022. The approved Borrow Pit footprint is expected to provide 374,000 m^3 of rock fill material which is considered to be sufficient to construct the permitted BRDA to Stage 10 (≈ 198,000 m^3) and to implement the closure design (≈ 106,000 m^3) with a contingency available (≈ 70,000 m^3).

The rock fill for the proposed BRDA Raise Development is expected to be sourced from the proposed Borrow Pit Extension and an estimated volume of ≈ 385,000 m^3 is required to construct the BRDA to Stage 16. Additional volumes are required to implement the closure design (≈ 62,000 m^3) and raise the SCDC (27,000 m^3), above the rock fill requirements for the construction of the BRDA to Stage 10. The total rock fill demand for the BRDA constructed to Stage 16 and for closure requirements is ≈ 778,000 m^3 (from April 2021). The existing and proposed Borrow Pits will provide 754,00 m^3 and there is ≈ 30,000 m^3 currently stockpiled on site.

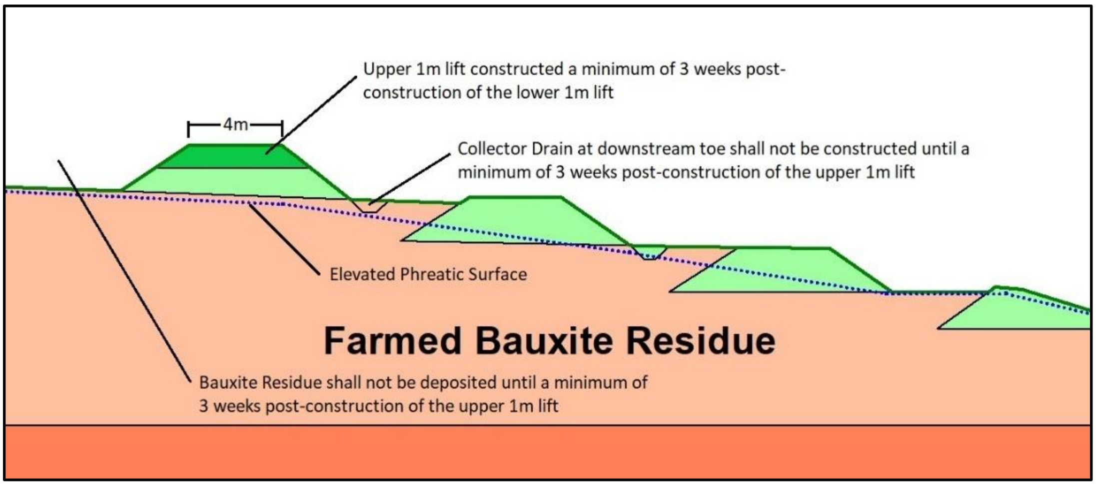

Figure 33: Stage Raise Construction Methodology

Stage raise construction follows the methodology described below and as shown in Figure 33 (Golder 2019C):

- The bauxite residue is deposited in layers, with the final layer filled to the elevation of the inner crest of the constructed stage raise, farmed and compacted.
- A minimum 14m width of subgrade, for the lateral extent of the stage raise to be constructed, is prepared for the construction of the subsequent stage raise, allowing 4m offset for the bench, 3m for the downstream slope at 1.5(H):1(V), 4m crest width and 1.5(H):1(V) upstream slope. Additional farmed mud is bulldozed into place and compacted to provide a level subgrade and/or to fill any low spots.

- A minimum 200 grms/m^2 separation geotextile is placed on the subgrade in the footprint of the proposed stage raise, approx. 10m width.
- The lower 1m lift of the stage raise is constructed with Type B rock fill and trimmed to the design profile. The rock fill is nominally compacted by tracking with plant.
- The upper 1m lift of the stage raise is constructed in a similar fashion following a minimum of 3 weeks has passed to allow for pore pressure dissipation. The final crest width is 4m at the design elevation.
- The excavation of the collector drain at the toe of the downstream slope and the deposition of bauxite residue ensues after a minimum of 3 weeks has passed since the construction of the upper 1m lift.

6.8 Stage Raise Phasing

Section 6.5 describes the current status of the BRDA (aerial survey from April 2021). It is expected that the Phase 1 BRDA will be fully constructed to Stage 10 and that all of Phase 2 BRDA will be raised to Stage 4 by the end of 2021.

For the permitted BRDA development to Stage 10, the bulk of bauxite residue will continue to be deposited in the Phase 2 BRDA and the rate of rise can be expected to be approx. 2m / year, or one stage raise / year constructed in the Phase 2 BRDA.

The phasing for the BRDA Raise Development would allow a more balanced deposition strategy between the Phase 1 and Phase 2 BRDA and there are a number of benefits to this approach:

- The footprint for deposition is greater is greater thus alleviating pressure on the mud-farming activities; and
- The annual rate of rise for the BRDA is reduced to between 1m and 1.5m / year.

The stage raise construction for the Phase 2 BRDA would continue to lag behind that of the Phase 1 BRDA by 4m to 6m (2 to 3 stage raises) until the Phase 1 BRDA reaches its design perimeter elevation of 36 mOD (Stage 16). The bulk of the bauxite residue deposition would then be deposited in the Phase 2 BRDA until the Stage 16 elevation is reached. Prior to the BRDA Closure, the design profile of the dome is formed with farmed bauxite residue and then the amended layer capping is constructed.

Table 14 and Figure 34 below outline the proposed construction phasing of the stage raises for the BRDA Raise Development based on the following assumptions:

- Approval for BRDA Raise Development at start of 2023.
- 14 m^3 of rock fill required per m length of stage raise constructed.
- Internal stage raises will continue to be constructed between the Phase 1 and Phase 2 BRDA.
- Bauxite residue is deposited at a characteristic density of 1.63 tonnes/m^3 dry or 2.19 tonnes/m^3 bulk when farmed, and filling a void of 950,000 m^3 / year, based on current rate of production.
- 30% of bauxite residue deposited to Phase 1 BRDA and 70% to Phase 2 BRDA from start of 2023 to completion of Stage 16 in the Phase 1 BRDA (2038).
- 10% to Phase 1 BRDA and 90% to Phase 2 BRDA otherwise.
- Total of 17.16 million m^3 bauxite residue deposited by 2039.

Table 14: Stage Raise Phasing for the BRDA Raise Development (2021 to 2039) from April 2021

Year	Phase 1 BRDA	Bauxite Residue (m³)	Rock fill (m³)	Phase 2 BRDA	Bauxite Residue (m³)	Rock fill (m³)
2021	Stage 10	-	4,200	Stage 4	500,000	0
2022	Stage 10	-	-	Stage 5	950,000	31,980
2023	Stage 10	95,000	-	Stage 6	855,000	32,630
2024	Stage 11	285,000	34,670	Stage 7	665,000	32,660
2025	-	285,000	-		665,000	
2026	-	285,000	-	Stage 8	665,000	32,250
2027	Stage 12	285,000	35,970		665,000	
2028	-	285,000	-	Stage 9	665,000	32,000
2029	-	285,000	-		665,000	
2030	Stage 13	285,000	31,540	Stage 10	665,000	31,640
2031	-	285,000	-		665,000	
2032	-	285,000	-	Stage 11	665,000	34,660
2033	Stage 14	285,000	30,820		665,000	
2034	-	285,000	-	Stage 12	665,000	35,960
2035	-	285,000	-	Stage 13	665,000	31,530
2036	Stage 15	95,000	30,170	Stage 14	855,000	30,810
2037	-	95,000	-	Stage 15	855,000	30,160
2038	Stage 16	95,000	29,490	Stage 16	855,000	29,480
2039	Dome	95,000	-	Dome	415,000	-
	Totals	**3,895,000**	**196,860**	**Totals**	**13,265,000**	**385,760**
	Total Rock fill Requirement = 582,620 m³					
	Total Bauxite Residue Void = 17,160,000 m³					

Note: *The annual volume of bauxite residue produced and deposited in the BRDA can vary as it is dependent on the grade of alumina in the bauxite ore.*

Figure 34: BRDA Raise Development Bauxite Residue Storage 2021 to 2039

6.9 Decant / Reclaim System

There are no decant structures associated with the operational BRDA i.e., spillways, decant towers etc., other than the caustic recovery system constructed within the Salt Cake Disposal Cell (see Section 13.0). The closure design for the BRDA will include spillways to channel flows from the dome to the PIC (see Section 8.3 and 8.4).

The bauxite residue is deposited centrally and grades at a slope of between 2% and 4% to the perimeter stage raises. The stage raises are constructed of permeable rock fill and allow the drainage of bauxite residue paste bleed water, surface water runoff, sprinkler water and seepage through the raise and into the collection drain excavated at the downstream toe of the uppermost stage raise.

The collection drain has a piped drainage system (450mm OD twin-walled HDPE pipes at max. 100m centres) which fast track the flows directly to the encompassing PIC. This system allows for the progressive restoration of lower benches as the BRDA increases in height by eliminating the trickle down of the alkaline waters over vegetation.

6.10 Lining System and Seepage Control Measures

Section 6.2 discusses the key design components of the BRDA basin and Table 15 below lists the lining systems installed for each design component.

The lining system for the BRDA basin is a mixture of natural and geosynthetic materials which have very low hydraulic conductivity. These lining systems provide the short-term containment as the BRDA basin is filled, the depth of deposited bauxite residue is increased, and consolidation occurs.

Section 4.3.1 of MWEI BREF 2018 discusses '*Techniques to prevent or minimize groundwater status deterioration and soil pollution*':

"After consolidation, extractive waste can have a similar permeability to a natural soil basal structure. In this case, the impermeable artificial basal structure provides the principal containment until the waste is consolidated. Thereafter the extractive waste tends to be the controlling basal structure."

Regarding the AAL BRDA, once a sufficient depth of bauxite residue has been deposited above the basal lining system (≈ 5m depth), then the bauxite residue itself becomes the controlling containment and long-term containment, owing to the following characteristics:

- Bauxite residue has a low hydraulic conductivity (see Section 4.3.1 and 5.3.4.1)
- Bauxite residue is farmed, and the consolidation benefits are achieved directly.
- No free water is stored on the BRDA.

Table 15: BRDA Lining Systems

BRDA Components	Lining System
Phase 1 BRDA	Low permeability estuarine deposits of varying depth (4m to 30m)
Phase 1 BRDA Extension	Composite Lined (1mm and 2mm HDPE over min. 0.6m depth of compacted till)
Phase 2 BRDA	Composite Lined (2mm HDPE over GCL and min. 0.5m depth of compacted till <u>or</u> min. 1m depth of compacted till)
IPW	■ Estuarine deposits underlie the IPW for the Phase 1 BRDA ■ Composite Lining System (as per BRDA) underlies the IPW for the Phase 1 BRDA Extension and the Phase 2 BRDA.
PIC	■ 2mm HPDE over estuarine deposits for the Phase 1 BRDA ■ Composite lining system for the Phase 1 BRDA Extension and Phase 2 BRDA (2mm HPDE over GCL and min 0.5m depth of compacted till)
OPW	Composite Lined on upstream slope (2mm HDPE over GCL and min. 1m depth of compacted till)
SWP	Composite lined basin and upstream side-slopes (2mm HDPE over GCL and min. 0.5m depth of compacted till)

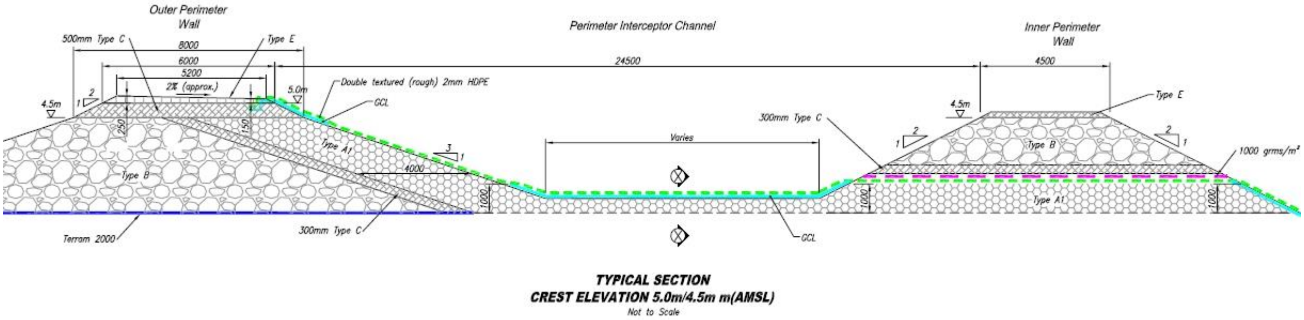

Figure 35: Representative Cross-Section showing the OPW, PIC and IPW for the Phase 2 BRDA

There are no leakage detection or leakage collection and removal systems installed directly above or below the BRDA lining system. These systems could only be expected to be operational in the short-term due to low permeability of the bauxite residue and the likelihood of clogging with fines. A coarse rock fill drainage bench was constructed above the lining system and along the upstream toe of the eastern sector of the Phase 2 BRDA to manage water ponding in the basin in the short-term until the bauxite residue was of sufficient elevation to form the PIC. The drainage bench was raised corresponding to the increase in bauxite residue and channelled flows to designed sumps which pumped the waters to operational segments of the PIC.

Seepage, bleed water, sprinkler water and surface water runoff percolate through the rock fill stage raises and discharge into the encompassing PIC. The PIC is composite lined and transfers the free water by gravity and pumping to the SWP, which is also composite lined. Bauxite residue sediment build-up at the base of the PIC and SWP is an issue which requires regular cleaning to maintain storage volumes but has the benefit of providing an additional level of low permeability barrier and a level of protection for the geomembrane. Waters stored in the SWP and subsequently pumped to the ECS for treatment prior to discharge and/or diverted to sprinkler systems for dusting prevention. AAL maintain varying volumes of water in the PIC and SWP to meet their water inventory targets for year which balances ensuring that there is sufficient water storage capacity for a significant storm event and ensuring there is sufficient water supplies available to supply the sprinkler system.

There is a potential for seepage to bypass beneath the PIC and/or for the PIC and SWP to have free water leakage from their lining systems. These risks are managed by the following infrastructure:

- A system of culverted drains and/or drains backfilled with rock fill exist beneath the BRDA (see Figure 5) which emerge at designated caissons. These caissons are monitored for water quality and flow and are pumped back into the PIC and/or discharged, depending on water quality assessment.
- Leak detection for the BRDA is monitored via the groundwater observation well (OW) network which comprises monitoring wells installed at the perimeter of the facility, offset from the downstream toe of the OPW, at maximum intervals of 150m. The monitoring wells extend for a minimum of 5m below the basin elevation of the BRDA into either the estuarine deposits, till or bedrock.
- A Toe Drain is excavated into the estuarine soils at the downstream toe of the OPW for the northern and western extents of the Phase 1 BRDA and for the northern extent of the SWP. The flow in the Toe Drain is monitored daily and a number of pumping stations have been established at known and historic seepage locations, which pump the waters back into the PIC.
- Abstraction wells have been installed at specific locations where geophysical surveys have identified potential flow paths and/or where seepage waters have been observed, which pump waters back into the PIC or SWP.
- An outer network of drains exists in the lands between the Toe Drain and the Flood Tidal Defence Berm (FTDB) which are monitored for water quality in accordance with the IEL.
- The outer drain network for the north and west flanks of the BRDA all converge to a single discharge point to the Robertstown River. This outlet is a one-way flap valve which only discharges at low tide. AAL have installed a penstock and manual shut-off to prevent discharge should an issue arise.

6.11 Stormwater Diversion and Control

The current BRDA water management infrastructure was designed to accommodate the BRDA development to Stage 10 and for an inflow design flood (IDF) with a return period of 1 in 200 years. There are currently no spillways or emergency discharge systems in the BRDA Water Management System to release waters in excess of this event.

The design flood events are managed by the BRDA water management infrastructure and all waters entering the PIC are pumped into the SWP and subsequently to the ECS and/or directly to the ECS.

The operational BRDA to date has provided sufficient internal flood attenuation capacity due to its topography i.e., the filling of the basin of the Phase 2 BRDA. As the Phase 2 BRDA reaches Stage 5 / Stage 6, the full PIC system will be constructed and the BRDA will have a consistent topography grading to the perimeter.

The proposed BRDA Raise Development will include modifications to the existing water management infrastructure to accommodate the Inflow Design Flood (IDF) determined by the CDA classification for the BRDA i.e., 1/3 between 1 in 1,000 and the PMF (see Section 2.3).

6.12 BRDA Water Management System

This section provides a summary of the current BRDA Water Management System, which is presented conceptually by the block flow diagram in Figure 36 below.

Section 7.8 provides a summary of the BRDA Raise Development Water Balance Assessment along with the proposed modifications. The full Assessment is provided in Appendix I.

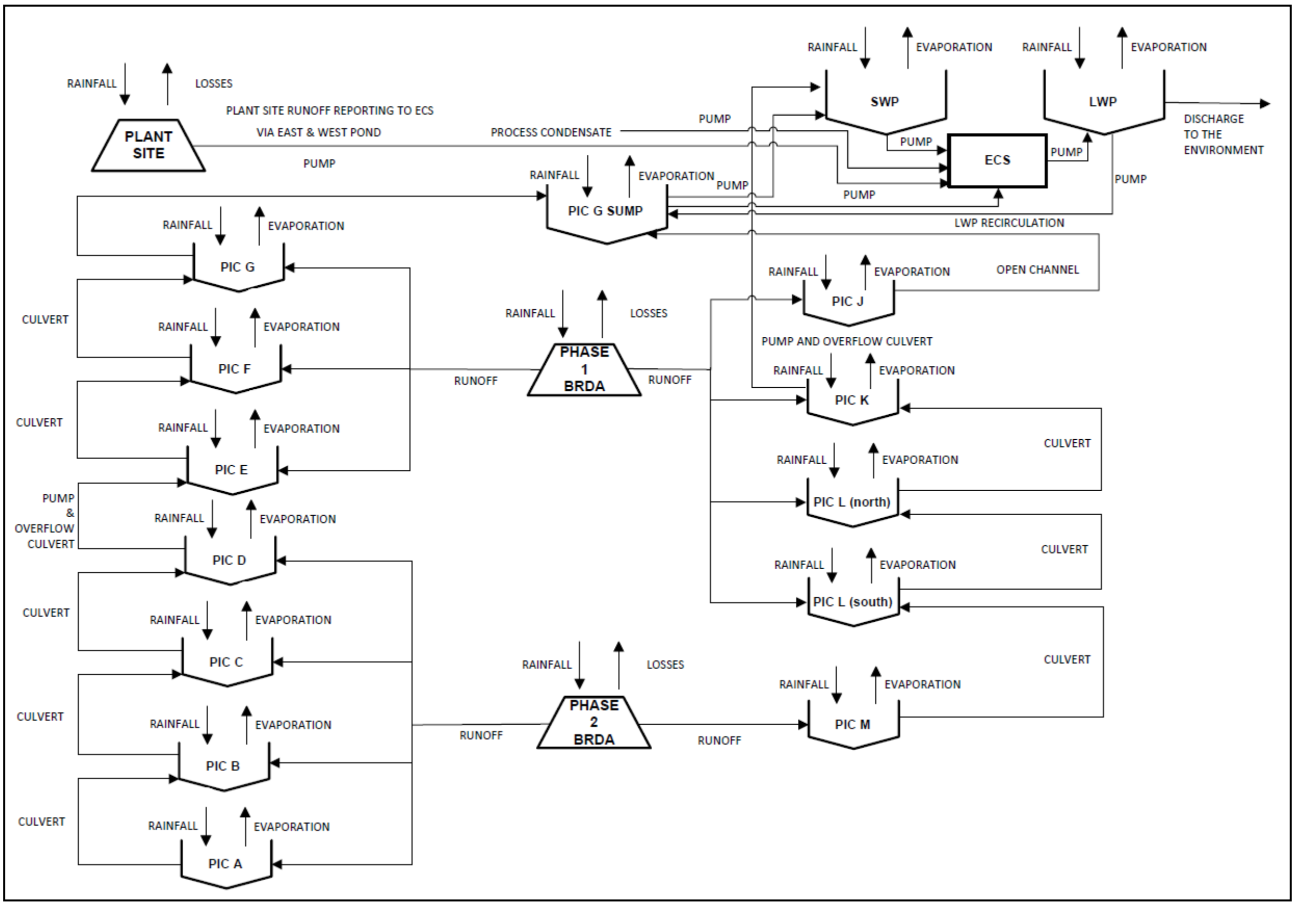

Figure 36: BRDA Water Management System - Block Flow Diagram

<u>Notes:</u>

1) *Catchment "Losses" presented in the flow diagram represent all hydrological losses from rainfall including evaporation, transpiration, infiltration, and losses due to surface depressions and ponding.*

The BRDA is surrounded by the Perimeter Interceptor Channel (PIC) which collects water emerging from the BRDA (seepage, bleed water, sprinkler water and surface water runoff) and conveys it via pumps either to the Effluent Clarification System (ECS) located in the plant and/or to the Storm Water Pond (SWP)

The SWP is located in the north-east sector of the BRDA, and its function is two-fold:

- To provide surge capacity for surface water that cannot be immediately processed by the ECS; and
- To provide a continuous flow of water that is used for dilution or wash water within some parts of the alumina plant.

Excess water from the PIC and SWP is pumped to the ECS via Pump 15, Pump 31 and Pump 32 at a maximum discharge capacity of 1,050 m^3/hr.

Note: *The ECS / LWP discharge capacity is 1,250 m^3/hr but also includes 200 m^3/hr of process condensate from the Plant. The SWP does not currently have an overflow spillway (during operation) but will be breached during the closure works for the post-closure period.*

The Liquid Waste Pond (LWP) is located adjacent to the SWP and receives treated water from the ECS and conditions this water (cooling and settlement) prior to discharging to one of the following:

- Controlled discharge into the River Shannon;
- Onto the surfaces of the BRDA by sprinkling during dry and windy weather, typically periodically during April to September; and/or
- Directly into the SWP if the effluent quality is off-specification i.e., recirculation of treated water.

The LWP does not currently have an overflow spillway (during operation) but will be breached during the closure works for the post-closure period. The current BRDA water inventory targets are presented below:

- **Winter** (October – March): 110,000 m^3 to ensure water storage capacity for stormwater.
- **Summer** (May – August): 180,000 m^3 to provide sufficient pre-treatment water storage for processing by the ECS prior to discharge for BRDA dusting prevention.
- **Transition Months** (April and September): 150,000 m^3.

Note: *The existing BRDA water inventory definition includes water stored in the PIC system and the SWP but does not include water stored in the LWP.*

6.12.1 Perimeter Interceptor Channel

The PICs are separated into PIC segments (PIC-A to PIC-G and PIC-J to PIC-M) that are separated by culverted ‘choke points’; these culverted sections provide vehicular access to the BRDA across the PICs. The Drawings showing the BRDA labelled water management system and the cross-sections for the respective PIC segments are provided in Appendix I.

There are 6 no. Phase 1 PICs segments that collect runoff from the Phase 1 BRDA.

- From the southwest corner of the Phase 1 BRDA, water flows clockwise through PIC-E, PIC-F and PIC-G, over a distance of ≈ 1,700m, to a sump located at the eastern extent of PIC-G, where water is pumped to the ECS and/or to the SWP via Pump 15 and Pump 33 / Pump 34, respectively.

  The constructed clockwise Phase 1 PIC segments have an upper crest width varying from 21.5m to 26.0m and base channel widths of 6.0m to 7.0m. The base elevations vary from 1.8 mOD to 0.9 mOD, the crest elevation is 4.7 mOD, and the operating freeboard is 0.5m.

- From the southeast corner of the Phase 1 BRDA Extension, water flows counter-clockwise through PIC-L, PIC-K and PIC-J, over a distance of ≈ 1,130m, to the sump located at the eastern extent of PIC-G.

  The constructed counter-clockwise Phase 1 PIC segments have an upper crest width varying from 20.0m to 25.0m and base channel widths of 4.0m to 21.0m. The base elevations vary from 15.7 mOD to 0.9 mOD, and the crest elevation varies from is 16.0 mOD to 4.7 mOD.

- The combined capacity of the constructed Phase 1 BRDA PIC is ≈ 116,500 m^3 at 0.5m freeboard and ≈ 155,500 m^3 at crest.

There are 5 no. Phase 2 PIC segments that collect runoff from the Phase 2 BRDA:

- From the north-east corner of the Phase 2 BRDA, water flows clockwise through PIC-A, PIC-B, PIC-C and PIC-D, over a distance of ≈ 2,140m, to a sump located at the northern extent of PIC-D, where water is pumped via Pump 24 into the Phase 1 BRDA PIC at the southern extent of PIC-E. There are also three overflow culverts installed which permit gravity flow from PIC-D to PIC-E, in the event of pump failure. The IPW for PIC-A has only been constructed during 2020, as the bauxite residue deposited in this sector attained the design elevation for the base of the channel, and the culverted connection to PIC-B is scheduled to be constructed during Q3 2021.

  The constructed counter-clockwise Phase 2 PIC segments have an upper crest width varying from 18.0m to 27.0m and base channel widths of 7.0m to 15.0m. The base elevations vary from 11.5 mOD to 1.0 mOD, the crest elevation varies from is 12.0 mOD to 5.0 mOD.

- At the northeast corner of the Phase 2 BRDA, PIC-M will flow counter-clockwise to connect with PIC-L, located at the southeast corner of the Phase 1 BRDA Extension. PIC-M is not yet constructed as the bauxite residue has not attained the design elevation of for the base of the channel. It is expected that PIC-M will be formed during 2022 / 2023.

  The design for PIC-M has an upper crest width of 13.5m and a base channel width of 5.5m. The base elevation varies form 13.5 mOD to 14.0 mOD, and the crest elevation varies from 16.0 mOD to 19.0 mOD.

- The combined capacity of the constructed Phase 2 BRDA PIC is ≈ 74,000 m^3 at 0.5m freeboard and ≈ 95,500 m^3 at crest.

6.12.2 Storm Water Pond

The SWP occupies a footprint of circa 6.0 ha. in the north-east sector of the Phase 1 BRDA and its layout is shown on Drawing 03.

The crest elevation of the SWP is 6.0 mOD along all its boundaries i.e., along the north dam wall bordering with the Bird Sanctuary, along the east dam dividing wall between the SWP and the LWP, along the south dam wall between the SWP and PIC-K (which doubles as the Central Access Ramp to the Phase 1 BRDA) and along the west dam dividing wall between the SWP and PIC-J. The north dam wall is the only external dam.

The base grades from ≈ 2.5 mOD at the south-west corner to ≈ 1.0 mOD at the north-east corner, where the extraction sump and fixed jetty are located.

The volumes retained by the SWP during operations and at maximum capacity are tabulated below.

Table 16: SWP Water Inventory

Facility	Operating Volume [(a)] (m³)	Freeboard Volume [(b)] (m³)	Total Volume (m³)	Crest Elevation (mOD)
SWP	182,000	58,000	240,000	6.0 mOD

Notes:

a) *The operational freeboard is 1.0m. Operational Volume is measured from base to 5 mOD.*

b) *Freeboard Volume is the additional 1.0m depth storage above the 100% Operating Volume i.e., from 5 mOD to 6 mOD*

6.12.3 Liquid Waste Pond

The LWP occupies a footprint of circa 1.8 ha. in the north-east sector of the Phase 1 BRDA and its layout is shown on Drawing 03.

The crest elevation of the LPW is 6.0 mOD along all its boundaries i.e., along the north dam wall bordering with the Bird Sanctuary, along the west dam dividing wall between the LWP and the SWP, along the south dam wall between the LWP and PIC-K (which doubles as the Central Access Ramp to the Phase 1 BRDA) and along the east dam wall which is at the toe of the outcropping bedrock. The north dam wall is the only external dam.

The base grades from ≈2.5 mOD at the south to ≈1.25 mOD at the north, where the extraction pumps on a floating jetty is located and a direct discharge pipe exits through the north dam wall.

The LWP is not managed as a minimum level pond, it is managed at an operating level to:

- Provide a reserve for sprinkling operations during the summer period; and
- Provide a cooling body for treated water entering prior to discharge into River Shannon.

The volumes retained by the LWP during operations and at maximum capacity are tabulated below.

Table 17: LWP Water Inventory

Facility	Operating Volume [(a)] (m³)	Freeboard Volume [(b)] (m³)	Total Volume (m³)	Crest Elevation (mOD)
LWP	47,400	8,900	56,300	6.0 mOD

Notes:

a) *The operational freeboard is 0.5m. Operational Volume is measured from base to 5.5 mOD.*

b) *Freeboard Volume is the additional 0.5m in the LWP above the 100% Operating Volume i.e., from 5.5 mOD to 6.0 mOD*

6.13 Other Infrastructure

6.13.1 BRDA Access Routes

Drawings 01, 02 and 03 shows the layout for the BRDA and the labels for the primary access routes.

- The BRDA is accessed via the Plant Site and subsequently via the security barrier positioned on the tarmacadam surfaced Access Road to the BRDA which approaches the BRDA at its north-east corner at an elevation of ≈ 14 mOD. This is a two-way route of ≈ 8.0m width.
- The Access Road has a turn-off to the west, which ramps down to the north crest of the LWP, at 6 mOD and then continues counter-clockwise along the crest of the north wall of the SWP. The road then ramps down slightly to the crest of the OPW at 4.7 mOD. or the Perimeter Access Road.
- The Perimeter Access Road encompasses the BRDA, the SWP and LWP and is a tarmacadam surfaced road with crash barrier on both sides, constructed on the crest of the OPW for the PIC and has a length of circa 5.2 km. This is a one-way route of ≈ 6.0m width and has a number of passing points. The Perimeter Access Road varies in elevation from 4.7 mOD along the north and west extents of the Phase 1 BRDA, to 5.0 mOD along the west extent of the Phase 2 BRDA, before it ramps up to 12 mOD at the south-east corner of the Phase 2 BRDA. The Perimeter Access Road continues north and ramps up to 16 mOD at the junction of the Phase 2 BRDA and the Phase 1 BRDA Extension, before ramping back down to 14 mOD along the east extent of the Phase 1 BRDA Extension (East Ridge Road). The Perimeter Access Road ends with a turn to the west back onto the Access Road to the BRDA.
- The Access Road continues to the south and ramps down to the south crest of the LWP, at 6 mOD, and then continues west along the crest of the south wall of the SWP and transitions into the Central Discharge Ramp, which is the primary access route for the Phase 1 BRDA and is a two-way internal rock fill surface ramp leading to the Phase 1 BRDA Discharge Platform and the SCDC.
- There are several crossing points from the Perimeter Access Road, over the PIC, into the BRDA and are rock fill surfaced routes that ramp up the side-slopes of the BRDA. There are 3 No. access ramps that enter the Phase 1 BRDA; North Ramp, North-West Ramp and the Central Discharge Ramp. There are 5 No. access ramps that enter the Phase 2 BRDA; West Ramp, South-West Ramp, South Ramp, South-East Ramp and East Ramp.
- Internal routes, crossing the BRDA and leading from the access ramps are constructed from rock fill and/or process sand. Vehicle access is also permitted on the crest of the constructed stage raise, which are 4m width, but have no crash barriers or berms.
- The East Ramp is the primary access route to the Phase 2 BRDA and leads to the Discharge Platform for the Phase 2 BRDA. The North-South Road spits the Phase 2 BRDA, constructed from Process Sand and is raised in elevation corresponding to the BRDA, and joins the East Ramp at its northern extent and Perimeter Access Road at its southern extent. The North-South Road varies from 11m to 13m width, has berms constructed along its crests and has a length of ≈ 900m.
- The Phase 2 Viewing Area or Observation Area is located along the Perimeter Access Ramp in the south-east sector of the Phase 2 BRDA.
- The Stockpile Yard is located to south-east of the Phase 2 BRDA and is accessed via a security gate in the perimeter fencing.
- A monitoring road is constructed at the downstream toe of the OPW for the extent of the Phase 2 BRDA, ramping down the west junction of the Phase 1 and 2 BRDAs and ramping back up at the east junction of the Phase 1 and Phase 2 BRDAs.

6.13.2 BRDA Sprinkler System

The surface of the BRDA is proactively managed by a network of sprinklers which cover the entire exposed bauxite residue surface on an approximately 75m x 75m grid. The sprinkler guns rotate and shoots water out at varying force (up to 50m radius) such that adjacent points in the grid form overlapping radii (max. 25m) to provide complete coverage. The Phase BRDA sprinkler system was installed in 2001 and the Phase 2 BRDA system was installed at the base of facility during construction. Figure 37 shows the current layout of the sprinkler lines for the BRDA and the sprinkler locations are shown on layout Drawings provided in the EIAR.

Figure 37: BRDA Sprinkler Line Layout (December 2020)

The sprinkler extension pipes are raised periodically, corresponding to the increase in elevation of the bauxite residue. Access to the sprinkler points is over deposited bauxite residue at varying stages of this mud-farming activities and is conducted using either the amphirol or the floating excavator, both of which have working platforms with handrails installed on their decks.

The sprinkler system is generally fed directly from the LWP with neutralized recycled water. During extended dry periods, the LWP provides a buffer storage for the sprinkler system. The sprinkler operational patterns and duration are decided daily based on an assessment of the of the weather forecasts and programmed by the BRDA Operations Department. In full operation, the sprinkler system can discharge at a rate of 650 to 750 m^3 / hour.

The Perimeter Access Road and internal road and ramps in the BRDA are swept clean using road sweepers and dust suppression is achieved using tractor pulled water bowsers.

6.13.3 Salt Cake Disposal Cell (SCDC)

Salt cake is a by-product of the process of purification of the caustic soda liquor used in the alumina extraction process from the bauxite ore and accounted for ≈ 1.0% or 15,300 tonnes during 2020 of the annual bauxite residue volume that is stored in the BRDA. Salt cake has a high concentration of caustic soda, is classified as a hazardous waste, and is required to be segregated from the other bauxite residues stored within the BRDA.

Since 2013, AAL have stored salt cake in an independent, composite lined, circa 1 ha. cell within the Phase 1 BRDA Extension (see Figure 1). Drawing 07 shows the salt cake disposal cell (SCDC) layout and the cross-sections. The SCDC has been constructed and later extended in footprint and elevation in three distinct phases:

- **2012 to 2013** – Phase 1 comprising a 'ring dam' composite lined cell constructed to a crest elevation of 24.0 mOD, over ≈ 18m depth of bauxite residue, which is also composite lined at the base of the Phase 1 BRDA Extension. The SCDC is triangular in shape (north, east and west dam walls), grades from a base elevation in the south corner of ≈ 20 mOD to the north-east corner at ≈ 19.0 mOD. A sump is located in the north-east corner, set a 1m below the cell base, and a decant tower and caustic recovery culvert system was constructed.
- **2016** – Phase 2 comprising a downstream raise of the Phase 1 cell constructed to a crest elevation of 26.0 mOD.
- **2018** – Phase 3 comprising a combined centreline and downstream raise of the Phase 2 cell constructed to a crest elevation of 29.0 mOD.

The SCDC is accessed from the Central Access Ramp to the Phase 1 BRDA, via a turn-off to the south onto the Access Ramp leading to crest of the SCDC. The salt cake is produced in the Plant and hauled to the SCDC in 40 tonne dumpers, where it is tipped into the cell at designated 'Tipping Points'. The process of loading, transporting and depositing the salt cake into the SCDC is managed by the AAL Standard Work Method (SWM) for Salt Cake Transportation and Storage (AAL 2021).

The west dam wall is the 'Tipping Wall' and has a width of ≈ 23.5m. The north and east dam walls are ≈ 8.0m width and they provide through access around the crest of the cell and to the Decant Tower. Crash barrier is installed at the outer and inner crests of the dam walls and rock fill berms are installed along the Access Ramp.

Salt cake partially dissolves in rainwater with the resultant leachate flowing through the perforated Decant Tower, which has a surround of non-calcareous filter rock fill, and subsequently by gravity flow via a culvert to a Caustic Leachate Transfer Facility located to the north and at a lower elevation on Stage 3. The leachate is pumped from here to the Plant.

AAL conducts monthly surveys of the SCDC to assess the remaining volume capacity. A sprinkler system is installed at the crest of the SCDC. AAL actively manage and clear the Tipping Points on the west dam wall, in accordance with the SWM (AAL 2021).

The total storage volume of the SDCD is estimated to be ≈ 72,800 m^3 at the crest (29 mOD). The remaining capacity at the end of April 2021 was estimated to be ≈ 12,500 m^3. The salt cake cell volume consumption rate within the cell is approx. 6,000 m^3/year i.e., salt cake is dissolved, and the caustic liquor is recovered, and the net rate of infill is circa 6,000 m^3 / year, meaning that the current cell capacity is expected to expire during 2023.

AAL are in the process of developing a Salt Cake Wet Oxidation Plant (SWOP) which will be located within the Plant, with the objective of removing salt cake from the waste stream.

7.0 BRDA RAISE DESIGN ASSESSMENTS

The following assessments were undertaken to support the design of the BRDA Raise.

- Geotechnical Analyses
- Seepage and Water Quality at Closure Assessment
- Water Balance and Hydrological Assessment

Summaries of these assessments are provided in the Section below, and the detailed assessments are provided in the Appendices.

7.1 Geotechnical Analyses

The following geotechnical analyses were undertaken to support the design of the BRDA Raise:

- Seismic Liquefaction Assessment (foundation soils and bauxite residue);
- Stability Assessment;
- Blast Assessment;
- Consolidation Assessment; and
- Breach Analysis.

7.2 Seismic Liquefaction Assessment

The seismic liquefaction assessment for the BRDA is provided in Appendix C and comprises an assessment of the seismic liquefaction potential of the underlying estuarine soils and the bauxite residue. A summary is provided below.

7.2.1 Methodology

The assessment has been undertaken according to the following procedure:

- Initial screening assessment of liquefaction susceptibility, based on Atterberg Limits, to determine if further analyses is required (Bray and Sancio 2006). This initial screening assessment is based on the in-situ material properties and does not consider the earthquake magnitude.
- If determined to be susceptible to liquefaction, a seismic liquefaction assessment was undertaken based on the design earthquake event to determine the **cyclic stress ratios (CSRs**) and the **cyclic resistance ratios (CRRs).**

7.2.2 Initial Screening

The initial liquefaction screening for the underlying estuarine deposits showed that the material would not be susceptible to liquefactions due its relative high PI. This outcome combined with the relatively low seismicity in Ireland, would direct that the estuarine deposits are regarded as not susceptible to seismic liquefaction and no further assessment was required.

The initial liquefaction screening for the bauxite residue showed that the material generally plotted in the range of moderate susceptibility. Further analyses were therefore required to establish liquefaction potential.

7.2.3 Bauxite Residue Liquefaction Assessment

In the liquefaction triggering analysis, the earthquake induced **CSRs** were compared to the **CRRs** to determine whether or not the bauxite residue will liquefy under the design earthquake loading. The design earthquake is 1 in 2,475-year event, Magnitude 5.0 with a peak ground acceleration of 0.05g and epicentre within 1km of the BRDA (Golder 2019A).

The **CSR** induced by the target magnitude earthquake was determined based on two methods:

- Seed simplified approach (Seed and Idriss 1971).
- 1-D and 2-D site response (shake) analyses using the finite element modelling software Quake/W, requiring the input of appropriately scaled earthquake records, was undertaken on representative sections to confirm the interpreted CSR from the Seed simplified approach.

The shake analyses were undertaken on two CPTu profiles determined during the 2018 CPT Investigation (Golder 2018), at the north-east sector of the Phase 1 BRDA Extension (at GA18-10C and GA18-10D, where the Seismic CPTu were conducted) and at the north-east corner of the Phase 1 BRDA (GA18-2A and GA18-2B, where the depth of estuarine soils was the deepest).

The CSRs determined from the three (3) methods (Seed simplified, 1-D and 2-D) are shown to vary with depth (confining pressure), with the stress induced by the earthquake greater near the surface where the confining stress is less. The results from the three (3) methods correlated reasonably well and allowed upper and lower bound values to be determined, generally a CSR of approx. 0.06 at the base of bauxite residue and approx. 0.08 at the mid-depth of bauxite residue.

The **CRR** is the capacity of the soil to resist liquefaction and has been determined based on two approaches:

- **The NCCER method**, which calculates the CRR from the tip resistance from the seismic CPTu cone, and applies adjustment factors based on the fines content, earthquake magnitude, in-situ stress level, and sloping ground.
- **State Parameter Approach** which is based on the correlation between state parameter and CSR as determined from the cyclic DSS testing (Golder 2018).

The same two CPTu profiles used for the CSR analyses were used in the determination of the CSR.

The results of the two (2) methods showed that they correlated reasonably well for the unfarmed bauxite residue (mid-depth and full depth) but that the interpretation of CRR from the state parameter approach is generally higher for the farmed bauxite residue and lower for the unfarmed bauxite residue.

The CRR from the state parameter approach is considered more representative as it is incorporated laboratory cyclic testing and the state of the bauxite residue. Generally, a CSR of 0.3 to 0.5 was determined in the upper farmed bauxite residue, a CSR of 0.1 to 0.2 at mid-depth and a CSR of 0.11 at the base of the bauxite residue.

The factor of safety (FoS) against liquefaction is the ratio of CRR over CSR. A FoS of greater than unity is required (Bulletin 139, ICOLD 2011) i.e., CRR > CSR.

The CSRs and the CRRs were plotted on the two CPTu profiles and the average FoS against liquefaction for the farmed and unfarmed bauxite residue is summarised in the Table 18.

The probability of liquefaction based on the factor of safety can be determined based on the relationship defined by Juang et. al. (2001).

Table 18: Factor of Safety and Seismic Liquefaction Probability (1 in 2,475 event, PGA = 0.05g)

CPTu Location	Bauxite Residue Description	Factor of Safety	Probability of Liquefaction		
			Probability	Probability of Occurrence	Description [a]
GA18-10C	Farmed	3.9	< 0.01	$< 4.0 \times 10^{-6}$	Almost Impossible or Negligible
	Unfarmed	1.6	0.1	$< 4.0 \times 10^{-5}$	Highly Improbable
GA18-10D	Farmed	4.7	< 0.01	$< 4.0 \times 10^{-6}$	Almost Impossible or Negligible
	Unfarmed	1.0	0.3	1.2×10^{-4}	Very Unlikely
GA18-2B	Farmed	6.1	< 0.01	$< 4 \times 10^{-6}$	Almost Impossible or Negligible
	Unfarmed	1.6	0.08	3.2×10^{-5}	Highly Improbable
GA18-2A	Unfarmed	2.7	< 0.01	$< 4 \times 10^{-6}$	Almost Impossible or Negligible

Notes:

a) *Description from Juang et. al. (2001)*

b) *Based on comparison with the Shake analysis using the Saguenay earthquake*

7.2.4 Summary and Discussion

The estuarine deposit has been screened to be not susceptible to liquefaction due to its relative high plasticity index.

The seismic liquefaction analyses for the bauxite residue meets the minimum required FoS of greater 1.0 against triggering liquefaction (ICOLD 2011) for the design earthquake event. The probability of liquefaction, based on the FoS for bauxite residue, is in the Highly Improbable to Almost Impossible range of probability of triggering liquefaction during the design earthquake event for the liquefaction analyses undertaken.

A small area adjacent to the SCDC where the bauxite residue is shown to be softer (GA18-10D), has a Very Unlikely potential to liquefy. This area was further investigated during the 2019 CPT investigation by conducting five (5 No.) additional CPTu locally and was determined to be a localized soft spot (Golder 2020). The CPTu soundings returned lower bound s_u/σ'_{v0} values ranging from 0.15 to 0.18 and further stability analyses were undertaken (Golder 2020) which returned satisfactory FoS values.

A sensitivity assessment was undertaken to determine the FoS for the BRDA for larger earthquake events (greater magnitude than 5.0) with greater return periods (> 1 in 2,475-year return period) and also with an epicentre within 1km of the BRDA.

The CPT sounding at GA18-2B was considered a reasonable estimate of the average bauxite residue properties around the BRDA facility to conduct the assessment. The following FoS values were returned:

- A PGA of 0.07g (1 in 5,000-year return period) results in an average factor of safety of 1.12 for the unfarmed bauxite residue, with very isolated layers showing a potential for liquefaction.
- A PGA of 0.08g (1 in 7,000-year return period) reduces the average factor of safety to 0.98, for the unfarmed bauxite residue, with more defined layers showing a potential for liquefaction.

- A PGA of 0.09g (1 in 10,000-year return period) further reduces the average factor of safety to 0.87, for the unfarmed bauxite residue, with liquefaction possible.

However, these greater PGA values would require a larger earthquake than a Magnitude 5.0. The HSE document, Seismic Hazard: UK Continental Shelf (HSE 2002) provides contour maps for UK and Ireland and a zonation model which lists the south-west coast of Ireland (zone A13) as an area with an earthquake magnitude observation threshold of 5.0.

7.3 Stability Assessment

The stability assessment for the BRDA to Stage 10 and Stage 16 is provided in Appendix D and a summary is provided below.

The BRDA has been divided into sectors which have similar foundation conditions, bauxite residue deposition characteristics and side-slope profile. These sectors are named based on their location e.g., North-East sector in the Phase 1 BRDA and vary in width around the perimeter of the BRDA, but are typically in the 200m to 350m width range. Stability sections lines have been assigned to each sector and monitoring instrumentation is installed along the alignment of the stability section lines on the side-slopes at designated elevation intervals as the BRDA is raised.

The stability sections assessed comprise the following and are shown on Drawing 12 and in Appendix M.

- **Phase 1 BRDA:** Section A-A, Section B-B, Section C-C, Section D-D, Section E-E, Section F-F, Section K-K and Section L-L.

  Note: *A number of stability sections were previously designated and assessed along the south face of the Phase 1 BRDA (Section G-G, Section H-H, Section I-I and Section J-J). This face is being merged with the Phase 2 BRDA and these stability sections are no longer assessed as the bauxite residue deposition provides a buttress for the slope. The monitoring instruments remaining in this sector are still read on a quarterly basis and the readings are assessed and reported in the quarterly memos and annual review.*

- **Phase 2 BRDA:** Section M-M, Section N-N, Section O-O, Section P-P, Section Q-Q, Section R-R, Section S-S, Section T-T, Section U-U, Section V-V, Section W-W and Section X-X.

  Note: *A number of stability sections in the Phase 2 BRDA have similar foundation conditions and will have bauxite residue deposition characteristics and side-slope profile when constructed, hence these stability Sections have been bundled into groups for analyses.*

7.3.1 Methodology

The stability analyses for the BRDA Raise Development were carried out using the limit equilibrium modelling software SLOPE-W Version 10.0.0.17401. The analytical method used was Morgenstern and Price method of slices, which satisfies both force and moment equilibrium. The analyses include both drained (effective stress) and undrained (total stress) strength conditions within the bauxite residue and the estuarine deposits

The stability models for each sector are constructed based on the stratigraphy identified by the CPTu profiles. The phreatic surfaces for the stability models constructed to Stage 16 are determined by using the current measured phreatic surfaces for the Phase 1 BRDA to Stage 10, which were then replicated using SEEP-W to assign hydraulic conductivity values for the farmed and unfarmed bauxite residue and subsequently modelled for the BRDA constructed to Stage 16.

The phreatic surfaces for the Stage 10 analyses conducted for the Phase 1 BRDA were determined by using the current measured phreatic surfaces.

The stability of each sector of the BRDA is analysed for the cases and to the FoS criteria listed in Table 19 below.

Table 19: Factor of Safety Criteria for the BRDA Raise Development based on International Guidelines

Loading Condition	Recommended Factor of Safety	
	CDA (2014)	ANCOLD (2012)
Short Term, Undrained (Total Stress)	Greater than 1.3 During, at, or end of Construction, depending on Risk Assessment	1.5 If loss of containment, Consolidated Undrained Strength
Global Slope [1]		
Upper Slope [2]		
Middle Slope [3]		
Lower Slope [4]		
Long Term, Drained, Steady State (Effective Stress)	1.5 Steady State, Phreatic Level	1.5 Effective Strength
Global Slope [1]		
Upper Slope [2]		
Middle Slope [3]		
Lower Slope [4]		
Pseudo-Static, Undrained (Total Stress)	1.0 [5]	Not required
Global Slope [1]		
Post-Earthquake, Undrained (Total Stress)	1.2 [5]	1.0 to 1.2 (residual undrained shear strength)
Global Slope [1]		

Notes:

1. *Global Slope is from the downstream toe of the OPW to Stage 16*
2. *Upper Slope is from Stage 10 to Stage 16*
3. *Middle Slope is from Stage 5 to Stage 10*
4. *Lower Slope is from the downstream toe of the OPW to Stage 16*
5. *Undrained shear strength values were reduced by 20% to allow for cyclic softening (Hynes and Franklin 1984) for pseudo-static and post-earthquake analyses.*

A minimum FoS of 1.5 is considered required for all static, long term, drained analysis. A reduced factor of safety of 1.3 may be considered acceptable for the short-term undrained condition following embankment construction, provided sufficient understanding of the material strength parameters and their behaviour exists, and an appropriate risk assessment has been undertaken.

The drained (effective stress) condition, which represents the standard ‘long term’ condition has been included in the current analyses. This condition would represent loading and shearing of the bauxite residue at a slow enough rate to limit the build-up of excess pore pressure, and typically produces a higher FoS and hence is not considered the critical case.

The analysis for undrained (total stress) condition within the bauxite residue is considered the critical case. While in general geotechnical terms and for other more free-draining tailings this is considered the ‘short term’, for the bauxite residue this represents a ‘long term’ condition that requires a minimum FoS of 1.5.

This total stress condition is considered the critical case as:

- An undrained condition for a material in a contractive state (unfarmed bauxite residue), generates excess pore pressure and results in a lower effective shear strength less than in the drained condition.
- The undrained condition when the material is in a relatively dense/stiff condition (farmed bauxite residue), dilates during shearing, generates negative pore pressure and may result in an effective shear strength greater than in the drained condition.

For the pseudo-static analysis, the coefficient of horizontal ground acceleration of 0.025 g (50% of PGA) was applied representing the 2,475-year return period earthquake, along with 20% strength reduction of the material strength parameters, as per the recommendations of Hynes-Griffin and Franklin (Hynes and Franklin 1984).

7.3.2 Parameter Selection

Geotechnical strength parameters can have a wide range, have a high likelihood of outliers, and are typically dependent on the selection of other parameters for their interpretation. The interpretation and determination of the geotechnical parameters can have a significant influence on the resulting FoS for a given stability model. The following methodology was adopted for the analyses of the stability sections.

- The undrained shear strength ratio (s_u/σ'_{v0}) is a key input for the stability analyses and is determined from the correlation of interpreted CPTu undrained shear strength with laboratory test data (primarily DSS test data). A summary of CPTu interpreted data for each stability section is provided in Appendix D-3.

  Mayne 2016 suggests that the DSS test is the most appropriate test to use when correlating the interpreted undrained strength from CPTu data as it presents undrained strength results that fall more-or-less mid-way between the other test modes (compression and extension) and thus provides an 'average' result. The undrained strength, s_u ,depends on the effective confining stress (σ'_{v0}) prior to shearing. The CPTu data is used to interpret the undrained shear strength (s_u) using the undrained strength factor, N_{kt}, and the following relationship : **s_u = (Net cone resistance) / N_{kt}**

  - An **N_{kt} value of 14** has been selected for the bauxite residue following correlation with laboratory testing and review of shear vane testing data.
  - Previously an N_{kt} value of 15 was used for the estuarine deposits which provided a reasonable estimate of the undrained shear strength ratio profile. The N_{kt} value currently adopted for interpretation of the undrained strength of the estuarine deposits is variable and is based on the normalized excess pore pressure parameter (B_q) which is measured during the CPTu sounding and reflects the permeability of the material it is passing through i.e., higher for clay soils and lower for silty / sandy soils. A trend line developed for Irish Clays based on, $\boldsymbol{B_q}$ (Long 2018).

  $$\boldsymbol{N_{kt} = 7.82\, B_q^{-0.65}} \quad \textbf{for Irish Clays.}$$

  The B_q value in the silty clay layer varies from 0.3 to 0.45 and returns N_{kt} values between 13.1 and 17.1.

- The stability model in SLOPE-W requires the input of a single undrained strength ratio (s_u/σ'_{v0}) value for each material layer. These single values are termed design or characteristic values.

  The characteristic value for the geotechnical strength parameters for use in the deterministic stability calculations is recommended to be selected to provide a high level of confidence that the measured values will be greater than the characteristic value. The confidence % (or equivalent percentile / fractile) of the characteristic value should be combined with the Factor of Safety (FoS) to determine the 99% exceedance probability (Been and Jefferies 2016) e.g., a 70% confidence value (or 30th percentile) combined with a FoS = 1.45 would provide the desired 99% exceedance probability. A range has been selected for the characteristic strength parameters as the value is determined for each stability section and layering within that stability section based on the interpreted CPTu strength, which is validated by laboratory testing of samples taken at the section and layer, where available.

Geotechnical index properties (i.e., dry density, bulk density, moisture content) typically have a narrower range and a lower likelihood of outliers for a particular soil type or tailings stream and the mean value is typically selected for the characteristic value, which is ≈ 50th percentile. Combined with a FoS = 1.45, would provide a 72.5% exceedance probability for measured values.

The characteristic undrained shear strength parameters selected are the 30th percentile for the estuarine deposits and the 10th percentile for the estuarine deposits the bauxite residue (farmed and unfarmed).

7.3.3 Phase 1 BRDA Stability Analyses Results

Stability analyses were conducted on select critical and representative stability Sections for the Phase 1 BRDA constructed to Stage 10, i.e., Section A-A, Section B-B, Section C-C, Section E-E and Section F-F. These stability Sections were re-assessed using the methodology described above as they are considered the critical stability sections for the Phase 1 BRDA. Section K-K and Section L-L are assessed on an annual basis and return values that are comfortably in exceedance of the FoS target criteria.

The geotechnical parameters selected for the estuarine deposits (where present) and the bauxite residue at each stability section have been determined following assessment of the field investigation data comprising in-situ testing, sampling, laboratory testing and interpretation by others prior to 2004 and by Golder after 2004.

Section B-B, Section C-C and Section E-E returned values for FoS in compliance with the target criteria listed in Table 19 for the BRDA constructed to Stage 10. Both Section A-A and Section F-F were determined to require toe buttresses to be constructed to meet the FoS target criteria. The toe buttresses for Section A-A and Section F-F have been constructed.

The stability analyses for the relevant stability Sections of the Phase 1 BRDA constructed to Stage 16, i.e. Section A-A (with toe buttress), Section B-B, Section C-C, Section D-D, Section E-E, Section F-F (with toe buttress), Section K-K and Section L-L, all returned FoS in compliance with the target criteria listed in Table 19.

7.3.4 Phase 2 BRDA Stability Analyses Results

Stability analyses were conducted on select critical and representative stability Sections for the Phase 2 BRDA constructed to Stage 16, i.e., Section N-N, Section P-P, Section R-R, Section T-T and Section V-V. These stability Sections were re-assessed using the methodology described above as they are considered the critical stability Sections for the Phase 2 BRDA.

- Section N-N is considered representative of Section M-M, Section N-N and Section O-O.
- Section P-P is considered representative of Section P-P and Section Q-Q
- Section R-R is considered representative of Section R-R and Section S-S
- Section V-V is considered representative of Section U-U, Section V-V, Section W-W and Section X-X

The geotechnical parameters selected for the estuarine deposits (where present) and the bauxite residue at each stability section have been determined following assessment of the field investigation data comprising in-situ testing, sampling, laboratory testing and interpretation by others prior to 2004 and by Golder after 2004.

All stability sections analysed returned FoS in compliance with the target criteria listed in Table 19 for the Phase 2 BRDA constructed to Stage 16, with no extra measures required e.g., buttressing.

7.3.5 Storm Water Pond and Liquid Waste Pond

The BRDA Raise Development does not require any adjustment, raise or increase in footprint to the structures of the Storm Water Pond (SWP) or the Liquid Waste Pond (LWP).

The stability analyses for the SWP and the LWP were mostly recently undertaken in March 2019 as part of the Risk Assessment for the BRDA (Golder 2019). The summary table of the stability analyses is provided in Table 20 below and the full assessment from March is included in Appendix D-2.

Table 20: SWP and LWP Stability Analyses Results

Sector	Slip Surface	Static Factor of Safety		Post-Earthquake Factor of Safety
		Effective Stress Analysis (Drained)	Total Stress Analysis (Undrained)	Total Stress Analysis (Undrained)
Sector H	SWP into Bird Sanctuary	1.9	1.6	1.3
Sector I	SWP into PIC	1.5	1.6	1.3
Sector J / Sector L	SWP into LWP LWP into SWP	2.1	1.9	1.5
Sector K	LWP into Bird Sanctuary	1.8	1.6	1.2
Sector M	LWP to PIC	1.7	1.3	1.2

- Based on the criteria presented in Table 1, Golder has classified the SWP and the LWP as dams having a **"Low**" HPC. The target levels for standards-based design criteria for tailings dams with a '**Low'** HPC would require a minimum static factor of safety of 1.3 for the during construction / end of construction period (short term) and 1.5 for the long term.
- For both the long-term conditions (drained and undrained), the factors of safety for all structures are considered adequate.
- All of the pond Sectors (SWP and LWP) returned FoS > 1.5 for the undrained analysis except for Sector M which returned an FoS < 1.5, however it does attain the original design FoS for this structure (1.3) and the long-term drained analysis does attain a FoS > 1.5.

7.3.6 Perimeter Interceptor Channel - Outer and Inner Perimeter Walls

The BRDA Raise Development does not require any adjustment, raise or increase in footprint to the structures of the Perimeter Interceptor Channel (PIC). However, it is proposed to provide additional storage capacity with the PIC system to manage the IDF event by extending the crest lining system for a number of PIC segments (see Section 7.8.2). This vertical raise of the downstream crest liner which will be supported by the existing crash barrier.

The stability analyses for the outer perimeter wall (OPW) and the inner perimeter wall (IPW) were mostly recently undertaken in March 2019 as part of the Risk Assessment for the BRDA (Golder 2019).

The summary table of the stability analyses is provided in Table 20 below and the full assessment from March is included in Appendix D.

Table 21: OPW and IPW Stability Analyses Results

Sector	Slip Surface	Static Factor of Safety		Post-Earthquake Factor of Safety
		Effective Stress Analysis (Drained)	Total Stress Analysis (Undrained)	Total Stress Analysis (Undrained)
PIC	Downstream failure of OPW	1.8	1.8	1.5
PIC	Downstream failure of IPW	2.0 to > 2.0	1.5 to 1.6	1.5

- Based on the criteria presented in Table 1, Golder has classified the OPW and the IPW as dams having a **"Low**" HPC. The target levels for standards-based design criteria for tailings dams with a '**Low'** HPC would require a minimum static factor of safety of 1.3 for the during construction / end of construction period (Short Term) and 1.5 for the long term.
- For the long-term conditions (drained and undrained), the factors of safety for all structures are adequate.

7.3.7 Summary and Discussion

The stability analyses for the Phase 1 BRDA and the Phase 2 BRDA have returned FoS in compliance with the target criteria listed in Table 19 for the BRDA constructed to Stage 10 and to Stage 16. These target FoS criteria are consistent with the current international guidelines for tailings dam safety management and best practice

Stability analyses for the supporting structures for the BRDA i.e., the water management structures have returned FoS values that are in compliance with their target criteria.

7.4 Blast Assessment

Golder 2017A provides an assessment for the blasting associated with a proposed Borrow Pit development, potentially impacting on the embankments and raises associated with Phase 1 BRDA.

The proposed Borrow Pit Development is located to the north-east of the BRDA and a component of the BRDA Raise Development is a further eastern expansion of the Borrow Pit.

The Golder 2017A report presented a stability review of the potentially impacted sector of the BRDA, and included:

- An interpretation of the Peak Particle Velocity (PPV) expected to be caused by the blasting based on a review of previous blasting at Aughinish during the construction of the Phase 2 BRDA;
- Stability review of the BRDA based on the blast vibration and the potential generation of excess pore pressure; and
- Recommendations for conducting the blasting and monitoring during the Borrow Pit Development.

A summary of the key elements is provided below, and the full assessment is provided in Appendix E.

7.4.1 BRDA Blast Stability Assessment

The stability of the sector of the BRDA nearby to the proposed blasting was assessed with two approaches:

- **Pseudo-Static Stability Assessment** - analysing the stability of slopes subject to blast vibration. The design blasts are limited to produce a maximum PPV of approximately 25 mm/s. Stability results are presented in Table 22 for three different PPV values. The average PPV over the entire slope is not expected to exceed 15 mm/s.

  The pseudo-static loading condition requires a minimum FoS of 1.0 according CDA guidelines. For the post-blast condition, where the generation of excess pore pressure may have decreased the material shear strength, a minimum FoS of 1.2 is recommended by CDA guidelines, and this was similarly applied to the pit-blasting analysis based on the residual excess pore pressure.

Table 22: Pseudo-Static Stability Analyses Results

Slip Surface	Static FoS	Pseudo-Static FoS		
		PPV = 15 mm/s	PPV = 20 mm/s	PPV = 25 mm/s
Overall Slope	1.6	1.1	1.1	1.0
Upper Slope	1.6	1.3	1.2	1.1
Lower Slope	1.6	1.2	1.1	1.1

Notes:

1. *FoS reported to one decimal place as is the industry standard*
2. *Results are based on total stress (undrained) analysis*
3. *Lower slope stability results are based on a slip surface depth of approximately 16 m.*

- **Post-Blast Analyses** - simulating the excess pore pressure in saturated soil which can potentially be generated by nearby blasting operations. One of the issues of conducting blasting nearby the BRDA is the potential for blast-induced residual pore pressure increases that reduce the shear strength for a time period long enough to allow gravity to cause the instability of the slope. The design blast is limited to produce a maximum PPV of approximately 25 mm/s. Three stages of explosive-induced pore pressure typically occur:

  - the peak transient pore pressure increase, which is directly associated with the passage of the compressive stress wave;
  - the residual pore pressure increase, which is induced by the passage of the stress wave but occurs after the passage of the stress wave; and
  - the residual pore pressure dissipation stage, which occurs as the soil consolidates.

  The residual pore pressure increase is the critical condition to be analysed as the peak transient pore pressure increase is a temporary increase and dissipates to the residual pore pressure relatively quickly.

  The slope/W software used for the analysis has two functions which allow excess pore to be analysed, these include the r_u coefficient and the B-bar coefficient. Both were used in the analysis and found to produce a similar result and represent the average pore pressure ratio (PPR) value. Typical values for the r_u encountered in practice range from 0.0 to 0.7 and it has been established that a linear relationship between FoS and r_u applies over this range. The threshold value for r_u is typically 0.3 and reflects a drop-off in FoS below minimum standards. Stability results are presented in Table 23 which provides a summary of the FoS based on the PPR, and the analysis results. Excess pore pressure is only assumed to be generated within the unfarmed bauxite residue.

The PPV generated by a nearby blast was determined to need to exceed 35 mm/s to generate sufficient excess pore pressure to reduce the FoS below the recommended 1.2 for post blast condition.

Table 23: Post-Blast Slope Stability Analyses with Excess Pore Pressure

r_u Coefficient	Average Pore Pressure Ratio (PPR) [a]	Factor of Safety (FoS) [b]	Equivalent PPV to produce Δu_{peak} (mm/s) [c]	Equivalent PPV to produce Δu_{res} (mm/s) [d]
0.1	0.20	1.4	~ 15	~ 15
0.2	0.35	1.3	~ 25	~ 80
0.3	0.50	1.2	~ 35	~ 300

Notes:

a) *Excess pore pressure assumed in the unfarmed red mud*
b) *FoS for Section K, overall slope instability and total stress (undrained) analysis*
c) *Calculated using Jacobs (1988)*
d) *Calculated using Veyera (1885) with an unfarmed bauxite residue relative density of 40%*

7.4.2 Summary and Discussion

The stability analyses undertaken found that the blast analysed resulting in a maximum PPV of approximately 25 mm/s at the BRDA embankment would not cause instability of the BRDA, due to vibration of the blast itself (pseudo-static analysis) or as a result of residual excess pore generated by the blast wave (post-blast analysis). This is consistent with the observations reported in case histories.

AAL received a Board Direction (BD-001560-18) to grant planning permission (Reference ABP 301-101118) from An Bord Pleanála in November 2018. The operation of the Borrow Pit subsequently required a review of the IE Licence to include Conditions for the operation of the Borrow Pit i.e., noise, vibration and air overpressure thresholds and monitoring locations. A new IE Licence (P0035-07) was issued by the EPA in September 2021, and it provides the Conditions for the operation of the Borrow Pit.

7.5 Consolidation Assessment

The consolidation assessment for the BRDA constructed from Stage 10 to Stage 16 is provided in Appendix F and a summary is provided below.

7.5.1 Methodology

The standard methods of comparing void ratios before and after loading and/or comparing void ratios at varying depths to estimate future settlement are not applicable for the unfarmed and farmed bauxite residue based on the site investigation data. The following two methods were conducted to provide an estimate for future settlement of the BRDA using the settlement tools provided by Civil Web (Version 01 March 2020).

- The coefficient of volume compressibility, m_v, is used to estimate the total consolidation and the coefficient of consolidation, c_v, is used to estimate the rate of consolidation.
- The CPTu measured tip resistance, q_c, is used to estimate the total consolidation (Robertson 2008 and Pishgah et al. 2013)

7.5.2 Parameter Selection

Sections 4.3.1 and 5.3.3 discusses the laboratory testing and the consolidation parameters established for the estuarine soils, the unfarmed and the farmed bauxite residue, and are summarized in Table 24 below.

Table 24: 1-D Oedometer Consolidation Parameters

Material	m_v (m^2 / MN)	c_v (m^2 / year) 50% consol.	c_v (m^2 / year) 90% consol.	Over Consolidation Ratio (OCR)
Estuarine Deposits (silty CLAY)	0.045 to 0.47	11 to 30	11 to 26	2 to 3.5
Estuarine Deposits (clayey SILT)	0.025 to 0.19	2.5 to 18	2.6 to 19	4 to 5
Unfarmed Bauxite Residue	0.30 to 3.00	1 to 8	3 to 32	≈ 1 (NC)
Farmed Bauxite Residue	0.020 to 0.081	33 to 96	34 to 100	3 to 7

Notes:

1. *The unfarmed bauxite residue is assumed to be normally consolidated*
2. *The farmed bauxite residue is considered artificially over consolidated as a result of the farming activities*

The coefficient of volume compressibility, m_v, is used to estimate the total consolidation and the coefficient of consolidation, c_v, is used to estimate the rate of consolidation. CPTu soundings have been conducted at many locations within the Phase 1 and Phase 2 BRDA and the average tip resistance values recorded, q_c, are summarized in Table 25 below.

Table 25: CPTu Consolidation Parameters

Material	q_c (MPa)
Estuarine Deposits (silty CLAY)	0.9
Estuarine Deposits (clayey SILT)	2.0
Unfarmed Bauxite Residue	1.0
Farmed Bauxite Residue	3.0

7.5.3 Summary and Discussion

The estimate settlement for the Phase 1 BRDA for the additional loading provided by the 12m raise from Stage 10 to Stage 16 is in the range of 380mm to 740mm at the location of the perimeter of Stage 16 at elevation 36 mOD and in the range of 555mm to 936mm at the location of the centre of the dome at elevation 44 mOD.

The estimate settlement for the Phase 2 BRDA for the additional loading provided by the 12m raise from Stage 10 to Stage 16 is in the range of 240mm to 315mm at the location of the perimeter of Stage 16 at elevation 36 mOD and in the range of 300mm to 475mm at the location of the centre of the dome at elevation 44 mOD.

It is expected that the final settlements will be the lower end of these scale based on the range of cumulative settlement to date and the thixotropic nature of the bauxite residue which may be partly restricting consolidation settlement and secondary settlement.

The largest expected settlement is in the unfarmed bauxite residue layer in the Phase 1 BRDA and, based on the c_v values the bulk of the settlement, can be expected to be complete during the deposition life of the BRDA (to 2039) leaving a minimal (< 100mm) long-term settlement in farmed bauxite residue layer.

7.6 Breach Analysis

A risk assessment update for the Bauxite Residue Disposal Area (BRDA) constructed to Stage 10 was undertaken by Golder (Golder 2019A). The assessment is considered appropriate for the BRDA constructed to Stage 16 as the BRDA footprint, the failure mechanisms and discharge pathways in a breach scenario remain unchanged. There is potential for increased volume of discharge and increased extent of discharge during a breach scenario due to the proposed increase in elevation of the BRDA to Stage 16 and these values has been reassessed. The breach analysis for the BRDA constructed to Stage 16 is provided in Appendix G and a summary is provided below.

7.6.1 Methodology

The estimated volume of bauxite residue that could potentially be released in a breach scenario has been assessed from the Tailings Flow Slide Calculator (WISE 2020) and from the size of the slope stability failures modelled in SLOPE/W (failure width is limited to the failure slope length). The extent of the tailings flow discharge or run-out distance has been assessed from the Tailings Flow Slide Calculator (WISE 2020).

7.6.2 Parameter Selection

The parameters inputted for the model are listed in Table 26 below and have been selected on historic laboratory testing, in-house AAL laboratory testing and in-situ shear vane testing.

Table 26: Model Parameters for Tailings Flow Slide Calculator (Wise-Uranium, Dec 2020 version)

Parameter	Selected Values
Geometry	
Initial height of BRDA Containment	35m above downstream elevation at Stage 16 (36 mOD)
Bed slope downstream of BRDA Containment	0 % at 1 mOD
Bauxite Residue Properties	
Unit Weight (Unfarmed and Farmed Bauxite residue)	21.5 kN/m^3
Bingham Yield Strength (Unfarmed Bauxite Residue)	4 kPa
Bingham Yield Strength (Farmed Bauxite Residue)	6 kPa
Bingham Plastic Viscosity (Unfarmed Bauxite Residue)	10 kPa.s
Bingham Plastic Viscosity (Farmed Bauxite Residue)	100 kPa.s

7.6.3 Summary and Discussion

The Phase 1 BRDA has a Very Unlikely (≈1 in 10,000) to Highly Improbable (≈1 in 100,000) annual risk of containment failure and Phase 2 BRDA has a Highly Improbable (≈1 in 100,000) to Almost Impossible (≈1 in 1,000,000) annual risk of containment failure These values are significantly less than the annual average probability of worldwide tailings dam failures based on statistical data (≈ 1 in 2,000), (Golder 2019A).

The water retaining structures (SWP, LWP and PICs) have an Unlikely (≈ 1 in 1,000) to Very Unlikely (≈1 in 10,000) annual risk of water release and is similarly less likely than the average probability of tailings dam failures based on statistical data (≈ 1 in 2,000), (Golder 2019A).

The impact of a breach scenario is largely dependent on the volume of material discharged and distance travelled by the material discharged. Both of these factors are dependent on the ability of the bauxite residue to liquefy. Where the bauxite residue is farmed, the material would slump rather than liquefy.

The estimated volume of bauxite residue that could potentially be released in a breach scenario has been assessed by two methods and the range is 40,000 m^3 to 90,000 m^3.

- Where the bauxite residue is farmed, the material would slump rather than liquefy. The distance travelled would be small, a distance of the order of 12.1m from the downstream toe of Phase 2 BRDA and into the Perimeter Interceptor Channel (PIC). Both the upper levels (above Stage 7) of the Phase 1 BRDA and all of Phase 2 BRDA would be expected to slump into the PIC or within ≈ 12m of the downstream toe.
- Where the material is potentially able to liquefy, which are confined to the lower slopes of the Phase 1 BRA to Stage 6 (16 mOD at perimeter to 20 mOD centrally), the distance travelled would be a maximum of 224m, although the presence of the PIC at the downstream toe may contain the flow even further, if intact. This run-out distance assumes that the farmed bauxite residue above the unfarmed bauxite residue also liquefies. If only the elevation of the unfarmed bauxite residue is considered, then the run-out distance is reduced to 52m.

The area between the Flood Tidal Defence Berm (FTDB) and the BRDA, Storm Water Pond (SWP) and Liquid Waste Pond (LWP) is at an elevation of approx. 1 mOD and has a footprint of ≈ 187,000 m^2, excluding the Bird Sanctuary, Special Protection Area (SPA) or Special Areas of Conservation (SAC) footprints and is therefore capable of retaining circa 750,000 m^3 of tailings and/or water provided that the FTDB at a crest elevation of 5 mOD remains intact.

In the event of a breach scenario resulting in bauxite residue flowing into the SWP and/or the PIC, the contaminant wastewater will be displaced and would flow via the open drainage network leading to the sluice gate valve in the West Drain (see Figure 39). AAL have installed a penstock valve on this sluice gate.

If the FTDB is breached due to a tidal surge, and a BRDA breach scenario occurred, the bauxite residue and containment wastewater would potentially be washed into the Robertstown and Shannon Rivers. However, the expected break-out volumes are relatively small.

7.7 Seepage and Water Quality at Closure Assessment

The seepage and water quality assessment for the BRDA constructed to Stage 16 and following closure is provided in Appendix H and a summary is provided below.

7.7.1 Model Parameters

The BRDA is comprised of multiple phases which contain:

1) Unfarmed bauxite residue (BR) on estuarine sediments for the original unlined Phase 1 BRDA, 72 ha
2) Unfarmed BR on composite liner for the Phase 1 Extension BRDA, 32 ha
3) Carbonated farmed BR on 1) and 2) since 2009
4) Carbonated farmed BR on composite liner for the Phase 2 BRDA, 80 ha

Upon closure, it is understood that the BRDA will be capped with a minimum 1m depth of amended bauxite residue, which comprises BR that has been mixed with neutralised process sand, gypsum, and organic material.

7.7.2 Methodology

The assessment comprised three (3) components:

- **Sampling and Seepage Water Quality Laboratory Testing:** The liquid present in the standpipe piezometers in the Phase 1 BRDA was sampled and tested as this liquid is considered the most direct analogue for seepage. This liquid represents older, unfarmed and unamended bauxite residue and is highly alkaline (pH circa 12 to 13) and has numerous trace metals present.

  In addition, leach testing in accordance with *EN 14405 Characterisation of waste – up-flow column percolation test* was carried out on the amended cap layer only, to assess the effects of the closure capping on infiltration and the rate of seepage through this layer. Metal analysis was carried out in the final 10:1 leachate from this test method.

- **Seepage Modelling:** A two-dimensional (2-D) numerical model was constructed in SEEP/W to provide an assessment of volume of potential seepage from the restored BRDA to Stage 16 into the encompassing perimeter interceptor channel (PIC) and through the base of the facility. The modelled design takes into consideration the changes in the lining system and material properties of the material deposited in the BRDA over time and the proposed restoration with grass at site closure.

- **Water Quality Assessment:** A mixing model was constructed to make a preliminary assessment of water quality in the Perimeter Interceptor Channel (PIC) upon closure. Geochemical mixing calculations were performed using water quality results from BRDA piezometers and leachate from amended layer (simulating runoff).

  The water quality predictions were completed using the geochemical code PHREEQC Version 3.3.7 (Parkhurst and Appelo, 1999). PHREEQC is a computer program that is used to simulate chemical reactions and transport processes in natural or contaminated water. The mixing model simulations performed with PHREEQC include aqueous speciation and saturation index modelling.

  The results of the water quality model included an evaluation of minerals capable of precipitating from solution that could control concentrations of parameters of potential environmental concern.

7.7.3 Summary and Discussion

During operation, all waters captured by the BRDA Water Management System i.e. PICs, SWP, Toe Drains, Abstraction Wells, plant side surface water drainage system etc. are returned to the Effluent Clarifier System (ECS) at the plant and subsequently re-used in the plant or BRDA operations or discharged at emission point W1-1 into the Shannon Estuary in accordance with the limit values listed in Schedule B: Emissions of Water of the licence issued by the EPA (IEL P0035-07). The bulk of this operational water comprises the water captured in the PIC and includes process water, irrigation water, seepage, and runoff.

Post-closure, it is anticipated that the PIC water will be discharged to local surface waterways encompassing the BRDA, via designated breach locations, subject to EPA licence amendment which will establish the new emission point locations and the new emission limit values. The bulk of the waters in the PIC at closure is expected to be comprised predominantly of runoff (dilute contact water from the dome and side-slope surfaces of the BRDA) and a minor amount of seepage (highly alkaline liquid held in the pore space of the bauxite residue, expressed slowly as seepage due to overlying pressure), that can be expected to reduce in the long-term.

The liquid sampled from the piezometers was compared to that sampled quarterly from the operational PICs and similarly returned a water quality with high pH (12 to 13) and elevated concentrations of particular metals i.e., arsenic, chromium, copper, lead, nickel, zinc, and mercury. The amended layer leachate testing demonstrates that the bauxite residue farming and amendment is successful in reducing the pH of the bauxite residue and improving the seepage water quality from the amended layer. Elevated concentrations for copper and lead were noted in the leach testing.

Note: *These metals arise from the bauxite ore itself and these concentrations are not necessarily indicative of constituents of potential concern in the final mixed seepage water quality.*

The seepage modelling provided an output for the cumulative annual flux volumes over the BRDA facility as a whole for an average year (based on 30 years of rainfall data from 01 Jan 1991 to 31 Dec 2020):

- Of the total water that accumulates in the PIC due to surface runoff and sidewall seepage, 93.7% arrives directly as surface water runoff from the dome and side slopes of the facility;
- The remaining 6.3% emanates from the facility slopes as sidewall seepage, and this is divided across four specific locations along the sidewalls – the Stage 5 bench, the Stage 10 bench and seepage directly into both the facility PICs from the Inner Perimeter Wall (IPW) and into the dome perimeter channels; and
- There is negligible seepage through the base of the facility, either in the unlined or lined phases.

The water quality of the resulting mixed solution (93.7% surface water and 6.3% seepage) was evaluated for two situations: (1) immediately after mixing and (2) after equilibration with atmospheric carbon dioxide and oxygen, precipitation of pertinent secondary mineral phases and sorption of trace metals onto precipitated iron hydroxides.

Immediately after mixing (step 1), the resulting water had a high alkalinity pH of 9.9 to 11, and elevated concentrations of arsenic, copper and zinc concentrations. The equilibration/precipitation/sorption simulations (step 2) resulted in a decrease in overall concentrations of metals and pH. Mineral precipitates capable of forming in solution were allowed to precipitate, and metals were allowed to sorb to the surface of iron oxide minerals. The mixed water quality was circa pH 8.8 due to dissolution of atmospheric carbon dioxide and the remaining metals with potentially elevated concentrations were 0.035 mg/L for arsenic, 0.0054 mg/L for copper, 0.31 for zinc.

Sensitivity analyses of seepage proportions from 2 – 10% show similar results, with highly alkaline pH upon immediate mixing and a decrease in pH upon equilibration. It is clear that even a small proportion of highly alkaline seepage is not easily diluted by a dilute runoff solution. Based on sensitivity analysis, equilibration and precipitation processes will have an important role in attenuation of discharge. The dissolution of atmospheric gases is a slow process, and it is envisaged that equilibrium will be achieved during the residence time of water in the PIC.

Following a review of the assessment, a concept design for converting the PICs to wetlands at closure and providing two (2) breach locations in the PIC for discharge was developed and these are discussed further in Section 8.5. Both breach locations have a sill elevation which will maintain a sufficient depth of water in the PICs throughout the year to preserve the wetlands, hence it is expected that discharge will only take place in the winter season (October to April), outside of extreme events during the rest of the year.

7.8 BRDA Hydrological Assessment (Water Balance)

A hydrological assessment for the BRDA water management system was conducted for the worst-case operational scenario when the final elevation of the Phase 1 and 2 BRDAs is increased to a crown of dome elevation of 44 mOD and a perimeter crest elevation of 36 mOD (Stage 16). In this scenario, there is no opportunity for storage of surface water on the topography of the BRDA, surface water runoff will report directly to the PIC segments and all of the waters are required to be managed within the water management system for the facility i.e., no emergency discharge permitted for the inflow design flood (IDF) event.

The assessment is provided in Appendix I and a summary is provided below.

7.8.1 Facility Water Management System

The assessment provides a description of the existing BRDA water management infrastructure, as well as a description of proposed upgrades to this water management system required to accommodate the IDF for the BRDA Raise Development. A description of the existing Plant Site water management system is also provided in the assessment, as a portion of the surface runoff generated on the Plant Site catchment is discharged to the BRDA water management system. The BRDA water management system is described in Section 6.12 and the Plant water management system is abridged below.

The Plant Site is the area where alumina refining activities are undertaken. Hydrologically, the Plant Site is divided into three main areas as follows (see Drawings in Appendix I):

- **Northern Area**: surface water runoff from this Raw Materials & Produce Storage Area (Non-Process) area is uncontaminated and discharges directly off site;
- **East Catchment**: surface water runoff from this area is potentially contaminated and drains to the East Pond for storage / attenuation prior to being pumped to the ECS; and
- **West Catchment**: surface water runoff from this area is potentially contaminated and drains to the West Pond for storage / attenuation prior to being pumped to either the ECS or to the Phase 1 BRDA PIC.

The Plant Site water management system for the hydrological assessment is presented conceptually by the block flow diagram in Figure 38.

The Plant Site hydrological analysis results are also presented in the assessment.

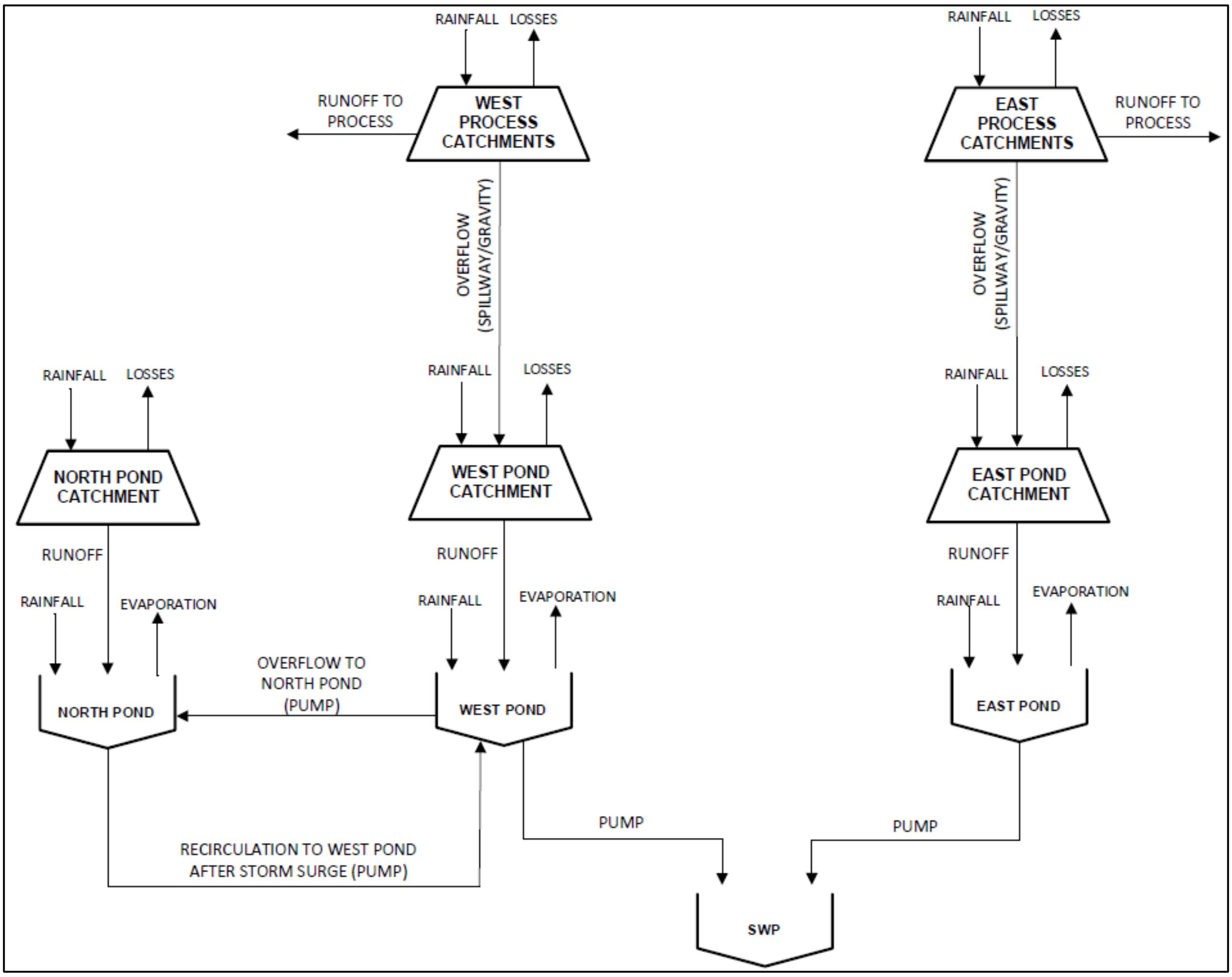

Figure 38: Plant Site Water Management System - Block Flow Diagram (see Appendix I)

Notes:

1) *The North Catchment is not presented in the flow diagram as this area discharges directly off site.*
2) *Catchment "Losses" presented in the flow diagram represent all hydrological losses from rainfall including evaporation, transpiration, infiltration, and losses due to surface depressions and ponding.*
3) *The East Pond and West Pond discharge to the ECS and the Phase 1 BRDA PIC system (which ultimately discharges to the ECS directly or via the SWP). For the purposes of this hydrological assessment these ponds have been modelled as discharging to the SWP (which ultimately discharges to the ECS). This is due to:*
   - i) *Limitations of the software used for the flood routing and storage capacity assessment; and*
   - ii) *A recommendation outcome from this study, that future flows discharging from the Plant Site to the BRDA water management system are discharged to the SWP rather than the Phase 1 PIC. This is intended to reduce the volume of water discharging to the PIC during the IDF and reduce the overall PIC pumping capacity required to accommodate the IDF.*

7.8.2 Upgrades to the BRDA Water Management System

The following upgrades to the BRDA water management system are proposed for the proposed BRDA Raise Development, which have been informed through the analysis undertaken in this study and are inherent in the methodology and results presented in the assessment. The results of the flood routing analysis of the upgraded BRDA water management system is also presented in the assessment.

- **PIC-A:** The channel for PIC-A to discharge directly to PIC-B was constructed during Q3 2021 in accordance with IEL (P0035-06) and the permitted development, and runoff from the bulk of the Phase 2 BRDA will flow clockwise from the north-east corner.

- **PIC-B to PIC-G:** Proposed increase to the crest elevation of segments PIC-B to PIC-G to 5.3 mOD from their existing crest elevations of 5.0 mOD (PIC-B to PIC-D) and 4.7 mOD (PIC-E to PIC-G). This is intended to provide additional storage capacity within the PIC system during the IDF and will be achieved through a vertical raise of downstream crest liner which will be supported by the existing crash barrier.

- **PIC D:** Proposed replacement of the existing three (3 no.) 0.3 m ID overflow culverts from PIC-D to PIC-E with two (2 no.) concrete box culverts (min.1.1m wide x 0.55m high), installed side-by-side, to provide improved conveyance capacity to accommodate the IDF.

- **PIC-M:** A new PIC (PIC-M) will be constructed at the northeast corner of the Phase 2 BRDA / southeast corner of the Phase 1 BRDA which will allow runoff to travel in a counter-clockwise direction from this area to PIC-L and then to PIC-K located at the northeast corner of the Phase 1 BRDA and directly south of the SWP. This will require the reconstruction of the existing ramp to the 'Merger Road' and the installation of a concrete box culvert to convey flows from PIC-M to PIC-L.

  Note: *The 'Merger Road' was the original perimeter road for the Phase 1 BRDA which subsequently separated the Phase 1 and Phase 2 BRDA basins and is currently covered over with the merging of the BRDAs.*

- **PIC-L:** The following upgrades are recommended for PIC-L:

  - A culverted embankment crossing is proposed to sub-divide the existing PIC-L into PIC-L (South) and PIC-L (North), as indicated on Drawing 03 (Appendix I). The purpose of this is to provide for flood attenuation storage within PIC-L (South) during the IDF by adding a culverted 'choke point' to attenuate flood discharges to the downstream water management system. Minimal flood storage volume is available in PIC-L (North) due to its steep invert gradient (approx. 1.3%), narrow base width (approx. 3.75 m) and low embankment crest level (approx. 11.5 mOD). However, PIC-L (South) will have a shallower invert gradient (approx. 0.4%), wider base width (approx. 15.75 m) and higher embankment crest elevation (16.0 mOD) allowing for significant attenuation storage.

  - Increase of the exiting PIC-L (North) embankment crest elevation by ≈ 1m height to 12.5 mOD, to provide additional storage capacity and prevent overtopping of the PIC during the IDF. The existing pipes draining this PIC discharge to a small intermediate pond prior to being culverted to PIC-K. This intermediate pond is unlikely to accommodate the IDF and hence replacement with a direct culvert between PIC-L (North) to PIC K has been recommended. The design for a replacement spillway structure is provided in Appendix L.

- **PIC K:** The following upgrades are recommended for PIC-K:

  - Proposed decommissioning of the existing culvert linking PIC-K to PIC-J, and installation of a pump and overflow culverts which will discharge flows from PIC-K to the SWP. The purpose of this improvement is to reduce the volumes of water discharging to PIC-G (via PIC J) and consequently minimise the PIC-G pump capacity upgrades required to accommodate the IDF within PIC-G. The PIC-K pump is intended to accommodate flows during regular meteorological conditions, while the overflow culverts are intended to accommodate flood flows up to the IDF.

- **PIC-G Pump Capacity:** Proposed upgrade of the PIC-G pumping capacity to allow the IDF to be accommodated within the PIC system.

- **Plant Site Discharges:** During a flood event pumping from the East and West Ponds on the Plant Site is proposed to be discharged to the SWP / ECS only and not discharge to the Phase 1 PIC, to minimise the overall PIC pumping capacity from PIC-G required to accommodate the IDF.

7.8.3 Methodology

The design criteria for the BRDA water management system have been selected to be in accordance with the Canadian Dam Association (CDA) (2007) and (2014) Guidelines. The BRDA has been identified to have a "**High**" HPC under the CDA Guidelines and therefore the Inflow Design Flood (IDF) will be 1/3 between the 1,000-year and the Probable Maximum Flood (PMF) events with a duration of 24 hours.

The Plant Site does not form part of the BRDA facility and the CDA guidelines for design rainfall events are not applicable to the Plant Site, given the CDA guidelines are intended for the design of dams / large impoundment facilities which pose a significantly higher hazard should their design criteria be exceeded. The BRDA is designed to function following cessation of operations at the site into the post-closure period. However, the Plant Site will have a relatively short life span as it will be decommissioned following cessation of operations at the site. The design flood event for the Plant Site water management system has been selected to be in accordance with the 'Flood Risk Management Plan – Shannon Estuary South' (OPW, 2018), the preferred standard of protection offered by flood protection measures in Ireland for fluvial flooding is the 100-year flood event. Storm water runoff discharging to the BRDA water management system from the Plant Site has been assessed for the 1 in 100-year +20% (climate change allowance) rainfall event with a duration of 24 hours.

The hydrological assessment of the proposed BRDA Raise Development water management system consisted of the following steps:

- Hydrological analysis of the Plant Site catchments and assessment of discharge rates from the Plant Site to the BRDA water management system. Runoff from the selected design rainfall events was routed through the Plant Site water management system using the US Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modelling System (HEC-HMS).
- Assessment of the inter-PIC flows for use in water balance modelling. The capacities of the PIC culverted 'choke points' were assessed using the US Department of Transportation Federal Highway Administration's HY-8 Culvert Hydraulic Analysis Program. The maximum discharges before overtopping were determined based on an assumed tailwater (water level at downstream end of the culvert) at the downstream end of the pipe culverts.
- Evaluation of the maximum operating water levels in the PICs, SWP and LWP under normal operating conditions for use as initial water levels in hydrologic / flood routing modelling. The 75th percentile water levels resulting from water balance modelling were assumed to represent the upper limit of normal operating conditions. The 75th percentile represents the value below which 75% of the modelled levels occur.
- Assessment of the performance of the PICs under IDF conditions using hydrologic modelling. The IDF was routed through the PICs using HEC-HMS. Improvements to the BRDA water management system were made where necessary to ensure the PICs did not overtop during this flood event (see Section 7.8.2).
- Assessment of the capacity of the SWP to manage inflows during the IDF using hydrologic modelling. The IDF was routed from the PICs through the SWP using HEC-HMS. Inflows from the Plant Site hydrological model (pumping from the East and West Ponds to the SWP) were also incorporated.

7.8.4 Summary and Discussion

- Peak runoff rates to the PIC segments from the BRDA range from 0.074 m^3/s (PIC-L North) to 1.115 m^3/s (PIC-E). The analysis shows that the proposed PIC system can accommodate the IDF without overtopping provided that proposed improvements to the PIC system are implemented (see Section 7.8.2). However, peak water levels exceed the target freeboard level of the PICs during the IDF (assumed to be 0.5 m below the PIC crest elevation) for all PICs except PIC-A and PIC-L (North and South segments).
- The BRDA water management system can accommodate the design rainfall event for the Plant Site, i.e., the 1 in 100-year + 20% climate change event.

  For rainfall events in excess of this event (e.g., the IDF for the BRDA) the estimated volume of excess runoff to be managed by the Plant Site during the IDF is 24,230 m^3. AAL proposes to temporarily retain and manage within the Plant Site surface water management system i.e., drains, sumps, bunded slabs and process tanks, up to the BRDA IDF rainfall event.
- The SWP and LWP have sufficient storage capacity to accommodate the IDF without overtopping provided the proposed improvements are made to the PICs and pumping systems i.e., no upgrades are required to the current structures.

8.0 BRDA RAISE CLOSURE CONSIDERATIONS

Condition 10 of the EPA issued licence (IEL P0035-07) requires AAL to have an approved plan in place for the orderly closure, decommissioning and aftercare of the facility. This plan is called the Closure, Restoration and Aftercare Management Plan (CRAMP) and covers both the Plant area and the BRDA.

The most recent CRAMP update was conducted by AAL during 2018 and subsequently approved by the EPA in October 2018. Financial provisions for the CRAMP are deposited by AAL annually into a Secured Fund and a Parent Company Guarantee (PCG) is in place to match the balance for the Secured Fund target value. AAL engage in progressive restoration of the BRDA and thus the CRAMP costing has reduced slightly in recent iterations.

This section provides a summary of the BRDA closure plans advanced to an engineering design level to Stage 16. These engineering designs are equally applicable for the BRDA constructed to Stage 10 and a further update to the CRAMP and financial provisions will be required in due course.

8.1 Capping Containment Objectives

The engineering design for the BRDA capping containment has considered the following objectives:

- Minimise the infiltration of water;
- Promotes surface drainage and maximises runoff;
- Provides a physical separation between stored bauxite reside and plant and animal life;
- Provides a platform for the landscaping for the BRDA;
- Provide water management infrastructure to discharge runoff resulting from the Inflow Design Flood (IDF); and
- Qualitive assessment for the provision of overflow / flood routing for flood events of greater magnitude than the IDF and/or for the potential of long-term clogging of elements of the system.

All capping containment details for the BRDA (dome and side-slopes) are required to comply with the Conditions of 8.5.6 and 8.5.7 of IEL P0035-07, namely:

Condition 8.5.6 – Design and construction details for all basal, side and capping containment engineering works proposed for any part of the BRDA shall be agreed in writing by the Agency prior to construction;

Condition 8.5.7 – All basal, side and capping containment engineering works proposed for any part of the BRDA shall be carried out under an Agency agreed Construction Quality Assurance Plan (CQA Plan).

8.2 Progressive Capping Containment

AAL have conducted small- and large-scale trials over the years to demonstrate proposed capping containment methodologies, amend the bauxite residue to reduce the pH and cultivate vegetation on the surface. AAL have partnered with a local university (University of Limerick) along with commercial consultancy services to aid the progress. Subsequently, the proposed 'amended mud' capping for exposed bauxite residue is now included as Condition 8.5.21 in IEL P0035-07.

8.2.1 Amended Layer

The final 1m depth of all exposed bauxite residue is required to comprise 'amended mud' or the 'amended layer'. Following large scale trials on the wide Stage 5 bench on the north and west sides of the Phase 1 BRDA (≈ 32m width x ≈ 1,200m length), the current specification for the amended layer, to be constructed in two 0.5m depth layers, to provide a neutralized soil material (< 9.0 pH) to support vegetation is listed below:

- Farmed or carbonated bauxite residue that has a pH < 11.5.
- Addition of washed process sand at rate of 1,250 m^3 / hectare / 0.5m depth layer and mixed thoroughly using a spader.
- Addition of gypsum at a rate of 90 tonnes / hectare / each 0.5m depth layer and mixed thoroughly using a spader.
- Addition of approved organic soil improver / compost at a rate of 550 tonnes / hectare / upper 0.5m depth layer and mixed thoroughly using a spader.
- Rotovation of the top surface, prior to grass seeding.

The current Condition 8.5.21 requires that the amended layer be underlain by a capillary break comprising process sand or equivalent approved. The requirement for this capillary break layer is subject to ongoing trials.

It is proposed that the capping containment for the wide benches at Stage 5 and at Stage 10 and the full extent of the BRDA dome would comprise the amended layer.

8.2.2 Phase 1 BRDA Side-Slopes

During 2013, AAL constructed the capping containment and landscaping of the north and west side-slopes of the Phase 1 BRDA from Stage 0 to Stage 8 (slope width ≈ 120m and length of ≈ 1,450m).

The works comprised the construction of a continuous rock fill blanket on the side-slopes (≈ 350mm depth), using rock fill material gained locally from the stage raises, and the construction of soil mounds above the rock fill to support vegetation. A piped drainage network was installed within the rock fill blanket to transfer bauxite residue influenced water from the uppermost stage raise directly to the PIC, in order to alleviate the trickle-down drainage during the operational life of the BRDA, which has the potential to impair the vegetation due to the high pH and to clog up and/or discolour the rock fill blanket with bauxite residue fines (Golder 2013).

The constructed works have performed well since commissioning and have only required minimal levels of maintenance. The operational drainage network has been extended upwards to the ditch excavated at the downstream toe of Stage 10 and the landscaping has established sufficiently to provide greening of the rock fill blanket.

8.2.3 Interim Landscaping

In recent years, AAL have progressed with interim landscaping of the upper stages raises of the Phase 1 BRDA (Stages 7 to 10) and the lower stage raises along the west side of the Phase 2 BRDA. These interim landscaping commenced during 2017 and comprise:

- Construction and maintenance of a ditch at the downstream toe of the uppermost stage raise;
- Installation of 450mm OD collector pipes from the ditch to the PIC at regular intervals (approx. 100m);
- Dressing the downstream slope of the rock fill stage raise with ≈ 100mm depth of screened subsoil. A ≈ 0.5m vertical gap is maintained at the downstream toe so as not to impede drainage; and
- Hydroseeding the subsoil layer with the design mix.

The performance of the operational drainage network and the hydro-seeding has been very successful, with only minimal levels of re-application of hydro-seeding required for patchy areas. The established areas do require watering during drought conditions as the subsoil layer is not of sufficient thickness to hold a reservoir of water.

8.3 BRDA Dome Closure

The engineering design and drawings for the BRDA Dome Closure are provided in Appendix J and the key design elements are summarized below.

- The BRDA dome grades at ≈ 4% from the inner crest of the Stage 16 raise at 36 mOD to the dome crown at 44 mOD.
- The dome for east sector of the Phase 1 BRDA, where the raised SCDC is located, blends into the overall BRDA dome. A specific capping containment design is proposed for the SCDC Raise (see Section 13.6), which is appropriate for the capping of a hazardous waste material and in accordance with the EPA approved design for the current SCDC (Golder 2017B).
- At closure, the last 1m depth of bauxite residue for the dome is amended layer, to be constructed in two 0.5m depth layers, to provide a neutralized soil material (< 9.0 pH) to support vegetation.
- The dome area of approx. 68.7 hectares in plan is spilt into seven (7) catchment areas (C-1 to C-7) for the primary dome and one (1) catchment area (C-8) for the east sector of the Phase 1 BRDA.
- Runoff from the dome is intercepted by sixteen (16) dome perimeter drainage channel segments which convey the intercepted runoff directly to the eight (8) spillways i.e., no storage or attenuation of waters. Each spillway is served by two (2) dome perimeter channel segments which are lined with concrete canvas, have a trapezoidal cross section, 1.5(H):1(V) side slopes, 1m design depth and are located adjacent to the upstream face of the Stage 16 raise (apart from C-8, for which they are located adjacent to the upstream face of Stage 11). The channels have been designed with a mild longitudinal slope of 0.13 % i.e., 750(H): 1(V), to ensure sub-critical flow conditions and low flow velocities along these channels.
- Each catchment has a designated spillway (SP-1 to SP-8), which are located approximately centrally along the catchment boundary, and are served two (2) segments of dome perimeter channel which are named for the spillways they serve i.e.,SP-1 is served by CHSP1-1 and CHSP1-2.
- The eight (8) spillways are distributed along the perimeter of the BRDA dome and traverse down the side-slopes, perpendicular to the respective PICs, at slopes varying between 6.3(H):1(V) and 6.8(H):1(V). The spillways are lined with concrete canvas, have rip-rap rock fill armouring to slow the flows and alleviate turbulence and hydraulic jumps, vary in base width from 6m to 8m (apart from SP-8 which is only 4m in width due to the smaller catchment), have a 1m design depth, and convey the runoff from the dome perimeter channels directly to the PICs.
- Gabion mattresses are provided for flow energy dissipation at entry to the PICs from the spillways.

8.4 BRDA Side-Slope Closure

The engineering design and drawings for the BRDA Side-Slope Closure are provided in Appendix K and the key design elements are summarized below.

- The side-slope area is approx. 103.3 hectares in plan (including the PICs footprint) and has a perimeter length of approx. 5,600m. The BRDA side-slopes have been divided into segments of 100m in width, extending from Stage 0 to Stage 16, with each segment forming an independent hydrological catchment i.e., runoff from a given segment is designed to be managed within that segment.

- At closure, the BRDA side slopes will be capped with a rock fill capping containment layer which will provide a continuous rock fill blanket across the entire footprint of the BRDA side slopes. The rock fill blanket will comprise the rock fill from which the stage raises have been constructed and additional rock fill placed over the exposed bauxite residue benches, interconnecting from stage raise to stage raise.
- Boundaries / cut-offs are constructed between each 100m width segment to provide controlled management of runoff from the side slopes and reduce the potential for runoff concentration along preferential flow pathways and localised overwhelming of the side slope drainage system.
- The downstream faces of the rock fill stage raises will be vegetated by hydroseeding a subsoil layer.
- The horizontal benches for each stage raise will have their rock fill capping containment layer (blanket) overlain by subsoil / topsoil layers and subsequently vegetated. However, a strip of the rock fill blanket (‘infiltration strip’) will remain exposed (i.e., not overlain with subsoil / topsoil or vegetated) which will allow surface water runoff to infiltrate into the rock fill blanket at each stage raise.
- The primary drainage system is an internal one i.e., within the rock fill blanket (300mm to 400mm depth, depending on the stage raise), with runoff entering via the infiltration strip and propagation of the IDF flows through the continuous rock fill blanket to the PICs.
- The secondary drainage system is a surface one i.e., via rip-rap lined overflow chutes from stage raise to stage raise (width varying from 0.5m to 2.0m, depending on the stage raise). This system has been designed to allow controlled discharge of the IDF in the event that the rock fill blanket, or a meaningful section thereof, does not have sufficient drainage capacity to accommodate the IDF e.g., due to long term clogging of the rock fill blanket and/or infiltration strip(s).
- The lateral spacing of the overflow chutes has been designed to be staggered in order to:
  - attenuate the IDF within the surface flow system and increase catchment times of concentrations by increasing the length of the surface flow paths
  - increase the potential for infiltration of surface water along the stage infiltration strips; and
  - Minimise potential visual impacts associated with the overflow chutes.
- The minimum lateral spacing between successive overflow chutes within each 100m side slope segment is 50m.

8.5 Discharge to the Environment

Following closure, AAL will enter into a minimum 5-year active aftercare period during which time all the waters from the BRDA will be captured and returned to the effluent clarification system (ECS) at the plant for treatment and subsequently to discharge via their licenced discharge point (W1-1).

During this 5-year period, AAL will complete the remaining closure works for the side-slopes and the dome, construct the passive treatment wetlands in the PIC and the SWP, construct the designated breach locations in the PIC and SWP and will allow time for the vegetation to establish sufficiently at each closure element.

AAL will continue to monitor the quality of the waters from the BRDA during the period, which is expected to improve significantly as the capping and closure works are completed and establish, and will apply for a discharge to the environment via two (2) proposed PIC breach spillway locations and subsequently to the Robertstown River (see Section 8.5.4) at appropriate water quality limits to be agreed with the EPA.

8.5.1 Water Quality at Closure Assessment

The seepage and water quality assessment for the BRDA constructed to Stage 16 and following closure is provided in Appendix H and is discussed in Section 7.7.

The findings from the assessment showed that there was potential for the mixed wates in the PIC to have pH > 9.0 and for some metals to have potentially elevated concentrations (subject to the agreed discharge limits). Attenuation, including passive treatment via a wetlands, of the waters in the PIC is considered to be required to provide residence time for equilibration to reduce pH and for removal of metals via plant species and mineral precipitates.

8.5.2 Wetland Pilot Scale Trials

AAL in conjunction with University of Limerick (UL) have conducted pilot scale plug horizontal surface flow wetland trials for a number of years at the BRDA with the purpose of investigating the effectiveness of wetlands to buffer the high pH (> 11.0) of the bauxite residue leachate. Several papers have been published on the wetland trials with the latest publication (O'Conner and Courtney, 2020) documenting the findings over a 52-month period (May 2015 to August 2019). The key elements of the trial were:

- Constructed wetland (4m wide x 11m long) with a 200mm soil substrate depth and a 100mm operational water depth.
- Wetland vegetation is *Phragmites Australis* (dominant species), *Typha Latifolia* and *Sparganiumerectum.*
- Inflow leachate was manufactured on site and comprised neat residue leachate extracted from the plant process with pH > 13 diluted with deionised (DI) water to provide a pH ≈ 11.
- Flow rate varied from 10 to 30 l/hr in the winter months to 45 to 55 l/hr in the summer months.
- Retention times varied from 9.16 days in the winter to 3.67 days in the summer, mean of 5.64 days.

The results of the 48-month operational period were very positive and consistent:

- Outflow leachate reduced in pH to a mean of 7.21, with all results having pH < 9.0. Suggested mechanisms for pH reduction in the wetlands are carbonation, microbial respiration as well as production of organic acids during decomposition and root exudates.
- Slight reduction in electrical conductivity.
- Decreases in concentration of metals, in particular for Al and V.

AAL are progressing with further pilot scale wetland trials using leachate extracted from the BRDA to provide an improved assessment of the potential metals present and their concentrations in the runoff at closure and to assess the effectiveness of the wetland and particular plant species to passively treat these metals and to continue to reduce the pH.

There are numerous papers and research projects detailing the effectiveness of wetlands for the reduction of metal concentrations from inflow waters in Ireland (Higgins 2007) and internationally for the passive remediation of metalliferous mine drainage (PIRAMID 2003). Based on the effectiveness of the pilot scale works conducted at AAL to the date, industry research results of the effectiveness of wetlands for passive remediation metalliferous waters and the level of metals concentrations identified in the seepage and water quality assessment, it is considered highly likely that a wetland will be an appropriate and effective passive treatment solution for the waters discharging from the BRDA at closure.

8.5.3 Wetland Sizing

A preliminary design layout for the wetland is provided on Drawings 10 and 11 is based on constructing a wetland in the existing PICs with a substrate depth of 0.3m, a permanent pond depth of 0.5m, an average 8.5m wet width for the Phase 1 BRDA PIC, an average 10.0m wet width for the Phase 2 BRDA and utilizing the existing grades of the PICs. The catchment area for the Phase 1 BRDA and Phase 2 BRDA is 116.1 ha and 72.8 ha, respectively. Based on the PIC lengths and the wet widths noted above, the wetland for the Phase 1 BRDA PIC will have a footprint of ≈ 25,500 m^2 and the wetland for the Phase 2 BRDA PIC will have a footprint of ≈ 24,500 m^2.

The preliminary sizing of the wetland is based on the Guidance Manual for Constructed Wetland (EA 2003) and 'The Suds Manual' (CIRIA C753, 2015).

- **Hydraulic Design** – the spillways at the two (2) PIC breach spillway locations have been designed to safely convey the BRDA IDF in aftercare and allow for attenuation of discharge to greenfield rates for events up to the 1 in 100-year event (see Section 8.5.4 and Appendix L).

  The wetlands proposed to be constructed in the PICs at closure have been hydraulically designed to achieve a minimum residence time of seven (7) days for rainfall events up to the 1-year, 1-hour duration rainfall event, corresponding to 10.8 mm rainfall depth. The residence time is based on the water quality at closure assessment (see Appendix H).

  For events up to the 1-year event, each PIC segment has been designed to contain and slowly release runoff reporting directly to the PIC segment. Release rates from each segment will be controlled through the implementation of a flow control device which will facilitate a minimum residence time for runoff of seven (7) days within the wetland systems.

  For larger (extreme) rainfall events up to and including the IDF, inter-PIC discharge will be provided via riprap lined overflow spillways provided at each PIC segment division. The invert level for these overflow spillways is set 0.7m above the estimated 1-year, 1- hour event design water level for each PIC segment.

- **Treatment Design** – retainment of short duration storms (1-year storm event) and/or a treatment volume of 10mm to 15mm of rainfall depth for the contributing catchment is recommended by CIRIA C753 and the Guidance Manual for Constructed Wetlands. This equates to a treatment volume of 17,415 m^3 for the Phase 1 BRDA and 10,920 m^3 for the Phase 2 BRDA, for a 15mm rainfall depth and corresponds to 1-year rainfall event with a 2.5-hour duration. The attenuation volumes for the preliminary design are ≈ 17,500 m^3 for the Phase 1 BRDA PIC and 12,000 m^3 for the Phase 2 BRDA PIC (based on a 0.5m permanent pond depth).

  Low-flow volume controls will be designed at the detailed design stage to permit normal discharge limited to the design treatment retention time i.e., target of 7 days, based on the pilot scale wetland trials to reduce pH < 9.0, at the elevation of the permanent pond for the wetland.

  Empirical guidelines for sizing of treatment wetland suggest a footprint area of 1% to 5% of the contributing catchment. This would equate to a footprint of 11,600 m^2 to 58,100 m^2 for the wetland in the Phase 1 BRDA PIC and a footprint of 7,300 m^2 to 36,400 m^2 for the wetland in the Phase 2 BRDA PIC. The footprints provided by the preliminary design correspond to ≈ 2.2% for the Phase 1 BRDA and ≈ 3.4% of the Phase 2 BRDA for the contributing catchment areas, respectively.

  Upstream sediment treatment / capture is provided by the side-slope and spillway rock fill construction layers on the flow path to the PICs.

- **Amenity Design** – The wetland (substrate and vegetation) will provide an effective protection layer for the geomembrane lining of the PIC and will provide a more suitable and natural form to conform with the side-slope and dome closure preliminary designs for the BRDA. An aquatic bench (zone of shallow water) will be formed along each side of the permanent pool on the upstream and downstream slopes of the PIC.
- **Biodiversity Design** – The final design of the wetland in the PIC is envisaged to incorporate a mixture of plant species, open water and benches and varying water depths. The 0.5m depth permanent pool will be maintained within PIC segments by the construction of intermediate berms to counter the grade at the base of the PIC, such that a decreasing staircase of water levels will be formed between ponds separated by the intermediate berms, leading to the two (2) designated breach locations.
- **Operation and Maintenance** – the expected life of a constructed wetland before significant maintenance is required, i.e., sediment or vegetation removal, is 20 to 25 years, provided regular inspection and regular maintenance is conducted during the period. An operation and maintenance schedule for the wetlands shall be prepared and costed for the CRAMP.

8.5.4 PIC Breach Assessment

The engineering design and drawings for the PIC breach spillways are provided in Appendix L and the key design elements are summarized below.

- Two (2) PIC breach spillway locations (Figure 39) have been selected as they correspond with the locations where invert elevations are lowest within the existing Phase 1 and Phase 2 BRDA PIC systems and therefore, facilitate drainage of the full system by gravity at closure, without the requirement for significant alteration of invert elevations or gradients.

Figure 39: Proposed PIC Breach Spillway Locations and Discharge Route (extract from Drawing 10)

- The surface water management design strategy post-closure for the PICs is summarised as follows:

  Phase 1 BRDA PICs:

  - Surface water runoff and seepage collected in the Phase 1 BRDA PICs (i.e., PIC-E, PIC-F, PIC-G, PIC-J, PIC-K, PIC-L and PIC-M) and the SWP will be discharged to the North Drain via PIC Breach Spillway #1, which is located centrally in PIC-G.
  - The existing PICs will drain by gravity to PIC-G. PIC Breach Spillway #1 will be constructed through the north-east embankment of the Outer Perimeter Wall for the PIC-G segment.
  - Waters in the North Drain then flow counter-clockwise to enter the northern section of the West Drain and subsequently to the penstock discharge point, locate to the west of the Phase 1 BRDA. The North Drain segment has a length of approximately 1.8 km and an estimated average gradient of 0.012%.
  - An additional spillway will be required at closure to convey flows from PIC-L to PIC-K. During operation these flows are conveyed with piped culverts which will be replaced by a spillway, pre- or post -closure.

  Phase 2 BRDA PICs:

  - Surface water runoff and seepage collected in the Phase 2 BRDA PICs (i.e., PIC-A, PIC-B, PIC-C and PIC-D) will be discharged to the West Drain via PIC Breach Spillway #2, which is located at the north-west corner of the Phase 2 BRDA (end of PIC-D, where the Phase 2 BRDA PICs meet with the Phase 1 BRDA PICs).
  - The existing PICs will drain by gravity to the north-west corner of the Phase 2 BRDA. PIC Breach Spillway #2 will be constructed through the outer perimeter wall for PIC-D.
  - Waters enter the southern section of the West Drain then flow clockwise to the north and subsequently to the penstock discharge point, locate to the west of the Phase 1 BRDA. The drain has a length of approximately 0.5 km and an estimated average gradient of 0.134%.

- Surface water from both PIC Breach Spillways is discharged into the Robertstown River, which is a tidal river that flows north before joining the River Shannon at the Shannon Estuary. The discharge takes place during low tides and is controlled by a sluice gate which has an invert level of approximately -1.1 mOD.
- The PIC breach spillways have been designed to safely convey the BRDA IDF post-closure and allow for attenuation of discharge to greenfield rates for events up to the 1 in 100-year event.
- The engineering design consists of two U-shaped concrete channels through the northern wall of the SWP (PIC Breach Spillway #1) and through the outer embankment of PIC-D (PIC Breach Spillway #2). The concrete channels have been designed with a width of 1.0m and discharge to trapezoidal rip-rap lined chutes. The minimum depths of the concrete channels are 1.0m (PIC Breach Spillway #1) and 1.5 m (PIC Breach Spillway #2). The rip-rap chutes have been designed with a bottom width of 3.0m and a depth of 1.0 m; and ultimately discharge to the existing downstream drains.
- Hydraulic modelling of the proposed spillways, PIC Breach Spillway #1, PIC Breach Spillway #2 and the North-East PIC Spillway, was completed using HEC-RAS software to assess the performance of the design during the considered events. The modelling demonstrated that the proposed design meets the design criteria

9.0 BRDA RAISE OPERATING PHILOSOPHY / REQUIREMENTS

9.1 General

Section 6.3 (BRDA Deposition), Section 6.4 (BRDA Raising) and Section 6.5 (BRDA Current Status) detail the operating philosophy of the BRDA. A summary of the key aspects is provided below:

- The BRDA is progressively raised by the upstream method in 2m high vertical lifts (stage raises).
- The bauxite residue is discharged as a paste centrally in layers and migrates to the perimeter at a shallow gradient, creating a domed profile.
- The bauxite reside is drained via the permeable stage and the BRDA retains no surface water.
- Deposited bauxite residue is mud-farmed to reduce the pH, to further reduce the moisture content and increase density and subsequently increase the strength parameters.
- The average rate of rise of the BRDA during its life is in the 1.0m to 2.0m range / year.
- An estimated 36.0 million tonnes of bauxite residue have been deposited in the BRDA from start-up in 1983 to October 2020.
- AAL have raised the Phase 1 BRDA to its design perimeter elevation (Stage 10 @ 24 mOD).
- The BRDA maintains the required target factors of safety (FoS) for stability in accordance with the CDA guidelines.

9.2 Deposition Plan

The current rate of production of bauxite residue is circa 1.57 million tonnes / year (dependent on grade of ore) and is deposited at a characteristic bulk density of 2.19 tonnes / m^3, following mud-farming activities. The rate of void consumption is between 900,000 to 1,000,000 m^3 / year for bauxite residue and approx. 35,000 m^3 / year for rock fill.

The internal footprint of the BRDA at Stage 10 (area available for deposition) is circa 102.3 hectares (1,023,000 m^2). The internal footprint of the BRDA at Stage 16 is circa 67.8 as (687,000 m^2). Hence, the rate of rise at current production levels can expected to be in the 1.0m to 1.5m range for the BRDA Raise Development, as the full footprint will be available for balance deposition.

Figure 31 in Section 6.3 provides a layout for the bauxite residue layered deposition cells which are partitioned by up to 1.5m high berms of farmed bauxite residue formed using a dozer; Phase 1 BRDA (Cells 1 to 25) and Phase 2 BRDA (Cells 26 to 46). Two layers are targeted to be deposited in each cell annually, after which the cell bunds are then re-formed from locally sourced farmed bauxite residue using a bulldozer.

9.3 Stage Raising Plan

Campaigns of stage raise construction are undertaken annually to meet deposition requirements and are typically of 10 to 12 weeks duration and occur twice per year.

Stage raises are constructed in sectors where the deposited bauxite residue has been deposited, has been sufficiently farmed to meet its pH threshold and is at the crest elevation of the current stage raise. Where low spots exist, locally farmed bauxite residue is relocated and tracked into placed to meet the required elevation for the footprint of the subsequent stage raise, prior to construction commencing.

Section 6.7 and Section 6.8 detail the stage raises construction methodology and the proposed stage raise construction phasing for the BRDA Raise Development.

The perimeter of the Stage 10 raise is ≈ 4,250m and the perimeter of the Stage 16 raise is ≈ 3,600m. Allowing for an annual rock fill consumption rate of approx. 35,000 m^3 and a stage raise requirement of approx. 14.0 m^3 / m length, means that an average length of ≈ 2,500m of stage raise are constructed annually. Generally, approx. 50% of the BRDA perimeter can be expected to be raised annually i.e., a 2m high stage raise constructed.

9.4 Water Management Plan

The water management system for the current BRDA is detailed in Section 3.5 and Section 6.12 and a summary of the water balance assessment for the BRDA Raise Development is discussed in Section 7.8 with the full assessment provided in Appendix I.

The water management system for the BRDA Raise Development is largely unchanged from the current system, as the catchment area has not increased. The current operational BRDA is compliant with the target criteria in accordance with the CDA guidelines as AAL can actively manage the IDF through pumping and through the temporary storage of water on the surface of the BRDA as a result of the present topography. However, as the BRDA increases in elevation, the structure of the BRDA will adjust to resemble its closure profile and a number of modifications are required for the current water management infrastructure to manage the IDF event during the operational and after-closure phases. These modifications are listed in Section 7.8 with the full assessment provided in Appendix I.

9.5 Bauxite Residue Surface Management

The AAL BRDA bauxite residue surface management system is discussed in Section 6.13.2 and comprises a system of sprinklers which cover the entirety of the exposed bauxite residue surface on an approximately 75m x 75m grid. This system has proven to be highly effective since installation in 2001.

The primary operational system for the BRDA Raise Development will be largely unchanged from the current system. The sprinklers will be raised accordingly with the increase in elevation of the bauxite residue and the system will be managed and be operational for as long as the bauxite residue remains exposed.

AAL have commissioned a range studies and trials during the past two (2) years to assess best practice solutions:

- Computational Fluid Dynamics (CFD) Modelling – the BRDA topography has been modelled at various stages of its storage life to Stage 16. Design storm events (various wind speeds and wind directions) have been applied to the models in order to measure key parameters and dust particles of various sizes have been introduced to assess potential dispersion paths and drop-out zones.
- Bauxite Residue Berms – the addition of additional bauxite residue berms at specific locations, alignments and heights determined to be beneficial from the CFD modelling is currently being trialled. These berms further segment the deposition cells and are being incorporated in the deposition planning.

Based on the findings to date, the operational best practice will be a blend of bauxite residue berms and surface conditioning to support the sprinkler system.

10.0 BRDA INSTRUMENTATION, MONITORING AND SURVEILLANCE

10.1 General

The objective of the instrumentation, monitoring and surveillance of the BRDA is to assess the performance of the facility and to mitigate the risk of instability in the short and long-term.

AAL are required to manage and operate the Plant and the BRDA under the conditions of the Industrial Emissions Licence (current revision is P0035-07, September 2021) issued by the EPA. Conditions relating to monitoring of the BRDA are replicated below.

- ***Condition 8.5.15*** requires that report on the status of the BRDA is required to be provided annually in the Annual Environmental Report (AER) for the facility. This report is required to contain, at a minimum, the elements detailed in *Schedule D: Annual BRDA Status Report* of the licence.
- ***Condition 8.5.25:*** The BRDA is required to be monitored as set out in *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence.
- ***Condition 8.5.26:*** AAL shall arrange for a Biennial Independent Audit. AAL shall arrange a Safety Evaluation of Existing Dams (SEED) Audit at a frequency agreed with the Agency, which shall substitute the Biennial Independent Audit for the same year of occurrence.

  Note: The SEED Audit or Dam Safety Review (DSR) for the CDA Guidelines is required to be undertaken by an independent consultant at a minimum frequency of 15 to 20 years for existing facilities. The most recent DSR for the BRDA was undertaken by SLR Consulting (Canada) Ltd. in April 2019.
- ***Condition 8.5.27:*** All inspections, monitoring, annual reviews and independent audits shall, where appropriate be carried out in accordance with the requirements of BAT and any technical guidance or decisions issued by, or on the behalf of, the Committee for the Adaption to Scientific and Technical Progress of Directive 2006/21/EC on the management of waste from extractive industries.

10.2 Physical Stability Monitoring Plan

A Physical Stability Monitoring Plan (the Plan) for the AAL BRDA has been developed by Golder as Engineer of Record (EoR) following an assessment of the current AAL licence (IEL P0035-07) and the 2018 Best Available Techniques (BAT) Reference Document for the Management of Waste from the Extractive Industries (BREF), in accordance with Directive 2006/21/EC (EUR 28963 EN), (MWEI BREF 2018).

The upstream raising of the BRDA is an ongoing operation during the operational life of the facility; therefore, the Plan is a live document requiring:

- Addition of instruments as the BRDA increases in elevation;
- Addition of interim instruments and monitoring programs to manage specific construction projects and/or events;
- Replacement of instruments resulting from damage / missing due to operations; and
- Removal of instruments from the plan as Phases of the BRDA overlap.

The Plan is updated on an annual basis and consists of scheduled installation and monitoring of geotechnical instruments installed within the facility, along with a series of scheduled audits, inspections and conformance checks to assess the performance of the BRDA.

The Plan for 2021 is included in Appendix M and a summary is provided in the Sections below.

10.2.1 Standard Walk Over Condition and Visual Assessment (BRDA, SWP and LWP, PICs, Toe Drains, North Drain, West Drain and FTDB)

In accordance with *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence the BRDA is visually inspected on weekdays (Monday to Friday).

A standard walk-over condition and visual assessment is conducted by the AAL BRDA Operator(s) and AAL BRDA Engineer of the predetermined check list items in accordance with the AAL documented procedures for the stability monitoring of the BRDA.

The AAL documented procedures for the stability monitoring of the BRDA include a visual inspection plan of the major components of the dam, criteria for observations, identification of key areas for intensive inspection, frequency of the inspections, reporting procedures, inspection procedure following significant events (flood event, earthquake, blast etc.), training and experience requirements of inspectors, procedures to escalate findings and data storage.

The control period for the daily conformance checks is for as long as the facility is in the operational phase.

10.2.2 Instrumentation

Instrumentation is installed periodically as the facility reached target elevations. The BRDA has a strong performance history showing consistently stable monitoring readings without significant fluctuations and achieving appropriate FoS.

The bulk of the installed instrumentation is in the Phase 1 BRDA, which has practically reached its current permitted perimeter elevation (Stage 10) for all sectors. The Phase 2 BRDA is just beginning to reach an elevation that is appropriate for instrumentation to be installed and several instruments have been installed during 2020 and 2021.

Temporary instrumentation has been and will continue to be installed for specific purposes and durations i.e., vibrating wire piezometers were installed for the early stage raises in the Phase 1 BRDA and during the construction of the starter dams for the Salt Cake Disposal Cell (SCDC). Additional vibrating wire piezometers are scheduled to be installed during Q1 2022 in the sector of the BRDA nearest to the permitted Borrow Pit. These temporary installations serve a purpose for a particular timeframe and are not retained in the long-term monitoring programme for the BRDA.

Drawings 01 to 04 in the Plan (Appendix M) show the locations of current instrumentation installed in the BRDA and Drawing 05 shows the locations of instrumentation proposed to be installed to Stage 10. Drawing 09 in Appendix B shows the proposed instrumentation plan for the BRDA Raise to Stage 16. Table 27 below shows a summary of the instrumentation currently installed in the BRDA (July 2021), the remaining instrumentation to be installed at Stage 10 and the instrumentation proposed to be installed at Stage 16.

Table 27: Summary of Active BRDA Monitoring Instrumentation

BRDA Status	Phase 1 BRDA		Phase 2 BRDA	
	Piezometers	Inclinometers	Piezometers	Inclinometers
July 2021	67	35	8	0
Stage 10	61 (+3, -9)	33 (+2, -4)	48 (+40)	16 (+16)
Stage 16	75 (+14)	45 (+12)	72 (+26)	28 (+12)

Note: *(-) refers to instruments lost on the south sector of the Phase 1 BRDA due to merger with the Phase 2 BRDA*

10.2.2.1 Phreatic Surface and Hydrostatic Pore Pressure

Piezometers are installed to measure the two phreatic surfaces that exist for the BRDA facility i.e., within the stored bauxite residue body and within the foundation estuarine soils. They are read at a quarterly frequency (at a minimum) and plotted on the respective stability Sections to determine the phreatic surface to be subsequently utilized in the stability assessments and to assess the hydrostatic pore pressures bauxite residue body and in the foundation soils. The new readings are compared to the readings from the previous quarter and to historic readings to monitor trends.

- Standpipe Piezometers are installed to varying depths within the farmed and unfarmed bauxite residue at multi-levels of the benches upstream of stage raises along defined stability Section lines.
- Casagrande Piezometers are installed into the estuarine soils beneath the Phase 1 BRDA (where no geosynthetic lining system is present) at multi-levels of the lower-level benches upstream of stage raises and Standpipe Piezometers are installed at the downstream toe i.e., downstream of the outer perimeter wall of the PIC. These installations are also aligned with the defined stability Section lines.

10.2.2.2 Lateral Movement

Inclinometers are installed to measure lateral movement or horizontal deflection of the BRDA side-slopes. They are installed to varying depths at multi-levels of the benches upstream of stage raises along defined stability Section lines and extend through the farmed bauxite residue and/or the unfarmed bauxite residue, the underlying estuarine soils (if present) and are anchored in the bedrock beneath the Phase 1 BRDA (where no geosynthetic lining system is present).

Inclinometers for the Phase 1 BRDA Extension are installed to a depth of approx. 2m above the base as a basal composite lining system is present. The inclinometers proposed for the Phase 2 BRDA will be installed to a similar depth as a basal composite lining system is also present.

All inclinometers have been installed with the A-axis perpendicular to the slope face, with the negative readings indicating displacement downslope and the positive readings indicating upslope movement. The B-axis indicates movement parallel to the slope and therefore tend to be less of a concern. The new readings are compared to the readings from the previous quarter and to historic readings to monitor trends.

10.2.2.3 Vertical Movement

The vertical movement (heave or settlement) of the bauxite residue is measured via extensometers (also known as spiders or magnets) which are installed in clusters of 2 to 6 at regular intervals along the shaft of select inclinometers. The extensometers can slide on the casing along the vertical axis of the inclinometer and their relative vertical movement is determined from a datum magnet which is installed near the base of the inclinometer. The uppermost extensometer indicates the maximum vertical movement. The new readings are compared to the readings from the previous quarter and to historic readings to monitor trends.

Extensometer clusters have been installed on 22 of the 35 inclinometers locations in the Phase 1 BRDA as rate and extent of settlement of the unfarmed bauxite residue was a central monitoring element in the early assessment of the BRDA performance. Since the farmed bauxite residue has been demonstrated to exhibit much lower levels of settlement, it is envisaged that a lesser frequency of extensometer installation will be required for the Phase 2 BRDA.

10.2.3 Water Balance (PIC, SWP and LWP)

The water level in the PICs, the SWP and LWP are measured in real time and continuously via Vega radar water elevation probes installed at various locations and these structures are inspected daily by the AAL BRDA Operators. The water inventory, flood capacity and freeboard are determined from the water levels.

10.2.4 Seepage

The PIC is designed to capture surface water runoff, sprinkler water, bauxite residue bleed water and seepages from the BRDA stack and return the waters to the ECS.

AAL have installed monitoring and redundancy systems downstream of the PIC to assess and manage seepages that bypass the primary system. These systems are discussed in detail in Section 6.10.

Flow rates from seepage / groundwater return systems are recorded and samples are taken for water quality testing to compare to previous data and assess trends.

10.2.5 Reporting, Review and Auditing Requirements

The Plan internal and external reporting, review and auditing requirements are listed below:

Internal

- AAL prepare an EoR Monthly Communication Report which is distributed to Golder.
- Golder provide a Quarterly Review Memorandum following the quarterly reading of the monitoring instrumentation, a visual inspection of the BRDA and the review of the EoR Monthly Communication Reports, prepared by AAL.
- AAL host a quarterly EoR meeting, at which is presented the quarterly EoR BRDA Review. The minutes of these meetings are stored internally by AAL along with the quarterly EoR BRDA Review presentations.
- In accordance with *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence the BRDA is subject to an Annual Review. The Annual Review has been conducted by Golder since 2004. A Report on the Annual Review and the Annual BRDA Status is compiled by AAL for inclusion in the AER which is submitted to the Agency.
- Internal Audits are conducted by AAL as part of Environment Management System which is certified to ISO14001:2015

External

External Audits are a system for evaluating the performance and safety of the BRDA on a regular basis by qualified and experienced experts and may be conducted by the Design Engineer(s) and/or EoR or they may be Independent External Audits i.e., someone(s) who was/is not involved with the design or overall service.

- In accordance with *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence the BRDA is subject to an Independent Audit every 2 years.
- The External Independent Audit is arranged by the EoR and is conducted by Senior Golder and/or other Senior Consultants who are external to the overall service, and a Report is submitted to AAL. The most recent Independent Audit was conducted by Golder Canada in 2018. COVID-19 restrictions prohibited an Independent Audit planned for Q1 2021 and it is currently rescheduled for Q1 2022.
- In accordance with *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence the BRDA is subject to a Safety Evaluation of Existing Dam (SEED) Audit at a minimum frequency of 15 to 20 years. The SEED Audit will be conducted in accordance with the Canadian Dam Association (CDA) Dam Safety Review (DSR) Guidelines (2014) by an external mine waste practitioner consultant who is independent of the EoR, and a Report is submitted to AAL. The most recent CDA DSR was conducted by SLR Consulting Limited in 2019.

10.3 BRDA Operational Control Documents

AAL maintain and update (minimum of annually) an Operation, Safety and Maintenance (OSM) Manual for the BRDA which is structured in accordance with the Mining Association of Canada (MAC) Guide for Developing an Operation, Maintenance and Surveillance Manual for Tailings and Water Management Facilities (MAC 2019).

In accordance with Conditions of 8.5.10, 8.5.11 and 8.5.12, AAL also maintain a detailed Operational Plan and Safety Manual for the BRDA.

The operating procedures for the BRDA are directed by a series of stand-alone Standard Work Method (SWM) documents which are prepared, maintained and updated by the AAL BRDA Engineering Team.

10.4 Engineer of Record

Golder has been the AAL appointed Engineer of Record (EoR) since 2018. Engaging EoR services is an industry-recognized best practice for owners seeking to reduce overall risk, to optimise practices, and to reduce costs associated with mine waste management.

The EoR is responsible for determining whether the final design, including drawings, technical specifications, and operating procedures, meets the applicable standards, criteria, and guidelines for the integrity of the BRDA facility. The EoR is responsible for the design satisfying the design criteria, including the performance criteria, established in the design report. The following comprise the EoR tasks and responsibilities for 2021:

- Attend quarterly EoR meetings with the AAL BRDA Management to to discuss the BRDA activities during the previous quarter and the planned BRDA activities for the upcoming quarter.
- Undertake quarterly monitoring of the geotechnical instruments installed at the facility, and review of the data to assess performance of the facility. A Quarterly Monitoring Technical Memorandum will be prepared to document the review.
- Perform regular EoR inspections of the BRDA. The frequency of inspections and reviews will be based on the performance of the facility but will be a minimum a 2-monthly site visit frequency i.e., 6 visits per year.
- An Annual Site Inspection and an Annual Review / Annual Audit of the BRDA will be conducted which comprises the inspection of the facility by an independent Golder Senior Mine Waste Practitioner and the summary and interpretation of the monitoring data reported during the year.
- Provide CQA site engineers, as required, during construction works. This will include oversight, monitoring and quality assurance services of construction and operations to assess compliance with the design.
- Review and comment on operational planning for dam raise construction, bauxite residue deposition and water management, as requested by AAL.
- Provide assistance and alerts on the operating review for any concern trends, issues, or unexpected performance and provides recommendations, advice, and action items to be taken.
- Review and comment on designs and construction of other structures associated with the BRDA that may affect the integrity of the structures of the BRDA or influence the performance.
- Conduct routine site investigations to characterise the bauxite residue and assess the performance of the facility. The site investigation typically consists of Cone Penetration Testing with pore pressure measurement (CPTu) performed every four years, or as required / requested.
- Additional design work or technical review of other structures which may impact the design and performance of the BRDA, as requested by AAL.

11.0 BRDA EMERGENCY PLANNING

Condition 9.4 of the IEL details the requirements for the emergency planning for the BRDA and documented procedures are detailed in the BRDA OMS Manual.

- AAL maintain and update a Major Accident Prevention Policy.
- AAL have a designated Safety Manager for implementation of Major Accident Prevention Policy.
- AAL maintain and update a Safety Management System to implement the Major Accident Prevention Policy.
- AAL maintain and update an Internal Emergency Plan.
- In accordance with Condition 9.4.5, AAL consult with the Local Authority and the Principal Response Agencies in relation to any information that may be required by them regarding external emergency planning for major accidents at the BRDA. Evidence of these consultations is provided in the Annual Environmental Report (AER).

12.0 BORROW PIT EXTENSION

12.1 Background and Current Status

AAL submitted a planning application in July 2017 to develop a Borrow Pit to extract rock in Aughinish East, based on a rock fill requirement of circa 374,000 m³ of rock fill (from 2018) to provide for ongoing construction of the BRDA over the lifetime of the permitted development to Stage 10.

The application was approved Board Order ABP-301011-18 in November 2018 and a new IE licence (P0035-07) was issued by the EPA in September 2021, which provides Conditions for the operation of the Borrow Pit (Conditions 5.12, 6.16, and 6.18).

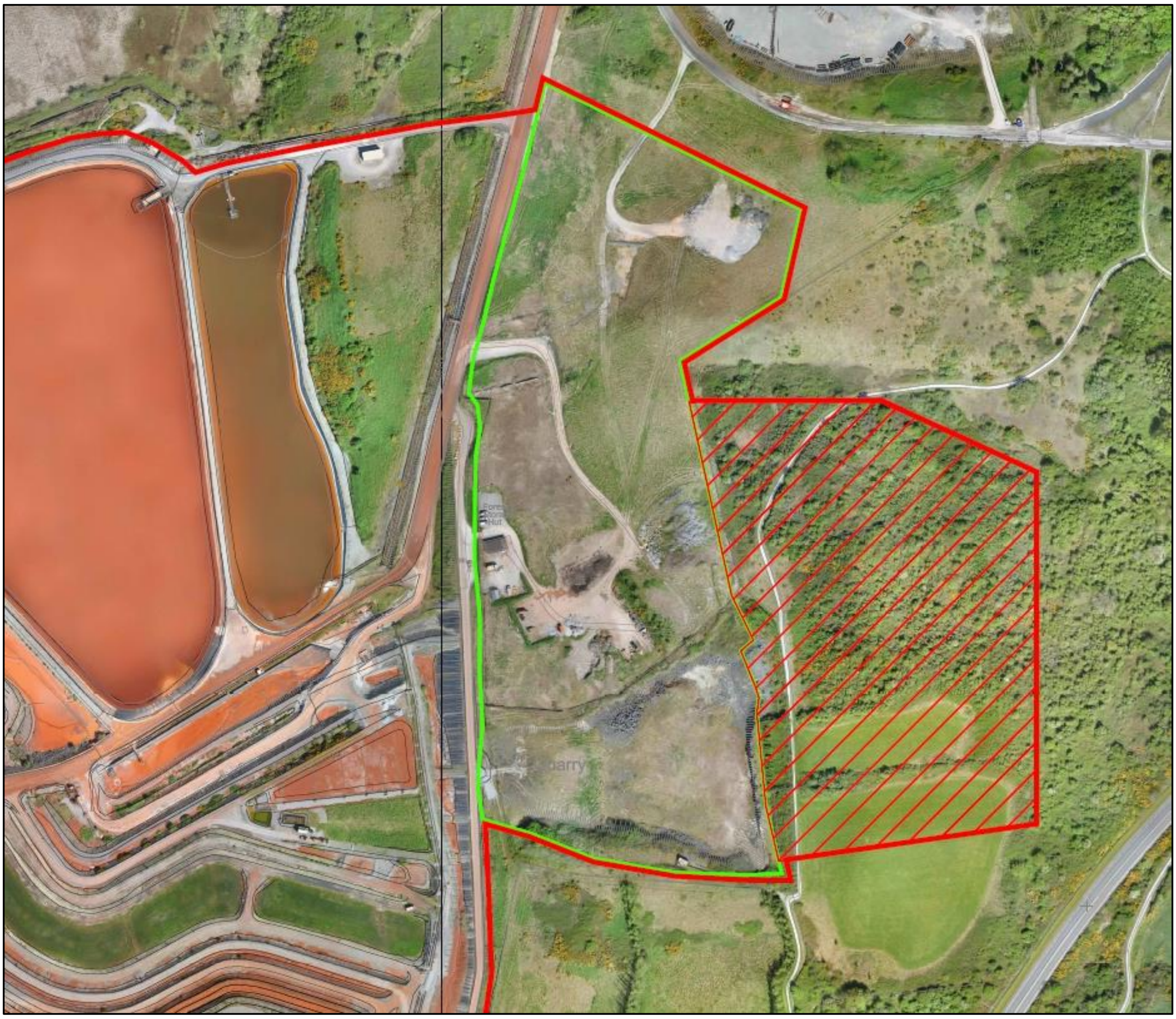

Figure 40: Permitted Borrow Pit Footprint (green) and the Proposed Borrow Pit Extension Footprint (red hatch), April 2021

The proposed Borrow Pit footprint is within the AAL facility and is located to the south of the Plant and to the east of the Phase 1 BRDA. It is an area of previously disturbed ground; the central and northern sectors had been historically used as rock fill stockpile yard and had since been rehabilitated during 2013. The southern sector comprised the compound area for the on-site Landscaping Contractor and a former Borrow Pit for rock that was operational in the early 1980s.

The approved Borrow Pit extraction area is c. 4.5 hectares with extraction permitted from surface at 16 mOD to 17 mOD elevation to a base elevation of 8.5 mOD, resulting in an average depth of circa 8m for the footprint. The overall development footprint is circa 7.0 hectares and includes the footprint of the previously developed Borrow Pit that has been extracted to been 8.0 mOD and 9.0 mOD. The floor of the previous Borrow Pit is proposed to be utilized for processing and stockpiling of the rock fill following blasting and extraction.

The proposed Borrow Pit Extension is an eastern expansion of the permitted footprint (see Figure 40) and is a 3.9 ha expansion of the extraction area to provide an additional ≈ 380,000 m^3 of rock fill material. The additional footprint for the overall expansion is 4.6 ha, which includes the offset from the extraction crest, screening berm, perimeter road and boundary fencing.

12.2 Site Conditions (Geotechnical and Hydrological)

The site conditions for the Borrow Pit Extension footprint and surroundings are discussed in Sections 3.2, 3.3, 3.5 and 3.6.

The bulk of the footprint has a shallow till covering and has a heavy vegetation overgrowth of scrub, bushes and gorse. Two meadows are present in the southern sector and the former nature trail traverses the western sector of the footprint from south to north. An assessment of the trees present in the footprint has been conducted and did not identify any trees of significance.

The average ground elevation in the footprint is ≈ 20 mOD, with a slightly mounded central portion at 21 mOD to 22 mOD reducing to ≈ 19 mOD at the north and south extents.

No services, utilities or water bodies are present within the footprint and the groundwater contours vary from approx., 6 mOD in the northern sector to approx. 1 mOD in the southern sector.

12.3 Geometric Design

The prosed design and phasing for the Borrow Pit Extension are shown on Drawings 08a and 08b and comprises the following:

- Eastern extension of the existing Borrow Pit footprint with new boundaries to the north, east and south.
- Overall site area of 4.6 ha with an extraction footprint of 3.9 ha to provide ≈ 380,000 m^3 of rock fill material.
- Extraction depth of 11.5m to 12.5m to a base elevation of 8.5 mOD
- Single excavated face at a maximum 70-degree angle.
- 3m offset from excavation crest to toe of landscaped screening berm, to be constructed to a minimum 2m height
- 5m offset from downstream toe of landscaped screening berm for perimeter road.
- Minimum 2m high security boundary fencing.

12.4 Life of Borrow Pit

The rock fill material requirement for the BRDA Raise Development is detailed in Table 14 and in Section 6.8 and rock fill material is expected to be required for stage raise construction up to Stage 16 and additional volumes will be required for the closure works.

The proposed development of the Borrow Pit Extension is depicted on Drawing 08c and envisages that the original Borrow Pit footprint will be advanced sufficiently to the north to expose the west face of the new Borrow

Pit Extension footprint. It is then proposed to alternate the development of both footprints i.e., from south to north for the original Borrow Pit and from west to east for the Borrow Pit Extension, based on an extraction volume of ≈ 50,000 m^3/year. It is anticipated that 6 to 8 blasts per year will be conducted.

The total extraction volume for the overall Borrow Pit is ≈ 754,000 m^3. It is considered likely that the Borrow Pit and Borrow Pit Extension will be fully developed prior to the end of storage life of the BRDA, and that sufficient rock fill material will be stockpiled for the remaining stage raise construction and closure works.

12.5 Borrow Pit Operation

The Borrow Pit Extension is proposed to be operated under similar conditions to those specified in the Board Order ABP-301011-18 and the Conditions of the new IEL (P0035-07) issued by the EPA in September 2021. The principal Conditions are listed below:

- No more than one blast per week.
- No blasting outside of the hours 08:00 to 18.00.
- No blasting from 01 October to 31 March, inclusive each year.
- The recommendations of Blast Vibration Assessment (Golder 2017A) shall be implemented for each blast or as otherwise approved by the EPA.
- All environmental mitigation and screening measures recommended in the EIAR shall be implemented in full.

It is envisaged that the Borrow Pit Extension will be managed by AAL in a similar fashion to the process in place for the current Borrow Pit, i.e., tendered to an external Contractor to operate, which comprises:

- Enabling Works Tender Package
  - Stripping of the vegetation and topsoil from the permitted footprint;
  - Construction of boundary fencing, bunds, entrances and access roads for the development; and
  - Demolition and removal of any structures present.
- Operational Works Tender Package (minimum 5-year period)
  - Blast design, drilling, coordination with explosive suppliers and appropriate authorities, blasting and monitoring (vibration and air overpressure) in accordance with the development and operational conditions for the Borrow Pit and agreements with stakeholders.
  - Loading, hauling, crushing, screening and stockpiling of rock fill material to the AAL requirements

12.6 Construction Materials and Quantities

The Contractor will be required to produce the required grades and quantities of materials from the rock fill generated from blasting, via processing through crushers and screeners. Three material types are envisaged to be required, which are termed:

- Type B material;
- Type C material;
- Type D material; and
- Type F material.

12.6.1 Type B Material

Type B material is primarily used for stage raise construction and is suitable rock fill material complying with the following:

- Blasted, ripped and/or excavated rock or weathered rock which is processed;
- Maximum particle size is 250mm in the minimum dimension.

12.6.2 Type C Material

Type C material is primarily used for road surfacing and liner protection and is suitable rock fil material complying with the following:

- Blasted, ripped and/or excavated rock or weathered rock which is processed;
- Well grade material and particle size distribution complying with Table 28 below:

Table 28: Type C Material - Particle Size Distribution Limits

Sieve Size (mm)	% Passing
50	100
20	65-90
6.3	30-60
1.18	15-30
0.300	5-20
0.075	0-10

12.6.3 Type D Material

Type D material is suitable drainage stone material complying with the following:

- Blasted, ripped and/or excavated rock or weathered rock which is processed;
- Maximum particle size of 60mm; and
- Minimum particle size of 20mm.

12.6.4 Type F Material

Type F material is suitable gabion basket fill material complying with the following:

- Hard, durable and free from deleterious materials;
- Maximum particle size of 150 mm; and
- Minimum particle size of 100 mm.

12.6.5 Material Quantities and Quality Control

A typical Contractor specification to produce and stockpile during the 5-year period is provided in Table 29 below

Table 29: Typical Contractor Specification for 5-year Borrow Pit Operation

Material Type	Year 1 (m^3)	Year 2 (m^3)	Year 3 (m^3)	Year 4 (m^3)	Year 5 (m^3)
Type B	48,500	46,500	48,500	47,500	48,500
Type C	1,500	1,500	1,500	1,500	1,500
Type D	-	1,000	-	1,000	-
Type F	-	1,000	-	-	-
	50,000	**50,000**	**50,000**	**50,000**	**50,000**

The material types shall be subject to inspection, sampling and conformance testing to determine the suitability of the material.

12.7 Blast Vibration Assessment

Golder 2017A provides an assessment for the blasting associated with a proposed Borrow Pit development, potentially impacting on the embankments and raises associated with Phase 1 BRDA and provides recommendations for conducting the blasting and monitoring during the Borrow Pit Development.

This assessment is summarized in Section 7.4 and the full assessment is provided in Appendix E. Drawing 08d provides an estimated blast PPV contour map based on a blast conducted at the south-east corner of the proposed extraction area and for site parameters and blast parameters selected. The assessment concluded that that the blasting at the Borrow Pit would not cause instability of the BRDA, due to vibration of the blast itself or as result of residual excess pore generated by the blast wave.

The new IEL (P0035-07) issued in September 2021 provides Conditions for the operation of the Borrow Pit i.e., Noise, Vibration and Air Overpressure thresholds and monitoring locations. The Contractor is required to design the blasts and operate the Borrow Pit Extension to comply with these Conditions and the additional stakeholder requirements i.e., PPV limits for the BRDA and for the Gas Transmission Pipeline.

The mitigating factors for the Borrow Pit Extension development are:

- The Borrow Pit Extension footprint is at a greater distance from the BRDA than the approved Borrow Pit which will reduce the potential effects recorded at the BRDA.
- A number of years of blasts will have completed at the Borrow Pit prior to the development of the Borrow Pit Extension which will permit fine-tuning of the site and blast design parameters.

12.8 Closure Plan

A restoration landscaping proposal was prepared by Brady Shipman Martin Landscape Architects (BSM) for the original Borrow Pit development which comprised a combination of natural regeneration of vegetation with additional hedge and tree planting.

BSM have updated the restoration landscaping proposal to encompass the enlarged footprint provided by Borrow Pit Extension and the drawing and details are provided in Chapter 8.0 of the EIAR.

13.0 SALT CAKE DISPOSAL CELL RAISE

13.1 Background and Current Status

As part of the planning application to extend the BRDA in 2005 (LCC 05/1836) a composite lined, independent waste management facility was proposed to be constructed in the north-east sector of the Phase 1 BRDA for the disposal of salt cake, as it constituted a hazardous waste in accordance with the Hazardous Waste Directive (91/689/EEC). Planning was granted for the BRDA extension in 2007 (ABP 13.217976) and the initial phase of the Salt Cake Disposal Cell (SCDC) was constructed during 2012/2013 to a crest elevation of 24 mOD.

The current SCDC is detailed in Section 6.13.3 and shown in Figure 41 below and comprises a triangular shaped, independent cell that has been constructed in three (3) phases above circa 18m depth of bauxite residue, which also is underlain by a basal composite lining system. The cell comprises the north, east and west embankments and a decant area located in the north-east corner. The west embankment is the tipping wall.

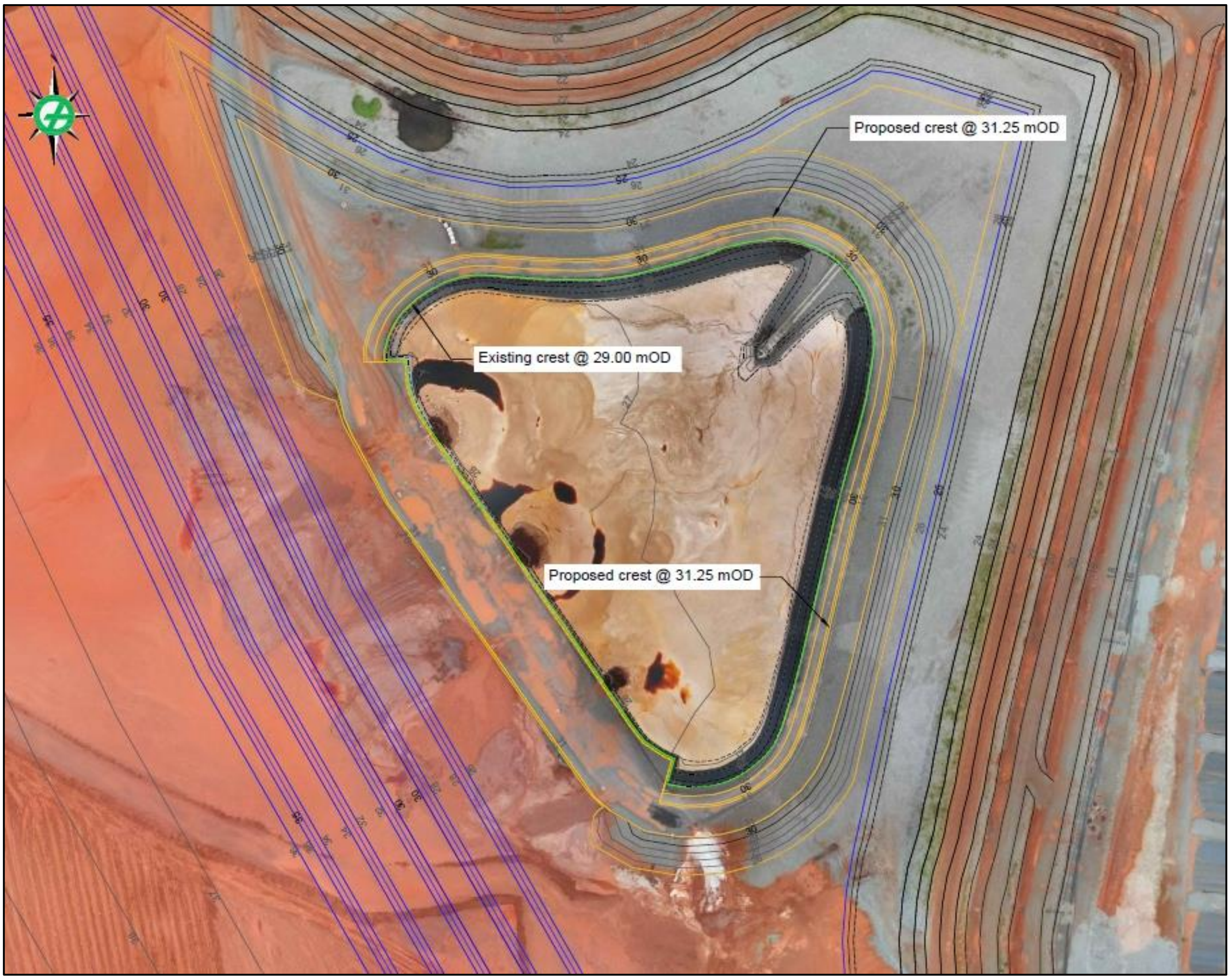

Figure 41: Salt Cake Disposal Cell and Surroundings (October 2020)

It is proposed the SCDC be vertically extended to accommodate further storage of salt cake within its current footprint (circa 22,500 m^3 of storage) and to provide the equivalent of 3 years storage capacity. This additional capacity will be utilized until the SWOP is commissioned and during downtime / maintenance of the SWOP for the operational life of the Plant and BRDA. The proposed increase in height is 2.25m which will comprise a single raise to provide a new perimeter crest elevation of 31.25 mOD.

13.2 Site Conditions (Geotechnical and Hydrological)

The site conditions for the SCDC footprint and surroundings are discussed in Sections 3.2, 3.3, 3.5 and 3.6.

The SCDC embankments are constructed of Type B rock fill material (see Section 6.7) which are composite lined on the upstream side-slopes with GCL overlain by 2mm HDPE geomembrane. The SCDC is encompassed by farmed bauxite residue to the west and south at an elevation of 25 mOD to 27 mOD, and by process sand overlying farmed bauxite residue to the north and east at an elevation of 24 mOD to 25 mOD. The process sand has been overlain with a thin layer of Type C rock fill material (< 20mm) to alleviate potential dusting.

The SCDC is accessed by a rock fill ramp extending to the north to join the Central Access Ramp for the Phase 1 BRDA.

13.3 Geometric Design

The prosed design for the SCDC Raise is shown on Drawing 07 and comprises the following:

- Downstream raise of the north and east embankments by 2.25m, from 29.0 mOD to 31.25 mOD, following a similar construction methodology to Phase 2 and Phase 3 of the SCDC:
  - The upstream toe shall be offset a minimum of 1m from the existing crest to permit at flat bench for the seaming of the lining system.
  - Embankments to be constructed of Type B rock fill material with a minimum 300mm depth of Type C rock fill material placed on the upstream slope to provide a suitable subgrade for the lining system and a minimum 200mm depth of Type C rock fill material at the crest for road surfacing.
  - The upstream slope shall be a maximum of 2.5(H):1(V), the overall crest width shall be a minimum of 8m, and the downstream slope shall be a maximum of 2.0(H):1(V)
  - Composite lining system on the upstream slope comprising a GCL overlain by a 2mm HDPE Geomembrane which is overlapped and seamed to the crest of the existing lining system and secured in an anchor trench at the new crest.
  - Vehicle crash barriers to be installed at the downstream crest and offset from the anchor trench on the upstream crest.
- Centre-line raising of the west embankment by 2.25m, from 29.0 mOD to 31.25 mOD, following a similar construction methodology to Phase 3 of the SCDC
  - The upstream toe shall be offset a minimum of 1.5m from the existing crest to permit at flat bench for the seaming of the lining system.
  - Embankment to be constructed using gabion terramesh baskets retaining walls on the upstream and downstream crests. Type D rock fill material shall be used to infill the gabions. Type B rock fill material to be backfilled in layers between the retaining walls and reinforced with geogrids connecting the terramesh tails. A minimum 200mm depth of Type C rock fill material at the crest for road surfacing.
  - The upstream and downstream slopes shall be approximately vertical, and the overall crest width shall be a minimum of 22m.
  - Composite lining system on the upstream slope comprising a protection geotextile on the face and crest of the gabion retaining wall, overlain by GCL, overlain by a 2mm HDPE Geomembrane which is overlapped and seamed to the crest of the existing lining system and secured in an anchor trench at the new crest.

- Tipping plates to be installed at designated locations on the upstream crest and protective layers for the lining system to be installed at the tipping locations comprising protection geotextile, tyres filled with Type C rock fill material and overlain with conveyor belt.
- Vehicle crash barriers to be installed at the downstream crest and offset from the anchor trench on the upstream crest.

- Raising and re-grading of the Access Ramp and Turning Circle using Type B rock fill material for the bulk of the construction and Type C rock fill material for road surfacing.
- Extension of all services for the SCDC i.e., lighting poles, sprinkler lines, sprinkler heads, DAP stations etc.
- Extension of the Decant Area including Decant Tower and Access Path.

13.4 Life of Facility

The existing SCDC has a storage footprint of approx. 1 ha. at a perimeter crest elevation of 29 mOD providing a maximum storage volume of ≈ 72,800 m^3 (no freeboard) and a storage depth of ≈ 9m. The remaining capacity at the end of June 2021 was estimated to be ≈ 12,400 m^3.

AAL are in the process of developing a Salt Cake Wet Oxidation Plant (SWOP) which will be located within the Plant, with the objective of removing salt cake from the waste stream. The proposed SDCC Raise will provide ≈ 22,500 m^3, which is the equivalent to 3 years storage capacity. This additional capacity will be utilized until the SWOP is commissioned and during downtime / maintenance of the SWOP for the operational life of the Plant and BRDA.

The storage footprint at the proposed crest elevation of 31.25 mOD is 1.45 ha and it will have a storage depth of ≈ 11m and an overall storage volume of ≈ 95,000 m^3 at a 1m freeboard.

13.5 SCDC Construction

It is envisaged that the construction of the SCDC will be managed by AAL in a similar fashion to the previous construction phases of the SCDC, namely:

- The Detailed Design shall be completed for the SCDC Raise.
- A Design Report and CQA Plan shall be submitted to the EPA for approval.
- A Tender Package comprising a Specification, Drawings and Bill of Quantities shall be prepared and a Contractor shall be selected for the Works following the tender process.
- CQA attendance shall be conducted during the Works.
- A CQA Validation Report shall be submitted to the EPA following completion of the Works.

The expected duration for the Works is 4 months and the target commencement date is March 2023.

13.5.1 Earthworks Materials

It is estimated that circa 27,000 m^3 of processed rock fill material will be required to construct the SCDC Raise. The rock fill materials are proposed to be sourced from the current permitted Borrow Pit and comprises three (3) material types which are termed:

- Type B material;
- Type C material;

- Type D material; and
- Type F material;

These rock fill material types are detailed in Section 12.6.

13.5.2 Geosynthetic Materials

It is estimated that circa 4,500 m^2 of composite lining is required to construct the SCDC Raise. The geosynthetic materials will be sourced by the Contractor, typically from within the European Union, and comprises four (5) material types:

- Gabion terramesh baskets;
- Geogrid (Paralink 300 or similar approved)
- Separation Geotextile (min. 200 grms/m^2)
- Protection Geotextile (min. 1,000 grms/m^2)
- GCL (min. 4,900 grms/m2)
- Geomembrane (2mm HDPE, Double-Textured)

13.5.3 Ancillary Materials

The following ancillary materials are considered to be required to complete the Works and will be sourced by the Contractor and/or supplied directly by AAL:

- Non-calcareous drainage and gabion rock fill (Type D1 and Type F1)
- Decant Tower - 1,050 mm diameter, 'Weholite' Structured Wall Pipe (or similar approved)
- Crash Barrier
- Concrete for posts, plinths and paths
- Conveyor Belt

13.6 Closure Plan (Dome and Side-Slopes)

A specific capping containment design, appropriate for the capping of a hazardous waste material, is proposed for the SCDC Raise which is accordance with the EPA approved design for the current SCDC (Golder 2017B).

The proposed capping containment design takes into account Condition 8.5.21 of the licence (IEL P0035-07) requiring the final 1m of all exposed bauxite residue deposited in Phases 1 and 2 of the BRDA shall comprise 'amended mud' and the on-going 'amended' layer trials at Aughinish.

The proposed capping containment capping design is depicted in Figure 2 and is detailed below:

- placement of process sand into the stored salt cake to consolidate the upper surface and form a working platform
- placement and grading of a minimum 0.5m layer depth of process sand above the working platform
- installation of a composite lining system comprising:
  - minimum 4,000 grms/m^2 GCL
  - single-textured 2mm HDPE geomembrane

- placement of a minimum 0.3m depth of capillary break (process sand / suitable processed rock fill)
- placement of a minimum 1.0m depth of 'amended mud' graded to the BRDA dome profile. The amended layer is composed of farmed bauxite residue, process sand, gypsum and organic soil improver and is the focus of on-going trials at Aughinish.
- Vegetative layer.

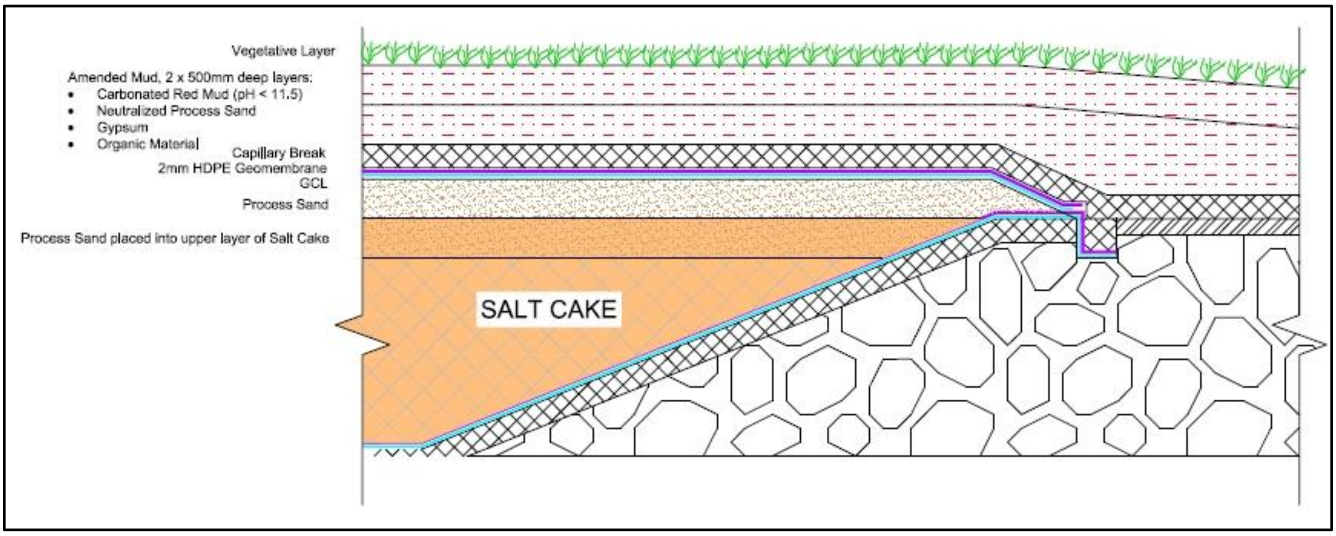

Figure 42: Proposed Capping Containment Design for the SCDC Raise

The prosed SCDC Raise dome blends into the overall BRDA dome at Stage 16. The final contoured closure dome for the proposed SCDC Raise is shown on Drawing 04 and comprises a surface profile which drains to the south from 35.5 mOD to 33.0 mOD. Spillways have been designed to carry the surface runoff from the SCDC cap down to Stage 11 and subsequently down to PIC-L, see Section 8.3 and Appendix J.

The side-slopes will be capped and remediated using a similar methodology to the rest of the BRDA, see Section 8.4 and Appendix K.

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APPENDIX A

Basis of Design and Design Criteria / Parameters

TECHNICAL MEMORANDUM

DATE 30 November 2021 **Project No.** 20143076.TM01.A3

TO Thomas Hartney, Aughinish Alumina Limited

CC Kevin McMahon (AAL), Gerd Janssens, Dave Buxton, Billy Murphy (Golder)

FROM Brian Keenan **EMAIL** bkeenan@golder.com

ENGINEERING DESIGN OF THE RAISE OF THE PHASE 1 AND 2 BRDA TO STAGE 16: BASIS OF DESIGN AND DESIGN CRITERIA

- This design criteria document was prepared in accordance with the Golder Tailings Design Practice Manual.
- This document has been prepared as a comprehensive template for all levels of design for tailings storage facilities. The user of this template should/may eliminate/add information to meet the needs of the specific project and client.
- This template includes general criteria for the design of the tailings thickening and distribution systems for front-end studies. As the project advances to pre-feasibility level design and beyond, the design criteria for mechanical, structural, electrical components are very detailed and outside the battery limits of this document.
- It is important that everyone on the project (Golder, client, other consultants, etc.) are working from the same set of design criteria. As such, this document should be considered a living document during all phases of the project and should updated and re-issued to the client as the project evolves.
- Separate design criteria documents may be required for different structures or ponds within the tailings management facility, i.e., spillways, salt cake disposal cell, wetlands area etc. Alternatively, some tables included in this document may be duplicated to account for the different design criteria for the various structures.

Prepared By:	Golder Associates Ireland Ltd	Brian Keenan	
	Project Manager	**Name**	**Signature**
Reviewed By:	Golder Associates Ireland Ltd	Gerd Janssens	
	Tailings Design	**Name**	**Signature**
Reviewed By:	Golder Associates Ireland Ltd	Christine Campbell	
	Hydrological Design	**Name**	**Signature**
Approved By:	Golder Associates UK Ltd	Dave Buxton	
	Project Director	**Name**	**Signature**
Approved By:	Aughinish Alumina Limited	Thomas Hartney	
	Client / Owner	**Name**	**Signature**

Table 1: Revision Record

Revision				Areas Revised	Remarks
No.	**By**	**Approved**	**Date**		
B.0	BK		21/08/2020		
A.0	BK		20/07/2021		Following review by AAL Team
A.1	BK		04/08/2021		Final review comments by AAL Team
A.2	BK		02/11/2021		Following review by SLR
A.3	BK		30/11/2021		Final comments by AAL Team

Instruction to Print Control (Indicate X where applicable):

☒ Entire design criteria revised – reissue all pages

☐ Partial revisions to design criteria – reissue all pages

☐ Reissue revised pages only

Stamp Design Criteria as Follows:

☒ Issued for review and comment

☐ Released for study

Table 2: Source and Discipline Code

Source		**Discipline**	
A	Assumed	CG	Civil/Geotechnical
B	Calculated	Wr	Water Resources
C	Client Information/Request	En	Environmental
G	Golder Associates Inc.	Se	Seismic
I	Industry Standard Practice	St	Structural
O	Information Provided by Others	Pr	Process
P	Published Information/Criteria	Pi	Piping
D	Database/Recommendation	El	Electrical
T	Testwork Data	In	Instrumentation
V	Vendor Data	Ar	Architectural
R	Regulatory/Code	Me	Mechanical

Table 3: List of System of Units, Abbreviations and Acronyms

Project System of Units: System International (SI)			
Mass/Density		**Other**	
kN	kilonewton	%	percent
g	gram	m^3/h	cubic meter per hour
kg	kilogram	cm/s	centimeters per second
tonne	metric ton (1,000 kg)	tpd	tonnes per day
kN/m3	kilonewton per cubic meter	g	gravitational acceleration (9.81 m/s^2, 32.2 ft/s^2)
kg/m3	kilogram per cubic meter	°C	degrees Celsius
		X(H):1(V)	X horizontal to 1 vertical
Length		PGA	peak ground acceleration
mm	millimeter	PHGA	peak horizontal ground acceleration
cm	centimeter	OBE	operating basis earthquake
m	meter	MDE	maximum design earthquake
km	kilometer	MCE	maximum credible earthquake
		FoS	factor of safety
		IDF	inflow design flood
		PMF	probable maximum flood
Area		PMP	probable maximum precipitation
m^2	square meter	mOD	meters above ordnance datum
ac	acre	P80	particle size (dia.) at which 80% (by mass) is finer
ha	hectare	TBD	to be determined
		NA	not applicable

Volume		**Environmental**	
cm^3	cubic centimeter	ARD	acid rock drainage
m^3	cubic meter	ML	metal leaching
		PAG	potential acid generation
Time		NPR	neutralizing potential ratio
yr	year	NAG	non-potentially acid generating
d	day	Mg/l	milligrams per litre (concentration)
h	hour	uS/cm	micro siemens per centimetre (conductivity)
min	minute	m/s	metres / second (hydraulic conductivity)
s	second		
		Pertinent Acronyms	
Electrical		ANCOLD	Australian National Committee on Large Dam
kW	kilowatt	BAT	Best Available Techniques
kWh	Kilowatt hour	BREF	Best Available Techniques Reference Documents
		CDA	Canadian Dam Association
Viscosity		EAP	Emergency Action Plan
Pa s	Pascal-second	EC	European Commission
		EMP	Environmental Management Plan
Pressure		EU	European Council
kN/m^2	kilonewton per square m	GARD	Global Acid Rock Drainage
kPa	kilopascal	GISTM	Global Industry Standard on Tailings Management
		HPC	Hazard Potential Classification
		IEL	Industrial Emissions Licence
		INAP	International Network for Acid Prevention
		LoF	Life of Facility
		MEND	Mine Environment Neutral Drainage
		MWEI	Mine Waste Extractive Industries
		OSM	Operational, Safety and Maintenance
		SWMs	Standard Work Methods
		UL	University of Limerick

1.0 INTRODUCTION

Aughinish Alumina Limited (AAL) is wholly owned by United Company RUSAL (UC Rusal) and operates the alumina refinery situated on Aughinish Island on the south side of the Shannon estuary. The Island is located between Askeaton and Foynes some 30 km west of Limerick and 10 km south-west of Shannon Airport. The Island is approximately 400 ha in area and is bounded by the River Shannon to the north, the Robertstown River to the west and south-west and the Poulaweala Creek to the east and south-east.

The plant and ancillary structures were constructed between 1978 and 1983 and are located at the northern top of the Island. Plant production has been increased since the commissioning of the plant in 1983 up to its current annual production of approximately 1.95 million tonnes of alumina. Bauxite residue from the production process is deposited in the BRDA located to the south-west of the process plant. The BRDA was constructed in three phases and comprises two distinct storage areas which are currently merging:

- The **Phase 1 BRDA** is formed from two facilities, the original Phase 1 BRDA constructed in the early 1980s, covering an area of 72 ha., and the Phase 1 BRDA Extension, constructed in the mid-to-late 1990s, covering an area of 32 ha.

  The Phase 1 BRDA Extension is lined with a 2 mm HDPE geomembrane while the original Phase 1 BRDA relies on the low permeability of the underlying estuarine deposits to minimise seepage from the base of the facility plus the inherent low permeability of the bauxite residue itself. The initial design for the Phase 1 BRDA was to provide storage to the year 2009 based on the facility constructed to Stage 7 (elevation 18 mOD), which equates to a central elevation of 27.5 mOD or 26m above original ground level. Approval was granted for the 2005 design report (Golder 2005C) proposing to raise the facility in three more stages (Stages 8, 9 and 10), resulting in a maximum perimeter elevation of 24 mOD and a maximum central elevation of 32 mOD and to construct the Phase 2 BRDA.

- The **Phase 2 BRDA** is a southern extension of the Phase 1 BRDA that was presented in the 2005 design report (Golder 2005D) and proposed a composite lined facility to Stage 10 with a maximum perimeter elevation of 24 mOD and a maximum central elevation of 32 mOD. The Phase 2 BRDA will overlap the southern extent of the Phase 1 BRDA, and the domes will merge. The Phase 2 BRDA covers an area of approximately 80 ha. and was commissioned in 2011.

The facilities are in the most part surrounded by a perimeter interceptor channel (PIC) which connects to the storm water pond (SWP). The method of raising the stack wall retaining the bauxite residue is by the upstream method which involves constructing a rock fill berm founded on previously deposited and farmed (since 2009) bauxite residue. The stack wall is raised systematically as the facility fills with bauxite residue in approximately 2 m high rock fill berms. The bauxite residue paste is mud-farmed, compacted and subsequently allowed to mature prior to placing the next layer. Since March 2009, the bauxite residue has been intensively mud-farmed. This process involves depositing the bauxite residue paste in purpose built internal cells within the BRDA and then using a specially adapted machine, the amphirol, which compresses the surface of the bauxite residue, reducing moisture and pH (via carbonation) and enhances the drying process by increasing the surface area of the bauxite residue exposed to the wind.

Unlike traditional upstream tailings raises using rock fill berms, where water is prevented from encroaching the stack wall, this facility retains no water. Bauxite residue is pumped to a central discharge as a paste forming a central dome. The bauxite residue migrates by gravity to the perimeter stack wall, producing averages grades between 2% and 4%. As a slope is formed towards the stack wall, surface water runoff from rainfall is diverted to the rock fill berms. The rock fill allows the runoff water and bauxite residue bleed water to pass through the berm and is transferred to the PIC at the base of the stack.

The facility is designed to operate with a high phreatic surface (design target is between 2m and 3m below surface) as the stack wall slopes are relatively shallow. The stack wall has an overall slope of 6.3(H):1(V) consisting of a lower and upper slope formed at 6(H):1(V) separated by a ≈ 28 m wide bench at Stage 5 (14 mOD).

Rusal Aughinish - BRDA April 2021

Figure 1: AAL BRDA April 2021 Survey (ASM Ireland Ltd)

<u>BRDA Current Status</u>

AAL have raised the stack wall for the **Phase 1 BRDA** to Stage 10 along the east, north-east and north-west sectors and have recently completed / currently constructing the south-west and south sectors to Stage 10. The elevation of bauxite residue deposited varies from approximately 32 mOD at the centre to approximately 22 mOD to 24 mOD at the perimeter stage raises.

For the **Phase 2 BRDA**, AAL have constructed to Stage 4 (12 mOD) along the west and south boundaries. Bauxite residue has been placed to approximately 11 mOD along the east perimeter wall, which will subsequently form the base of the internal PIC along this extent. The crest of east perimeter wall currently varies in elevation from Stage 6 (16 mOD) to Stage 4 (12 mOD) from its north-eastern extent to its eastern extent and transitions into the external PIC at the Observation Area located centrally on the east perimeter wall. The elevation of the bauxite residue deposited varies from approximately 12 mOD at the south end to approximately 20 mOD centrally along the internal access road (north-south road), splitting the Phase 2 BRDA into east and west sectors.

The Phase 1 and Phase 2 BRDAs are being progressively merged, with the Phase 2 BRDA overlapping on the upstream raises on the south face of the Phase 1 BRDA to a current elevation of approximately 15 mOD.

1.1 Project Description

On 14 February 2007, An Bord Pleanala (ABP) granted permission for the extension (addition of the Phase 2 BRDA footprint) and raising in elevation (Phase 1 and 2 BRDA to Stage 10) of the BRDA and associated modifications (perimeter interceptor channels, salt cake cell etc.) and retention for the increase in alumina production to 1.6 million tonnes per annum and for the increase in production capacity with the permitted integrated pollution licence to 1.95 million tonnes per annum, (Limerick County Council Planning Register Reference Number: 05/1836 and ABP Number: PL 13.217976).

This proposed application of the raising of the BRDA does not require any extension of footprint and will only require minor modifications to the existing ancillary facilities i.e., perimeter interceptor channel, pumping systems and culverts. However, it is proposed to raise the current permitted elevation of the BRDA at Stage 10 (perimeter crest @ 24 mOD and top of dome @ 32 mOD) by 12m to Stage 16 (perimeter crest at 36 mOD and top of dome at 44 mOD). This raise will provide an estimated 8.03 million m^3 of bauxite residue storage capacity and will extend the life of the facility (LoF) by approx. 9.0 years (based on April 2021 survey).

The existing salt cake disposal cell (SCDC) is sited in the north-east sector of the BRDA and is proposed to be raised to provide an additional approximately 23,000 m3 of storage capacity, providing the equivalent to 3 years of storage capacity.

1.2 Scope of Services

This document is intended to present the design basis and design criteria for the Aughinish Alumina Ltd (AAL) Bauxite Residue Disposal Area (BRDA). Golder is commissioned to:

- Prepare the Engineering Design Report for the Phase 1 and 2 BRDA constructed to the Stage 16 elevation and ancillary infrastructure (Salt Cake Disposal Cell and Borrow Pit Extension)
- Assist in the preparation of the Environmental Impact Assessment Report (EIAR) for the proposed development.

Table 4: General Information

Item	Value
Level of Study	Engineering Design for Planning and Approval Processes
Name of Facility	AAL BRDA
Focus of Study	Tailings Surface Disposal. Upstream raise of current facility from Stage 10 to Stage 16.
Mineral(s)	Alumina (aluminum oxide) produced from bauxite material and producing waste bauxite residue for disposal in the BRDA.

Table 5: Plant and BRDA Operation Information

Description	Quantity	Units	Data Source	Remarks / Assumptions
Plant and BRDA Information				
Ore Reserves (Mine)	N/A	tonnes	AAL AER 2020	Imported bauxite raw material, 70% Guinea and 30% Brazil
Ore Reserves (Plant)	≈ 250,000	tonnes	AAL	Maximum storage
Current Life of BRDA	9.8	years	AAL / Golder	Based on current rate of production from April 2021
Life of BRDA with proposed raise	18.5	years	AAL / Golder	Based on current rate of production for April 2021
Operation (annual)	365	days / year	AAL	-
Operation (daily)	24	hours / day	AAL	-
Ore Production (annual)	≈ 1.95 E6	tonnes / year	AAL	Alumina (aluminum oxide)
Ore Production (daily)	≈ 5,350	tonnes / day	AAL	Based on daily feed of 10,750 tonnes of bauxite raw material
Bauxite Residue Production (annual)	≈ 1.57 million	Dry tonnes / year	AAL / Golder	Waste residues by weight (AER 2020) are: • 90.7 % bauxite residue • 6.9 % process sand • 1.0 % salt cake • 1.4 % scales and sludges Bauxite residue storage ≈ 882,000 m^3 (for 2020) at 2.19 tonnes / m^3. Dry density varies from 1.58 to 1.63 tonnes/m^3 depending on unfarmed or farmed bauxite residue, respectively. Specific gravity varies from 3.3 to 3.4.
Bauxite Residue Production (daily)	≈ 4,300	Dry tonnes / day	AAL	Based on ≈ 50% grade by weight.
Rate of Rise	0.86m to 1.00m 1.25m to 1.75m	Phase 1 BRDA m / year Phase 2 BRDA m/year	AAL / Golder	Aerial survey data for Phase 1 and 2 BRDA from 2005 to April 2021 Rate of rise dependent on zonal deposition prioritization
Total Stored Residue	≈ 35.96 million	Dry tonnes	AAL AER 2020	1983 to December 2020

2.0 BATTERY LIMITS

Limitation of these design criteria and Golder's scope of services.

3.0 DESIGN BASIS

Table 6: Key Design Considerations

Objective	Basis	Remarks
Remain geotechnically stable under static conditions and seismic design events during operation and post-closure	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria	No national standard for Ireland. AAL have selected CDA standard taking into account MWEI BREF 2018
Provide adequate water inventory during operation and provide adequate storage and discharge capacity during operation and closure for inflow design flood	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria	No national standard for Ireland. AAL have selected CDA standard
Minimize impacts to environmental, biological, cultural, and social resources during construction, operation, and post-closure	■ Regulatory Requirements ■ Industry Standards & Guidelines ■ (ANCOLD, CDA and MWEI BREF 2018) ■ AAL Environmental Management Plan (EMP)	-
Maintain acceptable operating philosophy and management framework to prevent or restrict containment release	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ AAL Operation, Safety and Maintenance (OSM) manual ■ AAL Emergency Action Plan (EAP) ■ Design Criteria	-

Objective	**Basis**	**Remarks**
Constructed and operated in a safe and secure manner that minimize or eliminates impacts to persons' health, safety, and security	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ AAL OSM manual ■ AAL EAP	-
Designed with consideration of construction limitations for region	■ Design Criteria	Irish Standards and Eurocode
Closure Plan or Strategy developed during design and revised operation, prior to closure	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria	-

Table 7: General Considerations

Objective	Basis	Remarks
Provide tailings storage for the life of BRDA based on the nominal annual Bauxite RESIDUE production	Life of Facility (LoF) presented in Design Criteria	Outcome of conceptual design process
Optimize BRDA footprint at Stage 10 and side-slope gradient from Stage 10 to Stage 16	■ LoF requirements ■ Design Criteria	Stability Analyses at conceptual design stage
Provide suitable operational controls for dam construction	■ Regulatory Requirements: ▪ Design report, technical specifications and design and construction drawings ▪ AAL OSM manual ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria	AAL Standard Work Method for stage raise construction
Water management to be protective of the receiving environment	■ Regulatory Requirements ■ Design Criteria: ▪ BRDA water balance and hydrological assessment ▪ BRDA seepage model ▪ BRDA water quality assessment (seepage and surface water) ▪ BRDA freeboard assessment	AAL / University of Limerick (UL) ongoing wetland trials. Golder assessments and water quality prediction modelling.
Provide operability in all seasonal weather conditions	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria ■ AAL OSM Manual	AAL Standard Work Methods (SWMs)
Adequately define and quantify construction material resources required for construction and closure	■ Regulatory Requirements ■ Design Criteria ■ AAL OSM manual	Tally with Borrow Pit resources

Objective	Basis	Remarks
Adequately characterize residue, foundation soils and construction materials	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria ■ Closure Plan	Essential for geotechnical stability
Prevent disposal of waste materials other than residue into BRDA unless specifically designed for and permitted	■ Regulatory Requirements ■ Design Criteria ■ AAL OSM manual and AAL EMP	Essential for geotechnical stability
Ensure adequate farming of the bauxite residue to reduce pH and increase density	■ Regulatory Requirements ■ Design Criteria ■ AAL OSM manual ■ AAL SWMs	Essential for geotechnical stability
Minimize water inventory volumes in BRDA during operation of the facility	■ Design Criteria ▪ BRDA water balance and hydrological assessment ▪ Operational water inventory requirements ■ AAL OSM Manual	Minimize water storage / ponding on BRDA surface
Surface water will be routed to the greatest extent practicable to alleviate limit ponding against downstream slope of the outer embankment of the perimeter channel	■ Regulatory Requirements ■ Design Criteria ▪ BRDA water balance and hydrological assessment ■ Closure Plan ■ AAL OSM Manual	Essential during storm event to alleviate bauxite residue / contaminated water leaving facility
Consideration of Impacts to Facilities as a result of climate change	■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria ▪ BRDA Hydrological Assessment ■ Closure Plan ■ Dam Break Assessment	Coastal setting hazard. IDF event does not require adjustment for climate change.

Objective	Basis	Remarks
Minimize or mitigate seepage from BRDA during operation and post-closure.	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria ▪ BRDA seepage model ▪ BRDA water quality assessment (seepage and surface water) ■ Closure Plan	-
Construction, operation, and closure of the BRDA will avoid and/or mitigate impacts to protected habitats and cultural heritage (if applicable) areas.	Identified in consultation with [RELEVENT STAKEHOLDERS] and with approval from the [REGULATORS] ■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria ■ Closure Plan ■ In agreement with stakeholders	-
Design for closure methodology to provide a functional and sustainable landform consistent with surroundings	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Closure Plan	-
Identify borrow source materials for restoration, locations, and footprints	■ Regulation Requirements ■ Design Criteria ■ Closure Plan	-

Objective	Basis	Remarks
Minimize or mitigate dust emissions from BRDA surface during operation and post-closure	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria ■ Closure Plan ■ AAL OSM Manual and AAL EMP	-
Minimize or mitigate degradation of waterways and groundwater sources during operation and post-closure	■ Regulatory Requirements ■ Industry Standards & Guidelines (ANCOLD, CDA and MWEI BREF 2018) ■ Design Criteria ■ Closure Plan ■ AAL EMP	-
Minimize or mitigate impacts to local flora including abundance, diversity, distribution, and productivity	■ Regulatory guidelines ■ Industry Standards & Guidelines ■ (ANCOLD, CDA and MWEI BREF 2018) ■ AAL EMP	-

4.0 REGULATIONS, STANDARDS, AND GUIDELINES

This section includes only the regulatory, owner's and industry accepted standards and/or guidelines that will be used for the design of the project. This includes the recognized standards and guidelines used as references. Only list those standards or guidelines that will actually be used (to avoid conflict).

For example, the permit agencies may require designing to a given seismic or storm event, but the Owner may require design to a more stringent, international standard or guideline.

For definition, a standard is a requirement, and a guideline is considered optional.

- **Permitting and Jurisdictional Agencies**
  - **Planning:** Limerick County Council (Register Reference Number 05/1836) An Bord Pleanala (Reference Number 13.217976) for recent Phase 2 BRDA development and raising BRDA from Stage 7 to Stage 10.
  - **Operation and Management:** Environmental Protection Agency (EPA or 'the Agency') via Industrial Emissions Licence (P0035-07, issued in September 2021).
  - **Permitting Stages:**
    - Request to An Bord Pleanala (ABP) for Pre-application Consultation Meeting for 'Strategic Infrastructure Development'.
    - Pre-application Consultation Meeting with ABP to discuss proposed development.
    - Consultations with key stakeholders and with public.
    - Submittal of Planning Application, Environmental Impact Assessment Report (EIAR), Natura Impact Statement (NIS) and supporting Assessments (including this Engineering Design Report) for the proposed development to ABP.
    - Submittal of Industrial Emissions Licence Review to EPA.

Table 8: Summary References and Sources

Source	Reference Code	Regulatory	Owner's Standard and Guidelines	Industry Standard or Criteria	Other
Limerick County Council	1	X			
An Bord Pleanala	2	X			
Environmental Protection Agency	3	X			
Canadian Dam Association	4		X	X	
ANCOLD	5		X	X	
MWEI BREF 2018	6		X	X	
GISTM	7			X	
AAL OSM, EMP, EAP and SWMs	8		X		
AAL AER	9		X		
AAL IEL	10				
EN Codes / Standards	11	X		X	X
Ireland Code / Standards	12	X		X	X
Regulatory Requirements	13	X		X	X
Golder	14			X	X
Ercon	15			X	X
Geocon	16			X	X

5.0 PROJECT SPECIFIC DESIGN CRITERIA

This section includes the project specific criteria upon which the design will be based (examples: environmental management approach, design storm events, design seismic events, design factors of safety, etc.). The selected criteria or events are linked to the selected references in Section 3.0 but are more specific.

In accordance with MWEI BREF 2018 (4.2.1.3.4.3) and in the absence of a National or EN Standard, AAL have selected to undertake the classification of the BRDA and ancillary infrastructure in accordance Canadian Dam Association (CDA) Guidelines (CDA 2014) and to adopt the target level criteria for design parameters (inflow design flood, seismic event and factors of safety for static, pseudo-static and post-seismic stability which are dependent on the consequence of failure.

The CDA guidelines promote a risk-informed approach to dam safety analysis and assessment as it includes deterministic standards-based analysis among many considerations. Tailings dams are classified according to the consequence in the event of failure and takes into account the incremental loss of life, environmental impact and economic impact that a failure of the dam may inflict on downstream or upstream areas, or at the dam location itself. Incremental losses are those over and above losses that might have occurred in the same natural event or condition had the facility not failed. The classification assigned to a dam is the highest rank determined among the loss categories and range from Low, Significant, High, Very High and Extreme consequence.

The project specific design criteria are based on the dam hazard potential classification (HPC). The classification assigned to a dam is the highest rank determined among the 'incremental losses' categories. Golder has classified the BRDA, as a facility with a '**High**' HPC, while the SWP, the LWP and the PIC have been classified as dams having a "**Low**" HPC.

The hazard potential classification is based on the following:

- **Population at Risk / Loss of Life:** The population at risk is deemed to be 10 or fewer and is Temporary and loss of life is unspecified. This hazard puts the BRDA into the '**Significant**' HPC.

  The population at risk is confined to BRDA staff, subcontracted staff or third parties during its operation (40 hrs per week), subcontracted staff or third parties farming the land to the north of the BRDA (short period during summer months) and occasional attendance by inspection, monitoring or maintenance staff (subcontracted or third party) during its operation and following closure. There is no resident population downstream of the BRDA within the break-out zone;

- **Environmental and Cultural Values:** Even though a failure is likely to adversely affect wildlife habitat, the low mobility of the frictional granular flow and the consequence mitigating measures incorporated into the design of the facility will, in all likelihood, mean that restoration of the area is highly possible. This hazard puts the BRDA into the '**High**' HPC.

- **Infrastructure and Economics:** A failure of the BRDA will, in all likelihood, result in minimal economic losses to third parties i.e., beyond the footprint of lands owned by AAL and no impact to infrastructure or services. However, boundaries for special areas of conservation (SAC) and special protection areas (SPA) are present to the north and west of the BRDA and a failure of the BRDA has the potential to potential to impact on these areas. Significant costs may be associated with clean-up and restoration of affected area. This risk would put the BRDA in the **'Significant'** to '**High**' HPC.

The consequence category for the BRDA is classified as a **High** HPC to account for the clean-up and restoration costs of the adjacent SAC and SPA designated areas and for the potential for significant loss of important wildlife / fish habitat.

Table 9: Environmental Management

Description	Criteria / Value / Type	Units	Reference / Source	Comment	Remarks
Seepage and Groundwater					
Seepage Rate through base of facility	Estimated Seepage Volumes:	m^3/day	9, 14	Negligible seepage through the base of the facility, either in the unlined or lined phases, due to the low hydraulic conductivity of the bauxite residue once a sufficient depth (≈10m) is reached.	Appendix H - Seepage and Water Quality Assessment
Seepage Rate from PIC and/or SWP	Estimated Seepage Volumes:	m^3/day	9	Damage or defects in lining system for PIC and/or SWP have potential for seepage	Monitoring systems in place (Observation Wells and Toe Drain)
Seepage effluent quality and constituents	Water elevation pH Conductivity Caustic Soda Metals	pH units etc. mg/l uS/cm	8, 9, 10	Quarterly groundwater monitoring via Observation Wells installed around perimeter of Phase 1 and 2 BRDA in accordance with Schedule C.6 of IEL.	Reported in AER
Seepage collection methodology	Downstream of BRDA and PIC prior to entry to external environment	m^3/day	8, 9	BRDA seepage collection systems • **PIC** – tailings bleed water and surface water runoff, gravity flow and pumped to SWP at end of PIC. • **Toe Drain** – located downstream of PIC and SWP and pumped back to PIC / SWP at designated sumps • **Abstraction Wells and Chambers** – borehole well and manhole chambers installations targeted to intercept known seepage paths and/or sub-liner drainage systems and pumped back to PIC / SWP. • **Sub-Liner Drainage Systems** – rock fill backfilled drains reporting to Chambers for pumping to PIC	Levels in boreholes monitored by float switches and/or pumping rates on AAL system

Description	Criteria / Value / Type	Units	Reference / Source	Comment	Remarks
Lining System					
Basal Lining System	Natural Soils and Geosynthetic Single Layer and Composite Lining Systems	1 x 10^{-9} m/s	14, 15, 16	A lined basin is required for the BRDA as the BRDA is raised by the upstream method which involves constructing a rock fill berm founded on previously deposited and farmed bauxite residue. Rock fill berms permit bauxite residue bleed water and surface water runoff to trickle down to PIC.	-
BRDA Lining Systems	Natural Soils and Geosynthetic Single Layer and Composite Lining Systems	1 x 10^{-9} m/s	14, 15, 16	**Phase 1 BRDA:** Estuarine soils **Phase 1 BRDA Extension and Phase 2 BRDA:** Composite lining system comprising HDPE geomembrane overlying GCL or min. 1m depth compacted clay liner (CCL) **SWP:** Composite lining system comprising HDPE geomembrane over GCL over 0.3m CCL **LWP and PICs:** HDPE geomembrane lined over 1.0m x 10^{-9} m/s CCL **Salt Cake Disposal Cell:** Composite lining system comprising HDPE geomembrane over GCL over 19m of bauxite residue over BRDA basal lining system	SCDC (2012) and 2 x raises to date
Water Collection and Pond Lining System	Natural Soils and Geosynthetic Single Layer and Composite Lining Systems	1 x 10^{-9} m/s	14, 15, 16	**SWP:** Composite lining system comprising HDPE geomembrane over GCL over 0.3m CCL **or** 1.0m x 10-9 m/s CCL **LWP and PICs:** HDPE geomembrane lined over GCL over 0.3m CCL **or** 1.0m x 10^{-9} m/s CCL	SWP and LWP upgraded in 2007 and 2012

Description	Criteria / Value / Type	Units	Reference / Source	Comment	Remarks
Allowable Pore Pressure on Liner	Head of water	m	14	The facility is designed to operate with a high phreatic surface. Design target is 2m to 3m below BRDA sloped surface	-
Leak Detection System	Water elevation pH Conductivity Caustic Soda Metals	mg/l uS/cm	9, 10	Quarterly groundwater monitoring via Observation Wells installed around perimeter of Phase 1 and 2 BRDA in accordance with Schedule C.6 of IEL.	Seepages captured and returned to PIC / SWP
Hydrology					
Catchment Areas	BRDA 184 Plant Site 46.57	ha	14	104.0 ha for Phase 1 BRDA 80.0 ha for Phase 2 BRDA 21.90 ha for East Plant 23.13 ha for West Plant 1.54 ha for North Plant	Appendix I - BRDA Raise Water Balance Report
Modelling	GoldSim Monte Carlo Simulation Software	-	14	In-house software hydraulic and flood-routing software using starting inventory conditions for IDF event	As above
Design Storm Event managed by BRDA Water Infrastructure	BRDA IDF, 24 hr = 141.0 Plant IDF, 24-hr = 91.1	mm	14	BRDA IDF is 1/3 between the 1,000-year event and PMF (CDA criteria for ‘**High**’ HPC) Plant IDF is 100-year + 20% climate change factor IDF is fully contained within the BRDA Water infrastructure. Pump operation and capacities have been assessed to have adequate availability to manage the IDF.	At Closure, the BRDA is expected to be reduced to a facility with a CDA ‘**Significant**’ HPC
BRDA Water Storage Capacity	Average hydrological conditions	m^3	8	Phase 1 PIC = 103,000 (22,500) Phase 2 PIC = 67,000 (20,900) SWP = 182,000 (58,000) **Total = 352,000 (101,400)**	0.5m freeboard volume in brackets
BRDA Water Inventory	Average hydrological conditions	m^3	8	Winter Months = 110,000 (Reduced for storm management) Summer Months = 180,000 (Increased for dusting prevention)	Transition month volumes = 150,000

Description	Criteria / Value / Type	Units	Reference / Source	Comment	Remarks
Effluent Clarification System (ECS) and LWP	Average hydrological conditions	m^3 / hr m^3	8	Maximum inflow rate from ECS = Maximum permitted discharge rate = 1,250 LWP Volume = 45,700 (8,630)	0.5m freeboard volume in brackets
Freeboard for stage raises (Surface Water and Bauxite Residue Deposition)	Average hydrological and deposition conditions	m	8	• No water stored on BRDA. Side-slope drainage systems installed to augment free-draining rock fill. • 0.2m freeboard for stage raises during bauxite residue deposition attained by constructing small rock fill bund at upstream crest.	Filled to level with stage raise crest to provide foundation for next stage raise
Closure cover system and side-slope capping containment	Dome, Side-Slopes and Water Conveying Systems		8, 10, 14	1m capping layer of amended bauxite residue for 4% dome. Side-slope capping design and water conveying systems design in progress.	As-built side-slope capping containment systems

International guidelines for the required FoS for the AAL BRDA are as follows:

- Canadian Dam Association (CDA) Application of Dam Safety Guidelines for Mining Dams (CDA 2013, 2014);
- International Commission on Large Dams (ICOLD), Improving Tailings Dam Safety (ICOLD Bulletin 139, 2011)
- Australian National Conference on Large Dams (ANCOLD) Guidelines on Tailings Dams (ANCOLD 2012, 2019).

The Eurocode 7 design rules have not been applied as the code states that it applies to the embankments of 'small' dams. Small dams are not defined in Eurocode 7; however, it can be considered to be defined as less than 15 metres in height as ICOLD defines a 'large' dam being > 15m above its foundations.

The BRDA is considered a large dam, currently at a vertical design height of 24m and proposed to be increased to 36m, and as such, the above-mentioned guidelines, along with ICOLD bulletins, are considered more applicable.

Table 10: AAL BRDA Safety – Static, Seismic and Blast

Condition	CDA 2013, 2014	ANCOLD 2012, 2019	REMARKS
Short-Term Undrained, Static	Greater than 1.3 During, at, or end of Construction, depending on Risk Assessment	1.5 If potential loss of containment, 1.3 if no potential loss, Consolidated Undrained Strength	-
Long-term Drained, Static	1.5 Steady State, Phreatic Level	1.5 Effective Strength	-
Rapid Drawdown Full or Partial	1.2 to 1.3	Not required	N/A to BRDA
Seismic or Blast Loading, Pseudo-Static	1.0	Not required	20% reduction of material strength parameters
Post-Seismic or Post-Blast	1.2	1.0 to 1.2	20% reduction of material strength parameters
Seismic Liquefaction			
Screening liquefaction assessment of tailings and underlying soils. ■ **Bray and Sancio 2006:** Based on Moisture Content and Atterberg Limits. If determined to be susceptible, undertake seismic liquefaction assessment for design seismic event:			

MCE = M (earthquake moment magnitude) = 5.0 within 1km epicentre of the Site MDE = 1 in 2,475-year event (0.05g) for BRDA and 1 in 100-year event (0.009g) for SWP, LWP and PICs ■ **ICOLD Bulletin 139**: National Centre for Earthquake Engineering Research and National Science Foundation (NCEER/NSF) Method. Figure 1 – Flowchart. with addition of **Jefferies and Been 2016**: State Parameter Approach for Cyclic Resistance Ratio (CRR)
Blast Assessment
Interpret Peak Particle Velocity (PPV) expected to be produced by the blast event. Pseudo-Static Stability Assessment for maximum limiting PPV, FoS > 1.0 Post-Blast Stability Assessment, to account for excess pore pressure, FoS > 1.2

Table 11: Environmental Sustainability

Description	Criteria / Value	Units	Reference/ Source	Comment	Remarks
Embankment Fill Acid Generation Potential	Non-acid generating, chemically stable		INAP, MEND and GARD 8, 14, 15, 16	Stage raise constructed of rock fill derived from limestone	-
Embankment Fill potential for metal leaching	Non-metal leaching, chemically stable		INAP, MEND and GARD 8, 14, 15, 16	Stage raise constructed of rock fill derived from limestone	-
Seepage (Quantity/Quality)	Requirements of IEL	pH mg/l uS/cm	9, 10, 14	Appendix H - Seepage and Water Quality Assessment	-
Erosion Control	Slope and spillway design to minimize erosion.		9, 10, 14	Appendix J – BRDA Dome Closure Appendix K – BRDA Side-Slope Closure	-

6.0 SITE SPECIFIC DESIGN PARAMETERS

This section includes a summary of the input parameters that will be used for design. The section presents the actual values that will be uniformly applied / utilized throughout design, linked to the criteria summarized in Section 5.0.

Table 12: Site Location

Description	Criteria / Value	Units	Reference / Source	Comment	Revision
Site Coordinate System	Irish National Grid Transverse Mercator 127300E, 152200N	-	8, 9, 14	Spheroid: Airy Modified Datum: 1965	-
Site Location	Latitude: 52°36’53.19’’ Longitude: -9°4’25.53’’	-	8, 9	-	-
Site Elevation Range	1 to 32	mOD	8, 9, 14	Raise design to max 44mOD for Dome	-
Site Footprint	168.5	ha	8, 9	Area occupied by bauxite residue, excluding PICs: 94.5 ha for Phase 1 BRDA 74.0 ha for Phase 2 BRDA	-
Method of Deposition	Pumping and Trucking	-	8, 9	Bauxite residue is pumped All other waste streams are trucked.	-

Table 13: Weather Stations

Station Name	Climate Station ID	Latitude, Longitude, Elevation	Distance to BRDA (km)	Years of Data Available	Notes
Shannon Airport, Co. Clare	13	52.6807753, -8.9205048, 6 mOD	12.614 km	Daily from 1945 Monthly from 1850	ING 137792.00E, 159203.87N
Notes: 1. AAL have two on-site weather stations (BRDA and Jetty)					

Table 14: Shannon Airport Annual Air Temperature, Precipitation and Maximum Gust (1981-2010)

Month	Mean Air Temperature (ºC)	Mean Precipitation (mm)	Maximum Gust (m/s)	Remarks
January	6.0	102.3	38.6	30-year averages data set 1991-2020 dataset due out in Aug 2021
February	6.2	76.2	41.1	
March	7.8	78.7	33.4	
April	9.5	59.2	31.9	
May	12.1	64.8	30.3	
June	14.6	69.8	26.2	
July	16.4	65.9	26.7	
August	16.2	82.0	28.3	
September	14.2	75.6	31.9	
October	11.2	104.9	36.5	
November	8.3	94.1	33.9	
December	6.3	104.0	42.7	
Average Annual	**10.7**	**977.6**		
Reference	Met Eireann: https://www.met.ie/climate/30-year-averages			

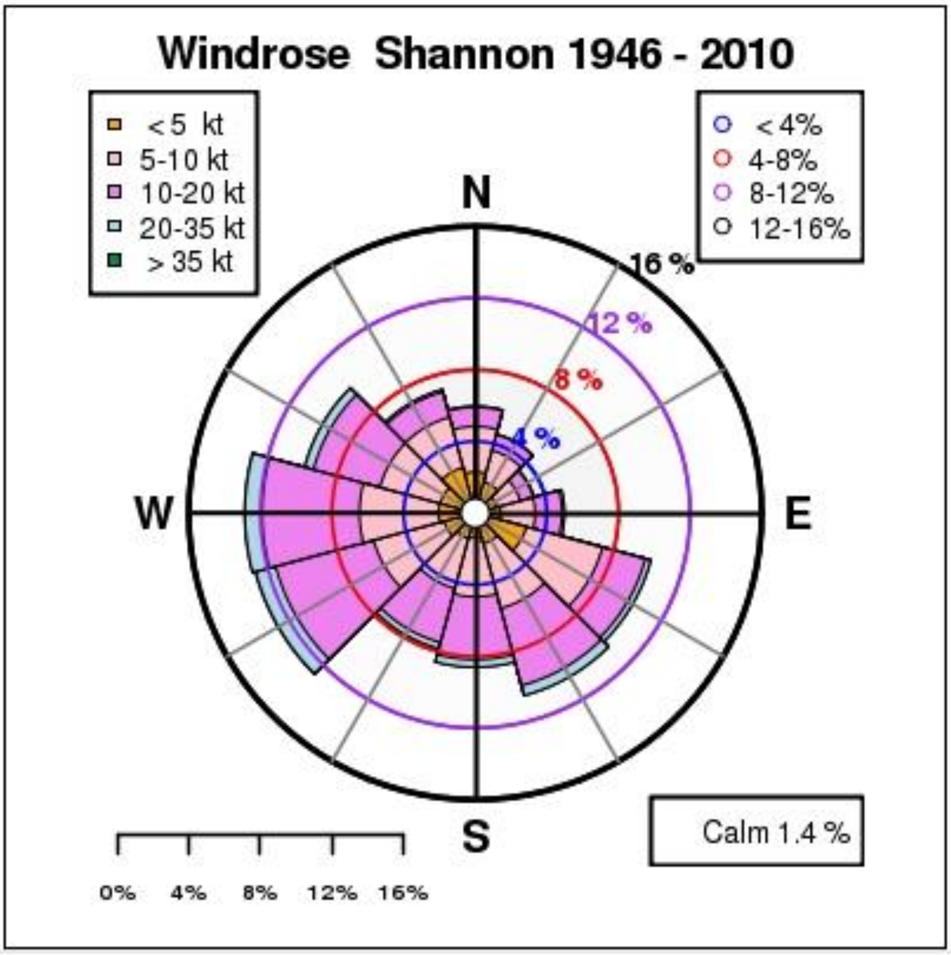


Figure 2: Shannon Airport - Wind Rose – 1946 to 2010

The Met Eireann website provides a wind rose plot for the nearest weather station to the Aughinish Site (Shannon Airport) for 1946 to 2010.

The wind rose shows the frequency of winds blowing from particular directions and indicates that the major wind directions are from approximately 110º to approximately 285º (north is 0º).

Table 15: Design Storm Precipitation Events

Return Period	24-Hour Rainfall Depth (mm)	Low (90% Confidence)	High (90% Confidence)	Reference/ Source	Revision
2-year	40.1	-	-	Met Eireann https://www.met.ie/climate/services The geographical descriptors of the Aughinish site are: ■ Irish Grid: Easting 127433 Northing 151917 ■ Altitude: 15 m Statistical estimates of the Probable Maximum Precipitation (PMP) for the site were determined using the procedures in WMO 2009 'Manual on Estimation of Probable Maximum Precipitation (PMP)', WMO-No. 1045.	-
5-year	48.6	-	-		-
10-year	54.4	-	-		-
30-year	64.0	-	-		-
50-year	68.9	-	-		-
100-year	75.9	-	-		-
200-year	83.7	-	-		-
500-year	95.1	-	-		-
1,000-year	107.5	-	-		-
2,500-year	123.7	-	-		-
5,000-year	137.5	-	-		-
10,000-year	152.9	-	-		-
PMP (12-hour)	181.14	-	-		-
PMP (24-hour)	215.32	-	-		-

Table 16: Dam Safety Input Values

Description	Criteria/ Value	Units	Reference/ Source	Comment	Revision
Geotechnical Stability and Risk Management					
Seismic Hazard Basis	Zonation Model (Zone A13) MCE = 5.0 M 1 in 2,475-year event (0.05g) for BRDA (**High**) 1 in 100-year event (0.009g) for SWP, LWP and PICs (**Low)**	M, g	Seismic Hazard: UK Continental Shelf (HSE 2002) CDA 2014	Seismic hazard contour map	-
Approach	The seismic hazard assessment is based on local and regional geotectonic information and statistical analysis of historical earthquakes experienced in the region. In areas of low seismicity and areas which lack direct seismic correlation with well-defined or active faults, a probabilistic approach is generally used, as is the case for Aughinish.	M, g	ICOLD Bulletin 139	The FoS against liquefaction is the ratio of CRR over CSR. FoS > 1.0 for the MDE is required	-
Seismic Response spectra	In view of the low seismicity of the region of the Aughinish Site and the classification of the facility, a site-specific seismic hazard assessment is not deemed to be required and the data provided by the HSE 2002 is appropriate for the engineering analyses.		N/A	N/A	-
Design Earthquake (ultimate height)	MDE = 5.0 within 1km epicentre of the Site (36 mOD at stage raise crest and 44mOD at dome for BRDA raise to Stage 16)	M	Seismic Hazard: UK Continental Shelf (HSE 2002)	The design earthquake is selected to prevent catastrophic embankment failure but may permit “acceptable deformations.” to 10,000-year return events.	-

Description	Criteria/ Value	Units	Reference/ Source	Comment	Revision
Geotechnical Stability and Risk Management					
Design Earthquake – Operation	MDE = 5.0 within 1km epicentre of the Site 1 in 2,475-year event (0.05g) for BRDA (**High**) 1 in 100-year event (0.009g) for SWP, LWP and PICs (**Low)**	M,g	Seismic Hazard: UK Continental Shelf (HSE 2002) CDA 2014	The operation-based earthquake is the event where damage / deformations are minimal so that the mine can resume operation in less than one week.	-
Design Earthquake – Closure / Post Closure	MDE = 5.0 within 1km epicentre of the Site 1 in 2,475-year event (0.05g) for BRDA (**Significant**) 1 in 100-year event (0.009g) for SWP, LWP and PICs (**Low)**		Seismic Hazard: UK Continental Shelf (HSE 2002) CDA 2014	At Closure / Post Closure, the BRDA is expected to be reduced to a facility with a '**Significan**t' HPC	-
PGA Reduction Factor	N/A		Hynes-Griffin, M.E., and Franklin, A. G. (1984)	N/A	-

Table 17: Storage and Deposition

Description	Criteria/ Value	Units	Reference / Source	Comment	Remarks
Current Life of BRDA Storage Capacity	53.1 million **Total** 14.9 million **Remaining (9.8 yrs)**	tonnes	8, 9, 14	Dry density (1.63 tonnes / m^3) of bauxite residues to Stage 10 & Dome Phase 1 and 2 BRDA survey from April 2021 and estimate of tonnes stored between 1983 and Dec 2020	-
Proposed Life of BRDA Storage Capacity	66.2 million **Total** 28.0 million **Remaining (18.5 yrs)**	tonnes	8, 9, 14	Dry density (1.63 tonnes/m^3) of bauxite residues to Stage 16 & Dome Phase 1 and 2 BRDA survey from April 2021 and estimate of tonnes stored between 1983 and Dec 2020	-
Residue Deposition Rate	≈ 4,300	tonnes / day	8, 9, 14	Based on ≈ 50% grade by weight.	-
Deposition Start Date	1983 2011 2023 / 2024	year	8, 9, 14	Phase 1 BRDA Phase 2 BRDA BRDA Raise above Stage 10	-
Design Life (Proposed)	Additional 9	year	8, 9, 14	Extent Life of BRDA from 2031 to 2040	-
Deposition Method	Hydraulic deposition discharge of bauxite residue paste from Mud Points located centrally within the BRDA Trucking for all other residues (process sand, salt cake and scales). Designated tipping areas for salt cake (salt cake cell) and scales in BRDA interior. Process sand utilized for constructing internal haul and access roads.		8, 9	Positive displacement high pressure pumping of bauxite residue paste at approx. 75% moisture content (≈ 58% solids) via distribution network to controlled sectors or cells within the BRDA Mud Points are located centrally and can be rotated. Bauxite residue paste migrates by gravity to perimeter stage raises and/or sector/cell bunds at between 2% and 4% stacking slope Thin layer deposition to dewater paste and facilitate mud farming. Nominal depth of 540mm. Mud farming reduces pH < 11.5, reduces moisture content, increases density and increase strength (see Table 25).	

7.0 OTHER RELATED DESIGN ASSUMPTIONS

This section is a catch all for other criteria or input assumptions that don't readily fall into the "input value" category. Examples include engineering judgment values, maximum slopes, lining system types, construction methods, material types, etc.

Table 18: BRDA and Salt Cake Cell Containment Systems

Description	**Criteria / Value**	**Reference / Source**	**Comment**	**Remarks**
Liner Type	Geosynthetic lining system only for Salt Cake Cell raise and for upgrade works for Perimeter Interceptor Channel	14	Refer to Table 9: Lining Systems, Leak Detection and Seepage Collection. No lining system required for proposed BRDA Raise as constructed upstream above existing BRDA. Composite lining system required for upstream slopes of SCDC raise.	-
BRDA Basin and SCDC Basal Lining System	**Natural:** Estuarine Soils **Geosynthetic** **Primary:** 2.0 mm HDPE Geomembrane **Secondary:** GCL over 0.3m CCL or 1.0m x 10^{-9} m/s CCL	14, 15, 16	Refer to Table 9: Lining Systems, Leak Detection and Seepage Collection for details of lining systems in different sectors of the BRDA and for water storage facilities Existing SCDC basal lining system comprises HDPE Geomembrane over GCL over 18m of bauxite residue over BRDA basin lining system.	No basal lining system required for BRDA or SCDC for this project
Upstream SCDC Raise Slope Lining System	**Geosynthetic** **Primary:** 2.0 mm HDPE Geomembrane **Secondary:** GCL	14, 15, 16	Refer to Table 9: Lining Systems, Leak Detection and Seepage Collection for details of lining systems in different sectors of the BRDA and for water storage facilities	Original SCDC in 2021 and 2 x raises to date
Perimeter Interceptor Channel Containment Lining System	**Geosynthetic** **Primary:** 2.0 mm HDPE Geomembrane **Secondary:** GCL over 0.3m CCL or 1.0m x 10^{-9} m/s CCL	14, 15, 16	Refer to Table 9: Lining Systems, Leak Detection and Seepage Collection for details of lining systems in different sectors of the BRDA and for water storage facilities	Upgrade works required for this project

Description	Criteria / Value	Reference / Source	Comment	Remarks
SWP and LWP Containment Lining System	**Geosynthetic** **Primary:** 2.0 mm HDPE Geomembrane **Secondary:** GCL over 0.3m CCL or 1.0m x 10^{-9} m/s CCL	14, 15, 16	Refer to Table 9: Lining Systems, Leak Detection and Seepage Collection for details of lining systems in different sectors of the BRDA and for water storage facilities	SWP upgraded in 2007 and SWP and LWP raised in 2012. No further upgrade required for current project

Table 19: Embankments

Description	Criteria/ Value	Units	Reference / Source	Comment	Remarks
BRDA Raise Construction Method	Upstream in 2m high lifts with a 4m wide crest and side-slopes of 1.5(H):1(V). Next stage raise is offset upstream by a bench width.	m	8, 9, 14	2m high stage raises constructed of 0 - 300mm rock fill derived from limestone on farmed and prepared bauxite residue footprint in accordance with AAL SWM for staged construction (1m lifts). Bench widths are 4m except for at Stage 5 at 14 mOD (≈ 28m) and Stage 10 at 24 mOD (12.5m)	Similar to previous stage raise construction method for unfarmed bauxite residue to ≈ Stage 7 in Phase 1 BRDA
BRDA Raise Downstream Slope	Overall slope of 6.3(H):1(V)	m	8, 9, 14	Consisting of a lower, middle and upper slopes formed at ≈ 6(H):1(V) separated by a ≈ 28 m wide bench at Stage 5 (14 mOD) and a 12.5m wide bench at Stage 10 (24 mOD)	-
BRDA Stage Raise Crest Width	4m	m	8, 9, 14	4m wide to facilitate plant traffic movements on the stage raises and for ease of construction	-

Description	Criteria/ Value	Units	Reference / Source	Comment	Remarks
BRDA Stage Raise Bench Width	4m ≈28m @ Stage 5 12.5m @ Stage 10	m	8, 9, 14	Bench widths are 4m except for at Stage 5 at 14 mOD (≈ 28m) and Stage 10 at 24 mOD (12.5m).	Stage 5 bench width varies from 25m to 34m for various sectors in the BRDA
SCDC Raise Construction Method	Downstream and Centre-Line in 2.25m high lift. **Downstream raises:** 2.0(H):1(V) downstream slope and 2.5(H):1(V) upstream slope **Centre-Line raise:** ≈ vertical slope with 1.5m offset bench on upstream slope.	m	14	Triangular shaped cell 2.25m high downstream raise on north and east embankment constructed of 0 - 300mm rock fill derived from limestone on prepared process sand footprint. 2.25m centre-line raise constructed of two terramesh gabion basket retaining walls anchored together and infilled with 0 - 300mm rock fill derived from limestone	Similar to raise construction methods for previous raises of the SCDC. Bauxite residue level rises accordingly around SCDC, buttressing previous raises.
SCDC Raise Crest Width	8m on north and east dam walls 22m on west dam wall	m	14	West wall is the tipping wall for the SCDC Crash barrier and light plant vehicle access on north and east dam walls	Similar to crest width for previous raises of the SCDC
Embankment Material (BRDA Raise and SCDC Raise)	Rock fill, termed Type B, 0 – 300mm	mm	8, 9, 14	Limestone derived rock fill, termed Type B material, 0 - 300mm grading, sourced from local quarries or from on-site Borrow Pit (blasted and crushed)	Similar to material used for previous stage raise construction
Stability	Instrumentation • Piezometers • Inclinometers • Extensometers		8, 9, 10, 14	Physical Stability Monitoring Plan and Schedule C.7 on IEL	Minimum of Quarterly Monitoring

Golder has adopted representative properties for the materials based on the available information and results of the recent and previous site investigation and laboratory testing programmes (Golder 2018, Golder 2019). The selected properties for the undrained and drained analysis are presented below.

Table 20: Geotechnical Design Parameters for Stability Analyses: Foundation Soils and Embankment Materials

Material Property	N_{kt}	Particle Density	Density / Unit Weight (Mg/m3 / kN/m3)		Strength		Comment
			Bulk	Dry	Effective	Undrained	
Bedrock					N/A Considered Impenetrable for stability analyses		Rathkeale Formation underlies the bulk of the BRDA, - dark grey, argillaceous (muddy) limestone and shaley mudstone Waulsortian Limestone outcrops along eastern flank of BRDA, - pale-grey massive, unbedded limestone
Estuarine Deposit (Sandy Silt Layer)	Calculated from B_q value measured in CPT soundings. Varies between 13 and 17	2.7	1.94 / 19.02	1.63 / 15.98	Ø = 30° c = 0 kPa	s_u/σ'_{v0} = 0.25 to 0.50 (Horizontal shear)	Effective strength from previous testing (Golder 2005). Undrained strength from shear vane and DSS testing (2018/19) Identify layers from CPTu soundings for each location. Min. of 10 kPa
Estuarine Deposit (Silty Clay Layer)	As above	2.7	1.82 / 17.85	1.31 / 12.85	Ø = 30° c = 0 kPa	s_u/σ'_{v0} = 0.20 to 0.30 (Horizontal shear)	As above. Present in thin layers (0.5m to 2m depth)
Process Sand	N/A	N/A	1.94 / 19.02	1.63 / 15.98	Ø = 33° c = 0 kPa	N/A	Laboratory Testing conducted in 2019
Rock fill	N/A	2.7	2.24 / 22	-	Ø = 45° c = 0 kPa	N/A	Typical for rock fill
Glacial Till	N/A	2.7	1.73 / 17	-	Ø = 36° c = 2 kPa	N/A	Typical glacial till in Ireland

Ø = friction angle; c = cohesion; s_u = undrained shear strength; σ'_{v0} = vertical effective confining stress

Table 21: Geotechnical Design Parameters for Stability Analyses: Bauxite Residue

Material Property	N_{kt}	Particle Density	Density / Unit Weight (Mg/m3 / kN/m3)		Strength		Comment
			Bulk	Dry	Effective	Undrained	
Unfarmed Bauxite Residue	14	3.4	2.19 / 21.48	1.58 / 15.49	Ø = 32° c = 0 kPa	s_u/σ'_{v0} = 0.20 to 0.25 (Horizontal Shear) s_u/σ'_{v0} = 0.50 to 0.70 (Compression)	Minimum undrained strength to be determined for each location Min. of 15 kPa.
Wick Drain Unfarmed Bauxite Residue	14	3.4	2.19 / 21.48	1.58 / 15.49	Ø = 32° c = 0 kPa	s_u/σ'_{v0} = 0.25 to 0.30 (Horizontal Shear) s_u/σ'_{v0} = 0.60 (Compression)	Minimum undrained strength to be determined for each location. Min of 15 kPa.
Farmed Bauxite Residue	14	3.4	2.19 / 21.48	1.63 / 15.98	Ø = 32° to 35° c = 0 kPa	s_u/σ'_{v0} = 0.60 (Horizontal shear)	Minimum undrained strength to be determined for each location Min. of 25 kPa.
Sludge Pond	14	-	2.24 / 22.00		Ø = 30° c = 0 kPa	s_u/σ'_{v0} = 0.15	Based on CPT data (2018). Sensitivity analyses conducted due to high variability in strength observed.
Salt Cake	14	-	1.22 / 12.00		N/A	s_u/σ'_{v0} = 0.05	Assumed lower bound strength based on CPT data (2018).

Ø=friction angle; c=cohesion; s_u=undrained shear strength; σ'_{v0}=vertical effective confining stress

General Description: AAL Bauxite Residue

'*The AAL bauxite residue consists of porous agglomerated particles containing some 70% to 80% of amorphous material (oxides, hydrated oxides and oxi-hydroxides such as boehmite, goethite and gibbsite) with fine crystals of quartz, heamatite, rutile and other opaque minerals. A limited number of very coarse heamatite and ilmenite crystals of 10 to 70 microns were observed whilst the remainder were less than 4 microns. Little or no clay minerals are present, and the quartz (silica) content is less than 1%*' (Delft 1988).

Bauxite residue is generally regarded as a thixotropic clayey silt and there is an indication that the bauxite residues may be cemented or aggregated. The bauxite residue particles are sub-rounded, friable with a low crushing strength.

Based on the mineralogy, it can be expected that the bauxite residue would not behave as a clay but would exhibit properties similar to those of a granular silt. However, unlike conventional soils, the amorphous particles could retain water which could have none or a limited effect on the properties of the material (Golder 2014).

Table 22: Other Geotechnical Parameters: Estuarine Soils and Bauxite Residue

Description	Criteria / Value	Units	Reference / Source	Comment	Remarks
Moisture Content	29 to 31% 34 to 44% 36 to 42% 33 to 35%	MC %	14	Estuarine Sandy Silt (30%) Estuarine Silty Clay (38%) Unfarmed Bauxite Residue (39%) Farmed Bauxite Residue (33%)	Consistent historic data Characteristic Values. Amorphous water content included (1.3% avg.)
Atterberg Limits	PI = 8 to 9 PI = 14 to 20 LL = 41 to 47 PL = 29 to 36	MC %	14, 15, 16	Estuarine Sandy Silt Estuarine Silty Clay Bauxite Residue - plots as a silt of intermediate plasticity on Casagrande chart	Consistent historic data
Particle Size Distribution	Bauxite Residue	%	14, 15, 16	Issues with evaluating the grain size of the material because of its caustic nature which interferes with the dispersion of the particles. Majority of the material is clay and silt size. About 90% by weight of the bauxite residue is finer than 40 microns and the D_{50} is between 2 and 5 microns.	Consistent historic data
Void Ratio	0.62 to 0.80 1.05 to 1.08 1.05 to 1.30	-	14, 15, 16	Estuarine Sandy Silt (0.71) Estuarine Silty Clay (1.08) Unfarmed Bauxite Residue (1.17) Farmed Bauxite Residue (1.09)	Characteristic Values

Specific Gravity (Particle Density)	2.71 2.67 3.2 to 3.7 3.2 to 3.7	-	14, 15, 16	Estuarine Sandy Silt (2.71) Estuarine Silty Clay (2.67) Unfarmed Bauxite Residue (3.4) Farmed Bauxite Residue (3.4)	Characteristic Values
Compressibility	Low – Medium Medium – High Medium -High Very Low - Low		14, 15, 16	Estuarine Sandy Silt Estuarine Silty Clay Unfarmed Bauxite Residue Farmed Bauxite Residue	Range of values and discussion in Golder 2019
Consolidation Duration	Medium Medium – High Medium – High Very Low - Low		14, 15, 16	Estuarine Sandy Silt Estuarine Silty Clay Unfarmed Bauxite Residue Farmed Bauxite Residue	Range of values and discussion in Golder 2019
Hydraulic Conductivity	1E-7 to 2E-10 5.0 E-9 (avg.) 1.9 E-8 (avg.)	m/s	14, 15, 16	Estuarine Soils (Sandy Silt - Silty Clay) Unfarmed Bauxite Residue Farmed Bauxite Residue	2005 SI Historic Data 2015 cores
NorSand and Critical State Parameters	$\Gamma = 1.35$ $\lambda_{10} = 0.129$ $M_{tc} = 1.4$ $N_{tc} = 0.25$ $\chi_{tc} = 4$ $G_{max\ @\ p\_atm}$ = 24.6 MPa $G_{exp} = 0.6$ $\nu = 0.15$ $H_0 = 75$ $H_{\psi} = 500$	-	14	The AAL bauxite residue is classified as a silty material of low plasticity. The typical range of Γ is 0.9 to 1.8 and the typical range for λ_{10} is from 0.10 to 0.25 for sandy silts to silts.	Historic data set aligned with 2018 and 2019 data. Stress – Stain Constitutive Modelling in 2021
State Parameter	< 0.05 - 0.05 to 0.05	-	14	Farmed bauxite residue, indicating strain hardening. Unfarmed bauxite residue, indicating potential strain softening.	CPT interpretation 2014, 2018 and 2019
Interface Friction	34 22	effective friction angle, degrees	14	Bauxite residue to textured geomembrane Bauxite residue to smooth geomembrane	Lab testing 2018
BRDA Slope	2 to 4	%	8, 9, 14	Central deposition sloping to perimeter stage raises and/or containment bunds	Aerial survey data

BRDA Slope Below Pool	N/A	%	8, 9, 14	BRDA surface does not store or detain water. AAL active management of ponding on BRDA surface.	Internal sumps and pumping systems

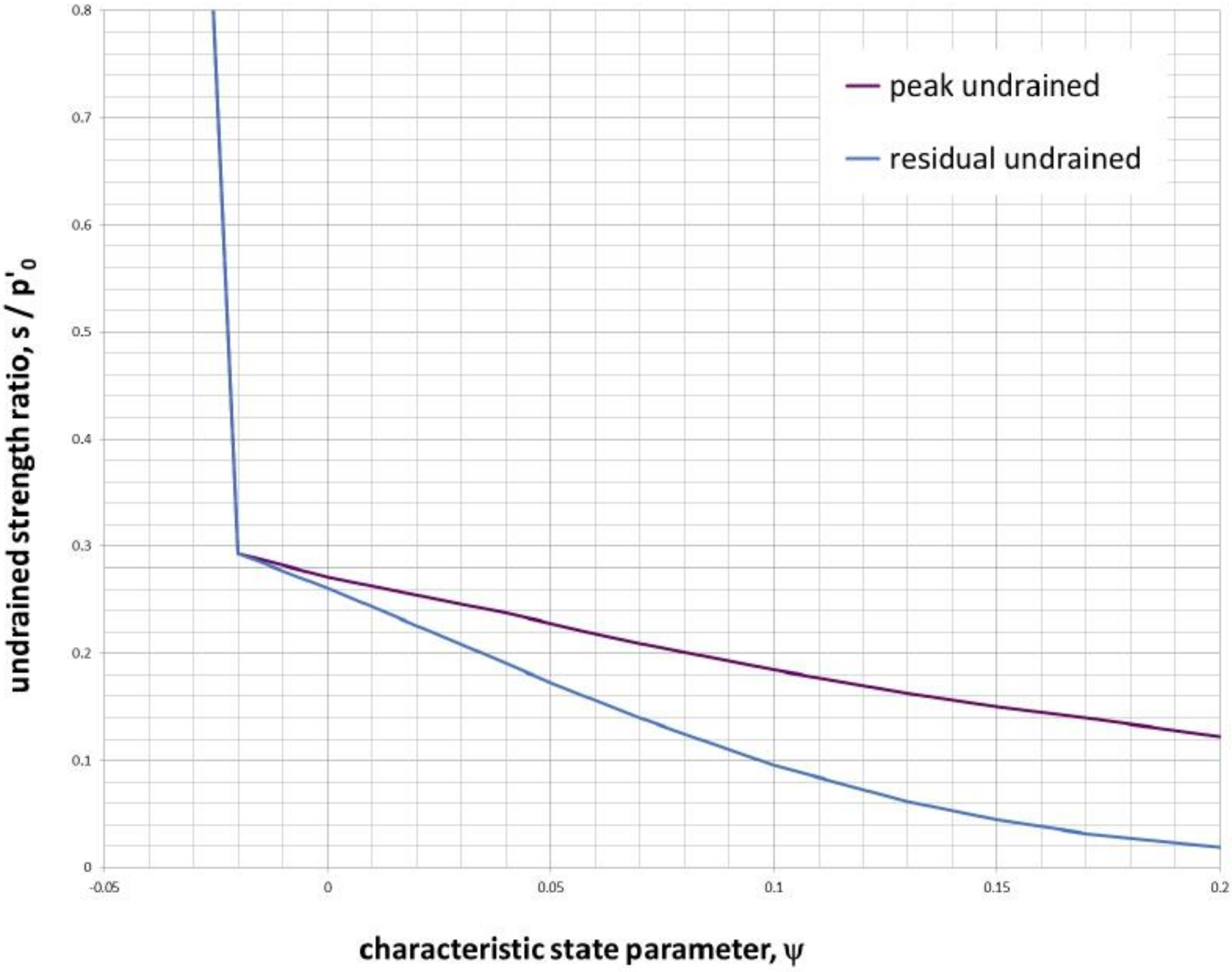


Figure 3: Computed effect of initial state (state parameter) on undrained strength ratio – Golder 2021

Table 23: Access Roads

Description	Criteria / Value	Units	Reference / Source	Comment	Remarks
Minimum Haul Road Width	6 Rock fill construction with asphalt surfacing 6 to 8 or 10 to 12 Process sand and/or rock fill construction	m m m	8, 12	20 tonne rigid tipper truck Plant access roads and perimeter interceptor channel / pond crest roads (with crash barrier) 40 tonne articulated dump truck Routes to designated tipping areas and salt cake cell (with safety berms and/or crash barriers)	One-way with pull-in bays One-way or Two-way
Maximum Haul Road Grade	10 to 16	%	8, 12	20 tonne rigid tipper truck (all routes) 40 tonne articulated dump truck (all routes)	One-way and Two-way
Internal Haul Road Construction	6 to 8 or 10 to 12	m m	8, 14	Process sand construction at max. 1.8(H):1(V) side-slopes and max. 2.5m above deposited bauxite residue for FoS > 1.3 Rock fill construction at 1.5(H):1(V) and max. 2.5m above deposited bauxite residue for FoS > 1.3	One-way and Two-way
Stage Raise Construction	4	m	8, 14	Rock fill construction to ≈ 2m high over prepared bauxite residue footprint in accordance with AAL SWM for staged construction i.e., 1m high lifts and 3 weeks duration between lifts for FoS > 1.5	One-way
Minimum Light Vehicle Road Width	4 Rock fill construction 6 Rock fill construction with asphalt surfacing	m m	8, 12	Stage raises (no berm or crash barrier) Plant access roads, perimeter interceptor channel / pond crest roads (with crash barrier/)	One-way One-way with pull-in bays

Maximum Light Vehicle Road Grade	10	%	8, 12	All routes	One-way
BRDA Security Measures	-	-	8	Security Hut and Barrier at Plant Entrance Security Barrier at BRDA Entrance	BRDA access via Plant

Table 24: Environmental Compliance Criteria (To be read in conjunction with IEL P0036-07)

Description	**Criteria / Value**	**Units**	**Point(s) of Measurement**	**Reference / Source**	**Comment**	**Revision**
Groundwater Discharge Control and Quality	Various Compliance Parameters	-	Groundwater monitoring points (GMPs) and Observation Wells (OWs)	8, 9	No discharge to groundwater from BRDA or Plant. Schedule C.6 Groundwater monitoring	Min. of Quarterly
Surface Water Discharge Control and Quality	Various Compliance Parameters	-	W1-1	8, 9, 10	Schedule C.2.1 and C.2.2 following treatment in AAL Effluent Clarification System (ECS)	Max. of 1,250 m^3/hr
Storm Water Discharge Control and Quality	Various Compliance Parameters	-	SS1 to SS5 and select Estuarine Streams (ES)	8, 9, 10	Schedule C.2.3 and C.6 Non-contact areas surrounding the BRDA and Plant Select ES are pumped to ECS	
Waste Monitoring Control and Quality	Various Compliance Parameters	-	-	8, 9, 10	Schedule C.4 Bauxite Residue, Process Sand, Salt Cake, Sludge, Leachate from BRDA	Min. of Quarterly
Noise	Various Compliance Parameters	-	NSL1 to NSL5 B1 to B9	8, 9, 10	Schedule C.5 Surveys at Daytime, Evening time and Night-time for various durations	Min. of Annually
Dust Emissions	Various Compliance Parameters	-	External and internal dust bowls	8, 9	Schedule C.6 Internal AAL procedures	Min of Quarterly
Soil	Various Compliance Parameters	-	Locations external to BRDA	8, 9, 10	Schedule C.6 Compared to baseline data	Min of 5 years

8.0 REFERENCES AND OTHER GUIDELINES / STANDARDS

AAL OMS, EMP, EAP and SWMs

Golder Tailings Design Practice Manual

Industry Standard and Guidelines

ANCOLD – Australian National Committee on Large Dams

2012 Guidelines on Tailings Dams

2019 Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure

CDA – Canadian Dam Association

2007 Dam Safety Guidelines (Revised 2013)

2014 Technical bulletin: Application of Dam Safety Guidelines to Mining Dams

2016 Technical Bulletin: Dam Safety Reviews

EU BAT – European Union Best Available Techniques

2018 Best Available Techniques Reference Document for the Management of Waste from Extractive Industries, in accordance with Directive 2006/21/EC (MWEI BREF, Dec 2018)

Global Industry Standard on Tailings Management

ICOLD – International Committee for Large Dams

2007 Dam Safety Guidelines (Revised 2013)

USCOLD – United States Committee on Large Dams

Standard Design Practices and Methods

ASTM International - American Society for Testing and Materials

BS – British Standard

EN – European Standard

IS – International Standard

ISO – International Organization for Standardization

NSAI – National Standard Authority of Ireland

Other National Standards

DIN - Deutsches Institut für Normung e.V. German national organisation for standards

TII – Transport Infrastructure Ireland, Specification for Roadworks, Series 600 – Earthworks (Publication No. CC-SPW-00600)

UK MCHW – Volume 1, UK Manual of Contract Documents for Highway Works, Specification for Highway Works, Series 600 (Earthworks)

Signature Page

Golder Associates Ireland Limited

Brian Keenan

Brian Keenan
Project Manager, Associate

Gerd Janssens
Senior Geotechnical Engineer

BK/GJ/ar

APPENDIX B

Drawings

OSI Map Series:
===========
1:2500

4795-B
REVISION DATE = 07-Jul-2014
SURVEY DATE = 31-Dec-1980

4795-D
REVISION DATE = 06-Jun-2014
SURVEY DATE = 30-Jun-2001

4796-A
REVISION DATE = 07-Jul-2014
SURVEY DATE = 30-Jun-2001

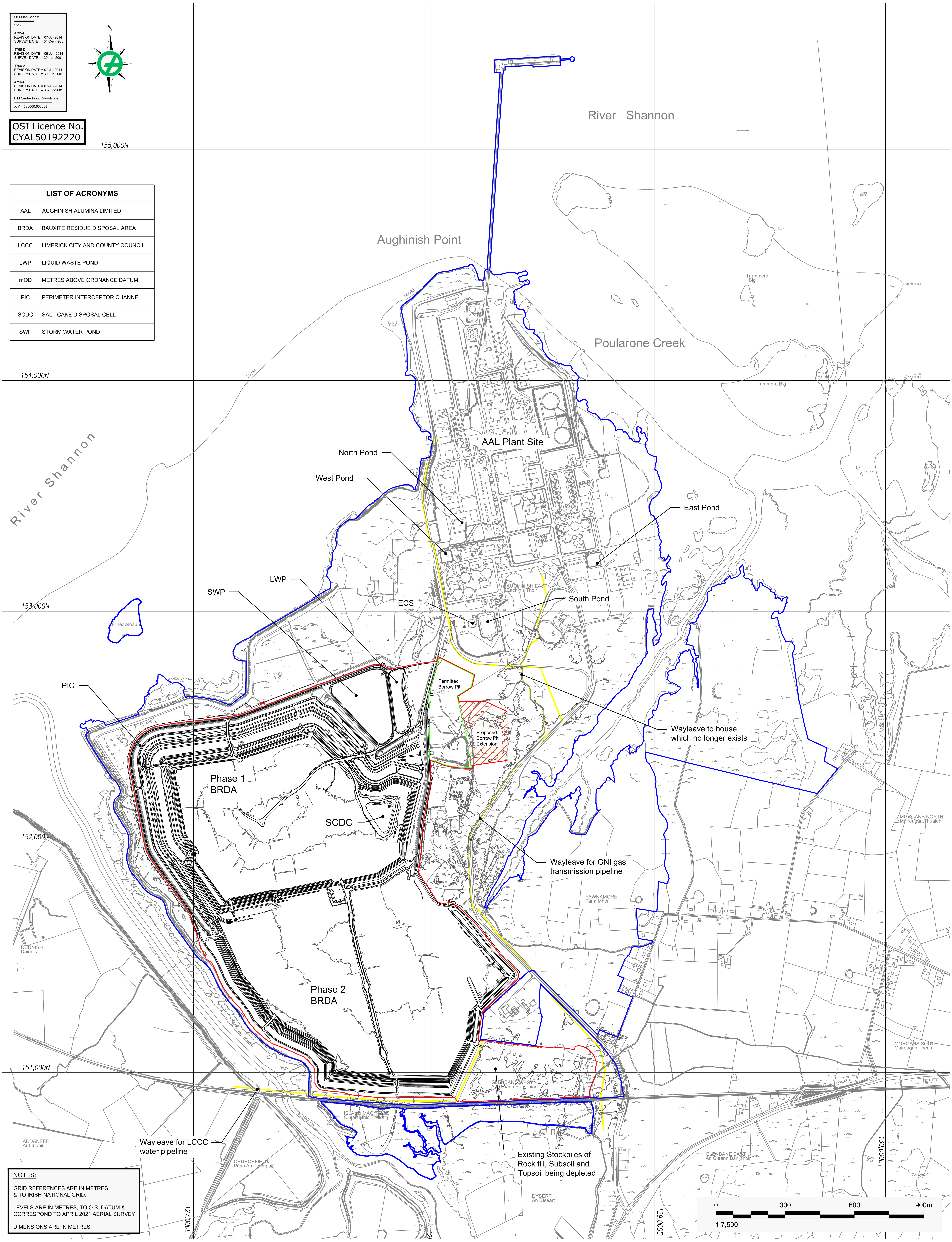
4796-C
REVISION DATE = 07-Jul-2014
SURVEY DATE = 30-Jun-2001

ITM Centre Point Co-ordinate:
====================
X,Y = 528060,652638

OSI Licence No.
CYAL50192220

LIST OF ACRONYMS

AAL	AUGHINISH ALUMINA LIMITED
BRDA	BAUXITE RESIDUE DISPOSAL AREA
LCCC	LIMERICK CITY AND COUNTY COUNCIL
LWP	LIQUID WASTE POND
mOD	METRES ABOVE ORDNANCE DATUM
PIC	PERIMETER INTERCEPTOR CHANNEL
SCDC	SALT CAKE DISPOSAL CELL
SWP	STORM WATER POND



NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

LEGEND

- AAL OWNERSHIP BOUNDARY & FORESHORE LEASE
- APPLICATION BOUNDARY
- WAYLEAVE
- 15 EXISTING GROUND CONTOUR (mOD), APRIL 2021
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION FOOTPRINT

CLIENT
AUGHINISH ALUMINA LTD.

