Response to Further Information Request (FIR)

12/09/2017

Response to Further Information Request 12/09/2017

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Item 1

The NIS states that "the footprint of the Stage 6 facility is confined to the existing northern and seven fields borrow areas. Virtually all the area is exposed indicating rock or glacial till over the site". The EIS states that " ... a resistivity and microgravity survey would be undertaken to delineate the presence of any palaeokarstic features... "

a. *Please provide a report of a comprehensive study of palaeokarst features and their potential impact on Stage 6 TMF stability. The study and report should detail the following as a minimum; site investigation plan, geotechnical logs, soil and rock laboratory testing, tailings geotechnical characterisation, geological plan and sections, and geotechnical parameters.*

b. *Table 5.1 of the EIS details the geotechnical parameters adopted in the stability modelling. Please compare these modelling parameters to the parameters obtained from the study requested in paragraph 1a above. A sensitivity analysis (undertaken to assess any potential variation in parameters for soil foundation, tailings and embankment construction material), should be included in the report requested in paragraph 1a above.*

Response to Item 1(a)

The approach taken to investigate the presence of possible palaeokrast features was to use a combination of tried and tested Geophysical methods; EM31, Electrical Resistivity Imaging (ERI), Seismic Refraction and Micro-Gravity.

Micro-Gravity is an industry wide method used to specifically target the presence of possible palaeokarst features, and was successfully used at Lisheen and Galmoy Mines on their Tailings Management Facilities.

The Geophysical investigations and follow-up intrusive investigations based on the findings of the Geophysics are provided in the following reports;

- Golder Associates (May 2017). Boliden Tara Mines Limited: Stage 6 Footprint Geophysical Surveys. Report reference 1775908.R01.B0
- Golder Associates (May 2017). Boliden Tara Mines Limited: Possible Borehole Locations – Pale Beds – Stage 6 ('Borrow Area'). Report reference 1775908.R02.B0
- Golder Associates (September 2017). Boliden Tara Mines Limited: Stage 6 Footprint Palaeokarst Investigation and Grouting

The investigation did not identify any large voids beneath the footprint of the Stage 6 facility that might pose a risk to the integrity of the facility and is presented in Appendix to FIR item 1.

Response to Item 1(b)

The successful backfilling of the paleokarstic features with grout as outlined in the report provides a foundation equal to the fractured limestone without any paleokarstic features. Therefore, the design parameters for the foundation rock were not changed.

It should be noted that no palaeokarstic features were found beneath the footprint of the dam wall.

Item 2

The EIS states that the "physical and chemical characteristics of the deposited tailings will remain unchanged from those deposited previously" and the NIS states that "the tailings are expected to be non-acid generating". Please provide an updated assessment of potential for acid generation to include consideration of the construction materials to be used.

Response to Item 2

An updated assessment of potential for acid generation in deposited tailings at the Randalstown TSF is presented in Appendix to FIR item 2.

The tailings have a high net neutralisation potential and results (past and on-going) indicate a considerable buffering capacity of the tailings to neutralise any acid generation

- Significant reductions in tailings pH or alkalinity are not expected to occur over time
- Acid mine drainage from the Randalstown tailings facility is not expected to be a significant environmental issue

All construction material to be used in construction of Stage 6 will have a neutralising potential ratio, determined on the basis of static test EN 15875, of greater than 3.

Item 3

The EIS refers to two alternatives used to assess the stage 6 TMF options for mine waste disposal namely; slurry and paste. This is not satisfactory. Please provide a more detailed report on the assessment of tailings disposal options from conventional slurry to dry stack. The selection of the preferred option should evaluate environmental and technical impacts and clearly demonstrate the selection process through scoring and ranking, using, for example, the multiple account analysis methods.

Response to Item 3

Surface paste disposal was one of the options considered for increasing the tailings storage of the TMF previously for stages 4 and 5. It was proposed to construct these stages as a dome on the existing facilities. Paste has the benefit of increasing the density of the deposited tailings and thereby reducing the volume of material to be stored. Also, the dome shape would increase the storage capacity for a given dam wall crest height and the inherent strength of recently deposited paste would have resulted in potentially lower costs for rehabilitation of the surface at closure.

Surface paste tailings disposal has significant advantages over conventional disposal of slurried tailings particularly when;

- reclamation of water has to be maximised such as in semi and arid areas;
- the reclaim water has significant value in terms of added chemicals which otherwise would have to be added in the process;
- space is a premium and the dam foot print is to be minimised; and
- the terrain is steep and tailings paste can be deposited down valley and above the containment dam wall thus maximising storage;

The distance of the TMF from the mill site is some 5km and at the limit for positive displacement pumps and therefore the paste plant for surface disposal of tailings paste would have to be located at the TMF site. The tailings would be pumped as a slurry to the TMF as currently undertaken and then processed through the paste plant and pumped to the facility using positive displacement pumps.

Stages 4 and 5 could have been domed from a central stack at the middle of the dividing wall originally between Stages I, II and III. This would not be the case for Stage 6 where the paste would initially be discharged from the crest until nearly full and then a central discharge tower constructed which could be used to form a dome. The tower required would be founded on fill above tailings and therefore subject to potential differential settlements.

The paste plant would operate 24hrs/day and the plant, the discharge points and eventually the tower would need to be adequately lit to allow for continuous observation/access.

The pasted tailings would form a slope between 2.5% and 5% depending on the final moisture content at discharge. Thus for a slope of 2.5%, the central tower height would need to be about 10m high for the paste to reach the Stage 6 crest wall at a distance of 400 m. Normally, the slope gradient will decrease as the distance from the discharge point increases and more discharge points may be required.

To prevent dusting of the paste an extensive system of sprinklers would be required and operated frequently during dry and windy periods. The exposed beaches would be subject to extensive dusting unless continually saturated. The sprinkler system would have to be continuously raised as paste is deposited. The sprinkler water and rainfall runoff from the slopes, which would be contaminated, would be collected around the upstream perimeter of the Stage 6 dam wall crest via a constructed channel. The water collected would be discharged into Stage 5. An emergency facility to hold slurried tailings would also be required when the paste plant is maintained or for some reason was non-operational. The facility would again be the existing Stage 5.

Tara Mines use the coarse fraction of the tailings (sands) for hydraulically back filling stopes underground as part of the mining operation. This procedure also reduces the amount of tailings discharging to the TMF. The fine tailings (slimes) are discharged into the TMF. When backfilling underground is not required, the total tailings (sand and slimes) are discharged directly to the TMF. Surface disposal of paste needs to be reasonably consistent in order that the geotechnical/fluid properties can be reliably used for design of the disposal facility. Therefore, the sand and slimes component of the tailings combined would have different rheological and geotechnical properties than each of the materials if separated. This would mean that the backfilling underground at Tara would also have to be paste and therefore a backfill plant would be required at the mill site as well as at the TMF.

There are a number of negative key issues relating to paste disposal in Stage 6 as discussed above. These are given below;

- potential for dusting of the paste surface;
- requirement for two paste plants at the TMF site and also to replace Tara's existing backfilling operation underground using cyclone coarse sands with paste; and
- operating the paste plant at the TMF site 24 hours a day, seven day a week with the associated noise and lighting.

Filter cake disposal has many of the advantages and disadvantages that apply to paste disposal. It is by far the safest option for tailings disposal and is reasonably commonly used in arid climates where water is a premium, the reclaimed water has a value and the filter cake can be dried in the heat and compacted. In Ireland, trafficking on filter cake, when wet, is very problematic. At Aughinish, where filter cake is produced, it is wetted up to form paste and pumped to the facility. The filter cake plant would have to be located at the TMF site and the material could either be placed by a conveyor system or trucks. The filter cake plant site, road ways and the placement areas would need to be lit up continuously.

Either paste or filter cake disposal operations would impact the neighbours due to light and noise pollution for the life of the facility. These options have been discussed with the stakeholders during previous consultation meetings and none would prefer a plant operating at the tailings sites 24 hours a day seven days a week. Currently, with slurry disposal, there is the minimum of activity on the TMF.

It was considered therefore that the negative aspects of surface paste or filter cake disposal outweighed the benefits and these methods of disposal were rejected in favour of conventional slurry discharge into Stage 6.

Item 4

It is noted that, as part of the EIS, a preliminary appraisal was carried out to address flood risk within the stage 6 TMF and environs. Section 3.9 of the EIS states that "there is adequate control of the tailings water level which can be achieved by adjusting the discharges into the TMF and removal of tailings water by pumping from the TSF to the plant site". Clarify why neither an emergency spillway nor the capability to store probable maximum precipitation of appropriate duration (months, not hours), is provided.

Response to Item 4

The stage 6 TMF is classified as a perimeter dam with no external catchment area. The facility will be operated with a 1 m freeboard. It is normal to operate a perimeter dam with no spillway and has been the case with Stages 1, 2, 3, 4 and 5 of the facility.

Similarly, tailings facilities at the Galmoy and Lisheen were operated without a spillway.

A spillway is not required because the majority of the water discharged into the facility is controlled by pumping and managed by Tara Mines. Thus the facility is operated by a 1m freeboard and if exceeded Tara mines has the option to stop pumping into the facility or increase pumping from the facility provided discharge criteria is meet.

The 1 m freeboard would accommodate the total average annual rainfall over the last 50 years (856 mm) and the maximum annual rainfall over the last 50 years (1136 mm).

Spillways will be installed at the end of the life of the facility as was the case at Galmoy and Lisheen facilities to protect the dam walls from being overtopped in the long term at closure (Refer to Section 6.15 of *Design for the Stage 6 Tailings Management Facility* report submitted as part of this IE License Review).

Item 5

The likelihood of seepage through the liner is discussed in section 3.5 of the EIS. However, no seepage analyses model and 2D sections through the proposed TMF have been provided, which are required to estimate phreatic surface for use in stability modelling.

a) *Please provide a seepage analyses model and 2D sections through the proposed TMF as part of the stability analysis.*

b) *Provide explanation why there is no allowance for an over liner and underliner drainage as part of seepage control during operation.*

Response to Item 5 (a)

Seepage from the TMF will be controlled by the low permeability composite lining system, and the low permeability of the tailings retained by the facility. On the upstream dam wall face there is Type A1 and A2 low permeability glacial till. Experience of a large number of quality assured and controlled geomembrane installations indicates the likely presence of between 2 and 5 leaks per hectare and these defects are generally less than 10 mm^2 in size.

Seepage calculations have been based on the design equations given in References 1 and 2 below for the worst case, when the facility is filled with tailings.

Ref 1 Leakage through Liners Constructed with Geomembranes. Part II. Composite Liners. J. P. Giroud, et al., Geotextiles and Geomembranes, 8(4) pp71-111 1989.

Ref 2 Rate of Leakage through a Composite Liner due to Geomembrane Defects. J. P. Giroud, et al., Geotextiles and Geomembranes, 11(1) pp1-28 1992

These equations also over estimate seepage flow from a lined tailings facility as any defect is choked by the tailings particles reducing the seepage significantly.

The volume of seepage flowing laterally through the dam wall via the GCL and some nominal defects in the lining system for a constant head of 20m (Phase 2) would be an average of about 1.5 m^3/day with a 10% probability of the seepage being less than 0.25 m^3/day and a 10% probability being greater than 5 m^3/day. The range represented in the figure below, is based on a number of variables including infiltration rate based on the range of vertical permeability values of the tailings, the range of defects in the lining per hectare, the effective permeability of the composite lining and head acting on the lining.

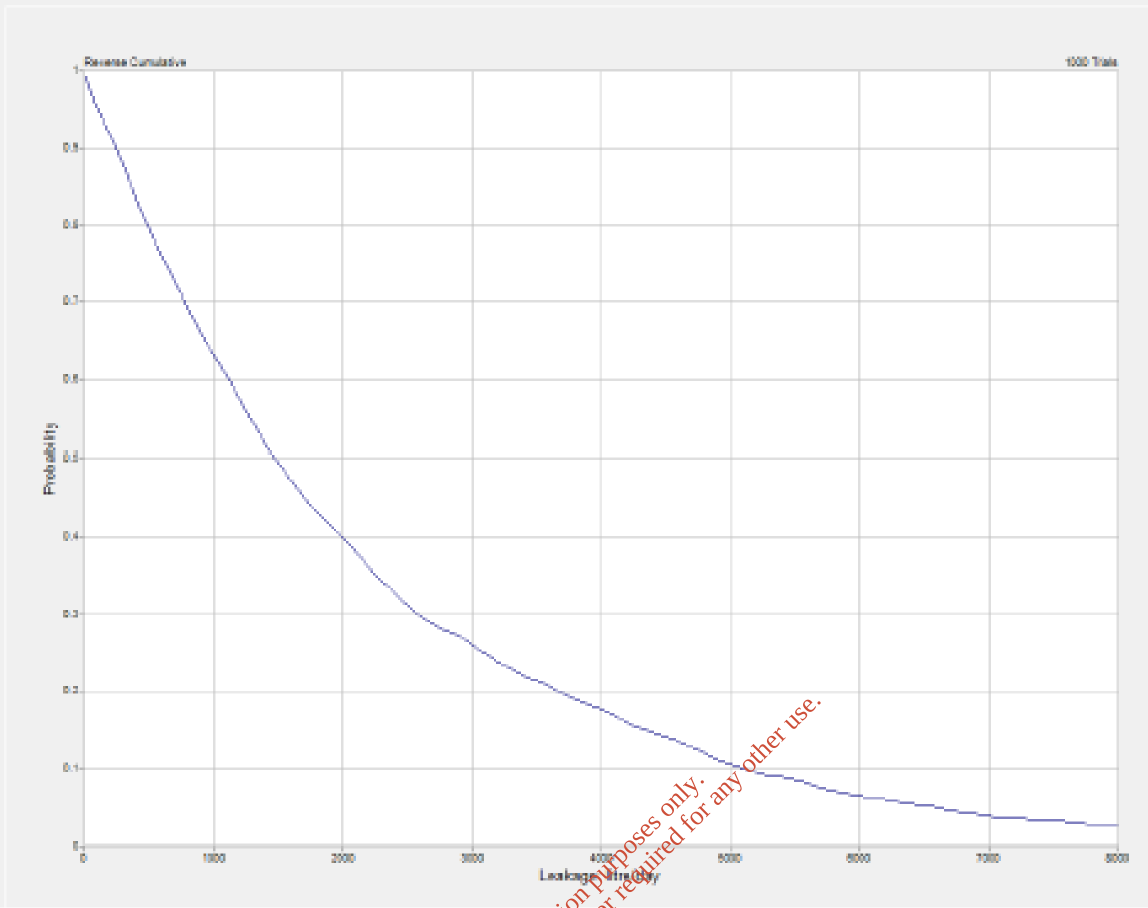

Figure 1: Probability of Seepage from Composite Lining System

The dam wall incorporates a basal granular drainage blanket and the downstream dam shell has been modified to be granular. Thus any seepage through the dam wall will be directed to the drainage blanket. Seepage through the basin will pass into the underlying limestone. It is not therefore not appropriate to undertake a seepage model under these circumstances.

Response to Item 5 (b)

Neither the main composited lined cells at Lisheen or the Phase 1 Extension and Phase 2 cells at Aughinish had over or under drainage systems.

Overdrainage:

There are a number of reasons why an overliner was not proposed detailed below:

- Unlike domestic land fill facilities, the tailings are a very low permeability material. Placing a drainage blanket above the lining would allow the drainage water to find and recharge any defects in the lining system.

- Most damage to a lining system in landfills is a consequence of placing the drainage blanket.
- The water storage capacity of the drainage system is relatively small and the recharge to the system via the low permeability tailings is slow. Thus pumps, which would have to pump over the dam wall would rapidly pump the drainage system followed by a long pause as the drainage blanket recharged.
- The basin area of the TMF is significantly greater than those used at domestic land fill sites. These are typically 1 Ha to 5 Ha. The basin area of the TMF is 43 Ha and would be difficult to shape with constant grades to a sump.
- Seepages are comparatively very low compared to the adjoining tailings facilities (Stages 1 to 5) which are lined with glacial till or clay blanket.

Underdrainage

Much of the footprint of the Stage 6 TMF is bed rock ranging from Lower Palaeozoic in age through to Lower Carboniferous Limestone.

The Lower Palaeozoic rocks are divided into four main rock types, consisting of Lower Palaeozoics, Red Beds (Old Red Sandstone), Mixed Beds and Pale Beds. The rocks are predominantly siltstones, sandstones, shales and greywackes with quartz conglomerate horizons. These Lower Palaeozoic rocks occur at rock head level to the east and north of the tailings facility and in a 500 m wide zone beneath the site, to the west side of the Randalstown fault. The fault cuts from northeast to southwest across the site.

The Pale Beds or the Meath Formation comprise a variety of pelletal, oolitic and bioclastic calcarenites which locally contain quartz sand and darker argillaceous layers. They are a dominantly massive group and are characterised by their low shale content. The base of the Pale Beds is defined by a pale micritic limestone known as the Micrite Unit, which varies greatly in thickness. The Pale Beds vary from 0 m to 140 m thick across the Randalstown area.

All rocks are fractured and the mass permeability exceeds the seepage derived from the low permeability of the tailings combined with any defects in the HDPE component of the composite lining. Thus an underliner drainage system would not collect the seepage and the seepage would pass vertically downwards and be diluted by the ground water.

A limited underdrainage system is incorporated to control groundwater during construction and particularly during the winter months in which the water table will rise. The main drainage system which exits in the north-west corner of the site will be monitored for water quality.

Item 6

Provide a slope stability analysis that addresses the following.

a) Short term (following construction) scenario.

b) Long term (TMF development and closure) scenario.

Response to Item 6(a)

Short term stability analysis was undertaken using a cohesion of 40 kPa and 60 kPa with no increase in strength with loading and with factors of 0.13 and 0.25 increase with increasing effective overburden pressure. Further analysis was undertaken using effective strength parameters and excess pore pressure as ru values. Results are presented in Attachment of FIR item 6.

Stability Modelling Results Short Term

Case	Condition	Location	FOS
1	Short term undrained shear strength of Type A1 and A2 materials 40kPa.	Upstream	
2	Short term undrained shear strength of Type A1 and A2 materials 60kPa.	Upstream	
3	Short term undrained shear strength of Type A1 and A2 materials 40kPa increasing strength factor of 0.13.	Upstream	
4	Short term undrained shear strength of Type A1 and A2 materials 40kPa increasing strength factor of 0.25.	Upstream	
5	Short term undrained shear strength of Type A1 and A2 materials 60kPa increasing strength factor of 0.13.	Upstream	
6	Short term undrained shear strength of Type A1 and A2 materials 60kPa increasing strength factor of 0.25.	Upstream	
7	Short term effective shear strength of Type A1 and A2 materials 32 degrees ru 0.25	Upstream	
8	Short term effective shear strength of Type A1 and A2 materials 32 degrees ru 0.45	Upstream	

Response to Item 6(b)

Long-term downstream embankment stability, with tailings level 1 m below the crest were undertaken for the HDPE liner intact and an assumed phreatic surface was drawn down through the embankment by the downstream drainage blanket and the rock used in downstream construction.

Stability analysis were undertaken in the long term assuming the HDPE had failed, the drainage system was no longer operational and the phreatic surface exits the dam wall in the slope at one third of its height. This is a very extreme scenario which is unlikely to develop. Further analyses were undertaken for the long term using pseudo-static conditions corresponding to a 0.06 g acceleration for the liner intact.

Results of the stability analyses are presented in the *Design for the Stage 6 Tailings Management Facility* report Section 6.14.4. This report is given again as Attachment to FIR item 6:

Stability Modelling Results Long Term

Case	Condition	Location	FOS
1	Long-term, tailings 1m below crest, phreatic surface at the toe of the dam. Static condition	Downstream	1.63
2	Long-term, tailings 1m below crest, phreatic surface at the toe of the dam. Ground Acceleration 0.06g. Pseudo static condition.	Downstream	1.38
3	Long-term, tailings 1m below crest, phreatic surface exists 1/3 dam height. Static condition.	Downstream	1.16
4	Long-term, tailings 1m below crest, phreatic surface exists 1/3 dam height. Ground Acceleration 0.06g. Pseudo static condition.	Downstream	0.99

Item 7

In relation to the description of the composite lining system:

(a) Provide specifications for the 500 and 1000g/m^2 geotextiles proposed

(b) Justify the need for the 1000g/m2 geotextile (e.g. is HDPE puncture of concern)?

(c) Clarify the term carbon-rich as it applies to the geotextile.

(d) Section 3 of the EIS made reference to leak detection. Clarify if a leak detection and collection system will be installed as part of the lining system.

Response to Item 7 (a)

A 1000 g/m2 non-woven needle punched geotextile is required to protect the GCL and HDPE geomembrane from the underlying Type C material which in turn is placed on top of the Type B material. A carbon rich 500 g/m2 non-woven needle punched geotextile is required in the anchor trench and on the surface of the HDPE to protect this material from the surcharge and movement of pipe work.

The physical and mechanical properties of the 1000 grm/m2 and 500 grm/m2 non-woven needle punched geotextile are presented in the *Design for the Stage 6 Tailings Management Facility* report Section 6.7.2:

Non-Woven Geotextile Properties

Parameter	**Specification 1000 g/m²**	**Specification 500 g/m²**
CBR Puncture Resistance	Minimum 10,000 N	Minimum 5,000 N
Wide Width Tensile Strength	Minimum 75 kN/m	Minimum 40 kN/m
Elongation at break	Minimum 50%	Minimum 50%
Thickness	Minimum 8.0 mm	Minimum 5.0 mm
Mass per unit area	Minimum 1000 g/m²	Minimum 500 g/m²

Response to Item 7 (b)

The use of 1000 g/m2 is a conservative approach but ensures with the protection layer that neither the GCL or HDPE would be punctured by any sharp protrusion present in the sub grade material.

Response to Item 7 (c)

This is a term to describe a black non-woven 500 grms/m^2 geotextile which has better resistance to ultra violet light if it were exposed.

Response to Item 7 (d)

A leak detection survey using DC electric current will be undertaken after the installation of the lining system and prior to commissioning. This geophysical method was used successfully for the Lisheen and Aughinish cells. An electric current is passed between two electrodes, one placed in either water ponded in the cell or by a water spray jetted onto the lining and the other in the peat outside the cell. With the geomembrane intact, the water in the cell will be electrically isolated from the external environment. The resulting potential field, measured as a difference between two non-polarising electrodes, is small but uniformly distributed over the geomembrane. If the geomembrane is defective, current will flow through the point of leakage and the measured potential will peak around the position of the defect.

Data acquisition is performed within the cell on a predetermined grid marked on the geomembrane at 2.0 m spacing. Two or more sets of data are obtained simultaneously, with information automatically stored on data loggers where possible. The results are processed and plotted on site, then overlaid on the plan of the cell to allow the immediate detection and location of the defect. The voltage used is 240 V and there is a strict safety protocol to follow to ensure no connection is made between the personnel, the ground and the water during the data gathering phase.

As the leak detection survey proposed will find all defects no permanent leak detection system will be installed underneath the lining. It should be noted that once tailings are placed in the facility no repairs of the lining system could be undertaken beneath the tailings. It also should be noted that at 2 m intervals, as per the leak detection survey, the amount of permanent cable to be installed over the entire TMF footprint would be in the order of 600 km.

A perimeter interceptor channel will be installed although virtually all the water collected would be surface water runoff from the dam wall and ground water.

Item 8

It is stated that the Stage 6 TMF extension will be operated in a manner similar to the existing TSF by discharging the tailings from spigots on the dam crest.

a) Provide a tailings deposition plan detailing the layout of the tailings pipeline, deposition points, decant location and technical specifications of the pipeline (including proposed pipe cross-section and long section).

b) Provide an explanation why burying the tailings pipeline is preferred to the over-ground piping, in relation to containment in the event of leakage.

Response to Item 8(a)

The tailings deposition systems for the Stage 6 crest are detailed in Figure 2, including spigot locations and long section (figure 3). The main header pipeline carrying the tailings is 800 mm OD, 559 mm ID, PE100 and SDR17. The spigot spacings will be at 75m. The tailings discharge pipe line at the spigots is 200 mm OD, 176 mm ID, PE 100 and SDR17. A typical spigot point is presented in Figure 4 below

During normal operation, tailings are cycloned to produce a coarse fraction used as backfill hydraulic fill for the mine. The fine fraction is pumped to the TSF. On occasion when hydraulic fill is not required by the mine, the processing plant tailings are pumped in total to the TSF.

Discharge of the total tailings fraction is from a single point, through a dedicated 800 mm OD pipeline of specification as described above. Discharge of this fine fraction occurs from 3 to 4 spigots at any one time for a period of one week and the tailings are deposited in layers. Discharge is rotated systematically along the dam wall crest.

The reclaim pump system is located in the south east corner and the water will be pumped back into the existing reclaim water pumping system.

Response to Item 8(b)

The header pipeline is buried on the crest to prevent excessive movement due to temperature related expansion/contraction of the pipe, protection from vehicle damage and to act as a landscape berm.

The fact that the pipeline is buried has no application to containment in the event of a leakage.

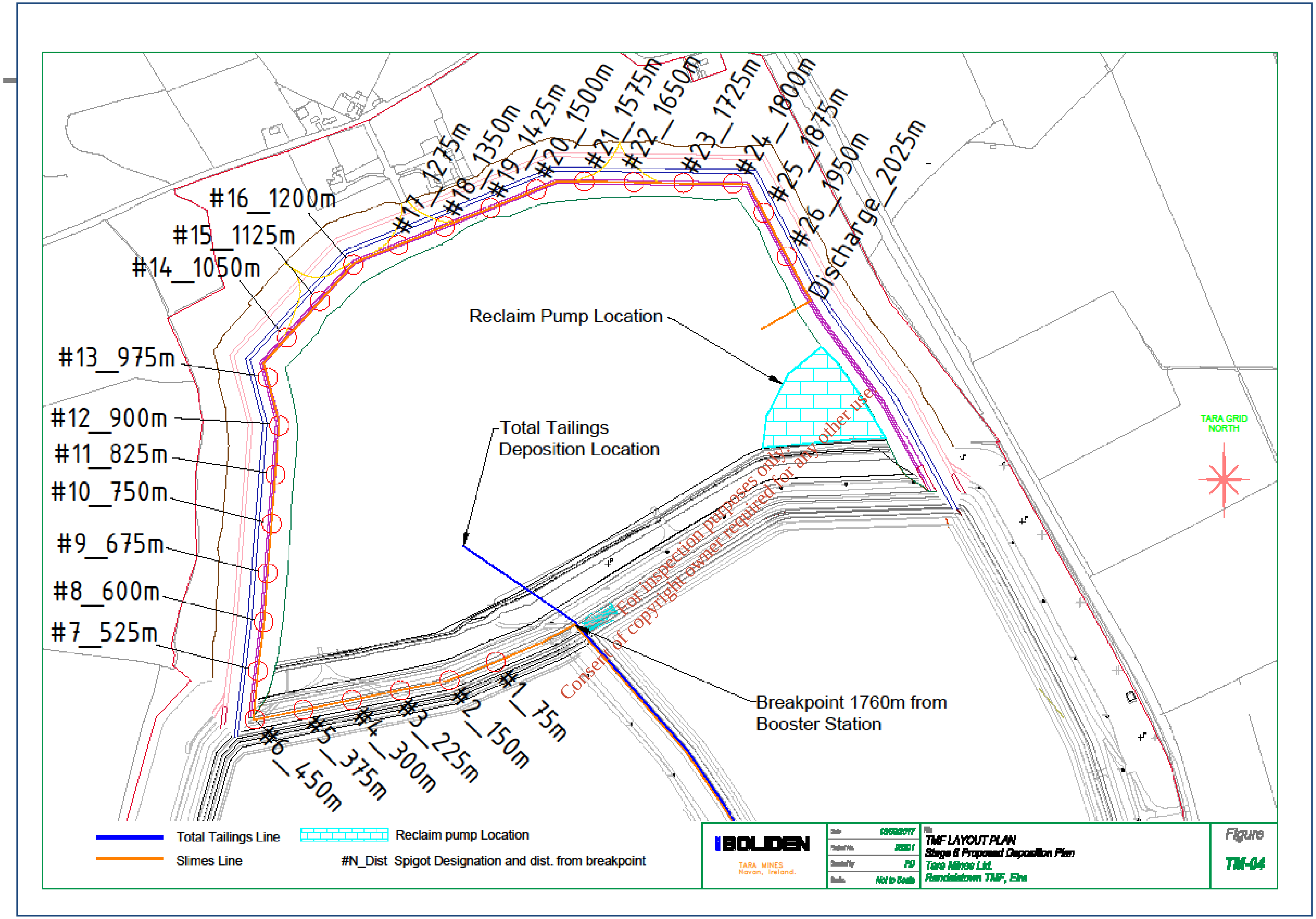

Figure 2 Stage 6 Tailings Deposition Plan

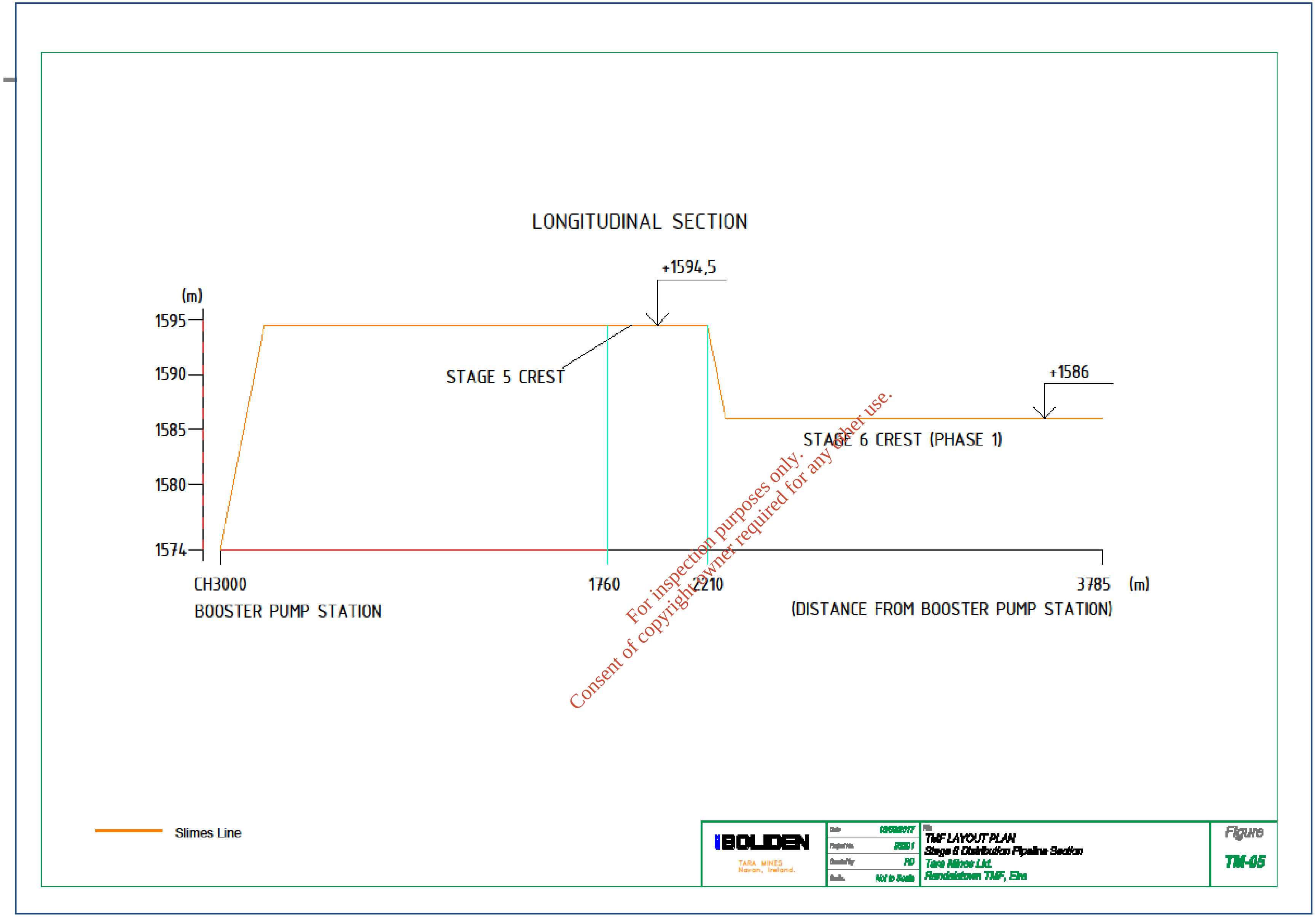

Figure 3 Stage 6 Layout Longitudinal Section

Figure 4 Typical Spigot Point

Item 9

Three sources of water that are considered as process water include water ingress to the mine; water from processing plant; and surface runoff.

