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Ref6786

Oceana Gold (New Zealand) Limited
P O Box 5442
Moray Place
DUNEDIN 9058

15 June 2011

Attention: Marty Hughes

Dear Marty,

RE: MACRAES PHASE 111 PROJECT-SECTION 92 REQUEST

Please find below our response to Questions 9a and 13a and to relevant sections of Questions 17 to 22.

Q9.a) Provide a detailed plan for managing the effects of lake stratification, including options for an intake structure that allows water to be taken from the surface of the lake.

To reduce the risk of discharge of deoxygenated water from the Camp Creek Dam we suggest a floating decant system for the Camp Creek Dam outlet. Figure 11 (attached) of the Camp Creek Dam Technical report has been revised to show the use of such a system. The decant consists of a 250 OD HDPE pipe. The pipe is anchored and weighted so that it has small negative buoyancy. The inlet consists of multiple small diameter holes drilled into the end of the pipe over an approximately 1.5m length. The end of the pipe is suspended from a buoy or small raft and the inlet depth can be easily varied. The buoy is tethered to the shore by ropes. We have used this design on a number of irrigation dams. It is simple and has proven to be easy to operate and maintain. Generally the inlet is set at about 1-2m below pond level. If this system is adopted the valve shown on the upstream end of the pipe beneath the dam could be omitted. However, in this case a plate that can be attached to the upstream end of the outlet pipe to block it off would need to be available to be installed by a diver if required.

Q13.a It has been proposed that a sump be installed in the bed of the truncated Tipperary Creek near the downstream foot of the TTTSF impoundment wall. The intention is that the sump would attract the flow of groundwater from beneath the tailings deposited onto the land surface and capture this groundwater seepage for re-circulation within the mine tailings water system. The following questions arise:

a. What extent and portion of groundwater emanating from beneath the TTTSF would become captured by the Tipperary Sump and why are other sumps proposed in the West Tipperary sub-catchment or Cranky Jims Creek headwaters abutting the TTTSF impoundment wall?

The primary purpose of the TTTSF Sump (seepage collection sump) is to act as a collection point for seepage collected in the underdrains and upstream cutoff drains (both underlying tailings) and the chimney drain in the embankment. It is also proposed that any groundwater seepage beneath the low permeability section of embankment (Zone A1) that discharges into the gully beneath the downstream shoulder of the embankment (i.e. below Zones B and C1) be picked up and discharged into the TTTSF Sump. There is no provision in the TTTSF Sump to collect deeper groundwater seepage that could emerge downstream of the embankment. Monitoring bores are proposed downstream of the embankment and if they detect contamination then another sump could be constructed further downstream to capture and allow water to be pumped back to the TTTSF Sump. Golders predict it will take some years for seepage of any consequence to pass downstream and so any additional downstream sump is unlikely to be required for some time.

The other sumps referred to in the question are small silt ponds for controlling sediment runoff during construction and operation of the TTTSF.

Q17.a) Please confirm the basis for the selected design shear strength parameters for the embankment fill and waste rock materials.

The selected shear strength parameters were based on large diameter triaxial (225mm diameter) tests conducted at Central Labs in the early stages of the project, and also based on review of test results from other projects where schist material had been used. These included tests by Ministry of Works for some irrigation projects in Otago and Kangaroo Creek dam in Australia.

Q17.b) Please provide further description of the engineering properties required for the embankment zone fill materials, and basis for these design criteria.

The design has been based on the assumption that the low permeability zone (Zone A1) of the TTTSF embankment should be less than 10^{-7}m/s. This is based on experience with other embankments at the Macraes Gold Project. The other engineering properties (e.g. strength, stiffness) are based on what the available material can achieve. Strength parameters are summarised in Sections 8.1 and 8.2 of the TTTSF Technical Report (Appendix 20 of the application). A description of the functions of the different embankment zones is given in Section 10.5 of the TTTSF Technical Report. The Construction Specification for the various zones of the TTTSF embankment is likely to be similar to that adopted for existing embankments. A copy of the relevant clauses from the existing Specification for construction of tailings storage embankments at the Macraes Gold Project covering material types (Section A4), embankment zone materials (Sections C1, C3, C6-C10) and earthworks (D1 – D6) is attached.

Q17.c) Please confirm the basis for the fill placement thicknesses of 350 mm (Type A1) and 600mm (Type B), with specific comment on previous construction precedent used for the site.

The current Specification states that the ‘permitted layer thickness depends on the Plant used by the Contractor’ (refer to Specification Clause D6.2 Layer thickness provided in response to Q17.b). The fill placement thicknesses of 350mm for Zone A1 and 600mm for Zone B represent maximum allowable lift heights that are permitted using a 90 tonne self propelled rubber tyred roller, a self propelled foot steel wheeled roller of not less than 30 tonnes static and a vibrating steel roller of not less than 18 tonnes static weight. If other plant is used then trials would be required to justify the layer thickness. In any event acceptance of the fill in Zone A is based on testing to confirm permeability, dry density, water content and particle size. For Zone B the fill must meet a minimum dry density standard. (Refer to Specification Sections C3 and C6 provided in response to Q17.b)

Q17.d) Figures D7 – D12 (appended to the report) show both yield accelerations and design loadings on the same figures. Please clarify the assessed yield accelerations and how these compare with the design earthquake loadings (i.e. OBE & MDE).

Figures D7 - D12 show the yield accelerations for critical circles (i.e. F = 1.0). The yield acceleration is equal to the 'Horiz Seismic Load' shown in the figures. The yield acceleration corresponds to the seismic horizontal load when FOS = 1.0. The actual design loadings and yield accelerations, where the FOS is less than 1.0, are summarised in Table D3 of Appendix 3 of the TTTSF Technical Report (Appendix 20 of the application). We note that Table D3 provided in the report has some errors. A correct version is shown below. There is no change to Figures D1 – D6. However we have made some amendments to Figures D7 – D12, specifically to include the design seismic load and the yield acceleration on the Figures and new versions of these Figures are appended. Figure D9 has no yield acceleration because the slope will not yield under the design seismic load. We also include an additional five figures (Figures D7a, D8a, D10a, D11a, D12a) to show the analysis of the embankment under the design seismic load. The FOS is generally less than 1.0 indicating yield and that permanent deformations can be expected.

Table D3. Simplified Deformation Analysis During Earthquake Shaking using Pseudostatic Stability Analyses

Loading	kh (g)(1)	kc (g)(2)	Predicted Permanent Deformation (3) (cm)		Depth of Failure Surface	Figure No.
			u50	u5		
475 year	0.33	0.31	0.01	0.02	H	D7
475 year	0.36	0.34	0.00	0.01	⅔H	D8
475 year	0.36	-	-	-	⅓H	D9
MDE	0.72	0.31	1.7	14	H	D10
MDE	0.72	0.34	1.2	9.5	⅔H	D11
MDE	0.8	0.47	0.5	4.1	⅓H	D12

Note: Three cases considered for each loading. Failure through top 1/3 and 2/3 of embankment and failure full depth through embankment to determine most critical displacement.

(1). kh (g) = average acceleration within the potential failure mass for various return period earthquakes (determined from QUAKE/W analyses)

(2). kc (g) = yield acceleration within the potential failure mass for a FOS = 1.0, determined using pseudostatic stability analyses.

(3). Simplified permanent deformation determined using Jibson (2007). u50 and u5 are displacements with 50% and 5% probably of exceedance respectively.

Q17.e) Please provide summary of SlopeW software results from the seismic analysis of the dam, including profiles showing accelerations through the embankment (for comparison with yield accelerations).

The summary of the results are shown in Appendix D of the TTTSF Technical Report (Appendix 20 of the application) and in Table D3 (included in the response to Q17.d).

Q18.a) Please confirm how the flood attenuation in McCormicks Creek was estimated (98m³/s presented) and provide details of attenuation method and assumptions.

The 1 in 100 AEP flood flow in McCormicks Creek at the confluence with the Shag River was estimated to be 98m^3/s. This location is of interest because of a road bridge on SH85 and two houses located nearby. The flood attenuation along the Tipperary and McCormicks Creeks was carried out by routing the breach hydrographs established for the Sunny Day and the Flood Induced failures through a HEC-RAS model. The hydrographs are presented in Figure 1.

The HEC-RAS model was developed from the 1.0m contour aerial maps established by Precision Aerial Surveys Ltd. The plan of the flood plain is presented in Figure 2 and the seventeen cross sections used to create the model are shown (Section A to Section Q).

The maximum 1 in 100 AEP flow in McCormicks Creek was estimated to be 130m^3/s at the confluence of McCormicks Creek and the Shag River by scaling flows recorded in the Shag River using the area-flow relationship as in the Regional Flood Estimation Procedure. The 100 AEP flood for the Shag River was estimated from flood measurements as 958m^3/s at "The Grange" by Otago Regional Council (ORC). The catchment of the gauging station was 319km^2. The catchment area of McCormicks Creek at the confluence with the Shag River is 44km^2.

The Manning's coefficient for Tipperary and McCormicks Creek was estimated to be 0.06. The Manning's coefficient for the Shag River was determined by calibrating observed flows at the gauge station and where the stream geometry and historic flood level were known.

HEC – RAS has an option for unsteady flow simulation as one of its analysis options. Thus it is capable of simulating one-dimensional unsteady flow through an open channel using the unsteady flow equation solver. HEC-RAS has an inherent storage routine that results in attenuation of flow through the model.

The HEC-RAS modelling indicates the dam breach flow was attenuated significantly by the time it reached any permanently inhabited buildings or important infrastructure.

Q18. b) Please provide further information to explain why breach failure towards the Frasers Pit was not considered further.

Breach failure towards the Frasers Pit was not considered because this pathway will become blocked by the Frasers East and Frasers North Waste Stacks.

Q18. c) Please confirm the source and accuracy of the downstream contour information presented.

The downstream contour information was provided by Precision Aerial Surveys and the contours were established by photogrammetry. They established good ground control and did tests comparing levels from the photogrammetry with corrected GPS levels. The error was up to 0.2m but generally on average was about 0.1m.

Q20.) The interaction of pit extensions and tailings storages is complex and to an extent uncertain, as highlighted by pit review reports and in Appendix 21. To assist in understanding this interaction, please provide a summary and discussion of the key risk factors (to both mining and tailings facilities), uncertainties, consequences and probable management/mitigation, including implications for post mine closure. This could take the form of a large summary table with some supporting discussion, and could be assembled by the applicant based on consultant's reports/advice and in-house knowledge.

A summary of key risks and possible management/mitigation measures for various facilities are provided in the attached Table of Risks. In broad terms the sequence pertaining to the mining of Round Hill and Southern Pit (RH-SP) is as follows.

- Decommission the MTI TSF.
- Decommission and excavate SP11A TSF and some of SP10 TSF. The excavated tailings are to be placed in the RTS, which lies on the central and eastern portion of the MTI TSF
- Excavate RH-SP. The west wall of the pit will be extensively monitored. This data will be used to control the rate and extent of mining.

  The main interaction of the RH-SP is with the MTI TSF as the Footwall Fault (FF) passes underneath part of the embankment. Because the magnitude of the movement along the FF increases as the pit is deepened, the effects of the interaction increase as the pit is deepened.

  The interaction between the RH-SP and the re-profiled SP10 TSF is relatively minor and occurs early on during mining.

Q21.) The following questions and further information requests have arisen from a preliminary review of Appendix 23, due to the appendix's lack of detailed information. In particular:
Q21. a) Please provide specific comment on what effect the in-situ rock on the downstream shoulder will have on the seismic performance and structural integrity of the embankment.

Foliation is adversely dipping in the upstream direction and this has been considered. Potential failure under drawdown conditions has been considered (refer to Figure A12 in Camp Creek Technical Report (Appendix 23 of the application)). At the final design stage consideration will also need to be given to the earthquake loading case.

Q21. b) Please provide comment on the effects of wind induced wavelap on the proposed design, and confirm specific design criteria to account for wind induced waves.

Design for wind effects will be based on USBR criteria. We propose to adopt the 1 in 10 AEP wind on top of the maximum design flood and the 1 in 100 AEP wind on top of normal top water level.

Q21. c) Provide details on the effects of the proposed reservoir on slope stability in the reservoir inundation area, and the assessed consequences of slope failure into the reservoir on the dam structure.

We have undertaken an assessment of the potential for reservoir induced land instability. The field mapping and review of aerial photographs indicated the presence of six landslides around the reservoir. Most are shown in Figure 6 of the Camp Creek Technical Report. They are also shown in the attached Figure 5 and are labelled as landslides L1 to L6. Inferred cross sections through the landslides are shown in Figures 6 and 7. All the landslides are located either completely or partially below the top normal reservoir water level. In all cases the toes of the landslides are at the bottom of the reservoir. A summary of the landslides is given in Table 1 (below) together with an assessment of the volume of each landslide. The main concern is with slopes on the southern side of the reservoir which are flatter than on the northern side due to foliation dipping to the north. The most significant landslides (L1 and L2) are about 130m and 170m respectively upstream of the dam on the south side of the reservoir. L3, the third largest potential landslide is located over 500m upstream of the dam. Landslides L4, L5 and L6 are much smaller (less than 750m^3) but are located immediately upstream of the dam on the north side of the reservoir.

The landslides on the north side of the reservoir, immediately upstream of the dam, are small and below normal top water level and would not be expected to have a significant adverse effect. The potential for landslides on the south side to impact on the reservoir are greater. An assessment of the potential height of a wave generated by landslide L1 had been made. If it is assumed to be fully submerged a wave height of 1.8m is predicted using the Grilli and Watts (2000) method. If it is assumed to be sub-aerial a wave height of 2.2m is predicted using Pugh and Chiang (1986). The toe of the landslide is at the bottom of the reservoir and so any landslide is likely to be low velocity, and therefore the wave heights predicted by the methods are likely to be over estimated. The freeboard at normal top water level is 2m. Consequently the risk of significant overtopping is small. However, it will be considered in more detail at final design. We note that if stability analyses indicate an unacceptable factor of safety it would be relatively easy to provide a toe buttress to landslides L1 and L2. Machine drilled boreholes are proposed at the dam site. Consideration will be given to drilling to confirm the depth of landslide L1, so that the effects of future movement of this landslide on the proposed reservoir and dam can be evaluated.

Table 1. Summary of Landslides within Camp Creek Reservoir

Landslide (refer Figure 5)	Estimated Volume (m^3)
L1	7,900
L2	5,800
L3	3,800
L4	740
L5	330
L6	290

Q21. d) Please confirm whether pressurised conduits will be used in the design.

The proposed outlet pipe will be pressurised. It is likely to be a 250 OD PE pipe which will be encapsulated in reinforced concrete over the upstream section as shown in Figures 11 and 12 of the Camp Creek Technical Report.

Q21.e) Please confirm the service spillway arrangements including routing the pipe through the abutment/embankment and present these on a drawing.

A long-section through the service spillway is shown in the attached Figure 4. It will be encapsulated in reinforced concrete to the downstream side of Zone A1. A filter/seepage interceptor drain will be located at the downstream end of the concrete encased section with granular drainage material around the pipe where it is located beneath Zone B.