PROJECT
BRDA RAISE DESIGN

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

TITLE
SITE LOCATION PLAN - Overall

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	01	A	1:7,500 A1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1
25 mm
0

OSI Map Series:
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1:2500

4795-B
REVISION DATE = 07-Jul-2014
SURVEY DATE = 31-Dec-1980

4795-D
REVISION DATE = 06-Jun-2014
SURVEY DATE = 30-Jun-2001

4796-A
REVISION DATE = 07-Jul-2014
SURVEY DATE = 30-Jun-2001

4796-C
REVISION DATE = 07-Jul-2014
SURVEY DATE = 30-Jun-2001

ITM Centre Point Co-ordinate:
===================
X,Y = 528060,652638

OSI Licence No.
CYAL50192220

LIST OF ACRONYMS	
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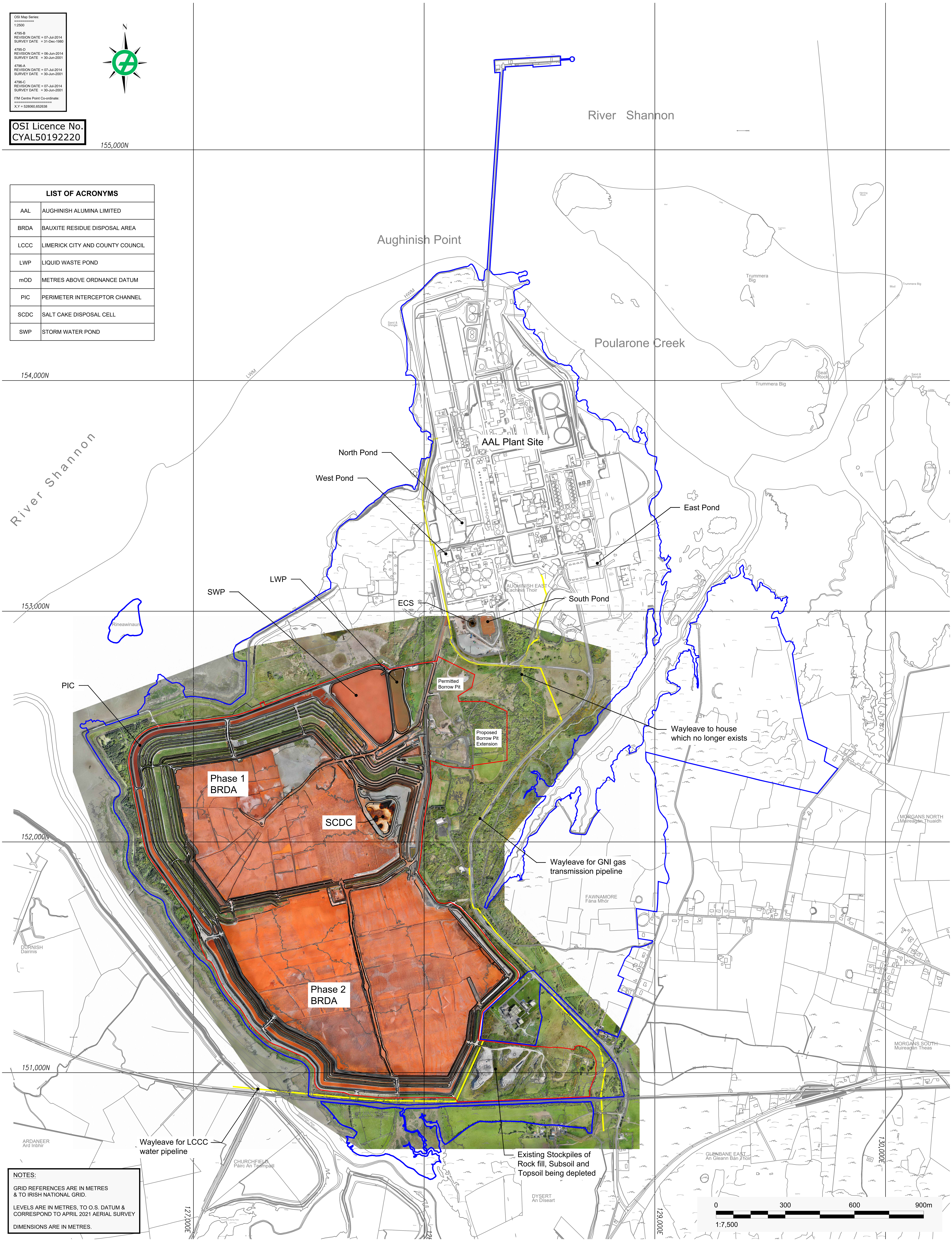
LWP	LIQUID WASTE POND
mOD	METRES ABOVE ORDNANCE DATUM
PIC	PERIMETER INTERCEPTOR CHANNEL
SCDC	SALT CAKE DISPOSAL CELL
SWP	STORM WATER POND



NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

LEGEND

- AAL OWNERSHIP BOUNDARY & FORESHORE LEASE
- APPLICATION BOUNDARY
- WAYLEAVE
- 15 EXISTING GROUND CONTOUR (mOD), APRIL 2021
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION FOOTPRINT

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

PROJECT

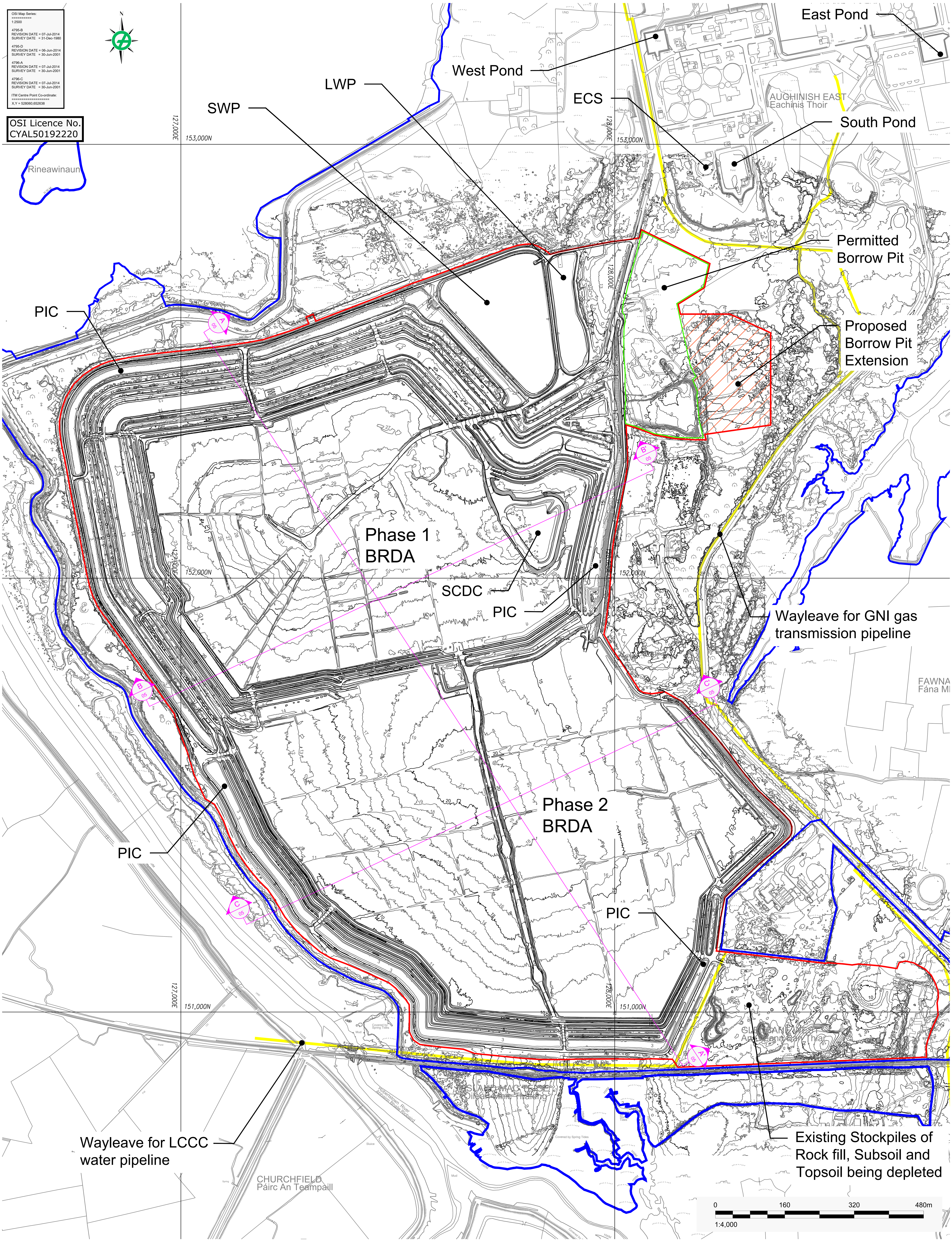
BRDA RAISE DESIGN

TITLE

SITE LOCATION PLAN - Overall with Aerial (April 2021)

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	01a	A	1:7,500 A1

0 25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1



LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 OGL CONTOUR (mOD), APRIL 2021
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION BOUNDARY

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

CLIENT
AUGHINISH ALUMINA LTD.

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

PROJECT
BRDA RAISE DESIGN

TITLE
SITE LOCATION PLAN - BRDA

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	02	A	1:4,000 A1

25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 OGL CONTOUR (mOD), APRIL 2021
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION BOUNDARY

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

PROJECT

BRDA RAISE DESIGN

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

TITLE

SITE LOCATION PLAN - BRDA with Aerial (April 2021)

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	02a	A	1:4,000 A1

25 mm
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

OSI Map Series:
==========
1:2500

4795-B
REVISION DATE = 07-Jul-2014
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4795-D
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SURVEY DATE = 30-Jun-2001

ITM Centre Point Co-ordinate:
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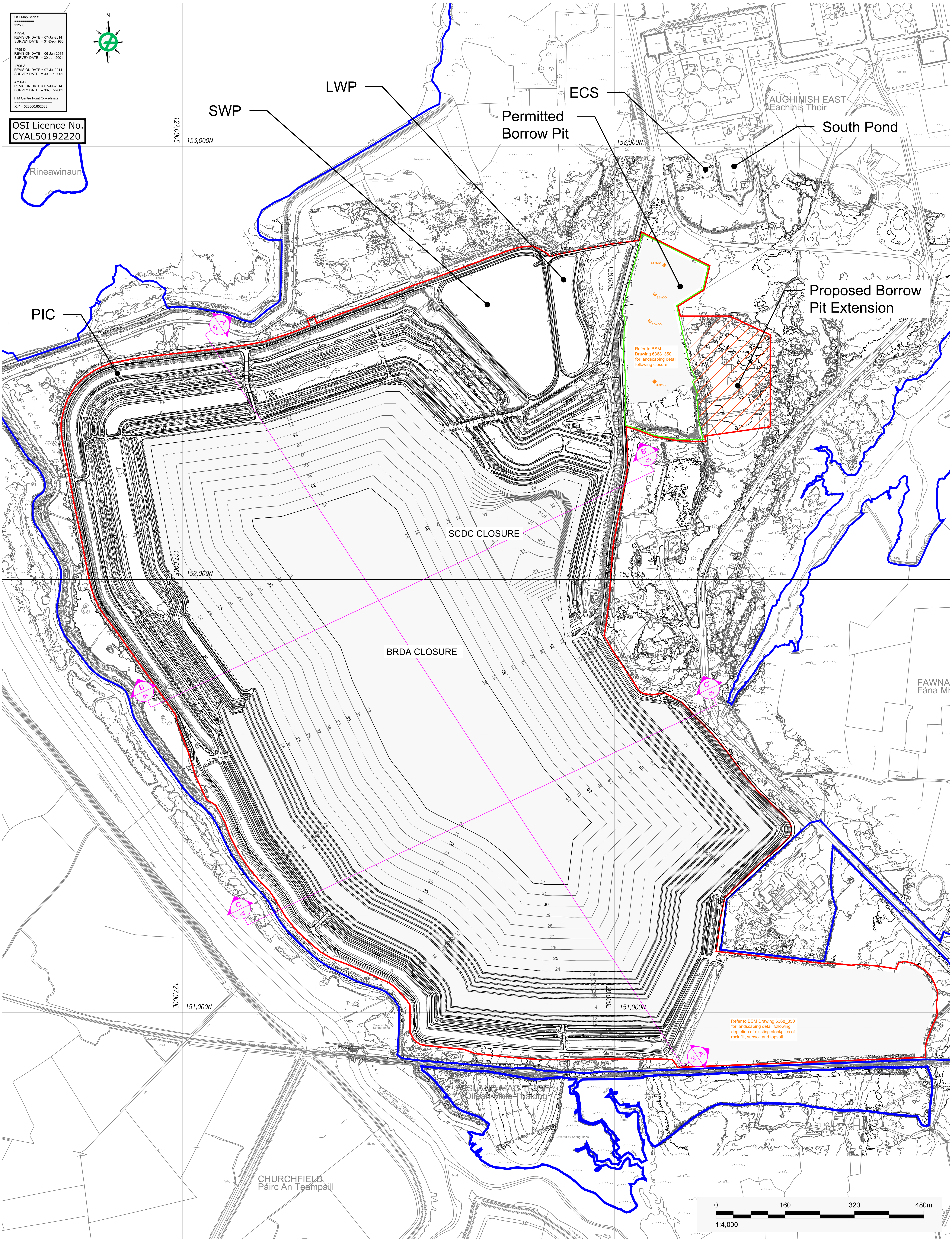
X,Y = 528060,652638

OSI Licence No. CYAL50192220



LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 GROUND CONTOUR (mOD)
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION BOUNDARY

NOTES:

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LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

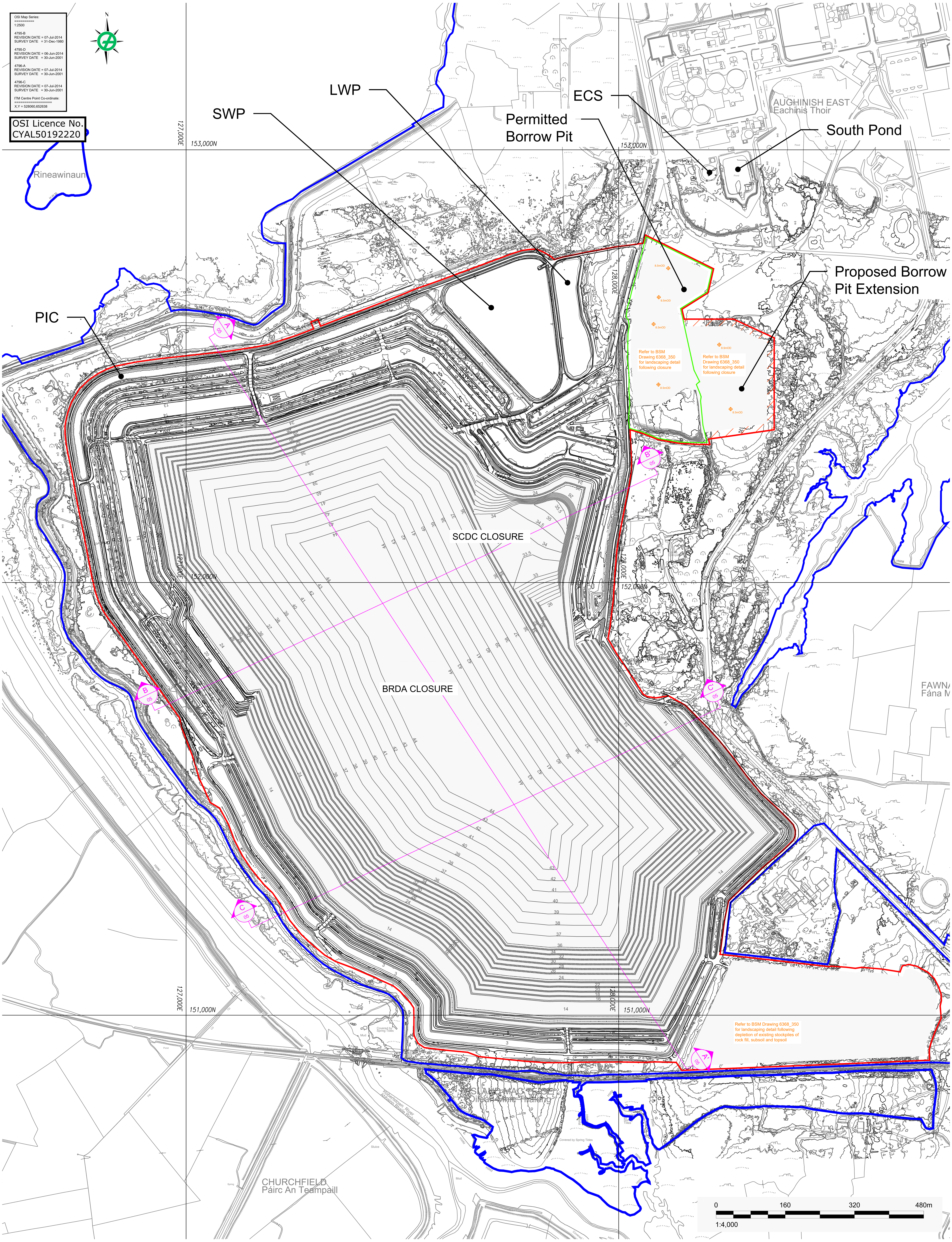
PROJECT

BRDA RAISE DESIGN

TITLE

PERMITTED BRDA TO STAGE 10 (24 mOD) WITH DOME

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	03	A	1:4,000 A1



LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 GROUND CONTOUR (mOD)
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION BOUNDARY

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

PROJECT

BRDA RAISE DESIGN

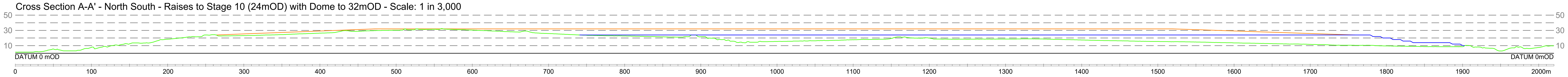
TITLE

PROPOSED BRDA RAISE TO STAGE 16 & SALT CAKE DISPOSAL CELL RAISE, BOTH WITH DOMES AT CLOSURE

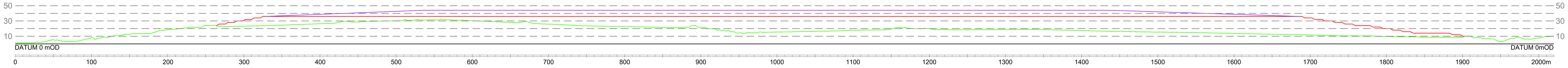
PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	04	A	1:4,000 A1

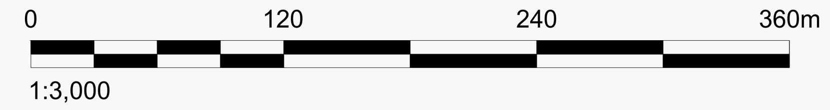
25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

Cross Section A-A' - North South - Raises to Stage 10 (24mOD) with Dome to 32mOD - Scale: 1 in 3,000

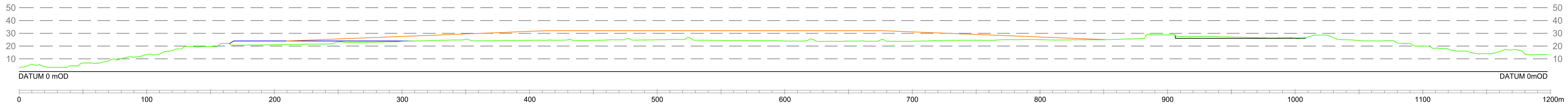


Cross Section A-A' - North South - Raises to Stage 16 (36mOD) with Dome to 44mOD - Scale: 1 in 3,000

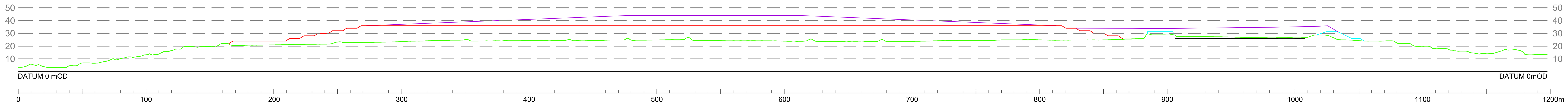


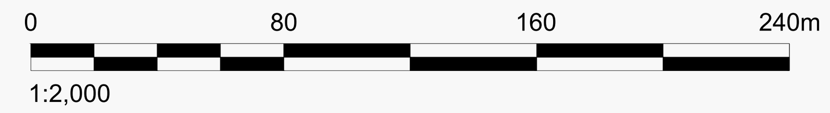


Cross Section B-B' - West East - Raises to Stage 10 (24mOD) with Dome to 32mOD - Scale: 1 in 2,000

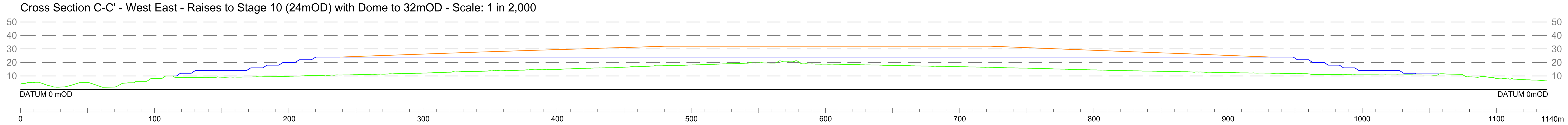


Cross Section B-B' - West East - Raises to Stage 16 (36mOD) with Dome to 44mOD & SCDC Raise - Scale: 1 in 2,000

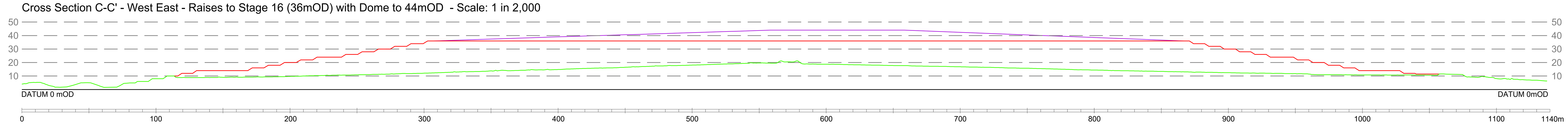


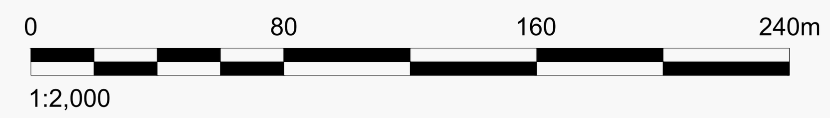


Cross Section C-C' - West East - Raises to Stage 10 (24mOD) with Dome to 32mOD - Scale: 1 in 2,000



Cross Section C-C' - West East - Raises to Stage 16 (36mOD) with Dome to 44mOD - Scale: 1 in 2,000





LEGEND

- EXISTING GROUND LEVEL, APRIL 2021
- RAISES TO STAGE 10 (24mOD)
- DOME FROM STAGE 10 (24mOD) TO 32mOD
- RAISES TO STAGE 16 (36mOD)
- DOME FROM STAGE 16 (36mOD) TO 44mOD
- SALT CAKE DISPOSAL CELL (SCDC) RAISE TO 31.25mOD

CLIENT

AUGHINISH ALUMINA LTD.

PROJECT

BRDA RAISE DESIGN

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

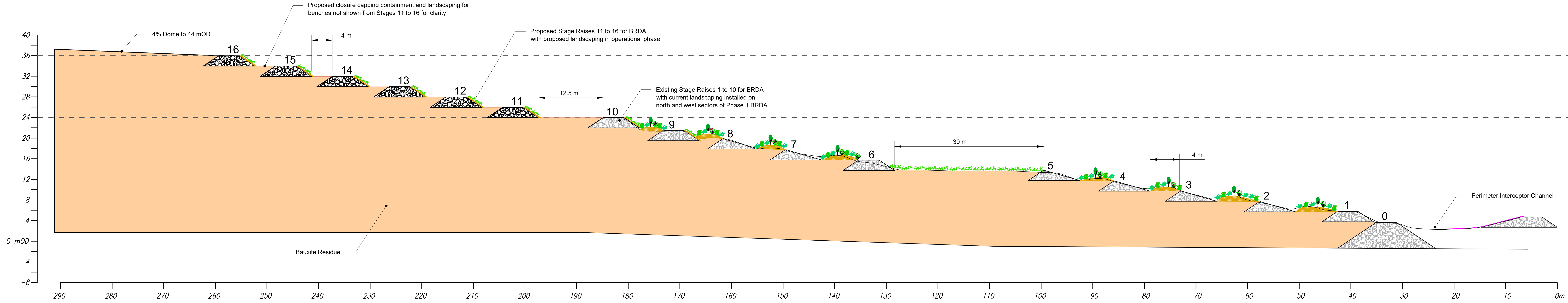
TITLE

BRDA CROSS SECTIONS

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	05	A	As shown A1

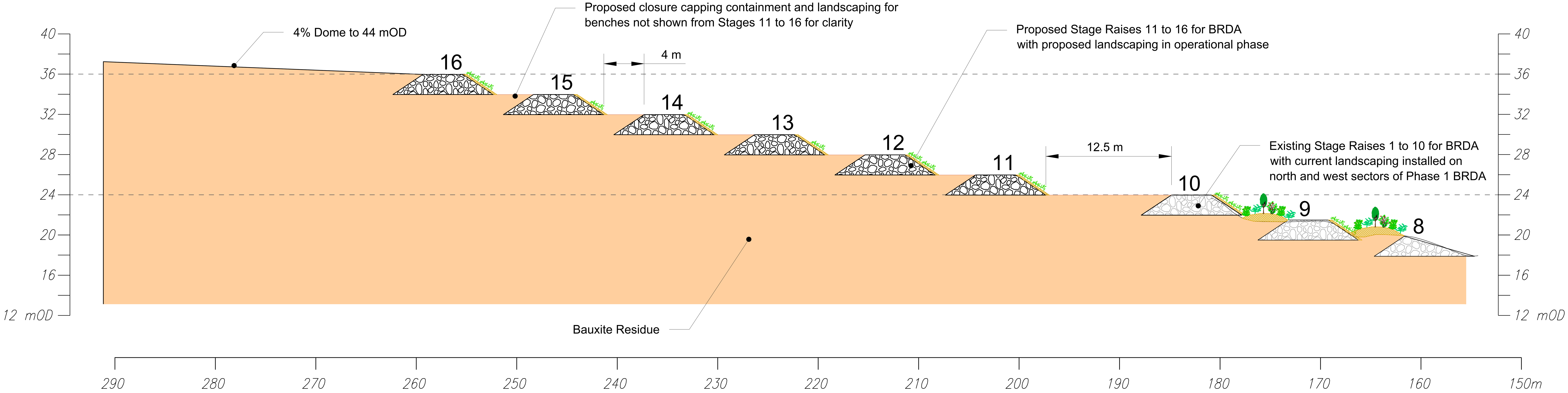
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

25 mm



TYPICAL BRDA SIDE-SLOPE PROFILE FROM EXISTING STAGE 0 TO STAGE 16

0 16 36 48m
1:400



TYPICAL BRDA SIDE-SLOPE PROFILE FROM PERMITTED STAGE 10 TO STAGE 16

0 8 16 24m
1:200

CLIENT
AUGHINISH ALUMINA LTD.

PROJECT
BRDA RAISE DESIGN

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	BK
DESIGN	BK
REVIEW	BK
APPROVED	BK

TITLE
TYPICAL BRDA SIDE-SLOPE PROFILE

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	06	A	As shown A1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

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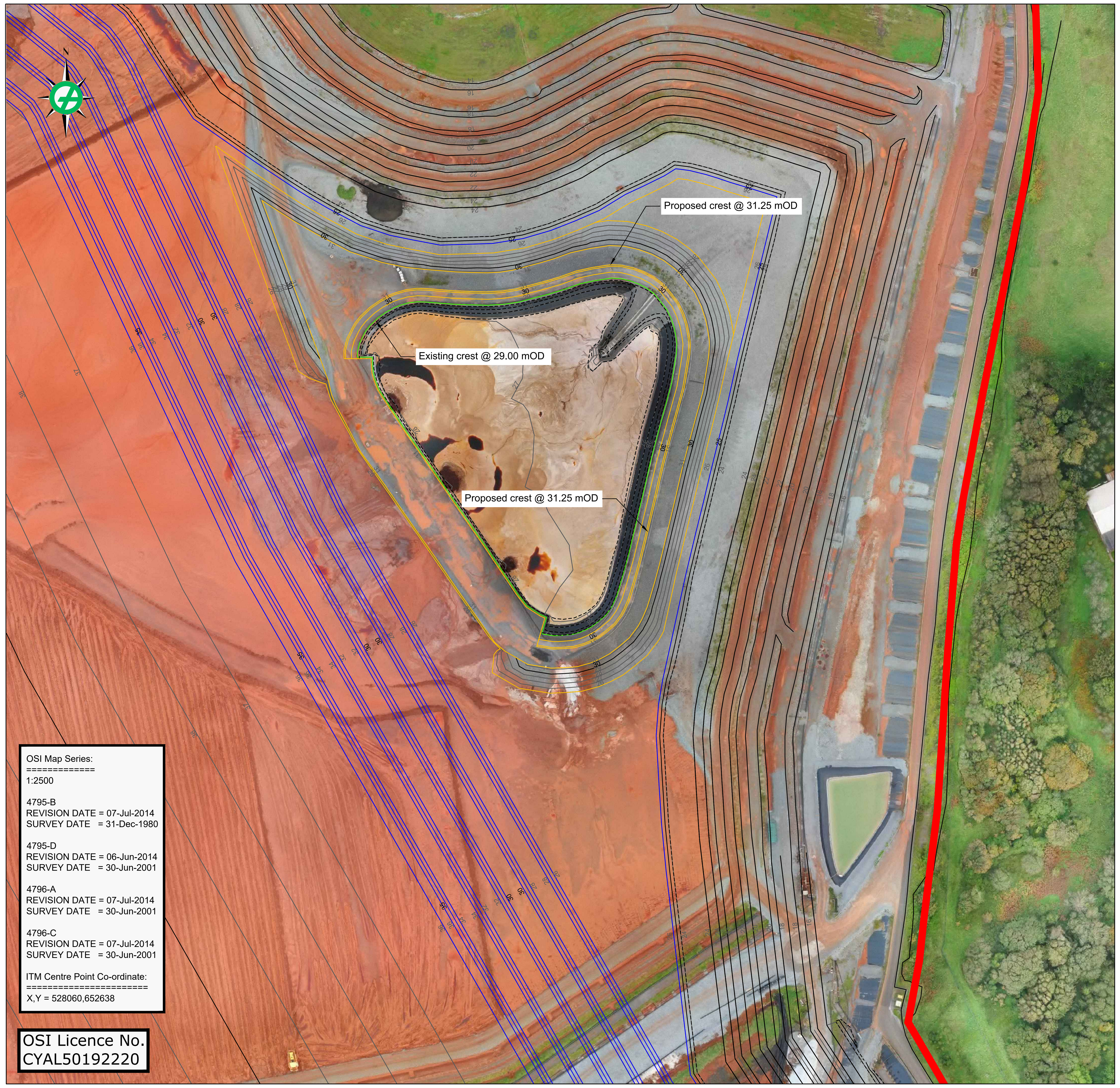
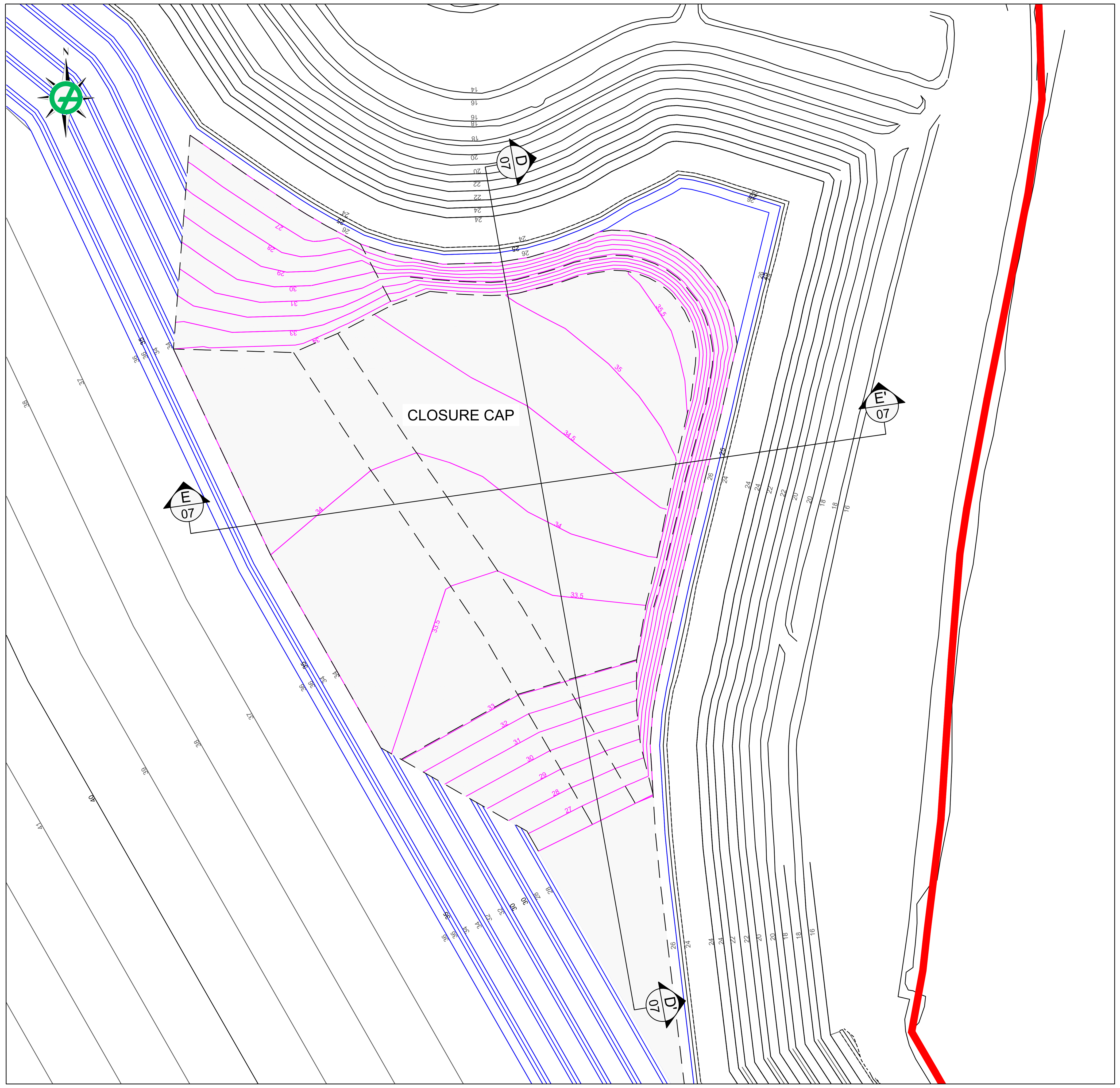
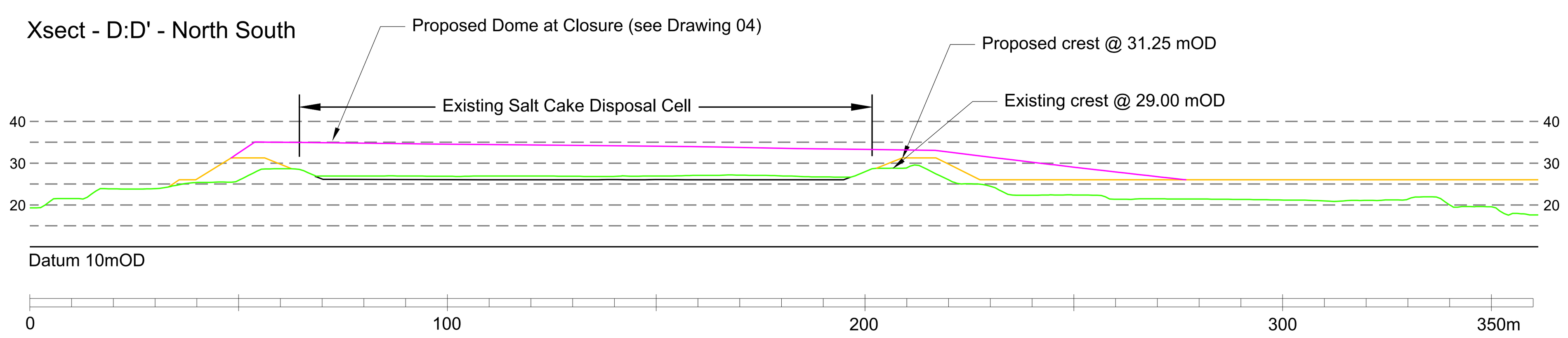
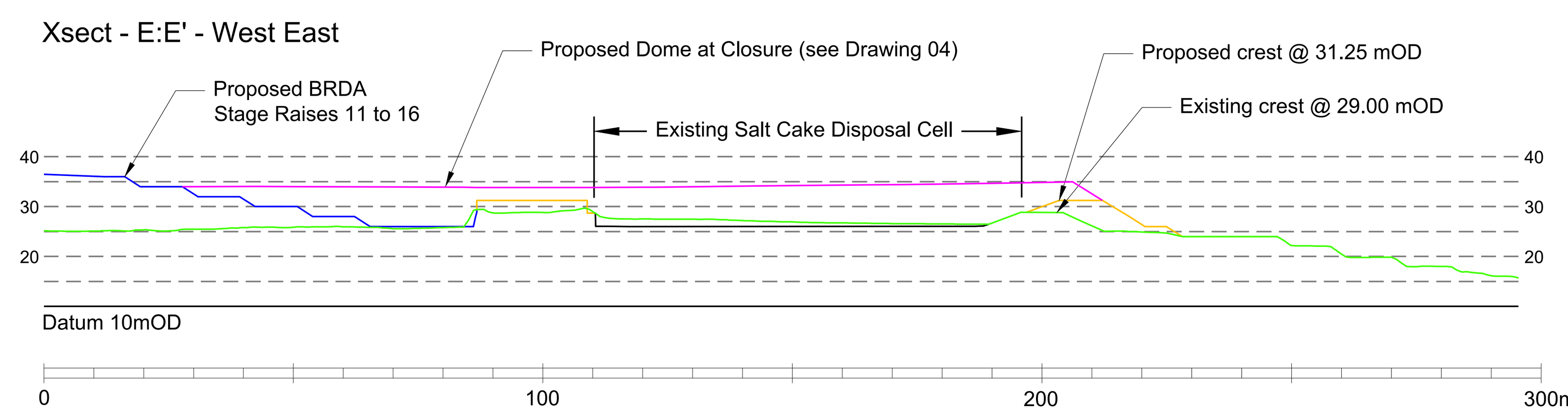


PROPOSED SALT CAKE DISPOSAL CELL RAISE: Plan View



PROPOSED SALT CAKE DISPOSAL CELL RAISE WITH CLOSURE CAP: Plan View





0 50 100 150m

1:1,250

LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 GROUND CONTOUR (mOD)
- EXISTING GROUND (mOD) FROM APRIL 2021 SURVEY
- PROPOSED SALT CAKE DISPOSAL CELL RAISE (mOD)
- PROPOSED RAISE TO BRDA (mOD), STAGE RAISES 11 TO 16
- PROPOSED DOME AT CLOSURE OVER THE SALT CAKE DISPOSAL CELL RAISE (mOD): SEE DRAWING 04

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

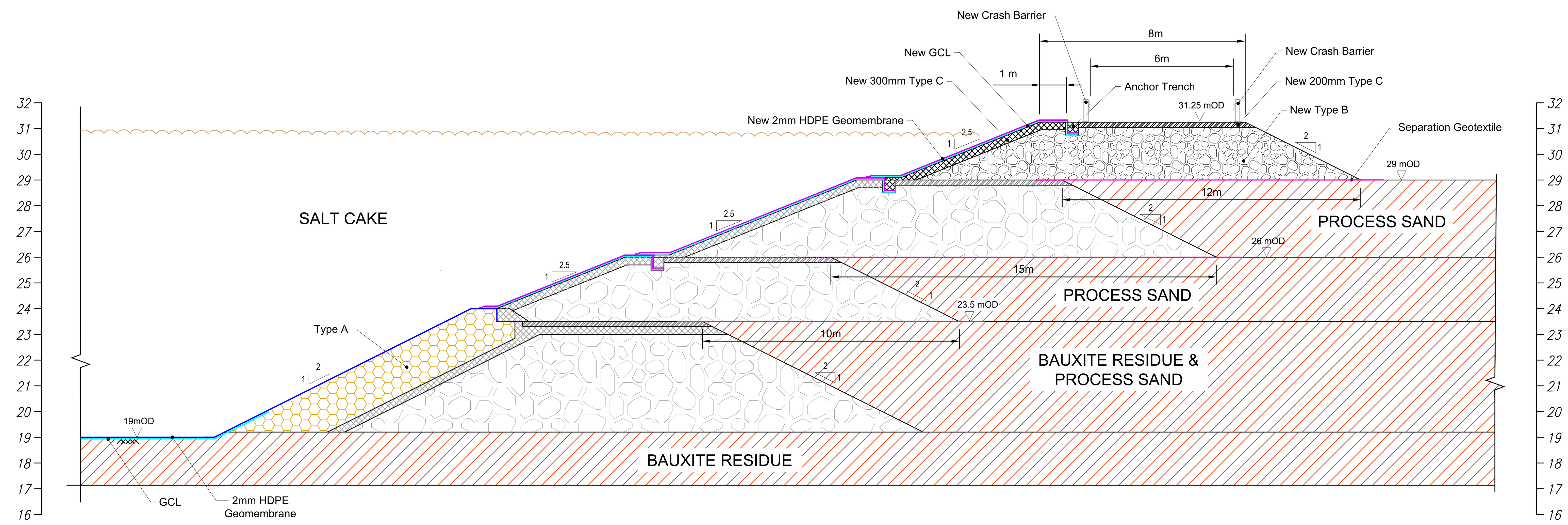
PROJECT

BRDA RAISE DESIGN

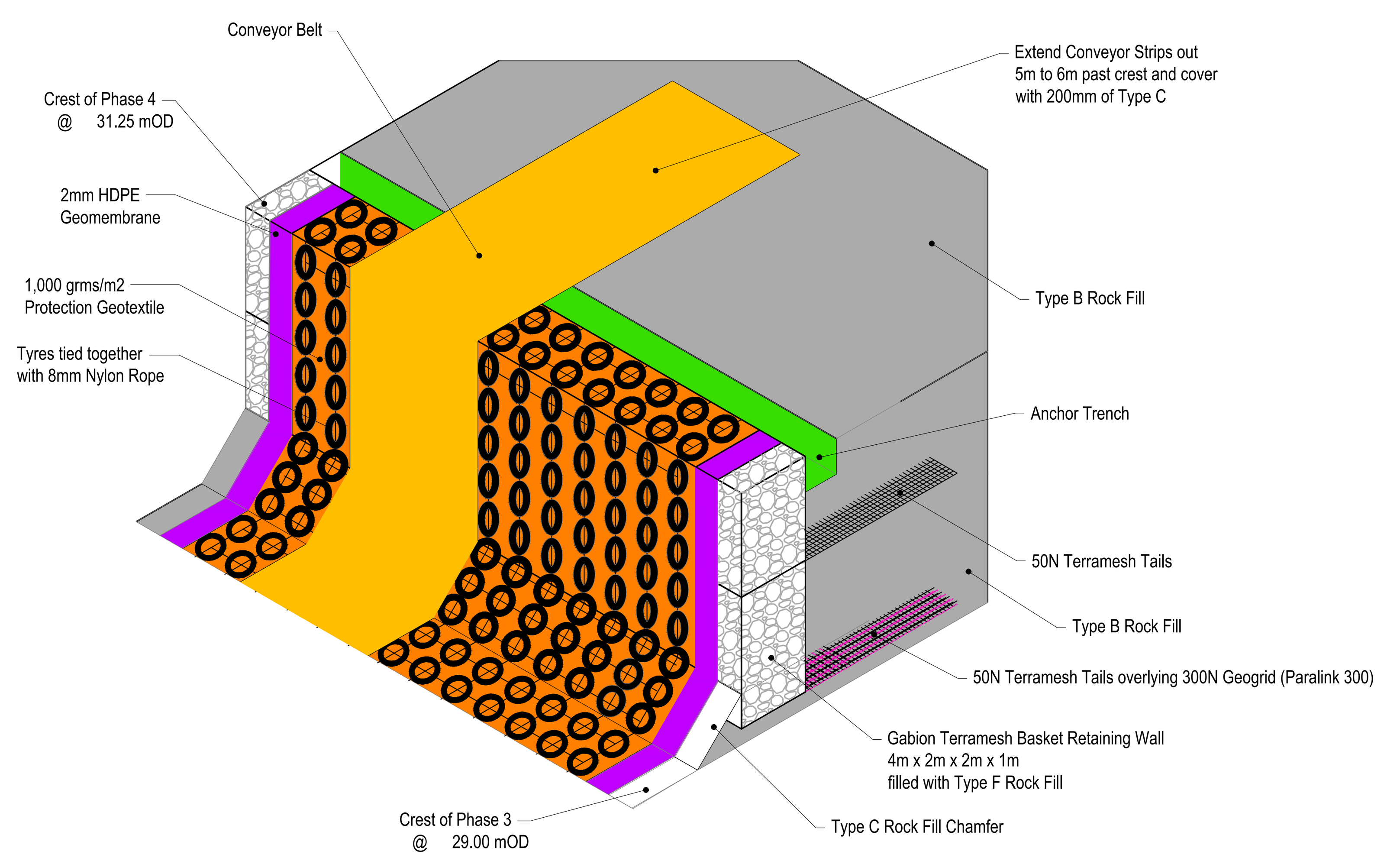
TITLE

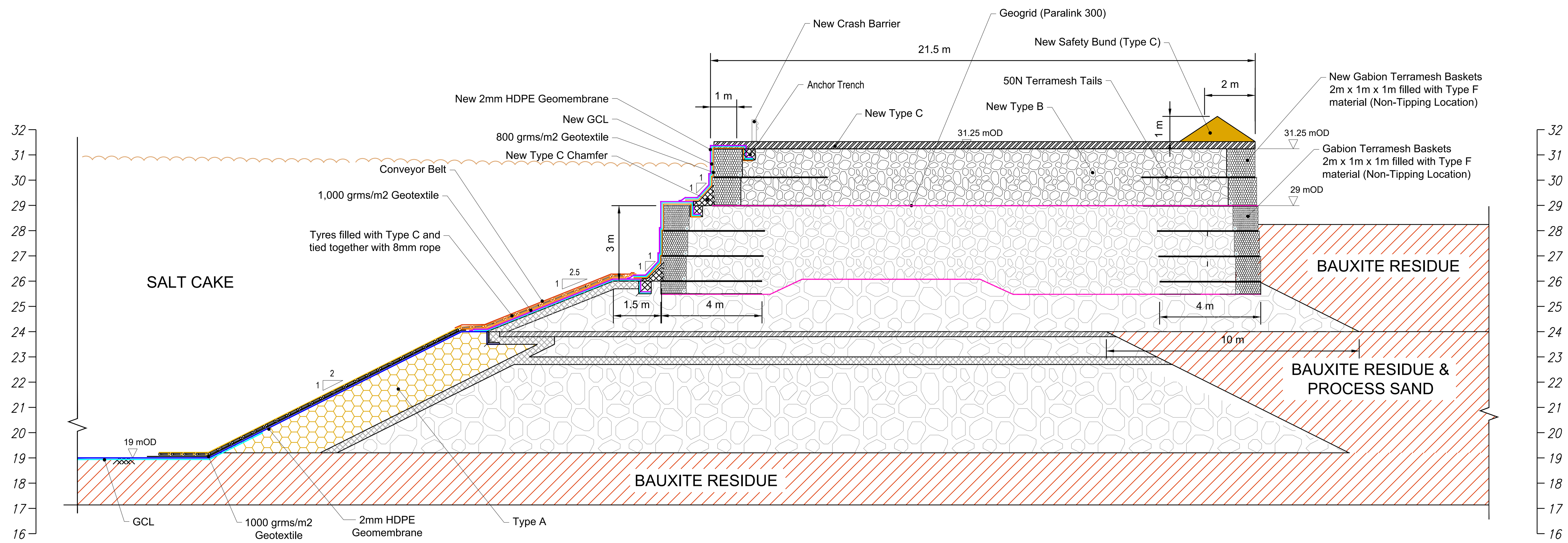
PROPOSED SALT CAKE DISPOSAL CELL RAISE: PLAN AND SECTIONS

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	07a	A	1:1,250 A1



TYPICAL SECTION FOR NORTH AND EAST CELL WALL





TYPICAL ISOMETRIC FOR LINER PROTECTION AT TIPPING POINT ON WEST CELL WALL

TYPICAL SECTION FOR WEST CELL WALL (NO LINER PROTECTION SHOWN FOR CLARITY)

0 5 10 15m

1:125

CLIENT

AUGHINISH ALUMINA LTD.

PROJECT

BRDA RAISE DESIGN

CONSULTANT

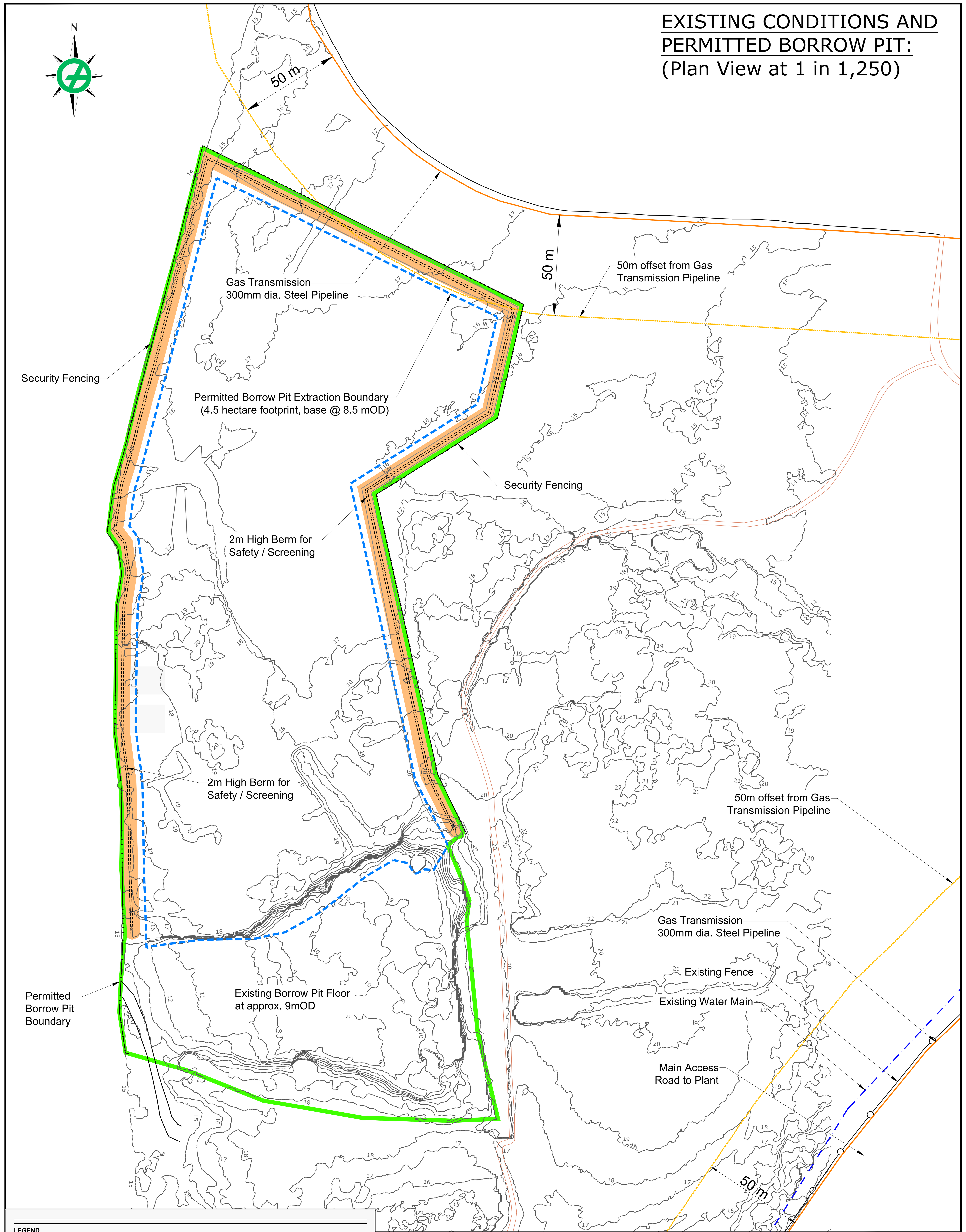
GOLDER
MEMBER OF WSP

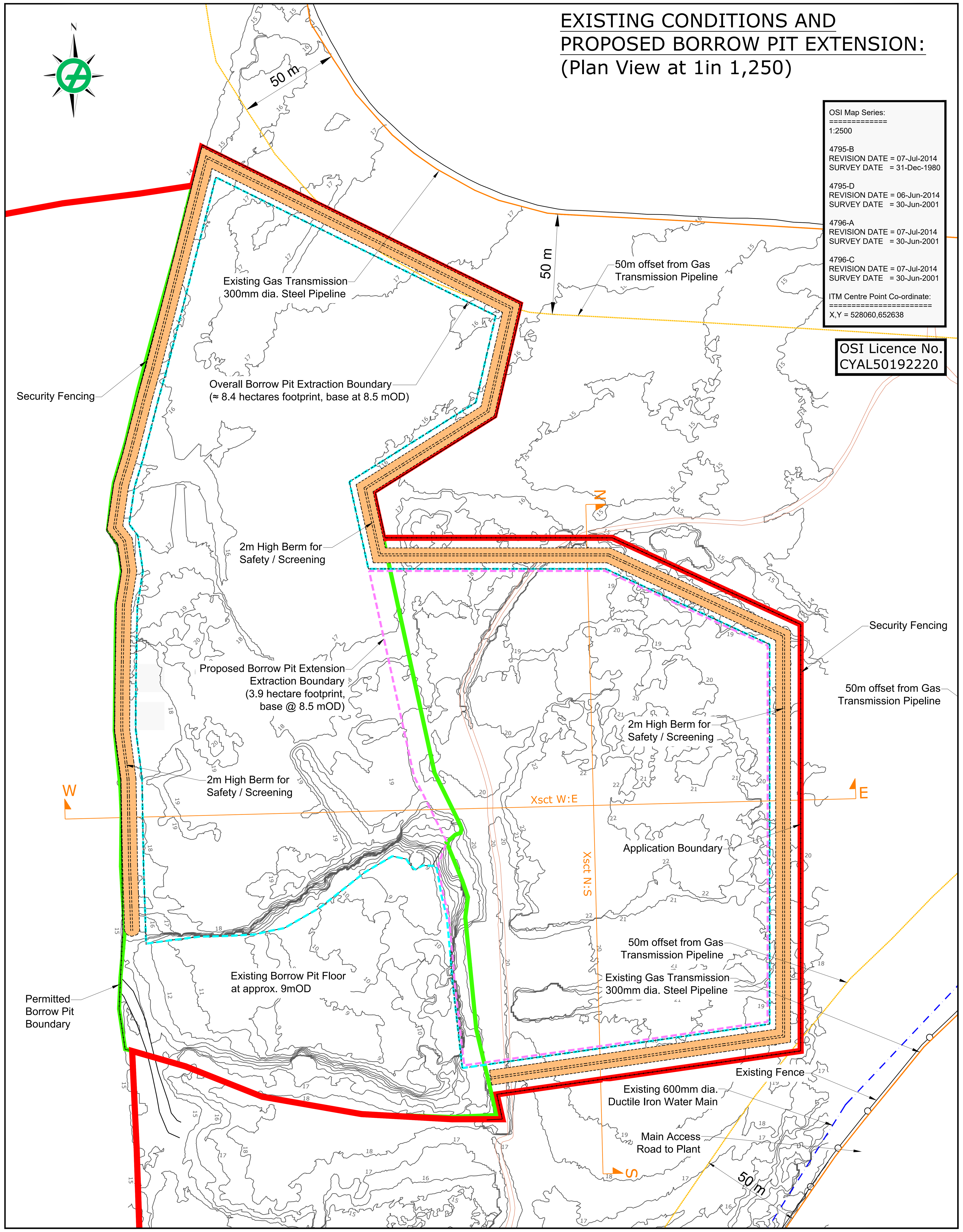
YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

TITLE

PROPOSED SALT CAKE DISPOSAL CELL RAISE:
CELL WALL SECTIONS

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	07b	A	1:125 A1





LEGEND

- APPLICATION BOUNDARY
- PERMITTED BORROW PIT BOUNDARY
- PERMITTED BORROW PIT EXTRACTION BOUNDARY
- PROPOSED BORROW PIT EXTENSION EXTRACTION BOUNDARY
- OVERALL EXTRACTION BOUNDARY (PERMITTED BORROW PIT AND PROPOSED EXTENSION)
- SAFETY / SCREENING BERM (MIN. 2m HIGH)
- SECURITY FENCING (SEE DETAIL A ON DRAWING 13b)
- 18 EXISTING CONDITIONS GROUND CONTOUR (mOD)
- GAS TRANSMISSION 300mm DIA. STEEL PIPELINE
- 50m OFFSET FROM GAS TRANSMISSION PIPELINE

0 50 100 150m

1:1,250

NOTES:

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LEVELS ARE IN METRES, TO O.S. DATUM &
CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

PROJECT

BRDA RAISE DESIGN

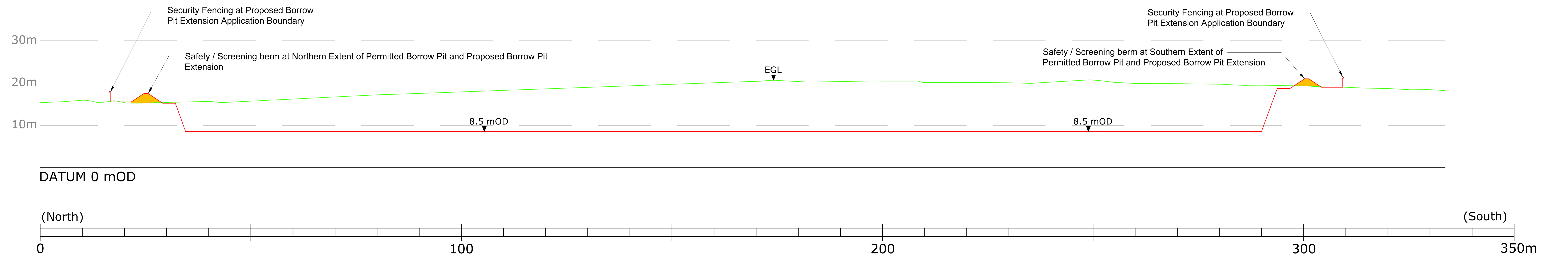
TITLE

PROPOSED BORROW PIT EXTENSION - Layout Plan

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	08a	A	1:1,250 A1

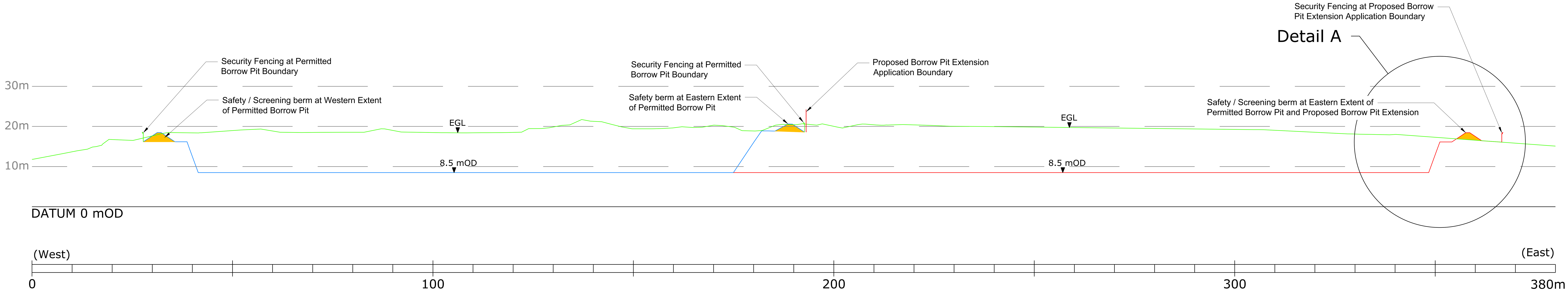
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

25 mm



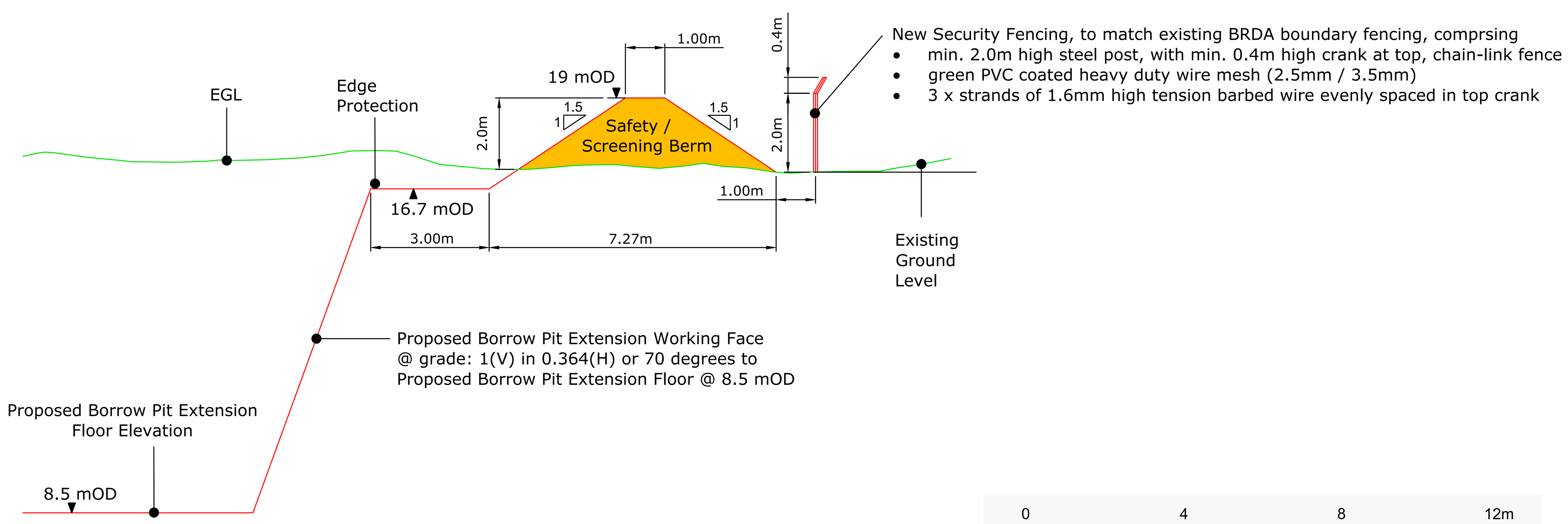
Cross Section NS: (North South)

0 20 40 60m
1:500



Cross Section WE: (West East)
Scale: 1 in 400

0 20 40 60m
1:500



Detail A: Safety / Screening Berm
Scale: 1 in 50

0 4 8 12m
1:100

LEGEND

- EXISTING GROUND LEVEL (EGL) FROM APRIL 2021 SURVEY (mOD)
- PROPOSED BORROW PIT EXTENSION (mOD)
- PERMITTED BORROW PIT (mOD)

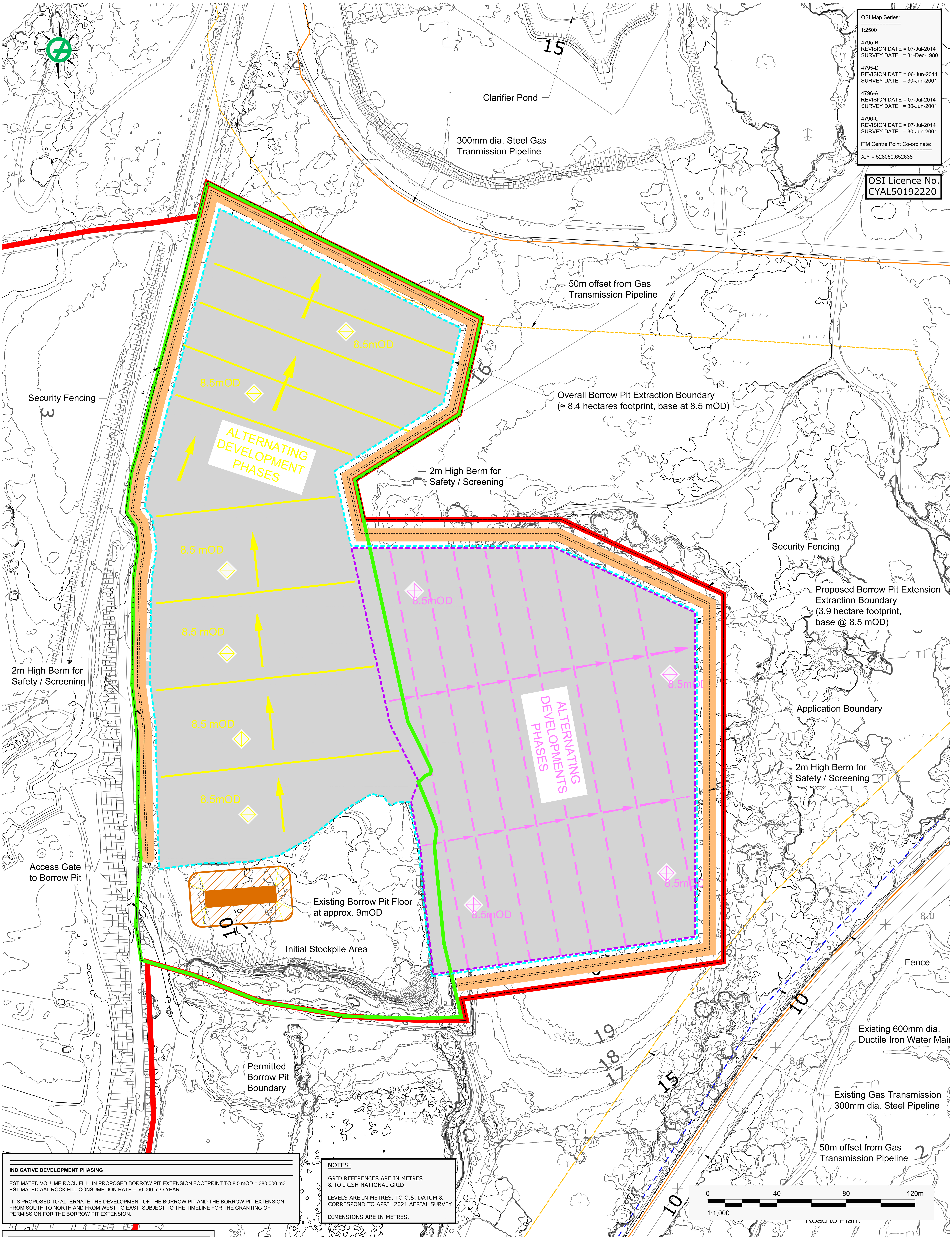
CLIENT	PROJECT
AUGHINISH ALUMINA LTD.	BRDA RAISE DESIGN

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

TITLE
PROPOSED BORROW PIT EXTENSION - Cross Sections

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	08b	A	As shown A1



INDICATIVE DEVELOPMENT PHASING

ESTIMATED VOLUME ROCK FILL IN PROPOSED BORROW PIT EXTENSION FOOTPRINT TO 8.5 mOD = 380,000 m3
ESTIMATED AAL ROCK FILL CONSUMPTION RATE = 50,000 m3 / YEAR

IT IS PROPOSED TO ALTERNATE THE DEVELOPMENT OF THE BORROW PIT AND THE BORROW PIT EXTENSION FROM SOUTH TO NORTH AND FROM WEST TO EAST, SUBJECT TO THE TIMELINE FOR THE GRANTING OF PERMISSION FOR THE BORROW PIT EXTENSION.

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

LEGEND

- APPLICATION BOUNDARY
- PERMITTED BORROW PIT BOUNDARY
- PERMITTED BORROW PIT EXTRACTION BOUNDARY
- PROPOSED BORROW PIT EXTENSION EXTRACTION BOUNDARY
- OVERALL EXTRACTION BOUNDARY (PERMITTED BORROW PIT AND PROPOSED EXTENSION)
- SAFETY / SCREENING BERM (MIN. 2m HIGH)
- SECURITY FENCING (SEE DETAIL A ON DRAWING 13b)
- 18 EXISTING CONDITIONS GROUND CONTOUR (mOD)
- PERMITTED BORROW PIT PHASING (INDICATIVE)
- PROPOSED BORROW PIT EXTENSION PHASING (INDICATIVE)
- GAS TRANSMISSION 300mm DIA. STEEL PIPELINE
- 50m OFFSET FROM GAS TRANSMISSION PIPELINE

CLIENT	PROJECT
AUGHINISH ALUMINA LTD.	BRDA RAISE DESIGN

CONSULTANT

GOLDER
MEMBER OF WSP

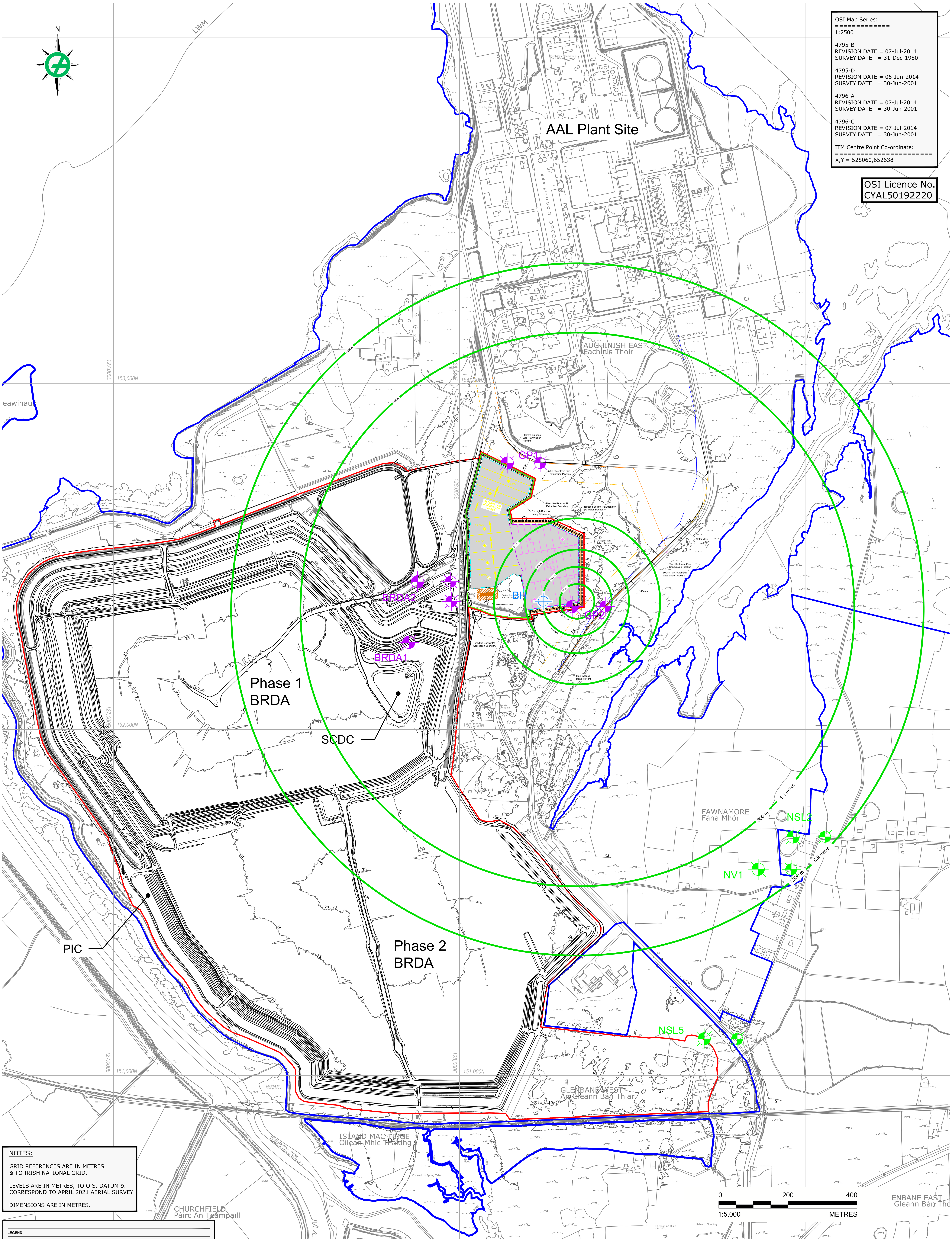
YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

TITLE

PROPOSED BORROW PIT EXTENSION: INDICATIVE PHASING

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	08c	A	1:1,000 A1

25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1



NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

LEGEND

- APPLICATION BOUNDARY
- PERMITTED BORROW PIT BOUNDARY
- PERMITTED BORROW PIT EXTRACTION BOUNDARY
- PROPOSED BORROW PIT EXTENSION EXTRACTION BOUNDARY
- OVERALL EXTRACTION BOUNDARY (PERMITTED BORROW PIT AND PROPOSED EXTENSION)
- SAFETY / SCREENING BERM (MIN. 2m HIGH)
- SECURITY FENCING (SEE DETAIL A ON DRAWING 13b)
- 18 EXISTING CONDITIONS GROUND CONTOUR (mOD)
- PERMITTED BORROW PIT PHASING (INDICATIVE)
- PROPOSED BORROW PIT EXTENSION PHASING (INDICATIVE)
- PEAK PARTICLE VELOCITY (PPV) CONTOUR IN mm/s
- NSL5 VIBRATION AND AIR OVERPRESSURE MONITORING
- GP2 VIBRATION MONITORING

CLIENT

AUGHINISH ALUMINA LTD.

PROJECT

BRDA RAISE DESIGN

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	POB
REVIEW	BK
APPROVED	BK / AAL

TITLE

Borrow Pit Extension: Estimated Peak Particle Velocity (PPV) Contour Plot (based on the selected design blast parameters)

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	08d	A	1:5,000 A1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

25 mm

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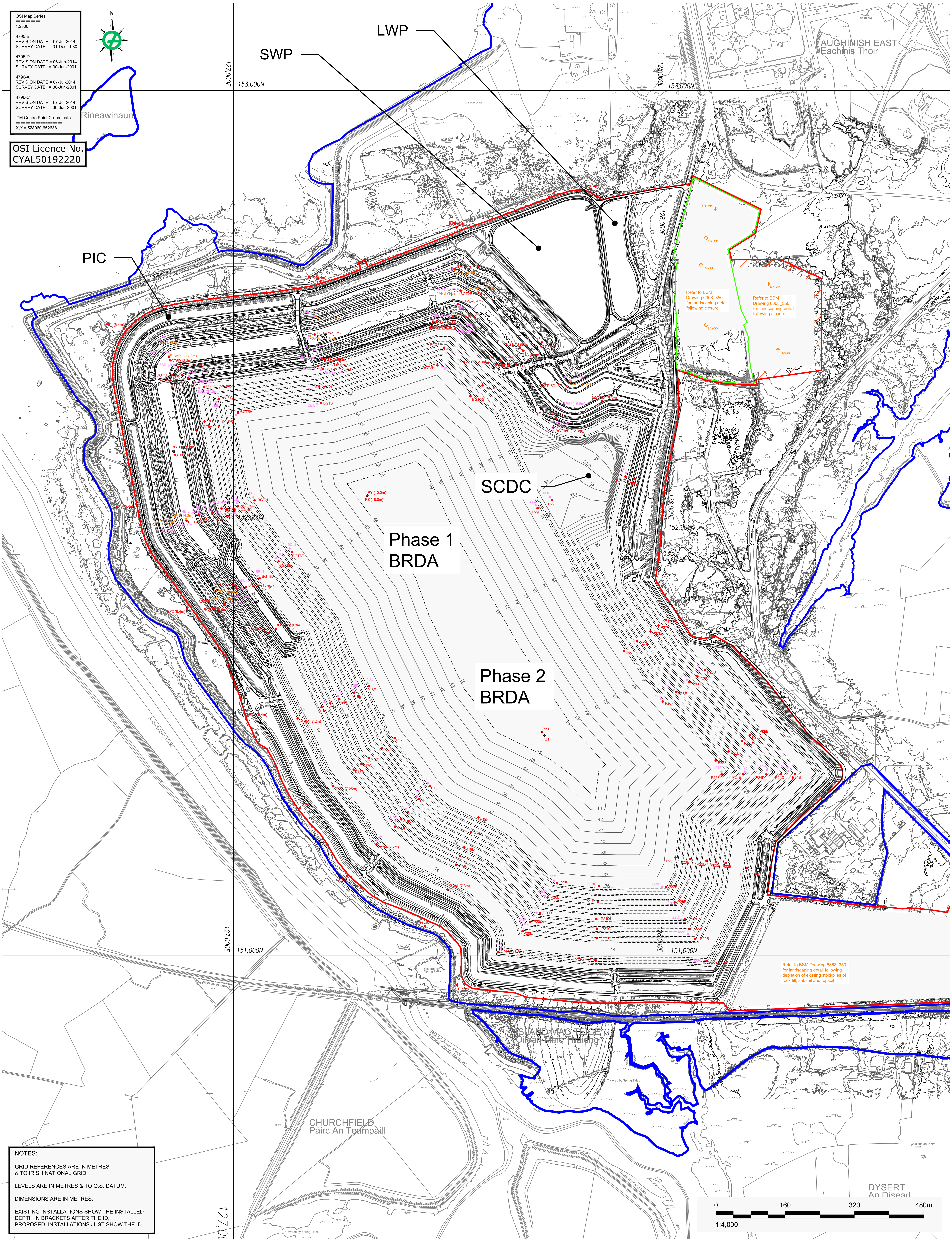


NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES & TO O.S. DATUM.

DIMENSIONS ARE IN METRES.

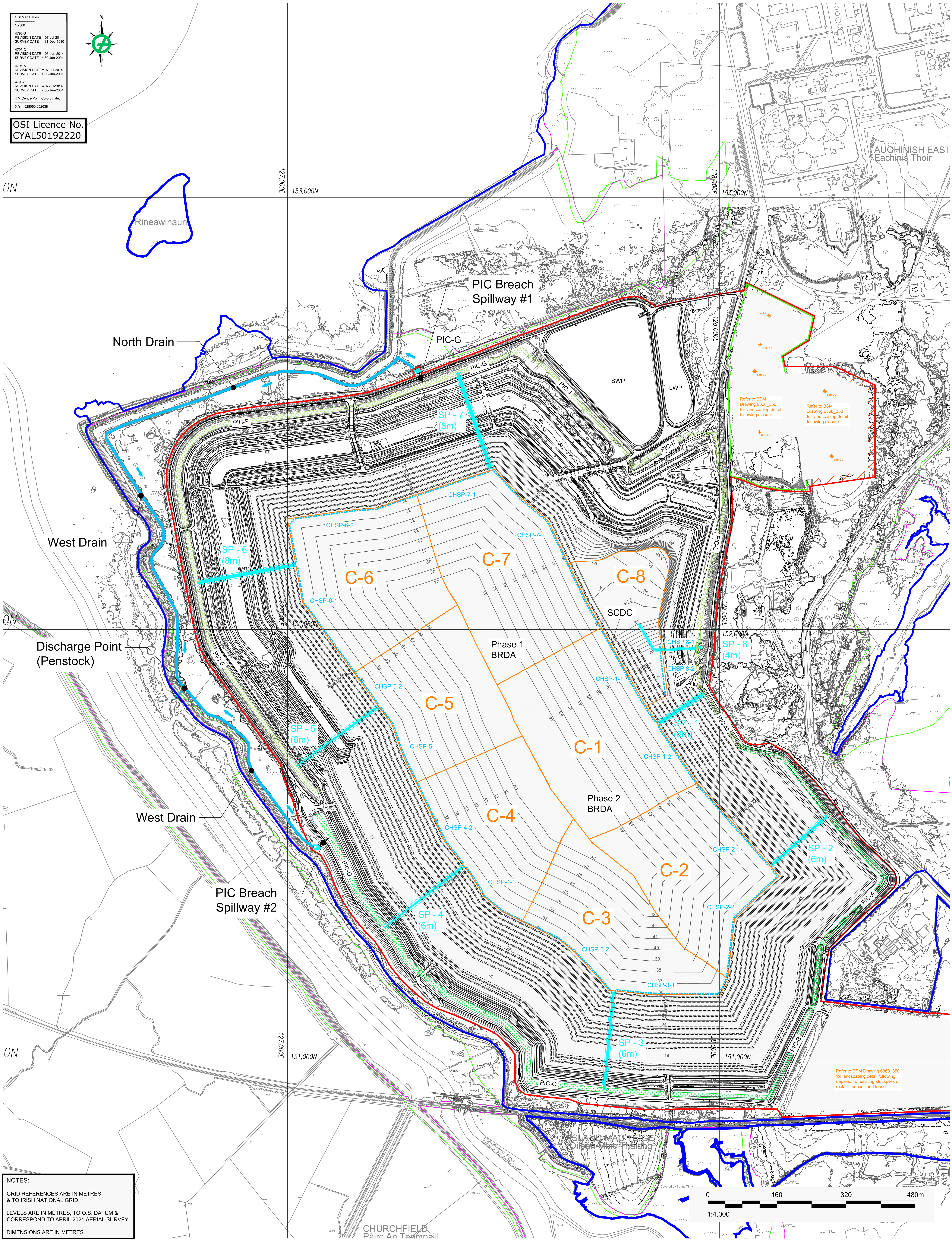
EXISTING INSTALLATIONS SHOW THE INSTALLED DEPTH IN BRACKETS AFTER THE ID. PROPOSED INSTALLATIONS JUST SHOW THE ID

LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 15 GROUND CONTOUR (mOD)
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION FOOTPRINT
- BGT / EP / P — STANDPIPE PIEZOMETER
- AIL / AIU — INCLINOMETER
- APL / APU — CASAGRANDE PIEZOMETER

CLIENT	AUGHINISH ALUMINA LTD.
PROJECT	BRDA RAISE DESIGN
CONSULTANT	GOLDER MEMBER OF WSP
YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	GJ
TITLE	MONITORING INSTRUMENTATION PLAN FOR BRDA RAISE TO STAGE 16
PROJECT No.	20143076
DRAWING No.	09
Rev.	A
SCALE	1:4,000 A1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1 0 25 mm



NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 15 EXISTING GROUND CONTOUR (mOD) APRIL 2021
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION FOOTPRINT
- SPECIAL PROTECTION AREA (SPA) BOUNDARY (JUNE 2020)
- SPECIAL AREA OF CONSERVATION (SAC) BOUNDARY (JUNE 2020)
- CATCHMENT BOUNDARY
- DOME PERIMETER CHANNEL
- SPILLWAY LOCATION (4m, 6m & 8m WIDTHS)
- PROPOSED DISCHARGE ROUTE AT CLOSURE
- WETLANDS IN PIC - DISCHARGE AT PIC BREACH SPILLWAY #1
- WETLANDS IN PIC - DISCHARGE AT PIC BREACH SPILLWAY #2

CLIENT
AUGHINISH ALUMINA LTD.

CONSULTANT
GOLDER
MEMBER OF WSP

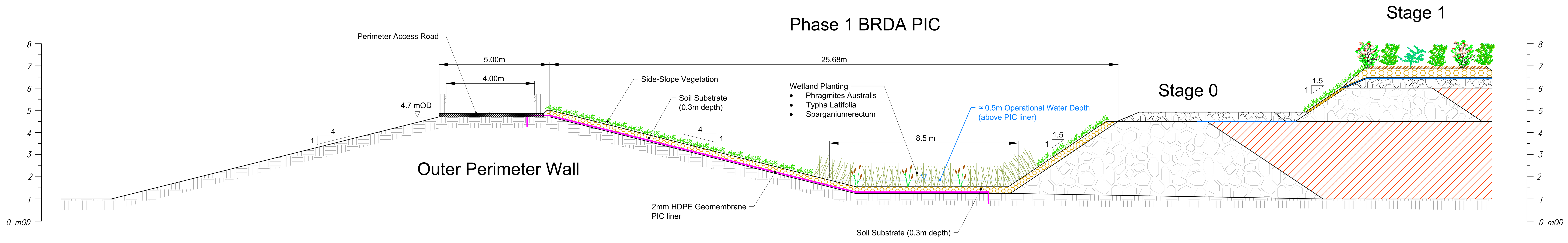
YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	GJ

PROJECT
BRDA RAISE DESIGN

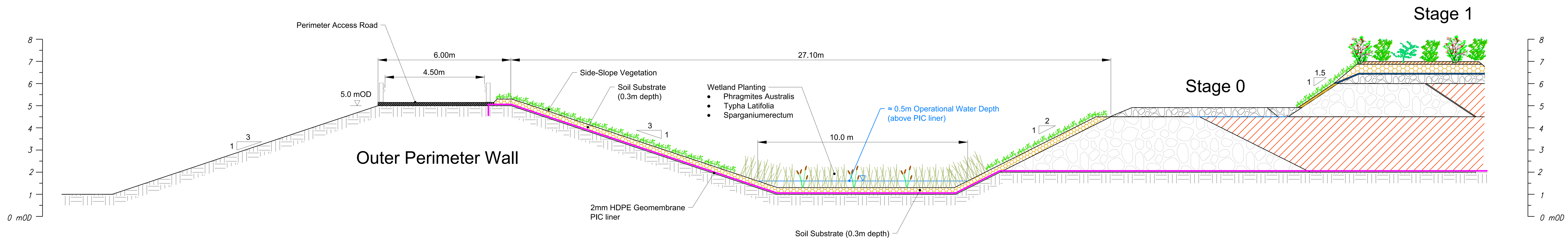
TITLE
WETLANDS AND DISCHARGE ROUTE AT CLOSURE

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	10	A	1:4,000 A1

25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1



TYPICAL SECTION FOR WETLANDS IN PHASE 1 BRDA PIC (PIC-G)



TYPICAL SECTION FOR WETLANDS IN PHASE 2 BRDA PIC (PIC-D)

0 4 8 12m
1:100

CLIENT
AUGHINISH ALUMINA LTD.

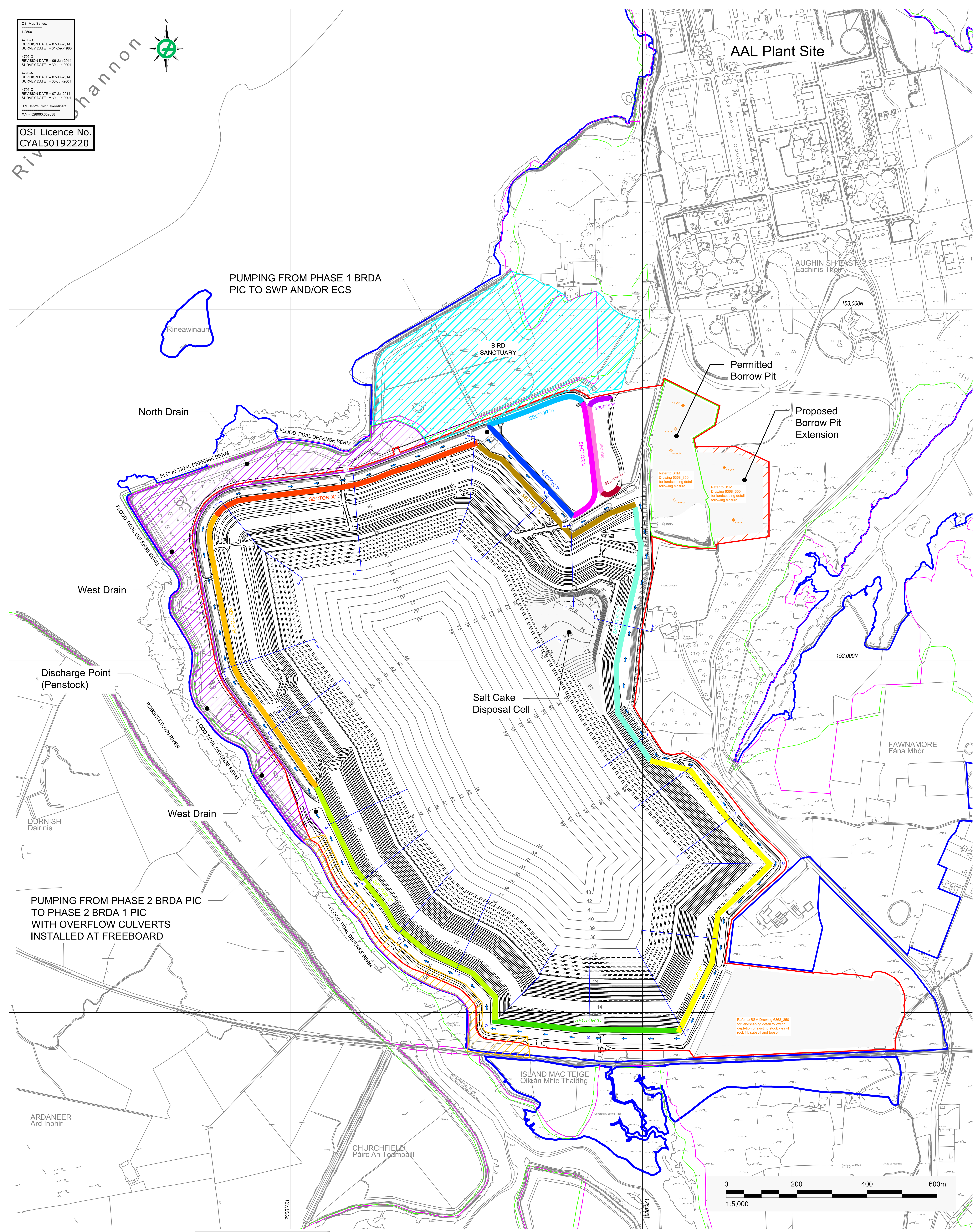
CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

PROJECT
BRDA RAISE DESIGN

TITLE
WETLANDS IN PIC: TYPICAL SECTIONS

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	11	A	1:100 A1



LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 OGL CONTOUR (mOD)
- PERMITTED BORROW PIT BOUNDARY
- PROPOSED BORROW PIT EXTENSION BOUNDARY
- SPECIAL PROTECTION AREA (SPA) BOUNDARY (JUNE 2020)
- SPECIAL AREA OF CONSERVATION (SAC) BOUNDARY (JUNE 2020)
- BIRD SANCTUARY FOOTPRINT WITHIN SAC AND SPA
- BREACH / FLOOD PLAIN TO THE NORTH-WEST AND WEST OF PHASE 1 BRDA
- BREACH / FLOOD PLAIN TO THE WEST OF PHASE 2 BRDA

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

PROJECT

BRDA RAISE DESIGN

CONSULTANT

GOLDER MEMBER OF WSP

YYYY-MM-DD	2021-Dec-01
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	BK

TITLE

STABILITY SECTIONS AND SECTORS FOR BREACH ANALYSIS

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	12	A	1:5,000 A1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

0 25 mm

APPENDIX C

Seismic Liquefaction Assessment

EXECUTIVE SUMMARY

An initial screening methodology was undertaken to assess the estuarine soils and the bauxite residue (farmed and unfarmed) for susceptibility to seismic liquefaction. The estuarine deposits were determined to be not susceptible, and the bauxite residue was determined to be in the range of moderate susceptibility, thus requiring further analyses.

The liquefaction analyses for the bauxite residue for the design earthquake (1 in 2,475-year return period, Magnitude 5.0 with an epicentre within 1km of the BRDA) meets the required Factor of Safety (FoS) against triggering liquefaction of > 1.0.

Based on the probabilities interpreted from the calculated factors of safety defined by Juang et. al. (2001):

- The unfarmed bauxite residue is generally in the 'Highly Improbable' to 'Almost Impossible or Negligible' range to liquefy during the design earthquake (1 in 2,475-year return period).
- The farmed bauxite residue is wholly in the 'Almost Impossible or Negligible' potential to liquefy during the design earthquake (1 in 2,475-year return period).

Sensitivity assessments concluded that an earthquake with a 1 in 7,000-year return period with an epicentre within 1k of the BRDA resulting in a peak ground acceleration (PGA) of 0.08g would be required to return a FoS < 1.0 for the unfarmed bauxite residue (a FoS value of 0.98 was returned).

However, this PGA value would require a larger earthquake than a Magnitude 5.0. The HSE document, Seismic Hazard: UK Continental Shelf (HSE 2002) provides contour maps for UK and Ireland and a zonation model which lists the south-west coast of Ireland (zone A13) as an area with an earthquake magnitude observation threshold of 5.0.

1.0 INTRODUCTION

The seismic liquefaction potential of the bauxite residue and underlying estuarine soils have been assessed and has been undertaken according to the following procedure:

- Initial screening assessment of liquefaction susceptibility to determine if further analyses is required. This initial screening assessment is based on the in-situ material properties.
- If determined to be susceptible to liquefaction, a seismic liquefaction assessment was undertaken based on the design earthquake event to determine the **cyclic stress ratios (CSRs**) and using two methods to determine the **cyclic resistance ratios (CRRs):**
  - National Center for Earthquake Engineering Research (NCEER) method (Youd et. al. 2001); and
  - State Parameter Approach (Jefferies and Been 2016).

2.0 INITIAL LIQUEFACTION SCREENING

A screening method for the liquefaction susceptibility of soils based on moisture content (MC), Liquid Limit (LL) and Plasticity Index (PI) was developed (Bray and Sancio 2006).

The screening method classifies material as either susceptible to liquefaction, where the PI is less than 12 (water content %) and the MC/LL ratio is greater than 0.85. Moderately susceptible is defined as material like clayey silts and silty clays of moderate plasticity (12 < PI < 18) at MC/LL > 0.8 which can undergo liquefaction when shaken intensely for a significant number of cycles of loading.

The atterberg limit results from the samples of the estuarine deposit collected during the 2018 investigation are plotted in Figure 1 and these results fall outside of the range of liquefaction susceptibility, due to the relatively high PI. These test values combined with the relatively low seismicity in Ireland, would direct that the estuarine deposits are regarded as not susceptible to seismic liquefaction.

The farmed and unfarmed bauxite residue test values generally plot in the range of moderate susceptibility. Further analyses are therefore required to establish liquefaction potential.

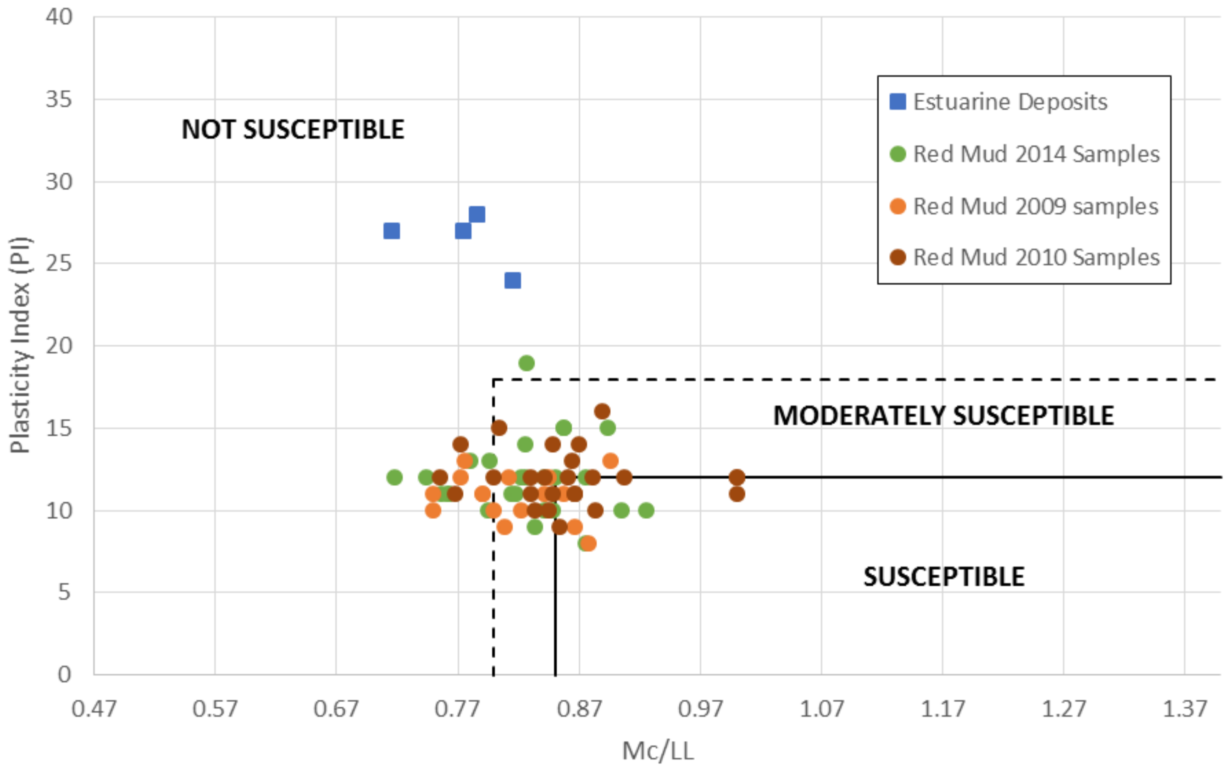


Figure 1: Liquefaction Screening Assessment for Estuarine Deposit and Bauxite Residue (Bray and Sancio 2006)

3.0 BAUXITE RESIDUE LIQUEFACTION ASSESSMENT

The liquefaction assessment was undertaken using the method prescribed by the NCEER (Youd et. al. 2001) and incorporating the State Parameter approach (Jefferies and Been 2006). This is the approach also recommended in ICOLD Bulletin 139 (ICOLD 2011) for evaluation of liquefaction potential (Figure 2).

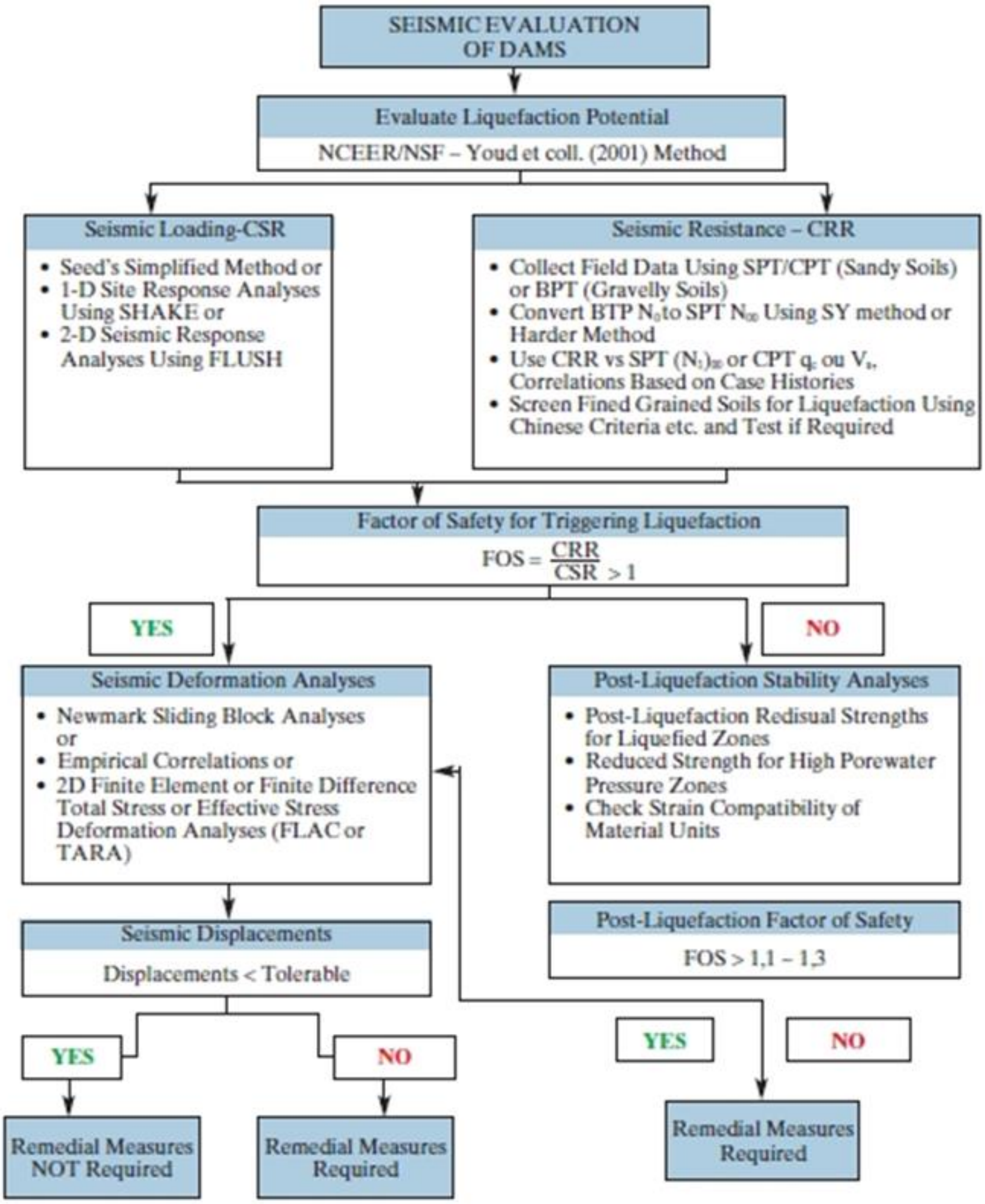


Figure 2: Seismic Evaluation Flowchart. Source: ICOLD Bulletin 139

In the liquefaction triggering analysis, the earthquake induced cyclic stress ratios (CSRs) was compared to the cyclic resistance ratios (CRRs) to determine whether or not the bauxite residue will liquefy under the design earthquake loading.

The seismicity of the area is required to determine the CSR. The BRDA would need to be designed to withstand an earthquake with a return period earthquake of 1 in 2,475-year based on the consequence classification of high (CDA 2014). This results in a design earthquake of Magnitude 5.0 with an epicentre within 1km of the BRDA resulting in a peak ground acceleration (PGA) of 0.05g (Golder 2019).

3.1 Cyclic Stress Ratio

The cyclic stress ratio (CSR) induced by the target magnitude earthquake was determined based on the Seed simplified approach (Seed and Idriss 1971). A 1-D and 2-D site response (shake) analyses using the finite element modelling software Quake/W was undertaken on representative sections to confirm the interpreted CSR from the Seed simplified approach.

Shake analyses is recommended as the preferred method to calculate CSR (Youd et. al. 2001).

3.1.1 Seed Simplified Approach (Seed and Idriss 1971)

The CSR can be calculated from the following formula:

$$CSR = (\tau_{av}/\sigma'_{vo}) = 0.65\ (a_{max}/g)\ (\sigma_{vo}/\ \sigma'_{vo})\ r_d$$

Where:

- a_{max} is the peak horizontal acceleration at the ground surface generated by the earthquake;
- σ_{vo} and σ'_{vo} are the total and effective vertical overburden stress; and
- r_d is the stress reduction coefficient which accounts for flexibility of the soil profile and was estimated by the procedure in Boulanger and Idriss, 2014.

The peak ground acceleration (PGA) determined from the assessment of the seismicity of the area is typically measured along the bedrock surface and requires amplification to be representative of the peak horizontal acceleration (PHA). An amplification factor of 2 has been selected based on comparison with the one and two dimensional shake analyses.

3.1.2 Shake Analyses

A 1-D and 2-D shake analyses was conducted using the software Quake/W (GeoSlope 2018).

An equivalent linear analyses was undertaken to determine the cyclic shear stress and the resulting CSR.

3.1.2.1 Input Parameters

The input parameters include equivalent earthquake loadings to the target magnitude earthquake and material parameters.

Earthquake loadings

The records of three earthquakes have been used to simulate a potential earthquake event. The earthquakes records were scaled to provide a peak acceleration of 0.05g. The following three earthquake records were used:

- Records from **Treasure Island and EL Centro in California**. The earthquakes magnitudes exceeded 6.0 and the time history was scaled accordingly and adjusted for a duration of 10 seconds, which is typical for a magnitude 5.0 earthquake (Chang and Krinitzsky 1977).
- Record from **Saguenay in Quebec, Canada** during the 1988 earthquake of magnitude 5.9. Due to the similarity with the target earthquake magnitude, no adjustment was made to the earthquake duration.

Plots for the earthquake records are shown in Figure 3.

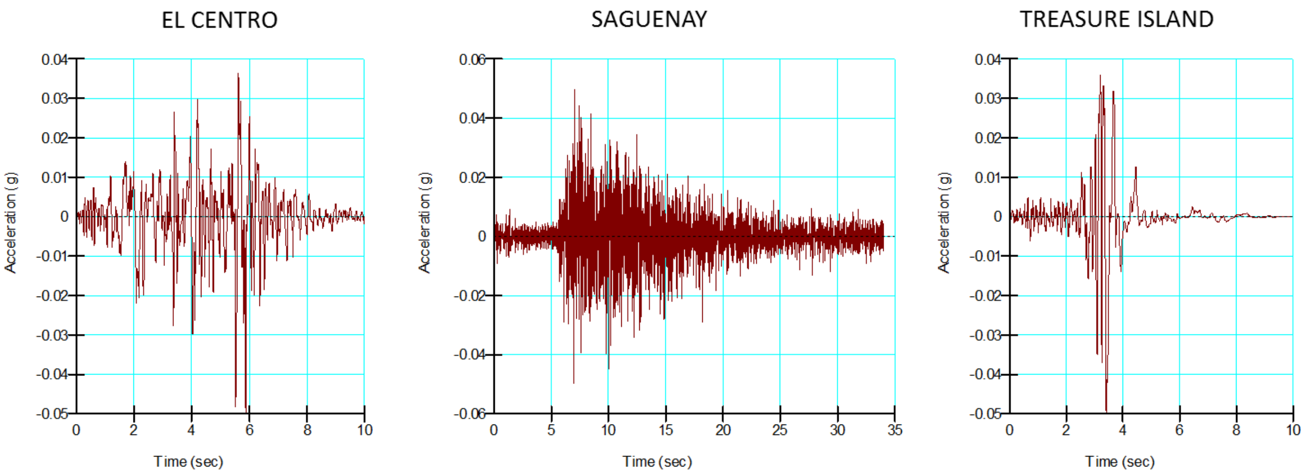


Figure 3: Earthquake Records used in Shake Analyses

Material Parameters

The material parameters which were required as input into the shake modelling are shown in Table 1. These material parameters include:

- Standard properties of bulk density, Poison's ratio and frictional strength parameters.
- Small Strain Shear Modulus (G_{max}): Typically a function of the stress state of the soil and generally increases with confining stress. A function was developed for the bauxite residue based on the results of the seismic CPTu data (Golder 2018) and Bender Element testing (Golder 2005). A constant value was used for the other materials as these are less critical.
- Small Strain Shear Modulus Reduction Function (G-reduction): Soils subjected to dynamic stresses tend to soften in response to cyclic shear strain, and this softening is typically described as a ratio to G_{max}.
- Cyclic Number function which is the relationship between CSR and the number of cycles required to produce liquefaction (N_L).
- Pore water pressure (PWP) function: Can be expressed as a ratio of the pore water pressure and static confining stress. The PWP pressure ratio can be expressed as a function of the equivalent number of uniform cycles (N), for a particular earthquake, and the number of cycles (N_L) which will cause liquefaction.
- Damping Ratio: Energy dissipation of the cyclic shear wave, which typically increases with increasing cyclic shear strain.

The Small Strain Shear Modulus (G_{max}) and the Damping Ratio are the parameters which will most effect the analyses. The bauxite residue parameters were interpreted from the Cyclic DSS testing, Seismic CPT (Golder 2018) and Bender Element testing in triaxial cell (Golder 2005).

Standard properties were used to estimate the material parameters for the estuarine, rockfill and rock due to the lack of specified testing. While the estuarine soils have been screened not to be susceptible to liquefaction, the material parameters are required for the ground profile for the Shake analyses.

Table 1: Material Parameters for Shake Analyses

Parameter	Bauxite Residue	Estuarine Deposit	Rock fill and Rock
Density	21.5 kN/m^3	19 kN/m^3	22 kN/m^3
Poisson's ratio	0.45	0.334	0.495
Drained strength Friction angle (Ø) and cohesion (c)	Ø = 32° c = 0 kPa	Ø = 30° c = 0 kPa	Ø = 36° c = 0 kPa
G-reduction function (See Figure)	From Cyclic DSS Testing and Ishibashi and Zang (1993) relationship	Typical for Sand [1]	Typical for Sand
PWP function (See Figure)	From Cyclic DSS Testing	Typical for Sand [1]	None
Cyclic Number function (See Figure)	Function developed from Cyclic DSS Testing	Typical for Sand [1]	None
Damping Ratio (See Figure)	From Cyclic DSS Testing and Ishibashi and Zang (1993) relationship	Typical for Sand [1]	Typical for Sand
G_{max}	Function with Effective stress Refer to Golder (2018)	150 MPa	500 MPa

Notes:

1. *A reduction function typical of a sand was selected for the estuarine deposit is also plotted. The reduction function typical of a clay was assessed for use and was found to be less conservative than that of sand, hence the more conservative approach (sand) was modelled.*

The Small Strain Shear Modulus Reduction Function (G-reduction) was interpreted based on the results of the cyclic DSS testing (blue line in Figure 4), and a relationship developed by Ishibashi and Zang (1993), which uses the plasticity Index (PI) and confining stress to estimate the reduction (black dashed line in Figure 4). The response of the bauxite residue is typical of a soil with a PI of 28.

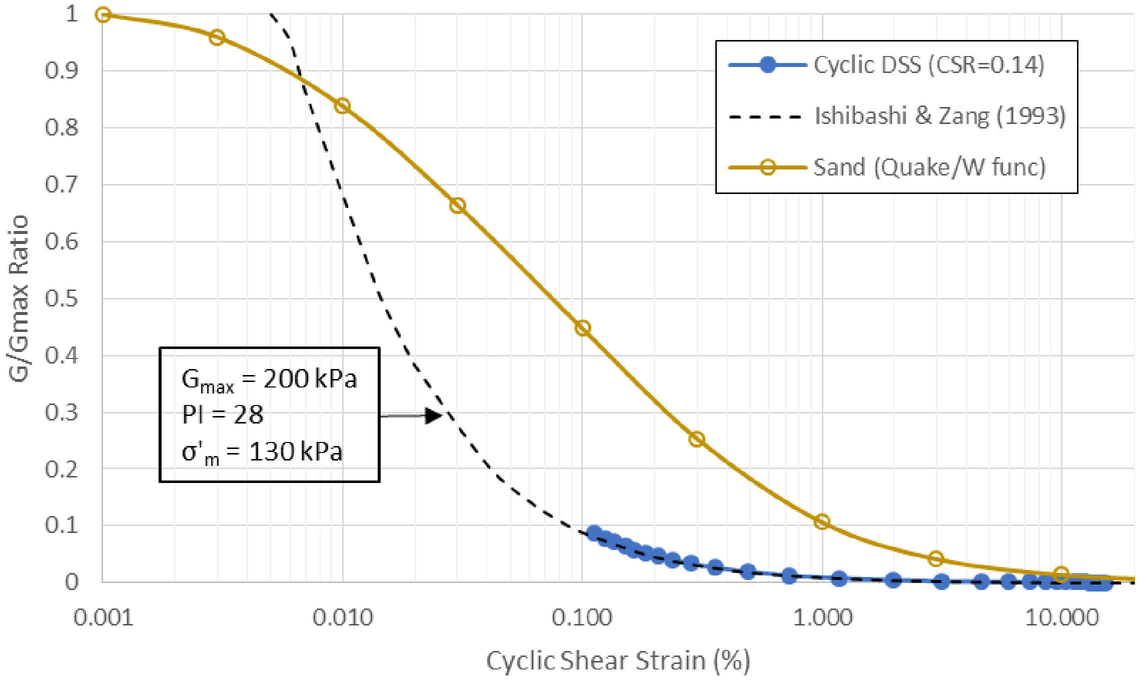


Figure 4: Bauxite Residue G/G_{max} ratio from Cyclic DSS Testing

The PWP ratio function for the bauxite residue was developed from the cyclic DSS testing. The function was chosen for the cyclic DSS testing conducted at a CSR of 0.14 (Figure 5). A typical function for sand is shown for comparison. The bauxite residue was shown to build up a greater pore pressure with generally less equivalent uniform cycles.

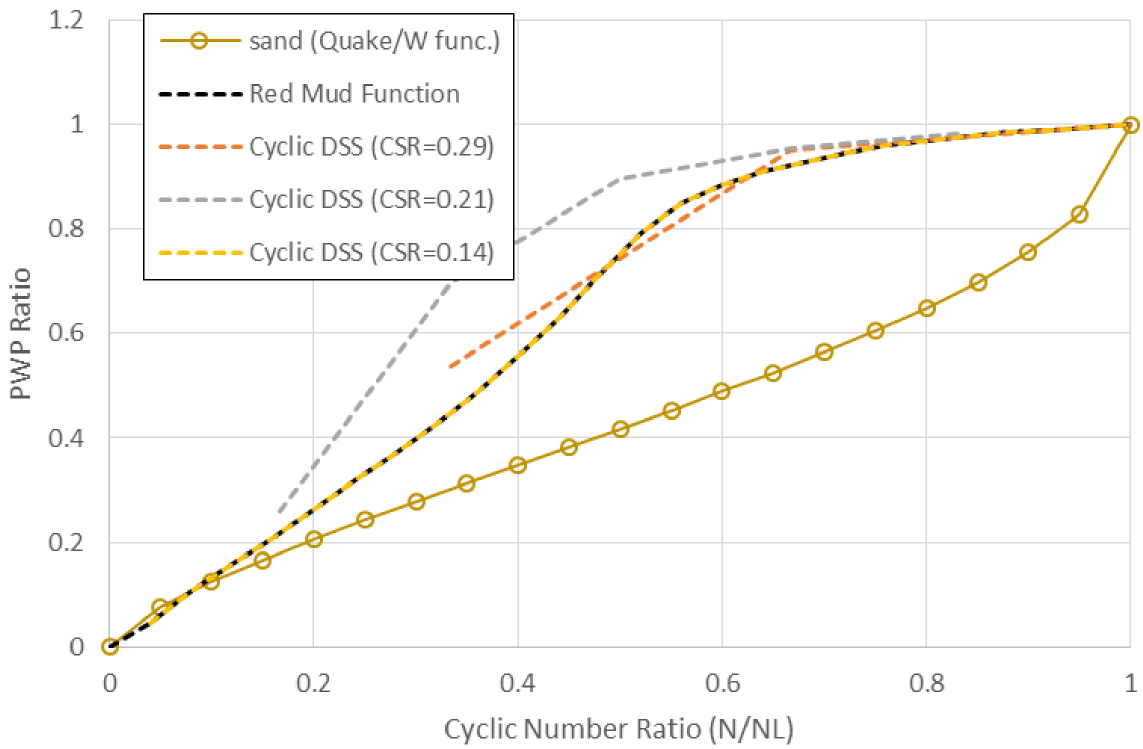


Figure 5: Bauxite Residue Pore Water Pressure Function from Cyclic DSS Testing

The cyclic number function was interpreted from the cyclic DSS testing based on approximately 13% strain required to induce liquefaction (Figure 6). A typical function for sand, chosen for the estuarine soils, shows that a greater CSR is typically required to induce liquefaction. However, the bauxite residue CSR is comparable with other copper and gold tailings (Wijewickreme et al. 2005).

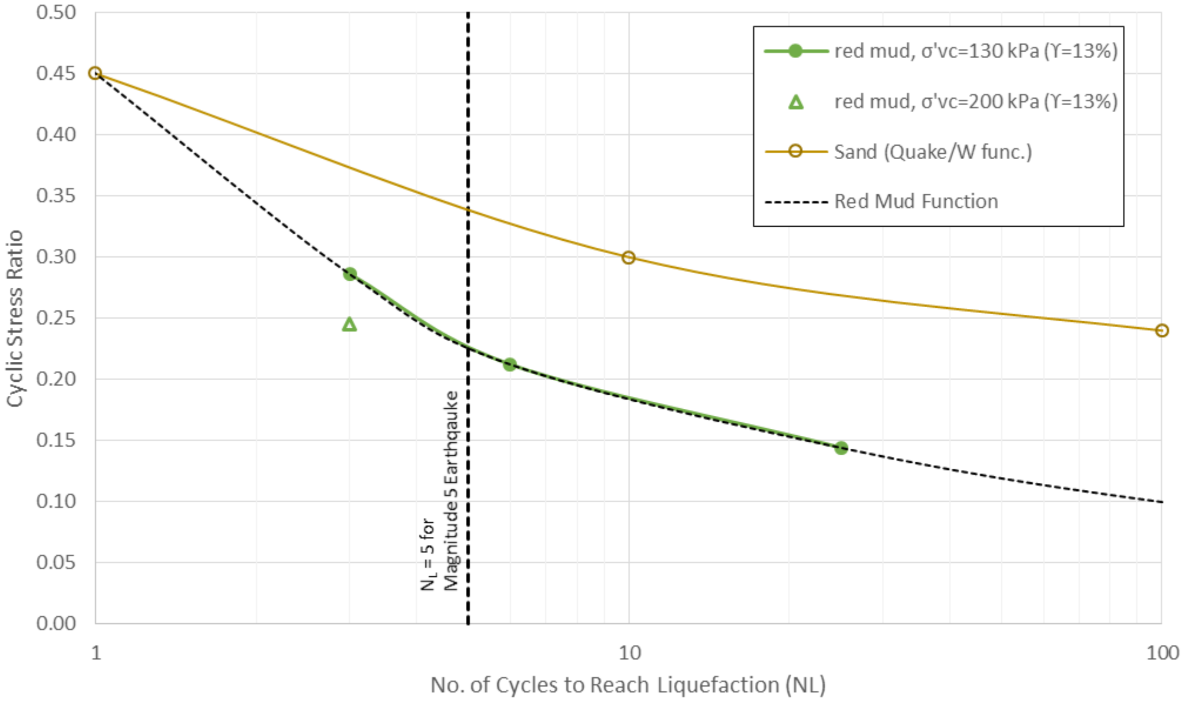


Figure 6: Bauxite Residue Cyclic Number Function from Cyclic DSS Testing

A bauxite residue damping ratio function was developed based on the small strain shear modulus reduction function and a relationship developed by Ishibashi and Zang (1993). The relationship is comparable to the cyclic DSS testing (Figure 7) and is less than a typical function for sand but typical for a material with a PI of 15 to 28.

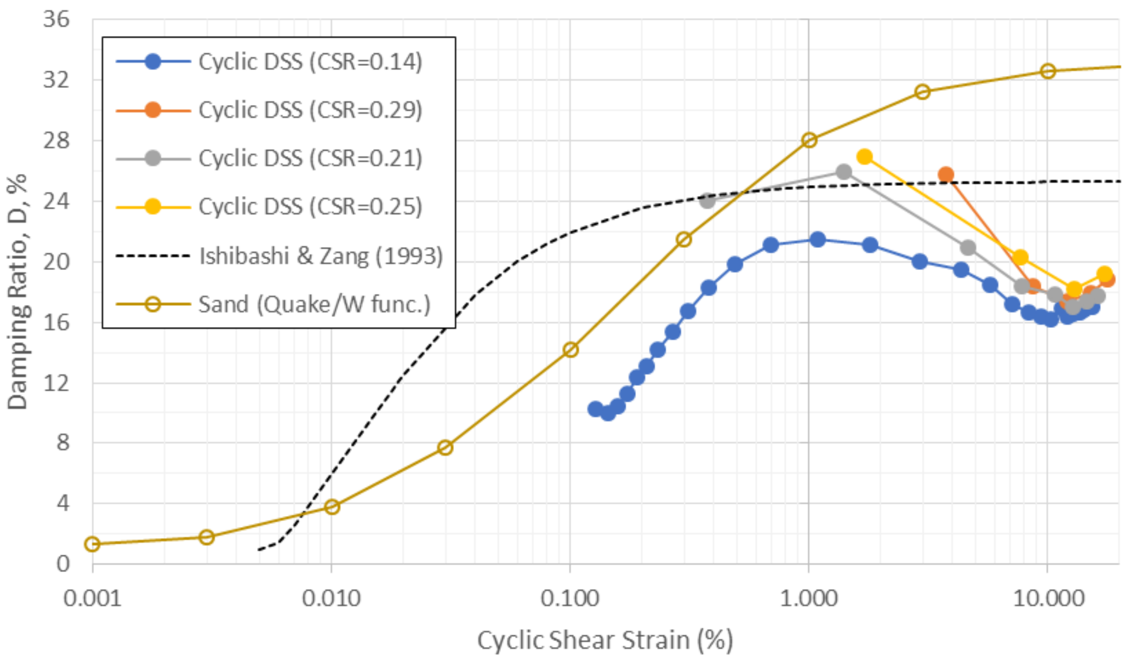


Figure 7: Bauxite Residue Damping Ratio Function from Cyclic DSS Testing

3.1.2.2 Analysis

1-D shake analyses were undertaken at the following 2018 CPTu locations:

- GA18-10C and GA18-10D, north of the salt cake disposal cell (stability Section K-K). This profile was selected as this was the location where the seismic CPTu were conducted.
- GA18-2B, at the north-east corner of the Phase 1 BRDA (stability Section B-B). This profile was selected as it is one of the critical sections where the depth of the foundation estuarine soils is the deepest.

The 1-D Quake/W models for these profiles are shown in Figure 8.

- Orange = farmed bauxite residue
- Pink = unfarmed bauxite residue
- Purple = estuarine deposit
- Grey = rock

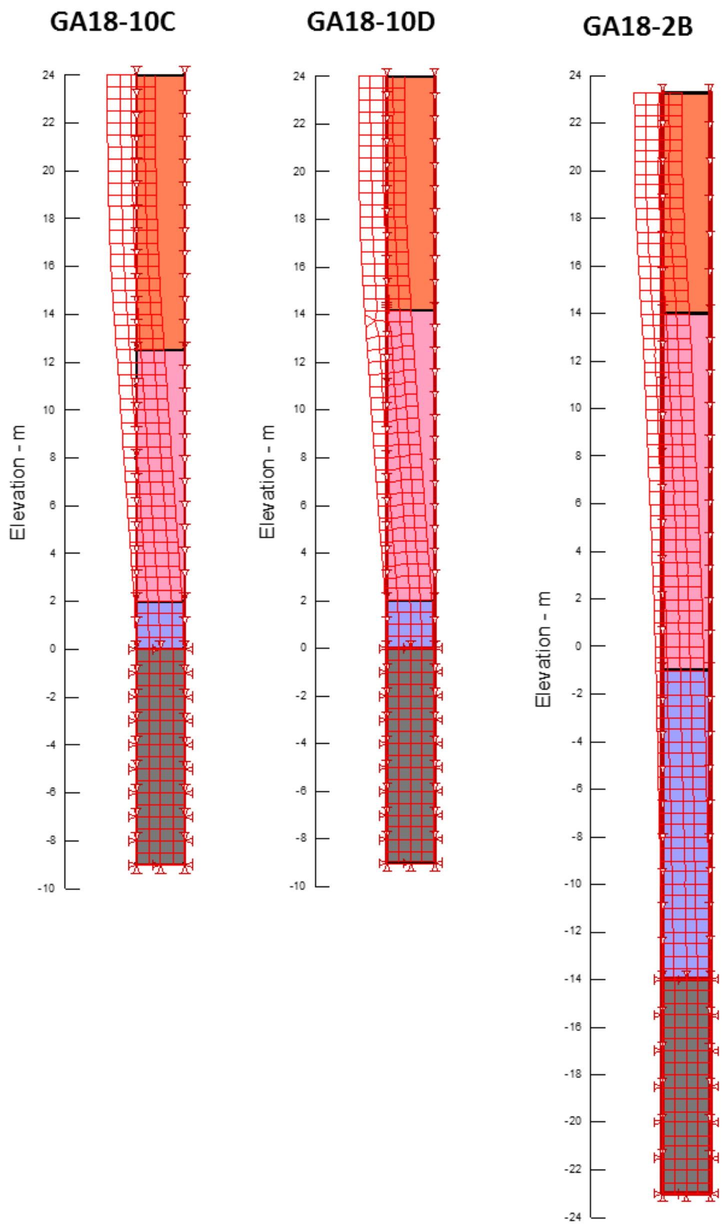


Figure 8: 1-D Shake Analysis Models

2-D shake analyses were conducted at the customary Section B (through CPT GA18-2A and 2B at the north-east corner of the BRDA) to allow comparison of the 1D and 2D shake analyses.

Section B-B was selected as this is a location where the foundation estuarine soils are generally the deepest. A cross-section of the 2D shake analyses through stability Section B-B is shown in Figure 9.

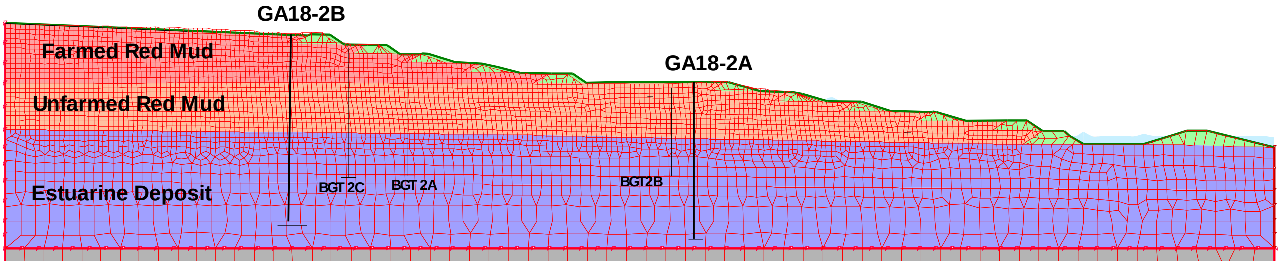


Figure 9: 2-D Shake Model in Quake/W for Section B-B

The CSR was calculated from the shake analyses based on the following equation:

CSR = q_d / 2$\sigma'_{v(static)}$, where q_d is the cyclic deviatoric stress and $\sigma'_{v(static)}$ is the initial effective vertical stress.

3.1.3 CSR Results

The results of the Seed Simplified approach were compared to the results of the Shake analyses and are shown in Figure 10. The CSR varies with depth (confining pressure), with the stress induced by the earthquake greater near the surface where the confining stress is less. This reduction is evident in all the plots.

The results from the shake analyses for all three earthquake histories are plotted at each CPTu profile analysed.

- The Treasure Island earthquake history produces the highest CSR values. This earthquake history was chosen for the 2D analyses as representing an upper bound value. The 2D analyses is comparable to the 1D analyses where both analyses were undertaken for the same profile i.e., GA18-2B for the Treasure Island earthquake.

  The Seed simplified approach generally matches that of the shake analyses for the Treasure Island earthquake record, if an amplitude factor of 2 is used. The Seed Simplified value is greater than the 1D shake analysis results at GA18-10C and GA18-10D, particularly at shallower depths. At GA18-2B, the Seed Simplified value is comparable to the Treasure Island earthquake at greater depths (greater than 10 m) but less at shallower depths.

- The Saguenay earthquake, which has a magnitude more closely matching the design earthquake, may be more representative of the stresses which can be expected. The Saguenay Earthquake results in lower CSR value than the Treasure Island earthquake and the Seed Simplified values in all instances.

- The El Centro earthquake history returns the lowest CSR values of the three (3) earthquakes for GA18-10C and GA18-10D and is an excellent match for the Seed Simplified values for GA18-2B.

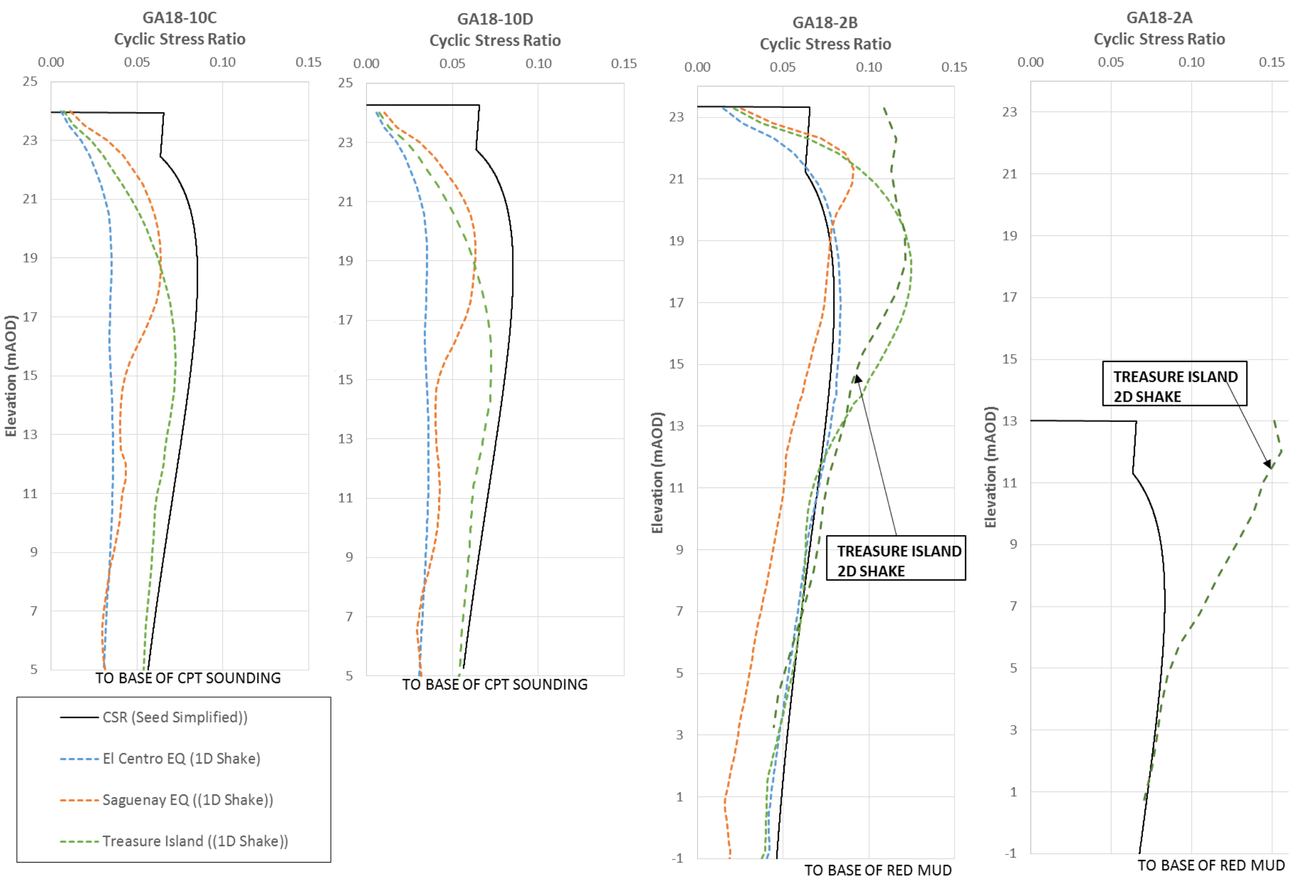


Figure 10: Cyclic Stress Ratio (CSR) Results

3.2 Cyclic Resistance Ratio

The CRR is the capacity of the soil to resist liquefaction. The cyclic resistance ratio (CRR) determined based on two approaches:

- **The NCCER method**, which calculates the CRR from the tip resistance from the cone, and applies adjustment factors based on the fines content, earthquake magnitude, in-situ stress level, and sloping ground.
- **State Parameter Approach** which is based on the correlation between state parameter and CSR as determined from the cyclic DSS testing (Golder 2018).

3.2.1 NCEER Method

For the NCEER approach (summarized in Youd et al, 2001) the CRR is computed from:

$$\mathbf{CRR = CRR_{7.5}\, K_M\, K_\sigma\, K_\alpha} \qquad \text{where:}$$

- $CRR_{7.5}$ is the cyclic strength interpreted for a “standard” Magnitude 7.5 earthquake ground motion (number of “significant” cycles). The $CRR_{7.5}$ is calculated from the normalised CPT tip resistance, with corrections made for fines content. The fines content correction was based on the soil behaviour type calculated from the CPT data.
- Magnitude Scaling Factor (K_M) is a correction factor to the earthquake magnitude being considered. K_M reflects that larger magnitude earthquakes extend over longer periods, for example about 10 seconds of shaking at Magnitude 5 compared to 30 seconds or more at Magnitude 7, and that soil strength depends

on the number of cycles imposed (much like other fatigue phenomena). The average K_M factor used is as recommended in Boulanger and Idriss (2014).

- Overburden correction factor K_σ is a correction factor from the reference 100 kPa stress level to that in-situ.
- Shear stress correction function K_α is a correction factor for initial static shear stresses and state of the soils. This factor has been set at 1 for the NCEER method due to the uncertainty in the correction. The correction should be incorporated in 2D Shake analyses which calculates the initial static shear stress, and the state parameter approach which considers the state of the soil.

3.2.2 State Parameter Approach

The laboratory testing conducted as part of the 2018 Investigation program included triaxial testing to confirm the critical state locus, and cyclic DSS testing to determine the CSR relationship with cyclic loading (Golder 2018). A relationship of CSR with state parameter (Ψ) was inferred from the testing and is plotted in Figure 11.

The state parameter (Ψ) was interpreted from the CPTu data using the relationship established in the 2018 Investigation report (Golder 2018). The variation in CRR with depth can, therefore, be interpreted at each CPTu location. The CRR has been set at a minimum of 0.05 as a reasonable lower bound.

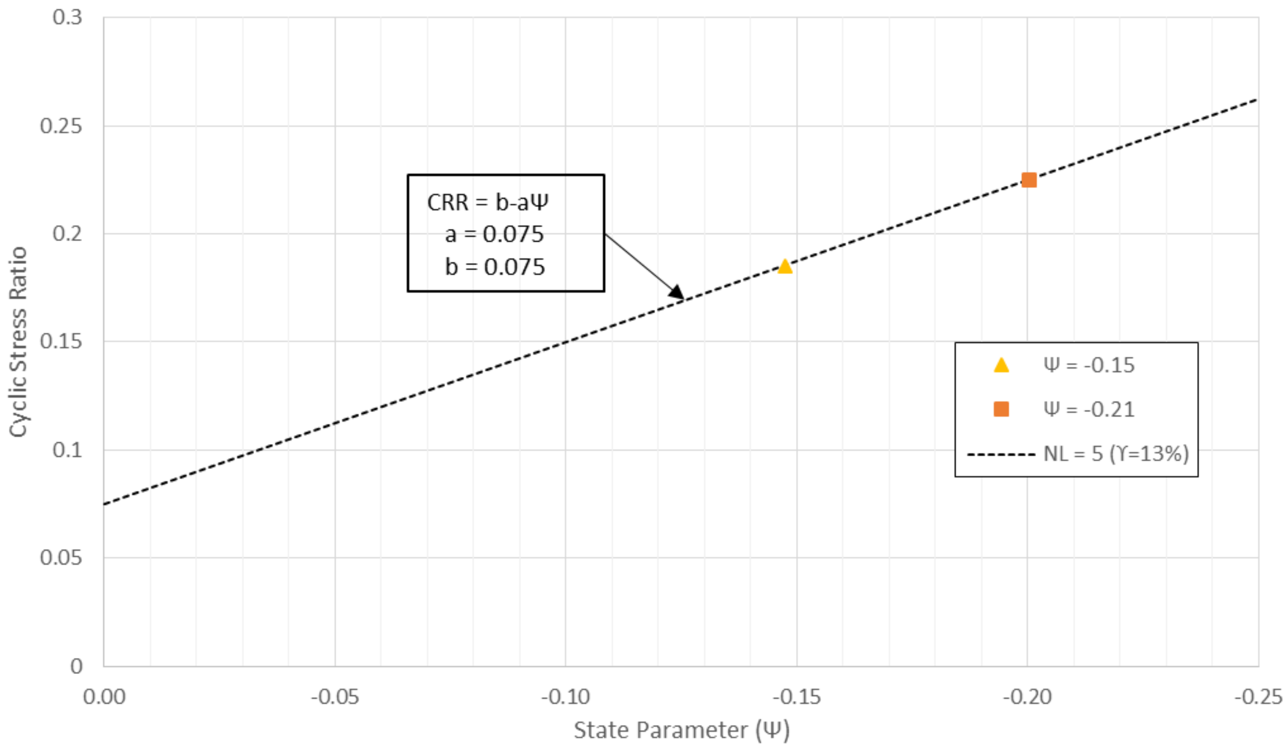


Figure 11: Relationship of State Parameter with Cyclic Stress Ratio (CSR) from cyclic DSS Testing

3.2.3 CRR Results

The results of the CRR analyses using the NCEER Method and State Parameter Approach are plotted in Figure 12. The interpretation of CRR from the state parameter approach is generally higher for the farmed bauxite residue and lower for the unfarmed bauxite residue.

The CRR for GA18-10D, where the CPTu data shows the unfarmed bauxite residue to be softer than typically encountered elsewhere for a localized area, is significantly lower, and is reflective of the soft nature.

The CRR from the state parameter approach is considered more representative as it is incorporated laboratory cyclic testing and the state of the bauxite residue.

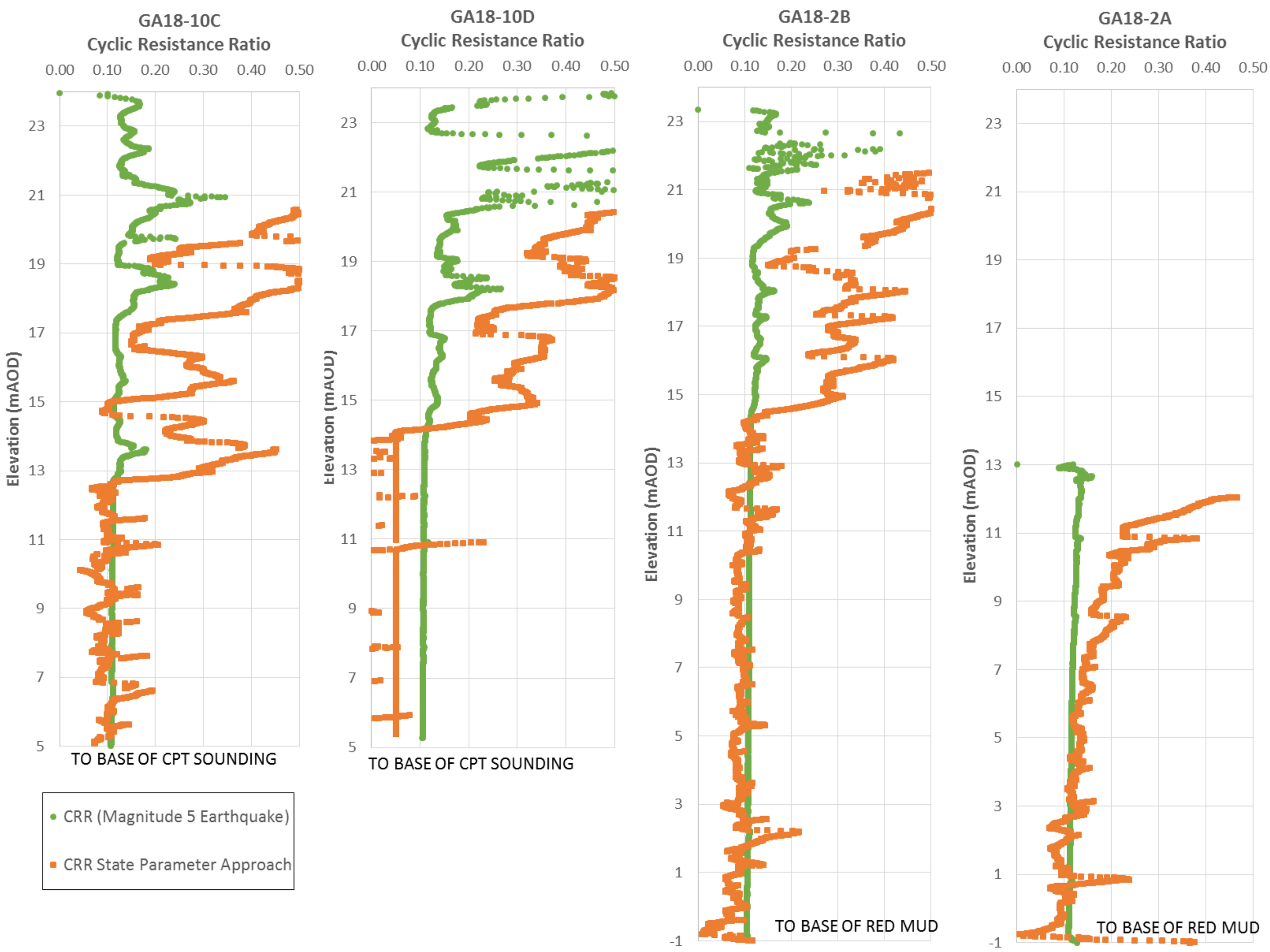


Figure 12: Cyclic Resistance Ratio (CRR) Results

3.3 Factor of Safety Against Liquefaction

The liquefaction potential is the ratio of imposed cyclic stresses in an earthquake (CSR) to the available cyclic strength (CRR), with a comparison of each plotted in Figure 13. The factor of safety against liquefaction is the ratio of CRR over CSR. Where the CRR is greater than CSR, the factor of safety against liquefaction is greater than unity. A Factor of Safety of greater than unity (1.0) is required (ICOLD 2011).

The CSR determined using the Seed Simplified approach has been used for the initial calculation of the factor of safety as it generally represents an upper bound value based on comparison with the Shake analysis using the Saguenay Earthquake record. The Saguenay Earthquake had a similar magnitude as the design earthquake for the Aughinish BRDA.

The CRR is greater than the Seed Simplified calculated CSR for all earthquakes assessed, except at GA18-10D (for the Treasure Island and the El Centro earthquakes) where the bauxite residue has been identified as being soft for a localized area. The CRR is greater than the CSR determined using the Saguenay Earthquake record, even at GA18-10D. The localized area at GA18-10D was re-tested during the 2019 investigation (Golder 2020). 5 No. CPTu soundings were conducted to determine the extent of the weak unfarmed bauxite residue; 1 no at the previous location, 2no. at 5m offsets east and west and 2 no. at 10 m offsets east and west. The lower bound undrained strength ratio (s_u/σ'_{v0}) = 0.08 returned at GA18-10D-1 was not repeated during the 2019 investigation and the soundings returned lower bound s_u/σ'_{v0} values ranging from 0.15 to 0.18. Further stability analyses were undertaken (Golder 2020) which returned satisfactory FoS values

The CSR using the Treasure Island Earthquake showed higher values at depths above 10m, where the depths of estuarine soils are deeper (GA18-2A and 2B). The CRR values are still shown to be greater indicating that liquefaction will not occur

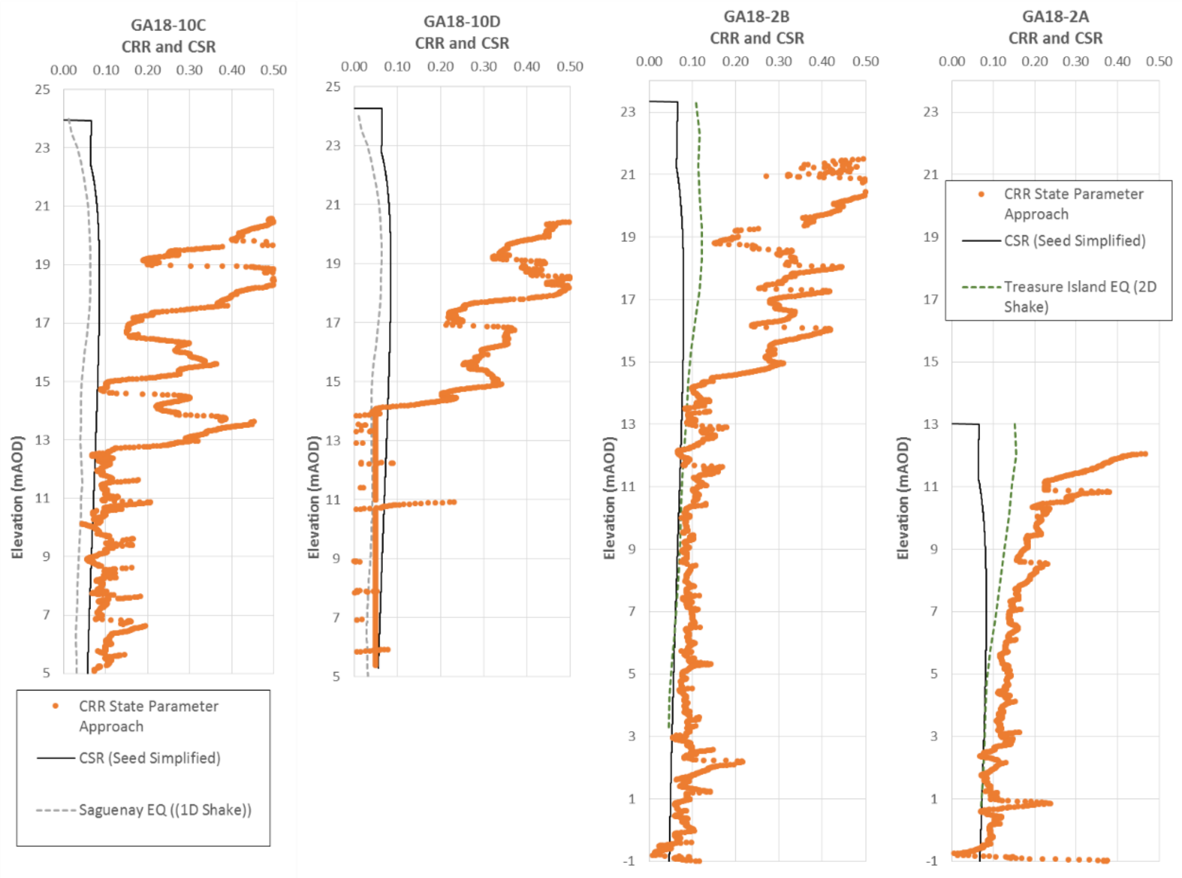


Figure 13: Comparison of Cyclic Resistance Ratio (CRR) to Cyclic Stress Ratio (CSR)

Notes:

1. *Localised thin layers within the CPT soundings assessed show that they have the potential to liquefy, however, it is considered unlikely to impact on the overall stability.*

The average factor of safety against liquefaction or the farmed and unfarmed bauxite residue, for an earthquake with a return period of 1 in 2,475-year, is summarised in Table 2. This relates to a design earthquake of Magnitude 5.0 with an epicentre within 1km of the BRDA resulting in a peak ground acceleration (PGA) of 0.05g.

The probability of liquefaction based on the factor of safety can be determined based on the relationship defined by Juang et. al. (2001).

Table 2: Factor of Safety and Seismic Liquefaction Probability (1 in 2,475-year event, PGA = 0.05g)

CPTu Location	Bauxite Residue Description	Factor of Safety	Probability of Liquefaction		
			Probability	Probability of Occurrence	Description [a]
GA18-10C	Farmed	3.9	< 0.01	$< 4.0 \times 10^{-6}$	Almost Impossible or Negligible
	Unfarmed	1.6	0.10	$< 4.0 \times 10^{-5}$	Highly Improbable
GA18-10D	Farmed	4.7	< 0.01	$< 4.0 \times 10^{-6}$	Almost Impossible or Negligible
	Unfarmed	1.0	0.30	1.2×10^{-4}	Very Unlikely
GA18-2B	Farmed	6.1	< 0.01	$< 4 \times 10^{-6}$	Almost Impossible or Negligible
	Unfarmed	1.6	0.08	3.2×10^{-5}	Highly Improbable
GA18-2A	Unfarmed	2.7	< 0.01	$< 4 \times 10^{-6}$	Almost Impossible or Negligible

Notes:

a) *Factor of safety based on comparison of CRR from State parameter approach and CSR from Seed Simplified, except for GA18-10D where CSR is determined from Shake Analyses using Saguenay Earthquake. See Section 4.3 for text on the re-testing of this area conducted during 2019 (Golder 2020)*

b) *Interpreted Probability Description from Juang et. al. (2001)*

3.5 Sensitivity Assessment

The target FoS against liquefaction is > 1.0 (ICOLD 2011), i.e., CRR is greater the CSR (see Figure 1).

A sensitivity assessment was undertaken to determine the FoS for the BRDA for larger earthquake events (greater magnitude than 5.0) with greater return periods (> 1 in 2,475-year return period) and with an epicentre within 1km of the BRDA.

The CPT sounding at GA18-2B was considered a reasonable estimate of the average bauxite residue properties around the BRDA facility to conduct the assessment. The following FoS values were returned:

- A PGA of 0.07g (1 in 5,000-year return period) results in an average factor of safety of 1.12 for the unfarmed bauxite residue, with very isolated layers showing a potential for liquefaction.
- A PGA of 0.08g (1 in 7,000-year return period) reduces the average factor of safety to 0.98, for the unfarmed bauxite residue, with more defined layers showing a potential for liquefaction.
- A PGA of 0.09g (1 in 10,000-year return period) further reduces the average factor of safety to 0.87, for the unfarmed bauxite residue, with liquefaction possible.

However, these greater PGA values would require a larger earthquake than a Magnitude 5.0. The HSE document, Seismic Hazard: UK Continental Shelf (HSE 2002) provides contour maps for UK and Ireland and a zonation model which lists the south-west coast of Ireland (zone A13) as an area with an earthquake magnitude observation threshold of 5.0.

4.0 CONCLUSIONS

An initial screening methodology was undertaken to assess the estuarine soils and the bauxite residue (farmed and unfarmed) for susceptibility to seismic liquefaction. The estuarine deposits were determined to be not susceptible, and the bauxite residue was determined to be in the range of moderate susceptibility, thus requiring further analyses.

The liquefaction analyses for the bauxite residue for the design earthquake (1 in 2,475-year return period) with an epicentre within 1km of the BRDA meets the minimum required Factor of Safety (FoS) against triggering liquefaction of > 1.0.

Based on the probabilities interpreted from the calculated factors of safety (FoS) defined by Juang et. al. (2001):

- The unfarmed bauxite residue is generally in the 'Highly Improbable' to 'Almost Impossible or Negligible' range to liquefy during the design earthquake (1 in 2,475-year return period).
- The farmed bauxite residue is wholly in the 'Almost Impossible or Negligible' potential to liquefy during the design earthquake (1 in 2,475-year return period).

Note: *The post-seismic condition is assessed in the stability assessment (Engineering Design Report - Appendix D)*

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APPENDIX D

Stability Assessment

1.0 EXECUTIVE SUMMARY

The stability of each sector of the BRDA is analysed for the cases and to the FoS criteria listed in Table 1.

Table 1: Factor of Safety Criteria for the BRDA Raise Development based on International Guidelines

Loading Condition	Recommended Factor of Safety	
	CDA (2014)	ANCOLD (2012, 2019)
Short Term, Undrained (Total Stress)	Greater than 1.3 During, at, or end of Construction, depending on Risk Assessment	1.5 If loss of containment, Consolidated Undrained Strength
Global Slope [1]		
Upper Slope [2]		
Middle Slope [3]		
Lower Slope [4]		
Long Term, Drained, Steady State (Effective Stress)	1.5 Steady State, Phreatic Level	1.5 Effective Strength
Global Slope [1]		
Upper Slope [2]		
Middle Slope [3]		
Lower Slope [4]		
Pseudo-Static, Undrained (Total Stress)	1.0 [5]	Not required
Global Slope [1]		
Post-Earthquake, Undrained (Total Stress)	1.2 [5]	1.0 to 1.2 (residual undrained shear strength)
Global Slope [1]		

Notes:

1. *Global Slope is from the downstream toe of the Outer Perimeter Wall (OPW) of the PIC to Stage 16*
2. *Upper Slope is from Stage 10 to Stage 16*
3. *Middle Slope is from Stage 5 to Stage 10*
4. *Lower Slope is from the downstream toe of the OPW to Stage 16*
5. *Undrained shear strength values were reduced by 20% to allow for cyclic softening (Hynes and Franklin 1984) for pseudo-static and post-earthquake analyses.*

The analysis for undrained (total stress) condition within the bauxite residue is considered the critical case. While in general geotechnical terms and for other more free-draining tailings this is considered the 'short term', for the bauxite residue this represents a 'long term' condition that requires a minimum FoS of 1.5.

- **Phase 1 BRDA -** Stability analyses were conducted on select critical and representative stability Sections for the Phase 1 BRDA constructed to Stage 10, i.e., Section A-A, Section B-B, Section C-C, Section E-E and Section F-F. Stability analyses were conducted for all the stability Sections of the Phase 1 BRDA constructed to Stage 16, i.e., Section A-A, Section B-B, Section C-C, Section D-D, Section E-E, Section F-F, Section K-K and Section L-L. All stability Sections returned FoS in compliance with the target criteria.

- **Phase 2 BRDA -** Stability analyses were conducted on select critical and representative stability Sections for the Phase 2 BRDA constructed to Stage 16, i.e., Section N-N, Section P-P, Section R-R, Section T-T and Section V-V. All stability Sections analysed returned FoS in compliance with the target criteria.

2.0 INTRODUCTION

Golder Associates Ireland Limited (Golder) conducted stability assessments for the BRDA as part of the compilation of the design report for the BRDA Raise Development.

The BRDA has been divided into sectors which have similar foundation conditions, bauxite residue deposition characteristics and side-slope profile. These sectors are named based on their location e.g., North-East sector in the Phase 1 BRDA and vary in width around the perimeter of the BRDA but are typically in the 200m to 350m range. Stability sections lines have been assigned to each sector and monitoring instrumentation is installed along the alignment of the stability section lines on the side-slopes at designated elevation intervals as the BRDA is raised.

The stability sections assessed comprise the following and are shown on Drawing 12 and in Appendix M of the Engineering Design Report.

- **Phase 1 BRDA:** Section A-A, Section B-B, Section C-C, Section D-D, Section E-E, Section F-F, Section K-K and Section L-L.

  Note: *A number of stability sections were previously designated and assessed along the south face of the Phase 1 BRDA (Section G-G, Section H-H, Section I-I and Section J-J). This face is being merged with the Phase 2 BRDA and these stability sections are no longer assessed as the bauxite residue deposition provides a buttress for the slope. The monitoring instruments remaining in this sector are still read on a quarterly basis and the readings are assessed and reported in the quarterly memos and annual review.*

- **Phase 2 BRDA:** Section M-M, Section N-N, Section O-O, Section P-P, Section Q-Q, Section R-R, Section S-S, Section T-T, Section U-U, Section V-V, Section W-W and Section X-X.

  Note: *A number of stability sections in the Phase 2 BRDA have similar foundation conditions and will have bauxite residue deposition characteristics and side-slope profile when constructed, hence these stability Sections have been bundled into groups for analyses.*

This assessment provides a summary of the findings for the critical stability conditions for the BRDA constructed to Stage 10 and to Stage 16 and provides recommendations for extra measures where needed. The methodology utilized for the stability assessment and the selection of the geotechnical parameters is also presented.

3.0 BACKGROUND

The original design for the BRDA to Stage 7 was based on the undrained strength parameters for the unfarmed bauxite residue and the effective strength parameters with increased pore pressure ratio of the underlying estuarine deposits. The short-term undrained condition (total stress) was considered the critical condition and the target factor of safety (FoS) was 1.3 (Golder 2005).

The design of the BRDA Raise to Stage 10 used similar strength parameters for the unfarmed bauxite residue and the underlying estuarine deposits. The short-term undrained condition (total stress) was considered the critical condition and the wide Stage 5 bench was introduced to improve the overall slope gradient to 6.3(H):1(V), in order to the achieve the target FoS of 1.3. The FoS returned was generally between 1.3 and 1.5 for the Phase 1 BRDA, apart from Section F-F in the south-west sector of the Phase 1 BRDA where a former Sludge Disposal Area was located and the FoS ≈ 1.2) and between 1.9 and 2.4 for the Phase 2 BRDA (Golder 2011).

Note: *The Sludge Disposal Area located in the south-west sector of the Phase 1 BRDA (Section F-F) was backfilled with bauxite residue, and wick drains were subsequently installed in the bauxite residue downstream of the Sludge Disposal Area to improve the strength parameters (Golder 2010). The overall slope design was steepened by eliminating the wide Stage 5 in this sector so that Stage raises 6, 7 and 8 were not constructed directly above the footprint of the Sludge Disposal Area and a FoS of 1.3 was attained. A toe buttress has been constructed for this sector to improve the FoS to 1.5*

Tailings dams are complex systems that have evolved over the years and will continue to evolve in the future. Since the design and approval of the Phase 2 BRDA and the overall BRDA Raise to Stage 10 (ABP, February 2007), international best practice guidelines for tailings dams have changed (ANCOLD 2012, 2019 and CDA 2013, 2014). The short-term undrained condition (total stress) is no longer considered appropriate for tailings which have very low permeability; they are now considered a long-term undrained condition (total stress) requiring a FoS of 1.5. As a result, AAL and Golder have implemented the following since the 2007 approval:

- AAL commenced mud-farming activities from 2009 when the Phase 1 BRDA was at ≈ 14 mOD (Stage 7) and prior to the Phase 2 BRDA. This has improved the strength parameters for the bauxite residue, improved the FoS, and has made feasible the raising of the BRDA beyond Stage 10.
- The underlying estuarine soils were previously assessed as a single layer with selected lower bound effective strength parameters and by increasing pore pressure ratio. Since 2018, the estuarine soils have been assessed similarly to the bauxite residue i.e., in an undrained condition (total stress) and as two distinct layers of geotechnical properties:
  - **Sandy Silt Layer** – Generally occurs as the surface layer and some underlying layers. Characterised by an irregular and higher undrained shear strength.
  - **Silty Clay Layer** – Generally occurs underlying the Sandy Silt layers. Characterised by a lower but more uniform undrained shear strength.
- AAL have adopted the CDA Guidelines for the BRDA in 2018 in preparation for the Dam Safety Review (DSR) conducted in 2019, and the BRDA is required to comply with the long-term undrained condition (total stress) requiring a FoS of 1.5.

  Whilst legacy facilities are largely considered to be exempt from compliance with FoS values that are currently greater than when they were permitted / approved when designed and constructed, owners are encouraged to make improvements where possible. AAL have enhanced Section A-A and Section F-F to meet the FoS criteria of 1.5 for the BRDA constructed to Stage 10.

  An extension or a raise to an existing tailings facility is assessed at the higher FoS target and will include the existing BRDA facility and the proposed BRDA Raise Development.

4.0 STABILITY ASSESSMENT METHODOLOGY

The stability analyses for the BRDA Raise Development were carried out using the limit equilibrium modelling software SLOPE-W Version 10.0.0.17401. The analytical method used was Morgenstern and Price method of slices, which satisfies both force and moment equilibrium. The analyses include both drained (effective stress) and undrained (total stress) strength conditions within the bauxite residue and the estuarine deposits

The stability models for each sector are constructed based on the stratigraphy identified by the CPTu profiles. The phreatic surfaces for the stability models constructed to Stage 16 are determined by using the current measured phreatic surfaces for the Phase 1 BRDA to Stage 10, which were then replicated using SEEP-W to assign hydraulic conductivity values for the farmed and unfarmed bauxite residue and subsequently modelled for the BRDA constructed to Stage 16. The phreatic surfaces for the Stage 10 analyses were determined by using the current measured phreatic surfaces.

The stability of each sector of the BRDA is analysed for the cases and to the FoS criteria listed in Table 2 below.

Table 2: Factor of Safety Criteria for the BRDA Raise Development based on International Guidelines

<table>
<tr><th rowspan="2">Loading Condition</th><th colspan="2">Recommended Factor of Safety</th></tr>
<tr><th>CDA (2014)</th><th>ANCOLD (2012, 2019)</th></tr>
<tr><td>Short Term, Undrained (Total Stress)</td><td rowspan="5">Greater than 1.3
During, at, or end of Construction,
depending on
Risk Assessment</td><td rowspan="5">1.5
If loss of containment,
Consolidated
Undrained Strength</td></tr>
<tr><td>Global Slope [1]</td></tr>
<tr><td>Upper Slope [2]</td></tr>
<tr><td>Middle Slope [3]</td></tr>
<tr><td>Lower Slope [4]</td></tr>
<tr><td>Long Term, Drained, Steady State (Effective Stress)</td><td rowspan="5">1.5
Steady State,
Phreatic Level</td><td rowspan="5">1.5
Effective
Strength</td></tr>
<tr><td>Global Slope [1]</td></tr>
<tr><td>Upper Slope [2]</td></tr>
<tr><td>Middle Slope [3]</td></tr>
<tr><td>Lower Slope [4]</td></tr>
<tr><td>Pseudo-Static, Undrained (Total Stress)</td><td rowspan="2">1.0 [5]</td><td rowspan="2">Not required</td></tr>
<tr><td>Global Slope [1]</td></tr>
<tr><td>Post-Earthquake, Undrained (Total Stress)</td><td rowspan="2">1.2 [5]</td><td rowspan="2">1.0 to 1.2
(residual undrained
shear strength)</td></tr>
<tr><td>Global Slope [1]</td></tr>
</table>

Notes:

1. *Global Slope is from the downstream toe of the OPW to Stage 16*
2. *Upper Slope is from Stage 10 to Stage 16*
3. *Middle Slope is from Stage 5 to Stage 10*
4. *Lower Slope is from the downstream toe of the OPW to Stage 16*
5. *Undrained shear strength values were reduced by 20% to allow for cyclic softening (Hynes and Franklin 1984) for pseudo-static and post-earthquake analyses.*

A minimum FoS of 1.5 is considered required for all static long term drained analysis. A reduced factor of safety of 1.3 may be considered acceptable for the short-term undrained condition following embankment construction, provided sufficient understanding of the material strength parameters and their behaviour exists, and an appropriate risk assessment has been undertaken.

The drained (effective stress) condition, which represents the standard 'long term' condition has been included in the current analyses. This condition would represent loading and shearing of the bauxite residue at a slow enough rate to limit the build-up of excess pore pressure, and typically produces a higher FoS and hence is not considered the critical case.

The analysis for undrained (total stress) condition within the bauxite residue is considered the critical case. While in general geotechnical terms and for other more free-draining tailings this is considered the 'short term', for the bauxite residue this represents a 'long term' condition that requires a minimum FoS of 1.5.

This total stress condition is considered the critical case as:

- An undrained condition for a material in a contractive state (unfarmed bauxite residue), generates excess pore pressure and results in a lower effective shear strength less than in the drained condition.
- The undrained condition when the material is in a relatively dense/stiff condition (farmed bauxite residue), dilates during shearing, generates negative pore pressure and may result in an effective shear strength greater than in the drained condition.

For the pseudo-static analysis, the coefficient of horizontal ground acceleration of 0.025 g (50% of PGA) was applied representing the 2,475-year return period earthquake, along with 20% strength reduction of the material strength parameters, as per the recommendations of Hynes-Griffin and Franklin (Hynes and Franklin 1984).

5.0 GEOTECHNICAL PARAMETER SELECTION

Geotechnical strength parameters can have a wide range, have a high likelihood of outliers, and are typically dependent on the selection of other parameters for their interpretation. The interpretation and determination of the geotechnical parameters can have a significant influence on the resulting FoS for a given stability model. The following methodology was adopted for the analyses of the stability sections.

The undrained shear strength ratio (s_u/σ'_{v0}) is a key input for the stability analyses and is determined from the correlation of interpreted CPTu undrained shear strength with laboratory test data (primarily DSS test data). A summary of CPTu interpreted data for each stability section is provided in Appendix D-3.

Mayne 2016 suggests that the DSS test is the most appropriate test to use when correlating the interpreted undrained strength from CPTu data as it presents undrained strength results that fall more-or-less mid-way between the other test modes (compression and extension) and thus provides an 'average' result. The undrained strength, s_u ,depends on the effective confining stress (σ'_{v0}) prior to shearing. The CPTu data is used to interpret the undrained shear strength (s_u) using the undrained strength factor, N_{kt}, and the following relationship : **s_u = (Net cone resistance) / N_{kt}**

- An **N_{kt} value of 14** has been selected for the bauxite residue following correlation with laboratory testing and review of shear vane testing data.
- Previously an N_{kt} value of 15 was used for the estuarine deposits which provided a reasonable estimate of the undrained shear strength ratio profile. The N_{kt} value currently adopted for interpretation of the undrained strength of the estuarine deposits is variable and is based on the normalized excess pore pressure parameter (B_q) which is measured during the CPTu sounding and reflects the permeability of the material it is passing through i.e., higher for clay soils and lower for silty / sandy soils. A trend line developed for Irish Clays based on, $\boldsymbol{B_q}$ (Long 2018).

$$\boldsymbol{N_{kt} = 7.82\, B_q^{-0.65}}$$ **for Irish Clays.**

The B_q value in the silty clay layer varies from 0.3 to 0.45 and returns N_{kt} values between 13.1 and 17.1.

- The stability model in SLOPE-W requires the input of a single undrained strength ratio (s_u/σ'_{v0}) value for each material layer. These single values are termed design or characteristic values.

The characteristic value for the geotechnical strength parameters for use in the deterministic stability calculations is recommended to be selected to provide a high level of confidence that the measured values will be greater than the characteristic value. The confidence % (or equivalent percentile / fractile) of the characteristic value should be combined with the Factor of Safety (FoS) to determine the 99% exceedance probability (Been and Jefferies 2016), e.g., a 70% confidence value (or 30th percentile) combined with a FoS = 1.45 would provide the desired 99% exceedance probability. A range has been selected for the characteristic strength parameters as the value is determined for each stability section and layering within that stability section based on the interpreted CPTu strength, which is validated by laboratory testing of samples taken at the section and layer, where available.

Geotechnical index properties (i.e., dry density, bulk density, moisture content) typically have a narrower range and a lower likelihood of outliers for a particular soil type or tailings stream and the mean value is typically selected for the characteristic value, which is ≈ 50th percentile. Combined with a FoS = 1.45, would provide a 72.5% exceedance probability for measured values.

The characteristic undrained shear strength parameters selected are the 30th percentile for the estuarine deposits and the 10th percentile for the bauxite residue (farmed and unfarmed).

6.0 MODEL CONFIGURATION AND ANALYSES TOOLS

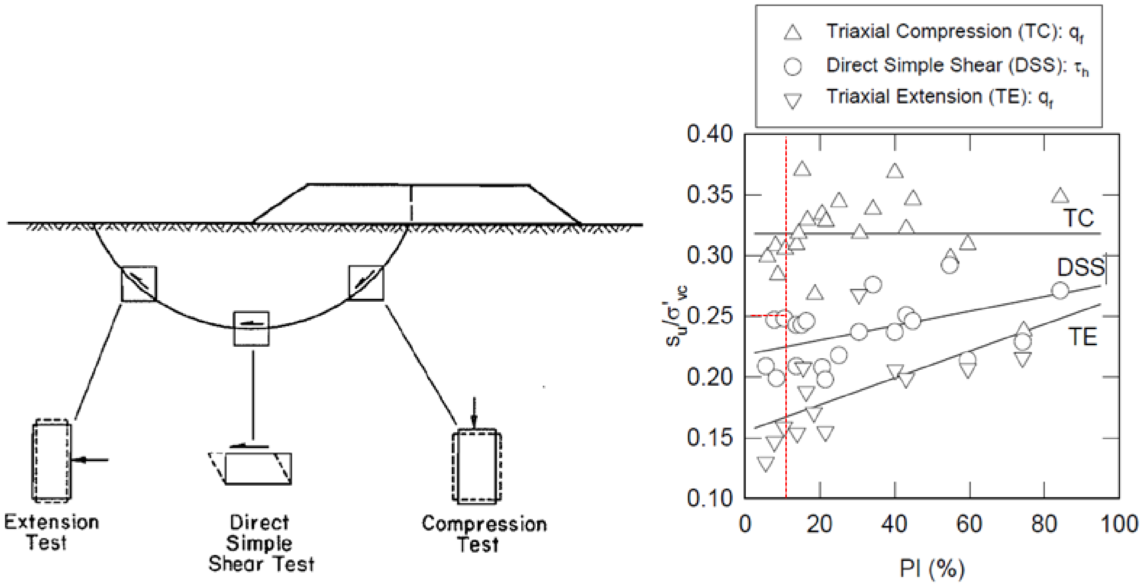
The set-up and execution of the models for stability analyses in SLOPE-W can have a significant impact on the outcome. The following methodology was adopted for the analyses of the stability sections.

- The stratigraphy of the Phase 1 BRDA can be broadly simplified, from bottom to top, as estuarine deposits, unfarmed bauxite residue and farmed bauxite residue. Specific layers can be introduced into the model and assigned distinct material properties, such as the splitting of the estuarine deposit into a Sandy Silt Layer and a Silty Clay Layer.

  Layering that can be clearly identified from the CPTu profiles in both the estuarine deposits and the bauxite residue has been constructed in the models for stability analyses and has been assigned specific parameters.

- Undrained shear strength, s_u, is not a unique soil property. It varies depending on the mode of failure, the stress state of the soil, anisotropic effects and rate of failure. The results of the CIU triaxial and DSS testing indicate that undrained strength anisotropy exists for the bauxite residue.

  Undrained shear strength anisotropy is the variation in undrained strength with direction of shearing. Undrained strength is typically greater in compression, less in horizontal shear, and the least in extension. Triaxial compression and extension tests are recommended for measuring undrained strength in compression and extension, and DSS for measuring horizontal shear, see Figure 1 below.



Source: Bjerrum (1972)   Source: Ladd and DeGroot (2004)

Figure 1: Undrained Shear Strength Anisotropy

Slope-W allows for use of anisotropic strengths for materials which are applied the relevant portions of the slope circle. Hence, the stability analyses benefit from the greater compressive strength of the farmed and unfarmed bauxite residue in the upper portion of the slip circle.

- The elevation of the piezometric line can have a significant influence on the FoS returned for the stability model. The piezometric lines inputted for the stability models are based on measured quarterly monitoring readings for the Phase 1 BRDA constructed to Stage 10, replicated by modelling using SEEP-W and subsequently projected to Stage 16 based on the hydraulic conductivity parameters of the bauxite residue.

7.0 STABILITY ANALYSES FOR PHASE 1 BRDA

This section provides the stability analyses for the Phase 1 BRDA comprising Section A-A, Section B-B, Section C-C, Section D-D, Section E-E, Section F-F, Section K-K and Section L-L.

The geotechnical parameters selected for the estuarine deposits (where present) and the bauxite residue at each stability section have been determined following assessment of the field investigation data comprising in-situ testing, sampling, laboratory testing and interpretation by others prior to 2004 and by Golder after 2004.

The undrained condition (total stress) is considered the critical case required a FoS of 1.5 for the Global, Upper, Middle and Lower Slope conditions as:

- An undrained condition for a material in a contractive state (unfarmed bauxite residue), generates excess pore pressure and results in a lower effective shear strength less than in the drained condition.
- The undrained condition when the material is in a relatively dense/stiff condition (farmed bauxite residue), dilates during shearing, generates negative pore pressure and may result in an effective shear strength greater than in the drained condition.

7.1 Section A-A

Table 3: Section A-A - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Undrained Strength			Comment
	Bulk	Dry	10th Percentile	30th Percentile	50th Percentile	
Estuarine (Upper) Deposit	1.94 / 19.0	1.63 / 16.0	$s_u/\sigma'_{v0} = 0.25$	$s_u/\sigma'_{v0} = 0.31$ to 0.33	$s_u/\sigma'_{v0} = 0.34$ to 0.38	Undrained strength from CPTu, shear vane and DSS testing
Estuarine (Lower) Deposit			$s_u/\sigma'_{v0} = 0.22$	$s_u/\sigma'_{v0} = 0.26$ to 0.28	$s_u/\sigma'_{v0} = 0.27$ to 0.29	
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 16.0	$s_u/\sigma'_{v0} = 0.30$ to 0.50 (Represents the 10th Percentile)			Undrained strength from CPTu, shear vane and DSS testing
Mixed (Farmed / Unfarmed) Bauxite Residue			$s_u/\sigma'_{v0} = 0.25$ (Represents the 10th Percentile)			
Farmed Bauxite Residue			$s_u/\sigma'_{v0} = 0.60$ (Represents the 10th Percentile)			

Ø=friction angle; c=cohesion; s_u=undrained shear strength; σ'_{v0}=vertical effective confining stress

Table 4: Section A-A – Stability Analyses Results at Stage 10

Section A-A @ Stage 10	Factor of Safety (Undrained)
	30th Percentile (Estuarine)
Global Slope Stability	1.73 **(A1)**
Upper Slope Stability	2.08
Lower Slope Stability	1.62 **(A2)**
Post-Seismic Stability	1.32
Pseudo-Static Stability	1.11

Table 5: Section A-A – Stability Analyses Results at Stage 16

Section A-A (Stage 16)	Factor of Safety (Undrained)		
	10th Percentile Estuarine	30th Percentile Estuarine	50th Percentile Estuarine
Global Slope Stability	1.30	1.48 **(A3)**	1.58 **(A3)**
Upper Slope Stability	2.06	2.05 **(A4)**	2.15 **(A4)**
Middle Slope Stability	1.78	1.94 **(A5)**	1.91 **(A5)**
Lower Slope Stability	1.27	1.48 **(A6)**	1.57 **(A6)**
Post-Seismic Stability	0.98	1.15 **(A7)**	1.22 **(A7)**
Pseudo-Static Stability	0.83	0.96 **(A8)**	1.03 **(A8)**

The stability analyses concludes that the as-constructed Section A-A slope profile attains the target FoS of 1.5 for the BRDA constructed to Stage 10 and for the proposed construction to Stage 16, using the 30th Percentile undrained strength ratio for the estuarine deposits.

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (Farmed-Undrained)	SHANSEP	21.5			25	0.6	
	Bauxite Residue (Mixed - Undrained)	SHANSEP	21.5			25	0.25	
	Bauxite Residue (Unfarmed - Undrained)	SHANSEP	21.5			12	0.3	Bauxite Residue
	Bauxite Residue (Unfarmed - Undrained) (0.5)	SHANSEP	21.5			12	0.5	Bauxite Residue
	Bedrock	Bedrock (Impenetrable)						
	Buttress	Mohr-Coulomb	22	0	45			
	Estuarine Lower GA18-1A&C	SHANSEP	19			5	0.26	
	Estuarine Lower GA18-1D	SHANSEP	19			5	0.28	
	Estuarine Upper GA18-1A&D	SHANSEP	19			5	0.31	
	Estuarine Upper GA18-1C	SHANSEP	19			5	0.33	
	Gabion	Mohr-Coulomb	22	200	45			
	Processed Glacial Till	Mohr-Coulomb	19	0	32			
	Rockfill	Mohr-Coulomb	22	0	45			

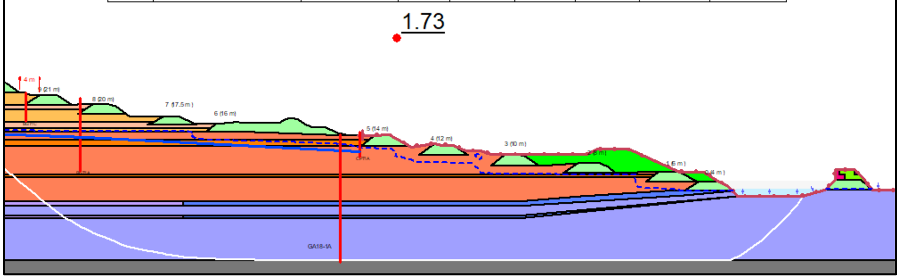


Figure A1: Global Slope Stability, Toe Buttress, Stage 10 = 1.73 (30th Percentile)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (Farmed - Undrained)	SHANSEP	21.5			25	0.6	
	Bauxite Residue (Mixed - Undrained)	SHANSEP	21.5			25	0.25	
	Bauxite Residue (Unfarmed - Undrained)	SHANSEP	21.5			12	0.3	Bauxite Residue
	Bauxite Residue (Unfarmed - Undrained) (0.5)	SHANSEP	21.5			12	0.5	Bauxite Residue
	Bedrock	Bedrock (Impenetrable)						
	Buttress	Mohr-Coulomb	22	0	45			
	Estuarine Lower GA18-1A&C	SHANSEP	19			5	0.26	
	Estuarine Lower GA18-1D	SHANSEP	19			5	0.28	
	Estuarine Upper GA18-1A&D	SHANSEP	19			5	0.31	
	Estuarine Upper GA18-1C	SHANSEP	19			5	0.33	
	Gabion	Mohr-Coulomb	22	200	45			
	Processed Glacial Till	Mohr-Coulomb	19	0	32			
	Rockfill	Mohr-Coulomb	22	0	45			

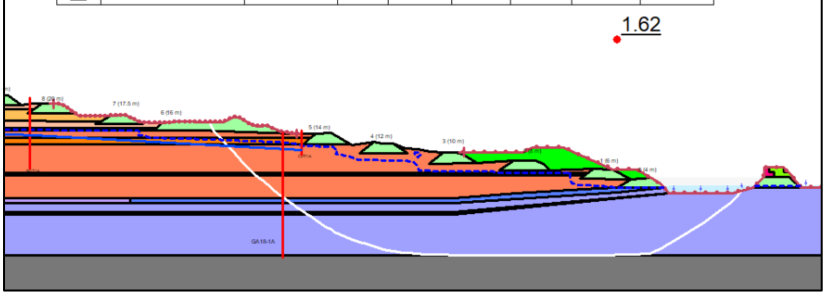
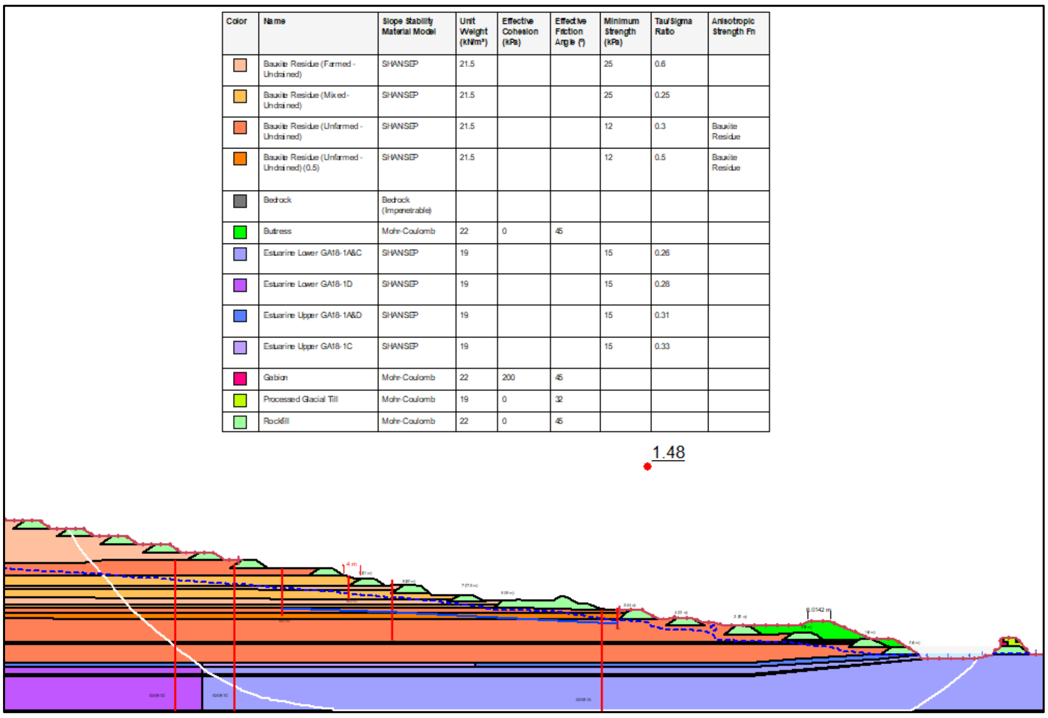
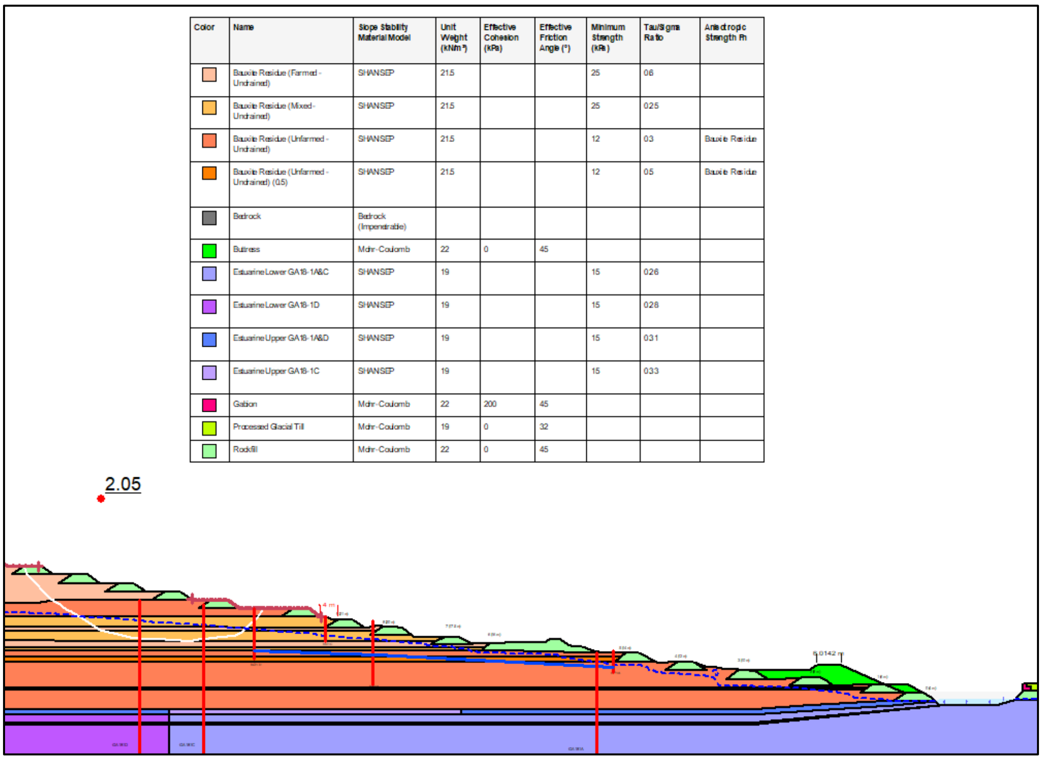


Figure A2: Lower Slope Stability, Toe Buttress, Stage 10 = 1.62 (30th Percentile)



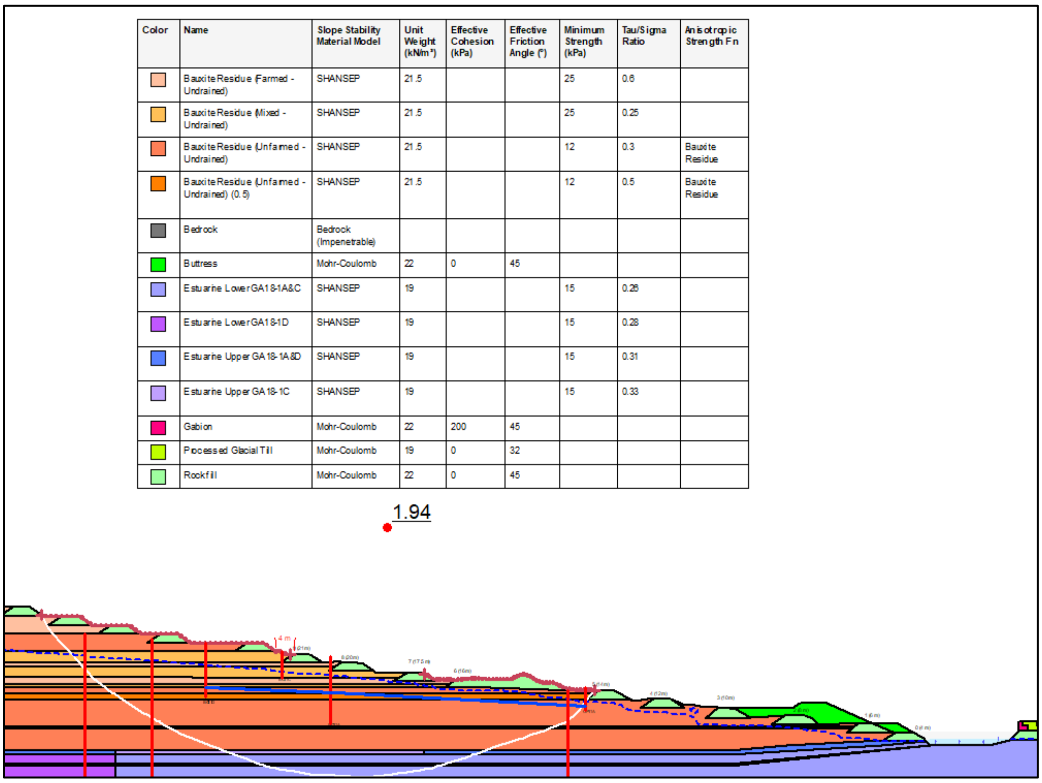
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (Farmed - Undrained)	SHANSEP	21.5			25	0.6	
	Bauxite Residue (Mixed - Undrained)	SHANSEP	21.5			25	0.25	
	Bauxite Residue (Unfarmed - Undrained)	SHANSEP	21.5			12	0.3	Bauxite Residue
	Bauxite Residue (Unfarmed - Undrained) (0.5)	SHANSEP	21.5			12	0.5	Bauxite Residue
	Bedrock	Bedrock (Impenetrable)						
	Buttress	Mohr-Coulomb	22	0	45			
	Estuarine Lower GA18-1A&C	SHANSEP	19			15	0.26	
	Estuarine Lower GA18-1D	SHANSEP	19			15	0.28	
	Estuarine Upper GA18-1A&D	SHANSEP	19			15	0.31	
	Estuarine Upper GA18-1C	SHANSEP	19			15	0.33	
	Gabion	Mohr-Coulomb	22	200	45			
	Processed Glacial Till	Mohr-Coulomb	19	0	32			
	Rockfill	Mohr-Coulomb	22	0	45			

Figure A3: Global Slope Stability, Toe Buttress, Stage 16 = 1.48 (30th Percentile) and = 1.58 (50th Percentile)



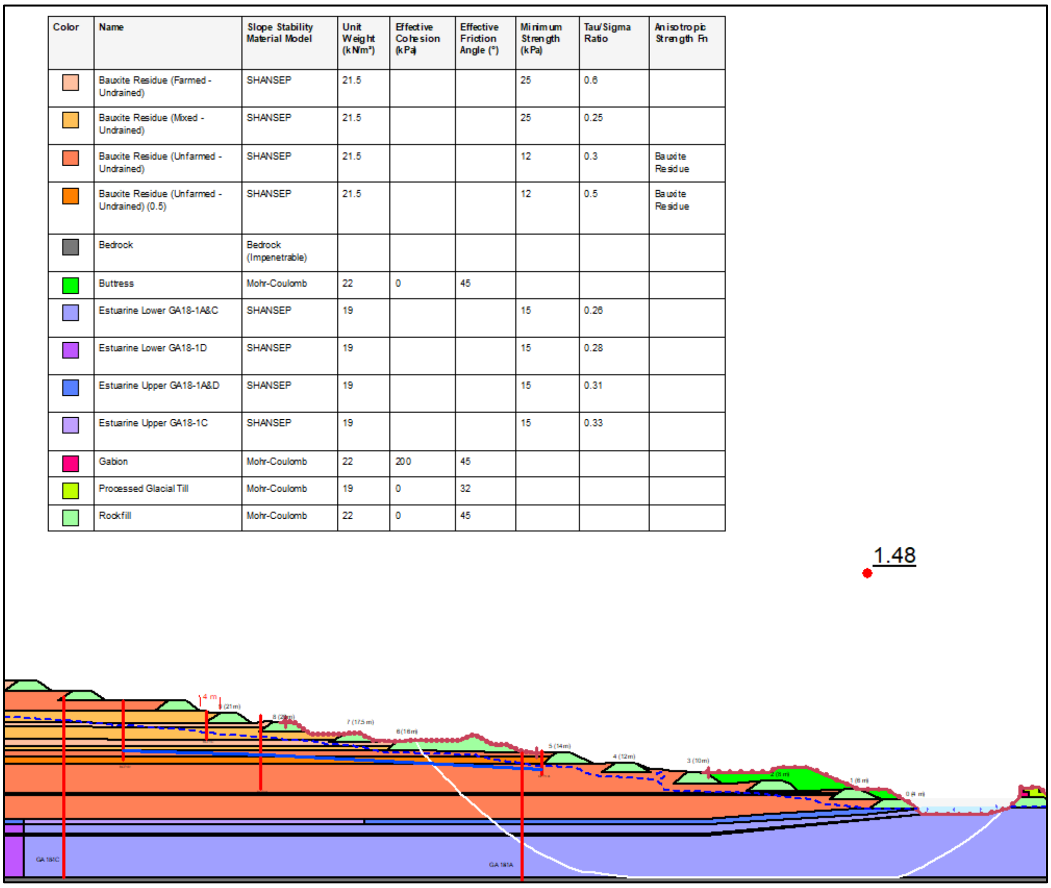
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (Farmed - Undrained)	SHANSEP	21.5			25	0.6	
	Bauxite Residue (Mixed - Undrained)	SHANSEP	21.5			25	0.25	
	Bauxite Residue (Unfarmed - Undrained)	SHANSEP	21.5			12	0.3	Bauxite Residue
	Bauxite Residue (Unfarmed - Undrained) (0.5)	SHANSEP	21.5			12	0.5	Bauxite Residue
	Bedrock	Bedrock (Impenetrable)						
	Buttress	Mohr-Coulomb	22	0	45			
	Estuarine Lower GA18-1A&C	SHANSEP	19			15	0.26	
	Estuarine Lower GA18-1D	SHANSEP	19			15	0.28	
	Estuarine Upper GA18-1A&D	SHANSEP	19			15	0.31	
	Estuarine Upper GA18-1C	SHANSEP	19			15	0.33	
	Gabion	Mohr-Coulomb	22	200	45			
	Processed Glacial Till	Mohr-Coulomb	19	0	32			
	Rockfill	Mohr-Coulomb	22	0	45			

Figure A4: Upper Slope Stability, Toe Buttress, Stage 16 = 2.05 (30th Percentile) and = 2.15 (50th Percentile)



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (Farmed - Undrained)	SHANSEP	21.5			25	0.6	
	Bauxite Residue (Mixed - Undrained)	SHANSEP	21.5			25	0.25	
	Bauxite Residue (Unfarmed - Undrained)	SHANSEP	21.5			12	0.3	Bauxite Residue
	Bauxite Residue (Unfarmed - Undrained) (0.5)	SHANSEP	21.5			12	0.5	Bauxite Residue
	Bedrock	Bedrock (Impenetrable)						
	Buttress	Mohr-Coulomb	22	0	45			
	Estuarine Lower GA18-1A&C	SHANSEP	19			15	0.26	
	Estuarine Lower GA18-1D	SHANSEP	19			15	0.28	
	Estuarine Upper GA18-1A&D	SHANSEP	19			15	0.31	
	Estuarine Upper GA18-1C	SHANSEP	19			15	0.33	
	Gabion	Mohr-Coulomb	22	200	45			
	Processed Glacial Till	Mohr-Coulomb	19	0	32			
	Rockfill	Mohr-Coulomb	22	0	45			

Figure A5: Middle Slope Stability, Toe Buttress, Stage 16 = 1.94 (30th Percentile) and = 1.91 (50th Percentile)



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (Farmed - Undrained)	SHANSEP	21.5			25	0.6	
	Bauxite Residue (Mixed - Undrained)	SHANSEP	21.5			25	0.25	
	Bauxite Residue (Unfarmed - Undrained)	SHANSEP	21.5			12	0.3	Bauxite Residue
	Bauxite Residue (Unfarmed - Undrained) (0.5)	SHANSEP	21.5			12	0.5	Bauxite Residue
	Bedrock	Bedrock (Impenetrable)						
	Buttress	Mohr-Coulomb	22	0	45			
	Estuarine Lower GA18-1A&C	SHANSEP	19			15	0.26	
	Estuarine Lower GA18-1D	SHANSEP	19			15	0.28	
	Estuarine Upper GA18-1A&D	SHANSEP	19			15	0.31	
	Estuarine Upper GA18-1C	SHANSEP	19			15	0.33	
	Gabion	Mohr-Coulomb	22	200	45			
	Processed Glacial Till	Mohr-Coulomb	19	0	32			
	Rockfill	Mohr-Coulomb	22	0	45			

Figure A6: Lower Slope Stability, Toe Buttress, Stage 16 = 1.48 (30th Percentile) and = 1.57 (50th Percentile)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (Farmed PS)	SHANSEP	21.5			20	0.48	
	Bauxite Residue (Mixed - Undrained) PS	SHANSEP	21.5			25	0.2	
	Bauxite Residue (Unfarmed - Undrained) (0.5) PS	SHANSEP	21.5			9.6	0.4	Bauxite Residue
	Bauxite Residue (Unfarmed PS)	SHANSEP	21.5			16	0.24	
	Bedrock	Bedrock (Impenetrable)						
	Buttress	Mohr-Coulomb	22	0	45			
	Estuarine Lower GA18-1A&C PS	SHANSEP	19			12	0.208	
	Estuarine Lower GA18-1D PS	SHANSEP	19			12	0.224	
	Estuarine Upper GA18-1A&D PS	SHANSEP	19			12	0.248	
	Estuarine Upper GA18-1C PS	SHANSEP	19			12	0.264	
	Gabion	Mohr-Coulomb	22	200	45			
	Processed Glacial Till	Mohr-Coulomb	19	0	32			
	Rockfill	Mohr-Coulomb	22	0	45			

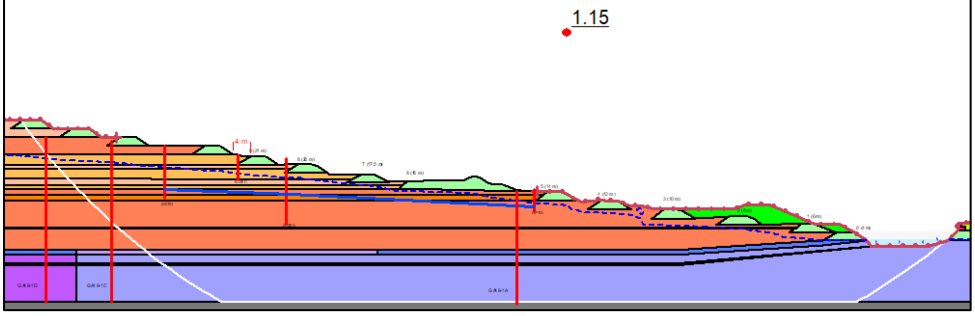


Figure A7: Undrained Post-Earthquake Analysis – Toe Buttress, Stage 16 = 1.15 (30th Percentile) and = 1.22 (50th Percentile)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (Farmed PS)	SHANSEP	21.5			20	0.48	
	Bauxite Residue (Mixed - Undrained) PS	SHANSEP	21.5			25	0.2	
	Bauxite Residue (Unfarmed - Undrained) (0.5) PS	SHANSEP	21.5			9.6	0.4	Bauxite Residue
	Bauxite Residue (Unfarmed PS)	SHANSEP	21.5			16	0.24	
	Bedrock	Bedrock (Impenetrable)						
	Buttress	Mohr-Coulomb	22	0	45			
	Estuarine Lower GA18-1A&C PS	SHANSEP	19			12	0.208	
	Estuarine Lower GA18-1D PS	SHANSEP	19			12	0.224	
	Estuarine Upper GA18-1A&D PS	SHANSEP	19			12	0.248	
	Estuarine Upper GA18-1C PS	SHANSEP	19			12	0.264	
	Gabion	Mohr-Coulomb	22	200	45			
	Processed Glacial Till	Mohr-Coulomb	19	0	32			
	Rockfill	Mohr-Coulomb	22	0	45			

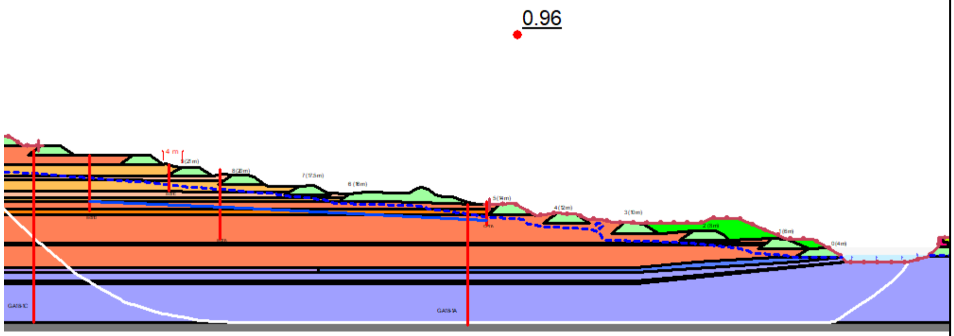


Figure A8: Undrained Pseudo-Static Analysis – Toe Buttress, Stage 16 = 0.96 (30th Percentile) and = 1.03 (50th Percentile)

7.2 Section B-B

Table 6: Section B-B - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Strength Undrained			Comment
	Bulk	**Dry**	**10th Percentile**	**30th Percentile**	**50th Percentile**	
Estuarine (Upper) Deposit	1.94 / 19.0	1.63 / 16.0	$s_u/\sigma'_{v0} = 0.23$ to 0.25	$s_u/\sigma'_{v0} = 0.26$ to 0.30	$s_u/\sigma'_{v0} = 0.28$ to 0.35	Undrained strength from CPTu, shear vane and DSS testing
Estuarine (Middle) Deposit			$s_u/\sigma'_{v0} = 0.20$ to 0.21	$s_u/\sigma'_{v0} = 0.22$ to 0.24	$s_u/\sigma'_{v0} = 0.22$ to 0.25	
Estuarine (Lower) Deposit			$s_u/\sigma'_{v0} = 0.28$ to 0.30	$s_u/\sigma'_{v0} = 0.28$ to 0.30	$s_u/\sigma'_{v0} = 0.30$ to 0.35	
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 16.0	$s_u/\sigma'_{v0} = 0.25$			Undrained strength from CPTu, shear vane and DSS testing
Farmed Bauxite Residue			$s_u/\sigma'_{v0} = 0.60$			

Ø=friction angle; c=cohesion; s_u=undrained shear strength; σ'_{v0}=vertical effective confining stress

Table 7: Section B-B – Stability Analyses Results at Stage 10

Section B-B @ Stage 10	Factor of Safety (Undrained) 30th Percentile (Estuarine)
Global Slope Stability	1.58 **(B1)**
Upper Slope Stability	2.04 **(B2)**
Lower Slope Stability	1.78 **(B3)**
Post-Seismic Stability	1.29 **(B4)**
Pseudo-Static Stability	1.06 **(B5)**

Table 8: Section B-B – Stability Analyses Results at Stage 16

Section B-B (Stage 16)	Factor of Safety (Undrained)	
	10th Percentile Estuarine	30th Percentile Estuarine
Global Slope Stability	1.28	1.49 **(B6)**
Upper Slope Stability	1.83	2.12 **(B7)**
Middle Slope Stability	1.48	1.73 **(B8)**
Lower Slope Stability	1.36	1.58 **(B9)**
Post- Seismic Stability	1.02	1.19 **(B10)**
Pseudo-Static Stability	0.86	1.00 **(B11)**

The stability analyses concludes that the as-constructed Section B-B slope profile attains the target FoS of 1.5 for the BRDA constructed to Stage 10 and for the proposed construction to Stage 16, using the 30th Percentile undrained strength ratio for the estuarine deposits.

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				15	0.26	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Lower) GA18-2B	SHANSEP	19				15	0.28	
	Estuarine (Middle) GA18-2A	SHANSEP	19				15	0.24	
	Estuarine (Middle) GA18-2B	SHANSEP	19				15	0.22	
	Estuarine (Upper) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Upper) GA18-2B	SHANSEP	19				15	0.26	

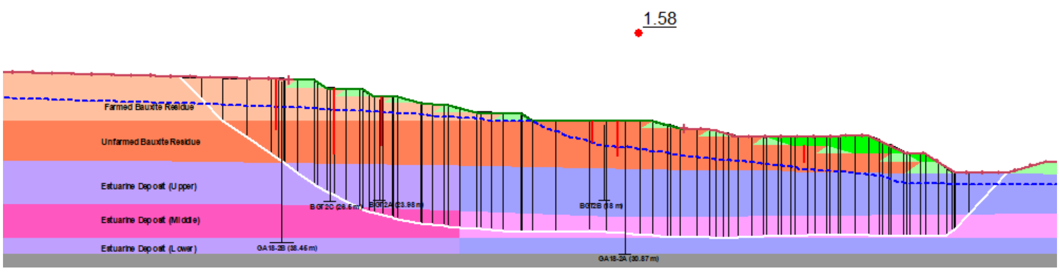


Figure B1: Undrained Static Analysis – Global Section Stage 10 (30th Percentile) – FoS = 1.58

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				15	0.26	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Lower) GA18-2B	SHANSEP	19				15	0.28	
	Estuarine (Middle) GA18-2A	SHANSEP	19				15	0.24	
	Estuarine (Middle) GA18-2B	SHANSEP	19				15	0.22	
	Estuarine (Upper) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Upper) GA18-2B	SHANSEP	19				15	0.26	

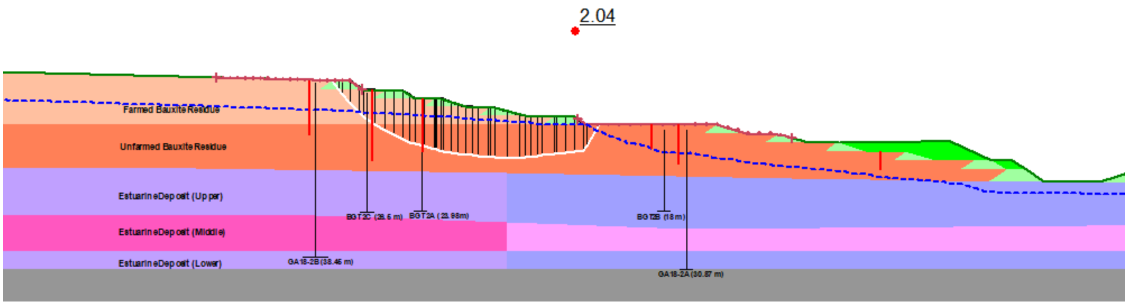


Figure B2: Undrained Static Analysis – Upper Section Stage 10 (30th Percentile) – FoS = 2.04

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				15	0.26	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Lower) GA18-2B	SHANSEP	19				15	0.28	
	Estuarine (Middle) GA18-2A	SHANSEP	19				15	0.24	
	Estuarine (Middle) GA18-2B	SHANSEP	19				15	0.22	
	Estuarine (Upper) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Upper) GA18-2B	SHANSEP	19				15	0.26	

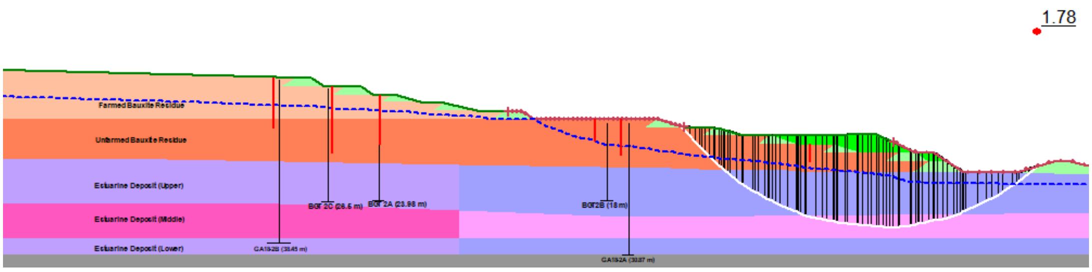


Figure B3: Undrained Static Analysis – Lower Section Stage 10 (30th Percentile) – FoS = 1.78

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	Bauxite Residue
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-2A (PS)	SHANSEP	19				12	0.24	
	Estuarine (Lower) GA18-2B (PS)	SHANSEP	19				12	0.224	
	Estuarine (Midde) GA18-2A (PS)	SHANSEP	19				12	0.192	
	Estuarine (Midde) GA18-2B (PS)	SHANSEP	19				12	0.176	
	Estuarine (Upper) GA-18-2A (PS)	SHANSEP	19				12	0.24	
	Estuarine (Upper) GA18-2B (PS)	SHANSEP	19				12	0.208	

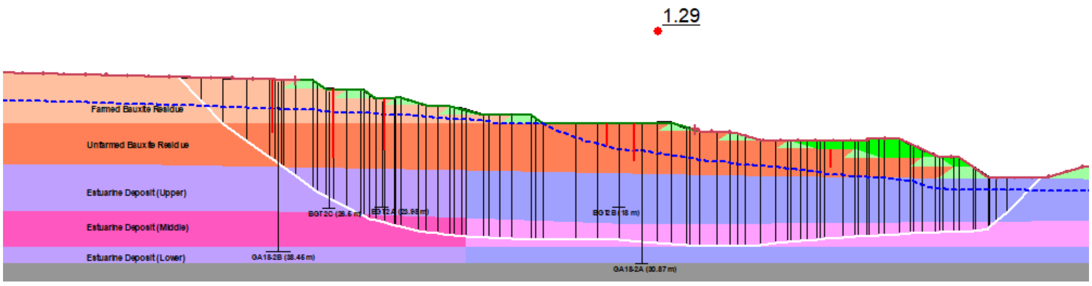


Figure B4: Undrained Post-Earthquake Analysis – Global Section Stage 10 (30th Percentile) – FoS = 1.29

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	Bauxite Residue
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-2A (PS)	SHANSEP	19				12	0.24	
	Estuarine (Lower) GA18-2B (PS)	SHANSEP	19				12	0.224	
	Estuarine (Middle) GA18-2A (PS)	SHANSEP	19				12	0.192	
	Estuarine (Middle) GA18-2B (PS)	SHANSEP	19				12	0.176	
	Estuarine (Upper) GA-18-2A (PS)	SHANSEP	19				12	0.24	
	Estuarine (Upper) GA18-2B (PS)	SHANSEP	19				12	0.208	

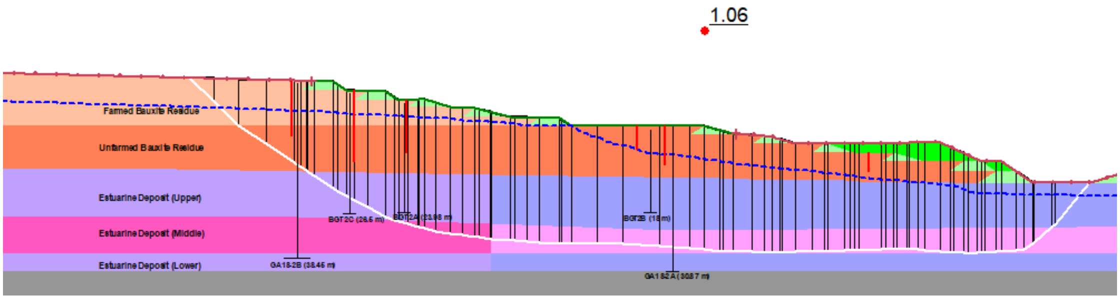


Figure B5: Section B-B; Undrained Pseudo-Static Analysis – Global Section Stage 10 (30th Percentile) – FoS = 1.06

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Butress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Lower) GA18-2B	SHANSEP	19				15	0.28	
	Estuarine (Middle) GA18-2A	SHANSEP	19				15	0.24	
	Estuarine (Middle) GA18-2B	SHANSEP	19				15	0.22	
	Estuarine (Upper) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Upper) GA18-2B	SHANSEP	19				15	0.26	

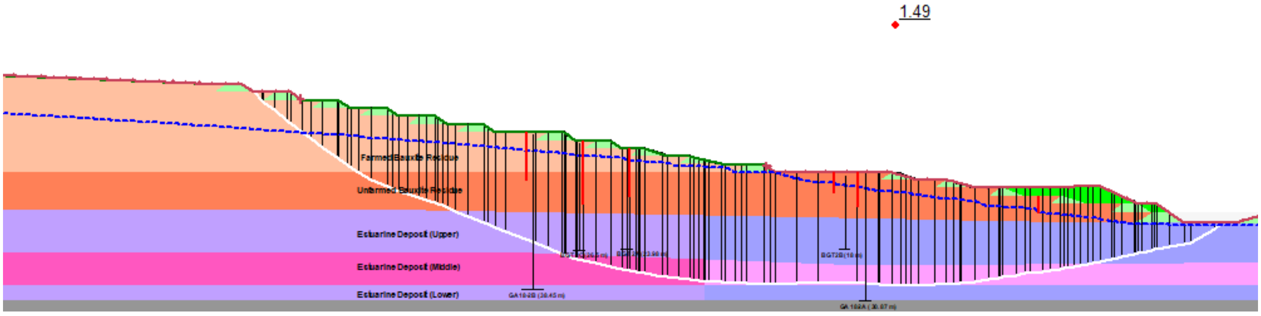


Figure B6: Undrained Static Analysis – Global Section with Buttress (30th Percentile) – FoS = 1.49

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Butress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Lower) GA18-2B	SHANSEP	19				15	0.28	
	Estuarine (Middle) GA18-2A	SHANSEP	19				15	0.24	
	Estuarine (Middle) GA18-2B	SHANSEP	19				15	0.22	
	Estuarine (Upper) GA18-2A	SHANSEP	19				15	0.3	
	Estuarine (Upper) GA18-2B	SHANSEP	19				15	0.26	

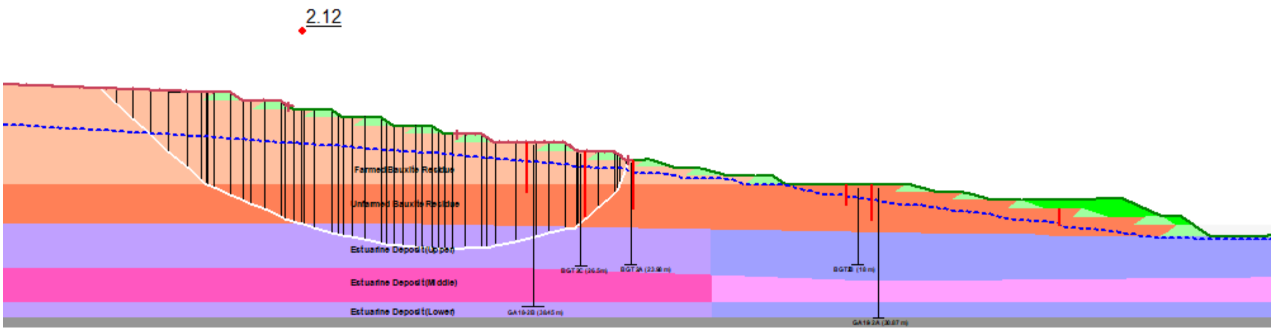


Figure B7: Undrained Static Analysis – Upper Section with Buttress (30th Percentile) – FoS = 2.12

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA182A	SHANSEP	19				15	0.3	
	Estuarine (Lower) GA182B	SHANSEP	19				15	0.28	
	Estuarine (Middle) GA182A	SHANSEP	19				15	0.24	
	Estuarine (Middle) GA182B	SHANSEP	19				15	0.22	
	Estuarine (Upper) GA182A	SHANSEP	19				15	0.3	
	Estuarine (Upper) GA182B	SHANSEP	19				15	0.26	

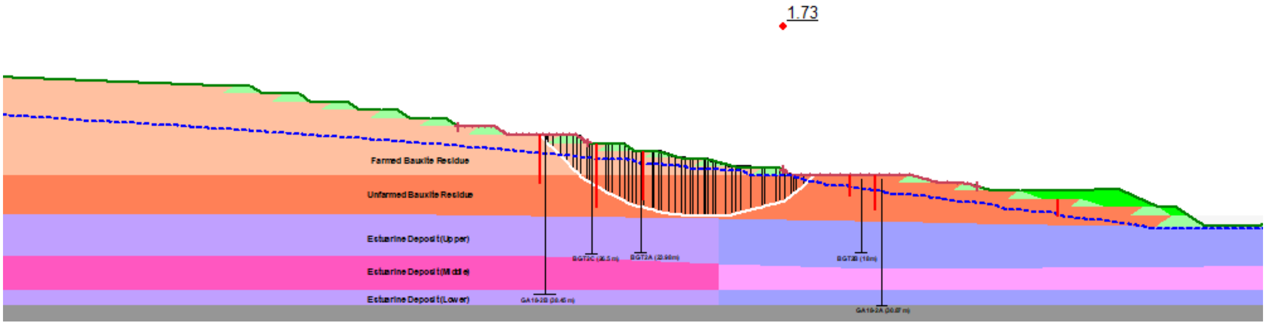


Figure B8: Undrained Static Analysis – Middle Section with Buttress (30th Percentile) – FoS = 1.73

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	Mohr-Coulomb	22	0	45	0			
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA182A	SHANSEP	19				15	0.3	
	Estuarine (Lower) GA182B	SHANSEP	19				15	0.28	
	Estuarine (Middle) GA182A	SHANSEP	19				15	0.24	
	Estuarine (Middle) GA182B	SHANSEP	19				15	0.22	
	Estuarine (Upper) GA182A	SHANSEP	19				15	0.3	
	Estuarine (Upper) GA182B	SHANSEP	19				15	0.26	

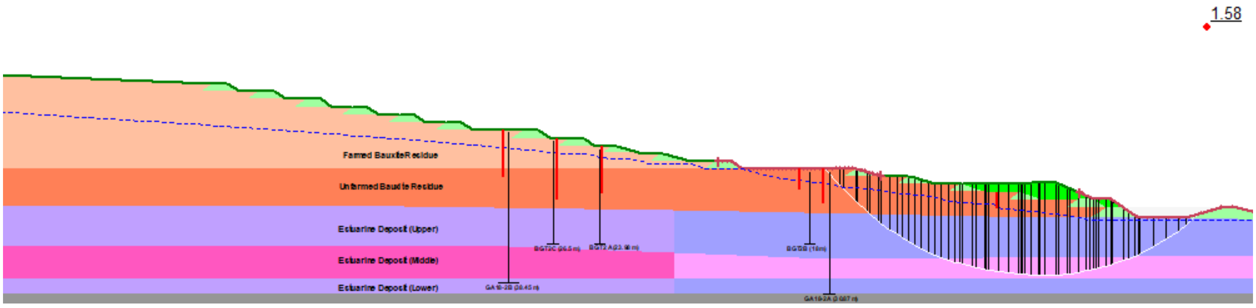


Figure B9: Undrained Static Analysis – Lower Section with Buttress (30th Percentile) – FoS = 1.58

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/ Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	Bauxite Residue
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	MohrCoulomb	22	0	45	0			
	Dyke Rockfill	MohrCoulomb	22	0	45	0			
	Estuarine (Lower) GA 18-2A (PS)	SHANSEP	19				12	0.24	
	Estuarine (Lower) GA 18-2B (PS)	SHANSEP	19				12	0.224	
	Estuarine (Middle) GA 18-2A (PS)	SHANSEP	19				12	0.192	
	Estuarine (Middle) GA 18-2B (PS)	SHANSEP	19				12	0.176	
	Estuarine (Upper) GA 18-2A (PS)	SHANSEP	19				12	0.24	
	Estuarine (Upper) GA 18-2B (PS)	SHANSEP	19				12	0.208	

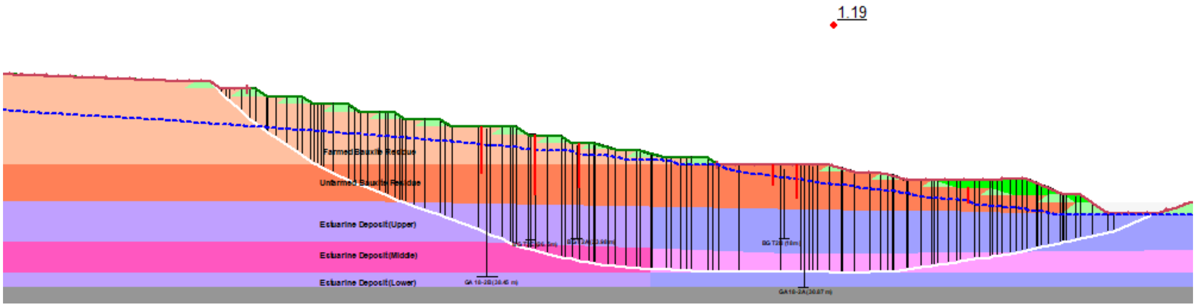


Figure B10: Undrained Post-Earthquake Analysis – Global Section with Buttress (30th Percentile) – FoS = 1.19

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	Bauxite Residue
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	
	Bedrock	Bedrock (Impenetrable)							
	Buttress	MohrCoulomb	22	0	45	0			
	Dyke Rockfill	MohrCoulomb	22	0	45	0			
	Estuarine (Lower) GA 18-2A (PS)	SHANSEP	19				12	0.24	
	Estuarine (Lower) GA18-2B (PS)	SHANSEP	19				12	0.224	
	Estuarine (Middle) GA18-2A (PS)	SHANSEP	19				12	0.192	
	Estuarine (Middle) GA18-2B (PS)	SHANSEP	19				12	0.176	
	Estuarine (Upper) GA18-2A (PS)	SHANSEP	19				12	0.24	
	Estuarine (Upper) GA18-2B (PS)	SHANSEP	19				12	0.208	

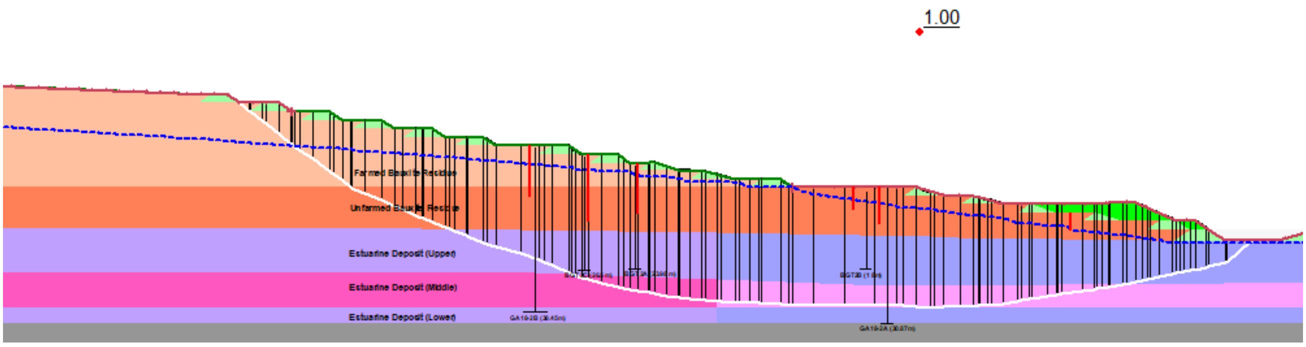


Figure B11: Undrained Pseudo-Static Analysis – Global Section with Buttress (30th Percentile) – FoS = 1.00

7.3 Section C-C

Table 9: Section C-C - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Strength			Comment
			Undrained			
	Bulk	Dry	10th Percentile	30th Percentile	50th Percentile	
Estuarine (Upper) Deposit	1.94 / 19.0	1.63 / 16.0	$s_u/\sigma'_{v0} = 0.23$	$s_u/\sigma'_{v0} = 0.39$	$s_u/\sigma'_{v0} = 0.45$	Undrained strength from CPTu, shear vane and DSS testing
Estuarine (Lower) Deposit			$s_u/\sigma'_{v0} = 0.35$	$s_u/\sigma'_{v0} = 0.44$	$s_u/\sigma'_{v0} = 0.47$	
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 16.0	$s_u/\sigma'_{v0} = 0.30$			Undrained strength from CPTu, shear vane and DSS testing
Farmed Bauxite Residue			$s_u/\sigma'_{v0} = 0.60$			

Ø=friction angle; c=cohesion; s_u=undrained shear strength; σ'_{v0}=vertical effective confining stress

Table 10: Section C-C – Stability Analyses Results at Stage 10

Section C-C @ Stage 10	Factor of Safety (Undrained)
	30th Percentile (Estuarine)
Global Slope Stability	2.00 **(C1)**
Upper Slope Stability	1.80 **(C2)**
Lower Slope Stability	1.81 **(C3)**
Post-Seismic Stability	1.86 **(C4)**
Pseudo-Static Stability	1.54 **(C5)**

Table 11: Section C-C – Stability Analyses Results at Stage 16

Section C-C (Stage 16)	Factor of Safety (Undrained)	
	10th Percentile Estuarine	30th Percentile Estuarine
Global Slope Stability	1.53 **(C6)**	1.84
Upper Slope Stability	2.08 **(C7)**	2.10
Middle Slope Stability	1.79 **(C8)**	1.65
Lower Slope Stability	1.50 **(C9)**	1.87
Post- Seismic Stability	1.17 **(C10)**	1.40
Pseudo-Static Stability	0.99 **(C11)**	1.21

The stability analyses concludes that the as-constructed Section C-C slope profile attains the target FoS of 1.5 for the BRDA constructed to Stage 10 and for the proposed construction to Stage 16, using the 10th Percentile undrained strength ratio for the estuarine deposits.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed)	SHANSEP	21.5				12	0.25
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower)	SHANSEP	19				15	0.44
	Estuarine (Upper)	SHANSEP	19				15	0.39

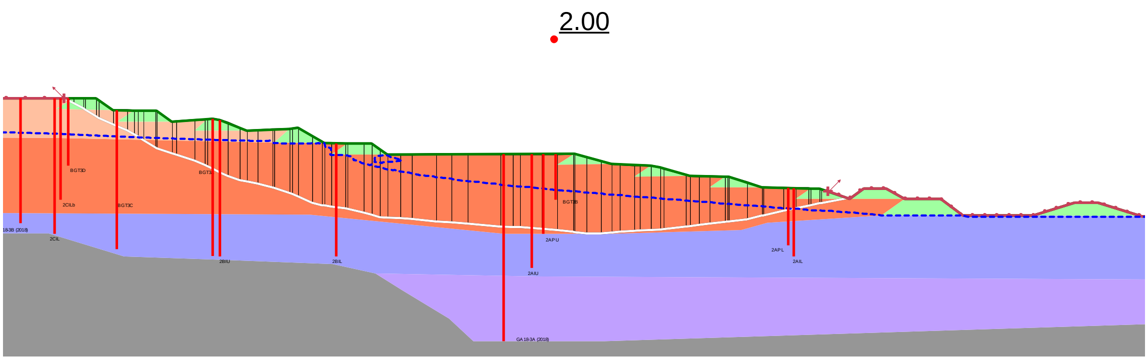


Figure C1: Undrained Static Analysis – Global Section Stage 10 (30th Percentile) – FoS = 2.00

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed)	SHANSEP	21.5				12	0.25
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower)	SHANSEP	19				15	0.44
	Estuarine (Upper)	SHANSEP	19				15	0.39

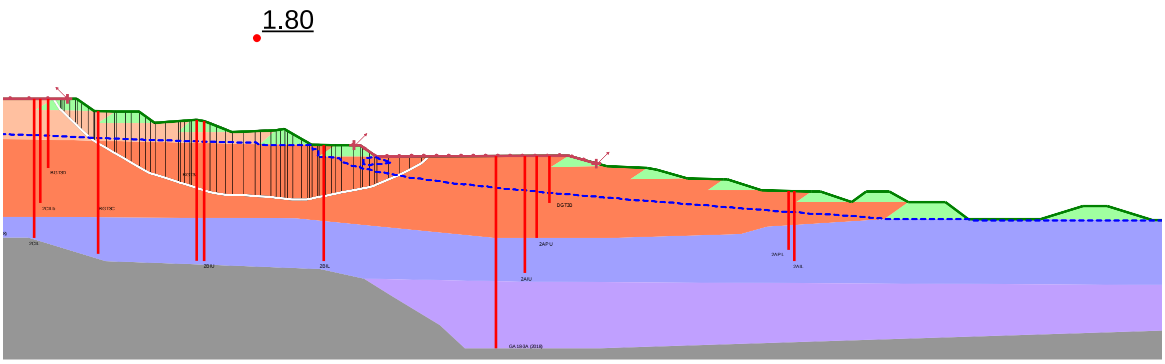


Figure C2: Undrained Static Analysis – Upper Section Stage 10 (30th Percentile) – FoS = 1.80

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed)	SHANSEP	21.5				12	0.25
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower)	SHANSEP	19				15	0.44
	Estuarine (Upper)	SHANSEP	19				15	0.39

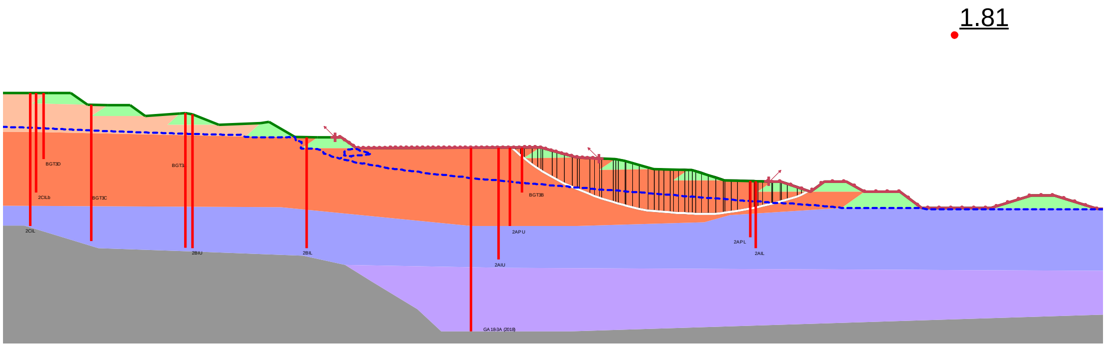


Figure C3: Undrained Static Analysis – Lower Section Stage 10 (30th Percentile) – FoS = 1.81

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower) (PS)	SHANSEP	19				12	0.352
	Estuarine (Upper) (PS)	SHANSEP	19				12	0.312

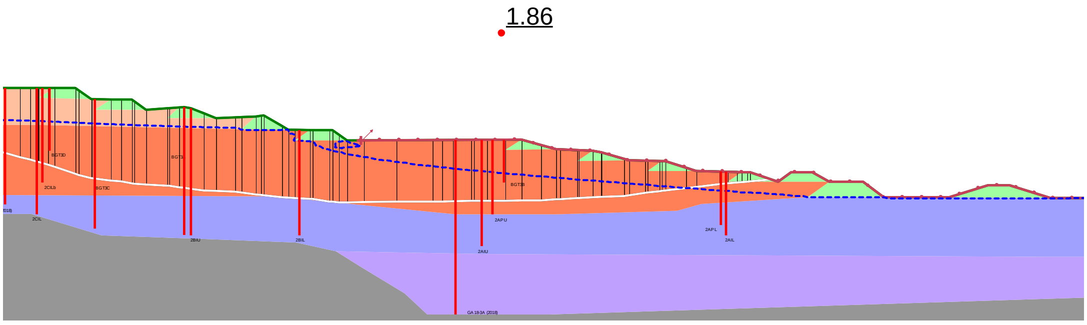


Figure C4: Undrained Post-Earthquake Analysis – Global Section Stage 10 (30th Percentile) – FoS = 1.86

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower) (PS)	SHANSEP	19				12	0.352
	Estuarine (Upper) (PS)	SHANSEP	19				12	0.312

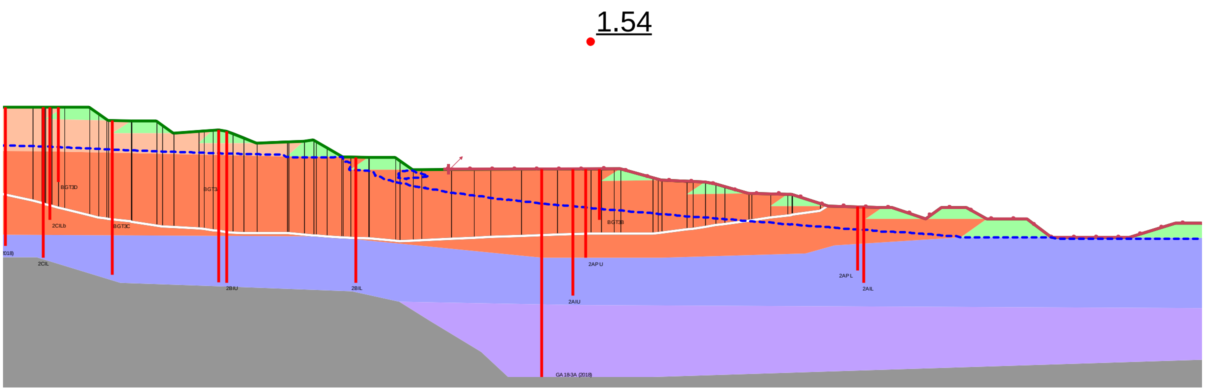


Figure C5: Undrained Pseudo-Static Analysis – Global Section Stage 10 (30th Percentile) – FoS = 1.54

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.3	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower)	SHANSEP	19				5	0.35	
	Estuarine (Upper)	SHANSEP	19				5	0.23	

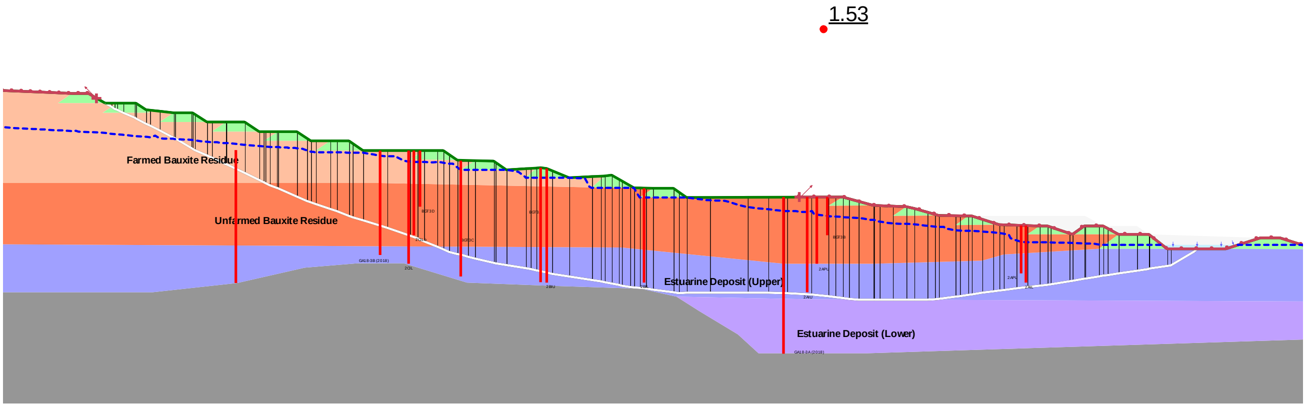


Figure C6: Undrained Static Analysis – Global Section (10th Percentile) – FoS = 1.53

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.3	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower)	SHANSEP	19				5	0.35	
	Estuarine (Upper)	SHANSEP	19				5	0.23	

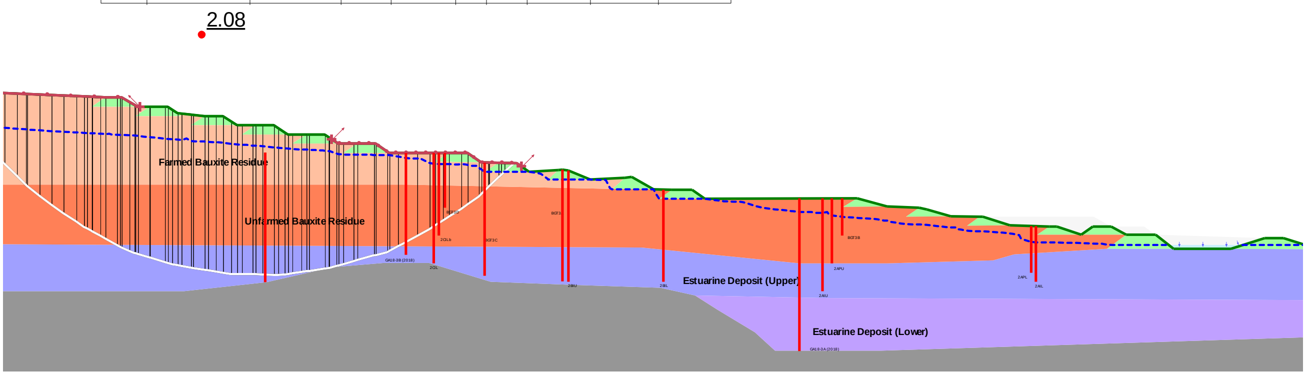


Figure C7: Undrained Static Analysis – Upper Section (10th Percentile) – FoS = 2.08

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.3	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower)	SHANSEP	19				5	0.35	
	Estuarine (Upper)	SHANSEP	19				5	0.23	

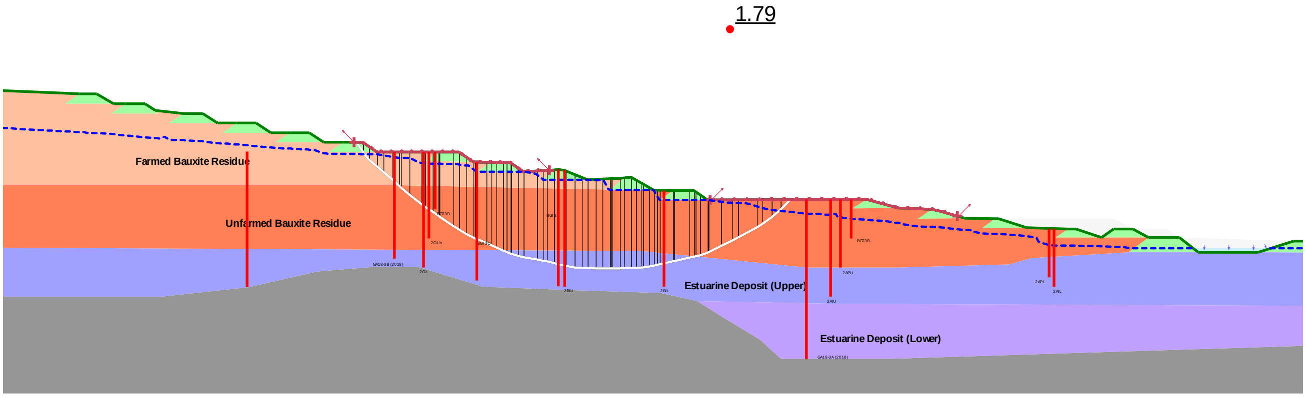


Figure C8: Undrained Static Analysis – Middle Section (10th Percentile) – FoS = 1.79

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.3	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower)	SHANSEP	19				5	0.35	
	Estuarine (Upper)	SHANSEP	19				5	0.23	

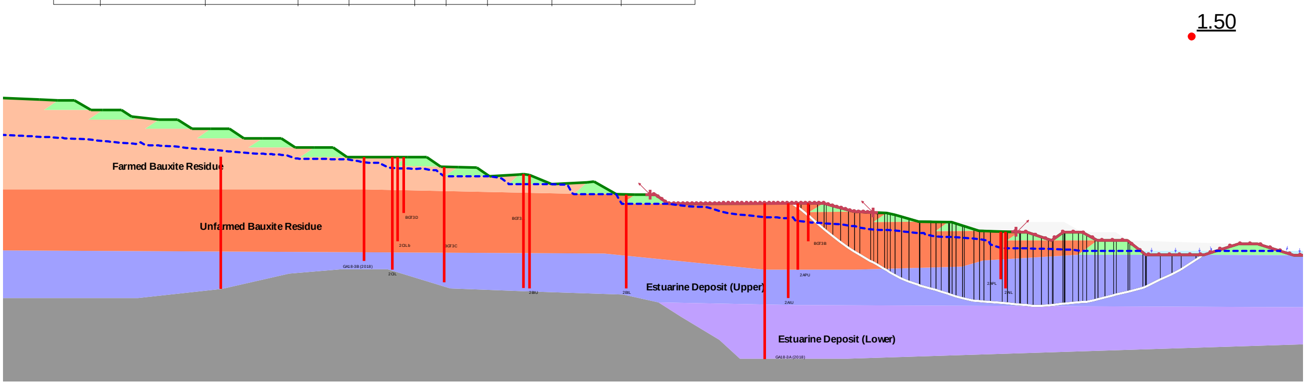


Figure C9: Undrained Static Analysis – Lower Section (10th Percentile) – FoS = 1.50

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.24
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower) (PS)	SHANSEP	19				4	0.28
	Estuarine (Upper) (PS)	SHANSEP	19				4	0.184

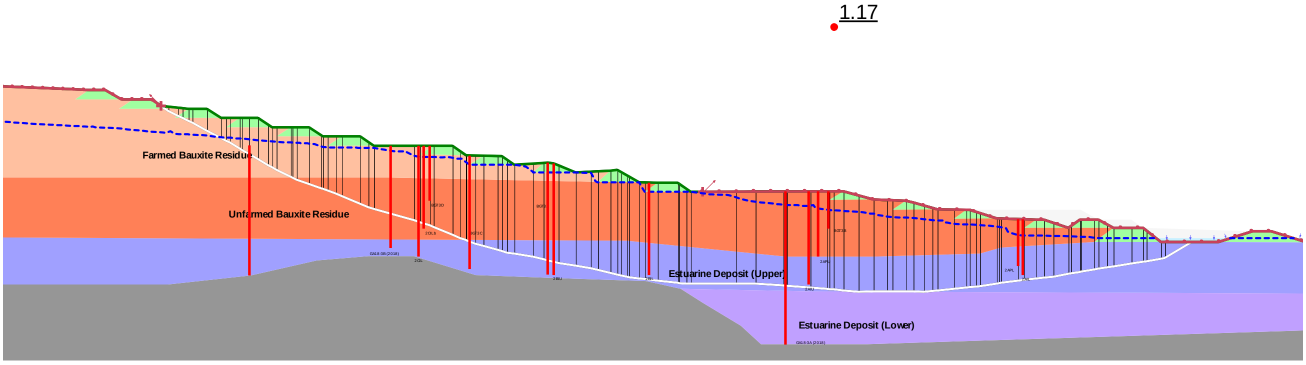


Figure C10: Undrained Post-Earthquake Analysis – Global Section (10th Percentile) – FoS = 1.17

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.24
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower) (PS)	SHANSEP	19				4	0.28
	Estuarine (Upper) (PS)	SHANSEP	19				4	0.184

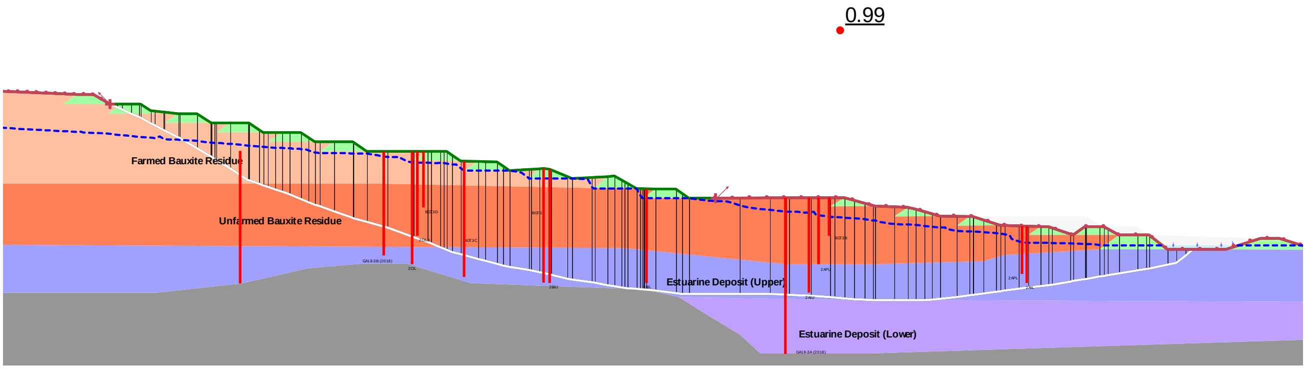


Figure C11: Undrained Pseudo-Static Analysis – Global Section (10th Percentile) – FoS = 0.99