(a) Clarify the statement, "All water from the process plant is pumped to the tailings facility".

(b) Provide a detailed explanation why collection systems for clean runoff and for seepage are not separate.

(c)Provide a detailed explanation why the interceptor channel used to collect and discharge seepage water into the water treatment system is not fully lined.

Response to Item 9(a)

The concentrator / processing water system is a closed loop system. All water streams (process water, storm water, runoff) drains centrally to the Mill tailings box and are pumped to the tailings storage facility as part of the 'tailings stream' for treatment and is subsequently returned to the reclaim pond on site for re-use.

This water is then either reused in the concentrator or further treated and blended in the sediment-aeration ponds prior to discharge to the River Boyne at Emission Point Reference SW1.

Response to Item 9(b)

As explained in response to item 5, as the facility is composite lined, the seepage is very low and virtually all the water collected in the perimeter interceptor channel will be derived from rainfall and ground water. Much of the water emanating from the basal drainage blanket will have derived from rainfall infiltrating the downstream shoulder of the dam wall.

Response to Item 9(c)

Water collected from the perimeter interceptor channel will be pumped back to the tailings facility. The interceptor channel will not be operated for water storage.

However, if monitoring indicates contaminates in excess of the discharge limits in the interceptor channel then the perimeter interceptor would be lined.

Item 10

It is proposed that the existing water treatment system will be used for both Stage 5 and Stage 6 TMF until an integrated constructed wetland system is put in place. Provide detailed information on the capability and suitability of the existing water treatment to accommodate the increased volumes following Stage 6 start-up and prior to Stage 5 decommissioning.

Response to Item 10

Process water flow from the TSF will not increase with the commissioning of Stage 6 as the combined inputs to both systems will remain the same.

All process water eminating from the TSF will continue to enter the water management system as at present. A wetlands water treamtment system will be utilised on decommissioning of Stage 5.

Item 11

It is noted that no tailings flow slide analysis is provided in the EIS in relation to TMF dam failure. Provide an assessment of the potential impact on the environment in the event of a TMF dam failure.

Response to Item 11

A dam break analysis for the existing and Stage 6 tailings facility has been undertaken and is presented in Appendix to FIR item 11.

Appendix to FIR Item 1

- Stage 6 TMF Footprint Palaeokarst Site Investigation and Grouting Works (Golder Associates)

September 2017

BOLIDEN TARA MINES LTD

Stage 6 TMF Footprint Palaeokarst Site Investigation and Grouting Works

Submitted to:
Boliden Tara Mines Ltd
Knockumber
Navan
Co Meath

REPORT

Report **Number** 1782163.R01.A0

Distribution:

Boliden Tara Mines - one copy
Golder Associates Ireland Ltd - one copy

Golder Associates

Study Limitations

IMPORTANT: This section should be read before reliance is placed on any of the opinions advice, recommendations or conclusions herein set out.

a) This report has been prepared for and at the request of Boliden Tara Mines Ltd ("the Client or Tara") for the purpose of undertaking a drilling investigation to its appointment of Golder Associates Ireland Limited (Golder) to act as Consultant;

b) Save for the Client no duty is undertaken or warranty or representation made to any party in respect of the opinions, advice, recommendations or conclusions herein set out;

c) Regard should be had to the agreements between Golder and the Client from the proposal and the subsequent Terms of Appointment issued by Golder;

d) All work carried out in preparing this report has used, and is based upon Golder's professional knowledge and understanding of the current relevant Irish, UK and European Community legislation.

   Changes in the legislation may cause the opinion, advice, recommendations or conclusions set out in this report to become inappropriate or incorrect. However, in giving its opinions, advice, recommendations and conclusions, Golder has considered pending changes to environmental legislation and regulations of which it is currently aware. Following delivery of this report, Golder will have no obligation to advise the Client of any such changes, or of their repercussions;

e) Golder acknowledges that it is being retained, in part because of its knowledge and experience with respect to engineering and environmental matters. Golder will consider and analyse all information provided to it in the context of its knowledge and experience and all other relevant information known to Golder. To the extent that the information provided to Golder is not inconsistent or incompatible therewith, Golder shall be entitled to rely upon and assume, without independent verification, the accuracy and completeness of all such information and Golder shall have no obligation to verify the accuracy and completeness of such information;

f) The content of this report represents the professional opinion of experienced engineering and environmental consultants, Golder does not provide specialist legal advice and the advice of lawyers will be required;

g) If the scope of the work includes borings, trial pits, or engineering interpretation of such information attention is drawn to the fact that special risks occur whenever engineering and related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with a professional Standard of Care may fail to detect certain conditions. The environmental, geologic, geotechnical, geochemical and hydrogeological conditions that Golder interprets to exist between sampling points may differ from those that actually exist. Passage of time, natural occurrences, and activities near the Site may substantially alter discovered conditions;

h) In the conclusion section of this report, Golder has set out its findings and provided a summary and overview of its advice, opinions and recommendations, however, other parts of this report will often indicate the limitations of the information obtained by Golder and therefore any advice, opinions or recommendations set out in the conclusion section ought not to be relied upon until considered in the context of the whole report; and

i) The contents of this report includes confidential information and should not be disclosed to third parties without prior written approval from Golder.

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1.0 BACKGROUND AND INTRODUCTION

Golder Associates (Golder) was retained by Boliden Tara Mines Ltd (Tara) to investigate potential palaeokarst features, interpreted during recent geophysics survey works (Electrical Resistivity Imaging (ERI), Seismic Refraction, EM31 Electromagnetic-Conductivity and Micro-Gravity), within the footprint of the proposed Stage 6 Tailings Management Facility (TMF) (the 'Site'). The 'Site' is located to the north of the existing TMF at Randalstown, Co. Meath (Figure 1).

Figure 1: Aerial Photograph of the Proposed Stage 6 TMF (Northern and Seven Fields Borrow Area)

The footprint of the Stage 6 TMF is confined to the existing 'Northern and Seven Fields Borrow Areas'. During the development phases of previous raises of the main TMF, the Site was used as a Borrow Area for construction material and occupies some 40 hectares.

The bedrock underlying the proposed Stage 6 TMF footprint comprises of two main units:

- Pale Beds (Carboniferous Limestones); and
- Non-calcareous Lower Palaeozoic (LP) sediments (Rathkenny Formation).

The geological map of the area (supplied by Tara) in Figure 2 below shows that the majority of the footprint to the east (including the area known as the Seven Fields Borrow Area) is underlain by black mudstone, siltstone and greywacke of the Rathkenny Formation, which is Lower Palaeozoic in age.

Pale Beds (PB) are understood to underlie much of the south-western part of the Site and present a potential risk for palaeokarst features; a number of anomalies were interpreted from the recent geophysical survey works which are the focus of this site investigation. Karst is defined as a geological feature formed by the dissolution of soluble rocks, and palaeokarst is a term used for ancient and inactive karst areas that have been buried or filled by later sediments.

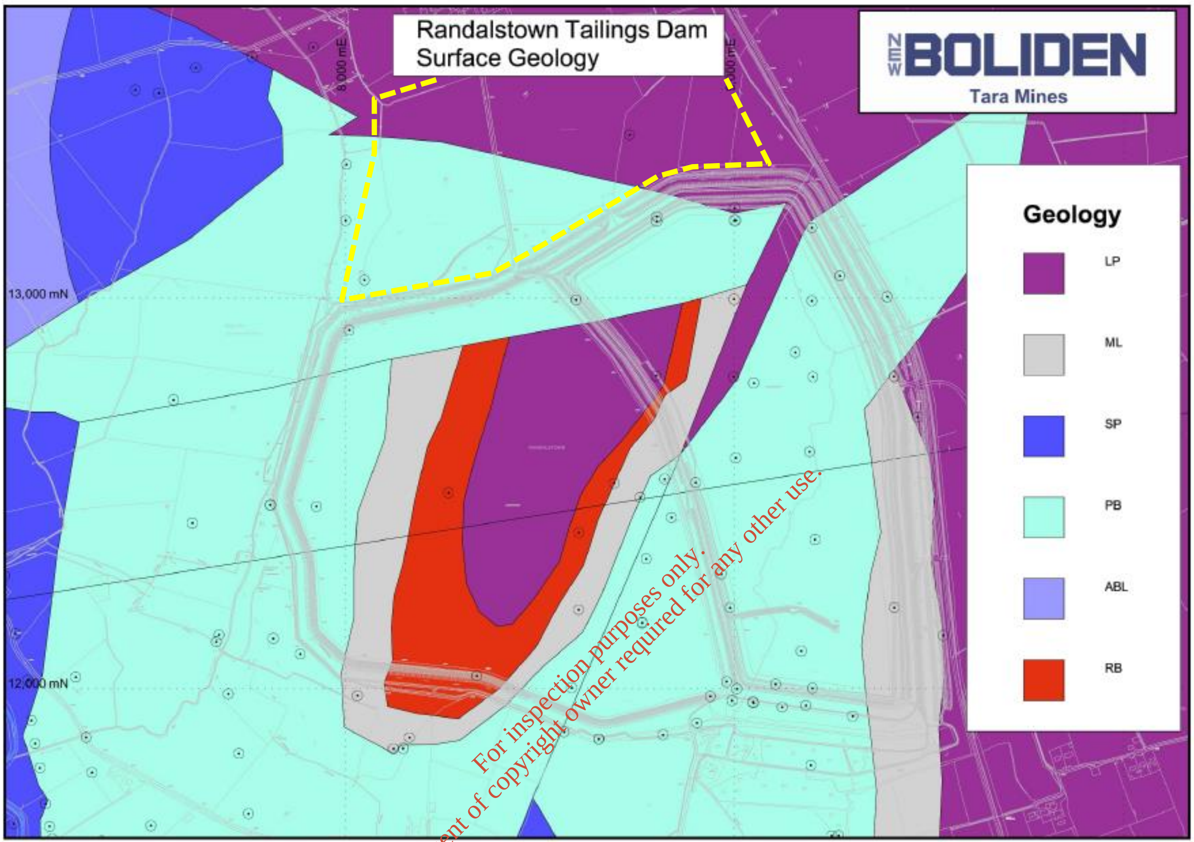

Figure 2: Geological Map of area around Tara TMF, Stage 6 TMF footprint is located to the north of the existing TMF (highlighted by yellow dashed line)

Golder have successfully undertaken surveys to locate and assess potential palaeokarst features at the Tara TMF and also at the Galmoy TMF in Co. Kilkenny, the Lisheen TMF in Co. Tipperary and at Aughinish Alumina Plant in Co. Limerick using a number of geophysical survey methods.

2.0 BOREHOLE TARGETING

The geophysical surveys undertaken over the proposed extension footprint for Stage 6 ('Borrow Area') of the TMF at Randalstown consisted of the following surveys (Golder Associates Report References 1775908.R01.B0 and 1775908.R02.B0):

- Seismic Refraction Survey – consisting of 102 profiles;
- Electrical Resistivity Imaging Survey (ERI) – consisting of 15 profiles,
- Micro-Gravity Survey – consisting of 2699 stations; and
- EM31 Electromagnetic-Conductivity Survey.

Results from the **Micro-Gravity** survey identified a number of gravity lows (Figure 3), indicating the possible presence of areas with *'low density sub-crop material'.* These gravity low areas may be due to palaeokarstic features within the underlying Pale Beds limestones. Modelling of the Micro-Gravity and comparison with the ERI pseudosections and Seismic Refraction profiles indicates that heavily weathered, fractured (and palaeokarstic) bedrock conditions may exist in the underlying Pale Bed limestones from near surface to depths of typically 40 to 50 m bgl. Five locations for boreholes, centred on the anomalies, were selected to test for the presence of palaeokarst features.

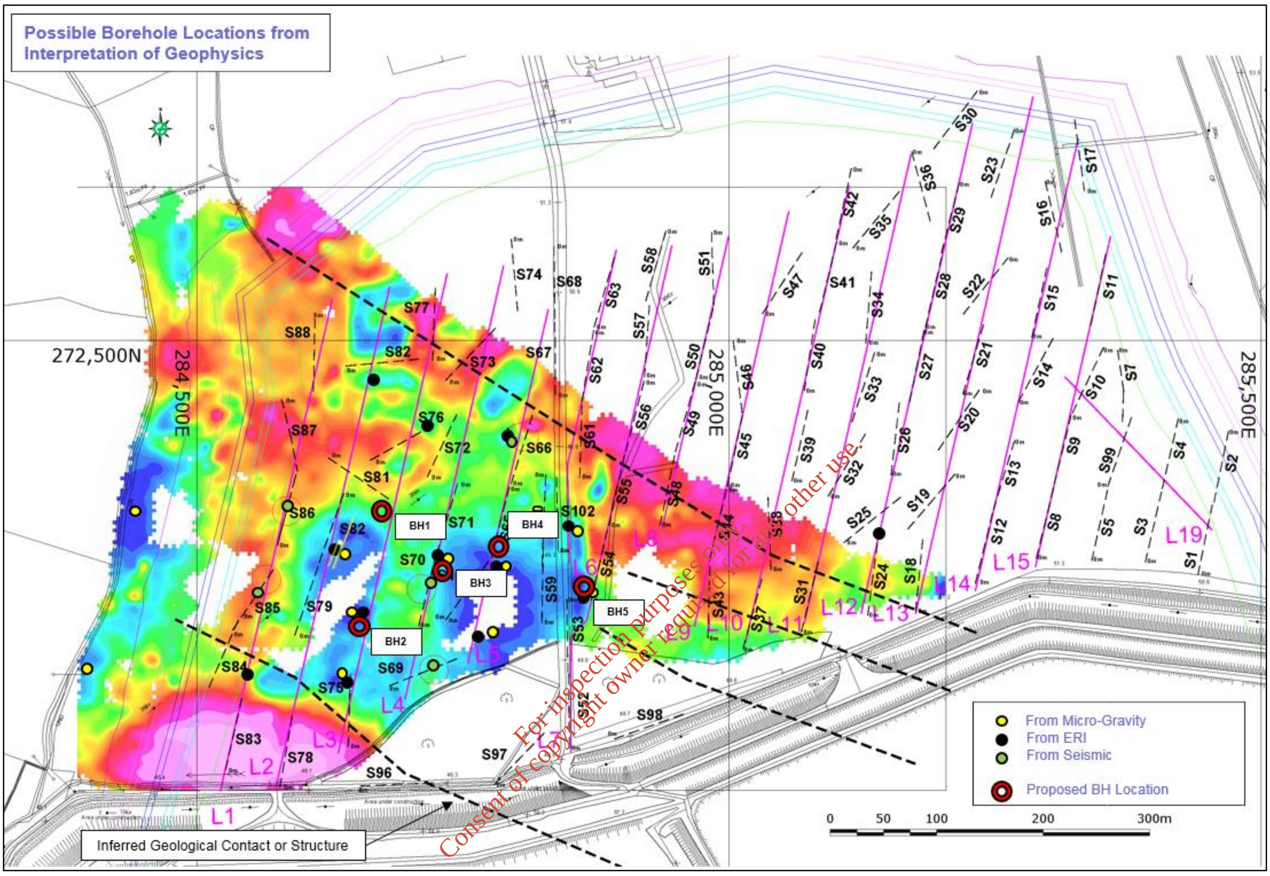

Figure 3: Residual Bouguer Gravity Map with Borehole Locations (red circles)

None of the anomalies interpreted are located beneath the dam wall footprint, all five are located in the south-west section of the base of the proposed Stage 6 TMF.

Previous site investigations (trial pitting) indicated that a thin layer of glacial till varying in thickness from 0.5 m to 5.0 m overlies much of the Site. In some instances overburden thicknesses in excess of 5 m occur, particularly overlying the Pale Beds in the western part of the Site. This material typically consists of grey clayey silty sands and gravels with boulders, and gravelly sandy clayey silts with boulders, and weathered brown clayey silty sands and gravels with occasional boulders. As the grey till approaches the underlying bedrock surface, the boulder content typically increases to over 50% of the material.

Figure 4 below presents the locations of the investigative boreholes plotted over a recent aerial photograph of the proposed Stage 6 extension to the TMF.

Figure 4: Location of Boreholes (plotted over recent aerial photograph of the Stage 6 TMF footprint)

Table 1 below presents the northing and easting coordinates for the boreholes in Irish National Grid along with the estimated depth of overburden from the seismic survey data.

Table 1: Coordinates for Proposed Borehole Locations in Irish National Grid

Borehole ID	Easting	Northing	Estimated Depth of Overburden from Geophysics
BH1	284634.2	272291.4	3.5 m
BH2	284653.1	272241.0	5.5 m
BH3	284733.0	272291.0	4.0 m
BH4	284783.4	272323.6	4.0 m
BH5	284870.9	272256.9	4.0 m

3.0 DRILLING AND CAMERA SURVEY RESULTS

The borehole drilling works were conducted by JS Drilling Ltd of Thomastown, Co Kilkenny with attendance provided by a Golder. Borehole locations were determined by an interpretation of the geophysical survey data and centred on the anomalies. A Golder geologist supervised the drilling works and completed geotechnical logs (to BS 5930:1999 + A2 2010) for each of the boreholes, which are provided in Appendix A. Following drilling each borehole, a camera survey was completed to assess for the presence of palaeokarst features.

A brief summary of each borehole is described below:

- **BH1** – Upper 6.0m comprised of made ground/overburden. Palaeokarst from 6m -17 m, in-filled with sands, gravels, and pebbles/cobbles, including a 2.5m section of weathered limestone. Water strike at 14.5m. Limestone bedrock encountered at 17.0m, with the borehole being terminated at **23.5m** (in bedrock).
- **BH2** – Upper 1.7m comprised of made ground/overburden. Palaeokarst from 1.7m - 40.3m, in-filled with sands, silts, clays, and gravels/cobbles. End of borehole at **40.3m** due drill rods becoming stuck.
- **BH3** - Upper 1.0m comprised of glacial till. Palaeokarst from 1.0m - 33.5m, in-filled with clays and gravels, becoming increasingly clay rich with depth. End of borehole at **33.5m** due to loss of air (very stiff clays).
- **BH4** – Upper 2.0m comprised of made ground/overburden. Weathered limestone from 2.0m - 18.5m, becoming less weathered at depth. Palaeokarst in-filled with clays, silt, sand, and rounded gravels from 18.5m - 23m. Limestone bedrock from 23.0m - 26.5m, becoming fractured at depth. Palaeokarst in-filled with clay and sand from 26.5m - 31m. End of borehole at **31m** due to drill rods becoming stuck.
- **BH5** – Upper 3.0m comprised of made ground, with glacial till between 3m -10m. Palaeokarst in-filled with clays, silts, and sands from 10m - 17m. End of borehole hole at **19.5m** due to drill rods becoming stuck.

4.0 GROUTING WORKS

Following from an assessment of the borehole logs and the camera surveys, the boreholes were decommissioned using gravity grouting techniques, proceeding from the bottom of the borehole to the top of the borehole. Four of the boreholes were infilled with a bentonite / cement grout mix to between 4m and 7 m below ground level and capped with a concrete plug while BH4 was infilled with the bentonite / cement grout mix to ground level.

The grout design mix selected had a ratio by weight of 8:3:1 for water: sodium bentonite powder: cement, respectively.

The supply water for the grout was filled into a tanker vehicle from a water tank used for the on-site truck wash. The grout was mixed in a high shear colloidal mixer to fully wet all the cement and bentonite particles prior to being pumped through the rods into the borehole. The percentage grout consumption ranged from 126% to 300% of the estimated extracted volume of the borehole by drilling. Additional grout consumption beyond 100% is attributed to minor voids created in the subsurface during the drilling of the boreholes. Table 2 provides a summary of the grouting works undertaken.

Table 2: Tara Mines Stage 6 Grouting Works

Borehole ID	Borehole Diameter (mm)	Borehole Total Depth (m) when drilled	Depth to top of Grout (m)	Volume of Grout Placed (m^3)	Est. Volume of Borehole for Grouting (m^3) ($V = d\pi r^2$)	% Grout Consumption	Concrete Volume used for Plug (m^3)
BH1	**100**	**23.5**	7	0.35	0.14	150	0.5
BH2	**100**	**40.3**	3.9	0.62	0.274	126	0.4
BH3	**100**	**33.5**	5.7	0.52	0.223	133	0.5
BH4	**100**	**31**	0	0.79	0.204	287	0
BH5	**100**	**19.5**	5	0.44	0.11	300	2.5

5.0 CONCLUSIONS

Five boreholes (BH1 to BH5) were drilled to depths of between 19.5m and 40.3 m centred over the interpreted geophysical anomalies to test for the presence of possible palaeokarst features in the underlying bedrock. The findings of the palaeokarst investigative drilling were in agreement with the results of the geophysical survey, indicating that the anomalies interpreted comprised of **infilled palaeokarst features**.

The boreholes intersected between ca. 1.0m - 6.0 m of glacial till/overburden underlain by weathered limestone and palaeokarst features infilled with sediments including clay/silt, sand, and rounded gravels (limestone and calcareous sandstones). Three boreholes were terminated before reaching bedrock due to drill rods becoming stuck. BH3 was terminated due to loss of air in stiff clays, and BH1 was terminated in limestone bedrock. Following drilling works, the boreholes were decommissioned with a bentonite-cement grout and capped with concrete.

The palaeokarst investigation did not identify any large voids beneath the footprint of the Stage 6 TMF Facility that might pose a risk to the integrity of the Stage 6 TMF Facility.

6.0 REFERENCES

1) Golder Associates (May 2017). Boliden Tara Mines Limited: Stage 6 Footprint Geophysical Surveys, 1775908.R01.B0

2) Golder Associates (May 2017). Boliden Tara Mines Limited: Possible Borehole Locations – Pale Beds – Stage 6 (‘Borrow Area’), 1775908.R02.B0

Report Signature Page

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APPENDIX A

Geotechnical Borehole Logs

Golder Associates

BOREHOLE LOG

Project				BOREHOLE No
Stage 6 - Karst features Site Investigantion				**BH1**
Job No: 1782163 - Karst SI	Date:	Ground Level (m): 1574.02	Co-Ordinates (Tara Mine Grid): E 8,216.8 N 13,272.8	
Contractor: JS Drilling				Sheet: 1 of 2

SAMPLES & TESTS / STRATA

Depth	Type No	Test Result	Water	Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	Geology	Instrument/ Backfill
				1571.02		(3.00) 3.00	MADE GROUND		
				1568.22		(2.80) 5.80	BOULDER CLAY		
				1565.52		(2.70) 8.50	PALEOKARST CAVITY - gravelly with rounded limestone pebbles BOULDER CLAY		
				1561.02		(4.50) 13.00	weathered LIMESTONE		
			1	1557.52		(3.50) 16.50	PALEOKARST CAVITY - gravelly BOULDER CLAY some cobbles		
				1557.02		17.00	PALEOKARST CAVITY - sands and gravels		
						(6.50)	Competant grey LIMESTONE BEDROCK		

Boring Progress and Water Observations

Date	Time	Depth	Casing Depth	Casing Dia. mm	Water Dpt

Chiselling / Water Added

Chiselling From	Chiselling To	Hours	Water Added From	Water Added To

GENERAL REMARKS

End of Hole at 23.5m - competant BEDROCK

All dimensions in metres Scale 1:143.75	Client: New Boliden - Tara Mines	Method/ Plant Used: Beretta T44	Logged By: MBD

BOREHOLE LOG

Golder Associates

Project				BOREHOLE No
Stage 6 - Karst features Site Investigantion				**BH1**
Job No 1782163 - Karst SI	Date	Ground Level (m) 1574.02	Co-Ordinates (Tara Mine Grid) E 8,216.8 N 13,272.8	
Contractor JS Drilling				Sheet 2 of 2

SAMPLES & TESTS			Water	STRATA				Geology	Instrument/ Backfill
Depth	Type No	Test Result		Reduced Level	Legend	Depth (Thick-ness)	DESCRIPTION		
				1550.52					

Boring Progress and Water Observations						Chiselling			Water Added		GENERAL REMARKS
Date	Time	Depth	Casing Depth	Casing Dia. mm	Water Dpt	From	To	Hours	From	To	
											End of Hole at 23.5m - competant BEDROCK

All dimensions in metres Scale 1:143.75	Client New Boliden - Tara Mines	Method/ Plant Used Beretta T44	Logged By

Golder Associates

BOREHOLE LOG

Project				BOREHOLE No
Stage 6 - Karst features Site Investigantion				**BH2**
Job No: 1782163 - Karst SI	Date	Ground Level (m): 1574.11	Co-Ordinates (Tara Mine Grid): E 8,220.2 N 13,186.1	
Contractor: JS Drilling				Sheet: 1 of 1

SAMPLES & TESTS: Depth	Type No	Test Result	Water	Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	Geology	Instrument/ Backfill
				1572.41		1.70	MADE GROUND		
				1571.11		3.00	PALEOKARST CAVITY - sands and pebbles		
				1567.91		(3.20) 6.20	PALEOKARST CAVITY - very fine grey silts and lac clays		
				1566.81		7.30	PALEOKARST CAVITY - boulder clay with cobbles		
			1	1565.11		9.00	PALEOKARST CAVITY - gravelly boulder clay with cobbles		
				1564.11		10.00	PALEOKARST CAVITY - gravelly boulder clay with cobbles		
				1535.61		(28.50) 38.50	PALEOKARST CAVITY - clay with sands and gravels		
				1533.81		40.30	PALEOKARST CAVITY - dead sand		

Boring Progress and Water Observations: Date	Time	Depth	Casing Depth	Casing Dia. mm	Water Dpt	Chiselling: From	To	Hours	Water Added: From	To	GENERAL REMARKS
											End of Hole at 40.3m - rods becoming stuck

All dimensions in metres Scale 1:251.875	Client: New Boliden - Tara Mines	Method/ Plant Used: Beretta T44	Logged By

Golder Associates

BOREHOLE LOG

Project				BOREHOLE No
Stage 6 - Karst features Site Investigantion				BH3
Job No: 1782163 - Karst SI	Date:	Ground Level (m): 1571.37	Co-Ordinates (Tara Mine Grid): E 8,294.3 N 13,245.7	
Contractor: JS Drilling				Sheet: 1 of 1

SAMPLES & TESTS			Water	STRATA				Geology	Instrument/ Backfill
Depth	Type No	Test Result		Reduced Level	Legend	Depth (Thickness)	DESCRIPTION		
				1570.37		1.00	Boulder black LIMESTONE		
				1558.37		(12.00) 13.00	PALEOKARST CAVITY - clay with boulders		
				1557.17		14.20	PALEOKARST CAVITY - gravel, with lots of water inflow (a "slurry")		
				1550.37		(6.80) 21.00	PALEOKARST CAVITY - orangy/brown clay		
				1548.27		(2.10) 23.10	PALEOKARST CAVITY - stiff clay		
				1545.17		(3.10) 26.20	PALEOKARST CAVITY - stiff clay, minor clasts		
				1539.67		(5.50) 31.70	PALEOKARST CAVITY		
				1539.17		32.20	PALEOKARST CAVITY - very stiff grey clay		
				1537.87		33.50	PALEOKARST CAVITY - very stiff grey clay		

Boring Progress and Water Observations						Chiselling			Water Added	
Date	Time	Depth	Casing Depth	Casing Dia. mm	Water Dpt	From	To	Hours	From	To

GENERAL REMARKS

End of Hole at 33.5m - rods pulled due to loss of air. Very stiff clays

All dimensions in metres Scale 1:209.375	Client: New Boliden - Tara Mines	Method/ Plant Used: Beretta T44	Logged By

Golder Associates

BOREHOLE LOG

Project				BOREHOLE No
Stage 6 - Karst features Site Investigantion				**BH4**
Job No	Date	Ground Level (m)	Co-Ordinates (Tara Mine Grid)	
1782163 - Karst SI		1572.54	E 8,336.5 N 13,276.7	
Contractor				Sheet
JS Drilling				1 of 1

SAMPLES & TESTS: Depth	Type No	Test Result	Water	STRATA: Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	Geology	Instrument/ Backfill
				1570.54		(2.00) 2.00	MADE GROUND		
				1567.24		(3.30) 5.30	fractured LIMESTONE with silts		
				1564.24		(3.00) 8.30	LIMESTONE - very heavily weathered in areas		
				1560.84		(3.40) 11.70	As above, but harder with depth		
				1560.04		12.50	As above, but harder with depth		
				1554.04		(6.00) 18.50	As above, but harder with depth		
			1	1549.54		(4.50) 23.00	PALEOKARST CAVITY - clays, silts, sands and rounded gravels		
				1549.04		23.50	very hard LIMESTONE		
				1547.04		(2.00) 25.50	very hard LIMESTONE		
				1546.04		26.50	soft - broken highly fractured LIMESTONE		
				1541.54		(4.50) 31.00	PALEOKARST CAVITY - clay filled with minor sand		

Boring Progress and Water Observations						Chiselling			Water Added		GENERAL REMARKS
Date	Time	Depth	Casing Depth	Casing Dia. mm	Water Dpt	From	To	Hours	From	To	
											End of Hole at 31.0m - rods becoming stuck

All dimensions in metres Scale 1:193.75	Client: New Boliden - Tara Mines	Method/ Plant Used: Beretta T44	Logged By

BOREHOLE LOG

Golder Associates

Project				BOREHOLE No
Stage 6 - Karst features Site Investigantion				BH5
Job No: 1782163 - Karst SI	Date:	Ground Level (m): 1573.66	Co-Ordinates (Tara Mine Grid): E 8,435.6 N 13,235.1	
Contractor: JS Drilling				Sheet: 1 of 1

SAMPLES & TESTS			Water	STRATA				Geology	Instrument/ Backfill
Depth	Type No	Test Result		Reduced Level	Legend	Depth (Thick-ness)	DESCRIPTION		
				1570.66		(3.00) 3.00	MADE GROUND		
				1569.66		(1.00) 4.00	BOULDER CLAY		
				1566.66		(3.00) 7.00	BOULDER CLAY		
				1563.66		(3.00) 10.00	silty BOULDER CLAY		
			▼	1562.66		(1.00) 11.00	PALEOKARST CAVITY - wet silts		
				1559.66		(3.00) 14.00	PALEOKARST CAVITY - clay/silts, increase in water		
				1556.66		(3.00) 17.00	PALEOKARST CAVITY - grey/brown coarse sand		
				1554.16		(2.50) 19.50	PALEOKARST CAVITY - sands and gravels		

Boring Progress and Water Observations						Chiselling			Water Added	
Date	Time	Depth	Casing Depth	Casing Dia. mm	Water Dpt	From	To	Hours	From	To

GENERAL REMARKS

End of Hole at 19.5m - rods becoming stuck

All dimensions in metres Scale 1:121.875	Client: New Boliden - Tara Mines	Method/ Plant Used: Beretta T44	Logged By

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Appendix to FIR Item 2

- Assessment of potential for acid generation in deposited tailings

An Assessment of Potential for acid generation in deposited Tailings at Randalstown

Physical Composition

The tailings comprise slurry containing approximately 10-15% solids. The solids are composed of particles, with a significant percentage being less than 11 microns in size, and 90% less than 44 microns.

Chemical Composition

The chemical composition of the tailings solids reflects a combination of the chemistry of the host rock, the ore mineralogy and the metallurgical processes.

Chemical Reagents

A number of chemical reagents are added during mineral processing. Some of the reagents are consumed or degraded in the process circuit. Small residual amounts may enter the tailings stream and be deposited in the TMF, however they are present in low concentrations and the organic component degrades rapidly.

Element Analysis

The primary economic minerals occurring within the orebody are galena and sphalerite. Secondary minerals include pyrite, marcosite, baryte, and some sulphosalts. These secondary minerals, together with residual amounts of galena and sphalerite, occur in the tailings that remain after minerals processing. As mineralisation has occurred within a Lower Carboniferous limestone host rock, the greater mass of tailings comprises calcitic and dolomitic limestone.

