REPORT

July 2017

AUGHINISH ALUMINA LIMITED

Borrow Pit: Phase 1 BRDA Blast Vibration Assessment

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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.

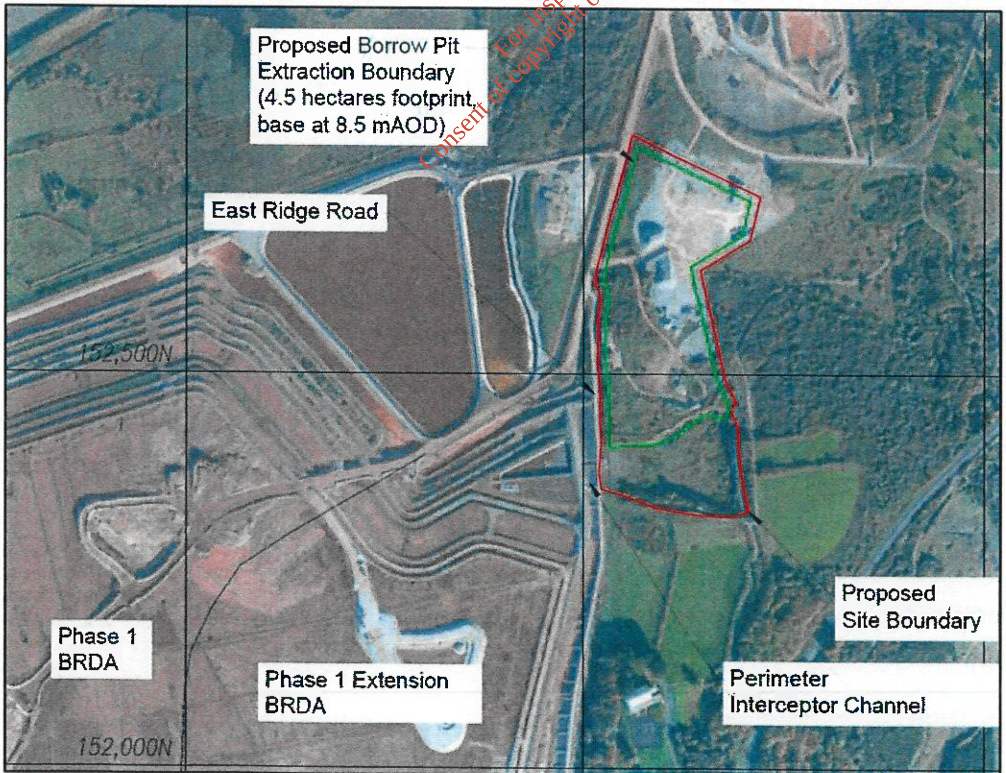

Figure 1: Location Map

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:

$$PPV = k\left(\frac{D}{\sqrt{W}}\right)^{-b}$$ **(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.

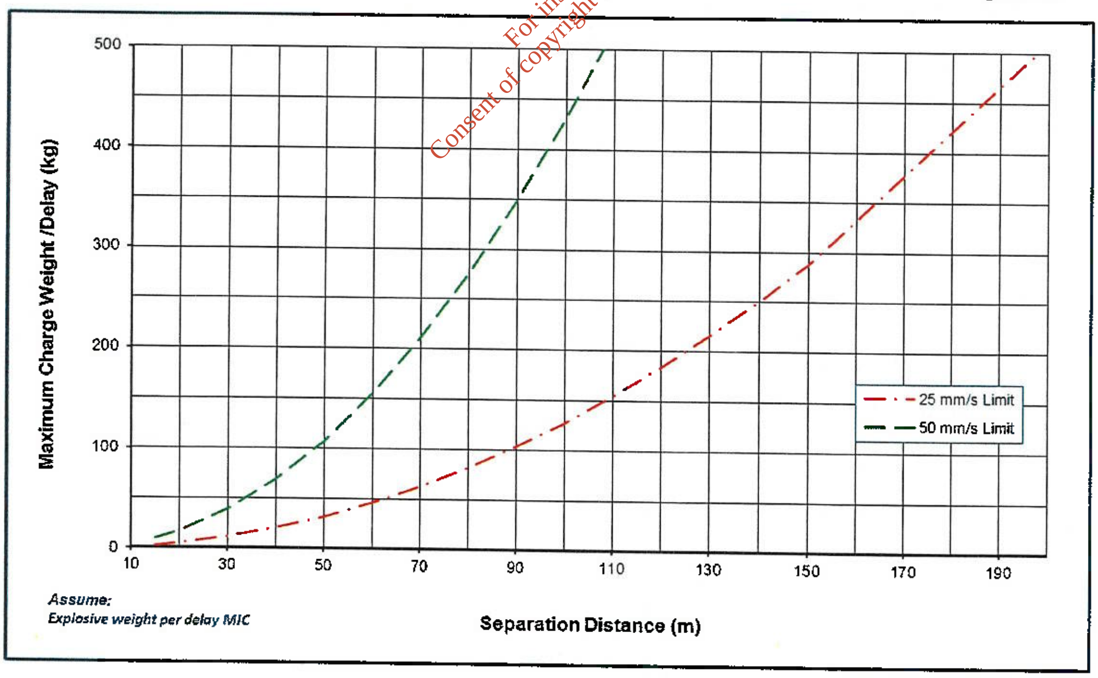

Figure 2: Maximum Explosive Charge Weight for a Range of Set-back Distances for the Assumed Attenuation Model

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

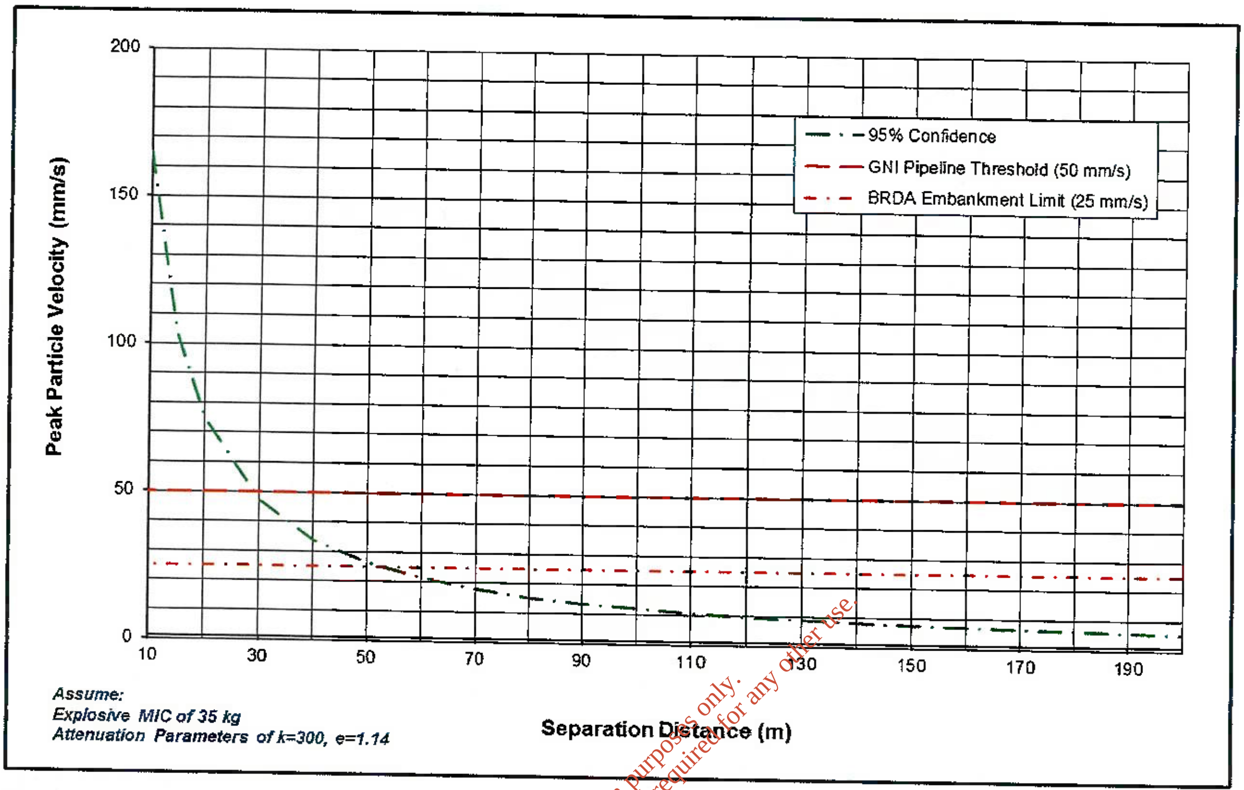

Figure 3: Estimated PPV for a range of set-back distances

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

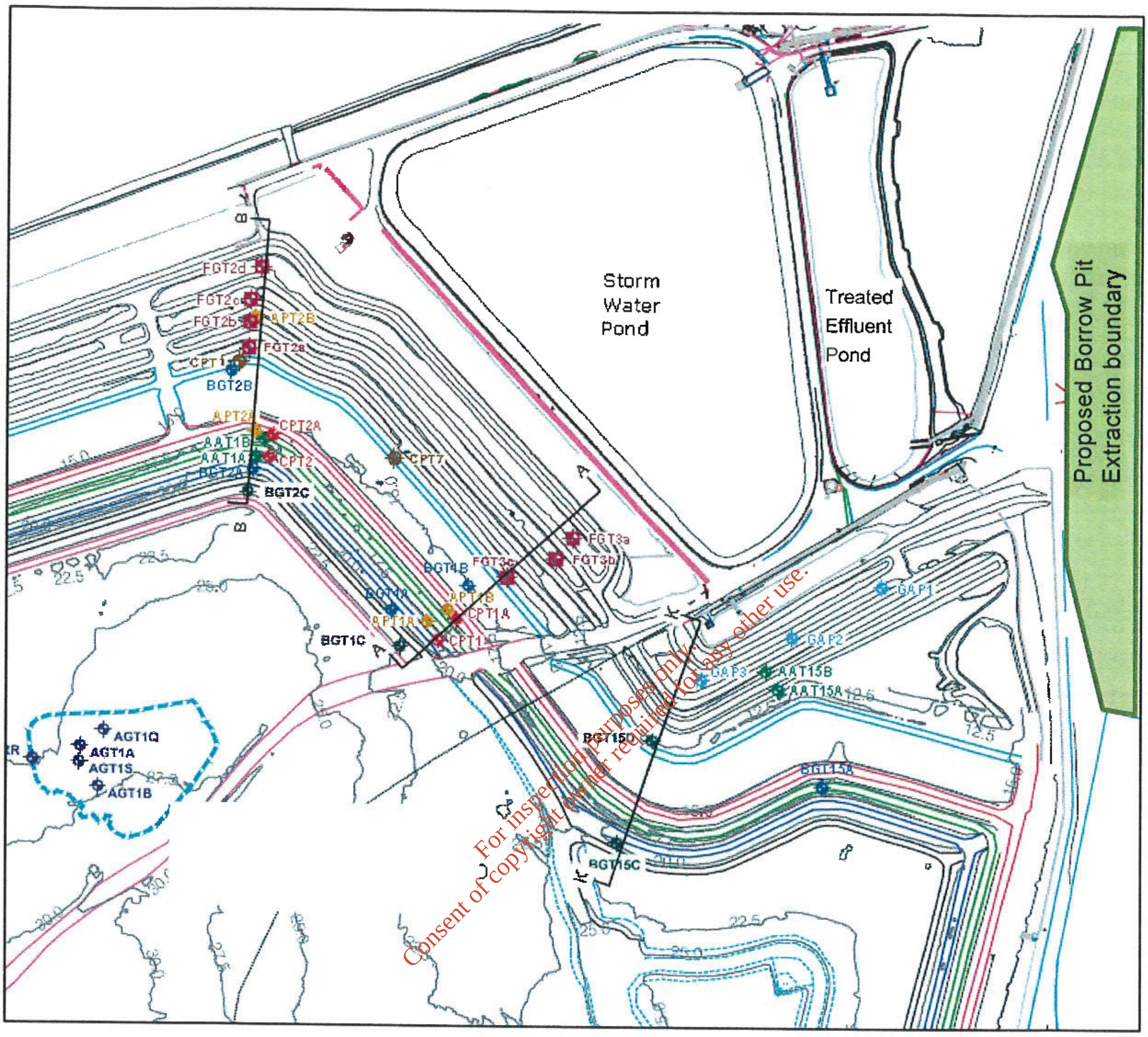

Figure 4: Location of Stability Analysis Section relative to the Proposed Borrow Pit

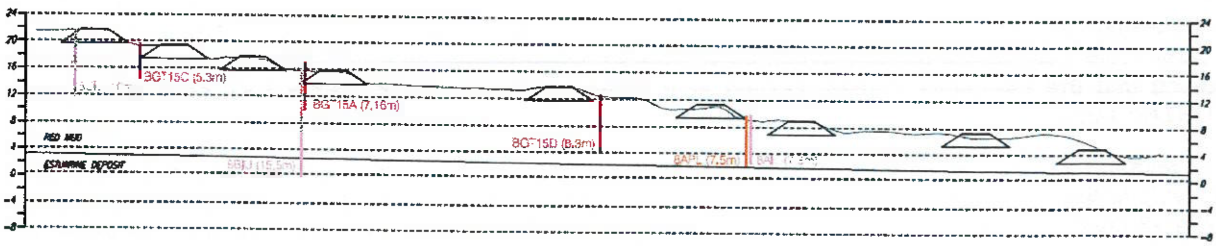

Figure 5: Section K-K

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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/σ'_v = 0.6 minimum of 60 kPa	Based on CPT analysis
Red Mud	22	s_u/σ'_v = 0.23 minimum of 25 kPa	Sensitivity analysis where s_u/σ'_v = 0.15 along the toe of the east slope
Estuarine Deposits	19	Φ = 30°, c=0 kPa	Sensitivity analysis where s_u/σ'_v = 0.23
Stage Raise Rock fill	22	Φ = 45°, 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:

$$F = W \times K$$

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:

$$K = K_a \left(\frac{PPA}{g}\right) \quad \text{(Wong and Pang 1992)}$$

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).