7.4 Section D-D

Table 12: Section D-D - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Strength			Comment
			Undrained			
	Bulk	Dry	10th Percentile	30th Percentile	50th Percentile	
Estuarine (Upper) Deposit	1.94 / 19.0	1.63 / 16.0	s_u/σ'_{v0} = 0.23 to 0.40	s_u/σ'_{v0} = 0.26 to 0.45	s_u/σ'_{v0} = 0.32 to 0.52	Undrained strength from CPTu, shear vane and DSS testing
Estuarine (Lower) Deposit			s_u/σ'_{v0} = 0.20 to 0.22	s_u/σ'_{v0} = 0.25 to 0.29	s_u/σ'_{v0} = 0.28 to 0.32	
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 16.0	s_u/σ'_{v0} = 0.23			Undrained strength from CPTu, shear vane and DSS testing
Farmed Bauxite Residue			s_u/σ'_{v0} = 0.60			

Ø=friction angle; c=cohesion; s_u=undrained shear strength; σ'_{v0}=vertical effective confining stress

Table 13: Section D-D - Stability Analyses Results at Stage 16

Section D-D (Stage 16)	Factor of Safety (Undrained)	
	10th Percentile Estuarine	30th Percentile Estuarine
Global Slope Stability	1.40	1.62 **(D1)**
Upper Slope Stability	2.39	2.33 **(D2)**
Middle Slope Stability	1.95	1.88 **(D3)**
Lower Slope Stability	1.28	1.55 **(D4)**
Post Seismic Stability	1.08	1.26 **(D5)**
Pseudo-Static Stability	0.91	1.07 **(D6)**

The stability analyses concludes that the Section D-D slope profile will attain the target FoS of 1.5 for the BRDA constructed to Stage 16, using the 30th Percentile undrained strength ratio for the estuarine deposits.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.23	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) (GA18-5A)	SHANSEP	19				5	0.29	
	Estuarine (Lower) (GA18-5B)	SHANSEP	19				5	0.25	
	Estuarine (Upper) (GA18-5A)	SHANSEP	19				20	0.45	
	Estuarine (Upper) (GA18-5B)	SHANSEP	19				10	0.26	

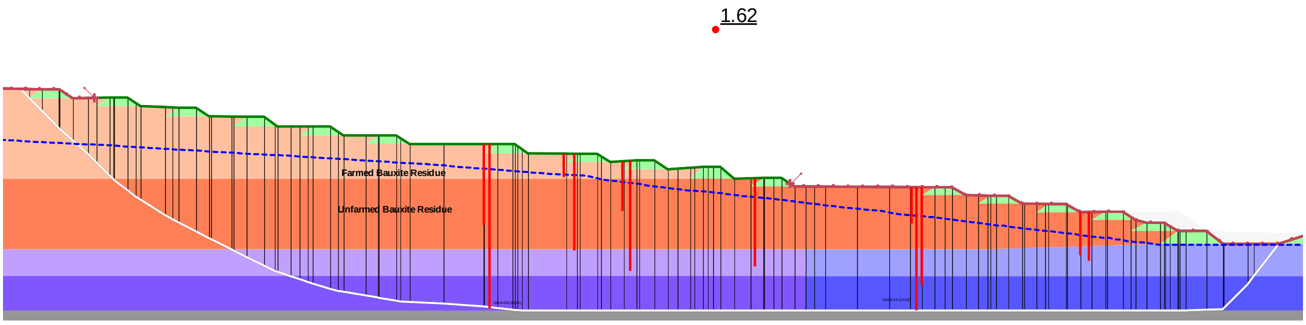


Figure D1: Undrained Static Analysis – Global Section (30th Percentile) – FoS = 1.62

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.23	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) (GA18-5A)	SHANSEP	19				5	0.29	
	Estuarine (Lower) (GA18-5B)	SHANSEP	19				5	0.25	
	Estuarine (Upper) (GA18-5A)	SHANSEP	19				20	0.45	
	Estuarine (Upper) (GA18-5B)	SHANSEP	19				10	0.26	

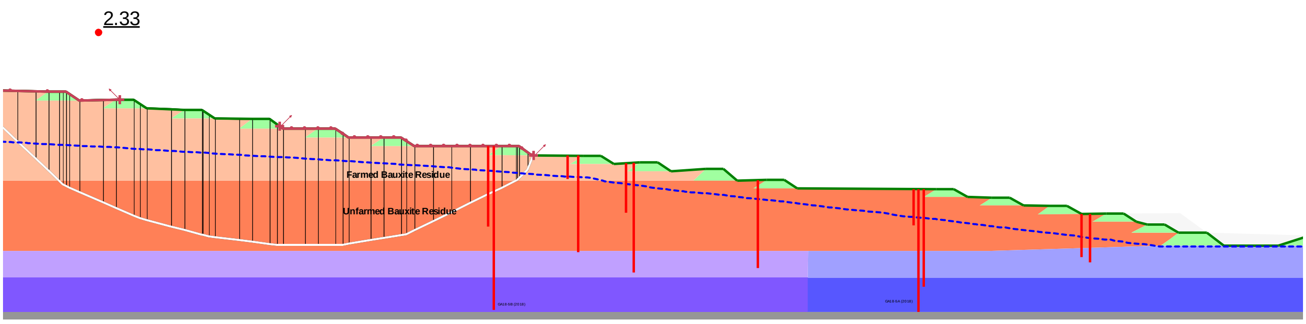


Figure D2: Undrained Static Analysis – Upper Section (30th Percentile) – FoS = 2.33

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.23	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) (GA18-5A)	SHANSEP	19				5	0.29	
	Estuarine (Lower) (GA18-5B)	SHANSEP	19				5	0.25	
	Estuarine (Upper) (GA18-5A)	SHANSEP	19				20	0.45	
	Estuarine (Upper) (GA18-5B)	SHANSEP	19				10	0.26	

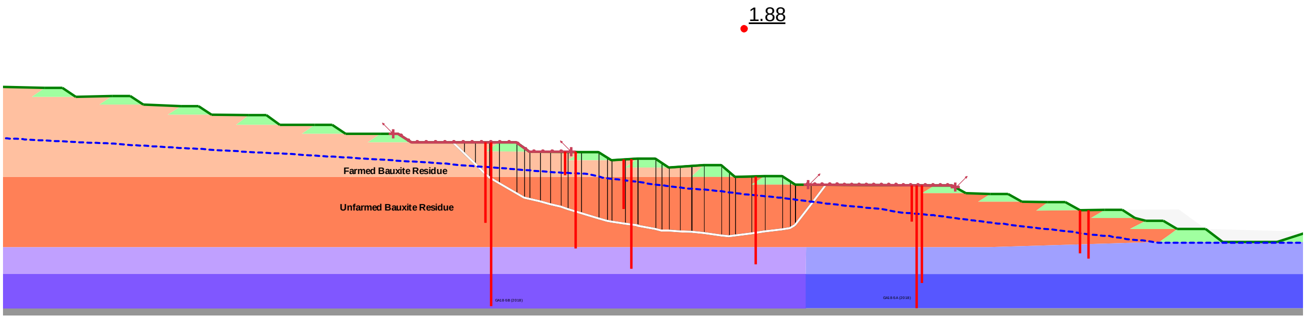


Figure D3: Undrained Static Analysis – Middle Section (30th Percentile) – FoS = 1.88

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.23	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) (GA18-5A)	SHANSEP	19				5	0.29	
	Estuarine (Lower) (GA18-5B)	SHANSEP	19				5	0.25	
	Estuarine (Upper) (GA18-5A)	SHANSEP	19				20	0.45	
	Estuarine (Upper) (GA18-5B)	SHANSEP	19				10	0.26	

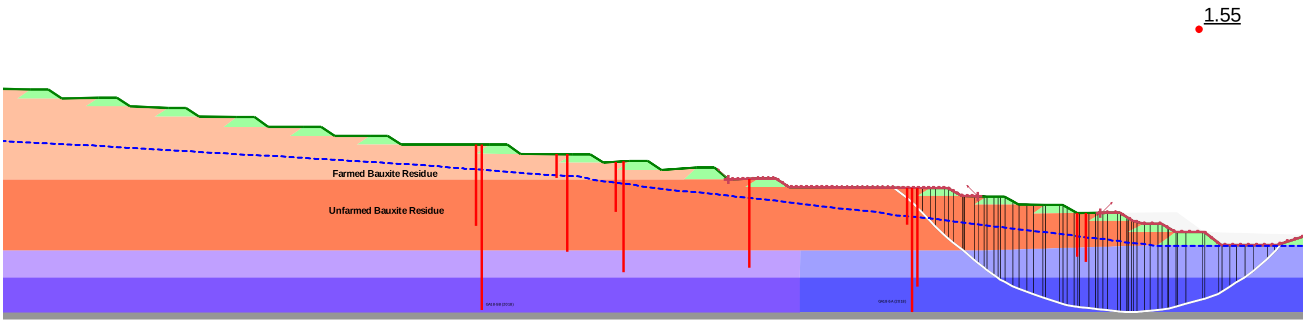


Figure D4: Undrained Static Analysis – Lower Section (30th Percentile) – FoS = 1.55

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.184
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower) (GA18-5A) (PS)	SHANSEP	19				4	0.232
	Estuarine (Lower) (GA18-5B) (PS)	SHANSEP	19				4	0.2
	Estuarine (Upper) (GA18-5A) (PS)	SHANSEP	19				16	0.36
	Estuarine (Upper) (GA18-5B) (PS)	SHANSEP	19				8	0.208

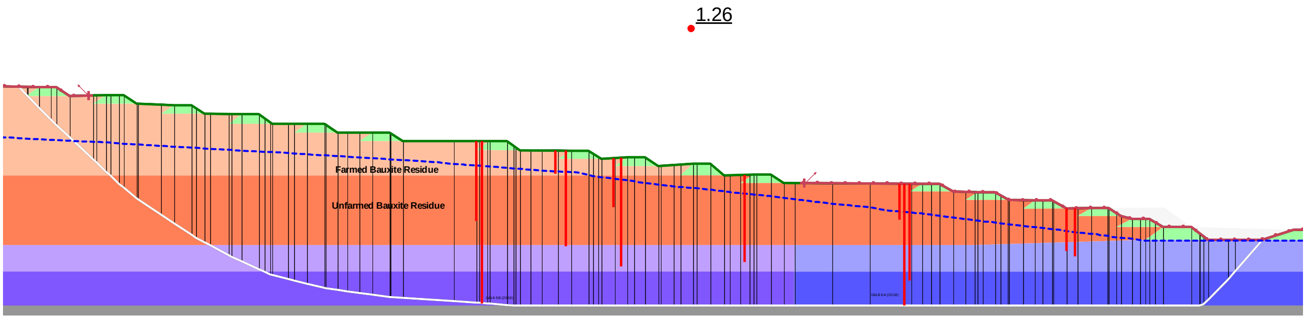


Figure D5: Undrained Post-Earthquake Analysis – Global Section (30th Percentile) – FoS = 1.26

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.184
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower) (GA18-5A) (PS)	SHANSEP	19				4	0.232
	Estuarine (Lower) (GA18-5B) (PS)	SHANSEP	19				4	0.2
	Estuarine (Upper) (GA18-5A) (PS)	SHANSEP	19				16	0.36
	Estuarine (Upper) (GA18-5B) (PS)	SHANSEP	19				8	0.208

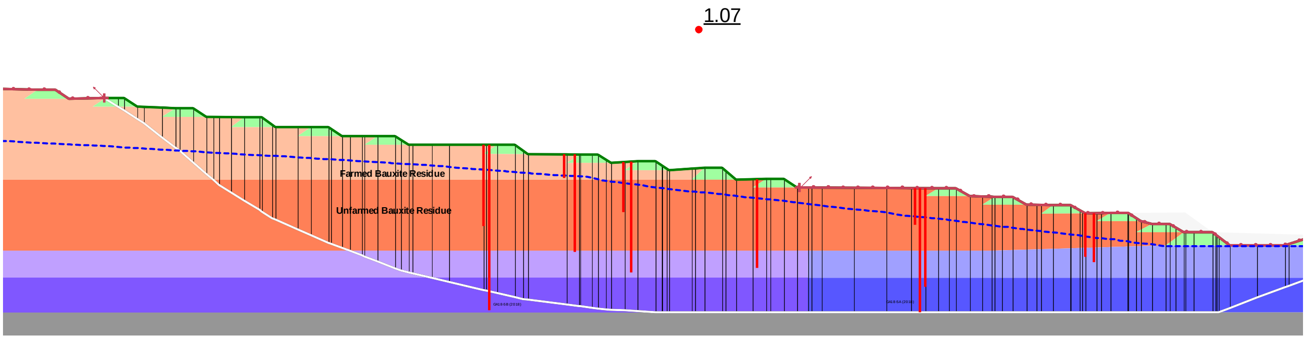


Figure D6: Undrained Pseudo-Static Analysis – Global Section (30th Percentile) – FoS = 1.07

7.5 Section E-E

Table 14: Section E-E - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Strength			Comment
			Undrained			
	Bulk	Dry	10^{th} Percentile	30^{th} Percentile	50^{th} Percentile	
Estuarine (Upper) Deposit	1.94 / 19.0	1.63 / 16.0	s_u/σ'_{v0} = 0.25 to 0.28	s_u/σ'_{v0} = 0.28 to 0.38	s_u/σ'_{v0} = 0.31 to 0.47	Undrained strength from CPTu, shear vane and DSS testing
Estuarine (Lower) Deposit			s_u/σ'_{v0} = 0.15 to 0.20	s_u/σ'_{v0} = 0.25 to 0.45	s_u/σ'_{v0} = 0.27 to 0.54	
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 16.0	s_u/σ'_{v0} = 0.25			Undrained strength from CPTu, shear vane and DSS testing
Farmed Bauxite Residue			s_u/σ'_{v0} = 0.60			

Ø=friction angle; c=cohesion; s_u=undrained shear strength; σ'_{v0}=vertical effective confining stress

Table 15: Section E-E – Stability Analyses Results at Stage 10

Section E-E @ Stage 10	Factor of Safety (Undrained)
	30^{th} Percentile (Estuarine)
Global Slope Stability	1.55 **(E1)**
Upper Slope Stability	1.93 **(E2)**
Lower Slope Stability	1.49 **(E3)**
Post-Seismic Stability	1.24 **(E4)**
Pseudo-Static Stability	1.05 **(E5)**

Table 16: Section E-E - Stability Analyses Results at Stage 16

Section E-E (Stage 16)	Factor of Safety (Undrained)	
	10th Percentile Estuarine	30th Percentile Estuarine
Global Slope Stability	1.09	1.48 **(E6)**
Upper Slope Stability	2.30	2.30 **(E7)**
Middle Slope Stability	1.47	1.61 **(E8)**
Lower Slope Stability	1.05	1.51 **(E9)**
Post Seismic Stability	0.79	1.15 **(E10)**
Pseudo-Static Stability	0.70	0.97 **(E11)**

The stability analyses concludes that the as-constructed Section E-E slope profile attains the target FoS of 1.5 for the BRDA constructed to Stage 10 and for the proposed construction to Stage 16, using the 30th Percentile undrained strength ratio for the estuarine deposits.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				15	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7B	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7C	SHANSEP	19				15	0.45	
	Estuarine (Upper) GA18-7A	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7B	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7C	SHANSEP	19				15	0.38	

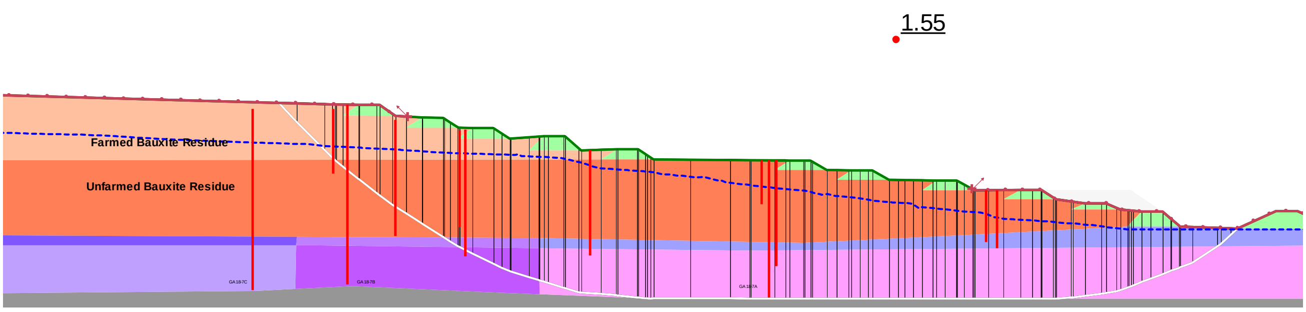


Figure E1: Undrained Static Analysis – Global Section Stage 10 (30th Percentile) – FoS = 1.55

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				15	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7B	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7C	SHANSEP	19				15	0.45	
	Estuarine (Upper) GA18-7A	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7B	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7C	SHANSEP	19				15	0.38	

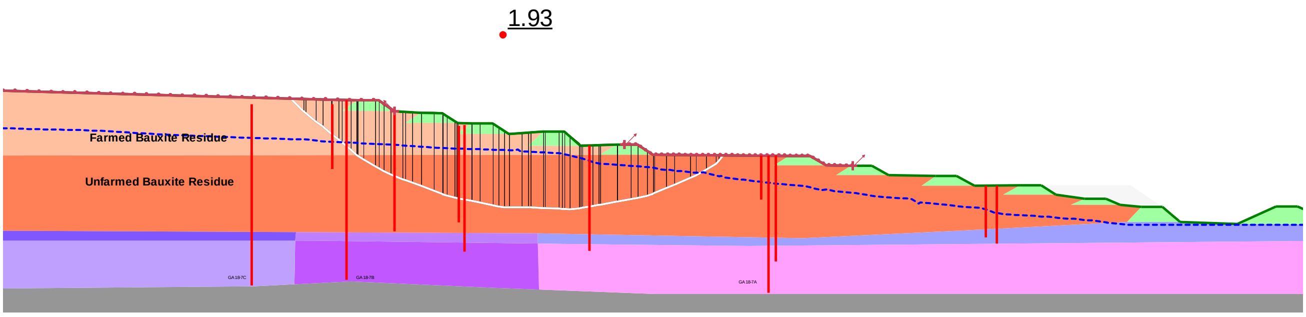


Figure E2: Undrained Static Analysis – Upper Section Stage 10 (30th Percentile) – FoS = 1.93

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				15	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7B	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7C	SHANSEP	19				15	0.45	
	Estuarine (Upper) GA18-7A	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7B	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7C	SHANSEP	19				15	0.38	

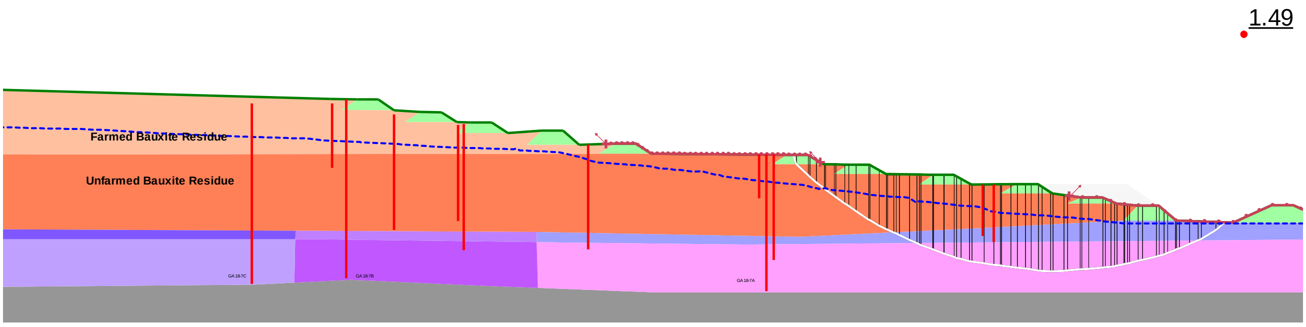


Figure E3: Undrained Static Analysis – Lower Section Stage 10 (30th Percentile) – FoS = 1.49

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	Bauxite Residue
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A (PS)	SHANSEP	19				12	0.2	
	Estuarine (Lower) GA18-7B (PS)	SHANSEP	19				12	0.2	
	Estuarine (Lower) GA18-7C (PS)	SHANSEP	19				12	0.36	
	Estuarine (Upper) GA18-7A (PS)	SHANSEP	19				12	0.224	
	Estuarine (Upper) GA18-7B (PS)	SHANSEP	19				12	0.224	
	Estuarine (Upper) GA18-7C (PS)	SHANSEP	19				12	0.304	

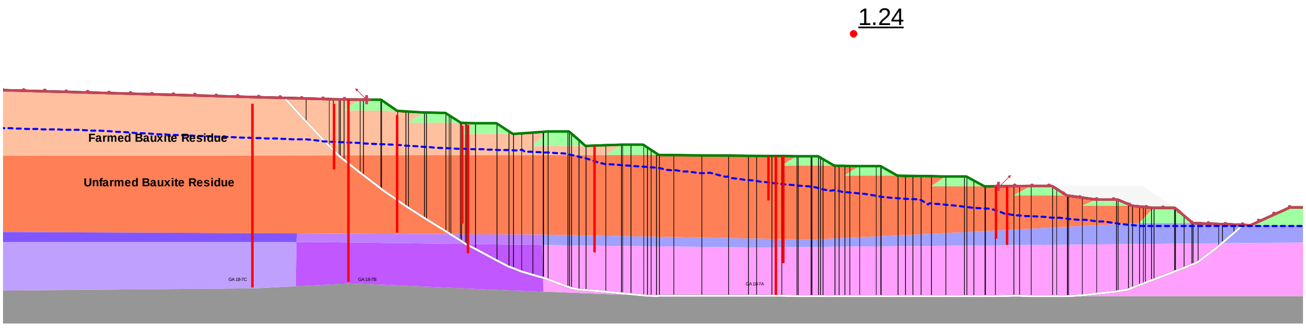


Figure E4: Undrained Post-Earthquake Analysis – Global Section Stage 10 (30th Percentile) – FoS = 1.24

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	Bauxite Residue
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A (PS)	SHANSEP	19				12	0.2	
	Estuarine (Lower) GA18-7B (PS)	SHANSEP	19				12	0.2	
	Estuarine (Lower) GA18-7C (PS)	SHANSEP	19				12	0.36	
	Estuarine (Upper) GA18-7A (PS)	SHANSEP	19				12	0.224	
	Estuarine (Upper) GA18-7B (PS)	SHANSEP	19				12	0.224	
	Estuarine (Upper) GA18-7C (PS)	SHANSEP	19				12	0.304	

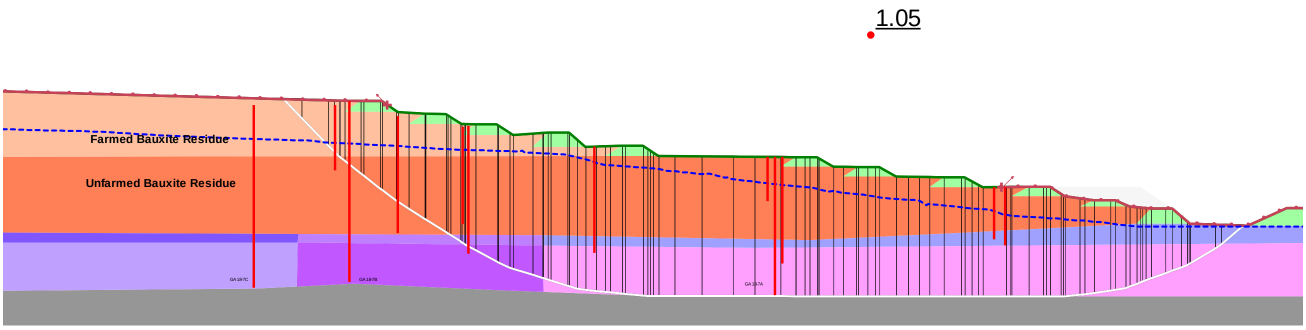


Figure E5: Undrained Pseudo-Static Analysis – Global Section Stage 10 (30th Percentile) – FoS = 1.05

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7B	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7C	SHANSEP	19				15	0.45	
	Estuarine (Upper) GA18-7A	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7B	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7C	SHANSEP	19				15	0.38	

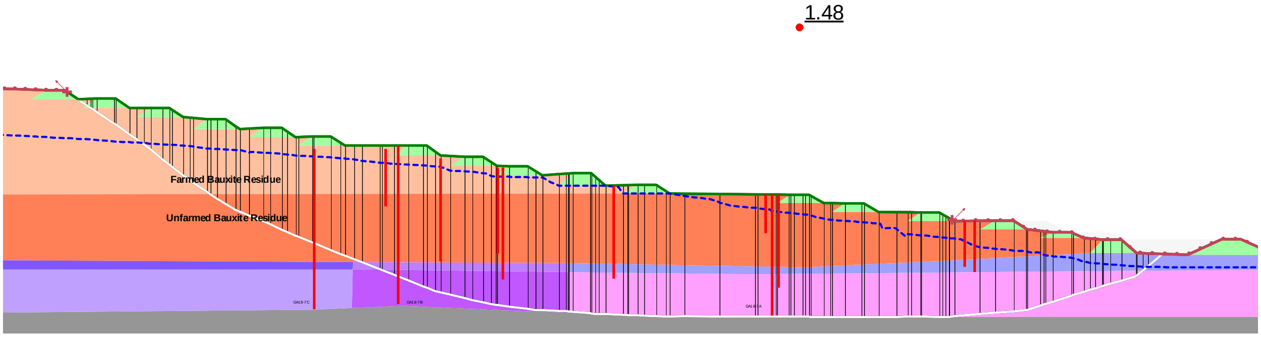


Figure E6: Undrained Static Analysis – Global Section (30th Percentile) – FoS = 1.48

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7B	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7C	SHANSEP	19				15	0.45	
	Estuarine (Upper) GA18-7A	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7B	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7C	SHANSEP	19				15	0.38	

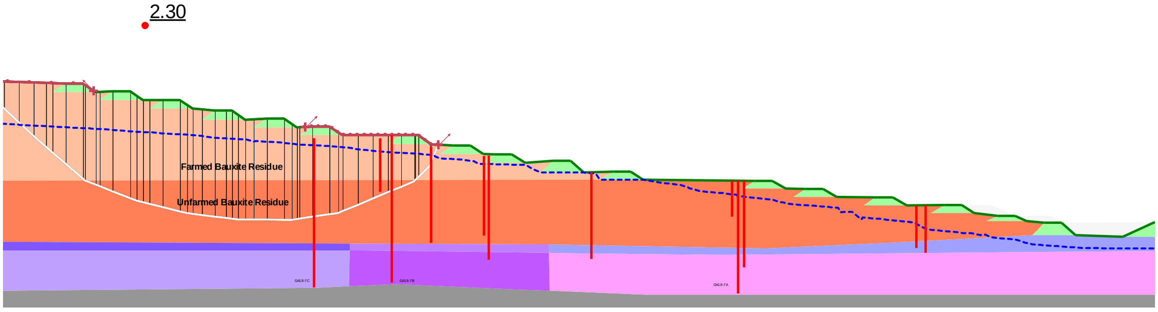


Figure E7: Undrained Static Analysis – Upper Section (30th Percentile) – FoS = 2.30

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7B	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7C	SHANSEP	19				15	0.45	
	Estuarine (Upper) GA18-7A	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7B	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7C	SHANSEP	19				15	0.38	

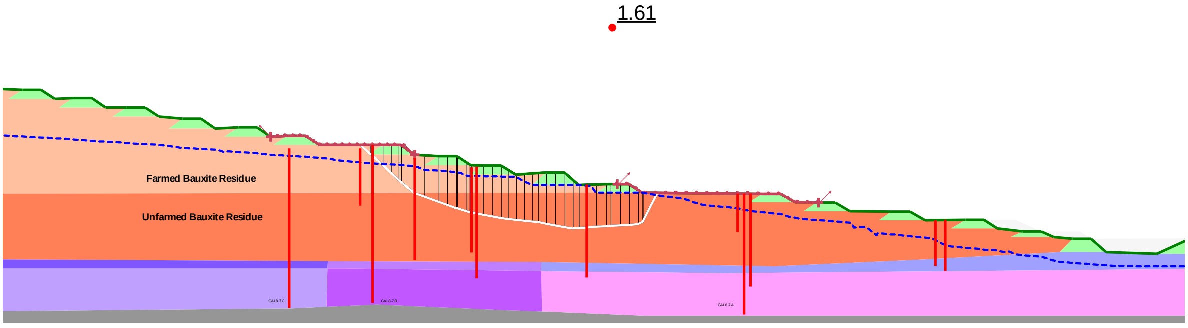


Figure E8: Undrained Static Analysis – Middle Section (30th Percentile) – FoS = 1.61

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Lower) GA18-7A	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7B	SHANSEP	19				15	0.25	
	Estuarine (Lower) GA18-7C	SHANSEP	19				15	0.45	
	Estuarine (Upper) GA18-7A	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7B	SHANSEP	19				15	0.28	
	Estuarine (Upper) GA18-7C	SHANSEP	19				15	0.38	

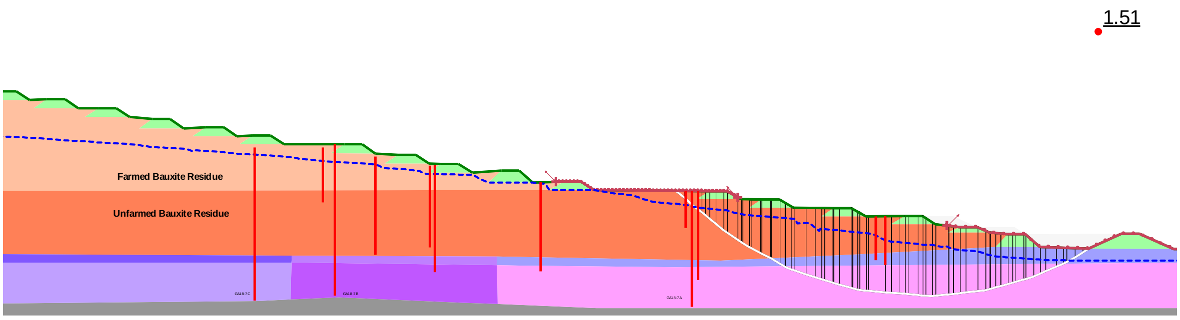


Figure E9: Undrained Static Analysis – Lower Section (30th Percentile) – FoS = 1.51

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower) GA18-7A (PS)	SHANSEP	19				12	0.2
	Estuarine (Lower) GA18-7B (PS)	SHANSEP	19				12	0.2
	Estuarine (Lower) GA18-7C (PS)	SHANSEP	19				12	0.36
	Estuarine (Upper) GA18-7A (PS)	SHANSEP	19				12	0.224
	Estuarine (Upper) GA18-7B (PS)	SHANSEP	19				12	0.224
	Estuarine (Upper) GA18-7C (PS)	SHANSEP	19				12	0.304

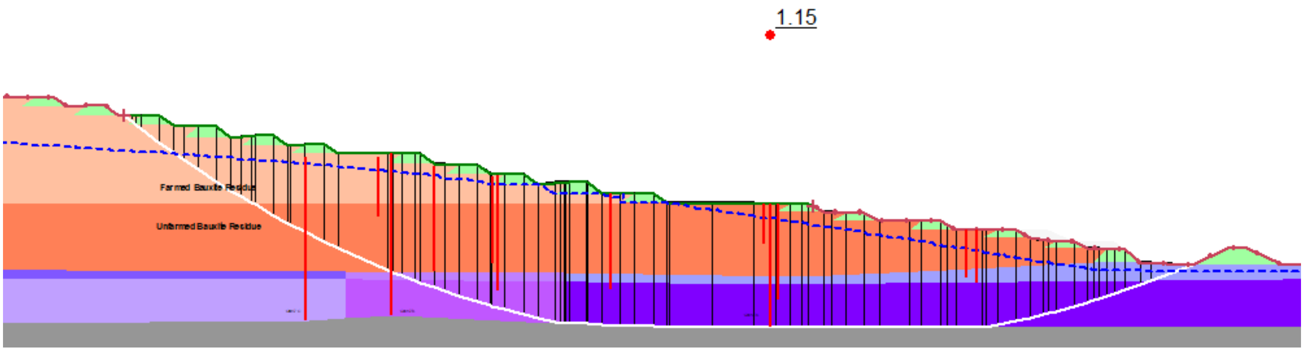


Figure E10: Undrained Post-Earthquake Analysis – Global Section (30th Percentile) – FoS = 1.15

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Lower) GA18-7A (PS)	SHANSEP	19				12	0.2
	Estuarine (Lower) GA18-7B (PS)	SHANSEP	19				12	0.2
	Estuarine (Lower) GA18-7C (PS)	SHANSEP	19				12	0.36
	Estuarine (Upper) GA18-7A (PS)	SHANSEP	19				12	0.224
	Estuarine (Upper) GA18-7B (PS)	SHANSEP	19				12	0.224
	Estuarine (Upper) GA18-7C (PS)	SHANSEP	19				12	0.304

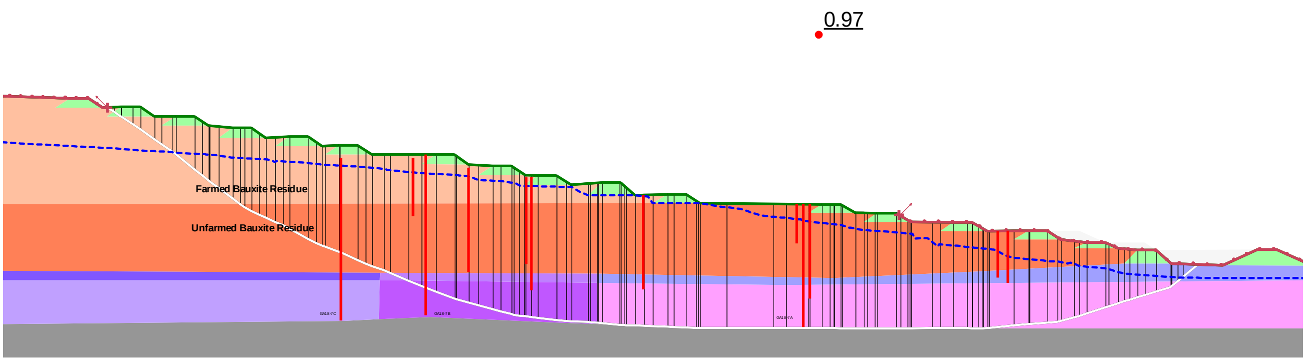


Figure E11: Section E-E; Undrained Pseudo-Static Analysis – Global Section (30th Percentile) – FoS = 0.97

7.6 Section F-F

Table 17: Section F-F - Characteristic Geotechnical Parameters

<table>
<tr><th rowspan="3">Material Property</th><th colspan="2" rowspan="2">Density / Unit Weight (Mg/m3 / kN/m3)</th><th colspan="4">Strength</th><th rowspan="3">Comment</th></tr>
<tr><th colspan="3">Undrained</th><th rowspan="2">Drained</th></tr>
<tr><th>Bulk</th><th>Dry</th><th>10th Percentile</th><th>30th Percentile</th><th>50th Percentile</th></tr>
<tr><td>Estuarine Deposit</td><td>1.94 / 19.0</td><td>1.63 / 16.0</td><td>s_u/σ'_{v0} = 0.25 to 0.50</td><td>-</td><td>-</td><td>-</td><td>Undrained strength from CPTu, shear vane and DSS testing</td></tr>
<tr><td>Rock fill</td><td>22</td><td></td><td></td><td></td><td></td><td>Ø = 45°</td><td>Typical for rock fill</td></tr>
<tr><td>Process Sand</td><td>19</td><td>-</td><td>-</td><td>-</td><td>-</td><td>Ø = 33°</td><td>Golder Direct Shear Testing in 2019</td></tr>
<tr><td>Sludge Disposal Area</td><td>22</td><td>-</td><td colspan="3">s_u/σ'_{v0} = 0.15 (Horizontal Shear)
s_u/σ'_{v0} = 0.30 (Compression)</td><td>-</td><td>Average based on 2018 CPTu data Sensitivity Analyses Conducted due to high variability in strength observed</td></tr>
<tr><td>Unfarmed Bauxite Residue</td><td rowspan="3">2.19 / 21.5</td><td rowspan="3">1.63 / 16.0</td><td colspan="3">s_u/σ'_{v0} = 0.25 (Horizontal Shear)
s_u/σ'_{v0} = 0.50 (Compression)</td><td>-</td><td rowspan="2">Undrained strength from CPTu, shear vane and DSS testing</td></tr>
<tr><td>Farmed Bauxite Residue</td><td colspan="3">s_u/σ'_{v0} = 0.60 (Horizontal Shear)</td><td>-</td></tr>
<tr><td>Unfarmed Bauxite Residue Wick Drain Area</td><td colspan="3">s_u/σ'_{v0} = 0.30 (Horizontal Shear)
s_u/σ'_{v0} = 0.60 (Compression)</td><td>-</td><td>Increased strength as a result of wick drain installation Based on 2019 CPTu data</td></tr>
</table>

Ø=friction angle; c=cohesion; s_u=undrained shear strength; σ'_{v0}=vertical effective confining stress

Table 18: Section F-F – Stability Analyses Results at Stage 10

Section F-F @ Stage 10	Factor of Safety (Undrained) 10th Percentile (Estuarine)
Global Slope Stability	1.55 **(F1)**
Upper Slope Stability	1.48 **(F2)**
Lower Slope Stability	1.50 **(F3)**
Post-Seismic Stability	1.15 **(F4)**
Pseudo-Static Stability	1.10 **(F5)**

Table 19: Section F-F – Stability Analyses Results at Stage 16

Section F-F (Stage 16)	Factor of Safety (Undrained) 10th Percentile Estuarine
Global Slope Stability	1.57 **(F6)**
Upper Slope Stability	2.38 **(F7)**
Middle Slope Stability	2.58 **(F8)**
Lower Slope Stability	1.49 **(F9)**
Post Seismic Stability	1.02 **(F10)**
Pseudo-Static Stability	1.26 **(F11)**

The stability analyses concludes that the as-constructed Section F-F slope profile attains the target FoS of 1.5 for the BRDA constructed to Stage 10 and for the proposed construction to Stage 16, using the 10th Percentile undrained strength ratio for the estuarine deposits.

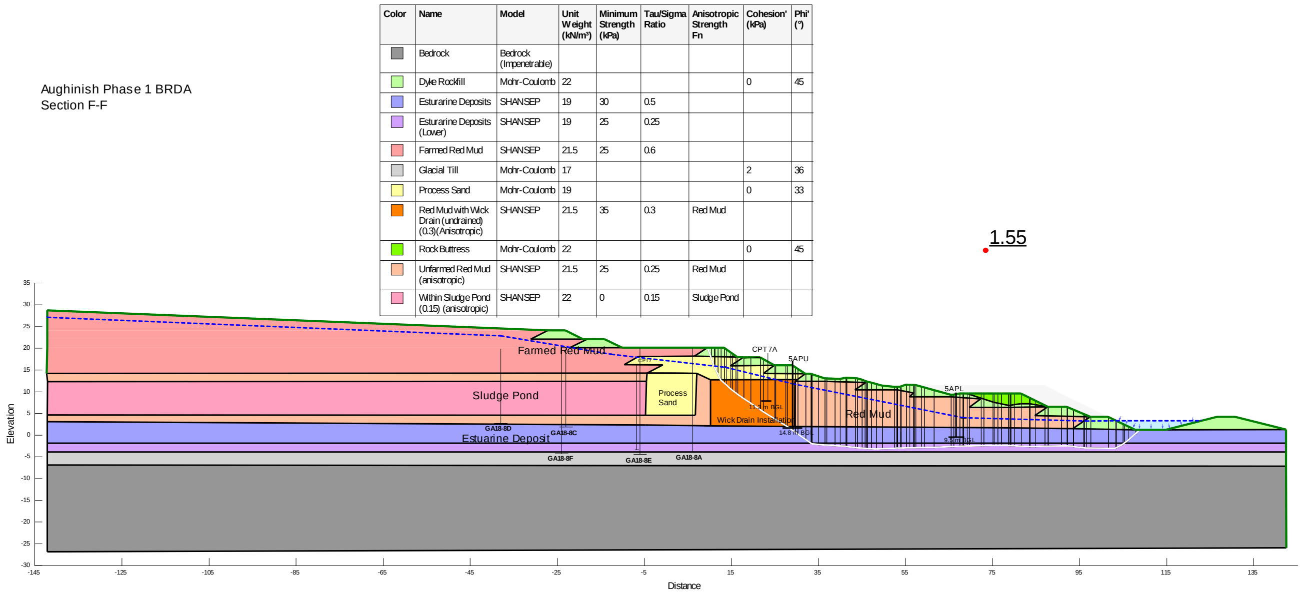


Figure F1: Undrained Static Analysis – Global Section Stage 10 (10th Percentile) – FoS = 1.55

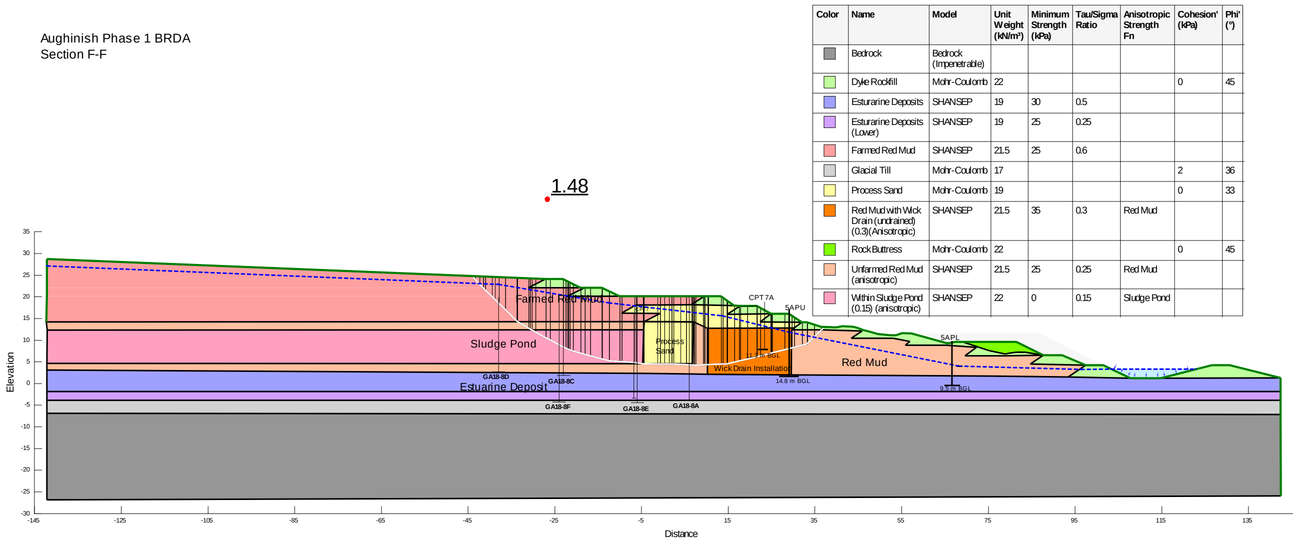
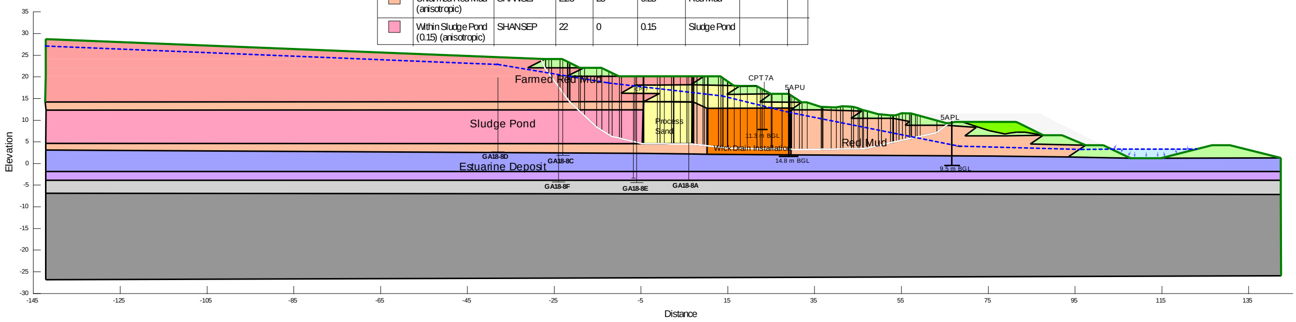
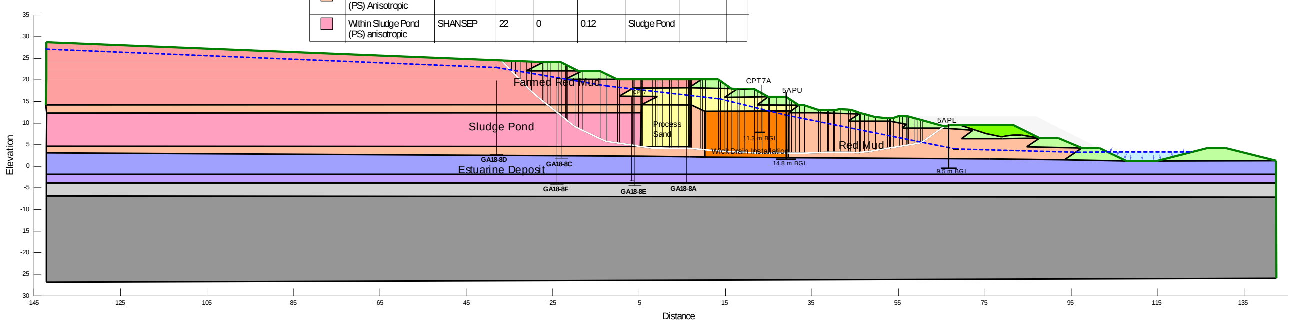


Figure F2: Undrained Static Analysis – Upper Section Stage 10 (10th Percentile) – FoS = 1.48



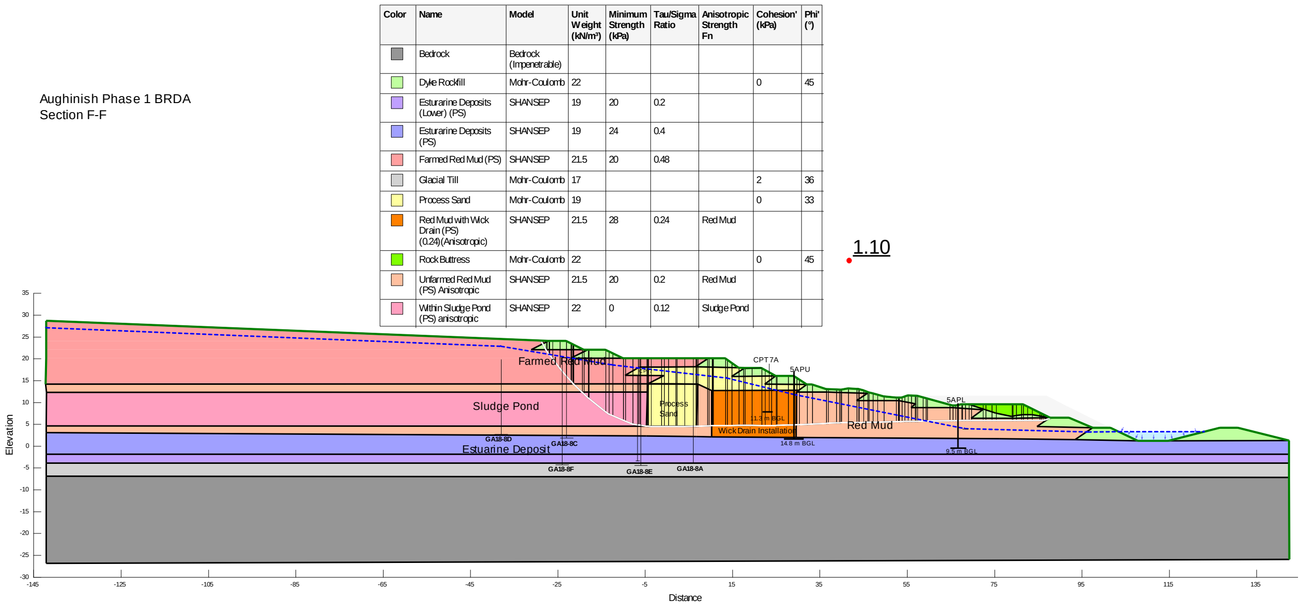
Color	Name	Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22				0	45
	Esturarine Deposits	SHANSEP	19	30	0.5			
	Esturarine Deposits (Lower)	SHANSEP	19	25	0.25			
	Farmed Red Mud	SHANSEP	21.5	25	0.6			
	Glacial Till	Mohr-Coulomb	17				2	36
	Process Sand	Mohr-Coulomb	19				0	33
	Red Mud with Wick Drain (undrained) (0.3)(Anisotropic)	SHANSEP	21.5	35	0.3	Red Mud		
	Rock Buttress	Mohr-Coulomb	22				0	45
	Unfarmed Red Mud (anisotropic)	SHANSEP	21.5	25	0.25	Red Mud		
	Within Sludge Pond (0.15) (anisotropic)	SHANSEP	22	0	0.15	Sludge Pond		

Figure F3: Undrained Static Analysis – Lower Section Stage 10 (10th Percentile) – FoS = 1.50



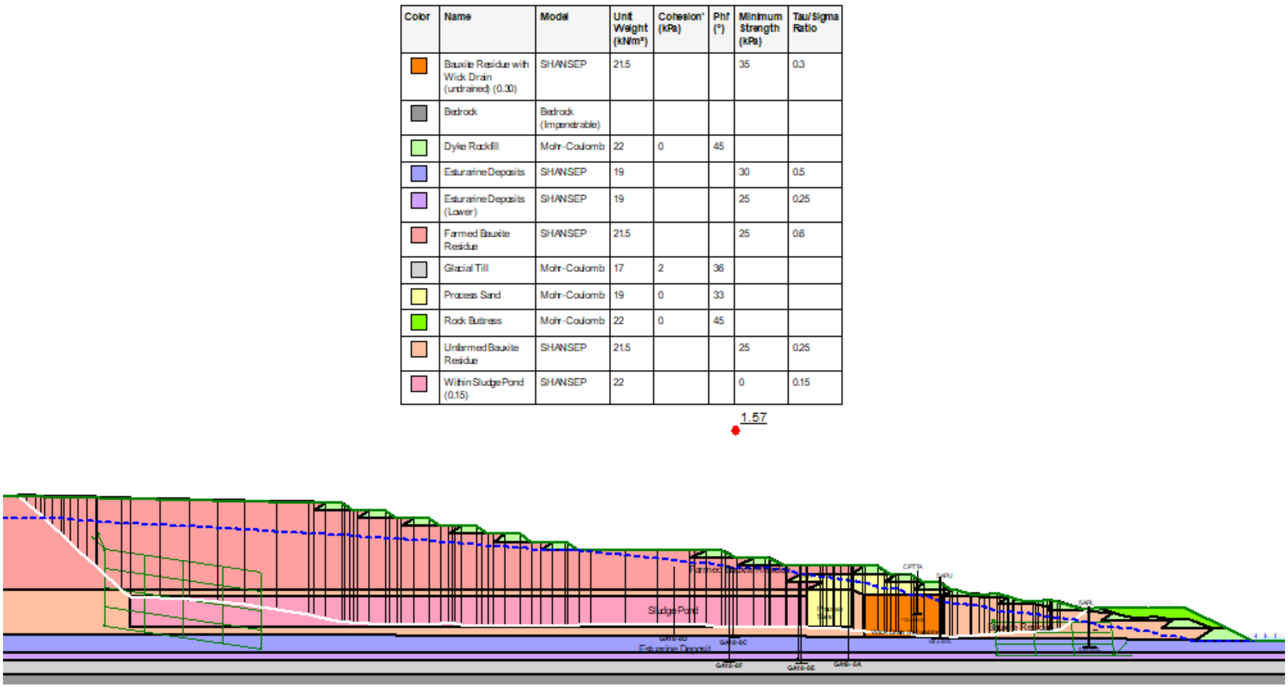
Color	Name	Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22				0	45
	Esturarine Deposits (Lower) (PS)	SHANSEP	19	20	0.2			
	Esturarine Deposits (PS)	SHANSEP	19	24	0.4			
	Farmed Red Mud (PS)	SHANSEP	21.5	20	0.48			
	Glacial Till	Mohr-Coulomb	17				2	36
	Process Sand	Mohr-Coulomb	19				0	33
	Red Mud with Wick Drain (PS) (0.24)(Anisotropic)	SHANSEP	21.5	28	0.24	Red Mud		
	Rock Buttress	Mohr-Coulomb	22				0	45
	Unfarmed Red Mud (PS) Anisotropic	SHANSEP	21.5	20	0.2	Red Mud		
	Within Sludge Pond (PS) anisotropic	SHANSEP	22	0	0.12	Sludge Pond		

Figure F4: Undrained Post-Earthquake Analysis – Global Section Stage 10 (10th Percentile) – FoS = 1.15



Color	Name	Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22				0	45
	Esturarine Deposits (Lower) (PS)	SHANSEP	19	20	0.2			
	Esturarine Deposits (PS)	SHANSEP	19	24	0.4			
	Farmed Red Mud (PS)	SHANSEP	21.5	20	0.48			
	Glacial Till	Mohr-Coulomb	17				2	36
	Process Sand	Mohr-Coulomb	19				0	33
	Red Mud with Wick Drain (PS) (0.24)(Anisotropic)	SHANSEP	21.5	28	0.24	Red Mud		
	Rock Buttress	Mohr-Coulomb	22				0	45
	Unfarmed Red Mud (PS) Anisotropic	SHANSEP	21.5	20	0.2	Red Mud		
	Within Sludge Pond (PS) anisotropic	SHANSEP	22	0	0.12	Sludge Pond		

Figure F5: Undrained Pseudo-Static Analysis – Global Section Stage 10 (10th Percentile) – FoS = 1.10



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue with Wick Drain (undrained) (0.30)	SHANSEP	21.5			35	0.3
	Bedrock	Bedrock (Impenetrable)					
	Dyke Rockfill	Mohr-Coulomb	22	0	45		
	Esturarine Deposits	SHANSEP	19			30	0.5
	Esturarine Deposits (Lower)	SHANSEP	19			25	0.25
	Farmed Bauxite Residue	SHANSEP	21.5			25	0.6
	Glacial Till	Mohr-Coulomb	17	2	36		
	Process Sand	Mohr-Coulomb	19	0	33		
	Rock Buttress	Mohr-Coulomb	22	0	45		
	Unfarmed Bauxite Residue	SHANSEP	21.5			25	0.25
	Within Sludge Pond (0.15)	SHANSEP	22			0	0.15

Figure F6: Undrained Static Analysis – Global Section with Buttress (10th Percentile) – FoS = 1.57

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue with Wick Drain (undrained) (0.30)	SHANSEP	21.5			35	0.3
	Bedrock	Bedrock (Impenetrable)					
	Dyke Rockfill	Mohr-Coulomb	22	0	45		
	Esturarine Deposits	SHANSEP	19			30	0.5
	Esturarine Deposits (Lower)	SHANSEP	19			25	0.25
	Farmed Bauxite Residue	SHANSEP	21.5			25	0.6
	Glacial Till	Mohr-Coulomb	17	2	36		
	Process Sand	Mohr-Coulomb	19	0	33		
	Rock Buttress	Mohr-Coulomb	22	0	45		
	Unfarmed Bauxite Residue	SHANSEP	21.5			25	0.25
	Within Sludge Pond (0.15)	SHANSEP	22			0	0.15

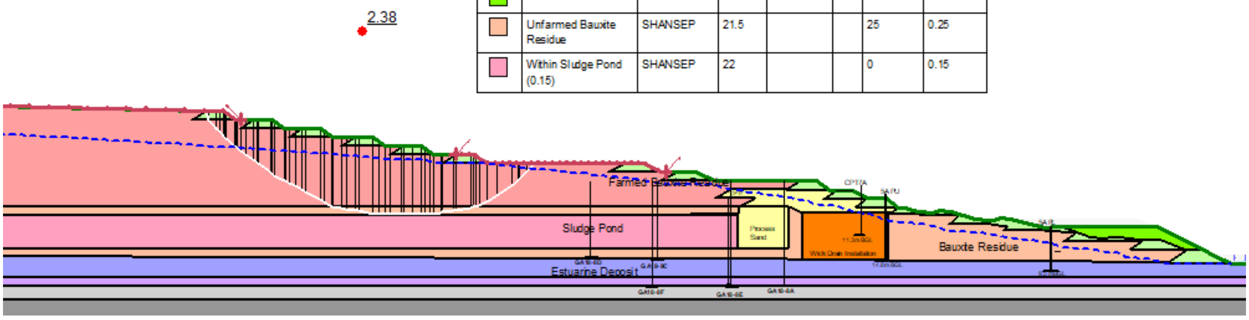


Figure F7: Undrained Static Analysis – Upper Section with Buttress (10th Percentile) – FoS = 2.38

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue with Wick Drain (undrained) (0.30)	SHANSEP	21.5			35	0.3
	Bedrock	Bedrock (Impenetrable)					
	Dyke Rockfill	Mohr-Coulomb	22	0	45		
	Esturarine Deposits	SHANSEP	19			30	0.5
	Esturarine Deposits (Lower)	SHANSEP	19			25	0.25
	Farmed Bauxite Residue	SHANSEP	21.5			25	0.6
	Glacial Till	Mohr-Coulomb	17	2	36		
	Process Sand	Mohr-Coulomb	19	0	33		
	Rock Buttress	Mohr-Coulomb	22	0	45		
	Unfarmed Bauxite Residue	SHANSEP	21.5			25	0.25
	Within Sludge Pond (0.15)	SHANSEP	22			0	0.15

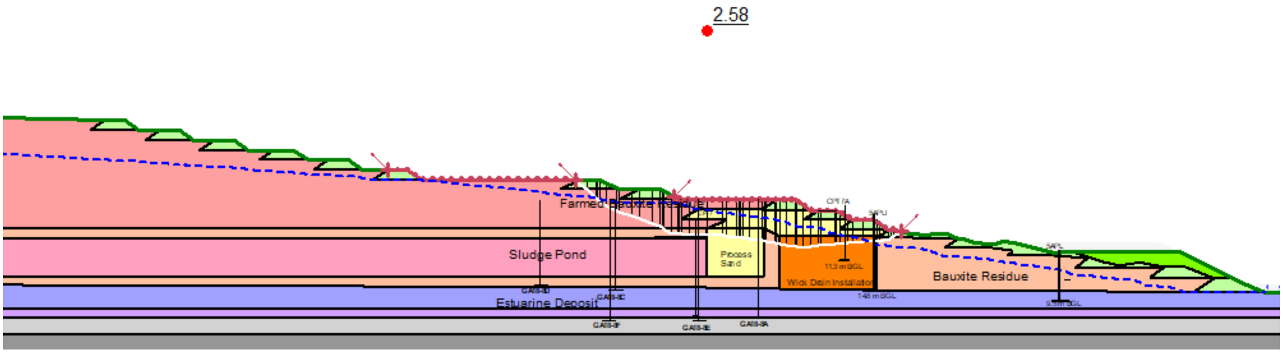


Figure F8: Undrained Static Analysis – Middle Section with Buttress (10th Percentile) – FoS = 2.58

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue with Wick Drain (undrained) (0.30)	SHANSEP	21.5			35	0.3
	Bedrock	Bedrock (Impenetrable)					
	Dyke Rockfill	Mohr-Coulomb	22	0	45		
	Estuarine Deposits	SHANSEP	19			30	0.5
	Estuarine Deposits (Lower)	SHANSEP	19			25	0.25
	Farmed Bauxite Residue	SHANSEP	21.5			25	0.6
	Glacial Till	Mohr-Coulomb	17	2	36		
	Process Sand	Mohr-Coulomb	19	0	33		
	Rock Buttress	Mohr-Coulomb	22	0	45		
	Unfarmed Bauxite Residue	SHANSEP	21.5			25	0.25
	Within Sludge Pond (0.15)	SHANSEP	22			0	0.15

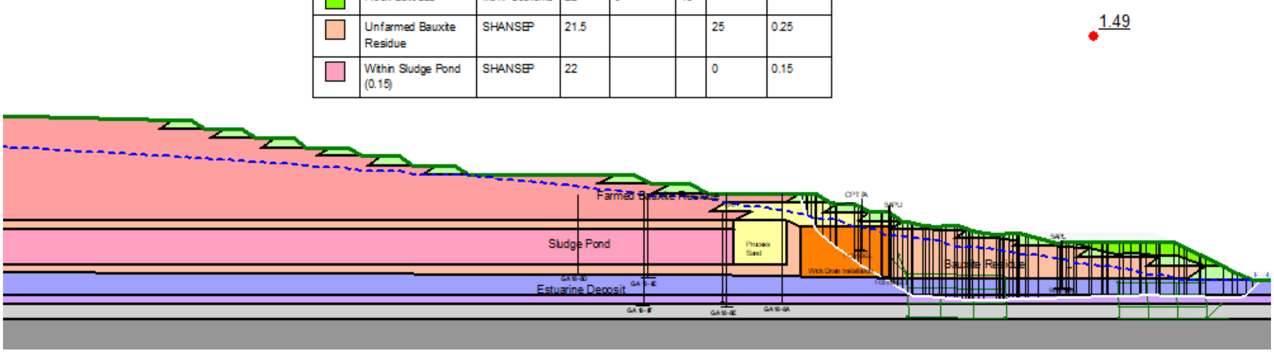


Figure F9: Undrained Static Analysis – Lower Section with Buttress (10th Percentile) – FoS = 1.49

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue with Wick Drain (undrained) (0.30) (PS)	SHANSEP	21.5			28	0.24
	Bedrock	Bedrock (Impenetrable)					
	Dyke Rockfill	Mohr-Coulomb	22	0	45		
	Esturaine Deposits (Lower) (PS)	SHANSEP	19			20	0.2
	Esturaine Deposits (PS)	SHANSEP	19			24	0.4
	Farmed Bauxite Residue (PS)	SHANSEP	21.5			20	0.48
	Glacial Till	Mohr-Coulomb	17	2	36		
	Process Sand	Mohr-Coulomb	19	0	33		
	Rock Buttress	Mohr-Coulomb	22	0	45		
	Unfarmed Bauxite Residue (PS)	SHANSEP	21.5			20	0.2
	Within Sludge Pond (PS)	SHANSEP	22			0	0.12

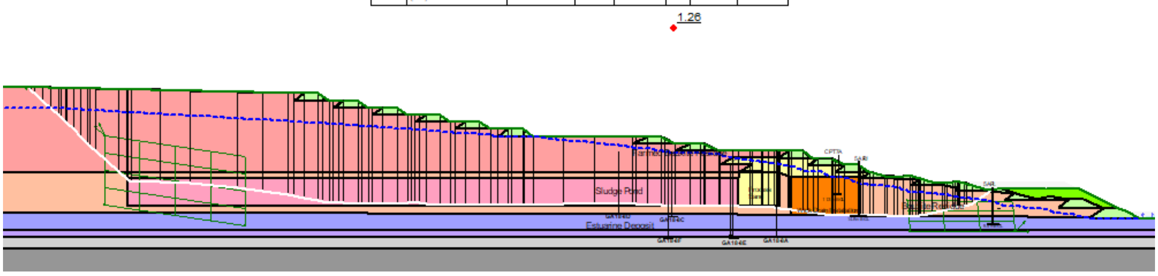


Figure F10: Undrained Post-Earthquake Analysis – Global Section with Buttress (10th Percentile) – FoS = 1.26

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue with Wick Drain (undrained) (0.30) (PS)	SHANSEP	21.5			28	0.24
	Bedrock	Bedrock (Impenetrable)					
	Dyke Rockfill	Mohr-Coulomb	22	0	45		
	Esturarine Deposits (Lower) (PS)	SHANSEP	19			20	0.2
	Esturarine Deposits (PS)	SHANSEP	19			24	0.4
	Farmed Bauxite Residue (PS)	SHANSEP	21.5			20	0.48
	Glacial Till	Mohr-Coulomb	17	2	36		
	Process Sand	Mohr-Coulomb	19	0	33		
	Rock Buttress	Mohr-Coulomb	22	0	45		
	Unfarmed Bauxite Residue (PS)	SHANSEP	21.5			20	0.2
	Within Sludge Pond (PS)	SHANSEP	22			0	0.12

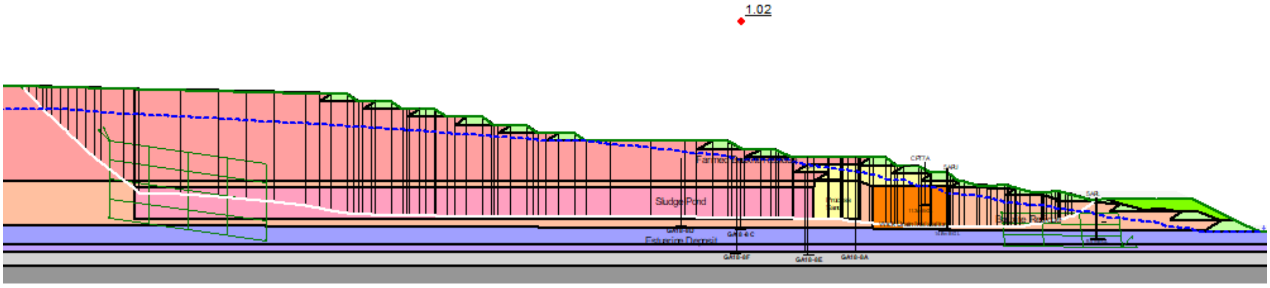


Figure F11: Undrained Pseudo-Static Analysis – Global Section with Buttress (10th Percentile) – FoS = 1.02

7.7 Section K-K

Table 20: Section K-K - Characteristic Geotechnical Parameters

<table>
<tr><th rowspan="3">Material Property</th><th colspan="2">Density / Unit Weight (Mg/m3 / kN/m3)</th><th colspan="4">Strength</th><th rowspan="3">Comment</th></tr>
<tr><th rowspan="2">Bulk</th><th rowspan="2">Dry</th><th colspan="3">Undrained</th><th rowspan="2">Drained</th></tr>
<tr><th>10th Percentile</th><th>30th Percentile</th><th>50th Percentile</th></tr>
<tr><td>Process Sand</td><td>19</td><td>-</td><td>N/A</td><td>N/A</td><td>N/A</td><td>$\Phi = 33°$</td><td>Golder Direct Shear Testing in 2019</td></tr>
<tr><td>Unfarmed Bauxite Residue</td><td rowspan="2">2.19 / 21.5</td><td rowspan="2">1.63 / 16.0</td><td colspan="3">$s_u/\sigma'_{v0} = 0.25$</td><td>-</td><td rowspan="2">Undrained strength from CPTu, shear vane and DSS testing</td></tr>
<tr><td>Farmed Bauxite Residue</td><td colspan="3">$s_u/\sigma'_{v0} = 0.60$</td><td>-</td></tr>
</table>

Table 21: Section K-K – Stability Analyses Results at Stage 16

Section K-K (Stage 16)	Factor of Safety (Undrained)
	No Estuarine
Global Slope Stability	2.58 **(K1)**
Upper Slope Stability	2.39 **(K2)**
Middle Slope Stability	1.52 **(K3)**
Lower Slope Stability	1.76 **(K4)**
Post Seismic Stability	2.02 **(K5)**
Pseudo-Static Stability	1.55 **(K6)**

The stability analyses concludes that the Section K-K slope profile will attain the target FoS of 1.5 for the BRDA constructed to Stage 16.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) Anisotropic	SHANSEP	21.5				10	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6		1
	Bedrock	Bedrock (Impenetrable)								1
	Clay	Undrained (Phi=0)	18			28				1
	Dyke Rockfill	Mohr-Coulomb	22	0	45					1
	Estuarine Deposits	SHANSEP	19				25	0.25		1
	Process Sand	Mohr-Coulomb	19	0	33					1
	Salt Cake	SHANSEP	12				5	0.05		2

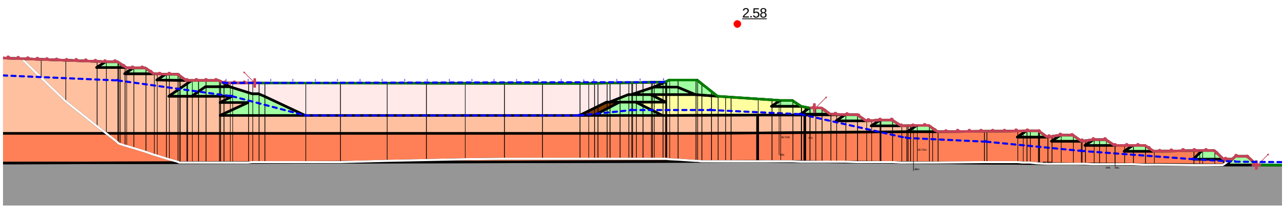


Figure K1: Undrained Static Analysis – Global Section (10th Percentile) – FoS = 2.58

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) Anisotropic	SHANSEP	21.5				10	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6		1
	Bedrock	Bedrock (Impenetrable)								1
	Clay	Undrained (Phi=0)	18			28				1
	Dyke Rockfill	Mohr-Coulomb	22	0	45					1
	Estuarine Deposits	SHANSEP	19				25	0.25		1
	Process Sand	Mohr-Coulomb	19	0	33					1
	Salt Cake	SHANSEP	12				5	0.05		2

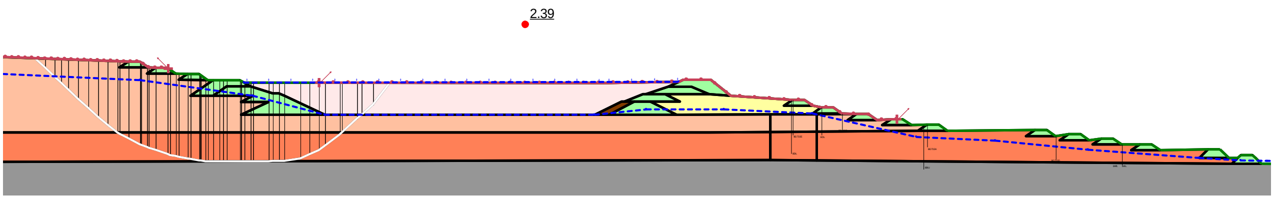


Figure K2: Undrained Static Analysis – Upper Section (10th Percentile) – FoS = 2.39

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) Anisotropic	SHANSEP	21.5				10	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6		1
	Bedrock	Bedrock (Impenetrable)								1
	Clay	Undrained (Phi=0)	18			28				1
	Dyke Rockfill	Mohr-Coulomb	22	0	45					1
	Estuarine Deposits	SHANSEP	19				25	0.25		1
	Process Sand	Mohr-Coulomb	19	0	33					1
	Salt Cake	SHANSEP	12				5	0.05		2

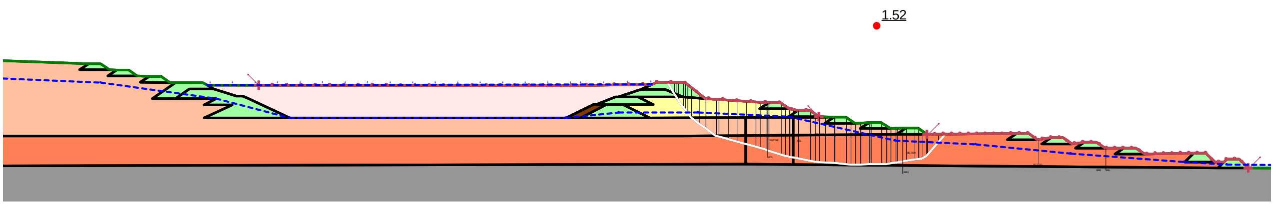


Figure K3: Undrained Static Analysis – Middle Section (10th Percentile) – FoS = 1.52

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) Anisotropic	SHANSEP	21.5				10	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				15	0.6		1
	Bedrock	Bedrock (Impenetrable)								1
	Clay	Undrained (Phi=0)	18			28				1
	Dyke Rockfill	Mohr-Coulomb	22	0	45					1
	Estuarine Deposits	SHANSEP	19				25	0.25		1
	Process Sand	Mohr-Coulomb	19	0	33					1
	Salt Cake	SHANSEP	12				5	0.05		2

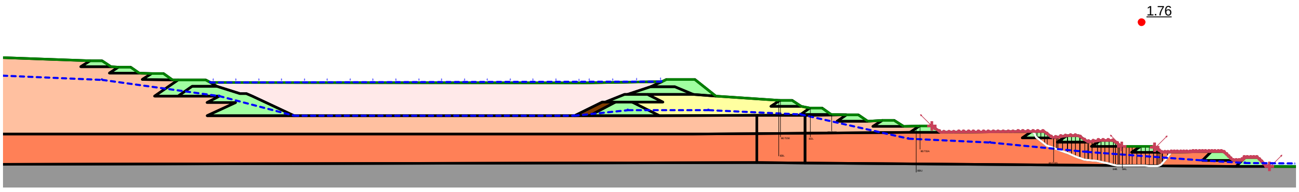


Figure K4: Section K-K; Undrained Static Analysis – Lower Section (10th Percentile) – FoS = 1.76

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	1
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				20	0.48	1
	Bedrock	Bedrock (Impenetrable)							1
	Clay	Undrained (Phi=0)	18			28			1
	Dyke Rockfill	Mohr-Coulomb	22	0	45				1
	Estuarine Deposits (PS)	SHANSEP	19				20	0.2	1
	Process Sand	Mohr-Coulomb	19	0	33				1
	Salt Cake	SHANSEP	12				5	0.05	2

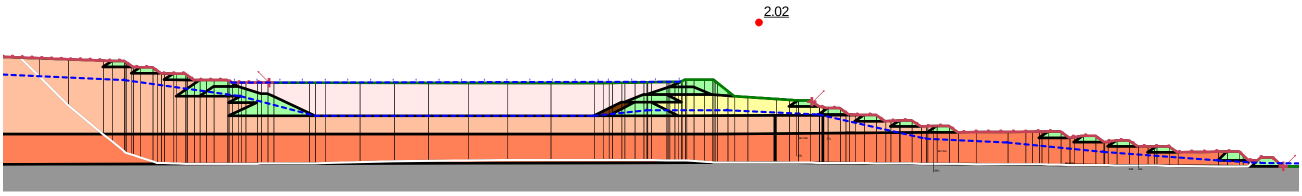


Figure K5: Undrained Post-Earthquake Analysis – Global Section (10th Percentile) – FoS = 2.02

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	1
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				20	0.48	1
	Bedrock	Bedrock (Impenetrable)							1
	Clay	Undrained (Phi=0)	18			28			1
	Dyke Rockfill	Mohr-Coulomb	22	0	45				1
	Estuarine Deposits (PS)	SHANSEP	19				20	0.2	1
	Process Sand	Mohr-Coulomb	19	0	33				1
	Salt Cake	SHANSEP	12				5	0.05	2

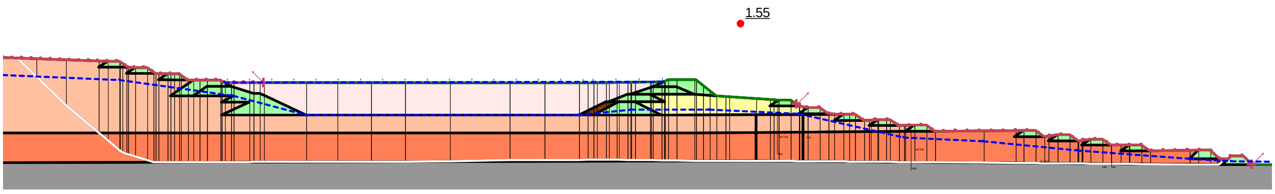


Figure K6: Undrained Pseudo-Static Analysis – Global Section (10th Percentile) – FoS = 1.55

7.8 Section L-L

Table 22: Section K-K - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Strength				Comment
			Undrained			Drained	
	Bulk	Dry	10th Percentile	30th Percentile	50th Percentile		
Glacial Till Deposit	1.94 / 19.0	1.63 / 16.0	N/A	N/A	N/A	Φ = 32°	Undrained strength from CPTu, shear vane and DSS testing
Process Sand	19		N/A	N/A	N/A	Φ = 33°	Golder Direct Shear Testing in 2019
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 16.0	s_u/σ'_{v0} = 0.25			-	Undrained strength from CPTu, shear vane and DSS testing
Farmed Bauxite Residue			s_u/σ'_{v0} = 0.60			-	

Table 23: Section L-L – Stability Analyses Results at Stage 16

Section K-K (Stage 16)	Factor of Safety (Undrained) (No Estuarine)
Global Slope Stability	3.28 **(L1)**
Upper Slope Stability	2.39 **(L2)**
Middle Slope Stability	1.52 **(L3)**
Lower Slope Stability	1.76 **(L4)**
Post Seismic Stability	3.92 **(L5)**
Pseudo-Static Stability	2.85 **(L6)**

The stability analyses concludes that the Section L-L slope profile will attain the target FoS of 1.5 for the BRDA constructed to Stage 16.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (Farmed)	SHANSEP	21.5			12	0.6		1
	Bauxite Residue (UnFarmed)	SHANSEP	21.5			10	0.25	Bauxite Residue	2
	Bedrock	Bedrock (Impenetrable)							2
	Clay	Mohr-Coulomb	18	0	28				2
	Dyke Rockfill	Mohr-Coulomb	22	0	45				2
	Glacial Till	Mohr-Coulomb	19	0	32				1
	Process Sand	Mohr-Coulomb	19	0	33				2
	Rockfill	Mohr-Coulomb	22	0	45				1
	Salt Cake	SHANSEP	12			5	0.05		1

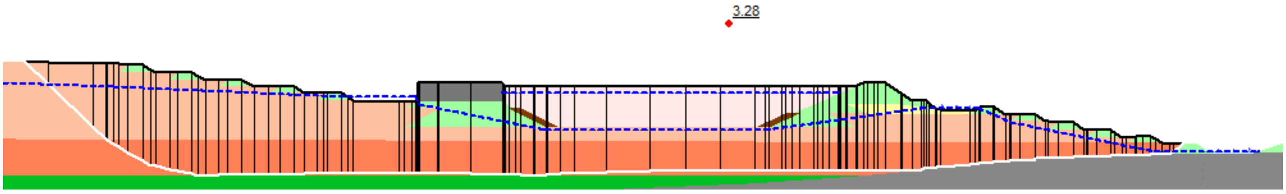


Figure L1: Undrained Static Analysis – Global Section Stage 16 – FoS = 3.28

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (Farmed)	SHANSEP	21.5			12	0.6		1
	Bauxite Residue (UnFarmed)	SHANSEP	21.5			10	0.25	Bauxite Residue	2
	Bedrock	Bedrock (Impenetrable)							2
	Clay	Mohr-Coulomb	18	0	28				2
	Dyke Rockfill	Mohr-Coulomb	22	0	45				2
	Glacial Till	Mohr-Coulomb	19	0	32				1
	Process Sand	Mohr-Coulomb	19	0	33				2
	Rockfill	Mohr-Coulomb	22	0	45				1
	Salt Cake	SHANSEP	12			5	0.05		1

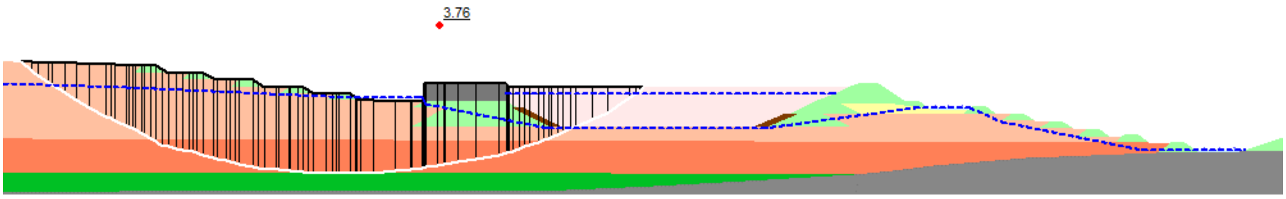


Figure L2: Undrained Static Analysis – Upper Section Stage 16 – FoS = 3.76

			Weight (kN/m³)	(kPa)	(°)	Strength (kPa)	Ratio	Strength Fn	Line
	Bauxite Residue (Farmed)	SHANSEP	21.5			12	0.6		1
	Bauxite Residue (UnFarmed)	SHANSEP	21.5			10	0.25	Bauxite Residue	2
	Bedrock	Bedrock (Impenetrable)							2
	Clay	Mohr-Coulomb	18	0	28				2
	Dyke Rockfill	Mohr-Coulomb	22	0	45				2
	Glacial Till	Mohr-Coulomb	19	0	32				1
	Process Sand	Mohr-Coulomb	19	0	33				2
	Rockfill	Mohr-Coulomb	22	0	45				1
	Salt Cake	SHANSEP	12			5	0.05		1

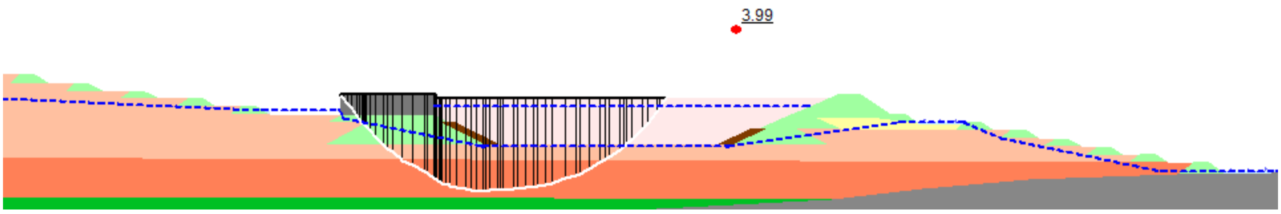


Figure L3: Undrained Static Analysis – Middle Section Stage 16 – FoS = 3.99

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (Farmed)	SHANSEP	21.5			12	0.6		1
	Bauxite Residue (UnFarmed)	SHANSEP	21.5			10	0.25	Bauxite Residue	2
	Bedrock	Bedrock (Impenetrable)							2
	Clay	Mohr-Coulomb	18	0	28				2
	Dyke Rockfill	Mohr-Coulomb	22	0	45				2
	Glacial Till	Mohr-Coulomb	19	0	32				1
	Process Sand	Mohr-Coulomb	19	0	33				2
	Rockfill	Mohr-Coulomb	22	0	45				1
	Salt Cake	SHANSEP	12			5	0.05		1

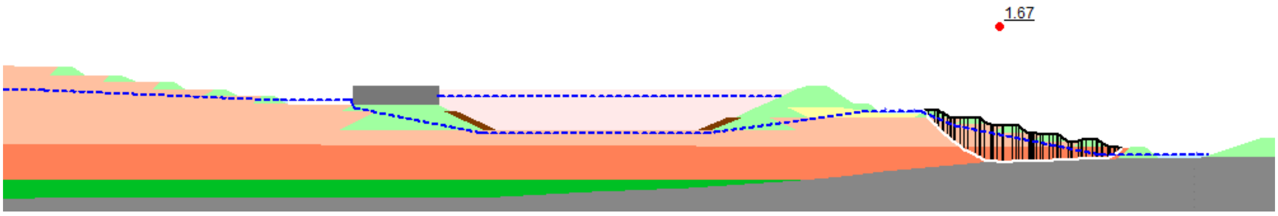


Figure L4: Undrained Static Analysis – Lower Section Stage 16 – FoS = 1.67

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (Farmed) PS	SHANSEP	21.5			9.6	0.48		1
	Bauxite Residue (UnFarmed) PS	SHANSEP	21.5			8	0.2	Bauxite Residue	1
	Bedrock	Bedrock (Impenetrable)							2
	Clay	Mohr-Coulomb	18	0	28				2
	Dyke Rockfill	Mohr-Coulomb	22	0	45				2
	Glacial Till	Mohr-Coulomb	19	0	32				1
	Process Sand	Mohr-Coulomb	19	0	33				2
	Rockfill	Mohr-Coulomb	22	0	45				1
	Salt Cake (PS)	SHANSEP	12			4	0.04		1

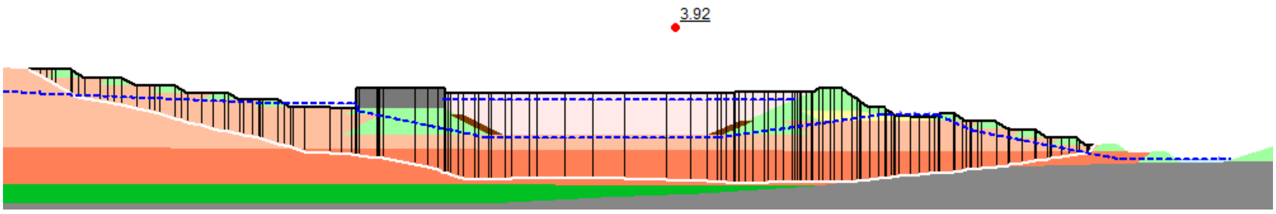


Figure L5: Undrained Post-Earthquake Analysis – Global Section Stage 16 – FoS = 3.92

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (Farmed) PS	SHANSEP	21.5			9.6	0.48		1
	Bauxite Residue (UnFarmed) PS	SHANSEP	21.5			8	0.2	Bauxite Residue	1
	Bedrock	Bedrock (Impenetrable)							2
	Clay	Mohr-Coulomb	18	0	28				2
	Dyke Rockfill	Mohr-Coulomb	22	0	45				2
	Glacial Till	Mohr-Coulomb	19	0	32				1
	Process Sand	Mohr-Coulomb	19	0	33				2
	Rockfill	Mohr-Coulomb	22	0	45				1
	Salt Cake (PS)	SHANSEP	12			4	0.04		1

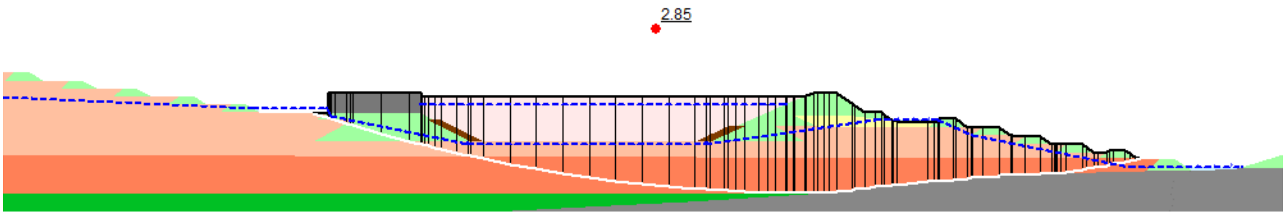


Figure L6: Undrained Pseudo-Static Analysis – Global Section Stage 16 – FoS = 2.85

8.0 STABILITY ANALYSES FOR PHASE 2 BRDA

This section provides the stability analyses for the Phase 2 BRDA comprising Section N-N, Section P-P, Section R-R, Section T-T and Section V-V.

- Section N-N is considered representative of Section M-M, Section N-N and Section O-O.
- Section P-P is considered representative of Section P-P and Section Q-Q
- Section R-R is considered representative of Section R-R and Section S-S
- Section V-V is considered representative of Section U-U, Section V-V, Section W-W and Section X-X

The geotechnical parameters selected for the estuarine deposits (where present) and the bauxite residue at each stability section have been determined following assessment of the field investigation data comprising in-situ testing, sampling, laboratory testing and interpretation by others prior to 2004 and by Golder after 2004.

The undrained condition (total stress) is considered the critical case required a FoS of 1.5 for the Global, Upper, Middle and Lower Slope conditions as:

- An undrained condition for a material in a contractive state (unfarmed bauxite residue), generates excess pore pressure and results in a lower effective shear strength less than in the drained condition.
- The undrained condition when the material is in a relatively dense/stiff condition (farmed bauxite residue), dilates during shearing, generates negative pore pressure and may result in an effective shear strength greater than in the drained condition.

8.1 Section N-N

Table 24: Section N-N - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Strength Undrained			Comment
	Bulk	Dry	10th Percentile	30th Percentile	50th Percentile	
Estuarine (Upper) Deposit	1.94 / 19.0	1.63 / 16.0	s_u/σ'_{v0} = 0.45	s_u/σ'_{v0} = 0.49	s_u/σ'_{v0} = 0.52	Undrained strength from CPTu, shear vane and DSS testing
Estuarine (Lower) Deposit			s_u/σ'_{v0} = 0.20	s_u/σ'_{v0} = 0.22	s_u/σ'_{v0} = 0.26	
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 16.0	s_u/σ'_{v0} = 0.25			Undrained strength from CPTu, shear vane and DSS testing
Farmed Bauxite Residue			s_u/σ'_{v0} = 0.60			

Table 25: Section N-N – Stability Analyses Results at Stage 16

Section N-N (Stage 16)	Factor of Safety (Undrained)	
	10th Percentile Estuarine	30th Percentile Estuarine
Global Slope Stability	1.76	1.87 **(N1)**
Upper Slope Stability	2.98	3.08 **(N2)**
Middle Slope Stability	2.31	2.41 **(N3)**
Lower Slope Stability	1.40	1.48 **(N4)**
Post Seismic Stability	1.40	1.49 **(N5)**
Pseudo-Static Stability	1.20	1.28 **(N6)**

The stability analyses concludes that the Section N-N slope profile will attain the target FoS of 1.5 for the BRDA constructed to Stage 16.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1
	Estuarine (Lower) GA19-5A	SHANSEP	19				12	0.22		2
	Estuarine (Upper) GA19-5A	SHANSEP	19				12	0.49		2

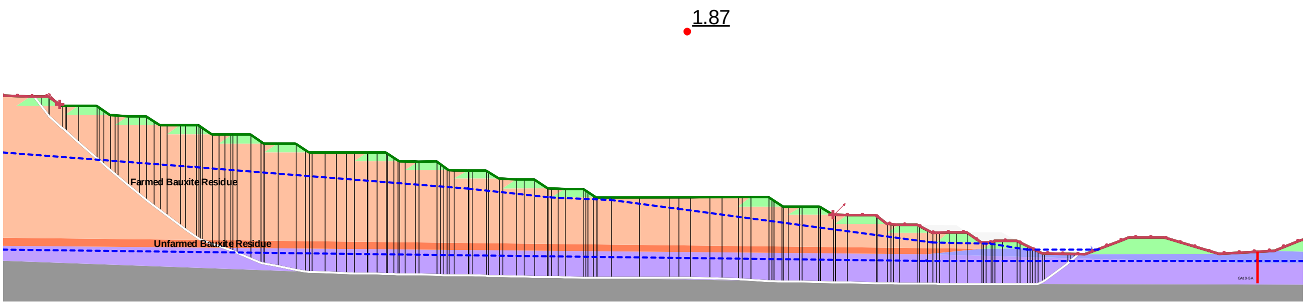


Figure N1: Undrained Static Analysis – Global Section (30th Percentile) – FoS = 1.87

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1
	Estuarine (Lower) GA19-5A	SHANSEP	19				12	0.22		2
	Estuarine (Upper) GA19-5A	SHANSEP	19				12	0.49		2

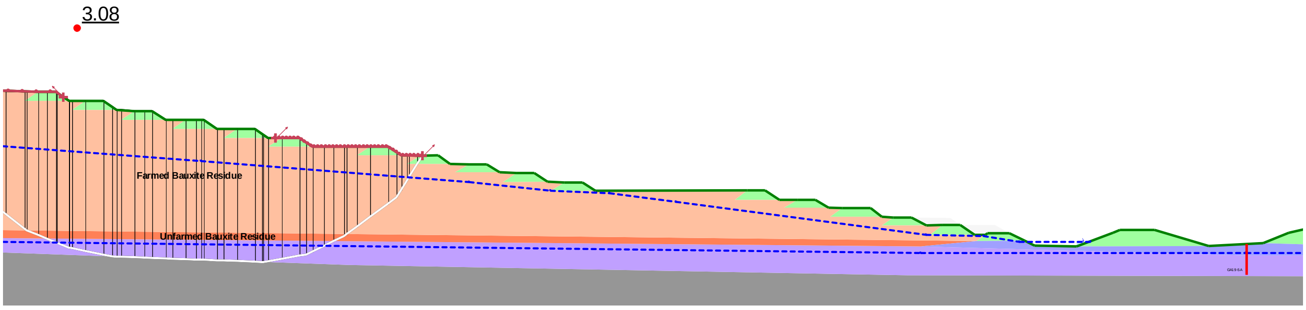


Figure N2: Undrained Static Analysis – Upper Section (30th Percentile) – FoS = 3.08

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1
	Estuarine (Lower) GA19-5A	SHANSEP	19				12	0.22		2
	Estuarine (Upper) GA19-5A	SHANSEP	19				12	0.49		2

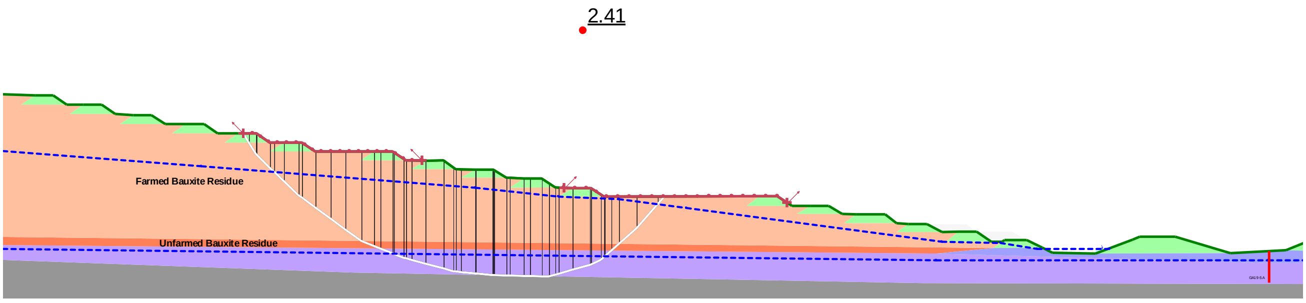


Figure N3: Undrained Static Analysis – Middle Section (30th Percentile) – FoS = 2.41

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1
	Estuarine (Lower) GA19-5A	SHANSEP	19				12	0.22		2
	Estuarine (Upper) GA19-5A	SHANSEP	19				12	0.49		2

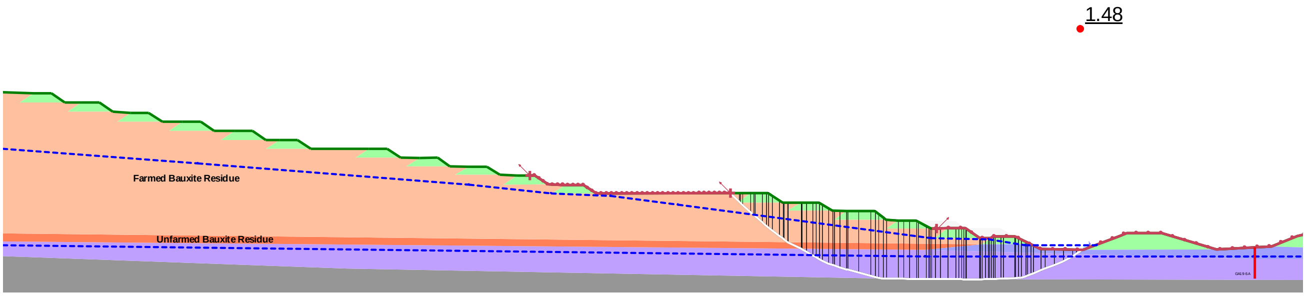


Figure N4: Undrained Static Analysis – Lower Section (30th Percentile) – FoS = 1.48

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	1
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	1
	Bedrock	Bedrock (Impenetrable)							2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			1
	Estuarine (Lower) (PS)	SHANSEP	19				9.6	0.176	2
	Estuarine (Upper) (PS)	SHANSEP	19				9.6	0.392	2

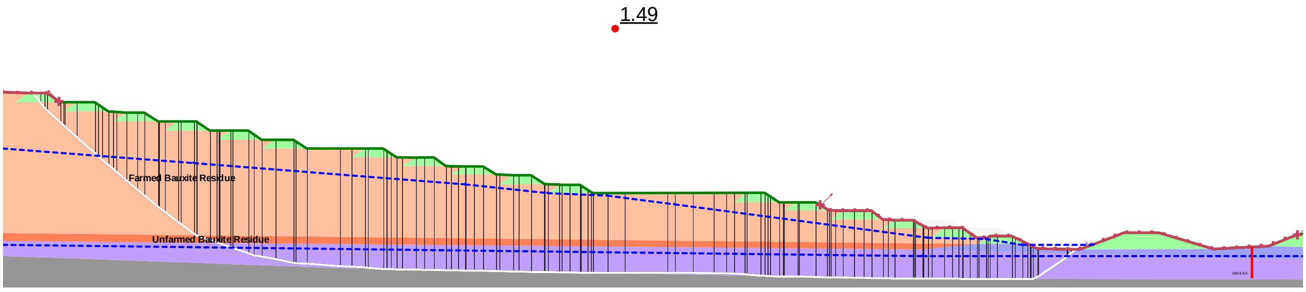


Figure N5: Undrained Post-Earthquake Analysis – Global Section (30th Percentile) – FoS = 1.49

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	1
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	1
	Bedrock	Bedrock (Impenetrable)							2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			1
	Estuarine (Lower) (PS)	SHANSEP	19				9.6	0.176	2
	Estuarine (Upper) (PS)	SHANSEP	19				9.6	0.392	2

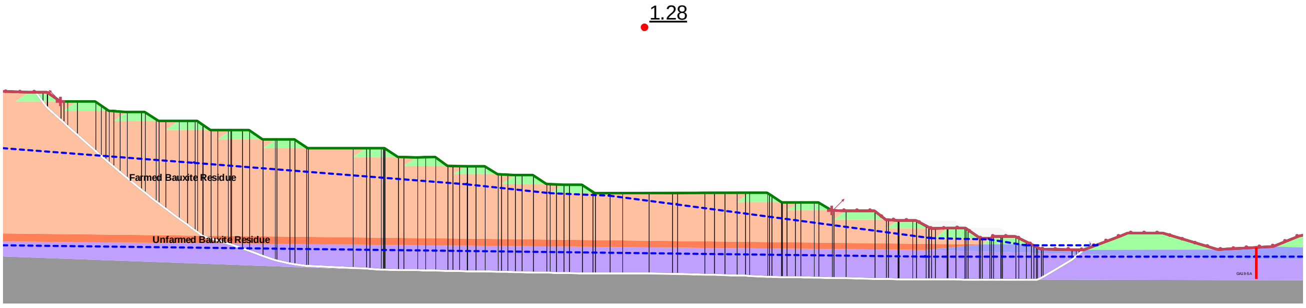


Figure N6: Undrained Pseudo-Static Analysis – Global Section (30th Percentile) – FoS = 1.28

8.2 Section P-P

Table 26: Section P-P - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Strength			Comment
			Undrained			
	Bulk	Dry	10th Percentile	30th Percentile	50th Percentile	
Estuarine (Upper) Deposit	1.94 / 19.0	1.63 / 16.0	s_u/σ'_{v0} = 0.50	s_u/σ'_{v0} = 0.70	s_u/σ'_{v0} = 0.73	Undrained strength from CPTu, shear vane and DSS testing
Estuarine (Lower) Deposit			s_u/σ'_{v0} = 0.25	s_u/σ'_{v0} = 0.36	s_u/σ'_{v0} = 0.39	
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 16.0	s_u/σ'_{v0} = 0.25			Undrained strength from CPTu, shear vane and DSS testing
Farmed Bauxite Residue			s_u/σ'_{v0} = 0.60			

Table 27: Section P-P – Stability Analyses Results at Stage 16

Section P-P (Stage 16)	Factor of Safety (Undrained)	
	10th Percentile Estuarine	30th Percentile Estuarine
Global Slope Stability	1.78 **(P1)**	1.94
Upper Slope Stability	3.04 **(P2)**	3.00
Middle Slope Stability	2.22 **(P3)**	2.20
Lower Slope Stability	1.52 **(P4)**	1.95
Post Seismic Stability	1.42 **(P5)**	1.55
Pseudo-Static Stability	1.22 **(P6)**	1.33

The stability analyses concludes that the Section P-P slope profile will attain the target FoS of 1.5 for the BRDA constructed to Stage 16.

Color	Name	Model	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1
	Estuarine (Lower) GA19-5C	SHANSEP	19				12	0.25		2
	Estuarine (Upper) GA19-5C	SHANSEP	19				12	0.5		2

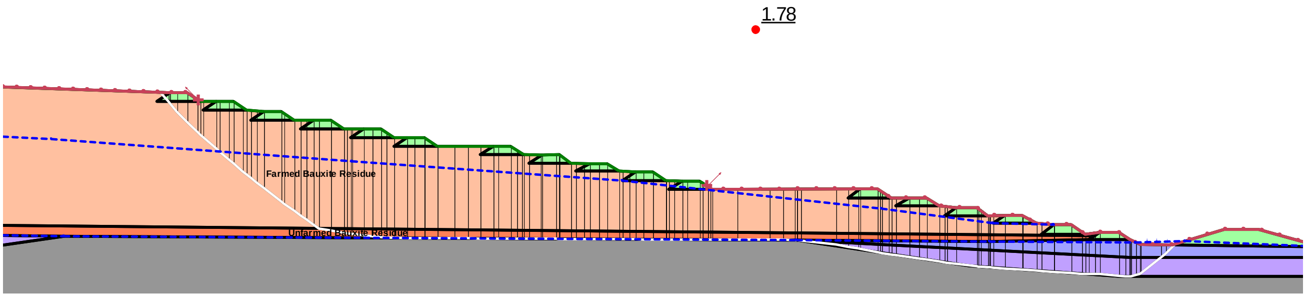


Figure P1: Undrained Static Analysis – Global Section (10th Percentile) – FoS = 1.78

Color	Name	Model	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1
	Estuarine (Lower) GA19-5C	SHANSEP	19				12	0.25		2
	Estuarine (Upper) GA19-5C	SHANSEP	19				12	0.5		2

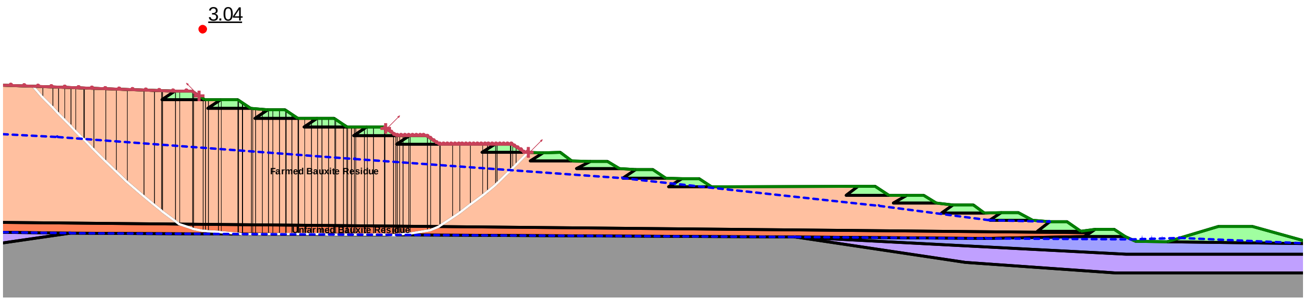


Figure P2: Undrained Static Analysis – Upper Section (10th Percentile) – FoS = 3.04

Color	Name	Model	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1
	Estuarine (Lower) GA19-5C	SHANSEP	19				12	0.25		2
	Estuarine (Upper) GA19-5C	SHANSEP	19				12	0.5		2

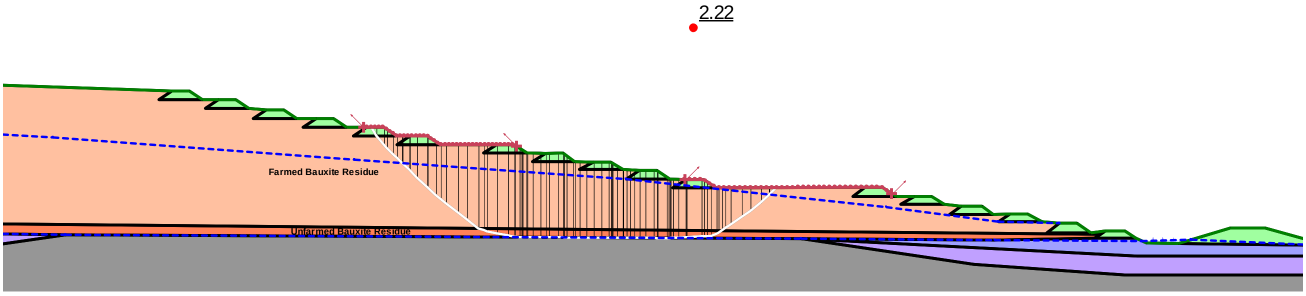


Figure P3: Undrained Static Analysis – Middle Section (10th Percentile) – FoS = 2.22

Color	Name	Model	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1
	Estuarine (Lower) GA19-5C	SHANSEP	19				12	0.25		2
	Estuarine (Upper) GA19-5C	SHANSEP	19				12	0.5		2

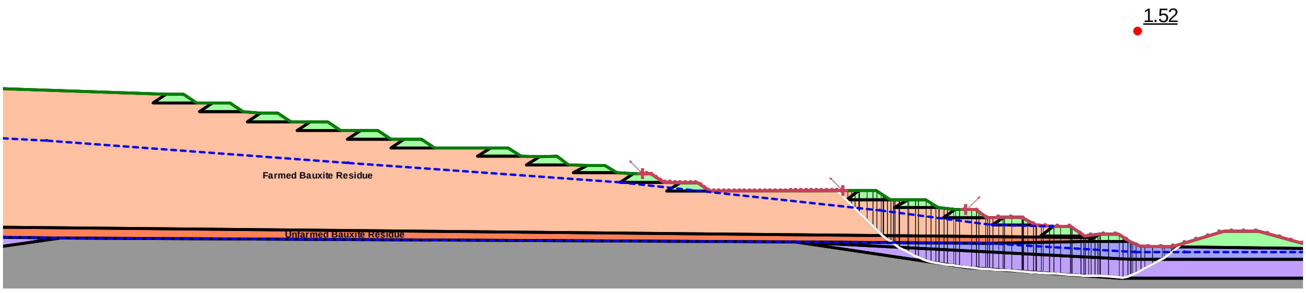


Figure P4: Undrained Static Analysis – Lower Section (10th Percentile) – FoS = 1.52

Color	Name	Model	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	1
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	1
	Bedrock	Bedrock (Impenetrable)							2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			1
	Estuarine (Lower) (PS)	SHANSEP	19				9.6	0.2	2
	Estuarine (Upper) (PS)	SHANSEP	19				9.6	0.4	2

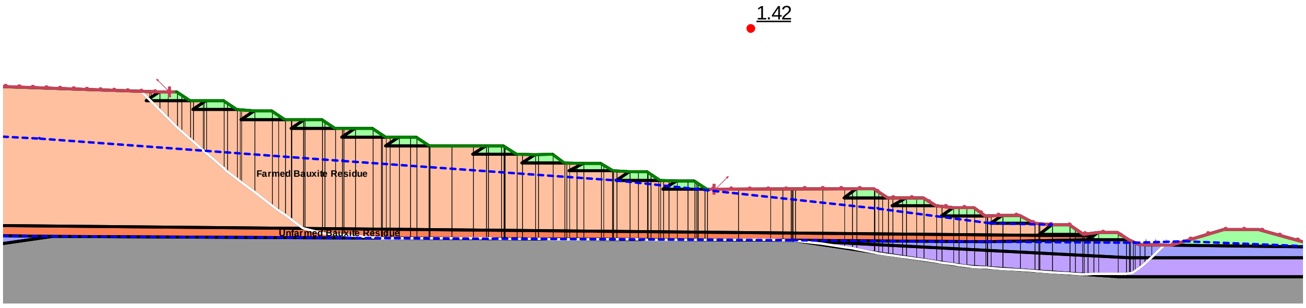


Figure P5: Undrained Post-Earthquake Analysis – Global Section (10th Percentile) – FoS = 1.42

Color	Name	Model	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2	1
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48	1
	Bedrock	Bedrock (Impenetrable)							2
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			1
	Estuarine (Lower) (PS)	SHANSEP	19				9.6	0.2	2
	Estuarine (Upper) (PS)	SHANSEP	19				9.6	0.4	2

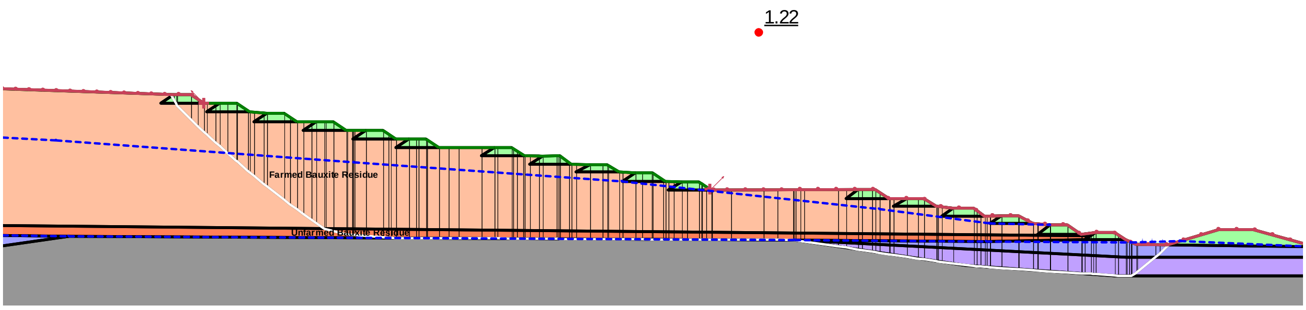


Figure P6: Undrained Pseudo-Static Analysis – Global Section (10th Percentile) – FoS = 1.22

8.3 Section R-R

Table 28: Section R-R - Characteristic Geotechnical Parameters

<table>
<tr><th rowspan="3">Material Property</th><th colspan="2">Density / Unit Weight (Mg/m3 / kN/m3)</th><th colspan="3">Strength</th><th rowspan="3">Comment</th></tr>
<tr><th colspan="2"></th><th colspan="3">Undrained</th></tr>
<tr><th>Bulk</th><th>Dry</th><th>10th Percentile</th><th>30th Percentile</th><th>50th Percentile</th></tr>
<tr><td>Estuarine (Upper) Deposit</td><td>1.94 / 19.0</td><td>1.63 / 16.0</td><td>$s_u/\sigma'_{v0} = 0.18$</td><td>N/A</td><td>N/A</td><td>Undrained strength from CPTu, shear vane and DSS testing</td></tr>
<tr><td>Unfarmed Bauxite Residue</td><td rowspan="2">2.19 / 21.5</td><td rowspan="2">1.63 / 16.0</td><td colspan="3">$s_u/\sigma'_{v0} = 0.25$</td><td rowspan="2">Undrained strength from CPTu, shear vane and DSS testing</td></tr>
<tr><td>Farmed Bauxite Residue</td><td colspan="3">$s_u/\sigma'_{v0} = 0.60$</td></tr>
</table>

Table 29: Section R-R – Stability Analyses Results at Stage 16

<table>
<tr><th rowspan="2">Section R-R (Stage 16)</th><th>Factor of Safety (Undrained)</th></tr>
<tr><th>No Estuarine</th></tr>
<tr><td>Global Slope Stability</td><td>1.85 (R1)</td></tr>
<tr><td>Upper Slope Stability</td><td>2.95 (R2)</td></tr>
<tr><td>Middle Slope Stability</td><td>2.20 (R3)</td></tr>
<tr><td>Lower Slope Stability</td><td>1.71 (R4)</td></tr>
<tr><td>Post Seismic Stability</td><td>1.47 (R5)</td></tr>
<tr><td>Pseudo-Static Stability</td><td>1.26 (R6)</td></tr>
</table>

The stability analyses concludes that the Section R-R slope profile will attain the target FoS of 1.5 for the BRDA constructed to Stage 16.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Upper)	SHANSEP	19				12	0.18	

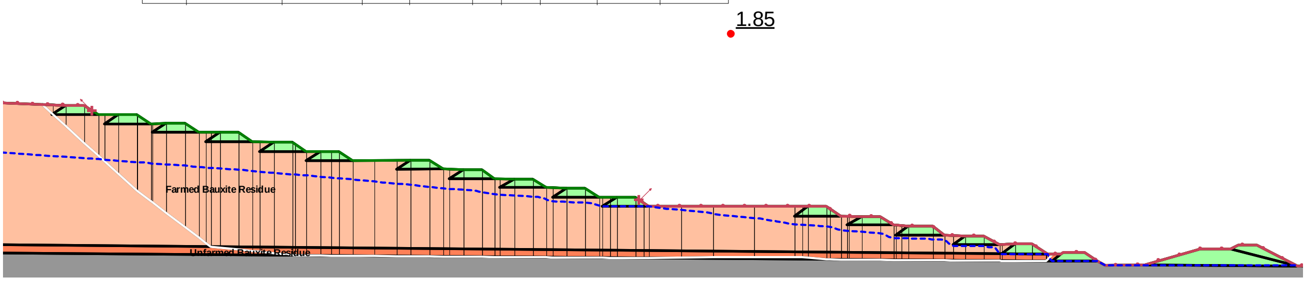


Figure R1: Undrained Static Analysis – Global Section (10th Percentile) – FoS = 1.85

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Upper)	SHANSEP	19				12	0.18	

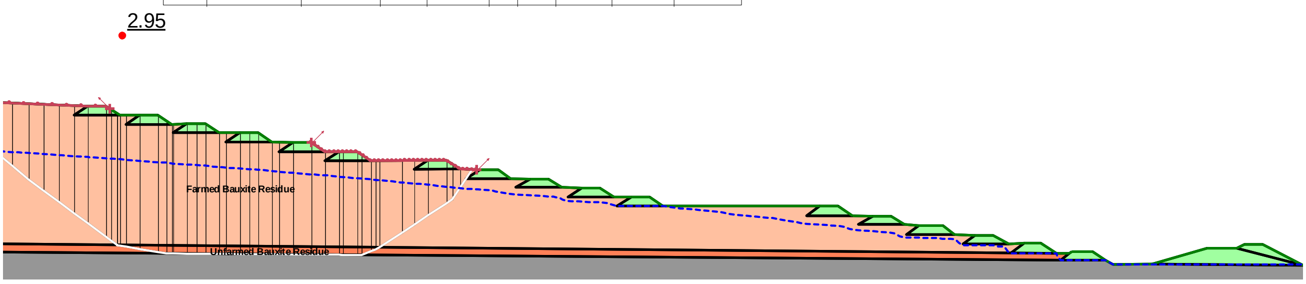


Figure R2: Undrained Static Analysis – Upper Section (10th Percentile) – FoS = 2.95

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Upper)	SHANSEP	19				12	0.18	

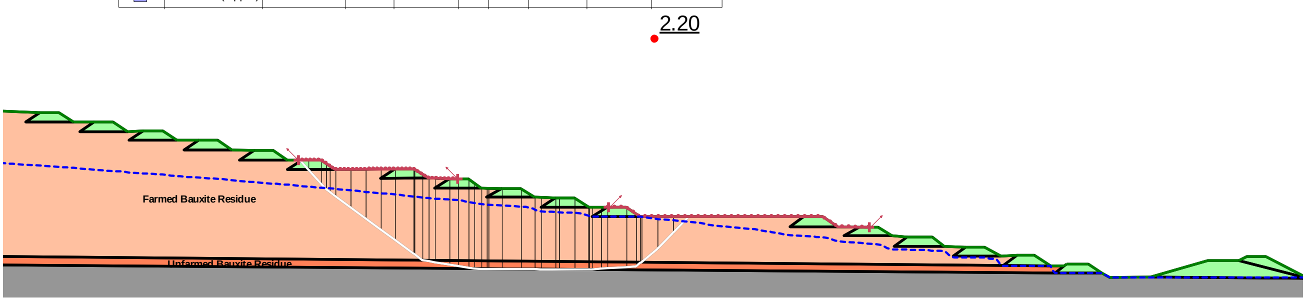


Figure R3: Undrained Static Analysis – Middle Section (10th Percentile) – FoS = 2.20

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			
	Estuarine (Upper)	SHANSEP	19				12	0.18	

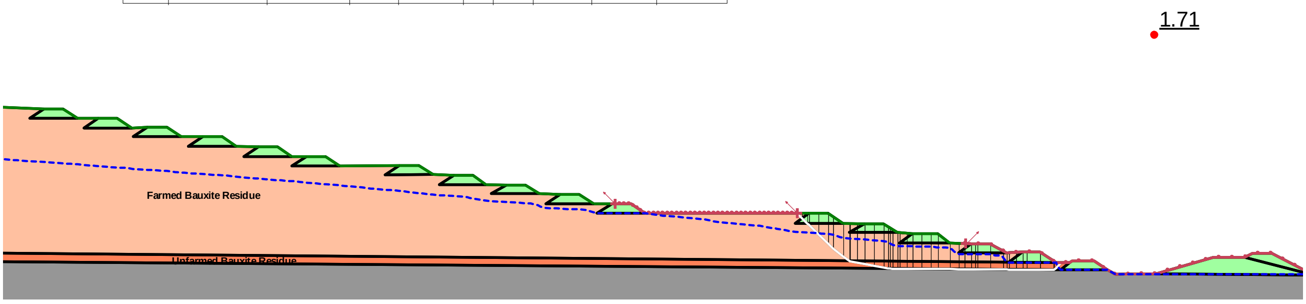


Figure R4: Undrained Static Analysis – Lower Section (10th Percentile) – FoS = 1.71

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Upper) (PS)	SHANSEP	19				9.6	0.144

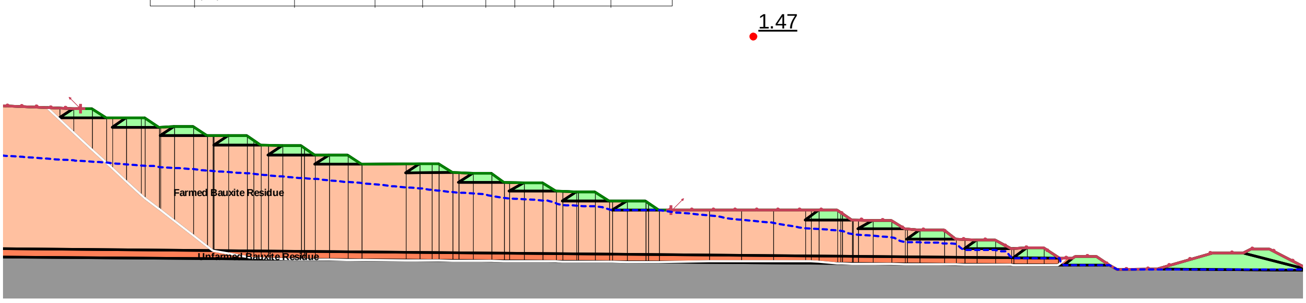


Figure R5: Undrained Post-Earthquake Analysis – Global Section (10th Percentile) – FoS = 1.47

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		
	Estuarine (Upper) (PS)	SHANSEP	19				9.6	0.144

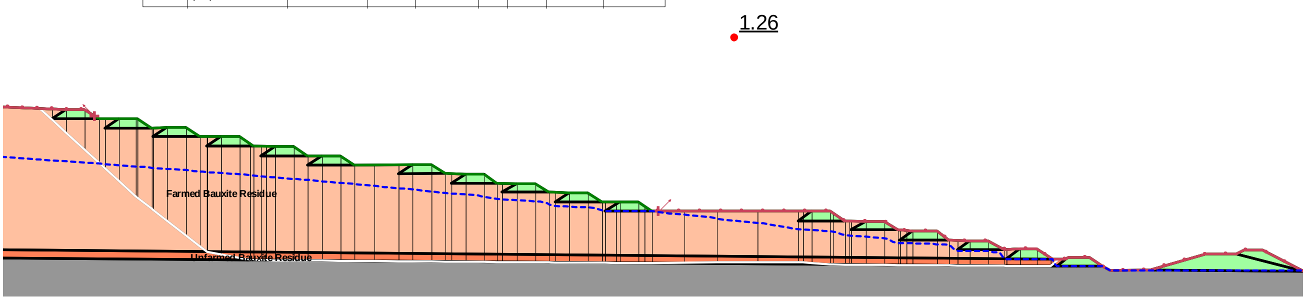


Figure R6: Undrained Pseudo-Static Analysis – Global Section (10th Percentile) – FoS = 1.26

8.4 Section T-T

Table 30: Section T-T - Characteristic Geotechnical Parameters

Material Property	Density / Unit Weight (Mg/m3 / kN/m3)		Strength			Comment
			Undrained			
	Bulk	Dry	10th Percentile	30th Percentile	50th Percentile	
Unfarmed Bauxite Residue	2.19 / 21.5	1.63 / 1.06	$s_u/\sigma'_{v0} = 0.25$			Undrained strength from CPTu, shear vane and DSS testing
Farmed Bauxite Residue			$s_u/\sigma'_{v0} = 0.60$			

Table 31: Section T-T – Stability Analyses Results at Stage 16

Section T-T (Stage 16)	Factor of Safety (Undrained)
	No Estuarine
Global Slope Stability	1.91 **(T1)**
Upper Slope Stability	3.01 **(T2)**
Middle Slope Stability	2.23 **(T3)**
Lower Slope Stability	2.17 **(T4)**
Post Seismic Stability	1.52 **(T5)**
Pseudo-Static Stability	1.30 **(T6)**

The stability analyses concludes that the Section T-T slope profile will attain the target FoS of 1.5 for the BRDA constructed to Stage 16.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								1
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1

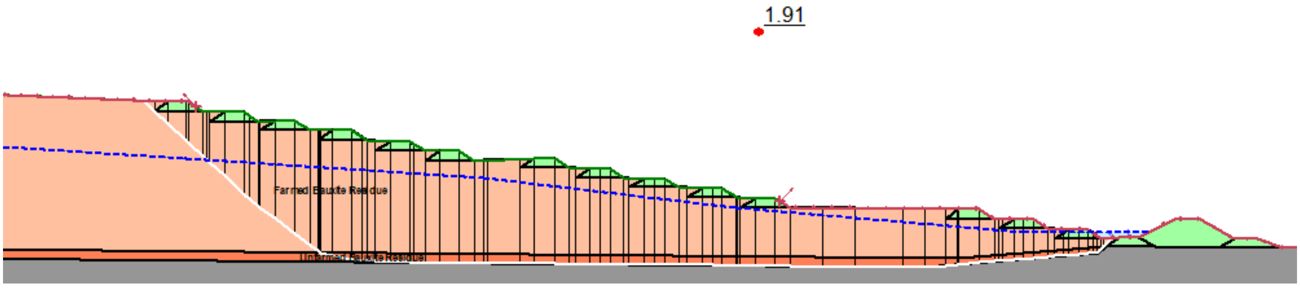


Figure T1: Undrained Static Analysis – Global Section (10th Percentile) – FoS = 1.91

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								1
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1

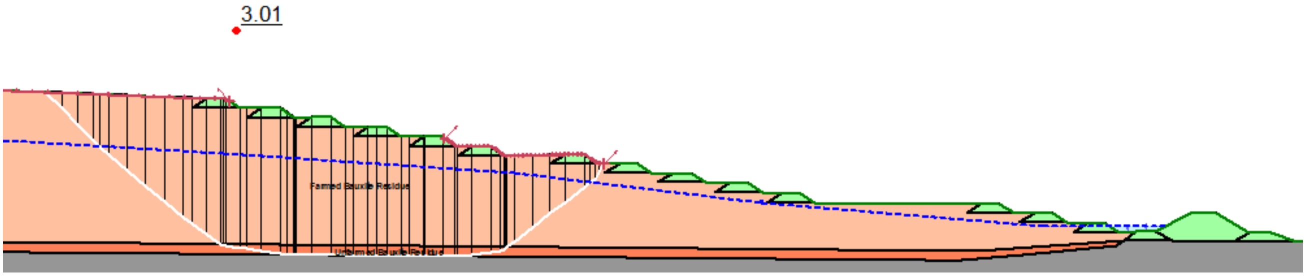


Figure T2: Undrained Static Analysis – Upper Section (10th Percentile) – FoS = 3.01

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6		1
	Bedrock	Bedrock (Impenetrable)								1
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0				1

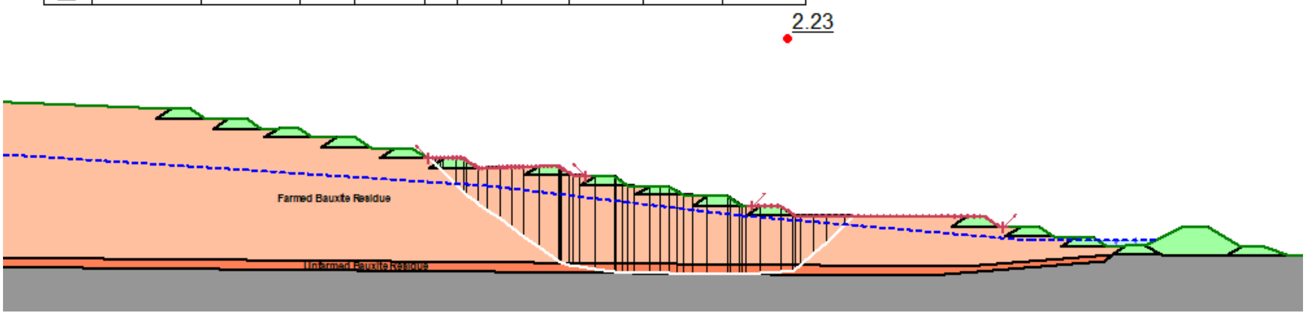


Figure T3: Undrained Static Analysis – Middle Section (10th Percentile) – FoS = 2.23

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5				12	0.25	Bauxite Residue
	Bauxite Residue Farmed	SHANSEP	21.5				12	0.6	
	Bedrock	Bedrock (Impenetrable)							
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0			

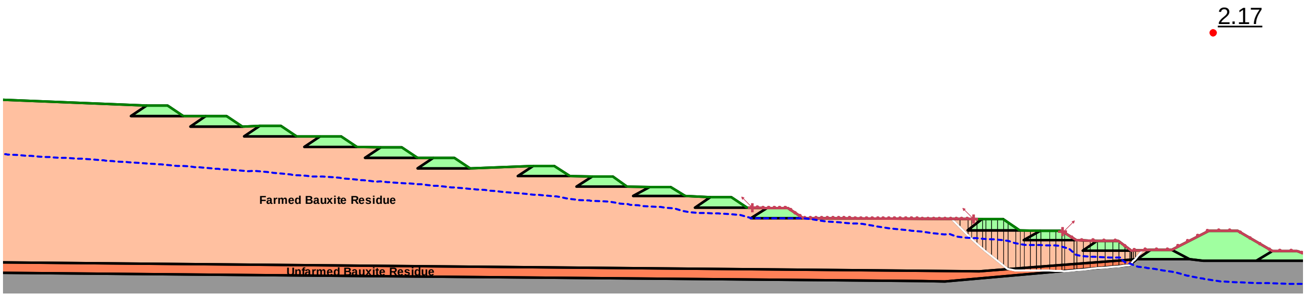


Figure T4: Undrained Static Analysis – Lower Section (10th Percentile) – FoS = 2.17

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		

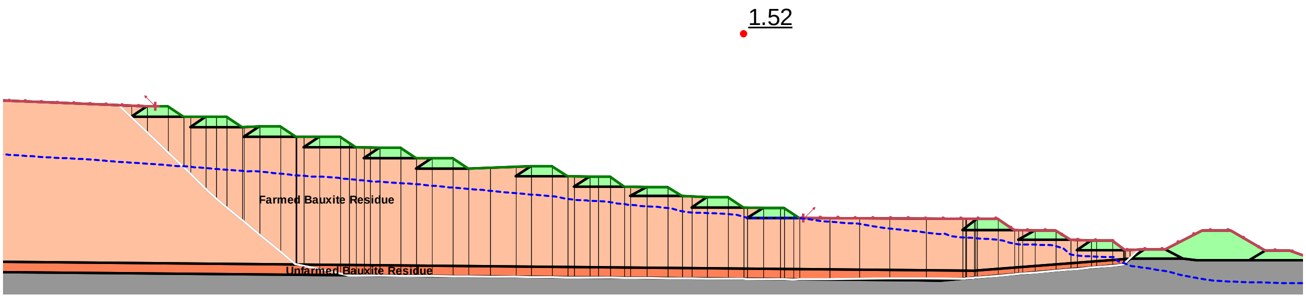


Figure T5: Undrained Post-Earthquake Analysis – Global Section (10th Percentile) – FoS = 1.52

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5				9.6	0.2
	Bauxite Residue Farmed (PS)	SHANSEP	21.5				9.6	0.48
	Bedrock	Bedrock (Impenetrable)						
	Dyke Rockfill	Mohr-Coulomb	22	0	45	0		

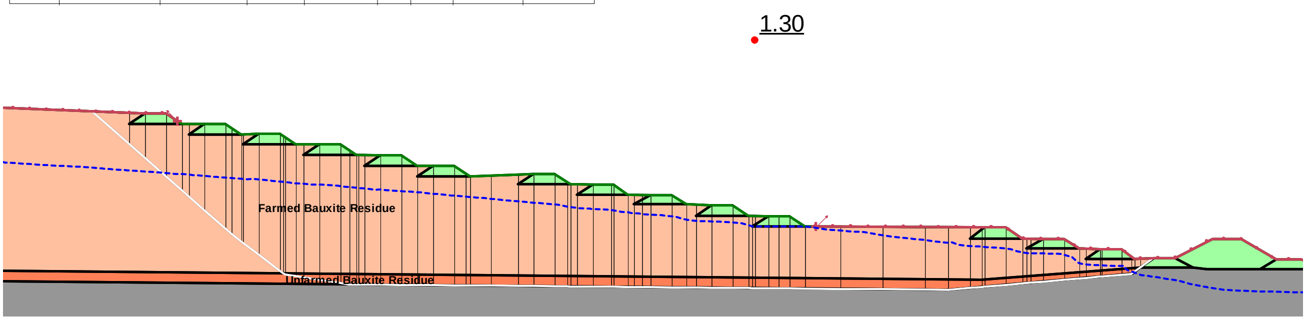


Figure T6: Section T-T; Undrained Pseudo-Static Analysis – Global Section (10th Percentile) – FoS = 1.30

8.5 Section V-V

Table 32: Section V-V- Characteristic Geotechnical Parameters

<table>
<tr><th rowspan="3">Material Property</th><th colspan="2">Density / Unit Weight (Mg/m3 / kN/m3)</th><th colspan="3">Strength</th><th rowspan="3">Comment</th></tr>
<tr><th colspan="2"></th><th colspan="3">Undrained</th></tr>
<tr><th>Bulk</th><th>Dry</th><th>10th Percentile</th><th>30th Percentile</th><th>50th Percentile</th></tr>
<tr><td>Unfarmed Bauxite Residue</td><td rowspan="2">2.19 / 21.5</td><td rowspan="2">1.63 / 16.0</td><td colspan="3">s_u/σ'_{v0} = 0.25</td><td rowspan="2">Undrained strength from CPTu, shear vane and DSS testing</td></tr>
<tr><td>Farmed Bauxite Residue</td><td colspan="3">s_u/σ'_{v0} = 0.60</td></tr>
</table>

Table 33: Section V-V – Stability Analyses Results at Stage 16

Section V-V (Stage 16)	Factor of Safety (Undrained)
	No Estuarine
Global Slope Stability	1.85 **(V1)**
Upper Slope Stability	2.66 **(V2)**
Middle Slope Stability	2.16 **(V3)**
Lower Slope Stability	2.08 **(V4)**
Post Seismic Stability	1.48 **(V5)**
Pseudo-Static Stability	1.26 **(V6)**

The stability analyses concludes that the Section V-V slope profile will attain the target FoS of 1.5 for the BRDA constructed to Stage 16.

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5			12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5			12	0.6		1
	Bedrock	Bedrock (Impenetrable)							1
	Clay	Mohr-Coulomb	22	0	28				1
	Dyke Rockfill	Mohr-Coulomb	22	0	45				1
	Rockfill	Mohr-Coulomb	22	0	45				1

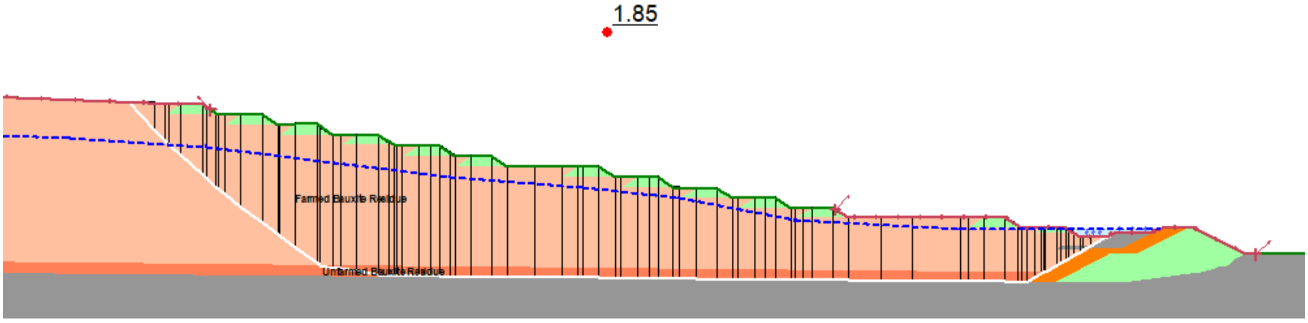


Figure V1: Undrained Static Analysis – Global Section (10th Percentile) – FoS = 1.85

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5			12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5			12	0.6		1
	Bedrock	Bedrock (Impenetrable)							1
	Clay	Mohr-Coulomb	22	0	28				1
	Dyke Rockfill	Mohr-Coulomb	22	0	45				1
	Rockfill	Mohr-Coulomb	22	0	45				1

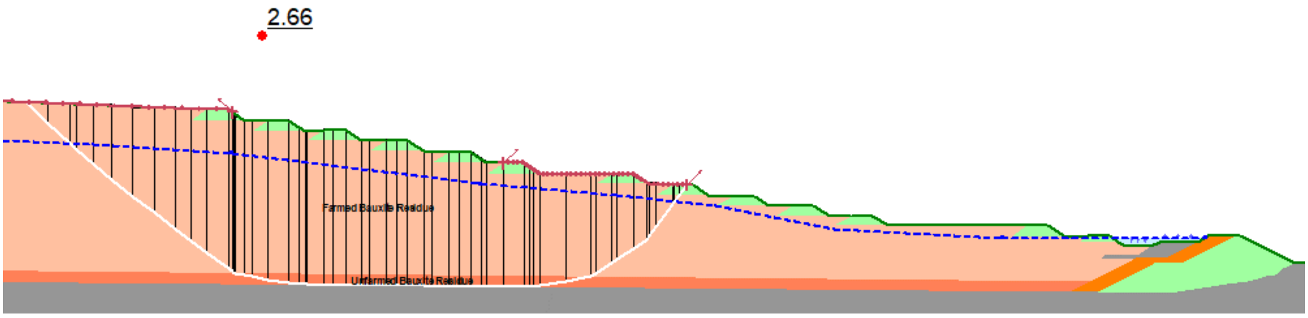


Figure V2: Undrained Static Analysis – Upper Section (10th Percentile) – FoS = 2.66

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5			12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5			12	0.6		1
	Bedrock	Bedrock (Impenetrable)							1
	Clay	Mohr-Coulomb	22	0	28				1
	Dyke Rockfill	Mohr-Coulomb	22	0	45				1
	Rockfill	Mohr-Coulomb	22	0	45				1

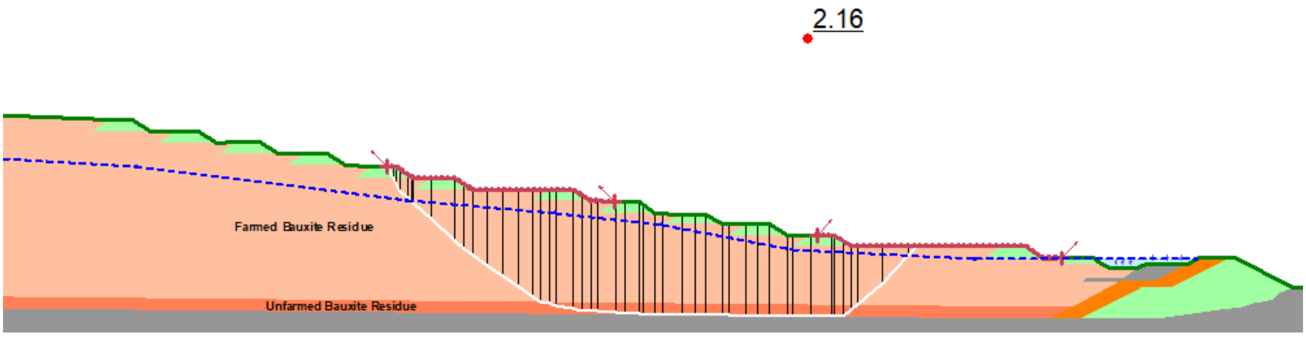


Figure V3: Undrained Static Analysis – Middle Section (10th Percentile) – FoS = 2.16

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Anisotropic Strength Fn	Piezometric Line
	Bauxite Residue (UnFarmed) (Anisotropic)	SHANSEP	21.5			12	0.25	Bauxite Residue	1
	Bauxite Residue Farmed	SHANSEP	21.5			12	0.6		1
	Bedrock	Bedrock (Impenetrable)							1
	Clay	Mohr-Coulomb	22	0	28				1
	Dyke Rockfill	Mohr-Coulomb	22	0	45				1
	Rockfill	Mohr-Coulomb	22	0	45				1

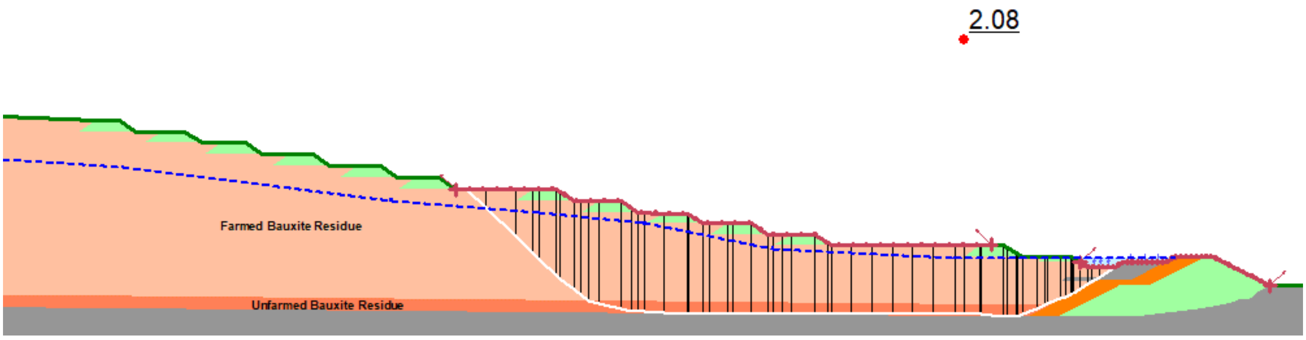


Figure V4: Undrained Static Analysis – Lower Section (10th Percentile) – FoS = 2.08

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5			9.6	0.2	1
	Bauxite Residue Farmed (PS)	SHANSEP	21.5			9.6	0.48	1
	Bedrock	Bedrock (Impenetrable)						1
	Clay	Mohr-Coulomb	22	0	28			1
	Dyke Rockfill	Mohr-Coulomb	22	0	45			1
	Rockfill	Mohr-Coulomb	22	0	45			1

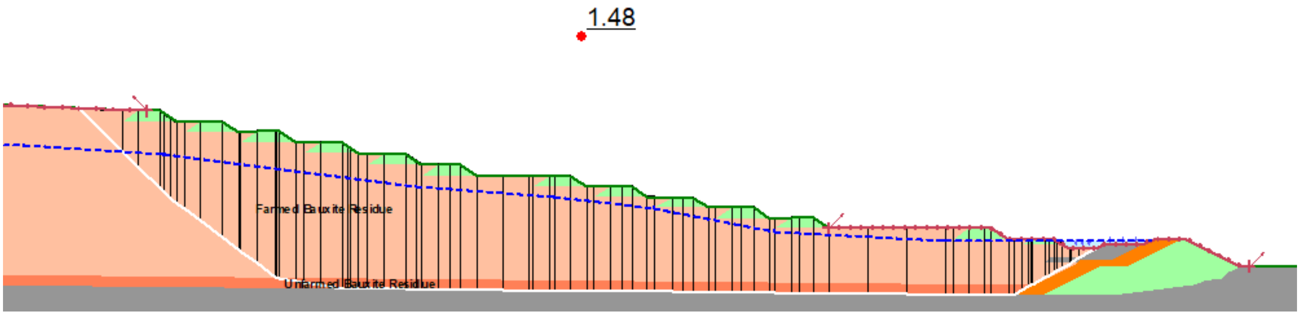


Figure V5: Undrained Post-Earthquake Analysis – Global Section (10th Percentile) – FoS = 1.48

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Bauxite Residue (UnFarmed) (PS)	SHANSEP	21.5			9.6	0.2	1
	Bauxite Residue Farmed (PS)	SHANSEP	21.5			9.6	0.48	1
	Bedrock	Bedrock (Impenetrable)						1
	Clay	Mohr-Coulomb	22	0	28			1
	Dyke Rockfill	Mohr-Coulomb	22	0	45			1
	Rockfill	Mohr-Coulomb	22	0	45			1

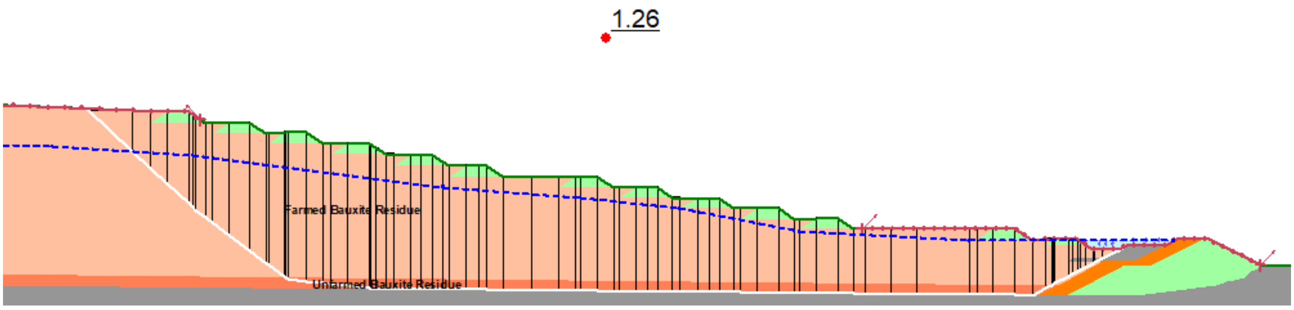


Figure V6: Undrained Pseudo-Static Analysis – Global Section (10th Percentile) – FoS = 1.26

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1.0 INTRODUCTION

Analyses of the stability of the Storm Water Pond (SWP), of the Liquid Waste Pond (LWP) and the Perimeter Interceptor Channels (PIC) were carried out using the limit equilibrium modelling software SLOPE-W Version 9.0.0.15234. The analytical method used was Morgenstern and Price method of slices, which satisfies both force and moment equilibrium.

Two conditions have been evaluated:

- Total stress analysis reflecting the short-term condition, including a post seismic event; and
- Effective stress analysis reflecting the long-term conditions.

2.0 ANALYSES CRITERIA

The International Guidelines, described in full in Appendix B, were also followed to determine the Factor of Safety (FoS) for the Aughinish Perimeter Interceptor Channels, Storm Water Pond and Liquid Waste Pond.

The Table below provides a summary of the FoS for slope stability analysis for each guideline.

Table 1: Factor of Safety Criteria based on International Guidelines

Loading Condition	Recommended Factor of Safety	
	CDA (2014)	ANCOLD (2012, 2019)
Short Term Undrained	Greater than 1.3 During, at, or end of Construction, depending on Risk Assessment	1.5 if loss of containment Consolidated Undrained Strength
Long Term Drained, Steady State	1.5 Steady State, Phreatic Level	1.5 Effective Strength
Pseudo-static	1.0 [1]	Not required
Post-Earthquake	1.2 [1]	1.0 to 1.2 (residual undrained shear strength)

Notes:

1. ***Undrained shear strength values were reduced by 20% to allow for cyclic softening (Hynes and Franklin 1984) for pseudo-static and post-earthquake analyses.***

3.0 MATERIAL PROPERTIES

Golder has adopted representative properties for the materials based on the available information and results of the recent and previous site investigation and laboratory testing programmes (Golder 2018). The selected properties for the undrained and drained analysis are presented below.

Table 2: Material Properties

Material Property	Density / Unit Weight (Mg/m^3 / kN/m^3)		Strength		Comment
	Bulk	Dry	Effective	Undrained	
Estuarine Deposit	1.73 / 17		Ø = 28° c = 0 kPa	$s_u/\sigma'_{v0} = 0.25$	Effective strength parameters determined from previous testing (Golder 2005) **Note:** Parameters determined from Golder 2018 testing are slightly improved – see Appendix B
Processed Glacial Till Fill	1.94 / 19		Ø = 32° c = 0 kPa	Not applicable	Typical for glacial till in Ireland
Glacial Till Fill	1.94 / 19		Ø = 32° c = 0 kPa	Not Applicable	Typical for glacial till in Ireland
Rock fill	2.24 / 22		Ø = 45° c = 0 kPa	Not applicable	Typical for Rock fill
Processed Rock fill	2.14 / 21		Ø = 40° c = 0 kPa	Not applicable	Typical for Rock fill
Gabion Mattress	2.24 / 22		Ø = 45° c = 200 kPa	Not applicable	Typical for Gabion baskets

Ø = friction angle; c = cohesion; s_u = undrained shear strength; σ'_{v0} = vertical effective confining stress.

4.0 STABILITY ANALYSES RESULTS

The stability analyses results for the sections analysed are summarised below and the analyses are presented on Figures D1 to D18 in Section 5.0 below. Drawing 12 shows the location of the Sectors for the SWP and LWP.

- The total stress analysis, using undrained shear strength parameters, returned FoS equal or greater than 1.3.
- The post-earthquake analyses results in FoS equal or greater than 1.2.
- The effective stress analysis, using drained strength parameters, returned FoS greater than 1.5.

Table 3: SWP, LWP and PIC Stability Results

Sector	Slip Surface	Static Factor of Safety		Post-Seismic Factor of Safety
		Effective Stress Analysis (Drained)	Total Stress Analysis (Undrained)	Total Stress Analysis (Undrained)
Sector H	SWP into Bird Sanctuary	1.9 **(D1)**	1.6 **(D2)**	1.3 **(D3)**
Sector I	SWP into PIC	1.5 **(D4)**	1.6 **(D5)**	1.3 **(D6)**
Sector J / Sector L	SWP into LWP LWP into SWP	2.1 **(D7)**	1.9 **(D8)**	1.5 **(D9)**
Sector K	LWP into Bird Sanctuary	1.8 **(D10)**	1.6 **(D11)**	1.2 **(D12)**
Sector M	LWP to PIC	1.7 **(D13)**	1.3 **(D14)** [1]	1.2 **(D15)**
PIC	Downstream failure of OPW	1.8 **(D16)**	1.8 **(D17)**	1.5 **(D18)**

Notes:

1. *All of the pond (SWP and LWP) Sectors returned FoS > 1.5 for the undrained analysis except for Sector M which returned an FoS < 1.5, however it does attain the original design FoS for this structure (1.3)* and the *long-term drained analysis does attain a FoS > 1.5.*

5.0 STABILITY ANALYSES FIGURES

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Estuarine	Mohr-Coulomb	17	0	28	0	1
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0	1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0	1
	Rockfill	Mohr-Coulomb	22	0	45	0	1

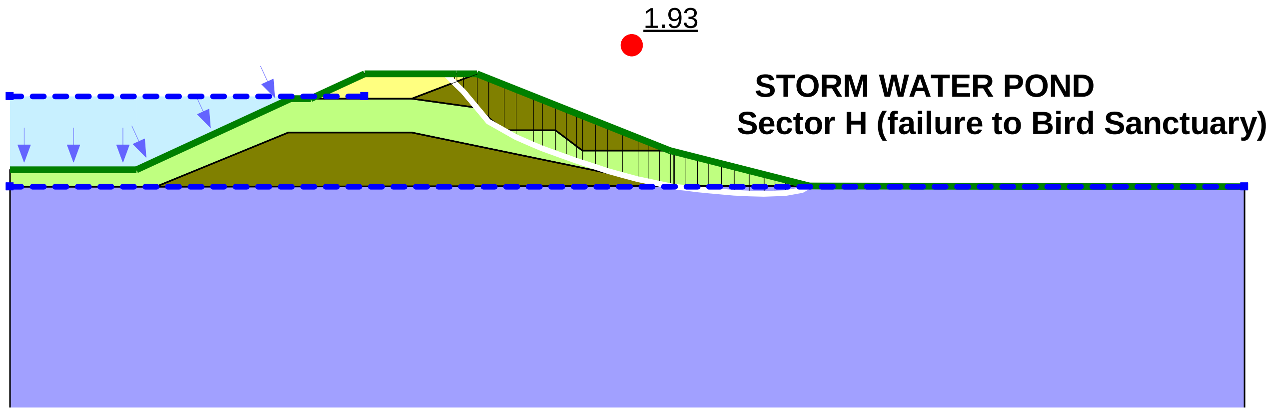


Figure D1: Drained (Effective Stress) Static Analysis – Sector H Failure to Bird Sanctuary

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Estuarine (Undrained)	SHANSEP	17				25	0.25	2
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0			1
	Rockfill	Mohr-Coulomb	22	0	45	0			1

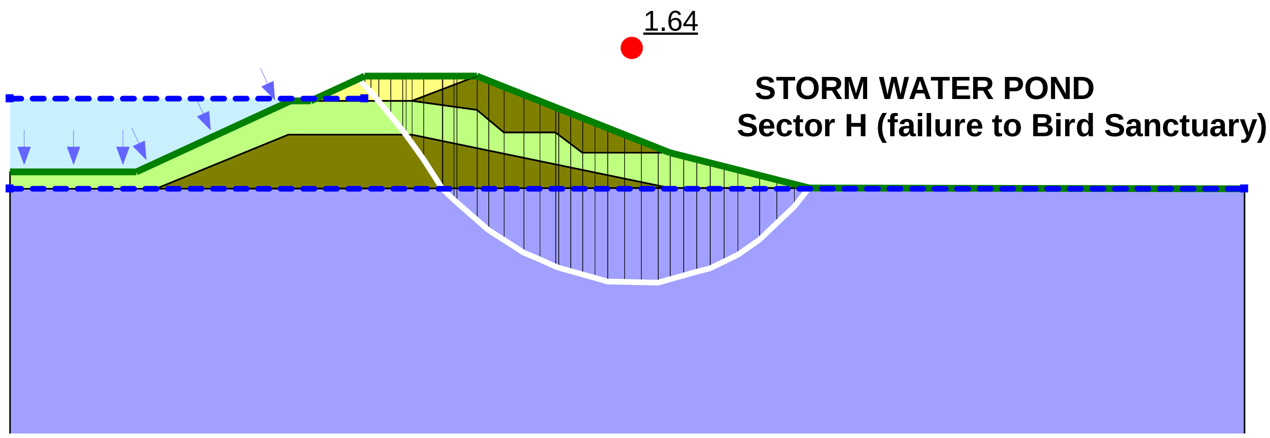


Figure D2: Undrained (Total Stress) Static Analysis – Sector H Failure to Bird Sanctuary

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Estuarine (Undrained) (PS)	SHANSEP	17				20	0.2	2
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0			1
	Rockfill	Mohr-Coulomb	22	0	45	0			1

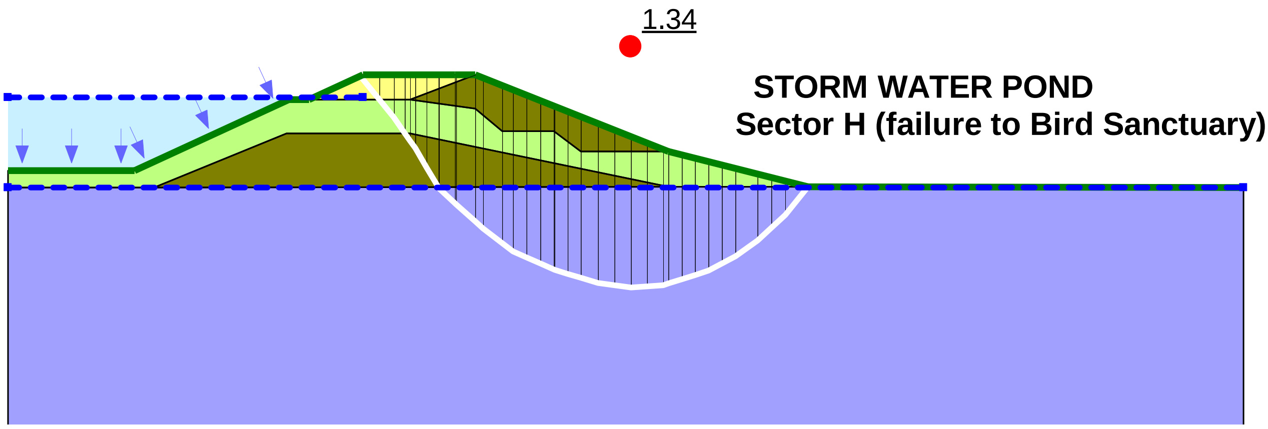


Figure D3: Undrained (Total Stress) Static Analysis (Post seismic) – Sector H Failure to Bird Sanctuary

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Estuarine	Mohr-Coulomb	17	0	28	0	2
	Gabion	Mohr-Coulomb	22	200	45	0	1
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0	1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0	1

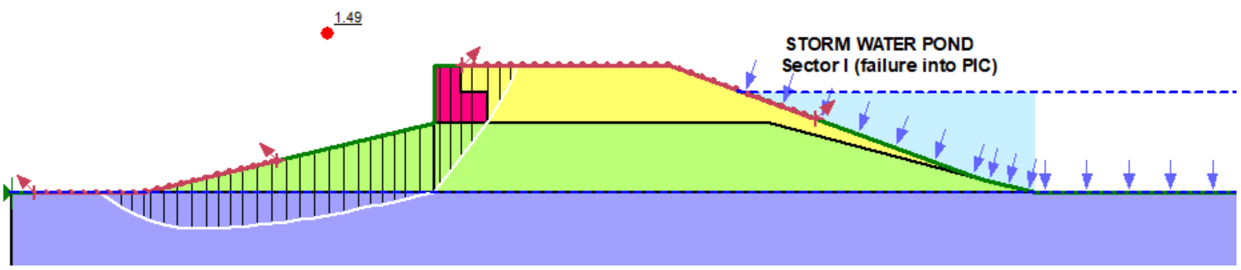


Figure D4: Drained (Effective Stress) Static Analysis – Sector I Failure to PIC

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Estuarine (Undrained)	SHANSEP	17				25	0.25	2
	Gabion	Mohr-Coulomb	22	200	45	0			1
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0			1

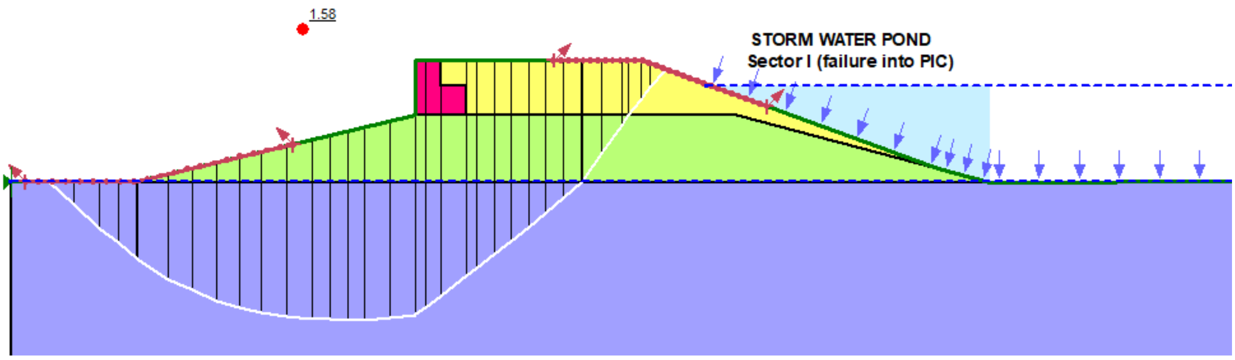


Figure D5: Undrained (Total Stress) Static Analysis – Sector I Failure to PIC

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Estuarine (Undrained) (PS)	SHANSEP	17				20	0.2	2
	Gabion	Mohr-Coulomb	22	200	45	0			1
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0			1

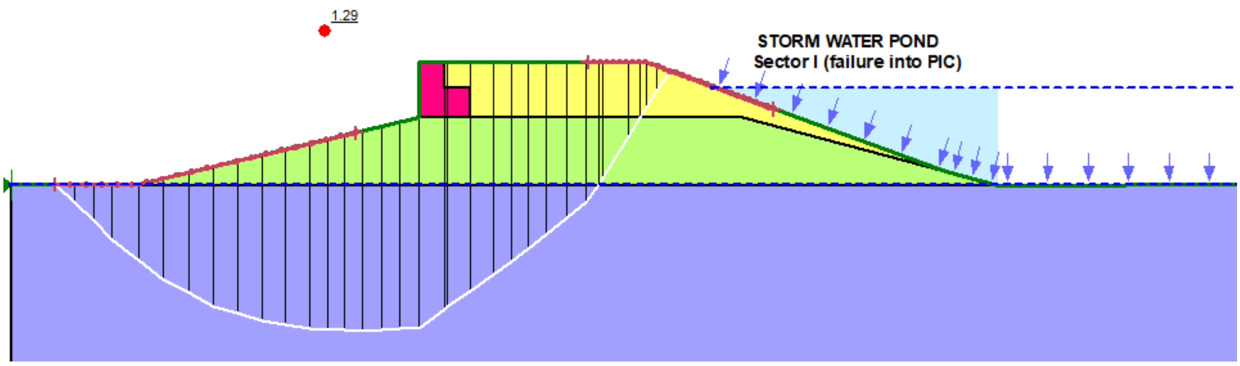


Figure D6: Undrained (Total Stress) Static Analysis (Post seismic) – Sector I Failure to PIC

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Estuarine	Mohr-Coulomb	17	0	28	0	2
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0	1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0	1
	Rock fill	Mohr-Coulomb	22	0	45	0	1

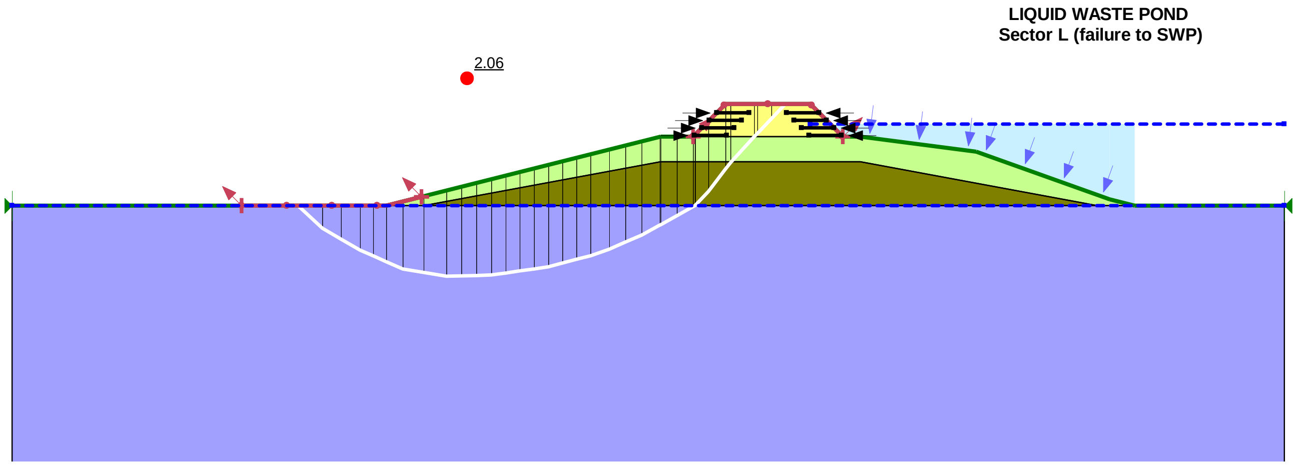


Figure D7: Drained (Effective Stress) Static Analysis – Sector J / L Failure to SWP

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Estuarine (undrained)	SHANSEP	17				25	0.25	2
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0			1
	Rock fill	Mohr-Coulomb	22	0	45	0			1

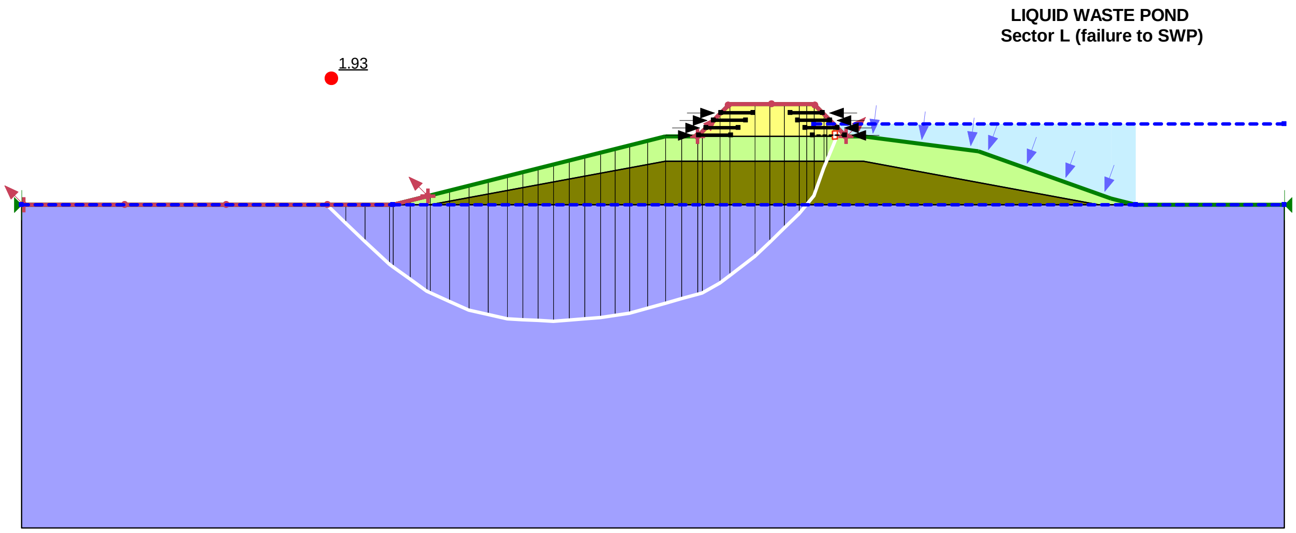


Figure D8: Undrained (Total Stress) Static Analysis – Sector J / L Failure to SWP

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Estuarine (undrained) (PS)	SHANSEP	17				20	0.2	2
	Processed Glacial Till	Mohr-Coulomb	19	0	32	0			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0			1
	Rock fill	Mohr-Coulomb	22	0	45	0			1

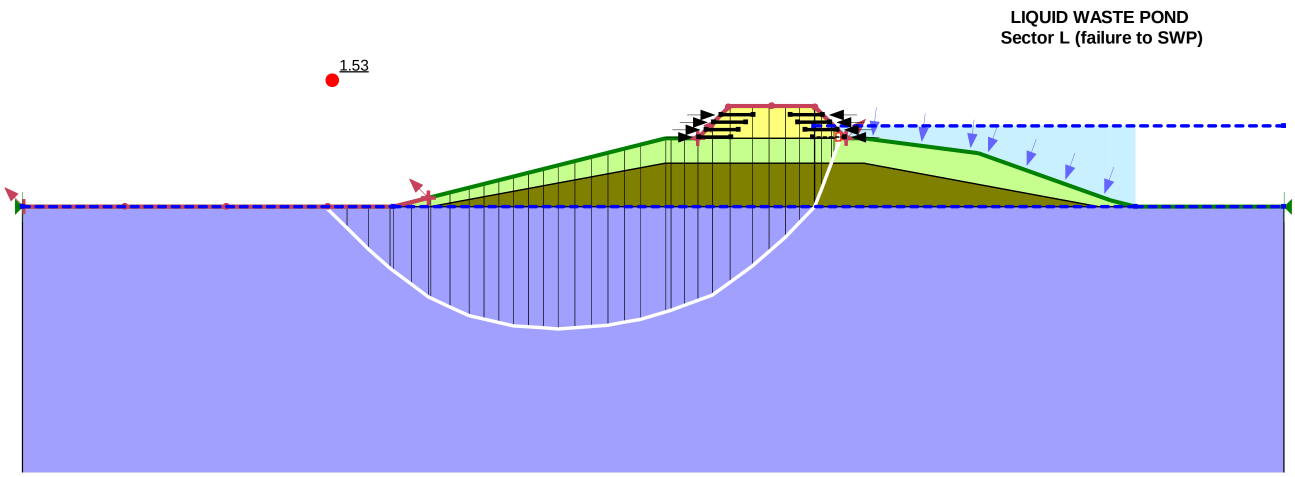


Figure D9: Undrained (Total Stress) Static Analysis (Post seismic) – Sector J / L Failure to SWP

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Concrete	Mohr-Coulomb	24	200	45	0	1
	Estuarine	Mohr-Coulomb	17	0	28	0	2
	Rock Fill	Mohr-Coulomb	22	0	45	0	1

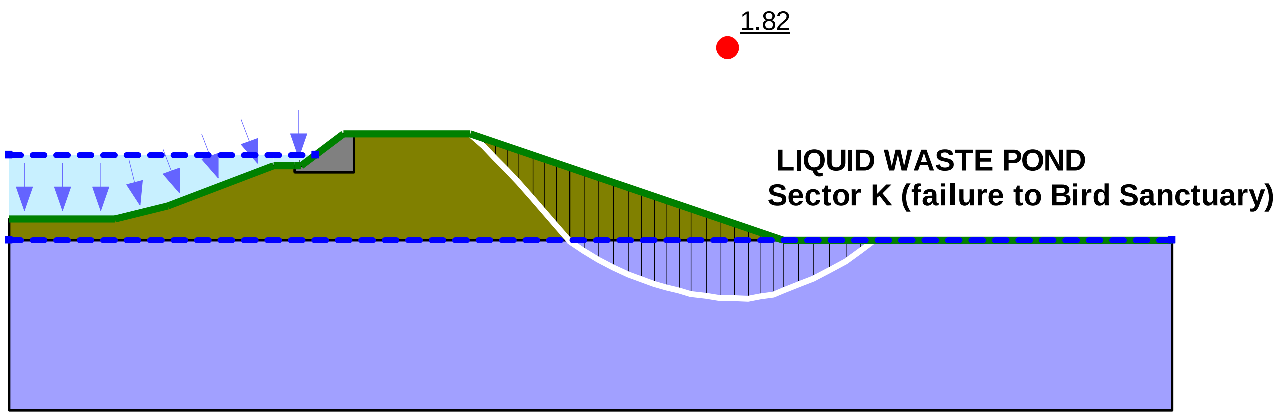


Figure D10: Drained (Effective Stress) Static Analysis – Sector K Failure to Bird Sanctuary

Color	Name	Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Concrete	Mohr-Coulomb	24			200	45	0	1
	Estuarine (undrained)	SHANSEP	17	25	0.25				2
	Rock Fill	Mohr-Coulomb	22			0	45	0	1

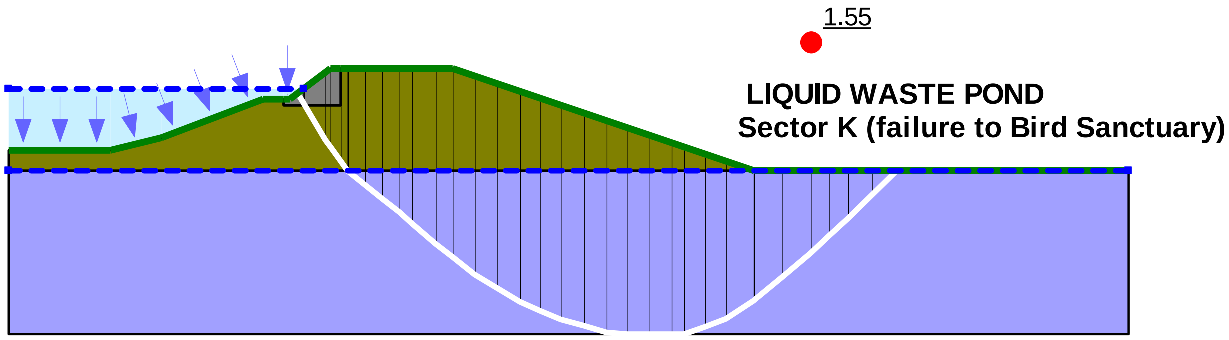


Figure D11: Undrained (Total Stress) Static Analysis – Sector K Failure to Bird Sanctuary

Color	Name	Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Concrete	Mohr-Coulomb	24			200	45	0	1
	Estuarine (undrained) (PS)	SHANSEP	17	20	0.2				2
	Rock Fill	Mohr-Coulomb	22			0	45	0	1

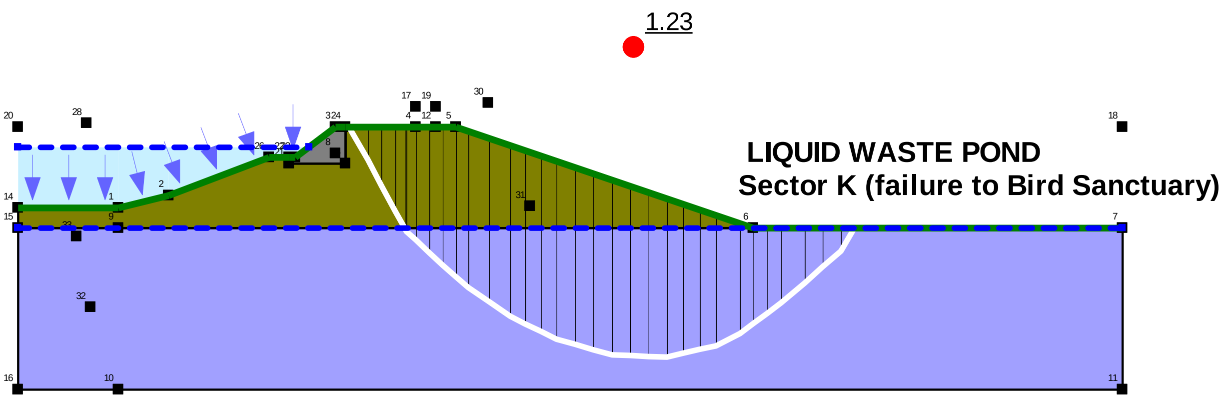


Figure D12: Undrained (Total Stress) Static Analysis (Post seismic) – Sector K Failure to Bird Sanctuary

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Esturaine	Mohr-Coulomb	17	0	28	0	2
	Gabion	Mohr-Coulomb	22	200	45	0	1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0	1
	Rock Fill	Mohr-Coulomb	22	0	45	0	1

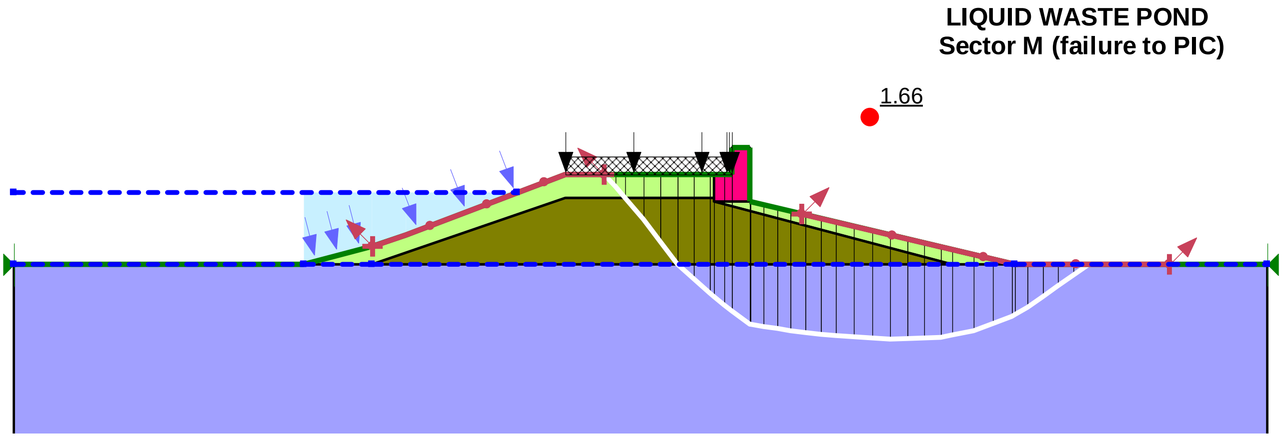


Figure D13: Drained (Effective Stress) Static Analysis – Sector M Failure to PIC

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Esturaine (Undrained)	SHANSEP	17				25	0.25	2
	Gabion	Mohr-Coulomb	22	200	45	0			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0			1
	Rock Fill	Mohr-Coulomb	22	0	45	0			1

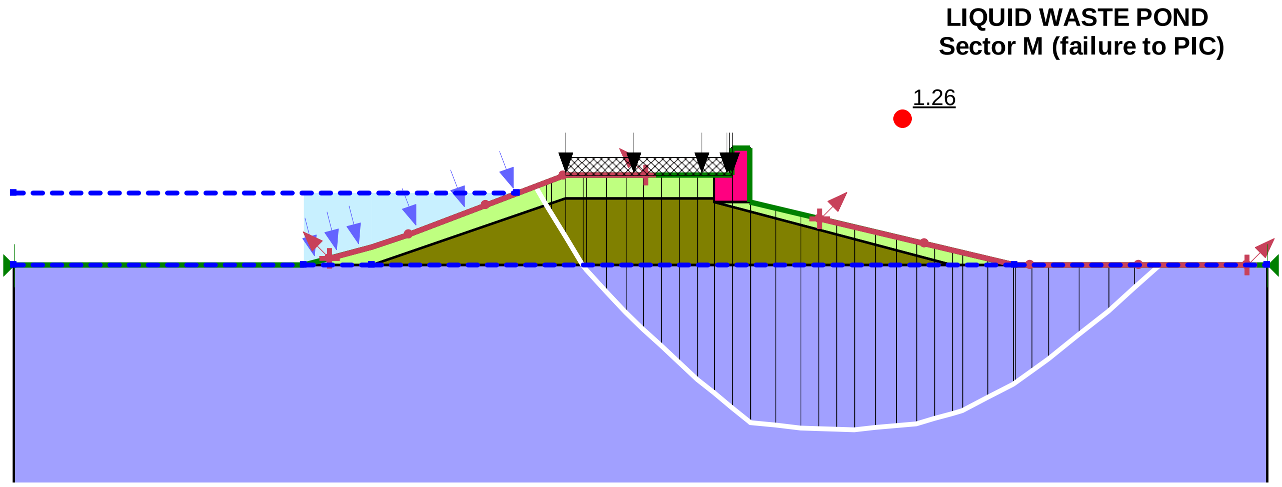


Figure D14: Undrained (Total Stress) Static Analysis – Sector M Failure to PIC

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Esturaine (Undrained) (PS)	SHANSEP	17				20	0.2	2
	Gabion	Mohr-Coulomb	22	200	45	0			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	0			1
	Rock Fill	Mohr-Coulomb	22	0	45	0			1

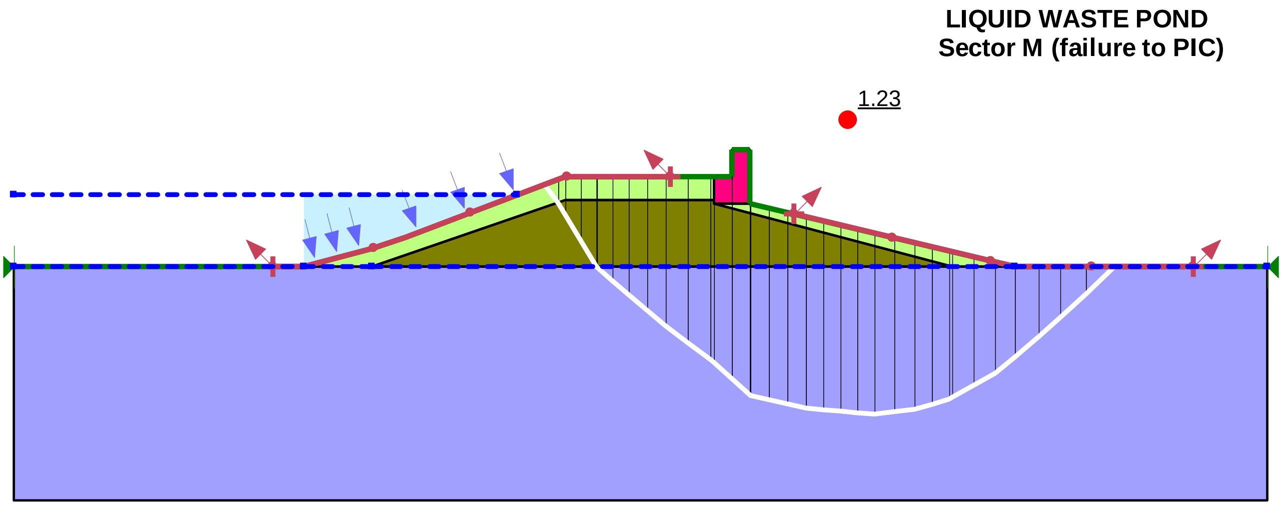


Figure D15: Undrained (Total Stress) Static Analysis (Post seismic) – Sector M Failure to PIC

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Estuarine Soil (Drained)	Mohr-Coulomb	17	0	28	2
	Processed Glacial Till	Mohr-Coulomb	20	0	32	1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32	1
	Processed Rock Fill	Mohr-Coulomb	21	0	40	
	Random Rock Fill	Mohr-Coulomb	22	0	45	

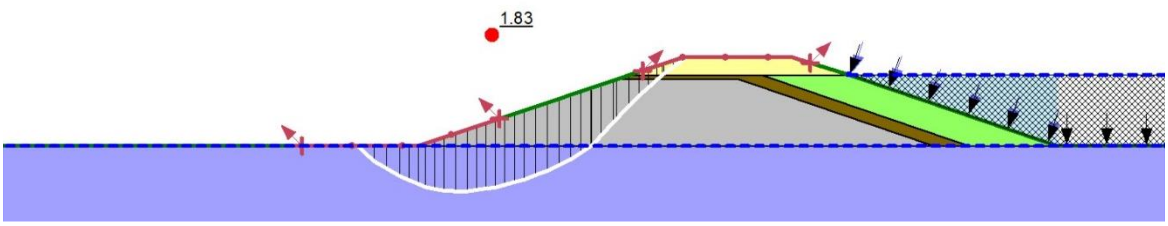


Figure D16: Drained (Effective Stress) Static Analysis – PIC Failure

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Estuarine (undrained)	SHANSEP	17			25	0.25	2
	Processed Glacial Till	Mohr-Coulomb	20	0	32			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32			1
	Processed Rock Fill	Mohr-Coulomb	21	0	40			
	Random Rock Fill	Mohr-Coulomb	22	0	45			

Figure D17: Undrained (Total Stress) Static Analysis – PIC Failure

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Estuarine (undrained) (PS)	SHANSEP	17			20	0.2	2
	Processed Glacial Till	Mohr-Coulomb	20	0	32			1
	Processed Glacial Till (2)	Mohr-Coulomb	19	0	32			1
	Processed Rock Fill	Mohr-Coulomb	21	0	40			
	Random Rock Fill	Mohr-Coulomb	22	0	45			

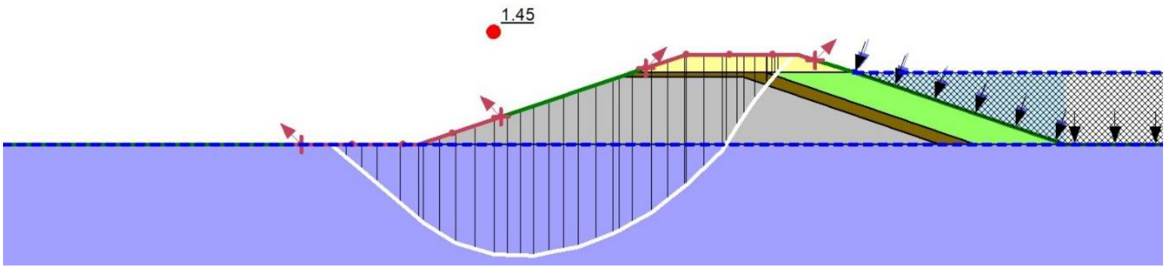
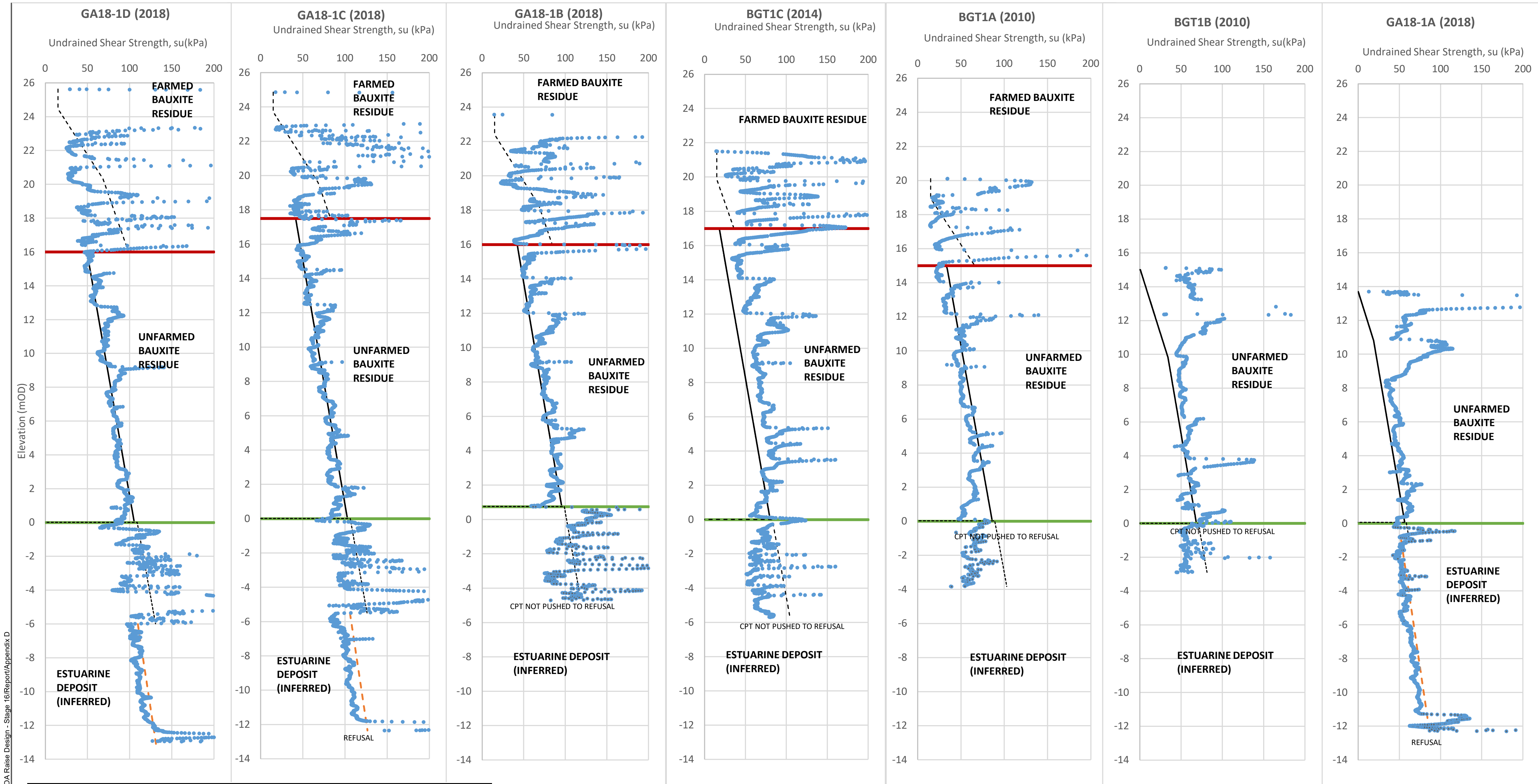


Figure D18: Undrained (Total Stress) Static Analysis (Post seismic) – PIC Failure

6.0 REFERENCES

Golder 2018, Golder Associates Ireland Limited (Golder), AAL BRDA 2018 CPT and Monitoring Instrument Installations, Phase 1 BRDA, 1895229.R01.A1, October 2018



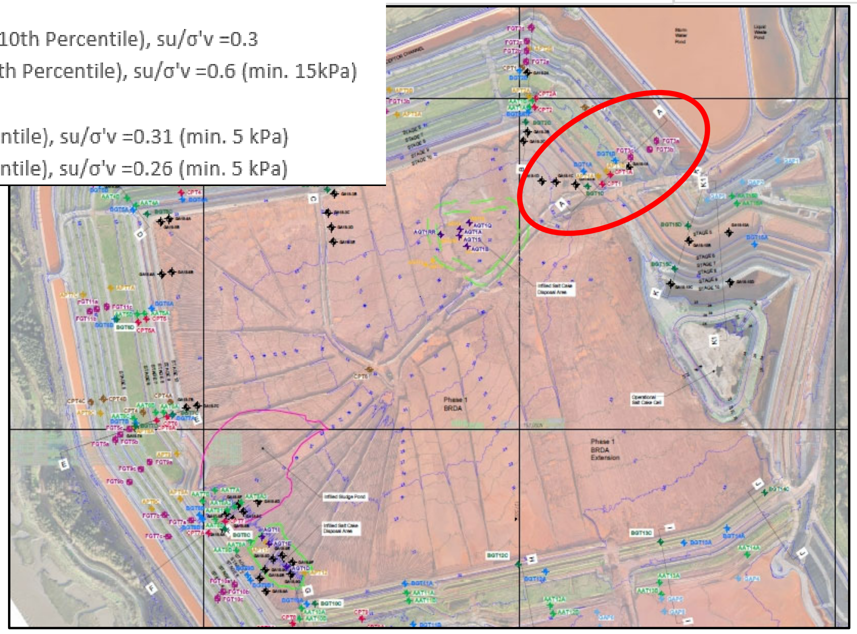
- su (Nkt=14)
- Unfarmed Bauxite Residue (10th Percentile), su/σ'v =0.3
- Farmed Bauxite Residue (10th Percentile), su/σ'v =0.6 (min. 15kPa)
- su (Nkt=15)
- Estuarine Upper (30th Percentile), su/σ'v =0.31 (min. 5 kPa)
- Estuarine Lower (30th Percentile), su/σ'v =0.26 (min. 5 kPa)

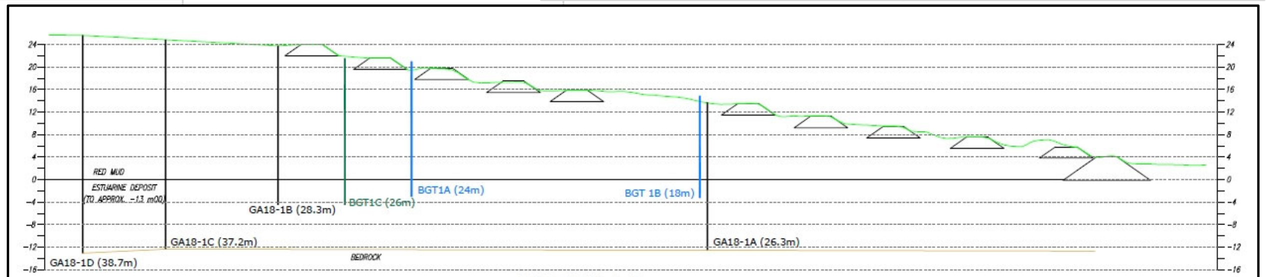
CPT LEGEND:

- GA18 Cone penetration test (2018)
- AGT Cone penetration test (Salt Cake 2014)
- AGT Cone penetration test (Red Mud 2014)
- BGT Cone penetration test (2014)
- BGT Cone penetration test (2010)
- AAT Cone penetration test (2009)
- CPT Cone penetration test (2007)
- APT Cone penetration test (2006)
- GAP Cone penetration test (2005)
- FGT Cone penetration test (2004)
- CPTS Cone penetration test (2003)

NOTES:

1. Elevations Shown are approximate



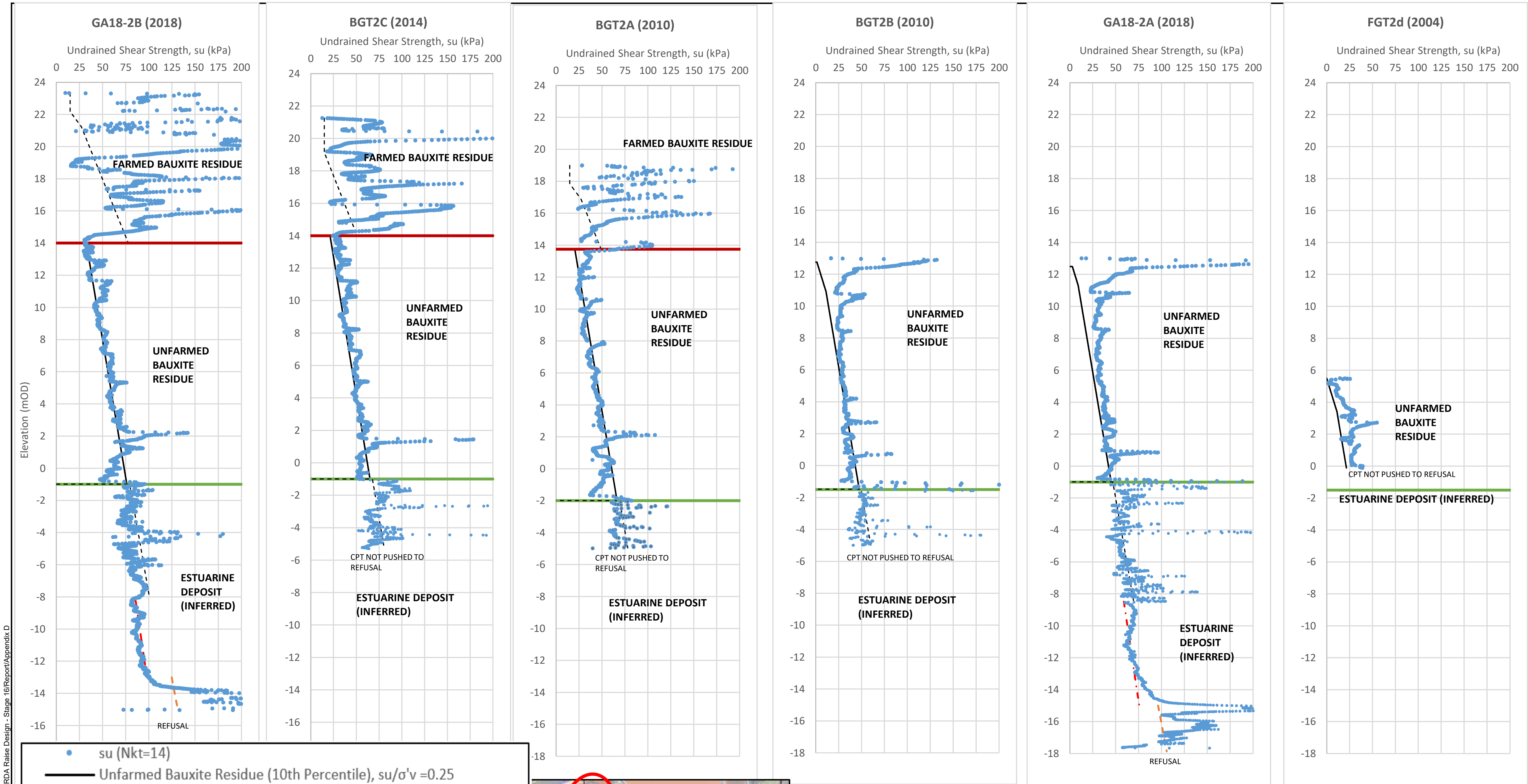


SECTION A-A

CLIENT	PROJECT
AUGHINISH ALUMINA LIMITED	BRDA Raise Development Appendix D-3: Stability Assessment

CONSULTANT	YYY/MM/DD	01/10/2021	TITLE
GOLDER MEMBER OF WSP	PREPARED	BM	SECTION A-A
	DESIGN	GLJ	UNDRAINED SHEAR STRENGTH FROM CPT DATA
	REVIEW	BK	
	APPROVED	GLJ	

PROJECT No.	Rev.	FIG
20143076	A	D-3-1



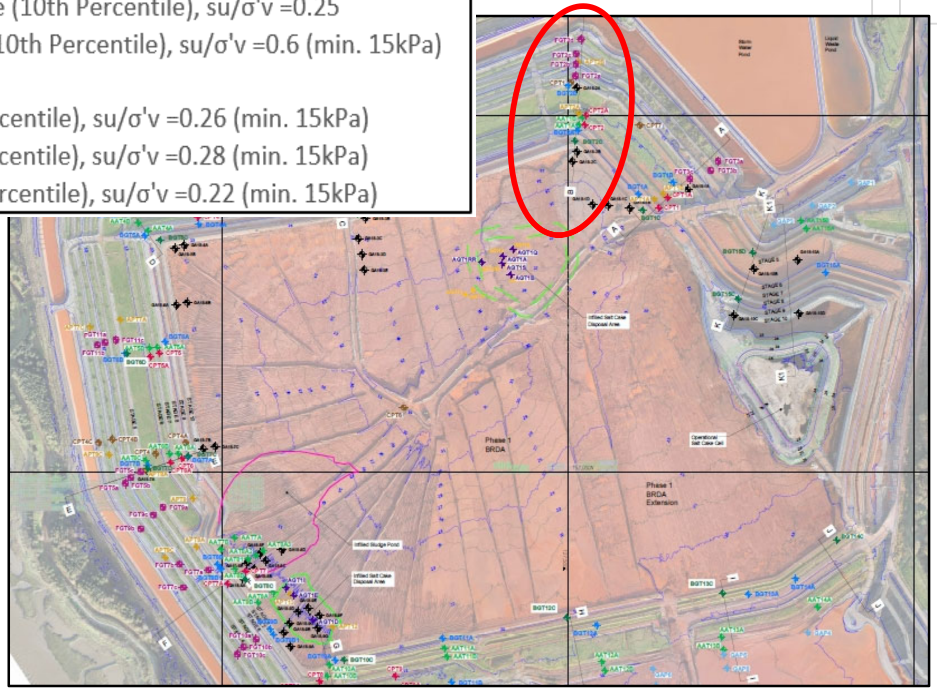
- su (Nkt=14)
- Unfarmed Bauxite Residue (10th Percentile), su/σ'v =0.25
- Farmed Bauxite Residue (10th Percentile), su/σ'v =0.6 (min. 15kPa)
- su (Nkt=15)
- Estuarine Upper (30th Percentile), su/σ'v =0.26 (min. 15kPa)
- Estuarine Lower (30th Percentile), su/σ'v =0.28 (min. 15kPa)
- Estuarine Middle (30th Percentile), su/σ'v =0.22 (min. 15kPa)

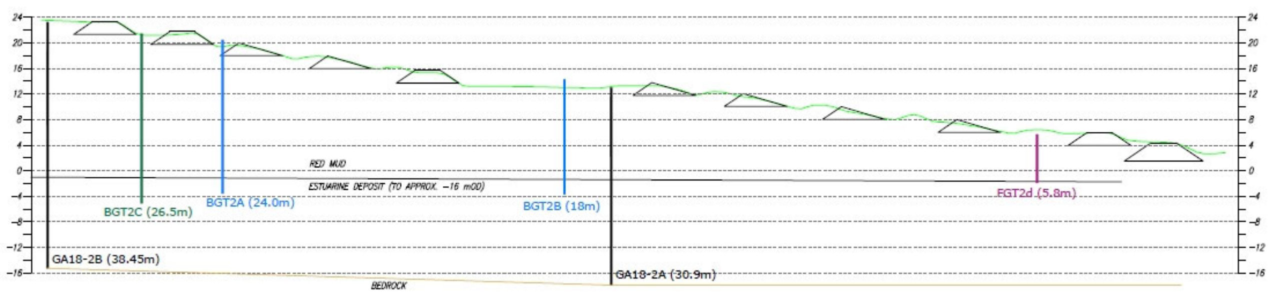
CPT LEGEND:

GA18	Cone penetration test (2018)
AGT	Cone penetration test (Salt Cake 2014)
AGT	Cone penetration test (Red Mud 2014)
BGT	Cone penetration test (2014)
BGT	Cone penetration test (2010)
AAT	Cone penetration test (2009)
CPT	Cone penetration test (2007)
APT	Cone penetration test (2006)
GAP	Cone penetration test (2005)
FGT	Cone penetration test (2004)
CPTS	Cone penetration test (2003)

NOTES:

1. Elevations Shown are approximate





SECTION B-B

CLIENT
AUGHINISH ALUMINA LIMITED

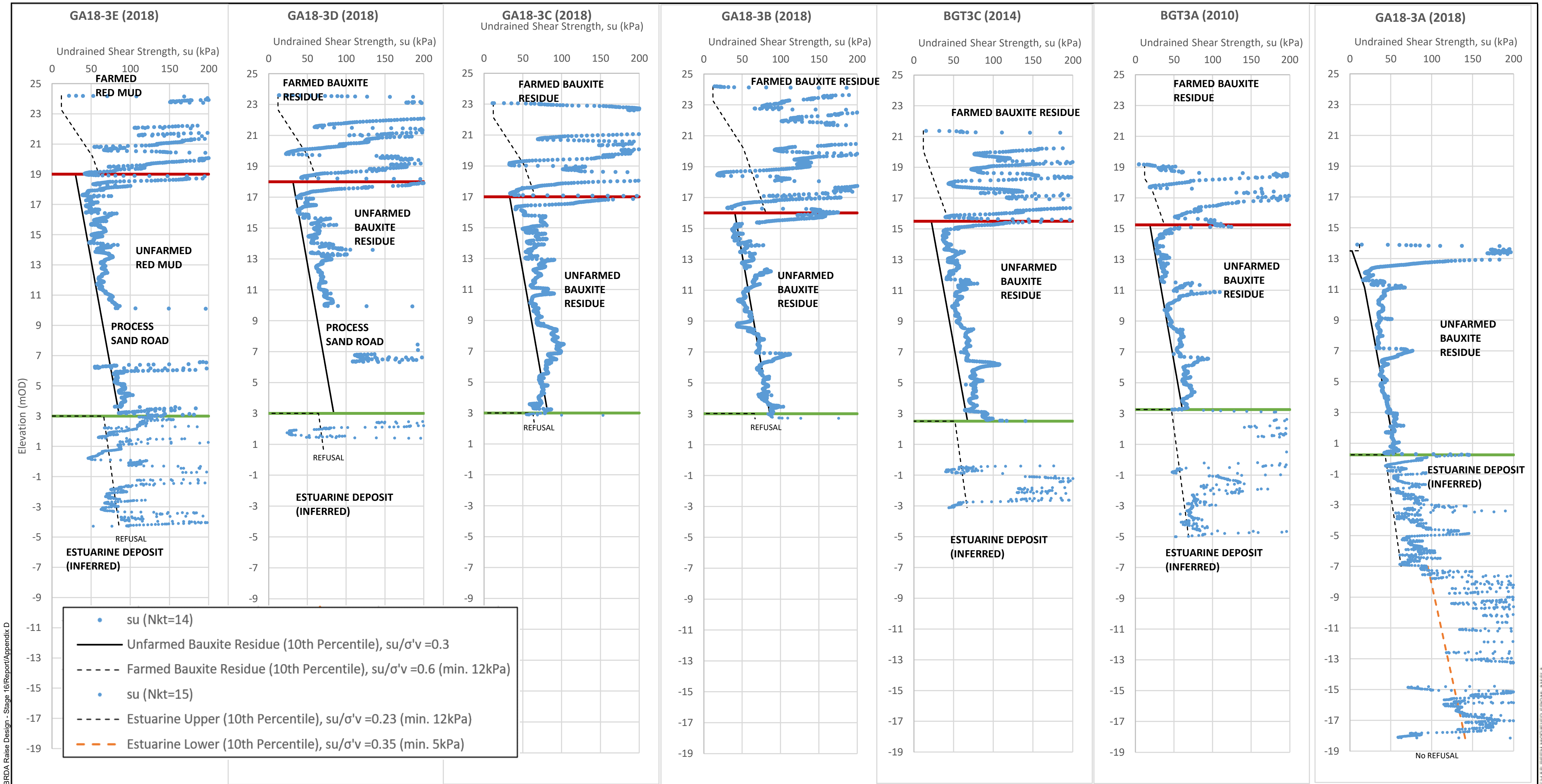
PROJECT
BRDA Raise Development
Appendix D-3: Stability Assessment

CONSULTANT
GOLDER MEMBER OF WSP

YYYY-MM-DD	01/10/2021
PREPARED	BM
DESIGN	GLJ
REVIEW	BK
APPROVED	GLJ

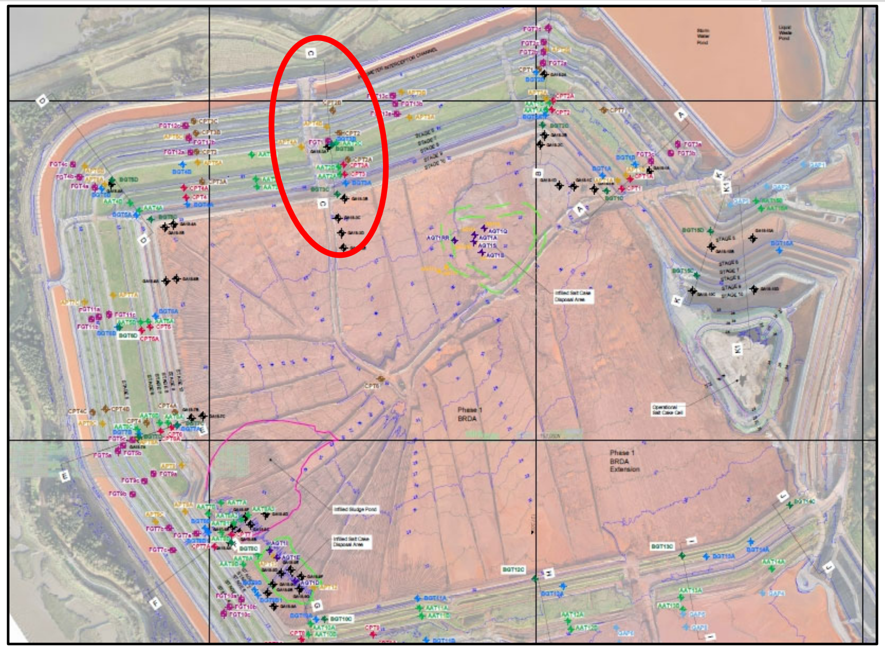
TITLE
SECTION B-B
UNDRAINED SHEAR STRENGTH FROM CPT DATA

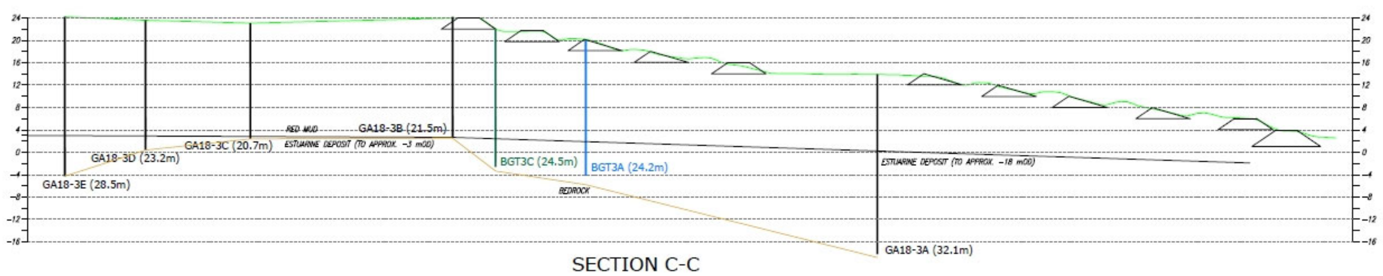
PROJECT No.	Rev.	FIG
20143076	A	D-3-2



CPT LEGEND:

GA18	Cone penetration test (2018)
AGT	Cone penetration test (Salt Cake 2014)
AGT	Cone penetration test (Red Mud 2014)
BGT	Cone penetration test (2014)
BGT	Cone penetration test (2010)
AAT	Cone penetration test (2009)
CPT	Cone penetration test (2007)
APT	Cone penetration test (2006)
GAP	Cone penetration test (2005)
FGT	Cone penetration test (2004)
CPTS	Cone penetration test (2003)





SECTION C-C

NOTES:

1. Elevations Shown are approximate

CLIENT
AUGHINISH ALUMINA LIMITED

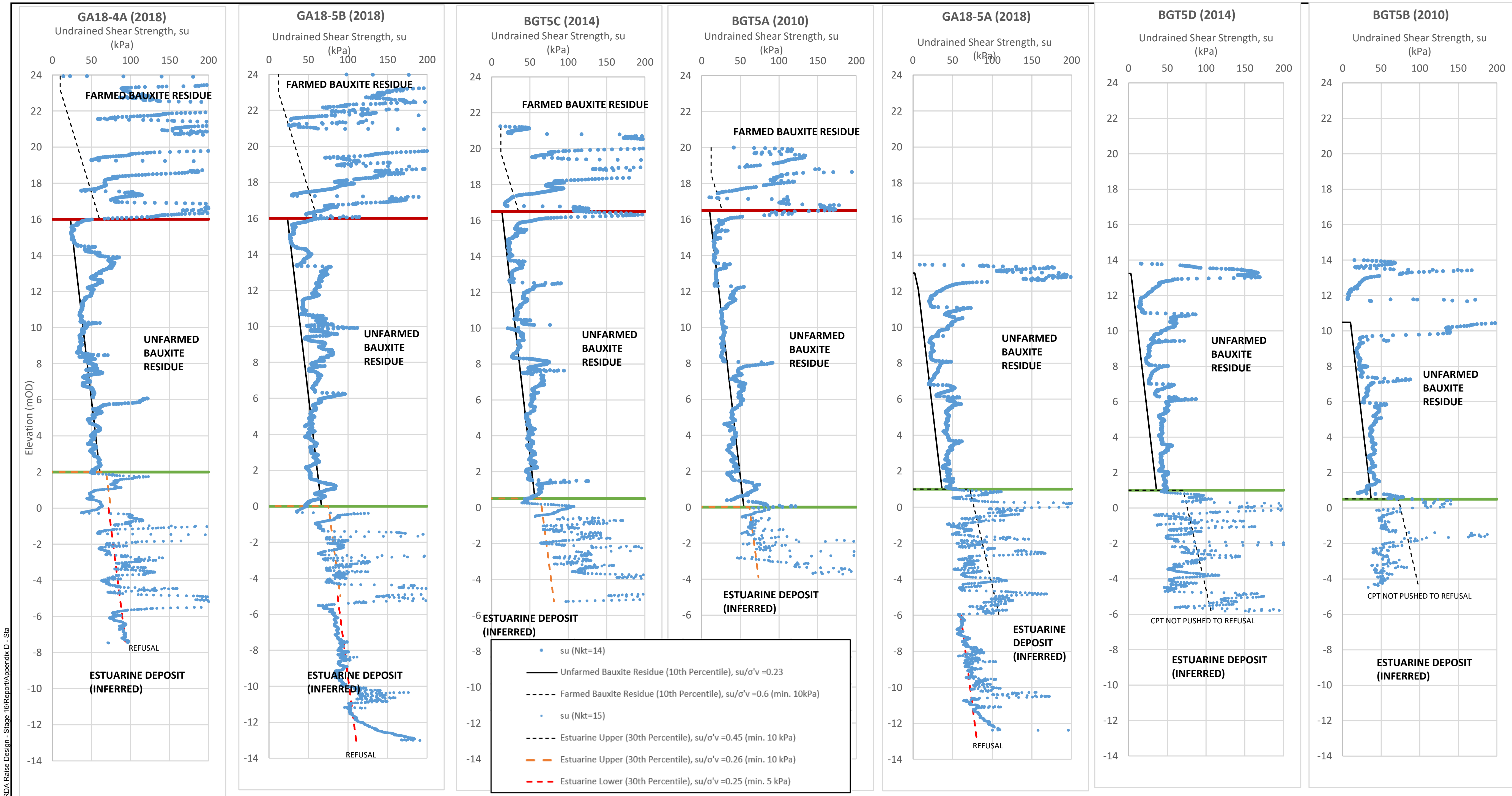
PROJECT
BRDA Raise Development
Appendix D-3: Stability Assessment

CONSULTANT
GOLDER MEMBER OF WSP

YYYY/MM/DD	01/10/2021
PREPARED	BM
DESIGN	GLJ
REVIEW	BK
APPROVED	GLJ

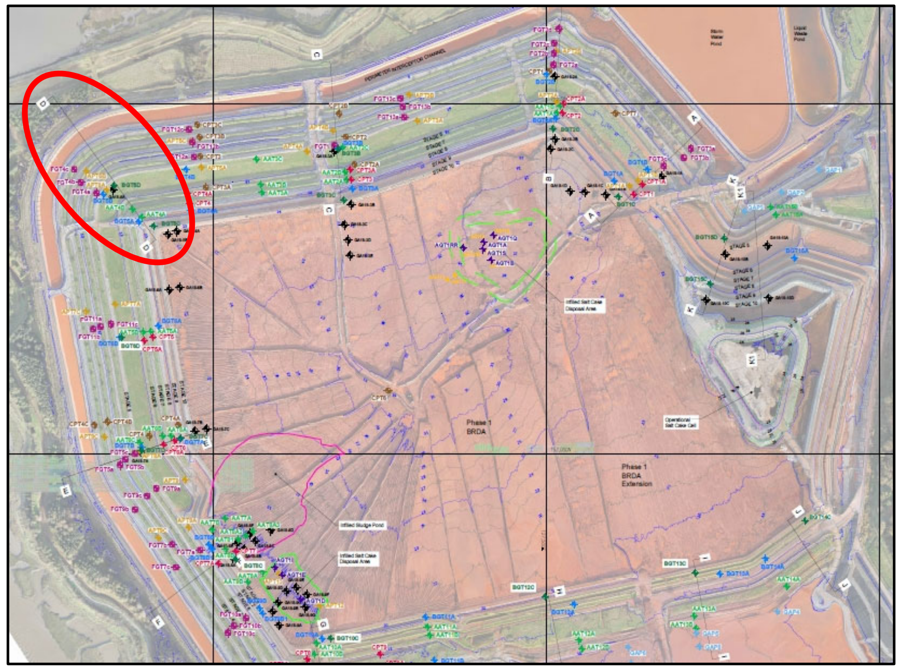
TITLE
SECTION C-C
UNDRAINED SHEAR STRENGTH FROM CPT DATA

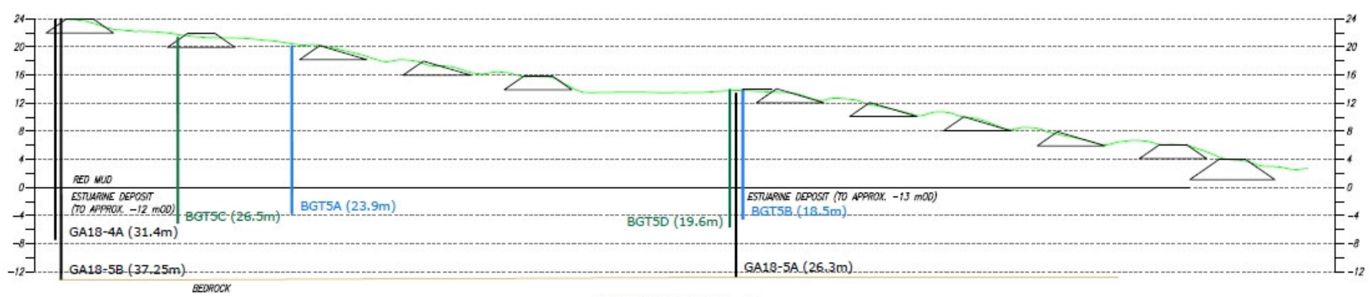
PROJECT No.	Rev.	FIG
20143076	A	D-3-3



CPT LEGEND:

GA18	Cone penetration test (2018)
AGT	Cone penetration test (Salt Cake 2014)
AGT	Cone penetration test (Red Mud 2014)
BGT	Cone penetration test (2014)
BGT	Cone penetration test (2010)
AAT	Cone penetration test (2009)
CPT	Cone penetration test (2007)
APT	Cone penetration test (2006)
GAP	Cone penetration test (2005)
FGT	Cone penetration test (2004)
CPTS	Cone penetration test (2003)





SECTION D-D

NOTES:
1. Elevations Shown are approximate

CLIENT
AUGHINISH ALUMINA LIMITED

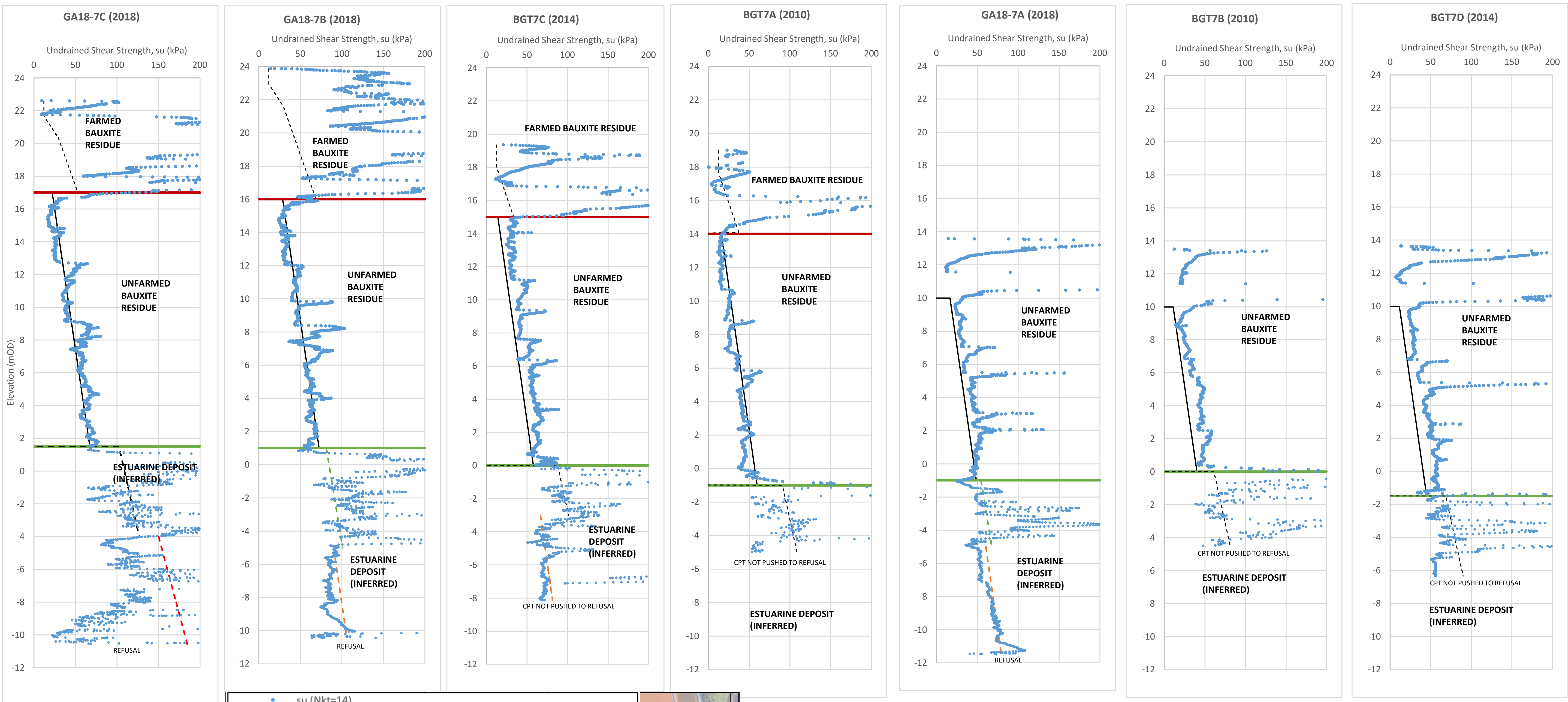
PROJECT
BRDA Raise Development
Appendix D-3: Stability Assessment

CONSULTANT
GOLDER MEMBER OF WSP

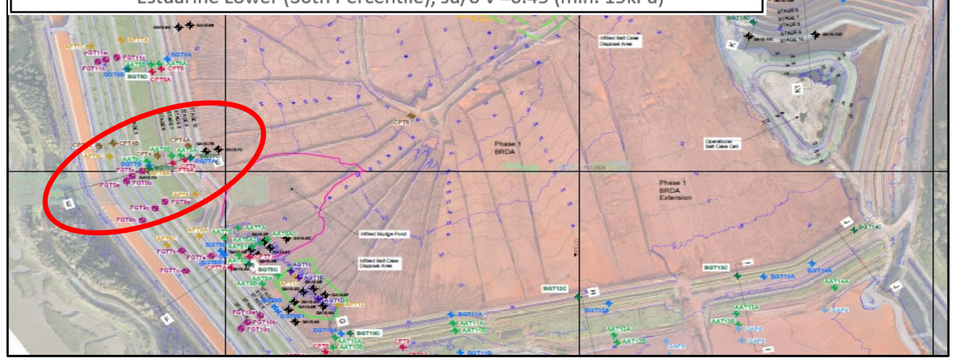
YYYY/MM/DD	01/10/2021
PREPARED	BM
DESIGN	GLJ
REVIEW	BK
APPROVED	GLJ

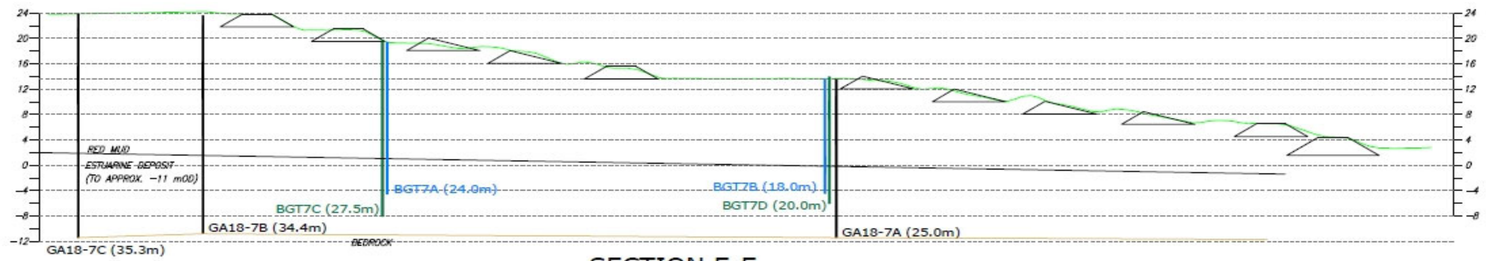
TITLE
SECTION D-D
UNDRAINED SHEAR STRENGTH FROM CPT DATA

PROJECT No. 20143076 Rev. A FIG D-3-4



- su (Nkt=14)
- Unfarmed Bauxite Residue (10th Percentile), su/σ'v =0.25
- Farmed Bauxite Residue (10th Percentile), su/σ'v =0.6 (min. 12kPa)
- su (Nkt=15)
- Estuarine Upper (30th Percentile), su/σ'v =0.28 (min. 15kPa)
- Estuarine Upper (30th Percentile), su/σ'v =0.38 (min. 15kPa)
- Estuarine Lower (30th Perentile), su/σ'v =0.25 (min. 15kPa)
- Estuarine Lower (30th Percentile), su/σ'v =0.45 (min. 15kPa)





SECTION E-E

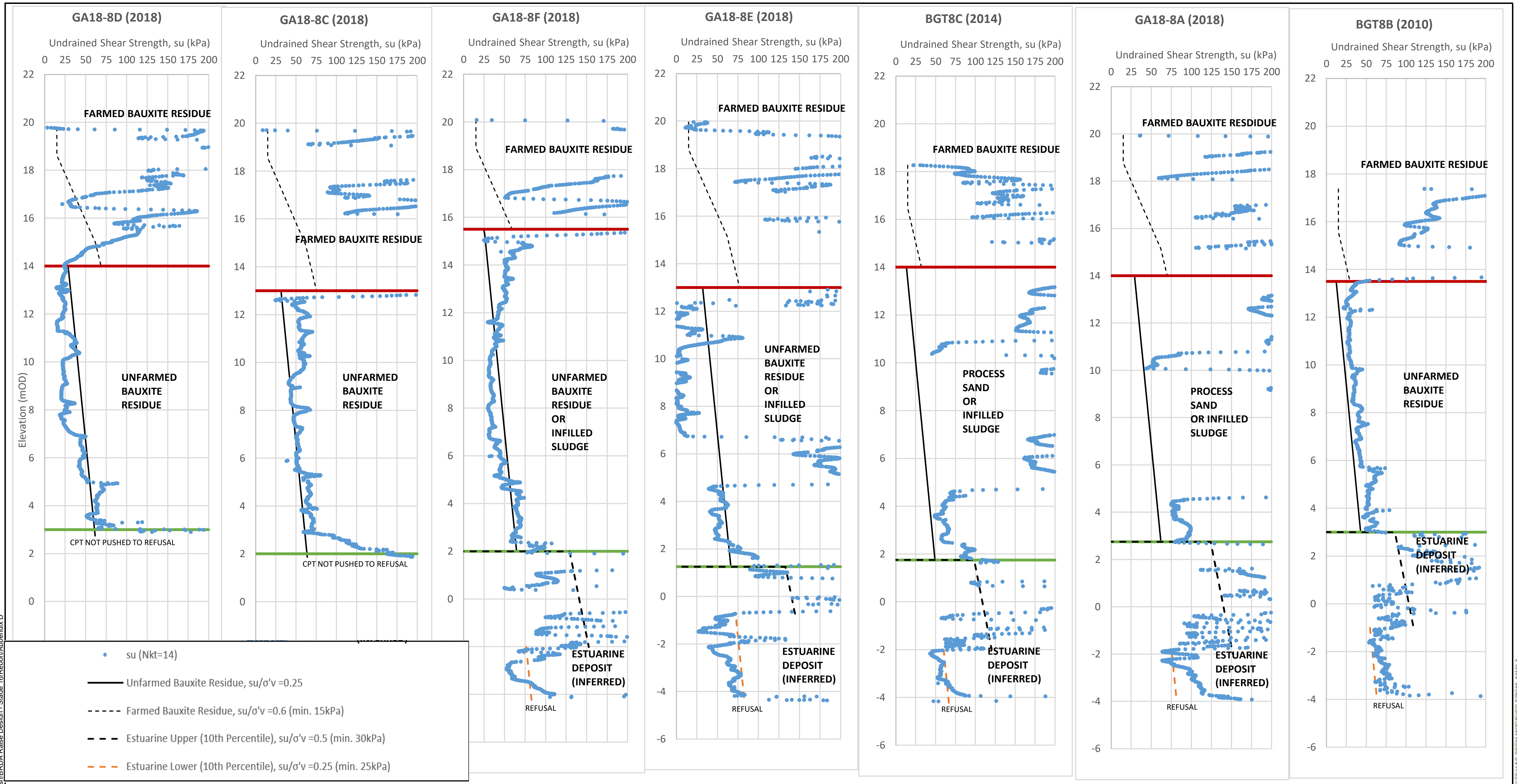
CPT LEGEND:

GA18	Cone penetration test (2018)
AGT	Cone penetration test (Salt Cake 2014)
AGT	Cone penetration test (Red Mud 2014)
BGT	Cone penetration test (2014)
BGT	Cone penetration test (2010)
AAT	Cone penetration test (2009)
CPT	Cone penetration test (2007)
APT	Cone penetration test (2006)
GAP	Cone penetration test (2005)
FGT	Cone penetration test (2004)
CPTS	Cone penetration test (2003)

NOTES:

1. Elevations Shown are approximate

CLIENT	AUGHINISH ALUMINA LIMITED
PROJECT	BRDA Raise Development Appendix D-3: Stability Assessment
CONSULTANT	GOLDER MEMBER OF WSP
YYYY/MM/DD	24/08/2018
PREPARED	BM
DESIGN	GLJ
REVIEW	BK
APPROVED	GLJ
TITLE	SECTION E-E UNDRAINED SHEAR STRENGTH FROM CPT DATA
PROJECT No.	20143076
Rev.	A
FIG	D-3-5

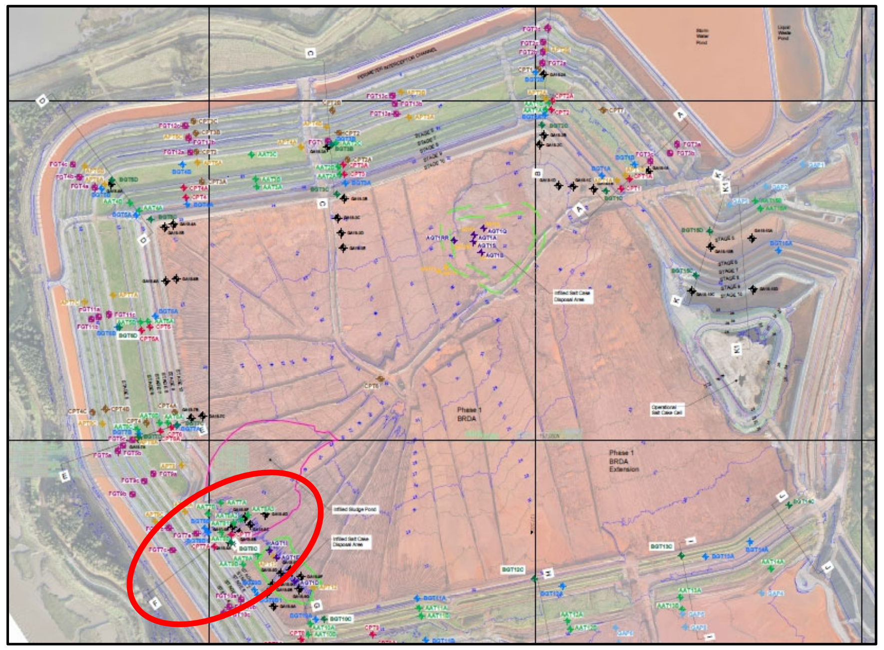


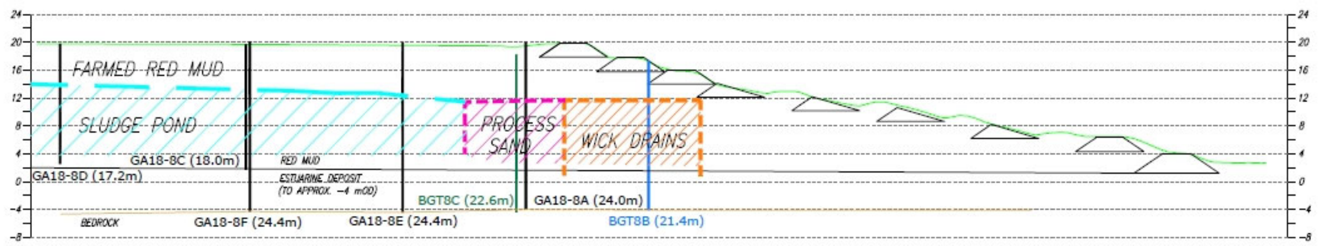
CPT LEGEND:

- GA18 Cone penetration test (2018)
- AGT Cone penetration test (Salt Cake 2014)
- AGT Cone penetration test (Red Mud 2014)
- BGT Cone penetration test (2014)
- BGT Cone penetration test (2010)
- AAT Cone penetration test (2009)
- CPT Cone penetration test (2007)
- APT Cone penetration test (2006)
- GAP Cone penetration test (2005)
- FGT Cone penetration test (2004)
- CPTS Cone penetration test (2003)

NOTES:

1. Elevations Shown are approximate





SECTION F-F (FIGURE 1)

CLIENT
AUGHINISH ALUMINA LIMITED

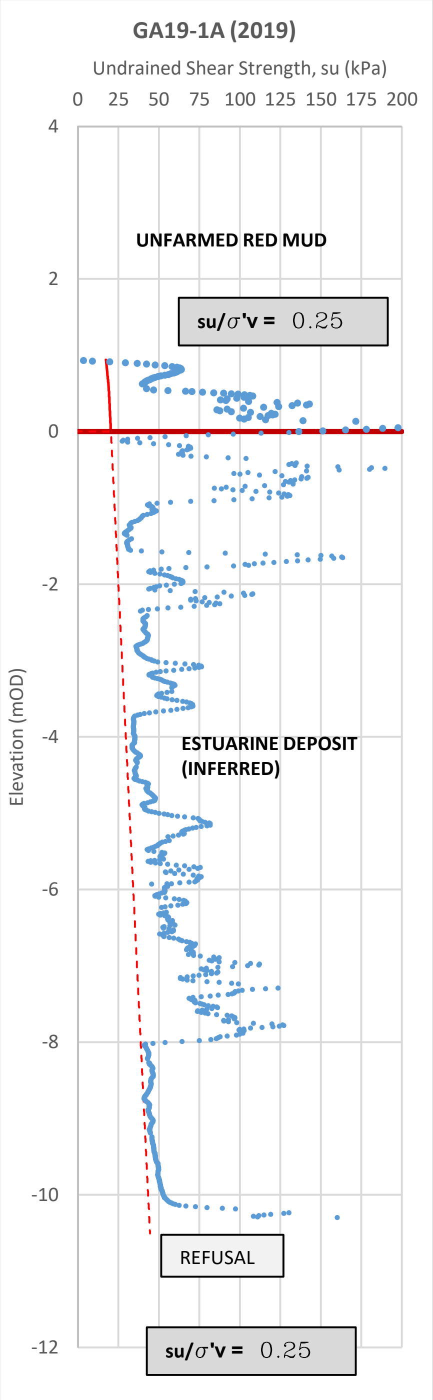
CONSULTANT
GOLDER MEMBER OF WSP

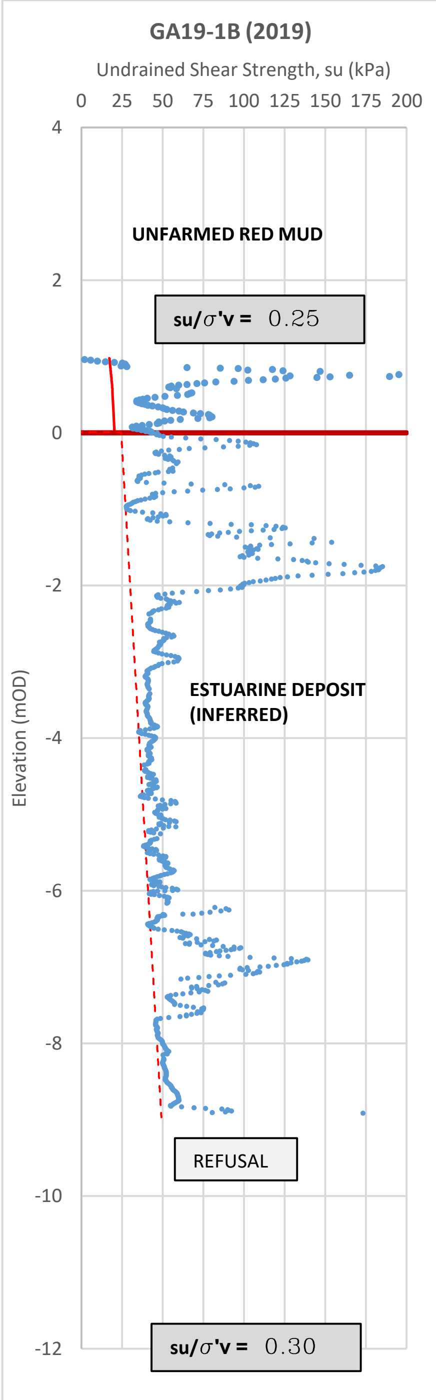
YYYY/MM/DD	01/10/2021
PREPARED	BM
DESIGN	GLJ
REVIEW	BK
APPROVED	GLJ

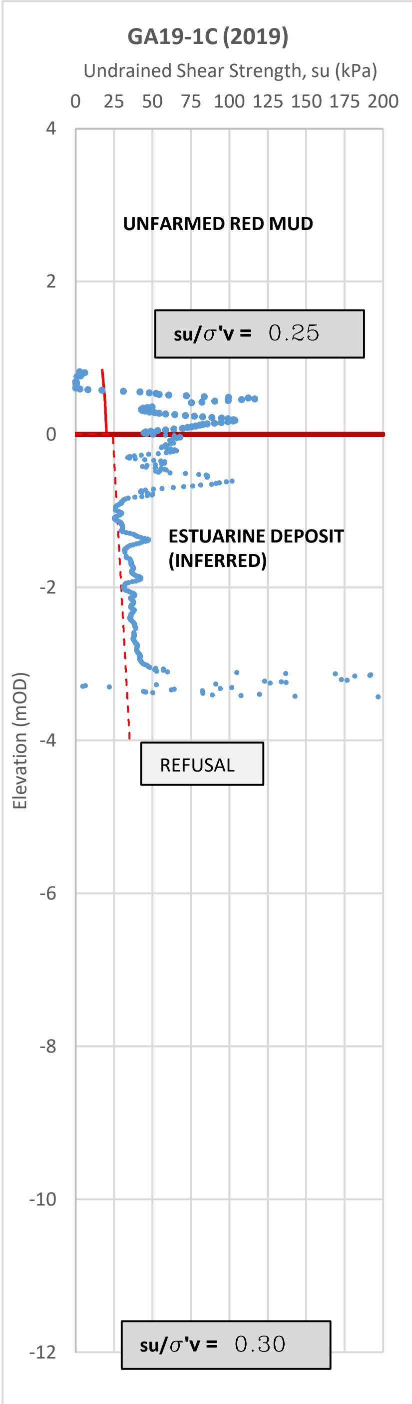
PROJECT
BRDA Raise Development
Appendix D-3: Stability Assessment

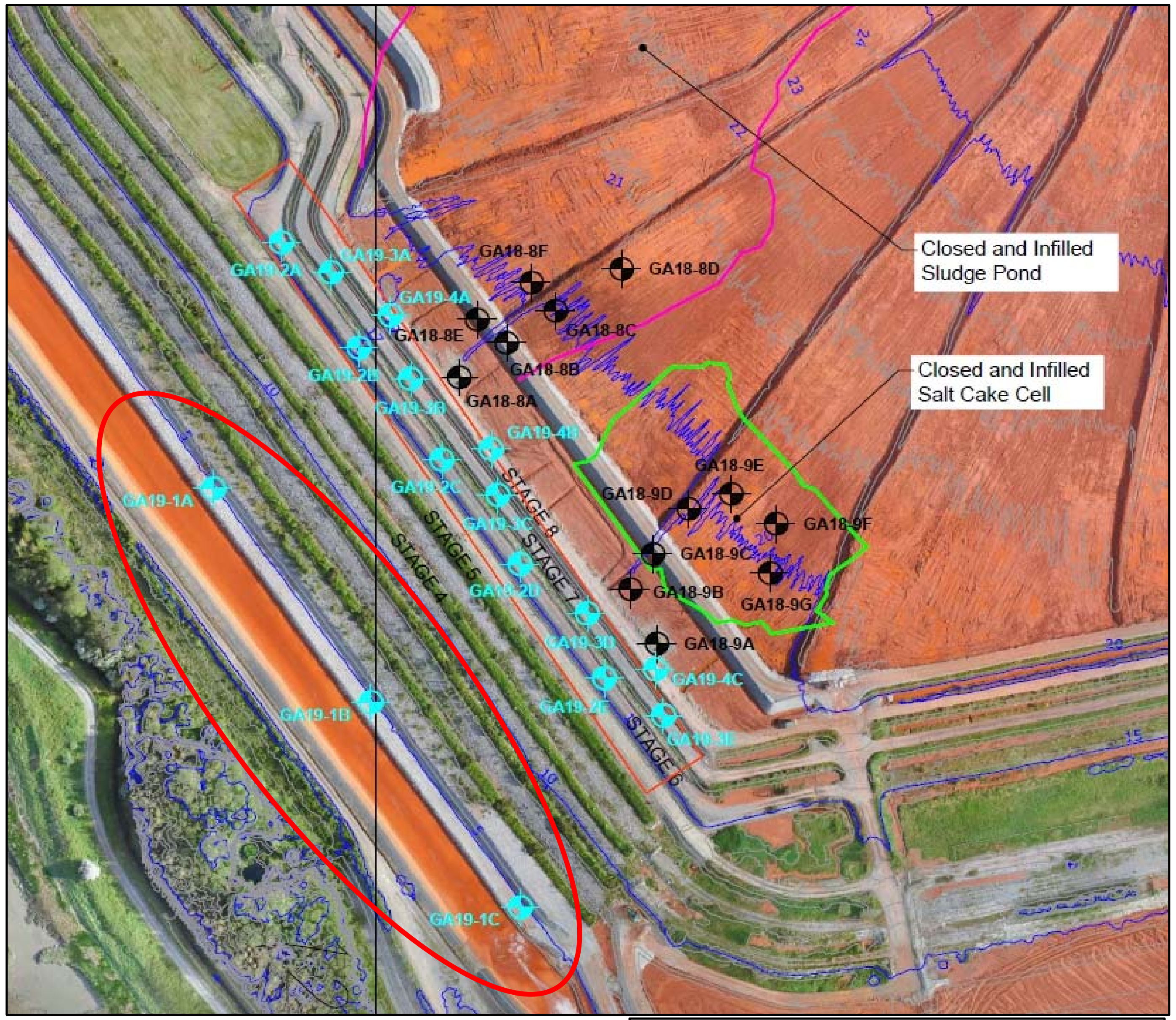
TITLE
SECTION F-F (FIGURE 1 OF 2)
UNDRAINED SHEAR STRENGTH FROM CPT DATA

PROJECT No.	Rev.	FIG
20143076	A	D-3-6









LEGEND

GA18-10D	2018 CPT PROFILE LOCATIONS
GA19-1A	2019 CPT PROFILE LOCATIONS

- su (Nkt=15 for Estuarine and = 14 for Bauxite Residue)
- Unfarmed Mud, su/σ'v =0.25 (min. 15 kPa)
- Estuarine, su/σ'v =0.25 to 0.30 (min. 5kPa)