Selective analyses of metals, metalloids, and non-metal components of the tailings are routinely performed by an external accredited laboratory as part of Tara Mines IEL.

Results of the analyses for 2016 are presented in Table 1:

The analyses are undertaken monthly for zinc, lead, arsenic, iron copper, mercury, cobalt, magnesium, sulphate and sulphide. Cyanide and pH are monitored weekly.

The tailings samples comprise significant concentrations of calcium and magnesium representative of the calcitic and dolomitic limestones from which they are derived. They also contain a few trace metals in concentrations which exceed 5 x crustal abundance. These include zinc, lead, arsenic, mercury and copper. The near-neutral pH indicates metal mobility will generally be low.

Table 1 Tailings Solid Chemistry 2016

Parameters		pH	Total Cyanide (mg/kg)	Weak Acid Dissociable Cn	Zinc (mg/kg)	Lead (mg/kg)	Arsenic (mg/kg)	Iron (mg/kg)
Tailings Solids	Average	7.9	<1.1	<3.4	2,410	1,690	531	21,173
	Minimum	7.7	<1.1	<3.4	1,470	733	302	13,300
	Maximum	8.3	<1.1	<3.4	3,130	2,590	832	30,300
	Median	8.0	<1.1	<3.4	2,420	1.640	442	20,600
Parameters		**Copper (mg/kg)**	**Mercury (mg/kg)**	**Cobalt (mg/kg)**	**Calcium (mg/kg)**	**Magnesium (mg/kg)**	**Sulphate (mg/l)**	**Sulphide (mg/kg)**
Tailings Solids	Average	75	<1.00	12.9	196,727	14,709	6,573	21,327
	Minimum	37	<1.00	9.0	175,000	8,900	3,400	11,900
	Maximum	126	<1.00	17.0	223,000	25,300	10,000	37,700
	Median	71	<1.00	13.0	194,000	13,900	6,500	19,500

Host Rock Chemistry & Mineralogy

A samples mineralogical composition is a critical aspect in the prediction of its acid generation, acid neutralisation, and metal leaching potential.

Tailings sample was submitted for XRD analyses in August 2015 to determine its mineralogical composition. The sample was found to contain the following:

Major abundance: Calcite (49.4%)

Moderate abundance: Quartz (16.2%), Dolomite (11.5%)

Minor abundance: Mica (8.3%), Barite (5.2%), K-feldspar (3.8%), Pyrite (2.7%), Albite (2.4%)

Trace abundance: Galena (<0.5%), Sphalerite (<0.5%)

The sulphur concentration likely to result from metal sulphides and sulphates within the tailings, particularly barite ($BaSO_4$) and pyrite (FeS_2), together with residual amounts of sphalerite (ZnS) and galena (PbS).

Acid-Base Accounting (ABA) and Acid Generation Potential

The generation of acidic conditions within deposited tailings or waste rock is generally encountered when pyrite is exposed to atmospheric oxygen and water. Pyrite oxidation is a complex, stepwise process, one end product of which is sulphuric acid. The associated reduction in pH of the waste mobilises metals as soluble salts which, with sulphuric acid, may cause severe environmental impacts if allowed to drain or run off from a tailings facility, in an uncontrolled manner. Acid rock drainage (ARD) can significantly compromise ground and surface water quality, and surface reclamation success, even if neutralisation reactions have occurred before discharge.

The results of ABA testing on tailings samples in 2016 are presented in Table 2.

Although tailings at Tara Mines contain both pyrite and trace amounts of its more reactive polymorph, marcasite, the geochemical and physical characteristics of the tailings mitigate against the formation of ARD.

At the Randalstown facility, oxidation of pyrite within the tailings is suppressed as a consequence of the fine texture and saturated condition of the tailings, both of which will restrict oxygen diffusion below the surface. Furthermore, the Carboniferous Limestone matrix of the tailings offers significant and excess neutralisation capacity should any pyrite oxidise to produce acid-sulphate tailings water.

Although no single standard exists for interpretation of ARD potential using ABA results, the MEND (2009) guidance report presents screening criteria for ARD potential that have gained widespread acceptance. These criteria indicate that where an NPR > 2, the material is expected to be non-acid generating.

Using 2016 data the median NPR is ~9.0 and is well in excess of 2; therefore the tailings are expected to be non-acid generating.

Furthermore, criteria based on the NNP is summarised in the GARD Guide (INAP, 2012). Material with an NNP >20kg $CaCO_3$/t is expected to be non-acid generating.

Using 2016 data the median tailings NNP is 526 kg $CaCO_3$/t and therefore classified as non-acid generating in terms of this criteria.

At present, there is no visible evidence of acid seepage from the Randalstown facility, or ponding of acidic water on the surface of the dam. Under conditions of net acid generation potential, ARD is usually detected as a yellow to ochre-red drainage, containing oxy-hydroxides of metals, particularly of iron, aluminium, and manganese. The absence of such drainage, and a tailings water median pH of 8.3, suggests that ARD is not being generated in the Randalstown tailings facility.

Based on the previous work and the ongoing monitoring it is concluded that Tara Mine's tailings at Randalstown have a high net neutralisation potential:

- Significant reductions in tailings pH or alkalinity are not expected to occur over time
- Acid mine drainage from the Randalstown tailings facility is not expected to be a significant environmental issue

The results indicated a considerable buffering capacity of the tailings to neutralise any acid generation.

Table 2 2016 ABA testing on tailings samples

	Acid Potential (AP)	Neutralisation Potential (NP)	Neutralisation Potential Ratio (NPR)	Net Neutralisation Potential (NNP)	Total C	Total Sulphur	Sulphate Sulphur	Sulphide Sulpher (Calc)	C organic	C inorganic
	kgCaCO3/t ore	kgCaCO3/t ore	unity	kgCaCO3/t ore	%	%	%	%	%	%
Jan-16	66	560	8.45	494	6.88	2.84	0.72	2.12	0.16	6.72
Feb-16	76	571	7.49	495	6.91	3.24	0.80	2.44	0.17	6.74
Mar-16	50	604	12.09	554	7.69	2.29	0.69	1.60	0.20	7.49
Apr-16	72	586	8.15	514	7.46	3.00	0.70	2.30	0.15	7.31
May-16	66	613	9.29	547	7.97	2.79	0.68	2.11	0.13	7.84
Jun-16	81	615	7.56	533	7.98	3.30	0.70	2.60	0.14	7.84
Jul-16	47	603	12.96	557	7.66	2.26	0.77	1.49	0.22	7.44
Aug-16	70	553	7.89	483	7.41	3.14	0.90	2.24	0.14	7.27
Sep-16	70	602	8.60	533	7.69	2.88	0.65	2.23	0.13	7.56
Oct-16	63	603	9.60	540	7.49	2.65	0.65	2.00	0.15	7.34
Nov-16	73	594	8.16	521	7.43	3.08	0.75	2.33	0.1	7.33
Dec-16	78	615	7.91	537	8.20	3.16	0.67	2.49	0.1	8.10
Average	**67.59**	**593.25**	**9.01**	**525.67**	**7.56**	**2.89**	**0.72**	**2.16**	**0.15**	**7.42**

Appendix to FIR Item 6

- Results of short term stability analysis
- Design for the Stage 6 Tailings Management Facility *(Golder Associates)*

March 2016

BOLIDEN TARA MINES

Design for the Stage 6 Tailings Management Facility

Submitted to:

Boliden Tara Mines Ltd

Knocknumber

Navan

Meath

REPORT

Report Number. 1532091.502/A.1

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Boliden Tara Mines Ltd - 1 copy (pdf)

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1.0 INTRODUCTION

This report presents the design of the proposed Stage 6 tailings management facility (TMF) at Boliden Tara Mines Ltd (Tara). Stage 6 is required to supplement the diminishing storage capacity of the main TMF and will allow for progressive closure of the existing Stage 5A and 5B facilities.

Tara Mines is Europe's largest zinc and lead mine located near the town of Navan in Co. Meath. The main orebody is hosted in Lower Carboniferous limestone and the process produces large quantities of tailings and approximately 1.15 Mt of tailings, which represents 60% of the total, were discharged into Stage 5A in 2015 whilst the remaining tailings were placed underground as backfill. Historically, approximately 48% to 52% of the mine tailings are discharged into the tailings facilities. For future estimates, the tailings discharge rate has been taken as 1.1Mt/y.

The existing facilities have been built in five key stages during the period between 1974 and present. Drawings 1.1 and 1.2 show the site location and Drawing 1.3 shows the layout plan for the existing tailings facilities, Stages 5A and 5B and the proposed Stage 6 location. All drawings are presented in Appendix A. Stages I and II were constructed to an elevation of 57.29 m AOD (Above Mine Datum) filled and the tailings surface temporarily re-vegetated in 1988 (Drawing 1.4). Stage III was constructed between 1985 and 1987 to an elevation of 57.29 m AOD and was filled in March 2003. Construction of Stage 4A, a raised facility over the existing tailings in Stages I and II to an elevation of 63.29 m AOD, began in late summer of 1998 and was completed in July 2000. The Stage 4A tailings facility was filled by the end of 2006. The construction of Stage 4B to an elevation of 63.29 m AOD, which is founded on the Stage III tailings, commenced in the summer of 2003 and the dam walls were completed in 2006. Stage 4B has been filled. Construction of Stage 5A above Stage 4A to an elevation of 67.29 m AOD has been completed and tailings deposition into 5A commenced in July 2013. To date the total capacity of the tailings facilities, Stages I, II, III, 4A, 4B and 5A is approximately 40.3 Mt and based on an average dry density of 1.42 t/m^3 equates to a volume of 28.4 Mm^3.

Tara Mines had completed the construction of Stage 5B in October 2015 which was constructed on the tailings of Stage 4B to a crest elevation of 67.29 m AOD. Stage 5B will provide additional tailings storage of 4.6 Mt (3.2 Mm^3).

An option study (Ref 1.1) and conceptual designs for Stage 6 were presented to Tara (Reference 1.2) in 2015. The decision by Tara was to adopt a new facility located along the northern sector of the existing facilities (Drawing 1.3) and confined within the northern and seven fields borrow areas (Drawing 1.4). These borrow areas have been extensively used for the construction of Stages 4B, 5A and 5B. Approximately 1.2 Mm^3 of material has been removed from the borrow area including some 300,000 m^3 of weathered rock.

The partially excavated borrow areas are shown in Figure 1 which is an aerial photograph of the borrow sites.

The northern boundary of the TMF footprint was restricted to a distance of at least 100 m from the public road.

The Stage 6 facility provides a potential struck storage volume of approximately 9.6 Mm^3 or 13.6 Mt at a crest elevation of 67.29 m AOD.

Figure 1: Aerial Photograph of the Northern and Seven Fields Borrow Area

2.0 GEOLOGY

2.1 Regional Geology

The bedrock geology underlying the Randalstown area is described in the Golder Associates palaeokarst report (Reference 2.1)

Rocks underlying the immediate area of the tailings impoundment range from Lower Palaeozoic in age through to Lower Carboniferous (Dwg. 2.1) as evaluated by Tara Mines geological department. From the Geological Survey of Ireland (GSI) website both borrow areas consist of the Lower Palaeozoic rocks termed the Rathkenny Formation also shown on Drawing 2.1. These are divided into four main rock types, consisting of Lower Palaeozoics, Red Beds (Old Red Sandstone), Mixed Beds and Pale Beds. These units are described briefly below. The Randalstown Fault cuts from northeast to southwest across the site.

The Lower Palaeozoic rocks consist predominantly of siltstones, sandstones, shales and greywackes with quartz conglomerate horizons. These Lower Palaeozoic rocks occur at rock head level to the east and north of the tailings impoundment and in a 500 m wide zone beneath the site, to the west side of the Randalstown fault.

The Red Beds, which are Lower Carboniferous in age, consist of dark red interbedded conglomerates and sandstones and unconformably overly the Lower Palaeozoic. The Red Beds are assumed to be thin in this area. Exploration boreholes drilled in this area encountered between 0 m to 12 m of Red Beds.

The Mixed Beds is a collective term for two units, which are the Laminated Beds and the Muddy Limestone and also termed the Liscarton Formation. The Laminated Beds consist of dark laminated siltstones, mudstones and shales with local sandstones and calcarenites. The thickness of the laminated beds beneath the basin area of the tailings facility, range from 0 m to 40 m. The overlying Muddy Limestone consists of dark, well bedded, argillaceous and crinoidal limestones with local coarser, bioclastic strata which are microconglomeratic in nature. The thickness of this unit in the tailings area ranges from 0 m to 10 m.

The Pale Beds or the Meath Formation comprise a variety of pelletal, oolitic and bioclastic calcarenites which locally contain quartz sand and darker argillaceous layers. They are a dominantly massive group and are characterised by their low shale content. The base of the Pale Beds is defined by a pale micritic limestone known as the Micrite Unit, which varies greatly in thickness. The Pale Beds vary from 0 m to 140 m thick across the Randalstown area.

The Pale Beds have distinctive marker horizons within them, such as nodular shaley units and silty horizons. Extensive fracturing and leaching of this horizon is common (Reference 2.1) but cavity formation is almost wholly restricted to those sections of the Pale Beds which do not have a cover of the Upper Dark Limestone (UDL). The UDL is a more argillaceous unit compared to the Pale Beds, and as such is less prone to the chemical action of the groundwater. In the tailings area the Pale Beds are not covered by the UDL.

2.2 Quaternary Deposits

The whole region was glaciated by ice sheets, in excess of 800 m thick, which covered Ireland during the Munsterian and Midlandian stages, approximately 15,000 years ago (Ref. 2.2 and 2.3). These left behind Quaternary overburden deposits in the area of the tailings impoundment consisting of three main soil types. The basal unit is a Quaternary glacial till consisting of consolidated lower grey silty clay to clayey silt with sand, gravel and some cobbles and boulders. This is directly overlain by the upper brown weathered glacial till also consisting of silty clay to clayey silt with sand, gravel with some cobbles and boulders. This is essentially a weathering of the underlying grey material and as such tends to have a higher clay content. Overlying some areas of the glacial till are lenses of possibly alluvial or out-wash granular materials comprising silt, sand and gravel.

The general thickness of overburden deposits recorded at Randalstown varies from about 1 m to 8 m and follows the somewhat random profile of the bedrock surface.

2.3 Structure

The major Randalstown Fault, running diagonally across the main tailings site area, trends in a northeast to southwest direction and brings the Pale Beds directly into contact with the Lower Palaeozoic rocks (Drawing 2.1 and Ref. 2.1). This is a reverse fault which dips to the northwest at a steep 75° angle in the tailings area. It is possible that associated with this structure is a zone of strong and extensive shearing, fracturing and subsequent calcite vein infilling.

Several minor faults are reported to exist beneath the tailings dam, notably along the Carboniferous/Lower Palaeozoic contact at the northern end of Stage II (Ref. 2.1).

The limestones within the tailings pond are believed to be draped about the northwest flank of a complex southwest plunging anticline of Lower Palaeozoic rocks.

2.4 Karstification

In carbonate rocks, the secondary permeability, where water moves through fissures, fractures, joints or along bedding planes, may be increased by solution during ground water movement. This process is termed karstification, and leaves interconnected cavities in the ground, and creates distinctive topographical features

(Reference 2.4). During the formation of some karst features, the development of a solution cavity in the underlying limestone may induce the overlying overburden material to migrate or slump into the cavity. This could result in the formation of a sinkhole, also termed doline, at the ground surface. The concerns of this risk relate to the foundation failures that may be induced by karstification, associated with the development of sinkholes.

Karstic features may be considered as either 'active', i.e. forming at the present time, or may have formed in the past and are now inactive, termed 'palaeokarst'. For karstification to be active at the site, there must be groundwater movement. A palaeokarst system is one in which the conditions which promote karstification are no longer present.

Palaeokarsts are present in the Randalstown area and investigations were previously carried out on this subject for the Stage 4 dam raise project (Reference 2.5). The outcome of these investigations indicated that the karsts at the tailings area comprises an immature palaeokarst system. It was termed an immature system because the individual cavities are small and are unlikely to be interconnected. It was considered as a palaeokarst due to the pre-glacial infill materials, such as sand, rock fragments and clay material, and also because the thickness of the overburden deposits indicate that there has been no active karstification in the Randalstown area within the past 15,000 years.

For Stage 6, some initial geophysics was undertaken to investigate the potential for excavating the rock within the basin of the facility which indicated some anomalies. We would propose undertaking a resistivity survey over the entire footprint and a microgravity survey over any anomalies found where the Pale Beds underly the site. Micro gravity is the best technique to determine any areas of mass deficiency representing palaeokarstic features. These types of survey were undertaken at Galmoy and Lisheen with considerable success.

3.0 SITE CONDITIONS

3.1 General

The footprint of the Stage 6 facility is confined to the existing northern and seven fields borrow areas. Virtually all the area is exposed indicating rock or glacial till over the site. It is anticipated that rock which can be excavated by a tracked excavator, ripped or removed by pneumatic hammer (breakable) will be used in construction of the dam walls and drainage. The foundation of the dam walls will either be glacial till or bedrock.

Some locations within the northern borrow area have been restored using topsoil or ameliorated soil and these materials will be removed and stockpiled within the mine boundary and outside the footprint of the facility for capping of Stage 5A. Topsoil stockpiles are also present mainly around the perimeter of the borrow area and they will remain in place if outside the Stage 6 footprint. Any topsoil stockpiles within the basin area will be removed to outside the dam or used for capping of Stage 5A.

A number of site investigations have been undertaken to evaluate the extent and suitability of the glacial till for construction of Stage 5B (References 3.1 and 3.2) in the northern and seven fields borrow areas and for the Simonstown borrow area (Reference 3.3) used in the construction of Stage 4. The depth and relative strength condition of the underlying bedrock (References 3.4 and 3.5) was also investigated by geophysics in the northern and seven fields borrow area together with a limited amount in the Simonstown borrow area.

3.2 Bedrock

The bedrock underlying the proposed Stage 6 footprint are the Pale Beds or Lower Palaeozoic rocks (Drawing. 2.1). Much of the area exposed in the north east sector of the Northern borrow area and the Seven Fields borrow area are Lower Palaeozoic rocks which are generally non calcareous with Pale Beds in the south west sector of the Northern borrow area which are calcareous.

Some removal of the weathered rock material has been undertaken during the construction of Stage 5B to a depth of at least 2 m below rock head level. The strength of the exposed rock is variable and dependent on the degree of weathering and structure. Where the rock has been removed by pneumatic hammer, the material is moderately strong to strong and would be suitable for the drainage protection materials.

3.3 Glacial Till

Above the bedrock is a thin layer of glacial till generally varying in thickness from 0.5 m to 5.0 m and is typically grey clayey silty sands and gravels with boulders and gravelly sandy clayey silts with boulders which is termed Type A2 material and weathered brown clayey silty sands and gravels with occasional boulders which is termed Type A1 material. As the grey till approaches the underlying bedrock surface the boulder content increases considerably to over 50% of the material.

Occasionally, there are bands and lenses of silty fine sand/fine sandy silt (Lacustrine Deposits) or sands and gravels within and above the glacial till and these materials are referred to as Type A3 material.

3.4 Site Work and Borrow Materials

3.4.1 Geophysics

A recent geophysical survey was undertaken in the basin area of Stage 6 consisting of electromagnetic conductivity, seismic refraction and electrical resistivity imaging. The surveys were undertaken to;

- Evaluate the nature and extent of overburden across the basin area for use as material for construction;
- Evaluate the nature and extent of 'rippable-breakable' rock for use as material for construction; and
- Evaluate the potential quantities of overburden and viable rippable–breakable rock available within the proposed footprint of the 'basin' area to be used in construction of the new facility.

The electromagnetic conductivity (EM31) survey is used to assist in the delineation of variations in the thickness of overburden. The seismic refraction survey is used to investigate areas of overburden and potential rippable-breakable rock. The electrical resistivity imaging (ERI) survey is used to map and delineate changes in material type and to determine overburden and rock-fill properties. The results are discussed in Reference 3.5 and the estimated volumes are given below.

Table 1: Estimated Volumes from the Geophysical Surveys Above 1572 m AMSL.

Western Cell	**Estimated Volume (m^3)**	**% Adjustment**	**Adjusted Volume (m^3)**	**Estimated Excavatability**
Overburden	170,000	30	119,000	Diggable
Weathered Bedrock	44,000	25	33,000	Rippable
Bedrock	3,500	25	2,600	Breaking
Total	**217,500**		**154,600**	-
Eastern Cell	**Estimated Volume (m^3)**	**% Adjustment**	**Adjusted Volume (m^3)**	**Estimated Excavatability**
Overburden	365,000	30	255,500	Diggable
Weathered Bedrock	260,000	25	195,000	Rippable
Bedrock	59,000	25	44,200	Breaking
Total	**684,000**		**494,700**	-

The adjustment for the overburden material is for boulders and cobbles which are too large to use as construction fill. The adjustment used for the available rock is precautionary. The total volume available for construction not requiring blasting is 0.65 Mm^3.

For the purpose of the geophysical survey, the basin area within Stage 6 was divided into two cells separated by the main haul road running through the area. The cells are presented in Figure 2.

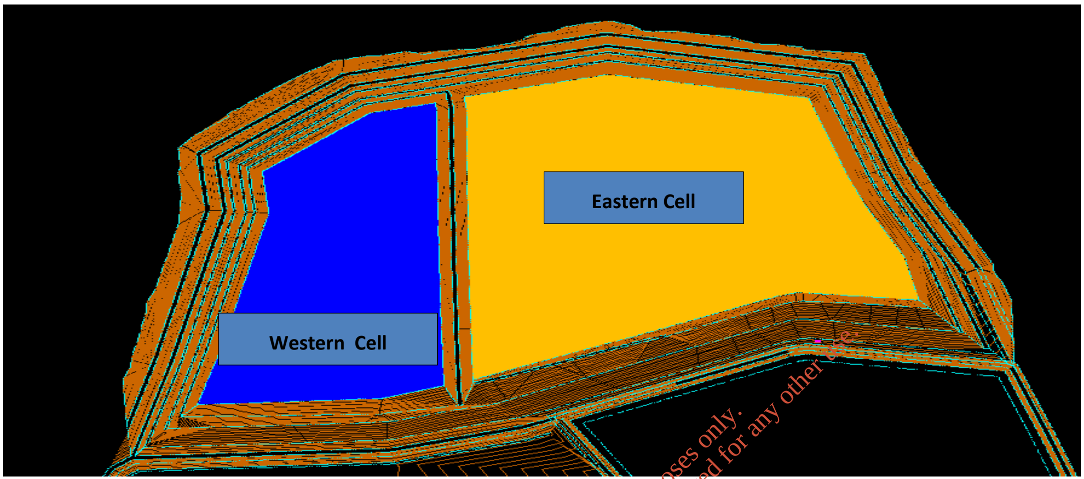

Figure 2: Western and eastern cells within the Stage 6 basin area.

A limited geophysical survey was also undertaken in the old Simonstown borrow area which was used extensively in the construction of Stage 4A. Two seismic profiles and a single ERI line were undertaken (Reference 3.5) to establish the depth of overburden material and rippable rock in the northern sector of the borrow area.

3.4.2 Trial Pitting Northern and Seven Fields Borrow Area

Twenty nine trial pits were excavated through the overburden material to near refusal either in bedrock or where hard digging due to the presence of numerous cobbles and boulders were observed within the basin area of Stage 6 in the northern and seven fields borrow areas. The location of the trial pits is given in Figure 3 and the logs presented in Appendix B and indicated that within the Stage 6 basin area of the western cell, the depth of overburden was generally between depths below ground level of 3.6 m and 5.0 m. In terms of elevation, the overburden extended to depths below 1570 m AMD and 1566.8 m AMD. In the eastern cell, overburden was found to depths of between 0.4 m and 3.5 m below ground level although in some areas the overburden has been removed completely to expose bedrock.

Table 2: Estimated Overburden Volumes from the Trial Pitting

Western Cell	**Estimated Volume (m^3)**	**% Adjustment**	**Adjusted Volume (m^3)**	**Comments**
Average Depth 4 m	520,000	30	364,000	Including material below an Elv. of 1570 m AMD
Average Depth 2 m	260,000	30	182,000	Material above an Elv. of 1570 m AMD
Eastern Cell	**Estimated Volume (m^3)**	**% Adjustment**	**Adjusted Volume (m^3)**	**Comments**

Western Cell	Estimated Volume (m^3)	% Adjustment	Adjusted Volume (m^3)	Comments
Average Depth 1.2 m	300,000	30	210,000	Above an Elv. of 1570 m AMD

Figure 3: Location of Trial Pits in the Northern and Seven Fields borrow area

3.5 Summary of Borrow Material

There are several sources of material that could be used for construction purposes for the dam wall and capping on land owned by Tara Mines and in the immediate vicinity of the existing TMF. The main sources of borrow material are:

- Within the basin area of Stage 6 which will need to be removed prior to completion of the first phase of construction; and
- The material outside of the TMF footprint and within the northern and seven fields borrow areas.

There are some discrepancies between the overburden evaluated by the geophysics and the trial pitting and it can be assumed that the latter is a more robust method of evaluation. In summary, the overburden material including diggable weathered bedrock within the basin area of Stage 6 is estimated to be between 392,000 m^3 and 574,000 m^3 depending on the depth achieved. This assumes that the depth of excavation in the western cell is approximately between 2 m and 4 m and generally above an elevation of between 1568 m AMD and 1572 m AMD. Assuming and average value (3m depth), the material available is 483,000 m^3. Where overburden is taken to depth and particularly where ground water is encountered, these excavations will be backfilled with boulders and cobbles removed from the glacial till.

From the geophysics, an estimate for the rippable rock volume is 228,000 m^3 and the volume of breaking rock with a pneumatic hammer is approximately 46,800 m^3.Therefore the total material available within the basin area of the TMF is approximately 757,000 m^3.

Table 3: Estimated Available Borrow Materials

Location	Estimated Volume (m^3)			
	Overburden Diggable	Rippable Rock	Breakable Rock	Total
TMF Basin Area	483,000	228,000	46,000	**757,000**
Outside TMF Footprint	329,000	175,000	37,000	**541,000**
Total	**812,000**	**403,000**	**83,000**	**1,298,000**

The remaining borrow area outside of the dam footprint would be used after the completion of Phase 1 construction.

The bulk of any short fall in borrow materials for Phase 2 construction would be obtained from land adjacent to the tailings facility owned by Tara and subject to planning. Additional materials such as drainage or protection material would be imported from local quarries together with any recycling of imported material from construction sites.

4.0 HYDROLOGY AND GROUNDWATER

4.1 Hydrology

The Randalstown tailings facility is located in a topographically flat area at approximately 50 m above sea level. There are four main drainage regimes in the vicinity (Dwg 4.1).

The Yellow River, to the west of the tailings facility, is close to the site and flows south. The Yellow River joins the Blackwater River about 0.75 km south west of the site.

The Blackwater River flows south east and into the River Boyne approximately 4 km downstream.

The Simonstown stream originally crossed the tailings site and was diverted during the construction of Stage I into a channel on the eastern side of Stages I and II. The stream flows into the Doug River which in turn flows into the Blackwater River.

4.2 Groundwater

Two hydrogeological units exist at the site:

- Quaternary sands, gravels and clays with an average thickness of 8 m beneath the tailings pond; and
- Lower Palaeozoic greywacke and Lower Carboniferous limestone bedrock.

The units are expected to be in hydraulic connection naturally. Hydraulic conductivity for Quaternary sands and gravels is likely to be of the order of 1E-4 m/s and 1E-5 m/s. Areas of high clay content are expected to have significantly lower hydraulic conductivity as represented for the clayey glacial till materials and as low as 1E-9 m/s. The hydraulic conductivity range for the bedrock hydrogeological unit is likely to be 1E-4 m/s and 1E-5 m/s (Reference 4.1) although results obtained from the two deep borehole investigations (Reference 3.1), indicated lower permeabilities at around 1E-6 m/s. The permeability of the rock will be dependent on frequency of fissuring and fissure infilling.

Groundwater flow in the Quaternary hydrogeological unit is expected to be restricted by the laterally discontinuous nature of the high hydraulic conductivity lenses.

Groundwater flow in the bedrock is expected to be locally very fast through faults, fissures and solution features in the limestone. High flows were recorded from the fissured bedrock (Reference 4.1).

Local groundwater flow is to the south west towards the Yellow River. Both hydrogeological units are expected to be in hydraulic connection with the Yellow River. A bedrock spring flows into the river near the north western edge of the Stage III tailings facility.

There are two private wells which draw water from bedrock in the Randalstown area and make contact with the Quaternary hydrogeological unit. Groundwater flow is expected to be to the south and southwest across the region, towards the Yellow and the Blackwater Rivers and River Boyne.

Groundwater is already encountered in the Northern and Seven Fields borrow areas and drainage measures have been installed to drain the site. During the summer season, the groundwater drops and rises in winter time. It is expected that over much of the basin area, rock will be exposed and therefore it would be prudent to place the lining system during late summer and early autumn.

5.0 SEISMICITY

5.1 Seismic Hazard

Ireland is characterised by very low levels of seismic activity. There have been no known occurrences of severe shaking over the past 10 centuries. The largest earthquake of recent time to shake the area was the magnitude 5.4 (Richter Scale) Wales earthquake of 1984, about 400 km from the site. The last reported earthquake in Ireland was in Wicklow on 28th April 1992 with a magnitude of 1.4 which is a very small event. The seismicity of Ireland is very low, particularly in the western part of the country and virtually the whole of Ireland is practically free of earthquakes (Reference 5.1).

Based on a review of historic data on earthquakes over the past 1,000 years within 1,000 km of the site, and broad estimates of attenuation, a design acceleration of 0.03g would provide for a 1 in 1,000 year event. Even up to a 1 in 10,000 year event, equivalent to the maximum credible earthquake (MCE), the acceleration should not exceed 0.06g.

The design criteria for the site have been evaluated using seismic data of the region to derive the Design Base Earthquake (DBE) and the Maximum Credible Earthquake (MCE). These are equated to a peak ground acceleration (PGA) for the site. This procedure follows international guidelines as set out by the International Commission on Large Dams (ICOLD) (Reference 5.2). The DBE equates to an earthquake event that usually has a return period equal to 10 to 50 times the life of the facility (500 years) and when occurring will not affect the performance of the structure. The MCE has a return period of some 10,000 years and when occurring will not cause total failure of the embankment wall but will result in severe damage i.e. slumping of the crest. The MCE is particularly applicable to the long term situation such as the close-out phase of the facility.

5.2 Seismic Vulnerability

Stage 6 will be constructed using the downstream method and therefore not vulnerable to the earthquakes anticipated for this site.

The DBE has been taken as 0.03g which is very low and would have little or no effect on any of the dam walls constructed. The MCE has been taken as 0.06g and has been used to determine the pseudo static stability of the dam walls. It is a very conservative method as it assumes a horizontal force in one direction whereas the action is both forwards and backwards and for a brief period of time at its peak. The normal consequence of seismicity is settlement of an engineered, constructed and compacted dam wall rather than dam failure. The MCE has been used in the long term stability analysis as a horizontal force acting on the stack wall of the TMF. This is termed the pseudo-static method of stability analysis and is a very conservative approach.

6.0 DESIGN

6.1 General

The adjoining cell is located along the northern sector of the of the existing TMF facility (Drawing 1.2 and 1.3) and confined within the northern and seven fields borrow areas. It is proposed to raise the dam in two phases (Dwg. 6.1), the first phase to 1586 m AMD and the second phase to 67.29 m AOD. External ramps will be provided (Dwg. 6.2) for access from the main haul roads. The total construction volumes and storage volumes for various elevations are presented in the following table.

Table 4: Struck Storage and Construction Volumes at given Crest Elvs.

Phases	Crest Elevation m AMD	Tailings Struck Storage Acc. Vol. m^3	Total Construction Acc. Volumes m^3	Ratio Tails/Con Vol. Ratio	Life in years
1	1586	5,300,000	533,800	10	6.5
2	1594	9,580,000	1,786,400	5.6	11.75

The ratio of tailings volume to construction volume indicates the additional volume gained in Phase 1 by filling the existing northern and seven fields borrow areas and borrowing the remaining material to construct the walls. The life in years has been based on a tailings discharge into the TMF of 1.1Mt/y tailings to TMF, a 0.5% beach slope and an average dry density of 1.42 t/m^3.