Q21.f) Please confirm the basis for selecting a 10 year ARI design flood for construction diversion works capacity. Provide further details of the effects of flood flows exceeding the design criteria of 10 year ARI during construction of the dam.

A 1 in 10 year ARI design flood has been selected for construction diversion works taking into consideration:

- NZSOLD Guidelines,

- The duration of construction when the dam could be vulnerable to overtopping (construction period likely to be about 9 months),
- There is a low risk to life if the design flood were to be exceeded as there are no permanently inhabited buildings in the potential flood plain until Waynestown which is located approximately 37km downstream.

Q21.g) Please provide further details on construction diversion works including the location of the diversion culvert and whether this will remain within the embankment.

The diversion works are likely to consist of a reinforced concrete pipe. It will be located on one side of the existing creek. It will be grouted up when the dam is commissioned.

Q21.h) Please confirm the basis for the fill placement thicknesses of 350 mm (Type A1) and 600mm (Type B).

Please refer to response to question 17.c.

Q22. a) Please confirm how the flood attenuation in Deepdell Creek was estimated and provide details of attenuation method and assumptions.

The flood attenuation along Camp Creek, Deepdell Creek and Shag River was carried out by routing the breach hydrographs established for the Sunny Day and Flood Induced failures through a HEC-RAS Model. The hydrographs are presented in Figure 3.
The HEC-RAS model was developed from 1.0m contour aerial maps established by Precision Aerial Surveys Ltd. The plan of the flood plain is presented in Figure 2 as well as the 118 cross sections that were used to create the model.

Under the flood induced failure the initial flow at each section on the downstream of the breach is assumed to be equal to the 1 in 100 AEP flow at the section. The flood induced breach hydrograph was then routed through the model.

The Manning's coefficient was estimated to be 0.06 based on the calibration undertaken. Sensitivity analyses indicate that the flow depth was not sensitive to the value of Manning's coefficient.

HEC – RAS has an option for unsteady flow simulation as one of its analysis options. Thus it is capable of simulating one-dimensional unsteady flow through an open channel using the unsteady flow equation solver. HEC-RAS has an inherent storage routine that results in attenuation of flow through the model.

The HEC-RAS modelling indicates the dam breach flow was attenuated significantly by the time it reached any permanently inhabited buildings or important infrastructure.

Q22.b) Please confirm the source and accuracy of the downstream contour information presented.

The downstream contour information was provided by Precision Aerial Surveys and the contours were established by photogrammetry. They established good ground control and did tests comparing levels from the photogrammetry with corrected GPS levels. The error was up to 0.2m but generally on average was about 0.1m.

Q22.c) Please provide further comment on the expected itinerant population and how this has been accounted for in the PAR assessment, with specific comment on traffic volumes on Golden Point Road at the crossing over Deepdell Creek, including predicted future traffic volumes.

Golden Point Road, at the crossing over Deepdell Creek, is closed to the public and it is intended to be permanently closed. There are a number of people who visit the Historic Mining Reserve as members of bus tour party groups. There are approximately 6 tours per week in summer and 3 tours per week in winter. Numbers vary between 2 to 54 people with an average of about 18. They are at the site for approximately 20 minutes. If conditions are inclement tour clients are offered the opportunity to visit the reserve but are discouraged from exiting the bus. In addition to tour parties there are occasional independent travellers and DOC staff that visit, but such visits are low in number. We believe the risk to itinerants is not high because although the time for the flood wave to arrive from a breach is 15 minutes, the time for peak water depth to occur is 45 minutes (refer to Table 5 of Camp Creek Dam Breach Report). A steadily rising water level in Deepdell Creek would be a warning for people to move to higher ground. It is relatively open ground in this area and the ground rises quickly so it would be easy to reach high ground. We note that there is no certainty that tour groups will continue after the mine closes. The risks to itinerants at the Historic Mining Reserve could be managed by appropriate warning signs and briefing tour operators of the risks and what action to take in the unlikely event of a breach. We acknowledge there is some uncertainty in the assessment of risk but note that even though we assess the dam as low PIC we recommend adoption of design standards (e.g. earthquake design loads and design floods) that are appropriate for a medium PIC dam.

Q22.d) Please confirm the expected damage to Golden Point Historic Mining Reserve or other historic structures downstream due to dam failure, and comment on the potential for people to be within these areas during a dam failure event.

The main elements of the Golden Point Historic Mining Reserve are the old Battery which is located on the northern bank, immediately adjacent to Deepdell Creek, the old Mine Managers house located on the south bank of Deepdell Creek approximately 4m above Deepdell Creek and three mud-brick buildings located on the south side of Deepdell Creek. One of the mud-brick buildings is at the same level as the Old Managers house. The other two are located about 11m above Deepdell Creek. In the event of a hypothetical breach of Camp Creek Dam the predicted water level in Deepdell Creek will rise to completely inundate the Battery, one of the mud-brick buildings and the old Mine Managers house. The depth of inundation would likely result in substantial damage to these structures. Two of the mud-brick buildings would be above the predicted dam breach flood level. There is low potential for people to be present at the Golden Point Historic Mining Reserve. This is commented on in some detail in the response to Q22c. As noted in Q22c, we believe the risk to people who happen to be in the Historic Mining Reserve at the time of a breach is low because although the time for the flood wave to arrive from a breach is 15 minutes, the time for peak water depth to occur is 45 minutes. A steadily rising water level in Deepdell Creek would be a warning for people to move to higher ground.

Q22.e) Please provide comment on the likelihood of loss of life and provide supporting information to confirm whether the assessed loss of life is less than 1 person.

The predicted breach flood water level for a hypothetical sunny day failure is below the floor levels of all permanently inhabited houses downstream of the proposed Camp Creek Dam. In the event of a 1 in 100 AEP flood induced breach the water level would exceed the floor level in 15 houses. However, the incremental flood water depth (i.e. the increased water level above the 1 in 100AEP flood event on its own) is small and is assessed to have minimal risk consequences. This is discussed in more detail in section 8.3 of the Camp Creek Dam Breach Report. The report concludes that "a breach of Camp Creek Dam does not result in an increased number of houses with significant danger to life".

The risk to life of itinerants is also assessed to be low. The area of highest risk is probably associated with the Golden Point Historic Mining Reserve. We believe the likelihood of loss of life at this location is very low because:

i) There are no permanent inhabitants

ii) There are only occasional visitors to the site. Currently regular bus tour groups visit the site 6 times per week in summer and 3 times in winter for approximately 20 minutes. There would also be occasional independent travellers. DOC staff also visit the site to undertake maintenance.

iii) As noted in the responses to questions Q22c and Q22d the breach flood wave associated with Camp Creek is predicted to arrive 15 minutes after commencement of a breach, but the time for peak flood water depth to occur is 45 minutes. A steadily rising water level in Deepdell Creek would be a warning for people to move to higher ground. The ground rises quickly in the area and is relatively open so it is easy for people to move to safe ground.

The risk to life of recreational users of Camp Creek, Deepdell Creek or Shag River (e.g. swimmers or fishermen) is assessed to be very low. The Camp and Deepdell Creeks have no to minimal recreational use as many sections of the water courses are only accessible from private land. The time for arrival of the breach flood wave to arrive in the Shag River is approximately 60 minutes (refer to Table 5 of the Camp Creek Dam Breach Report). The time for the peak water depth to occur is approximately 40 minutes later. A steadily rising water level in the Shag River associated with a breach of Camp Creek Dam Creek would be a warning for people to move to higher ground.

Some minor bridges are predicted to be flooded in the event of a breach of Camp Creek. All concerned are either regularly or occasionally flooded by flood events and so local residents are used to this hazard. If water was above bridge deck level the expectation is that people would not attempt to drive across.

Yours faithfully
ENGINEERING GEOLOGY LTD

T. Matuschka (CPEng)

Encl: TABLE OF RISKS
EXTRACT FROM EXISTING SPECIFICATION
Figures 1 – 7
Figure 11 (Camp Creek Dam Technical Report, revised version of Figure 11)
Figures D7 – D12a

TABLE OF RISKS

Facility	Risk	Comments	Management/Mitigation
Deconstruct SP11A TSF	Difficult for large plant to traffic over tailings	Greatest risk when excavation gets deeper and close to or below saturated tailings.	• Use vehicles with low track/tyre pressure. • Access heavy vehicles around the perimeter of TSF on the embankment and natural ground. • Form dedicated haul roads using waste rock, possibly together with geotextile reinforcing. • Implement drainage measures to accelerate drainage of the tailings (e.g. drainage trenches with sump pumping). • Control stormwater runoff. • Monitor depth of phreatic surface.
	Instability of excavated tailings slopes	Greatest risk at depth where the excavation is below saturated tailings/phreatic surface.	• Limit heights of cuts and adopt gentle cut slopes. • Monitor slopes as excavation progresses to determine practical safe cut depths and slopes. • Implement drainage measures to control seepage out of cut slopes. • Control surface water runoff (minimize ponding of water on tailings). • Monitor depth of phreatic surface.
Mining RH-SP	Increased deformation on FF	Greater deformation than expected may occur. Mining to be carried out on a stop/start basis controlled by rate of observed deformation.	• Further machine boreholes and groundwater monitoring to be carried out for detailed design. • Rigorous stability analyses to be carried out. • Deformation and groundwater monitoring to be implemented. • Revise design of RH-SP, if required. • Increase dewatering measures. • Backfill areas of RH-SP
	Increased deformation on FF by Process Plant	Mining to be limited on the northern side to minimize	• Deformation monitoring to be implemented. • Revise design of RH-SP, if required.

Facility	Risk	Comments	Management/Mitigation
		deformation in the Process Plant area.	
	Uncontrolled deformation on FF	Creep movement may not reduce, or increase, during stop/start mining. In the extreme uncontrolled failure may occur.	• Deformation and piezometer monitoring to be implemented. • The extent of deformation will increase as the depth of excavation increases allowing ongoing back analysis to predict future deformation. Mining will progress sufficiently slowly such that greater than expected deformation during shallower depth mining should provide early warning of future potential problems. • Revise design of RH-SP, if required. • Emergency backfill of RH-SP.
Re-profile SP10 TSF	Difficult for large plant to traffic over tailings	Greatest risk at depth close to or below saturated tailings.	• Use vehicles with low track/tyre pressure. • Access heavy vehicles around the perimeter of TSF on the embankment and natural ground. • Form dedicated haul roads using waste rock, possibly with geotextile reinforcing. • Implement drainage measures to drain tailings. • Control stormwater runoff. • Monitor phreatic surface.
	Instability of the final tailings cut slope.	Greatest risk during/immediately after earthquake loading (i.e. liquefaction of saturated tailings). The risk will reduce with time as the tailings drain.	• Confirm and monitor depth of phreatic surface. • Rigorous stability analyses to be carried out. • Monitor deformation and piezometric levels. • Rehabilitate surface and control surface water runoff (i.e. minimise ponding and infiltration). • Trim to flatter slope if required.
	Instability of Embankment due to mining RH-SP	Excavation of RH-SP close to toe of the embankment could affect stability, mainly due to	• Drill machine boreholes downstream of the embankment to check rock structure prior o mining. • Rigorous stability analyses to be carried out.

Facility	Risk	Comments	Management/Mitigation
		unfavourable dipping discontinuities in the insitu rock.	• Monitor deformation. • Mine RH-SP in shallow benches towards SP10 Embankment. • Geologically map face of RH-SP as mining extends towards SP10 embankment. • Revise RH-SP profile if unfavourable dipping discontinuities could affect stability and/or significant deformation indicates potential stability concerns. • Backfill portion of RH-SP to improve stability, if necessary.
MTI TSF	Movement of FF affecting performance of MTI TSF	Previous mining has caused up to 5m of movement on the FF beneath the MTI Embankment and this has not significantly affected the performance of the embankment and TSF. The embankment includes mitigation measures in the design.	• Decant pond maintained clear of area where movement occurs on the FF. When RH-SP is mined there will be no pond as the TSF will be closed. • Tailings are generally cohesionless with high friction angle and monitoring shows that they are reasonably well drained. • The embankment includes a wider chimney drain over most of the area affected by movement on the FF. • The embankment includes a significant downstream rockfill shoulder which is not susceptible to erosion. • The TSF includes an extensive system of subsoil drains. • Pumpwells are located beneath the downstream shoulder to lower the water pressure on the FF. These will be maintained and increased during mining of RH-SP. • Extensive deformation, piezometer and seepage monitoring is carried out. Historical monitoring data provides good information for prediction of future performance.

Facility	Risk	Comments	Management/Mitigation
			• OceanGold has considerable experience operating a stop/start process using deformation monitoring data for mining of historical and existing pits. • State of the art dynamic deformation analyses are proposed to assess the performance of the MTI TSF under the anticipated deformation on the FF.
	Significant deformation on FF.	Deformation on the FF due to mining RH-SP could be 10m or more. The mode of deformation causes mainly shear across the embankment with some extension. Shearing could increase seepage loss from the tailings to the FF aggravating the situation.	• Where the outcrop of the FF crosses beneath the embankment the shear deformation is 5% or more of the distance between the upstream and downstream toe of the embankment. • A single shear crack is possible at the bottom of the embankment but higher up it is likely to manifest as a series of smaller cracks over some distance along the crest. • Zone A is widest at the bottom of the embankment (about 20m where the FF outcrop crosses) and more able to minimize seepage loss. • Deep piezometers are to be installed in the tailings over the FF to monitor changes in piezometric level due to deformation. Depending on the measured piezometric levels, and other monitoring, additional pumpwells may be installed. • Extensive deformation monitoring will be carried to monitor the performance of the embankment and will be used to assess the safe continuation of mining and design of RH-SP.
	Uncontrolled movement on FF.	Uncontrolled movement on the FF could result in a failure of the MTI Embankment and loss of tailings. The risk of uncontrolled movement on the FF increases as mining of	• Breach analyses for possible failure scenarios indicate that the tailings flowing through the breach would be contained within the existing Golden Point Pit and RH-SP.