NOTES:
1. Elevations Shown are approximate

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CLIENT
AUGHINISH ALUMINA LIMITED

CONSULTANT
GOLDER

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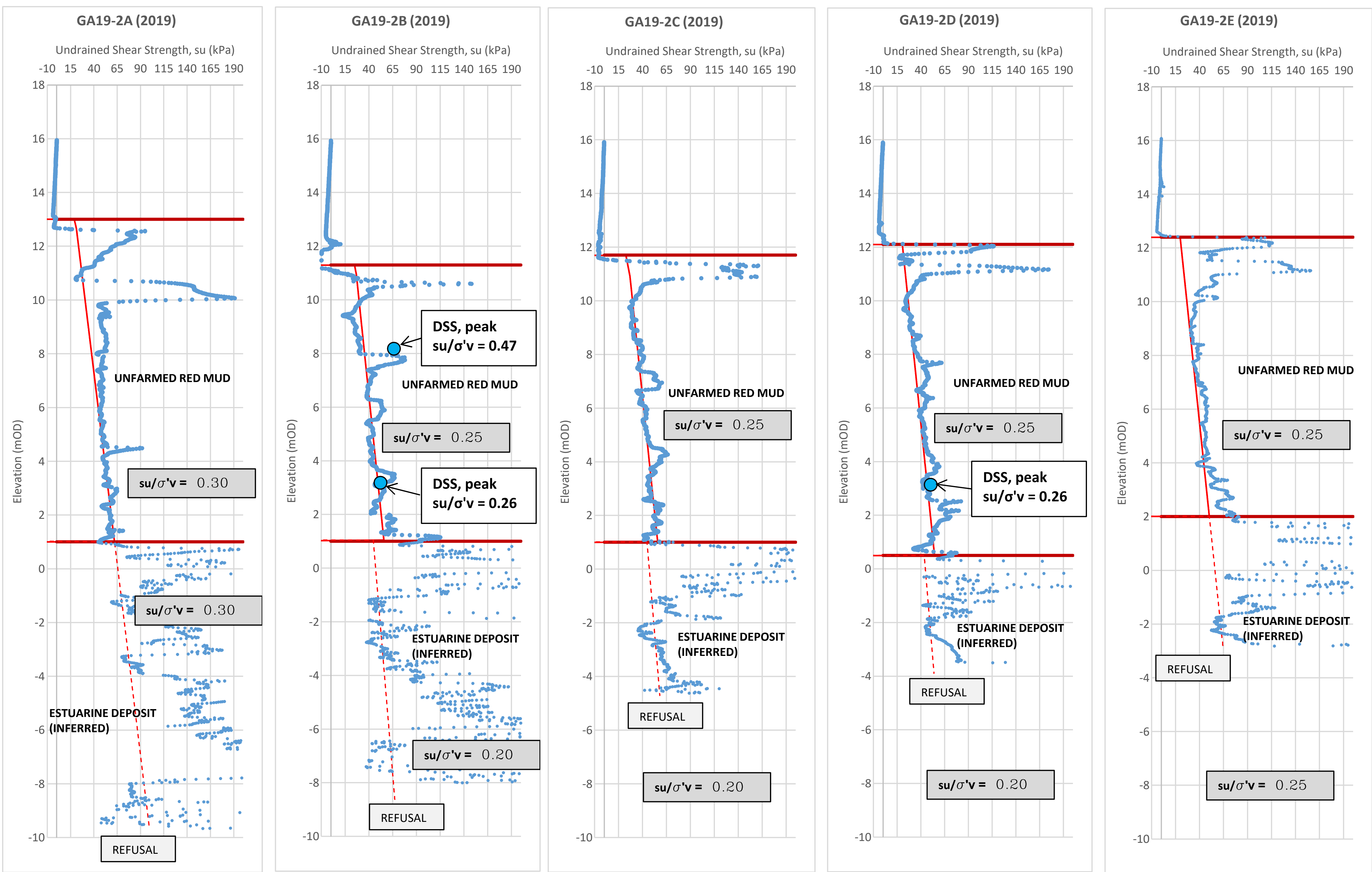
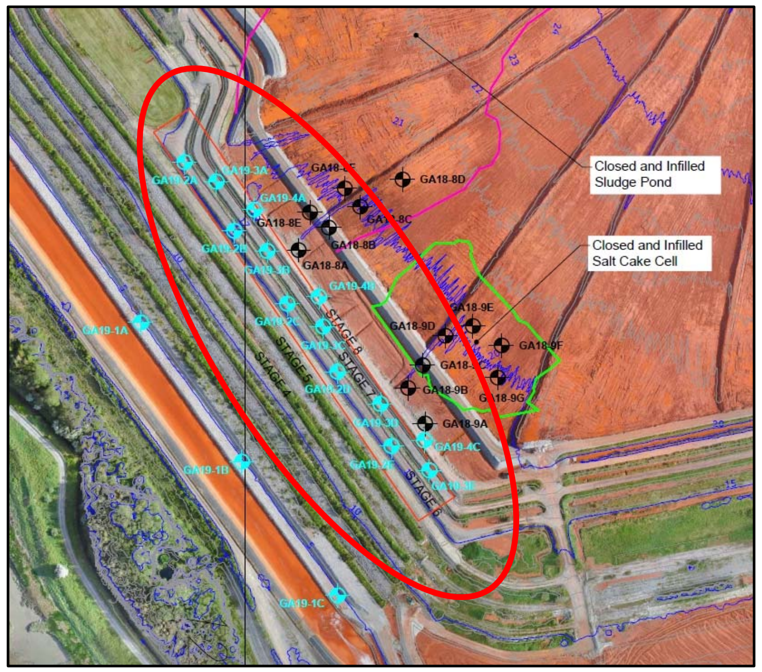
PREPARED	BM
DESIGN	BM
REVIEW	BK
APPROVED	GJ

PROJECT
AAL BRDA
CPT 2019 INVESTIGATION

TITLE
SECTION F-F, STAGE 0
UNDRAINED SHEAR STRENGTH FROM CPT DATA

PROJECT No.	Rev.	FIG
20143076	A	D-3-7





●	su (Nkt=15 for Estuarine and =14 for Bauxite Residue)
———	Unfarmed Mud, su/σ'v =0.25 (min. 15 kPa)
- - - -	Estuarine, su/σ'v =0.25 to 0.30 (min. 5kPa)

LEGEND

GA18-10D	2018 CPT PROFILE LOCATIONS
GA19-1A	2019 CPT PROFILE LOCATIONS

NOTES:

1. Elevations Shown are approximate

CLIENT
AUGHINISH ALUMINA LIMITED

CONSULTANT
GOLDER

YYYY/MM/DD	14/04/2020
PREPARED	BM
DESIGN	BM
REVIEW	BK
APPROVED	GJ

PROJECT
AAL BRDA
CPT 2019 INVESTIGATION

TITLE
SECTION F-F, STAGE 6
UNDRAINED SHEAR STRENGTH FROM CPT DATA

PROJECT No.	Rev.	FIG
20143076	A	D-3-8

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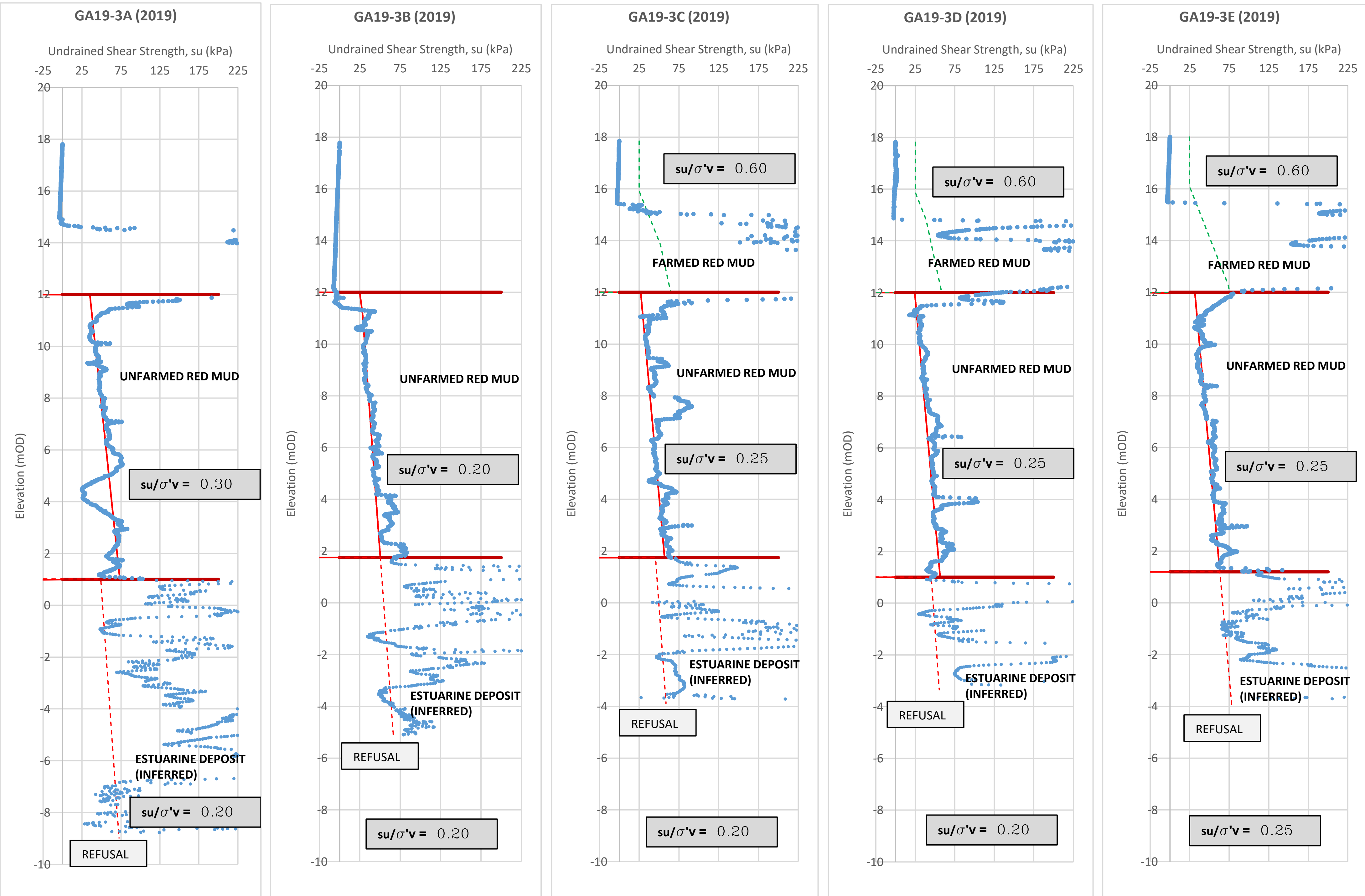
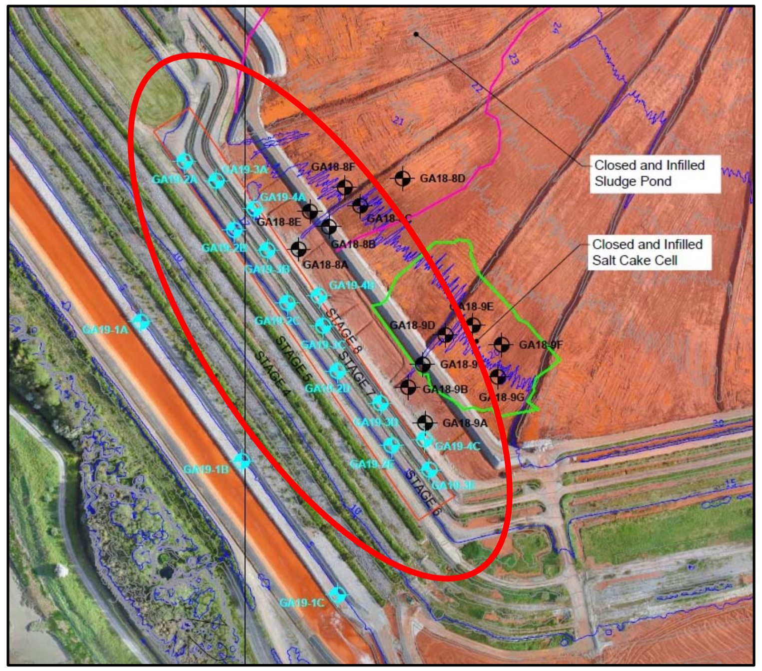
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A



su (Nkt=15 for Estuarine and = 14 for Bauxite Residue)
Farmed Mud, su/σ'v =0.6 (min. 25kPa)
Unfarmed Mud, su/σ'v =0.25 (min. 15 kPa)
Estuarine, su/σ'v =0.20 to 0.25 (min. kPakPa)



LEGEND

GA18-10D	2018 CPT PROFILE LOCATIONS
GA19-1A	2019 CPT PROFILE LOCATIONS

NOTES:

1. Elevations Shown are approximate

CLIENT	AUGHINISH ALUMINA LIMITED
CONSULTANT	GOLDER
YYYY/MM/DD	14/04/2020
PREPARED	BM
DESIGN	BM
REVIEW	BK
APPROVED	GJ
PROJECT	AAL BRDA CPT 2019 INVESTIGATION
TITLE	SECTION F-F, STAGE 7 UNDRAINED SHEAR STRENGTH FROM CPT DATA
PROJECT No.	20143076
Rev.	A
FIG	D-3-9

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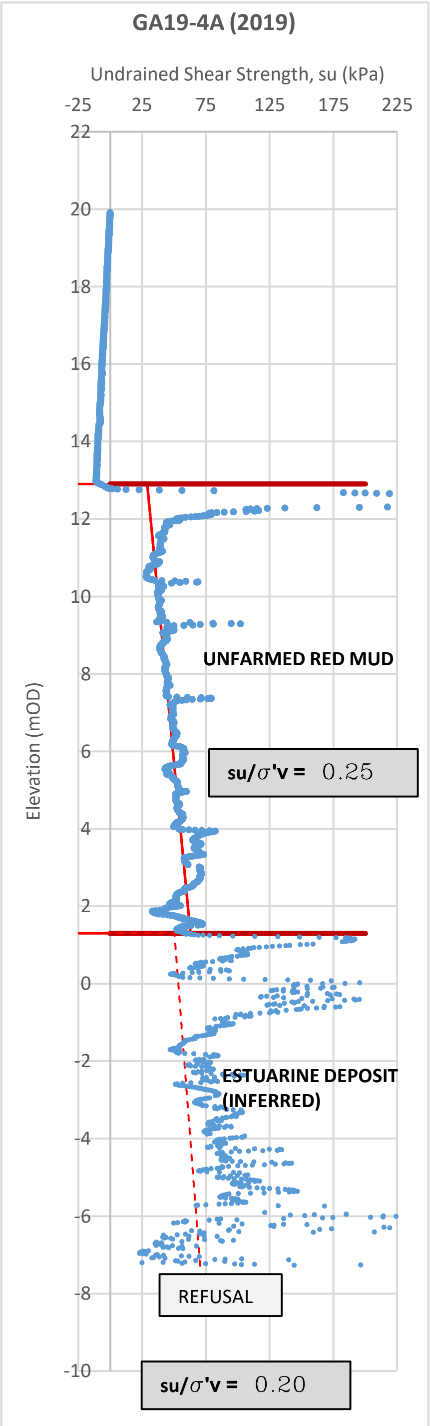
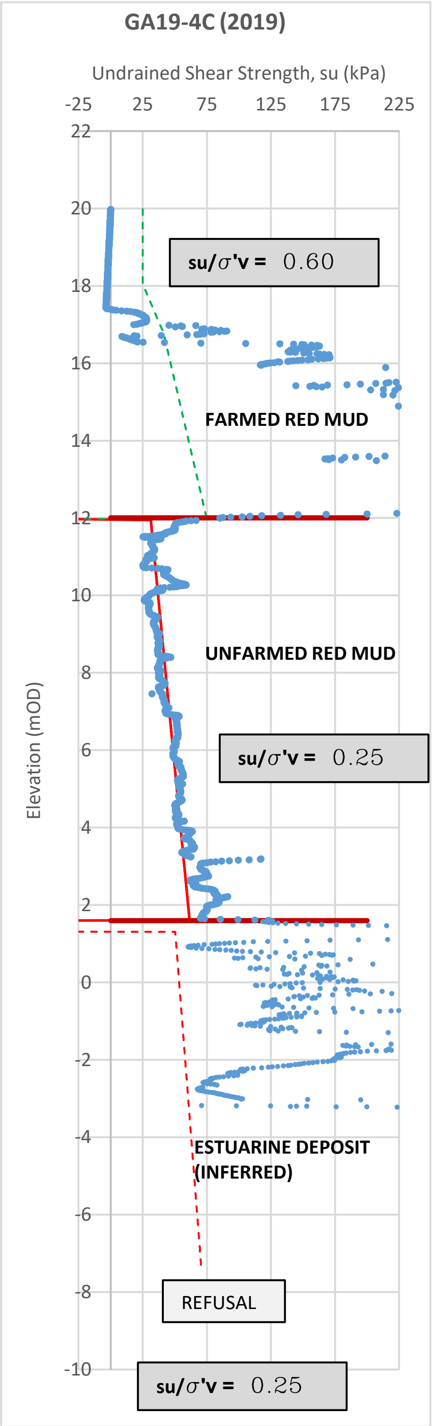
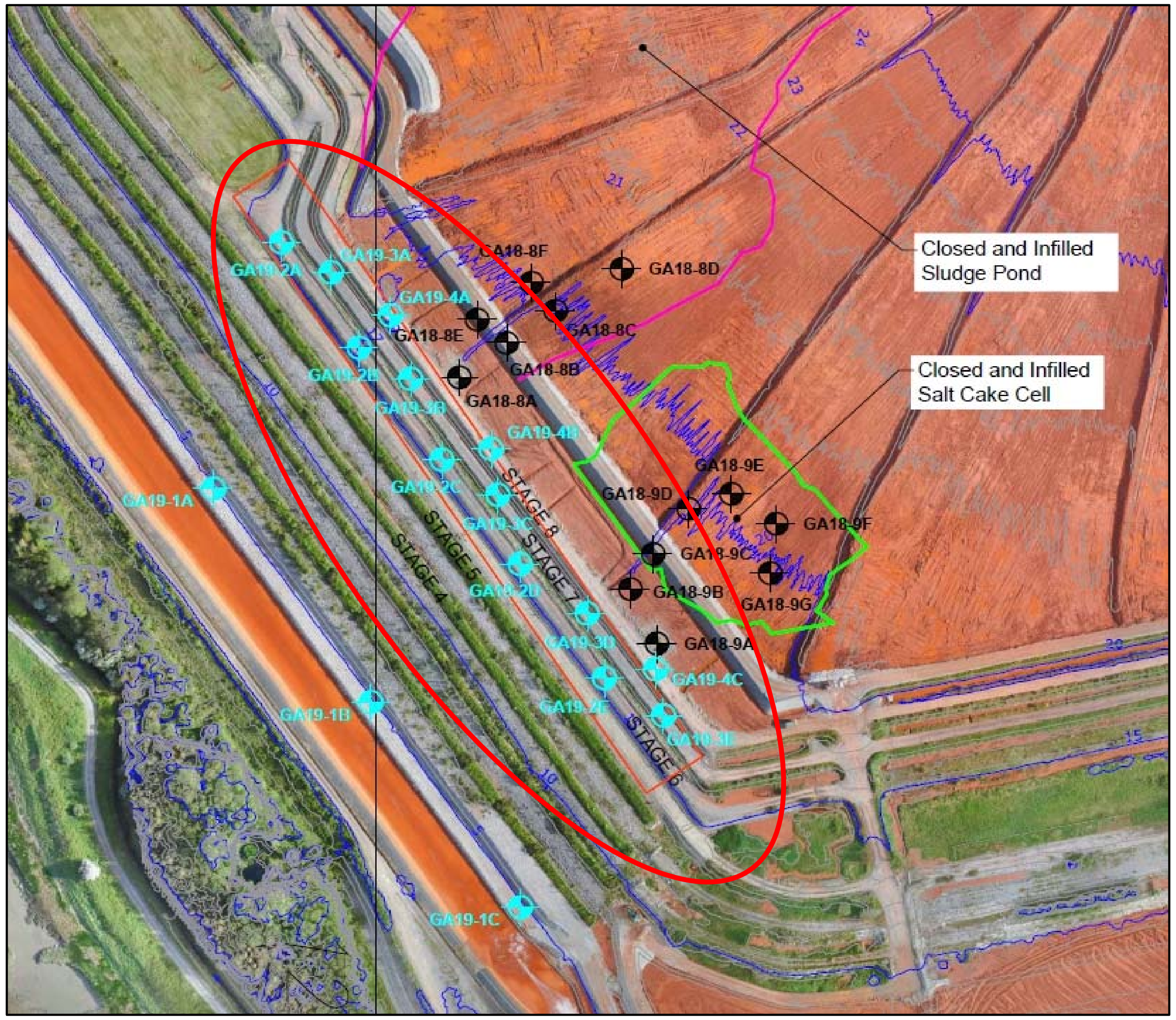




- su (Nkt=15 for Estuarine and = 14 for Bauxite Residue)
- Farmed Mud, su/σ'v =0.6 (min. 25kPa)
- Unfarmed Mud, su/σ'v =0.25 (min. 15 kPa)
- Estuarine, su/σ'v =0.20 to 0.25 (min. kPakPa)



LEGEND

GA18-10D	2018 CPT PROFILE LOCATIONS
GA19-1A	2019 CPT PROFILE LOCATIONS

NOTES:
1. Elevations Shown are approximate

CLIENT
AUGHINISH ALUMINA LIMITED

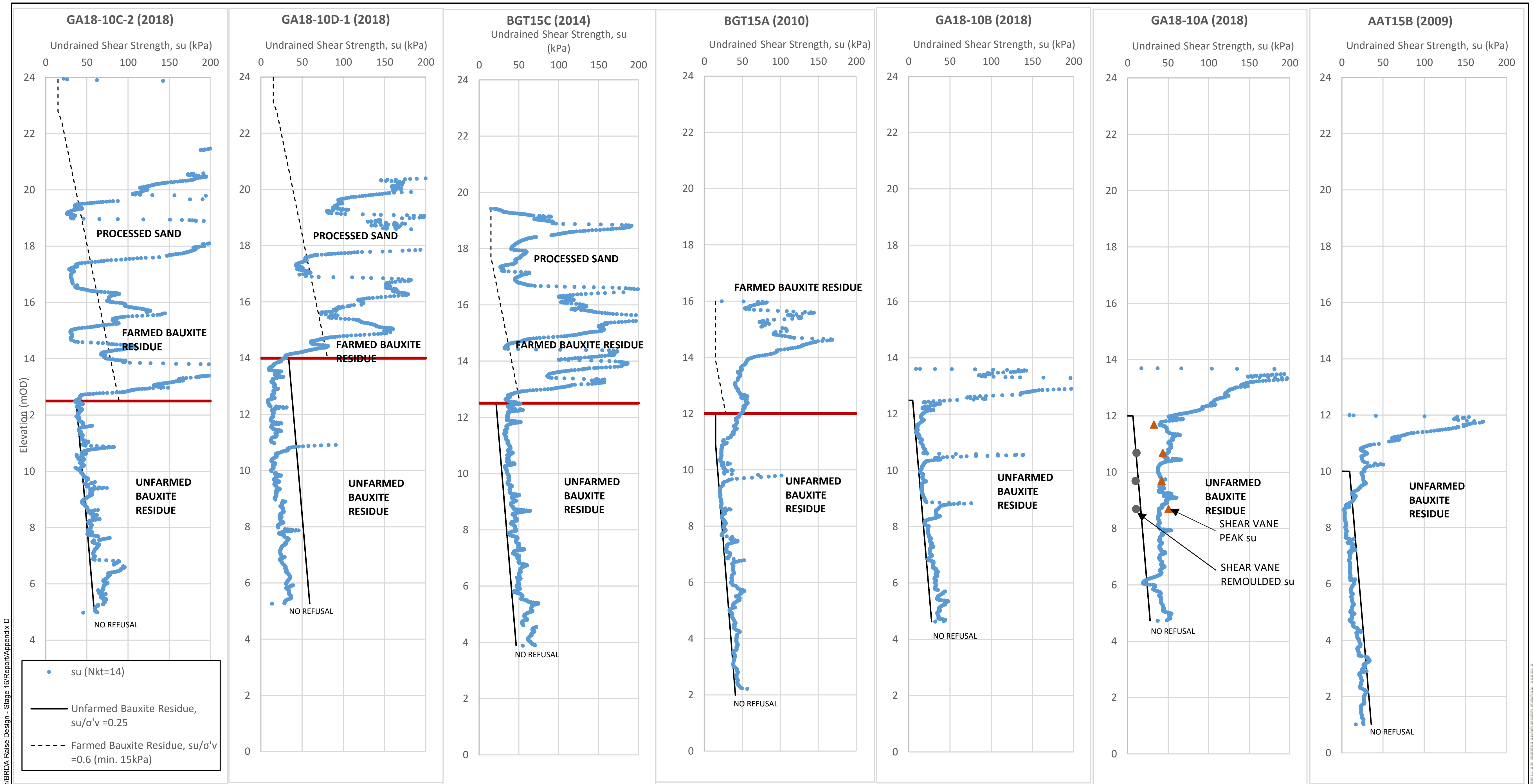
CONSULTANT
GOLDER

YYY/MM/DD	14/04/2020
PREPARED	BM
DESIGN	BM
REVIEW	BK
APPROVED	GJ

PROJECT
AAL BRDA
CPT 2019 INVESTIGATION

TITLE
SECTION F-F, STAGE 8
UNDRAINED SHEAR STRENGTH FROM CPT DATA

PROJECT No. 20143076 Rev. A FIG. D-3-10

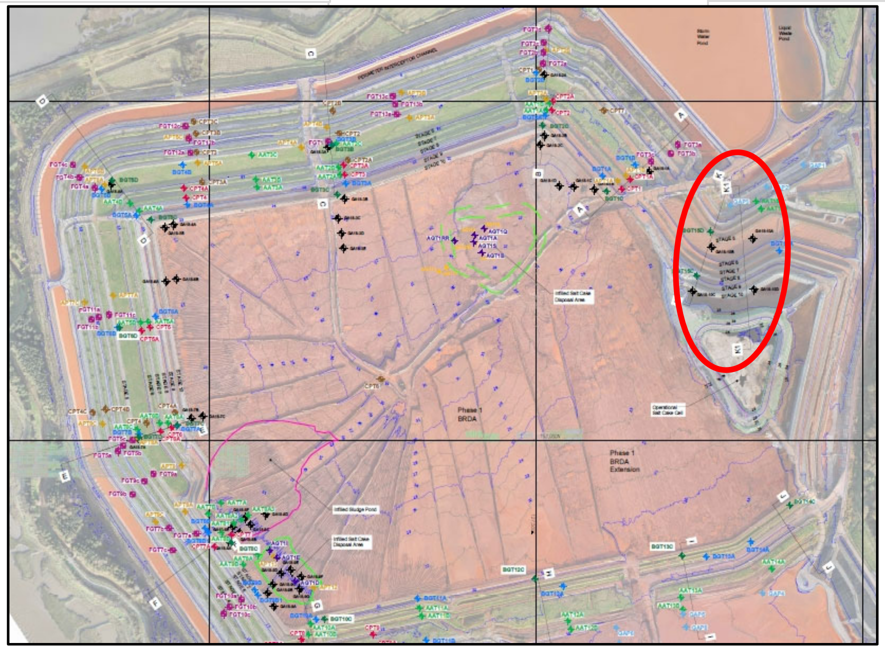


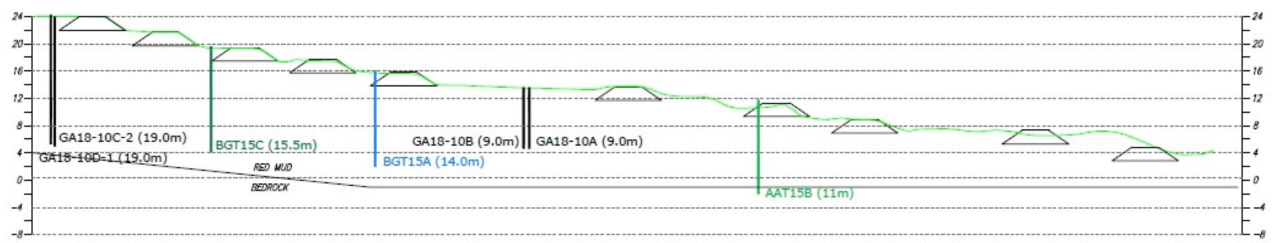
CPT LEGEND:

GA18	Cone penetration test (2018)
AGT	Cone penetration test (Salt Cake 2014)
AGT	Cone penetration test (Red Mud 2014)
BGT	Cone penetration test (2014)
BGT	Cone penetration test (2010)
AAT	Cone penetration test (2009)
CPT	Cone penetration test (2007)
APT	Cone penetration test (2006)
GAP	Cone penetration test (2005)
FGT	Cone penetration test (2004)
CPTS	Cone penetration test (2003)

NOTES:

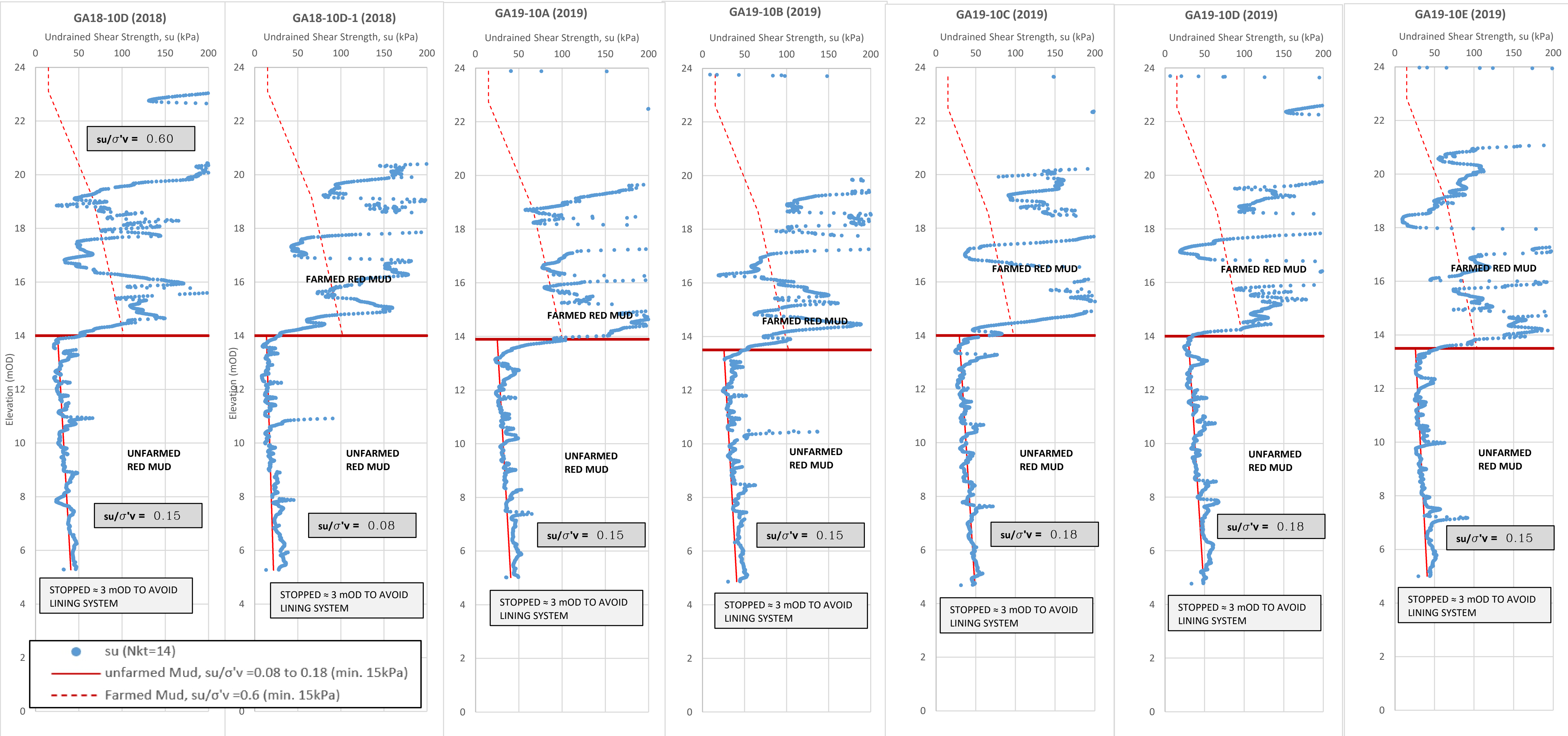
1. Elevations Shown are approximate

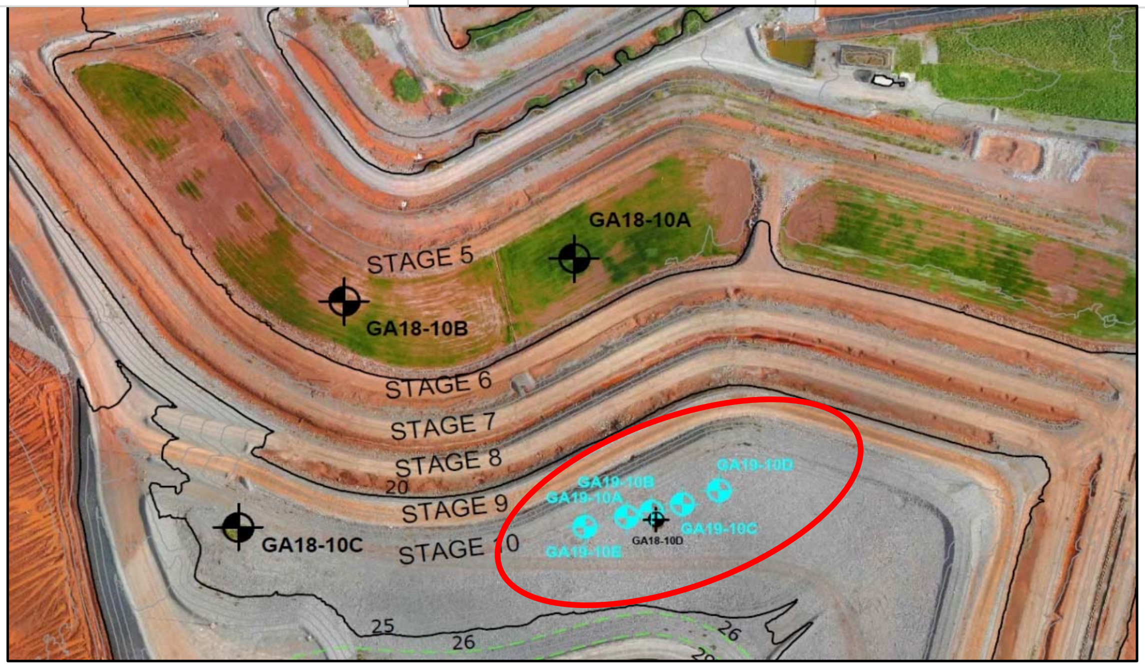


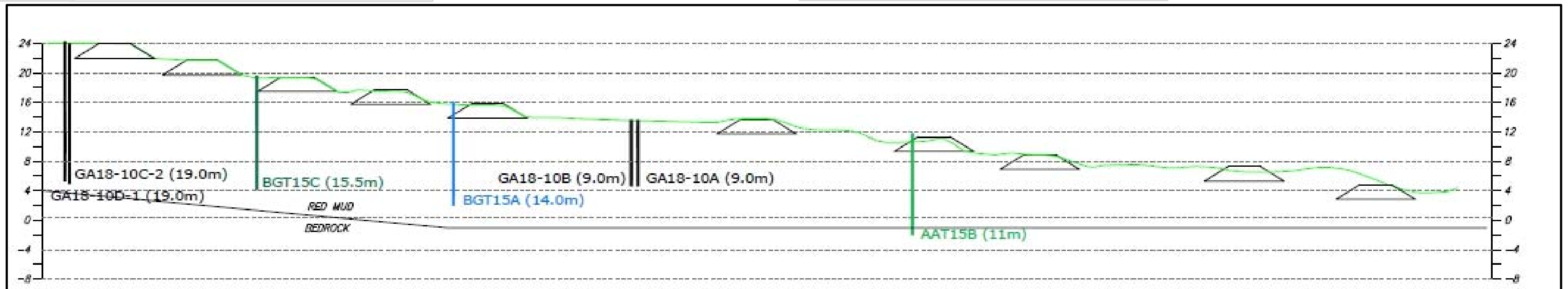


SECTION K-K

CLIENT	AUGHINISH ALUMINA LIMITED
PROJECT	BRDA Raise Development Appendix D-3: Stability Assessment
CONSULTANT	GOLDER MEMBER OF WSP
YYYY/MM/DD	01/10/2021
PREPARED	BM
DESIGN	GLJ
REVIEW	BK
APPROVED	GLJ
TITLE	SECTION K-K UNDRAINED SHEAR STRENGTH FROM CPT DATA
PROJECT No.	20143076
Rev.	A
FIG	D-3-11







SECTION K-K

LEGEND

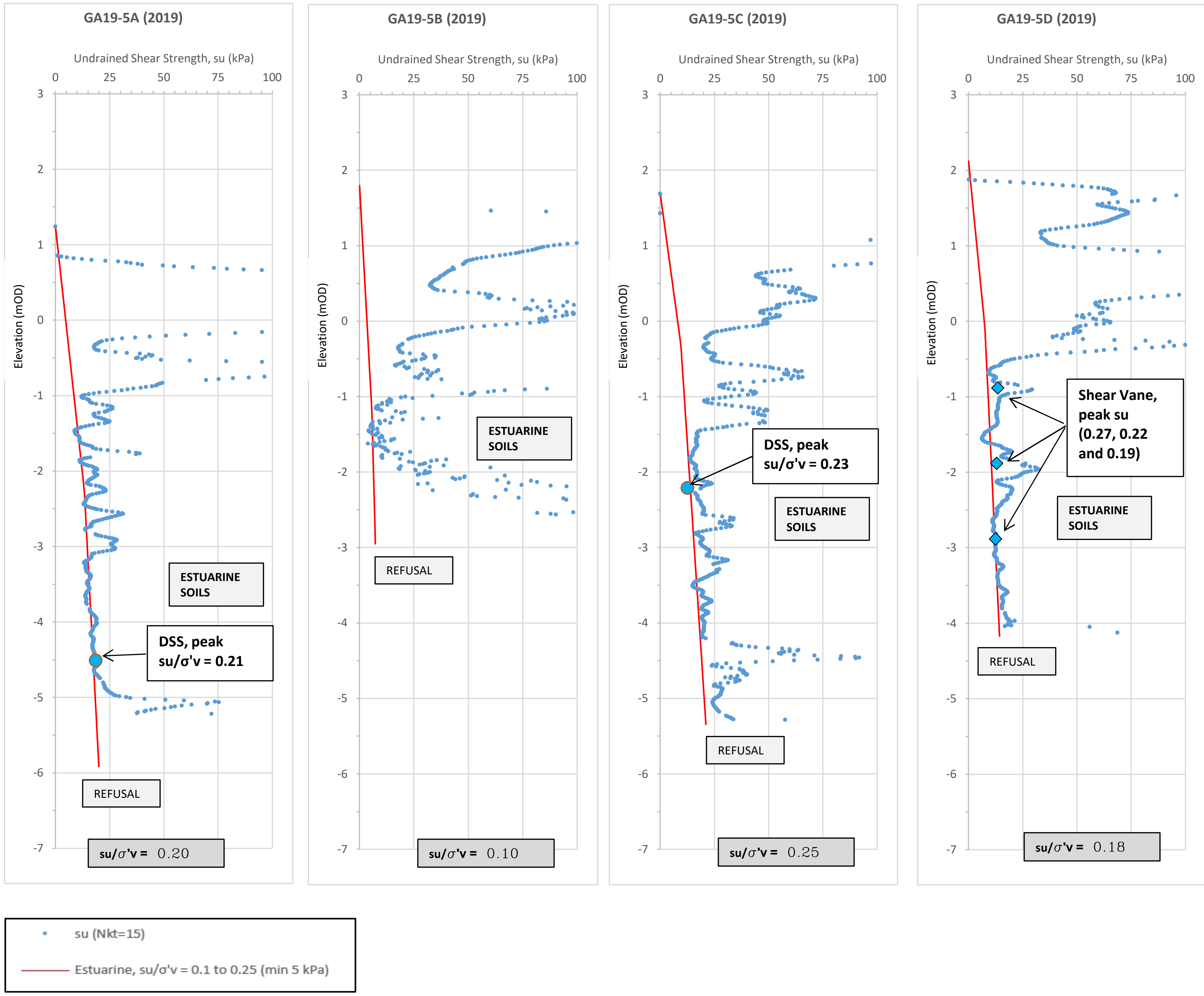
GA18-10D 2018 CPT PROFILE LOCATIONS

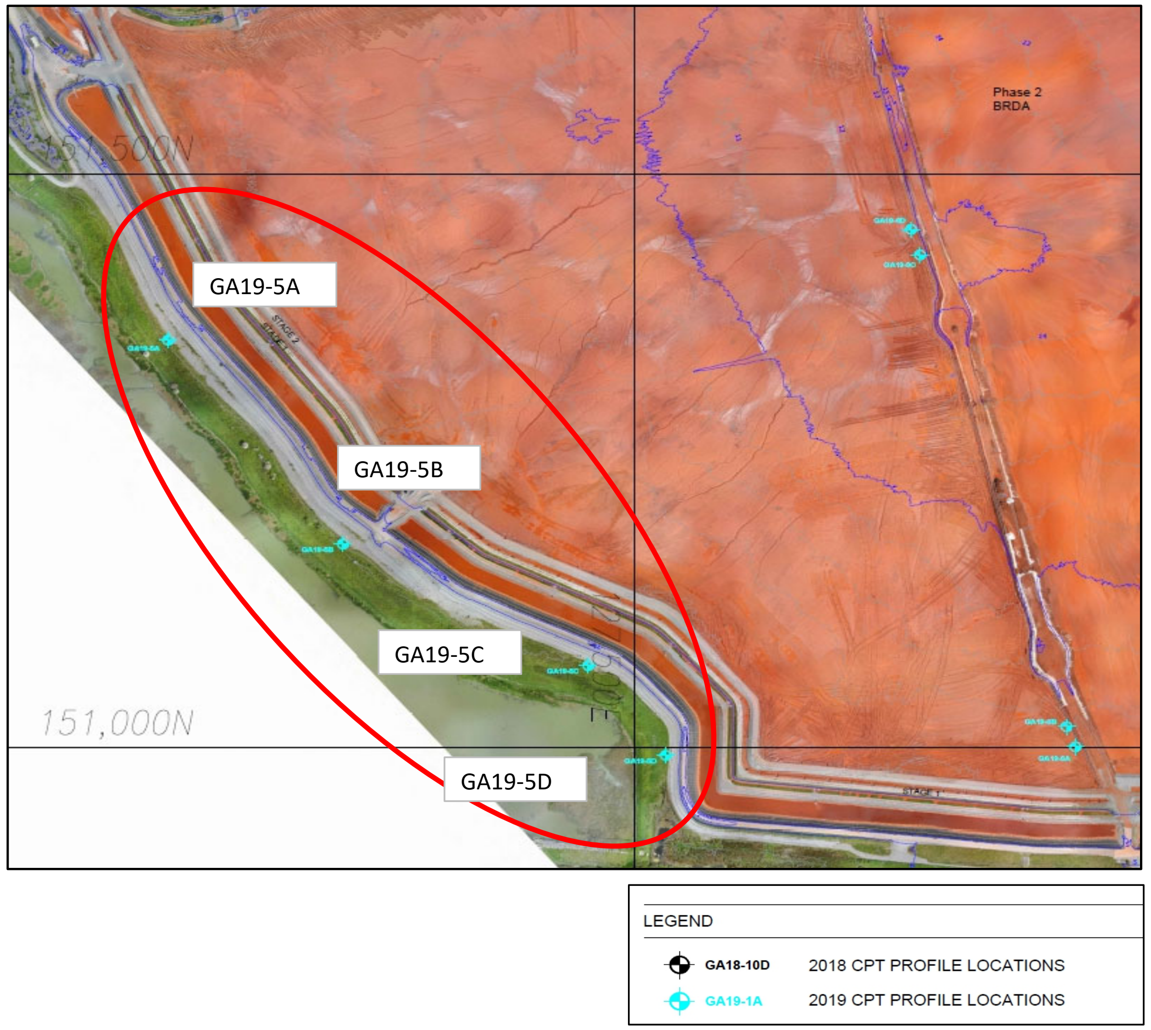
GA19-1A 2019 CPT PROFILE LOCATIONS

NOTES:

1. Elevations Shown are approximate

CLIENT	AUGHINISH ALUMINA LIMITED
CONSULTANT	GOLDER
YYY/MM/DD	14/04/2020
PREPARED	BM
DESIGN	BK
REVIEW	BK
APPROVED	GJ
PROJECT	AAL BRDA CPT 2019 INVESTIGATION
TITLE	SECTION K-K UNDRAINED SHEAR STRENGTH FROM CPT DATA
PROJECT No.	20143076
Rev.	A
FIG	D-3-12

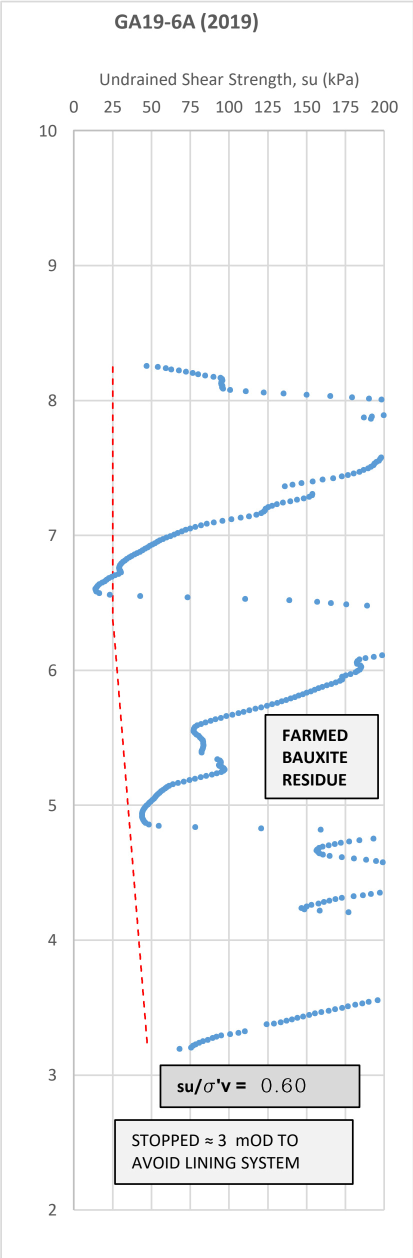


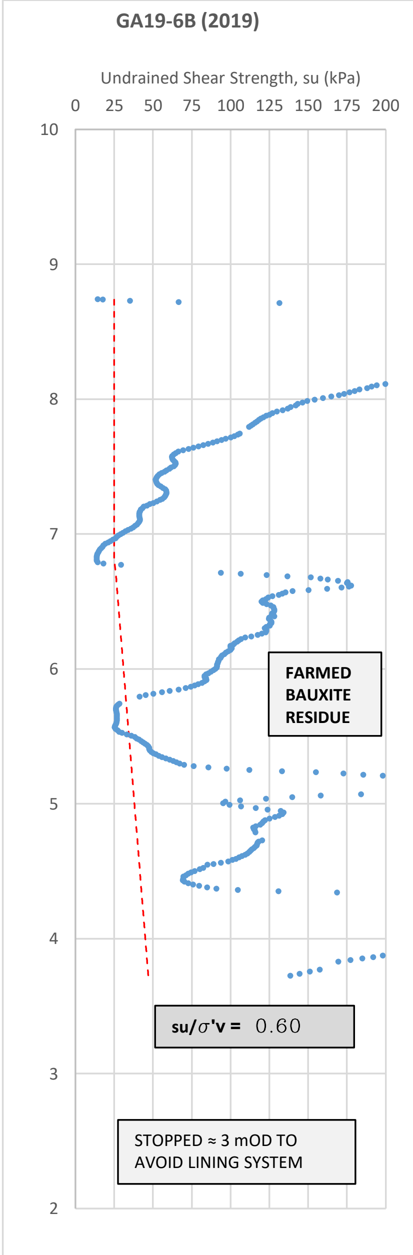


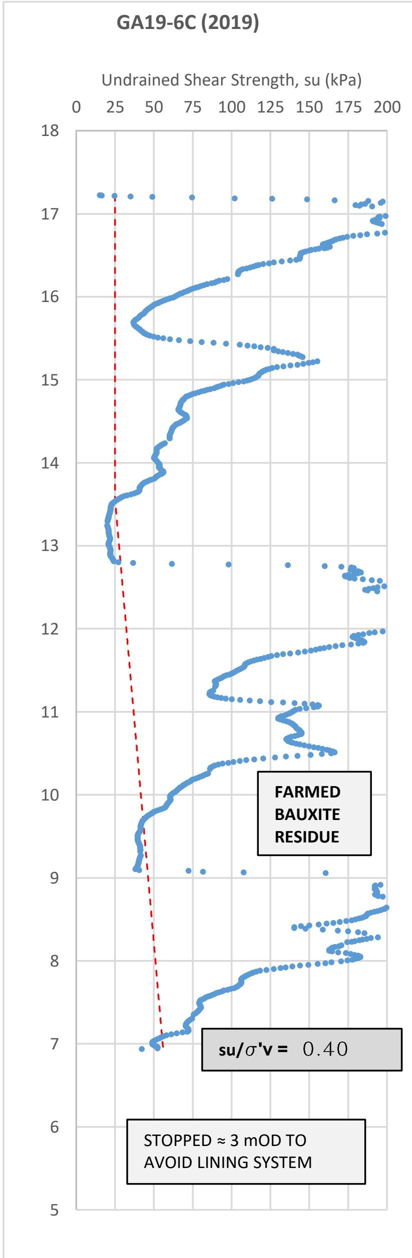
NOTES:

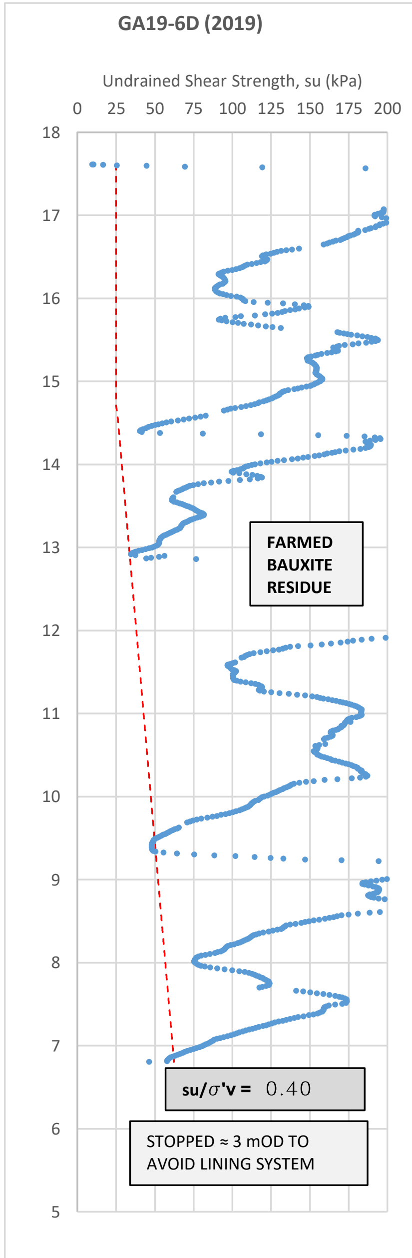
1. Elevations Shown are approximate

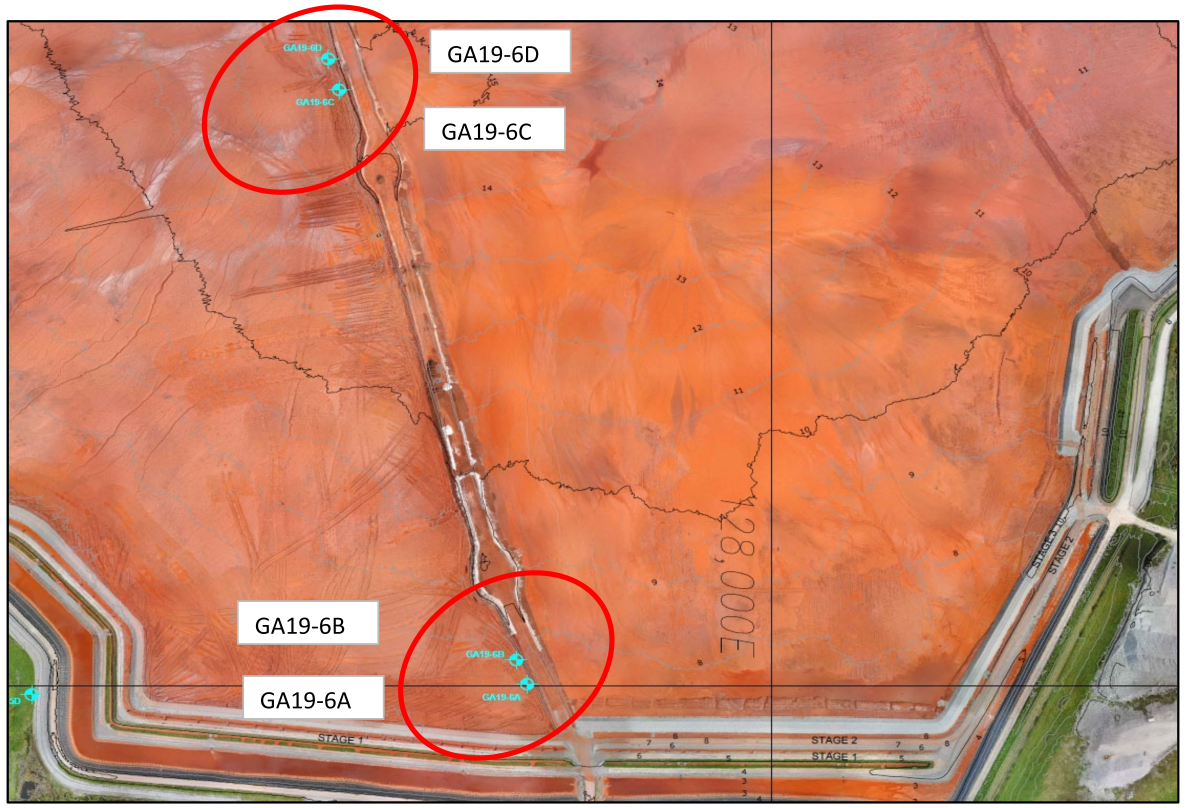
CLIENT	AUGHINISH ALUMINA LIMITED	PROJECT	BRDA Raise Development Appendix D-3: Stability Assessment
CONSULTANT	GOLDER MEMBER OF WSP	TITLE	DOWNSTREAM TOE OF PHASE 2 BRDA (ESTUARINE SOILS) UNDRAINED SHEAR STRENGTH FROM CPT DATA
YYYY/MM/DD	01/10/2021	PROJECT No.	20143076
PREPARED	BM	Rev.	A
DESIGN	BK	FIG	D-3-13
REVIEW	BK		
APPROVED	GJ		











LEGEND

- GA18-10D 2018 CPT PROFILE LOCATIONS
- GA19-1A 2019 CPT PROFILE LOCATIONS

- su (Nkt=14)
- unfarmed Mud, su/σ'v =0.25
- Farmed Mud, su/σ'v =0.40 to 0.60 (min. 25kPa)

NOTES:

1. Elevations Shown are approximate

CLIENT
AUGHINISH ALUMINA LIMITED

PROJECT
BRDA Raise Development
Appendix D-3: Stability Assessment

CONSULTANT
GOLDER MEMBER OF WSP

YYYY-MM-DD	01/10/2021
PREPARED	BM
DESIGN	BK
REVIEW	BK
APPROVED	GJ

TITLE
PHASE 2 BRDA: FARMED BAUXITE RESIDUE AREAS
UNDRAINED SHEAR STRENGTH FROM CPT DATA

PROJECT No.	Rev.	FIG
20143076	A	D-3-14

APPENDIX E

Blast Assessment

July 2017

AUGHINISH ALUMINA LIMITED

Borrow Pit: Phase 1 BRDA Blast Vibration Assessment

Submitted to:
Aughinish Alumina Limited
Aughinish Island
Askeaton
Co. Limerick

REPORT

Report **Number** 1667376.R01.A1

Distribution:

Aughinish Alumina Limited - 1 copy (pdf)
Golder Associates Ireland Limited - 1 copy (pdf)

Golder Associates

Record of Issue

Company	Client Contact	Version	Date Issued	Method of Delivery
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Aughinish Alumina Ltd	Tom Hartney	1667376.R01.A1	24 July 2017	Email

Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

Aughinish Alumina Limited (AAL) have engaged Golder Associates Ireland Limited (Golder) to assess the blasting from a the proposed Borrow Pit development impacting on the embankments and raises associated with Phase 1 Extension Bauxite Residue Disposal Area (BRDA). The BRDA is located to the west and south-west of the footprint of the proposed Borrow Pit.

Figure 1 below shows the extraction boundary (green line) for the proposed Borrow Pit along the Phase 1 Extension BRDA located to the west and south-west. Drawings 1 and 2 provided in Appendix A show the overall location plan and Section A-A.

The elevation of the basin of the Phase 1 BRDA is at approximately 2 metres above ordnance datum (mAOD). A rock fill perimeter dam wall was initially constructed, to approximately 4 mAOD crest elevation, and the facility perimeter has been raised using the upstream method of construction with 2 m high rock fill stage raises, to its current elevation of 20 mAOD (Stage 8) to 24 mAOD (Stage 10). A perimeter channel encircles the facility collecting surface water runoff from the red mud and reports these flows back to the Storm Water Pond (SWP).

The Phase 1 Extension BRDA is an eastern extension of the Phase 1 BRDA, continuing with a basin elevation of approximately 2 mAOD but ramping upwards to the east tracking the increase in elevation of the bedrock in this area. The road separating the proposed Borrow Pit and the Phase 1 Extension BRDA is known as the 'East Ridge Road' and is at approximately 16 mAOD for the common section. A lined perimeter channel is constructed to the west of the 'East Ridge Road' and a rock fill dam wall is constructed to the west of the perimeter channel at a crest elevation of approximately 12 mAOD, ramping downwards to the north. The basin of the Phase 1 BRDA is lined with a smooth HDPE liner, except along the edges where double sided textured geomembrane was used.

The foundation materials for the BRDA basin local to the proposed Borrow Pit footprint are a combination of existing bedrock and placed rock fill and the depth of red mud stored locally (within 100 m) is estimated to be in the 2 m to 4 m range.

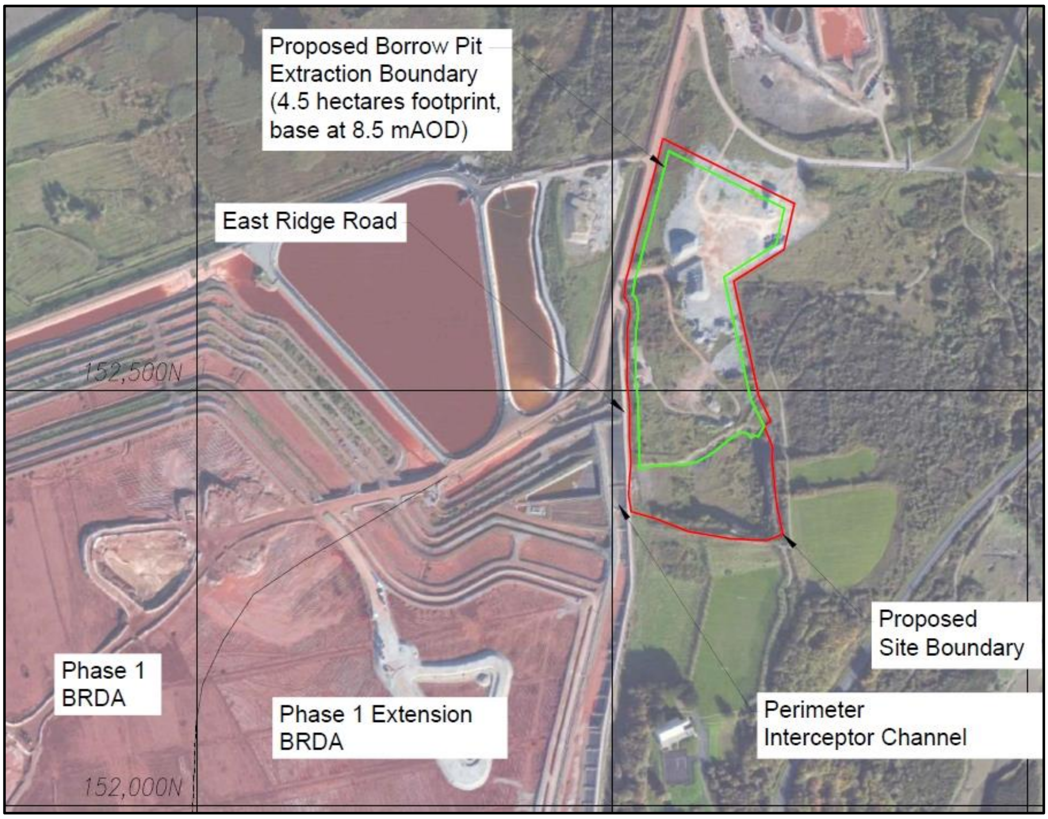

Figure 1: Location Map

2.0 SCOPE OF ASSESSMENT

AAL wish to determine the potential blasting effects during the proposed Borrow Pit operation and conduct a stability review on the Phase 1 Extension BRDA embankment. It is Golder's understanding that the planning application and EIA for the proposed Borrow Pit will be headed by Tom Phillips and Associates Limited (TPA) and that this analysis will be included in the appendix of the EIA.

This report presents the stability review of the Phase 1 Extension BRDA, and includes:

- An interpretation of the expected Peak Particle Velocity (PPV) caused by the blasting based on a review of previous blasting at Aughinish;
- Typical blast limits recommended for structures similar to the BRDA;
- Stability review of the BRDA based on the blast vibration and the potential generation of excess pore pressure; and
- Recommendations for conducting the blasting and monitoring during the borrow pit development.

3.0 BLAST VIBRATIONS

The intensity of ground vibrations, which is an elastic effect measured in units of Peak Particle Velocity (PPV), is defined as the speed of excitation of particles within the ground resulting from vibratory motion. The PPV is the most commonly used measure of the intensity of the ground vibration due to blasts. For the purposes of this report, PPV is measured in mm/s. While ground vibration is an elastic effect, one must also consider the plastic or non-elastic effect produced locally by each detonation when assessing the effects on structures such as earth embankments. The detonation of an explosive produces a very rapid and dramatic increase in volume due to the conversion of the explosive from a solid to a gaseous state. When this occurs within the confines of a borehole, it has the following effects:

- The bedrock in the area immediately adjacent to the explosive product is crushed;
- As the energy from the detonation radiates outward from the borehole, the bedrock between the borehole and quarried face becomes fragmented and is displaced while there is minimal fracturing of the bedrock behind the borehole; and
- Energy not used in the fracturing and displacement of the bedrock dissipates in the form of ground vibrations, sound and airblast. This energy attenuates rapidly from the blast site due to geometric spreading and natural damping.

The rate at which ground vibrations attenuate from a blast site is dependent on a number of variables. These include the characteristics of the blast (delay timing, type of explosive, etc.), topography of the site, as well as the characteristics of the bedrock and/or soil materials. The intensity of ground vibrations from blasting operations is primarily a function of the maximum explosive weight detonated per delay period, and the distance between the blast and the receptor location.

The industry equation for predicting the PPV from surface blasting is shown below:

$$\boldsymbol{PPV = k\left(\frac{D}{\sqrt{W}}\right)^{-b}}$$ **(United States Bureau of Mines, 1959)**

Where:

PPV = Peak Particle Velocity (mm/s)

D = Distance (m).

W = Explosive Charge Weight per Delay (kg)

k and b are site-specific factors – typically determined by on-site measurement

The Distance is the plan distance measured from the charge location to the receptor. The explosive charge weight per delay is the Maximum Instantaneous Charge (MIC).

For the data in a given study, the 95% confidence curve for that data is typically used to define the ground vibration attenuation model. The purpose of the model equation is not so much to predict what a given vibration level would be at a particular location for a given blast, but to indicate the probability that the peak vibration would fall below the level indicated by the equation for a given distance and maximum explosive weight. The equation is therefore a useful blast design tool in establishing maximum explosive charge weights per delay for various distances from a blast site for a given maximum ground vibration level (i.e. limit).

4.0 BLAST VIBRATION LIMITS

Ground vibration guidelines are typically established for blasting sites to prevent damage to adjacent facilities or infrastructure. Exceeding these levels does not in itself imply that damage has occurred but only increases the potential that damage might occur.

4.1 Blasting Near Dams and Embankments

The designed seismic ground vibration limit for tailings embankments is typically based on a peak ground acceleration (PGA). The PGA assessment procedure is typically used to assess the potential instability of embankments related to earthquake-induced seismicity.

However, there are fundamental differences between blast-induced ground vibrations from construction or mining operations and ground vibrations caused by earthquakes. Earthquake-induced vibrations are typically very low frequency, very large displacement and long duration. Ground vibrations initiated by open pit blasts typically contain less energy, have a higher spectral frequency content, and have significantly shorter time duration (less than a second versus more than half a minute to several minutes) than earthquake-induced ground vibrations. The dominant frequency of blast-induced ground vibrations depend on the site geology, distance to the blast and delay sequencing of the blast. The dominant frequency from surface mine blasts typically range from 30 Hz to 100 Hz. Thus, although the PGA of the blast-induced ground vibrations may exceed the designed seismic limits, the PPV and displacements may be a small fraction of those anticipated for an earthquake induced event. Large earthquakes generate large strains. The long wavelengths would typically shake dams as a unit, simultaneously throughout. Additionally, the damage potential increases with the duration of the event. With blast vibrations, the wavelengths are significantly shorter and the various parts of the embankment are unlikely to be in phase.

Appropriate limits for blast-induced vibrations at earth dams and embankments have been discussed in numerous publications. Blasting near earth-fill and tailings dams has the potential to increase residual pore pressure, reduce the dam's stability, induce settlements, or cause other damages. Charlie et al. (1987) suggested the following criteria for blasting near dams (**Table 1**), based on liquefaction potential and susceptibility to pore pressure increases.

Table 1: General Guidelines to Vibration Damage Thresholds for Blasting Near Dams

Dam Construction	PPV Limit (mm/s)
Dams constructed of or having foundation materials consisting of loose sand or silts that are sensitive to vibration.	25
Dams having medium dense sand or silts within the dam or foundation materials	50
Dams having materials insensitive to vibrations in the dam or foundation materials	100

Notes: *From Charlie et al. (1987)

The information presented in **Table 1** can be used as general guidelines for assessing the potential for blast vibration damage to structures. Considering the material types present within the dam walls and the BRDA foundations a conservative PPV limit of 25 mm/s would be recommended for the embankment.

Charlie et. al (2001) show that significant residual pore pressure increase, at peak particle velocity exceeding 15 mm/s , may occur at shallow depths, and recommends that peak particle velocity and pore pressure should be monitored and evaluated at several locations in the dam, foundation soils, and abutments.

4.2 Blasting near GNI Transmission Line

A 300 mm diameter steel transmission gas pipeline is present along the north extent of the proposed Borrow Pit footprint. The PPV is limited by Gas Networks Ireland (GNI) to 75 mm/s. Golder proposes to set this threshold at 50 mm/s. A minimum set-back distance of 50 m was provisionally agreed with GNI.

An assessment for PPV for blast vibration in the vicinity of the GNI pipeline was undertaken by Golder using more conservative **k and b values of 3,352 and 1.95** respectively. The values selected are considered representative of highly fractured limestone.

5.0 REVIEW OF PREVIOUS AAL BLAST AND VIBRATION DATA

Many studies have been conducted around the world to develop values for k and b for various the rock types. However, these values are very site-specific and are dependent on a number of factors including rock mass formation, jointing, direction of planes etc. Golder conducted a review of the measured vibration data from the blasting conducted during the construction of the Phase 2 BRDA (2008 to 2011) to back-calculate appropriate parameters for k and b. Unfortunately, the data scatter was sufficiently large that a reliable estimate was not able to be used.

In order to implement a reliable model it was decided to use the results of a Golder vibration attenuation study carried out at the former Galmoy Mine blasting operations. The 95% confidence curve for the data was established with **k and b values of 300 and 1.14** respectively. Based on the results of the ground vibration monitoring to date, a maximum explosive charge weight for a given set-back distance from a blast to a given receptor while maintaining a PPV within the 25 mm/s and 50 mm/s limits is shown in Figure 2.

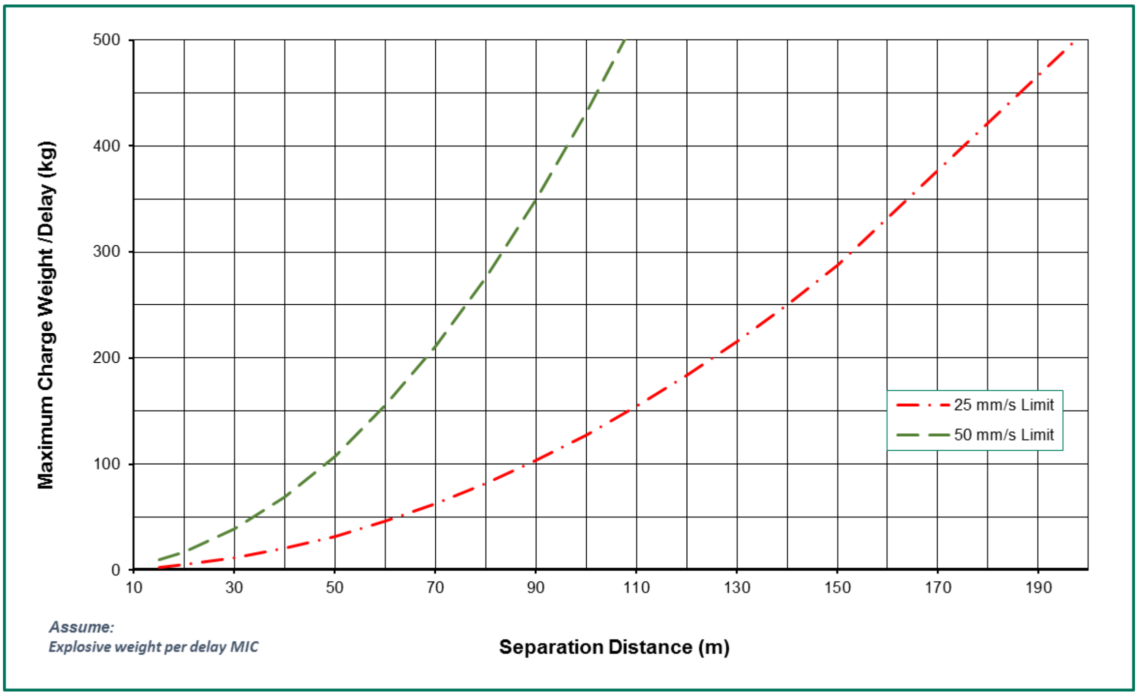

Figure 2: Maximum Explosive Charge Weight for a Range of Set-back Distances for the Assumed Attenuation Model

In order to assess the applicability of the model described above, the Phase 2 BRDA blast data was reviewed to select blast reports that had near sensor blast locations i.e. < 150 m. The data from five blasts conducted in September 2010 was compared to those estimated with the model.

Table 2 lists the measurements on sensors located at the Asbestos Water Pipe and Water Works during Phase 2 BRDA construction and also shows the predicted PPV.

Table 2: AAL Phase 2 BRDA Blast Data (Aug/Sept 2010)

Blast ID (date)	Sensor Location	D (m)	W (kg)	k	b	Predicted PPV (mm/s)	Measured PPV (mm/s)
17 (27/08/10)	Water Works	150	44	300	1.14	8.57	4.06
20 (03/09/10)	Water Works	150	33	300	1.14	7.28	1.90
25 (15/09/10)	Water Works	150	15	300	1.14	4.64	2.03
26 (15/09/10)	Water Works	150	13	300	1.14	4.28	2.54
26 (15/09/10)	Asbestos Pipe	100	13	300	1.14	6.79	2.41

A comparison between the predicted and measured PPV values would suggest that the parameter values assigned to k and b are appropriate, although somewhat conservative.

6.0 ASSESSMENT OF GROUND VIBRATIONS FOR BORROW PIT BLASTING

Previous blasting reports from the Phase 2 BRDA development suggest that the value for W is expected to be in the 30 kg to 40 kg range. A values of 35 kg shall be used for the assessment. As discussed above, the following parameters shall be used for the assessment:

- k = 300; and
- b = 1.14

Figure 3 shows the estimated PPV at a range of set-back distances and the assumed vibration limit for the BRDA and assumed threshold for the GNI pipeline.

Table 3: Predicted PPV for AAL Borrow Pit Blasting at Gas Pipeline

Sensor Location	D (m)	W (kg) assumed	k	B	Predicted PPV (mm/s)
Ground above Gas Pipeline	20	35	300	1.14	75
Ground above Gas Pipeline	30	35	300	1.14	47
Ground above Gas Pipeline	40	35	300	1.14	34
Ground above Gas Pipeline	50	35	300	1.14	26

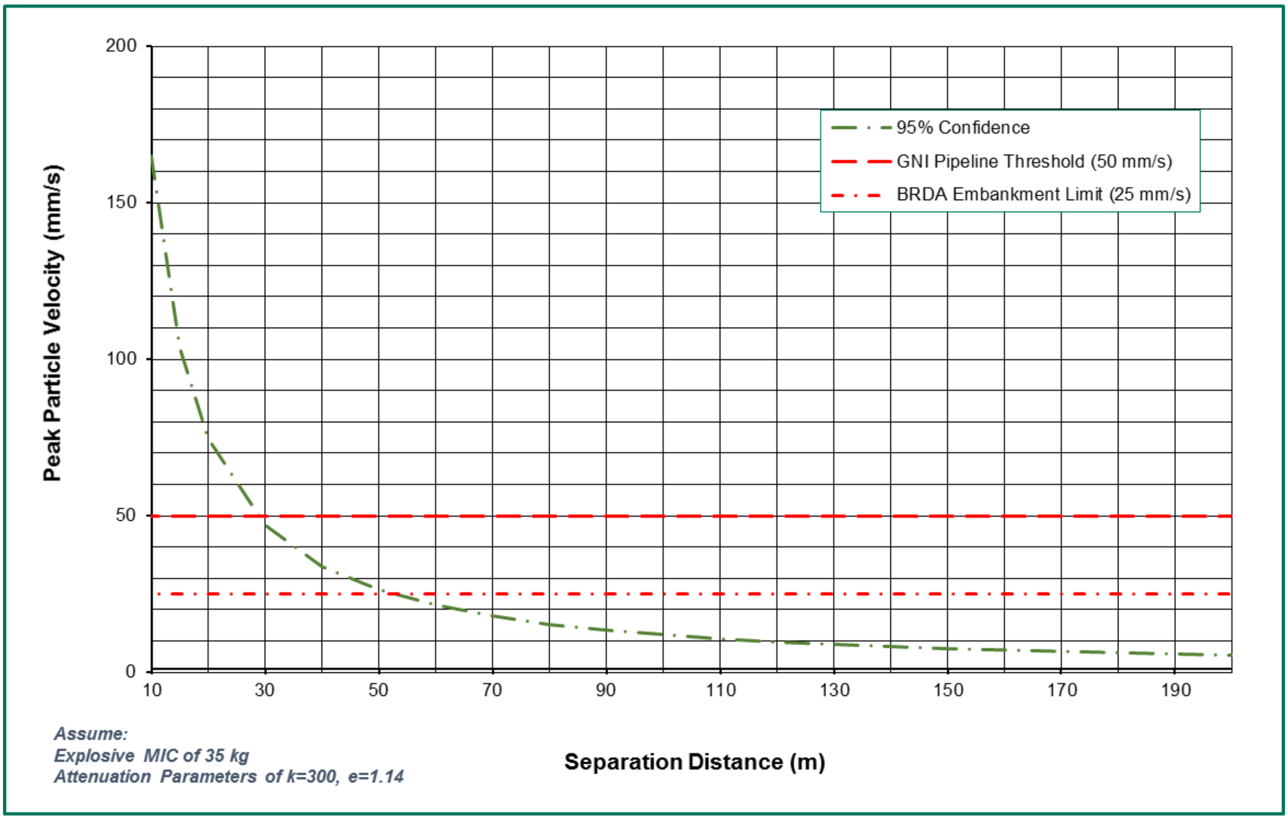

Figure 3: Estimated PPV for a range of set-back distances

As shown in Figure 3, the estimated set-back distance to remain compliant with the embankment and pipeline limits, assuming an MIC of 35 kg, are as follows:

- BRDA Embankment (25 mm/s) – 53 m; and
- GNI Pipeline (50 mm/s) – 29 m.

Based on the assigned parameter values for k and b, set-back distances of 53 m and 29 m are recommended from the nearest point of a given blast of the Borrow Pit to the BRDA embankment and gas transmission pipeline respectively. Any blasting within these set-backs may necessitate a reduction in the maximum explosive weight detonated per delay period so that the peak ground vibration levels could be maintained below assumed vibration limits.

Table 4 shows predicted PPV levels at other receptors of concern for the assumed parameters discussed above.

Table 4: Predicted PPV at Other Receptors of Concern for AAL Borrow Pit Blasting

Sensor Location	D (m)	W (kg) assumed	k	B	Predicted PPV (mm/s)
Clarifier Pond	175	35	300	1.14	6.3
Clarifier Tank	225	35	300	1.14	4.7
Nearest House	800	35	300	1.14	1.1

A plan showing contours of the estimated reduction in PPV with distance from the proposed Borrow Pit is shown in Drawing A3 provided in Appendix A. The PPV values are based on the calculation and input parameters assumed above.

7.0 STABILITY ASSESSMENT DUE TO BLAST VIBRATION

The stability of the Phase 1 Extension BRDA due to nearby blasting was assessed with two approaches:

- **Pseudo-static Stability Assessment** - analysing the stability of slopes subject to blast vibration; and
- **Post-blast Analysis** - simulating the excess pore pressure in saturated soil which can potentially be generated by nearby blasting operations.

Both approaches make use of two-dimensional limit equilibrium analysis which provides a Factor of Safety (FoS) against slope instability. The FoS is defined as the ratio of resisting forces to driving forces. Slope instability occurs when the driving forces exceed the resisting forces. Driving forces typically include gravity induced loading, seepage forces and blast induced vibration. The primary resisting force is the shear strength of the material.

An analysis of the stability of the Phase 1 BRDA was carried out using the limit equilibrium modelling software SLOPE-W Version 8.11.1. The analytical method used was Morgenstern and Price method of slices, which satisfies both force and moment equilibrium.

The analysis was carried out for undrained (total stress) condition within the red mud as this will be the critical case. An undrained condition for a material in a contractive state (non-farmed red mud), generates excess pore pressure and results in a lower effective shear strength less than in the drained condition. The undrained condition when the material is in a relatively dense/stiff condition (farmed red mud), dilates during shearing, generates negative pore pressure and may result in an effective shear strength greater than in the drained condition.

The drained (effective stress) condition, which represents the long term condition has not been included in the current analyses. This condition represents loading and shearing of the red mud at a slow enough rate to limit the build-up of excess pore pressure, and typically produces a higher FoS.

The phreatic surface used within the model is based on monitoring data from standpipe piezometers nearby the location analysed.

7.1 Analysis Criteria

International guidelines used to develop the required factor of Safety (FoS) for the Aughinish BRDA are as follows:

- Canadian Dam Association (CDA) Application of Dam Safety Guidelines for Mining Dams (CDA 2014); and
- Australian National Conference on Large Dams (ANCOLD) Guidelines on Tailings Dams (ANCOLD 2012.

Table 5 provides a summary of the FoS for slope stability analysis for each guideline.

The Eurocode 7 design rules have not been applied as the code states that it applies to the embankments of small dams. The BRDA is considered a large dam and as such the above mentioned guidelines, along with ICOLD bulletins, are considered more applicable.

Table 5: Factor of safety Criteria based on International Guidelines

Loading Condition	Recommended Factor of Safety	
	CDA (2014)	ANCOLD (2012)
Short Term Undrained	Greater than 1.3 During, at, or end of Construction, depending on Risk Assessment [a]	1.5 if loss of containment, Consolidated Undrained Strength
Long Term Drained	1.5 SteadyState Phreatic level	1.5 Effective Strength
Pseudo-static	1.0	Not analysed
Post-earthquake	1.2	1.0 to 1.2 Post Seismic material shear strength

a) The CDA guidelines typically require a FoS of 1.3 for the short term undrained loading condition, but does state that this may not apply to tailings dams that are constructed over time, similar to the BRDA.

A minimum FoS of 1.5 is generally considered required for all long term static analysis. ICOLD Bulletin 139 (ICOLD 2011) discusses the potential for static liquefaction of loose saturated tailings due to a "trigger" mechanism, and that a FoS of 1.5 is generally accepted as adequate when using the maximum deviator shear stress.

The European Union Reference Document on Best Available Techniques for Management of Tailings in Mining Activities (EU 2009) recommends a FoS between 1.3 and 1.5 for the short and long term. This document is in the process of being updated and the Draft version dated June 2016 recommends similar FoS for the short and long term at 1.5.

The pseudo-static loading condition requires a FoS of 1.0 according CDA guidelines. The recommendation for the blast vibration assessment (Wong and Pang 1992) similarly recommend a FoS of 1.0.

For the post-earthquake condition, where the generation of excess pore pressure may have decreased the material shear strength, a FoS of 1.2 is recommended, and this was similarly applied to the pot-blasting analysis based on the residual excess pore pressure.

7.2 Model Configuration

Section K-K cut along the north-east slope of the Phase 1 BRDA, which is located closest to the proposed quarry, has been used in the stability analysis model. Figure 4 shows the location of Section K-K, and Figure 5 shows Section K-K as used in the model. Section K-K used is considered to be the critical section for a full slope length closest to the proposed Borrow Pit. The overall slope height for the BRDA next to the proposed Borrow Pit footprint is much less than that analysed for Section K-K.

The embankment crest is at elevation 22 mAOD, with the red mud surface sloping up at a grade of approximately 2% to 4%. The current overall slope of the perimeter wall of the Phase 1 BRDA is 6.3(H):1(V), consisting of a lower slope of 6(H):1(V) and an upper slope of 6(H):1(V) broken by an intermediate (upper level) bench at Stage 5, 14 mAOD.

The facility is raised by constructing perimeter walls on the red mud by the upstream method; this method involves the construction of a series of retaining bunds upstream of the toe of the BRDA facility and so forming a supporting face to the overall structure. The stack wall is raised systematically as the facility fills with red mud in approximately 2 m high stage raises

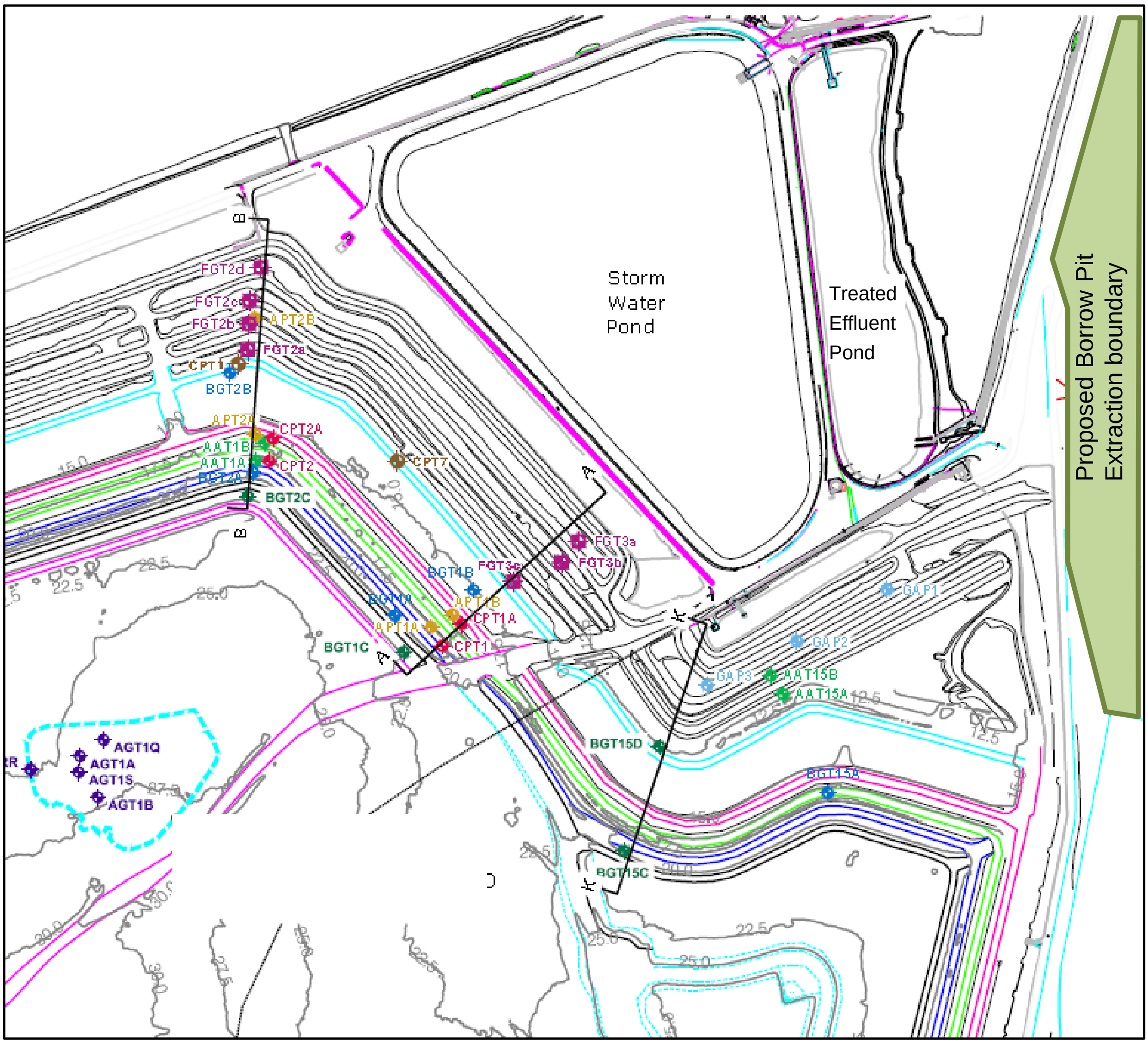

Figure 4: Location of Stability Analysis Section relative to the Proposed Borrow Pit

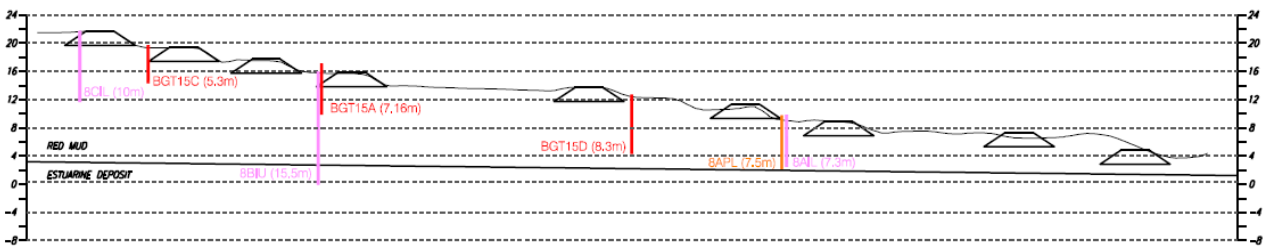

Figure 5: Section K-K

7.3 Material Properties

Golder has adopted representative properties for the materials based on the available information and results of extensive site investigation and laboratory testing programmes conducted previously. The selected properties for the analyses are presented in Table 6. The material parameters chosen for the analysis are based on the average material parameters established previously through review of all the site investigations conducted over the life of the facility. Where required, these have been updated to reflect the specific conditions within the area closest to the proposed borrow pit area. Six CPT soundings were conducted in the area between 2005 and 2014. The sections that follow provide further description on how the material strength parameters were developed.

Table 6: Material Parameters used in the Stability Analysis

Material	Unit Weight (kN/m^3)	Shear Strength	Comment
Farmed Red Mud	22	$s_u/\sigma'_v = 0.6$ minimum of 60 kPa	Based on CPT analysis
Red Mud	22	$s_u/\sigma'_v = 0.23$ minimum of 25 kPa	Sensitivity analysis where $s_u/\sigma'_v = 0.15$ along the toe of the east slope
Estuarine Deposits	19	$\Phi = 30^\circ$, c=0 kPa	Sensitivity analysis where $s_u/\sigma'_v = 0.23$
Stage Raise Rock fill	22	$\Phi = 45^\circ$, c=0 kPa	Estimate for average rock fill strength

Notes:

s_u = undrained shear strength; σ'_v = vertical effective stress; Φ = Friction angle; c = cohesion

The red mud shear strength is represented as an undrained shear strength. The peak undrained strength is the shear strength of a soil when sheared at a rate such that shear-induced pore pressures are unable to dissipate. The undrained strengths are typically referenced as a ratio to the pre-shear vertical effective stress of the element of soil under consideration.

The Estuarine deposits and rock fill is represented as a drained (fictional) strength.

7.3.1 Red Mud Undrained Shear Strength

The red mud undrained shear strength was determined based laboratory triaxial test results, and in-situ testing consisting of Cone Penetration Testing (CPT) and Shear Vane Testing.

The CPT data have been transformed to **undrained shear strength (s_u)** using an **undrained strength factor N_{kt}**. The relationship is defined as **s_u = (Net cone resistance)/N_{kt}**, which is the standard approach. An N_{kt} value of approximately 15, which is the generally accepted value for all tailings (Golder experience), provides a reasonable estimate of the undrained shear strength when compared to the Geonor In-Situ Shear Vane Testing and the laboratory Triaxial Testing from samples taken adjacent to the CPT soundings via the MOSTAP sampling tubes.

The undrained shear strength, of which the lowest bound value for the undrained shear strength ratio (s_u/σ'_v) for the non-farmed red mud was estimated to be 0.23. This equates to a total stress frictional angle (φ) of 13 degrees which is approximately 60% below the lower bound red mud effective frictional angle and represents excess pore pressure generation in the red mud due to its contractive state. A minimum undrained shear strength of 25 kPa was selected based on the desiccated red mud increasing the near surface strength (at low effective stress), and represents a lower bound value observed.

CPT soundings conducted along the eastern slope in 2009 (AAT15A and AAT15B) indicate potentially lower undrained shear strengths than observed in the remainder of the facility and other nearby CPT soundings conducted prior to and after 2004. It is likely that the red mud has further consolidated at the location of these soundings with the undrained shear strength similar to that observed for the rest of the BRDA.

A sensitivity analysis was carried out to determine the influence on the FoS if the red mud along the toe had a reduced strength.

The interpreted undrained shear strength values from CPT soundings conducted along the eastern slope are included in Figure B1 and B2.

7.3.2 Farmed Red Mud Undrained Shear Strength

Since March 2009, the red mud has been intensively mud-farmed. This process involves discharging the red mud in thin layers (< 300 mm), in purpose built internal cells within the BRDA, and then using a specially adapted machine, the amphiroll, which compresses the surface of the red mud, reducing moisture and enhances the drying process by creating furrows, thus increasing the surface area of the red mud exposed for drying. Prior to placement of subsequent layers of red mud, the furrows are levelled and the surface is track-compacted using a dozer.

The farmed red mud is expected to have a higher shear strength ratio (s_u/σ'_v) than the non-farmed red mud due to the lower initial void ratio (higher density).

The farmed red mud strength was similarly based on the review of the CPT data and calculation of the undrained shear strength using an average N_{kt}. A minimum undrained shear strength of 60 kPa was selected and is based on approximately the minimum 20th percentile of interpreted shear strength values. The undrained shear strength ratio selected has a higher shear strength ratio than the effective shear strength and is reflective of dilatant behaviour.

A CPT screening tool which can be used to determine potential dilatant and contractive nature of silt and sands is presented in Shuttle and Cunning (2008), and has previously been plotted for a number of CPTs. From this it was determined that the farmed red mud is likely dilatant, and is unlikely to generate excess pore pressure during shearing.

7.3.3 HDPE Geomembrane Interface

The base of the BRDA Phase 1 extension has been lined with a HDPE geomembrane, of which most was smooth but a portion along the northern edge in the area closest to the proposed borrow pit, was textured.

The geomembrane interface strength between the overlying red mud and between the underlying soil has been assumed to be greater than 13 degrees, which is the equivalent shear strength of the red mud. This interface has therefore not been modelled separately within the stability analysis.

The sensitivity of the geomembrane interface shear strength to the FoS was analysed by reducing the interface shear strength to a residual value of 10 degrees. The interface was modelled as a 1 m thick layer within the model and represents the interface between the geomembrane and either the red mud or underlying soil.

7.3.4 Estuarine deposit

The estuarine deposit along the foundation has been analysed as an effective (drained) strength, consistent with previous analysis. This is the normal procedure for analysing the behaviour of soils which are loaded slowly and where there is no build-up of excess pore pressure. Additional analysis has also been included to assess the sensitivity on the factor of safety if the estuarine deposits were to exhibit undrained shear strength properties.

7.4 Pseudo-Static Stability Assessment

A pseudo-static approach to analysing the stability of slopes subjected to blast vibrations, characterised by high frequency pulses, has been developed by Wong and Pang (1992). The blasting vibration at the bedrock is modelled as a simple harmonic motion in the horizontal direction.

The disturbing effect of the blast is modelled as an equivalent inertia force F, and calculated according to formula:

F = W x K

where

- W is the weight of the soil mass above the potential slip surface; and
- K is the response peak ground acceleration coefficient (in g) of the soil mass.

This pseudo-static approach is well established and commonly used for seismic analysis. The response peak ground acceleration coefficient is used as an input in a limit equilibrium slope stability analysis software, and a Factor of Safety (FoS) against instability calculated.

7.4.1 Response Peak Ground Acceleration Coefficient

The response peak ground acceleration coefficient (K) at the soil mass is calculated from the input bedrock motion and the dynamic response of the soil slope, and can be expressed as:

$$\boldsymbol{K = K_a\left(\frac{PPA}{g}\right)}$$ **(Wong and Pang 1992)**

where

- K_a is the magnification factor determined from response analysis, and based on the damping factor and fundamental period of the slope;
- PPA is the peak particle acceleration (in m/s^2) caused by the blast vibration; and
- g is the acceleration due to gravity (9.81 m/s^2)

A soil damping factor (λ) of 0.2 for the fundamental period of vibration, and an infinite duration of input bedrock ground motion were assumed in the response analysis.
Values of K_a has been assessed by modelling the slope as a multi degree system taking into consideration the higher vibration modes of the slope.
The magnification factor (K_a) varies with the frequency of the earthquake blast and the fundamental period of the slope, which is in turn dependant on the height of the slope (H) and shear wave velocity (V_s) of the red mud.

The frequency of the blast typically varies between 30 Hz to 100 Hz (Wong and Pang 1992), and frequency of 30 Hz was conservatively assumed for the analysis.

The shear wave velocity of the red mud varies between approximately 178 m/s to 355 m/s (at low strains around 1%), based on results of bender element laboratory testing conducted in 2004. A conservative average value around 300 m/s was chosen for the analysis. The height of the slope in the region closest to the blast is approximately 20 m, resulting in a ratio of Vs/H of approximately 15. K_a can then be determined from Figure 6 (from Wong and Pang 1992).

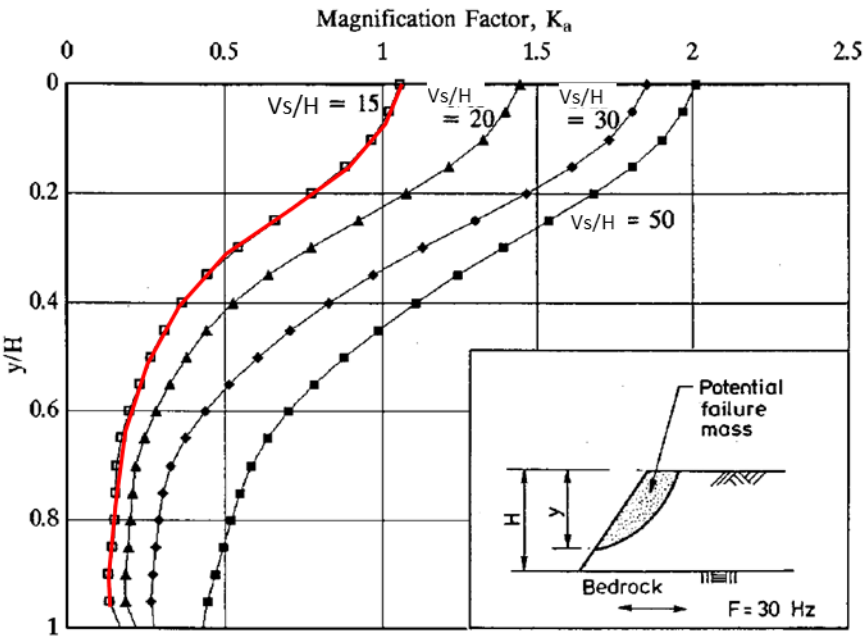

Figure 6: Magnification Factor vs Depth of Sliding Mass (from Wong and Pang 1992)

The values for the response peak ground acceleration coefficient (K) used in the stability analyses are shown in Table 7.

Table 7: Peak Ground Acceleration Coefficient

Ratio of Slip Surface Depth (y/H)	Response peak ground acceleration coefficient (K)		
	PPV = 15 mm/s	PPV = 20 mm/s	PPV = 25 mm/s
1	0.049	0.065	0.081
0.7	0.045	0.060	0.075

7.4.2 Pseudo-Static Analysis Results

The pseudo-static stability analyses results on the sections analysed are summarised in Table 8, and the analyses presented in Appendix C. The static FoS against slope instability is included as a reference. All analysis shown are based in total stress (undrained) stability analysis which assumes undrained material strength parameters, and represents the worst case scenario. Due to the shallow slope, a number of slip surfaces were analysed, which include:

- Overall slope – The slip surface extends from the crest to toe;
- Upper slope – The slip surface is limited to the upper slope extending from the crest to the middle bench (El. 14 mAOD); and
- Lower Slope – The slip surface is limited to the lower portion of the slope from approximately El. 14 mAOD to the toe.

Table 8: Pseudo-static Stability Analysis Results

Slip Surface Location	Static Factor of Safety (FoS)	Pseudo-static Factor of Safety (FoS)			Figures (Appendix C)
		PPV = 15 mm/s	PPV = 20 mm/s	PPV = 25 mm/s	
Overall Slope	1.6	1.1	1.1	1.0	C1, and C4 to C6
Upper Slope	1.6	1.3	1.2	1.1	C2, and C7 to C9
Lower Slope	1.6	1.2	1.1	1.1	C3, and C10 to C12

Notes:

1. **FoS reported to one decimal place as is the industry standard**
2. **Results are based on total stress (undrained) analysis**
3. **Lower slope stability results are based on a slip surface depth of approximately 16 m.**

Figure 7 below plots the reduction in FoS with an increase in PPV for the slip surface extending through the overall slope. The calculated peak response ground acceleration used in the pseudo-static analysis, based on the PPV, is plotted on the y axis. It is evident from the chart that if the PPV is limited to 20 mm/s, the FoS remains above the minimum recommended factor of safety of 1.0.

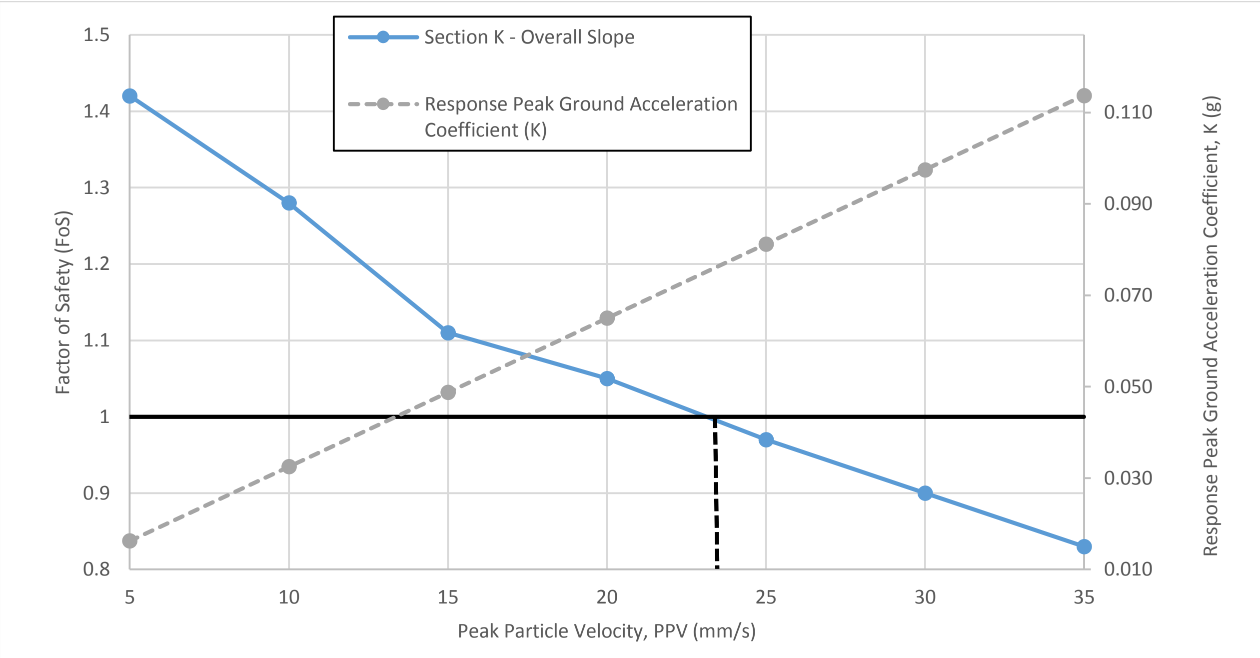

Figure 7: Pseudo-Static Stability Analysis Results, Overall Slope Slip Surface

Additional analyse were conducted to determine the sensitivity of the FoS to a reduction in the undrained shear strength of the red mud along the toe of the slope, along with undrained material strength parameter for the underlying estuarine deposit. There is a slight reduction in the FoS, and the PPV should be limited to 15 mm/s for the FoS to remain above the minimum recommended factor of safety of 1.0 (Figure 8). The sensitivity analysis is included in Appendix C, Figure C13.

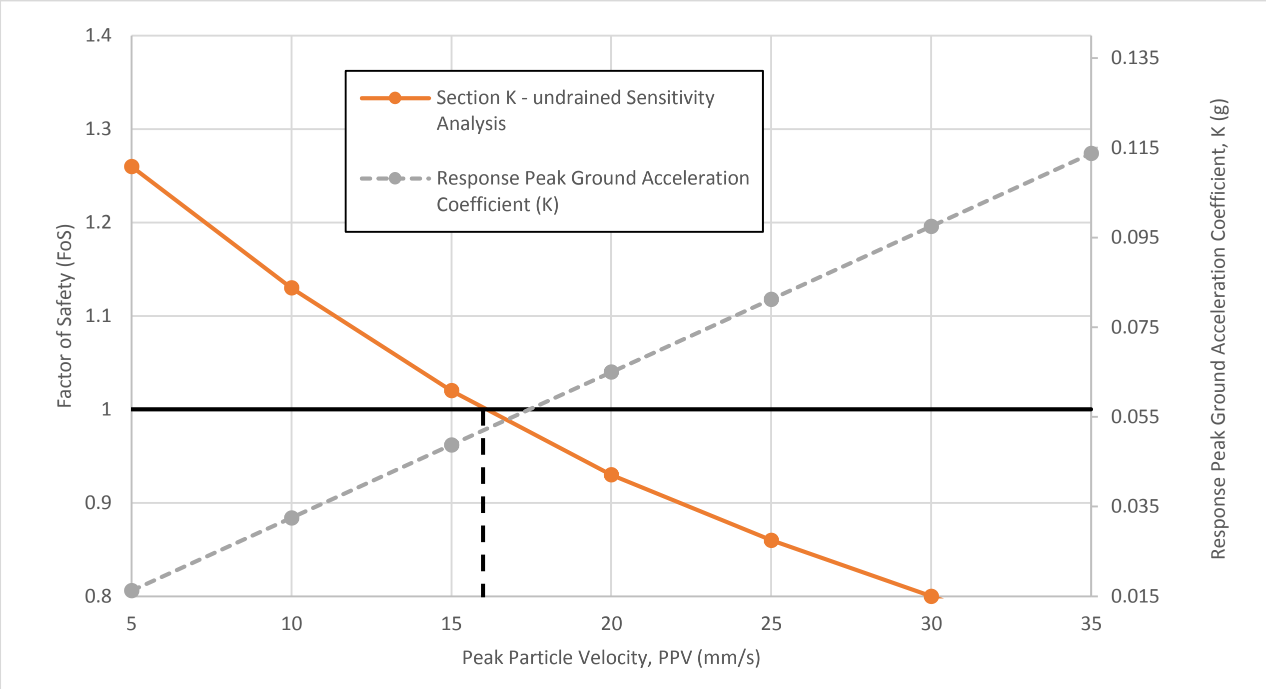

Figure 8: Sensitivity Pseudo-static Stability Analysis, Reduced Red Mud Strength along the Toe

A reduction in the geomembrane interface shear strength, along the base of the Phase 1 BRDA extension, was also analysed A reduction in shear strength to 10 degrees would still maintain a FoS of 1.0 against instability, with a PPV of 15 mm/s (Figure C14 in Appendix C).

7.5 Post- Blast Stability Analysis

Following a blast, there is a rapid release of energy which generates a compressive stress wave producing intense radial compressive strains. one of the issues of conducting blasting nearby the BRDA is the potential for blast-induced residual pore pressure increases that reduce the shear strength for a time period long enough to allow gravity to cause the instability of the slope. Three stages of explosive-induced pore pressure typically occur:

1) the peak transient pore pressure increase, which is directly associated with the passage of the compressive stress wave;

2) the residual pore pressure increase, which is induced by the passage of the stress wave but occurs after the passage of the stress wave; and

3) the residual pore pressure dissipation stage, which occurs as the soil consolidates.

The section analyses the potential slope instability brought on by an increase in pore pressure, and a subsequent decrease in the overall effective stress which results in a decrease in the overall strength of the material. The residual pore pressure increase is the critical condition to be analysed. The peak transient pore pressure increase is a temporary increase and dissipates to the residual pore pressure relatively quickly.

Liquefaction, a concern for any loose hydraulically placed material, occurs when the excess pore pressure approaches the initial vertical effective stress within the soil structure. The analysis presented in this section, therefore, models the potential for cyclic liquefaction of the red mud. The risk of cyclic liquefaction due to blasting is typically less than that due to an earthquake, and is as a result of the shorter time frame of the blast. Cyclic induced liquefaction is the increase pore pressure with each cyclic loading. A minimum number of cycles, depending on the nature of the material, is required to induce liquefaction.

7.5.1 Peak Excess Pore Pressure

The compressive stress wave induced peak transient pore pressure (Δu_{peak}) can be estimated based on an empirical relationship (Jacobs 1988) for charges detonated in saturated soils:

$$\Delta u_{\text{peak}} = 100{,}300 \left(\frac{R}{M^{1/3}}\right)^{-2.67}$$ **in kPa (Jacobs 1988)**

where

- R is the is the distance (in m) between the explosive and recording site; and
- M is the mass (in kg) of TNT, a high explosive.

The distance (R) and explosive mass (M) can also be used to calculate the PPV according to the following equation also by Jacobs (1988), and which can be inserted in the above equation:

$$\boldsymbol{PPV = 12.9(\frac{R}{M^{1/3}})^{-2.21}}$$ **in m/s**

The calculated peak transient pore pressure increase is for the PPV values identified in the pseudo-static analyses are included in Table 8. The values presented are conservative as they are based on an empirical relationship for charges detonated within the saturated and not outside, as is the situation for the proposed borrow pit development.

Table 9: Calculated Peak Transient Pore Pressure Increase

Peak Particle Velocity, PPV (mm/s)	Peak Transient Pore Pressure Increase, Δu_{peak} (kPa)
15	28.6
20	40.5
25	53.0

7.5.2 Residual Excess Pore Pressure

A residual increase in pore pressure occurs when a contractive (relatively loose) soil responds plastically to the blast-induced strain, resulting in compression and subsequent increase in pore pressure. Minimal increase in pore pressure would occur in a denser soil due to minimal compression of the soil structure. A number of empirical relationships have been developed to determine the expected residual excess pore pressure (Δu_{res}) based on the PPV. Three of these relationships are presented in this report, and are calculated as follows:

$$\boldsymbol{PPR = \frac{\Delta u_{res}}{\sigma'_0} = 6.67\, PPV^{0.33}\, \sigma'_0{}^{-0.31}\, D_r{}^{-0.179}}$$ **(Veyera 1985)**

$$\boldsymbol{PPR = \frac{\Delta u_{res}}{\sigma'_0} = 10.59\, (\frac{100PPV}{Vs})^{0.43}\, \sigma'_0{}^{-0.17}\, D_r{}^{-0.18}}$$ **(Hubert 1986)**

$$\boldsymbol{PPR = \frac{\Delta u_{res}}{\sigma'_0} = 16\, (\frac{100PPV}{Vs})^{0.33}\, \sigma'_0{}^{-0.31}\, D_r{}^{-0.179}}$$ **(Veyara and Charlie 1990)**

where

- PPR is the pore pressure ratio;
- σ'_0 is the initial effective stress (in kPa)
- V_s is the compression wave velocity (in m/s), estimated to be 1,600 m/s (saturated clay / sand);
- PPV is the peak particle velocity (in m/s), taken as the limiting value of 25 mm/s; and

- D_r is the relative density of the material, and is a measure of the in-place density with respect to the densest and loosest states the material can attain. A value of 40% is used for the assessment.

It is conservatively estimated that the relative density of the unfarmed red mud varies between approximately 40% and 60%, and will vary with depth as the red mud consolidates under its own weight. This estimate is based on laboratory testing conducted on the red mud.

A comparison of the different empirical relationships for a constant effective stress (σ'_0) of 100 kPa is shown in Figure 9. From this it is seen that the relationship developed by Veyara and Charlie (1990) typically results in the highest PPR and the Hubert relationship typically produces the lowest PPR.

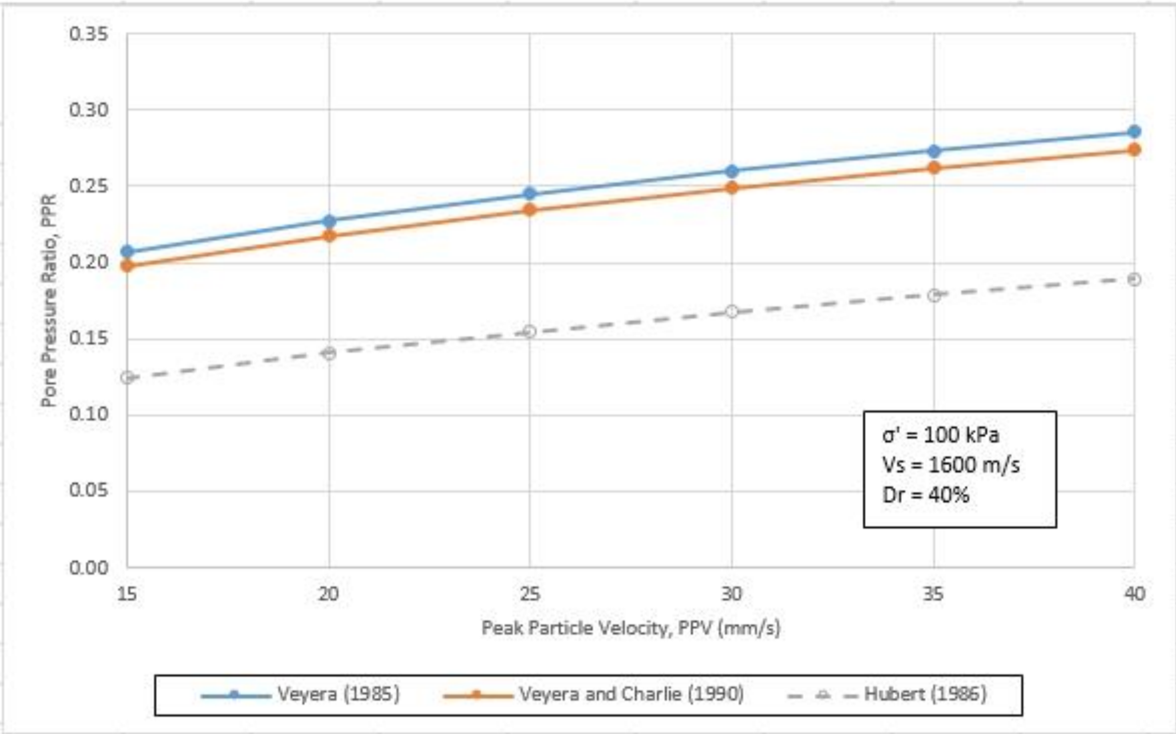

Figure 9: Empirical Relationships of Peak Particle Velocity (PPV) versus Pore Pressure Ratio (PPR)

The increase in pore pressure is presented in the following section and is plotted based on the effective stress along the slip surface analysed.

7.5.3 Post-Blast Stability Analysis Results

The slope/W software used for the analysis has two functions which allow excess pore to be analysed, these include the r_u coefficient and the B-bar coefficient. Both were used in the analysis and found two produce a similar result, and represent the average pore pressure ratio (PPR) value.

Table 10 provides a summary of the FoS based on the PPR, and the analysis results included in Appendix C, Figures C15 to C17. Excess pore pressure is only assumed to be generated within the unfarmed red mud. An equivalent PPV value is provided based on a conservative red mud relative density of 40%. The initial pore pressure, and the total pore pressure generated for increasing PPR along the base of the slip surface is plotted in Figure 10. The calculated peak and residual pore pressure for a PPV of 25 mm/sec (and unfarmed red mud DR = 40%) is shown as a reference. The equivalent PPV in Table 10, and pore pressure in Figure 10 is calculated according to the relationship by Veyera (1985).

Table 10: Slope Stability Analysis with Excess Pore Pressure

r_u Coefficient	Average Pore Pressure Ratio (PPR) [a]	Factor of Safety (FoS) [b]	Equivalent PPV to produce Δu_{peak} (mm/s) [c]	Equivalent PPV to produce Δu_{res} (mm/s) [d]
0.1	0.20	1.4	~ 15	~ 15
0.2	0.35	1.3	~ 25	~ 80
0.3	0.50	1.2	~ 35	~ 300

Notes:

a) Excess pore pressure assumed in the unfarmed red mud.

b) FoS for Section K, overall slope instability and total stress (undrained) analysis.

c) Calculated using Jacobs (1988).

d) Calculated using Veyera (1885) with an unfarmed red mud relative density of 40%.

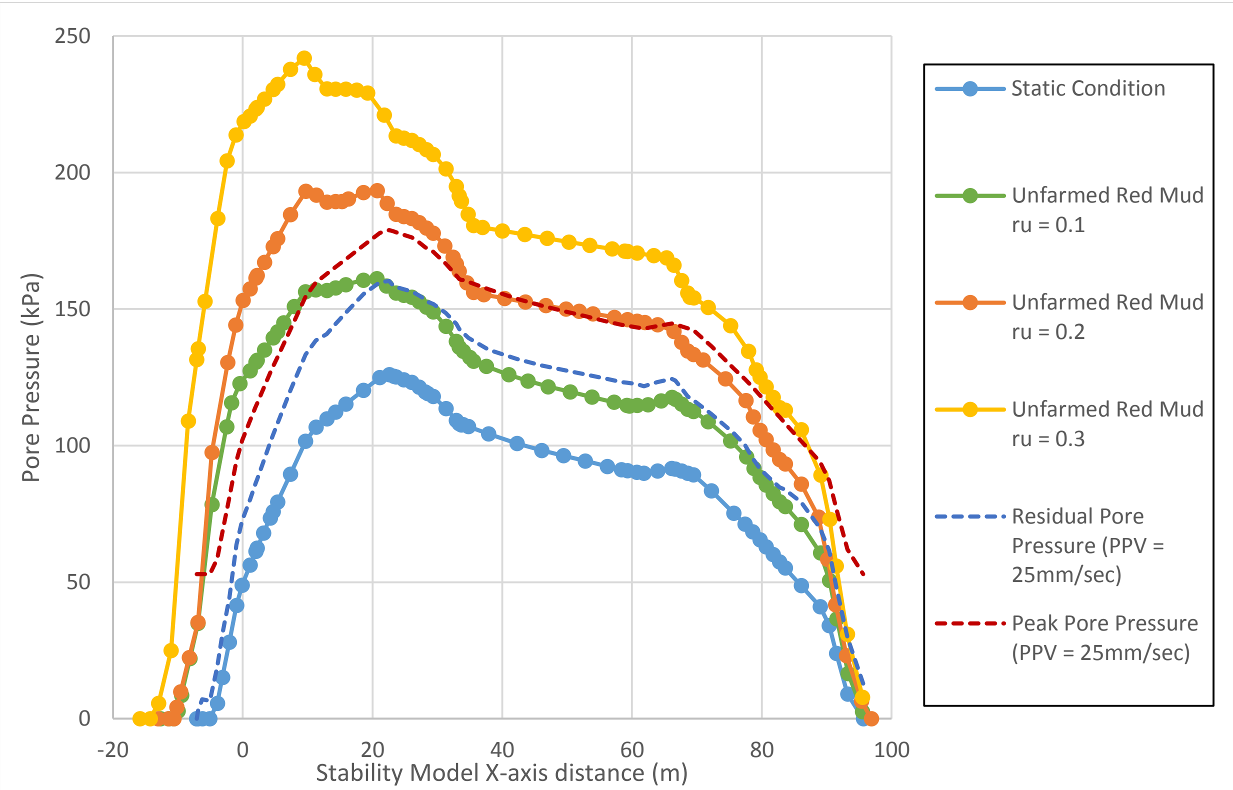

Figure 10: Pore Pressure Generated along the Slip Surface for Various PPR

The PPV generated by a nearby blast would need to exceed 350 mm/s to generate sufficient excess pore pressure to reduce the FoS below the recommended 1.2 for post blast condition (Veyera 1985). A PPV exceeding 40 mm/s would be required to produce a PPR of 0.5, if the Veyara and Charlie (1990) relationship were used.

It is estimated that the peak transient pore pressure increase associated with a PPV of 35 mm/s will only result in an average PPR of 0.3.

7.6 Stability Analysis Results Summary

A blast producing a PPV of approximately 25 mm/s is very unlikely to result in instability due to vibration of the blast itself (pseudo-static analysis), and as a result of residual excess pore generated by the blast wave (post- blast analysis). This is consistent with the observations reported in case histories (Refer to Section 4.1).

Sensitivity analysis representing the worst case condition of the red mud forming the slope has shown that a PPV of 15 mm/sec will produce a FoS against slope instability of greater than unity. From Figure A1 it is evident that the PPV reduces down to 15 mm/s near the toe of the slope, so that even if there has been no increase in the strength of the red mud in this area since 2009, the resulting FoS is still within recommended limits.

It is to be further noted that pseudo-static analysis presented here is considered conservative, as reported in Wong and Pang (1992).

Blast-induced vibrations and dissipation of residual pore pressure may also induce settlement but which is anticipated to be relatively minor based on the results of the Pseudo-static and post blast analysis. The anticipated extent of the settlement, if any, has not been assessed as part of this analysis as this is less of a concern for the BRDA as no water is installed on the facility which could overtop.

Figure A3 plots the decrease in PPV with distance from a blast on the boundary of the proposed borrow pit, and represents the maximum PPV the BRDA would be subjected to, based on a 35 kg blast (Refer to Section 6.0). A PPV of 25 mm/s is experienced on the edge of the BRDA, and reduces to below 15 mm/s before the blast wave reaches any significant slope height.

The risk of slope instability occurring due to blasting at the proposed borrow pit is considered highly unlikely based on the analysis presented. Careful coordination of the blast and continued monitoring during the pit development to confirm the parameters established in the assessment would further reduce any risk of instability.

8.0 SUMMARY AND RECOMMENDATIONS

The effect of blasting within the footprint of the proposed Borrow Pit was evaluated and found to pose a very unlikely risk to the stability of the adjacent BRDA. The intensity of ground vibrations due to the blasting, expressed as a peak particle velocity (PPV) was calculated based on the type and size of blast and characteristics of the area. This was then calibrated with previous blasting in the area. The PPV reduces with distance from the blast.

The stability analyses undertaken found that the calculated PPV, for the blast analysed, would not cause instability of the BRDA. The stability analysis consisted of a pseudo-static analysis which evaluated the stability base on the blast vibration; and a post-blast analysis which evaluated the stability due to an increase in pore pressure within the red mud.

The following are recommendations for blasting at the proposed Borrow Pit.

- Estimated set-back distances from blasts at the Borrow Pit to limit the PPV to < 25 mm/s, assuming a maximum instantaneous explosive charge weight of 35 kg (MIC), are:
  - **53 m to the BRDA embankment**, and
  - **50 m at the end of the life of the Borrow Pit to the GNI gas transmission pipeline**.
- Initial blasts shall be conducted on the eastern extent of the face of the proposed Borrow Pit, to maintain the furthest distance from the BRDA (approximately 150 m);
- Results of the initial blast vibration monitoring can be used to calibrate the PPV prediction model and refine the values for k and b; and

- Run the calibrated prediction model to determine a maximum explosive charge weight (MIC) to remain compliant with the designated PPV limits for the extent of the Borrow Pit that is close to the BRDA.

Note: there may be other structures that require lower PPV limits and that these may then become the controlling factors.

The following monitoring is recommended to be conducted during the blasts at the Borrow Pit:

- Blast vibration monitoring at various locations within the BRDA, and should include at a minimum the following:
  - At the toe of the slope at the location closest to the Borrow pit to provide an indication of the maximum PPV that the red mud would be exposed to.
  - Monitoring at the mid-point and crest of the slope, to provide an indication of the reduction in PPV with distance from the blast, but also potential amplification through the depth of the red mud.
- Pore pressure monitoring at various locations within the red mud through the installation of vibrating wire piezometers. These will measure any excess pore pressure induced by the blasting and ensuring that sufficient time is maintained between blasts to let any residual pore pressure increase to dissipate. These would be located close to the blast vibrating monitoring points to allow calibration of increased pore pressure with PPV;
- Monitor inclinometers and extensometers after each blast to confirm that there were no displacements or settlements as a result of the blast; and
- A recommended threshold criteria and response framework is presented in Table 11 below.

Table 11: Response Framework for Blasting at Aughinish Borrow Pit

	Threshold Criteria During Blasting		
	Acceptable Situation	**Concern**	**High Risk Situation**
PPV Criteria (mm/s)	+25% of predicted PPV [a]	+25% to +50% of predicted PPV [a]	> +50% of predicted PPV [a]
Pore Pressure Criteria (kPa)	Less than 25% increase	25 to 75 % increase	Greater than 75% increase
Inclinometer and extensometer displacement criteria	Less than 5 mm	Between 5 and 10 mm	Greater than 10 mm
Action Required	• Continue to conduct monitoring and visual assessment in accordance with the QC/QA requirements set out for pit blasting	• Drill & Blast Engineer to assess the situation • Document location, visually assess and photograph • Increase visual inspection • Identify potential causes • Implement blast design review • Plan and take appropriate mitigation measures following blast design review	• Temporarily suspend access to the critical area and suspend activities • Assess the situation, update planning and take appropriate mitigation measures with blast design review

	Threshold Criteria During Blasting		
Instrumentation Monitoring	• Continue to monitor dissipation of excess pore pressure (if any). • Pore pressure increase to be less than 10% before conducting next blast.	• Continue to monitor dissipation of excess pore pressure (if any). • Pore pressure increase to be less than 10% before conducting next blast.	• Continue to monitor dissipation of excess pore pressure (if any). • Take readings of all inclinometers and extensometers, and take readings daily. • Pore pressure increase to be less than 10% before conducting next blast.
Personnel Notified	AAL Attendant Person Drill & Blast Engineer	AAL Attendant Person Drill & Blast Engineer AAL BRDA Supervisor Golder Engineer	AAL Attendant Person Drill & Blast Engineer AAL BRDA Supervisor Golder Engineer AAL General Manager Golder Blast Team

Notes:

a) Predicted PPV is a live number as the current estimations are based on using k and b values of 300 and 1.14 respectively. Following an assessment of the monitoring data from the initial blasts and subsequent blasts in the Borrow Pit at conservative distances from the BRDA, these k and b values may be adjusted to better calibrate the model.

■ A number of measures can be put in place to reduce the PPV should the initial blast monitoring record values in excess of the predicted maximum values. These measures may include using smaller explosive charge weights per borehole or a process called 'decking' in which either the charge load per hole is reduced, the amount of explosives detonated per delay is reduced, or both. This process would require individual borehole and blast design assessments.

9.0 REFERENCES

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Report Signature Page

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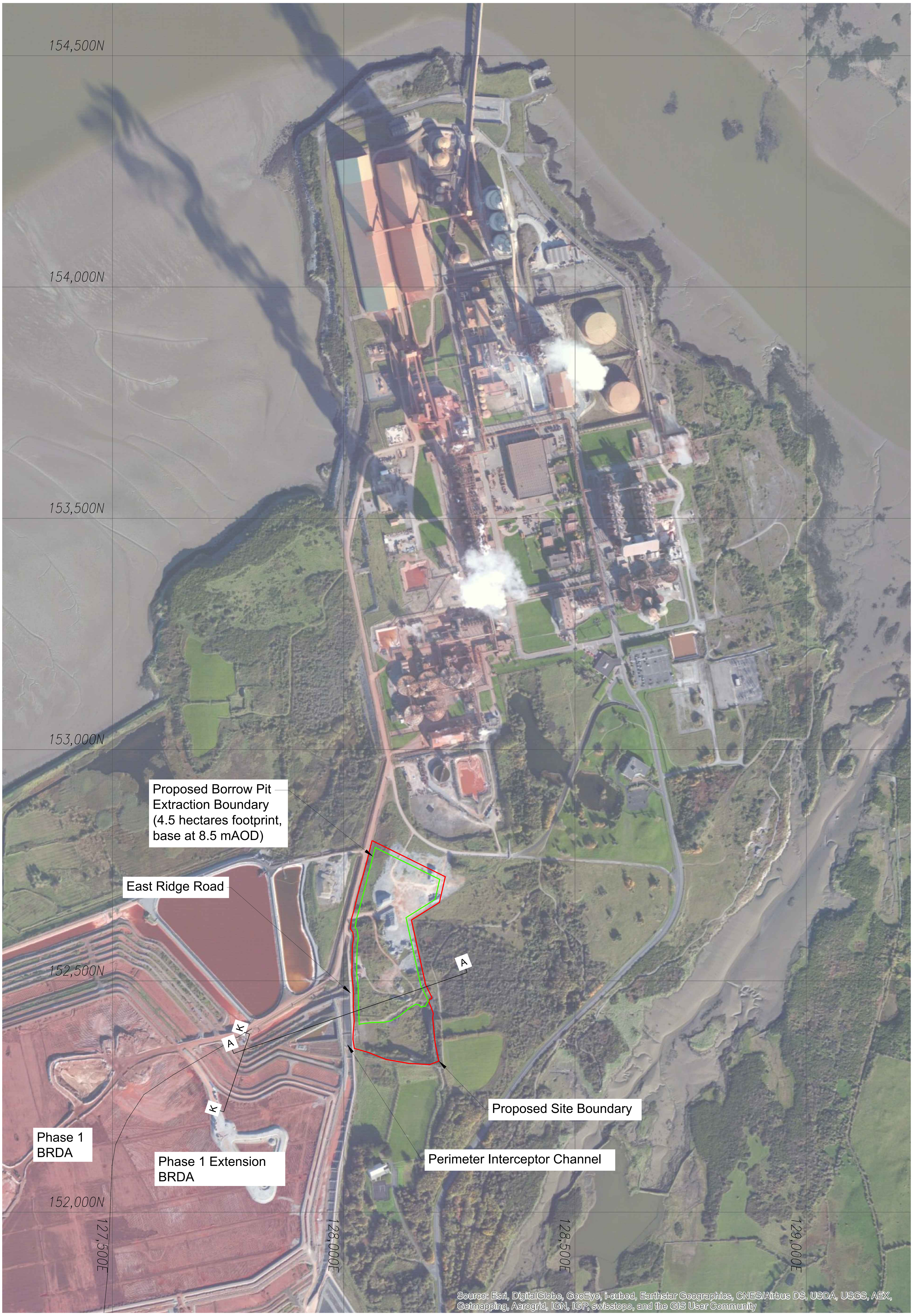
APPENDIX A

Drawings

Path: ---- | File Name: 01 - Site Location.dwg



LEGEND

- APPLICATION EXTRACTION BOUNDARY
- APPLICATION SITE BOUNDARY

DRAFT

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES & TO O.S. DATUM.

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

Golder Associates

YYYY-MM-DD	2017-Jun-13
PREPARED	POB
DESIGN	POB
REVIEW	BB
APPROVED	BB

PROJECT

BORROW PIT DEVELOPMENT

TITLE

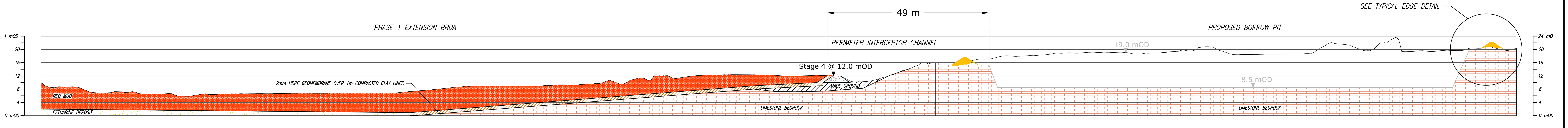
SITE LOCATION

PROJECT No.	DRAWING No.	Rev.	SCALE
1667376	01	A	1:4,000 A1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

0 25 mm

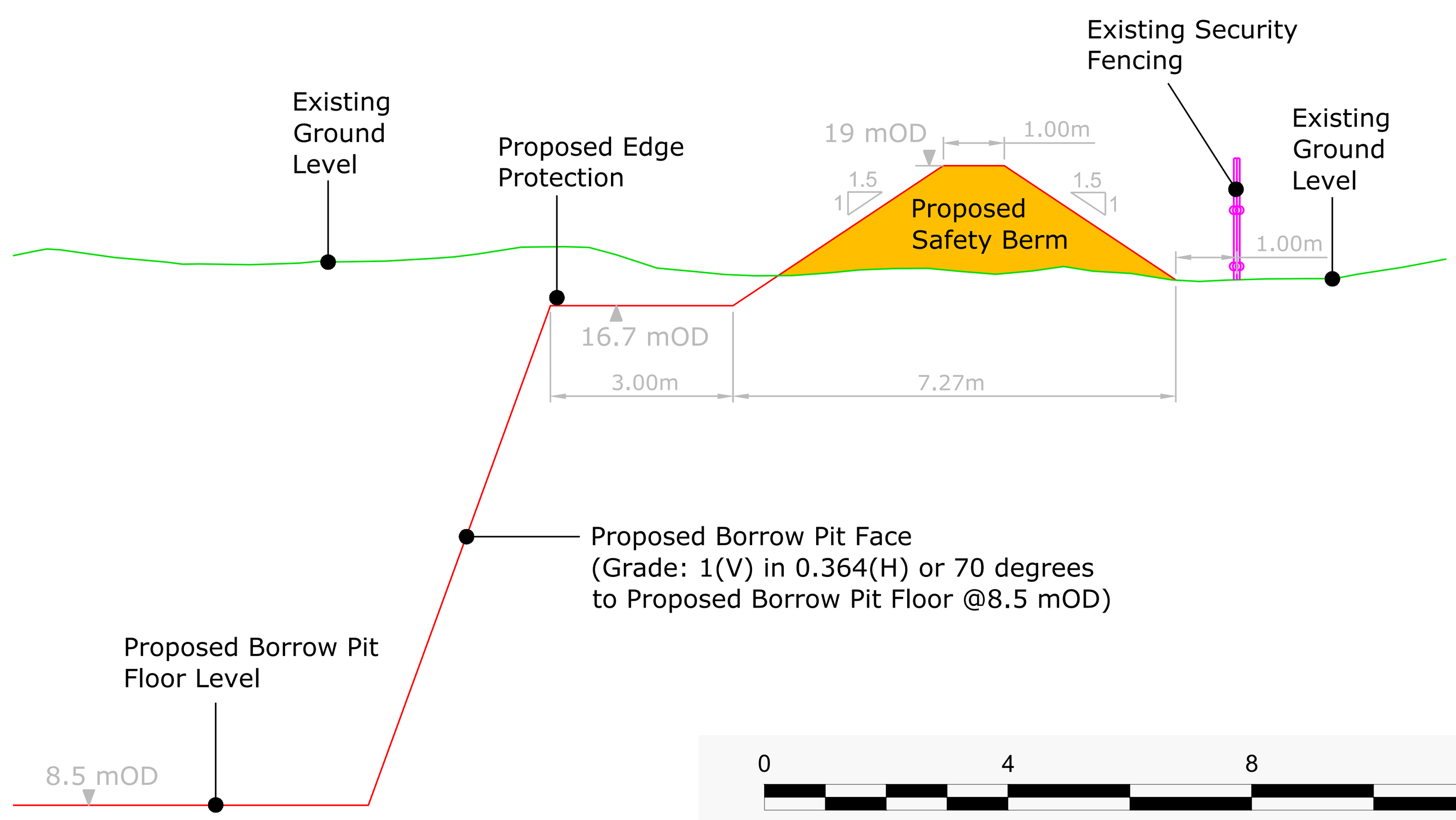
Cross Section WE (West East)



SECTION A-A

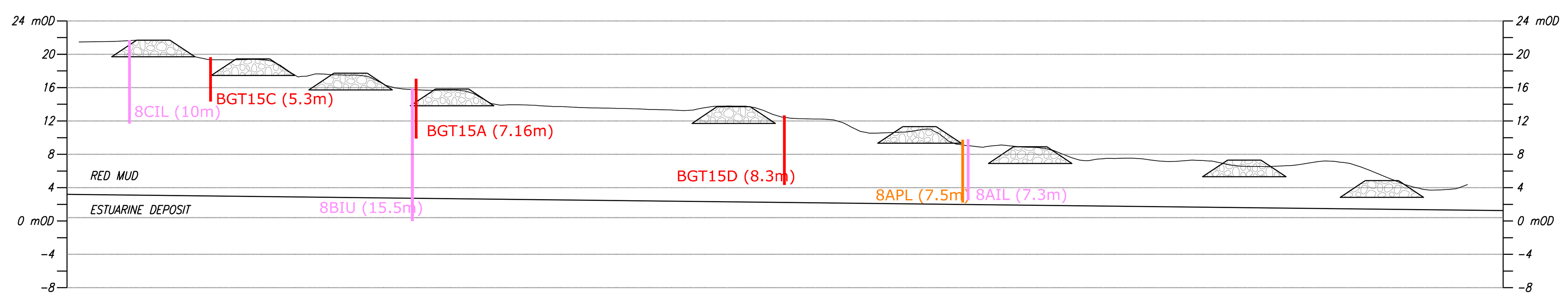
0 24 48 72m
1:600

Typical Edge Detail



0 4 8 12m
1:100

Cross Section SW-NE (South-West - North-East)



SECTION K-K

0 20 40 60m
1:500

LEGEND

EXISTING GROUND CONTOUR (mAOD)

PROPOSED BORROW PIT DESIGN CONTOUR (mAOD)

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

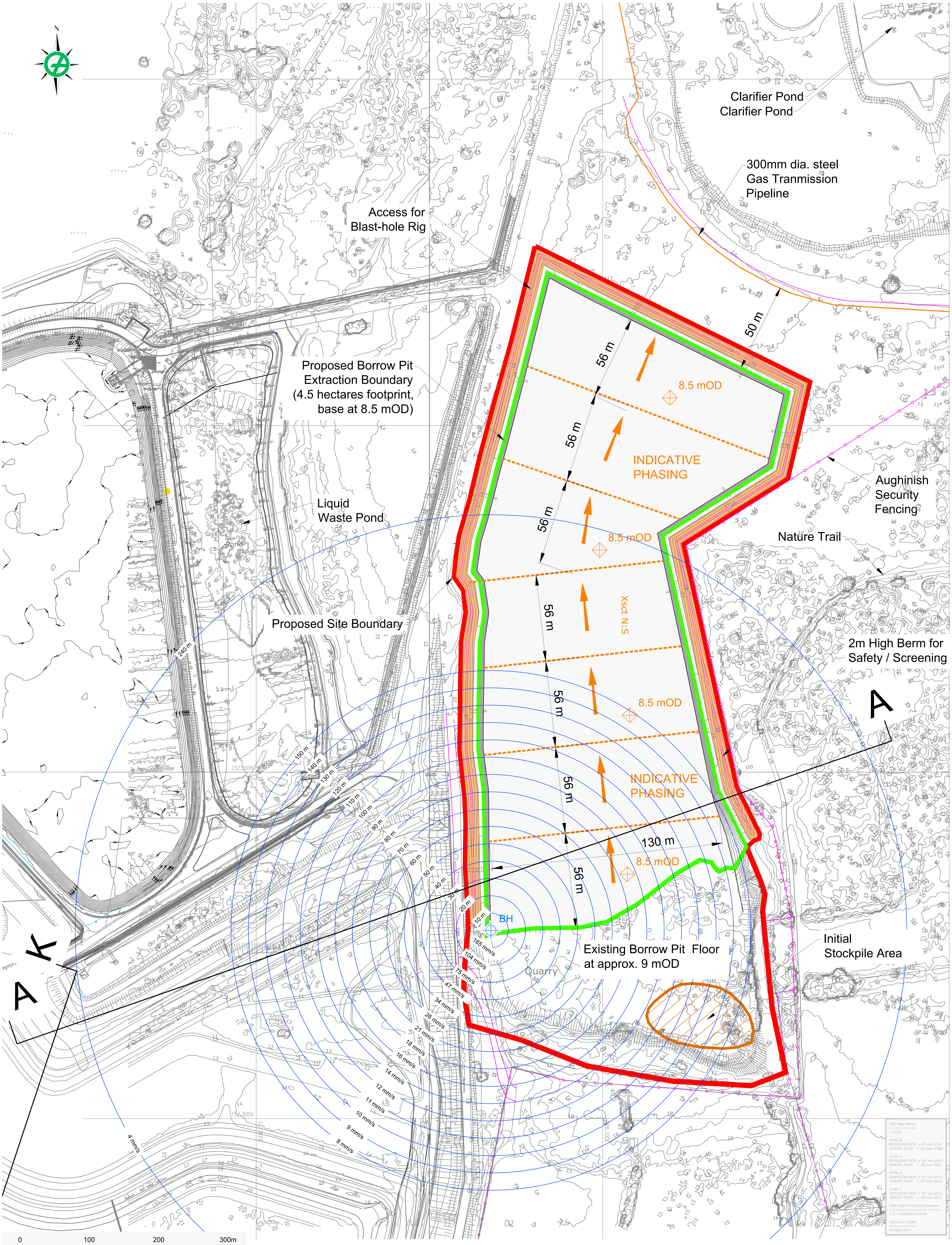
LEVELS ARE IN METRES & TO O.S. DATUM.

DIMENSIONS ARE IN METRES.

REFER TO DRAWINGS PA-02, PA-03 & PA-04 FOR SECTION DETAIL LOCATIONS

CLIENT	AUGHINISH ALUMINA LTD.
PROJECT	BORROW PIT DEVELOPMENT
CONSULTANT	Golder Associates
YYYY-MM-DD	2017-May-03
PREPARED	POB
DESIGN	POB
REVIEW	BB
APPROVED	BB
TITLE	CROSS SECTIONS
PROJECT No.	1667376
DRAWING No.	02
Rev.	A
SCALE	As shown A1

Path: ---- | File Name: 03 - PPV Contour Plot.dwg



0 100 200 300m

1:2,500

LEGEND

- APPLICATION EXTRACTION BOUNDARY
- APPLICATION SITE BOUNDARY
- EXISTING GROUND CONTOUR (mAOD)
- PPV CONTOUR IN mm/s FOR W = 35 kg

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES & TO O.S. DATUM.

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

Golder Associates

YYYY-MM-DD	2017-Jun-19
PREPARED	POB
DESIGN	POB
REVIEW	BB
APPROVED	BB

PROJECT

BORROW PIT DEVELOPMENT

TITLE

PPV CONTOUR PLOT FOR W = 35 kg

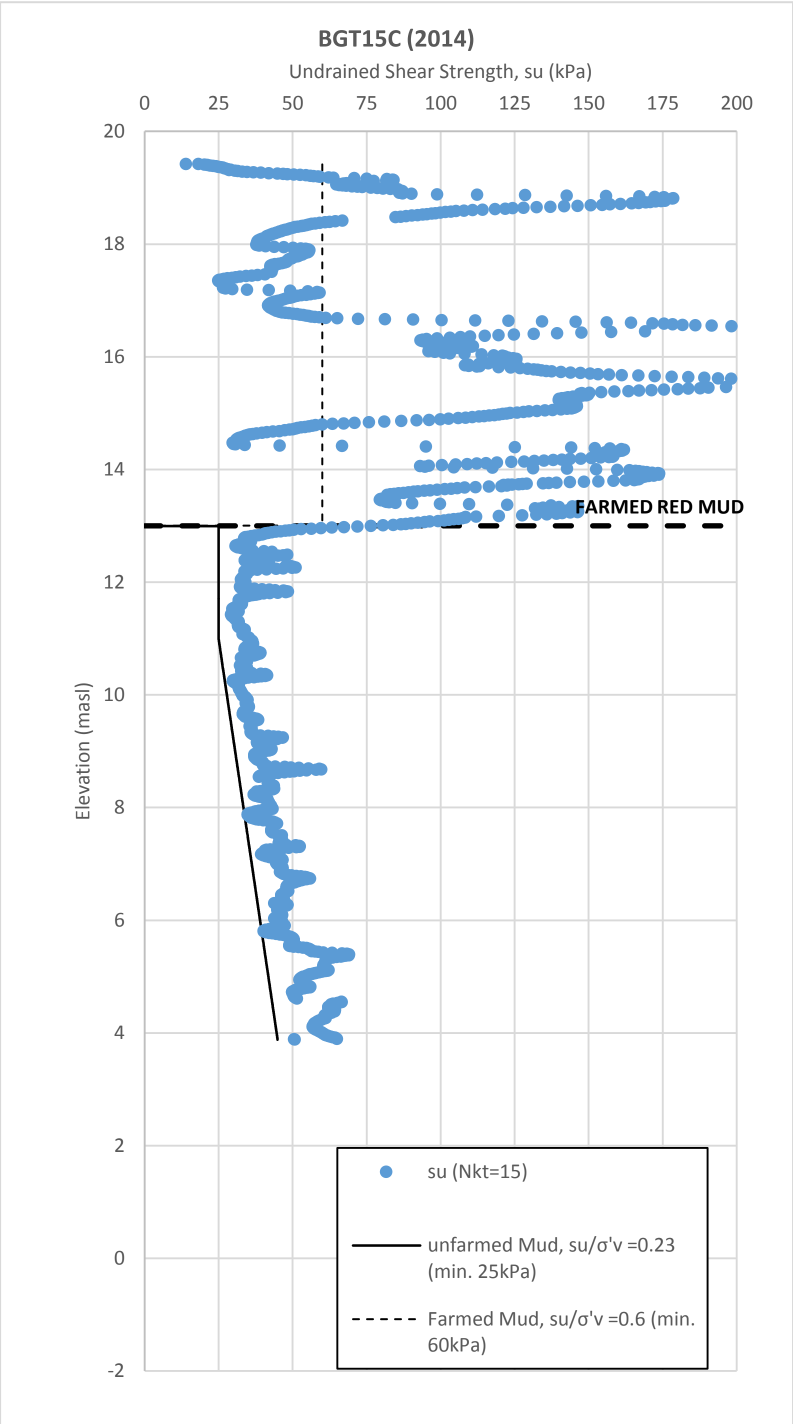
PROJECT No.	DRAWING No.	Rev.	SCALE
1667376	03	A	1:1,000 A1

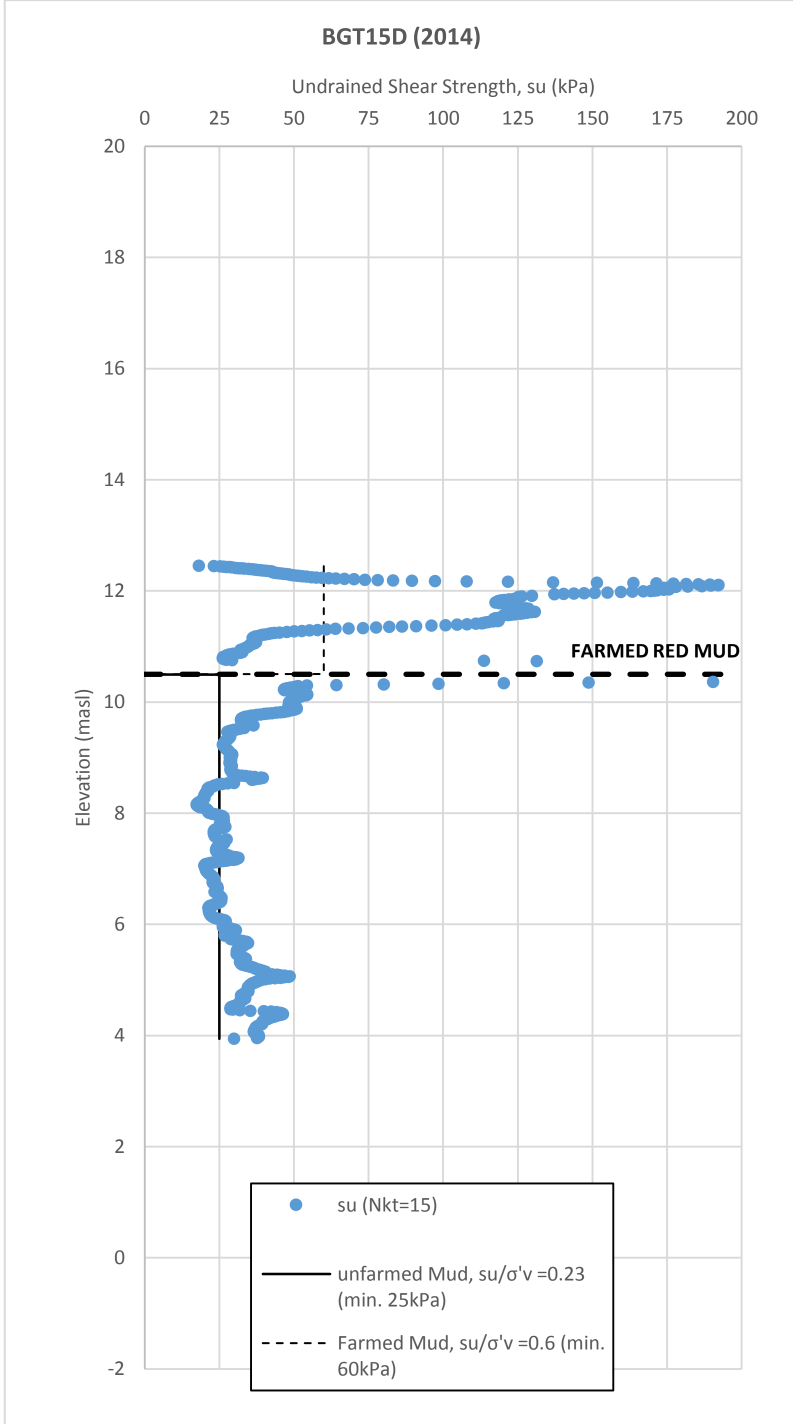
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

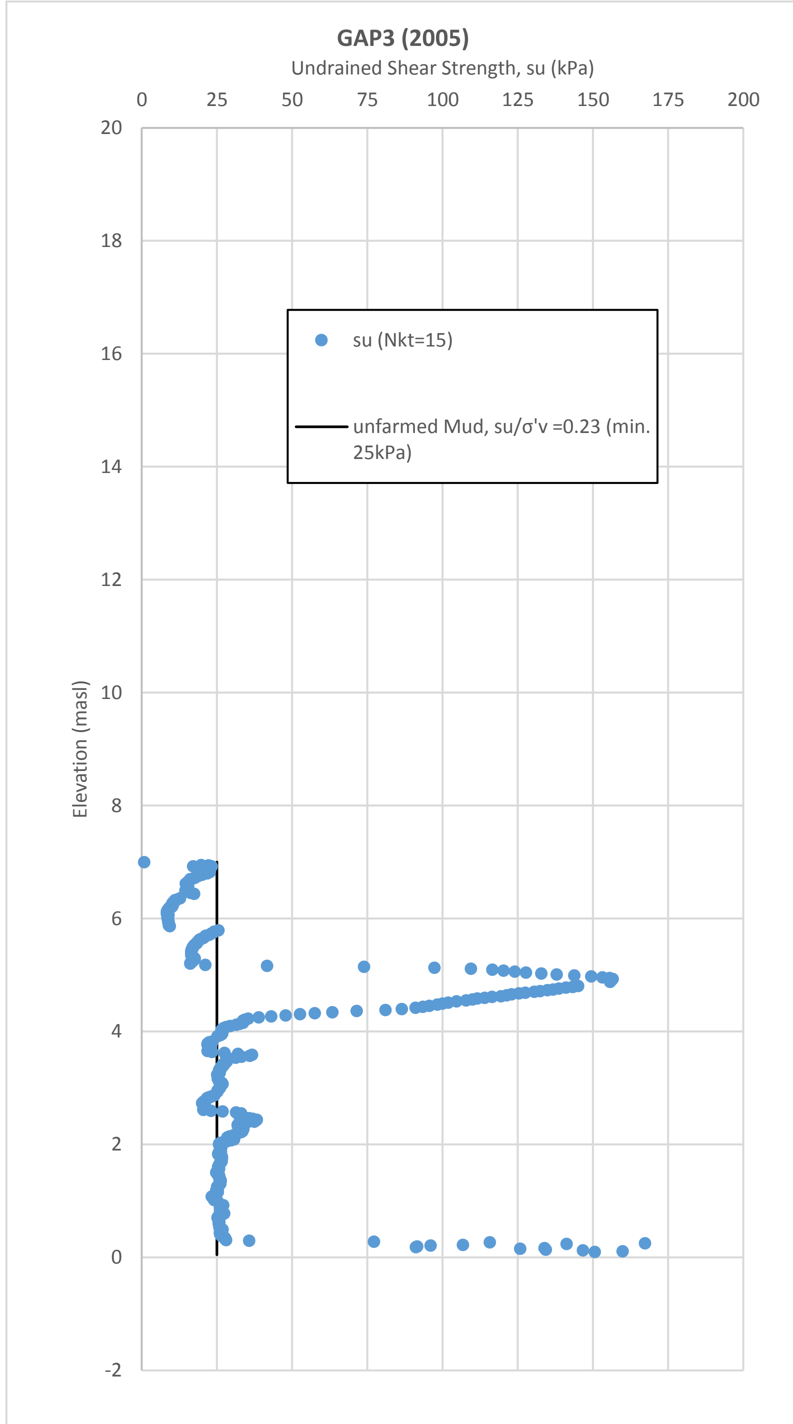
0 25 mm

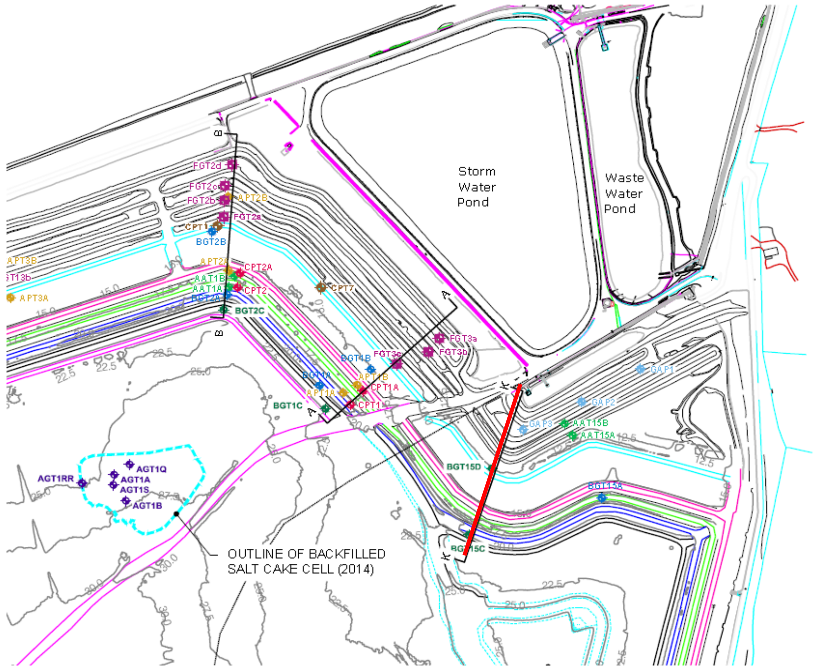
APPENDIX B

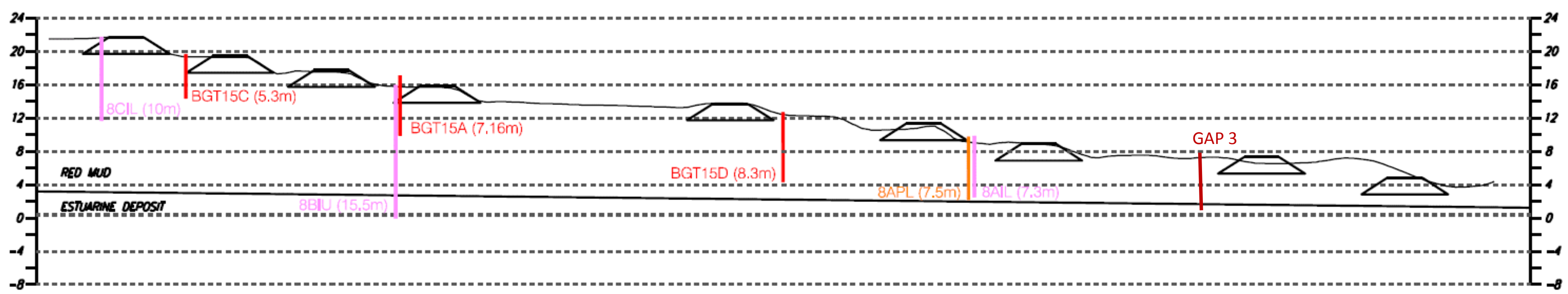
Cone Penetration Testing Analysis











Notes:

1. Elevations Shown are approximate
2. Undrained shear calculated according to the formula s_u = (net cone resistance)/N_{kt}, where Nkt is undrained strength factor determined to be 15

CLIENT
AUGHINISH ALUMINA LIMITED

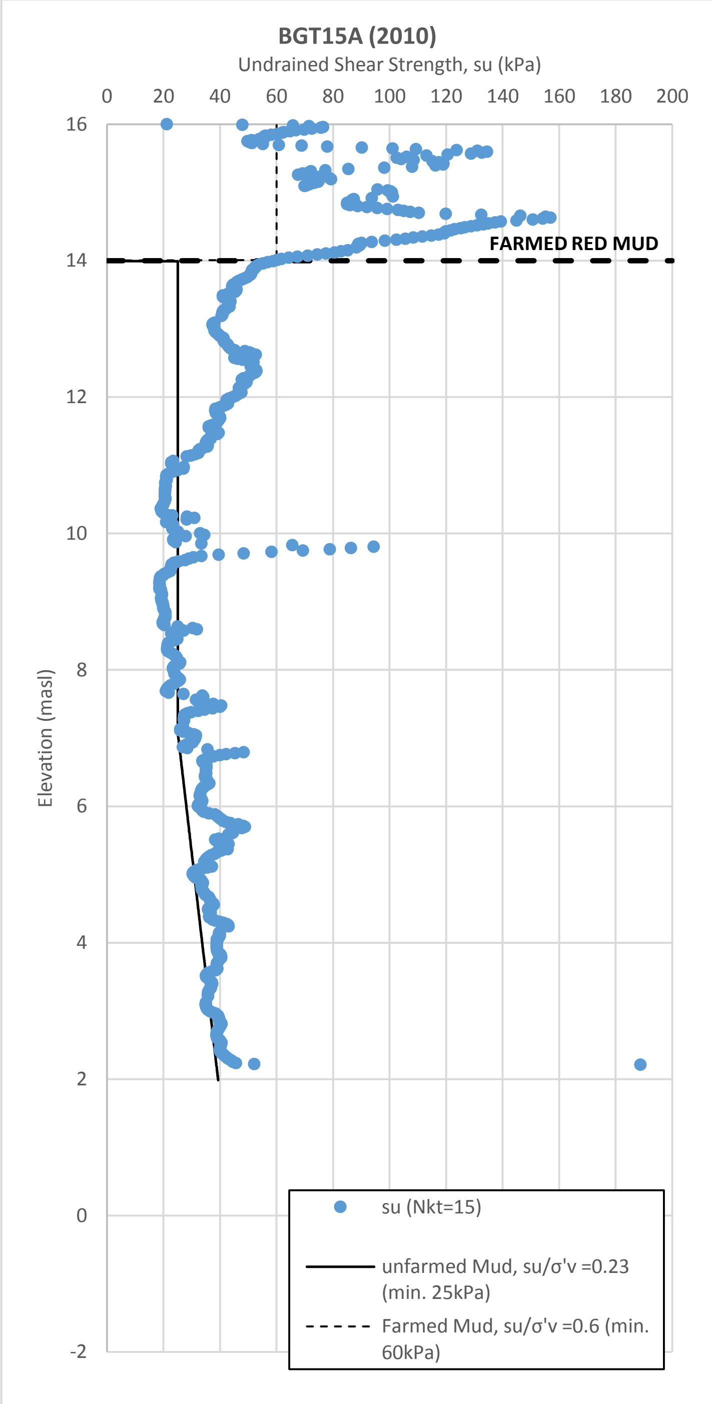
CONSULTANT
Golder Associates

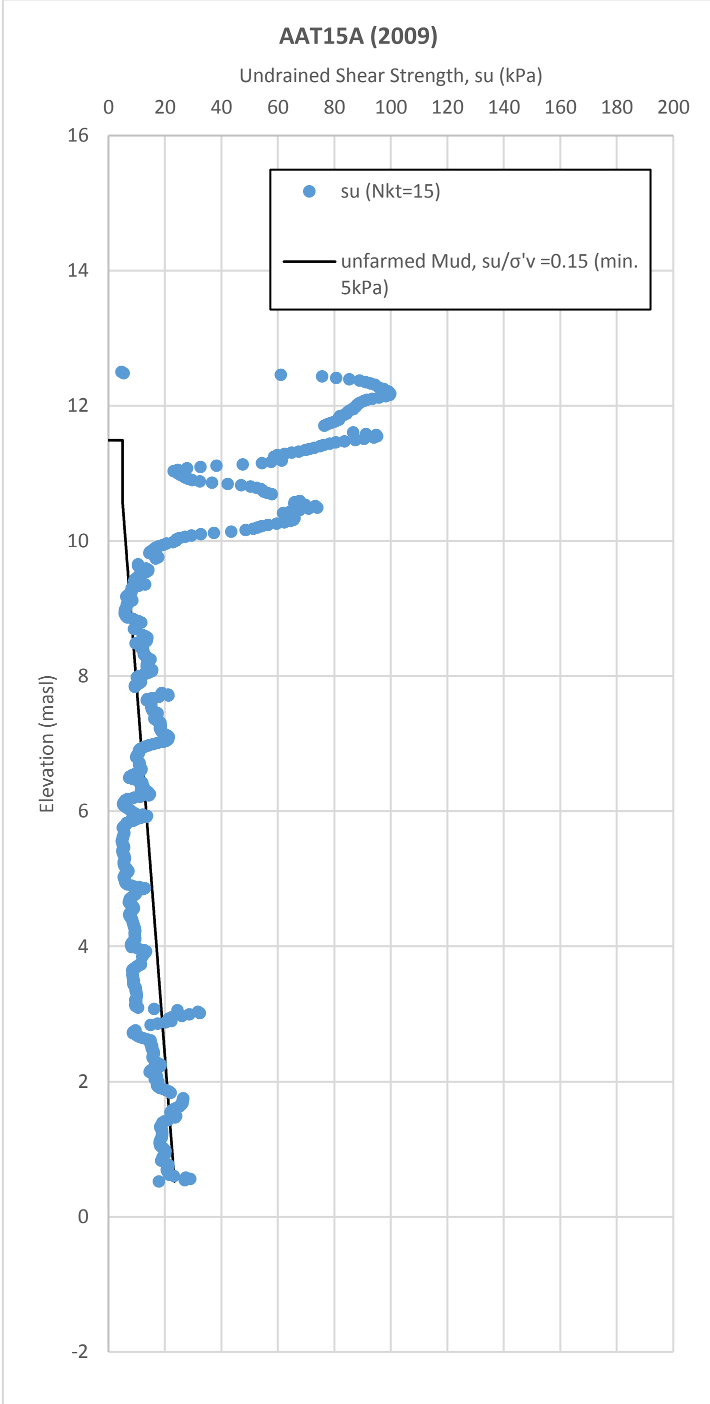
YYY/MM/DD	15/06/2017
PREPARED	GLJ
DESIGN	GLJ
REVIEW	BK
APPROVED	RW

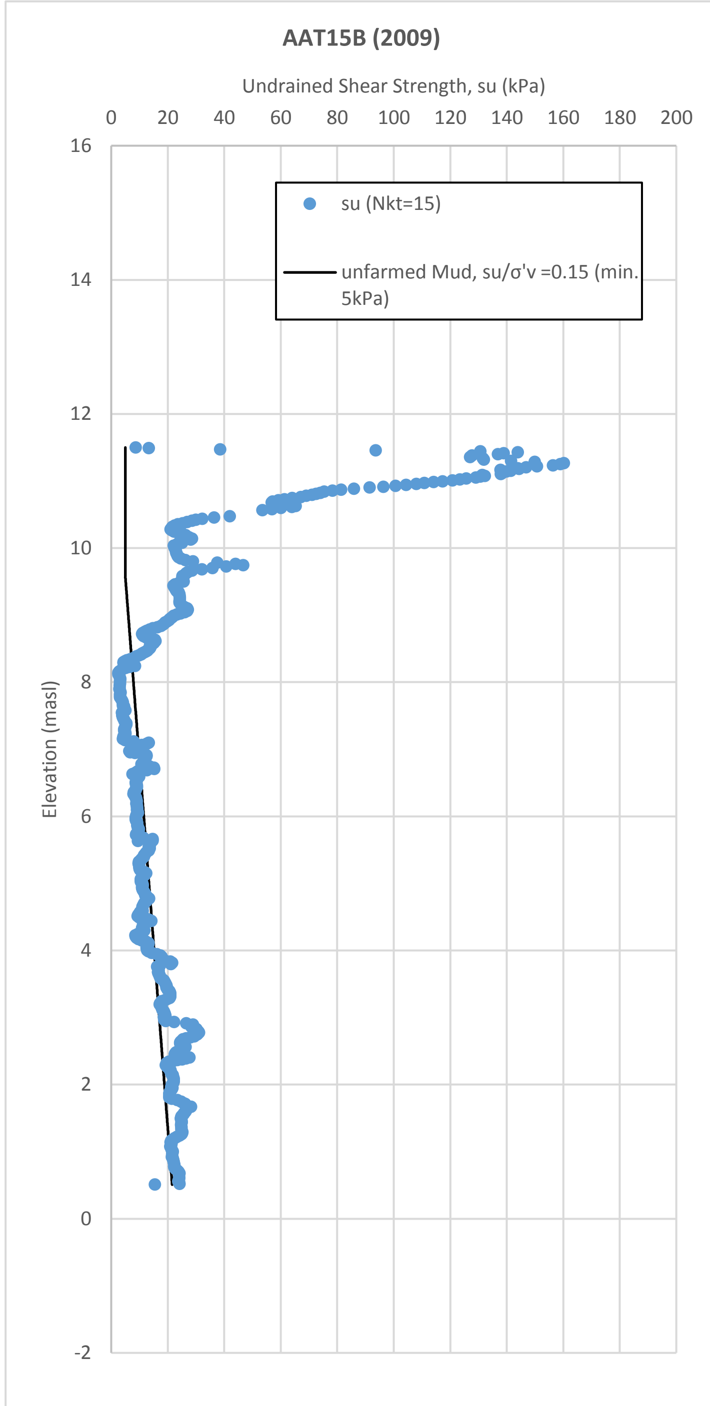
PROJECT
BRDA PHASE 1
BLAST VIBRATION ASSESSMENT

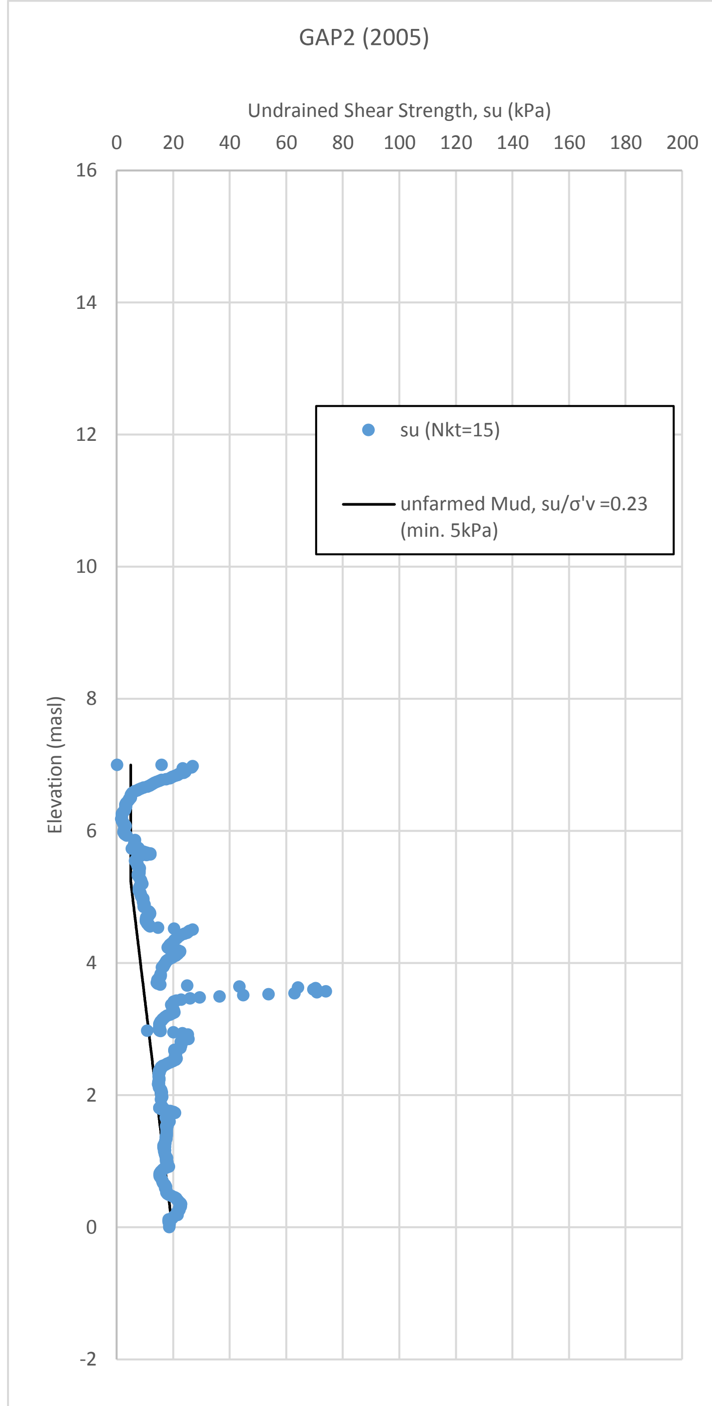
TITLE
SECTION K
UNDRAINED SHEAR STRENGTH FROM CPT DATA

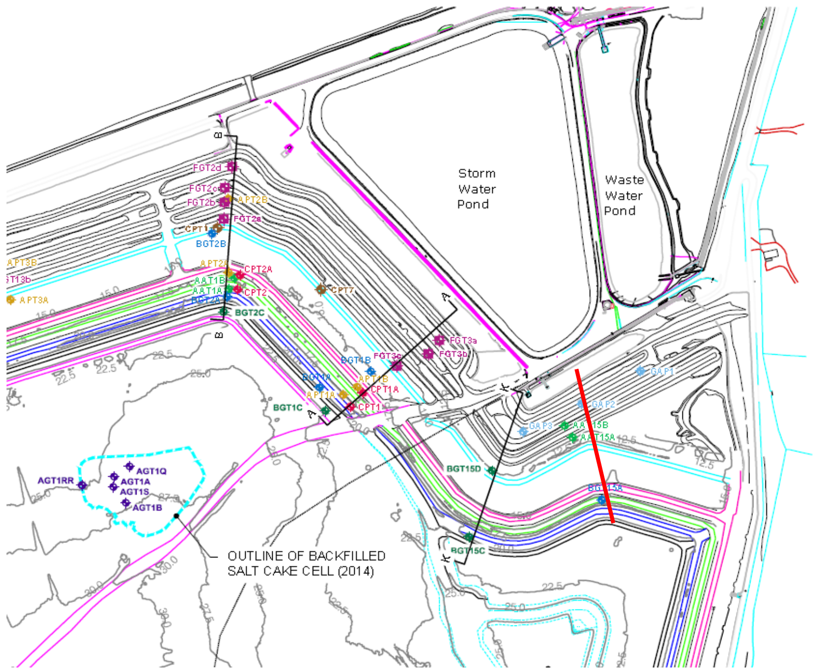
PROJECT No. 1667376 Rev. 0 FIG B1











Notes:

1. Elevations Shown are approximate
2. Undrained shear calculated according to the formula s_u = (net cone resistance)/N_{kt}, where Nkt is undrained strength factor determined to be 15

CLIENT
AUGHINISH ALUMINA LIMITED

CONSULTANT
Golder Associates

YYYY/MM/DD	15/06/2017
PREPARED	GLJ
DESIGN	GLJ
REVIEW	BK
APPROVED	RW

PROJECT
BRDA PHASE 1
BLAST VIBRATION ASSESSMENT

TITLE
EAST OF SECTION K
UNDRAINED SHEAR STRENGTH FROM CPT DATA

PROJECT No.	Rev.	FIG
1667376	0	B2

APPENDIX C

Pseudo-Static Analyses

1.0 UNDRAINED STATIC STABILITY ANALYSIS

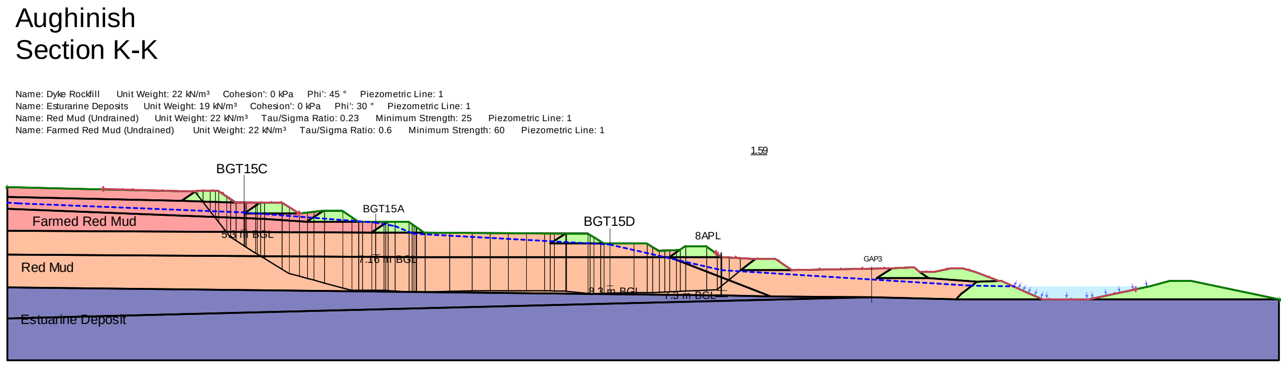


Figure C1: Undrained Static Stability Analysis, Overall slope

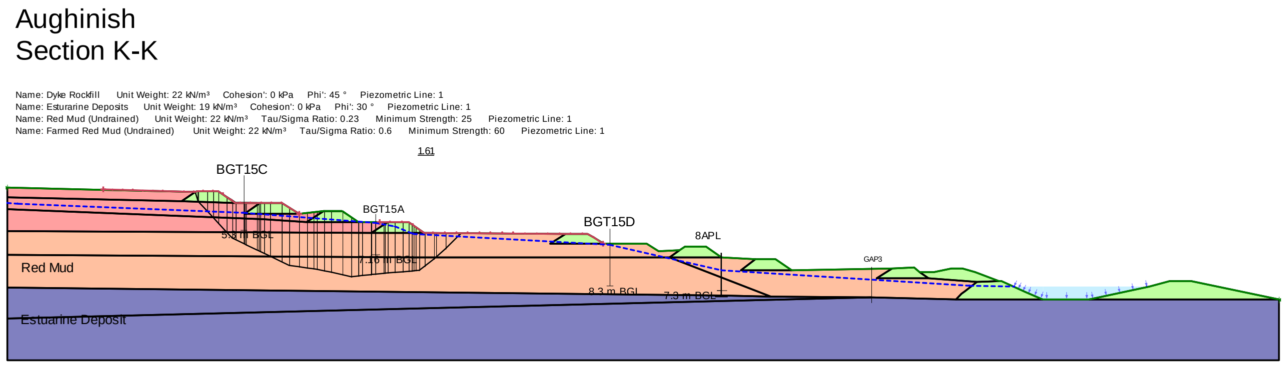


Figure C2: Undrained Static Stability Analysis, Upper slope

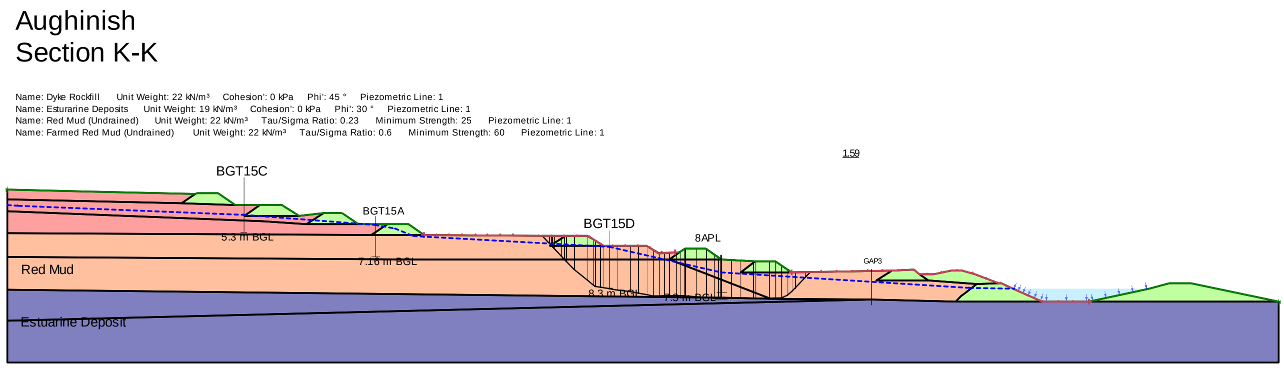


Figure C3: Undrained Static Stability Analysis, Upper slope

2.0 PSEUDO-STATIC STABILITY ANALYSIS

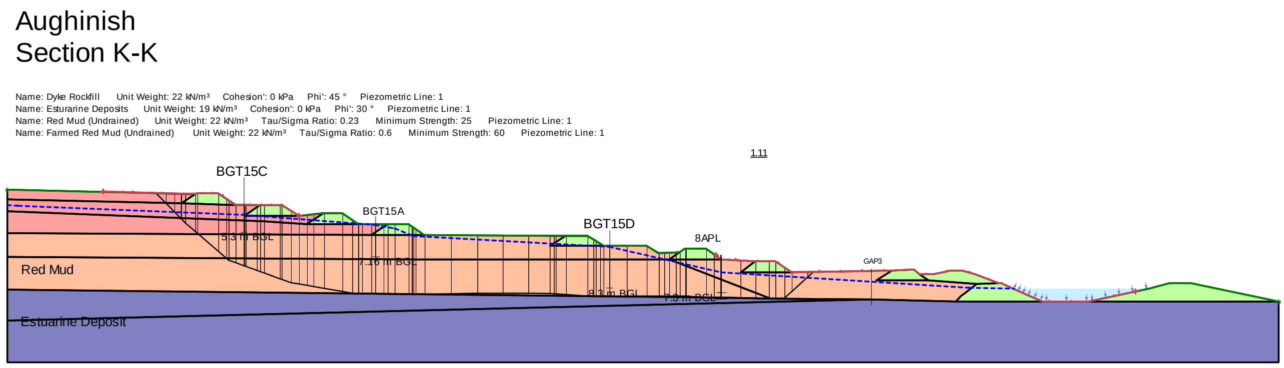


Figure C4: Undrained Pseudo-static Stability Analysis, PPV = 15 mm/s, Overall Slope

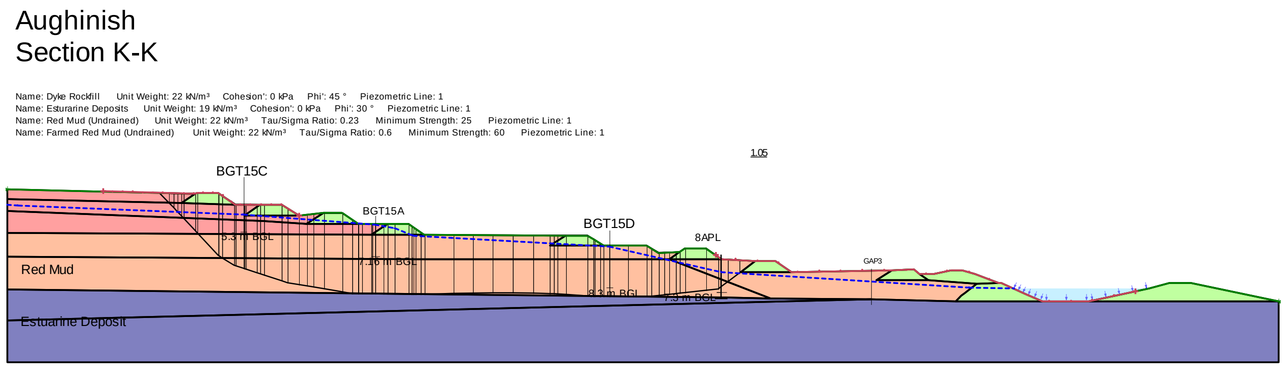


Figure C5: Undrained Pseudo-static Stability Analysis, PPV = 20 mm/s, Overall Slope

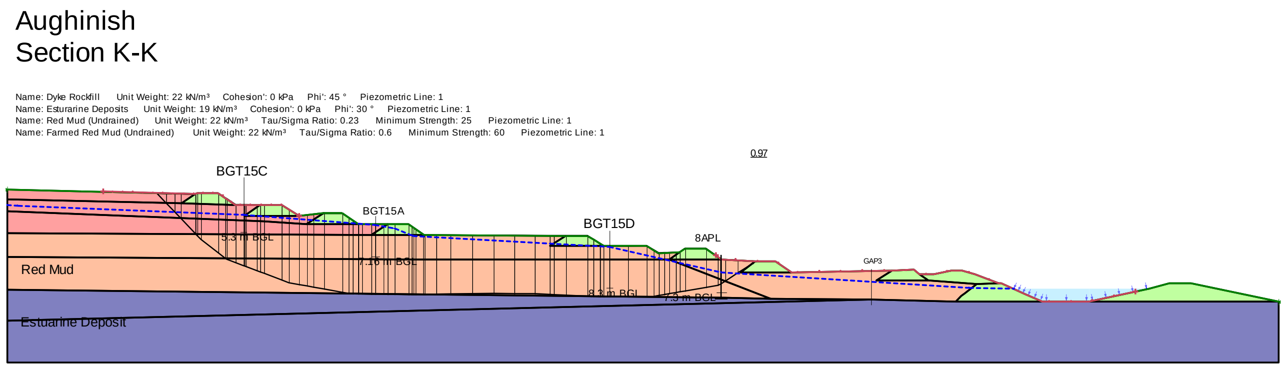


Figure C6: Undrained Pseudo-static Stability Analysis, PPV = 25 mm/s, Overall Slope

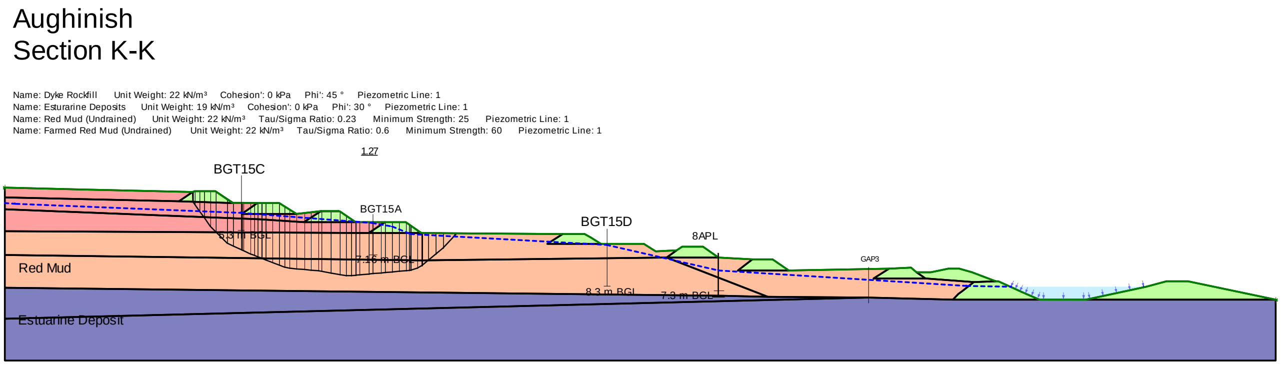


Figure C7: Undrained Pseudo-static Stability Analysis, PPV = 15 mm/s, Upper Slope

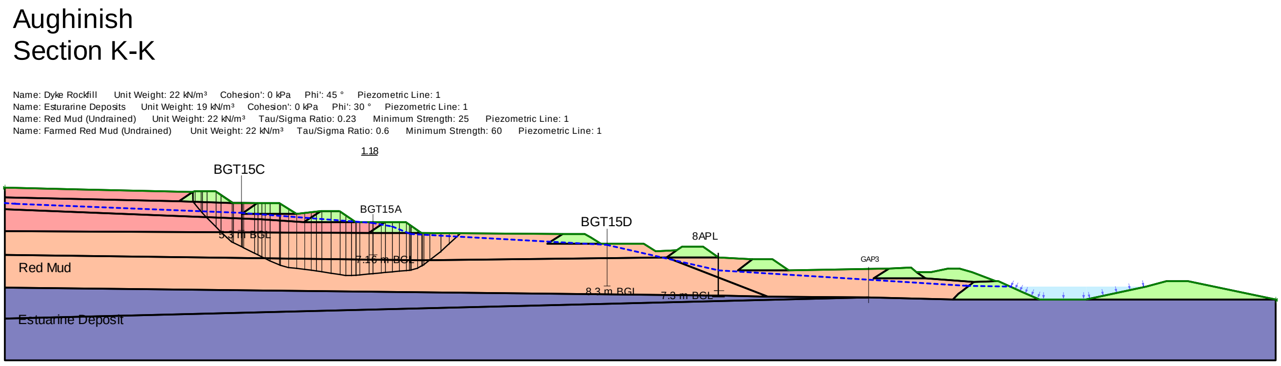


Figure C8: Undrained Pseudo-static Stability Analysis, PPV = 20 mm/s, Upper Slope

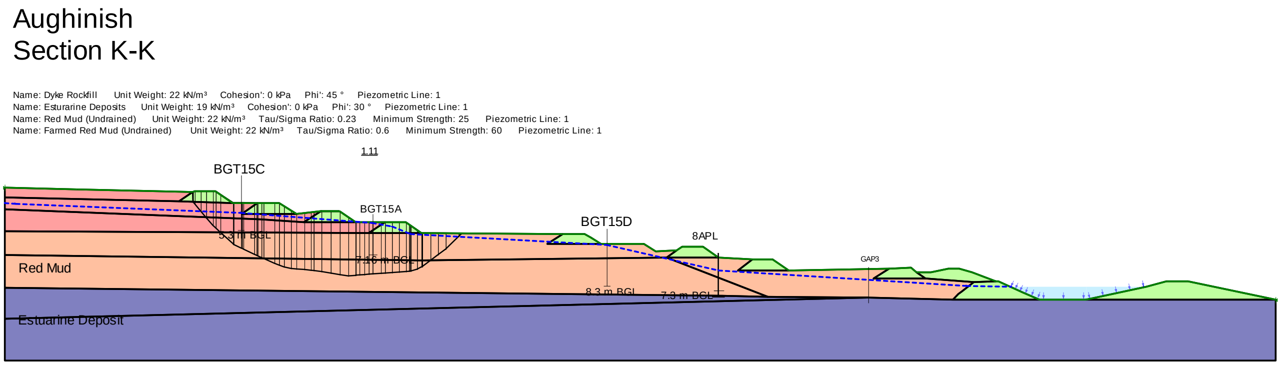


Figure C9: Undrained Pseudo-static Stability Analysis, PPV = 25 mm/s, Upper Slope

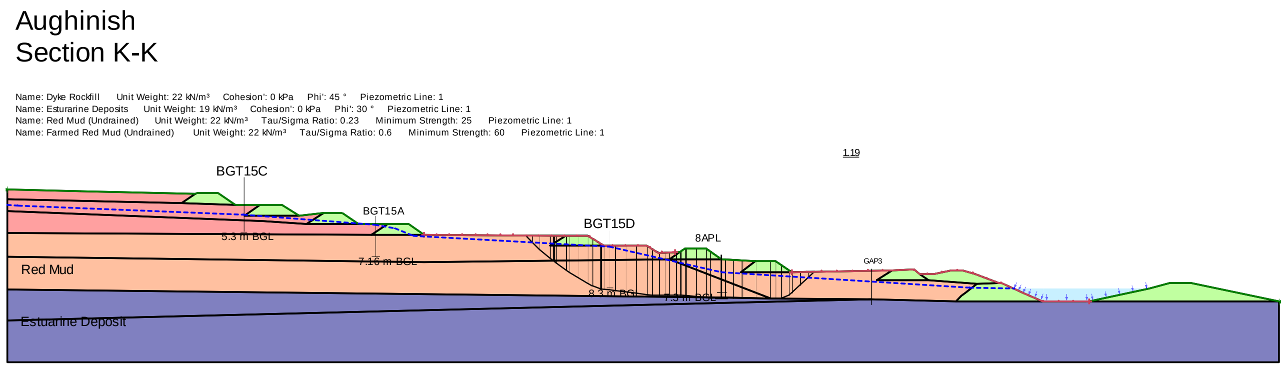


Figure C10: Undrained Pseudo-static Stability Analysis, PPV = 15 mm/s, Lower Slope

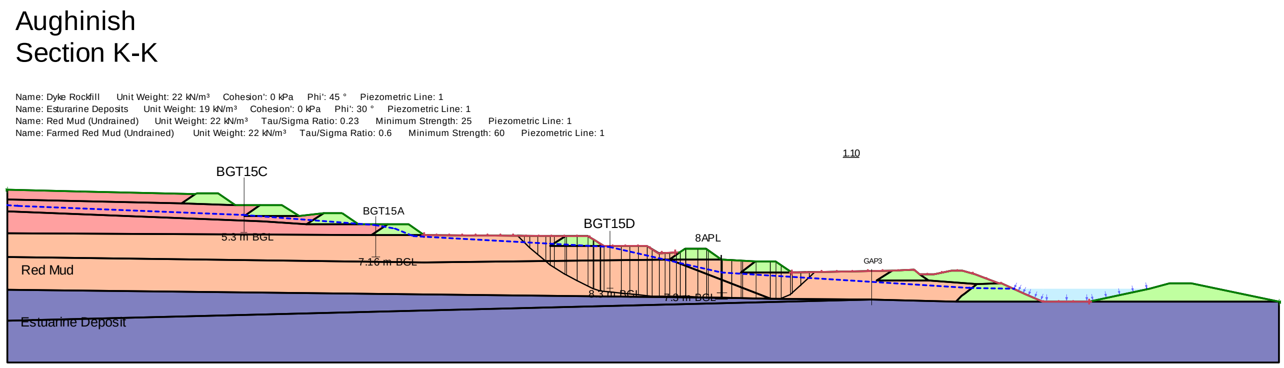


Figure C11: Undrained Pseudo-static Stability Analysis, PPV = 20 mm/s, Lower Slope

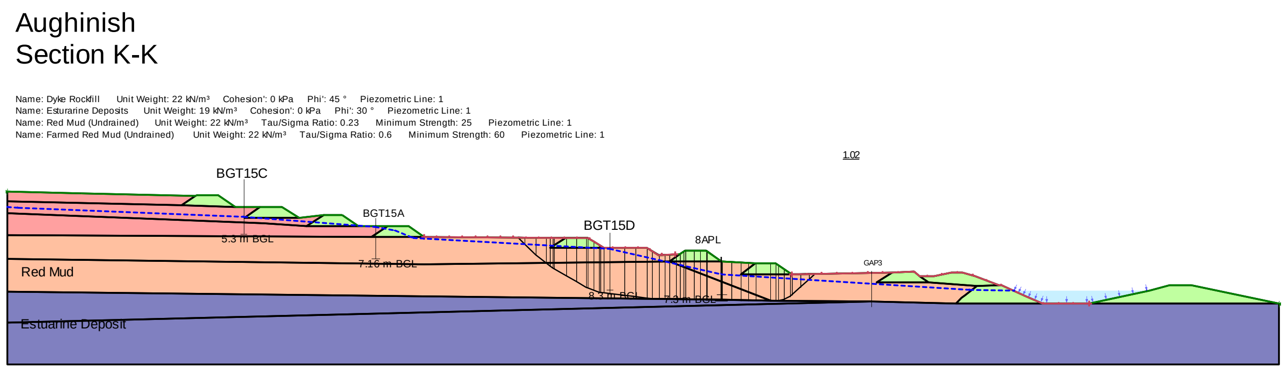


Figure C12: Undrained Pseudo-static Stability Analysis, PPV = 25 mm/s, Lower Slope

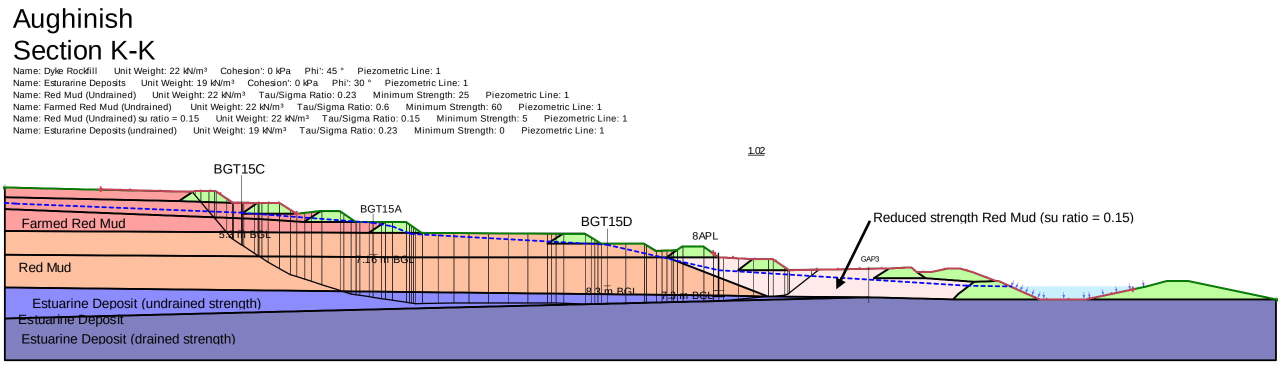


Figure C13: Undrained Pseudo-static Sensitivity Stability Analysis, PPV = 15 mm/s, Lower Slope

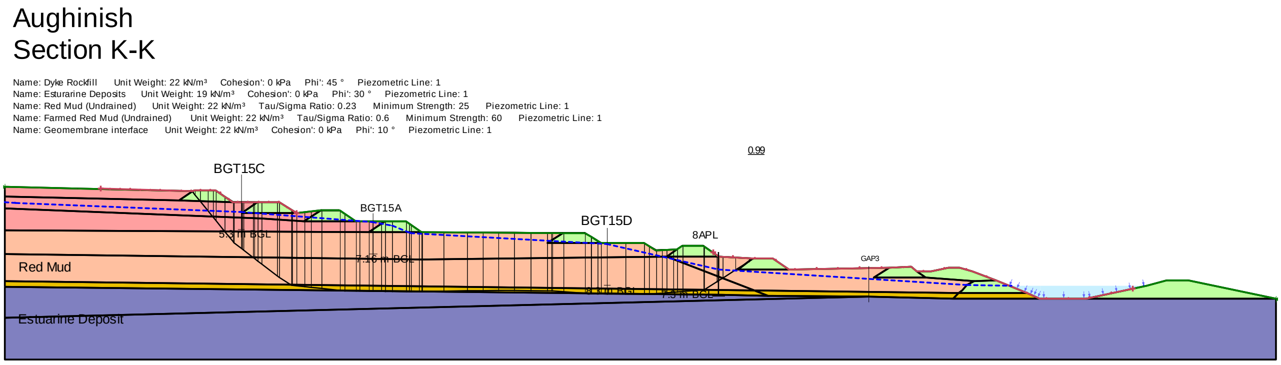


Figure C14: Pseudo-static Sensitivity Stability Analysis with reduced Geomembrane interface strength, PPV = 15 mm/s, Lower Slope

3.0 POST-BLAST (EXCESS PORE PRESSURE) STABILITY ANALYSIS

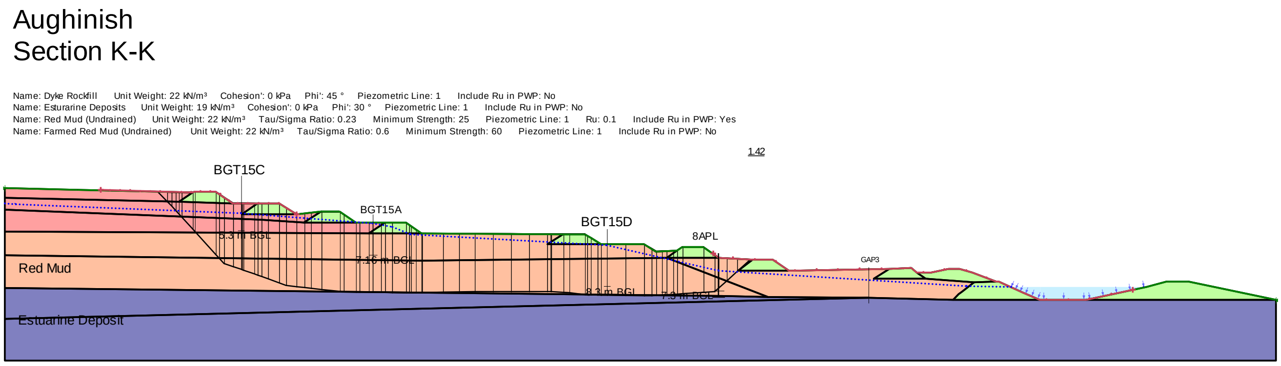


Figure C15: Undrained Post-Blast Stability Analysis, Average PPR = 0.1 (overall slope)

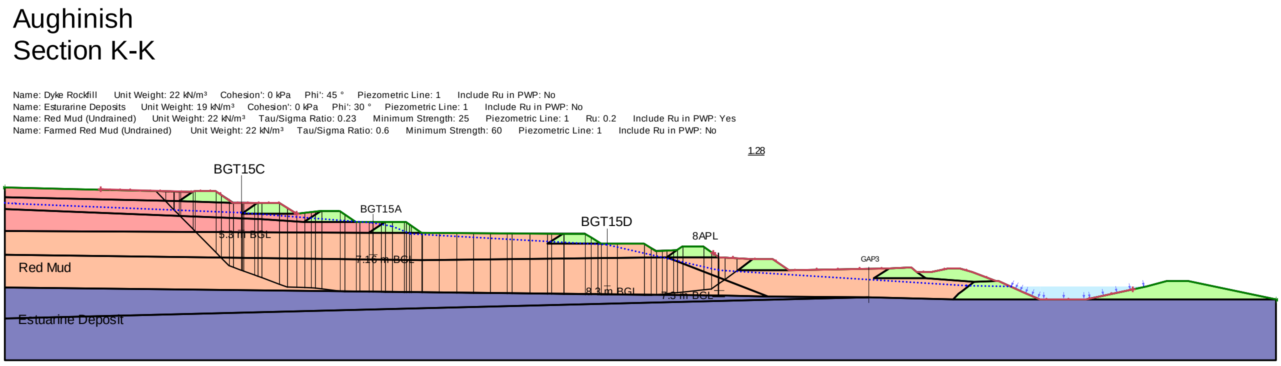


Figure C16: Undrained Post-Blast Stability Analysis, Average PPR = 0.2 (overall slope)

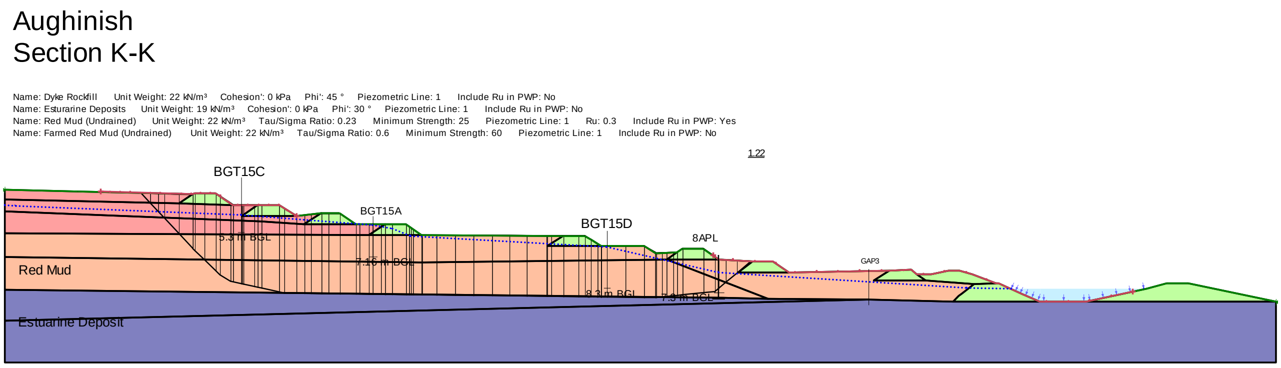


Figure C17: Undrained Post-Blast Stability Analysis, Average PPR = 0.3 (overall slope)

\\naa1-s-main01\company\projects\2016\1667376 - aughinish - borrow pit\9. working notes\brda blast vibration assessment\report\a1 version\appendix c - stability analyses\appendix c - stability.docx

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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Golder Associates

APPENDIX F

Consolidation Assessment

1.0 INTRODUCTION

Golder Associates Ireland Limited (Golder) conducted consolidation assessments for the BRDA as part of the compilation of the design report for the BRDA Raise Development.

When soil layers covering a large area are loaded vertically, as is the case with the BRDA, the compression can be assumed to be one-dimensional (1-D). Total settlement of a loaded soil is comprised of three components:

- Immediate settlement which is typically assumed to be elastic and occurs in a short time after the initial loading.
- Consolidation settlement which is a time dependent process that occurs in saturated fine-grained soils which have a low hydraulic conductivity. The rate of settlement will depend on the rate of pore water drainage to alleviate increase in pore water pressure due to the loading.
- Secondary settlement which is also a time dependent process but occurs at a constant effective stress and no change in pore water pressure.

The immediate settlement can be assumed to occur during the mud-farming process and considered a component of the deposition of the layer and is not included in the assessment. Secondary settlement can also be discounted from the assessment as it is considered to be inhibited by the thixotropic nature of the bauxite residue (see Section 3.0 below). The consolidation settlement element comprises the bulk of the total settlement expected for the foundation soils and the bauxite residue deposited in the BRDA and is assessed below.

2.0 STRATIGRAPHY OF THE BRDA

The stratigraphy of the BRDA has been established from site investigations, primarily CPTu profiles.

2.1 Phase 1 BRDA.

The foundation conditions for the Phase 1 BRDA can be summarized as:

- The estuarine deposits are present beneath the north and west flanks in varying depths from 10m to 30m. These estuarine deposits thin out and are largely absent beneath the central and east sectors.
- The eastern sector of the Phase 1 BRDA (the Phase 1 BRDA Extension) is constructed over a ridge of outcropping rock, sloping upwards from west to east, with intermittent thin layers of till material.

The north and west side-slopes of the Phase 1 BRDA constructed to Stage 10, are overlying estuarine deposits which over a period of ≈ 40 years have reached an elevation of 24 mOD. The central bulk of the Phase 1 BRDA (area encompassed by Stage 10) is predominately overlying thin and/or intermittent estuarine and till layers over bedrock. The proposed raises from Stage 11 to Stage 16 will be constructed upstream of Stage 10, hence no additional loading is placed on the north and west side-slopes overlying the deeper estuarine deposits.

During 2009, AAL started farming activities. The BRDA was approximately at Stage 6 (16 mOD) during 2009 with the central interior area being at approximately 20 mOD, representing an 16m to 20m depth of unfarmed bauxite residue deposited over a period of ≈ 27 years.

Since 2009, the perimeter has been raised to Stage 10 (24 mOD) and the central area is currently at approximately 32 mOD, representing an 8m to 12m depth of farmed bauxite residue deposited over a period of ≈ 12 years. Bauxite residue will continue to be placed to the proposed Stage 16 elevation over a duration of ≈ 18.5 years (to 2039).

2.2 Phase 1 BRDA.

The foundation conditions for the Phase 2 BRDA can be summarized as:

- The estuarine deposits are present beneath the north-western flank of the Phase 2 BRDA in depths varying from 0m to 8m. These estuarine deposits thin out progressively eastwards and are absent beneath the bulk of the Phase 2 BRDA footprint.
- The eastern sector of the Phase 2 BRDA is constructed over a ridge of outcropping rock, sloping upwards from west to east, with intermittent thin layers of till material.

The Phase 2 BRDA has been operational since 2011 and has a basin elevation varying between 1 mOD and 2 mOD. The depth of bauxite residue placed has not been mud-farmed due to the risk of damaging the geomembrane liner with the amphirol screws. Subsequently, all the bauxite residue placed has been farmed and bauxite residue will continue to be placed to the proposed Stage 16 elevation over a duration of ≈ 18.5 years (to 2039). Currently, the Phase 2 BRDA is at approx. 11.5 mOD at the perimeter and at approx. 20 mOD in the central area.

3.0 HISTORIC SETTLEMENT IN THE BRDA

Estimation of settlement in tailings facilities is difficult to determine with confidence due to the layered deposition methodology and the inherent variability in the geotechnical properties of the layers. Estimation of settlement in bauxite residue facilities is further complicated by the thixotropic nature of the bauxite residue, which leads to a build up of strength over time by forming a structure, which may be partly restricting consolidation settlement and secondary settlement.

No significant variation in density and void ratio in samples taken for unfarmed bauxite residue at various depths was noticeable from the site investigations conducted between 2004 and 2018. The void ratio of unfarmed bauxite residue typically varies between 1.00 and 1.30 and the void ratio for farmed bauxite residue typically varies between 0.90 and 1.15, but both shows little correlation with depth.

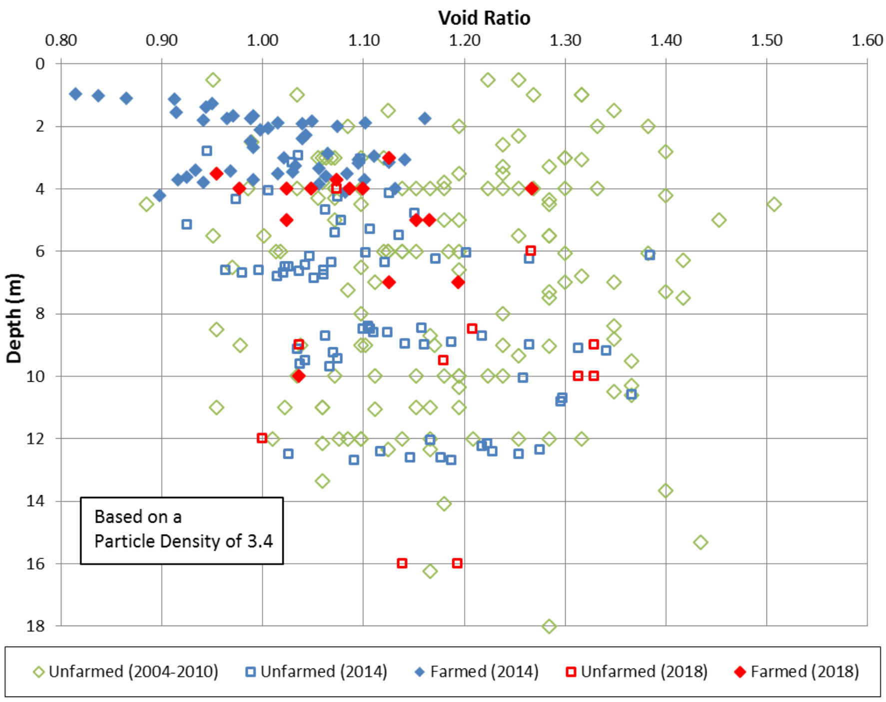

Figure 1: Bauxite Residue Void Ratio Variation from 2004 to 2018

Extensometers (spiders) have been installed in the Phase 1 BRDA, in conjunction with the inclinometer installations, during various monitoring instrument installations phases since 2007 between Stage 3 and Stage 10. These instruments are monitored quarterly, and the cumulative uppermost extensometer (typically at a depth of ≈ 4m below surface) is reported in the Annual Review (Golder 2021). The range of cumulative settlement within the bauxite residue, during the 14-year period, varies from 25mm to 317mm with an average of 101mm recorded.

No extensometers have been installed in the Phase 2 BRDA as of yet as the facility has not yet reached a sufficient elevation to warrant the installation of inclinometers.

4.0 METHODOLOGY

The standard methods of comparing void ratios before and after loading (see Figure 1) and/or comparing void ratios at varying depths to estimate future settlement are not applicable for the unfarmed and farmed bauxite residue based on the site investigation data.

The following two methods were conducted to provide an estimate for future settlement of the BRDA:

- The coefficient of volume compressibility, m_v, is used to estimate the total consolidation and the coefficient of consolidation, c_v, is used to estimate the rate of consolidation.
- The CPTu measured tip resistance, q_c, is used to estimate the total consolidation (Robertson 2008 and Pishgah et al. 2013)

4.1 Coefficient of Volume Compressibility Method

Laboratory consolidation testing has been conducted on samples of the estuarine deposits, the unfarmed bauxite residue and the farmed bauxite during various site investigation campaigns and are summarized in Table 1 below.

Table 1: 1-D Oedometer Consolidation Parameters

Material	m_v (m^2 / MN)	c_v (m^2 / year) 50% consol.	c_v (m^2 / year) 90% consol.	Over Consolidation Ratio (OCR)
Estuarine Deposits (silty CLAY)	0.045 to 0.47	11 to 30	11 to 26	2 to 3.5
Estuarine Deposits (clayey SILT)	0.025 to 0.19	2.5 to 18	2.6 to 19	4 to 5
Unfarmed Bauxite Residue	0.30 to 3.00	1 to 8	3 to 32	≈ 1 [1]
Farmed Bauxite Residue	0.020 to 0.081	33 to 96	34 to 100	3 to 7 [2]

Notes:

1. *The unfarmed bauxite residue is assumed to be normally consolidated*
2. *The farmed bauxite residue is considered artificially over consolidated as a result of the farming activities*

The coefficient of volume compressibility, m_v, is used to estimate the total consolidation and the coefficient of consolidation, c_v, is used to estimate the rate of consolidation. The estimates are based on the following assumptions:

- Based on the cv values for 90% consolidation and the timeline for deposition, the bulk of the consolidation of estuarine deposits layers beneath the BRDA can be expected to have already occurred for the current depth of bauxite residue deposited.

- Settlement is calculated at two locations for both the Phase 1 and Phase 2 BRDA: Below the crest of Stage 16 at 36 mOD and below the centre of the BRDA dome at 44 mOD. Both of these locations are considered to have little to no underlying estuarine deposits.
- For the Phase 1 BRDA, it is 12 years since the unfarmed bauxite residue layer has been placed and the layer can be expected to be well advanced in its consolidation due to the additional loading from the current depth of farmed bauxite residue layer deposited above. Hence, the lower value for m_v is selected (0.30). The higher m_v value (3.0) is considered to represent bauxite residue that is recently deposited, remains unfarmed and has only been subject to short-term loading from additional deposited layers.
- Similarly for the Phase 2 BRDA, it is 10 years since the unfarmed bauxite residue has been placed at the base of the facility and the layer can be expected to be well advanced in its consolidation due to the additional loading from the current depth of farmed bauxite residue layer above. Hence, the lower value for m_v is selected (0.30).
- The farmed bauxite residue layer can be expected to be still undergoing consolidation however, the rate of rise has been slow. Hence, the average value for m_v is selected (0.050).

4.1.1 Settlement Estimate

The settlement has been estimated using the consolidation settlement tool provided by CivilWeb (Version 01 March 2020) and is summarized in Table 2 below. The loading applied is equivalent to a 12m depth of farmed bauxite residue placed on the BRDA from Stage 10 with a perimeter elevation of 24 mOD and a central elevation of 32 mOD.

Table 2: Settlement Estimate using Coefficient of Volume Compressibility Method

Location	Density FBR (kN/m^3)	Depth of new FBR (m)	m_v UFBR (m^2 / MN)	m_v UFBR (m^2 / MN)	Settlement in UFBR (mm)	Settlement in FBR (mm)	Total Remaining Settlement (mm)
Phase 1 BRDA at Stage 16 (32 mOD)	21.5	12	0.3	0.05	680	80	740
Phase 1 BRDA at Dome crown (44 mOD	21.5	12	0.3	0.05	345	80	936
Phase 2 BRDA at Stage 16 (32 mOD)	21.5	12	0.3	0.05	90	150	240
Phase 2 BRDA at Stage 16 (32 mOD)	21.5	12	0.3	0.05	90	210	300

The highest settlement is expected for under the dome Phase 1 BRDA (936mm remaining for BRDA constructed to Stage 16) due to the underlying depth unfarmed bauxite residue layer and the greatest elevation of bauxite residue deposition. The majority of this settlement is in the unfarmed bauxite residue layer and the bulk of the settlement can be expected to be complete during the deposition life of the BRDA (to 2039) leaving a minimal (< 100mm) long-term settlement in farmed bauxite residue layer.

4.2 CPTu Measured Tip Resistance Method

CPTu soundings have been conducted at many locations within the Phase 1 and Phase 2 BRDA and the average tip resistance values recorded, q_c, are summarized in Table 3 below.

Table 1: CPTu Consolidation Parameters

Material	q_c (MPa)
Estuarine Deposits (silty CLAY)	0.9
Estuarine Deposits (clayey SILT)	2.0
Unfarmed Bauxite Residue	1.0
Farmed Bauxite Residue	3.0

4.2.1 Settlement Estimate

The settlement has been estimated using the settlement CPT tool provided by CivilWeb (Version 01 March 2020). The loading applied is equivalent to a 12m depth of farmed bauxite residue placed on the BRDA from Stage 10 with a perimeter elevation of 24 mOD and a central elevation of 32 mOD.

Table 2: Settlement Estimate using CPTu Measured Tip Resistance Method

Location (Stage 10 to -)	Density FBR (kN/m^3)	Depth of new FBR (m)	q_c UFBR (MPa)	q_c UFBR (MPa)	Settlement in UFBR (mm)	Settlement in FBR (mm)	Total Remaining Settlement (mm)
Phase 1 BRDA at Stage 16 (32 mOD)	21.5	12	1.0	3.0	340	40	380
Phase 1 BRDA at Dome crown (44 mOD	21.5	12	1.0	3.0	490	65	555
Phase 2 BRDA at Stage 16 (32 mOD)	21.5	12	1.0	3.0	190	125	315
Phase 2 BRDA at Stage 16 (32 mOD)	21.5	12	1.0	3.0	285	190	475

The highest settlement is expected for under the dome of the Phase 1 BRDA (555mm for the BRDA constructed to Stage 16) due to the underlying unfarmed bauxite residue layer and the greatest elevation of bauxite residue deposition. The majority of this settlement is in the unfarmed bauxite residue layer and the bulk of the settlement can be expected to be complete during the deposition life of the BRDA (to 2039) leaving a minimal (< 100mm) long-term settlement in farmed bauxite residue layer.

5.0 DISCUSSION

The estimate settlement for the Phase 1 BRDA for the additional loading provided by the 12m raise from Stage 10 to Stage 16 is in the range of 380mm to 740mm at the location of the perimeter of Stage 16 at elevation 36 mOD and in the range of 555mm to 936mm at the location of the centre of the dome at elevation 44 mOD.

The estimate settlement for the Phase 2 BRDA for the additional loading provided by the 12m raise from Stage 10 to Stage 16 is in the range of 240mm to 315mm at the location of the perimeter of Stage 16 at elevation 36 mOD and in the range of 300mm to 475mm at the location of the centre of the dome at elevation 44 mOD.

It is expected that the final settlements will be the lower end of these scale based on the range of cumulative settlement to date and the thixotropic nature of the bauxite residue which may be partly restricting consolidation settlement and secondary settlement.

The largest expected settlement is in the unfarmed bauxite residue layer in the Phase 1 BRDA and, based on the c_v values the bulk of the settlement, can be expected to be complete during the deposition life of the BRDA (to 2039) leaving a minimal (< 100mm) long-term settlement in farmed bauxite residue layer.

6.0 REFEREENCES

Pishgah, Pouya & Chenari, Reza. 2013, Reliability measures for consolidation settlement by means of CPT data, Geo Montreal 2013.

Robertson, P.K., 2009. CPT Interpretation – A Unified Approach, Canadian Geotechnical Journal,46: 1-19.

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APPENDIX G

Breach Analysis

EXECUTIVE SUMMARY

A risk assessment update for the Bauxite Residue Disposal Area (BRDA) constructed to Stage 10 was undertaken by Golder (Golder 2019). The assessment is considered appropriate for the BRDA constructed to Stage 16 as the BRDA footprint, the failure mechanisms and discharge pathways in a breach scenario remain unchanged. There is potential for increased volume of discharge and increased extent of discharge during a breach scenario due to the proposed increase in elevation of the BRDA to Stage 16 and these values has been reassessed.

The Phase 1 BRDA has a Very Unlikely (≈1 in 10,000) to Highly Improbable (≈1 in 100,000) annual risk of containment failure and Phase 2 BRDA has a Highly Improbable (≈1 in 100,000) to Almost Impossible (≈1 in 1,000,000) annual risk of containment failure These values are significantly less than the annual average probability of worldwide tailings dam failures based on statistical data (≈ 1 in 2,000).

The water retaining structures (SWP, LWP and PICs) have an Unlikely (≈ 1 in 1,000) to Very Unlikely (≈1 in 10,000) annual risk of water release and is similarly less likely than the average probability of tailings dam failures based on statistical data (≈ 1 in 2,000).

The impact of a breach scenario is largely dependent on the volume of material discharged and distance travelled by the material discharged. Both of these factors are dependent on the ability of the bauxite residue to liquefy. Where the bauxite residue is farmed, the material would slump rather than liquefy.

The estimated volume of bauxite residue that could potentially be released in a breach scenario has been assessed by two methods and the range is 40,000 m^3 to 90,000 m^3.

- Where the bauxite residue is farmed, the material would slump rather than liquefy. The distance travelled would be small, a distance of the order of 12.1m from the downstream toe of Phase 2 BRDA and into the Perimeter Interceptor Channel (PIC). Both the upper levels (above Stage 7) of the Phase 1 BRDA and all of Phase 2 BRDA would be expected to slump into the PIC or within ≈ 12m of the downstream toe.
- Where the material is potentially able to liquefy, which are confined to the lower slopes of Phase 1 to the Stage 6 elevation (16 mOD at perimeter to 20 mOD centrally), the distance travelled would be a maximum of 224m, although the presence of the PIC at the downstream toe may contain the flow even further, if intact. This run-out distance assumes that the farmed bauxite residue above the unfarmed bauxite residue also liquefies. If only the elevation of the unfarmed bauxite residue is considered, then the run-out distance is reduced to 52m.

The area between the Flood Tidal Defence Berm (FTDB) and the BRDA, Storm Water Pond (SWP) and Liquid Waste Pond (LWP) is at an elevation of approx. 1 mOD and has a footprint of ≈ 187,000 m^2, excluding the Bird Sanctuary, Special Protection Area (SPA) or Special Areas of Conservation (SAC) footprints and is therefore capable of retaining circa 750,000 m^3 of tailings and/or water provided that the FTDB at a crest elevation of 5 mOD remains intact.

In the event of a breach scenario resulting in bauxite residue flowing into the SWP and/or the PIC, the contaminant wastewater will be displaced and would flow via the open drainage network leading to the sluice gate valve in the Robertstown Ditch. AAL have installed a penstock valve on this sluice gate.

If the FTDB is breached due to a tidal surge, and a BRDA breach scenario occurred, the bauxite residue and containment wastewater would potentially be washed into the Robertstown and Shannon Rivers. However, the expected break-out volumes are relatively small.

1.0 INTRODUCTION

Golder Associates Ireland Limited (Golder) has conducted an update to the previous breach analysis for the Bauxite Residue Disposal Area (BRDA) constructed to Stage 10 (Golder 2019) as part of the compilation of the design report for the BRDA Raise Development, for the BRDA constructed to Stage 16.

2.0 RISK ASSESSMENT FOR THE BRDA

A risk assessment update for the Bauxite Residue Disposal Area (BRDA) constructed to Stage 10 was undertaken by Golder (Golder 2019). The assessment is considered appropriate for the BRDA constructed to Stage 16 as the BRDA footprint, the failure mechanisms and discharge pathways in a breach scenario remain unchanged. There is potential for increased volume of discharge and increased extent of discharge during a breach scenario due to the proposed increase in elevation of the BRDA to Stage 16 and these values has been reassessed (see Sections 3.1 and 3.2).

The risk assessment presented an update of the previous risk assessments and breach analysis completed in in 2006 and 2013, for the BRDA, the Storm Water Pond (SWP), the Liquid Waste Pond (LWP) and the Perimeter Interceptor Channel (PIC) at AAL. The update incorporates potential developments in the vicinity of the BRDA i.e., the Borrow Pit, the Canadian Dam Association Guidelines for tailings dams (CDA 2014), and the results of the geotechnical test work carried out on the bauxite residue since 2013. A summary of the risk assessment update is provided in the Sections below.

2.1 BRDA Classification

The classification of the BRDA and ancillary infrastructure has been undertaken in accordance with CDA 2014, which proposes target level design criteria specific for tailings dams i.e., inflow design flood, seismic event and factors of safety for static, pseudo-static and post-seismic stability.

The CDA guidelines promote a risk-informed approach to dam safety analysis and assessment as it includes deterministic standards-based analysis among many considerations. Tailings dams are classified according to the consequence in the event of failure and takes into account the incremental loss of life, environmental impact and economic impact that a failure of the dam may inflict on downstream or upstream areas, or at the dam location itself. Incremental losses are those over and above losses that might have occurred in the same natural event or condition had the facility not failed. The classification assigned to a dam is the highest rank determined among the loss categories and range from Low, Significant, High, Very High and Extreme consequence.

Golder has classified the BRDA, as a facility with a '**High**' hazard potential classification (HPC) while the SWP, the LWP and the PIC have been classified as dams having a "**Low**" HPC.

The consequence category for the BRDA is classified as '**High'** HPC primarily to account for the clean-up and restoration costs of the adjacent Special Area of Conservation (SAC) and Special Protection Area (SPA) designated lands located to the north of the SWP and LWP.

2.2 Breach Pathways and Scenarios

The initial step in a breach analysis involves the identification of potential "pathways" of the BRDA dam wall breaches that could conceivably result in release of significant volumes of material to the downstream environment. The perimeter of the BRDA has been divided into seven sectors (A through to G) and the respective pathways are illustrated on Drawing 12.

Plausible breach scenarios were then established that would lead to the loss of bauxite residue and/or bauxite residue influenced into the environment:

- **Displacement of bauxite residue influenced water in the PIC as a result of a tidal surge and/or wave event** without breaching the embankment wall and indirect displacement of the bauxite residue influenced water in the SWP and LWP based on a sea level rise to 2200.
- **Containment failure of the SWP and LWP for a tidal surge and/or wave event**. Although not a significant hazard at the current sea level, a significant return period storm event may breach the Flood Tidal Defence Berm (FTDB).
- **Containment failure of the PICs for a tidal surge and/or wave event**. Although this is not such an significant hazard at the current sea level, a significant return period storm event may breach the FTDB.
- **Slope failure of the containment walls for the SWP, LWP, and the Outer Perimeter Wall (OPW) of the PICs** under static load conditions;
- **Containment failure of the Phase 1 BRDA as a result of a significant earthquake event** causing liquefaction of the unfarmed bauxite residue; and
- **Overtopping of the LWP as a result of a significant earthquake event** causing settlement of the crest.

2.3 Breach / Failure Mechanisms

A review of the statistical tailings facility failures was conducted to identified the main breach / failure mechanisms for tailings facilities. These were referred to in development of the potential breach / failure mechanisms leading to potential breach scenarios for the BRDA, which include:

- **Earthquake Event -** leading to Slope Failure or Dynamic Liquefaction.
- **Tidal Surge or Wave Event -** leading to Erosion Induced Slope Failure. As sectors of the BRDA are located close to the River Shannon, erosion resulting from a Tidal Surge or Wave Event is also considered as a possible failure mechanism.
- **Rainfall Event -** leading to Erosion Induced Slope Failure.
- **Blast Event -** leading to Static Liquefaction induced Slope Failure or Dynamic Liquefaction. Controlled Blast Events are proposed to take place in the permitted Borrow Pit located adjacent to the north-east perimeter of the BRDA during Q2 2022.
- **Slope Instability** – as a result of either strength failure through bauxite residue or erosion of the side-slopes.
- **Static Liquefaction -** of the Unfarmed Bauxite residue **(**leading to lower or overall slope failure) or farmed bauxite residue (leading to Upper Slope Failure). Trigger Events such as Rate of Rise, Excessive Strain/Creep, Foundation Creep or a Rainfall Event are potential mechanisms that could result in static liquefaction.
- **Foundation Failure –** as a result of strength failure through the foundation soils leading to Overall Slope Failure via Static Liquefaction.
- **Overtopping Event (Discharged Bauxite residue) -** leading to erosion induced slope failure.

Having established a number of cause / consequence trees that model the potential pathways from the hazards to the target, probabilities were assigned to the cause/consequence trees, to produce fault / event trees.

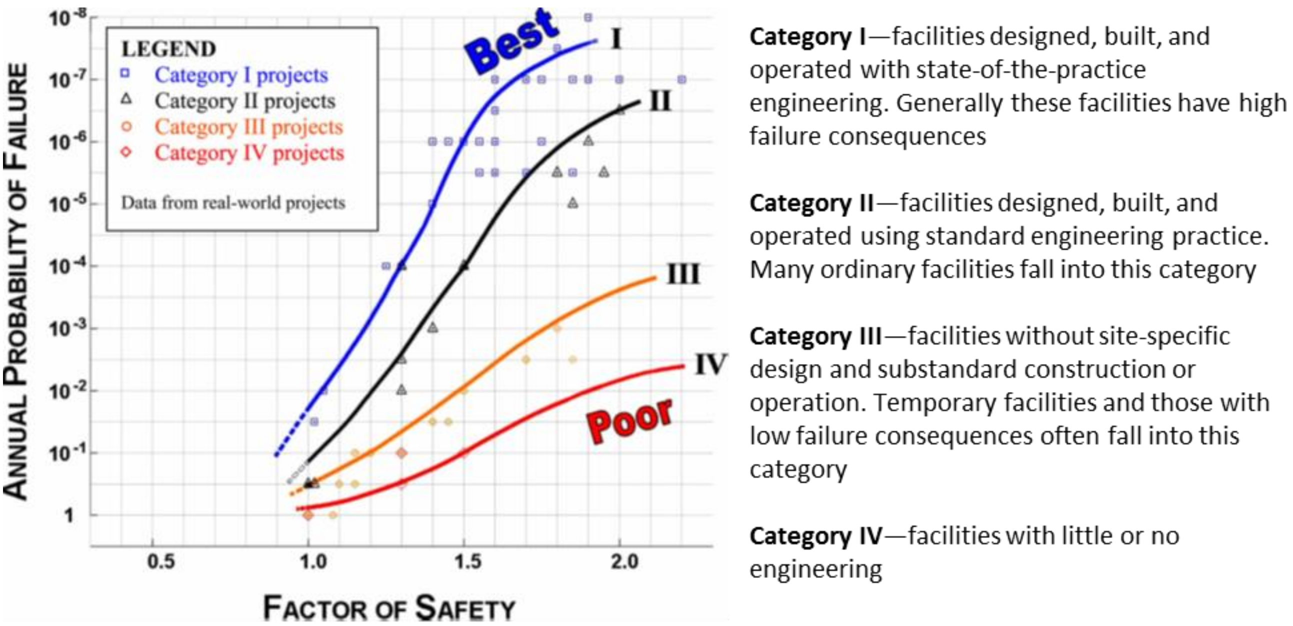
2.4 Fault / Event Trees

A semi-quantitative fault / event tree analysis was the method adopted for the risk assessment. Information for input to the fault / event tree analysis has been collated from both precedent information of previous tailings dam facility failures and from site specific or site generated data such as material strength, rainfall, earthquakes, wave heights and climatic impacts. Having established a number of cause / consequence trees that model the potential pathways from the hazards to the target, probabilities were assigned to the cause/consequence trees, to produce fault / event trees.

Fault / event tree model the system faults (or failure events) that lead to initiation of the ‘top fault’ (containment failure) leading to release of residue and/or water from the BRDA or ancillary structures. The probabilities were assigned on the basis of calculations and/or professional judgement, where appropriate.

Where stability analyses were undertaken to produce a factor of safety (drained unless otherwise indicated), the probability relationship developed by Silva et. al. (2008) has been applied for modern tailings facilities designed, built, and operated using standard engineering practice.

The AAL BRDA and ancillary structures have been considered as Category II facilities for the assessment.



The annual probability of failure for modern engineered embankment dams ranges from **1.43 E-3 to 4.26 E-4 or 1 in 700 to 1 in 2,350** which equates to a **factor of safety (FoS) range** in terms of stability, of **1.33 to 1.42,** based on the FoS – Annual Probability of Failure relationship for a Category II tailings facility.

2.5 Annual Probabilities of Failure

The probability of the release of bauxite residue was calculated for each of the failure mechanisms. These probabilities have been presented as average annual probabilities of failure. Presentation of the results in this way allows easy comparison with the worldwide statistical failure rate for dams, from which the standard of care of the overall BRDA facility can be judged.

A summary of the annual probabilities of failure for the BRDA and ancillary facilities for various mechanisms leading to containment breach or bauxite residue release is presented in Table A below. The annual probability of failure from proposed blast events was also considered.

The most significant hazards identified are colour-coded; annual probabilities of **E-04 (1 in 1,000 year) are colour-coded orange** and annual probabilities of **E-05 (1 in 10,000 year) are colour-coded yellow**.

These probabilities are interpreted against the description of probability context in Table B subsequently.

Table A: Annual Probability of Containment Breach / Bauxite Residue Release

Pathway Sector	Annual Probability of Containment Breach / Bauxite Residue Release							
	Blasting	Overtopping	Earthquake	Slope Instability	Foundation Failure	Shannon / Robertstown Tidal Surge and/or Wave Event	Combined	Probability Range Context
Phase 1 BRDA	8.93 E-06 **Sectors F & G**	N/A	5.00 E-05 **All Sectors**	1.45 E-05 **Sector B**	7.94 E-06 **Sector B**	9.60 E-07 **Sectors A & G**	**2.64 E-04**	**Unlikely to Very Unlikely**
Phase 2 BRDA	N/A	N/A	5.00 E-06 **All Sectors**	2.50 E-08 **Sectors D & E**	8.61 E-12 **Sector C**	2.40 E-09 **Sector C**	**1.01 E-05**	**Highly Improbable**
SWP	N/A	6.11 E-06	1.00 E-07	3.59 E-04	5.93 E-05	2.00 E-04	**6.24 E-04**	**Unlikely to Very Unlikely**
LWP	N/A	1.51 E-05	1.00 E-06	2.26 E-04	7.68 E-05	2.00 E-04	**5.19 E-04**	**Unlikely to Very Unlikely**
Phase 1 PIC	N/A	6.11 E-06	1.00 E-07	2.13 E-04	1.84 E-06	1.00 E-04	**3.21 E-04**	**Unlikely to Very Unlikely**
Phase 2 PIC	N/A	6.11 E-06	1.00 E-07	2.13 E-04	1.84 E-06	1.00 E-04	**3.21 E-04**	**Unlikely to Very Unlikely**

N/A = Not Applicable

Table B: Description of Probability Range Context

Annual Probability of Occurrence	Description
1E-6 (1 in 1 million)	**Almost Impossible or Negligible** (no published information on a similar case exists)
1E-5 (1 in 100,000)	**Highly Improbable** (published information exists, but in a slightly different context)
1E-4 (1 in 10,000)	**Very Unlikely** (it has happened elsewhere, but some time ago)
1E-3 (1 in 1,000)	**Unlikely** (recorded recently elsewhere)
1E-2 (1 in 100)	**Possible** (could have occurred already without intervention)
0.1 (1 in 10)	**Highly Probable** (a previous incident of a similar nature has occurred already)
0.2 – 0.5 (1 in 5 to 1 in 2)	**Uncertain** (nearly equal chance of occurring to that of not occurring)
0.5 - 0.9 (>1 in 2)	**Nearly Certain** (one or more incidents of a similar nature have occurred recently)
1 (or 0.999)	**Certain** (or as near to, as makes no significant difference)

Note: *Table extracted from Triple Bottom Line Risk Management: Enhancing Profit, Environmental Performance and Community Benefits, Adrian R. Bowden, Malcolm R. Lane, Julia H. Martin, John Wiley & Sons, 2002*

A summary of the annual probabilities of displacement of water from the SWP, the LWP and the Phase 1 & 2 BRDA PICs for the primary cause (tidal surge and/or wave event) is presented in the table below.

Table 1: Summary of Annual Probability of Displacement of Water

Facility	Event	Annual Probability of Displacement
SWP	Tidal Surge and/or Wave Event	1.96 E-04
LWP	Tidal Surge and/or Wave Event	1.96 E-04
Phase 1 PIC	Tidal Surge and/or Wave Event	9.8 E-04
Phase 2 PIC	Tidal Surge and/or Wave Event	4.9 E-04

These annual probabilities of failures are plotted in the Figure 1 below along with the range for annual probability of failure for modern engineered embankment dams (**1.43 E-3 to 4.26 E-4 or 1 in 1,700 to 1 in 2,350)** based on the data provided above for a Category II tailings facility (Silva et al., 2008).

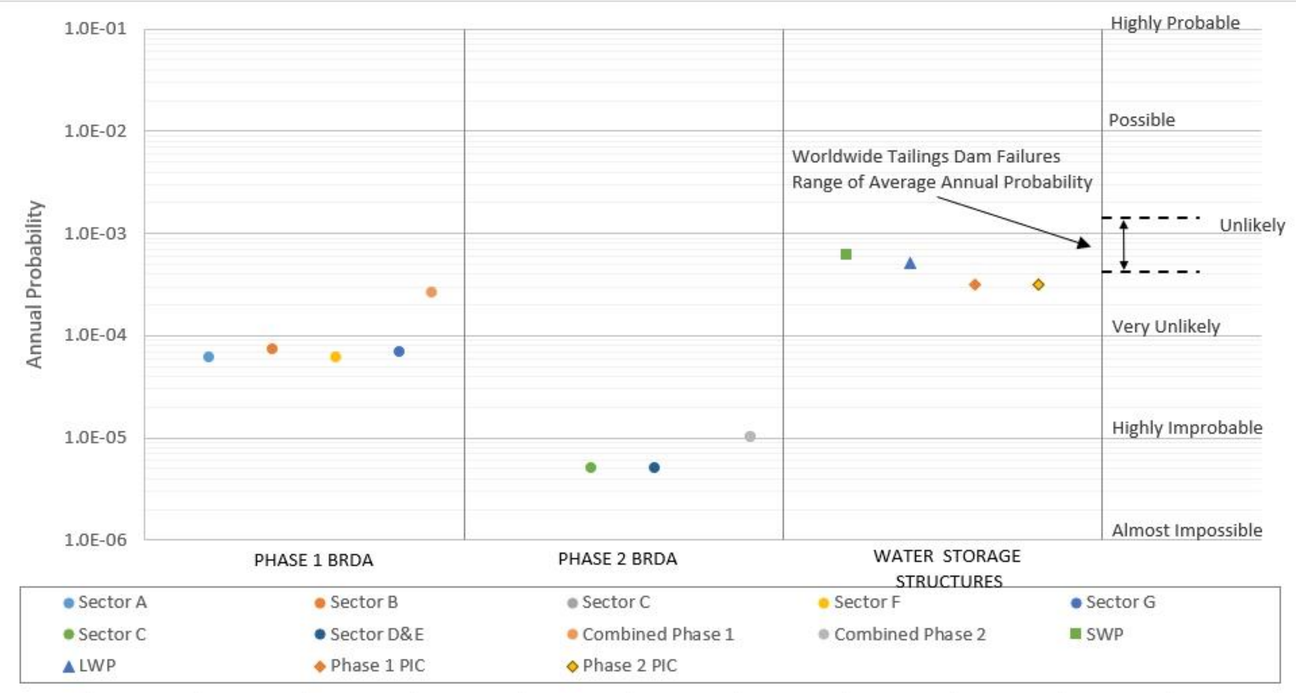


Figure 1: BRDA and Water Retaining Structures Failure Probability

2.6 Results and Interpretation

The Phase 1 BRDA has a **Highly Improbable** to **Very Unlikely** annual risk of containment failure and Phase 2 BRDA has a **Highly Improbable** to **Almost Impossible** annual risk of containment failure These values are significantly less than the annual average probability of worldwide tailings dam failures based on statistical data.

The water retaining structures (**SWP, LWP and PICs**) have **an Unlikely** to **Very Unlikely** annual risk of water release and is similarly less likely than the average probability of tailings dam failures based on statistical data.

A summary of the annual probability for each identified failure mechanism / hazard is detailed below.

2.6.1 Phase 1 BRDA

- An earthquake event has the highest annual probability of bauxite residue containment loss for the BRDA, with a **Very Unlikely** to **Highly Improbable** risk. This is due to a 1 in 10,000-year return event earthquake resulting in dynamic liquefaction of the unfarmed bauxite residue . This hazard has an order of magnitude greater likelihood of occurring than all other identified hazards. It is to be noted that the earthquake magnitude for this high return period event is an extrapolation of limited seismic historical data which was undertaken for a study focussing more on the UK than Ireland.
- Future blasting at the borrow pit has an annual probability interpreted as **Almost Impossible** to **Highly Improbable,** to lead to slope instability at Sectors F and G. The size of blast can be controlled and the response within the bauxite residue monitored, to alleviate the risk.
- Slope instability and failure from tidal surge and/or wave events have an **Almost Impossible** risk of occurrence. The slope stability includes an evaluation of failure of the bauxite residue, both within the farmed and unfarmed zones, erosion induced slope failure and undrained slope failure where a trigger event would initiate undrained conditions within the bauxite residue .
- Foundation failure also has an **Almost Impossible** risk of occurrence. The bedrock is well defined underneath the facility and the till and estuarine strengths are well characterised. The most likely slip surface would be through the weaker zone identified in the estuarine deposits, which are located primarily

along the northern section of the Phase 1 BRDA and along the western sections of the Phase 1 and Phase 2 BRDA.

- Overtopping is not considered a hazard as no water is stored on the BRDA.

2.6.2 Phase 2 BRDA

- An earthquake event has the higher probability of bauxite residue containment loss for the BRDA, with a **Highly Improbable** to **Almost Impossible**. The farmed bauxite residue has a lower annual probability of failure than the unfarmed bauxite residue and is less likely to dynamically liquefy .
- Slope instability has an **Almost Impossible** risk of occurrence due to the slip surface going predominantly through the farmed bauxite residue, and with only small volume of unfarmed bauxite residue along the base of the facility.
- Foundation instability and failure from surge event both have an **Almost Impossible** risk and are only applicable for Sector C along the Robertson River. The estuarine deposit does not extend significantly along Sectors D and E.

Slope instability of the bauxite residue is not considered sufficient in itself to cause notable loss of containment beyond the extent of the PIC. Static flow liquefaction of the bauxite residue would be required, which further reduces the risk of catastrophic containment failure. For the Phase 2 BRDA, it would require flow liquefaction of the farmed bauxite residue, which would potentially require a significantly higher return period earthquake (1 in 100,000-year return, i.e., a **Highly Improbable** event).