The main benefit of two phases is that the maximum amount of fill can be borrowed from the basin area to use in construction. Once the first phase is complete, the basin area will be sterilised to any future borrowing by the placement of the lining system. It will therefore be necessary to remove all available material within the basin area and any addition materials found would be stored on the downstream side of Phase 1 or as ramps and ancillary structures. The floor plan area of the TMF basin is approximately 38.5 Ha.

6.2 Dam Sections

The construction of the walls would be different from all previous dam walls forming the existing facility because of the design for a composite lining on the upstream face of the dam walls and basin area although the dam wall will be zoned (Dwg. 6.3) and the figure below.

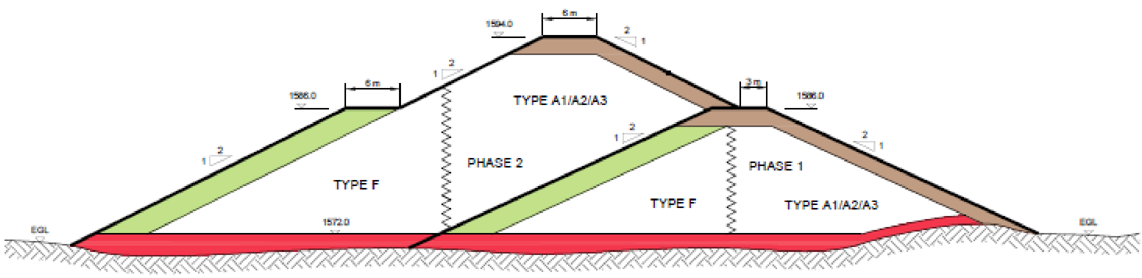

Figure 4: Typical Dam Section

The upstream slope will be constructed with a slope of 2H:1V and for the first phase, will be keyed into the existing Stage I, II, III and Stage 4, with a ramp down to 57.29 m AOD and a ramp up to 63.29 m AOD (Dwg 6.4A). The second phase will be keyed into Stage 5 at 67.29 m AOD. The overall downstream slope will be approximately 2.25H:1V consisting of two 2H:1V slopes with a 6m bench at 1586 m AMD for the second phase. This will allow access to Stages I, II and III by ramping down to 57.29 m AOD and Stage 4 by ramping upwards to an elevation of 1590.0 m AMD (Dwg 6.4B). The bench will also be used to install some of the monitoring instruments. The crest width of the dam walls for all phases is 6 m and the maximum wall height is approximately 20 m for the final elevation of 67.29 m AOD.

Sections through the dam wall are presented in Drawing 6.5 and the section locations are shown in Drawing 6.6. The current ground level is variable and generally between elevations of 1570 m AMD and 1580 m AMD. Much of the dam wall footprint is within the existing areas which have been previously borrowed for Stage 5.

The upstream sector of the dam (Dwg. 6.3) consists of a 6m wide unit of clayey glacial till (Type A1 and/or Type A2). The footprint of the dam wall will be constructed with a 1m minimum thickness of Type D2 rock which acts as a drainage blanket. On the lower parts of the downstream sector of the dam wall, a 6 m width of Type A2 glacial till will be placed. Between the upstream Type A1/A2 zone and the downstream Type A2 zone is an upstream core consisting of Type A material and a downstream core consisting of Type F material, a wethered limestone.

No chimney drainage system is needed for this structure since the basal Type D2 will act as a drainage blanket. The composite lining will minimise seepage through the dam wall provided it is not damaged during operations. Measures will be undertaken to prevent damage of the lining system during operations using protection measures as discussed later.

The existing northern dam wall, perimeter interceptor channel and finger drain channels would be stripped of topsoil and vegetation prior to receiving the protection material and lining system forming Stage 6. The existing manholes will be trimmed below ground level so there are no protrusions that could damage the lining system. To prevent the lining system collapsing into the manholes it is proposed to backfill them with uniform 50 mm cobbles (Type E). The open concrete chutes will also be backfilled with Type E material. The location of these structures are presented on aerial photographs in Appendix C.

Once the existing northern dam wall is lined, rainfall infiltration into the dam wall will be significantly reduced and the lower flow rates measured in the weirs of Stage 4 and Stage 5 will be applicable.

The finger drain channels and the perimeter interceptor channel at the base of the existing northern wall of the facility would also be backfilled with Type E material. A 300 mm diameter perforated pipe would be installed into the base of the perimeter interceptor channel on a bedding of Type C material.

A 100 mm layer of processed rockfill Type C material would be placed over the Type A materials on the upstream side of the new Stage 6 dam walls and on the northern face of the existing embankment wall (Dwg.6.7) to provide protection for the lining system. The maximum particle size of the Type C would be 20 mm and the material would be well graded. A 1,000 g/m^2 non-woven geotextile is placed on top of the Type C material prior to placement of the lining system. The adjoining wall would be cleaned of vegetation, trimmed to receive the 100 mm layer of Type C followed by the 1,000 g/m^2 non-woven geotextile. The Type C material on the dam walls would continue over the backfilled finger channels and backfilled perimeter interceptor channel. The Type C will intercept any seepage at the downstream toe of the northern dam wall and into the perimeter interceptor channel. Where Type C is placed on the Type E material backfill it will be separated by Terram 1000 geotextile or equivalent. Terram may also be required in soft areas on the borrow area floor and downstream toe of the existing northern dam wall. Once the Type C material is placed, the composite lining would be formed along the base, up the slope and anchored on the crest.

A perimeter roadway would be constructed and merge with the roads around the existing TMF. A new security fence is required along the north perimeter access road (Dwg 6.8) and tied into the existing fences on the eastern and western site boundaries.

A road surfacing material Type B, 200 mm thick would be required at the top of the dam crest and the lower perimeter road, intermediate bench at 1586 m AMD and any permanent ramps.

It is anticipated that at least 3 external ramps will be installed along the northern wall of Stage 6. The first ramp would be in a central location (Dwg.6.2) and be accessed via the main tarmac road from the site offices. Two other ramps would be located opposite the east and west sectors of the northern borrow area. These ramp would allow access to the remaining borrow materials and stockpiles to complete the second phase of Stage 6, the capping of Stage 5B and the eventual capping of Stage 6.

Temporary internal ramps will also be required but these would be removed systematically as the lining system is installed.

6.3 Fill material

6.3.1 General

The dam wall raise is constructed from several materials which include cohesive glacial till, granular glacial material and processed rock fill. It consists of the following material types:

- Type A1 and A2 Cohesive Glacial Till;
- Type A3 Glacial Granular Material;
- Type B Road Material;
- Type C Protection Material;
- Type D1 Backfilling Existing Borrow Area Drains;
- Type D2 Dam Footprint Drainage Blanket Material; and
- Type E 75 mm Coarse Drainage Backfill For Perimeter Interceptor Channel and Concrete Chutes.

6.3.2 Type A Materials

6.3.2.1 Type A1

Type A1 will be suitable material complying with the following:

- Material Type A shall be glacial silty sands and gravels obtained from the borrow area with a Plasticity Index greater than 10%;
- The moisture content shall be between -2% and +4% of optimum as measured in the Standard Proctor method and the material shall be free of all unsuitable material and compacted to 95% of Standard Proctor in 300 mm lifts; and
- Maximum particle size is 200 mm in the minimum dimension.

6.3.2.2 Type A2 Material

Type A2 will be suitable material complying with the following:

- Material Type A2 shall be glacial silty sands and gravels obtained from the borrow area with a Plasticity Index less than 10% but greater than 4%;
- The moisture content shall be between -2% and +4% of optimum as measured in the Standard Proctor method and the material shall be free of all unsuitable material and compacted to 95% of Standard Proctor in 300 mm lifts; and
- Maximum particle size is 200 mm in the minimum dimension.

6.3.2.3 Type A3 Material

Type A3 will be suitable material complying with the following:

- Material Type A3 shall be glacial silty sands and gravels obtained from the borrow area with a Plasticity Index less than 4%;
- The moisture content shall be between -2% and +4% of optimum as measured in the Standard Proctor method and the material shall be free of all unsuitable material and compacted to 95% of Standard Proctor in 300 mm lifts; and
- Maximum particle size is 200 mm in the minimum dimension.

6.3.3 Granular Rock Material

6.3.3.1 General

There is a requirement for a large quantity of granular rock material which will be either imported or derived from quarrying in the borrow area and these are discussed below.

6.3.3.2 Type B Road Surface Material

The specification for this material which conforms to the 804 specification is tabulated below.

Table 5: Type B Grading Envelope

Sieve Size (mm)	Coarse (% passing)	Fine (% passing)
63	100	
31.5	90	80
16	85	55
8	65	35
4	50	22
2	40	15
1	35	10
0.5	20	0
0.063	7	0

The material will be compacted with four passes of a 10 tonnes smooth vibratory roller.

6.3.3.3 Type C Protection Material

The specification for this material is tabulated below.

Table 6: Type C Material

Sieve Size (mm)	% Passing
20	100
6.3	60-95
1.18	35-80
0.300	15-60
0.075	0-30

This will be nominally compacted with a smooth roller on the basin floor. The protection material placed on the dam wall side slopes will also be nominally compacted with a smooth roller winched down from the crest or by an alternative method agreed by the Engineer.

6.3.3.4 Type D1 Backfilling Existing Borrow Area Drains

The specification for this material is tabulated below.

Table 7: Type D1 Grading Envelope

Sieve Size (mm)	Coarse (% passing)	Fine (% passing)
250	100	
100	100	30
75	60	10
50	40	0
20	15	0
10	10	0
1	6	0

This material will be placed and nominally compacted with the construction plant.

6.3.3.5 Type D2 Dam Footprint Drainage Blanket Material

The specification for Type D2 material is that it is granular and 100% passing 250 mm. The material will be placed in maximum lift size of 500mm and compacted with a 10 tonnes vibratory roller using 6 passes.

6.3.3.6 Type E Coarse Drainage Material

This material is uniformly graded at 50 mm diameter.

6.4 Fill Material Quantities

The volumes of material for construction are tabulated as follows.

Table 8: Material Quantities

Description	Unit	1st Stage Qty	2nd Stage Qty	Total Qty
Total Dam Fill Volumes	**m3**	**533,800**	**1,173,400**	**1,707,200**
Type A1/A2 Upstream Wall Construction	m3	105,500	110,600	**216,100**
Type A 2 Downstream Wall Construction	m3	107,900	111,900	**219,800**
Type B Road Surfacing	m3	4,900	5,100	**7,400**
Type C Protection Material	m3	52,280	7,400	**59,700**
Type D1 Drainage	m3	600		**600**
Type D2 Dam Footprint Drainage	m3	85,400	98,900	**184,300**
Type A1/A2/A3 Upstream Core	m3	117,500	426,000	**543,500**
Type F Downstream Core	m3	117,500	426,000	**543,500**
Type E Coarse Drainage	m3	11,500		**11,500**
Total Construction Volumes	**m3**	**600,500**	**1,185,900**	**1,786,400**

The total dam fill volume excludes Type B, Type C, Type D1 and Type E material which is typically imported although could be produced on site from crushing and grading strong rock as previously undertaken for Stage 5B.

6.5 Site Preparation

There is a considerable amount of site preparation that is required to develop this site of which some could be undertaken pre-construction while other activities would be done during construction.

Blakes Stream cuts across the Northern borrow area and the Seven Fields borrow area and is located in the basin area of the proposed Stage 6 facility. It enters the site in the north east corner and exits the site in the south west corner via a settling pond complex before entering the Yellow River. It is proposed to divert Blakes Stream to the east as it enters the site (Dwg 6.9) and into a disused channel which runs along the eastern boundary prior to entering the Simonstown Stream. It will be necessary to compare the invert levels of Blake's Stream and the existing channel to determine the optimum location for the two to join.

There are a number of trees that will require removal from the site. The majority are immediately north of the existing northern dam wall but also occur around the Contractors compound as well as the central site road and along established drainage ditches. All vegetation will be removed from the footprint of the proposed Stage 6 TMF.

There are several power lines that cross the site. These are shown on Drawing 6.10. A 10 KV power line runs along the downstream toe of the existing northern dam wall. This will require realignment. Another power line carrying 38 KV follows the eastern boundary of the seven fields borrow area and this will need to be realigned further to the east beyond the footprint of the TMF. A power line crosses diagonally from north east to south west of the seven fields and the northern borrow area. This is a 38 KV line and will need to be moved.

The Contractors compound will require removal and the septic tanks removed and backfilled with compacted fill or left and backfilled with concrete. Fencing along the downstream side of the existing northern wall should be removed from site into a suitable landfill.

The elevations within the existing borrow area forming the footprint Stage 6 is variable and the Drawing 6.11 is a recent survey showing the cut and fill depths to an elevation of 1572 m AMD. After removal of the materials from the borrow area some regarding of the surface will be required to receive the lining system. The existing drainage system will be backfilled with Type D1 material to prevent ponding during the placement of the lining system. Boulders from Types A1 and A2, and Type D2 material will be used to fill hollows.

Type D2 material will be used to bring the level of the subgrade to an approximate elevation of 1571 m AMD within the footprint of the dam wall. On subgrade above 1571 m AMD, a 1m layer of Type D2 will be placed. Any unsuitable materials in the basin area will be removed and stockpiled outside the TMF footprint and used for capping restoration.

Groundwater may be an issue and it may be difficult to remove all water from the excavation and some boulders and Type D2 material may have to be placed into water to raise the excavation level prior to installation of the lining system. Care is required as the lining system will be placed during the summer when the water table is at its lowest. However, as the water table rises during the approach of autumn and winter, it is important to surcharge the liner to prevent the lining lifting. The most practical method is to place the tailings into the facility soon after the lining is placed or mine water and at the end of summer.

After regarding of the floor area, a 100 mm layer of Type C is placed on top of the floor where the lining system is to be placed in the basin area. In soft areas, Terram 1000 would be placed prior to Type C placement.

Generally, prior to the preparation of the foundations, a resistivity and microgravity survey would be undertaken to delineate the presence of any palaeokarstic features in the limestone bedrock. Some anomalies result from rapid changes in the thickness of the glacial till above the bedrock or highly weathered and fractured near surface bedrock. Any anomalies, if found, would be investigated by drilling. However, as discussed in Sections 2 and 3, the majority of the Stage 6 footprint is founded on Palaeozoic rocks which are non-calcareous rocks and it is only the south west corner of the Northern borrow area where limestone may be present as part of the Pale Beds. The geophysical work undertaken in the borrow area to determine the excavatability of the bedrock indicated the possibility of palaeokarst in five locations as presented below.

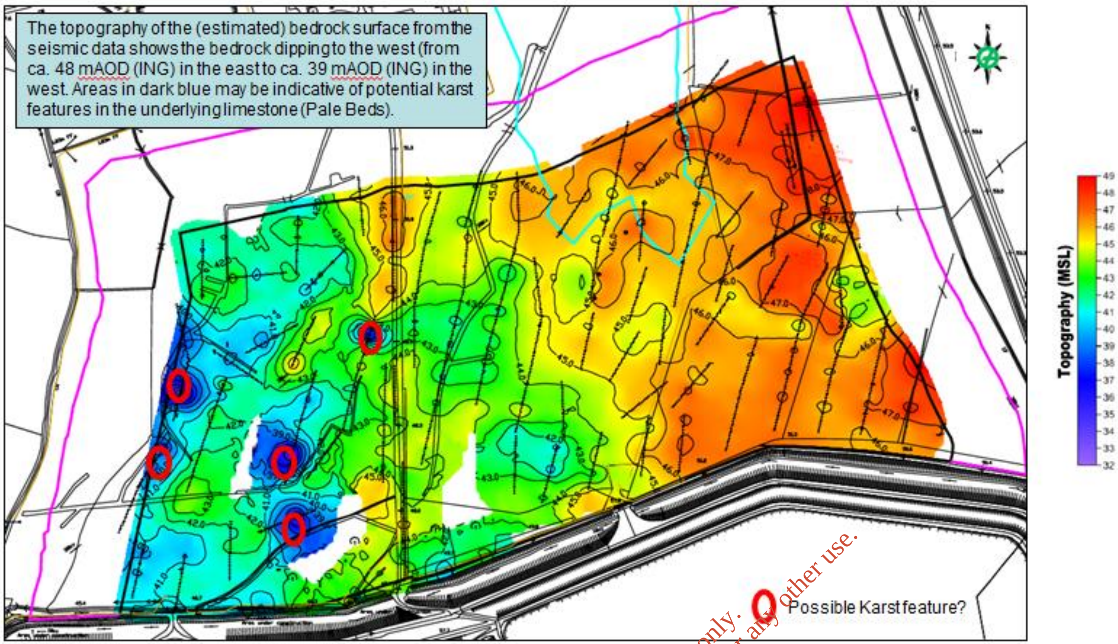

Figure 5: Possible Palaeokarstic Features (Ref.3.5)

6.6 Composite Lining

The TMF incorporates a composite lining system which consists of high density polyethylene (HDPE) geomembrane over a geosynthetic clay liner (GCL). The composite lining overlies a1000 g/m^2 which in turn overlies 100 mm of Type C protection material as shown in section on Drawing 6.12.

6.6.1 High Density Polyethylene (HDPE)

The HDPE will be 2 mm thick, double textured and is placed directly over GCL on the 2H:1V upstream slopes of the dam wall. In the basin area the HDPE is 2 mm thick and smooth and is placed directly over the GCL. The HDPE and GCL are anchored along the dam crest at 1586 m AMD and along the existing north wall of Stage 4 at an elevation of 63.29 m AOD for the first phase of Stage 6. Similarly, the lining system is anchored along the dam crest at 67.29 m AOD for the second phase of Stage 6.

The geomembrane materials shall consist of high density polyethylene, produced from new resins and containing no fillers, plasticisers or additives of any kind with the exception of carbon black.

The geomembrane shall comply with the requirements set out in the table below for 2.0 mm double textured and smooth geomembrane.

Table 9: HDPE Properties

Parameters	Properties	
Material	Double Textured 2 mm	Smooth 2 mm
Thickness (minimum average)	nom. (mil)	
■ Lowest individual of 10 values	-10%	-10%
Density mg/l (minimum.)	0.940 g/cc	0.940 g/cc
Tensile Properties (minimum average)		
■ Yield strength	29 kN/m	29 kN/m

Parameters	Properties	
■ Break strength	21 kN/m	53 kN/m
■ Yield elongation	>12%	>12%
■ Break elongation	>100%	>700%
Tear Resistance (minimum average)	250 N	249 N
Puncture Resistance (minimum average)	500 N	640 N
Stress Crack Resistance	300 hr.	300 hr.
Carbon Black Content (range)	2.0 - 3.0%	2.0 - 3.0%

The geomembrane installation will be independently supervised and will be subjected to a strict CQA procedure.

6.6.2 Geosynthetic Clay Liner (GCL)

The GCL is placed beneath the HDPE geomembrane and over the Type C sub-base material on the upstream dam slopes and basin area of the facility.

The GCL shall be Bentofix or an approved equivalent and the grade shall be 3600 g/m^2 dry weight with a maximum permeability of 5E-11 m/s. The GCL shall consist of a layer of natural sodium bentonite clay encapsulated between two polypropylene textiles (geotextile) and have the following properties:

Table 10: GCL Properties

Parameter			Properties
Bentonite Layer	Swell Index, Minimum		24 ml/2g
	Fluid Loss, Maximum		18 ml
	Sodium Bentonite, Minimum		3600 g/m^2
Geotextile Mass (Minimum Average Roll Value)	Slit-Film Woven		105 g/m^2
	Nonwoven Needle Punched		200 g/m^2
Index Flux (Maximum)			8x10^{-9} m^3/m^2/s
Hydraulic Conductivity (Maximum)			5x10^{-11} m/s
Peel Strength (Minimum)			240 N/m
Strip Tensile Strength (Minimum)	Machine Direction	Tensile Strength	8 kN/m
		Elongation	8%
	Across Machine Direction	Tensile Strength	8 kN/m
		Elongation	8%
Hydrated Internal Shear Strength (Minimum)			24 kPa

For purposes of strength, performance, and integrity, the GCL shall be manufactured by mechanically bonding the geotextile using a needle-punching process. Needle-punched GCLs are those which, by the use of a needling board, have fibres of the non-woven geotextile pushed through the bentonite clay layer and integrated into a woven or non-woven geotextile.

The bentonite sealing compound or bentonite granules used to seal penetrations and make repairs shall be made of the same natural sodium bentonite as the GCL and shall be as recommended by the GCL Manufacturer. All GCL shall be free of damage or defect.

The GCL installation will be independently supervised and will be subjected to a strict CQA procedure.

6.7 Geotextile

6.7.1 General

Three types of geotextile material are required, a 1000 g/m^2 non-woven needled punched geotextile to protect the lining system from underneath, a carbon rich 500 g/m^2 non-woven needled punched geotextile to protect the lining system from above on the slope and a Terram 1000 or equivalent as a separation medium.

6.7.2 500 g/m^2 and 1000 g/m^2 Non-Woven Geotextile

A 1000 g/m^2 non-woven needle punched geotextile is required to protect the GCL and HDPE geomembrane from the underlying Type C material which in turn is placed on top of the Type B material. A carbon rich 500 g/m^2 non-woven needle punched geotextile is required in the anchor trench and on the surface of the HDPE to protect this material from the surcharge and movement of pipe work.

The physical and mechanical properties of the 1000 grm/m^2 and 500 grm/m^2 non-woven needle punched geotextile are given below.

Table 11: Non-Woven Geotextile Properties

Parameter	Specification 1000 g/m^2	Specification 500 g/m^2
CBR Puncture Resistance	Minimum 10,000 N	Minimum 5,000 N
Wide Width Tensile Strength	Minimum 75 kN/m	Minimum 40 kN/m
Elongation at break	Minimum 50%	Minimum 50%
Thickness	Minimum 8.0 mm	Minimum 5.0 mm
Mass per unit area	Minimum 1000 g/m^2	Minimum 500 g/m^2

6.7.3 Terram 1000

A separation geotextile will be required in some glacial foundation areas exposed, to prevent the Type C material punching into any soft and wet material. It will not always be possible to remove these materials or prevent water accumulating on the surface of the glacial till particularly in localised depressions. The geotextile would also be used to separate the coarse drainage material (Type D1 and Type E) and finer material. The separation textile would be Terram 1000 or equivalent.

6.8 Geofabric Material Quantities

The areas of material for construction are tabulated as follows.

Table 12: Material Quantities

Description	Unit	Phase 1 Qty	Phase 2 Qty	Total Qty
1000 g/m^2 Geotextile	m^2	529,262	74,255	**603,517**
500 g/m^2 Geotextile	m^2	99,262	74,255	**173,517**
Terram 1000	m^2	176,421	24,752	**201,172**
GCL	m^2	529,262	74,255	**603,517**
HDPE (Smooth) 2 mm	m^3	430,000		**430,000**
HDPE (Textured) 2 mm	m^3	99,262	74,255	**173,517**

6.9 Anchor Trench

The geosynthetics will be fixed in an anchor trench excavated along the crest of the dam wall stages as shown in section in Drawing 6.12. The trenches will be excavated with rounded shoulders where the geotextile and geomembrane lining will adjoin the trench in order to avoid sharp curvatures in the membrane material. The trench will be backfilled with screened Type C material and compacted in layers not exceeding 150 mm deep.

6.10 Permanent Surcharge

A permanent surcharge is required on the exposed geomembrane to prevent the HDPE from being lifted and damaged by wind action and minimise damage from pipe movements. It also acts as a ladder if someone accidently falls into the facility. The surcharge will consist of car tyres in filled with Type C material and placed on the carbon rich 500 g/m^2 non-woven needle punched geotextile. Each line of surcharge shall be a maximum of 2 m centres longitudinally and anchored on the crest and the weights consisting of car tyres shall be a maximum of 1.5 m apart down slope and where applicable on the lining placed on the floor of the tailings and a minimum distance of 1 tyre width beyond the downstream toe. The two tyres at the toe of the slope and 1 tyre width beyond will be tied together (Dwg 6.13). The tyres will be attached by suitable rope with a minimum life of 10 years.

6.11 Leak Detection

A leak detection survey using DC electric current will be undertaken after the installation of the lining system. This geophysical method was previously used. An electric current is passed between two electrodes, one placed in either water ponded in the cell or by a water spray jetted onto the lining and the other in the peat outside the cell. With the geomembrane intact, the water in the cell will be electrically isolated from the external environment. The resulting potential field, measured as a potential difference between two non-polarising electrodes, is small but uniformly distributed over the geomembrane. If the geomembrane is defective, current will flow through the point of leakage and the measured potential will peak around the position of the defect.

Data acquisition is performed within the cell on a predetermined grid marked on the geomembrane at 2.0 m spacing. Two or more sets of data are obtained simultaneously, with information automatically stored on data loggers where possible. The results are processed and plotted on site, then overlaid on the plan of the cell to allow the immediate detection and location of the defect.

The voltage used is about 240 V and there is a strict safety protocol to follow to ensure no connection is made between the personnel, the ground and the water during the data gathering phase.

It is also anticipated that some difficulties will arise in physically walking on the flooded geomembrane. The surface of the geomembrane forming the basin area will be extremely slippery and to overcome this problem the operators use specialised waterproof clothing and footwear developed in the sailing industry.

6.12 Seepage

Seepage from the TMF will be controlled by the low permeability composite lining system, and the low permeability of the tailings retained by the facility. Experience of a large number of quality assured and controlled geomembrane installations indicates (Reference 6.1) the presence of between 2 and 5 leaks per hectare and these are generally less than 10 mm^2 in size.

Seepage calculations have been based on the design equations given in References 6.1 and 6.2 and for the worst case, when the facility is filled with tailings.

The volume of seepage flowing laterally through the dam wall via the GCL and some nominal defects in the lining system for a constant head of 20m (Phase 2) would be an average of about 1.5 m^3/day with a 10% probability of the seepage being less than 0.25 m^3/day and a 10% probability of greater than being 5 m^3/day. The range represented in the figure below, is based on a number of variables including infiltration rate based on the range of vertical permeability values of the tailings, the range of defects in the lining per hectare, the effective permeability of the composite lining and head acting on the lining.

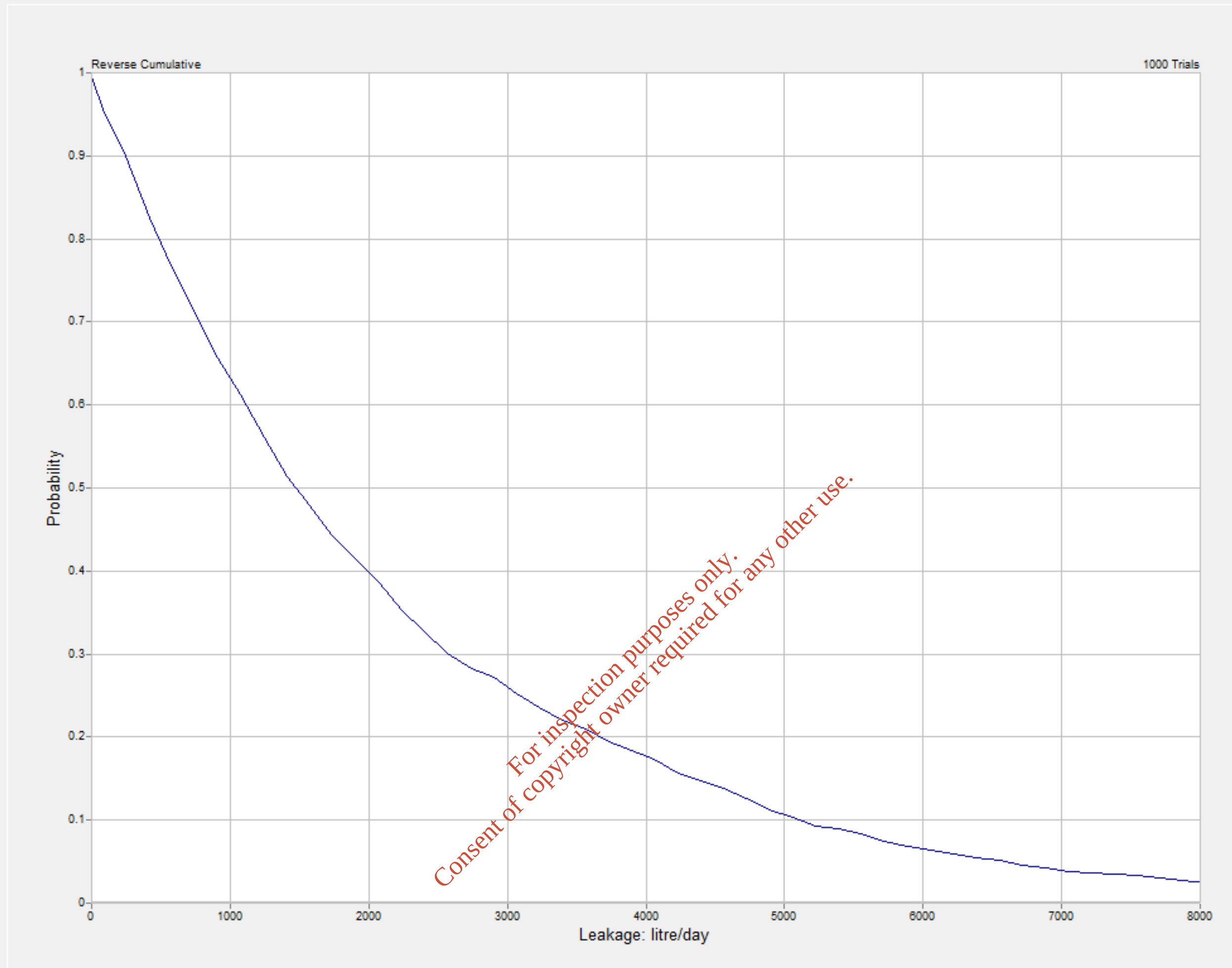

Figure 6: Probability of Seepage from Composite Lining System

6.13 Perimeter Interceptor Channel

The regional groundwater flow is from the north east to the south west and the minimal seepage passing through defects in the lining system will mix and be diluted with the ground water. The perimeter interceptor channel (Drawing 6.8) will collect surface water runoff from the dam walls, perimeter road and borrow areas. This seepage will be insignificant when compared with the clean surface water runoff from the dam walls and surrounding land.

During winter, sections of the perimeter interceptor channel will intercept the groundwater as it rises and this will be monitored for quality. It is expected that water quality will be suitable for discharge into the environment directly but initially it will be collected.

The final profile of the channel will be fixed during construction after the removal of unsuitable materials and also after borrowing suitable material. The perimeter interceptor channel will discharge into the existing east and west perimeter interceptor channel. The longitudinal profile is considerably variable and generally between 1569 m AMD and 1580 m AMD as shown on Drawing 6.14, which is the profile for the second phase. The plan showing the chainage of the longitudinal profile is shown on Drawing 6.15. Because of the presence of low spots there could be issues with potential ponding in the channel unless pumping is undertaken from these low points. The eastern sector of the Stage 6 perimeter interceptor channel has a low spot at about 1573 m AMD which is 1.8m below the Stage I and II eastern perimeter interceptor channel where it would join. The western sector of the Stage 6 western perimeter interceptor channel has a low spot at 1569 m AMD which is 0.5 m below the Stage III perimeter interceptor channel where it would join.

Where practicable the low spots will be backfilled and high spots removed and the channel depth will be dependent on achieving where possible, gravity flow to the two outlets which are the existing interceptor channels. Also, any low spots in the borrow area adjacent to the perimeter access road will also be backfilled with boulders and cobbles rejected from the Type A glacial materials. It is not anticipated that the channel in the glacial till or bedrock will be significantly deeper than 1 m and the side slopes would vary from 1H:2V to 1H:1V to 3H:2V depending on the material excavated.

For Phase 1, the road and perimeter interceptor channel could be raised and constructed in fill used for the next phase (Dwg 6.16). However, that could result in ponding in low spots against the construction fill and also water from the channel seeping into the fill.

6.14 Embankment Stability Modelling

The stability analysis for the new cell was carried out using commercially available limit-equilibrium slope stability software, SLOPE/W version 7.15 (Reference 6.3). The analytical method used was Morgenstern and Price method of slices, which satisfies both force and moment equilibrium. The stability analysis indicates that the dam walls have adequate Factor of Safety (FoS) under the modelled conditions. Details of the modelling approach and the results are presented below.