Facility	Risk	Comments	Management/Mitigation
		RH-SP gets deeper.	• In the event of a failure of the embankment it is possible that some of the tailings loss could be deflected north towards the Process Plant and Deepdell Creek. • If the deformation monitoring indicates that failure is possible a deflection berm can be constructed across the ROM Pad to deflect the tailings back towards Golden Point Pit and RH-SP. Alternatively the ROM pad could be re-graded at the start of mining to act as a permanent deflection berm.
	Breach of MTI Embankment due to deformation on the FF resulting in loss/erosion of tailings.	This scenario does not include a large scale failure of the embankment, which is covered above for uncontrolled movement on the FF.	• The MTI TSF will be closed when mining RH-SP starts and therefore there will be no operational water ponding on the TSF to exacerbate erosion in the event of a shallow breach. • Significant visual and instrument monitoring will be carried out during mining and any progressive shallow slumping or cracking of the embankment will remediated as soon as it appears. • Test pits dug to about 3m depth in the tailings do not experience any significant slumping indicating that the tailings at the surface of the MTI are unlikely to slump through a shallow breach in the embankment. • Larger cracks could occur deeper down within the embankment which cannot be remediated. Loss of tailings could occur through these cracks, especially if the tailings liquefy during earthquake loading. However, the tailings are relatively dense at depth and will tend to dilate on shearing reducing mobility, and the rockfill in the downstream shoulder will inhibit loss of tailings. The loss of tailings is likely to be contained within the rockfill and tailings escaping from the downstream shoulder of the embankment should be

Facility	Risk	Comments	Management/Mitigation
			minimal.
	Deformation causing loss of subsoil drains.	Previous deformation of the embankment due to movement on the FF has already damaged some subsoil drains (i.e. no seepage at the outlets).	• Monitoring to date indicates that the loss of the subsoil drains during previous mining activity has not significantly affected the performance of the TSF in the area of the FF. • Seepage loss is likely to end up either in RH-SP or on the Plant Site where it drains to the environmental sump. Seepage loss will therefore generally be contained within the site and collected as part of the formal seepage collection system. • After closure of the MTI TSF, and as the tailings drain with time, so the need for the subsoil drains becomes less critical.
	Fracturing of bedrock resulting in loss/erosion of tailings and/or increased seepage loss.	Tension cracks will occur in the bedrock due to movement on the FF and increased seepage loss could occur. This could elevate the water pressure on the FF exacerbating the movement.	• Increased seepage loss will tend to follow the cracks and end up in RH-SP where it can be collected and treated. • Monitoring piezometers will be installed in the vicinity of the FF and additional pumpwells installed if it appears that elevated piezometric levels are causing additional risk
Construct RTS	Instability of RTS.	The greatest risk is during earthquake loading where liquefaction of the MTI tailings could occur. The RTS tailings will be partially saturated and therefore liquefaction of these tailings is not feasible.	• Further investigation and testing is proposed for the design of the RTS to confirm the properties of the tailings and the piezometric levels. • For detailed design state of the art dynamic deformation analyses are proposed to assess the performance of the RTS under earthquake loading. • Rigorous stability analyses will be carried out for detailed design of the RTS, together with monitoring of deformation and piezometric levels during and after

Facility	Risk	Comments	Management/Mitigation
			construction. • The performance of the RTS will be monitored during construction and the RTS design revised if necessary. • Careful stormwater controls will be implemented to collect and discharge stormwater in a controlled manner to avoid erosion and minimize wetting of the tailings.

Abbreviations

RH-SP	Round Hill – Southern Pit
SP11A TSF	Southern Pit Option 11A Tailings Storage Facility.
SP10 TSF	Southern Pit Option 10 Tailings Storage Facility.
MTI TSF	Mixed Tailings Impoundment Tailings Storage Facility.
RTS	Reclaimed Tailings Stack
FF	Footwall Fault

EXTRACT FROM EXISTING SPECIFICATION

MATERIALS

A4. WASTE MATERIAL TYPES

Mine waste material is categorized by "type" defined in terms of the mechanical or geotechnical properties of the waste after placement into its final condition. The categories are not necessarily closely related to the geological origin, state of weathering, degree of oxidation, hardness, average size or other such property.

Description of the primary functions and earth fill specifications for the embankment fill are given in the "Embankment Fill Material" Annexure.

To achieve the requirements of structural strength, tailings retention capability and control of seepage, the embankments at the Macraes Gold Operations incorporate various waste zones. These zones fulfil different functions and only certain specifications of waste rock are required or permitted within the zones as follows.

A4.1 Type 1 – Waste Material

Type 1 waste material shall consist of completely, highly or moderately weathered schist rock which after placement, conditioning and compaction as specified has maximum limits on particle size and is capable of forming a strong, dense, low permeability fill.

Schist rock fulfilling the criteria for Type 1 waste material is generally but not necessarily expected to be found at depths of up to 10 m below the existing ground surface at the Site and up to about 3 m below existing ground surface elsewhere.

Type 1 waste material is found in the mine pits, haul roads, in making preparations for embankment foundation, and other borrow areas outside of the embankment footprint.

A4.2 Type 2 - Waste Material

Type 2 waste material shall consist of loess, colluvium, solifluction or any combination of these which after blending with Type 1 or Type 3 waste material, placement, conditioning and compaction, is within the maximum limits on particle size and is capable of forming a strong, dense, low permeability fill.

Type 2 waste material deposits are found in limited quantities in the mine pits, embankment foundation preparations, haul roads and diversion drains. The loess, colluvium and solifluction deposits are often intermixed in limited zones up to about 3 m deep overlying weathered schist.

A4.3 Type 3 - Waste Material

Type 3 waste material shall consist of slightly weathered or fresh schist rock which after placement and compaction has maximum limits on particle size and constitutes a strong dense fill.

Type 3 waste material is primarily found in the mine pits, but some small quantities may be found in making preparations for embankment foundation.

A4.4 Type 4 - Waste Material

Type 4 waste material shall comprise all mine waste and materials from embankment foundation preparation and other works (e.g. haul roads) and which is not Type 1, 2, 3 or 5 waste material.

A4.5 Type 5 - Waste Material
Type 5 waste material will be reject material which will usually only come from the excavations required to expose the foundations or profile of the Works and will comprise weak or organic material, largely excavated from the bases of natural drainage channels and the like.

EMBANKMENT ZONE MATERIALS

C1. TESTING
Embankment fill materials shall be subject to compliance testing and/or Acceptance Testing as reasonably required by the Principals Representative and/or these Specifications.

C3. ZONE A1 MATERIALS
Embankment fill material specified as “Zone A1” controls seepage and contributes to embankment strength and shall be constructed from either Type 1 waste material or partially weathered Type 3 waste material blended with Type 2 waste material.

Zone A1 material shall have the following properties after placement, conditioning and compaction:

PROPERTY	SPECIFICATION
permeability	not more than 10^{-7}m/s
dry density	any individual test result shall not be less than 2.05t/m^3
water content	any individual test result shall not be less than 5.5%.

Note: dry density and water content specification may be changed in the event of a significant change in the type of material being compacted.

C3.1 Particle Size
Zone A1 material shall have the following particle size grading after compaction within the following limits:

SIEVE SIZE	%PASSING BY DRY WEIGHT
150mm	100
75mm	85 – 100
37.5mm	75 – 100
19m	60 – 100
9.5mm	45 – 92
4.75mm	30 – 82
2.36mm	22 – 72
600 microns	15 – 58
75 microns	9 – 40

Steps shall be taken to ensure that the maximum particle size is not exceeded within Zone A1 fill layer.

C3.2 Compaction

The loose layer thickness for each lift, including compaction equipment and number of passes shall meet the specifications required to achieve the desired fill properties. Where the Zone A1 fill appears loose, is poorly compacted or contains a high proportion of fines (greater than 25% passing the 75 micron sieve) the Contractor shall provide proof rolling to the satisfaction of the Principals Representative. Under the passage of a rubber tyred vehicle of no less than 90 tonne weight, there shall be no appreciable deformation of Zone A1. The Principals Representative may, at its sole discretion, permit the use of a lesser weight vehicle at their discretion and if there is deformation of Zone A1, then the substandard fill shall be removed and replaced.

C6. ZONE B MATERIALS

Zone B forms the remainder of the structural fill portions of the tailings and silt pond embankments. It shall comprise Type 1, 2 or 3 waste material and have the following properties after placement and compaction:

PROPERTY	SPECIFICATION
dry density	the mean value of the results of any ten consecutive tests shall not be less than $2.1t/m^3$
	any individual test result shall not be less than $2.0t/m^3$

The maximum particle size for material in Zone B shall not exceed 500mm.

Care shall be taken to avoid placing coarse rockfill (material without any appreciable quantity of silt, sand and finer gravel) in direct contact with Zones A, A1, A2 and A3. Well graded material to a width of at least 1m shall be placed against Zones A, A1, A2 and A3 to ensure a transition from finer to coarser graded material.

C7. ZONE B1 MATERIALS

Zone B1 is a structural fill zone placed between Zone A, A1 and A2 and Zone B to provide an intermediate particle size distribution between the two fill types for improved filter compatibility. It shall comprise Type 1, 2 or 3 waste material and be specifically selected, or reworked, to include more fines and a smaller rock size (maximum 400mm diameter) than Zone B. Zone B1 shall have the following properties after placement and compaction.

PROPERTY	SPECIFICATION
dry density	the mean value of the results of any ten consecutive tests shall not be less than $2.1t/m^3$
	any individual test result shall not be less than $2.0t/m^3$

C8. ZONE C MATERIALS

Zone C forms the downstream shoulder of the downstream tailings embankment, contributing to strength and providing for bulk disposal of waste material. Zone C can comprise Type 1, 2, 3 or 4 waste material. Type 5 waste is specifically excluded.

a) Zone C1. All waste material in Zone C1 shall be placed in lifts no greater than 2.5 m high and shall be spread and compacted with a tracked bull dozer appropriate for the purpose to form an even surface. The route dump trucks use to reach the tip head shall be controlled so that each lift receives a relatively uniform level of compaction from the truck traffic; and

b) Zone C2. Waste material in Zone C2 shall be placed in lifts no greater than 7.5 m high.

C9. ZONE D MATERIALS

Zone D is a chimney drain. Its primary function is to intercept seepage and to limit the development of pore pressures in the downstream shoulders of the embankment. Zone D shall be constructed primarily from Type A1 drainage material except for the base collector which shall be of Type B drainage material. Type A1 drainage material shall achieve a relative density of greater than 65 percent after placement and compaction.

C10. ROCKFILL MATERIALS

Rockfill shall comprise predominantly Type 3 waste material except in the lift immediately adjacent to Zone A, A1, A2 and A3. This lift shall also include Type 1 or 2 waste so as to form a transition layer between the rockfill and Zone A, A1, A2 and A3 materials. Rockfill shall be placed in loose lifts of no greater than 0.9m thick and be compacted by the systematic passage of the dump trucks delivering the rockfill.

EARTHWORKS

D1. TESTING

Compliance and/or Acceptance Tests shall be undertaken to confirm the specified fill requirements meet or exceed these Specifications. Such tests shall be undertaken as specified or by rapid methods as reasonably required by the Principals Representative on a day to day basis. Where an adequate correlation is established between the rapid and specified methods, the Principals Representative may rely on the results of the rapid methods (including for issue of certificates) except where the Contractor's work is considered inadequate, in which case the specified tests shall be undertaken as well.

If there are any differences between the results of tests carried out by the Principals Representative and the Contractor, the Principals Representative's test results shall prevail.

D2. TEMPORARY WORKS

The Contractor shall be solely responsible for the sufficiency, stability and safety of all temporary earth works.

D3. SELECTION OF EMBANKMENT FILL

Selection of suitable materials at the nominated point of excavation for embankment fill shall be undertaken by the Contractor's Embankment Supervisor.

All fill shall be placed to the lines and levels shown on the drawings, or otherwise established by the Principals Representative

Only Materials detailed on the Drawings and which comply with these Specifications for quality, grading and properties shall be used in the embankment and any fill material which does not comply with these Specifications shall be removed and replaced at no Cost to the Principal.

D4. PLACEMENT OF EMBANKMENT FILL

All fill shall be placed at water content within the specified limits. No fill within Zone A, A1, A2 and A3 shall be placed on foundations exhibiting groundwater seepage, nor shall it be placed in puddles of water or wet areas. Drainage works required to keep foundation surfaces dry shall be subject to the Principals Representative's approval.

Where the Contractor wishes to leave construction drainage permanently in place, the Principals Representative may require pipes and cavities to be fully grouted and made impermeable after use at no cost to the Principal. Where directed by the Principals Representative such drainage will be connected to the permanent foundation drainage zones. Any fill material that becomes excessively wet, regardless of the cause, shall be removed from the embankment by the Contractor. Such fill may be re-used at a later time, provided it has been dried to the required water content and fully meets specification.

Particular care shall be taken in placing and compacting the first several lifts above the foundation surface. It is particularly important that segregation of the fill material is avoided and that any protuberances on the compactor do not penetrate the fill and damage the foundation. Placement of the initial fill parallel to the foundation surface (as opposed to a horizontal lift) for foundation surfaces flatter than 10H:1V is acceptable, provided that the shear stress on the fill created by the compactor climbing up the slope does not loosen or disturb the previously compacted layer.

Where structural fill (i.e. Zones A, A1, A2, A3 and B) is compacted against steep surfaces the fill surface shall be ramped toward the steep surface at a slope of between 6H:1V to 10H:1V, over a distance of 3 m, so that a component of the compactive force acts towards the steep surface.

Fill placement procedures shall ensure that fill materials are forced into intimate contact with the foundation surface and that the required density is achieved. The compaction method employed will depend upon the steepness of the surface to be filled against and the nature of irregularities in the foundation surface.

D5. CONDITIONING OF EMBANKMENT FILL

Conditioning shall include the spreading of waste rock or borrow material on the fill platform, reducing the particle size of rock by crushing with a protruding foot compactor if required, and the watering, mixing and final spreading using a motor grader carried out to ensure that the fill material is uniform with respect to particle size distribution, water content and layer thickness and meets the requirements of this specification.

Prior to placing any new fill, the Zone A, A1, A2 and A3 surface shall be lightly scarified to the satisfaction of the Principals Representative. For the tailings embankment level of Zone B shall be maintained at 600mm above the Zone A, A1, A2 or A3 level or as otherwise determined by the Principals Representative. All fill placed within Zone A, A1, A2 or A3 shall be conditioned thoroughly so that immediately prior to compaction, the water content of the fill is as uniform as possible within any one area. It is the Contractor's responsibility to determine the type of Plant and the sequence of operation necessary to condition a particular type of material. Once the sequence has been adopted it shall not be changed without prior discussion with the Principals Representative.

D6. COMPACTION OF EMBANKMENT FILL

Hand-tamping or mechanical compacters shall be used to compact fill in or against irregular surfaces on abutments, in potholes or depressions not accessible by heavy compaction equipment. Hand-tamped fill shall be placed in 100 mm maximum loose layer lifts.

New fill shall not be placed over previously placed fill that has failed to achieve the required standard of compaction, or which has become contaminated, or which has deteriorated from the required fill standards. Previously placed fill which does not comply shall be reinstated or removed at no Cost to the Principal. No fill shall be placed over frozen material within Zones A, A1, A2 and A3 unless it is a light frost that can be easily broken up with 2 passes of a 30 tonne self propelled steel wheeled sheepsfoot type roller to the satisfaction of the Principals Representative. Such material shall then be incorporated in the next overlying lift of fill by thorough mixing. In all other situations, frozen material shall be removed prior to placement of new fill. The Contractor shall maintain positive and effective drainage during filling operations to minimise deterioration of material exposed and the upper fill layers.

D6.1 Compaction Equipment

Compaction within Zones A, A1, A2, A3 and B shall be carried out using the most appropriate compaction equipment operated in accordance with the procedure specified by the Principals Representative. For the tailings embankment such Plant would be expected to include a large, self propelled rubber tyred roller of at least 90 tonne (e.g. a 50,000l watercart or equivalent), a self propelled protruding foot steel wheeled roller of not less than 30 tonnes weight, and a vibrating steel roller of not less than 18 tonnes static weight. For the silt pond embankments a large towed sheepsfoot compactor (12 tonne static weight) would be acceptable for Zone A2. A vibrating drum roller 9 tonne static weight) would be acceptable for Zone B and possibly Zone A2. The compaction of Zone A3 will have to be confirmed by field trial tests.