2.6.3 SWP, LWP and PIC

- Slope instability has the greatest risk of occurrence at **Very Unlikely**. Slope instability includes embankment failure, erosion induced slope failure (surface erosion as well as piping), and subsequent destruction following flow failure of the bauxite residue .
- A tidal surge and/or wave event which leads to erosion of the toe and subsequent slope failure has a relatively similar risk of occurrence at **Very Unlikely**. The storm event has to overtop the FTDB, which is linked to have a 1 in 1,000-year return period storm and tidal surge event.
- Foundation failure leading to slope instability has a risk of **Very Unlikely** to **Highly Improbable**. The risk of foundation failure is through the estuarine deposit.
- Overtopping has a **Highly Improbable** to **Almost Impossible** risk of occurrence. Overtopping includes during operation when AAL maintain the pond level through pumping and following closure when a spillway will be installed to limit the risk of overtopping. The design storm event required for overtopping, based on the minimum required freeboard, is a 1 in 200-year event. Earthquake induced crest settlement, leading to overtopping has also been evaluated.
- An earthquake event leading to slope instability has the lowest risk of occurrence.
- The annual probabilities of displacement of water from the SWP, LWP and PICs have been assessed. The values returned are higher for the SWP and LWP due to the greater crest elevation of 6.0 mOD (**1.96 E-04**) compared to the PICs with a crest elevation of 5.0 mOD (**4.9 E-4 to 9.8 E-4**). The Phase 2 BRDA PIC has a lower annual probability as it borders the Robertstown River along its western extent and is protected by Foynes Island.

3.0 BREACH ANALYSIS FOR BRDA CONSTRUCTED TO STAGE 16

A potential breach scenario resulting from one or a combination of the failure mechanism identified has been determined to have a **Very Unlikely** to **Negligible** probability.

The impact of a breach scenario is largely dependent on the volume of material discharged and distance travelled by the material discharged. Both of these factors are dependent on the ability of the bauxite residue to liquefy. Where the bauxite residue is farmed, the material would slump rather than liquefy.

The specified area to be affected by a potential breach scenario has been published in Appendix C of the 2019 Limerick City and County Council External Emergency Response Plan for the BRDA in accordance with S.I. 566 of Waste Management (Management of Waste from Extractive Industries) Regulations 2009.

3.1 Discharge Volume

A number of failed tailings facilities were monitored during their failures. Based on the statistical data of conventional ponded tailings storage failures, the volume of material to be discharged from a facility in the event of a dam break can be estimated as the sum of the pond water plus a percentage of the total volume of tailings stored in those ponded storage facilities (US COLD 1994). This percentage is generally estimated at about 20%. A summary graph is presented below.

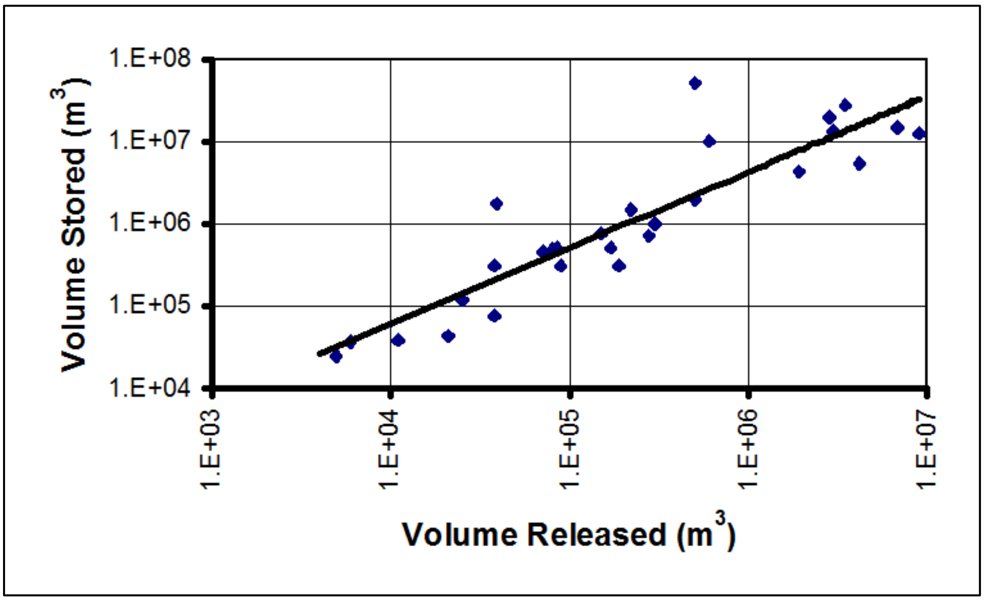


Figure 1: Estimate of Tailings Discharge Volume (extracted from Tailings Dam Incidents, US COLD 1994)

It is acknowledged that the tailings discharge volume graph in the Figure 1 above is based on conventional ponded tailings storage facilities, which contain significant amounts of surface water.

However, this is not the case for the BRDA where the bauxite residue is thickened and discharged as a paste and then farmed to produce a residue with moisture content of approx. 30% and a minimum undrained shear strength ratio of s_u/σ'_{v0} = 0.6 (farmed bauxite residue).

On the lower slopes of the Phase 1 BRDA, which have not been farmed (unfarmed), then there is a potential for the bauxite residue to liquefy as a consequence of large strains. However, this is a Very Unlikely event (Golder 2019).

As the older unfarmed bauxite residue consolidates over time and reaches a critical density beyond which the potential for the material to liquefy is significantly reduced, it can be expected that the potential volume of bauxite residue that could be released from the facility in the event of a breach would be reduced.

Several authors such as Jayapalan et al.1983, Vick 1991, Blight et al. 1981 and Wise Uranium Project (WISE 2002, 2014 and 2020) have indicated techniques to model discharge of tailings resulting from a dam breach. The latter have developed a flow model which has been used to estimate the volume of bauxite residue discharged and the distance travelled.

The estimated volume of bauxite residue that could potentially be released in a breach scenario has been assessed from the Tailings Flow Slide Calculator (WISE 2020) and from the size of the slope stability failures modelled in SLOPE/W. A summary of results from the two approaches is provided in Table 1 below.

Table 1: Estimated Bauxite Residue Release in a Very Unlikely Breach Event

Methodology	Volume of Bauxite Residue Released (m^3)
Flow Slide Calculator [1]	70,000 to 90,000
SLOPE/W Stability [2]	40,000 to 50,000

Notes:

1. *The Tailings Flow Slide Calculator produces a 2D section of the flow material to the point where it comes to rest. Only the lower stages of the Phase 1 BRDA along the northern and western sectors are considered to liquefy. The remaining areas, above Stage 6, in the Phase 1 BRDA area and all the Phase 2 BRDA area are farmed, or will be farmed, and are Very Unlikely to liquefy during dynamic conditions and Unlikely to Very Unlikely to liquify under static liquefaction.*
2. The volumes are based on the range of failures from the SLOPE/W modelling, using the failure sectional areas for the lower slope and assuming a maximum failure width equal to the slope length of the lower slope failure.

3.2 Extent of Bauxite Residue Release

In the Very Unlikely to Negligible probability of a breach, the distance travelled by the bauxite residue has been estimated using the flow model developed for the Wise Uranium Project (WISE 2020). The model does not consider frictional resistance from the flow channel (downstream of dam) and may overestimate the inundation distance, hence providing a conservative run-out value.

The model simulates flow of liquefied mill tailings using a Bingham plastic model to simulate the flow behaviour of the tailings. Bingham plastic fluids are time-independent viscous fluids for which the apparent viscosity decreases with shear rate and motion only commences when the yield stress is exceeded. The plastic viscosity of bauxite residue may vary with shear rate due its thixotropic behaviour i.e. if the shear rate is increased, shear thinning, or alignment of particles will exhibit a decrease in the plastic viscosity and greater run-out. Similarly, dilatant or shear thickening behaviour would exhibit an increase in plastic viscosity and lesser run-out.

The Phase 1 BRDA is comprised of unfarmed bauxite residue to approx. Stage 7 (18m), while the upper Stages, the dome and all of the Phase 2 BRDA will be comprised of farmed bauxite residue. Hence, the selection of Bingham property values for the unfarmed and farmed bauxite residue is critical for the analysis. Bauxite residue is discharged as a paste at a concentration of approx. 58% (assuming SG = 3.4). Following deposition, dewatering and deposition of subsequent layers, the concentration of bauxite residue increases to approx. 75%. At these solid concentrations, a failure flow would resemble that of a frictional granular flow rather than a mud flow or a slurry.

The Bingham yield strength value, or residual shear strength, is selected based on historic laboratory testing, in-house AAL laboratory testing and in-situ shear vane testing with values ranging from 2 to 13 kPa.

The Bingham plastic viscosity value is selected based on measured values for varying clay types at varying water contents (Ghezzehei and Or 2001). Unfarmed bauxite residue typically has a moisture content of ≈ 38% while farmed bauxite residue has a moisture content of ≈ 34%. These moisture contents in a clay type material would provide Bingham plastic viscosity values in the 10 to 100 kPa.s range, respectively.

The parameters inputted for the model are listed in Table 2 below.

Table 2: Model Parameters for Tailings Flow Slide Calculator (Wise-Uranium, Dec 2020 version)

Parameter	Selected Values
Geometry	
Initial height of BRDA Containment	35 above downstream elevation at Stage 16 (36 mOD)
Bed slope downstream of BRDA Containment	0 % at 1 mOD
Bauxite Residue Properties	
Unit Weight (Unfarmed and Farmed Bauxite residue)	21.5 kN/m^3
Bingham Yield Strength (Unfarmed Bauxite Residue)	4 kPa
Bingham Yield Strength (Farmed Bauxite Residue)	6 kPa
Bingham Plastic Viscosity (Unfarmed Bauxite Residue)	10 kPa.s
Bingham Plastic Viscosity (Farmed Bauxite Residue)	100 kPa.s

The distance travelled by the bauxite residue for the volume of material expected to be released by a breach scenario will be dependent on the ability of the bauxite residue to liquefy.

Where the bauxite residue is farmed, the material would slump rather than liquefy. The distance travelled would be small, a distance of the order of 12.1m from the downstream toe of Phase 2 BRDA and into the Perimeter Interceptor Channel (PIC). Both the upper levels (above Stage 7) of the Phase 1 BRDA and all of Phase 2 BRDA would be expected to slump into the PIC or within ≈ 12m of the downstream toe.

Where the material is potentially able to liquefy, which are confined to the lower slopes of Phase 1 to the Stage 6 elevation (16 mOD at perimeter to 20 mOD centrally), the distance travelled would be a maximum of 224m, although the presence of the PIC at the downstream toe may contain the flow even further, if intact. This run-out distance assumes that the farmed bauxite residue above the unfarmed bauxite residue also liquefies. If only the elevation of the unfarmed bauxite residue is considered, then the run-out distance is reduced to 52m.

The area between the Flood Tidal Defence Berm (FTDB) and the BRDA, Storm Water Pond (SWP) and Liquid Waste Pond (LWP) is at an elevation of approx. 1 mOD and has a footprint of ≈ 187,000 m^2, excluding the Bird Sanctuary, Special Protection Area (SPA) or Special Areas of Conservation (SAC) footprints (see Drawing 10 and Drawing 12), and is therefore capable of retaining circa 0.75 million m^3 of tailings and/or water provided that the FTDB at a crest elevation of 5 mOD remains intact.

In the event of a breach scenario resulting in bauxite residue flowing into the SWP and/or the PIC, the contaminant wastewater will be displaced and would flow via the open drainage network leading to the sluice gate valve and subsequently into the Robertstown River. AAL have installed a penstock valve on this sluice gate.

If the FTDB is breached due to a tidal surge and a BRDA breach scenario occurred, the bauxite residue and containment wastewater would potentially be washed into the Robertstown and Shannon Rivers. However, the expected breach volumes are relatively small.

The perimeter of the BRDA is essentially square with the southeast and northeast corners cut-off. This forms six main embankment walls termed the north, west, south, southeast, east and northeast:

- The northern wall is adjacent to the Bird Sanctuary and then beyond that the River Shannon;
- The western wall is adjacent to the Robertstown River;
- The southern wall abuts the railway line;
- The south-eastern wall is situated adjacent to the Limerick City and County Council (LCCC) Wastewater Treatment Plant (WWTP) and farmland;
- The eastern wall is situated adjacent to the Aughinish Sports ground and complex and amenity lands; and
- The north-east wall is adjacent to the SWP and LWP and then into the Bird Sanctuary.

Drawing 01 and Drawing 12 indicate the site setting of the BRDA. The River Shannon lies to the north with all tributary rivers feeding into the river. The tributaries include the Robertstown River and Poulaweala Creek.

The perimeter of the BRDA has been divided in seven sectors (A through to G) as previously discussed. The common characteristic for each sector is that the released bauxite residue, should a failure occur, will eventually follow similar pathways (Pathways A through to G, respectively).

The seven identified sectors and respective pathways are illustrated in Drawing 12, which shows the likely extent of a bauxite residue flow from the BRDA.

3.3 Breach Pathways

The pathways / sectors for a bauxite residue release in the Very Unlikely to Negligible probability of a breach are described below:

- **Pathway A** – breach of the northern embankment wall of the Phase 1 BRDA resulting in the release of liquefiable bauxite residue directly into the PIC and then into the AAL owned lands between the Outer Perimeter Wall (OPW) of the Perimeter Interceptor Channel (PIC) and the Flood Tidal Defence Berm (FTDB). Bauxite residue infilling the PIC could breach/overtop the OPW which will result in the release of alkaline water. Both bauxite residue and water would be contained by the FTDB. However, if the FTDB is breached by a tidal surge then the bauxite residue and alkaline water will enter the River Shannon. The probability of failure of this section of the embankment wall and release of bauxite residue is **6.05 E-05.** The earthquake event and the higher liquefaction risk of the unfarmed bauxite residue is controlling.
- **Pathway B** – breach of western embankment wall of the Phase 1 BRDA and release of liquefiable bauxite residue into the PIC and then into the AAL owned lands between the OPW and the FTDB. Bauxite residue infilling the PIC could breach/overtop the OPW which will result in the release of alkaline water. Both bauxite residue and water would be contained by the FTDB. The probability of failure of this section of the embankment wall and release of bauxite residue is **7.29 E-05.** The earthquake event and the higher liquefaction risk of the unfarmed bauxite residue is controlling.
- **Pathway C** – breach of the western embankment wall of the Phase 2 BRDA resulting in the slumping of bauxite residue into the PIC. Bauxite residue slumping into the PIC could breach/overtop the OPW which will result in the release of alkaline water although this would be contained by the FTDB. The FTDB is protected by Foynes Island and reduces the risk to be breached by a tidal surge. The probability of failure of this section of the embankment wall and release of bauxite residue is **5.03 E-06**. The earthquake event is controlling but the annual probability of failure is an order of magnitude less due to the farmed bauxite residue.

- **Pathway D** – breach of the south-western embankment wall of the Phase 2 BRDA resulting in the slumping of bauxite residue into the PIC. Bauxite residue slumping into the PIC could breach/overtop the OPW which will result in the release of alkaline water although this would be contained by the FTDB and the railway embankment. The probability of failure of this section of the embankment wall and release of bauxite residue is **5.02 E-06.** The earthquake event is controlling but the annual probability of failure is an order of magnitude less due to the farmed bauxite residue.

- **Pathway E** – breach of the south eastern and eastern embankment wall of the Phase 2 BRDA resulting in the slumping of bauxite residue into the PIC, which would be maintained dry along this section due to the gradient of the invert level of the channel. The probability of failure of this section of the embankment wall and release of bauxite residue is **5.02 E-06.** The earthquake event is controlling but the annual probability of failure is an order of magnitude less due to the farmed bauxite residue.

- **Pathway F** – breach of the eastern wall of the Phase 1 BRDA resulting in the slumping of bauxite residue into the PIC formed against the eastern ridge road. The PIC along this section would be dry due to the gradient of the channel. The probability of failure of this section of the embankment wall and release of bauxite residue is **6.14 E-05.** The earthquake event and the higher liquefaction risk of the unfarmed bauxite residue is controlling.

- **Pathway G** – breach of the north-eastern wall of the Phase 1 BRDA resulting in the release of liquefiable bauxite residue directly into the PIC, SWP and LWP. These structures could consequently breach/overtop, and the bauxite residue and water will spill into the AAL owned lands between the OPW and the FTDB. A breach of the OPW will also result in the release of alkaline water. Both bauxite residue and water would be contained by the FTDB. However, if the FTDB is breached by a tidal surge then the bauxite residue and alkaline water will enter the River Shannon. The probability of failure of this section of the embankment wall and release of bauxite residue is **6.90 E-05.** The earthquake event and the higher liquefaction risk of the unfarmed bauxite residue is controlling.

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APPENDIX H

Seepage and Water Quality Assessment

GOLDER
MEMBER OF WSP

REPORT

Aughinish Alumina Bauxite Residue Disposal Area

Engineering Design: Closure Water Quality Predictions

Submitted to:

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19132440.R03.A2

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Distribution List

Aughinish Alumina Ltd - one copy (pdf)

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Table of Contents

TABLES

APPENDICES

1.0 INTRODUCTION

1.1 Assignment

Golder Associates Ireland Limited ('Golder') was commissioned by Aughinish Alumina Limited ('AAL') to assess the potential water quality following closure of the bauxite residue disposal area ('BRDA'), to be constructed to Stage 16 at AAL's alumina refinery in Askeaton, Co. Limerick, Ireland.

The BRDA is comprised of multiple phases which contain:

1) Unfarmed bauxite residue (BR) on estuarine sediments for the original unlined Phase 1 BRDA, 72 ha
2) Unfarmed BR on composite liner for the Phase 1 Extension BRDA, 32 ha
3) Carbonated farmed BR on 1) and 2) since 2009
4) Carbonated farmed BR on composite liner for the Phase 2 BRDA, 80 ha

The farming process that is carried out by AAL consists of ploughing and aerating bauxite residue to reduce the pH, whereby carbon dioxide in the atmosphere reacts with the high pH hydroxide components of the bauxite residue, forming carbonates. This process is known as carbonation. The farming process also improves the geotechnical properties of the bauxite residue. Upon closure, it is proposed that the BRDA will be capped with amended bauxite residue, which comprises BR that has been mixed with neutralised process sand, gypsum, and organic material. This amended layer has been demonstrated on Site in several locations.

The requirements for the study were agreed between AAL and Golder and subsequently set out in proposal P19132440.P04.B0, dated 21 January 2021. This report has been prepared in general accordance with the proposal.

The first aspect of this study included leachate analysis of amended bauxite residue and chemical analysis of piezometer liquids (Golder Associates, 19132440.R01.A1 *Amended Bauxite Residue Leaching and Piezometer Liquid Analysis*, dated August 2021). This Analysis Report forms Appendix B of this report. The second aspect of the study entailed a closure seepage study carried out under the same terms of appointment and issued previously (Golder Associates, 19132440.R02.A1 *Closure Study – Seepage Assessment: Bauxite Residue Disposal Area to Stage 16*, dated August 2021). This Closure Seepage Study forms Appendix C of this report. This report uses the findings of the closure seepage study to estimate the mixed water quality in the Perimeter Interceptor Channel (PIC) for the period after 5 years post closure using a geochemical mixing model, whereby BRDA seepage and runoff will inform the water quality.

1.2 Site Setting

The alumina refinery is situated on Aughinish Island on the south side of the Shannon estuary, Co. Limerick. The Island is located between Askeaton and Foynes and is approximately 30 kilometres (km) west of Limerick and 15 km southwest of Shannon Airport. The Island has an area of approximately 400 hectares (ha) and is bounded by the River Shannon to the north, the Robertstown River to the west and southwest and the Poulaweala creek to the east and southeast.

The Phase 1 BRDA is located southwest of the process plant and is formed of two facilities: the original Phase 1 BRDA, which covers an area of 72 ha and the eastern Phase 1 BRDA Extension, which covers an area of 32 ha. The Phase 2 BRDA adjoins the southern extent of the Phase 1 BRDA and covers an area of 80 ha.

The BRDA is surrounded by perimeter interception channels (PICs), which collect seepage and run-off from Phase 1 and Phase 2 BRDA and convey it via pumps either to the Effluent Clarification System (ECS) or the Storm Water Pond (SWP). Both the ECS and the SWP are situated to the northeast of the Phase 1 BRDA. All treated surface water is discharged to River Shannon via discharge point W1-1 in accordance with AAL's EPA licence.

In the five-year closure period, all treated surface water will continue to be discharged to the River Shannon via the W1-1 discharge point.

Post-closure (period after the 5 years closure process is completed), the PICs will be modified to discharge surface water off-site via designated breach locations, for which new licence discharge points and conditions will be developed from licence review.

2.0 BACKGROUND

AAL has requested that Golder carry out works to update predictions for water quality and quantity of seepage from the bauxite residue, which will inform requirements for water management and treatment post-closure.

The leachate from bauxite residue is of a high pH due to the presence of sodium hydroxide (caustic soda) which is used in the alumina extraction process. The alkaline buffering capacity of high pH bauxite residue systems containing hydroxide species is very high, i.e. even very large volumes of flushing water are unlikely to reduce the pH significantly. This is due to the salinity of bauxite residue and the hydroxide complexing that takes place whereby hydroxide binds to any positive valance ions in solution (such as calcium, magnesium, potassium, iron, and aluminium). This complexing creates a store of alkalinity which is released from the residue into solution when the pore water is flushed.

Previous studies to assess long-term water quality and seepage behaviour are summarised in Section 2.1 and Section 2.2 below.

2.1 Previous Sampling

Golder Associates, 19132440.R01.A1 Aughinish Alumina – Amended Bauxite Residue Leaching and Piezometer Liquid Analysis, August 2021 (included as Appendix B of this report)

The most direct analogue for seepage from unamended bauxite residue is liquid present in the piezometers installed in this material, which has been extracted and analysed in this scope of works. In addition, leach testing in accordance with *EN 14405 Characterisation of waste – up-flow column percolation test* was carried out on the amended cap layer only, to assess the effects of the closure capping on infiltration and the rate of seepage through this layer. Metal analysis was carried out on the leachate from this test method.

Piezometer water analysis from selected piezometers on the Phase 1 BRDA, representing older unfarmed, unamended bauxite residue reveals that this piezometer liquid is highly alkaline (pH circa 12-13). Constituents of potential concern in relation to surface water and groundwater criteria in the piezometer liquid include arsenic, zinc, and sometimes nickel, lead, chromium, and copper. These metals arise from their natural occurrence in bauxite ore and these exceedances are not necessarily indicative of constituents of potential concern in the final mixed seepage water quality.

Based on alkalinity testing, the buffering capacity of these piezometer liquids is very high, with the alkalinity existing predominantly as hydroxide alkalinity.

The amended layer leachate testing demonstrates that the bauxite residue farming and amendment is successful in reducing the pH of the bauxite residue and improving the seepage water quality from the amended layer.

2.2 Seepage Modelling

Golder Associates, 19132440.R02.A1 Closure Study – Seepage Assessment: Bauxite Residue Disposal Area to Stage 16, August 2021 (included as Appendix C of this report)

In this study, a two-dimensional numerical model was constructed in SEEP/W to provide an assessment of potential seepage from the restored BRDA to Stage 16. The modelled design takes into consideration the changes in the lining system and material properties of the material deposited in the BRDA over time and the proposed vegetative restoration at site closure.

Based on the modelling results, simple calculations have been performed to scale the two-dimensional model outputs to cumulative annual flux volumes over the BRDA facility as a whole. The results of this assessment predict the following for an average year (based on 30 years of rainfall data from 01 Jan 1991 to 31 Dec 2020):

- Of the total water that accumulates in the PIC due to surface runoff and sidewall seepage, 93.7% arrives directly as surface water runoff from the dome and side slopes of the facility;
- The remaining 6.3% emanates from the facility slopes as sidewall seepage, and this is divided across four specific locations along the sidewalls, as per design – the Stage 5 bench, the Stage 10 bench and seepage directly into both the facility PICs from the Inner Perimeter Wall (IPW) and into the dome perimeter channels; and
- There is negligible seepage through the base of the facility, either in the unlined or lined phases.

Based on observations made during the modelling, it is evident that the model is sensitive to input parameters such as rooting depth (which has a strong influence on the rate and amount of water lost via transpiration), and the hydraulic properties of the rock fill stage raises / rock fill blanket which are proposed to provide the capping containment and the drainage structure for the side-slopes of the BRDA at closure.

3.0 OBJECTIVE AND SCOPE OF WORK

The objective of the mixing model was to make a preliminary assessment of water quality in the Perimeter Interceptor Channel (PIC) upon closure. Geochemical mixing calculations were performed using water quality results from BRDA piezometers and leachate from amended layer (simulating runoff).

The water quality predictions were completed using the geochemical code PHREEQC Version 3.3.7 (Parkhurst and Appelo, 1999). PHREEQC is a computer program that is used to simulate chemical reactions and transport processes in natural or contaminated water. The mixing model simulations performed with PHREEQC include aqueous speciation and saturation index modelling.

The results of the water quality model included an evaluation of minerals capable of precipitating from solution that could control concentrations of parameters of potential environmental concern.

4.0 EXISTING WATER QUALITY – PIC AND FUTURE RECEPTOR

4.1 Existing Water Quality – PIC

The refinery is currently in operation and PIC water is sampled quarterly as ‘bauxite residue stack leachate’ and reported in accordance with the conditions of the Site’s licence (Reg. No P0035-06)..

The PIC water is currently treated prior to discharge and therefore the concentrations reported in Table 1 do not reflect the composition of the treated water discharged off-Site. The PIC water has pH values ranging from 11-12 and variable concentrations of metals. As part of the Site’s Licence Review for the post-closure period, new discharge limits will be agreed with the Environmental Protection Agency.

Table 1: Existing PIC Water Quality – 2020 Quarterly Monitoring (all units in mg/L except pH, which is in pH units)

Parameter	Q1 -2020	Q2-2020	Q3-2020	Q4-2020
pH	11.8	12.2	10.9	11.9
Alkalinity	3301.00	2060.50	5802.00	5649.50
Cl	68.60	228.60	209.30	133.40
F	23.10	34.00	27.90	33.30
Soda	2.76	5980.00	4650.00	4960.00
Al	160.632	9.714	45.9168	8.209
As	244.0	69.4	0.1	11.3
Cd	<5	1.000	<0.0005	<0.5
Cr	34	95	0.0119	3.4000
Cu	26	58	0.0115	10
Fe	1077	34.8	0.1019	2.2
Pb	2	0.2	0.0002	<0.2
Mg	<1	14.3	<1.0	<1.0
Hg	<.1	<0.1	0.001	0.2
Ni	2	7.2	0.0042	2.2
Ti	<5	5	<0.005	5
Zn	37	<0.9	0.0014	13

4.2 Future Receiving Water Quality

Following the five year closure period, e.g. post closure, it is anticipated that the PIC water will be discharged to local surface waterways encompassing the BRDA, via designated breach locations, subject to licence amendment and agreement of discharge limits.

AAL carries out monthly monitoring for pH, conductivity and soda content on surface water points in the area of the BRDA including Mangan’s Lough, the OPW Channel and the Phase 2 BRDA West Robertstown Gate. The current suite of analysis (pH, conductivity, and soda content) is designed to assess potential impact from caustic soda. The pH values in these three locations ranged from 6.8-8.8 in quarterly monitoring for 2020.

In 2021, Aughinish undertook additional water quality monitoring on the receiving water quality (West Robertstown Drain Middle/OPW Channel/Mangan’s Lough), with analysis for a full suite of parameters (including major ions and metals). This baseline water quality analysis of future receiving water can be used to assess potential impacts to the receptor and to inform regulatory thresholds for discharge under future licencing, as has been done at other mining sites in Ireland.

Table 2: Baseline Water Quality Monitoring (mg/L) in surface water areas surrounding the BRDA (pH results in pH units)

Parameter	Mangans Lough	OPW	West Robertstown Drain Middle
pH	7.13	7.69	8.22
Alkalinity	163.9	287.6	584.7
Cl	121.4	669.9	1010.5
SO4	<1.39	130	186.1
Mg	7.3	53.6	65.3
Al	0.012	<0.030	0.006
As	0.001	0.001	0.014
Cd	<0.01	<0.01	<0.01
Cr	0.001	0.001	0.002
Fe	0.047	0.024	0.062
Ni	<0.010	<0.010	0.002
Pb	<0.01	<0.01	<0.01
Ti	<0.05	<0.05	<0.05
Zn	0.011	0.011	0.011
Hg	0.00009	0.00008	0.00021

5.0 MODELLING METHODOLOGY

The mixing water quality model was completed using the geochemical code PHREEQC Version 3.3.7 (Parkhurst and Appelo, 1999). PHREEQC is an equilibrium speciation and mass-transfer code developed by the United States Geological Survey (USGS). This code has the ability to simulate the pertinent geochemical processes occurring in a water body, such as mixing of multiple solutions, precipitation/dissolution of selected solids, redox reactions, evaporation, interaction with atmospheric gases, and adsorption of metals. PHREEQC was chosen because it combines thermodynamic and adsorption capabilities with the ability to conduct mixing and reaction path modelling. The code has gained widespread use and global acceptance by regulatory and technical communities in North America, Europe and further afield.

The sequence of geochemical modelling followed the steps listed below:

Determine Input Water Qualities: Piezometer liquid and amended layer leachate water qualities have been used as proxies for seepage and runoff, respectively. Water that infiltrates into the restored BRDA may be expressed as seepage at the base of the side slopes (entry to the PICs) or surface water seeps in the side slopes.

Liquid samples from piezometers in the BRDA were used as a proxy for seepage contribution. Run-off is dilute contact water running over the surface vegetation of the closed and restored BRDA. Leachate testing of the amended layer surface of the BRDA in accordance with the BS 12457-3 test (a standard European test method to characterise release of soluble constituents from waste materials) were used as a proxy for runoff. This is considered conservative, as the leachate produced in the test method is in prolonged contact with the amended layer and future runoff is likely to be more dilute. The analytical results for this sampling are included in Appendix A.

Using a conservative approach, concentrations at less than detection limit were assumed to equal the analytical detection limits in model simulations.

Calculate Mixing Ratios: Mixing ratios were derived from seepage modelling (Golder 2021), which assessed the anticipated inputs to the water in the PIC post closure. One mixing ratio was calculated for input into PHREEQC based on the following mix percentages: 94% runoff and 6% seepage. The 2:1 and the 8:1 concentration from the amended layer leachate were used in two different models as a sensitivity analysis.

The four piezometer liquid samples were used in equal proportion as a component of seepage, therefore each liquid comprised 1.5% of the mix model for a total of 6% seepage.

Mix and Calculate Water Composition: The input volumes were mixed in the appropriate proportion using a mass balance calculation. The resultant water composition was determined including metals speciation, redox (Eh) and pH, alkalinity, and saturation indices for mineral phases.

Evaluate Geochemically-Credible Solubility Controls: Supersaturated mineral phases were identified and evaluated for their likelihood to precipitate from the mixed solutions. Geochemically credible mineral phases were included in this evaluation based on considerations related to precipitation kinetics and observational evidence at mining/industrial sites, existing literature (i.e. Nordstrom, 2009), and best professional judgement. Geochemically credible mineral phases that were considered in PHREEQC are presented in Table 3.

Equilibrate with Selected Solid Phases / Adsorbents and Recalculate Solution: After equilibration with the selected solid phases, the solution composition was reassessed. If ferrihydrite [$Fe(OH)_3$] was identified as being supersaturated, adsorption onto this phase was simulated. The number of available adsorption sites was calculated assuming 0.005 strong bonding sites and 0.2 weak bonding sites per mole of ferrihydrite. A specific surface area of 600 m^2/g was assumed. All three values are default values developed by Dzombak and Morel (1990).

Table 3: Geochemical mineral phases considered in the PHREEQC model

Mineral Phase	Chemical Formula	Mineral Phase	Chemical Formula
Amorphous aluminium hydroxide	$Al(OH)_3$*(am)*	Azurite	$Cu_3(CO_3)_2(OH)_2$
Amorphous Silica	SiO2 (am)	Ferrihydrite	$Fe(OH)_3$
Witherite	$BaCO_3$	Siderite	$FeCO_3$
Barite	$BaSO_4$	Melanterite	$FeSO_4$ $7H_2O$

Mineral Phase	Chemical Formula	Mineral Phase	Chemical Formula
Calcite	$CaCO_3$	Cobalt(II)Carbonate	$CoCO_3$
Fluorite	CaF_2	Magnesite	$MgCO_3$
Gypsum	$CaSO_4:2H_2O$	Rhodochrosite	$MnCO_3$
Otavite	$CdCO_3$	Manganite	MnOOH
Goslarite	$ZnSO_4$ $7H_2O$	Birnessite	MnO_2
Smithsonite	$ZnCO_3$	Nickel Hydroxide	$Ni(OH)_2$
Copper Hydroxide	$Cu(OH)_2$	Cerussite	$PbCO_3$
Malachite	$Cu_2(OH)_2CO_3$	Amorphous zinc hydroxide	$Zn(OH)_2$(am)
Zinc Carbonate	$ZnCO_3:1H_2O$		

6.0 MODEL ASSUMPTIONS AND UNCERTAINTY

Care was taken to incorporate known processes that occur in surface water bodies, as understood during model development. However, in natural systems and complex man-made systems, observed conditions, particularly on a daily basis, will almost certainly vary with respect to estimated conditions. Water quality modelling requires the use of many assumptions due to the uncertainty related to determining the physical and geochemical characteristics of a complex system.

Given all of the inherent uncertainties, the results of the water quality model should be used as a tool to aid in the design of monitoring programs and closure planning, to develop best practice strategies and to outline potential risks rather than to indicate absolute concentrations.

The major assumptions used in the geochemical modelling are listed below:

1) **The water chemistries used in the modelling are representative of their respective input sources.** This assumption is a *sine qua non*; without this assumption, it is not possible to proceed with mixing modelling. Input water qualities were derived from piezometer liquid and amended layer leachate testing. Data were selected to generate input water qualities based on best professional judgement. As a conservative approach, measured water quality parameters that were less than the analytical detection limit have been assumed to be equal to the detection limit for modelling purposes.

2) **There will be complete mixing of mass from the water sources in the Perimeter Interceptor Channel.** The geochemical model assumes the water in the PIC is completely mixed, and density stratification will not occur.

3) **Water in the PIC is in full thermodynamic equilibrium.** The equilibrium assumption is the standard computational basis of PHREEQC. On a detailed scale, such equilibrium is unlikely to be the case for all chemical components throughout the entire water body. In particular, redox equilibrium is not commonly achieved in large bodies of water, even when exposed to the atmosphere. Moreover, gas exchange with the atmosphere (dissolution or de-gassing of oxygen and carbon dioxide) is a slow process that may take hours or days to reach equilibrium.

4) **The PHREEQC model appropriately simulates chemical reactions and contains the appropriate thermodynamic constants.** This is an assumption common to virtually all geochemical modelling efforts. The PHREEQC code and Minteq.V4 thermodynamic database used in this modelling exercise are widely accepted to simulate mining and industrial waters and have shown to provide reliable results in a wide range of settings.

5) **Seasonal Trends and Ephemeral Events.** The current modelling effort evaluates water quality based on single water samples from monitoring and leachate testing, and average seepage ratios. Upper and lower concentrations, which may occur in response to temporal changes in the physical and chemical system, are therefore not specifically captured.

The data and approach used to estimate future water quality are commensurate with industry best practices (INAP 2009) and are believed to provide a reasonable approximation of the system, as currently understood, within the context of the assumptions used in the model.

7.0 QUALITY CONTROL

PHREEQC was used to calculate charge balance errors for the input water qualities. The charge balance error serves as an indication of the quality of the input data. Anion and cation sums, when expressed as milliequivalents per litre, should balance because all waters are electrically neutral. The test is based on the percentage difference defined as follows:

$$\%\,\boldsymbol{difference} = \mathbf{100}\,\frac{\sum \boldsymbol{cations} - \sum \boldsymbol{anions}}{\sum \boldsymbol{cations} + \sum \boldsymbol{anions}}$$

Concentrations reported below detection limits were taken at the detection limit. A charge balance error less than 5% is generally accepted as indicative of a good analysis (Hounslow, 1995), however Nordstrom et al. (2009) considers imbalances up to 20% to be adequate.

Charge balance errors for the input solutions ranged from -14% to 30%. Of the input water qualities that were used for water quality predictions, one sample (piezometer liquid BGT3B) had charge imbalances greater than 20%. It was still included in the assessment to have additional data on piezometer liquid from the unfarmed Phase 1 BRDA. The data is considered useable for its intended purpose (i.e. to provide an indication of potential water quality in the Perimeter Interceptor Channel at closure and identify contaminants of potential concern).

8.0 INPUT DATA

8.1 Piezometer Liquid Quality Data – Seepage

Analytical results from piezometer liquid samples were used as a proxy for the contribution from seepage to the geochemical mixing model.

The piezometer details are summarised in Table 4 below.

Table 4: Monitoring Data Points - Piezometers

Monitoring Point	Monitoring Location	Comments
BGT3B	Phase I BRDA	Unfarmed bauxite residue
BGT3D	Phase I BRDA	Unfarmed bauxite residue
P17A	Phase 2 BRDA	Farmed bauxite residue
P20A	Phase 2 BRDA	Farmed bauxite residue

Analytical results from the four piezometer liquid samples taken in January 2021 are presented in Table 5. The liquid is characterised by alkaline pH (circa 11-12), which is comparable to pH values in existing PIC water samples (see Section 4.1). Results can be variable between piezometers and typically do not show significant difference between the Phase 1 and Phase 2 BRDA piezometers, except with regards to zinc. Zinc concentrations in the Phase 1 BRDA piezometers are orders of magnitude higher than the Phase 2 BRDA piezometers.

Table 5: January 2021 Piezometer Liquid Samples (mg/l)

Parameter	BGT3B	BGT3D	P17A	P20A
pH	12.21	12.92	12.56	11.73
Alkalinity	965	12,256	4,023	1,279
Cl	14.5	122	73.4	46.6
F	8.7	67.6	16.8	4
Sulphate (SO_4)	67.3	1117	186.4	89
Ca	0.7	0.8	1.7	0.9
Mg	0.1	0.1	0.1	0.1
K	9.5	63.2	22.7	7.1
Na	663.4	5719.1	1772.6	557.2
Si	7.598	2.203	2.699	2.237
Ag	0.005	0.005	0.005	0.005
Al	50.282	564.269	229.372	57.23
As	0.0455	1.5155	0.4985	0.0716
B	0.049	0.538	0.222	0.063
Ba	0.003	0.003	0.003	0.003
Be	0.0005	0.0005	0.0005	0.0005
Cd	0.0005	0.0005	0.0005	0.0005
Co	0.002	0.002	0.002	0.002
Cr	0.0048	0.0048	0.0063	0.0015
Cu	0.01	0.09	0.069	0.017
Fe	0.051	0.066	0.06	0.126
Li	0.005	0.005	0.005	0.005
Mn	0.002	0.002	0.002	0.002
Mo	0.056	2.386	1.136	0.207
Ni	0.002	0.009	0.005	0.002

Parameter	BGT3B	BGT3D	P17A	P20A
Pb	0.019	0.005	0.005	0.005
Sb	0.002	0.002	0.002	0.002
Se	0.021	0.824	0.105	0.025
Sn	0.005	0.025	0.025	0.025
Sr	0.005	0.005	0.005	0.005
Ti	0.005	0.025	0.005	0.005
U	0.005	0.005	0.005	0.005
V	1.1774	14.0734	7.2434	1.4511
Zn	3.24	62.063	0.563	0.011
Hg	0.001	0.001	0.001	0.001

8.2 Amended Layer Leachate – Runoff

A 1 kg sample of amended layer material was collected from Stage 5 and subjected to BS 12457-3 two-stage batch leach testing. The BS 12457-3 test is a standard European test method to characterise release of soluble constituents from waste materials. This test method reacts granular solid material at liquid:solid ratios of 2l:1kg and 8l:1kg with combined reporting at a 10l:1kg ratio. The combined 10l:1kg leachate concentrations have been used in this report as a proxy of the runoff contribution to the PIC as they are considered the most reasonable analogue to dilute contact water. This test method as a proxy for runoff is considered conservative, as in actuality a large proportion of the runoff would not be in direct extended contact with the amended layer but rather travel across the surface of the vegetation and may more likely resemble rainwater. Table 6 below presents the concentrations of the 2:1, 8:1 and 10:1 leachate.

Table 6: Concentrations of liquid:solid (L:S) ratios of 2:1 and 8:1 (mg/L) and combined 10:1 (mg/L)

Parameter	L:S 2:1	L:S 8:1	L:S 10:1
pH	7.07	6.8	
Alkalinity	180	38	61.5
Cl	8	0.3	1.3
F	0.4	0.3	0.3
SO4	16.2	0.7	3.4
Ca	28.8	10.4	13.4
Mg	2.5	0.4	0.7
K	3.3	0.2	0.7
Na	31.1	2.8	7.5
Si	3.6	0.9	1.3
Ag	0.005	0.005	0.005
Al	0.46	0.75	0.7
As	0.0038	0.0026	0.0028

Parameter	L:S 2:1	L:S 8:1	L:S 10:1
B	0.039	<0.012	<0.012
Ba	0.003	0.003	0.003
Be	<0.0005	<0.0005	<0.0005
Cd	<0.0005	<0.0005	<0.0005
Co	0.002	<0.002	<0.002
Cr	0.0128	0.0022	0.004
Cu	0.017	<0.007	<0.007
Fe	1.4	0.27	0.46
Li	0.039	0.012	0.016
Mn	<0.002	<0.002	<0.002
Mo	0.006	<0.002	<0.002
Ni	0.003	<0.002	<0.002
Pb	<0.005	<0.005	<0.005
Sb	0.004	0.002	0.002
Se	<0.003	<0.003	<0.003
Sn	<0.005	<0.005	<0.005
Sr	0.073	0.016	0.025
Ti	0.129	0.025	0.042
U	<0.01	<0.01	<0.01
V	0.2422	0.1191	0.1395
Zn	0.007	<0.003	<0.003
Hg	<0.0006	<0.0006	<0.0006

9.0 MIXING MODEL RESULTS

9.1 Water Quality Predictions

The results of the water quality predictions mixing runoff (amended layer leachate) and seepage (piezometer liquid water quality) are summarized in Table 6, including the immediate mixed water quality comprised of runoff and seepage (Step 1, first column) and following equilibration with atmospheric carbon dioxide and oxygen and precipitation of saturated species (Step 2, second column).

The Step 1 pH values exhibit elevated pH. After equilibration and precipitation, the Step 2 pH values are below pH 9 (8.8). The pH value decreases due to the dissolution of carbon dioxide in the highly alkaline mixed water (which transforms into carbonic acid). Pilot wetland trials of alkaline water at AAL demonstrate that it is possible to enhance equilibration of the mixed water with carbon dioxide by routing the seepage and runoff through an engineered wetland with sufficient surface area and hydraulic residence time. These pilot trials achieve a pH reduction from 10.3 to circa pH 7. This has been discussed further in Section 10.0.

Table 7: Summary of Results of Mixing Modelling, results in mg/L

	94% runoff, 6% seepage	
Parameter	**Step 1 Mixing**	**Step 2 After equilibration**
pH	10.59	8.83
Alkalinity	350	300
Cl	5.27	5.27
SO_4	26.24	26.24
Al	14.89	10.79
Ag	0.0050	0.0050
As	0.036	0.035
B	0.025	0.025
Ba	0.003	0.003
Be	0.0005	0.00003
Ca	12.62	2.50
Cd	0.0005	0.0005
Co	0.002	0.0019
Cu	0.0095	0.0054
Cr	0.004	0.004
F	1.53	1.53
Fe	0.44	0.0 (fully precipitated)
K	2.28	2.28
Li	0.015	0.015
Mg	0.66	0.66
Mn	0.002	0.0 (fully precipitated)
Mo	0.06	0.06

	94% runoff, 6% seepage	
Parameter	Step 1 Mixing	Step 2 After equilibration
Na	144.65	144.65
Ni	0.002	0.002
Pb	0.005	0.0023
Sb	0.002	0.002
Se	0.018	0.018
Si	1.45	1.45
Sn	0.006	0.006
Sr	0.024	0.024
U	0.01	0.01
V	0.51	0.49
Zn	1.04	0.31
Hg	0.0006	0.0006

9.2 Sensitivity Analysis

Additional modelling runs have been carried out to assess the mixed water quality at variable quantities of seepage. Uncertainties relating to the proportion of seepage and runoff may arise from a number of aspects outlined in the seepage modelling report, including a conservative worst-case scenario adopted for the study which assumes a near-ground level phreatic surface providing high driving heads for sidewall seepage rates. This worst-case assumption may therefore maximise the proportion of seepage estimated and impact on the relative proportions of runoff and seepage which enters the PICs. Model runs at lower quantities of seepage (2% and 4%) have been assessed. Alternatively, a higher seepage proportion of 10% was also assessed to see if this would drive higher pH following the Step 2 equilibration step. Therefore, sensitivity mix models in PHREEQC were carried out for the following proportions: 10% seepage, 4% seepage, and 2% seepage.

The sensitivity analysis results for key parameters (pH, arsenic, copper, zinc) are included in Table 8, below. For all proportions, from 2% seepage to 10% seepage, the Step 1 mix exhibits elevated pH discharge with respect to pH and the Step 2 equilibrated solution is slightly less than pH 9. From sensitivity analysis, it is clear that even a small proportion of highly alkaline seepage is not easily diluted by a dilute runoff solution (represented here by 10:1 leachate from the amended layer). Based on this, it is expected that the main control on pH discharge will be the equilibration and precipitation processes which happen passively but require some time to occur.

Table 8: Sensitivity Analysis Modelling (Variable Percentages of Seepage), Results in mg/L

	10% Seepage		4% Seepage		2% Seepage	
Parameter	Step 1 Mixing	Step 2 After equilibration	Step 1 Mixing	Step 2 After equilibration	Step 1 Mixing	Step 2 After equilibration
pH	11.09	8.98	10.16	8.68	9.51	8.52
As	0.024	0.023	0.024	0.023	0.013	0.012
Cu	0.0110	0.0072	0.0086	0.0045	0.0078	0.0030
Zn	1.65	0.28	0.66	0.36	0.33	0.27

9.3 Saturation Index Calculations and Equilibration with Selected Mineral Phases

The potential for mineral precipitation was assessed by the PHREEQC model, according to the saturation index (SI) calculated according to the following equation:

$$SI = \log \frac{IAP}{K_{sp}}$$

The saturation index is the logarithm of the ratio of the ion activity product (IAP) for a given mineral and the solubility product (K_{sp}). An SI greater than zero indicates that the water is supersaturated with respect to a particular mineral phase, and precipitation of this mineral may occur. An evaluation of precipitation kinetics is then required to evaluate the likelihood that a supersaturated mineral will actually form. An SI less than zero denotes undersaturation and indicates that the mineral in question will have a general propensity to dissolve rather than precipitate. To account for uncertainties associated with analytical results and thermodynamic data, for the purpose of this evaluation, phases reporting SI values between 1 and -1 are considered to be in equilibrium with the solution.

The results of the geochemical modelling indicate that mixed solutions are in equilibrium or supersaturated with respect to a few secondary minerals, which could potentially precipitate and create a crust along the PIC.

Iron hydroxides (such as ferrihydrite) were oversaturated in the mixed water solutions; precipitation of ferrihydrite decreased iron concentrations in the mixed solutions.

Metal concentrations (for example, lead and manganese) decrease as a function of sorption to the surface of iron oxhydroxide minerals.

Solutions were also supersaturated with respect to some carbonate phases (i.e. calcite).

10.0 PROPOSED WETLAND TREATMENT

Constructed wetland trials have been conducted at AAL as a potential means of treating the alkaline leachates post closure. The use of such wetlands as passive treatment systems has been successfully demonstrated in acidic mine water treatment as well as highly alkaline leachate from steel slag wastes (Mayes et al. 2008, Mayes et al., 2009), with potential for extending their use for the treatment of alkaline leachates associated with bauxite residues. The pH of the alkaline leachates can be decreased by carbonation with carbon dioxide released by microbial respiration, and production of organic acids within the constructed wetland systems (O'Connor & Courtney, 2020). Two research studies have demonstrated the potential effectiveness for a passive constructed wetland system at the Aughinish BDRA site (Higgins et al., 2017; O'Connor & Courtney, 2020).

Higgins et al. (2017) conducted a study across a one-year period (August 2013-August 2014), receiving plant liquor through a constructed wetland system at AAL. A horizontal surface flow constructed wetland cell of 44 m^2 along with a programmable logic controller (PLC) leachate mixing system were applied, following an initial six-month acclimatisation of the wetland vegetation using freshwater only. The PLC system was used to apply leachate (mixed with tap water) at a known pH and flow rate to the wetland system from an inflow, flowing across the network of locally sourced wetland plants and soil to an outflow. Residence time within the wetland system was calculated with tracer tests to be 4-6 days. The monthly inflow of mixed leachate was shown to decrease from a mean pH of 10.3, to a mean of pH 8.1 in the treated outflow leachate. In addition to a reduction in the pH, concentrations of dissolved trace elements were also consistently reduced when compared with the inflow. Aluminium concentrations reduced by 97%, arsenic by 84%, and vanadium by 86% during the study period. Major elements (Ca, Mg, Na) were also reduced but on a smaller order (31-53%). Calcium and sodium are noted to be removed from the system through carbonation to form carbonate fractions within the soils. Some variability is noted across the results, attributed to varying input residue chemistries and increased plant growth and microbial activity during the summer months.

O'Connor and Courtney (2020) completed a follow-up study to Higgins' experimental site at AAL. Rather than treatment with tap water in the PLC mixing system, which led to the formation of calcite crusts, de-ionised water was used to dilute the leachate. The leachate was added to the wetland at a set pH of c. 11, and flow rate of 10-30 L/h in winter and 45-55 L/h in summer, across a 52-month period (May 2015-August 2019). The inflow leachate pH (mean 11.23) reduced to a mean of 7.21 across 48-months of operation. Trace elements were only measured across the final fourth year of operation but also showed a decrease in concentrations for aluminium and vanadium. Mean aluminium concentrations decreased from 17,256 µg/L in the inflow, to a mean of 330 µg/L in the outflow, whilst vanadium decreased from a mean of 140 µg/L in the inflow, to a mean of 13 µg/L in the outflow. Notably, calcium concentrations increased between the inflow and outflow in this study, with the dominant inflow cation as sodium.

When combined, both studies (Higgins et al., 2017; O'Connor & Courtney, 2020) demonstrate the wetland provides an effective treatment for reducing pH and trace element concentrations in outflow leachates across a sustained five-year period within the constructed wetland system at Aughinish (O'Connor & Courtney, 2020). The systems may be variable according to inflow leachate chemistry in addition to weather or seasonal factors affecting the wetland hydraulic regime but show a sustained and effective decrease in pH and trace metals in bauxite residue leachates (O'Connor & Courtney, 2020).

Wetland performance data provided by AAL up to March 2021 indicates that the system continues to perform effectively, with inflow pH ranging from 10-12 being reduced to circa pH 7.

11.0 SUMMARY

In the post-closure conditions for the BRDA, the water quality in the perimeter interceptor channel (PIC) is expected to be comprised predominantly of runoff (dilute contact water over the surface of the BRDA) and a minor amount of seepage (highly alkaline liquid held in the pore space of the bauxite residue, expressed slowly as seepage due to overlying pressure). Conceptual water quality predictions were performed to evaluate mixed water quality arising from average proportions of BRDA runoff and seepage in the PIC in closure. The results of the water quality predictions will be used to inform the closure and restoration plan for the BRDA.

Piezometer liquid analysis and amended layer leachate results were used as input water qualities and a single mixing model was performed assuming a ratio of 94% runoff and 6% seepage. The water quality of the resulting mixed solution was evaluated for two situations: (1) immediately after mixing and (2) after equilibration with atmospheric carbon dioxide and oxygen, precipitation of pertinent secondary mineral phases and sorption of trace metals onto precipitated iron hydroxides.

Immediately after mixing (step 1), the resulting water had pH of 9.9 to 11. The equilibration/precipitation/sorption simulations (step 2) resulted in a decrease in overall concentrations of metals and pH. Mineral precipitates capable of forming in solution were allowed to precipitate, and metals were allowed to sorb to the surface of iron oxide minerals. The mixed water quality was circa pH 8.8 due to dissolution of atmospheric carbon dioxide, below pH 9.

Sensitivity analyses of seepage proportions from 2 – 10% show similar results, with an alkaline pH upon immediate mixing and a decrease in pH upon equilibration. Based on sensitivity analysis, equilibration and precipitation processes will have an important role in attenuation of discharge. The immediate mixed water quality is of a high pH, and following equilibration, pH values decrease and metal concentrations reduce. PHREEQC modelling indicates that the solutions are in equilibrium or supersaturated with respect to oxyhydroxide minerals, which could potentially precipitate and create a crust along the PIC channel. These minerals include iron hydroxides (such as ferrihydrite), zinc carbonate, calcite and magnesite. Precipitation of these minerals could control dissolved concentrations of iron, aluminium, calcium, zinc and manganese. Metal concentrations (for example, lead) could decrease as a function of sorption to the surface of iron oxhydroxide minerals.

These modelling results should be considered in the ongoing closure design of the PIC, which anticipates that the PIC will be converted to a wetland for passive water treatment and that water can be held in the PIC or additional water management infrastructure (e.g. basins, engineered wetlands) to confirm water quality prior to discharge into the receiving waters. Wetland trials at AAL demonstrate that passive treatment using wetlands is effective at reducing pH and metal concentrations.

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Signature Page

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10 August 2021

MBD/TR/BK/ab

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APPENDIX A

Analytical Results - January 2021 Sampling

element

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Golder Associates Ltd
Town Centre House
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Attention :	Martha Buckwalter-Davis
Date :	10th August 2021
Your reference :	19132440
Our reference :	Test Report 21/1325 Batch 1
Location :	AAL
Date samples received :	2nd February, 2021
Status :	Final report
Issue :	3

Five samples were received for analysis on 2nd February, 2021 of which five were scheduled for analysis. Please find attached our Test Report which should be read with notes at the end of the report and should include all sections if reproduced. Interpretations and opinions are outside the scope of any accreditation, and all results relate only to samples supplied.
All analysis is carried out on as received samples and reported on a dry weight basis unless stated otherwise. Results are not surrogate corrected.

Authorised By:

S. Gomery

Simon Gomery BSc
Project Manager

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis
EMT Job No: 21/1325

Report : **Liquid**

Liquids/products: V=40ml vial, G=glass bottle, P=plastic bottle
$H=H_2SO_4$, Z=ZnAc, N=NaOH, $HN=HNO_3$

Please see attached notes for all abbreviations and acronyms

EMT Sample No.	1-9	10-18	19-27	28-36									
Sample ID	B6T3B	B6T3D	P17A	P20A									
Depth													
COC No / misc													
Containers	V HN	V HN	V HN	V HN									
Sample Date	28/01/2021	28/01/2021	28/01/2021	28/01/2021									
Sample Type	Liquid	Liquid	Liquid	Liquid									
Batch Number	1	1	1	1							LOD/LOR	Units	Method No.
Date of Receipt	02/02/2021	02/02/2021	02/02/2021	02/02/2021									
Dissolved Aluminium	50282 AC	564269 AF	229372 AF	57230 AC							<20	ug/l	TM30/PM14
Dissolved Antimony	<2	<2	<2	<2							<2	ug/l	TM30/PM14
Dissolved Arsenic	45.5	1515.5	498.5	71.6							<2.5	ug/l	TM30/PM14
Dissolved Barium	<3	<3	<3	<3							<3	ug/l	TM30/PM14
Dissolved Beryllium	<0.5	<0.5	<0.5	<0.5							<0.5	ug/l	TM30/PM14
Dissolved Boron	49	538	222	63							<12	ug/l	TM30/PM14
Dissolved Cadmium	<0.5	<0.5	<0.5	<0.5							<0.5	ug/l	TM30/PM14
Dissolved Calcium	0.7	0.8	1.7	0.9							<0.2	mg/l	TM30/PM14
Total Dissolved Chromium	4.8	4.8	6.3	1.5							<1.5	ug/l	TM30/PM14
Dissolved Cobalt	<2	<2	<2	<2							<2	ug/l	TM30/PM14
Dissolved Copper	10	90	69	17							<7	ug/l	TM30/PM14
Total Dissolved Iron	51	66	60	126							<20	ug/l	TM30/PM14
Dissolved Lead	19	<5	<5	<5							<5	ug/l	TM30/PM14
Dissolved Lithium	<5	<5	<5	<5							<5	ug/l	TM30/PM14
Dissolved Magnesium	<0.1	<0.1	<0.1	<0.1							<0.1	mg/l	TM30/PM14
Dissolved Manganese	2	<2	<2	<2							<2	ug/l	TM30/PM14
Dissolved Mercury	<1	<1	<1	<1							<1	ug/l	TM30/PM14
Dissolved Molybdenum	56	2386	1136	207							<2	ug/l	TM30/PM14
Dissolved Nickel	<2	9	5	<2							<2	ug/l	TM30/PM14
Dissolved Potassium	9.5	63.2	22.7	7.1							<0.1	mg/l	TM30/PM14
Dissolved Selenium	21	824 AC	105	25							<3	ug/l	TM30/PM14
Dissolved Silicon	7598	2203	2699	2237							<100	ug/l	TM30/PM14
Dissolved Silver	<5	<5	<5	<5							<5	ug/l	TM30/PM14
Dissolved Sodium	663.4 AA	5719.1 AE	1772.6 AE	557.2 AA							<0.1	mg/l	TM30/PM14
Dissolved Strontium	<5	<5	<5	<5							<5	ug/l	TM30/PM14
Dissolved Tin	<5	<25 AA	<25 AA	<25 AA							<5	ug/l	TM30/PM14
Dissolved Titanium	<5	<25 AA	<5	<5							<5	ug/l	TM30/PM14
Dissolved Uranium	<5	<5	<5	<5							<5	ug/l	TM30/PM14
Dissolved Vanadium	1177.4	14073.4 AC	7243.4 AC	1451.1							<1.5	ug/l	TM30/PM14
Dissolved Zinc	3240	62063 AC	563	11							<3	ug/l	TM30/PM14
Fluoride	8.7 AB	67.6 AB	16.8 AD	4.0 AB							<0.3	mg/l	TM173/PM0
Sulphate as SO4	67.3	1117.0	186.4	89.0							<0.5	mg/l	TM38/PM0
Chloride	14.5	122.0	73.4	46.6							<0.3	mg/l	TM38/PM0
Total Alkalinity as CaCO3	965	12256	4023	1279							<1	mg/l	TM75/PM0
P Alkalinity as CaCO3	778	9396	2965	781							<1	mg/l	TM75/PM0
pH	12.21	12.92	12.56	11.73							<0.01	pH units	TM73/PM0

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis
EMT Job No: 21/1325

Report : **CEN 10:1 2-Batch**

Solids: V=60g VOC jar, J=250g glass jar, T=plastic tub

Please see attached notes for all abbreviations and acronyms

EMT Sample No.	37-38	LOD/LOR	Units	Method No.
Sample ID	STAGE 5			
Depth				
COC No / misc				
Containers	T			
Sample Date	28/01/2021			
Sample Type	Soil			
Batch Number	1			
Date of Receipt	02/02/2021			
Dissolved Aluminium (C2)	0.46	<0.02	mg/l	TM30/PM14
Dissolved Aluminium (C8)	0.75	<0.02	mg/l	TM30/PM14
Dissolved Aluminium (A2)	0.92	<0.04	mg/kg	TM30/PM14
Dissolved Aluminium (A2-10)	7.0	<0.2	mg/kg	TM30/PM14
Dissolved Beryllium (C2)	<0.0005	<0.0005	mg/l	TM30/PM14
Dissolved Beryllium (C8)	<0.0005	<0.0005	mg/l	TM30/PM14
Dissolved Beryllium (A2)	<0.001	<0.001	mg/kg	TM30/PM14
Dissolved Beryllium (A2-10)	<0.005	<0.005	mg/kg	TM30/PM14
Dissolved Boron (C2)	0.039	<0.012	mg/l	TM30/PM14
Dissolved Boron (C8)	<0.012	<0.012	mg/l	TM30/PM14
Dissolved Boron (A2)	0.078	<0.024	mg/kg	TM30/PM14
Dissolved Boron (A2-10)	<0.12	<0.12	mg/kg	TM30/PM14
Dissolved Calcium (C2)	28.8	<0.2	mg/l	TM30/PM14
Dissolved Calcium (C8)	10.4	<0.2	mg/l	TM30/PM14
Dissolved Calcium (A2)	57.5	<0.4	mg/kg	TM30/PM14
Dissolved Calcium (A2-10)	134	<2	mg/kg	TM30/PM14
Dissolved Cobalt (C2)	<0.002	<0.002	mg/l	TM30/PM14
Dissolved Cobalt (C8)	<0.002	<0.002	mg/l	TM30/PM14
Dissolved Cobalt (A2)	<0.004	<0.004	mg/kg	TM30/PM14
Dissolved Cobalt (A2-10)	<0.02	<0.02	mg/kg	TM30/PM14
Dissolved Iron (C2)	1.40	<0.02	mg/l	TM30/PM14
Dissolved Iron (C8)	0.27	<0.02	mg/l	TM30/PM14
Dissolved Iron (A2)	2.80	<0.04	mg/kg	TM30/PM14
Dissolved Iron (A2-10)	4.6	<0.2	mg/kg	TM30/PM14
Dissolved Lithium (C2)	0.039	<0.005	mg/l	TM30/PM14
Dissolved Lithium (C8)	0.012	<0.005	mg/l	TM30/PM14
Dissolved Lithium (A2)	0.08	<0.01	mg/kg	TM30/PM14
Dissolved Lithium (A2-10)	0.16	<0.05	mg/kg	TM30/PM14
Dissolved Magnesium (C2)	2.5	<0.1	mg/l	TM30/PM14
Dissolved Magnesium (C8)	0.4	<0.1	mg/l	TM30/PM14
Dissolved Magnesium (A2)	5.0	<0.2	mg/kg	TM30/PM14
Dissolved Magnesium (A2-10)	7	<1	mg/kg	TM30/PM14
Dissolved Manganese (C2)	0.002	<0.002	mg/l	TM30/PM14
Dissolved Manganese (C8)	<0.002	<0.002	mg/l	TM30/PM14
Dissolved Manganese (A2)	<0.004	<0.004	mg/kg	TM30/PM14
Dissolved Manganese (A2-10)	<0.02	<0.02	mg/kg	TM30/PM14
Dissolved Potassium (C2)	3.3	<0.1	mg/l	TM30/PM14
Dissolved Potassium (C8)	0.2	<0.1	mg/l	TM30/PM14
Dissolved Potassium (A2)	6.6	<0.2	mg/kg	TM30/PM14
Dissolved Potassium (A2-10)	7	<1	mg/kg	TM30/PM14
Dissolved Silicon(C2)	3.6	<0.1	mg/l	TM30/PM14
Dissolved Silicon (C8)	0.9	<0.1	mg/l	TM30/PM14
Dissolved Silicon (A2)	7.2	<0.2	mg/kg	TM30/PM14
Dissolved Silicon (A2-10)	13	<1	mg/kg	TM30/PM14

Element Materials Technology

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis
EMT Job No: 21/1325

Report : CEN 10:1 2-Batch

Solids: V=60g VOC jar, J=250g glass jar, T=plastic tub

Please see attached notes for all abbreviations and acronyms

EMT Sample No.	37-38										LOD/LOR	Units	Method No.
Sample ID	STAGE 5												
Depth													
COC No / misc													
Containers	T												
Sample Date	28/01/2021												
Sample Type	Soil												
Batch Number	1												
Date of Receipt	02/02/2021												
Dissolved Silver (C2)	<0.005										<0.005	mg/l	TM30/PM14
Dissolved Silver (C8)	<0.005										<0.005	mg/l	TM30/PM14
Dissolved Silver (A2)	<0.01										<0.01	mg/kg	TM30/PM14
Dissolved Silver (A2-10)	<0.05										<0.05	mg/kg	TM30/PM14
Dissolved Sodium (C2)	31.1										<0.1	mg/l	TM30/PM14
Dissolved Sodium (C8)	2.8										<0.1	mg/l	TM30/PM14
Dissolved Sodium (A2)	62.1										<0.2	mg/kg	TM30/PM14
Dissolved Sodium (A2-10)	75										<1	mg/kg	TM30/PM14
Dissolved Strontium (C2)	0.073										<0.005	mg/l	TM30/PM14
Dissolved Strontium (C8)	0.016										<0.005	mg/l	TM30/PM14
Dissolved Strontium (A2)	0.15										<0.01	mg/kg	TM30/PM14
Dissolved Strontium (A2-10)	0.25										<0.05	mg/kg	TM30/PM14
Dissolved Tin (C2)	<0.005										<0.005	mg/l	TM30/PM14
Dissolved Tin (C8)	<0.005										<0.005	mg/l	TM30/PM14
Dissolved Tin (A2)	<0.01										<0.01	mg/kg	TM30/PM14
Dissolved Tin (A2-10)	<0.05										<0.05	mg/kg	TM30/PM14
Dissolved Titanium (C2)	0.129										<0.005	mg/l	TM30/PM14
Dissolved Titanium (C8)	0.025										<0.005	mg/l	TM30/PM14
Dissolved Titanium (A2)	0.26										<0.01	mg/kg	TM30/PM14
Dissolved Titanium (A2-10)	0.42										<0.05	mg/kg	TM30/PM14
Dissolved Uranium (C2)	<0.01										<0.01	mg/l	TM30/PM14
Dissolved Uranium (C8)	<0.01										<0.01	mg/l	TM30/PM14
Dissolved Uranium (A2)	<0.02										<0.02	mg/kg	TM30/PM14
Dissolved Uranium (A2-10)	<0.1										<0.1	mg/kg	TM30/PM14
Dissolved Vanadium (C2)	0.2422										<0.0015	mg/l	TM30/PM14
Dissolved Vanadium (C8)	0.1191										<0.0015	mg/l	TM30/PM14
Dissolved Vanadium (A2)	0.484										<0.003	mg/kg	TM30/PM14
Dissolved Vanadium (A2-10)	1.395										<0.015	mg/kg	TM30/PM14
Total Alkalinity as CaCO3 (C2)	180										<1	mg/l	TM75/PM0
Total Alkalinity as CaCO3 (C8)	38										<1	mg/l	TM75/PM0
Total Alkalinity as CaCO3 (A2)	360										<2	mg/kg	TM75/PM0
Total Alkalinity as CaCO3 (A2-10)	615										<10	mg/kg	TM75/PM0
P Alkalinity as CaCO3 (C2)	<1										<1	mg/l	TM75/PM0
P Alkalinity as CaCO3 (C8)	<1										<1	mg/l	TM75/PM0
P Alkalinity as CaCO3 (A2)	<2										<2	mg/kg	TM75/PM0
P Alkalinity as CaCO3 (A2-10)	<10										<10	mg/kg	TM75/PM0
Sulphate as SO_4 (C2)	16.2										<0.5	mg/l	TM38/PM0
Sulphate as SO_4 (C8)	0.7										<0.5	mg/l	TM38/PM0
Sulphate as SO_4 (A2)	32										<1	mg/kg	TM38/PM0
Sulphate as SO_4 (A2-10)	34										<5	mg/kg	TM38/PM0

Please include all sections of this report if it is reproduced
All solid results are expressed on a dry weight basis unless stated otherwise.

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis

EMT Job No.	Batch	Sample ID	Depth	EMT Sample No.	Analysis	Reason
No deviating sample report results for job 21/1325						

Please note that only samples that are deviating are mentioned in this report. If no samples are listed it is because none were deviating.
Only analyses which are accredited are recorded as deviating if set criteria are not met.

SOILS

Please note we are only MCERTS accredited (UK soils only) for sand, loam and clay and any other matrix is outside our scope of accreditation.

Where an MCERTS report has been requested, you will be notified within 48 hours of any samples that have been identified as being outside our MCERTS scope. As validation has been performed on clay, sand and loam, only samples that are predominantly these matrices, or combinations of them will be within our MCERTS scope. If samples are not one of a combination of the above matrices they will not be marked as MCERTS accredited.

It is assumed that you have taken representative samples on site and require analysis on a representative subsample. Stones will generally be included unless we are requested to remove them.

All samples will be discarded one month after the date of reporting, unless we are instructed to the contrary.

If you have not already done so, please send us a purchase order if this is required by your company.

Where appropriate please make sure that our detection limits are suitable for your needs, if they are not, please notify us immediately.

All analysis is reported on a dry weight basis unless stated otherwise. Limits of detection for analyses carried out on as received samples are not moisture content corrected. Results are not surrogate corrected. Samples are dried at 35°C ±5°C unless otherwise stated. Moisture content for CEN Leachate tests are dried at 105°C ±5°C.

Where Mineral Oil or Fats, Oils and Grease is quoted, this refers to Total Aliphatics C10-C40.

Where a CEN 10:1 ZERO Headspace VOC test has been carried out, a 10:1 ratio of water to wet (as received) soil has been used.

% Asbestos in Asbestos Containing Materials (ACMs) is determined by reference to HSG 264 The Survey Guide - Appendix 2 : ACMs in buildings listed in order of ease of fibre release.

Sufficient amount of sample must be received to carry out the testing specified. Where an insufficient amount of sample has been received the testing may not meet the requirements of our accredited methods, as such accreditation may be removed.

Negative Neutralization Potential (NP) values are obtained when the volume of NaOH (0.1N) titrated (pH 8.3) is greater than the volume of HCl (1N) to reduce the pH of the sample to 2.0 - 2.5. Any negative NP values are corrected to 0.

The calculation of Pyrite content assumes that all oxidisable sulphides present in the sample are pyrite. This may not be the case. The calculation may be an overesitimate when other sulphides such as Barite (Barium Sulphate) are present.

WATERS

Please note we are not a UK Drinking Water Inspectorate (DWI) Approved Laboratory .

ISO17025 accreditation applies to surface water and groundwater and usually one other matrix which is analysis specific, any other liquids are outside our scope of accreditation.

As surface waters require different sample preparation to groundwaters the laboratory must be informed of the water type when submitting samples.

Where Mineral Oil or Fats, Oils and Grease is quoted, this refers to Total Aliphatics C10-C40.

DEVIATING SAMPLES

All samples should be submitted to the laboratory in suitable containers with sufficient ice packs to sustain an appropriate temperature for the requested analysis. The temperature of sample receipt is recorded on the confirmation schedules in order that the client can make an informed decision as to whether testing should still be undertaken.

SURROGATES

Surrogate compounds are added during the preparation process to monitor recovery of analytes. However low recovery in soils is often due to peat, clay or other organic rich matrices. For waters this can be due to oxidants, surfactants, organic rich sediments or remediation fluids. Acceptable limits for most organic methods are 70 - 130% and for VOCs are 50 - 150%. When surrogate recoveries are outside the performance criteria but the associated AQC passes this is assumed to be due to matrix effect. Results are not surrogate corrected.

DILUTIONS

A dilution suffix indicates a dilution has been performed and the reported result takes this into account. No further calculation is required.

BLANKS

Where analytes have been found in the blank, the sample will be treated in accordance with our laboratory procedure for dealing with contaminated blanks.

NOTE

Data is only reported if the laboratory is confident that the data is a true reflection of the samples analysed. Data is only reported as accredited when all the requirements of our Quality System have been met. In certain circumstances where all the requirements of the Quality System have not been met, for instance if the associated AQC has failed, the reason is fully investigated and documented. The sample data is then evaluated alongside the other quality control checks performed during analysis to determine its suitability. Following this evaluation, provided the sample results have not been effected, the data is reported but accreditation is removed. It is a UKAS requirement for data not reported as accredited to be considered indicative only, but this does not mean the data is not valid.
Where possible, and if requested, samples will be re-extracted and a revised report issued with accredited results. Please do not hesitate to contact the laboratory if further details are required of the circumstances which have led to the removal of accreditation.

REPORTS FROM THE SOUTH AFRICA LABORATORY

Any method number not prefixed with SA has been undertaken in our UK laboratory unless reported as subcontracted.

Measurement Uncertainty

Measurement uncertainty defines the range of values that could reasonably be attributed to the measured quantity. This range of values has not been included within the reported results. Uncertainty expressed as a percentage can be provided upon request.

ABBREVIATIONS and ACRONYMS USED

#	ISO17025 (UKAS Ref No. 4225) accredited - UK.
SA	ISO17025 (SANAS Ref No.T0729) accredited - South Africa
B	Indicates analyte found in associated method blank.
DR	Dilution required.
M	MCERTS accredited.
NA	Not applicable
NAD	No Asbestos Detected.
ND	None Detected (usually refers to VOC and/SVOC TICs).
NDP	No Determination Possible
SS	Calibrated against a single substance
SV	Surrogate recovery outside performance criteria. This may be due to a matrix effect.
W	Results expressed on as received basis.
+	AQC failure, accreditation has been removed from this result, if appropriate, see 'Note' on previous page.
>>	Results above calibration range, the result should be considered the minimum value. The actual result could be significantly higher, this result is not accredited.
*	Analysis subcontracted to an Element Materials Technology approved laboratory.
AD	Samples are dried at 35°C ±5°C
CO	Suspected carry over
LOD/LOR	Limit of Detection (Limit of Reporting) in line with ISO 17025 and MCERTS
ME	Matrix Effect
NFD	No Fibres Detected
BS	AQC Sample
LB	Blank Sample
N	Client Sample
TB	Trip Blank Sample
OC	Outside Calibration Range
AA	x5 Dilution

AB	x10 Dilution
AC	x20 Dilution
AD	x25 Dilution
AE	x50 Dilution
AF	x200 Dilution

HWOL ACRONYMS AND OPERATORS USED

HS	Headspace Analysis.
EH	Extractable Hydrocarbons - i.e. everything extracted by the solvent.
CU	Clean-up - e.g. by florisil, silica gel.
1D	GC - Single coil gas chromatography.
Total	Aliphatics & Aromatics.
AL	Aliphatics only.
AR	Aromatics only.
2D	GC-GC - Double coil gas chromatography.
#1	EH_Total but with humics extracted.
#2	EU_Total but with fatty acids extracted.
_	Operator - underscore to separate acronyms (exception for +).
+	Operator to indicate cumulative e.g. EH+HS_Total or EH_CU+HS_Total
MS	Mass Spectrometry.

EMT Job No: 21/1325

Test Method No.	Description	Prep Method No. (if appropriate)	Description	ISO 17025 (UKAS/SANAS)	MCERTS (UK soils only)	Analysis done on As Received (AR) or Dried (AD)	Reported on dry weight basis
PM4	Gravimetric measurement of Natural Moisture Content and % Moisture Content at either 35°C or 105°C. Calculation based on ISO 11465:1993(E) and BS1377-2:1990.	PM0	No preparation is required.			AR	
TM30	Determination of Trace Metals by ICP-OES (Inductively Coupled Plasma – Optical Emission Spectrometry): WATERS by Modified USEPA Method 200.7, Rev. 4.4, 1994; Modified EPA Method 6010B, Rev.2, Dec 1996; Modified BS EN ISO 11885:2009: SOILS by Modified USEP	PM14	Preparation of waters and leachates for metals by ICP OES/ICP MS. Samples are filtered for Dissolved metals, and remain unfiltered for Total metals then acidified				
TM30	Determination of Trace Metals by ICP-OES (Inductively Coupled Plasma – Optical Emission Spectrometry): WATERS by Modified USEPA Method 200.7, Rev. 4.4, 1994; Modified EPA Method 6010B, Rev.2, Dec 1996; Modified BS EN ISO 11885:2009: SOILS by Modified USEP	PM14	Preparation of waters and leachates for metals by ICP OES/ICP MS. Samples are filtered for Dissolved metals, and remain unfiltered for Total metals then acidified			AR	Yes
TM38	Soluble Ion analysis using Discrete Analyser. Modified US EPA methods: Chloride 325.2 (1978), Sulphate 375.4 (Rev.2 1993), o-Phosphate 365.2 (Rev.2 1993), TON 353.1 (Rev.2 1993), Nitrite 354.1 (1971), Hex Cr 7196A (1992), NH4+ 350.1 (Rev.2 1993) – All anions comparable to BS ISO 15923-1: 2013l	PM0	No preparation is required.				
TM38	Soluble Ion analysis using Discrete Analyser. Modified US EPA methods: Chloride 325.2 (1978), Sulphate 375.4 (Rev.2 1993), o-Phosphate 365.2 (Rev.2 1993), TON 353.1 (Rev.2 1993), Nitrite 354.1 (1971), Hex Cr 7196A (1992), NH4+ 350.1 (Rev.2 1993) – All anions comparable to BS ISO 15923-1: 2013l	PM0	No preparation is required.			AR	Yes
TM73	Modified US EPA methods 150.1 (1982) and 9045D Rev. 4 - 2004) and BS1377-3:1990. Determination of pH by Metrohm automated probe analyser.	PM0	No preparation is required.				
TM75	Modified US EPA method 310.1 (1978). Determination of Alkalinity by Metrohm automated titration analyser.	PM0	No preparation is required.				
TM75	Modified US EPA method 310.1 (1978). Determination of Alkalinity by Metrohm automated titration analyser.	PM0	No preparation is required.			AR	Yes
TM173	Analysis of fluoride by ISE (Ion Selective Electrode) using modified ISE method 9214 - 340.2 (EPA 1998)	PM0	No preparation is required.				
TM173	Analysis of fluoride by ISE (Ion Selective Electrode) using modified ISE method 9214 - 340.2 (EPA 1998)	PM0	No preparation is required.			AR	Yes

EMT Job No: 21/1325

Test Method No.	Description	Prep Method No. (if appropriate)	Description	ISO 17025 (UKAS/SANAS)	MCERTS (UK soils only)	Analysis done on As Received (AR) or Dried (AD)	Reported on dry weight basis
NONE	No Method Code	PM18	Modified method BS EN12457-3:2002 . As received solid samples are leached with water in a 2:1 water to solid ratio for 6 hours, the same aliquot of solid is then re-leached with water in an 8:1 water to solid ratio for 18 hours, the moisture content of the sample is included in the ratio. This preparation produces two eluates.			AR	Yes
NONE	No Method Code	PM4	Gravimetric measurement of Natural Moisture Content and % Moisture Content at either 35°C or 105°C. Calculation based on ISO 11465:1993(E) and BS1377-2:1990.			AR	

APPENDIX B

Amended Bauxite Residue Leaching and Piezometer Liquid Analysis Report

REPORT

Aughinish Alumina Limited

Amended Bauxite Residue Leaching and Piezometer Liquid Analysis

Submitted to:

Aughinish Alumina Ltd
Aughinish Island
Askeaton
Co Limerick

Submitted by:

Golder Associates Ireland Limited
Town Centre House, Dublin Road, Naas,
Co. Kildare, W91 TD0P Ireland

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19122440.R01.A2

August 2021

Distribution List

AAL - one copy (PDF)

Golder Associate Ireland Ltd - one copy (PDF)

Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

Golder Associates Ireland Limited (Golder) has been engaged by Aughinish Alumina Ltd (AAL) to assess long-term water quality and behaviour of seepage emanating from the bauxite residue (BR) disposal area (BRDA) at AAL's facility in Askeaton, Co Limerick (the Site).

The BRDA is comprised of multiple phases which contain:

1) Unfarmed BR on estuarine sediments on original unlined Phase 1, 72 ha
2) Unfarmed BR on composite liner Phase 1 Extension, 32 ha
3) Carbonated farmed BR on 1) and 2) since 2009
4) Carbonated farmed BR on composite liner in Phase 2, 80 ha

The farming process that is carried out by AAL consists of ploughing and aerating bauxite residue to reduce the pH whereby carbon dioxide in the atmosphere reacts with the high pH hydroxide components of the bauxite residue, forming carbonates. The farming process also improves the geotechnical properties of the bauxite residue. Upon closure, it is proposed that the BRDA will be capped with a minimum 1 m depth of amended bauxite residue which has been mixed with neutralised process sand, gypsum, and organic material. This amended layer is present on the Site on several closed cells in the Phase 1 BRDA.

2.0 BACKGROUND

AAL has requested that Golder advise them on the potential water quality and quantity of seepage from the bauxite residue, which will inform requirements for water management and treatment post-closure.

The leachate from bauxite residue is of a high pH due to the presence of sodium hydroxide (caustic soda) which is used in the alumina extraction process. The alkaline buffering capacity of high pH bauxite residue systems containing hydroxide species is very high, i.e. even very large volumes of flushing water are unlikely to reduce the pH significantly. This is due to the salinity of bauxite residue and the hydroxide complexing that takes place whereby hydroxide binds to any positive valance ions in solution (such as calcium, magnesium, potassium, iron, and aluminium). This complexing creates a large store of alkalinity which is released from the residue into solution when the pore water is flushed.

To assess the potential for seepage, Golder has reviewed historic hydraulic conductivity testing for the bauxite residue material[1]. Due to very low hydraulic conductivity values, seepage is expected to be very slow with an estimated 4 mm of vertical seepage per day or 1.4 m per year. Therefore, it would take approximately 20 years for one pore volume to flow through a 30 m thickness of residue. Saturated column leach testing of the unamended bauxite residue material is therefore challenging. The most direct analogue for seepage from unamended and unfarmed bauxite residue is liquid present in the piezometers installed in this material, which has been extracted and analysed in this scope of works.

Hydraulic conductivity values are expected to be higher for the amended capping layer due to the amendments with growth media including compost to allow for a vegetated domed cap on the BRDA. Golder recommended that leach testing in accordance with *EN 14405 Characterisation of waste – up-flow column percolation test* be carried out on the amended cap layer only, to assess the effects of the closure capping on infiltration and the

[1] Golder Associates Ireland Ltd. 2016. Aughinish Alumina Limited: Process Sand and Red Mud Tailings Geotechnical Laboratory Testing. Report Number 1435512.R02.B0 dated January 2016.

rate of seepage through this layer which is expected to be a major contributor to seepage from the BRDA into the perimeter interceptor channel.

3.0 METHODS

3.1 Amended Layer Leach Testing and Hydraulic Conductivity

3.1.1 Sampling Methods

Golder attended the site on 26 June 2020 with Metlab to collect amended bauxite residue samples from the Phase 1 BRDA. The locations of these samples and sample photos are shown in Figure 1, below. Golder retained a 5 kg sample of the amended bauxite residue surface layer, which was collected from Stage 5, located on the Phase 1 BRDA. Stage 5 has been successfully revegetated and the grass layer was stripped to allow for collection of the amended layer material. Another 5 kg sample was collected from Cell 6, also located on the Phase 1 BRDA at a higher level. This material has been amended and farmed but has not been vegetated. Cell 6 contains fresher bauxite residue material than Stage 5.

Metlab retained circa 20 kg bulk disturbed samples of amended bauxite residue from two locations: a vegetated area on Stage 5 and an unvegetated area on Cell 6 (Stage 10) and retained undisturbed core samples from Stage 5. An undisturbed core could not be retained from Cell 6.

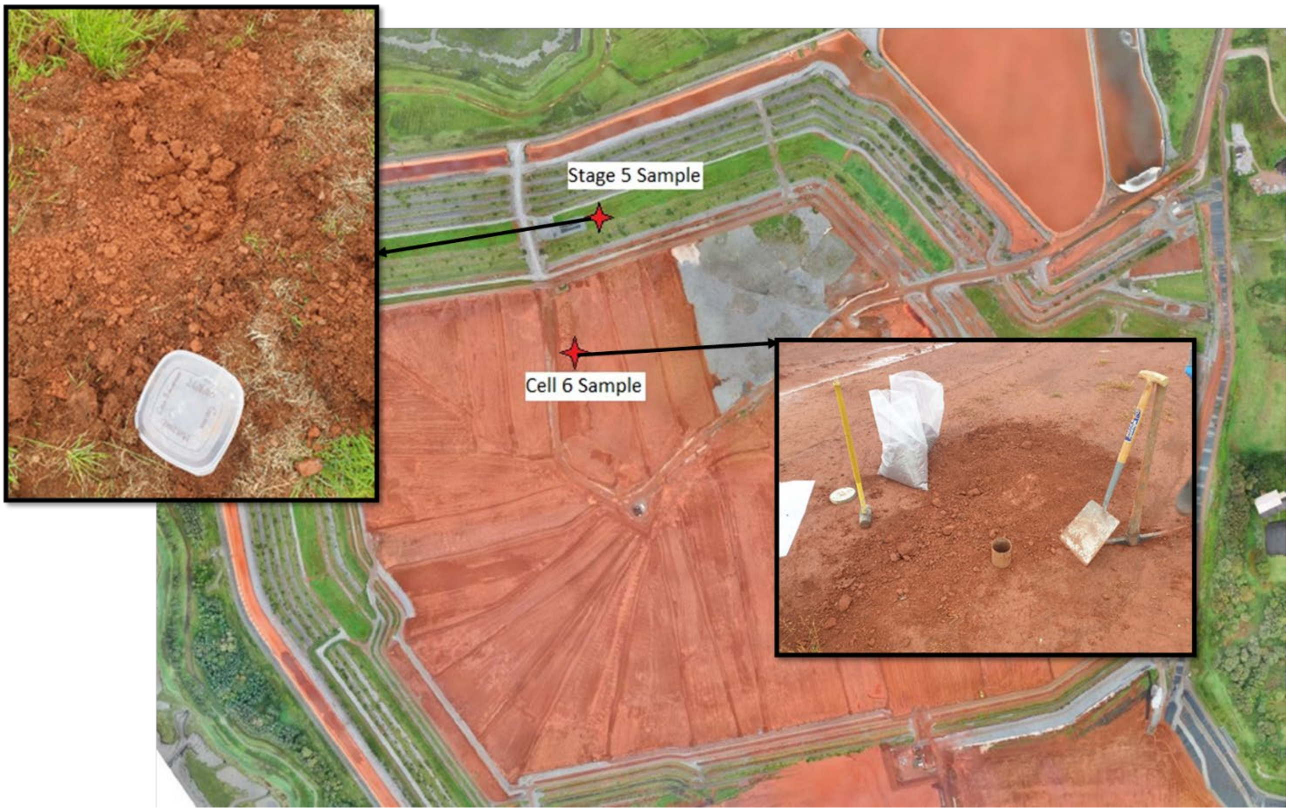


Figure 1: Sampling Locations and Photos

3.1.2 EN 14405 Leachate Testing

The two-no. 5 kg amended bauxite residue samples from Stage 5 and Cell 6 were submitted to SGS Intron, a laboratory in the Netherlands, for EN 14405 testing. The EN 14405 test is a standard European leaching test to assess the constituents that can be released from waste materials. Seven eluate fractions are collected over the course of the leaching test, starting with a liquid to solid ratio of 0.1:1 and increasing to a cumulative liquid to solid ratio of 10:1 (L/S = 0.1, 0.2, 0.5, 1, 2, 5, and 10). Two up-flow column tests of the amended capping layer were carried out with analysis of the seven stages of leachate for pH and conductivity and the total 10:1 leachate for other relevant parameters (major anions and metals) for purposes of seepage water quality assessment.

3.1.3 Nuclear Density Testing

Metlab carried out nuclear density testing on site to assess in situ conditions (density and moisture content).

3.2 Piezometer Liquid Analysis

3.2.1 Sampling

Golder attended the Site on 8 June and 26 June 2020 to collect liquid from selected piezometers on the Phase 1 BRDA representing unfarmed, unamended bauxite residue. Samples were collected using a Waterra foot valve and Waterra tubing. Field parameters (temperature, pH and conductivity) were assessed using a portable water meter.

3.2.2 Laboratory Analysis

The water samples were dispatched to Element in Deeside, UK, and analysed for p-alkalinity (measure of the amount of acid required to drop the pH to 8.3), and m-alkalinity (total alkalinity, e.g. measure of the amount of acid it takes to drop the pH to 4.3). In addition, metals analysis was carried out for arsenic, barium, beryllium, boron, cadmium, chromium, copper, iron, lead, mercury, nickel, selenium, vanadium, titanium, and zinc.

4.0 RESULTS

4.1 Geotechnical Results, Amended Layer

The Metlab reports are included in Appendix A and summarised below.

4.1.1 Nuclear Density

The average nuclear density test results for each sampling location are summarised in Table 1, below.

Table 1: Average Nuclear Density Results, Amended Layer Testing

	Test Depth (mm)	Bulk Density (Mg/m^3)	Dry Density (Mg/m^3)	Moisture Content (%)	Corrected Compaction (%)
Stage 5 (Field Test)	250 mm	2.115	1.672	26.4	95.6%
Cell 6 (Stage 10) (Field Test)	250 mm	1.937	1.588	22.0	90.7

4.1.2 Dry Density and Moisture Content

The dry density and moisture content testing is summarised in Table 2, below.

Table 2: Dry density and moisture content test results

	Maximum Dry Density (Mg/m^3)	Optimum Moisture Content (%)	Natural Moisture Content (%)
Stage 5 Bulk disturbed sample	1.75	21.0	17.2
Cell 6 (Stage 10) Bulk disturbed sample	1.74	20.0	27.9

4.1.3 Core Density Determination

The core density results for the undisturbed core sample from Stage 5 are summarised in Table 3, below.

Table 3: Core Density Determination

	Bulk Density (Mg/m^3)	Dry Density (Mg/m^3)	Moisture Content (%)
Stage 5 (undisturbed core)	2.11	1.67	26.4
Cell 6 (Stage 10)	Fresher material, could not be cored		

4.1.4 Permeability in a Triaxial Cell

The triaxial permeability analysis was subcontracted by Metlab to GEO Site & Testing Services Ltd in Wales, UK. The results are summarised in Table 4, below, with reported values in the 10^{-10} m/s range. These permeability values are lower than anticipated, given that previous testing on farmed BR samples in 2015 was in the 10^{-8} to 10^{-9} m/s range[2]. It may be that the compost addition in the sample has an impact on the laboratory permeability testing.

Table 4: Triaxial Permeability Results, Amended Layer

Sample Name	Vertical Permeability (m/s)
Stage 5	1.04×10^{-10}
Cell 6 (Stage 10)	0.97×10^{-10}

[2] Golder Associates Ireland Ltd. 2016. Aughinish Alumina Limited: Process Sand and Red Mud Tailings Geotechnical Laboratory Testing. Report Number 1435512.R02.B0 dated January 2016.

4.2 Amended Layer Leaching

The EN 14405 testing was carried out in July and August 2020. Both column tests percolated successfully, indicating that the saturated hydraulic conductivity is sufficient to allow for flow through the sample material.

The SGS Intron laboratory report is included in Appendix B.

4.2.1 Leachate Step Results – pH and Conductivity

At each of the seven leachate steps, pH and conductivity were measured. These parameters were chosen as pH is the primary parameter of interest from bauxite residue and conductivity is a measurement of the overall solute release from the sample.

In the Cell 6 sample, representing fresher amended material which is unvegetated, pH ranges from 8.98 to 9.78 and conductivity ranges from 455 to 6,345 µS/cm (See Figure 2 and Figure 3). The higher conductivity is caused by higher concentrations of solutes released from the sample (See Section 4.2.2).

In the Stage 5 sample, pH ranges from 8.37 to 8.74 and conductivity ranges from 335 to 1,045 uS/cm (see Figure 2 and Figure 3). These results indicate that the amendment process and revegetation is successful at reducing pH in seepage to the legislative limit for direct discharge.

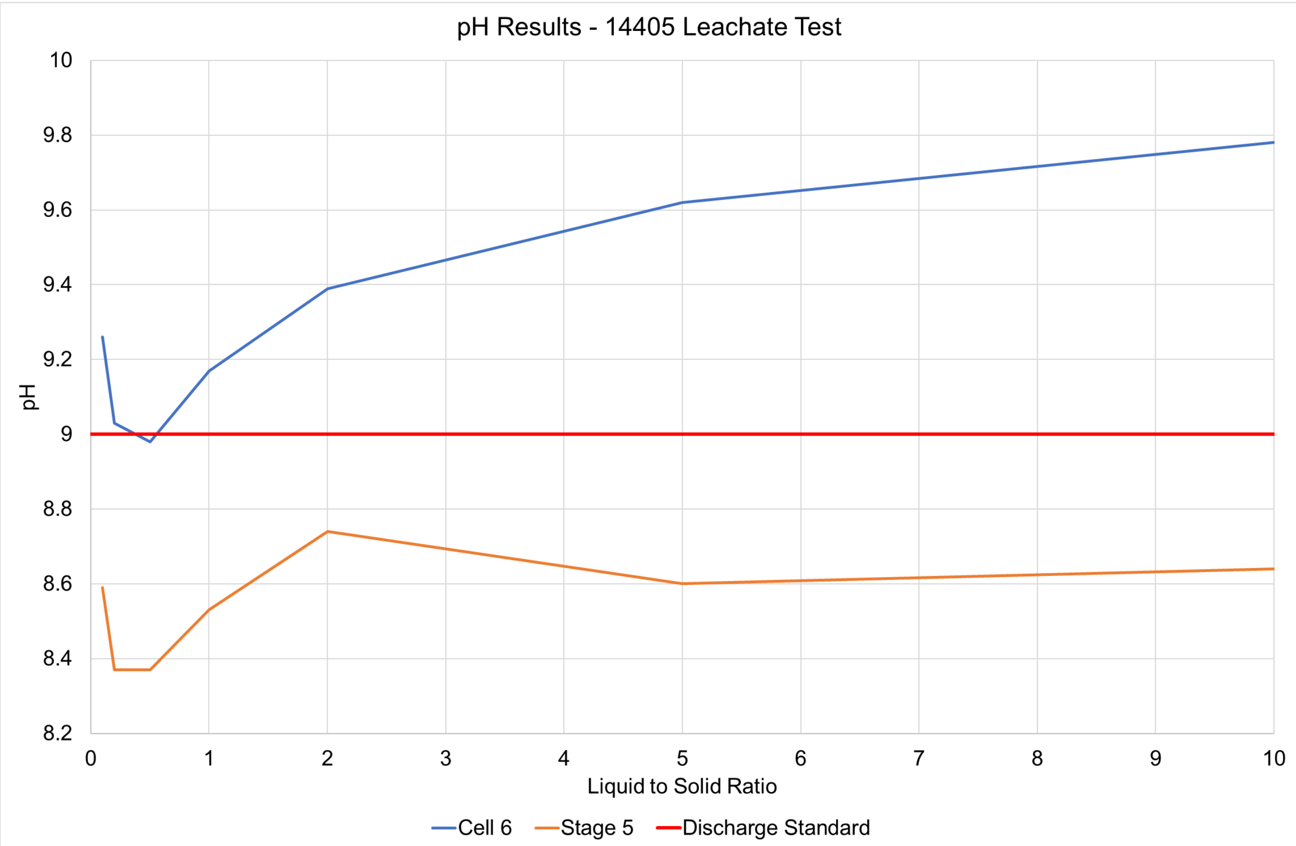


Figure 2: pH Results - CEN 14405 leachate test on amended bauxite layer

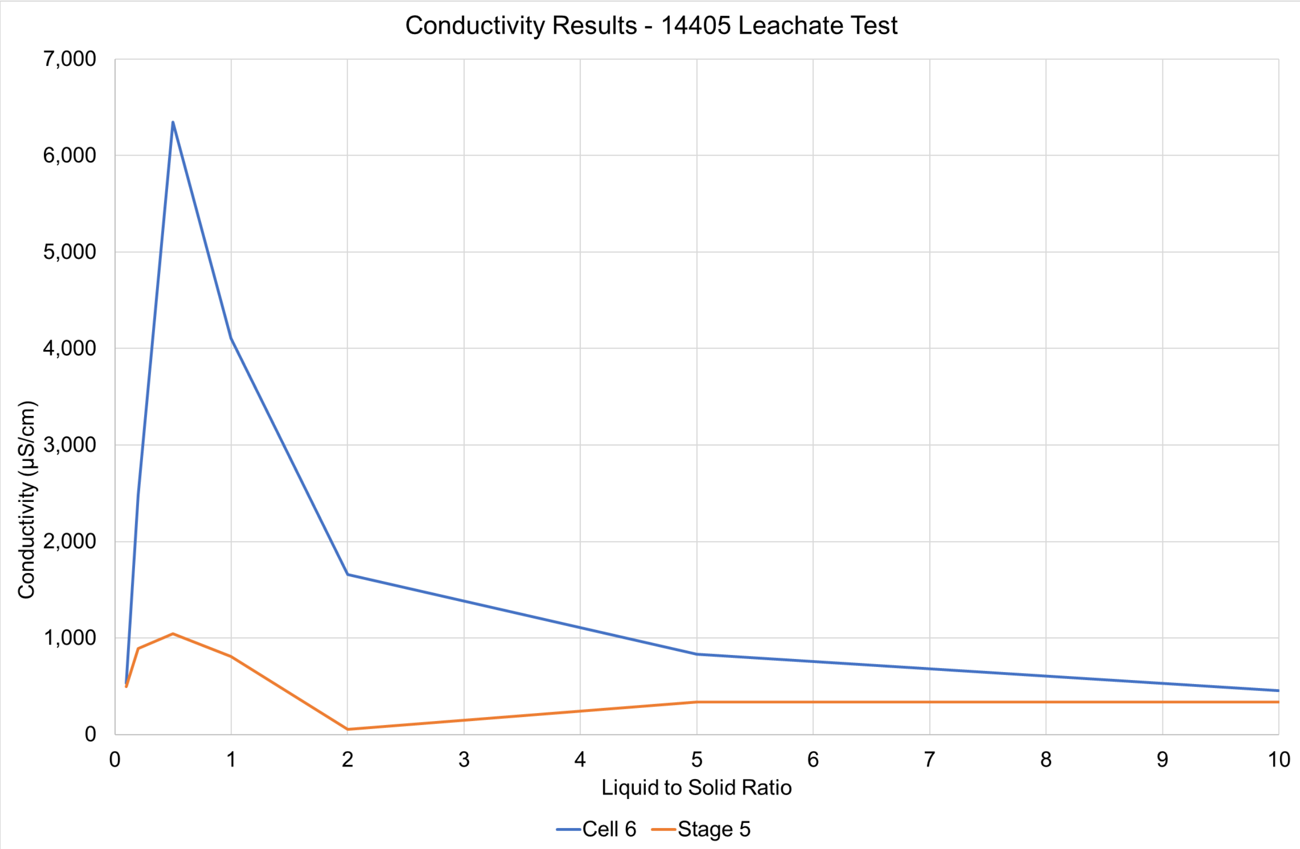


Figure 3: Conductivity Results - CEN 14405 leachate test on amended bauxite layer

4.2.2 10:1 Leachate Results – Metals

The final 10:1 leachate results for metals from the Stage 5 and Cell 6 (Stage 10) samples are presented in Table 5, below. In general, the metal results are lower in the Stage 5 sample. The Stage 5 material is older amended material and has been vegetated. The reduced concentrations and lower pH and conductivity in Stage 5 indicates that the process of farming, amendment, and vegetated cover is effective in neutralising the bauxite residue.

Table 5: Amended Layer Leachate Testing

Parameter	Emission (mg/kg)		Emission (mg/L)	
	Stage 5	Cell 6	Stage 5	Cell 6
Dry matter (% m/m)	77.5	78.2		
Antimony (Sb)	< 0.004	0.015	<0.0004	0.0015
Arsenic (As)	0.069	0.068	0.0069	0.0068
Barium (Ba)	< 0.06	< 0.06	<0.006	<0.006
Cadmium (Cd)	< 0.001	0.0032	<0.0001	0.00032
Chromium (Cr)	< 0.10	0.25	<0.010	0.025
Cobalt (Co)	< 0.03	< 0.03	<0.003	<0.003

Parameter	Emission (mg/kg)		Emission (mg/L)	
	Stage 5	Cell 6	Stage 5	Cell 6
Cupper (Cu)	0.080	0.084	0.0080	0.0084
Mercury (Hg)	< 0.0004	0.0007	<0.00004	0.00007
Molybdenum (Mo)	0.015	0.051	0.0015	0.0051
Lead (Pb)	< 0.10	0.17	<0.010	0.017
Nickel (Ni)	< 0.05	0.084	<0.005	0.0084
Selenium (Se)	< 0.007	< 0.007	<0.0007	<0.0007
Tin (Sn)	< 0.02	< 0.02	<0.002	<0.002
Vanadium (V)	2.4	27	0.24	2.7
Zinc (Zn)	< 0.02	0.07	<0.002	0.007
Fluoride (F)	36	60	3.6	6.0
Chloride (Cl)	21	86	2.1	8.6
Sulphate (SO4)	190	950	19.0	95.0
Bromide (Br)	< 0.8	< 0.8	<0.08	<0.08
Titanium (Ti)	3.0	9.2	0.30	0.92
Total Dissolved Solids (TDS)	2,360	9,050	236.0	905.0

4.3 Phase 1 BRDA Piezometer Liquid Quality

Field parameters (temperature, pH, and conductivity) are summarised in Table 6 . Laboratory analysis including alkalinity and metals are shown Table 8 and Table 9. Laboratory certificates are included in Appendix C.

4.3.1 Field pH and Conductivity

Water quality parameters (pH and conductivity) from the piezometer liquid samples were assessed in the field and the results are summarised in Table 6, below.

pH values are highly alkaline, ranging from 12 to 13 and conductivity ranges from 10.8 to 20.7 mS/cm in the Phase 1 piezometers. These piezometers are installed in older bauxite residue material that is unamended and unfarmed.

Table 6: Piezometer Liquid, Field pH and Conductivity Results

Piezometer	Depth (m)	08 June 2020		26 June 2020	
		pH	Conductivity (mS/cm)	pH	Conductivity (mS/cm)
BGT3D	12.3	13.01	19.9	13.22	20.7
2APL	9.7	12.86	10.6	12.72	10.8

Piezometer	Depth (m)	08 June 2020		26 June 2020	
		pH	Conductivity (mS/cm)	pH	Conductivity (mS/cm)
BGT3B	15.3	11.35	10.38	13.19	11.8
1APL	9.3	12.95	11.9	13.05	20.6
BGT2B	7.3	12.55	19.08	12.71	13.15
BGT2C	15.3	12.66	17.2	12.82	17.2
BGT5B		12.82	14.7	13.19	20.7
BGT5E	18.3	12.88	15.0	13.00	13.3

4.3.2 Alkalinity Analysis and Discussion

P-alkalinity and total (M) alkalinity were measured from the piezometer liquid samples and the results are summarised in Table 8 and Table 9, below. Alkalinity is a measurement of the buffering capacity of a liquid, e.g. its capacity to resist acidification. In this case, acidification is desirable to bring the high pH of the strongly alkaline piezometer liquid found in the Phase I BRDA (with pH values ranging from 12 to 13) down below pH 9, which is the legislative discharge criterion.

Total (M) Alkalinity (as $CaCO_3$) ranges from 882 mg/L to 16,380 mg/L and P alkalinity (as $CaCO_3$) ranges from 634 to 13,506 mg/L. Values over 150 mg/L for alkalinity are considered high. The determination of P and M alkalinity allow for the separate contributions to alkalinity from caustic, carbonate, and bicarbonate to be estimated using the relationship table (Table 7). For all samples, the P alkalinity is greater than half of the M alkalinity and therefore concentrations of hydroxide (OH) and carbonate (CO_3) concentrations have been calculated in accordance with these relationships: OH = 2P – M, CO_3 = 2(M-P), and HCO_3 = 0.

Table 7: Alkalinity Relationships (P & M)

If	OH (Hydroxide, Caustic) (mg/L)	CO_3 (Carbonate) (mg/L)	HCO_3 (bicarbonate) (mg/L)
P = 0	0	0	M
P < M/2	0	2P	M = 2P
P = M/2	0	2P	0
P > M/2	2P - M	2 (M-P)	0
P = M	M	0	0

4.3.3 Metals Analysis

The results of metals analysis from the piezometer liquid samples are presented in Table 8 and Table 9, below. The Site does not have limits for metal concentrations in discharge on their Industrial Emissions Licence (Reference P0035-06).

Table 8: Piezometer Liquid Metals Analysis - 08 June 2020

Piezo Ref	As	Ba	Be	B	Cd	Cr	Cu	Fe	Pb	Hg	Ni	Se	V	Zn	M-alk (total)	P-alk	Calc OH	Calc CO_3
	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	mg/L	mg/L	mg/L	mg/L
BGT3D	1896	<1.8	<0.5	527	<0.03	2.1	83	8.9	<0.4	<0.5	8.8	866.4	30,280	66,300	1676	1344	1012	664
2APL	958.4	<1.8	<0.5	777	<0.03	4.5	10	144.2	<0.4	<0.5	11.9	148.1	16,970	256	882	634	386	496
BGT3B	139.6	5.1	<0.5	112	<0.03	1.4	8	52.9	43.7	<0.5	<0.2	66.1	3,903	14,160	3226	3168	3110	116
1APL	1216	<1.8	<0.5	1083	<0.03	2.2	11	42.7	273.1	<0.5	12	847.1	1,372	104,000	1460	1146	832	628
BGT2B	525.4	<1.8	<0.5	308	<0.03	8	52	19.3	914.1	<0.5	4.2	293.9	11,710	16,540	5610	5200	4790	820
BGT2C	1366	<1.8	<0.5	382	<0.03	3.7	33	34.1	14	<0.5	11.3	198.4	10,920	47,810	976	760	544	432
BGT5B	863.6	<1.8	<0.5	461	<0.03	5.7	73	143.9	<0.4	<0.5	3.5	287.2	10,180	11,780	1086	848	610	476
BGT5E	1430	<1.8	<0.5	388	<0.03	8.9	55	13.4	<0.4	<0.5	9	1248	10,660	19,790	946	664	382	564

Table 9: Piezometer Liquid from Phase 1 BRDA, Metals Analysis – 26 June 2020

Piezo	As	Ba	Be	B	Cd	Cr	Cu	Fe	Pb	Hg	Ni	Se	Ti	V	Zn	M-alk (total)	P-alk	Calc OH	Calc CO_3
	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	µg/L	mg/L	mg/L	mg/L	mg/L
BGT3D	1,721	<1.8	<0.5	550	<0.03	5.8	59	34	<0.4	<0.5	8.9	669.7	224	18,980	63,580	14,580	11,624	8668	5912
2APL	1,261	<1.8	<0.5	2,130	1.11	56	18	1,061	<0.4	<0.5	56.5	211	165	13,010	679	9,140	6,478	3816	5324
BGT3B	267.3	<1.8	<0.5	223	<0.03	4.7	26	273.2	326.6	<0.5	2.3	130.5	11	4,982	21,160	4,903	4,178	3453	1450
1APL	1,263	<1.8	<0.5	1,145	<0.03	1.3	10	14.6	<0.4	<0.5	13.9	726.8	<5	1,532	99,820	15,200	11,956	8712	6488
BGT2B	267.7	<1.8	<0.5	184	<0.03	107.9	75	27.1	24.6	<0.5	2.2	264.8	<5	5,491	3,509	3,065	2,152	1239	1826
BGT2C	291	<1.8	<0.5	164	<0.03	19.9	40	26.2	1	<0.5	7.8	89.7	<5	6,869	3,958	3,723	2,588	1453	2270
BGT5B	1,244	<1.8	<0.5	800	<0.03	1.7	49	42.3	<0.4	<0.5	5.2	357.9	<5	14,210	24,770	16,380	13,506	10632	5748
BGT5E	1,625	<1.8	<0.5	499	<0.03	9.4	44	52.3	<0.4	<0.5	11.6	1265	19	17,580	41,480	12,160	9,610	7060	5100

5.0 CONCLUSIONS

The amended layer leachate testing demonstrates that the bauxite residue farming and amendment is successful in reducing the pH of the bauxite residue and improving the seepage water quality from the amended layer.

Piezometer water analysis from selected piezometers on the Phase 1 BRDA, representing older unfarmed, unamended bauxite residue reveals that this piezometer liquid is highly alkaline (pH circa 12-13). The buffering capacity of these piezometer liquids is very high and the alkalinity exists predominantly as hydroxide alkalinity.

This work represents the first phase of work to inform mixed water quality in the Perimeter Interceptor Channel post-closure and should be read in conjunction with 19122440.R02.A2 *Aughinish Alumina Limited: Preliminary Engineering Closure Study – Seepage Assessment for BRDA at Stage 16* dated August 2021 and 19122440.R03.A2 *Aughinish Alumina Bauxite Residue Disposal Area: Preliminary Engineering Closure Water Quality Predictions* dated August 2021.

Signature Page

Golder Associates Ireland Limited

Martha Buckwalter-Davis	Brian Keenan
Geochemist	*Project Manager*

Date: 10 August 2021

MBD/BK/ab

Registered in Ireland Registration No. 297875
Town Centre House, Dublin Road, Naas, Co. Kildare, W91 TD0P, Ireland
Directors: S. Copping, A. Harris, DRV Jones, A-L Oberg-Hogsta
VAT No.: 8297875W

APPENDIX A

Metlab Test Results

METLAB LTD

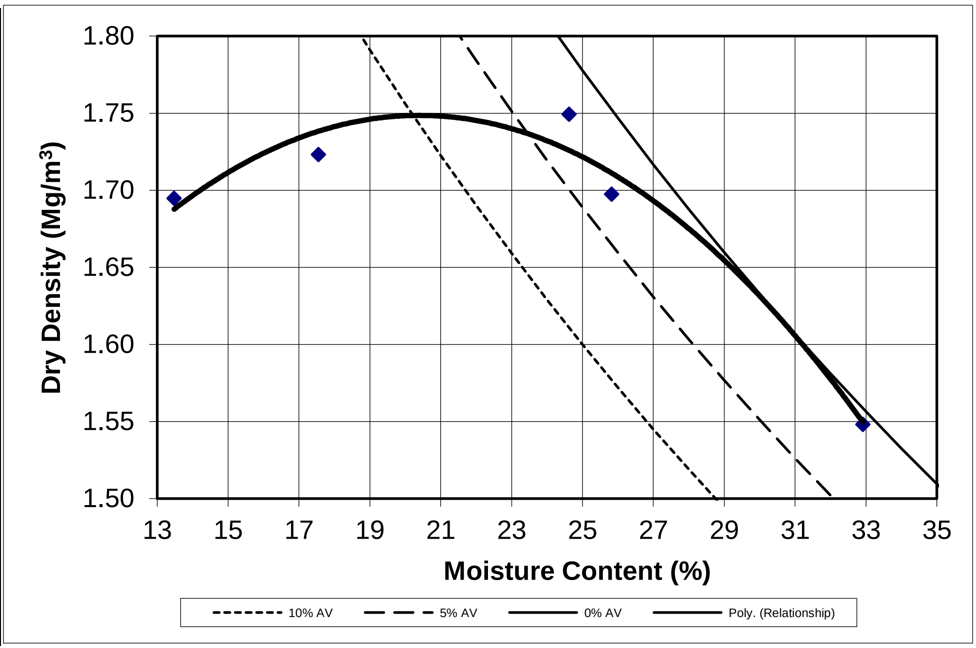
Unit 6 Airways Technology Park, Ballygarvan, Cork.
T: (021) 4311614 F: (021) 4311714 email: www.metlab.ie
Metlab Ltd - registered in Ireland (No. 84894)

DRY DENSITY/MOISTURE CONTENT RELATIONSHIP TEST REPORT
TESTED IN ACCORDANCE WITH BS 1377:PART 4:1990

Contract No	**J3451**	Report No	**9266-OMC.03**
Client	**Golder Associates**	Lab No	**9266**
Client Contact	**Mr. Brian Keenan**	Sample Type	**Bulk Disturbed**
Contract	**Tailings**	Method of Prep	**Multiple Points**
	Aughinish Alumina	Sampling Certificate	**Yes**
	Co. Limerick	Sampled By	**ROB**
Site Ref	**N/A**	Date Sampled	**26 June 2020**
Client Ref	**OMC.03**	Date Received	**6 July 2020**
Specification	**Site**	Date Started	**7 July 2020**
Location	**Cell 5**		
Sample Description	**Red Mud**		

TEST RESULTS		
Maximum Dry Density	**1.75**	**Mg/m3**
Optimum Moisture Content	**21.0**	**%**
Natural Moisture Content	**17.2**	**%**

GRAPH DATA	
Dry Density (Mg/m^3)	**Moisture Content (%)**
1.695	13.48
1.723	17.55
1.749	24.62
1.697	25.82
1.548	32.91



Notes: (1) Air voids determined using an assumed specific gravity (Gs) = 3.20 Mg/m^3
(2) % Retained on 37.5mm Sieve = 0 % Retained on 20mm Sieve = 0 %
(3) Tested using a 4.5 kg rammer in accordance with clause 3.5 of the test standard.

Remarks:

Approved

14 July 2020
Date Reported

Approval By:	
Technical Manager	■
Dep Technical Manager	☐
Laboratory Supervisor	☐

End of Test

Form No: CMT 004 Rev 2

METLAB LTD

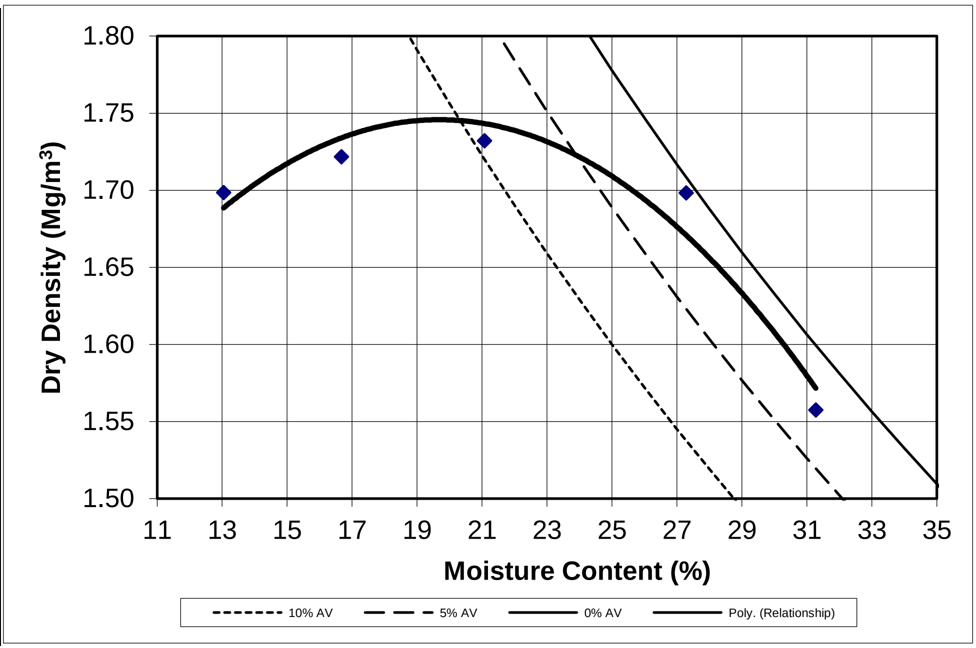
Unit 6 Airways Technology Park, Ballygarvan, Cork.
T: (021) 4311614 F: (021) 4311714 email: www.metlab.ie
Metlab Ltd - registered in Ireland (No. 84894)

DRY DENSITY/MOISTURE CONTENT RELATIONSHIP TEST REPORT
TESTED IN ACCORDANCE WITH BS 1377:PART 4:1990

Contract No	J3451	Report No	9267-OMC.04
Client	Golder Associates	Lab No	9267
Client Contact	Mr. Brian Keenan	Sample Type	Bulk Disturbed
Contract	Tailings	Method of Prep	Multiple Points
	Aughinish Alumina	Sampling Certificate	Yes
	Co. Limerick	Sampled By	ROB
Site Ref	N/A	Date Sampled	26 June 2020
Client Ref	OMC.04	Date Received	6 July 2020
Specification	Site	Date Started	7 July 2020
Location	Cell 6		
Sample Description	Farmed Bauxite Residue		

TEST RESULTS		
Maximum Dry Density	1.74	Mg/m3
Optimum Moisture Content	20.0	%
Natural Moisture Content	27.9	%

GRAPH DATA	
Dry Density (Mg/m^3)	Moisture Content (%)
1.699	13.04
1.722	16.67
1.732	21.07
1.698	27.28
1.558	31.27



Notes: (1) Air voids determined using an assumed specific gravity (Gs) = 3.20 Mg/m^3
(2) % Retained on 37.5mm Sieve = 0 % Retained on 20mm Sieve = 0 %
(3) Tested using a 4.5 kg rammer in accordance with clause 3.5 of the test standard.

Remarks:

Approved

14 July 2020
Date Reported

Approval By:	
Technical Manager	■
Dep Technical Manager	☐
Laboratory Supervisor	☐

End of Test

Form No: CMT 004 Rev 2

Permeability in a Triaxial Cell

BS 1377 : Part 6 : 1990 Clause 6

Specimen Details

Borehole		Cell 5
Sample No.		
Depth	m	
Date		16-09-20
Disturbed / Undisturbed		Undisturbed

Description of Specimen

Reddish brown fine gravelly sandy silty CLAY with some rootlets.

Initial Specimen Conditions

Height	mm	127.00
Diameter	mm	100.00
Area	mm^2	7853.98
Volume	cm^3	997.46
Mass	g	2106.13
Dry Mass	g	1670.00
Density	Mg/m^3	2.11
Dry Density	Mg/m^3	1.67
Moisture Content	%	26.1
Voids Ratio		1.031
Specific Gravity	Mg/m^3	3.40
	(assumed/measured)	assumed

Final Specimen Conditions

Moisture Content	%	27.93
Density	Mg/m^3	2.18
Dry Density	Mg/m^3	1.70

Test Setup

Date started	04-09-20
Date Finished	15-09-20
Top Drain Used	y
Base Drain Used	y
Pressure System Number	PCell 10
Cell Number	CCell 10

D P Nan

Checked and Approved By

16-09-20
Date

Client Ref

GSTL
GEO Site & Testing Services Limited

Aughinish Alumina

Contract No

UKAS TESTING 2788

50025

Permeability in a Triaxial Cell

BS 1377 : Part 6 : 1990 Clause 6

Specimen Details

Borehole		Cell 5
Sample No.		
Depth	m	
Date		16-09-20

Saturation

Cell Pressure Incr.	kPa	50.00
Back Pressure Incr.	kPa	48.00
Differential Pressure	kPa	2.00
Final Cell Pressure	kPa	250.00
Final Pore Pressure	kPa	143.00
Final B Value		0.96

Consolidation

Effective Pressure	kPa	100.00
Cell Pressure	kPa	250.00
Back Pressure	kPa	150.00
Excess Pore Pressure	kPa	100.00
Pore Pressure at End	kPa	150.00
Consolidated Volume	cm^3	980.86
Consolidated Height	mm	126.30
Consolidated Area	mm^2	7766.84
Vol. Compressibility	m^2/MN	2.5139
Consolidation Coef.	$m^2/yr.$	0.1664
Final Voids Ratio		0.997

Permeability

Cell Pressure	kPa	250.00
Effective Cell Pressure	kPa	100.00
Back Pressure Diff.	kPa	20.00
Mean Rate of Flow	ml/min	0.00079
Average Temperature	'C	20

Vertical Permeability Kv	**m/s**	**1.04 x 10-10**

D P Evans

Checked and Approved By

16-09-20
Date

Client Ref

GSTL
GEO Site & Testing Services Limited

Aughinish Alumina

Contract No

50025

UKAS TESTING 2788

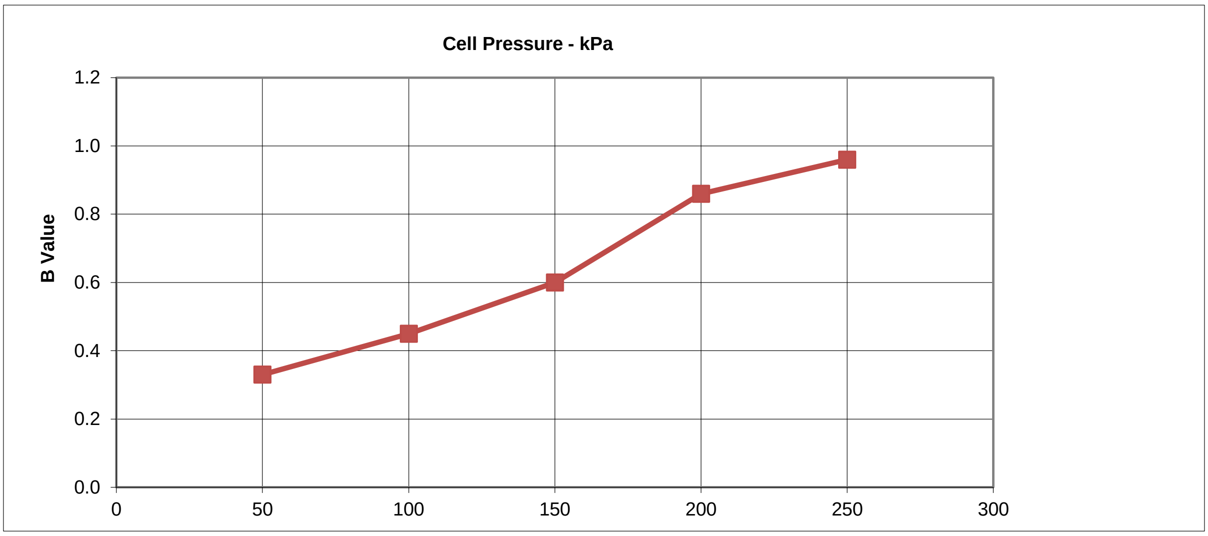
Permeability in a Triaxial Cell

BS 1377 : Part 6 : 1990 Clause 6

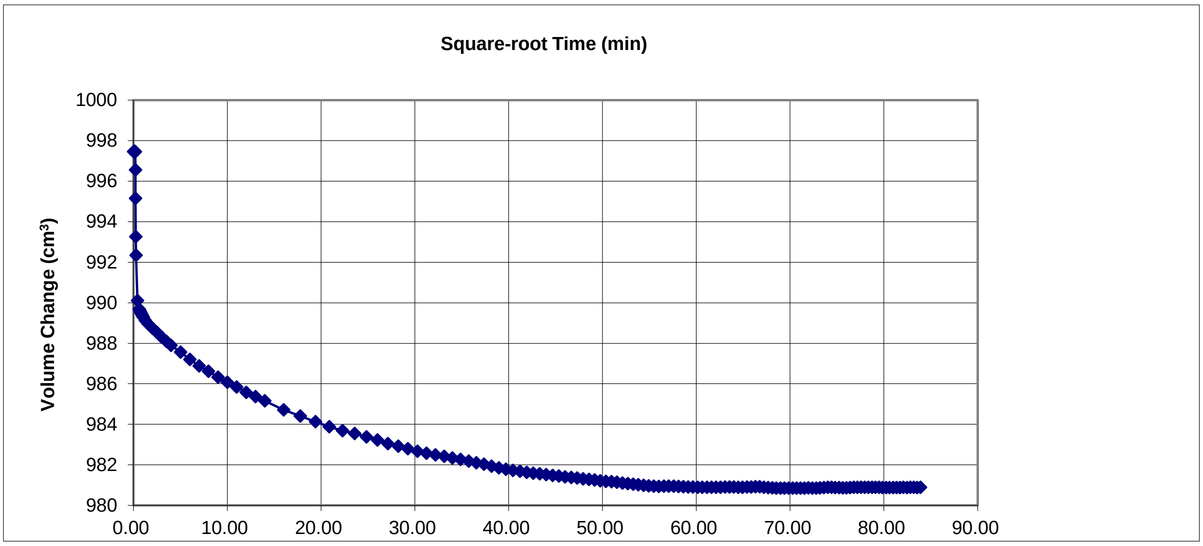
Specimen Details

Borehole		Cell 5
Sample No.		
Depth	m	
Date		16-09-20

Saturation Stage



Consolidation Stage



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Checked and Approved By

16-09-20
Date

Client Ref

GSTL
GEO Site & Testing Services Limited

Aughinish Alumina

Contract No

50025

UKAS TESTING 2788

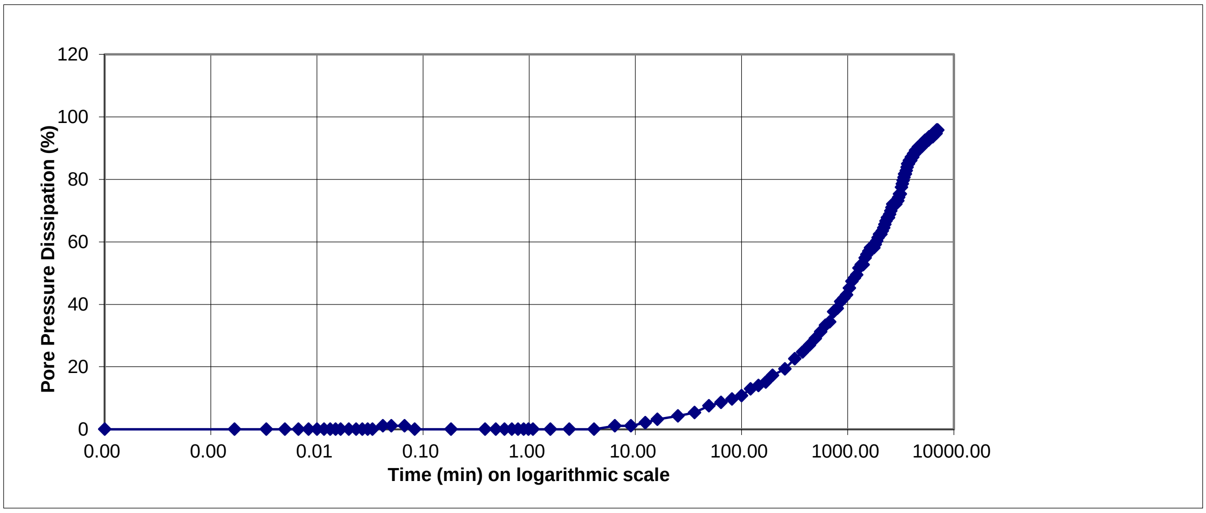
Permeability in a Triaxial Cell

BS 1377 : Part 6 : 1990 Clause 6

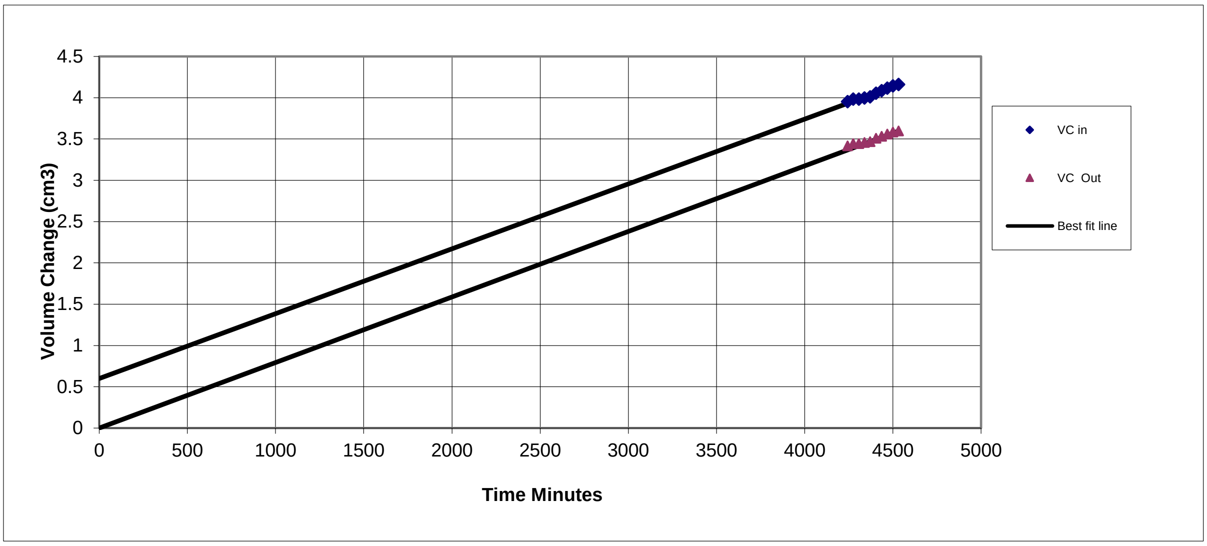
Specimen Details

Borehole		Cell 5
Sample No.		
Depth	m	
Date		16-09-20

Consolidation Stage



Permeability Stage



DP Gray

Checked and Approved By

16-09-20
Date

Client Ref

Contract No

Aughinish Alumina

50025

GSTL
GEO Site & Testing Services Limited

UKAS TESTING 2788

Permeability in a Triaxial Cell

BS 1377 : Part 6 : 1990 Clause 6

Specimen Details

Borehole		Cell 6
Sample No.		
Depth	m	
Date		16-09-20
Disturbed / Undisturbed		Undisturbed

Description of Specimen

Reddish brown fine gravelly sandy silty CLAY

Initial Specimen Conditions

Height	mm	130.00
Diameter	mm	100.50
Area	mm^2	7932.72
Volume	cm^3	1031.25
Mass	g	1997.55
Dry Mass	g	1638.76
Density	Mg/m^3	1.94
Dry Density	Mg/m^3	1.59
Moisture Content	%	21.9
Voids Ratio		1.140
Specific Gravity	Mg/m^3	3.40
	(assumed/measured)	assumed

Final Specimen Conditions

Moisture Content	%	23.85
Density	Mg/m^3	1.99
Dry Density	Mg/m^3	1.61

Test Setup

Date started	04-09-20
Date Finished	15-09-20
Top Drain Used	y
Base Drain Used	y
Pressure System Number	PCell 11
Cell Number	CCell 11

DP Glass

Checked and Approved By

16-09-20
Date

Client Ref

Aughinish Alumina

Contract No

50025

GSTL
GEO Site & Testing Services Limited

UKAS TESTING 2788

Permeability in a Triaxial Cell

BS 1377 : Part 6 : 1990 Clause 6

Specimen Details

Borehole		Cell 6
Sample No.		
Depth	m	
Date		16-09-20

Saturation

Cell Pressure Incr.	kPa	50.00
Back Pressure Incr.	kPa	48.00
Differential Pressure	kPa	2.00
Final Cell Pressure	kPa	200.00
Final Pore Pressure	kPa	190.00
Final B Value		0.96

Consolidation

Effective Pressure	kPa	100.00
Cell Pressure	kPa	200.00
Back Pressure	kPa	100.00
Excess Pore Pressure	kPa	98.00
Pore Pressure at End	kPa	100.00
Consolidated Volume	cm^3	1018.35
Consolidated Height	mm	129.46
Consolidated Area	mm^2	7866.56
Vol. Compressibility	m^2/MN	1.7941
Consolidation Coef.	$m^2/yr.$	0.1276
Final Voids Ratio		1.113

Permeability

Cell Pressure	kPa	200.00
Effective Cell Pressure	kPa	100.00
Back Pressure Diff.	kPa	20.00
Mean Rate of Flow	ml/min	0.00073
Average Temperature	'C	20

Vertical Permeability Kv	**m/s**	**9.77 x 10-11**

DP Grans

Checked and Approved By

16-09-20
Date

Client Ref

GSTL
GEO Site & Testing Services Limited

Aughinish Alumina

Contract No

50025

UKAS TESTING 2788

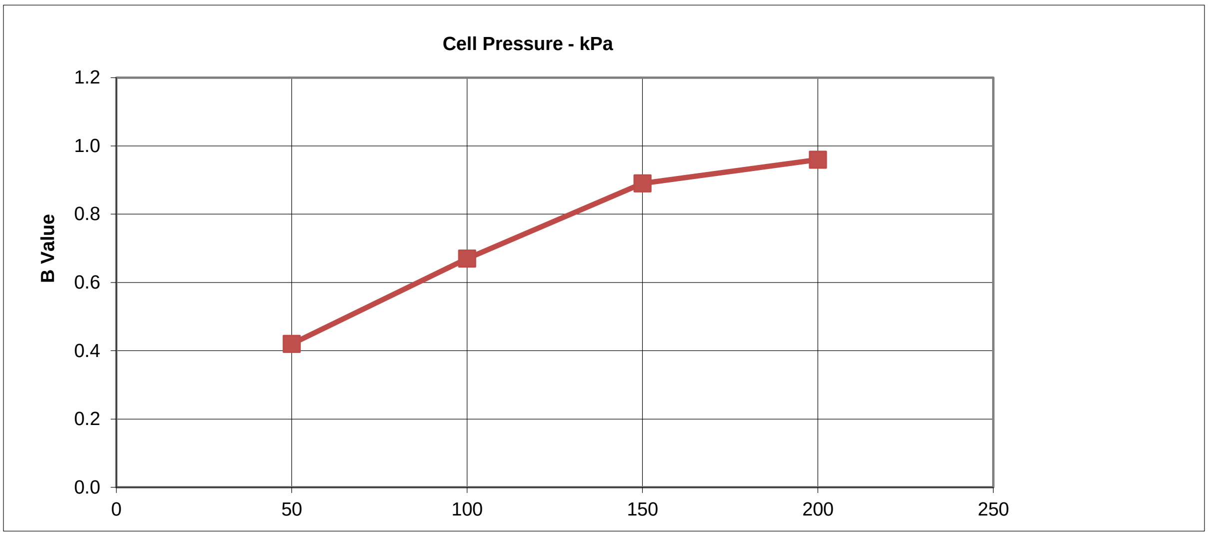
Permeability in a Triaxial Cell

BS 1377 : Part 6 : 1990 Clause 6

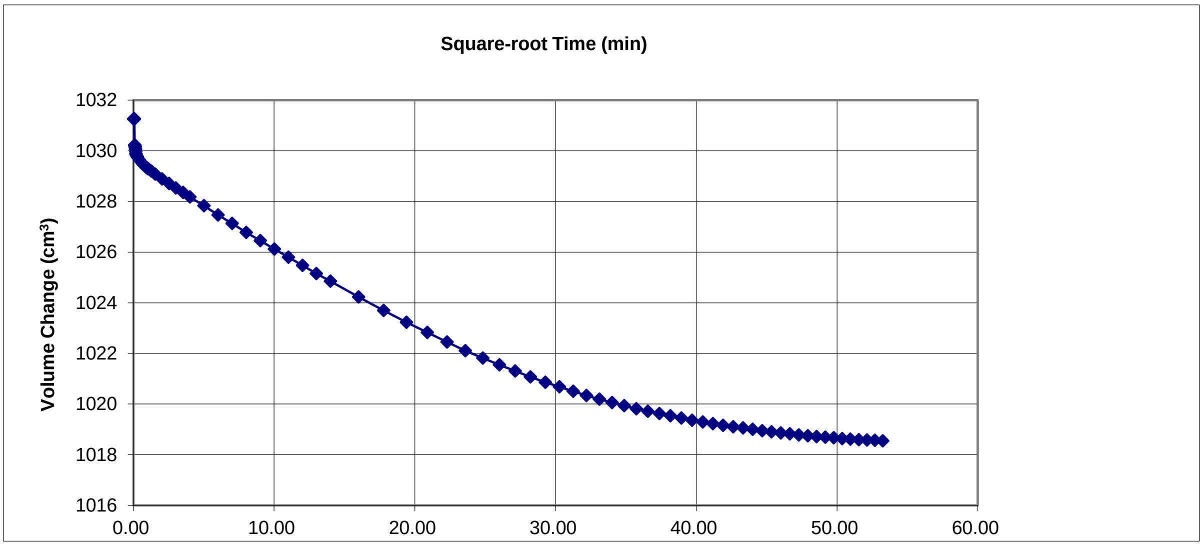
Specimen Details

Borehole	Cell 6
Sample No.	
Depth m	
Date	16-09-20

Saturation Stage



Consolidation Stage



DP Grang

Checked and Approved By

16-09-20
Date

Client Ref

GSTL
GEO Site & Testing Services Limited

Aughinish Alumina

Contract No

50025

UKAS TESTING 2788

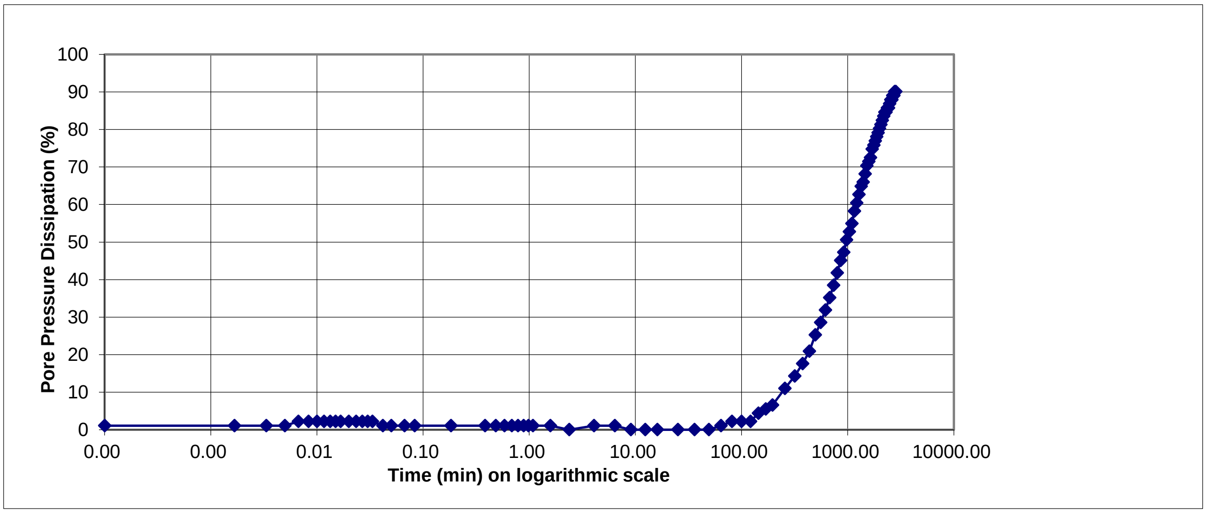
Permeability in a Triaxial Cell

BS 1377 : Part 6 : 1990 Clause 6

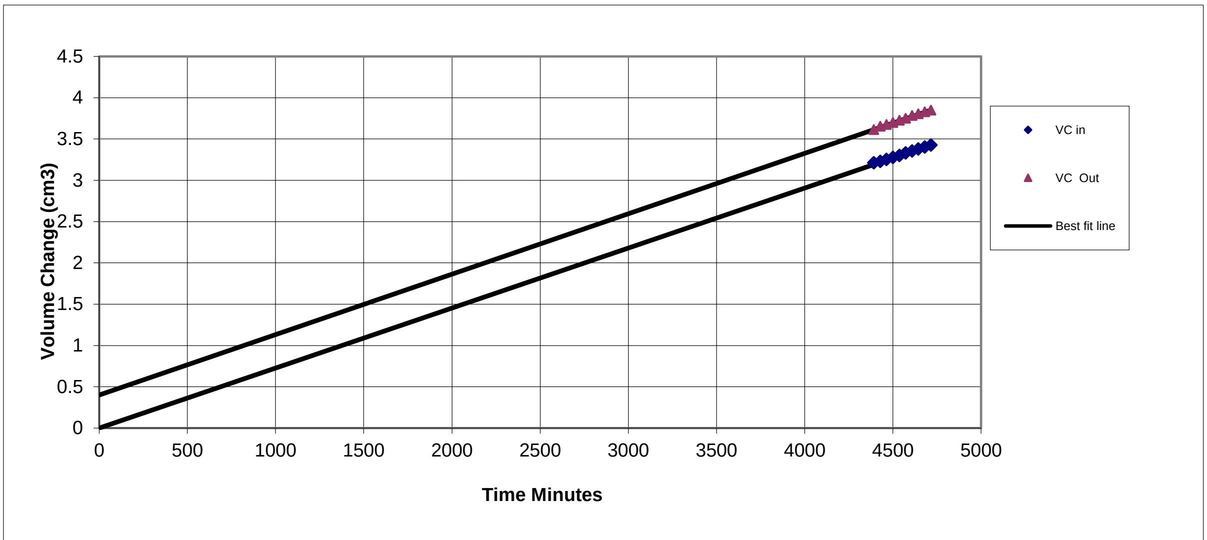
Specimen Details

Borehole		Cell 6
Sample No.		
Depth	m	
Date		16-09-20

Consolidation Stage



Permeability Stage



D P Wang

Checked and Approved By

16-09-20
Date

Client Ref

GSTL
GEO Site & Testing Services Limited

Aughinish Alumina

Contract No

50025

UKAS TESTING 2788

METLAB

Metlab Ltd: Unit 20/21 Finglas Business Park, Tolka Valey Road, Finglas, Dublin 11, Ireland
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Web:www.metlab.ie

CORE CUTTER DENSITY DETERMINATION TEST REPORT
TESTED IN ACCORDANCE WITH BS 1377:PART 9:1990 METHOD 2.4

Contract No	**J3451**	Report No	**J3451-CD.10**
Client	**Mr. Brian Keenan**	Date	**26 June 2020**
Client Contact	**Golder Associates**	Sample Type	**Undisturbed Core**
Contract	**Aughinish Alumina**	Method of Prep	**N/A**
	Co. Limerick	Groundwater Present	**No**
		Sampling Certificate	**Yes**
Specification	**Project Specification**	Sample Description	**Red Mud**

TEST RESULT

Lab Number:	CD:	Site Ref:	Location:	Bulk Density (Mg/m3)	Dry Density (Mg/m3)	Moisture Content (%)
9266	10	N/A	Cell 5	2.11	1.67	26.4%

Remarks:

Approved

14 July 2020
Date Reported

Approval By:	
Technical Manager	■
Deputy Technical Manager	☐
Laboratory Supervisor	☐

End of Test

Form No: CMT 020 Rev 0

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Web:www.metlab.ie

Contract No	**J3451**	Report No	**J3451-NDM.01-02**
Client	**Golder Associates**	Tested By	**Ryan O Brien**
Client Contact	**Mr. Brian Keenan**	Date Tested	**Indicated per Test**
Contract	**Aughinish Alumina**	Gauge Model	**Troxler 3440**
	Co. Limerick	Gauge Serial No	**19025**
		Proctor (Kg/m^3)	**1750**
Material	**Red Mud/Farmed Bauxite Residue**	Sheet No	**1 of 1**

Bulk Density Corection Factor	**1.01**	Moisture Content Correction Factor	**1.01**

NDM NO	**1**	Test Date:	26 June 2020		*LOCATION* :		Cell 5		
Site Ref	**N/A**	DEPTH (mm)	BULK DENSITY (Kg/m^3)		DRY DENSITY (Kg/m^3)		MOISTURE CONTENT(%)		Corrected Compaction (%)
Measurement Ref				Corrected		Corrected		Corrected	
A		250	2089	2110	1655	1668	26.2	26.5	95.3
B			2101	2122	1660	1673	26.6	26.8	95.6
C			2091	2112	1663	1676	25.7	26.0	95.8
AVERAGE FOR GROUP			2094	**2115**	1659	**1672**	26.2	**26.4**	**95.6**

NDM NO	**2**	Test Date:	26 June 2020		*LOCATION* :		Cell 6		
Site Ref	**N/A**	DEPTH (mm)	BULK DENSITY (Kg/m^3)		DRY DENSITY (Kg/m^3)		MOISTURE CONTENT(%)		Corrected Compaction (%)
Measurement Ref				Corrected		Corrected		Corrected	
A		250	1925	1944	1580	1593	21.8	22.1	91.0
B			1917	1936	1574	1587	21.8	22.0	90.7
C			1911	1930	1570	1583	21.7	21.9	90.5
AVERAGE FOR GROUP			1918	**1937**	1575	**1588**	21.8	**22.0**	**90.7**

Remarks:

Ryan O Brien
Approved

14 July 2020
Date Reported

Approval By:	
Laboratory Manager	☐
Field CQA Engineer	■

End of Test

Form No: CMT 007 Rev 0

APPENDIX B

SGS Intron Amended Layer Leach Results

Leaching properties of red mud

Report SGS INTRON B.V.

Status: Final report (Revision a)
Date: 9 August 2021
Document number: A117360/R20201136a

WHEN YOU NEED TO BE SURE

Colophon

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Golder Associates Ireland Limited
attn Mrs. M. Buckwalter-Davis
Town Centre House, Duplin Road
W91 TDOP NAAS , CO. KILDARE

Email address:
Martha_Buckwalter-Davis@golder.com

Quotation:
A117360/Leaching properties of red mud

Date:
20 April 2020

Purchase order:
Purchase order nr. GAIRL 001575

Date:
28 May 2020

Order taker:
SGS INTRON B.V.

Contact:
Huub Creuwels

Telephone number:
+31882145290

Mobile number:
+31653725899

Email address:
huub.creuwels@sgs.com

Author:
ing. H.J.M.A. Creuwels

Signature:

Authorizer:
Ir. R. Leppers

Signature:

Date:
12 October 2020 (final report)
9 August 2021 (revision version a)

Reason of change:

At the request of the client revision to an analysis report.

Disclaimer

Contents

1. Samples

Golder Associates Ireland Limited sent 2 samples of red mud for a leaching test according to EN 14405.

2. Executed work

In order to determine the leaching behaviour of this waste material the harmonised European standard EN 14405 (up flow percolation test) was used. This percolation test is a 7-stage column test to give information on the leaching characteristics of waste. The 7 individual eluates are collected at a cumulative liquid to solid ratio of 0.1 – 0.2 - 0.5 – 1.0 – 2.0 – 5.0 and 10.0. In the 7 individual eluate fractions the pH and conductivity are analysed. In the cumulative mixed sample from the 7 eluate fractions the concentrations of the 15 heavy metals: antimony (Sb), arsenic (As), barium (Ba), cadmium (Cd), chromium (Cr), cobalt (Co), copper (Cu), mercury (Hg), molybdenum (Mo), nickel (Ni), selenium (Se), lead (Pb), tin (Sn), vanadium (V) zinc (Zn) and the 4 anions fluoride (F), chloride (Cl), bromide (Br), sulphate (SO_4) is analysed and the emission is calculated. In addition to these parameters the emission of titanium (Ti) and the amount of total dissolved solids is analysed.

The column tests were performed on the samples designated as red mud cell 5 amended (sample 1) and mud cell 6 amended (sample 2)

The development of pH and conductivity during the 7-stage column test are presented in figure 1 and 2. The emission of the different elements are presented in table 1.

3. Results

Both column tests percolated successfully. There is no difference in permeability compared to aggregates with a fineness D_{95} < 4mm.

Below you find a summary of the results:

- There is a significant difference in alkalinity between both red mud samples, see figure 1. This will result in difference in leaching for the pH dependent elements. Red mud cell 6 has a higher alkalinity than cell 5
- There is a significant difference in conductivity between both red mud samples, see figure 2. The higher conductivity is caused by higher concentrations of substances present in the leached eluates.

Figure 1: Alkalinity of the 2 samples red mud during the 7-stage column test.

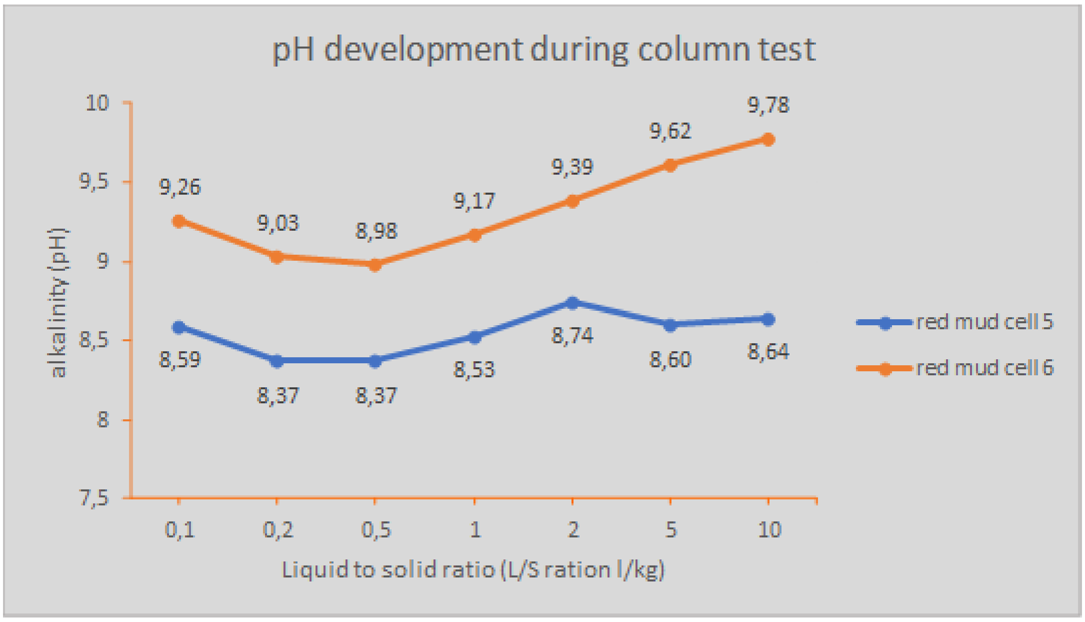

Figure 2: Conductivity of the 2 samples red mud during the 7-stage column test.

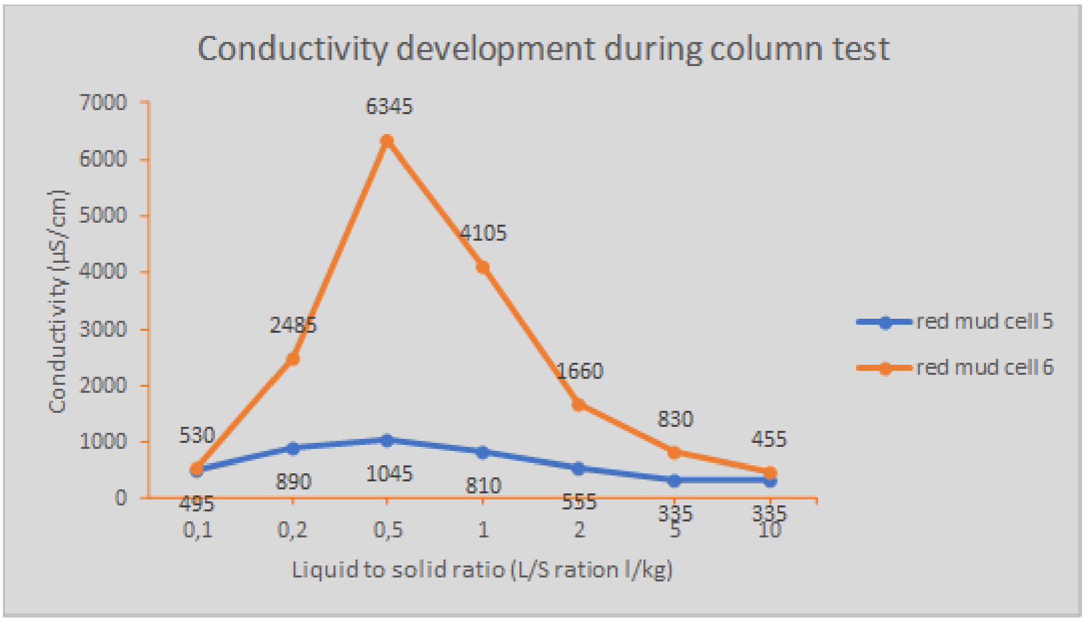

Table 1: Emission of the tested elements of the red mud samples.

Parameter	Emission (mg/kg d.m.)	
	Red mud Cell 5 amended	Red mud Cell 6 amended
Dry matter (% m/m)	77.5	78.2
Antimony (Sb)	< 0.004	0.015
Arsenic (As)	0.069	0.068
Barium (Ba)	< 0.06	< 0.06
Cadmium (Cd)	< 0.001	0.0032
Chromium (Cr)	< 0.10	0.25
Cobalt (Co)	< 0.03	< 0.03
Cupper (Cu)	0.080	0.084
Mercury (Hg)	< 0.0004	0.0007
Molybdenum (Mo)	0.015	0.051
Lead (Pb)	< 0.10	0.17
Nickel (Ni)	< 0.05	0.084
Selenium (Se)	< 0.007	< 0.007
Tin (Sn)	< 0.02	< 0.02
Vanadium (V)	2.4	27
Zinc (Zn)	< 0.02	0.07
Fluoride (F)	36	60
Chloride (Cl)	21	86
Sulphate (SO_4)	190	950
Bromide (Br)	< 0.8	< 0.8
Titanium (Ti)	3.0	9.2
Total dissolved solids (TDS)*	2360	9050

*The results of total dissolved solids are determined after filtration through a 45µm filter. After filtration the eluate was still turbid due to particles < 45µm. The results of the total dissolved solids are included the particles < 45µm.

Appendix A. Analysis report 20.2656b

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tel: +31 (0) 88 - 2 145 204

Analyserapport

Revisie b vervangt revisie a d.d. 25-9-2020

SGS INTRON B.V.
t.a.v. de heer ing. H.J.M.A. Creuwels
Postbus 5187
6130 PD SITTARD
Nederland

Datum	:	08-10-2020
Betreft	:	Leaching behavior of red mud
Uw code	:	A117360
Laboratoriumnummer	:	202656b
Monsterneming	:	26-6-2020 te Ireland door Opdrachtgever
Periode onderzoek	:	17-07-2020 t/m 08-10-2020

MONSTERGEGEVENS

Monsternummer	Monstertype	Monstercode	Acceptatiedatum
1	red mud	Red mud cell 5 amended	17-07-2020
2	red mud	Red mud cell 6 amended	17-07-2020
171	eluaat kolomtest	171	17-07-2020
172	eluaat kolomtest	172	17-07-2020
173	eluaat kolomtest	173	17-07-2020
174	eluaat kolomtest	174	17-07-2020
175	eluaat kolomtest	175	17-07-2020
176	eluaat kolomtest	176	17-07-2020
177	eluaat kolomtest	177	17-07-2020
178	eluaat kolomtest	178 mengextract	17-07-2020
271	eluaat kolomtest	271	17-07-2020
272	eluaat kolomtest	272	17-07-2020
273	eluaat kolomtest	273	17-07-2020
274	eluaat kolomtest	274	17-07-2020
275	eluaat kolomtest	275	17-07-2020
276	eluaat kolomtest	276	17-07-2020
277	eluaat kolomtest	277	17-07-2020
278	eluaat kolomtest	278 mengextract	17-07-2020

Opgesteld door: ing. A. Meijs
accountmanager

Geautoriseerd door: ing. W. Ubachs
accountmanager

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ANALYSEMETHODEN

Analyse	Analysetechniek	Methode	Q	u
Antimoon	AAS hydride generatie	NVN 7323 (1997), AP04-E-XIII	Q	
Arseen	ICP	NEN 6966, AP04-E-V	Q	
Barium	ICP	NEN 6966, AP04-E-X	Q	
Breken < 4 mm		AP04		
Bromide	HPLC	NEN-EN-ISO 10304-1, AP04-E-XVII, NEN-EN 16192	Q	
Cadmium	ICP	NEN 6966, AP04-E-II	Q	
Chloride	HPLC	NEN-EN-ISO 10304-1, AP04-E-XVII, NEN-EN 16192	Q	
Chroom totaal	ICP	NEN 6966, AP04-E-VI	Q	
Conserveren		Eigen methode		
Droge stof 105°C analysemonster kolomproef	gravimetrie	AP04-V	Q	
Fluoride	HPLC	NEN-EN-ISO 10304-1, AP04-E-XVIII, NEN-EN 16192	Q	
Geleidbaarheid 25°C	conductometrie	AP04-U-V, gelijkwaardig aan NEN-EN 16192 (NEN-ISO 7888)	Q	
Kobalt	ICP	NEN 6966, AP04-E-XII	Q	
Kolomproef 7 fracties	Kolomproef	EN 14405		
Koper	ICP	NEN 6966, AP04-E-VII	Q	
Kwik	koude damp AAS	NEN 7324 (2001), AP04-E-VIII	Q	
Lood	ICP	NEN 6966, AP04-E-I	Q	
Molybdeen	ICP	NEN 6966, AP04-E-IX	Q	
Nikkel	ICP	NEN 6966, AP04-E-IV	Q	
pH	potentiometrie	NEN-ISO 10523, AP04-U-IV	Q	
Seleen	AAS hydride generatie	NVN 7323 (1997), AP04-E-XIV	Q	
Sulfaat	HPLC	NEN-EN-ISO 10304-1, AP04-E-XVII, NEN-EN 16192	Q	
TDS (Total Dissolved Solids)	gravimetrie	Eigen methode		
Tin	ICP	NEN 6966, AP04-E-XI	Q	
Titaan	ICP	Eigen methode		u
Vanadium	ICP	NEN 6966, AP04-E-XV	Q	
Zink	ICP	NEN 6966, AP04-E-III	Q	

Q = geaccrediteerd door RvA, u = uitbesteed bij onderaannemer, Qu = geaccrediteerd bij de onderaannemer

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RESULTATEN

leaching test EN14405

Sample description:	Red mud
Sampling location:	Ireland

Client:	SGS INTRON B.V.
Labnumber:	202656-1
Date:	24-8-2020

Data sample		
dry mass m_d	691	[g]
maximum particle size	4	[mm]
percentage particle < 4 mm	100	[% m/m]

Data eluates		
flow rate:		[ml/hr]
temperature:	20 ± 2	[°C]

Fraction		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**mix**
L/S ratio:	[l/kg]	0,1	0,2	0,5	1,0	2,0	5,0	10,0	
acidity:	[pH]	8,6	8,4	8,4	8,5	8,7	8,6	8,6	
conductivity:	[µS/cm]	495	890	1045	810	555	335	215	

Analyse	Eenheid	178	1 (mg/kg d.s.)
Antimoon	µg/l	< 0,4	< 0,004
Arseen	µg/l	6,9	0,069
Barium	µg/l	< 60	< 0,060
Cadmium	µg/l	< 0,10	< 0,0010
Kobalt	µg/l	< 3,0	< 0,03
Chroom	µg/l	< 10	< 0,10
Koper	µg/l	8,0	0,08
Kwik	µg/l	< 0,04	< 0,0004
Molybdeen	µg/l	1,5	0,015
Nikkel	µg/l	< 5,0	< 0,05
Lood	µg/l	< 10	< 0,10
Seleen	µg/l	< 0,7	< 0,007
Tin	µg/l	< 2,0	< 0,02
Titaan	µg/l	300	3,0
Vanadium	µg/l	240	2,40
Zink	µg/l	< 20	< 0,02
Fluoride vrij	µg/l	3600	36
Chloride	µg/l	2100	21
Bromide	µg/l	< 80	< 0,08
Sulfaat	µg/l	19000	190
TDS	µg/l	236000	2360

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leaching test EN14405

Sample description:	red mud
Sampling location:	Ireland

Client:	SGS INTRON B.V.
Labnumber:	202656-2
Date:	24-8-2020

Data sample

dry mass m_d	695	[g]
maximum particle size	4	[mm]
percentage particle < 4 mm	100	[% m/m]

Data eluates

flow rate:		[ml/hr]
temperature:	20 ± 2	[°C]

Fraction		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**mix**
L/S ratio:	[l/kg]	0,1	0,2	0,5	1,0	2,0	5,0	10,0	
acidity:	[pH]	9,3	9,0	9,0	9,2	9,4	9,6	9,8	
conductivity:	[µS/cm]	530	2485	6345	4105	1660	830	455	

Analyse	Eenheid	278	2 (mg/kg d.s.)
Antimoon	µg/l	1,5	< 0,004
Arseen	µg/l	68	0,068
Barium	µg/l	< 60	< 0,060
Cadmium	µg/l	0,32	0,0032
Kobalt	µg/l	< 3,0	< 0,03
Chroom	µg/l	25	0,25
Koper	µg/l	84	0,084
Kwik	µg/l	0,07	0,0007
Molybdeen	µg/l	5,1	0,051
Nikkel	µg/l	8,4	0,084
Lood	µg/l	17	0,17
Seleen	µg/l	< 0,7	< 0,007
Tin	µg/l	< 2,0	< 0,02
Titaan	µg/l	920	9,2
Vanadium	µg/l	2700	27
Zink	µg/l	70	0,07
Fluoride vrij	µg/l	6000	60
Chloride	µg/l	8600	86
Bromide	µg/l	< 80	< 0,08
Sulfaat	µg/l	95000	950
TDS	µg/l	905000	9050

kwaliteitswaarborg bodembeheer SIKB AP04

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OPMERKINGEN BIJ DEZE REVISIE B:

Omrekening van µg/l naar mg/kg d.s. toegevoegd. TDS (Total Dissolved Solids) en Titaan zijn bijgevoegd.

INFORMATIE OVER DE GESCHIKTHEID VAN DE MONSTERS VOOR ANALYSE

SGS INTRON is conform internationale voorschriften (NEN-EN-ISO/IEC 17025) verplicht te controleren of aangeboden monsters geschikt zijn voor het beoogde onderzoek en moet borgen dat monsters niet achteruit gaan voordat het gehalte is zekergesteld. Het vereist daarom ook dat de leveranciers van monsters ze tijdig en op een juiste wijze verpakt en geconserveerd aanleveren bij het laboratorium.

Er zijn geen verschillen met de richtlijnen geconstateerd die mogelijk de betrouwbaarheid van de resultaten van onderstaande monsters of analyses hebben beïnvloed.

Het monster is niet geconserveerd aangeleverd.

Betreft monsters:	

Het monster is voor de volgende analyse in een ongeschikte verpakking aangeleverd.

Betreft monsters:	

De conserveringstermijn is voor de volgende analyse overschreden.

Analyse(s)	monster(s)

DISCLAIMER

- Dit rapport mag zonder schriftelijke toestemming van het SGS INTRON laboratorium uitsluitend in zijn geheel worden gereproduceerd.
- De resultaten hebben uitsluitend betrekking op de onderzochte monsters.
- De NEN-EN-ISO/IEC 17025 accreditatie omvat alle resultaten behorende bij analyses die bij analysemethoden met een Q zijn gemarkeerd.
- De meetonzekerheid van de gerapporteerde resultaten en overige prestatiekenmerken kunt u opvragen bij SGS INTRON
- Op verzoek kan een lijst van de geaccrediteerde analysemethodes opgevraagd worden, welke de relatie (conform, gelijkwaardig, eigen methode) met de onderliggende norm beschrijft.

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ABOUT SGS

SGS is the world's leading inspection, verification, testing and certification company and is recognized as the global benchmark for quality and integrity. With more than 89.000 employees, SGS operates a network of over 2.600 offices and laboratories around the world.

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APPENDIX C

Element Deeside Piezometer Liquid Results

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F: +44 (0) 1244 833781

W: www.element.com

Golder Associates Ltd
Town Centre House
Dublin Road
Naas
Co Kildare
Ireland

Attention :	Martha Buckwalter-Davis
Date :	24th June, 2020
Your reference :	19132440
Our reference :	Test Report 20/7679 Batch 1
Location :	AAL
Date samples received :	16th June, 2020
Status :	Final report
Issue :	1

Eight samples were received for analysis on 16th June, 2020 of which eight were scheduled for analysis. Please find attached our Test Report which should be read with notes at the end of the report and should include all sections if reproduced. Interpretations and opinions are outside the scope of any accreditation, and all results relate only to samples supplied.
All analysis is carried out on as received samples and reported on a dry weight basis unless stated otherwise. Results are not surrogate corrected.

Authorised By:

S. Gomery

Simon Gomery BSc
Project Manager

Element Materials Technology Environmental UK Limited
Registered in England and Wales
Registered Office: 10 Lower Grosvenor Place, London, SW1W 0EN
Company Registration No: 11371415

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis
EMT Job No: 20/7679

Report : **Liquid**

Liquids/products: V=40ml vial, G=glass bottle, P=plastic bottle
H=H_2SO_4, Z=ZnAc, N=NaOH, HN=HN0_3

Please see attached notes for all abbreviations and acronyms

EMT Sample No.	1-4	5-8	9-12	13-16	17-20	21-24	25-28	29-32			LOD/LOR	Units	Method No.
Sample ID	BGT3D	2APL	BGT3B	1APL	BGT2B	BGT2C	BGT5B	BGT5E					
Depth													
COC No / misc													
Containers	V HN	V HN	V HN	V HN	V HN	V HN	V HN	V HN					
Sample Date	08/06/2020	08/06/2020	08/06/2020	08/06/2020	08/06/2020	08/06/2020	08/06/2020	08/06/2020					
Sample Type	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid					
Batch Number	1	1	1	1	1	1	1	1					
Date of Receipt	16/06/2020	16/06/2020	16/06/2020	16/06/2020	16/06/2020	16/06/2020	16/06/2020	16/06/2020					
Dissolved Arsenic	1896.0	958.4	139.6	1216.0	525.4	1366.0	863.6	1430.0			<0.9	ug/l	TM30/PM14
Dissolved Barium	<1.8	<1.8	5.1	<1.8	<1.8	<1.8	<1.8	<1.8			<1.8	ug/l	TM30/PM14
Dissolved Beryllium	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5			<0.5	ug/l	TM30/PM14
Dissolved Boron	527	777	112	1083	308	382	461	388			<12	ug/l	TM30/PM14
Dissolved Cadmium	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03			<0.03	ug/l	TM30/PM14
Total Dissolved Chromium	2.1	4.5	1.4	2.2	8.0	3.7	5.7	8.9			<0.2	ug/l	TM30/PM14
Dissolved Copper	83	10	8	11	52	33	73	55			<3	ug/l	TM30/PM14
Total Dissolved Iron	8.9	144.2	52.9	42.7	19.3	34.1	143.9	13.4			<4.7	ug/l	TM30/PM14
Dissolved Lead	<0.4	<0.4	43.7	273.1	914.1	14.0	<0.4	<0.4			<0.4	ug/l	TM30/PM14
Dissolved Mercury	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5			<0.5	ug/l	TM30/PM14
Dissolved Nickel	8.8	11.9	<0.2	12.0	4.2	11.3	3.5	9.0			<0.2	ug/l	TM30/PM14
Dissolved Selenium	866.4$_{AA}$	148.1	66.1	847.1$_{AA}$	293.9	198.4	287.2	1248.0$_{AA}$			<1.2	ug/l	TM30/PM14
Dissolved Vanadium	30280.0$_{AA}$	16970.0$_{AA}$	3903.0$_{AA}$	1372.0	11710.0$_{AA}$	10920.0$_{AA}$	10180.0$_{AA}$	10660.0$_{AA}$			<0.6	ug/l	TM30/PM14
Dissolved Zinc	66300.0$_{AA}$	256.2	14160.0$_{AA}$	104000.0$_{AA}$	16540.0$_{AA}$	47810.0$_{AA}$	11780.0$_{AA}$	19790.0$_{AA}$			<1.5	ug/l	TM30/PM14
Total Alkalinity as CaCO3	1676	882	3226	1460	5610	976	1086	946			<1	mg/l	TM75/PM0
P Alkalinity as CaCO3	1344	634	3168	1146	5200	760	848	664			<1	mg/l	TM75/PM0
Electrical Conductivity @25C	22110	12765	6642	23002	11570	19006	17106	21175			<2	uS/cm	TM76/PM0
pH	13.12	12.84	12.73	13.13	12.77	12.96	13.00	12.80			<0.01	pH units	TM73/PM0

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis

EMT Job No.	Batch	Sample ID	Depth	EMT Sample No.	Analysis	Reason
No deviating sample report results for job 20/7679						

Please note that only samples that are deviating are mentioned in this report. If no samples are listed it is because none were deviating.
Only analyses which are accredited are recorded as deviating if set criteria are not met.

EMT Job No.: 20/7679

SOILS

Please note we are only MCERTS accredited (UK soils only) for sand, loam and clay and any other matrix is outside our scope of accreditation.

Where an MCERTS report has been requested, you will be notified within 48 hours of any samples that have been identified as being outside our MCERTS scope. As validation has been performed on clay, sand and loam, only samples that are predominantly these matrices, or combinations of them will be within our MCERTS scope. If samples are not one of a combination of the above matrices they will not be marked as MCERTS accredited.

It is assumed that you have taken representative samples on site and require analysis on a representative subsample. Stones will generally be included unless we are requested to remove them.

All samples will be discarded one month after the date of reporting, unless we are instructed to the contrary.

If you have not already done so, please send us a purchase order if this is required by your company.

Where appropriate please make sure that our detection limits are suitable for your needs, if they are not, please notify us immediately.

All analysis is reported on a dry weight basis unless stated otherwise. Limits of detection for analyses carried out on as received samples are not moisture content corrected. Results are not surrogate corrected. Samples are dried at 35°C ±5°C unless otherwise stated. Moisture content for CEN Leachate tests are dried at 105°C ±5°C.

Where Mineral Oil or Fats, Oils and Grease is quoted, this refers to Total Aliphatics C10-C40.

Where a CEN 10:1 ZERO Headspace VOC test has been carried out, a 10:1 ratio of water to wet (as received) soil has been used.

% Asbestos in Asbestos Containing Materials (ACMs) is determined by reference to HSG 264 The Survey Guide - Appendix 2 : ACMs in buildings listed in order of ease of fibre release.

Sufficient amount of sample must be received to carry out the testing specified. Where an insufficient amount of sample has been received the testing may not meet the requirements of our accredited methods, as such accreditation may be removed.

Negative Neutralization Potential (NP) values are obtained when the volume of NaOH (0.1N) titrated (pH 8.3) is greater than the volume of HCl (1N) to reduce the pH of the sample to 2.0 - 2.5. Any negative NP values are corrected to 0.

The calculation of Pyrite content assumes that all oxidisable sulphides present in the sample are pyrite. This may not be the case. The calculation may be an overesitimate when other sulphides such as Barite (Barium Sulphate) are present.

WATERS

Please note we are not a UK Drinking Water Inspectorate (DWI) Approved Laboratory .

ISO17025 accreditation applies to surface water and groundwater and usually one other matrix which is analysis specific, any other liquids are outside our scope of accreditation.

As surface waters require different sample preparation to groundwaters the laboratory must be informed of the water type when submitting samples.

Where Mineral Oil or Fats, Oils and Grease is quoted, this refers to Total Aliphatics C10-C40.

DEVIATING SAMPLES

All samples should be submitted to the laboratory in suitable containers with sufficient ice packs to sustain an appropriate temperature for the requested analysis. The temperature of sample receipt is recorded on the confirmation schedules in order that the client can make an informed decision as to whether testing should still be undertaken.

SURROGATES

Surrogate compounds are added during the preparation process to monitor recovery of analytes. However low recovery in soils is often due to peat, clay or other organic rich matrices. For waters this can be due to oxidants, surfactants, organic rich sediments or remediation fluids. Acceptable limits for most organic methods are 70 - 130% and for VOCs are 50 - 150%. When surrogate recoveries are outside the performance criteria but the associated AQC passes this is assumed to be due to matrix effect. Results are not surrogate corrected.

DILUTIONS

A dilution suffix indicates a dilution has been performed and the reported result takes this into account. No further calculation is required.

BLANKS

Where analytes have been found in the blank, the sample will be treated in accordance with our laboratory procedure for dealing with contaminated blanks.

NOTE

Data is only reported if the laboratory is confident that the data is a true reflection of the samples analysed. Data is only reported as accredited when all the requirements of our Quality System have been met. In certain circumstances where all the requirements of the Quality System have not been met, for instance if the associated AQC has failed, the reason is fully investigated and documented. The sample data is then evaluated alongside the other quality control checks performed during analysis to determine its suitability. Following this evaluation, provided the sample results have not been effected, the data is reported but accreditation is removed. It is a UKAS requirement for data not reported as accredited to be considered indicative only, but this does not mean the data is not valid.
Where possible, and if requested, samples will be re-extracted and a revised report issued with accredited results. Please do not hesitate to contact the laboratory if further details are required of the circumstances which have led to the removal of accreditation.

REPORTS FROM THE SOUTH AFRICA LABORATORY

Any method number not prefixed with SA has been undertaken in our UK laboratory unless reported as subcontracted.

Measurement Uncertainty

Measurement uncertainty defines the range of values that could reasonably be attributed to the measured quantity. This range of values has not been included within the reported results. Uncertainty expressed as a percentage can be provided upon request.

ABBREVIATIONS and ACRONYMS USED

#	ISO17025 (UKAS Ref No. 4225) accredited - UK.
SA	ISO17025 (SANAS Ref No.T0729) accredited - South Africa
B	Indicates analyte found in associated method blank.
DR	Dilution required.
M	MCERTS accredited.
NA	Not applicable
NAD	No Asbestos Detected.
ND	None Detected (usually refers to VOC and/SVOC TICs).
NDP	No Determination Possible
SS	Calibrated against a single substance
SV	Surrogate recovery outside performance criteria. This may be due to a matrix effect.
W	Results expressed on as received basis.
+	AQC failure, accreditation has been removed from this result, if appropriate, see 'Note' on previous page.
>>	Results above calibration range, the result should be considered the minimum value. The actual result could be significantly higher, this result is not accredited.
*	Analysis subcontracted to an Element Materials Technology approved laboratory.
AD	Samples are dried at 35°C ±5°C
CO	Suspected carry over
LOD/LOR	Limit of Detection (Limit of Reporting) in line with ISO 17025 and MCERTS
ME	Matrix Effect
NFD	No Fibres Detected
BS	AQC Sample
LB	Blank Sample
N	Client Sample
TB	Trip Blank Sample
OC	Outside Calibration Range
AA	x20 Dilution

EMT Job No: 20/7679

Test Method No.	Description	Prep Method No. (if appropriate)	Description	ISO 17025 (UKAS/SANAS)	MCERTS (UK soils only)	Analysis done on As Received (AR) or Dried (AD)	Reported on dry weight basis
TM30	Determination of Trace Metals by ICP-OES (Inductively Coupled Plasma – Optical Emission Spectrometry): WATERS by Modified USEPA Method 200.7, Rev. 4.4, 1994; Modified EPA Method 6010B, Rev.2, Dec 1996; Modified BS EN ISO 11885:2009: SOILS by Modified USEP	PM14	Preparation of waters and leachates for metals by ICP OES/ICP MS. Samples are filtered for Dissolved metals, and remain unfiltered for Total metals then acidified				
TM73	Modified US EPA methods 150.1 (1982) and 9045D Rev. 4 - 2004) and BS1377-3:1990. Determination of pH by Metrohm automated probe analyser.	PM0	No preparation is required.				
TM75	Modified US EPA method 310.1 (1978). Determination of Alkalinity by Metrohm automated titration analyser.	PM0	No preparation is required.				
TM76	Modified US EPA method 120.1 (1982). Determination of Specific Conductance by Metrohm automated probe analyser.	PM0	No preparation is required.				

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Golder Associates Ltd
Town Centre House
Dublin Road
Naas
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Ireland

Attention :	Martha Buckwalter-Davis
Date :	8th July, 2020
Your reference :	19132440
Our reference :	Test Report 20/8529 Batch 1
Location :	AAL
Date samples received :	2nd July, 2020
Status :	Final report
Issue :	1

Nine samples were received for analysis on 2nd July, 2020 of which nine were scheduled for analysis. Please find attached our Test Report which should be read with notes at the end of the report and should include all sections if reproduced. Interpretations and opinions are outside the scope of any accreditation, and all results relate only to samples supplied.
All analysis is carried out on as received samples and reported on a dry weight basis unless stated otherwise. Results are not surrogate corrected.

Authorised By:

S. Go

Simon Gomery BSc
Project Manager

Element Materials Technology Environmental UK Limited
Registered in England and Wales
Registered Office: 10 Lower Grosvenor Place, London, SW1W 0EN
Company Registration No: 11371415

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis
EMT Job No: 20/8529

Report : **Liquid**

Liquids/products: V=40ml vial, G=glass bottle, P=plastic bottle
H=H_2SO_4, Z=ZnAc, N=NaOH, HN=$HN0_3$

Please see attached notes for all abbreviations and acronyms

EMT Sample No.	1-4	5-8	9-12	13-16	17-20	21-24	25-28	29-32	33-36		LOD/LOR	Units	Method No.
Sample ID	BGT3D	2APL	BGT3B	1APL	BGT2B	BGT2C	BGT5B	BGT5E	DUP1				
Depth													
COC No / misc													
Containers	V HN	V HN	V HN	V HN	V HN	V HN	V HN	V HN	V HN				
Sample Date	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020				
Sample Type	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid				
Batch Number	1	1	1	1	1	1	1	1	1				
Date of Receipt	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020				
Dissolved Arsenic	1721.0	1261.0	267.3	1263.0	267.7	291.0	1244.0	1625.0	490.4		<0.9	ug/l	TM30/PM14
Dissolved Barium	<1.8	<1.8	<1.8	<1.8	<1.8	<1.8	<1.8	<1.8	<1.8		<1.8	ug/l	TM30/PM14
Dissolved Beryllium	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	ug/l	TM30/PM14
Dissolved Boron	550	2130	223	1145	184	164	800	499	415		<12	ug/l	TM30/PM14
Dissolved Cadmium	<0.03	1.11	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03		<0.03	ug/l	TM30/PM14
Total Dissolved Chromium	5.8	56.0	4.7	1.3	107.9	19.9	1.7	9.4	2.1		<0.2	ug/l	TM30/PM14
Dissolved Copper	59	18	26	10	75	40	49	44	20		<3	ug/l	TM30/PM14
Total Dissolved Iron	34.0	1061.0	273.2	14.6	27.1	26.2	42.3	52.3	181.0		<4.7	ug/l	TM30/PM14
Dissolved Lead	<0.4	<0.4	326.6	<0.4	24.6	1.0	<0.4	<0.4	186.1		<0.4	ug/l	TM30/PM14
Dissolved Mercury	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	ug/l	TM30/PM14
Dissolved Nickel	8.9	56.5	2.3	13.9	2.2	7.8	5.2	11.6	5.2		<0.2	ug/l	TM30/PM14
Dissolved Selenium	669.7$_{AA}$	211.0	130.5	726.8$_{AA}$	264.8	89.7	357.9	1265.0$_{AA}$	219.6		<1.2	ug/l	TM30/PM14
Dissolved Vanadium	18980.0$_{AA}$	13010.0$_{AA}$	4982.0$_{AA}$	1532.0	5491.0$_{AA}$	6869.0$_{AA}$	14210.0$_{AA}$	17580.0$_{AA}$	6969.0$_{AA}$		<0.6	ug/l	TM30/PM14
Dissolved Zinc	63580.0$_{AB}$	679.0	21160.0$_{AA}$	99820.0$_{AB}$	3509.0	3958.0	24770.0$_{AA}$	41480.0$_{AA}$	42430.0$_{AA}$		<1.5	ug/l	TM30/PM14
Total Alkalinity as CaCO3	14580	9140	4903	15200	3065	3723	16380	12160	7194		<1	mg/l	TM75/PM0
P Alkalinity as CaCO3	11624	6478	4178	11956	2152	2588	13506	9610	5718		<1	mg/l	TM75/PM0
Electrical Conductivity @25C	40647	23399	17812	43380	10006	11195	43949	33616	24123		<2	uS/cm	TM76/PM0
pH	13.16	12.80	12.90	13.21	12.46	12.51	13.23	13.06	12.93		<0.01	pH units	TM73/PM0

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis

EMT Job No.	Batch	Sample ID	Depth	EMT Sample No.	Analysis	Reason
No deviating sample report results for job 20/8529						

Please note that only samples that are deviating are mentioned in this report. If no samples are listed it is because none were deviating.
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EMT Job No.: 20/8529

SOILS

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All analysis is reported on a dry weight basis unless stated otherwise. Limits of detection for analyses carried out on as received samples are not moisture content corrected. Results are not surrogate corrected. Samples are dried at 35°C ±5°C unless otherwise stated. Moisture content for CEN Leachate tests are dried at 105°C ±5°C.

Where Mineral Oil or Fats, Oils and Grease is quoted, this refers to Total Aliphatics C10-C40.

Where a CEN 10:1 ZERO Headspace VOC test has been carried out, a 10:1 ratio of water to wet (as received) soil has been used.

% Asbestos in Asbestos Containing Materials (ACMs) is determined by reference to HSG 264 The Survey Guide - Appendix 2 : ACMs in buildings listed in order of ease of fibre release.

Sufficient amount of sample must be received to carry out the testing specified. Where an insufficient amount of sample has been received the testing may not meet the requirements of our accredited methods, as such accreditation may be removed.

Negative Neutralization Potential (NP) values are obtained when the volume of NaOH (0.1N) titrated (pH 8.3) is greater than the volume of HCl (1N) to reduce the pH of the sample to 2.0 - 2.5. Any negative NP values are corrected to 0.

The calculation of Pyrite content assumes that all oxidisable sulphides present in the sample are pyrite. This may not be the case. The calculation may be an overesitimate when other sulphides such as Barite (Barium Sulphate) are present.

WATERS

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DEVIATING SAMPLES

All samples should be submitted to the laboratory in suitable containers with sufficient ice packs to sustain an appropriate temperature for the requested analysis. The temperature of sample receipt is recorded on the confirmation schedules in order that the client can make an informed decision as to whether testing should still be undertaken.

SURROGATES

Surrogate compounds are added during the preparation process to monitor recovery of analytes. However low recovery in soils is often due to peat, clay or other organic rich matrices. For waters this can be due to oxidants, surfactants, organic rich sediments or remediation fluids. Acceptable limits for most organic methods are 70 - 130% and for VOCs are 50 - 150%. When surrogate recoveries are outside the performance criteria but the associated AQC passes this is assumed to be due to matrix effect. Results are not surrogate corrected.

DILUTIONS

A dilution suffix indicates a dilution has been performed and the reported result takes this into account. No further calculation is required.

BLANKS

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NOTE

Data is only reported if the laboratory is confident that the data is a true reflection of the samples analysed. Data is only reported as accredited when all the requirements of our Quality System have been met. In certain circumstances where all the requirements of the Quality System have not been met, for instance if the associated AQC has failed, the reason is fully investigated and documented. The sample data is then evaluated alongside the other quality control checks performed during analysis to determine its suitability. Following this evaluation, provided the sample results have not been effected, the data is reported but accreditation is removed. It is a UKAS requirement for data not reported as accredited to be considered indicative only, but this does not mean the data is not valid.
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REPORTS FROM THE SOUTH AFRICA LABORATORY

Any method number not prefixed with SA has been undertaken in our UK laboratory unless reported as subcontracted.

Measurement Uncertainty

Measurement uncertainty defines the range of values that could reasonably be attributed to the measured quantity. This range of values has not been included within the reported results. Uncertainty expressed as a percentage can be provided upon request.

ABBREVIATIONS and ACRONYMS USED

#	ISO17025 (UKAS Ref No. 4225) accredited - UK.
SA	ISO17025 (SANAS Ref No.T0729) accredited - South Africa
B	Indicates analyte found in associated method blank.
DR	Dilution required.
M	MCERTS accredited.
NA	Not applicable
NAD	No Asbestos Detected.
ND	None Detected (usually refers to VOC and/SVOC TICs).
NDP	No Determination Possible
SS	Calibrated against a single substance
SV	Surrogate recovery outside performance criteria. This may be due to a matrix effect.
W	Results expressed on as received basis.
+	AQC failure, accreditation has been removed from this result, if appropriate, see 'Note' on previous page.
>>	Results above calibration range, the result should be considered the minimum value. The actual result could be significantly higher, this result is not accredited.
*	Analysis subcontracted to an Element Materials Technology approved laboratory.
AD	Samples are dried at 35°C ±5°C
CO	Suspected carry over
LOD/LOR	Limit of Detection (Limit of Reporting) in line with ISO 17025 and MCERTS
ME	Matrix Effect
NFD	No Fibres Detected
BS	AQC Sample
LB	Blank Sample
N	Client Sample
TB	Trip Blank Sample
OC	Outside Calibration Range
AA	x10 Dilution

AB	x100 Dilution

EMT Job No: 20/8529

Test Method No.	Description	Prep Method No. (if appropriate)	Description	ISO 17025 (UKAS/SANAS)	MCERTS (UK soils only)	Analysis done on As Received (AR) or Dried (AD)	Reported on dry weight basis
TM30	Determination of Trace Metals by ICP-OES (Inductively Coupled Plasma – Optical Emission Spectrometry): WATERS by Modified USEPA Method 200.7, Rev. 4.4, 1994; Modified EPA Method 6010B, Rev.2, Dec 1996; Modified BS EN ISO 11885:2009: SOILS by Modified USEP	PM14	Preparation of waters and leachates for metals by ICP OES/ICP MS. Samples are filtered for Dissolved metals, and remain unfiltered for Total metals then acidified				
TM73	Modified US EPA methods 150.1 (1982) and 9045D Rev. 4 - 2004) and BS1377-3:1990. Determination of pH by Metrohm automated probe analyser.	PM0	No preparation is required.				
TM75	Modified US EPA method 310.1 (1978). Determination of Alkalinity by Metrohm automated titration analyser.	PM0	No preparation is required.				
TM76	Modified US EPA method 120.1 (1982). Determination of Specific Conductance by Metrohm automated probe analyser.	PM0	No preparation is required.				

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W: www.element.com

Golder Associates Ltd
Town Centre House
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Ireland

Attention :	Martha Buckwalter-Davis
Date :	13th July, 2020
Your reference :	19132440
Our reference :	Test Report 20/8529 Batch 1 Schedule D
Location :	AAL
Date samples received :	2nd July, 2020
Status :	Final report
Issue :	1

Nine samples were received for analysis on 2nd July, 2020 of which nine were scheduled for analysis. Please find attached our Test Report which should be read with notes at the end of the report and should include all sections if reproduced. Interpretations and opinions are outside the scope of any accreditation, and all results relate only to samples supplied.
All analysis is carried out on as received samples and reported on a dry weight basis unless stated otherwise. Results are not surrogate corrected.

Authorised By:

S. Go

Simon Gomery BSc
Project Manager

Element Materials Technology

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis
EMT Job No: 20/8529

Report : **Liquid**

Liquids/products: V=40ml vial, G=glass bottle, P=plastic bottle
$H=H_2SO_4$, Z=ZnAc, N=NaOH, $HN=HN0_3$

Please see attached notes for all abbreviations and acronyms

EMT Sample No.	1-4	5-8	9-12	13-16	17-20	21-24	25-28	29-32	33-36				
Sample ID	BGT3D	2APL	BGT3B	1APL	BGT2B	BGT2C	BGT5B	BGT5E	DUP1				
Depth													
COC No / misc													
Containers	V HN	V HN	V HN	V HN	V HN	V HN	V HN	V HN	V HN				
Sample Date	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020	26/06/2020				
Sample Type	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid	Liquid				
Batch Number	1	1	1	1	1	1	1	1	1		LOD/LOR	Units	Method No.
Date of Receipt	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020	02/07/2020				
Dissolved Titanium	224	165	11	<5	<5	<5	<5	19	<5		<5	ug/l	TM30/PM14

Please include all sections of this report if it is reproduced
All solid results are expressed on a dry weight basis unless stated otherwise.

Client Name: Golder Associates Ltd
Reference: 19132440
Location: AAL
Contact: Martha Buckwalter-Davis

EMT Job No.	Batch	Sample ID	Depth	EMT Sample No.	Analysis	Reason
No deviating sample report results for job 20/8529						

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EMT Job No.: 20/8529

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WATERS

Please note we are not a UK Drinking Water Inspectorate (DWI) Approved Laboratory .

ISO17025 accreditation applies to surface water and groundwater and usually one other matrix which is analysis specific, any other liquids are outside our scope of accreditation.

As surface waters require different sample preparation to groundwaters the laboratory must be informed of the water type when submitting samples.

Where Mineral Oil or Fats, Oils and Grease is quoted, this refers to Total Aliphatics C10-C40.

DEVIATING SAMPLES

All samples should be submitted to the laboratory in suitable containers with sufficient ice packs to sustain an appropriate temperature for the requested analysis. The temperature of sample receipt is recorded on the confirmation schedules in order that the client can make an informed decision as to whether testing should still be undertaken.

SURROGATES

Surrogate compounds are added during the preparation process to monitor recovery of analytes. However low recovery in soils is often due to peat, clay or other organic rich matrices. For waters this can be due to oxidants, surfactants, organic rich sediments or remediation fluids. Acceptable limits for most organic methods are 70 - 130% and for VOCs are 50 - 150%. When surrogate recoveries are outside the performance criteria but the associated AQC passes this is assumed to be due to matrix effect. Results are not surrogate corrected.

DILUTIONS

A dilution suffix indicates a dilution has been performed and the reported result takes this into account. No further calculation is required.

BLANKS

Where analytes have been found in the blank, the sample will be treated in accordance with our laboratory procedure for dealing with contaminated blanks.

NOTE

Data is only reported if the laboratory is confident that the data is a true reflection of the samples analysed. Data is only reported as accredited when all the requirements of our Quality System have been met. In certain circumstances where all the requirements of the Quality System have not been met, for instance if the associated AQC has failed, the reason is fully investigated and documented. The sample data is then evaluated alongside the other quality control checks performed during analysis to determine its suitability. Following this evaluation, provided the sample results have not been effected, the data is reported but accreditation is removed. It is a UKAS requirement for data not reported as accredited to be considered indicative only, but this does not mean the data is not valid.
Where possible, and if requested, samples will be re-extracted and a revised report issued with accredited results. Please do not hesitate to contact the laboratory if further details are required of the circumstances which have led to the removal of accreditation.

REPORTS FROM THE SOUTH AFRICA LABORATORY

Any method number not prefixed with SA has been undertaken in our UK laboratory unless reported as subcontracted.

Measurement Uncertainty

Measurement uncertainty defines the range of values that could reasonably be attributed to the measured quantity. This range of values has not been included within the reported results. Uncertainty expressed as a percentage can be provided upon request.

ABBREVIATIONS and ACRONYMS USED

#	ISO17025 (UKAS Ref No. 4225) accredited - UK.
SA	ISO17025 (SANAS Ref No.T0729) accredited - South Africa
B	Indicates analyte found in associated method blank.
DR	Dilution required.
M	MCERTS accredited.
NA	Not applicable
NAD	No Asbestos Detected.
ND	None Detected (usually refers to VOC and/SVOC TICs).
NDP	No Determination Possible
SS	Calibrated against a single substance
SV	Surrogate recovery outside performance criteria. This may be due to a matrix effect.
W	Results expressed on as received basis.
+	AQC failure, accreditation has been removed from this result, if appropriate, see 'Note' on previous page.
>>	Results above calibration range, the result should be considered the minimum value. The actual result could be significantly higher, this result is not accredited.
*	Analysis subcontracted to an Element Materials Technology approved laboratory.
AD	Samples are dried at 35°C ±5°C
CO	Suspected carry over
LOD/LOR	Limit of Detection (Limit of Reporting) in line with ISO 17025 and MCERTS
ME	Matrix Effect
NFD	No Fibres Detected
BS	AQC Sample
LB	Blank Sample
N	Client Sample
TB	Trip Blank Sample
OC	Outside Calibration Range

EMT Job No: 20/8529

Test Method No.	Description	Prep Method No. (if appropriate)	Description	ISO 17025 (UKAS/SANAS)	MCERTS (UK soils only)	Analysis done on As Received (AR) or Dried (AD)	Reported on dry weight basis
TM30	Determination of Trace Metals by ICP-OES (Inductively Coupled Plasma – Optical Emission Spectrometry): WATERS by Modified USEPA Method 200.7, Rev. 4.4, 1994; Modified EPA Method 6010B, Rev.2, Dec 1996; Modified BS EN ISO 11885:2009: SOILS by Modified USEP	PM14	Preparation of waters and leachates for metals by ICP OES/ICP MS. Samples are filtered for Dissolved metals, and remain unfiltered for Total metals then acidified				

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APPENDIX C

Seepage Modelling Report

REPORT

Aughinish Alumina Limited

Engineering Design Closure Study - Seepage Assessment for BRDA at Stage 16

Submitted to:

Aughinish Alumina Limited

Aughinish Island
Askeaton
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Submitted by:

Golder Associates Ireland Limited

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19132440.R02.A2

August 2021

Distribution List

Aughinish Alumina Limited - 1 copy (PDF)

Golder Associates Ireland Limited - 1 copy (PDF)

Table of Contents

TABLES

FIGURES

1.0 INTRODUCTION

1.1 Terms of Reference

Golder Associates Ireland Limited ('Golder') was commissioned by Aughinish Alumina Limited ('AAL') to assess the potential behaviour of seepage emanating from Phase 1 and Phase 2 of the bauxite residue disposal area ('BRDA') constructed to Stage 16 at AAL's alumina refinery in Askeaton, Co. Limerick, Ireland.

1.2 Site Setting

The alumina refinery is situated on Aughinish Island on the south side of the Shannon estuary, Co. Limerick. The Island is located between Askeaton and Foynes and is approximately 30 kilometres (km) west of Limerick and 15 km southwest of Shannon Airport. The Island has an area of approximately 400 hectares (ha) and is bounded by the River Shannon to the north, the Robertstown River to the west and southwest and the Poulaweala creek to the east and southeast.

The Phase 1 BRDA is located southwest of the process plant and is formed of two facilities: the original Phase 1 BRDA, which covers an area of 72 ha and the eastern Phase 1 BRDA Extension, which covers an area of 32 ha. The Phase 2 BRDA adjoins the southern extent of the Phase 1 BRDA and covers an area of 80 ha.

The BRDA is surrounded by perimeter interception channels (PICs), which collect leachate water and run-off from Phase 1 and Phase 2 BRDA and convey it via pumps either to the Effluent Clarification System (ECS) or the Storm Water Pond (SWP). Both the ECS and the SWP are situated to the northeast of the Phase 1 BRDA. At closure, the PICs will be modified to discharge surface water off-site via designated breach locations.

1.3 Study Context

The Phase 1 BRDA is currently at Stage 9/10 and the Phase 2 BRDA is currently at Stage 3/4. As part of the design report assessments for the BRDA Raise to Stage 16 Development, AAL has commissioned studies for closure of the BRDA at Stage 16. To support this, Golder has prepared two preliminary engineering design reports as follows:

- Dome Water Management Infrastructure Design. Bauxite Residue Disposal Area – Closure Design at Stage 16, 20143063.R01.B0, (Golder 2020a); and
- BRDA Side Slope Closure Design at Stage 16. Surface Water Management Design, 20143076.R02.B1, (Golder 2020b).

These reports consider the closure design for water management infrastructure to transfer run-off from the BRDA dome to the PICs and the design of surface water management on the side slopes to control discharges into the PICs. The reports propose that for the dome catchment there will be a direct transfer of water, falling on this area, to the PICs via designated spillways, whilst for the side slopes, water will be allowed to trickle down the rock-filled blanket to the PICs.

The focus of this report is to assess the additional inputs that may enter the PICs as a consequence of infiltration into the restored BRDA Stage 16. Such inputs may express themselves as seepage at the base of the side slopes (at the point of entry to the PICs), through the lining system at the base of the BRDA (basal liner), or as surface water seeps in the side slopes.

Golder has undertaken modelling of the restored BRDA to assess the potential for seepage to occur from the infiltration into the restored BRDA at Stage 16 and to quantify these potential discharges should these be identified. The findings are then utilized in the assessment for the conceptual water quality in the PICs following closure of the BRDA at Stage 16 (19132440.R03.A0).

1.4 Modelling Objective

The primary objective of this study is to construct a model of the BRDA at Stage 16 and use the simulation results to estimate the potential volumes of seepage generated along the side slopes of the restored Stage 16 BRDA, as well as the surface water runoff from the facility dome. The study outcomes will be used to inform an assessment of water quality in the PIC and to assist in any refinement of design studies previously undertaken by Golder (as referenced in Section 1.3).

Note: This study deals with the post-closure phase of the BRDA, i.e. following restoration at Stage 16, and no consideration of changes in the structure and/or properties of the BRDA are included in model simulations.

2.0 MODEL SET UP

2.1 Code Selection and Modelling Approach

The modelling software selected for this study is Seequent's Geoslope SEEP/W package (version 11.1.1.22085) (GEOSLOPE 2012). SEEP/W is a finite element code which is widely used for modelling groundwater flow in both saturated and unsaturated porous media. The software also offers close coupling to other types of engineering simulations, such as the allied SLOPE/W code for slope stability modelling.

The modelling approach adopted for this study is to simulate a single 'representative' year (i.e. January to December) of surface layer processes over the BRDA, such as runoff, infiltration and water storage, using a transient analysis.

Initial conditions for the transient modelling are inherited from a steady-state 'parent' model. As noted above, both the transient model and its steady-state parent model assume that the BRDA is at Stage 16 from the start and no modelling of the construction phase is included. It is further assumed that the hydraulic properties of the BRDA's constituent materials and its underlying geology, as well as the characteristics of the vegetation cover on its upper surface, remain constant over the simulated period.

2.2 Model Domain

The first step in the construction of the finite element model in SEEP/W is the definition of the model domain. This essentially refers to the model extents, its geometry and its constituent material types, which are assigned to different sub-regions of the model domain. A two-dimensional model domain was adopted for this study, based on a vertical slice taken through the BRDA along an approximately north-south heading. Specifically, the slice is taken from a point just beyond the northern PIC (E: 127287, N: 1526100) to a point immediately south of the southern PIC (E: 127877, N: 150899), see Figure 1. This slice was selected to encompass both the Phase 1 and Phase 2 BRDA and is approximately 1,810 m in length with a nominal slice width of 1m.

The geometry of the facility and its underlying geology was obtained from a 3D CAD model of the Stage 16 BRDA. The sections were imported into SEEP/W and used to define the model regions, which collectively delineate its constituent material types. The resulting model geometry is illustrated in Figure 2, with a factor three vertical scaling applied to enable easier visualisation.

In the vertical direction, the model extends from the peak of the Stage 16 cap, at 44 mOD, to a point in the bedrock at -23 mOD. The thickness of the modelled bedrock layer ranges from 10m at the model's northern extent, where it is overlain by approx. 13m depth of estuarine soils, to 22.5 m at the model's southern limit, where it is overlain by approx. 0.5 depth of till. The estuarine soils are modelled as pinching out beneath the edge of the Phase 1 BRDA base, at which point the Phase 2 basal lining system starts. This consists of a composite liner which directly overlies the shallow till layer and limestone bedrock beneath the Phase 2 BRDA.

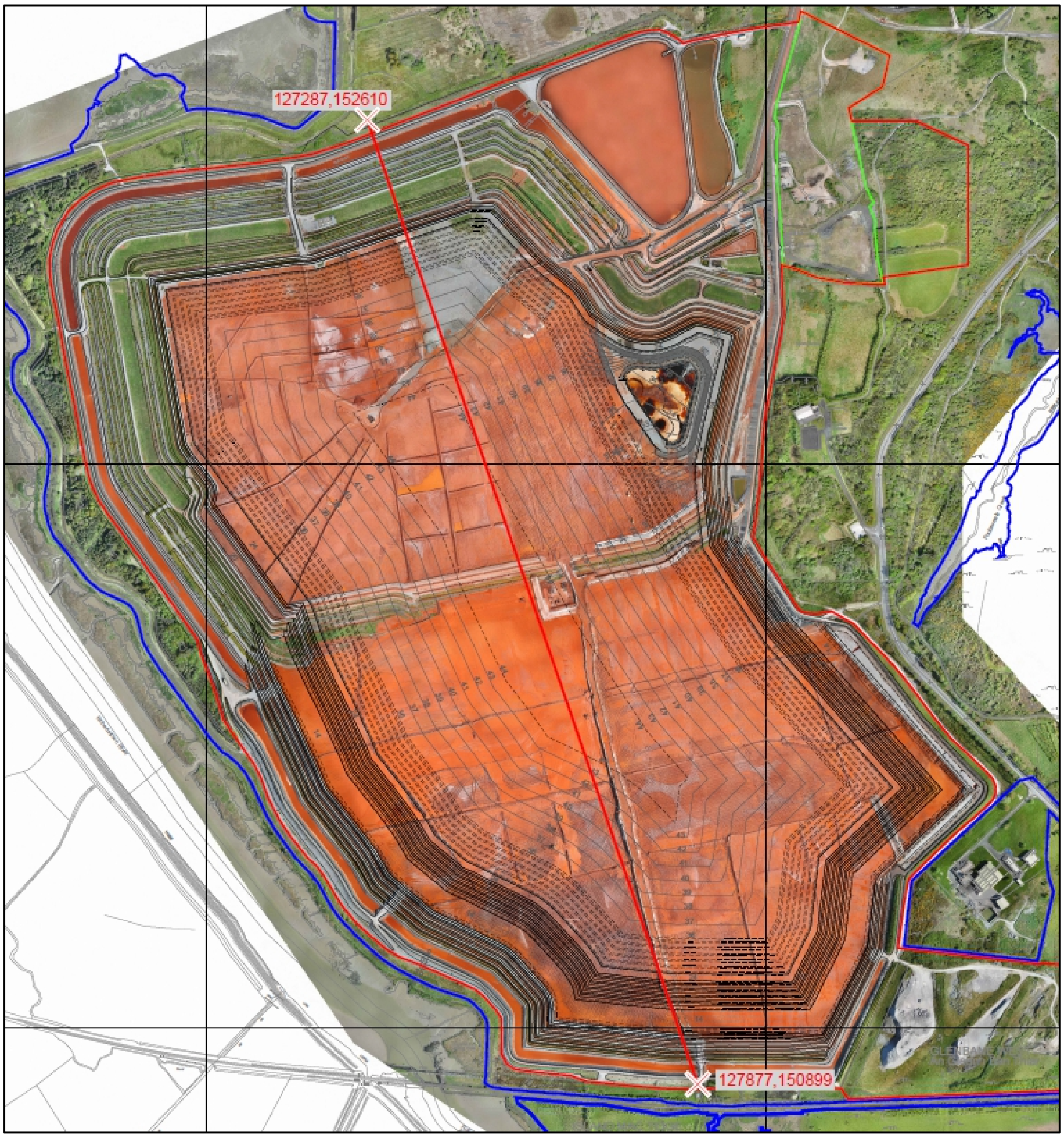

Figure 1: Location of vertical, 1m width slice through the BRDA at Stage 16 with Closure Dome, (Background Aerial Image from April 2021)

Model regions are assigned one of seven material types, as follows:

- **Bauxite Residue (Amended) –** Represents the upper ~1m of amended bauxite residue at the facility surface, which will serve as a final capping layer for the BRDA.
- **Bauxite Residue (Farmed) –** Represents the bauxite residue which has been subjected to the farming process, i.e. from Stage 5 of the Phase 1 BRDA and from the basin of the Phase 2 BRDA.
- **Bauxite Residue (Unfarmed) –** Represents the bauxite residue which has not been subjected to the farming process, i.e. from the basin up to Stage 5 of the Phase 1 BRDA.
- **Rock Fill –** Represents the rock-fill stage raises and blanket which will cover the flanks of the BRDA.
- **Composite Lining System –** Represents the lining system that covers the base of the Phase 2 BRDA basin and the Phase 1 BRDA extension basin (the footprint of Phase 1 BRDA extension does not extend below the slice). This consists of a combination of a primary liner (2 mm HPDE geomembrane) over a secondary liner (either a geosynthetic clay liner (GCL) and a 0.3 to 0.5m thickness of compacted glacial till, or a minimum 1m depth of till).

- **Till –** A thin till layer (0.5m to 1.0m) is intermittently present between the estuarine soils and the bedrock beneath the Phase 1 BRDA but is omitted from the model due to its inconsistency. The till layer is modelled as 0.5m depth beneath the composite lining system (geomembrane and GCL) beneath the Phase 2 BRDA.
- **Estuarine Soils –** Represents the estuarine deposits which underlie the Phase 1 BRDA; typically clays and silts.
- **Bedrock –** Represents the bedrock underlying the estuarine soils and Phase 2 BRDA, i.e. primarily Rathkeale Formation limestones.

Hydraulic properties of the modelled material types are described in the following section of this report.

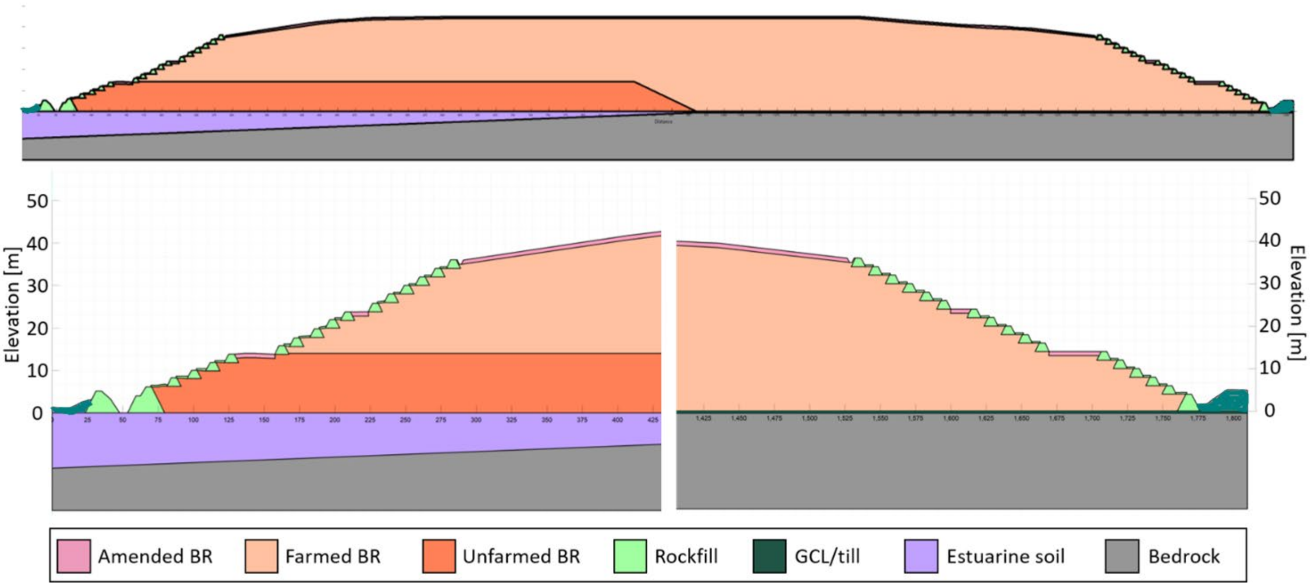

Figure 2: Model Geometry for BRDA at Stage 16 with Closure Dome, showing the northern slope (left), the internal storage area and dome, and the southern slope (right). Scaling Factor 3V:1H.

2.3 Hydraulic Properties

The hydraulic properties of the modelled materials strongly influence water circulation patterns within the saturated and unsaturated layers of the model. Therefore, it is important that these appropriately reflect the properties of the constituent materials of the BRDA and its underlying geology. The existing phreatic surface measured via piezometers at Stage 10 of the Phase 1 BRDA was reproduced in a SEEP/W model to validate the K_v and K_h values selected for the farmed and unfarmed bauxite residue seepage modelling at Stage 16.

- Golder (Golder 2015) undertook hydraulic conductivity testing on three farmed bauxite residue core samples and returned values ranging from 3.7×10^{-8} m/s to 8.5×10^{-9} m/s, with a characteristic value of 1.9×10^{-8} m/s determined for the vertical component of conductivity, K_v. Recent testing of the bauxite residue at solids contents between 30% and 70% return vertical conductivity values between 1.0×10^{-8} m/s and 1.0×10^{-9} m/s, with the lower values corresponding to the higher solids content (Golder 2021). The bauxite residue is deposited at a solids content of approx. 57% and the mud-farmed bauxite residue has a solids content of approx. 74%.

  The deposition of bauxite residue layers and the farming process is considered to result in a bedded structure that provides a horizontal conductivity (K_h) of approximately 10 times greater than that of the vertical (Golder 2005). Accordingly, for the farmed bauxite residue, the horizontal and vertical components of hydraulic conductivity were set at 1.0×10^{-7} m/s and 1.0×10^{-8} m/s, respectively. A saturated volumetric water content of 0.33 (33%) was assumed, based on testing by Golder (Golder 2014).

- The unfarmed bauxite residue has a lower permeability than the farmed material. Furthermore, the unfarmed residue is understood to exhibit minimal differences between the vertical and horizontal components of hydraulic conductivity (Golder 2005). *In situ* conductivity testing by Golder and others (Delft 1988), (URS 2002), (Golder 2005) has provided values between 1×10^{-10} m/s and 5.6×10^{-9} m/s. Therefore, a bulk saturated conductivity of 1.0×10^{-9} m/s was assigned for the unfarmed bauxite residue in the model. The saturated volumetric water content is somewhat higher due to the absence of the farming process and is assigned a characteristic value of 0.39 (39%), based on testing (Golder 2014).

- The hydraulic properties of the amended layer are more uncertain. On the basis that this will consist of a mixture of farmed bauxite residue, compost and sand, which generally exhibit higher permeabilities than that of the unamended residue, for modelling purposes it is assumed that the conductivity of the amended material is somewhat greater. Furthermore, this layer is not anticipated to exhibit a significant degree of anisotropy. A bulk hydraulic conductivity of 1.0×10^{-6} m/s was assigned for the amended bauxite residue. A saturated volumetric water content of 0.35 (35%) is assumed for this material.

  Note*:* *The hydraulic conductivity value used in the model for the amended layer is greater than the conductivity values suggested by recent triaxial permeability tests* (Golder 2021)*. However, as noted in the testing report, the laboratory results are thought to be impacted by the presence of compost in the amended material, and the true hydraulic conductivity values are expected to be higher for the amended capping layer.*

- The Phase 2 BRDA basal lining system is a composite liner consisting of a 2 mm HDPE geomembrane combined with either a GCL over a 0.3 to 0.5m thickness of compacted glacial till or a minimum 1m depth of compacted till. In both cases, the hydraulic conductivity of the geomembrane is assumed to exert a controlling influence on the vertical conductivity of the combined lining system layer. The vertical conductivity of the modelled layer is therefore estimated by scaling the conductivity of the 2 mm geomembrane (i.e., 1×10^{-14} m/s) over the 0.5m thickness of the layer in the model, to obtain a value of 2.5×10^{-12} m/s. The horizontal conductivity is assumed to be an order of magnitude greater as this will be controlled by the properties of the compacted till. A volumetric water content of 0.3 (30%) is assumed for this layer (Hanson, Risken and Yeşiller 2013).

- The estuarine soils beneath the BRDA generally consist of clays and silts. There has been no recent permeability testing of the estuarine soils and the results available date from pre-2004 and are indicative of the in-situ conditions prior to the Phase 1 BRDA development. Laboratory testing on undisturbed samples returned permeabilities ranging from 1.0×10^{-7} to 1.0×10^{-10} m/s whilst in-situ testing returned permeabilities ranging from 5.0×10^{-5} to 5.0×10^{-9} m/s, which is considered to be due to the bedded nature of the deposits. Recompacted samples on estuarine returned permeabilities ranging from 2.1×10^{-9} to 2.5×10^{-10} m/s (Golder 2005). Therefore, a saturated hydraulic conductivity of $K_{sat} = 1.0 \times 10^{-8}$ m/s was assigned for the estuarine soils in the model, together with an K_v/K_h ratio of 0.1 to reflect the transverse anisotropy induced by the bedding layers. The saturated volumetric water content was assumed to be 0.4 (40%), based on a combination of literature values (McWhorter and Sunada 1977) and laboratory testing undertaken by Golder (Golder 2019).

- The mapped bedrock geology comprises Waulsortian Formation limestones on the eastern side of the BRDA and the overlying Rathkeale Formation limestones and mudstones on the western side. The Rathkeale Formation is characterised as impure muddy limestones and shaley mudstones. The Waulsortian Formation is characterised as a medium bedded to massive, fine to coarsely crystalline, blue grey limestone. Structurally no major faults and no karst features have been identified in the footprint of the BRDA. Discrete fracture zones / palaeokarst features and weathering profiles near surface have been identified by borehole logs but are considered to be hydrogeologically unconnected.

The limestone bedrock is modelled as fully saturated with a hydraulic conductivity of 1×10^{-8} m/s, based on recovery tests carried out on groundwater observation wells (Golder 2005). A volumetric water content of 0.35 was assumed, based on literature values (McWhorter and Sunada 1977).

Volumetric water content (VWC) functions for the (unsaturated) modelled materials are presented in Figure 3 and the associated hydraulic conductivity functions are shown in Figure 4.

VWC functions for the bauxite residue and estuarine soils are estimated, based on the assumption that these materials exhibit soil moisture retention characteristics equivalent to those of a silty sand. For the composite lining system, clay-like soil moisture characteristics are adopted, whereas for the rock fill, sand-like properties were assigned, as gravel-like properties were found to cause severe numerical convergence problems in the model solution. The hydraulic conductivity function for each material, which characterises how the material's conductivity responds to changes in soil moisture content, is estimated from its respective VWC function using the Van Genuchten method (Van Genuchten 1980).

A summary of the modelled materials' hydraulic properties is presented in Table 1.

Table 1: Summary of the Material Hydraulic Properties used in the Model

Material	Saturated K_h [m/s]	Anisotropy Ratio (K_h/K_v)	Saturated VWC [-]	VWC function material type
Amended BR	1.0×10^{-6}	1.0	0.35	Silty Sand
Farmed BR	1.0×10^{-7}	10.0	0.33	Silty Sand
Unfarmed BR	1.0×10^{-9}	1.0	0.39	Silty Sand
Rock fill	1.0×10^{-2}	1.0	0.35	Sand
Composite Liner	2.5×10^{-11}	10.0	0.30	Clay
Estuarine soils	1.0×10^{-8}	10.0	0.40	Silty Sand
Bedrock	1.0×10^{-8}	1.0	0.35	Rock

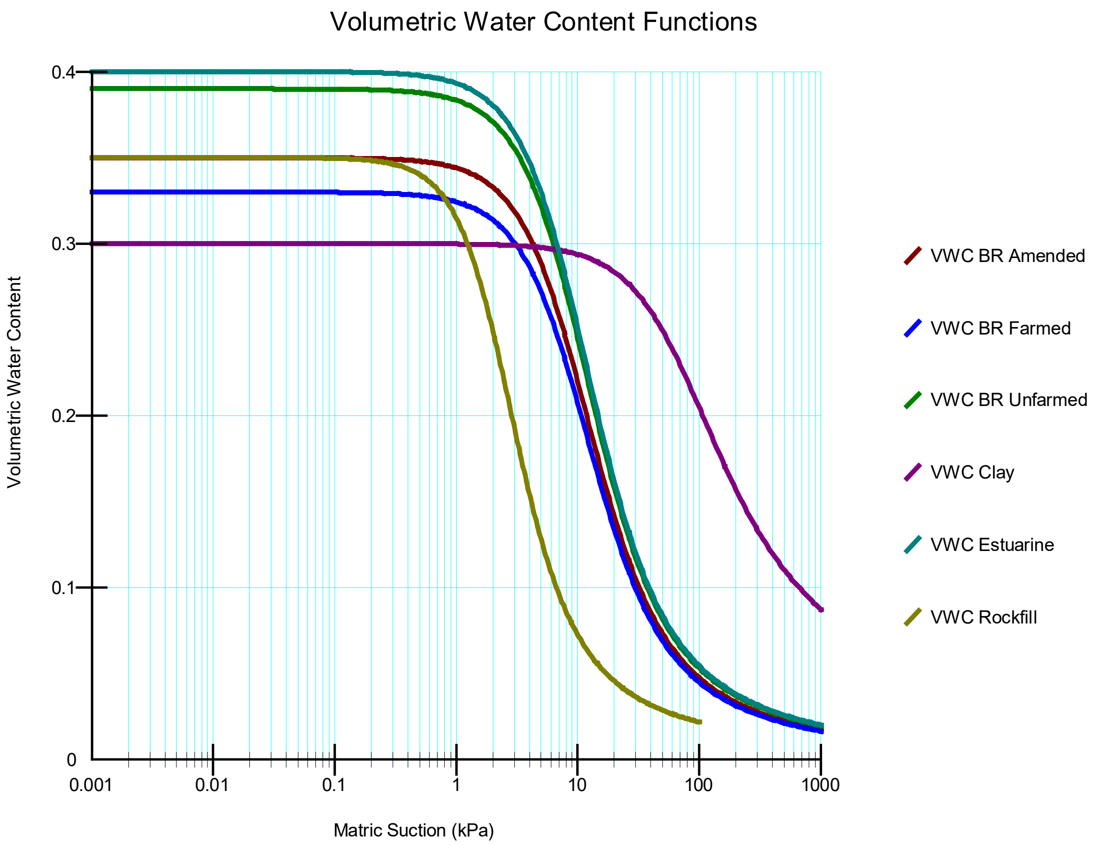

Figure 3: Volumetric Water Content Functions for the Six Unsaturated Material Types in the Model

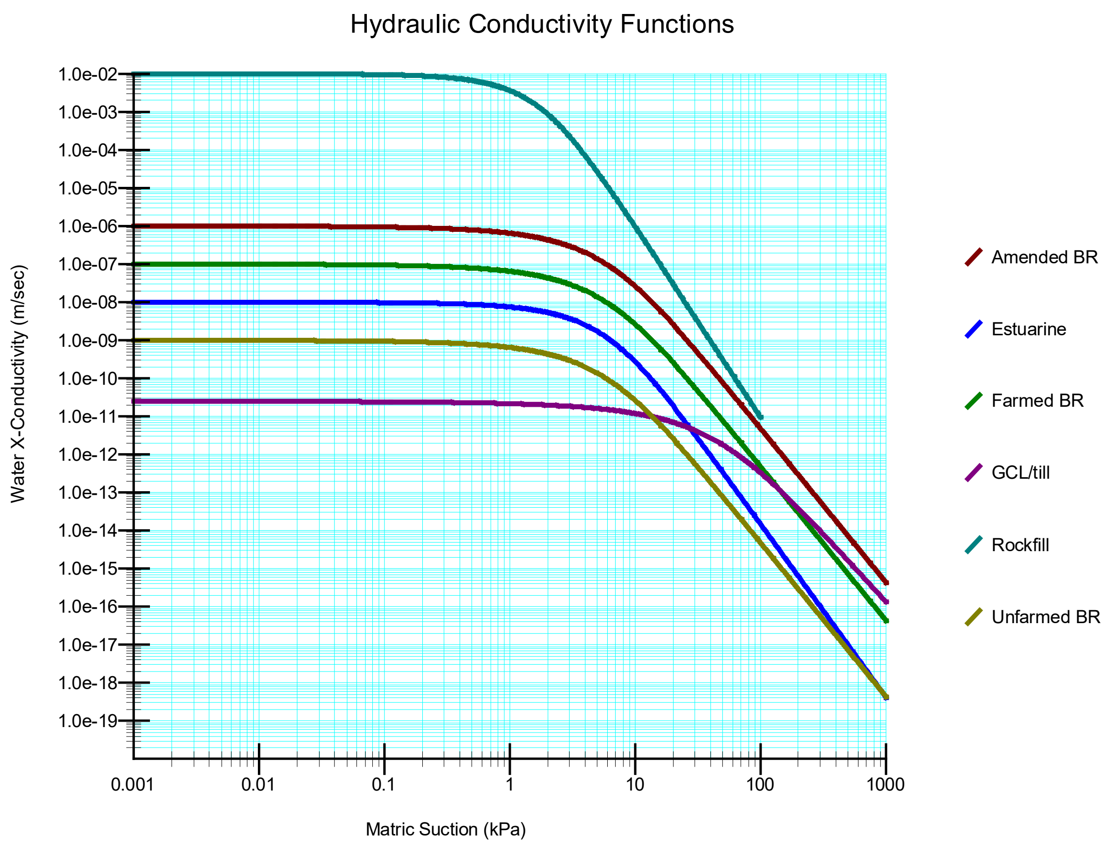

Figure 4: Hydraulic Conductivity Functions for the Six Unsaturated Material Types in the Model

2.4 Boundary and Initial Conditions

The primary objective of this modelling study is to estimate the likely volumes of seepage generated along the side slopes of the restored Stage 16 BRDA, as well as the surface water runoff from the facility dome. In this sense, the boundary condition at the model's upper surface (i.e. the dome and side slopes) cannot be precisely specified in terms of a known flux per unit area, but rather is part of the solution being sought.

Nonetheless, for a transient analysis it is essential to define an initial starting point, which is specified in terms of the total head at each node in the model grid. To this end (as noted in Section 2.1 of this report), the initial conditions for the transient analysis of this study are provided by the results of a steady-state 'parent' model.

2.4.1 Steady-State Boundary Conditions

For the steady-state parent model, a no-flow boundary condition is applied at the model's base, reflecting the absence of any significant vertical head gradient in the bedrock at this depth. Constant head boundary conditions are applied at the model sides (i.e. the northern and southern limits) and are both set to 0m such that no pre-existing head gradient is applied across the site. No-flow boundary conditions are applied at the base of the PIC and on the inner face of the Outer Perimeter Wall (OPW) to replicate the hydraulic properties of the composite lining system at these locations.

At the model's upper surface, a constant recharge boundary condition is applied to simulate recharge due to infiltration from precipitation. The applied flux is equivalent to an estimated recharge rate of 50 mm/year (EPA 2019) in which runoff and evapotranspiration are accounted for.

Note: The model's upper surface is modelled as a 'potential seepage face', meaning that a free surface (i.e. discharge) can develop at any point along the boundary. In the case that the applied water flux exceeds the infiltration capacity of the soil at any point, the pore-water pressure along the discharge surface is set to zero, thereby ensuring a physically realistic solution.

Current-day phreatic surface is measured via piezometers at 1m to 3m below surface on the facility side slopes, and several metres below surface at the upper surface i.e. Stage 9 or Stage 10. It is expected that the phreatic surface for the BRDA will drop further following closure due to the dome shape, the vegetative cover and the low permeability of the bauxite residue (amended and farmed layers) reducing the opportunity for infiltration and recharge. However, a 'worst-case' scenario to maximize seepage is adopted for this study, with the initial phreatic surface assumed at close to the surface for the side-slopes and the upper surface.

A SEEP/W model to Stage 10 was constructed using the measured upper phreatic surface depth and varying the K_v and K_h values of the farmed and unfarmed bauxite residue in order to reproduce the measured side-slope phreatic surface. This model was then projected to Stage 16 and the same upper phreatic surface depth, K_v and K_h values were then applied. The model showed the phreatic surface for the side-slopes reducing in depth and becoming near surface, in particular at Stages 5 to 7 and at Stage 10, when the BRDA reached Stage 16.

2.4.2 Transient Boundary Conditions

Boundary conditions for the transient analysis are equivalent to those applied for the steady-state analysis at the model's base, sides and in the PIC drainage channels. The only difference is at the model's upper surface, for which a 'Land-Climate Interaction' (LCI)-type boundary condition is applied in place of the constant recharge used for the steady-state analysis. This enables the simulation of soil-vegetation-atmosphere transfers across the ground surface and can be used to compute the surface water balance resulting from these interactions. The mathematical formulation of the LCI boundary condition lies beyond the scope of this report; however, further details are available from the relevant SEEP/W documentation (GEOSLOPE 2020).

The LCI boundary condition requires a number of input parameters relating to both the meteorological conditions over the site and the nature and extent of vegetation cover over the modelled surface layer. Historical data were obtained from the Irish Meteorological Service website (MET Éireann 2021). Average daily values for precipitation, air temperature and relative humidity were then calculated from the previous 30 years of data (01 Jan 1991 to 31 Dec 2020) from the nearby Shannon Airport monitoring station (grid ref. E: 137900, N: 160300), to obtain an average annual profile of atmospheric inputs.

The planned vegetation cover over the restored Stage 16 BRDA is understood to be primarily grass. Based on this, a Leaf Area Index (LAI) of 2.45 is assumed (Ramirez-Garcia, Almendros and Quemada 2012). The LAI characterises the plant leaf area per unit ground surface area and is modelled as a constant function with time under the assumption that grass cover varies minimally throughout the year in western Ireland. From this, a soil cover fraction (SCF) function can be calculated (Ritchie 1972).

Other important parameters for the LCI boundary condition relate to the maximum depth and density profile of plant roots, since these parameters control the rate of water removal via transpiration. Based on guidance in (Allen, et al. 1998) and assuming a sward of mixed grass types, the average rooting depth is estimated at 0.3m. A simple linear function is adopted for the normalised root density (Prasad 1988).

Finally, the plant moisture limiting function, which controls the relationship between transpiration and soil matric suction, is based on that used by (Huang, Barbour and Carey 2015).

2.5 Model Convergence

A seepage analysis in unsaturated porous media is a highly nonlinear process, and the iterative numerical methods used to solve this type of problem are not guaranteed to converge. Therefore, it is important to check whether the solution is sufficiently converged before the results can be interpreted with any confidence.

A useful way to assess this in SEEP/W is to plot graphs of conductivity vs. matric suction for both the material property input functions (i.e. the user-specified hydraulic conductivity functions described in Section 2.3) and the actual material properties assigned for each node within the model domain, for each of the unsaturated materials.

Convergence is deemed to have been attained when the two sets of data overlie for each material. Such plots are shown in Figure 5 and Figure 6 for the steady-state and transient analyses, respectively. From the graphs, it is evident that both analyses are well converged. Alternative measures of convergence (not shown here) were also deemed satisfactory for both analyses.

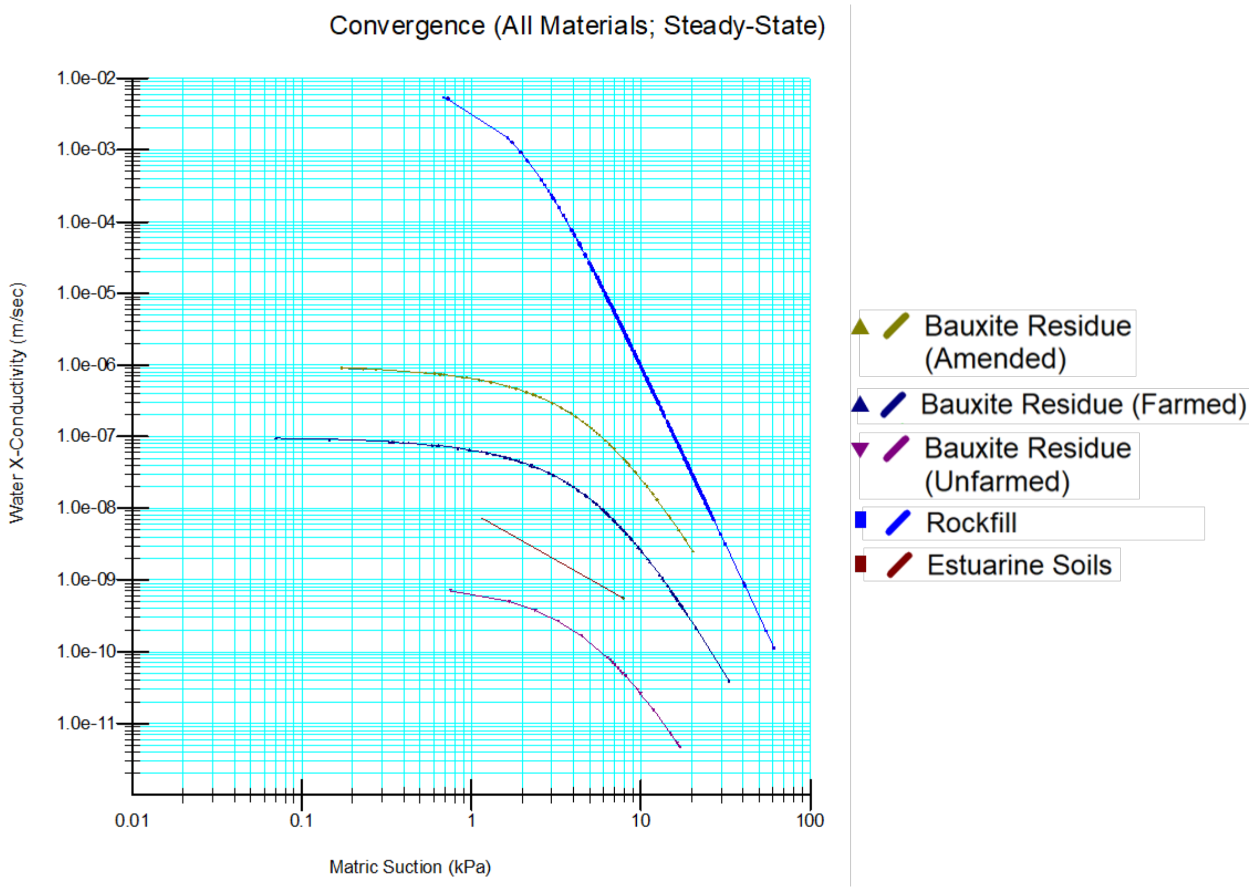

Figure 5: Model Convergence Checks for the Steady-State Analysis

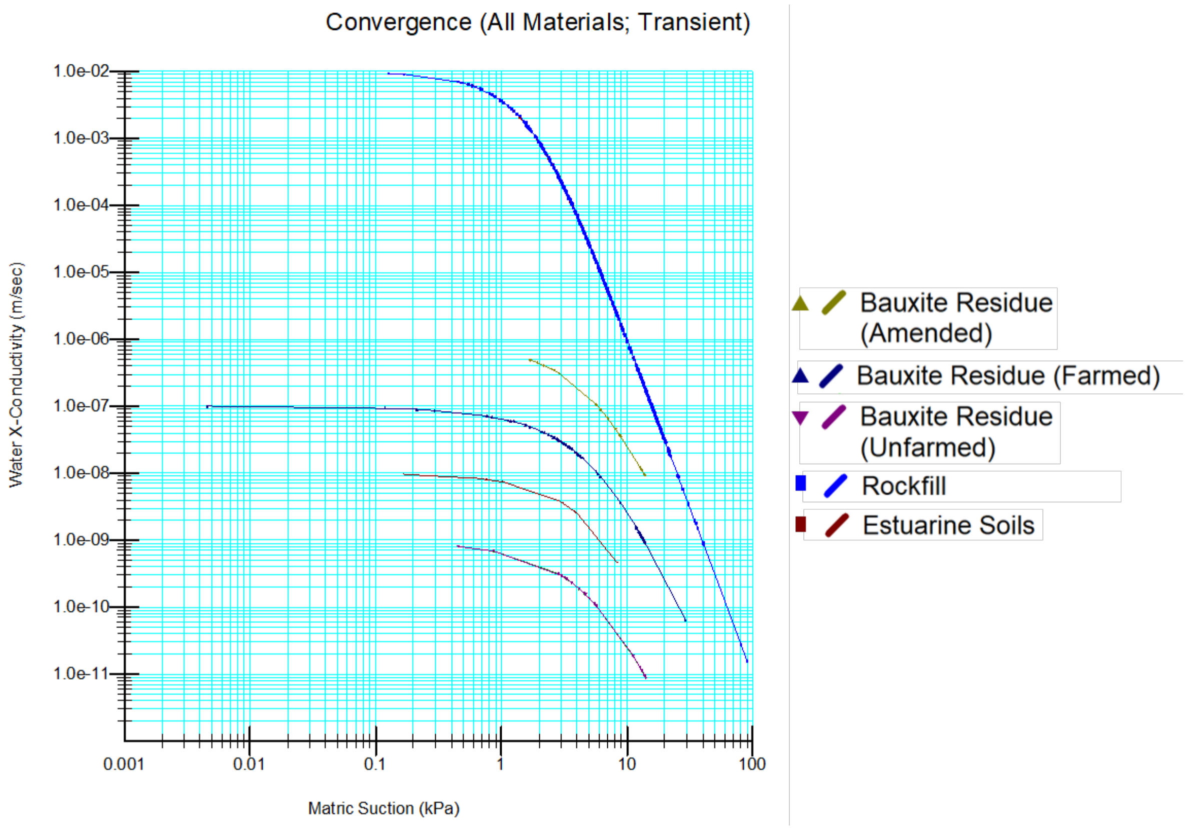

Figure 6: Model Convergence Checks for the Transient Analysis (all time steps)

3.0 ASSESSMENT OF POTENTIAL SEEPAGE

3.1 Preamble

As noted in the modelling objectives in Section 1.4, the primary goal of this study is to provide quantitative estimates of surface runoff and sidewall seepage from the BRDA, which will be used as inputs for a PHREEQC mixing model to predict the post-closure water quality in the PIC.

For the purposes of the mixing calculations, the key inputs of interest from seepage modelling are the relative quantities of water in the PIC that arise from different origins, i.e. due to surface runoff and seepage from different parts of the facility. However, since the model represents a nominal 1m wide ‘slice’ through the BRDA, the absolute modelled seepage volumes (see Table 2) are not particularly meaningful. Therefore, the model outputs must be scaled to the site surface area before they can be used in water quality calculations (see Table 3). Furthermore, since the model results indicate only a relatively minor difference between summer and winter conditions in terms of the relative runoff and seepage contributions, average annual values are reported for this study.

3.2 Surface Runoff and Sidewall Seepage

As noted in the discussion of steady-state boundary conditions in Section 2.4.1, a ‘worst-case’ scenario to maximize seepage is assumed for this study. Therefore, the initial condition from the steady-state parent simulation provides a starting phreatic surface either at, or very close to, the ground surface over the BRDA dome, and just below the base of the stage raises and rock-filled blanket on the facility side slopes. This is likely to yield a worse-case estimate of water quality, as the high-water levels provide large driving heads for generating seepage at the model side slopes.

Less than 1% of incident precipitation is lost to evaporation in the model. This is due to the high average rainfall (which causes evaporative flux to be set to zero in the model) and high relative humidity in western Ireland, as well as the fact that the upper surface of the model is assumed to be fully vegetated, and thus water is predominantly removed via transpiration instead. Of the water which enters the model’s upper surface, a substantial proportion – equivalent to 30% of precipitation – is lost to transpiration.

The position of the phreatic surface is not seen to change substantially over the course of the one-year transient simulation period, dipping by at most 0.5m below the dome surface during the summer months. Over the model surface, a significant proportion of the incident rainfall – equivalent to approximately two thirds of precipitation – is rejected as runoff by the model.

Of the total runoff which enters the PICs, 70% is generated over the facility dome. Based on the most recent dome water management plans (Golder 2020a), runoff from the dome is understood to be intercepted by dome perimeter channels which convey water via a series of spillways directly to the PICs. Therefore, no losses are assumed between the net runoff water generated over the dome and the volume which enters the PICs. The remaining 30% of runoff is generated over the model side slopes.

<u>**Note**</u>: *In the model this water is not strictly treated as runoff because precipitation applied over the side slopes is intercepted by the high-permeability rock fill blanket which covers this area. However, under the assumption that precipitation applied over the rock fill blanket is rapidly routed to the PICs as shallow interflow, the net water flux into the rock fill blanket is treated as runoff for reporting purposes.*

The evolution of modelled water fluxes over the 1-year simulation period is presented in Figure 7. Slight seasonal fluctuations are apparent in both the transpiration and runoff time-series, as reflected by the weakly ‘s-shaped’ profiles of these curves.

Note: The water balance in Figure 7 should not be expected to zero (i.e. inputs equal to outputs). There are two reasons for this:

- Firstly, as described above, water that enters the model over the side slopes as precipitation (minus evapotranspiration) is not accounted for in the total modelled runoff, since this is treated as interflow through the rock fill blanket which subsequently discharges directly into the PIC at the downstream toe of the side-slopes.
- Secondly, transpiration is not included in the water balance because root water uptake occurs below the ground surface.

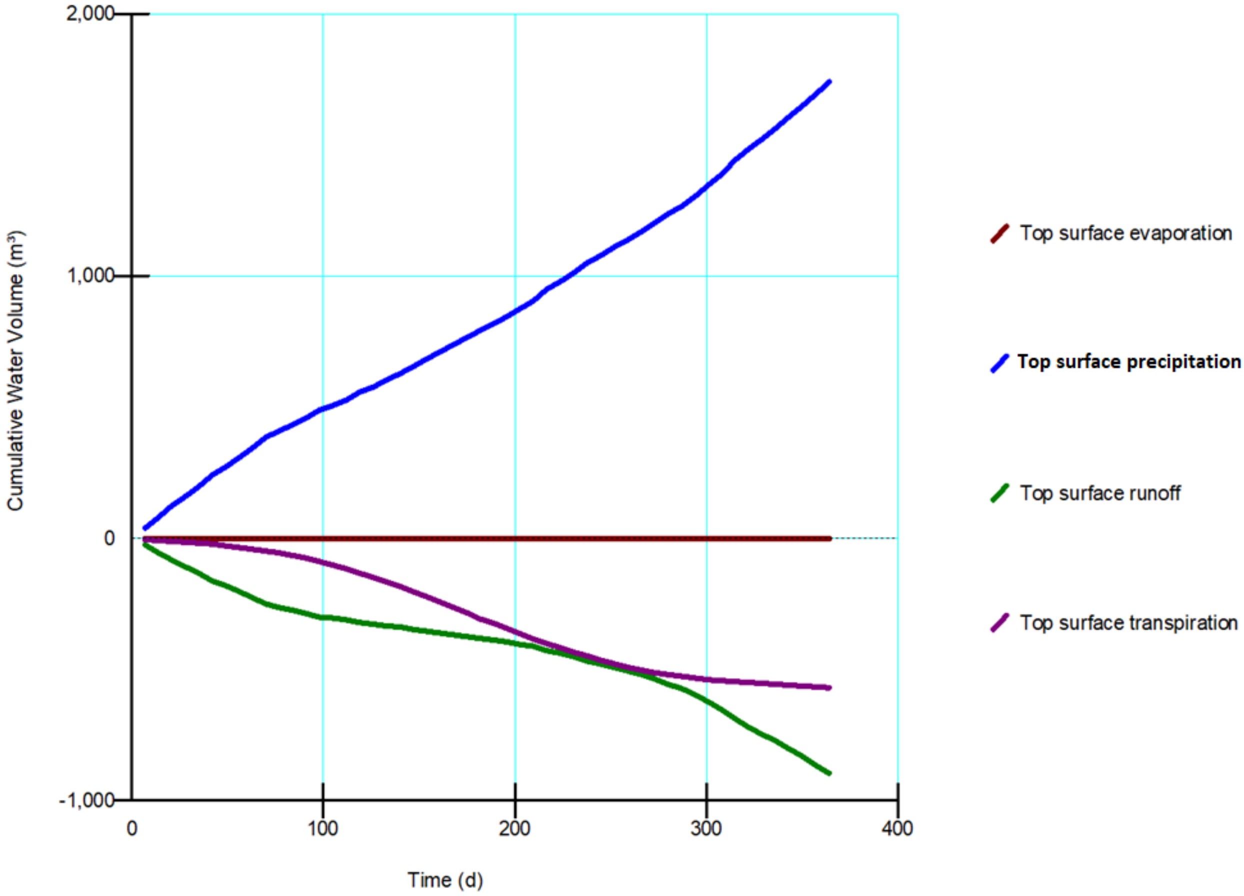

Figure 7: Cumulative Water Fluxes with Losses over the 1-year Simulation Period

Water that infiltrates into the model past the plant rooting layer either discharges as seepage from the facility side slopes or contributes to storage. However, discharge volumes due to seepage are small compared with those associated with surface runoff.

Quantitatively, only about 4.5% of the water which enters the PICs is attributable to sidewall seepage, and this occurs at discrete locations along the facility side slopes. Around 7% of the modelled sidewall seepage discharges directly into the PICs from the Inner Perimeter Wall (IPW), this being split equally between the North (Phase 1 BRDA) and South (Phase 2 BRDA) PICs.

Greater seepage rates are seen to arise along a free surface which develops along the broad ‘bench’ at the Stage 5 raise, with around 24% and 48% of the total sidewall seepage discharging from the north and south slopes at this level, respectively. The difference between the Phase 1 and 2 BRDA modelled discharges at this elevation is likely due to the presence of lower-permeability unfarmed bauxite residue beneath the Stage 5 in the Phase 1 BRDA.

A further 17% of sidewall seepage is attributable to discharges at the Stage 10 bench, which is again somewhat wider than those of other stage raises, and thus the phreatic surface meets the side-slope surface at this elevation. Modelled seepage volumes at Stage 10 are split roughly equally between the Phase 1 and Phase 2 BRDAs. Seepage discharging from both the Stage 5 and 10 benches is assumed to drain directly into the PICs via the rock fill blanket. The remaining 4% of seepage discharges from the side-slope of the dome and into the dome perimeter channels, which is then assumed to be routed along with surface runoff via the spillway channels to the PICs.