6.14.1 Model Geometry

The model geometry is based on a typical cross-section of perimeter dam wall at full height (67.29 m AOD) including the bench at 1586.0 m AMD. The Stage 6 height is approximately 20 m above the glacial till/bedrock foundation level. The modelled sub-surface conditions are based on the geotechnical investigation information at the site of the existing TMF.

6.14.2 Geotechnical Parameters

The geotechnical strength parameters adopted in the stability modelling are presented in the following table and the geometric configuration presented in Figure 7.

Table 13: Geotechnical Parameters for Stability Analysis

Material Type	Unit Weight (kN/m^3)	Effective Strength	
		c' (kPa)	ø' (deg)
Type A	19	0	32
Type D1 and D2	21	0	37
Tailings	18	0	32
In-situ Glacial Till	20	5	32
Limestone Bedrock*	-	-	-

*Limestone bedrock was considered impenetrable by slope failure.

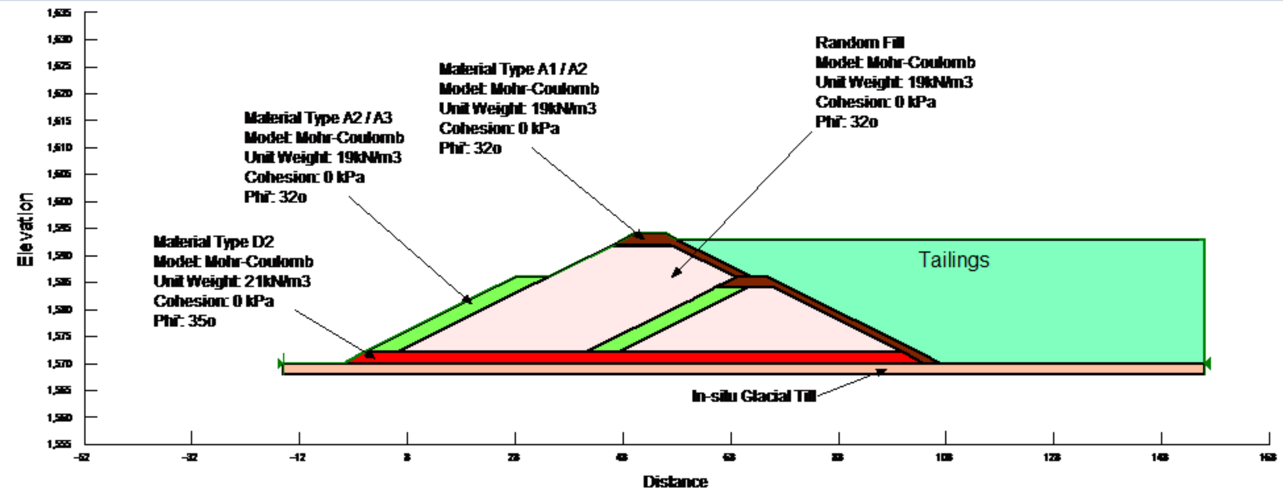

Figure 7: Model with material details

Effective strength parameters are typically used to assess long-term stability (after closure), whereby the excess pore pressures developed in any fine grained soils have dissipated.

6.14.3 Stability Modelling

The stability of the TMF wall for each option was considered under the following conditions:

Long-term downstream embankment stability, with tailings level 1 m below the crest. The HDPE liner was assumed to be intact and an assumed phreatic surface was drawn down through the embankment by the downstream drainage blanket and the rock used in downstream construction.

Long-term downstream embankment stability, with tailings level 1 m below the crest. The HDPE liner was considered to undergo complete failure and the drainage system was no longer free draining and the phreatic surface exits the dam wall in the slope at one third of its height. This is a very extreme scenario which is unlikely to develop.

Further analyses were undertaken using pseudo-static conditions corresponding to a 0.06 g acceleration for the liner intact.

6.14.4 Results of Stability Modelling

The results of the stability analyses are presented in Table 14 below in the form of the FoS for the most critical slip surface and in Figures 8 to 11. The required minimum FoS was exceeded under all conditions analysed.

Table 14: Stability Modelling Results

Case	Condition	Location	FOS
1	Long-term, tailings 1m below crest, phreatic surface at the toe of the dam. Static condition	Downstream	1.63
2	Long-term, tailings 1m below crest, phreatic surface at the toe of the dam. Ground Acceleration 0.06g. Pseudo static condition.	Downstream	1.38
3	Long-term, tailings 1m below crest, phreatic surface exists 1/3 dam height. Static condition.	Downstream	1.16

4	Long-term, tailings 1m below crest, phreatic surface exists 1/3 dam height. Ground Acceleration 0.06g. Pseudo static condition.	Downstream	0.99

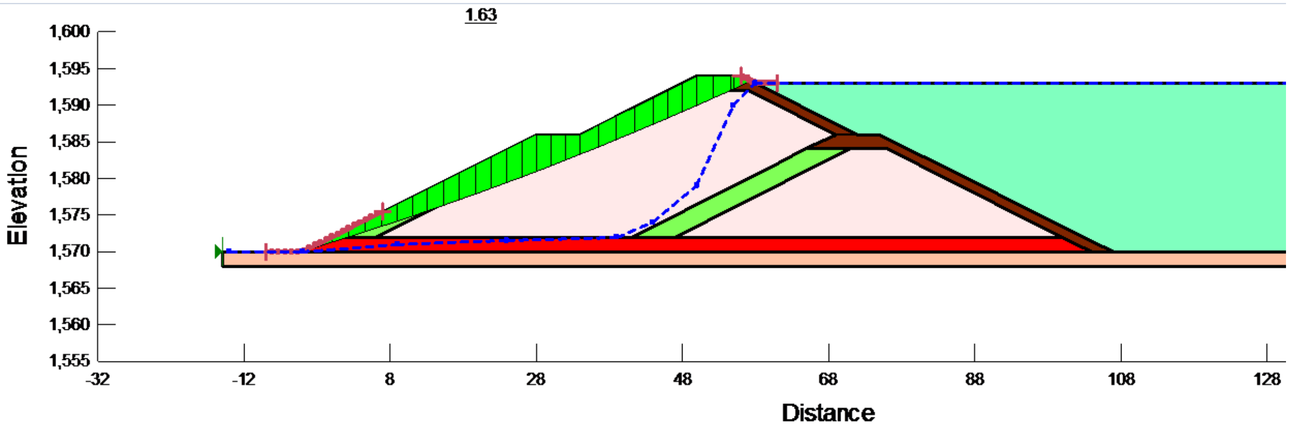

Figure 8: Static Analysis with Drainage Blanket operational (Phreatic Surface at the Dam toe)

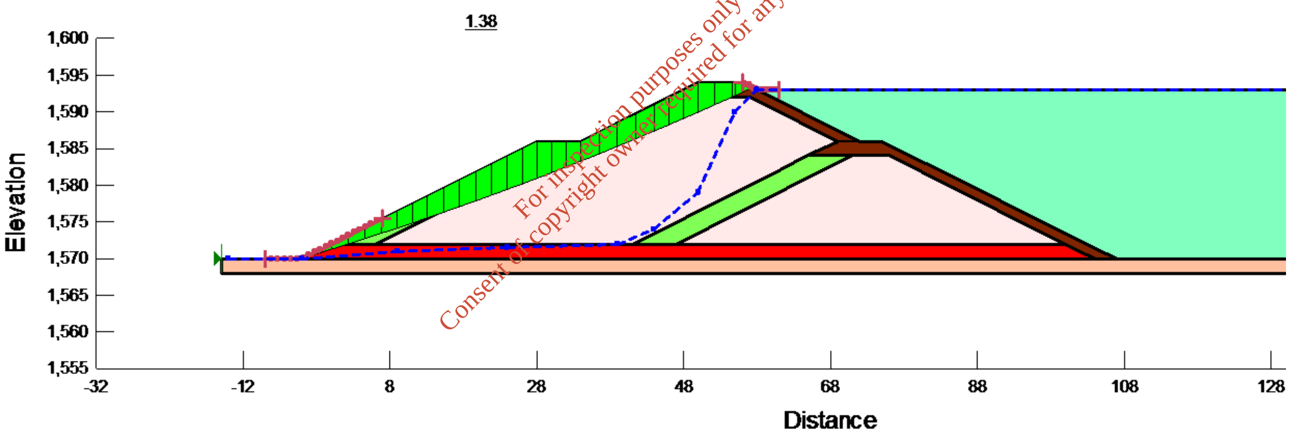

Figure 9: Pseudo – Static analysis with Drainage Blanket operational (Phreatic Surface at the Dam toe)

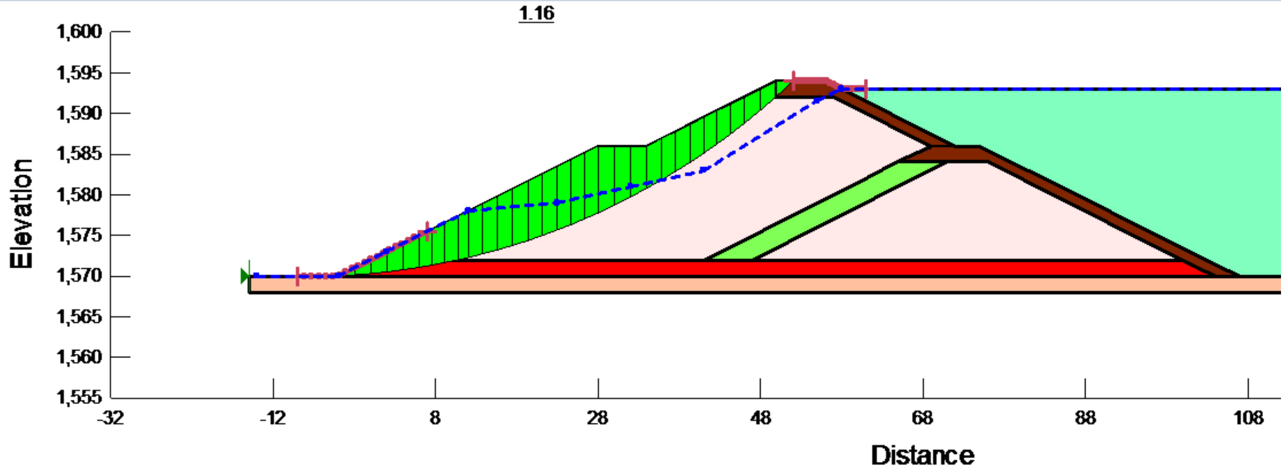

Figure 10: Static Analysis with Drainage Blanket blocked (Phreatic Surface at 1/3 Dam height)

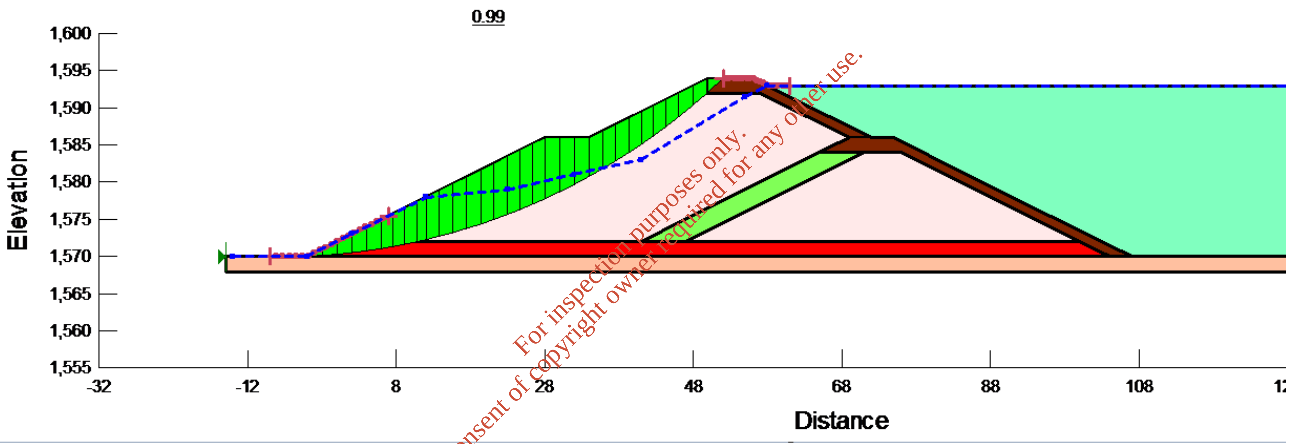

Figure 11: Pseudo - Static Analysis with Drainage Blanket blocked (Phreatic Surface at 1/3 Dam height)

The factor of safety for the static and pseudo-static cases for the internal drainage system operational are satisfactory. Where the drainage system is non-operational, the factor of safety for the static condition is satisfactory. However, if an earthquake occurs, then the factor of safety is 0.99 which under these conservative conditions, is acceptable. As the dam wall incorporates a downstream drainage blanket and an upstream zone of Type A1 and A2 low permeability material together with the lining system, it is very unlikely that the phreatic surface would exit at a point one third of the dam height.

6.15 Spillway

This type of perimeter dam used for tailings storage, which has no external catchment area, is normally operated without spillways. There is always adequate control on the tailings water level which can be achieved by adjusting the discharges into the TMF and removal of tailings water by pumping from the TMF to the plant site. However, a spillway is required post-closure to control water levels and discharges.

The spillway will be constructed prior to closure of the new cell. The level of the tailings water in the TMF will be dependent on the discharge from the mill into the facility, water reclaim from the facility and the effective rainfall. Both the discharge and reclaim are controlled by the mill operators and can be adjusted at short notice. The probable maximum precipitation (PMP) for this area will be of the order of 250 mm over 24 hours which is readily retained within the freeboard of the facility.

The spillway will be located and designed in detail closer to the end of the life of the Stage 6 facility. The cap of Stage 5 adjacent to the northern wall will be at about 67.29 m AOD. Ideally, the spillway would be located in the south east corner of the Stage 6 dam wall where the reclaim pumps are located. For premature closure the wall would be cut down to accommodate a lower tailings level. The water will be discharged into the eastern perimeter interceptor channel. However, the final location will be dependent on the location of the return water pumps which will ensure the lowest tailings level and the location of the sand ramp where the shortest route from the mill is also the south east corner of Stage 6. If this was chosen for the sand line, then the return pumps and spillway would be located in the south west corner of the Stage 6 dam wall.

7.0 OPERATIONS

7.1 General

Stage 6 is composite lined and special care will be required from the operators in order to prevent damage to the lining system. Generally, the typical causes of damage are from dropping equipment on the lining, dropping and dragging pipelines or the installation and movement of the return water pumps.

Stage 6 will be operated similar to the existing TMF by discharging the tailings from spigots from the dam crest. The spigots would be open sequentially and the tailings will be discharged uniformly over the TMF. The discharge points should be every 50 m rather than currently 70 m and two tailings lines should be operated around the facility in opposite directions. Unlike the main Stage 5 operation, there will be a considerable depth between the crest and the basin floor initially. A slotted discharge pipe from the valve to the floor of the basin will be required (Dwg. 7.1). The slotted discharge pipe will be anchored on top of the in filled tyre surcharge which is place directly on 500 g/m^2 carbon rich geotextile which in turn is placed on the lining. Alternatively to slots, the end of the pipe could be cut off as the tailings rises although this would be a more hazardous operation as the operator would have to climb down the dam slope using the tyres supporting the pipe with a chain saw

The tailings will be placed to a final elevation of 1593.0 m AMD (Phase 2) and it is expected that the tailings will beach at a slope gradient of about 0.5% towards the reclaim pumps. The beach slope will be checked during operations. During Phase 1 and Phase 2, the tailings level should not exceed a depth less than 1m below the dam crest. The pond water level should be kept to 1.5m below the crest during operations particularly where the water is against the lining and at the corners of the facility. Spray from waves in the corners can be problematic and where possible a tailings beach should be developed to prevent this occurrence.

Where the reclaim pumps are located, a continuous protection of in filled tyres will be required some 20 m wide. This is illustrated on Drawing 7.1. This will need to be further detailed after discussions with yourselves on the method to be adopted particularly if a barge is to be used. The optimum location of the reclaim pumps is firstly along the existing northern wall in the south east sector where the future spillway can discharge into the eastern perimeter interceptor channel. The second option is along the existing northern wall but in the south west sector where the future spillway can discharge into the western perimeter interceptor channel. The existing northern wall is preferable as the composite lining and lining protection can be installed for all two phases from the beginning. This is further discussed in Section 8.

The sand line also poses a potential challenge as this would be discharged from a 600 mm diameter pipe. Again the liner will be protected with a minimum 20 m wide continuous in filled tyre protection as indicated in Drawing 7.1. Ideally this should be located on the existing northern wall where the composite lining and lining protection can be installed for both the phases from the beginning. This is further discussed in Section 8. During previous operations, the sand line has been extended into the facility because of the higher beach slope generated by the depositing sands. The other issue relates to discharging from the base of the facility. It would be possible to slot the pipe as per the normal spigot lines. But it is likely that the sand from the total tailings would form a delta at the discharge point and at a higher elevation than the surrounding settled slimes which could then be prone to dusting. The deposited sand could be redistributed by placing a slimes spigot adjacent to the sand line. It would also be possible to construct a berm into the facility, on the tailings at a later stage of the life of Phase 1 and also for Phase 2 if the sand line had to be extended into the facility.

To prevent damage of the lining at the point of discharge and to provide surcharge for the base, the basin should be filled with a minimum of 1 m of water equating to approximately 430,000 m^3.

7.2 Piezometer and Groundwater Monitoring

The groundwater monitoring system for the TMF is presented on Drawing 7.2 and indicates piezometers and monitoring wells at 5 locations. The monitoring wells will be located beyond the downstream toe and adjacent to the perimeter access road but subject to any anomalies indicated by the resistivity and microgravity survey which would indicate the presence of any water bearing fissures. The piezometers would be installed into the foundations and above the drainage blanket at the base of the dam fill from the crest at 1586 m AMD for the first phase of Stage 6. These would then be replaced during the construction of the second phase of Stage 6 with two piezometers installed on the crest and two piezometers on the intermediate bench. The construction of the piezometers and wells are shown on Drawings 7.3.

The water level and water quality sampling will be undertaken in accordance with the IPPC license.

7.3 Settlement Monitoring Points

Settlement of the dam wall will be undertaken at seven locations as shown on Drawing 7.2 and the construction shown on Drawing 7.3

8.0 CONSTRUCTION

8.1 General

Unlike Stages 4 and 5, there is no time restraint for placing the construction materials because there will be little to no excess pore pressure development in the foundation glacial tills compared to Stages 4 and 5 founded on tailings. Therefore, there is no restriction on the rate of rising of the dam wall or on the size or number of trucks. We would expect the size of dump trucks to be 40 tonnes rather than the 25 tonnes used to build the Stage 5B raise. We would also expect the dozers spreading on the dam wall to be the size of a D8. Using larger plant and increasing the number of dump trucks will significantly increase productivity.

The total bulk volume of fill to be removed from within the basin area of Stage 6 is 757,000 m^3. The total bulk volume of construction material to build Phase 1 is 530,000 m^3 plus 27,000 m^3 towards ramp construction and a further 200,000 m^3 to allow for capping of Stage 5A. For the second phase of Stage 6, to an elevation of 1594.0 m AMD the volume of construction fill material is 1,170,000 m^3. A further 300,000 m^3 is required for capping of Stage 5B. Assuming a construction season per year of 30 weeks, the average volume of material placed for the construction of Stage 6 only for a given number of seasons is tabulated below.

Table 15: Average Production Values For Construction of Stage 6

Construction Fill Vol. m^3 Phase 1 1586 m AMD	**557,000**	**557,000**	**557,000**	**557,000**	**557,000**
Season	1	1.5	2	2.5	3
Weeks	30	45	60	75	90
Average Vol. m^3/week	18,567	12,378	9,283	7,427	6,189
Construction Fill Vol. m^3	**1,170,000**	**1,170,000**	**1,170,000**	**1,170,000**	**1,170,000**

Phase 2 67.29 m AOD					
Season	1	1.5	2	2.5	3
Weeks	30	45	60	75	90
Average Vol. m^3/week	39,000	26,000	19,500	15,600	13,000

To place a weekly average of above 18,000 m^3 of construction fill would be optimistic although very much subject to the weather conditions, plant levels and distance to the borrow areas. During the construction of Stage III, the maximum fill rate per week achieved was 24,000 m^3 although the overall average over the season was nearer 15,000 m^3/week and that was over a relatively dry construction season. Over two seasons, the average was closer to 12,000 m^3/week and for three seasons the average reduced to 10,000 m^3/week.

8.2 Phase 1 Construction

If construction commenced in the spring of 2018, the earthworks could be completed by mid-summer of 2019, a total of 45 weeks and requiring an average weekly production rate of 12,378 m^3/week. This would allow a further 15 weeks to complete the remaining lining over the basin area, the dam walls and install the pumps and pipeline for discharge by the beginning of 2020.

The total area to be lined is about 530,000 m^2 of which about 81% is on the basin floor. Liner production is again dependent on the weather and is impacted by rainfall, wind and temperature. Placing lining on the basin floor is quicker than placing liner on the side slopes because of the greater length of roll that can be placed and the less welds required. We would expect an average weekly production rate of at least 20,000 m^3 for installation of the lining system based on four crews. This equates to a 27 week construction duration. Thus, sections of the wall and floor will need to be prepared by the beginning of the 2019 construction season to allow commencement of lining installation.

The lining will need to extend onto the existing dam wall to Stage 4 at an elevation of 63.29 m AOD as there is no bench at 1586 m AMD, the crest elevation of Stage 6 Phase 1.

The maximum number of seasons if work commenced in the spring of 2017 is essentially 2.5 which would equate to 3,400 m^3/week which is reasonable. This operation will be impacted by excess pore pressure development in the tailings so a slow construction pace is required.

8.3 Phase 2 Construction

The volume of fill required to construct Phase 2 is 1,170,000 m^3 plus an additional 300,000 m^3 of material to cap Stage 5B.

From Table 15, the weekly construction rate for 3 seasons is nearly 13,000 m^3 which should be achievable.

The area of the dam wall to be lined is 74,000m^2 which should be completed in about 7 weeks.

The Phase 1 facility needs to be operational during the construction of Phase 2. During the earthworks phase of construction, the pipes discharging into Phase 1 of Stage 6 will be located at edge of the Phase 1 crest. There is a provision of 3m between the upstream crest edge of Phase 1 and the upstream toe of Phase 2. However, once the Phase 2 earthworks is complete, the pipes are removed (dragged up the dam wall face) prior to preparation of the slope for receiving the composite lining. During this stage of the operation, the composite lining would need to have been placed on the existing northern dam wall (dividing wall between existing facility and Stage 6) so that tailings discharge can continue into the facility while the composite lining is being installed on the new dam walls of Phase 2.

Both the reclaim pump and sand line location require continuous tyre protection (Dwg. 7.1) of the composite lining system. The reclaim pumps and sand line will need to operate continuously and it would be appropriate to locate them on the existing northern wall and install the composite lining plus protection to the final dam crest height of 67.29 m AOD. The pipelines for these operations are currently operating on Stage 5 at an elevation of 67.29 m AOD and will need to be brought down the existing northern wall to the Stage 4 crest at 63.29 m AOD. The Phase 1 of Stage 6 is at a crest elevation of 1586 m AMD.

9.0 CLOSURE

Stage 6 will be capped in accordance with the procedures developed for capping Stages 5A and 5B (Reference 10.1). This will include approximately 350 mm of capping material. The arrangement of drainage and shape of the cap will need to be detailed once the location of the sand line and reclaim pumps are determined by Tara Mines. The spillway arrangements will be engineered once the facility is approaching closure. A detailed design of the closure is currently being prepared separately.

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Report Signature Page

GOLDER ASSOCIATES (UK) LTD

Roger White Prinicpal/Project Manager	Martyn Willan Geotechnical Engineer

Date: March 2016

RSW/MW/gg

APPENDIX A

Drawings

Drawing No. 1.1 – Site Location Plan

Drawing No. 1.2 – Mine Site and Randalstown Tailings Facility Location Plan

Drawing No. 1.3 – Plan of Stage 6 Raise

Drawing No. 1.4 – Plan of Dam Stages and the Northern and Sevenfields Borrow Areas

Drawing No. 2.1 – Randalstown Tailings Dam Surface Geology

Drawing No. 4.1 – Principal Drainage Flowpaths in the Randalstown Vicinity

Drawing No. 6.1A – Stage 6 TMF Layout Plan Phase 1

Drawing No. 6.1B – Stage 6 TMF Layout Plan Phase 1 and 2

Drawing No. 6.1C - Stage 6 TMF Layout Plan Phase 1, 2 and 3

Drawing No. 6.2 – Permanent Ramp Locations

Drawing No. 6.3 – Typical Dam Wall Section

Drawing No. 6.4A – Western and Eastern Connections Details Phase 1

Drawing No. 6.4B – Western and Eastern Connections Details Phase 2

Drawing No. 6.5 – Stage 6 Sections Through Dam Wall Phase 1 and 2

Drawing No. 6.6 – Stage 6 Plan Showing Chainage of Sections

Drawing No. 6.7 – Typical Section of Existing Northern Dam Wall

Drawing No. 6.8 – Perimeter Roadway, Fence and Interceptor Channel.

Drawing No. 6.9 – Diversion of Blakes Stream

Drawing No. 6.10 – Existing Powerlines

Drawing No. 6.11– Survey of the Borrow Area Within the Footprint of Stage 6 Showing Cut and Fill Depth to 1573 mAMD

Drawing No. 6.12 –Typical Composite Lining System Detail

Drawing No. 6.13 – Tyre Ballast for Embankment Slopes

Drawing No. 6.14 – Longitudinal Profile Along Perimeter Interceptor Channel

Drawing No. 6.15 – Plan Showing Perimeter Interceptor Channel Chainage

Drawing No. 6.16 – Raised Perimeter Roadway and Interceptor Channel Constructed in the Next Phase

Drawing No. 7.1 – Pipeline and Reclaim Pump and Sand Line Protection

Drawing No. 7.2 – Piezometer and Monitoring Location Plan and Section

Drawing No. 7.3 – Instrumentation Details

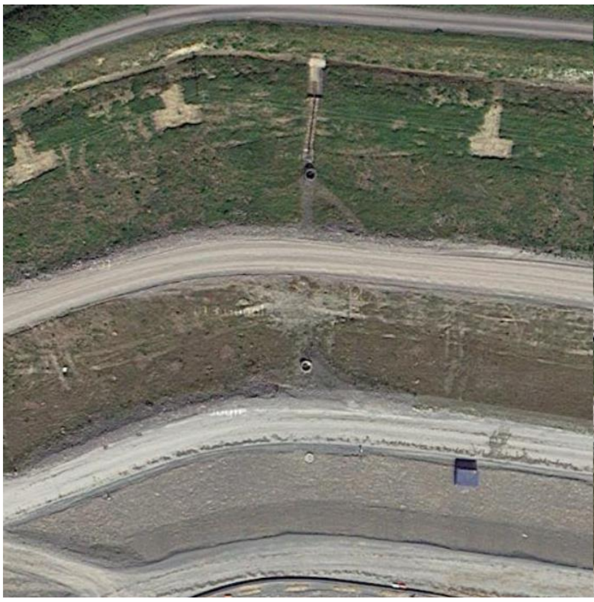
APPENDIX B

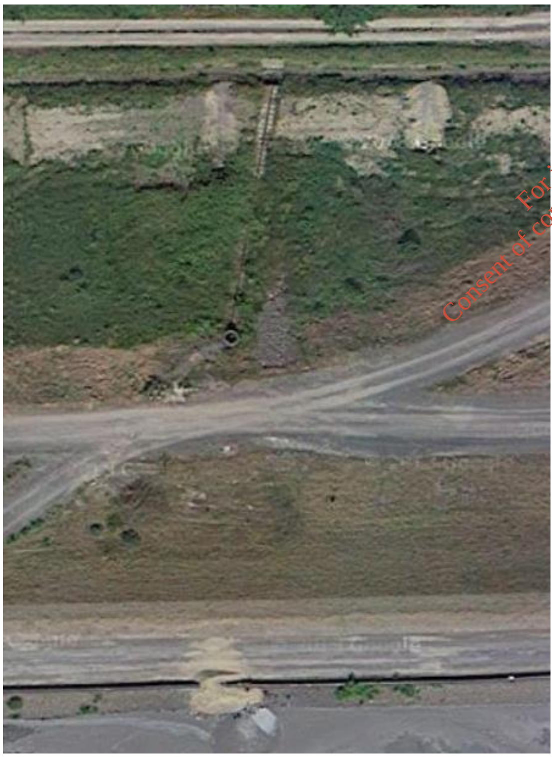
Trial Pit Logs

APPENDIX C

Aerial Photograph

Photos showing locations of finger channels on the downstream side of the existing northern wall to be backfilled





Photos showing locations of chutes and manholes on the downstream side of the existing northern wall to be backfilled

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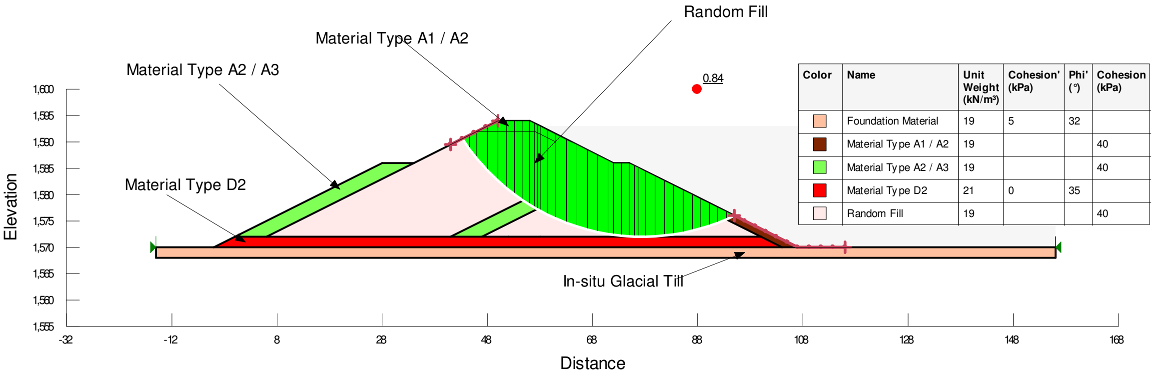

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	Material Type A2 / A3	19			40
	Material Type D2	21	0	35	
	Random Fill	19			40

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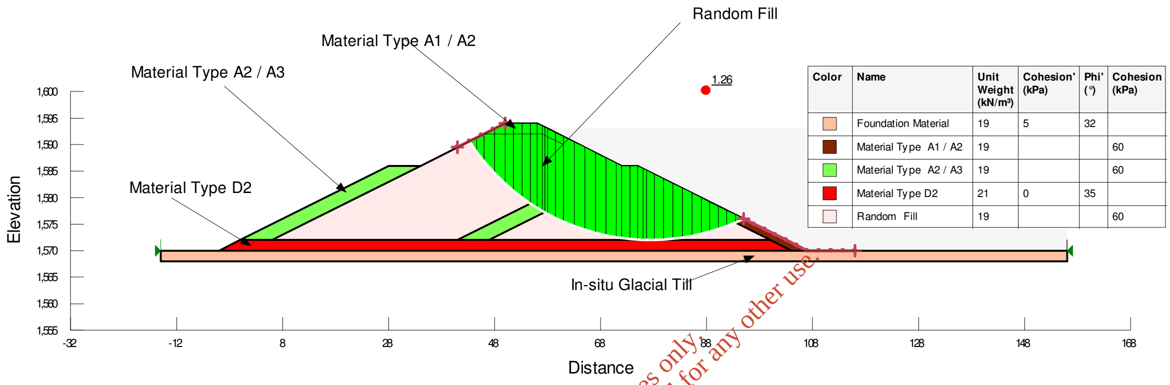