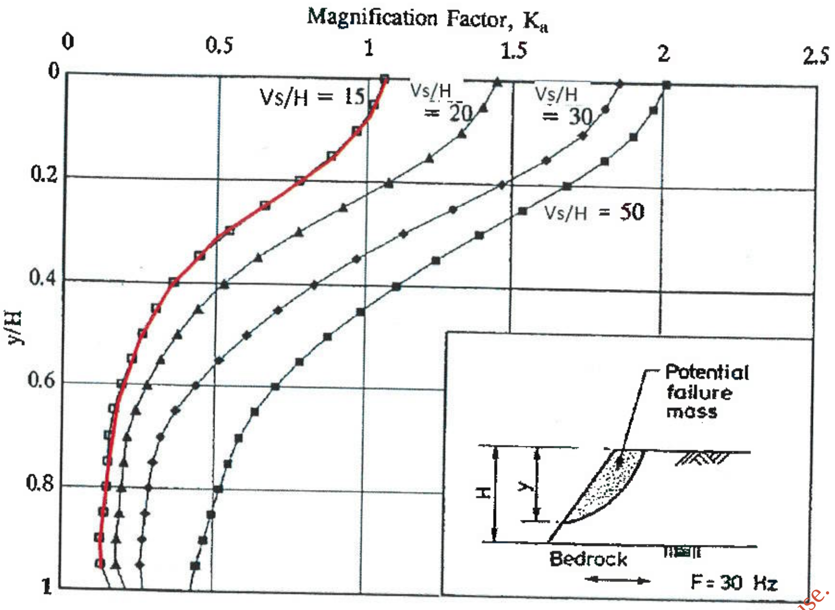

Figure 6: Magnification Factor vs Depth of Sliding Mass (from Wong and Pang 1992)

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.

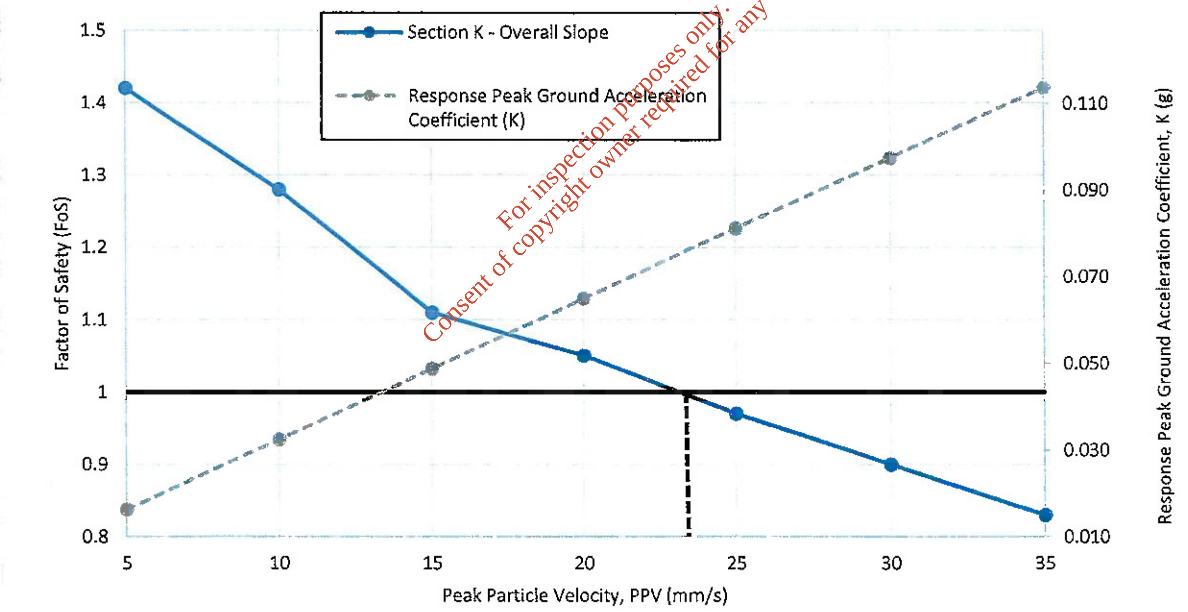

Figure 7: Pseudo-Static Stability Analysis Results, Overall Slope Slip Surface

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.

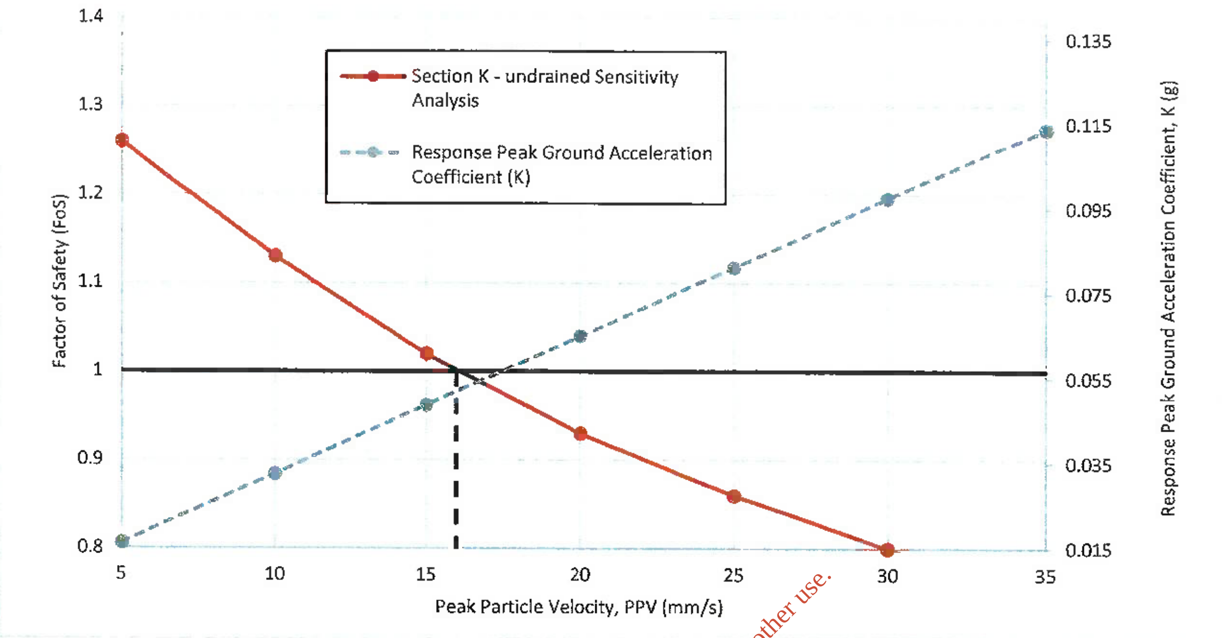

Figure 8: Sensitivity Pseudo-static Stability Analysis, Reduced Red Mud Strength along the Toe

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:

$$PPV = 12.9\left(\frac{R}{M^{1/3}}\right)^{-2.21}$$ **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:

$$PPR = \frac{\Delta u_{res}}{\sigma'_0} = 6.67\, PPV^{0.33}\, {\sigma'_0}^{-0.31}\, D_r^{-0.179}$$ **(Veyera 1985)**

$$PPR = \frac{\Delta u_{res}}{\sigma'_0} = 10.59\left(\frac{100PPV}{Vs}\right)^{0.43} {\sigma'_0}^{-0.17}\, D_r^{-0.18}$$ **(Hubert 1986)**

$$PPR = \frac{\Delta u_{res}}{\sigma'_0} = 16\left(\frac{100PPV}{Vs}\right)^{0.33} {\sigma'_0}^{-0.31}\, D_r^{-0.179}$$ **(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.

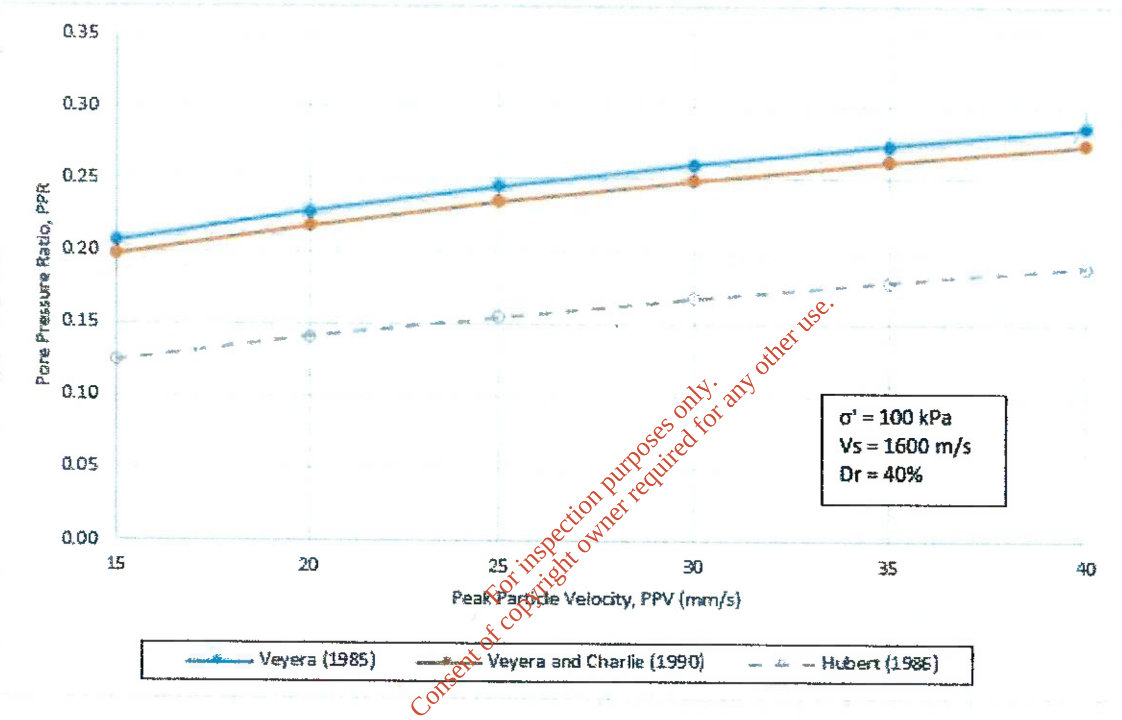

Figure 9: Empirical Relationships of Peak Particle Velocity (PPV) versus Pore Pressure Ratio (PPR)