In summary, of the total modelled runoff which accumulates in the PICs, 95.5% arrives directly as surface water runoff from both the facility dome and side slopes. The remaining 4.5% emanates from the side slopes as seepage, and this is divided across four specific locations along the slopes, as discussed above. The cumulative modelled discharge due to seepage at these locations, for both the Phase 1 and Phase 2 BRDA slopes is shown in Figure 8.

<u>Note:</u> *The absolute volumes in this plot are not particularly meaningful since these relate to the 1m wide 'slice' through the BRDA represented by the model and have not yet been scaled to the surface area of the facility. As with the previous figure, slight seasonal effects are discernible in some of the time-series.*

A summary of the cumulative annual water fluxes through the model dome and side slopes is provided in Table 2. As above, the absolute volumes in this table pertain to the 1m wide slice represented by the model.

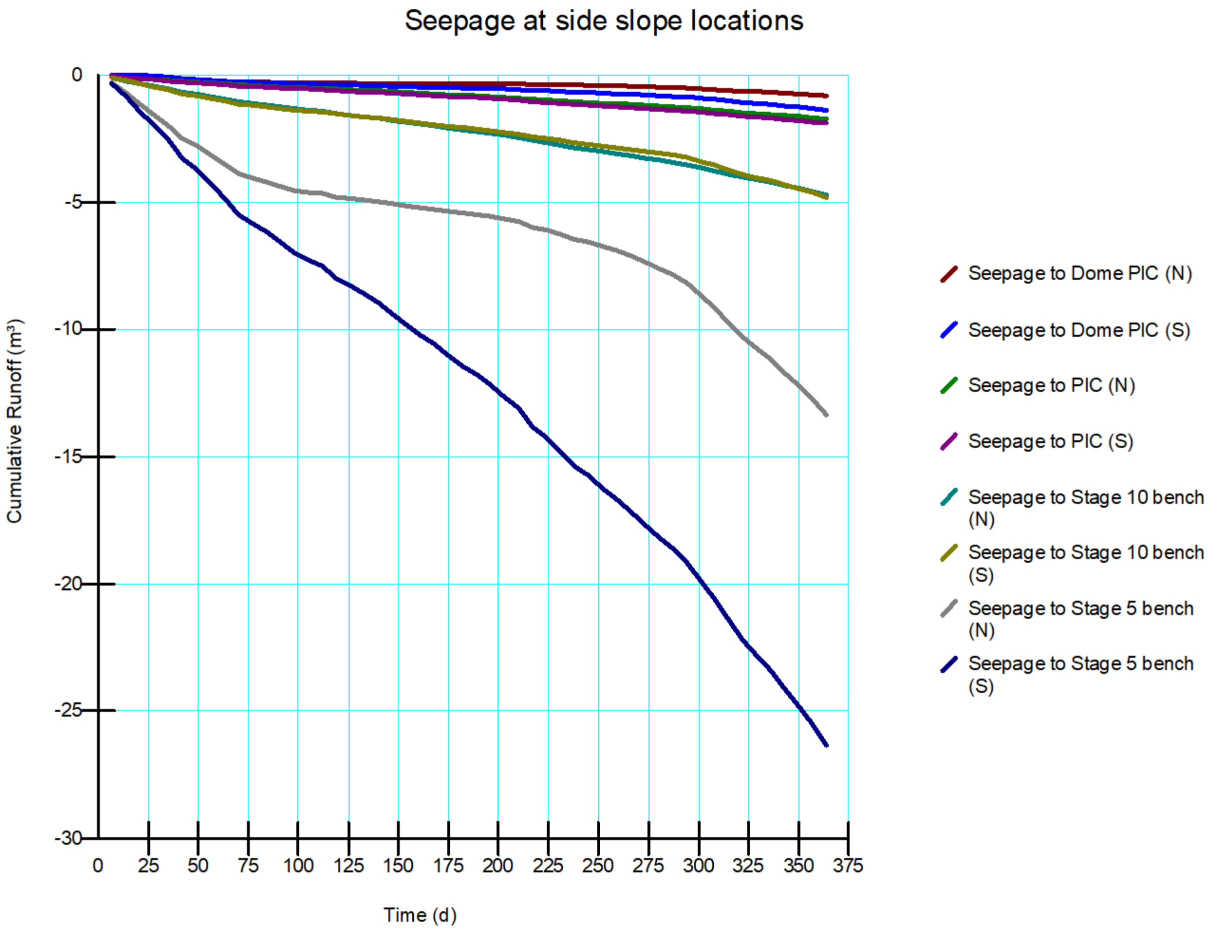

Figure 8: Cumulative Runoff due to Seepage at Discharge Locations along the BRDA side-slopes

Table 2: Modelled Cumulative Annual Water Fluxes (for 1m 'slice' through the BRDA)

Source / Destination	Cumulative Annual Water Flux (m^3/year)			
	Dome	Side-Slope (North Phase 1)	Side-Slope (South Phase 2)	Sum
Precipitation	1,255	240	250	**1,745**
Evapotranspiration	-464	-37	-36	**-537**
Surface Runoff	-843	-178	-159	**-1,180**
Net Infiltration				**28**
Sidewall Seepage (Into Dome PIC)	-	0.8	1.4	**2.2**
Sidewall Seepage (Stage 10 bench)	-	4.7	4.8	**9.5**
Sidewall Seepage (Stage 5 bench)	-	13.3	26.3	**39.6**
Sidewall Seepage (Into PIC)	-	1.7	1.9	**3.6**
Sidewall Seepage (Total)	**-**	20.5	34.4	**54.9**

Notes:

1. ***The water balance is not fully resolved by the model over a one-year period and the discrepancy can be attributed to storage and/or consolidation***.

3.3 Basal Seepage

The model constructed for this study can also be used to provide an estimate of the level of seepage through the base of the BRDA facility.

Simulation results indicate that the total volume of seepage through the base of the Phase 1 BRDA sector of the model is negligible. Even after scaling to the 104 ha. basal footprint of the Phase 1 BRDA, the total seepage over the 1-year simulation period is on the order of 2×10^{-8} m^3/year, see Table 3. This is considered to be due to the very low permeability of the (unfarmed) bauxite residue, which effectively becomes the primary liner for the Phase 1 BRDA in the long term. Similarly, seepage through the 80 ha. basal footprint of the Phase 2 sector of the model is so low (2×10^{-8} m^3/year, see Table 3) as to be effectively considered zero

Note: Assumes that the Phase 2 basal liner is free of any defects or imperfections. Previous studies (Golder 2005) have considered the potential impact of the presence of holes and tears in the basal liner, and a similar approach could feasibly be adopted for the current analysis. However, given the low permeability of the lining system and the significant depth of the overlying low permeability bauxite residue, it is considered unlikely that the presence of minor defects in the liner would make a substantive difference to the basal seepage rates once a threshold depth of approx. 10m is deposited.

3.4 Scaled Results

The modelled cumulative water fluxes discussed above relate to the 1m wide ‘slice’ through the centre of the BRDA represented by the model. These results could feasibly be used directly to provide quantitative estimates of surface runoff and sidewall seepage volumes for water quality modelling work. However, it is preferable to base the water quality modelling on cumulative water fluxes over the entire facility surface area, as this approach takes into account the fact that the ratio of side slope surface area to dome surface area is greater when calculated over the BRDA as a whole.

It is important to note the various assumptions that have been made to scale the two-dimensional model results to the entire BRDA surface area. In particular, it is noted that cumulative flux calculations for the BRDA dome and side slopes are treated separately. This is done to avoid either double-accounting of flux volumes over the dome surface, as would occur if the model results were simply multiplied by the facility perimeter, or under-representation of the total sidewall seepage volumes by neglecting to account for contributions from the eastern and western slopes of the facility.

Accordingly, total flux volumes for the BRDA dome are calculated by multiplying the modelled annual fluxes over the dome 'length' (~1,235 m) by the projected dome 'width' at Stage 16, which is estimated at 700m. By contrast, flux volumes for the BRDA slopes are calculated by multiplying the modelled sidewall seepage by the projected facility perimeter at Stage 8 (i.e. an average perimeter for the Stage 16 BRDA). This is estimated at 2.25 km and 2.35 km for the Phase 1 and Phase 2 sectors, respectively.

For the basal seepage, cumulative annual volumes are calculated by assuming a 104-ha footprint for the Phase 1 BRDA and an 80-ha footprint for the Phase 2 BRDA.

The resulting estimated annual water flux volumes through the BRDA dome, side-slopes and base are presented in Table 3. It is important to note that, due to the differential treatment of runoff and seepage volumes on the facility dome and side slopes, the relative water volumes generated by surface runoff and sidewall seepage are slightly different to those reported in Table 2. In particular, the proportion of water which enters the PICs that is attributable to sidewall seepage is now somewhat greater at approximately 6.3%. Whilst this is not substantially different to the ratio of seepage to runoff reported for the 2D model (~4.5%), it is likely preferable to use the greater estimate (i.e. the scaled results) as inputs to water quality modelling.

Table 3: Estimated Annual Water Fluxes through the BRDA Stage 16 Dome, Side-Slopes and Base

Source / Destination	Cumulative Annual Water Flux (m^3/year)			
	Dome	Side-Slope (North Phase 1)	Side-Slope (South Phase 2)	Sum
Precipitation	878,500	540,000	587,500	**2,006,000**
Evapotranspiration	-324,800	-83,250	-84,600	**-492,650**
Surface Runoff	-590,100	-400,500	-373,650	**-1,364,250**
Net Infiltration				**149,100**
Sidewall Seepage (Into Dome PIC)	-	1,800	3,290	**5,090**
Sidewall Seepage (Stage 10 bench)	-	10,575	11,280	**21,855**
Sidewall Seepage (Stage 5 bench)	-	29,925	61,805	**91,730**
Sidewall Seepage (Into PIC)	-	3,825	4,465	**8,290**
Sidewall Seepage (Total)	**-**	**46,125**	**80,840**	**126,965**
Basal Seepage (Phase 1 BRDA)	**~2 × 10^{-8}**			
Basal Seepage (Phase 2 BRDA)	**~2 × 10^{-11}**			

Notes:

1. ***The water balance is not fully resolved by the model over a one-year period and the discrepancy can be attributed to storage and/or consolidation.***

4.0 MODEL SENSITIVITIES

A formal model sensitivity analysis is not included in the scope of this study, and therefore no quantitative statements about the sensitivity of model outputs to different parameters can be made in this report. However, during construction of the model a number of observations about particularly influential parameters were made. These are discussed in this section.

As noted in the discussion of the modelled sidewall seepage in Section 3.2, a significant proportion of the water which enters the model as infiltration is lost to the atmosphere as transpiration. This process is primarily mediated by the parameters that specify the characteristics of the top surface vegetation cover in the LCI boundary condition. In particular, it is noted that the rooting depth parameter has a strong influence on the rate and amount of water lost via transpiration. The rooting depth parameter value of 0.3 m was established based on guidance (Ramirez-Garcia, Almendros and Quemada 2012) and is at the lower end of the range suggested for grassland (0.3m to 0.5m), on the basis that vegetation productivity may be suppressed by the high-pH soils which make up the amended bauxite capping layer.

The total volume of water lost to transpiration influences the amount of water rejected as surface runoff, and therefore the relative volumes of runoff and seepage are sensitive to the rooting depth and other parameters which control the rate of transpiration. Accordingly, the uncertainty associated with these parameters directly influence the uncertainty associated with the model outputs.

A further observation from model construction is that the depth to the phreatic surface is primarily controlled by the material properties of the farmed bauxite residue. This is consistent with conceptual understanding of the site hydrogeology since this material makes up the bulk of the BRDA, and therefore controls both the rate of vertical percolation through the facility (via the vertical component of conductivity), as well as the rate of seepage through the side slopes (mainly via the horizontal component of conductivity).

As noted in Section 2.3, the hydraulic properties of the stage raise rock fill material, and in particular the slope of its volumetric water content (VWC) function, were found to have a strong influence on model convergence. Including adjacent materials with extreme contrasts in material properties in the model (in this case, seven orders of magnitude difference in K_{sat} for the unfarmed bauxite and stage raise rock fill materials) presents a significant numerical challenge to the modelling software, making convergence extremely difficult to achieve. Therefore, a slightly less aggressive VWC function was assigned for the stage raise rock fill material by assuming sand-like, rather than gravel-like, soil moisture retention characteristics. This modelling decision was justified on the basis that some fines are likely to migrate into the stage raises from the bauxite residue, thereby altering their hydraulic properties over time.

5.0 UNCERTAINTY AND MODEL LIMITATIONS

The numerical model developed for this study is a greatly simplified representation of the conceptual model for the BRDA and has been designed to address the specific objectives of this study, which are primarily to provide inputs for geochemical water quality modelling. Accordingly, there are a number of model assumptions and uncertainties that should be considered when interpreting the model results and making decisions based upon them. These include the following:

- As noted in the discussion of modelling objectives in Section 1.4, the approach adopted for this study assumes that the BRDA is at Stage 16 (i.e. post-closure) from the start of the simulation period, and no modelling of the construction phase is included. Furthermore, it is assumed that the hydraulic properties of the model's constituent materials, as well as the characteristics of the vegetation cover on its upper surface, remain constant over the simulated period.
- As discussed in Section 2.3, the hydraulic properties of the modelled materials – and in particular, those of the amended bauxite layer – are uncertain, in part due to the variability of laboratory and *in situ* test results, as well as the inherent spatial heterogeneity of hydraulic properties within a given material type. To the extent that the model predictions of interest are sensitive to the material properties, there will be a resulting level of uncertainty associated with those predictions.
- Current-day phreatic surfaces are measured via piezometers at 1m to 3m below surface on the facility side slopes, and several metres below the upper surface i.e. at Stage 9 or Stage 10. It is expected that the phreatic surface for the BRDA will drop further following closure due to the dome shape, the vegetative cover and the low permeability of the bauxite residue (amended and farmed layers) reducing the opportunity for infiltration and recharge. However, a 'worst-case' scenario to maximize seepage is adopted for this study, with the initial phreatic surface assumed at close to the surface for the side-slopes and the upper surface providing high driving heads for the sidewall seepage rates.
- As noted in the discussion of model results in Section 3.2, surface runoff generated on the facility dome is assumed to be routed directly into the PICs via dome perimeter interception channels and spillways, without any significant losses. Uncertainty regarding the proportion of surface runoff which enters the PIC will influence the uncertainty associated with the relative proportions of runoff and seepage in the PICs.
- As discussed in Section 4.0, there is significant uncertainty associated with the LCI boundary condition parameters which specify the characteristics of vegetation on the model's upper surface. In particular, the modelled transpiration rate was found to be highly sensitive to the plant rooting depth parameter, and therefore so too are the model results relating to surface runoff.
- As noted in the discussion of model results in Section 3.4, a number of assumptions are made when scaling the model results to the BRDA surface area. These are discussed in greater detail in the relevant section, where it is noted that the scaled water volumes should be treated with caution, given the assumptions which underpin their calculation.

6.0 CONCLUSIONS

A two-dimensional numerical model has been constructed in SEEP/W to provide an assessment of potential seepage from the restored BRDA to Stage 16. The modelled design takes into consideration the changes in the lining system and material properties of the material deposited in the BRDA over time and the proposed restoration with grass at site closure.

Based on the modelling results, simple calculations have been performed to scale the two-dimensional model outputs to cumulative annual flux volumes over the BRDA facility as a whole. The results of this assessment predict the following:

- Of the total water that accumulates in the PIC due to surface runoff and sidewall seepage, 93.7% arrives directly as surface water runoff from the dome and side slopes of the facility;
- The remaining 6.3% emanates from the facility slopes as sidewall seepage, and this is divided across four specific locations along the sidewalls – the Stage 5 bench, the Stage 10 bench and seepage directly into both the facility PICs from the Inner Perimeter Wall (IPW) and into the dome perimeter channels; and
- There is negligible seepage through the base of the facility, either in the unlined or lined phases.

  The Phase 2 BRDA is underlain by a composite geosynthetic lining, which even assuming that it is free of any defects or imperfections, is susceptible to seepage, although at a very minor level due to the low hydraulic conductivity properties of the composite lining materials. For both the Phase 1 and Phase 2 BRDAs, the deposited bauxite residue effectively becomes the primary liner in the long term as a result of its low hydraulic conductivity properties and its significant depth, in comparison to the thickness of either the underlying geosynthetic layers and/or the estuarine soils present.

Based on observations made during the modelling, it is evident that the model is sensitive to input parameters such as rooting depth (which has a strong influence on the rate and amount of water lost via transpiration), and the hydraulic properties of the stage raise / rock blanket rock fill.

Given the inevitable uncertainties in numerical modelling and the model sensitivity to various input parameters, then the model results presented in this report should be considered within the decision-making process alongside other information, such as experience of ground conditions at the BRDA, monitoring data, and other design considerations.

In the event that there are any changes to the design assumptions, material properties etc., used in the model, then the modelling approach should be revisited and examined to confirm it remains appropriate and relevant to those changes.

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APPENDIX I

BRDA Hydrological Assessment (Water Balance)

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REPORT

Hydrological Assessment for the Perimeter Interceptor Channels, Storm Water Pond and Liquid Waste Pond

BRDA Raise Development to Stage 16

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Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

Golder Associates Ireland Limited (Golder) has been engaged by Aughinish Alumina Limited (AAL) to undertake a hydrological assessment to appraise the capacities of the Perimeter Interceptor Channels (PICs), Storm Water Pond (SWP) and Liquid Waste Pond (LWP) to inform the engineering design of the proposed BRDA Raise Development constructed to Stage 16.

The proposed BRDA Raise Development will comprise the raising of the facility on the existing footprint to Stage 16 (36 mOD perimeter crest elevation). The Phase 1 BRDA is currently constructed to Stage 10 and the Phase 2 BRDA is currently constructed to Stage 4.

The objective of this report is to present the methodologies used for and outcomes from the hydrological assessment, and to provide recommendations for improvements to the existing BRDA water management system for the engineering design for the proposed BRDA Raise Development.

2.0 BACKGROUND

AAL is wholly owned by United Company RUSAL and operates the alumina refinery situated on Aughinish Island on the south side of the Shannon estuary. The island is located between Askeaton and Foynes and is approximately 30 km west of Limerick and 10 km southwest of Shannon Airport. The Island has an area of approximately 400 ha and is bounded by the River Shannon to the north, the Robertstown River to the west and south-west and the Poulaweala creek to the east and south-east. The Phase 1 BRDA is located southwest of the Plant Site and is formed from two facilities, the original Phase 1 BRDA, covering an area of 72 ha and the eastern Phase 1 BRDA Extension covering an area of 32 ha. The Phase 2 BRDA adjoins the southern extent of the Phase 1 BRDA and covers an area of 80 ha.

For the proposed BRDA Raise Development, the design criteria for the BRDA water management system have been selected to be in accordance with the Canadian Dam Association (CDA) (2007) and (2014) Guidelines. The BRDA has been identified to have a "**High**" hazard potential classification (HPC) rating under the CDA Guidelines and therefore the Inflow Design Flood (IDF) will be 1/3 between the 1,000-year and the Probable Maximum Flood (PMF) events. The assessment considers the ultimate operational configuration for the BRDA, i.e., BRDA fully operational and constructed to Stage 16, as this configuration is considered to be representative of the worst-case operational conditions for assessment of the BRDA water management system.

Golder previously completed an assessment of the performance of the existing water management system for various design flood events for the existing Stage 10 BRDA facility (Golder, 2019); this technical memorandum provided recommendations for improvements to the system to accommodate the IDF. However, this was a preliminary assessment and did not consider the PIC culverted 'choke points' in the Phase 1 and 2 BRDA PICs.

During 2019, SLR Consulting was retained by AAL to conduct an independent dam safety review (DSR) of the BRDA (SLR 2019). The DSR provided a number of recommendations related to the BRDA water management system. The following recommendations from the DSR have been considered in this assessment:

- A storm duration of 24 hours has been considered; this duration is commonly used for hydrological analysis when the intent is to maximise the volume of water to be stored in the facilities of the water management system;
- The time of concentration calculations for the Phase 1 and 2 BRDAs have been revised, and the resulting values input to hydrologic / flood routing modelling; and
- The pump operation / capacities have been reviewed to assess adequate available capacity in the BRDA water management system to accommodate the IDF.

3.0 SITE WATER MANAGEMENT SYSTEM DESCRIPTION

The following sections of this report provide a description of the existing BRDA water management infrastructure, as well as a description of proposed upgrades to this water management system required to accommodate the IDF. A description of the existing Plant Site water management system is also provided, as a portion of the surface runoff generated on the Plant Site catchment is discharged to the BRDA water management system.

3.1 BRDA and Plant Site Water Management System

3.1.1 Existing BRDA Water Management System

The BRDA is surrounded by PICs, which collect bleed water and runoff from the Phase 1 and Phase 2 facilities and convey it via pumps either to the Effluent Clarifier System (ECS) or to the SWP. It is formed by the construction of the outer and inner perimeter embankment walls, with the inner embankment wall also being the starter stage raise (Stage 0).

The existing PICs are separated into PIC segments (PIC-A to PIC-L) that are separated by culverted 'choke points'; these culverted sections provide vehicular access to the BRDA across the PICs. An additional PIC (PIC-M) is planned as part of the current BRDA (to Stage 10); PIC-M is not yet constructed but will be located at the southeast corner of the Phase 2 BRDA.

The SWP and LWP are located in the north-east sector of the Phase 1 BRDA. Drawings 01 and 02 provided in Appendix A show the existing site layout and the proposed BRDA Raise Development is shown on Drawing 03.

Drawing 02 shows the layout of the existing BRDA water management system. There are seven (6 no.) Phase 1 PIC segments that collect runoff from the Phase 1 BRDA and one (1 no.) Phase 2 PIC segment (PIC-M) that collects runoff from the north-east sector of the Phase 2 BRDA that contributes directly to the Phase 1 PIC flows:

- From the southwest corner of the Phase 1 BRDA, water flows clockwise through PIC-E, PIC-F and PIC-G to a sump where water is pumped to the ECS or to the SWP.
- From the north-east corner of the Phase 2 facility and the southeast corner of the Phase 1 facility, water flows counter-clockwise through PIC-M (to be constructed), PIC-L, PIC-K and PIC-J to the sump (as above).

  Note: *At the northeast corner of the Phase 2 BRDA, PIC-M will flow counter-clockwise to connect with PIC-L, located at the southeast corner of the Phase 1 BRDA Extension. PIC-M is not yet constructed as the bauxite residue has not attained the design elevation of for the base of the channel. It is expected that PIC-M will be formed during 2022 / 2023.*

There are four (4 no.) Phase 2 PIC segments that collect runoff from the Phase 2 BRDA and transfer it to the Phase 1 PIC:

- Water flows clockwise around the Phase 2 facility to its northwest corner (PIC-A to PIC-D). Water is pumped from the northern extent of PIC-D to the southwest corner of the Phase 1 PIC (PIC-E); there are also three overflow culverts installed here which permit overflow from PIC-D to PIC-E.

Table 5 in Section 5.4 and Drawings 08 to 12 in Appendix A show the dimensions of constructed and future PIC sections. The water collected in the Phase 2 PICs is pumped into the Phase 1 PICs and subsequently to the ECS or to the SWP. The function of the SWP is two-fold:

- To provide surge capacity for surface water that cannot be immediately processed by the ECS; and

- To provide a continuous flow of recycled water that is used for dilution or wash water within some parts of the alumina plant.

Excess water from the SWP is pumped to the ECS. The SWP does not currently have an overflow spillway (during operation) but will have a breach installed during the closure works for the post-closure period.

The LWP receives treated water from the ECS, where the water cools and settles prior to discharge to the licensed emission point. The LWP also provides water for BRDA dusting prevention during the summer.

Details of the existing BRDA pumping systems are summarised in the update of extreme rainfall and inflow design floods for the PIC and SWP (Golder 2019), which is provided in Appendix C.

The current BRDA water inventory targets are presented below; AAL's Control Room Operator (CRO) is responsible for ensuring the inventory targets are met:

- **Winter** (October – March): 110,000 m^3 to ensure storage capacity for stormwater.
- **Summer** (May – August): 180,000 m^3 to provide sufficient pre-treatment water storage prior to processing by the ECS and discharge for BRDA dusting prevention.
- **Transition Months** (April and September): 150,000 m^3.

Note: *The existing BRDA water inventory definition includes water stored in the PIC system and the SWP but does not include water stored in the LWP.*

3.1.2 Existing Plant Site Surface Water Management System

The Plant Site is the area where alumina refining activities are undertaken. Hydrologically, the Plant Site is divided into three main areas as shown on Drawing 04 and as follows:

- **Northern Area**: surface water runoff from this Raw Materials & Produce Storage Area (Non-Process) area is uncontaminated and discharges directly off site;
- **East Catchment**: surface water runoff from this area is potentially contaminated and drains to the East Pond for storage / attenuation prior to being pumped to the ECS; and
- **West Catchment**: surface water runoff from this area is potentially contaminated and drains to the West Pond for storage / attenuation prior to being pumped to either the ECS or to the Phase 1 BRDA PIC.

Within the east and west catchments there are process area sub-catchments (labelled 'PBI Boundaries' on Drawing 04) where surface water runoff is initially drained onto the various process areas' bunded slabs. Up to 10,000 m^3 of surface water runoff will be pumped into process vessels from these catchments during a storm event.

There is also a small catchment area draining to the North Pond (or 'Containment Pond'), which is used to contain process water if there is an issue with the process system; otherwise, this pond typically remains unused. The North Pond can be used to provide additional storage / attenuation volume for surface water runoff if required.

The East and West catchments are comprised of the following main land cover types:

- Greenfield Areas with grass cover;
- Hardstand Areas (various e.g., road paving, concrete, roofs etc.); and
- Process Areas where surface water runoff is collected in sumps and used in the Plant Site process system.

Runoff generated from the East and West catchments (with the exception of runoff from the process area sub-catchments) is routed to the East and West Ponds (respectively), via a gravity drainage system, comprising open channels, pipes and culverts.

The Plant Site water management system considered for the hydrological assessment is presented conceptually by the block flow diagram in Figure 1 below and provided on Drawing 07 in Appendix A. The results of the Plant Site hydrological analysis are presented in Section 9.0.

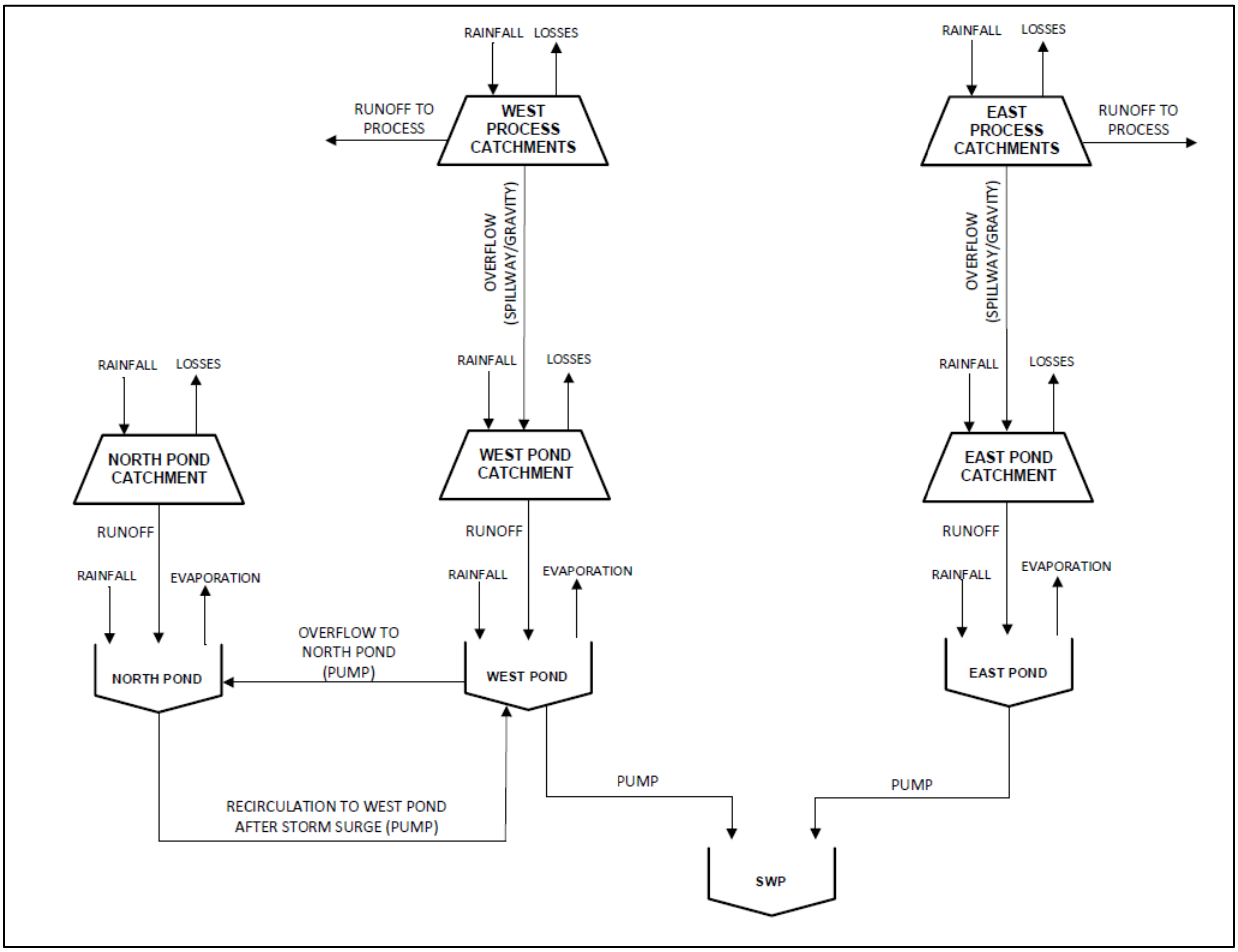

Figure 1: Plant Site Water Management System - Block Flow Diagram (see Drawing 07)

Notes:

1) *The North Catchment is not presented in the flow diagram as this area discharges directly off site.*

2) *Catchment "Losses" presented in the flow diagram represent all hydrological losses from rainfall including evaporation, transpiration, infiltration, and losses due to surface depressions and ponding.*

3) *As outlined above, the East and West Pond discharge to the ECS and the Phase 1 BRDA PIC system (which ultimately discharges to the ECS directly or via the SWP). For the purposes of this hydrological assessment (and the corresponding flow diagram), these ponds have been modelled as discharging to the SWP (which ultimately discharges to the ECS). This is due to:*

   i) *Limitations of the software used for the flood routing and storage capacity assessment (Section 8.0); and*

   ii) *A recommendation outcome from this study, that future flows discharging from the Plant Site to the BRDA water management system are discharged to the SWP rather than the Phase 1 PIC. This is intended to reduce the volume of water discharging to the PIC during the IDF and reduce the overall PIC pumping capacity required to accommodate the IDF.*

3.1.3 Proposed Upgrades to the BRDA Water Management System for the BRDA Raise Development

The following upgrades to the BRDA water management system are proposed for the proposed BRDA Raise Development, which have been informed through the analysis undertaken in this study and are inherent in the methodology and results presented in this report. Drawing 03 provided in Appendix A show the proposed site layout for the BRDA Raise Development.

- **PIC-A:** The channel for PIC-A to discharge directly to PIC-B is scheduled to be constructed during Q3 2021, and runoff from the bulk of the Phase 2 BRDA will flow clockwise from the northeast corner.
- **PIC-B to PIC-G:** Proposed increase to the crest elevation of segments PIC-B to PIC-G to 5.3 mOD. This is intended to provide additional storage capacity within the PIC system during the IDF and will be achieved through a vertical raise of downstream crest liner which will be supported by the existing crash barrier located along the PIC perimeter. The existing crest elevations of PIC-B to PIC-D and PIC-E to PIC-G are 5.0 mOD and 4.7 mOD, respectively.
- **PIC D:** Proposed replacement of the existing three (3 no.) 0.3 m ID overflow culverts from PIC-D to PIC-E with two (2 no.) concrete box culverts (min.1.1m wide x 0.55m high), installed side-by-side, to provide improved conveyance capacity to accommodate the IDF (See Table 7 in Section 5.4 for detailed culvert information).
- **PIC-M:** A new PIC (PIC-M) will be constructed at the northeast corner of the Phase 2 BRDA / southeast corner of the Phase 1 BRDA which will allow runoff to travel in a counter-clockwise direction from this area to PIC-L and then to PIC-K located at the northeast corner of the Phase 1 BRDA and directly south of the SWP. This will require the reconstruction of the existing ramp to the 'Merger Road' and the installation of a concrete box culvert to convey flows from PIC-M to PIC-L.

  Note: *The 'Merger Road' was the original perimeter road for the Phase 1 BRDA which subsequently separated the Phase 1 and Phase 2 BRDA basins.*
- **PIC-L:** The following upgrades are recommended for PIC-L:
  - A culverted embankment crossing is proposed to sub-divide the existing PIC-L into PIC-L (South) and PIC-L (North), as indicated on Drawing 03. The purpose of this is to provide for flood attenuation storage within PIC-L (South) during the IDF by adding a culverted 'choke point' to attenuate flood discharges to the downstream water management system. Minimal flood storage volume is available in PIC-L (North) due to its steep invert gradient (approx. 1.3%), narrow base width (approx. 3.75 m) and low embankment crest level (approx. 11.5 mOD). However, PIC-L (South) will have a shallower invert gradient (approx. 0.4%), wider base width (approx. 15.75 m) and higher embankment crest elevation (16.0 mOD) allowing for significant attenuation storage.
  - Increase of the exiting PIC-L (North) embankment crest elevation by approximately 1m height to 12.5 mOD, to provide additional storage capacity and prevent overtopping of the PIC during the IDF. The existing pipes draining this PIC discharge to a small intermediate pond prior to being culverted to PIC-K. This intermediate pond is unlikely to accommodate the IDF and hence replacement with a direct culvert between PIC-L (North) to PIC K has been recommended (See Table 6 in Section 5.4 for detailed culvert information). The design for a replacement spillway structure is provided in Appendix L.
- **PIC K:** The following upgrades are recommended for PIC-K:
  - Proposed decommissioning of the existing culvert linking PIC-K to PIC-J, and installation of a pump and overflow culverts which will discharge flows from PIC-K to the SWP. The purpose of this

improvement is to reduce the volumes of water discharging to PIC-G (via PIC J) and consequently minimise the PIC-G pump capacity upgrades required to accommodate the IDF within PIC-G.

- The PIC-K pump is intended to accommodate flows during regular meteorological conditions, while the overflow culverts are intended to accommodate flood flows up to the IDF. Refer to Sections 5.4 and 8.0 for information relating to the proposed overflow culvert and pump arrangements, respectively.

- **PIC-G Pump Capacity:** Proposed upgrade of the PIC-G pumping capacity to allow the IDF to be accommodated within the PIC system. Please refer to Section 8.0 for information relating to the proposed pumping capacity.
- **Plant Site Discharges:** During a flood event pumping from the East and West Ponds on the Plant Site is proposed to be discharged to the SWP / ECS only and not discharge to the Phase 1 PIC, to minimise the overall PIC pumping capacity from PIC-G required to accommodate the IDF.
- The proposed BRDA Raise Development water management system is presented conceptually by the block flow diagram in

Figure 2 below and provided on Drawing 05 in Appendix A. The results of the flood routing analysis of the upgraded BRDA water management system is presented in Section 8.0.

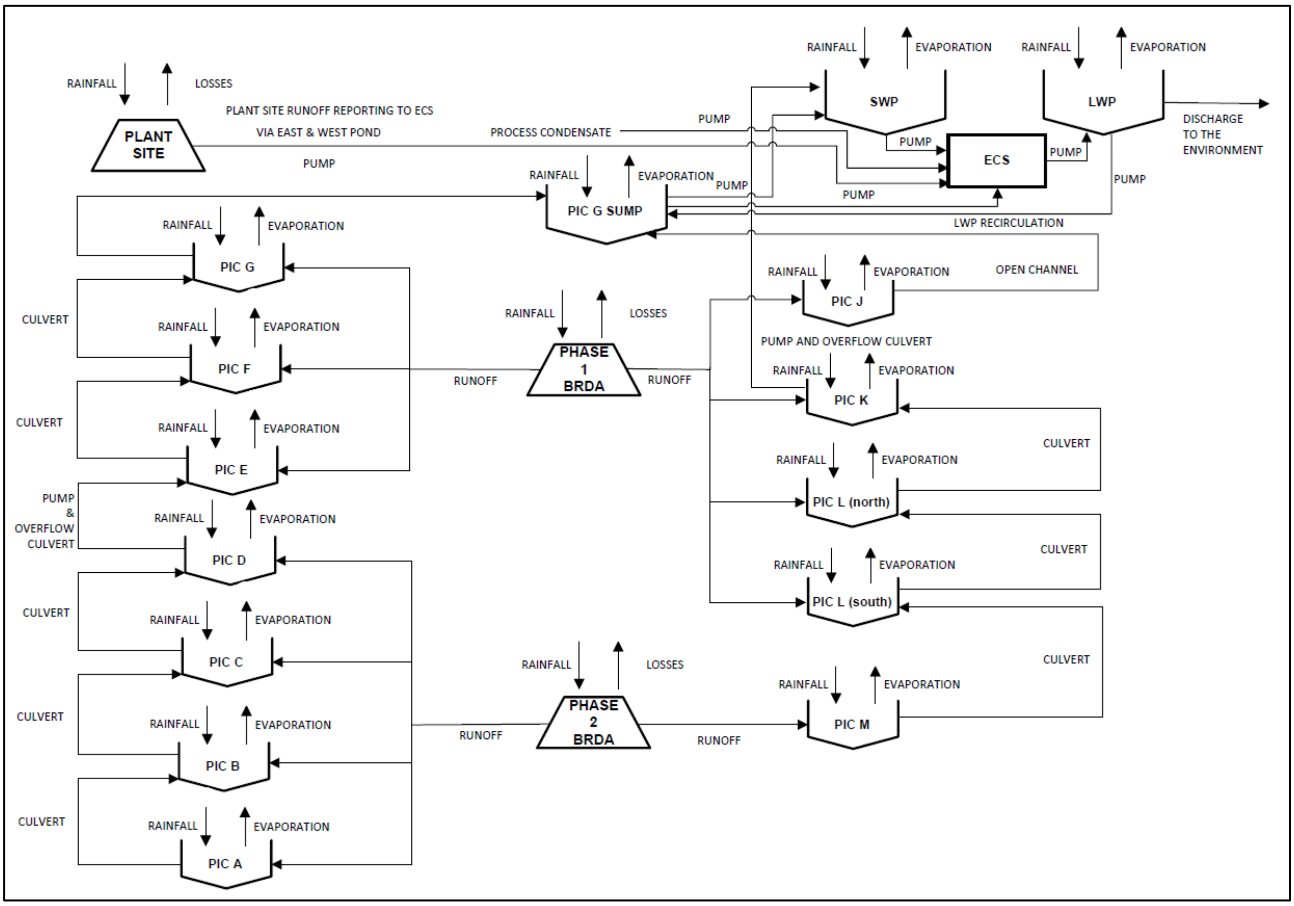

Figure 2: Proposed BRDA Raise Development Water Management System - Block Flow Diagram (see Drawing 05)

Notes:

1) *Catchment "Losses" presented in the flow diagram represent all hydrological losses from rainfall including evaporation, transpiration, infiltration, and losses due to surface depressions and ponding.*

4.0 APPROACH / METHODOLOGY

The hydrological assessment of the proposed BRDA Raise Development water management system consisted of the following steps:

- Hydrological analysis of the Plant Site catchments and assessment of discharge rates from the Plant Site to the BRDA water management system. Runoff from the selected design rainfall events was routed through the Plant Site water management system using the US Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modelling System (HEC-HMS).
- Assessment of the inter-PIC flows for use in water balance modelling. The capacities of the PIC culverted 'choke points' were assessed using the US Department of Transportation Federal Highway Administration's HY-8 Culvert Hydraulic Analysis Program. The maximum discharges before overtopping were determined based on an assumed tailwater (water level at downstream end of the culvert) at the downstream end of the pipe culverts.
- Evaluation of the maximum operating water levels in the PICs, SWP and LWP under normal operating conditions for use as initial water levels in hydrologic / flood routing modelling. The 75th percentile water levels resulting from water balance modelling were assumed to represent the upper limit of normal operating conditions. The 75th percentile represents the value below which 75% of the modelled levels occur.
- Assessment of the performance of the PICs under IDF conditions using hydrologic modelling. The IDF was routed through the PICs using HEC-HMS. Improvements to the BRDA water management system were made where necessary to ensure the PICs did not overtop during this flood event (see Section 3.1.3).
- Assessment of the capacity of the SWP to manage inflows during the IDF using hydrologic modelling. The IDF was routed from the PICs through the SWP using HEC-HMS. Inflows from the Plant Site hydrological model (pumping from the East and West Ponds to the SWP) were also incorporated.

5.0 DESIGN BASIS

The main components of the design basis, upon which the hydrological analyses have been undertaken, are presented in the following sections:

- The BRDA Inflow Design Flood (Section 5.1);
- The Plant Site Design Flood (Section 5.2);
- BRDA and Plant Site Catchment Data (Section 5.3);
- Perimeter Interceptor Channel and Culvert Data (Section 5.4); and
- SWP, LWP and ECS Data (Section 5.5).

Information relating to the proposed improvements to the PICs has been presented previously in Section 3.1.3. Information relating to the assessment of PIC pumping requirements and capacity of the SWP to accommodate the IDF are presented in Section 8.0; while Section 9.0 presents information relating to the hydrological assessment of the Plant Site.

5.1 BRDA Inflow Design Flood

Rainfall frequency analysis for the BRDA has previously been undertaken by Golder (2020); Table 1 presents the results of this analysis for a range of design events and durations.

Table 1: Design Rainfall Depths (BRDA)

Rainfall Duration (hrs)	Rainfall Depths (mm)			
	200-year	1,000-year	1/3 between 1,000-year and PMP	PMP
6	58.9	80.9	102.2	144.7
12	70.2	93.6	120.6	174.6
24	83.7	107.6	141.0	208.0

In keeping with recommendations provided by SLR (2019), an event duration of 24 hours was considered for the purposes of the analyses described in this report.

Based on guidelines provided by the CDA (2007 and 2014), the selection for the BRDA of a “**High**” HPC determines the appropriate IDF to be the 1/3 between the 1,000-year and the PMF events. Therefore, the design rainfall depth considered for the purposes of flood routing was 141.0 mm.

Notes:

1. *The PMF is the most extreme meteorological event, among extreme events, corresponding to a theoretical maximum flood with an undefined return period (i.e., greater than 1 in 10,000 years). The methods for estimating the PMF include accounting for climate change (WMO 2009) and no additional factors are required to be applied to the PMF or the IDF (which is derived from the PMF).*
2. *In aftercare, the PIC is breached at two locations and spillways have been designed to transfer the IDF to discharge; the BRDA water management system is no longer required to contain the IDF, as iit would be in operational life (which is limited).*

The 50% summer design storm profile recommended in Volume 4 of the Flood Estimation Handbook (Houghton-Carr, 1999) was applied to the IDF rainfall depth. This storm profile represents the one that is, on average, peakier than 50% of UK summer storms and peakier than the 75% winter storm profile. These design storm profiles were originally developed under the Flood Studies Report (NERC, 1975), which was a study based on extensive research and analysis of available hydrometric and rainfall records in Britain and Ireland; and is therefore considered applicable for Ireland (Cunnane & Lynn, 1975).

The estimated peak runoff rates and runoff volumes to each PIC segment catchment are presented in Section 8.0.

5.2 Plant Site Design Flood

Hydrological annual exceedance (return period) criteria are used to design water management infrastructure. It is not economical to design structures for a limiting value and criteria are selected based on an acceptable level of risk of failure within the expected lifetime of the structure. High risk facilities (e.g., dams) are typically designed using more stringent (higher return period) hydrologic design criteria compared to lower risk facilities (e.g., road culverts).

As the Plant Site does not form part of the BRDA facility, the CDA guidelines for design rainfall events are not applicable to the Plant Site, given the CDA guidelines are intended for the design of dams / large impoundment facilities which pose a significantly higher hazard should their design criteria be exceeded. Additionally, the BRDA facility is designed to function following cessation of operations at the site into the post-closure period; however, the Plant Site will have a relatively short life span as it will be decommissioned following cessation of operations at the site. Therefore, more stringent (high return period) hydrologic design criteria are warranted for the BRDA.

As cited in the ‘Flood Risk Management Plan – Shannon Estuary South’ (OPW, 2018), the preferred standard of protection offered by flood protection measures in Ireland for fluvial flooding is the 100-year flood event. For drainage system design, ‘The Planning System and Flood Risk Management Guidelines for Planning Authorities (Technical Appendices)’ (OPW, 2009) refer to the ‘Greater Dublin Regional Code of Practise for Drainage Works (Dublin City Council, 2020).

In the absence of a comprehensive national design standard; this code of practice states the following requirements for stormwater drainage design:

- “Current design criteria normally require no flooding occurs up to the 30-year return period, and properties are protected against flooding for the 100-year return period”; and
- “Attenuation structures such as ponds have to have the ability to deal with events up to a 100-year return period”.

On this basis, the 1 in 100-year storm plus an allowance for Climate Change, is considered to be an appropriate standard of protection for the Plant Site water management system. Therefore, for the purposes of this assessment, storm water runoff discharging to the BRDA water management system from the Plant Site has been assessed for the 1 in 100-year +20% rainfall event, as set out in Table 2 and explained in the notes below.

The rainfall time series applied for the flood routing modelling was established by applying the Flood Estimation Handbook (FEH) - Summer storm distribution (Houghton-Carr, 1999) to the design rainfall depth.

Table 2: Design Rainfall Depths (Plant Site)

Rainfall Duration (hrs)	Rainfall Depth (mm)	
	100-year	100-year plus Climate Change [1]
24	75.9	**91.1**

Notes:

1) *A climate change allowance of +20% has been applied to the design rainfall depth in accordance with the Limerick County Development Plan (as extended) – Strategic Flood Risk Assessment (Limerick City & County Council, 2018).*

Extreme low probability rainfall events in excess of the 1 in 100-year storm, i.e., the IDF, would result in the generation of additional runoff volumes which AAL proposes to retain and manage within the Plant Site up to the BRDA IDF rainfall event. The Plant Site hydrological analysis and results are presented in Section 9.0.

5.3 Hydrological Catchment Data

5.3.1 BRDA Catchment Data

Golder has reviewed the topography and catchment data for the proposed (ultimate configuration) BRDA Raise Development, which will see the final elevation of the Phase 1 and 2 BRDAs increased to a top dome elevation of 44 mOD with a perimeter crest elevation of 36 mOD.

The BRDA has been divided into sub-catchments which drain to respective PIC segments. The characteristics for each sub-catchment, which have been used in the hydrological assessment, are presented in Table 3. Times of concentration were estimated using the Watershed Lag Method (NRCS, 2010). The selected curve number for the ultimate BRDA configuration corresponds with a contoured row crops soil cover, on hydrologic soil group D, with poor hydrologic condition (NRCS, 1986).

Table 3: BRDA Catchment Characteristics

Catchment	Area (km^2)	Curve Number	Lag Time (mins)	Time of Concentration (mins)
Phase 2 BRDA				
PIC-A	0.1637	88	22	36
PIC-B	0.1387	88	19	32
PIC-C	0.2389	88	24	39
PIC-D	0.1840	88	17	28
Phase 1 BRDA				
PIC-E	0.3057	88	23	39
PIC-F	0.1201	88	14	23
PIC-G	0.1606	88	21	34
PIC-J	0.0747	88	15	24
PIC-K	0.1042	88	9	15

Catchment	Area (km²)	Curve Number	Lag Time (mins)	Time of Concentration (mins)
PIC-L (North)	0.0190	88	6	10
PIC-L (South)	0.1813	88	17	29
PIC-M	0.1192	88	12	19

Notes:

1) *PIC-M has been included in the catchment for the Phase 1 BRDA, although it will be located in the north-east sector of the Phase 2 BRDA, as it will flow counter-clockwise to contribute directly to PIC-L without pumping required.*

The lag times / times of concentration determined for each sub-catchment to the Stage 16 elevations are considerably shorter than those estimated previously for the total Phase 1 BRDA and Phase 2 BRDA catchments (Golder, 2019). This is the result of a number of factors including the assessment of the sub-catchment to each PIC segment individually (reduced flow lengths) and the increase in the elevation of the top of the BRDA for the proposed BRDA Raise Development, resulting in steeper sub-catchment slopes.

5.3.2 Plant Site Catchment Data

Golder has reviewed available information relating to the Plant Site and assessed the characteristics for those catchments draining to the East, West and North Ponds.

The characteristics for each Plant Site sub-catchment, which have been used in the hydrological assessment, are presented in Table 4 below.

Times of concentration were estimated using the Watershed Lag Method (NRCS, 2010).

Table 4: Plant Site Catchment Data

Catchment	Area (Km²)	% Impervious	Curve Number	Lag Time (mins)	Time of Concentration (mins)
East	0.2190 [3]	70	91 [1]	30	49
West	0.2321 [3]	71	91 [1]	43	72
North Pond	0.0154	100	98 [2]	10	17
PBI East	0.0567	100	98 [2]	4	6
PBI West	0.0315	100	98 [2]	8	13

Notes:

1) *Curve numbers for the East Pond and West Pond catchments are composite curve numbers for the entire catchment considering both hardstand and greenfield areas. Composite curve numbers were selected based on NRCS (1986) values for industrial areas in hydrologic soil group 'C', with average percent impervious area of 72%.*

2) *Curve numbers for the North Pond, PBI East, and PBI West catchments correspond with impervious area land use, for all hydrologic soil groups (NRCS, 1986).*

3) *East and West Catchment areas reported exclude PBI East and PBI West sub-catchments.*

5.4 Perimeter Interceptor Channel and Culvert Data

A review of available information (including historical drawings) relating to the Phase 1 and 2 BRDA PICs has been undertaken, and summary data are presented in Table 5 along with the proposed design parameters for the future PIC-M and proposed sub-division of PIC-L.

Typical cross section drawings for the PICs are provided in Appendix A and estimated elevation-volume-area relationships for each PIC segment are provided in Appendix B.

Table 5: PIC Summary Data

PIC Segment	Base Width (m)	Downstream Invert Level (mOD)	Downstream Crest Level (mOD)	Upstream Invert Level (mOD)	Upstream Crest Level (mOD)	Length (m)
PIC-A	Varies	10.5	12.0	11.5	12.0	564
PIC-B	7.0	1.5	5.0 [1]	5.0	8.5	441
PIC-C	8.0	1.5	5.0 [1]	1.5	5.0 [1]	688
PIC-D	8.1	1.0	5.0 [1]	1.0	5.0 [1]	440
PIC-E	6.0	1.8	4.7 [1]	1.8	4.7 [1]	700
PIC-F	6.0	1.6	4.7 [1]	1.8	4.7 [1]	587
PIC-G	7.0	0.9	4.7 [1]	1.6	4.7 [1]	373 [3]
PIC-J	8.56	1.0	6.2	1.75	6.2	287
PIC-K	Varies	1.95	6.0	3.5	6.0	150
PIC-L (North)	3.75	9.88	11.5 [2]	13.5	16.0	155
PIC-L (South)	15.74	13.5	16.0	15.0	19.0	371
PIC-M	5.5	13.5	19.0	14.0	16.0	210

Notes:

1) *Existing crest elevations shown. Increase to crest elevation at 5.3 mOD proposed as a PIC water management system improvement measure (See Section 3.1.3)*

2) *Existing crest elevation shown. Increase to crest elevation at 12.5 mOD proposed as a PIC water management system improvement measure (See Section 3.1.3)*

3) *Reported length excludes PIC-G sump area.*

In addition to the summary data in Table 5, Golder has assumed that:

- The existing western and northern Phase 1 PICs (PIC-E to PIC-G) have side slopes constructed at 1(V):4(H) on the downstream bank and 1(V):1.5(H) on the upstream bank;
- The existing Phase 2 PICs (PIC-B to PIC-D) have side slopes constructed at 1(V):3(H) on the downstream bank and 1(V):2(H) on the upstream bank;

Assumptions have been made for the PIC culverts for the purpose of the analyses. A summary of the assumed information for existing culverts that are intended to be retained are presented in Table 6. All culverts are taken to be circular HDPE pipes with inlets projecting from the embankment. Existing culverts from PIC-D to PIC-E and PIC-L to PIC-K are not included as these are intended to be upgraded (See Table 7).

Table 6: Existing PIC Culvert Summary Data (Assumed Data)

Culvert	Diameter (m) [1]	Number of Barrels [4]	Inlet Invert Level (mOD) [2]	Outlet Invert Level (mOD) [2]	Length (m) [3]	Culvert Arrangement [4]
PIC-A to PIC-B	0.45	3	10.5	6.4	172	Two lower-level barrels side-by-side and one upper-level barrel directly on top of the lower-level barrels
PIC-B to PIC-C	0.60	3	2.6	2.575	25	
PIC-C to PIC-D [6]	0.60	3	2.0	1.970	30	
PIC-E to PIC-F [6]	0.60	3	2.0	1.985	15	
PIC-F to PIC-G [6]	0.60	3	2.0	1.975	25	

Notes:

1) *Culvert diameters estimated from site photos, where available.*

2) *Inlet or outlet invert level estimated from site photos and topography data and an assumed culvert gradient of 0.001 m/m, with the exception of culvert A which was assumed to have a gradient 0.024 m/m with an upstream invert level at the invert level of PIC A.*

3) *Culvert lengths estimated from topography data.*

4) *Culvert arrangement observed from site photos where visible, however some culverts were covered by water and so the culvert arrangement has been assumed.*

5) *Culverts are covered by water in site photos (and during the site visit) and therefore all culvert parameters have been assumed.*

Additional culverts proposed to be installed as proposed improvements to the BRDA water management system are presented in Table 7 below.

It should be noted that the three (3 no.) PIC-A to PIC-B culverts presented in Table 6 were previously installed but not connected to PIC-A, pending construction of PIC-A. It is now proposed that one (1 no.) culvert is connected to drain PIC-A as the reduced conveyance capacity achieved utilising one (1 no.) pipe provides additional flood attenuation during the IDF (compared to an additional number of pipes), while maintaining sufficient conveyance capacity to accommodate the IDF without overtopping. Given the long length of this culvert section, two (2 no.) culverts would be installed (side by side) with one culvert operational to drain PIC-A and the other culvert closed at its inlet with a removeable cap which would act as a backup culvert in the case of a blockage in the operational culvert (i.e., inlet cap removed). However, for the purpose of these analysis the single operational culvert has been considered.

Table 7: Proposed Additional or Replacement PIC Culvert Summary Data (Assumed Data)

Culvert	Dimension (m)	No. Barrels	Inlet Invert Level (mOD)	Outlet Invert Level (mOD)	Length (m)	Culvert Type and Arrangement
PIC-A to PIC-B	0.45 m diameter	1	10.5	6.4	172	HDPE pipe culvert.
PIC-D to PIC-E	1.1 x 0.55 m [Span x Rise]	2	4.75	4.725	25	Concrete box culverts. Laid side-by-side.
PIC-K to SWP	1.1 x 0.55 m [Span x Rise	2	5.45	5.29	24	Concrete box culverts. Laid side-by-side.
PIC-L (North) to PIC-K	0.45 m diameter	1	9.88	5.15	60	HDPE pipe culvert.
PIC-L (South) to PIC-L (North)	0.45 m diameter	1	13.5	13.475	25	HDPE pipe culvert.
PIC-M to PIC-L (South)	0.6 x 0.4 m [Span x Rise]	1	15.03	15.00	30	Concrete box culvert.

5.5 SWP, LWP and ECS Data

The elevation-volume-area relationships for the existing SWP and LWP that have been incorporated in the hydrological analysis are presented in Table 8 and Table 9, respectively.

Table 8: SWP Elevation-Volume-Area Relationship

Elevation (mOD)	Volume (m^3)	Water Surface Area (m^2)
0.8	0	0
1.0	28	158
2.0	23,861	44,239
3.0	73,393	51,725
4.0	126,431	54,359
5.0	182,225	57,217
6.0	240,271	58,902

Table 9: LWP Elevation-Volume-Area Relationship

Elevation (mOD)	Volume (m^3)	Water Surface Area (m^2)
1.1	0	0
2.0	3,410	6,386
3.0	11,903	10,267
4.0	23,829	13,655
5.0	39,068	16,338
6.0	56,103	17,765

For the purposes of this study a constant LWP operating level / water storage level (for water inventory requirements) of 5.5 mOD has been assumed.

Details of the ECS (e.g., storage volumes, evaporative losses) have not been modelled in this study, however, the following key flow rate characteristics have been incorporated in the analysis:

- ECS treatment capacity / LWP discharge capacity = 1,250 m^3/hr;
- Process condensate discharged rate to the ECS = 200 m^3/hr; and
- Maximum discharge capacity from the SWP to the ECS = 1,050 m^3/hr.

A simulated stage-storage curve has been applied for the ECS within the modelling to allow the models to function. This is not reported herein as it is considered irrelevant for the purposes of modelling given inflows equal outflows and therefore modelled storage within the ECS remains static.

5.6 BRDA Water Inventory Targets

Two BRDA water inventory target Scenarios have been assessed in this study to allow comparison of the BRDA water management system performance and PIC pump upgrade requirements. The Scenarios assessed are as follows:

- **Scenario 1:** existing BRDA water inventory targets (as described in Section 3.1.1); and
- **Scenario 2:** water inventory targets reduced by 30,000 m^3 from existing targets (as requested by AAL).

The monthly BRDA water inventory target values applied for each Scenario in the water balance modelling assessment (Section 7.0) are presented in Table 10 below.

Table 10: BRDA Modelled Water Inventory Targets

Month	Scenario 1: Existing BRDA Water Inventory Targets (m^3)	Scenario 2: Reduced BRDA Water Inventory Targets (m^3)
January	110,000	80,000
February	110,000	80,000
March	110,000	80,000
April	150,000	120,000
May	180,000	150,000
June	180,000	150,000
July	180,000	150,000
August	180,000	150,000
September	150,000	120,000
October	110,000	80,000
November	110,000	80,000
December	110,000	80,000

Notes:

1) *The definition of water inventory applied in the water balance modelling is water stored in the PIC system, SWP and LWP; however, the existing AAL definition of the water inventory target does not include LWP water.*

Golder considers the modelled definition of water inventory to be reasonable for the purposes of water balance modelling given:

- Water from the LWP (only) is used in the sprinkler system; and
- Golder considers the LWP to be a prudent store to maintain a significant inventory of water within the BRDA water management system, i.e., to maintain water inventory targets; the risk of the LWP overtopping is less than that for the SWP since the permitted discharge rate equals the maximum inflow rate from the ECS (1,250 m^3/hr). Additionally, the storage of a volume of water in the LWP allows residence time for water to cool and settle prior to discharge to the environment.

6.0 PIC CULVERT CAPACITY ASSESSMENT

The estimated maximum conveyance capacity of each PIC culvert was assessed using the US Department of Transportation Federal Highway Administration's HY-8 Culvert Hydraulic Analysis Program. The maximum discharges before overtopping were determined based on an assumed tailwater elevation. The tailwater assumptions and results are presented in Table 11 below.

Table 11: PIC Culvert Maximum Capacity Assessment (normal meteorological conditions)

Culvert	Maximum Discharge Before Overtopping (m^3/s)	Notes
PIC-A to PIC-B	0.41	Tailwater calculated in HY-8 assuming trapezoidal downstream channel (PIC-B).
PIC-B to PIC-C	2.68	Assumed tailwater elevation of 2.3 mOD (PIC-C lower-level culvert pipes centreline elevation).
PIC-C to PIC-D	2.98	Assumed tailwater elevation of 1.35 mOD (turn off level of PIC-D to PIC-E pump).
PIC-D to PIC-E	1.18	Assumed tailwater elevation of 2.3 mOD (PIC-E lower-level culvert pipes centreline elevation).
PIC-E to PIC-F	2.89	Assumed tailwater elevation of 2.3 mOD (PIC-F lower-level culvert pipes centreline elevation).
PIC-F to PIC-G	2.87	Assumed tailwater elevation of 2.0 mOD (proposed turn off level of PIC-G to SWP pump).
PIC-M to PIC-L (South)	0.43	Tailwater calculated in HY-8 assuming trapezoidal downstream channel (PIC-L (South)).
PIC-L (South) to PIC-L (North)	0.45	Tailwater calculated in HY-8 assuming trapezoidal downstream channel (PIC-L (North)).
PIC-L (North) to PIC-K	0.57	Assumed tailwater elevation of 5.45 mOD (PIC-K overflow culvert inlet invert elevation).
PIC-K to SWP	1.28	Assumed no tailwater given water levels in the SWP are likely to be well below the outlet invert level of the culvert for normal meteorological conditions.

The capacities in Table 11 have been used in the water balance modelling (Section 7.0) as an estimate of the maximum permissible flow rate between PIC segments during normal operating conditions.

These capacities have not been used to assess the performance of the PIC system during the IDF. The performance of the PIC system was assessed via hydrologic / flood routing modelling taking variable tailwater conditions into account; this analysis is described is Section 8.0

7.0 WATER BALANCE MODELLING

Golder has developed a water balance model for the proposed BRDA Raise Development and the Plant Site area using GoldSim Monte Carlo simulation software.

Water balance modelling was used to evaluate the water volumes in the BRDA water management system under normal operating and meteorological conditions. The results of this modelling were used as initial conditions (PIC, SWP, and LWP water levels) in hydrologic / flood routing modelling under IDF conditions (Section 8.0).

7.1 Water Balance Model Structure and Input Data

The water balance model has been constructed at a daily time step, with daily rainfall and evaporation data utilised to estimate daily runoff volumes from the BRDA over a 27,394-day (75 years) duration. Runoff reports to the PICs and is conveyed through the PIC system before being pumped to the SWP or ECS; for modelling purposes it has been assumed that all water pumped from the PIC system is pumped to the SWP. From the SWP, water is pumped to the ECS for treatment before being discharged to the environment via the LWP.

A block flow diagram showing the conceptual water balance model is presented in Figure 3 below which is a simplified version of Figure 2 (Section 3.1.3) for the purposes of modelling. Within the water balance model PIC segments E, F, G and J are modelled as one reservoir due to their consistent crest level and assumptions made regarding the culverts between these PIC segments.

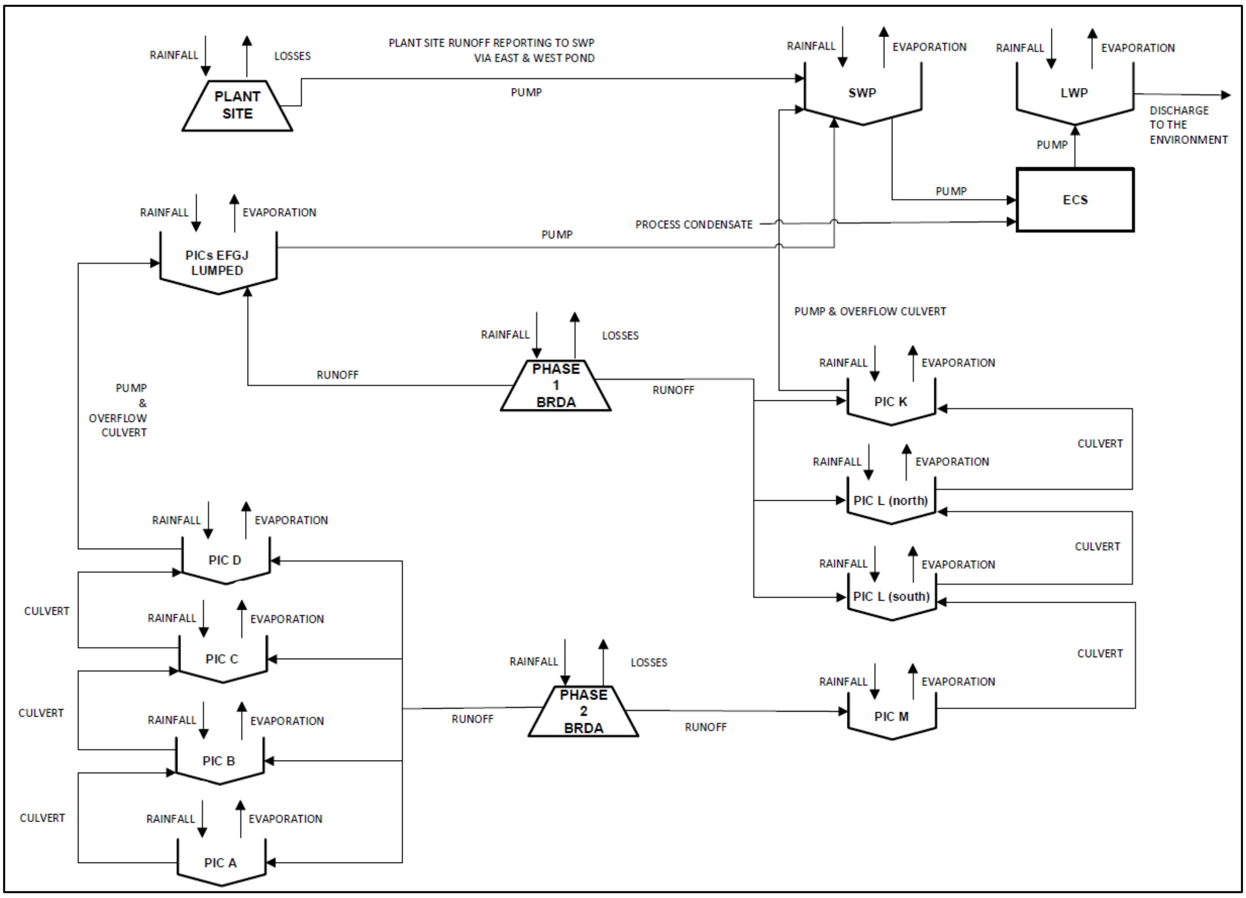

Figure 3: Modelled BRDA Raise Development Water Management System

Notes:

1) *Catchment "Losses" presented in the flow diagram represent all hydrological losses from rainfall including evaporation, transpiration, infiltration, and losses due to surface depressions and ponding.*

Figure 1 (Section 3.1.2) presents the conceptual block flow diagram for the Plant Site water management system. For the purposes of the water balance modelling exercise, due to the small catchment area draining to the North Pond, this catchment area has been included in the West Pond catchment area. This is considered appropriate as during normal operating and meteorological conditions, low water levels are maintained in the North Pond with pumping from the North Pond undertaken when necessary. The North Pond and its catchment have, however, been analysed separately within the Plant Site flood routing modelling to assess storage capacity requirements (Section 9.0).

Daily runoff volumes have been estimated using the SCS Curve Number Method (USDA, 1986) for the catchment areas and Curve Numbers presented in Section 5.3. Discharge through PIC pumps as well as the East Pond and West Pond pumps has been set to occur when water levels are above the pump off levels. However, pumping from the SWP to the ECS only occurs when water inventory targets (Section 7.2) are exceeded. The pump details assumed for the water balance analysis are provided in Section 7.1.2.

Water usage for dusting prevention has not been incorporated in the water balance modelling. Additionally, water usage from the SWP for dilution within the Plant Site has not been assessed. This allows for a worst-case estimate of water volumes in the SWP under normal operating and meteorological conditions.

7.1.1 Meteorological Data

A 75-year time series (01 September 1945 to 31 August 2020) of observed daily rainfall and evaporation data for Shannon Airport has been used as input meteorological data to the water balance model (Met Éireann, 2021). The model runs up to 75 different simulations using ‘time-shifting’ of these data series (shifting by one year for each simulation) to allow statistics to be generated and extracted from the model results.

Snowpack and snowmelt have not been incorporated in the water balance modelling as Golder considers snow accumulation at the site to be negligible. Long term weather averages for Shannon Airport (1981 – 2010) show that the mean number of days with snow lying at 09:00 UTC is 0.9 days per year (Met Éireann, 2019).

7.1.2 Pump and Overflow / Culvert Data

The BRDA and Plant Site pump systems within the water balance have been modelled at a daily time step assuming pumping occurs when water levels exceed the relevant pump off level. Details of these modelled pump data are provided in Table 12.

Table 12: Water Balance Model Pump Data

Pump	Pump Off Level (mOD)	Maximum Discharge Rate (m^3/s)	Notes
PIC-D to PIC-E	1.35	0.238	Off level and maximum discharge rate correspond to the current off level and maximum discharge rate for the existing ‘Pump 24’ located within PIC-D which currently pumps from the PIC-D to PIC-E.
PIC-G sump to SWP	2.0	0.238	Off level set below the existing ‘Pump 33’ off level at 2.0 mOD to increase the storage capacity within PIC G to attenuate flood events. Maximum discharge rate corresponds with that of ‘Pump 33’; this rate is significantly lower than the maximum rate required in PIC-G to manage the IDF (Section 8.0) but is sufficient to manage the volumes in PIC-G under normal operating and meteorological conditions, at the modelled daily time-step.

Pump	Pump Off Level (mOD)	Maximum Discharge Rate (m^3/s)	Notes
PIC-K to SWP	3.0	0.041	Proposed new pump at PIC-K, with assumed maximum discharge rate capable of accommodating flows during regular meteorological conditions. Off level set at 0.5 m above main channel invert level.
SWP to ECS	n/a	0.291	No off-level set; pumping only occurs if water inventory targets are exceeded. Maximum pumping rate assumed to be 0.291 m^3/s which is equal to the maximum possible discharge rate from the SWP to the ECS of 1,050 m^3/hr.
East Pond to SWP	3.9	0.036	Off level corresponds to the estimated existing system off level. Maximum discharge rate equal to the estimated maximum discharge rate achievable for the existing East Pond pump.
West Pond to SWP	6.5	0.055	Off level corresponds to the estimated existing system off level. Maximum discharge rate equal to the estimated maximum discharge rate achievable for the existing West Pond pump.

Flows to and from the ECS and LWP have not been modelled using pump off rules or maximum pump rates. These processes have been simplified in the model with the pumped discharge from the SWP reporting to the LWP via the ECS, followed by discharge to the environment at the current permitted rate.

Overflow culverts within the PIC system (i.e., PIC-D and PIC-K overflow culverts) have not been considered in the water balance model as no discharge is expected to occur through these features under normal operating and meteorological conditions; i.e., water levels and inter-PIC flows are managed by the pumps and PIC culverts only. However, these features have been included in the hydrologic / flood routing modelling (Section 8.0).

7.2 Water Balance Model Results

The primary objective of water balance modelling was to establish appropriate initial conditions i.e., storage volumes / water levels, in the PICs, SWP and LWP to be used for hydrologic / flood routing analysis. For both water inventory Scenarios, a total of 75 simulations of the water balance model were run to allow statistical analysis of the modelling results. These statistics have been used to select appropriate initial volumes / water levels to be used in the hydrologic / flood routing analysis. The maximum modelled water inventory deficit (target minus achieved) throughout the 75-year time series is 77,000 m^3 and 75,000 m^3 for Scenarios 1 and 2, respectively (see Table 10). Peak water inventory deficits are typically observed in the early summer months (May or June) with reducing inventory deficits in the following months. This is attributable to the large increase in water inventory requirements from March (110,000 m^3) to May (180,000 m^3) and the time required accumulate the required additional storage volumes within the system.

The 50th percentile and 75th percentile volumes / water levels from the water balance modelling are presented in Table 13 and Table 14 for water inventory Scenario 1 and 2, respectively. Golder has selected the 75th percentile values as initial conditions for the hydrologic / flood routing analysis as these values represent wetter conditions compared to the 50th percentile values and may be considered maximum normal operating values.

Table 13: Water Inventory Scenario 1 - Volume / Water Level Result Statistics (Normal Operating and Meteorological Conditions)

Facility	75th Percentile Volume (m^3)	75th Percentile Level (mOD)	50th Percentile Volume (m^3)	50th Percentile Level (mOD)
SWP	118,175	3.85	116,195	3.81
LWP	47,414	5.50	47,414	5.50
PIC-A	1,657	10.48	1,523	10.44
PIC-B	605	2.50	585	2.49
PIC-C	2,729	1.93	2,468	1.89
PIC-D	3,639	1.79	2,450	1.57
PIC-E	896	2.00	896	2.00
PIC-F	1,216	2.00	1,216	2.00
PIC-G/J	5,620	1.93	4,645	1.82
PIC-K	28	3.00	28	3.00
PIC-L (North)	0	9.88	0	9.88
PIC-L (South)	99	13.57	0	13.50
PIC-M	2,016	15.00	1,881	14.93
East Pond [1]	2,653	3.90	2,653	3.90
West Pond [1]	421	6.50	421	6.50
Total BRDA Volumes (Excluding East and West Ponds) [1]	184,094	N/A	179,301	N/A

Notes:

1) *Volumes in the East and West Ponds (Plant Site ponds) are not included in the BRDA water inventory calculation. Therefore, these volumes have been excluded from the total BRDA volumes reported in the above table.*

Table 14: Water Inventory Scenario 2 - Volume / Water Level Result Statistics (Normal Operating and Meteorological Conditions)

Facility	75th Percentile Volume (m^3)	75th Percentile Level (mOD)	50th Percentile Volume (m^3)	50th Percentile Level (mOD)
SWP	88,169	3.28	86,063	3.24
LWP	47,414	5.50	4,7414	5.50
PIC-A	1,645	10.48	1,523	10.44
PIC-B	604	2.51	585	2.49
PIC-C	2,738	1.94	2,472	1.89
PIC-D	3,799	1.83	2,524	1.58
PIC-E	896	2.00	896	2.00
PIC-F	1,216	2.00	1,216	2.00
PIC-G/J	5,603	1.93	4,559	1.82
PIC-K	28	3.00	28	3.00
PIC-L (North)	0	9.88	0	9.88
PIC-L (South)	94	13.57	0	13.50
PIC-M	2,025	15.00	1,893	14.94
East Pond[1]	2,653	3.90	2,653	3.90
West Pond[1]	421	6.50	421	6.50
Total BRDA Volumes (Excluding East and West Ponds)[1]	154,231	N/A	149,173	N/A

Notes:

1) *Volumes in the East and West Ponds (Plant Site ponds) are not included in the BRDA water inventory calculation. Therefore, these volumes have been excluded from the total BRDA volumes reported in the above table.*

8.0 BRDA DESIGN FLOOD ROUTING AND STORAGE CAPACITY ASSESSMENT

Golder has undertaken a flood routing and storage capacity analysis for the proposed BRDA Raise Development using the US Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modelling System (HEC-HMS). The objectives of this analysis are:

- To assess the performance of the PICs for the Inflow Design Flood (IDF) and provide recommendations for improvement of the PIC system, including options to increase PIC storage capacity; and
- To assess the storage capacity of the existing SWP and LWP, and the ability of the system to accommodate the IDF with zero discharge to the environment (other than permitted existing treated discharge via the ECS and LWP).

The conceptual hydrologic model is shown in Figure 4 below. There are 14 reservoir elements in the model representing the 11 PIC segments, the SWP, the ECS and the LWP. Given the design of the PICs (i.e., blind channels with culverted, pumped and overflow outflows), these facilities have been represented as storage facilities rather than channels within the model. In reality there will be a pump discharging from PIC-G to the ECS; however due to limitations of the modelling software the analysis has been undertaken assuming all pumping from PIC-G discharges to the SWP (and ultimately to the ECS via the SWP).

An inflow hydrograph to the SWP from the Plant Site has also been incorporated, which corresponds to the 24-hour 100-year plus climate change design rainfall event. The Plant Site flood routing analysis is described in Section 9.0.

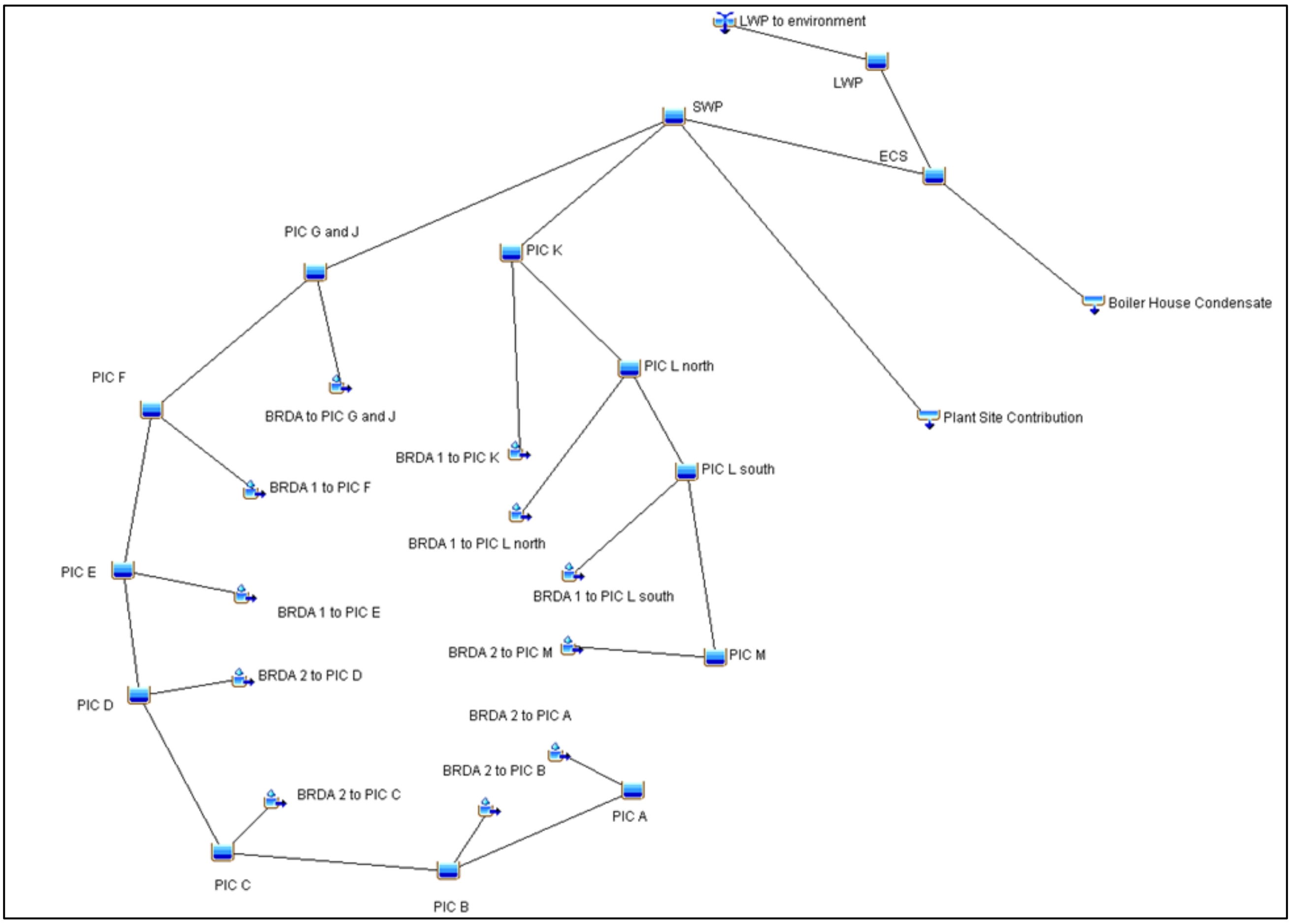

Figure 4: BRDA Raise Development - Conceptual HEC-HMS Flood Routing Model

The peak flow rates and runoff volumes from the BRDA sub-catchments for the IDF are presented in Table 15.

Table 15: BRDA Peak Runoff Rates and Runoff Volumes for the IDF

BRDA Sub-Catchment	Peak Runoff Rate (m^3/s)	Runoff Volume (1,000 m^3)
PIC-A	0.602	17.44
PIC-B	0.515	14.78
PIC-C	0.871	25.45
PIC-D	0.690	19.60
PIC-E	1.115	32.57
PIC-F	0.455	12.80
PIC-G/J	0.870	25.06
PIC-K	0.404	11.10
PIC-L (North)	0.074	2.02
PIC-L (South)	0.679	19.32
PIC-M	0.456	12.70

The PIC pumping arrangement will require upgrading from the existing arrangement (Section 7.1.2) to accommodate the IDF. This includes upgrading of the PIC-G pumping capacity to the SWP, and installation of a new pump which will discharge flows from PIC-K to the SWP.

The pumping system characteristics assumed in the model for the pumps discharging from PIC-D, PIC-G, PIC-K and the SWP are presented in Table 16 for water inventory Scenario 1 (i.e. existing water inventory targets) (Section 5.6). The pump from PIC-D has been modelled applying the pump curve for the existing pump in this location i.e., 'Pump 24'. All other pumps have been modelled assuming a constant pump rate.

The SWP pump has been modelled using a constant pump rate equal to the maximum permissible discharge rate from the SWP (1,050 m^3/hr). For PIC-G the pumping characteristics required to accommodate the IDF (without overtopping of the PIC or overtopping of the SWP through excessive pumping) consider a lower-level pump (for normal operating and meteorological conditions) and an upper-level pump (for flood conditions).

Table 16: Water Inventory Scenario 1 - Pumping System Characteristics for PICs and SWP

Parameter	PIC-D Pump	PIC-G Lower Level Pump (Normal Conditions)	PIC-G Upper-Level Pump (Flood Conditions)	PIC-K Pump	SWP Pump
Pump intake elevation (m)	0.50	1.4	1.4	2.5	1.30
Pump line elevation (m)	5.50	6.0	6.0	6.0	17.00
Pump switch ON level	2.75	2.5	3.6	3.5	3.5
Pump switch OFF level	1.35	2.0	3.5	3.0	1.9
Maximum modelled pump rates (m^3/s)	0.222	0.194	0.500	0.041	0.291
		0.694 (combined)			
Maximum modelled pump rates (m^3/hr)	800	700	1,800	150	1,050
		2,500 (combined)			

For water inventory Scenario 2, i.e. existing water inventory targets reduced by 30,000 m^3, (see Section 5.6), a small decrease to the PIC-G pumping rate requirement is achievable while maintaining sufficient capacity for the IDF; this is due to the increased capacity available in the SWP allowing the PIC-G upper-level (flood) pump to discharge over a longer period of time (due to its larger operational range) at a slightly reduced (100 m^3/hr) pump rate. The PIC-G pump characteristics modelled for water inventory Scenario 2 are presented in Table 17; the remaining pumps remain unchanged from those presented in Table 16.

Table 17: Water Inventory Scenario 2 - Pumping System Characteristics for PIC-G

Parameter	PIC-G Lower-Level Pump (Normal Conditions)	PIC-G Upper-Level Pump (Flood Conditions)
Pump intake elevation (m)	1.4	1.4
Pump line elevation (m)	6.0	6.0
Pump switch ON level	2.5	3.25
Pump switch OFF level	2.0	3.0
Maximum modelled pump rates (m^3/s)	0.194	0.472
	0.666 (combined)	
Maximum modelled pump rates (m^3/hr)	700	1,700
	2,400 (combined)	

The pump curve that has been applied to PIC-D within the models is presented in Table 18, which is the pump curve for the existing pump located at PIC-D ("Pump 24") provided by AAL. As described previously, the remaining pumps have been analysed assuming a constant pump rate and therefore no specific pump curve has been applied for those pumps.

Table 18: PIC-D Pump Curve Data ("Pump 24")

Head (m)	Flow Rate (m^3/hr)
6.0	855
8.0	761
12.0	576
16.0	427
20.0	278
24.0	124
26.0	0

As referred to in Section 6.0, variable tailwater conditions have been taken into account in the hydrologic / flood routing modelling for the analysis of the PIC system performance. To consider these tailwater conditions, the models were run iteratively, with the modelled water level time series within each PIC segment used as the tailwater elevation time series for the culvert immediately upstream in the subsequent model iteration. A number of iterations were analysed to allow convergence of the estimated tailwater time series. The worst-case performance results for each PIC segment i.e., maximum water level, have been reported following comparison of modelling results for each tailwater iteration.

Similarly, the assessment of the PIC-G pump requirements and the SWP capacity assessment have considered the worst-case tailwater iteration Scenario.

For the first tailwater iteration, a constant tailwater elevation was considered as set out in Table 19 below.

Table 19: First Iteration Tailwater Elevation Selection

PIC	Tailwater Elevation (mOD)	Basis of Elevation Selection
PIC-B	3.175	Elevation at the top of the PIC-B culvert lower-level pipe outlet.
PIC-C	2.570	Elevation at the top of the PIC-C culvert lower-level pipe outlet.
PIC-D	N/A	No tailwater elevation set for first tailwater iteration given the PIC-D overflow culverts have an invert level substantially higher than the inter-PIC culverts from PIC-B to PIC-G. Following the first iteration, variable tailwater time series have been applied for subsequent iterations.
PIC-E	2.585	Elevation at the top of the PIC-E culvert lower-level pipe outlet.
PIC-F	3.60 [1] / 3.25 [2]	Elevation equal to the "On" elevation of the upper-level (flood conditions) pump in PIC G

Notes:

1) *Tailwater elevation of 3.60 mOD applicable to water inventory Scenario 1.*
2) *Tailwater elevation of 3.25 mOD applicable to water inventory Scenario 2.*

These tailwater elevations are higher than those presented previously in Table 11 (Section 6.0) for the assessment of maximum culvert capacities during normal operating and meteorological conditions. In this instance, tailwater conditions during the IDF (flood conditions) are being considered and therefore higher tailwater elevations as set out in Table 19 are considered appropriate for tailwater estimation.

Notes:

1) *PIC-A, PIC-K, PIC-L (North and South) and PIC-M were found to not be significantly affected by tailwater conditions during the IDF and therefore are not included in the table below; this is due to their respective geometries, invert gradients and elevation differences.*

8.1 PIC Performance and Improvement Recommendations

A number of improvements to the existing PICs are required to prevent overtopping during the IDF; these improvements were outlined previously in Sections 3.1.3, 5.4, and 8.0, and these anticipated upgrades have been incorporated in the analysis performed herein.

The modelled results of the PIC system performance, i.e., peak water levels and freeboard, during the IDF for water inventory Scenarios 1 and 2 are presented in Table 20 and Table 21, respectively.

Table 20: PIC Peak Water Levels and Freeboard (Scenario 1)

PIC Segment	Peak Water Level (mOD)	Freeboard (m)
PIC-A	11.278	0.722
PIC-B	5.127	0.173
PIC-C	5.111	0.189
PIC-D	5.145	0.155
PIC-E	5.159	0.141
PIC-F	5.140	0.160
PIC-G/J	5.121	0.179
PIC-K	5.812	0.188
PIC-L (North)	11.850	0.650
PIC-L (South)	15.455	0.545
PIC-M	15.660	0.340

Table 21: PIC Peak Water Levels and Freeboard (Scenario 2)

PIC Segment	Peak Water Level (mOD)	Freeboard (m)
PIC-A	11.278	0.722
PIC-B	5.128	0.172
PIC-C	5.112	0.188
PIC-D	5.147	0.153
PIC-E	5.229	0.071
PIC-F	5.211	0.089
PIC-G/J	5.206	0.094
PIC-K	5.812	0.188
PIC-L (North)	11.850	0.650
PIC-L (South)	15.455	0.545
PIC-M	15.660	0.340

Conclusions from the hydrologic / flood routing modelling of the PIC system are:

- The hydrologic / flood routing modelling results have demonstrated that it is feasible for the IDF to be accommodated within the PIC system without overtopping, provided the PIC and pumping capacity improvements described in this report are implemented and pre-flood water levels are maintained at or below the levels applied in the flood routing analysis.
- The assumed initial water volumes and levels in the water management system have a significant impact on the system performance during the IDF. These are influenced by both the BRDA water inventory targets and the On/Off levels of the PIC pumps. Although the results presented herein demonstrate only a small improvement in PIC performance (i.e., small reduction in PIC maximum pump rate requirement) through a reduction in water inventory; further improvement may be achieved if it is possible to lower the On/ Off levels of the PIC pumps, primarily in PIC-G. A reduction in water inventory alone, without a reduction in PIC pump operating levels, limits any benefit to the PICs and instead the benefit is realised within the SWP.
- The peak water levels reached during the IDF are typically above the target freeboard level of 0.5m below dam crest elevations. There is minimal scope for further attenuation within the PIC system in order to reduce pumping requirements to the SWP or storage within the SWP.

8.2 SWP and LWP Performance

The ability of the SWP and LWP to accommodate the IDF has been analysed using hydrologic / flood routing modelling. As the IDF is routed through the BRDA water management system, the maximum storage requirement within these facilities can be assessed.

The results of the SWP analyses for water inventory Scenarios 1 and 2 (Section 5.6) are presented in Table 22 below.

Table 22: SWP Analysis Results

Water Inventory Scenario	SWP Maximum Storage Requirement (m^3)	Remaining Capacity (m^3)	Peak Water Surface Elevation (mOD)	Freeboard at Peak Water Surface Elevation (m)
Scenario 1	231,739	8,532	5.9 (approx.)	0.1 (approx.)
Scenario 2	210,729	29,542	5.5 (approx.)	0.5 (approx.)

Notes:

1) *Although the water inventory target has been reduced by 30,000 m^3 for Scenario 2, a capacity improvement of approximately 21,000m^3 has been realised in the SWP. The discrepancy is due to the adjustment in PIC-G pumping capacities applied for Scenario 2 (See Table 17).*

Figure 5 below presents a model extract (for Scenario 1) of graphed time series results for the SWP, showing storage volumes / levels within the SWP as well as inflow / outflow rates. The maximum storage requirement presented in Table 22 is slightly lower than that indicated on Figure 5, as spare storage capacity within the Plant Site North Pond (3,671 m^3) will be utilised during the IDF which is not accounted for within the modelling due to limitations of the modelling software (see Section 9.0 for further information).

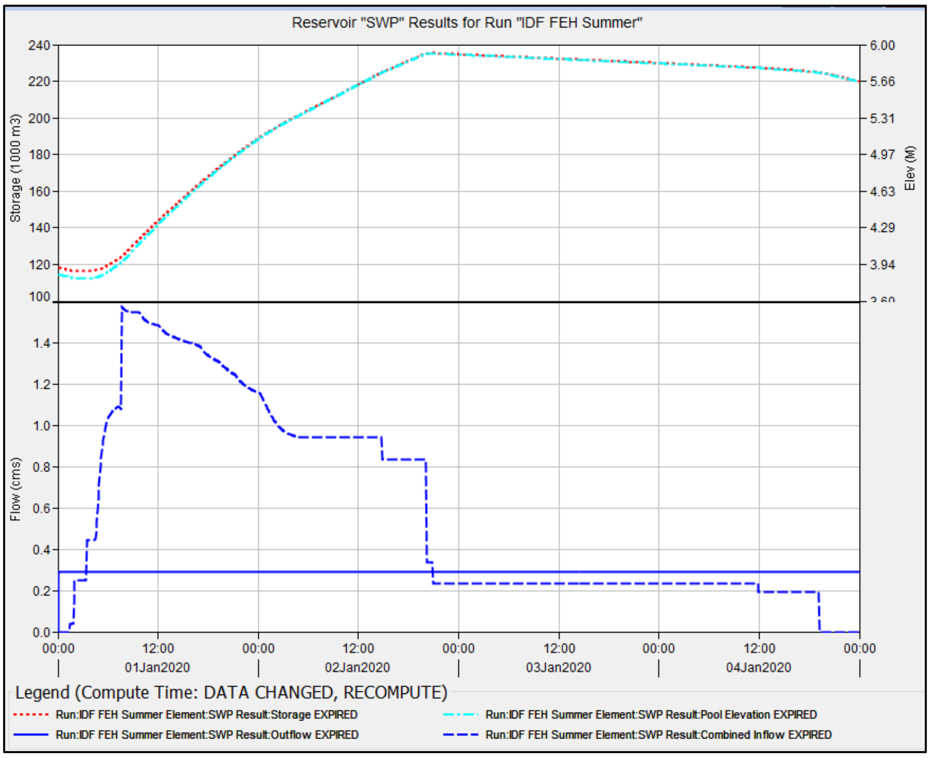

Figure 5: Modelled Storage Volumes / Levels and Inflows / Outflows for the SWP (Scenario 1)

As the LWP inflows (1,250 m^3/hr inflow from the ECS) equal the LWP outflows (1,250 m^3/hr discharge to environment), the storage capacity within the LWP remains unchanged throughout the model duration.

Conclusions from the hydrologic / flood routing modelling of the BRDA water management system relevant to the SWP and LWP are:

- The LWP storage requirement is independent of the BRDA IDF as the inflow rate from the ECS equals the discharge rate from the LWP. Therefore, the LWP can accommodate routing of the IDF through the BRDA water management system.
- The hydrologic / flood routing modelling results have demonstrated that it is feasible for the IDF to be routed through the BRDA water management system, without overtopping of the SWP (or any other part of the system).

9.0 PLANT SITE DESIGN FLOOD ROUTING AND STORAGE CAPACITY ASSESSMENT

Golder has undertaken a flood routing and storage capacity analysis for the Plant Site water management system using HEC-HMS. The objective of this analysis is to estimate a discharge hydrograph for surface water runoff generated on and pumped from the Plant Site to the BRDA, for inclusion in the BRDA IDF routing analysis (Section 8.0). The conceptual hydrologic model is shown in Figure 6 below.

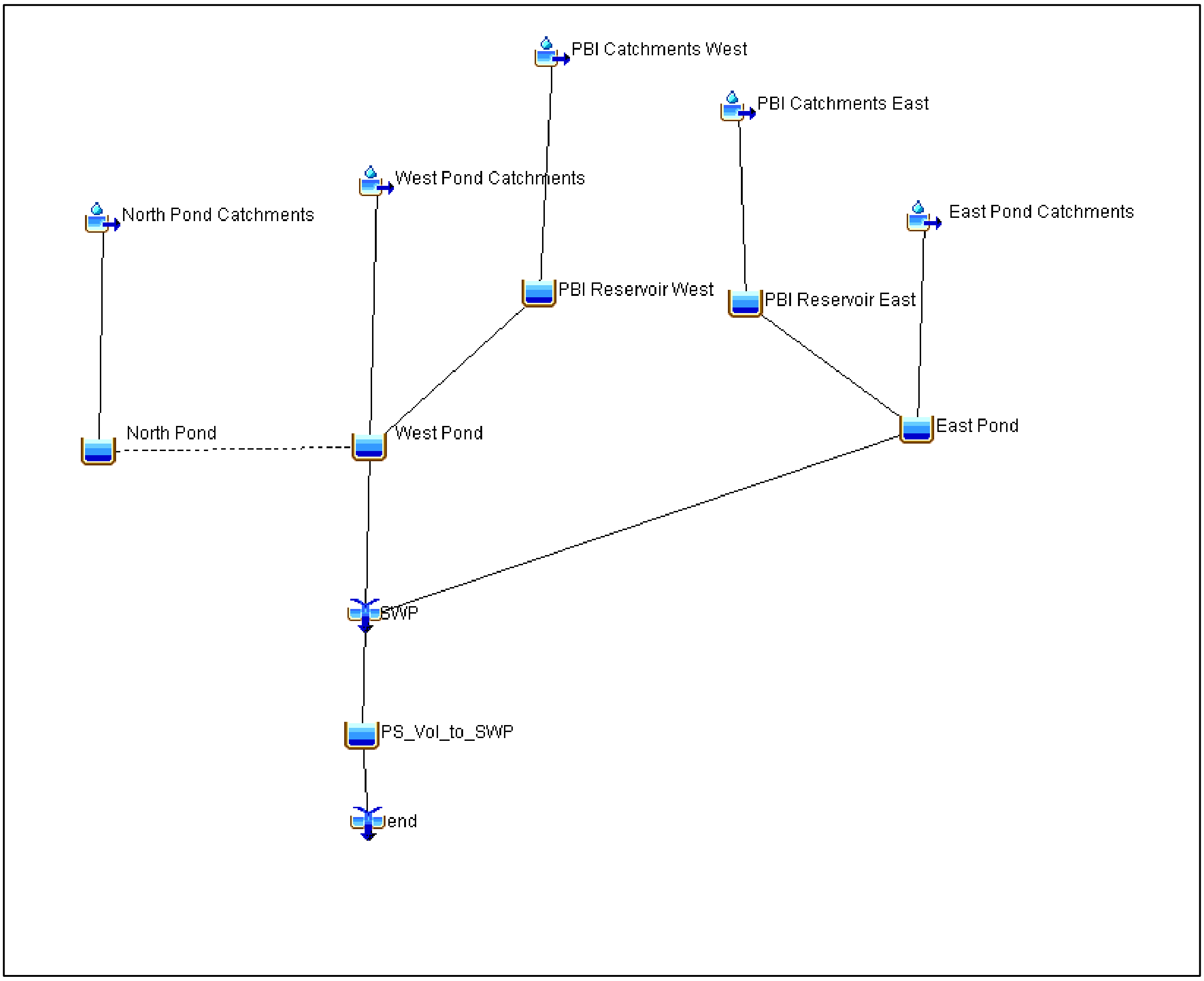

Figure 6: Plant Site - Conceptual HEC - HMS Flood Routing Model

Golder understands that up to 10,000 m^3 of surface water runoff will be pumped into process vessels from the process area catchments (labelled ‘PBI Catchments’ in Figure 6) within the Plant Site during a storm event. This system storage has been modelled using reservoir storage elements for the East and West Pond catchments (totalling 10,000 m^3) on a pro rata basis based on the proportion of process catchment area within the East Pond catchment (64%) and West Pond catchment (36%).

It is intended that during extreme rainfall events, spare storage capacity within the North Pond will be utilised through pumping from the West Pond to the North Pond. Due to limitations of the modelling software, the North Pond is modelled independently to the West Pond with no transfers of flow between the ponds; instead, the available spare capacity within the North Pond at the end of the storm event is assessed and subtracted from the SWP sizing requirements.

The existing maximum capacity of the East, West and North Ponds have been estimated to be 9,407 m^3 and 8,591 m^3, 6,861 m^3, respectively. The spare capacity in the North Pond during the IDF is estimated to be 3,671 m^3.

As outlined in Section 5.2, the 1 in 100-year plus climate change design criterion is considered to be an appropriate standard of protection for the Plant Site water management system. Therefore, for the purposes of this assessment the storm water runoff discharging to the BRDA water management system from the Plant Site has been assessed for the 1 in 100-year plus climate change rainfall event.

Extreme low probability rainfall events in excess of this storm i.e., the BRDA IDF, would result in the generation of additional runoff volumes which AAL propose to temporarily retain and manage within the Plant Site surface water management system i.e., drains, sumps, bunded slabs and kerbed road infrastructure, up to the BRDA IDF rainfall event. The Plant Site hydrological analysis results are summarised in Table 23.

Table 23: Plant Site Hydrological Analysis Results Summary

Parameter	Plant Site 100 Year (+ 20% Climate Change) Rainfall Event	BRDA IDF Rainfall Event
East Catchment Runoff Volume (m^3)	14,590	25,130
East PBI Catchment Runoff Volume (m^3)	5,160	7,990
West Catchment Runoff Volume (m^3)	15,470	26,640
West PBI Catchment Runoff Volume (m^3)	2,870	4,440
North Catchment Runoff Volume (m^3)	1,310	2,070
North Pond Spare Storage Capacity (m^3)	4,421	3,671
Volume absorbed by East PBI (m^3)	5,160	6,340
Volume absorbed by West PBI (m^3)	2,870	3,570
Volume Discharged to the BRDA (m^3)	30,070	30,070 (limited to 1 in 100-year plus CC rainfall event)
Excess Runoff Volume to be Managed on the Plant Site during the BRDA IDF (m^3)	N/A	24,230

The assumed pumping rates to discharge the 1 in 100-year plus climate change surface runoff from the Plant Site ponds to the BRDA (SWP) are as follows:

- East Pond pumping rate: 0.11 m^3/s (396 m^3/hr) over a duration of approximately 37 hours; and
- West Pond pumping rate: 0.10 m^3/s (360 m^3/hr) over a duration of approximately 43 hours.

10.0 CONCLUDING REMARKS AND RECOMMENDATIONS

- Golder has estimated the peak runoff rates from the proposed Phase 1 and 2 to Stage 16 BRDA Raise Development during the IDF to the PIC system; this has included analysis of each PIC segment (divided by culverted 'choke points' in the PIC system) and its sub-catchment. The IDF for the BRDA facility is 1/3 between the 1000-year event and the PMF with an event duration of 24 hours. Peak runoff rates to the PIC segments from the BRDA range from 0.074 m^3/s (PIC-L North) to 1.115 m^3/s (PIC-E) (Table 15).
- The analysis shows that the proposed PIC system can accommodate the IDF (without overtopping) provided that improvements to the PIC system, outlined in Sections 3.1.3, 5.0, and 8.0, are implemented. However, peak water levels exceed the target freeboard level of the PICs (assumed to be 0.5 m below the PIC crest elevation) for all PICs except PIC-A and PIC-L (North and South segments).
- The assumed initial water volumes and levels in the water management system have a significant impact on the system performance during the IDF. These are influenced by both the BRDA water inventory targets and the On/Off levels of the PIC pumps. Although the results presented herein demonstrate only a small improvement in PIC performance, i.e., small reduction in PIC maximum pump rate requirement through a 30,000 m^3 reduction in water inventory, further improvement may be achieved if it is possible to lower the On/Off levels of the PIC pumps, primarily in PIC-G. A reduction in water inventory alone, without a reduction in PIC pump operating levels, limits any benefit to the PICs and instead the benefit is realised within the SWP.
- Golder has assessed the storage capacities for the existing SWP and LWP during the IDF. The results of these assessments are presented in Section 8.2, and demonstrate that the SWP and LWP can accommodate the IDF without overtopping provided the proposed improvements are made to the PICs and pumping systems.
- Golder has undertaken a hydrological assessment for the Plant Site catchments and estimated pumping rates and runoff volumes corresponding to the 1 in 100-year plus climate change rainfall event; runoff volumes up to this magnitude of event will be pumped to the BRDA water management system and therefore have been incorporated in the hydrological assessment of the BRDA water management system. For rainfall events in excess of this event i.e., the BRDA IDF, the surplus runoff volumes are proposed by AAL to be temporarily retained and managed within the Plant Site surface water management system up to the BRDA IDF rainfall event. The hydrological analysis of the Plant Site is described in Section 9.0.
- Golder recommends that all of the proposed BRDA water management improvement measures outlined in Sections 3.1.3, 5.0, and 8.0, are implemented. Upgrades which will be implemented are:
  - Provision of additional culverts for PIC-A, PIC-D, PIC-M, L and PIC-K, including the sub-division of PIC-L into northern and southern segments and the decommissioning of the existing culvert from PIC-K to PIC-J;
  - Increases to PIC crest elevations for PICs B to G and PIC-L (North);
  - Construction of PIC-M;
  - PIC pump arrangement upgrades for PIC-G and PIC-K;
  - Pumped flows from the Plant Site to discharge to the SWP rather than the PIC system. This is intended to reduce the volume of water discharging to the PIC during the IDF and reduce the overall PIC pumping capacity required to accommodate the IDF.
- Golder recommends that during a large magnitude flood event such as the IDF, that the spare storage capacity of the North Pond is utilised via pumping from the West Pond.

11.0 STUDY LIMITATIONS AND ASSUMPTIONS

The analysis undertaken has been based upon a number of limitations and assumptions, as described throughout the report. The assumptions made may significantly influence the results of the analysis and should be verified where possible and the analysis updated if necessary. Key limitations / assumptions include:

- PIC Geometry Data: The PIC geometry utilised in the modelling for storage volumes has been determined from a number of sources including; survey of PIC segments as they've been drained down and cleaned out and review of historical design drawings and site topographical data. Greater certainty may be obtained from survey of the remaining PIC segments when they are being cleaned out.
- PIC Culvert Data: The PIC culvert data has been assumed based on photographs taken during a site visit, historical drawings and site topographic data, in the absence of detailed survey information. For some PIC segments the culverts were covered by water during the site visit and therefore culvert data was based fully on assumption. Improved data in relation to the culvert arrangements will allow greater accuracy in modelling inter-PIC flows and assessment of the capacities of the culverts.
- BRDA Water Management Improvement Measures: The analysis undertaken and results presented are based on the assumption that all of the BRDA water management improvement measures proposed within this report (i.e. Sections 3.1.3, 5.0, and 8.0) ~~are~~ will be implemented.

  The definition of water inventory applied in the water balance modelling is water stored in the PIC system, SWP and LWP; however, the existing AAL definition of the water inventory target does not include LWP water. Golder considers the modelled definition of water inventory to be reasonable for the purposes of water balance modelling as outlined in Section 5.6.
- A constant LWP operating / water storage level of 5.5 mOD has been assumed.
- Water usage for dusting prevention has not been incorporated in the water balance modelling. Additionally, water usage from the SWP for dilution within the Plant Site has not been assessed. This allows for a conservative worst-case estimate of water volumes in the SWP under normal operating and meteorological conditions.
- Due to limitations of the flood routing modelling software, all pumping from PIC-G has been modelled as discharging to the SWP (and ultimately to the ECS via the SWP). In reality there will be a pump discharging from PIC-G directly to the ECS in addition to pumping to the SWP, as per the existing pumping arrangement. Nevertheless, the modelled pumping arrangement could be considered by AAL when implementing proposed PIC pump arrangement improvements.
- The performance of the PICs and SWP during the IDF are sensitive to both the proposed PIC-G pumping rates and the pumping ON / OFF levels, as outlined in Table 16 and Table 17. Divergence from the parameters set out may mean that the PICs or the SWP can no longer accommodate the IDF.
- The ability of the existing Plant Site water management infrastructure to accommodate design rainfall events has not been assessed in this study. Pumping rates required to discharge Plant Site surface water runoff for the 1 in 100-year + 20% climate change event have been incorporated as an inflow hydrograph to the SWP for the purposes of assessing the BRDA water management system performance.

  The estimated volume of excess runoff to be managed by the Plant Site during the IDF is 24,230 m^3 (Table 23). AAL proposes to temporarily retain and manage within the Plant Site surface water management system i.e., drains, sumps, bunded slabs and kerbed road infrastructure, up to the BRDA IDF rainfall event.

12.0 REFERENCES

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Signature Page

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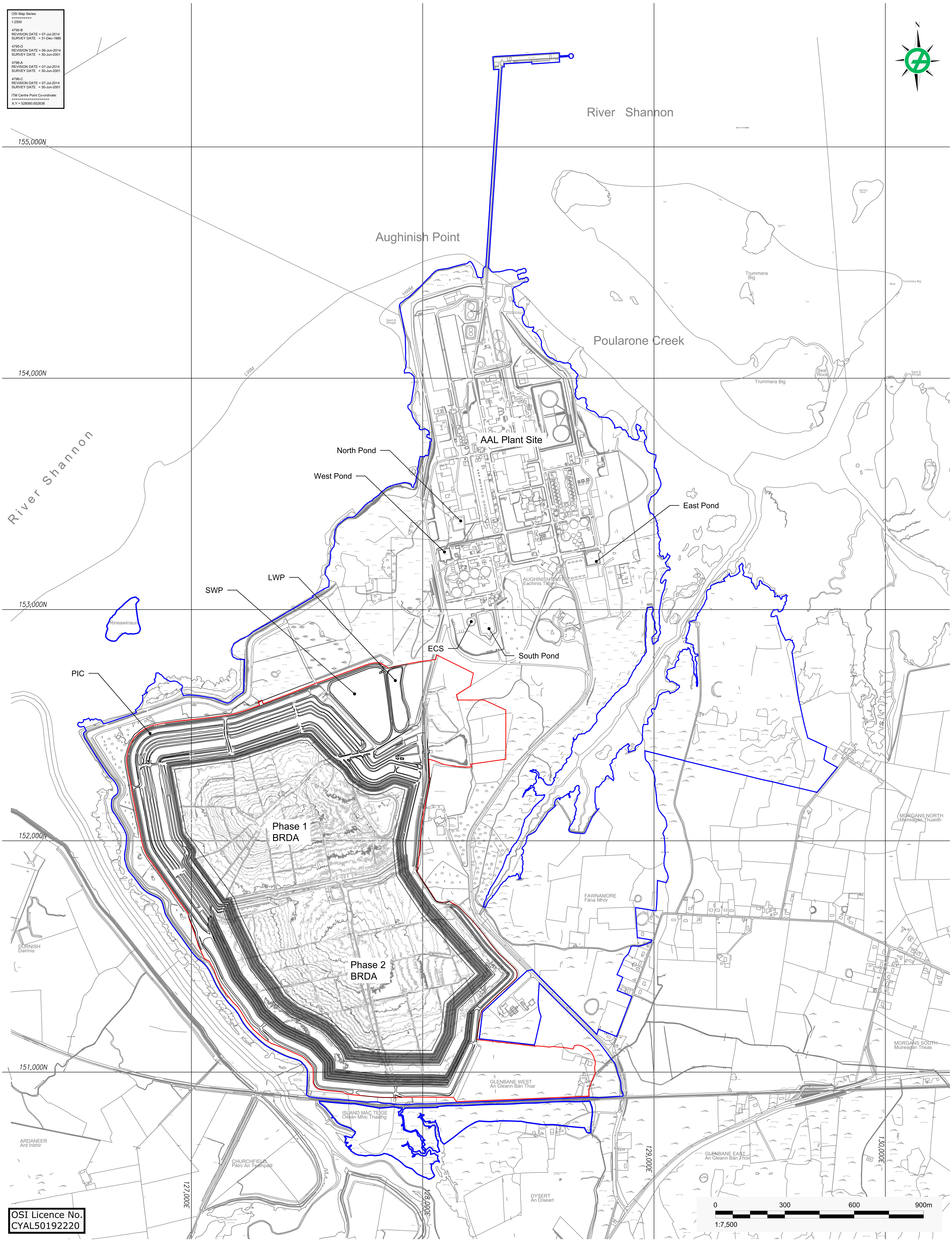
APPENDIX A

Drawings



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AAL OWNERSHIP BOUNDARY

APPLICATION BOUNDARY

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

CLIENT

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CONSULTANT

GOLDER
MEMBER OF WSP

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APPROVED	BK/DH

PROJECT

HYDROLOGICAL ASSESSMENT FOR THE BRDA RAISE DESIGN

TITLE

SITE LOCATION PLAN - Overall

PROJECT No.	DRAWING No.	Rev.	SCALE
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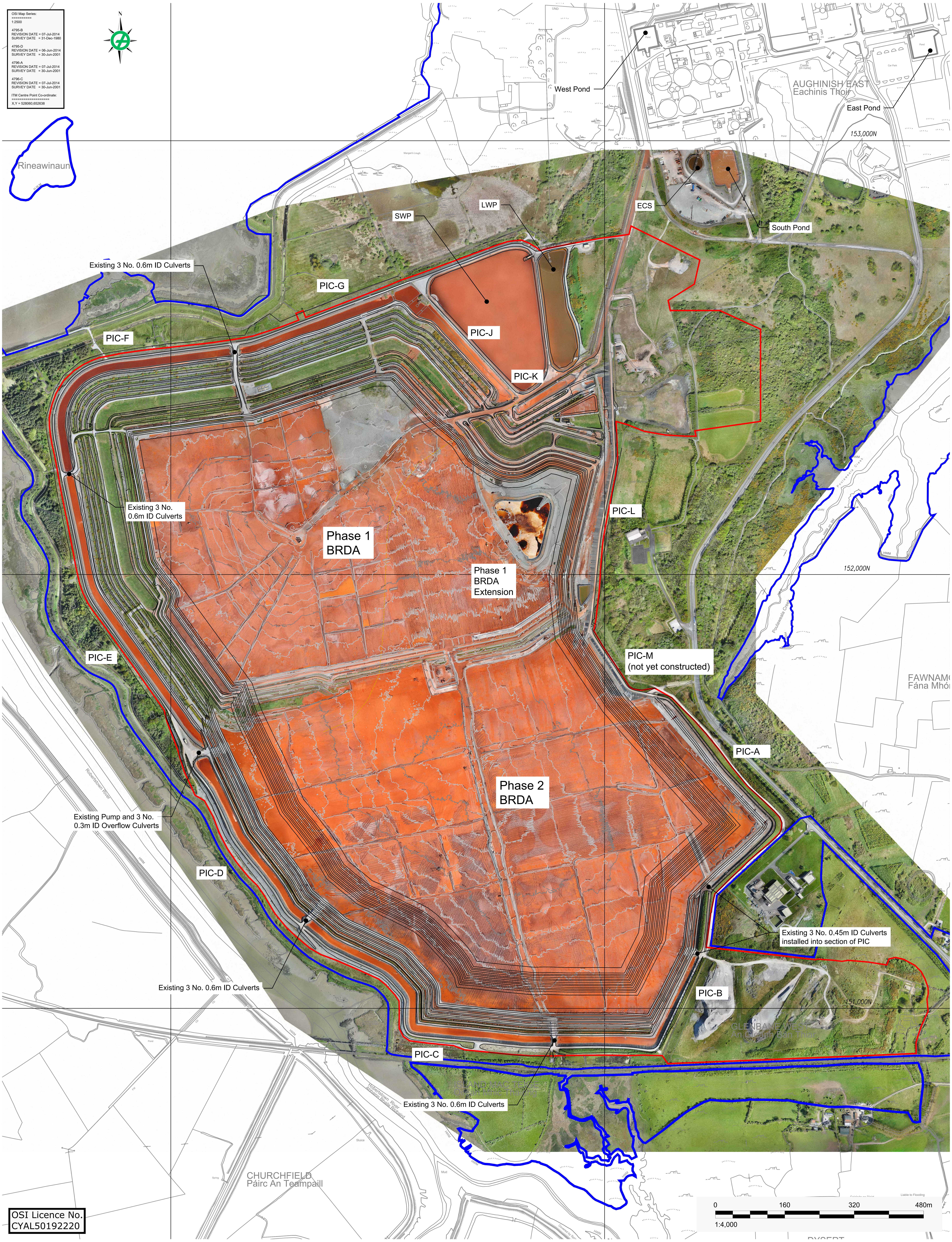

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- AAL OWNERSHIP BOUNDARY
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PROJECT

HYDROLOGICAL ASSESSMENT FOR THE BRDA RAISE DESIGN

TITLE

BRDA LOCATION PLAN TO STAGE 10

PROJECT No.	DRAWING No.	Rev.	SCALE
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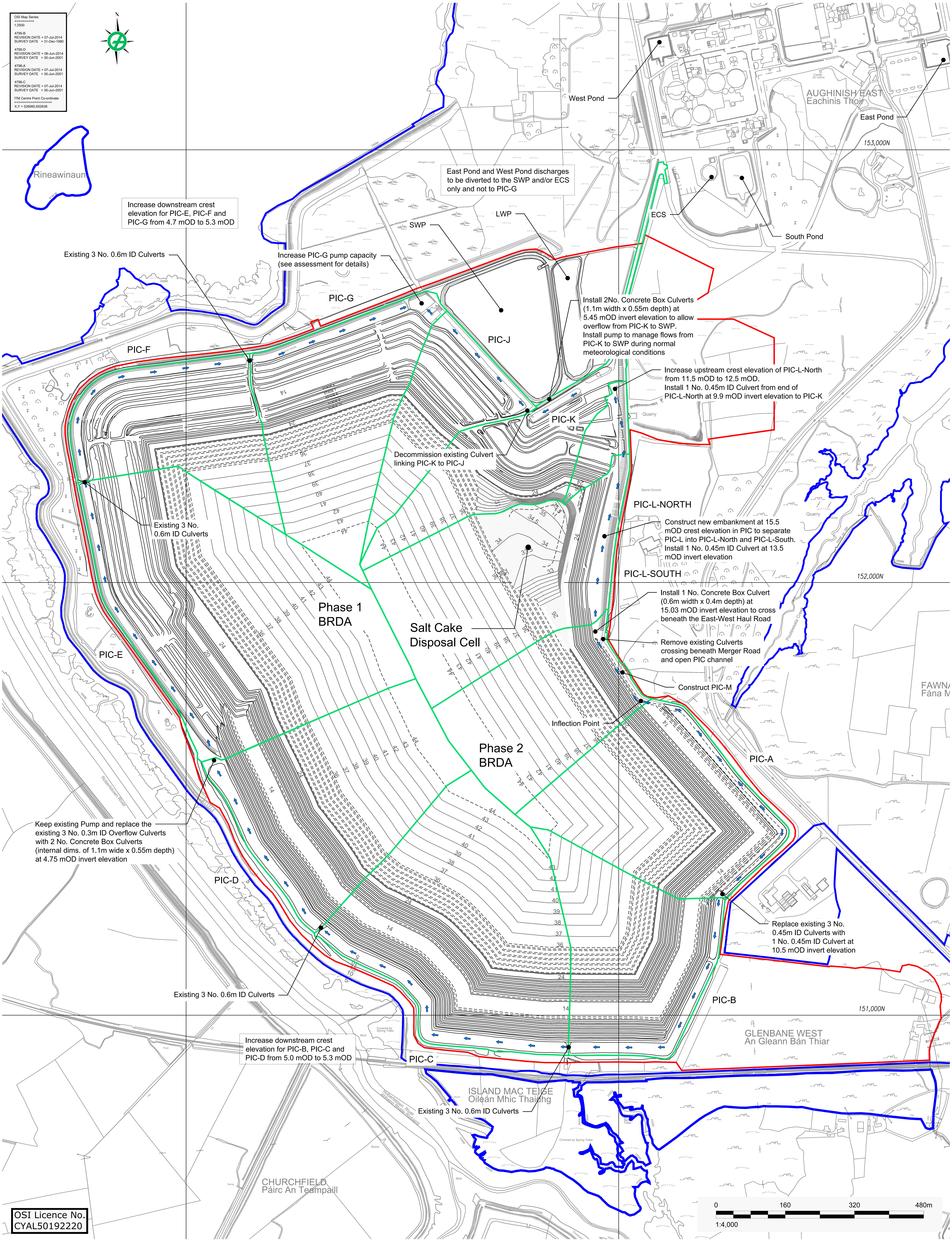

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- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 OGL CONTOUR (mOD)
- PIC CATCHMENT DELINIATIONS

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PROJECT

HYDROLOGICAL ASSESSMENT FOR THE BRDA RAISE DESIGN

TITLE

PROPOSED BRDA RAISE TO STAGE 16 (PRE-CLOSURE) AND UPGRADES TO THE WATER MANAGEMENT SYSTEM

PROJECT No.	DRAWING No.	Rev.	SCALE
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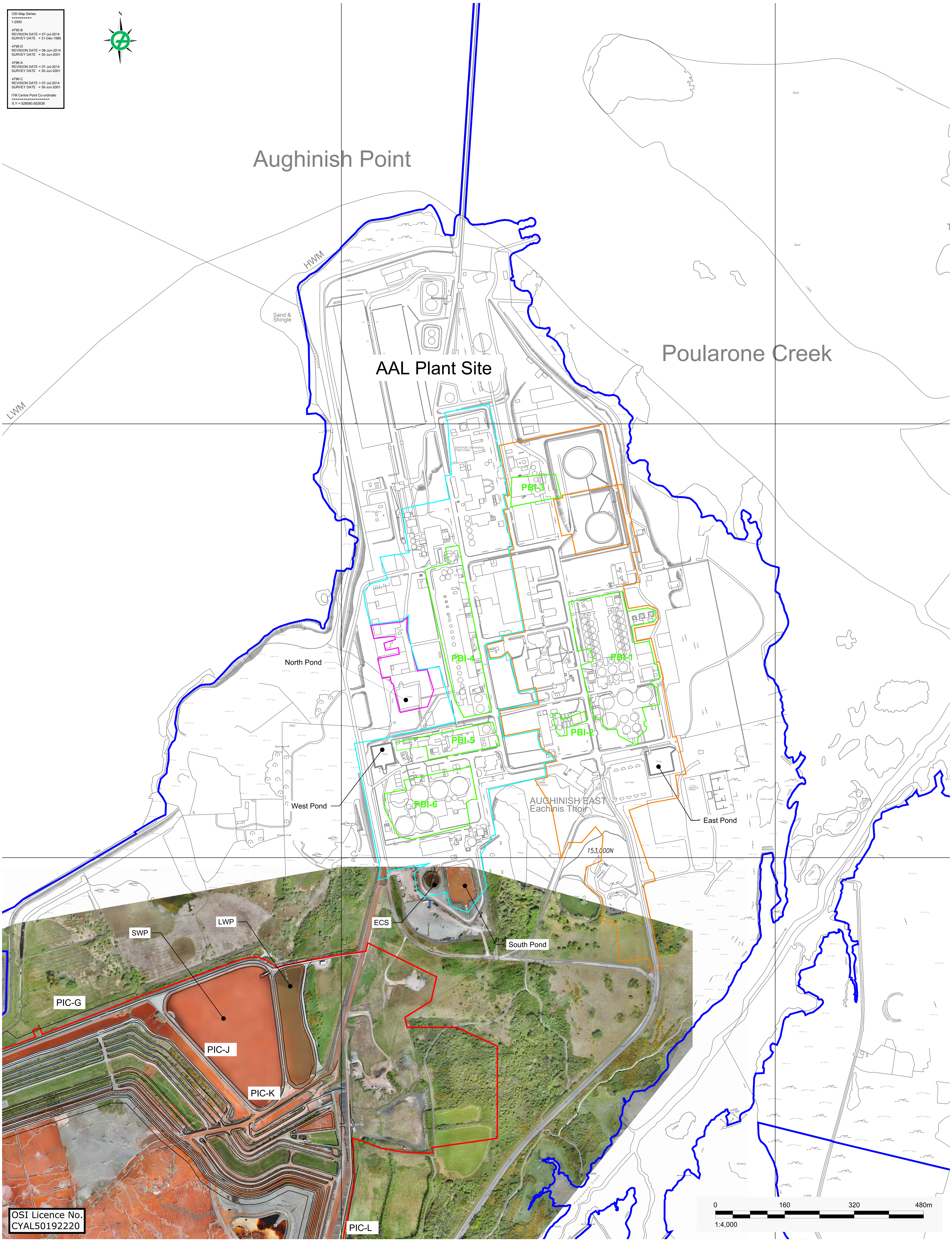

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- APPLICATION BOUNDARY
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- WEST POND CATCHMENT BOUNDARY
- PROCESS AREA SUB-CATCHMENT BOUNDARIES (PBI-1 TO PBI-6)

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HYDROLOGICAL ASSESSMENT FOR THE BRDA RAISE DESIGN

TITLE

PLANT LOCATION PLAN

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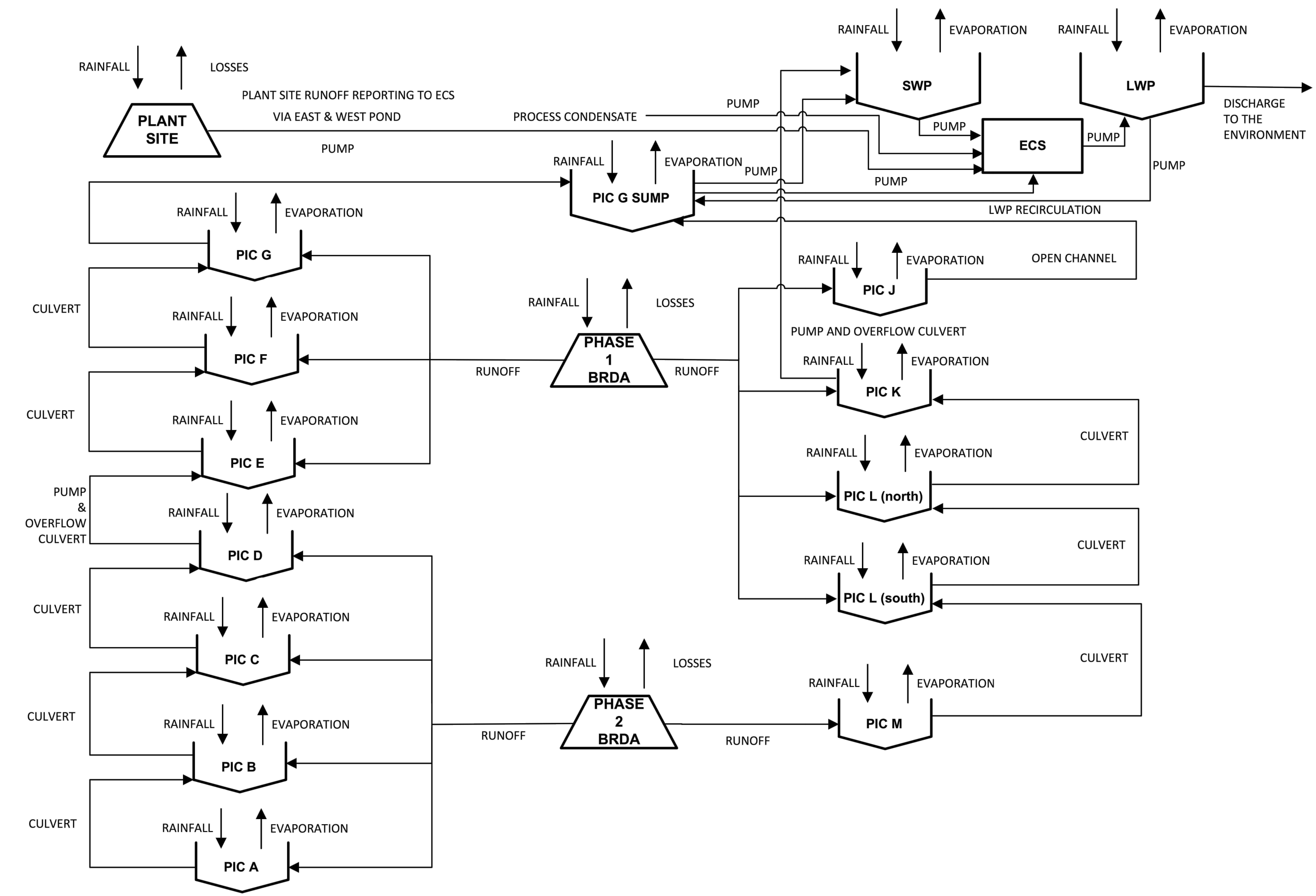

NOTES

1) *Catchment "Losses" presented in the flow diagram represent all hydrological losses from rainfall including evaporation, transpiration, infiltration, and losses due to surface depressions and ponding.*

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TITLE
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BLOCK FLOW DIAGRAM

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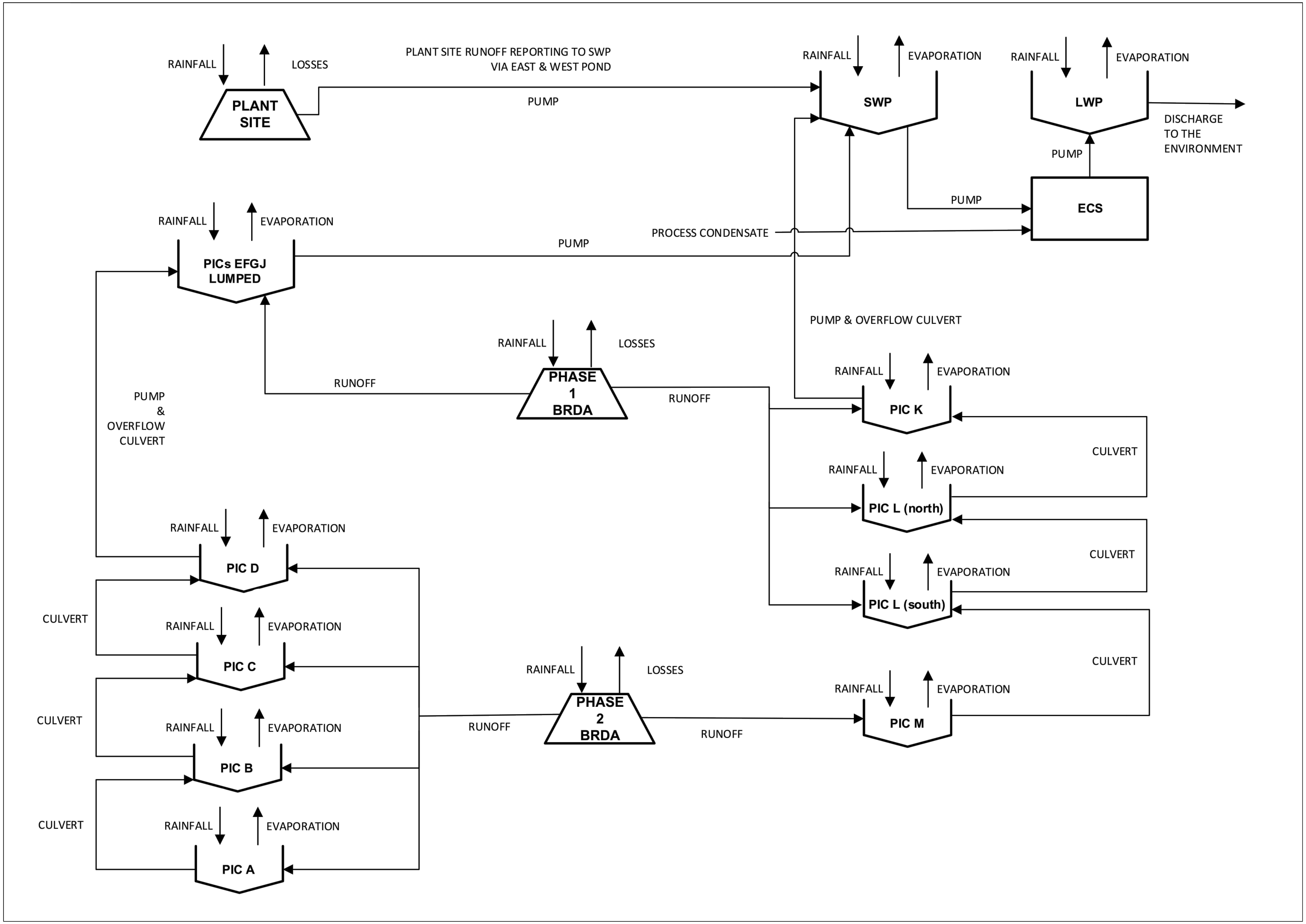

NOTES

1) *Catchment "Losses" presented in the flow diagram represent all hydrological losses from rainfall including evaporation, transpiration, infiltration, and losses due to surface depressions and ponding.*

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PROJECT
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TITLE
BRDA WATER MANAGEMENT SYSTEM MODELLED BLOCK FLOW DIAGRAM

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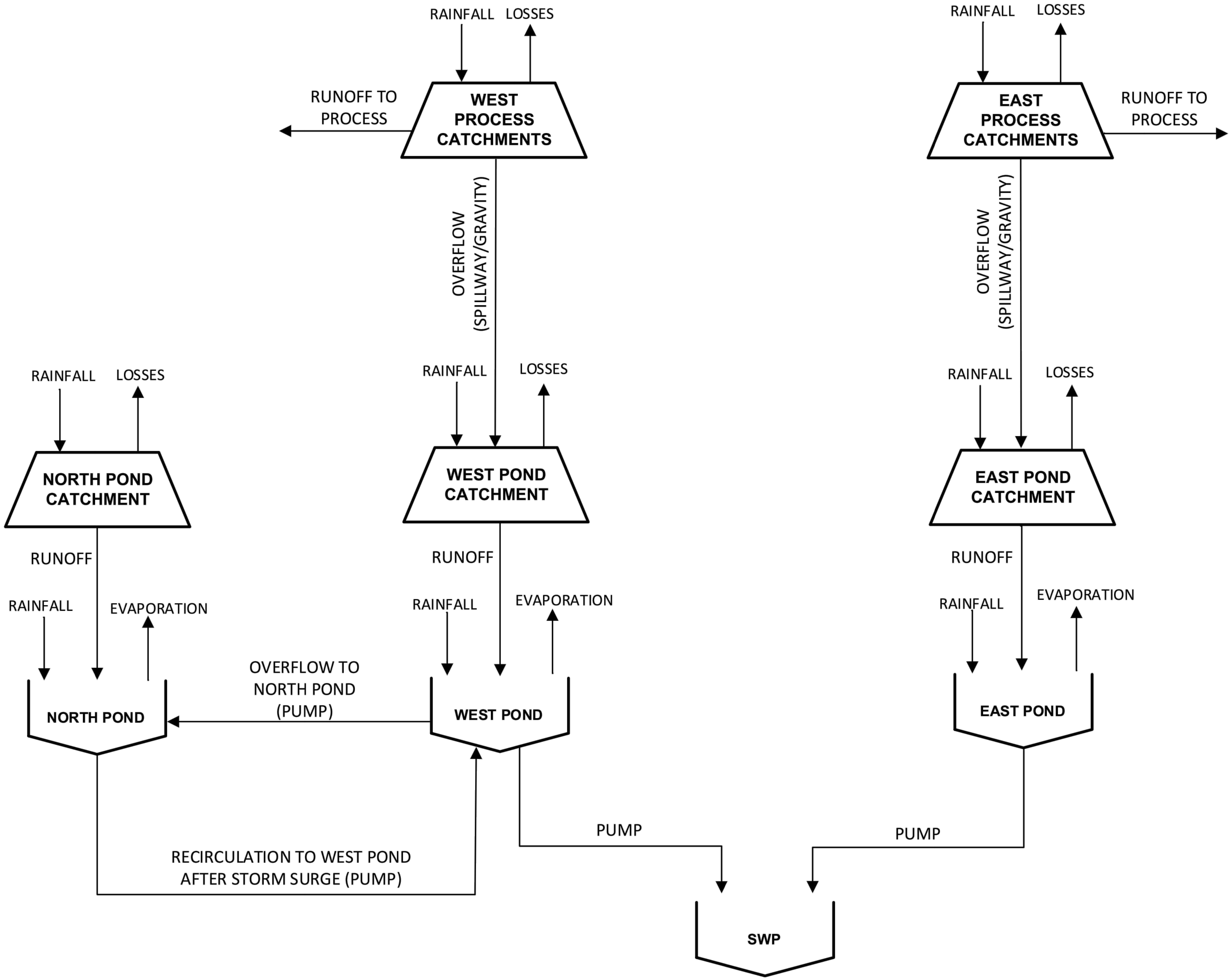

NOTES

1) *Catchment "Losses" presented in the flow diagram represent all hydrological losses from rainfall including evaporation, transpiration, infiltration, and losses due to surface depressions and ponding.*

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DESIGN	DH
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TITLE	**PLANT WATER MANAGEMENT SYSTEM BLOCK FLOW DIAGRAM**

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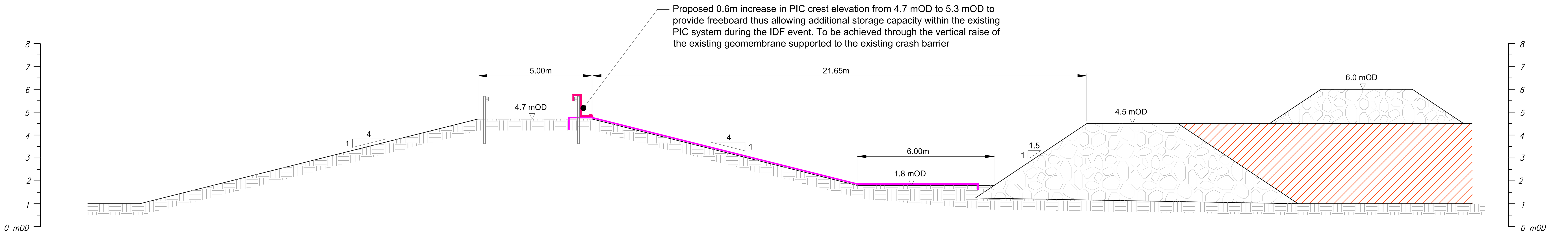

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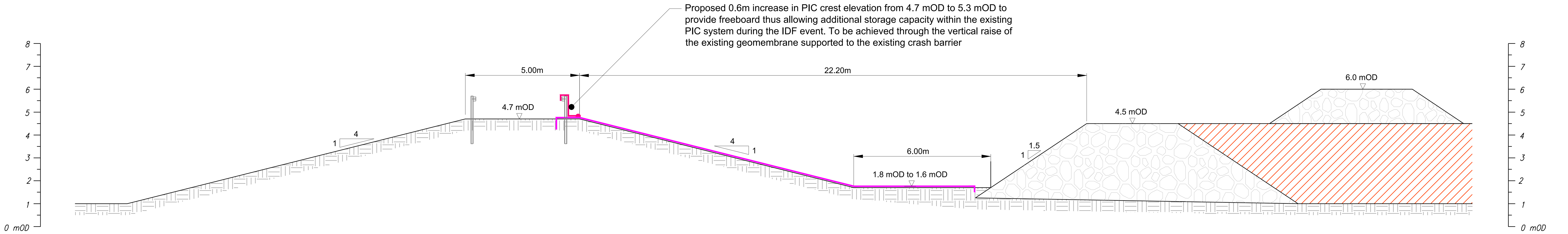

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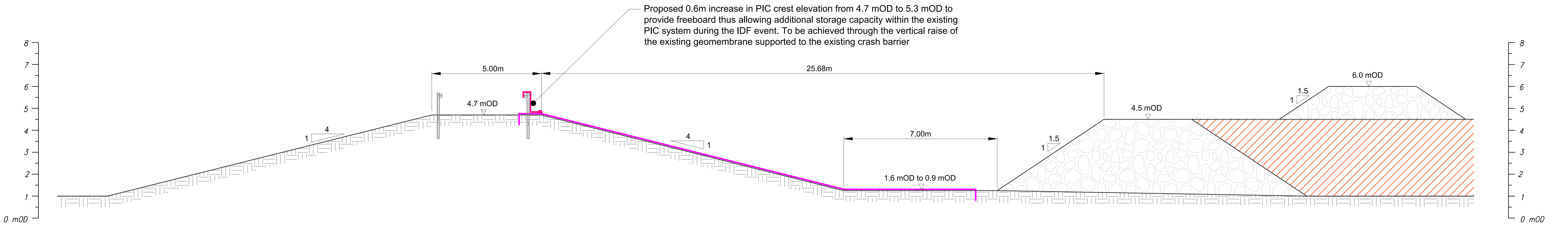

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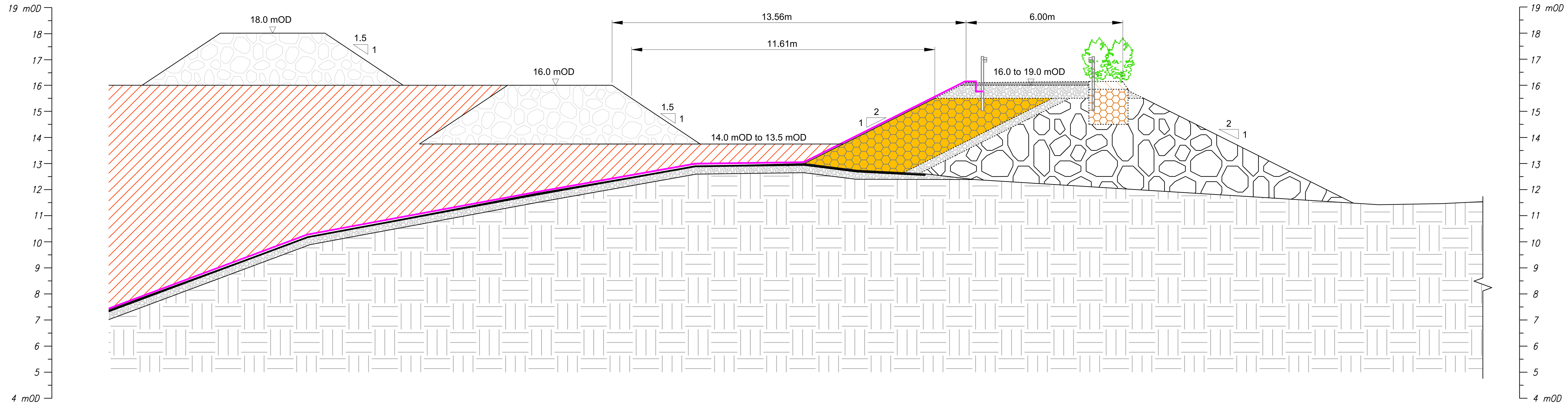

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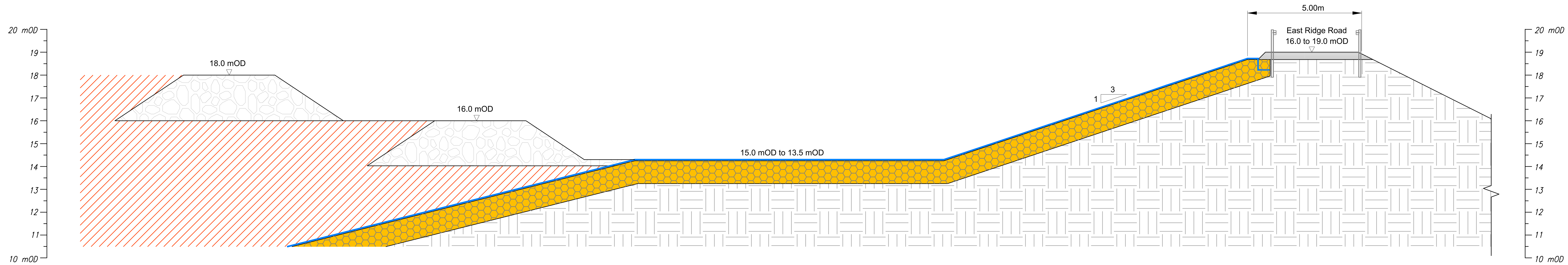

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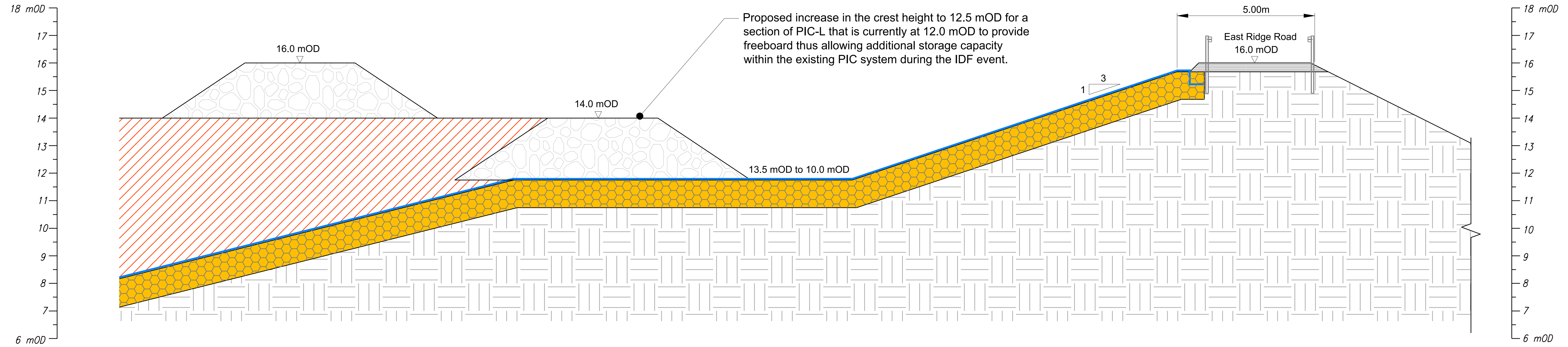

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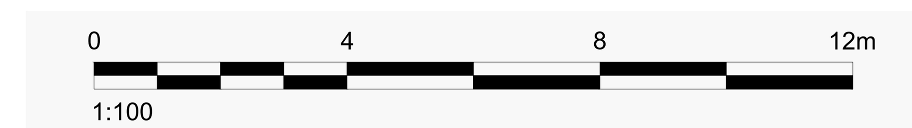

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PIC CROSS SECTIONS 2 OF 5

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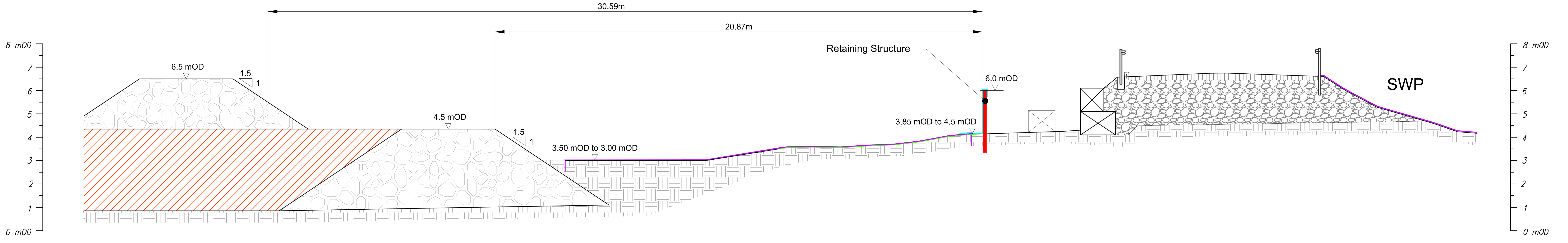

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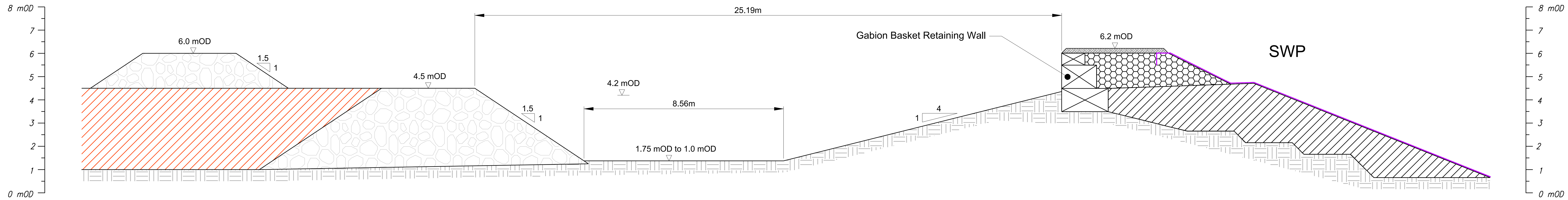

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HYDROLOGICAL ASSESSMENT FOR THE BRDA RAISE DESIGN

TITLE
PIC CROSS SECTIONS 3 OF 5

PROJECT No.	DRAWING No.	Rev.	SCALE
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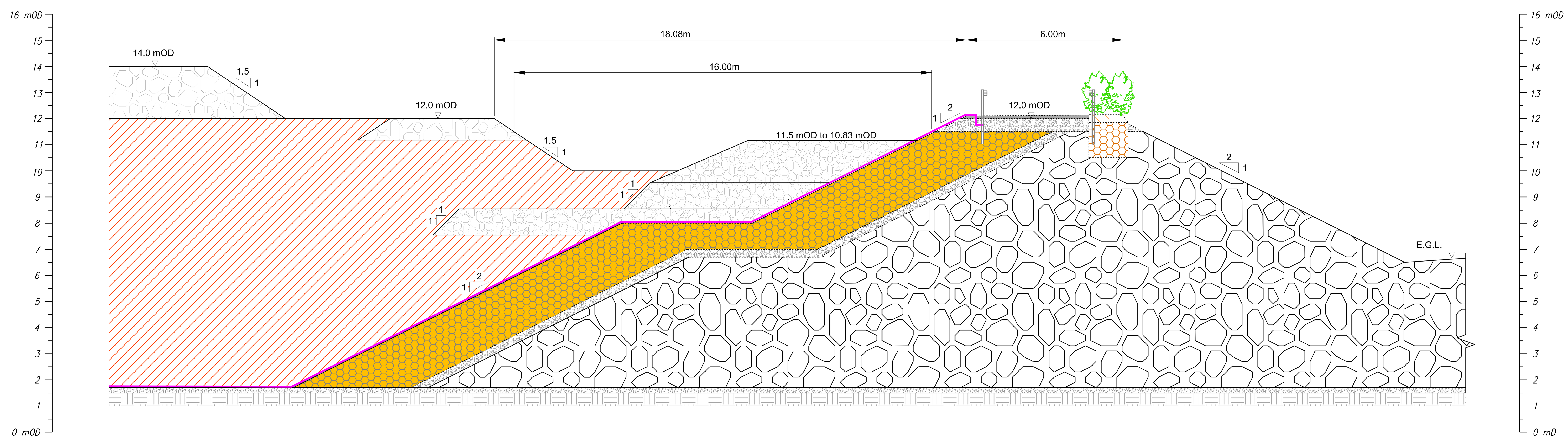

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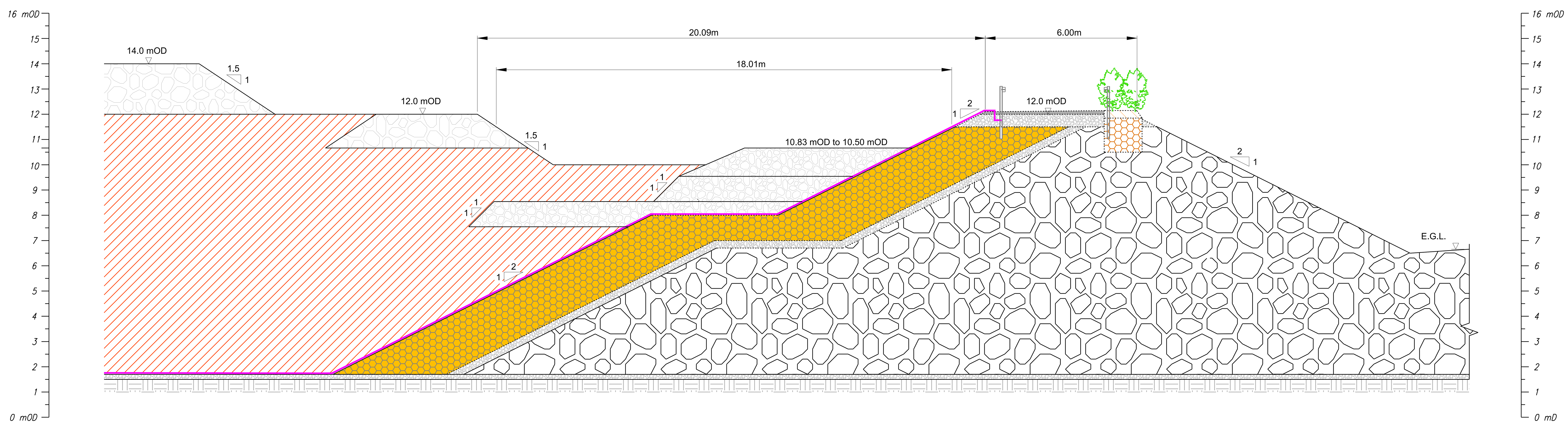

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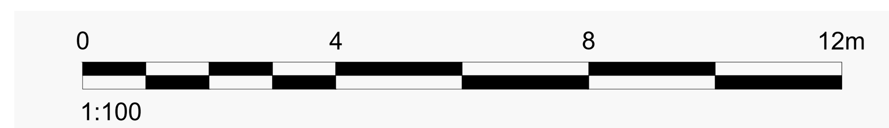


CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Nov-15
PREPARED	POB
DESIGN	DH
REVIEW	BK
APPROVED	BK/DH

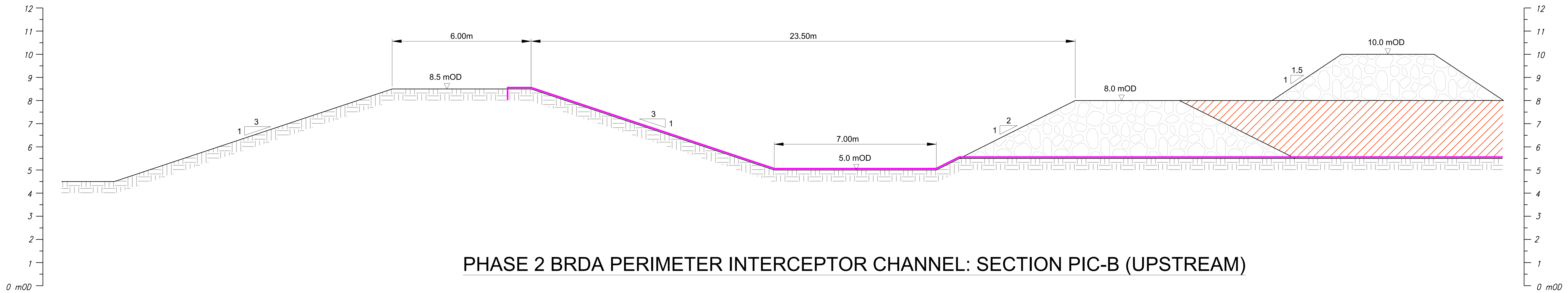
PROJECT

HYDROLOGICAL ASSESSMENT FOR THE BRDA RAISE DESIGN

TITLE

PIC CROSS SECTIONS 4 OF 5

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076.R02	11	A	1:100 A1



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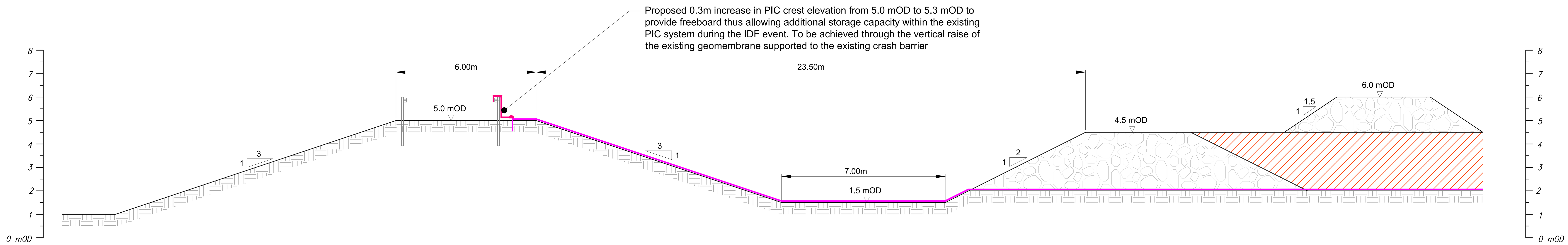

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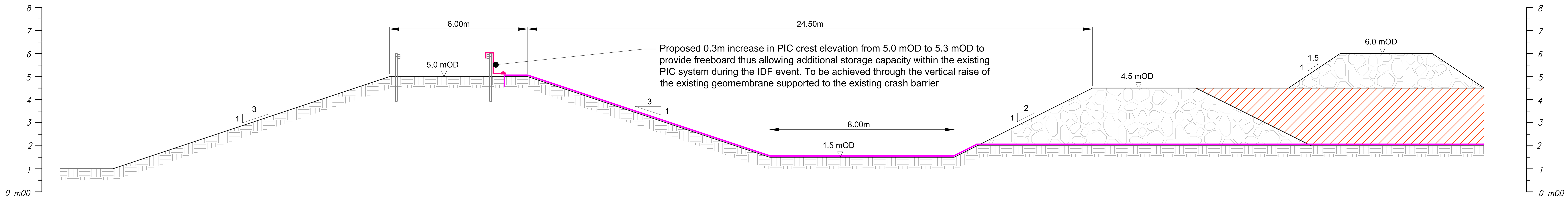

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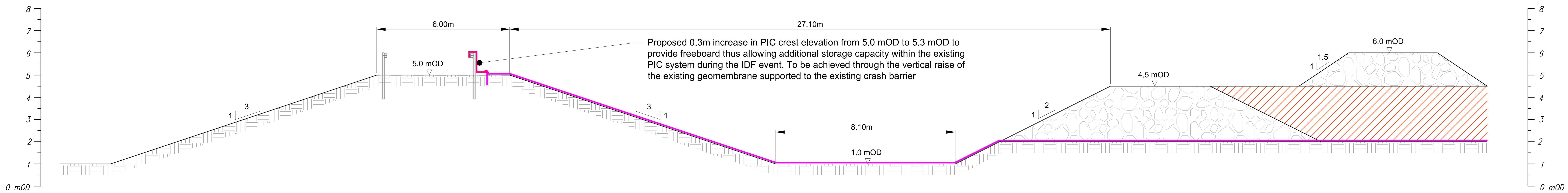

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PHASE 2 BRDA PERIMETER INTERCEPTOR CHANNEL: SECTION PIC-D

CLIENT	AUGHINISH ALUMINA LTD.
PROJECT	HYDROLOGICAL ASSESSMENT FOR THE BRDA RAISE DESIGN
CONSULTANT	GOLDER MEMBER OF WSP
YYYY-MM-DD	2021-Nov-15
PREPARED	POB
DESIGN	DH
REVIEW	BK
APPROVED	BK/DH
TITLE	PIC CROSS SECTIONS 5 OF 5
PROJECT No.	20143076.R02
DRAWING No.	12
Rev.	A
SCALE	1:100 A1

APPENDIX B

Estimated PIC Volume-Level-Surface Area Relationships

Table B.1: PIC-A Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
10.00	0	0
10.50	3,724	1,727
10.51	3,745	1,765
11.00	6,050	4,385
11.50	9,479	8,372
12.00	10,466	13,358

Table B.2: PIC-B Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
1.50	0	0
1.51	9	0.04
2.00	520	130
2.60	1,351	691
3.10	2,218	1,583
3.60	3,241	2,948
4.10	4,423	4,864
4.50	5,481	6,845
5.00	7,606	11,770
5.30	8,267	15,077

Table B.3: PIC-C Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
1.50	0	0
1.51	5,538	28
2.00	7,224	3,154
2.50	8,944	7,196
3.00	10,664	12,098
3.50	12,384	17,860
4.00	14,104	24,482
4.50	15,824	31,964
5.00	20,296	41,510
5.30	21,328	47,393

Table B.4: PIC-D Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
0.30	0	0
0.301	66	0.03
1.00	120	65
1.01	3,586	84
1.50	4,664	2,105
2.00	5,764	4,712
2.50	6,864	7,869
3.00	7,964	11,576
3.50	9,064	15,833
4.00	10,164	20,640
4.50	11,264	25,997
5.00	14,124	32,674
5.30	14,784	36,777

Table B.5: PIC-E Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
1.80	0	0
1.81	4,238	21
2.00	4,970	896
2.30	6,125	2,560
2.80	8,050	6,104
3.30	9,975	10,610
3.80	11,900	16,079
4.20	13,440	21,147
4.50	14,595	25,352
4.70	18,165	28,887
5.30	20,475	39,660

Table B.6: PIC-F Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
1.60	0	0
1.61	177	1
1.80	3,845	383
2.00	4,491	1,216
2.30	5,459	2,709
2.80	7,073	5,842
3.30	8,688	9,782
3.80	10,302	14,530
4.20	11,593	18,909
4.50	13,207	22,629
4.70	15,555	25,658
5.30	17,493	34,885

Table B.7: Combined PIC-G (including pump sump area) Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
0.90	0	0
1.15	2,497	527
1.40	3,828	1,334
1.60	5,025	2,209
1.80	5,534	3,236
2.30	6,808	6,160
2.80	8,081	9,598
3.30	9,354	13,549
3.80	10,627	18,012
4.20	11,646	21,952
4.50	12,410	25,123
4.70	14,411	27,624
5.30	15,939	35,226

Table B.8: Combined PIC-J Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
1.00	0	0
1.15	1,248	31
1.40	2,497	62
1.60	2,813	594
1.80	3,128	1,185
2.30	3,918	2,947
2.80	4,707	5,106
3.30	5,496	5,496
3.50	5,812	8,785
3.80	6,285	10,602
4.20	6,917	13,239
4.50	7,230	16,101
4.70	9,612	17,289
5.30	9,870	23,132

Table B.9: Combined PICs EFG and J Lumped Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
0.90	0	0
1.15	3,746	558
1.40	6,326	1,396
1.60	7,838	2,803
1.80	12,508	4,803
2.30	22,310	14,377
2.80	27,911	26,649
3.30	33,513	39,437
3.80	39,115	59,222
4.20	43,596	75,247
4.50	47,442	89,204
4.70	57,744	99,458
5.30	63,777	132,903

Table B.10: PIC-K Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
1.95	0	0
3.00	59	28
3.50	1,630	670
4.00	2,814	1,881
4.50	4,364	3,595
5.00	4,477	5,806
5.50	4,589	8,073
6.00	4,702	10,396

Table B.11: PIC-L (North) Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
9.88	0	0
10.50	218	44
11.00	394	80
11.50	569	116
12.00	745	152
12.50	1,094	611

Table B.12: PIC-L (South) Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
13.50	0	0
14.00	3,116	670
14.50	6,232	1,340
15.00	7,066	4,665
15.50	7,901	8,406
16.00	8,736	12,566

Table B.13: PIC-M Stage Storage Surface Area Relationship

Elevation (m)	Water Surface Area (m^2)	Volume (m^3)
13.50	0	0
14.00	1,336	311
14.50	1,703	1,071
15.00	2,071	2,014
15.50	2,438	3,141
16.00	2,806	4,452

APPENDIX C

Golder 2019 - Tech. Memo.

MEMORANDUM

DATE 13 March 2019 **Project No.** 1897858

TO B. Keenan

FROM G. Antognazza, C. Campbell **EMAIL** CHCampbell@golder.com

UPDATE OF EXTREME RAINFALL & INFLOW DESIGN FLOODS FOR THE PERIMETER INTERCEPTOR CHANNELS AND THE STORM WATER POND - AUGHINISH BAUXITE RESIDUE DISPOSAL AREA

1.0 INTRODUCTION

Golder Associates Ireland Limited (Golder) has been engaged by Aughinish Alumina Ltd (AAL) to conduct an update to the Risk Assessment for the Bauxite Residue Disposal Area (BRDA) at Aughinish Island. The update incorporates design changes to the BRDA, potential developments in the vicinity of the BRDA, the results of additional geotechnical test work, and design criteria in accordance with the Canadian Dam Association (CDA) Guidelines 2014.

This memorandum presents the following information in support of the Risk Assessment:

- Updated estimates of 1 000, 2 500, 5 000 and 10 000-year rainfall depths;
- Estimates of the Probable Maximum Precipitation (PMP);
- Updated estimates of the Inflow Design Floods and Probable Maximum Floods (PMFs) to the Perimeter Interceptor Channels (PICs) and Storm Water Pond (SWP) which receive runoff from the BRDA, and
- Preliminary results with respect to the performance of the PICs and SWP under the Inflow Design Floods and PMF.

2.0 SITE DESCRIPTION

2.1 Site Location

The site is situated on Aughinish Island approximately 30 km west of Limerick and 15 km southwest of Shannon Airport. The island is bounded by Shannon River to the north, the Robertstown River to the west and southwest and the Poulaweala creek to the east and south east.

2.2 Water Management

The BRDA is located southwest of the existing process plant, with Phase 2 situated immediately south of the existing Phase 1 facility. The BRDA is surrounded by the PICs, which collect runoff from the Phase 1 and Phase 2 facilities and convey it via pumps either to the Effluent Clarifier System (ECS) or to the SWP. The SWP is in the northeast corner of the Phase 1 BRDA.

The Phase 2 PIC collects runoff from the Phase 2 BRDA; water flows clockwise around the Phase 2 facility to its northwest corner, where it is pumped and, depending on the water level, discharged via two overflow culverts into the Phase 1 PIC. A short section of the Phase 2 BRDA PIC, in the north-east corner, flows counter-clockwise to connect with the Phase 1 BRDA PIC at the south-east corner of the Phase 1 BRDA.

There are two Phase 1 PICs that collect runoff from the Phase 1 BRDA. From the southwest corner of the Phase 1 BRDA, water flows clockwise. From the southeast corner of the Phase 1 facility, water flows counter-clockwise. Both PICs converge in the northeast corner of the facility, where water is pumped to the ECS and into the SWP. Overflow culverts are not provided for the Phase 1 PICs.

The function of the SWP is two-fold:

- To provide surge capacity for surface water that cannot be immediately processed by the ECS; and
- To provide a continuous flow of “pond water” that is used for dilution within some parts of the alumina plant.

Excess water from the SWP is pumped to the ECS. The SWP does not have an overflow spillway during operation but will have one installed during the closure works for the post-closure period.

Key design parameters for the PICs and SWP are summarized in Table 1.

Table 1: Key Design Parameters for the PICs and SWP

Facility	Operating Volume (1,000 m³)	Freeboard Volume [(a)] (1,000 m³)	Total Volume (1,000 m³)	Crest Elevation (mOD)
Phase 1 PIC	103.0	22.5	125.5	4.7 (west and north sections)
Phase 2 PICs	67.0 [b]	20.9	87.9	5.0 (west and south sections)
SWP	182.0	58.0	240.0	6.0

Notes:

a) Freeboard is the additional 1 m in the SWP and the additional 0.5 m in the PICs above the 100% operating volume;

b) The operating volume of the Phase 2 PIC will increase when the eastern section of PIC is constructed at elevation 11 mOD.

AAL provided the following information regarding the pumps:

- Pump 24, which discharges from the Phase 2 PIC to the Phase 1 PIC, is unmetered but operates over a water level range of 3.5 m. The pump is level controlled and switches ON at a water level of approximately 2.8 m and switches OFF at a water level of approximately 1.4 m.
- Three pumps discharge water from the Phase 1 PIC:
  - Pump 15 discharges water to the ECS with a flowrate between 100 m^3/hr and 700 m^3/hr over a water level range of 3.1 m. The pump is level controlled but the system can be overridden by the Control Room Operator (CRO). The pump is run continuously for majority of the year. The pump switches ON at a water level of approximately 3.9 m and switches OFF at a water level of approximately 3.0 m.
  - Pumps 33 and 34 discharge water to the SWP. The pumps are not level controlled and are switched ON and OFF by the CRO to manage the overall BRDA water inventory (a low inventory for stormwater management and a high inventory for dust prevention depending on season). The pumps are operated over a water level range of 3.1 m, similar to Pump 15.

- Pumps 31 and 32 discharge water from the SWP to the ECS. Pump 31 is the duty pump and Pump 32 is the stand-by pump. The flowrate from Pump 31 to the ECS ranges between 50 m^3/hr and 300 m^3/hr. The pump is level controlled and operates over a water level range of 4.4 m. The pump switches ON at a water level of approximately 4.6 m and switches OFF at a water level of approximately 1.9 m.

The overall BRDA water inventory targets are 110,000 m^3 in the winter (October – March) to ensure water storage capacity for storm rainfall, and 180,000 m^3 in the summer (May – August) to provide enough water supply for dusting suppression, with two transition months (April and September) where the target is 150,000 m^3. The CRO is responsible for making sure that the inventory targets are met.

3.0 EXTREME RAINFALL AND PROBABLE MAXIMUM PRECIPITATION

3.1 Extreme Rainfall

Rainfall depth-duration-frequency (DDF) data were determined for the location from Met Eireann (https://www.met.ie/climate/services/rainfall-return-periods). The geographical descriptors of the site are:

- Irish Grid: Easting 127433 Northing 151917
- Altitude: 15 m

Rainfall depths were provided for durations ranging from 5 minutes to 25 days and for return periods ranging from 6 months to 500 years.

Rainfall depths for durations of 6, 12, 24 and 48 hours and return periods of 1 000, 2 500, 5 000 and 10 000 years were extrapolated from the data and are presented in Table 2 and Figure 1.

Table 2: Derived Rainfall Depths

Rainfall Duration (hrs)	Rainfall Depths (mm) for varying Return Periods			
	1,000-year	2,500-year	5,000-year	10,000-year
6	80.9	96.5	110.1	125.6
12	93.6	109.9	123.8	139.5
24	107.6	123.7	137.5	152.9
48	131.3	150.1	166.1	183.9

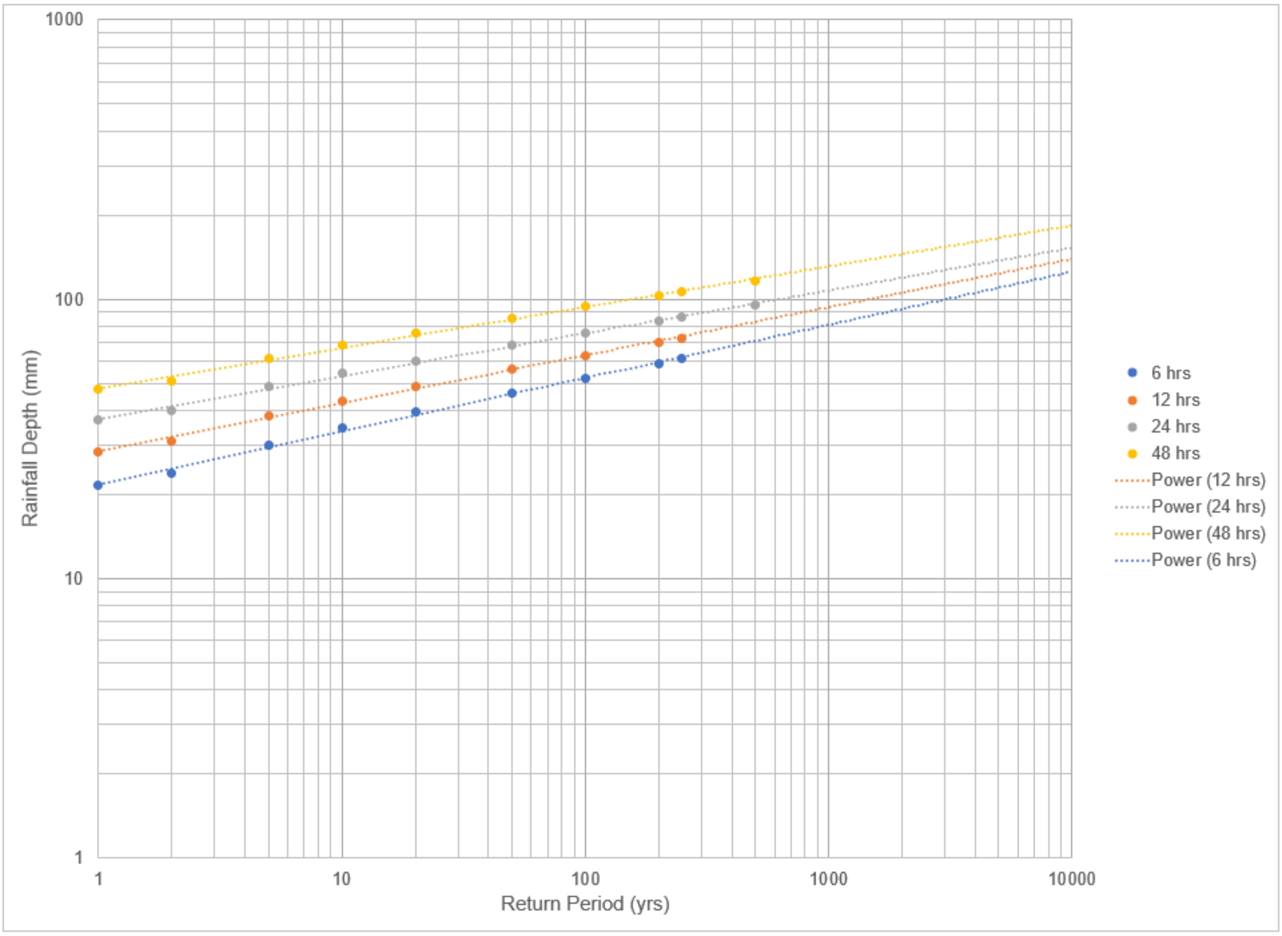

Figure 1: Selected Rainfall Depth-Duration-Frequency Curves

3.2 Probable Maximum Precipitation

Statistical estimates of the Probable Maximum Precipitation (PMP) for the site were determined using the procedures in WMO 2009 'Manual on Estimation of Probable Maximum Precipitation (PMP)', WMO-No. 1045.

Hourly and daily rainfall data for the analysis were downloaded from the Met Eireann website (https://www.met.ie/climate/available-data/historical-data). The data used in this study are presented in Table 3.

Table 3: Provided Rainfall Data

Type of Data	Period Available
Hourly	1989 – 2018
Daily	1945 – 2018

The hourly data was used to estimate the PMP for durations of 6, 12, 24 and 48 hours and the daily data was used to estimate the PMP for durations of 24 and 48 hours. Both datasets were used to estimate the 24-hour and 48-hour PMP for the purposes of comparison. The derived PMPs are presented in Figure 2 and Table 4. Longer records provide results with less statistical uncertainty; the hourly data has a record of 30 years whereas the daily data has a record of 74 years. Therefore, this study considers the results from the daily data. PMP estimates for durations of 6 and 12 hours were extrapolated from the estimates for durations of 24 and 48 hours.

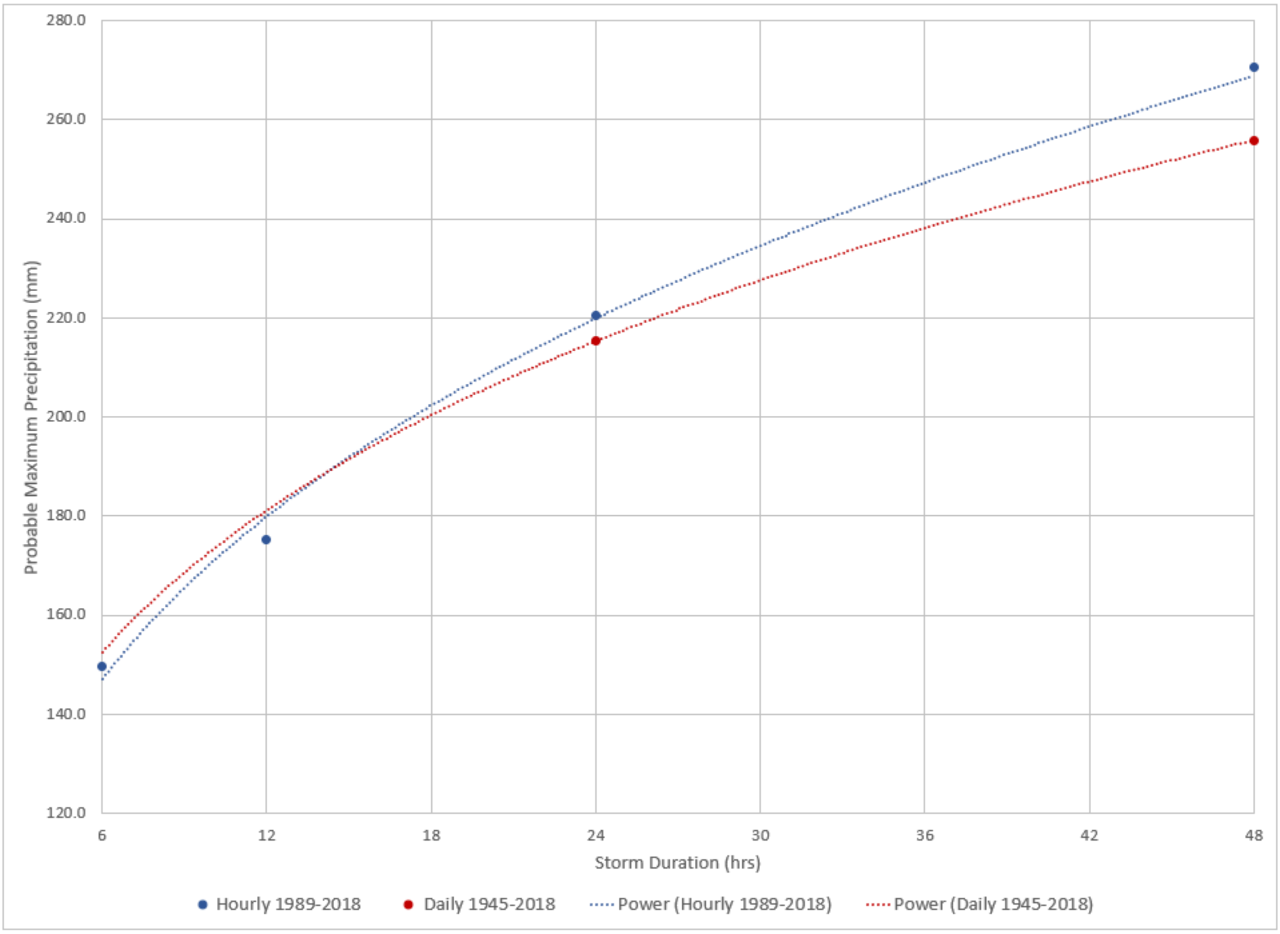

Figure 2: Derived Probable Maximum Precipitation Depths

Table 4: Derived Probable Maximum Precipitation Depths

Rainfall Duration (hrs)	PMP – Hourly Data 1987 – 2018 (mm)	PMP – Daily Data 1945 – 2018 (mm)	Considered PMP (mm)
6	149.56	-	144.7
12	175.39	-	174.60
24	220.39	215.32	208.00
48	270.69	255.92	255.92

4.0 INFLOW DESIGN FLOODS

Based on the Canadian Dam Association Guidelines 2014, the Phase 1 PIC, the Phase 2 PIC and the LWP are deemed to have a **"Low"** risk rating, which sets the target level for the Inflow Design Flood as the 100-yr event. However, the capacity of these structures together with the pumping to the Effluent Clarifier System have been designed to cope with the 200-yr flood event.

The BRDA has a **"High"** risk rating. The Inflow Design Flood will be 1/3 between the 1000-yr event and the PMF. The PMF is the runoff generated during the PMP event.

A HEC-HMS model was developed to simulate the runoff generated by the BRDA during the 200-yr, 1,000-yr, 1/3 between the 1,000-yr and PMP rainfall events and the PMP for durations of the 6, 12, 24 and 48 hours, as shown in Table 5. The model is described in section 5.1 below.

Table 5: Simulated Rainfall Depths

Rainfall Duration (hrs)	Rainfall Depths (mm)			
	200-year	1,000-year	1/3 between 1,000-year and PMP	PMP
6	58.9	80.9	102.2	144.7
12	70.2	93.6	120.6	174.6
24	83.7	107.6	141.0	208.0
48	103.3	131.3	172.8	255.9

The drainage characteristics assigned to the BRDA in the model are summarized in Table 6. The time of concentration, which is the time for the entire BRDA to contribute runoff is 24.7 hours. Hence, the critical storm duration in order to consider the highest peak flow at the outlet of the system will be between 24 and 48 hours.

Table 6: BRDA Drainage Characteristics

Facility	Area (km^2)	Curve Number	Time Lag (hrs)	Time of Concentration (hrs)
Phase 1 BRDA	1.029	90	9.8	16.3
Phase 2 BRDA	0.743	90	14.8	24.7

The resulting peak runoff rates and runoff volumes are provided in Table 7 and Table 8.

Table 7: BRDA Peak Flows and Runoff Volumes (200-year and 1,000-year rainfall events)

Rainfall Duration (hrs)	Phase 1 BRDA				Phase 2 BRDA			
	200-yr Peak Flow (m^3/s)	200-yr Runoff Volume ($1000m^3$)	1,000-yr Peak Flow (m^3/s)	1,000-yr Runoff Volume ($1000m^3$)	200-yr Peak Flow (m^3/s)	200-yr Runoff Volume ($1000m^3$)	1,000-yr Peak Flow (m^3/s)	1,000-yr Runoff Volume ($1000m^3$)
6	0.649	35.818	1.027	56.508	0.334	25.742	0.528	40.617
12	0.839	46.221	1.250	68.714	0.432	33.220	0.642	49.393
24	1.073	58.990	1.493	82.069	0.551	42.401	0.767	58.996
48	1.418	77.958	1.919	105.585	0.728	56.039	0.986	75.905

Table 8. BRDA Peak Flows and Runoff Volumes (1/3 between 1,000-year and PMP, and the PMF)

Rainfall Duration (hrs)	Phase 1 BRDA				Phase 2 BRDA			
	1/3 1,000-yr & PMP Peak Flow (m^3/s)	1/3 1,000-yr & PMP Runoff Volume ($1,000m^3$)	PMF Peak Flow (m^3/s)	PMF Runoff Volume ($1,000m^3$)	1/3 1,000-yr & PMP Peak Flow (m^3/s)	1/3 1,000-yr & PMP Runoff Volume ($1,000m^3$)	PMF Peak Flow (m^3/s)	PMF Runoff Volume ($1,000m^3$)
6	1.447	79.523	2.299	126.655	0.743	57.164	1.181	91.056
12	1.768	97.249	2.818	155.527	0.908	69.911	1.449	111.819
24	2.137	117.648	3.437	190.140	1.098	84.579	1.768	144.413
48	2.668	147.157	4.173	231.415	1.371	105.800	2.148	166.395

5.0 PIC AND SWP PERFORMANCE

5.1 Model Set Up

To assess the response of the PICs and SWP to the design Inflow Design Floods in Table 5 above, the facilities were modelled in HEC-HMS. The conceptual model is shown in Figure 3.

There are five reservoir elements in the model representing the Phase 1 PIC, the Phase 2 PIC, the SWP, the ECS and the LWP. Given the design of the PICs (i.e. blind channels with pumped outflows), these facilities have been represented as storage facilities and not as channels in the model.

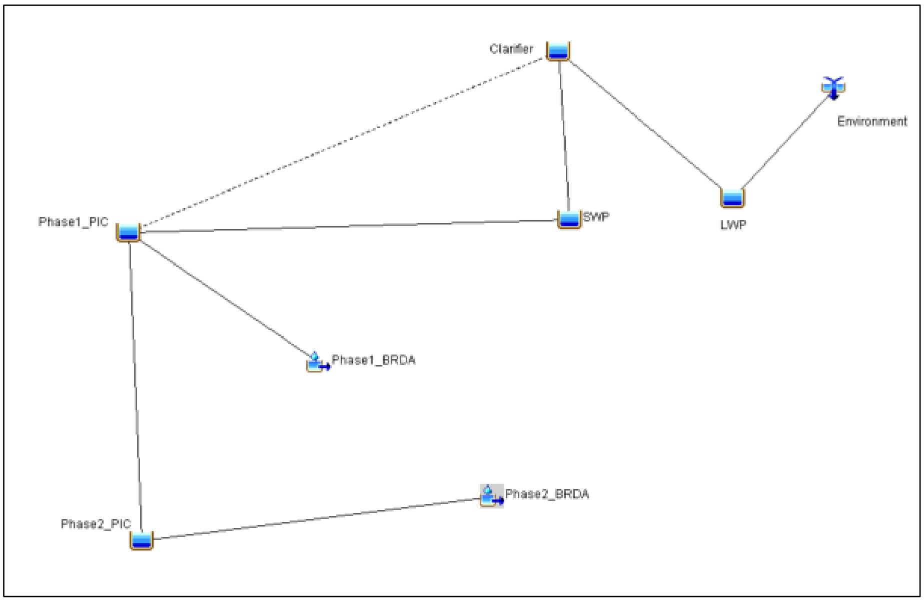

Figure 3: Conceptual Drainage Model

The reservoir elements representing the Phase 1 and 2 PICs have been assigned the elevation-volume curves in Table 9 and Table 10, respectively. The curve for the Phase 1 PIC was provided by AAL in '6th Feb data for PMF calculation.xlsx'. The curve for the Phase 2 PIC was taken from 'Rusal Aughinish Standard Work Method, Water Management', revised on 12/09/2018.

Table 9: Phase 1 PIC Elevation-Volume Curve

Elevation (m)	Volume (m^3)	Remarks
0	0	Bottom of channel
0.5	200	
1.0	550	
1.5	2,130	
2.0	11,295	
2.5	25,745	
3.0	42,700	
3.5	63,035	
4.0	86,755	
4.2	103,000	Freeboard level
4.5	113,855	
4.7	125,606	Top of channel

Table 10: Phase 2 PIC Elevation-Volume Curve

Elevation (m)	Volume (m^3)	Remarks
0	0	Bottom of channel
4.5	67,000	Freeboard level
5.0	87,900	Top of channel

The elevation-volume curve for the SWP was taken from the as-built drawing titled 'SW-pond – asbuilt.dwg' and is shown in Figure 4. The bottom of pond level is 0.8 m and the top of pond level is 5.9 m; the SWP has a total capacity of 234,410 m³. The operating volume of the SWP is 182,000 m³ which corresponds to a freeboard level of approximately 5.0 m.

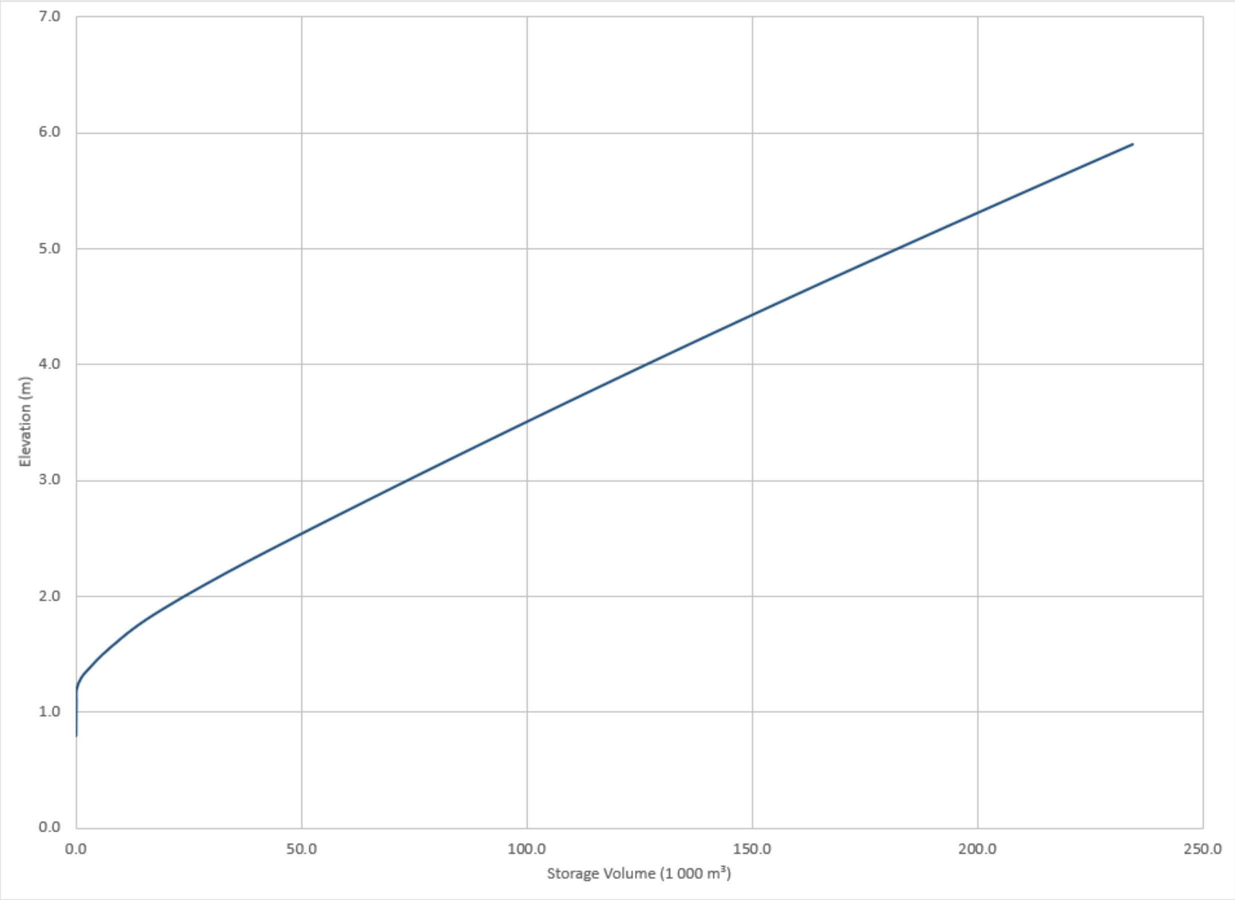

Figure 4: SWP Elevation-Storage Curve

The elevation-storage curve for the LWP was taken from the as-built drawing titled 'LW-pond – asbuilt.dwg' and is shown in Figure 5. The bottom of pond level is 1.1 m and the top of pond level is 5.9 m; the LWP has a total capacity of 54,300 m³. An operating volume of 39,070 m³ has been assumed which corresponds to a freeboard level of 5.0 m.

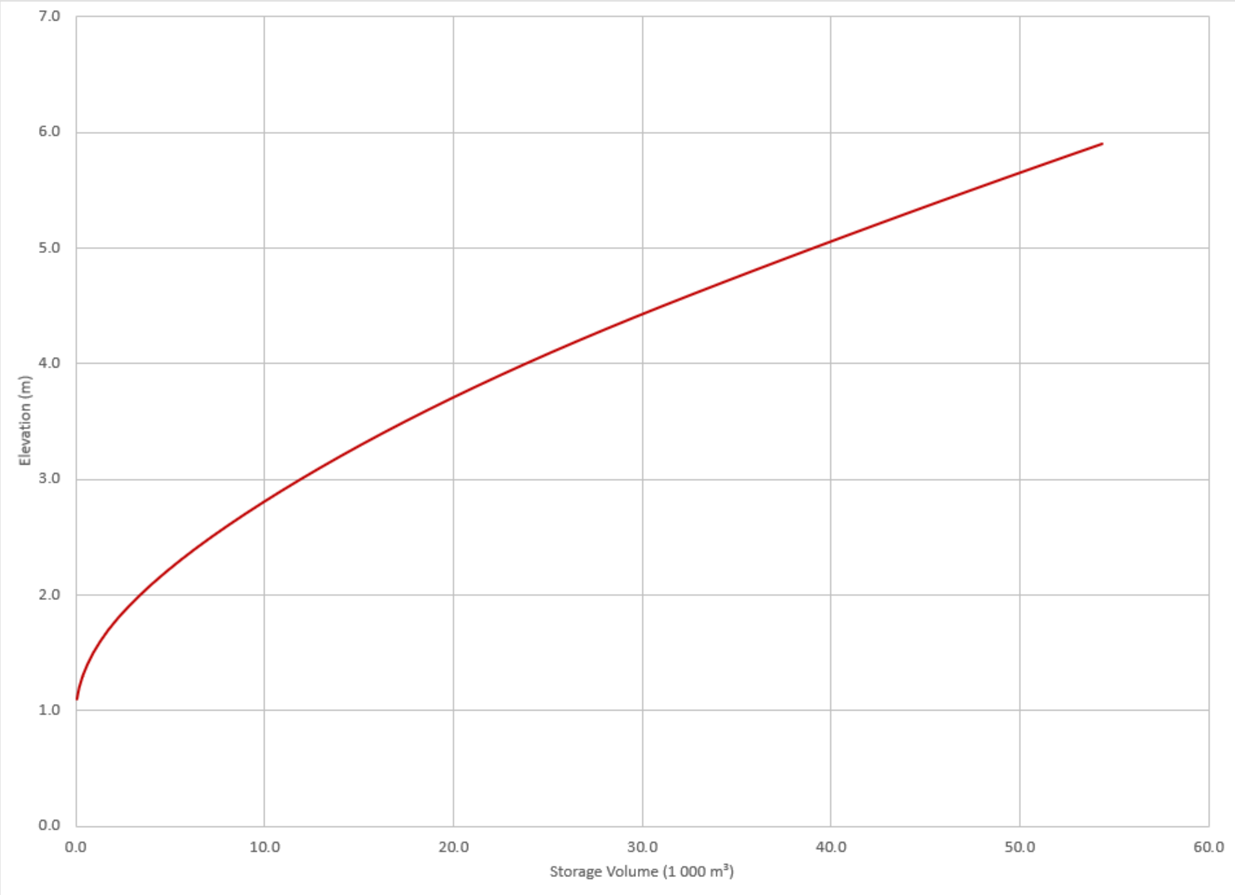

Figure 5. LWP Elevation-Storage Curve

The ECS consists of several tanks and ponds for which design details are unknown, and an elevation-volume curve was assumed for the system to allow the model to function. The resulting elevation-volume curve is presented in Table 11.

Table 11: Clarifier System Elevation-Storage Curve

Elevation (m)	Volume ($m^{3)}$)
12.0	0
13.0	19,910
14.0	40,440
15.0	61,600
16.0	83,390
17.0	105,800

The model also includes the following outflow structures:

- Pump 24 which discharges water from the Phase 2 PIC to the Phase 1 PIC;

- Two overflow pipe culverts which connect the Phase 2 PIC to the Phase 1 PIC;
- Three pumps which discharge water from the Phase 1 PIC:
  - Pump 15 which discharges into the ECS; and
  - Pump 33 and Pump 34 which discharge to the SWP. It is assumed that both pumps operate simultaneously.
- Pump 31 and/or Pump 32 which discharge water from the SWP to the ECS.

The default pumping system characteristics used in the model are provided in Table 12. The pump curves provided by AAL are summarised in Table 13.

Table 12: Pumping System Characteristics

Parameter	Pump 24	Pump 15	Pumps 33 and 34	Pump 31 or 32
Pump intake elevation (m)	0.5	0.5	0.5	1.3(1)
Pump line elevation (m)	5.0	19.0	5.0	19.0
Pump switch ON level	2.8	3.9	3.9	4.6
Pump switch OFF level	1.4	3.0	2.0(2)	1.9
Equipment Loss (m)	6.0	17.0	6.0	33.0

NOTES: (1) Pump intake elevation assumed to be 0.5 m above SWP invert level of 0.8 m; (2) pump switch OFF level assumed to be at the level of the upstream invert of the lowest overflow culvert from the Phase 2 PIC to the Phase 1 PIC.

Table 13: Pump Curves

Pumps 24, 33 and 34		Pump 15		Pumps 31 or 32	
Head (m)	Flow (m^3/hr)	Head (m)	Flow (m^3/hr)	Head (m)	Flow (m^3/hr)
6	855	30.5	1181	45.2	587
8	761	33.5	1136	48.0	548
12	576	36.6	1067	52.0	485
16	427	39.6	999	56.0	439
20	278	42.7	908	60.0	374
24	124	45.7	818	64.0	289
26	0	48.8	704	68.0	200

Pumps 24, 33 and 34		Pump 15		Pumps 31 or 32	
Head (m)	Flow (m^3/hr)	Head (m)	Flow (m^3/hr)	Head (m)	Flow (m^3/hr)
		51.8	613	72.0	135
		54.9	488	76.0	39
		57.9	318	79.4	0
		60.7	0		

Design characteristics for the overflow pipe culverts between the Phase 2 PIC and the Phase 1 PIC are summarised in Table 13.

Table 14: Phase 2 PIC Overflow Pipe Culverts

Characteristic	Culvert 1	Culvert 2
Shape	Circular	Circular
Material	HDPE	HDPE
Inlet Type	Projecting from embankment	Projecting from embankment
Length (m)	56	56
Diameter (m)	0.6	0.6
Inlet Elevation (m)	2.0	2.6
Outlet Elevation (m)	1.8	2.4

For the purposes of this assessment, the following have also been assumed:

- The Effluent Clarifier System supplies water to the LWP at a constant flow rate of 1,200 m^3/hr; and
- The LWP discharges to the environment at a constant rate of 1,200 m^3/h.

5.2 Model Scenarios

The model was also used to simulate the performance of the Phase 1 PIC, the Phase 2 PIC and the SWP during storm durations of 6, 12, 24 and 48 hours for the following events:

- Rainfall depth with a 200-year return period;
- Rainfall depth with a return period of 1,000 years;
- Rainfall depth equal to 1/3 between the 1,000-year rainfall and the PMP; and
- The PMP.

Thus, a total of 16 rainfall scenarios were modelled. The default initial water volumes and levels in the modelled facilities, at the start of each simulation, are provided in Table 15.

Values for the Phase 2 PIC, the Phase 1 PIC and the SWP sum to 180,000 m³, the BRDA water inventory target in the summer months. It is assumed that the water level in the Phase 1 PIC is at the level of the upstream invert of the lowest overflow culvert from the Phase 2 PIC, the water level in the Phase 2 PIC is at the switch OFF level for Pump 24, and that remainder of the inventory is stored in the SWP. Values for the ECS and the LWP correspond to a depth one metre below the freeboard levels in these facilities.

Table 15: Initial Water Volumes and Levels in BRDA Facilities

Facility	Initial Water Volume (m³)	Initial Water Level (m)
Phase 2 PIC	20,100	1.35
Phase 1 PIC	11,295	2.00
SWP	148,605	4.40
ECS	61,600	15.0
LWP	22,480	3.9

5.3 Results

5.3.1 Rainfall Depth, 200-year Return Period

The results of the simulations for the 200-year rainfall event presented in Table 16 and summarised below:

- The system performs without overtopping for all the storm durations with the assumed initial water levels and pumping system characteristics.
- The peak water levels reached in the Phase 1 PIC and the Phase 2 PIC are 3.9 m and 3.8 m, respectively. These water levels occur during the 48-hour event. Both levels are below the freeboard levels of the facilities (4.2 m for the Phase 1 PIC and 4.5 m for the Phase 2 PIC).
- The peak water level reached in the SWP is 5.1 m and occurs during the 48-hour event. This level is above the freeboard level of the facility (4.9 m).

Table 16. Peak Discharges, Water Levels and Volumes - 200-year Rainfall

Rainfall Duration (hrs)	Phase 1 PIC			Phase 2 PIC			SWP		
	Peak Discharge(1) (m³/s)	Peak Water Level (m)	Peak Storage (1,000m³)	Peak Discharge (m³/s)	Peak Water Level (m)	Peak Storage (1,000m³)	Peak Discharge (m³/s)	Peak Water Level (m)	Peak Storage (1,000m³)
6	0	3.59	67.07	0.201	2.75	40.95	0	4.40	148.39
12	0.433	3.89	81.53	0.203	2.84	42.23	0	4.43	150.24
24	0.432	3.89	81.53	0.207	3.17	47.25	0.155	4.84	173.03
48	0.433	3.89	81.59	0.254	3.81	56.78	0.156	5.09	187.43

NOTE: (1) Discharge to the SWP only; does not include discharge to the ECS.

5.3.2 Rainfall Depth, 1,000-year Return Period

The results of the simulations for the 1,000-year rainfall event are presented in Table 17. For the 1,000-year rainfall event:

- The system performs without overtopping for all the storm durations with the assumed initial water levels and pumping system characteristics.
- The peak water levels in the Phase 1 PIC and the Phase 2 PIC are 4.1 m and 4.6 m, respectively, during the 48-hour rainfall event. The water level in the Phase 1 PIC is slightly below the freeboard level of the facility (4.2 m), but the water level in the Phase 2 PIC is slightly above the freeboard level of the channel (4.5 m).
- The peak water level reached in the SWP is 5.2 m during the 48-hour event. This level exceeds the freeboard level of the facility (4.9 m).

Table 17. Peak Discharges, Water Levels and Volumes – 1,000-year Rainfall

Rainfall Duration (hrs)	Phase 1 PIC			Phase 2 PIC			SWP		
	Peak Discharge(1) (m^3/s)	Peak Water Level (m)	Peak Storage (1,000m^3)	Peak Discharge (m^3/s)	Peak Water Level (m)	Peak Storage (1,000m^3)	Peak Discharge (m^3/s)	Peak Water Level (m)	Peak Storage (1,000m^3)
6	0.433	3.89	81.54	0.212	3.08	45.91	0.155	4.79	170.29
12	0.433	3.89	81.57	0.217	3.47	51.63	0.156	5.01	182.72
24	0.433	3.89	81.63	0.255	3.94	58.61	0.156	5.12	188.98
48	0.437	4.05	90.97	0.455	4.55	68.89	0.157	5.22	194.96

NOTE: (1) Discharge to the SWP only; does not include discharge to the ECS.

5.3.3 Rainfall Depth, 1/3 between 1,000-year Event and PMP

The results of the simulations for the rainfall depth 1/3 between the 1,000-year event and the PMP are presented in Table 18 and are summarised below:

- The system performs without overtopping for storm durations of 6, 12 and 24 hours with the assumed initial water levels and pumping system characteristics.
- The peak water levels in the Phase 1 PIC, the Phase 2 PIC and the SWP during the 24-hour event are 4.1 m, 4.8 m and 5.3 m, respectively. The water level in the Phase 1 PIC is slightly below the freeboard level of the facility (4.2 m), but the water levels in the Phase 2 PIC and the SWP exceed the freeboard levels of these facilities (4.5 m for the Phase 2 PIC and 4.9 m for the SWP).
- Overtopping occurs at the Phase 1 PIC for the 48-hour duration under the default settings.
- For the Phase 1 PIC to perform without overtopping during the 48-hour duration, the capacities of Pumps 33 and 34 which discharge to the SWP will need to be increased. A total pumping rate of approximately 2,300 m^3/hr will be required (1,150 m^3/hr for each pump).

- If the capacities of Pumps 33 and 34 are increased, the peak water levels in the Phase 1 PIC, the Phase 2 PIC and the SWP during the 48-hour rainfall events are 4.7 m, 4.8 m, and 5.8 m, respectively. These water levels exceed the freeboard levels in all the facilities (4.2 m for the Phase 1 PIC, 4.5 m for the Phase 2 PIC, and 4.9 m for the SWP). The water level in the Phase 1 PIC is at the level of the top of the facility, and the water levels in the Phase 2 PIC and the SWP are only slightly below the tops of these facilities (5.0 m and 5.9 m, respectively).

Table 18. Peak Discharges, Water Levels and Volumes – 1/3 between 1,000-year Event and PMP

Rainfall Duration (hrs)	Phase 1 PIC			Phase 2 PIC			SWP		
	Peak Discharge[(1)] (m^3/s)	Peak Water Level (m)	Peak Storage ($1,000m^3$)	Peak Discharge (m^3/s)	Peak Water Level (m)	Peak Storage ($1,000m^3$)	Peak Discharge (m^3/s)	Peak Water Level (m)	Peak Storage ($1,000m^3$)
6	0.433	3.89	81.58	0.253	3.84	57.24	0.156	5.10	188.00
12	0.435	3.99	86.45	0.395	4.31	64.20	0.157	5.19	193.22
24	0.439	4.14	98.40	0.513	4.79	79.07	0.157	5.26	196.97
48 [(2)]	0.635	4.67	123.71	0.650	4.77	78.47	0.159	5.83	230.54

NOTES: (1) Discharge to the SWP only; does not include discharge to the ECS. (2) Assumes that the capacities of Pumps 33 and 34 have been increased by a factor of 1.4.

5.3.4 Rainfall Depth, PMP

- The system performs under the default settings without overtopping for the 6-hour duration storm with the assumed initial water levels and pumping system characteristics.
- The peak water levels in the Phase 1 PIC, the Phase 2 PIC and the SWP are 4.6 m, 4.5 m and 5.3 m, respectively, during the 6-hour storm (Table 19). The water levels in the Phase 1 PIC and the SWP exceed the freeboard levels of these facilities (4.2 m and 4.9 m), whereas the water level in the Phase 2 PIC is at the freeboard level.
- Overtopping occurs at the Phase 1 PIC for storm durations of 12, 24 and 48 hours under the default settings.
- For the system to perform without overtopping during storms with durations of 12, 24 and 48 hours, the capacities of either the Phase 1 PIC or the SWP, or both facilities, and the capacities of Pumps 33 and 34 will need to be increased. The runoff volumes from the BRDA for the 12, 24 and 48-hour duration PMP are 267,350 m^3, 334,550 m^3 and 397,810 m^3, respectively, but the total available volume in the Phase 1 PIC, the Phase 2 PIC and the SWP is 267,910 m^3 above the summer inventory target of 180,000 m^3.

Table 19. Peak Discharges, Water Levels and Volumes – PMP

Rainfall Duration (hrs)	Phase 1 PIC			Phase 2 PIC			SWP		
	Peak Discharge (m^3/s)	Peak Water Level (m)	Peak Storage ($1,000m^3$)	Peak Discharge (m^3/s)	Peak Water Level (m)	Peak Storage ($1,000m^3$)	Peak Discharge (m^3/s)	Peak Water Level (m)	Peak Storage ($1,000m^3$)
6	0.436	4.52	115.25	0.670	4.58	70.19	0.157	5.26	197.05

6.0 CONCLUDING REMARKS

- Updated estimates of 1,000, 2,500, 5,000 and 10,000-yr rainfall depths corresponding to storm durations of 6, 12, 24 and 48 hours are provided based on rainfall depth-duration-frequency data for the site sourced from Met Eireann. These return periods correspond to those recommended for Inflow Design Floods in the Canadian Dam Association Guidelines 2014.
- Statistical estimates of the PMP for storm durations of 6, 12, 24 and 48 hours are provided. These have been determined using the procedures in WMO 2009 'Manual on Estimation of Probable Maximum Precipitation (PMP)', WMO-No. 1045. PMP values range from 144.7 mm for a 6 hours event to 255.9 mm for a 48 hours event.
- Updated estimates of the Inflow Design Floods to the PICs, which receive runoff from the BRDA, have been determined using hydrologic modelling. Peak flows from the Phase 1 BRDA for the 200-yr event vary from 0.649 m^3/s (6-hour storm duration) to 1.418 m^3/s (48-hour storm duration). Peak flows from the Phase 2 BRDA for the same event range from 0.334 m^3/s (6-hour storm duration) to 0.728 m^3/s (48-hour storm duration).
- Estimates of the PMF to the PICs have also been determined. Peak flows from the Phase 1 BRDA vary from 2.299 m^3/s (6-hour storm duration) to 4.173 m^3/s (48-hour storm duration). Peak flows from the Phase 2 BRDA for the same event range from 1.181 m^3/s (6-hour storm duration) to 2.148 m^3/s (48-hour storm duration).
- Based on the model results, the facilities will not be overtopped during 200-yr and 1,000-yr rainfall events with durations between 6 and 48 hours with the assumed initial water levels and pumping system characteristics.
- The facilities will also not be overtopped for rainfall depths for 1/3 between the 1,000-year rainfall depth and the PMP with durations of 6, 12 and 24 hours under the default settings.
- However, overtopping of the Phase 1 PIC will occur for the rainfall depth 1/3 between the 1,000-year rainfall and the PMP with a duration of 48 hours. For the Phase 1 PIC to perform without overtopping for this rainfall depth, the capacities of Pumps 33 and 34 which discharge to the SWP will need to be increased. A total pumping rate of approximately 2,300 m^3/hr will be required (1,150 m^3/hr for each pump).
- The facilities will not be overtopped during the 6-hour duration PMP event with the assumed initial water levels and pumping system characteristics.

- However, overtopping of the Phase 1 PIC will occur during PMP events with durations between 12 and 48 hours. For the system to perform without overtopping for PMP events with durations between 12 and 48 hours, the capacities of either the Phase 1 PIC or the SWP, or both facilities, and the capacities of Pumps 33 and 34 which discharge from the Phase 1 PIC to the SWP will need to be increased.

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APPENDIX J

BRDA Dome Closure Assessment

REPORT

Dome Water Management Infrastructure Design

Bauxite Residue Disposal Area - Closure Design at Stage 16

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Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

Golder Associates Ireland Limited (Golder) has been engaged by Aughinish Alumina Ltd. (AAL) to provide engineering design services for the closure of the proposed Phase 1 and 2 Bauxite Residue Disposal Area (BRDA) constructed to Stage 16.

AAL requires that a closure design for water management infrastructure to transfer runoff from the BRDA dome to the PICs. The design will also be used to inform visual assessments and support the strategic infrastructure development (SID) application for the proposed development of the BRDA to stage 16.

This report considers the management of runoff from the dome of the complete Phase 1 and 2 BRDA to Stage 16 (36 mOD) and the raised Salt Cake Cell (crest at 31.25 mOD) at closure, which is located in the east sector of the Phase 1 BRDA. This study also assumes that the dome capping containment works have been completed and that the side slope capping containment works from Stage 0 to Stage 16 have been completed at closure.

The scope of the services is to undertake the engineering design of water management infrastructure (i.e., channels and spillways / chutes) to transfer the Inflow Design Flood (IDF) from the BRDA dome to the perimeter interceptor channels (PICs), and includes the following tasks:

- **Task 1:** Geometric design of Phase 1 and 2 BRDA to Stage 16;
- **Task 2:** Hydrological assessment for peak flows during the IDF;
- **Task 3:** Hydraulic design of upper level (dome) drainage channels at Stage 16; and
- **Task 4:** Hydraulic design of spillways, chutes and/or energy dissipators required to transfer runoff from the BRDA to the PICs.

The objective of this report is to outline the methodologies and outcomes for these tasks.

Note: The design of the PIC's or the side slope drainage at closure, are not considered as part of this study.

2.0 BACKGROUND

AAL is wholly owned by United Company RUSAL and operates the alumina refinery situated on Aughinish Island on the south side of the Shannon estuary. AAL own a circa 601.22 ha. landholding (the Site) on Aughinish Island. The Island is predominantly rural in character with the remaining land usage comprising agriculture, single low density residential housing and protected habitats (wetlands and grasslands).

Aughinish Island is located on the south banks of the Shannon Estuary, at approximately 50km from the outlet to the North Atlantic, in the south-west of Ireland, and is bounded by the River Shannon to the north, the Robertstown River to the west and southwest and the Poulaweala creek to the east and southeast. The nearest towns are Askeaton (ca. 6.0 km to the east) and Foynes (ca. 3.5 km to the west) and the Site is located circa 30 km west of Limerick City.

The Phase 1 BRDA is located southwest of the process plant and is formed of two facilities: the original Phase 1 BRDA, which covers an area of 72 ha and the eastern Phase 1 BRDA Extension, which covers an area of 32 ha. The Phase 2 BRDA adjoins the southern extent of the Phase 1 BRDA and covers an area of 80 ha.

The BRDA is surrounded by PICs, which collect bleed water and runoff from the Phase 1 and Phase 2 facilities and currently convey it via pumps either to the Effluent Clarification System (ECS) or the Storm Water Pond (SWP). Both the ECS and the SWP are situated to the northeast of the Phase 1 BRDA. At closure, the PICs will be designed to discharge surface water off-site.

3.0 BRDA DOME PERIMETER DRAINAGE CHANNEL AND SPILLWAY ARRANGEMENT

3.1 Runoff Management Strategy and Spillway Layout

The surface water runoff management strategy for the dome of the Phase 1 and Phase 2 BRDA to Stage 16 (at closure) considers the following main principles:

- Runoff from the dome is intercepted by the dome perimeter drainage channels which convey the intercepted runoff to spillways. The overtopping elevation of the dome perimeter channels will be the crest elevation of the Stage 16 raise (i.e., 36 mOD);
- Seven proposed spillways are distributed along the perimeter of the BRDA dome. The spillways will convey runoff from the dome perimeter channels to the PICs;
- The proposed locations of the spillways have been selected based on the following criteria:
  - Catchment areas resulting in similar design flow rates;
  - Ease of construction (e.g., avoidance of spillways at bends in the PIC / stage raises);
  - Distribution of runoff across the different segments of the PIC system; and
  - Avoidance of spillways discharging close to the PIC culvert crossings which could impact the hydraulic performance of these culverts during an extreme storm event.

The design layout for the dome perimeter channels and spillways are presented in Drawing 01 (Appendix A).

- A further spillway and two perimeter drainage channels sections are proposed below the dome elevation to convey surface water runoff from the raised and capped Salt Cake Disposal Cell area to the adjacent section of PIC. The Salt Cake Disposal Cell is located to the east of the Phase 1 BRDA Stage 16 dome and is sited between the footprints for Stage 11 and Stage 12 in this sector and is proposed to be raised to a crest elevation of 31.25 mOD. The capped surface of this area at closure will comprise an upper-level area (surface elevation ranging from approximately 35.5 mOD to 33 mOD) and a lower-level area (surface elevation of approximately 26 mOD).

  The surface of the upper-level area will be graded to direct surface water runoff towards the upper spillway inlet to the south, with a bund constructed at the crest of capped slope to prevent runoff from discharging down the capped side slopes and divert it into the upper spillway inlet.

  The surface of the lower-level area will also be graded to direct surface water to the two sections of perimeter drainage channels and divert them to the lower spillway inlet.

3.2 Catchment Data

The catchments draining to each spillway are presented in Drawing 01 (Appendix A), and their main hydrological properties are provided in Table 1.

Times of concentration were estimated using the Watershed Lag Method, developed by the National Resources Conservation Service (NRCS, 2010). The selected curve number for the closure scenario corresponds with a pasture / grassland / range soil cover, on hydrologic soil group D, with fair hydrologic condition (NRCS, 1986).

Table 1: Spillway Catchment Properties

Catchment	Spillway	Area (ha)	Flow Length (m)	Curve Number	Average Land Slope (%)	Lag Time (min)	Time of Concentration (min)
C - 1	SP - 1	11.9	533	84	2.8	16	26
C - 2	SP - 2	9.1	534	84	3.1	15	25
C - 3	SP - 3	8.8	538	84	3.6	14	23
C - 4	SP - 4	8.6	493	84	2.9	14	24
C - 5	SP - 5	8.5	465	84	3.0	13	22
C - 6	SP - 6	10.1	645	84	2.8	19	32
C - 7	SP - 7	11.7	713	84	3.0	17	28
C - 8	SP - 8	4.9	336	84	3.0	10	17

4.0 PMP AND IDF CALCULATION

4.1 BRDA Classification

For the operational phase of the facility, the BRDA has been classified to have a “**High**” hazard potential classification (HPC) under the Canadian Dam Association (CDA) Guidelines (CDA, 2014) and therefore, the IDF during this phase is 1/3 between the 1,000-year and the Probable Maximum Flood (PMF).

For the closure phase of the facility, Golder considers that the BRDA classification would be reduced to a “**Significant**” HPC which also corresponds to an IDF 1/3 between the 1,000-year and the Probable Maximum Flood (PMF).

During 2019, SLR Consulting was retained by AAL to conduct an independent dam safety review (DSR) of the BRDA (SLR 2019). The DSR report also states that this reduced classification could be justified for closure.

4.2 PMP Assessment

During 2019, Golder undertook an analysis of the Probable Maximum Precipitation (PMP) and IDF for the BRDA for a range of rainfall event durations ranging from 6 to 48 hours; these analyses were used to inform an assessment of the performance and capacities of the PICs and the BRDA Storm Water Pond (SWP) (Golder, 2019). However, given the short times of concentrations for the dome catchments, an updated PMP / IDF assessment was undertaken for shorter duration storm events, which is required for the assessment of design runoff rates appropriate for the spillway and dome perimeter channel designs.

The times of concentration of the eight delineated catchments reporting to the spillways range between 17 and 32 minutes. Golder has selected a critical storm duration of 30 minutes for the assessment of design runoff rates, and therefore an assessment of the PMP and IDF for a storm duration of 30 minutes has been undertaken. The selection of the assumed rainfall duration is discussed further in Section 4.4.

Statistical estimates of the PMP for 1-hour, 6-hour and 24-hour events were determined using the procedures described in ‘Manual on Estimation of Probable Maximum Precipitation (PMP)’ (WMO, 2009). From these estimates, the 30-minute event PMP was estimated by extrapolation assuming a power trend.

Thirty-one years (1989 to 2019) of hourly rainfall data from the climate station at Shannon Airport were used in the analysis (Met Éireann, 2020). The 1-hour, 6-hour and 24-hour PMP estimates are presented in Figure 2 along with the extrapolated 30-minute PMP estimate.

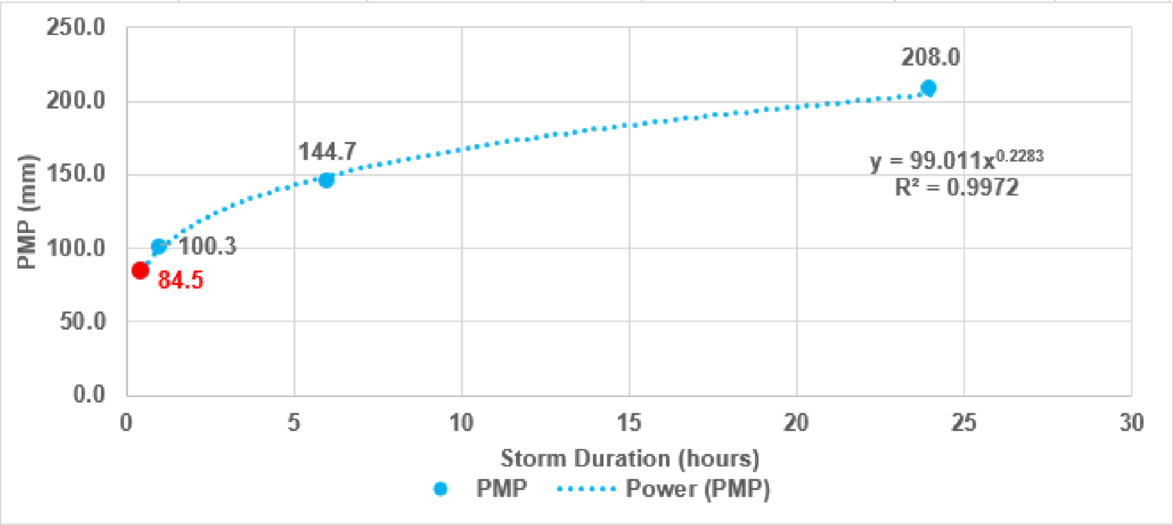

Figure 1: Estimated PMP Depths

4.3 1,000-Year Rainfall Depth Assessment

To estimate the 1 in 1,000-year event precipitation, rainfall depth-duration-frequency (DDF) data was downloaded from the Met Eireann website for the location of the BRDA (Met Éireann, 2019). Rainfall depths were provided for durations ranging from 5 minutes to 25 days and for return periods ranging between 6 months and 500 years. The rainfall depth for a duration of 30 minutes and a return period of 1,000 years was extrapolated from the available data as presented in Figure 2 and Table 2.

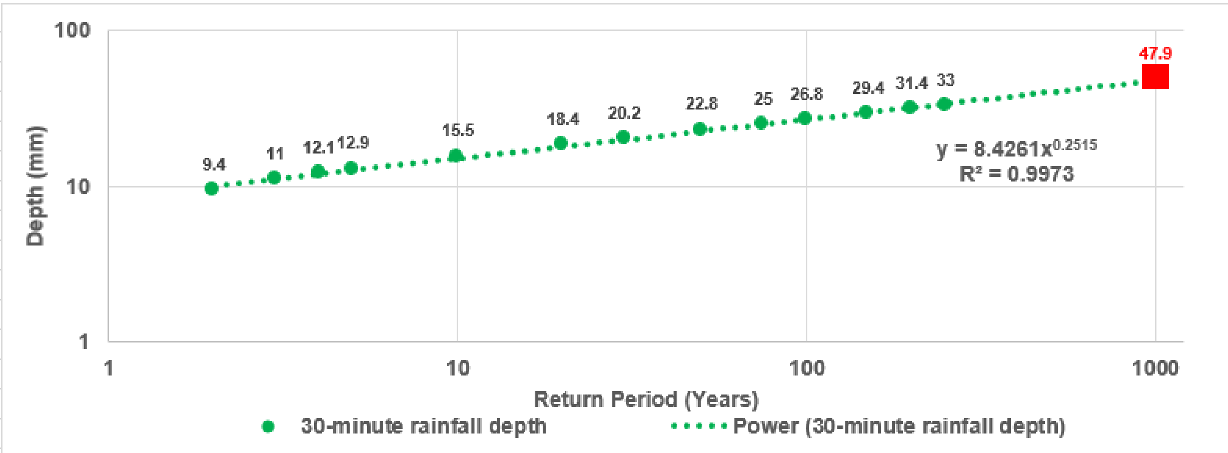

Figure 2: 30-minute Rainfall Depth-Frequency Data

Table 2: 30-minute Rainfall Depth-Frequency Data (mm)

Return Period (Years)													
2	3	4	5	10	20	30	50	75	100	150	200	250	1,000
9.4	11.0	12.1	12.9	15.5	18.4	20.2	22.8	25.0	26.8	29.4	31.4	33.0	**47.9**

4.4 IDF Assessment

As outlined in Section 4.1, the IDF corresponding to a BRDA HPC of “Significant” is 1/3 between the 1,000-year event and the Probable Maximum Flood (PMF). The IDF is the peak runoff rate generated during the 30-minute rainfall depth estimated as 1/3 between the 1,000-year and PMP rainfall depths, as presented in Table 3 below.

Table 3: Estimated Design Rainfall Depths

Rainfall Duration (min)	Rainfall Depths (mm)		
	1,000-year	PMP	IDF (1/3 between 1,000-year and PMP)
30	47.9	84.5	**60.1**

Peak flows for the IDF were calculated by applying the Rational Method to each catchment. The determination of rainfall intensities for this method assumes that the rainfall duration is equal to the time of concentration of the catchment.

As outlined in Sections 3.2 and 4.2, the time of concentration of the catchments are close to 30 minutes. Therefore, rainfall intensities based on a critical storm duration of 30 minutes have been used for the estimation of IDF peak flows. The rainfall depth that is 1/3 between the 1,000-year and the PMP for a 30-minute duration event is 60.1 mm, which corresponds to a rainfall intensity of 120.2 mm/hr.

A runoff coefficient of 0.6 was assumed for each catchment, which is considered representative of Pasture/Range/Meadow terrain types with average slopes i.e., 2% to 7%, during high return period rainfall events (Chow et al, 1988).

The IDF peak flow calculation for each catchment is presented in Table 4.

Table 4: IDF Peak Flows Calculation (Rational Method)

Catchment	Spillway	Area (ha)	Runoff Coefficient	Rainfall Duration (min)	Rainfall Depth (mm)	Rainfall Intensity (mm/hr)	Estimated Peak Flow (m^3/s)
C - 1	SP - 1	11.9	0.6	30	60.1	120.2	2.39
C - 2	SP - 2	9.1	0.6	30	60.1	120.2	1.83
C - 3	SP - 3	8.8	0.6	30	60.1	120.2	1.76
C - 4	SP - 4	8.6	0.6	30	60.1	120.2	1.73
C - 5	SP - 5	8.5	0.6	30	60.1	120.2	1.70
C - 6	SP - 6	10.1	0.6	30	60.1	120.2	2.02
C - 7	SP - 7	11.7	0.6	30	60.1	120.2	2.35
C - 8	SP - 8	4.9	0.6	30	60.1	120.2	0.97

5.0 HYDRAULIC DESIGN OF THE DOME PERIMETER CHANNELS

5.1 Dome Perimeter Channel Layout and Design Flow Rates

The overall dome perimeter channel comprises a series of sixteen (16) perimeter drainage channel segments have been designed to intercept surface water runoff from the BRDA dome and convey the runoff to the eight (8) spillways. Each spillway conveys the runoff carried by two (2) dome perimeter drainage channel segments that merge at the spillway inlet. The layout designs for the sixteen (16) dome perimeter drainage channel segments are presented in Drawing 01 (Appendix A) and design details are presented in Drawings 02 to 04, along with spillway design details.

The catchment areas and corresponding design flow rates for each dome perimeter drainage channel segment are presented in Table 5. These design flow rates have been calculated in accordance with the approach described in Section 4.4.

Table 5: Dome Perimeter Channel Segment Catchment Data

Downstream Spillway	Channel	Area (ha)	Design Flow Rate (m^3/s)
SP-1	CHSP 1 – 1	7.4	1.48
	CHSP 1 – 2	4.5	0.91
SP-2	CHSP 2 – 1	5.0	1.00
	CHSP 2 – 2	4.1	0.83
SP-3	CHSP 3 – 1	3.0	0.61
	CHSP 3 – 2	5.8	1.15
SP-4	CHSP 4 – 1	3.4	0.68
	CHSP 4 – 2	5.2	1.05
SP-5	CHSP 5 – 1	6.0	1.20
	CHSP 5 – 2	2.5	0.50
SP-6	CHSP 6 – 1	4.9	0.99
	CHSP 6 – 2	5.1	1.03
SP-7	CHSP 7 – 1	2.8	0.57
	CHSP 7 – 2	8.9	1.78
SP-8	CHSP 8 – 1(1)	0.2	0.04
	CHSP 8 – 2(1)	1.4	0.28

Notes:

1. *The total Salt Cake Disposal Cell catchment area draining to Spillway SP-8 is 4.9 ha. However, the upper elevation Salt Cake Disposal Cell area drains directly to the upper section spillway and hence the sum of the catchment areas draining to these lower perimeter drainage channels is less than 4.9 ha.*

5.2 Dome Perimeter Drainage Channel Segment Geometry

The dome perimeter drainage channel segments have been designed with a trapezoidal cross section and 1.5(H):1(V) side slopes and will be located adjacent to the upstream face of the Stage 16 raise, and to the Stage 11 raise for the perimeter drainage channel segments discharging to SP-8, with an overtopping elevation equal to the crest elevation of the stage raise. The channels have been designed with a mild longitudinal slope of 0.13 % i.e., 750(H): 1(V), to ensure sub-critical flow conditions and low flow velocities along these channels.

Further geometric channel design information is provided on Drawings 02 to 04 (Appendix B) and in Table 6 below.

Table 6: Dome Perimeter Drainage Channel Segments - Geometric Design Properties

Channel	Length (m)	Bottom Width (m)	Overtopping Elevation (m OD)	Downstream Invert Level (m OD)	Upstream Invert Level (m OD)
CHSP 1 – 1	261	2.0	36	35	35.35
CHSP 1 – 2	163	2.0	36	35	35.22
CHSP 2 – 1	256	2.0	36	35	35.34
CHSP 2 – 2	307	2.0	36	35	35.41
CHSP 3 – 1	280	2.0	36	35	35.37
CHSP 3 – 2	266	2.0	36	35	35.36
CHSP 4 – 1	192	2.0	36	35	35.26
CHSP 4 – 2	225	2.0	36	35	35.30
CHSP 5 – 1	195	2.0	36	35	35.26
CHSP 5 – 2	97	2.0	36	35	35.13
CHSP 6 – 1	286	2.0	36	35	35.38
CHSP 6 – 2	396	2.0	36	35	35.53
CHSP 7 – 1	183	2.0	36	35	35.25
CHSP 7 – 2	447	2.5	36	35	35.60
CHSP 8 – 1	9	1.0	26	25.5	25.51
CHSP 8 – 2	99	1.0	26	25.5	25.63

5.3 Dome Perimeter Drainage Channel Segments Lining Design

The design of the dome perimeter drainage channel segments includes a ‘Concrete Canvas’ liner to protect from erosion of the underlying bauxite residue, prevent contamination of the water management system by bauxite residue leaching or seepage and to provide protection from accidental and UV damage during its operational life. Concrete Canvas (‘CC’) is a Geosynthetic Cementitious Composite Mat (GCCM) that is commonly used for channel lining applications. CC consists of a 3-dimensional fibre matrix containing a specially formulated dry concrete mix; a PVC backing ensures the material has a low permeability (Concrete Canvas, 2020a). A summary of key properties of the CC liner considered in the design are summarised below (Concrete Canvas, 2020b and 2020c):

- Manning’s roughness coefficient = 0.011;
- Low level of permeability similar to compacted clay liner (hydraulic conductivity = $1x10^{-8}$ m/s); and
- Durable product with a minimum BBA certified design life of 120 years;

The CC liner is available in three thicknesses (5 mm, 8 mm and 13 mm) depending on the intended use of the liner and site-specific design considerations. The 13 mm thickness CC liner (CC13™) is proposed for the dome perimeter drainage channel segment lining, to be consistent with the CC liner thickness selected for the spillway lining design (see Section 6.3.1).

5.4 Dome Perimeter Drainage Channel Segment Hydraulic Analysis

A hydraulic model of each perimeter drainage channel segment pair (discharging to a single spillway) and spillway confluence was developed using the United States Army Corps of Engineers Hydrologic Engineering Center’s River Analysis System (HEC-RAS) software (USACE, 2020). One-dimensional steady-state flow analysis with a mixed flow regime was completed to assess the design flow depths, flow velocities and the flow regimes (i.e., subcritical or supercritical) along the dome perimeter drainage channels.

The channel confluences at each spillway were included in the models to ensure that the potential effects of these junctions on the flow regime within the channels were considered (e.g., backwater effects, reduced velocities, change in flow regime). The momentum equation was used to compute the energy losses through the junctions within the models. A Manning’s roughness coefficient of 0.011 was applied for the channels which is representative of the roughness characteristics for the designed channel liner. The model steady flow boundary conditions were the Normal Depth (S = 0.00133) at the upstream ends of the channels and critical depth at the downstream end of the model (downstream extent of the models extended down the spillway to a point where critical depth is achieved).

The dome perimeter drainage channel segments were designed and assessed against the following hydraulic design criteria:

- No overtopping (Freeboard ≥ 0 m);
- Sub-critical flow regime (Froude No < 1) maintained along the entire length of the channels; and
- Flow velocities < 5 m/s.

<u>Note:</u> *A 0m freeboard has been selected as the structures are not intended to store water during normal conditions. The structures have been assessed for the IDF event and do not overtop for this event. Overtopping of either the dome perimeter channel or the spillways will result in the water entering the side-slope rock fill drainage system and it is not considered to be significant issue.*

The hydraulic modelling results demonstrate that the designed channels meet these design criteria. Table 7 below presents a summary of the results for each channel, and further detailed (tabulated) model outputs are provided in Appendix B.

Table 7: Dome Perimeter Channel Segments - Hydraulic Analysis Results Summary

Channel	Design Flow (m^3/s)	Depth (m)		Velocity (m/s)		Froude No		Freeboard (m)	
		Min	Max	Min	Max	Min	Max	Min	Max
CHSP 1 – 1	1.48	0.39	0.48	1.13	1.44	0.58	0.81	0.26	0.52
CHSP 1 – 2	0.91	0.32	0.48	0.69	1.12	0.36	0.68	0.46	0.52
CHSP 2 – 1	1.00	0.31	0.44	0.86	1.28	0.47	0.80	0.34	0.56
CHSP 2 – 2	0.83	0.28	0.44	0.72	1.22	0.39	0.79	0.31	0.56
CHSP 3 – 1	0.61	0.24	0.42	0.55	1.09	0.30	0.77	0.39	0.58
CHSP 3 – 2	1.15	0.34	0.42	1.04	1.36	0.57	0.81	0.31	0.58
CHSP 4 – 1	0.68	0.26	0.42	0.61	1.09	0.34	0.73	0.48	0.58
CHSP 4 – 2	1.05	0.32	0.42	0.95	1.30	0.52	0.80	0.38	0.58
CHSP 5 – 1	1.20	0.35	0.41	1.12	1.36	0.62	0.80	0.39	0.59
CHSP 5 – 2	0.50	0.29	0.41	0.47	0.70	0.26	0.45	0.58	0.59
CHSP 6 – 1	0.99	0.31	0.44	0.84	1.28	0.45	0.80	0.31	0.56
CHSP 6 – 2	1.03	0.31	0.44	0.87	1.31	0.47	0.81	0.15	0.56
CHSP 7 – 1	0.57	0.26	0.45	0.48	0.94	0.25	0.64	0.50	0.55
CHSP 7 – 2	1.78	0.38	0.45	1.25	1.50	0.66	0.84	0.02	0.55
CHSP 8 – 1	0.04	0.28	0.29	0.1	0.10	0.06	0.07	0.21	0.21
CHSP 8 – 2	0.28	0.24	0.31	0.61	0.87	0.4	0.64	0.13	0.20

6.0 HYDRAULIC DESIGN OF THE SPILLWAYS

6.1 Spillway Layout and Design Flow Rates

As described in Section 3.0, seven spillways are proposed, distributed along the perimeter of the BRDA dome to convey surface water runoff from the dome perimeter drainage channels to the PICs. A further spillway is located to the east of the BRDA dome to convey runoff from the capped Salt Cake Disposal Cell area to the adjacent PIC. The layout of these spillways is presented in Drawing 01 (Appendix A) and design details are presented in Drawings 02 to 07. The catchment areas and corresponding design flow rates for each spillway have been presented in Table 4 (Section 4.4 above).

6.2 Spillway Geometry

The spillways have been designed as trapezoidal chutes with side slopes of 2.5(H):1(V). The spillway profiles will match the BRDA closure side slope, with local longitudinal slopes typically ranging from 3.5(H):1(V) (spillway profile between adjacent stage raises) to flat (spillway sections along stage raises). The overall longitudinal slope for the spillway will correspond to the overall side-slope of the BRDA (Drawing 06 and Drawing 07). The design depth and the bottom widths of the spillways vary based on the design flow rates, and three spillway design arrangements have been developed, as follows:

- **8m base width spillways:** The 8m base width spillway design arrangement is applied to the three spillways with the largest design flow rates draining the BRDA dome (SP-1, SP-6 and SP-7). These spillways are designed to have a depth of 1m.
- **6m base width spillways:** The 6m base width spillway design arrangement is applied to the four remaining spillways draining the BRDA dome (SP-2 to SP-5). These spillways are designed to have a depth of 1m.
- **4m base width spillway:** The 4m base width spillway design arrangement is applied to spillway SP-8 which drains the capped Salt Cake Cell area. This spillway has a considerably lower design flow rate compared to the BRDA dome spillways and is designed to have a depth of 0.5m.

The spillways will be lined with riprap on the base, which is placed over the Concrete Canvas underlayer lining. The Concrete Canvas provides a similar function and benefits to those listed for the Dome Perimeter Channel (see Section 5.3). The spillway base widths were designed with consideration for the required riprap rock sizes, to achieve a rock gradation that is not difficult to source or place and to achieve a consistent riprap sizing specification that can be applied to all spillways (see Section 6.3 for further information).

Design details for these spillway arrangements are presented in Drawings 02 to 05 (Appendix A), and long section profiles are presented in Drawings 06 and 07. The overall gradient of the spillways matches that of the BRDA and is approx. 6.8(H):1(V).

The long section profiles for the spillways draining the BRDA dome are similar with the exception of spillway SP-5 which has short (1m) horizontal sections at Stage raises 6, 7 and 8. To ensure that the hydraulic design criteria (Section 6.4.1) are met and a hydraulic jump is achieved at these stage raises, a 0.1 m deep riprap basin (formed of additional riprap fill) has been incorporated in the SP-5 spillway design at Stages 6 to 8; this detail is presented on Drawing 07.

6.3 Spillway Lining Design

The spillway chutes have been designed incorporating an angular rock riprap lining layer to dissipate a portion of the design flow energy and to reduce design flow velocities.

Riprap lining is commonly used to protect underlaying soil surfaces from erosion; however, given the underlying bauxite residue may be highly susceptible to erosion an additional ‘Concrete Canvas’ (‘CC’) lining layer has been incorporated beneath the riprap lining to provide additional erosion protection.

6.3.1 Spillway Concrete Canvas Lining

A general description of the CC liner product has been provided in Section 5.3. The spillway design considers a CC lining thickness of 13 mm (CC13™) which represents the thickest currently available proprietary CC liner product. The CC13™ liner generally has superior mechanical performance properties compared to the thinner CC products that are available and is suitable for applications including where trafficking is required; where there are high velocity or turbulent flow conditions; or for conditions subject to design requirements for impact or dynamic loads (Concrete Canvas, 2020c and 2020d).

Therefore, given an angular rock riprap lining layer will be installed along the spillways over the concrete canvas layer, and the expected turbulent flow conditions during the IDF, the CC13™ product has been selected for the design of the spillways.

6.3.2 Spillway Riprap Lining

The spillway riprap lining has been designed in accordance with guidance developed by Robinson et. al. (1998), which provides a methodology for sizing the median or D_{50} rock size for chutes based on the chute slope and unit flow rate (flow rate per unit width). The study also provides a methodology for estimating the Manning's roughness coefficient of the rock chute based on the D_{50} and chute slope.

These calculations were undertaken for each spillway considering a 3.5(H):1(V) spillway slope (i.e. steep portion of BRDA closure side slope) including a Factor of Safety (FoS) for the D_{50} rock size of 1.4. As discussed in Section 6.2, the spillway base widths (and therefore the spillway unit flow rate) were selected to achieve a design rock size that is not overly large, and to achieve a consistent riprap gradation specification that can be applied to all spillways.

The resulting spillway riprap sizing design requirements and estimated Manning's roughness coefficient are as follows:

- Riprap material = angular rock.
- D_{50} (median) riprap rock size = 250 mm.
- Riprap layer thickness (2 x D_{50}) = 500 mm.
- Manning's roughness coefficient = 0.055.

The riprap gradation requirement was assessed in accordance with the United States Department of Agriculture Natural Resources Conservation Service design procedures for rock-lined chutes (USDA, 2018), and are presented in Table 8.

Table 8: Riprap Gradation Requirements

Passing by Weight (%)	Lower Envelope Gradation (mm)	Upper Envelope Gradation (mm)
100	375	500
85	325	450
50	250	375
10	200	325

6.4 Spillway Hydraulic Analysis

6.4.1 Hydraulic Analysis and Results

A hydraulic model of each spillway was developed using HEC-RAS software (USACE, 2020). A one-dimensional steady-state flow analysis with mixed flow regime was completed to assess the design flow depths, flow velocities and the flow regimes (i.e., subcritical or supercritical) along the spillways.

A Manning's roughness coefficient of 0.055 was calculated for the spillways (see Section 6.3.2) and applied in the models. The model steady flow boundary conditions were 'Critical Depth' at the upstream end of the model and 'Known Water Surface Elevation' (assumed water level within the adjacent PIC) at the downstream end of the model.

The Spillways were designed and assessed against the following hydraulic design criteria:

- No overtopping (Freeboard > 0 m);
- Hydraulic jump achieved at each stage raise (flat sections of spillway); i.e. change in flow regime from super-critical (Froude no. > 1) to sub-critical (Froude no. < 1); and
- Flow velocities < 5 m/s.

The hydraulic modelling results demonstrated that the designed spillways meet these design criteria.

Table 9 presents a summary of the hydraulic analysis results for each channel, and further detailed (tabulated) model outputs are provided in Appendix B.

Table 9: Spillways - Hydraulic Analysis Results Summary

Spillway	Design Flow (m^3/s)	Depth (m)		Velocity (m/s)		Froude No		Freeboard (m)	
		Min	Max	Min	Max	Min	Max	Min	Max
SP – 1	2.39	0.08	0.41	0.65	3.71	0.34	4.27	0.59	0.92
SP – 2	1.83	0.10	0.46	0.55	2.94	0.28	3.03	0.54	0.90
SP – 3	1.76	0.08	0.48	0.51	3.93	0.25	4.73	0.52	0.92
SP – 4	1.73	0.07	0.53	0.45	4.14	0.21	5.15	0.47	0.93
SP – 5	1.70	0.09	0.48	0.50	2.95	0.25	3.16	0.52	0.91
SP – 6	2.02	0.06	0.44	0.51	3.75	0.26	4.71	0.56	0.94
SP – 7	2.35	0.09	0.47	0.55	3.84	0.27	4.54	0.53	0.91
SP – 8	0.97	0.08	0.40	0.17	2.78	0.06	3.16	0.10	0.42

7.0 CLOSING REMARKS

Golder has completed engineering designs for water management infrastructure to transfer runoff from the BRDA dome to the PICs during closure. The designs will also be used to inform visual assessments and support the strategic infrastructure development (SID) application for the proposed development of the BRDA to stage 16 and the level of design detail / analysis undertaken is considered appropriate for these purposes.

Further design detailing and design information would be required to progress these designs to ‘tender’ or ‘construction’ stage designs. At that point further detailed design optimisation or value engineering of the system could be considered.