Compaction equipment shall be trafficked in an orderly and systematic manner always as nearly as practical in a direction parallel to the embankment crest and so as to ensure that the entire surface of each fill layer is traversed by a uniform compactive effort and meets or exceeds these Specifications everywhere.

The Contractor shall determine the type of Plant, the number of passes and the sequence of operation necessary to compact a particular type of material to the specified standard. The sequence so determined shall not be varied without prior agreement with the Principals Representative.

D6.2 Layer thicknesses

The permitted layer thickness depends on the Plant used by the Contractor. Where Plant meets or exceeds the requirements specified in (a) above fill material shall be spread and compacted in layers which do not exceed 350 mm loose thickness in Zones A and A1 prior to compaction, provided there is no evidence of segregation, 250mm loose thickness prior to compaction in Zone A2, and 600 mm loose thickness prior to compaction in Zone B.

Where other Plant approved by the Principals Representative is used the maximum loose layer thickness for Zones A, A1, A2 and A3 shall not exceed 200mm, unless trials are undertaken and it can be demonstrated that a greater layer thickness is acceptable. If segregation of Zone A, A1, A2 and A3 fill occurs during placement, the layer thickness shall be reduced as required by the Principals Representative. Any irregularities in the loose layer thickness shall be levelled out by blade prior to or during the first pass of compaction equipment.

D6.3 Compactive effort

Each layer of fill within Zone A, A1, A2, A3 and B shall, after spreading, be compacted so as to meet all required Specifications. The productivity of available spreading and compaction Plant shall always exceed that required to place and compact the fill at the rate it is delivered to the fill.

D6.4 Fill slopes

Fill slopes steeper than 20% gradient shall be overfilled as necessary and trimmed back to ensure that all fill in the slope face is adequately compacted. Erosion of temporary slopes shall be repaired to the specified standards of the fill before any further fill is placed on the slope.

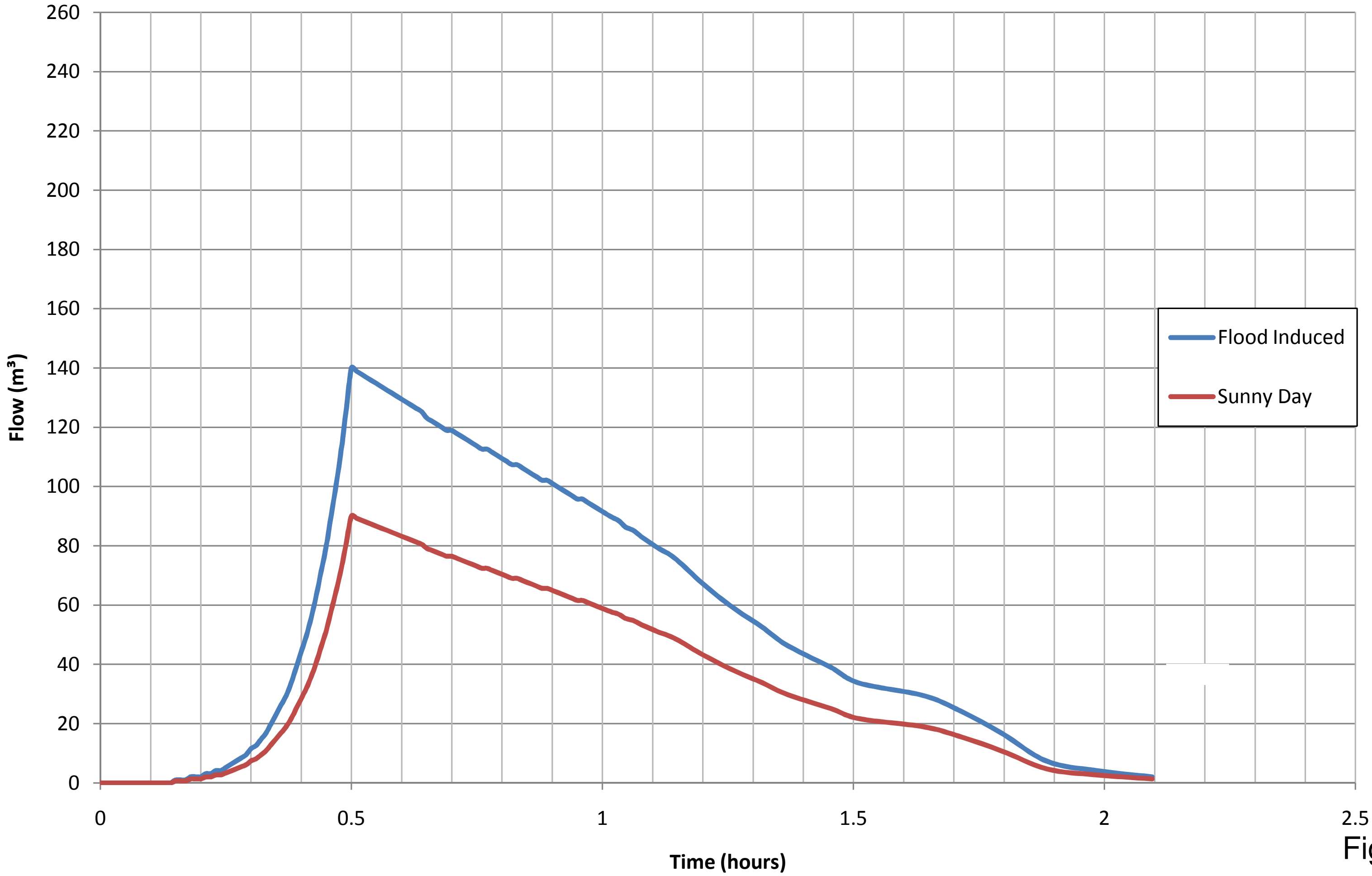

Figure 1

Engineering Geology Ltd
Unit 7C, 331 Rosedale Rd, PO Box 301054,

OCEANA GOLD (NZ) LTD, MACRAES GOLD PROJECT
TOP TIPPERARY TAILINGS STORAGE FACILITY
BREACH HYDROGRAPHS

Ref. No: 6786
Date: 3 June 2011
Drawn: SFC
File: Hydrograhs.xlsx

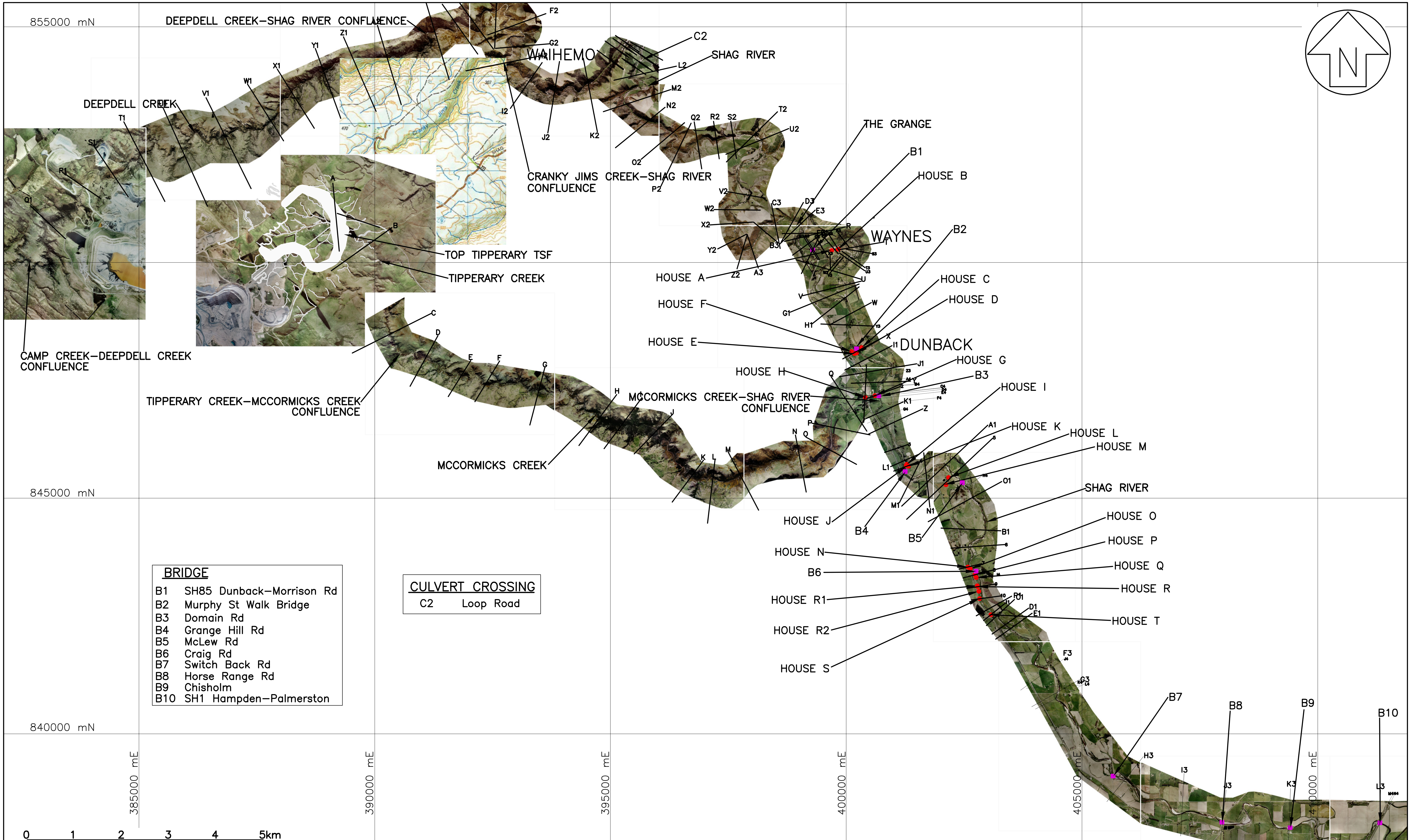

Figure 2

ENGINEERING GEOLOGY LTD
Unit 7C, 331 Rosedale Rd, PO Box 301054, Albany
Ph (09)486–2546, Fax (09)486–2556

OCEANA GOLD (NZ) LTD, MACRAES GOLD PROJECT
Top Tipperary TSF Dam Breach Study
Site Plan

Drawing No.	6846–Figure 3
Date:	25 Mar 2011
Drawn:	BL
Scale:	1:75000 (@A3)
Filename:	6846–Figure 3.dwg