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%.**

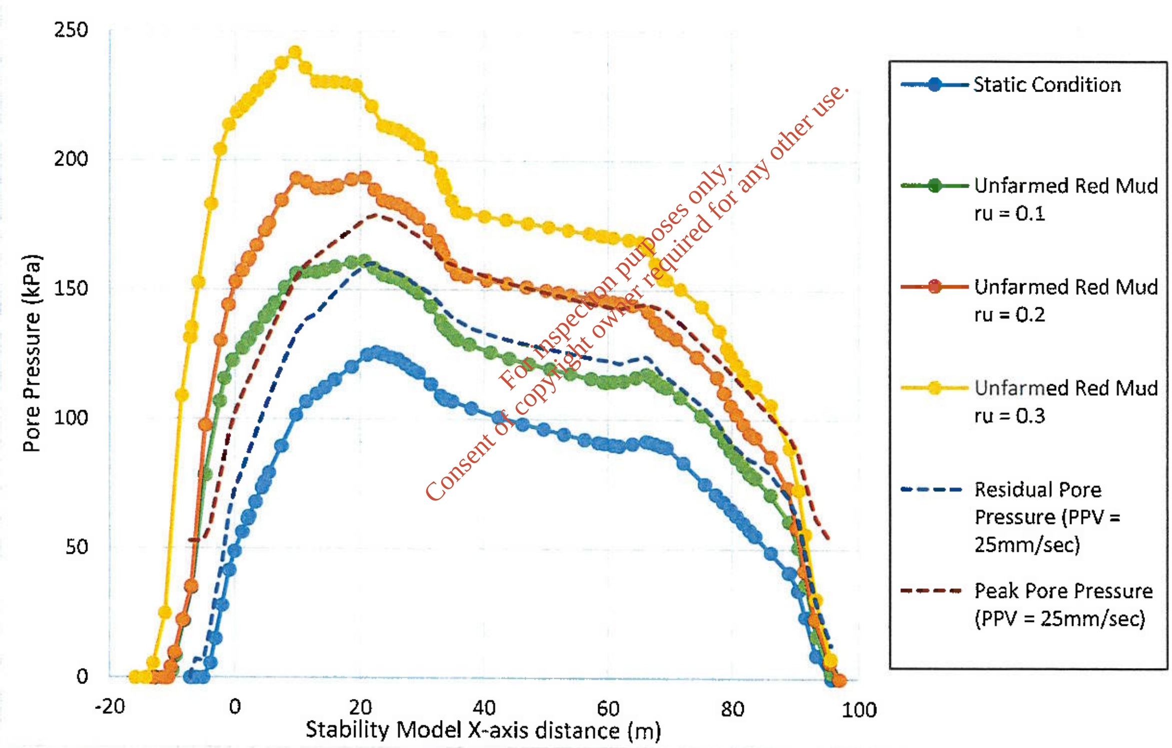

Figure 10: Pore Pressure Generated along the Slip Surface for Various PPR

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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Gerd Janssens
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Roger White
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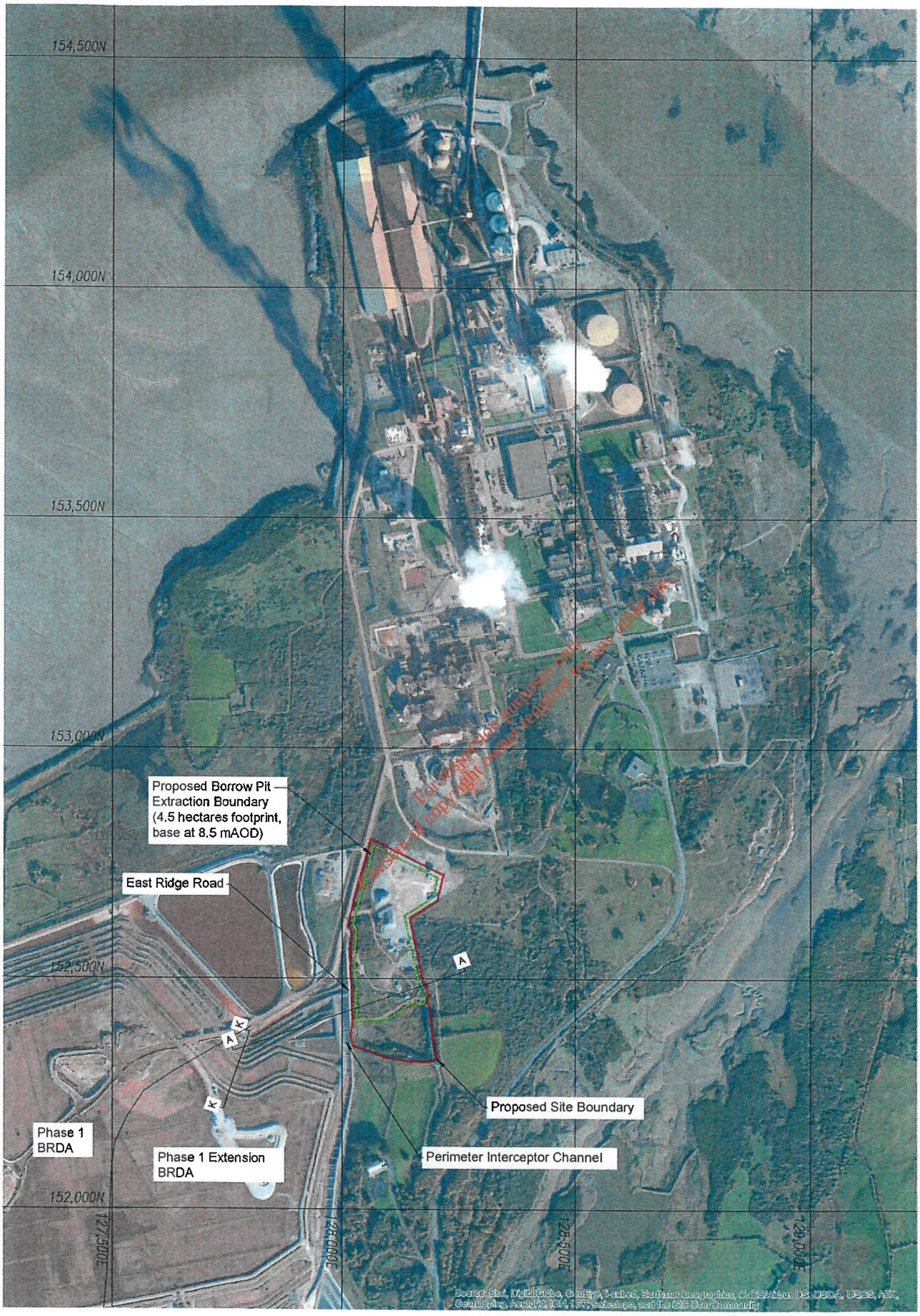
Directors: S. Copping, A. Harris (British), DRV Jones
VAT No.: 8297875W

APPENDIX A

Drawings



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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

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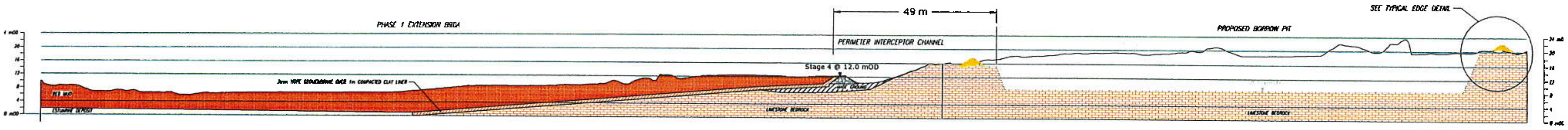

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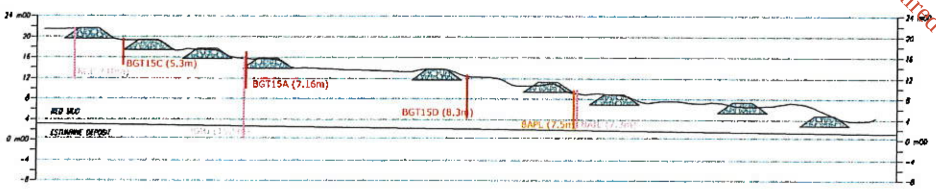

SECTION K-K

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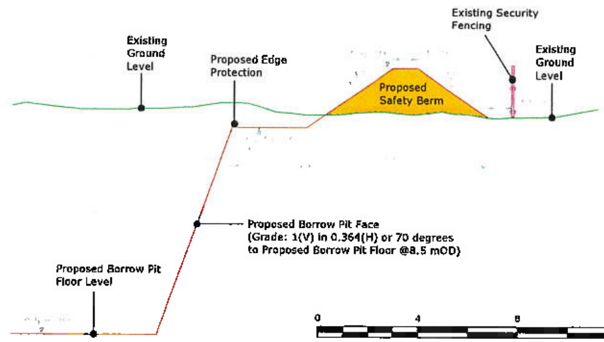
Typical Edge Detail



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- EXISTING GROUND CONTOUR (mAOD)
- PROPOSED BORROW PIT DESIGN CONTOUR (mAOD)

NOTES:

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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.	
CONSULTANT	Golder Associates	
	YYYY-MM-DD	2017-May-03
	PREPARED	POB
	DESIGN	POB
	REVIEW	BB
	APPROVED	BB

PROJECT	BORROW PIT DEVELOPMENT		
TITLE	CROSS SECTIONS		
PROJECT No. 1667376	DRAWING No. 02	Rev. A	SCALE As shown 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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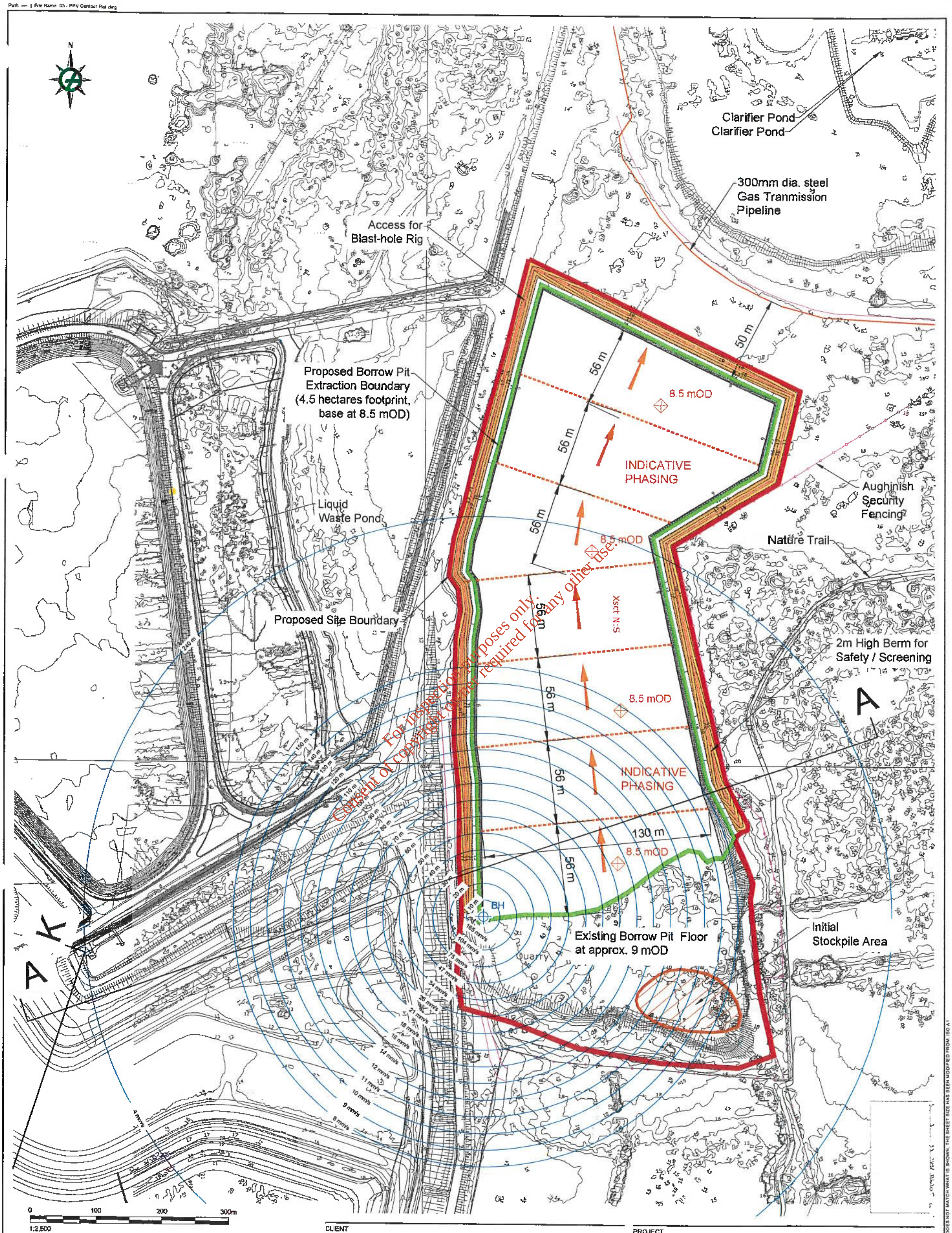