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Signature Page

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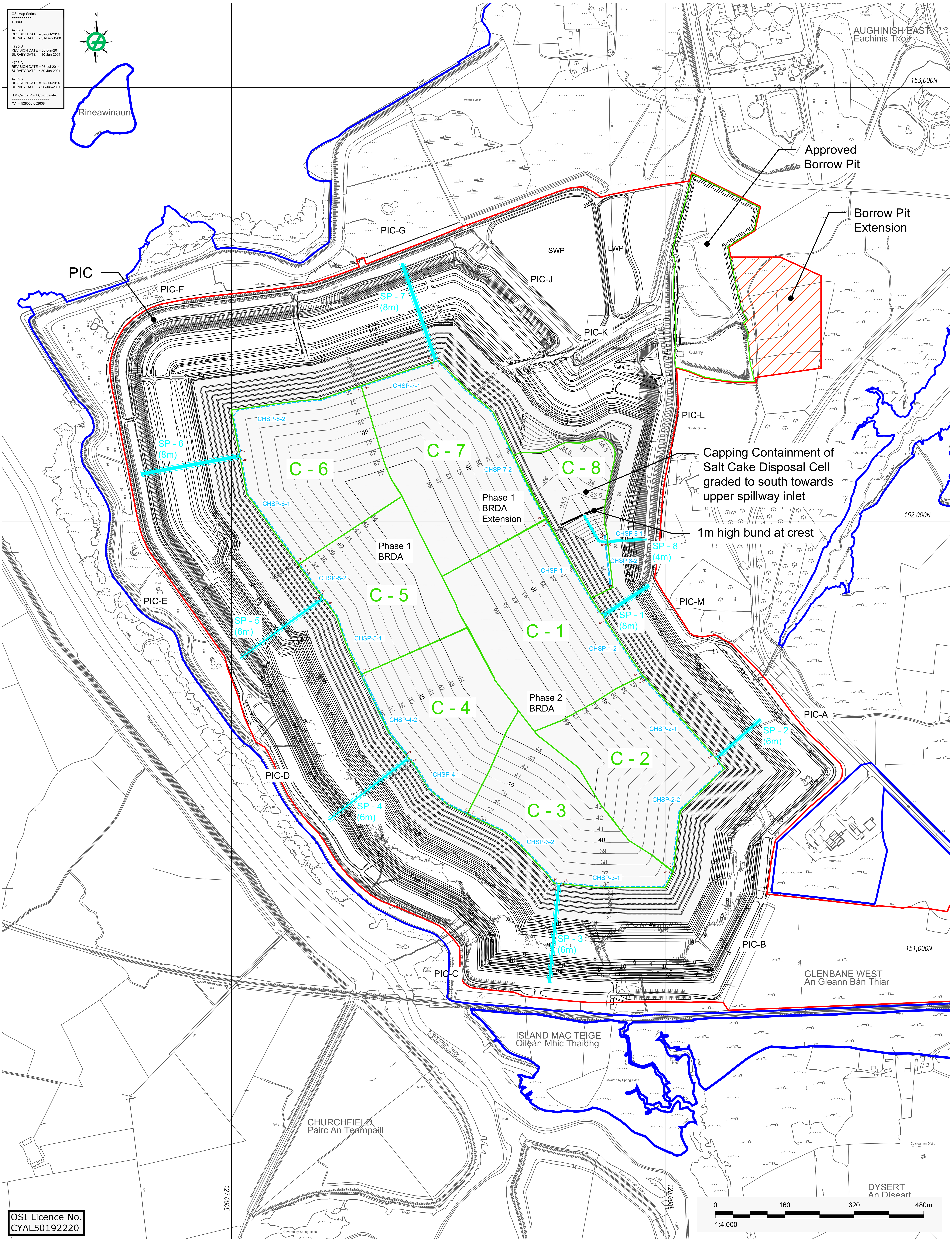
APPENDIX A

Drawings



LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- SPILLWAY LOCATION
- DOME PERIMETER CHANNEL
- CATCHMENT BOUNDARY
- 22 OGL CONTOUR (mOD)
- APPROVED BORROW PIT
- PROPOSED BORROW PIT EXTENSION

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES & TO O.S. DATUM.

DIMENSIONS ARE IN METRES.

CLIENT	AUGHINISH ALUMINA LTD.
PROJECT	BRDA CLOSURE DESIGN AT STAGE 16
CONSULTANT	GOLDER MEMBER OF WSP
YYYY-MM-DD	2021-Aug-11
PREPARED	POB
DESIGN	GA/DH
REVIEW	CC
APPROVED	BK/DH
TITLE	BRDA DOME, CATCHMENTS AND SPILLWAY LOCATIONS
PROJECT No.	20143063
DRAWING No.	01
Rev.	A
SCALE	1:4,000 A1

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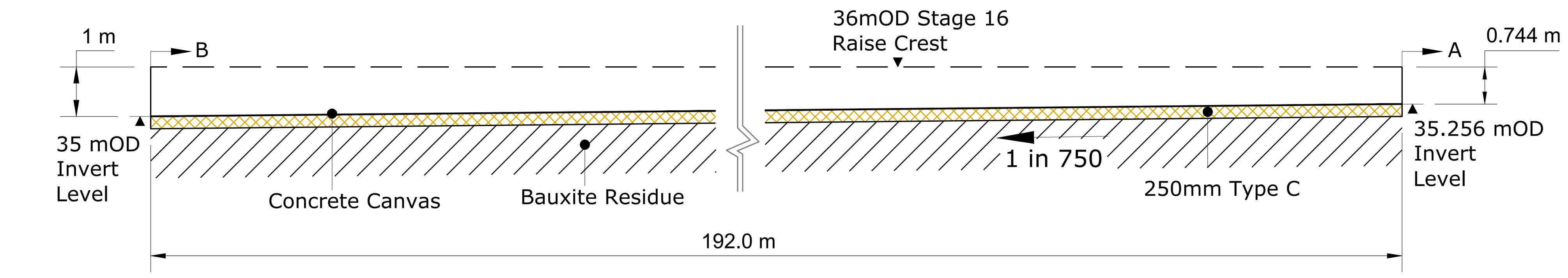
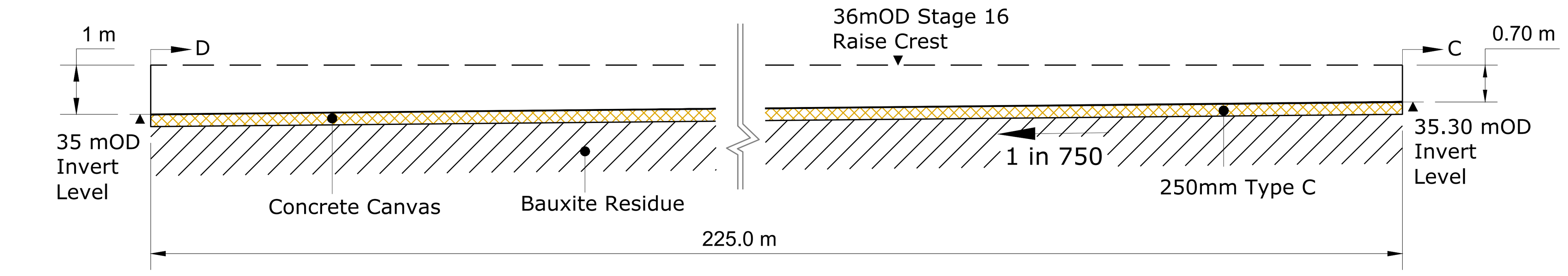


DOME PERIMETER CHANNEL PROFILE CHSP-4-1: SEE TABLE FOR OTHER CHANNEL INVERTS AND LENGTHS
(Scale - 1:100 @ A1)



DOME PERIMETER CHANNEL PROFILE CHSP-4-2: SEE TABLE FOR OTHER CHANNEL INVERTS AND LENGTHS
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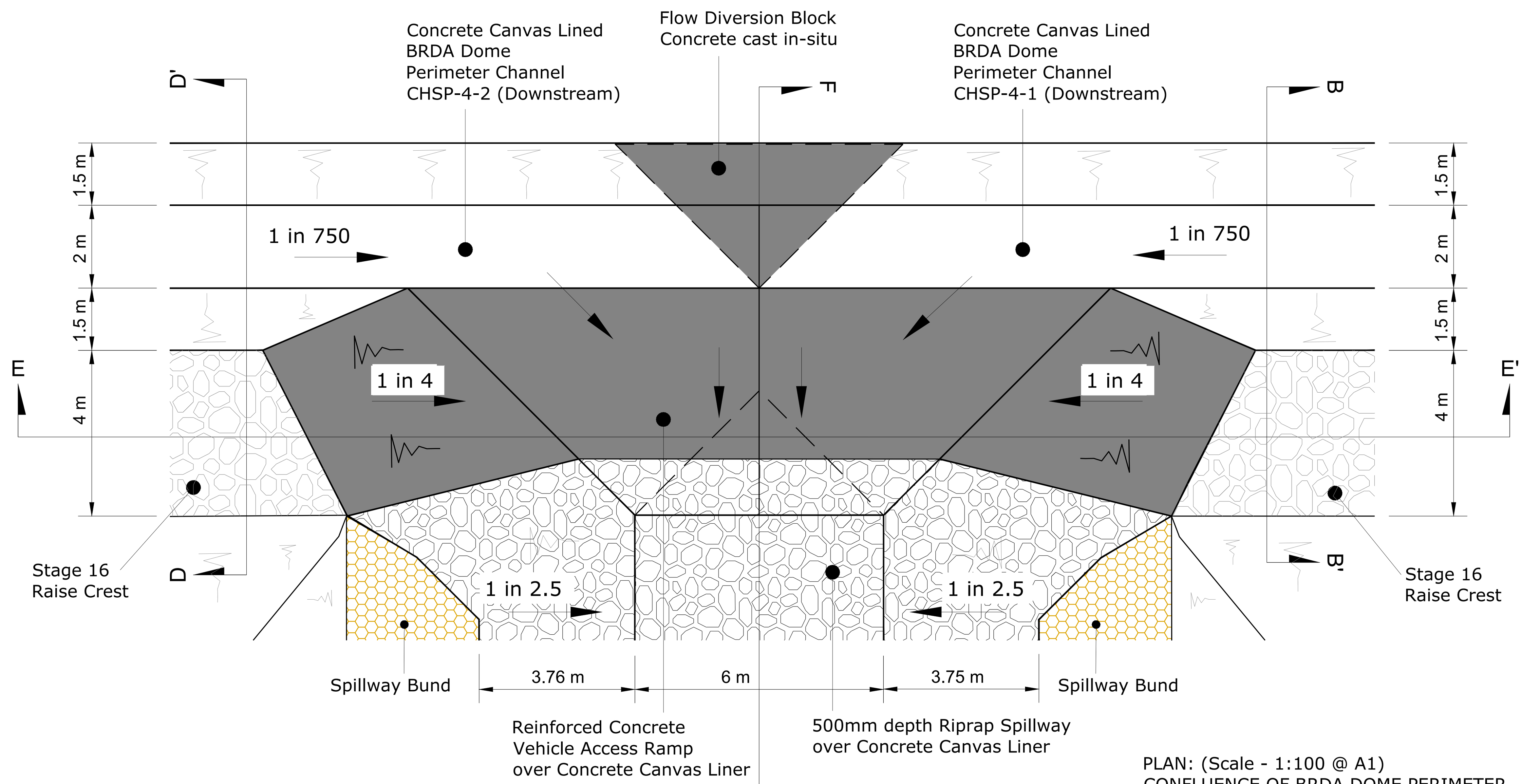
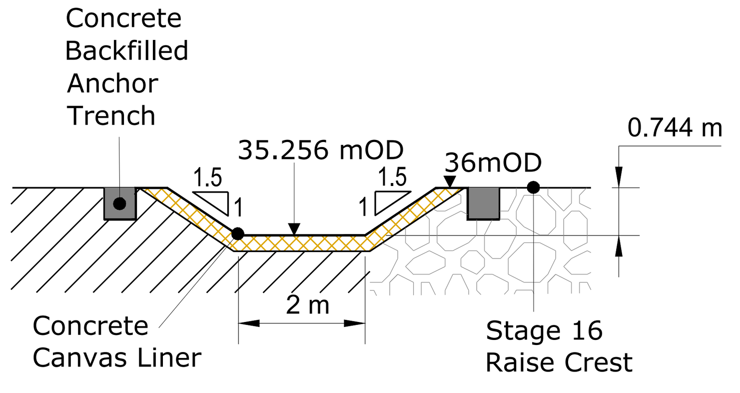
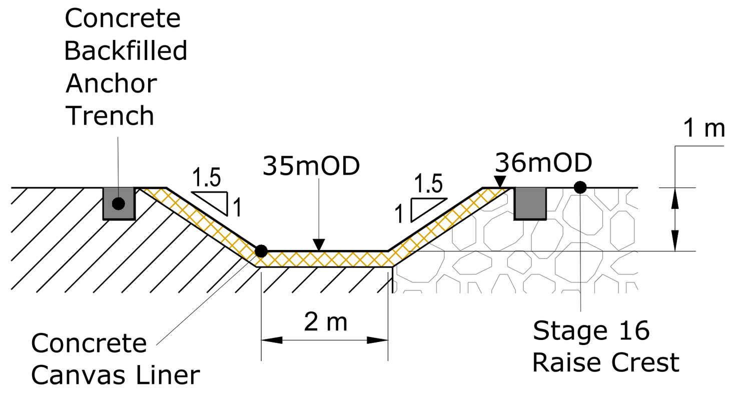


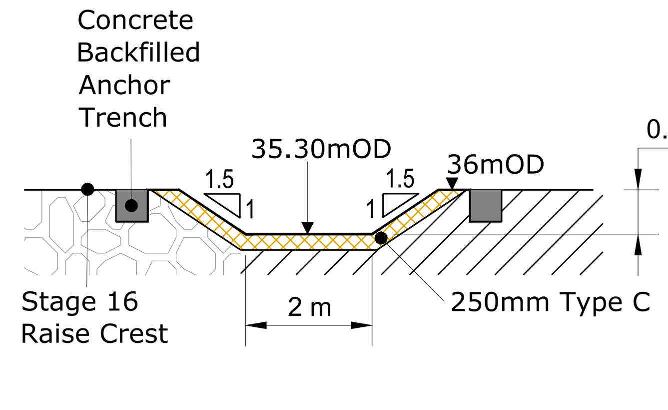
PLAN: (Scale - 1:100 @ A1)
CONFLUENCE OF BRDA DOME PERIMETER CHANNELS & RIPRAP SPILLWAY SP-4



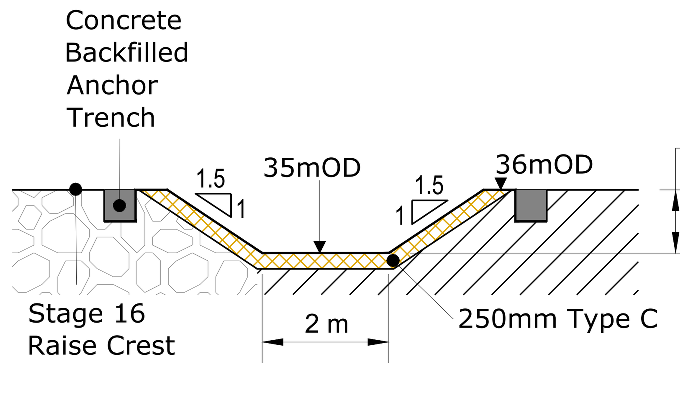
CROSS SECTION A:A' - CHSP-4-1 Upstream:
(Scale - 1:100 @ A1)



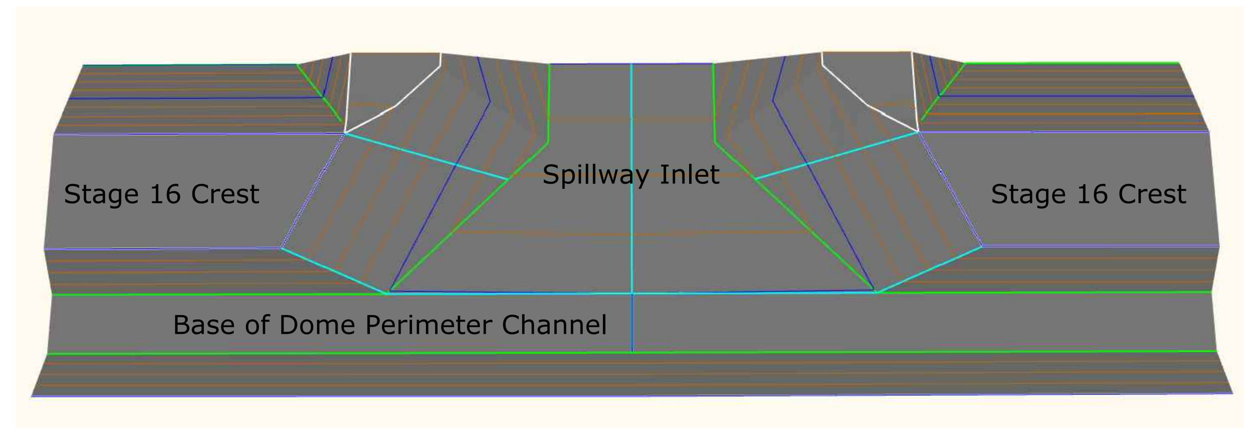
CROSS SECTION B:B' - CHSP-4-1 Downstream:
(Scale - 1:100 @ A1)



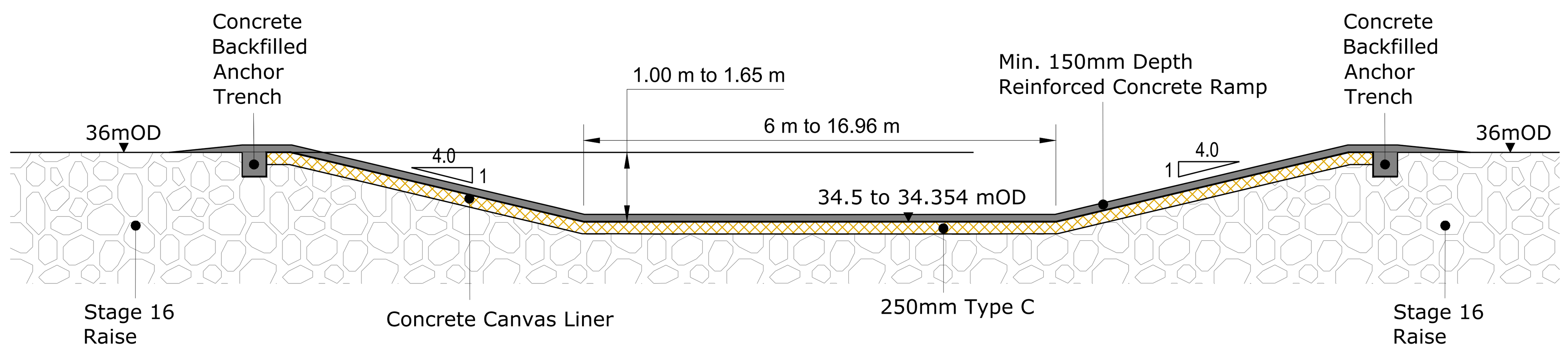
CROSS SECTION C:C' - CHSP-4-2 Upstream:
(Scale - 1:100 @ A1)



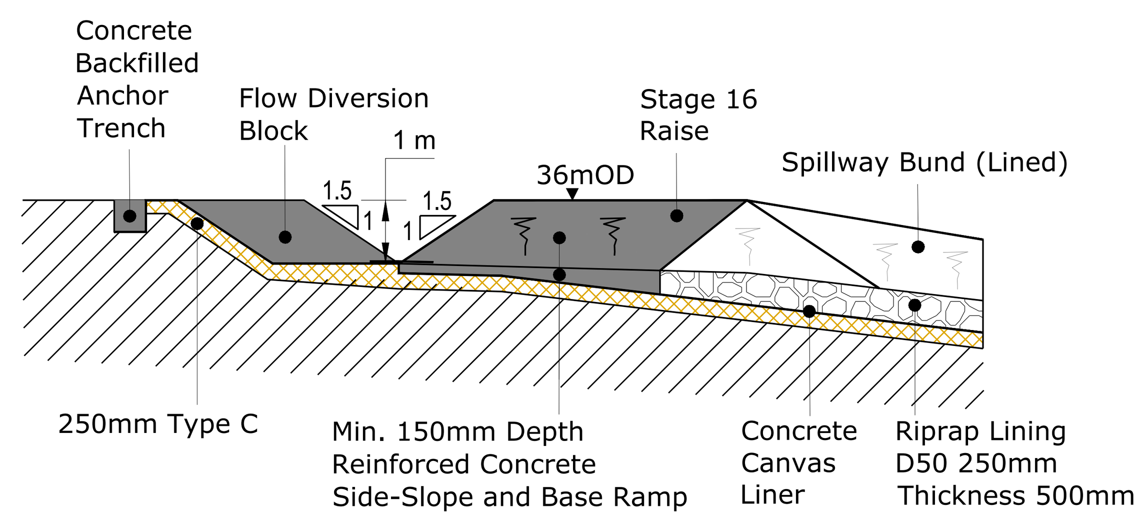
CROSS SECTION D:D' - CHSP-4-2 Downstream:
(Scale - 1:100 @ A1)



Simplifed Spillway Confluence - Liner Formation 3D Model
(NTS @ A1)

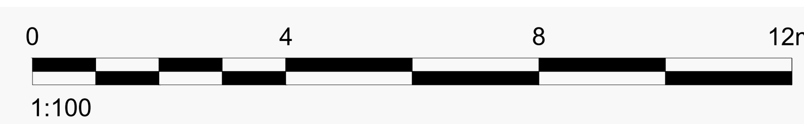


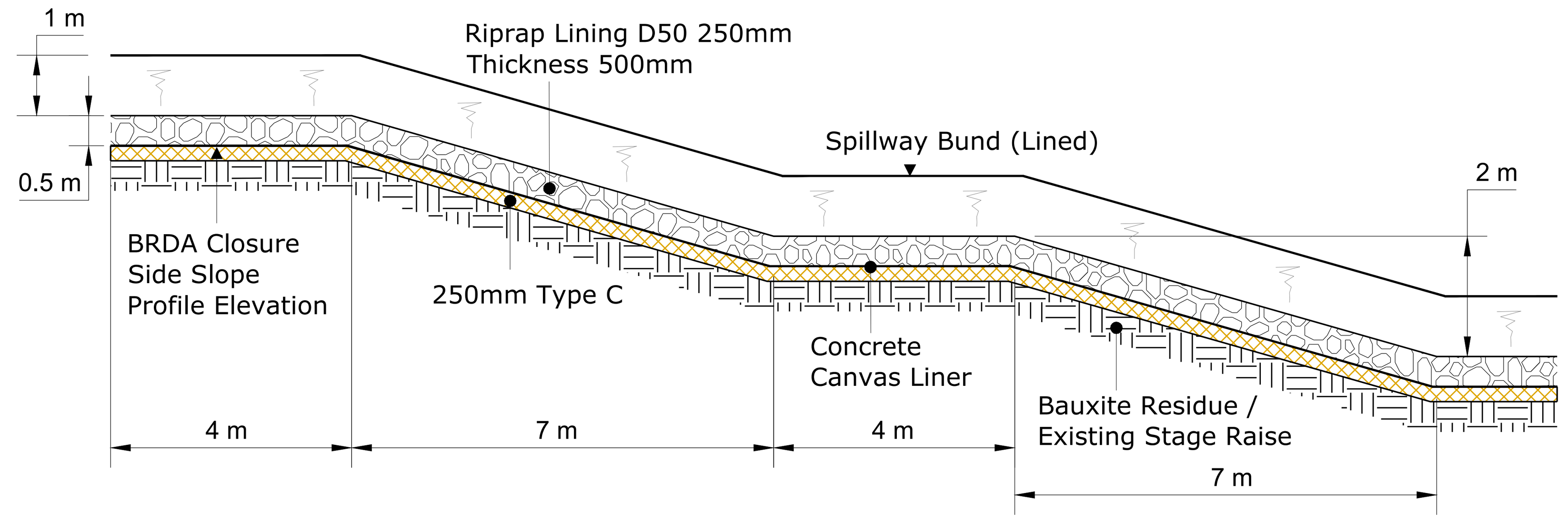
CROSS SECTION E:E' - Spillway Crest SP-4: SEE TABLE FOR OTHER CHANNEL INVERTS AND LENGTHS
(Scale - 1:100 @ A1)



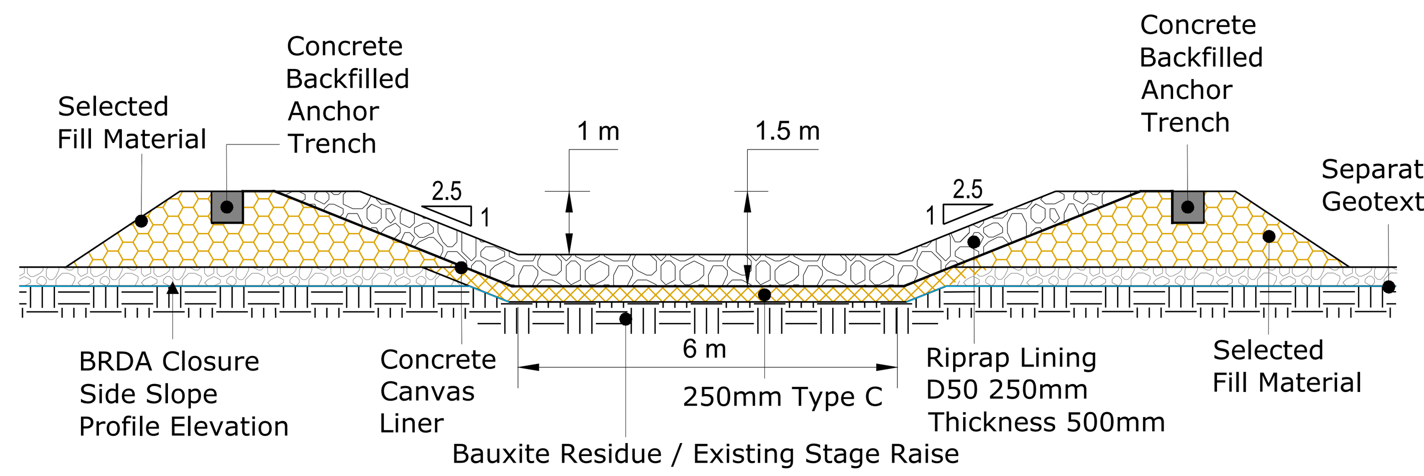
CROSS SECTION F:F' - Spillway Crest SP-4:
(Scale - 1:100 @ A1)

Simplifed Spillway Confluence - Liner Formation 3D Model
(NTS @ A1)





RIPRAP SPILLWAY SP-4 - Typical Profile:
(Scale - 1:100 @ A1)



RIPRAP SPILLWAY SP-4 - Typical Cross Section:
(Scale - 1:100 @ A1)

Spillway	Dome Perimeter Channel	Channel Length (m)	Channel Upstream Invert (mOD)
SP2	CHSP-2-1	256	35.342
	CHSP-2-2	307	35.409
SP3	CHSP-3-1	280	35.374
	CHSP-3-2	266	35.355
SP4	CHSP-4-1	192	35.256
	CHSP-4-2	225	35.300
SP5	CHSP-5-1	195	35.260
	CHSP-5-2	97	35.130

NOTE(S)

1. RIPRAP SPILLWAY TYPICAL PROFILE DIMENSIONS MAY VARY DEPENDING ON LOCATION ALONG SPILLWAY PROFILE
2. ANCHOR TRENCHES FOR THE DOME PERIMETER CHANNEL SHALL BE BACKFILLED WITH CONCRETE
3. ANCHOR TRENCHES FOR THE SPILLWAY SHALL BE BACKFILLED WITH ARISING FROM THE EXCAVATION OF THE SELECT FILL MATERIAL FOR THE BUND
4. DOUBLE LAPPING OF CONCRETE CANVAS MAY BE REQUIRED AT INTERFACE OF DOME PERIMETER CHANNEL AND REINFORCED CONCRETE SLAB CROSSING (SEE SECTION E-E' AND F-F')

CLIENT
AUGHINISH ALUMINA LTD.

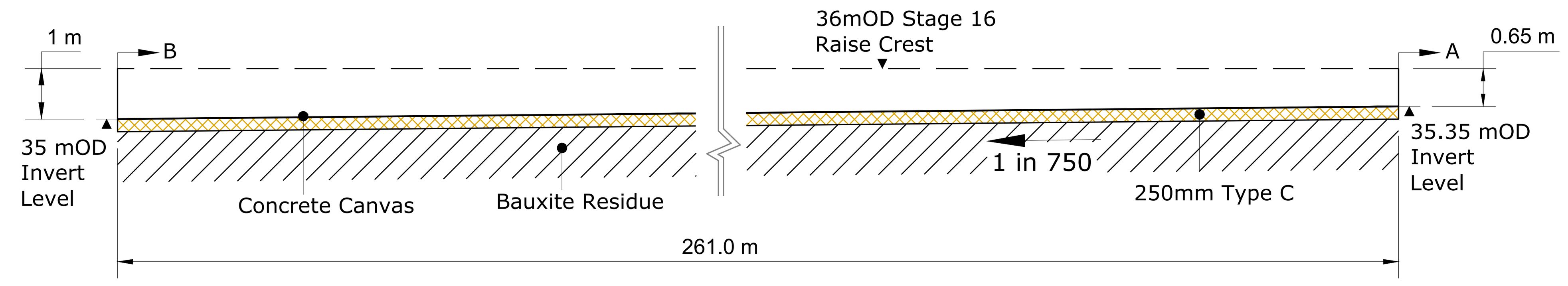
CONSULTANT
GOLDER

YYYY-MM-DD	2021-Aug-11
PREPARED	POB
DESIGN	GA
REVIEW	BK
APPROVED	GA/DH

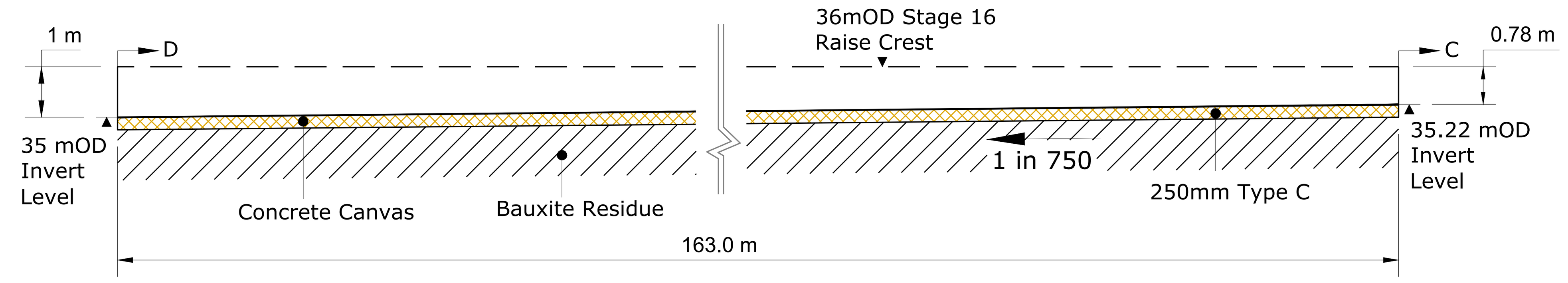
PROJECT
BRDA CLOSURE DESIGN

TITLE
6m WIDTH SPILLWAY DETAILS FOR SP-4 ANALOGOUS FOR SP-2, SP-3 AND SP-5

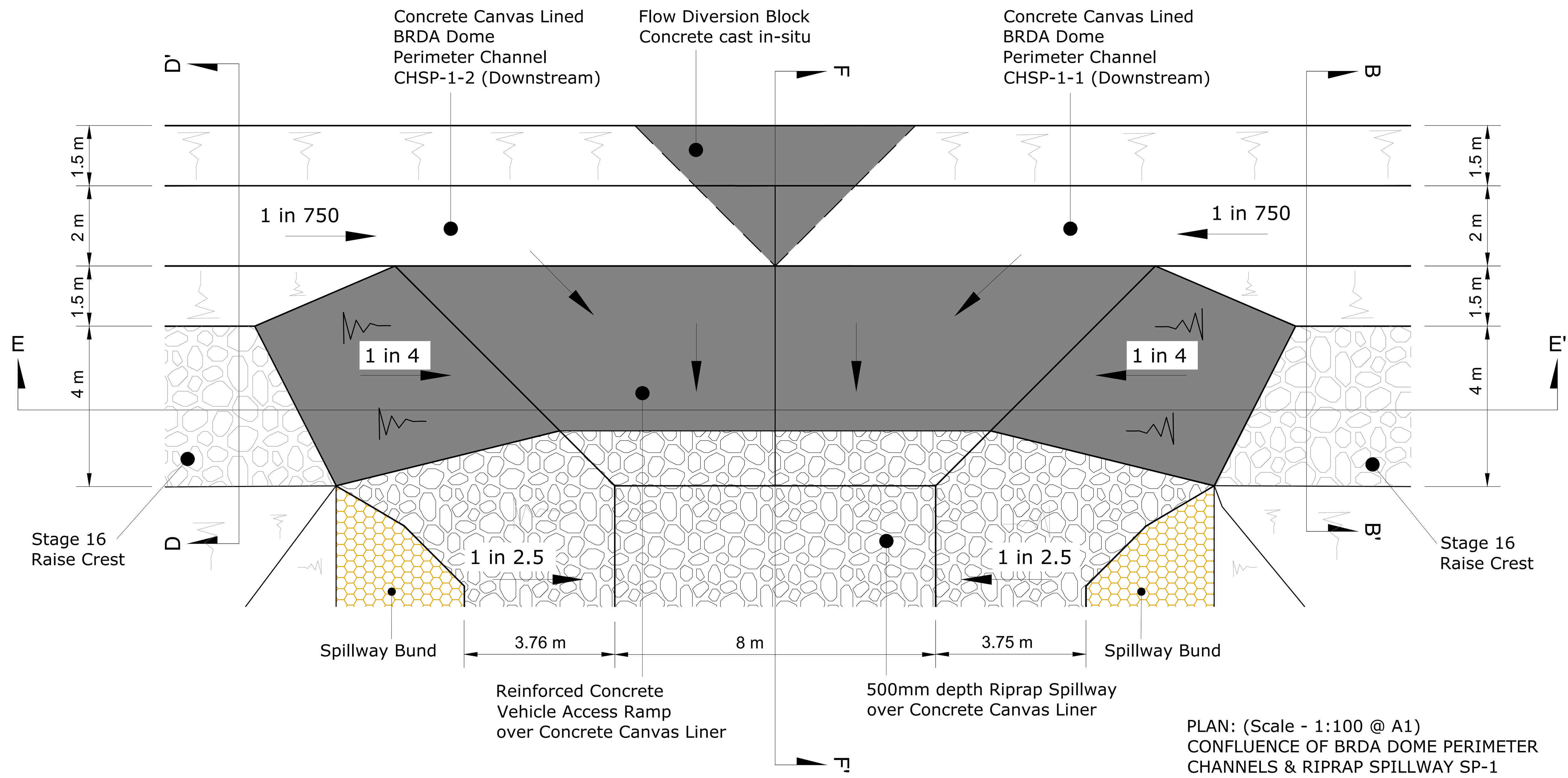
PROJECT No.	DRAWING No.	Rev.	SCALE
20143063	02	A	1:100 A1



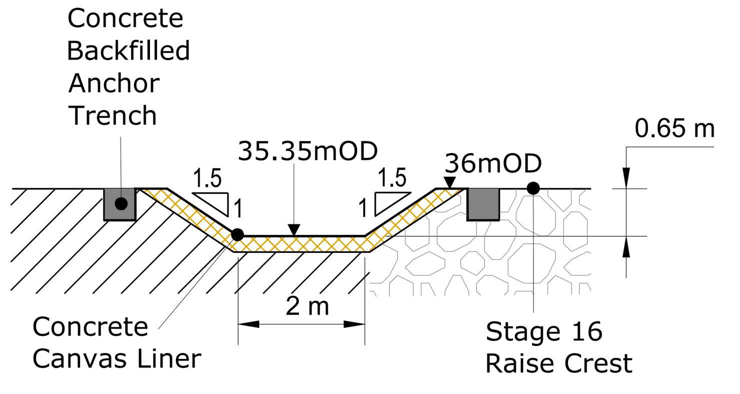
DOME PERIMETER CHANNEL PROFILE CHSP-1-1: SEE TABLE FOR OTHER CHANNEL INVERTS AND LENGTHS
(Scale - 1:100 @ A1)



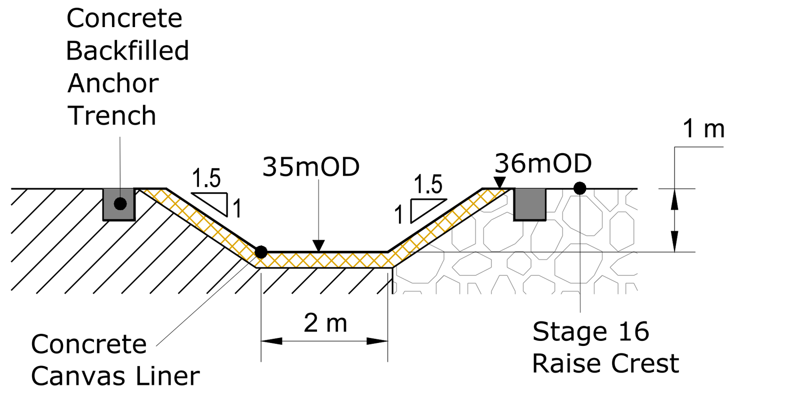
DOME PERIMETER CHANNEL PROFILE CHSP-1-2: SEE TABLE FOR OTHER CHANNEL INVERTS AND LENGTHS
(Scale - 1:100 @ A1)



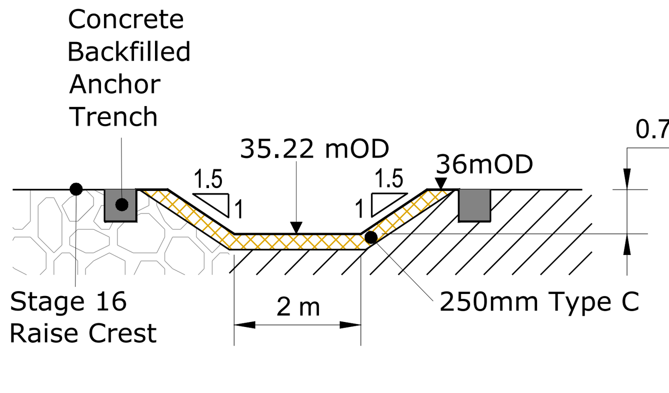
PLAN: (Scale - 1:100 @ A1)
CONFLUENCE OF BRDA DOME PERIMETER
CHANNELS & RIPRAP SPILLWAY SP-1



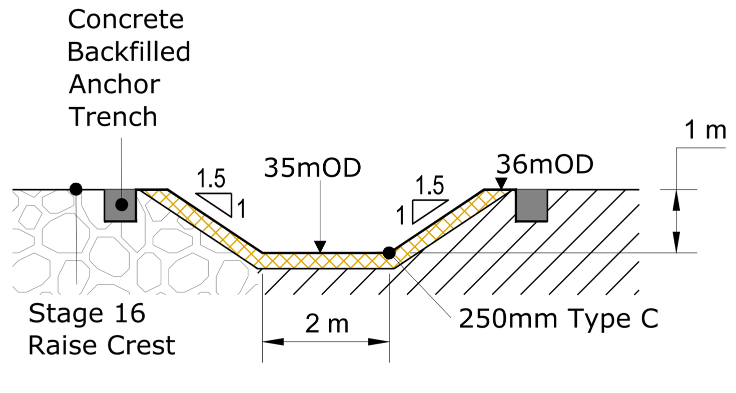
CROSS SECTION A:A' - CHSP-1-1 Upstream:
(Scale - 1:100 @ A1)



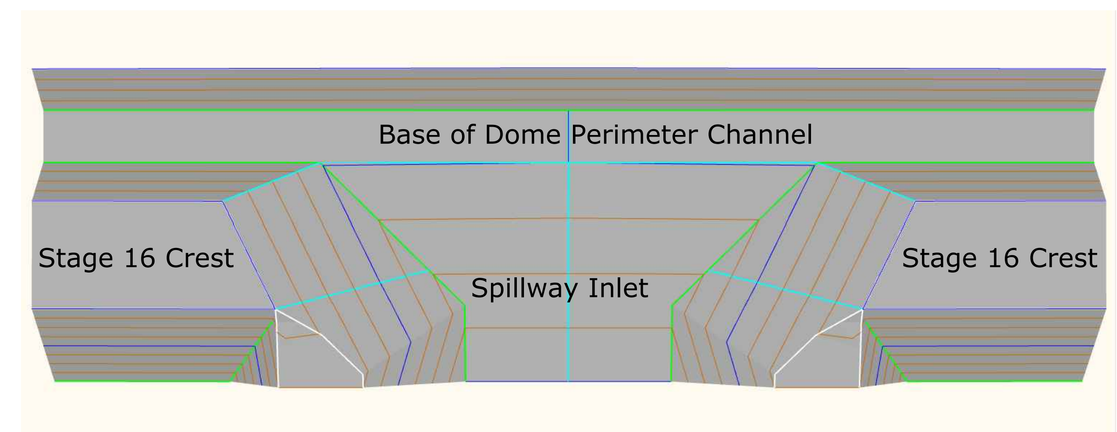
CROSS SECTION B:B' - CHSP-1-1 Downstream:
(Scale - 1:100 @ A1)



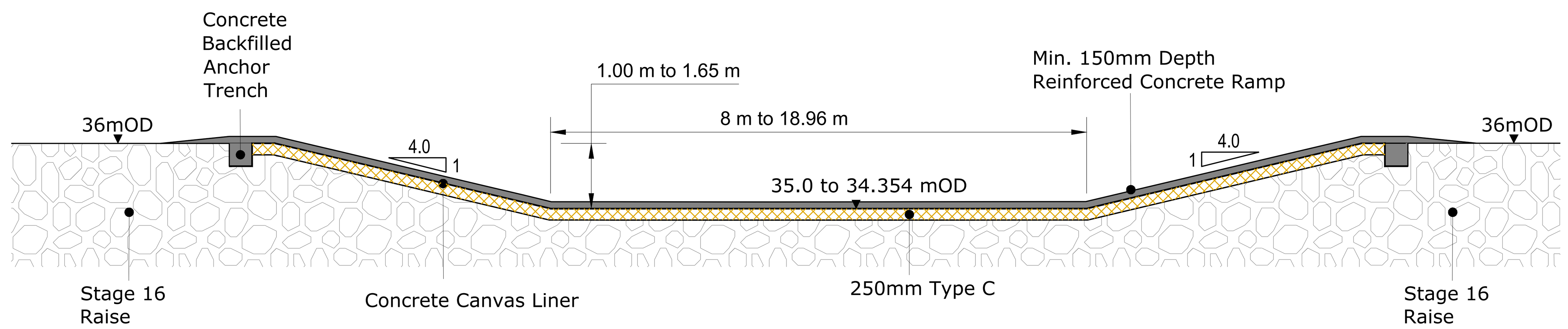
CROSS SECTION C:C' - CHSP-1-2 Upstream:
(Scale - 1:100 @ A1)



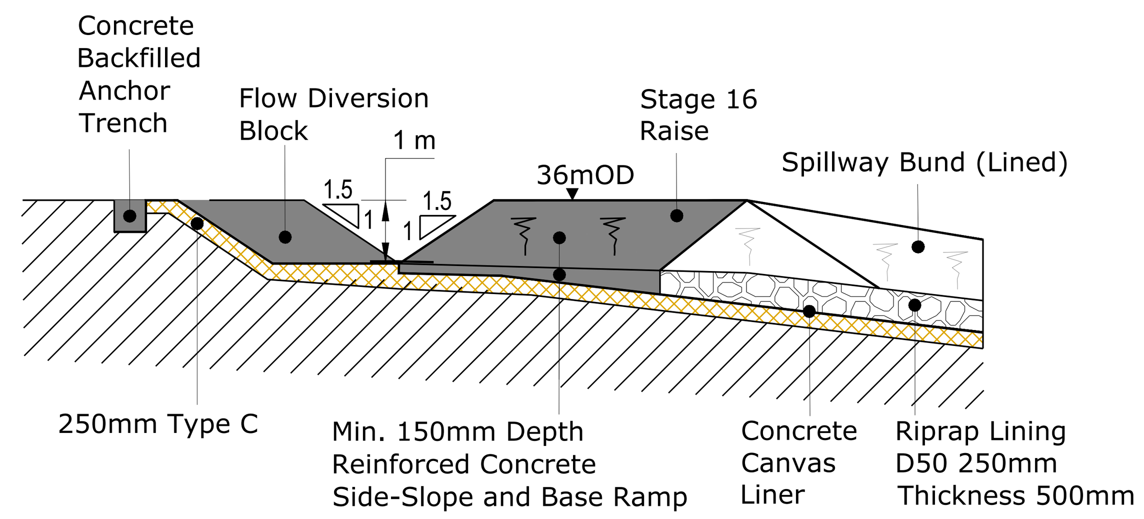
CROSS SECTION D:D' - CHSP-1-2 Downstream:
(Scale - 1:100 @ A1)



Simplified Spillway Confluence - Liner Formation 3D Model

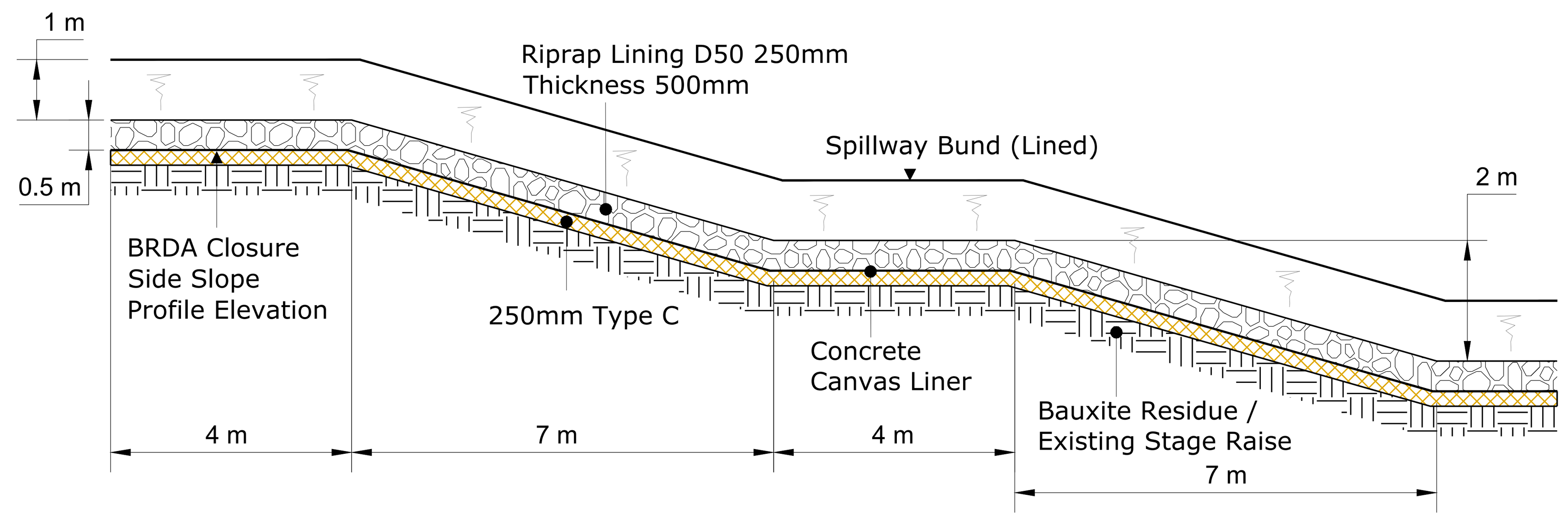


CROSS SECTION E:E' - Spillway Crest SP-1:
(Scale - 1:100 @ A1)

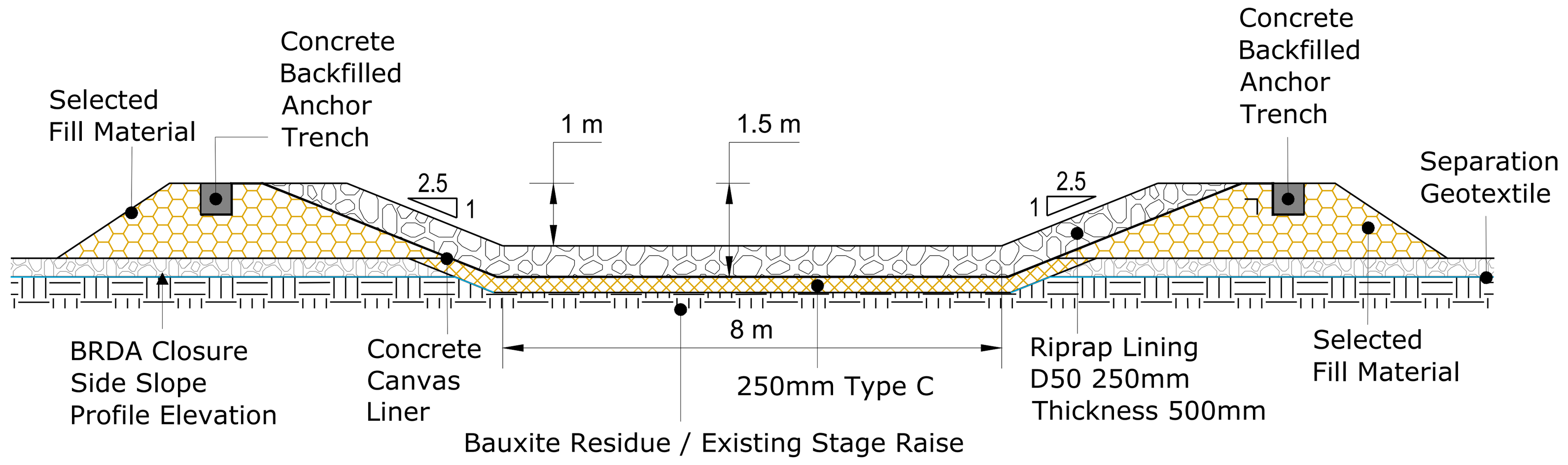


CROSS SECTION F:F' - Spillway Crest SP-1:
(Scale - 1:100 @ A1)

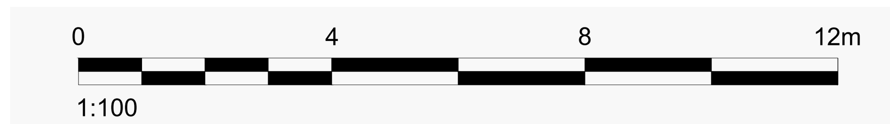
Simplified Spillway Confluence - Liner Formation 3D Model
(NTS @ A1)



RIPRAP SPILLWAY SP-1 - Typical Profile:
(Scale - 1:100 @ A1)



RIPRAP SPILLWAY SP-1 - Typical Cross Section:



Spillway	Dome Perimeter Channel	Channel Length (m)	Channel Upstream Invert (mOD)
SP1	CHSP-1-1	261	35.348
	CHSP-1-2	163	35.217
SP6	CHSP-6-1	286	35.381
	CHSP-6-2	396	35.528
SP7	CHSP-7-1	183	35.245
	CHSP-7-2	447	35.597

NOTE(S)

1. RIPRAP SPILLWAY TYPICAL PROFILE DIMENSIONS MAY VARY DEPENDING ON LOCATION ALONG SPILLWAY PROFILE
2. ANCHOR TRENCHES FOR THE DOME PERIMETER CHANNEL SHALL BE BACKFILLED WITH CONCRETE
3. ANCHOR TRENCHES FOR THE SPILLWAY SHALL BE BACKFILLED WITH ARISING FROM THE EXCAVATION OF THE SELECT FILL MATERIAL FOR THE BUND
4. DOUBLE LAPPING OF CONCRETE CANVAS MAY BE REQUIRED AT INTERFACE OF DOME PERIMETER CHANNEL AND REINFORCED CONCRETE SLAB CROSSING (SEE SECTION E-E' AND F-F')
5. THE DESIGNED BASE WIDTH OF DOME PERIMETER CHANNEL CHSP-7-2 IS 2.5m AS OPPOSED TO THE 2.0m INDICATED ON THE TYPICAL CROSS SECTIONS; ALL OTHER DOME PERIMETER CHANNELS HAVE DESIGNED BASE WIDTHS OF 2.0m, AS PER THE TYPICAL CROSS SECTIONS

CLIENT
AUGHINISH ALUMINA LTD.

CONSULTANT
GOLDER

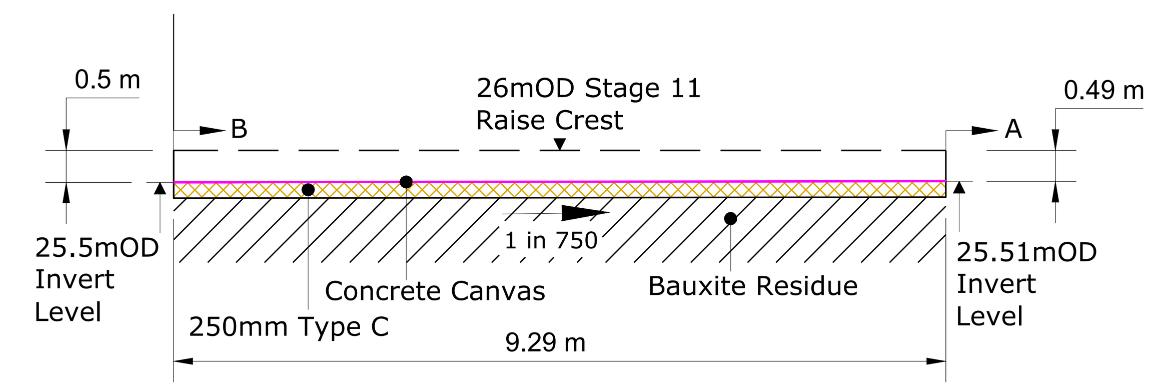
YYYY-MM-DD	2021-Aug-11
PREPARED	POB
DESIGN	GA
REVIEW	BK
APPROVED	GA/DH

PROJECT
BRDA CLOSURE DESIGN

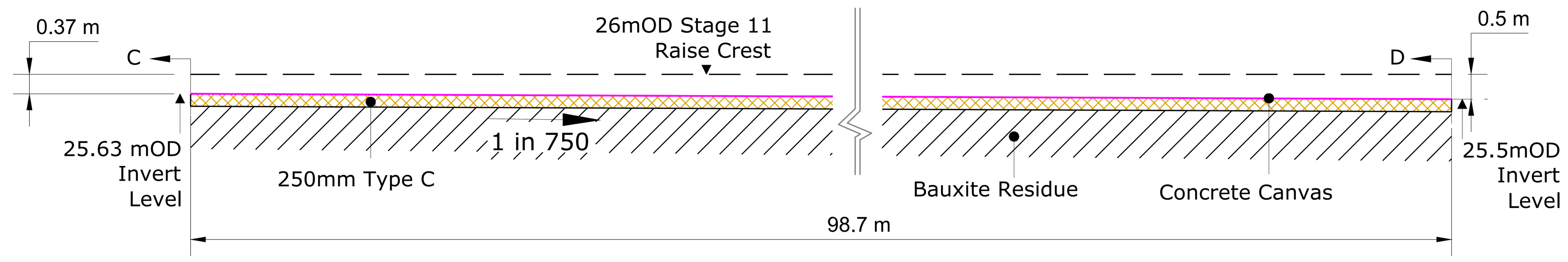
TITLE
8m WIDTH SPILLWAY FOR SP-1
ANALOGOUS FOR SP-6 AND SP-7

PROJECT No.	DRAWING No.	Rev.	SCALE
20143063	03	A	1:100 A1

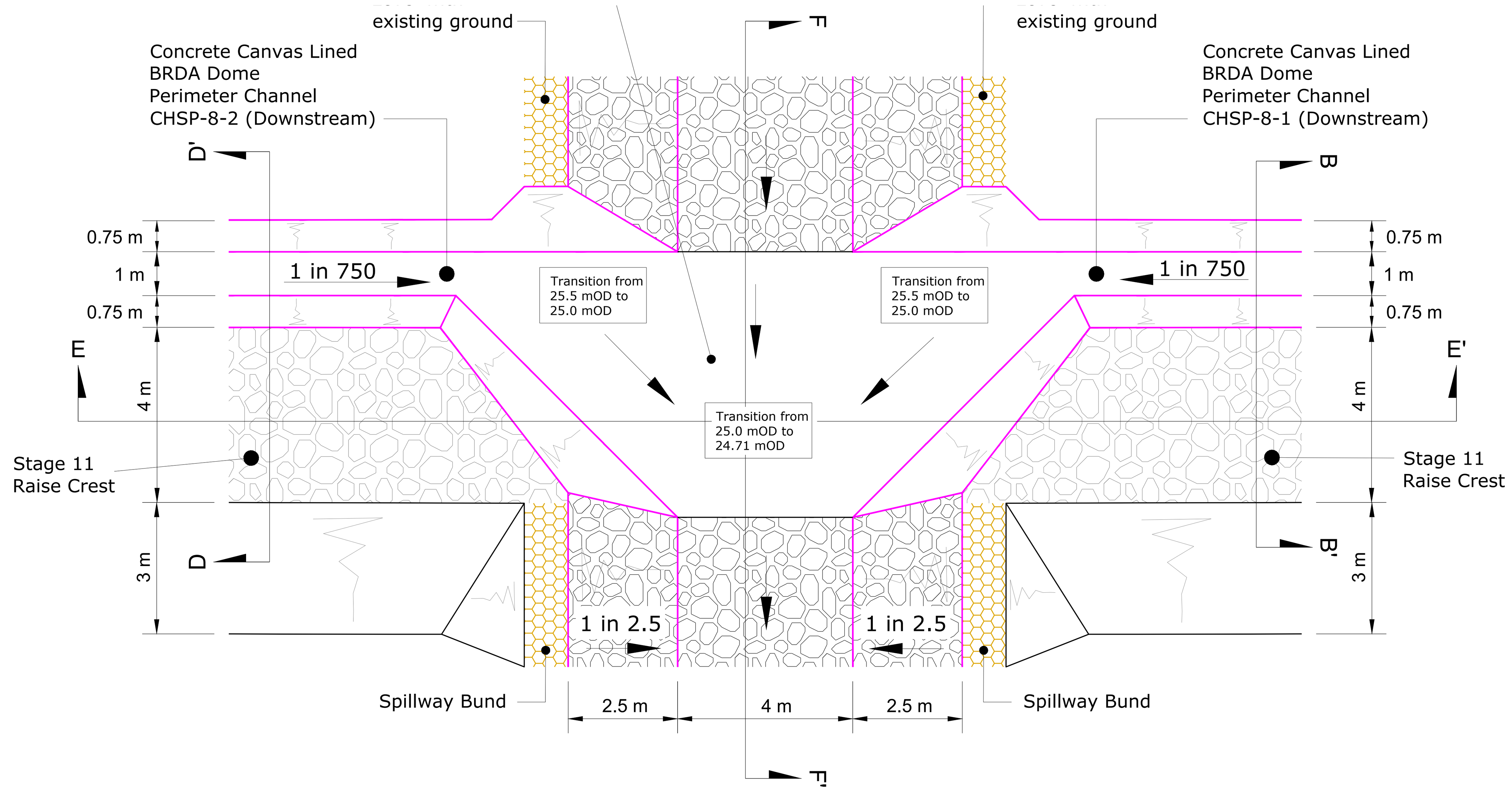
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1
25 mm
0



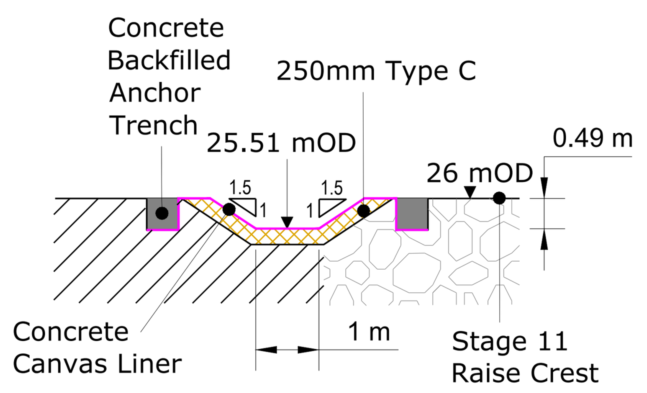
CHANNEL PROFILE CHSP-8-1:
(Scale - 1:100 @ A1)



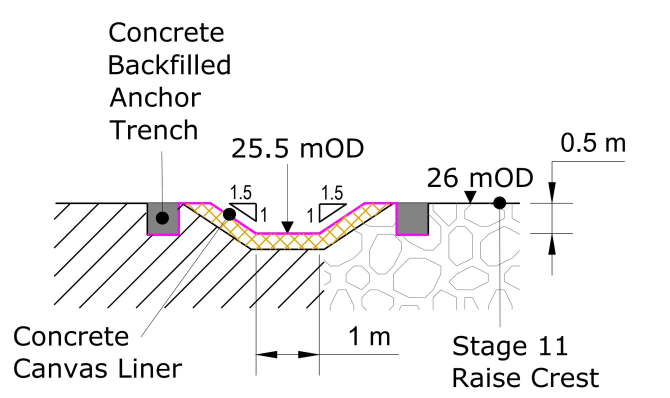
CHANNEL PROFILE CHSP-8-2:
(Scale - 1:100 @ A1)



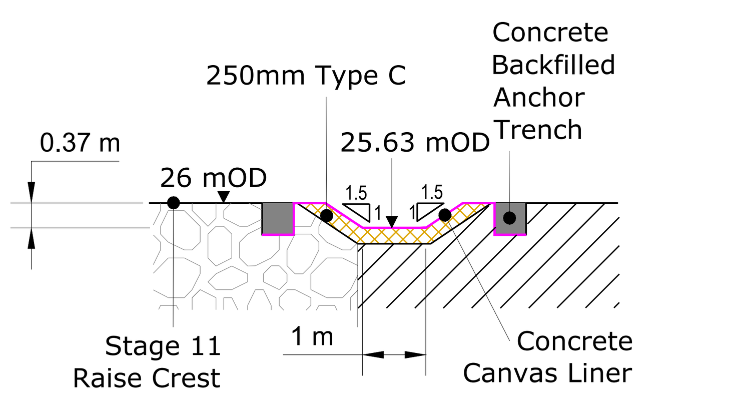
PLAN: (Scale - 1:100 @ A1)
CONFLUENCE OF BRDA DOME PERIMETER
CHANNELS & RIPRAP SPILLWAY SP8



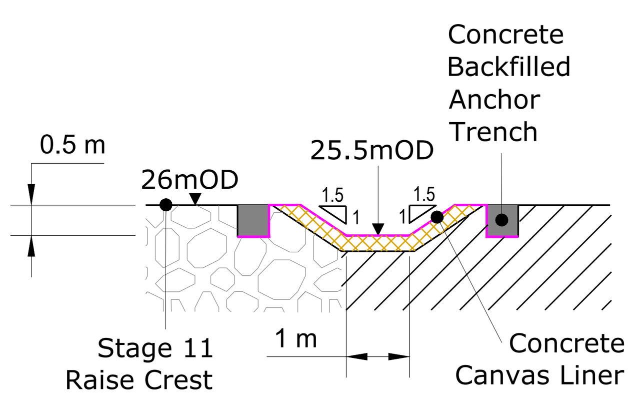
CROSS SECTION A:A' - CHSP-8-1 Upstream:
(Scale - 1:100 @ A1)



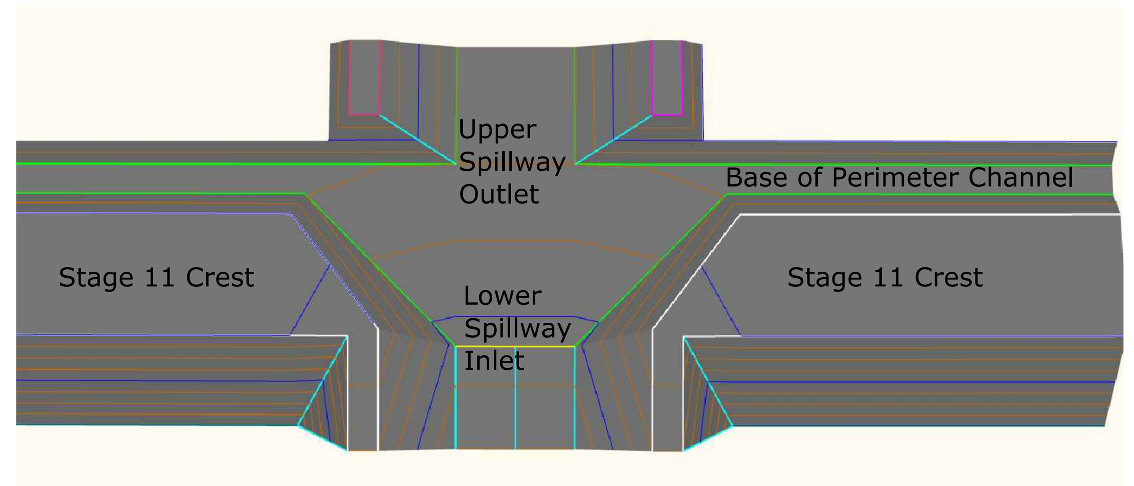
CROSS SECTION B:B' - CHSP-8-1 Downstream:
(Scale - 1:100 @ A1)



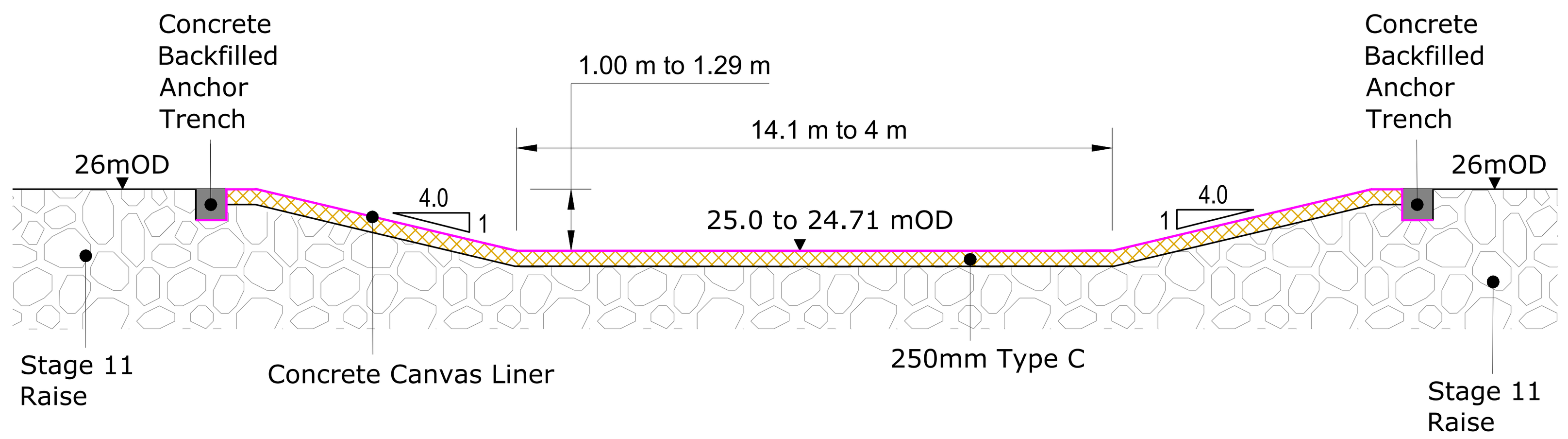
CROSS SECTION C:C' - CHSP-8-2 Upstream:
(Scale - 1:100 @ A1)



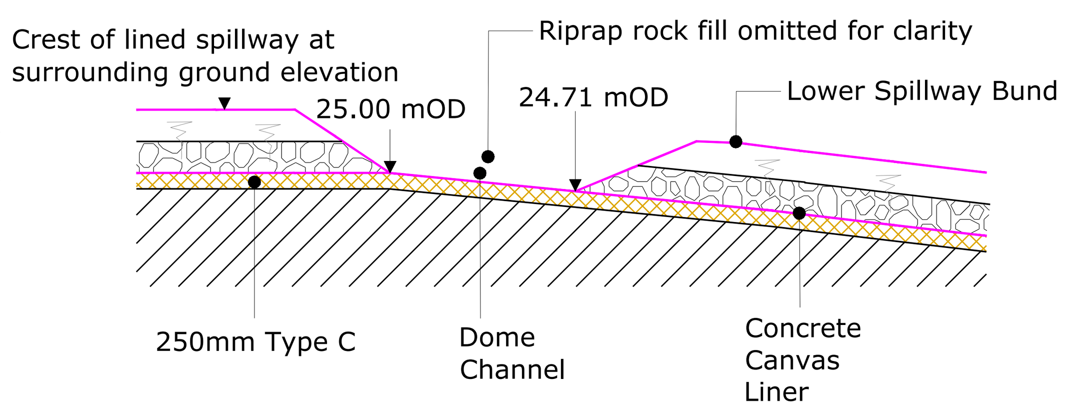
CROSS SECTION D:D' - CHSP-8-2 Downstream:
(Scale - 1:100 @ A1)



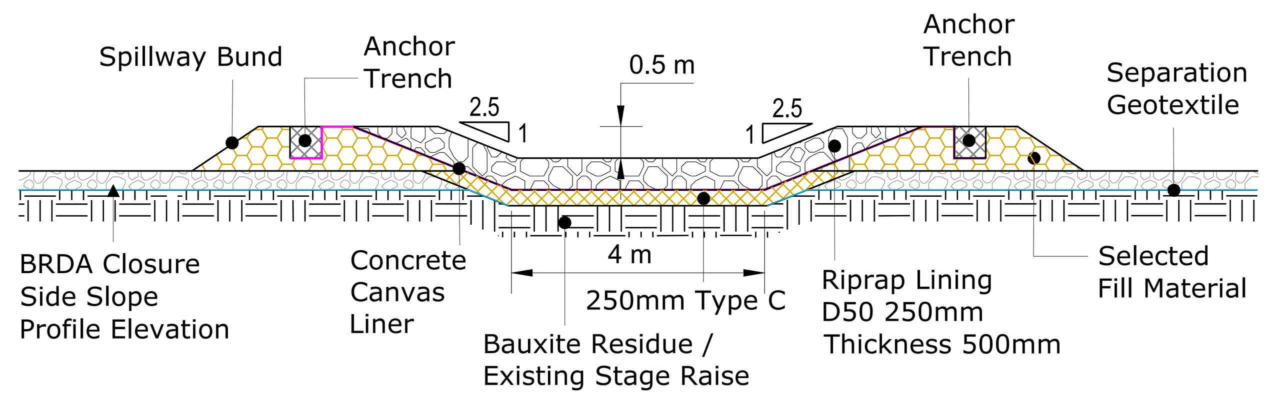
Spillway Confluence - Liner Formation 3D Model
(NTS @ A1)



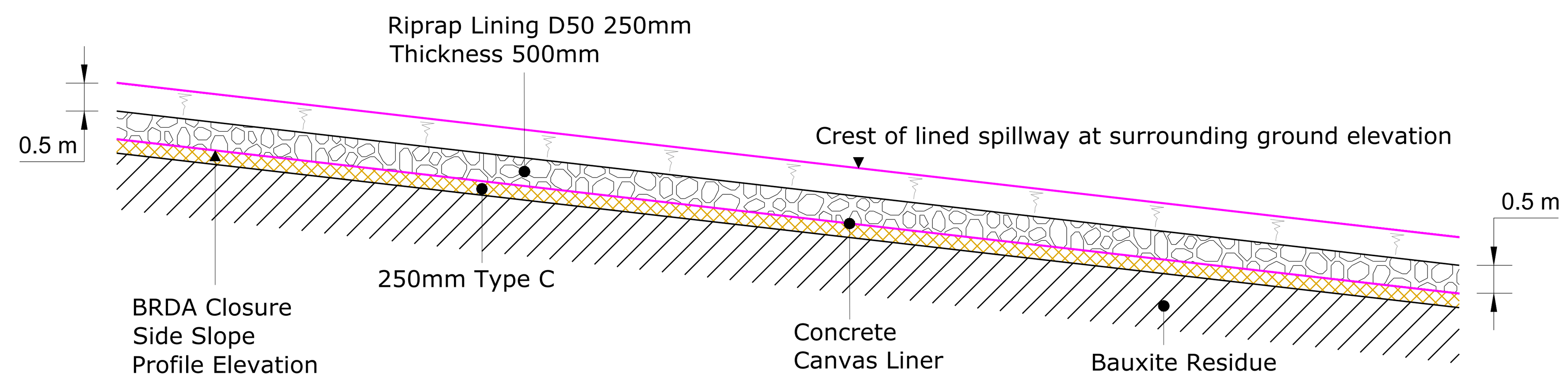
CROSS SECTION E:E' - SPILLWAY CREST SP8:
(Scale - 1:100 @ A1)



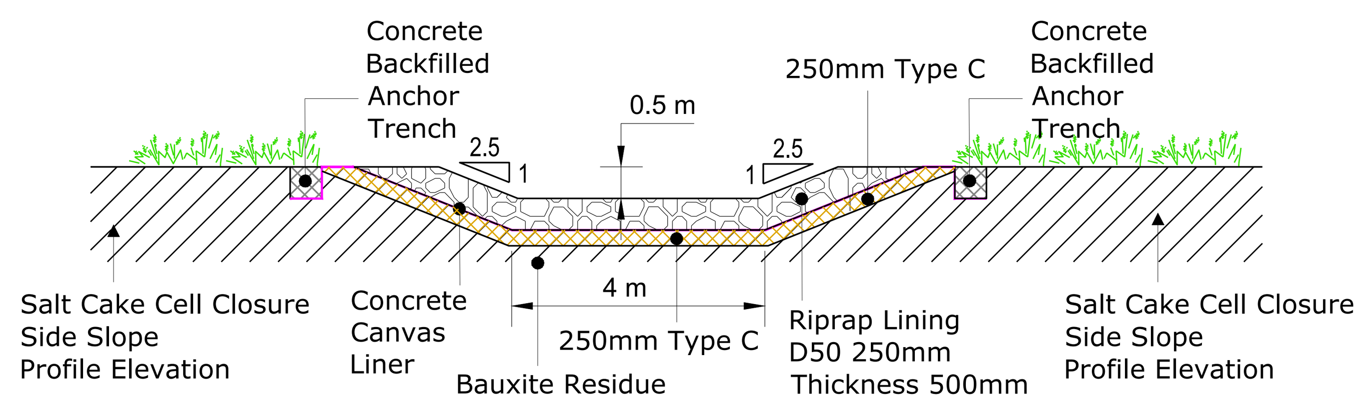
CROSS SECTION F:F' - Spillway Crest SP8:
(Scale - 1:100 @ A1)



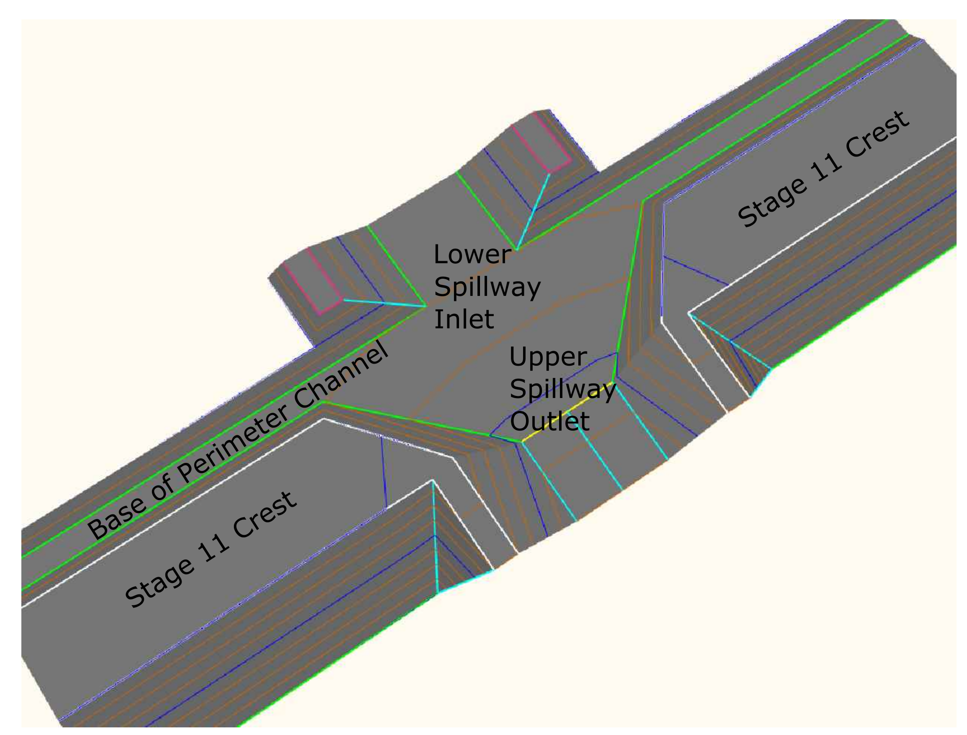
RIPRAP SPILLWAY SP8 - Typical Cross Section: Lower Slope
(Scale - 1:100 @ A1)



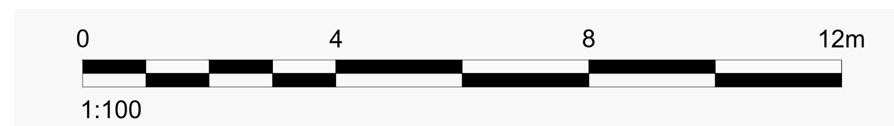
RIPRAP SPILLWAY SP8 - Typical Upper Slope Profile:
(Scale - 1:100 @ A1)

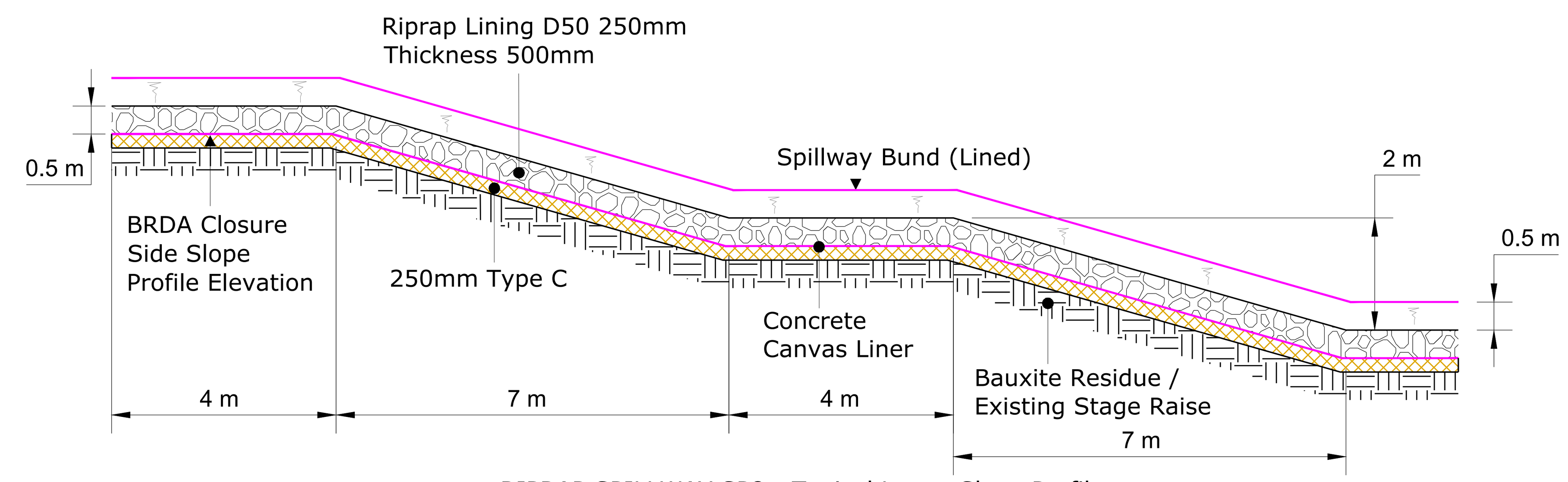


RIPRAP SPILLWAY SP8 - Typical Cross Section: Upper Slope
(Scale - 1:100 @ A1)



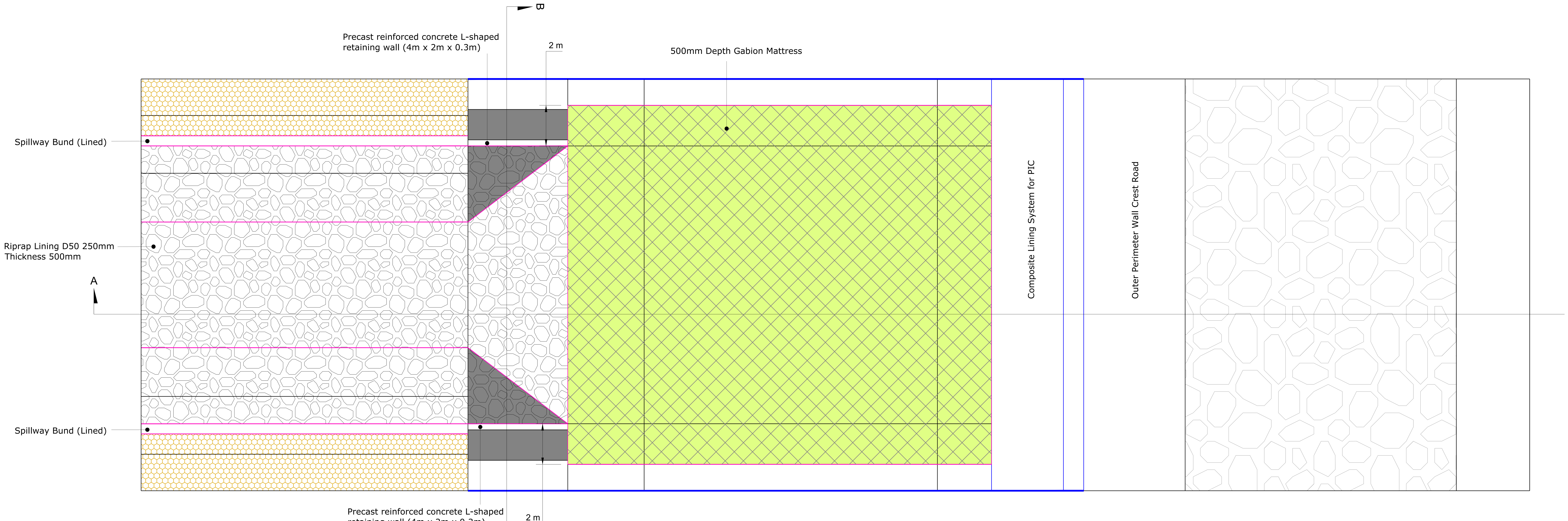
Spillway Confluence - Liner Formation 3D Model
(NTS @ A1)



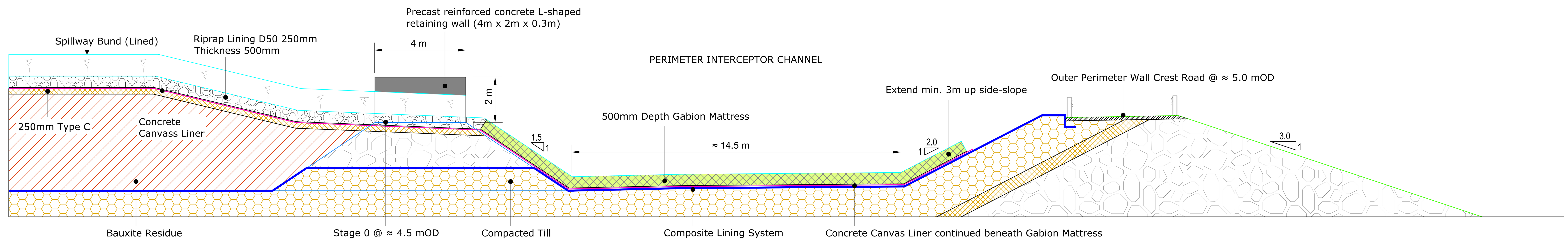


RIPRAP SPILLWAY SP8 - Typical Lower Slope Profile:

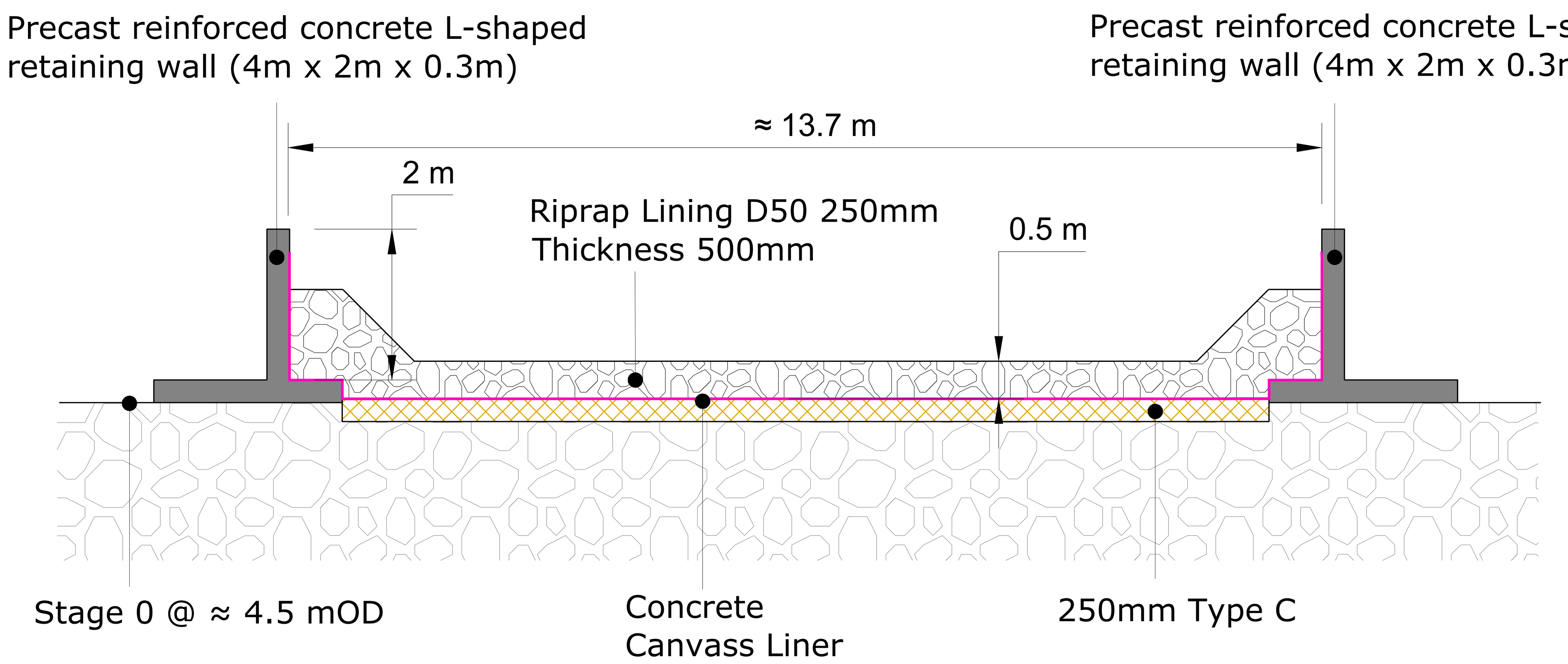
CLIENT		PROJECT
AUGHINISH ALUMINA LTD.		BRDA CLOSURE DESIGN
CONSULTANT	YYYY-MM-DD: 2021-Aug-11	TITLE
GOLDER	PREPARED: POB	4m WIDTH SPILLWAY DETAILS FOR SP-8
	DESIGN: BK	
	REVIEW: BK	PROJECT No. 20143063 · DRAWING No. 04 · Rev. A · SCALE 1:100 A1
	APPROVED: DH/GA	



PLAN: 6m WIDTH SPILLWAY DISCHARGE INTO PIC - Typical Plan
(Scale - 1:100 @ A1)



CROSS-SECTION A-A': 6m WIDTH SPILLWAY DISCHARGE INTO PIC - Typical Cross Section:
(Scale - 1:100 @ A1)



CROSS-SECTION B-B': 6m WIDTH RIPRAP SPILLWAY ENTRANCE TO PIC - Typical Cross Section
(Scale - 1:100 @ A1)

NOTE(S)

1. TYPICAL DETAILS SHOWN FOR 6m WIDTH SPILLWAY. DIMENSIONS AND DETAILS WILL VARY FOR 4m AND 8m WIDTH SPILLWAYS

0 4 8 12m
1:100

CLIENT
AUGHINISH ALUMINA LTD.

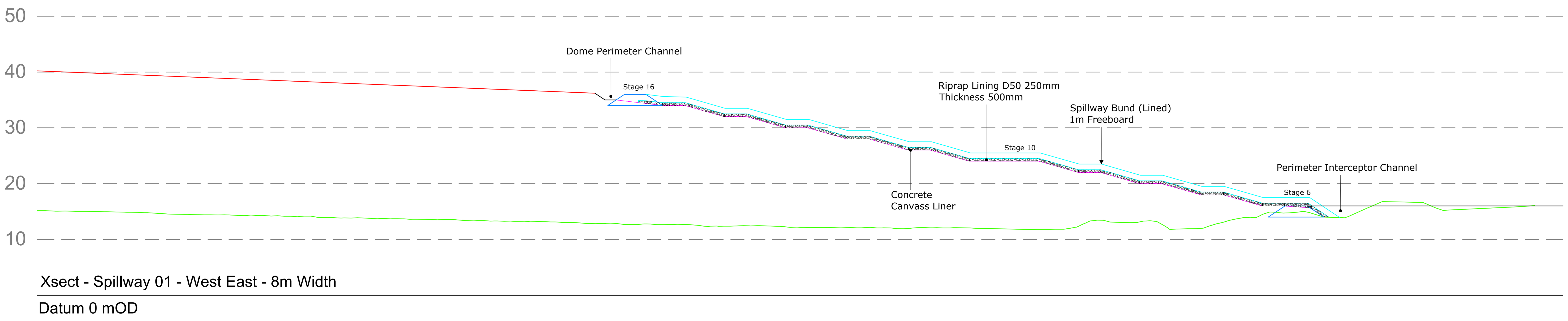
PROJECT
BRDA CLOSURE DESIGN

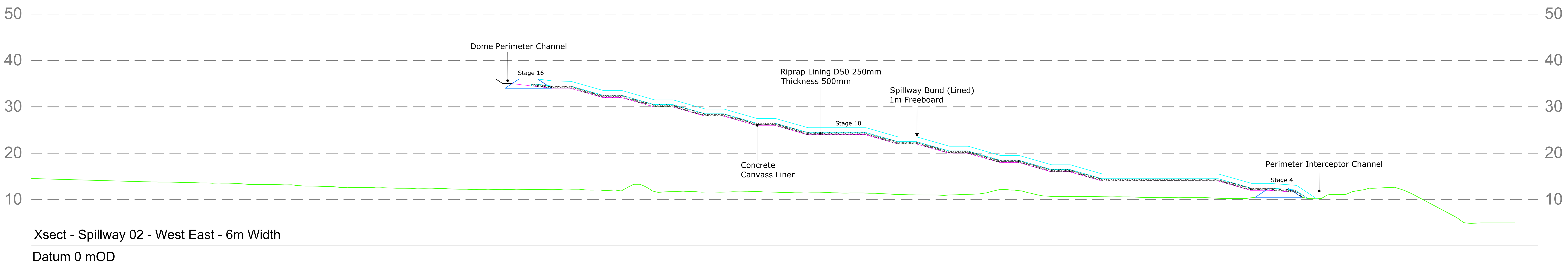
CONSULTANT
GOLDER

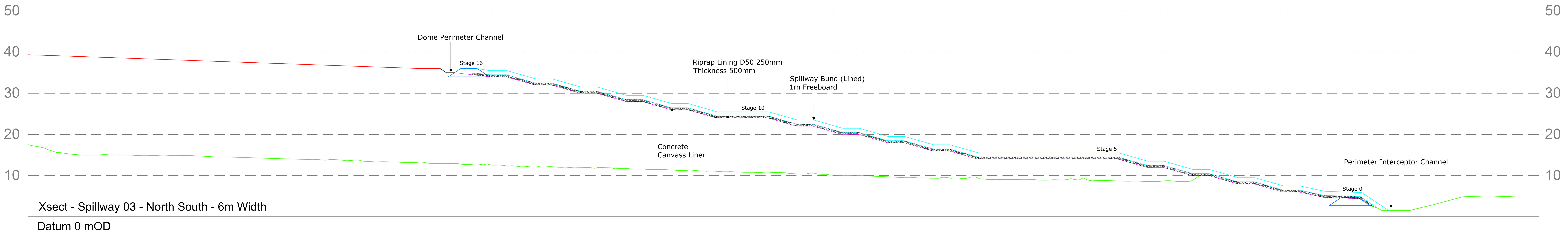
YYYY-MM-DD	2020-Sept-21
PREPARED	POB
DESIGN	GA
REVIEW	BK
APPROVED	GA/DH

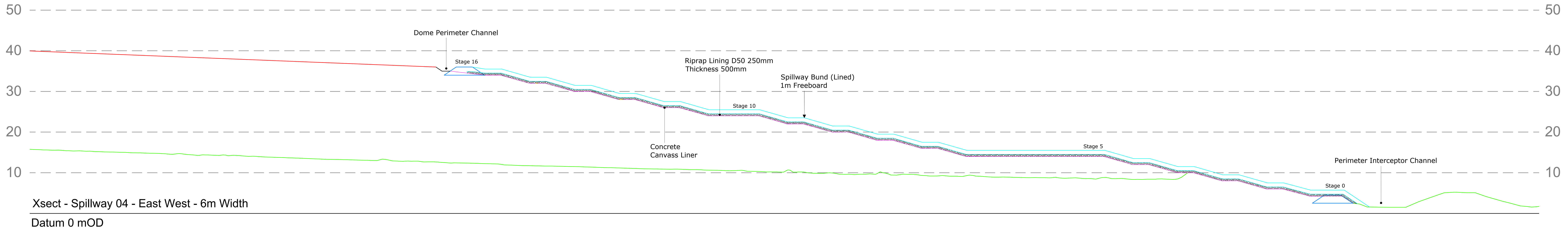
TITLE
TYPICAL DETAILS OF SPILLWAY DISCHARGE INTO PIC

PROJECT No.	DRAWING No.	Rev.	SCALE
20143063	05	A	1:100 A1









LEGEND

- EXISTING TOPOGRAPHY (APRIL 2020)
- PROPOSED TOPOGRAPHY TO STAGE 16
- CREST OF SPILLWAY BUND
- LINED BASE OF SPILLWAY

0 20 40 60m
1:500

CLIENT
AUGHINISH ALUMINA LTD.

PROJECT
BRDA CLOSURE DESIGN

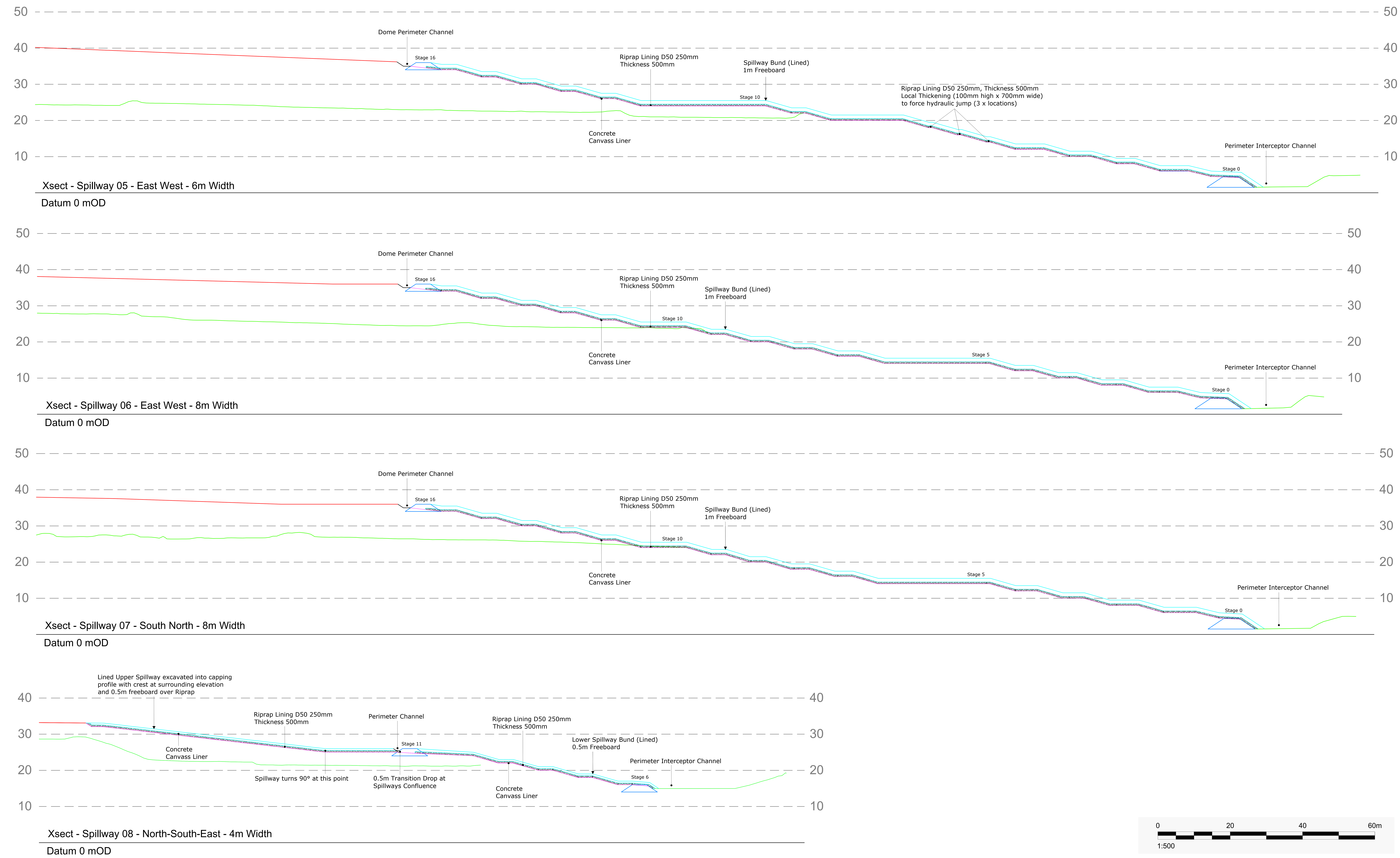
CONSULTANT
GOLDER

YYYY-MM-DD	2020-Sep-02
PREPARED	POB
DESIGN	GA
REVIEW	BK
APPROVED	DH/GA

TITLE
SPILLWAY LONG-SECTION PROFILES: SP-1 TO SP-4

PROJECT No.	DRAWING No.	Rev.	SCALE
20143063	06	A	1:500 A1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1
25 mm
0



LEGEND

- EXISTING TOPOGRAPHY (APRIL 2020)
- PROPOSED TOPOGRAPHY TO STAGE 16
- CREST OF SPILLWAY BUND
- LINED BASE OF SPILLWAY

CLIENT	AUGHINISH ALUMINA LTD.
PROJECT	BRDA CLOSURE DESIGN
CONSULTANT	GOLDER
YYYY-MM-DD	2020-Sep-21
PREPARED	POB
DESIGN	GA
REVIEW	BK
APPROVED	DH/GA
TITLE	SPILLWAY LONG-SECTION PROFILES: SP-5 TO SP-8
PROJECT No.	20143063
DRAWING No.	07
Rev.	A
SCALE	1:500 A1

APPENDIX B

Hydraulic Analysis Results

Dome Perimeter Channel CHSP-1-1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 1 1	260.8	PF 1	1.48	35.35	35.74	35.69	35.85	0.001261	1.44	1.03	3.21	0.81	0.39	0.26
CHSP 1 1	7.25	PF 1	1.48	35.01	35.49		35.55	0.000653	1.15	1.29	3.43	0.6	0.48	0.51
CHSP 1 1	0	PF 1	1.48	35	35.48		35.55	0.000623	1.13	1.31	3.44	0.58	0.48	0.52

Dome Perimeter Channel CHSP-1-2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 1 2	162.9	PF 1	0.91	35.22	35.54	35.47	35.61	0.000934	1.12	0.81	2.96	0.68	0.32	0.46
CHSP 1 2	7.1	PF 1	0.91	35.01	35.48		35.51	0.000251	0.71	1.28	3.42	0.37	0.47	0.52
CHSP 1 2	0	PF 1	0.91	35	35.48		35.51	0.000236	0.69	1.31	3.44	0.36	0.48	0.52

Spillway 1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Flow Depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
SP1	125.499	PF 1	2.39	34.9	35.14	35.1	35.21	0.031087	1.17	2.04	9.19	0.79	0.24	0.76
SP1	124.1383	PF 1	2.39	34.85	35.05	35.05	35.15	0.052635	1.38	1.73	9.02	1.01	0.2	0.8
SP1	121.1277	PF 1	2.39	34.5	34.84	34.7	34.87	0.009295	0.8	3	9.7	0.46	0.34	0.66
SP1	117.0869	PF 1	2.39	34.5	34.7	34.7	34.8	0.052635	1.38	1.73	9.02	1.01	0.2	0.8
SP1	109.8067	PF 1	2.39	32.5	32.84	32.7	32.87	0.009153	0.79	3.02	9.71	0.45	0.34	0.66
SP1	105.8067	PF 1	2.39	32.5	32.7	32.7	32.8	0.052648	1.38	1.73	9.02	1.01	0.2	0.8
SP1	98.5266	PF 1	2.39	30.5	30.84	30.7	30.87	0.009153	0.79	3.02	9.71	0.45	0.34	0.66
SP1	94.5266	PF 1	2.39	30.5	30.7	30.7	30.8	0.052628	1.38	1.73	9.02	1.01	0.2	0.8
SP1	87.2465	PF 1	2.39	28.5	28.84	28.7	28.87	0.009153	0.79	3.02	9.71	0.45	0.34	0.66
SP1	83.2465	PF 1	2.39	28.5	28.7	28.7	28.8	0.052635	1.38	1.73	9.02	1.01	0.2	0.8
SP1	75.9664	PF 1	2.39	26.5	26.84	26.7	26.87	0.009153	0.79	3.02	9.71	0.45	0.34	0.66
SP1	71.9664	PF 1	2.39	26.5	26.7	26.7	26.8	0.052648	1.38	1.73	9.02	1.01	0.2	0.8
SP1	64.6866	PF 1	2.39	24.5	24.91	24.7	24.93	0.004978	0.65	3.68	10.04	0.34	0.41	0.59
SP1	52.1866	PF 1	2.39	24.5	24.7	24.7	24.8	0.052628	1.38	1.73	9.02	1.01	0.2	0.8
SP1	44.9068	PF 1	2.39	22.5	22.84	22.7	22.87	0.009153	0.79	3.02	9.71	0.45	0.34	0.66
SP1	40.9065	PF 1	2.39	22.5	22.7	22.7	22.8	0.052648	1.38	1.73	9.02	1.01	0.2	0.8
SP1	33.6267	PF 1	2.39	20.5	20.84	20.7	20.87	0.009153	0.79	3.02	9.71	0.45	0.34	0.66
SP1	29.6264	PF 1	2.39	20.5	20.7	20.7	20.8	0.052648	1.38	1.73	9.02	1.01	0.2	0.8
SP1	22.3466	PF 1	2.39	18.5	18.84	18.7	18.87	0.009153	0.79	3.02	9.71	0.45	0.34	0.66
SP1	18.3463	PF 1	2.39	18.5	18.7	18.7	18.8	0.052652	1.38	1.73	9.02	1.01	0.2	0.8
SP1	11.0665	PF 1	2.39	16.5	16.84	16.7	16.87	0.009217	0.79	3.01	9.7	0.45	0.34	0.66
SP1	7.1175	PF 1	2.39	16.5	16.7	16.7	16.8	0.05264	1.38	1.73	9.02	1.01	0.2	0.8
SP1	2.6358	PF 1	2.39	16.16	16.34	16.36	16.47	0.078147	1.56	1.53	8.9	1.2	0.18	0.82
SP1	2.3669	PF 1	2.39	16.06	16.2	16.26	16.42	0.204313	2.11	1.13	8.68	1.86	0.14	0.86
SP1	2.15	PF 1	2.39	15.93	16.05	16.13	16.36	0.339074	2.47	0.97	8.58	2.34	0.12	0.88
SP1	0.1014	PF 1	2.39	14.33	14.41	14.53	15.11	1.279754	3.71	0.64	8.39	4.27	0.08	N/A (PIC)
SP1	0	PF 1	2.39	14.26	14.34	14.46	14.97	1.063975	3.5	0.68	8.42	3.93	0.08	N/A (PIC)

Dome Perimeter Channel CHSP-2-1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 2 1	256.33	PF 1	1	35.34	35.66	35.61	35.74	0.001276	1.28	0.78	2.96	0.8	0.32	0.34
CHSP 2 1	6.337	PF 1	1	35.01	35.44		35.48	0.000426	0.88	1.14	3.29	0.48	0.43	0.56
CHSP 2 1	0	PF 1	1	35	35.44		35.47	0.000404	0.86	1.16	3.31	0.47	0.44	0.56

Dome Perimeter Channel CHSP-2-2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 2 2	306.856	PF 1	0.83	35.41	35.69	35.65	35.77	0.001292	1.21	0.68	2.86	0.79	0.28	0.31
CHSP 2 2	166.432	PF 1	0.83	35.22	35.52		35.59	0.001069	1.14	0.73	2.89	0.72	0.3	0.48
CHSP 2 2	29.75	PF 1	0.83	35.04	35.44		35.47	0.000374	0.79	1.05	3.21	0.44	0.4	0.56
CHSP 2 2	23.368	PF 1	0.83	35.03	35.44		35.47	0.000349	0.77	1.07	3.23	0.43	0.41	0.56
CHSP 2 2	6.273	PF 1	0.83	35.01	35.44		35.46	0.000295	0.73	1.14	3.29	0.4	0.43	0.56
CHSP 2 2	0	PF 1	0.83	35	35.44		35.46	0.000278	0.72	1.16	3.31	0.39	0.44	0.56

Spillway 2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
SP2	192.9143	PF 1	1.83	34.9	35.14	35.1	35.21	0.02737	1.11	1.65	7.24	0.74	0.24	0.76
SP2	191.5531	PF 1	1.83	34.86	35.06	35.06	35.16	0.053022	1.37	1.33	7.02	1.01	0.2	0.8
SP2	188.5425	PF 1	1.83	34.5	34.84	34.7	34.87	0.009316	0.78	2.34	7.7	0.45	0.34	0.66
SP2	184.5017	PF 1	1.83	34.5	34.7	34.7	34.8	0.053022	1.37	1.33	7.02	1.01	0.2	0.8
SP2	177.2215	PF 1	1.83	32.5	32.84	32.7	32.87	0.009175	0.78	2.35	7.71	0.45	0.34	0.66
SP2	173.2215	PF 1	1.83	32.5	32.7	32.7	32.8	0.053022	1.37	1.33	7.02	1.01	0.2	0.8
SP2	165.9414	PF 1	1.83	30.5	30.84	30.7	30.87	0.009175	0.78	2.35	7.71	0.45	0.34	0.66
SP2	161.9414	PF 1	1.83	30.5	30.7	30.7	30.8	0.053039	1.37	1.33	7.02	1.01	0.2	0.8
SP2	154.661	PF 1	1.83	28.5	28.84	28.7	28.87	0.009175	0.78	2.35	7.71	0.45	0.34	0.66
SP2	150.6613	PF 1	1.83	28.5	28.7	28.7	28.8	0.053036	1.37	1.33	7.02	1.01	0.2	0.8
SP2	143.3812	PF 1	1.83	26.5	26.84	26.7	26.87	0.009175	0.78	2.35	7.71	0.45	0.34	0.66
SP2	139.3808	PF 1	1.83	26.5	26.7	26.7	26.8	0.053022	1.37	1.33	7.02	1.01	0.2	0.8
SP2	132.101	PF 1	1.83	24.5	24.91	24.7	24.93	0.004975	0.64	2.87	8.05	0.34	0.41	0.59
SP2	119.6	PF 1	1.83	24.5	24.7	24.7	24.8	0.053039	1.37	1.33	7.02	1.01	0.2	0.8
SP2	112.3208	PF 1	1.83	22.5	22.84	22.7	22.87	0.009175	0.78	2.35	7.71	0.45	0.34	0.66
SP2	108.3204	PF 1	1.83	22.5	22.7	22.7	22.8	0.053022	1.37	1.33	7.02	1.01	0.2	0.8
SP2	101.0406	PF 1	1.83	20.5	20.84	20.7	20.87	0.009175	0.78	2.35	7.71	0.45	0.34	0.66
SP2	97.0402	PF 1	1.83	20.5	20.7	20.7	20.8	0.053022	1.37	1.33	7.02	1.01	0.2	0.8
SP2	89.7604	PF 1	1.83	18.5	18.84	18.7	18.87	0.009175	0.78	2.35	7.71	0.45	0.34	0.66
SP2	85.7601	PF 1	1.83	18.5	18.7	18.7	18.8	0.05304	1.37	1.33	7.02	1.01	0.2	0.8
SP2	78.4803	PF 1	1.83	16.5	16.84	16.7	16.87	0.009175	0.78	2.35	7.71	0.45	0.34	0.66
SP2	74.48	PF 1	1.83	16.5	16.7	16.7	16.8	0.053028	1.37	1.33	7.02	1.01	0.2	0.8
SP2	67.2002	PF 1	1.83	14.5	14.96	14.7	14.98	0.00321	0.55	3.32	8.32	0.28	0.46	0.54
SP2	42.2005	PF 1	1.83	14.5	14.7	14.7	14.8	0.053039	1.37	1.33	7.02	1.01	0.2	0.8
SP2	34.9204	PF 1	1.83	12.5	12.84	12.7	12.87	0.009239	0.78	2.34	7.71	0.45	0.34	0.66
SP2	30.9712	PF 1	1.83	12.5	12.7	12.7	12.8	0.053028	1.37	1.33	7.02	1.01	0.2	0.8
SP2	26.3008	PF 1	1.83	12.15	12.33	12.35	12.46	0.078385	1.55	1.18	6.91	1.2	0.18	0.82
SP2	25.5103	PF 1	1.83	11.71	12.16	11.91	12.18	0.003488	0.57	3.23	8.27	0.29	0.45	0.55
SP2	12.9295	PF 1	1.83	11.7	12.11		12.13	0.005119	0.64	2.85	8.03	0.34	0.41	0.59
SP2	1.0717	PF 1	1.83	11.7	11.9	11.9	12	0.05303	1.37	1.33	7.02	1.01	0.2	0.8
SP2	0	PF 1	1.83	11.26	11.37	11.46	11.71	0.395589	2.58	0.71	6.56	2.51	0.11	0.89

Dome Perimeter Channel CHSP-3-1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 3 1	280.313	PF 1	0.61	35.37	35.61	35.57	35.67	0.001278	1.09	0.56	2.72	0.77	0.24	0.39
CHSP 3 1	250.854	PF 1	0.61	35.33	35.57		35.63	0.001229	1.08	0.56	2.72	0.76	0.24	0.43
CHSP 3 1	238.946	PF 1	0.61	35.32	35.56		35.62	0.001225	1.08	0.57	2.72	0.76	0.24	0.44
CHSP 3 1	6.398	PF 1	0.61	35.01	35.42		35.44	0.000183	0.56	1.08	3.24	0.31	0.41	0.58
CHSP 3 1	0	PF 1	0.61	35	35.42		35.44	0.00017	0.55	1.11	3.27	0.3	0.42	0.58

Dome Perimeter Channel CHSP-3-2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 3 2	266.057	PF 1	1.15	35.35	35.69	35.65	35.79	0.001312	1.36	0.85	3	0.81	0.34	0.31
CHSP 3 2	176.42	PF 1	1.15	35.23	35.58		35.67	0.00122	1.32	0.87	3.04	0.79	0.35	0.42
CHSP 3 2	132.4	PF 1	1.15	35.18	35.53		35.61	0.001146	1.29	0.89	3.06	0.76	0.35	0.47
CHSP 3 2	10.404	PF 1	1.15	35.01	35.43		35.48	0.000648	1.06	1.08	3.24	0.59	0.42	0.57
CHSP 3 2	6.49	PF 1	1.15	35.01	35.43		35.48	0.000635	1.05	1.09	3.25	0.58	0.42	0.57
CHSP 3 2	0	PF 1	1.15	35	35.42		35.48	0.000604	1.04	1.11	3.27	0.57	0.42	0.58

Spillway 3: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
SP3	225.8646	PF 1	1.76	34.9	35.14	35.1	35.2	0.029802	1.13	1.56	7.19	0.77	0.24	0.76
SP3	224.503	PF 1	1.76	34.86	35.06	35.06	35.15	0.053409	1.36	1.3	7	1.01	0.2	0.8
SP3	221.4928	PF 1	1.76	34.5	34.83		34.86	0.009228	0.77	2.28	7.67	0.45	0.33	0.67
SP3	217.452	PF 1	1.76	34.5	34.7	34.7	34.79	0.053409	1.36	1.3	7	1.01	0.2	0.8
SP3	210.1718	PF 1	1.76	32.5	32.83	32.7	32.86	0.009228	0.77	2.28	7.67	0.45	0.33	0.67
SP3	206.131	PF 1	1.76	32.5	32.7	32.7	32.79	0.053409	1.36	1.3	7	1.01	0.2	0.8
SP3	198.8508	PF 1	1.76	30.5	30.83	30.7	30.86	0.009227	0.77	2.28	7.67	0.45	0.33	0.67
SP3	194.81	PF 1	1.76	30.5	30.7	30.7	30.79	0.053429	1.36	1.3	7	1.01	0.2	0.8
SP3	187.5298	PF 1	1.76	28.5	28.83	28.7	28.86	0.009227	0.77	2.28	7.67	0.45	0.33	0.67
SP3	183.489	PF 1	1.76	28.5	28.7	28.7	28.79	0.053432	1.36	1.3	7	1.01	0.2	0.8
SP3	176.2249	PF 1	1.76	26.5	26.83	26.7	26.86	0.00926	0.77	2.28	7.67	0.45	0.33	0.67
SP3	172.2102	PF 1	1.76	26.5	26.7	26.7	26.79	0.053409	1.36	1.3	7	1.01	0.2	0.8
SP3	164.9304	PF 1	1.76	24.5	24.9	24.7	24.92	0.004913	0.63	2.81	8.01	0.34	0.4	0.6
SP3	152.43	PF 1	1.76	24.5	24.7	24.7	24.79	0.053429	1.36	1.3	7	1.01	0.2	0.8
SP3	145.1662	PF 1	1.76	22.5	22.83	22.7	22.86	0.00926	0.77	2.28	7.67	0.45	0.33	0.67
SP3	141.1515	PF 1	1.76	22.5	22.7	22.7	22.79	0.053409	1.36	1.3	7	1.01	0.2	0.8
SP3	133.8877	PF 1	1.76	20.5	20.83	20.7	20.86	0.00926	0.77	2.28	7.67	0.45	0.33	0.67
SP3	129.873	PF 1	1.76	20.5	20.7	20.7	20.79	0.053409	1.36	1.3	7	1.01	0.2	0.8
SP3	122.5932	PF 1	1.76	18.5	18.84	18.7	18.87	0.009087	0.77	2.3	7.68	0.45	0.34	0.66
SP3	118.5929	PF 1	1.76	18.5	18.7	18.7	18.79	0.05343	1.36	1.3	7	1.01	0.2	0.8
SP3	111.3131	PF 1	1.76	16.5	16.84	16.7	16.87	0.009087	0.77	2.3	7.68	0.45	0.34	0.66
SP3	107.3128	PF 1	1.76	16.5	16.7	16.7	16.79	0.053417	1.36	1.3	7	1.01	0.2	0.8
SP3	100.033	PF 1	1.76	14.5	14.98	14.7	15	0.002576	0.51	3.48	8.42	0.25	0.48	0.52
SP3	66.0329	PF 1	1.76	14.5	14.7	14.7	14.79	0.053428	1.36	1.3	7	1.01	0.2	0.8
SP3	58.7528	PF 1	1.76	12.5	12.84	12.7	12.87	0.009087	0.77	2.3	7.68	0.45	0.34	0.66
SP3	54.7523	PF 1	1.76	12.5	12.7	12.7	12.79	0.053417	1.36	1.3	7	1.01	0.2	0.8
SP3	47.4727	PF 1	1.76	10.5	10.84	10.7	10.87	0.009087	0.77	2.3	7.68	0.45	0.34	0.66
SP3	43.4726	PF 1	1.76	10.5	10.7	10.7	10.79	0.05342	1.36	1.3	7	1.01	0.2	0.8
SP3	36.1925	PF 1	1.76	8.5	8.84	8.7	8.87	0.009087	0.77	2.3	7.68	0.45	0.34	0.66
SP3	32.1924	PF 1	1.76	8.5	8.7	8.7	8.79	0.053424	1.36	1.3	7	1.01	0.2	0.8
SP3	24.9123	PF 1	1.76	6.5	6.84	6.7	6.87	0.009107	0.77	2.29	7.68	0.45	0.34	0.66
SP3	20.9283	PF 1	1.76	6.5	6.7	6.7	6.79	0.053418	1.36	1.3	7	1.01	0.2	0.8
SP3	14.8315	PF 1	1.76	5.2	5.33	5.4	5.56	0.211122	2.09	0.84	6.66	1.88	0.13	0.87
SP3	5.8989	PF 1	1.76	4.88	5.08	5.08	5.17	0.053424	1.36	1.3	7	1.01	0.2	0.8
SP3	0	PF 1	1.76	2.07	2.5	2.27	2.52	0.003867	0.58	3.04	8.15	0.3	0.43	N/A (PIC)

Dome Perimeter Channel CHSP-4-1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Flow Depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
CHSP 4 1	191.914	PF 1	0.68	35.26	35.52	35.47	35.58	0.001128	1.09	0.62	2.77	0.73	0.26	0.48
CHSP 4 1	138.007	PF 1	0.68	35.18	35.47		35.52	0.00085	0.99	0.69	2.85	0.64	0.29	0.53
CHSP 4 1	85.84	PF 1	0.68	35.11	35.44		35.47	0.000531	0.84	0.81	2.98	0.52	0.33	0.56
CHSP 4 1	6.155	PF 1	0.68	35.01	35.42		35.44	0.000224	0.62	1.09	3.25	0.34	0.41	0.58
CHSP 4 1	0	PF 1	0.68	35	35.42		35.44	0.000213	0.61	1.11	3.26	0.34	0.42	0.58

Dome Perimeter Channel CHSP-4-2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Flow Depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
CHSP 4 2	224.872	PF 1	1.05	35.3	35.62	35.58	35.71	0.00127	1.3	0.81	2.97	0.8	0.32	0.38
CHSP 4 2	76.312	PF 1	1.05	35.1	35.46		35.53	0.000891	1.15	0.91	3.08	0.68	0.36	0.54
CHSP 4 2	6.319	PF 1	1.05	35.01	35.42		35.47	0.000533	0.96	1.09	3.24	0.53	0.41	0.58
CHSP 4 2	0	PF 1	1.05	35	35.42		35.47	0.000508	0.95	1.11	3.26	0.52	0.42	0.58

Spillway 4: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
SP4	226.4703	PF 1	1.73	34.9	35.14	35.1	35.2	0.025973	1.07	1.62	7.22	0.72	0.24	0.76
SP4	225.1091	PF 1	1.73	34.86	35.06	35.06	35.15	0.053602	1.35	1.28	6.99	1.01	0.2	0.8
SP4	222.09	PF 1	1.73	34.5	34.83	34.7	34.86	0.009203	0.77	2.26	7.66	0.45	0.33	0.67
SP4	218.06	PF 1	1.73	34.5	34.7	34.7	34.79	0.053602	1.35	1.28	6.99	1.01	0.2	0.8
SP4	210.78	PF 1	1.73	32.5	32.83	32.7	32.86	0.009052	0.76	2.27	7.66	0.45	0.33	0.67
SP4	206.78	PF 1	1.73	32.5	32.7	32.7	32.79	0.053583	1.35	1.28	6.99	1.01	0.2	0.8
SP4	199.5	PF 1	1.73	30.5	30.83	30.7	30.86	0.009052	0.76	2.27	7.66	0.45	0.33	0.67
SP4	195.5	PF 1	1.73	30.5	30.7	30.7	30.79	0.053582	1.35	1.28	6.99	1.01	0.2	0.8
SP4	188.21	PF 1	1.73	28.5	28.83	28.7	28.86	0.009054	0.76	2.27	7.66	0.45	0.33	0.67
SP4	184.22	PF 1	1.73	28.5	28.7	28.7	28.79	0.053598	1.35	1.28	6.99	1.01	0.2	0.8
SP4	176.94	PF 1	1.73	26.5	26.83	26.7	26.86	0.009052	0.76	2.27	7.66	0.45	0.33	0.67
SP4	172.94	PF 1	1.73	26.5	26.7	26.7	26.79	0.053583	1.35	1.28	6.99	1.01	0.2	0.8
SP4	165.66	PF 1	1.73	24.5	24.9	24.7	24.92	0.004886	0.62	2.78	7.99	0.34	0.4	0.6
SP4	153.16	PF 1	1.73	24.5	24.7	24.7	24.79	0.053582	1.35	1.28	6.99	1.01	0.2	0.8
SP4	145.88	PF 1	1.73	22.5	22.83	22.7	22.86	0.009052	0.76	2.27	7.66	0.45	0.33	0.67
SP4	141.88	PF 1	1.73	22.5	22.7	22.7	22.79	0.053583	1.35	1.28	6.99	1.01	0.2	0.8
SP4	134.6	PF 1	1.73	20.5	20.83	20.7	20.86	0.009052	0.76	2.27	7.66	0.45	0.33	0.67
SP4	130.6	PF 1	1.73	20.5	20.7	20.7	20.79	0.053583	1.35	1.28	6.99	1.01	0.2	0.8
SP4	123.32	PF 1	1.73	18.5	18.83	18.7	18.86	0.009052	0.76	2.27	7.66	0.45	0.33	0.67
SP4	119.32	PF 1	1.73	18.5	18.7	18.7	18.79	0.05359	1.35	1.28	6.99	1.01	0.2	0.8
SP4	112.04	PF 1	1.73	16.5	16.83	16.7	16.86	0.009051	0.76	2.27	7.66	0.45	0.33	0.67
SP4	108.04	PF 1	1.73	16.5	16.7	16.7	16.79	0.053604	1.35	1.28	6.99	1.01	0.2	0.8
SP4	100.76	PF 1	1.73	14.5	14.98	14.7	14.99	0.00256	0.5	3.45	8.4	0.25	0.48	0.52
SP4	66.76	PF 1	1.73	14.5	14.7	14.7	14.79	0.053611	1.35	1.28	6.99	1.01	0.2	0.8
SP4	59.48	PF 1	1.73	12.5	12.83	12.7	12.86	0.009235	0.77	2.26	7.65	0.45	0.33	0.67
SP4	55.48	PF 1	1.73	12.5	12.7	12.7	12.79	0.053604	1.35	1.28	6.99	1.01	0.2	0.8
SP4	48.2	PF 1	1.73	10.5	10.83	10.7	10.86	0.009051	0.76	2.27	7.66	0.45	0.33	0.67
SP4	44.2	PF 1	1.73	10.5	10.7	10.7	10.79	0.053604	1.35	1.28	6.99	1.01	0.2	0.8
SP4	36.92	PF 1	1.73	8.5	8.83	8.7	8.86	0.009052	0.76	2.27	7.66	0.45	0.33	0.67
SP4	32.92	PF 1	1.73	8.5	8.7	8.7	8.79	0.053598	1.35	1.28	6.99	1.01	0.2	0.8
SP4	25.64	PF 1	1.73	6.5	6.83	6.7	6.86	0.009057	0.76	2.27	7.66	0.45	0.33	0.67
SP4	21.64	PF 1	1.73	6.5	6.7	6.7	6.79	0.053596	1.35	1.28	6.99	1.01	0.2	0.8
SP4	14.75	PF 1	1.73	4.73	5.11	4.93	5.13	0.005983	0.66	2.6	7.88	0.37	0.38	0.62
SP4	5.89	PF 1	1.73	4.73	4.93	4.93	5.02	0.053598	1.35	1.28	6.99	1.01	0.2	0.8
SP4	4.19	PF 1	1.73	3.73	3.8	3.93	4.67	1.963201	4.14	0.42	6.34	5.15	0.07	N/A (PIC)
SP4	1.86	PF 1	1.73	2.67	2.79	2.87	3.06	0.296581	2.31	0.75	6.59	2.19	0.12	N/A (PIC)
SP4	0	PF 1	1.73	1.97	2.5	2.17	2.51	0.001803	0.45	3.88	8.65	0.21	0.53	N/A (PIC)

Dome Perimeter Channel CHSP-5-1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 5 1	194.751	PF 1	1.2	35.26	35.61	35.56	35.7	0.001271	1.36	0.88	3.05	0.8	0.35	0.39
CHSP 5 1	52.971	PF 1	1.2	35.07	35.45		35.53	0.000972	1.24	0.97	3.13	0.71	0.38	0.55
CHSP 5 1	6.569	PF 1	1.2	35.01	35.41		35.48	0.000764	1.14	1.05	3.21	0.63	0.4	0.59
CHSP 5 1	0	PF 1	1.2	35	35.41		35.47	0.000734	1.12	1.07	3.23	0.62	0.41	0.59

Dome Perimeter Channel CHSP-5-2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 5 2	97.496	PF 1	0.5	35.13	35.42	35.31	35.45	0.000413	0.7	0.71	2.88	0.45	0.29	0.58
CHSP 5 2	6.45	PF 1	0.5	35.01	35.41		35.42	0.000137	0.48	1.04	3.2	0.27	0.4	0.59
CHSP 5 2	0	PF 1	0.5	35	35.41		35.42	0.000127	0.47	1.07	3.23	0.26	0.41	0.59

Spillway 5: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
SP5	234.4185	PF 1	1.7	34.89	35.14	35.09	35.2	0.022053	1.01	1.68	7.27	0.67	0.25	0.75
SP5	233.0573	PF 1	1.7	34.86	35.06	35.06	35.15	0.053776	1.34	1.27	6.98	1.01	0.2	0.8
SP5	230.0467	PF 1	1.7	34.5	34.83	34.7	34.86	0.00913	0.76	2.24	7.64	0.45	0.33	0.67
SP5	225.9917	PF 1	1.7	34.5	34.7	34.7	34.79	0.053776	1.34	1.27	6.98	1.01	0.2	0.8
SP5	218.7275	PF 1	1.7	32.5	32.83	32.7	32.86	0.009181	0.76	2.24	7.64	0.45	0.33	0.67
SP5	214.7131	PF 1	1.7	32.5	32.7	32.7	32.79	0.053798	1.34	1.27	6.98	1.01	0.2	0.8
SP5	207.449	PF 1	1.7	30.5	30.83	30.7	30.86	0.00918	0.76	2.24	7.64	0.45	0.33	0.67
SP5	203.4346	PF 1	1.7	30.5	30.7	30.7	30.79	0.053801	1.34	1.27	6.98	1.01	0.2	0.8
SP5	196.1545	PF 1	1.7	28.5	28.83	28.7	28.86	0.009016	0.76	2.25	7.65	0.44	0.33	0.67
SP5	192.1545	PF 1	1.7	28.5	28.7	28.7	28.79	0.053776	1.34	1.27	6.98	1.01	0.2	0.8
SP5	184.8744	PF 1	1.7	26.5	26.83	26.7	26.86	0.009016	0.76	2.25	7.65	0.44	0.33	0.67
SP5	180.8744	PF 1	1.7	26.5	26.7	26.7	26.79	0.053798	1.34	1.27	6.98	1.01	0.2	0.8
SP5	173.6106	PF 1	1.7	24.5	24.98	24.7	24.99	0.002514	0.5	3.43	8.39	0.25	0.48	0.52
SP5	138.9792	PF 1	1.7	24.5	24.7	24.7	24.79	0.053801	1.34	1.27	6.98	1.01	0.2	0.8
SP5	131.7003	PF 1	1.7	22.5	22.83	22.7	22.86	0.009015	0.76	2.25	7.65	0.44	0.33	0.67
SP5	127.7	PF 1	1.7	22.5	22.7	22.7	22.79	0.053798	1.34	1.27	6.98	1.01	0.2	0.8
SP5	120.4202	PF 1	1.7	20.5	20.93	20.7	20.95	0.00362	0.56	3.04	8.15	0.29	0.43	0.57
SP5	100.4678	PF 1	1.7	20.5	20.7	20.7	20.79	0.053798	1.34	1.27	6.98	1.01	0.2	0.8
SP5	93.1889	PF 1	1.7	18.5	18.88	18.7	18.9	0.005581	0.65	2.63	7.9	0.36	0.38	0.62
SP5	92.8899	PF 1	1.7	18.5	18.88		18.9	0.005681	0.65	2.62	7.89	0.36	0.38	0.62
SP5	92.5258	PF 1	1.7	18.6	18.8	18.8	18.89	0.053778	1.34	1.27	6.98	1.01	0.2	0.8
SP5	84.8817	PF 1	1.7	16.5	16.88	16.7	16.9	0.005585	0.65	2.63	7.89	0.36	0.38	0.62
SP5	84.5827	PF 1	1.7	16.5	16.88		16.9	0.005685	0.65	2.62	7.89	0.36	0.38	0.62
SP5	84.2217	PF 1	1.7	16.6	16.8	16.8	16.89	0.053779	1.34	1.27	6.98	1.01	0.2	0.8
SP5	76.5776	PF 1	1.7	14.5	14.88	14.7	14.9	0.005585	0.65	2.63	7.89	0.36	0.38	0.62
SP5	76.2786	PF 1	1.7	14.5	14.88		14.9	0.005685	0.65	2.62	7.89	0.36	0.38	0.62
SP5	75.9176	PF 1	1.7	14.6	14.8	14.8	14.89	0.053786	1.34	1.27	6.98	1.01	0.2	0.8
SP5	68.2731	PF 1	1.7	12.5	12.87	12.7	12.89	0.006279	0.67	2.53	7.83	0.38	0.37	0.63
SP5	60.27	PF 1	1.7	12.5	12.7	12.7	12.79	0.053779	1.34	1.27	6.98	1.01	0.2	0.8
SP5	52.9937	PF 1	1.7	10.5	10.85	10.7	10.88	0.007343	0.71	2.41	7.75	0.4	0.35	0.65
SP5	46.9931	PF 1	1.7	10.5	10.7	10.7	10.79	0.053779	1.34	1.27	6.98	1.01	0.2	0.8
SP5	39.713	PF 1	1.7	8.5	8.84	8.7	8.87	0.008139	0.73	2.33	7.7	0.42	0.34	0.66
SP5	34.713	PF 1	1.7	8.5	8.7	8.7	8.79	0.05378	1.34	1.27	6.98	1.01	0.2	0.8
SP5	27.4329	PF 1	1.7	6.5	6.87	6.7	6.89	0.006337	0.67	2.53	7.83	0.38	0.37	0.63
SP5	19.5589	PF 1	1.7	6.5	6.7	6.7	6.79	0.053778	1.34	1.27	6.98	1.01	0.2	0.8
SP5	13.1211	PF 1	1.7	4.88	5	5.08	5.24	0.244207	2.16	0.79	6.62	2	0.12	0.88
SP5	5.2187	PF 1	1.7	4.68	4.88	4.88	4.97	0.053778	1.34	1.27	6.98	1.01	0.2	0.8
SP5	3.7199	PF 1	1.7	4.2	4.29	4.4	4.74	0.667053	2.95	0.58	6.46	3.16	0.09	N/A (PIC)
SP5	0	PF 1	1.7	2.07	2.5	2.27	2.52	0.003608	0.56	3.04	8.15	0.29	0.43	N/A (PIC)

Dome Perimeter Channel CHSP-6-1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 6 1	285.722	PF 1	0.99	35.38	35.69	35.65	35.78	0.001284	1.28	0.77	2.96	0.8	0.31	0.31
CHSP 6 1	87.417	PF 1	0.99	35.12	35.48		35.54	0.000775	1.08	0.92	3.08	0.63	0.36	0.52
CHSP 6 1	6.937	PF 1	0.99	35.01	35.44		35.48	0.0004	0.86	1.16	3.31	0.46	0.43	0.56
CHSP 6 1	0	PF 1	0.99	35	35.44		35.48	0.000377	0.84	1.18	3.33	0.45	0.44	0.56

Dome Perimeter Channel CHSP-6-2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Water depth (m)	Freeboard (m)
CHSP 6 2	395.95	PF 1	1.03	35.53	35.85	35.81	35.93	0.001328	1.31	0.78	2.94	0.81	0.32	0.15
CHSP 6 2	299.268	PF 1	1.03	35.4	35.72		35.8	0.0013	1.3	0.79	2.95	0.81	0.32	0.28
CHSP 6 2	109.156	PF 1	1.03	35.15	35.5		35.57	0.000927	1.16	0.89	3.05	0.69	0.35	0.5
CHSP 6 2	92.336	PF 1	1.03	35.12	35.48		35.55	0.000839	1.12	0.92	3.08	0.66	0.36	0.52
CHSP 6 2	6.944	PF 1	1.03	35.01	35.44		35.49	0.000432	0.89	1.16	3.31	0.48	0.43	0.56
CHSP 6 2	0	PF 1	1.03	35	35.44		35.48	0.000408	0.87	1.18	3.33	0.47	0.44	0.56

Spillway 6: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
SP6	230.78	PF 1	2.02	34.89	35.12	35.07	35.17	0.02581	1.04	1.95	9.14	0.72	0.23	0.77
SP6	229.4208	PF 1	2.02	34.85	35.03	35.03	35.12	0.053974	1.31	1.55	8.91	1	0.18	0.82
SP6	226.4102	PF 1	2.02	34.5	34.81	34.68	34.84	0.008918	0.74	2.73	9.56	0.44	0.31	0.69
SP6	222.3694	PF 1	2.02	34.5	34.68	34.68	34.77	0.053974	1.31	1.55	8.91	1	0.18	0.82
SP6	215.0892	PF 1	2.02	32.5	32.81	32.68	32.84	0.008795	0.74	2.75	9.56	0.44	0.31	0.69
SP6	211.0892	PF 1	2.02	32.5	32.68	32.68	32.77	0.053977	1.31	1.55	8.91	1	0.18	0.82
SP6	203.809	PF 1	2.02	30.5	30.81	30.68	30.84	0.008795	0.74	2.75	9.56	0.44	0.31	0.69
SP6	199.809	PF 1	2.02	30.5	30.68	30.68	30.77	0.053969	1.31	1.55	8.91	1	0.18	0.82
SP6	192.5288	PF 1	2.02	28.5	28.81	28.68	28.84	0.008795	0.74	2.75	9.56	0.44	0.31	0.69
SP6	188.5288	PF 1	2.02	28.5	28.68	28.68	28.77	0.053974	1.31	1.55	8.91	1	0.18	0.82
SP6	181.2486	PF 1	2.02	26.5	26.81	26.68	26.84	0.008795	0.74	2.75	9.56	0.44	0.31	0.69
SP6	177.2486	PF 1	2.02	26.5	26.68	26.68	26.77	0.053977	1.31	1.55	8.91	1	0.18	0.82
SP6	169.9688	PF 1	2.02	24.5	24.88	24.68	24.89	0.004714	0.6	3.36	9.88	0.33	0.38	0.62
SP6	157.4684	PF 1	2.02	24.5	24.68	24.68	24.77	0.053969	1.31	1.55	8.91	1	0.18	0.82
SP6	150.1886	PF 1	2.02	22.5	22.81	22.68	22.84	0.008795	0.74	2.75	9.56	0.44	0.31	0.69
SP6	146.1883	PF 1	2.02	22.5	22.68	22.68	22.77	0.053977	1.31	1.55	8.91	1	0.18	0.82
SP6	138.9085	PF 1	2.02	20.5	20.82	20.68	20.85	0.007924	0.71	2.84	9.61	0.42	0.32	0.68
SP6	133.9066	PF 1	2.02	20.5	20.68	20.68	20.77	0.053977	1.31	1.55	8.91	1	0.18	0.82
SP6	126.6265	PF 1	2.02	18.5	18.82	18.68	18.85	0.007924	0.71	2.84	9.61	0.42	0.32	0.68
SP6	121.6265	PF 1	2.02	18.5	18.68	18.68	18.77	0.053972	1.31	1.55	8.91	1	0.18	0.82
SP6	114.3464	PF 1	2.02	16.5	16.83	16.68	16.86	0.007147	0.69	2.93	9.66	0.4	0.33	0.67
SP6	108.3464	PF 1	2.02	16.5	16.68	16.68	16.77	0.053974	1.31	1.55	8.91	1	0.18	0.82
SP6	101.0663	PF 1	2.02	14.5	14.94	14.68	14.95	0.002748	0.51	4	10.2	0.26	0.44	0.56
SP6	72.0663	PF 1	2.02	14.5	14.68	14.68	14.77	0.053975	1.31	1.55	8.91	1	0.18	0.82
SP6	64.7862	PF 1	2.02	12.5	12.82	12.68	12.85	0.007927	0.71	2.84	9.61	0.42	0.32	0.68
SP6	59.7874	PF 1	2.02	12.5	12.68	12.68	12.77	0.053974	1.31	1.55	8.91	1	0.18	0.82
SP6	52.5078	PF 1	2.02	10.5	10.82	10.68	10.85	0.007924	0.71	2.84	9.61	0.42	0.32	0.68
SP6	47.5062	PF 1	2.02	10.5	10.68	10.68	10.77	0.053974	1.31	1.55	8.91	1	0.18	0.82
SP6	40.2261	PF 1	2.02	8.5	8.83	8.68	8.86	0.007147	0.69	2.93	9.66	0.4	0.33	0.67
SP6	34.2261	PF 1	2.02	8.5	8.68	8.68	8.77	0.053967	1.31	1.55	8.91	1	0.18	0.82
SP6	26.946	PF 1	2.02	6.5	6.85	6.68	6.87	0.006109	0.65	3.09	9.74	0.37	0.35	0.65
SP6	18.9569	PF 1	2.02	6.5	6.68	6.68	6.77	0.053975	1.31	1.55	8.91	1	0.18	0.82
SP6	12.5195	PF 1	2.02	5	5.12	5.18	5.33	0.22407	2.03	0.99	8.6	1.91	0.12	0.88
SP6	4.9933	PF 1	2.02	4.7	4.88	4.88	4.97	0.053973	1.31	1.55	8.91	1	0.18	0.82
SP6	0	PF 1	2.02	2.2	2.29	2.38	2.67	0.574617	2.72	0.74	8.45	2.93	0.09	N/A (PIC)

Dome Perimeter Channel CHSP-7-1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 7 1	183.47	PF 1	0.57	35.24	35.5	35.44	35.54	0.00087	0.94	0.61	2.77	0.64	0.26	0.5
CHSP 7 1	5.165	PF 1	0.57	35.01	35.45		35.46	0.000127	0.49	1.17	3.32	0.26	0.44	0.55
CHSP 7 1	0	PF 1	0.57	35	35.45		35.46	0.00012	0.48	1.2	3.34	0.25	0.45	0.55

Dome Perimeter Channel CHSP-7-2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
CHSP 7 2	447.437	PF 1	1.78	35.6	35.98	35.94	36.1	0.001328	1.5	1.19	3.65	0.84	0.38	0.02
CHSP 7 2	173.759	PF 1	1.78	35.23	35.62		35.73	0.001245	1.47	1.21	3.68	0.82	0.39	0.38
CHSP 7 2	2.262	PF 1	1.78	35	35.45		35.53	0.000793	1.26	1.41	3.84	0.66	0.45	0.55
CHSP 7 2	0	PF 1	1.78	35	35.45		35.53	0.000785	1.25	1.42	3.84	0.66	0.45	0.55

Spillway 7: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
SP7	234.3962	PF 1	2.35	34.89	35.13	35.09	35.2	0.025491	1.13	2.08	9.21	0.76	0.24	0.76
SP7	233.035	PF 1	2.35	34.85	35.05	35.05	35.15	0.047201	1.37	1.71	9.01	1.01	0.2	0.8
SP7	230.0244	PF 1	2.35	34.5	34.83	34.7	34.87	0.008638	0.8	2.94	9.66	0.46	0.33	0.67
SP7	225.8957	PF 1	2.35	34.5	34.7	34.7	34.8	0.047201	1.37	1.71	9.01	1.01	0.2	0.8
SP7	218.6126	PF 1	2.35	32.5	32.83	32.7	32.86	0.008778	0.8	2.92	9.65	0.47	0.33	0.67
SP7	214.6108	PF 1	2.35	32.5	32.7	32.7	32.8	0.047216	1.37	1.71	9.01	1.01	0.2	0.8
SP7	207.3277	PF 1	2.35	30.5	30.83	30.7	30.86	0.008779	0.8	2.92	9.65	0.47	0.33	0.67
SP7	203.3259	PF 1	2.35	30.5	30.7	30.7	30.8	0.047195	1.37	1.71	9.01	1.01	0.2	0.8
SP7	196.0428	PF 1	2.35	28.5	28.83	28.7	28.86	0.008778	0.8	2.92	9.65	0.47	0.33	0.67
SP7	192.0397	PF 1	2.35	28.5	28.7	28.7	28.8	0.04723	1.37	1.71	9.01	1.01	0.2	0.8
SP7	184.7579	PF 1	2.35	26.5	26.83	26.7	26.86	0.008775	0.8	2.92	9.65	0.47	0.33	0.67
SP7	180.753	PF 1	2.35	26.5	26.7	26.7	26.8	0.047216	1.37	1.71	9.01	1.01	0.2	0.8
SP7	173.4729	PF 1	2.35	24.5	24.9	24.7	24.92	0.004746	0.66	3.56	9.98	0.35	0.4	0.6
SP7	160.9661	PF 1	2.35	24.5	24.7	24.7	24.8	0.047195	1.37	1.71	9.01	1.01	0.2	0.8
SP7	153.6843	PF 1	2.35	22.5	22.83	22.7	22.86	0.008772	0.8	2.92	9.65	0.47	0.33	0.67
SP7	149.6762	PF 1	2.35	22.5	22.7	22.7	22.8	0.047216	1.37	1.71	9.01	1.01	0.2	0.8
SP7	142.8649	PF 1	2.35	20.5	20.84	20.7	20.87	0.00835	0.79	2.97	9.68	0.46	0.34	0.66
SP7	138.4214	PF 1	2.35	20.5	20.7	20.7	20.8	0.047216	1.37	1.71	9.01	1.01	0.2	0.8
SP7	131.1011	PF 1	2.35	18.5	18.84	18.7	18.87	0.007837	0.78	3.03	9.71	0.44	0.34	0.66
SP7	126.1007	PF 1	2.35	18.5	18.7	18.7	18.8	0.047215	1.37	1.71	9.01	1.01	0.2	0.8
SP7	118.8206	PF 1	2.35	16.5	16.84	16.7	16.87	0.007837	0.78	3.03	9.71	0.44	0.34	0.66
SP7	113.8206	PF 1	2.35	16.5	16.7	16.7	16.8	0.047219	1.37	1.71	9.01	1.01	0.2	0.8
SP7	106.5405	PF 1	2.35	14.5	14.97	14.7	14.98	0.002678	0.55	4.29	10.34	0.27	0.47	0.53
SP7	75.5405	PF 1	2.35	14.5	14.7	14.7	14.8	0.047208	1.37	1.71	9.01	1.01	0.2	0.8
SP7	68.2604	PF 1	2.35	12.5	12.85	12.7	12.88	0.007089	0.75	3.13	9.76	0.42	0.35	0.65
SP7	62.2604	PF 1	2.35	12.5	12.7	12.7	12.8	0.047219	1.37	1.71	9.01	1.01	0.2	0.8
SP7	54.9803	PF 1	2.35	10.5	10.85	10.7	10.88	0.007089	0.75	3.13	9.76	0.42	0.35	0.65
SP7	48.9803	PF 1	2.35	10.5	10.7	10.7	10.8	0.047219	1.37	1.71	9.01	1.01	0.2	0.8
SP7	41.7002	PF 1	2.35	8.5	8.87	8.7	8.89	0.006091	0.72	3.29	9.84	0.4	0.37	0.63
SP7	33.7002	PF 1	2.35	8.5	8.7	8.7	8.8	0.047212	1.37	1.71	9.01	1.01	0.2	0.8
SP7	26.4201	PF 1	2.35	6.5	6.87	6.7	6.9	0.005736	0.7	3.35	9.87	0.38	0.37	0.63
SP7	17.431	PF 1	2.35	6.5	6.7	6.7	6.8	0.047216	1.37	1.71	9.01	1.01	0.2	0.8
SP7	10.99	PF 1	2.35	5	5.13	5.2	5.38	0.229364	2.25	1.05	8.63	2.06	0.13	0.87
SP7	4.9306	PF 1	2.35	4.67	4.87	4.87	4.97	0.050393	1.4	1.68	8.99	1.04	0.2	0.8
SP7	0	PF 1	2.35	2.2	2.3	2.4	2.73	0.538013	2.92	0.8	8.49	3.03	0.1	N/A (PIC)

Dome Perimeter Channel CHSP-8-1: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Flow Depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
CHSP 8 - 1	12.2	PF 1	0.04	25.51	25.79	25.57	25.79	0.000011	0.1	0.4	1.86	0.07	0.28	0.21
CHSP 8 - 1	2.908	PF 1	0.04	25.5	25.79		25.79	0.000009	0.1	0.42	1.87	0.06	0.29	0.21
CHSP 8 - 1	0	PF 1	0.04	25.5	25.79		25.79	0.000099	0.1	0.42	1.87	0.06	0.29	0.21

Dome Perimeter Channel CHSP-8-2: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Water depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
CHSP 8 - 2	102.187	PF 1	0.28	25.63	25.87	25.81	25.91	0.000951	0.87	0.32	1.71	0.64	0.24	0.13
CHSP 8 - 2	3.478	PF 1	0.28	25.5	25.81		25.83	0.000352	0.61	0.46	1.93	0.4	0.31	0.19
CHSP 8 - 2	0	PF 1	0.28	25.5	25.8		25.82	0.004302	0.64	0.44	1.9	0.43	0.3	0.2

Spillway 8: HEC-RAS Hydraulic Analysis Results

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Water depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
SP8 R1	151.9214	PF 1	0.65	32.55	32.68	32.68	32.75	0.060964	1.11	0.58	4.67	1.01	0.13	0.37
SP8 R1	90.2069	PF 1	0.65	25.5	25.86	25.63	25.86	0.002134	0.37	1.75	5.78	0.22	0.36	0.14
SP8 R1	73.2469	PF 1	0.65	25.5	25.79		25.8	0.004485	0.48	1.36	5.44	0.3	0.29	0.21
SP8 R2	73.0969	PF 1	0.97	25.5	25.67		25.75	0.056288	1.25	0.77	4.87	1	0.17	0.33
SP8 R2	64.5383	PF 1	0.97	24.5	24.86	24.67	24.88	0.004544	0.55	1.77	5.81	0.32	0.36	0.14
SP8 R2	51.9971	PF 1	0.97	24.5	24.67	24.67	24.75	0.056295	1.25	0.77	4.87	1	0.17	0.33
SP8 R2	44.717	PF 1	0.97	22.5	22.8	22.67	22.82	0.008632	0.68	1.43	5.5	0.42	0.3	0.2
SP8 R2	40.717	PF 1	0.97	22.5	22.67	22.67	22.75	0.056295	1.25	0.77	4.87	1	0.17	0.33
SP8 R2	33.4369	PF 1	0.97	20.5	20.8	20.67	20.82	0.008632	0.68	1.43	5.5	0.42	0.3	0.2
SP8 R2	29.4369	PF 1	0.97	20.5	20.67	20.67	20.75	0.056293	1.25	0.77	4.87	1	0.17	0.33
SP8 R2	22.1568	PF 1	0.97	18.5	18.8	18.67	18.82	0.008632	0.68	1.43	5.5	0.42	0.3	0.2
SP8 R2	18.1568	PF 1	0.97	18.5	18.67	18.67	18.75	0.056292	1.25	0.77	4.87	1	0.17	0.33
SP8 R2	10.8767	PF 1	0.97	16.5	16.8	16.67	16.82	0.00869	0.68	1.43	5.5	0.43	0.3	0.2
SP8 R2	6.9276	PF 1	0.97	16.5	16.67	16.67	16.75	0.056283	1.25	0.77	4.87	1	0.17	0.33
SP8 R2	1.9556	PF 1	0.97	16.12	16.28	16.29	16.38	0.082257	1.42	0.68	4.78	1.19	0.16	0.34
SP8 R2	0	PF 1	0.97	14.96	15.95	15.13	15.95	0.000161	0.19	5.21	6.48	0.07	0.99	N/A (PIC)

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APPENDIX K

BRDA Side-Slope Closure Assessment

REPORT

BRDA Side Slope Closure Design at Stage 16

Surface Water Management Design

Submitted to:

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Distribution List

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Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

Golder Associates Ireland Limited (Golder) has been engaged by Aughinish Alumina Limited (AAL) to provide engineering design services for the closure of the Phase 1 and 2 Bauxite Residue Disposal Area (BRDA) proposed to be constructed to Stage 16. The Phase 1 BRDA is currently at Stage 10 and the Phase 2 BRDA is currently at Stage 4/5.

AAL requires that a engineering closure design for surface water management on the side slopes of the BRDA to be undertaken. The design will be used to inform visual impact assessments and support a strategic infrastructure development (SID) application for the proposed development of the BRDA to Stage 16.

This report considers the management of surface water runoff from the side slopes of the constructed Phase 1 and 2 BRDA to Stage 16 (crest elevation at 36 mOD) and assumes that the side slope capping containment and landscaping works from Stage 0 to Stage 16 have been completed at closure.

The objective of this report is to outline the methodologies used and outcomes from the engineering hydraulic engineering design of the side slope water management system. The outcomes of this report are intended to inform the wider side slope design requirements i.e., landscaping design, visual impact assessments and quantity and cost estimates, which are reported separately.

2.0 BACKGROUND

AAL is wholly owned by United Company RUSAL (UC RUSAL) and operates the alumina refinery situated on Aughinish Island on the south side of the Shannon estuary. AAL own a circa 601.22 ha. landholding (the Site) on Aughinish Island. The Island is predominantly rural in character with the remaining land usage comprising agriculture, single low density residential housing and protected habitats (wetlands and grasslands).

Aughinish Island is located on the south banks of the Shannon Estuary, at approximately 50km from the outlet to the North Atlantic, in the south-west of Ireland, and is bounded by the River Shannon to the north, the Robertstown River to the west and southwest and the Poulaweala creek to the east and southeast. The nearest towns are Askeaton (ca. 6.0 km to the east) and Foynes (ca. 3.5 km to the west) and the Site is located circa 30 km west of Limerick City.

The Phase 1 BRDA is located southwest of the process plant and is formed of two facilities: the original Phase 1 BRDA, which covers an area of 72 ha and the eastern Phase 1 BRDA Extension, which covers an area of 32 ha. The Phase 2 BRDA adjoins the southern extent of the Phase 1 BRDA and covers an area of 80 ha.

The BRDA is surrounded by PICs, which collect bleed water and runoff from the Phase 1 and Phase 2 facilities and currently convey it via pumps either to the Effluent Clarification System (ECS) or the Storm Water Pond (SWP). Both the ECS and the SWP are situated to the northeast of the Phase 1 BRDA. At closure, the PICs will be designed to discharge surface water off-site.

The design of drainage infrastructure for the BRDA dome is considered separately (Golder 2020). Drainage is intercepted at the upstream crest of Stage 16 by dome perimeter channels and discharged to the PIC via designated spillways.

3.0 CONCEPT AND CATCHMENT ANALYSIS

3.1 Side Slope Surface Water Management Design Concept

At closure, the BRDA side slopes will be capped with a rock fill capping containment layer which will provide a continuous rock fill blanket across the entire footprint of the BRDA side slopes. The rock fill blanket will comprise the rock fill from which the stage raises have been constructed and additional rock fill placed over the exposed bauxite residue benches, interconnecting from stage raise to stage raise.

The downstream faces of the rock fill stage raises will be vegetated by hydroseeding an approximately 100 mm depth of sub soil dressed on the upstream slope (to within 500 mm of the toe of the stage raise). The horizontal benches for each stage raise will have their rock fill capping containment layer (blanket) overlain by subsoil / topsoil layers and subsequently vegetated, However, a strip of the rock fill blanket (‘infiltration strip’) will remain exposed (i.e., not overlain with subsoil / topsoil or vegetated) which will allow surface water runoff to infiltrate into the rock fill blanket at each stage raise.

The side slope water management system has been designed to discharge runoff resulting from the Inflow Design Flood (IDF), from the side slopes to the PICs, through the following two processes:

- **Rock fill layer drainage:** The side slopes have been designed to accommodate infiltration of rainfall / surface water into the rock fill blanket (capping containment layer) through an infiltration strip, with propagation of the IDF flows through the continuous rock fill blanket to the PICs.
- **Stage raise overflow chutes:** The rock fill blanket drainage system, described above, is intended to be the primary means of discharging the IDF from the BRDA side slopes; however, a secondary overflow drainage system has been designed to allow controlled discharge of the IDF in the event that the rock fill blanket, or a meaningful section thereof, does not have sufficient drainage capacity to accommodate the IDF e.g. due to long term clogging of the rock fill blanket and/or infiltration strip(s). This overflow system will comprise riprap lined overflow chutes which will allow for the controlled discharge of flows from each stage raise to the stage raise below.

The design includes the division of the BRDA side slopes into segments 100m in width, with each segment forming an independent hydrological catchment i.e., runoff from a given segment is designed to be managed within that segment. The division of the BRDA side slope into catchments facilitates the controlled management of runoff from the side slopes and reduces the potential for runoff concentration along preferential flow pathways and localised overwhelming of the side slope drainage system.

For the purposes of this engineering design, one 100m width side slope segment has been analysed which is considered to provide side slope drainage design parameters for each stage raise, that are representative for the majority of the BRDA side slopes. This segment is located along the northern perimeter of the Phase 1 BRDA, where the BRDA side slopes have their maximum lengths and therefore segment catchment areas and design flow rates are at their maximum. At the detailed design stage, each side slope segment should be checked to ensure these design parameters remain representative for a given segment or are updated as necessary. For example, the geometry of the side slope at the south-western section of the Phase 1 BRDA is markedly different to the side slope geometries elsewhere around the BRDA perimeter, and therefore will likely require modification to the side slope drainage parameters.

Drawing 01 (Appendix A) presents a general layout plan of the BRDA and Drawing 02 presents a general arrangement plan for a typical side slope segment drainage design. The design includes staggering (lateral spacing) of the locations of the raise overflow chutes, as presented on Drawing 02. In the event of flood routing via these overflow chutes, their staggered locations will provide flood routing attenuation and increased times of concentration within the system; this is described further in Section 4.2.

3.2 Side Slope Water Management Catchment Analysis

3.2.1 IDF Rainfall Depth Estimation

Rainfall frequency analysis and the estimation of IDF rainfall depths for the BRDA at closure has been previously undertaken by Golder (Golder 2020).

For the operational phase of the facility, the BRDA has been classified as having a "**High**" hazard potential classification (HPC) rating under the Canadian Dam Association (CDA) Guidelines (CDA, 2014) and therefore, the IDF during this phase is 1/3 between the 1,000-year flood and the Probable Maximum Flood (PMF). For the closure phase of the facility, Golder considers that the BRDA classification would be reduced to a "**Significant**" HPC which also corresponds to an IDF 1/3 between the 1,000-year flood and the Probable Maximum Flood (PMF).

Table 1 presents the estimated IDF rainfall depths for the BRDA at closure for a range of storm durations; the methodology employed for this IDF analysis is described in detail in the Golder 2020 report.

Table 1: Estimated Design Rainfall Depths

Rainfall Duration (hour)	Rainfall Depths (mm)		
	1,000-year	PMP	IDF (1/3 between 1,000-year and PMP)
0.5	47.9	84.5	**60.1**
1	55.4	100.3	**70.4**
2	64.1	116.0	**81.4**
3	69.8	127.2	**89.0**
6	80.9	144.7	**102.2**
9	88.1	163.5	**113.3**
12	93.6	174.6	**120.6**
24	107.6	208.0	**141.0**

3.2.2 Hydrological Catchment Analysis

As described in Section 3.1, the BRDA side slopes will be divided into 100m wide segments, with each segment forming an independent hydrological catchment. The typical design details for the side slope segment breaks are presented on Drawing 03. One side slope segment has been analysed herein, which is considered to provide side slope drainage design parameters for each stage raise, that are representative for the majority of the BRDA side slopes.

The hydrological properties for each stage raise within the representative segment have been analysed and are presented in Table 2 and Table 3 below.

Hydrological properties for each independent stage raise are required for the assessment of the rock fill layer drainage capacity (Table 2); while cumulative hydrological properties for each stage raise are required for the assessment of the stage raise overflow chute design requirements (Table 3).

Lag times / times of concentration were estimated using the Watershed Lag Method, developed by the National Resources Conservation Service (NRCS, 2010). The curve number applicable for the side slopes at closure is estimated to be 82 which corresponds to a contoured row crop ground cover, on hydrologic soil group C, with good hydrologic condition (NRCS, 1986). These hydrological properties have been used to estimate IDF runoff rates for the design of the rock fill layer and stage overflow chutes, as described in Section 4.0.

Note: *No catchment analysis has been undertaken for Stage 16 for the side slope drainage as runoff from the catchment area upgradient of Stage 16 will be intercepted by the proposed dome perimeter channel and discharged to the PICs via spillways (Golder 2020). However, the crest surface (4 m plan length) and the downstream slope (3 m plan length) for the Stage 16 embankment raise have been included in the Stage 15 catchment data presented below.*

Table 2: BRDA Side Slope Catchment (100m width segment): Independent Stage Raise Hydrological Properties

Stage Raise	Area (m^2)	Flow Length (m)[1]	Lag Time (mins)	Time of Concentration (mins)
15	1,500	80	3.9	6.5
14	1,100	76	3.6	6.1
13	1,100	76	3.6	6.1
12	1,100	76	3.6	6.1
11	1,100	76	3.6	6.1
10	1,950	85	4.2	6.9
9	1,100	76	3.6	6.1
8	1,200	77	3.7	6.2
7	1,200	77	3.7	6.2
6	1,300	78	3.8	6.3
5	3,600	101	5.3	8.8
4	1,300	78	3.8	6.3
3	1,300	78	3.8	6.3
2	1,500	80	3.9	6.5
1	1,600	81	3.9	6.6
0	1,300	78	3.8	6.3

Notes:

1. *Calculated assuming an average lateral distance between successive raise overflow chutes of 65m.*

Table 3: BRDA Side Slope Catchment (100m width segment): Cumulative Stage Raise Hydrological Properties

Stage Raise	Cumulative Area (m^2)	Cumulative Flow Length (m)[1]	Lag Time (mins)	Time of Concentration (mins)
15	1,500	80	3.9	6.5
14	2,600	156	6.5	10.9
13	3,700	232	8.9	14.9
12	4,800	308	11.2	18.7
11	5,900	384	13.3	22.2
10	7,850	469	15.8	26.3
9	8,950	545	17.8	29.6
8	10,150	622	19.7	32.9
7	11,350	699	21.6	36.1
6	12,650	777	23.6	39.3
5	16,250	878	26.3	43.9
4	17,550	956	28.2	46.9
3	18,850	1,034	29.9	49.9
2	20,350	1,114	31.8	53.0
1	21,950	1,195	33.7	56.1
0	23,250	1,273	35.4	59.0

Notes:

1. *Calculated assuming an average lateral distance between successive raise overflow chutes of 65m.*

4.0 HYDRAULIC DESIGN

As described in Section 3.1, the side slope water management system has been designed to discharge runoff, resulting from the Inflow Design Flood (IDF), from the side slopes to the PICs via two drainage mechanisms i.e., a primary drainage system through the rock fill blanket and a contingency drainage system via overflow chutes. The hydraulic design of these side slope drainage mechanisms is described in Sections 4.1 and 4.2, respectively.

During the design process, the potential consequences resulting from a flood event of greater magnitude to the IDF have been considered qualitatively. If such a (low probability and high magnitude) flood event were to occur which resulted in the side slope drainage system being overwhelmed, it is expected that damage to the side slope landscaping may occur. However, significant damage to the BRDA side slopes (e.g., significant erosion of raise embankments) is not anticipated for such a scenario as the continuous rock fill blanket (present across the entire footprint of the BRDA side slopes) would act as an erosion protection layer. Furthermore, the overflow chutes have been designed for peak flow rates that do not consider attenuation storage throughout the side slope drainage system; in reality some flood attenuation would be available and therefore the overflow chutes would accommodate peak flow rates in excess of the IDF.

4.1 Rock Fill Blanket Drainage

4.1.1 Rock Fill Blanket Drainage Capacity

The drainage capacity of the rock fill blanket was assessed by applying Wilkins' equation (Wilkins 1956) for non-Darcy flow through porous media, in accordance with methodologies set out by Garga et al. (1990).

For the purposes of this assessment the following assumptions have been made regarding the material properties of the rock fill blanket and the application of Wilkins' equation:

- Rock fill blanket porosity = 25%;
- Relative surface area efficiency of particles (R_{SAE}) = 1.22;
- Wilkins' equation 'N' parameter = 1.852; and
- Minimum rock fill blanket invert gradient = 1%.

Historical rock fill particle size distribution (PSD) data for rock fill material used to construct the existing BRDA rock fill embankment raises have been assessed and are considered to be representative of the rock fill material that is likely to be used to construct further embankment raises and the BRDA side slope capping containment layer. These historical rock fill PSD data sets (14 in total) are presented in Appendix B.

For each PSD dataset, a weighted average hydraulic mean radius for the rock fill was estimated and from these a range of rock fill layer drainage capacity estimates were calculated. The hydraulic mean radius parameter is a measure of the average pore size through which flow takes place (Garga et al., 1990).

Table 4 presents a summary of the estimated rock fill blanket flow capacities for varying layer thickness.

Table 4: Rock Fill Blanket Flow Capacity Summary Data

Weighted Mean Hydraulic Radius		Void Velocity (m/s)	Bulk Velocity (m/s)	Rock fill Blanket Drainage Capacity (per 100m width) (L/s)		
Statistic	Dimension (mm)			0.3m Thickness	0.4m Thickness	0.5m Thickness
Minimum	**1.18**	**0.015**	**0.0038**	**113**	**150**	**188**
25th Percentile	2.06	0.020	0.0049	148	198	247
Median	2.85	0.023	0.0058	175	233	291
75th Percentile	3.04	0.024	0.0060	180	240	301
Maximum	3.35	0.025	0.0063	189	252	315

For the purposes of this assessment, the minimum calculated hydraulic mean radius (1.18 mm) and the corresponding estimated flow capacities were selected as the design basis for the rock fill blanket. This approach yields a Factor of Safety (FoS) for the rock fill blanket drainage capacity of > 1.5 when compared with the flow capacities associated with the median calculated mean hydraulic radius (i.e., median of the weighted average hydraulic mean radii calculated for each of the 14 PSD data sets).

4.1.2 Routing Assessment of IDF Through Rock fill Blanket

Golder has undertaken a flood routing analysis for the proposed BRDA side slope rock fill blanket drainage system, using the US Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modelling System (HEC-HMS) software (USACE 2020).

The objective of this assessment is to simulate routing of the IDF progressively through the rock fill blanket and to inform the rock fill blanket thickness requirements to accommodate discharge of the IDF to the PICs, without surface discharge via the contingency overflow chutes (see Section 4.2). One side slope drainage segment (100 m width) has been assessed. The conceptual hydrologic model is shown in Figure 1 below.

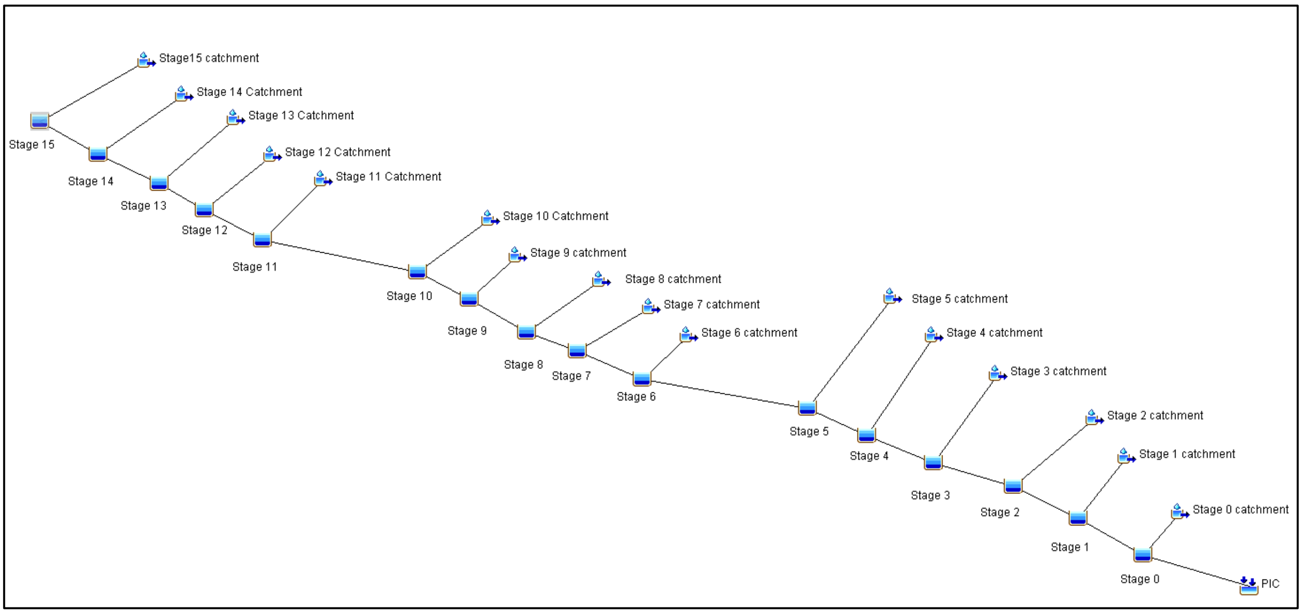

Figure 1: Conceptual HEC-HMS Rock fill Layer Flood Routing Model

The model includes 16 No. reservoir elements representing the storage / attenuation capacity at each stage raise i.e., storage along the infiltration strips from Stages 0 to 15, and 16 No. sub-basin elements which simulate runoff generation from the catchment area for each stage. The model estimates catchment runoff using the SCS curve number method (USDA 1986), utilising the stage raise catchment hydrological properties presented in Table 2.

The modelling assessment was undertaken for three storm durations (0.5-hour, 1-hour, and 2-hour) so that the critical storm duration for each stage raise is assessed. The IDF rainfall depth for these storm durations are provided in Table 1. The modelled rainfall time series for each storm event was established by applying Huff's dimensionless mass curves (1st quartile curve) (Mays 2001) to the IDF rainfall depths.

As shown on Drawing 03, the BRDA side slope and overflow chute designs allow fora maximum surface water ponding depth of 0.25 m along each stage raiser infiltration strip. Surface flow via the stage raise overflow chutes would commence should the ponding depth along the infiltration strip exceed 0.25 m. The primary design outcomes from the flood routing analysis are presented in Table 5 below.

Table 5: Rock fill Blanket Flood Routing Modelling Results Summary

Stage Raise ID	Rock fill Blanket Thickness Design Requirement (mm)	Maximum Simulated Surface Water Ponding Depth (mm)
0 to 4	400	231
5[1]	300	111
6 to 8	400	158
8 to 15	300	153

Notes:

1. *The Stage 5 rock fill blanket thickness is designed at 300 mm to provide flood attenuation (reduced flow capacity compared to 400 mm) and utilise the attenuation potential of this Stage which is far greater than that of other Stages due to its long bench length. This approach also reduces the overall rock fill volume requirement for the side slopes.*

Golder understands that as part of the side slope landscape design, landscape variations (i.e., planting mounds) are intended to be constructed within the boundaries of each side slope segment; these mounds may span a number of raises and resulting in localised infilling of sections of the rock fill infiltration strips (see Drawing 04 for typical detail).

Golder has assessed the maximum allowable width of these planting mounds that could be constructed at each stage, without compromising the capacity of the rock fill drainage system to accommodate the IDF (i.e., without surface discharge via the overflow chutes). Table 6 presents the maximum allowable planting mound widths, which corresponds with the maximum length of infiltration strip that can be infilled. The profile of these mounds should be constructed so that surface water runoff from the mounds then reports to the adjacent stage raise landscaping / infiltration strip, and therefore does not drain to downgradient stages.

Note: *The landscape variation planting mounds will not be constructed at locations along the stage raise overflow path (i.e., surface flow path between overflow chutes – see Drawing 02), as this would compromise the design of the overflow chute drainage system design.*

Table 6: Maximum Allowable Planting Mound Widths per 100m Side Slope Segment

Stage(s)	Maximum Allowable Planting Mound Base Width (m) [1]
0	5
1 to 2	10
3 to 4	20
5	5
6 to 7	20
8	10
9 to10	20
11	30
12 to 15	40

Notes:

1. *Maximum allowable base width includes the width of mounds forming segment breaks.*

Based on the design parameters presented herein, the results of this modelling assessment demonstrate that a BRDA side slope rock fill blanket can fully accommodate discharge of the IDF from each stage raise to the PIC's. The rock fill blanket thickness required to achieve this are presented in Table 5, for each stage raise.

4.2 Stage Raise Overflow Chutes

4.2.1 Design Layout

The BRDA side slope drainage system has been designed to accommodate discharge of the IDF from each stage raise to the PICs via flow through the proposed rock fill capping containment layer and rock fill stage raise embankments (see Sections 3.1 and 4.1). However, the design also incorporates a contingency surface flow drainage system via a series of stage raise overflow chutes which will allow controlled discharge of the IDF, in the event that the rock fill layer (or a meaningful section thereof) does not have sufficient drainage capacity to accommodate the IDF e.g., due to long term clogging of the rock fill blanket and/or the infiltration strip(s).

Within each side slope segment, one riprap lined overflow chute will be located on each stage raise to facilitate controlled discharge of the IDF to the stage raise directly below. The design layout for this system is shown on Drawing 02. The lateral spacing of the overflow chutes has been designed to be staggered in order to:

- Attenuate the IDF within the surface flow system and increase catchment times of concentrations by increasing the length of the surface flow paths;
- Increase the potential for infiltration of surface water along the stage infiltration strips; and
- Minimise potential visual impacts associated with the overflow chutes.

The lateral spacing of the overflow chutes from Stages 0 to 15 has been designed based on the following criteria:

- Minimum lateral spacing between successive overflow chutes = 50m; and
- Average lateral spacing between all overflow chutes within each side slope segment = 65m.

Note: *A reduction of the designed average lateral chute spacing below 65m may adversely affect the design outcomes as catchment times of concentration would reduce and may result in increased design flow rates.*

4.2.2 Hydrological Modelling and Chute Sizing

To estimate the IDF design flow rates for each overflow chute, Golder developed a hydrologic model for each stage raise catchment using HEC-HMS software. The model estimates total catchment runoff for each stage (within the side slope segment) using the SCS curve number method (USDA 1986), utilising the (cumulative) stage raise catchment hydrological properties presented in Table 3. As per Section 4.1.2, the modelling assessment was undertaken for storm durations ranging from 0.5 hour to 2 hours, and the modelled rainfall time series for each storm event was established by applying Huff's dimensionless mass curves (Mays 2001) to the IDF rainfall depths.

As shown on Drawing 03, the spill elevation of each overflow chute is set at 0.25m above the invert of the adjacent stage infiltration strip. This provides flood attenuation storage along each raise and also encourages infiltration and drainage into the rock fill blanket. The overflow chutes will be locally cut into the rock fill stage raise embankments and constructed at a slope of 2.5(H):1(V).

Chute sizing requirements (widths) were determined using the broad crested weir equation (USACE, 2000) with a maximum flow depth at the upstream end of the chute of 0.15m (corresponding with a maximum ponded water elevation equal to the elevation of the top of the landscape berm sub soil).

For ease of construction, the stage overflow chute design widths have been clustered into a series of standard design widths. Table 7 presents the designed chute sizes and design peak flow rates considered for each stage raise.

Table 7: Overflow Chute Design Flow Rates and Widths

Stage Raise ID	Design Flow Rate (m^3/s)	Overflow Chute Width (m)
0 to 2	0.181	2.0
3 to 5	0.161	1.75
6	0.124	1.5
7 to 9	0.117	1.25
10 to 11	0.096	1.0
12 to 13	0.070	0.75
14 to 15	0.047	0.5

4.2.3 Chute Riprap Sizing and Energy Dissipation Requirements

The stage overflow chute riprap lining has been designed in accordance with guidance developed by Robinson et. al. (Robinson et al. 1998), which provides a methodology for sizing the median or D_{50} rock size for chutes based on the chute slope and unit flow rate (flow rate per unit width).

These calculations were undertaken for each standard chute design size and corresponding design flow rate (see Table 7) considering a 2.5(H):1(V) chute slope and including a Factor of Safety (FoS) for the D_{50} rock size of 1.4. The resulting chute riprap sizing design requirements are as follows:

- Riprap material = angular rock;
- D_{50} (median) riprap rock size = 150 mm; and
- Riprap layer thickness along sloped section of chute (2 x D_{50}) = 300 mm.

A reduced riprap layer thickness of 250 mm at the upstream end of the chutes (horizontal section) is considered appropriate given the flow velocities along this section of each chute will be small.

The riprap gradation requirements were assessed in accordance with the United States Department of Agriculture Natural Resources Conservation Service design procedures for rock-lined chutes (USDA 2018) and are presented in Table 8 below.

Table 8: Overflow Chute Riprap Gradation Requirements

Passing by Weight (%)	Lower Envelope Gradation (mm)	Upper Envelope Gradation (mm)
100	225	300
85	195	270
50	150	225
10	120	195

As shown on Drawing 02, the 300 mm thickness chute riprap will be extended at the base of each chute to form an energy dissipation zone; the riprap will also extend laterally along the infiltration strip beyond the extent of the chute by a minimum of 2.25 m i.e., 15 x $D_{50,}$ in accordance with recommendations by Robinson et. al. (1998). The riprap lining will be extended up the slope of the landscape berm to the elevation of the top of the sub soil layer i.e., 400mm above the invert of the receiving infiltration strip. The maximum estimated hydraulic jump depth within the energy dissipation zone is approximately 185 mm.

<u>Note:</u> *Drawing 02 provides a typical detail for the energy dissipation zone where the chute is located at a laterally centralised location within the side slope segment i.e., not adjacent to a side slope segment break, where discharge can propagate away from the chute in two directions. However, for chutes located adjacent to side slope segment breaks from stage raises 0 to 6, an increased energy dissipation zone base width of 0.75m (compared to 0. m elsewhere) is required to facilitate the hydraulic jump. This is due to the propagation of flows being restricted to one direction and is applicable only to Stages 0 to 6 (due to the increased design flow rates for these stages compared to stages at higher elevations). The increased base width of the energy dissipation zone and infiltration strip should extend to a minimum distance of 5m from the chute location.*

5.0 CLOSING REMARKS

Golder has completed an engineering design for surface water management of the BRDA side slopes to allow controlled discharge of runoff from the BRDA side slopes to the PICs during closure. The design will be used to inform visual impact assessments and support a strategic infrastructure development (SID) application for the proposed development of the BRDA to Stage 16.

Further design detailing and design information would be required to progress these designs to ‘tender’ or ‘construction’ stage designs. The design information presented herein is subject to a number of assumptions/ limitations which should be checked / validated during detailed design, and designs updated as necessary; these include:

- One side slope segment of 10 m width has been analysed which is considered to provide side slope drainage design parameters for each stage raise that are representative for the majority of the BRDA side slopes (see Section 3.1). Validation of the applicability of this design parameter to each proposed side slope segment should be undertaken, and updates made where necessary e.g., due to significant changes to side slope geometry.
- The stage raise overflow chutes have been designed assuming an average lateral chute spacing of 65 m. A reduced average lateral spacing may adversely affect the design outcomes as catchment times of concentration would reduce and may result in increased design flow rates (See Section 4.2.1). Validation of overflow chute spacing, and designs should be undertaken for each side slope segment, and updates made if necessary.

Indicative variations to the landscape berms and vehicle access requirements for the Stage raises are shown on Drawing 04.

Final landscaping details for the landscape berms (Drawing 03) and the variations (Drawing 04) will be provided by BSM (AAL nominated landscape architect consultant).

6.0 REFERENCES

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Garga, V.K., Hansen, D., Townsend, R.D., (1990). Considerations on the Design of Flowthrough Rock fill Drains, Proceedings of the 14th Annual British Columbia Mine Reclamation Symposium in Cranbrook, BC, 1900. The Technical and Research Committee on Reclamation.

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Wilkins, J.K., (1956). The flow of water through rock fill and its application to the design of dams. Proc. 2nd Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering, pp.141-149

Signature Page

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Date: October 2020

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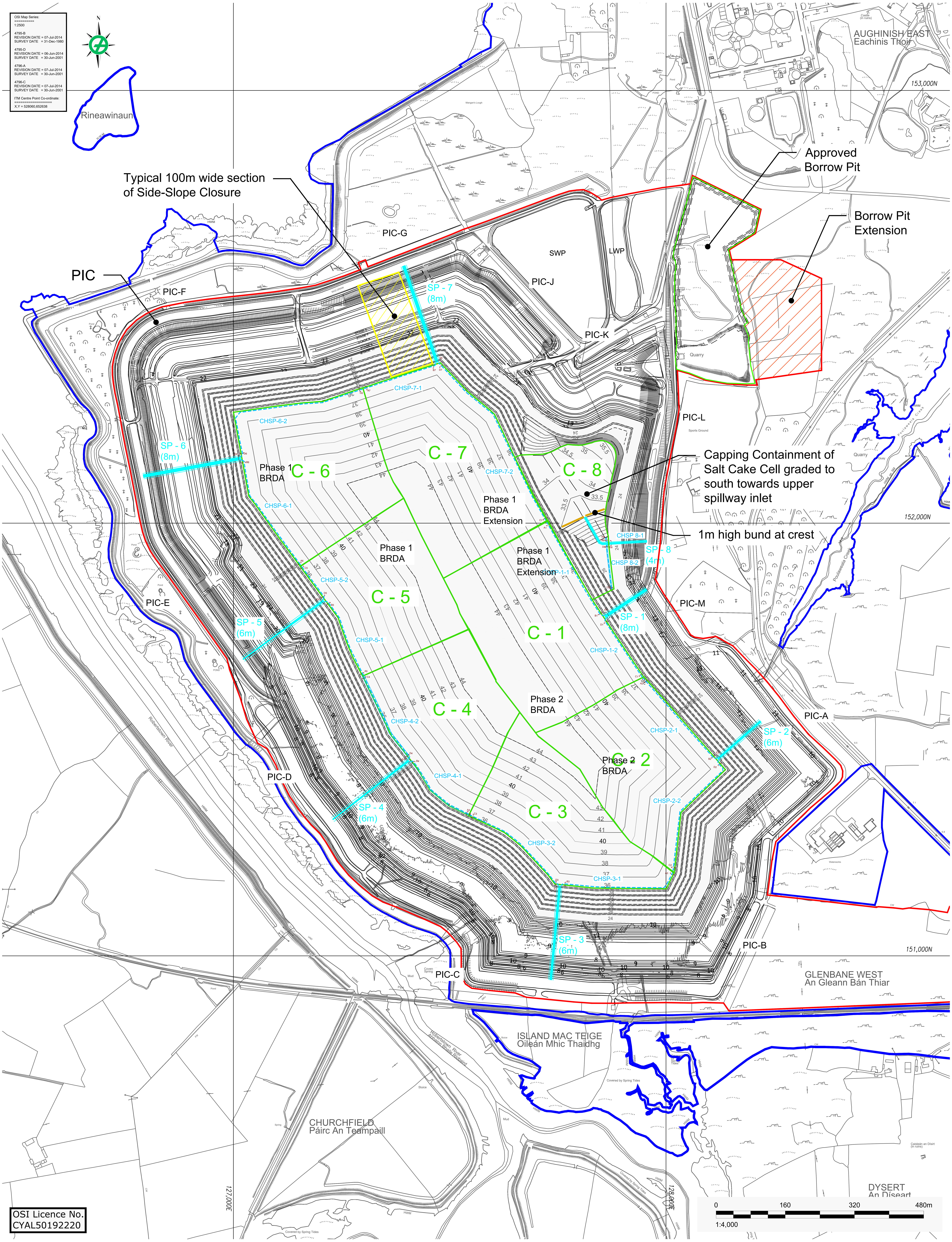
Directors: S. Copping, A. Harris, DRV Jones, A.L. Oberg-Hogsta
VAT No.: 8297875W

APPENDIX A

Drawings



LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- SPILLWAY LOCATION
- DOME PERIMETER CHANNEL
- CATCHMENT BOUNDARY
- 22 OGL CONTOUR (mOD)
- APPROVED BORROW PIT
- PROPOSED BORROW PIT EXTENSION
- BOUNDARY FOR TYPICAL SEGMENT: 100m WIDTH OF SIDE-SLOPE CLOSURE

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES & TO O.S. DATUM.

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER MEMBER OF WSP

YYYY-MM-DD	2021-Aug-11
PREPARED	POB
DESIGN	DH/BK
REVIEW	CC
APPROVED	BK/DH

PROJECT

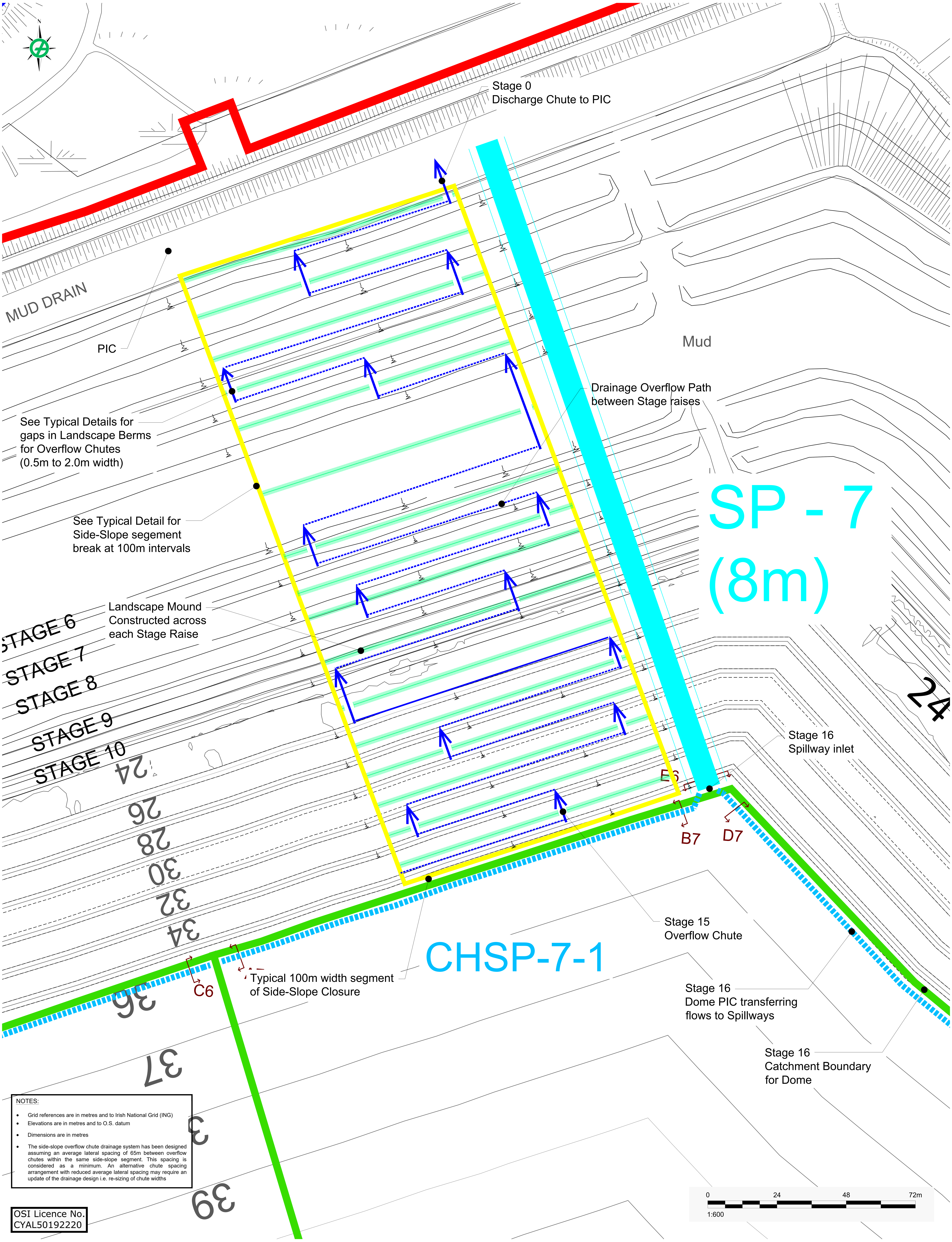
BRDA SIDE-SLOPE CLOSURE DESIGN AT STAGE 16

TITLE

BRDA LOCATION PLAN AT CLOSURE

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	01	A	1:4,000 A1

25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1



NOTES:

- Grid references are in metres and to Irish National Grid (ING)
- Elevations are in metres and to O.S. datum
- Dimensions are in metres
- The side-slope overflow chute drainage system has been designed assuming an average lateral spacing of 65m between overflow chutes within the same side-slope segment. This spacing is considered as a minimum. An alternative chute spacing arrangement with reduced average lateral spacing may require an update of the drainage design i.e. re-sizing of chute widths

OSI Licence No.
CYAL50192220

LEGEND

AAL OWNERSHIP BOUNDARY
APPLICATION BOUNDARY
SPILLWAY LOCATION
DOME PERIMETER CHANNEL
CATCHMENT BOUNDARY
22 OGL CONTOUR (mOD)
BOUNDARY FOR TYPICAL SEGMENT: 100m WIDTH OF SIDE-SLOPE CLOSURE
LANDSCAPE MOUNDS
STAGE RAISE DRAINAGE OVERFLOW PATH
OVERFLOW CHUTE BETWEEN STAGE RAISES I.E. GAP IN THE LANDSCAPE BERM

CLIENT
AUGHINISH ALUMINA LTD.

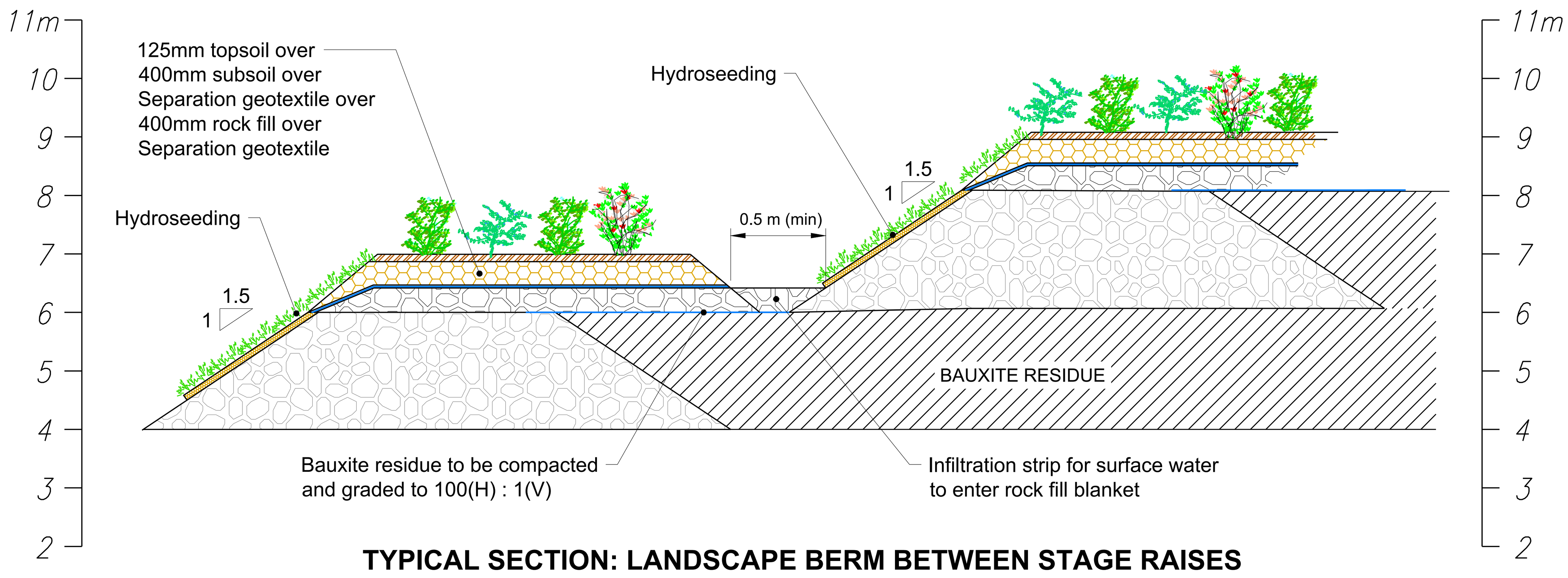
CONSULTANT
GOLDER

YYYY-MM-DD	2021-Aug-11
PREPARED	POB
DESIGN	DH/BK
REVIEW	CC
APPROVED	BK/DH

PROJECT
BRDA SIDE-SLOPE CLOSURE DESIGN AT STAGE 16

TITLE
TYPICAL SEGMENT: 100m WIDTH OF SIDE-SLOPE CLOSURE

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	02	A	1:600 A1



TYPICAL SECTION: LANDSCAPE BERM BETWEEN STAGE RAISES
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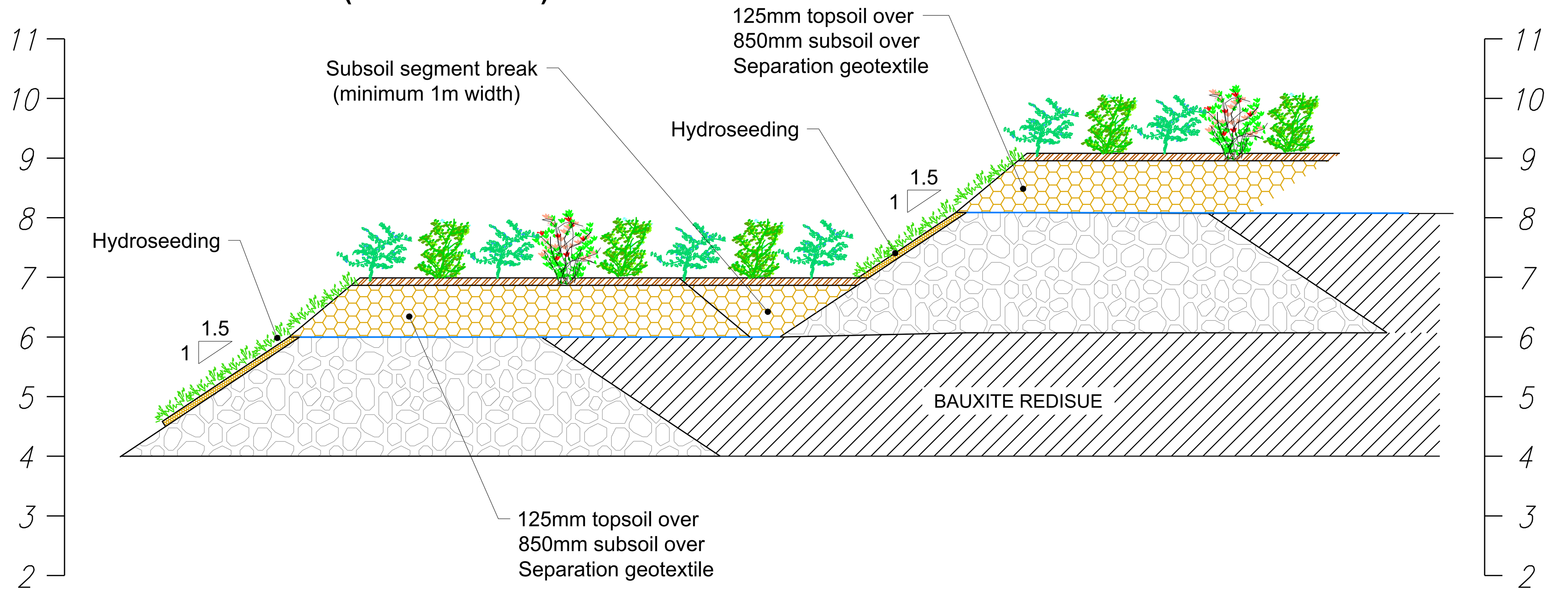
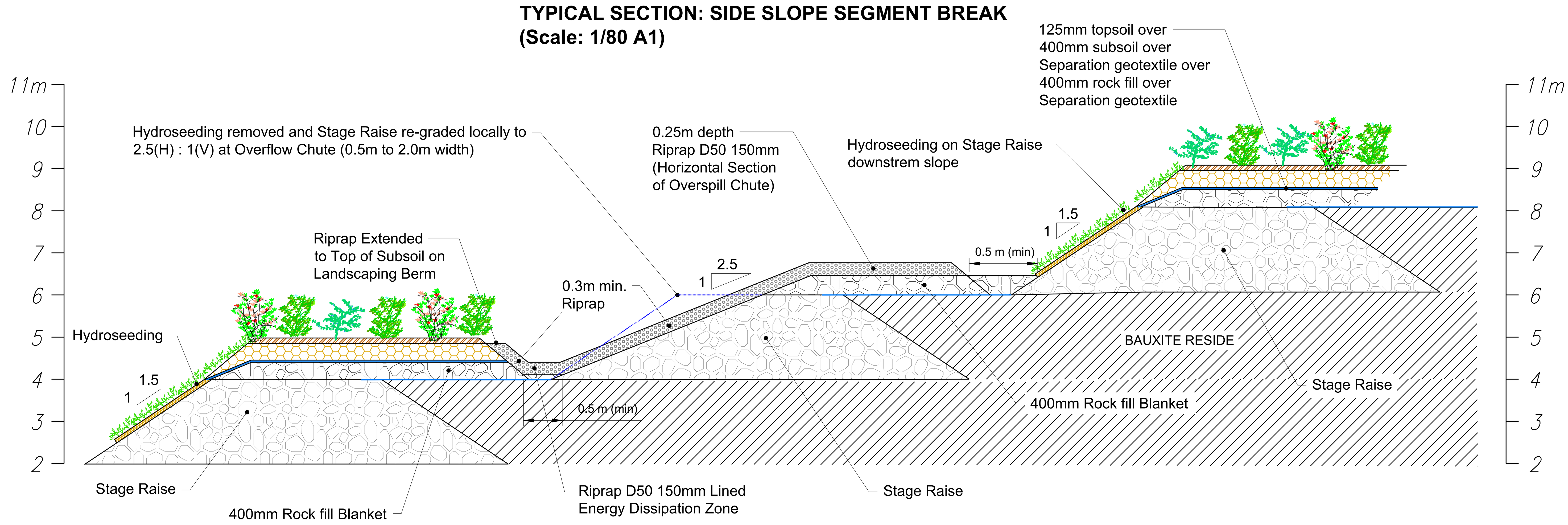
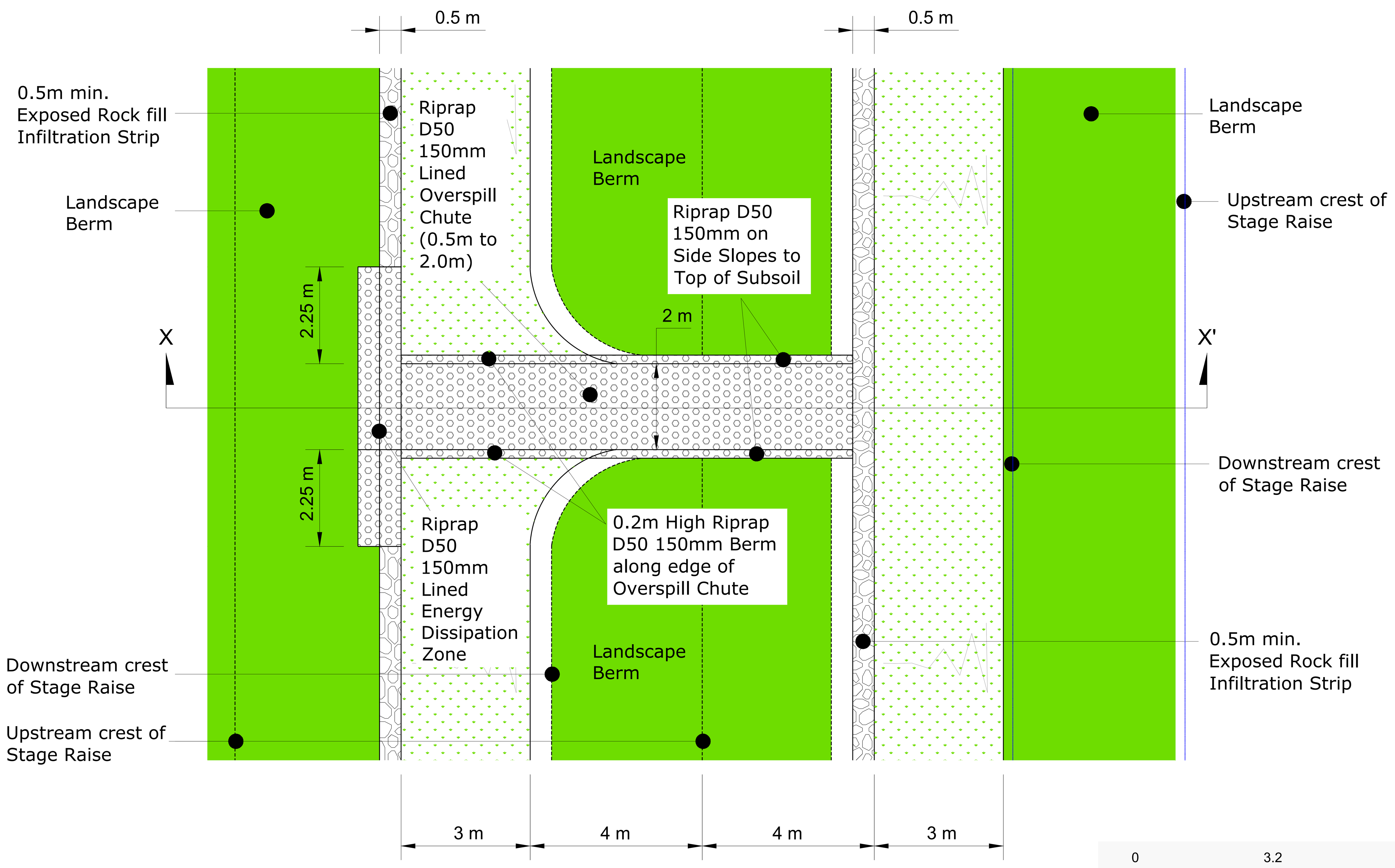
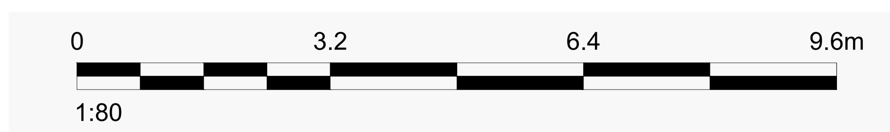


TYPICAL SECTION: SIDE SLOPE SEGMENT BREAK
(Scale: 1/80 A1)



TYPICAL SECTION X-X': OVERFLOW CHUTE BETWEEN STAGE RAISES
(Scale: 1/80 A1)





TYPICAL PLAN: OVERFLOW CHUTE BETWEEN STAGE RAISES
(Scale: 1/80 A1)

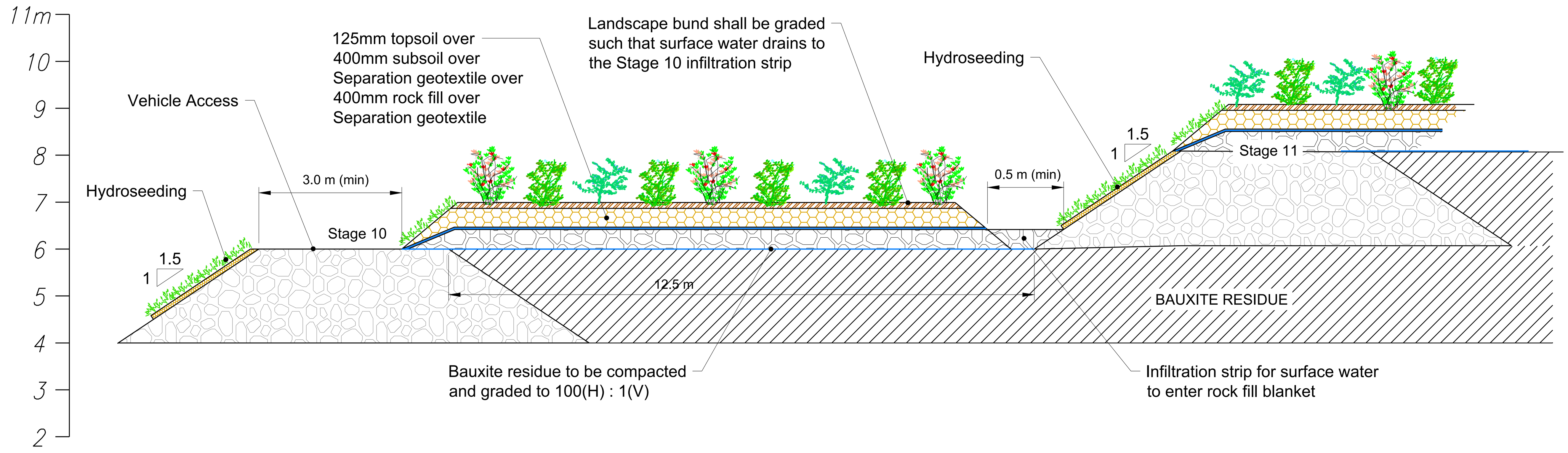
NOTES:

- Overflow chute widths vary from 0.5m to 2.0m, depending on the Stage Raise elevation.
  See design report for information on widths for respective Stage Raises.
  A 2.0m width chute is shown in the Typical Plan detail.
- Rock fill blanket depth varies between 0.3m and 0.4m thicknesss, depending on Stage Raise elevation.
  See design report for information on depths for respecitve Stage Raises.
  A 0.4m depth of Rock fill blanket is shown on the Typical Sections
- Bauxite residue benches between Stage Raises shall be compacted and graded at 100(H) : 1(V)
- Infiltration strip width for Stage 5 is 1.5m and is 0.5m for all other Stages.
- Vehicle access on Stage Raise crests shall be maintained at Stage 0, Stage 5, Stage 10 and Stage 16.
  See Drawing 04 for typical details and variations and Drawing 05 for long slope section.
- Subsoil segment breaks shall be a minimum of 1m plan width
- Landscape details shown are indicative. Final landscaping design to be provided by BSM

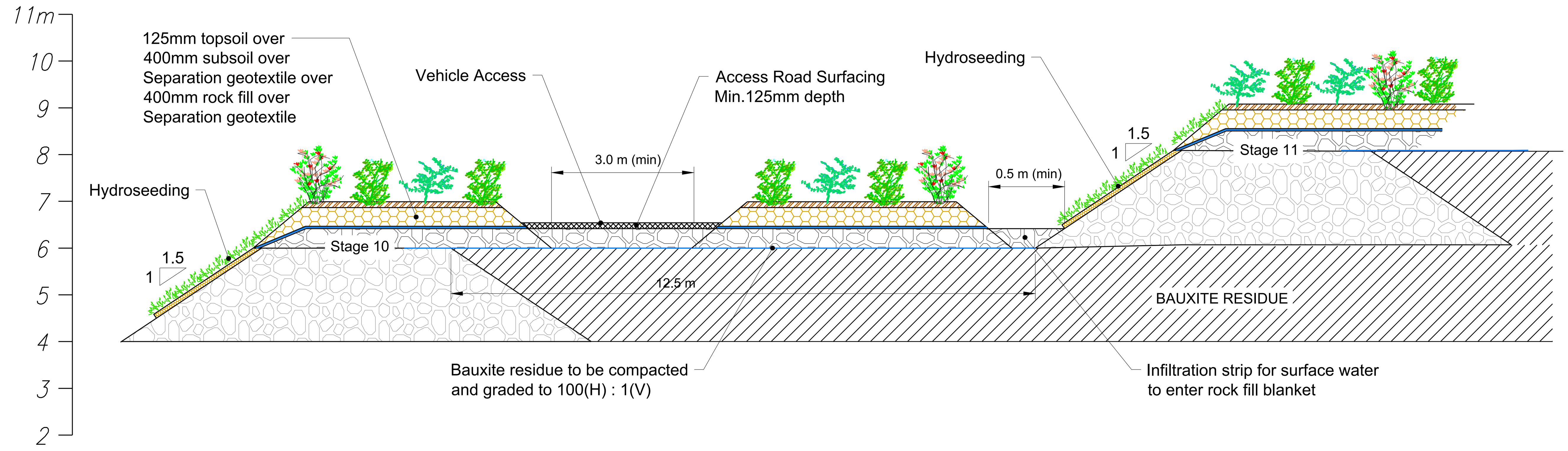
CLIENT	AUGHINISH ALUMINA LTD.	PROJECT	BRDA SIDE-SLOPE CLOSURE DESIGN AT STAGE 16
CONSULTANT	GOLDER	TITLE	SIDE-SLOPE CLOSURE: TYPICAL DETAILS

YYYY-MM-DD	2021-Aug-11
PREPARED	POB
DESIGN	DH/BK
REVIEW	CC
APPROVED	BK/DH

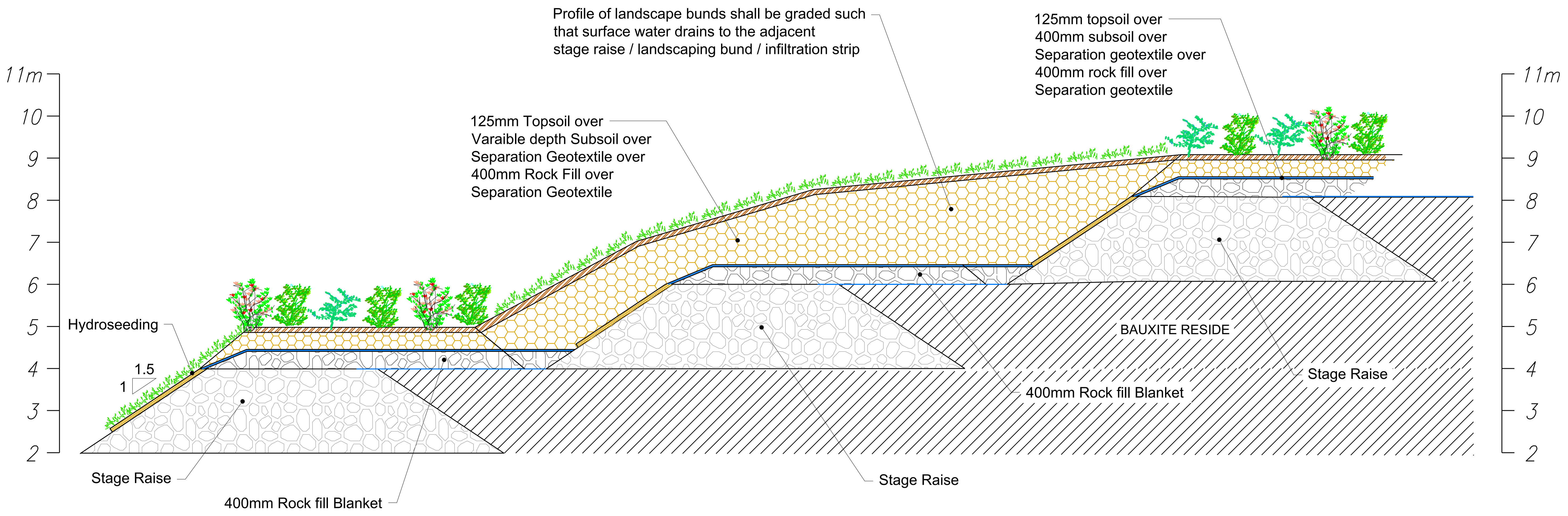
PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	03	A	1:80 A1



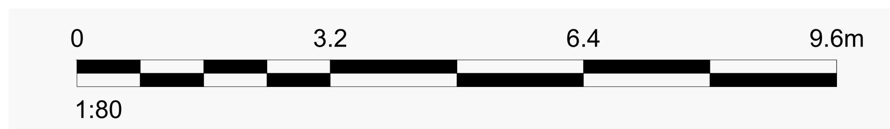
TYPICAL SECTION: VEHICLE ACCESS ON STAGE RAISE (STAGE 10)
(Scale: 1/80 A1)

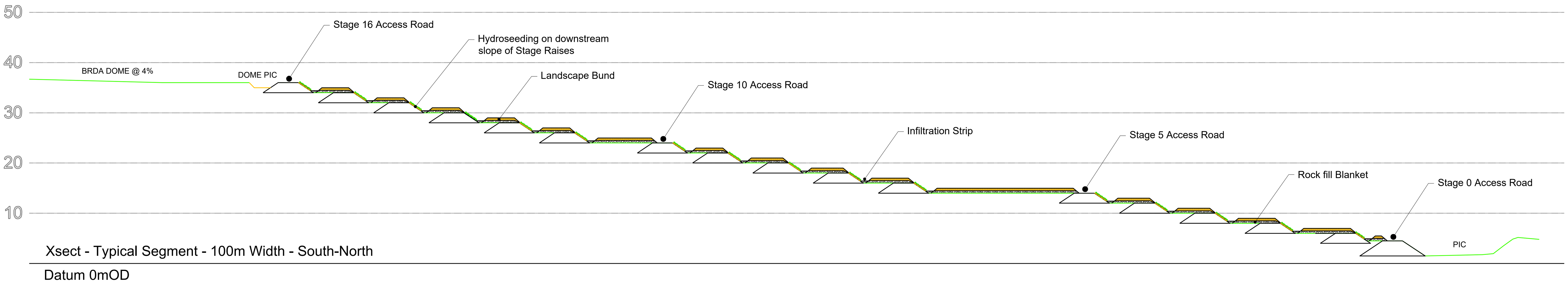


TYPICAL SECTION: VARIATION FOR VEHICLE ACCESS ON STAGE RAISE (STAGE 10)
(Scale: 1/80 A1)

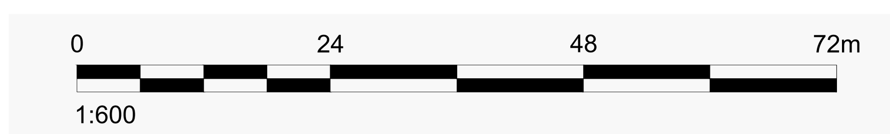


TYPICAL SECTION: LOCAL VARIATION FOR PLANTING BUNDS SPANNING A NUMBER OF STAGE RAISES
(Scale: 1/80 A1)





TYPICAL SECTION: LONG SECTION OF 100m WIDTH SEGMENT FROM STAGE 0 TO STAGE 16
(Scale: 1/600 A1)



NOTES:

- Infiltration strip width is 1.5m width for Stage 5 and 0.5m width for all other Stages
- Vehicle Access on Stage Raise crests shall be maintained at Stage 0, Stage 5, Stage 10 and Stage 16.
- Typical Section shown for Vehicle Access on Stage 10, which has a 12.5m offset to Stage 11
- Profiling for landscape bunds on Stage 5 and Stage 10 shall ensure that surface water drains to the infiltration strip of their respective stage raise
- Landscape details shown are indicative. Final landscaping design to be provided by BSM
- Landscape design to implement variations for Vehicle Access and Planting Bunds spanning across Stage Raises

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER

YYYY-MM-DD	2021-Aug-11
PREPARED	POB
DESIGN	DH/BK
REVIEW	CC
APPROVED	BK/DH

PROJECT

BRDA SIDE-SLOPE CLOSURE DESIGN AT STAGE 16

TITLE

SIDE-SLOPE CLOSURE: LONG SECTION AND VARIATIONS FOR ACCESS ROADS AND LANDSCAPING

PROJECT No.	DRAWING No.	Rev.	SCALE
20143076	04	A	1:80 and 1:600 A1

APPENDIX B

Rock Fill Historical PSD Data

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
New road
Thomondgate
Limerick
Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
seamusoconnell@bhp.ie

Client: Priority Construction Ltd.
162, Clontarf Road
Dublin 3.

BHP Ref. No.: 12/10/007
Order No.: 1032
Date Received: 27/09/2012
Date Tested: 05/10/2012
Test Spec: Customer Spec
Item: Type B fill

F.T.A.O.: Mr. Conor McMahon

Client Ref: HWC Project at Rusal, Limerick.
Sampling Certificate Provided: Yes

IS EN 933-1 : 1998 Cl. 7 (Particle Size Distribution)

BHP Reference	12/10/007			SPECIFICATION LIMITS
Client Reference				
Sieve Size (mm)	% Passing	% Passing	% Passing	
500	100			
125	74			
80	55			
63	44			
45	31			
40	29			
31.5	26			
20	19			
16	17			
14	15			Not Applicable
12.5	14			
10	13			
8	12			
6.3	11			
4	9			
2.80	8			
2	8			
1	7			
500µm	6			
425µm	6			
250µm	5			
125µm	5			
63µm	4.0			

Remarks:
Details of any material not representative of the bulk sample found : None found.
The uniformity coefficient is 16.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 27th November 2012

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
New road
Thomondgate
Limerick
Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
seamusoconnell@bhp.ie

Client: Priority Construction Ltd.
162, Clontarf Road
Dublin 3.

BHP Ref. No.: 12/10/008
Order No.: 1032
Date Received: 27/09/2012
Date Tested: 05/10/2012
Test Spec: Customer Spec
Item: Type B fill

F.T.A.O.: Mr. Conor McMahon

Client Ref: HWC Project at Rusal, Limerick.
Sampling Certificate Provided: Yes

IS EN 933-1 : 1998 Cl. 7 (Particle Size Distribution)

BHP Reference	12/10/008			**SPECIFICATION LIMITS**
Client Reference				
Sieve Size (mm)	**% Passing**	**% Passing**	**% Passing**	
500	100			Not Applicable
125	67			
80	35			
63	29			
45	29			
40	29			
31.5	26			
20	22			
16	20			
14	19			
12.5	18			
10	16			
8	15			
6.3	13			
4	11			
2.80	10			
2	9			
1	8			
500µm	7			
425µm	7			
250µm	6			
125µm	5			
63µm	4.7			

Remarks:
Details of any material not representative of the bulk sample found : None found.
The uniformity coefficient is 37.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 27th November 2012

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
New road
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Tel +353 61 455399
Fax +353 61 455447
E Mail
seamusoconnell@bhp.ie

Client: Priority Construction Ltd.
162, Clontarf Road
Dublin 3.

BHP Ref. No.: 12/10/009
Order No.: 1032
Date Received: 27/09/2012
Date Tested: 05/10/2012
Test Spec: Customer Spec
Item: Type B fill

F.T.A.O.: Mr. Conor McMahon

Client Ref: HWC Project at Rusal, Limerick.
Sampling Certificate Provided: Yes

IS EN 933-1 : 1998 Cl. 7 (Particle Size Distribution)

BHP Reference	12/10/009			SPECIFICATION LIMITS
Client Reference				
Sieve Size (mm)	% Passing	% Passing	% Passing	
500	100			
125	58			
80	52			
63	48			
45	42			
40	41			
31.5	37			
20	29			
16	26			
14	24			
12.5	23			Not Applicable
10	21			
8	18			
6.3	16			
4	14			
2.80	12			
2	11			
1	9			
500µm	8			
425µm	8			
250µm	7			
125µm	7			
63µm	5.9			

Remarks:
Details of any material not representative of the bulk sample found : None found.
The uniformity coefficient is 67.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 27th November 2012

ISO 17025 INAB ACCREDITED TESTING DETAILED IN SCOPE REG NO.005T

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
New road
Thomondgate
Limerick
Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
seamusoconnell@bhp.ie

Client: Priority Construction Ltd.
162, Clontarf Road
Dublin 3.

BHP Ref. No.: 12/10/236
Order No.: 1032
Date Received: 10/10/2012
Date Tested: 20/10/2012
Test Spec: Customer Spec
Item: Type B fill

F.T.A.O.: Mr. Conor McMahon

Client Ref: HWC Project at Rusal, Limerick.
Sampling Certificate Provided: Yes

IS EN 933-1 : 1998 Cl. 7 (Particle Size Distribution)

BHP Reference	12/10/236			SPECIFICATION LIMITS
Client Reference				
Sieve Size (mm)	% Passing	% Passing	% Passing	
500	100			
125	74			
80	54			
63	39			
45	30			
40	27			
31.5	22			
20	16			
16	14			
14	13			Not Applicable
12.5	13			
10	11			
8	10			
6.3	9			
4	7			
2.80	6			
2	6			
1	5			
500µm	4			
425µm	4			
250µm	3			
125µm	3			
63µm	2.5			

Remarks:
Details of any material not representative of the bulk sample found : None found.
The uniformity coefficient is 10.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 30th November 2012

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
New road
Thomondgate
Limerick
Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
bhpcon2@bhp.ie

Client: Priority Construction Ltd.
162, Clontarf Road
Dublin 3.

BHP Ref. No.: 12/11/256
Order No.: 1032
Date Received: 21/11/2012
Date Tested: 01/12/2012
Test Specification: Customer spec
Item: Type B fill

F.T.A.O.: Mr. Conor McMahon

Client Reference: HWC Project at Rusal, Limerick.

Sampling Certificate Provided: No

BS 1377:Part 2:1990, Cl.9.2 (Particle Size Distribution)

BHP Reference	12/11/256			CLIENT REFERENCE
Sieve Size (mm)	**% Passing**	**% Passing**	**% Passing**	**Customer Ref** Type B fill **Source:** site won material
125	100			
100	68			
75	53			
63	48			
50	31			
37.5	29			
28	20			
20	15			
14	14			
10	11			
6.3	9			
5.0	8			
3.35	7			
2.00	5			
1.18	4			
600µm	4			
425µm	3			
300µm	3			
212µm	3			
150µm	3			
63µm	2			

Remarks:
Nil

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 20th December 2012

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
New road
Thomondgate
Limerick
Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
bhpcon2@bhp.ie

Client: Priority Construction Ltd.
162, Clontarf Road
Dublin 3.

BHP Ref. No.: 12/11/257
Order No.: 1032
Date Received: 21/11/2012
Date Tested: 01/12/2012
Test Specification: Customer spec
Item: Type B fill

F.T.A.O.: Mr. Conor McMahon

Client Reference: HWC Project at Rusal, Limerick.

Sampling Certificate Provided: No

BS 1377:Part 2:1990, Cl.9.2 (Particle Size Distribution)

BHP Reference	12/11/257			CLIENT REFERENCE
Sieve Size (mm)	**% Passing**	**% Passing**	**% Passing**	**Customer Ref** Type B fill **Source:** site won material
125	72			
100	72			
75	64			
63	48			
50	39			
37.5	32			
28	26			
20	21			
14	18			
10	16			
6.3	12			
5.0	10			
3.35	9			
2.00	7			
1.18	6			
600µm	5			
425µm	5			
300µm	4			
212µm	4			
150µm	4			
63µm	3			

Remarks:
Nil

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 20th December 2012

Form No.:BHP/MTI/1021 1.1 27/9/06

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
New road
Thomondgate
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Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
bhpcon2@bhp.ie

Client:	Murphy International Ltd.	**BHP Ref. No.:**	16/01/113
	Great Connell	**Order No.:**	Not Supplied
	Newbridge	**Date Received:**	22/01/2016
	Co. Kildare	**Date Tested:**	28/01/2016
		Test Specification:	Customer spec
F.T.A.O.:	Mr. Jason Doherty	**Item:**	Type B rock fill

Client Reference: Salt cake Cell Downstream Raise Construction Project at AAL.

Sampling Certificate Provided: Yes

BS 1377:Part 2:1990, Cl.9.2 (Particle Size Distribution)

BHP Reference	16/01/113			**CLIENT REFERENCE**
Sieve Size (mm)	**% Passing**	**% Passing**	**% Passing**	**Customer Ref** Type B Processed Rock fill **Source:** Site stockpile
300	100			
125	59			
100	52			
75	47			
63	44			
50	39			
37.5	32			
28	27			
20	21			
14	16			
10	13			
6.3	10			
5.0	9			
3.35	8			
2.00	7			
1.18	6			
600µm	5			
425µm	4			
300µm	4			
212µm	4			
150µm	4			
63µm	3			

Remarks:

The sample, as supplied does comply with the grading requirements of a Type B material, as detailed in cl. 6.3 of the AAL Specification for the Salt Cake Cell Raise Construction QA Plan.

The maximum particle size found is 200mm. The uniformity coefficient is 22.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 12th February 2016

Form No.:BHP/MTI/1021 1.1 27/9/06

ISO 17025 INAB ACCREDITED TESTING DETAILED IN SCOPE REG NO.0051

TEST REPORT

Analysing
Testing
Consulting
Calibration

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New road
Thomondgate
Limerick
Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
bhpcon2@bhp.ie

Client:	Murphy International Ltd.	**BHP Ref. No.:**	16/01/114
	Great Connell	**Order No.:**	Not Supplied
	Newbridge	**Date Received:**	22/01/2016
	Co. Kildare	**Date Tested:**	28/01/2016
		Test Specification:	Customer spec
F.T.A.O.:	Mr. Jason Doherty	**Item:**	Type B rock fill

Client Reference: Salt cake Cell Downstream Raise Construction Project at AAL.

Sampling Certificate Provided: Yes

BS 1377:Part 2:1990, Cl.9.2 (Particle Size Distribution)

BHP Reference	16/01/114			**CLIENT REFERENCE**
Sieve Size (mm)	**% Passing**	**% Passing**	**% Passing**	**Customer Ref** Type B Processed Rock fill **Source:** Site stockpile
300	100			
125	63			
100	63			
75	54			
63	54			
50	42			
37.5	34			
28	26			
20	20			
14	17			
10	14			
6.3	11			
5.0	9			
3.35	8			
2.00	6			
1.18	5			
600µm	5			
425µm	4			
300µm	4			
212µm	4			
150µm	4			
63µm	3			

Remarks:
The sample, as supplied does comply with the grading requirements of a Type B material, as detailed in cl. 6.3 of the AAL Specification for the Salt Cake Cell Raise Construction QA Plan.
The maximum particle size found is 200mm. The uniformity coefficient is 16.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 12th February 2016

Form No.:BHP/MTI/1021 1.1 27/9/06

ISO 17025 INAB ACCREDITED TESTING DETAILED IN SCOPE REG NO.005T

TEST REPORT

Analysing
Testing
Consulting
Calibration

Client:	Murphy International Ltd.	**BHP Ref. No.:**	16/01/115
	Great Connell	**Order No.:**	Not Supplied
	Newbridge	**Date Received:**	22/01/2016
	Co. Kildare	**Date Tested:**	28/01/2016
		Test Specification:	Customer spec
F.T.A.O.:	Mr. Jason Doherty	**Item:**	Type B rock fill

BHP
BHP
New road
Thomondgate
Limerick
Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
bhpcon2@bhp.ie

Client Reference: Salt cake Cell Downstream Raise Construction Project at AAL.

Sampling Certificate Provided: Yes

BS 1377:Part 2:1990, Cl.9.2 (Particle Size Distribution)

BHP Reference	16/01/115			**CLIENT REFERENCE**
Sieve Size (mm)	**% Passing**	**% Passing**	**% Passing**	**Customer Ref** Type B Processed Rock fill **Source:** Site stockpile
300	100			
125	67			
100	67			
75	47			
63	47			
50	43			
37.5	34			
28	25			
20	19			
14	16			
10	14			
6.3	12			
5.0	11			
3.35	10			
2.00	8			
1.18	7			
600µm	6			
425µm	6			
300µm	5			
212µm	5			
150µm	5			
63µm	4			

Remarks:

The sample, as supplied does comply with the grading requirements of a Type B material, as detailed in cl. 6.3 of the AAL Specification for the Salt Cake Cell Raise Construction QA Plan.

The maximum particle size found is 200mm. The uniformity coefficient is 27.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 12th February 2016

Form No.:BHP/MTI/1021 1.1 27/9/06

ISO 17025 INAB ACCREDITED TESTING DETAILED IN SCOPE REG NO.005T

TEST REPORT

Analysing
Testing
Consulting
Calibration

Client:	Murphy International Ltd.	**BHP Ref. No.:**	16/01/116
	Great Connell	**Order No.:**	Not Supplied
	Newbridge	**Date Received:**	22/01/2016
	Co. Kildare	**Date Tested:**	28/01/2016
		Test Specification:	Customer spec
F.T.A.O.:	Mr. Jason Doherty	**Item:**	Type B rock fill

BHP
BHP
New road
Thomondgate
Limerick
Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
bhpcon2@bhp.ie

Client Reference: Salt cake Cell Downstream Raise Construction Project at AAL.

Sampling Certificate Provided: Yes

BS 1377:Part 2:1990, Cl.9.2 (Particle Size Distribution)

BHP Reference	16/01/116			**CLIENT REFERENCE**
Sieve Size (mm)	**% Passing**	**% Passing**	**% Passing**	**Customer Ref** Type B Processed Rock fill **Source:** Site stockpile
300	100			
125	69			
100	69			
75	57			
63	57			
50	53			
37.5	38			
28	29			
20	21			
14	19			
10	16			
6.3	13			
5.0	12			
3.35	10			
2.00	8			
1.18	7			
600µm	6			
425µm	6			
300µm	5			
212µm	5			
150µm	5			
63µm	4			

Remarks:
The sample, as supplied does comply with the grading requirements of a Type B material, as detailed in cl. 6.3 of the AAL Specification for the Salt Cake Cell Raise Construction QA Plan.
The maximum particle size found is 200mm. The uniformity coefficient is 24.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 12th February 2016

Test results relate to the samples as supplied. This test report shall not be duplicated,except in full and only with the permission of the test laboratory.
Sampling details where supplied are held on file.

ISO 17025 INAB ACCREDITED TESTING DETAILED IN SCOPE REG NO.005T

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
BHP
New road
Thomondgate
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Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
seamusoconnell@bhp.ie

Client: J Murphy & Sons Ltd.
Great Connell
Newbridge
Co. Kildare

BHP Ref. No.: 17/08/2322
Order No.: Not supplied
Date Received: 28/08/2017
Date Tested: 31/08/2017
Test Spec: Customer Spec.
Item: Type B rock fill

F.T.A.O.: Mr. Alan Judge

Client Ref: SCC Phase 3 - Aughinish, Co. Limerick
Sampling Certificate Provided: No

EN 933-1 : 2012 Cl. 7 (Particle Size Distribution)

BHP Reference	17/08/2322	Specification	Specification	**SPECIFICATION LIMITS**
Client Reference	Type B rock fill			
Sieve Size (mm)	**% Passing**	**% Passing (minimum)**	**% Passing (maximum)**	
500	100			
300	100			
125	76			
80	76			
63	76			
45	67			
40	66			
31.5	59			
20	47			
16	42			
14	41			**Not Applicable**
12.5	38			
10	35			
8	31			
6.3	27			
4	23			
2.80	20			
2	18			
1	15			
500µm	13			
425µm	12			
250µm	11			
125µm	9			
63µm	7.8			

Remarks:

The sample, as supplied is a well graded crushed rock. The material has a maximum particle size 300mm with a uniformity coefficient > 100.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 27th September 2017

Analysing
Testing
Consulting
Calibration

TEST REPORT

Client:	J Murphy & Sons Ltd. Great Connell Newbridge Co. Kildare	**BHP Ref. No.:**	17/08/2323
		Order No.:	Not supplied
		Date Received:	28/08/2017
		Date Tested:	31/08/2017
		Test Spec:	Customer Spec.
		Item:	Type B rock fill
F.T.A.O.:	Mr. Alan Judge		
Client Ref:	SCC Phase 3 - Aughinish, Co. Limerick.		

Sampling Certificate Provided: No

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New road
Thomondgate
Limerick
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Tel +353 61 455399
Fax +353 61 455447
E Mail
seamusoconnell@bhp.ie

EN 933-1 : 2012 Cl. 7 (Particle Size Distribution)

BHP Reference	17/08/2323	Specification	Specification	**SPECIFICATION LIMITS**
Client Reference	Type B rock fill			
Sieve Size (mm)	**% Passing**	**% Passing (minimum)**	**% Passing (maximum)**	
500	100			
300	100			
125	78			
80	78			
63	73			
45	60			
40	56			
31.5	51			
20	42			
16	38			
14	36			**Not Applicable**
12.5	34			
10	31			
8	28			
6.3	25			
4	20			
2.80	18			
2	16			
1	13			
500µm	12			
425µm	11			
250µm	10			
125µm	9			
63µm	7.5			

Remarks:
The sample, as supplied is a well graded crushed rock. The material has a maximum particle size 300mm with a uniformity coefficient > 100.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 28th September 2017

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP
New road
Thomondgate
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Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
seamusoconnell@bhp.ie

Client: J Murphy & Sons Ltd.
Great Connell
Newbridge
Co. Kildare

BHP Ref. No.: 17/12/0783
Order No.: Not supplied
Date Received: 08/12/2017
Date Tested: 11/12/2017
Test Spec: Customer Spec.
Item: Type B rock fill

F.T.A.O.: Mr. Alan Judge

Client Ref: SCC Phase 3 - Aughinish, Co. Limerick.
Sampling Certificate Provided: No

EN 933-1 : 2012 Cl. 7 (Particle Size Distribution)

BHP Reference	17/12/0783	Specification	Specification	SPECIFICATION LIMITS
Client Reference	Type B rock fill			
Sieve Size (mm)	% Passing	% Passing (minimum)	% Passing (maximum)	
500	100			
300	100			
125	100			
80	84			
63	84			
45	71			
40	71			
31.5	66			
20	54			
16	48			
14	44			Not Applicable
12.5	42			
10	36			
8	32			
6.3	28			
4	22			
2.80	19			
2	17			
1	14			
500µm	12			
425µm	12			
250µm	11			
125µm	9			
63µm	7.3			

Remarks:
The sample, as supplied is a well graded crushed rock. The material has a maximum particle size 300mm with a uniformity coefficient > 100.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 21st December 2017

INAB ISO 17025 ACCREDITED TESTING DETAILED IN SCOPE REG NO.005T

TEST REPORT

Analysing
Testing
Consulting
Calibration

BHP

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New road
Thomondgate
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Ireland
Tel +353 61 455399
Fax +353 61 455447
E Mail
seamusoconnell@bhp.ie

Client: J Murphy & Sons Ltd.
Great Connell
Newbridge
Co. Kildare

BHP Ref. No.: 17/12/0784
Order No.: Not supplied
Date Received: 08/12/2017
Date Tested: 11/12/2017
Test Spec: Customer Spec.
Item: Type B rock fill

F.T.A.O.: Mr. Alan Judge

Client Ref: SCC Phase 3 - Aughinish, Co. Limerick.
Sampling Certificate Provided: No

EN 933-1 : 2012 Cl. 7 (Particle Size Distribution)

BHP Reference	17/12/0784	Specification	Specification	SPECIFICATION LIMITS
Client Reference	Type B rock fill			
Sieve Size (mm)	% Passing	% Passing (minimum)	% Passing (maximum)	
500	100			
300	100			
125	77			
80	62			
63	62			
45	55			
40	53			
31.5	50			
20	42			
16	38			
14	37			Not Applicable
12.5	35			
10	32			
8	29			
6.3	25			
4	20			
2.80	17			
2	16			
1	13			
500µm	12			
425µm	11			
250µm	10			
125µm	9			
63µm	7.1			

Remarks:
The sample, as supplied is a well graded crushed rock. The material has a maximum particle size 300mm with a uniformity coefficient > 100.

Seamus O'Connell
Laboratory Technical Manager
For and On Behalf of BHP Laboratories

Issue Date: 21st December 2017

GOLDER
MEMBER OF WSP

golder.com

APPENDIX L

PIC Breach and Wetlands Assessment

REPORT

Perimeter Interceptor Channel (PIC) Breach and Wetlands Assessment

Bauxite Residue Disposal Area - Closure Design at Stage 16

Submitted to:

Aughinish Alumina Limited

Aughinish Alumina Limited
Aughinish Island
Askeaton
County Limerick
Ireland

Submitted by:

Golder Associates Ireland Limited

Town Centre House, Dublin Road, Naas,
Co. Kildare, W91 TD0P Ireland

+353 45 810 200

20143076.R04.A1

November 2021

Distribution List

Aughinish Alumina Limited - 1 Copy (pdf)

Golder Associates Ireland Limited - 1 Copy (pdf)

Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

Golder Associates Ireland Ltd. (Golder) has been engaged by Aughinish Alumina Ltd. (AAL) to provide design services for the closure of the proposed Phase 1 and 2 Bauxite Residue Disposal Area (BRDA) constructed to Stage 16.

An engineering level closure design for water management infrastructure to transfer runoff from the BRDA dome at Stage 16 to the perimeter interceptor channels (PICs) has previously been completed (Golder, 2020). The surface water runoff management strategy for the dome of the Phase 1 and Phase 2 BRDA to Stage 16 considered the following main principles.

- Runoff from the dome is intercepted by the dome perimeter channels which convey the intercepted runoff to spillways.
- Seven spillways (i.e., SP-1 to SP-7) are distributed along the perimeter of the BRDA dome. The spillways will convey runoff from the dome perimeter channels to the PICs.
- A further spillway (SP-8) and two dome perimeter channels sections are proposed below the dome elevation to convey surface water runoff from the raised and capped Salt Cake Disposal Cell (SCDC) area to the adjacent section of the PIC.

Further to the dome water management infrastructure design (Golder, 2020), AAL contracted Golder to complete engineering level closure designs for:

- Water management infrastructure to transfer runoff from the PIC system and the Storm Water Pond (SWP) to the adjacent existing external drains to the north and west of the BRDA;
- Water management infrastructure to transfer runoff from PIC L (located to the east of the Phase 1 BRDA) to PIC K (located to the south of the SWP); and
- Hydraulic design for the wetlands proposed to be constructed in the PICs at closure.

The scope of services included the following tasks:

- Hydrological assessment of peak flows to the PICs and the SWP during the 1 in 100-year event and the BRDA Inflow Design Flow (IDF) post-closure;
- Hydraulic design of two spillways to safely transfer the IDF from the PICs and the SWP to the existing downstream external drains after closure. Spillway design shall allow for attenuation of discharge to greenfield rates for events up to the 1 in 100-year event.
- Hydrological assessment of peak flows to PIC-L during the IDF after closure; and
- Hydraulic design of one spillway to safely transfer the IDF from PIC-L to PIC-K after closure, i.e., 'the North-East PIC Spillway'.
- Hydraulic assessment of wetlands proposed to be constructed in the PICs at closure to achieve a minimum residence time of seven days for rainfall events up to the 1-year, 1-hour duration rainfall event, corresponding to 10.8 mm rainfall depth. The residence time is based on the water quality at closure assessment (see Appendix H of the EIAR).

The objective of this report is to outline the methodologies utilized and the outcomes from these tasks. The study assumes that the dome and the side-slope capping containment works from Stage 0 to Stage 16 have been completed at closure.

2.0 BACKGROUND

AAL is wholly owned by United Company RUSAL and operates the alumina refinery situated on Aughinish Island on the south side of the Shannon estuary. AAL own a circa 601.22 ha. landholding (the Site) on Aughinish Island. The Island is predominantly rural in character with the remaining land usage comprising agriculture, single low density residential housing and protected habitats (wetlands and grasslands).

Aughinish Island is located on the south banks of the Shannon Estuary, at approximately 50km from the outlet to the North Atlantic, in the south-west of Ireland, and is bounded by the River Shannon to the north, the Robertstown River to the west and southwest and the Poulaweala creek to the east and southeast. The nearest towns are Askeaton (ca. 6.0 km to the east) and Foynes (ca. 3.5 km to the west) and the Site is located circa 30 km west of Limerick City.

The Phase 1 BRDA is located southwest of the process plant and is formed of two facilities: the original Phase 1 BRDA, which covers an area of 72 ha and the eastern Phase 1 BRDA Extension, which covers an area of 32 ha. The Phase 2 BRDA adjoins the southern extent of the Phase 1 BRDA and covers an area of 80 ha.

The BRDA is surrounded by PICs, which collect seepage and runoff from the Phase 1 and Phase 2 facilities and currently convey it via pumps either to the Effluent Clarification System (ECS) or the Storm Water Pond (SWP). Both the SWP and the ECS are situated to the northeast of the Phase 1 BRDA.

Following closure, AAL will enter into a minimum 5-year active aftercare period during which time all the waters from the BRDA will be captured and returned to the ECS for treatment and subsequently to discharge at the licenced discharge point. AAL will continue to monitor the quality of the waters from the BRDA during the 5-year period, which is expected to improve significantly as the closure works are completed and will apply for a discharge to the environment at the two (2) proposed breach locations in the PICs and at appropriate water quality limits to be agreed with the EPA. It is proposed that a wetlands be constructed in the base of the PICs at closure, through which the runoff and seepage emanating from the BRDA will flow, prior to discharge to the downstream environment.

For the operational life of the facility, the BRDA has been classified to have a “High” hazard potential classification (HPC) under the Canadian Dam Association (CDA) Guidelines (CDA, 2014) which designates the Inflow Design Flood (IDF) to be 1/3 between the 1,000-year and the Probable Maximum Flood (PMF).

For the closure phase of the facility, Golder considers that the BRDA classification would be reduced to a “Significant” HPC which also corresponds to an IDF 1/3 between the 1,000-year and the Probable Maximum Flood (PMF).

During 2019, SLR Consulting was retained by AAL to conduct an independent dam safety review (DSR) of the BRDA (SLR 2019). The DSR report also states that this reduced classification could be justified for closure.

3.0 PIC BREACH CLOSURE SPILLWAY DESIGN

As discussed in Section 2.0, two (2) PIC breach spillways will discharge surface water off-site at closure (Figure 1). The PIC breach spillway locations have been selected as they correspond with the locations where invert elevations are lowest within the existing Phase 1 and Phase 2 BRDA PIC systems and therefore, facilitate drainage of the full system by gravity at closure, without the requirement for significant alteration of invert elevations or gradients.

The location of the North-East PIC spillway, connecting PIC-L to PIC-K (see Section 4.0) is also indicated on Figure 1. A site layout plan is provided on Drawing 1 (Appendix A).

