Midnite Mine Superfund Site
100 Percent Design

Appendix D – Mine Waste Excavation and Containment

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LIST OF ACRONYMS

ARD	acid rock drainage
ASTM	American Society for Testing and Materials
BODR	Basis of Design Report
BMP	Best Management Practice
BPA	Backfilled Pits Area
CD	Consent Decree
COC	constituents of concern
CRSP	Colorado Rockfall Simulation Program
CSWPPP	Contractor Stormwater Pollution Prevention Plan
CSZ	construction support zone
cy	cubic yard
DMC	Dawn Mining Company LLC
EPA	U.S. Environmental Protection Agency
ESD	Explanation of Significant Difference
GDL	geocomposite drainage layer
gpm	gallons per minute
GSR	Green and Sustainable Remediation
HASP	Health and Safety Plan
HDPE	high density polyethylene
HSWRP	Hillside Waste Rock Pile
LiDAR	Light Detection and Ranging
LLDPE	linear low-density polyethylene
MA	mined area
MAA	mine affected area
mg/L	milligrams per liter
Newmont	Newmont USA Limited
NPDES	National Pollutant Discharge Elimination System
NRC	U.S. Nuclear Regulatory Commission
NUREG	U.S. Nuclear Regulatory Commission regulation
O&M	Operations and Maintenance
OM&M	Operations, Maintenance and Monitoring
pCi	picocuries
PCP	Pollution Control Pond
PVC	Polyvinyl Chloride
RA	Remedial Action
RAO	Remedial Action Objective

RD	Remedial Design
RI/FS	Remedial Investigation/Feasibility Study
ROD	Record of Decision
Site	Midnite Mine Superfund Site
SMP	Site-wide Monitoring Plan
SOW	statement of work
SWMP	Stormwater Management Plan
SWRP	South Waste Rock Pile
TDS	total dissolved solids
Tribe	Spokane Tribe of Indians
USBOM	Bureau of Mines
WCA	Waste Containment Area
WTP	water treatment plant

D1.0 INTRODUCTION

This appendix to the Midnite Mine Superfund Site Basis of Design Report (BODR) presents the detailed design information for the Mine Waste Excavation and Containment components at the Midnite Mine Superfund Site (Site). Mine Wastes include above-grade mine wastes, contaminated soil and sediment, mine drainage sediments, and mine road materials.

Mine waste generally will be excavated, transported, and placed in the Waste Containment Areas (WCA) in a continuous operation, without stockpiling excavated material prior to placement in Pit 4 and Pit 3. As such, Mine Waste Excavation and Containment are considered to be a single task and both are illustrated in the same section in the design drawings (Section 4 of Volume II) and described in this appendix.

The configuration of the waste containment upon completion of the Remedial Action (RA), as shown on the Section 4 Drawings in Volume II of the BODR, reflect the maximum waste storage that could be realized using the existing pits for storage. The storage available, assuming this configuration, exceeds the range of current estimates of waste volumes at the Site as discussed in Section D4.0.

The proposed backfilling sequence consists of completing placement of waste within the Pit 4 to its final configuration, followed by placement of waste within the Pit 3. As such, the Pit 4 WCA shown on the Drawings in Section 4 is considered “fixed”, whereas the final Pit 3 configuration has room to be expanded or reduced and as a result, may “float” depending on the actual volume of waste encountered during cleanup. Backfilling Pit 4 prior to placing waste in Pit 3 provides the most flexibility with respect to allowing additional waste material if significant changes in volume are identified in the future. Because the Pit 3 WCA is larger, more flexibility in waste storage volume storage can be accommodated relative to the Pit 4 without significantly altering the design. In addition, lessons learned during backfilling of the smaller Pit 4 can be used to improve designs, operations and procedures for the larger Pit 3 backfilling effort. Regrading of the Backfilled Pits Area (BPA) and Area 5 (the area between Pit 3 and Pit 4) will be performed concurrently with backfilling of Pit 3. Although backfilling both pits simultaneously was initially considered, it was not deemed feasible given space limitations and water management considerations discussed in Appendix E – Water Management Ponds.

Mine waste excavation and placement within the WCA is divided into three phases:

Phase 1 – Includes excavation and placement of mine wastes in Pit 4. Pit 4 will be dewatered and impacted Pit 4 water will be collected, conveyed and stored in Pit 3 during this Phase. Primary work objectives during Phase 1 include:

1) Preparation and construction of the site access road and construction support facilities described in Appendix B. The construction support zone (CSZ) and water treatment plant (WTP) footprints will require soil cleanup prior to construction of these facilities. This cleanup will include demolition of existing structures within the CSZ. These activities are known as the Early Works.

2) Initial processing of drain materials from the Hillside Waste Rock Pile (HSWRP).

3) Excavation of the Pit 4 Overburden Pile, with placement of this and other mine waste in the upper Eastern Drainage in Pit 4.

4) Excavation and consolidation of Ore/Protore Piles within Pit 4.

5) Excavation of parts of the South Waste Rock Pile (SWRP) where the South Pond will be constructed and parts of the HSWRP, with consolidation of excavated materials in Pit 4.

6) Construction of the South Pond in the SWRP. The South Pond will be used to store water during the Phase 2 construction activities when Pit 3 is taken off-line to begin remediation.

7) Construction of the new WTP at the end of Phase 1 prior to commencing with Phase 2 (providing the NPDES permitting approvals have been received and final design has been completed).

8) Capping and revegetating the final surface at Pit 4.

Phase 2 – Includes regrading the BPA and excavation and placement of mine wastes into Pit 3. Pit 3 will be dewatered and impacted water from the Site will be collected, conveyed, and stored in a lined, temporary storage pond located immediately south of Pit 3 on the SWRP (i.e., the South Pond discussed in Appendix E). Primary work objectives during Phase 2 include:

1) Removal all waste from, and meeting soil and sediment cleanup standards in the Western and Eastern Drainages so that surface water runoff in these drainage basins can be released to Blue Creek without retention and treatment at the WTP.
2) Regrading Area 5 (located between Pit 3 and Pit 4).
3) Demolition of the existing WTP and associated facilities provided the NPDES permitting is complete for the new facility, with placement of the demolition debris in Pit 3.
4) Construction of the West Pond; the West Pond will be used to store water during Phase 3 when the South Pond is taken off-line.
5) Demolition the other existing site facilities, with placement of the demolition debris in Pit 3.
6) Capping and revegetating areas where final grade has been established in the Pit 3 WCA, the BPA, and Area 5 to the extent practical near the end of Phase 2.

Phase 3 – Includes excavation and placement of remaining mine wastes in Pit 3, capping of remaining uncovered areas in the Pit 3 waste containment area, and revegetation remaining disturbed areas. Upon completion of Phase 2 work, the only significant volume of mine waste requiring excavation and containment will be that located in the Central Drainage in the vicinity of the South Pond and Pollution Control Pond (PCP). At that point, the South Pond can be decommissioned and replaced with a smaller retention pond in the Western Drainage (i.e., the West Pond) while Phase 3 of the remedial construction proceeds in the last remaining areas containing mine waste in the Central Drainage. Demolition and/or removal of temporary mobile and prefab support facilities in the CSZ and other areas of the Site will occur near the end of Phase 3.

Upon completion of Phase 3, it is anticipated that the only remaining material on Site requiring relocation and containment will be any impacted sediment that may have accumulated within the West Pond over its operational life and its liner system. The waste materials from the West Pond will be relocated to a separate waste containment cell located in the Contingency Storage Area in the upper portion of the Area 5/Pit 3 WCA as shown on Drawing 4-74. The conceptual design for the Contingency Storage Area is shown on Drawing 4-97. It is anticipated that the Contingency Storage Area would have capacity for at least 250,000 cubic yards of waste. The West Pond will not be decommissioned until flows at the Site have decreased to the point where

the available storage in the equalization ponds at the new WTP can handle the storage requirements of the Site.

Contingent Action Wastes - Waste materials may also include materials from outside the mined area (e.g., Blue Creek and Delta sediments) that may need to be excavated as part of the Contingent Action. This material would be placed in additional cells constructed in the Contingency Storage Area which would be constructed on top of the an existing cover, and these wastes will be completely encapsulated with a separate underliner and drainage system beneath the waste, and a composite cover system over the waste as shown on Drawing 4-97.

The remainder of this appendix contains the following information in subsections and attachments:

- Demonstration that the design will attain the Mine Waste Excavation and Containment Performance Standards identified in the Consent Decree (CD, EPA, 2011) (Section D2.0).
- Calculations, assumptions, and parameters for the design such as waste material volume estimates, materials management strategies, anticipated limits of excavations, and erosion and surface water controls (provided in attachments to this appendix).
- Excavation and grading plans for the above-grade mine wastes, contaminated soil and sediment, mine drainage sediments, and mine road materials at key points during RA construction (provided in Section 4 of the Drawings in Volume II).
- Strategy for managing surface water runoff in waste excavation areas. Details of surface water and sediment management at key points during the RA are included in Appendix F.
- Management of near-surface impacted groundwater in areas where it may impact surface water runoff quality.
- Considerations for Green and Sustainable Remediation (GSR) are in Section D13.0.
- Processes for verifying that cleanup levels have been achieved in the excavated areas are included in Appendix S.

D.1.1 DESIGN CHANGES

Final configuration of the WCA is shown on Drawings 4-17, 4-46, 4-55 and 4-74. This configuration differs somewhat from the conceptual design for the Selected Remedy as

presented on Figure 12-1 of the Midnite Mine Record of Decision (ROD, EPA, 2006). These major differences are discussed below:

BPA/Pit 3 Continuous Cover Design. The WCA configuration shown on Figure 12-1 of the ROD shows the BPA and the Pit 3 WCA as two discrete waste containment areas, with a gap in the cover system between the BPA and the Pit 3 WCA. This gap results in a thin strip of uncovered area in a valley formed at the low point between the covered waste areas in Pit 3 and the BPA where meteoric water may infiltrate into either the BPA, Pit 3, or both. The conceptual design shown on Figure 12-1 of the ROD included a surface water/shallow groundwater interceptor trench that was included in the design as a means to try to collect water that may infiltrate in this uncovered area.

Rather than allowing this water to infiltrate, and then try to intercept or collect is afterwards, the design as presented in Drawing 4-55 includes a continuous cover system between Pit 3 and the BPA, without this gap, and without the shallow surface water/groundwater interceptor trench. Continuous capping allows for interception and diversion of meteoric water from the BPA and Pit 3 WCA before it becomes groundwater and potentially contacts waste materials.

Elimination of Shallow Groundwater Interceptor Trench. The groundwater interception trench, located between the BPA and the Pit 3 WCA as shown on Figure 12-1 is superfluous given that a continuous cover system is being installed between the two areas to intercept and divert meteoric water before it becomes groundwater. In addition, the shallow interceptor system shown on Figure 12-1 lies almost on top of the topographic divide in the mine subwaste surface between areas draining to Pit 3 and areas draining to the BPA. Thus, although this trench could serve to intercept surface water if the WCA was shaped as shown in Section 12 of the ROD, it would be ineffective at collecting shallow groundwater since shallow groundwater flow has been shown to be topographically controlled (URS, 2002).

It also should also be noted that Figure 12-1 in the ROD was prepared using the pre-mine topography, rather than the topography prepared by the U.S. Bureau of Mine (USBOM), which includes modifications to the topography resulting from the mining excavations at the Site that dramatically changed the pre-mined topography. Evaluations of the USBOM topography, as well as evaluations of historic mining activity by Peters (1999), show that surficial materials and bedrock excavation occurred in this area to depths of 40 feet or more in the vicinity of the proposed surface water/groundwater interceptor trench, making it highly unlikely that any alluvium or alluvial groundwater will exist in this area after removal of mine waste.

Pit 4/Pit 3 Continuous Cover Design. Figure 12-1 in the ROD shows waste rock from Area 5 (the area between Pit 4 and Pit 3) being removed, leaving an approximately 10-acre area uncovered depression between Pit 3 and Pit 4 where surface water would be trapped and ponding would occur upgradient of Pit 3. Based upon the subwaste topography, it is clear that this area is the source of water for seeps expressing themselves in the north wall of Pit 3. Allowing ponding of surface water in this area will result in increased seepage into Pit 3 and should be avoided. As a result, the design presented in this BODR for the WCA (the Section 4 Drawings) includes grading the waste rock to provide positive drainage away from Area 5, and capping the surface to form a continuous cover over the backfilled Pit 4, Area 5, and Pit 3 to prevent infiltration of meteoric water. Capping of Area 5 is expected to reduce or eliminate the seeps in the north wall of Pit 3.

Excavation of the Adit Pit and Pit 2 West. The excavation and waste containment plans presented in the 60% BODR, 90% BODR, and this 100% BODR for the Adit Pit and Pit 2 West (the two small backfilled pits included as part of the BPA that are located in the Western Drainage) vary from that shown in the Preliminary (30%) BODR (MWH, 2012a). In the 30% BODR, the waste backfill in these two pits remained in place, with the waste surface graded to conform to the surrounding topography and capped to prevent infiltration of meteoric water. However, during the July 22, 2013 Technical Meeting held in Wellpinit, WA, EPA requested that the excavation and containment plan for these two pits be reevaluated and that additional consideration be given to removing the waste from the pits.

Upon additional consideration, it was determined that although removal of mine waste materials from these two pits would require excavation, transport, and consolidation of approximately 100,000 cubic yards of additional mine waste in the WCA, it does present a number of advantages. These include:

1. All waste material will be removed from the Western Drainage and consolidated in the Central Drainage. Removing all waste sources from the Western Drainage can be expected to reduce the potential for contaminant loading to groundwater in the Western Drainage while not measurably increasing the loading in the Central Drainage.
2. There is some indication that the Adit Pit was used as an ore load-out area during mining in the BPA (Peters, 1999). This suggests that some of the material in the partially backfilled Adit Pit may be higher activity material and possibly higher reactivity material that could be more effectively isolated within the Pit 3 backfill.

3. Although the groundwater well installed through the existing waste rock into the bottom of Pit 2 West (GW-52) West has always been dry, the mine waste subgrade topography indicates the pit bottom is not graded in a configuration that would be freely draining and has the potential to pond approximately 10-feet of water in the pit bottom. Excavation of all waste rock from Pit 2 West will allow for recontouring of the pit bottom to a configuration that will not have the potential to impound water in pit bottom. Although there is some potential for flushing of contaminants from the newly exposed pit walls in surface water runoff after excavation of the mine waste, should this occur it likely would only be for a relatively short period of time. In addition, it will be much easier to collect and treat this surface water runoff, if necessary for a short time, than if the impacted water occurred as groundwater (i.e., if the materials in Pit 2 West were left in place) over the long term.
4. Both the review of historical aerial photographs (Peters, 1999) and groundwater level measurements in the monitoring well installed in the Adit Pit (GW-55) indicate that groundwater has not been encountered in the Adit Pit either before or after backfilling. Thus, it appears that additional grading of the pit bottom will not be necessary to avoid ponding after removal of mine wastes. As was stated above for Pit 2 West, there is some potential for initial flushing of contaminants from the newly exposed pit walls in surface water runoff. If this does occur, it should be for a relatively short period of time and will be easily controlled.
5. If the waste was allowed to remain in the Adit Pit and Pit 2 West, the regraded waste surfaces would need to be very steep in order to conform to surrounding terrain. This would result in a very-steeply sloped composite cover system over both areas. The very-steeply sloped composite covers represent significant veneer slope stability and erosional stability challenges and would likely require long-term maintenance.
6. Although the removal of the mine waste from the Adit Pit and Pit 2 West was not explicitly discussed in the CD Statement of Work (SOW, EPA, 2011) (references are only made to regrading and capping the BPA waste in general), removal of waste from these two pits appears to conform more closely with the conceptual configuration of the WCA as shown in Figure 12-1 of the ROD (EPA, 2006).

Based upon these positive outcomes, it was decided to remove the mine waste from the Adit Pit and Pit 2 West and consolidate them within the Pit 4 WCA during Phase1 of the RA. Final soil

cleanup and verification of the Adit Pit and Pit 2 West will be completed in Phase 2 following cleanup of areas surrounding these pits. Wastes generated from final soil cleanup in these pits will be placed in Pit 3. This is presented in the Section 4 design Drawings.

It also was requested by the Spokane Tribe of Indians (the Tribe) representatives that consideration be given to removing all waste material from the BPA and consolidating them within the Pit 3. EPA indicated this change would be considered a major change to the Selected Remedy and that data would be necessary to support this proposed change. EPA also explained that submittal and approval of an Explanation of Significant Difference (ESD) would be necessary before designs could be prepared for removal of all waste from the BPA. During the meeting it was agreed there is some merit to removal of all the backfilled pit waste; however, it was decided to not pursue this alternative any further given the amount of time that would be necessary to:

- Prepare the ESD and obtain technical and public consensus and approval,
- Alter or completely stop the ongoing Remedial Design (RD) process, and
- Ultimately the delay the Midnite Mine RA construction schedule.

D2.0 PERFORMANCE STANDARDS

The Performance Standards presented herein are defined in the Consent Decree Statement of Work, and were developed to define attainment of the Remedial Action Objectives (RAOs) of the Selected Remedy. The Performance Standards include both general and specific standards applicable to the Selected Remedy work elements and associated work components. All of the Performance Standards, as well as a summary of where or how they are addressed in the RD, are summarized on Table 4-6 of the BODR. The Performance Standards applicable to the Mine Waste Excavation and Mine Waste Containment are listed in Tables D-1 and D-2 below. Performance Standards applicable to the remediation of the BPA are included as Table D-3. Since regrading of the BPA will be performed concurrently with Pit 3 backfilling during Phase 2 and the BPA surface cover will be contiguous with the Pit 3 surface cover, performance standards associated BPA remediation are addressed under Pit 3 waste consolidation performance standards, with the exception of performance standards for groundwater removal from the BPA.

Table D-1 — Performance Standards Applicable to Mine Waste Excavation

Performance Standard No. in CD SOW	Performance Standard	Comments
2.3 General Standards Applicable to All Work Elements and Work Components		
2.3.15 E.	Removals and other excavations conducted as part of the construction activities shall be performed in a manner that allows for proper drainage from the excavated area. Drainage from Work Areas that may have come into contact with contaminants shall be captured and conveyed to the water treatment plant for treatment. No drainage from Work Areas that may have come into contact with contaminants shall be allowed to infiltrate or discharge to natural drainages where water treatment collection and conveyance controls are not in place and operating.	To the extent practical, mine waste excavations will be completed beginning at the upstream (northern) end of the Western, Central, and Far Eastern Drainages and continued in a downstream direction. Excavation areas will be graded in a manner that contains surface water runoff from excavation areas wholly within the excavation areas, from where it will either drain by gravity, or be pumped to the storage pond and ultimately the WTP for treatment. Additional details of excavation procedures to be used during mine waste removal are presented in Technical Specification Section 02205 – Mine Waste Excavation and Disposal.
2.3.15 H.	To the extent practicable, construction activities shall be conducted in a manner that does not result in the re-contamination of areas already remediated or contamination of areas that were previously uncontaminated. Any such re-contaminated or newly contaminated areas shall be addressed by the Settling Defendants in a manner that is subject to the review and approval of EPA.	The proposed phasing of construction activities will avoid the recontamination of remediated areas. Contamination of previously uncontaminated areas will be avoided to the maximum extent practical, and stockpiling of contaminated materials in uncontaminated areas will be avoided. If routing of construction traffic, or other operations that may potentially result in re-contamination of remediated areas or contamination of previously uncontaminated areas is unavoidable, these operations will be limited, the EPA will be notified, and these areas will be addressed in a manner that is subject to the review and approval of the EPA. Requirements for the construction contractor to execute the work in a manner that does not result in the re-contamination of areas already remediated or contamination of areas that were previously uncontaminated is provided in Technical Specification Section 02205 – Mine Waste Excavation and Disposal.
2.3.18	Best Management Practices (BMPs) shall be used as specified below during all construction activities to minimize the transport of disturbed material by water, wind erosion or vehicles. The Settling Defendants shall develop a catalog of BMPs that shall be used at the Site and shall identify the primary activities requiring those BMPs. The BMP catalog shall be comprehensive and is subject to the review and approval of EPA. The minimum BMPs that must be contained in the BMP catalog are presented below. The Settling Defendants shall include these BMPs in the BMP catalog along with additional BMPs that may be necessary to complete the Work. A Storm	The Master SWMP included in Appendix O describes the over-arching framework for how stormwater and surface water will be managed to limit the release of sediment, pollutants, and deleterious debris to downstream areas during and following the RAs. The Master SWMP is the foundation document that provides the catalog of BMPs that will be applied to reduce the adverse impacts of stormwater. The RA Contractor will be required to prepare a Construction Stormwater Pollution Prevention Plan (CSWPPP) that presents the stormwater management protocol and procedures that are specific to the phased construction activities. The RA Contractor's CSWPPP will reference the Master SWMP for general stormwater management practices and will identify

Table D-1 — Performance Standards Applicable to Mine Waste Excavation
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Performance Standard No. in CD SOW	Performance Standard	Comments
	Water Management Plan (SWMP) shall be prepared which contains the BMP catalog and identifies BMPs and specific sediment control measures to be employed before, during, and after construction.	the BMPs that are applicable to the scheduled construction activities.
2.3.18. A	The Work shall be conducted in a manner that minimizes the generation of fugitive dust. If the application of water or other dust suppressants to Work Areas is used to control the generation and migration of fugitive dust, such application of dust suppressants shall comply with the following requirements:	It is anticipated that dust suppression will be required for this work and primarily will consist of dust suppressant watering. “Free” water will not be allowed to run off as a result of this activity. Dust suppressant additives may be added to permanent access roads or haul roads, subject to prior EPA approval. Technical specification Section 01560 – Temporary Environmental Controls describes dust suppression methods and procedures and will be subject to EPA review and approval.
2.3.18. A.i	Dust suppressants containing brine, or other materials that are harmful to surface water or vegetation shall not be used. Subject to EPA approval, water treated to meet the WTP discharge limits may be used for dust suppression in the Work Area, provided it will not result in releases to surface water or adversely affect worker health and safety.	See response top 2.3.18. A above. The design assumes that water from the WTP, which has elevated levels of TDS and sulfates, will be used for dust control in the Mine Area on contaminated materials. It is assumed that this water would not be used on areas that are outside of the Mine Area or have been cleaned up to applicable standards.
2.3.18. A.ii	Application of dust suppressants shall be performed in a manner that minimizes surface water runoff, over spray of chemical suppressants into surface water bodies, wetlands or other sensitive habitats, and/or generation of muddy conditions.	See response top 2.3.18. A above.
2.3.18. B	At a minimum, the following BMPs shall be used to minimize the transport of sediment from Work Areas:	BMPs to minimize sediment transport from the Work Area are identified in the Master SWMP for this work. The Master SWMP is provided in Appendix O.
2.3.18 B.i	Staging areas, accumulation areas and other areas where Work is to be performed on exposed slopes shall be isolated with appropriate BMPs to minimize transport of potentially contaminated sediments from the Work Areas by surface water runoff.	The Master SWMP in Appendix O contains the BMP catalog, including BMPs to minimize the transport of sediments. As described in 2.3.15.E above, excavations will be conducted beginning with upstream areas within each drainage and working in a downstream direction, with the working excavation areas being shaped to retain surface water runoff. In areas where this is not possible, other BMPs will be utilized to minimize the transport of potentially contaminated sediments from the work areas by surface water runoff.
2.3.18 B.iii	Work that occurs within surface water bodies shall be performed in accordance with the requirements of the SWMP in the approved Remedial Action Work Plan to minimize sediment migration from the Work Area and mitigate damage to existing vegetation. All such Work shall be performed in a manner that limits harm to wetlands and surface water. In	The Master SWMP in Appendix O contains the BMP catalog, including BMPs to minimize the transport of sediments and to limit harm to wetlands and surface water during the RA. With a few specific exceptions (i.e., sediment cleanup within drainages) this work will not occur within surface water bodies. To the maximum extent practical, sediment cleanup within drainages will be conducted within drier parts of the

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	addition, the Work shall be performed in a manner that minimizes the release of sediments beyond the Work Area. BMPs shall be employed and refined as necessary to minimize the release of sediment.	year (summer and early autumn) to avoid unnecessary impacts to surface water bodies.
2.3.18 B.iv	Any dewatering or diversion of surface water and groundwater shall be performed in a manner that minimizes the release of sediments to the extent practicable beyond the Work Area and limits harm to wetlands and surface water.	See response to 2.3.18 B.iii
2.4.2.3 Mine Waste Excavation Work Component		
A. Mine Waste Excavation		
2.4.2.3.2 A.i.	Above-Grade Mine Waste Excavation - Mine Wastes located above the premining topographic surface within the MA with the exception of mine wastes currently located in the Backfilled Pit Area (BPA) shall be excavated. All of the above materials located in the MA that exceed the cleanup levels identified in Table 4-1 shall be excavated for consolidation and containment in Pits 3 and 4.	Above-grade mine wastes located above the pre-mining topographic surface are shown on Drawing 4-1 and shall be excavated to the pre-mining topography as shown on Drawings 4-2, 4-24, and 4-51 and relocated in the Pit 3 and Pit 4 backfill areas. The Pit 3 and Pit 4 Mine Waste Containment Areas will be contiguous and continuously capped. As such, Area 5 between Pits 3 and Pit 4 will be regraded and capped in-place, as shown on Drawing 4-46, as opposed to being excavated and placed in either Pit 3 or Pit 4.
2.4.2.3.2 A.ii.	Contaminated Soils and Sediments Excavation - Contaminated soils (impacted by roads or other areas of mine waste) and sediments located in the MA and MAA that exhibit contaminant concentrations above the cleanup levels in BODR Tables 4-1 and 4-2, shall be excavated for consolidation and containment in Pits 3 and 4.	Delineations of extents and volume estimates for contaminated soil cleanup within the MA and MAA are based on data and information provided in the *RI Report* (EPA, 2005) and *Mine Waste Investigations Report* (MGC, 2011a).The estimated cleanup limits shown in the Section 4 of the Drawings are based upon these delineations. As indicated on the Drawings, the actual extent of soil contamination and cleanup will be determined in the field using procedures defined in the "Analytical Support and Verification Plan for Remediation of Surface Materials and Sediments" is included in Appendix S.
2.4.2.3.2 A.iii.	Mine Drainage Sediments Excavation - Mine Drainage Sediments located in drainages downstream of the MA in the MAA have been impacted by the release of contaminated materials from the MA. Mine Drainage Sediments that exhibit contaminant concentrations above the cleanup levels presented in Table 4-2 shall be excavated for consolidation and containment in Pits 3 and 4. The extent of contaminated sediments requiring removal in the mine drainages shall be determined during RD.	See Response to 2.4.2.3.2 A.ii, above.
2.4.2.3.2 A.iv	Road Materials Excavation – Mine wastes used for the construction of roads and any	See Response to 2.4.2.3.2 A.ii, above.

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.	soils and sediments below, adjacent to, and downstream of the roads that exceed the cleanup levels presented in BODR Table 4-2 shall be excavated for consolidation and containment in Pits 3 and 4. The extent of contaminated materials requiring excavation shall be determined during RD	
2.4.2.3.2 A.v.	Soil/sediment sampling shall be conducted following removals to ensure that remaining soils and sediments meet cleanup levels identified in Tables 4-1 and 4-2. The sampling design and frequency shall be developed using methodology that conforms with EPA guidance for the development of sampling and analysis plans and quality assurance project plans.	See Response to 2.4.2.3.2 A.ii, above.
2.4.2.3.2 A.vi.	A layer of suitable soil or soil amendments, as determined during RD, shall be placed over areas cleared of mine waste. Such areas shall be graded and re-vegetated to minimize erosion and ARD formation and to channel water away from waste containment areas.	Areas cleared of mine waste will be graded to conform to the pre-mining topography as shown on Drawings 4-2, 4-24, and 4-51 to the extent practical. In areas cleared of mine waste where subsoil excavation and removal is required, one foot of clean soil from an approved borrow source or soil amendments will be placed to enhance revegetation and minimize erosion.
B. Surface Water and Stormwater Management and Controls During Excavation		
2.4.2.3.2 B.i.	During the excavation of contaminated materials, surface water and stormwater BMPs shall be applied to prevent, to the extent practicable, sediment transport and the contact of clean surface water and stormwater with contaminated materials.	Appendix F entitled “Surface Water and Sediment Controls” and Section 6 of the Drawings in Volume II describe the Surface Water and Sediment Controls which will be used to shed clean water away from contaminated areas during various stages of the RA. The Master SWMP is contained in Appendix O and includes a BMP catalog. The SWMP defines the requirements for inspecting, maintaining, and repairing sedimentation controls and maintaining BMPs throughout construction.
2.4.2.3.2 B.ii.	To the extent practicable, clean water coming into contact with contaminated materials in the excavation areas that results in surface water concentrations exceeding the surface water cleanup levels identified in Table 4-3 shall be collected and conveyed to the WTP for treatment.	To the extent practicable, the mine waste excavations will occur in a downhill direction, and be bermed and contoured such that such that all surface water that enters the excavations (and potentially contacts mine wastes) will be captured in the excavation. This water will either gravity drain or be pumped to the temporary storage impoundments pending treatment at the operating WTP. These details are included in Appendix D – Mine Waste Excavations and Containment and Appendix F – Surface Water and Sediment Controls.
2.4.2.3.2 B.iii.	Sediments captured by surface water and stormwater controls shall be contained and removed to an approved location designed to prevent redistribution of the sediments to the surrounding environment. The disposition of	Sediments will be captured during construction in a variety of temporary surface water and sediment controls structures discussed in Appendix F and BMPs identified in Appendix O (Master SWMP). The process for verifying Site COC concentrations in

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	the sediments shall be determined by sampling the sediments at a frequency and for analytes determined during RD.	sediments is included in the Analytical Support and Verification Plan for Remediation of Surface Materials and Sediments contained in Appendix S. Sediment determined to be contaminated (or assumed to be contaminated based on the location of the BMP) will be incorporated into the waste containment areas in Pits 3 and 4. Captured sediments that are determined to be clean may be incorporated into soil cover layers as part of remedial construction.
2.4.2.3.2 B.iv.	Surface water and stormwater controls and water collection and conveyance systems shall remain in place and be monitored for effectiveness until such a time as all contaminated materials requiring excavation have been removed for consolidation and containment in Pits 3 and 4.	The surface water and sediment controls (described in Appendix F), and water collection and conveyance systems (described in Appendix J) will be constructed, operated and removed according to a phased construction approach as described in Appendix A – General Design Information and in this Appendix D. These temporary structures and systems will remain in place until permanent structures/systems are built and water in the remediated areas can be shed to the natural drainages down gradient of the Site. The Operations Maintenance and Monitoring Plan (OM&M Plan) in Appendix P defines O&M requirements for the surface and stormwater controls during the RA activities. In addition, surface water down gradient of the Site will be monitored in accordance with the Site-Wide Monitoring Plan (SMP), contained in Appendix Q, to evaluate the effectiveness of these engineering controls during the RA.
2.4.2.3.2 B.v.	The Settling Defendants shall develop a monitoring program to ensure that the concentrations of contaminants in surface water leaving the MA are below those listed in Table 4-3. If concentrations are greater than those listed in Table 4-3, the water shall be collected and conveyed to the water treatment plant for treatment.	To the extent practicable, all surface water that contacts mining wastes within the MA will continue to be captured during the RA activities and conveyed to the operating WTP. These details are described in this Appendix D – Mine Waste Excavation and Containment, Appendix E – Water Management Ponds, and Appendix F – Surface Water and Sediment Controls. However, as noted in the ROD, achievement of the surface water cleanup levels down gradient of the MA will require a period for natural attenuation to occur after the remedy is completed. Therefore, the design does not include provisions to capture and treat surface water down gradient of the MA. The SMP in Appendix Q defines the monitoring program that will be implemented both during and following the RA to evaluate contaminant concentrations in surface water down gradient of the MA. The SMP defines the action levels that will be used during the RA to evaluate if mine-related contaminants are being released to surface water as a result of the RA activities. The SMP also describes how surface water will be monitored following the RA

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2.4.2.3.2 B.vi.	If, during the course of excavation, the surface water and stormwater BMPs in the BMP Catalog are found to be insufficient to address surface water and stormwater management issues, the Settling Defendants shall develop and implement new BMPs, subject to EPA review and approval.	for comparison with the cleanup levels listed on Table 4-3. As described in the Master SWMP included in Appendix O, the Project Engineer will perform periodic inspections and monitoring to confirm that the BMPs are adequate and functioning as intended, or to determine if additional BMPs are necessary. If necessary, the Project Engineer will immediately initiate actions to correct existing BMPs or develop and implement new BMPs.
C. Excavated Materials Staging/Stockpiling		
2.4.2.3.2 C. i.	If it is determined during design that staging of excavated materials prior to their consolidation and containment is necessary, a Staging/Temporary Stockpile Plan shall be developed and included in the RD.	A Staging/Temporary Stockpiling Plan is included as Appendix R. Generally, staging and stockpiling will not be necessary as most of mine waste material will be directly loaded and hauled to its final destination in the waste containment areas (see Technical Specification 02205 – Mine Waste Excavation and Disposal; Sections 3.3 and 3.4). The only material anticipated to require stockpiling prior to placement in the containment area are 1) the pit-bottom sediments, which will be stockpiled on the waste rock piles, 2) material from the topsoil stockpile at the WTP site, 3) excavation spoils from the construction of the groundwater control system, and 4) the drain rock that will be processed from the HSWRP, which will occur in Area 5.
2.4.2.3.2 C. ii.	The Staging/Temporary Stockpile Plan shall include a list of BMPs that complies with applicable worker protection requirements. In addition, the BMPs shall ensure, to the extent practicable, that staged/stockpiled materials are isolated from contact with surface water and stormwater and that staging/stockpiling processes do not result in the generation of ARD and/or conditions that could lead to the migration of contaminants to the surrounding environment.	A Staging/Temporary Stockpiling Plan is provided in Appendix R. Temporary stockpiling of contaminated materials is designed to occur within existing mine waste areas (i.e., all runoff from the stockpiled materials will be captured and treated); therefore BMPs (other than those described in the Master SWMP) will not be needed. Engineering controls to capture stormwater and surface water in the mine waste areas are described in Appendix F (Surface Water and Sediment Controls) and are depicted in the Section 6 Drawings included in Volume II. Applicable worker protection requirements for construction activities are included in Appendix L – RA Health and Safety Plan (HASP).

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A. Temporary Facilities during Construction Activities		
2.4.2.4.2 A.i.	During performance of the Pits 3 and 4 Component of Work, temporary facilities, such as covers, runoff controls, temporary sumps, and water capture and removal systems, shall be provided, as determined in the SWMP and RD. Water requiring treatment shall be conveyed as soon as practicable to the WTP for storage and treatment.	Appendix F entitled "Surface Water and Sediment Controls" contains text, calculations and references Section 6 of the Drawings in Volume II. Appendix E entitled "Water Management Ponds" and references Section 5 of the Drawings in Volume II. These documents and drawings illustrate how surface water and impacted site water will be managed upon completion of each major phase of construction. The Master SWMP is contained in Appendix O. As required by the SOW in the CD, this SWMP will be updated on an annual basis, at a minimum, and will describe the intermediate phases and temporary facilities to be employed to capture and convey water to the WTP, as well as diversion of clean water around work areas, as construction progresses.
B. Groundwater Intrusion into Pits 3 and 4		
2.4.2.4.2 B.i.	Groundwater adjacent to each pit shall be collected and diverted away from the pits or blocked from flowing into the pits, as practicable, by methods determined during RD.	The primary mechanism proposed for diverting groundwater from the pits is to provide a continuous surface cover system over the majority of the contributing areas (to Pit 4, Pit 3, and the BPA) where surface infiltration provides a recharge source for groundwater reporting to the pits. This cap will extend beyond the pit crests and include areas that currently infiltrate and contribute to pit seepage (e.g., Area 5). Water from the surface cover system that historically has reported to the pits will be collected in the surface water diversions and routed around the pit areas.
2.4.2.4.2 B.ii.	To the degree practicable, clean groundwater shall be segregated from contaminated waters to minimize water volumes requiring treatment.	See response to 2.4.2.4.2 B.i. above. In addition, the construction specifications included as Appendix K (i.e., Section 02200 – Earthwork) require slush grouting of sections of perimeter collection channel that are excavated into bedrock and contain open fractures. The construction specifications also require shotcrete lining of high permeability weathered bedrock or other high permeability sections of perimeter channel excavations that cannot be treated by slush grouting.
2.4.2.4.2 B.iii.	To the degree practicable, groundwater entering the pits shall not contact reactive mine waste or waste capable of causing groundwater contamination.	An underdrain system constructed of non-reactive rock will be installed in the bottoms of Pits 3 and 4 to collect groundwater before it contacts reactive mine waste backfill in the pits, as shown in Section 4 of the Drawings in Volume II. The pit bottom drainage system will be separated from overlying reactive mine waste backfill by a synthetic geomembrane. In addition, a 20-foot thick layer of less reactive waste rock will be placed above the geomembrane to provide additional separation between pit

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		groundwater and more reactive mine waste. The drain system will be extended up the pit walls, as shown on Drawings 4-15 and 4-39, in areas where pit wall seepage is occurring in order to intercept these seeps and convey them to the underdrain system before they contact reactive mine waste in the backfill. A separate Waste Rock Dewatering System will be installed above the geomembrane liner to collect water that infiltrates through the overlying waste rock and collects on the geomembrane liner.
2.4.2.4.2 B.iv.	Contaminated groundwater shall be captured and treated in the WTP.	This Appendix D - Mine Waste Excavation and Containment contains text, calculations, and references drawings in Volume II to describe/illustrate the contaminated groundwater pump-back system, which includes wells installed above and below the lower liner.
C. Surface Water Management - Pits 3 and 4		
2.4.2.4.2 C.i.	Surface water and stormwater management shall be conducted in accordance with the SWMP. Surface water and stormwater management BMPs shall be developed and constructed to divert clean surface water and stormwater away from the pits during construction. Surface water and stormwater that enters the pits shall be captured and conveyed to the WTP. Surface water and stormwater BMPs constructed shall remain in place and be monitored for effectiveness until consolidation and containment of excavated materials in the pits is completed and permanent surface water and stormwater management facilities are in place and functional.	Appendix F entitled “Surface Water and Sediment Controls” contains text, calculations and references drawings in Volume II that show how water will be captured and routed around construction activities at completion of the three major phases of construction. Per the SOW in the CD, the SWMP in Appendix O will be updated on an annual basis to reflect the most current construction status. The SWMP includes a BMP catalog, and will describe the temporary facilities to be employed to capture and convey impacted water to the WTP, and clean water around the work areas, at intermediate phases of construction.
2.4.2.4.2 C.ii.	Facilities shall be constructed to divert clean surface water away from the pits. The diversion facilities shall be designed using standard engineering techniques for capacity and erosional stability to convey the 100-year, 24 hour storm event in a stable manner and to withstand a 500-year, 24 hour storm event.	Clean surface water will be diverted away from the pits via a series of diversion channels and the grading of the final cover system. Appendix F (Stormwater and Surface Water Controls) includes the design information for the diversion channels and the phased stormwater controls are shown on the Section 6 Drawings (located in Volume II). The conveyance capacity of these facilities has been designed for the 500-year, 24-hour storm event. Erosional stability of the cover system has been designed for the 100-year, 24-hour event as described in this Appendix D (Mine Waste Excavation and Containment). The construction specifications in Appendix K require slush grouting of sections of perimeter collection channel that are excavated into bedrock and contain open fractures.

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		The construction specifications also require shotcrete lining of high permeability weathered bedrock or other high-permeability sections of perimeter channel excavations that cannot be treated by slush grouting
2.4.2.4.2 C.iii.	To the degree practicable, clean surface water shall be segregated from contaminated water to minimize water volumes requiring treatment.	The RA will be performed in phases such that surface water from remediated areas can be shed away from the active excavation areas as soon as practicable. Surface water will be segregated by site grading to manage and direct drainage, and using permanent and temporary drainage channels to divert clean surface water away from the active construction areas. Appendix D (Mine Waste Excavation and Containment) describes the phased excavation activities and the site topography at the end of each Phase is depicted on the Section 1 Drawings (located in Volume II). Appendix F (Stormwater and Surface Water Controls) includes the design information for the diversion channels and the phased stormwater controls are shown on the Section 6 Drawings.
2.4.2.4.2 C.iv.	Contaminated surface water shall be captured and treated in the WTP.	Excavation activities will be performed such that drainage patterns are maintained to shed potentially contaminated surface water to diversion channels and temporary impoundments, and ultimately to the operating WTP. This Appendix D (Mine Waste Excavation and Containment) describes the excavation activities. Appendix F - Surface Water and Sediment Controls contains text, calculations, and references drawings in Volume II that show the temporary engineering controls (e.g., temporary drainage channels) that will be constructed to capture and convey contaminated water to the Water Management Ponds (Appendix E). Water from these ponds will be conveyed to the WTP for treatment.
D. Pits 3 and 4 Preparation and Mine Waste Excavation		
2.4.2.4.2 D.i.	Each pit shall be dewatered prior to any mine waste emplacement.	Pits 3 and 4 will be dewatered prior to construction activities as described in Sections 6.2 and 7.2 of this appendix and shown on the Remedial Action Schedule (Appendix X).
2.4.2.4.2 D.ii.	Water removed during such dewatering shall be conveyed to and treated at the WTP.	Water removed during dewatering of Pits 3 and 4 shall be conveyed to the WTP (either via the intermediate storage pond or directly to the WTP, depending on the WTP operating requirements) for treatment.
2.4.2.4.2 D.iii.	To the extent practicable, water shall be kept from accumulating in the pits during and after construction of the containment system. If water accumulates in the pits during construction, the water shall be	The underdrain sump/dewatering system shown on Drawings 4-12, 4-15, 4-37, 4-38, and 4-39 will be installed upon completion of pit-bottom grading and preparation and will remain operational through backfilling and completion of RA construction.

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	collected and conveyed for treatment at the WTP.	Likewise, the mine waste dewatering system shown on Drawings 4-13, 4-16, 4-40, and 4-41 will be installed upon completion of the geomembrane liner and will remain operational from that point forward. Duplicate dewatering risers, including pumps and piping are proposed to avoid shutdowns in the dewatering system due to maintenance or mechanical failure.
2.4.2.4.2 D.iv.	Existing sediments which have collected at the bottom of the pits shall be removed prior to preparation of the pit floors. Such removed sediments shall be staged for subsequent re-emplacement in the pits. The need and process for dewatering of the sediments and conveyance and treatment of water from the sediments shall be determined during RD.	Pit-bottom sediments shall be removed as described in Sections 6.3 and 7.3 of this appendix and stockpiled for replacement in the pits as described in an approved Staging/Temporary Stockpiling Plan. The Staging/Temporary Stockpiling Plan is included as Appendix R.
2.4.2.4.2 D.v.	As determined during RD, pit walls shall be prepared to ensure worker health and safety during construction.	Pit rockfall protection measures are described in Sections 6.1 and 7.1 of this appendix.
2.4.2.4.2 D.vi.	The pit surfaces shall be contoured to efficiently drain water entering the pits to low points located below the drainage layer. The need to perform additional excavation of the current pit bottoms to ensure gravity drainage to the low points shall be determined during RD.	Pit bottom surface preparation and grading is discussed in Sections 6.3 and 7.3 of this appendix, and shown on Drawings 4-12 and 4-37. Pit 4 will require recontouring and excavation of a sump so that gravity flow in the pit bottom can be accomplished. Pit 3 will require some cleanup, but in general water in Pit 3 gravity flows to the last mined area (drop cut) which forms the low point of the pit.
E. Drainage Layer – Pits 3 and 4		
2.4.2.4.2 E.i.	A continuous drainage layer of non-reactive rock or other suitable material, approved by EPA, shall be constructed overlying the base of the pit and extending up the sides of each pit as necessary to intercept groundwater entering the pit.	Pit underdrain systems are described in Sections 6.4 and 7.4 of this appendix and shown on Drawings 4-12, 4-15, 4-37, 4-38, and 4-39.
2.4.2.4.2 E.ii.	If during RD suitable material for the drainage layer can be found on site, EPA may approve the use of such materials, following consultation with the Tribe.	Results of investigations presented in the *Mine Waste Investigations Report* (MGC, 2011a) and the *Addendum to the Mine Waste Investigations Report* (WME, 2012) indicate that suitable material for the drainage layer can be processed from the HSWRP. It is anticipated that this material will be used for construction of the drainage layer.
2.4.2.4.2 E.iii.	The drainage layers shall extend vertically along the side walls of each pit to elevations determined during RD, to keep water entering the pits from contacting mine waste and to effectively channel water to the pit bottoms.	Locations of pit wall seeps were mapped as part of investigations for the *Geologic Investigations of Pits and Assessment of Pit Sediments Design Investigation Report* (MGC, 2011b) and the *Pit Seep Monitoring Report for Pit 3 and Pit 4* (Plumley and Assoc., 2012). These seeps are shown on Drawing 4-38 and the drain configuration shown is designed to intercept these seeps and convey them to the pit-bottom sump without contacting reactive mine waste

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		materials in the pit backfill.
2.4.2.4.2 E.iv.	The drainage layers shall be designed and constructed in a manner to provide efficient drainage of water along the sidewalls and bottoms of each pit.	See response to 2.4.2.4.2 E.iii, above.
2.4.2.4.2 E.v.	Water entering the pits and transported through the drainage layers shall be collected in a sump or sumps placed at the bottom of the pits. The water collection sump(s) shall be constructed in the lowest portion of the pit bottom and gravity drainage from the pit walls and pit bottom shall be used to direct water to the sump. The design of such sump(s) may require additional excavation into the pit bottom to ensure gravity drainage.	Pit bottom grading, drainage sumps, and drain placement are discussed in Sections 6.3, 6.4, 7.3, and 7.4 of this appendix and shown in Section 4 of the Drawings (Volume II).
2.4.2.4.2 E.vi.	The installation of the drainage layers along the pit walls and bottoms shall be coordinated with the emplacement of mine wastes into the pits and the sub-waste liners, described below.	The sequence for drain installation and waste placement are discussed in appendices D and X (RA Schedule), and shown on Section 4 of the Drawings.
2.4.2.4.2 E.vii.	Water levels in the sumps shall be maintained at elevations determined during RD which minimize hydraulic head, scaling, and fouling, and prevent water contact with the mine waste. Water collected in the sumps shall be conveyed by pumping or gravity for treatment at the WTP.	The anticipated range of operating water levels within the underdrain (pit bottom) and waste rock dewatering (overliner) sumps are shown on Drawings 4-79 and 4-82, respectively. The proposed range of water level fluctuations will ensure that the water level will remain within coarse drain rock of the sump backfill, thus avoiding water level fluctuations over the greater pit floor and liner surfaces, while avoiding drawing the water levels down to the elevation of the screened sections of dewatering risers. The pumping levels presented in the 100% BODR are consistent with these water level requirements. The pumping levels are set below the bottoms of the pits (i.e., in the sumps) in order to maintain a groundwater flow direction from surrounding areas toward the pits, prevent contact of groundwater with the mine waste, and minimize the potential for scaling and fouling of the pit dewatering wells.
F. Sub-waste Liner – Pits 3 and 4		
2.4.2.4.2 F.i.	A sub-waste liner shall be constructed in each pit below and adjacent to the emplaced mine wastes in locations and to vertical elevations determined during remedial design.	Sub-waste liners will be placed between the drain systems and overlying mine waste in Pit 3 and Pit 4 as shown on Drawings 4-15 and 4-41.
2.4.2.4.2 F.ii.	The sub-waste liners shall be placed between the mine wastes and the drainage layers: additional materials shall be placed, as necessary, to protect the integrity of the sub-waste liners, as	Sub-waste liners will be placed between the drain systems and overlying mine waste in Pit 3 and Pit 4 as shown in Setion 4 of the Drawings. The liner section will include a geomembrane cushion (geofabric layer) under the geomembrane, and an

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	determined during RD.	overliner cushion layer of fine-grained soil as discussed in Attachment D-1 to Appendix D. The plans to minimize fluctuations in water levels in an attempt to minimize scaling and fouling while preventing direct contact with the mine waste rock will be described in the OM&M Plan for Water Management in Appendix P.
2.4.2.4.2 F.iii.	The sub-waste liners shall be constructed of a synthetic material determined during RD.	It is proposed that the sub-waste liner be constructed of High-Density Polyethylene geomembrane as discussed in Sections 6.5 and 7.6 of Appendix D.
2.4.2.4.2 F.iv.	The sub-waste liners shall be designed to effectively isolate the mine waste and minimize the passage of both water and mine waste particles between the adjacent drainage layers and the emplaced mine wastes.	See responses to 2.4.2.4.2 F.i, 2.4.2.4.2 F.ii, and 2.4.2.4.2 F.iii, above.
2.4.2.4.2 F.v.	The sub-waste liners shall be constructed in such a way as to transmit water collected on the liners to sump(s) located above the liner at its low point. The sumps shall be constructed in such a manner that water from the mine waste materials shall concentrate in the sump area using gravity drainage.	Proposed grading for the sub-waste liners are shown on Drawings 4-13 and 4-40. This grading provides for gravity drainage of water on the liner surface toward sumps, which will be dewatered by pumping from waste rock dewatering risers located within the sumps.
G. Pits 3 and 4 Mine Waste Consolidation		
2.4.2.4.2 G.i.	All materials excavated as part of the Mine Waste Excavation Component of Work and existing sediments from the pit bottoms shall be consolidated in the pits.	Materials excavated during Mine Waste Excavation will be consolidated in the pits as described in Section D4.0 (Material Balance) of this appendix and shown in Section 4 of the Drawings.
2.4.2.4.2 G.ii.	Mine waste shall be emplaced in lifts above the sub-waste liner and any protective layer determined necessary during RD. Placement shall minimize settling.	It is proposed that Mine Waste be placed in 10-foot maximum horizontal loose lifts over the protective overliner cushion layer.
2.4.2.4.2 G.iii.	The emplacement of mine waste lifts shall be coordinated with the installation of the adjacent sub-waste liner and drainage layer along the pit walls and bottoms, as determined during RD.	Where required, drainage layer placement along the pit walls will occur concurrently with Mine Waste placement as shown on Drawing 4-90.
2.4.2.4.2 G.iv.	Mine waste emplaced in the pits shall be compacted to design specifications during backfilling.	It is proposed that Mine Waste be placed by dumping from trucks and spreading in 10-foot maximum horizontal loose lifts as discussed in Sections D6.7, D7.8 and D8.1 of this appendix.
2.4.2.4.2 G.v.	Emplacement of mine waste in the pits shall ensure efficient drainage to sumps constructed above the sub-waste liner.	See responses to 2.4.2.4.2 G.ii and 2.4.2.4.2 G.iv above.
2.4.2.4.2 G.vi.	Water levels in the sumps above the sub-waste liner shall be maintained at an elevation determined during RD, which	See response to 2.4.2.4.2 E.vii above.

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Performance Standard No. in CD SOW	Performance Standard	Comments
	minimizes hydraulic head, scaling, fouling and infiltration through the sub-waste liner.	
2.4.2.4.2 G.vii.	Water collected in such sumps shall be conveyed by pumping or gravity for treatment at the WTP.	A typical Waste Dewatering Sump Detail is shown on Drawing 4-82. Water collected in these sumps will be pumped to the WTP through dewatering risers that will be raised concurrently with the rise of the waste backfill surface.
2.4.2.4.2 G.viii.	As determined during RD, the least reactive (ARD generating) mine waste materials shall be placed in portions of the pits below the surrounding groundwater level.	The first 20 feet of waste placed above the sub-waste liners will be lower-activity and low ARD potential waste as illustrated in Section 4 of the Drawings.
2.4.2.4.2 G.ix.	As determined during RD, materials with high radon-generating ability, such as ore and proto-ore, shall be placed in the pits so as to minimize radon flux at the top of the backfill and below the cover.	As shown in Section 4 of the Drawings, Ore, Protore, or other materials identified as having high radon-generating ability will be excluded from the 20 feet of waste immediately underlying the cover in the containment areas.
2.4.2.4.2 G.x.	The mine waste materials shall be mounded above the top elevation of each pit and sloped to support a cover and surface water management system designed to maximize runoff and minimize infiltration into the mine wastes, while preserving slope stability.	The top surfaces of the waste containment areas will be graded as shown in Section 4 of the Drawings to provide positive drainage of surface water from the cover surface. Erosional and slope stability calculations for the proposed cover surface are provided in Attachments D-5 and D-6 to this appendix.
H. Pits 3 and 4 Cover Construction		
2.4.2.4.2 H.i.	A cover made of geologic material and a synthetic liner shall be constructed over the emplaced mine waste in each pit in such a way as to permanently meet the ROD cleanup standards for soil and radon flux and to minimize the infiltration of water into the pits.	A cover system consisting of a synthetic linear low-density polyethylene (LLDPE) geomembrane overlain by a soil cover and 0.5 feet of growth media (soil cover or topsoil material, as shown on Drawing 4-83). The soil cover thickness will be a minimum of 3 feet thick and will be obtained from the Rhoads Property Borrow area. On sloped areas steeper than 6.6:1 (horizontal:vertical) a geocomposite drainage layer (GDL) will be included between the geomembrane and soil cover layers in order to reduce the potential for positive pore pressure and cover instability at the geomembrane soil interface.
2.4.2.4.2 H.ii.	Cover specifications shall be determined during RD and shall ensure that the thickness of the geologic materials alone shall be sufficient to limit the radon flux rate to less than 20 pCi/m^2/sec as required in Section 8 of the ROD, in accordance with the Nuclear Regulatory Commission guidance document NUREG 1620 (NRC, 2000). Radon flux shall be measured using standard NRC techniques presented in 40 CFR Part 61, Appendix B, Method 115 to ensure that the average radon flux from the cover	Radon flux calculations were performed for the selected cover borrow source and have been included as Attachment D-3 to this appendix.

Table D-2 — Performance Standards Applicable to Mine Waste Containment in Pits 3 and 4

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Performance Standard No. in CD SOW	Performance Standard	Comments
	remains less than 20 pCi/m2/sec.	
2.4.2.4.2 H.iii.	The cover shall be constructed in compacted lifts and include a synthetic liner of a material determined during design, to minimize infiltration of precipitation into the underlying mine wastes.	The soil cover system described in response to item 2.4.2.4.2 H.i has been proposed to meet this performance objective. The cover soil will be placed as described in Section D.10 of this appendix.
2.4.2.4.2 H.iv.	The cover shall be constructed to efficiently minimize infiltration of water, while preserving slope stability, minimizing erosion and biointrusion, and supporting vegetation. The cover shall be designed using standard engineering techniques and a factor of safety of 1.3 for static and 1.0 for dynamic slope stability. The cover shall be erosionally stable under the 100-year, 24-hour storm event.	The results of infiltration analyses of the cover system are included as Attachment D-4 to this appendix. Erosional and slope (veneer) stability calculations are included as Attachments D-6 and D-7 to this appendix, respectively.
2.4.2.4.2 H.v.	The cover shall overlay mounded mine waste and shall slope out to a surface water management system to maximize runoff and minimize infiltration into the mine wastes, while preserving slope stability.	See response to 2.4.2.4.2 G.x, above.
2.4.2.4.2 H.vi.	Once constructed, the cover shall be vegetated as determined during RD, in consultation with the Tribe, for purposes of evapotranspiration, ecological habitat, slope stability, and long-term effectiveness.	Infiltration calculations are presented in Attachment D-4 to this appendix. Vegetation parameters for infiltration analyses were selected based on proposed species provided in the Attachment D-12 (Revegetation Plan). Selection of species incorporated input from the Tribe as well as other factors as discussed in Attachment D-12.

Table D-3 — Performance Standards Applicable to Groundwater Removal from BPA

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Performance Standard No. in CD SOW	Performance Standard	Comments
A. Temporary Facilities During Construction Activities		
2.4.2.5 A.i.	During performance of the BPA Component of Work, temporary facilities, such as covers, runoff controls, temporary sumps, and water capture and removal systems, shall be provided, as determined in the SWMP and RD. Water requiring treatment shall be conveyed as soon as practicable to the WTP for storage and treatment.	This work will be performed as part of the Phase 2 Pit 3 remediation. Refer to 2.4.2.4.2 A.i above.
B. Groundwater Diversion - Backfilled Pit Area		
2.4.2.5 B.i.	Groundwater adjacent to the BPA shall be collected and diverted away or blocked from flowing into the BPA, as practicable, by methods determined during RD.	This work will be performed as part of the Phase 2 Pit 3 remediation. Refer to 2.4.2.4.2 B.i above.
2.4.2.5 B.ii.	To the degree practicable, clean ground water shall be segregated from contaminated ground water to minimize water volumes requiring treatment.	This work will be performed as part of the Phase 2 Pit 3 remediation. Refer to 2.4.2.4.2 B.ii above.
2.4.2.5 B.iii.	Contaminated groundwater shall be captured and treated in the WTP.	This work will be performed as part of the Phase 2 Pit 3 remediation. Refer to 2.4.2.4.2 B.iii above.
C. Surface Water - Backfilled Pit Area		
2.4.2.5 C.i.	Facilities shall be constructed to divert surface water away from the BPA. The diversion facilities shall be designed using standard engineering techniques for capacity and erosional stability to convey the 100-year, 24 hour storm event in a stable manner and to withstand a 500-year, 24 hour storm event.	This work will be performed as part of the Phase 2 Pit 3 remediation. Refer to 2.4.2.4.2 C.i above.
2.4.2.5 C.ii.	To the degree practicable, clean surface water shall be segregated from contaminated water to minimize water volumes requiring treatment.	This work will be performed as part of the Phase 2 Pit 3 remediation. Refer to 2.4.2.4.2 C.ii above.
2.4.2.5 C.iii.	Contaminated surface water shall be captured and treated in the WTP.	This work will be performed as part of the Phase 2 Pit 3 remediation. Refer to 2.4.2.4.2 C.iii above.
D. Groundwater Removal from Backfilled Pit Area		
2.4.2.5 D.i.	Water in the BPA shall be removed using wells or other methods approved by EPA during RD, to elevations determined during RD which minimize hydraulic head in the pit, scaling, and fouling.	The groundwater pump-back systems using extraction wells installed in the BPA are described in Section D7.5 of Appendix D entitled “Mine Waste Excavation and Containment”, and references drawings in Volume II to illustrate this contaminated groundwater pump-back system in the BPA. In general, wells currently on site that are effective at removing contaminated groundwater will be saved for continued use during the RA. Additional extraction wells may be installed and/or planned for installation in the BPA and conveyed to the WTP for treatment as described in this appendix.

Table D-3 — Performance Standards Applicable to Groundwater Removal from BPA
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Performance Standard No. in CD SOW	Performance Standard	Comments
2.4.2.5 D.ii.	Water removed from the BPA shall be conveyed to the WTP for treatment.	Water removed from the BPA will be conveyed to the WTP, either via the storage ponds or directly to the WTP, depending on WTP operating conditions at the time of removal.
E. Mine Waste Excavation and Consolidation		
2.4.2.5 E.i.	As approved during RD, mine waste materials shall be mounded above the top elevation of the BPA and sloped to support a cover and surface water management system designed to maximize runoff and minimize infiltration into the mine wastes, while preserving slope stability.	The elevation of the upper surface consisting of mine waste rock in the BPA will be greater than the current edge of the BPA as discussed in Appendix D and depicted on the drawings referenced in Volume II. This will allow the upper liner coming from Pit 3 to extend beyond this edge so that precipitation will run off the cover surface and be channeled away from the BPA. Cap slope stability also is discussed in Appendix D and there are calculations supporting the cover design including the slopes presented. Surface Water management designs are presented in Appendix F entitled “Surface Water and Sediment Controls”.
F. Cover Construction		
2.4.2.5 F.i.	A cover made of geologic material and a synthetic liner shall be constructed over the mounded mine waste in the BPA in such a way as to permanently meet the ROD cleanup standards for soil and radon flux and to minimize the infiltration of water into the pits.	Refer to 2.4.2.4.2 H.i. above
2.4.2.5 F.ii.	Cover specifications shall be determined during remedial design and shall ensure that the thickness of the geologic materials alone shall be sufficient to limit the radon flux rate to less than 20 pCi/m2/sec as required in Section 8 of the ROD, in accordance with the Nuclear Regulatory Commission guidance document NUREG 1620 (NRC 2000). Radon flux shall be measured using standard NRC techniques presented in 40 CFR Part 61, Appendix B, Method 115 to ensure that the average radon flux from the cover remains less than 20 pCi/m2/sec.	Refer to 2.4.2.4.2 H.ii. above.
2.4.2.5 F.iii.	The cover shall be constructed in compacted lifts and include a synthetic liner of a material determined during design, to minimize infiltration of precipitation into the underlying mine wastes.	Refer to 2.4.2.4.2 H.iii. above.
2.4.2.5 F.iv.	The cover shall be constructed to efficiently minimize infiltration of water, while preserving slope stability, minimizing erosion and biointrusion, and supporting vegetation. The cover shall be designed using standard engineering techniques and a factor of safety of	Refer to 2.4.2.4.2 H.iv. above.

Table D-3 — Performance Standards Applicable to Groundwater Removal from BPA
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Performance Standard No. in CD SOW	Performance Standard	Comments
	1.3 for static and 1.0 for dynamic slope stability. The cover shall be erosionally stable under the 100-year, 24-hour storm event.	
2.4.2.5 F.v.	The cover shall overlay mounded mine waste and shall slope out to a surface water management system to maximize runoff and minimize infiltration into the mine wastes, while preserving slope stability.	Refer to 2.4.2.5 E.i. above

As described above, the regrading and capping of BPA will be performed concurrently with Pit 3 backfilling during Phase 2 and the BPA surface cover will be contiguous with the Pit 3 surface cover. As such, performance standards applicable to regrading and capping of the BPA are addressed under Pit 3 waste consolidation performance standards.

D3.0 ENGINEERING DESIGN DRAWINGS

The engineering design drawings are contained in Volume 2 of the BODR. The drawings related to Mine Waste Excavation and Containment (Table D-4) are located in Section 4 and include:

Table D-4 — Mine Waste Excavation and Containment (Section 4) Drawing List

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Sheet Number	Description
4-1	Mine Waste Location Map
4-2	Phase 1 - Waste Excavation and Pit 4 Backfill
4-3	Phase 1 – Hillside Waste Rock Pile Excavation Plan
4-4	Phase 1 – Pit 4 Overburden Pile Excavation Plan
4-5	Phase 1 – Stockpiles 1 and 2 Excavation Plan
4-6	Phase 1 – Stockpiles 3, 4, 5 and 8 Excavation Plan
4-7	Phase 1 - Stockpiles 6 and 7 Excavation Plan
4-8	Phase 1 – South Pond Bench Excavation Plan
4-9	Phase 1 – Western Drainage Waste Excavation Plan
4-10	Phase 1 – Pit 4 Cover Tie-In Grading Plan
4-11	Phase 1 – Pit 4 Cover Tie-In Grading Point Table
4-12	Phase 1 – Pit 4 Bottom Excavation and Grading Plan
4-13	Phase 1 – Pit 4 Sub-Waste Liner Installation Plan
4-14	Phase 1 - Pit 4 Sub-Waste Liner Installation Grading Points
4-15	Phase 1 – Pit 4 Sub-Waste Liner Installation Sections
4-16	Phase 1 – Pit 4 Infiltration Collection System
4-17	Phase 1 - Pit 4 Top of Cover Grading Plan
4-18	Phase 1 – Pit 4 Top of Cover Gading Point Table
4-19	Phase 1 – Extent of Pit 4 Geocomposite Drainage Layer
4-20	Phase 1 - Pit 4 Backfill Sections
4-21	Phase 1 – Pit 4 North Subgrade Grading Plan
4-22	Phase 1 – Pit 4 North Cover Grading Plan
4-23	Phase 1 - Pit 4 North Sections
4-24	Phase 2 – Waste Excavation and Pit 3 and BPA Backfill
4-25	Phase 2 – Hillside Waste Rock Pile Excavation Plan
4-26	Phase 2 – Western Drainage Excavation Plan
4-27	Phase 2 – East Waste Rock Pile Excavation Plan
4-28	Phase 2 – Pit 2 West Subgrade Grading Plan
4-29	Phase 2 – Contaminated Soil and Sediment Location Plan
4-30	Phase 2 – East Access Road Materials Excavation Plan
4-31	Phase 2 – Western Drainage Sediments Excavation Plan
4-32	Phase 2 – Eastern Drainage Sediments Excavation Plan
4-33	Phase 2 – Internal Mine Roads Materials Excavation Plan
4-34	Phase 2 – Pit 3 Cover Tie-In Grading Plan
4-35	Phase 2 – Pit 3 and Area 5 Cover Tie-In Grading Point Table
4-36	Phase 2 – Area 5 Cover Tie-In Grading Plan
4-37	Phase 2 – Pit 3 Bottom Excavation and Grading Plan
4-38	Phase 2 – Pit 3 Underdrain Dewatering Plan
4-39	Phase 2 – Pit 3 Underdrain Dewatering Sections
4-40	Phase 2 – Pit 3 Sub-Waste Liner Installation Plan
4-41	Phase 2 – Pit 3 Sub-Waste Liner Installation Sections
4-42	Phase 2 – Pit 3 and BPA Top of Cover Grading Plan
4-43	Phase 2 – Pit 3 and BPA Top of Cover Grading Point Table
4-44	Phase 2 – Pit 3 and BPA Backfill Sections (1 of 2)
4-45	Phase 2 – Pit 3 and BPA Backfill Sections (2 of 2)
4-46	Phase 2 – Area 5 Cover Grading Plan

Table D-4 — Mine Waste Excavation and Containment (Section 4) Drawing List
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Sheet Number	Description
4-47	Phase 2 – Area 5 Sections
4-48	Phase 2 – Pit 2 West Cover Grading Plan
4-49	Phase 2 – Adit Pit Cover Grading Plan
4-50	Phase 2 – Pit 2 West and Adit Pit Sections
4-51	Phase 3 – Waste Excavation and Pit 3 and BPA Backfill
4-52	Phase 3 – Central Drainage Excavation Plan
4-53	Phase 3 – Contaminated Soil and Sediment Excavation
4-54	Phase 3 – BPA Dewatering and Infiltration Collection Plan
4-55	Phase 3 – Pit 3 and BPA Top of Cover Grading Plan
4-56	Phase 3 – Pit 3 and BPA Top of Cover Grading Point Table
4-57	Phase 3 – Pit 3 Toe Area Grading Plan
4-58	Phase 3- Extent of Pit 3 and Area 5 Geocomposite Drainage Layer
4-59	Phase 3 – Pit 3 and BPA Backfill Sections (1 of 2)
4-60	Phase 3 – Pit 3 and BPA Backfill Sections (2 of 2)
4-61	Permanent Maintenance Roads Key Map
4-62	Pit 4 Maintenance Road Plan and Profile – Station 0+00 to 12+00
4-63	Pit 4 Maintenance Road Plan and Profile – Station 12+00 to 24+00
4-64	Pit 4 Maintenance Road Plan and Profile – Station 24+00 to 36+00
4-65	Pit 4 Maintenance Road Plan and Profile – Station 36+00 to 48+00
4-66	Pit 4 Maintenance Road Plan and Profile – Station 48+00 to 60+00
4-67	Pit 4 Maintenance Road Plan and Profile – Station 60+00 to End
4-68	Pit 3 Maintenance Road Plan and Profile – Station 0+00 to End
4-69	BPA Maintenance Road Plan and Profile – Station 0+00 to 8+50
4-70	BPA Maintenance Road Plan and Profile – Station 8+50 to 17+00
4-71	BPA Maintenance Road Plan and Profile – Station 17+00 to End
4-72	Permanent Maintenance Roads Line and Curve Tables
4-73	End of Phase 3 – Interim Fencing Plan
4-74	Final Remediation Grading Plan
4-75	Final Remediation – West Pond Regrading Plan
4-76	WCA Settlement Monitoring Points Plan
4-77	Final Remediation – Permanent Access Barrier Plan
4-78	Revegetation Areas
4-79	Details and Typical Sections (1 of 20)
4-80	Details and Typical Sections (2 of 20)
4-81	Details and Typical Sections (3 of 20)
4-82	Details and Typical Sections (4 of 20)
4-83	Details and Typical Sections (5 of 20)
4-84	Details and Typical Sections (6 of 20)
4-85	Details and Typical Sections (7 of 20)
4-86	Details and Typical Sections (8 of 20)
4-87	Details and Typical Sections (9 of 20)
4-88	Details and Typical Sections (10 of 20)
4-89	Details and Typical Sections (11 of 20)
4-90	Details and Typical Sections (12 of 20)
4-91	Details and Typical Sections (13 of 20)
4-92	Details and Typical Sections (14 of 20)

Table D-4 — Mine Waste Excavation and Containment (Section 4) Drawing List
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Sheet Number	Description
4-93	Details and Typical Sections (15 of 20)
4-94	Details and Typical Sections (16 of 20)
4-95	Details and Typical Sections (17 of 20)
4-96	Details and Typical Sections (18 of 20)
4-97	Details and Typical Sections (19 of 20)
4-98	Details and Typical Sections (20 of 20)

D4.0 MATERIAL BALANCE

The configuration of the WCA and major above grade waste excavation areas at the end of Phase 1, Phase 2, and Phase 3 are shown on Drawings 4-2, 4-24, and 4-51 respectively. The storage capacities of the WCA and estimated material excavation volumes are summarized in Table D-5 (Pit 4) and Table D-6 (Pit 3).

The Pit Waste Capacities shown in Tables D-5 and Table D-6 reflect the entire volume of the WCA exclusive of the soil cover volume. As noted in Section D1.0, the configuration of the WCA shown reflects the maximum waste storage. As can be seen in Table D-5 and D-6, the available disposal volume is greater than maximum anticipated volume of material requiring disposal.

Material volumes of above-grade excavations were calculated based upon topographic differences between the current ground surface (i.e., post mining) and the pre-mine ground surface in waste removal areas. Volumes estimates of Ore and Protore stockpiles, which are located on top of low-activity/reactivity waste surfaces, required the estimation of an intermediate, top of low-activity/reactivity waste rock surface. These intermediate surfaces were estimated based on drilling results presented in the *Mine Waste Investigations Report* (MGC, 2011a) and interpretations of the surrounding waste rock contours. Volume estimates for Contaminated Soil and Sediment, Mine Drainage Sediment, and Road Material excavations were obtained from the *Mine Waste Investigations Report*.

The majority of the high-activity/reactivity waste on Site will be placed in Pit 4. In order to assure there is sufficient capacity in the Pit 4 WCA to store this material, the storage capacity of the containment area, excluding zones that are: 1) within 20-feet of the underdrain, 2) within 20 feet of the surface cover, and 3) within 20 feet of the pit walls, was calculated and is listed as “Pit 4 High Activity/Reactivity Waste Capacity” in Table D-5. Prior to placement of high

activity/reactivity waste in Pit 4, at least 20 feet of low activity/reactivity mine waste will be placed above the underdrain layer. It is anticipated that the sources of low-activity/reactivity mine waste placed above the drain layer in Pit 4 (prior to placement of any high activity/high reactivity waste) may include:

1) Rejected material from processing of the HSWRP placed as liner protection for the Subwaste Liner (3 feet thick)
2) Pit 4 Overburden Pile material (estimated to be approximately 440,000 cubic yards) and,
3) Other HSWRP reject material from initial drain material processing (total reject including overliner protection layer estimated to be 133,000 cubic yards).

The available volume of low-activity/reactivity mine waste from these sources is much greater than the estimated 140,000 cubic yards material needed to provide the proposed 20-foot thick minimum cover of low-activity/reactivity mine waste over the Pit 4 drainage layer.

Sources of low-activity/reactivity waste that will be placed above the drain layer in Pit 3 prior to placement of any high activity/ reactivity waste include:

1) Drain rock processing rejects from HSWRP processing for drain material, and
2) The remainder of the unprocessed material from the HSWRP.

The combined volume of low-activity/reactivity mine waste from these HSWRP materials is estimated to be approximately 1,550,000 cubic yards, which is much greater than the estimated 349,000 cubic yards material needed for the proposed 20-foot thick cover of low-activity/reactivity mine waste over the Pit 3 drainage layer. Thus, for both Pit 3 and Pit 4, the material available during early stages of backfilling, as identified in the RA Schedule presented in Appendix X, is more than sufficient to provide a low activity/reactivity layer at least 20 feet in thickness, prior to placement of higher activity/reactivity wastes.

Table D-5 — Phase 1 – Material Balance to Pit 4

Source of Mine Waste	Volume (c.y.)	
Low-Activity/Reactivity Waste Sources:		
Hillside Waste Rock Pile - Drain Gravel	95,700	
Pit 4 Subwaste Liner Bedding Layer	5.200	
Hillside Waste Rock Pile - Reject	133,000	
Pit 4 Overburden Pile	440,000	
Pit 4 Rockfall Debris and Grading Spoils	9,000	
South Pond Grading & Excavation	631,000	
West Access Road Cleanup	8,000	
Other South Waste Rock Pile Excavation	4,201,400	
Total Low-Activity/Reactivity Waste		**5,523,300**
High Activity/Reactivity Waste Sources:		
Protore Stockpile 1	127,000	
Protore Stockpile 2	60,000	
Ore Stockpile 3	34,200	
Ore Stockpile 4	379,000	
Ore & Protore Stockpile 5	79,900	
Ore & Protore Stockpile 6	185,000	
Ore Stockpile 7	78,600	
Lime Protore Stockpile 8	490,000	
Pit 4 Bottom Sediments	1,000	
Total High-Activity/Reactivity Wastes		**1,434,700**
Total Calculated Mine Waste Volume to Pit 4		**6,958,000**
Pit 4 High-Activity/Reactivity Waste Capacity		**3,549,000**
Total Pit 4 Waste Capacity*		**6,958,000**

*Capacity calculated using bathymetric surface below the water levels in pits 3 and 4 and LiDAR topographic data presented in the *Survey Design Investigation Report* (Tetra Tech, 2010).

A shrinkage factor of 1.0 (no shrink or swell) was assumed between the excavated volume and in-place volume in the waste containment facilities when evaluating the available storage capacity. The estimated shrinkage factor is for end-of-construction conditions and includes settlement that occurs during the fill process, but does not include volume reductions that will occur due to long-term settlement (estimated in Attachment D-13). Although experience indicates that some shrinkage often occurs when regrading loose-dumped mine waste rock piles, a shrinkage factor of 1.0 is considered conservative, but reasonable in this case due to the age of the waste rock piles. This and other assumptions used to complete these initial

material balance calculations should be reviewed, and the calculations updated as additional information becomes available during RA construction.

Existing topsoil stockpiles located in the CSZ, which includes the construction support facility and the WTP and equalization ponds, are shown on Drawings 2-1, 2-2, and 2-3. At present, these topsoil stockpiles have not been included in the Phase 1 material balance as material to be placed in Pit 4. These topsoil materials will be excavated and relocated as part of initial site preparation work. The Staging/Temporary Stockpiling Plan in Appendix R discusses where these materials may be stored during the RA depending on the results of sampling and analyses performed on these materials during the RA to determine if they meet soil cleanup criteria.

Preliminary testing performed on soils in the southern topsoil stockpile shown on Drawing 2-2 indicate these materials meet soil cleanup standards (MWH, 2013a). If further testing during the RA verifies that these soils, and any soils in the two smaller stockpiles shown on Drawing 2-3, meet soil cleanup standards, they will be used for clean soil cover during RA construction rather than being placed as mine waste in Pit 4 during Phase 1. If these materials do not meet soil cleanup standards, they will be temporarily stockpiled within existing mine waste areas. The Staging/Temporary Stockpiling Plan (Appendix R) describes where, and how, these materials will be temporarily stored during Phase 1 and then used either as cover material if clean, or backfilled into Pit 4 if concentrations are in excess of the cleanup limits.

Vegetation removed during site preparation will be disposed of as a thin layer in Pit 3. The volume of material is estimated to be approximately 20,000 cy based on tree counts conducted in May 2014. This material will be treated as low-activity/reactivity when placed within the WCA. Due to the small volume relative to the size of the WCA, differential settlement due to decomposition of this material over time is expected to be insignificant so long as the material is placed in a thin, relatively uniform layer over the WCA.

Table D-6 — Phases 2 and 3 Material Balance to Pit 3

Source of Mine Waste	Volume (c.y.)	
Low-Activity/Reactivity Waste Sources:		
Hillside Waste Rock Pile - Drain Gravel, Bottom	151,000	
Pit 3 Sub-Waste Liner Bedding Layer	14,700	
Hillside Waste Rock Pile - Drain Gravel, Slopes	210,000	
Hillside Waste Rock Pile - Reject & Unprocessed	1,551,000	
East Waste Rock Pile	1,132,000	
Eastern Drainage Sediments	30,000	
Pit 3 Rockfall Debris and Grading Spoils	31,700	
Western Drainage Sediments	80,000	
Western Drainage - Waste Rock & Sediments	4,381,600	
Impacted Foundation Sediments/Soils	995,000	
Access Road Materials	79,000	
Central Drainage Sediments	50,000	
Central Drainage - Remaining SWRP Materials	2,103,000	
Wood from Tree Removal	20,000	
Total Low-Activity/Reacitity Waste		**10,829,000**
High Activity/Reactivity Waste Sources:		
Pit 3 Bottom Sediments	3,300	
Adit Pit Waste	15,000	
PCP Soils	35,800	
Total High-Activity/Reactivity Waste		**54,100**
Total Calculated Mine Waste Volume to Pit 3		**10,883,100**
Total Pit 3 Waste Capacity*		**15,394,000**
Excess Capacity per Current Design		**4,510,900**

*Capacity calculated using bathymetric surface below the water surface in Pits 3 and 4 and LiDAR topographic data presented in the *Survey Design Investigation Report* (Tetra Tech, 2010).

The assumptions discussed above at arriving at this material balance will be reviewed as additional information becomes available and material balance calculations updated as needed. The CSZ is shown on Drawing 2-13. The volume of material to be excavated during contaminated soil and sediment excavation during preparation of the Construction Support Facilities Zone (including sediments in isolated areas within the Whitetail Creek [referred to as the Far West Drainage in the RI/FS (URS, 2002)]) will be determined during construction. It is

expected to be a relatively small volume, and will be handled in the same manner as any contaminated topsoil stockpile materials encountered.

The results of the materials balance calculations summarized in Tables D-5 and D-6 indicate there is sufficient capacity in the WCA to consolidate the estimated volumes of mine wastes from the Site. In addition, Pit 4 has sufficient capacity in the designated High Activity/Reactivity waste storage zone to consolidate all Ore and Protore Stockpiles on the Site. The calculated capacity for the Pit 3 backfill configuration reflects the maximum storage volume that can be achieved in this area. This configuration will be reviewed periodically during RA construction and the configuration modified as the waste volume estimates, as well as the estimated shrink/swell factor, become more refined.

D5.0 MINE WASTE EXCAVATIONS

The mine wastes that will be excavated for consolidation in the WCA include:

- Above-Grade Mine Wastes, which include wastes that are piled above the pre-mining topographic surface.
- Contaminated Soils and Sediments, which are materials located in the MA and mine affected area (MAA) that exhibit contaminant concentrations above the cleanup levels in BODR Tables 4-1 and 4-2. This includes sediments located in drainages downstream of the MA in the MAA, sediments in isolated locations in the Whitetail Creek (Far West) Drainage, and mine wastes used for the construction of roads and any soils and sediments below, adjacent to, and downstream of the roads that exceed the cleanup levels.

General excavation procedures that will be used for the excavation of all waste types are discussed in the following section, and excavation procedures specific to each of these waste types in subsequent sections.

D5.1 GENERAL EXCAVATION METHODS AND PROCEDURES

This section describes the general excavation procedures that are common to the various mine wastes.

- Excavations will commence from higher to lower elevation (i.e., in a downhill direction), with a horizontal working excavation surface and an elevated surface at the downhill portion of the active excavation area to retain storm water within the excavation area. To

the extent practicable, excavated materials will be directly loaded into haul trucks, transported, and placed within the WCA. Materials temporarily stockpiled will include:

- Relocation of Existing Topsoil Stockpiles
- Demolition Debris from Structures in the CSZ
- Phase I CSZ Soil Remediation Materials, including Whitetail Creek (Far West Drainage) soil cleanup materials
- CSZ Grading Materials
- HSWRP Process Materials
- Pit 4 – Bottom Cleanup and Grading
- Groundwater Controls Systems Interceptor Trench Excavated Material
- Pit 3 - Bottom Cleanup and Grading

- All temporarily stockpiled material will be handled in accordance with the Staging/Temporary Stockpiling Plan contained in Appendix R.
- Surface water and stormwater management will be in accordance with the *Stormwater Management Plan (SWMP)* included as Appendix O. The SWMP identifies the BMPs that will be implemented to control surface water and minimize the transport of sediments during RA construction.
- Captured sediments that are confirmed to be below cleanup levels may be incorporated into constructed soil cover or revegetation soil layers. Captured sediments above cleanup levels will be consolidated with other mine wastes in the WCA.
- Maintenance and monitoring requirements for surface and stormwater controls are described in the *Operations Maintenance and Monitoring (OM&M) Plan* (Appendix P).
- Verification that excavated areas meet cleanup levels and additional contaminant delineation performed during the RA will be performed in accordance with the *Analytical Support and Verification Plan for Remediation of Surface Materials and Sediments* (Appendix S).

- The equipment and procedures for mine waste excavation is presented in the RA Work Plan. In general, mine waste excavation will be performed using standard excavating equipment and haul trucks.
- To the extent possible, excavation of sediments in the drainages will only occur during drier parts of the year (summer and early autumn) in order to avoid flowing water.
- Following removal of mine wastes but prior to final grading and installation of cover (i.e., completion of remediation), shallow test trenches will be excavated in areas where seeps are visible or are likely to contain shallow groundwater (i.e. valley bottoms or areas of wet ground). These shallow test trenches will be excavated to evaluate the possible existence of shallow groundwater that may interact with surface water after removal of mine waste and contaminated surface materials. If groundwater is observed in a test trench, samples will be collected and assessed relative to cleanup levels as discussed in Appendix O. If shallow groundwater is identified that does not meet cleanup standards, it will be collected and conveyed for treatment to minimize the co-mingling of contaminated water with clean water to the extent practicable.

  The precise details of shallow groundwater collection systems for as-yet unidentified areas where shallow groundwater may interact with surface water are not known at this time. These details will depend on the location and extent of these occurrences. Depending on the locations of the seeps, the water would be conveyed either by gravity, or be pumped to the water management pond or other impacted water collection point (e.g. the Western Drainage Seep pump back system).
- Areas cleared of above-grade mine waste and meeting soil cleanup criteria will be graded to conform to the pre-mining topography as shown on Drawing 4-74, to the extent practical, covered, and revegetated as described in Section D11.0. In areas cleared of mine waste where the underlying native soils do not meet soil cleanup criteria, additional Contaminated Soils and Sediments Excavation will be performed as described in Section D5.3.

D5.2 Above-Grade Mine Waste Excavation

Above-grade mine wastes generally overlie the pre-mining topographic surface and are shown on Drawing 4-1. Above-Grade Mine Waste Excavation Plans have been prepared for the three key phases of construction as shown on Drawings 4-2, 4-24, and 4-51 and are discussed below.

The above-grade wastes will be excavated to the native ground surface, which has been estimated based upon the pre-mining topography and will be verified visually in the field during excavation. The extents of contaminated soil cleanup below the above-grade mine waste will be determined during RA construction, and is not reflected in the finished grades shown on the Drawings.

Procedures for removal of contaminated soils beneath the above-grade mine wastes are described under "Contaminated Soils and Sediments Excavation," Section D5.3 and estimated volumes for these materials are included as "Impacted Foundation Sediments/Soils" in the material balance presented in Section D4.0.

D5.2.1 Phase I Excavation

Excavated surfaces upon completion of Phase 1 construction are shown on Drawing 4-2. Primary above-grade excavations that will occur during this phase include:

1) Topsoil Stockpiles – Topsoil Stockpiles located in the CSZ are located in the southwestern corner of the Site as shown on Drawing 4-2. These Topsoil Stockpiles will be relocated by excavating to the native ground surface as part of initial Site preparation work and moved to temporary stockpile locations as discussed in Appendix R. The final ground surface shown is based upon the pre-mine topography in this area.

2) Pit 4 Overburden Pile – will be excavated to the native ground contact. The final ground surface shown is based upon the pre-mine topography in this area.

3) HSWRP – will be excavated as required to provide material needed to produce Pit 4 drain material. Excavation will occur starting from the eastern edge of the HSWRP and working in a westerly direction as shown on Drawing 4-3 in order to provide the needed drain material and clear HSWRP material from the final footprint of the Pit 4 WCA.

4) Ore/Protore Stockpiles - will be removed down to the "base of Protore surfaces" which were estimated as described in Section D4.0.

5) South Waste Rock Pile Waste material in the vicinity of the South Pond - will be excavated to provide a level bench at an elevation of approximately 2683 ft. as shown on Drawing 4-8. The South Pond Excavation will be completed in waste rock to the final pond configuration and the surrounding waste rock surface graded to drain by gravity to the South Pond to the extent practical as shown on Drawing 4-8.

6) The uphill (northern) portion of the South Waste Rock Pile located in the Western Drainage - will be excavated to the native ground contact. The excavation will start from the uphill end of the South Waste Rock Pile and work in a downslope or downstream manner, removing all waste rock from the Western Drainage channel section as the excavation progresses. The excavated surface shown on the Drawing 4-9 in areas of the Western Drainage that have been cleared of waste rock reflect the pre-mine topography. During excavation, the active waste rock excavation surface will be sloped to drain toward the north in order to prevent surface water runoff from the excavation as shown on Drawing 4-9.

7) Adit Pit and Pit 2 West – mine waste will be excavated to native ground contact. Final soil clean-up and verification in these areas will be completed in Phase 2. Final grading and backfilling of the Adit Pit and Pit 2 West is not practical during Phase 1 due to the extent of contaminated areas remaining around these pits. These remaining areas of contamination at the end of Phase 1 will pose a very significant risk for recontamination of the Adit Pit and Pit 2 West areas. Therefore, any associated soil cleanup and verification, final grading, and cover soil placement will not be completed until Phase 2 after cleanup of the areas adjacent to the Adit Pit and Pit 2 West has been completed. Any mine waste associated with soil cleanup in the Adit Pit and Pit 2 West will be placed in Pit 3. If water does accumulate in the two pits during the time period after removal of waste rock, but prior to final soil cleanup, grading, and cover soil placement, the pits may be dewatered by pumping to the water management system for treatment.

D5.2.2 Phase 2 Excavation

Excavated surfaces at the completion of Phase 2 construction are shown on Drawing 4-24. Primary above-grade excavations that will occur during Phase 2 will include:

1) Completion of the excavation of the HSWRP to the native ground contact. The final ground surface shown on Drawing 4-25 is based upon the pre-mine topography in this area.

2) Removal of all remaining Above-Grade Mine Waste from the Western Drainage. The final ground surface shown on Drawing 4-26 is based upon the pre-mine topography in this area. Additional stormwater and sediment control structures required below the downstream toe of the waste rock upon completion of excavation and soil sediment

cleanup in the Western Drainage are discussed in Appendix F and shown on the Section 6 Drawings.

3) Removal of all Above-Grade Mine Waste in the East Waste Rock Pile located in the Eastern Drainage. The final ground surface shown on Drawing 4-27 is based upon the pre-mine topography in this area. Additional storm water and sediment control structures required below the downstream toe of the waste rock upon completion of excavation and soil sediment cleanup in the Eastern Drainage are discussed in Appendix F and shown on the Section 6 Drawings.

4) Excavations associated with regrading and capping of Area 5 (between Pit 3 and Pit 4) once it is no longer needed for drain material processing and stockpiling. The final regraded surface of Area 5 is shown on Drawing 4-46.

D5.2.3 Phase 3 Excavation

Drawing 4-51 shows the excavated surfaces at the completion of Phase 3 construction. The primary Above-Grade Excavation that will occur during Phase 3 will consist of removing the South Pond and remaining South Waste Rock Pile from the Central Drainage downgradient of the Pit 3 WCA as shown on Drawing 4-52. The final ground surface shown on Drawing 4-52 is based upon the pre-mine topography in this area. Additional stormwater/sediment engineering controls (i.e., BMPs) as described in Appendix F and shown in Section 6 of the Drawings will be required below the toe of the South Waste Rock Pile during the final phase of excavation in the Central Drainage.

D5.3 CONTAMINATED SOILS AND SEDIMENTS EXCAVATION

Contaminated soils (impacted by roads or other areas of mine waste) and sediments are materials located in the MA and MAA that exhibit contaminant concentrations above the cleanup levels in BODR Tables 4-1 and 4-2. These materials include sediments located in drainages downstream of the MA in the MAA and sediments in localized areas within the Whitetail Creek (referred to as the Far West Drainage in the RI/FS). Contaminated soils and sediments may also include mine wastes used for the construction of roads and any soils and sediments below, adjacent to, and downstream of the roads that exceed the cleanup levels. These materials will be excavated for consolidation and containment in Pits 3 and 4.

Areas where contaminated soils and sediments have been identified or will be investigated during early phases of RA construction are shown on Drawings 2-1 through 2-4. Investigations

of the extent contaminated soils and sediments, and volume estimates for contaminated soil cleanup within the MA and MAA are based on data and information provided in the *RI Report* (EPA, 2005), *Mine Waste Investigations Report* (MGC, 2011a), and the *White Tail Creek Sediment Evaluation – Phase 2 Data Transmittal Report* (WME, 2014). In addition to those areas identified on the drawings, it is assumed that an average of 1-foot of contaminated soils and sediments exist under areas overlain by Above-Grade Mine Waste and will require excavation and relocation in the Pit 3 and Pit 4 WCA. The actual extent of soil contamination and cleanup will be determined during RA construction using procedures defined in the *Analytical Support and Verification Plan for Remediation of Surface Materials and Sediments* (Appendix S).

5.3.1 Phase 1 Soil/Sediment Removal

The Far-Western Drainage (White Tail Creek) was identified in the ROD as being potentially impacted. However, the FS (URS, 2002) assumed that no sediments would be removed from this area. Additional characterization of this drainage were performed during the fall of 2013 and summarized in the *White Tail Creek Sediment Evaluation Data Transmittal Report* (WME, 2014). The results of the sediment sampling indicated two locations in this area that will require cleanup of sediments. This work will be completed prior to Phase 1 and the approximate cleanup limits are shown on Drawing 2-4. Cleanup volumes are not significant (estimated to be approximately 5,000 cy).

The location of additional potentially contaminated sediments that may require cleanup during early stages of Phase 1 construction (Early Works) include the entire fenced area within the CSZ as shown on Drawings 2-1 and 2-13. Although Contaminated Soils and Sediments have not been identified in the area delineated on this drawing with the exception of the Whitetail Creek Investigation area lying outside of the perimeter fence (WME, 2014), this area will be the site of the new WTP and Construction Support Facilities. As such, an evaluation of the potential for contamination and any necessary cleanup will occur as part of the initial RA construction. Contaminated material excavated from this area will be stockpiled as described in the *Staging and Stockpiling Plan* (Appendix R). Additional contaminated sediment cleanup during Phase 1 excavation will include the West Access Road, as shown on Drawings 2-1 and 2-4. The West Access Road is no longer used for Site access. Delineations of the extents of contaminated soils along the West Access Road are based on data and information provided in the *Mine*

Waste Investigations Report (MGC, 2011a). The actual extent of soil contamination and cleanup will be determined during RA.

5.3.2 Phase 2 Soil/Sediment Removal

The identified extents of contaminated soils and sediments that will be removed during Phase 2 construction are shown on Drawings 4-29 through 4-33. These areas of contamination include:

1) Western Drainage Sediments Excavation (Drawing 4-31). The delineation of extents of contaminated sediment cleanup in the Western Drainage between the toe of the SWRP and the confluence with the Eastern Drainage is shown on Drawing 4-31 and was obtained from the *Mine Waste Investigations Report* (MGC, 2011a). The actual extent of soil contamination and cleanup will be determined during RA. It is anticipated that this cleanup will occur immediately prior to completion of the cleanup of Above-Grade Waste and contaminated soils in the upper portion of the drainage. This will enable the release of storm water from the upper Western Drainage once cleanup is complete without remobilizing sediments located further downstream.

2) Eastern Drainage Sediments Excavation (Drawing 4-32). The delineation of Eastern Drainage contaminated sediment cleanup in the Far East Drainage between the toe of the East Waste Rock Pile, and the confluence with the Eastern Drainage, and in the Eastern Drainage between the East Access Road and the confluence with the Western Drainage was obtained from the *Mine Waste Investigations Report* (MGC, 2011a). The actual extent of soil contamination and cleanup will be determined during RA. It is anticipated that this cleanup will occur immediately prior to completion of the cleanup of Above-Grade Waste and contaminated soils in the East Waste Rock Pile area. This will enable the release of stormwater from the East Waste Rock Pile area upon completion of cleanup without remobilizing sediments located further downstream.