LEGEND

- APPLICATION EXTRACTION BOUNDARY
- APPLICATION SITE BOUNDARY
- 35 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.	PROJECT	BORROW PIT DEVELOPMENT
CONSULTANT	Golder Associates	TITLE	PPV CONTOUR PLOT FOR W = 35 kg
YYYY-MM-DD	2017-Jun-19	PROJECT No.	1667375
PREPARED	POB	DRAWING No.	03
DESIGN	POB	Rev.	A
REVIEW	BB	SCALE	1:1,000 A1
APPROVED	BB		

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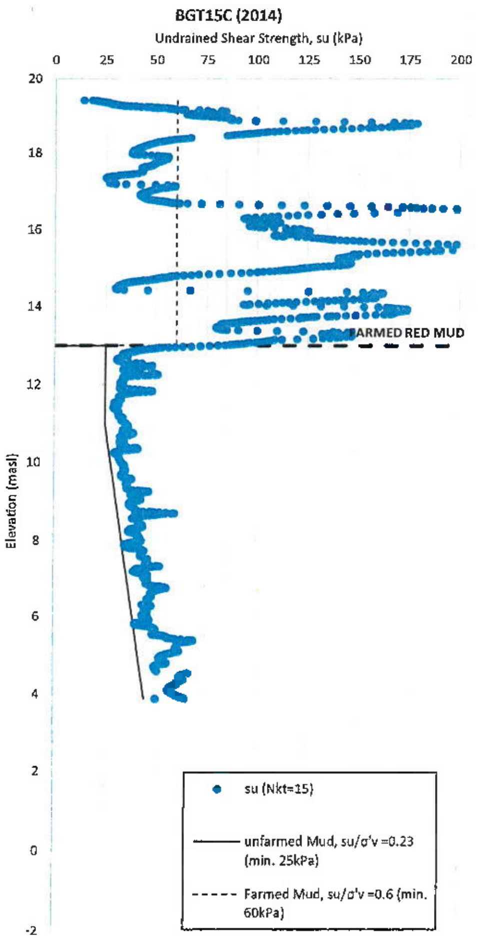
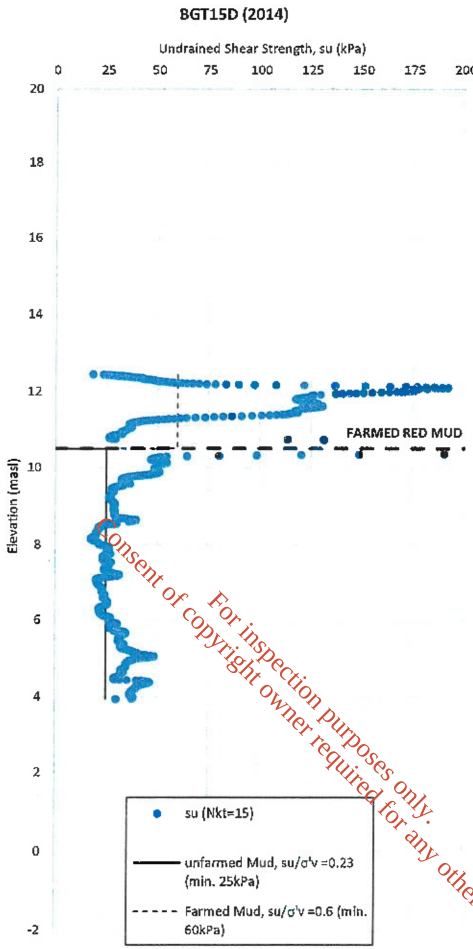
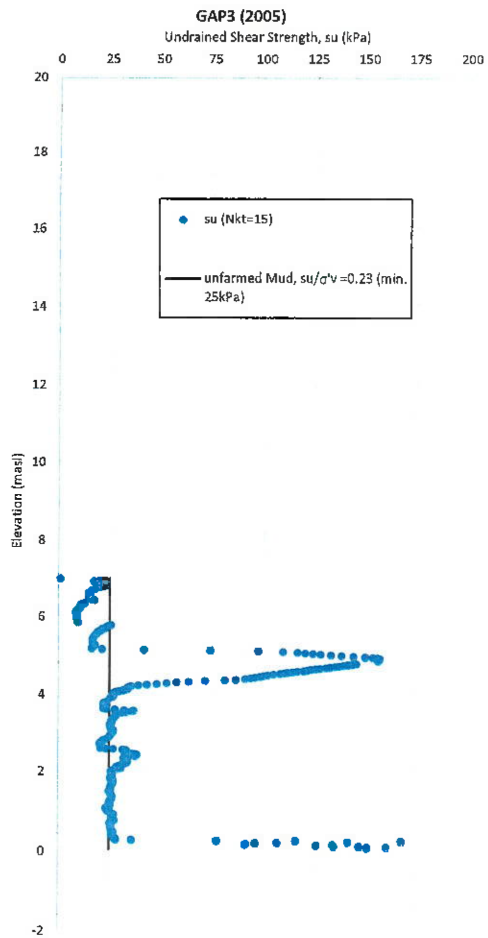
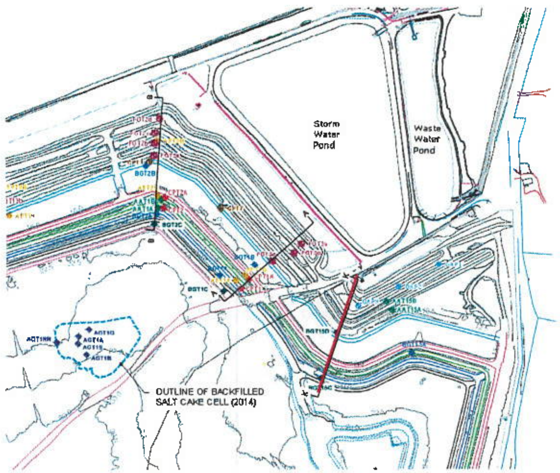
APPENDIX B

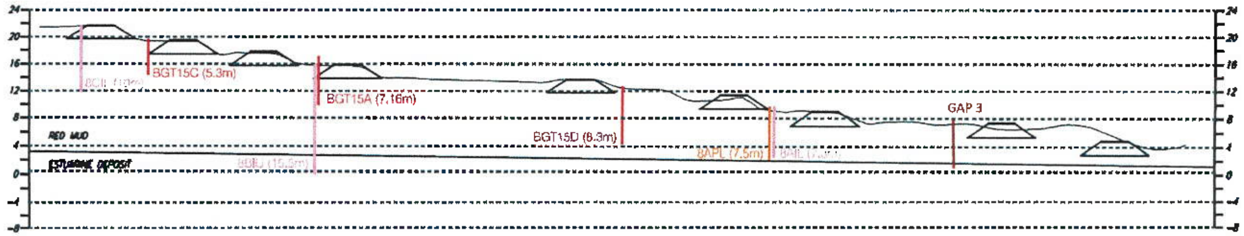
Cone Penetration Testing Analysis











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

AUGHINISH ALUMINA LIMITED

Golder Associates

16/08/2017
GLJ
GLJ
BK
RW

BRDA PHASE 1
BLAST VIBRATION ASSESSMENT

SECTION K
UNDRAINED SHEAR STRENGTH FROM CPT DATA

1667376 | 0 | B1

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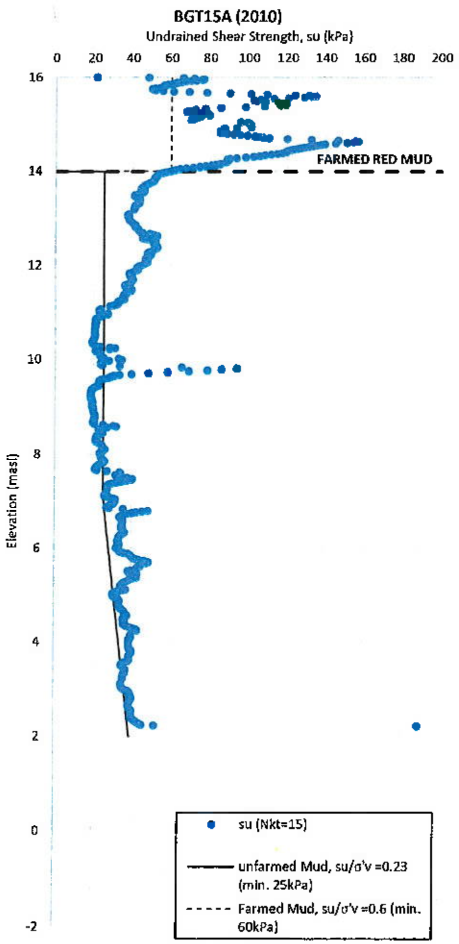