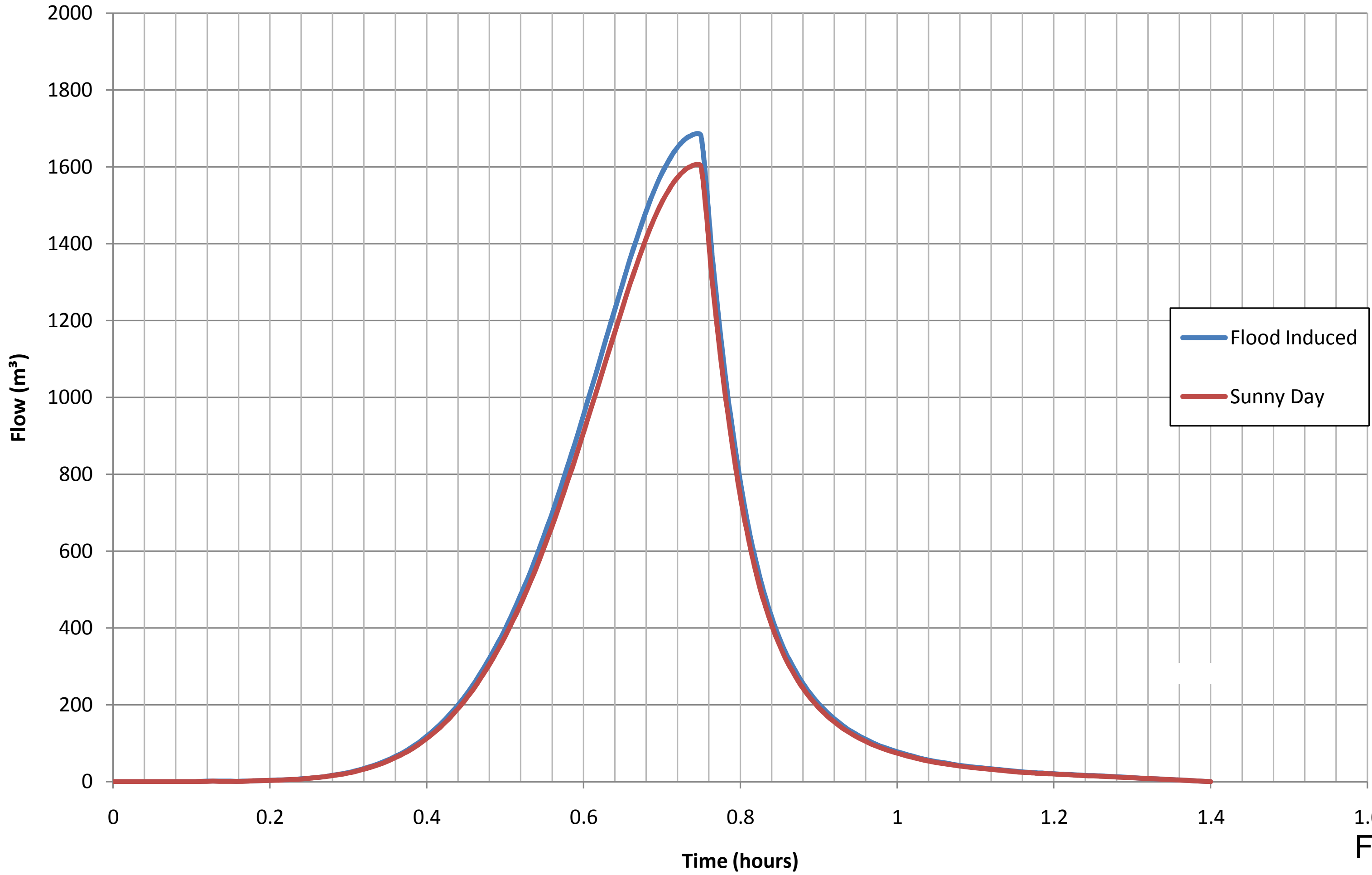

Figure 3

Engineering Geology Ltd
Unit 7C, 331 Rosedale Rd, PO Box 301054,

OCEANA GOLD (NZ) LTD, MACRAES GOLD PROJECT
CAMP CREEK WATER STORAGE DAM BREACH STUDY
BREACH HYDROGRAPHS

Ref. No: 6786
Date: 3 June 2011
Drawn: SFC
File: Hydrographs.xlsx

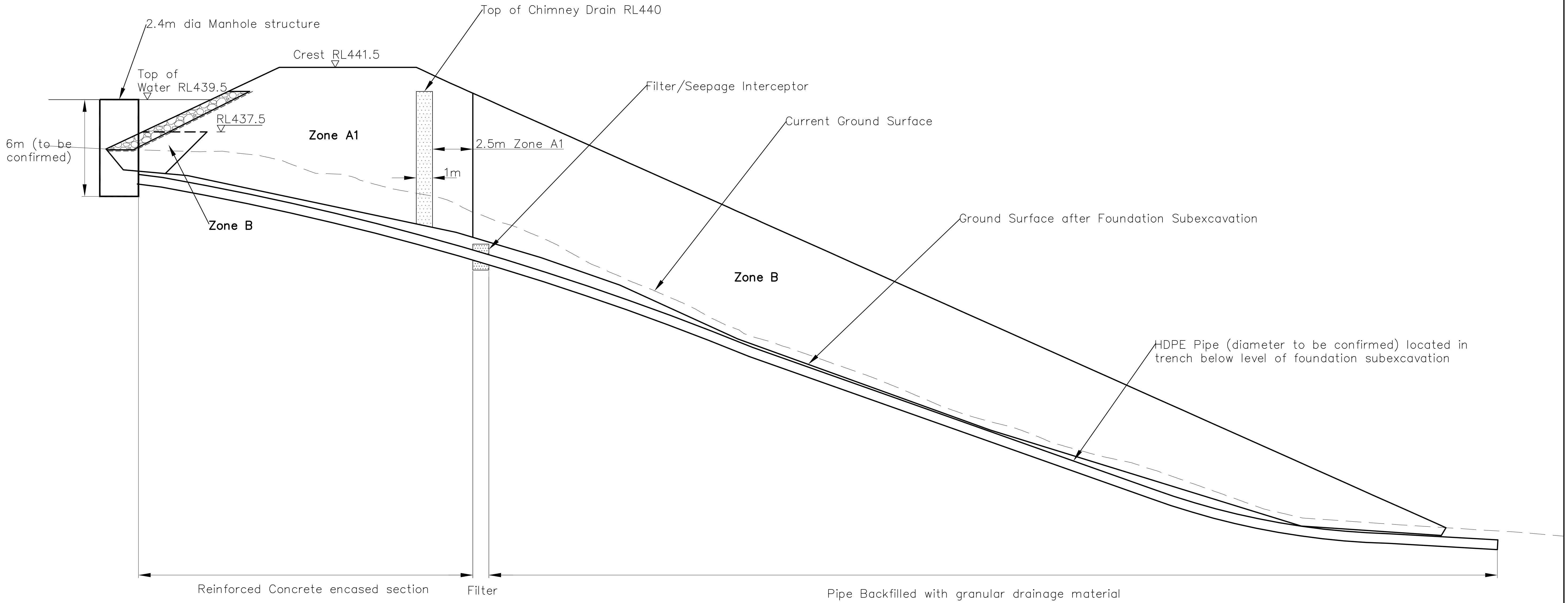

Figure 4

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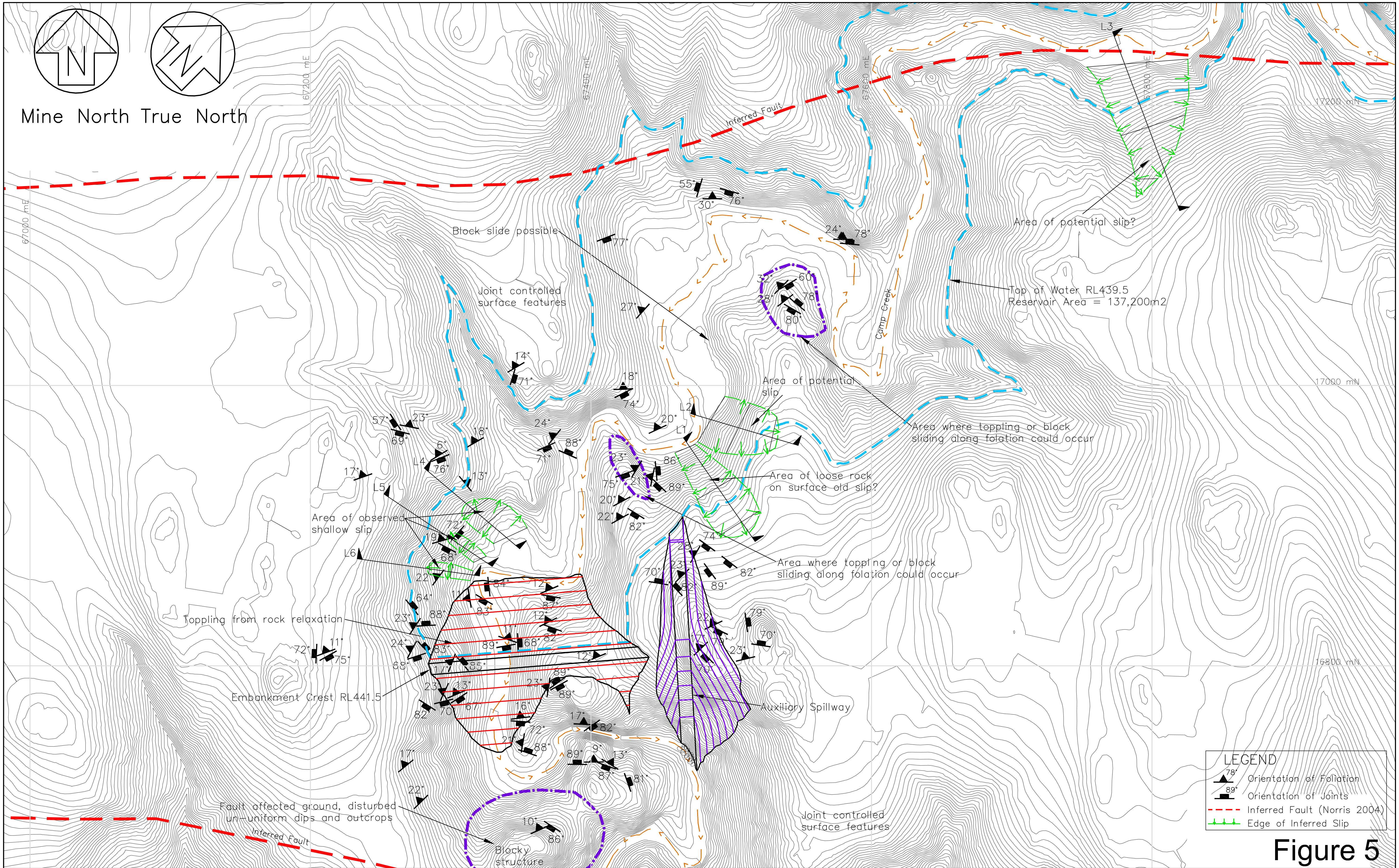
OCEANA GOLD (NZ) LTD, MACRAES GOLD PROJECT
Camp Creek Water Storage Dam
Long Section Along Primary/Service Spillway

Drawing No.	6786-Fig 4
Date:	9 June 2011
Drawn:	BL
Scale:	1:250 (@A3)
Filename:	6786-Fig4.dwg



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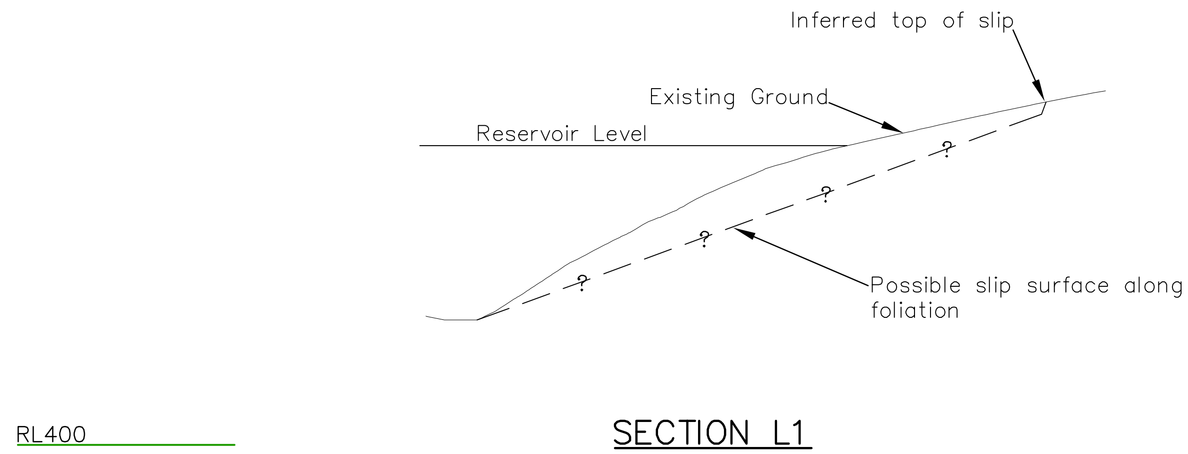
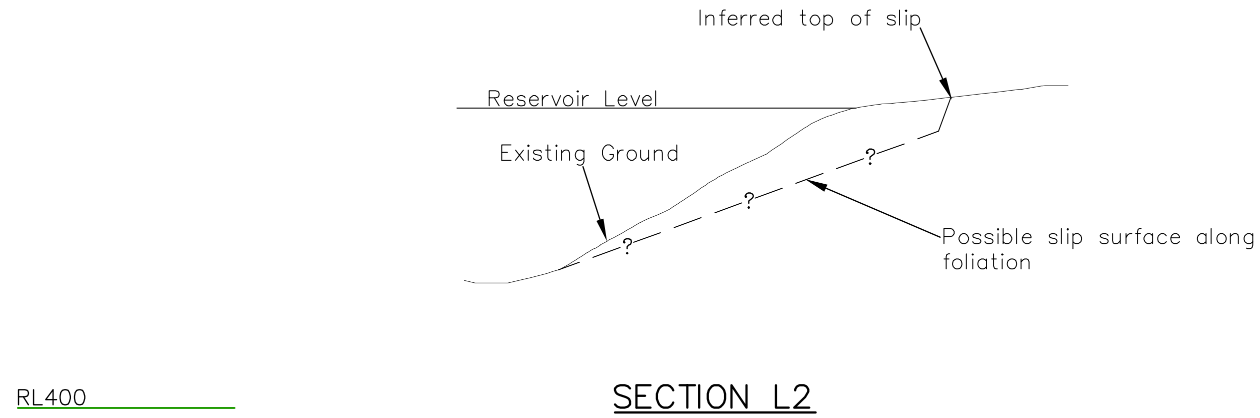
OCEANA GOLD (NZ) LTD, MACRAES GOLD PROJECT
Camp Creek Water Storage Dam
Geological Mapping

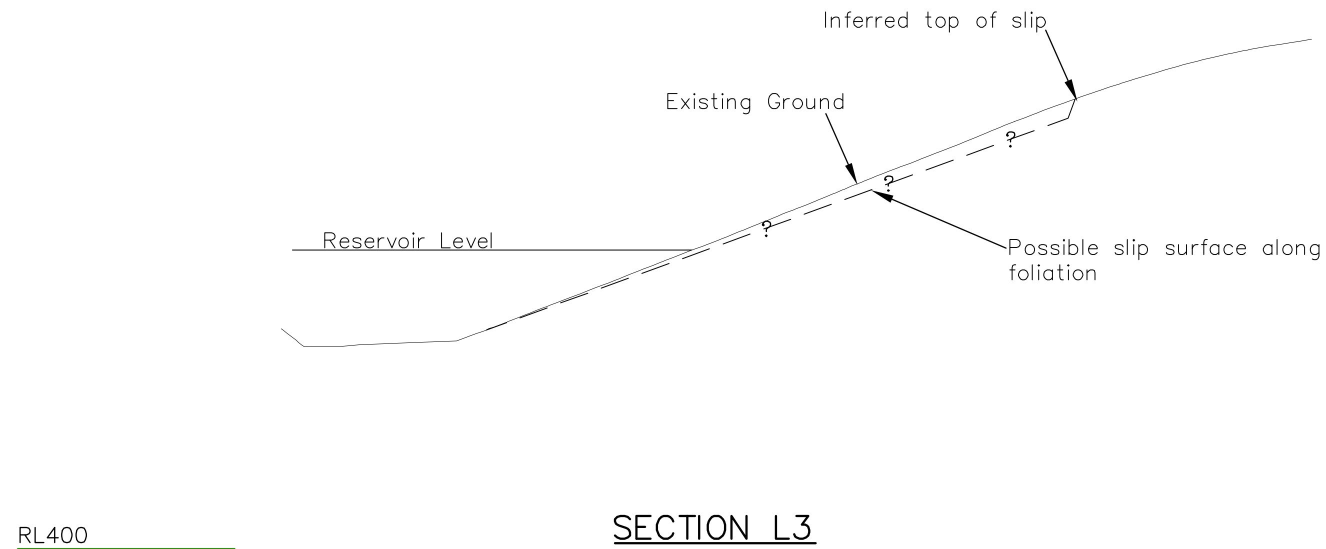
Drawing No. 6914-Fig 6
Date: 21 Jan 2011
Drawn: MC/BL
Scale: 1:2500 (@A3)
Filename: 6914-Fig 6.dwg



SECTION L1



SECTION L2



SECTION L3

Figure 6

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Ph (09)486–2546, Fax (09)486–2556

OCEANA GOLD (NZ) LTD, MACRAES GOLD PROJECT
Camp Creek Water Storage Dam
Cross Sections L1, L2 and L3

Drawing No.	6914–Fig 6
Date:	Jun 2011
Drawn:	DM
Scale:	1:1000 (@A3)
Filename:	6914–Fig 6.dwg