3) The East Access Road (Drawing 4-30). This road currently provides access to the Site and existing WTP. A surface course of clean fill has been placed over the East Access Road in 2012 in order to reduce the potential for transport of contaminated sediments on vehicles travelling to and from the Site. It is anticipated that this road may continue to be used as a secondary access point during early phases of the RA, and that cleanup will not occur until later stages of Phase 2 construction. Delineation of extents of contaminated soils along the East Access Road is based on data and information provided in the *Mine Waste*

Investigations Report (MGC, 2011a). The actual extent of soil contamination and cleanup will be determined during RA.

4) Internal Mine Roads (Drawing 4-33). The Internal Mine Roads south of the PCP provide access to wells and pumps in the Western Drainage and to wells and the PCP pumpback system located in the Central Drainage. As such, the cleanup and or removal of these roads must be coordinated with water management activities and may not occur until later stages of Phase 2, or during Phase 3 of construction. Delineations of the extents of contaminated soils along the Internal Mine Roads are based on data and information provided in the *Mine Waste Investigations Report* (MGC, 2011a). The actual extent of soil contamination and cleanup will be determined during RA.

5.3.3 Phase 3 Soil/Sediment Removal

The identified extents of contaminated sediments that will be removed during Phase 3 will include of cleanup of Central Drainage Sediments as shown on Drawing 4-53. The delineation of Central Drainage contaminated sediment cleanup between the toe of the PCP and culvert crossing of the West End Road (Ford-Wellpinit Road) was obtained from the *Mine Waste Investigations Report* (MGC, 2011a). The actual extent of soil contamination and cleanup will be determined during RA. It is anticipated that this cleanup will occur immediately after completion of the cleanup of Above-Grade Waste and contaminated soils in the SWRP and PCP area. Although not delineated during the mine waste investigation, it is anticipated that cleanup also will require dewatering and cleanup of soils in the pond area at inlet to the Ford-Wellpinit Road culvert crossing. This will enable the release of stormwater from the Central Drainage upon completion of cleanup without remobilizing sediments located further downstream. As discussed above, it is also possible that cleanup of the Internal Mine Roads will be delayed until this time so they can continue to provide access to PCP pumpback systems.

D6.0 PHASE 1 – PIT 4 WASTE CONTAINMENT

The sequence for backfilling of Pit 4 is discussed below, from the initial rockfall protection (D6.1) for worker safety to the final containment and capping of the pit (Section D6.9).

D6.1 PIT 4 ROCKFALL PROTECTION

In the fall of 2013, a Site visit and independent assessment of potential rockfall mitigation measures that could be implemented at the Site was performed by a specialist rockfall

mitigation engineer/contractor. A recommended Rockfall Mitigation Plan has been developed by the specialist rockfall engineer/contractor based on this information and is included as Attachment D-11 to this Appendix.

Previous analyses of rockfall hazard and potential mitigation measures were performed for Pits 3 and 4 as part of the *Geologic Investigation of Pits and Assessment of Pit Sediments Design Investigation Report* (MGC, 2011b). The rockfall hazard analyses were made using the Colorado Rockfall Simulation Program (CRSP) (Jones, et al 2000) and incorporated assumed parameters based upon photographs and mapping of Site conditions available at the time. Updated rockfall simulations were performed as part of the Rockfall Mitigation Plan using CRSP, and incorporating the results of observations made while at the Site, as well as the results of rockfall monitoring that has been performed at the Site since 2011 (MWH, 2013b).

Conclusions made based upon the Site visit and updated rockfall hazard simulations included:

1. Physical and hydraulic scaling of the pit walls should be conducted to reduce the rockfall hazard prior to initiating work in the pits. Scaling should include removal, or identification and monitoring, of rockfall sources larger than 3-feet in size as appropriate.
2. The rockfall catch berm/ditch design (10-feet deep and 15-feet wide horizontally) and work sequence proposed should significantly reduce the risk of rockfall impacting the work areas during pit backfilling operations. The dimensions and construction sequence for maintaining the proposed rockfall berm/ditch is shown on Drawing 4-81.
3. A portable rockfall barrier, or an approved alternative system should be used in areas where personnel need to work outside of construction equipment prior to construction of rockfall catch berms (i.e. during sump drilling/blasting, sump excavation, drainage system construction, and liner placement) or in areas where rockfall catch berms cannot be constructed due to Site space constraints. Preliminary sketches of the proposed portable rockfall barriers are included in Attachment D-11 to this appendix.
4. Although the Rockfall Hazard Monitoring Program has provided useful information relative to the rockfall hazard at the Site, continued monitoring is unlikely to provide additional information that would be useful in designing rockfall protection measures.

The recommendations in the Rockfall Mitigation Plan included as Attachment D-11 were made by the specialist rockfall mitigation engineer/contractor and provide the general guidelines that should be followed for rockfall protection to personnel working within the boundaries of the pits.

The RA Contractor will be responsible for preparing a specific plan for rockfall mitigation that incorporates the listed rockfall mitigation and protection measures in Attachment D-11. This has been noted in the specifications. The specific rockfall mitigation plan will be subject to approval by Newmont/DMC and the EPA.

D6.2 PIT 4 DEWATERING

Pit 4 will be dewatered prior to commencement of pit bottom cleanup. Historically (WTP operating years 2001 through 2011), the peak accumulated water volume in Pit 4 has ranged from 9 million gallons to 25 million gallons, with an average peak storage of 15 million gallons. Based on these typical peak storage volumes, it is estimated that dewatering of Pit 4 will take approximately 20 days to complete. Water removed from Pit 4 during dewatering will be conveyed either directly to the WTP or to Pit 3 for intermediate storage prior to being conveyed to the WTP for treatment, depending on the WTP operating schedule at the time of dewatering.

D6.3 PIT 4 PIT BOTTOM CLEANUP AND GRADING

Sediments that have accumulated in the Bottom of Pit 4 were characterized as part of the field investigations performed for the *Geologic Investigations of Pits and Assessment of Pit Sediments Design Investigation Report* (MGC, 2011b). As part of those investigations, Pit 4 was dewatered to the point where approximately 0.5 acres of the pit bottom remained underwater. This allowed for the observation, measurement, and sampling of fine-grained sediments in the pit bottom. It was estimated that approximately 2,400 cubic yards of sediment had accumulated in the pit bottom. These sediments were found to be one to two feet thick in the pit bottom and thinner near the pit walls (0.1 to 0.3 feet thick). The sediments were noted to be predominantly saturated, silt-sized material with coarser material occurring around the margins of the pit floor. In addition to the materials investigated in the pit bottom, a significant amount of gravel to boulder-sized material are visible on the pit floor, both below and above the pool level. It is anticipated that additional coarse rock material will accumulate on the pit floor as a result of pit-slope rock scaling operations that will be performed to reduce rockfall hazard as described in Section D6.1.

The sediments and coarse rock material that has accumulated in the bottom of Pit 4 will be removed after completion of dewatering operations and prior to pit-bottom grading. Based upon the results of the pit sediments assessment (MGC, 2011b) it is anticipated that much of the material that has accumulated in the pit bottom can be removed using conventional earth-

moving equipment (excavators, loader, and haul trucks). Prior to excavation, the material will be dried, either due to natural evaporation, or by adding drying agents if needed for very fine-grained, saturated sediments. These drying agents could include fine-grained waste rock or Site soils, imported fly ash (ASTM C618 Class C or Class F), or other materials and would be mixed with the pit-bottom sediments by bucket mixing using an excavator of front-end loader. At this time, the need for drying agents to stabilize/dewater the pit sediments is not anticipated. Should such a need be identified, any amendments will be subject to prior EPA and Tribe approval.

If it is determined that the amount of fine-grained sediments remaining after bulk cleanup using earth-moving equipment could be detrimental to the performance of the underdrain system, final cleanup by hydraulic-monitoring jetting of remaining sediments may be required. Hydraulic jetting would involve washing the remaining fine-grained materials using a high-pressure water jet to a low-point in the pit bottom where they could be collected using a slurry pump and conveyed to a dewatering area where they would be pumped into geotubes for dewatering.

Sediments and coarse rock material removed during pit-bottom cleanup operations will be stockpiled for replacement in Pit 4 as described in an approved *Staging/Temporary Stockpiling Plan* (Appendix R). Material removed during pit-bottom cleanup will not be placed in zones within the Pit 4 backfill that have been designated for low-activity/reactivity waste.

Upon completion of cleanup, the bottom of Pit 4 will be graded in preparation for placement of drain material as shown on Drawing 4-12. An underdrain sump will be excavated by drilling and blasting in the low area located in central portion of the pit bottom as shown on Drawing 4-12 and 4-79. Generally, the pit is sloped to drain to this area in its current configuration. Areas where ponding may occur in the pit floor, or otherwise will not flow by gravity toward the underdrain sump will be reworked to the extent possible without ripping, drilling or blasting.

Due to the nature of the rock formation in the pit bottom, aggressive reworking of the pit floor (e.g. ripping or blasting) to remove smaller irregularities (i.e. less than 2 feet high) that result in areas of ponding is likely to result in irregular rock breakout and creation of other areas of ponding. As a result, grading the pit bottom to a perfectly smooth, free-draining surface is considered unrealistic, and is unnecessary given the inward hydraulic gradients toward the pit. Instead, if areas of ponding are noted during the jetting operation and can be removed by scraping the pit floor with the toothed bucket of a hydraulic excavator, this will be performed. In addition, the required minimum thickness of drainage layer in the pit bottom (discussed in the following section) will be maintained as measured above any high point or “sills” in the pit

bottom that obstruct flow and create ponding. Thus any areas of ponding will be shallow "dead pools" with very small volumes that form below the required minimum thickness of drainage layer and will not affect drain capacity. Material removed during grading and sump excavation will be stockpiled along with the coarse rock material removed during pit-bottom cleanup as described in the *Staging/Temporary Stockpiling Plan* (in Appendix R) and will be placed in Pit 4 during backfilling. Due to their in-situ proximity to mineralized areas, material removed during grading and sump excavation will not be placed in zones within the Pit 4 backfill that have been designated for low-activity/reactivity waste.

D6.4 PIT 4 UNDERDRAIN SYSTEM

An underdrain system constructed of crushed and screened, non-reactive rock from the Hillside Waste Rock dump will be installed in the bottom of Pit 4. This underdrain will collect groundwater before it contacts reactive mine waste backfill in the pit. A liner bedding layer will be placed over the underdrain prior to covering the underdrain system with a geomembrane liner (i.e., a sub-waste liner). The sub-waste liner is intended to isolate the underdrain from seepage through the overlying waste rock to the extent practical, a condition that is primarily a concern during construction. The configuration of the Pit 4 underdrain system is shown Drawings 4-13 and 4-15. The extent of the underdrain drain gravel and sub-waste liner are shown on Drawing 4-13.

In addition to the pit-bottom underdrain system, performance criteria 2.4.2.4.2 E.iii of the SOW in the CD stipulates that "*The drainage layers shall extend vertically along the side walls of each pit to elevations determined during RD, to keep water entering the pits from contacting mine waste and to effectively channel water to the pit bottoms."* Locations of pit wall seeps were mapped during late summer of 2010 as part of investigations for the *Geologic Investigations of Pits and Assessment of Pit Sediments Design Investigation Report* (MGC, 2011b) and additional seep mapping was performed in the spring of 2012 to provide data during wetter portions of the year (Plumley and Assoc., 2012). As communicated, both in the summary report and in the approved work plan for the spring 2012 seep mapping (MWH, 2012b): "*The specific objectives of the additional pit wall seep monitoring were to verify seeps previously identified in the Geologic Report; and identify new seeps in the pit walls above the existing water level that might be evident in the spring. Data gathered from this additional monitoring is intended to help define the positioning of drains that will be included in the remedial design for interception of pit wall seeps.*" No signs of seepage were observed in Pit 4 during either of these investigations

and as a result, the drain configuration shown on Drawing 4-12 does not include drainage layers extending up the pit walls in Pit 4.

The material proposed for the underdrain construction will be obtained by processing material from the HSWRP. Results of investigations presented in the *Mine Waste Investigations Report* (MGC, 2011a) and the *Addendum to the Mine Waste Investigations Report* (WME, 2012) indicate that processed by screening and crushing coarser fractions will have suitable durability and geochemical characteristics for use in the underdrain layer. Gradation specifications have been developed for the underdrain materials for three distinct material drain materials; (1) sump drain rock, (2) drain gravel, and (3) liner bedding.

The sump drain rock material will be used as backfill in the underdrain sump as shown on Drawing 4-79, and will be 2-inch to 6-inch sized material with no more than 3-percent by weight material finer than a #200 sieve. The drain rock is intended to have a very high permeability and large pore spaces to reduce the potential for plugging of the underdrain sump by the migration of fine-grained soils or accumulation of chemical precipitates.

The overlying drain gravel layer will cover the bottom and sump area of Pit 4 as shown on Drawings 4-12, 4-13, and 4-15. The drain gravel layer will consist of medium to coarse gravel, and will have sufficiently high permeability to allow for gravity conveyance of groundwater and seepage water to the sump area without developing a significant saturated zone within the drain system. The gradation requirements for the drain gravel are developed in Attachment D-9, and summarized in Table D-7.

Table D-7 — Drain Gravel Gradation Requirements

U.S Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
3-inch	76.2	100	100
1 1/2-inch	38.1	100	0
1-inch	25.4	90	0
3/4-inch	19.05	40	0
1/2 inch	12.7	20	0
3/8-inch	9.5	10	0
No. 4	4.76	5	0
No. 200	0.075	5	0

The estimated range of hydraulic conductivity for the drain gravel is $7x10^{-1}$ to $9x10^{0}$ cm/sec based on the gradation range listed in Table D-7. Calculations for estimated permeability of the drain gravel are presented in Attachment D-9.

The uppermost soil layer in the underdrain system will be a liner-bedding layer placed over the surface of the drain gravel as shown on the Sub-Waste Geomembrane Liner Detail on Drawing 4-81. This bedding layer will reduce the potential for puncture of the overlying sub-waste geomembrane by coarse fragments within the drain gravel layer. The bedding layer will have a maximum particle size (D_{max}) of 1.5-inch or smaller and will meet filter (retention) compatibility requirements with the underlying drainage gravel layer. The gradation requirements for the bedding layer are developed in Attachment D-9 and summarized in Table D-8. Due to its coarse nature and low fines content, the bedding layer also will have a very high hydraulic conductivity, estimated to range between 0.3 and 1 cm/sec based on the specified gradation (see Attachment D-9).

Table D-8 — Liner Bedding Gradation Specifications[(a)]

U.S. Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
1 1/2-inch	38.1	100	100
1-inch	25.4	100	90
3/4-inch	19.05	85	40
1/2 inch	12.7	40	10
3/8-inch	9.5	15	0
No. 4	4.76	5	0

Notes: [a/] Gradation based ASTM C33 No 56 Coarse Aggregate

The materials to be used in three layers within the underdrain system are designed to meet gradational stability (filter) criteria and prevent migration of the finer overlying materials into the underlying layers. Additional details of the liner bedding layer and the puncture resistance of the overlying geomembrane liner are included in Section D6.5 and Attachment D-1.

As currently configured, the underdrain system will have much greater flow capacity than is required to convey the groundwater flow rates measured in the *Groundwater Investigations Design Investigation Report* (MGC, 2011c). The drain gravel layer in the Pit 4 underdrain system as shown in the drawings with a 1) 5-foot minimum thickness, 2) hydraulic conductivity of $7x10^{-1}$ to $9x10^{0}$ cm/sec (1.38 to 17.7 ft/min), and 3) 5 percent minimum liner slope has the capacity to conduct between 2.6 and 33.1 gpm per foot of drain width to the underdrain sump.

Given the total measured groundwater inflow rate of 13.5 gpm in Pit 4 (MGC, 2011c), the underdrain system as designed has sufficient capacity in each 0.4 ft to 5 ft-wide section of underdrain to convey the entire estimated Pit 4 inflow to the underdrain sump at the minimum specified drain thickness. Therefore, the drain system placed over the entire bottom of Pit 4, has much more hydraulic capacity than necessary to convey the anticipated flow. Also, as shown in Section C on Drawing 4-15, the underdrain has a thickness considerably greater than the specified 5-foot minimum thickness at the critical location in the vicinity of the underdrain sump. Thus, the actual as-designed flow capacity will be much greater than required.

Underdrain Dewatering Design. To the extent practicable, water shall be kept from accumulating in Pit 4 during and after consolidation of waste within the pit. Water that accumulates in the underdrain sump during, and after construction, will be conveyed for treatment at the WTP by pumping through an underdrain sump dewatering system.

The underdrain sump dewatering system design is shown on Drawings 4-12, 4-15, and 4-79. Dewatering risers will be installed in the underdrain sump prior to placement of the sump drain rock. Backfill will be paced around the dewatering risers in a uniform manner using an excavator or similar placement technique (as opposed to pushing backfill around the risers using dozers) in order maintain a uniform horizontal stress distribution, and reduce lateral displacement of the risers during construction. The dewatering risers will be constructed from stainless-steel well casing, which will be extended during construction to remain above the backfill surface as waste is being placed in Pit 4. Stainless steel well casing was selected for the dewatering risers for its superior strength, which will allow it to withstand anticipated loadings from the waste backfill (relative to other corrosion-resistant pipe materials such as PVC and HDPE). In addition, it will be much less susceptible to corrosion relative to other high-strength pipe materials.

The design for dewatering risers incorporate oversized friction sleeves that prevent backfill-settlement-induced dragdown forces from loading the riser pipes. The friction sleeves are constructed of high-strength carbon steel pipe of a larger inside diameter than the outside diameter of the riser casing. In addition, two layers of 60-mil smooth HDPE sheeting will be wrapped around the friction sleeve as shown on Detail 5 of Drawing 4-80 to provide an additional slip surface and further reduce drag down forces. Similar riser designs have been used successfully at a number of waste containment facilities where very high settlements were anticipated, including the Kettleman Hills hazardous and municipal waste facility in Kings County, California.

The underdrain sumps will be dewatered using submersible pumps installed within the drain risers casing after backfilling of the sump. The well-discharge pipes will be extended in coordination with the extension of well casings so as to remain above the backfill surface, and allow near continuous dewatering as waste is being placed in Pit 4.

Duplicate dewatering risers are proposed to avoid long shutdowns in the dewatering system due to maintenance or mechanical failure during the RA and post-RA. The underdrain configuration shown on the drawings provides approximately 1,100,000 gallons of storage (assuming an active porosity of 30 percent) between the top of the sump and the lowest point in the overlying subwaste liner. This allows storage for approximately 58 days of groundwater inflow at the measured inflow rate of 13.5 gpm for Pit 4 without operating the underdrain dewatering system before the overlying bedding layer and liner system are at risk of becoming saturated.

Water Levels. The design range of operating water levels within the underdrain sumps is shown on Drawing 4-79. The proposed range of water level fluctuations was selected to ensure that the water level will remain within coarse drain rock of the sump backfill, thus avoiding water level fluctuations over the greater pit floor and liner surfaces, while at the same time avoiding drawing the water levels down to the elevation of the screened sections of dewatering risers.

D6.5 PIT 4 SUB-WASTE LINER SYSTEM

Prior to placement of mine waste in Pit 4, a sub-waste liner will be installed over the underdrain system as shown on Drawings 4-13 and 4-15. The intent of the sub-waste liner is to isolate the mine waste from the underdrain system to the extent practical and minimize the passage of both water and mine waste particles from the mine wastes into the underdrain system. The potential for migration of water and fines through the mine waste and into the underdrain will be greatest during mine waste placement, when the rates of infiltration of meteoric water will be greatest. Once the RA construction is complete, and the surface cover system is in place over the Pit 4 waste, the rates of water migration through the waste rock will diminish to low levels as discussed in Section D10.0

The liner subgrade (i.e. underdrain surface) will be shaped as shown on Drawing 4-13. The proposed grading for the sub-waste liner shown on Drawing 4-13 will provide for gravity drainage of water that collects on the liner surface toward a waste rock dewatering sump located at the low-point of the liner surface. Prior to liner installation, a geofabric cushion layer will be placed over the bedding layer of the underdrain system. The liner will be constructed of

HDPE geomembrane as discussed in Section D9.0. Prior to placement of mine waste, an overliner protection layer will be placed over the geomembrane to protect it from damage from waste loading or construction equipment.

D6.6 PIT 4 WASTE ROCK DEWATERING SYSTEM

The proposed Pit 4 waste dewatering system is shown on Drawings 4-13, 4-15, and 4-82. The purpose of the waste rock dewatering system is to collect and convey water, primarily precipitation that during the backfilling operations collects on the sub-waste liner. This water will be conveyed from the sub-waste liner to a waste dewatering sump located on the low point of the liner, where it then will be pumped through dewatering risers to the WTP for treatment. It is anticipated that flow rates to the waste dewatering sump will be highest during construction and that once the surface cover system is in place over the Pit 4 waste, the rates of water migration through the waste rock will diminish to very low rates as discussed in Section D10.0.

Waste Dewatering System. Water that accumulates in the waste rock dewatering sump during, and after construction, will be pumped to the WTP through waste rock dewatering risers located in the sump. In order to increase hydraulic efficiency, the waste rock dewatering sump will be backfilled as shown on Drawing 4-82 with drain gravel of similar specification to the drain gravel for the underdrain system discussed in Section D6.4. To provide separation and prevent fines migrations from the overlying mine waste into the drain gravel, a layer of ASTM C33 Fine Aggregate will be placed as a filter layer between the drain gravel and waste rock. In addition, the results of a filter-compatibility evaluation (Attachment D-10) indicate that an intermediate filter layer having a gradation consistent with ASTM C33 No.67 Coarse Aggregate will be needed between the drain gravel and the finer-grained (ASTM C33) waste filter sand. The required gradation of the drain gravel layer is summarized in Table D-7 above. The gradation requirements for the filter layers are developed in Attachment D-10 and summarized in Tables D-9 and D-10. These intermediate filter layers will most likely be sourced from an off-site borrow source.

Table D-9 — Filter Sand Layer (ASTM C33 Fine Aggregate) Particle Size Distribution

U.S. Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
3/8 inch	9.5	100	100
No.4	4.75	95	100
No.8	2.36	80	100
No. 16	1.18	50	85
No. 30	0.6	25	60
No. 50	0.3	5	30
No. 100	0.15	0	10
No. 200	0.075	0	5

Table D-10 — Intermediate Filter (ASTM C33 No.67 Coarse Aggregate Gradation

U.S. Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
1-inch	25.4	100	100
3/4-inch	19.05	100	90
1/2 inch	12.7	55	20
3/8-inch	9.5	10	0
No. 4	4.76	5	0

The Waste Rock Dewatering Risers will be similar to the risers discussed for the Pit 4 underdrain dewatering system in Section D6.4. These risers will be extended periodically during construction to maintain a top elevation higher than the surrounding waste backfill surface. As with the underdrain system, the subwaste dewatering system will have duplicate dewatering risers to avoid shutdowns in the dewatering system due to maintenance or mechanical failure (i.e., a primary and a backup dewatering riser).

Water Levels. The design range of operating water levels within the waste rock dewatering sumps is shown on Drawing 4-82. The range of water level fluctuations will ensure that the water level will remain within the sump backfill, thus avoiding water level fluctuations within the mine waste, while avoiding drawing the water levels down to the elevation of the screened sections of dewatering risers in order to reduce the potential for scaling and plugging of the intake system.

As currently configured, the waste dewatering sump, with an estimated backfill hydraulic in the range of $7x10^{-1}$ to $9x10^{0}$ cm/sec and an effective sump radius of approximately 30 feet, will have much greater flow capacity than is required to convey the long-term infiltration rates estimated in the infiltration analyses summarized in Section D10.3.

Additional Infiltration Collection. Additional mine waste infiltration collectors will be installed on the hillside west of the Pit 4 crest as shown on Drawing 4-16. The mine waste infiltration collectors will be French-drain style infiltration collectors installed in lined trenches as shown in detail on Drawing 4-82. The purpose of these trenches is to collect water that may infiltrate through the mine waste at the native ground contact, and route it to the top of the sub-waste liner and waste dewatering system. This system is primarily intended to collect meteoric water that infiltrates through the waste rock during construction.

D6.7 PIT 4 MINE WASTE PLACEMENT

Once the sub-waste liner has been installed, Pit 4 will be ready to receive mine wastes from various sources at the Site. In general, Pit 4 will contain most of the high-activity/reactivity waste sources at the Site as shown in Table D-5.

The waste containment capacity of the configuration shown for Pit 4 is approximately 6.72 to 6.96 million cubic yards. The Pit 4 sections shown on Drawing 4-20 delineate backfill zones where higher activity/reactivity wastes will be excluded. The material balance for Pit 4 is described in Detail in Section D4.0. The zone that can be used to contain higher activity/reactivity waste within Pit 4 has an estimated storage capacity of 3.5 million cubic yards, whereas the estimated volume of higher-activity/reactivity waste scheduled to be placed in Pit 4 is 1.43 million cubic yards. The material balance calculations indicate sufficient capacity exists for all waste scheduled for placement within Pit 4 as detailed in the material balance section.

As proposed, the waste placement sequencing in Pit 4 will be as follows:

1) Sump drain rock in underdrain sump (D6.4)
2) Underdrain gravel layer (D6.4)
3) Liner bedding (1.5” D_{max}) layer (D6.4)
4) Geofabric for liner cushion layer (D6.5)
5) Subwaste geomembrane liner(D6.5)
6) Overliner protection layer of fine-grained (1/4” D_{max} or finer) soil (D6.5)
7) Reject material from HSWRP
8) Pit 4 Overburden Pile material
9) Ore and Protore material

10) Sediments from cleanup of Pit 4 bottom

11) Pit 4 rockfall debris and pit-bottom grading spoils

12) South Pond grading and excavation spoils

13) Other SWRP Excavation

These wastes will be hauled to Pit 4 in either mine haulage trucks or off-road dump trucks, dumped, and spread in horizontal lifts with maximum thickness of 10 feet. No additional compaction of the waste material is planned beyond that which results from spreading and incidental traffic over the waste surface by construction equipment.

The final configuration of the Pit 4 backfill upon completion of construction is shown on Drawings 4-17 and 4-20. The configurations shown on these drawings reflect final cover surface grading details including drainage benches, concave geomorphic design of intrabench slopes, etc.

Mine Waste Settlement. Settlement of the mine waste backfill in Pit 4 will result in deformation and strains in the final cover, as well as induce lateral displacement and drag-down forces on the underdrain and waste dewatering risers if not accommodated. Deformation analyses were performed to estimate the amount of settlement that can be expected in the mine waste both during and after construction. Evaluations also were made of the impact of the deformation on the cover system and underdrain dewatering risers. Description of the methods, assumptions, and material properties used in the analyses, as well as the results of the analyses are provided in Attachment D-13.

Two-dimensional (2D) finite element analyses were performed along critical sections in Pit 4 to provide estimates of settlement for sections with significant variations in fill thickness over short horizontal distances. The 2D analyses were performed for initial settlement (occurring during construction) and long-term (post-cover construction) creep settlement. The lateral displacements of the underdrain and waste dewatering risers due to construction and long-term creep deformations of the mine waste also were estimated.

A summary of the settlement for the two critical sections analyzed for Pit 4 are provided in Table D-11. The creep settlement is based on estimates of the settlement that will occur within the first 50 years of completion of construction. The relationship of creep settlement with time

initially shows a rapid rate of settlement immediately after completion of construction. The rate of settlement then slows considerably about 10 years after final placement.

Table D-11 — Maximum Calculated Settlements in Pit 4 Mine Waste

Section	End of Construction (ft)	50 years (Creep) (ft)	Total (ft)
Pit 4 – Section C	2.2	3.7	5.9
Pit 4 – Section D	2.8	4.5	7.3

The calculated in-section lateral displacements in the dewatering risers are presented in Table D-12. In addition, the total estimated horizontal displacement and deviation of the dewatering risers from vertical were calculated and the results are also summarized in Table D-12.

Table D-12 — Maximum Calculated Lateral Displacements at the Pit 4 Risers

Section	In-Section Displacement			Resultant Displacement			
	EOC (ft)	50 years (Creep) (ft)	Total (ft)	EOC (ft)	50 years (Creep) (ft)	Total (ft)	Deviation from Vertical
Pit 4 – Section C	0.10	0.31	0.4	0.42	0.42	1.3	0.4%
Pit 4 – Section D	0.41	0.82	1.2				

Estimates of three-dimensional (3D) deformation for the cover system used the relationship developed in the 2D analysis between vertical creep settlement and the thickness of mine waste backfill. The maximum calculated geomembrane liner strains and differential settlements were estimated based upon the 3D distributions of settlement and are provided in Table D-13. The results the 3D deformations in the cover system also were used to evaluate the impact of long-term creep settlement along the relatively flat bench channels. Analyses of the Pit 4 cover and drainage bench designs prior to the 100% submittal indicates that post-settlement bed slopes along the drainage benches will exceed 0.5 percent at all locations, which exceeds the minimum slopes needed to convey stormwater from the WCA while providing sufficient freeboard against overtopping of drainage bench channels.

Table D-13 — Maximum Calculated Liner Strains and Differential Settlements for Pit 4 Cover

Max. Liner Strain (%)	Max. Differential Settlement (ft/ft)
0.42	0.021

The magnitude of the settlements listed in Table D-11 is considered reasonable for the evaluated loading conditions. Long-term maximum creep settlements of approximately 4.5 feet

in Pit 4 are anticipated; however the differential settlement between adjacent points will be much lower. Overall, the estimated long-term creep settlements will not result in significant changes to the cover geometry or flow directions of precipitation on the cover due to the relatively steep grades of the as-designed cover surface.

The amounts of lateral displacement of the underdrain and waste dewatering riser pipes due to construction loadings, and from long-term creep settlement will result in deflections of the risers of approximately 0.4 percent. These deviations from vertical are not sufficient to adversely impact the functioning of the risers as the predicted deviations from vertical are relatively minor and occur in a uniform manner with fill height.

Lateral deformations and strains that may develop within the cover system due to long-term creep also were estimated from the results of the 2D finite-element modelling and summarized in Attachment D-13. The estimated lateral deformations were used to evaluate the potential for long-term creep to induced excessive strains on the cover geomembrane. The results indicated that strains induced by post-construction creep will be significantly less than the maximum strain the geomembrane is able to withstand.

The maximum tensile strain developed in the geomembrane due to post-construction creep is estimated to be approximately two orders of magnitude lower than the specified break strain for the LLDPE geomembrane. As such, the longitudinal strains induced in the geomembrane liner by creep settlement are considered acceptable and will not cause failure of the liner.

The estimated lateral deformations also were used to evaluate the potential for post-construction creep to cause excessive slippage between geomembrane layers in the non-welded cover geomembrane overlap at the drainage benches, which could result in development of a gap in the cover system geomembrane layer. The results of the evaluation indicate that, due to the flexible nature and relatively high interface shear strength of the textured LLDPE geomembrane material selected for cover construction, very little slippage (less than one inch) is expected at the non-welded overlaps at the drainage benches. Therefore, the proposed 5-foot overlap in these areas is considered sufficient.

D6.8 PIT 4 GLOBAL STABILITY

The results of global stability analyses of the final backfilled Pit 4 configuration are in Attachment D-5. The analyses are focused on global slope stability of potential failure surfaces located at moderate to large depths in the mine waste and foundation layers. The stability of the

surficial cover system and potential for slope failures along cover interface elements are summarized in a separate attachment (Attachment D-7) and Section D10.5.

The proposed cover system will consist of a uniform soil layer overlying a geomembrane as shown on Drawing 4-83. On steeper sloped areas of mine waste (greater than 15 percent), the cover system also will include a GDL between the soil and geomembrane layers to reduce potential pore water build up in the slope and increase slope stability. The extent of the GDL is shown on Drawing 4-19. For the purposes of these global stability analyses, the soil cover was included, primarily to provide for a more complete accounting of weight forces, but localized, shallow failure surfaces within the cover layers were not considered in the global stability analyses. A 3-foot-thick cover soil layer was used in the analyses.

Criteria for minimum factors of safety for the stability of the final configurations for the mine WCA are specified in the CD SOW (EPA, 2011). These criteria include that a minimum factor of safety of 1.3 be maintained for static conditions and a minimum factor of safety of 1.0 be maintained under pseudo-static earthquake loading conditions. In addition, a post-earthquake analysis was made for the section selected since alluvial clays are thought to exist within the foundation. These clays likely are saturated and may experience strain-softening under earthquake loadings. In the post-earthquake analyses, the colluvium shear strength was modeled assuming (1) clay behavior, (2) sand behavior and (3) a conservative combination of sand-clay behavior. A minimum required factor of safety of 1.0 was selected as the design criteria for analysis of post-earthquake conditions.

Input parameters including section locations, sections geometries, material parameters, and seismic loading conditions are described in Attachment D-5. The results from the analyses of the backfilled Pit 4 containment area are summarized in Table D-14.

Table D-14 — Factors of Safety for Global Stability

Cross Section	Factor of Safety		
	Static	Pseudo-Static	Post-Earthquake
Design Criterion Minimum Factor of Safety	1.3	1.0	1.0
Pit 4 – Cross-Section 1	3.3	2.2	2.9 (clay behavior) 3.3 (sand behavior) 2.9 (sand-clay behavior)

These results indicate that the required minimum factors of safety for global stability are satisfied for the proposed final configuration at Pit 4 for the critical section that was analyzed.

D6.9 PIT 4 COVER SYSTEM

Typical surface cover details are shown on Drawings 4-83 through 4-94. The design criteria for the surface cover are common to all WCA. As such, the design calculations and details are described in Section D10. Specific design details for the tie-in of the Pit 4 surface cover into the Area 5 surface cover, to be completed later in Phase 2, is shown on Drawing 4-88.

D7.0 PHASE 2 –PIT 3 WASTE CONTAINMENT AND BPA GRADING

Regrading of the BPA, regrading of Area 5, and initial waste containment within Pit 3 will occur simultaneously and will result in a single contiguous, capped WCA. Therefore, with the exception of the groundwater dewatering systems in the BPA and Pit 3, all three work components are treated as a single entity. Below the construction activities follow a similar progression to those described in Section D6.0 and as a result, where construction elements and their designs are the same, the text below references previous discussions in Section D6.0.

D7.1 PIT 3 ROCKFALL PROTECTION

As discussed in Section D6.1, a Site visit and independent assessment of potential rockfall mitigation measures was performed by a specialist rockfall mitigation engineer/contractor. A recommended Rockfall Mitigation Plan was developed by the specialist and is included as Attachment D-11 to this appendix. The recommendations and conclusions for the designs for rockfall protection discussed for Pit 4 in Section D6.1 apply to Pit 3 as well.

D7.2 PIT 3 DEWATERING

Pit 3 will be dewatered prior to commencement of pit bottom cleanup. Peak accumulated water volumes in Pit 3 have historically ranged from 33 million gallons to 83 million gallons (WTP operating years 2001 through 2011), with an average peak storage of 58 million gallons. Based on these typical peak storage volumes, it is estimated that dewatering of Pit 3 could take approximately 80 days to complete. If necessary to meet the construction schedule, the Pit 3 dewatering time may be reduced considerably by drawing the water storage volume down in advance by extending the WTP operating schedule.

Water removed from Pit 3 during dewatering will be conveyed either directly to the WTP or to South Pond for intermediate storage prior to being conveyed to the WTP for treatment, depending on the WTP operating schedule at the time of dewatering.

D7.3 PIT 3 PIT BOTTOM CLEANUP AND GRADING

Sediments that have accumulated in the Bottom of Pit 3 were characterized as part of the field investigations performed for the *Geologic Investigations of Pits and Assessment of Pit Sediments Design Investigation Report* (MGC, 2011b). As part of those investigations, Pit 3 was dewatered to the point where only the lowest point of the pit bottom near the drop cut along the western pit wall remained underwater. It was estimated that approximately 3,300 cubic yards of sediment cover the Pit 3 bottom, approximately 3-inches thick in the pit floor and a somewhat thicker layer around the perimeter of the pit floor. The sediments were noted to be predominantly saturated, silt-sized material with coarser material occurring around the margins of the pit floor. In addition to the fine-grained sediments in the pit bottom, some gravel to boulder-sized materials are located in isolated piles on the pit floor. It is anticipated that additional coarse rock material will accumulate on the pit floor as a result of pit-slope rock scaling operations. Cleanup operations will be similar to those described for Pit 4 in Section D6.3. The bottom of Pit 3 will be graded in preparation for placement of drain material as shown on Drawing 4-37. An underdrain sump will be excavated by drilling and blasting in the low area located in northwestern portion of the pit bottom as shown on Drawing 4-37 and 4-79. Generally, the pit is sloped to drain to this area in its current configuration. As with Pit 4, areas where ponding may occur in the pit floor or otherwise will not flow by gravity toward the underdrain sump will be reworked to the extent possible without ripping, drilling or blasting.

Due to the nature of the rock formation in the pit bottom, aggressive reworking of the pit floor (e.g. ripping or blasting) to remove smaller irregularities (i.e. less than 2 feet high) that result in areas of ponding is likely to result in irregular rock breakout and creation of other areas of ponding. As a result, grading the pit bottom to a perfectly smooth, free-draining surface is considered unrealistic and unnecessary. Instead, if areas of ponding are noted during the jetting operation and can be removed by scraping the pit floor with the toothed bucket of a hydraulic excavator, this will be performed. In addition, the required minimum thickness of drainage layer in the pit bottom (discussed in the following section) will be maintained as measured above any high point or “sills” in the pit bottom that obstruct flow and create ponding.

Thus any areas of ponding will be "dead pools" that form below the required minimum thickness of drainage layer and will not affect drain capacity.

Material removed during grading and sump excavation will be stockpiled along with the coarse rock material removed during pit-bottom cleanup as described in an approved *Staging/Temporary Stockpiling Plan* (Appendix R), and will be replaced in Pit 3 during backfilling. Due to their in-situ proximity to mineralized areas, material removed during grading and sump excavation will not be placed in zones within the Pit 3 backfill that have been designated for low-activity/reactivity waste.

D7.4 PIT 3 UNDERDRAIN SYSTEM

An underdrain system constructed of non-reactive rock will be installed in the bottoms of Pit 3 to collect groundwater before it contacts reactive mine waste backfill in the pits. The Pit 3 underdrain system will be similar to the Pit 4 underdrain system described in Section D6.4 and those details are not repeated in this section. The configuration of the Pit 3 underdrain system is shown Drawings 4-38, 4-39, 4-40, and 4-41. Performance criteria 2.4.2.4.2 E.iii of the SOW in the CD (EPA, 2011) stipulates that "*The drainage layers shall extend vertically along the side walls of each pit to elevations determined during RD, to keep water entering the pits from contacting mine waste and to effectively channel water to the pit bottoms.*" Locations of pit wall seeps were mapped during late summer of 2010 as part of investigations for the *Geologic Investigations of Pits and Assessment of Pit Sediments Design Investigation Report* (MGC, 2011b) and additional seep mapping was performed in the spring of 2012 to provide data during wetter portions of the year (Plumley and Assoc., 2012). As communicated, both in the summary report and in the approved work plan for the spring 2012 seep mapping (MWH, 2012b): "*The specific objectives of the additional pit wall seep monitoring were to verify seeps previously identified in the Geologic Report; and identify new seeps in the pit walls above the existing water level that might be evident in the spring. Data gathered from this additional monitoring is intended to help define the positioning of drains that will be included in the remedial design for interception of pit wall seeps.*" The locations of seeps observed during these investigations are shown on Drawing 4-38 and the slope drain configurations shown on this drawing are extended up the pit walls to intercept these seeps. At a minimum, the slope drains as shown extend a distance of 25 feet vertically above, and 50 feet horizontally beyond, the limits of these seep areas. In areas where mine waste will be placed directly against drain rock for the slope drains

(see Drawings 4-38 and 4-44), only coarse-grained mine waste will be placed within 20-feet of the drain rock to protect from fines intrusion.

Details for the gradation requirements for the drain materials are developed in Attachment D-9 and summarized in Tables D-7 and D-8 in Section D6.4. As discussed in Section D6.4, the drain system has the capacity to conduct between 2.6 and 33.1 gallons per minute (gpm) per foot of drain width to the underdrain sump. Given the measured Pit 3 groundwater inflow rate ranged from 15.1 gpm to 19.9 gpm (MGC, 2011c), the underdrain system as designed has sufficient capacity in each 0.5 ft to 7.6 ft-wide section of underdrain to convey the entire estimated Pit 3 inflow to the underdrain sump at the minimum specified drain thickness. Therefore, the drain system placed over the bottom of Pit 3 will have much greater hydraulic capacity than the anticipated flows. The drain capacity on the pit slopes will be considerably higher due to the steeper gradients.

As shown in Section I on Drawing 4-39, the underdrain as currently designed has a thickness considerably greater than the specified 5-foot minimum thickness at the critical location in the vicinity of the underdrain sump. Thus the actual as-designed capacity will be much greater. Based on these considerations, it can be concluded that the underdrain system, as currently configured, will have much greater flow capacity than is required to convey the measured groundwater flow rates reported in the *Groundwater Investigations Design Investigation Report (*MGC, 2011c).

As with Pit 4, water shall be kept from accumulating in Pit 3 during and after consolidation of mine waste within the pit to the extent practical. Water accumulating in the underdrain sump during, and after construction, will be conveyed for treatment at the WTP by pumping through an underdrain sump dewatering system. The underdrain sump will be dewatered using submersible pumps located in dewatering risers that connect the sumps to the ground surface. Details of the dewatering riser design are described in Section D7.4. Similar to Pit 4, duplicate dewatering risers are proposed to avoid extended shutdowns in the dewatering system due to maintenance or mechanical failure during the RA and post-RA.

The underdrain configuration shown on the drawings provides approximately 370,000 gallons of storage (assuming an active porosity of 30 percent) between the top of the sump and the lowest point in the overlying subwaste liner. This allows for approximately 15 days of storage within the underdrain system at an average estimated inflow rate of 17.5 gpm for Pit 3 without

operating the underdrain dewatering system before the overlying bedding layer, liner system and waste rock are at risk of becoming saturated.

D7.5 BPA DEWATERING SYSTEM

The two main backfilled pits in the BPA (i.e., the Boyd Pit and the smaller, upgradient Pit 2) were dewatered as part of a BPA dewatering investigation. The general location of these two pits are shown on Figure 1-3 in the BODR. Wells located in the two smaller pits associated with the BPA, the Adit Pit and Pit 2 currently are dry. The dewatering investigation was conducted based on the approved work plan entitled, *Backfilled Pits Area Pumping Plan* (WME, 2013). This investigation provided information regarding the configuration of dewatering wells and water levels to be maintained in the permanent BPA dewatering system. The two existing dewatering wells, GW-54 (in the Boyd Pit) and GW-58 (in Pit 2) were used for BPA dewatering, with the groundwater from these wells being piped to Pit 3 for storage prior to treatment. These are the same wells that were used in a previous BPA dewatering investigation performed in 1999 and 2000 (SMI, 2001, URS, 2002). The locations of GW-54 and GW-58 are shown on Drawing 4-54.

The objectives of the investigation included:

1. Evaluating the operating condition of existing wells and equipment in Pit 2 and the Boyd Pit for use in initial dewatering of the BPA.
2. Initial dewatering in Pit 2 and the Boyd Pit to evaluate the effect of varying water levels and pumping schemes on groundwater water levels within the waste rock backfill and surrounding bedrock to provide optimal water levels and dewatering well configurations for long-term dewatering of the BPA.

The initial phase of the BPA investigation consisted of testing the condition of Wells GW-54 and GW-58 and the associated piping and pumping systems to verify that they were operational and could be used for dewatering during the investigation program. This phase was completed in July 2013 and confirmed that the two dewatering wells and the associated equipment were functional. Pumping then commenced in GW-54, located in the larger, downgradient Boyd Pit in September 2013. Dewatering of the Boyd Pit was conducted in stages, with the groundwater level in the pit drawn down to predetermined levels and held at these lower level for a minimum of two weeks in order to measure inflow rate into the pit while maintaining a constant water level. Drawdown to the final stage within approximately 5 feet of the pit bottom was completed

in late March 2014. Once the final drawdown stage in the Boyd Pit was attained, dewatering commenced in the smaller, upgradient Pit 2, while the pumping continued in the Boyd Pit. Dewatering of Pit 2 also was conducted in steps and the final drawdown level was attained in late June 2014 (at approximately 3 feet above the pit bottom). In accordance with the approved Work Plan, dewatering of the Boyd Pit and Pit 2 will be continued throughout the design process, the RA construction, and after construction.

The results of BPA dewatering have been presented in Data transmittal reports provided to EPA in Midnite Mine monthly reports beginning in August 2013 and are summarized in Attachment D-14. Generally, the results of BPA pumping confirm the conceptual model of the Site hydrogeology presented in the *Backfilled Pits Area Pumping Plan* (WME, 2013).

The BPA pumping investigation summary included as Attachment D-14 includes the following conclusions:

1. The water levels in wells surrounding the Boyd Pit and Pit 2 indicate the pits act as hydraulic sinks when dewatered. In addition, water levels indicate Pit 2 acts as a hydraulic sink even without active pumping.
2. Long-term base inflow rates into the Boyd Pit and Pit 2 appear to be lower than previously estimated. The long-term late season (2014) inflow rates were measured to be approximately 2.5 gpm into the Boyd Pit, and the 2.0 gpm into Pit 2.
3. Pumping from the Pit 2 (GW-58) dewatering well did not significantly affect the direction of hydraulic gradients in the vicinity of Pit 2. Evaluation of paired deep and shallow wells in Pit 2 indicates flow from the deeper bedrock aquifer toward Pit 2 whether or not GW-58 was operational. This is likely due to the water-level control provided by the hydraulic connection to the Boyd Pit once a groundwater level above approximately 2,759 ft is reached in Pit 2.
4. There is connection between the groundwater levels measured in bedrock wells BOM-3M/BOM-3D and the dewatering of the Boyd Pit. However, additional pumping from Pit 2 did not have a noticeable effect on groundwater levels in BOM-3M/BOM-3D. Neither pumping from the Boyd Pit, nor Pit 2 had a measureable effect on the shallow well BOM-3S.
5. Groundwater levels down gradient of the Boyd Pit at BOM-4 and GW-43 declined during the recent pumping campaign. Although it is possible this decline may be due to a decrease in the supply groundwater in the shallow buried alluvial channel down gradient

of the Boyd Pit after the groundwater level in the pit dropped below the overflow elevation (2,679 ft); it may also be the result of normal seasonal variations. Additional pumping from Pit 2 during the later stages of the BPA dewatering program did not affect the rate of decline in groundwater levels at BOM-4 and GW-43.

6. Operation of the pumpback system during the testing period and the measured water chemistry at GW-54 and GW-58 indicate a potential for the formation of scale and potential fouling of the dewatering system (pumps and pipes). Therefore, routine maintenance of the dewatering system is recommended to ensure efficient operation.
7. Based on the above observations and conclusions above, it appears that effective hydraulic control of the BPA groundwater system can be accomplished by pumping groundwater from the Boyd Pit only, and that pumping from Pit 2 can be discontinued. The volumetric rate of water capture when operating only the Boyd Pit pumping well (GW-54) at the later drawdown stages (6.6 gpm at elevation 2,662) was similar to the rate of capture when both the Boyd Pit and Pit 2 wells were operating, indicating little or no increase in capture efficiency by operating both well systems. This is consistent with the conceptual flow model based on BPA pre-backfill topography, which indicates water flowing from Pit 2 to the Boyd Pit.
8. It is recommended that an additional (redundant) dewatering well be installed in the Boyd Pit in the vicinity (within 25 feet) of the existing GW-54 dewatering well to provide a backup, and to limit the length of disruptions to operation of the dewatering system in the event of a catastrophic well failure.
9. The target operating water level range for the Boyd Pit dewatering system should be between 2,660 and 2,665 ft, which is sustainable as a groundwater level within this range was maintained for approximately 9 months during the BPA dewatering program in 2014.
10. It is estimated that the long-term, late-season dewatering rate (prior to capping of the BPA) will be approximately 4.5 gpm based upon the current combined flow rates from both the Boyd Pit and Pit 2. It is likely that the flow rates from the BPA dewatering system will decrease below this level once the area has been capped and isolated from infiltration of meteoric water above the BPA.

The permanent dewatering system for the BPA will incorporate GW-54 in the Boyd Pit for continued use in the dewatering system. In addition, a second backup dewatering well will be installed in the Boyd Pit to provide redundancy in the permanent dewatering system. This second well will be of similar in design and construction, including:

1. Installation in a 10.5-inch nominal borehole, drilled across the waste/bedrock interface
2. 6-inch, Schedule 40 PVC well casing, with the lower 40 feet being screened containing 0.020 inch milled slots.
3. Placement of 10-20 silica sand filter around the screened section to an elevation at least 10 feet above the top of well screen.
4. A bentonite seal above the sand filter to the ground surface.

The backup well will be installed after completion of regrading of the BPA in the vicinity of GW-54 early in Phase 3 or RA construction. It is anticipated that the backup well will be installed within approximately 25 feet of the GW-54 along the GW-54 access road as shown on Drawings 4-54 and 4-55. It is also recommended that GW-58 be maintained as part of the permanent BPA monitoring system to verify acceptable control of groundwater levels in Pit 2.

D7.6 PIT 3 SUB-WASTE LINER SYSTEM

Prior to placement of mine waste in Pit 3, a sub-waste liner will be installed over the underdrain system as shown on Drawings 4-40 and 4-41. The function of the sub-waste liner system is described in Section D6.5. Details of the proposed liner system are shown on Drawing 4-81 and discussed in Section D6.5 and D9.0. The liner subgrade (i.e. underdrain surface) will be shaped as shown on Drawing 4-40. The proposed grading for the sub-waste liner shown on Drawing 4-40 will provide for gravity drainage of water that collects on the liner surface toward a waste dewatering sump located at the low-point of the liner surface. The waste dewatering sump will be dewatered by pumping from risers located within the sumps as discussed in Section D7.7.

D7.7 PIT 3 WASTE ROCK DEWATERING SYSTEM

The proposed Pit 3 waste rock dewatering system is shown on Drawings 4-40, 4-44, and 4-82. The design and operation of the waste dewatering system is similar to that described for Pit 4 in Section D6.6 and those details are not repeated in this section. It is anticipated that flow rates to Pit 3 sub-waste dewatering sump will be highest during construction and that once the surface cover system is in place over the Pit 3 waste, the rates of water migration through the waste

rock will diminish to very low levels as discussed in Section D10.0. As currently configured, the waste dewatering sump, with an estimated backfill hydraulic conductivity in the range of $7x10^{-1}$ to $9x10^{0}$ cm/sec and an effective sump radius of approximately 30 feet, will have much greater flow capacity than is required to convey the long-term infiltration rates estimated in the infiltration analyses summarized in Section D10.3.

D7.8 PHASE 2 PIT 3 MINE WASTE PLACEMENT/BPA REGRADING

Once the sub-waste liner has been installed in Pit 3, the pit and the BPA will be ready to receive mine wastes from various sources at the Site. In general, Pit 3 will contain most of the lower activity/reactivity waste sources at the Site listed in Table D-6. The configuration of Pit 3 as shown on the drawings, and the associated volumes listed in Table D-6 are for the maximized storage volume. As such, the waste storage capacity in the Pit 3/BPA WCA (15.4 million cy) is much larger than the current waste volume estimate (11.1 million cy), and the current configuration should be able to accommodate unanticipated wastes encountered during the final stages of the RA.

The configuration of the Pit 3 backfill upon completion of Phase 2 is shown on Drawings 4-42, 4-44 and 4-45. As discussed in Section D7.0, regrading of the BPA, regrading of Area 5, and waste containment in Pit 3 will occur simultaneously and will result in a single contiguous, surface cover. Therefore, BPA regrading, Area 5 regrading, and Pit 3 backfilling are treated as a single work component.

The configurations shown on the drawings reflect surface grading details such as drainage benches, concave, geomorphic design of intrabench slopes, etc.

As proposed, the waste placement sequencing in Pit 3 during Phase 2 backfill operations will be as follows:

1) Sump drain rock in underdrain sump (D7.4)
2) Underdrain gravel layer (D7.4)
3) Sub-waste liner bedding (1.5" D_{max}) layer (D7.6)
4) Geofabric for geomembrane cushion layer (D7.6)
5) Synthetic geomembrane sub waste liner (D7.6)
6) Fine-grained (1/4" D_{max} or finer) soil liner cushion layer (D7.6)
7) Reject material from HSWRP
8) Remainder of HSWRP material
9) Pit 3 rockfall debris and pit-bottom grading spoils
10) Sediments from cleanup of Pit 3 bottom
11) Remainder of South Dump Material from the Western Drainage
12) Contaminated soil cleanup from the Western Drainage
13) Western Drainage Sediment Cleanup
14) Existing WTP and other demolition debris
15) East Waste Rock Pile materials
16) Contaminated soil cleanup from East Waste Rock Pile area
17) Eastern and Far East Drainage Sediment Cleanup
18) East Access Road Cleanup
19) Other internal mine roads and contaminated soil cleanup

These mine wastes will be hauled to Pit 3 in mine haul trucks or off-road dump trucks, dumped, and spread in horizontal lifts with maximum thickness of 10 feet. No additional compaction of the waste material is planned beyond that which results from spreading and incidental traffic over the waste surface by construction equipment.

D7.9 AREA 5 REGRADING

The area located between the uphill (northern) edge of Pit 3 and the downhill (southern) edge of Pit 4 is referred to as Area 5. This area is currently covered with waste rock with a relatively flat surface area that will be used for processing and stockpiling of drain rock material during Phases 1 and 2 of RA. Prior to mining, an alluvial channel cut through this area, sloping in a southerly direction, and into Pit 3. If waste rock were removed from this area to the pre-mine surface, as shown on Figure 12-1 of the ROD, an area of ponding with no surface drainage outlet would be formed immediately upstream of the Pit 3 WCA. This area of ponding would almost certainly result in seepage from into Pit 3. As a result, the plan for Area 5 is based upon grading this area to achieve a mounded, free-draining surface and covering it in a manner consistent with other waste areas (i.e. geomembrane cap and revegetated soil cover).

The configuration of the Area 5 surface upon completion of regrading is shown on Drawings 4-46 and 4-47. The surface cover will be tied into the Phase 3, Pit 3 cover and the Phase 1, Pit 4 cover as shown on Drawing 4-88. The configurations shown on the drawings reflect surface grading details such as drainage benches, concave, geomorphic design of intrabench slopes, etc.

D7.10 PIT 3 PHASE 2 COVER SYSTEM

Upon completion of Phase 2 waste placement in Pit 3, a surface cover will be installed to the extent practical over those portions of the Pit 3 area where additional waste placement or regrading will not be occurring. This cover will serve to reduce the infiltration of meteoric water through the waste materials. As discussed in Section D7.8, the configuration of the Phase 2 Pit 3 backfill shown on Drawings 4-42, 4-44, and 4-45 reflects surface grading details such as drainage benches, concave, geomorphic design of intrabench slopes, etc. The cover design is described in Section D10.0. Specific design details for the tie-in of the Phase 2 - Pit 3 surface cover into the Pit 3 surface cover to be completed later in Phase 3, are shown on Drawing 4-92.

D8.0 PHASE 3 –PIT 3 WASTE CONTAINMENT

D8.1 PHASE 3 PIT 3 MINE WASTE PLACEMENT

The configuration of the Pit 3 backfill upon completion of Phase 3 is shown on Drawings 4-55, 4-59 and 4-60. The configurations shown on the drawings reflect surface grading details such as drainage benches, concave, geomorphic design of intrabench slopes, etc. The waste

containment capacity of the configuration shown for Pit 3 at the end of Phase 3 is approximately 15,400,000 cubic yards.

As proposed, the waste placement sequencing in Pit 3 during Phase 3 backfilling will be as follows:

1) Debris from South Pond demolition.
2) Remainder of SWRP material and PCP in Central Drainage.
3) Central Drainage soil and sediment cleanup.
4) Other contaminated soil cleanup.

These wastes will be hauled to Pit 3 in either mine haul trucks or off-road dump trucks, dumped, and spread in horizontal lifts with maximum thickness of 10 feet. No additional compaction of the waste material is planned beyond that which results from spreading and incidental traffic over the waste surface by construction equipment.

Mine Waste Settlement. As discussed in Section D6.7, settlement of the mine waste backfill results in deformation and strains in the final cover, as well as induces lateral displacement and drag-down forces on the underdrain and waste dewatering risers if not accommodated for in design and construction. Deformation analyses were performed for the Pit 3 mine waste using the same methods and assumptions presented in Section D6.7 for Pit 4.

A summary of the settlement for the two critical sections analyzed for Pit 3 are provided in Table D-15.

Table D-15 — Maximum Calculated Settlements in Pit 3 Mine Waste

Section	End of Construction (ft)	50 years (Creep) (ft)	Total (ft)
Pit 3 – Section A	4.6	6.4	11
Pit 3 – Section B	5.7	7.2	13

The calculated in-section lateral displacements in the underdrain and waste dewatering risers are provided in Table D-16. In addition, the total estimated horizontal displacement and deviation of the dewatering risers from vertical were calculated and the results are also summarized in Table D-16.

Table D-16 — Maximum Calculated Lateral Displacements at the Pit 3 Risers

Section	In-Section Displacement			Resultant Displacement			
	EOC (ft)	50 years (Creep) (ft)	Total (ft)	EOC (ft)	50 years (Creep) (ft)	Total (ft)	Deviation from Vertical
Pit 3 – Section A	0.96	1.7	2.7	0.97	1.9	2.8	0.6%
Pit 3 – Section B	0.13	0.78	0.9				

The maximum calculated geomembrane liner strains and differential settlements are provided in Table D-17. Analyses of the current (100% Design) Pit 3 cover and drainage bench design indicates that post-settlement bed slopes along the drainage benches will exceed 0.5 percent at all locations. This exceeds the minimum bed slope needed to convey stormwater from the WCA while providing sufficient freeboard against overtopping of drainage bench channels.

Table D-17 — Maximum Calculated Liner Strains and Differential Settlements for Pit 3 Cover

Max. Liner Strain (%)	Max. Differential Settlement (ft/ft)
0.33	0.027

The magnitude of the settlements listed in Table D-15 is considered reasonable for the evaluated loading conditions. Long-term maximum creep settlements of approximately 7.2 feet in Pit 3 are anticipated, however the differential settlement between adjacent points will be much lower. Overall, the estimated long-term creep settlements will not result in significant changes to the cover geometry or flow directions due to the relatively steep grades of the majority of the as-designed cover surface.

The amounts of lateral displacement of the underdrain and waste dewatering riser pipes due to construction loadings, and from long-term creep settlement will result in deflections of the risers of approximately 0.6 percent. These deviations from vertical are not sufficient to adversely impact the functioning of the risers as the predicted deviations from vertical are relatively minor and occur in a uniform manner with fill height.

Lateral deformations and strains that may develop within the Pit 3 cover system due to long-term creep were estimated in a manner similar to that described for Pit 4 in Section 6.7 and are summarized in Attachment D-13. Like the Pit 4 analyses, analyses of lateral deformations and differential settlement indicate that deformations will not result in excessive strains developing

within the cover geomembrane system, nor will they result in unacceptable slippage and separation at the non-welded overlaps at the drainage benches.

D8.2 PIT 3 GLOBAL STABILITY

The results of global stability analyses of the final backfilled Pit 3 configuration are included in Attachments D-5. The global stability analyses are focused on potential failure surfaces located at moderate to large depths in the mine waste and foundation layers. The stability analyses of the surficial cover system and potential for slope failures along cover interface elements (veneer stability) are presented in a separate attachment (Attachment D-7) and summarized in Section D10.5.

The proposed cover system will consist of a uniform soil layer overlying a geomembrane as shown on Drawing 4-83. On steeper sloped areas of mine waste (greater than 15 percent), the cover system will also include a GDL between the soil and geomembrane layers to reduce potential pore water build up in the slope and increase slope stability. For the purposes of these global stability analyses, the soil cover was included, primarily to provide for a more complete accounting of weight forces, but localized, shallow failure surfaces within the cover layers were not considered in the global stability analyses. A 3-foot-thick cover soil layer was used in the analyses.

Criteria for minimum factors of safety for the stability of the final configuration of the WCA are specific in the CD SOW (EPA, 2011). These criteria include that a minimum factor of safety of 1.3 be maintained for static conditions and a minimum factor of safety of 1.0 be maintained under pseudo-static earthquake loading conditions. In addition, a post-earthquake analysis was made for one section (Section 3) where it appears that alluvial clays may exist within the foundation. These clays likely are saturated and may experience strain-softening under earthquake loadings. In the post-earthquake analyses, the colluvium shear strength was modeled assuming: (1) clay behavior, (2) sand behavior and (3) a conservative combination of sand-clay behavior. A minimum required factor of safety of 1.0 was selected as the design criteria for analysis of post-earthquake conditions.

Input parameters including section locations, sections geometries, material parameters, and seismic loading conditions are described in Attachment D-5. The results from the analyses of the backfilled Pit 3 containment area are summarized in Table D-18.

Table D-18 — Factors of Safety for Global Stability

Cross Section	Factor of Safety		
	Static	Pseudo-Static	Post-Earthquake
Pit 3 – Cross-Section 2	2.9	2.0	-NA
Pit 3 – Cross-Section 3 (top of slope	3.0	2.1	3.0(clay behavior) 3.0 (sand behavior) 3.0 (sand-clay behavior)
Pit 3 – Cross-Section 3 (toe of slope)	2.3	2.0	2.6 (clay behavior) 1.7 (sand behavior) 1.6 (sand-clay behavior)

N/A – not applicable, as no alluvium clays are present in this section.

These results indicate that the required minimum factors of safety for global stability are satisfied for the proposed final configuration at Pit 3 at both critical section locations that were analyzed.

D8.3 PHASE 3 COVER SYSTEM

A surface cover will be placed over the remaining uncovered WCA upon completion of Phase 3 waste relocation and regrading activities. The configuration of the Phase 3 surfaces shown on Drawings 4-55, 4-59, and 4-60 reflect surface grading details such as drainage benches, concave, geomorphic design of intrabench slopes, etc. Details of the cover design are presented in Section D10.0. Specific design details for the tie-in of the Phase 3 - Pit 3 surface cover into the Pit 3 surface cover that was completed in Phase 2, and the Pit 3 cover tie-in to the Area 5 cover area shown on Drawing 4-86.

D8.4 PHASE 3 DEWATERING SYSTEM

During Phase 3, French-drain style infiltration collectors will be installed at the toe of the waste backfill slopes in two locations as shown on Drawing 4-57. The native ground surface at these two locations slopes toward the edge of the waste containment cover. The purpose of the infiltration collectors is to reduce the potential for head buildup beneath the liner at the waste/native ground contact in these two locations. Water in these collectors will drain by gravity to dewatering sumps located outside of the limits of waste containment cover. Water removed from the infiltration collectors will be conveyed to the WTP, either via the storage ponds or directly to the WTP, depending on WTP operating conditions at the time of removal.

D9.0 SUB-WASTE LINER/DEWATERING SYSTEM DESIGN

Sub-waste liners will overlie the drain rock in Pit 3 and Pit 4 as described in sections D6.5 and D7.6 and as shown on the drawings. The sub-waste liners in both pits have common design elements with common design considerations, which are summarized in this section.

The sub-waste liner will be placed over the underdrain and will serve to separate the overlying mine waste from the underdrain. The upper layer of the underdrain will consist of a minimum 1-foot thick layer of liner bedding material. The liner bedding material will consist of gravel processed from the HSWRP with a maximum particle size of 1.5 inches, and placed as the final lift on the surface of the drain rock to provide padding of the synthetic layers that follow. The required gradation for the liner bedding material is summarized on Table D-9 and the criteria used for development of this specification are summarized in Attachment D-9.

The proposed sub-waste liner will consist of a:

1) Geofabric cushion layer placed over the liner bedding layer to provide additional puncture protection for the overlying geomembrane liner.
2) Geomembrane liner constructed from 80-mil HDPE geomembrane. HDPE was selected as the geomembrane material due to its resistance to degradation when exposed to a wide range of chemicals and durability under severe loading conditions.
3) Over-liner protection layer, consisting of a 3-foot minimum thickness of fine-grained material, with a maximum particle size of ¼ inches, in order to protect the geomembrane from damage due to waste loading or construction equipment.

Specific geomembrane design considerations including chemical compatibility of the selected geomembrane material with Site waters, and puncture resistance under anticipated waste loadings are discussed in the following sections.

D9.1 CHEMICAL COMPATABILITY CONSIDERATIONS

A review of the compatibility considerations between the HDPE geomembrane proposed for the sub-waste liner to chemical and radioactive degradation is summarized in Attachment D-2.

Water from two monitoring wells in the BPA (GW-54 and GW-58) are assumed to be representative of some of the most aggressive Site waters that might be in contact with the HDPE geomembrane liner and was therefore assumed in the evaluation.

Based on the results of water quality testing from the BPA wells, the primary constituents that may potentially have a detrimental effect on the proposed HDPE geomembrane are sulfuric acid and sulfates in the leachate solution. The values for these water quality parameters measured during design investigations in 2010 (MGC, 2011c) are:

- pH of approximately 3.5 to 4.0, and
- Sulfates concentrations of approximately 12,500 milligrams per liter (mg/L).

Available chemical resistance information for HDPE indicates that, at these concentrations, sulfuric acid and sulfates will not damage the liner. In addition, the HDPE geomembrane that has been proposed for the sub-waste liners will not be exposed to temperatures greater than about 20 degrees Celsius. Koerner (2005) lists HDPE as having “generally good resistance” to inorganic acids and salts at temperatures ranging from 38 to 70 degrees Celsius. Manufacturer’s literature notes that non-oxidizing acids and salts have little to no effect on an HDPE geomembrane (Poly-Flex, 2010). The literature also indicates that there is no mechanical or chemical degradation at sulfuric acid concentrations up to 50 percent and in high-sulfate solutions at temperatures ranging from 20 to 60 degrees Celsius (GSE, 2012). Based on review of available information, no measurable chemical degradation of the HDPE materials is expected for many hundreds of years.

Attachment D-2 cites a number of sources in the literature that document studies illustrating the compatibility of HDPE geomembrane liner material with acidic process solutions. Most of these studies are associated with municipal and industrial landfills; however, three studies that specifically address the compatibility of HDPE geomembranes with mine waste solutions containing low-pH mine water are discussed in Attachment D-2. All of these studies indicate that any chemical effects on the HDPE subwaste liner material properties would take many hundreds of years to occur.

Radioactive degradation of the 80 mil HDPE liner is not expected to be a concern based on Site-specific gamma measurements that indicate upper bound radiation absorption for the geomembrane of 3000 rads over 1,000 years. This is significantly less than the 10^6 to 10^7 rads noted to be the lower bound for the start of polymer degradation. Additional discussion is provided in Attachment D-2.

D9.2 STRESS CONDITIONS/PUNCTURE PROTECTION

The sub-waste geomembrane liner protection requirements are evaluated in Attachment D-1. The potential for liner puncture and need for a geomembrane cushion layer are evaluated for liner bedding materials with angular particles and a maximum particle size (D_{max}) of 1.5 inches.

The results of the liner puncture design calculations are summarized in Table D-19. Table D-19 presents the required mass per unit area of geotextile cushioning for the maximum fill heights in Pit 3 and Pit 4 based upon the current detailed grading plan.

Table D-19 — Summary of Linear Puncture Protection Requirements

Location	Maximum Fill Height, Including Cover(ft.)	Geotextile Required for Puncture Protection
Pit 4	260	20 oz./s.y. NW GT
Pit 3	436	32 oz./s.y. NW GT

NW GT = nonwoven geotextile

The required protection for an 80-mil HDPE geomembrane consists of a geotextile with a minimum unit density of 20 oz./yd^2 in Pit 4 to 32 oz./yd^2 in Pit 3 for a geomembrane overlying bedding material with a maximum particle size (D_{max}) of 1.5-inches. A geotextile cushion layer is not required when the geomembrane is in contact with fine-grained soil containing no material larger than ¼-inch in size such as the overliner protection layer.

D10.0 COVER SYSTEM DESIGN

Surface covers will be placed over Pit 4, Pit 3, the BPA, and Area 5 as described in Sections D6.9, D7.10, and D8.3, and as shown on the Section 4 Drawings in Volume II. The proposed cover system in all locations will consist of a:

1) Three-foot continuous soil cover layer without a separate topsoil or growth media layer based on an evaluation of Rhoads Borrow Area soil properties presented in the Revegetation Plan (Attachment D-12). These analyses indicate the Rhoads' soils will provide a suitable plant growth media and no further amendment or additional topsoil is necessary.

2_ Geomembrane Layer. The geomembrane for the cover system will be 40 mil LLDPE with a textured top surface or similar.

3) GDL on areas of mine waste that are sloped at steeper than a 15 percent grade. The GDL will be installed between the soil and geomembrane layers to reduce potential pore water build up in the slope to enhance slope stability.

LLDPE geomembrane material was selected for the surface cover system due to the flexibility of the material, which allows the material to undergo large strains without damage. This material was also selected based on the interface shear strength with the proposed cover borrow soil and with a GDL. The design details and calculations presented in this section are common for the entire WCA at the Site.

D10.1 COMPATIBILITY CONSIDERATIONS

A review of the compatibility of the LLDPE geomembrane proposed for the cover system to chemical and radioactive degradation is included in Attachment D-2.

Chemical compatibility for the cover geomembrane is less of a concern than for the liner system geomembrane. As noted in the *EPA (Draft) Technical Guidance For RCRA/CERCLA Final Covers* (EPA, 2004)*: "It is important that the requirements of a GM for a liner system not be confused with requirements for a cover system. In a typical liner system application, the GM is exposed to leachate and subjected to relatively high normal stresses. Replacement or repair of the GM after waste placement is not typically possible.*"

Leachate exposure for the cover geomembrane will be in the form of condensate on the bottom side of the geomembrane. The upper side of the geomembrane will have exposure to only meteoric water. The condensate that the cover geomembrane will be exposed to is expected to be much less aggressive than the leachate from the BPA described in Section D9.1.

Radioactive degradation of the LLDPE is not expected to be a concern based on site specific gamma measurements that indicate upper bound radiation absorption for the geomembrane of 3000 rads over 1,000 years. This is significantly less than the 10^6 to 10^7 rads noted to be the lower bound for the start of polymer degradation.

D10.2 RADON MODELING

Attachment D-3 summarizes the radon modeling that was performed as part of the surface cover design. The performance standard used for cover design is the U.S. Nuclear Regulatory Commission (NRC) average long-term radon emanation standard of 20 picocuries per square meter per second (pCi/m^2-sec). The proposed surface cover system consists of a uniform soil

material overlying a geomembrane above mine waste with a relatively low activity concentration of radium-226. The geomembrane was not included in the analyses. Radon modeling was performed for the selected borrow source for the cover material (Rhoads Property Borrow Area).

The thickness of the reclamation cover needed to limit radon emanation from the backfilled pits was analyzed using the NRC RADON model (NRC, 1989). The model utilizes the one-dimensional radon diffusion equation, which uses the physical and radiological characteristics of the mine waste and overlying materials to calculate the rate of radon emanation through the cover. The model was used to calculate the cover thickness required to limit the radon emanation rate through the top of the cover to 20 pCi/m^2-s, following the guidance presented in NRC publications NUREG/CR-3533 (NRC, 1984) and Regulatory Guide 3.64 (NRC, 1989). The rate of emanation standard is applied to the average emanation over the entire surface of the backfilled pits.

Model input parameters, including soil and waste properties are discussed in detail in Attachment D-3. The results of radon modeling are shown on Table D-20. Included with these results is the thickness for the cover soil needed to reduce the rate of radon emanation to values below the limit of 20 pCi/m^2-s averaged over the entire WCA surface. The results also indicate that a cover thickness of 1.8 feet of Rhoads Property borrow material will be sufficient to limit radon emanation acceptable levels. However, it is recommended that a minimum cover thickness of 3.0 feet be used based upon slope stability, liner protection, and construction considerations.

Table D-20 — Summary of Radon Modeling Results

Model Parameters	Model Layer	
	Layer 1 Mine Waste	Layer 2 Cover
Porosity	0.38	0.38
Specific Gravity	2.83	2.63
Dry Density (g/cc)	1.76	1.63
Radium-226 Activity Concentration (pCi/g)	32	0
Emanation Coefficient	0.35	0.35
Long-Term Moisture Content (%)	5.4	10.4
Calculated Radon Diffusion Coefficient (cm^2/sec)	0.0296	0.0142
Required Cover Thickness (cm)	----	53 (1.8 ft)

D10.3 INFILTRATION ANALYSIS

This section summarizes the results of analyses of infiltration through the cover system proposed for the Site. Details of the infiltration analyses are provided in Attachment D-4. These analyses were conducted to evaluate percolation (leakage) through the cover system under:

1) As-constructed conditions with the geomembrane intact
2) Long-term degraded conditions when the geomembrane has degraded and is no longer effective at limiting percolation.

For the as-constructed case, percolation through the cover system and into the underlying waste materials was assumed to occur through small defects in the geomembrane. For the infiltration analyses, the WCA cover was divided into three distinct areas depending on surface geometry and cover system components as listed in Attachment D-4. Separate percolation calculations were made for each of these areas, and a composite, area-weighted percolation rate was calculated for the entire cover system.

In order to evaluate upper-bound percolation conditions in the extreme long-term for degraded liner conditions, analyses were made assuming the geomembrane and GDL no longer exist or have no effect on the hydraulics of the cover system and percolation would flow unimpeded through the bottom of the soil cover. As with the analyses of as-constructed conditions, the WCA cover was subdivided based upon surface drainage conditions as listed in Attachment D-4. The composite percolation rate through the cover system for the upper-bound long-term degraded conditions was then calculated as the sum of the weighted area leakage rates.

Infiltration modelling was performed using one-dimensional numerical models to provide input for the cover leakage calculations. Runoff was incorporated in the 1D model for sloped surfaces by not allowing surface water to pond on the climate boundary (ground surface) if the 24-hour precipitation exceeded the 24-hour infiltration rates. The development of the numerical model is described below. Recommendations provided in NUREG/CR-7028 (Benson et al., 2011) and Albright et al. (2010) for modeling water balance covers were followed where applicable. Infiltration analyses were conducted using the computer program Vadose/W (Geo-Slope International, Ltd, 2012). Vadose/W is a finite-element-based program that can be used to model movement and distribution of pore water within porous material. Model input data including climate data, soil properties, and vegetation properties are presented in detail in Attachment D-4.

Infiltration simulations were performed for typical and wet period climate conditions. Typical conditions were represented by modeling one typical year, with analyses started using the initial conditions developed as described in Attachment D-4. Wet year conditions were analyzed based upon the third of three consecutive wet years following a typical year, and using initial conditions developed as described in Attachments D-4. The results are presented below.

The cumulative percolation through the cover for as-constructed conditions are presented in Table D-21. Sensitivity simulations included cases to evaluate the influence of cover thickness on calculated infiltration rates. The results show that the cover system as designed and under as-constructed conditions will reduce infiltration to 0.015 percent or less of annual precipitation under both typical and wet year climate conditions. As discussed in Attachment D-4, the majority of the percolation occurs in the relatively flat drainage bench areas where flow concentrations and surface ponding may occur. As a consequence, a secondary low-permeability barrier in the form of a GCL has been included in the design details for the drainage bench channel bottoms as shown on Drawing 4-85.

Table D-21 — Infiltration Model Results for As-Constructed Conditions

Cover Thickness (ft)	Percolation through cover (mm/yr)		Percolation as Percent of Annual Precipitation (%)	
	Typical Year Climate Conditions	Wet Period Climate Conditions*	Typical Year Climate Conditions	Wet Period Climate Conditions*
2	0.030	0.109	0.006	0.017
3	0.016	0.097	0.003	0.015
6	0.052	0.168	0.011	0.026

*Results are for the final model year of three consecutive wet years following a typical climate year.

As an upper bound (i.e., worst case) estimate for percolation through the cover system under long-term conditions, the cover system was modeled assuming the geosynthetic layers will completely degrade and no longer have any effect on the hydraulic characteristics of the cover system (i.e. the effects of the geomembrane and GDL layers are completely ignored). The results for these analyses and summarized in Table D-22. The results show that a 3-foot cover system using Rhoads Property borrow soils (which is the designed thickness) will reduce infiltration to approximately 0.7 percent of annual precipitation under typical year, and approximately 2.3 percent of annual precipitation under wet year climate conditions.

Table D-22 — Infiltration Model Results for Long-Term Degraded Conditions

Cover Thickness (ft)	Percolation (mm/yr)		Percolation as Percent of Annual Precipitation (%)	
	Typical Year Climate Conditions	Wet Period Climate Conditions*	Typical Year Climate Conditions	Wet Period Climate Conditions*
2	5.90	16.41	1.25	2.51
3	3.09	15.17	0.66	2.32
6	2.00	14.19	0.42	2.17

*Results are for the final model year of three consecutive wet years following a typical climate year.

D10.4 EROSIONAL STABILITY

The results of erosional stability analyses for the cover system proposed for the RA Construction at the Midnite Mine are presented in this section. Details of the analysis procedures and input parameters are presented in Attachment D-6.

The most critical slopes for evaluating erosional stability are the steepest interbench slopes with the longest uninterrupted slope runs. The proposed side slope geometry for the cover slopes includes drainage benches at a 50-foot vertical spacing. The steepest intrabench slopes will be 3H:1V. The drainage benches will be shaped to provide a drainage channel on the bench surface and prevent over-crest runoff.

Erosional stability analyses were performed for the borrow source selected for cover material, the Rhoads Property Borrow Area. Material properties used in the analysis for these soils are included in Attachment D-6.

Erosional stability of the cover was evaluated for the 100-year, 24-hour storm event (EPA, 2011). The Rational Method as outlined in WDOE (2004) was used to calculate peak flows from the design storm.

The erosional stability of vegetated slopes was evaluated using the methods recommended in NRC (1990) and Johnson (2002). Temple et al. (1987) outlines procedures for grass-lined channel design. It is assumed that the soil covers will not be erosionally stable, and that repair and maintenance will be required if the design storm event occurs prior to vegetation being established on the soil cover. Therefore, the stresses were only evaluated for the condition where vegetation has been established. The erosional stability of the cover surfaces was evaluated by calculating a factor of safety against erosion due to the peak runoff from the 100-year, 24-hour storm event. The surfaces were evaluated for two conditions: 1) resistance of the

vegetation, and 2) resistance of the cover system soil layer. The peak unit discharge flow was conservatively multiplied by a flow concentration factor of three.

Calculated factors of safety for erosional stability are presented in Table D-23. Calculated factors of safety less than 1.0 are an indication that a specific failure mode (either soil erosion or vegetation loss) can be expected during the design storm event. For the cover slopes, the calculated factors of safety show that for established vegetation conditions, slopes will be erosionally stable during peak discharge from the 100-year, 24-hour storm event.

Table D-23 — Factors of Safety for Erosion Protection of Cover

Interbench Slope	Description of Erosion Protection	Factor of Safety for Soil on Vegetated Slope	Factor of Safety for Vegetation
3H:1V	Vegetation and Top Soil	16.4	1.1

These analyses indicate that cover slopes constructed as vegetated slopes without rock for erosion protection will be erosionally stable once vegetation has been established. The calculated factors of safety for both soil erosion and vegetation loss are above 1.0.

Soil loss estimates from the surface covers for sheet flow were estimated for a 1,000-year period using Version 2 of the Revised Universal Soil Loss Equation (RUSLE2) as summarized in Attachment D-6. The vegetation conditions assumed for the 1,000-year time period varied from bare ground for the initial two years, to cool season grasses with poor stand for the remaining years. The parameters used in, and results of, the analyses are summarized in Table D-24. The results show that the expected surficial soil loss is not significant and is calculated to be less than one inch over the 1,000-year period analyzed.

Table D-24 — Summary of RUSLE2 Model Parameters and Results

Model Parameter	Value
Soil Texture	Sandy Loam
Climate	Stevens County, WA Annual Precip.18 – 20 inches
Cover Slope (%)	33
Cover Slope Length (ft)	171
Vegetation Conditions	
Initial two years	bare ground
2 to 1,000 years	cool season grasses (poor stand)
Soil loss for bare ground conditions with rough surface (tons/acre/year)	14.00
Soil loss for cool season grasses (poor stand) vegetation conditions (tons/acre/year)	0.12
Soil loss (inches/1,000 years)	**0.74**

D10.5 VENEER STABILITY

This section summarizes the results of slope stability (veneer) analyses that were conducted for the cover system that will be placed over the mine waste as part of the RA at the Site. This analysis is presented in Attachment D-7.

The cover system will consist of a uniform soil layer overlying a synthetic geomembrane. On steeply sloped areas (greater than 15 percent slopes), the cover system will also include a GDL between the soil and geomembrane layers. The GDL layer was added based on results of veneer stability analyses that indicated that stability is not satisfactory for steeper slopes if significant positive pore pressure develops above the geomembrane. The borrow source selected for the cover material, Rhoads Property Borrow Area, was evaluated.

Analyses were performed for: 1) drained conditions under static and pseudo-static loading, and (2) saturated conditions under static loading. For drained conditions, the GDL is assumed to have adequate capacity to preclude the development of positive pore pressures on the geomembrane liner and within the cover soil. The longest 3H:1V interbench cover slope was selected for evaluating the drained conditions. For saturated conditions, the slope angle resulting in a factor of safety of 1.3 was back calculated under static conditions assuming the cover soil is fully saturated. The back calculation of the slope angle for saturated conditions was used to estimate the slope angle at which the slope would become unstable if a GDL layer is not included and the soil cover becomes fully saturated.

The failure (sliding) surface is assumed to occur along the weakest interface, which corresponds to the surface with the lowest interface shear strength. Consequently, when a GDL is included in the cover system, the failure surface is assumed to occur along the GDL and geomembrane interface based upon the previous test results (MGC, 2011a) and experience on other projects that indicate the interface strengths for the mine soil-geosynthetic interfaces will be higher than for the GDL-geomembrane interface.

Analyses of stability in areas where a GDL is included in the cover system were used to back-calculate the minimum interface shear strength (as represented by an angle of interface friction and no adhesion) needed to meet project slope stability criteria. The results for required interface strength can then be used to evaluate the suitability of various geomembrane liner and GDL combinations in terms of required interface shear strength. The results of the analyses are presented in Table D-25. The project requirements for minimum factors of safety are 1.3 under static conditions and 1.0 under pseudo-static conditions as outlined in EPA (2011). For drained conditions, the minimum interface frictions angles needed to meet the project factor of safety requirements are 22.0° and 23.0° for static and pseudo-static conditions, respectively.

Table D-25 — Factors of Safety for Veneer Stability

Failure Surface	Back-Calculated Minimum Required Interface Friction Angle	
	Peak (used for static loading conditions, FS = 1.3)	Post-Peak (used for pseudo-static loading conditions, FS = 1.0)
GDL to Geomembrane Interface	22.0	23.0

For saturated conditions, the steepest slope that will still result in a factor of safety of 1.3 under static loading is approximately 9 degrees (16 percent). As a result, the cover system in areas steeper than 15 percent will include a GDL to prevent pore water buildup and increase slope stability. Top cover slopes which have flatter slopes (less than 15 percent) will not require a GDL except for the top cover slope of Pit 3 where there is a drainage swale and concentrated flows may occur. The extents of GDL coverage over the Pit 4 and Pit 3 WCA are shown on Drawings 4-19 and 4-58 respectively.

GDL/Geomembrane interface strength testing was conducted, using site specific soils as substrate and superstrate in the test setup, as part of the 60% design to measure interface

strength parameters for the specific materials considered for construction. The results of the testing are summarized in Attachment D-7 and in Table D-26.

Table D-26 — Results for Measured Interface Friction Angles for Materials

Failure Surface	Friction Angle	
	Peak	Post-Peak
PVC/Single Fabric-Sided Geocomposite	18.3	16.3
PVC/Double Fabric-Sided Geocomposite	21.0	21.6
LLDPE/Single Fabric-Sided Geocomposite	18.5	15.9
LLDPE/Double Fabric-Sided Geocomposite	34.0	25.4

Based on the testing results, the interface friction angle for a LLDPE geomembrane/double fabric-sided drainage geocomposite does meet the minimum required interface friction angle to needed to satisfy veneer stability design criteria. Testing was also conducted for the interface between a 40 mil LLDPE Agru Super Gripnet Liner and geotextile. Although the test results indicate significant curvature in the failure envelop, which precludes simplified interpretation of interface strength using a friction angle, the test results indicate that a 40-mil LLDPE Agru Super Gripnet Liner/geotextile interface will meet the minimum shear strength requirements for veneer stability over the range of potential cover loadings. Additional discussion on the testing results is provided in Attachment D-7. Neither the PVC geomembrane tested with a single and double-fabric faced GDL nor the textured LLDPE geomembrane tested with a single-fabric-faced GDL met the interface strength criteria.

D10.6 REQUIRED GEOCOMPOSITE (GDL) CAPACITY

This section summarizes the seepage transmission capacity requirement for the GDL that is to be placed over steeper sloped surfaces of the cover system for the RA Construction at the Midnite Mine.

The design method used for estimating the acceptable transmissivity of a geocomposite drain is presented in Koerner (2005). This method was used in conjunction with the peak weekly percolation rate into the GDL at the base of the soil cover calculated as part of the infiltration analysis (Attachment D-4). The results present an upper bound value for the required GDL capacity. The peak weekly percolation rate calculated from the infiltration analysis is 29.73 millimeters/day. Two slope sections were evaluated and included: (1) the longest length for the steepest slope; and (2) the longest length for slopes steeper than 15 percent. As surface ponding is unlikely for either of these slope configurations, these GDL capacity analyses are

considered to be conservative. Table D-27 lists the minimum recommended laboratory measured transmissivity values for the GDL.

Table D-27 — Summary of Recommended Minimum Laboratory Transmissivity Results

Slope	Gradient	Cover Loading (psf)	Recommended Minimum Laboratory Measured Transmissivity, θ_{ult} (m^2/s), for GDL
5.4H:1V	0.2	< 1000	9.3×10^{-4}
3H:1V	0.3	< 1000	3.4×10^{-4}

m^2/s = square meters per second; GDL = geocomposite drainage layer

D11.0 REVEGETATION OF DISTURBED AREAS

The revegetation approach for the Site during the RA is described in the Revegetation Plan provided as Attachment D-12. The Revegetation Plan includes:

- A description of the borrow source soil
- Where and where not additional soil will be necessary off the WCA
- Revegetation techniques in various disturbed area
- Appropriate seed mixtures and the use of shrub and tree seedlings
- Necessary inspections and maintenance, along with a weed management plan.

The Revegetation Plan divides the Site revegetation into four distinct areas based primarily on the slope angle that will be revegetated using different approaches. The areas described in the plan include the WCA, flat-lying disturbed areas, steeper areas that are greater than 3:1 (h:v) but flatter than 2:1 (h:v), areas steeper than 2:1 (h:v), and downstream drainages. Each area of these areas has distinct revegetation approaches related to soil layers, plant species, and mulching techniques that are recommended to establish self-sustaining native plant communities and meet the proposed land use goals of suitable wildlife habitat and traditional land uses.

D12.0 SURFACE WATER AND STORMWATER MANAGEMENT AND CONTROLS

The Mine Waste Excavation and Containment design provides for capture and treatment of mine-impacted surface water and stormwater, and for clean water to be shed away from contaminated areas. Appendix F and Section 6 of the drawings in Volume II describe the Surface Water and Sediment Controls, which will be used to shed clean water away from contaminated areas at the end of each of the three major phases of RA construction. The Master SWMP (included in Appendix O) identifies BMPs that will be applied to reduce the adverse impacts of stormwater and specific sediment control measures that will be employed before, during, and after construction for both sediment and stormwater control. The RA Contractor will be required to prepare a CSWPPP that presents the stormwater management protocol and procedures that are specific to the phased construction activities. The RA Contractor's CSWPPP will reference the Master SWMP for general stormwater management practices and will identify the BMPs that are applicable to the scheduled construction activities. The CSWPPP will be updated on an annual basis, at a minimum, and will describe the intermediate phases and temporary facilities to be employed in storm water and surface water management as construction progresses.

To the extent practical, above-grade mine waste excavations and excavation of underlying contaminated soils will be conducted beginning with upstream areas within each drainage and working in a downstream direction, with the working excavation areas being shaped to retain surface water runoff. In areas where this is not possible, other BMPs will be utilized to minimize the transport of potentially contaminated sediments from the work areas by surface water runoff. With a few specific exceptions (e.g., sediment cleanup within drainages) work will not occur within surface water bodies. To the maximum extent practical, sediment cleanup within drainages will be conducted within drier parts of the year (summer and early autumn) to avoid unnecessary impacts to surface water bodies.

Mine-impacted sediments captured in the stormwater BMPs will be characterized and, if above sediment cleanup levels, consolidated with the mine wastes. Maintenance and monitoring requirements for surface and stormwater controls is described in the Operations Maintenance and Monitoring Plan (OM&M Plan) in Appendix P.

D13.0 GREEN AND SUSTAINABLE REMEDIATION CONSIDERATIONS

The GSR considerations for the Mine Waste Excavation and Containment activities are presented below. GSR considerations were evaluated for: 1) Construction Materials (characteristics and manufacturing considerations), 2) Construction Methods, and 3) Low Impact/Sustainability measures undertaken during construction.

D13.1 CONSTRUCTION MATERIAL CONSIDERATIONS

The sub-waste liner and cover liner materials were carefully selected based on chemical compatibility and protection from punctures and stress/strain conditions. These characteristics will help ensure long-term viability and environmental protection.

The Rhoads Property Borrow Area soils were selected as the borrow source for the cover material because the fine-grained material: 1) promotes vegetation growth, 2) minimizes the final cover thickness based on radon emanation evaluations (which in turn reduces the number of truck loads to cover the Site), and 3) provides an erosionally stable cover material (which likely will require less long-term maintenance).

The most significant GSR opportunity regarding the cover material borrow source is limiting fuel consumption and greenhouse gas emissions associated with necessary truck haul distances. The Rhoads Property Borrow Area is located adjacent to the RA (where the covers will be installed). The proximity of the Rhoads Property Borrow Area significantly reduces the fuel required to transport the cover soils to the Site thereby significantly reducing the greenhouse gas emissions. The total number of truckloads necessary to cap the Site has been minimized by selecting the Rhoads Property Borrow Area soils versus other borrow sources that would likely require a thicker cover. Rhoads Property soil also provides a more favorable growth medium for plants and is less erosive than the other borrow sources evaluated, all of which support GSR principles.

D13.2 CONSTRUCTION METHODS

Mine waste will generally be excavated, transported, and placed in the mine WCA in the pits in a continuous operation, without stockpiling excavated material. Additionally, the excavation and hauling equipment used will be appropriately sized. These methods minimize the double handling of excavated materials, greenhouse gas emissions, fugitive dust generation, and erosion.

The proposed use of suitable material from the on-site HSWRP for the drainage layers underlying Pit 3 and Pit 4 prevents excavation and hauling of 515,000 cy of suitable material from an off-site borrow area. The benefits in reduced vehicle traffic, greenhouse gas emissions, fugitive dust generation, and erosion are substantial. The use of on-site materials also prevents habitat destruction at an off-site borrow location.

The proposed phasing of construction activities will avoid the recontamination of remediated areas. In addition, mine waste excavations will be completed beginning at the upstream end of the drainages and continue in a downstream direction thereby resulting in “clean” drainages. The added benefit of excavating in this manner is that any precipitation that falls during the work will be contained at the working face of the excavation and avoid contaminating downstream locations. Sediment cleanup within drainages will be conducted within the drier parts of the year to avoid unnecessary impacts to surface water bodies and wildlife.

Dust suppression will be utilized in the work areas and on the access roads to decrease visible dust related emissions. On-Site vehicle speeds will be restricted to accommodate safe roadway conditions based on roadway grade, roadway soil conditions, roadway congestion, and the need to limit air emissions caused by roadway fugitive dust. Dust suppressant water used for the construction and excavation activities likely will be taken from the WTP effluent, thus significantly reducing the need to import water to the Site from great distances. Dust suppressant additives likely will be used on semi-permanent access roads or haul roads, subject to prior EPA approval.

Construction workers will be instructed to avoid engine idling and using machinery with automatic idle-shutdown devices will be suggested. Ultra-low sulfur diesel will be used in excavation and hauling equipment as well as support vehicles.

The SWMP (included in Appendix O) identifies BMPs and specific sediment control measures that will be employed before, during, and after construction for both sediment and storm water control. The Surface Water and Sediment Controls will be used to shed clean water away from contaminated areas thereby reducing the volume of mine-impacted water requiring treatment and will contain contaminated water within the contaminated areas preventing recontamination of remediated areas.

D13.3 LOW IMPACT DEVELOPMENT/SUSTAINABILITY

A thoughtful approach was taken to optimize the route of the access roads from the excavation location to the disposal location to minimize Site disruption and vehicle mileage.

Maintaining a single point of entry/exit to the MA helps prevent re-contamination of areas already remediated or contamination of areas that were previously uncontaminated. This single point of entry/exit also minimizes the required support facilities and associated infrastructure.

Carpool locations in Wellpinit, Ford, and Spokane for worker transportation to and from the MA during excavation and construction activities will reduce traffic to and from the Site, fugitive dust generation, gasoline and diesel use, and greenhouse gas emissions.