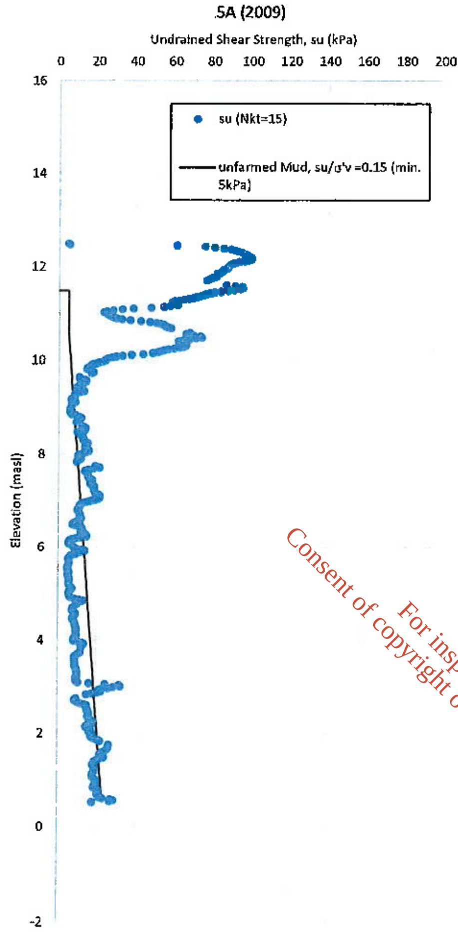

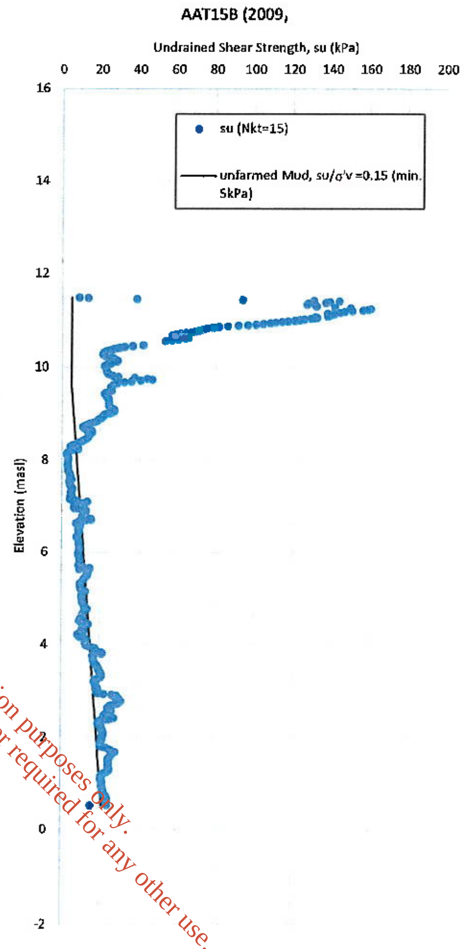

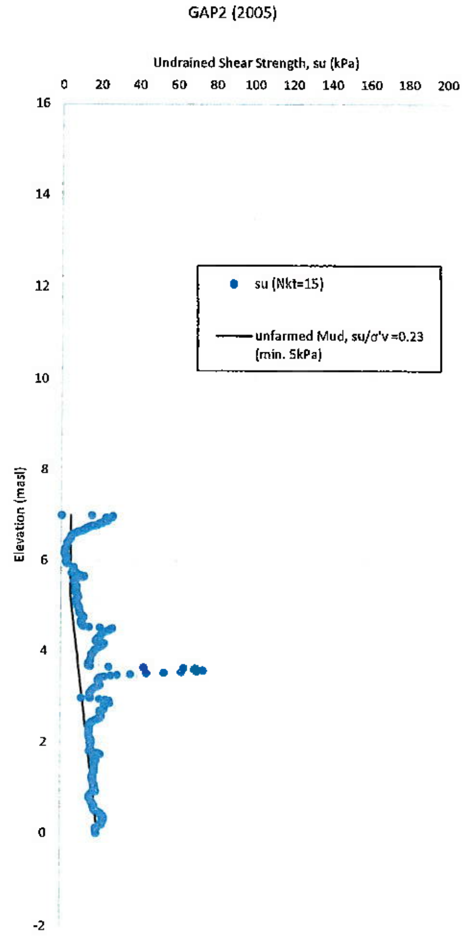

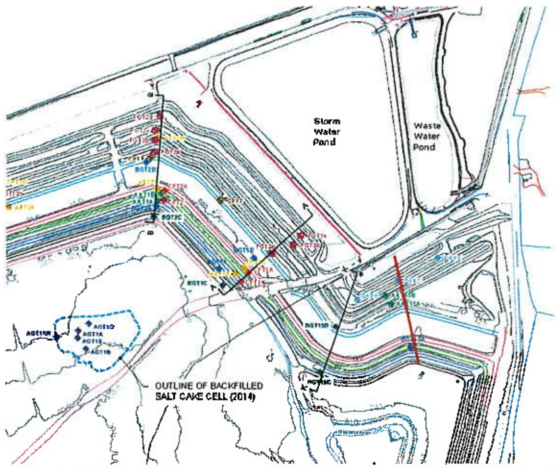

Notes:

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

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	15/08/2017
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	GLJ
	BK
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BRDA PHASE 1
BLAST VIBRATION ASSESSMENT

EAST OF SECTION K
UNDRAINED SHEAR STRENGTH FROM CPT DATA

1667376 0 B2

C:\Users\gjanssens\Documents\projects\Aughinish\Blasting\CPT - Section K 19June2017.xlsx

APPENDIX C

Pseudo-Static Analyses

1.0 UNDRAINED STATIC STABILITY ANALYSIS

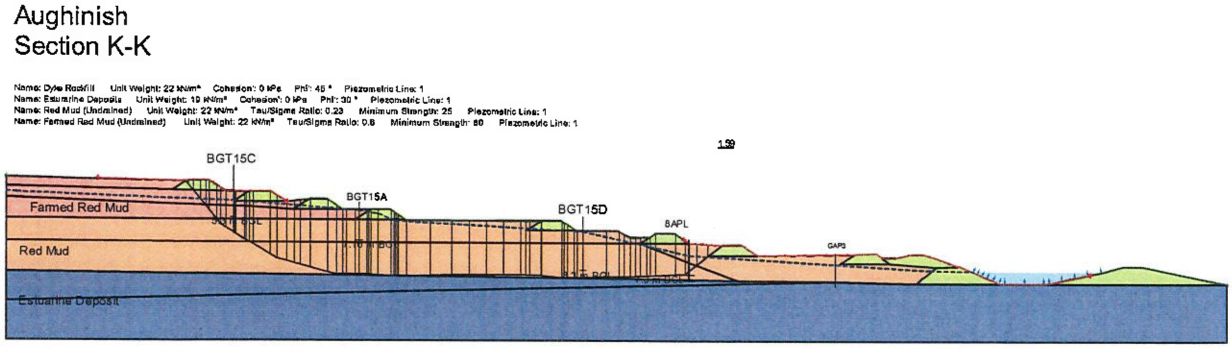

Figure C1: Undrained Static Stability Analysis, Overall slope

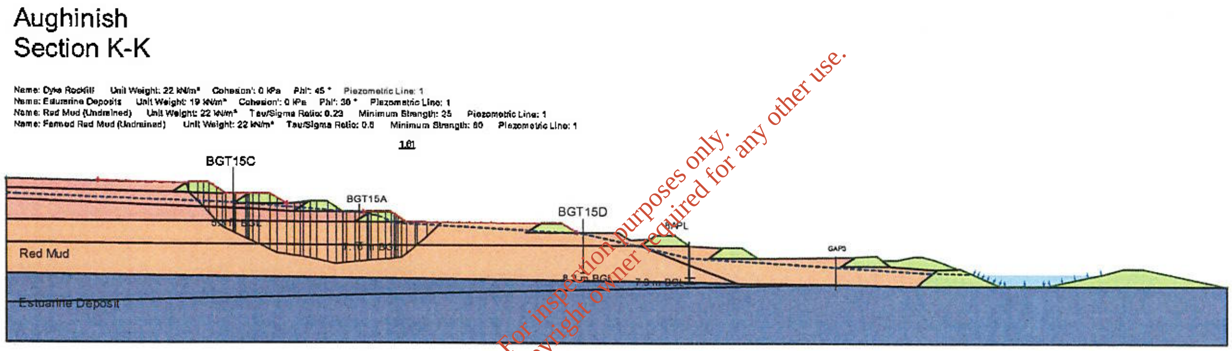

Figure C2: Undrained Static Stability Analysis, Upper slope

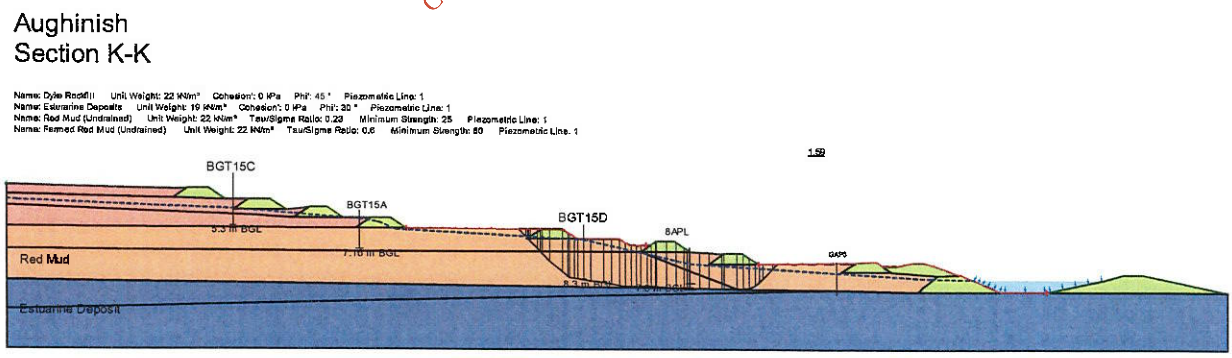

Figure C3: Undrained Static Stability Analysis, Upper slope

2.0 PSEUDO-STATIC STABILITY ANALYSIS

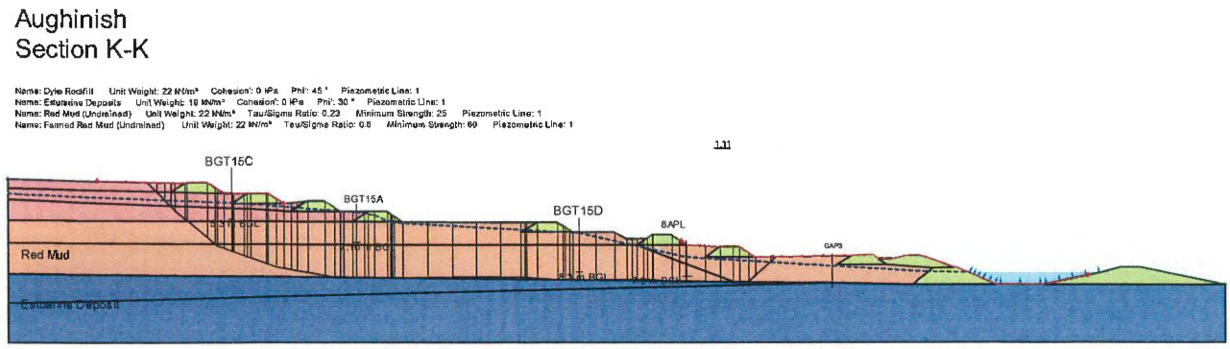

Figure C4: Undrained Pseudo-static Stability Analysis, PPV = 15 mm/s, Overall Slope

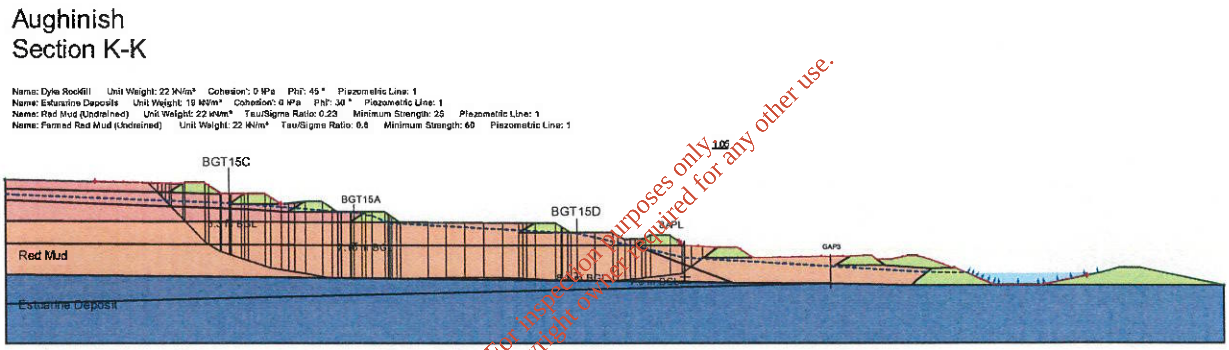

Figure C5: Undrained Pseudo-static Stability Analysis, PPV = 20 mm/s, Overall Slope

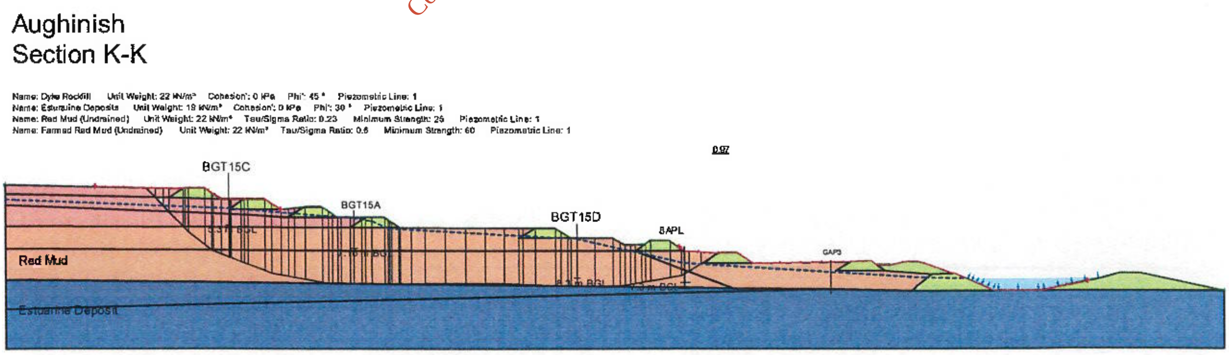

Figure C6: Undrained Pseudo-static Stability Analysis, PPV = 25 mm/s, Overall Slope