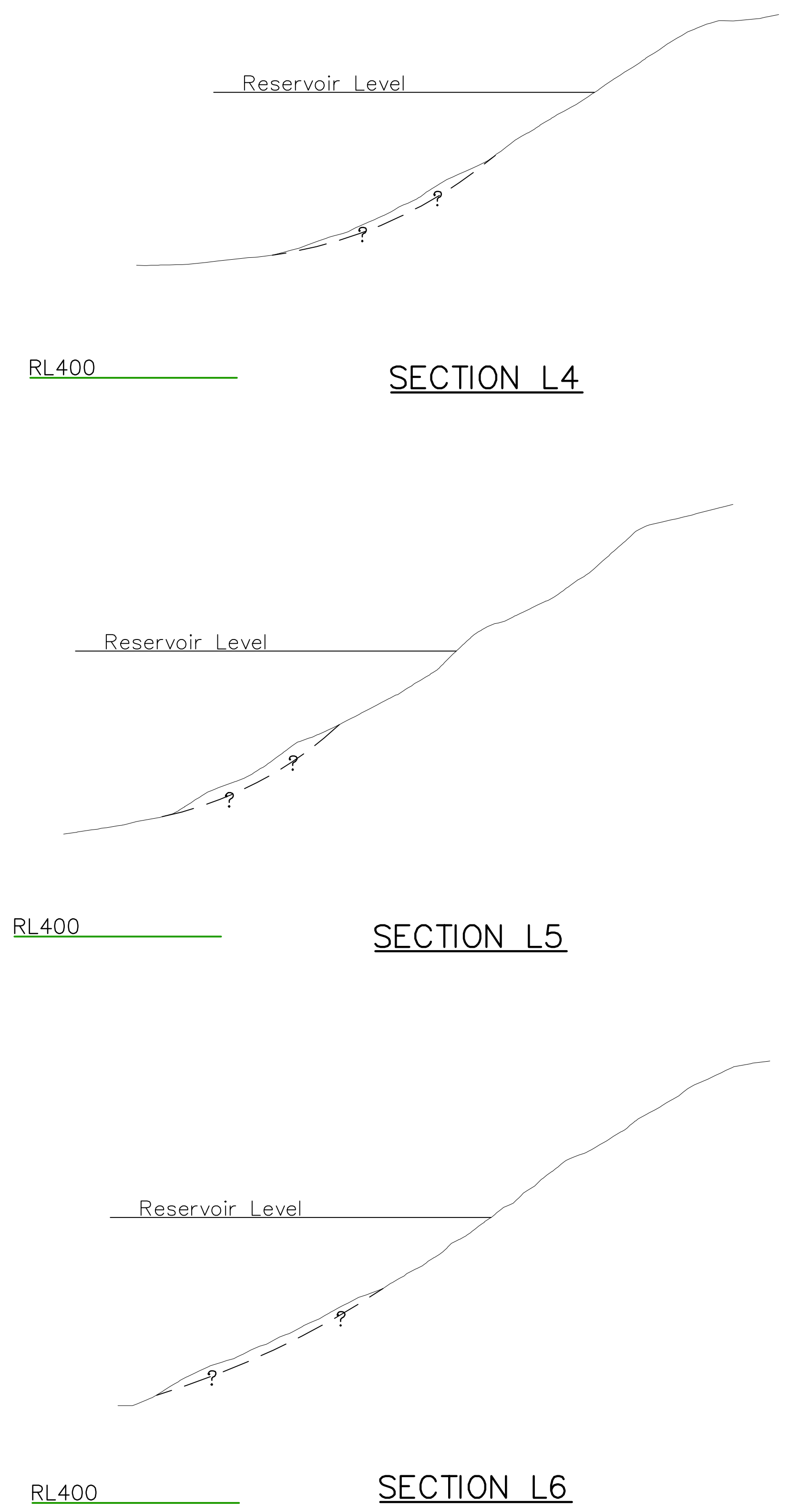

Figure 7

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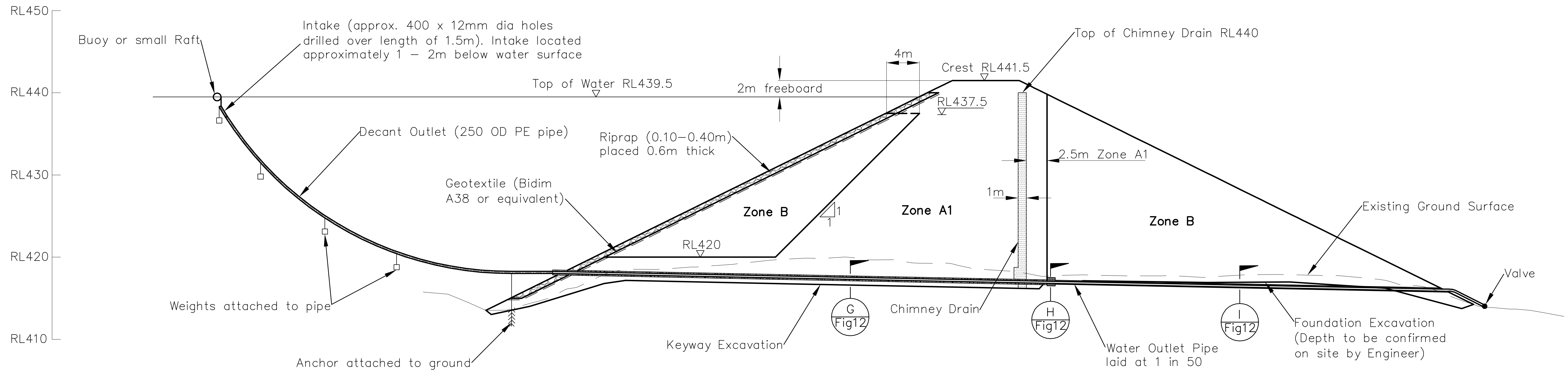
OCEANA GOLD (NZ) LTD, MACRAES GOLD PROJECT
Camp Creek Water Storage Dam
Cross Sections L4, L5 and L6

Drawing No.	6914-Fig 6
Date:	Jun 2011
Drawn:	DM
Scale:	1:1000 (@A3)
Filename:	6914-Fig 6.dwg



EMBANKMENT CROSS – SECTION F – F' AT WATER OUTLET PIPE

Figure 11

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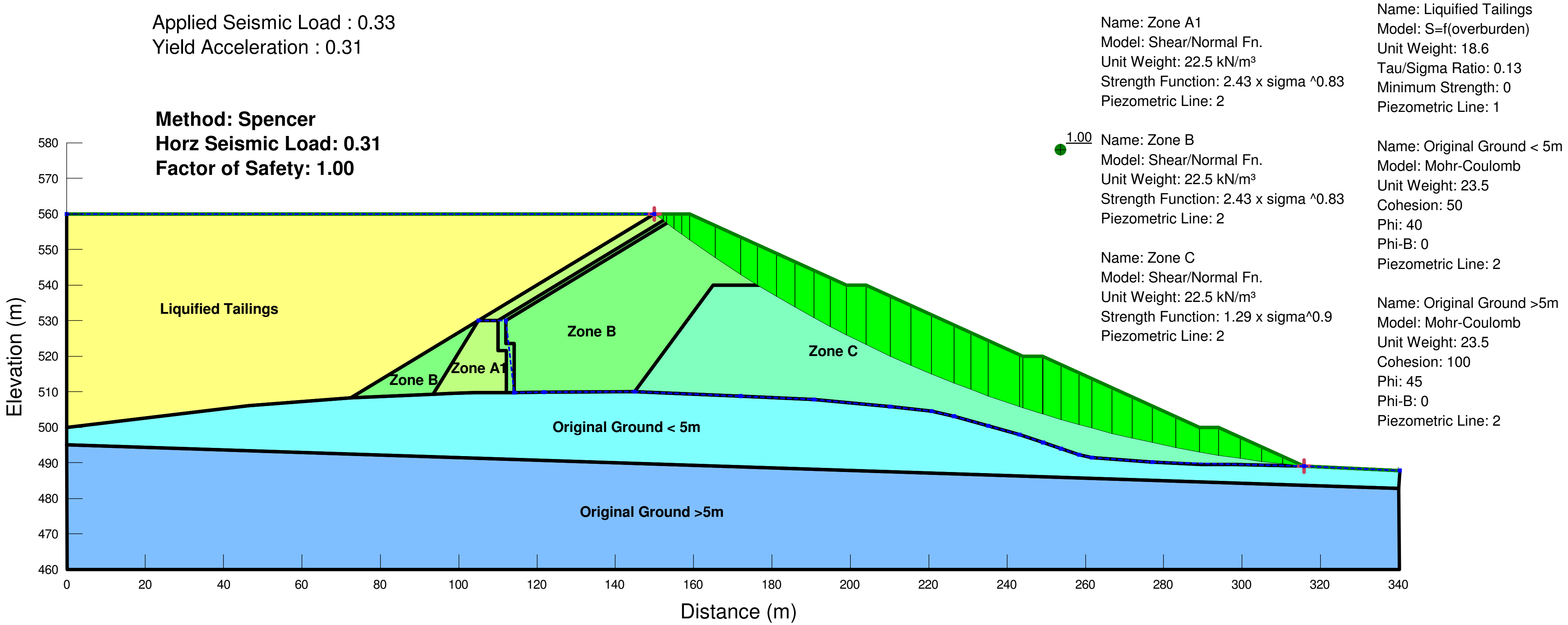
Ph (09)486–2546, Fax (09)486–2556

OCEANA GOLD (NZ) LTD, MACRAES GOLD PROJECT
Camp Creek Water Storage Dam
Section at Water Outlet Pipe

Drawing No.	6914–Fig 11 Rev.A
Date:	14 June 2011
Drawn:	BL
Scale:	1: 500 (@A3)
Filename:	6914–Fig 11–A.dwg

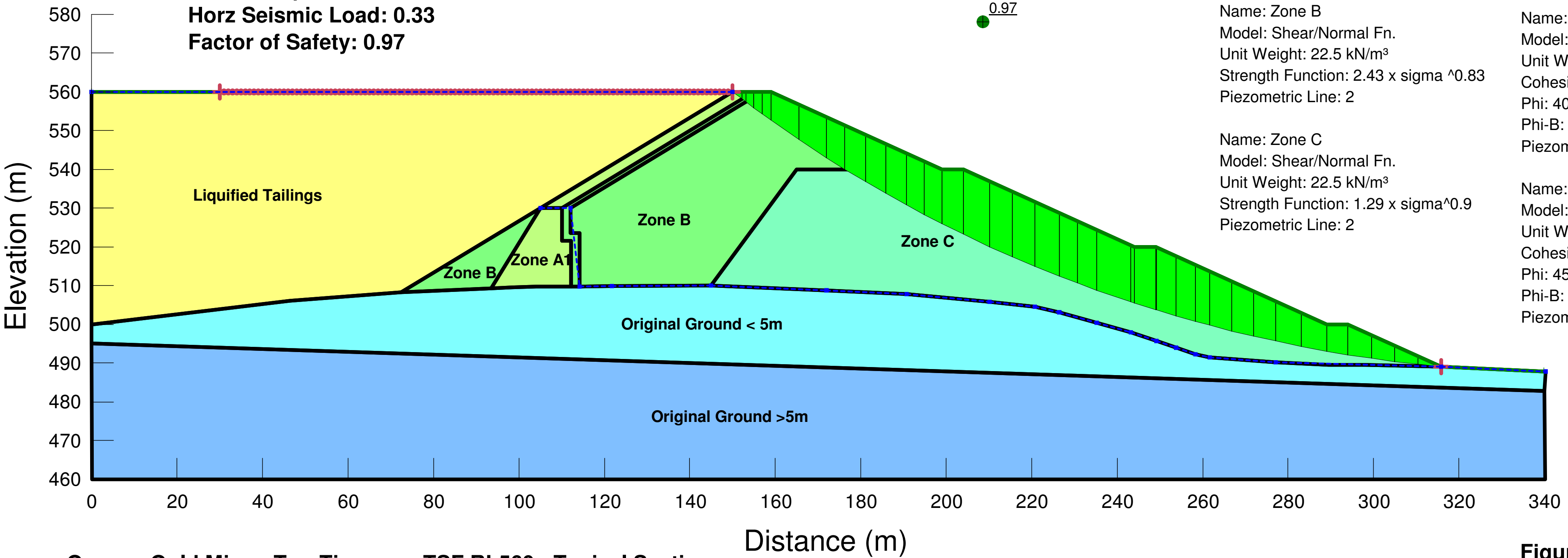


Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 475 Yr Return Period - Full Dam Height (Yield Acceleration)

Figure D7
3:12:50 p.m.
1/03/2011

Applied Seismic Load : 0.33
Yield Acceleration : 0.31

Method: Spencer
Horz Seismic Load: 0.33
Factor of Safety: 0.97



Name: Zone A1
Model: Shear/Normal Fn.
Unit Weight: 22.5 kN/m³
Strength Function: 2.43 x sigma ^0.83
Piezometric Line: 2

Name: Zone B
Model: Shear/Normal Fn.
Unit Weight: 22.5 kN/m³
Strength Function: 2.43 x sigma ^0.83
Piezometric Line: 2

Name: Zone C
Model: Shear/Normal Fn.
Unit Weight: 22.5 kN/m³
Strength Function: 1.29 x sigma^0.9
Piezometric Line: 2

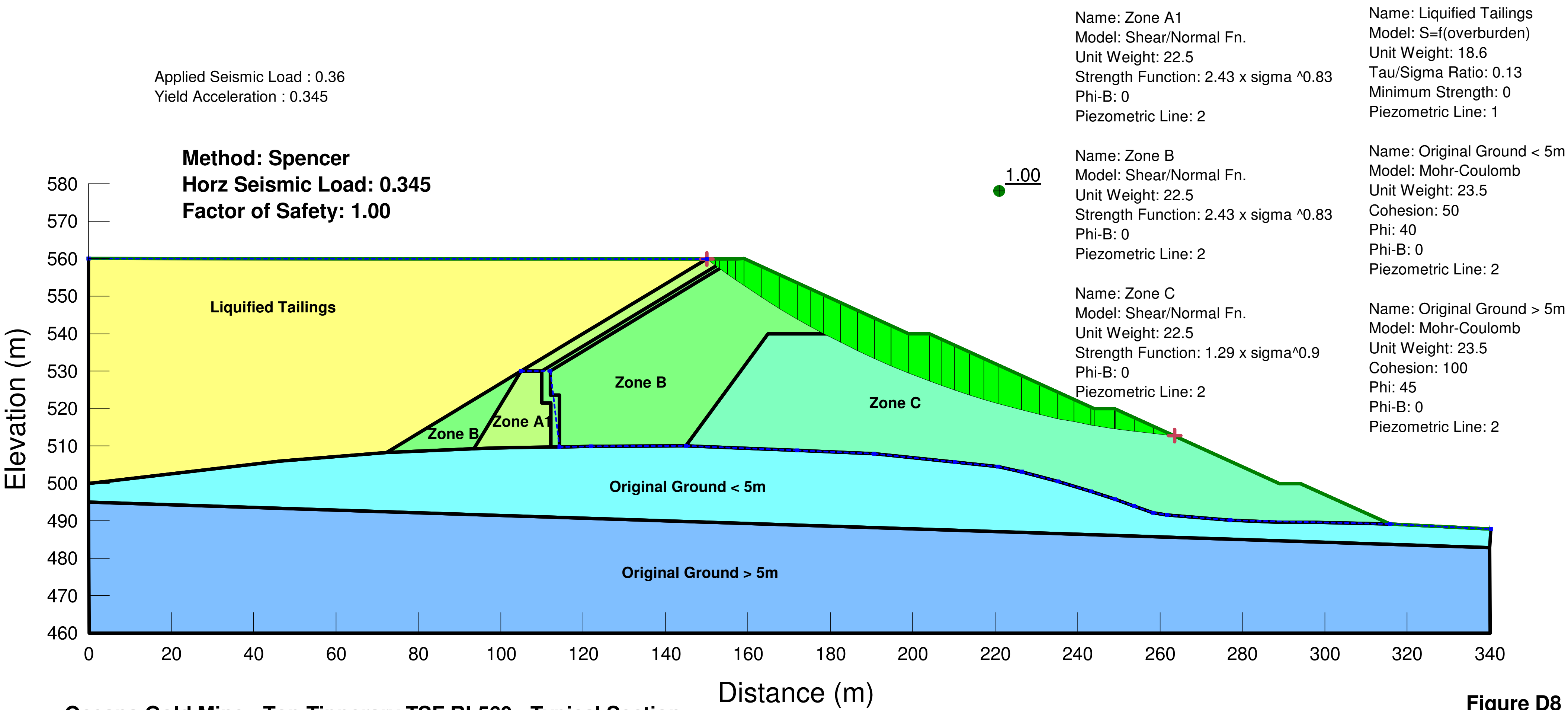
Name: Liquified Tailings
Model: S=f(overburden)
Unit Weight: 18.6
Tau/Sigma Ratio: 0.13
Minimum Strength: 0
Piezometric Line: 1