Areas cleared of mine waste will be graded to conform to the pre-mining topography to restore the natural pre-existing landscape to the extent practical while meeting reuse goals. Mimicking rather than altering the Site's natural setting will improve the cover's long-term performance and protect local ecosystems. Revegetation efforts with an approved, native seed mix will commence promptly after excavation and construction activities are complete to restore habitat, improve infiltration, and reduce soil erosion.

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Attachment D-1

Liner Puncture Protection

ATTACHMENT D-1

MIDNITE MINE REMEDIAL ACTION LINER PUNCTURE PROTECTION

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*19-Nov-12*	*30% Design*	*Melanie Davis*	*Clint Strachan*	*19-Nov-12*	*Clint Strachan*
1	*02-Dec-13*	*60% Design*	*Melanie Davis*	*Jason Cumbers*	*02-Dec-13*	*Clint Strachan*
2	*16-May-14*	*90% Design (title update only)*	*Melanie Davis*	*Jason Cumbers*	*16-May-14*	*Clint Strachan*
2	*13-May-15*	*100% Design – no changes*				

Location and Format

Electronic copies of these calculations are located in the project files system at:

\\usftc2s01\Projects\Newmont\Midnite Mine_2011\6.0 Studies & Reports\6.2 Technical\Remedial Action Liner Design\Liner Puncture Protection\60 % Design

The following calculations were generated using the following software:

Microsoft Office Professional Plus 2010 (Microsoft Excel)

Supplement

Supplement D-1.1 – Liner Puncture Protection Calculations

1.0 BACKGROUND

This calculation brief has been prepared to evaluate the requirements for a cushion layer beneath and/or above the sub-waste 80 mil HDPE geomembrane liner for Pits 3 and 4. The sub-waste geomembrane liner will be placed over the pit-bottom underdrain layer as part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). It is expected that the pit-bottom underdrain will consist of angular, gravel-sized material (drain gravel). It is proposed that uppermost layer of the underdrain will consist of a finer-grained gravel (liner bedding), on which the geomembrane will be placed. This finer-grained gravel bedding layer will have a specified maximum particle size of 1.5 inches. It is currently proposed that the sub-waste geomembrane liner will be covered with a layer of 0.25-inch-minus soil (if available) as an overliner protection layer prior to placement of mine waste in the pits.

For waste containment facilities, a geotextile cushion typically is recommended for HDPE (high density polyethylene) geomembrane liners when the liner bedding or cover material has a maximum particle size of greater than 0.25 inch (Daniel and Koerner, 2007). In comparison, a geotextile cushion is typically not used to protect geomembrane liners for heap leach facilities with heights up to 300 feet (Theil and Smith, 2003) due to the sloped surfaces often encountered the heap-leach pad footprints and concerns over introducing a potentially weak sliding surface along the cushion-geomembrane interface. Data presented in Theil and Smith indicates a geomembrane liner under high loads may not require a cushioning fabric. For the specific application for the Midnite Mine sub-waste liner, it was assumed that a geotextile would be incorporated under the geomembrane as a cushion layer, consistent with the recommendations and procedures recommended discussed in Section 1.1. If a geotextile cushion layer is not included over the underdrain bedding layer, then material testing should be performed to evaluate puncture resistance using the specific materials proposed for use at the site.

This attachment presents the geotextile puncture protection requirements using an empirical method, where liner bedding material under the liner will have a specified maximum size of 1.5 inches. It is anticipated that soils used for construction of the overliner cushion layer will have a maximum size of 0.25 inches and that an overliner geotextile cushion layer will not be necessary if this material is available.

1.1 METHODS

The design methods presented in Wilson-Fahmy et al. (1996), Narejo et al. (1996), and Koerner et al. (1996), and summarized in Koerner (2005) was used to evaluate the requirements for a geotextile cushion layer beneath the sub-waste geomembrane liner for puncture protection. The following equations were used to calculate the required thickness of the geotextile to cushion a geomembrane.

$$FS = p_{allow}/p_{reqd}$$

$$p_{allow} = (50 + 0.00045\frac{M}{H^2} - 130000(1.5 - t)(H \text{ x } 1000)^{-2.4})[\frac{1}{M_{FS} \text{ x } M_{PD} \text{ x } M_{FA}}][\frac{1}{RF_{CR} \text{ x } RF_{CBD}}]$$

where

p_{allow} = allowable pressure on the geomembrane (kPa)
M = geotextile mass per unit area (g/m^2)
H = protrusion height (m)
t = geomembrane thickness (mm)
M_{FS} = modification factor for protrusion shape
M_{PD} = modification factor for packing density
M_{FA} = modification factor for arching in solids
RF_{CR} = reduction factor for long-term creep
RF_{CBD} = reduction factor for long-term chemical/biological degradation

$$p_{reqd} = H * \gamma$$

where

p_{reqd} = overburden pressure (KN/m^2)
H = height of fill, (m)
γ = total unit weight of fill, (kN/m^3)

The recommended factor of safety for HDPE geomembrane puncture protection presented in Koerner (2005) is 3.0. The geomembrane thickness that was evaluated is 80 mil (2 mm). The equation provided above for allowable pressure includes an adjustment to the allowable pressure for varying geomembrane liner thicknesses based on equations presented in Narejo et al. (1996) and Narejo and Corcoran (2002). The protrusion height is estimated as equal to half of the specified maximum diameter (D_{max}) of 1.5 inches. The modification and reduction factors were selected from tables presented in Koerner (2005) and Narejo et al. (1996) and included consideration of the compatibility of the HDPE liner to potential degradation mechanisms. The liner bedding layer is expected to consist of angular particles, and the modification factor for protrusion shape reflects this assumption. The overburden pressure was calculated based on the maximum thickness of mine waste to be placed in Pits 3 and 4 of 436 and 260 feet, respectively, and included the maximum cover soil thickness of 3 feet that may be required. The total unit weight of the mine waste was estimated based on densities presented in EPA (2002) for the South Dump. The total unit weight of the cover material used in the calculations is based on laboratory testing results for the Rhoads Property Borrow Area (MGC, 2011a, b). The placed densities were estimated based on: (1) the average laboratory-measured maximum standard Proctor dry densities (rock correction applied where applicable); (2) average measured in-situ moisture contents for collected samples from the borrow area; and (3) assuming the cover materials will be placed at approximately 85 percent of standard Proctor compaction.

1.2 RESULTS

Calculations to evaluate the weight of geotextile cushion required beneath the sub-waste geomembrane liners in Pit 3 and Pit 4 were performed using the equations presented in Section 1.1. Calculation sheets are provided in Supplement D-1.1 and include the input values selected for each case evaluated. Table 1 presents the required weight per unit area of geotextile cushioning beneath the geomembrane for a maximum D_{100} of 1.5 inches. The required protection beneath an 80 mil HDPE geomembrane under the anticipated waste loadings ranges from a geotextile with a unit weight of 20 to 32 ounces per square yard (oz/yd^2) for Pit 4 and Pit 3, respectively. It is not anticipated that a geotextile cushion layer will be necessary above the

geomembrane due to the fine-grained (minus 0.25-inch) nature of the soils proposed for the overliner cushion layer.

Table 1. Summary of Liner Puncture Protection Requirements

Location	Assumed Maximum Fill Height, Including Cover (ft)	Geotextile Required for Puncture Protection
Pit 4	260	20 oz/yd^2 NW GT
Pit 3	436	32 oz/yd^2 NW GT

NW GT = nonwoven geotextile

If it is determined that a geotextile cushion layer is not desirable beneath the geomembrane due to concerns over the potential for sliding on sloped surfaces or other reasons, it may be possible to omit this layer. However, further testing with the specific materials proposed for use in construction of the sub-waste liner and underdrain systems should be performed using procedures that have been developed for heavily-loaded heap-leach pads (Theil and Smith, 2003) to evaluate the potential for liner puncture without a cushion layer. In addition, if material larger than 0.25-inch-minus soil is selected for the overliner protective layer, liner puncture will be evaluated for this material.

2.0 REFERENCES

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Supplement D-1.1

Liner Puncture Protection Calculations

Client:	Dawn Mining Company/Newmont USA Limited	Job No.:	1011322
Project:	Midnite Mine	Date:	12/2/2013
Description:	Remedial Action Liner Design	Computed By:	M. Davis
Detail:	Liner Puncture Protection Calculations	Checked By:	J.Cumbers

Objective: Determine puncture protection required for 80 mil HDPE subwaste liner in Pits 3 and 4

Summary of Puncture Protection Required

Location	Fill Height, including cover (ft)	Puncture Protection
Pit 4	260	20 oz/sy NW GT
Pit 3	436	32 oz/sy NW GT

NW GT = nonwoven geotextile

Client:	Dawn Mining Company/Newmont USA Limited	Job No.:	1011322
Project:	Midnite Mine	Date:	12/2/2013
Description:	Remedial Action Liner Design	Computed By:	M. Davis
Detail:	Liner Puncture Protection Calculations	Checked By:	J. Cumbers

Objective: Dete Determine puncture protection for 80 mil HDPE liner to be placed in Pit 4

Method: Use procedures presented in Narejo et al. (1996, 2002) and Koerner (2005)

Equations: $FS = p_{allow}/p_{reqd}$

$p_{allow} = (50 + 0.00045\ M/H^2 - 130000*(1.5-t)*(H*1000)^{-2.4})*[1/(MF_S \times MF_{PD} \times MF_A)]*[1/(R_{FCR} \times RF_{CBD})]$

p_{allow} = allowable pressure (kPa)
M = geotextile mass per unit area (g/m^2)
H = protrusion height (m)
t = geomembrane thickness (mm)
MF_S = modification factor for protrusion shape
MF_{PD} = modification factor for packing density
MF_A = modification factor for arching in solids
RF_{CR} = reduction factor for long-term creep
RF_{CBD} = reduction factor for long-term chemical/biological degredation

$p_{reqd} = H_f * \gamma$

p_{reqd} = overburden pressure (KN/m^2)
H = height of fill, (m)
γ = total unit weight of fill, (kN/m^3)

Minimum Recommended Factor of Safety	3	from Koerner (2005)

p_{reqd}, (KN/m^2)

Unit weight of mine waste, γ_{mw} (pcf)	116	Estimate for waste rock based on testing results on South Dump samples presented in EPA (2002)
Maximum thickness of mine waste, H_{mw} (ft)	257	From 60 percent design
Unit weight of cover soil, γ_{cs} (pcf)	110	From test results from MGC (2011a, b)
Maximum design height of cover soil, H_{cs} (ft)	3	From 60 percent design
p_{reqd}, (psf)	30,142	calculated
p_{reqd}, (kPa)	1,443	calculated

p_{allow}, (KN/m^2)

Geomembrane thickness, t (mm)	2	80 mil HDPE
Geotextile mass per unit area, M (g/m2)	675	*Iterate until factor of safety is greater than 3.0.*
Protrusion height, H (m)	0.01905	Dmax/2 (1.5 inches/2) for liner bedding material, equal to 19.05 mm (0.01905 m)
Modification factor for protrusion shape, MF_S	1	From Table 5.18 in Koerner (2005) for angular particles
Modifcation factor for packing density, MF_{PD}	0.6	From Table 5.18 in Koerner (2005) for dense packing and protrusion height between 12 and 25 mm
Modification factor for arching in solids, MF_A	0.25	From Table 5.18 in Koerner (2005) for deep geostatic pressure
Reduction factor for long-term creep, RF_{CR}	1.25	Interpolated from Table 5.28 in Koerner (2005) and Table 12 from Narejo, et. al. (1996) for d_{50} between 12 and 25 mm for 810 g/m^2 geotextile
Reduction factor for long-term chemical/biological degredation, RF_{CBD}	1.1	From Table 5.18 in Koerner (2005) for mild leachate
p_{allow} (kPa)	4,568	calculated
Factor of Safety = p_{allow}/p_{reqd}	3.2	calculated

References:

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Client:	Dawn Mining Company/Newmont USA Limited	Job No.:	1011322
Project:	Midnite Mine	Date:	12/2/2013
Description:	Remedial Action Liner Design	Computed By:	M. Davis
Detail:	Liner Puncture Protection Calculations	Checked By:	J. Cumbers

Objective: Det Determine puncture protection for 80 mil HDPE liner to be placed in Pit 3

Method: Use procedures presented in Narejo et al. (1996, 2002) and Koerner (2005)

Equations: $FS = p_{allow}/p_{reqd}$

$p_{allow} = (50 + 0.00045\ M/H^2 - 130000*(1.5-t)*(H*1000)^{-2.4})*[1/(MF_S \times MF_{PD} \times MF_A)]*[1/(R_{FCR} \times RF_{CBD})]$

- p_{allow} = allowable pressure (kPa)
- M = geotextile mass per unit area (g/m^2)
- H = protrusion height (m)
- t = geomembrane thickness (mm)
- MF_S = modification factor for protrusion shape
- MF_{PD} = modification factor for packing density
- MF_A = modification factor for arching in solids
- RF_{CR} = reduction factor for long-term creep
- RF_{CBD} = reduction factor for long-term chemical/biological degredation

$p_{reqd} = H_f * \gamma$

- p_{reqd} = overburden pressure (KN/m^2)
- H = height of fill, (m)
- γ = total unit weight of fill, (kN/m^3)

Minimum Recommended Factor of Safety 3 from Koerner (2005)

p_{reqd}, (KN/m^2)

Unit weight of mine waste, γ_{mw} (pcf)	116	Estimate for waste rock based on testing results on South Dump samples presented in EPA (2002)
Maximum thickness of mine waste, H_{mw} (ft)	433	From 60 percent design
Unit weight of cover soil, γ_{cs} (pcf)	110	From test results from MGC (2011a, b)
Maximum design height of cover soil, H_{cs} (ft)	3	From 60 percent design
p_{reqd}, (psf)	50,558	calculated
p_{reqd}, (kPa)	2,421	calculated

p_{allow}, (KN/m^2)

Geomembrane thickness, t (mm)	2	80 mil HDPE
Geotextile mass per unit area, M (g/m2)	1100	assumed
Protrusion height, H (m)	0.01905	Dmax/2 (1.5 inches/2) for liner bedding material, equal to 19.05 mm (0.01905 m)
Modification factor for protrusion shape, MF_S	1	From Table 5.18 in Koerner (2005) for angular particles
Modifcation factor for packing density, MF_{PD}	0.6	From Table 5.18 in Koerner (2005) for dense packing and protrusion height between 12 and 25 mm
Modification factor for arching in solids, MF_A	0.25	From Table 5.18 in Koerner (2005) for deep geostatic pressure
Reduction factor for long-term creep, RF_{CR}	1.15	Interpolated from Table 5.28 in Koerner (2005) and Table 12 from Narejo, et. al. (1996) for d_{50} between 12 and 25 mm for 1100 g/m^2 geotextile
Reduction factor for long-term chemical/biological degredation, RF_{CBD}	1.1	From Table 5.18 in Koerner (2005) for mild leachate
p_{allow} (kPa)	7,742	calculated

Factor of Safety = p_{allow}/p_{reqd} 3.2 calculated

References:

Koerner, R. 2005. Designing with Geosynthetics. 5th Edition.

Narejo, D., and Corcoran, G. 2002. Geomembrane Protection Design Manual. First Edition. March.

Narejo, D., Koerner, R.M., and Wilson-Fahmy, R.F., 1996. Puncture Protection of Geomembranes, Part II Experimental. Geosynthetics Intl., vol. 3, no. 5, pp 629-653.

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Attachment D-2

Cover and Liner Geomembrane Compatibility

ATTACHMENT D-2

MIDNITE MINE REMEDIAL ACTION
COVER AND LINER GEOMEMBRANE COMPATIBILITY

Revisioning					
Rev.	**Date**	**Description**	**By**	**Reviewed**	**Date**
0	*19-Nov-12*	*30% Design*	*Melanie Davis*	*Tom Kelley*	*23-Nov-12*
1	*26-Nov-13*	*60% Design*	*Melanie Davis*	*Tom Kelley*	*09-Dec-13*
2	*16-May-14*	*90% Design*	*Melanie Davis*	*Clint Strachan*	*16-May-14*
2	*14-May-15*	*100% Design – no changes*			

Location and Format
Electronic copies of this memorandum and pertinent references are located in the project files system at: *\\usslc1s01\IFO\Industrial Projects\MIDNITE MINE\Deliverables_Working Documents\Basis of Design Rpt\Appendix D - Mine Waste Excavation and Containment\90% Submittal\Att D-2 Geomembrane Compatibility*

1.0 GEOMEMBRANE COMPATIBILITY

This memorandum evaluates the long-term compatibility of the liners proposed for use in Pit 3 and Pit 4 as part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The proposed liners include the sub-waste 80 mil high density polyethylene (HDPE) geomembrane liner, and the 40 mil linear low density polyethylene (LLDPE) geomembrane for the cover system. Evaluation of a 40 mil polyvinyl chloride (PVC) geomembrane has been removed from this memorandum since this material is no longer being considered for use in the cover system. Interface-friction testing results indicate a cover system that includes a PVC geomembrane will not meet veneer stability on steep slopes (see Attachment D-7).

Potential long-term aging and degradation mechanisms have been considered for the geomembranes, depending on the type of polymer and environmental exposure. Potential degradation mechanisms include: chemical degradation, ultraviolet degradation, biological degradation, degradation due to swelling, degradation by extraction, thermal degradation, oxidative degradation, and radioactive degradation, (Koerner, et al., 1990; Koerner, 2005). The potential degradation mechanisms evaluated for the geomembranes proposed for use in the liner and cover systems as part of the RA include chemical degradation, oxidative degradation and radioactive degradation. These mechanisms are discussed in the following sections.

1.1 CHEMICAL DEGRADATION

1.1.1 Liner System Geomembrane

Groundwater monitoring wells GW-54 and GW-58 in the existing Backfilled Pits Area (BPA) are expected to be representative of the most aggressive leachate (i.e., low pH and high sulfates) that might contact the sub-waste HDPE geomembrane liner in the Pit 3 and Pit 4. The constituents that were evaluated as potentially detrimental to the liner are sulfuric acid and sulfates in the leachate solution. Based on the results of water quality testing in wells GW-54 and GW-58 (presented in the *Groundwater Investigations Design Investigation Report* [MGC, 2011]), the lower bound expected for the pH of the leachate solution is 3.5 (representing sulfuric acid), and the maximum concentration for sulfates is estimated as approximately 12,500 milligrams per liter (mg/L).

Available chemical resistance information indicates that HDPE is resistant to sulfuric acid and sulfates. Table 5.8 in Koerner (2005) lists HDPE as having “generally good resistance” to inorganic acids and salts at temperatures ranging from 38 to 70 degrees Celsius. Reactions leading to geomembrane degradation are generally accelerated at higher temperatures. It is anticipated that temperatures within the backfilled pits will be below the lower end of this temperature range. Poly-Flex (2010) technical literature notes that non-oxidizing acids and salts have little or no effect on a HDPE geomembrane. In addition, GSE Lining Technology, LLC (GSE, 2012) technical literature notes that HDPE is resistant to sulfuric acid and shows no mechanical or chemical degradation at concentrations up to 50 percent and at temperatures ranging from 20 to 60 degrees Celsius. GSE (2012) technical literature also lists HDPE as showing no mechanical or chemical degradation to sulfates in soluble solutions at temperatures ranging from 20 to 60 degrees Celsius.

There are a number of sources that document studies illustrating the compatibility of HDPE geomembrane liner material with acidic process solutions. Numerous studies have been conducted on the effect of various solutions on geomembranes primarily associated with municipal and industrial landfills. There are limited studies that have been conducted to evaluate the effect of mine waste leachates on geomembranes. Two of these studies are discussed below.

- Mitchell (1985) performed geomembrane chemical compatibility tests with simulated uranium mill process solution for three types of geomembranes: HDPE, CSPE (chlorosulfonated polyethylene), and PVC. The simulated solution consisted primarily of water and sulfuric acid at pH values ranging from 1.5 to 2.5. The HDPE geomembrane samples used for the testing consisted of a section of 40 mil HDPE geomembrane which included a fillet-welded field seam. Temperatures used during the testing ranged from 18 to 76 degrees Celsius. The results of the testing indicated that the acid process solution was "not very aggressive with any of the materials or seams [tested]." The HDPE geomembrane performed better and was more stable than the other geomembranes; however, the CSPE and PVC performed satisfactorily.

- Gulec, et al. (2004 and 2005) presented results of chemical resistance to short-term degradation of mechanical and polymer properties of a 60 mil HDPE geomembrane using a synthetic acid mine drainage (AMD) solution and an acidic solution. The AMD solution included iron, zinc, and copper, as well as sulfates (4,500 mg/L). Sulfuric acid was added to the AMD solution to obtain a pH of 2.1. The acidic solution consisted of deionized water with sulfuric acid added to obtain a pH of 2.1. The temperature range used for the study was 20 to 60 degrees Celsius. Testing was performed over a 22 month period. The results of the study indicate that the HDPE geomembrane is resistant to short-term degradation of mechanical properties for the solutions used in the study. In regards to the short-term degradation of the polymer properties of the geomembrane, this study showed little effect on the polymer properties, except with regards to oxidation induction time. The testing results indicated a decrease in the oxidation induction time with exposure time for all exposure conditions. This general trend is similar to results for studies conducted on geomembranes and leachate from municipal waste landfills.

1.1.2 Cover System Geomembrane

Chemical compatibility for the cover geomembrane is less of a concern than for the sub-waste liner system geomembrane. As noted in the *EPA (Draft) Technical Guidance For RCRA/CERCLA Final Covers:* **"**It is important that the requirements of a GM [geomembrane] for a liner system not be confused with requirements for a cover system. In a typical liner system application, the GM is exposed to leachate and subjected to relatively high normal stresses. Replacement or repair of the GM after waste placement is not typically possible" (EPA, 2004). Leachate exposure for the cover geomembrane will be in the form of condensate on the bottom side of the geomembrane. The upper side of the geomembrane will have exposure to only meteoric water. The condensate that the cover geomembrane will be exposed to is expected to be much less aggressive than the leachate from the BPA described in Section 1.1.1.

The polymeric compound of LLDPE is the same as HDPE. The main difference between LLDPE and HDPE is that LLDPE has a lower density and lower crystallinity than HDPE

(Koerner et al., 2005). The lower density of the LLDPE results in reduced chemical resistance and allows for oxygen to diffuse more quickly into the polymer structure. However, LLDPE is less susceptible to stress cracking and is able to sustain more strain (Lupo and Morrison, 2005). The LLDPE geomembrane to be used on the cover system is expected to be chemically compatible with the condensate from the mine waste. Poly-Flex (2010) technical literature notes that non-oxidizing acids and salts have little or no effect on a LLDPE geomembrane. Kay et al. (2004) notes that polyethylenes are highly resistant to chemicals and acidic agents, due to their non-polar nature. The condensate is expected to be much less aggressive than the leachate used in the testing by Mitchell (1985) and Gulec et al. (2004 and 2005) discussed in Section 1.1.1. Although this testing was conducted on a HDPE geomembrane, the results indicate that a LLDPE geomembrane would be compatible with a less aggressive leachate such as is expected for the condensate (i.e. higher pH and lower sulfate concentration).

1.2 OXIDATIVE DEGRADATION

1.2.1 Liner System Geomembrane

Oxidative degradation is considered to be the most harmful degradation mechanism to polyethylenes (Kay et al., 2004; Islam and Rowe, 2007). Antioxidants are normally used to retard oxidative reactions and increase the long-term durability of polyethylenes, and more than one antioxidant is normally used by manufacturers to provide greater long-term stability. Oxidation of the geomembrane is thought to start only after depletion of the antioxidant, and only if oxygen is available. Oxidation of a polyethylene geomembrane can result in a loss of molecular weight and an increase in susceptibility to environmental stress cracking. LLDPE is less sensitive to environmental stress cracking than HDPE (Kay et al, 2004). Some factors that affect oxidation include temperature, the composition of the solution the geomembrane is in contact with, and oxygen levels.

The results for the Gulec et al. (2004) study conducted on geomembranes and acidic solutions (AMD and synthetically prepared solutions) were used to estimate the depletion time for the antioxidant in a 60 mil HDPE geomembrane. For one-sided exposure and field temperatures ranging from 15 to 20 degrees Celsius, the range of estimated antioxidant depletions times for a 60 mil HDPE geomembrane was 210 to 426 years. This estimation was for the AMD solution used for the study with a pH of 2.1. Thus the solution used in this Gulec study was approximately 50 times more acidic than the water from the BPA monitoring wells.

1.2.2 Cover System Geomembrane

As noted in Section 1.1.2, oxidative degradation is considered to be the most harmful degradation mechanism to polyethylenes. LLDPE has a lower density than HDPE which allow oxygen to diffuse quicker into the polymer structure (Koerner et al., 2005). It is expected that the depletion time for the antioxidants in an LLDPE will be less than an HDPE, however data is not available to quantify the actual decrease expected (Koerner et al., 2005). The study by Gulec et al. (2004) discussed in Section 1.2.1 would indicate a possible upper bound result for LLDPE for similar test conditions.

1.3 RADIOACTIVE DEGRADATION

Surface gamma measurements of the mine waste to be placed in Pit 3 and Pit 4 above the HDPE geomembrane liner and below the cover geomembrane are summarized in AES (2011). Surface gamma measurements were performed of waste rock, stockpiles, haul and access roads, and mine drainage sediments. The mean and maximum gamma exposure rates for all locations in the mine area were measured as 36.1 and 289 microrads/hr, respectively (AES, 2011). The effect of beta and alpha radiation on geomembranes is essentially the same as gamma radiation (Koerner, 2010). Radioactivity above approximately 10^6 to 10^7 rads can start to cause polymer degradation (Eby, 1979; Phillips, 1988; Koerner et al, 1990; Koerner, 2005). Koerner notes that geomembranes can be used with low level activity waste containment with radioactivity measured below 10^6 to 10^7. Damage to the geomembrane is caused by the total dose of radiation absorbed (Kircher and Bowman, 1964). Using the maximum measured gamma exposure rates for the site of approximately 300 microrads/hr, the upper bound gamma exposure of the cover and sub-waste liner geomembranes is approximately 3000 rads over 1,000 years. This value is significantly less than the 10^6 to 10^7 rads noted to be the lower bound for the start of polymer degradation.

Koerner (2005) emphasizes there is limited literature on radioactive degradation of geomembranes and that additional research should be conducted; however, there have been some studies done to evaluate radioactive degradation of polyethylenes and they are listed below.

- Test results listed in Kircher and Bowman (1964) for polyethylene films at 3 to 15 mil thickness indicated sensitivity to absorbed radiation doses above 4×10^6 rads. Results showed that for thicker films (greater than 15 mil), high density polyethylene behaves similarly to low density polyethylene.

- Testing conducted on a 60 mil HDPE geomembrane for the W-025 Radioactive Mixed Waste Disposal Facility showed that the HDPE met acceptance criteria based on guidelines provided in Koerner (1990) for changes in material properties when exposed to a radiation of 50 krads (Golder, 1991).

- Badu-Tweneboah et al. (1999) evaluated the long-term performance of a 60 mil HDPE geomembrane component of a low-level radioactive waste disposal landfill. Results of the analysis indicated that the HDPE geomembrane would perform as intended for containment for the 500-year design period.

- Compatibility testing conducted for the Hanford Grout facility on a 60-mil HDPE geomembrane showed that the HDPE had a 25 percent decrease in geomembrane strength and elasticity when exposed to a total radiation dose of 4×10^7 rads (INEEL, 2004).

2.0 CONCLUSIONS

2.1.1 Liner System Geomembrane

Studies discussed above confirm that a HDPE membrane is chemically resistant to acidic solutions containing sulfuric acid and sulfates and remains viable for hundreds of years. The testing conducted by Mitchell (1985) and Gulec et al. (2004 and 2005) provides laboratory data to support the use of an HDPE geomembrane liner within Pit 3 and Pit 4. In both cases, the results indicated that HDPE geomembranes were chemically resistant to acidic solutions containing sulfuric acid. The testing conducted by the Gulec et al. (2004 and 2005) indicates an HDPE geomembrane also is chemically resistant to sulfates.

An 80 mil HDPE geomembrane has been proposed for the liner system within pits. It is anticipated that the geomembrane liner will not be exposed to temperatures greater than about 20 degrees Celsius, which will result in slower reaction rates than seen in the laboratory studies cited above. Based on review of available information, no measurable chemical degradation of the HDPE materials is expected for short-term conditions. Long-term, exposures of the HDPE liner to sulfuric acid likely will result in antioxidant depletion. Once the available antioxidant has been depleted, geomembrane oxidation may occur, eventually resulting in decreased environmental stress crack resistance and other deleterious effects. Based on Gulec study (2004) the antioxidant depletion time should exceed 210 years for one-sided exposure of a 60 mil HDPE geomembrane at field temperatures ranging from 15 to 20 degrees Celsius to a sulfuric-acid containing solution with a pH of 2.1. Given the site water may have a pH of 3.5 or higher, it is anticipated that the HDPE antioxidant depletion time will be considerably longer. The study by Gulec et al. (2004) did not evaluate the effect of the thickness of the geomembrane on the antioxidant depletion times. However, studies conducted for chemical compatibility of municipal waste landfill leachates and geomembranes (Rowe et al., 2010) have shown a significant increase in the antioxidant depletion times with an increase in thickness of the geomembrane.

Radioactive degradation of the 80 mil HDPE liner is not expected to be a concern based on surface gamma measurements that indicate upper bound radiation absorption for the geomembrane of 3000 rads over 1,000 years. This is significantly less than the 10^6 to 10^7 rads noted to be the lower bound for the start of polymer degradation.

2.1.2 Cover System Geomembrane

Leachate exposure for the cover geomembrane will be in the form of condensate on the bottom side of the geomembrane. The condensate is considered to be much less aggressive than the leachate within the bottom of Pits 3 and 4. Studies by Mitchell (1985) and Gulec et al. (2004 and 2005) on HDPE indicate that a LLDPE geomembrane would be compatible the condensate which is expected to have a higher pH and lower sulfate concentration than the leachate used for the studies.

Oxidation is the primary concern for degradation of the LLDPE. The discussion provided in Section 2.1.1 regarding oxidation depletion time for the HDPE liner is indicative of expected antioxidant depletion times for the LLDPE. Although it is expected that antioxidant depletion

times are less for HDPE than LLDPE, the exposure conditions for the LLDPE cover geomembrane will be less aggressive than for the HDPE liner.

Radioactive degradation of the LLDPE is not expected to be a concern based on surface gamma measurements that indicate upper bound radiation absorption for the geomembrane of 3000 rads over 1,000 years. This is significantly less than the 10^6 to 10^7 rads noted to be the lower bound for the start of polymer degradation.

3.0 REFERENCES

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Attachment D-3

Cover Design, Radon Emanation Modeling

ATTACHMENT D-3

MIDNITE MINE REMEDIAL ACTION COVER DESIGN RADON EMANATION MODELING

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*23-Oct-12*	*30% Design*	*Melanie Davis*	*Clint Strachan*	*23-Oct-12*	*Clint Strachan*
1	*11-Nov-13*	*60% Design*	*Melanie Davis*	*Clint Strachan*	*15-Nov-13*	*Clint Strachan*
2	*15-May-14*	*90% Design (title change only)*	*Melanie Davis*	*Clint Strachan*	*15-May-14*	*Clint Strachan*
2	*14-May-15*	*100% Design – no changes*				

Location and Format

Electronic copies of these calculations are located in the project files system at:

\\usftc2s01\Projects\Newmont\Midnite Mine_2011\6.0 Studies & Reports\6.2 Technical\Remedial Action Cover Design\Radon Modeling\60% Design

The following calculations were generated using the following software:

Microsoft Office Professional Plus 2010 (Microsoft Excel)
RADON Model

Supplements

Supplement D-3.1 – Radon Model Output
Supplement D-3.2 – Radon Model Output for Sensitivity Analysis

1.0 BACKGROUND

This calculation brief presents the results of radon emanation modeling through the proposed cover system that will be part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The cover system will be placed over the mine waste that will be consolidated in Pit 3 (including the Backfilled Pit Area, or BPA) and Pit 4, and over the regraded surface of Area 5 (area between Pits 3 and 4). This modeling was performed to provide estimates of the required thickness of the soil cover.

The proposed cover system design consists of a 3-ft thick uniform soil layer overlying a synthetic geomembrane. On steeply sloped areas (25 to 33 percent slopes), the cover system will also include a geocomposite drainage layer (GDL) between the soil and geomembrane layers to reduce the potential pore-water build-up over the geomembrane and thereby enhance slope stability. Although a synthetic geomembrane liner will be incorporated into the cover system, its effectiveness at limiting radon emanation from the mine waste containment areas has not been included in the radon emanation modeling. This conservative approach is consistent with U.S. Nuclear Regulatory Commission guidelines.

The Rhoads Property Borrow Area located adjacent the Site is analyzed as the soil cover borrow material. The predominant material above bedrock at this potential borrow source consists of a residual deposit of clayey sand (MGC, 2011a, b). The upper layer of bedrock consists of weathered quartz monzonite which has weathered to primarily a residual clayey sand, with some slightly clayey or silty sand. It is assumed that both overlying soils and the underlying weathered bedrock residuum from this borrow area would be used for the cover system.

The preliminary design for the cover system called for the top 0.5 feet of the soil cover to be topsoil or amended borrow soil to promote vegetation growth. However based upon data presented in the Borrow Source Design Investigation Report (MGC, 2011a), it does not appear that amendment of soil from the Rhoads Property Borrow Area will be necessary to promote vegetation growth (see the Revegetation Plan, Attachment D-12). Therefore, for the purposes of these analyses, the top 0.5 feet of the cover system is assumed to have similar material properties as the underlying soil cover material. Additional information on the borrow source is provided in MGC (2011a, b).

The performance standard used in the cover design modeling is the U.S. Nuclear Regulatory Commission (NRC) long-term radon emanation standard of 20 picocuries per square meter per second (pCi/m^2-sec) (10 CFR 40, Appendix A). This maximum emanation rate is an average over the entire cover area. For radon emanation modeling, the proposed cover system consists of a uniform cover soil overlying a geomembrane above mine waste, which would have a relatively low activity concentration of radium-226. Consistent with NRC guidelines, the geomembrane was conservatively not included in the analyses of radon emanation from the cover system.

Details of the radon model, its input variables, and descriptions of input values used to represent the cover system and waste materials are discussed below.

2.0 DESCRIPTION OF MODEL AND INPUT VALUES

The thickness of the cover system necessary to limit radon emanation from the mine waste was analyzed using the NRC RADON model (NRC, 1989). The model utilizes the one-dimensional radon diffusion equation, which uses the physical and radiological characteristics of the mine waste and overlying materials to calculate the rate of radon emanation through the cover. This model was used to calculate the cover thickness required to limit the radon emanation rate through the top of the cover to 20 pCi/m^2-s, following the guidance presented in NRC publications NUREG/CR-3533 (NRC, 1984) and Regulatory Guide 3.64 (NRC, 1989). The 20 pCi/m^2-s limit for emanation is applied to the average emanation rate over the entire surface of the waste containment areas.

The input parameters used in the model are based on engineering experience with similar projects and available measured data for the borrow source and for the mine wastes. The measured data was the result of previous work by others, including MGC (2011a, b) and AES (2011). The input parameters and values used in the model are outlined below.

2.1 THICKNESS OF CONTAMINATED MATERIAL

As documented in NRC Regulatory Guide 3.64 (NRC, 1989), a material thickness of 500 cm (16.4 feet) may be used in RADON to represent an equivalent infinitely-thick source for modeling radon emanation. The proposed plan for mine waste placement in Pits 3 and 4 includes placement of waste rock with lower radium activity concentrations as the top layer of mine waste (minimum thickness of 500 cm). This would reduce the radon emanation from mine waste with higher radium activity concentrations that would be placed lower in the pits. Material from the South Waste Rock Pile is currently proposed to comprise the upper-most layer of mine waste in Pits 3 and 4 with a minimum thickness of 20 feet. For modeling the cover over the BPA, the radium activity concentration from the South Waste Rock Pile was used for mine waste. The same activity concentration was used because actual radon emanation measurements (EPA, 2005) showed that emanation from the BPA was within the range of values measured on the South Waste Rock Pile.

2.2 RADIUM ACTIVITY CONCENTRATION

Mine Waste. The mine waste radium-226 activity concentration value used in the modeling presented in this appendix is 32 picocuries per gram (pCi/g), based on the average laboratory-measured values of samples from the South Waste Rock Pile as documented in the Technical Memorandum Mine Waste Characterization (AES, 2011). For comparison, AES (2011) presented the average radium-226 activity concentration for the South Waste Rock Pile material (calculated from the natural uranium ore grade) as 23 pCi/g. Since the average measured value for radium-226 activity concentration of the mine waste (i.e., 32 pCi/g) is higher than the average calculated value (23 pCi/g), the higher average measured value was conservatively selected for use in the model.

Cover Material. The radium activity of the cover material was assumed to be zero, based on guidance in Regulatory Guide 3.64 (NRC, 1989) which states that radium activity in the cover soils may be neglected for cover design purposes provided the cover soils are obtained from

background materials that are not associated with ore formations or other radium-enriched materials. Results of radiological testing of the material from the Rhoads Property Borrow Area for radium concentration activities range from 1.0 to 3.0 pCi/g, with an average value of 1.6 pCi/g (MGC, 2011a). These values are below the cleanup level and site background limit listed in the Record of Decision (EPA, 2006) of 4.7 pCi/g. In addition, inclusion of the average radium activity concentration for cover material from the Rhoads Property Borrow Area does not increase the required thickness of the cover.

2.3 RADON EMANATION COEFFICIENT

The radon emanation coefficient used in the model for the cover layers and mine waste is 0.35. This is the conservative default value used in the RADON model (NRC, 1989).

2.4 SPECIFIC GRAVITY, DENSITY AND POROSITY

The specific gravity of the mine waste was estimated as 2.83 based on the average measured value for test pit samples from the South Waste Rock Dump (EPA, 2002). The dry density of the waste rock was estimated at 110 pounds per cubic foot (pcf) based on the average in-situ densities measured for test pits excavated in the South Waste Rock Dump (EPA, 2002). The porosity of the waste rock was calculated using the estimated specific gravity and dry density based on the following equation:

$$n = 1 - \left(\frac{\gamma_d}{G_s \gamma_w}\right) \qquad \text{(Eq. 1)}$$

where

n = porosity,
γ_d = dry density of soil in pcf,
G_s = specific gravity of soil, and
γ_w = unit weight of water in pcf.

The dry density and porosity values used in the model for the waste rock are listed in Table 1.

The specific gravity and dry density values of the cover material used in the model are based on laboratory testing results for the Rhoads Property Borrow Area (MGC, 2011a, b). The porosity value for the cover material was calculated using Equation 1 and is provided in Table 1. The specific gravity value listed is the average laboratory measured value for samples collected from the borrow area. The density value was estimated based on the average laboratory measured maximum standard Proctor dry densities (with the rock correction applied where applicable) for collected samples from the borrow area and assuming the cover material will be placed at an average of approximately 85 percent of standard Proctor compaction.

Table 1. Density and Porosity Values

Material	Specific Gravity	Dry Density (pcf)	Dry Density (g/cc)	Porosity
Cover Layers (Rhoads Borrow Area)	2.63	101.7	1.63	0.38
Waste Rock (South Waste Rock Dump)	2.83	110	1.76	0.38

2.5 LONG-TERM MOISTURE CONTENT

NRC Regulatory Guide 3.64 (NRC, 1989) recommends several alternative methods for estimating the long-term moisture content for cover soils including: 1) measuring natural moisture contents at depths between 120 and 500 cm (thereby excluding measurements at shallow depths that may be affected by seasonal variations and excluding measurements near the groundwater table); 2) laboratory testing of 15-bar moisture contents; and 3) using the empirical equation by Rawls and Brakenseik (1982) which is a function of percent clay of the cover material. The specific alternative methods used in estimating the long-term moisture contents of the mine waste and cover materials are discussed below.

Mine Waste Material. The long-term moisture content of the mine waste was estimated as the average natural moisture content at depths below 120 cm (4 feet). This average value was 5.4 percent, based on 40 samples collected from South Waste Rock Dump test pits and borings and documented in EPA (2002). Groundwater was not encountered in the test pits or borings, so all the measurements for natural water content below four feet were included in determining the average long-term moisture content. From these 40 samples, three samples were above 500 cm depth, and had an average measured moisture content of 11.1 percent. This method was selected for determination of long-term moisture content for modeling the mine waste material due to the lack of measured data for 15-bar moisture contents and percent clay of the material.

Cover Material. The long-term moisture content of cover materials from the Rhoads Property Borrow Area was estimated as 10.4 percent, based on the average measured 15-bar gravimetric moisture content for samples collected from the borrow area (MGC, 2011a). This method was selected due to available measured 15-bar moisture content measurements and limited natural moisture content data at depths below four feet. In addition, the measured 15-bar moisture contents were compared with estimated long-term moisture contents using Rawls and Brakenseik (1982). For all the samples with measured 15-bar moisture contents, the 15-bar moisture contents were higher than the estimated long-term moisture content using Rawls and Brakenseik (1982). The average activity of the fines for the Rhoads Property Borrow material indicates the behavior of the material is similar to what would be expected for an illite clay. The measured 15-bar moisture content would be a little higher than the estimated value based on empirical relationships of clay content versus 15-bar moisture contents. Therefore, the average measured 15-bar moisture content of 10.4 percent was used in the model.

The average long-term moisture contents used in the radon modeling are summarized in Table 2.

Table 2. Estimated Long-Term Moisture Contents

Material	Gravimetric Moisture Content (%)
Cover Layers (Rhoads Borrow Area)	10.4
Mine Waste (South Waste Rock Dump)	5.4

2.6 DIFFUSION COEFFICIENT

The radon diffusion coefficient used in the RADON model either can be calculated within the model (based on an empirical relationship dependent upon porosity and the degree of saturation) or input directly in the model using values measured from laboratory testing. Laboratory test data were not available for the mine waste and cover materials. Therefore, the empirical relationship in RADON was used, resulting in the calculated values summarized in Table 3 below.

Table 3. Calculated Radon Diffusion Coefficients

Material	Diffusion Coefficient (cm^2/s)
Cover Layers (Rhoads Borrow Area)	0.0142
Mine Waste (South Waste Rock Dump)	0.0296

2.7 LONG-TERM EFFECTIVENESS

NRC (1989) notes that potential defects could occur in the cover due to pedogenic processes including soil desiccation, erosion cracking and piping, and biointrusion due to animal burrowing and root penetration. The cover system will be placed at a low relative compaction (approximately 85 percent of standard Proctor compaction) to represent long-term conditions and minimize potential changes to the cover system due to pedogenic processes as recommended in NUREG CR-7028 (Benson et al., 2011). In addition, the proposed cover soils (sand to clayey sand) are relatively permeable. NUREG CR-7028 indicates that long-term changes in cover material properties are more prone to occur for less permeable soils compared to more permeable soils. Cover thickness loss due to erosion is estimated to be less than 1 inch (see Attachment D-6). A correction to account for potential pedogenic processes is not warranted at this time because the physical and hydraulic properties of the proposed cover soils at the proposed emplaced conditions are not likely to result in significant post-construction changes.

3.0 MODEL RESULTS FOR COVER THICKNESS

The radon emanation modeling results summarized in Table 4 show the thickness for the cover system required to limit the rate of radon emanation from the waste containment areas to values below the limit of 20 pCi/m^2-s. The results indicate a cover thickness of 1.8 feet would be sufficient to limit radon emanation acceptable levels. The cover system design has a minimum cover thickness of 3.0 feet (including soil cover and topsoil/growth media layers). The RADON model output is provided in Supplement D-3.1.

Table 4. Summary of Results

Model Parameters	Model Layer	
	Layer 1 Mine Waste	Layer 2 Cover
Porosity	0.38	0.38
Specific Gravity	2.83	2.63
Dry Density (g/cc)	1.76	1.63
Radium-226 Activity Concentration (pCi/g)	32	0
Emanation Coefficient	0.35	0.35
Long-Term Moisture Content (%)	5.4	10.4
Calculated Radon Diffusion Coefficient (cm^2/sec)	0.0296	0.0142
Required Cover Thickness (cm)	**----**	**53 (1.8 ft)**

4.0 SENSITIVITY ANALYSIS

A sensitivity analysis was conducted to evaluate the range of cover density compaction of 80 to 90 percent of standard Proctor compaction on the calculated rate of radon emanation. The results are summarized in Table 5 and indicate that replacing the average dry density with values representing 80 to 90 percent of standard Proctor compaction results in an increase of 0.7 feet (for 80 percent compaction) or a decrease of 0.8 feet (for 90 percent compaction), in the required cover thickness. The resulting cover thicknesses from the sensitivity analysis (2.5 and 1.0 feet) are lower than the design thickness of 3.0 feet. The RADON model output for the sensitivity analysis is provided in Supplement D-3.2.

Table 5. Summary of Sensitivity Analysis Results

Model Parameters	Model Layer		
	Layer 1 Mine Waste	Layer 2 Cover at 80% compaction	Layer 2 Cover at 90% compaction
Porosity	0.38	0.42	0.34
Specific Gravity	2.83	2.63	2.63
Dry Density (g/cc)	1.76	1.53	1.73
Radium-226 Activity Concentration (pCi/g)	32	0	0
Emanation Coefficient	0.35	0.35	0.35
Long-Term Moisture Content (%)	5.4	10.4	10.4
Calculated Radon Diffusion Coefficient (cm^2/sec)	0.0296	0.0195	0.0091
Required Cover Thickness (cm)	**----**	**76 (2.5 ft)**	**31 (1.0 ft)**

5.0 REFERENCES

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Supplement D-3.1

Radon Model Output

-----*****! RADON !*****-----

Version 1.2 - MAY 22, 1989 - G.F. Birchard tel.# (301)492-7000
U.S. Nuclear Regulatory Commission Office of Research

RADON FLUX, CONCENTRATION AND TAILINGS COVER THICKNESS
ARE CALCULATED FOR MULTIPLE LAYERS

OUTPUT FILE: Rhoads

DESCRIPTION: Midnite Mine Remedial Action Cover Design
Cover Material from Rhoads Borrow Area

CONSTANTS

RADON DECAY CONSTANT	.0000021	s^-1
RADON WATER/AIR PARTITION COEFFICIENT	.26	
DEFAULT SPECIFIC GRAVITY OF COVER & TAILINGS	2.65	

GENERAL INPUT PARAMETERS

LAYERS OF COVER AND TAILINGS	2	
DEFAULT RADON FLUX LIMIT	20	pCi m^-2 s^-1
NO. OF THE LAYER TO BE OPTIMIZED	2	
DEFAULT SURFACE RADON CONCENTRATION	0	pCi l^-1
SURFACE FLUX PRECISION	.001	pCi m^-2 s^-1

LAYER INPUT PARAMETERS

LAYER 1 Mine Waste

THICKNESS	500	cm
POROSITY	.38	
MEASURED MASS DENSITY	1.76	g cm^-3
MEASURED RADIUM ACTIVITY	32	pCi/g^-1
DEFAULT LAYER EMANATION COEFFICIENT	.35	
CALCULATED SOURCE TERM CONCENTRATION	1.089D-04	pCi cm^-3 s^-1
WEIGHT % MOISTURE	5.4	%
MOISTURE SATURATION FRACTION	.250	
CALCULATED DIFFUSION COEFFICIENT	2.963D-02	cm^2 s^-1

LAYER 2 Cover

THICKNESS	1	cm
POROSITY	.38	
MEASURED MASS DENSITY	1.63	g cm^-3
MEASURED RADIUM ACTIVITY	0	pCi/g^-1
DEFAULT LAYER EMANATION COEFFICIENT	.35	
CALCULATED SOURCE TERM CONCENTRATION	0.000D+00	pCi cm^-3 s^-1
WEIGHT % MOISTURE	10.4	%
MOISTURE SATURATION FRACTION	.446	
CALCULATED DIFFUSION COEFFICIENT	1.417D-02	cm^2 s^-1

DATA SENT TO THE FILE `RNDATA' ON DRIVE A:

N	F01	CN1	ICOST	CRITJ	ACC
2	-1.000D+00	0.000D+00	2	2.000D+01	1.000D-03

LAYER	DX	D	P	Q	XMS	RHO
1	5.000D+02	2.963D-02	3.800D-01	1.089D-04	2.501D-01	1.760
2	1.000D+00	1.417D-02	3.800D-01	0.000D+00	4.461D-01	1.630

BARE SOURCE FLUX FROM LAYER 1: 4.843D+01 pCi m^-2 s^-1

RESULTS OF THE RADON DIFFUSION CALCULATIONS

LAYER	THICKNESS (cm)	EXIT FLUX (pCi m^-2 s^-1)	EXIT CONC. (pCi l^-1)
1	5.000D+02	2.428D+01	2.549D+04
2	5.268D+01	2.002D+01	0.000D+00

Supplement D-3.2

Radon Model Output for Sensitivity Analysis

-----*****! RADON !*****-----

Version 1.2 - MAY 22, 1989 - G.F. Birchard tel.# (301)492-7000
U.S. Nuclear Regulatory Commission Office of Research

RADON FLUX, CONCENTRATION AND TAILINGS COVER THICKNESS
ARE CALCULATED FOR MULTIPLE LAYERS

OUTPUT FILE: Rhoads80

DESCRIPTION: Midnite Mine Remedial Action Cover Design
Cover Material from Rhoads Borrow Area at 80% compaction

CONSTANTS

RADON DECAY CONSTANT	.0000021	s^-1
RADON WATER/AIR PARTITION COEFFICIENT	.26	
DEFAULT SPECIFIC GRAVITY OF COVER & TAILINGS	2.65	

GENERAL INPUT PARAMETERS

LAYERS OF COVER AND TAILINGS	2	
DEFAULT RADON FLUX LIMIT	20	pCi m^-2 s^-1
NO. OF THE LAYER TO BE OPTIMIZED	2	
DEFAULT SURFACE RADON CONCENTRATION	0	pCi l^-1
SURFACE FLUX PRECISION	.001	pCi m^-2 s^-1

LAYER INPUT PARAMETERS

LAYER 1 Mine Waste

THICKNESS	500	cm
POROSITY	.38	
MEASURED MASS DENSITY	1.76	g cm^-3
MEASURED RADIUM ACTIVITY	32	pCi/g^-1
MEASURED EMANATION COEFFICIENT	.35	
CALCULATED SOURCE TERM CONCENTRATION	1.089D-04	pCi cm^-3 s^-1
WEIGHT % MOISTURE	5.4	%
MOISTURE SATURATION FRACTION	.250	
CALCULATED DIFFUSION COEFFICIENT	2.963D-02	cm^2 s^-1

LAYER 2 Cover

THICKNESS	1	cm
POROSITY	.42	
MEASURED MASS DENSITY	1.53	g cm^-3
MEASURED RADIUM ACTIVITY	0	pCi/g^-1
DEFAULT LAYER EMANATION COEFFICIENT	.35	
CALCULATED SOURCE TERM CONCENTRATION	0.000D+00	pCi cm^-3 s^-1
WEIGHT % MOISTURE	10.4	%
MOISTURE SATURATION FRACTION	.379	
CALCULATED DIFFUSION COEFFICIENT	1.948D-02	cm^2 s^-1

DATA SENT TO THE FILE `RNDATA' ON DRIVE A:

N	F01	CN1	ICOST	CRITJ	ACC
2	-1.000D+00	0.000D+00	2	2.000D+01	1.000D-03

LAYER	DX	D	P	Q	XMS	RHO
1	5.000D+02	2.963D-02	3.800D-01	1.089D-04	2.501D-01	1.760
2	1.000D+00	1.948D-02	4.200D-01	0.000D+00	3.789D-01	1.530

BARE SOURCE FLUX FROM LAYER 1: 4.843D+01 pCi m^-2 s^-1

RESULTS OF THE RADON DIFFUSION CALCULATIONS

LAYER	THICKNESS (cm)	EXIT FLUX (pCi m^-2 s^-1)	EXIT CONC. (pCi l^-1)
1	5.000D+02	2.650D+01	2.315D+04
2	7.554D+01	2.002D+01	0.000D+00

-----*****! RADON !*****-----

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U.S. Nuclear Regulatory Commission Office of Research

RADON FLUX, CONCENTRATION AND TAILINGS COVER THICKNESS
ARE CALCULATED FOR MULTIPLE LAYERS

OUTPUT FILE: Rhoads90

DESCRIPTION: Midnite Mine Remedial Action Cover Design
Cover Material from Rhoads Borrow Area at 90% compaction

CONSTANTS

RADON DECAY CONSTANT	.0000021	s^-1
RADON WATER/AIR PARTITION COEFFICIENT	.26	
DEFAULT SPECIFIC GRAVITY OF COVER & TAILINGS	2.65	

GENERAL INPUT PARAMETERS

LAYERS OF COVER AND TAILINGS	2	
DEFAULT RADON FLUX LIMIT	20	pCi m^-2 s^-1
NO. OF THE LAYER TO BE OPTIMIZED	2	
DEFAULT SURFACE RADON CONCENTRATION	0	pCi l^-1
SURFACE FLUX PRECISION	.001	pCi m^-2 s^-1

LAYER INPUT PARAMETERS

LAYER 1 Mine Waste

THICKNESS	500	cm
POROSITY	.38	
MEASURED MASS DENSITY	1.76	g cm^-3
MEASURED RADIUM ACTIVITY	32	pCi/g^-1
DEFAULT LAYER EMANATION COEFFICIENT	.35	
CALCULATED SOURCE TERM CONCENTRATION	1.089D-04	pCi cm^-3 s^-1
WEIGHT % MOISTURE	5.4	%
MOISTURE SATURATION FRACTION	.250	
CALCULATED DIFFUSION COEFFICIENT	2.963D-02	cm^2 s^-1

LAYER 2 Cover

THICKNESS	1	cm
POROSITY	.34	
MEASURED MASS DENSITY	1.73	g cm^-3
MEASURED RADIUM ACTIVITY	0	pCi/g^-1
DEFAULT LAYER EMANATION COEFFICIENT	.35	
CALCULATED SOURCE TERM CONCENTRATION	0.000D+00	pCi cm^-3 s^-1
WEIGHT % MOISTURE	10.4	%
MOISTURE SATURATION FRACTION	.529	
CALCULATED DIFFUSION COEFFICIENT	9.120D-03	cm^2 s^-1

DATA SENT TO THE FILE `RNDATA' ON DRIVE A:

N	F01	CN1	ICOST	CRITJ	ACC
2	-1.000D+00	0.000D+00	2	2.000D+01	1.000D-03

LAYER	DX	D	P	Q	XMS	RHO
1	5.000D+02	2.963D-02	3.800D-01	1.089D-04	2.501D-01	1.760
2	1.000D+00	9.120D-03	3.400D-01	0.000D+00	5.292D-01	1.730

BARE SOURCE FLUX FROM LAYER 1: 4.843D+01 pCi m^-2 s^-1

RESULTS OF THE RADON DIFFUSION CALCULATIONS

LAYER	THICKNESS (cm)	EXIT FLUX (pCi m^-2 s^-1)	EXIT CONC. (pCi l^-1)
1	5.000D+02	2.224D+01	2.764D+04
2	3.083D+01	2.001D+01	0.000D+00

Attachment D-4

Cover Design, Infiltration Analyses

ATTACHMENT D-4

MIDNITE MINE REMEDIAL ACTION COVER DESIGN INFILTRATION ANALYSES

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*11-Dec-12*	*30% Design*	*Jean-Baptiste Varnier*	*Melanie Davis*	*12-Dec-12*	*Tom Kelley*
1	*3-Jul-13*	*Interim Submittal to EPA*	*Jean-Baptiste Varnier*	*Melanie Davis*	*10-Jul-13*	*Tom Kelley*
2	*2-Dec-13*	*60% Design*	*Jean-Baptiste Varnier*	*Melanie Davis*	*2-Dec-13*	*Tom Kelley*
3	*13-Jun-14*	*90% Design*	*Jean-Baptiste Varnier/Melanie Davis*	*Melanie Davis*	*17-Jul-14*	*Tom Kelley*
3	*14-May-15*	*100% Design – no changes*				

Location and Format

Electronic copies of these calculations are located in the project files system at:

\\usftc2s01\Projects\Newmont\Midnite Mine_2011\6.0 Studies & Reports\6.1 Reports\MWH Reports\90% Design Report

The following calculations were generated using the following software:

Microsoft Office Professional Plus 2010 (Microsoft Excel)
Geo-Slope International, Ltd, 2012 (Vadose/W)

Supplements

Supplement D-4.1 – Cover Design Infiltration Analyses Calculations

1.0 BACKGROUND

This calculation brief presents the methods and results of analyses of percolation through the cover system that will be part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The cover system will be placed over mine waste that will be consolidated in Pit 3 (including the Backfilled Pits Area, or BPA) and Pit 4, and over the regraded surface of Area 5 (area between Pits 3 and 4). This calculation brief is an update to the calculation brief provided in the 60 percent Basis of Design Report (BODR) (MWH, 2013).

The proposed cover system consists of a uniform soil layer overlying a synthetic geomembrane. On steeper sloped areas (greater than 15 percent slopes), the cover system will also include a geocomposite drainage layer (GDL) between the soil and geomembrane layers to reduce the potential pore-water build over the geomembrane and enhance slope stability. The GDL will consist of a non-woven, needle-punched geotextile heat-bonded to a geonet.

The Rhoads Property Borrow Area located adjacent to the Site is analyzed as the soil cover borrow material. The predominant soil in this borrow area consists of clayey sand (MGC, 2011a, b). The upper layer of bedrock consists of weathered quartz monzonite which has weathered to a residual soil (residuum). It is assumed that both materials may be used for the cover system. A minimum thickness of 1.8 feet is required for this cover soil based on radon emanation modeling results (refer to Attachment D-3); however, a minimum soil cover thickness of 3.0 feet has been proposed based on construction and slope stability considerations.

The preliminary design for the cover system called for the top 0.5 feet of the soil cover to be topsoil or amended borrow soil to promote vegetation growth. However based upon data presented in the Borrow Source Design Investigation Report (MGC, 2011a), it does not appear that amendment of soil from the Rhoads Property Borrow Area will be necessary to promote vegetation growth (see the Revegetation Plan, Attachment D-12). Therefore, for the purposes of these analyses, the top 0.5 feet of the cover system is assumed to have similar material properties as the underlying soil cover material. Additional information on the borrow source is provided in MGC (2011a, b).

2.0 CASES ANALYZED

Analyses were conducted to evaluate percolation (leakage) through the cover system under: 1) as-constructed conditions with the geomembrane intact, and 2) long-term degraded conditions when the geomembrane has degraded and is no longer effective at limiting percolation. The cases are described further below.

2.1 AS-CONSTRUCTED CONDITIONS

For the as-constructed case, percolation through the cover system and into the underlying waste materials was assumed to occur through small defects in the geomembrane. The basis for assumptions regarding defect geometry and frequency are discussed in Section 3. For the infiltration analyses, the waste containment area cover was divided into three distinct areas depending on surface geometry and cover system components. Separate percolation

calculations were made for each of these areas, and a composite, area-weighted leakage rate was calculated for the entire cover system. The properties of these three areas are shown on Figure 1 and listed in Table 1.

Table 1. Cover Areas Analyzed for As Constructed Conditions

Case	Description	% of Total Area
A1	Mildly sloped areas (typically less than 15%) with no surface ponding and no GDL overlying the geomembrane (GM)	18.2
A2	Flat areas on the drainage benches where flow is concentrated and/or surface ponding may occur and there is no GDL overlying the GM. The percentage of the total area shown is based upon the assumption that 50% of the total drainage bench area will fall into this classification.	5.7
A3	Sloped areas (typically greater than 15%) with no surface ponding and a GDL overlies the GM	76.1

The approximate percentages of the total area for each case listed in Table 1 are based on the current cover grading plan for the waste containment areas. These percentages will be updated as needed due to subsequent revisions of the grading plan. The areas are designated on Figure 2 and Figure 3 for Pit 3 and Pit 4, respectively.

In addition to analyzing the design base case for cover thickness (3 feet), sensitivity analyses were conducted to evaluate the effect of cover thicknesses of 2 feet and 6 feet.

2.2 LONG-TERM DEGRADED CONDITIONS

Under very long-term conditions, plasticizers in geomembrane materials will be lost due to oxidization, leaching, and other processes and as a result, the geomembrane material is expected to stiffen and become more susceptible to cracking. In order to evaluate upper bound percolation conditions in the extreme long-term for degraded liner conditions, analyses were made assuming the GM and GDL no longer exist or have no effect on the hydraulics of the cover system and percolation would flow unimpeded through the bottom of the soil cover. As with the analyses of as constructed conditions, the waste containment area cover was subdivided based upon surface drainage conditions as listed in Table 2.

Table 2. Cases Analyzed for Long-Term Conditions (Degraded, No GM Effect)

Case	Description	% of Total Area
B1	Mildly sloped and sloped areas with no surface ponding, GM and GDL no longer has any effect	94.3
B2	Flat drainage bench areas where surface ponding may occur, GM no longer has any effect	5.7

In addition to analyzing the design base case for cover thickness (3 feet), sensitivity analyses were conducted to evaluate the effect of cover thicknesses of 2 feet and 6 feet.

3.0 LEAKAGE CALCULATIONS

3.1 AS-CONSTRUCTED CONDITIONS

Small geomembrane defects may occasionally be undetected during construction (Giroud et al. 1997 and EPA 1992). Although there is a minute rate of leakage through the geomembrane via permeation or diffusion rate, the permeation rate is insignificant when compared to the leakage rate through small defects in the installed liner. For as-constructed conditions, it was assumed that all leakage would occur through these small defects and other, less significant transport mechanisms were neglected.

The leakage rates through assumed geomembrane defects were calculated using empirical equations from Bonaparte et al. (1989) for leakage through a quasi-composite liner (e.g., the geomembrane is underlain by a high permeability material and overlain by a medium permeability material or a drain material). The equations used are:

$$Q = 0.21 \times a^{0.1} \times h^{0.9} \times k_s^{0.74} \quad \text{(Eqn. 1)}$$

Where:

Q is the leakage rate through defect (m^3/s)
a is the defect area (m^2)
h is the head of water on top of the geomembrane (m)
k_s is the permeability of the material underlying the geomembrane (m/s)

or

$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{0.5} \quad \text{(Eqn. 2)}$$

Where:

Q is the leakage rate through defect (m^3/s)
a is the defect area (m^2)
h is the head of water on top of the geomembrane (m)
k_d is the permeability of the material overlying or underlying the geomembrane (m/s)

The typical diameter for small geomembrane defects is 1 to 2 millimeters (Giroud et al., 1997 and EPA, 1992). A value of 2 millimeters was used for these leakage calculations. A defect frequency of 1 hole per acre, as recommended by EPA (1992), was used in this analysis. The head of water on the geomembrane used to calculate the leakage rates was based on the results of a one-dimensional numerical model (i.e., Vadose/W described in Section 4.0) assuming no leakage through the geomembrane. This head basis is somewhat conservative as it does not account for localized reduction in liner head that may occur in the vicinity of the defect due downward flow through the semipermeable cover material (see Figure 1). Based on recommendations in Bonaparte et al. (1989), Equation 1 was used for Cases A1 and A2, and the geometric mean of both equations was used for Case A3. For Cases A1 and A2, the permeability used in the equation is the permeability of the cover soil of 5.9×10^{-6} m/s (MGC, 2011a). For Case A3, an assumed permeability for the waste rock was used in the equations. This value was assumed to be the geometric mean of 1×10^{-4} to 1×10^{-5} m/s (3.2×10^{-5} m/s). The composite leakage rate through the cover system was then calculated as the sum of the

weighted area leakage rates for Cases A1 through A3 using the area percentages listed in Table 1.

3.2 LONG-TERM DEGRADED CONDITIONS

Percolation rates through the cover system under very long-term conditions were calculated assuming the geosynthetic layers are either no longer present or have no impact on the hydraulic behavior of the cover system. This represents an upper bound case for leakage under extremely long-term conditions.

The upper-bound case for leakage for long-term degraded conditions, assuming the geosynthetic layers have no effect on cover hydraulics, was evaluated using a one-dimensional numerical model (i.e. Vadose/W described in Section 4.0). The composite percolation rate through the cover system for the upper bound long-term degraded conditions was then calculated as the sum of the weighted area leakage rates for Cases B1 and B2 using the area percentages listed in Table 2.

4.0 NUMERICAL MODEL

4.1 DESCRIPTON OF MODEL

4.1.1 General

One-dimensional numerical models were used to provide input for the cover leakage calculations discussed in Section 3.0. The development of the numerical models is described below. Recommendations provided in NUREG/CR-7028 (Benson et al., 2011) and Albright et al. (2010) for modeling unsaturated flow through cover systems were followed where applicable.

4.1.2 Description of Models

Infiltration analyses were conducted using the computer program Vadose/W (Geo-Slope International, Ltd, 2012). Vadose/W is an accepted model for cover infiltration analyses. (Albright et al., 2010). Vadose/W is a finite-element-based program that is used to model movement and distribution of pore water within porous material. Vadose/W can model both saturated and unsaturated flow in response to atmospheric conditions. This makes it possible to analyze seepage as a function of time while considering infiltration, precipitation, surface water runoff and ponding, plant transpiration and evaporation based on computed soil water stress conditions. The physical relationships required for rigorous calculation of evaporation include fully coupled heat and mass transfer with vapor flow in the soil and across the soil-atmosphere continuum. Evaporation is computed in the Vadose/W model using the Penman-Wilson (Wilson, 1990) method.

The one-dimensional model was used to analyze the head of water on the geomembrane that would develop if there were no defects present, and for percolation out of the bottom of the soil cover if no geomembrane were present. A one-dimensional model is the preferred approach for cover infiltration modeling (Albright et al, 2010 and Benson, 2013). To simulate lateral flow of excess precipitation for areas where ponding is not expected to occur (mildly sloped and sloped cover surfaces), runoff was allowed from the ground surface. Runoff can be handled in a one-

dimensional model in Vadose/W by not allowing rainfall in excess of maximum infiltration rates to accumulate on the climate boundary surface. In these analyses, daily precipitation values are assumed to occur in a uniform manner over a 24-hour period. This is considered reasonable and somewhat conservative as it will results in higher total volume of infiltration into the cover than if the precipitation rate was assumed to occur in a non-uniform manner or over a shorter duration. Assuming that daily precipitation amounts occur uniformly over a 24-hour period means that even the "no-ponding" boundary condition effectively allows for some ponding, but that the amount of surface ponding is limited to the amount that can infiltrate during a 24-hour period and that all additional excess precipitation (that is at a rate in excess of the infiltration that can infiltrate into the soil cover over a 24-hour period) is assumed to run off and not enter the cover system. For areas where ponding may occur (i.e. flat drainage benches), surface water was allowed to pond on the climate boundary if the daily precipitation values exceeded the infiltration rate and could only be removed via evaporation or infiltration. To handle snow, the model allows snow pack to develop on the ground surface during periods when precipitation occurs with air temperatures below freezing. A simplified thermal model was used for the analyses.

4.1.3 Model Geometry

One-dimensional models were created to simulate infiltration through the cover for as-constructed and degraded geomembrane conditions. The model geometries, including the finite element meshes, are presented in Figures 4 through 6 for the various cover thicknesses and bottom-surface boundary conditions analyzed. The geometries of the models shown in Figures 4 through 6 represents the soil cover overlying a geocomposite drainage layer (free flow) and a geomembrane layers (no flow). The dimensions of the models are 0.61 m (2 ft), 0.91 m (3 ft) and 1.83 m (6 ft) high and 0.01 m wide. The fluxes were evaluated at the base of the cover for each of the cover thicknesses evaluated (2 ft, 3 ft and 6 ft).

4.2 MODEL INPUT PARAMETERS

4.2.1 Climate Data

Climate data are available from the Midnite Mine climate station (NWS Station No. 452913). This station is a Remote Automated Weather Station (RAWS) that was installed by the U.S. Department of Interior, Bureau of Land Management. The period of record for the Midnite Mine climate station is 1991 through 2012 (22 years). Climate data used in the analyses included daily precipitation, maximum and minimum air temperatures, average wind speed, and maximum and minimum relative humidity. Data was obtained from the Western Regional Climate Center (WRCC, 2012).

A "typical year" and "wet year" were selected for use for the analyses based on evaluation of the data. A "typical year" and "wet year" are commonly used for cover infiltration modeling (Albright et al., 2010). Months that had greater than five missing daily measurements were not included in the calculation of the average and maximum precipitation over the 1991-2012 time period in order to be consistent with the approach used by the Western Regional Climate Center (WRCC, 2012). The climate at the site is characterized by snow accumulation starting in October, and continuing through February, with the majority of snow melt occurring in March. Due to the typical climate variations at the site, the annual precipitation was evaluated based on a water

year from October 1 through September 30 to reduce the chances that the model year ends in the middle of a significant hydrologic event (e.g. wet period or snowpack accumulation). Using this criterion, the station has an average measured annual precipitation value of 18.35 inches and a maximum measured annual precipitation of 25.77 inches.

The typical year precipitation was selected as that which occurred between October 1, 2002 and September 30, 2003, with an annual precipitation value of 18.54 inches which is close to the average annual precipitation value for Midnite Mine station of 18.35. The wet year was selected as October 1, 2005 through September 30, 2006, with an annual precipitation value of 25.77 inches, which is the maximum annual precipitation for the climate station record. The monthly and annual precipitation values for the model years selected are summarized in Table 3. The daily precipitation values for the typical and wet year are shown in Figures 7 and 8, respectively.

Table 3. Midnite Mine Monthly Precipitation for Model Years

Month	Typical Year (Midnite Mine Climate Station Year 2002-2003) Precipitation[a] (in)	Wet Year (Midnite Mine Climate Station Year 2005-2006) Precipitation[a] (in)
Oct	0.01	1.51
Nov	1.74	2.78
Dec	3.46	4.23
Jan	4.86	7.24
Feb	0.77	1.37
Mar	2.78	1.52
Apr	1.92	2.33
May	1.47	0.97
Jun	0.14	2.49
Jul	0.00	0.46
Aug	0.21	0.32
Sep	1.18	0.55
Total	**18.54**	**25.77**

[a]Daily data is used in the model.

4.2.2 Vegetation Data

The vegetation species currently being considered for the cover are listed in Table 4 and discussed in more detail in Attachment D-12.

Vegetation parameters for the numerical model were selected to represent average, or typical conditions for these plant species. The vegetation parameters that define how plants remove water from the soil profile in the model are the root depth, leaf area index (LAI), and the plant moisture limiting function. Cover profiles were modeled using average vegetation parameters.

The rooting depth in the model determines the depth at which plants are able to extract water. Only the fully developed cover profile was simulated; therefore, root growth was not included in the model. The range and average rooting depths for the species are listed in Table 4. The average long-term rooting depth of the vegetated species is approximately 24 inches. A rooting depth of 22 inches was used in the model. This value was used for the 60% design and since

this value is slightly less (more conservative) than the average rooting depth of the proposed species, the value was not revised for 90% design.

Table 4. Rooting Depths for Proposed Vegetation Species

Vegetation Species	Rooting Depth[a]		
	Minimum	**Maximum**	**Average**
Bluebunch wheatgrass (*Pseudoroegneria spicata*)	254 mm (10 in)	1219 mm (48 in)	737 mm (29 in)
Idaho fescue (*Festuca idahoensis*)	356 mm (14in)	610 mm (24 in)	483 mm (19 in)
Slender wheatgrass (*Elymus trachycaulus*)	305 mm (12 in)	610 mm (24 in)	457 mm (18 in)
Sandberg bluegrass (Poa secunda)	254 mm (10 in)	457 mm (18 in)	356 mm (14 in)
Canada bluegrass (*Poa compressa*)	254 mm (10 in)	457 mm (18 in)	356 mm (14 in)
Indian ricegrass (*Achnatherum hymenoides*)	457 mm (18 in)	610 mm (24 in)	534 mm (21 in)
Green needlegrass (Nassella viridula)	356 mm (14 in)	610 mm (24 in)	483 mm (19 in)
Western yarrow (*Achillea millefolium*)	203 mm (8 in)	914 mm (36 in)	559 mm (22 in)
Silvery lupine (*Lupinus argenteus*)	152 mm (6 in)	610 mm (24 in)	381 mm (15 in)
Arrowleaf balsamroot (*Balsamorhiza sagittata*)	356 mm (14 in)	914 mm (36 in)	635 mm (25 in)
Snowberry (*Symphoricarpos albus*)	457 mm (18 in)	1524 mm (60 in)	991 mm (39 in)
Wood's rose (*Rosa woodsii*)	305 mm (12 in)	1219 mm (48 in)	762 mm (30 in)
Serviceberry (*Amelanchier alnifolia*)	610 mm (24 in)	1524 mm (60 in)	1067 mm (42 in)
Average	***330 mm (13 in)***	***868 mm (34 in)***	***600 mm (24 in)***

[a]From Redente (2014)

The LAI is used as an indicator of growing season and transpiration, and is used to determine the potential transpiration from potential evapotranspiration. In reality, LAI values are expected to vary from year to year, with typically heavier LAI development during wetter years. However, the model results are not sensitive to these variations. Therefore, the LAI function was assumed to be the same for each year of the simulation.

The LAI function used in the model was developed by fitting a function through the monthly values provided by Redente (2012) for the proposed plant species for the 60% BODR (MWH, 2013) for average vegetation conditions. The monthly LAI values used to develop the function for the model are shown in Table 5. The average LAI values for the proposed species for the 90% design listed in Table 4 are also shown in Table 5. Since the monthly LAI values used for the 60% design are equal to or slightly lower (more conservative) than the average monthly LAI

values for the proposed species for the 90% design, the LAI function was not revised for 90% design.

Table 5. Monthly LAI Values

Month	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
LAI_{ave}[a] (used in model)	1.0	0.2	0.0	0.0	0.0	0.5	1.0	1.6	2.4	2.2	2.0	1.8
LAI_{ave}[b] for 90% Design Species	1.6	0.2	0.0	0.0	0.0	0.8	1.6	2.5	3.0	3.0	2.5	2.0

[a] From Redente (2012)
[b] From Redente (2014)

The plant moisture limiting function determines the percentage decrease in the plants ability to draw water as the negative water pressure increases in the unsaturated ground. A typical function from Geo-Slope International, Ltd (2012) was used in the model which assumes the wilting point suction for the soil is 1500 kilopascal (kPa) and is consistent with recommendations provided in Albright et al. (2010).

4.2.3 Soil Properties

The soil parameters for the materials used in the analyses are summarized in Table 6.

Table 6. Cover Material Properties

Parameter	Value
Thickness	0.91m (3.0 ft)
Dry Density (g/cc)	1.63
Coefficient of Volume Change, m_v (1/kPa)	1×10^{-5}
Saturated Water Content, θ_s (-)	0.38
Residual Water Content, θ_r (-)	0.0
SWCC coefficient[(a)], a (kPa)	14.75
SWCC coefficient[(a)], n (-)	0.62
SWCC coefficient[(a)], m (-)	0.59
Saturated Hydraulic Conductivity, K_{sat} (cm/s)	5.9×10^{-4}

(a) Fredlund and Xing (1994) soil water retention and hydraulic conductivity function parameters

The function for the soil water characteristic curve (SWCC) for the Rhoads Property Borrow Area was estimated based on measured SWCCs with the Fredlund and Xing (1994) fit and estimated coefficient of volume change. The SWCCs were adjusted to account for gravel using the method presented by Bower and Rice (1984). The coefficient of volume change was estimated from the following equation:

$$m_v = \frac{1}{D} \qquad \text{(Eqn. 4)}$$

where D is the constrained modulus (Lambe and Whitman, 1969). The constrained modulus was estimated from the following equation:

$$D = \frac{E(1-\mu)}{(1+\mu)(1-2\mu)} \qquad \text{(Eqn. 5)}$$

where E is the Young's modulus, and μ is the Poisson's ratio (Lambe and Whitman, 1969).

Typical m_v values for the cover soils were calculated using the above equations. Typical Young's modulus and Poisson's ratio values for clayey sand (Bowles, 1988) are used in Equation 5. The m_v value used in the model was 1 x 10^{-5} 1/kPa.

The SWCCs were adjusted to match the estimated saturated water content at a suction of zero. The saturated volumetric water content was estimated based on the porosity of the cover material. The porosity was calculated using the average measured specific gravity and maximum standard Proctor dry density for collected samples from the borrow area. It was assumed that the cover materials will be placed at approximately 85 percent of standard Proctor compaction. The average SWCC is presented in Figure 9. The average SWCC was used for the model runs.

The function for unsaturated hydraulic conductivity was estimated using Fredlund and Xing (1994) estimation method in VADOSE/W with the estimated SWCC and geometric mean of measured saturated hydraulic conductivity for samples from the borrow area (MGC, 2011a). The hydraulic conductivity function is presented in Figure 10.

The thermal conductivity and volumetric specific heat values used in the model were estimated based on typical values provided in Vadose/W (Geo-Slope International Ltd., 2012).

The physical and hydraulic properties of the cover soils are not expected to experience post-construction changes due to the proposed emplaced conditions. As noted in NUREG CR-7028 (Benson et. al, 2011), potential changes to the cover can be minimized by designing the cover system to be as close as practical to the anticipated equilibrium state under long-term conditions; furthermore, their study also noted that long-term changes are more prone to occur for less permeable soils compared to more permeable soils. As noted in Attachment D-3, the cover system will be placed at a low relative compaction (average of approximately 85 percent of standard Proctor compaction) to match long-term conditions and minimize potential changes to the cover system due to pedogenic processes.

4.3 MODEL CONDITIONS

4.3.1 Initial Conditions

The initial conditions in the cover profile at the start of the simulation years were established from the typical climate data year (refer to Section 4.2.1). In order to establish the initial conditions, simulations were run repeatedly with the typical year climate data until successive year-end nodal pressure values throughout the profile converged. For the 2 ft and 3 ft models, this occurred within four years. For the 6 ft models, this occurred within 14 years. The

converged nodal pressure distribution at the end of the 4th year model run (or 14th year for 6 ft model cases) was used as the initial condition for the final model runs. VADOSE/W computes the initial water content from the initial pressure head using the soil water characteristic curve parameters.

4.3.2 Boundary Conditions

The upper boundary of the domain was assigned an atmospheric boundary condition to allow time-variable inputs of precipitation and potential evapotranspiration. This is consistent with recommendations in GEOSLOPE (2012) and Albright et al. (2010). The atmospheric boundary condition was specified to allow water unable to infiltrate the soil surface to be removed from the model as surface runoff or to pond on the surface. The atmospheric boundary condition was discretized in the time domain on a daily (24-hour) basis. As described in Section 4.1.2, a 24-hour discretization is equivalent to allowing limited surface ponding and tends to underestimate runoff (for the cases where runoff was allowed) and overestimate infiltration during intense precipitation events on sloped ground surfaces. However, a 24-hour discretization corresponds to the historical reporting increment for the RAWS data and is considered reasonably conservative.

For Cases A1 and A2, the bottom boundary condition was assigned as a no flow boundary to estimate the head of water that develops on an impermeable geomembrane over time. For Case A3, B1, and B2, the bottom boundary was assigned a “seepage face with review” boundary condition. The “seepage face with review” boundary condition does not allow flow to occur across the boundary in either direction under unsaturated (negative pressure head) conditions, but maintains a free-flowing, zero-pressure-head state under saturated conditions.

Upper and lower boundary conditions are summarized in Table 7.

Table 7. Model Boundary Conditions

Cases Analyzed	Upper Boundary Condition	Lower Boundary Condition
Case A - As Constructed Conditions		
Case A1 – mildly sloped areas, no GDL	No ponding	No flow boundary
Case A2 – flat areas, no GDL	Ponding allowed	No flow boundary
Case A3 – sloped areas, GDL overlies GM	No Ponding	Seepage face with review
Case B - Long-Term Degraded Conditions		
Case B1 – mildly and sloped areas, no GDL or GM	No ponding	Seepage face with review
Case B2 – flat areas, no GDL or GM	Ponding allowed	Seepage face with review

4.3.3 Node Spacing and Convergence Tolerances

Node spacings used for the finite-element mesh were selected to provide reasonable convergence and water-balance error limits, and follow general recommendations provided by Albright et al. (2010) and GEOSLOPE (2012). A tighter spacing of nodes were incorporated at the soil surface, where pore pressure and hydraulic conductivity variations are the greatest.

The 10 mm wide model was constructed using a variable node spacing ranging from 5 mm at the soil surface to approximately 30 mm. The convergence tolerances for the simulations were set at 0.001 m for the pressure head and 0.01 degrees Celsius for the temperature. The combination of node spacing and convergence tolerances resulted in mass balance errors over the entire model domain of less than one percent for all model runs. As a general rule, errors of five percent or less are considered acceptable.

5.0 RESULTS

The results of analyses of as-constructed conditions are summarized in Section 5.1. Results for analyses of long-term conditions where the geomembrane has degraded to the extent that it no longer has any effect on the hydraulic of the cover system (i.e. the geomembrane is ignored) are included in Section 5.2.

Model simulations and leakage calculations were performed for typical and wet period climate conditions. Typical conditions were represented by modeling one typical climate year, with analyses started using the initial conditions developed as described previously. Wet year conditions were analyzed based upon the third of three consecutive wet years following a typical year, and using initial conditions developed as described previously.

5.1 AS-CONSTRUCTED CONDITIONS

The cumulative leakage rate through the cover for as constructed conditions, assuming 2-mm diameter defects occur in the geomembrane at a rate of one defect per acre, are presented in Figures 11, 12, and 13 and are summarized in Table 8. The annual cumulative percolation at the bottom of the cover system (presented as a percentage of annual precipitation) for typical and wet year conditions are shown on Figure 11. The results show that the cover system as designed (3-foot thickness) under as-constructed conditions will reduce infiltration to 0.015 percent or less of annual precipitation under both typical and wet year climate conditions. The majority of the percolation occurs in the relatively flat drainage bench areas where surface ponding may occur.

The annual cumulative percolation at the bottom of the cover system (presented as a percentage of annual precipitation) is plotted versus cover thickness for typical and wet year conditions on Figure 11. Sensitivity analyses for cover thickness indicate that cover thickness has very little impact on the leakage rate under as-constructed conditions. Although slightly higher percolation rates were calculated for the 2 and 6 feet cases versus the 3 feet case, this may largely be the result of the uncoupled calculation procedure used to estimate leakage through the geomembrane. The leakage calculations for the as-constructed conditions calculate geomembrane leakage as a function of the head of water on the geomembrane. As the head on the geomembrane was calculated from the Vadose/W analyses where no leakage was allowed to occur through the geomembrane, these heads (and consequently the leakage through the geomembrane) would be somewhat overestimated in the vicinity of the defect. This effect may be more pronounced in the case of the 6-foot cover, where the additional domain from 3 to 6 feet results in a larger head build up on the geomembrane.

Table 8. Percolation Results for As Constructed Conditions (Case A)

Cover Thickness (ft)	Area	Percolation (mm/yr)		Percolation as Percent of Annual Precipitation (%)	
		Typical Year Climate Conditions	Wet Period Climate Conditions*	Typical Year Climate Conditions	Wet Period Climate Conditions*
2	A1	0.000	0.063	0.000	0.010
	A2	0.530	1.636	0.112	0.250
	A3	0.000	0.005	0.000	0.001
	Entire Cover	**0.030**	**0.109**	**0.006**	**0.017**
3	A1	0.000	0.000	0.000	0.000
	A2	0.276	1.707	0.059	0.261
	A3	0.000	0.000	0.000	0.000
	Entire Cover	**0.016**	**0.097**	**0.003**	**0.015**
6	A1	0.000	0.000	0.000	0.000
	A2	0.919	2.949	0.195	0.450
	A3	0.000	0.000	0.000	0.000
	Entire Cover	**0.052**	**0.168**	**0.011**	**0.026**

*Results are for the final model year of three consecutive wet years following a typical climate year.

5.2 LONG-TERM DEGRADED CONDITIONS

As an upper bound for long-term degraded conditions, percolation through the cover system was modeled assuming the geosynthetic layers will completely degrade and no longer have any effect on the hydraulic of the cover system (i.e. the effects of the geomembrane and GDL layers are completely ignored). The results for these analyses are presented in Figures 14, 15, and 16 and summarized in Table 9. The results show that the 3-foot cover system will reduce infiltration to approximately 0.7 percent of annual precipitation under typical year, and approximately 2.3 percent of annual precipitation under wet year climate conditions. As with the previously-described cases, the majority of the percolation occurs in the relatively flat drainage bench areas where surface ponding may occur.

The annual cumulative percolation through the completely degraded geomembrane (presented as a percentage of annual precipitation) is plotted versus cover thickness for typical and wet year conditions on Figure 14. The results indicate that percolation is decreased by approximately 50 percent under typical year conditions if the soil cover thickness is increased from 2 feet to 3 feet; however, the reduction in percolation becomes less pronounced (30 percent) as the cover thickness is increased to 6 feet. The effect of cover thickness on percolation rates is less significant when considering wet year conditions.

Table 9. Percolation Results for Long-Term Degraded Conditions (Cases B1 and B2, no effect of geosynthetic layers on flow)

Cover Thickness (ft)	Area	Percolation (mm/yr)		Percolation as Percent of Annual Precipitation (%)	
		Typical Year Climate Conditions	Wet Period Climate Conditions*	Typical Year Climate Conditions	Wet Period Climate Conditions*
2	B1	0.00	0.00	0.00	0.00
	B2	104	288	22.0	44.0
	Entire Cover	**5.90**	**16.41**	**1.25**	**2.51**
3	B1	0.00	0.00	0.00	0.00
	B2	54	266	11.4	40.6
	Entire Cover	**3.09**	**15.17**	**0.66**	**2.32**
6	B1	0.00	0.00	0.00	0.00
	B2	35	249	7.4	38.0
	Entire Cover	**2.00**	**14.19**	**0.42**	**2.17**

*Results are for the final model year of three consecutive wet years following a typical climate year.

6.0 CONCLUSIONS

The results of cover system infiltration analyses under as-constructed and long-term degraded geomembrane conditions indicate the following:

1. As-constructed percolation rates for the 3-foot cover presented in the 60% BODR using Rhoads Property Borrow Area soils are expected to be very low. The estimated percolation rates through the synthetic geomembrane liner geomembrane are 0.016 mm/year under typical climate conditions and 0.097 mm/year under wet period climate conditions. This indicates that percolation through the cover system will be 0.015 percent, or less, of the annual precipitation at the site under both typical and wet period conditions. A significant portion of the water percolating through the cover system and into the underlying waste is expected to originate from the relatively flat drainage bench areas that experience long-term ponding on the ground surface.

2. Percolation rates for the 3-foot cover using Rhoads Property Borrow Area soils are also expected to be low under upper bound long-term degraded conditions. The estimated percolation rates through the degraded synthetic geomembrane liner are 3.1 mm/year under typical climate conditions and 15.2 mm/year under wet period climate conditions. The estimates for percolation under degraded conditions represent upper bound conditions where the synthetic liner is completely disintegrated to the point that it has no influence on the hydraulics of the cover system. These rates indicate that the annual percolation through the cover system could be as high as 0.7 percent of the annual precipitation during typical years, and 2.3 percent of annual precipitation during wet period conditions. As with the as-constructed condition, a significant portion of the water percolating through the cover system is expected to originate from the relatively flat drainage bench areas that experience long-term ponding on the ground surface.

3. The effect of soil cover thickness on percolation through the cover system and into the underlying waste rock was relatively minor for the range of soil covers evaluated. Increasing the soil cover from 3 feet to 6 feet resulted in a moderate increase in estimated percolation under as-constructed conditions due to the potential for increased ponding on the geomembrane below the root zone. As discussed previously, this increase may be the result of increased conservatism in the de-coupled calculation procedure used to estimate percolations rates as soil cover thicknesses increases. In reality, the percolation rates for the 3-foot and 6-foot cover systems under as-constructed conditions are expected to be similar. Under completely degraded conditions where the geomembrane is assumed to no longer have any effect on flow through the cover system, percolation rates may decrease by a minor amount under typical year, and an insignificant amount under wet year conditions, as the cover thickness is increased from 3 feet to 6 feet.

4. Grading of the surface of the cover system to reduce the potential for long-term ponding is seen to have a very significant impact on minimizing percolation rates through the cover system. For as-constructed conditions, estimated annual percolation rates for the 3-foot cover system increase from 0 mm/year under “no ponding” conditions to 0.3 mm/year under “ponding” conditions for a typical-year and from 0 mm/year to 1.7 mm/year under wet year conditions. Under fully-degraded conditions where the geomembrane liner is assumed to have no effect on flow through the 3-foot cover system, annual percolation rates increase from 0 mm/year under “no ponding” conditions to 54 mm/year under typical year conditions and from 0 mm/year to 266 mm/year under wet year conditions. Based upon these results, reducing areas where long-term ponding can occur will have the most significant effect on reducing percolation through the cover system, both in the short-term and long-term. In order to minimize infiltration, it is recommended that the relatively flat top-surfaces and bench areas be graded to slopes ranging from 2 to 5 percent wherever possible.