The surface water management design strategy at closure for the PICs is summarised as follows:

- **Phase 1 BRDA PICs:**
  - Surface water runoff and seepage collected in the Phase 1 BRDA PICs (PIC-E, PIC-F, PIC-G, PIC-J, PIC-K and PIC-L) and in a short segment of the Phase 2 BRDA PICs (PIC-M) will be discharged to the North Drain via PIC Breach Spillway #1, which is located centrally in PIC-G,
  - The existing PICs will drain by gravity to PIC-G. PIC Breach Spillway #1 will be constructed through the north-east embankment of the Outer Perimeter Wall (OPW) of the PIC-G segment.
  - Waters in the North Drain will then flow counter-clockwise to enter the northern section of the West Drain and subsequently to the south to the penstock discharge point, located to the west of the Phase 1 BRDA. The North Drain segment has a length of approximately 1.3 km and the northern section of the West Drain has a length of approximately 0.6 km. The gradient along the North Drain and the northern section of the West Drain is approximately 0.013%.
  - An additional spillway will be required at closure to convey flows from PIC-L to PIC-K (North-East PIC). During operation these flows are conveyed by piped culverts which will be replaced by a spillway at closure.
- **Phase 2 BRDA PICs:**
  - Surface water runoff and seepage collected in the Phase 2 BRDA PICs (PIC-A, PIC-B, PIC-C and PIC-D) will be discharged to the southern section of the West Drain via PIC Breach Spillway #2, which is located at the north-west corner of the Phase 2 BRDA (end of PIC-D, where the Phase 2 BRDA PICs meet with the Phase 1 BRDA PICs).
  - The existing PICs will drain by gravity to the north-west corner of the Phase 2 BRDA. PIC Breach Spillway #2 will be constructed through the OPW of the PIC-D segment.
  - Waters entering the southern section of the West Drain will then flow clockwise to the north and subsequently to the penstock discharge point, located to the west of the Phase 1 BRDA. The southern section of the West Drain has a length of approximately 0.5 km and an estimated average gradient of 0.134%.
- Surface water from both PIC Breach Spillways is discharged into the Robertstown River, which is a tidal river that flows north before joining the River Shannon at the Shannon Estuary. The discharge takes place during low tides and is controlled by a sluice gate which has an invert level of approximately -1.1 mOD.
- The PIC breach spillways have been designed to safely convey the BRDA IDF post-closure and allow for attenuation of discharge to greenfield rates for events up to the 1 in 100-year event.

- The preliminary design consists of two U-shaped concrete channels through the OPW of PIC-G (PIC Breach Spillway #1) and through the OPW of PIC-D (PIC Breach Spillway #2). The concrete channels have been designed with a width of 1.0m and discharge to trapezoidal rip-rap lined chutes. The minimum depths of the concrete channels are 2.15m (PIC Breach Spillway #1) and 1.80m (PIC Breach Spillway #2). The inlet width of both PIC breach spillways is reduced to 0.5m to limit the discharge rate from the PICs to the allowable discharge rate for the 1 in 100-year event. The rip-rap chutes have been designed with a bottom width of 3.0m and a depth of 1.0 m; and ultimately discharge to the existing downstream drains.
- Hydraulic modelling of the proposed spillways, PIC Breach Spillway #1, PIC Breach Spillway #2 and the North-East PIC Spillway, was completed using HEC-RAS software to assess the performance of the design during the considered events. The modelling demonstrated that the proposed design meets the design criteria.

Figure 1: Key Features of the PIC Drainage System at Closure

3.1 PIC Breach Spillway Hydrological Assessment

3.1.1 Hydrologic Design Criteria

The PIC Breach Spillways have been designed in accordance with the following hydrological design criteria:

- The spillways shall accommodate safe discharge the BRDA IDF post-closure ; and
- The spillways shall satisfy River Regime Protection storm water design criteria (for control of runoff rates) in the Volume Two of the Greater Dublin Strategic Drainage Study (SDCC, 2005):
  - 1 in 1-year discharge rates from the PIC breach spillways shall not exceed the 1 in 1-year greenfield peak runoff rate or 2 L/s/ha (whichever is greater);
  - 1 in 100-year discharge rates from the PIC breach spillways shall not exceed the 1 in 100-year greenfield peak runoff rates; and
  - Site critical duration storms shall be used to assess attenuation storage volume requirements.

The Greater Dublin Strategic Drainage Study (SDCC, 2005) also specifies River Flood Protection storm water design criteria and recommends the provision of long-term storage for runoff volumes in excess of the greenfield runoff volumes.

During extreme rainfall events, where flooding is likely to occur in the receiving river system, it is important to limit these runoff volumes. This can be achieved by spilling from the drainage system to an area which will drain very slowly, preferably by infiltration.

Golder proposes the area to the north and west of the BRDA, enclosed between the Flood Tidal Defence Berm (FTDB) and the OPW for the PICs (not including the footprint of the Special Area of Conservation (SAC) or the Special Protection Area (SPA), also known as the ‘Bird Sanctuary’, located to the north of the SWP, see Drawing 1 in Appendix A), acts as a designated flood area that provides temporary storage for spillway discharges during extreme events.

3.1.2 PIC Closure Catchments

The catchments draining to the PICs are presented in Drawing 1 (Appendix A), and their main hydrological properties are provided in Table 1. Each catchment is formed by a portion of the BRDA dome and a portion of the BRDA embankment side slope.

Times of concentration were estimated using the Watershed Lag Method (NRCS 2010). Curve numbers (CN) were estimated as composite values and are presented in Table 2.

Table 1: PIC Breach Spillways Catchment Properties

Spillway	Area (ha)	Flow Length (m)	Composite CN	Average Land Slope (%)	Lag Time (min)	Time of Concentration (min)
PIC Breach Spillway #1	116.1	2,091	83	2.04	57	95
PIC Breach Spillway #2	72.8	2,874	83	1.50	86	143

Table 2: Composite Curve Numbers Calculations (PIC Breach Spillway #1 and PIC Breach Spillway #2)

Spillway	PIC Breach Spillway #1	PIC Breach Spillway #2
Dome catchment area (ha)	47.1	26.5
Embankment side slope area (ha)	69.0	46.3
Dome catchment CN (1)	84	84
Embankment side slope CN (2)	82	82
Total area (ha)	116.1	72.8
Estimated Composite CN	83	83

Notes:

(1) Pasture / grassland / range soil cover, on hydrologic soil group D, with fair hydrologic condition (NRCS 1986).

(2) Contoured row crop ground cover, on hydrologic soil group C, with good hydrologic condition (NRCS 1986).

3.1.3 PIC Closure Maximum Allowable Release Rates

As mentioned in Section 3.1.1, the PIC breach spillways were designed to satisfy River Regime Protection storm water design criteria (for control of runoff rates) in Volume Two of the Greater Dublin Strategic Drainage Study (SDCC, 2005). Maximum allowable release rates were evaluated based on assessment of the following:

- 1 in 1-year greenfield peak runoff rates;
- Release rates equal to 2 L/s/ha; and
- 1 in 100-year greenfield peak runoff rates.

The greenfield runoff rate for a specific return period is defined as the peak rate of runoff due to rainfall falling on a given area of vegetated land, prior to any development. The 1 in 1-year and 1 in 100-year greenfield runoff rates for each catchment were calculated using the Greenfield Runoff Rate Estimation Tool (HR Wallingford, 2021) and are presented in Table 3, together with estimates of the 2 L/s/ha release rates.

The maximum allowable release rates to be discharged from the PIC spillways to achieve the specified design criteria were assessed based on these values. No limiting release rate is required for extreme low probability events in excess of the 1 in 100-year event, i.e., the IDF.

Table 3: PIC Closure Allowable Release Rates

Spillway	1 in 1-Year Storm			1 in 100-Year Storm	
	Greenfield Runoff Rate (L/s)	2 L/s/ha Release Rate (L/s)	Allowable Release Rate (L/s)	Greenfield Runoff Rate (L/s)	Allowable Release Rate (L/s)
PIC Breach Spillway #1	227.7	232.2	232.2	696.4	696.4
PIC Breach Spillway #2	150.3	145.6	150.3	459.8	459.8

3.1.4 PIC Closure – Runoff Rates Assessment

In accordance with the design criteria (Section 3.1.1), the site critical duration storms were used to assess the required attenuation storage volume in the PICs to achieve the allowable release rates.

Rainfall depth-duration-frequency (DDF) data was downloaded from the Met Eireann website for the location of the BRDA (Met Eireann 2021). Rainfall depths were obtained for durations from 5 minutes to 25 days and for return periods ranging between 6 months and 500 years.

The BRDA IDF is the flood generated from a rainfall depth 1/3 between the 1 in 1,000-year rainfall depth and the Probable Maximum Precipitation (PMP) at closure (Golder 2020). This rainfall depth has previously been assessed for a range of storm durations in accordance with the methodology outlined in the design report for the BRDA dome water management design at closure (Golder 2020).

The 50% summer design storm profile, i.e., the proportion of the total storm depth occurring at intervals throughout the storm duration, recommended in the Volume 4 of the Flood Estimation Handbook (Houghton-Carr, 1999) was applied for the assessment of each design storm rainfall depth. This profile represents the one that is, on average, peakier than 50% of UK summer storms, and peakier than the 75% winter storm profile.

Rainfall-runoff processes at closure for the Phase 1 and 2 BRDA constructed to Stage 16 were simulated using the United States Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) (USACE, 2021a). Model inputs included the estimated catchment areas, composite curve numbers and lag times provided in Table 1 for the PIC breach spillway catchments, and the design storm profiles described above.

Different duration storms (i.e., 1, 2, 3, 4, 6, 9, 12, 24 and 48 hours) were routed through the PICs in the HEC-HMS hydrologic model. The catchment critical duration storms were assumed to be the storms which yielded the greatest spillway discharges and attenuation storage requirements.

For the 1 in 100-year event, the catchment critical duration storms were the 12-hour duration storm for PIC Breach Spillway #1 and 9-hour duration storm for PIC Breach Spillway #2.

For the IDF event, the critical duration storm was the 4-hour duration storm for both the PIC Breach Spillway #1 and the PIC Breach Spillway #2.

Design rainfall depths for the critical duration storms for each PIC Breach spillway catchment are summarised in Table 4 and Table 5 .

Table 4: Critical Duration Storm Rainfall Depths (1 in 1-Year and 1 in 100-Year Events)

Spillway	Catchment Critical Storm Duration (hours)	1 in 1-Year Storm Rainfall Depth (mm)	1 in 100-Year Storm Rainfall Depth (mm)
PIC Breach Spillway #1	12	28.4	63.0
PIC Breach Spillway #2	9	25.4	58.3

Table 5. Critical Duration Storm Rainfall Depths (IDF Event)

Spillway	Catchment Critical Storm Duration (hours)	IDF Storm Rainfall Depth (mm)
PIC Breach Spillway #1	4	95.0
PIC Breach Spillway #2	4	95.0

Peak runoff rates from each PIC breach spillway catchment for the 1 in 1-year and 1 in 100-year, and IDF critical duration storms were assessed using the HEC-HMS hydrologic model and are presented in Table 6.

Runoff rates for the 1 in 100-year event to PIC Breach Spillway #1 and PIC Breach Spillway #2 are 1.8 and 1.7 times the corresponding allowable release rates in Table 3, indicating the requirement for attenuation storage to limit spillway discharge rates.

Runoff rates for the 1 in 1-year event are lower than the corresponding allowable release rates indicating that the allowable release rate for the 1 in 1-year event will be achieved without the requirement for attenuation storage.

Table 6. PIC Closure Peak Runoff Rates

Catchment	1 in 1-Year Peak Runoff Rate (L/s)	1 in 100-Year Peak Runoff Rate (L/s)	IDF Peak Runoff Rate (L/s)
PIC Breach Spillway #1	207	1,243	6,038
PIC Breach Spillway #2	111	800	3,210

3.2 PIC Breach Spillways Hydraulic Design

3.2.1 PIC Breach Spillways Proposed Layout and Geometry

The locations of the proposed PIC breach spillways are presented in Drawing 1 (Appendix A).

The spillways have been designed as U-shaped concrete channels installed through the OPW of PIC-G (PIC Breach Spillway #1) and through the OPW of PIC-D (PIC Breach Spillway #2) with a constant longitudinal slope. The concrete channels will discharge to riprap-lined trapezoidal chutes with side slopes of 2.5(H):1(V). These riprap-lined chutes will ultimately discharge to the existing downstream drains.

The depths and widths of the concrete channel and the chutes have been designed to safely convey the IDF while limiting discharge rates to the allowable release rates in Table 3 for events up to the 1 in 100-year event. Geometric channel design information is provided in Drawings 02 and 03 (Appendix A) and in Table 7.

Both concrete channels have been designed with a throughflow width of 1m. The minimum depths are 2.15m and 1.80m for PIC Breach Spillway #1 and PIC Breach Spillway #2, respectively.

The inlet width of both PIC breach spillways is reduced to 0.5m to limit the discharge rate from the PICs to the allowable discharge rate for the 1 in 100-year event. Details showing the transitions from 0.5m width channel to 1m width channel in both PIC breach spillways are provided in Drawings 02 and 03 (Appendix A).

The riprap chutes have been designed with a constant bottom width of 3m. The PIC Breach Spillway #1 riprap chute has a depth of 1.5m and the PIC Breach Spillway #2 riprap chute has a depth of 1.0m.

Table 7: PIC Breach Spillway Geometric Design Properties

Spillway	Channel	Length (m)	Bottom Width (m)	Minimum Depth (m)	Upstream Invert Level (mOD)	Downstream Invert Level (mOD)	Longitudinal Slope (%)
PIC Breach Spillway #1	Concrete Channel	32	1 (1)	2.15	2.25	1.30	2.9
	Riprap Chute	11	3	1.50	1.30	1.00	2.9
PIC Breach Spillway #2	Concrete Channel	27	1 (1)	1.80	2.10	1.39	2.4
	Riprap Chute	9	3	1.00	1.39	1.00	3.0

Notes:

(1) Width at the inlet is reduced to 0.5 m. See Drawing 03 (Appendix A)

3.2.2 PIC Attenuation Capacity Assessment

PIC Breach Spillways #1 and #2 will both transfer surface water from their upstream PIC segments to the existing downstream drains, i.e., the North Drain and southern section of the West Drain, respectively. These PIC segments will provide attenuation storage capacity within the BRDA water management system post-closure, prior to discharge to the downstream drains.

The existing operational elevation-storage relationships for the PICs were estimated and reported previously by Golder (2021). At closure it is anticipated that PIC-E, PIC-F, PIC-G and PIC-J will provide attenuation storage capacity for PIC Breach Spillway #1; whilst PIC-B, PIC-C and PIC-D will provide attenuation storage for PIC Breach Spillway #2.

For the purposes of this assessment, it has been assumed that during closure the storage capacities within these facilities will be reduced by 35% to allow for infilling associated with possible wetland construction.

The estimated combined elevation-storage relationships at closure considered for this assessment are presented in Figure 2 for both spillway locations. The estimated maximum capacity of PIC-E, PIC-F, PIC-G, and is PIC-J is 64,647 m^3 at 4.7 mOD, and the estimated maximum capacity of PIC-B, PIC-C and PIC-D is 55,870 m^3 at 5.0 mOD.

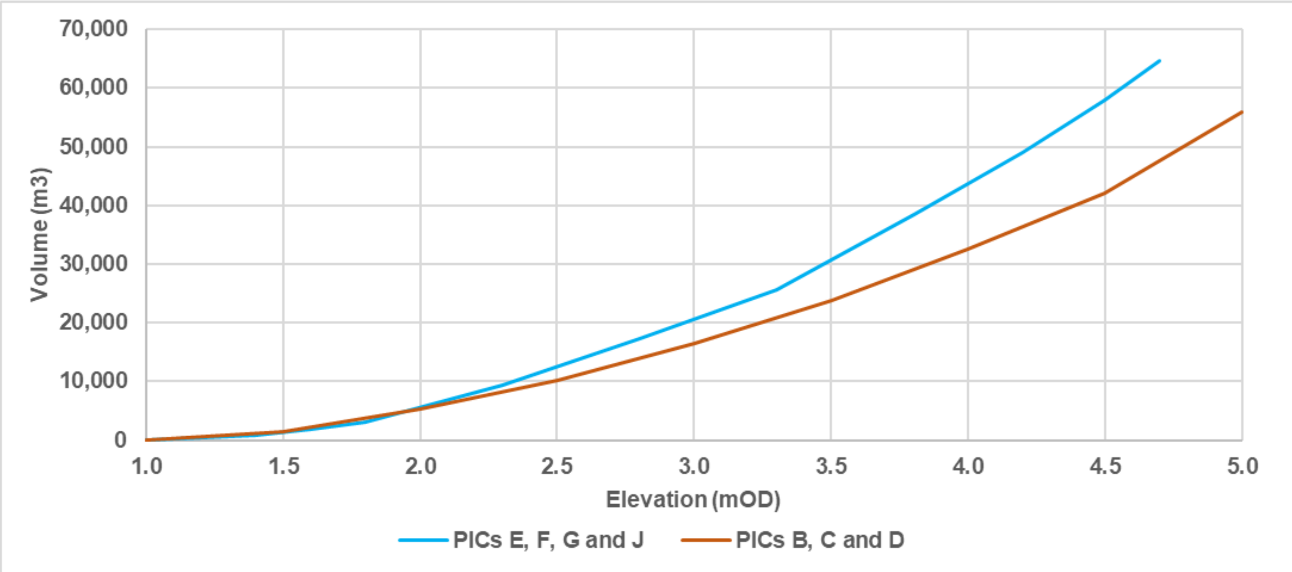

Figure 2: Estimated PICs Elevation-Storage Relationships at Closure

To assess design compliance with the maximum allowable release rates, the PICs attenuation capacities and spillway discharges were assessed for the 1 in 1-year and 1 in 100-year critical duration storms using the HEC-HMS hydrologic model and the results are summarised in Table 8.

The initial water level in the PICs at the start of the simulations was assumed to be at the inlet invert elevation of the spillways. The modelled peak discharge rates comply with the maximum allowable release rates in Table 3, and the 1 in 100-year peak water levels indicate a minimum freeboard to the embankment crest of 1.62m in the PIC Breach Spillway #1 2.25m in the PIC Breach Spillway #2 for events up to the 1 in 100-year event.

Table 8: Attenuation Capacity Assessment Results (1 in 1-year and 1 in 100-year events)

Spillway	1 in 1-Year Storm				1 in 100-Year Storm			
	Peak Inflow Rate (L/s)	Max. Storage Volume (m^3)	Peak Water Level (mOD)	Peak Discharge Rate (L/s)	Peak Inflow Rate (L/s)	Max. Storage Volume (m^3)	Peak Water Level (mOD)	Peak Discharge Rate (L/s)
PIC Breach Spillway #1	206	11,898	2.46	85	1,243	21,915	3.08	655
PIC Breach Spillway #2	111	7,716	2.25	50	800	13,350	2.75	458

Routing of the IDF was also assessed using the HEC-HMS hydrological model and the results of this assessment are presented in Table 9. The IDF peak water levels indicate a minimum freeboard to the embankment crest of 0.62m in PIC Breach Spillway #1 and 1.39m in PIC Breach Spillway #2.

Table 9: Attenuation Capacity Assessment Results (IDF Event)

Spillway	IDF Peak Inflow Rate (L/s)	IDF Max. Storage Volume (m^3)	IDF Peak Water Level (mOD)	IDF Peak Discharge Rate (L/s)
PIC Breach Spillway #1	6,038	45,773	4.08	2,155
PIC Breach Spillway #2	3,210	25,683	3.61	1,610

3.2.3 PIC Breach Spillways Lining Design

The PIC breach spillways will consist of concrete U-channels which discharge to downstream riprap-lined chutes. These chutes have been designed incorporating an angular rock riprap lining layer to dissipate a portion of the flow energy and to reduce design flow velocities, prior to discharge to the existing downstream drains. Riprap lining is commonly used to protect underlying soil surfaces from erosion. An additional separation geotextile has been provided beneath the riprap lining as a filter layer.

The chutes have been designed as rock chutes in accordance with guidance in Robinson et. al. (1998). The methodology allows for evaluation of the minimum stable median (D_{50}) rock size for a given cross section based on the chute slope and the unit flow rate (flow rate per unit width). Robinson et. Al. (1998) also provides an empirical equation for estimating the Manning's roughness coefficients of the chutes based on the D_{50} and chute slope.

The design geometric properties for the spillways in Table 7 were used in the calculations. The design flow rate for each spillway was taken as the modelled peak discharge rate for the IDF as shown in Table 9. A design D_{50} of 250 mm was selected for both chutes as this riprap sizing is consistent with the riprap material considered for the design of the BRDA dome closure spillways (Golder, 2020). This represents a Factor of Safety (FoS) of 3.2 and 4.1 applied to the minimum D_{50} rock size for PIC Breach Spillway #1 and PIC Breach Spillway #2, respectively.

The resulting spillway riprap design requirements and estimated Manning's roughness coefficients are presented in Table 10.

Table 10: Spillway Riprap Sizing – PIC Breach Spillways

Spillway	PIC Breach Spillway #1	PIC Breach Spillway #2
Minimum stable median (D_{50}) riprap rock size (mm)	79	61
Design median (D_{50}) riprap rock size (mm)	250	250
Riprap layer thickness (2 x D_{50}) (mm)	500	500
Manning's roughness coefficient	0.039	0.038

The riprap gradation requirement was assessed in accordance with the United States Department of Agriculture Natural Resources Conservation Service design procedures for rock-lined chutes (USDA, 2018). The results are presented in Table 11.

Table 11: Riprap Gradation Requirements – PIC Breach Spillways

Passing by Weight (%)	Lower Envelope Gradation (mm)	Upper Envelope Gradation (mm)
100	375	500
85	325	450
50	250	375
10	200	325

3.2.4 PIC Breach Spillways Hydraulic Analysis

Hydraulic modelling was undertaken to assess the performance of the spillways. A hydraulic model of each spillway was developed using the United States Army Corps of Engineers Hydrologic Engineering Center's River Analysis System (HEC-RAS) (USACE, 2021b). The models also included approximately 50m of the receptor drains downstream of the confluence with the spillways.

The design geometric properties for the spillways in Table 7 were used in the model. A Manning's roughness coefficient of 0.013 was used for the concrete channel (USACE 2008). The Manning's roughness coefficients in Table 10 were applied to the riprap lined chutes.

Geometric properties of the North Drain and southern section of the West Draain were defined using site topographic data, where available. It was assumed that the North Drain and the West Drain are trapezoidal channels with a base width of 4.4 m, side slopes of 1.2(H):1(V) and a depth of 1.6m. A Manning's roughness coefficient of 0.030 was applied for the North Drain and the West Drain which is assumed to be representative of their roughness characteristics (earth, winding and sluggish channel with grass and some weeds) (USDA 2008).

The steady-state flow data applied in the HEC-RAS model consisted of the estimated peak discharge rates for the IDF as provided in Table 9. The model steady flow boundary conditions were 'Critical Depth' at the upstream end of the model and 'Normal Depth' at the downstream end of the model (based on the estimated average longitudinal slopes along the drains input to the model).

One-dimensional (1-D) steady-state flow analysis with a mixed flow regime was completed to assess the design flow depths, flow velocities and flow regimes (i.e., subcritical or supercritical) along the spillways. The results were assessed against the following hydraulic design criteria:

- No overtopping of the structures (i.e., freeboard > 0.3m); and
- Hydraulic jump achieved either at or upstream of the confluence with the receptor drain, i.e., change in flow regime from supercritical (Froude no. > 1) to subcritical (Froude no. < 1).

A summary of the hydraulic modelling results during the IDF event for each spillway is presented in Table 12, and further detailed (tabulated) model outputs are provided in Appendix B. Modelling results demonstrate that the spillways meet the above-specified design criteria.

Table 12: Hydraulic Analysis Results Summary – PIC Breach Spillways

Spillway	Channel	Design Flow (L/s)	Depth (m)		Velocity (m/s)		Froude No.		Freeboard (m)	
			Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
PIC Breach Spillway #1	Concrete Channel	2,160	0.13	1.24	3.48	5.48	1.00	4.83	0.91 (1)	2.02 (1)
	Riprap Chute	2,160	0.80	1.12	0.39	0.62	0.14	0.25	0.38	0.70
	Existing North Drain	2,160	1.13	1.13	0.33	0.33	0.11	0.11	0.47	0.47
PIC Breach Spillway #2	Concrete Channel	1,610	0.10	1.02	3.17	5.08	1.00	5.36	0.78 (2)	1.70 (2)
	Riprap Chute	1,610	0.16	0.28	1.56	2.94	1.03	2.48	0.72	0.84
	Existing West Drain	1,610	0.16	0.49	0.67	2.39	0.33	1.90	1.11	1.44

Notes:

(1) Freeboard calculated assuming a channel depth of 2.15 m.

(2) Freeboard calculated assuming a channel depth of 1.80 m.

During the IDF event, the modelling indicates the following;

- In PIC Breach Spillway #1, a hydraulic jump will occur approximately immediately downstream of the transition from the concrete channel to the riprap channel; and
- In PIC Breach Spillway #2, a hydraulic jump will occur approximately 10m downstream of the confluence with the southern section of the West Drain.

The spillway design includes an extension of the riprap lining for a distance of 10m along the existing receptor drains to protect the channels against erosion at the location of the hydraulic jump.

4.0 NORTH-EAST PIC SPILLWAY

A further PIC spillway is required at closure to convey runoff from PIC-L to PIC-K, i.e., the North-East PIC Spillway. This spillway has been designed as a riprap-lined trapezoidal chute. The design methodology and outcomes are detailed in the following sections.

4.1 North-East PIC Spillway IDF Assessment

The catchment draining to the North-East PIC Spillway is presented in Drawing 1 (Appendix A) and its main hydrological properties are provided in Table 13. The curve number (CN) was estimated as a composite value as presented in Table 14.

The catchment includes the area which drains into PIC-L, which consists of the following:

- Catchments contributing to the BRDA dome closure spillways SP-1 and SP-8 (Golder 2020); and
- Embankment side slopes draining to PIC-L and PIC-M.

Table 13. North-East PIC Spillway Catchment Properties

Spillway	Area (ha)	Flow Length (m)	Composite CN	Average Land Slope (%)	Lag Time (min)	Time of Concentration (min)
North-East PIC Spillway	26.0	1,120	83	8.4	17	28

Table 14. Composite Curve Number Calculations (North-East PIC Spillway)

Parameter	Value
Dome catchment area (ha)	14.6
Embankment side slope area (ha)	11.4
Dome catchment CN (1)	84
Embankment side slope CN (2)	82
Total area (ha)	26.0
Estimated Composite CN	83

Notes:

(1) Pasture / grassland / range soil cover, on hydrologic soil group D, with fair hydrologic condition (NRCS 1986).

(2) Contoured row crop ground cover, on hydrologic soil group C, with good hydrologic condition (NRCS 1986).

The North-East PIC Spillway has been designed to safely convey the BRDA Inflow Design Flood (IDF). As outlined in Section 3.1.4, the rainfall depth associated with the BRDA IDF has been assessed for a range of storm durations in accordance with the methodology outlined in the design report for the BRDA dome water management design at closure (Golder 2020).

Peak flows for the IDF were calculated by applying the Rational Method to the contributing catchment. The determination of rainfall intensities for this method assumes that the rainfall duration is equal to the time of concentration of the catchment. The estimated time of concentration of the catchment draining to the North-East PIC Spillway is 28 minutes, and therefore a storm duration of 30 minutes was considered for the spillway design. The estimated IDF rainfall depth for a 30-minute storm duration is 60.1 mm, corresponding to a rainfall intensity of 120.2 mm/hr.

A runoff coefficient of 0.6 was assumed for the catchment, which is considered representative of Pasture/Range/Meadow terrain types with steep slopes, i.e., >7%, during high return period rainfall events (Chow et al 1988). The IDF peak flow calculation is presented in Table 15.

Table 15: IDF Peak Flow Calculation (Rational Method)

Catchment	Area (ha)	Runoff Coefficient	Rainfall Duration (min)	Rainfall Depth (mm)	Rainfall Intensity (mm/hr)	Estimated Peak Flow (m^3/s)
North-East PIC Spillway	26.0	0.6	30	60.1	120.2	5.2

4.2 North-East PIC Spillway Layout and Geometry

The proposed inlet to the North-East PIC Spillway is located at the northwest corner of PIC-L (Drawing 1, Appendix A). Geometric channel design information is provided in Drawing 04 (Appendix A) and in Table 16.

Table 16. Geometric Design Properties – North-East Spillway

Spillway	Length (m)	Bottom Width (m)	Depth (m)	Side Slope (H:1V)	Upstream Invert Level (mOD)	Downstream Invert Level (mOD)	Longitudinal Slope (%)
North-East PIC Spillway	69	10	1	2.5	10.0	3.5	9.4

4.3 North-East PIC Spillway Lining Design

The North-East PIC Spillway has been designed incorporating an angular rock riprap lining layer to dissipate a portion of the flow energy and to reduce flow velocities.

Like the riprap lining for the PIC breach spillways the North-East PIC Spillway riprap lining has been designed in accordance with guidance in Robinson et. al. (1998). The geometric design properties for the spillway in Table 16 were used in the calculations, and the design flow rate for the spillway was the IDF peak flow rate outlined in Table 15.

A design D_{50} of 250 mm was selected for the spillway as this riprap sizing is consistent with the riprap material considered for the design of the BRDA dome closure spillways (Golder, 2020), and the PIC breach spillways. The selected riprap material achieves a Factor of Safety (FoS) for the D_{50} rock size of 1.5. The resulting spillway riprap sizing design requirements and estimated Manning's roughness coefficient are presented in Table 17.

Table 17. Spillway Riprap Sizing – North-East Spillway

Spillway	North-East PIC Spillway
Minimum stable median (D_{50}) riprap rock size (mm)	165
Design median (D_{50}) riprap rock size	250
Riprap layer thickness (2 x D_{50}) (mm)	500
Manning's roughness coefficient	0.044

The riprap gradation requirement was assessed in accordance with the United States Department of Agriculture Natural Resources Conservation Service design procedures for rock-lined chutes (USDA, 2018), and the results are presented in Table 18.

Table 18: Riprap Gradation Requirements – North-East PIC Spillway.

Passing by Weight (%)	Lower Envelope Gradation (mm)	Upper Envelope Gradation (mm)
100	375	500
85	325	450
50	250	375
10	200	325

The riprap lining will be underlain by a ‘Concrete Canvas’ liner to protect from erosion of the underlying bauxite residue, prevent contamination of the water management system by bauxite residue leaching or seepage and to provide protection from accidental and UV damage during its operational life.

Concrete Canvas (‘CC’) is a Geosynthetic Cementitious Composite Mat (GCCM) that is commonly used for channel lining applications. CC consists of a 3-dimensional fibre matrix containing a specially formulated dry concrete mix; a PVC backing ensures the material has a low permeability (Concrete Canvas, 2020a). A summary of key properties of the CC liner considered in the design are summarised below (Concrete Canvas, 2020b and 2020c):

- Manning’s roughness coefficient = 0.011;
- Low level of permeability similar to compacted clay liner (hydraulic conductivity = 1×10^{-8} m/s); and
- Durable product with a minimum BBA certified design life of 120 years;

The CC liner is available in three thicknesses (5 mm, 8 mm and 13 mm) depending on the intended use of the liner and site-specific design considerations. The 13 mm thickness CC liner (CC13™) is proposed for the spillway lining design.

4.4 North-East PIC Spillway Hydraulic Analysis

A hydraulic model of the North-East PIC Spillway was developed using HEC-RAS software (USACE 2021b) to assess the performance of the spillway during the IDF. The design geometric properties for the spillways in Table 16 were used in the model. The model also included PIC-K and cross sections were cut from a 3-dimensional (3D) surface of the proposed channel developed using AutoCAD Civil 3D.

The Manning’s roughness coefficient applied for the spillway was based on the selected riprap size (Table 17). A Manning’s roughness coefficient of 0.030 was applied for PIC K which is considered representative of its roughness characteristics at closure (earth, winding and sluggish channel with grass and some weeds) (USDA 2008).

Steady-state flow data applied in the HEC-RAS model consisted of the estimated IDF peak flow rate in Table 15. The model steady flow boundary conditions were ‘Critical Depth’ at the upstream end of the model and ‘Normal Depth’ at the downstream end of the model (based on the estimated average longitudinal slope along PIC-K input to the model). The spillways were designed and assessed against the following hydraulic design criteria:

- No overtopping of the structure (i.e., freeboard > 0.3m); and
- Hydraulic jump achieved either at or upstream of the confluence with PIC K, i.e., change in flow regime from supercritical (Froude no. >1) to subcritical (Froude no. <1).

One-dimensional steady-state flow analysis with a mixed flow regime was completed to assess the design flow depths, flow velocities and the flow regimes (i.e., subcritical or supercritical) along the spillway.

Table 19 presents a summary of the results, and further detailed (tabulated) model outputs are provided in Appendix B. The hydraulic modelling results demonstrate that the above-specified design criteria are satisfied.

Table 19. Hydraulic Analysis Results Summary – North-East PIC Spillway.

Channel	Design Flow (m^3/s)	Depth (m)		Velocity (m/s)		Froude No		Freeboard (m) (1)	
		Min	Max	Min	Max	Min	Max	Min	Max
North-East PIC Spillway	5.2	0.21	0.48	0.76	2.35	0.38	1.68	0.52	0.79
PIC K	5.2	0.47	0.62	0.62	1.02	0.28	0.57	0.38	0.53

Notes:

(1) Freeboard calculated assuming a channel depth of 1 m.

5.0 WETLANDS IN PIC

The wetlands proposed to be constructed in the PICs at closure have been hydraulically designed to achieve a minimum residence time of seven (7) days for rainfall events up to the 1-year, 1-hour duration rainfall event, corresponding to 10.8 mm rainfall depth. The residence time is based on the water quality at closure assessment (see Appendix H of the Design Report).

The time of concentration at closure for each PIC segment has been estimated to be approximately 1-hour, and the 1- year return period event has been selected in accordance with the following design guidance documents:

- The CIRIA SuDS Manual – design water quality event for components that treat runoff as it flows through vegetation (CIRIA, 2015); and
- The Greater Dublin Regional Code of Practise for Drainage Works – design criteria for river water quality protection.

For events up to the 1-year event, each PIC segment has been designed to contain and slowly release runoff reporting directly to the PIC segment. Release rates from each segment will be controlled through the implementation of a flow control device which will facilitate a minimum residence time for runoff of seven (7) days within the wetland systems.

The preliminary design flow control release rates for the 1-year, 1-hour rainfall event are 2.4 litres per second for PIC segments A, B, C and D; and 2.7 litres per second for all other PIC segments.

For larger (extreme) rainfall events up to and including the IDF, inter-PIC discharge will be provided via riprap lined overflow spillways provided at each PIC segment division. The invert level for these overflow spillways is set 0.7m above the estimated 1-year, 1- hour event design water level for each PIC segment. These design details are presented on Drawing 05 and the hydraulic assessment calculations are provided in Appendix C.

6.0 CLOSING REMARKS

Golder has completed the engineering level closure designs for water management infrastructure to transfer runoff from the PIC system and the SWP to the downstream adjacent external drains. The spillways have been designed to safely convey the BRDA IDF post-closure and allow for attenuation of discharge to greenfield rates for events up to the 1 in 100-year event.

Golder's design consists of two U-shaped concrete channels through the OPW of PIC-G (PIC Breach Spillway #1) and through the OPW of PIC-D (PIC Breach Spillway #2). The concrete channels have been designed with a width of 1.0m and discharge to trapezoidal riprap lined chutes. The minimum depths of the concrete channels are 2.15m (PIC Breach Spillway #1) and 1.80m (PIC Breach Spillway #2). The riprap chutes have been designed with a bottom width of 3.0 m and a depth of 1.5m (PIC Breach Spillway #1) and 1.0m (PIC Breach Spillway #2); and ultimately discharge to the existing downstream drains.

Golder has completed the engineering level closure design for water management infrastructure to transfer the IDF from PIC-L to PIC-K. The design consists of a riprap lined trapezoidal channel with a bottom width of 10.0 m and a depth of 1.0 m.

Hydraulic modelling of the proposed spillways, PIC Breach Spillway #1, PIC Breach Spillway #2 and the North-East PIC Spillway, was completed using HEC-RAS software to assess the performance of the design during the considered events. The modelling demonstrated that the proposed design meets the design criteria. The minimum freeboard values achieved during the IDF event were 0.38 m, 0.72 m and 0.52 m in the PIC Breach Spillway #1, the PIC Breach Spillway #2 and the North-East PIC Spillway, respectively. For the three proposed spillways, hydraulic jumps occurred upstream or at the confluence of the spillway and the receptor.

The wetlands proposed to be constructed in the PICs at closure have been hydraulically designed to achieve a minimum residence time of seven (7) days for rainfall events up to the 1-year, 1-hour duration rainfall event, corresponding to 10.8 mm rainfall depth. The residence time is based on the water quality at closure assessment (see Appendix H of the EIAR).

Further design information and detailing will be required to progress these designs to 'construction' stage designs. At that point, design optimisation or value engineering of the system could be considered.

7.0 REFERENCES

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Signature Page

Golder Associates Ireland Limited

Dhealy

Brian Keenan

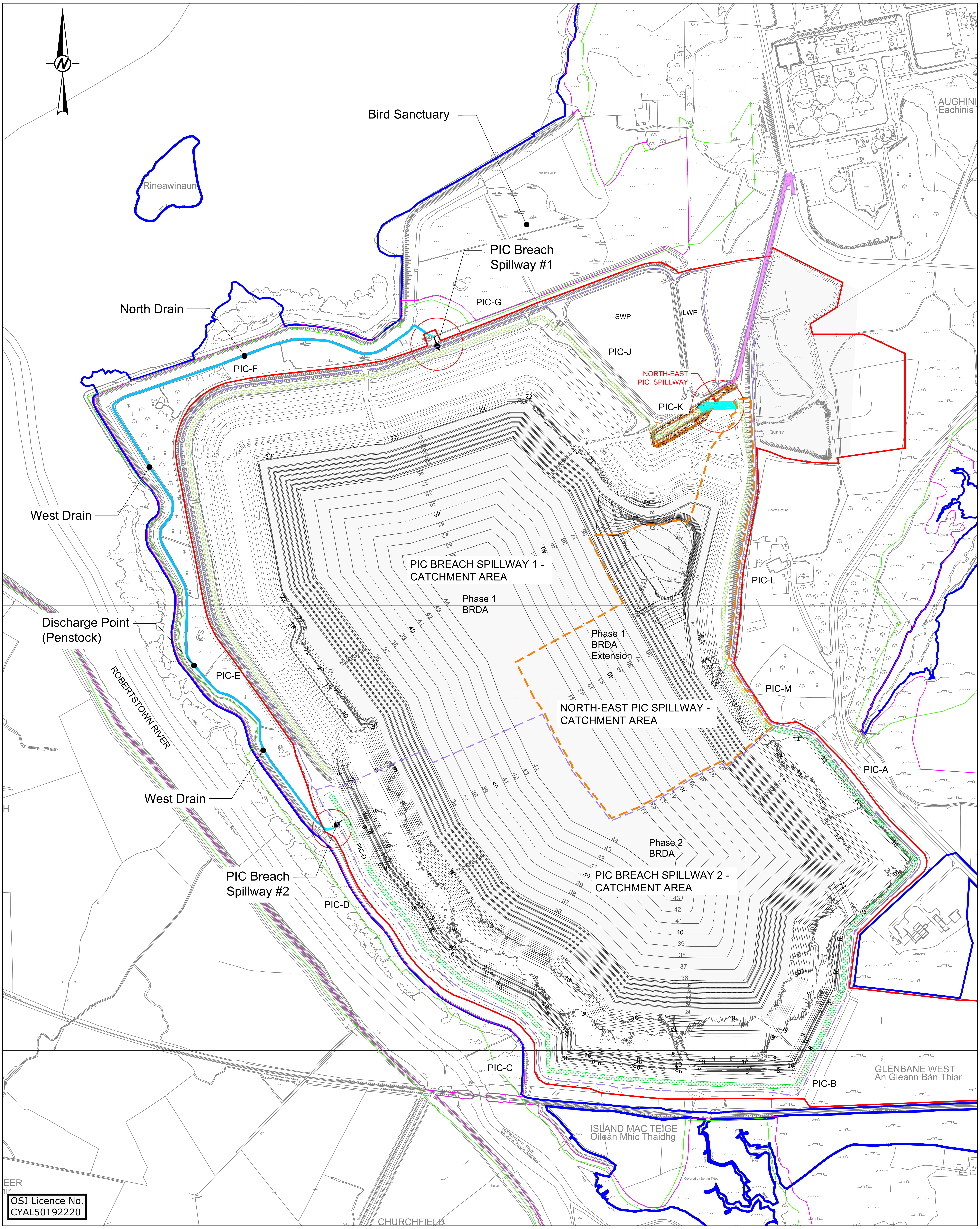
Darren Healy
Senior Water Resources Engineer

Brian Keenan
Project Manager / Associate

GLA/BK

APPENDIX A

Drawings



LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 OGL CONTOUR (mOD)
- WETLANDS IN PHASE 1 BRDA (8.5m WIDTH)
- WETLANDS IN PHASE 2 BRDA (10.0m WIDTH)
- PROPOSED DISCHARGE ROUTE VIA NORTH DRAIN AND ROBERTSTOWN DITCH
- SPECIAL PROTECTION AREA (SPA) BOUNDARY (JUNE 2020)
- SPECIAL AREA OF CONSERVATION (SAC) BOUNDARY (JUNE 2020)
- PIC SPILLWAY CATCHMENTS (SPILLWAY #1 AND SPILLWAY #2)
- NORTH-EAST PIC SPILLWAY CATCHMENT

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES, TO O.S. DATUM & CORRESPOND TO APRIL 2021 AERIAL SURVEY

DIMENSIONS ARE IN METRES.

0 200 400
1:4,000 METRES

CLIENT
AUGHINISH ALUMINA LTD

PROJECT
BRDA RAISE DESIGN

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-11-15
DESIGNED	GA
PREPARED	ECS
REVIEWED	DH
APPROVED	BK

TITLE
PIC SPILLWAY BREACH LOCATIONS AND CATCHMENTS

PROJECT NO.	SCALE	REV.	DRAWING NO.
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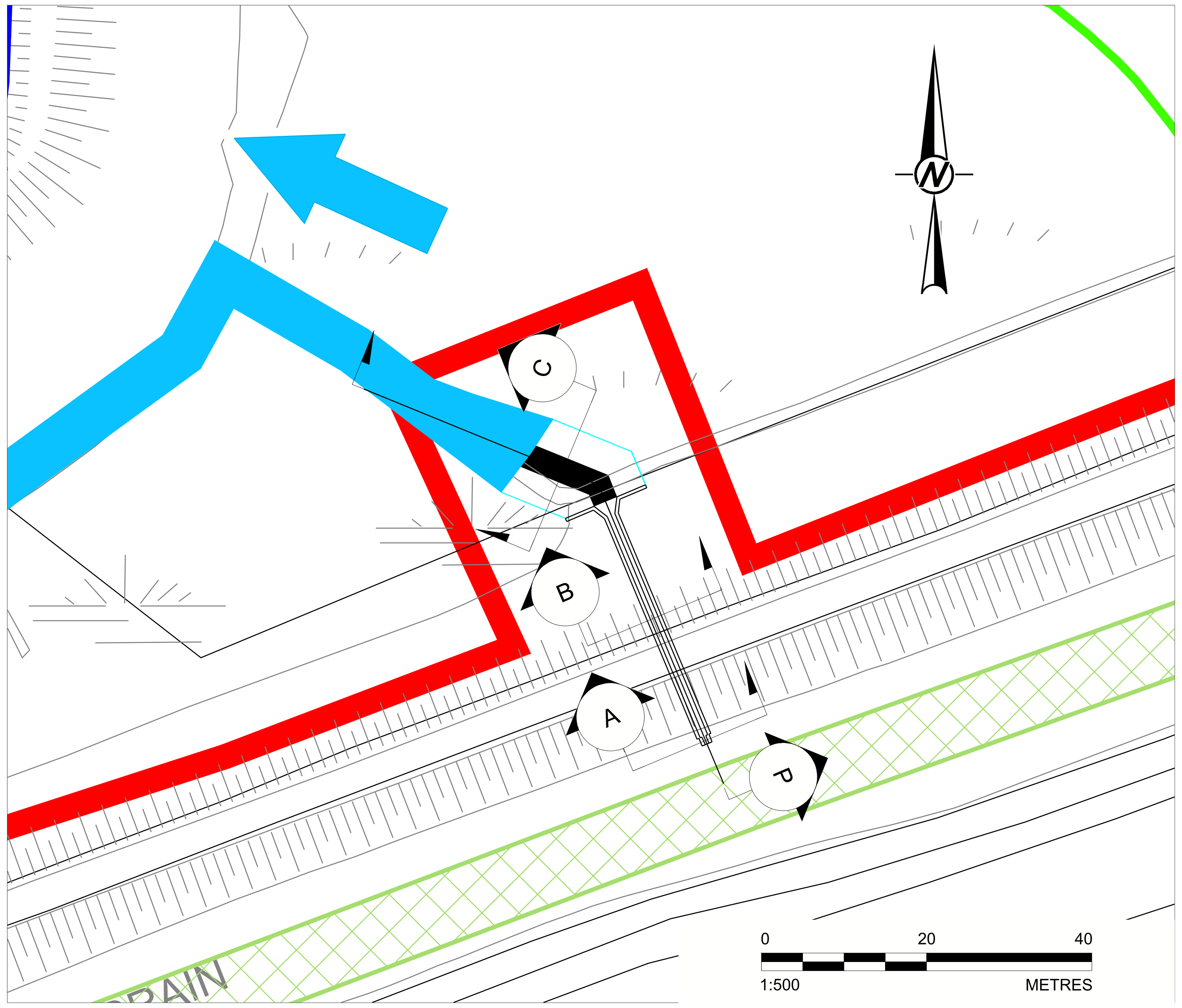

PIC BREACH SPILLWAY 1 - PLAN
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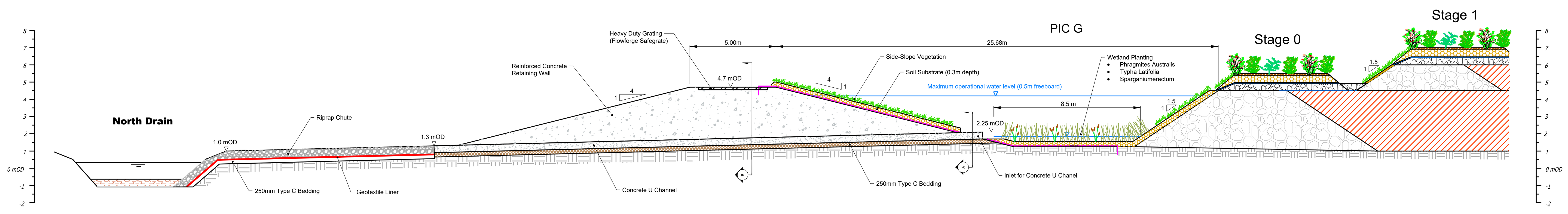

PROFILE P - PIC BREACH SPILLWAY #1
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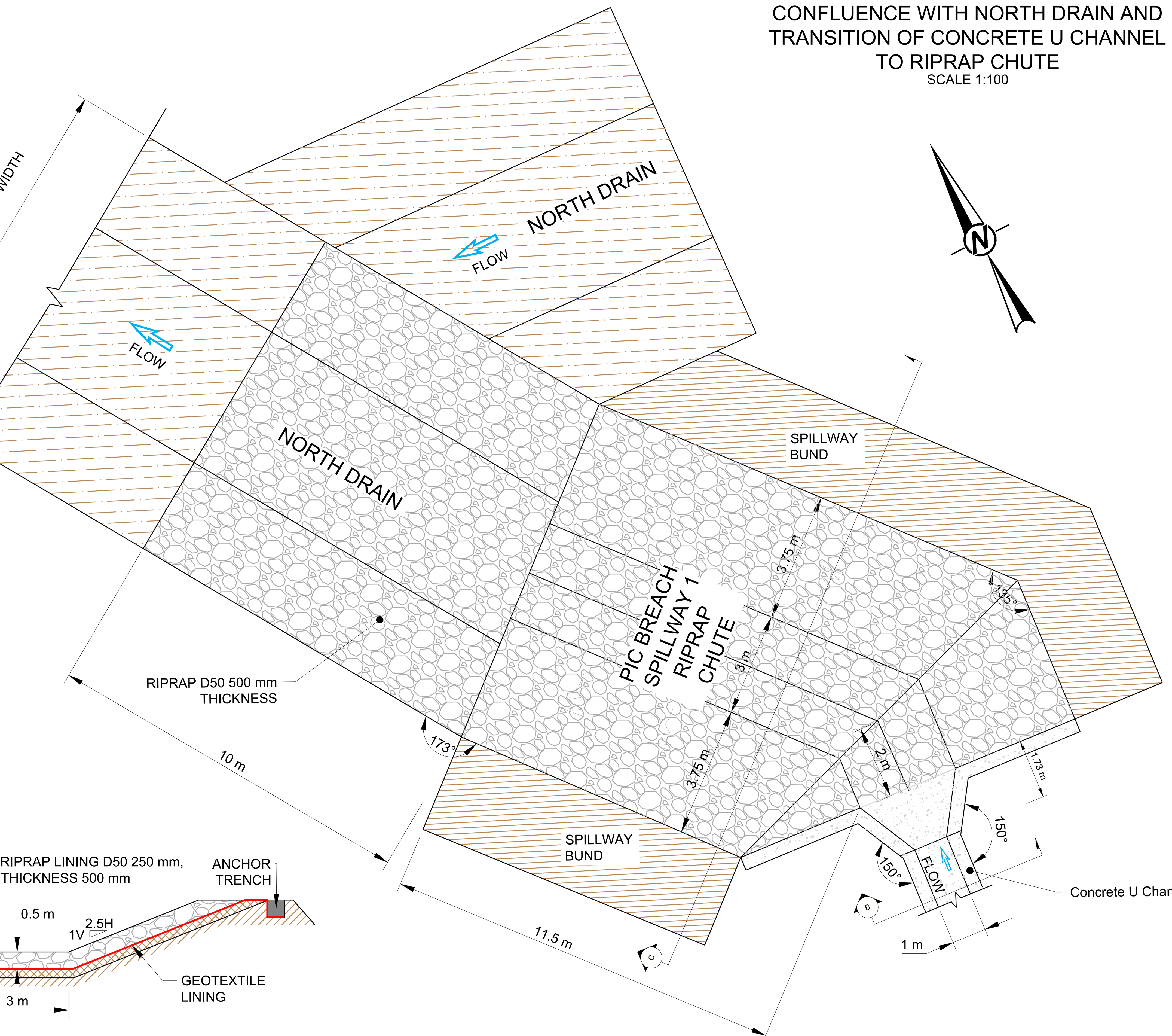
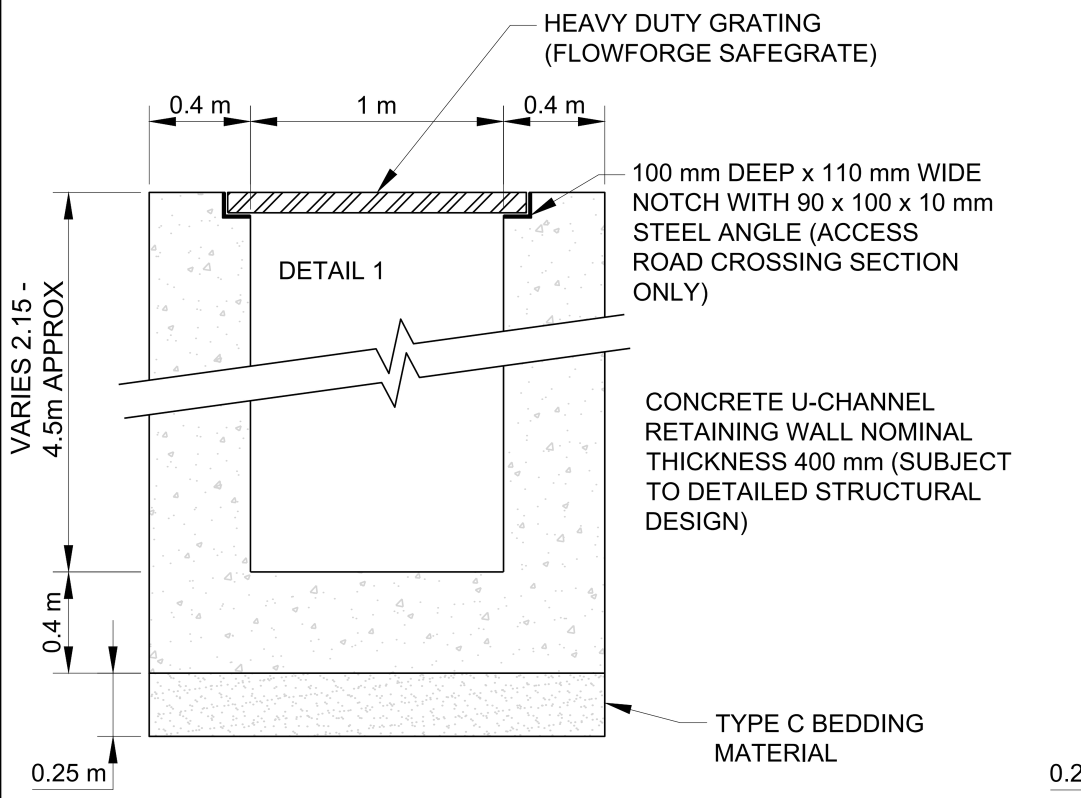
CONFLUENCE WITH NORTH DRAIN AND TRANSITION OF CONCRETE U CHANNEL TO RIPRAP CHUTE
SCALE 1:100





SECTION B - CONCRETE U CHANNEL
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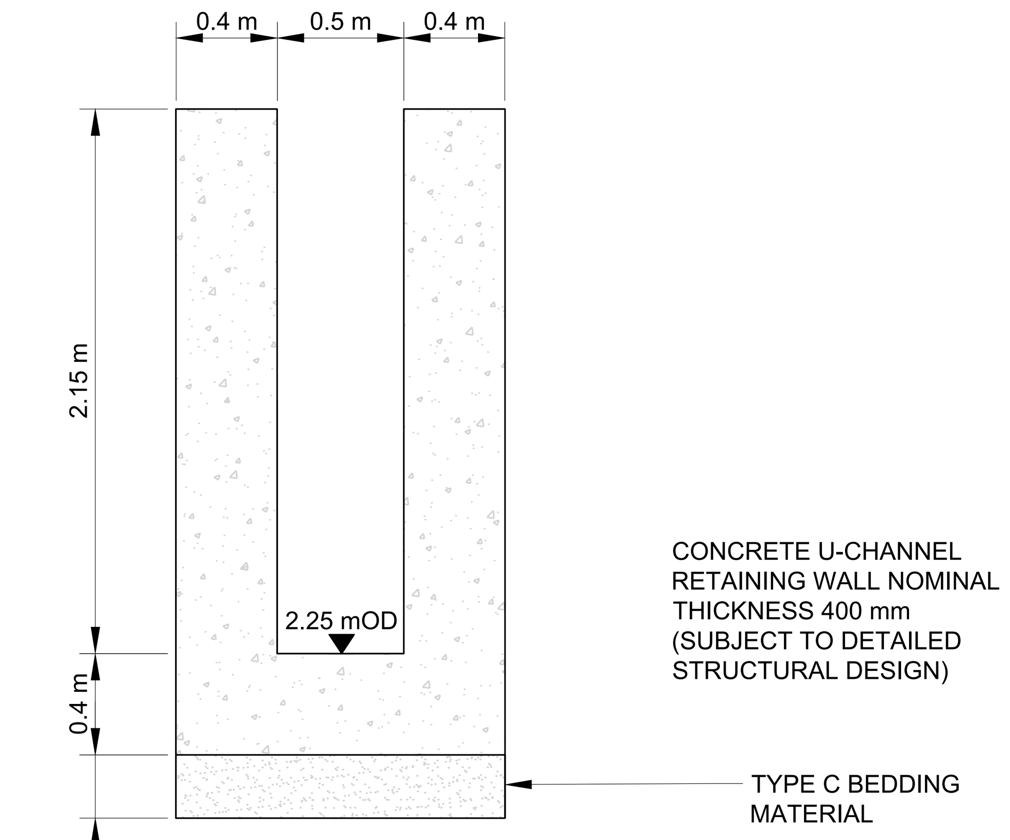

SECTION A - CONCRETE U CHANNEL INLET DETAIL
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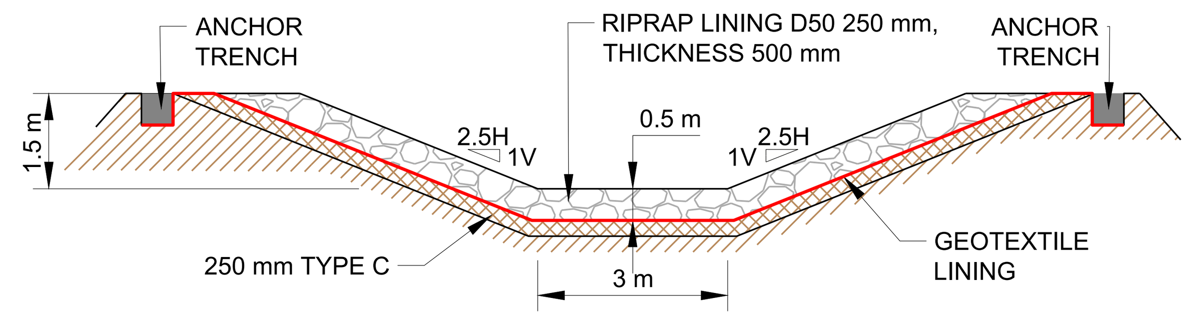
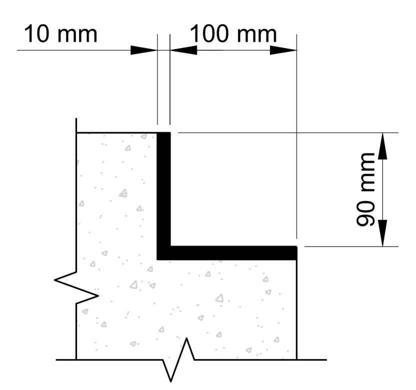


SECTION C - RIPRAP CHUTE TYPICAL DETAIL
SCALE 1:100



DETAIL 1 - ACCESS ROAD CROSSING DETAIL
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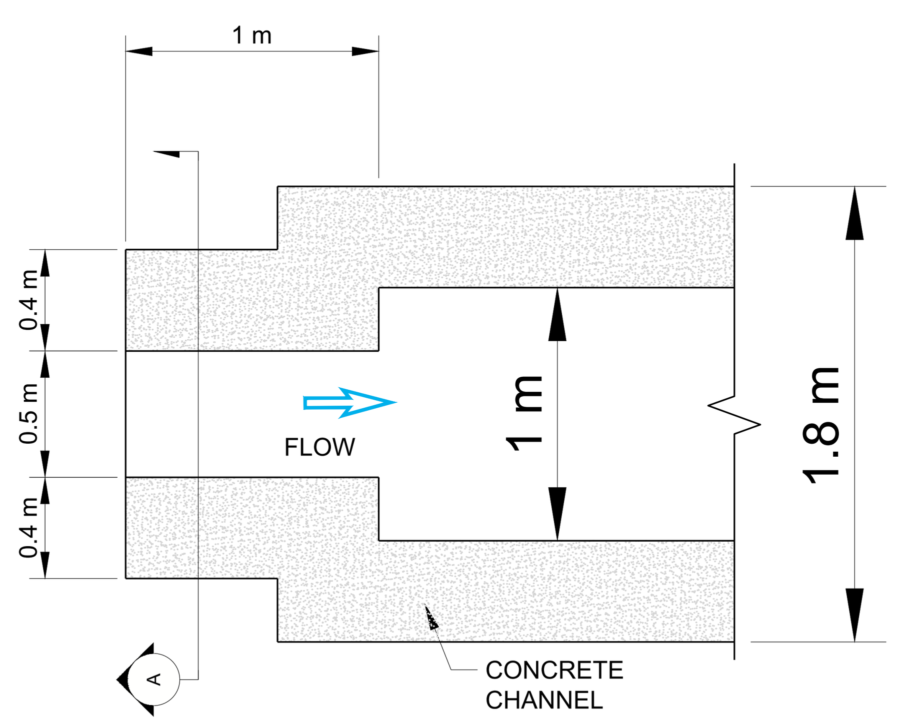

CONCRETE U CHANNEL INLET DETAIL - PLAN
SCALE 1:25

LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 OGL CONTOUR (mOD)
- WETLANDS IN PHASE 1 BRDA (8.5m WIDTH)
- WETLANDS IN PHASE 2 BRDA (10.0m WIDTH)
- PROPOSED DISCHARGE ROUTE
- SPECIAL PROTECTION AREA (SPA) BOUNDARY (JUNE 2020)
- SPECIAL AREA OF CONSERVATION (SAC) BOUNDARY (JUNE 2020)

CLIENT
AUGHINISH ALUMINA LTD

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-10-29
DESIGNED	GA
PREPARED	ECS
REVIEWED	GA
APPROVED	BK

PROJECT
BRDA RAISE DESIGN

TITLE
PIC BREACH SPILLWAY #1: DESIGN DETAILS

PROJECT NO.	SCALE	REV.	DRAWING NO.
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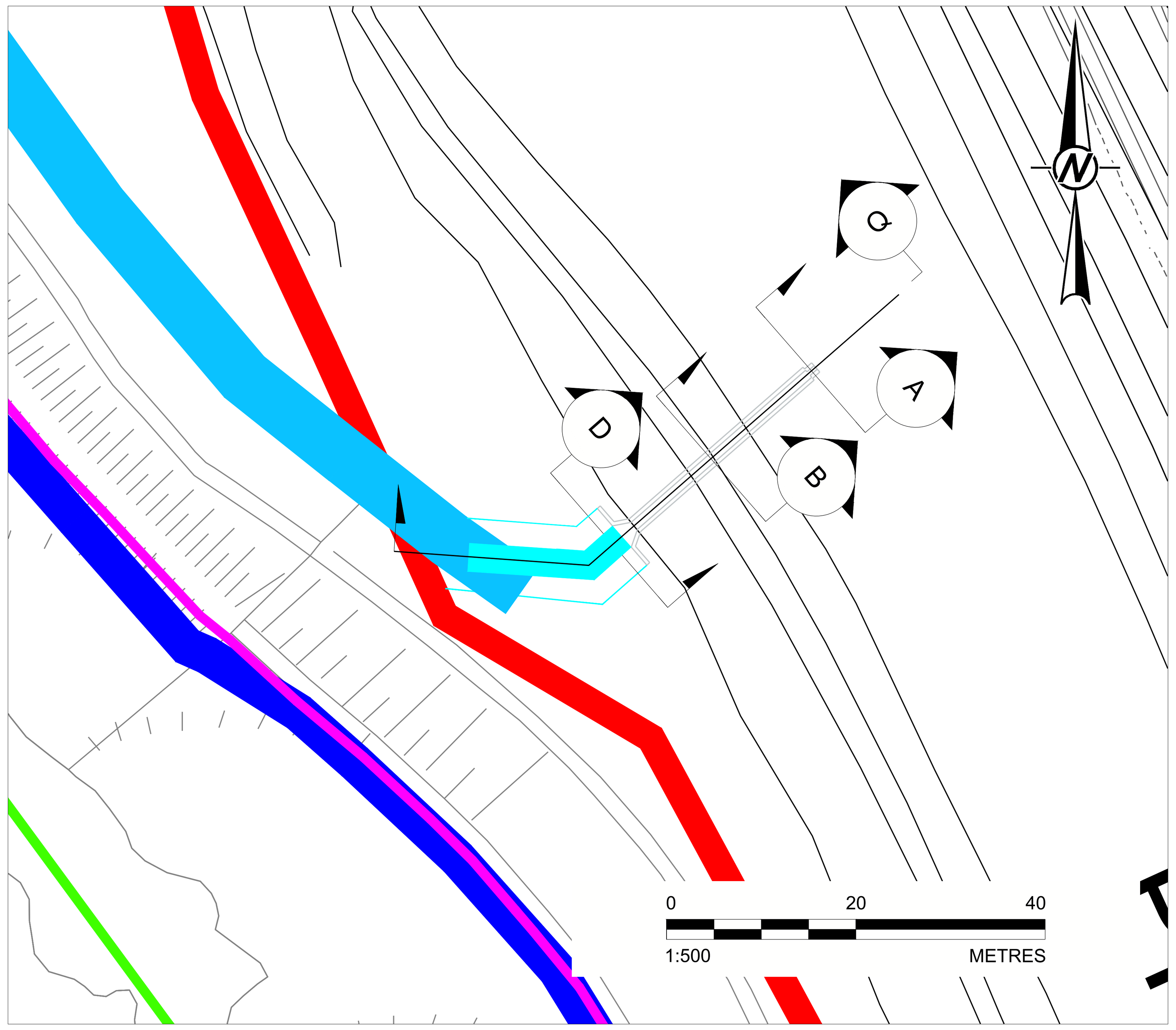

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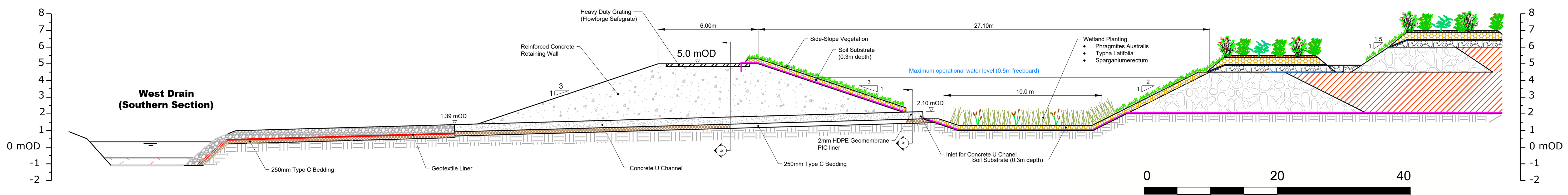

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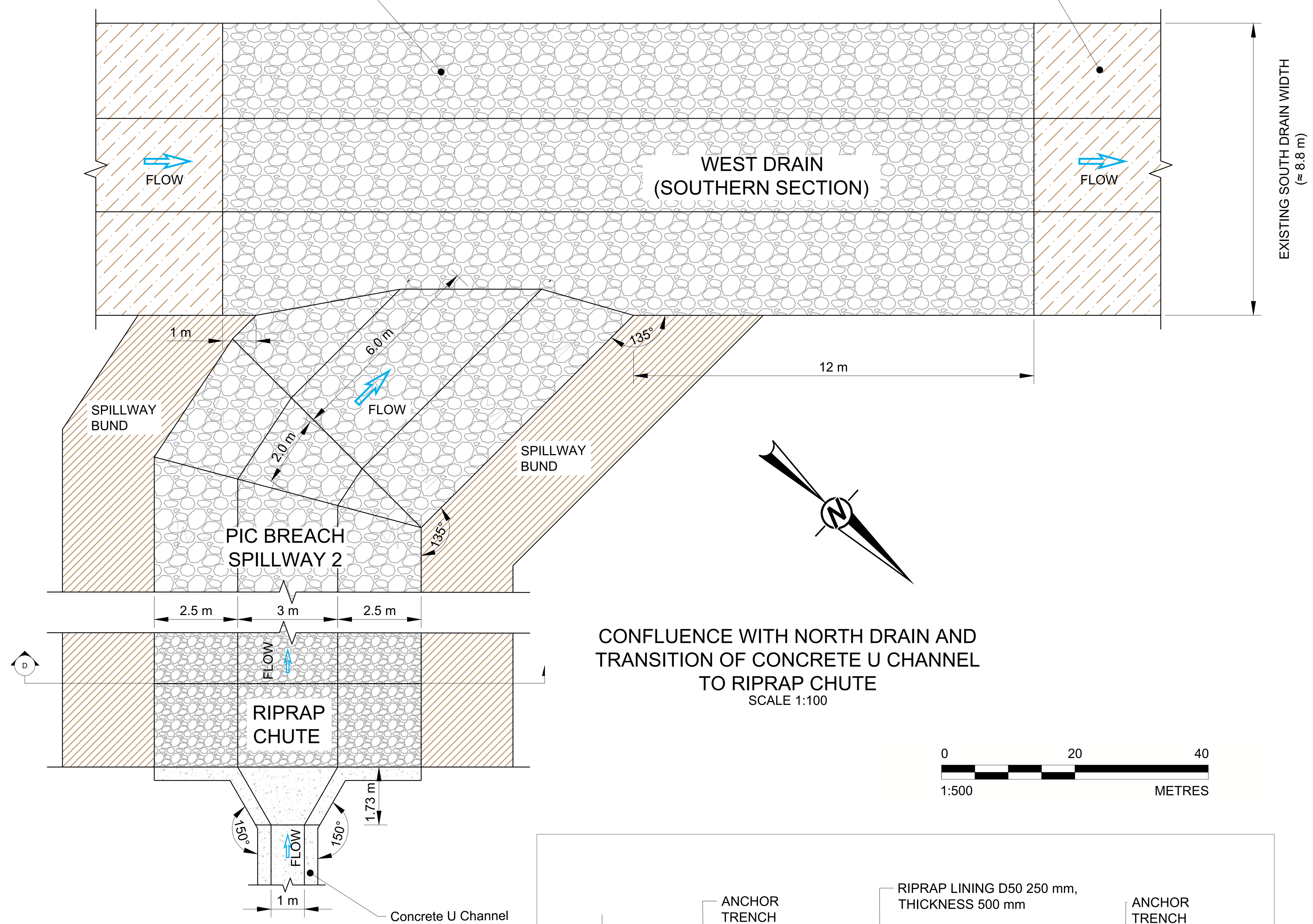

CONFLUENCE WITH NORTH DRAIN AND TRANSITION OF CONCRETE U CHANNEL TO RIPRAP CHUTE
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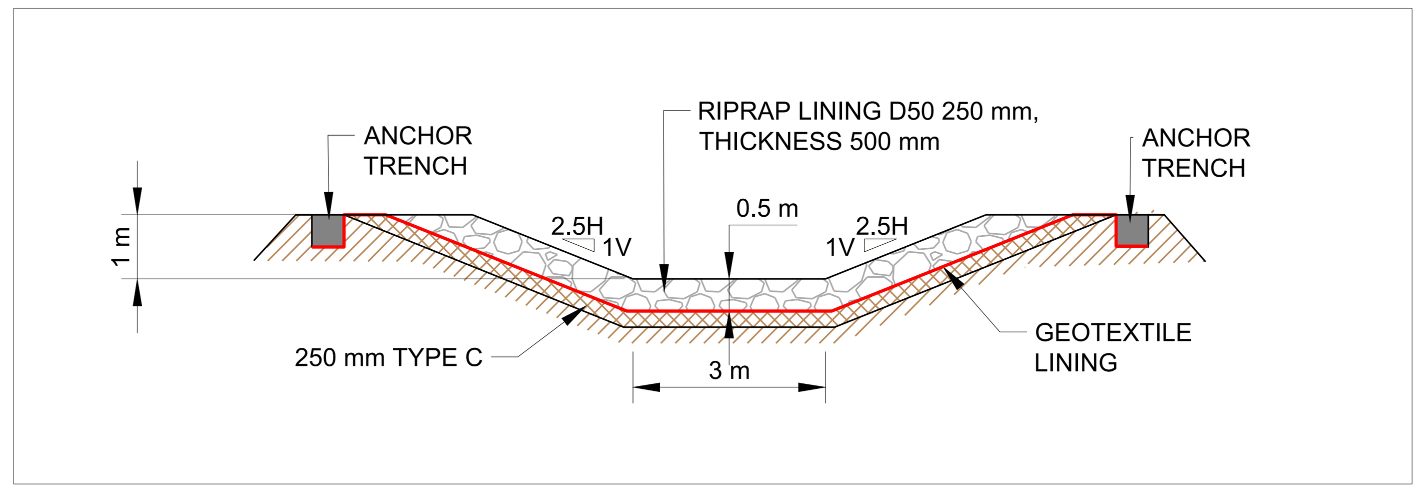

SECTION D - RIPRAP CHUTE TYPICAL DETAIL
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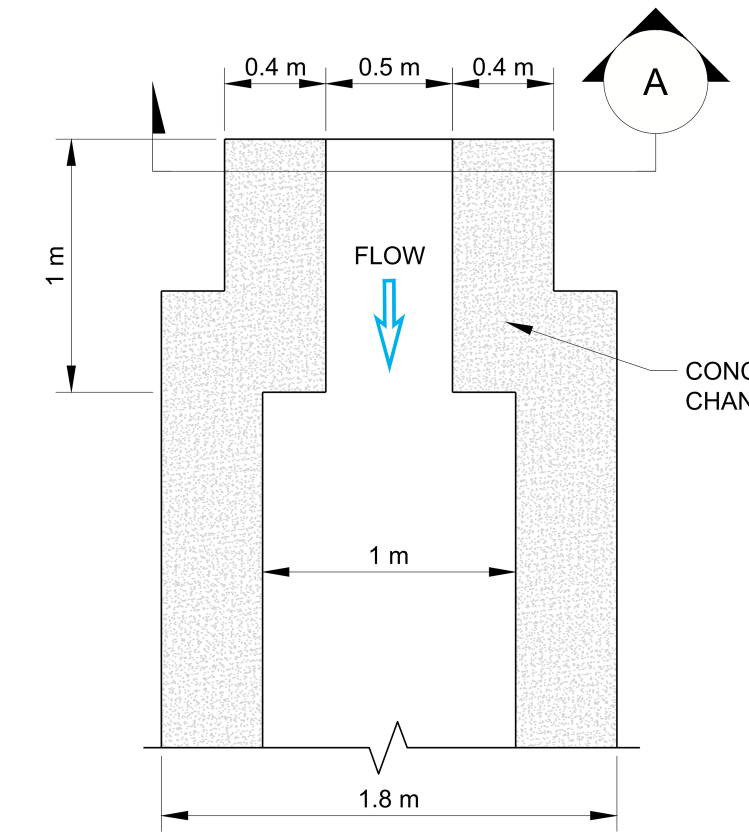

CONCRETE U CHANNEL - INLET DETAIL PLAN
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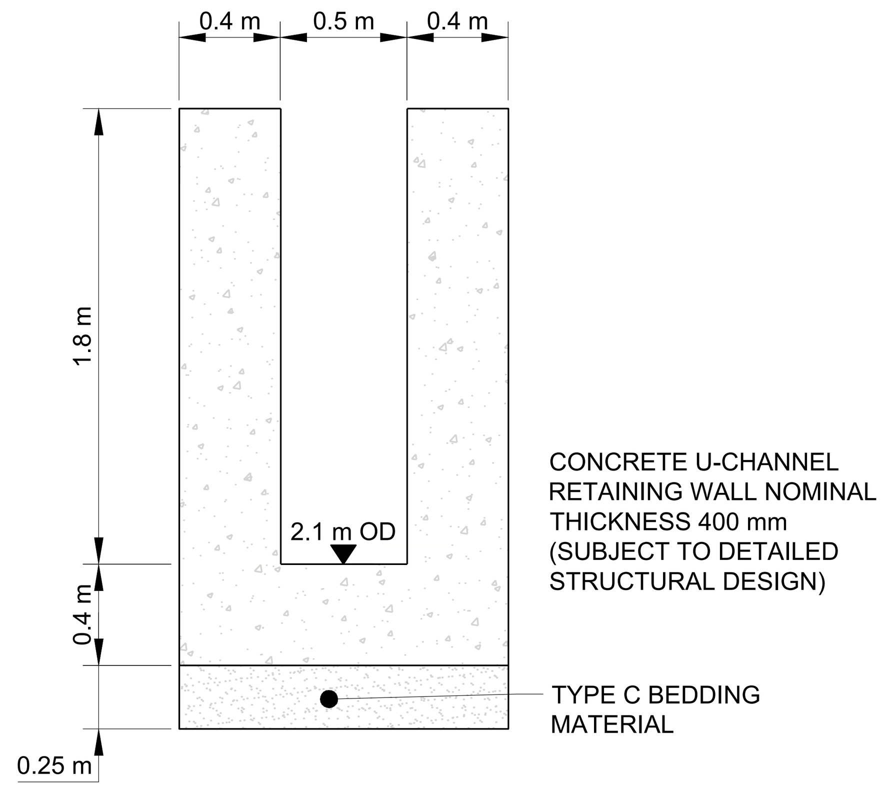

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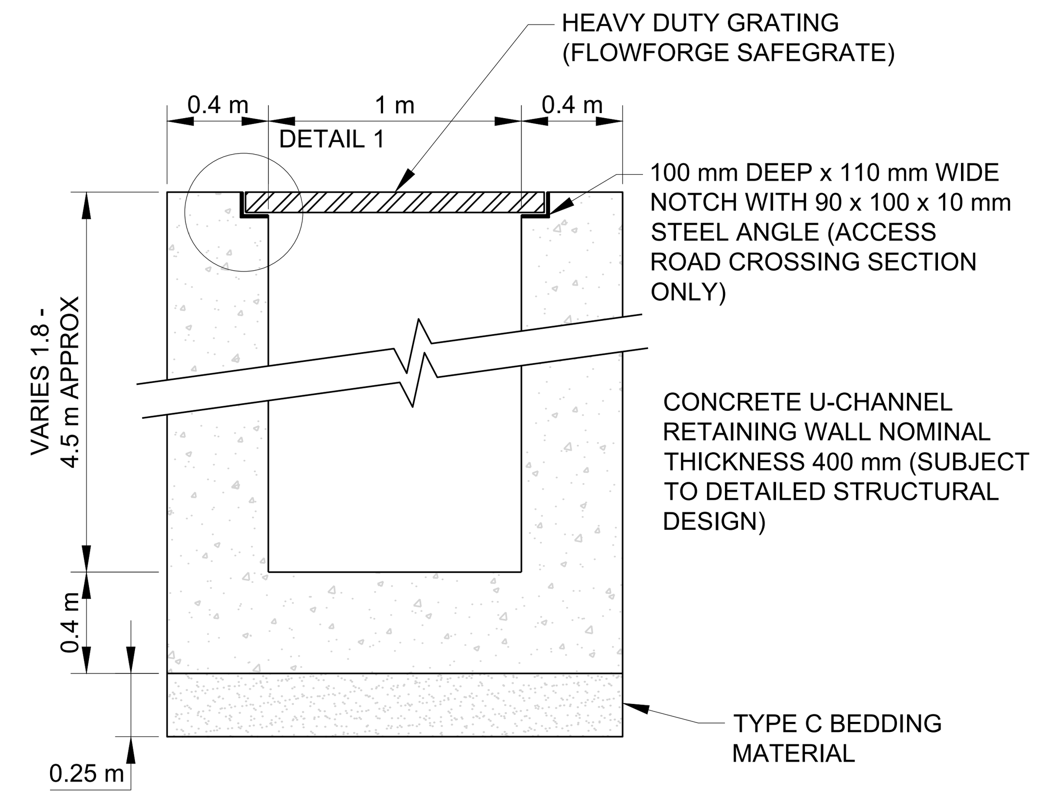

SECTION B - CONCRETE U CHANNEL
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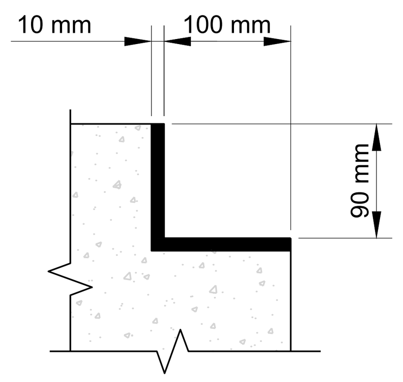

DETAIL 1 - ACCESS ROAD CROSSING DETAIL
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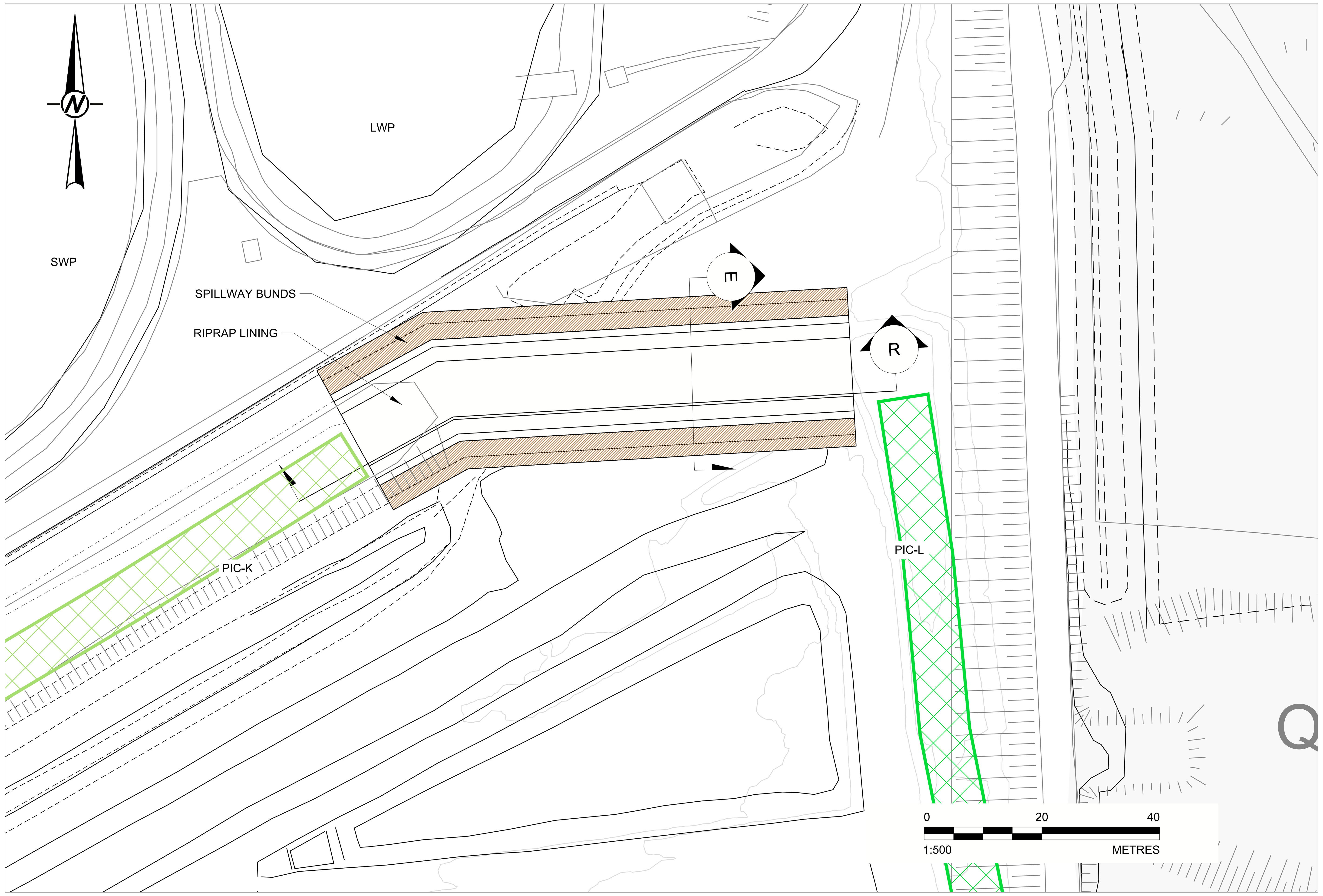
LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 OGL CONTOUR (mOD)
- WETLANDS IN PHASE 1 BRDA (8.5m WIDTH)
- WETLANDS IN PHASE 2 BRDA (10.0m WIDTH)
- PROPOSED DISCHARGE ROUTE
- SPECIAL PROTECTION AREA (SPA) BOUNDARY (JUNE 2020)
- SPECIAL AREA OF CONSERVATION (SAC) BOUNDARY (JUNE 2020)

CLIENT	AUGHINISH ALUMINA LTD
CONSULTANT	GOLDER MEMBER OF WSP
YYYY-MM-DD	2021-11-15
DESIGNED	GA
PREPARED	ECS
REVIEWED	GA
APPROVED	BK
PROJECT	BRDA RAISE DESIGN
TITLE	PIC BREACH SPILLWAY #2: DESIGN DETAILS
PROJECT NO.	20143076
SCALE	As shown A1
REV.	A
DRAWING NO.	03



NORTH-EAST PIC SPILLWAY - PLAN
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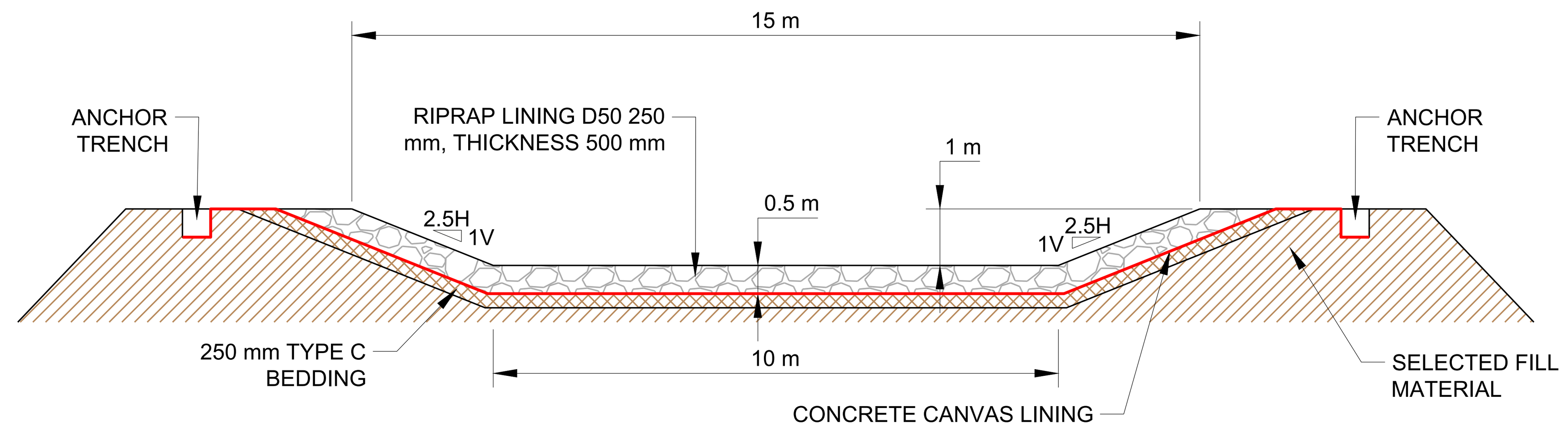

SECTION E - NORTH-EAST PIC SPILLWAY TYPICAL DETAIL
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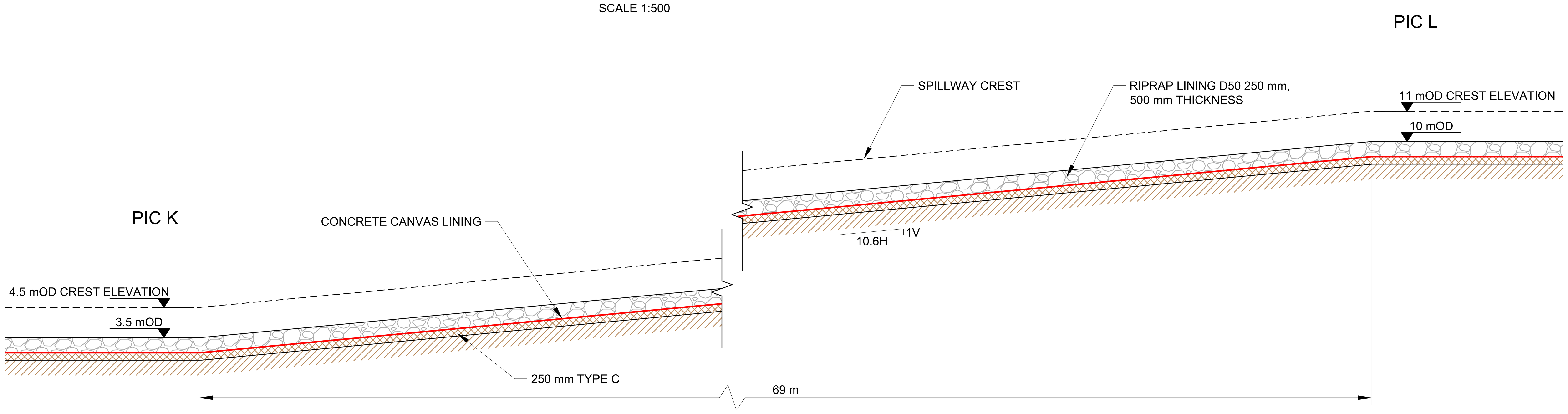

PROFILE R - NORTH-EAST PIC SPILLWAY
SCALE 1:100

0 5 10
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LEGEND

- AAL OWNERSHIP BOUNDARY
- APPLICATION BOUNDARY
- 22 OGL CONTOUR (mOD)
- WETLANDS IN PHASE 1 BRDA (8.5m WIDTH)
- WETLANDS IN PHASE 2 BRDA (10.0m WIDTH)
- PROPOSED DISCHARGE ROUTE
- SPECIAL PROTECTION AREA (SPA) BOUNDARY (JUNE 2020)
- SPECIAL AREA OF CONSERVATION (SAC) BOUNDARY (JUNE 2020)

CLIENT
AUGHINISH ALUMINA LTD

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-10-29
DESIGNED	GA
PREPARED	ECS
REVIEWED	GA
APPROVED	BK

PROJECT
BRDA RAISE DESIGN

TITLE
NORTH-EAST PIC SPILLWAY: DESIGN DETAILS

PROJECT NO.	SCALE	REV.	DRAWING NO.
20143076	As shown A1	A	04

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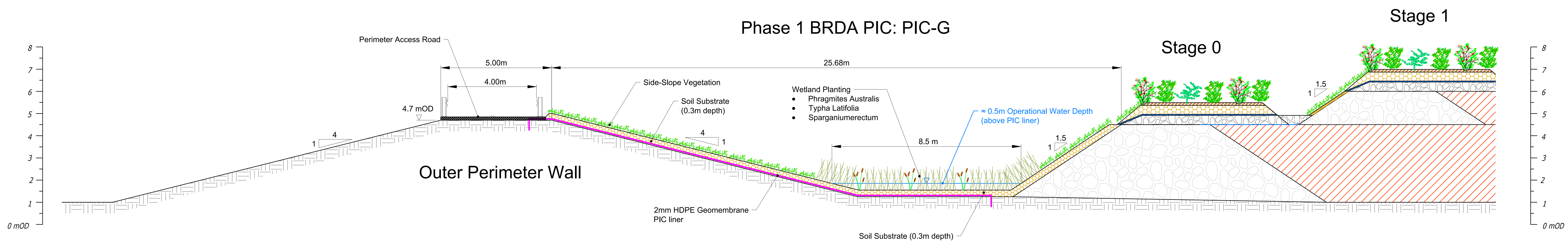

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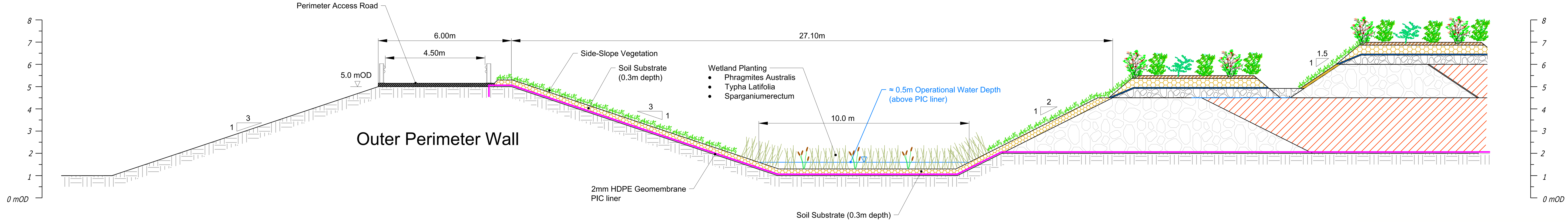
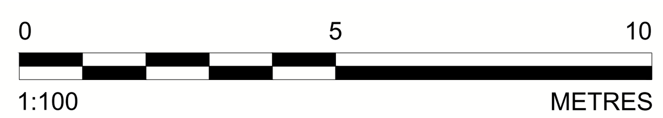
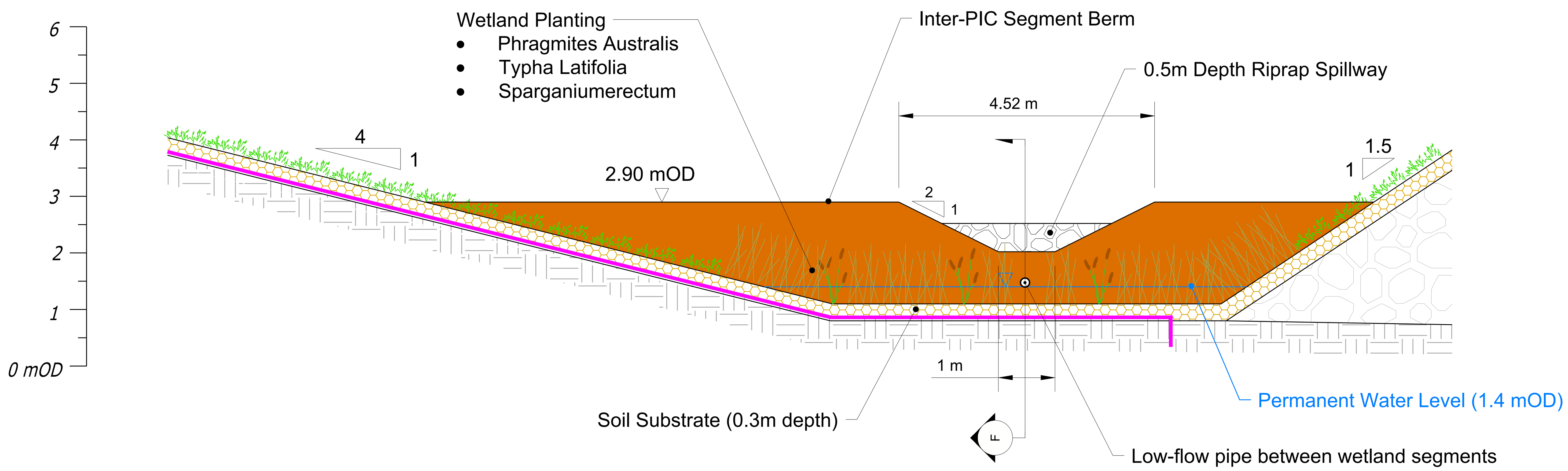
Phase 2 BRDA PIC: PIC-D

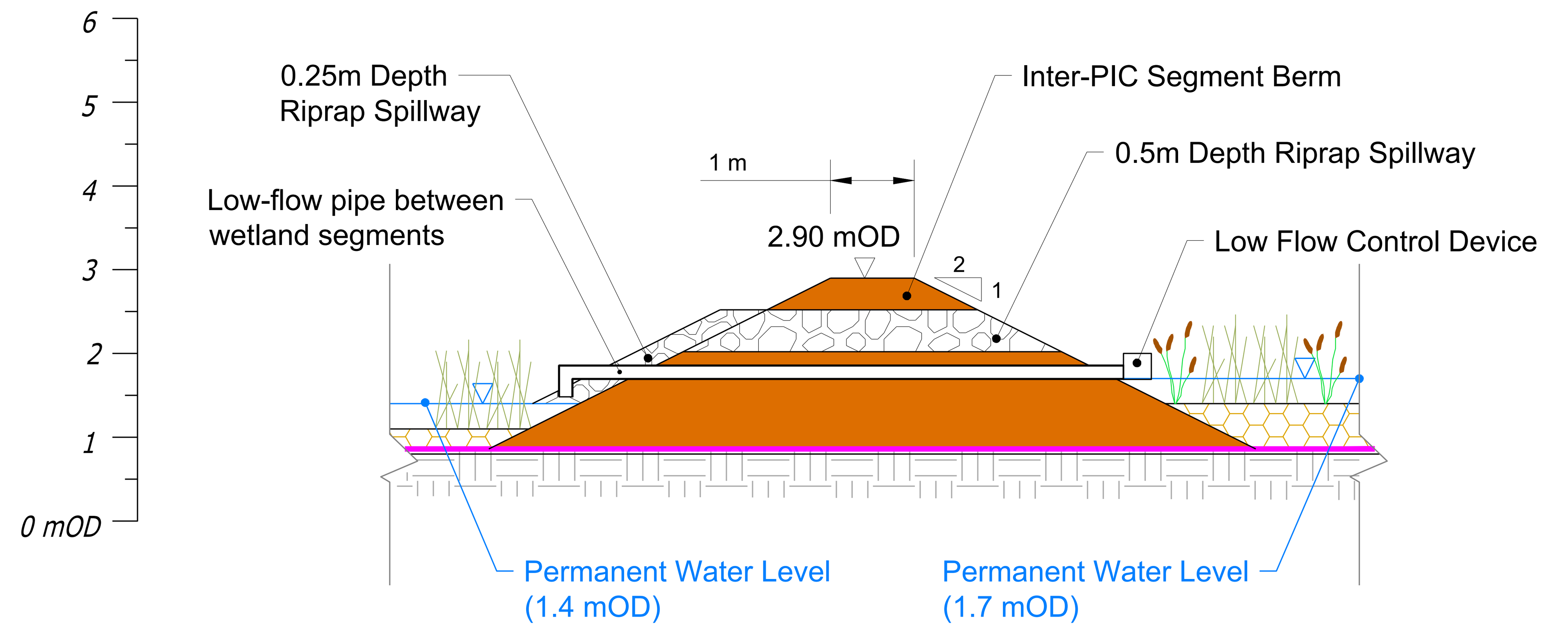


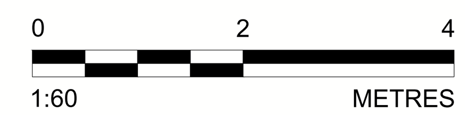


CROSS-SETION: WETLANDS CONSTRUCTED IN PIC-D (PHASE 2 BRDA)
SCALE 1:100



ELEVATION: INTER-PIC-G SEGMENT BERM
SCALE 1:60





SECTION F: INTER-PIC-G SEGMENT BERM
SCALE 1:60

PIC-G: Representative Inter-PIC Segment Berm
- Permanent Water Elevation = 1.40 mOD
- Peak Water Elevation (1-year, 1-hour) = 1.82 mOD
- Base elevation of Rip-rap Overflow = 2.02 mOD
- Top elevation of Rip-rap Overflow = 2.52 mOD
- Crest of Inter-PIC Segment Berm = 2.90 mOD

CLIENT
AUGHINISH ALUMINA LTD

PROJECT
BRDA RAISE DESIGN

CONSULTANT
GOLDER MEMBER OF WSP

YYYY-MM-DD	2021-10-29
DESIGNED	GA
PREPARED	ECS
REVIEWED	GA
APPROVED	BK

TITLE
WETLANDS IN PIC: DESIGN DETAILS

PROJECT NO.	SCALE	REV.	DRAWING NO.
20143076	As shown A1	A	05

APPENDIX B

HEC-RAS Analysis

PIC Breach Spillway #1: HEC-RAS Hydraulic Analysis Results (1 in 1-year event)

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Flow Depth (m)	Freeboard (m)
PIC Breach Spillway 1	103.1	1 year	0.09	2.25	2.39	2.39	2.47	0.005852	1.19	0.07	0.5	1.01	0.14	0.86
PIC Breach Spillway 1	102.15	1 year	0.09	2.22	2.33	2.36	2.45	0.013072	1.57	0.05	0.5	1.53	0.11	0.89
PIC Breach Spillway 1	102.1	1 year	0.09	2.22	2.27	2.31	2.44	0.041324	1.88	0.05	1	2.81	0.05	0.95
PIC Breach Spillway 1	100.16*	1 year	0.09	2.16	2.21	2.25	2.37	0.031902	1.73	0.05	1	2.49	0.05	0.95
PIC Breach Spillway 1	98.23*	1 year	0.09	2.11	2.16	2.2	2.31	0.030476	1.71	0.05	1	2.44	0.05	0.95
PIC Breach Spillway 1	96.29*	1 year	0.09	2.05	2.1	2.14	2.25	0.030476	1.71	0.05	1	2.44	0.05	0.95
PIC Breach Spillway 1	94.36*	1 year	0.09	2	2.05	2.09	2.19	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	92.42*	1 year	0.09	1.94	1.99	2.03	2.14	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	90.49*	1 year	0.09	1.88	1.93	1.97	2.08	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	88.55*	1 year	0.09	1.83	1.88	1.92	2.02	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	86.62*	1 year	0.09	1.77	1.82	1.86	1.97	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	84.68*	1 year	0.09	1.72	1.77	1.81	1.91	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	82.74*	1 year	0.09	1.66	1.71	1.75	1.86	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	80.81*	1 year	0.09	1.6	1.65	1.69	1.8	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	78.87*	1 year	0.09	1.55	1.6	1.64	1.74	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	76.94*	1 year	0.09	1.49	1.54	1.58	1.69	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	75.00*	1 year	0.09	1.44	1.49	1.53	1.63	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	73.066	1 year	0.09	1.38	1.43	1.47	1.58	0.029526	1.69	0.05	1	2.41	0.05	0.95
PIC Breach Spillway 1	71.3013	1 year	0.09	1.33	1.35	1.37	1.49	0.10448	1.65	0.05	3	4	0.02	0.98
PIC Breach Spillway 1	71.2013	1 year	0.09	1.32	1.34	1.36	1.46	0.743185	1.53	0.06	3.06	3.61	0.02	0.98
PIC Breach Spillway 1	69.5162	1 year	0.09	1.28	1.33	1.32	1.35	0.02172	0.52	0.16	3.18	0.73	0.05	0.95
PIC Breach Spillway 1	68.250*	1 year	0.09	1.25	1.31		1.32	0.018598	0.5	0.17	3.19	0.68	0.06	0.94
PIC Breach Spillway 1	66.9828	1 year	0.09	1.23	1.28		1.29	0.029314	0.57	0.15	3.16	0.84	0.05	0.95
PIC Breach Spillway 1	65.212*	1 year	0.09	1.17	1.22	1.21	1.23	0.036327	0.61	0.14	3.15	0.92	0.05	0.95
PIC Breach Spillway 1	63.442*	1 year	0.09	1.11	1.16		1.18	0.02704	0.56	0.15	3.17	0.81	0.05	0.95
PIC Breach Spillway 1	61.672*	1 year	0.09	1.06	1.16		1.17	0.002217	0.26	0.33	3.35	0.26	0.1	0.9
PIC Breach Spillway 1	59.9013	1 year	0.09	1	1.16		1.16	0.000516	0.16	0.53	3.54	0.13	0.16	0.84
PIC Breach Spillway 1	59.8013	1 year	0.09	0.99	1.16		1.16	0.000128	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	57.808*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	55.815*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	53.821*	1 year	0.09	0.99	1.16		1.16	0.000128	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	51.828*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	49.834*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83

PIC Breach Spillway 1	47.841*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	45.848*	1 year	0.09	0.99	1.16		1.16	0.000128	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	43.854*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	41.861*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	39.868*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	37.874*	1 year	0.09	0.99	1.16		1.16	0.000128	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	35.881*	1 year	0.09	0.99	1.16		1.16	0.000128	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	33.887*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	31.894*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	29.901*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	27.907*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	25.914*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	23.921*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	21.927*	1 year	0.09	0.99	1.16		1.16	0.000131	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	19.934*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	17.940*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	15.947*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	13.954*	1 year	0.09	0.99	1.16		1.16	0.000131	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	11.960*	1 year	0.09	0.99	1.16		1.16	0.000128	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	9.967*	1 year	0.09	0.99	1.16		1.16	0.000129	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	7.974*	1 year	0.09	0.99	1.16		1.16	0.00013	0.11	0.77	4.8	0.09	0.17	0.83
PIC Breach Spillway 1	5.980*	1 year	0.09	0.99	1.15		1.16	0.00013	0.11	0.77	4.8	0.09	0.16	0.84
PIC Breach Spillway 1	3.987*	1 year	0.09	0.99	1.15		1.16	0.000129	0.11	0.77	4.8	0.09	0.16	0.84
PIC Breach Spillway 1	1.993*	1 year	0.09	0.99	1.15		1.16	0.000129	0.11	0.77	4.8	0.09	0.16	0.84
PIC Breach Spillway 1	0	1 year	0.09	0.99	1.15	1.02	1.15	0.00013	0.11	0.77	4.8	0.09	0.16	0.84

PIC Breach Spillway #1: HEC-RAS Hydraulic Analysis Results (1 in 100-year event)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # C	Flow Depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
PIC Breach Spillway 1	103.1	100 year	0.66	2.25	2.81	2.81	3.09	0.009724	2.35	0.28	0.5	1.01	0.56	0.44
PIC Breach Spillway 1	102.15	100 year	0.66	2.22	2.72	2.78	3.07	0.012558	2.61	0.25	0.5	1.18	0.5	0.5
PIC Breach Spillway 1	102.1	100 year	0.66	2.22	2.41	2.57	3.04	0.030201	3.53	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	100.16*	100 year	0.66	2.16	2.35	2.52	2.98	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	98.23*	100 year	0.66	2.11	2.29	2.46	2.93	0.030061	3.52	0.19	1	2.61	0.18	0.82
PIC Breach Spillway 1	96.29*	100 year	0.66	2.05	2.24	2.4	2.87	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	94.36*	100 year	0.66	2	2.18	2.35	2.81	0.030061	3.52	0.19	1	2.61	0.18	0.82
PIC Breach Spillway 1	92.42*	100 year	0.66	1.94	2.13	2.29	2.76	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	90.49*	100 year	0.66	1.88	2.07	2.24	2.7	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	88.55*	100 year	0.66	1.83	2.01	2.18	2.65	0.030061	3.52	0.19	1	2.61	0.18	0.82
PIC Breach Spillway 1	86.62*	100 year	0.66	1.77	1.96	2.12	2.59	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	84.68*	100 year	0.66	1.72	1.9	2.07	2.53	0.030061	3.52	0.19	1	2.61	0.18	0.82
PIC Breach Spillway 1	82.74*	100 year	0.66	1.66	1.85	2.01	2.48	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	80.81*	100 year	0.66	1.6	1.79	1.95	2.42	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	78.87*	100 year	0.66	1.55	1.73	1.9	2.37	0.030061	3.52	0.19	1	2.61	0.18	0.82
PIC Breach Spillway 1	76.94*	100 year	0.66	1.49	1.68	1.84	2.31	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	75.00*	100 year	0.66	1.44	1.62	1.79	2.25	0.030061	3.52	0.19	1	2.61	0.18	0.82
PIC Breach Spillway 1	73.066	100 year	0.66	1.38	1.57	1.73	2.2	0.030061	3.52	0.19	1	2.61	0.19	0.81
PIC Breach Spillway 1	71.3013	100 year	0.66	1.33	1.39	1.5	2.1	0.109057	3.73	0.18	3	4.93	0.06	0.94
PIC Breach Spillway 1	71.2013	100 year	0.66	1.32	1.38	1.48	2.07	0.979658	3.68	0.18	3.19	4.98	0.06	0.94
PIC Breach Spillway 1	69.5162	100 year	0.66	1.28	1.56	1.44	1.59	0.004558	0.67	0.98	3.94	0.43	0.28	0.72
PIC Breach Spillway 1	68.250*	100 year	0.66	1.25	1.56		1.58	0.003422	0.61	1.08	4.02	0.37	0.31	0.69
PIC Breach Spillway 1	66.9828	100 year	0.66	1.23	1.56		1.57	0.002635	0.56	1.18	4.11	0.33	0.33	0.67
PIC Breach Spillway 1	65.212*	100 year	0.66	1.17	1.56		1.57	0.001522	0.46	1.42	4.29	0.26	0.39	0.61
PIC Breach Spillway 1	63.442*	100 year	0.66	1.11	1.56		1.57	0.000944	0.39	1.67	4.48	0.21	0.45	0.55
PIC Breach Spillway 1	61.672*	100 year	0.66	1.06	1.56		1.56	0.000621	0.34	1.92	4.67	0.17	0.5	0.5
PIC Breach Spillway 1	59.9013	100 year	0.66	1	1.56		1.56	0.000424	0.3	2.2	4.86	0.14	0.56	0.44
PIC Breach Spillway 1	59.8013	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	0.43
PIC Breach Spillway 1	57.808*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	55.815*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.72	0.1	0.57	1.03
PIC Breach Spillway 1	53.821*	100 year	0.66	0.99	1.56		1.56	0.000129	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	51.828*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	49.834*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03

PIC Breach Spillway 1	47.841*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.86	5.72	0.1	0.57	1.03
PIC Breach Spillway 1	45.848*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	43.854*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	41.861*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.72	0.1	0.57	1.03
PIC Breach Spillway 1	39.868*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.86	5.72	0.1	0.57	1.03
PIC Breach Spillway 1	37.874*	100 year	0.66	0.99	1.56		1.56	0.000129	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	35.881*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	33.887*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	31.894*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.72	0.1	0.57	1.03
PIC Breach Spillway 1	29.901*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.86	5.72	0.1	0.57	1.03
PIC Breach Spillway 1	27.907*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	25.914*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.87	5.73	0.1	0.57	1.03
PIC Breach Spillway 1	23.921*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.86	5.72	0.1	0.57	1.03
PIC Breach Spillway 1	21.927*	100 year	0.66	0.99	1.56		1.56	0.00013	0.23	2.86	5.72	0.1	0.57	1.03
PIC Breach Spillway 1	19.934*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.87	5.73	0.1	0.56	1.04
PIC Breach Spillway 1	17.940*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.87	5.72	0.1	0.56	1.04
PIC Breach Spillway 1	15.947*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.86	5.72	0.1	0.56	1.04
PIC Breach Spillway 1	13.954*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.86	5.72	0.1	0.56	1.04
PIC Breach Spillway 1	11.960*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.87	5.73	0.1	0.56	1.04
PIC Breach Spillway 1	9.967*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.87	5.72	0.1	0.56	1.04
PIC Breach Spillway 1	7.974*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.86	5.72	0.1	0.56	1.04
PIC Breach Spillway 1	5.980*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.86	5.72	0.1	0.56	1.04
PIC Breach Spillway 1	3.987*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.87	5.73	0.1	0.56	1.04
PIC Breach Spillway 1	1.993*	100 year	0.66	0.99	1.55		1.56	0.00013	0.23	2.87	5.72	0.1	0.56	1.04
PIC Breach Spillway 1	0	100 year	0.66	0.99	1.55	1.12	1.55	0.00013	0.23	2.86	5.72	0.1	0.56	1.04

PIC Breach Spillway #11: HEC-RAS Hydraulic Analysis Results (IDF event)

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # C	Flow Depth (m)	Freeboard (m)
PIC Breach Spillway 1	103.1	IDF	2.16	2.25	3.49	3.49	4.11	0.016618	3.48	0.62	0.5	1	1.24	-0.24
PIC Breach Spillway 1	102.15	IDF	2.16	2.22	3.39	3.46	4.08	0.019002	3.7	0.58	0.5	1.09	1.17	-0.17
PIC Breach Spillway 1	102.1	IDF	2.16	2.22	2.63	3	4.01	0.033037	5.2	0.41	1	2.58	0.41	0.59
PIC Breach Spillway 1	100.16*	IDF	2.16	2.16	2.58	2.94	3.95	0.032706	5.18	0.42	1	2.56	0.42	0.58
PIC Breach Spillway 1	98.23*	IDF	2.16	2.11	2.53	2.88	3.88	0.032372	5.16	0.42	1	2.55	0.42	0.58
PIC Breach Spillway 1	96.29*	IDF	2.16	2.05	2.47	2.83	3.82	0.032062	5.14	0.42	1	2.54	0.42	0.58
PIC Breach Spillway 1	94.36*	IDF	2.16	2	2.42	2.77	3.75	0.031762	5.12	0.42	1	2.52	0.42	0.58
PIC Breach Spillway 1	92.42*	IDF	2.16	1.94	2.36	2.72	3.69	0.031498	5.11	0.42	1	2.51	0.42	0.58
PIC Breach Spillway 1	90.49*	IDF	2.16	1.88	2.31	2.66	3.63	0.031256	5.09	0.42	1	2.5	0.43	0.57
PIC Breach Spillway 1	88.55*	IDF	2.16	1.83	2.25	2.6	3.57	0.031033	5.08	0.42	1	2.49	0.42	0.58
PIC Breach Spillway 1	86.62*	IDF	2.16	1.77	2.2	2.55	3.51	0.030816	5.07	0.43	1	2.48	0.43	0.57
PIC Breach Spillway 1	84.68*	IDF	2.16	1.72	2.14	2.49	3.45	0.030632	5.06	0.43	1	2.47	0.42	0.58
PIC Breach Spillway 1	82.74*	IDF	2.16	1.66	2.09	2.44	3.38	0.030465	5.05	0.43	1	2.47	0.43	0.57
PIC Breach Spillway 1	80.81*	IDF	2.16	1.6	2.03	2.38	3.33	0.030465	5.05	0.43	1	2.47	0.43	0.57
PIC Breach Spillway 1	78.87*	IDF	2.16	1.55	1.98	2.32	3.27	0.030298	5.04	0.43	1	2.46	0.43	0.57
PIC Breach Spillway 1	76.94*	IDF	2.16	1.49	1.92	2.27	3.21	0.030298	5.04	0.43	1	2.46	0.43	0.57
PIC Breach Spillway 1	75.00*	IDF	2.16	1.44	1.86	2.21	3.16	0.030298	5.04	0.43	1	2.46	0.42	0.58
PIC Breach Spillway 1	73.066	IDF	2.16	1.38	1.81	2.16	3.1	0.030298	5.04	0.43	1	2.46	0.43	0.57
PIC Breach Spillway 1	71.3013	IDF	2.16	1.33	1.46	1.7	2.99	0.085219	5.48	0.39	3	4.83	0.13	0.87
PIC Breach Spillway 1	71.2013	IDF	2.16	1.32	2.12	1.67	2.14	0.001235	0.62	3.48	5.68	0.25	0.8	0.2
PIC Breach Spillway 1	69.5162	IDF	2.16	1.28	2.12		2.14	0.001033	0.58	3.71	5.81	0.23	0.84	0.16
PIC Breach Spillway 1	68.250*	IDF	2.16	1.25	2.12		2.14	0.00092	0.56	3.87	5.9	0.22	0.87	0.13
PIC Breach Spillway 1	66.9828	IDF	2.16	1.23	2.12		2.14	0.000826	0.54	4.02	5.98	0.21	0.89	0.11
PIC Breach Spillway 1	65.212*	IDF	2.16	1.17	2.12		2.13	0.000655	0.49	4.37	6.17	0.19	0.95	0.05
PIC Breach Spillway 1	63.442*	IDF	2.16	1.11	2.12		2.13	0.000526	0.46	4.73	6.36	0.17	1.01	-0.01
PIC Breach Spillway 1	61.672*	IDF	2.16	1.06	2.12		2.13	0.000428	0.42	5.09	6.55	0.15	1.06	-0.06
PIC Breach Spillway 1	59.9013	IDF	2.16	1	2.12		2.13	0.000351	0.39	5.47	6.74	0.14	1.12	-0.12
PIC Breach Spillway 1	59.8013	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	-0.13
PIC Breach Spillway 1	57.808*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	55.815*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	53.821*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	51.828*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	49.834*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47

PIC Breach Spillway 1	47.841*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	45.848*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	43.854*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	41.861*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	39.868*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	37.874*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	35.881*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	33.887*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	31.894*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	29.901*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	27.907*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	25.914*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	23.921*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	21.927*	IDF	2.16	0.99	2.12		2.13	0.00013	0.33	6.46	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	19.934*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	17.940*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	15.947*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	13.954*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.46	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	11.960*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	9.967*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	7.974*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	5.980*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.46	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	3.987*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	1.993*	IDF	2.16	0.99	2.12		2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47
PIC Breach Spillway 1	0	IDF	2.16	0.99	2.12	1.27	2.12	0.00013	0.33	6.47	7.04	0.11	1.13	0.47

PIC Breach Spillway #2: HEC-RAS Hydraulic Analysis Results (1 in 1-year event)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Flow Depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
PIC Breach Spillway 2	95.366	1yr	0.05	2.1	2.2	2.2	2.25	0.005572	0.99	0.05	0.5	1	0.1	1.4
PIC Breach Spillway 2	94.416	1yr	0.05	2.07	2.14	2.17	2.24	0.014456	1.36	0.04	0.5	1.61	0.07	1.43
PIC Breach Spillway 2	94.366	1yr	0.05	2.07	2.1	2.13	2.23	0.048479	1.61	0.03	1	2.91	0.03	1.47
PIC Breach Spillway 2	92.487*	1yr	0.05	2.01	2.05	2.08	2.15	0.032365	1.42	0.04	1	2.42	0.04	1.46
PIC Breach Spillway 2	90.607*	1yr	0.05	1.95	1.99	2.02	2.09	0.030947	1.4	0.04	1	2.37	0.04	1.46
PIC Breach Spillway 2	88.728*	1yr	0.05	1.9	1.93	1.96	2.03	0.030947	1.4	0.04	1	2.37	0.03	1.47
PIC Breach Spillway 2	86.848*	1yr	0.05	1.84	1.87	1.9	1.97	0.030765	1.4	0.04	1	2.36	0.03	1.47
PIC Breach Spillway 2	84.969*	1yr	0.05	1.78	1.82	1.85	1.92	0.030765	1.4	0.04	1	2.36	0.04	1.46
PIC Breach Spillway 2	83.089*	1yr	0.05	1.72	1.76	1.79	1.86	0.030481	1.39	0.04	1	2.35	0.04	1.46
PIC Breach Spillway 2	81.210*	1yr	0.05	1.67	1.7	1.73	1.8	0.029482	1.38	0.04	1	2.32	0.03	1.47
PIC Breach Spillway 2	79.330*	1yr	0.05	1.61	1.64	1.67	1.74	0.030213	1.39	0.04	1	2.34	0.03	1.47
PIC Breach Spillway 2	77.451*	1yr	0.05	1.55	1.59	1.61	1.69	0.029999	1.39	0.04	1	2.33	0.04	1.46
PIC Breach Spillway 2	75.571*	1yr	0.05	1.49	1.53	1.56	1.63	0.030472	1.39	0.04	1	2.35	0.04	1.46
PIC Breach Spillway 2	73.692*	1yr	0.05	1.43	1.47	1.5	1.57	0.031696	1.41	0.04	1	2.39	0.04	1.46
PIC Breach Spillway 2	71.812*	1yr	0.05	1.38	1.41	1.44	1.51	0.031696	1.41	0.04	1	2.39	0.03	1.47
PIC Breach Spillway 2	69.933	1yr	0.05	1.32	1.36	1.38	1.46	0.031696	1.41	0.04	1	2.39	0.04	1.46
PIC Breach Spillway 2	68.201	1yr	0.05	1.27	1.28	1.3	1.36	0.084797	1.25	0.04	3	3.46	0.01	1.49
PIC Breach Spillway 2	68.101	1yr	0.05	1.26	1.27	1.29	1.34	0.514621	1.12	0.04	3.07	2.97	0.01	1.49
PIC Breach Spillway 2	66.297*	1yr	0.05	1.21	1.24	1.24	1.25	0.034863	0.49	0.1	3.16	0.88	0.03	0.97
PIC Breach Spillway 2	64.493	1yr	0.05	1.15	1.19	1.18	1.2	0.022703	0.43	0.12	3.19	0.73	0.04	0.96
PIC Breach Spillway 2	62.826*	1yr	0.05	1.1	1.13	1.13	1.15	0.040869	0.52	0.1	3.16	0.95	0.03	0.97
PIC Breach Spillway 2	61.160*	1yr	0.05	1.05	1.09		1.1	0.02202	0.43	0.12	3.19	0.72	0.04	0.96
PIC Breach Spillway 2	59.493	1yr	0.05	1	1.03	1.03	1.05	0.046242	0.54	0.09	3.15	1	0.03	0.97
PIC Breach Spillway 2	57.762*	1yr	0.05	0.84	0.86	0.87	0.9	0.20688	0.83	0.06	3.32	1.96	0.02	1.58
PIC Breach Spillway 2	56.031*	1yr	0.05	0.68	0.7	0.71	0.73	0.110026	0.67	0.08	3.55	1.46	0.02	1.58
PIC Breach Spillway 2	54.301*	1yr	0.05	0.52	0.55	0.55	0.57	0.0799	0.59	0.08	3.78	1.26	0.03	1.57
PIC Breach Spillway 2	52.570*	1yr	0.05	0.37	0.39	0.39	0.41	0.104981	0.63	0.08	4	1.42	0.02	1.58
PIC Breach Spillway 2	50.839*	1yr	0.05	0.21	0.23	0.23	0.25	0.07996	0.56	0.09	4.23	1.24	0.02	1.58
PIC Breach Spillway 2	49.108	1yr	0.05	0.05	0.11	0.07	0.11	0.00143	0.19	0.27	4.55	0.25	0.06	1.54

PIC Breach Spillway 2	47.144*	1yr	0.05	0.05	0.11		0.11	0.001408	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	45.179*	1yr	0.05	0.04	0.1		0.11	0.001382	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	43.215*	1yr	0.05	0.04	0.1		0.1	0.001465	0.19	0.26	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	41.251*	1yr	0.05	0.04	0.1		0.1	0.001448	0.19	0.26	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	39.286*	1yr	0.05	0.04	0.1		0.1	0.001428	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	37.322*	1yr	0.05	0.03	0.09		0.09	0.001405	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	35.358*	1yr	0.05	0.03	0.09		0.09	0.001379	0.19	0.27	4.55	0.24	0.06	1.54
PIC Breach Spillway 2	33.393*	1yr	0.05	0.03	0.09		0.09	0.00146	0.19	0.26	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	31.429*	1yr	0.05	0.02	0.08		0.09	0.001443	0.19	0.26	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	29.465*	1yr	0.05	0.02	0.08		0.08	0.001422	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	27.500*	1yr	0.05	0.02	0.08		0.08	0.001399	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	25.536*	1yr	0.05	0.02	0.08		0.08	0.001371	0.19	0.27	4.55	0.24	0.06	1.54
PIC Breach Spillway 2	23.572*	1yr	0.05	0.01	0.07		0.07	0.001451	0.19	0.26	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	21.608*	1yr	0.05	0.01	0.07		0.07	0.001432	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	19.643*	1yr	0.05	0.01	0.07		0.07	0.00141	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	17.679*	1yr	0.05	0	0.06		0.07	0.001384	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	15.715*	1yr	0.05	0	0.06		0.06	0.001382	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	13.750*	1yr	0.05	0	0.06		0.06	0.001435	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	11.786*	1yr	0.05	0	0.06		0.06	0.001413	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	9.822*	1yr	0.05	-0.01	0.05		0.06	0.001388	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	7.857*	1yr	0.05	-0.01	0.05		0.05	0.001359	0.19	0.27	4.55	0.24	0.06	1.54
PIC Breach Spillway 2	5.893*	1yr	0.05	-0.01	0.05		0.05	0.001353	0.19	0.27	4.55	0.24	0.06	1.54
PIC Breach Spillway 2	3.929*	1yr	0.05	-0.01	0.05		0.05	0.001399	0.19	0.27	4.55	0.25	0.06	1.54
PIC Breach Spillway 2	1.964*	1yr	0.05	-0.02	0.04		0.04	0.001372	0.19	0.27	4.55	0.24	0.06	1.54
PIC Breach Spillway 2	0	1yr	0.05	-0.02	0.04	0	0.04	0.001341	0.18	0.27	4.55	0.24	0.06	1.54

PIC Breach Spillway #2: HEC-RAS Hydraulic Analysis Results (1 in 100-year event)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # C	Flow Depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
PIC Breach Spillway 2	95.366	100yr	0.46	2.1	2.54	2.54	2.76	0.008526	2.09	0.22	0.5	1.01	0.44	1.06
PIC Breach Spillway 2	94.416	100yr	0.46	2.07	2.45	2.51	2.74	0.011884	2.38	0.19	0.5	1.23	0.38	1.12
PIC Breach Spillway 2	94.366	100yr	0.46	2.07	2.22	2.35	2.72	0.030827	3.15	0.15	1	2.64	0.15	1.35
PIC Breach Spillway 2	92.487*	100yr	0.46	2.01	2.16	2.29	2.66	0.030418	3.13	0.15	1	2.62	0.15	1.35
PIC Breach Spillway 2	90.607*	100yr	0.46	1.95	2.1	2.23	2.6	0.030418	3.13	0.15	1	2.62	0.15	1.35
PIC Breach Spillway 2	88.728*	100yr	0.46	1.9	2.04	2.17	2.54	0.030553	3.14	0.15	1	2.63	0.14	1.36
PIC Breach Spillway 2	86.848*	100yr	0.46	1.84	1.98	2.12	2.49	0.030709	3.14	0.15	1	2.63	0.14	1.36
PIC Breach Spillway 2	84.969*	100yr	0.46	1.78	1.93	2.06	2.43	0.030709	3.14	0.15	1	2.63	0.15	1.35
PIC Breach Spillway 2	83.089*	100yr	0.46	1.72	1.87	2	2.37	0.030576	3.14	0.15	1	2.63	0.15	1.35
PIC Breach Spillway 2	81.210*	100yr	0.46	1.67	1.81	1.94	2.31	0.030366	3.13	0.15	1	2.62	0.14	1.36
PIC Breach Spillway 2	79.330*	100yr	0.46	1.61	1.75	1.88	2.26	0.030511	3.14	0.15	1	2.62	0.14	1.36
PIC Breach Spillway 2	77.451*	100yr	0.46	1.55	1.7	1.83	2.2	0.030511	3.14	0.15	1	2.62	0.15	1.35
PIC Breach Spillway 2	75.571*	100yr	0.46	1.49	1.64	1.77	2.14	0.030676	3.14	0.15	1	2.63	0.15	1.35
PIC Breach Spillway 2	73.692*	100yr	0.46	1.43	1.58	1.71	2.08	0.030406	3.13	0.15	1	2.62	0.15	1.35
PIC Breach Spillway 2	71.812*	100yr	0.46	1.38	1.52	1.65	2.02	0.030406	3.13	0.15	1	2.62	0.14	1.36
PIC Breach Spillway 2	69.933	100yr	0.46	1.32	1.47	1.6	1.97	0.030546	3.14	0.15	1	2.63	0.15	1.35
PIC Breach Spillway 2	68.201	100yr	0.46	1.27	1.32	1.4	1.87	0.11529	3.3	0.14	3	4.9	0.05	1.45
PIC Breach Spillway 2	68.101	100yr	0.46	1.26	1.31	1.39	1.84	0.983015	3.23	0.14	3.23	4.93	0.05	1.45
PIC Breach Spillway 2	66.297*	100yr	0.46	1.21	1.31	1.34	1.41	0.065834	1.39	0.33	3.51	1.44	0.1	0.9
PIC Breach Spillway 2	64.493	100yr	0.46	1.15	1.26	1.28	1.35	0.056672	1.32	0.35	3.53	1.35	0.11	0.89
PIC Breach Spillway 2	62.826*	100yr	0.46	1.1	1.21	1.23	1.29	0.047492	1.25	0.37	3.56	1.24	0.11	0.89
PIC Breach Spillway 2	61.160*	100yr	0.46	1.05	1.17	1.18	1.24	0.042643	1.21	0.38	3.58	1.18	0.12	0.88
PIC Breach Spillway 2	59.493	100yr	0.46	1	1.12	1.13	1.19	0.039301	1.18	0.39	3.59	1.14	0.12	0.88
PIC Breach Spillway 2	57.762*	100yr	0.46	0.84	0.92	0.96	1.07	0.130458	1.69	0.27	3.58	1.96	0.08	1.52
PIC Breach Spillway 2	56.031*	100yr	0.46	0.68	0.77	0.8	0.88	0.083354	1.44	0.32	3.8	1.59	0.09	1.51
PIC Breach Spillway 2	54.301*	100yr	0.46	0.52	0.6	0.64	0.72	0.098554	1.49	0.31	3.97	1.71	0.08	1.52
PIC Breach Spillway 2	52.570*	100yr	0.46	0.37	0.45	0.48	0.55	0.094313	1.44	0.32	4.17	1.67	0.08	1.52
PIC Breach Spillway 2	50.839*	100yr	0.46	0.21	0.28	0.31	0.39	0.098169	1.43	0.32	4.37	1.69	0.07	1.53
PIC Breach Spillway 2	49.108	100yr	0.46	0.05	0.28	0.15	0.28	0.001392	0.43	1.05	4.93	0.3	0.23	1.37

PIC Breach Spillway 2	47.144*	100yr	0.46	0.05	0.27		0.28	0.001386	0.43	1.05	4.93	0.3	0.22	1.38
PIC Breach Spillway 2	45.179*	100yr	0.46	0.04	0.27		0.28	0.001379	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	43.215*	100yr	0.46	0.04	0.27		0.28	0.001399	0.43	1.05	4.93	0.3	0.23	1.37
PIC Breach Spillway 2	41.251*	100yr	0.46	0.04	0.26		0.27	0.001393	0.43	1.05	4.93	0.3	0.22	1.38
PIC Breach Spillway 2	39.286*	100yr	0.46	0.04	0.26		0.27	0.001386	0.43	1.05	4.93	0.3	0.22	1.38
PIC Breach Spillway 2	37.322*	100yr	0.46	0.03	0.26		0.27	0.001379	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	35.358*	100yr	0.46	0.03	0.26		0.27	0.001372	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	33.393*	100yr	0.46	0.03	0.25		0.26	0.001392	0.43	1.05	4.93	0.3	0.22	1.38
PIC Breach Spillway 2	31.429*	100yr	0.46	0.02	0.25		0.26	0.001385	0.43	1.05	4.93	0.3	0.23	1.37
PIC Breach Spillway 2	29.465*	100yr	0.46	0.02	0.25		0.26	0.001378	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	27.500*	100yr	0.46	0.02	0.25		0.25	0.001371	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	25.536*	100yr	0.46	0.02	0.24		0.25	0.001363	0.43	1.06	4.94	0.3	0.22	1.38
PIC Breach Spillway 2	23.572*	100yr	0.46	0.01	0.24		0.25	0.001383	0.43	1.05	4.93	0.3	0.23	1.37
PIC Breach Spillway 2	21.608*	100yr	0.46	0.01	0.24		0.25	0.001376	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	19.643*	100yr	0.46	0.01	0.23		0.24	0.001369	0.43	1.06	4.94	0.3	0.22	1.38
PIC Breach Spillway 2	17.679*	100yr	0.46	0	0.23		0.24	0.001361	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	15.715*	100yr	0.46	0	0.23		0.24	0.00136	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	13.750*	100yr	0.46	0	0.23		0.24	0.001372	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	11.786*	100yr	0.46	0	0.22		0.23	0.001364	0.43	1.06	4.94	0.3	0.22	1.38
PIC Breach Spillway 2	9.822*	100yr	0.46	-0.01	0.22		0.23	0.001357	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	7.857*	100yr	0.46	-0.01	0.22		0.23	0.001349	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	5.893*	100yr	0.46	-0.01	0.22		0.23	0.001347	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	3.929*	100yr	0.46	-0.01	0.21		0.22	0.001358	0.43	1.06	4.94	0.3	0.22	1.38
PIC Breach Spillway 2	1.964*	100yr	0.46	-0.02	0.21		0.22	0.00135	0.43	1.06	4.94	0.3	0.23	1.37
PIC Breach Spillway 2	0	100yr	0.46	-0.02	0.21	0.08	0.22	0.001342	0.43	1.06	4.94	0.3	0.23	1.37

PIC Breach Spillway #2: HEC-RAS Hydraulic Analysis Results (IDF event)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # C	Flow Depth	Freeboard
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		(m)	(m)
PIC Breach Spillway 2	95.366	IDF	1.61	2.1	3.12	3.12	3.63	0.014476	3.17	0.51	0.5	1	1.02	0.48
PIC Breach Spillway 2	94.416	IDF	1.61	2.07	3.01	3.09	3.61	0.017169	3.42	0.47	0.5	1.12	0.94	0.56
PIC Breach Spillway 2	94.366	IDF	1.61	2.07	2.41	2.71	3.55	0.031798	4.73	0.34	1	2.59	0.34	1.16
PIC Breach Spillway 2	92.487*	IDF	1.61	2.01	2.35	2.65	3.49	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	90.607*	IDF	1.61	1.95	2.3	2.59	3.43	0.031683	4.72	0.34	1	2.58	0.35	1.15
PIC Breach Spillway 2	88.728*	IDF	1.61	1.9	2.24	2.54	3.38	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	86.848*	IDF	1.61	1.84	2.18	2.48	3.32	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	84.969*	IDF	1.61	1.78	2.12	2.42	3.26	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	83.089*	IDF	1.61	1.72	2.06	2.36	3.2	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	81.210*	IDF	1.61	1.67	2.01	2.31	3.14	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	79.330*	IDF	1.61	1.61	1.95	2.25	3.09	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	77.451*	IDF	1.61	1.55	1.89	2.19	3.03	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	75.571*	IDF	1.61	1.49	1.83	2.13	2.97	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	73.692*	IDF	1.61	1.43	1.78	2.07	2.91	0.031683	4.72	0.34	1	2.58	0.35	1.15
PIC Breach Spillway 2	71.812*	IDF	1.61	1.38	1.72	2.02	2.86	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	69.933	IDF	1.61	1.32	1.66	1.96	2.8	0.031683	4.72	0.34	1	2.58	0.34	1.16
PIC Breach Spillway 2	68.201	IDF	1.61	1.27	1.38	1.58	2.69	0.095554	5.08	0.32	3	4.99	0.11	1.39
PIC Breach Spillway 2	68.101	IDF	1.61	1.26	1.36	1.54	2.67	0.918559	5.07	0.32	3.49	5.36	0.1	1.4
PIC Breach Spillway 2	66.297*	IDF	1.61	1.21	1.37	1.49	1.81	0.169859	2.94	0.55	3.8	2.48	0.16	0.84
PIC Breach Spillway 2	64.493	IDF	1.61	1.15	1.38	1.44	1.58	0.052228	2	0.81	4.13	1.44	0.23	0.77
PIC Breach Spillway 2	62.826*	IDF	1.61	1.1	1.38	1.39	1.51	0.024983	1.56	1.03	4.4	1.03	0.28	0.72
PIC Breach Spillway 2	61.160*	IDF	1.61	1.05	1.31	1.33	1.46	0.030438	1.67	0.97	4.32	1.12	0.26	0.74
PIC Breach Spillway 2	59.493	IDF	1.61	1	1.27	1.28	1.4	0.028483	1.63	0.99	4.35	1.09	0.27	0.73
PIC Breach Spillway 2	57.762*	IDF	1.61	0.84	1.03	1.12	1.31	0.086768	2.34	0.69	4.06	1.82	0.19	1.41
PIC Breach Spillway 2	56.031*	IDF	1.61	0.68	0.86	0.95	1.15	0.095498	2.39	0.67	4.14	1.89	0.18	1.42
PIC Breach Spillway 2	54.301*	IDF	1.61	0.52	0.69	0.78	0.98	0.09755	2.37	0.68	4.28	1.9	0.17	1.43
PIC Breach Spillway 2	52.570*	IDF	1.61	0.37	0.53	0.62	0.81	0.097281	2.34	0.69	4.43	1.89	0.16	1.44
PIC Breach Spillway 2	50.839*	IDF	1.61	0.21	0.37	0.45	0.64	0.097826	2.31	0.7	4.59	1.89	0.16	1.44
PIC Breach Spillway 2	49.108	IDF	1.61	0.05	0.53	0.28	0.55	0.00137	0.68	2.38	5.53	0.33	0.48	1.12

PIC Breach Spillway 2	47.144*	IDF	1.61	0.05	0.53		0.55	0.001367	0.68	2.38	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	45.179*	IDF	1.61	0.04	0.52		0.55	0.001363	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	43.215*	IDF	1.61	0.04	0.52		0.54	0.001373	0.68	2.38	5.52	0.33	0.48	1.12
PIC Breach Spillway 2	41.251*	IDF	1.61	0.04	0.52		0.54	0.001369	0.68	2.38	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	39.286*	IDF	1.61	0.04	0.52		0.54	0.001366	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	37.322*	IDF	1.61	0.03	0.51		0.54	0.001362	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	35.358*	IDF	1.61	0.03	0.51		0.53	0.001358	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	33.393*	IDF	1.61	0.03	0.51		0.53	0.001368	0.68	2.38	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	31.429*	IDF	1.61	0.02	0.51		0.53	0.001364	0.67	2.39	5.53	0.33	0.49	1.11
PIC Breach Spillway 2	29.465*	IDF	1.61	0.02	0.5		0.53	0.00136	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	27.500*	IDF	1.61	0.02	0.5		0.52	0.001357	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	25.536*	IDF	1.61	0.02	0.5		0.52	0.001353	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	23.572*	IDF	1.61	0.01	0.49		0.52	0.001362	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	21.608*	IDF	1.61	0.01	0.49		0.52	0.001358	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	19.643*	IDF	1.61	0.01	0.49		0.51	0.001355	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	17.679*	IDF	1.61	0	0.49		0.51	0.001351	0.67	2.39	5.53	0.33	0.49	1.11
PIC Breach Spillway 2	15.715*	IDF	1.61	0	0.48		0.51	0.00135	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	13.750*	IDF	1.61	0	0.48		0.5	0.001356	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	11.786*	IDF	1.61	0	0.48		0.5	0.001352	0.67	2.39	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	9.822*	IDF	1.61	-0.01	0.48		0.5	0.001348	0.67	2.4	5.53	0.33	0.49	1.11
PIC Breach Spillway 2	7.857*	IDF	1.61	-0.01	0.47		0.5	0.001344	0.67	2.4	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	5.893*	IDF	1.61	-0.01	0.47		0.49	0.001343	0.67	2.4	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	3.929*	IDF	1.61	-0.01	0.47		0.49	0.001349	0.67	2.4	5.53	0.33	0.48	1.12
PIC Breach Spillway 2	1.964*	IDF	1.61	-0.02	0.47		0.49	0.001345	0.67	2.4	5.53	0.33	0.49	1.11
PIC Breach Spillway 2	0	IDF	1.61	-0.02	0.46	0.21	0.49	0.001341	0.67	2.4	5.53	0.33	0.48	1.12

North East Breach Spillway: HEC-RAS Hydraulic Analysis Results (IDF event)

North East Breach Spillway	190.159	IDF	5.2	10	10.29	10.29	10.43	0.029589	1.64	3.16	11.47	1	0.29	0.71
North East Breach Spillway	188.18*	IDF	5.2	9.83	10.04	10.12	10.32	0.092799	2.35	2.21	11.05	1.68	0.21	0.79
North East Breach Spillway	186.21*	IDF	5.2	9.66	9.87	9.95	10.14	0.08491	2.29	2.27	11.08	1.62	0.21	0.79
North East Breach Spillway	184.23*	IDF	5.2	9.49	9.7	9.78	9.97	0.086973	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	182.26*	IDF	5.2	9.31	9.53	9.61	9.8	0.08714	2.31	2.25	11.07	1.63	0.22	0.78
North East Breach Spillway	180.28*	IDF	5.2	9.14	9.36	9.44	9.63	0.08714	2.31	2.25	11.07	1.63	0.22	0.78
North East Breach Spillway	178.30*	IDF	5.2	8.97	9.18	9.27	9.46	0.087109	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	176.33*	IDF	5.2	8.8	9.01	9.09	9.29	0.087109	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	174.35*	IDF	5.2	8.63	8.84	8.92	9.11	0.087109	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	172.38*	IDF	5.2	8.46	8.67	8.75	8.94	0.087109	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	170.40*	IDF	5.2	8.29	8.5	8.58	8.77	0.087109	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	168.42*	IDF	5.2	8.11	8.33	8.41	8.6	0.087109	2.31	2.25	11.07	1.63	0.22	0.78
North East Breach Spillway	166.45*	IDF	5.2	7.94	8.16	8.24	8.43	0.087109	2.31	2.25	11.07	1.63	0.22	0.78
North East Breach Spillway	164.47*	IDF	5.2	7.77	7.98	8.07	8.26	0.086995	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	162.50*	IDF	5.2	7.6	7.81	7.89	8.09	0.086995	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	160.52*	IDF	5.2	7.43	7.64	7.72	7.91	0.086995	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	158.54*	IDF	5.2	7.26	7.47	7.55	7.74	0.086989	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	156.57*	IDF	5.2	7.09	7.3	7.38	7.57	0.086989	2.31	2.25	11.07	1.63	0.21	0.79
North East Breach Spillway	154.59*	IDF	5.2	6.91	7.13	7.21	7.4	0.086969	2.31	2.25	11.07	1.63	0.22	0.78
North East Breach Spillway	152.62*	IDF	5.2	6.74	6.96	7.04	7.23	0.086969	2.31	2.25	11.07	1.63	0.22	0.78
North East Breach Spillway	150.64*	IDF	5.2	6.57	6.79	6.87	7.06	0.08648	2.3	2.26	11.07	1.63	0.22	0.78
North East Breach Spillway	148.66*	IDF	5.2	6.4	6.61	6.69	6.88	0.08648	2.3	2.26	11.07	1.63	0.21	0.79
North East Breach Spillway	146.69*	IDF	5.2	6.23	6.44	6.52	6.71	0.086336	2.3	2.26	11.07	1.63	0.21	0.79
North East Breach Spillway	144.71*	IDF	5.2	6.06	6.27	6.35	6.54	0.087239	2.31	2.25	11.07	1.64	0.21	0.79
North East Breach Spillway	142.74*	IDF	5.2	5.89	6.1	6.18	6.37	0.087239	2.31	2.25	11.07	1.64	0.21	0.79
North East Breach Spillway	140.76*	IDF	5.2	5.71	5.93	6.01	6.2	0.087239	2.31	2.25	11.07	1.64	0.22	0.78
North East Breach Spillway	138.78*	IDF	5.2	5.54	5.76	5.84	6.03	0.087239	2.31	2.25	11.07	1.64	0.22	0.78
North East Breach Spillway	136.81*	IDF	5.2	5.37	5.58	5.67	5.86	0.087239	2.31	2.25	11.07	1.64	0.21	0.79
North East Breach Spillway	134.83*	IDF	5.2	5.2	5.41	5.49	5.69	0.087239	2.31	2.25	11.07	1.64	0.21	0.79
North East Breach Spillway	132.86*	IDF	5.2	5.03	5.24	5.32	5.51	0.087239	2.31	2.25	11.07	1.64	0.21	0.79
North East Breach Spillway	130.88*	IDF	5.2	4.86	5.07	5.15	5.34	0.087238	2.31	2.25	11.07	1.64	0.21	0.79

North East Breach Spillway	128.90*	IDF	5.2	4.69	4.9	4.98	5.17	0.087238	2.31	2.25	11.07	1.64	0.21	0.79
North East Breach Spillway	126.93*	IDF	5.2	4.51	4.73	4.81	5	0.087238	2.31	2.25	11.07	1.64	0.22	0.78
North East Breach Spillway	124.95*	IDF	5.2	4.34	4.56	4.64	4.83	0.087238	2.31	2.25	11.07	1.64	0.22	0.78
North East Breach Spillway	122.98*	IDF	5.2	4.17	4.38	4.47	4.66	0.087238	2.31	2.25	11.07	1.64	0.21	0.79
North East Breach Spillway	121	IDF	5.2	4	4.21	4.29	4.49	0.087238	2.31	2.25	11.07	1.64	0.21	0.79
North East Breach Spillway	119.00*	IDF	5.2	3.94	4.21	4.23	4.37	0.037151	1.77	2.94	11.38	1.11	0.27	0.73
North East Breach Spillway	117.00*	IDF	5.2	3.87	4.16	4.17	4.31	0.032098	1.69	3.08	11.44	1.04	0.29	0.71
North East Breach Spillway	115.00*	IDF	5.2	3.81	4.1	4.11	4.24	0.032097	1.69	3.08	11.44	1.04	0.29	0.71
North East Breach Spillway	113	IDF	5.2	3.75	4.04	4.04	4.18	0.032098	1.69	3.08	11.44	1.04	0.29	0.71
North East Breach Spillway	111.20*	IDF	5.2	3.67	3.92	3.96	4.11	0.048614	1.92	2.7	11.27	1.25	0.25	0.75
North East Breach Spillway	109.40*	IDF	5.2	3.58	3.85	3.88	4.02	0.042915	1.85	2.81	11.32	1.18	0.27	0.73
North East Breach Spillway	107.6	IDF	5.2	3.5	3.91	3.79	3.98	0.009325	1.14	4.57	12.07	0.59	0.41	0.59
North East Breach Spillway	106.20*	IDF	5.2	3.47	3.91		3.97	0.008101	1.09	4.78	12.16	0.55	0.44	0.56
North East Breach Spillway	104.80*	IDF	5.2	3.45	3.9		3.95	0.007062	1.04	4.99	12.25	0.52	0.45	0.55
North East Breach Spillway	103.4	IDF	5.2	3.42	3.89		3.94	0.006066	0.99	5.25	12.35	0.49	0.47	0.53
North East Breach Spillway	101.4	IDF	5.2	3.42	3.9		3.93	0.001778	0.76	6.87	17.25	0.38	0.48	0.52
North East Breach Spillway	99.53*	IDF	5.2	3.41	3.89		3.93	0.002242	0.82	6.37	17.05	0.43	0.48	0.52
North East Breach Spillway	97.66*	IDF	5.2	3.4	3.88		3.92	0.002825	0.87	5.94	17.05	0.47	0.48	0.52
North East Breach Spillway	95.79*	IDF	5.2	3.39	3.87		3.91	0.003571	0.94	5.55	17.12	0.53	0.48	0.52
North East Breach Spillway	93.91*	IDF	5.2	3.37	3.85		3.91	0.004327	1	5.18	16.65	0.57	0.48	0.52
North East Breach Spillway	92.04*	IDF	5.2	3.36	3.84		3.9	0.00396	1.02	5.08	14.88	0.56	0.48	0.52
North East Breach Spillway	90.17*	IDF	5.2	3.35	3.84		3.89	0.003599	1	5.21	14.69	0.54	0.49	0.51
North East Breach Spillway	88.30*	IDF	5.2	3.34	3.83		3.88	0.003176	0.96	5.41	14.72	0.51	0.49	0.51
North East Breach Spillway	86.43*	IDF	5.2	3.32	3.83		3.87	0.002696	0.91	5.7	14.82	0.47	0.51	0.49
North East Breach Spillway	84.56*	IDF	5.2	3.31	3.83		3.87	0.002261	0.86	6.04	15	0.43	0.52	0.48
North East Breach Spillway	82.69*	IDF	5.2	3.3	3.83		3.86	0.00187	0.81	6.44	15.24	0.4	0.53	0.47
North East Breach Spillway	80.81*	IDF	5.2	3.29	3.83		3.86	0.00155	0.76	6.86	15.53	0.36	0.54	0.46
North East Breach Spillway	78.94*	IDF	5.2	3.28	3.83		3.85	0.001281	0.71	7.33	15.84	0.33	0.55	0.45
North East Breach Spillway	77.07*	IDF	5.2	3.26	3.83		3.85	0.001051	0.66	7.84	16.17	0.3	0.57	0.43
North East Breach Spillway	75.2	IDF	5.2	3.25	3.83		3.85	0.000872	0.62	8.38	16.56	0.28	0.58	0.42
North East Breach Spillway	73.321*	IDF	5.2	3.25	3.83		3.85	0.000905	0.63	8.29	16.61	0.28	0.58	0.42

North East Breach Spillway	71.443*	IDF	5.2	3.25	3.83		3.85	0.000937	0.63	8.22	16.66	0.29	0.58	0.42
North East Breach Spillway	69.564*	IDF	5.2	3.25	3.82		3.84	0.000968	0.64	8.15	16.72	0.29	0.57	0.43
North East Breach Spillway	67.686*	IDF	5.2	3.25	3.82		3.84	0.000997	0.64	8.08	16.78	0.3	0.57	0.43
North East Breach Spillway	65.807*	IDF	5.2	3.25	3.82		3.84	0.001025	0.65	8.03	16.84	0.3	0.57	0.43
North East Breach Spillway	63.929*	IDF	5.2	3.25	3.82		3.84	0.00105	0.65	7.98	16.9	0.3	0.57	0.43
North East Breach Spillway	62.050*	IDF	5.2	3.25	3.81		3.84	0.001072	0.65	7.94	16.96	0.31	0.56	0.44
North East Breach Spillway	60.171*	IDF	5.2	3.25	3.81		3.83	0.001092	0.66	7.91	17.03	0.31	0.56	0.44
North East Breach Spillway	58.293*	IDF	5.2	3.25	3.81		3.83	0.001107	0.66	7.89	17.09	0.31	0.56	0.44
North East Breach Spillway	56.414*	IDF	5.2	3.25	3.81		3.83	0.00112	0.66	7.88	17.16	0.31	0.56	0.44
North East Breach Spillway	54.536*	IDF	5.2	3.25	3.81		3.83	0.001127	0.66	7.87	17.22	0.31	0.56	0.44
North East Breach Spillway	52.657*	IDF	5.2	3.25	3.8		3.83	0.001127	0.66	7.88	17.24	0.31	0.55	0.45
North East Breach Spillway	50.779*	IDF	5.2	3.25	3.8		3.82	0.001127	0.66	7.89	17.29	0.31	0.55	0.45
North East Breach Spillway	48.9	IDF	5.2	3.25	3.8		3.82	0.001125	0.66	7.91	17.39	0.31	0.55	0.45
North East Breach Spillway	47.171*	IDF	5.2	3.25	3.8		3.82	0.001189	0.67	7.78	17.39	0.32	0.55	0.45
North East Breach Spillway	45.443*	IDF	5.2	3.25	3.79		3.82	0.00126	0.68	7.64	17.4	0.33	0.54	0.46
North East Breach Spillway	43.714*	IDF	5.2	3.25	3.79		3.81	0.001339	0.69	7.51	17.41	0.34	0.54	0.46
North East Breach Spillway	41.986*	IDF	5.2	3.25	3.79		3.81	0.001428	0.71	7.36	17.42	0.35	0.54	0.46
North East Breach Spillway	40.257*	IDF	5.2	3.25	3.78		3.81	0.001529	0.72	7.22	17.44	0.36	0.53	0.47
North East Breach Spillway	38.529*	IDF	5.2	3.25	3.78		3.81	0.001644	0.74	7.07	17.47	0.37	0.53	0.47
North East Breach Spillway	36.800*	IDF	5.2	3.25	3.77		3.8	0.001778	0.75	6.91	17.5	0.38	0.52	0.48
North East Breach Spillway	35.071*	IDF	5.2	3.25	3.77		3.8	0.001934	0.77	6.74	17.54	0.4	0.52	0.48
North East Breach Spillway	33.343*	IDF	5.2	3.25	3.76		3.8	0.002122	0.79	6.56	17.59	0.41	0.51	0.49
North East Breach Spillway	31.614*	IDF	5.2	3.25	3.76		3.79	0.002345	0.82	6.38	17.64	0.43	0.51	0.49
North East Breach Spillway	29.886*	IDF	5.2	3.25	3.75		3.79	0.002616	0.84	6.18	17.71	0.45	0.5	0.5
North East Breach Spillway	28.157*	IDF	5.2	3.25	3.74		3.78	0.002963	0.87	5.96	17.77	0.48	0.49	0.51
North East Breach Spillway	26.429*	IDF	5.2	3.25	3.73		3.78	0.003414	0.91	5.72	17.83	0.51	0.48	0.52
North East Breach Spillway	24.7	IDF	5.2	3.25	3.72		3.77	0.00405	0.96	5.44	17.87	0.55	0.47	0.53
North East Breach Spillway	22.800*	IDF	5.2	3.23	3.72		3.76	0.003959	0.96	5.43	17.49	0.55	0.49	0.51
North East Breach Spillway	20.900*	IDF	5.2	3.21	3.71		3.75	0.0039	0.96	5.42	17.24	0.55	0.5	0.5
North East Breach Spillway	19.000*	IDF	5.2	3.19	3.7		3.75	0.003855	0.96	5.42	17.07	0.54	0.51	0.49
North East Breach Spillway	17.100*	IDF	5.2	3.17	3.69		3.74	0.003824	0.96	5.41	16.86	0.54	0.52	0.48
North East Breach Spillway	15.200*	IDF	5.2	3.15	3.69		3.73	0.003795	0.96	5.4	16.66	0.54	0.54	0.46
North East Breach Spillway	13.300*	IDF	5.2	3.13	3.68		3.73	0.003771	0.97	5.38	16.47	0.54	0.55	0.45
North East Breach Spillway	11.400*	IDF	5.2	3.12	3.67		3.72	0.003763	0.97	5.36	16.26	0.54	0.55	0.45
North East Breach Spillway	9.500*	IDF	5.2	3.1	3.66		3.71	0.003766	0.97	5.34	16.08	0.54	0.56	0.44
North East Breach Spillway	7.600*	IDF	5.2	3.08	3.66		3.7	0.003789	0.98	5.3	15.89	0.54	0.58	0.42
North East Breach Spillway	5.700*	IDF	5.2	3.06	3.65		3.7	0.00383	0.99	5.26	15.71	0.54	0.59	0.41
North East Breach Spillway	3.800*	IDF	5.2	3.04	3.64		3.69	0.003878	1	5.22	15.54	0.55	0.6	0.4
North East Breach Spillway	1.900*	IDF	5.2	3.02	3.63		3.68	0.003962	1.01	5.16	15.35	0.55	0.61	0.39
North East Breach Spillway	0	IDF	5.2	3	3.62	3.49	3.67	0.004082	1.02	5.1	15.17	0.56	0.62	0.38

APPENDIX C

Wetlands Analysis

Design Scenario: 1-Year, 1-Hour rainfall event (10.8 mm) with 70% assumed Runoff Proportion									
PIC	**Existing Base Elevation (mOD)**	**Proposed top of Wetland Soil layer (mOD)**	**Proposed wetland permanent water elevation (mOD)**	**Proposed low flows outlet invert level elevation (mOD)**	**Peak water surface elevation during design event (mOD)**	**Proposed base elevation of Riprap overflow spillway (mOD)**	**Top elevation of riprap overflow spillway (mOD)**	**Estimated Runoff Volume (m3)**	**Maximum outflow rate to achieve 7 - day residence time (L/s)**
PIC A	10.000	10.300	10.500	10.500	10.937	11.137	11.637	1,495	2.5
PIC B	1.500	2.146	2.346	2.346	3.125	3.325	3.825	772	1.3
PIC C	1.500	1.800	2.000	2.000	2.346	2.546	3.046	1,802	3.0
PIC D	1.000	1.300	1.500	1.500	1.928	2.128	2.628	1,438	2.4
PIC E	1.800	2.134	2.334	2.334	2.905	3.105	3.605	2,739	4.5
PIC F	1.600	1.900	2.100	2.100	2.334	2.534	3.034	780	1.3
PIC G	0.900	1.200	1.400	1.400	1.820	2.020	2.520	1,647	2.7
PIC J	N/A PIC G and J combined								
PIC K	1.950	2.250	2.450	2.450	3.709	3.909	4.409	606	1.0
PIC L (North)	N/A PIC L (North) to discharge via spillway to PIC K at closure without retention within PIC L (North)								
PIC L (South)	13.500	13.800	14.000	14.000	14.666	14.866	15.366	1,835	3.0
PIC M	N/A - PIC M to be merged with PIC L (south at closure)								

Overflow Riprap Spillways			
	PIC's B and C	PIC's L(south), K, E and F	PIC A
Base Width (m)	7.5	7.5	7.5
Height (m)	0.4	0.6	0.35
Side Slopes	to suit	to suit	to suit

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APPENDIX M

Physical Stability Monitoring Plan 2021

TECHNICAL MEMORANDUM

DATE 15 November 2021 **Project No.** 21452853.TM06.A1

TO Kevin McMahon, Aughinish Alumina Limited

CC Tom Hartney, Colm Cribbin, Louise Clune, Rory O'Dwyer (AAL)

FROM Brian Keenan; Gerd Janssens **EMAIL** bkeenan@golder.com

PHYSICAL STABILITY MONITORING PLAN FOR THE AAL BRDA: 2021 UPDATE

1.0 INTRODUCTION

This Physical Stability Monitoring Plan for the AAL BRDA has been developed by Golder Associates Ireland Ltd as Engineer of Record (EoR) following an assessment of the current AAL licence (IEL P0035-07) and the 2018 Best Available Techniques (BAT) Reference Document for the Management of Waste from the Extractive Industries (BREF), in accordance with Directive 2006/21/EC (EUR 28963 EN), (MWEI BREF 2018).

2.0 LICENCE (IEL P0035-07)

AAL are required to manage and operate the BRDA under the Conditions of the Industrial Emissions Licence (current revision is P0035-07 issued on 28 September 2021) issued by the Irish Environmental Protection Agency (Agency). Conditions relating to monitoring of the BRDA are replicated below.

- ***Condition 8.5.10*** requires that that AAL submit to the Agency an Operation Plan (OP) for the BRDA. This plan is required to be reviewed annually and amendments notified to the Agency. This OP is required to include an Environmental Monitoring Program which shall, where appropriate, be in accordance with BAT and have regard to the Landfill Monitoring Manual published by the Agency.

- ***Condition 8.5.15*** requires that report on the status of the BRDA is required to be provided annually in the Annual Environmental Report (AER) for the facility. This report is required to contain, at a minimum, the elements detailed in *Schedule D: Annual BRDA Status Report* of the licence.

- ***Condition 8.5.25:*** The BRDA is required to be monitored as set out in *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence.

- ***Condition 8.5.26:*** AAL shall arrange for a Biennial Independent Audit. AAL shall arrange a Safety Evaluation of Existing Dam (SEED) Audit at a frequency agreed with the Agency, which shall substitute the Biennial Independent Audit for the same year of occurrence.

- ***Condition 8.5.27:*** All inspections, monitoring, annual reviews and independent audits shall, where appropriate be carried out in accordance with the requirements of BAT and any technical guidance or decisions issued by, or on the behalf of, the Committee for the Adaption to Scientific and Technical Progress of Directive 2006/21/EC on the management of waste from extractive industries.

- ***Condition 9.4.4:*** AAL shall maintain an Internal Emergency Plan (EP) specifying the measures to be taken in the event of an accident at the BRDA.

- ***Condition 9.4.5:*** AAL shall, at a minimum of annually, consult with the Local Authority and the Principal Response Agencies in relation to any information that may be required by them regarding external emergency planning for major accidents at the BRDA. Evidence of these consultations is required to be provided in the Annual Environmental Report (AER) for the facility.
- ***Condition 10.3.1:*** AAL shall maintain and submit to the Agency for review, a fully detailed and costed plan for aftercare (Aftercare Plan) for a minimum aftercare period of 30 years. The Aftercare Plan shall be reviewed at a minimum of annually, with amendments notified to the Agency for their approval prior to their implementation.
- ***Condition 11.1 (iii):*** AAL shall notify the Agency by both telephone and email or webform, as soon as practicable after the occurrence of any malfunction or breakdown of key control or monitoring equipment set out in Schedule C of the licence, which is likely to lead to the loss of control of the abatement system.
- ***Condition 11.3.1:*** AAL shall notify the Agency by both telephone and email or webform, as soon as practicable after the occurrence of i) any event that is likely to affect the stability of the BRDA or ii) any significant effects on the environment revealed by the control and monitoring procedures on the BRDA.

3.0 PHYSICAL STABILITY MONITORING PLAN

A physical stability monitoring plan (Plan) has been established for the BRDA and consists of scheduled installation and monitoring of geotechnical instruments installed within the facility, along with a series of scheduled audits, inspections and conformance checks to assess the performance of the BRDA.

A comprehensive physical stability monitoring plan is developed in the planning and design phase and is implemented and adapted based on the monitoring findings in the operational phase and in the closure and after-closure phases. The upstream raising of the BRDA is an ongoing operation during the operational life of the facility; therefore, the Plan is a live document requiring:

- Addition of instruments as the BRDA increases in elevation;
- Addition of interim instruments and monitoring programs to manage specific construction projects and/or events;
- Replacement of instruments resulting from damage / missing due to operations; and
- Removal of instruments from the plan as Phases of the BRDA overlap.

3.1 Who should read and use this Plan

The following people/personnel should familiarize themselves with the Plan:

- Operational personnel responsible for BRDA;
- BRDA safety monitoring personnel;
- BRDA environmental monitoring personnel;
- BRDA geotechnical monitoring personnel;
- Personnel responsible for maintenance of plant and instruments located in the BRDA; and
- The Design Engineer, Contractor(s) and Work Supervisor(s) responsible for construction activities at the BRDA.

3.2 Objective

The objective of the monitoring of the BRDA is to assess the performance of the facility and to mitigate the risk of instability in the short and long-term.

An updated Risk Assessment and Break-Out Study Summary Report (Golder 2019) has been undertaken which identified the mechanisms which could negatively affect the partial or total structural stability of the BRDA. The following is a list of the identified mechanisms:

- Earthquake induced Slope Failure (including Dynamic Liquefaction);
- Blast induced Slope Failure (including Dynamic Liquefaction);
- Static Slope Failure (including Static Liquefaction);
- Slope Instability (including Static Liquefaction);
- Foundation Instability;
- Slope Failure from Tidal Surge or Wave Event;
- Slope Failure from Erosion (Rainfall Event)
- Slope Failure from Erosion (Overtopping)

The target levels for standards-based design criteria for tailings dams during Construction, Operation, Closure and After-Closures are determined from the tailings dam classification, which are based on the consequences in the event of failure.

The design of the BRDA incorporates the facility's ability to withstand the design events, with an appropriate factor of safety, based on the consequence classification for the facility and design return periods for the events.

Instrumentation has been installed in the bauxite residue and underlying estuarine soils to monitor slope stability and static liquefaction parameters. Additional instrumentation will be installed prior to planned blast events.

No instrumentation has been put in place to monitor highly improbable and infrequent events like an earthquake, a tidal surge or wave event, a flood event or operational hazards. Water level instrumentation is in place for real-time monitoring of water inventories in the PICs, the SWP and the LWP (Vega Radar Water Elevation Probes). In addition, there are a number of CCTV installations in the BRDA which may be utilized to monitor unforeseen or operational hazards.

Audits, inspections and conformance checks are utilised to reduce the risk of the other mechanisms leading to instability.

3.3 Monitoring Instrumentation

This section deals with the MWEI BREF 2018 guidance for the inclusion of the following aspects in the Plan:

- Number and Location of Control Stations (understood to monitoring instrument installations);
- Type and Purpose of Monitoring Measure;
- Appropriate Instrumentation Selection;
- Frequency of Monitoring; and
- Responsible Person(s).

The location of the current configuration of Phase 1 BRDA monitoring instruments are shown in Drawings 01 to 04 in Appendix A and the Summary Tables are provided in Appendix B (as of September 2021).

The type and purpose of the monitoring instrumentation and the parameters to be monitored are summarised in Table 1 below. This Table follows the format as per Table 4.18 (*Reported physical stability monitoring parameters and frequencies for ponds, dams and heaps*) and Table 4.19 ((*Reported physical stability monitoring instrumentation for ponds, dams and heaps*) of the MWEI BREF 2018.

During the construction of the lower stages of the Phase 1 BRDA, pore pressures in the bauxite residue had previously been monitored through vibrating wire piezometers installed beneath the footprint of the rock fill stage raises. These former piezometers are no longer operational and new vibrating wire piezometers are not installed for stage raise construction as an understanding of the allowable rate of construction was developed and AAL have adopted mud farming methods.

Blasting may be undertaken nearby the BRDA to source rock fill for the embankment raises. Should this work commence, strong motion accelerographs will be positioned locally on the BRDA to monitor the blast vibration and real-time monitoring vibrating wire piezometers will be installed locally in the BRDA to monitor the dynamic pore pressure, if any.

3.4 Monitoring Frequencies

Schedule C.7: Monitoring at the Bauxite Residue Disposal Area of the licence lists the minimum monitoring frequencies for geotechnical parameters of the BRDA.

Table 1: BRDA Geotechnical Parameter Monitoring Frequencies

Parameter	Licence Monitoring Frequency	BRDA Monitoring Frequency	Proposed MWEI BREF 2018 Monitoring Frequency
Water Levels	Weekly	Continuously	Weekly to Continuously
Standard Walk-Over Condition and Stability Checks	Daily	Daily	
Phreatic Surface	Quarterly	Quarterly	Weekly to Monthly
Hydrostatic Pore Pressure	Quarterly	Quarterly	Weekly to Monthly
Seismicity	As required by Agency	Annually	Continuously (seismic areas)
Dynamic Pore Pressure and Liquefaction	Not listed	Pore Pressures (Quarterly) Liquefaction Potential (every 4 years and/or as required)	Weekly to Fortnightly (seismic areas)
Settlement / Movement	Quarterly	Quarterly	Continuously to Monthly
Geotechnical Parameters of Extractive Waste	Not listed	Daily for specific parameters Thorough review every 4 years	Weekly to Monthly

Parameter	Licence Monitoring Frequency	BRDA Monitoring Frequency	Proposed MWEI BREF 2018 Monitoring Frequency
Extractive Waste Placement	Annually	Pumped residue tonnages calculated monthly All trucked residue logged daily and weekly Annual topographical survey to calculate volume of placed residue.	Weekly to Monthly

The monitoring frequencies for the geotechnical parameter of the BRDA are in accordance with the licence Conditions but generally are at greater intervals than those listed in Table 4.18 (*Reported physical stability monitoring parameters and frequencies for ponds, dams and heaps*) of the MWEI BREF 2018. The justifications for the physical stability monitoring methods and frequencies for the BRDA are listed below:

- Facility is sited in a region characterised by very low levels of seismic activity.
- Active facility for over 30 years with a significant database of extractive waste and site characteristics supporting an effectual understanding of the performance and stability.
- The Phase 1 BRDA has reached its current permitted design height in several sectors (Stage 10) and is performing in accordance with the design criteria and CDA guidelines. Instrumentation have been installed periodically as the facility reached target elevations. The required number of instruments and surveillance frequency is evaluated as part of the Annual Review.
- BRDA has a strong performance history showing consistently stable monitoring readings without significant fluctuations and achieving appropriate FoS.
- Trigger levels have been set, which when exceeded, will require the instruments to be read at a higher frequency interval.
- Although, the bauxite residue in the Phase 1 BRDA (below Stage 7 at elevation 18 mOD) has been identified to have the potential for liquefaction, the probability of liquefaction based on the factor of safety determined is in the Highly Improbable to Almost Impossible range. Trigger mechanisms would be required to initiate liquefaction, and these have been identified as part of the risk assessment, and extreme rainfall events, increased rate of rise, seismic loading and foundation creep. The risk of liquefaction has been reduced through design (undrained stability analyses), monitoring surveillance (piezometers and inclinometers) and good operational practices.
- AAL engage in mud-farming (increases undrained shear strength parameters) and have good operational practices.
- No storage of free water on the BRDA surface and/or in an upper-level perimeter interceptor channel (the upper-level PIC was not constructed following the Kolontár failure in 2010).

Table 2: BRDA Inspections and Monitoring Instrumentation

Parameter	Instrumentation	Location	Responsible Person(s)	Frequency & Reporting	Monitoring Purpose
Standard Walk-Over Condition and Stability Check	Visual Inspection	BRDA and ancillary infrastructure	▪ AAL Operators ▪ AAL BRDA Engineer (for protection of physical assets of the facility)	▪ Daily ▪ Checklist and Log	▪ Conformance check by Operators ▪ Conformance check by BRDA Engineer
Water level in Phase 1 PIC, Phase 2 PIC and SWP	Automatic Elevation Readers (Vega Radar Water Elevation Probes)	▪ Phase 1 PIC ▪ Phase 2 PIC ▪ SWP	▪ AAL Control Room Operator (CRO) (Water Management Standard Work Method	▪ Continuously ▪ Checklist and Log ▪ Weekly and Monthly Reports	▪ Measures Water Inventory ▪ Measures Flood Capacity ▪ Measures Freeboards
Pore Pressure and Phreatic Surface Position (permanent)	Standpipe and Casagrande Piezometers	**Bauxite Residue** ▪ Multi-level , upstream of stage raises at defined section lines **Estuarine Soils** ▪ Multi-level , upstream of stage raises at defined section lines ▪ Downstream of Phase 1 and Phase 2 PIC OPWs at defined section lines	▪ Golder as EoR ▪ AAL BRDA Engineer (for protection of physical assets of the geotechnical instruments)	▪ Quarterly ▪ Quarterly Review Memorandum ▪ Annual Review	▪ Measures phreatic surface position in bauxite residue ▪ Measures pore pressure in bauxite residue ▪ Measures pore pressure in foundation estuarine soils ▪ Measures groundwater elevation in foundation estuarine soils ▪ Enables stability assessments

Parameter	Instrumentation	Location	Responsible Person(s)	Frequency & Reporting	Monitoring Purpose
Pore Pressure (temporary)	Vibrating Wire Piezometers	▪ Below construction work footprint into Bauxite Residue ▪ Adjacent slope to Borow Pit and into Bauxite Residue	▪ Golder as EoR ▪ AAL BRDA Engineer	▪ As required by Method Statement / Construction Quality Assurance Plan ▪ CQA Validation Report	▪ Measure temporary pore pressure increases in bauxite residue for construction projects involving > 3m height loading over bauxite residue i.e., SCDC raises ▪ Measure possible pore pressure increases during blasting
Seismicity	Not measured on site (not a seismic area)	Sourced from DIAS, BGS, SHARE 2013 and UK HSE 2002	▪ Golder as EoR	▪ Annual Review	▪ Enables seismic assessments
Horizontal Movement (permanent)	Inclinometers	Multi-level, upstream of stage raises at defined section locations into Bauxite Residue, Estuarine Soils and underlying bedrock	▪ Golder as EoR ▪ AAL BRDA Engineer	▪ Quarterly ▪ Quarterly Review Memorandum ▪ Annual Review	▪ Measures lateral displacement of the red mud and foundation estuarine soils ▪ Enables assessment of physical stability
Vertical Movement and Deformation (permanent)	▪ Extensometer (installed in clusters on select inclinometer casings) ▪ Aerial Surveys	▪ Multi-level, upstream of stage raises at defined section locations into Bauxite Residue ▪ Phase 1 and 2 BRDAs	▪ Golder as EoR ▪ AAL BRDA Engineer	▪ Quarterly ▪ Quarterly Review Memorandum ▪ Annual Review	▪ Measures settlement (deformation) of the bauxite residue at defined sections. ▪ Measures volume of bauxite residue placed and rate of rise ▪ Enables assessment of physical stability

Parameter	Instrumentation	Location	Responsible Person(s)	Frequency & Reporting	Monitoring Purpose
Horizontal and Vertical Movement and Deformation (temporary)	▪ Geodetic Points	▪ Multi-level at crest of stage raises in NE and SW sectors of Phase 1 BRDA	▪ Golder as EoR ▪ AAL BRDA Engineer	▪ As required, typically quarterly or annually ▪ Quarterly Review Memorandum ▪ Annual Review	▪ Measures 3D movement in area showing movements above thresholds values or continuing trends
Real Time Monitoring Systems	Water level and CCTV real-time monitoring installed on site Geotechnical real-time monitoring proposed to be installed for specific projects i.e. Blasting near BRDA	▪ Geotechnical real-time monitoring not considered to be currently required for general BRDA perimeter due to active farming, good operational practices, consistent stable readings without significant fluctuations and achieving appropriate FoS. ▪ Particular segments of the BRDA perimeter i.e., adjacent to Borrow Pit	▪ Golder as EoR ▪ AAL BRDA Engineer	As required, real-time vibrating wire piezometers proposed to be installed for development of Borrow Area requiring blasting	▪ Measures pore pressures and/or horizontal and vertical movements and deformations ▪ Security
Geotechnical Parameters	▪ Cone Penetration Testing (CPTu) and MOSTAP sampling ▪ Seismic CPTu	Multi-level, upstream of stage raises at defined section locations into Bauxite Residue and Estuarine Soils	▪ Golder as EoR ▪ AAL BRDA Engineer	▪ As required, typically every 4 years ▪ Summary Report ▪ Parameters updated in Annual Review	▪ Measures undrained shear strength ▪ Measures shear wave velocity ▪ Samples taken for geotechnical laboratory testing (index, strength and SG testing)

Parameter	Instrumentation	Location	Responsible Person(s)	Frequency & Reporting	Monitoring Purpose
Extractive Waste Placement	▪ Flow meters ▪ Aerial Surveys	▪ Bauxite Residue Pipelines ▪ Phase 1 and 2 BRDAs	▪ AAL BRDA Engineer	▪ Continuous Flow Measurement of pumped residue ▪ Daily and Weekly logs of trucked residue ▪ Annual Capacity and Rate of Raise Check ▪ Daily Mud Farming Logs ▪ Monthly Tonnage Deposition Calculations ▪ Annual Volume Capacity Check	▪ Measures volume of bauxite residue placed and rate of rise

3.5 Scheduling (Control Periods and Conformance Checks by Operators)

In accordance with *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence the BRDA is visually inspected weekdays (Monday to Friday). A standard walk-over condition and visual check is conducted by the AAL BRDA Engineer.

The control period for the daily conformance checks is for as long as the facility is in the operational phase. The reduction in frequency of this monitoring condition in the closure and after-closure phases will be agreed with the Agency.

3.6 Conformance Check Methods and Evaluation

AAL documented procedures for stability monitoring of BRDA .

Golder as Design Engineer and EoR are notified as required.

3.7 Internal Audits

Conducted by AAL as part of Environment Management System which is certified to ISO14001:2015

3.8 External Audits

External Audits are a system for evaluating the performance and safety of the BRDA on a regular basis by qualified and experienced experts and may be conducted by the Design Engineer(s) and/or EoR or they may be Independent External Audits i.e., someone(s) who was/is not involved with the design or overall service.

- Golder provide a Quarterly Review Memorandum following the quarterly reading of the monitoring instrumentation, a visual inspection of the BRDA and the review of the EoR Monthly Communication Reports, prepared by AAL.
- In accordance with *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence the BRDA is subject to an Annual Review. The Annual Review has been conducted by Golder since 2004.
- In accordance with *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence the BRDA is subject to an Independent Audit every 2 years.

  The most recent Independent Audit was conducted by Golder Canada in 2018. COVID-19 restrictions prohibited an Independent Audit planned for Q1 2021 and it is currently rescheduled for Q1 2022.
- In accordance with *Schedule C.7: Monitoring at the Bauxite Residue Disposal Area* of the licence the BRDA is subject to a SEED Audit at a minimum frequency of 15 to 20 years.

  The SEED Audit will be conducted in accordance with the Canadian Dam Association (CDA) Dam Safety Review (DSR) Guidelines (2014) by an external geotechnical consultant who is independent of the EoR. The most recent CDA DSR was conducted by SLR Consulting Limited in 2019.

3.9 Responsible Person(s)

The Responsible Person(s) for the Monitoring and Reporting tasks are listed in Table 2.

3.10 Data Storage and Reporting

Data Storage as detailed by AAL. The BRDA physical stability monitoring reporting requirements are listed below:

- AAL prepare an EoR Monthly Communication Report which is distributed to Golder.
- The Quarterly Review is conducted by Golder and a Quarterly Review Memorandum is submitted to AAL.
- AAL host a quarterly EoR meeting, at which is presented the quarterly EoR BRDA Review. The minutes of these meetings are stored internally by AAL along with the quarterly EoR BRDA Review presentations.
- The Annual Review is conducted by Golder and a Report is submitted to AAL.
- A Report on the Annual Review and the Annual BRDA Status is compiled by AAL for inclusion in the AER which is submitted to the Agency.
- The External Independent Audit is arranged by the EoR and is conducted by Senior Golder and/or other Senior Consultants who are external to the overall service, and a Report is submitted to AAL which is subsequently submitted to the Agency.
- The SEED Audit is conducted Audit is conducted by an external geotechnical consultant who is independent of the EoR as per Schedule C.7. frequency and a Report will be submitted to AAL.

3.11 Criteria for Assessment

The assessment criteria for the BRDA physical stability monitoring parameters are discussed below.

Threshold values for monitoring parameters and actions ranked in level of threshold exceedance are presented in Table 3.

3.11.1 Standard Walk-Over Condition and Stability Check

Visual assessment of the predetermined check list items in accordance with the AAL documented procedures for the stability monitoring of the BRDA.

3.11.2 Inclinometers

All inclinometers have been installed with the A-axis perpendicular to the slope face, and the negative readings indicating displacement downslope. The B-axis indicates movement parallel to the slope and therefore tend to be less of a concern. Current inclinometer readings are taken and compared to the readings from the previous quarter and to historic readings.

3.11.3 Extensometers

Settlement markers installed in clusters of 2 to 6 at varying depths along the inclinometer casing are referred to as Spiders or Extensometers. Current extensometer readings are taken and compared to the readings from the previous quarter and to historic readings.

3.11.4 Piezometers

Piezometers are installed to varying depths within the bauxite residue and the estuarine soils. Current piezometer readings are taken and compared to the readings from the previous quarter and to historic readings.

Table 3 below lists the Physical Stability Monitoring Criteria along with the proposed action items which are ranked in order of the degree of importance and/or exceedance of the thresholds i.e., a minor exceedance may only require notification and an assessment while significant exceedances will trigger the additional actions.

Table 3: BRDA Physical Stability Monitoring Criteria

Task / Instrument	Threshold(s)	Minor Exceedances Action(s)	Significant Exceedances Action(s)
Standard Walk-Over Condition and Stability Check	Noticeable cracking, deformation, settlement, bulging at toe, water seeping at downstream toe of outer perimeter wall of PIC or east side of Phase 2 BRDA.	▪ Notify AAL BRDA Engineer and EoR ▪ Assessment of potential instigating factors	▪ Increased frequency of monitoring ▪ Isolation of sector of BRDA ▪ Investigation of issue ▪ Installation of additional monitoring instrumentation ▪ Design and Construction of remedial works
Inclinometers	Greater than 5mm movement from previous quarter and/or greater than 20mm trend movement in one year	▪ Noted in Quarterly Memorandum ▪ Assessment of potential instigating factors	▪ Notify AAL BRDA Engineer ▪ Increased frequency of monitoring ▪ Installation of additional monitoring instrumentation ▪ Isolation of sector of BRDA ▪ Investigation of issue ▪ Design and Construction of remedial works
Extensometers	Greater than 5mm movement from previous quarter and/or greater than 20mm trend movement in one year	▪ Noted in Quarterly Memorandum ▪ Assessment of potential instigating factors	▪ Notify AAL BRDA Engineer ▪ Increased frequency of monitoring ▪ Installation of additional monitoring instrumentation ▪ Isolation of sector of BRDA ▪ Investigation of issue ▪ Design and Construction of remedial works
Piezometers in Bauxite Residue	Greater than 1.5m movement from previous quarter and/or greater than 2.0m net movement in one year	▪ Noted in Quarterly Memorandum ▪ Assessment of potential instigating factors	▪ Notify AAL BRDA Engineer ▪ Increased frequency of monitoring ▪ Installation of additional monitoring instrumentation ▪ Isolation of sector of BRDA ▪ Investigation of issue ▪ Design and Construction of remedial works

Task / Instrument	Threshold(s)	Minor Exceedances Action(s)	Significant Exceedances Action(s)
Piezometers in Estuarine Soils	Greater than 0.5m movement from previous quarter and/or greater than 1.0m net movement in one year	▪ Noted in Quarterly Memorandum ▪ Assessment of potential instigating factors Design and Construction of remedial works	▪ Notify AAL BRDA Engineer ▪ Increased frequency of monitoring ▪ Installation of additional monitoring instrumentation ▪ Isolation of sector of BRDA ▪ Investigation of issue

3.12 Schedule for Plan Review

The Physical Stability Monitoring Plan requires updating at a minimum frequency of annually. The Responsible Persons are the Golder EoR and the AAL BRDA Engineer.

3.13 BRDA Monitoring Plan

AAL have documented procedures for the stability monitoring of the BRDA which are documented in the Operation, Safety and Maintenance (OSM) manual. These procedures include a visual inspection plan of the major components of the dam, criteria for observations, identification of key areas for intensive inspection, frequency of the inspections, reporting procedures, inspection procedure following significant events (storm event), training and experience requirements of inspectors, procedures to escalate findings and data storage.

3.14 Emergency Planning

Documented procedures in place detailed in the BRDA OSM manual.

In accordance with Condition 9.4.5 of the licence, AAL consult with the Local Authority and the Principal Response Agencies in relation to any information that may be required by them regarding external emergency planning for major accidents at the BRDA. Evidence of these consultations is provided in the Annual Environmental Report (AER).

3.15 Real-Time Monitoring Systems

Real-time monitoring systems are not considered to be currently required for BRDA, see Section 3.3.

Real-time monitoring will be considered for specific purposes within or close to the BRDA and for closure and after-closure phases.

- One such purpose was undertaken between 30 November 2020 and 09 March 2021. Four (4) level loggers and a baro logger were installed in standpipe piezometers along the north sector of the BRDA to continuously monitor water levels and barometric pressure during the winter season. Overall, the range of piezometric elevations during the winter period can be considered stable with only low levels of change recorded in the approx. 3-month period.
- Real-time monitoring for the potential of dynamic pore pressure increases in the bauxite residue during blasting of the Borrow Pit, in the sector of the BRDA nearest to the Borrow Pit, i.e., Section K-K, is scheduled to be installed during Q1 of 2022.

4.0 SUMMARY

The current location and number and monitoring instruments currently installed within the Phase 1 and 2 BRDAs and the frequency of readings are considered sufficient to monitor the performance of the BRDA.

The following comments and recommendations can be made regarding current and future monitoring installations and construction works:

- Vibrating wire piezometers to monitor dynamic pore pressure and liquefaction are not currently required due to the very low seismic region, the relatively constant phreatic level within the BRDA and the controlled bauxite residue deposition plan.
- Static liquefaction concerns for upstream raised tailings facilities are currently particularly relevant globally. The facility has recently undergone its four-year Geotechnical Review and the updating of the Risk Assessment and Break-Out Study. The facility maintains stability FoS in accordance with current CDA guidelines.
- Should blasting be undertaken in the vicinity of the BRDA. Strong motion accelerographs will be positioned locally on the BRDA to monitor the blast vibration and vibrating wire piezometers will be installed locally in the BRDA to monitor the dynamic pore pressure.
- An instrumentation plan covering the life of the facility has been developed to provide a schedule and understanding of when future instrumentation will be installed. This instrumentation plan is provided in Appendix C and shall be reviewed on an annual basis.

5.0 REFERENCES

BGS, British Geological Survey, Keyworth, Nottingham, NG12 5GG, UK, www.bgs.ac.uk

CDA 2013, Canadian Dam Association, Dam Safety Guidelines 2007, Revised 2013

CDA 2014, Canadian Dam Association, Technical Bulletin 2014, Application of Dam Safety Guidelines to Mining Dams

DIAS, Dublin Institute for Advanced Studies, 10 Burlington Road, Dublin 4, Ireland, www.dias.ie

HSE 2002, Seismic Hazard: UK Continental Shelf, 2002/005

MAC 2019, The Mining Association of Canada, Developing an Operation, Maintenance and Surveillance (OMS) Manual for Tailings and Water Management Facilities, 2nd Edition, February 2019

MWEI BREF 2018, Best Available Techniques (BAT) Reference Document for the Management of Waste from Extractive Industries, in accordance with Directive 2006/21/EC; EUR 28963 EN; Publications Office of the European Union, Luxembourg, 2018; ISBN 978-92-79-77178-1; doi:10.2760/35297, JRC109657

SHARE 2013, Seismic Hazard Harmonization in Europe (SHARE) Project, ENV.2008.1.3.1.1, Development of a common methodology and tools to evaluate earthquake hazard in Europe, 2013

Signature Page

Golder Associated Ireland Limited

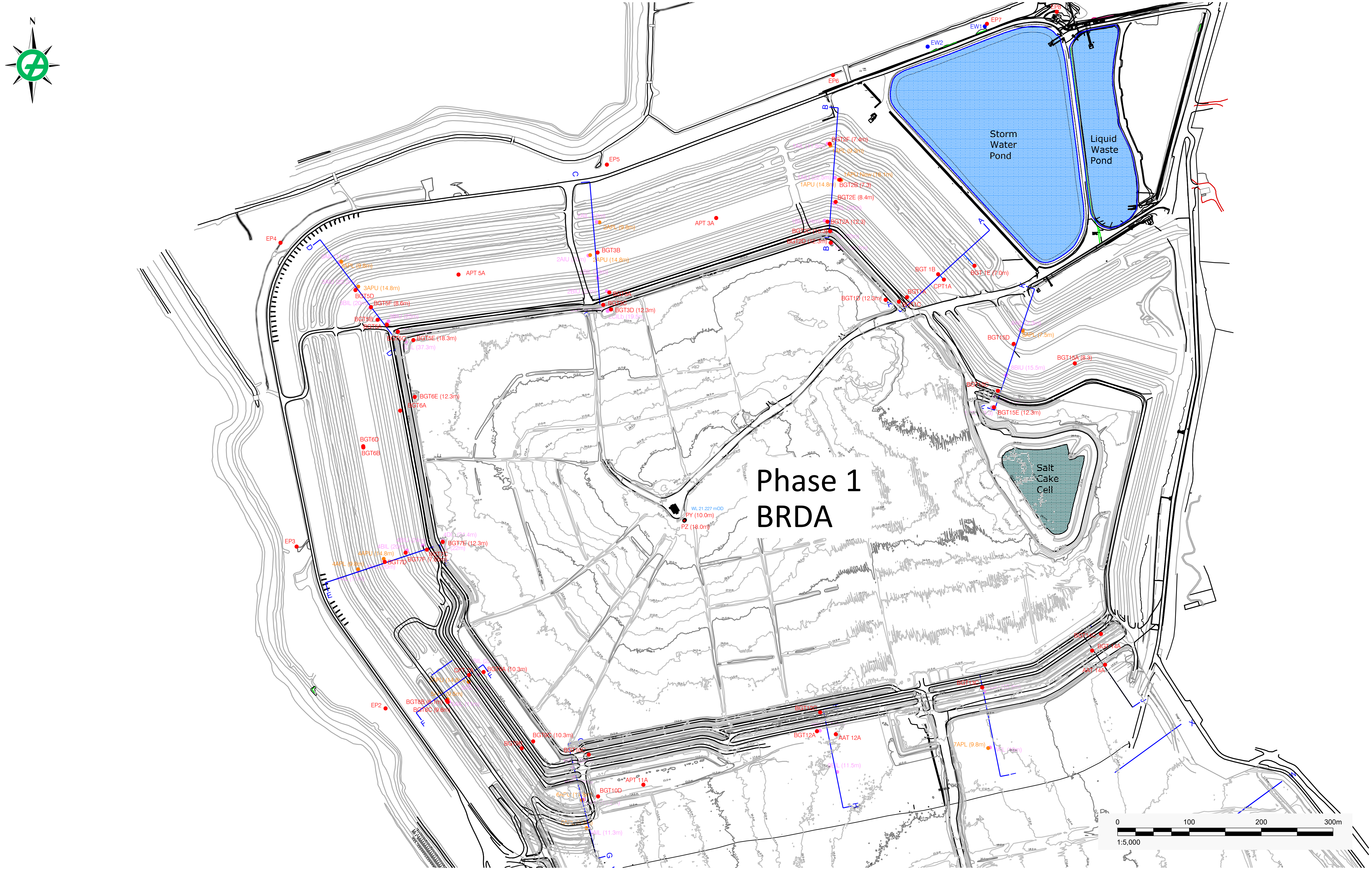
Gerd Janssens
Geotechnical Engineer

Brian Keenan
Project Manager, Associate

GJ/BK/ar

APPENDIX A

Current Drawings (July 2021)



LEGEND:

- Abstraction Well
- Standpipe
- Inclinometer
- Piezometer

NOTE(S)

1. BRDA CONTOURS AND MAP UPDATED FOLLOWING DRONE SURVEY CONDUCTED BY ASM LTD (APRIL 2021)
2. NEW INSTRUMENTS INSTALLED IN OCTOBER / NOVEMBER 2020
3. NEW INSTRUMENTS INSTALLED IN JUNE 2021

CLIENT	AUGHINISH ALUMINA LTD
PROJECT	AUGHINISH BRDA 2021 MONITORING
CONSULTANT	GOLDER MEMBER OF WSP
YYYY-MM-DD	2021-Oct-22
PREPARED	POB
DESIGN	BK
REVIEW	DC
APPROVED	BK
TITLE	PHASE 1 BRDA MONITORING INSTRUMENT LOCATION PLAN: Q3 2021
PROJECT No.	21452853
DRAWING No.	01 of 04
Rev.	A
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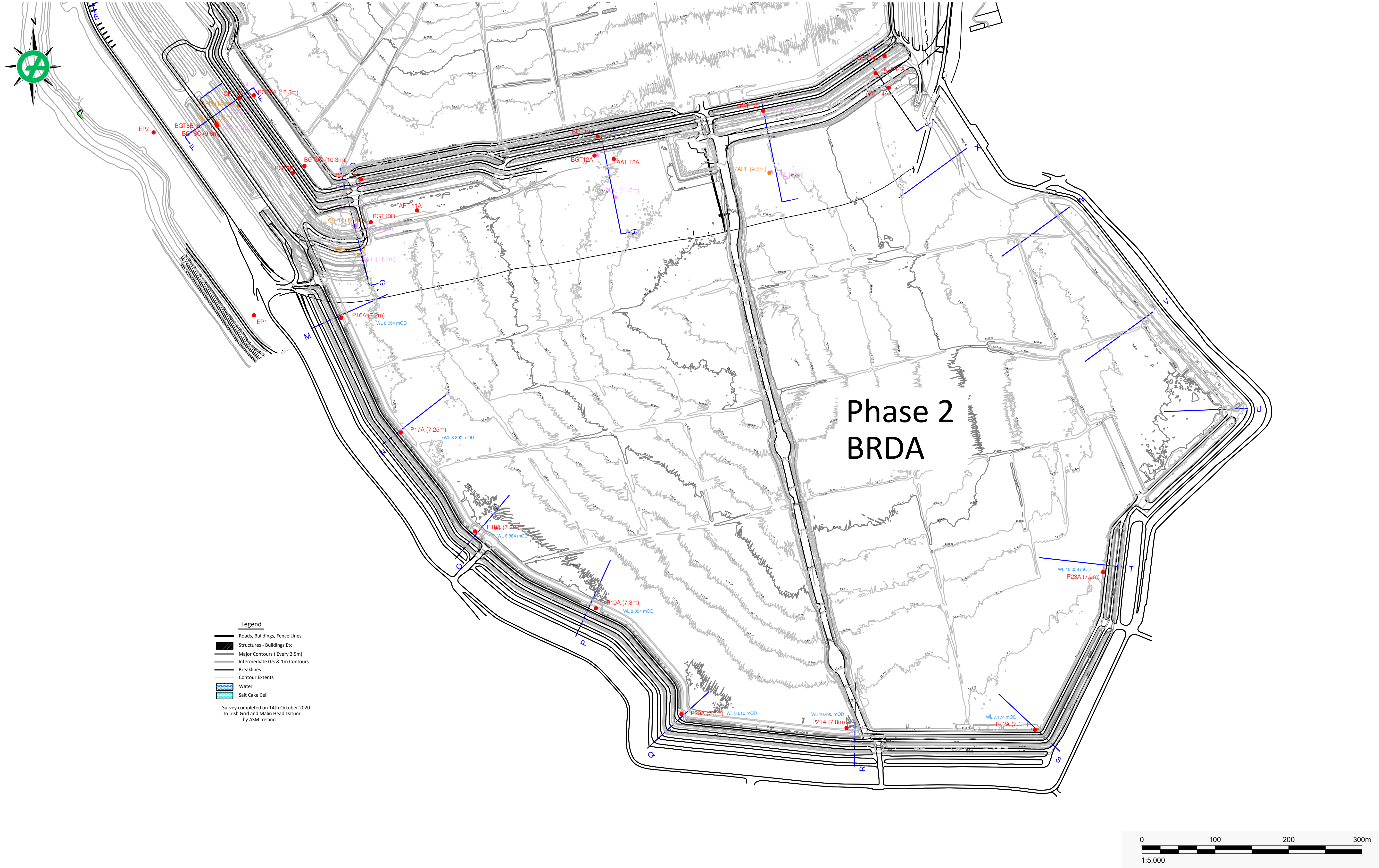

LEGEND:

- Abstraction Well
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- Inclinometer
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CLIENT

AUGHINISH ALUMINA LTD

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Oct-22
PREPARED	BK
DESIGN	POB
REVIEW	DC
APPROVED	BK

PROJECT

AUGHINISH BRDA 2021 MONITORING

TITLE

PHASE 2 BRDA
MONITORING INSTRUMENT LOCATION PLAN: Q3 2021

PROJECT No.	DRAWING No.	Rev.	SCALE
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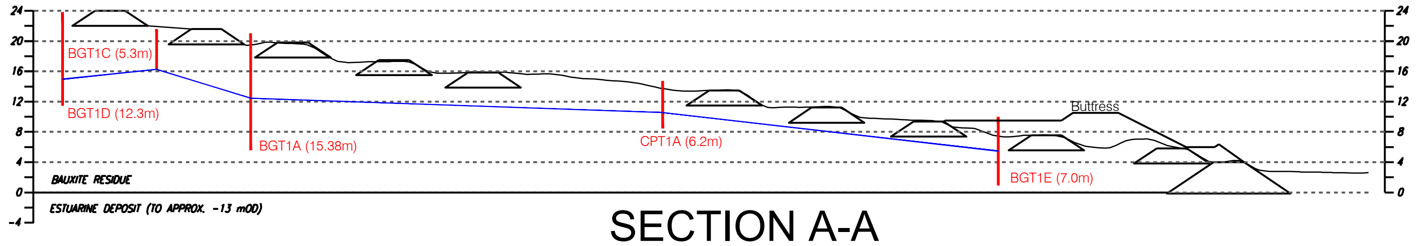

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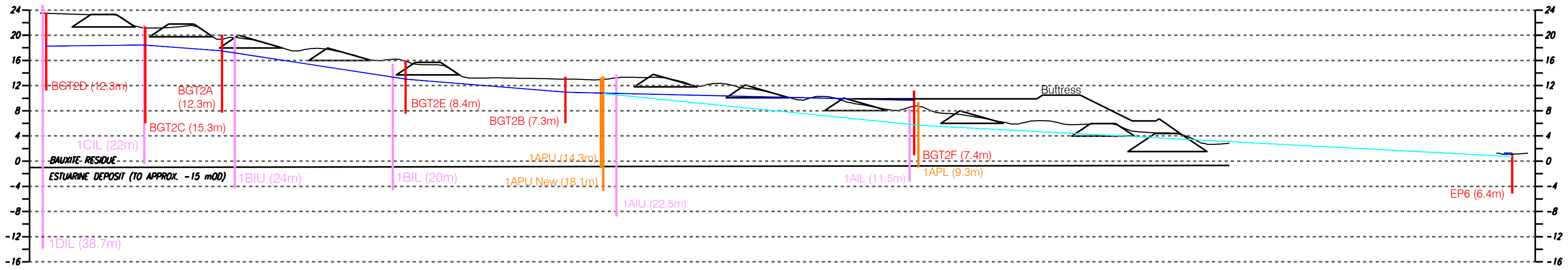

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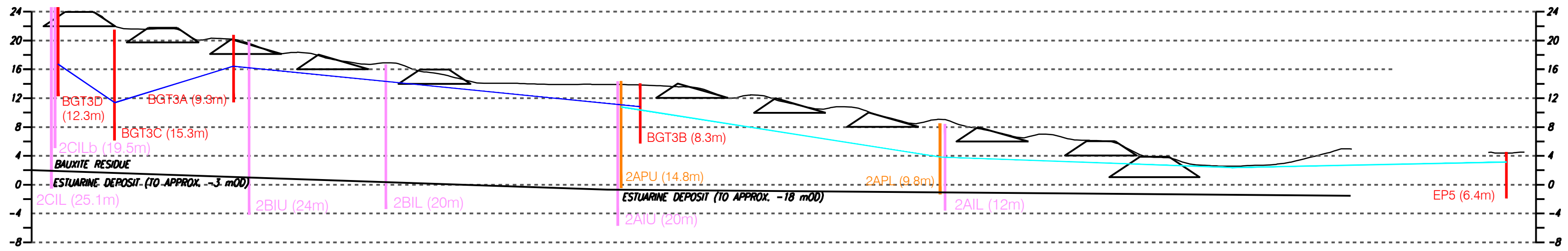

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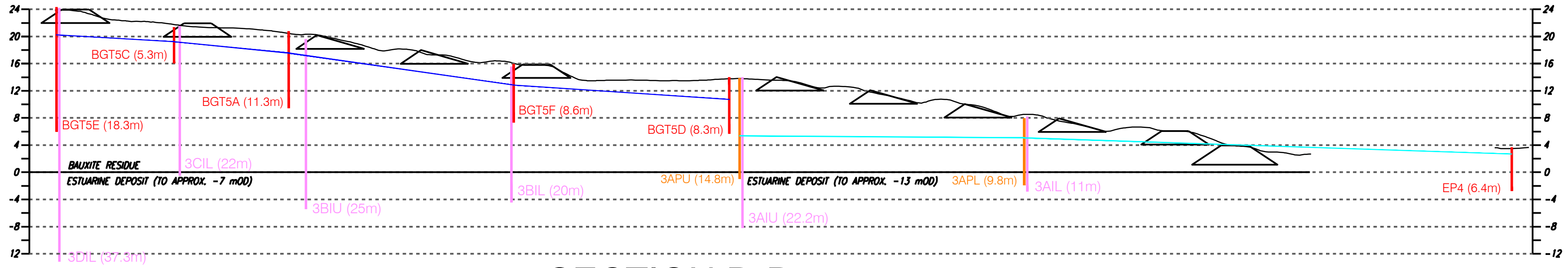

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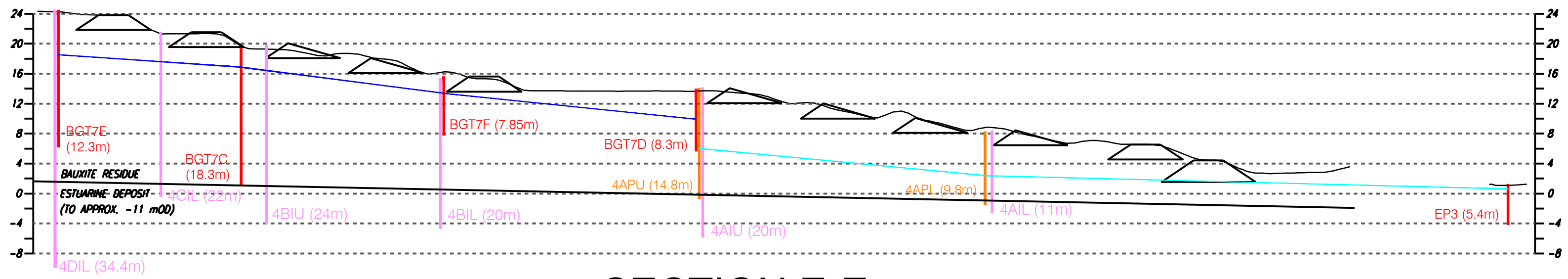

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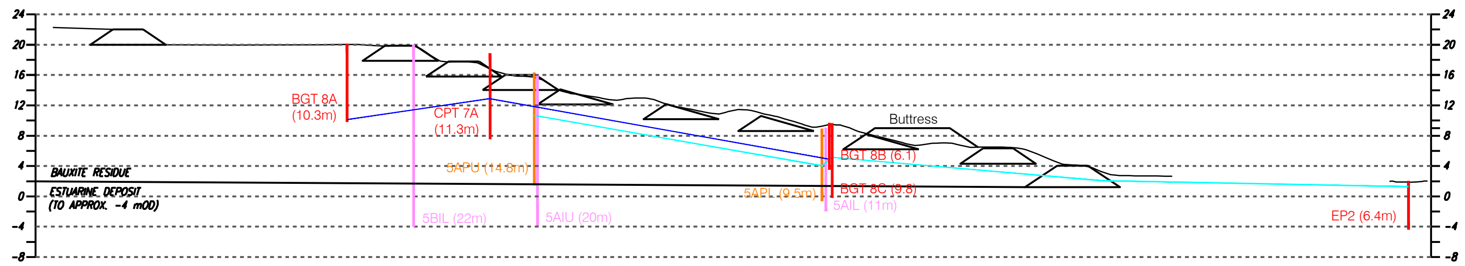


SECTION F-F

PIEZOMETRIC LINE IN ESTUARINE SOILS

PIEZOMETRIC LINE IN BAUXITE RESIDUE

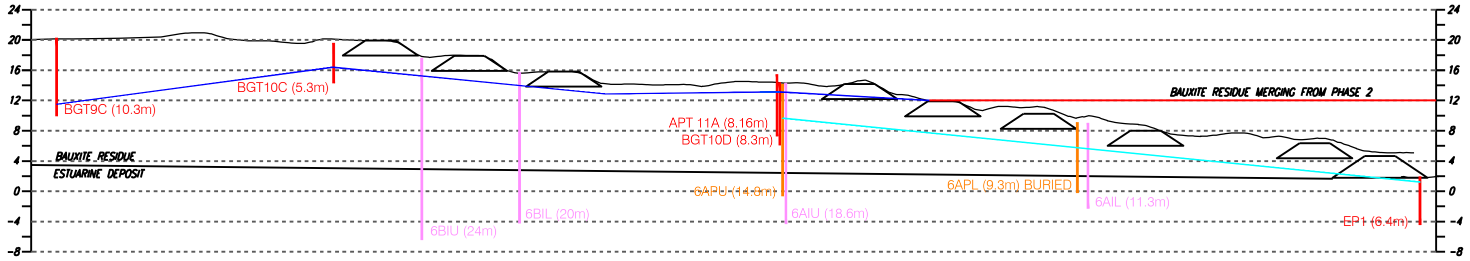
NOTE(S)

1. BRDA INSTRUMENT SECTIONS UPDATED WITH NEW INSTRUMENTS INSTALLED DURING JUNE 2021

0 20 40 60m

1:1,000

CLIENT		PROJECT	
AUGHINISH ALUMINA LTD		AUGHINISH BRDA 2020 MONITORING	
CONSULTANT	YYYY-MM-DD: 2021-Oct-22	TITLE	
GOLDER MEMBER OF WSP	PREPARED: POB	PHASE 1 BRDA INSTRUMENTATION SECTIONS Q3 2021 (1 of 2)	
	DESIGN: BK		
	REVIEW: DC		
	APPROVED: BK	PROJECT No. 21452853 / DRAWING No. 03 of 04 / Rev. A	SCALE 1:1,000 A3



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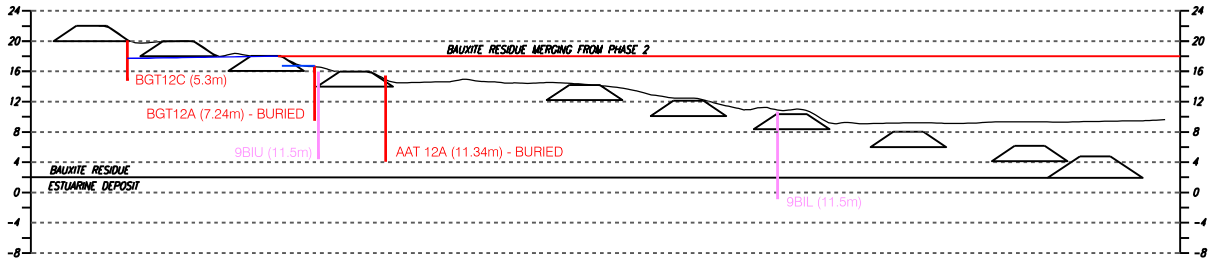

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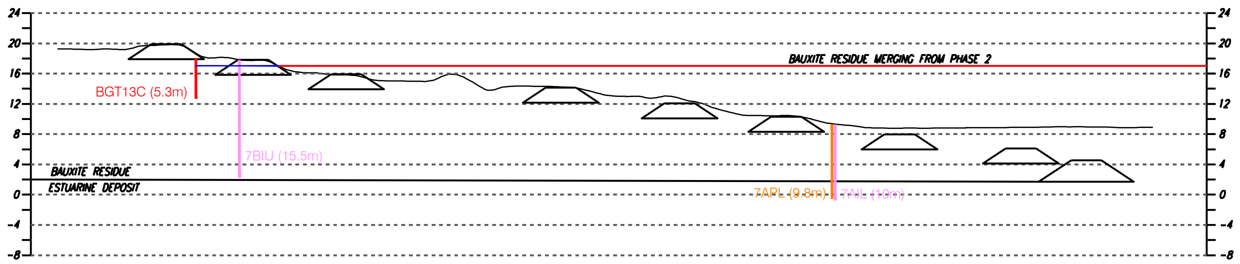

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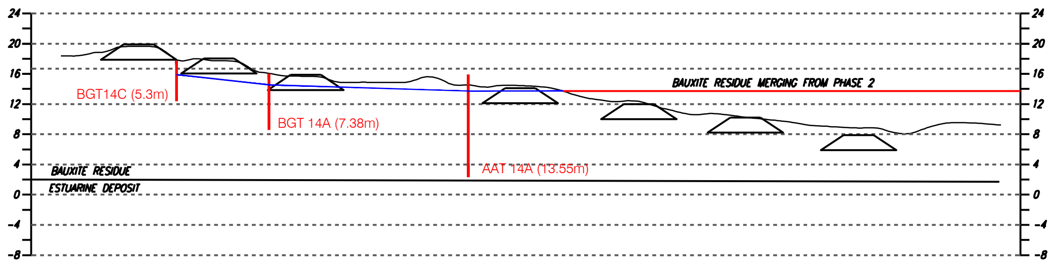

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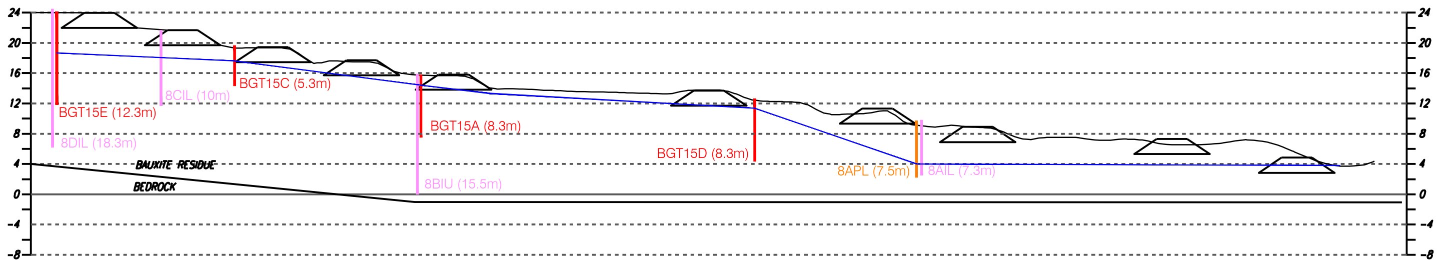


SECTION K-K

PIEZOMETRIC LINE IN ESTUARINE SOILS

PIEZOMETRIC LINE IN RED MUD

NOTE(S)

1. BRDA INSTRUMENT SECTIONS UPDATED WITH INSTRUMENTS INSTALLED DURING JUNE 2021

0 20 40 60m

1:1,000

CLIENT	AUGHINISH ALUMINA LTD	PROJECT	AUGHINISH BRDA 2020 MONITORING
CONSULTANT	GOLDER MEMBER OF WSP	YYYY-MM-DD	2021-Oct-22
		PREPARED	POB
		DESIGN	BK
		REVIEW	DC
		APPROVED	BK

TITLE
PHASE 1 BRDA INSTRUMENTATION SECTIONS Q3 2021 (2 of 2)

PROJECT No.	DRAWING No.	Rev.	SCALE
21452853	04 of 04	A	1:1,000 A3

APPENDIX B

Summary Sheets

Phase 1 BRDA - Aughinish Alumina Instrumentation Monitoring - Q3 Sept 2021

STANDPIPE PIEZOMETER MONITORING RECORD

	Instrument Identifier															
	BGT 1A	BGT1C	CPT 1A	BGT 2A	BGT 2B	BGT 2C	BGT 3A	BGT 3B	BGT 3C	BGT 5A	BGT 5B	BGT 5C	BGT 5D	BGT 6A	BGT 6D	CPT 7A
Elevation at Piezo Top (mOD)	20.980	21.540	14.190	20.475	14.310	21.910	20.765	14.020	21.460	20.780	19.105	21.370	13.990	19.620	13.930	18.860
Piezometer Depth (m)	15.380	5.300	5.680	11.280	5.190	15.300	9.000	8.300	5.380	11.300		5.300	8.300	9.280	8.300	11.320
Elevation at Piezo Tip (mOD)	5.600	16.240	8.510	9.195	9.120	6.610	11.765	5.720	16.080	9.480		16.070	5.690	10.340	5.630	7.540
2018-2020 Elevation Readings (mOD)																
11/12/2019	13.030	16.250	11.320	17.100	11.386	19.203	16.775	11.190	11.910	18.310	N/A	19.410	11.630	17.570	11.980	13.830
16/03/2020	12.650	16.370	11.050	17.920	11.386	19.013	16.835	11.260	11.910	18.310	N/A	19.340	11.780	17.660	12.160	14.070
08/06/2020	13.000	16.180	10.030	16.860	9.071	17.943	16.945	10.270	11.640	N/A	15.735	18.825	9.460	17.310	9.480	13.580
06/09/2020	12.605	16.240	10.910	17.690	11.340	17.993	16.725	11.090	11.865	N/A	15.145	19.260	11.450	17.490	12.020	13.940
14/12/2020	12.930	17.820	11.100	18.030	11.446	19.193	16.765	11.390	11.930	N/A	16.145	19.510	12.600	17.490	12.210	13.870
02/03/2021	12.407	16.174	10.441	17.231	11.180	18.484	16.500	10.931	11.458	N/A	16.061	19.262	11.375	17.502	11.931	13.886
14/06/2021	12.570	17.260	10.300	17.420	11.026	18.343	16.375	10.760	11.320	N/A	15.875	19.115	10.960	17.325	11.505	13.815
06/09/2021	12.460	16.250	10.050	17.510	10.946	18.453	16.445	10.810	11.380	N/A	15.895	19.240	10.740	17.280	11.320	12.880

	Instrument Identifier															
	BGT 7C	BGT 7D	BGT 9B	BGT 10C	BGT 10D	APT 11A	BGT 12A	AAT 12A	BGT 12C	BGT 13C	BGT 14A	AAT 14A	BGT 14C	BGT 15A	BGT 15C	BGT 15D
Elevation at Piezo Top (mOD)	19.430	13.960	16.940	19.600	14.400	15.450	16.750	15.435	20.100	18.000	16.700	15.880	17.700	16.665	19.640	12.650
Piezometer Depth (m)	18.300	8.300	9.340	5.300	8.300	8.160	7.240	11.340	5.300	5.300	7.295	13.540	5.300	7.090	5.300	8.300
Elevation at Piezo Tip (mOD)	1.130	5.660	7.600	14.300	6.100	7.290	9.510	4.095	14.800	12.700	9.405	2.340	12.400	9.575	14.340	4.350
2019-2020 Elevation Readings (mOD)																
11/12/2019	18.530	10.510	12.545	16.480	13.030	12.205	13.740	-	19.080	16.920	-	13.520	16.340	15.895	18.230	11.450
16/03/2020	17.750	10.450	12.660	16.780	13.110	12.530	13.740	-	18.880	17.570	15.720	13.320	16.140	15.835	17.930	11.490
08/06/2020	17.590	9.320	12.310	16.290	11.800	11.090	13.550	-	18.450	16.710	15.130	13.080	16.145	14.905	17.590	10.440
06/09/2020	18.235	10.430	12.620	16.680	12.970	12.560	14.120	-	17.970	17.140	15.140	13.370	16.510	15.525	17.680	11.440
14/12/2020	18.460	Blocked	12.500	16.670	13.060	12.470	N/A	N/A	18.290	16.980	15.580	13.520	16.190	15.885	17.740	11.500
02/03/2021	17.737	Blocked	12.393	16.228	12.732	12.333	N/A	N/A	17.481	16.779	15.005	14.188	16.076	15.618	17.577	11.398
14/06/2021	17.000	Blocked	12.160	16.430	12.920	12.525	N/A	N/A	17.550	17.040	15.230	14.230	16.230	15.505	17.585	11.385
06/09/2021	16.870	Blocked	12.000	16.390	13.140	12.480	N/A	N/A	17.730	17.060	15.250	13.720	15.900	15.275	17.630	11.360

	Instrument Identifier							
	EP1	EP2	EP3	EP4	EP5	EP6	EP7	EP8
Elevation at Piezo Top (mOD)	1.770	1.896	1.106	1.342	2.151	1.175	2.405	1.889
Piezometer Depth (m)	6.400	6.400	5.400	6.400	6.400	6.400	6.400	6.400
Elevation at Piezo Tip (mOD)	-4.630	-4.504	-4.294	-5.058	-4.249	-5.225	-3.995	-4.511
2019-2020 Elevation Readings (mOD)								
11/12/2019	1.150	1.356	0.466	1.022	1.031	0.735	0.435	1.189
16/03/2020	1.150	1.526	0.566	1.292	1.231	0.835	Flooded	Flooded
08/06/2020	0.610	0.876	0.496	0.462	0.791	0.405	1.645	Flooded
06/09/2020	0.930	1.386	0.526	1.142	1.051	0.755	1.235	1.499
14/12/2020	1.010	Flooded	0.456	No Access	1.111	0.715	0.835	1.439
02/03/2021	1.025	1.371	0.510	0.955	0.981	0.729	0.729	1.518
14/06/2021	1.040	1.141	0.441	No Access	No Access	0.588	0.625	1.519
06/09/2021	1.020	1.146	0.476	0.367	0.751	0.645	0.565	1.519

	Instrument Identifier																	
	BGT 1D	BGT 1E	BGT 2D	BGT 2E	BGT 2F	BGT 3D	BGT 5E	BGT 5F	BGT 6E	BGT 7E	BGT 7F	BGT 8A	BGT 8B	BGT 8C	BGT 9C	BGT 15E	PY	PZ
Elevation at Piezo Top (mOD)	24.790	7.944	24.820	15.952	8.368	24.600	24.300	15.935	24.260	24.110	15.619	20.130	9.660	9.630	20.250	24.150	31.737	31.573
Piezometer Depth (m)	13.300	7.000	13.800	8.400	7.400	12.300	18.300	8.600	12.300	12.300	7.850	10.300	6.110	9.830	10.300	12.300	10.000	18.000
Elevation at Piezo Tip (mOD)	11.490	0.944	11.020	7.552	0.968	12.300	6.000	7.335	11.960	11.810	7.769	9.830	3.550	-0.200	9.950	11.850	21.737	13.573
2019-2020 Elevation Readings (mOD)																		
11/12/2019	16.405		19.823			17.780	20.240		21.520	19.590		10.430			12.200	19.030		
16/03/2020	16.260		19.753			18.480	20.340		21.480	19.500		10.800			12.360	19.030		
08/06/2020	15.750		19.503			15.365	19.020		21.190	19.260		10.550			11.800	18.630		
06/09/2020	15.940		19.813			18.145	20.408		21.335	19.320		10.775			12.130	18.940		
14/12/2020	18.660	5.504	19.723	13.302	7.278	18.050	20.450	13.055	21.310	19.200	13.649	10.590	5.080	4.660	12.270	18.850		
02/03/2021	16.253	5.255	19.597	13.177	7.183	17.287	20.331	12.936	21.021	18.836	13.579	10.338	4.871	4.934	11.899	18.741		
14/06/2021	15.900	5.294	19.503	13.177	6.948	16.335	20.604	12.885	21.085	18.810	13.554	10.125	5.030	5.080	12.040	18.735		
06/09/2021	15.990	6.417	19.543	13.032	9.701	16.730	20.260	12.835	21.130	14.130	13.389	10.150	4.900	5.150	11.500	18.690	21.227	-

Phase 1 BRDA - Aughinish Alumina Instrumentation Monitoring - Q3 Sept 2021

CASAGRANDE PIEZOMETER MONITORING RECORD.

	Instrument Identifier												
	1APL	2APL	3APL	4APL	5APL	6APL	8APL	1APU	2APU	3APU	4APU	5APU	6APU
Elevation of top of piezo (mOD)	8.200	8.444	7.884	8.231	8.900	9.072	9.211	13.251	14.298	13.733	14.005	16.173	14.141
Depth of piezometer (m)	9.865	9.875	6.485	7.225	10.615	10.315	7.735	14.085	8.915	14.835	14.955	8.525	13.545
Elevation of piezo tip (mOD)	-1.633	-1.396	1.442	1.030	-1.691	-1.207	1.594	-0.810	5.433	-0.982	-0.910	4.760	0.625
2018-2020 Elevation Readings (mOD)													
01/06/2018	Blocked	4.034	4.297	2.390	3.854	3.268	3.289	10.415	10.778	5.353	6.211	11.115	9.710
05/09/2018	Blocked	3.869	4.327	2.245	3.794	3.198	3.384	10.695	10.288	5.353	5.960	11.080	8.955
06/12/2018	Blocked	3.929	4.472	2.460	3.794	3.538	3.769	11.250	10.283	5.353	6.225	11.100	9.640
08/03/2019	Blocked	4.099	4.417	2.865	3.974	3.878	3.509	11.275	10.398	5.353	6.615	11.190	9.890
18/06/2019	2.178	4.081	4.484	2.845	3.914	3.728	3.449	10.785	10.626	5.353	6.595	11.015	9.850
13/09/2019	2.248	4.049	4.517	2.250	3.914	3.548	3.469	10.980	10.748	5.353	6.185	11.015	9.460
11/12/2019	2.248	3.909	4.507	2.435	3.814	3.818	3.509	11.260	10.768	5.263	6.365	11.115	10.140
16/03/2020	2.428	4.149	4.607	2.495	4.164	3.888	3.639	11.295	10.878	5.333	6.495	11.345	10.140
08/06/2020	2.038	3.944	3.397	2.005	3.964	-	3.459	10.295	10.788	5.323	6.065	10.945	9.490
06/09/2020	2.195	3.889	3.677	2.445	4.254	-	3.549	10.996	10.818	5.323	6.185	11.205	9.700
14/12/2020	2.208	3.979	4.527	3.675	4.164	-	3.579	11.365	10.828	5.413	6.405	11.185	9.890
04/03/2021	2.048	3.823	4.763	5.634	4.206	-	3.615	11.005	10.859	5.283	6.258	11.195	9.827
14/06/2021	2.158	3.829	5.007	2.250	4.084	-	3.549	10.785	10.858	#VALUE!	6.155	10.915	9.690
06/09/2021	2.849	3.829	5.077	2.395	4.044	-	3.609	10.635	10.798	5.373	6.035	10.675	9.730
						Gone							

Phase 2 BRDA - Aughinish Alumina Instrumentation Monitoring - Q3 Sept 2021

STANDPIPE PIEZOMETER MONITORING RECORD

	Instrument Identifier							
	P16A	P17A	P18A	P19A	P20A	P21A	P22A	P23A
Elevation at Piezo Top (mOD)	10.684	10.430	10.584	10.604	10.590	12.585	12.584	12.788
Piezometer Depth (m)	7.200	7.250	7.200	7.300	7.500	7.780	7.070	7.880
Elevation at Piezo Tip (mOD)	3.484	3.180	3.384	3.304	3.090	4.805	5.514	4.908
2020 Elevation Readings (mOD)								
14/12/2020	8.204	9.160	8.934	8.814	8.650	-	-	-
02/03/2021	8.056	9.158	8.979	8.885	8.447	-	-	-
14/06/2021	8.024	9.080	9.064	8.959	8.715	-	-	-
06/09/2021	8.054	8.890	8.884	8.854	8.610	10.495	7.174	10.058

Phase 1 BRDA - Aughinish Alumina Instrumentation Monitoring - Q3 Sept 2021

EXTENSOMETERS (SPIDERS) MONITORING RECORD.

Date			08/06/2020		06/09/2020		14/12/2020		02/03/2021		14/06/2021		06/09/2021	
		Hole Depth	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2
1AIU	Datum	23.06	22.539	22.579	22.541	22.579	22.545	22.575	22.542	22.570	22.545	22.575	22.545	22.575
	Spider 1		21.401	21.441	21.404	21.435	21.410	21.440	21.401	21.433	21.405	21.435	21.405	21.435
	Spider 2		18.669	18.698	18.670	18.701	18.675	18.705	18.666	18.700	18.670	18.705	18.680	18.710
	Spider 3		15.632	15.668	15.634	15.664	15.640	15.670	15.634	15.665	15.640	15.670	15.640	15.670
	Spider 4		12.723	12.758	12.728	12.756	12.735	12.765	12.728	12.758	12.730	12.760	12.735	12.765
	Spider 5		9.792	9.823	9.795	9.826	9.800	9.830	9.795	9.825	9.800	9.830	9.805	9.835
	Spider 6		6.776	6.815	6.779	6.812	6.785	6.815	6.780	6.813	6.780	6.815	6.790	6.820
2AIU	Datum	19.900	19.276	19.317	19.280	19.315	19.280	19.310	19.278	19.312	19.280	19.310	19.280	19.310
	Spider 1		18.448	18.478	18.450	18.480	18.450	18.480	18.453	18.482	18.450	18.480	18.455	18.485
	Spider 2		15.303	15.337	15.305	15.337	15.305	15.340	15.309	15.337	15.305	15.340	15.305	15.335
	Spider 3		12.461	12.491	12.465	12.490	12.465	12.495	12.462	12.495	12.465	12.495	12.470	12.500
	Spider 4		9.936	9.963	9.940	9.965	9.950	9.975	9.954	9.980	9.950	9.980	9.965	9.995
	Spider 5		6.528	6.558	6.530	6.560	6.530	6.560	6.534	6.563	6.530	6.565	6.540	6.570
	Spider 6		2.056	2.089	2.060	2.090	2.060	2.090	2.063	2.094	2.060	2.090	2.070	2.100
3AIU	Datum	20.59	20.074	20.099	20.070	20.098	20.085	20.115	20.070	20.099	20.065	20.095	20.070	20.100
	Spider 1		18.866	18.896	18.862	18.896	18.875	18.905	18.860	18.893	18.860	18.895	18.860	18.895
	Spider 2		16.338	16.371	16.338	16.370	16.350	16.380	16.334	16.364	16.330	16.365	16.330	16.365
	Spider 3		13.447	13.485	13.445	13.485	13.470	13.500	13.454	13.485	13.450	13.485	13.450	13.485
	Spider 4		10.644	10.674	10.640	10.675	10.655	10.685	10.639	10.672	10.650	10.680	10.650	10.680
	Spider 5		7.819	7.850	7.815	7.850	7.835	7.865	7.816	7.845	7.815	7.845	7.820	7.850
	Spider 6		5.539	5.570	5.538	5.570	5.555	5.585	5.539	5.569	5.540	5.570	5.540	5.570
4AIU	Datum	18.92	18.520	18.556	18.525	18.555	18.525	18.555	18.525	18.556	18.525	18.555	18.549	18.579
	Spider 1		17.239	17.268	17.248	17.275	17.250	17.280	17.244	17.276	17.245	17.275	17.240	17.270
	Spider 2		14.663	14.695	14.675	14.705	14.675	14.705	14.671	14.701	16.670	16.700	14.666	14.696
	Spider 3		11.603	11.633	11.612	11.645	11.615	11.645	11.613	11.640	11.615	11.640	11.606	11.636
	Spider 4		8.552	8.582	8.560	8.590	8.565	8.595	8.560	8.590	8.560	8.590	8.557	8.587
	Spider 5		6.171	6.201	6.180	6.215	6.185	6.215	6.180	6.210	6.180	6.210	6.174	6.204
	Spider 6		2.845	2.875	2.855	2.880	2.860	2.890	2.852	2.886	2.855	2.885	2.852	2.882
5AIU	Datum	19.87	21.936	21.969	21.936	21.969	21.940	21.970	21.933	21.964	21.940	21.970	21.935	21.965
	Spider 1		20.682	20.711	20.681	20.711	20.680	20.710	20.678	20.707	20.680	20.710	20.680	20.710
	Spider 2		18.020	18.050	18.020	18.050	18.025	18.055	18.017	18.045	18.020	18.050	18.020	18.050
	Spider 3		15.114	15.148	15.115	15.150	15.120	15.150	15.111	15.145	15.120	15.145	15.120	15.150
	Spider 4		12.285	12.317	12.285	12.317	12.290	12.320	12.283	12.316	12.285	12.320	12.285	12.320
	Spider 5		9.340	9.371	9.340	9.370	9.345	9.375	9.339	9.369	9.340	9.375	9.340	9.370
	Spider 6		7.202	7.234	7.202	7.235	7.205	7.235	7.202	7.233	7.205	7.235	7.205	7.235
6AIU	Datum	18.31	17.850	17.890	17.850	17.890	17.850	17.890	17.853	17.893	17.850	17.890	17.860	17.890
	Spider 1		17.126	17.157	17.125	17.155	17.125	17.155	17.130	17.160	17.125	17.155	17.130	17.160
	Spider 2		13.851	13.883	13.845	13.875	13.850	13.880	13.853	13.885	13.850	13.880	13.855	13.885
	Spider 3		10.861	10.902	10.865	10.900	10.865	10.900	10.872	10.903	10.865	10.905	10.875	10.905
	Spider 4		7.874	7.904	7.870	7.900	7.875	7.905	7.876	7.906	7.870	7.905	7.880	7.910
	Spider 5		4.872	4.902	4.875	4.900	4.875	4.905	4.878	4.909	4.875	4.905	4.880	4.910
	Spider 6		3.205	3.243	3.210	3.240	3.210	3.240	3.211	3.242	3.210	3.240	3.215	3.245

Date			08/06/2020		06/09/2020		14/12/2020		02/03/2021		14/06/2021		06/09/2021	
		Hole Depth	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2
1DIL	Datum	38.7	37.120	37.150	37.124	37.155	37.125	37.155	37.120	37.150	37.115	37.145	36.660	36.690
	Spider 1		22.160	22.190	22.162	22.193	22.160	22.190	22.156	22.187	22.155	22.185	21.700	21.730
	Spider 2		16.271	16.301	16.273	16.304	16.275	16.305	16.269	16.301	16.269	16.300	15.810	15.840
	Spider 3		10.224	10.255	10.225	10.256	10.225	10.260	10.222	10.256	10.224	10.254	9.770	9.800
	Spider 4		2.998	3.028	3.000	3.030	3.000	3.030	3.001	3.030	3.001	3.031	2.540	2.570
2CIL	Datum	25.1	23.570	23.600	23.568	23.598	23.570	23.600	23.568	23.601	23.571	23.601	23.570	23.600
	Spider 1		16.343	16.372	16.344	16.375	16.345	16.375	16.348	16.376	16.345	16.375	16.350	16.380
	Spider 2		10.339	10.368	10.343	10.375	10.340	10.370	10.345	10.375	10.365	10.395	10.345	10.375
	Spider 3		4.545	4.575	4.548	4.575	4.550	4.575	4.552	4.580	4.550	4.580	4.555	4.585
2CILB	Datum	19.5	18.022	18.052	18.017	18.048	18.015	18.045	18.016	18.046	18.014	18.044	18.020	18.050
	Spider 1		11.995	12.028	11.995	12.021	11.995	12.025	11.997	12.026	11.995	12.025	12.000	12.030
	Spider 2		4.828	4.858	4.824	4.856	4.825	4.855	4.824	4.855	4.826	4.856	4.835	4.865
3DIL	Datum	37.3	35.714	35.745	35.705	35.736	35.710	35.740	35.740	35.775	35.700	35.730	35.700	35.730
	Spider 1		20.815	20.842	20.811	20.837	20.810	20.840	20.820	20.844	20.814	20.844	20.810	20.840
	Spider 2		14.527	14.562	14.525	14.557	14.530	14.560	14.536	14.565	14.535	14.565	14.530	14.560
	Spider 3		8.815	8.845	8.811	8.841	8.820	8.850	8.826	8.855	8.826	8.856	8.820	8.850
	Spider 4		2.895	2.925	2.893	2.922	2.900	2.930	2.901	2.933	2.902	2.932	2.900	2.930
4DIL	Datum	34.4	31.125	31.164	31.122	31.152	31.120	31.150	31.115	31.148	31.122	31.152	31.115	31.145
	Spider 1		21.478	21.514	21.481	21.512	21.480	21.510	21.476	21.565	21.482	21.512	21.480	21.510
	Spider 2		14.204	14.238	14.206	14.240	14.210	14.240	14.208	14.242	14.215	14.245	14.215	14.245
	Spider 3		8.022	8.052	8.021	8.056	8.025	8.055	8.025	8.057	8.035	8.065	8.035	8.065
	Spider 4		3.118	3.149	3.118	3.151	3.125	3.155	3.126	3.157	3.133	3.163	3.135	3.165
8DIL	Datum	18.3	16.778	16.812	16.770	16.810	16.775	16.805	16.774	16.806	16.755	16.805	16.780	16.810
	Spider 1		13.826	13.852	13.824	13.855	13.825	13.855	13.823	13.850	13.825	13.855	13.825	13.855
	Spider 2		7.832	7.862	7.831	7.863	7.830	7.860	7.833	7.860	7.830	7.860	7.840	7.870
	Spider 3		3.542	3.572	3.542	3.573	3.545	3.575	3.543	3.570	3.545	3.575	3.550	3.580

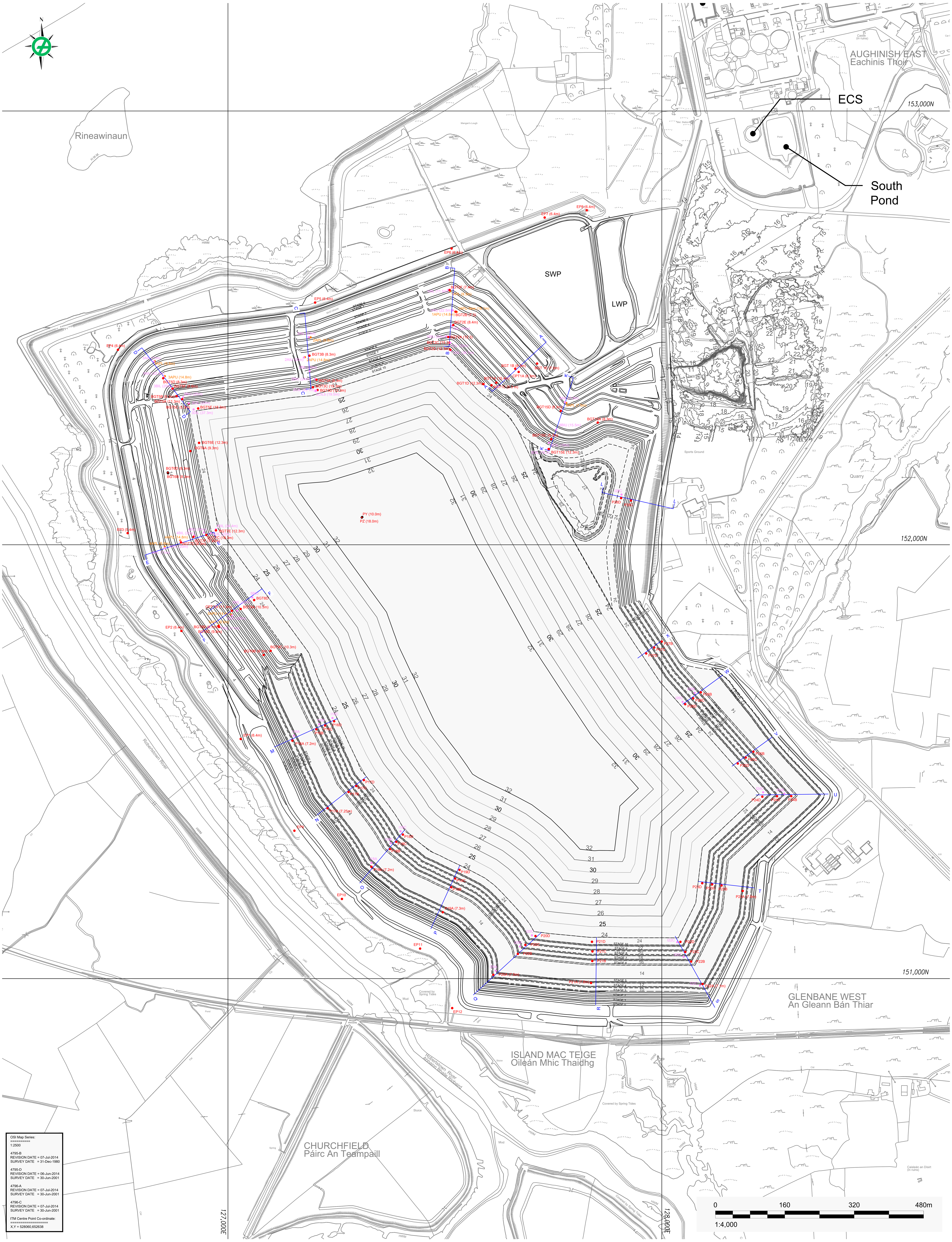
2.814m increase in top of pipe

Date			08/06/2020		06/09/2020		14/12/2020		02/03/2021		14/06/2021		06/09/2021	
		Hole Depth	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2
1AIL	Datum	11.77	11.330	11.365	11.330	11.362	11.335	11.365	11.331	11.360	11.330	11.365	14.145	14.170
	Spider 1		10.970	11.000	10.965	10.998	10.970	11.000	10.962	10.995	10.970	10.995	13.895	13.925
	Spider 2		7.380	7.410	7.378	7.405	7.385	7.415	7.378	7.408	7.380	7.405	10.340	10.370
	Spider 3		4.495	4.524	4.492	4.522	4.505	4.535	4.498	4.525	4.500	4.525	7.495	7.525
2AIL	Datum	10.78	10.256	10.285	10.255	10.285	10.260	10.290	10.253	10.286	10.255	10.285	10.255	10.285
	Spider 1		9.226	9.256	9.230	9.260	9.230	9.260	9.225	9.257	9.225	9.260	9.230	9.260
	Spider 2		6.512	6.542	6.512	6.545	6.515	6.545	6.511	6.538	6.510	6.540	6.515	6.545
	Spider 3		3.202	3.237	3.203	3.240	3.205	3.235	3.205	3.234	3.205	3.235	3.210	3.240
3AIL	Datum	10.43	9.979	10.010	9.980	10.010	9.975	10.005	9.975	10.010	9.975	10.010	9.975	10.010
	Spider 1		9.516	9.545	9.520	9.550	9.515	9.545	9.516	9.545	9.515	9.545	9.515	9.545
	Spider 2		7.375	7.405	7.375	7.405	7.375	7.405	7.375	7.405	7.375	7.405	7.385	7.415
	Spider 3		4.218	4.245	4.220	4.245	4.215	4.245	4.220	4.251	4.220	4.250	4.220	4.250
4AIL	Datum	10.96	10.510	10.535	10.510	10.540	10.510	10.540	10.495	10.525	10.510	10.540	10.515	10.540
	Spider 1		9.619	9.649	9.615	9.645	9.620	9.650	9.600	9.630	9.615	9.650	9.625	9.655
	Spider 2		6.635	6.663	6.630	6.655	6.635	6.665	6.615	6.645	6.635	6.665	6.635	6.665
	Spider 3		3.579	3.609	3.575	3.605	3.580	3.610	3.561	3.589	3.575	3.610	3.590	3.620
5AIL	Datum	10.82	10.070	10.110	10.075	10.110	10.080	10.110	10.079	10.109	10.070	10.110	10.085	10.115
	Spider 1		9.073	9.105	9.075	9.105	9.080	9.110	9.076	9.107	9.080	9.110	9.080	9.110
	Spider 2		5.954	5.986	5.955	5.985	5.960	5.990	5.957	5.987	5.960	5.990	5.960	5.990
	Spider 3		3.011	3.040	3.010	3.040	3.015	3.045	3.012	3.041	3.015	3.045	3.020	3.050
6AIL	Datum	11.59												
	Spider 1		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger	
	Spider 2													
	Spider 3													
7AIL	Datum	9.73												
	Spider 1		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger		Buried by Red Mud at Phase 1 to 2 Merger	
	Spider 2													
	Spider 3													
8AIL	Datum	7.31	6.870	6.905	6.875	6.905	6.875	6.905	6.875	6.906	6.875	6.905	6.880	6.910
	Spider 1		5.692	5.722	5.695	5.725	5.695	5.725	5.693	5.722	5.695	5.725	5.700	5.730
	Spider 2		3.054	3.084	3.060	3.090	3.060	3.090	3.058	3.090	3.060	3.090	3.065	3.095

Date			08/06/2020		06/09/2020		14/12/2020		02/03/2021		14/06/2021		06/09/2021	
		Hole Depth	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2	Read 1	Read 2
1BIU	Datum		24.230	24.255	24.225	24.257	24.220	24.250	24.219	24.246	24.220	24.250	24.220	24.250
	Spider 1		17.825	17.856	17.827	17.858	17.820	17.850	17.822	17.850	17.820	17.850	17.820	17.850
	Spider 2		11.637	11.682	11.638	11.666	11.630	11.660	11.633	11.662	11.630	11.660	11.630	11.660
	Spider 3		4.635	4.666	4.639	4.669	4.630	4.660	4.635	4.665	4.630	4.665	4.640	4.670
2BIU	Datum		22.989	23.020	22.980	23.011	22.985	23.015	22.982	23.014	22.985	23.015	22.985	23.015
	Spider 1		17.120	17.151	17.117	17.146	17.120	17.150	17.119	17.145	17.820	17.850	17.120	17.150
	Spider 2		10.886	10.917	10.882	10.914	10.885	10.915	10.885	10.912	10.885	10.915	10.890	10.920
	Spider 3		4.956	4.986	4.951	4.980	4.950	4.980	4.952	4.980	4.950	4.980	4.955	4.985

APPENDIX C

Instrumentation Plan (July 2021)



OSI Map Series:
==========
1:2500

4795-B
REVISION DATE = 07-Jul-2014
SURVEY DATE = 31-Dec-1980

4795-D
REVISION DATE = 06-Jun-2014
SURVEY DATE = 30-Jun-2001

4796-A
REVISION DATE = 07-Jul-2014
SURVEY DATE = 30-Jun-2001

4796-C
REVISION DATE = 07-Jul-2014
SURVEY DATE = 30-Jun-2001

ITM Centre Point Co-ordinate:
===================
X,Y = 528060,652638

LEGEND:

- Standpipe Piezometer
- Inclinometer
- Casagrande Piezometer

NOTES:

GRID REFERENCES ARE IN METRES & TO IRISH NATIONAL GRID.

LEVELS ARE IN METRES & TO O.S. DATUM.

DIMENSIONS ARE IN METRES.

CLIENT

AUGHINISH ALUMINA LTD.

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2021-Jul-07
PREPARED	POB
DESIGN	BK
REVIEW	BK
APPROVED	GJ

PROJECT

PHYSICAL STABILITY MONITORING PLAN - JULY 2021

TITLE

INSTRUMENTATION PLAN

PROJECT No.	DRAWING No.	Rev.	SCALE
21452853	05	A	1:4,000 A1

25 mm | IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

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