Name: Original Ground < 5m
Model: Mohr-Coulomb
Unit Weight: 23.5
Cohesion: 50
Phi: 40
Phi-B: 0
Piezometric Line: 2

Name: Original Ground >5m
Model: Mohr-Coulomb
Unit Weight: 23.5
Cohesion: 100
Phi: 45
Phi-B: 0
Piezometric Line: 2

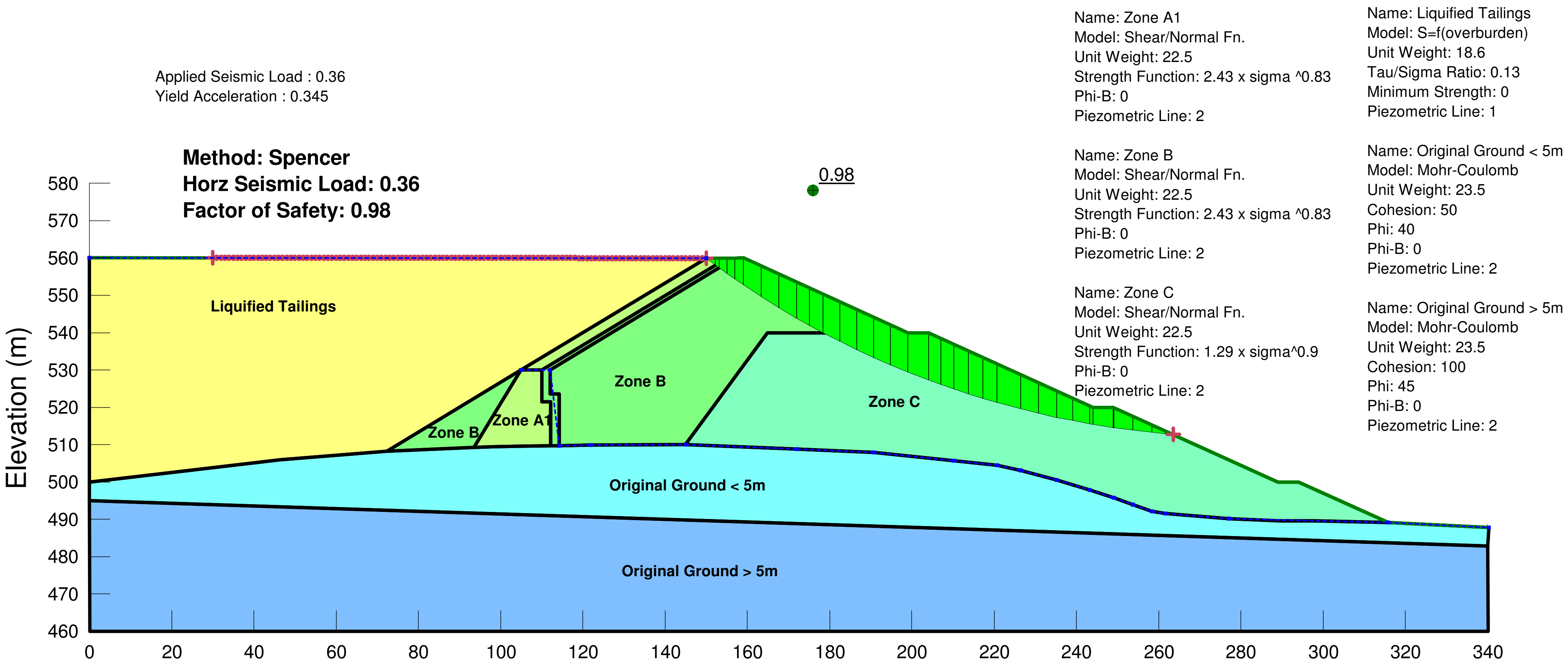
Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 475 Yr Return Period - Full Dam Height (Applied Seismic Load)

Figure D7a
8:37:00 a.m.
6/06/2011



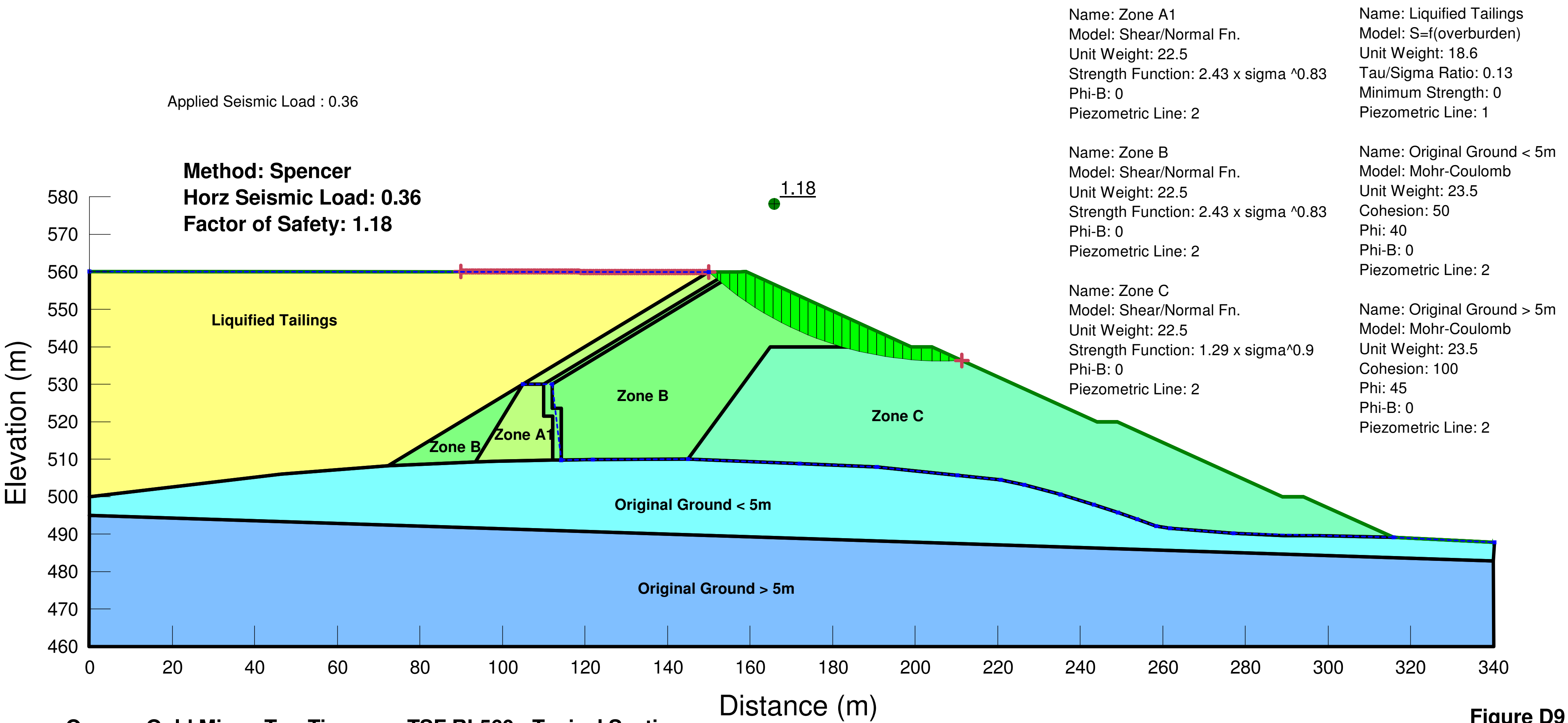
Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 475 Yr Return Period - Two Thirds of Dam Height (Yield Acceleration)

Figure D8
8:58:49 a.m.
6/06/2011



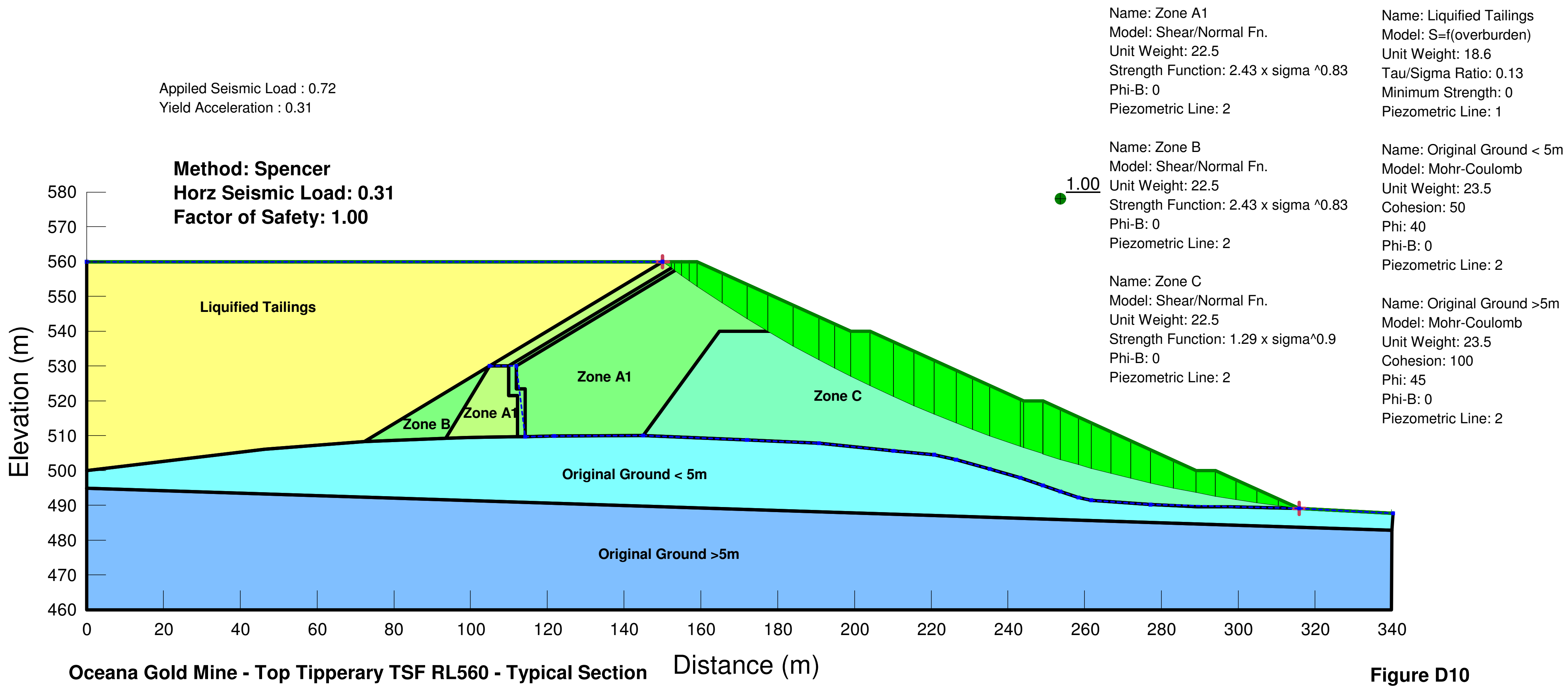
Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 475 Yr Return Period - Two Thirds of Dam Height (Applied Seismic Load)

Figure D8a
9:01:55 a.m.
6/06/2011



Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 475 Yr Return Period - One Third of Dam Height (Applied Seismic Load)

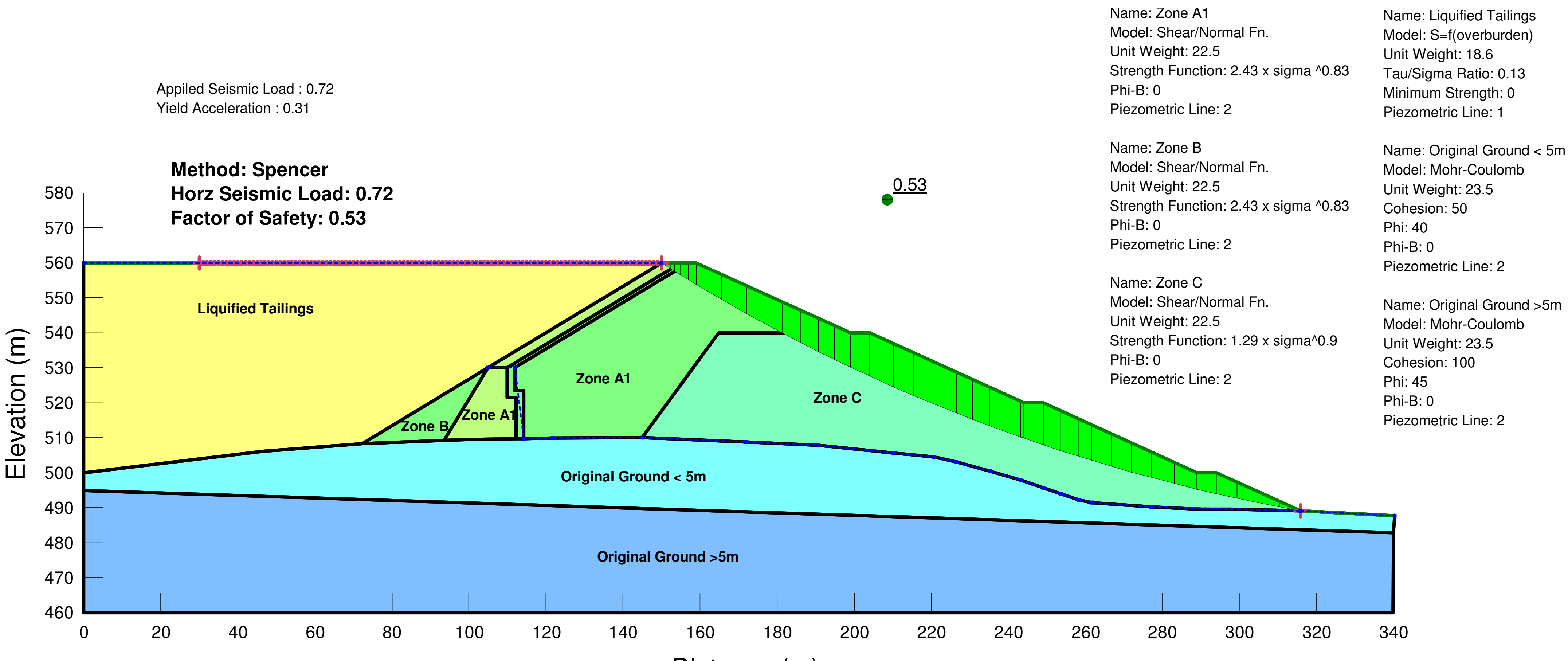
Figure D9
2:57:53 p.m.
1/03/2011



Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 2500 Yr Return Period - Full Dam Height (Yield Acceleration)