7.0 REFERENCES

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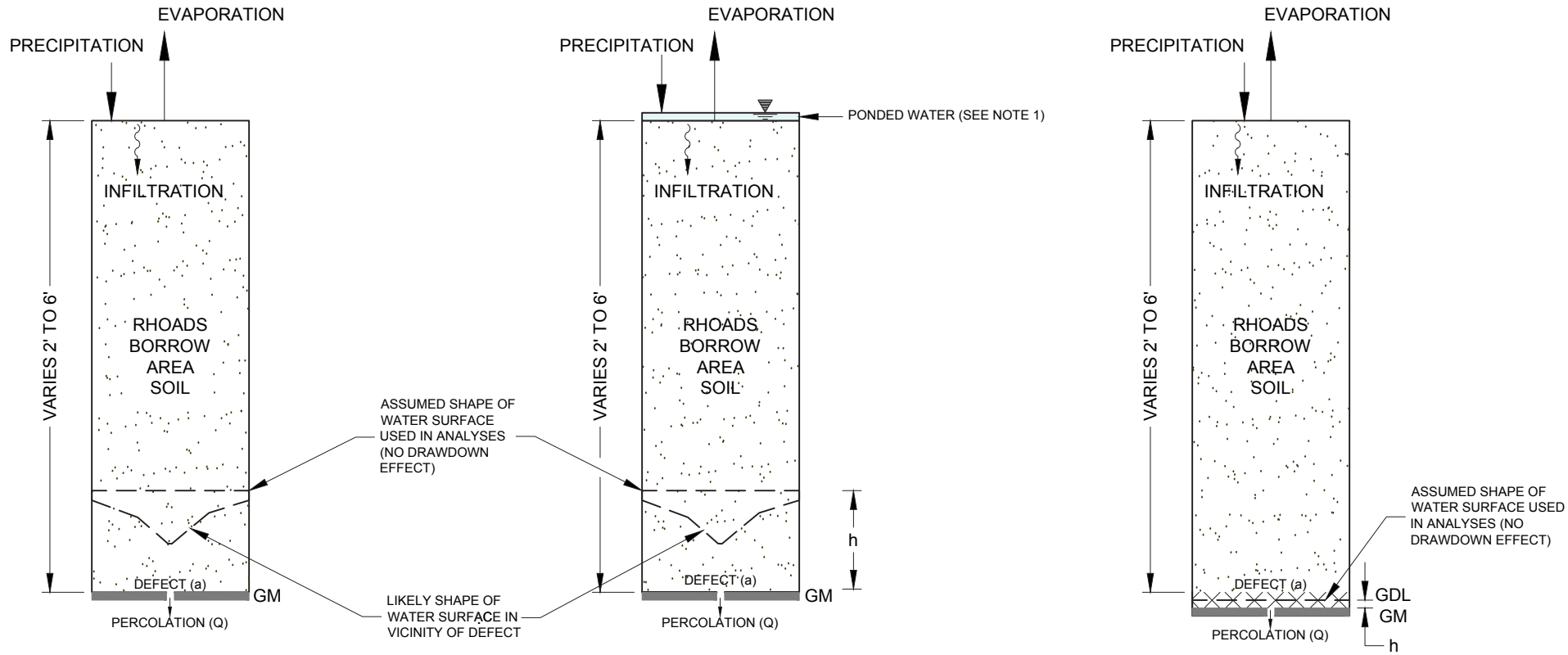
FIGURES

CASE A. AS CONSTRUCTED CONDITIONS



CASE A1: MILDLY SLOPED AREAS WITH NO SURFACE PONDING, NO GDL ABOVE THE GM

CASE A2: FLAT DRAINAGE BENCH AREAS WHERE SURFACE PONDING MAY OCCUR, NO GDL ABOVE THE GM

CASE A3: SLOPED AREAS WITH NO SURFACE PONDING, GDL OVERLIES GM

NOTES:

1. PONDING OCCURS WHEN DAILY PRECIPITATION EXCEEDS SUM OF DAILY INFILTRATION AND DAILY EVAPORATION. PONDED WATER CAN ONLY BE REMOVED BY EVAPORATION OR INFILTRATION.
2. 2 mm DIAMETER DEFECT PER ACRE ASSUMED FOR EACH CASE
3. GM = GEOMEMBRANE
4. GDL = GEOCOMPOSITE DRAINAGE LAYER

NEWMONT

PROJECT: MIDNITE MINE

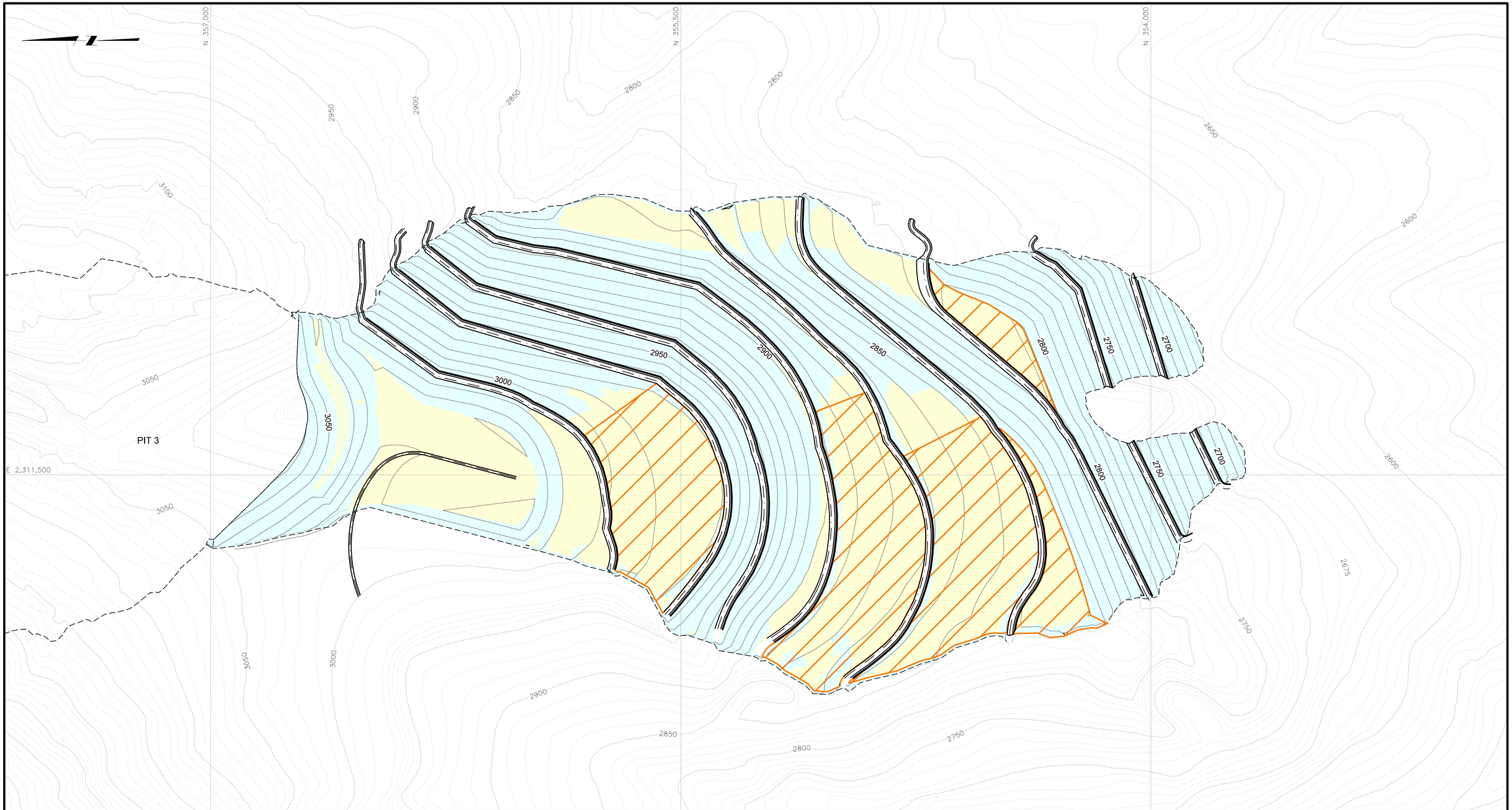
TITLE: CASE A. AS CONSTRUCTED CONDITIONS

MWH

DATE: NOV 2013

FIGURE 1

FILE NAME: 2013-06-28 GM CONDITIONS



LEGEND:

- AREA SLOPE < 15.0%
- AREA SLOPE > 15.0%
- NO GDL

ACRONYMS:

GDL GEOCOMPOSITE DRAINAGE LAYER
GM GEOMEMBRANE

DESIGNATION	DESCRIPTION	AREA (SF)
A1 (PIT 3)	MILDLY SLOPED AREAS (LESS THAN 15%) WITH NO GDL OVERLYING GM	769,182
A2 (PIT 3)	FLAT AREAS ON THE DRAINAGE BENCHES WHERE FLOW IS CONCENTRATED AND SURFACE PONDING MAY OCCUR AND THERE IS NO GDL OVERLYING THE GM. (THIS AREA LISTED IS BASED UPON THE ASSUMPTION THAT 50% OF THE TOTAL DRAINAGE BENCH AREA WILL FALL INTO THIS CLASSIFICATION)	177,046
A3 (PIT 3)	STEEPER SLOPED AREAS WITH NO SURFACE PONDING AND A GDL OVERLIES THE GM	2,452,991

0 150' 300'
CONTOUR INTERVAL = 10'

DESIGNED BY	M. DAVIS	06-14
DRAWN BY	D. MIRANDA	06-14
CHECKED BY	T. KELLEY	06-14
APPROVED BY	T. KELLEY	06-14
PROJECT MANAGER	V. DRAIN	06-14
CLIENT APPROVAL	----	----
CLIENT REFERENCE NO.	----	

NEWMONT

PROJECT LOCATION: WELLPINIT, WASHINGTON
PROJECT: MIDNITE MINE
TITLE: PIT 3 DESIGNATION OF COVER AREAS WITH AND WITHOUT GEOCOMPOSITE DRAINAGE LAYER

MWH

FIGURE 2
REVISION 0
FILE NAME pit 3
DATE JUNE 2014

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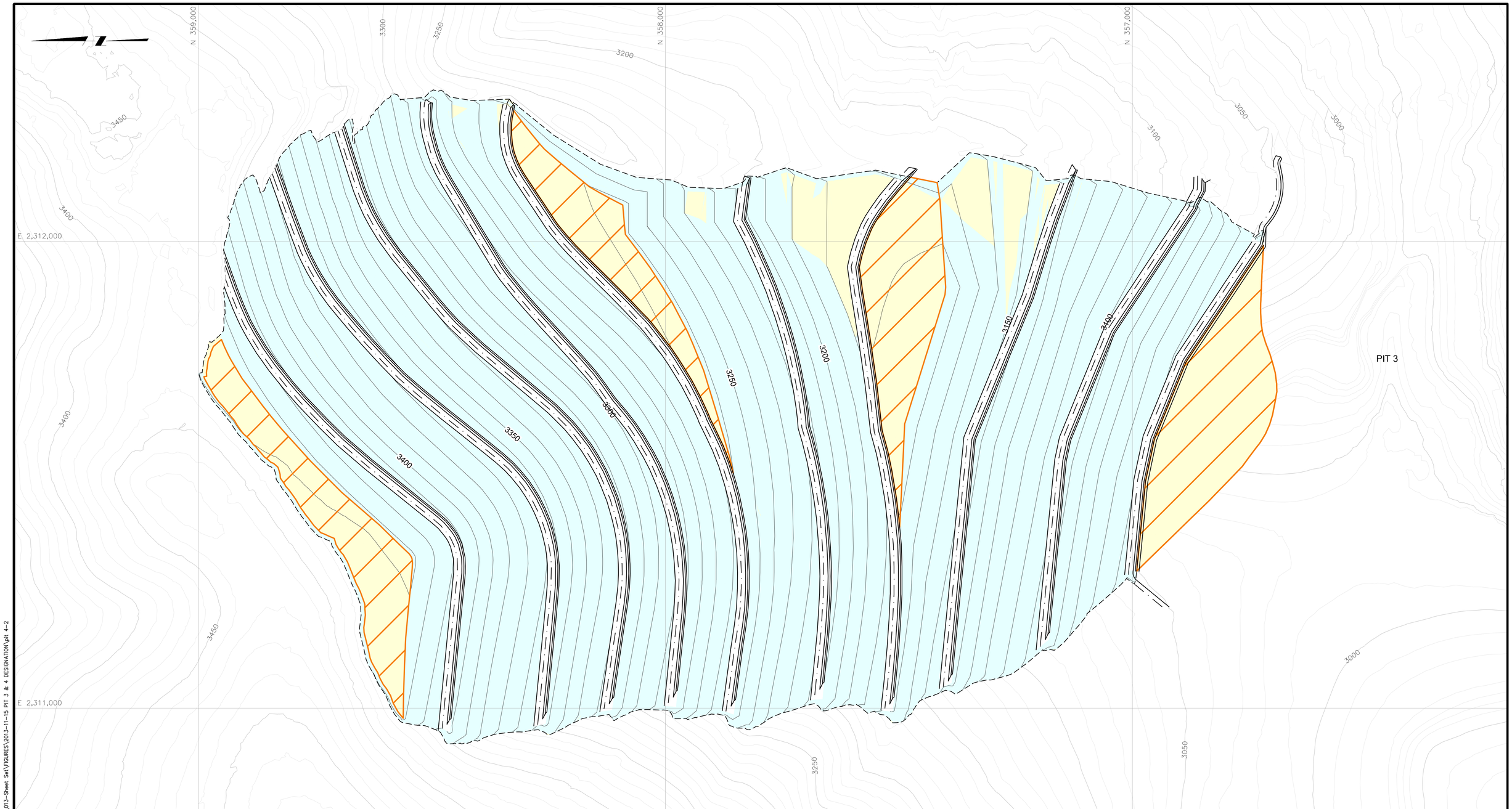


LEGEND

- AREA SLOPE < 15.0%
- AREA SLOPE > 15.0%
- NO GDL

ACRONYMS:

GDL GEOCOMPOSITE DRAINAGE LAYER
GM GEOMEMBRANE

DESIGNATION	DESCRIPTION	AREA (SF)
A1 (PIT 4)	MILDLY SLOPED AREAS (LESS THAN 15%) WITH NO GDL OVERLYING GM	289,028
A2 (PIT 4)	FLAT AREAS ON THE DRAINAGE BENCHES WHERE FLOW IS CONCENTRATED AND SURFACE PONDING MAY OCCUR AND THERE IS NO GDL OVERLYING THE GM. (THIS AREA LISTED IS BASED UPON THE ASSUMPTION THAT 50% OF THE TOTAL DRAINAGE BENCH AREA WILL FALL INTO THIS CLASSIFICATION)	152,361
A3 (PIT 4)	STEEPER SLOPED AREAS WITH NO SURFACE PONDING AND A GDL OVERLIES THE GM	1,972,484

0 100' 200'
CONTOUR INTERVAL = 10'

DESIGNED BY	M. DAVIS	06-14
DRAWN BY	D. MIRANDA	06-14
CHECKED BY	T. KELLEY	06-14
APPROVED BY	T. KELLEY	06-14
PROJECT MANAGER	V. DRAIN	06-14
CLIENT APPROVAL	----	----
CLIENT REFERENCE NO.	----	

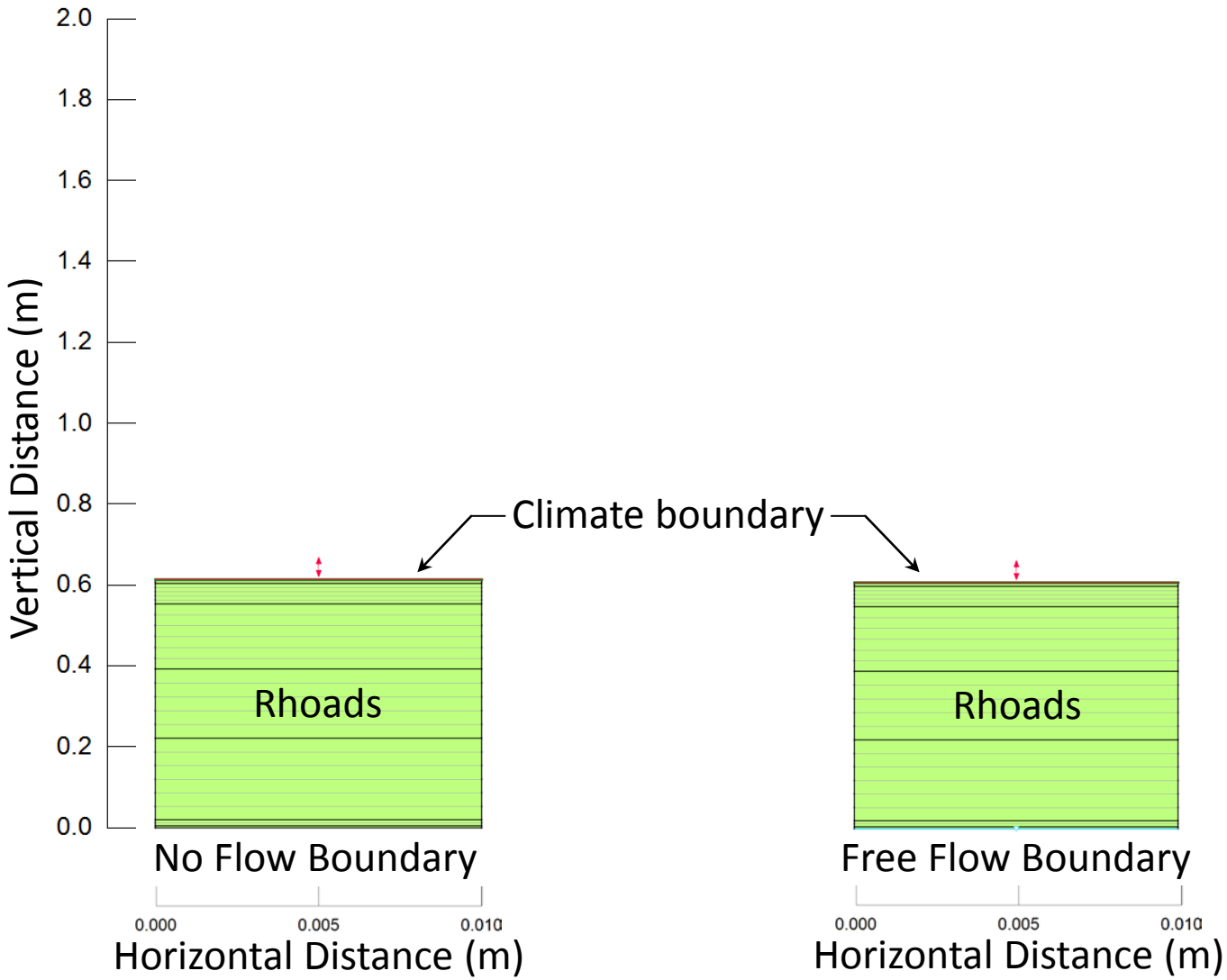
NEWMONT

PROJECT LOCATION WELLPINIT, WASHINGTON
PROJECT MIDNITE MINE
TITLE PIT 4 DESIGNATION OF COVER AREAS WITH AND WITHOUT GEOCOMPOSITE DRAINAGE LAYER

MWH

FIGURE 3
REVISION 0
FILE NAME pit 4-2
DATE JUNE 2014

L:\Design-Drafting\Clients-I-P\NEWMONT\101322 MIDNITE MINE\013-Sheet Set\FIGURES\2013-11-15 PIT 3 & 4 DESIGNATION\pit 4-2



NEWMONT

PROJECT
MIDNITE MINE

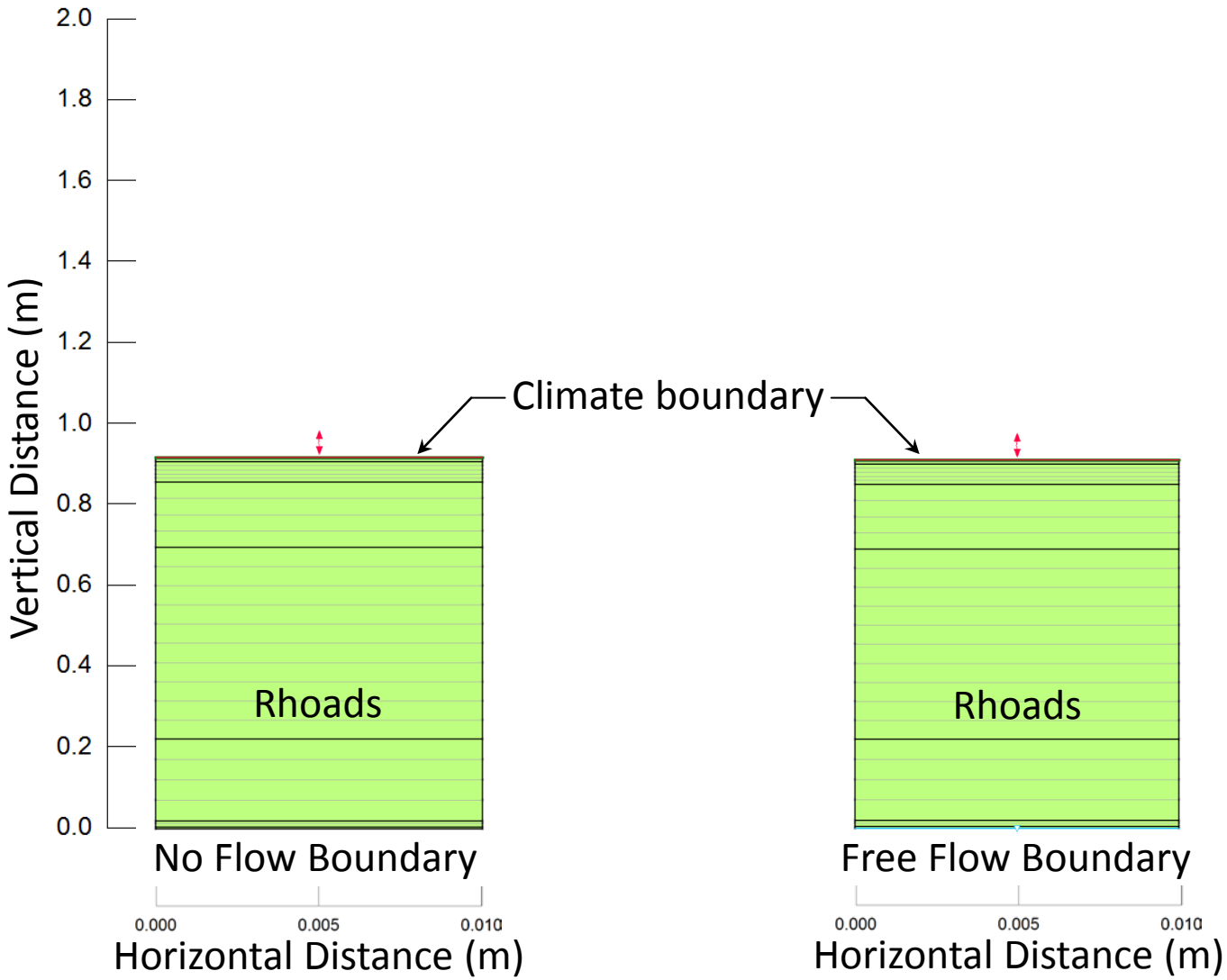
TITLE
Model Geometry – 2 ft (0.61 m) models
Rhoads Property Borrow

MWH

DATE July 2013

FIGURE 4

FILENAME Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

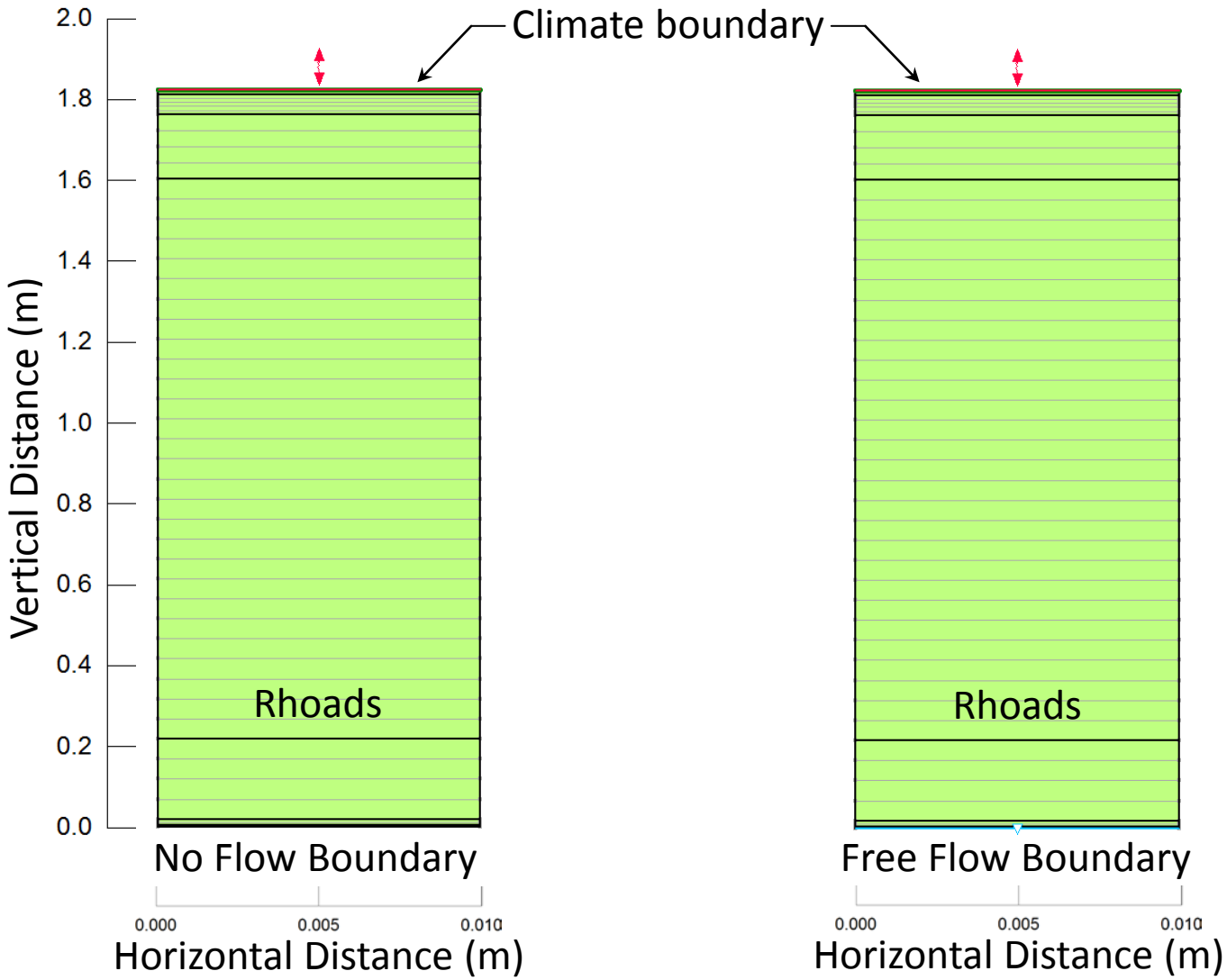
TITLE
Model Geometry – 3 ft (0.91 m) models
Rhoads Property Borrow

MWH

DATE July 2013

FIGURE 5

FILENAME Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

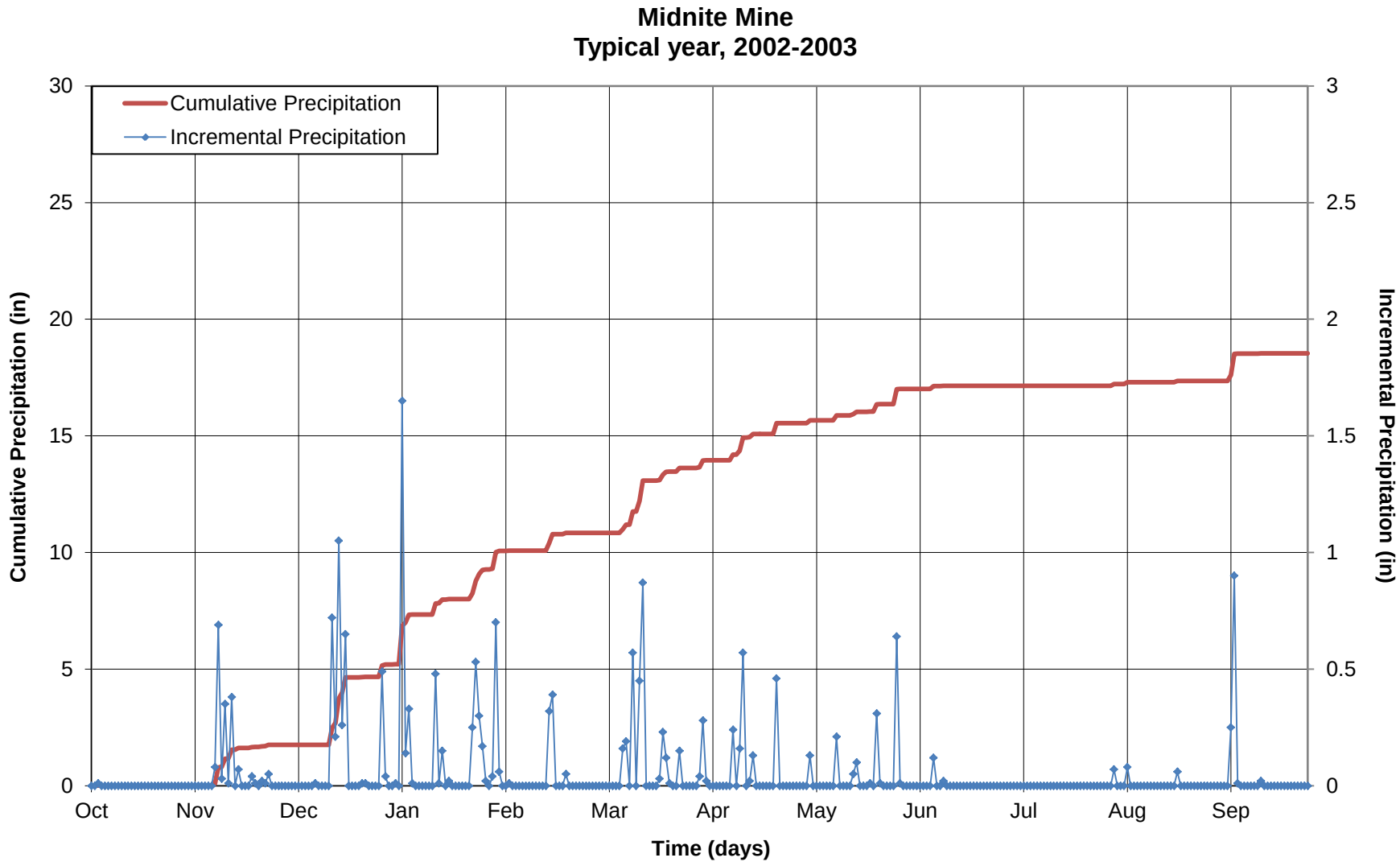
TITLE
Model Geometry – 6 ft (1.83 m) models
Rhoads Property Borrow

MWH

DATE July 2013

FIGURE 6

FILENAME Infil Calc Brief Figures 60%.pptx



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PROJECT
MIDNITE MINE

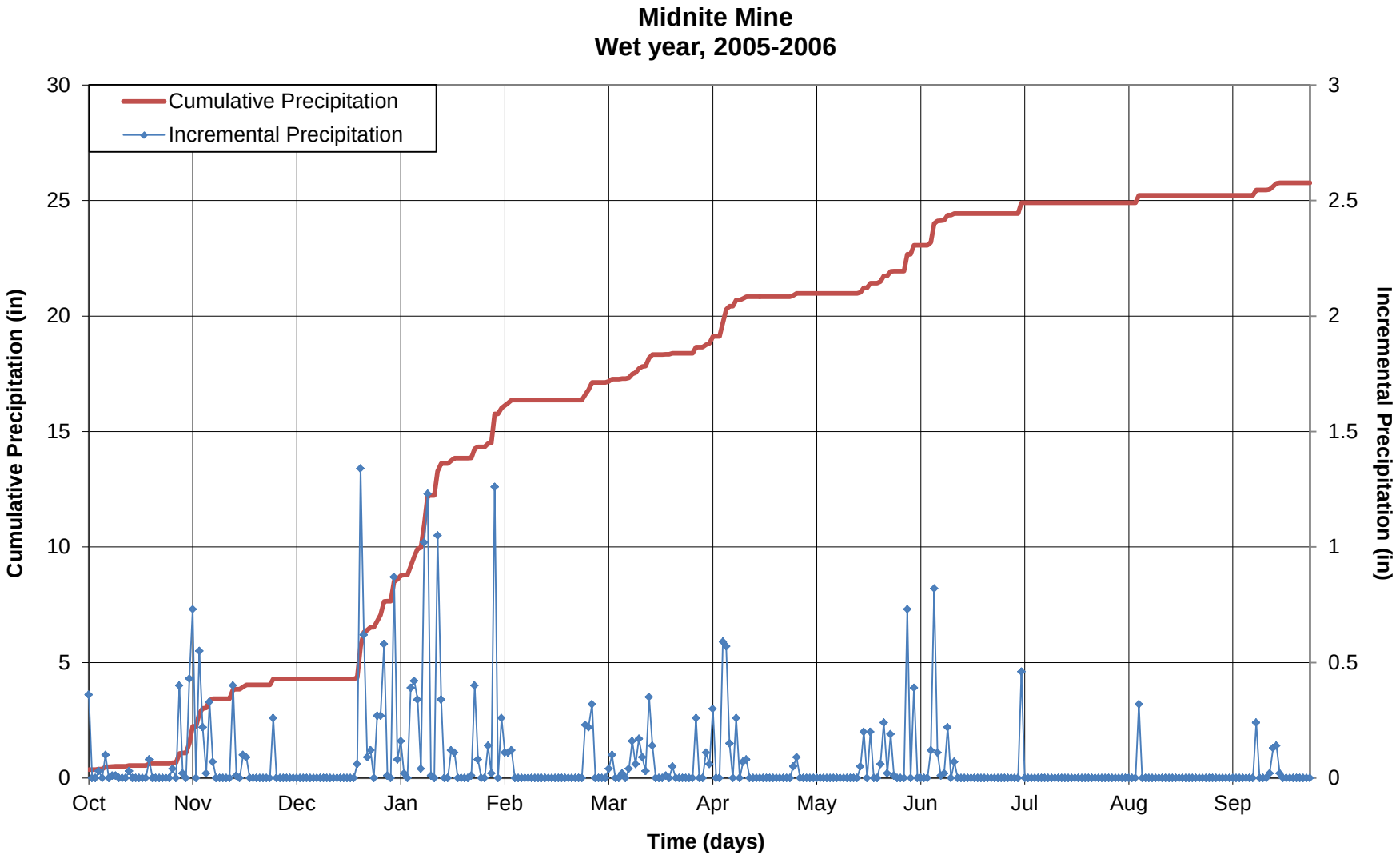
TITLE
Typical Year Daily Precipitation Record

MWH

DATE
July 2013

FIGURE 7

FILENAME
Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

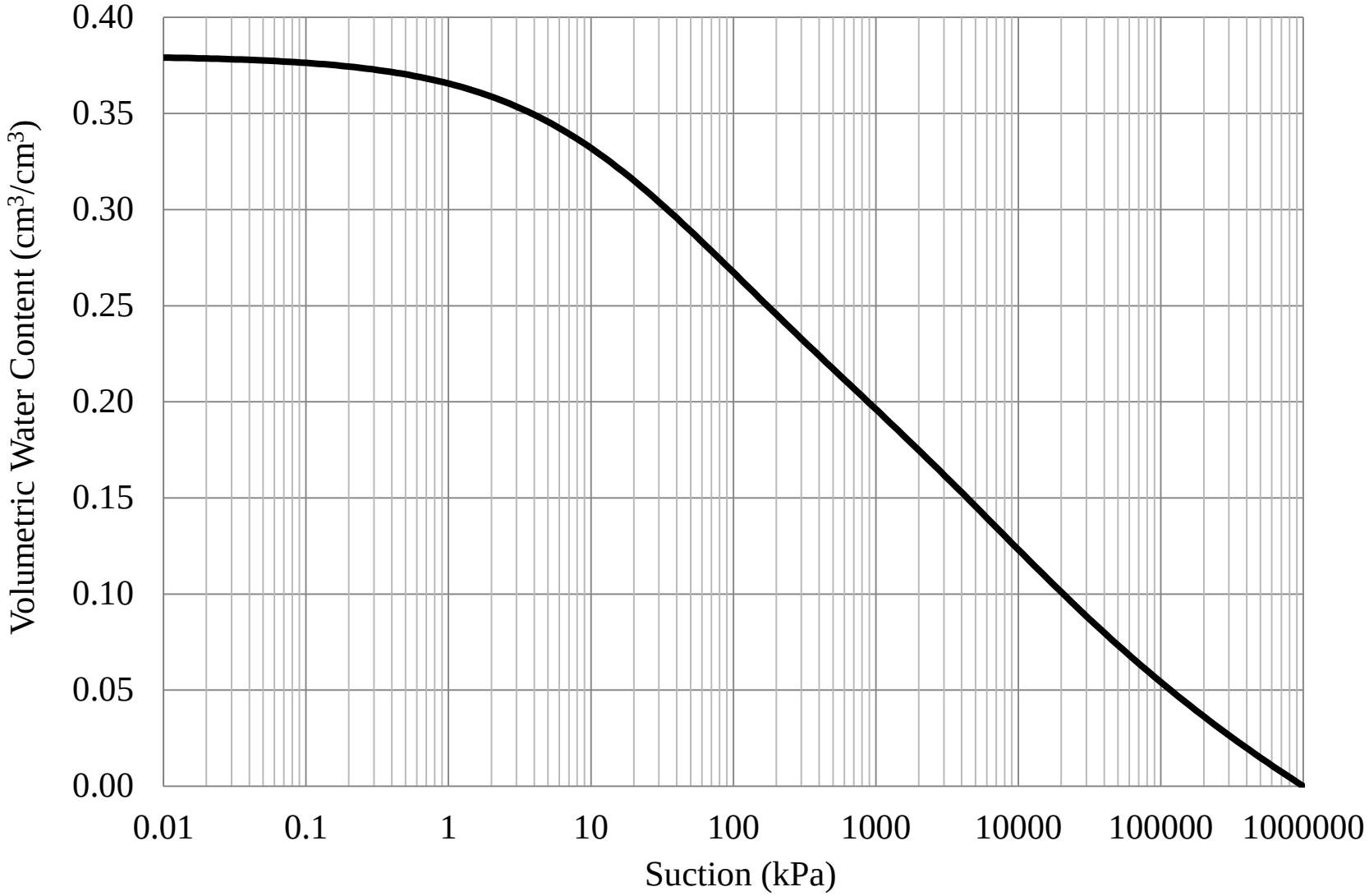
TITLE
Wet Year Daily Precipitation Record

MWH

DATE
July 2013

FIGURE 8

FILENAME
Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

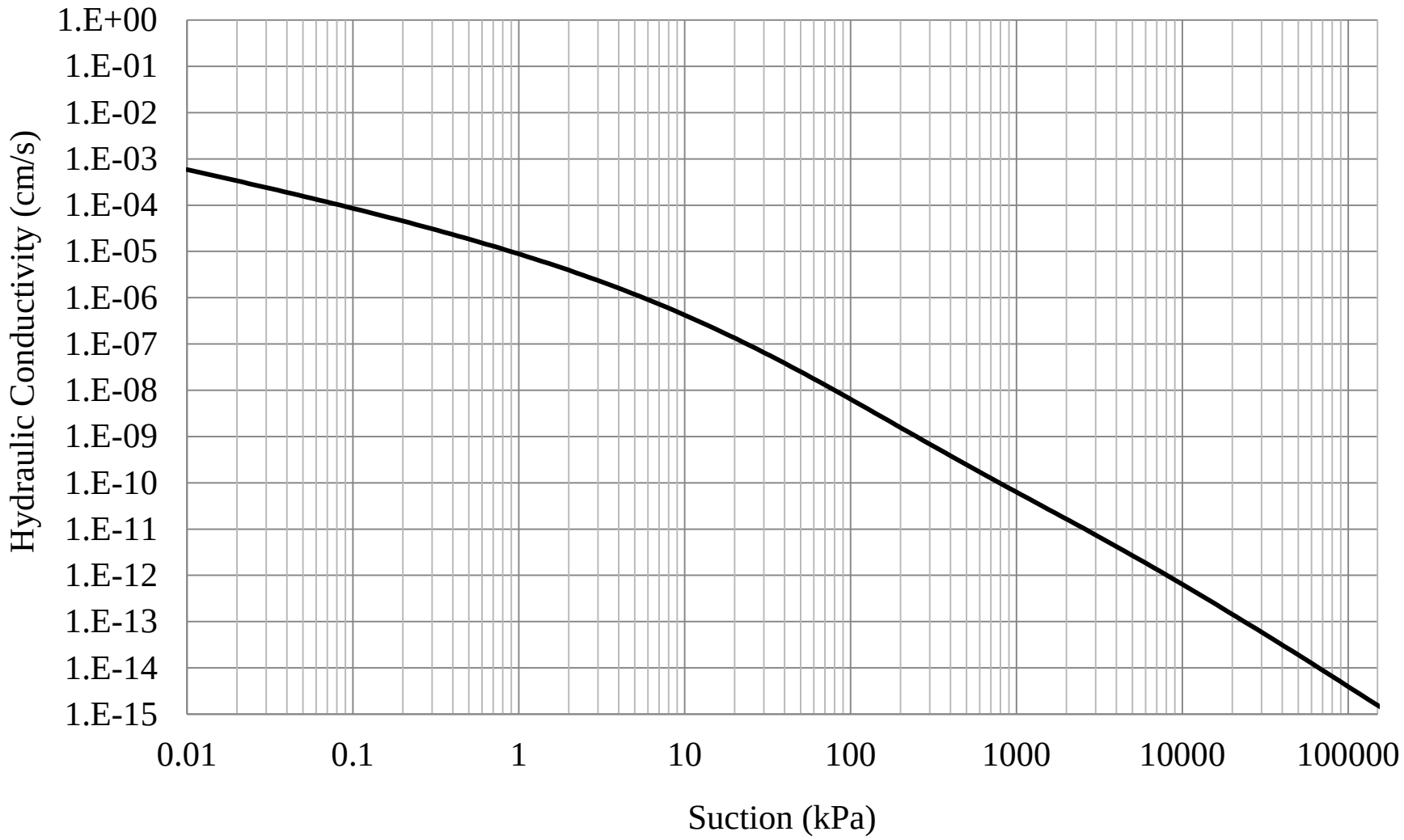
TITLE
Estimated Soil Water Characteristic Curve for Rhoads Property Borrow

MWH

DATE July 2013

FIGURE 9

FILENAME Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

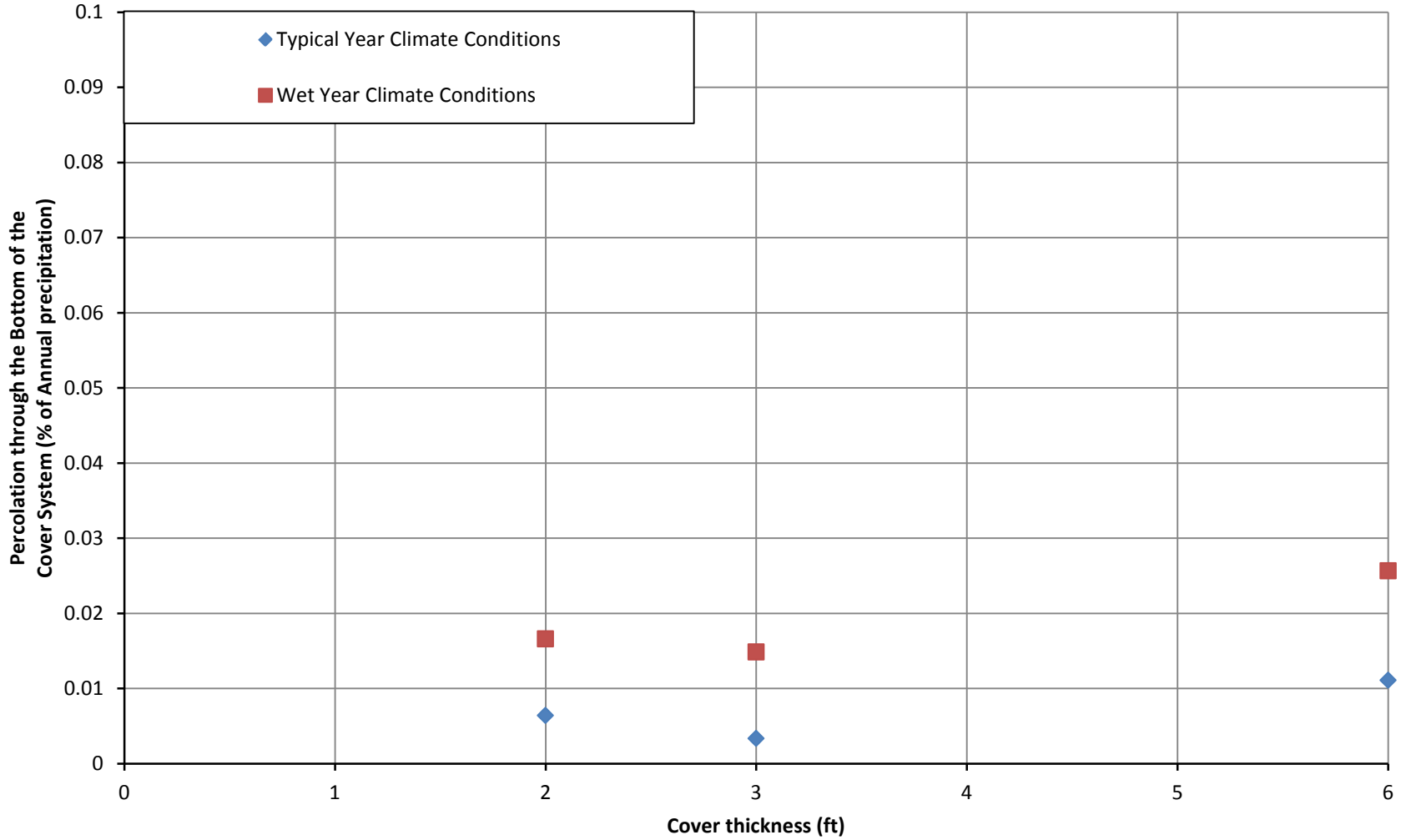
TITLE
Estimated Hydraulic Conductivity Curve for Rhoads Property Borrow

MWH

DATE
July 2013

FIGURE 10

FILENAME
Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

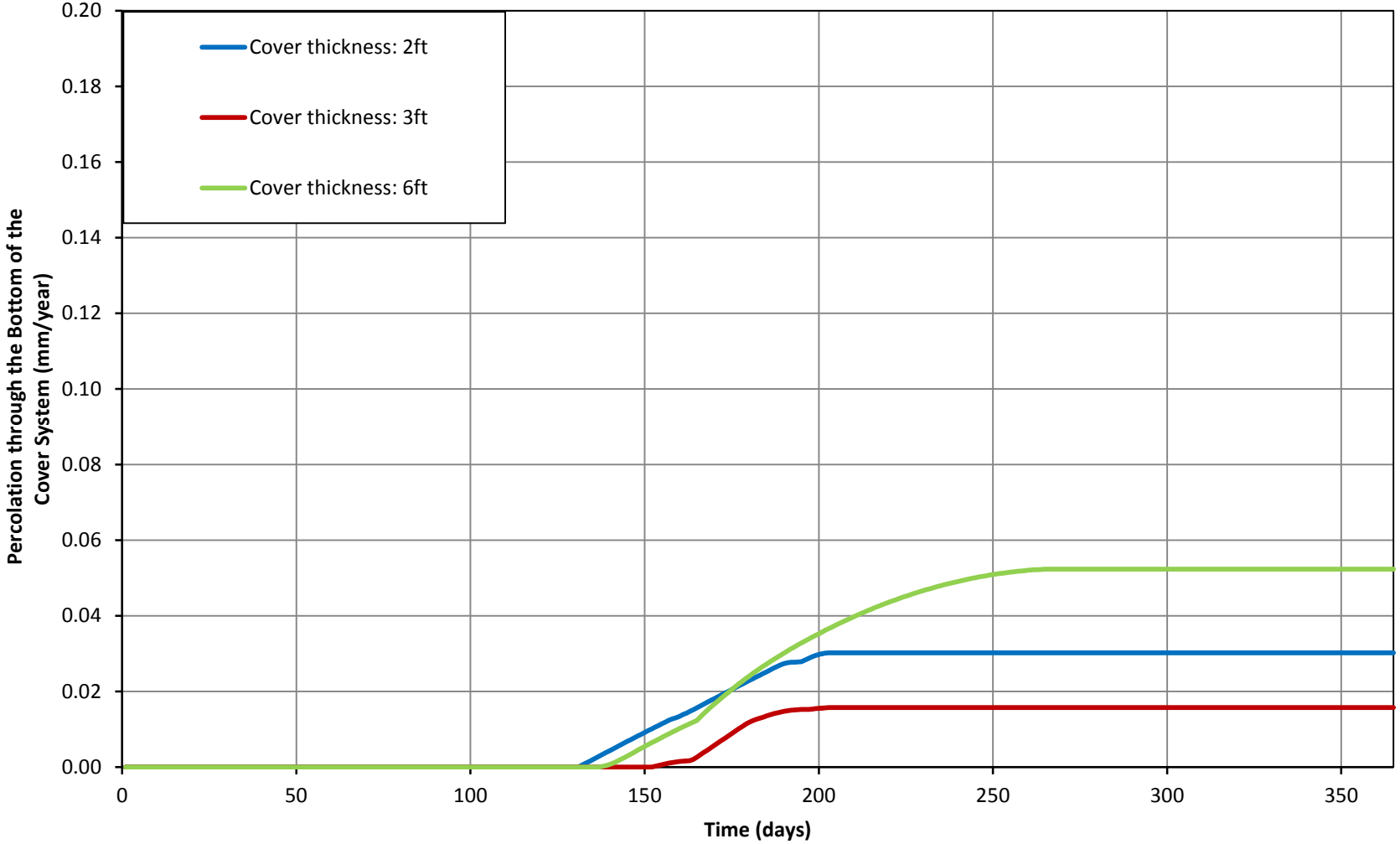
TITLE
As Constructed Conditions
Percolation as a Percent of Annual Precipitation
Rhoads Property Borrow

MWH

DATE
July 2013

FIGURE 11

FILENAME
Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

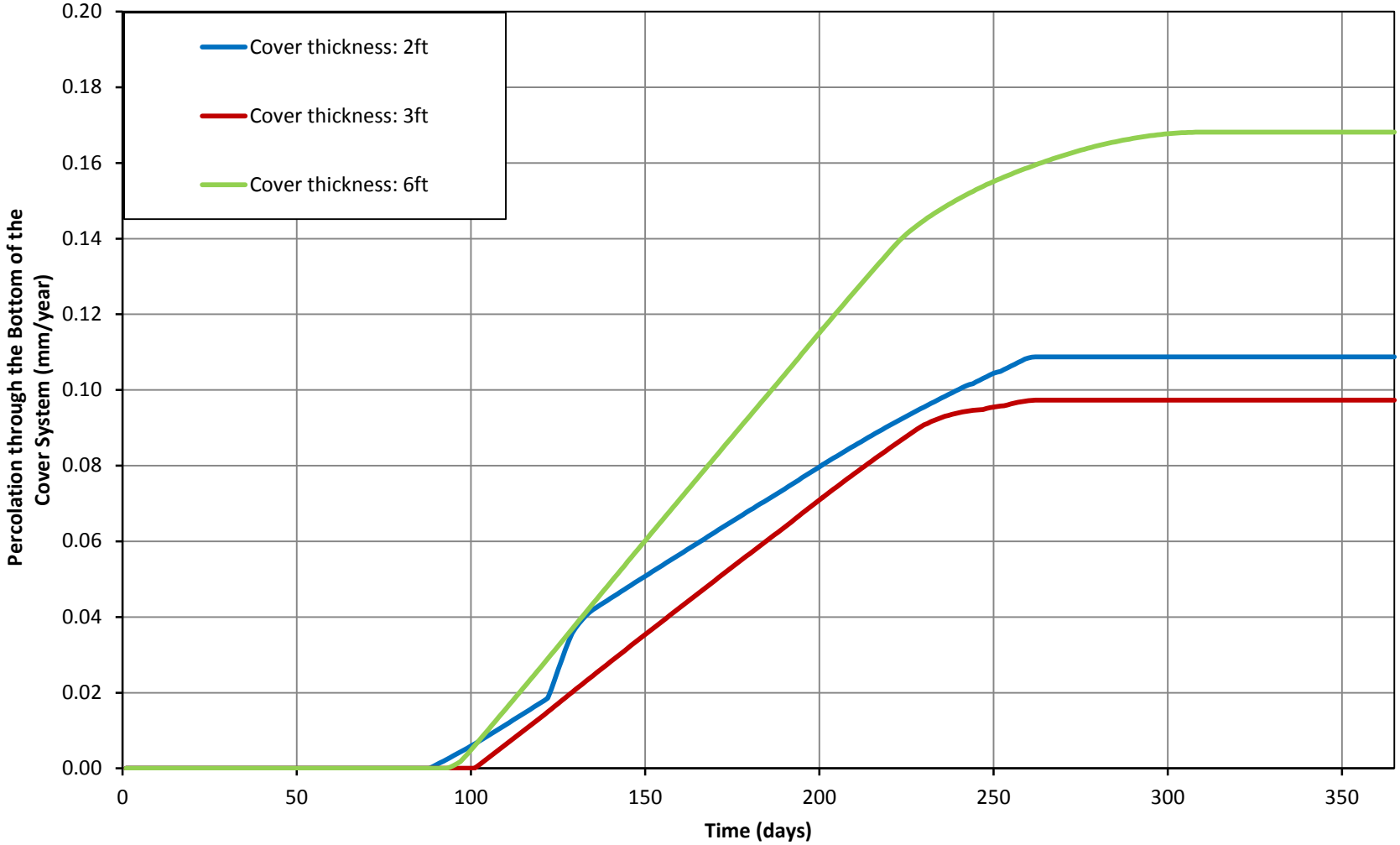
TITLE
As Constructed Conditions
Cumulative Percolation - Typical Year Conditions
Rhoads Property Borrow

MWH

DATE
July 2013

FIGURE 12

FILENAME
Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

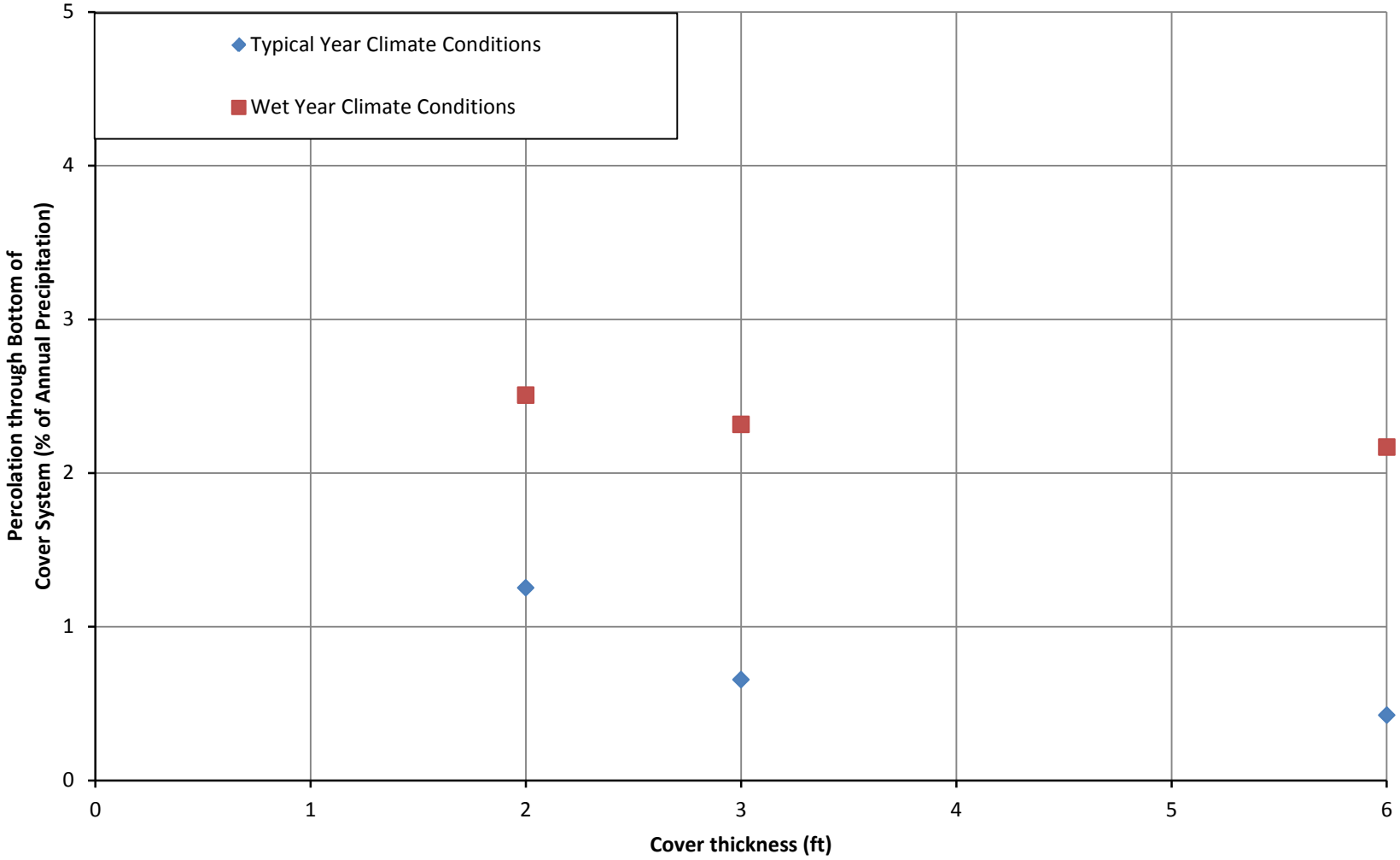
TITLE
As Constructed Conditions
Cumulative Percolation – Wet Year Conditions
Rhoads Property Borrow

MWH

DATE
July 2013

FIGURE 13

FILENAME
Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

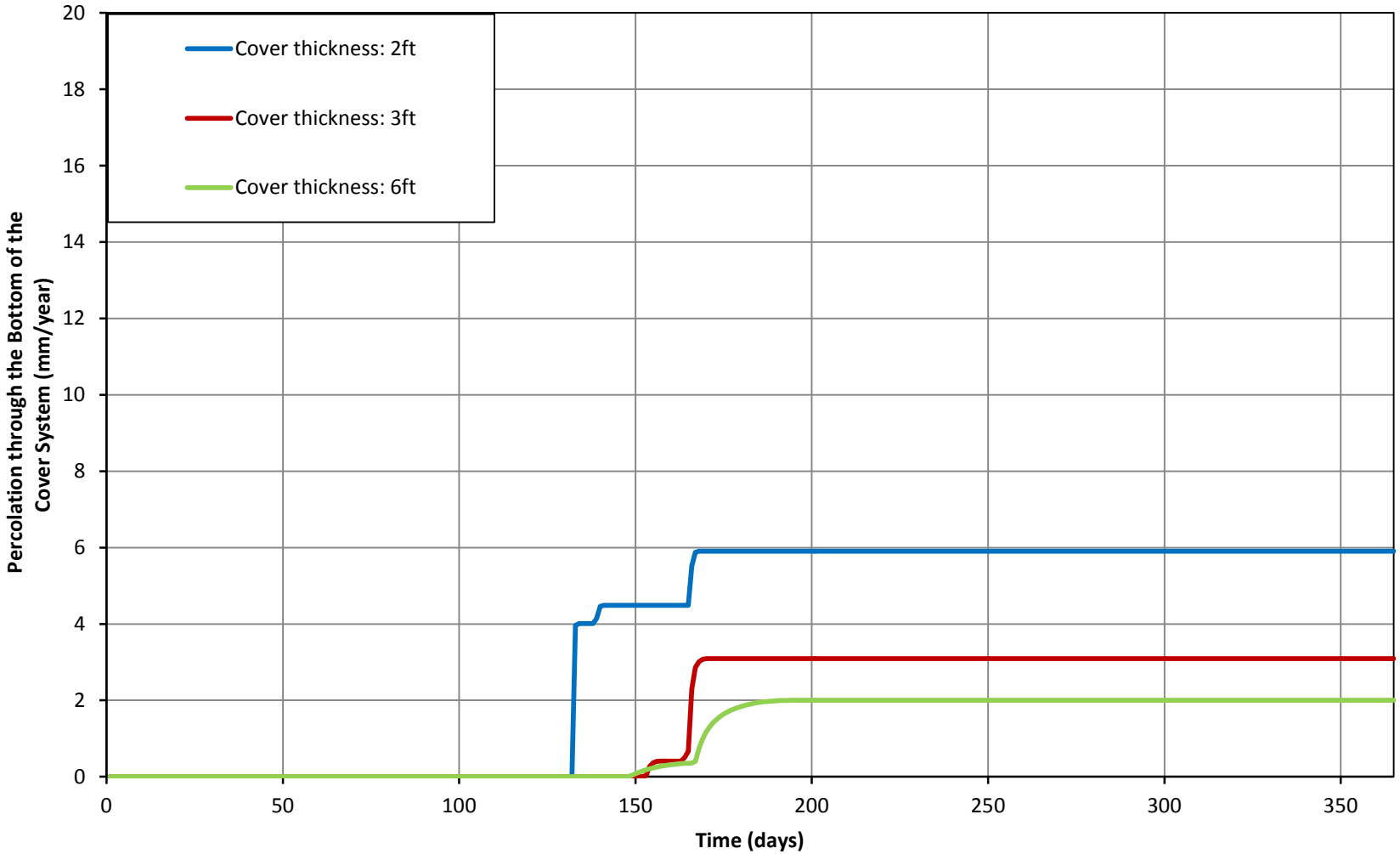
TITLE
Long-Term Degraded Conditions (no GM)
Percolation as a Percent of Annual Precipitation
Rhoads Property Borrow

MWH

DATE
July 2013

FIGURE 14

FILENAME
Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

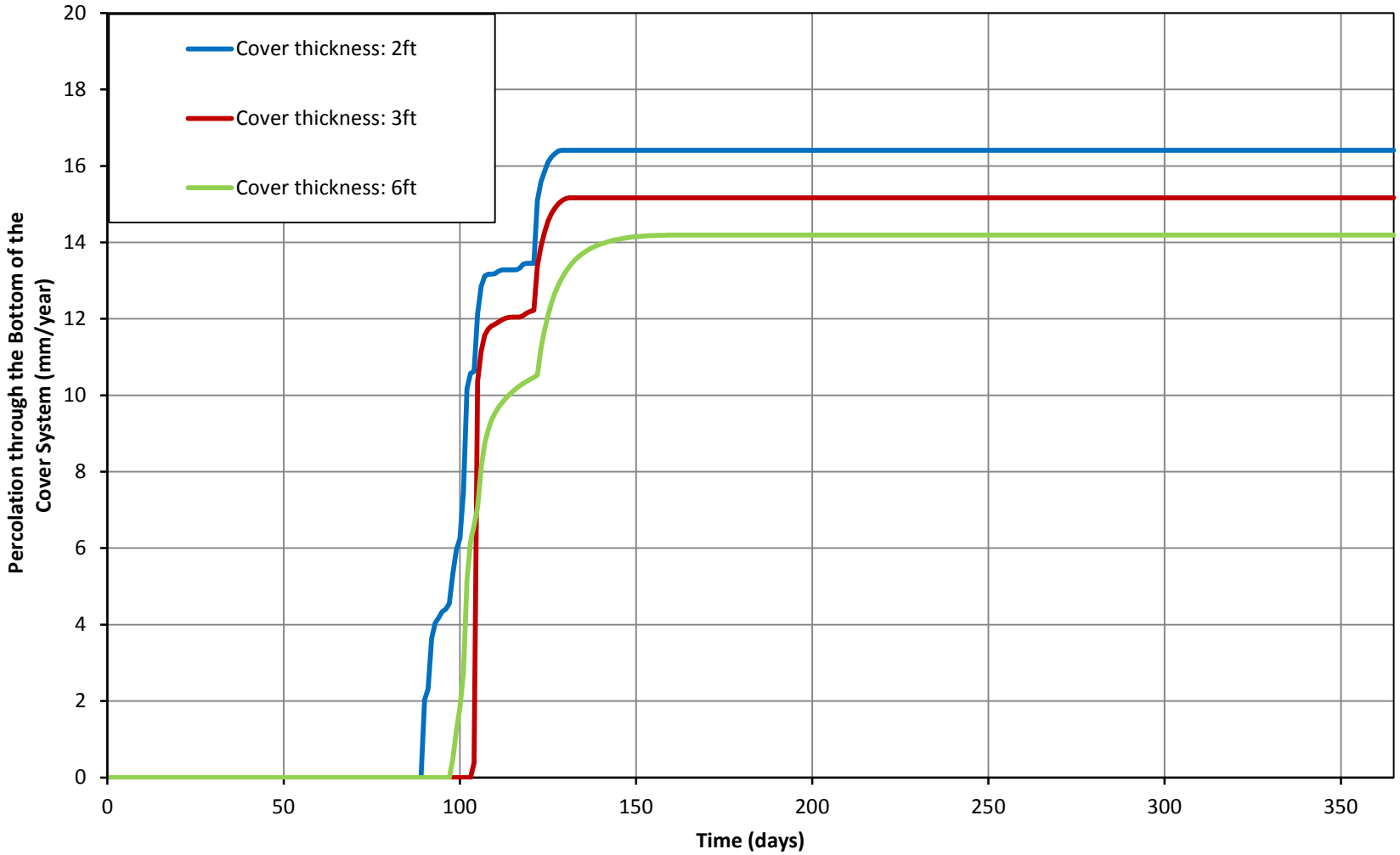
TITLE
Long-Term Degraded Conditions (no GM)
Cumulative Percolation - Typical Year Conditions
Rhoads Property Borrow

MWH

DATE
July 2013

FIGURE 15

FILENAME
Infil Calc Brief Figures 60%.pptx



NEWMONT

PROJECT
MIDNITE MINE

TITLE
Long-Term Degraded Conditions (no GM)
Cumulative Percolation – Wet Year Conditions
Rhoads Property Borrow

MWH

DATE
July 2013

FIGURE 16

FILENAME
Infil Calc Brief Figures 60%.pptx

Supplement D-4.1

Cover Design Infiltration Analyses Calculations

MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Leakage rate through cover geomembrane - Case A	Job No. : 1011322

Case A - As constructed condition:

Case A1: Mildly sloped areas with no surface ponding, no GDL above the GM
Case A2: Flat Drainage bench areas where surface ponding may occur, no GDL above the GM
Case A3: Sloped areas with no surface ponding, GDL overlies GM

								Average Year (a)			Wet Year (b)		
Case	Flow Condition	Cover Thickness (ft)	Ponding Condition	Cover Soil Permeability (m/s)	Underlying Bedding Material Permeability (m/s)	Permeability used in equation (m/s)	Equation #	Maximum head of water (m)	Average head of water (m)	Annual Leakage rate (mm/yr)	Maximum head of water (m)	Average head of water (m)	Annual Leakage rate (mm/yr)
A1	No Flow (with GM only)	2	No Ponding	5.9E-06	-	5.9E-06	4	0.000	0.000	0.000	0.610	0.012	0.063
		3	No Ponding	5.9E-06	-	5.9E-06	4	0.000	0.000	0.000	0.000	0.000	0.000
		6	No Ponding	5.9E-06	-	5.9E-06	4	0.000	0.000	0.000	0.000	0.000	0.000
A2	No Flow (with GM only)	2	Ponding	5.9E-06	-	5.9E-06	4	0.671	0.110	0.530	0.906	0.362	1.636
		3	Ponding	5.9E-06	-	5.9E-06	4	0.940	0.057	0.276	1.172	0.403	1.707
		6	Ponding	5.9E-06	-	5.9E-06	4	1.543	0.195	0.919	2.036	0.783	2.949
A3	Free Flow (with GDL)	2	No Ponding	5.9E-06	3.2E-05	3.2E-05	Geometric mean of 2 and 4	0.000	0.000	0.000	0.002	3.84E-05	0.005
		3	No Ponding	5.9E-06	3.2E-05	3.2E-05	Geometric mean of 2 and 4	0.000	0.000	0.000	0.000	0.00E+00	0.000
		6	No Ponding	5.9E-06	3.2E-05	3.2E-05	Geometric mean of 2 and 4	0.000	0.000	0.000	0.000	0.00E+00	0.000

Case B - Degraded condition:

Case B1: Mildly sloped areas with no surface ponding, without GM
Case B2: Flat Drainage bench areas where surface ponding may occur, without GM

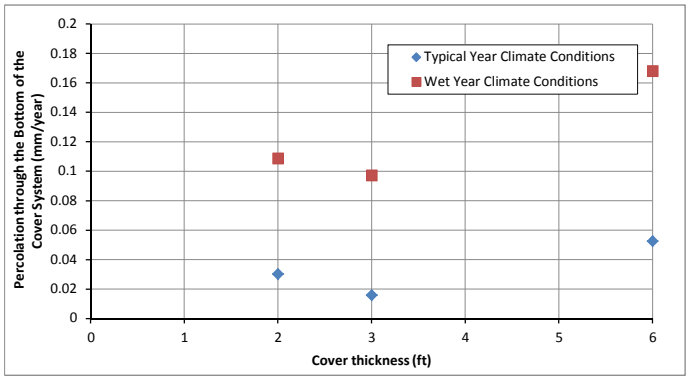
Case	Flow Condition	Cover Thickness (ft)	Ponding Condition	Cover Soil Permeability (m/s)	Average Year	Wet Year
B1	Free Flow	2	No Ponding	5.9E-06	0	19
	Free Flow	3		5.9E-06	0	0
	Free Flow	6		5.9E-06	0	0
B2	Free Flow	2	Ponding	5.9E-06	104	288
	Free Flow	3		5.9E-06	54	266
	Free Flow	6		5.9E-06	35	249

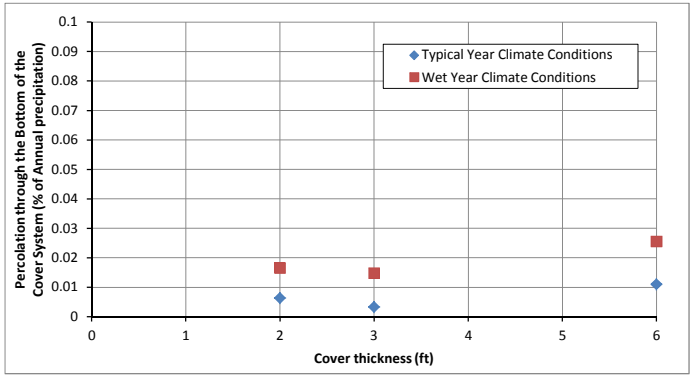
MWH	By : Jean-Baptiste Varnier	Date : 11/20/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Leakage rate through cover geomembrane - Case A	Job No. : 1011322

Case A - As constructed condition:

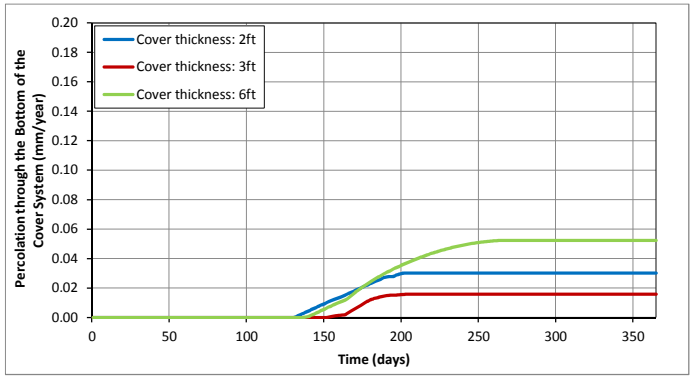
Area 1 - Mildly sloped areas with no surface ponding, no GDL above the geomembrane = 18.2%
Area 2 - Flat Drainage bench areas where surface ponding may occur, no GDL exists = 5.7%
Area 3 - Sloped Areas where no surface ponding occurs, GDL overlies geomembrane = 76.1%

Cover thickness (ft)	Area 1 - No Ponding, No GDL				Area 2 - Ponding, No GDL				Area 3 - No Ponding, With GDL				Total Infiltration Rate (mm/year)		Total Infiltration Rate percentage of total precipitation (%)	
	Case #	Infiltration Rate (mm/year)		Area (%)	Case #	Infiltration Rate (mm/year)		Area (%)	Case #	Infiltration Rate (mm/year)		Area (%)				
		Average year	Wet year			Average year	Wet year			Average year	Wet year		Average Year	Wet Year	Average Year	Wet Year
2	A1	0.000	0.063	18.2	A2	0.530	1.636	5.7	A3	0.000	0.005	76.1	0.030	0.109	0.006	0.017
3		0.000	0.000	18.2		0.276	1.707	5.7		0.000	0.000	76.1	0.016	0.097	0.003	0.015
6		0.000	0.000	18.2		0.919	2.949	5.7		0.000	0.000	76.1	0.052	0.168	0.011	0.026

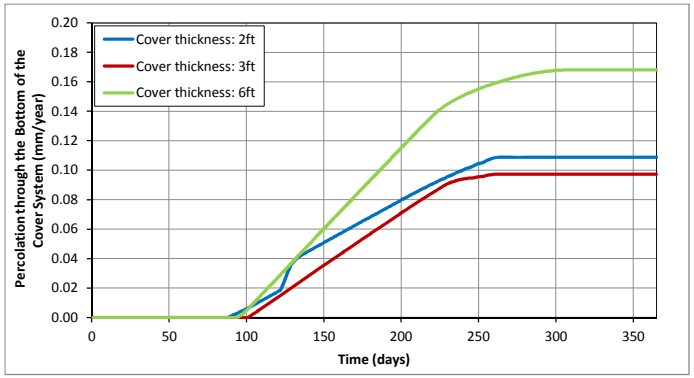




*** AVERAGE YEAR**



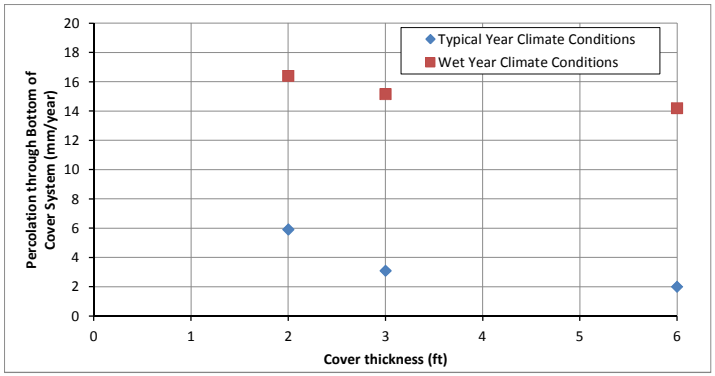
*** WET YEAR**

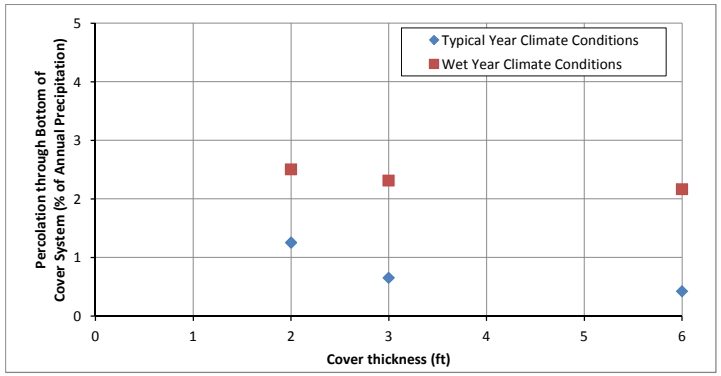


Case B - Degraded condition - No Geomembrane:

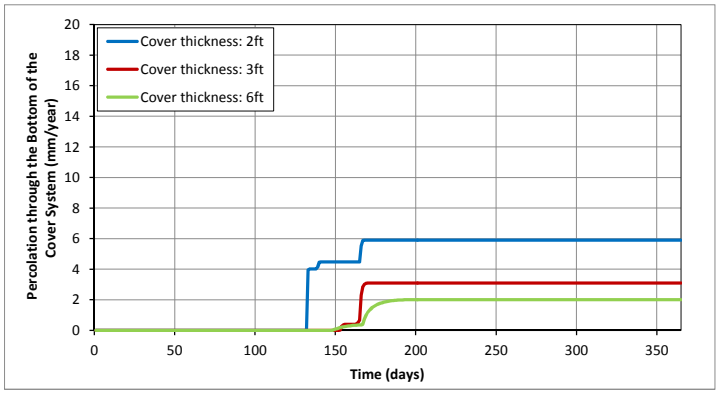
Areas with no ponding, no GDL over degraded geomembrane = 94.3%
Areas with Surface Ponding, no GDL over degraded geomembrane = 5.7%

Cover thickness (ft)	Area 1 - No Ponding, No GDL				Area 2 - With Ponding, No GDL				Total Infiltration Rate (mm/year)		Total Infiltration Rate percentage of total rain (%)	
	Case #	Infiltration Rate (mm/year)		Area (%)	Case #	Infiltration Rate (mm/year)		Area (%)				
		Average year	Wet year			Average year	Wet year		Average Year	Wet Year	Average Year	Wet Year
2	B1	0	0	94.3	B2	104	288	5.7	5.90	16.41	1.25	2.51
3		0	0	94.3		54	266	5.7	3.09	15.17	0.66	2.32
6		0	0	94.3		35	249	5.7	2.00	14.19	0.42	2.17

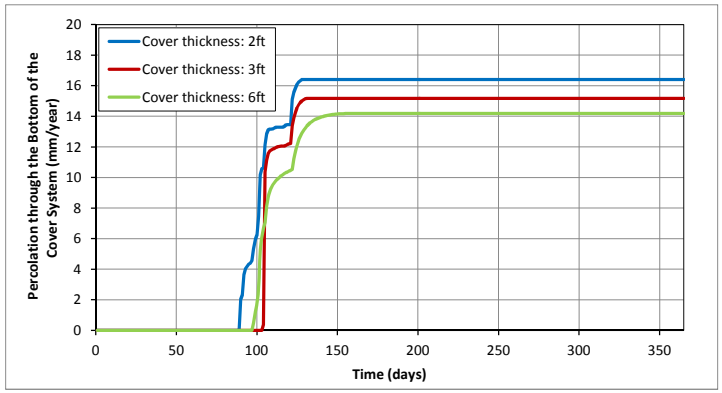




*** AVERAGE YEAR**



*** WET YEAR**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A	Job No. : 1011322

Case A1, 2 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

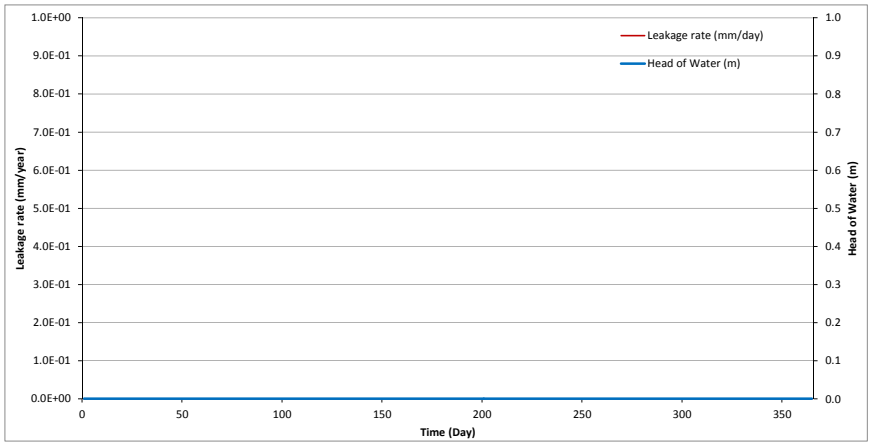
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A1	Job No. : 1011322

Case A1, 2 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

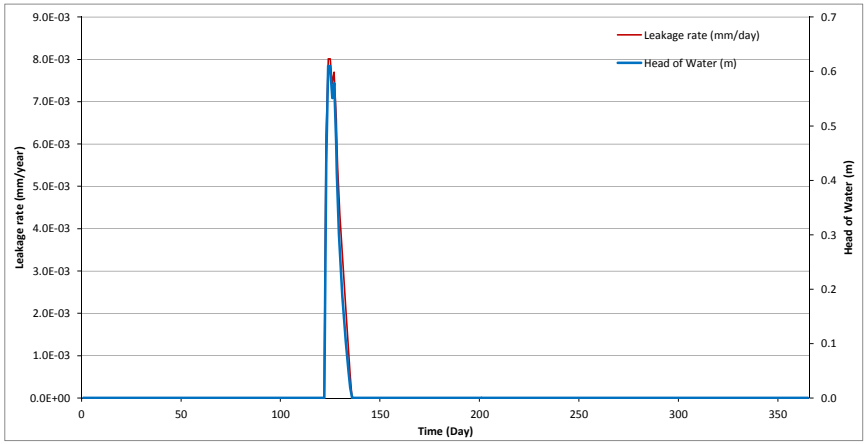
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{\ 0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	6.100E-01
Average head of water (m)	1.194E-02
Maximum daily leakage rate (mm/day)	8.013E-03
Annual Leakage rate (mm/year)	**6.330E-02**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A2	Job No. : 1011322

Case A2, 2 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

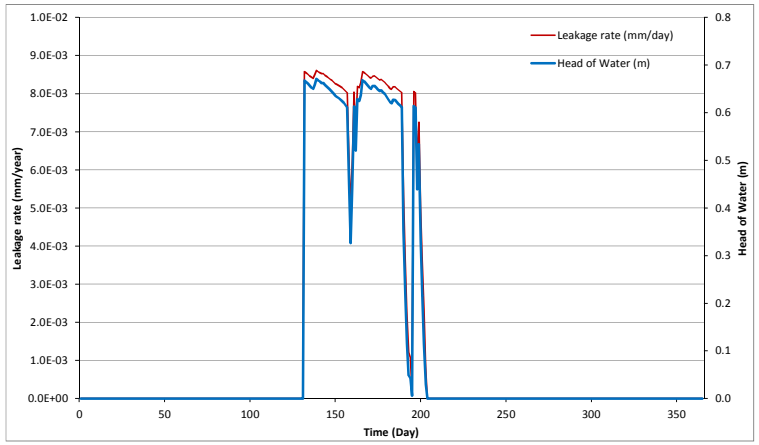
$$Q = 3 \times a^{0.75} \times h^{0.75} \times {k_d}^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	6.712E-01
Average head of water (m)	1.099E-01
Maximum daily leakage rate (mm/day)	8.609E-03
Annual Leakage rate (mm/year)	**5.301E-01**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A2	Job No. : 1011322

Case A2, 2 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

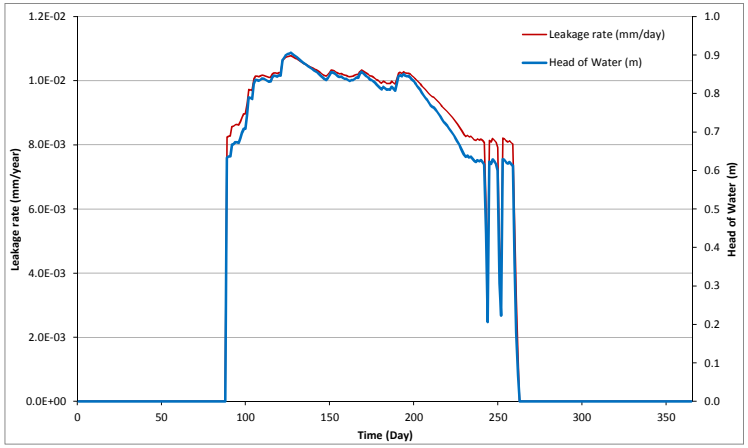
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{\ 0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	9.062E-01
Average head of water (m)	3.618E-01
Maximum daily leakage rate (mm/day)	1.078E-02
Annual Leakage rate (mm/year)	**1.636E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A1	Job No. : 1011322

Case A1, 3 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

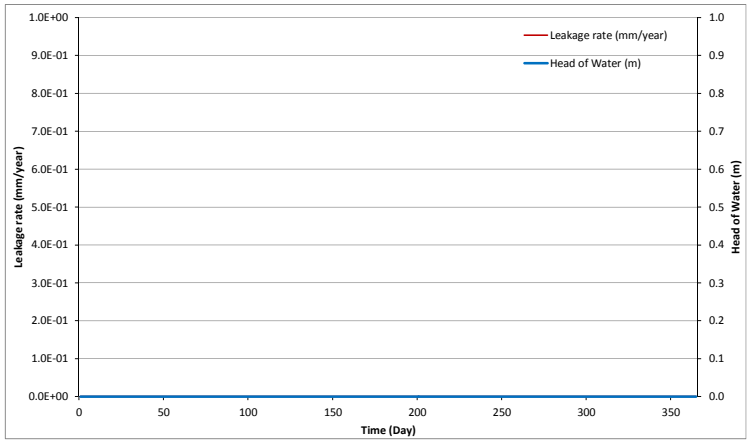
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A1	Job No. : 1011322

Case A1, 3 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

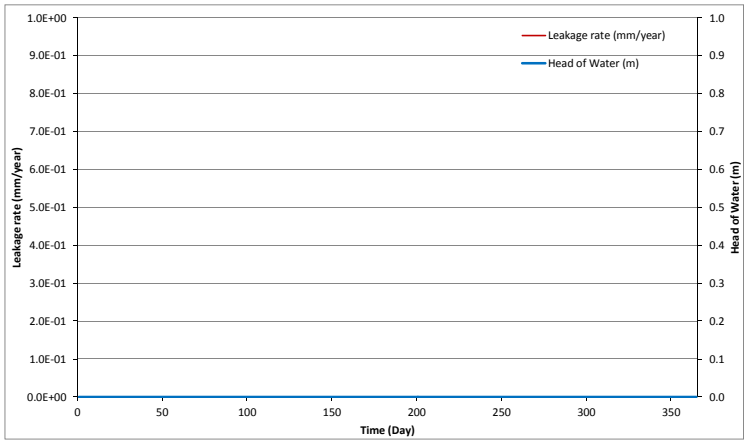
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A2	Job No. : 1011322

Case A2, 3 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

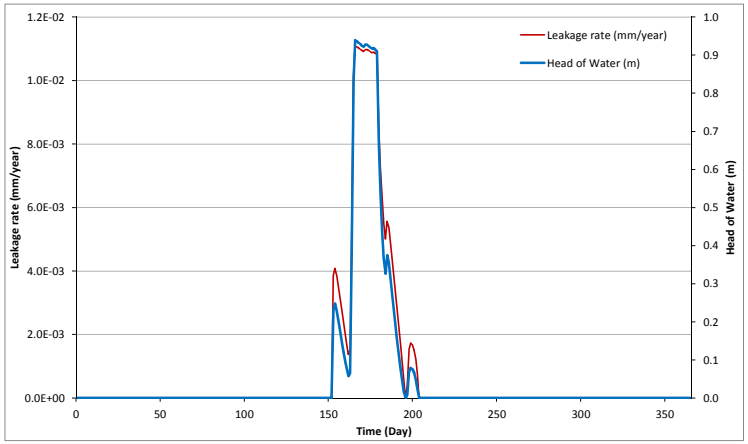
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{\ 0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	9.397E-01
Average head of water (m)	5.657E-02
Maximum daily leakage rate (mm/day)	1.108E-02
Annual Leakage rate (mm/year)	**2.762E-01**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A2	Job No. : 1011322

Case A2, 3 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

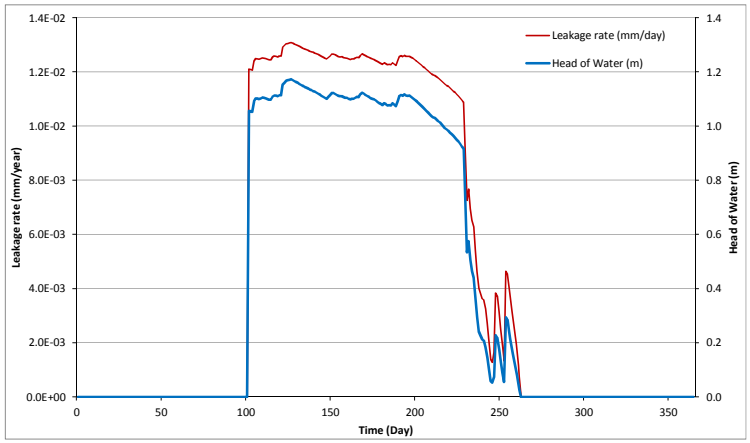
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	1.172E+00
Average head of water (m)	4.031E-01
Maximum daily leakage rate (mm/day)	1.308E-02
Annual Leakage rate (mm/year)	**1.707E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A1	Job No. : 1011322

Case A1, 6 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

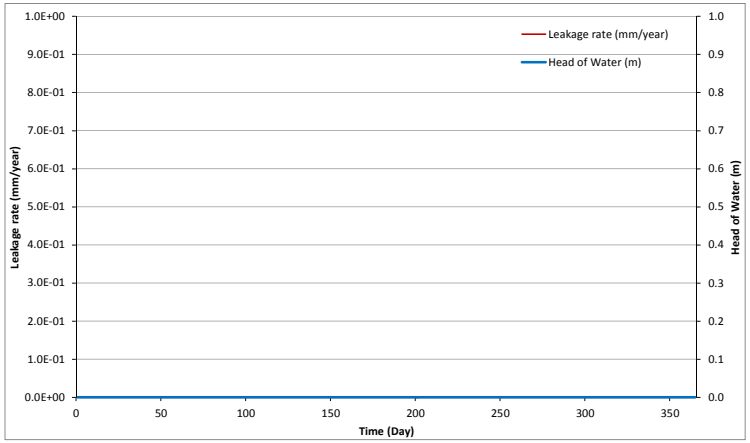
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{\ 0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A1	Job No. : 1011322

Case A1, 6 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

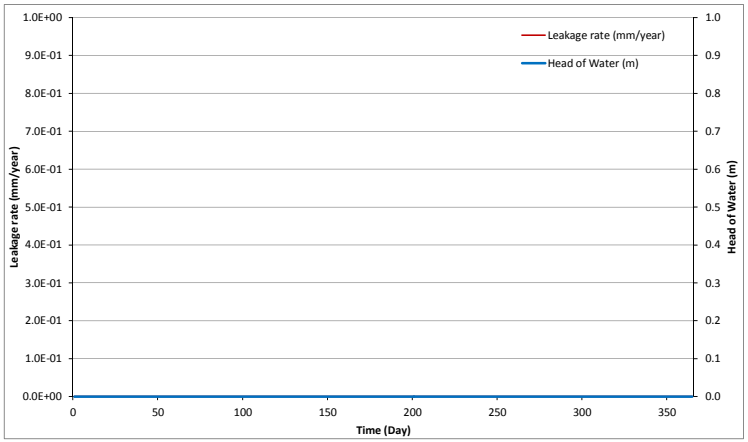
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A2	Job No. : 1011322

Case A2, 6 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

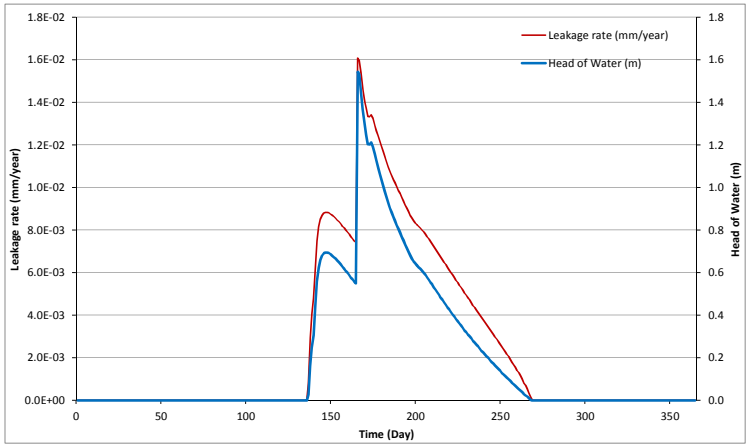
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{\ 0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	1.543E+00
Average head of water (m)	1.950E-01
Maximum daily leakage rate (mm/day)	1.607E-02
Annual Leakage rate (mm/year)	**9.189E-01**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A2	Job No. : 1011322

Case A2, 6 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

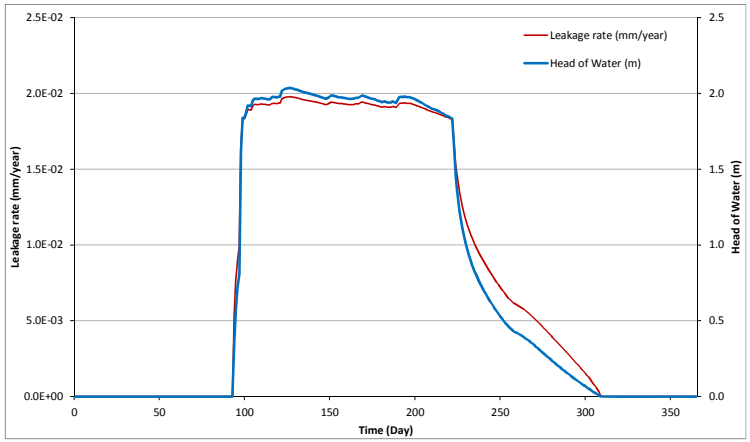
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_d^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of cover (Rhoads)	k_d	5.9E-06	m/s	

Summary table

Maximum head of water (m)	2.036E+00
Average head of water (m)	7.827E-01
Maximum daily leakage rate (mm/day)	1.979E-02
Annual Leakage rate (mm/year)	**2.949E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A3	Job No. : 1011322

Case A3, 2 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

$$Q = 0.21 \times a^{0.1} \times h^{0.9} \times {k_s}^{0.74}$$ (Bonaparte et al., 1989, Equation 2)

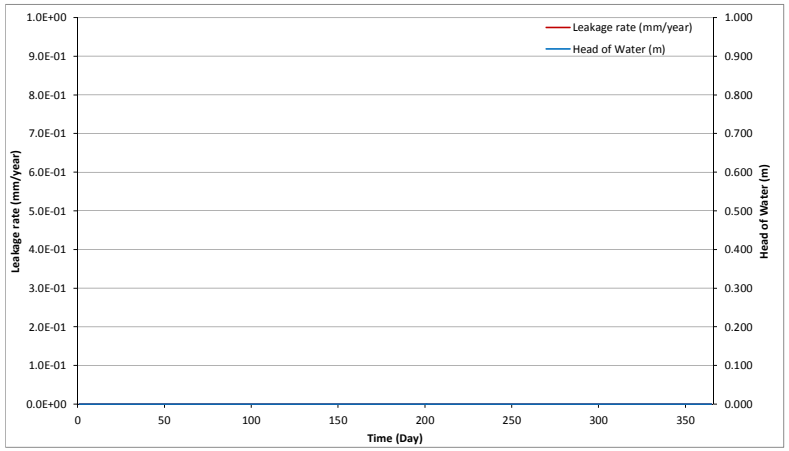
$$Q = 3 \times a^{0.75} \times h^{0.75} \times {k_s}^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of waste rock	k_s	3.2E-05	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A3	Job No. : 1011322

Case A3, 2 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

$$Q = 0.21 \times a^{0.1} \times h^{0.9} \times {k_s}^{0.74}$$ (Bonaparte et al., 1989, Equation 2)

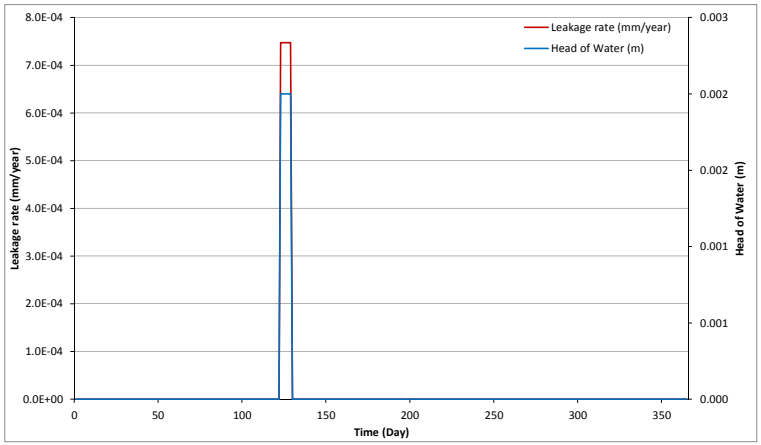
$$Q = 3 \times a^{0.75} \times h^{0.75} \times {k_s}^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of waste rock	k_s	3.2E-05	m/s	

Summary table

Maximum head of water (m)	2.000E-03
Average head of water (m)	3.836E-05
Maximum daily leakage rate (mm/day)	7.476E-04
Annual Leakage rate (mm/year)	**5.233E-03**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/20133	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A3	Job No. : 1011322

Case A3, 3 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

$$Q = 0.21 \times a^{0.1} \times h^{0.9} \times {k_s}^{0.74}$$ (Bonaparte et al., 1989, Equation 2)

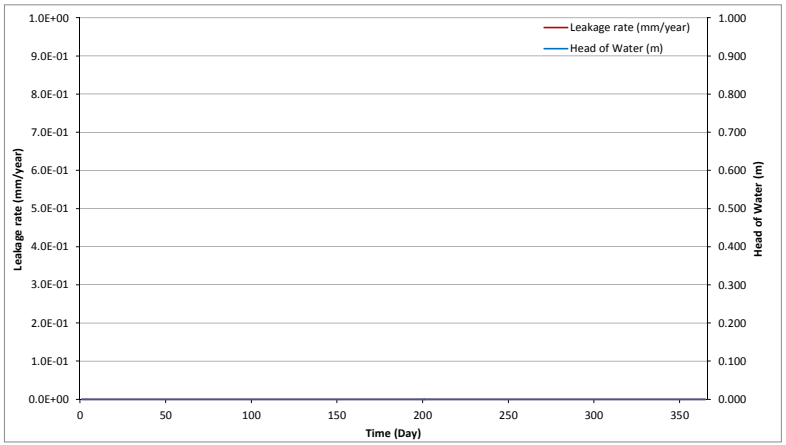
$$Q = 3 \times a^{0.75} \times h^{0.75} \times {k_s}^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of waste rock	k_s	3.2E-05	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A3	Job No. : 1011322

Case A3, 3 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

$$Q = 0.21 \times a^{0.1} \times h^{0.9} \times {k_s}^{0.74}$$ (Bonaparte et al., 1989, Equation 2)

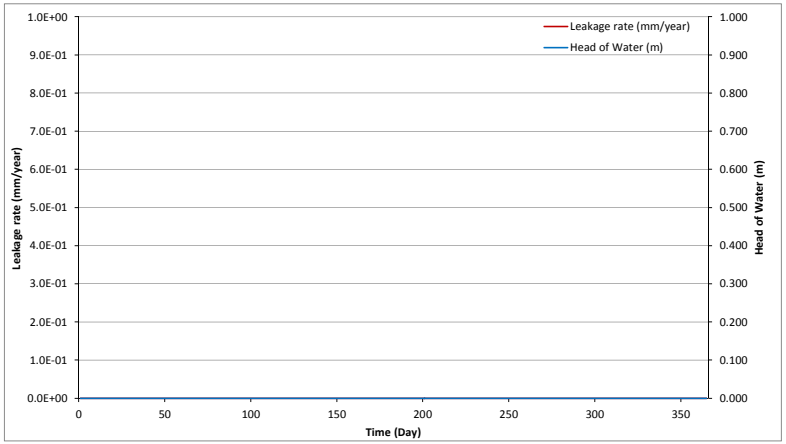
$$Q = 3 \times a^{0.75} \times h^{0.75} \times {k_s}^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of waste rock	k_s	3.2E-05	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Mine
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A3	Job No. : 1011322

Case A3, 6 ft cover, average year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

$$Q = 0.21 \times a^{0.1} \times h^{0.9} \times k_s^{0.74}$$ (Bonaparte et al., 1989, Equation 2)

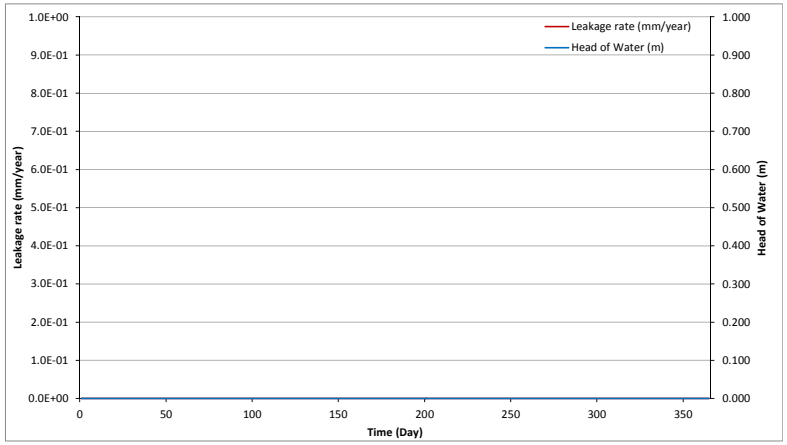
$$Q = 3 \times a^{0.75} \times h^{0.75} \times k_s^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of waste rock	k_s	3.2E-05	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



MWH	By : Jean-Baptiste Varnier	Date : 07/10/2013	Client : Midnite Min
	Checked by : Melanie Davis	Description : Daily leakage rate through cover geomembrane - Case A3	Job No. : 1011322

Case A3, 6 ft cover, wet year climate conditions

References:

(1) Bonaparte, R., Giroud, J.P., and Gross, B.A., 1989. Rates of Leakage Through Landfill Liners, Geosynthetics 1989 conference, San Diego, pp. 18-29.

$$Q = 0.21 \times a^{0.1} \times h^{0.9} \times {k_s}^{0.74}$$ (Bonaparte et al., 1989, Equation 2)

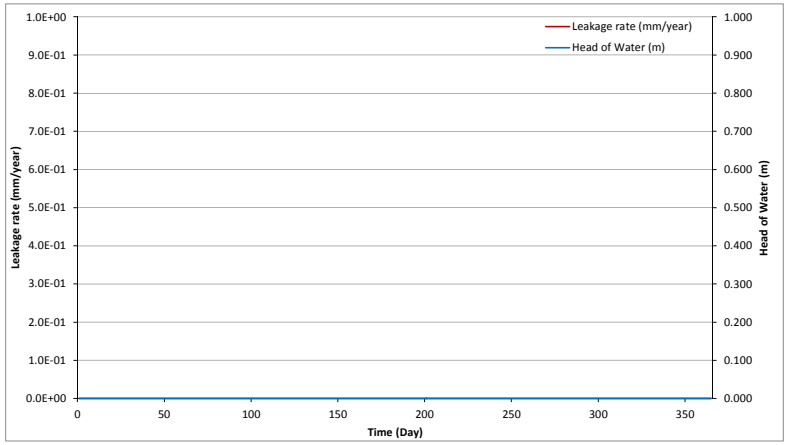
$$Q = 3 \times a^{0.75} \times h^{0.75} \times {k_s}^{0.5}$$ (Bonaparte et al., 1989, Equation 4)

where Q is the leakage rate through defect (m^3/s), a is the defect area (m^2), h is the head of water (m), k_d is the hydraulic conductivity of the material overlying the geomembrane (m/s).