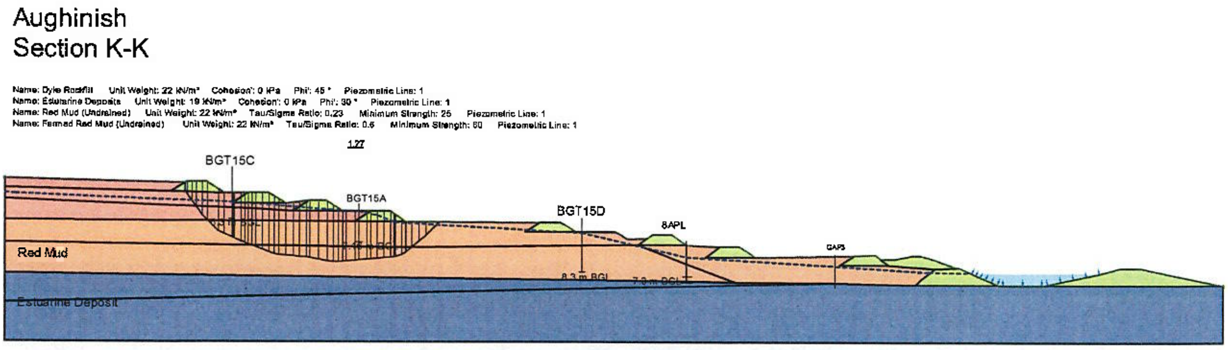

Figure C7: Undrained Pseudo-static Stability Analysis, PPV = 15 mm/s, Upper Slope

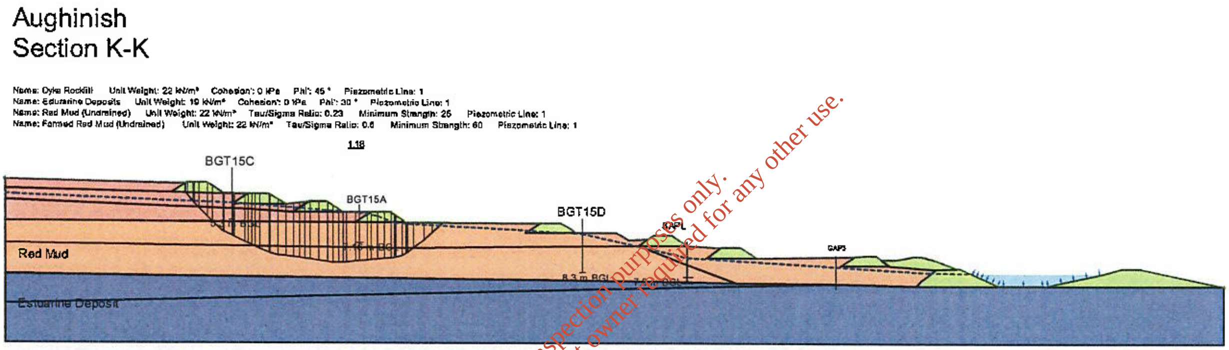

Figure C8: Undrained Pseudo-static Stability Analysis, PPV = 20 mm/s, Upper Slope

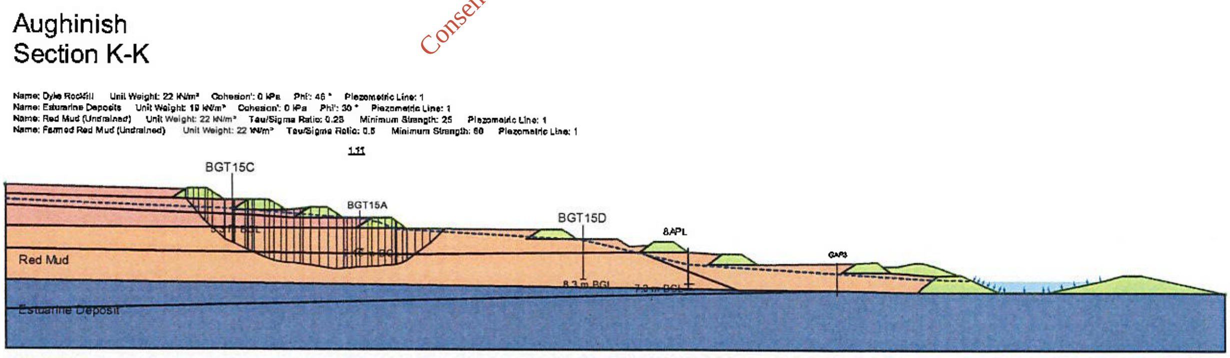

Figure C9: Undrained Pseudo-static Stability Analysis, PPV = 25 mm/s, Upper Slope

Aughinish
Section K-K

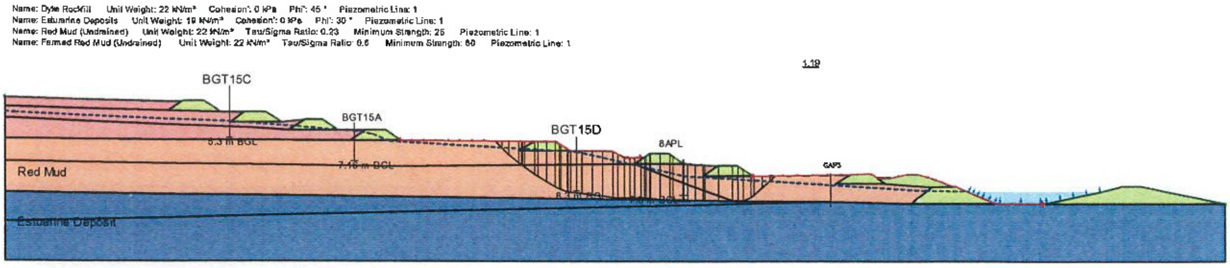

Figure C10: Undrained Pseudo-static Stability Analysis, PPV = 15 mm/s, Lower Slope

Aughinish
Section K-K

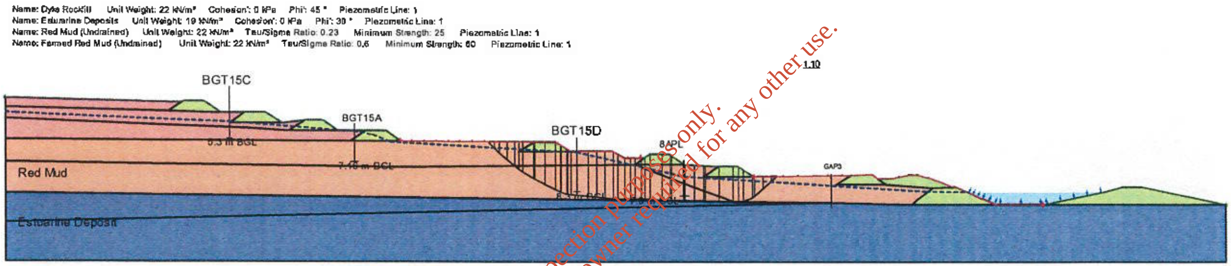

Figure C11: Undrained Pseudo-static Stability Analysis, PPV = 20 mm/s, Lower Slope

Aughinish
Section K-K

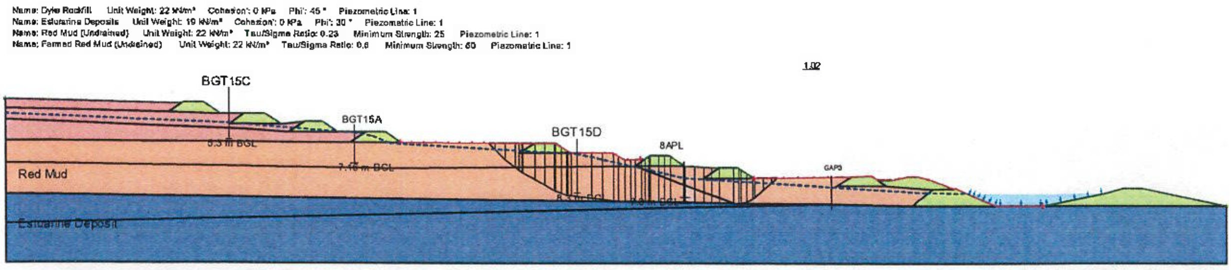

Figure C12: Undrained Pseudo-static Stability Analysis, PPV = 25 mm/s, Lower Slope

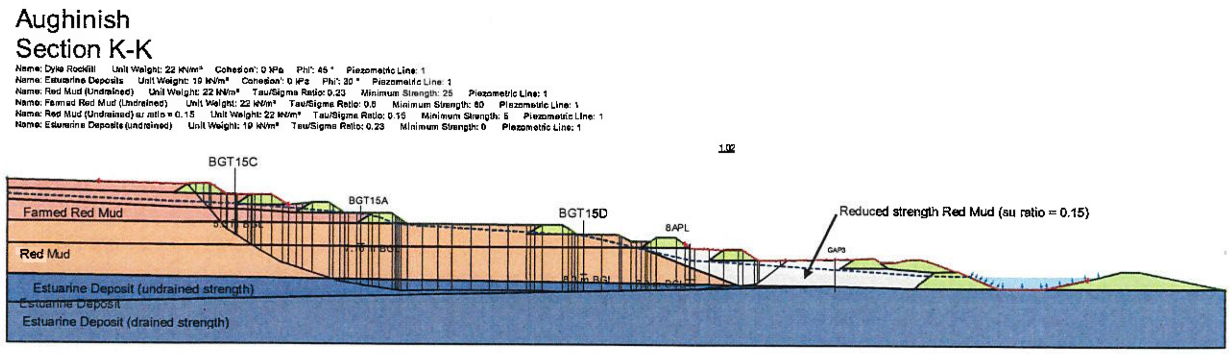

Figure C13: Undrained Pseudo-static Sensitivity Stability Analysis, PPV = 15 mm/s, Lower Slope

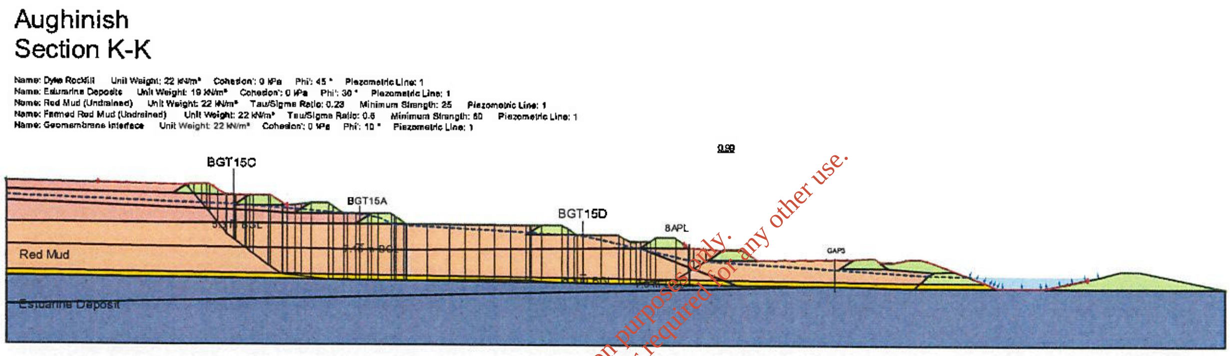

Figure C14: Pseudo-static Sensitivity Stability Analysis with reduced Geomembrane interface strength, PPV = 15 mm/s, Lower Slope

3.0 POST-BLAST (EXCESS PORE PRESSURE) STABILITY ANALYSIS

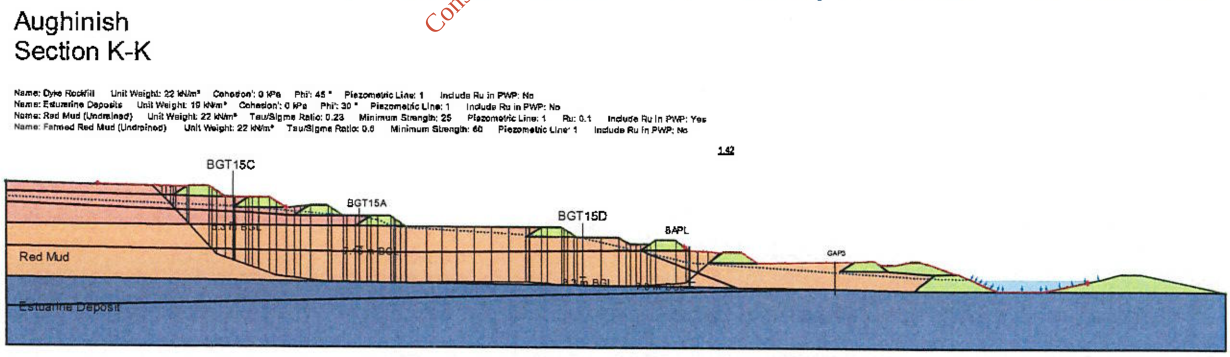

Figure C15: Undrained Post-Blast Stability Analysis, Average PPR = 0.1 (overall slope)