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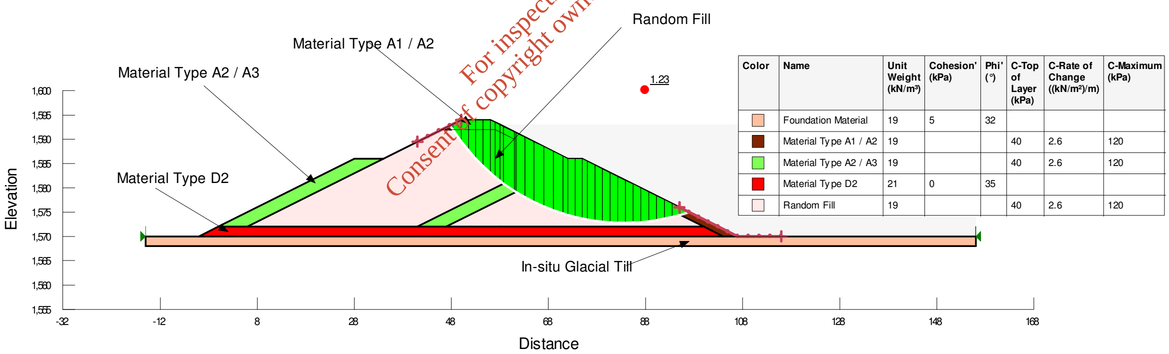

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	Material Type A1 / A2	19			40	2.6	120
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	Material Type D2	21	0	35			
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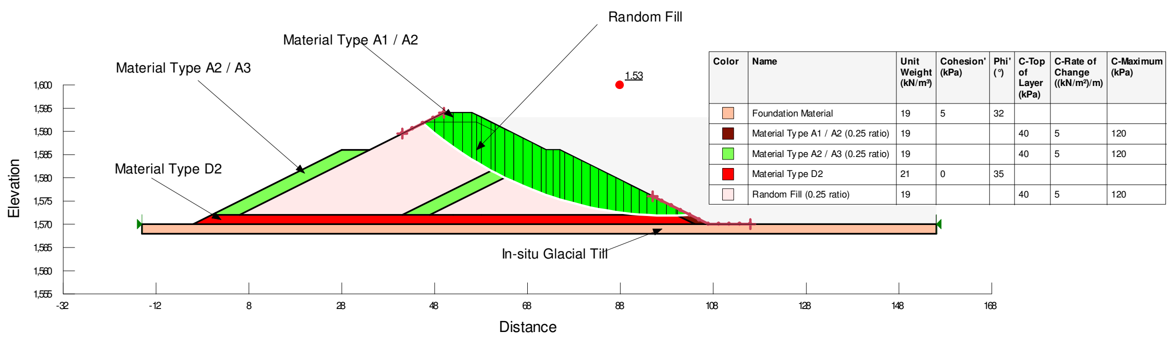
Case 3



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	Material Type A1 / A2 (0.25 ratio)	19			40	5	120
	Material Type A2 / A3 (0.25 ratio)	19			40	5	120
	Material Type D2	21	0	35			
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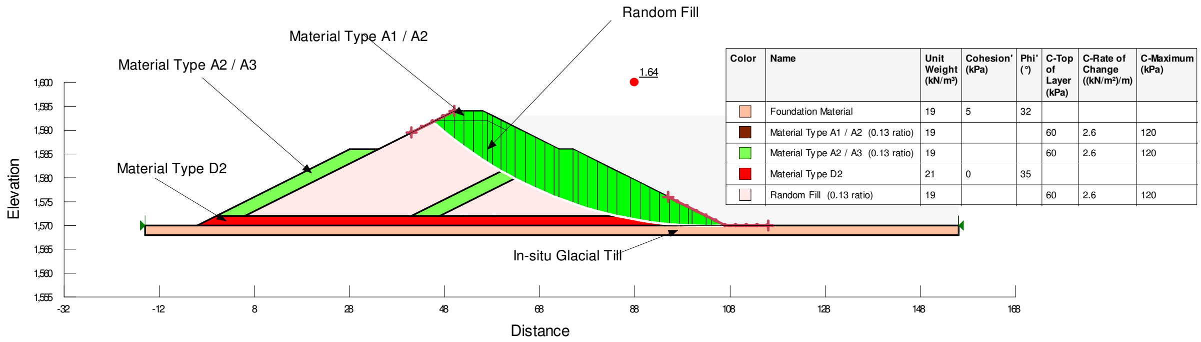

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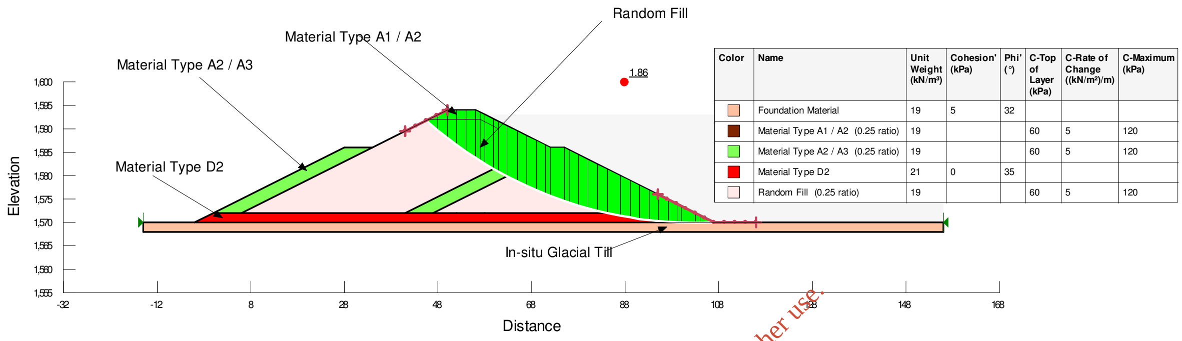

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	Material Type A1 / A2 (0.25 ratio)	19			60	5	120
	Material Type A2 / A3 (0.25 ratio)	19			60	5	120
	Material Type D2	21	0	35			
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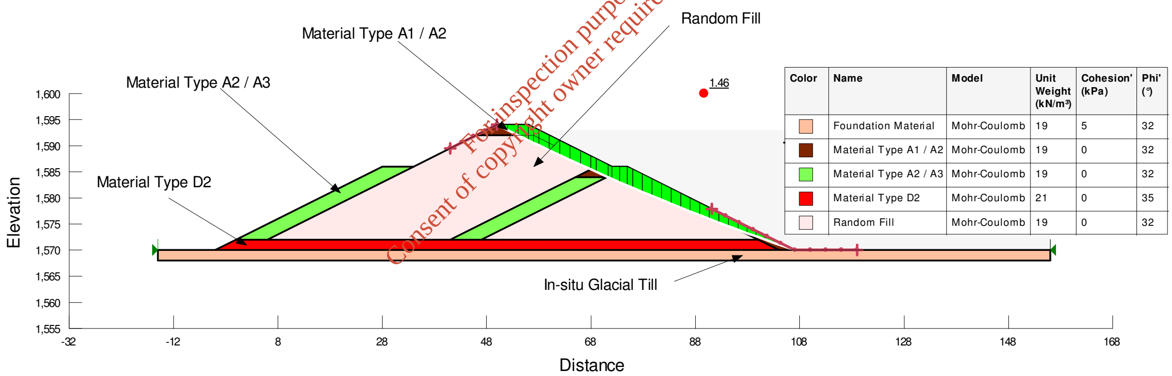

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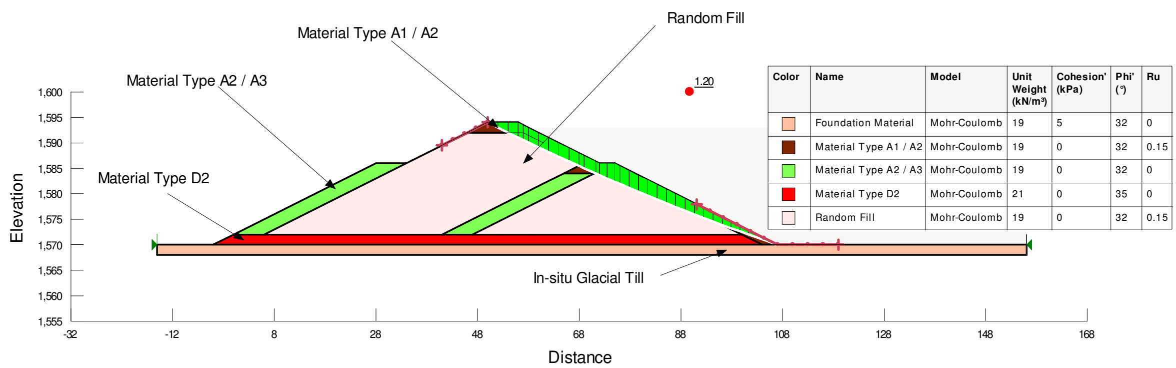
Case 7



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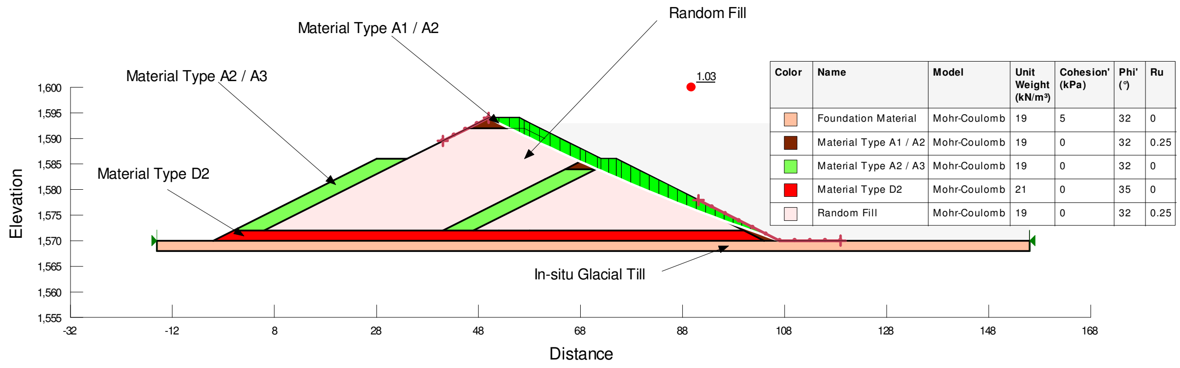
Case 8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88
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Material Type D2 | Mohr-Coulomb | 21 | 0 | 35 | 0
Random Fill | Mohr-Coulomb | 19 | 0 | 32 | 0.25

Case 9

Appendix to FIR Item 11

- Tailings Management Facility Dam Break Study *(Golder Associates)*

July 2016

BOLIDEN TARA MINES LIMITED

Tara Tailings Management Facility Dam Break Study

Submitted to:
Boliden Tara Mines Limited
Knockumber
Navan
Co. Meath
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Report **Number** 1651706.500/A.1

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REPORT

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APPENDICES

1.0 INTRODUCTION

Boliden Tara Mines Limited (Tara) has commissioned Golder Associates (UK) Limited to undertake a dam break analysis study for the Randalstown Tailings Management Facility (TMF) which includes Stages 5A, 5B and 6. The purpose of the study is to determine the extent of flooding and potential pathways of an uncontrolled discharge of tailings into the downgradient environment during operations and closure. The results of the dam break analysis will be used to aid in future emergency planning for the Site.

2.0 BACKGROUND

The TMF, which is raised above surrounding ground levels, has been developed in stages, initially commencing in 1974. Stage 5A and 5B have been constructed on top of the existing TMF, to a crest level of 1,594 m above sea level (asl) and Stage 6 (planned) will be located north of the existing TMF, also to a crest level of 1,594 m.

Each stage will hold approximately 500,000 m^3 of supernatant water during operation. At closure it is planned that an engineered cap and growth media will be placed above the tailings. Surface drains will manage surface water runoff so that the closed facility is effectively drained. Spillways will be installed at closure to manage the 1 in 10,000 year flood event. It is planned that the closure profile of Stage 5A and 5B will slope southwards towards a spillway from Stage 5A linking 5B. Another spillway will then discharge flood water from Stage 5B, to the Simonstown Stream, south of the TMF. The proposed Stage 6 spillway is located at the south-eastern corner of Stage 6, discharging to a perimeter drain.

Figure 1 presents the TMF layout, which includes the proposed Stage 6. Drawing 1 presents the location of the TMF and nearby watercourses.

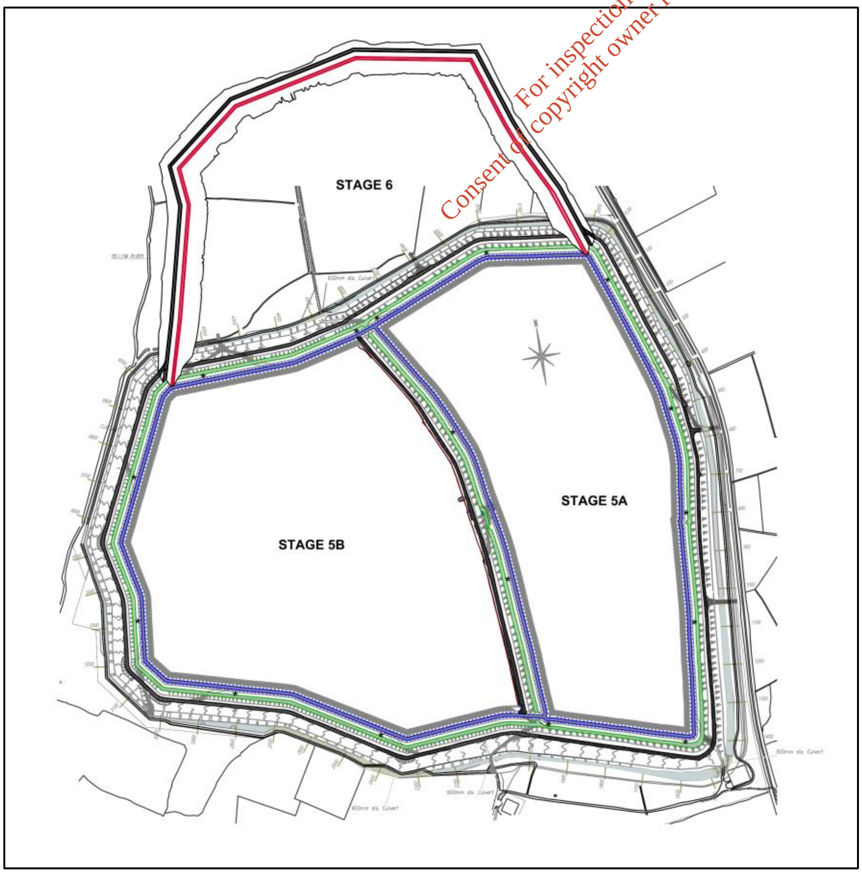

Figure 1: TMF Layout

3.0 DAM BREAK MODELLING

Due to the proximity of downstream identified receptors (residential properties) on all aspects (north, east, south and west) of the planned TMF, the critical (or 'worst case') scenario considered assumes failure occurs in the immediate proximity to nearby receptors. Drawing 2 and 3 presents the locations and details of the nearby receptors. Given the small catchment of the TMF, active management during operations, designed spillway capacity and proposed closure profile, the likelihood of overtopping is very small and therefore only piping failure has been considered in this study. Operational and closure failure scenarios have been considered for each TMF cell. Due to the number of nearby receptors, Stage 5B and Stage 6 consider two separate breach location scenarios. Stage 6 failure scenarios are located either side of a topographical high point. Five failure scenario locations have been considered during both operations and closure, totalling 10 failure scenarios and are as follows:

- Stage 5A – East;
- Stage 5B – West;
- Stage 5B – South;
- Stage 6 – North-west; and
- Stage 6 – North-east.

Sequential failure of multiple Stages has not been considered.

The aforementioned failure scenarios have been modelled using the following three steps:

- **Establish dam break parameters;**
  - Dam break geometry parameters have been estimated using the Froehlich Method (Froehlich, 2008). The parameters are a required input for determining dam break hydrographs.
- **Develop dam break hydrograph;**
  - Dam break hydrographs were determined using HEC-HMS (Hydrologic Engineering Centres Hydrologic Modelling System) software.
- **Assess potential impact downstream (flood routing);**
  - Dam break hydrographs were applied as boundary conditions to XPSWMM/TUFLOW (2D hydraulic model), providing a representation of downstream flood extents and time to inundation, based on the variable topography surrounding the TMF.

3.1 Dam Break Modelling Process

Dam break hydrographs have been estimated using the level pool routing function in HEC-HMS. Input parameters are provided in Section 3.1.1, which includes failure formation time and breach geometry.

3.1.1 Dam Break Parameters

Dam break parameters have been determined using the Froehlich method; an empirical model used to determine dam failure geometry. The Froehlich Method uses the following equation:

Average Breach Width = $0.27k_oV_w^{0.32}h_b^{0.04}$

Where:

k_o = 1.3, if overtopping failure, 1.0, if piping failure

V_w = Reservoir Volume Released (m^3)

h_b = Breach Height (m)

Side slope = 1.0, if overtopping failure, 0.7, if piping failure.

The dam break geometry for each scenario is presented in Table 1.

The volume of tailings released from the facility is based on the methodology presented in *Tailings mobilization estimates for dam breach studies* (Knight Piesold, 2015) and the Froehlich Method (2008). The mass of mobilized tailings is estimated assuming full mixing of supernatant water, with the tailings solids and interstitial water at a selected solids content. In this instance, instantaneous mixing at 65% solids content by mass has been applied, which is considered a conservative upper limit to Newtonian-like fluid behaviour. The supernatant water also includes the 24 hour 10,000 year return period rainfall event which is based on Met Eireann rainfall data for Randalstown. The Met Eireann rainfall frequency analysis data yields a 24 hour 10,000 year rainfall event of 210 mm (Figure 2).

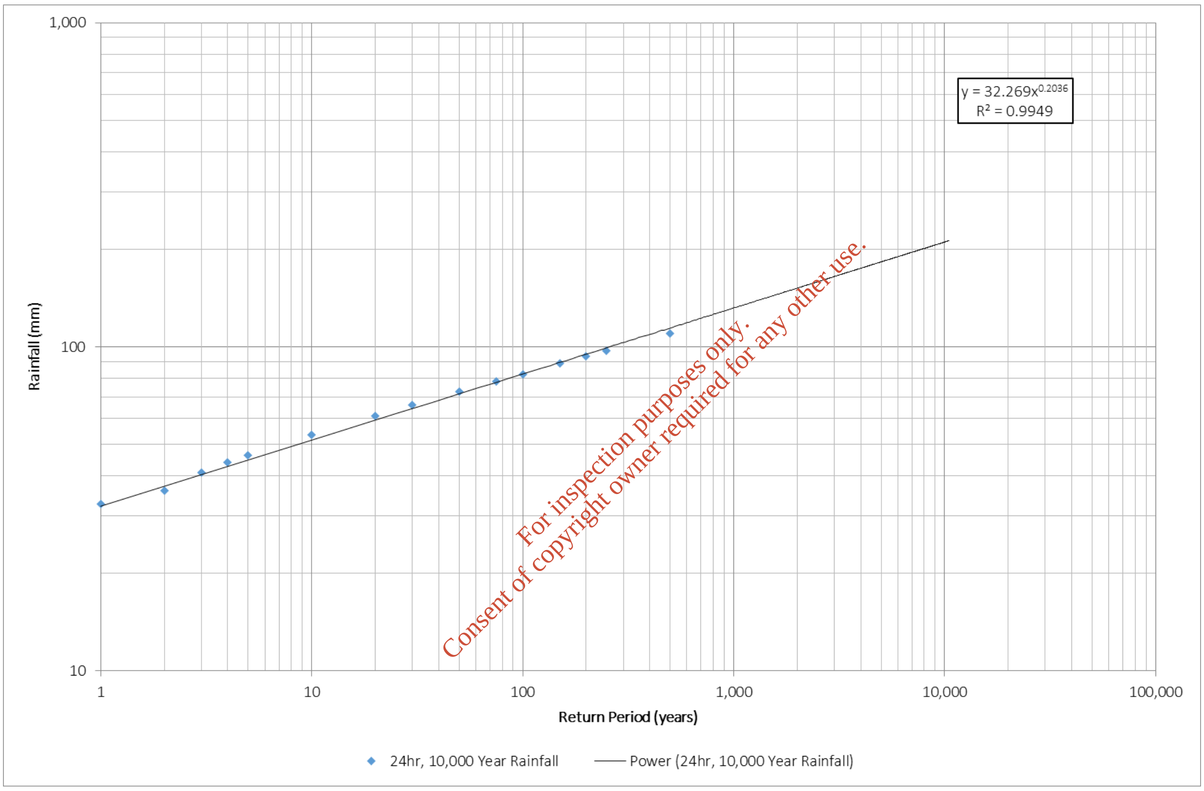

Figure 2: Rainfall Data

Failure formation time has been estimated using the equation developed by Froehlich (2008). The equation is presented below:

$$T = 0.0175\sqrt{\frac{Vw}{gHb^2}}$$

V_w = Reservoir Volume Released (m^3)

h_b = Breach Height (m)

g = gravity (m/s^2)

Breach depth is based on the current crest elevation of 1,594 masl, which also applies to the closure scenario. Breach formation times are presented in Table 1.

Table 1: Dam Break Parameters

Scenario	Total Volume Released (Mm³) [1]	Supernatant Water (Mm³)	24hr 10,000 yr Rainfall (Mm³)	Breach height (m)	Failure Formation Time (hr)	Side Slope (1:H)	Breach Base Width (m)
Stage 5A, Operation	2.711	0.5	0.126	20	0.46	0.7	20.8
Stage 5A, Closure	0.547	-	0.126	20	0.21	0.7	6.9
Stage 5B, Operation	2.893	0.5	0.168	20	0.48	0.7	21.6
Stage 5B, Closure	0.729	-	0.168	20	0.22	0.7	8.9
Stage 6, Operation	2.694	0.5	0.122	20	0.46	0.7	20.8
Stage 6, Closure	0.530	-	0.122	20	0.2	0.7	6.7

[1] ***Total volume releases includes supernatant water stored (500,000 m3), water stored in the tailings (estimated at 65% solids content) and the 24 hour 10,000 year return period rainfall event for Randalstown (210 mm depth).***

3.2 Model Results – Dam Break Hydrographs

The level-pool routing function in HEC-HMS was used to generate dam break hydrographs based on the parameters presented in Table 1 (breach geometry and failure formation time). Figure 3 below presents the hydrographs derived from the HEC-HMS modelling that were subsequently applied to the XPSWMM/TUFLOW model as an upstream boundary condition.

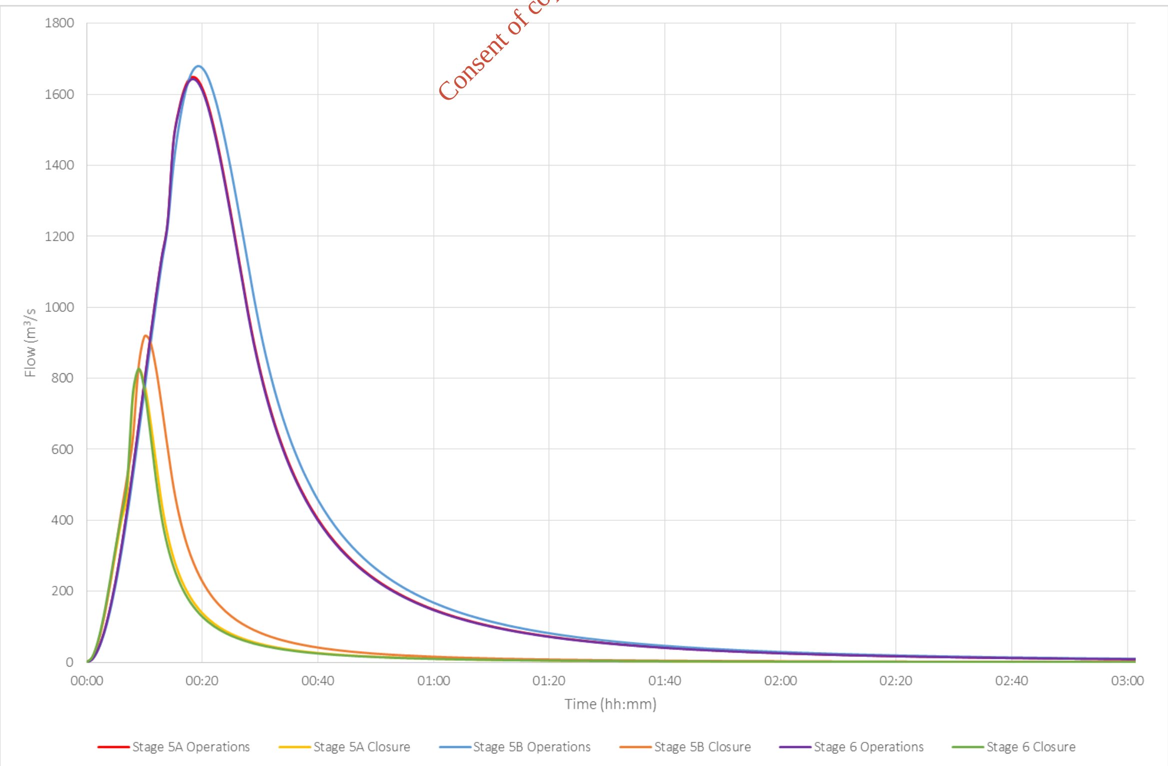

Figure 3: Dam Break Hydrographs

Peak flow is greater during operation due to the large volume of water contained within the TMF, primarily supernatant water. Note that dam break models are very sensitive to each of the input parameters, particularly the formation time; reducing the formation time of the failure and keeping all other parameters equal can lead to significantly higher peak flows.

3.3 Flood Routing

Flood routing downstream of a breach from the TMF has been modelled using XPSWMM/TUFLOW. TUFLOW is a 2D flood and tide simulation model. The 2D solution, which has been utilised to model flood routing of a dam break, simulates the hydrodynamics of floods based on the Stelling finite difference, alternating direction implicit (ADI) scheme that solves the full 2D free surface shallow water equations.

3.3.1 Model Inputs

The model was constructed using a grid where each point represents the modelled topography. The hydrographs applied to the model are presented in Figure 3. Flow is then calculated between each cell. Model inputs are provided in Table 2.

Table 2: TUFLOW Model Inputs

Parameter	Model Input	Comments	Sources
2D Grid	12 m Cell Size	Digital Elevation Model (DEM)	Tara
Time Step	3 seconds	-	-
Downstream Boundary Condition	Located approximately 3 - 4 km downstream, along River Blackwater.	Downstream boundary condition set to automatically calculate based on Manning's equation (Manning's is an empirical formula used for estimating open channel flow and is a function of channel velocity, flow area, surface roughness and channel slope.) and a downstream slope, derived from the DEM, of 0.005 m/m.	-
Upstream Boundary Condition	Hydrographs developed using HEC-HMS.	See Section 3.1.2.	-
Roughness	0.05	A relatively high Manning's coefficient (resistance of ground surface to the flow of water) was selected for the surrounding land use (arable and pasture) due to the presence of mobilised tailings.	HEC-RAS Hydraulic Reference Manual (HEC-RAS, 2010).

The Yellow River and Simonstown Stream have been modelled in 2D by cutting the stream into the DEM. The interceptor ditch circumnavigating the existing Stage 5A and 5B has been extended around the proposed Stage 6 of the TMF. A bund along the northern extent of the Site has also been incorporated into the model by raising ground levels by 2 m.

The River Blackwater is represented in the model by the data from the DEM. It is assumed the DEM provides the elevation of the water surface at the time of survey. It has therefore not been possible to fully assess the flow in the River Blackwater i.e. tailings and river flow. The likely impact on flows in the Blackwater have been quantified later in this report.

3.4 Model Results – Flood Routing

The maximum extent and depth of flooding is presented in Drawings 3 to 13.

Table 3 provides a summary of modelling results and flood propagation for various locations downstream of the TMF. These locations are presented in Drawings 3 to 13.

Table 3: Model Outputs

Section Line	West	South	East	RB1	RB2
Stage 5A, Dam Break East, Operation Phase, Drawing 4					
Peak Flow (m^3/s)	4	160	500	25	180
Available Flood Warning (minutes)*	53	15	5	53	60
Number of Properties Impacted	9 receptor locations (RH1 to RH6) and (RH83 to RH86)				
Stage 5A, Dam Break East, Closure, Drawing 5					
Peak Flow (m^3/s)	-	70	250	4	20
Available Flood Warning (minutes)*	-	13	4	60	75
Number of Properties Impacted	3 receptors locations (RH84, RH85 and RH86)				
Stage 5B Dam Break West, Operation Phase, Drawing 6					
Peak Flow (m^3/s)	1,170	21	-	580	360
Available Flood Warning (minutes)*	10	36	-	27	39
Number of Properties Impacted	1 receptor locations (RH18)				
Stage 5B Dam Break West, Closure Phase, Drawing 7					
Peak Flow (m^3/s)	560	16	-	135	84
Available Flood Warning (minutes)*	8	38	-	25	43
Number of Properties Impacted	1 receptor locations (RH18)				
Stage 5B Dam Break South, Operation Phase, Drawing 8					
Peak Flow (m^3/s)	27	1,400	-	50	240
Available Flood Warning (minutes)*	27	8		35	39
Number of Properties Impacted	7 receptor locations (RH1 to RH7)				
Stage 5B Dam Break South, Closure Phase, Drawing 9					
Peak Flow (m^3/s)	13	710	-	7	41
Available Flood Warning (minutes)*	29	6	-	35	45
Number of Properties Impacted	4 (RH3 to RH6)				
Stage 6 Dam Break North-west, Operation Phase, Drawing 10					
Peak Flow (m^3/s)	1,000	50	35	450	300
Available Flood Warning (minutes)*	24	55	37	42	55
Number of Properties Impacted	6 (RH18, RH19, RH23, RH24, RH48 and RH49)				
Stage 6 Dam Break North-west, Closure Phase, Drawing 11					
Peak Flow (m^3/s)	120	20	10	40	40
Available Flood Warning (minutes)*	23	60	39	54	92

Section Line	West	South	East	RB1	RB2
Number of Properties Impacted	4 receptor locations (RH18, RH19, RH48 and RH49)				
Stage 6 Dam Break North-east, Operation Phase, Drawing 12					
Peak Flow (m^3/s)	120	125	320	70	140
Available Flood Warning (minutes)*	36	39	24	75	93
Number of Properties Impacted	9 receptor locations (RH4 to RH6, RH48 to RH50, RH52, RH55 and RH84)				
Stage 6 Dam Break North-east, Closure Phase, Drawing 13					
Peak Flow (m^3/s)	39	53	56	12	25
Available Flood Warning (minutes)*	35	43	23	80	107
Number of Properties Impacted	3 receptor locations (RH4, RH5 and RH55)				

** From beginning of breach.*

The results of the modelling predicts a number of properties are likely to be at risk of inundation, particularly those in proximity to Stage 6, Yellow River and Simonstown River. A flood warning system will be adopted. Residents at high risk and / or incapacitated will have systems installed in their properties (light and alarm). The early warning system will also include text messages to residents, notifications to the emergency services and a localised siren. It is worth noting that a number of receptors are located in very close proximity to the TMF and are inundated in less than 15 minutes, particularly properties north of Stage 6. The presence of a bund along the north boundary of the Site will have a negligible impact to warning times for a dam break of the scale considered but could provide a tangible benefit for smaller releases from the TMF.

A failure along the northern boundary of Stage 6 presents the biggest risk to nearby properties. Stage 6, dam break north-west, operation scenario results indicate 10 properties are at risk and the peak flow entering the River Blackwater is approximately 450 m^3/s. Stage 6, dam break north-east, operation scenario results impacts 11 properties, however the flow entering the River Blackwater is attenuated to 140 m^3/s as tailings is conveyed along the eastern perimeter of the TMF.

During operation, a failure along the western embankment of Stage 5B presents the lowest risk but would still affect a single property and peak flow to the River Blackwater would be 580 m^3/s. At closure the property would still be affected, however peak flow to the River Blackwater decreases to 135 m^3/s.

Although the modelling results for all scenarios predicts limited flooding prior to reaching Navan, it is highly likely that such an increase in flow in the Blackwater would cause significant flooding as the Blackwater enters Navan, especially where hydraulic control structures such as bridges form constrictions.

3.5 Limitations

The analysis models the propagation of liquefied tailings as a Newtonian fluid (i.e. water). The viscosity of water is significantly lower than tailings, therefore the assessment is considered to be conservative for areas downstream. In addition, the model does not account for reduced pore water following capping and closure. Therefore dam break hydrographs and model outputs for the closure scenario are considered to be particularly conservative.