Data Inputs	Variable	Value	Units	Source
Defect diameter	D_d	0.079	in	Typical value of 2 mm, Giroud et al. (1997) and EPA (1992)
		0.0020	m	
Rate of defects	r	1	defect/acre	Typical value EPA (1992)
Hydraulic permeablity of waste rock	k_s	3.2E-05	m/s	

Summary table

Maximum head of water (m)	0.000E+00
Average head of water (m)	0.000E+00
Maximum daily leakage rate (mm/day)	0.000E+00
Annual Leakage rate (mm/year)	**0.000E+00**



Attachment D-5

Global Stability Analysis

ATTACHMENT D-5

MIDNITE MINE REMEDIAL ACTION GLOBAL STABILITY ANALYSIS

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*21-Nov-12*	*30% Design*	*Jean-Baptiste Varnier and Melanie Davis*	*Tom Kelley*	*21-Nov-12*	*Tom Kelley*
1	*2-Dec-13*	*60% Design*	*Jean-Baptiste Varnier*	*Melanie Davis*	*5-Dec-13*	*Tom Kelley*
2	*30-Jun-14*	*90% Design*	*Jean-Baptiste Varnier and Melanie Davis*	*Melanie Davis*	*18-Jul-2014*	*Tom Kelley*
2	*14-May-15*	*100% Design – no changes*				

Location and Format
Electronic copies of these calculations are located in the project files system at: *\\usftc2s01\Projects\Newmont\Midnite Mine_2011\6.0 Studies & Reports\6.2 Technical\Remedial Action Cover Design\Global Stability\90% Design* *The following calculations were generated using the following software:* *Geo-Slope International, Ltd, 2012 (Slope/W)*

1.0 BACKGROUND AND PURPOSE

This calculation brief presents the methods, input parameters, and results of the global slope stability analyses that were conducted for Pit 3 and Pit 4 after placement of mine waste within the pits and construction of the cover for the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The analyses summarized in this brief focus on the global slope stability of potential failure surfaces located at moderate to large depths in the mine waste and foundation layers lying beneath certain areas of the backfilled areas in and adjacent to the pits. These analyses were conducted according to applicable stability performance standards as required by Appendix B to the Remedial Design/Remedial Action (RD/RA) Consent Decree (CD) (EPA, 2011) under static and seismic loading conditions. In addition to the analyses specified in the CD, which consisted of analysis of static and pseudo-static earthquake conditions, analyses also were performed for post-earthquake strength conditions in areas underlying the cover (i.e., foundation areas) that may contain saturated, potentially-liquefiable colluvium. These analyses have been updated from the 60 percent Basis of Design Report (BODR) (MWH, 2013) to incorporate minor grading changes and refine the slip surface entry and exit ranges.

For the purposes of these global stability analyses, the soil cover was included, primarily to provide for a more complete accounting of weight forces, but localized, shallow failure surfaces within the cover layers were not considered for this calculation brief. The stability of the surficial cover system and potential for slope failures along cover interface elements are summarized in a separate calculation brief (i.e. Attachment D-7). The proposed cover system will consist of a uniform soil layer overlying a geomembrane. On steeper sloped areas of mine waste (greater than 15 percent), the cover system also will include a geocomposite drainage layer (GDL) between the soil and geomembrane layers to reduce potential water build up in the slope and increase slope stability.

1.1 DESCRIPTION OF SLOPE STABILITY ANALYSES

1.1.1 General

Geotechnical slope stability analyses were performed to evaluate the stability of the reclaimed slopes of Pits 3 and 4. Cross-Sections 2 and 3 for Pit 3 and Cross Section 1 for Pit 4 (see Figure 1) were selected as the most critical cross-sections based on topography, slope height, and foundation conditions. The analyses were conducted for conditions at closure with the remediation cover in place. As discussed above, the stability of the surficial cover system was not considered in this global stability evaluation, as it is part of the Veneer Cover Stability Analysis calculation brief (Attachment D-7). The slope failure surfaces were limited to slip surfaces with maximum depths of at least 25 feet (i.e. shallow, near surface failures were not included).

Analyses were conducted for static and pseudo-static loading conditions. Post-earthquake conditions were also evaluated in areas where the potential of encountering saturated colluvium in the foundation exists. Analyses presented in MWH (2012a) indicate that some of the colluvium could potentially suffer strength reductions as a result of dynamic loading. Post-earthquake stability analyses were performed assuming cyclical softening will occur in the

saturated colluvium. The model profiles used for the slope stability analysis are shown in Figures 2 through 21 and are based on: (1) the pre-mining surface contours, (2) the subgrade surface contours, (3) the existing surface contours, and (4) the design contours for the reclamation cover.

Although Cross Section 1 and Cross Section 3 contain bends in the section in plan view, as shown on Figure 1, in order to follow the most critical downs-slope direction and/or subsurface features (e.g. colluvium in the valley bottom), these sections were analyzed as planar, straight line sections. This is conservative as three-dimensional effects of the change in section direction that will result in an increase in resistance to movement are being neglected.

1.1.2 Selected Model and Conditions

Slope stability analyses were performed using two-dimensional limit equilibrium methods with the aid of the program SLOPE/W (GEO-SLOPE, 2012). The Morgenstern-Price method (Morgenstern and Price, 1965) with a half-sine function for inter-slice forces was used for the analysis. This method can be used to evaluate both circular and non-circular slip surfaces, and satisfies both force and moment equilibrium in the factor of safety calculation (GEO-SLOPE, 2012). Analyses were performed by calculating the factors of safety along circular failure surfaces. In cases where the assumed circular failure surfaces penetrated the underlying bedrock, the failure surfaces were truncated at the bedrock surface and sliding was assumed to occur along the interface in the weaker overlying material. Numerous trial failure surfaces were analyzed to find the lowest factor of safety. Long-term, steady-state pore water pressure conditions, represented by a phreatic surface at the base of waste rock, were used in the analyses. Effective-stress strength parameters were used to represent the mine waste.

2.0 MODEL MATERIAL TYPES

The material types for the cross-sections used in the slope stability analyses include: (1) cover soil from the Rhoads Property Borrow Area; (2) mine waste; (3) colluvium; and (4) bedrock. The parameters for each material are discussed below and summarized in Table 1.

2.1 COVER SOIL

Material from the Rhoads Property Borrow Area was considered for the cover material for the global stability analyses. A soil cover thickness of 3.0 feet has been included in the stability sections. The borrow material consists of: (1) clayey sand; and (2) weathered quartz monzonite that has weathered to a residual soil (residuum) (MGC, 2011a, b).

The density value of the cover material used in the analyses is based on laboratory testing results for the Rhoads Property Borrow Area (MGC, 2011a, b). The placed density value was estimated based on the average laboratory measured maximum standard Proctor dry density (rock correction applied where applicable) and average measured in-situ moisture content for samples from the borrow area, and assuming the cover material will be placed at approximately 85 percent of standard Proctor compaction.

Material strength parameters for the Rhoads Property Borrow cover soil were based upon the results of laboratory testing provided in MGC (2011a). Fully drained conditions were assumed for the cover soil due to the inclusion of the GDL in the cover system. Effective strength parameters used in the analyses correspond to the average values of the laboratory tests.

2.2 MINE WASTE

The density and effective strength parameters of the mine waste used in the analyses were based on subsurface information provided in URS (2001) and material parameters used for stability analyses presented in URS (2002). Effective strength parameters were used to represent the mine strengths due to the relatively coarse, free-draining nature of the mine waste.

2.3 COLLUVIUM

The density of the colluvium was calculated from the measured in-place dry density and the natural water content for a sample representative of the colluvium in the Central Drainage south of Pit 3, sample CD-B4 at 6 to 7.5 feet, presented MWH (2012b).

Under static conditions the colluvium has no excess pore pressure and was therefore modeled using effective shear strength parameters. The effective friction angle used for the colluvium was estimated based on the average measured plasticity index (PI) of colluvium samples located above weathered bedrock or bedrock from MWH (2012b) and using the relationship between PI and the effective angle of internal friction angle presented in Duncan and Wright (2005). The effective cohesion value was assumed equal to 0. This estimated effective friction angle is very similar to that obtained from triaxial testing performed as part of the storage pond investigation (MWH, 2012b) as discussed in the following paragraph.

A consolidated undrained triaxial test, with pore pressure measurements was performed on specimens of low-strength colluvium (alluvial clay) from Test Pit CDTP2 (7 to 14 foot depth), located adjacent to CD-B2. Although the triaxial tests were run on remolded specimens compacted to 98 percent of the standard Proctor maximum dry density at 1 percent above the optimum moisture content, the post-peak results at large strain (15 percent) for the specimen tested at the highest effective consolidation pressure (8,640 psf) can reasonably be expected to represent the normally-consolidated, constant-volume shear strength of the low-strength colluvium. Analysis of the result from this test specimen indicates a constant-volume effective stress friction angle slightly higher (32.4 degrees) than the estimated value of 30.8 degrees described in the preceding paragraph above. The lower estimated value of 30.8 degrees was used as a conservative estimate of effective stress friction angle.

For the analyses of earthquake (pseudo-static) loading conditions, the colluvium shear strength was modeled using undrained strengths to account for rapid loading conditions during the design seismic event. Undrained shear strengths recommended in Attachment E-3 were used in the analysis for colluvium. The undrained shear strengths are based upon the interpretations of recent piezocone (CPT) investigations along the toe of the South Waste Rock Pile (SWRP) in the alluvial clays. The undrained shear strengths were represented using c/p' ratios recommended in Attachment E-3 to Appendix E. The normally-consolidated c/p' ratio are used, in conjunction with the SHANSEP parameters recommended in Attachment E-3, to calculate the shear strengths at various waste rock burial depths in the model. The use of the normally

consolidated c/p' ratio and SHANSEP parameters from the CPT interpretations, which were run on alluvial clays in the valley bottom of the SWRP to represent all of the colluvium in the foundation, is considered very conservative.

Residual, post-earthquake strengths recommended in Supplement E-2.2 to Attachment E-2 in Appendix E were used in the analyses of post-earthquake conditions. In the post-earthquake analyses, the colluvium shear strength was modeled assuming (1) clay behavior, (2) sand behavior and (3) a conservative combination of sand-clay behavior. The first two cases (clay behavior and sand behavior) correspond to general categories of post-earthquake material behavior of the colluvium. The third case (sand-clay behavior) corresponds to a hybrid model where interbedded sand and clay layers exist, producing a worst case scenario where strength behavior producing the lowest strength at each overburden stress level was used in the analyses. The three behaviors used in the stability analyses are described in the following subsections.

2.3.1 Clay Behavior

For the clay behavior analysis under post-earthquake conditions, the colluvium shear strength was modeled assuming the undrained shear strength parameters described in Supplement E-2.2 of Attachment E-2 to Appendix E. The undrained shear strengths were calculated based upon the results of CPT soundings along the toe of the SWRP. Undrained strengths were characterized for four overburden stress levels: free field, 30 ft waste rock fill, 65 ft waste rock fill and 120 ft waste rock fill. The free field case indicates that no overburden pressure is applied to the colluvium. Due to the overconsolidated state of the near-surface clays, the undrained shear strengths in the free-field conditions are modestly higher than will exist under long-term drained loading as represented by the effective stress friction angle.

2.3.2 Sand Behavior

Recommendations from Supplement E-2.2 of Attachment E-2 to Appendix E were also used in modelling the shear strength of the colluvium exhibiting sand behavior. For the stability analyses under sand behavior for post-earthquake conditions, colluvium with less than 15 feet of the waste rock overburden was assumed to liquefy, consistent with the results of the CPT investigation. The liquefied residual strength used in the analyses was based upon the liquefied strength in the liquefiable zones under free-field stress conditions. The strength of the un-liquefied colluvium assumed a Mohr-Coulomb (internal friction angle only) failure envelop. The angle of internal friction used to represent the un-liquefied colluvium (28 degrees) was selected as a conservative representation of internal friction angle calculated from CPT sounding results. It should be noted that the assumption of sand behavior results in lower post-earthquake strengths in areas of low stress (i.e. free field areas with less than 15 feet of overlying waste rock), and results higher post-earthquake shear strengths in more highly confined areas with more 15 feet of overlying waste rock relative to the clay behavior case.

2.3.3 Sand-Clay Behavior

The sand-clay behavior analysis under post-earthquake conditions is a hybrid model that assumes sand behavior (i.e. liquefied residual strengths) when less than 15 feet of the waste rock fill is located above the colluvium and clay behavior (i.e. residual clay strengths) when more than 15 feet of waste rock overlies the colluvium. This assumed behavior is considered to be very conservative, but could be representative of a condition where interbedded sand and

clay layers exist, and parallel the underlying weathered bedrock contact, thus allowing slippage along the weakest parallel layers within the colluvium at varying overburden stress levels.

2.4 BEDROCK

The bedrock was modeled as an impenetrable surface where the slip surfaces being analyzed were not allowed to cut into this stronger material. Instead, slip surfaces were truncated at the bedrock contact and sliding was assumed to occur along the interface in the weaker overlying material.

2.5 MATERIAL PROPERTY SUMMARY

The table below provides a summary of the material properties used in the global slope stability analyses.

Table 1. Material Properties Used in Stability Analyses under Static and Pseudo-Static Conditions

Material Type	Total Density (pcf)	Effective Friction Angle, ϕ' (deg)	Effective Cohesion, c' (psf)	Undrained Shear Strength ratio s_u/σ_v' [(b)]
Cover Soil (Rhoads Property Borrow Area)	110	35.2	0	-
Mine Waste	135	38.0	0	-
Colluvium	122	30.8[(a)]	0[(a)]	2.10 (Free Field) 0.56 (30' Waste Rock Fill) 0.46 (65' Waste Rock Fill) 0.41 (120' Waste Rock Fill)
Bedrock	Impenetrable			-

Notes: (a) Static conditions
(b) Pseudo-static conditions

Table 2. Material Properties Used in Stability Analysis under Post-Earthquake Conditions

Material Type	Total Density (pcf)	Effective Friction Angle, ϕ' (deg)	Effective Cohesion, c' (psf)	Undrained Shear Strength Ratio, su/σv'
Cover Soil (Rhoads Property Borrow Area)	110	35.2	0	-
Mine Waste	135	38.0	0	-
Colluvium, Clay behavior	122	-	-	1.55 (Free Field) 0.42 (30' Waste Rock Fill) 0.34 (65' Waste Rock Fill) 0.31 (120' Waste Rock Fill)
Colluvium, Sand behavior	122	- 28	- 0	0.08 (Free Field) -
Colluvium, Sand-Clay behavior	122	-	-	0.08 (Free Field) 0.42 (30' Waste Rock Fill) 0.34 (65' Waste Rock Fill) 0.31 (120' Waste Rock Fill)
Bedrock	Impenetrable			-

3.0 SEISMIC LOADING

The global slope stability analyses under seismic loading conditions were conducted using a pseudo-static approach, where a horizontal inertial force is applied to the numerical models. This horizontal inertial force simulates the effects of accelerations during a seismic event. The peak ground acceleration (PGA) for a probability of exceedance of 10 percent in 250 years at the Site for site Class B (rock) is 0.131g (MGC, 2010), where g is gravity. The PGA at the Site was adjusted to site Class C (very dense soil and soft rock) to represent waste rock for the slope stability analyses. The recommendations presented in USGS (2013) were used to calculate the adjusted PGA. The Class C PGA at the Site is 0.157g. The procedure of representing the seismic coefficient as a function of the PGA has been adopted in review of uranium facility design and documented by the U.S. Department of Energy (DOE, 1989). DOE (1989) recommends the use of a horizontal seismic coefficient of $^{2}/_{3g}$ of the PGA for pseudo-static stability analyses for post-reclamation conditions. The U.S. Nuclear Regulatory Commission (NRC, 2003) recommends the seismic coefficient be either 67 percent of the PGA or 0.1, whichever is greater. The horizontal seismic coefficient used for the pseudo-static stability analyses is 0.105.

4.0 RESULTS

The required minimum factors of safety are 1.3 under static conditions and 1.0 under pseudo-static conditions as outlined in EPA (2011). A minimum required factor of safety of 1.0 also was used for post-earthquake conditions. Results of the analyses are summarized in Table 3 and in Figures 2 to 21.

Table 3. Factors of Safety for Global Stability

Loading Conditions		Minimum Required FOS	Pit 4	Pit 3		
			Cross-Section 1	Cross-Section 2	Cross-Section 3 Top of Slope	Cross-Section 3 Toe of Slope
Static		1.3	3.3	2.9	3.0	2.3
Pseudo-Static		1.0	2.2	2.0	2.1	2.0
Post-Earthquake	Clay Behavior	1.0	2.9	NA	3.0	2.6
	Sand Behavior		3.3	NA	3.0	1.7
	Sand-Clay Behavior		2.9	NA	3.0	1.6

Due to the effects of overconsolidation of the near-surface clay colluvium, the calculated factors of safety under post-earthquake conditions with assumed clay behavior at Cross Section 3 are slightly higher than for static, long-term drained (effective stress). The factors of safety under static, pseudo-static and post-earthquake conditions for all cases exceed the minimum required factors of safety.

5.0 REFERENCES

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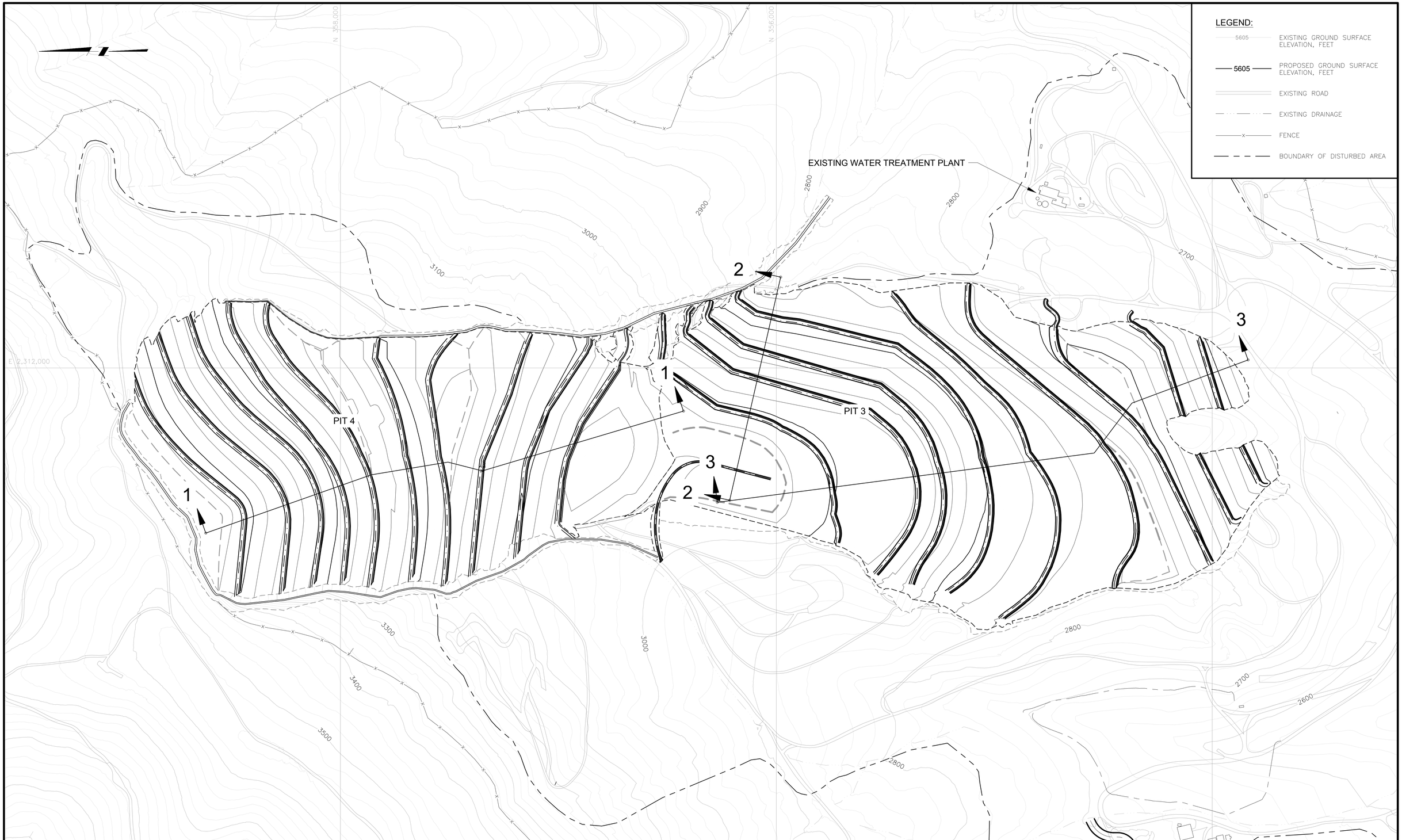
FIGURES



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NEWMONT

PROJECT LOCATION: WELLPINIT, WASHINGTON

PROJECT: MIDNITE MINE

TITLE: CROSS SECTION LOCATIONS

MWH

FIGURE 1

REVISION: A

FILE NAME: 1011322 PLAN

DATE: JUNE 2014

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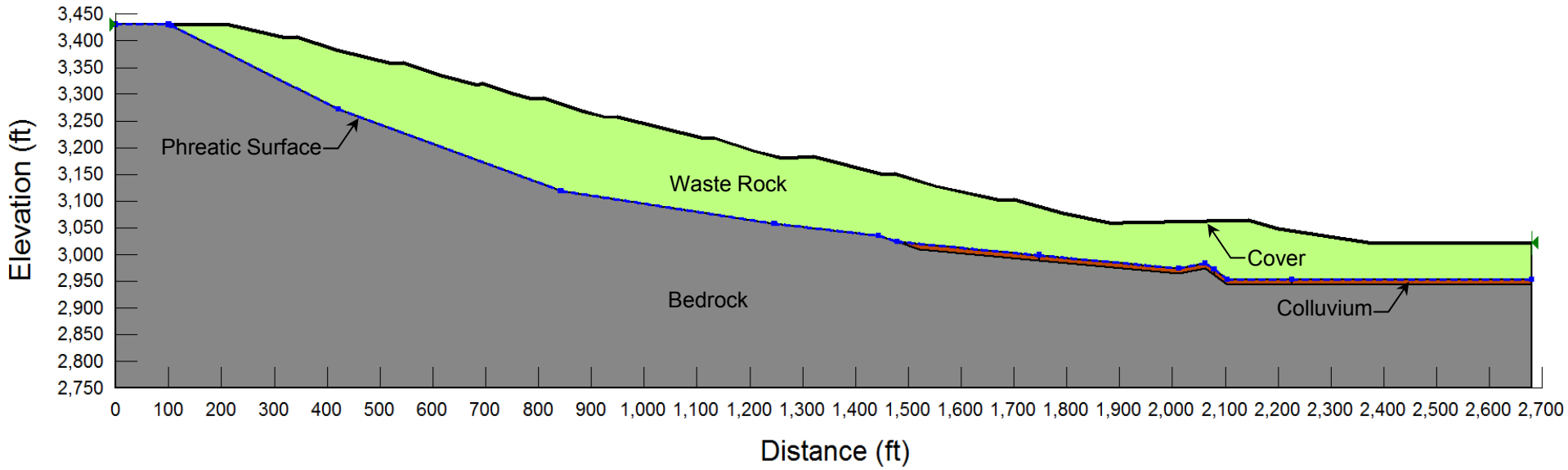
Midnite Mine
Slope Stability Analysis
Pit 4 - Cross-Section 1
Geometry

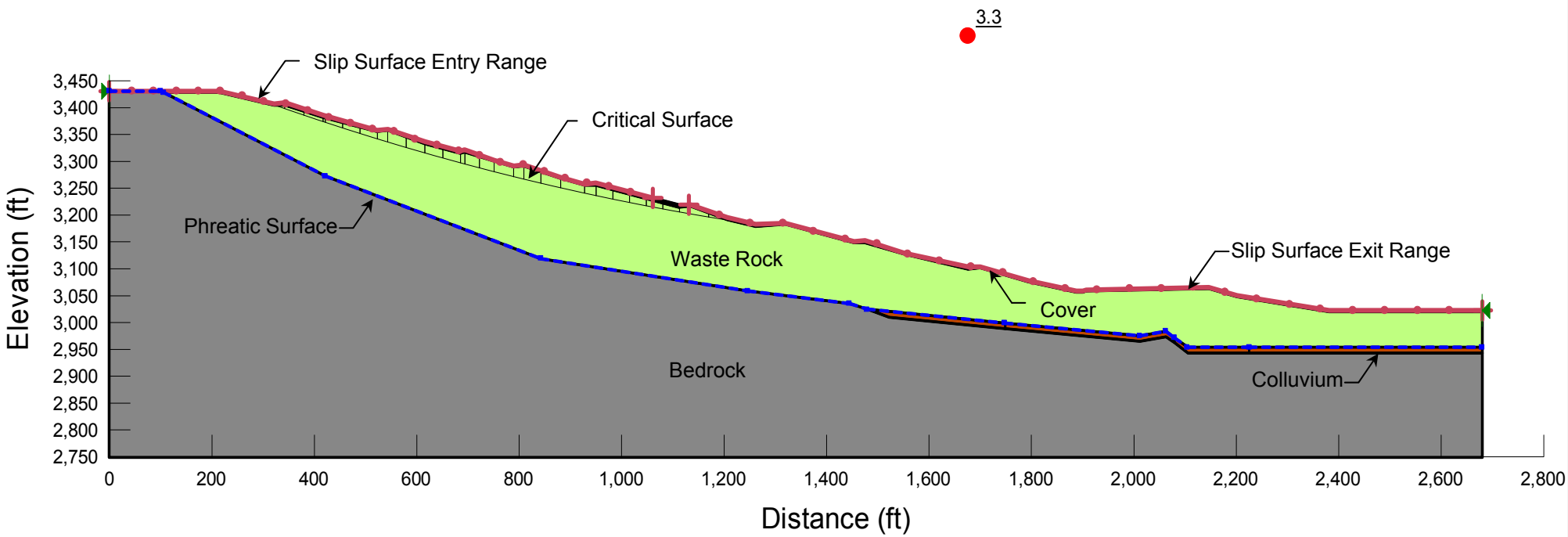


NEWMONT	PROJECT: MIDNITE MINE	MWH
	TITLE: Slope Stability Analysis – Geometry Pit 4 - Cross-Section 1	DATE: JUN 2014 — FIGURE 2
		FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 4 - Cross-Section 1
Static Conditions

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)
Cover	110	35.2	0
Waste Rock	135	38.0	0
Colluvium	122	30.8	0
Bedrock	Impenetrable		



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PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Static Conditions
Pit 4 - Cross-Section 1

MWH

DATE: JUN 2014

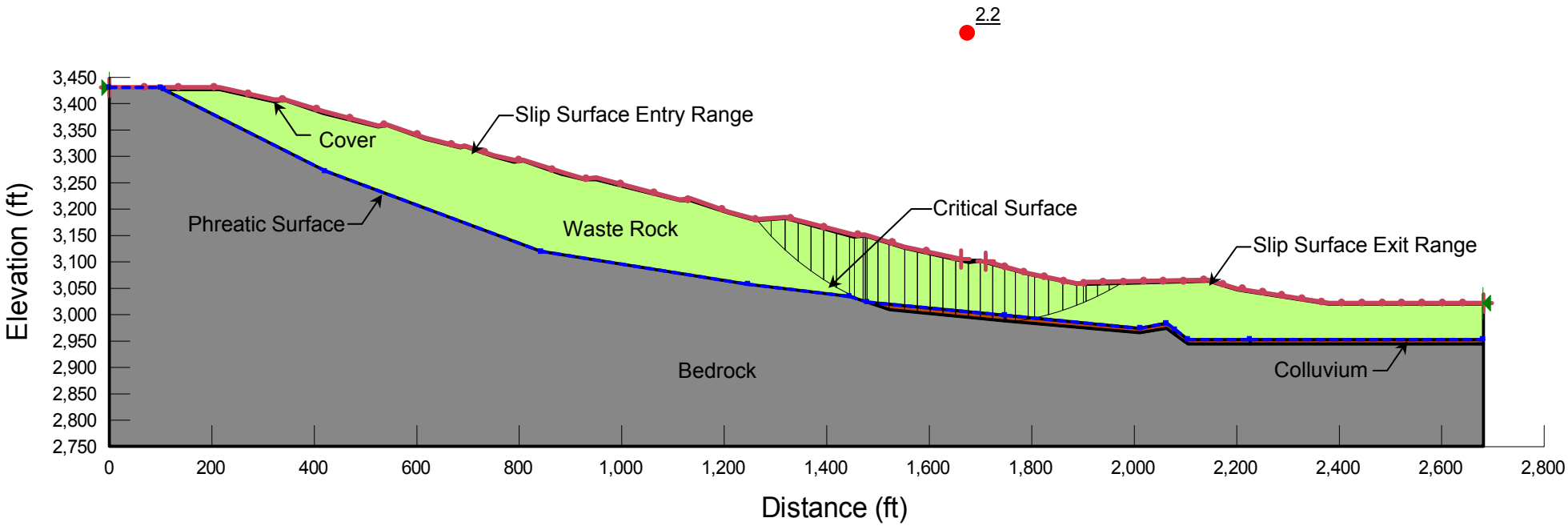
FIGURE 3

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 4 - Cross-Section 1
Pseudo-Static Conditions (K_h=0.105)

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	2.10 (Free Field) 0.56 (30' Waste Rock Fill) 0.46 (65' Waste Rock Fill) 0.41 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Pseudo-Static Conditions
Pit 4 - Cross-Section 1

MWH

DATE: JUN 2014

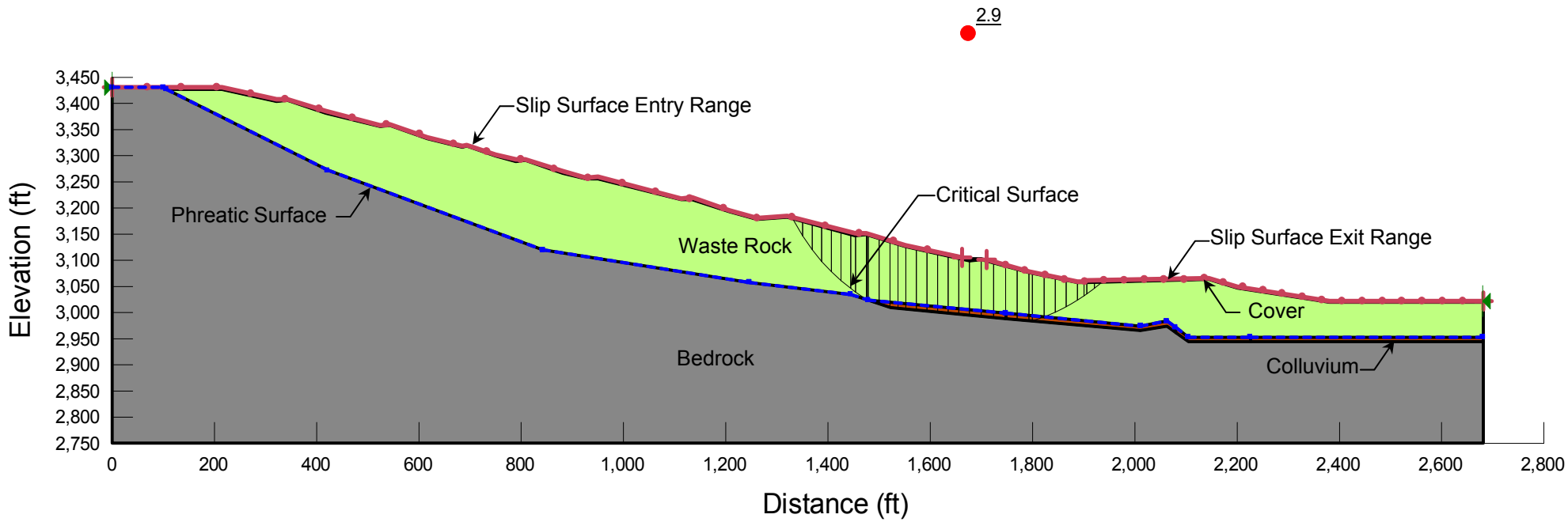
FIGURE 4

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 4 - Cross-Section 1
Post-Earthquake Conditions
Clay Behavior

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	1.55 (Free Field) 0.42 (30' Waste Rock Fill) 0.34 (65' Waste Rock Fill) 0.31 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT
MIDNITE MINE

TITLE
Slope Stability Analysis - Post-Earthquake Conditions
Clay Behavior
Pit 4 - Cross-Section 1

MWH

DATE
JUN 2014

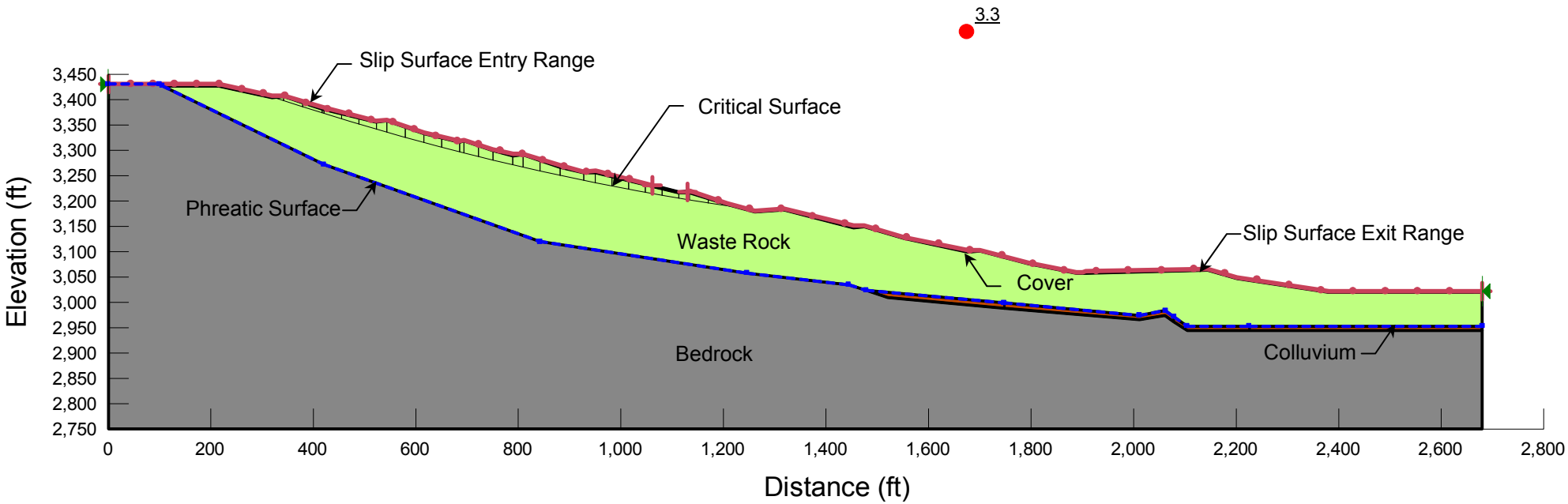
FIGURE 5

FILENAME
Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 4 - Cross-Section 1
Post-Earthquake Conditions
Sand Behavior

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	- 28	- 0	0.08 (Free Field) -
Bedrock	Impenetrable			-



NEWMONT

PROJECT
MIDNITE MINE

TITLE
Slope Stability Analysis - Post-Earthquake Conditions
Sand Behavior
Pit 4 - Cross-Section 1

MWH

DATE
JUN 2014

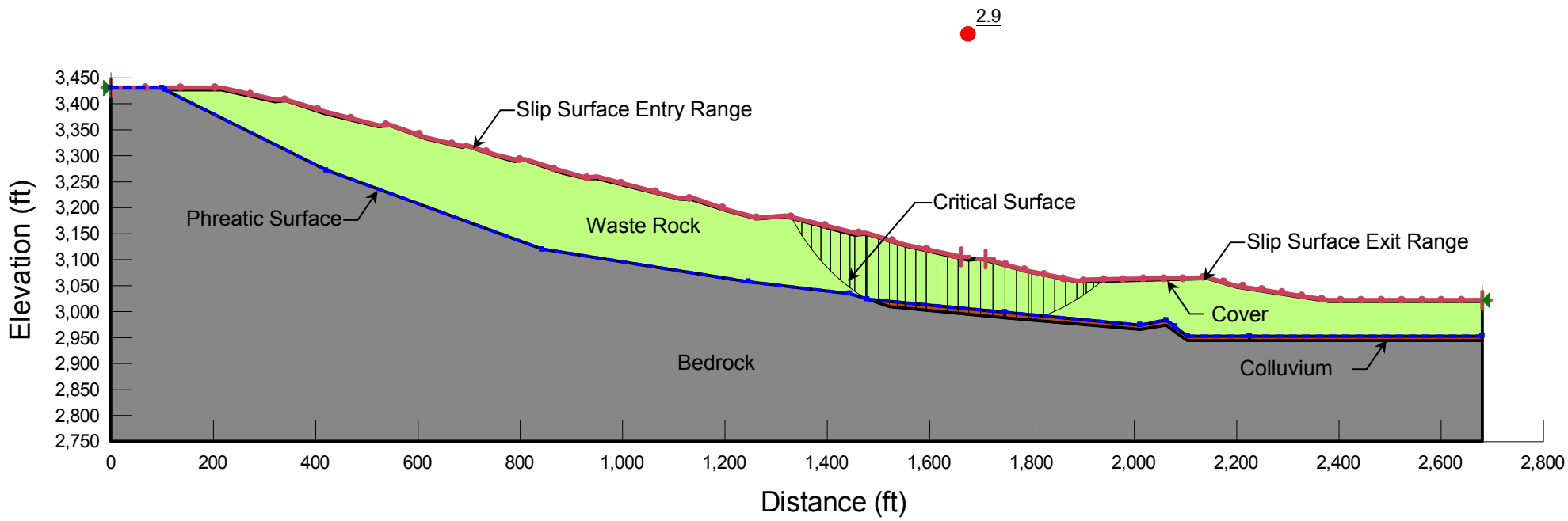
FIGURE 6

FILENAME
Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 4 - Cross-Section 1
Post-Earthquake Conditions
Sand-Clay Behavior

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	0.08 (Free Field) 0.42 (30' Waste Rock Fill) 0.34 (65' Waste Rock Fill) 0.31 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT
MIDNITE MINE

TITLE
Slope Stability Analysis - Post-Earthquake Conditions
Sand-Clay Behavior
Pit 4 - Cross-Section 1

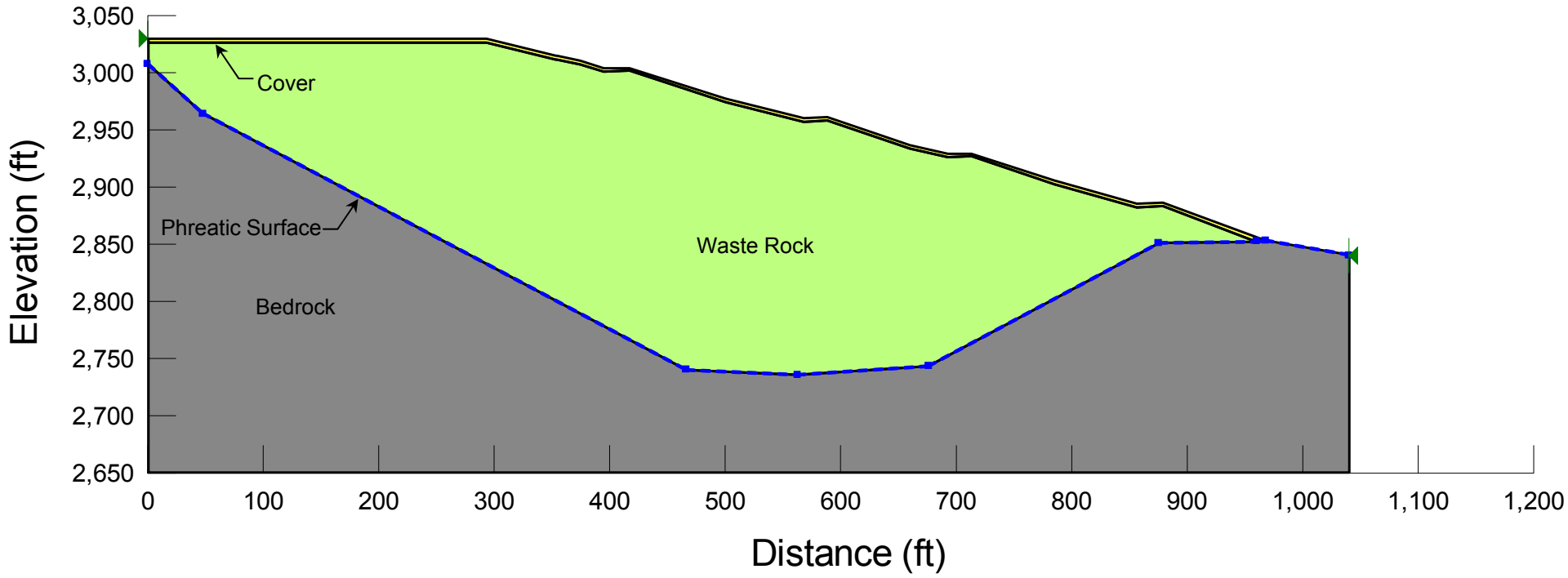
MWH

DATE
JUN 2014

FIGURE 7

FILENAME
Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 2
Geometry

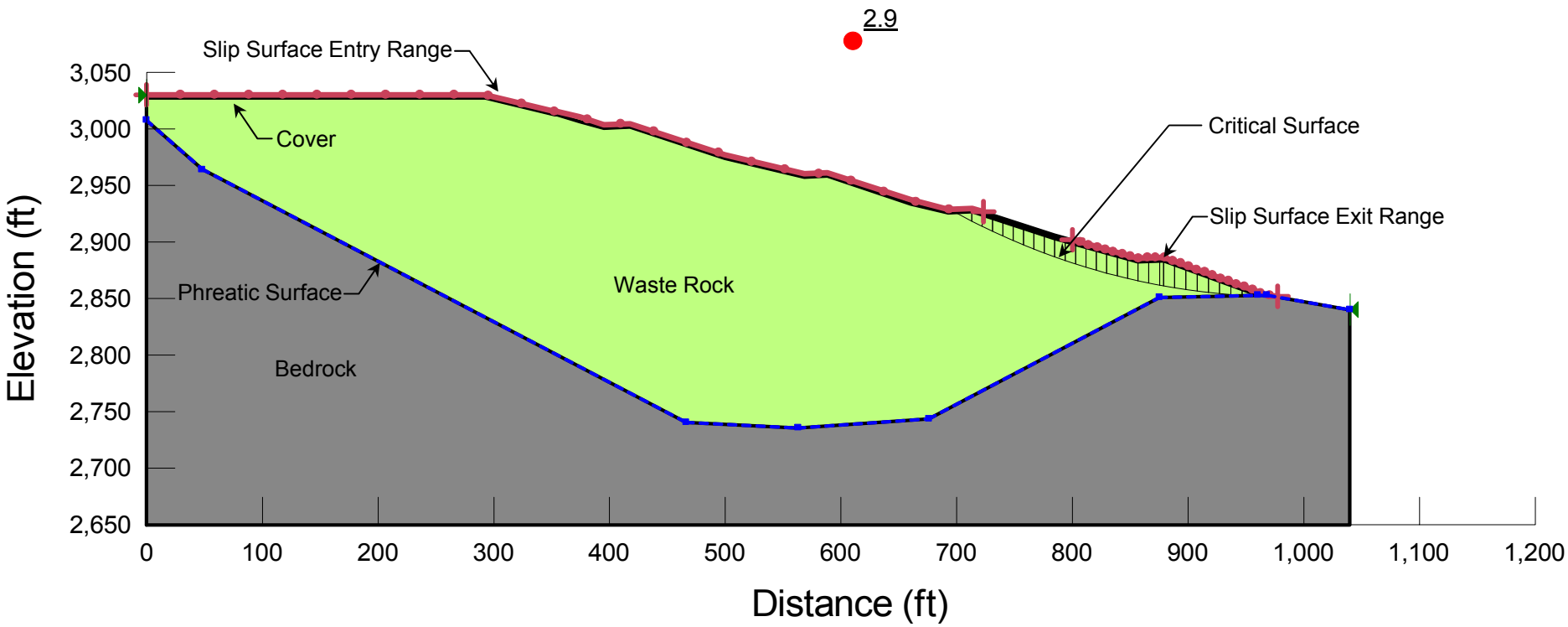


NEWMONT	PROJECT: MIDNITE MINE	MWH
	TITLE: Slope Stability Analysis - Geometry Pit 3 - Cross-Section 2	DATE: JUN 2014 — FIGURE 8
		FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 2
Static Conditions

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)
Cover	110	35.2	0
Waste Rock	135	38.0	0
Bedrock	Impenetrable		



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PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Static Conditions
Pit 3 - Cross-Section 2

MWH

DATE: JUN 2014

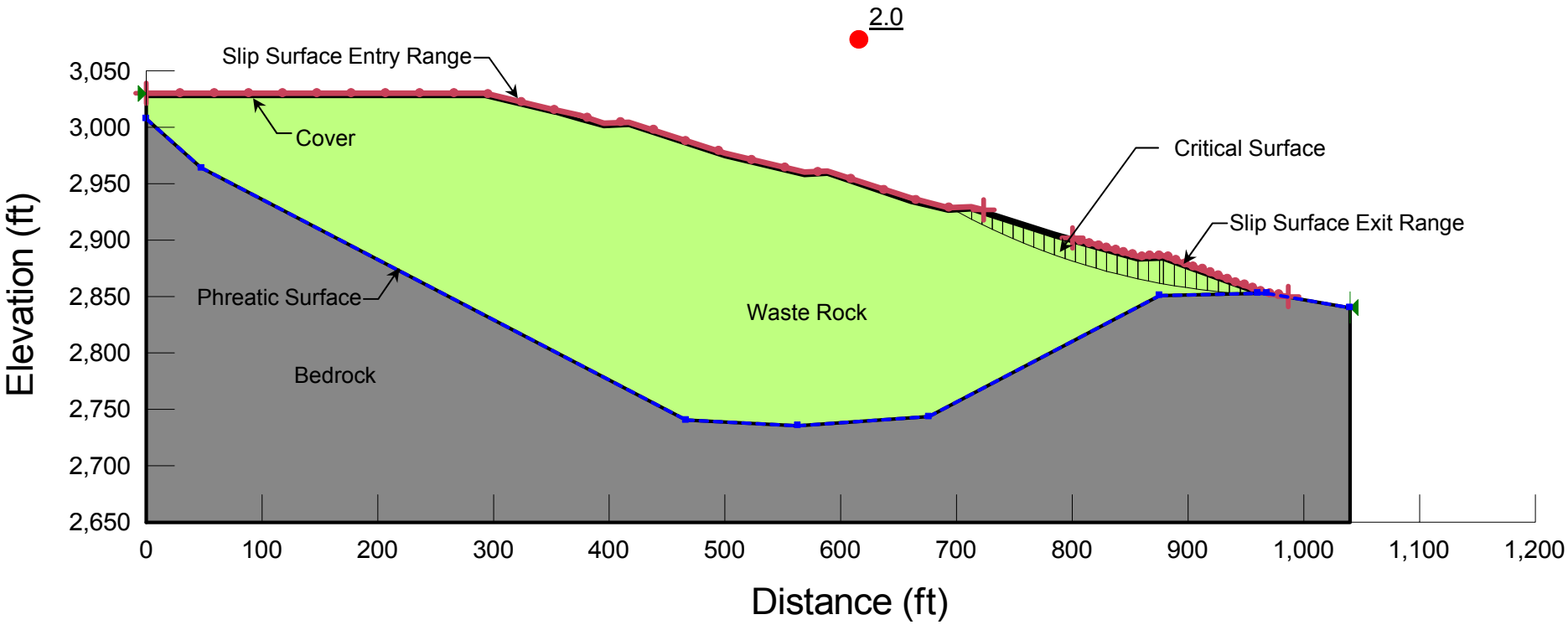
FIGURE 9

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 2
Pseudo-Static Conditions (K_h=0.105)

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Bedrock	Impenetrable			-



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PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Pseudo-Static Conditions
Pit 3 - Cross-Section 2

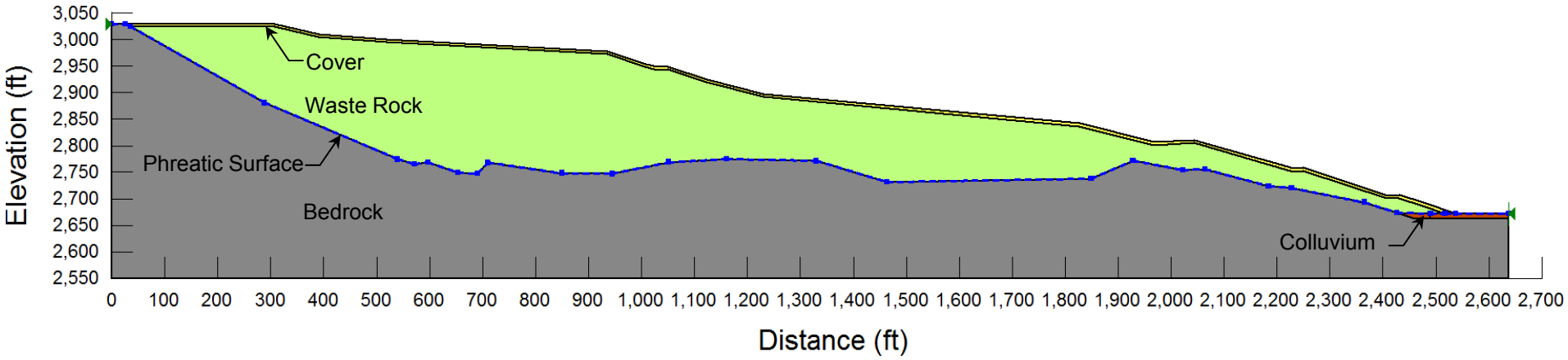
MWH

DATE: JUN 2014

FIGURE 10

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Geometry



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PROJECT
MIDNITE MINE

TITLE
Slope Stability Analysis - Geometry
Pit 3 - Cross-Section 3
Top of Slope

MWH

DATE
JUN 2014

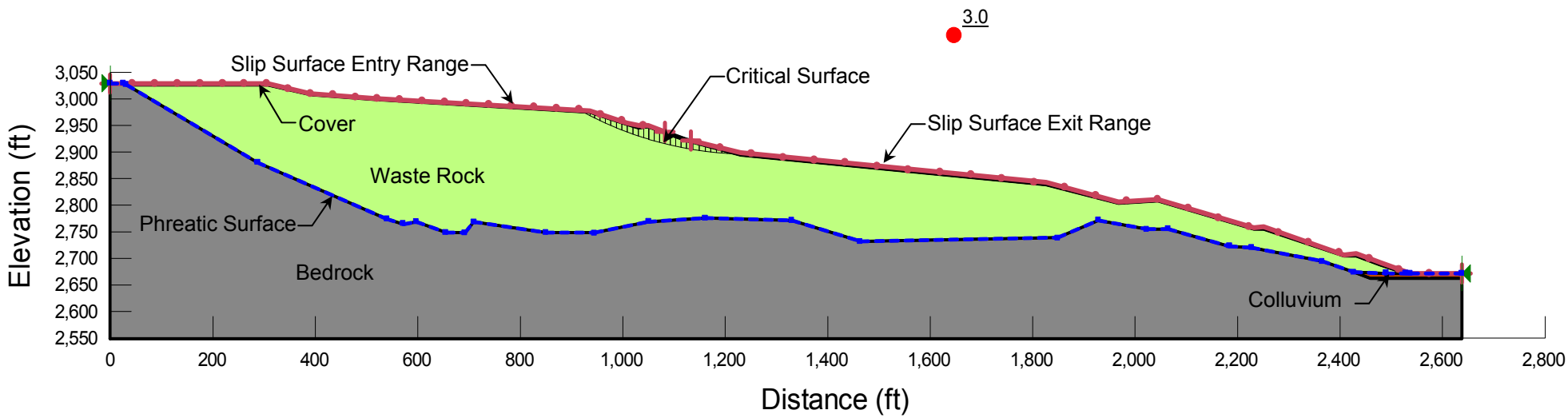
FIGURE 11

FILENAME
Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Static Conditions
Top of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)
Cover	110	35.2	0
Waste Rock	135	38.0	0
Colluvium	122	30.8	0
Bedrock	Impenetrable		



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PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Static Conditions
Pit 3 - Cross-Section 3
Top of Slope

MWH

DATE: JUN 2014

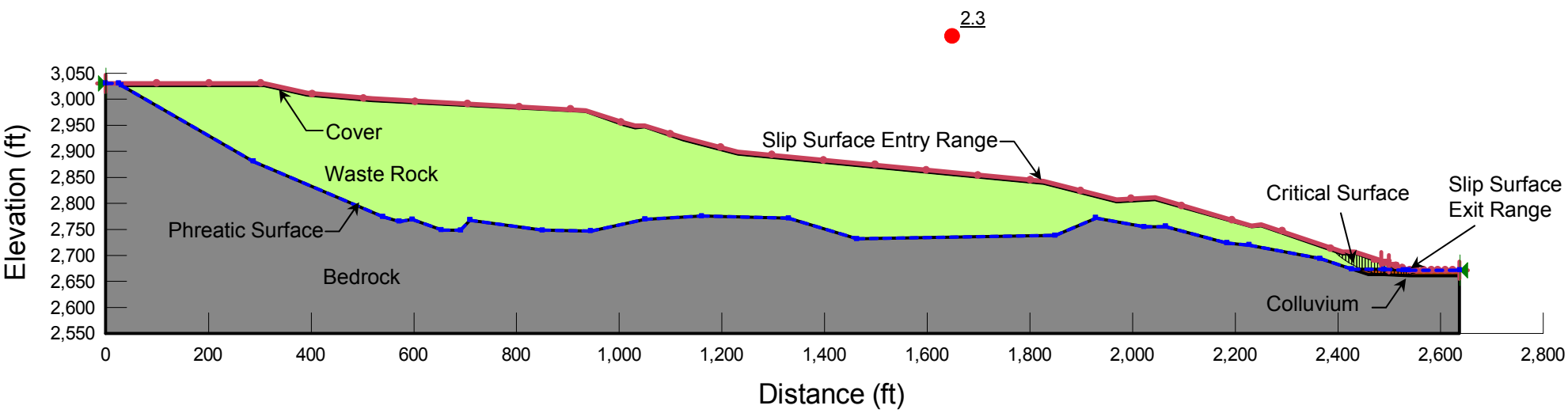
FIGURE 12

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Static Conditions
Toe of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)
Cover	110	35.2	0
Waste Rock	135	38.0	0
Colluvium	122	30.8	0
Bedrock	Impenetrable		



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PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Static Conditions
Pit 3 - Cross-Section 3
Toe of Slope

MWH

DATE: JUN 2014

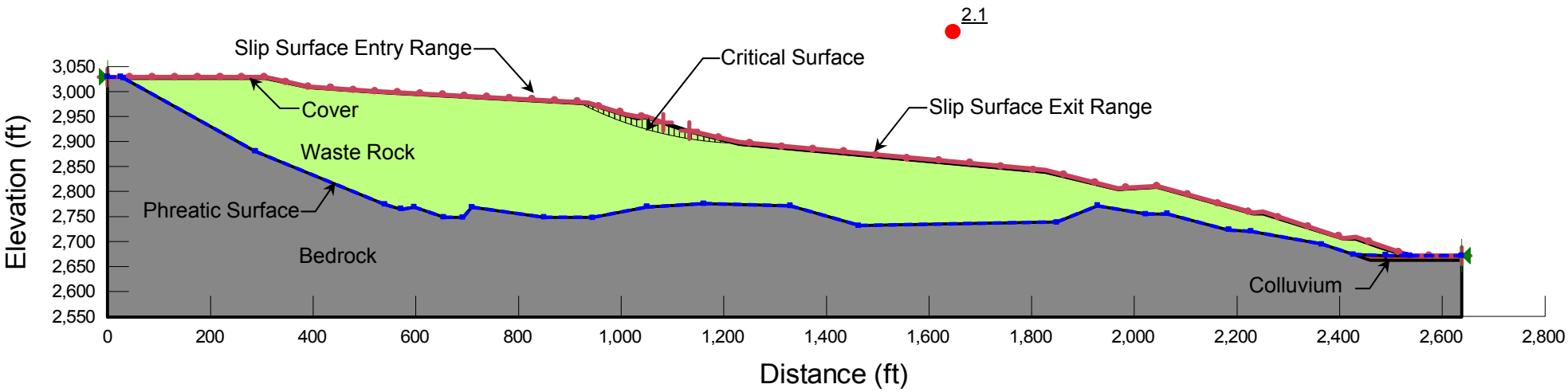
FIGURE 13

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Pseudo-Static Conditions (K_h=0.105)
Top of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	2.10 (Free Field) 0.56 (30' Waste Rock Fill) 0.46 (65' Waste Rock Fill) 0.41 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Pseudo-Static Conditions
Pit 3 - Cross-Section 3
Top of Slope

MWH

DATE: JUN 2014

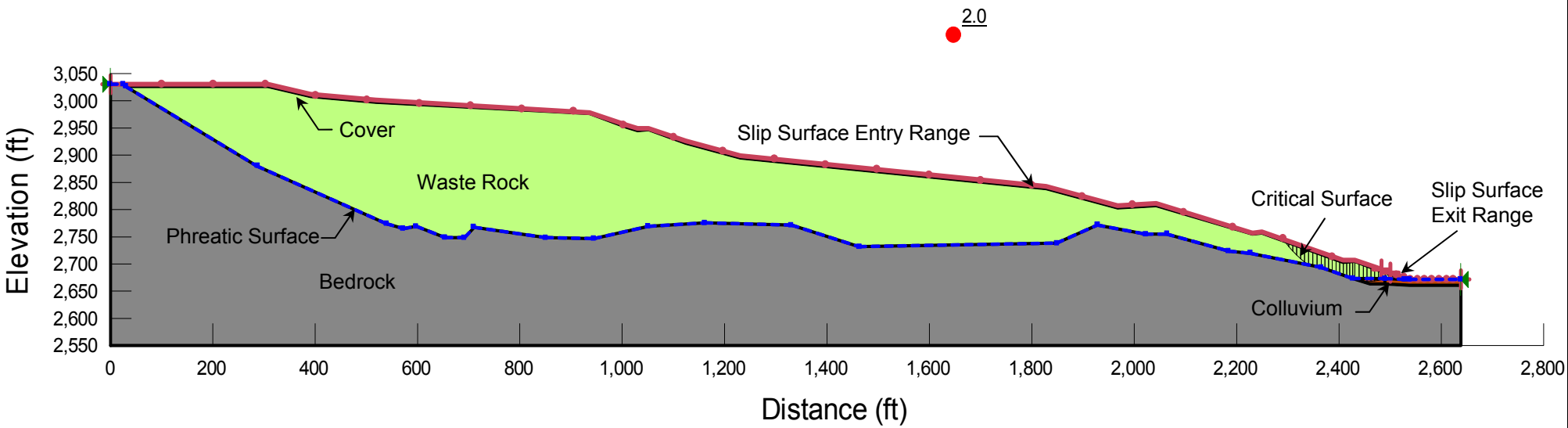
FIGURE 14

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Pseudo-Static Conditions (K_h=0.105)
Toe of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	2.10 (Free Field) 0.56 (30' Waste Rock Fill) 0.46 (65' Waste Rock Fill) 0.41 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Pseudo-Static Conditions
Pit 3 - Cross-Section 3
Toe of Slope

MWH

DATE: JUN 2014

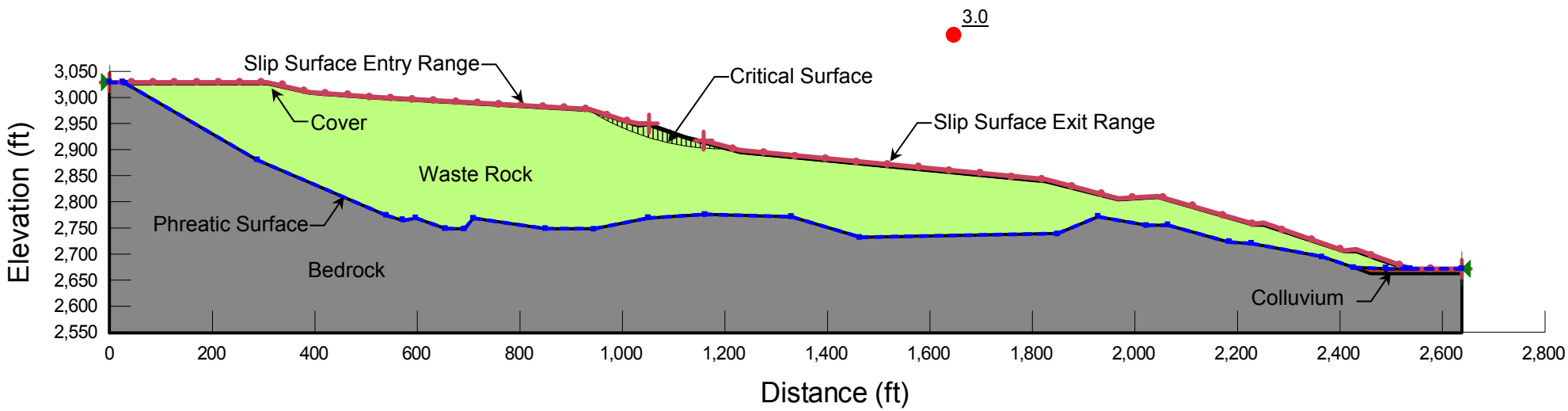
FIGURE 15

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Post-Earthquake Conditions
Clay Behavior
Top of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	1.55 (Free Field) 0.42 (30' Waste Rock Fill) 0.34 (65' Waste Rock Fill) 0.31 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT
MIDNITE MINE

TITLE
Slope Stability Analysis - Post-Earthquake Conditions
Clay Behavior
Pit 3 - Cross-Section 3 – Top of Slope

MWH

DATE
JUN 2014

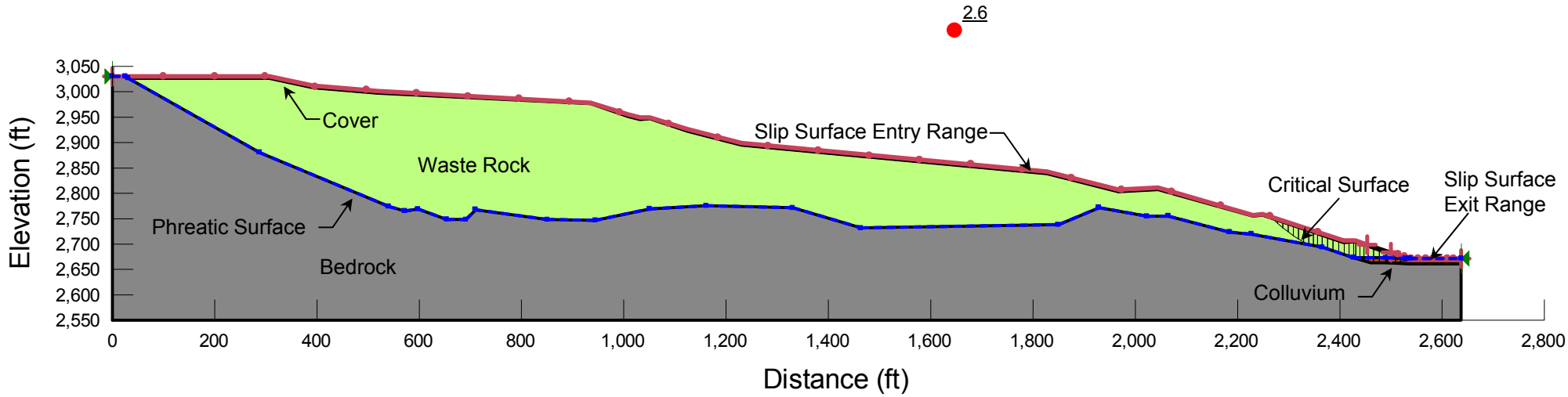
FIGURE 16

FILENAME
Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Post-Earthquake Conditions
Clay Behavior
Toe of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	1.55 (Free Field) 0.42 (30' Waste Rock Fill) 0.34 (65' Waste Rock Fill) 0.31 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Post-Earthquake Conditions
Clay Behavior
Pit 3 - Cross-Section 3 – Toe of Slope

MWH

DATE: JUN 2014

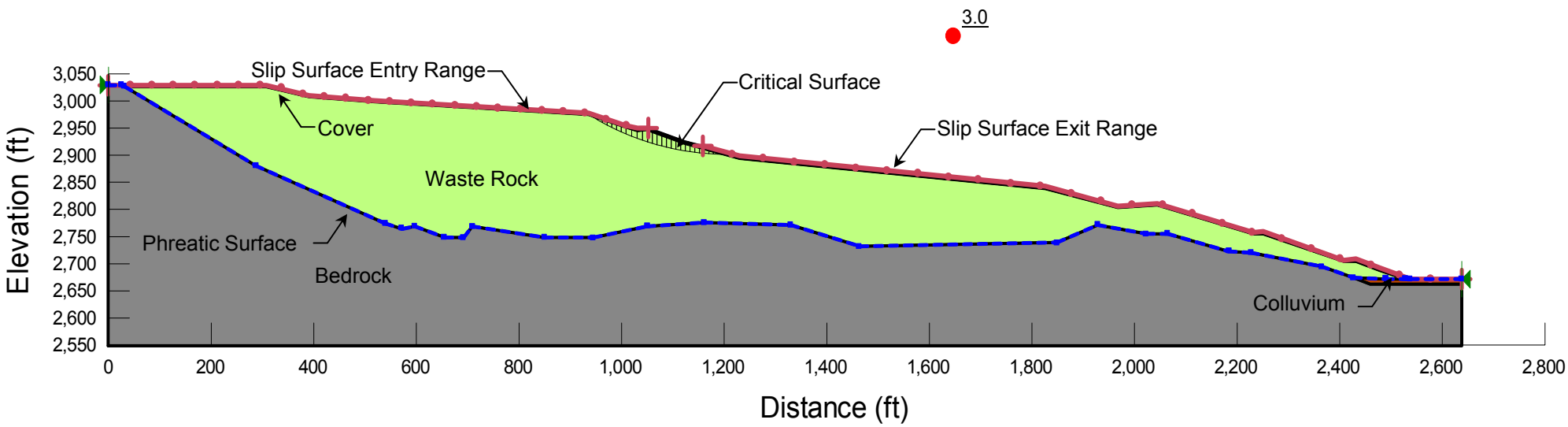
FIGURE 17

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Post-Earthquake Conditions
Sand Behavior
Top of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	- 28	- 0	0.08 (Free Field) -
Bedrock	Impenetrable			-



NEWMONT

PROJECT
MIDNITE MINE

TITLE
Slope Stability Analysis - Post-Earthquake Conditions
Sand Behavior
Pit 3 - Cross-Section 3 – Top of Slope

MWH

DATE
JUN 2014

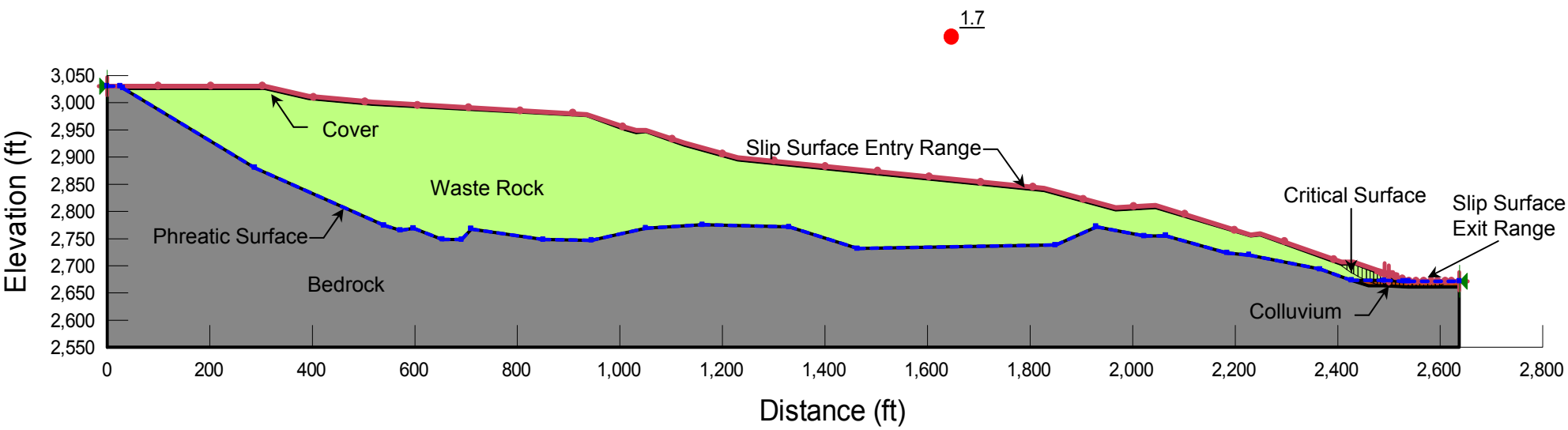
FIGURE 18

FILENAME
Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Post-Earthquake Conditions
Sand Behavior
Toe of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	- 28	- 0	0.08 (Free Field) -
Bedrock	Impenetrable			-



NEWMONT

PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Post-Earthquake Conditions
Sand Behavior
Pit 3 - Cross-Section 3 – Toe of Slope

MWH

DATE: JUN 2014

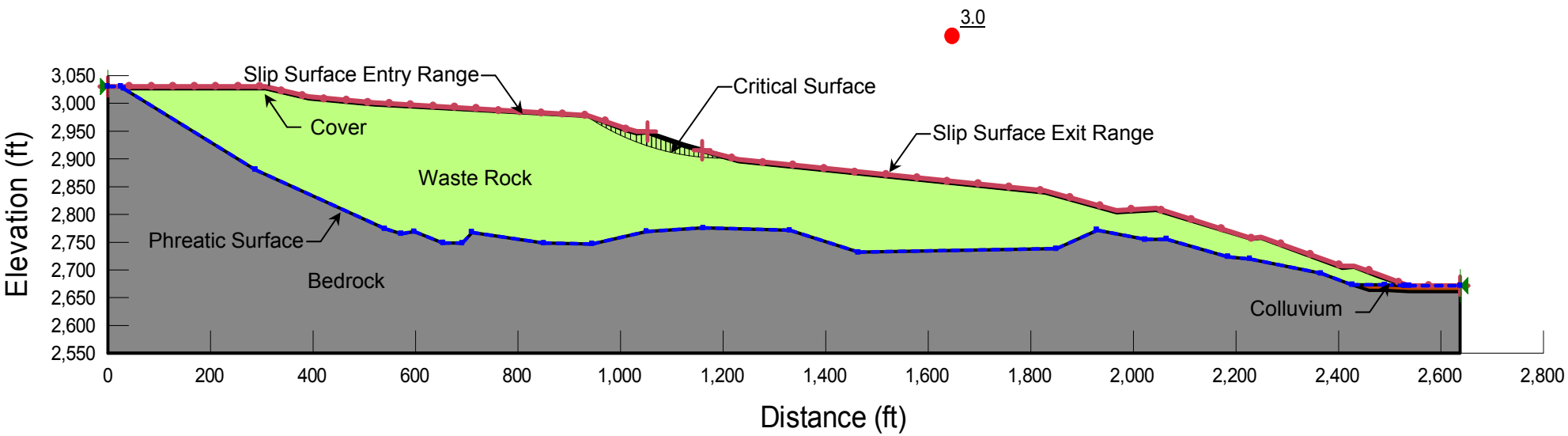
FIGURE 19

FILENAME: Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Post-Earthquake Conditions
Sand-Clay Behavior
Top of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	0.08 (Free Field) 0.42 (30' Waste Rock Fill) 0.34 (65' Waste Rock Fill) 0.31 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT
MIDNITE MINE

TITLE
Slope Stability Analysis - Post-Earthquake Conditions
Sand-Clay Behavior
Pit 3 - Cross-Section 3 – Top of Slope

MWH

DATE
JUN 2014

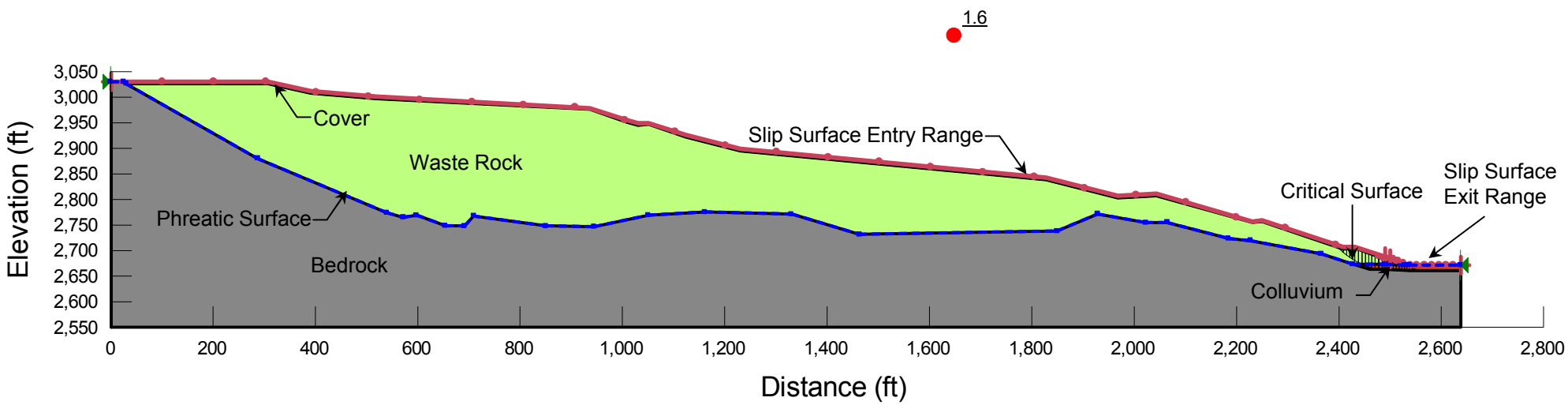
FIGURE 20

FILENAME
Pit3 & 4 – Figures.pptx

Midnite Mine
Slope Stability Analysis
Pit 3 - Cross-Section 3
Post-Earthquake Conditions
Sand-Clay Behavior
Toe of Slope

Material Parameters Used in Stability Analyses

Material Type	Total Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Cohesion, c' (psf)	s_u/σ_v'
Cover	110	35.2	0	-
Waste Rock	135	38.0	0	-
Colluvium	122	-	-	0.08 (Free Field) 0.42 (30' Waste Rock Fill) 0.34 (65' Waste Rock Fill) 0.31 (120' Waste Rock Fill)
Bedrock	Impenetrable			-



NEWMONT

PROJECT: MIDNITE MINE

TITLE: Slope Stability Analysis - Post-Earthquake Conditions
Sand-Clay Behavior
Pit 3 - Cross-Section 3 – Toe of Slope

MWH

DATE: JUN 2014

FIGURE 21

FILENAME: Pit3 & 4 – Figures.pptx

Attachment D-6

Erosional Stability Analyses

ATTACHMENT D-6

MIDNITE MINE REMEDIAL ACTION EROSIONAL STABILITY ANALYSES

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*15-Nov-12*	*30% Design*	*Melanie Davis*	*Tom Kelley*	*21-Nov-12*	*Tom Kelley*
1	*02-Dec-13*	*60% Design*	*Melanie Davis*	*Jason Cumbers*	*18-Nov-13*	*Clint Strachan*
2	*21-May-14*	*90% Design*	*Melanie Davis*	*Jared Erickson/ Jason Cumbers*	*21-May-14*	*Clint Strachan*
2	*14-May-15*	*100% Design – no changes*				

Location and Format

Electronic copies of these calculations are located in the project files system at:

\\usslc1s01\IFO\Industrial Projects\MIDNITE MINE\Deliverables_Working Documents\Basis of Design Rpt\Appendix D - Mine Waste Excavation and Containment\90% Submittal\Att D-6 Erosional Stability

The following calculations were generated using the following software:

Microsoft Office Professional Plus 2010 (Microsoft Excel)
United States Department of Agriculture RUSLE2 Program Version 1.26.6.4, 2006 (RUSLE2)

Supplements

Supplement D-6.1 –Time of Concentration and Unit Discharge Calculation Sheets
Supplement D-6.2 –Temple Method Erosion Protection Calculation Sheets
Supplement D-6.3 –Average Depth of Erosion Calculation Sheet

1.0 BACKGROUND

This calculation brief presents the results of erosional stability analyses for the cover system that will be part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The cover system will be placed over mine waste that will be consolidated in Pit 3 (including the Backfilled Pit Area, or BPA) and Pit 4, and over the regraded surface of Area 5 (the area between Pits 3 and 4).

The proposed cover system consists of a uniform soil layer overlying a synthetic geomembrane. On steeply sloped areas (25 to 33 percent slopes), the cover system will also include a geocomposite drainage layer (GDL) between the soil and geomembrane layers to reduce the potential pore-water build-up over the geomembrane and thereby enhance slope stability. The GDL will consist of a non-woven, needle-punched geotextile heat-bonded to an underlying geonet.

The soil from the Rhoads Property Borrow Area located adjacent to the Site is analyzed as the soil cover borrow material. The predominant soil in this borrow area consists of clayey sand (MGC, 2011a, b). The upper zone of bedrock consists of weathered quartz monzonite which has weathered to a residual soil (residuum). Both the clayey sand and the residuum may be used for the cover system. A minimum thickness of 1.8 feet is required for this cover soil based on radon emanation modeling results (refer to Attachment D-3); however a minimum soil cover thickness of 3.0 feet has been proposed, based on construction and slope stability considerations.

The preliminary design for the cover system called for the top 0.5 feet of the soil cover to be topsoil or amended borrow soil to promote vegetation growth to enhance erosional stability. However based upon data presented in the Borrow Source Design Investigation Report (MGC, 2011a), it does not appear that amendment of soil from the Rhoads Property Borrow Area will be necessary to promote vegetation growth (see the Revegetation Plan, Attachment D-12). Therefore, for the purposes of these analyses, the top 0.5 feet of the cover system is assumed to have similar material properties as the underlying soil cover material. Additional information on the borrow source is provided in MGC (2011a, b).

The most critical slopes selected for evaluating erosional stability are the steepest interbench slopes. The proposed side slope geometry for the cover slopes include drainage benches at a 50-foot vertical spacing, with a 25-foot width based on recommendations in Best Management Practices for Reclaiming Surface Mines in Washington and Oregon (Norman et. al, 1997). The steepest interbench slope will be 3H:1V. The benches will be shaped to provide a drainage channel on the bench surface and prevent over-crest runoff. In addition, the benches will be sloped toward the edge of cover to convey surface water runoff towards stormwater diversion channels located off the waste containment surfaces.

2.0 DESIGN STORM EVENT

The design storm for erosional stability of the cover is the 100-year, 24-hour storm event (EPA, 2011). The Rational Method as outlined in WDOE (2004) was used to calculate peak flows from the design storm, and is discussed in the following section.

3.0 CALCULATION OF PEAK DISCHARGE

The following inputs were used in the calculation of peak discharge:

1. The peak discharge calculations were made using the Rational Method as recommended in the *Surface Water Design Investigation Report* and consistent with WDOE (2004) guidance. The Rational Method is also recommended for evaluation of erosional stability of cover systems in Johnson (2002) and Nelson et al. (1986). The procedure used is as described in WDOE (2004). Peak discharge was converted to a peak unit discharge for the analysis. The equation used for peak unit discharge is as follows:

$$Q_{unit} = C*I*L/43{,}200$$

   Where Q_{unit} = peak unit discharge(cfs/ft)
   C = runoff coefficient (dimensionless)
   I = peak rainfall intensity (in/hr)
   L = slope length (ft), maximum slope length used for analysis
   A factor of 1/43,200 is used to convert in/hr to ft/s.

2. The runoff coefficient was selected based on soil type, vegetation and referenced values in WDOE (2004). The analysis was conducted for established vegetation conditions.

3. The cover was represented as side slopes/turf with slopes greater than 10 percent and a corresponding runoff coefficient of 0.30 from Table 4.7.1 in WDOE (2004). As recommended in Section 4.7.1 of WDOE (2004), this value was increased by 25 percent for a 100-year storm event, with a resulting runoff coefficient of 0.375.

4. The time of concentration was calculated for the 3H:1V interbench cover slopes by the equation listed on page 4-57 in WDOE (2004) and shown below:

$$T_c = L / k * S^{0.5}$$

   Where T_c = time of concentration (min)
   L = slope length (ft), maximum slope length used for analysis
   k = ground cover coefficient (ft/min) from Table 4.7.3 in WDOE (2004)
   S = slope (ft/ft), maximum interbench slope used for analysis

   A minimum of 5 minutes was used for the time of concentration as recommended in WDOE (2004). The cover was represented as “short pasture, grass, or lawn” with a corresponding ground cover coefficient, k, of 420 ft/min from Table 4.7.3 in WDOE (2004).

5. The peak rainfall intensity was calculated using the second equation listed on page 4-56 in WDOE (2004) and shown below:

$$I = m / (T_c)^n$$

Where I = peak rainfall intensity (in/hr)
T_c = time of concentration (min)
m and n = rainfall intensity coefficients (dimensionless), from Table 4.7.2 in WDOE (2004)

The values of the rainfall intensity coefficients, m and n, from Table 4.72 in WDOE (2004) are 12.33 and 0.643, respectively, for the 100-year mean recurrence interval for Spokane, WA.

The 100-year, 24-hour peak unit discharge result is presented in Table 1. The discharge represents downslope flow for a unit-width of the slope. A calculation sheet is provided in Supplement D-6.1.

Table 1. Peak Reclaimed Surface Discharge

Interbench Slope	Slope Length (feet)	Time of Concentration (min)	Rainfall Intensity (in/hr)	Runoff Coefficient	Peak Unit Discharge (cfs/ft)
3H:1V	171	5.0	4.38	0.375	0.006

in/hr = inches per hour; cfs/ft = cubic feet per second per foot

The peak unit discharge value in Table 1 above was used to evaluate the erosional stability of the reclaimed surfaces. The erosional stability evaluation is presented below.

4.0 EROSIONAL STABILITY

4.1 VEGETATED SLOPES

The surfaces of the vegetated slopes were evaluated for erosional stability using the methods recommended in NRC (1990) and Johnson (2002).

Temple Method. Temple et al. (1987) outlines procedures for grass-lined channel design. These procedures are recommended in Johnson (2002) for areas of vegetated cover and include methods for estimating stresses on channel vegetation as well as the channel surface soils. The evaluation for the vegetated cover slopes used the peak discharge values from the 100-year, 24-hour storm event (summarized in Table 1) to represent the effective stresses from runoff on the cover surface. It is assumed that the soil covers will not be erosionally stable, and that repair and maintenance will be required if the design storm event occurs prior to vegetation being established on the soil cover. Therefore, the stresses were only evaluated for the scenario after vegetation establishment.

The erosional stability of the cover surfaces was evaluated by calculating a factor of safety against erosion due to the peak runoff from the 100-year, 24-hour storm event. Factor of safety values were calculated as the ratio of the allowable stresses (the resisting strength of the cover vegetation or soils) to the effective stresses (the stresses imparted by the runoff flowing over the

cover). The surfaces were evaluated for two conditions: (1) resistance of the vegetation, and (2) resistance of the cover system layer.

The peak unit discharge flow for the 3H:1V interbench slope (from Table 1) was conservatively multiplied by a flow concentration factor of three to estimate the design flow, as recommended in Johnson (2002). The equations listed below for calculating effective stresses are applicable for a range of design flows (Temple et al., 1987) of:

$$0.0025C_I^{2.5} < q_{design} \leq 36$$

Where C_I = retardance curve index (defined below)
q_{design} = design flow (cfs/ft)

The calculated design flow is below this range. As per recommendations in Temple et al. (1987), the lower bound value of this range was used as the design flow in the calculations for effective shear stresses.

Allowable stresses. Allowable stresses for the cover soil were calculated using the equations in Temple et al. (1987). Material planned for the cover soil consists of borrow from the Rhoads Property Borrow Area. The Rhoads Property Borrow Area is classified as clayey sand (MGC, 2011a, b) and is considered a cohesive soil. For cohesive soils, erosional resistance is based on the plasticity index (PI) and void ratio of the material. From laboratory testing (MGC, 2011a, b), the average PI of this material is 18. The void ratio is estimated as 0.61 which is equivalent to 85 percent of average standard Proctor dry density of the borrow samples using a laboratory measured specific gravity of 2.63.

The equation for allowable shear strength for cohesive soils used for the Rhoads Property Borrow Area is:

$$\tau_a = \tau_{ab} C_e^2$$

Where τ_a = allowable shear strength (in psf)
τ_{ab} = base allowable shear strength = $(1.07\,[PI]^2+14.3[PI]+47.7)\times10^{-4}$
C_e = void ratio correction factor = 1.42 - 0.61e, where e is the void ratio

For a vegetated surface primarily of mixed grasses, the allowable vegetation shear strength is:

$$\tau_{va} = 0.75C_I$$

Where τ_{va} = allowable vegetation shear strength (in psf)
C_I =retardance curve index = $2.5\,[h(M)^{1/2}]^{1/3}$
h = stem length (in ft)
M = stem density factor

The vegetated shear strength was calculated for conditions after vegetation is established.

Effective stresses. The effective shear stress on soil due to peak runoff from the 100-year, 24-hour storm event was calculated as:

$$\tau_e = \gamma dS\left(1 - C_f\right)(n_s/n)^2$$

Where τ_e = effective shear stress (in psf)

γ = unit weight of water = 62.4 pounds per cubic foot (pcf)
d = depth of flow (ft)
S = slope of cover surface (ft/ft), from Table 1
C_f = cover factor (0.7 for established vegetation)
n_s = soil grain roughness factor (0.0156 for cohesive soil with a PI>10), and
n = Manning's roughness coefficient for vegetated surface

$$n = e^{C_i\left(0.0133[\ln q]^2 - 0.0954 \ln q + 0.297\right) - 4.16}$$

The effective shear stress on vegetation is calculated as:

$$\tau_v = \gamma dS - \tau_e$$

Where τ_v = effective vegetal stress (in psf)

The factor of safety, defined as the ratio of allowable to effective (applied) shear stress for both soil and vegetation with established vegetation conditions on the cover surfaces are summarized in Table 2. A factor of safety of less than 1.0 indicates failure, as defined by erosion of the cover soil, or loss of vegetative cover, during the design storm event. Calculation sheets are provided in Supplement D-6.2.

These analyses indicate that cover slopes constructed as vegetated slopes without rock for erosion protection will be erosionally stable for established vegetation conditions. The calculated factors of safety are at or above 1.0.

In addition, a sensitivity analysis was conducted to identify the steepest interbench slope that would achieve a minimum factor of safety of 1.0 for a maximum slope length of 200 ft. Calculation sheets are provided in Supplements D-6.1 and D-6.2. The analysis results (summarized in Table 2) indicate that a vegetated interbench slope up to 2.5H:1V would be erosionally stable.