Figure D10

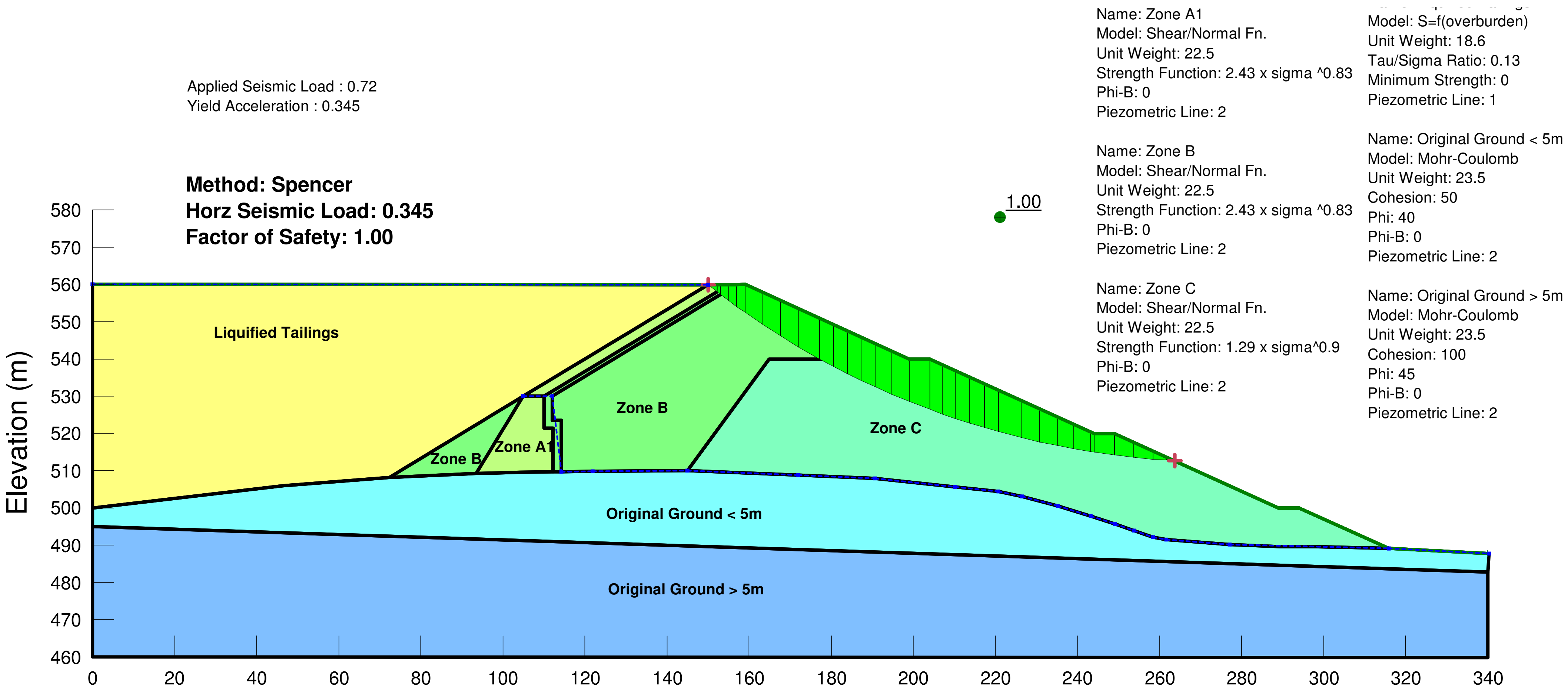
9:29:21 a.m.
6/06/2011



Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 2500 Yr Return Period - Full Dam Height (Applied Seismic Load)

Figure D10a

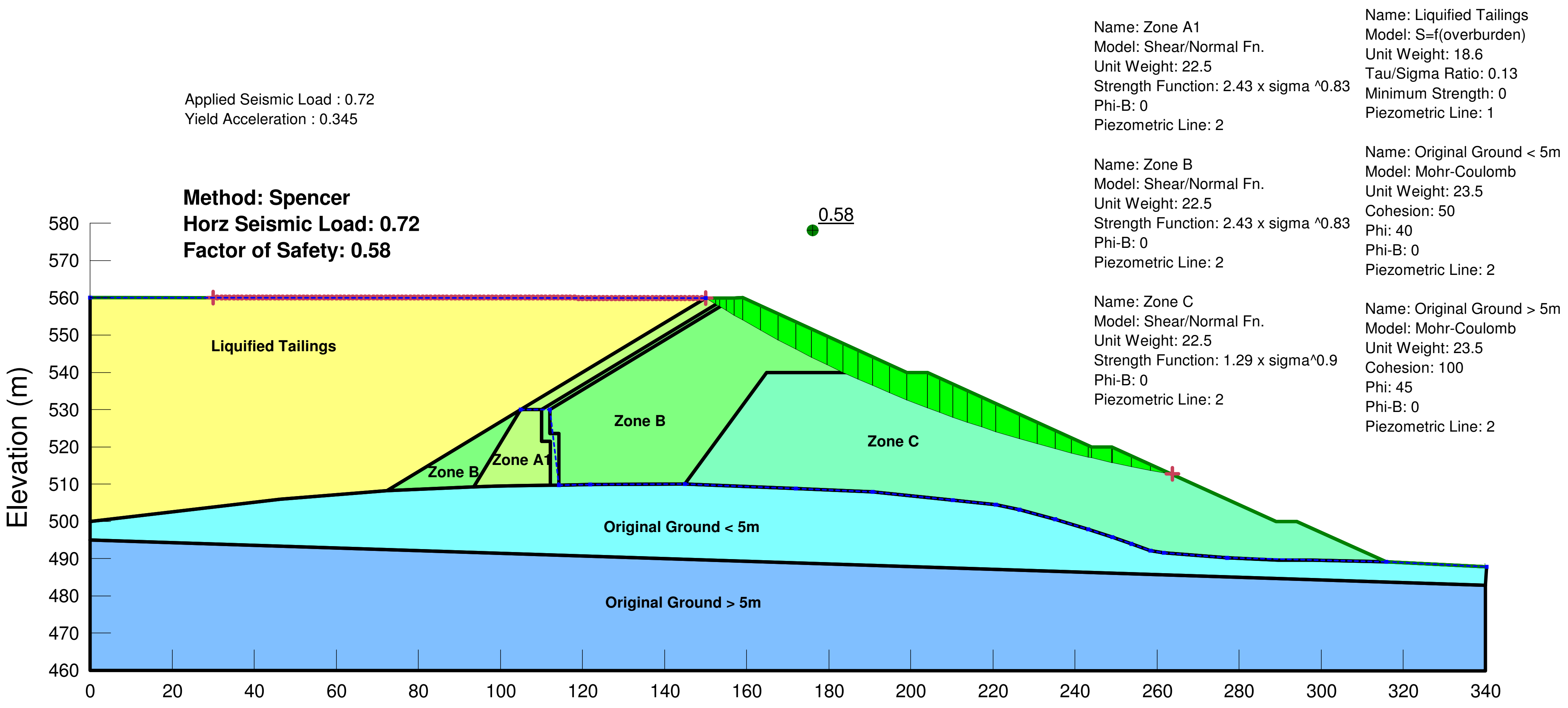
9:37:10 a.m.
6/06/2011



Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 2500 Yr Return Period - Two Thirds of Dam Height (Yield Acceleration)

Figure D11

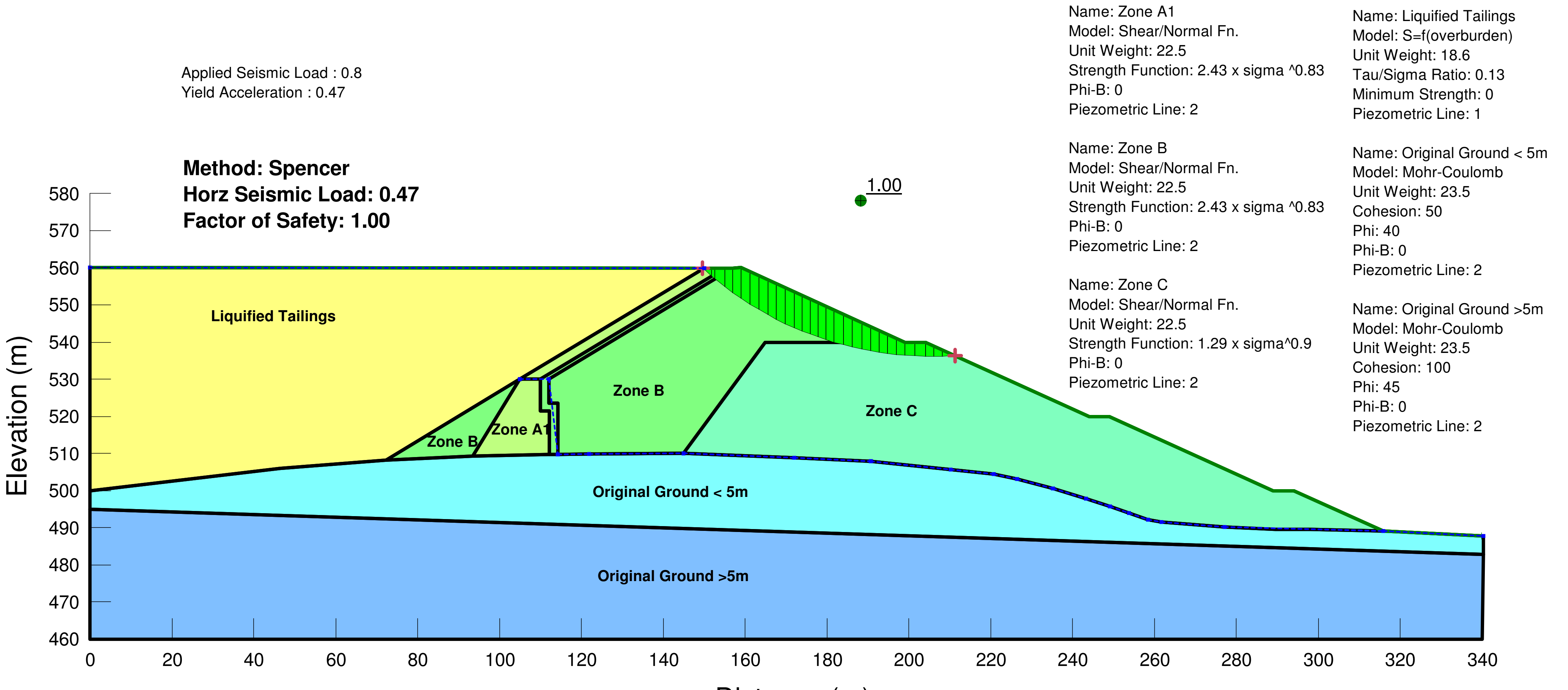
9:44:05 a.m.
6/06/2011



Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 2500 Yr Return Period - Two Thirds of Dam Height (Applied Seismic Load)

Figure D11a

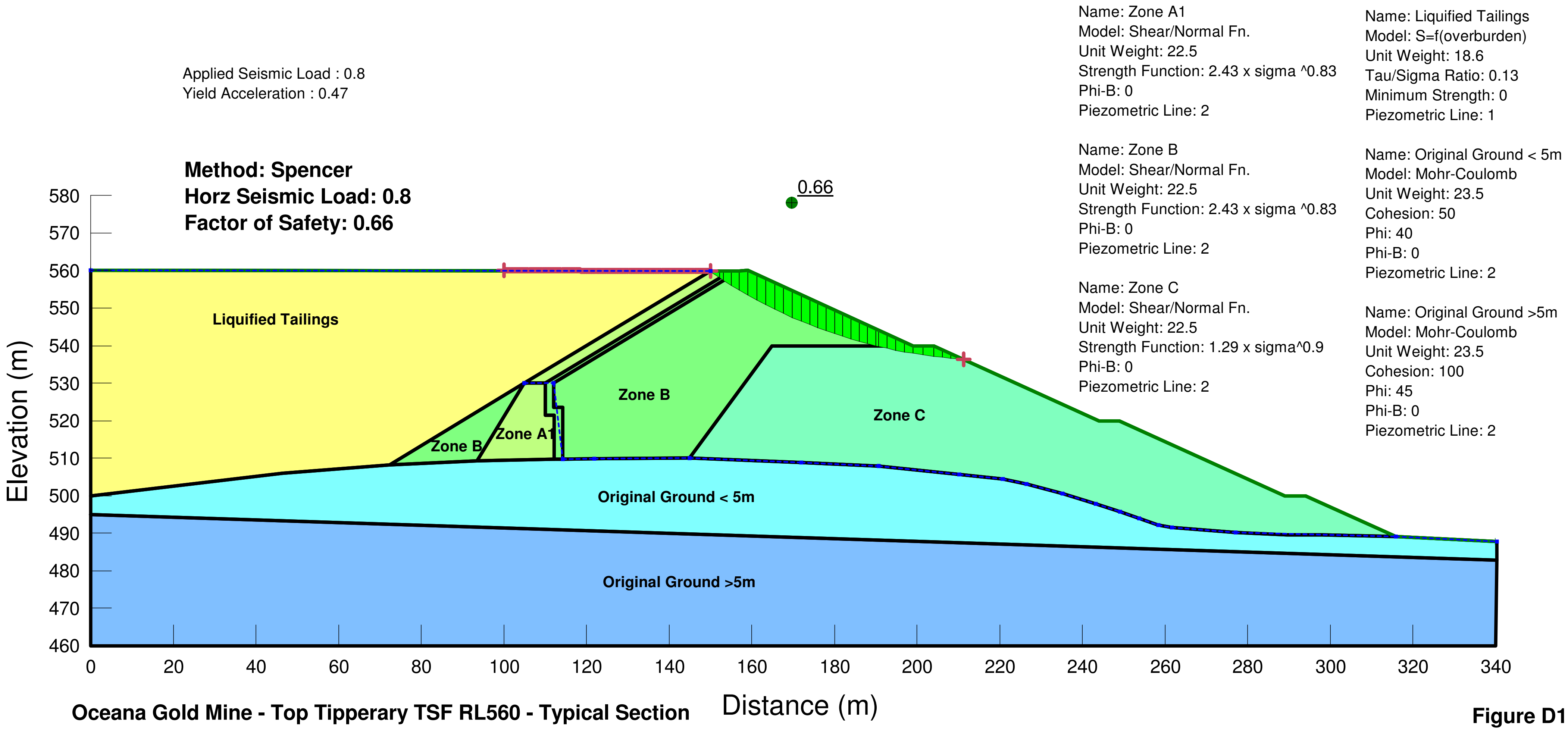
9:46:57 a.m.
6/06/2011



Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 2500 Yr Return Period - One Third of Dam Height (Yield Acceleration)

Figure D12

1:36:38 p.m.
1/03/2011



Oceana Gold Mine - Top Tipperary TSF RL560 - Typical Section
Earthquake 2500 Yr Return Period - One Third of Dam Height (Apllied Seismic Load)

Figure D12a

10:03:36 a.m.
6/06/2011