4.0 CONCLUSIONS

Only a piping failure of the TMF has been considered for dam break scenarios during operation and closure; as there is no catchment upstream of the TMF, and during closure the TMF will not hold any supernatant water. The introduction of spillways during closure will reduce the likelihood of a dam break by minimising the volume of water above the tailings. In addition, pore water in the tailings will reduce following capping, further reducing the likelihood of failure.

A failure of the TMF embankment in proximity to nearby properties and during operation represents the greatest risk to the public. During operation, a failure along the western embankment of Stage 5B presents the lowest risk but would still affect a single property and peak flow to the River Blackwater would be 580 m^3/s

In the event of a failure of the TMF, properties in proximity to Simonstown Stream, Yellow River and Stage 6 are particularly at risk of inundation. An early warning system will be implemented for those at risk, however residents will have very limited time in which to react.

Given the high consequence (on local residents) of a dam break failure in the immediate vicinity of the TMF facility at Tara as demonstrated in this study, we would recommended that future consideration is given to undertaking a further assessment based on non-Newtonian tailings dam breach (i.e. tailings viscosity, rather than assuming water). The software required to complete this type of study (Flow3D) is state-of-the-art and Golder is a leading provider in these types of assessments.

5.0 REFERENCES

1) Froehlich, 2008. Embankment Dam Breach Parameters and Their Uncertainties. ASCE, Journal of Hydraulic Engineering, Vol. 134, No. 12, December 2008.

2) Knight Piesold, 2015. Tailings mobilization estimates for dam breach studies. Proceedings Tailings and Mine Waste 2015. D, Fontaine & V, Martin.

3) USACE, 2010. HEC-RAS Hydraulic Reference Manual. Version 4.1.

4) USACE, 2014. Using HEC-RAS for Dam Break Studies. August 2014. TD-39.

Report Signature Page

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08 July 2016

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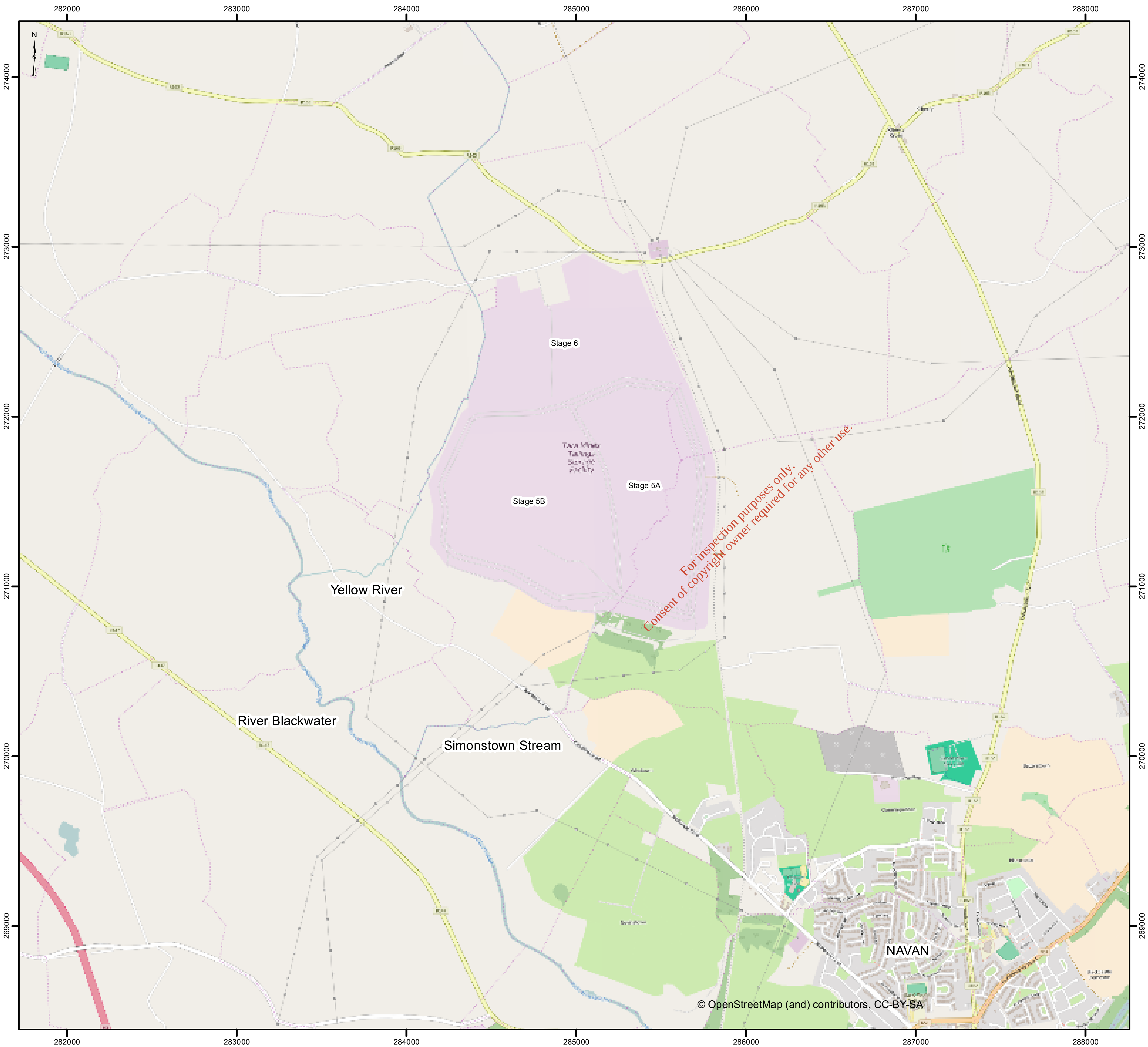
APPENDIX A

Drawings



LEGEND

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500

Meters

1:22,000

CLIENT

BOLIDEN TARA LTD

PROJECT

TARA DAM BREAK STUDY 2016

TITLE

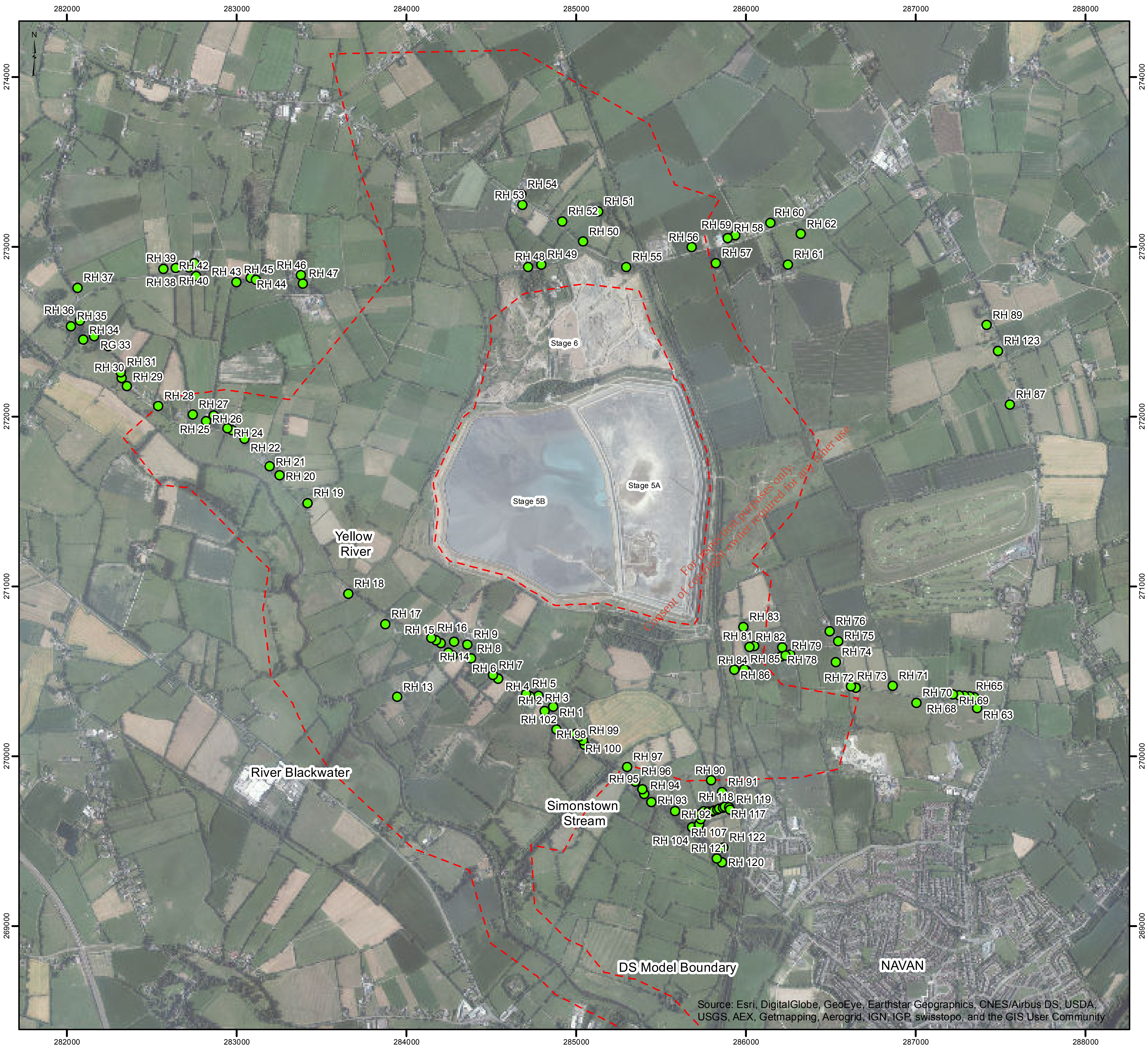
SITE LOCATION PLAN

CONSULTANT

Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0011	B	1



LEGEND

- Receptor
- Model Boundary

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

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Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

PROJECT
TARA DAM BREAK STUDY 2016

TITLE
RECEPTOR LOCATIONS

CONSULTANT

Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0001	A	2

Receptor Reference Number	Dwelling No	Name	Address	Address Line 1	Address Line 2	East	North
RH 1	1476	Paddy & Mary Gore	Randalstown	NAVAN		284866.719	270290.16
RH 2	1477	Joe & Geraldine Fitzpatrick	Randalstown	NAVAN		284815.000	270263.75
RH 3	1441	Siobhan & Lorcan O'Dowd	Randal Lodge, Randalstown	RANDALSTOWN	NAVAN	284780.156	270353.47
RH 6	1479	Richie & Grainne Cahill	Randalstown	DONAGHPATRICK	CO. MEATH	284543.156	270453.75
RH 7	706	Mrs. Smith	Randalstown	DONAGHPATRICK	CO. MEATH	284510.438	270475.91
RH 8	1443	Sarah & Nathan Doyle	Randalstown	RANDALSTOWN	NAVAN	284381.844	270574.97
RH 9	1444	Pat Kennedy	Randalstown	DONAGHPATRICK	CO. MEATH	284360.094	270656.81
RH 10	1442	Aidan & Sarah Kennedy	Randalstown	DONAGHPATRICK	CO. MEATH	284273.875	270592.38
RH 11	1480	Philip & Geraldine O'Dea	Randalstown	DONAGHPATRICK	CO. MEATH	284248.625	270608.94
RH 12	280	Mr. Ennis	Randalstown	RANDALSTOWN	NAVAN	284281.031	270673.22
RH 13	267	Brendan & Susan O' Neill	Mulleard	NAVAN		283947.000	270347.34
RH 14	1445	Vacant - Duignan's	Tatestown	DONAGHPATRICK	CO. MEATH	284204.219	270664.66
RH 15	707	Eimear & Karl Brennan	Tatestown	DONAGHPATRICK	CO. MEATH	284176.594	270680.81
RH 16	910	Dympna & M ichael Dolan	Tatestown	NAVAN		284146.750	270695.13
RH 17	713	Phillip & Mary Brady	Tatestown	NAVAN		283876.781	270775.09
RH 18	1446	Kevin & Geraldine Thompson	Riverview House, Tatestown	TATESTOWN	CO. MEATH	283659.188	270955.53
RH 19	1447	Jack Smith	Tatestown	NAVAN		283419.625	271488.28
RH 20	555	Dick & Eithne Brady	Tatestown Lodge, Tatestown	NAVAN		283252.719	271653.94
RH 21	1530	John & Majella Brady	Tatestown	TATESTOWN	CO. MEATH	283194.031	271707.31
RH 22	786	John & Mary Boyle	Tatestown	NAVAN		283047.188	271867.84
RH 23	1481	Joan & Pat O'Toole	Tatestown	NAVAN		282973.125	271919.78
RH 24	1411	Michael & Patsy Monaghan	Tatestown	NAVAN		282945.313	271929.03
RH 25	1412	Mrs Kelly	Tatestown	NAVAN		282866.063	272004.06
RH 26	1413	Paul & Joanne Fisher	Tatestown	NAVAN		282821.375	271972.88
RH 27	1482	John & Orla Molloy	Tatestown	NAVAN		282741.313	272010.81
RH 28	1414	Raymond Quail	Tatestown	NAVAN		282538.781	272061.31
RH 32	1483	Paul Byrne	Tatestown	NAVAN		282245.063	272414.31
RG 33	714	Aidan & Mary Jordan	Tatestown	NAVAN		282161.438	272471.63
RH 34	1494	Tom & Carol Noonan	Tatestown	DONAGHPATRICK	CO. MEATH	282097.719	272452.91
RH 36	553	Mr. & Mrs. Kevin Martin	Tatestown	DONAGHPATRICK	CO. MEATH	282076.031	272565.69
RH 37	1493	Mr. Eddie Joyce	Donaghpatrick	DONAGHPATRICK	CO. MEATH	282064.219	272758.09
RH 38	1431	Mr. Johnny Joyce	Donaghpatrick	DONAGHPATRICK	CO. MEATH	282568.000	272869.28
RH 39	1455	Thomas & Mandy Maree	Donaghpatrick	DONAGHPATRICK	CO. MEATH	282641.094	272873.59
RH 40	1531	John & Christina Maree	Donaghpatrick	DONAGHPATRICK	CO. MEATH	282719.531	272878.22
RH 41	1429	Mrs. Catherine Lally	Donaghpatrick	DONAGHPATRICK	CO. MEATH	282750.281	272903.41
RH 42	1430	Mr. Tommy Henehan	Donaghpatrick	DONAGHPATRICK	CO. MEATH	282762.781	272827.44
RH 43	1454	Dick Brady	Donaghpatrick	DONAGHPATRICK	CO. MEATH	282998.719	272790.47
RH 44	1428	Sean & Ann Lally	Donaghpatrick	DONAGHPATRICK	CO. MEATH	283082.531	272816.41
RH 45	1427	Thomas & Nora Lally	Donaghpatrick	DONAGHPATRICK	CO. MEATH	283113.125	272804.56
RH 46	1453	Peter Brady	Donaghpatrick	DONAGHPATRICK	CO. MEATH	283379.500	272831.78
RH 47	720	Dick Brady	Donaghpatrick	DONAGHPATRICK	CO. MEATH	283391.063	272781.88
RH 48	1426	John & Olive Kiely	Randalstown	DONAGHPATRICK	CO. MEATH	284719.063	272870.84
RH 50	1423	Renter	Woodview Lodge	RANDALSTOWN	NAVAN	285041.478	273029.91
RH 51	1422	Benny Sheridan	Woodview House	RANDALSTOWN	NAVAN	285131.969	273208.31
RH 52	178	Gabriel Caldwell	Randalstown	DONAGHPATRICK	CO. MEATH	284918.469	273148.72
RH 53	1451	Gerry Cauldwell	Randalstown	DONAGHPATRICK	CO. MEATH	284685.781	273245.19
RH 54	1421	Kevin Ward	Randalstown	DONAGHPATRICK	CO. MEATH	284677.781	273309.16
RH 55	1384	Willie Noonan	Randalstown	NAVAN		285296.469	272881.38
RH 56	1383	Padraic Heaney	Randalstown	NAVAN		285683.406	272996.75
RH 57	1450	Fiacra & Mary O'Cinneide	Randalstown	SILLOGUE	CO. MEATH	285823.750	272902.59
RH 59	1382	Cynthia & Eamonn Dempsey	Randalstown	SILLOGUE	CO. MEATH	285939.531	273068.13
RH 60	1380	Mrs. Rose Heaney	Randalstown	NAVAN		286147.531	273140.06
RH 61	1381	Mrs. Finnegan	Randalstown	NAVAN		286250.656	272895.56
RH 62	1449	Mr. & Mrs. Thomas Finnegan	Randalstown	SILLOGUE	NAVAN	286326.594	273074.94
RH 63	195	David Mullin	Proudstown	NAVAN		287365.719	270281.34
RH 64	190	Paul Mc Donagh	Proudstown	NAVAN		287348.125	270346.13
RH65	191	Brendan O'Reilly	Proudstown	PROUDSTOWN	CO. MEATH	287321.750	270350.91
RH 67	192	Gerry Mc Donagh	Proudstown	NAVAN		287289.781	270354.13
RH 68	193	Mike O'Shea	Proudstown	NAVAN		287257.781	270356.50
RH 69	194	Eric Brady	Proudstown	NAVAN		287223.406	270362.91
RH 70	1002	Paddy Markey	Proudstown	NAVAN		287008.344	270311.75
RH 71	796	Keith & Tracey Brady	Proudstown	NAVAN		286868.344	270412.59
RH 72	967	Michael Mc Ateer	Simonstown	NAVAN		286648.500	270399.94
RH 73	968	Tommy Mc Ateer	Simonstown	NAVAN		286621.688	270409.13
RH 74	282	Pat Conroy	Simonstown	NAVAN		286531.000	270551.84
RH 75	281	Mrs. Callaghan	Simonstown	NAVAN		286546.906	270675.53
RH 76	1003	Michael Clark	Simonstown	NAVAN		286495.031	270733.28
RH 77	1516	Tony & Rosaemary Curran	Simonstown	NAVAN		286265.125	270596.97
RH 80	969	Noel Clark	Simonstown	NAVAN		286216.625	270639.75
RH 81	970	Raymond & Caitriona Brady	Simonstown	NAVAN		286051.000	270646.09
RH 82	196	Derek & Ann Lister	Simonstown	NAVAN		286023.469	270640.28
RH 83	197	Brendan & Claire Brady	Simonstown	NAVAN		285988.188	270758.94

Receptor Reference Number	Dwelling No	Name	Address	Address Line 1	Address Line 2	East	North
RH 86	1005	Gerry Fitzpatrick	Simonstown	NAVAN		285935.000	270508.34
RH 87	1000	Tony Heanay	Proudstown	NAVAN		287558.188	272070.16
RH 89	1399	Jimmy Connolly	Proudstown	NAVAN		287421.000	272538.13
RH 90	787	John & Helen Burns	Windtown	NAVAN		285796.781	269854.78
RH 91	506	Eithne Cantwell	WIndtown	NAVAN		285861.188	269787.13
RH 92	507	John Sherlock	Linton House, Windtown	NAVAN		285585.094	269674.38
RH 93	508	Eamon & Ann Kearney	Windtown	NAVAN		285444.125	269727.44
RH 94	509	Joseph Downey	Windtown	NAVAN		285403.656	269773.69
RH 95	510	Andy & Geraldine O'Connor	Windtown	NAVAN		285391.438	269802.63
RH 96	511	Martina & Gabriel Hamilton	Windtown	NAVAN		285350.313	269848.88
RH 97	512	Mary & Ciaran Mangan	Rathaldron	NAVAN		285301.469	269934.34
RH 98	1440	Helen Duignan	Rathaldron	NAVAN		285051.875	270067.09
RH 99	1478	Bernie Ladd & Brian Collins	Rathaldron	NAVAN		285041.281	270089.69
RH 100	996	Jim Mc Hugh	Rathaldron	NAVAN		285018.688	270112.84
RH 101	997	Meath Disabilities Services	Shalimar House, Rathaldron	RATHALDRON	CO. MEATH	285000.188	270129.47
RH 102	708	Padraig & Louise Fitzsimons	Rathaldron	NAVAN		284887.031	270154.50
RH 103	489	O'Briens	1 Windtown	KINGDOM HALL	NAVAN	285701.813	269569.59
RH 104	490	Kingdom Hall	2 Windtown	NAVAN		285685.906	269578.47
RH 105	491	Mr. & Mrs. Muffit	3 Windtown	NAVAN		285724.188	269586.75
RH 106	492	Fr. Ray Husband	4 Windtown	NAVAN		285728.469	269596.53
RH 107	493	Mrs. Shiela Joyce	5 Windtown	NAVAN		285734.594	269626.53
RH 108	494	Mrs. Johnston	6 Windtown	NAVAN		285730.875	269651.31
RH 109	495	Mr. Wogan	7 Windtown	NAVAN		285737.000	269660.81
RH 110	496	Mrs. Mc Ateer	8 Windtown	NAVAN		285754.156	269674.88
RH 111	497	Mr. & Mrs. Markey	9 Windtown (Rosewood)	NAVAN		285779.281	269671.22
RH 112	498	Mr. Jack Meighan	10 Windtown	NAVAN		285803.000	269672.00
RH 113	499	Ms. Bridget & Kathleen Walsh	11 Windtown	NAVAN		285813.406	269676.28
RH 114	500	Mr. Cassidy (2 brothers)	12 Windtown	NAVAN		285834.250	269683.03
RH 115	501	Mrs. Baker	13 Windtown	NAVAN		285845.250	269687.31
RH 116	502	Mrs. Alexander	14 Windtown	NAVAN		285866.094	269694.03
RH 117	503	Mr. & Agnus Mc Govern	15 Windtown	NAVAN		285878.344	269698.94
RH 118	504	Mrs. Kathleen Markey	16 Windtown	NAVAN		285904.063	269696.50
RH 119	505	Mrs. Brennan	17 Windtown	NAVAN		285907.719	269683.03
RH 120	484	Derelict	Windtown	NAVAN		285860.000	269373.91
RH 121	485	Mr. & Mrs. John Lynch	Windtown	NAVAN		285830.688	269395.41
RH 122	486	Tom Hoskins	Windtown	WINDTOWN	CO. MEATH	285869.781	269458.91
RH 123	187	Mr. Colm Lynch	Windtown	NAVAN		287488.531	272386.59
RH 4	1700	Will Kearney	Randalstown			284728.220	270349.68
RH 5	1701	Richard & Caroline Brennan	Randalstown			284705.730	270366.00
RH 29	1703	Michael & Mary Corrigan	Tatestown			282353.470	272180.00
RH 30	1704	David & Annmarie Cassidy	Tatestown			282322.900	272227.39
RH 31	1705	Stephen Corrigan	Tatestown			282318.000	272259.69
RH 35	1706	George & Julie Mc Dermott	Tatestown			282023.380	272531.77
RH 49	1707	Robbie O'Brien	Randalstown			284797.600	272895.31
RH 58	1708	Brian & Marie Heaney	Randalstown			285895.600	273050.19
RH 78	1004	David Smith	Simonstown			286210.340	270585.25
RH 79	618	John & Ann O'Hare	Simonstown			286236.160	270590.47
RH 84	971 A	Patsy & Rita Brady	Simonstown			286003.160	270501.56
RH 85	971 B	Joe & Collette Mooney	Simonstown			285993.000	270510.85

CLIENT
BOLIDEN TARA LTD

PROJECT
TARA DAM BREAK STUDY 2016

TITLE
SPREADSHEET OF RECEPTOR LOCATIONS

CONSULTANT

Golder Associates

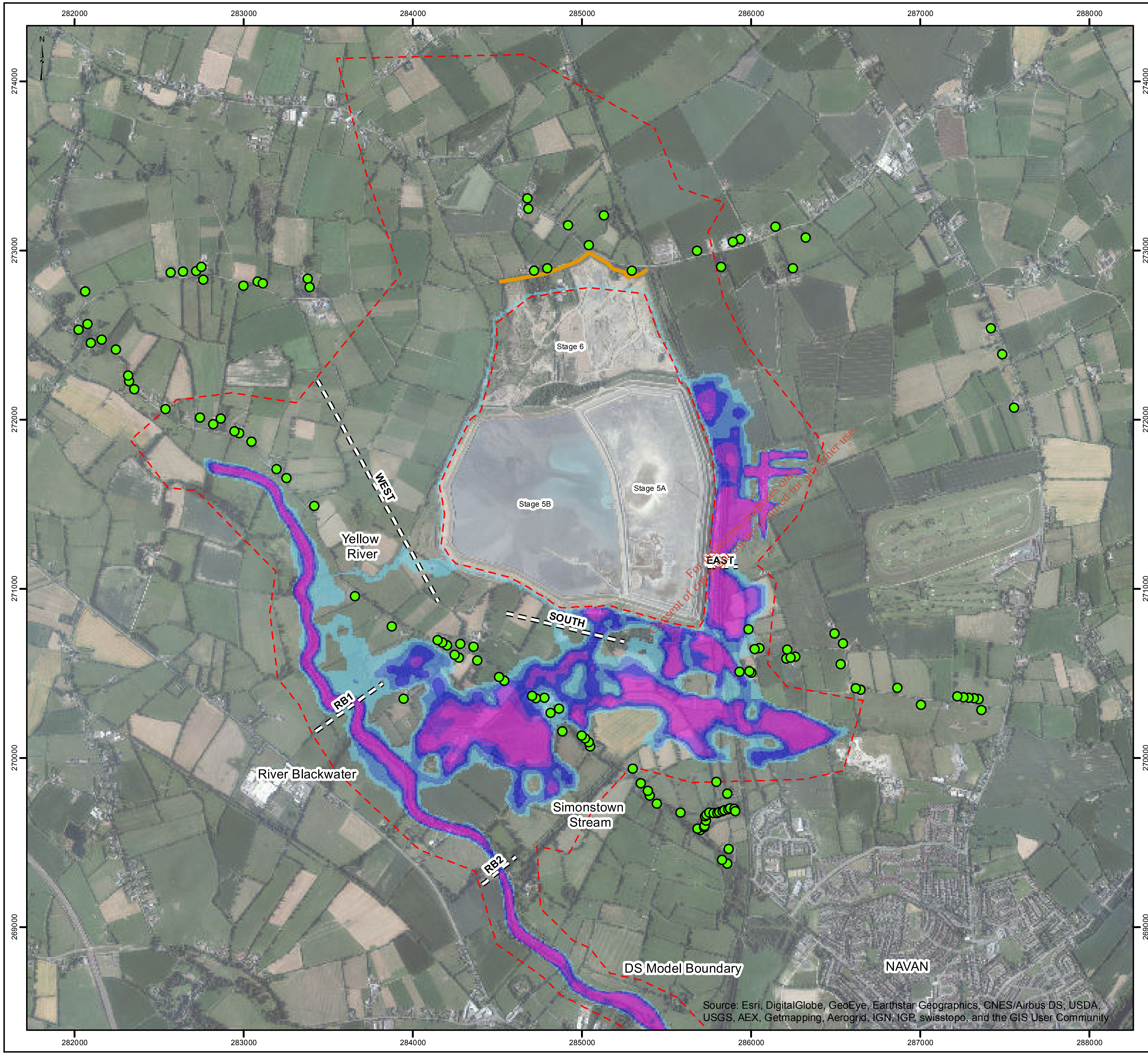
YYYY-MM-DD	06 JUL 2016
DESIGNED	BK
PREPARED	ECS
REVIEWED	BK
APPROVED	MG

PROJECT NO.	CONTROL	REV.	DRAWING
1651706	1001-DB-0013	A	3

Path: \\lond1-s-main01\CADD_GIS_GRAPHICS\Boliden Tara Mines Ltd\Dam_Break\99_PROJECTS\1651706_Stage_6\1001_Dam_Break_Stage_6\02_PRODUCTION\DWG\ | File Name: 1651706-1001-DB-0013.dwg

25 mm

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A3



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500
Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

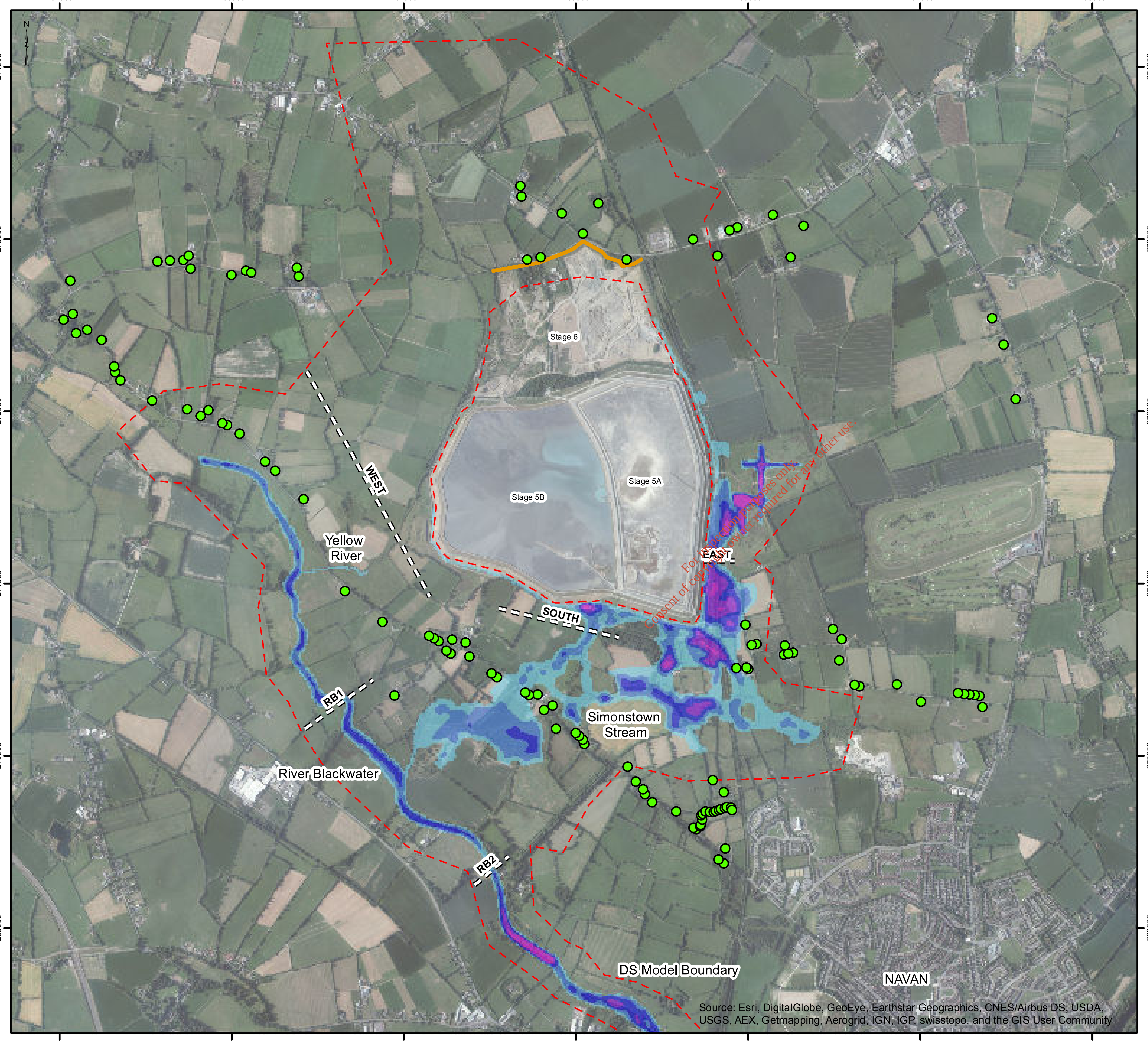
PROJECT
TARA DAM BREAK STUDY 2016

TITLE
STAGE 5A, DAM BREAK EAST, OPERATION PHASE

CONSULTANT
Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0001	B	4



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500

Meters

1:22,000

CLIENT

BOLIDEN TARA LTD

PROJECT

TARA DAM BREAK STUDY 2016

TITLE

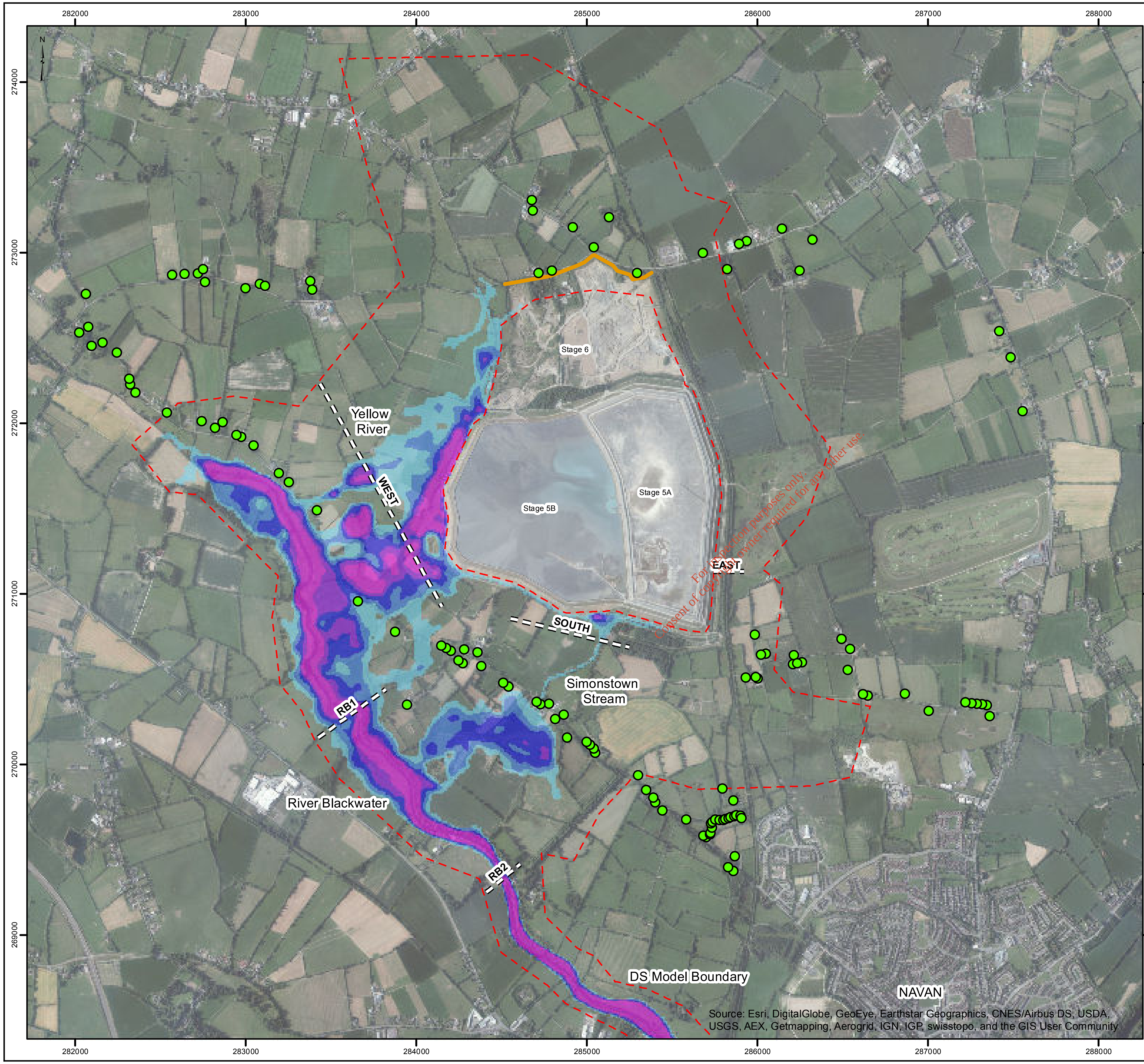
STAGE 5A, DAM BREAK EAST, CLOSURE PHASE

CONSULTANT

Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0002	B	5



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500
Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