Table 2. Factors of Safety for Erosion Protection of Cover

Interbench Slope	Slope Length (ft)	Description of Erosion Protection	Factor of Safety for Soil on Vegetated Slope	Factor of Safety for Vegetation
3H:1V	171	Vegetation and Top Soil	16.4	1.1
2.5H:1V	200	Vegetation and Top Soil	14.4	1.0

5.0 SHEET EROSION OF VEGETATED SLOPES

Version 2 of the Revised Universal Soil Loss Equation (RUSLE2) was used to evaluate the potential for soil loss due to sheet flows across the surface cover side slopes.

The RUSLE2 equation is defined as: $a = n * k * l * S * c$

Where: a = average soil loss for each day, in tons per acre per year
n = erosivity factor
k = soil erodibility
l = soil length factor
S = slope steepness factor
c = cover management factor

The factors listed above are daily values that can vary daily depending on the input parameters to the program. The average annual soil loss is the sum of the daily soil loss values.

The climate data used in the program was selected from the RUSLE2 database for Stevens County, Washington for an annual precipitation ranging from 18 to 20 inches. The average annual precipitation at the Midnite Mine site based on the on-site climate station is 18.35 inches. The soil texture was selected in the model as sandy loam for the Rhoads Property Borrow Area soils based on results of laboratory testing (MGC, 2011a, b).

The steepest interbench slopes are proposed to be 3H:1V, with a maximum length of 171 feet. The vegetation conditions assumed for the 1,000 year time period include bare ground with a rough surface for the initial two years, cool season grasses with poor stand for years 2 to 1000 for the remaining years. The parameters used in the program and the results are summarized in Table 3. Calculation sheets are provided in Supplement D-6.4.

The soil loss was calculated assuming a total unit weight of the cover soil of 110 pcf. The results show potential for surficial soil loss of 0.74 inches over 1,000 years. This surficial surface loss is less than the minimum design thickness of 6 inches for the topsoil layer and indicates the proposed configuration has acceptable long-term resistance against soil loss.

Table 3. Summary of RUSLE2 Model Parameters and Results

Model Parameter	Value
Soil Texture	Sandy Loam
Climate	Stevens County, WA Annual Precip.18 – 20 inches
Cover Slope (%)	33
Cover Slope Length (ft)	171
Vegetation Conditions	
Initial two years	bare ground
2 to 1,000 years	cool season grasses (poor stand)
Soil loss for bare ground conditions with rough surface (tons/acre/year)	14.00
Soil loss for cool season grasses (poor stand) vegetation conditions (tons/acre/year)	0.12
Soil loss (inches/1,000 years)	**0.74**

6.0 REFERENCES

Johnson, T.L., 2002. Design of Erosion Protection for Long-Term Stabilization. U.S. Nuclear Regulatory Commission (NRC), *NUREG-1623*. September.

Miller Geotechnical Consultants (MGC), 2011a. Borrow Source Design Investigation Report. Revision 2. Prepared on behalf of Newmont USA Limited and Dawn Mining Company for U.S. Environmental Protection Agency, Region 10. May 6.

Miller Geotechnical Consultants (MGC), 2011b. Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Revision 1. Prepared on behalf of Newmont USA Limited and Dawn Mining Company for U.S. Environmental Protection Agency, Region 10. December 2.

Nelson, J., S. Abt, R. Volpe, D. van Zyl, N. Hinkle, and W. Staub, 1986. Methodologies for Evaluation of Long-term Stabilization Designs of Uranium Mill Tailings Impoundments. *NUREG/CR-4620,* U.S. Nuclear Regulatory Commission. June.

Norman, D.K., P. Wampler, A. Throop, E.F. Schnitzer, and J.M. Roloff, 1997. Best Management Practices for Reclaiming Surface Mines in Washington and Oregon. Washington Division of Geology and Earth Resources, Open File Report 96-2. Oregon Department of Geology and Mineral Industries, Open File Report 96-2. December.

Temple, D.M., K.M. Robinson, R.A. Ahring, and A.G. Davis, 1987. Stability Design of Grass-Lined Open Channels. USDA Handbook 667.

United States Department of Agriculture, Natural Resources Conservation Service (USDA-NRCS), 2006. RUSLE2 program. Version 1.26.6.4. November 13.

U.S. Environmental Protection Agency (EPA), 2011. Statement of Work for the Remedial Design and Remedial Action for the Midnite Mine Superfund Site; Spokane Indian Reservation, Washington. August.

U.S. Nuclear Regulatory Commission (NRC), 1990. Final Staff Technical Position, Design of Erosion Protective Covers for Stabilization of Uranium Mill Tailings Sites. August.

Washington State Department of Ecology (WDOE), 2004. Stormwater Management Manual for Eastern Washington, Water Quality Program. September.

Supplement D-6.1

Time of Concentration and Unit Discharge Calculation Sheets

Client: Dawn Mining Company/Newmont USA Limited
Project: Midnite Mine
Detail: Remedial Action Cover Design - Cover Slopes , Rhoads Property borrow
Job No.: 1011322
Date: 4/18/2014
Calc. By: M.Davis
Checked By: J. Erickson

TIME OF CONCENTRATION AND RUNOFF CALCULATIONS

Notes

Summary of Results

Description	Slope (ft/ft)	Slope Length (ft)	T_c (min)	Intensity (in/hr)	Runoff Coeff.	Unit Discharge (cfs/ft)
Entire Interbench Slope at 3H:1V	0.333	171	5.00	4.38	0.375	0.006

Entire Interbench Slope at 3H:1V

Design Geometry

54	Elevation Difference, ΔH (ft)	Calculated from design geometry, maximum slope used for analysis
0.333	Inter-Bench Slope, S (ft/ft)	Calculated from design geometry, maximum slope used for analysis
170.8	Slope Length, L (ft)	Calculated from design geometry, maximum slope length used for analysis

Time of Concentration

420	Ground Cover Coefficient, k (ft/min)	From Table 4.7.3 in WDOE (2004) for "short pasture grass or lawn"
0.704	Time of Concentration, T_c (min)	Calculated using equation on pg. 4-57 in WDOE (2004)
5.00	**Time of Concentration, T_c (min)**	Use minimum of 5 minutes (WDOE, 2004)

Unit Discharge

12.33	Rainfall Intensity Coefficient, m	From Table 4.7.2 in WDOE (2004) for 100-year MRI for Spokane
0.643	Rainfall Intensity Coefficient, n	From Table 4.7.2 in WDOE (2004) for 100-year MRI for Spokane
4.38	Rainfall Intensity (in/hr)	Calculated using equation on pg. 4-56 in WDOE, 2004
0.375	Runoff Coefficient, C	Value estimated from WDOE (2004) Table 4.7.1 for side slopes (turf) of 0.3, increased by 25% for 100-yr storm
0.006	**Unit Discharge, Q_{unit} (cfs/ft)**	Calculated using equation on pg. 4-56 in WDOE, 2004

Reference

Washington State Department of Ecology (WDOE), 2004. Stormwater Management Manual for Eastern Washington, Water Quality Program; September.

Client: Dawn Mining Company/Newmont USA Limited
Project: Midnite Mine
Detail: Remedial Action Cover Design - Cover Slopes, Sensitivity (Steepest interbench slope for 200' slope length)
Job No.: 1011322
Date: 5/21/2014
Calc. By: M.Davis
Checked By: J. Erickson

TIME OF CONCENTRATION AND RUNOFF CALCULATIONS

Notes

Summary of Results

Description	Slope (ft/ft)	Slope Length (ft)	T_c (min)	Intensity (in/hr)	Runoff Coeff.	Unit Discharge (cfs/ft)
Entire Interbench Slope at 2.5H:1V	0.400	200	5.00	4.38	0.375	0.008

Entire Interbench Slope at 2.5H:1V

Design Geometry

74	Elevation Difference, ΔH (ft)	Calculated from geometry
0.400	Inter-Bench Slope, S (ft/ft)	Back-calculated for minimum factor of safety of 1.0
200.0	Slope Length, L (ft)	Recommended maximum interbench slope length

Time of Concentration

420	Ground Cover Coefficient, k (ft/min)	From Table 4.7.3 in WDOE (2004) for "short pasture grass or lawn"
0.753	Time of Concentration, T_c (min)	Calculated using equation on pg. 4-57 in WDOE (2004)
5.00	**Time of Concentration, T_c (min)**	Use minimum of 5 minutes (WDOE, 2004)

Unit Discharge

12.33	Rainfall Intensity Coefficient, m	From Table 4.7.2 in WDOE (2004) for 100-year MRI for Spokane
0.643	Rainfall Intensity Coefficient, n	From Table 4.7.2 in WDOE (2004) for 100-year MRI for Spokane
4.38	Rainfall Intensity (in/hr)	Calculated using equation on pg. 4-56 in WDOE, 2004
0.375	Runoff Coefficient, C	Value estimated from WDOE (2004) Table 4.7.1 for side slopes (turf) of 0.3, increased by 25% for 100-yr storm
0.008	**Unit Discharge, Q_{unit} (cfs/ft)**	Calculated using equation on pg. 4-56 in WDOE, 2004

Reference

Washington State Department of Ecology (WDOE), 2004. Stormwater Management Manual for Eastern Washington, Water Quality Program; September.

Supplement D-6.2

Temple Method Erosion Protection Calculation Sheets

Client:	Dawn Mining Company/Newmont USA Limited
Project:	Midnite Mine
Detail:	Remedial Action Cover Design - Cover Slopes , Rhoads Property borrow
Job No.:	1011322
Date:	4/18/2014
Calc. By:	M.Davis
Checked By:	J. Erickson

TEMPLE METHOD FOR EROSION OF VEGETATED SLOPES

Notes

	100-yr, 24-hr design storm.	Per the requirements established in Appendix B of the Consent Decree (EPA, 2011)

Slope Geometry

Entire Interbench Slope at 3H:1V

54	Elevation Difference, H_0 (ft)	Design geometry, maximum
3	Original Inter-Bench Slope, (Xhoriz:1vert)	Design geometry
0.333	Original Inter-Bench Grade, S_0 (ft/ft)	Calculated from design geometry
18.4	Original Inter-Bench Slope Angle, θ_0 (deg)	Calculated from design geometry
162.0	Horizontal Distance from Toe to Crest of Inter-Bench Slope, X_0 (ft)	Calculated from design geometry
170.8	Original Slope Length, L_0 (ft)	Calculated from design geometry, longest slope used for analysis

Flow Characteristics

0.006	Unit discharge (cfs/ft)	Calculated using equation on pg. 4-56 in WDOE, 2004
3	Concentration Factor, F	As recommended in NUREG-1623 (NRC, 2002); Appendix A, Page A-7, Step 3.
0.019	Concentrated Design Flow, Q (cfs/ft)	Calculated per NUREG-1623 (NRC, 2002); Appendix A, Page A-7, Step 5.

Cover Soil Properties

18.0	Plasticity Index, PI	Calculated from Atterberg Limits analyses for Rhoads Property soils (MGC, 2011).
101.8	Dry Density (pcf)	Calculated from standard Proctor compaction tests for Rhoads Property soils (MGC, 2011) and assuming 85% compaction.
2.63	Specific Gravity	Calculated from specific gravity tests for Rhoads Property soils (MGC, 2011).
0.62	Void ratio	Calculated
0.0649	Diameter for which 75% of the Material is Finer, d_{75} (in)	Calculated from grain size analyses for Rhoads Property soils (MGC, 2011). Not required for cohesive soils.

Vegetation

1	Representative Stem Length, h_{stem} (ft)	Lower bound stem length for proposed cover vegetation (Redente, 2013)
200	Representative Stem Density, M_{stem} (stems/ft^2)	Temple et. al., 1987. page: 44 Table 3.1. Grass mixture with good coverage selected to represent established cover vegetation.
0.7	Cover Factor, C_f	Estimated cover factor after vegetation is established (Redente, 2013)

Other

62.4	Unit Weight of Water, γ_w (pcf)	

CALCULATIONS

6.05	Retardance Curve Index, C_i	Calculated per Temple et. al., 1987. Equation 1.3
4.53	Allowable Shear Stress on Vegetation, t_{va} (psf)	Calculated per Temple et. al., 1987. Equation 1.17
0.0652	base allowable tractive shear stress (psf) t_{ab} (psf)	Calculated per Temple et. al., 1987. Table 3.3, Page 46. For cohesive soils, SC with 10 <PI <20
1.04	void ratio correction factor, C_e	Calculated per Temple et. al., 1987. Table 3.3, Page 46. For cohesive soils, SC with 10 <PI <20
0.0711	Allowable Shear Stress on Soil of Vegetated Slope, t_a (psf)	Calculated per Temple et. al., 1987. Table 3.1, Page 43.
0.0156	Soil Grain Roughness, n_s	Estimated per Temple et. al., 1987. Table 3.3, Page 46. For cohesive soils with PI>10.
0.2247	Design Flow, q design (cfs/ft)	Use design flow if 0.0025 (C_i)^0.25 ≤ q ≤ 36 (Equation 4.8a, pg. 53), otherwise use upper or lower bound value (Temple et al., 1987)
0.2661	Manning's Coefficient for Vegetated Conditions, n	Calculated per Temple et. al., 1987. Equation 4.1a
0.202	Assumed Depth of Flow, d (ft)	Iterate d until q calculated equals q design

Client:	Dawn Mining Company/Newmont USA Limited
Project:	Midnite Mine
Detail:	Remedial Action Cover Design - Cover Slopes , Rhoads Property borrow
Job No.:	1011322
Date:	4/18/2014
Calc. By:	M.Davis
Checked By:	J. Erickson

TEMPLE METHOD FOR EROSION OF VEGETATED SLOPES

0.22	q calc (cfs/ft), with veg	Calculated per Manning's equation
0.00	qcalc - qdesign	Iterate d until q calculated equals q design
1.11	Average Flow Velocity, V (ft/sec)	Calculated as q/d
0.00434	Effective Stress on the Soil, t_e (psf)	Calculated per Temple et. al., 1987. Equation 4.3a
4.20	Effective Stress on the Vegetation, t_{ve} (psf)	Calculated per Temple et. al., 1987. Equation 4.9a
16.4	**Ratio of Allowable Stress on Soil to Effective Stress on Soil, FS_{soil}**	Calculated
1.1	**Ratio of Allowable Stress on Veg. to Effective Stress on Veg., FS_{veg}**	Calculated

References

Chow, Ven Te, 1959. Open-Channel Hydraulics. McGraw-Hill Publishing Company, New York.

Miller Geotechnical Consultants (MGC), 2011. Borrow Source Design investigation Report, Revision 2. Prepared on Behalf of: Newmont USA, Ltd. and Dawn Mining Company. Prepared for U.S. EPA. May 6, 2011.

Redente, E.F., 2013. Personal communications with Dr. Ed Redente, Redente Ecological Consultants, to Melanie Davis, MWH Americas, Inc., regarding vegetation parameters for proposed species for final cover. August 26.

Temple, D.M., K.M. Robinson, R.M. Ahring, and A.G. Davis. 1987. Stability Design of Grass-Lined Open Channels. U.S. Department of Agriculture, Agriculture Handbook 667.

U.S. Environmental Protection Agency (EPA), 2011. Statement of Work for the Remedial Design and Remedial Action for the Midnite Mine Superfund Site; Spokane Indian Reservation, Washington. August, 2011.

U.S. Nuclear Regulatory Commision (NRC), 2002. Design of Erosion Protection for Long-Term Stabilization; NUREG-1623. September 2002.

Washington State Department of Ecology (WDOE), 2004. Stormwater Management Manual for Eastern Washington, Water Quality Program; September.

Client:	Dawn Mining Company/Newmont USA Limited
Project:	Midnite Mine
Detail:	Remedial Action Cover Design - Cover Slopes, Sensitivity (Steepest interbench slope for 200' slope length)
Job No.:	1011322
Date:	5/21/2014
Calc. By:	M.Davis
Checked By:	J. Erickson

TEMPLE METHOD FOR EROSION OF VEGETATED SLOPES

Notes

	100-yr, 24-hr design storm.	Per the requirements established in Appendix B of the Consent Decree (EPA, 2011)

Slope Geometry

Entire Interbench Slope at 2.5H:1V

74	Elevation Difference, H_0 (ft)	Calculated from geometry
2.5	Inter-Bench Slope, (Xhoriz:1vert)	Back-calculated for minimum factor of safety of 1.0
0.400	Inter-Bench Grade, S_0 (ft/ft)	Calculated from geometry
21.8	Inter-Bench Slope Angle, θ_0 (deg)	Calculated from geometry
185.7	Horizontal Distance from Toe to Crest of Inter-Bench Slope, X_0 (ft)	Calculated from geometry
200.0	Slope Length, L_0 (ft)	Recommended maximum interbench slope length

Flow Characteristics

0.008	Unit discharge (cfs/ft)	Calculated using equation on pg. 4-56 in WDOE, 2004
3	Concentration Factor, F	As recommended in NUREG-1623 (NRC, 2002); Appendix A, Page A-7, Step 3.
0.023	Concentrated Design Flow, Q (cfs/ft)	Calculated per NUREG-1623 (NRC, 2002); Appendix A, Page A-7, Step 5.

Cover Soil Properties

18.0	Plasticity Index, PI	Calculated from Atterberg Limits analyses for Rhoads Property soils (MGC, 2011).
101.8	Dry Density (pcf)	Calculated from standard Proctor compaction tests for Rhoads Property soils (MGC, 2011) and assuming 85% compaction.
2.63	Specific Gravity	Calculated from specific gravity tests for Rhoads Property soils (MGC, 2011).
0.62	Void ratio	Calculated
0.0649	Diameter for which 75% of the Material is Finer, d_{75} (in)	Calculated from grain size analyses for Rhoads Property soils (MGC, 2011). Not required for cohesive soils.

Vegetation

1	Representative Stem Length, h_{stem} (ft)	Lower bound stem length for proposed cover vegetation (Redente, 2013)
200	Representative Stem Density, M_{stem} (stems/ft^2)	Temple et. al., 1987. page: 44 Table 3.1. Grass mixture with good coverage selected to represent established cover vegetation.
0.7	Cover Factor, C_f	Estimated cover factor after vegetation is established (Redente, 2013)

Other

62.4	Unit Weight of Water, γ_w (pcf)	

CALCULATIONS

6.05	Retardance Curve Index, C_i	Calculated per Temple et. al., 1987. Equation 1.3
4.53	Allowable Shear Stress on Vegetation, t_{va} (psf)	Calculated per Temple et. al., 1987. Equation 1.17
0.0652	base allowable tractive shear stress (psf) t_{ab} (psf)	Calculated per Temple et. al., 1987. Table 3.3, Page 46. For cohesive soils, SC with 10 <PI <20
1.04	void ratio correction factor, C_e	Calculated per Temple et. al., 1987. Table 3.3, Page 46. For cohesive soils, SC with 10 <PI <20
0.0711	Allowable Shear Stress on Soil of Vegetated Slope, t_a (psf)	Calculated per Temple et. al., 1987. Table 3.1, Page 43.
0.0156	Soil Grain Roughness, n_s	Estimated per Temple et. al., 1987. Table 3.3, Page 46. For cohesive soils with PI>10.
0.2247	Design Flow, q design (cfs/ft)	Use design flow if 0.0025 (C_i)^2.5 ≤ q ≤ 36 (Equation 4.8a, pg. 53), otherwise use upper or lower bound value (Temple et al., 1987)
0.2661	Manning's Coefficient for Vegetated Conditions, n	Calculated per Temple et. al., 1987. Equation 4.1a
0.191	Assumed Depth of Flow, d (ft)	Iterate d until q calculated equals q design

Client:	Dawn Mining Company/Newmont USA Limited
Project:	Midnite Mine
Detail:	Remedial Action Cover Design - Cover Slopes, Sensitivity (Steepest interbench slope for 200' slope length)
Job No.:	1011322
Date:	5/21/2014
Calc. By:	M.Davis
Checked By:	J. Erickson

TEMPLE METHOD FOR EROSION OF VEGETATED SLOPES

0.22	q calc (cfs/ft), with veg	Calculated per Manning's equation
0.00	qcalc - qdesign	Iterate d until q calculated equals q design
1.17	Average Flow Velocity, V (ft/sec)	Calculated as q/d
0.00467	Effective Stress on the Soil, t_e (psf)	Calculated per Temple et. al., 1987. Equation 4.3a
4.52	Effective Stress on the Vegetation, t_{ve} (psf)	Calculated per Temple et. al., 1987. Equation 4.9a
14.4	**Ratio of Allowable Stress on Soil to Effective Stress on Soil, FS_{soil}**	Calculated
1.0	**Ratio of Allowable Stress on Veg. to Effective Stress on Veg., FS_{veg}**	Calculated

References

Chow, Ven Te, 1959. Open-Channel Hydraulics. McGraw-Hill Publishing Company, New York.

Miller Geotechnical Consultants (MGC), 2011. Borrow Source Design investigation Report, Revision 2. Prepared on Behalf of: Newmont USA, Ltd. and Dawn Mining Company. Prepared for U.S. EPA. May 6, 2011.

Redente, E.F., 2013. Personal communications with Dr. Ed Redente, Redente Ecological Consultants, to Melanie Davis, MWH Americas, Inc., regarding vegetation parameters for proposed species for final cover. August 26.

Temple, D.M., K.M. Robinson, R.M. Ahring, and A.G. Davis. 1987. Stability Design of Grass-Lined Open Channels. U.S. Department of Agriculture, Agriculture Handbook 667.

U.S. Environmental Protection Agency (EPA), 2011. Statement of Work for the Remedial Design and Remedial Action for the Midnite Mine Superfund Site; Spokane Indian Reservation, Washington. August, 2011.

U.S. Nuclear Regulatory Commision (NRC), 2002. Design of Erosion Protection for Long-Term Stabilization; NUREG-1623. September 2002.

Washington State Department of Ecology (WDOE), 2004. Stormwater Management Manual for Eastern Washington, Water Quality Program; September.

Supplement D-6.3

Average Depth of Erosion Calculation Sheet

Client: Dawn Mining Company/Newmont USA Limited
Project: Midnite Mine
Detail: Remedial Action Cover Design - Cover Slopes, Rhoads Property borrow
Job No.: 1011322
Date: 10/9/2013
Author: M.Davis
Checked: J. Cumbers

CALCULATION OF AVERAGE DEPTH OF EROSION

Assumptions:

Cover material originates from Rhoads Property Borrow Site (Sandy Loam with Low-Moderate Organic Material)
Unit soil loss (ton/acre/year) for each ground cover type is calculated from Rusle2 program using parameters presented in calculation brief.
110 Unit Weight of in-place cover material (pcf), calculated from standard Proctor compaction tests for Rhoads Property soils (MGC, 2011) and assuming 85% compaction at insitu water content.
Assume 2 years of bare ground with rough surface, 8 years of Cool Season Grass (poor stand) for 10-yr soil loss
Assume 2 years of bare ground with rough surface, 998 years of Cool Season Grass (poor stand) for 1000-yr soil loss

Average Depth of Erosion for 3H:1V Interbench Slope

Ground Cover	Unit Soil Loss (t/ac/yr)	Unit Soil Loss (lb/ac/yr)	Unit Soil Loss (lb/sf/yr)	Average Depth of Yearly Erosion (in/yr)
Bare Ground	14.00	28,000	0.6428	0.0701
Cool Season Grass (Poor Stand)	0.12	240	0.0055	0.0006

Soil Loss Time Period	Unit Soil Loss (t/ac)	Unit Soil Loss (lb/ac)	Unit Soil Loss (lb/sf)	Average Depth of Erosion (in)
10-yr Soil Loss	28.96	57,920	1.33	**0.15**
1000-yr Soil Loss	148	295,520	6.78	**0.74**

Reference:

Miller Geotechnical Consultants (MGC), 2011. *Borrow Source Design investigation Report, Revision 2* . Prepared on Behalf of: Newmont USA, Ltd. and Dawn Mining Company. Prepared for U.S. EPA. May 6, 2011.
United States Department of Agriculture, Natural Resources Conservation Service (USDA-NRCS), 2006. RUSLE 2 program. Version 1.26.6.4. November 13.

Attachment D-7

Cover System Veneer Slope Stability Analyses

ATTACHMENT D-7

MIDNITE MINE REMEDIAL ACTION COVER SYSTEM VENEER SLOPE STABILITY ANALYSIS

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*15-Nov-12*	*30% Design*	*Jean-Baptiste Varnier*	*Melanie Davis*	*21-Nov-12*	*Tom Kelley*
1	*26-Nov-13*	*60% Design*	*Jean-Baptiste Varnier*	*Melanie Davis*	*05-Dec-13*	*Tom Kelley*
2	*23-May-14*	*90% Design*	*Jean-Baptiste Varnier*	*Melanie Davis*	*18-Jul-14*	*Tom Kelley*
2	*14-May-15*	*100% Design – no changes*				

Location and Format
Electronic copies of these calculations are located in the project files system at: *\\usftc2s01\Projects\Newmont\Midnite Mine_2011\6.0 Studies & Reports\6.2 Technical\Remedial Action Cover Design\Cover Stability\60% Design* *The following calculations were generated using the following software:* *Microsoft Office Professional Plus 2010 (Microsoft Excel)*

Supplement
Supplement D-7.1 – Veneer Cover Soil Stability Calculations *Supplement D-7.2 – Veneer Cover Soil Stability Sensitivity Analysis Calculations* *Supplement D-7.3 – Laboratory Testing Results*

1.0 BACKGROUND

This calculation brief presents the methods, input and results of slope stability (veneer) analyses that were conducted for the cover system that will be part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The cover system will be placed over the mine waste that will be consolidated in Pit 3 (including the Backfilled Pit Area, or BPA) and Pit 4, and over the regraded surface of Area 5 (area between Pits 3 and 4). These analyses were conducted according to applicable stability criteria as required by Appendix B to the Remedial Design/Remedial Action (RD/RA) Consent Decree (EPA, 2011) under static and seismic loading conditions.

The proposed cover system will consist of a uniform soil layer overlying a synthetic geomembrane. On steeply sloped areas (greater than 15 percent slopes), the cover system also will include a geocomposite drainage layer (GDL) between the soil and geomembrane layers. The GDL layer was added based on results of veneer stability analyses that indicate that stability is not satisfactory for steeper slopes if significant positive pore pressure develops above the geomembrane. Therefore, a drainage layer will need to be placed above the geomembrane to prevent water build up on the geomembrane. For these analyses, the GDL is assumed to be a non-woven, needle-punched geotextile heat-bonded to both the top and bottom faces of a geonet. Three types of geomembranes were evaluated for the cover system and include: 1) 40 mil polyvinyl chloride (PVC) with a faille top surface; 2) 40 mil linear low-density polyethylene (LLDPE) with a textured top surface; and 3) 40 mil LLDPE with spikes on the underside of the liner and drainage studs on the upper side of the liner (Agru Super Gripnet Liner).

Material from the Rhoads Property Borrow Area located adjacent to the site is analyzed as the soil cover borrow material. The predominant material above bedrock in this borrow source consists of clayey sand (MGC, 2011a, b). The upper layer of bedrock consists of weathered quartz monzonite which is weathered to a residual soil (residuum). It is assumed that both materials may be used for the cover system. A minimum thickness of 1.8 feet is required for this cover soil based on radon emanation modeling results (refer to Attachment D-3); however a minimum soil cover thickness of 3.0 feet has been proposed based on construction and slope stability considerations.

The preliminary design for the cover system called for the top 0.5 feet of the soil cover to be topsoil or amended borrow soil to promote vegetation growth. However, based upon data presented in the Borrow Source Design Investigation Report (MGC, 2011a), it does not appear that amendment of soil from the Rhoads Property Borrow Area will be necessary to promote vegetation growth (see the Revegetation Plan, Attachment D-12). Therefore, for the purposes of these analyses, the top 0.5 feet of the cover system is assumed to have similar material properties as the underlying soil cover material. Additional information on the borrow source is provided in MGC (2011a, b).

2.0 DESCRIPTION OF VENEER STABILITY ANALYSES

2.1 GENERAL

The veneer stability analyses were performed using an analytical limit-equilibrium solution for finite-length cover slopes underlain by geosynthetics proposed by Koerner and Soong (1998, 2005). This method of stability analysis assumes a failure along planar weak interfaces in the cover system. The factor of safety is calculated from the forces acting on the active and the passive wedge. Figure 1 shows the forces used in the limit equilibrium analyses under static and pseudo-static loading conditions and assuming no positive pore pressure development above the geomembrane. Figure 2 shows forces used in limit equilibrium analyses for static conditions assuming positive pore pressure development above the geomembrane.

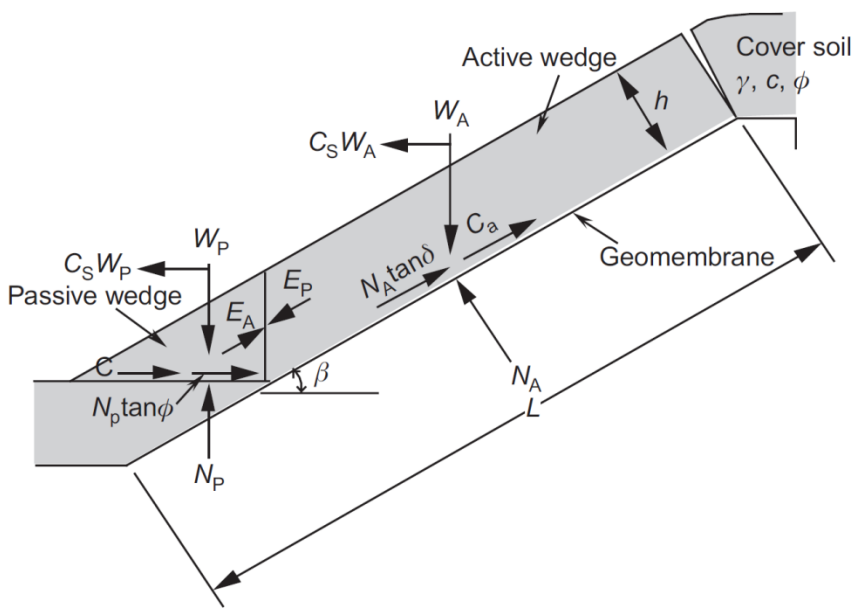


Figure 1. Limit equilibrium forces involved in static (C_s=0) and pseudo-static analysis from Koerner and Soong, 2005.

The parameters shown in Figure 1 are defined as follows:

L, length of slope measured along the geomembrane
h, total cover soil thickness
γ, cover soil unit weight of the soil
ϕ, cover soil friction angle
c, cover soil cohesion
δ, interface friction angle between the geonet and the PVC geomembrane
β, slope angle
C, cohesive force along failure plane of the passive wedge
C_a, adhesive force between the geonet and the PVC geomembrane
N_A, effective force normal to the failure plane of the active wedge
N_p, effective force normal to the failure plane of the passive wedge
W_A, total weight of the active wedge

W_p, total weight of the passive wedge
E_A, interwedge force acting on the active wedge from the passive wedge
E_p, interwedge force acting on the passive wedge from the active wedge
C_s, horizontal seismic coefficient

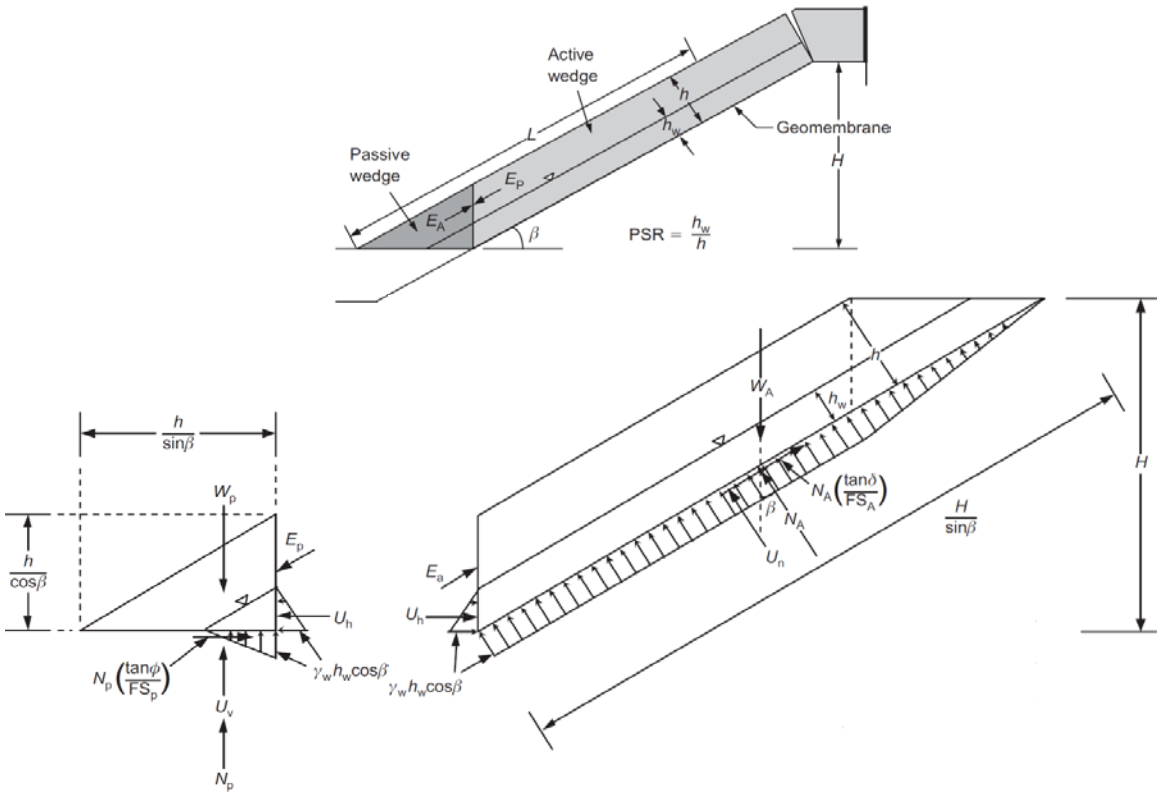


Figure 2. Limit equilibrium forces involved in finite-length slope of uniform cover soil with parallel-to-slope seepage build-up for the active and passive wedges after Koerner and Soong, 2005.

The parameters shown in Figure 2 are defined as follows:

H, vertical height of the slope measured from the toe
h, total cover soil thickness
h_w, height of free water surface measured in the direction perpendicular to the slope
γ_w, unit weight of water
δ, interface friction angle between the geomembrane and cover soil
β, slope angle
U_h, resultant of the pore pressures acting on the interwedge surfaces
U_n, resultant of the pore pressures acting perpendicular to the slope
U_v, resultant of the vertical pore pressures acting on the passive wedge.
N_A, effective force normal to the failure plane of the active wedge
N_p, effective force normal to the failure plane of the passive wedge
W_A, total weight of the active wedge
W_p, total weight of the passive wedge

The equation for factor of safety is:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

where a, b and c for drained conditions are defined as follows:

$$a = (C_s W_A - N_A sin\beta)cos\beta + C_s W_p cos\beta$$

$$b = -[C_s W_A + N_A sin\beta)sin\beta tan\phi + (N_A tan\delta + C_a)cos^2\beta + (C + W_p tan\phi)cos\beta]$$

$$c = (N_A tan\delta + C_a)cos\beta\ sin\beta tan\phi$$

where, a, b and c for saturated conditions are defined as follows:

$$a = W_A sin\beta\ cos\beta - U_h cos^2\beta + U_h$$

$$b = -W_A sin^2\beta\ tan\phi + U_h sin\beta\ cos\beta\ tan\phi - N_A cos\beta\ tan\delta - \left(W_p - U_v\right)tan\phi$$

$$c = N_A\ sin\beta\ tan\delta tan\phi$$

2.2 MODEL GEOMETRY AND SITE-SPECIFIC CONDITIONS

Based upon the detailed grading plan developed as part of the 60% design, the steepest cover interbench slopes vary from 3 horizontal to 1 vertical (3H:1V) to 4H:1V, with horizontal lengths of approximately 150 and 200 feet, respectively. Top cover slopes will be graded to less than 15 percent, as well as some select areas of the interbench slopes. The cover soil thickness of 3.0 feet (ft) was used in these analyses. Analyses were performed for: 1) drained conditions under static and pseudo-static loading, and (2) saturated conditions under static loading. The two geomembrane materials described in Section 1.0 (PVC and LLDPE – 2 configurations) were considered. For drained conditions, the GDL is assumed to have adequate capacity to preclude the development of positive pore pressures on the geomembrane liner and within the cover soil. The longest 3H:1V interbench cover slope was selected for evaluating the drained conditions. For saturated conditions, the slope angle was back calculated to achieve a factor of safety of 1.3 under static conditions assuming the cover soil is fully saturated. The longest cover slope length was used in the analysis. The back calculation of the slope angle for saturated conditions was used to estimate the slope angle at which the slope would become unstable if a GDL layer is not included and the soil cover becomes fully saturated.

The failure (sliding) surface is assumed to occur along the weakest interface, which corresponds to the surface with the lowest interface shear strength. Consequently, the failure surface is assumed to occur along the GDL and geomembrane interface based upon the previous test results (MGC, 2011a) and experience on other projects that indicate the interface strengths for the mine soil-geosynthetic interfaces will be higher than for the GDL-geomembrane interface. This assumption is discussed further in the next section.

The analyses conservatively neglect the tensile strength of the GDL that would, in reality, be mobilized if sliding were to initiate along the GDL-geomembrane interface.

2.3 MATERIAL PROPERTIES

Strength parameters for the Rhoads Property Borrow Area cover soil were based upon the results of laboratory testing performed for and reported in the *Borrow Source Design Investigation Report* (MGC, 2011a). The friction angle used in the analyses corresponds to the average values obtained from interpretation of the laboratory tests. The density value of the cover soil used in the calculations is based on laboratory testing results (MGC, 2011a, b). The in-placed density value was estimated based on the:

- Average laboratory measured maximum standard Proctor dry densities (rock correction applied where applicable)
- Average measured in-situ water content for samples from the borrow area, and
- Assuming the cover materials will be placed at approximately 85 percent of standard Proctor maximum dry density.

As mentioned previously, the sliding surface is assumed to occur along the GDL and geomembrane interface using the Koerner and Soong (1998, 2005) method of slope failure analysis. This assumption was based upon review of interface shear strength test results performed for the 60% Design, for previous borrow investigations (MGC, 2011a), from published values (Koerner and Narejo, 2005), and on prior experience on other projects that indicate the GDL-geomembrane contact will be the weakest interface in the cover system and the location where shear failure will be initiated. Although literature values are assumed to be reasonably conservative, interface strength testing using site-specific soils and geosynthetics being considered for use in cover construction was conducted to determine geosynthetic interface strengths. These test results are discussed in Section 3.0.

For the veneer stability analysis under drained conditions, the GDL to geomembrane peak and post-peak shear strength (in terms of interface friction angles) needed to achieve factors of minimum factors of safety of 1.3 and 1.0 for static and pseudo-static loading conditions, respectively, were back-calculated. This value represents the minimum required peak interface friction angle for the geomembrane to GDL interface for the cover system.

For the analysis of maximum stable slopes without a GDL under saturated conditions, the peak interface friction angle for the cover soil and a LLDPE geomembrane was used. The LLDPE had the lowest soil/geomembrane interface strength of the geomembranes under consideration (MGC, 2011a) and the results can be considered conservative if applied to a PVC geomembrane.

The properties used for the analyses are summarized in Table 1.

Table 1. Material Properties

Material	Value
Cover Soil Thickness, h_{cs}	3.00 ft
Cover Soil Density, d_{cs}	110 pcf
Cover Soil Saturated Density, $d_{cs,sat}$	125.5 pcf
Cover Soil Peak Friction Angle, ϕ_{cs}	35.2°
Cover Soil Post-Peak Friction Angle, ϕ_{cs}	33.4°
Cover Soil Cohesion, c_{cs}	0 psf
Cover Soil/LLDPE Geomembrane Peak Friction Angle, δ (saturated static loading conditions without GDL)	22.7°
GDL/Geomembrane Interface Peak Friction Angle, δ (drained static loading conditions)	Back-calculated for FS = 1.3
GDL/Geomembrane Interface Post-Peak Friction Angle, δ (drained pseudo-static loading conditions)	Back-calculated for FS = 1.0
Adhesion between GDL/Geomembrane, c_a	0 psf

pcf =pounds per cubic foot, psf = pounds per square foot

2.4 SEISMIC LOADING

The veneer stability analyses under seismic loading conditions were conducted using a pseudo-static approach, where a horizontal inertial force is applied to the numerical models. This horizontal inertial force simulates accelerations due to a seismic event. The peak ground acceleration (PGA) for a probability of exceedance of 10 percent in 250 years at the Site for site Class B (rock) is 0.131g (MGC, 2010), where g is gravity. The PGA at the Site was adjusted to site Class C (very dense soil and soft rock) to represent waste rock for the veneer stability analyses. The recommendations presented in USGS (2013) were used to calculate the adjusted PGA. The Class C PGA at the Site is 0.157g. A peak ground acceleration value greater than the 0.1g requires seismic forces be considered for the slope stability analysis of cover soils (EPA, 2004). EPA (2004) lists the use of the pseudo-static factor of safety method as acceptable and conservative method to evaluate seismic forces.

The procedure of representing the seismic coefficient as a function of the PGA has been adopted in review of uranium facility design and documented by the U.S. Department of Energy (DOE, 1989). DOE (1989) recommends the use of a horizontal seismic coefficient of $^{2}/_{3g}$ of the PGA for pseudo-static stability analyses for post-reclamation conditions. The U.S. Nuclear Regulatory Commission (NRC, 2003) recommends the seismic coefficient be either 67 percent of the PGA or 0.1, whichever is greater. The horizontal seismic coefficient used for the pseudo-static stability analyses is 0.105.

3.0 RESULTS

Results of the veneer stability analyses for drained and undrained conditions are summarized in Sections 3.1 and 3.2, respectively, and calculations sheets are provided in Supplement D-7.1.

3.1 DRAINED CONDITIONS

The required minimum factors of safety are 1.3 under static conditions and 1.0 under pseudo-static conditions as outlined in EPA (2011). For drained conditions, the minimum interface frictions angles required to meet the factor of safety requirements are 22.0° and 23.0° static and pseudo-static conditions, respectively. The required interface friction angles were calculated for the most critical design slope (i.e., longest), with an unbroken 3H:1V slope length of 171ft.

Table 2. Factors of Safety for Veneer Stability for Drained Conditions under Static and Pseudo-Static Loading Conditions

Failure Surface	Back-Calculated Minimum Required Interface Friction Angle	
	Peak (used for static loading conditions, FS = 1.3)	Post-Peak (used for pseudo-static loading conditions, FS = 1.0)
GDL to Geomembrane Interface	22.0	23.0

In addition, a sensitivity analysis was conducted to identify the required interface friction angle that would achieve a minimum factor of safety of 1.3 and 1.0 for static and pseudo-static loading conditions, respectively, for a maximum cover slope length of 200 ft. Slopes ranging from 2.7H:1V to 3H:1V were used in the analyses. Calculation sheets are provided in Supplement D-7.2 and analysis results are presented in Figure 3.

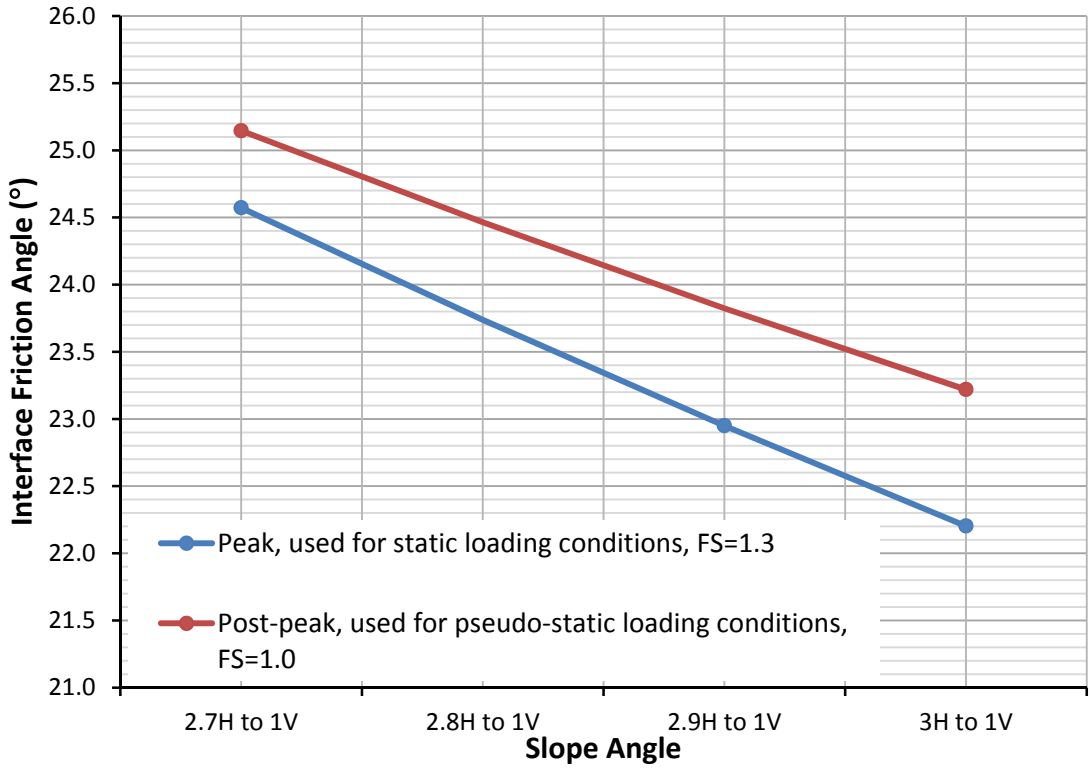


Figure 3. Minimum Required Interface Friction Angle for Veneer Stability for Drained Conditions under Static and Pseudo-Static Loading Conditions (Slope Length = 200ft)

3.2 UNDRAINED (NO GDL) CONDITIONS

For saturated conditions, the maximum slope angle to achieve a factor of safety of 1.3 for under static loading is approximately 9 degrees (16.6 percent). Areas steeper than 15 percent will include a GDL to prevent pore water buildup and increase slope stability. Top cover slopes which have flatter slopes (less than 15 percent) will not require a GDL except for the top cover slope of Pit 3 where there is a drainage swale and concentrated flows may occur. Cover areas designated to have GDL are shown in Figures 2 and 3 of Attachment D-4.

Table 3. Back-Calculated Cover Slope Angle for Saturated Conditions under Static Loading Condition

Failure Surface	Factor of Safety	Longest Cover Slope Length	Back-Calculated Maximum Slope Angle
GDL to Geomembrane Interface	1.3	721 ft	9.4° (6.0H:1V)

3.3 GDL/GEOMEMBRANE INTERFACE TESTING

GDL/Geomembrane interface strength testing has been conducted as part of the 60% design to measure interface strength parameters for the specific materials being considered for construction. The results of the testing are summarized in Table 4 and the laboratory data is attached to this calculation brief as Supplement D-7.3.

Table 4. Preliminary Results for Measured Interface Friction Angles for Materials

Failure Surface	Friction Angle	
	Peak	Post-Peak
PVC/Single Fabric-Sided Geocomposite	18.3	16.3
PVC/Double Fabric-Sided Geocomposite	21.0	21.6
Textured LLDPE/Single Fabric-Sided Geocomposite	18.5	15.9
Textured LLDPE/Double Fabric-Sided Geocomposite	34.0	25.4

The geosynthetic cover materials used for testing were selected in part based on the minimum interface friction angles required to meet the minimum factors of safety under drained conditions. Based on the testing results, the interface friction angle for a textured LLDPE geomembrane/double fabric-sided drainage geocomposite is above the minimum interface friction angle required to meet veneer stability criteria for the most critical design slope (i.e., the longest continuous slope). In addition, the textured LLDPE/Double fabric-sided GDL interface friction is sufficient to meet veneer stability criteria on slopes as steep as 2.7H:1V with a maximum continuous slope length of 200 feet (see results of sensitivity analysis presented Figure 3). The other potential combinations of GDL and geomembranes listed in Table 4 did not meet the veneer stability criteria.

Testing was also conducted for the interface between a 40 mil LLDPE Agru Super Gripnet Liner and geotextile. Test results are provided in Supplement D-7.3 and show a relatively high adhesion value for peak and post-peak conditions. The test results indicate a highly non-linear interface failure envelope, which is likely the result of mechanical interlocking between the overlying fabric and the very rough Agru Super Gripnet geomembrane surface. This interface cannot be accurately characterized using a simple linear (frictional) failure envelope model without adhesion and thus are not included in the summary in Table 4. It may be unconservative to interpret this interface with a linear failure envelop model that includes both friction (δ) and adhesion (c_a) and the most prudent way to characterize this material interface would be to model the interface strength using a non-linear failure envelop. Regardless, the measured shear strength of the Agru Super Gripnet Liner/geotextile interface exceeded the required interface shear strength (see Table 2) over the range of normal loadings that can be expected for the cover system for both peak and post-peak conditions. The shear strength comparison is provided in Supplement D-7.3. The results indicate that a 40 mil LLDPE Agru Super Gripnet Liner to geotextile interface will meet the minimum shear strength requirements for veneer stability for a 3-ft cover loading.

In conclusion, it appears that either textured LLDPE geomembrane used in conjunction with a double-fabric-faced GDL, or the Agru Super Gripnet LLDPE Geomembrane used with a geofabric drainage facing will result in a cover system with acceptable veneer stability. The other geomembrane/GDL combinations tested do not meet the veneer stability requirements. As discussed previously, these analyses conservatively neglected the tensile strength of the GDL. The tensile strength of the GDL will, in reality, provide additional resisting forces in the cover system and further increase the veneer stability.

4.0 REFERENCES

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Miller Geotechnical Consultants (MGC), 2011b. Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Revision 1. Prepared on behalf of Newmont USA Limited and Dawn Mining Company for U.S. Environmental Protection Agency, Region 10. December 2.

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Supplement D-7.1

Veneer Cover Soil Stability Calculations

MWH	By: Jean-Baptiste Varnier	Date: 10/29/2013	Project: Midnite Mine	Sheet 1 of 2
	Checked By: Melanie Davis	Description: Rhoads Property Borrow under Static Conditions - 3H to 1V slope		Job No. : 1011322

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49

(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.

(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.

(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.

(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.

(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	170.76	ft	Design (maximum length for 3H:1V)
	L	52.05	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
	h_{cs}	0.91	m	
Slope ratio	S_r	3	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	35.2	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	0.614	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	22.0	degrees	Backcalculated for FS = 1.3
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.322	rad	ATAN(1/S_r)
	β	18.4	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	778	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	738	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	24	kN/m	Equation 8 of reference (1)

Factor of Safety	FOS	1.30	
	a	73.8	kN/m
	b	-112.1	kN/m
	c	21.0	kN/m

MWH	By: Jean-Baptiste Varnier	Date: 10/29/2013	Project: Midnite Mine	Sheet 2 of 2
	Checked By: Melanie Davis	Description: Rhoads Property Borrow under Pseudo-Static Conditions - 3H to 1V slope		Job No. : 1011322

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49
(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.
(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.
(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.
(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.
(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.
(7) U.S. Geological Survey (USGS), 2013. U.S. Seismic Design Maps Application. Accessed on October 22, 2013 from http://geohazards.usgs.gov/designmaps/us/application.php

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	170.76	ft	Design (maximum length for 3H:1V)
		52.05	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
		0.91	m	
Slope ratio	S_r	3	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	33.4	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	*0.583*	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	23.0	degrees	Backcalculated for FS = 1.0
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed
Seismic				
Peak Ground Acceleration	PGA	0.1572	g	From reference (4) adjusted for waste rock using reference (7)

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.322	rad	ATAN(1/S_r)
	β	18.4	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	778	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	738	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	24	kN/m	Equation 8 of reference (1)
Seismic				
Horizontal seismic coefficient	C_s	0.10	-	2/3*PGA/g , minimum 0.1 - references (5) and (6)
Factor of Safety	**FOS**	**1.00**		
	a	301.1	kN/m	
	b	-363.1	kN/m	
	c	62.1	kN/m	

MWH	By: Jean-Baptiste Varnier	Date: 11/26/2013	Project: Midnite Mine	Sheet 1 of 1
	Checked By: Melanie Davis	Description: Cover Soil Stability - Rhoads Property Borrow under Saturated and Static Conditions		Job No. : 1011322

Cover Materials:

Cover soil overlying geomembrane liner.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49

(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.

(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	721.0	ft	Longest slope length from design
	L	219.76	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
	h_{cs}	0.91	m	
Slope angle	β	9.4	degrees	Calculated for FS = 1.30
	β	0.164	rad	
Soil and Geocomposite Properties				
Parellel Submergence Ratio	PSR	1		Assumed fully Saturated
Maximum dry density	$d_{max,dry}$	119.7	pcf	From test results - references (2) and (3)
		18.81	kN/m^3	
Cover soil dry density	$d_{cs,\ dry}$	101.7	pcf	Calculated as 85% of average maximum standard Proctor density
	$d_{cs,\ dry}$	15.99	kN/m^3	
Porosity		0.38		Calculated from test results - reference (2)
Water density	d_{water}	62.4	pcf	
		9.81	kN/m^3	
Cover soil saturated density	$d_{cs,\ sat}$	125.5	pcf	
	$d_{cs,\ sat}$	19.71	kN/m^3	
Cover soil friction angle	ϕ_{cs}	35.2	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	0.614	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^3	
Interface friction angle	δ	22.7	degrees	Average peak interface friction angle for LLDPE and Rhoads Property Borrow from test results - reference (2)
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^3	Assumed

Calculated Values	Variable	Value	Units	Source
Slope				
Height of free water surface	h_w	0.91	m	$PSR*h_{cs}$
Slope ratio	S_r	6.0	to 1	
Height of the Slope	H	35.94	m	$L*sin(\beta)$
Soil and Geocomposite				
Active wedge weight	W_A	3910.5	kN/m	Equation 32 of reference (1)
Resultant of pore pressures acting perpendicular to the slope	U_n	1918.8	kN/m	Equation 33 of reference (1)
Resultant of pore pressures acting on the interwedge surface	U_h	4.1	kN/m	Equation 34 of reference (1)
Resultant vertical pore pressure acting on the passive wedge	U_v	24.7	kN/m	Equation 29 of reference (1)
Effective normal force of the active wedge	N_A	1939.7	kN/m	Equation 26 of reference (1)
Passive wedge weight	W_P	51.1	kN/m	Equation 35 of reference (1)
Factor of Safety	**FOS**	**1.30**		Equation 15 of reference (1)
	a	631.0	kN/m	Equation 31 of reference (1)
	b	-892.3	kN/m	Equation 31 of reference (1)
	c	93.6	kN/m	Equation 31 of reference (1)

Supplement D-7.2

Veneer Cover Soil Stability Sensitivity Analysis Calculations

MWH	By: Jean-Baptiste Varnier	Date: 5/30/2014	Project: Midnite Mine	Sheet 1 of 2
	Checked By: Melanie Davis	Description: Sensitivity Analysis, Static Conditions - 2.7H to 1V slope and 200 ft slope length		Job No. : 1011322

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49

(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.

(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.

(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.

(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.

(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	200.00	ft	Design (maximum length for 3H:1V)
	L	60.96	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
	h_{cs}	0.91	m	
Slope ratio	S_r	2.7	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	35.2	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	0.614	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	24.6	degrees	Backcalculated for FS = 1.3
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.355	rad	ATAN(1/S_r)
	β	20.3	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	923	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	866	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	22	kN/m	Equation 8 of reference (1)

Factor of Safety	FOS	1.30	
	a	104.4	kN/m
	b	-161.6	kN/m
	c	33.7	kN/m

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49
(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.
(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.
(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.
(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.
(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.
(7) U.S. Geological Survey (USGS), 2013. U.S. Seismic Design Maps Application. Accessed on October 22, 2013 from http://geohazards.usgs.gov/designmaps/us/application.php

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	200.00	ft	Design (maximum length for 3H:1V)
		60.96	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
		0.91	m	
Slope ratio	S_r	2.7	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	33.4	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	*0.583*	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	25.1	degrees	Backcalculated for FS = 1.0
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed
Seismic				
Peak Ground Acceleration	PGA	0.1572	g	From reference (4) adjusted for waste rock using reference (7)

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.355	rad	ATAN(1/S_r)
	β	20.3	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	923	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	866	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	22	kN/m	Equation 8 of reference (1)
Seismic				
Horizontal seismic coefficient	C_s	0.10	-	2/3*PGA/g, minimum 0.1 - references (5) and (6)
Factor of Safety	**FOS**	**1.00**		
	a	374.8	kN/m	
	b	-462.1	kN/m	
	c	87.3	kN/m	

MWH	By: Jean-Baptiste Varnier	Date: 5/30/2014	Project: Midnite Mine	Sheet 1 of 2
	Checked By: Melanie Davis	Description: Sensitivity Analysis, Static Conditions - 2.8H to 1V slope and 200 ft slope length		Job No. : 1011322

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49

(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.

(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.

(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.

(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.

(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	200.00	ft	Design (maximum length for 3H:1V)
	L	60.96	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
	h_{cs}	0.91	m	
Slope ratio	S_r	2.8	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	35.2	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	0.614	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	23.7	degrees	Backcalculated for FS = 1.3
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.343	rad	ATAN(1/S_r)
	β	19.7	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	922	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	868	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	23	kN/m	Equation 8 of reference (1)

Factor of Safety	FOS	1.30	
	a	98.2	kN/m
	b	-151.1	kN/m
	c	30.5	kN/m

MWH	By: Jean-Baptiste Varnier	Date: 5/30/2014	Project: Midnite Mine	Sheet 2 of 2
	Checked By: Melanie Davis	Description: Sensitivity Analysis, Pseudo-Static Conditions - 2.8H to 1V slope and 200 ft slope length		Job No. : 1011322

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49
(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.
(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.
(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.
(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.
(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.
(7) U.S. Geological Survey (USGS), 2013. U.S. Seismic Design Maps Application. Accessed on October 22, 2013 from http://geohazards.usgs.gov/designmaps/us/application.php

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	200.00	ft	Design (maximum length for 3H:1V)
		60.96	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
		0.91	m	
Slope ratio	S_r	2.8	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	33.4	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	*0.583*	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	24.5	degrees	Backcalculated for FS = 1.0
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed
Seismic				
Peak Ground Acceleration	PGA	0.1572	g	From reference (4) adjusted for waste rock using reference (7)

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.343	rad	ATAN(1/S_r)
	β	19.7	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	922	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	868	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	23	kN/m	Equation 8 of reference (1)
Seismic				
Horizontal seismic coefficient	C_s	0.10	-	2/3*PGA/g, minimum 0.1 - references (5) and (6)
Factor of Safety	**FOS**	**1.00**		
	a	368.2	kN/m	
	b	-450.7	kN/m	
	c	82.5	kN/m	

MWH	By: Jean-Baptiste Varnier	Date: 5/30/2014	Project: Midnite Mine	Sheet 1 of 2
	Checked By: Melanie Davis	Description: Sensitivity Analysis, Static Conditions - 2.9H to 1V slope and 200 ft slope length		Job No. : 1011322

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49

(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.

(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.

(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.

(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.

(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	200.00	ft	Design (maximum length for 3H:1V)
	L	60.96	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
	h_{cs}	0.91	m	
Slope ratio	S_r	2.9	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	35.2	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	0.614	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	22.9	degrees	Backcalculated for FS = 1.3
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.332	rad	ATAN(1/S_r)
	β	19.0	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	920	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	870	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	24	kN/m	Equation 8 of reference (1)

Factor of Safety	FOS	1.30	
	a	92.5	kN/m
	b	-141.5	kN/m
	c	27.6	kN/m

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49
(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.
(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.
(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.
(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.
(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.
(7) U.S. Geological Survey (USGS), 2013. U.S. Seismic Design Maps Application. Accessed on October 22, 2013 from http://geohazards.usgs.gov/designmaps/us/application.php

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	200.00	ft	Design (maximum length for 3H:1V)
		60.96	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
		0.91	m	
Slope ratio	S_r	2.9	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	33.4	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	*0.583*	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	23.8	degrees	Backcalculated for FS = 1.0
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed
Seismic				
Peak Ground Acceleration	PGA	0.1572	g	From reference (4) adjusted for waste rock using reference (7)

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.332	rad	ATAN(1/S_r)
	β	19.0	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	920	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	870	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	24	kN/m	Equation 8 of reference (1)
Seismic				
Horizontal seismic coefficient	C_s	0.10	-	2/3*PGA/g, minimum 0.1 - references (5) and (6)
Factor of Safety	**FOS**	**1.00**		
	a	361.7	kN/m	
	b	-439.8	kN/m	
	c	78.1	kN/m	

MWH	By: Jean-Baptiste Varnier	Date: 10/29/2013	Project: Midnite Mine	Sheet 1 of 2
	Checked By: Melanie Davis	Description: Sensitivity Analysis, Static Conditions - 3H to 1V slope and 200 ft slope length		Job No. : 1011322

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49

(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.

(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.

(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.

(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.

(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	200.00	ft	Design (maximum length for 3H:1V)
	L	60.96	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
	h_{cs}	0.91	m	
Slope ratio	S_r	3	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	35.2	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	0.614	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	22.2	degrees	Backcalculated for FS = 1.3
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.322	rad	ATAN(1/S_r)
	β	18.4	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	919	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	872	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	24	kN/m	Equation 8 of reference (1)

Factor of Safety	FOS	1.30	
	a	87.2	kN/m
	b	-132.7	kN/m
	c	25.1	kN/m

MWH	By: Jean-Baptiste Varnier	Date: 10/29/2013	Project: Midnite Mine	Sheet 2 of 2
	Checked By: Melanie Davis	Description: Sensitivity Analysis, Pseudo-Static Conditions - 3H to 1V slope and 200 ft slope length		Job No. : 1011322

Cover Materials:

Cover soil overlying geocomposite and geomembrane liner. Geocomposite is non-woven needle-punched geotextile heat-bonded on both sides to underlying geonet.

References:

(1) Koerner, R.M., and Soong, T-Y. (2005). Analysis and design of veneer cover soils. Geosynthetics International, Special Issue on the Giroud Lectures, 12, No. 1, 28-49
(2) Miller Geotechnical Consultants (2011). Borrow Source Design Investigation Report. Revision 2. May 6, 2011.
(3) Miller Geotechnical Consultants (2011). Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Rev. 1. December 2.
(4) MGC (2010). Midnite Mine - Site Seismicity Analysis. Technical Memorandun. January 25, 2010; updated April, 2010.
(5) U.S. Department of Energy (DOE) (1989). Technical Approach Document, Revision II, UMTRA-DOE/AL 050425.0002, Uranium Mill Tailings Remedial Action Project, Albuquerque, New Mexico.
(6) U.S. Nuclear Regulatory Commission (NRC), 2003. Standard Review Plan for the Review of a Reclamation Plan for Mill Tailings Sites Under Title II of the Uranium Mill Tailings Remediation Control Act of 1978. NUREG 1620, Revision 1, June.
(7) U.S. Geological Survey (USGS), 2013. U.S. Seismic Design Maps Application. Accessed on October 22, 2013 from http://geohazards.usgs.gov/designmaps/us/application.php

Data Inputs	Variable	Value	Units	Source
Gravity	g	9.81	$m.s^{-1}$	Standard Gravitational Acceleration
Slope Geometry				
Slope length measured along the geomembrane	L	200.00	ft	Design (maximum length for 3H:1V)
		60.96	m	
Cover soil thickness	h_{cs}	3.00	ft	Design
		0.91	m	
Slope ratio	S_r	3	to 1	Design
Soil and Geocomposite Properties				
Cover soil density	d_{cs}	110	pcf	Calculated as 85% of average maximum standard Proctor density at average insitu water content based on test results - references (2) and (3)
	d_{cs}	1769	kg/m^3	
Cover soil friction angle	ϕ_{cs}	33.4	degrees	Average peak friction angle from test results - reference (2)
	ϕ_{cs}	*0.583*	rad	
Cover soil cohesion	c_{cs}	0	psf	From test results - reference (2)
	c_{cs}	0.0	kN/m^2	
Interface friction angle (geocomposite/geomembrane)	δ	23.2	degrees	Backcalculated for FS = 1.0
	δ	0.4	rad	
Adhesion at interface	c_a	0	kN/m^2	Assumed
Seismic				
Peak Ground Acceleration	PGA	0.1572	g	From reference (4) adjusted for waste rock using reference (7)

Calculated Values	Variable	Value	Units	Source
Slope				
Total cover soil thickness	h	0.91	m	Sum of cover soil component depths
Slope angle	β	0.322	rad	ATAN(1/S_r)
	β	18.4	degrees	
Soil and Geocomposite				
Total cover soil density	d_{tcs}	1769	kg/m^3	Weighted average of cover soil densities
Total cover soil unit weight	γ_{tcs}	17.4	kN/m^3	d_{tcs}*g/1000
Cohesive force along failure plane of the passive wedge	C	0	kN/m	Equation 10 of reference (1)
Adhesive force between Geonet/PVC Geomembrane	C_a	0	kN/m	Equation 5 of reference (1)
Active wedge weight	W_A	919	kN/m	Equation 3 of reference (1)
Effective normal force of the active wedge	N_A	872	kN/m	Equation 4 of reference (1)
Passive wedge weight	W_P	24	kN/m	Equation 8 of reference (1)
Seismic				
Horizontal seismic coefficient	C_s	0.10	-	2/3*PGA/g , minimum 0.1 - references (5) and (6)
Factor of Safety	**FOS**	**1.00**		
	a	355.4	kN/m	
	b	-429.4	kN/m	
	c	74.0	kN/m	

Supplement D-7.3

Laboratory Testing Results

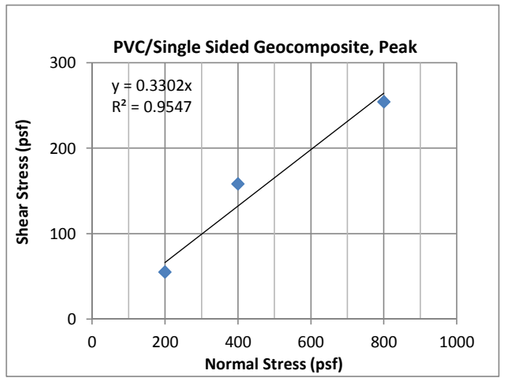
MWH

By: Jean-Baptiste Varnier | Date: 5/30/2014 | Project: Midnite Mine | Sheet 1 of 3

Checked By: Melanie Davis | Description: Interface Direct Shear Test Results - PVC | Job No. : 1011322

PVC/Single Sided Geocomposite, Peak

Normal stress (psf)	Displacement (in)	Shear stress (psf)
0	0	0
200	0.142	55
400	0.142	158
800	0.503	254

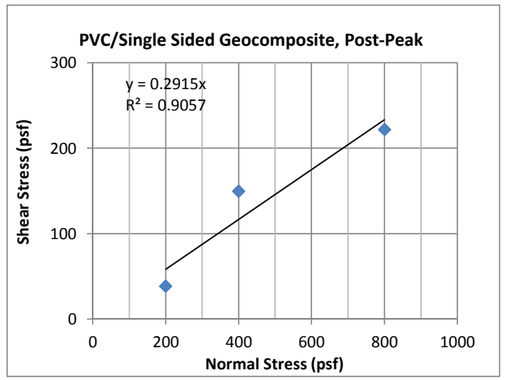


y/x = 0.3302

ϕ (°) = 18.27

PVC/Single Sided Composite, Post-Peak

Normal stress (psf)	Displacement (in)	Shear stress* (psf)
0	0	0
200	>1 in	38.5
400	>1 in	149.5
800	>1 in	221.7



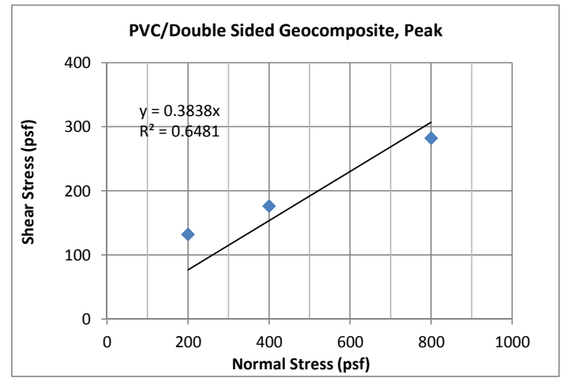
y/x = 0.2915

ϕ (°) = 16.25

*Post-peak shear stress estimated as the average for > 1 inch of displacement since values fluctuate.

PVC/Double Sided Geocomposite, Peak

Normal stress (psf)	Displacement (in)	Shear stress (psf)
0	0	0
200	0.383	132
400	0.383	176
800	0.499	282

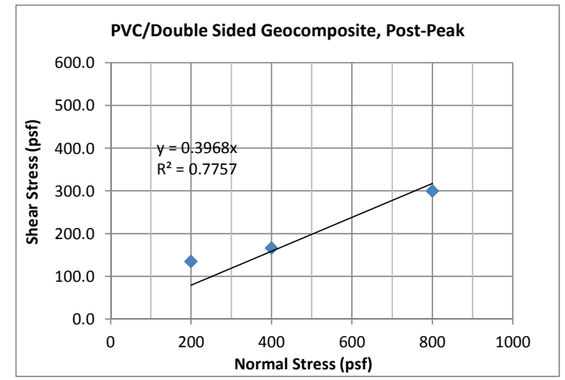


y/x = 0.3838

ϕ (°) = 21.00

PVC/Double Sided Composite, Post-Peak

Normal stress (psf)	Displacement (in)	Shear stress* (psf)
0	0	0
200	>1 in	134.8
400	>1 in	166.4
800	>1 in	299.8



y/x = 0.3968

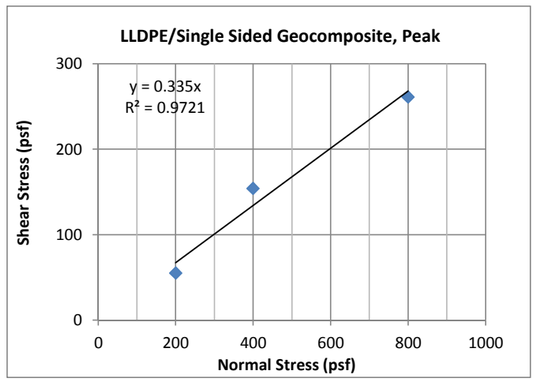
ϕ (°) = 21.64

*Post-peak shear stress estimated as the average for > 1 inch of displacement since values fluctuate.

MWH	By: Jean-Baptiste Varnier	Date: 12/4/2013	Project: Midnite Mine	Sheet 2 of 3
	Checked By: Melanie Davis	Description: Interface Direct Shear Test Results - PVC		Job No. : 1011322

LLDPE/Single Sided Geocomposite, Peak

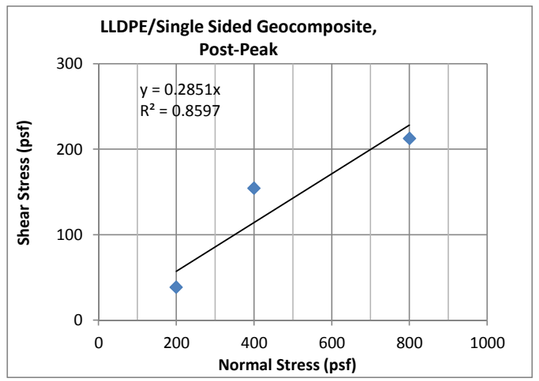
Normal stress (psf)	Displacement (in)	Shear stress (psf)
0	0	0
200	0.142	55
400	0.203	154
800	0.203	261



y/x = 0.335
ϕ (°) = 18.52

LLDPE/Single Sided Composite, Post-Peak

Normal stress (psf)	Displacement (in)	Shear stress* (psf)
0	0	0
200	>1 in	38.5
400	>1 in	154.3
800	>1 in	212.6

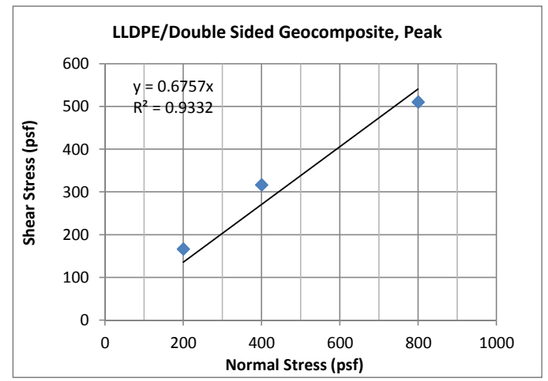


y/x = 0.2851
ϕ (°) = 15.91

*Post-peak shear stress estimated as the average for > 1 inch of displacement since values fluctuate.

LLDPE/Double Sided Geocomposite, Peak

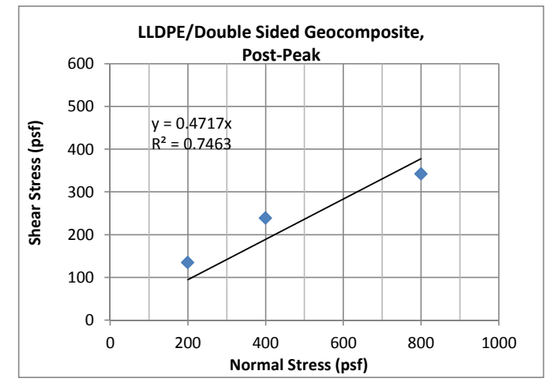
Normal stress (psf)	Displacement (in)	Shear stress (psf)
0	0	0
200	0.56	166
400	0.498	316
800	0.68	510



y/x = 0.6757
ϕ (°) = 34.05

LLDPE/Double Sided Composite, Post-Peak

Normal stress (psf)	Displacement (in)	Shear stress* (psf)
0	0	0
200	1.146	135
400	1.611	239
800	1.844	342



y/x = 0.4745
ϕ (°) = 25.38

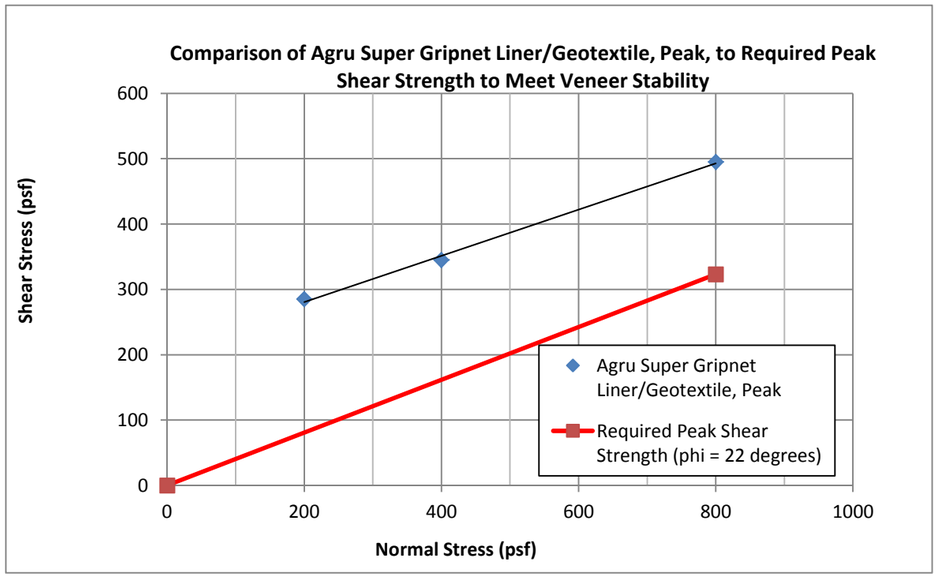
MWH	By: Jean-Baptiste Varnier	Date: 12/4/2013	Project: Midnite Mine	Sheet 3 of 3
	Checked By: Melanie Davis	Description: Interface Direct Shear Test Results - Agru Super Gripnet Liner		Job No. : 1011322

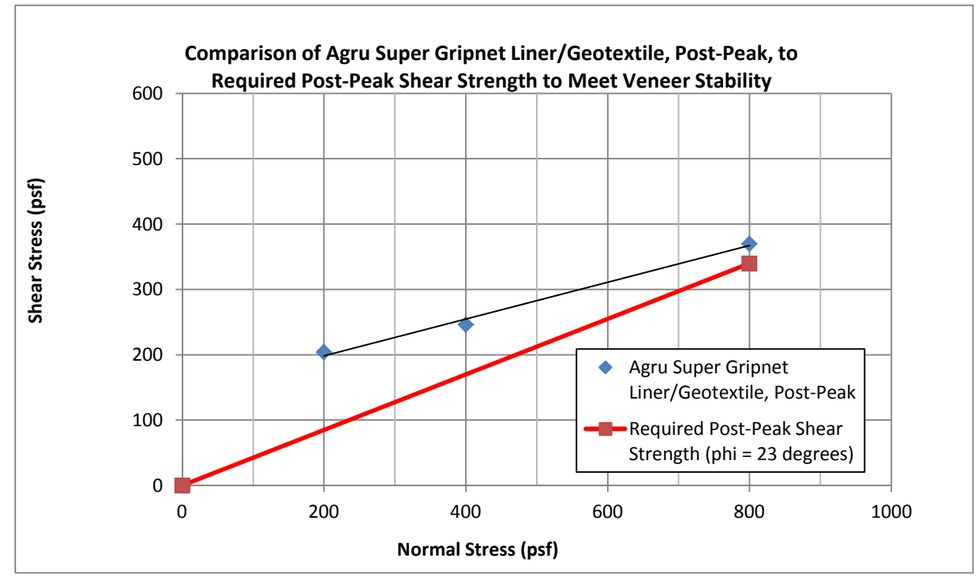
Agru Super Gripnet Liner/Geotextile, Peak

Normal stress (psf)	Displacement (in)	Shear stress (psf)
0	0	0
200	0.459	285
400	0.397	345
800	0.893	495

Agru Super Gripnet Liner/Geotextile, Post-Peak

Normal stress (psf)	Displacement (in)	Shear stress (psf)
0	0	0
200	1.993	204
400	1.993	246
800	1.993	370





Large Scale Interface Direct Shear
ASTM D5321

Client: MWH
Job Number.: 2512-76
Project Number: 1011322
Conditions: Inundated
Location: Newmont Midnite Mine
Project:

Speed Rate (in/min): 0.050
Normal Stress 1 (psf): 200
Normal Stress 2 (psf): 400
Normal Stress 3 (psf): 800

Test Date: 41219
Technician: TMR
Data Entered By: TMR
Date: 11/8/2013
Data Files: MNLDPG2A.TXT,MNLDPG2B.TXT,MNLDPG2C.TXT

Displacement (in)	Normal Stress 1 (psf):	Normal Stress 2 (psf):	Normal Stress 3 (psf):
0.000	0	0	0
0.019	40	118	150
0.080	52	148	201
0.142	55	158	226
0.203	54	156	230
0.260	52	156	235
0.324	45	153	244
0.383	40	148	253
0.445	37	143	253
0.503	32	136	254
0.566	29	133	254
0.625	32	131	249
0.687	42	141	246
0.742	50	154	243
0.803	49	158	236
0.864	49	156	231
0.926	45	154	230
0.982	40	153	225
1.044	37	149	226
1.104	34	144	225
1.164	32	144	226
1.220	34	138	226
1.281	39	143	228
1.340	42	151	226
1.400	44	153	221
1.458	44	156	220
1.517	45	156	218
1.576	42	154	216
1.637	39	153	216
1.693	34	149	218
1.753	34	146	218
1.813	35	144	221
1.871	39	151	221
1.926	42	161	221

Top Layer
Boring: Superstrate
Depth:
Sample Number:
Density (pcf): 111
% Moisture: 14

Bottom Layer
Boring: Substrate
Depth:
Sample Number:
Density (pcf): 117
% Moisture: 8

Top Geosynthetic
Material Type: GSE Single Sided Composite
Roll Number: 7739
Lot Number:

Bottom Geosynthetic
Material Type: 40 PVC
Roll Number: 3
Lot Number: FMA0037-3

Interface
Top Geosynthetic vs. Bottom Geosynthetic

	Peak	Ultimate
Friction Angle	17.6	15.5
Adhesion (psf)	8.5	12.0

	Peak Strength(psf)	Ultimate Strength(psf)
Normal Stress 1	55	42
Normal Stress 2	161	161
Normal Stress 3	254	221

Data Checked By DPM Date 11/8/13
File Name: 2512_76_largeScaleDirectShear-ASTMD3080m-5321-6243-R0_01.xls

ADVANCED TERRA TESTING

Sample Configuration

Top Layer

Boring: Superstrate

Depth:

Sample Number:

Density (pcf): 111

% Moisture: 14

Bottom Layer

Boring: Substrate

Depth:

Sample Number:

Density (pcf): 117

% Moisture: 8

Top Geosynthetic

Material Type: GSE Single Sided Composite

Roll Number: 7739

Lot Number:

Bottom Geosynthetic

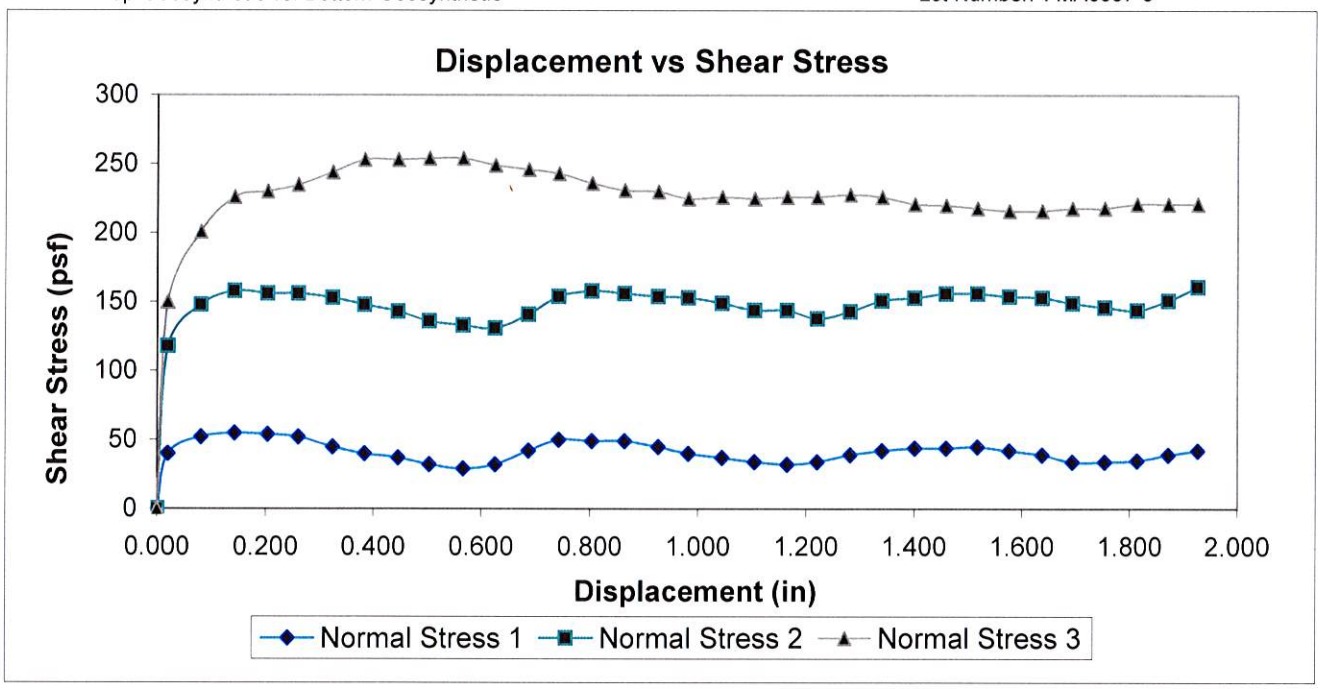
Material Type: 40 PVC

Roll Number: 3

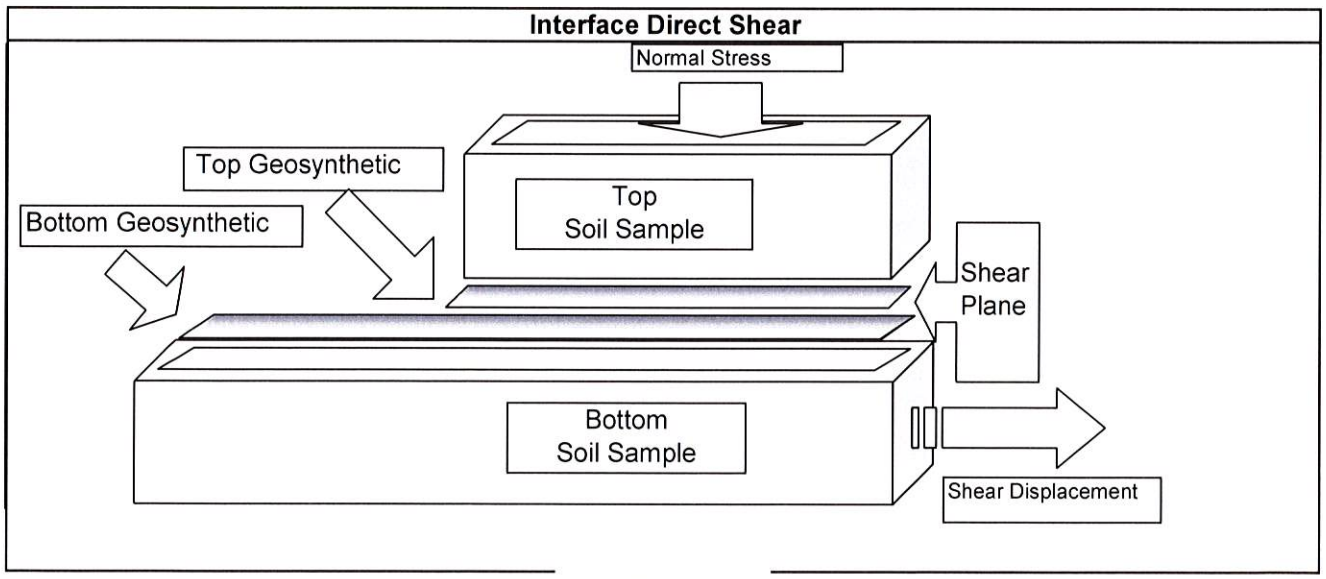
Lot Number: FMA0037-3

Interface

Top Geosynthetic vs. Bottom Geosynthetic

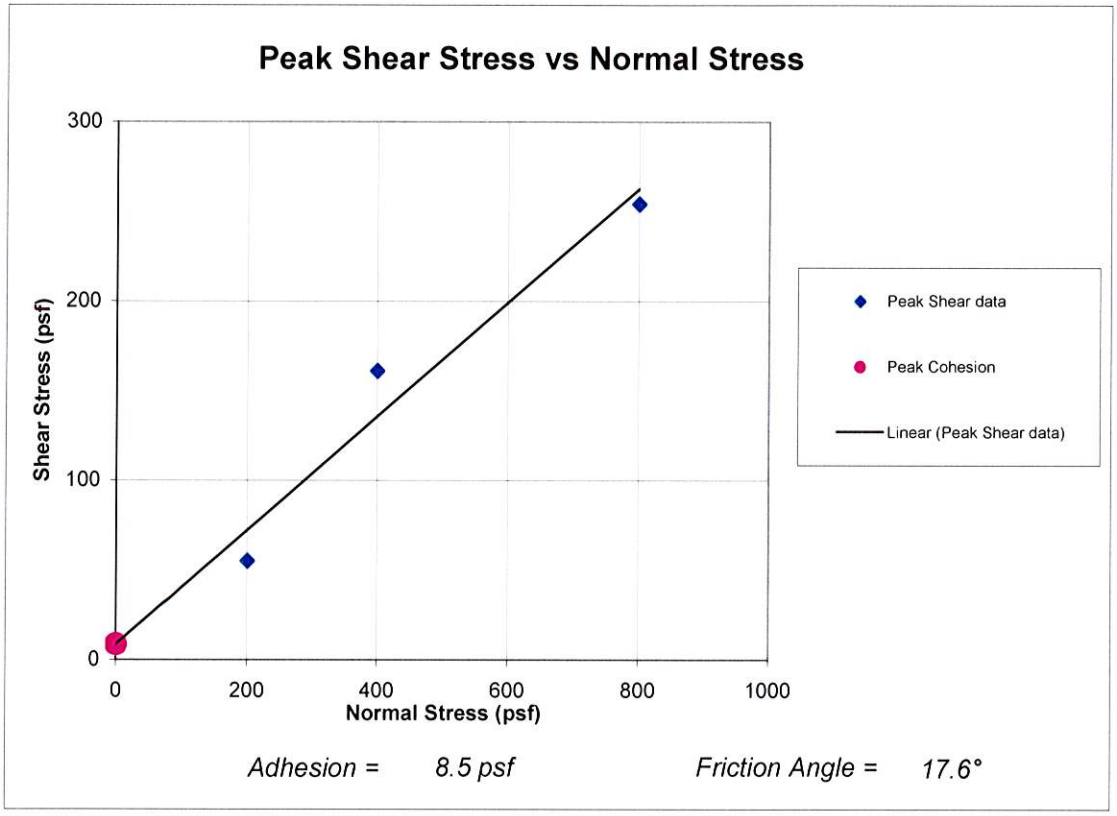


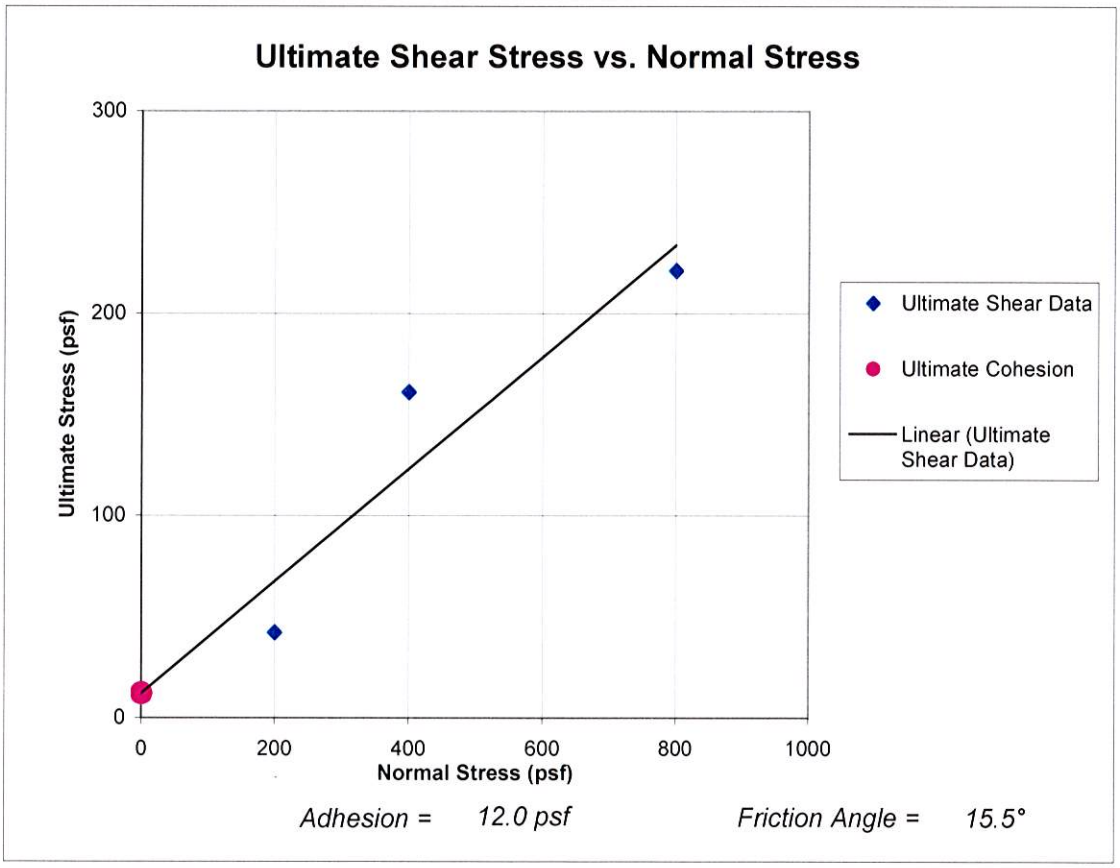
Diagram



ATT

ADVANCED TERRA TESTING





ATT
ADVANCED TERRA TESTING

Large Scale Interface Direct Shear
ASTM D5321

Client: MWH
Job Number.: 2512-76
Project Number: 1011322
Conditions: Inundated
Location: Newmont Midnite Mine
Project:

Speed Rate (in/min): 0.050
Normal Stress 1 (psf): 200
Normal Stress 2 (psf): 400
Normal Stress 3 (psf): 800