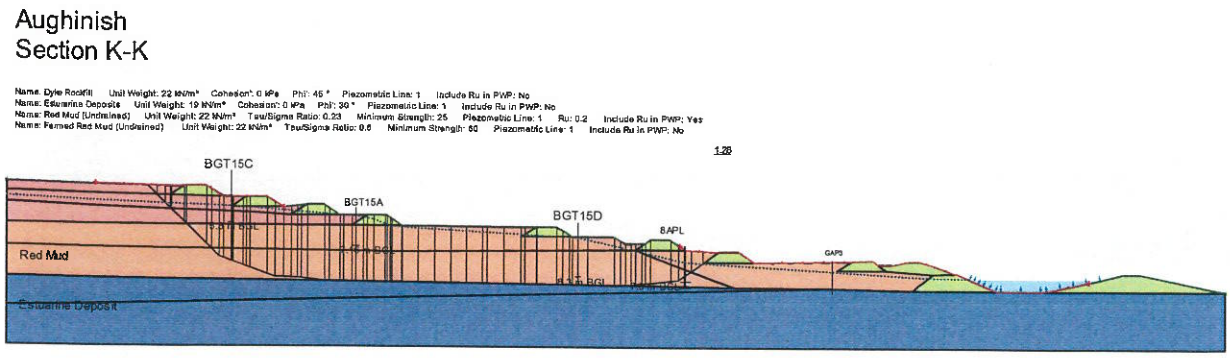

Figure C16: Undrained Post-Blast Stability Analysis, Average PPR = 0.2 (overall slope)

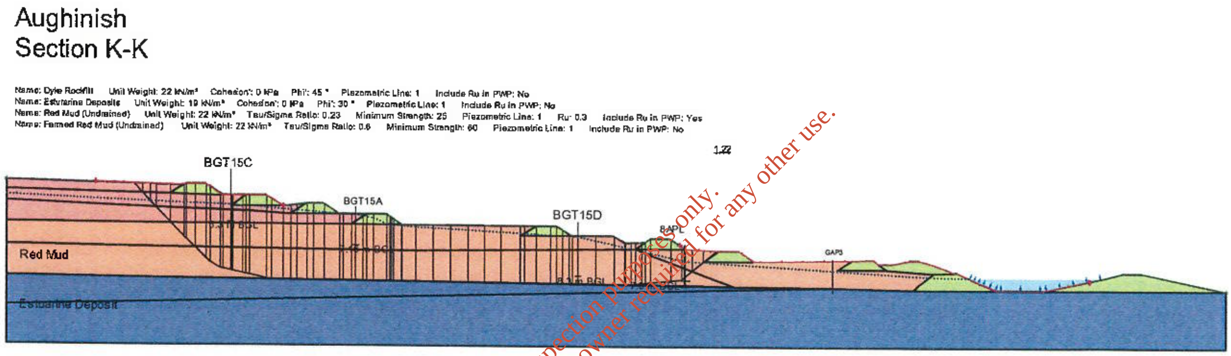

Figure C17: Undrained Post-Blast Stability Analysis, Average PPR = 0.3 (overall slope)

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Golder Associates

12.0 LANDSCAPE AND VISUAL IMPACT ASSESSMENT

12.1 Preamble

The Landscape Chapter of the EIAR was prepared by Brady Shipman Martin and provides an assessment of the likely landscape and visual impacts of the proposal by Aughinish Alumina Ltd. to develop a Borrow Pit of c. 4.5 hectares for the extraction of rock, together with associated crushing and stockpiling of the rock.

The proposed development will located within the wider and existing Aughinish Alumina Ltd. lands, and on previously disturbed grounds including part of a former borrow pit. The proposed borrow pit will provide rock material for the ongoing works associate with the BRDA, and also provides for a restoration plan of the extracted area upon completion.

A detailed project description is provided in *Chapter 3 Description of the Proposed Development* of the EIAR.

12.2 Assessment Methodology

12.2.1 Guidelines, Legislation, Policies and Plans

This landscape and visual assessment has been undertaken with regard to the following publications:

- Guidelines on the recommended information to be contained in Environmental Impact Statements published by the Environmental Protection Agency (EPA 2002);
- Advice Notes on Current Practice in the Preparation of Environmental Impact Statements (EPA 2003);
- Revised guidelines on the information to be Contained in Environmental Impact Statements (Draft), September 2015
- Advice Notice for Preparing Environmental Impact Statements (Draft), September 2015;
- Limerick County Development Plan 2010 – 2016; and
- Regard was also had to the new EIA Directive 2014/52/EU adopted on 16th April 2014, and in anticipation of it coming into force later in the year.

12.2.2 Assessment Approach and Significance Criteria

Impact on the landscape arising from development has two distinct but closely related aspects. The first is **impact on character** in the form of change to the character of the landscape that arises from the insertion of the proposed development into the environment. The combined impacts will elicit responses, the significance of which will be partially dependent on how people perceive a particular setting or landscape and how much the changes will matter in relation to other senses as experienced and valued by those concerned.

The second aspect, *visual impact*, is considered as *'visual intrusion'* and *'visual obstruction'*, where:

Visual intrusion is concerned with the relative perception of visual impact based on the degree to which the proposed development impinges on a view without blocking it.

Visual obstruction is defined as the full or substantial blocking of a view by the proposed development or by constituent elements of the development.

This landscape and visual assessment entailed:

- Visiting the site on numerous occasions between September 2016 and March 2017, reviewing the condition of the site, views to and from the sites, preparing a photographic record of the main landscape and visual elements, noting features and characteristics, etc.;
- Undertaking a desktop study of the location and context of the site, using information gathered from site visits, and reviewing the County Development Plan for landscape and visual aspects such as protected views, landscape features, trees, etc., and studying ordnance mapping and aerial photography;
- Reviewing engineering proposals, in plan, section and elevation on an on-going basis;
- Reviewing emerging engineering, construction and other environmental assessment reports; and
- Reviewing the series of photomontages prepared for the proposed developments.

The assessment of landscape and visual impact considers the significant and sensitive characteristics of the environment and notes the character of the impacts, their magnitude, duration and consequences. Such impacts may be direct or indirect, cumulative or arise as a result of particular interactions and may or may not be residual in nature.

The significance criteria used for the assessment is in accordance with the criteria suggested in the EPA "Guidelines on information to be contained in an EIAR" as follows:

Imperceptible: An impact capable of measurement but without noticeable consequences.
Slight: An impact which causes noticeable changes in the character of the environment without affecting its sensitivities.
Moderate: An impact that alters the character of the environment in a manner that is consistent with existing and emerging trends.
Significant: An impact which, by its character, magnitude, duration or intensity alters a sensitive aspect of the environment.
Profound: An impact which obliterates sensitive characteristics.
In each instance the landscape and or visual impact may be *positive, neutral or negative* and duration may be considered as being either *temporary* (lasting 1 year or less); *short-term* (lasting 1 to 7 years), *medium-term* (lasting 7 to 15 years); *long-term* (lasting 15 to 60 years) or *permanent* (lasting over 60 years).

12.2.3 Photomontages

A series of photomontages were prepared to accurately represent the physical and visual characteristics of the proposed development within its setting. The Photomontages are included as Appendix 12.1 of the EIAR.

The proposed development is entirely contained within a distinct part of the existing Aughinish Alumina Ltd. lands and principally comprises excavation into those lands. The views selected are from the surrounding landscape context, from publicly accessible areas, and are selected to be representative of predicted visibility of the proposed development from that context.

The location of all views is shown on Figure 1.0 of Appendix 12.1 of the EIAR, and for each view, the 'As Existing' and 'As Proposed' version of the view is presented.

12.3 Baseline Environment

12.3.1 Description of the Receiving Environment

Aughinish Island is located in a rural, low-lying area dominated by the Shannon estuary with its associated wetlands, mud-flats and large areas of open water.

12.3.2 Aughinish Island Landscape Context

Low-lying, agricultural landscape extends south and west of the island from Foynes east through Barrigone to east and southeast of Askeaton. The low-lying terrain does include localised variations in topography and field and road boundaries are typically defined by hedge and tree rows. Pasture predominates as a land use and there is little arable farming in the area. Residential property is generally dispersed along local roads with increased density notable at settlements such Barrigone, Fawnamore and along the N69 leading into Foynes and Askeaton.

Rising and upland agricultural land with panoramic views over surrounding areas lies immediately southwest of Foynes. The land rises notably to 172m AOD at Knockpatrick Hill where a graveyard and old church are prominently located on the exposed hilltop. The elevated vantage offers expansive panoramic views east and northeast over the surrounding countryside, Aughinish Island and the Shannon Estuary.

The shoreline of the Shannon Estuary includes generally small scale spaces around the various creeks and inlets of the Shannon estuary. These areas are frequently intimate settings characterised by mud-flats and wetlands. While they include manmade interventions such as flood defence walls and embankments, areas of semi-wilderness also remain, principally as wetlands, reed beds, salt marshes and mud flats. AAL have developed as series of popular nature trails to the east of the facility and which take-in the margins of Poulaweala Creek and the Shannon Estuary.

The large expanse of open water in the Shannon Estuary is the most significant landscape element in the area. The water brings character and setting to surrounding landscape elements and provides a base against which the topographical variation and the visual distinctiveness of the surrounding landscape are viewed.

The Aughinish Alumina Ltd. facility, comprising both the production plant as well as the Bauxite Residue Disposal Area (BRDA), is located on Aughinish Island. The overall facility extends c 3.5km from the dis-used Limerick to Foynes railway line to the mudflats of the Shannon Estuary, and up to 1.5km or more in the east to west direction. It one of the more prominent industrial features located within the Shannon Estuary landscape.

12.3.3 Development Area

The development area extends to c. 7 hectares with an extraction area of c. 4.5 hectares located within the overall Aughinish Island, and with the Aughinish Alumina Ltd. facility. The lands include a contractor's compound, stock piling areas, and part of a former borrow pit.

Existing ground levels at the site vary from 16m AOD to c. 20m AOD, with the exception of the former borrow pit that is a base level of c. 9m AOD. The site is located immediately east of the northernmost part of the Phase 1 BRDA which will rise to c. 32m AOD, and due south of the production facility that includes extensive industrial production buildings and infrastructure.

To the immediate east of the site, and extending further south to the Phase 2 BRDA, an area of c. 500m width comprising mixed scrub woodland and grassland leads to the mudflats between Aughinish Island and Poulaweala Creek beyond. This area includes the access roadway to the Aughinish facility, and also includes a network of popular nature trails, a butterfly sanctuary, a public carpark and a number of related community amenity facilities.

The development area is perceived primarily as connected to the main Aughinish facility, and is generally at a distance of c. 100m or more from the publicly accessible areas, and separated by areas of mixed scrub woodland and grassland. The westernmost nature trail does run c. 20m from the edge of the former borrow pit, and is separated by a landscape buffer zone including planted berms as well as a safety fence.

12.3.4 Landscape Planning Context

The Strategic Integrated Framework Plan (SIFP) for the Shannon Estuary designates Aughinish Island as a Strategic Development Location, and this is further supported by the County Development Plan. The SIFP recognises the Shannon Estuary as *"an immensely important asset and one of the most valuable natural resources in Ireland".* The County Development Plan also includes policies and objectives which respond to this strategic industrial importance of the area while simultaneously considering the sensitives of the surrounding landscape, including *Policy ED P10: Ensuring no adverse environmental impacts*, and *Objective ED 04: Safeguard Strategic Development locations along the estuary.*

Notwithstanding this industrial characteristic, the County Development Plan also identifies key landscape aspects of the area. Of particular importance is the scenic route in proximity to the site, which extends along the N69 Coastal Road through the town of Glin to the west and eastwards as far as Foynes.

Policies and objectives in the County Development Plan that may be relevant to the proposed development in the context of its landscape setting and are outlined below:

Objective EH O6: Landscaping and Development

It is the objective of the Council to

- Ensure the adequate integration of development into the landscape by the retention of existing trees and landscape features and/or suitable planting.
- Encourage, where appropriate, the use of native species. The layout of landscaping planting and features to act as wildlife corridors within developments, particularly residential developments, and linking with other habitats in the area will be encouraged.
- Resist the removal of substantial lengths of roadside boundaries. Where an alternative, suitable site is available for the development, applicants should consider such an alternative on the basis that avoids the necessity for widespread boundary removal. Only in exceptional circumstances should roadside boundaries be removed.