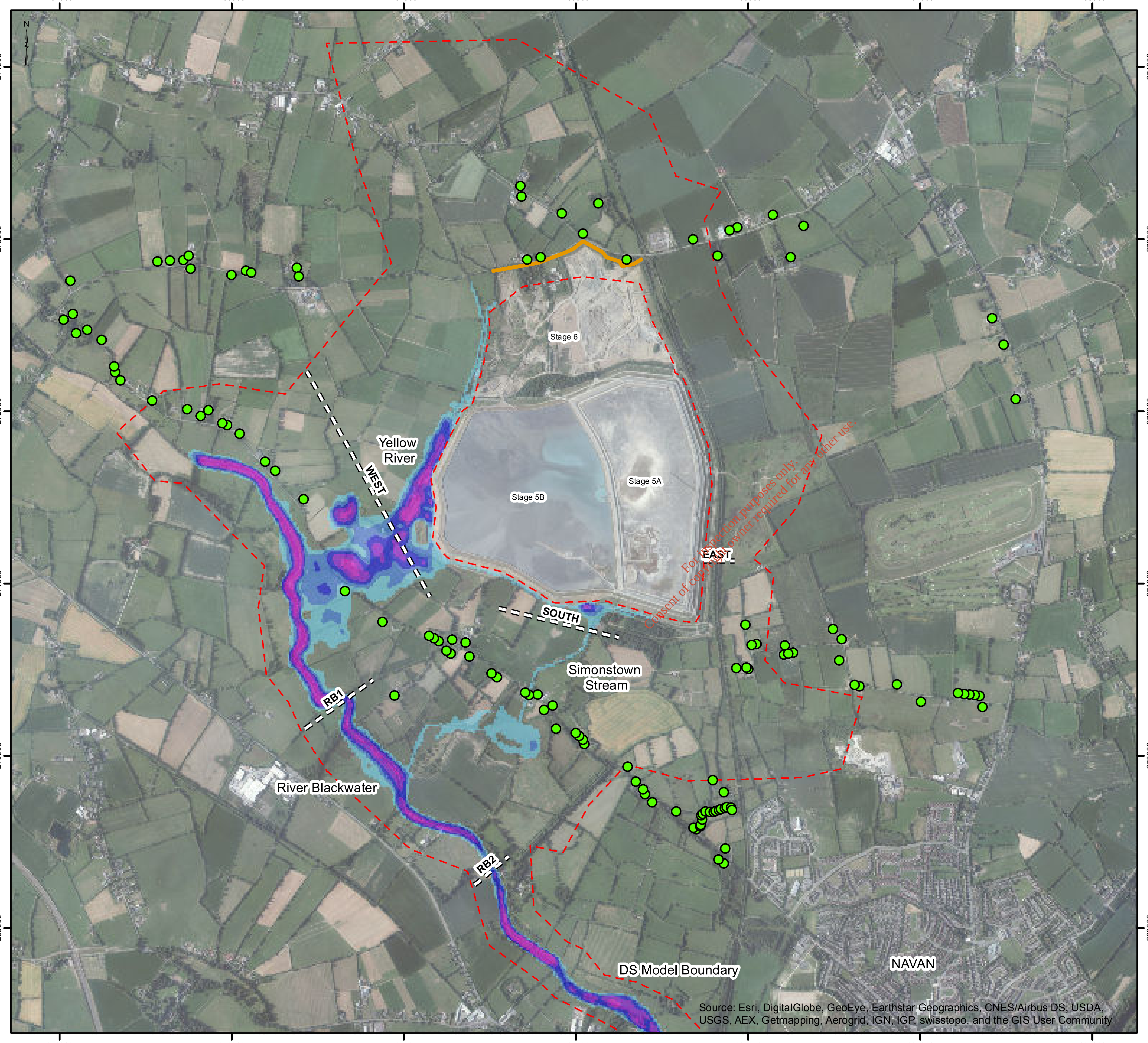
PROJECT
TARA DAM BREAK STUDY 2016

TITLE
STAGE 5B, DAM BREAK WEST, OPERATION PHASE

CONSULTANT
Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0003	B	6



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500
Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

PROJECT
TARA DAM BREAK STUDY 2016

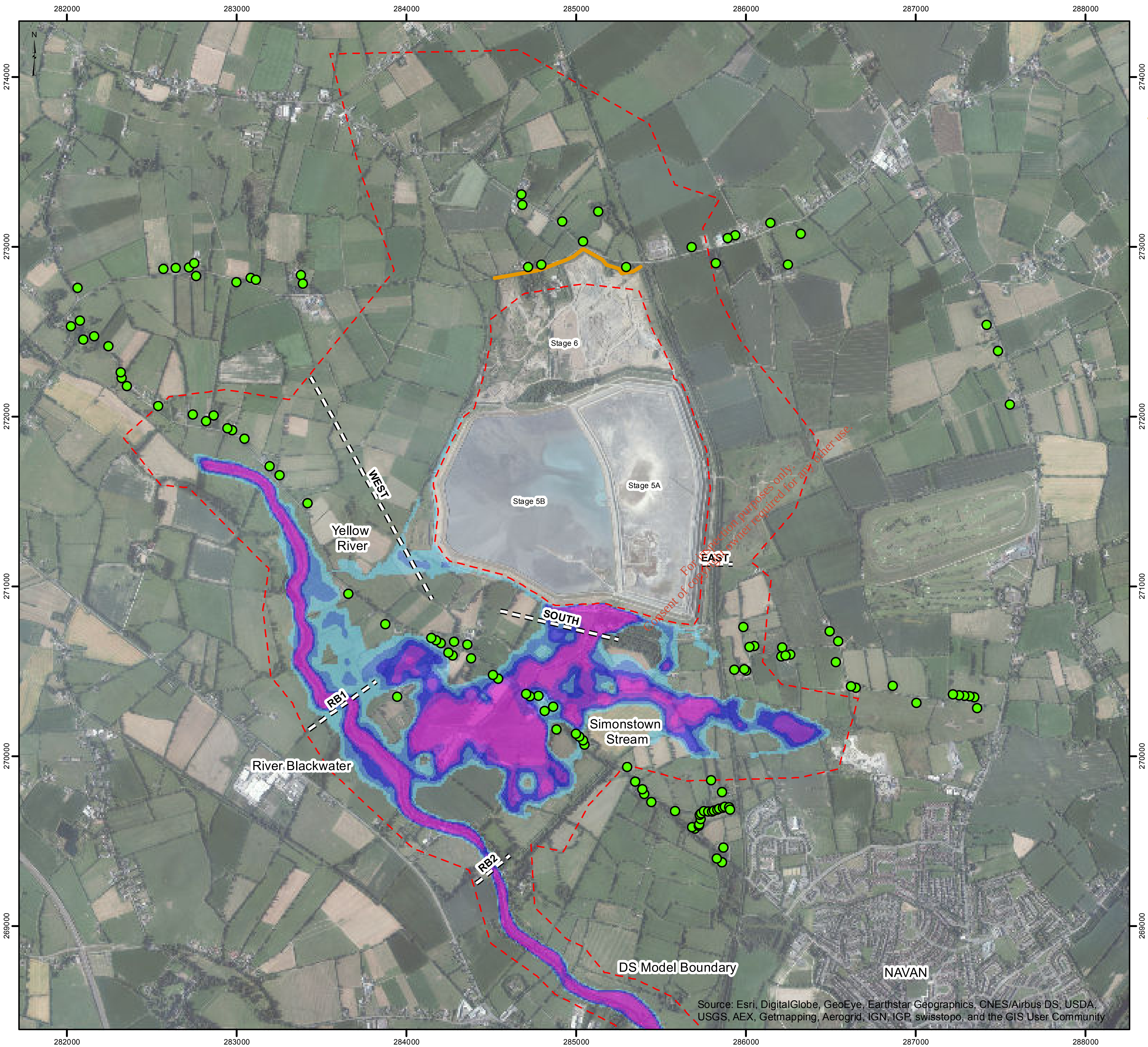
TITLE
STAGE 5B, DAM BREAK WEST, CLOSURE PHASE

CONSULTANT

Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0004	B	7



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500
Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

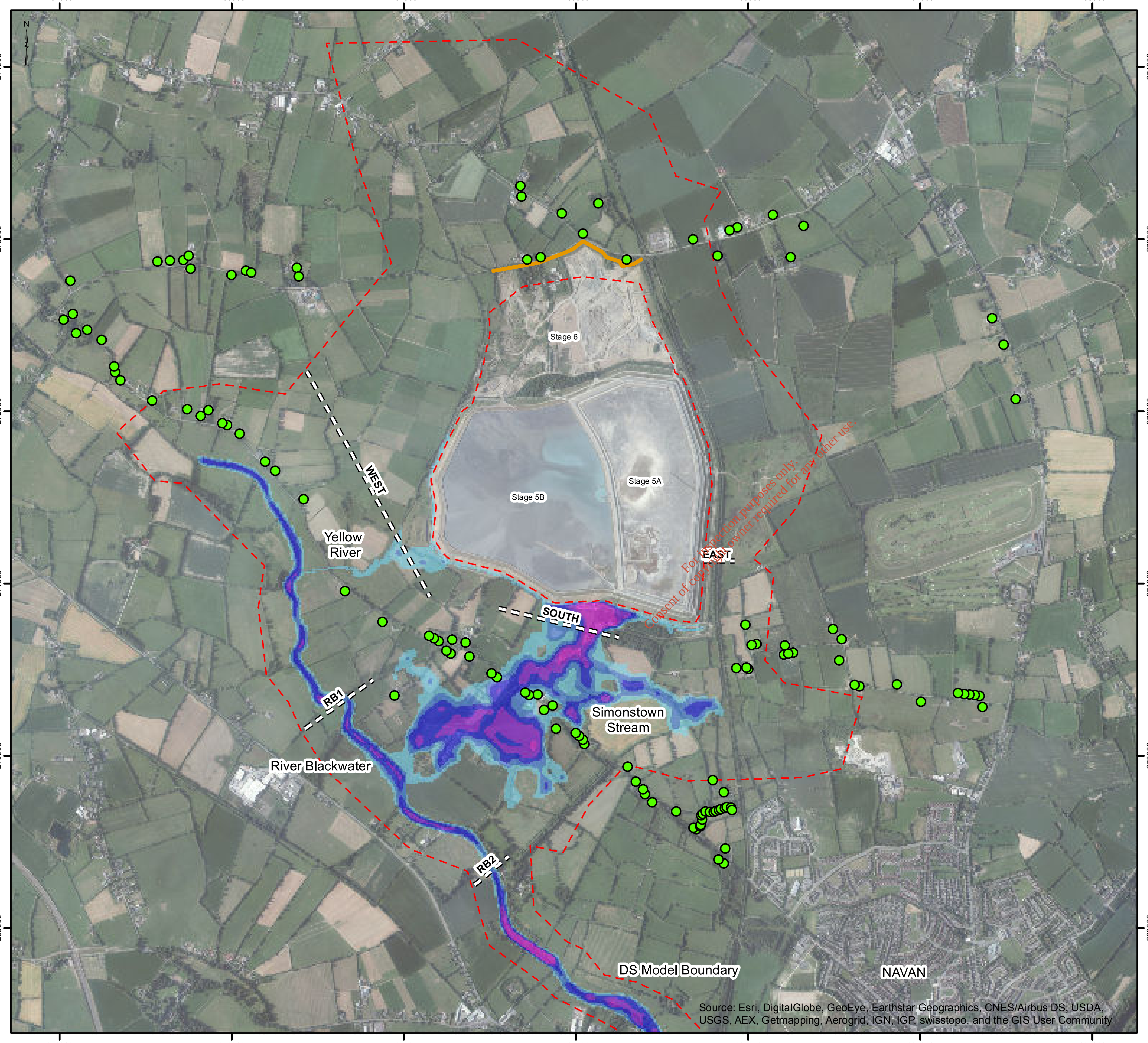
PROJECT
TARA DAM BREAK STUDY 2016

TITLE
STAGE 5B, DAM BREAK SOUTH, OPERATION PHASE

CONSULTANT
Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0005	B	8



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500
Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

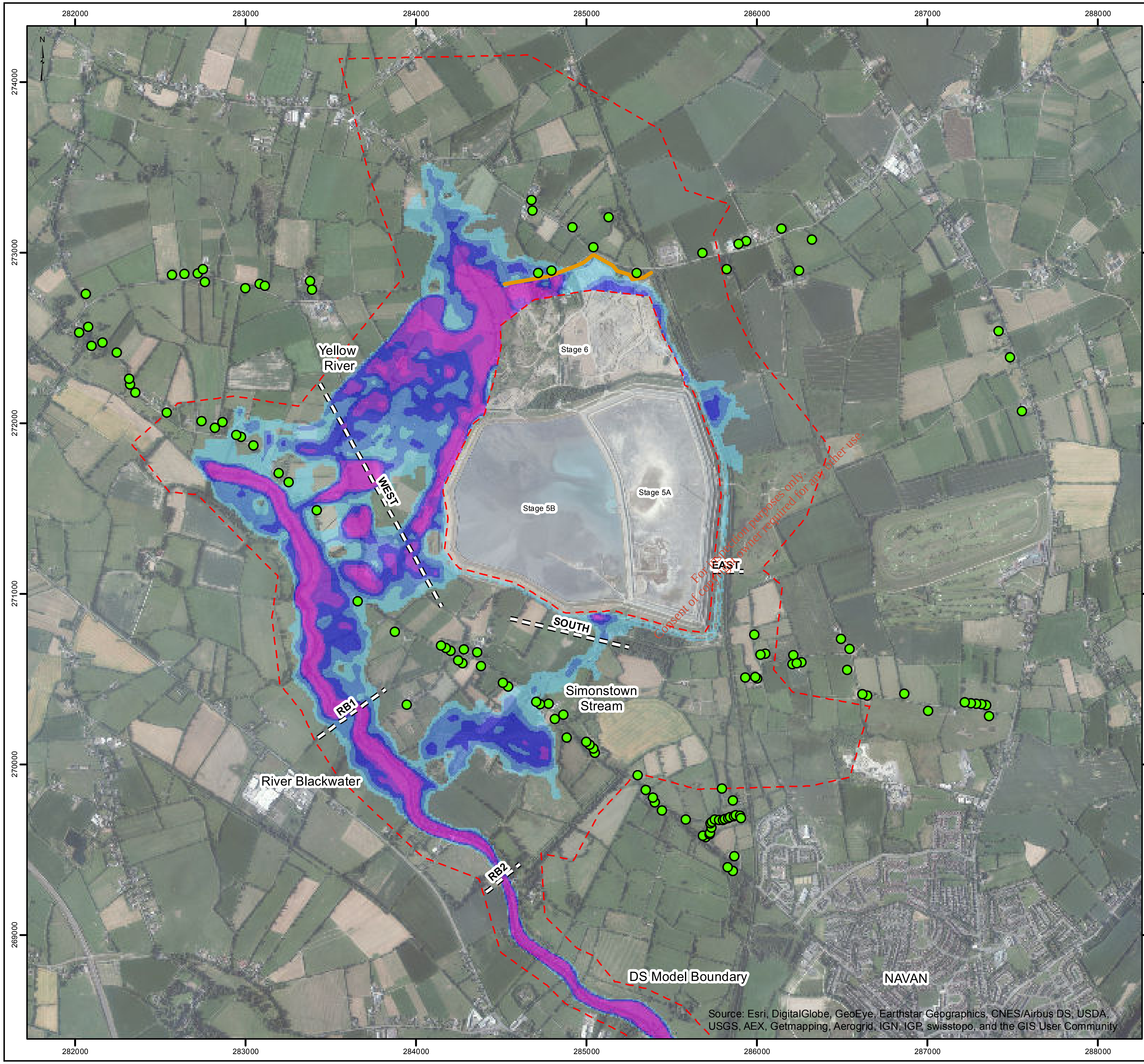
PROJECT
TARA DAM BREAK STUDY 2016

TITLE
STAGE 5B, DAM BREAK SOUTH, CLOSURE PHASE

CONSULTANT
Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0006	B	9



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500
Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

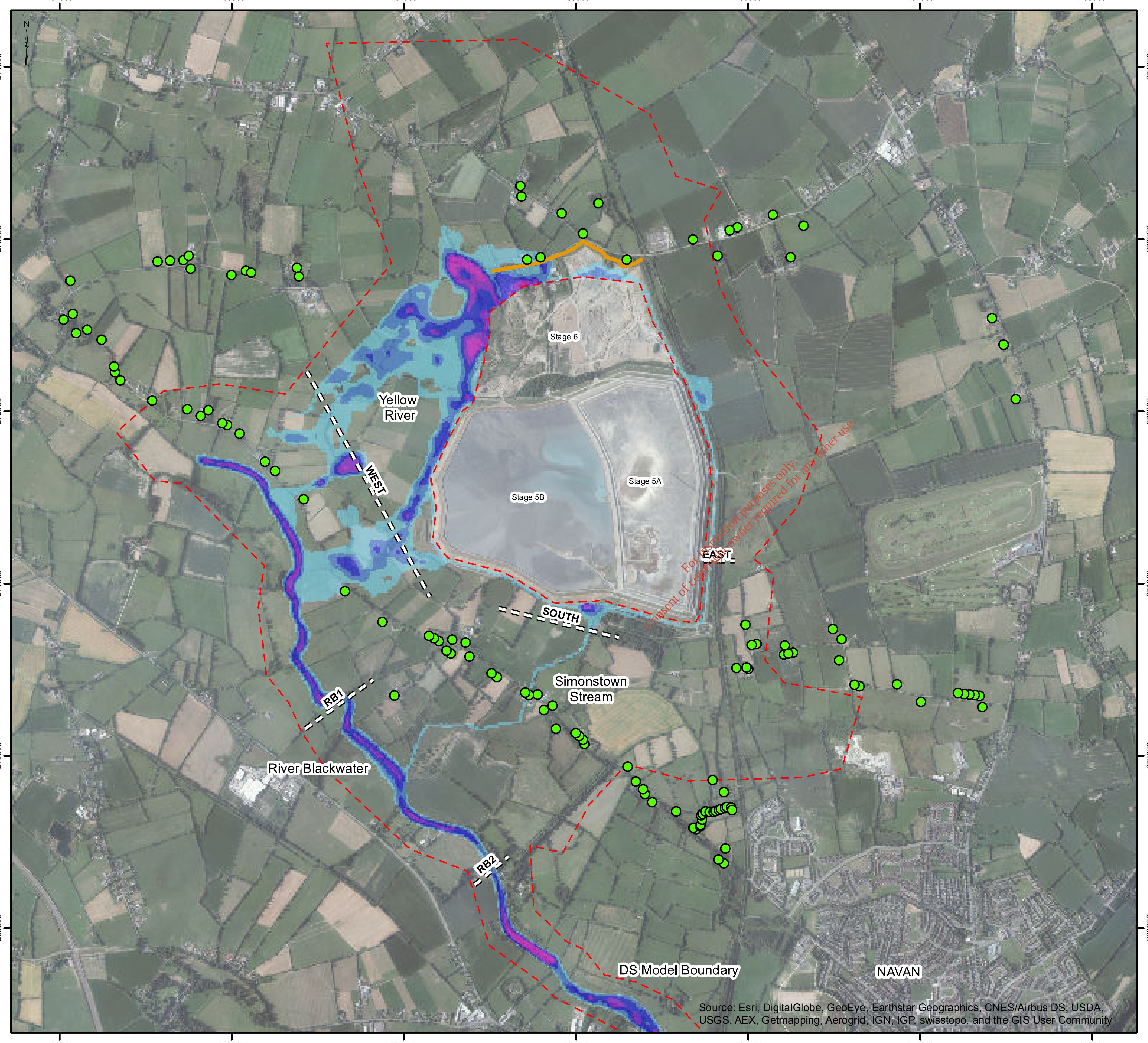
PROJECT
TARA DAM BREAK STUDY 2016

TITLE
STAGE 6, DAM BREAK NORTH-WEST, OPERATION PHASE

CONSULTANT
Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0007	B	10



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500
Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

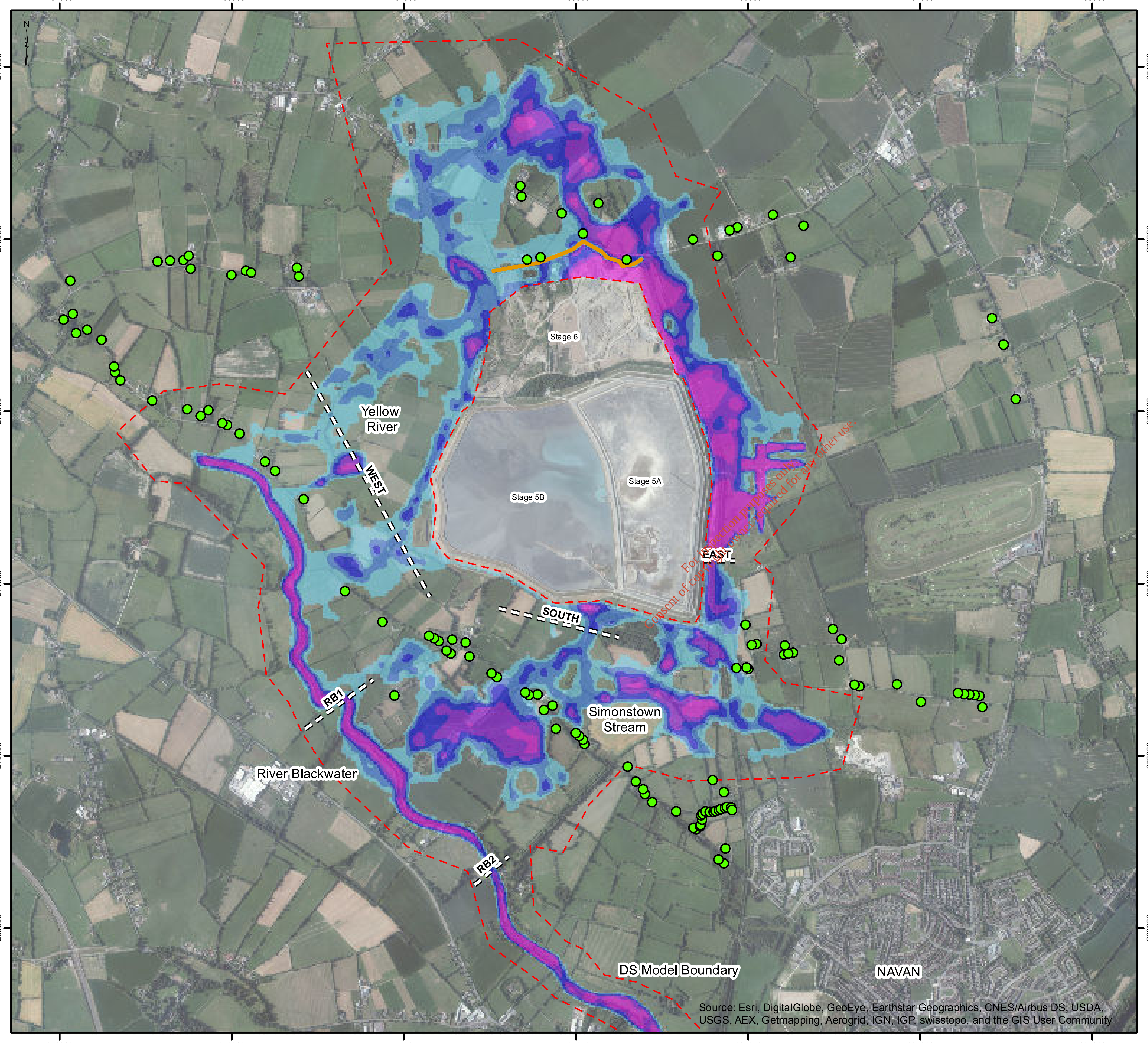
PROJECT
TARA DAM BREAK STUDY 2016

TITLE
STAGE 6, DAM BREAK NORTH-WEST, CLOSURE PHASE

CONSULTANT
Golder Associates

YYYY-MM-DD	08 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0008	B	11



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500

Meters

1:22,000

CLIENT

BOLIDEN TARA LTD

PROJECT

TARA DAM BREAK STUDY 2016

TITLE

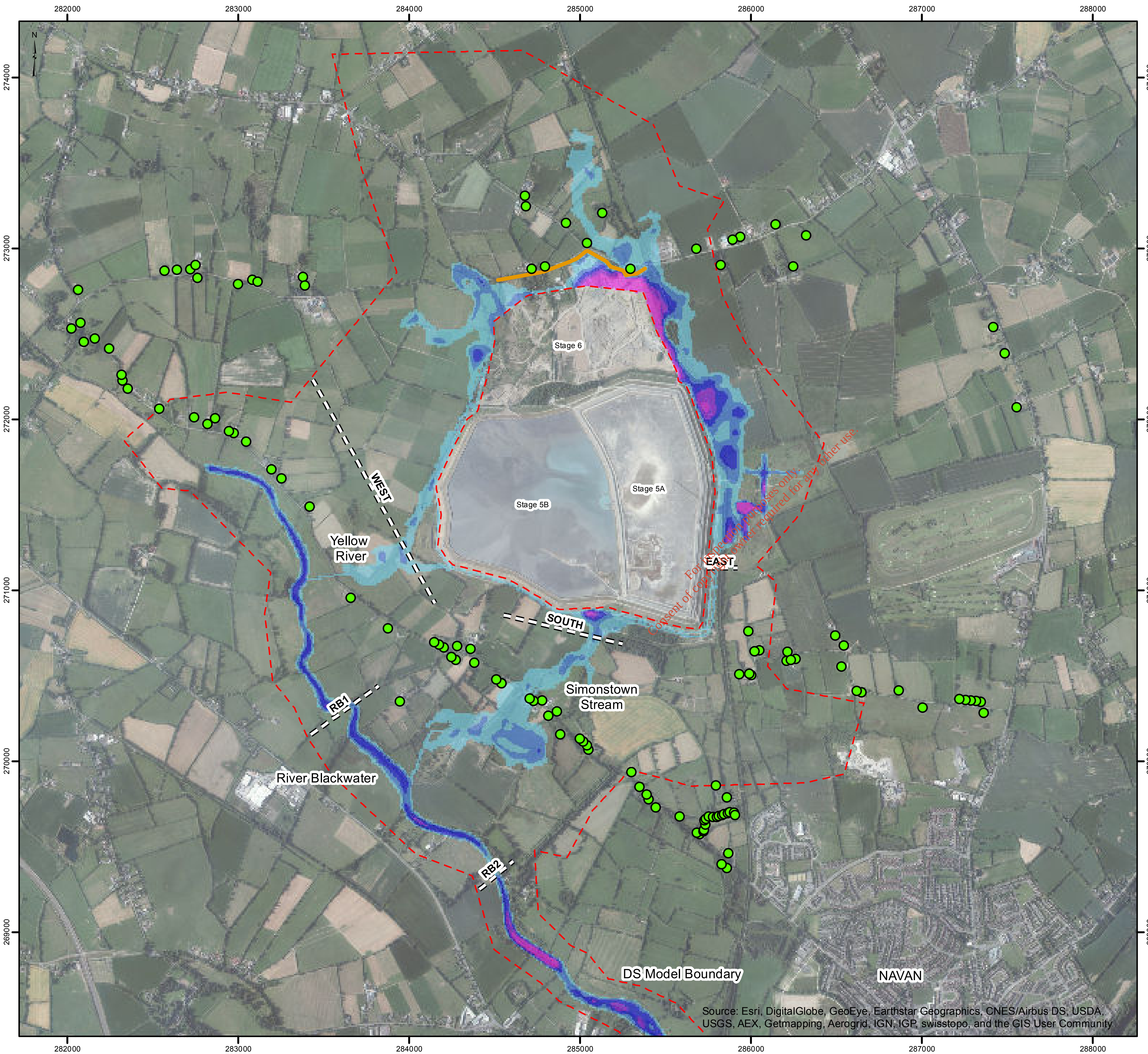
STAGE 6, DAM BREAK NORTH-EAST, OPERATION PHASE

CONSULTANT

Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0009	B	12



LEGEND

- Receptor
- Section line (flow)
- Model Boundary
- North embankment

Flood Depth (m)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0+

NOTES

REFERENCE

COORDINATE SYSTEM: TM65 IRISH GRID

500 250 0 500
Meters

1:22,000

CLIENT
BOLIDEN TARA LTD

PROJECT
TARA DAM BREAK STUDY 2016

TITLE
STAGE 6, DAM BREAK NORTH-EAST, CLOSURE PHASE

CONSULTANT

Golder Associates

YYYY-MM-DD	06 JUL 2016
PREPARED	ECS
DESIGN	RE
REVIEW	RE
APPROVED	MG

PROJECT No.	CONTROL	REV	DRAWING
1651706	1001-DB-0010	B	13

APPENDIX B

Dam Break Video Files (CD)

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