Test Date: 41590
Technician: TMR
Data Entered By: TMR
Date: 1/0/1900

Data Files: MNLDPG2A.TXT,MNLDPGB.TXT,MNLDPGC.TXT

Displacement (in)	Normal Stress 1 (psf):	Normal Stress 2 (psf):	Normal Stress 3 (psf):
0.000	0	0	0
0.019	40	105	143
0.080	52	146	203
0.142	55	151	243
0.203	54	154	261
0.260	52	153	259
0.324	45	145	251
0.383	40	140	240
0.445	37	131	218
0.503	32	130	200
0.566	29	135	185
0.625	32	141	183
0.687	42	149	196
0.742	50	156	218
0.803	49	153	221
0.864	49	153	223
0.926	45	148	221
0.982	40	143	213
1.044	37	143	200
1.104	34	143	191
1.164	32	151	190
1.220	34	153	195
1.281	39	158	206
1.340	42	156	215
1.400	44	158	221
1.458	44	154	218
1.517	45	153	220
1.576	42	153	216
1.637	39	151	206
1.693	34	153	206
1.753	34	154	213
1.813	35	159	225
1.871	39	166	235
1.926	42	163	245

Top Layer
Boring: Superstrate
Depth: --
Sample Number: --
Density (pcf): 117
% Moisture: 8

Bottom Layer
Boring: Substrate
Depth: --
Sample Number: --
Density (pcf): 111
% Moisture: 14

Top Geosynthetic
Material Type: GSE Single Sided Composite
Roll Number: 7739
Lot Number: --

Bottom Geosynthetic
Material Type: Textured LLDPE
Roll Number: --
Lot Number: --

Interface
Top Geosynthetic vs. Bottom Geosynthetic

	Peak	Ultimate
Friction Angle	18.2	17.7
Adhesion (psf)	7.5	1.0

	Peak Strength(psf)	Ultimate Strength(psf)
Normal Stress 1	55	42
Normal Stress 2	166	163
Normal Stress 3	261	245

Data Checked By DPM Date 11/14/13
File Name: 2512_76_largeScaleDirectShear-ASTMD3080m-5321-6243-R0_02.xls

ATT
ADVANCED TERRA TESTING

Sample Configuration

Top Layer

Boring: Superstrate

Depth: --

Sample Number: --

Density (pcf): 117

% Moisture: 8

Bottom Layer

Boring: Substrate

Depth: --

Sample Number: --

Density (pcf): 111

% Moisture: 14

Top Geosynthetic

Material Type: GSE Single Sided Composite

Roll Number: 7739

Lot Number: --

Bottom Geosynthetic

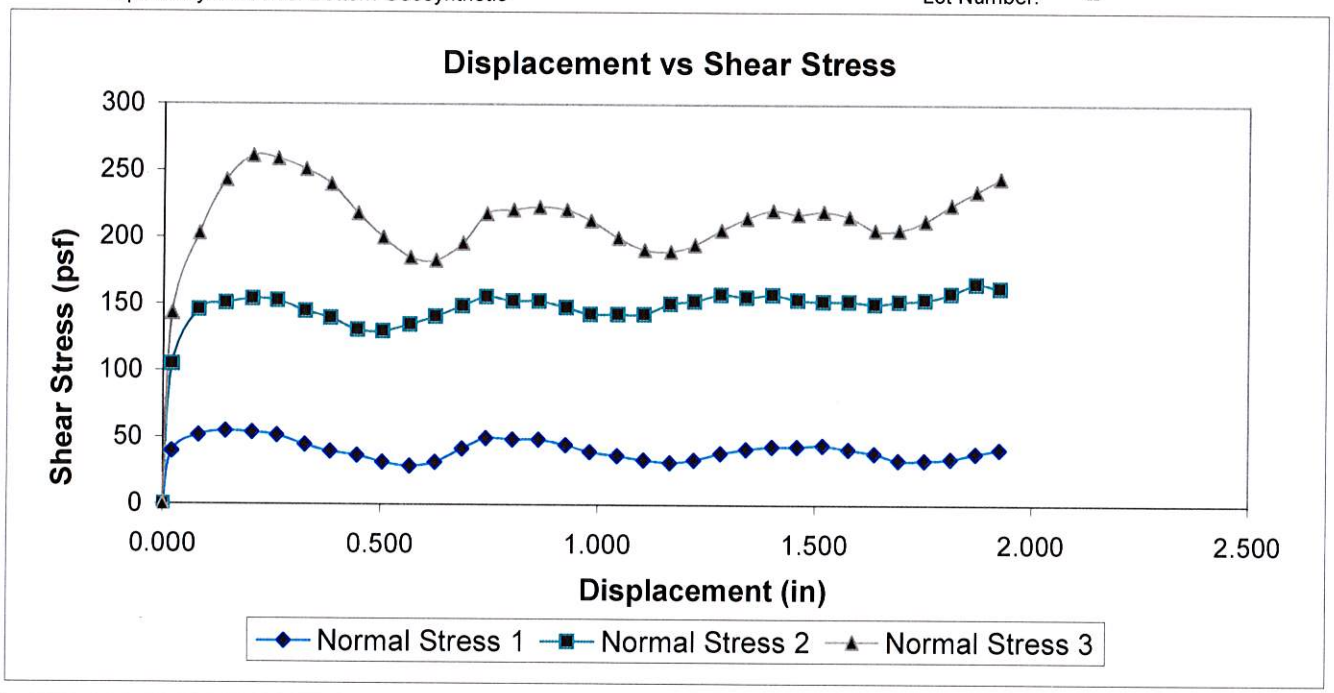
Material Type: Textured LLDPE

Roll Number: --

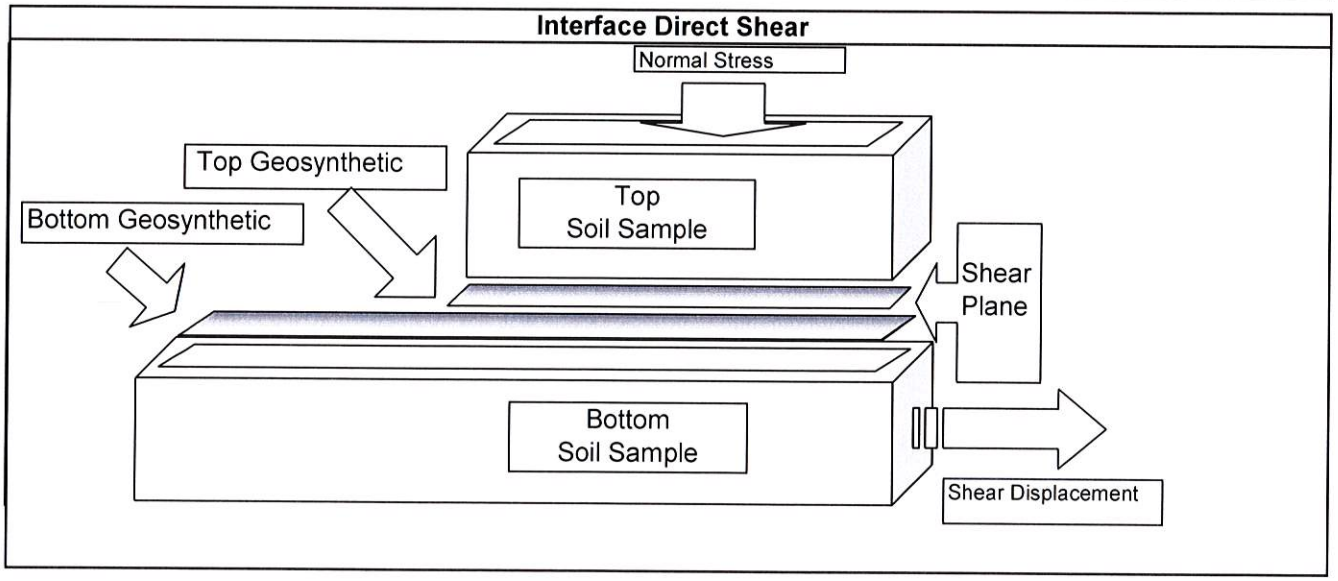
Lot Number: --

Interface

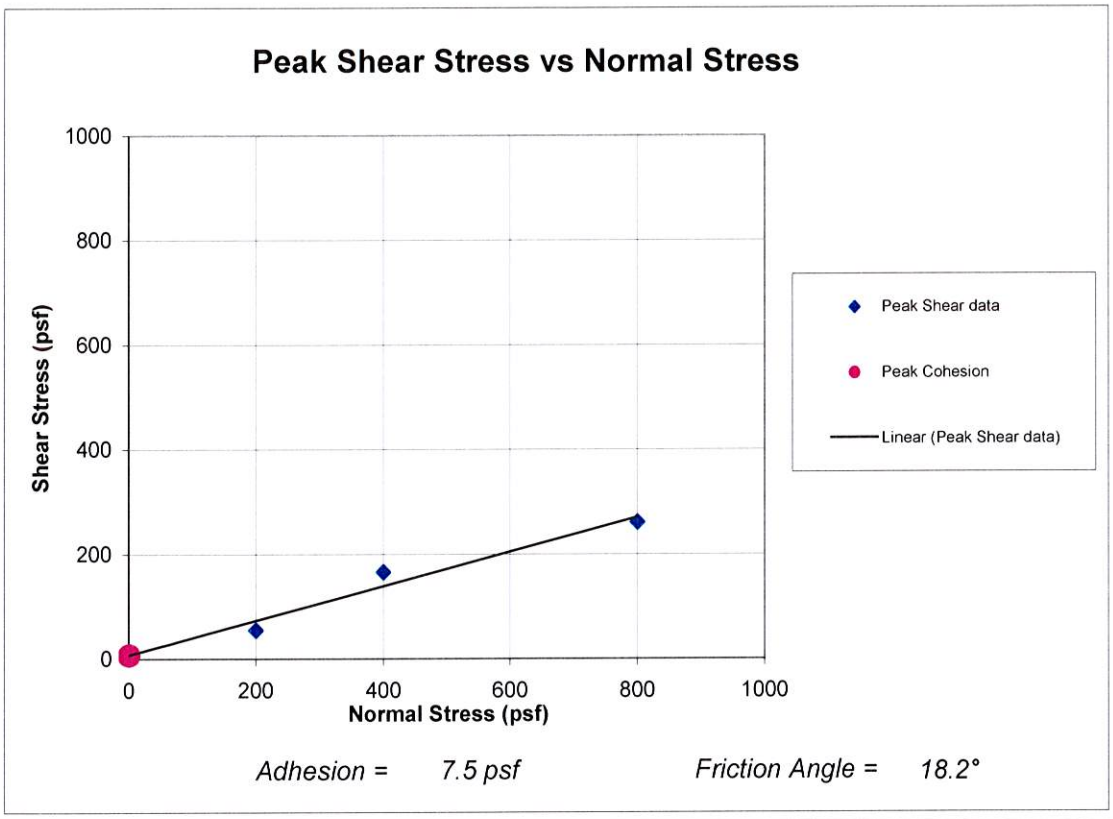
Top Geosynthetic vs. Bottom Geosynthetic

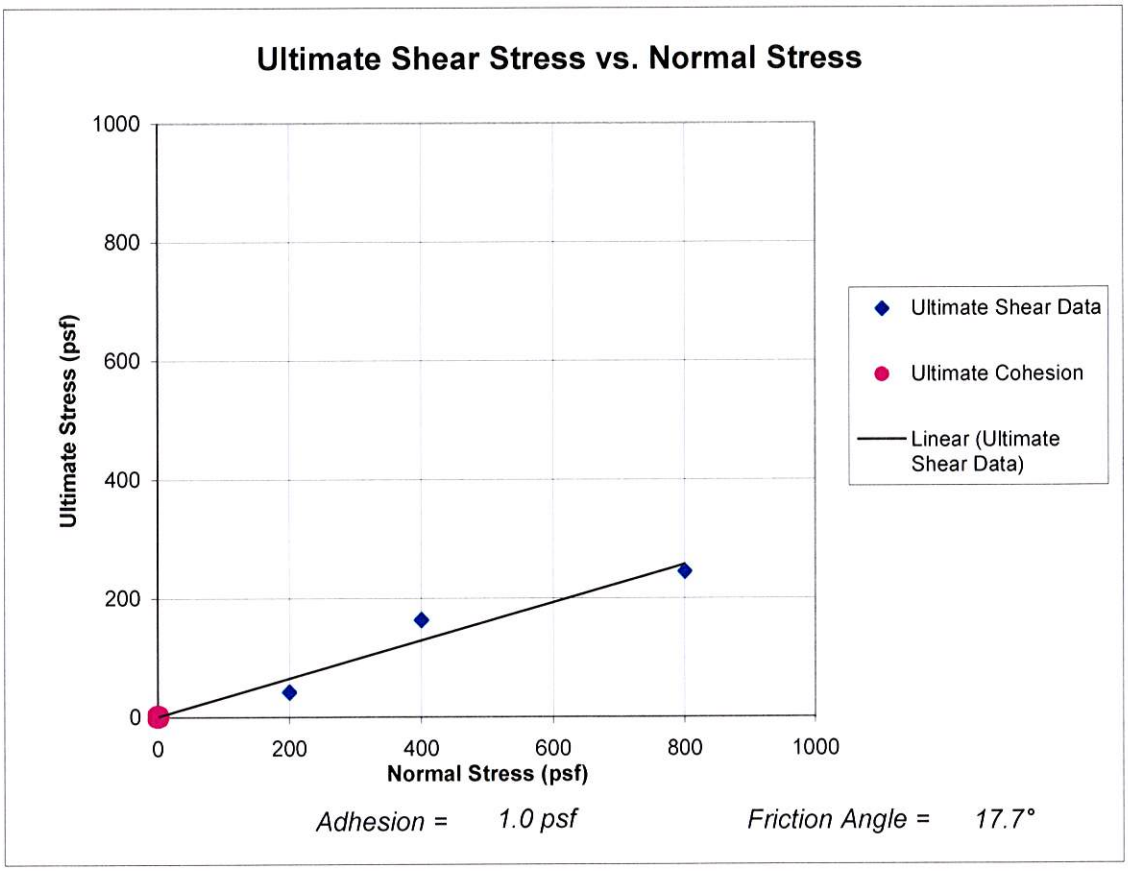


Diagram



ATT
ADVANCED TERRA TESTING





ATT
ADVANCED TERRA TESTING

Large Scale Interface Direct Shear
ASTM D5321

Client: MWH
Job Number.: 2512-76
Project Number: 1011322
Conditions: Inundated
Location: Newmont Midnite Mine
Project:

Speed Rate (in/min): 0.050
Normal Stress 1 (psf): 200
Normal Stress 2 (psf): 400
Normal Stress 3 (psf): 800

Test Date: 11/20/2013
Technician: TMR
Data Entered By: TMR
Date: 11/22/2013

Data Files: MNLDTG2A.TXT,MNLDTG2B.TXT,MNLDTG2C.TXT

Displacement (in)	Normal Stress 1 (psf):	Normal Stress 2 (psf):	Normal Stress 3 (psf):
0.000	0	0	0
0.026	107	156	206
0.087	135	221	294
0.147	142	252	347
0.206	147	272	389
0.263	152	286	417
0.322	155	296	440
0.382	161	306	459
0.443	165	311	479
0.498	165	316	490
0.560	166	311	500
0.619	166	309	505
0.680	166	309	510
0.734	163	301	505
0.795	165	287	482
0.854	161	284	465
0.914	158	281	459
0.969	152	279	450
1.029	150	271	440
1.088	150	267	434
1.146	147	269	432
1.203	147	257	412
1.262	147	257	407
1.321	147	251	401
1.379	143	247	392
1.435	145	249	381
1.493	145	246	374
1.552	142	243	372
1.611	142	239	366
1.667	138	239	359
1.726	137	238	356
1.786	135	241	349
1.844	140	238	342
1.902	135	236	341

Top Layer
Boring: Superstrate
Depth: --
Sample Number: --
Density (pcf): 117
% Moisture: 8

Bottom Layer
Boring: Substrate
Depth: --
Sample Number: --
Density (pcf): 111
% Moisture: 14

Top Geosynthetic
Material Type: GSE Double Sided Composite
Roll Number:
Lot Number: --

Bottom Geosynthetic
Material Type: Textured LLDPE
Roll Number: --
Lot Number: --

Interface
Top Geosynthetic vs. Bottom Geosynthetic

	Peak	Ultimate
Friction Angle	29.3	18.4
Adhesion (psf)	69.0	82.5

	Peak Strength(psf)	Ultimate Strength(psf)
Normal Stress 1	166	135
Normal Stress 2	316	236
Normal Stress 3	510	341

Data Checked By KR Date 11/22/13
File Name: 2512_76_largeScaleDirectShear-ASTMD3080m-5321-6243-R0_03.xls

ADVANCED TERRA TESTING

Sample Configuration

Top Layer

Boring: Superstrate

Depth: --

Sample Number: --

Density (pcf): 117

% Moisture: 8

Bottom Layer

Boring: Substrate

Depth: --

Sample Number: --

Density (pcf): 111

% Moisture: 14

Top Geosynthetic

Material Type: GSE Double Sided Composite

Roll Number:

Lot Number: --

Bottom Geosynthetic

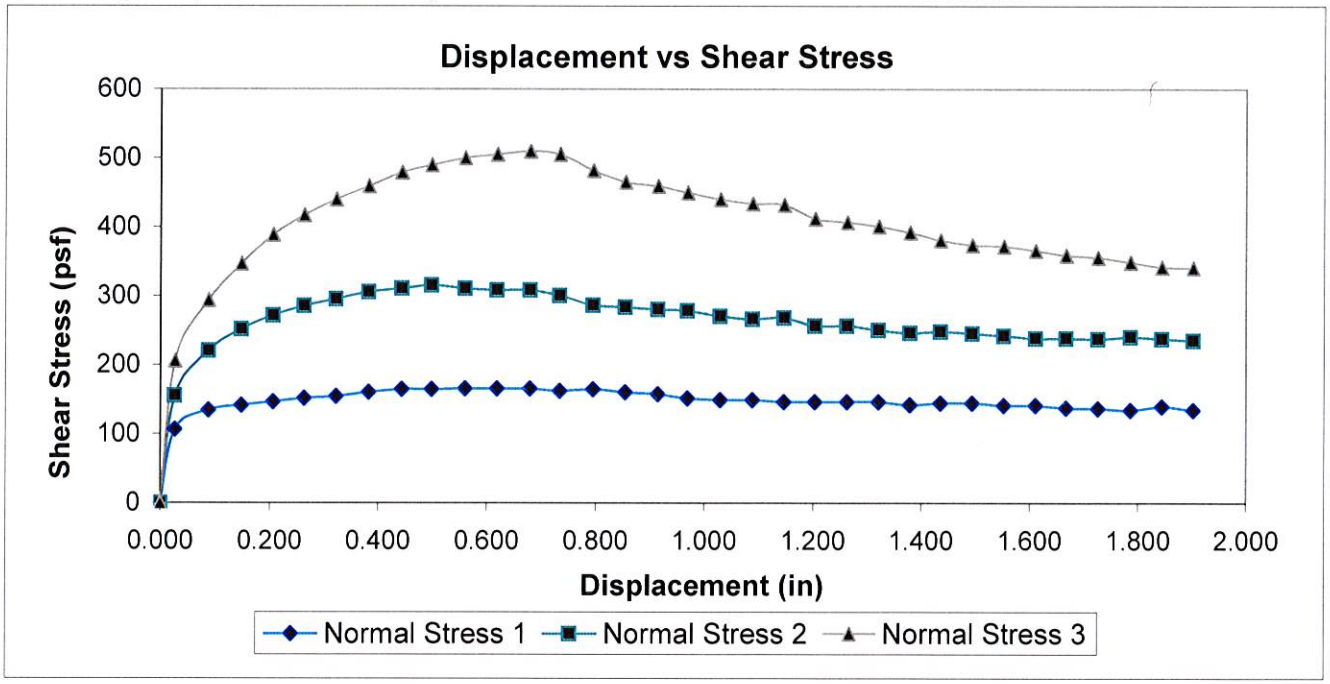
Material Type: Textured LLDPE

Roll Number: --

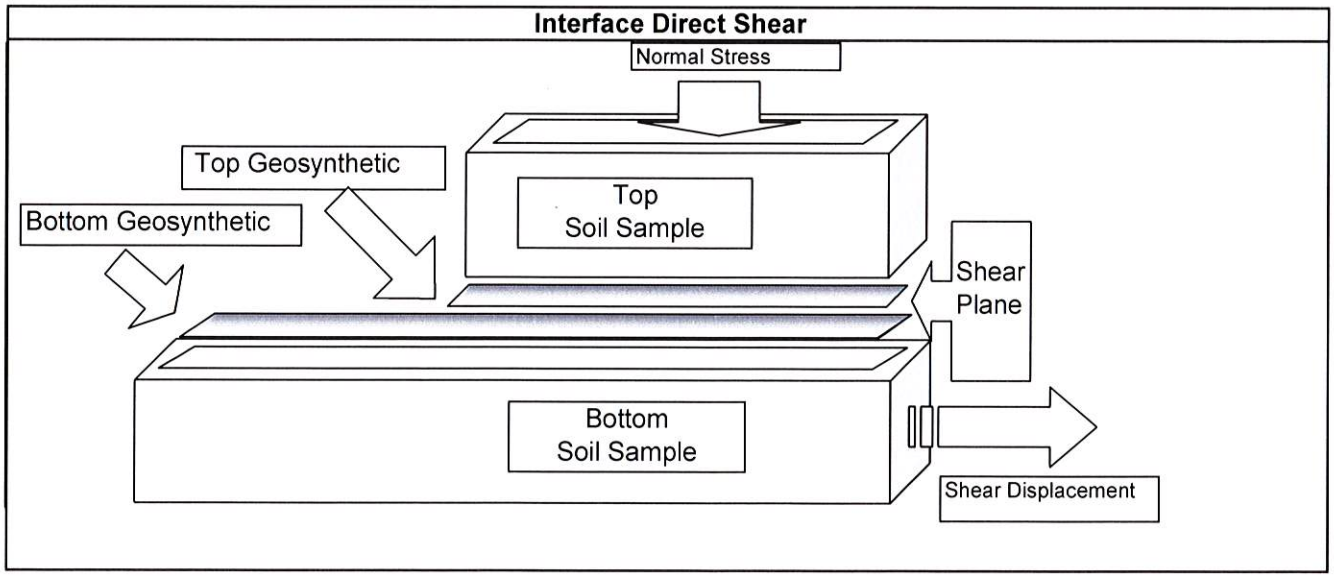
Lot Number: --

Interface

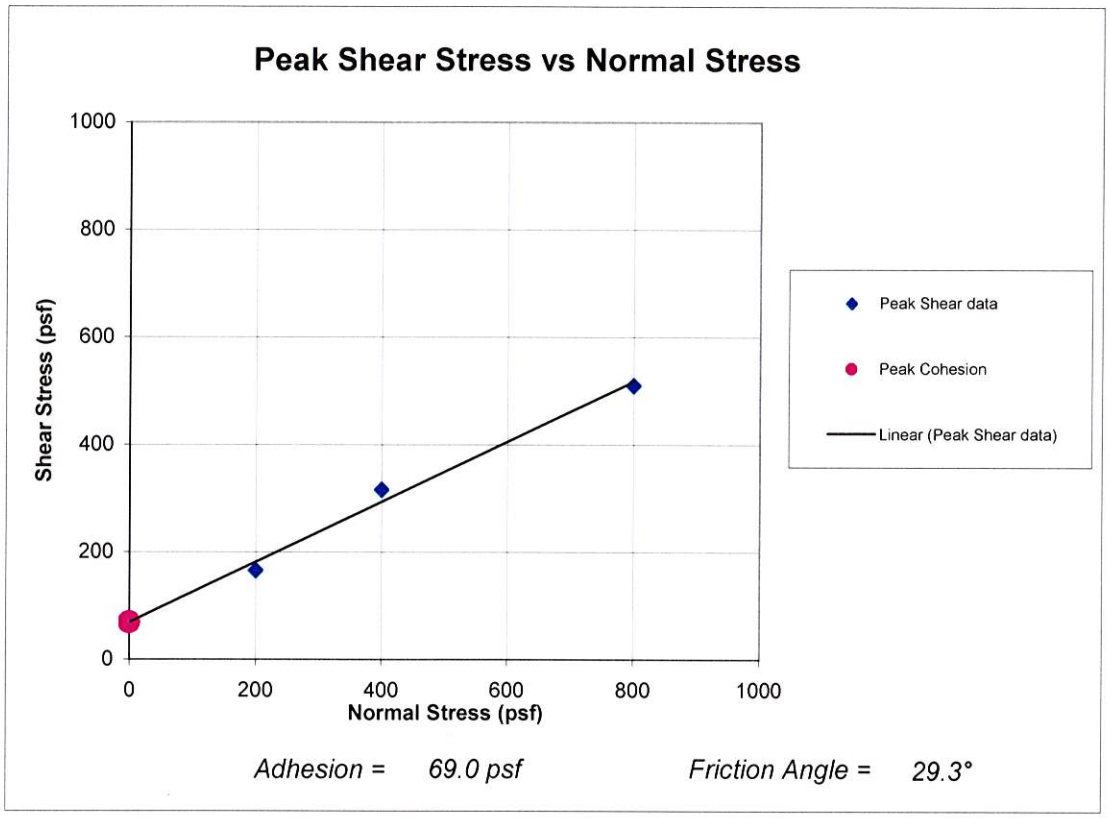
Top Geosynthetic vs. Bottom Geosynthetic



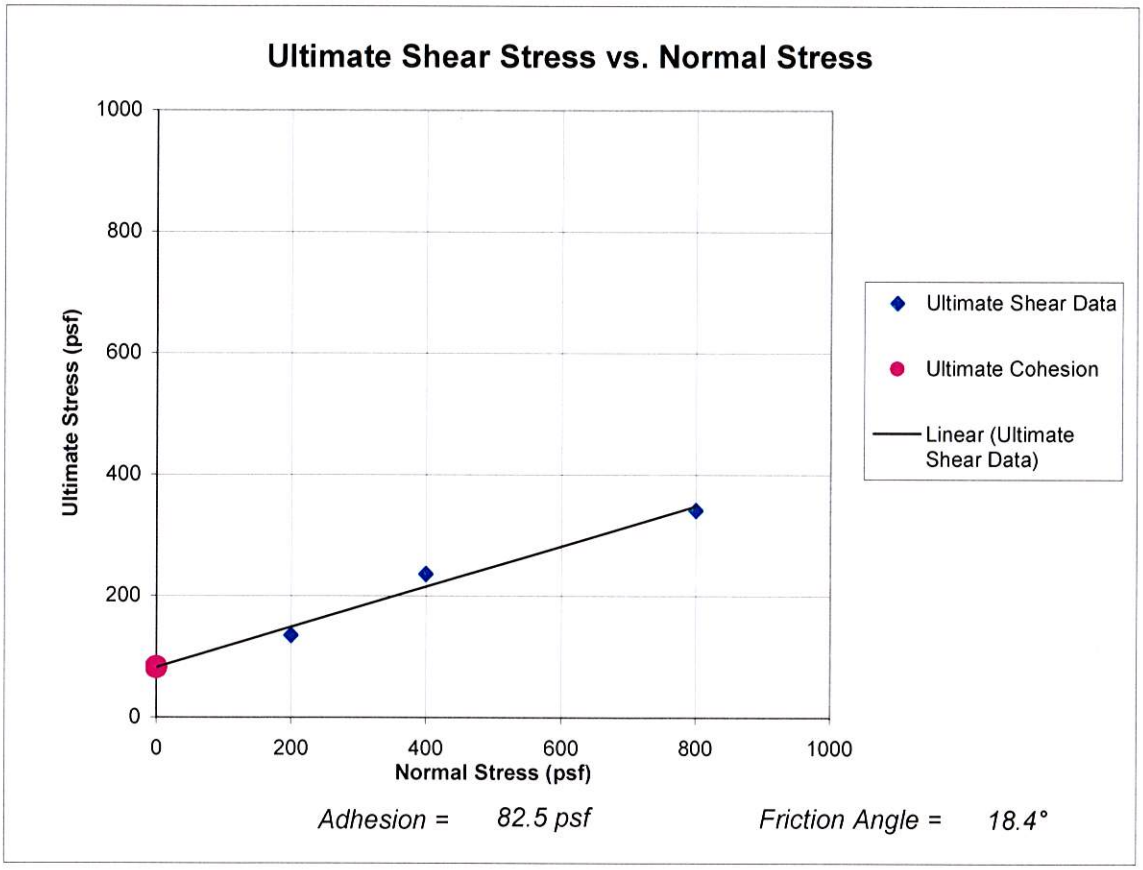
Diagram



ATT
ADVANCED TERRA TESTING



Adhesion = 69.0 psf *Friction Angle = 29.3°*



Adhesion = 82.5 psf *Friction Angle = 18.4°*

ADVANCED TERRA TESTING

Large Scale Interface Direct Shear
ASTM D5321

Client:	MWH	Speed Rate (in/min):	0.050	Test Date:	11/29/2013
Job Number.:	2512-76	Normal Stress 1 (psf):	200	Technician:	TMR
Project Number:	1011322	Normal Stress 2 (psf):	400	Data Entered By:	TMR
Conditions:	Inundated	Normal Stress 3 (psf):	800	Date:	12/2/2013
Location:	Newmont Midnite Mine				
Project:		Data Files:	MNLDAGRA.TXT,MNLDAGRB.TXT,MNLDAGRC.TXT		

Displacement (in)	Normal Stress 1 (psf):	Normal Stress 2 (psf):	Normal Stress 3 (psf):
0.000	0	0	0
0.021	196	196	178
0.082	242	234	237
0.147	259	269	287
0.209	267	294	325
0.268	277	317	362
0.333	282	340	390
0.397	282	345	412
0.459	285	342	433
0.520	279	340	452
0.582	277	347	467
0.645	269	340	478
0.708	269	339	486
0.767	262	339	491
0.830	255	329	493
0.893	255	324	495
0.957	254	314	490
1.015	249	309	486
1.076	259	302	488
1.139	249	307	486
1.201	237	294	485
1.261	231	284	472
1.322	224	279	470
1.385	214	276	467
1.445	216	279	462
1.507	209	267	448
1.568	206	267	437
1.630	201	262	423
1.691	211	251	408
1.748	197	244	403
1.810	204	239	400
1.873	206	236	390
1.933	204	244	380
1.993	204	246	370

Top Layer
Boring: Superstrate
Depth: --
Sample Number: --
Density (pcf): 117
% Moisture: 8

Bottom Layer
Boring: Substrate
Depth: --
Sample Number: --
Density (pcf): 111
% Moisture: 14

Top Geosynthetic
Material Type: Geofabric
Roll Number: 1118022288
Lot Number: --

Bottom Geosynthetic
Material Type: AGRU Spike
Roll Number: 911333-12
Lot Number: --

Interface
Top Geosynthetic vs. Bottom Geosynthetic

	Peak	Ultimate
Friction Angle	19.4	15.7
Adhesion (psf)	211.0	142.0

	Peak Strength(psf)	Ultimate Strength(psf)
Normal Stress 1	285	204
Normal Stress 2	347	246
Normal Stress 3	495	370

Data Checked By BKL Date 12/2/13

File Name: 2512_76_largeScaleDirectShear-ASTMD3080m-5321-6243-R0_04.xls

ATT
ADVANCED TERRA TESTING

Sample Configuration

Top Layer

Boring: Superstrate

Depth: --

Sample Number: --

Density (pcf): 117

% Moisture: 8

Bottom Layer

Boring: Substrate

Depth: --

Sample Number: --

Density (pcf): 111

% Moisture: 14

Top Geosynthetic

Material Type: Geofabric

Roll Number: 1118022288

Lot Number: --

Bottom Geosynthetic

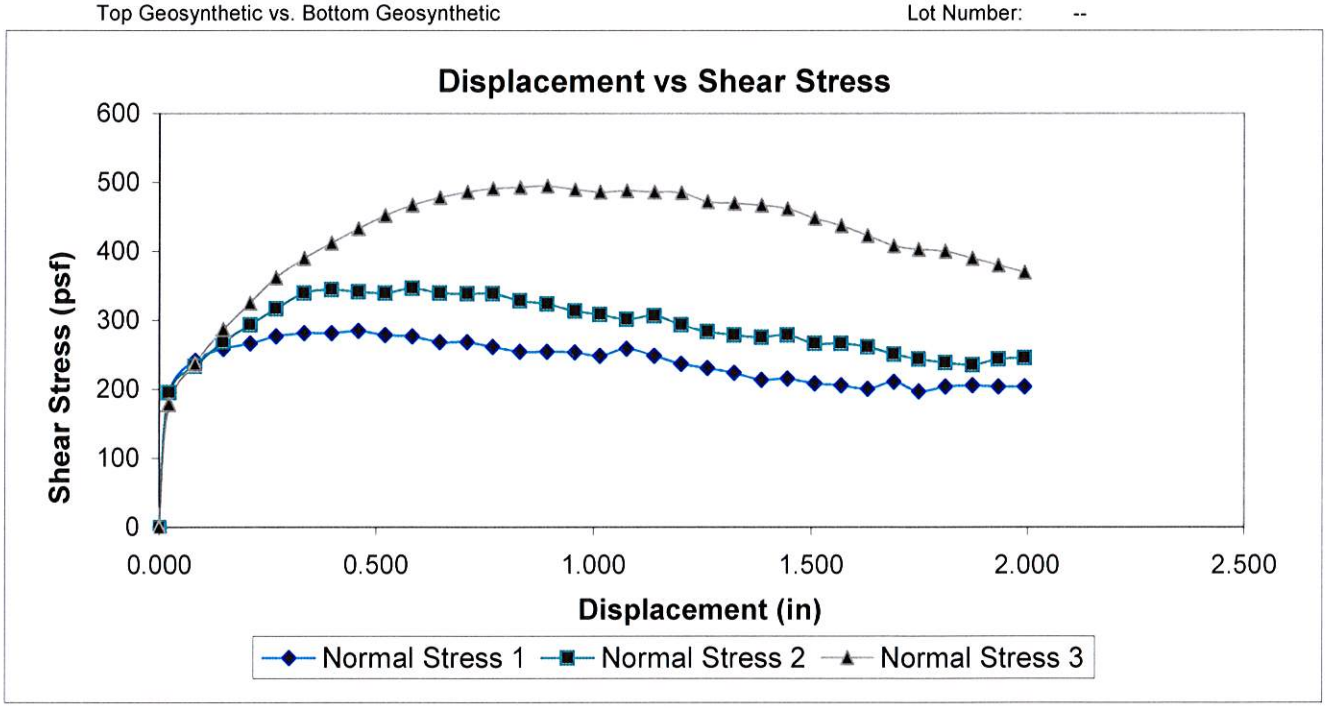
Material Type: AGRU Spike

Roll Number: 911333-12

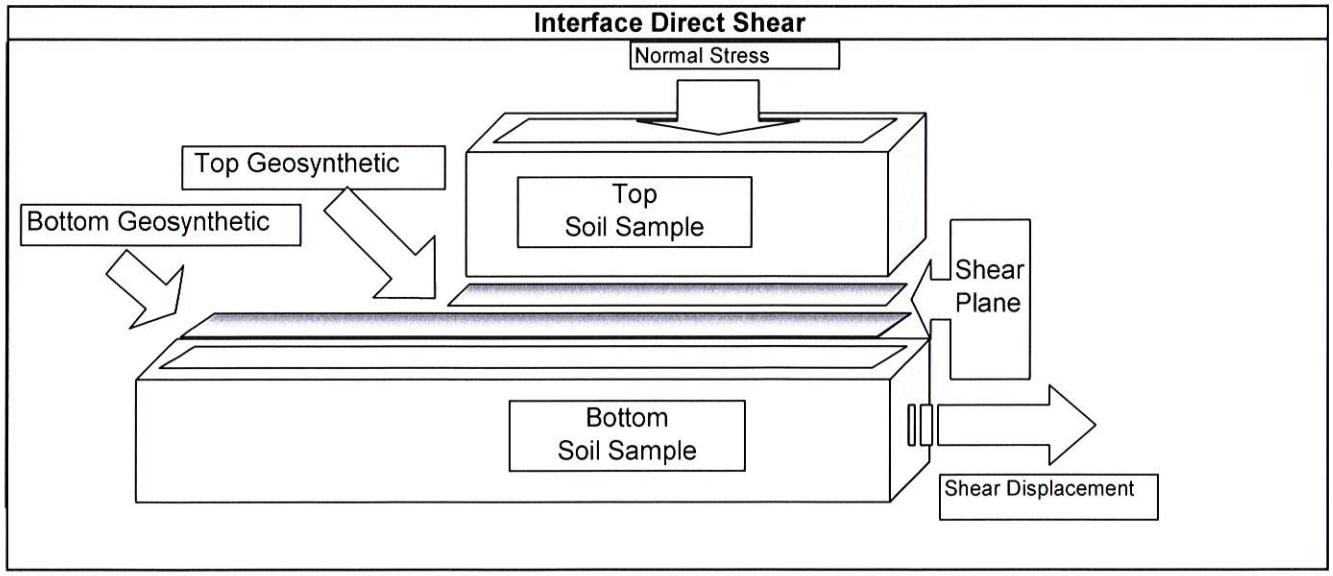
Lot Number: --

Interface

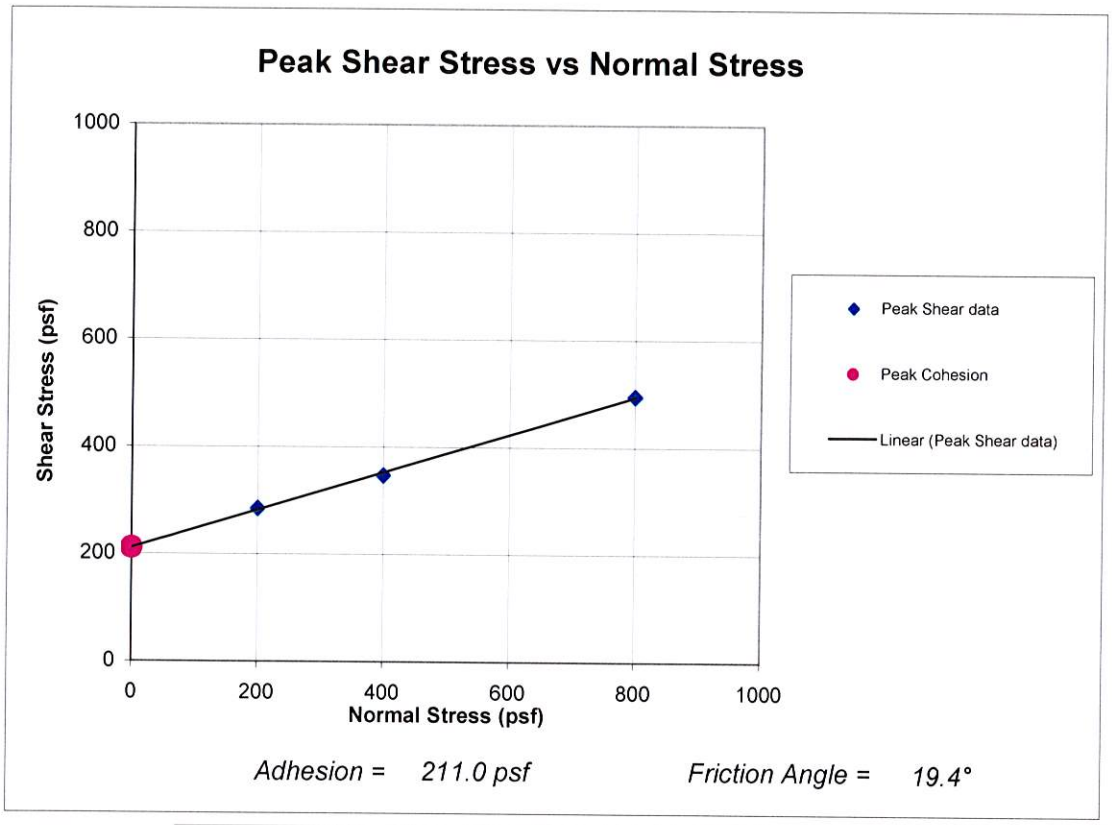
Top Geosynthetic vs. Bottom Geosynthetic

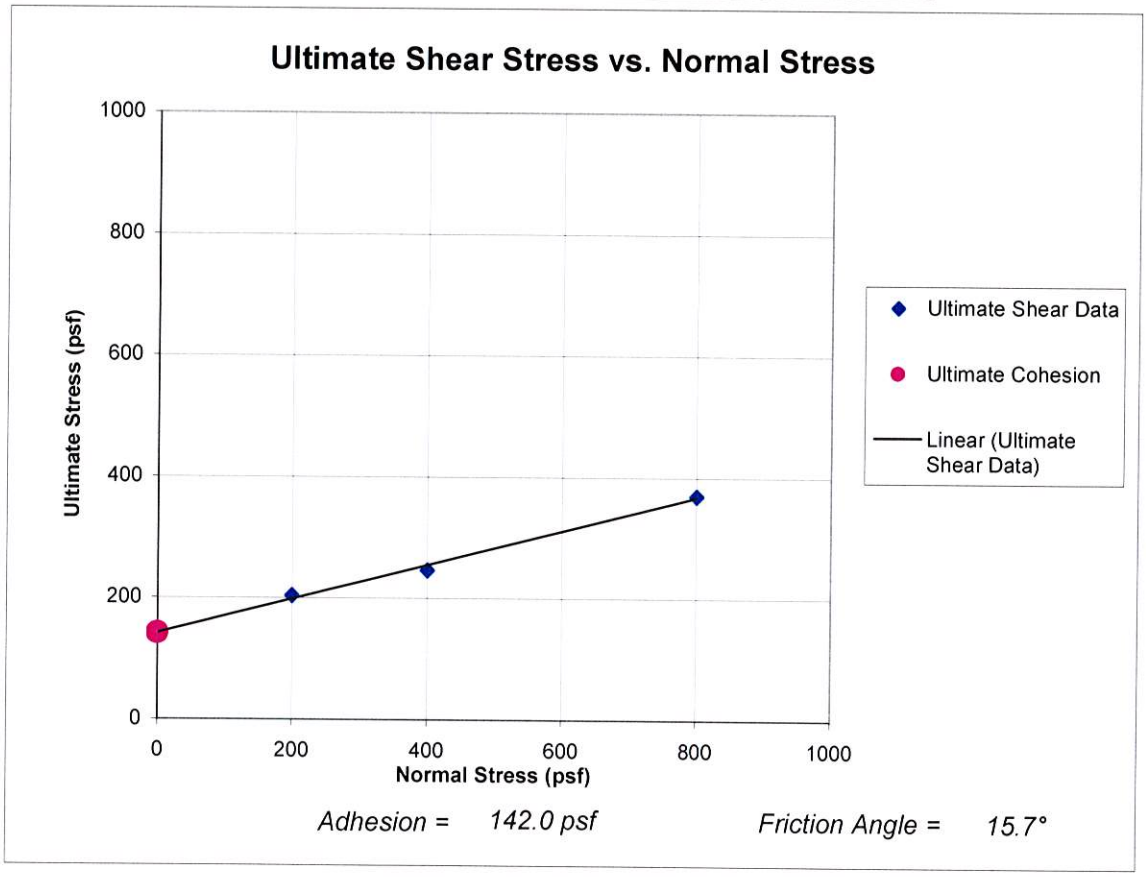


Diagram



ATT
ADVANCED TERRA TESTING





ATT
ADVANCED TERRA TESTING

Large Scale Interface Direct Shear
ASTM D5321

Client: MWH
Job Number.: 2512-76
Project Number: 1011322
Conditions: Inundated
Location: Newmont Midnite Mine
Project:

Speed Rate (in/min): 0.050
Normal Stress 1 (psf): 200
Normal Stress 2 (psf): 400
Normal Stress 3 (psf): 800

Test Date: 12/11/2013
Technician: TMR
Data Entered By: TMR
Date: 12/11/2013

Data Files: MNLDP2DA.TXT,MNLDP2DB.TXT,MNLDP2DC.TXT

Displacement (in)	Normal Stress 1 (psf):	Normal Stress 2 (psf):	Normal Stress 3 (psf):
0.000	0	0	0
0.026	110	105	159
0.086	127	147	201
0.145	133	151	214
0.201	133	158	222
0.264	130	168	242
0.324	130	175	259
0.383	132	176	272
0.439	128	176	274
0.499	125	175	282
0.560	123	170	282
0.619	125	168	282
0.676	128	165	282
0.735	132	163	280
0.796	133	160	282
0.854	133	163	284
0.910	133	165	287
0.969	133	168	294
1.030	133	170	299
1.088	132	170	300
1.145	132	170	300
1.204	130	166	294
1.263	132	166	297
1.322	135	163	295
1.376	137	165	297
1.437	138	165	300
1.495	140	165	302
1.554	137	165	304
1.608	137	165	302
1.667	135	166	300
1.725	133	168	300
1.785	135	166	302
1.839	135	165	300
1.898	135	168	304

Top Layer
Boring: Superstrate
Depth: --
Sample Number: --
Density (pcf): 117
% Moisture: 8

Bottom Layer
Boring: Substrate
Depth: --
Sample Number: --
Density (pcf): 111
% Moisture: 14

Top Geosynthetic
Material Type: GSE Double Sided Composite
Roll Number: --
Lot Number: --

Bottom Geosynthetic
Material Type: 40 PVC
Roll Number: 3
Lot Number: FMA0037-3

Interface
Top Geosynthetic vs. Bottom Geosynthetic

	Peak	Ultimate
Friction Angle	15.6	16.2
Adhesion (psf)	76.0	67.0

	Peak Strength(psf)	Ultimate Strength(psf)
Normal Stress 1	140	135
Normal Stress 2	176	168
Normal Stress 3	304	304

Data Checked By BKC Date 12/11/13
File Name: 2512_76_largeScaleDirectShear-ASTMD3080m-5321-6243-R0_05.xls

ATT
ADVANCED TERRA TESTING

Sample Configuration

Top Layer

Boring: Superstrate

Depth: --

Sample Number: --

Density (pcf): 117

% Moisture: 8

Bottom Layer

Boring: Substrate

Depth: --

Sample Number: --

Density (pcf): 111

% Moisture: 14

Top Geosynthetic

Material Type: GSE Double Sided Composite

Roll Number: --

Lot Number: --

Bottom Geosynthetic

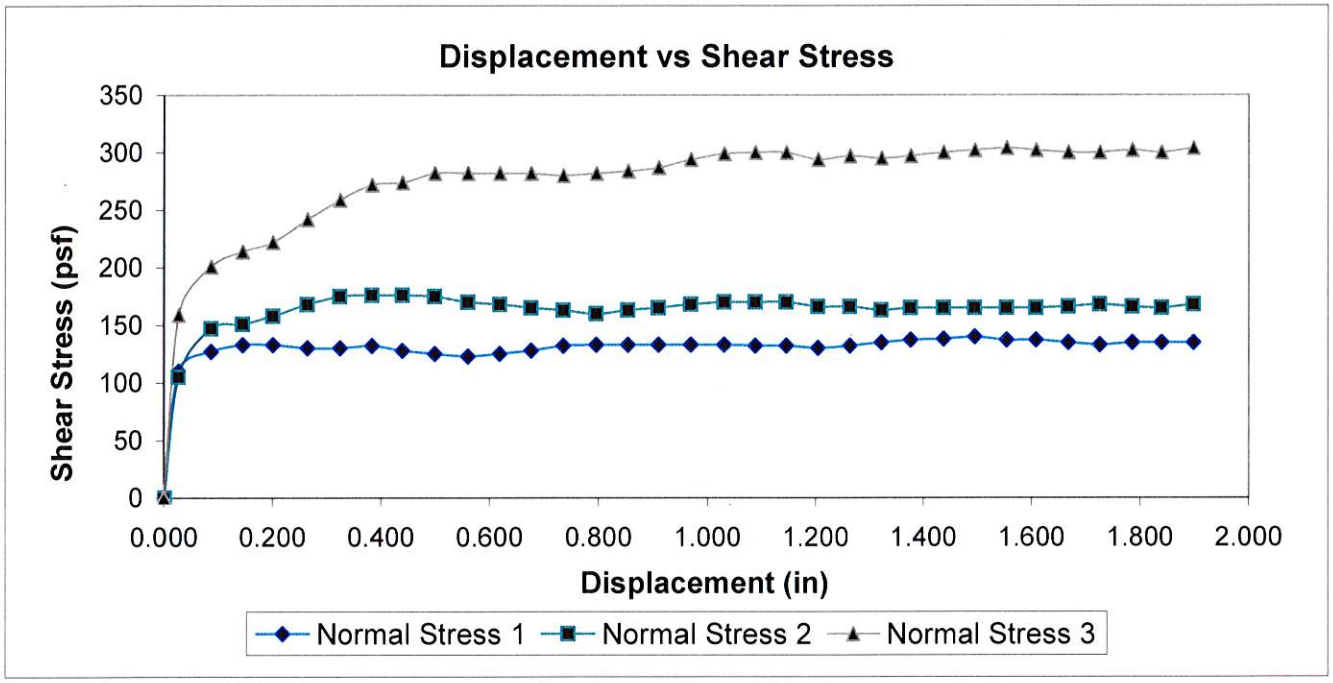
Material Type: 40 PVC

Roll Number: 3

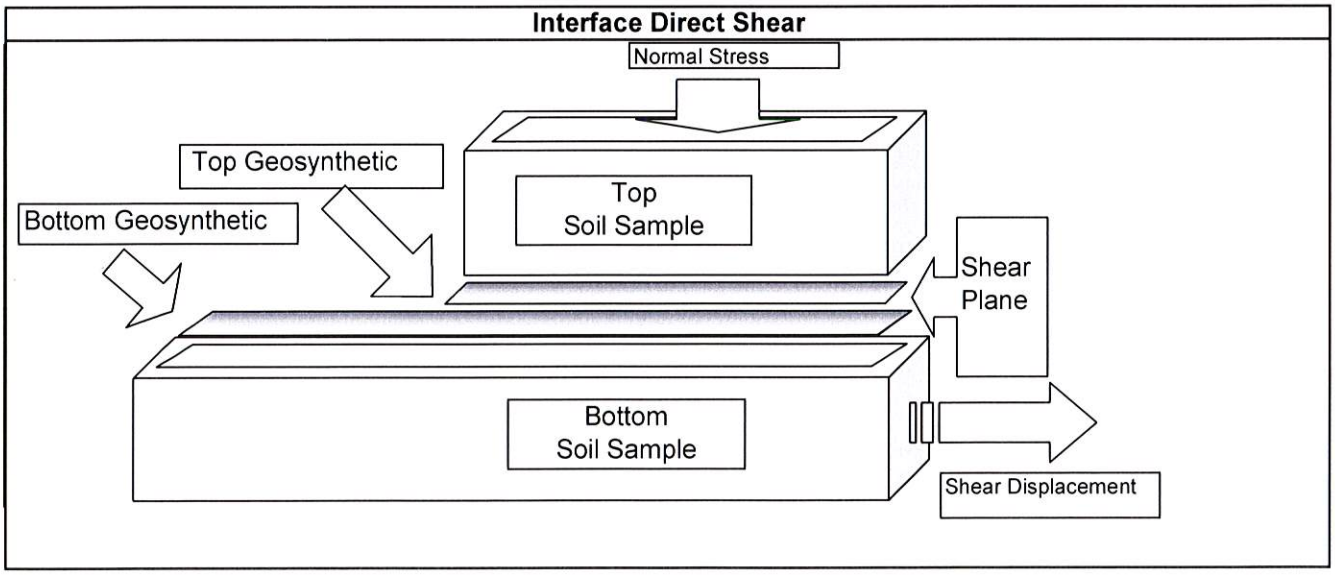
Lot Number: FMA0037-3

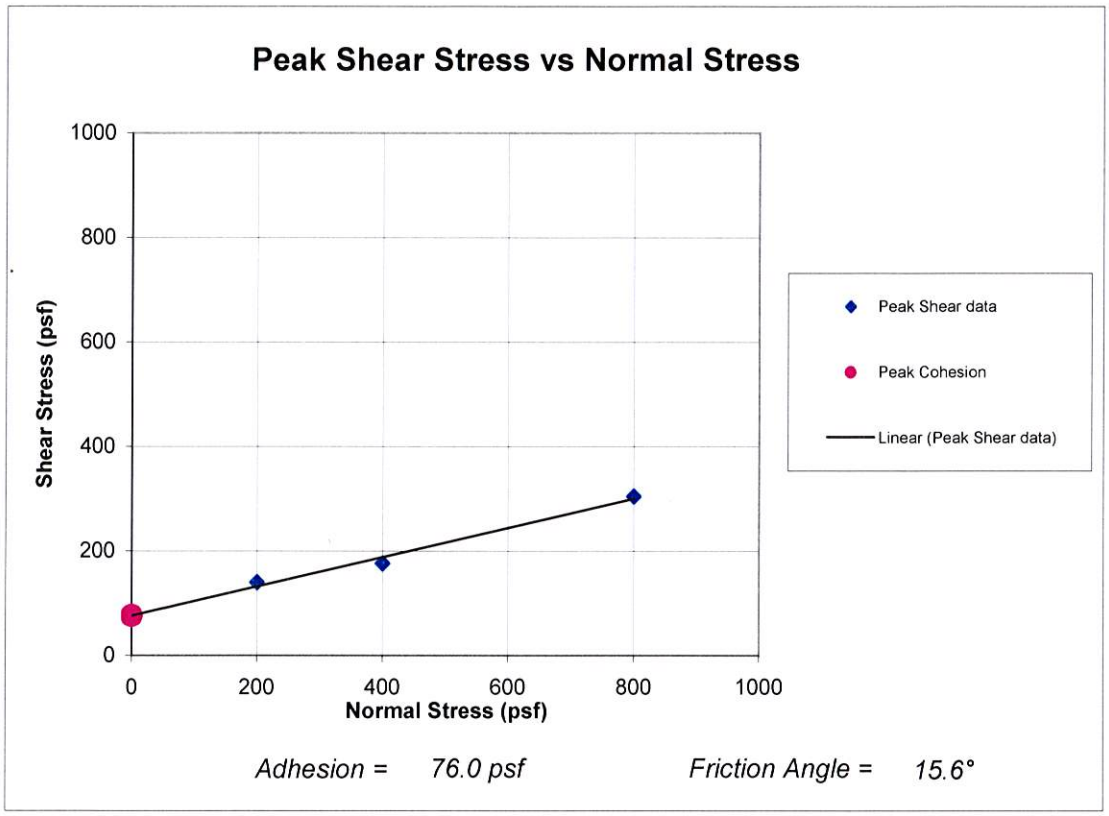
Interface

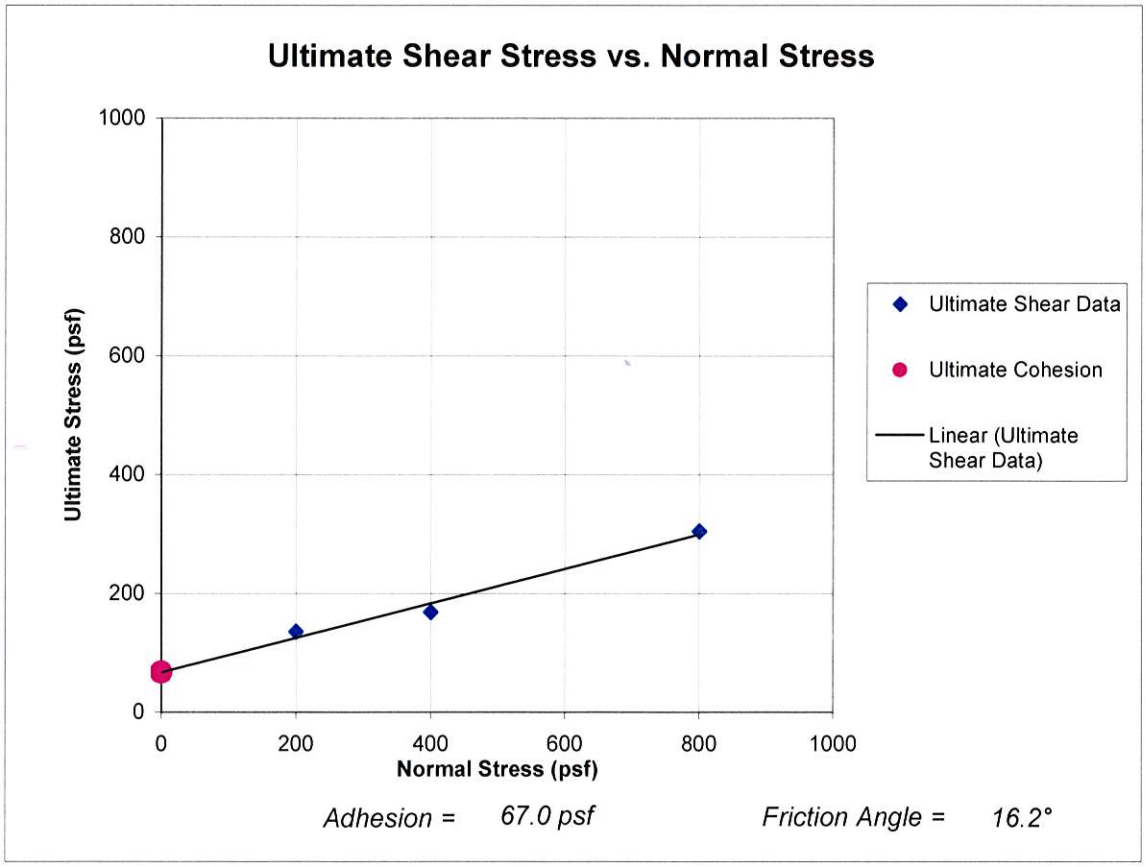
Top Geosynthetic vs. Bottom Geosynthetic



Diagram







ATT
ADVANCED TERRA TESTING

Attachment D-8

Cover Design, Geocomposite Capacity Requirement

ATTACHMENT D-8

MIDNITE MINE REMEDIAL ACTION COVER DESIGN GEOCOMPOSITE CAPACITY REQUIREMENT

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*11-Dec-12*	*30% Design*	*Melanie Davis*	*Tom Kelley*	*11-Dec-12*	*Tom Kelley*
1	*20-Nov-13*	*60% Design*	*Jean-Baptiste Varnier*	*Melanie Davis*	*03-Dec-13*	*Tom Kelley*
2	*30-May-14*	*90% Design (minor text changes)*	*Jean-Baptiste Varnier*	*Melanie Davis*	*20-Jun-14*	*Tom Kelley*
2	*14-May-15*	*100% Design – no changes*				

Location and Format
Electronic copies of these calculations are located in the project files system at: *\\usslc1s01\IFO\Industrial Projects\MIDNITE MINE\Deliverables_Working Documents\Basis of Design Rpt\Appendix D - Mine Waste Excavation and Containment\90% Submittal\Att D-4 Infiltration Analysis* *The following calculations were generated using the following software:* *Microsoft Office Professional Plus 2010 (Microsoft Excel)*

Supplement
Supplement D-8.1 – Geocomposite Capacity Calculations

1.0 BACKGROUND

This calculation brief has been prepared to estimate the seepage transmission capacity requirement for the geocomposite drainage layer (GDL) that is proposed for placement over the steeper sloped surfaces as part of the cover system for the Remedial Action Construction at the Midnite Mine Superfund Site (the Site). The cover system will be placed over mine waste that will be consolidated in the Waste Containment Area which includes: Pit 3 (including the Backfilled Pits Area, or BPA) and Pit 4, and the regraded surface of Area 5 (area between Pits 3 and 4). This calculation brief provides the methods, input parameters, and results of analyses for determining the capacity requirements for the GDL that will be placed over the geomembrane on sloped portions of the Waste Containment Area.

The side slopes for the Waste Containment Area cover will include drainage benches at a typical 50-foot vertical spacing. The interbench slopes will typically range from 3H:1V (Horizontal to Vertical) to 4H:1V. The benches will be shaped to provide drainage channels on the bench surfaces to prevent over-crest runoff as shown in Section 4 of the Drawings. These drainage channels will slope toward the edges of the cover to convey surface water and overliner seepage collected in the GDL off of the waste containment surfaces.

The proposed cover system will consist of a uniform soil layer overlying a geomembrane. On steeper sloped areas (15 percent or steeper) of mine waste, the cover system will also include a GDL between the soil and geomembrane layers to reduce potential pore pressure build up in the slope and increase slope stability. Slopes less than 15 percent will be stable based on veneer stability analyses (Attachment D-7) even if a GDL is not provided and fully saturated conditions develop within the cover soil profile. However, a GDL will be installed on the relatively flat top section of Pit 3, where the slope forms a shallow drainage swale and concentrated flows may occur, and could potentially result in an increase in percolation through the cover if internal drainage is not provided. It is assumed that the GDL will consist of non-woven, needle-punched geotextile heat-bonded to a geonet.

Material from the Rhoads Property Borrow Area located adjacent to the Site is analyzed as the soil cover borrow material in this calculation brief. The predominant soil in this borrow area consists of clayey sand (MGC, 2011a, b). The upper layer of bedrock consists of weathered quartz monzonite which has weathered to a residual soil (residuum). It is assumed that both materials may be used for the cover system. A minimum thickness of 1.8 feet is required for this cover soil based on radon emanation modeling results (refer to Attachment D-3); however, a minimum soil cover thickness of 3.0 feet has been proposed based on construction and slope stability considerations.

The preliminary design for the cover system called for the top 0.5 feet of the soil cover to be topsoil or amended borrow soil to promote vegetation growth. However based upon data presented in the Borrow Source Design Investigation Report (MGC, 2011a), it does not appear that amendment of soil from the Rhoads Property Borrow Area will be necessary to promote vegetation growth (see the Revegetation Plan, Attachment D-12). Therefore, for the purposes of these analyses, the top 0.5 feet of the cover system is assumed to have similar material properties as the underlying soil cover material. Additional information on the borrow source is provided in MGC (2011a, b).

2.0 METHODS

The design methods used for estimating the acceptable transmissivity of a geocomposite drain is presented in Koerner (2005). As presented by Koerner, the following equations were used to calculate the allowable transmissivity of the GDL for the cover system:

$$\theta_{allow} = FS \times \theta_{reqd}$$

Where:

FS = factor of safety, assumed as 2.0 based on recommendations in Koerner (2005)
θ_{allow} = allowable transmissivity
θ_{reqd} = required transmissivity

The required transmissivity was calculated using the equation listed below:

$$\theta_{reqd} = (q \times L)/i$$

Where:

q = percolation rate into geocomposite (m/sec)
L = horizontal slope length (m)
i = gradient of the flow within the geocomposite = $\sin\beta$, where β is the slope angle in degrees

The steeper side slopes of the cover (greater than 15 percent) are the critical slopes of concern for GDL capacity due to potential slope stability issues if pore pressure builds up in the slope. To minimize the potential for ponding to occur on the steeper side slopes of the cover, the GDL capacity was estimated using: (1) an infiltration model that allowed surface water to pond on the ground surface; and (2) wet period climate conditions. The peak weekly percolation rate into the GDL was calculated at the base of the soil cover as part of the infiltration analyses (ponding analysis with seepage face boundary at the base of soil cover presented in Attachment D-4) and was used to estimate the required GDL capacity. The results present an upper bound value for the required GDL capacity. The peak weekly percolation rate calculated from the infiltration analysis is 29.73 millimeters/day. Two slope sections were evaluated and included: (1) the longest length for the steepest slope (170.8 feet at 3H:1V); and (2) the longest length for slopes steeper than 15 percent. For the detailed grading plan developed as part of the 90% Design, the longest slope steeper than 15 percent has a length of 248 ft and a slope of 18.6 percent. As surface ponding is unlikely for either of these slope configurations, these GDL capacity analyses are considered to be conservative.

When evaluating the acceptability of transmissivity values obtained from laboratory testing, the allowable transmissivity values calculated above should be increased to account for long-term factors such as intrusion, creep, chemical clogging, and biological clogging, as reflected in the following equation:

$$\theta_{ult} = \theta_{allow} \times [RF_{IN} \times RF_{CR} \times RF_{CC} \times RF_{BC}]$$

Where:

θ_{ult}= ultimate (acceptable) transmissivity determined from laboratory testing
θ_{allow} = allowable transmissivity
RF_{IN} = reduction factor for intrusion of the adjacent geosynthetics into the geonet's core space, 1.4
RF_{CR} = reduction factor for creep deformation, 1.25
RF_{CC} = reduction factor for chemical clogging, 1.1
RF_{BC} = reduction factor for biological clogging, 1.75

The values listed above for the reduction factors are from Table 4.2 in Koerner (2005) for covers and were used in calculating the ultimate (acceptable) transmissivity for the GDL.

3.0 RESULTS

Calculations for the minimum required laboratory-measured transmissivity value for the GDL were performed as described in the preceding section. Calculation sheets are provided in the attached Supplement D-8.1 and include the input values selected for each case evaluated. Table 1 lists the minimum recommended laboratory measured transmissivity values for the geocomposite drainage layer.

Table 1. Summary of Results

Slope	Gradient	Cover Loading (psf)	Recommended Minimum Laboratory Measured Transmissivity, θ_{ult} (m^2/s), for GDL
5.4H:1V	0.2	< 1000	9.3×10^{-4}
3H:1V	0.3	< 1000	3.4×10^{-4}

m^2/s = square meters per second; GDL = geocomposite drainage layer

4.0 REFERENCES

Koerner, R., 2005. Designing with Geosynthetics. 5th Edition. Pearson Prentice Hall, Upper Saddle River, NJ.

Miller Geotechnical Consultants (MGC), 2011a. Borrow Source Design Investigation Report. Revision 2. Prepared on behalf of Newmont USA Limited and Dawn Mining Company for U.S. Environmental Protection Agency, Region 10. May 6.

Miller Geotechnical Consultants (MGC), 2011b. Technical Memorandum – Rhoads Property Borrow Investigation, Phase II. Revision 1. Prepared on behalf of Newmont USA Limited and Dawn Mining Company for U.S. Environmental Protection Agency, Region 10. December 2.

Supplement D-8.1

Geocomposite Capacity Calculations

Project:	Midnite Mine	Job No.:	1011322
Description:	Remedial Action Design	Date:	11/20/2013
Detail:	Geocomposite Drainage Layer Design	Computed By:	J.B. Varnier
		Checked By:	M.Davis

Objective: Determine required transmissivity for geocomposite drainage layer on cover slopes (5.4H:1V slope)

Assumptions: Geocomposite consists of a nonwoven, needle-punched geotextile heat-bonded to an underlying geonet. Cover soil is from Rhoads Property Borrow Area. Wet period climate conditions. Peak weekly flux rate through cover used for calculations.

Method: Use procedures presented in Koerner (2005)

Equations: Factory of Safety (FS) = $\theta_{allow}/\theta_{reqd}$

$\theta_{allow} = \theta_{reqd} \times FS$

$\theta_{reqd} = (q \times L)/i$

$\theta_{ult} = \theta_{allow} \times [RF_{IN} \times RF_{CR} \times RF_{CC} \times RF_{BC}]$

θ_{allow} = allowable transmissivity, m^2/s
θ_{reqd} = required transmissivity from design, m^2/s
q = percolation rate into geocomposite (m/sec)
L = horizontal slope length (m)
i = gradient of the flow within the geocomposite = sin β, where β is the slope angle
θ_{ult} = ultimate transmissivity from laboratory testing
RF_{IN} = reduction factor for intrusion of the geosynthetics into the geonet's core space
RF_{CR} = reduction factor for creep deformation
RF_{CC} = reduction factor for chemical clogging
RF_{BC} = reduction factor for biological clogging

θ_{reqd} (m^2/s)

q, maximum weekly percolation rate through cover (mm/day)	29.73	From infiltration modeling for cover soil from Rhoads Property Borrow Area (MWH, 2013)
q, maximum weekly percolation rate through cover (m/s)	3.44E-07	Calculated
l, slope length (ft)	248.0	Longest slope length from 60 percent design for slopes >15%
L, horizontal (interbench) slope length (ft)	243.9	Calculated
L, horizontal (interbench) slope length (m)	74.3	Calculated
i, gradient	0.19	From 60 percent design, 5.4H:1V slope
β, slope angle (degrees)	10.5	Calcuated based on gradient
θ_{reqd} (m^2/s)	1.38E-04	Calculated

θ_{all}(m^2/s)

Factor of Safety	2	Koerner (2005)
θ_{allow} (m^2/s)	2.76E-04	Calculated

θ_{ult}(m^2/s)

RF_{IN}	1.4	From Table 4.2 in Koerner (2005) for covers
RF_{CR}	1.25	From Table 4.2 in Koerner (2005) for covers
RF_{CC}	1.1	From Table 4.2 in Koerner (2005) for covers
RF_{BC}	1.75	From Table 4.2 in Koerner (2005) for covers
The laboratory measured transmissivity, θ_{ult}(m^2/s), to meet a FS of 2.0 =	**9.3E-04**	Calculated

Note: Cover loading for transmissivity is estimated as 330 psf based on a 3' cover thickness and a total unit weight of 110 pcf (based on design).

References:

Koerner, R. 2005. Designing with Geosynthetics. 5th Edition.
MWH, Americas, Inc. (MWH), 2013. Midnite Mine RA Cover Design Infiltration Modeling, Attachment D-3 of Appendix D of the 60% BODR. December.

Project:	Midnite Mine	Job No.:	1011322
Description:	Remedial Action Design	Date:	11/20/2013
Detail:	Geocomposite Drainage Layer Design	Computed By:	J.B. Varnier
		Checked By:	M.Davis

Objective: Determine required transmissivity for geocomposite drainage layer on cover slopes (3H:1V slope)

Assumptions Geocomposite consists of a nonwoven, needle-punched geotextile heat-bonded to an underlying geonet. Cover soil is from Rhoads Property Borrow Area. Wet period climate conditions. Peak weekly flux rate through cover used for calculations.

Method: Use procedures presented in Koerner (2005)

Equations: Factory of Safety (FS) = $\theta_{allow}/\theta_{reqd}$

$\theta_{allow} = \theta_{reqd} \times FS$

$\theta_{reqd} = (q \times L)/i$

$\theta_{ult} = \theta_{allow} \times [RF_{IN} \times RF_{CR} \times RF_{CC} \times RF_{BC}]$

θ_{allow} = allowable transmissivity, m^2/s
θ_{reqd} = required transmissivity from design, m^2/s
q = percolation rate into geocomposite (m/sec)
L = horizontal slope length (m)
i = gradient of the flow within the geocomposite = $\sin\beta$, where β is the slope angle
θ_{ult} = ultimate transmissivity from laboratory testing
RF_{IN} = reduction factor for intrusion of the geosynthetics into the geonet's core space
RF_{CR} = reduction factor for creep deformation
RF_{CC} = reduction factor for chemical clogging
RF_{BC} = reduction factor for biological clogging

θ_{reqd} (m^2/s)

q, maximum weekly percolation rate through cover (mm/day)	29.73	From infiltration modeling for cover soil from Rhoads Property Borrow Area (MWH, 2013)
q, maximum weekly percolation rate through cover (m/s)	3.44E-07	Calculated
l, slope length (ft)	170.8	Longest slope length from 60 percent design for steepest slopes (3H:1V)
L, horizontal (interbench) slope length (ft)	162.0	Calculated
L, horizontal (interbench) slope length (m)	49.4	Calculated
i, gradient	0.33	From 60 percent design
β, slope angle (degrees)	18.4	Calcuated based on gradient
θ_{reqd} (m^2/s)	5.10E-05	Calculated

θ_{all}(m^2/s)

Factor of Safety	2	Koerner (2005)
θ_{allow} (m^2/s)	1.02E-04	Calculated

θ_{ult}(m^2/s)

RF_{IN}	1.4	From Table 4.2 in Koerner (2005) for covers
RF_{CR}	1.25	From Table 4.2 in Koerner (2005) for covers
RF_{CC}	1.1	From Table 4.2 in Koerner (2005) for covers
RF_{BC}	1.75	From Table 4.2 in Koerner (2005) for covers
The laboratory measured transmissivity, θ_{ult}(m^2/s), to meet a FS of 2.0 =	**3.4E-04**	Calculated

Note: Cover loading for transmissivity is estimated as 330 psf based on a 3' cover thickness and a total unit weight of 110 pcf (based on design).

References:

Koerner, R. 2005. Designing with Geosynthetics. 5th Edition.
MWH, Americas, Inc. (MWH), 2013. Midnite Mine RA Cover Design Infiltration Modeling, Attachment D-3 of Appendix D of the 60% BODR. December.

Attachment D-9

Filter and Hydraulic Conductivity Calculations for Pit 3 and Pit 4 Sub-Waste Liner Bedding and Drain Gravel

ATTACHMENT D-9

MIDNITE MINE REMEDIAL ACTION
Filter and Hydraulic Conductivity Calculations for Pit 3 and Pit 4 Sub-Waste Liner Bedding and Drain Gravel

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*02-Dec-13*	*60% Design*	*Chad Tomlinson*	*Melanie Davis*	*04-Dec-13*	*Tom Kelley*
0	*14-May-15*	*90% and 100% Design – no changes*				

Location and Format

Electronic copies of these calculations are located in the project files system at:

\\usftc2s01\Projects\Newmont\Midnite Mine_2011\6.0 Studies & Reports\6.2 Technical\Remedial Action Liner Design\Liner Bedding and Drainage Gravel Design

The following calculations were generated using the following software:

Microsoft Office Professional Plus 2010 (Microsoft Excel)

Supplements

Supplement D-9.1 – Filter Gradation Calculations
Supplement D-9.2 – Hydraulic Conductivity Calculations

1.0 BACKGROUND

This calculation brief has been prepared to evaluate whether the gradations specified for the drain gravel and sub-waste liner bedding materials meet filter compatibility requirements between these two materials. Hydraulic conductivity estimates for these materials are also provided. The sub-waste geomembrane liner will be placed over the pit-bottom underdrain layer in Pit 3 and Pit 4 as part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The pit-bottom underdrain will consist of angular, gravel-sized material (drain gravel). A finer-grained gravel layer will be placed as liner bedding between the geomembrane and drain gravel to reduce the potential for puncturing the geomembrane as mine waste is placed over the geomembrane. The gradation requirements for this liner bedding layer was identified in the liner puncture calculations summarized in Attachment D-1 to Appendix D.

2.0 METHODS

2.1 FILTER COMPATABILITY

The design methods used to evaluate the filter compatibility between the drain gravel drain and liner bedding material are based on guidance provided in Chapter 26 – Gradation Design of Sand and Gravel Filters of the United States Department of Agriculture (USDA) Part 633 National Engineering Handbook (NEH) (USDA, 1994). As the basis of the filter calculations, the liner bedding was considered as the base soil. The maximum and mean particle size criteria for the liner bedding material were developed as part of liner puncture protection calculations (see Attachment D-1 to Appendix D) are summarized in Table 1 and were used to develop gradation specifications for the liner bedding material.

Table 1. Liner Bedding Gradation Requirements for Puncture Resistance

U.S. Standard Sieve Size	Opening Size (mm)	Minimum Percent Passing (%)
1 ½ inch	38.1	100
¾ inch	19.05	50

A liner bedding gradation conforming to the gradation for American Society for Testing and Materials (ASTM) C33 No 56 Coarse Aggregates was selected to meet the gradation requirements listed in Table 1. The gradation specifications for the liner bedding material are summarized in Table 2.

Table 2. Liner Bedding Gradation Specifications[a]

U.S. Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
1 1/2-inch	38.1	100	100
1-inch	25.4	100	90
3/4-inch	19.05	85	40
1/2 inch	12.7	40	10
3/8-inch	9.5	15	0
No. 4	4.76	5	0

Notes: [a/] Gradation based ASTM C33 No 56 Coarse Aggregate

A drain gravel gradation was selected that roughly conforms, at its finer limit to the coarse limit of the liner bedding material specification, and at its coarse limit is restricted to a 3-inch maximum particle size in accordance with NEH recommendations to minimize the potential for particle segregation during placement. The proposed drain gravel specification is summarized in Table 3.

Table 3. Underdrain Drainage Rock Gradation Requirements

U.S Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
3-inch	76.2	100	100
1 1/2-inch	38.1	100	0
1-inch	25.4	90	0
3/4-inch	19.05	40	0
1/2 inch	12.7	20	0
3/8-inch	9.5	10	0
No. 4	4.76	5	0
No. 200	0.075	5	0

The filter compatibility calculations and gradation chart for the liner bedding, required filter band and the specified gravel drain rock gradation are presented in Supplement D-9.1. The primary concern regarding filter compatibility is on preventing particle migration from the finer-grained bedding layer into the coarser-grained drain gravel layer. Grain-sized based hydraulic conductivity estimates were made using procedures summarized in Section 2.2 that these materials (the bedding layer as well as the drain gravel) have sufficient hydraulic conductivity to function as drain materials. The filter compatibility calculations are presented in Supplement D-9.1 and summarized in Section 3.0.

2.2 ESTIMATION OF HYDRAULIC CONDUCTIVITY

Estimates of the hydraulic conductivity of both the gravel drain rock and liner bedding were made for the specified gradations. For this estimation, the average of results from calculations using 1) the Fair-Hatch equation and 2) the Harlman Equation were utilized to estimate the intrinsic permeability and are presented below:

Fair-Hatch Equation (Equation 3-25, McWhorter & Sunada, 1977)

$$k = 1/A[(1-\emptyset)^2/\emptyset^3 * (B/100*\sum F/d_m)^2]^{-1}$$

Where:

k = intrinsic permeability [cm^2]
A = Packing Factor = ±5
B = Particle Shape Factor [7 for crushed rock]
F =% by weight between two particle sizes
d_m = geometric mean of the particle sizes corresponding to F [cm]

Harlman (Equation 3-26, McWhorter & Sunada, 1977)

$$k = (6.54 * 10^4) * d_{90}^2$$

Where:

D_{90} = Particle size with 90% finer [cm]

The hydraulic conductivity was calculated using the intrinsic permeability calculated from the equations above and following relationship:

$$K = k * \rho g/\mu$$

Where:

K = Hydraulic Conductivity [cm/sec]
ρ = density of water [1 g/cm^3]
g = gravitational acceleration [981 cm/sec^2]
μ = viscosity of water [1 g/cm-s]

Calculations for hydraulic conductivity are presented in Supplement D-9.2 and results summarized in Section 3.0.

3.0 RESULTS

Based on the calculations and reasoning presented in Supplement D-9.1, the specified liner bedding and gravel drain rock gradations presented in Tables 2 and 3 demonstrate acceptable compatibility from a filter-criteria standpoint. Although the finer limit for the specified gradation for the drain gravel is finer the minimum specified limit based upon hydraulic conductivity recommendations, the coarse nature of the base soil (i.e. the bedding layer) make the minimum gradation limit irrelevant so long as the drain gravel is coarser than the fine limit of the bedding layer.

The estimated hydraulic conductivity of the liner bedding and gravel drain rock based on the calculations presented in Supplement D-9.2 are summarized in Table 4.

Table 4. Estimated Hydraulic Conductivities

Material	Minimum Hydraulic Conductivity (cm/sec)	Maximum Hydraulic Conductivity (cm/sec)
Liner Bedding [a/]	3×10^{-1}	1×10^{0}
Drain Gravel	7×10^{-1}	9×10^{0}

Notes: [a/] Intrinsic permeability calculations based on ASTM C33 No. 56 Coarse Aggregate

4.0 REFERENCES

American Society for Testing and Materials (ASTM), 2008. Designation ASTM C 33-07, Standard Specification for Concrete Aggregates. West Conshohocken, PA.

McWhorter, D.B. and D.K. Sunada, 1977. Ground-Water Hydrologry and Hydraulics. Water Resources Publications, Colorado

United States Department of Agriculture (USDA), 1994. Chapter 26. Gradation Design of Sand and Gravel Filters. Part 633. National Engineering Handbook. National Resource Conservation Service. Washington D.C.

Supplement D-9.1

Filter Gradation Calculations

Client: Dawn Mining Company/Newmont USA Limited
Project: Midnite Mine
Description: Remedial Action Pit 3 and Pit 4 Sub-Waste Liner Bedding and Underdrain Drain Gravel
Detail: Liner Bedding and Drain Gravel Design

Job No. 1011322
Date: 12/2/2013
Computed By: C. Tomlinson
Checked By: M. Davis

U.S and Metric Sieve Sizes

Sieve Size	Size [mm]	%Pass
12-inch	304.8	0
	304.8	100
6-inch	152.4	0
	152.4	100
3-inch	76.2	0
	76.2	100
1-1/2 inch	38.1	0
	38.1	100
1	25.4	0
	25.4	100
3/4 inch	19.05	0
	19.05	100
No.4	4.75	0
	4.75	100
No. 10	2	0
	2	100
No.20	0.84	0
	0.84	100
No.60	0.25	0
	0.25	100
		0
No. 140	0.105	0
	0.105	100
No. 200	0.075	0
	0.075	100

Max Liner Bedding Size based on liner puncture requirements

U.S.Sieve Size	Size Opening [mm]	%Pass
1 1/2-inch	38.1	100
3/4 inch	19.05	50
3/4 inch	19.05	0

Liner Bedding Gradation (ASTM C33, No. 56 Coarse

	Sieve Size	Size Opening [mm]	Min %Pass	Max %Pass
	1 1/2-inch	38.1	100	100
	1 inch	25.4	90	100
	3/4 inch	19.05	40	85
	1/2 inch	12.7	10	40
	3/8 inch	9.5	0	15
	No. 4	4.76	0	5

Correction Factor = 100/Percent Passing No.4 Sieve
Correction Factor = Regrading not Required due to lack of material finer than #4

Corrected Filter Gradation (Not Required)

	Sieve Size	Sieve (mm)	% passing
	1 1/2- inch	38.1	100
	3/4 inch	19.05	90
	No.4	4.76	40
	No.10	2	10
	No. 20	0.84	0
	No. 60	0.25	0
	No. 140	0.105	0
	No. 200	0.074	0

Specified Gravel Filter Criterion

U.S. Sieve Size	Size [mm]	%Pass Max	% Passing Min
3-inch	76.2	100	100
1 1/2-inch	38.1	100	0
1-inch	25.4	90	0
3/4-inch	19.05	40	0
1/2 inch	12.7	20	0
3/8-inch	9.5	10	0
No. 4	4.76	5	0
No. 200	0.075	5	0

Based on Finer Gradation C33 No 56 Coarse Material
Filter Calcs, Based Upon USBR Class III (Choke Filter) Criteria

d_{85base} = 19.05 mm

$D_{15Filter(max)}$ 76.2 $4*d_{85base} \geq D_{15}$, for Category 4 Base Soil Control Point 1

Since particle migration is primary concern (permeability of base soil bedding sufficient for drain)

5 $D_{15max}/D_{15min} \leq 5$ per NRCS mm

$D_{15FIlter\ (min)}$ 15.24 mm Control Point 2

To prevent gap graded range filter, Coefficient of Uniformity of 6 and ratio of max to min dia at all % passing values of 60 or less < 5

$D_{10max} = D_{15max} / 1.2 =$ 63.5 mm

$D_{60max} = D_{10max} * 6 =$ 381.00 mm Control Point 3

$D_{60min} = D_{60max} / 5 =$ 76.20 mm Control Point 4

Determine minimum and maximum particle size (D_5 and D_{100})

D_5 = greater than or equal to 0.075 mm Control Point 5

D_{100} = 76.2 mm Control Point 6

To minimize segregation, need to define relationship between D_{90max} and D_{10min}

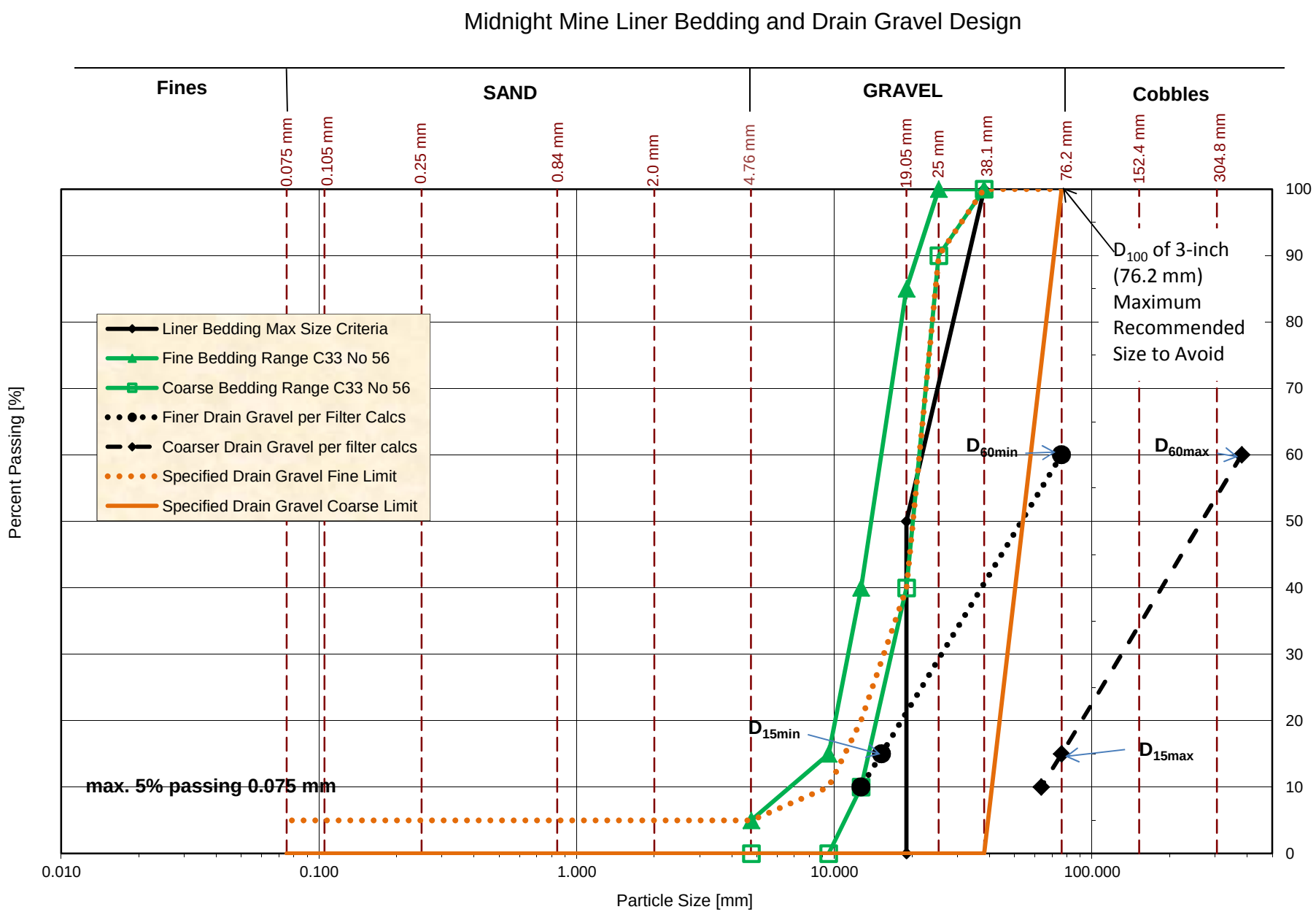
$D_{10min} = D_{15min} / 1.2$ 12.7

Since D_{10min} is 12.7 mm then maximum D_{90} for the filter is 60 Control Point 7 from Table 26-6

Connect Control Points 4,2 and 5 to form design for the fine side of the filter band.
Connect Control Points 6,7,3 and 1 to form a partial design for the coarse side of the filter band.

Filter Design Criteria

Name	Size (mm)	% Passing	Description
Control Point 8		100	Extrapolated D_{100min}
Control Point 4	76	60	D_{60min}
Control Point 2	15.24	15	D_{15min}
Control Point 5	12.7	10	$D_{5\ Min}$
Control Point 6		100	D_{100}
Control Point 7	60	90	D_{90max}
Control Point 3	381	60	D_{60max}
Control Point 1	76.2	15	D_{15max}
	63.5	10	D10


Midnight Mine Liner Bedding and Drain Gravel Design
Fines
SAND
GRAVEL
Cobbles
0.075 mm
0.105 mm
0.25 mm
0.84 mm
2.0 mm
4.76 mm
19.05 mm
25 mm
38.1 mm
76.2 mm
152.4 mm
304.8 mm
Liner Bedding Max Size Criteria
Fine Bedding Range C33 No 56
Coarse Bedding Range C33 No 56
Finer Drain Gravel per Filter Calcs
Coarser Drain Gravel per filter calcs
Specified Drain Gravel Fine Limit
Specified Drain Gravel Coarse Limit
D100 of 3-inch (76.2 mm) Maximum Recommended Size to Avoid
D60min
D60max
D15min
D15max
max. 5% passing 0.075 mm
Percent Passing [%]
Particle Size [mm]
0.010
0.100
1.000
10.000
100.000
0
10
20
30
40
50
60
70
80
90
100

Supplement D-9.2

Hydraulic Conductivity Calculations

Client: Dawn Mining Company/Newmont USA Limited
Project: Midnite Mine
Description: Remedial Action Pit 3 and Pit 4 Sub-Waste Liner Bedding and Underdrain Drain Gravel
Detail: Liner Bedding and Drain Gravel Hydraulic Conductivity Estimates

Job No. 1011322
Date: 12/2/2013
Computed By: C. Tomlinson
Checked By: M. Davis

Fair-Hatch Equation (Equation 3-25, McWhorter & Sunata, 1977)

$$k = 1/A[(1-\emptyset)^2/\emptyset^3 * (B/100*\sum F/d_m)^2]^{-1}$$

$$K = k * \mathbf{r}g/\mathbf{m}$$

k = intrinsic permeability [cm^2]
A = Packing Factor = 5 +/-
B = Particle Shape Factor = 6 (spherical) to 7.7(highly angular), use 7 for crushed rock.
F = % by weight between two particle sizes.
d_m = Geometric mean of particle sizes corresponding to F [cm]
K - Hydraulic Conductivity (cm/s)
r = density of water = 1 g/cm3
g = gravitational acceleration 981 cm/s^2
m = viscosity of water 1 g/cm-s

Harlman (Equation 3-26, Mcwhorter & Sunata, 1977)

$$k = (6.54 * 10^4) * d_{90}^2$$

k = intrinsic permeability [cm^2]
d_{90} = Particle size with 90% finer [cm]

Material: Bedding Layer ASTM C33, No. 56 Fine Limit

A = 5 ϕ = 0.33 B = 7

Sieve	Size (mm)	% Finer	d_m (cm)	F (%)	F/d_m
1 1/2-inch	38.1	100			
1 inch	25.4	100	3.11	0	0.00
3/4 inch	19.05	85	2.20	15	6.82
1/2 inch	12.7	40	1.56	45	28.93
3/8 inch	9.5	15	1.10	25	22.76
No. 4	4.76	5	0.67	10	14.87
No. 200	0.075	3	0.06	2	33.47

$\sum F/d_m$ = 106.85

k = 0.000286182 cm^2 Fair-Hatch
d_{90} (cm) = 0.7 k = 0.000266523 cm^2 Harlman
K = 3.E-01 cm/s Fair-Hatch
K = 3.E-01 cm/s Harlman
Mean K = 3.E-01 cm/s

Material: Drain Gravel, Fine Limit

A = 5 ϕ = 0.33 B = 7

Sieve	Size (mm)	% Finer	d_m (cm)	F (%)	F/d_m
3-inch	76.2	100			
1 1/2-inch	38.1	100	5.39	0	0.00
1-inch	25.4	90	3.11	10	3.21
3/4-inch	19.05	40	2.20	50	22.73
1/2 inch	12.7	20	1.56	20	12.86
3/8-inch	9.5	10	1.10	10	9.10
No. 4	4.76	5	0.67	5	7.44
No. 200	0.075	5	0.06	0	0.00

$\sum F/d_m$ = 55.34

k = 0.001067 cm^2 Fair-Hatch
d_{90} = 0.95 k = 0.000491 cm^2 Harlman
K = 1.E+00 cm/s Fair-Hatch
K = 5.E-01 cm/s Harlman
Mean K = 7.E-01 cm/s

Material: Bedding Layer ASTM C33, No. 56 Coarse Limit

A = 5 ϕ = 0.33 B = 7

Sieve	Size (mm)	% Finer	d_m (cm)	F (%)	F/d_m
1 1/2-inch	38.1	100			
1 inch	25.4	90	3.11	10	3.21
3/4 inch	19.05	40	2.20	50	22.73
1/2 inch	12.7	10	1.56	30	19.29
3/8 inch	9.5	0	1.10	10	9.10
No. 4	4.76	0	0.67	0	0.00
No. 200	0.075	0	0.06	0	0.00

$\sum F/d_m$ = 54.34

k = 0.001106742 cm^2 Fair-Hatch
d_{90} = 1.27 k = 0.000877296 cm^2 Harlman
K = 1.E+00 cm/s Fair-Hatch
K = 9.E-01 cm/s Harlman
Mean K = 1.E+00 cm/s

Material: Drain Gravel, Coarse Limit

A = 5 ϕ = 0.33 B = 7

Sieve	Size (mm)	% Finer	d_m (cm)	F (%)	F/d_m
3-inch	76.2	100			
1 1/2-inch	38.1	0	5.39	100	18.56
1-inch	25.4	0	3.11	0	0.00
3/4-inch	19.05	0	2.20	0	0.00
1/2 inch	12.7	0	1.56	0	0.00
3/8-inch	9.5	0	1.10	0	0.00
No. 4	4.76	0	0.67	0	0.00
No. 200	0.075	0	0.06	0	0.00

$\sum F/d_m$ = 18.56

k = 0.009487 cm^2 Fair-Hatch
d_{90} = 4 k = 0.008703 cm^2 Harlman
K = 9.E+00 cm/s Fair-Hatch
K = 9.E+00 cm/s Harlman
Mean K = 9.E+00 cm/s

Attachment D-10

Filter Calculations for Pit 3 and Pit Waste Dewatering Sumps

ATTACHMENT D-10

MIDNITE MINE REMEDIAL ACTION
Filter Calculations for Pit 3 and Pit 4 Waste Dewatering Sumps

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*2-Dec-13*	*60% Design*	*Chad Tomlinson*	*Melanie Davis*	*04-Dec-13*	*Tom Kelley*
0	*14-May-15*	*90% and 100% Design – no changes*				

Location and Format

Electronic copies of these calculations are located in the project files system at:

\\usftc2s01\Projects\Newmont\Midnite Mine_2011\6.0 Studies & Reports\6.2 Technical\Remedial Action Liner Design\Dewatering Sumps

The following calculations were generated using the following software:

Microsoft Office Professional Plus 2010 (Microsoft Excel)

Supplement

Supplement D-10.1 – Filter Gradation Calculations for Dewatering Sumps

1.0 BACKGROUND

This Attachment has been prepared to summarize the evaluation of filter compatibility between materials within the waste dewatering sumps that will be constructed as part of RA construction during Pit 3 and Pit 4 backfilling. The waste dewatering sumps will be used to collect and remove water that percolates through the waste rock, primarily during backfilling operations before placement of the final cover over the waste. The sumps will overlay the sub-waste geomembrane liner which will be placed over the pit-bottom underdrain layer in Pit 3 and Pit 4 as part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The waste dewatering sump backfill will consist of angular, gravel-sized material (drain gravel). The 30% design called for a geofabric to separate and prevent fines migration from the overlying waste rock into the drain gravel. However, due to concerns regarding the potential for plugging and longevity of the geofabric that were raised during the review of the 30% design, the design of the waste dewatering sumps has been revised to replace the geofabric with natural aggregates. It is proposed that drain gravel in the waste dewatering sump will have the same gradation as the drain gravel being used for the underdrain collection system. To segregate the drain gravel from the waste, a layer of ASTM C33 Fine Aggregate will be placed as a filter layer between the drain gravel and waste rock. In addition, the results of this evaluation indicate that an intermediate filter layer having a gradation consistent with ASTM C33 No.67 Coarse Aggregate will be needed between the drain gravel and the finer-grained (ASTM C33) waste filter sand.