The CDP outlines specific objectives for the landscape character type within which the proposed site is set. The majority of these are in the context of housing control, and as a result the following is an outline of key extracts relevant to the proposal in question:

Objective EH O12: Shannon Coastal Zone Landscape Character Area

It is the objective of the Council:

- To protect the views and prospects along the N69 as a priority for the Planning Authority. Only in exceptional circumstances ... will development be allowed between the road and the estuary ...
- To encourage the use of site-specific designs with careful attention to landscaping ...
- All of the above (a to c) does not apply within the settlements of the Shannon Coastal zone.
- To rigidly adhere to best practice in the installation and use of wastewater treatment systems,
- given the proximity of the Shannon and the importance of water-based habitats in the area, to ensure that no deterioration in water quality takes place.
- Development identified under the Strategic Integrated Framework Plan will adhere to the mitigation measures for landscape management as appropriate.

With regard to scenic views and prospects, the following objective is identified:

Objective EH O17: Scenic Views and Prospects

- It is the objective of the Council to safeguard the scenic views and prospects by integrating them into landscape character areas, which will ensure a more balanced approach towards landscape issues within the County.
- In areas where scenic views and prospects are listed in Map 7.6 there will be a presumption against development except that which is required in relation to

farming and appropriate tourism and related activities, or a dwelling required by a long term land owner or his/her family that can be appropriately designed so that it can be integrated into the landscape.

- The Planning Authority will exercise a high level of control (layout design, siting, materials used, landscaping) on developments in these areas. In such areas site specific designs are required. It should be noted that in areas outside these delineated areas, high standards will also be required.

The nature and scale of the proposed development, in the context of the existing physical environment is such that it will be readily absorbed within the existing industrial facility, and will not give rise to additional adverse landscape and visual impacts.

12.3.5 Overall Summary, Significance and Sensitivity

The scale of the proposed development site in the context of the existing Aughinish Alumina facility on Aughinish Island is such that it is unlikely to give rise to any significant landscape and visual impacts within the existing landscape context. Additionally, the extractive nature of the proposed development is such that the emerging and resulting site profile will be less visible than the existing site area, if not entirely invisible, from vantage points within the low lying rural context.

Glimpse views may be possible from more elevated vantage points, however, these would be from a considerable distance to the south west, and the BRDA and the production facility structures are the dominant elements within the landscape and in front of the proposed development area when viewed from this direction.

In relation to the landscape planning context, the proposed development will be on previously disturbed land, and will have negligible impact on existing vegetation. The development includes the provision of extensive additional new native trees and vegetation along the proposed perimeter berms and also within the borrow pit upon restoration.

The scale and location of the development is such that it will not be impact on the views and prospects from the section of the N69 between Foynes and Glin as identified in the County Development Plan.

12.4 Description of the Proposed Development

The proposed borrow pit will include extraction of rock over an area of c. 4.5 hectares to a depth of c. 8m, and to a finished level of c. 8.5m AOD. Extraction will take place, starting in the former borrow pit and moving in the northern direction over a 10 year period.

Upon commencement, the construction stage will include removal of the existing contractor's compound and any associated storage, the erection of a 2m high security fence around the site perimeter, the stripping of topsoil from the full site area, and formation of 8m wide and 2m high berms along the northern, western and eastern edges of the development. The berms will be planted with mixed woodland and understorey shrubs so as to facilitate early establishment of the longer term perimeter screening. The planted berms will establish to provide a continuous screen separation between the development area and the adjoining lands.

The security fence and berms will generally be 70m or more from the existing nature trails. Where the proposed borrow pit joins the former borrow pit, a short section of the westernmost part of the nature trail will be 10 to 20m from the fence where it runs closer to the former borrow pit and the borrow pits and boundaries will tie into each other.

The operational stage will include excavation, rock crushing and stockpiling, and will be phased from the former borrow pit at the southern part of the development area extending northwards towards the production facility. Excavated material will be crushed at the lower excavated level throughout the phasing. Crushed rock will be stockpiled towards the southern end of the borrow pit at the excavated level. The volume of the stockpile will vary over time, but in any event will not be higher than the depth of the borrow pit and will not protrude above the adjoining ground level. Rock from the stockpile will be transported by truck along existing internal construction tracks to the BRDA.

Closure will take place upon completion of excavation and will include the implementation of a landscape restoration plan.

12.5 Predicted Impacts

Potential landscape and visual impacts arising from the proposed development include direct and indirect impacts. Direct impacts, in general, will include excavation of soil and rock from the site, and may also include visibility of exposed excavation faces and the proposed perimeter berms and planting from beyond the site. Indirect impacts, In general, may include temporary or short-term visibility of construction activity on the site.

It is noted that the majority of the development including excavation, crushing, stockpiling and haulage activities, will occur at the reduced level of the former and proposed borrow pit. The presence of extractive activity and extraction related vehicles at the existing ground level will be limited to the initial soil stripping of the site and establishment of the perimeter berms, and temporarily in preparation of sequential phases for excavation.

12.5.1 Landscape Impacts

The nature of the proposed development is such that there will be direct and permanent impacts on the landscape of the development area as vegetation and topsoil is removed, and as rock is excavated to the a depth of c. 8m throughout.

The existing site area, in the context of the much wider Aughinish Alumina Ltd. facility, and comprising contractor's compound and storage, limited vegetation and previously disturbed ground, is considered to be of low landscape value. The development area partly extends to a former borrow pit, and the proposed development is essentially an extension of that borrow pit. In that context, the direct landscape impacts are considered to be *moderate, neutral and permanent,* and will be mitigated through the implementation of a restoration plan as will be described below.

12.5.2 Visual Impacts

Given the low-lying character of the landscape context, and the more elevated BRDA immediately to the west of the proposed development area, the potential of the proposed development to become visible is greatest from the area to the east at Fawnamore and Poulaweala Creek, and also from elevated and more distant ground to the south west at Knockpatrick.

A series of Accurate Visual Representations (AVRs), of photomontages, have been prepared from the area east of the proposed development, and also from the elevated ground at Knockpatrick Cemetery to the southwest. These, together with a view location map, are included in Appendix 12.1 of the EIAR.

View 1 is from the quay at the northern end of Poulaweala Creek looking south west towards the development site. The *As Existing* view clearly shows the undulating and higher ground on the opposite side of the river inlet to be prominent in the middle ground and restricting visibility to the Aughinish access road and beyond to the BRDA.

The *As Proposed* view indicates the 8m deep profile of the full extent of the proposed borrow pit in a red outline, and indicates that neither the borrow pit nor the perimeter berm and security fence will be visible from this location. There will be **no visual impact** from this location.

View 2 is from the local access road leading to Poulaweala Creek and looking north west towards the development site. The *As Existing* view shows middle ground vegetation and topography limiting visibility in the direction of the site, with only glimpse views above the of occasional structures or land profiles associates with the Aughinish facility.

The *As Proposed* view indicates the 8m deep profile of the full extent of the proposed borrow pit in a red outline, and indicates that it will not be possible to see into the borrow pit from this location. There may be glimpse views of the planting on the perimeter berms, however, the landscape and visual impact arising will be **imperceptible and neutral.**

Upon commencement, some construction activity will be apparent as topsoil is stripped and the perimeter berms are established and planted. This construction activity will be substantially screened by intervening topography and vegetation, will be temporary, and will not give rise to any significant visual impact.

View 3 is from the local road at Fawnamore, west of the main road that leads to Poulaweala Creek, and looking north west towards the development site. The *As Existing* view shows middle ground vegetation and topography limiting visibility in the direction of the site, and includes views of some of the taller and more substantial structures associated with the production facility. At the centre of the view, a green field and hedgerow can be seen on the horizon. This field is the grass field located immediately east of the former borrow pit, and the hedgerow runs along the northern boundary of the field.

The *As Proposed* view indicates the 8m deep profile of the full extent of the proposed borrow pit in a red outline, and indicates that neither the borrow pit nor the perimeter berm and security fence will be visible from this location. There will be **no visual impact** from this location.

View 4 and View 5 are from the local road at Fawnamore in the vicinity of existing residential dwellings and looking north west towards the development site. The *As Existing* views show foreground and middle ground vegetation that restricts visibility in the direction of the site.

The *As Proposed* views indicate the 8m deep profile of the full extent of the proposed borrow pit in a red outline, and indicates that neither the borrow pit nor the perimeter berm and security fence will be visible from these locations. There will be **no visual impact** from these locations.

View 6 is from the access road to Aughinish Island just north of the railway overbridge and looking north west towards the development site. The *As Existing* view shows glimpse views of parts of the BRDA and of the taller elements of the production plant, and also includes views towards some of the residences at Fawnamore. Middle ground vegetation restricts visibility in the direction of the development site.

The *As Proposed* views indicate the 8m deep profile of the full extent of the proposed borrow pit in a red outline, and indicates that neither the borrow pit nor the perimeter berm and security fence will be visible from this location. There will be **no visual impact** from this location.

View 7 is from the Knockpatrick Cemetery some 4.5 km to the south west but at an elevation of c. 170m AOD. The *As Existing* view shows the panoramic nature of the view from here that includes the Shannon Estuary, Foynes, Aughinish Island, and expansive views to the north and around to the east. The proposed development site is discernible to the right of the production facility structures and beyond the Phase 1 BRDA. It is possible to see some of the rock face of the former borrow pit, as well as the existing contractor's compound and associated storage containers and vehicles.

The *As Proposed* view includes the full extent of the proposed borrow pit as well as the perimeter berms and planting. Upon commencement, the contractor's compound will be removed, and there will be construction activity and construction vehicles as the topsoil is stripped and the perimeter berms are formed.

Following the establishment of the site, extractive activity will mostly be confined to the reduced level of the former borrow pit where rock crushing, stock piling and haulage will take place. This will comprise limited amounts of equipment and will mostly be screened by virtue of being located at the reduced level.

As each new phase of excavation commences, there will be temporary activity at the upper level and the next phase is prepared for extraction. Over the duration of the 10 year extraction programme, the planted perimeter berms will become more established and effective in screening operational activity within the borrow pit.

The landscape and visual impact is considered to be **imperceptible, neutral and permanent.**

12.6 Mitigation

As set out in the previous sections, the nature of the proposed development is such that there will be limited landscape and visual impact, save for temporary construction activity associated with site preparation and the establishment of perimeter berms. Mitigation measures can be described in three separate categories, including Design Mitigation, Construction Mitigation, and Landscape Reinstatement.

Design Mitigation

The proposed scheme incorporates inherent mitigation as the majority of construction activity, as well as the final development, is at a reduced level relative to the existing ambient ground levels, and therefore will be self-screening by its nature.

The proposed development includes the early stage establishment of planted perimeter berms that will serve to mitigate at-grade construction activity, and also to provide a longer term integration within the immediate and wider landscape context. The planted perimeter berms will also provide early stage screening from the portion of the nature trail that extends from the former borrow pit and where the tow borrow pits will join.

Construction Mitigation

Construction activity will be both at-grade and also at the reduced level of the borrow pit. At-grade construction will be temporary as topsoil is stripped and perimeter berms are formed. The early establishment of berms and proposed planting will mitigate subsequent temporary at-grade activity as new phases of the development are prepared for excavation.

Activity at the reduced level of the borrow pit has inherent mitigation by virtue of being at the lower level of the borrow pit. For the most part, this activity will not visible from beyond the development site itself.

Landscape Restoration

Upon cessation of extraction activity, a landscape restoration plan will be implemented so as to enhance the landscape and ecological value of the resulting borrow pit.

Soil pockets will be established at the toe of the excavations, and localised areas of the base level of the borrow pit will be filled with topsoil. These areas will be planted with native species including Willow, Alder, Birch Hawthorn and Blackthorn. Additionally, dry calcareous type grass will naturally establish over much of the rest of the borrow pit base.

12.7 Residual Impacts

Residual impacts will include the direct impacts of the proposed development area as the ground level reduced permanently by some 8m. The proposed development includes the establishment of planted perimeter berms that will be permanent and at the existing pre-development ground levels, and also a comprehensive final restoration plan within the excavated ground that will include planting of native species trees and shrub vegetation.

Within the immediate site area, there will be permanent change, however, such change is consistent with other previous and ongoing activities at the facility, and the scale of the proposed development can readily be absorbed within the existing wider facility.

From beyond the site, residual landscape and visual impacts will be either none or imperceptible.

APPENDIX 12.1: Photomontages prepared by Brady Shipman Martin Landscape Architects