1.1 METHODS

The design methods used to develop the filter design for the dewatering sump were based on guidance provided in Chapter 26 – Gradation Design of Sand and Gravel Filters of the United States Department of Agriculture (USDA) Part 633 National Engineering Handbook (USDA, 1994). As the basis of the filter calculations, a filter sand having a gradation conforming to requirements for ASTM C33 Fine Aggregate was considered as the base soil. The gradation of ASTM C33 Fine Aggregate is provided in Table 1.

Table 1. ASTM C33 Fine Aggregate Particle Size Distribution

U.S. Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
3/8 inch	9.5	100	100
No.4	4.75	95	100
No.8	2.36	80	100
No. 16	1.18	50	85
No. 30	0.6	25	60
No. 50	0.3	5	30
No. 100	0.15	0	10
No. 200	0.075	0	5

In order to provide for consistency of materials during construction, the same gravel gradation used for the drain gravel in the underdrain system (see Attachment D-9) was selected for use in the waste dewatering sump. The gradation of the drain gravel is provided in Table 2.

Table 2. Drain Gravel Gradation

U.S. Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
3-inch	76.2	100	100
1 1/2-inch	38.1	100	0
1-inch	25.4	90	0
3/4-inch	19.05	40	0
1/2 inch	12.7	20	0
3/8-inch	9.5	10	0
No. 4	4.76	5	0
No. 200	0.075	5	0

Based on a review of Table 1 and 2 above and the grain size distribution curves provided in Supplement D-10.1, an intermediate filter is needed to provide filter compatability between the drain gravel and ASTM C33 Fine Aggregate filter sand layer. The filter calculations and gradation chart developed to specify the 2nd stage filter gradation are presented in Supplement D-10.1 and the results are summarized in Section 1.2.

1.2 RESULTS

Based on the calculations and rationale presented in Supplement D-10.1, a gradation consistent with an ASTM C33 No.67 Coarse Aggregate was determined to be suitable for use as the intermediate filter between the drain rock and ASTM C33 Fine Aggregate filter sand layer. The gradation for the ASTM C33 No.67 Coarse Aggregate is presented in Table 3.

Table 3. ASTM C33 No.67 Coarse Aggregate Gradation

U.S. Standard Sieve Size	Opening Size (mm)	Percent Passing (%)	
		Maximum	Minimum
1-inch	25.4	100	100
3/4-inch	19.05	100	90
1/2 inch	12.7	55	20
3/8-inch	9.5	10	0
No. 4	4.76	5	0

2.0 REFERENCES

U.S. Department of Agriculture (USDA), 1994. Chapter 26. Gradation Design of Sand and Gravel Filters. Part 633. National Engineering Handbook. National Resource Conservation Service. Washington D.C.

Supplement D-10.1

Filter Gradation Calculations for Dewatering Sumps

Client: Dawn Mining Company/Newmont USA Limited
Project: Midnite Mine
Description: Remedial Action Pit 3 and Pit 4 Dewater Sump
Detail: Drain Gravel and Fine Aggregate Filter Design

Job No. 1011322
Date: 12/2/2013
Computed By: C. Tomlinson
Checked By: M. Davis

US and Metric Sieve Sizes		
US Sieve Size	Size [mm]	%Pass
3-inch	76.2	0
	76.2	100
1-1/2 inch	38.1	0
	38.1	100
1	25.4	0
	25.4	100
3/4 inch	19.05	0
	19.05	100
No.4	4.75	0
	4.75	100
No. 10	2	0
	2	100
No.20	0.84	0
	0.84	100
No.60	0.25	0
	0.25	100
		0
No. 140	0.105	0
	0.105	100
No. 200	0.075	0
	0.075	100

ASTM C33 Fine Aggregate (Sand) Gradation			Coarse Range	Fine Range
			Max	Max
	Sieve Size	Size Opening [mm	%Pass	%Pass
	3/8 inch	9.5	100	100
	No.4	4.75	95	100
	No.8	2.36	80	100
	No. 16	1.18	50	85
	No. 30	0.6	25	60
	No. 50	0.3	5	30
	No. 100	0.15	0	10
	No. 200	0.075	0	5

Correction Factor = 100/Percent Passing No.4 Sieve
Correction Factor = 1.052632 1

Corrected Filter Gradation			Coarse	Fine
	Sieve Size	Sieve (mm)	% passing	% passing
	3/8 inch	9.5		
	No.4	4.75	100	100
	No.8	2.36	84	100
	No. 16	1.18	53	85
	No. 30	0.6	26	60
	No. 50	0.3	5	30
	No. 100	0.15	0	10
	No. 200	0.075	0	5

ASTM C33 Type 56 Coarse Aggregate (Gravel)				Drain Gravel Specification			No. 67 Coarse Aggregate (2nd Stage Filter)	
		Coarse	Fine	Coarse	Fine	Opening	Coarse	Fine
U.S. Sieve Size	Size [mm]	%Pass Min	% Passing Max	%Pass Min	% Passing Max	Size [mm]	%Pass Min	% Passing Max
3-inch	75			100	100			
1 1/2-inch	37.5	100	100	100	0			
1 -inch	25	Calcs	100	90	0	25	100	100
3/4-inch	19	40	85	40	0	19	90	100
1/2-inch	12.5	10	40	20	0	9.5	20	55
3/8-inch	9.5	0	15	10	0	4.75	0	10
No. 4	4.75	0	5	5	0	2.36	0	5
No. 8	2.36							
No. 16	1.18							

Based on % passing #200 seive, material 4

Filter Calcs — Note: Per USBR Design Manual pg 10, D15f/D85b ratio for 2nd stage filter between 2 processed materials can be 9 (use 7

d_{85} = 1.18 mm

$D_{(15max)}$ 10.62 $9*d_{85base} \geq D_{15max}$ **Control Point 1**

For Permeability $D_{15min} \geq 4xD_{15\ base\ before\ regarding}$

$D15_{min}$ 1.6 mm

Check for Gap Grading $D_{15max}/D_{15min} \leq 5$

6.6375

Adjust $D15_{min}$ to maintain $D15_{max}/D_{15}min$ ratio of no more than 5

$D15_{min}$ 2.124 mm **Control Point 2**

To prevent broad range of particle sizes, Coefficient of Uniformity of 6 and ratio of max to min dia at all % passing values of 60 or less < 5

$D_{10max} = D_{15max} / 1.2 =$ 8.85 mm

$D_{60max} = D_{10max} * 6 =$ 53.10 mm

$D60_{max} > D_{90}$ below, adjust to D_{90Max} 30 mm **Control Point 3**

$D_{60min} = D_{60max} / 5 =$ 6.00 mm **Control Point 4**

Determine minimum and maximum particle size (D_5 and D_{100})

D_5 = greater than or equal to 0.075 mm **Control Point 5**

D_{100} = 75 mm **Control Point 6**

To minimize segregation, need to define relationship between D_{90max} and D_{10min}

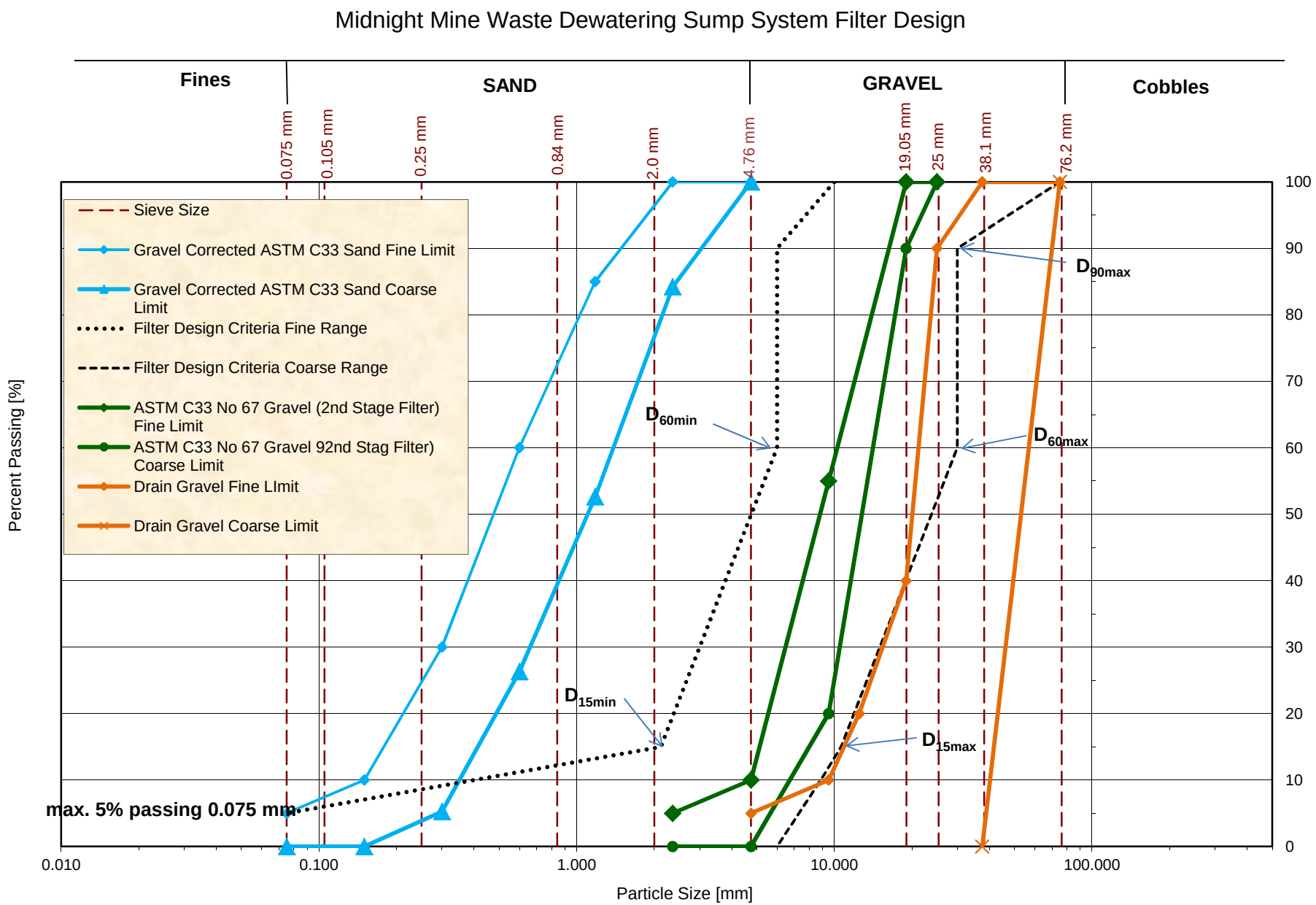
$D_{10min} = D_{15min} / 1.2$ 1.77

Since D_{10min} is 1.77 mm then maximum D90 for the filter is 30 mm per Table 26-6 **Control Point 7**

Connect Control Points 4,2 and 5 to form design for the fine side of the filter band. Connect Control Points 6,7,3 and 1 to form a partial design for the coarse side of the filter band.

Filter Design Criteria

	Control Point	Size (mm)	% Passing	Description
Fine	Control Point 8	10	100	Extrapolated D_{100min}
Fine		6	90	D_{90min}
Fine	Control Point 4	6.00	60	D_{60min}
Fine	Control Point 2	2.124	15	D_{15min}
Fine	Control Point 5	0.075	5	D_5
Coarse	Control Point 6	75	100	D_{100}
Coarse	Control Point 7	30	90	D_{90max}
Coarse	Control Point 3	30.0	60	D_{60max}
Coarse	Control Point 1	10.62	15	D_{15max}
Coarse		8.85	10	D_{10max}
Coarse		6	0	


Midnight Mine Waste Dewatering Sump System Filter Design
Fines
SAND
GRAVEL
Cobbles
0.075 mm
0.105 mm
0.25 mm
0.84 mm
2.0 mm
4.76 mm
19.05 mm
25 mm
38.1 mm
76.2 mm
Sieve Size
Gravel Corrected ASTM C33 Sand Fine Limit
Gravel Corrected ASTM C33 Sand Coarse Limit
Filter Design Criteria Fine Range
Filter Design Criteria Coarse Range
ASTM C33 No 67 Gravel (2nd Stage Filter) Fine Limit
ASTM C33 No 67 Gravel 92nd Stag Filter) Coarse Limit
Drain Gravel Fine LImit
Drain Gravel Coarse Limit
D90max
D60min
D60max
D15min
D15max
max. 5% passing 0.075 mm
Percent Passing [%]
Particle Size [mm]
0.010
0.100
1.000
10.000
100.000
0
10
20
30
40
50
60
70
80
90
100

Attachment D-11

Report from Rock Solids Solution – Midnite Mine Rockfall Mitigation Plan

ROCKFALL MITIGATION PLAN

MIDNITE MINE – Wellpinit, Washington

July 10, 2014

195 Lodgepole Circle, Parachute, CO 81635
Office: (970) 987-4247

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SUPPLEMENT 1 Rock Solids – Midnite Mine Rockfill Mitigation Memorandum

1.0 INTRODUCTION

This Rockfall Mitigation Plan (the Plan) describes the Scaling and Engineering Controls that are recommended for the Midnite Mine Remediation Project near Wellpinit, Washington. This Plan provides details regarding the appropriate equipment and procedures to be used during the implementation of highwall scaling and the necessary minimum engineering controls for rockfall mitigation during remedial construction within Midnite's Pit 3 and 4. Supplement 1 contains the Midnite Mine Rockfall Mitigation Memorandum that was included in the submittal for the 60% Basis of Design Report (BODR). This document provides the majority of the background data and other information evaluated to assist in defining the rockfall mitigation measures presented in this Plan. Other documents reviewed as part of preparation of this Plan are referenced in the following sections.

1.1 PROJECT BACKGROUND

Pit 3 and Pit 4 at the Midnite Mine were excavated in the 1970s with a 1981 completion of the mining operations. The mine has remained inactive with the exception of water management activities at the site. Based upon our observations, Pits 3 and 4 were excavated with benches on 50± elevations with some controlled blasting techniques utilized along the final perimeter wall to control rockfall during the mining operations. The benches, for the most part, were filled with slope debris and talus material, creating irregular slopes, rock faces with talus/colluvial-filled benches and in some areas a fairly uniform slope.

The remedial design requires the dewatering of Pits 3 and 4, cleanup of the sediments in the pit bottoms, excavation of a new drain sump (which will require blasting), placement of a drainage system installed in the sump, placement of drain rock over the entire pit bottom, installation of a low permeable liner placed over the basal drain rock layer, and backfilling the remainder of Pits 3 and 4 with mine waste (primarily waste rock and soil cleanup materials). In addition, drain rock will be extended up the pit walls in parts of Pit 3 where seepage has been occurring.

High exposure of personnel to rockfall hazards will occur during sediment removal, sump excavation, initial drain rock placement and low permeable liner installation (because personnel will be working outside the safety of construction equipment). Once the operation to place mine waste on top of the sub-waste liner system begins, MWH has developed an operational deposition plan that utilizes a perimeter ditch system for the majority of the work to reduce the risk of rockfall hazard to construction personnel. There will be a few locations initially (at low elevations) where trenches are not feasible due to the narrow geometry. These locations have been limited, to the extent possible, to areas with flatter slopes where rockfall hazards are less. In addition, once mine rock backfill operations start, there should be little need for personnel to be outside of equipment within the confines of the pits.

MWH implemented a Rockfall Hazard Monitoring Program (RHMP) in the winter of 2011/2012. The results are presented in the "Midnite Mine Rockfall Hazard Monitoring Report, July 2013 RHMP" (MWH, 2013) and documented that a rockfall hazard exists in both Pit 3 and Pit 4. The report documented the accumulation of over 1,000 rocks on the pit lake ice and included video records of 50 rockfall events during the 17-month monitoring period. The video monitoring

system clearly identifies a potential hazard during the remedial construction activities. It is clear that rockfall is and will continue to be a hazard for personnel working within Pits 3 and 4. Due to this hazard, engineering measures should be implemented just prior to, and during, initial remedial construction.

1.2 ENGINEERING CONTROLS FOR ROCKFALL MITIGATION

1.2.1 Scaling

As rockfall is very unpredictable, a systematic approach to the construction should be implemented to reduce the exposure to personnel working within Pits 3 and 4. Initial rockfall hazard mapping (MGC, 2011) indicated that rocks up to 6 to 10-feet (in diameter) exist on the slopes and could generate a rockfall hazard during construction. This size of rock represents a significantly high energy rockfall event that will be difficult to dampen and/or control with any reasonable rockfall protection system. As such, inspection and scaling of the pit slopes should be performed prior to construction activities in Pits 3 and 4 to remove and/or identify potential rockfall sources that may generate rocks larger than 3-feet in diameter. Scaling operations should be conducted by experienced scalers to systematically remove and/or identify rockfall sources in excess of 3 feet in diameter. Larger rocks that have been identified, but **cannot** be scaled should be monitored either by theodolite monitoring of movement points or other remote monitoring systems.

Hydraulic scaling using high-pressure water jets is an option for areas that are within 100-feet (vertically) of the current pool levels in the pit bottoms. Hydraulic scaling would reduce the dust generation during scaling and remove loose material in the lower pit wall areas. Implementation of hydraulic scaling should be conducted after scaling of the upper wall has been completed and prior to pumping out of the pit lake. Hydraulic scaling will remove a significant amount of loose and raveling rock that represents a rockfall source.

1.2.2 Rockfall Barrier Fencing

A portable rockfall barrier fence can be used and would be effective for capturing and retaining rocks that are up to 3-feet in diameter. A portable rockfall fence system designed to provide protection for personnel working in the pit bottom outside the protection of construction equipment should be implemented after scaling has been completed within the selected area. Several portable systems (each 12-feet high, 100-feet long with 15-feet spans, supported by posts placed on skids) placed adjacent to the high wall in the bottom of the pit designed to capture and retain (3-foot minus) rockfall, thereby protecting the work area should be incorporated in the work plan. The portable system can be utilized for initial work (e.g. pit floor cleanup) prior to construction of the rockfall protection berm system proposed by MWH, or in areas the berm system **cannot** be constructed due to space constraints within the work area. Washington DOT has successfully utilized a moveable rockfall barrier system for some of their operations (. An example of a movable/portable barrier system that could be utilized at the Midnite Mine (with some modifications) is shown as Attachment C of the Midnite Mine Rockfall Mitigation Memorandum (Supplement 1).

1.3 SUMMARY

- Rockfall is a hazard in Pits 3 and 4 at the Midnite Mine and can be very unpredictable. Analyses indicate that, once rockfall is initiated a significantly high percentage of rockfall will make it down to the remedial work areas in the pit bottom.

- The Rockfall Hazard Monitoring Program (RHMP) has been a useful tool for providing information relative to the rockfall hazard.

- Physical and hydraulic scaling of (and/or identification of) larger rockfall sources (larger than 3-feet in size) should be conducted prior to dewatering the pit lakes and work effort in the pit bottom areas.

- MWH designed a rockfall catch berm/ditch (10-feet deep and 15-feet wide horizontally) system and work sequence to significantly reduce the risk of rockfall impacting the work areas during the majority of the pit backfilling operations.

- A portable rockfall barrier, or an approved alternative system designed to contain a 3-foot minus rockfall event should be used in areas where personnel need to work outside of construction equipment prior to construction of the MWH rockfall catch berms (i.e. sump drilling/blasting, sump excavation, drainage system construction, and liner placement) or in areas where rockfall catch berms cannot be constructed due to site space constraints.

1.4 DEFINITIONS:

The following words are used in the remainder of this plan and are defined below:

Access Zone: The area in which people are at risk of falling. (e.g. while on rope or close to a vertical drop of more than 6 feet) or work area below the unprotected highwalls.

Aid Climbing: A method of accessing a structure or site where the rope access operative moves by attaching directly to anchor points with special footloops and support devices. The method allows scalers/workers to climb horizontally underneath structures.

Anchor: A fixed attachment point, or series of points on a structure which support the rope systems and other connections to personnel. A critical anchor is an anchor in which failure would result in serious consequences to the safety of the scaler/climber or integrity of the system.

Belay: A method or system used to manage the slack in the safety rope which provides fall protection to rope access operatives in the event of a fall or failure in the main support system.

Certificate of Conformity: Document provided by the manufacturer or a third party testing lab showing the conformity of equipment to a specific standard.

Scaler/Climber: A term used generally to refer to a rope-access operative that may be climbing, descending, traversing a rope or structure. The personnel will engage in the activity of

inspecting, clearing and removal of rockfall hazards greater than 3-feet in diameter from the highwall slopes.

Emergency Medical System (EMS): The local system of professional first aid, law enforcement, fire and rescue assistance.

Fall Arrest: A system designed to stop a fall and reduce the risk of injury resulting from a fall.

Fall Protection: A term used generally to describe any system designed to reduce the risk of personnel from falling or sustaining and injury resulting from a fall.

Fall Restraint: A system that prevents a person from entering an area where the risk of falling exists.

Fall Factor: A method of describing the proportional seriousness of a fall. The fall factor is defined as the maximum distance a worker can fall divided by the length of rope (connection) between the falling worker and the anchorage point.

Fall Line: The straight path defined by an object/rockfall when it is dropped/rolled and subjected to gravity.

Fixed Rope: A rope securely attached to an anchor point that will be utilized by the scaler/climber to provide static support during scaling/working operations.

Hazard Zone: Any area where workers or the public are at risk of injury from the actual work being performed (e.g. rockfall, falling tools, debris flow, etc.).

Job Hazard Analyses: A written evaluation produced by the team of scalers/climbers/workers of the hazards likely to be encountered while performing the work on a project, and how those hazards are best mitigated by planning, precautions, equipment use, and PPE. The document will describe how a particular job or series of jobs will be performed to minimize the risks to the safety of the workers, others workers and the public.

Kernmantle Rope: A synthetic rope with a load-bearing internal core (kern) and a protective outer sheath (mantle).

Lead Scaler/Climbing: A method of protecting the first person who is scaling/climbing or traversing a structure, when rope or safety systems are not already in place or pre-installed. The safety rope is connected to the anchorage points at regular intervals as the scaler/climber progresses.

Peak Impact Force: The maximum force experienced by the body during a fall.

Personal Protective Equipment (PPE): Equipment designed to protect workers from hazards in the work environment (e.g. helmets, gloves, safety glasses, safety boots, fall protection harnesses, etc.).

Rescue: The act of moving an injured or incapacitated worker(s) from the Hazard Zone and/or Access Zone to an area where more definitive medical care can be administered.

Rockfall Barrier: An engineered system that will restrain or contain a rockfall event of a maximum magnitude without complete destruction of the system. The barrier can be as simple as a ditch, wall, netting, etc. that will have a service load and life for the control of rockfall events.

Rope Access: Techniques by which access is gained to slopes, structures, etc. where ropes are the primary means of support, positioning, or safety protection.

Safe Working Load (SWL): The maximum load which an item of equipment may raise, lower or suspend under repeated particular service conditions.

Safety (Back-up) Rope: Rope used to protect the worker in the event of a fall if the primary means of support (e.g. ladder, working line, anchor, etc.) fails.

Safe Zone: Any area outside the Hazard Zone or the Access Zone on a project.

Working Rope: The primary load-bearing rope used for descending, ascending or positioning.

Work Positioning: Techniques for supporting a person while working by using specialized equipment in tension in such a way as to prevent a fall.

2.0 ROCKFALL PROTECTION TEAM MEMBERS, EQUIPMENT AND PROCEDURES

2.1 ROPE ACCESS TEAM MEMBERS

The scaling/climbing team will consist of a group of professionals trained and experienced in the practice of identifying and conducting the safe removal of potential rockfall from the slopes of Pit 3 and Pit 4. The work shall be conducted by the team in advance of the work to be conducted in the bottom of the pit areas and shall be conducted prior to the requirements of the work activities in areas below the Hazard Area. Follow-up inspections and scaling should be performed during work progress to ensure safe conditions are being maintained.

Scaling/Climbing Team. A scaling/climbing team consists of at least three climbers approved by the Team Leader to perform rope-supported work together.

Scaling Supervisor: Each scaling team of three or more personnel working in close proximity to each other shall have a scaling supervisor that will control communications between his/her scaling team and the Team Leader, Lookout Person and other project personnel.

Team Leader. Appoint a team leader to represent the activities of several groups requiring rope-supported work. The Team Leader must be a qualified individual and will have overall

responsibility for the coordination and conduct of several concurrent the scaling operations and rope supported work activities.

Lookout Person. A lookout person (located at the bottom or top) must be on duty at all times when employees are working on ropes. The lookout person's duty will be to ensure that anchorages remain secure and undisturbed and that no activities take place that could endanger employees working on the ropes. The lookout person must also help respond to emergency situations.

Communications. The scaling supervisor for each scaling team shall maintain reliable voice/radio communications between employees on his/her team on ropes with the top slope or belay personnel. If distance or background noise interferes with voice communications, use a two-way radio shall be used.

Rescue. Before starting rope-supported work, make arrangements for rescue. Provisions for rescue must include self-rescue, rescue using onsite personnel, and rescue requiring off-site personnel.

2.1.1 Operational Duties of Team Members

The **Scaling Supervisor** shall be designated by the contractor to be the key individual on a group of scalers that have a designated area to inspect and/or remove rockfall hazards from the slope. The supervisor shall be the individual responsible for assessing the skills of the team of scalers/climbers for the scaling operations.

The Scaling Supervisor shall control the chain of command for the work, communications, etc. of the team and maintain a safe and orderly operation for the contractor such that safety, communication, and planning of the work can be orchestrated with other contractors on the site. The supervisor shall control the chain of command for the team, such that control of the work is maintained.

The **Lookout Person** shall be provided for each scaling/climbing team that accesses the site. This person shall assist the scaling team with access issues, observe the site around the scaling team and other workers of potential hazards and shall communicate those hazards to the scaling team as well as other workers that may be within the area. A lookout person shall also be provided when personnel are to be working behind the portable/temporary rockfall barrier to provide early warning of eminent threat of rockfall from the pit slope. The lookout person shall have the authority to stop work and modify the work effort until improved safety systems have been established.

Communications between the scaling crews and other worker in the area will be the collective responsibility of the scaling supervisor and the designated lookout person for that work area.

If a **rescue** operation is required, the scaling supervisor and designated Lookout Person shall maintain control of the communications on site, such that the risk to other personnel and rescue workers can be minimized and a safe successful rescue can be coordinated and accomplished.

2.2 SCALING PROCEDURES

The scaling operations Team Leader (coordinating several scaling supervisors) shall ensure the following steps are taken:

1) All scaling team members are briefed of the proposed daily activities
2) Work with the scaling supervisors and scalers to develop the daily Job Safety Analyses (JSA) and control the documentation of the JSAs for the work and project Manager.

The Scaling Supervisor shall prepare the daily JSA for the designated work area and review that JSA with the scalers directly under his/her supervision, develop the access zones, routes and establish a daily safety/rescue plan for his/her team. This information shall be documented on the JSA and provided to the Team Leader prior to accessing the work areas.

Each Scaling Supervisor and scaler shall:

A. Identify rock scaling work zone
   1. Unstable rock formation(s)
   2. Other work activities in close proximity
   3. Impact on equipment and vehicle operations and travel

B. Identify anchor points
   1. Ground pins
   2. Existing anchorages
   3. Trees or fixed objects

C. Identify and establish the safe entrance and exit points
   1. Safe entrance and exit for climbers
   2. Safe entrance and exit for other work activities in close proximity

D. Identify and establish the Controlled Access Zone
   1. Establish and communicate the Controlled Access Zone with all personnel and workers working within the Controlled Access Zone

E. Identify means of rockfall control or removal
   1. Hand Scaling (hand held scale bars)
   2. Mechanical (wire rope and equipment and / or air bag)
   3. Controlled Blasting

F. Review site after scaling operation is complete
   1. Intended unstable rock is removed or identified for the team
   2. Notify all workers and contracting officers if the operation is not complete and the Controlled Access Zone is unsafe

2.2.1 Equipment

Scaling Supervisors shall inspect all climbing gear and equipment each day before and after use to ensure its adequacy. Documentation of this inspection shall be provided to the Team Leaders on the JSA daily. As a minimum, the Supervisor shall inspect:

A. Ropes
   1. Man-made fiber ropes specifically made for climbing and rescue.
   2. All ropes are capable of supporting a 5,000 Lbs. (22.2 kN) static load without failure.
   3. All ropes are of kernmantle construction and have a minimum diameter of 7/16" (11 mm) and a maximum diameter ½" (13 mm)

B. Mining Rods
   Hand-held mining rods or rock bars (pry bars) shall be used for the rock scaling activities.

C. Hardware
   1. Connecting hardware is capable of supporting a 5,000 Lbs. (22.2 kN) static load without failure.
   2. Only locking carabineers shall be used.

D. Body Harnesses
   1. Only full body harnesses shall be used.
   2. All harness connections shall be capable of supporting a 5,000 Lbs. (22.2 kN) static load without failure.

E. Equipment Inspection
   1. Scaling Supervisors and Team Leaders shall inspect all climbing gear and equipment each day before and after use to ensure its adequacy.
   2. All scalers/climbers shall remove from the rock scaling operations any equipment identified as inadequate for use.

F. Equipment Retirement
   1. All equipment and gear subjected to severe shock shall be removed from the rock scaling activities.
   2. Any equipment and gear exceeding the manufacturer's recommendations regarding wear shall be removed from the rock scaling operations.
   3. All rope, webbing, harnesses and cord shall be removed from rock scaling activities after no more than 5 years of use from the date it was first put into service.

G. Anchorages
   1. Two independent anchorages shall be used by each scaler/climber during rock scaling activities.
   2. All anchorages shall be capable of supporting a 5,000 Lbs. (22.2 kN) static load without failure.

3. Steps shall be taken to limit rope extensions upon an anchor failure.
4. Anchors shall be separated independent so that if one anchor fails it will not cause the second anchor to fail.

2.2.2 Methods of Ensuring Safe Access and Egress

A. All scalers/climbers are to review anchorage points prior to descending to the work zone(s).

B. All scalers/climbers are to keep visual and verbal contact with the designated Lookout Person while descending to the work zone(s).

C. Scalers/climbers are to choose descent routes that will not expose an existing climber/worker to falling rock, loose soils and other debris that may cause harm.

D. Scalers/climbers are to communicate to the Lookout Person as well as other climbers/workers when entering and exiting the work zone.

E. Belay system
1. Scalers shall use an independent line that is self-controlled by each scaler/climber.
2. All belay ropes shall be manmade fiber ropes specifically designed for climbing or rescue.
3. All ropes shall be capable of supporting a 5,000 Lbs. (22.2 kN) static load without failure.
4. All climbing hardware shall be capable of supporting a 5,000 Lbs. (22.2kN) static load without failure.
5. When not using a standard belay, a deceleration device shall be implemented to limit fall forces to less than 1,800 Lbs.
6. The maximum free fall distance shall not exceed 6 feet.

2.2.3 Controlled Access Zone (Rock Scaling Area)

Rock scaling personnel and approved personnel for the project are the only authorized people permitted to enter controlled access zones and areas where rock scaling activities are taking place. All other workers shall be prohibited from entering controlled access zones.

A. Controlled access zones shall be defined by areas and locations that people and personnel are exposed to hazards as a result of the rock scaling activities. The controlled access zones shall:

1. Utilize traffic control and flagging, if necessary during the rock scaling operations to control vehicle and foot travel access to the controlled access zones.
2. Enforce road area closures during rock scaling activities.
3. Place clear and visible signage at each end around the perimeter of the controlled access zones communicating that communicate the activities and potential hazards.

4. Maintain communication between rock scaling crew, equipment operators and traffic control personnel throughout the rock scaling operations.

B. **Teams Leader and Scaling Supervisors**

It is the responsibility of the Teams Leader and Scaling Supervisors to implement this program by:

1. Maintaining constant communication with fellow workers, equipment operators, traffic control personnel and other personnel involved with the project.
2. Establishing the controlled access zone (rock scaling areas) with fellow workers, equipment operators, traffic control personnel and other personnel involved with the project.
3 Performing routine safety checks of work operations; is the method of scaling is safe and effective. If not, discontinue method and redevelop a safer and more effective method of removing unstable rock formations.
4. Enforcing safety policy and procedures.
5. Correcting any unsafe practices or conditions immediately.

C. **Scalers/Climbers**:

It is the responsibility of all Scalers/Climbers to:

1. Maintain constant communication with fellow workers, equipment operators, traffic control personnel and other personnel involved with the project.
2. Understand and adhere to the procedures outlined in this Job Plan.
3. Follow the instructions of the Scaler Team Leader.
4. Bring to management's attention any unsafe or hazardous conditions or practices that may cause injury to either themselves or any other employees; and report any incident that causes injury to an employee, regardless of the nature of the injury. If the method of scaling is unsafe or not effective, it is the responsibility of Scalers/Climbers to discontinue the method and redevelop a safer and more effective method of removing unstable rock formations.

2.3 HYDRAULIC SCALING

Scale crews must implement a controlled access zone for the rock scaling operations to protect workers and employees from falling material. Scaling operations shall be coordinated by the Teams Leader to provide horizontal segments of the pit highwall that are completed such that other reclamation operations can be conducted near the pit floor. The mechanical scaling operations shall be completed top down to within 100 vertical feet of the limits of the pit bottom work area prior to personnel conducting Hydraulic Scaling down to the pit floor.

Hydraulic scaling will be utilized on the highwall in the area that is within 100 vertical feet of the pit pool elevation. The objective of the hydraulic scaling operation is to utilize a dust suppression system that can remove potential rockfall hazards more efficiently and more effectively than conventional mechanical scaling methods.

Hydraulic scaling will consist of utilizing the existing pit pool as a source of water for the pumping purposes. A diesel or gas operated sump pump capable of providing a minimum of 100 psi pressure 100 feet above the pool level will be installed via boot near the limits of the current hydraulic scaling operations. The pump will be attached to a barge type structure that is anchored in three locations to keep the pump barge from being damaged by the scaling operations. The hose line will run vertically along the limits of an already deemed previously "scaled" area to a point or bench on the slope where hydraulic scaling is to be conducted. A temporary hose anchor system shall be installed to control the hose during pumping operations. The scale crew will wash down the slope in a top down manner with the high pressure water system to remove loose debris and rocks that pose a future threat to workers in the bottom of the pit floor. The hydraulic scaling crew shall work in horizontal segments of the highwall such that an entire horizontal segment of pit floor/pool can be safely accessed for future work.

The hydraulic scaler shall install temporary hose anchors for the hose and separate anchors for the scaler/hose operator to conduct the work. The hose pressure and discharge control system will be adjusted to allow for safe yet effective operation of the hydraulic scaling operation. Care shall be taken not to "mine" out portions of the highwall with the hydraulic scaling system. The goal of the hydraulic scaling will be to remove potential rockfall that has a size of 3-feet or greater and to allow the water to solidify the ravel of finer material once dried.

2.4 GROUND OPERATIONS AND PROCEDURES FOR PROTECTING EMPLOYEES IN THE PIT FROM FALLING MATERIAL

2.4.1 Portable Pit Floor Rockfall Barrier System

Once the highwall scaling operations have been completed within a significant reach of pit highwall, pit floor dewatering can be initiated. The use of a portable rockfall barrier has been developed to allow another level of safety for workers conducting pit floor operations. The portable barrier shall be designed to control rockfall up to 3-feet in diameter and shall be portable via trackhoe or dozer pull operations, which can pull the skid mounted fence and post elements into strategic locations within the current work area. The fence post skid plates will be of sufficiently large surface area such that stability of the fence system during a rockfall event will allow the fence to contain the rockfall event and continue to perform during clean-out operations.

The fence post skid plate systems shall be connected via cables such that the entire fence length will behave as a single element to control rockfall. No manpower or personnel should have to be exposed (outside of heavy equipment) to the highwall system during placement or relocation of the fence. The 100 ft long, 15 panel fence system will be pulled into place from the safe erection area to the work area and secured by mass (weights) added to the post skid plate areas to maintain stability during operations. The end anchorages shall be massive deadman anchor blocks that can be moved and relocated from the safety of the heavy equipment.

All operations required to be conducted behind the safety of the portable rockfall barrier shall be no closer than 20-feet from the ends of the fence. The rockfall catchment fence and netting system will deform during an impact. All efforts shall be made by the work force to stay far enough away from the downstream impact face of the fence to maintain as safe as is practicable of spaces to get the work completed. As such all work operations should consider the risk of the work relative to the location of the fence. A **Lookout Person** shall be provided during the work operations to assist in providing an early warning system to all workers in the area of an observed rockfall event happening. The **Lookout Person** shall be responsible for providing the daily safety meeting, preparation of the work area JSA and control of the site.

Relocation of the fence system will have to be coordinated with all work schedules and efforts to maintain the optimum protection from the system for all workers required to be in the pit floor area. An exclusion zone perimeter shall be cordoned off to warn workers of the limited safe work area available. It will be the responsibility of the **Lookout Person** to maintain the cordoned area and notify workers of encroachment of the limited safe work zone.

2.5 RECORD KEEPING

Records of training provided to employees shall be maintained by the Project Safety Director. All JSAs for the daily operations of scaling, hydraulic scaling and work zone behind the portable rockfall barrier will be provided to the Site Supervisor and/or Project Manager daily. No work shall be conducted without the clear understanding of the hazards and documentation by the work force that the hazard has been recognized. Signature of all workers on the JSA will be mandatory.

3.0 PERSONAL PROTECTIVE EQUIPMENT (PPE)

Personal protective equipment (PPE) is designed to provide an effective barrier between a worker and potentially dangerous objects, substances and processes.

American National Standards Institute (ANSI) standards are shown throughout this section in the absence of other applicable local safety legislation. Where local safety legislation applies, those standards will take precedence. When operations and/or policies dictate the use of personal protective equipment, it will be used in accordance with applicable standards for worker protection.

3.1 MANDATORY REQUIREMENTS

1. Hard hat at all times
2. Safety eye wear appropriate to hazard exposure at all times
3. Appropriate safety footwear
4. Hearing protection (in areas of noise levels above 85 dba)
5. Drug/alcohol free work place.

The Site Supervisor (SS) or Project Manager (PM) shall monitor and evaluate the use and effectiveness of all personal protective equipment. The Scaling Team Leader and Scaling

Supervisors shall provide the SS/PM with JSA documentation of the daily site analyses and equipment inspections prior to the start of the work. They shall provide inspections of all:

a. Safety devices or safeguards, which may include personal protective equipment, shall be acceptable as the proper type, design, strength and quality and shall be at least equivalent to those complying with the standards approved by The American National Standards Institute, or other recognized authorities, except that where no authoritative standard exists for a safety device or safeguard, the use of such safeguard or safety device shall be subject to inspection and acceptance or rejection by the Scaling Team Leader.
b. Protective equipment shall be distinctly marked so as to facilitate identification of the manufacturer.
   Exception: Employer manufactured shields, barriers, etc.
c. The contractor shall assure that the employee is instructed in the use of, and uses protective equipment in accordance with the manufacturer's instructions.
d. The Contractor shall assure that all personal protective equipment, whether employer-provided or employee-provided, complies with the applicable standards for the equipment. The contractor shall assure this equipment is maintained in a safe, sanitary condition.
e. Protectors shall be of such design, fit and durability as to provide adequate protection against the hazards for which they are designed. They shall be reasonably comfortable and shall not unduly encumber the employee's movements necessary to perform his work.

3.2 BASIC PERSONAL PROTECTIVE EQUIPMENT

It is the responsible of the Site Supervisor, Project Manager, Individual Team Leaders and Scaling Supervisors to review personal protective equipment requirements and availability at each project location.

- Hard hats and/or climber safety helmets certified by UIAA/CE will be required depending on job requirements for all personnel
- Climbing hard hats must be greater than ANSI Z89.1 Type II hard hats with chin straps for rock scaling
- Appropriate safety footwear
- Full body safety harnesses for fall protection and full body safety climbing harnesses for scalers will be required
- Lanyards and rope grabs
- Respiratory protective equipment, as dictated by project hazard
- Dust masks
- Safety glasses, goggles and face shields with hardhat adapters
- Hearing protectors
- Cutting goggles
- Welding hoods with hard hat adapters and proper lens
- Welding gloves
- Welding jackets, chaps and sleeves

- Personal Flotation Device (PFD)

Note: The above list represents a cross section of personal protective equipment, which may be required. Additions or deletions may be necessary depending upon site conditions. The daily JSA will determine what is the appropriate PPE is appropriate for each job that is to be performed.

3.3 SELECTING PERSONAL PROTECTIVE EQUIPMENT

PPE must meet the following requirements:

- Provide desired protection against the hazard to which the worker will be exposed as identified in the project Job Safety Analysis (JSA) and safety "toolbox" meetings
- Provide maximum comfort coupled with minimum weight
- Result in minimal restrictions of essential body movement and vision requirements
- Durability

Note: Any defective or damaged PPE shall not be used on any the project or job site.

3.4 FALL PROTECTION AND ROPE SUPPORTED WORK PROGRAM

Protect employees who work on slopes steeper than 1½:1, H:V slippery footing, or who could fall from heights of 6 feet or more (if not protected by fixed scaffolding, guardrails, or safety nets) by a personal fall protection system. The fall protection system must meet the requirements of this section. However, these requirements do not apply to rope supported work (high angle work) such as high scaling, geologic mapping, structural inspections, or other operations that require specialized rope equipment or techniques. Refer to the section on "Rope-Supported Work."

3.4.1 Hardware

Connectors must be drop forged, pressed, or formed steel or equivalent materials. Connectors must have a corrosion-resistant finish, and edges must be smooth to prevent damage to interfacing parts of the system. D-rings and snap-hooks must be able to sustain a minimum tensile load of 5,000 pounds (22.2 kilonewton [kn]). D-rings and snap-hooks must be proof-tested to a minimum tensile load of 3,600 pounds (16 kN) without cracking, breaking, or being permanently deformed.

a. Snap-Hooks. Snap-hooks must be of a locking type designed to prevent disengagement of the snap-hook by contact of the snap-hook keeper with the connected member.

b. Use. Connect snap-hooks to items 1-5 below only when designed for that use:

1. Directly to webbing, rope, wire rope, or chain.
2. To each other.
3. To a D-ring to that has another snap-hook or other connector is attached.
4. To a horizontal lifeline.

5. To any object with an incompatible shape or dimension that could disengage the snap-hook (e.g., the connected object depressing the snap-hook keeper and releasing itself).

Lanyards and Lifelines. Lanyards and vertical lifelines that tie off one employee must have a minimum breaking strength of 5,000 pounds (22.2 kN). Self-retracting lifelines and lanyards that automatically limit free fall distance to 2 feet (.61 meter [m]) or less must have components that can sustain a minimum static load of 3,000 pounds (13.3 kn) applied to the device, with the lifeline or lanyard fully extended. Self-retracting lifelines and lanyards that do not limit free fall distance to 2 feet (.61 m) or less, rip stitch lanyards, and tearing and deforming lanyards must be able to sustain a minimum tensile load of 5,000 pounds (22.2 kN), applied to the device with the lifeline or lanyard fully extended. A qualified person must design, install, and supervise the use of horizontal lifelines, as part of a complete personal fall arrest system that maintains a safety factor of at least two. Restraint lines must be able to sustain a tensile load of at least 3,000 pounds (13.3 kN). Lifelines and carriers must not be made of natural fiber rope.

Anchorages. Anchorages must be able to support at least 5,000 pounds (22.2 kN) per employee attached, or must be designed, installed, and used under the supervision of a qualified person as part of a complete fall protection system that maintains a safety factor of at least two.

a. Rebar. Anchorages may not be made from drill steel or reinforcing bar less than #8 GR 75

b. #8 GR 75 All Thread Steel Anchor pins. Williams form engineering #8 Grade 75 bar having a minimum embedment of 4 ft can be used as anchoring pins. The WFE #8 GR75 bar has an allowable shear strength of 20,000 lbs.

c. Mobile Anchorages. Anchorages must not be attached to mobile equipment or other items that can move while the anchorage is in use.

3.4.2 Procedures

Use personal fall protection systems and their components only for employee fall protection. Inspect lifelines, lanyards, belts, hardware, and anchorages before use each day and discard questionable devices. Use and care for fiber lifelines and lanyards according to recommendations contained in the Rigging Manual and in these standards, whichever is more protective.

a. Lifelines. Provide each employee with a separate lifeline when using vertical lifelines.

b. Rescue. Make provisions to promptly rescue employees who fall or provide the means for self-rescue.

c. Protection. Protect lifelines from being cut, abraded, or damaged in any way.

d. Maintenance, Inspection, Testing. All personal fall protection systems must follow the manufacturers' recommendations for maintenance, inspection, and testing.

e. Training. Before using personal fall protection systems, and after changing any component or system, train employees in the application limits of the equipment, proper hookup, anchoring, and tie-off techniques, methods of use, and proper methods of equipment inspection and storage.

3.4.3 Personal Fall Arrest System

A system used to arrest an employee in a fall from a working level consists of an anchorage, connectors, and a body harness. The system may also include a lanyard, deceleration device, lifeline, or suitable combination of these.

a. Performance Criteria. Personal fall arrest systems must, when stopping a fall:

1. Limit maximum arresting force on an employee to 1,800 pounds (8 kN), when used with a body harness.
2. Bring an employee to a complete stop and limit the maximum deceleration distance an employee travels to 3.5 feet (1.07 m).
3. Have sufficient strength to withstand twice the potential impact energy of an employee freefalling a distance of 6 feet (1.8 m), or the free fall distance permitted by the system, whichever is less.

b. Performance Test. When used by employees with a combined person and tool weight of less than 310 pounds (140 kg), personal fall arrest systems meeting the criteria and protocol contained in 29 CFR 1926 Subpart M, must comply with these requirements.

c. Use. Rig personal fall arrest systems to prevent an employee from falling more than 6 feet or contacting any lower level. Employees must wear a personal fall arrest system with the attachment point of the body harness in the center of the back near shoulder level or above the head. When connected to a horizontal lifeline that could become vertical, connectors must be able to lock in either direction on the lifeline.

d. Maintenance. Maintenance is a critical element in personal fall arrest systems. Follow manufacturer’s recommendations. At least one employee, certified and trained by the manufacturer as qualified to inspect and maintain personal fall arrest systems, must be available when such systems are in use.

e. Impact Loading. When a personal fall arrest system has been subjected to shock loading, immediately remove it from service until a qualified person inspects it and determines it suitable for reuse.

3.4.4 Positioning Device Systems

Positioning device systems include equipment or hardware that, when used with its body belt or body harness, supports an employee on an elevated vertical surface (such as a wall or a rebar

mat), allowing both hands freedom of movement. Such systems also refer to devices attached between the employee and an anchorage to prevent an accidental fall from an elevated surface.

a. **Performance Criteria**. Positioning device systems must withstand, without failure, a 4-foot (1.2-m) drop of a 250 pound (113-kg) weight.

b. **Performance Test**. Positioning device systems comply with these requirements if they meet the test contained in 29 CFR 1926 Subpart M, Appendix D. Restraint line systems must be designed to meet the same test requirements as other positioning device systems.

Personal Fall Protection Systems for Ladder Climbing: Employees wear, or are attached to, personal fall protection systems to prevent injuries and falls.

3.4.5 Design Criteria for System Components

a. **Operation Criteria:** The system must permit the employee to ascend or descend with both hands free for climbing, without having to hold, push, or pull any part of the system. The connection between the carrier or lifeline and the point of attachment to the body belt or harness must be no more than 9 inches (23 cm) long. The system must activate within 2 feet (0.61 m) after a fall occurs to limit the descending velocity of an employee to 7 feet per second (2.1 meters per second) or less.

b. **Performance Criteria:** Ladder safety devices and their support systems must withstand, without failure, an 18-inch (0.41-m) drop of a 500-pound (226-kg) weight. All other personal fall protection systems for climbing activities must withstand, without failure, a 4-foot (1.2-m) drop of a 250-pound (113-kg) weight.

c. **Installation:** Attach mountings for rigid carriers at each end of the carrier. Attach intermediate mounting, as necessary, spaced along the entire length of the carrier to provide the strength necessary to stop employee falls. Attach mounting for flexible carriers at each end of the carrier. When the system is exposed to wind, install cable guides used with a flexible carrier with a minimum spacing of 25 feet (7.6 m) and a maximum spacing of 40 feet (12.2 m) along the entire length of the carrier, to prevent wind damage to the system. The design and installation of mountings and cable guides must not reduce the design strength of the ladder.

3.4.6 Requirements for Linemen's Belts and Lifelines

a. **Belts and Lifeline Requirements:** Use linemen's body belts and safety straps when working above ground levels on wood poles, steel towers, communication towers, and other transmission line, substation, and switchyard structures.

b. **Body Belts, Safety Straps, and Lanyards:** Linemen's body belts, safety straps, and lanyards must meet the current design specifications in 29 CFR 1926.959. The manufacturer must certify compliance.

c. **Nonconductive Rope Lifelines**. Nonconductive rope lifelines must have a minimum breaking strength of 5,000 pounds and be able to withstand an alternating current dielectric test of at least 25,000 volts per foot "dry" for 3 minutes without visible deterioration.

3.4.7 Rope Supported Safety Requirements

The requirements in this section apply when an employee performs rope supported work on high angle slopes, where the rope is the primary means of support, and where the employee must manipulate the rope and its attachments, while using technical climbing techniques to obtain access to the work area. This includes such work as high scaling, geologic mapping, rock bolting, structural inspections, construction, operations, and maintenance activities. Permit rope supported work only when other means of access are not feasible or methods other than rope-supported work expose employees to greater danger.

Belays. Use a safety (belay) rope for work covered by this section. The belay line must be separate and independent from the support (load) line. Another employee may control the belay or the belay may be self-controlled. Use a belay line, if feasible, for jobs where the work surface provides the primary support once the work site is reached. The standard belay line (safety line) must be a dynamic rope that meets the minimum strength requirements of this section. When not using a standard belay, use a deceleration device to limit fall forces to less than 1,800 pounds. The maximum free fall distance must not exceed 6 feet.

Equipment. The employee who performs the inspection must know how the entire system will function under various slope conditions. Equipment strengths must be certified, listed as meeting, or shown by testing to meet the requirements of ANSI A10.14, ANSI Z359.1, the European Union (designated by the "CE" marking), Union Internationale des Associations d'Alpinisme (UIAA), or other recognized certification organization. Knots, friction devices, ascenders, and other hardware will decrease the overall strength of the rope support system. Before use, evaluate the complete support system, with all of its parts, for adequacy.

Anchorage. Use at least two independent anchorages. Each anchorage must be able to support at least 5,000 pounds (22.2 kN) static load. Locate anchorages far enough apart so that failure of one anchor will not cause failure of another anchor. If possible, locate anchors in line with the direction of rope pull; otherwise, take steps to limit the rope extension if one anchor fails. All climber-installed anchorages, except directional anchors, must meet the minimum strength requirements of this section. Directional anchors are anchors used to laterally position a climber who is supported by two anchors meeting the requirements of this section. Directional anchors do not take the place of the main support anchors.

Rope. Support or safety ropes must be manmade fiber ropes, specifically designed for climbing or rescue applications and must be capable of supporting 5,000 pounds (22.2 kN) static load without failure. Ropes must be of kernmantle construction with a minimum diameter of 7/16 inch (11 mm) and a maximum diameter of ½ inch (13 mm).

Hardware. All connecting hardware used in the support system must be capable of supporting 5,000 pounds (22.2 kN) static load without failure. Use only locking carabineers in the support system. You may substitute locking shackles for locking carabineers in anchor connections.

Body Harnesses. Use only full body harnesses, which distribute fall arrest forces over at least the upper thighs, pelvis, waist, chest and shoulders. Harness connections must be designed for work positioning and fall arrests and be capable of supporting 5,000 pounds (22.2 kN) static load without failure. Separate waist and chest harnesses are permitted if designed to be buckled together.

Equipment Inspection. Inspect all equipment used for rope supported work before and after each use, using the manufacturer's recommended procedures. Maintain a rope log for each rope.

Equipment Retirement. If any equipment used for rope-supported work is subjected to severe impact or shock loading, immediately remove it from service. Do not use it for employee protection until it has been inspected and determined suitable for reuse. The employee who inspects the equipment must be knowledgeable about equipment specifications. Remove from service any equipment that exceeds manufacturer's recommended wear or shows other defects. Automatically retire ropes, webbing, accessory cord, and harnesses from service after 5 years, regardless of condition or use history.

4.0 EMERGENCY PROCEDURES

Before starting rope-supported work, make arrangements for rescue. Provisions for rescue must include self-rescue, rescue using onsite personnel, and rescue requiring off-site personnel. The contractor's Team Leader and Scaling Supervisor are designated to respond to and participate in any emergency that may occur. The Team Leader is trained and certified in American Red Cross First Aid and CPR. All subcontractors should participate by identifying their qualified first aid personnel. The first aid team's main responsibility during an emergency is to respond to the call for emergency help.

During an emergency, all applicable radio traffic will be dedicated to the emergency.

All personnel working on the project are directed to the following five-step procedure should they need emergency help.

a. Notify supervisors or fellow workers of an emergency to then notify the nearest medical assistance.
b. Go to the emergency meeting point (street address and/or intersection as designated on the emergency plan identified in the HES Plan for the Project). An employee of the contractor should be sent to this meeting point to direct traffic.
c. If medical assistance is not available within 3-4 minutes of the jobsite, then the onsite certified first aid responder (Teams Leader) shall administer first aid and CPR treatment.
d. Notify medical assistance dispatch of the nature of the emergency (injury, fire, etc.). BE SPECIFIC.
e. Direct the emergency vehicle crew to the scene.

4.1 NOTIFICATION OF EMERGENCY EVACUATION

Notification of an emergency evacuation shall be either by:

a. Word of mouth
b. Radio contact
c. Repeated crane horn, whistle, or prearranged signal

All personnel working on the project will be required to observe the following procedure in the event of an evacuation of the project:

Upon notification

a. Stop all work.
b. All loads to be lowered, if possible.
c. Equipment and energy sources to be shut down.
d. All employees to proceed to the nearest designated emergency meeting point/muster point.
e. Employees to report to a supervisor for a name check-off (foremen will assist in name check-off). Scaling supervisors and Team Leader conduct a personnel count.
f. Site security measures to be established in the area as necessary to keep non-essential people well back (for safety reasons).
g. Work to be resumed only under the direction of the Team Leader or his/her designee.

4.2 FIRST AID AND MEDICAL EMERGENCIES

First Aid for Eye Emergencies
All eye flushing kits shall be located with the first aid and medical supplies on each jobsite, facility or project as specified in the Emergency Response Plan.

Be Prepared

- Wear eye protection for all hazardous activities and sports – at school, home, and on the job.
- Stock a first aid kit with a rigid eye shield and commercial eyewash before an eye injury happens.
- **DO NOT** assume that any eye injury is harmless. When in doubt, see a doctor immediately.

Chemical Burns to the Eye
In all cases of eye contact with chemicals:

- Immediately flush the eye with water or any other drinkable liquid. Hold the eye under a faucet or shower, or pour water into the eye using a clean container. Keep the eye open and as wide as possible while flushing. Continue flushing for at least 15 minutes.
- **DO NOT** use an eyecup.
- If a contact lens is in the eye, begin flushing over the lens immediately. This may wash away the lens.

- **DO NOT** bandage the eye.
- Seek immediate medical treatment after flushing.

Specks in the Eye

- **DO NOT** rub the eye
- Try to let tears wash the speck out or use an eyewash.
- Try lifting the upper eyelid outward and down over the lower lid.
- If the speck does not wash out, keep the eye closed, bandage it lightly, and see a doctor.

Blows to the Eye

- Apply a cold compress without putting pressure on the eye. Crushed ice in a plastic bag can be taped to the forehead to rest gently on the injured eye.
- In cases of pain, reduced vision, or discoloration (black eye), seek emergency medical care. Any of these symptoms could mean internal eye damage.

Cuts and Punctures of the Eye or Eyelid

- **DO NOT** wash out the eye with water or any other liquid.
- **DO NOT** try to remove an object that is stuck in the eye.
- Cover the eye with a rigid shield without applying pressure. The bottom half of a paper cup can be used.
- See a doctor at once.

4.3 MEDIA RELATIONS

The Site Supervisor or Project Manager shall address all media relations. Under no circumstances shall an employee of the contractor discuss events of an emergency with personnel other than his/her team leader.

The Emergency Response Plan designates the companies' Project Manager as media spokesman in most cases. Employees as well as the Project Manager must be aware of this policy and be encouraged to politely direct any media inquiries to the Project Manager.

4.4 EMERGENCY MEDICAL SERVICES

The Contractor shall have a designated protocol to follow to the medical offices for medical care from work related injuries occurring on their projects requirements. One of the following facilities will be used. On projects or jobsites that a 911 medical response cannot be received the appropriate medical facilities for medical treatment shall be posted. The posting shall include the name of the facility, phone number(s), and address.

Newmont Mining Emergency Contact:

Mr. Bobby Nelson, Site Manager
Phone: (509) 258-4511
Cell Phone: (509) 936-5272

Nearest Hospital – Spokane, Washington:

Sacred Heart Medical Center
101 W. 8th Avenue
Spokane, Washington 99204
Telephone: (509) 474-3131

ALL SERIOUS ACCIDENTS OR INJURIES MUT BE REPORTED AND DOCUMENTED IMMEDIATELY

4.5 STRUCTURAL AND NATURAL HAZARDS PROCEDURES

4.5.1 Structure or Equipment

Should the situation arise where a structure has collapsed or equipment has been involved in an accident, the following general procedures should be followed:

a. The normal emergency assistance procedures should be followed
b. The nearest supervisors are to be notified
c. People are to be kept out of the area except for those rendering medical assistance
d. Area utilities are to be turned off as quickly as possible, providing it is safe to do so and lights are not needed to render medical assistance
e. Attempts to clean-up or repair should not be made until clearance has been given by the Project Manager or Director of Safety

4.5.2 Hazardous Substance Release or Spill

Immediately upon a release of a spill, steps should be taken to implement the spill plan for that the project. The plan shall be spelled out as part of the Project Safety Plan and should comprise the following basic steps:

a. The Material Safety Data Sheets (MSDS) are to be referred to for detailed procedures
b. Contact the Site Supervisor or Project Managers HES representative immediately and secure the area
c. If unable to be contained by competent persons in area, contact local fire department (911)
d. In most cases clean-up procedures should start as soon as possible to prevent further spread of the substance into flowing water, ground water, or sewer systems

Many urban centers offer weather watch warning and the following storms conditions fall into that category. When a project is in the vicinity of these storms, the following procedures should be observed.

4.5.3 Fire Suppression

Project Safety Policies that have been implemented imply that "if you attempt to put out a fire in its first 30 seconds, you stand a good chance of being successful." During the first 30 seconds, emergency assistance procedures should be followed by anyone discovering or involved in a fire.

a. The fire is to be evaluated in regards to controlling it
b. Verify that the Fire Department has been contacted and given information as to location, type of fire and any injuries if known
c. If in a remote location, dispatch an employee to direct emergency teams to location of incident
d. Necessary evacuation steps are to be taken
e. The evacuation procedure is to be carried out
f. Site security measures are to be established as necessary to keep non-essential people out and to safeguard records and equipment

When directed to evacuate a building under threat of fire, employees should observe the following points:

a. Permanent elevators are not to be used unless directed
b. Lights to be left on, doors, and windows closed but not locked
c. Employees to stay as low as possible and to try to keep out of the smoke (possibly toxic)
d. Should clothes catch fire, the best thing to do is to drop and roll

4.5.4 Severe Weather Conditions

Many urban centers offer weather watch warnings and the following storms conditions fall into that category. When a project is in the vicinity of these storms, the following procedures should be observed. Employees working in remote areas where weather warnings are not applicable to the area should maintain an awareness of prevailing weather conditions and be equipped with clothing and equipment appropriate for the anticipated weather conditions.

a. Severe Lightning Storm

1. Lakes, sloughs, or any open body of water is to be avoided
2. Tops of buildings, high lines, vessels or crane operation to be avoided
3. Construction equipment to be avoided
4. Vehicles to be pulled off the road and the 4-way flashers activated until the storm has passed

b. Tornado

1. Below grade shelter to be found (staying inside structures is recommended but exterior doors and windows are to be avoided)
2. Operation of tower cranes to be suspended (the hook should be brought up and the trolley brought in, the power disconnected and the crane allowed to weather vane)

3. Operation of mobile crane to be suspended-boom should be laid down if time permits or the load line hooked to either the house or the structure at some low point. The equipment should be left and refuge taken in a shelter
4. Operation of drilling equipment to be suspended and if time permits the drilling tower should be lowered into transport position then take refuge away from the equipment
5. Should shelter not be available, employees should proceed to low ground or a ditch and lie down with the head, eyes, and ear protected

c. Blizzard

1. Clothing appropriate for conditions: e.g. heavy duty winter protection or survival equipment
2. Sweating and/or exposure to the cold are to be avoided. In cold weather, employees should work in pairs and conduct ongoing buddy checks for frostbite
3. Pedestrian travel routs should be known. Safety ropes should be erected as a guide
4. Employees should not travel alone unnecessarily. Survival equipment must be carried

4.5.6 Earthquake

Should the situation arise when there is a structural collapse or the threat of imminent collapse, the following general procedures shall be followed:

a. Normal emergency assistance procedures
b. The affected area of the earthquake should be secured
c. The nearest supervisor should be notified
d. People should be kept out of the area except for those rendering emergency aid
e. All the area utilities should be turned off and locked as quickly as possible providing it is safe to do so

4.5.7 Landslide or Rockfall

No action to be taken except to preserve life and prevent injury. Shelter to be found immediately.

5.0 REFERENCES

Miller Geotechnical Consultants (MGC), 2011. Geologic Investigation of Pits and Assessment of Pit Sediments, Design Investigation Report, Revision 2. Prepared on behalf of Newmont USA Limited and Dawn Mining Company for U.S. Environmental Protection Agency, Region 10. April 29.

MWH, 2013. Midnite Mine Rockfall Hazard Monitoring Summary Report, Revision 0. Prepared for Dawn Mining Company, LLC and Newmont USA Limited. July.

SUPPLEMENT 1

Rock Solids – Midnite Mine Rockfill Mitigation Memorandum

MEMORANDUM

To: **MWH Global** **November 27, 2013**
Attention: **Tom Kelley & Vance Drain**

Project: **Midnight Mine – Rockfall Mitigation – Washington State**
Re: **Rock Mitigation – Project# 2013-043**

Mr. Kelley & Mr. Drain,

In accordance with our contract No.S1011322-100033-OM MWH Job No.1011322.313403 Rock Solid Solutions conducted a site visit at the Midnite Mine near Wellpinit, Washington to assess rockfall hazards. Mr. Don Berger of Rock Solid visited the site on October 22 & 23, 2013 with Mr. Tom Kelley of MWH Global and Mr. AJ Austin of Newmont USA. Mr. Lou Miller of Worthington Miller Environmental (WME) participated in office discussions associated with the site observations and proposed remedial construction. The site visit was conducted to observe field conditions and to make an initial assessment of: 1) current condition of the rockfall hazards, and 2) what might be needed in the way of engineering controls for rockfall mitigation during remedial construction within Pit #3 and #4.

Project:

We understand that the Pit #3 and Pit #4 was excavated in the 1970's with a 1981 completion of the mining operations. The mine has remained inactive with the exception of water management activities at the site. See the map attached. Based upon our observations, mining in Pits #3 and #4 were excavated with benches on 50± elevations with some controlled blasting techniques utilized along the final perimeter wall to control rockfall during the mining operations. The benches, for the most part have been filled with slope debris and talus material, creating irregular slopes, rock faces with talus/colluvial-filled benches and in some areas a fairly uniform slope.

We also understand that the remedial design will require dewatering of Pits #3 and #4, cleanup of the sediments in the pit bottoms, excavation of a new drain sump (which will require blasting), placement of a drainage system installed in the sump, placement of drain rock over the entire pit bottom, installation of a low permeable liner placed over the basal drain rock

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layer, and backfilling the remainder of Pits #3 and #4 with mine waste (primarily waste rock and soil cleanup materials). In addition, drain rock will be extended up the pit walls in parts of Pit #3 where seepage has been occurring.

High exposure of personnel to rockfall hazards will occur during sediment removal, sump excavation, initial drain rock placement and low permeable liner installation occurs (personnel will be working outside the safety of construction equipment). Once the mine waste is being placed on top of the sub-waste liner system, MWH has developed an operational deposition plan that utilizes a perimeter ditch system at all times to reduce the risk of rockfall hazard to construction personnel. In addition, once mine rock backfill operations start, there should be little need for personnel to be outside of equipment within the confines of the pits.

MWH has implemented a Rockfall Hazard Monitoring Program (RHMP) since the winter of 2011/2012. The results are presented in the "Midnite Mine Rockfall Hazard Monitoring Report, July 2013 RHMP" (MWH, 2013) and documented that a rockfall hazard exists in both Pit #3 and Pit #4. The report is concise yet thorough and documented the accumulation of over 1,000 rocks on the pit lake ice and included video records of 50 rockfall events during the 17-month monitoring period. The video monitoring system clearly identifies a potential hazard during the remedial construction activities. It is clear that rockfall is and will continue to be a hazard for personnel working within Pits #3 and #4. Due to this hazard, engineering measures should be implemented just prior to, and during, initial remedial construction.

Observations:

The site visit was conducted to assess the rockfall hazard and identify potential rockfall protection measures. Photographs taken during the site visit are included as Attachment A. The bottom of Pit #4 was inspected from the south end up to the far north end. It is apparent that rockfall has occurred over the past few years. The timing of rock fall events was not obvious in Pit #4, however, the size of rock that fell during these events in the past ranged from small rocks (less than 1-foot in diameter) to 5-foot plus diameter. However, it is not clear if the larger rocks were the result of post-mining rockfall or were remnants from the mining operations. Pit #3 has a significantly higher and larger highwall area for a rockfall source. The terminal locations of rockfall events in Pit #3 are generally underwater in the pit lake bottom, so they cannot be observed. It is obvious that rockfall is an ongoing occurrence based upon the accumulation of rock on the catch benches. Remedial construction must consider the impacts of rockfall on the construction operation and safety of personnel.

The RHMP has been a useful tool to provide information relative to the rockfall hazard; however, continuation of the program is unlikely to provide additional useful information. Further monitoring by the video system will only confirm that a risk exists and will not enhance the design of engineered systems needed to reduce the risk to personnel from the rockfall hazard.

Updated Rockfall (CRSP) Analyses:

A review of the previous Colorado Rockfall Simulation Program (CRSP) modeling included in the "Geologic Investigations of Pits and Assessment of Pit Sediments Design Investigation Report" (MGC, 2011) was conducted by our staff to evaluate the impact energy and bounce height of various rock sizes. The previous analyses generally appear to be consistent with what is observed in the field and with the results of additional modeling that we have performed as outlined in this memorandum. The rock size has a direct relationship to the rockfall energy and potential for run-out. Larger rocks originating from the higher levels of the mine will have a tendency to break apart and become numerous smaller rocks once they reach the lower levels in the pit. Therefore, Rock Solid focused on modeling a 3-foot rock and what the behavior of those rocks will be at various sections in the open pits. The bounce height and impact energy (ft-lbs) are the useful output from the analyses, as this information indicates the location and capacity requirements for the rockfall mitigation system.

A summary of additional CRSP Modeling performed by Rock Solid Solutions is included as Attachment B. The analyses point for all sections was selected at 6-feet from the toe of the slope. This point was selected to represent the physical placement of a catchment system that could be safely installed. The analyses indicated that for all sections, between 5% and 68% of all rocks that initiate from randomly-selected locations on the pit walls will reach the pit floor/work area. The analyses for most of the pit slope geometries indicate that the bounce height would be less than 10-feet.

Study section 3D represented the section with the highest potential rockfall energy and bounce heights. As such, the area in the vicinity of Section 3D (the east-northeast sector of Pit 3) represents the highest risk of a rockfall hazard during construction. We understand that sump construction and drainage systems are planned in the drop-cut along the west wall on the opposite side of Pit #3 from this area. Drain rock placement and subwaste liner placement next to this area (3D) should include a slightly larger offset from the highwall (12 to 15-feet horizontally) from the toe to reduce the risk of rockfall hazard to construction personnel.

Engineering Controls for Rockfall Mitigation:

Scaling: As rockfall is very unpredictable, a systematic approach to the construction should be implemented to reduce the exposure to personnel working within Pits #3 & #4. Initial rockfall hazard mapping (MGC, 2011) indicated that rocks up to 6 and 10-feet (diameter) exist on the slopes and could generate a rockfall hazard during construction. This represents a significantly high energy rockfall event that will be difficult to dampen and/or control with any reasonable rockfall protection system. As such, inspection of, and scaling of, the pit slopes should be performed prior to construction activities in Pits #3 & #4 to remove and/or identify potential

rockfall sources that may generate rocks larger than 3-feet in diameter. Scaling operation should be conducted by a team of experienced scalers to systematically remove and/or identifying rockfall sources in excess of 3 feet in diameter. Larger rocks that have been identified but **cannot** be scaled should be monitored either by theodolite monitoring of movement points or other remote monitoring systems.

Hydraulic scaling using high-pressure water jets is an option for areas that are within 100-feet (vertically) of the current pool levels in the pit bottoms. Hydraulic scaling would reduce the dust generation during scaling and remove loose material in the lower pit wall areas. Implementation of hydraulic scaling should be conducted after scaling of the upper wall has been completed and prior to pumping out of the pit lake. Hydraulic scaling will remove a significant amount of loose and raveling rock that represents a rockfall source.

Rockfall Barrier Fencing: A portable rockfall barrier fence can be used and would be effective for capturing and retaining rocks that are up to 3-feet in diameter. The portable rockfall fence system should be designed to provide protection to personnel working in the pit bottom outside the protection of construction equipment. Several portable systems (each with a 12-foot height, 100-feet long with 15-feet spans, supported by posts placed on skids) placed adjacent to the high wall in the bottom of the pit could be designed to capture and retain rockfall, thereby protecting the work area. The portable system could be utilized for initial work (e.g. pit floor cleanup) prior to construction of the rockfall protection berm system proposed by MWH, or in areas the berm system **cannot** be constructed due to space constraints within the work area. Washington DOT has successfully utilized a moveable rockfall barrier system for some of their operations (ROTEC International, Nov 2013). An example of a movable/portable barrier system that could be utilized at the Midnite Mine (with some modifications) is shown as Attachment C.

Summary:

- Rockfall is a hazard in Pits #3 and #4 at the Midnite Mine and can be very unpredictable. Analyses indicate that, once rockfall is initiated a significantly high percentage of rockfall will make it down to the remedial work areas in the pit bottom.

- The RHMP has been a useful tool for providing information relative to the rockfall hazard; however, continuation of the program is unlikely to provide additional information that would be useful in designing rockfall protection measures.

- Physical and hydraulic scaling of (and/or identification of) larger rockfall sources (larger than 3-feet in size) should be conducted prior to dewatering the pit lakes.

- The MWH rockfall catch berm/ditch design (10-feet deep and 15-feet wide horizontally) and work sequence proposed should significantly reduce the risk of

rockfall impacting the work areas during pit backfilling operations.

- A portable rockfall barrier could be used in areas where personnel need to work outside of construction equipment prior to construction of rockfall catch berms (i.e. sump drilling/blasting, sump excavation, drainage system construction, and liner placement) or in areas where rockfall catch berms cannot be constructed due to site space constraints.

This site visit memorandum provides a discussion of the observations and conceptual rockfall mitigation measures that may be applicable for the Midnite Mine proposed remedial construction and backfilling operations. Rock Solid is pleased to be part of the Midnite Mine remedial design team and looks forward to assisting the MWH/ Newmont team in developing procedures to safely complete the required work. Please call/email if any questions or concerns.

Respectfully,

Donald J. Berger P.E.
Rock Solid Solutions
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(970) 987-2743
dberger@rocksolidblasting.com

References:

Miller Geotechnical Consultants (MGC), 2011. *Geologic Investigations of Pits and Assessment of Pit Sediments*. Prepared on Behalf of Newmont USA Limited/Dawn Mining Company, January 14.

MWH, 2013. *Midnite Mine Rockfall Hazard Monitoring Summary Report, Revision 0*. Prepared for Dawn Mining Company, LLC and Newmont USA Limited. July.

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E.Hoek & J.W.Bray, Third Edition 1981 *Rock Slope Engineering*, Institute of Mining and Metallurgy, London

Jones, C.L., J.D. Higgins, and R.D. Andrew, 2000. *Colorado Rockfall Simulation Program (CRSP), Version 4.0* Colorado Geological Survey.

Thommen, Robert III, Nov 2013. ROTEC International *Moveable Barrier Systems for Rockfall*, Prepared for Washington DOT

Attachment A
Site Visit Photos

Photo #1- Pit#4 Looking South from the Observation Level

Photo #2- Pit #4 Pool Area

Photo #3 – Pit#4 Bottom Pool Looking North at Highwall and Upper Observation Area

Photo #4 – Pit#4 East Wall, Note Interbench slope failure and talus/collivial filled benches

Photo #5 – Pit#3 East Wall & Pool area from North Observation Area

Photo #6 – Pit#3 West and South Highwalls

Photo #7 - Pit#3 West Wall from the Pool level

Photo #8 – Rockslide area observed in Pit #3 NW corner

Photo #9 – Pit#3 North Wall from the Pool Level

Photo #10 – Pit #3 Rock Slope NE side

Attachment B
CRSP Analyses Summary

CRSP (Colorado Rockfall Simulation Program) Review and Summary

Rock Solid Solutions

Rock Solid utilized surface roughness, tangential coefficients (R_t), and normal coefficients (R_n) per page 26 of the MGC Design Investigation Report dated 4/29/2011 and MWH as well. The coefficients that MWH chose for design are consistent with the range of values given in Chapter 5, Tables 4 and 5 of the CRSP Manual dated March 2000.

- SR for rock and talus = 1.0*Rock Radius = 1.5 for a 3-foot diameter rock
- SR for stiff soil/talus = 0.75*Rock Radius = 1.12 for a 3-foot diameter rock
- Rock R_t = 0.85, R_n = 0.22
- Talus R_t = 0.80, R_n = 0.16
- Stiff Soil/Talus R_t = 0.70, R_n = 0.12

Rock Solid first input sections with the MWH ditch constructed at the toe to double check their results. This also checked Rock Solids data input and checked the consistency in the analyses between users. Then we changed the surface roughness values for a 3-foot rock and took out the MWH ditch at the bottom to rerun the analysis for a 3-foot rock for all of the sections.

We placed the analysis point 6 feet from the toe to represent the moveable fence option. The 6 feet served to account for the width of skids used to support the fence system.

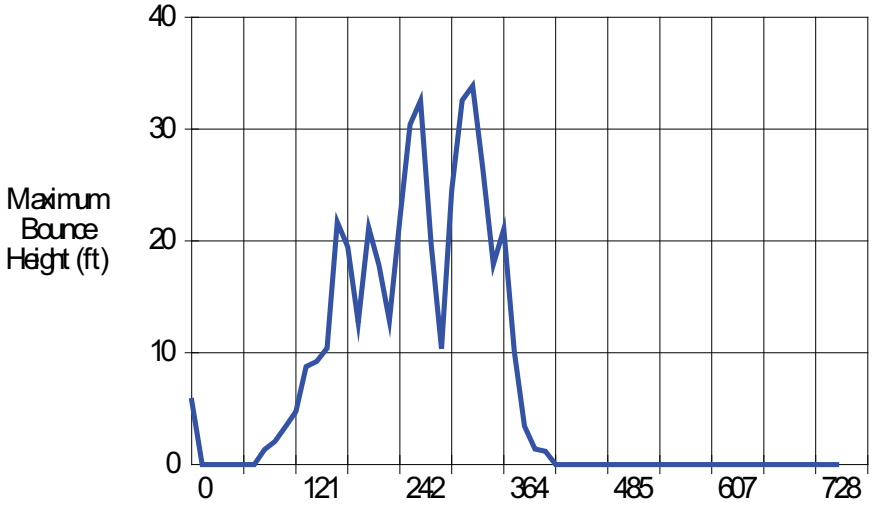
Summary

Section	Max Bounce Height (feet)	Max Energy (ft-lbs)	# of Rocks hitting fence out of 1,000 initiations	Rock Shape
3A	6.3	176,006	226	Spherical
3A with bench cleaned	5.2	188,442	172	Spherical
3B	6.2	94,391	126	Spherical
3C	9.1	72,914	222	Discoidal
3D	15.2	207,798	682	Discoidal
3D fence 12' from toe	9.8	103,112	519	Discoidal
3D with bench cleaned	10.4	159,978	313	Discoidal
4A	No rocks hit fence			Discoidal
4B	1.7	37,423	47	Spherical
4C	3.3	46,581	183	Spherical

Comments about the results:

Section 3A:
There is a bench at Elevation 2630 that could be cleaned (if we can get something on the bench to do the job) to make the bench 10feet wide with a 1foot tall berm on the outside edge. CRSP analysis results with the cleaned bench indicate that only about 7.7% of the rocks were stopped on the bench. This would indicate that extensive efforts to clean this bench would not prove to be cost beneficial.

Section 3D:
The bounce height 6ft from the toe is 15.2 feet. If the fence is moved out to 12 feet from the toe, the bounce height is reduced to a maximum of 9.8 feet. There is a bench at Elevation 2570 that could be cleaned. The bench was modeled as a10 foot wide bench with a 1foot high berm on the outside edge. About 46% of the rocks were stopped on the bench. This would indicate that a low cost to benefit ratio for cleaning this bench exists. An analysis point was evaluated on the outside edge of the cleaned bench to see if a fence could be constructed on the bench. The maximum bounce height was 28 feet at this point, so we do not believe a fence is a viable option on the bench.



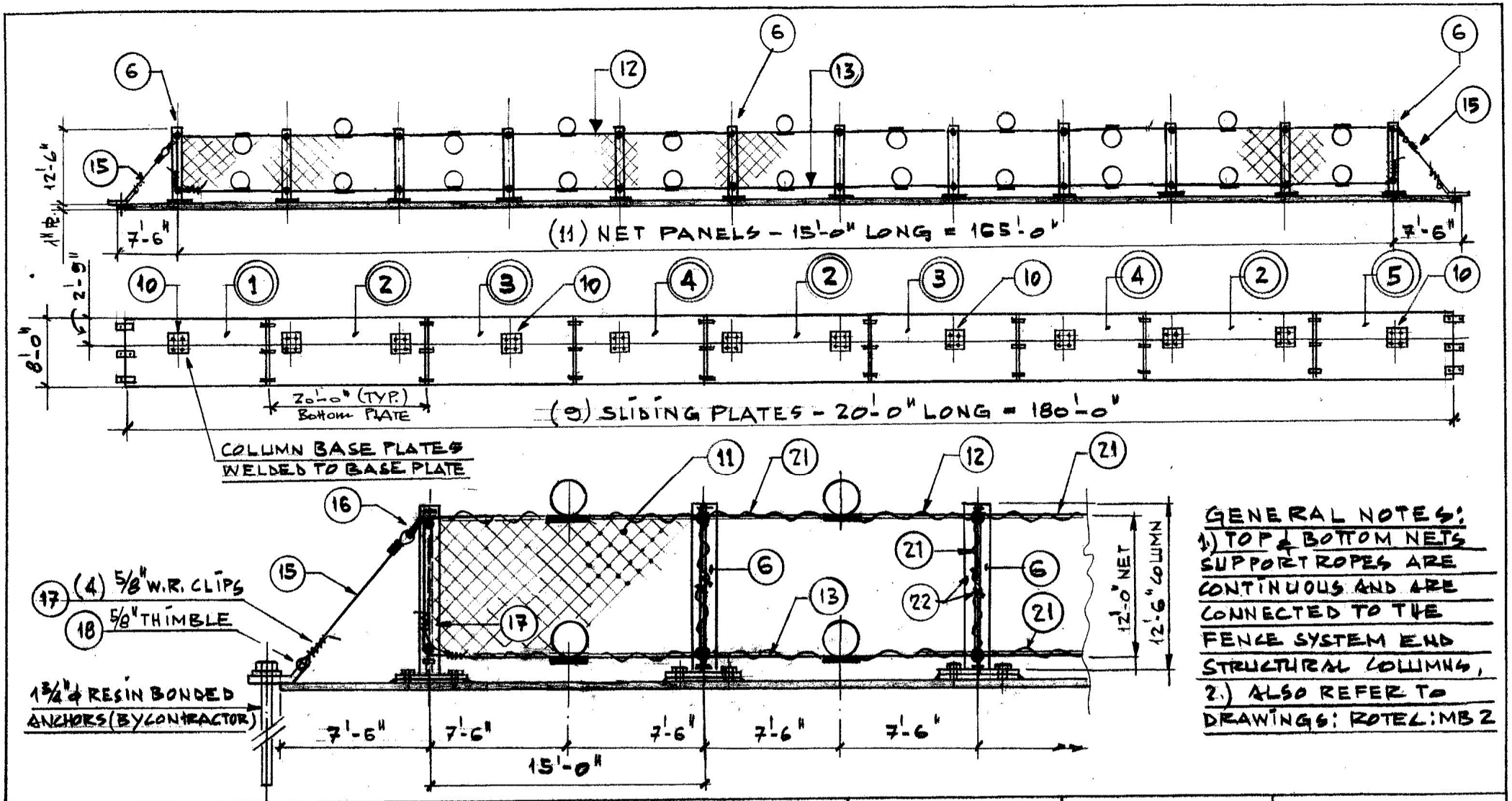
Cross-section 3D, 3-foot diameter rock

Rock Solid then modeled cross-section 3D with a 2-foot diameter rock since scaling will likely take care of anything bigger than a 2-foot rock. The energy was drastically reduced at the fence location, but the maximum bounce heights remained about the same. Therefore, the options for section 3D are to 1) position the moveable fence out 12 feet from the toe along this side of the pit, 2) clean the bench or 3) do both. We believe the preferred option is to move the fence out because the maximum energy at this location is much lower. It also eliminates the need for workers to be exposed to rockfall hazards while cleaning the bench.

The MWH trench/ditch system (pg 27 Design Report) was evaluated for section 3D, using a 15-foot

deep by 15-foot wide horizontal trench, all rocks were stopped by the trench/berm system. Rock Solid reran the analyses with a 10-foot deep by 15-foot wide berm/ditch and all 3-foot diameter rocks were stopped by the system as well. We recommend that the trench depth be reduced to 10-foot depth for ease of construction to match the proposed backfill lift thickness.

Attachment C
Example of
Movable Rockfall Barrier
ROTEC International

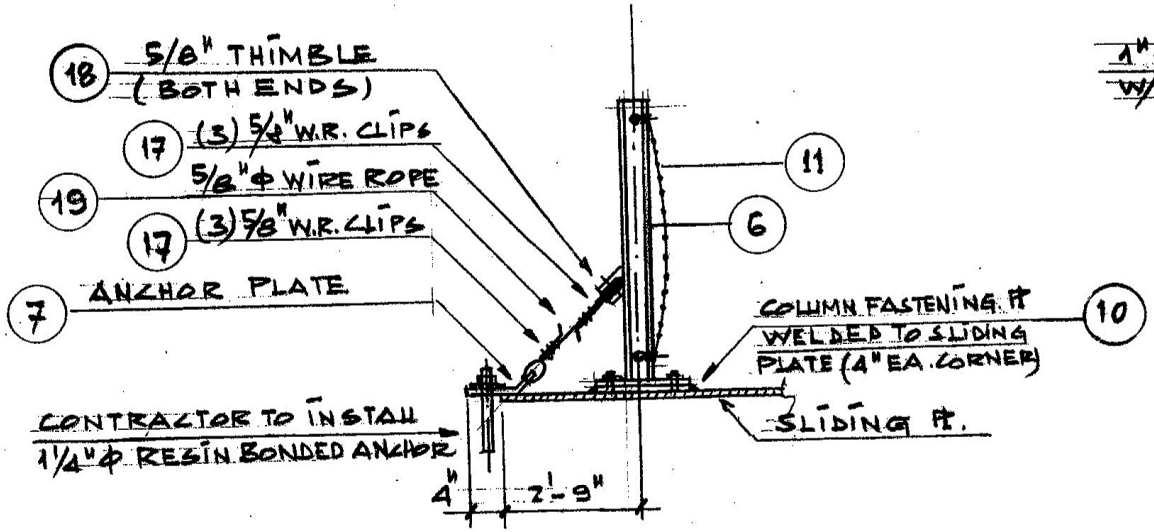


GENERAL NOTES:

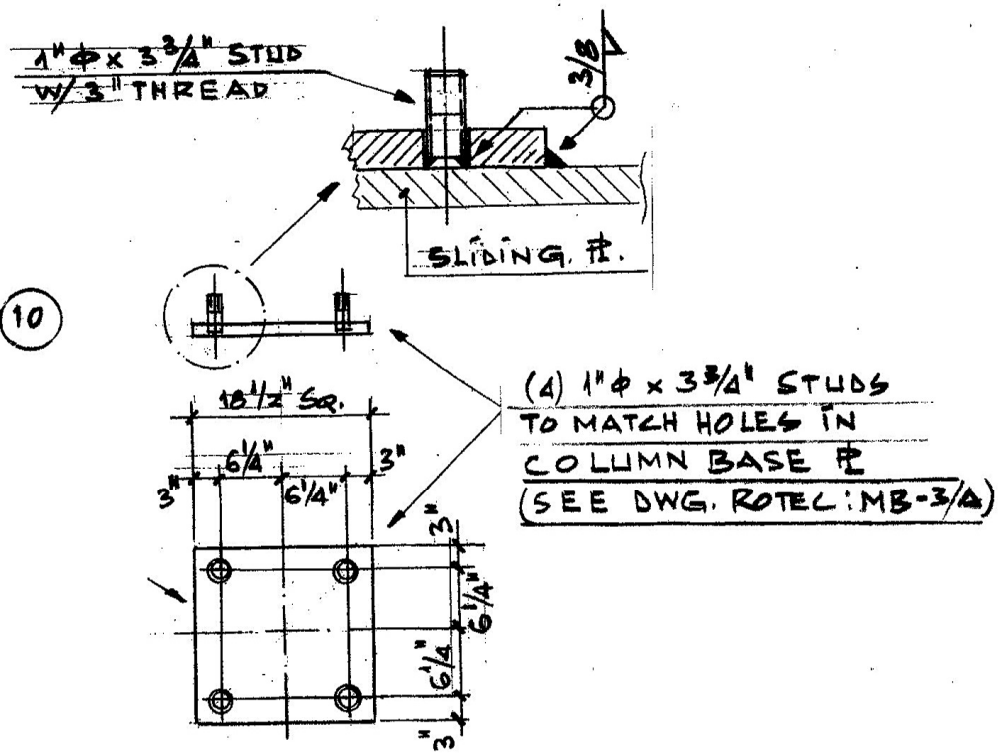
1.) TOP & BOTTOM NETS SUPPORT ROPES ARE CONTINUOUS AND ARE CONNECTED TO THE FENCE SYSTEM END STRUCTURAL COLUMNS.

2.) ALSO REFER TO DRAWINGS: ROTEC:MB 2

ROTEC	MOVABLE BARRIER FOR ROCKFALL	DRAWN /DATE: 11/25/13	SCALE: N.T.S	DRAWING NO: A
		CLIENT CONTRACT NO: —	ROTEC PROJECT NO: —	



TYPICAL CROSS-SECTION



NOTE:
ALSO, REFER TO DRAWINGS:
ROTEC: MB-1 AND MB-2

(10) COLUMN FASTENING PLATE
18 1/2" x 18 1/2" x 1 THK

ROTEC	Movable Rockfall Barrier	DRAWN /DATE: 11/25/13	SCALE: N.T.S.	DRAWING NO: B
		CLIENT CONTRACT NO: —	ROTEC PROJECT NO: —	

Attachment D-12

Revegetation Plan

ATTACHMENT D-12

MIDNITE MINE REMEDIAL ACTION REVEGETATION PLAN

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
1	*16-April-14*	*90% Design*	*Ed Redente*	*n/a*		*Tom Kelley/ Vance Drain*
2	*9-July-14*	*90% Design (added information from 2014 HEP survey)*	*Ed Redente*	*n/a*	*10-July-14*	*Vance Drain/ Melanie Davis*
2	*22-May-15*	*100% Design – no changes*				

Location and Format
Electronic copies of these files are located in the project files system at: *\\usslc1s01\IFO\Industrial Projects\MIDNITE MINE\Deliverables_Working Documents\Basis of Design Rpt\Appendix D - Mine Waste Excavation and Containment\90% Submittal\Att D-12*

Supplement
Supplement D-12.1 – Weed Management Plan for the Rhoads Property

1.0 INTRODUCTION

This attachment describes the manner that Midnite Mine remedial actions will be stabilized and revegetated as part of the remediation process. The objectives of revegetation are to establish conditions that support wildlife habitat and traditional land uses. Traditional land uses include gathering plants for food and medicinal purposes and outdoor recreation including hunting. Revegetation also will create a self-sustaining plant cover that will reduce infiltration, control erosion and produce a stable landform. For purposes of developing this plan for revegetation of disturbed areas, the terrain at the site was divided into the following four categories:

1. Flat-lying Waste Containment Capped Areas where a soil cover will be underlain by an impermeable geomembrane, and where trees species will be excluded.
2. Other flat-lying disturbed areas which are sloped at 3:1 horizontal to vertical (h:v) or flatter.
3. Other steeper disturbed areas. These areas were further subdivided into areas that:
    a. Are steeper than 3:1(h:v) but flatter than 2:1(h:v). It is assumed that if soil cleanup results in removal of all soil overlying bedrock in these areas, that a 1-foot thick soil layer can be placed over the ground surface prior to revegetating.
    b. Are steeper than 2:1(h:v) and will therefore be too steep for placement of a soil layer in areas where soil cleanup results in excavation to bedrock.
4. Downstream drainage areas**.**

The estimated extent of each of these areas is shown on Figure 1.

1.1 BORROW SOURCE FOR COVER SOIL

The Rhoads Property Borrow Area will serve as the source of cover soil for final Site reclamation. A description of the soil from the Rhoads Property Borrow Area follows, along with treatments needed to provide a plant growth medium that will support a self-sustaining plant community.

The Rhoads Property Borrow Area is an 81-acre parcel of land situated just southwest of the Site (Figure 2). Borrow investigations identified that this area may yield more than 600,000 cubic yards of clayey sand materials that have a USDA textural classification of loam. The organic matter content of this soil ranges from 0.4 to 2.7 percent, with a pH of 6.0 to 6.6 (MGC, 2011). Plant available phosphorous (P) and potassium (K) concentrations are adequate for plant growth, with an average P concentration of 11 mg/kg and an average K concentration of 260 mg/kg (MGC, 2011). Inorganic nitrogen (N) concentrations are low, with nitrate-N concentrations averaging less than 1.0 mg/kg.

The soil from this borrow source will not require any organic amendments or pH adjustments to serve as soil cover and plant growth medium. The only nutrient that may be low for sustained plant growth is nitrate-N. However, since the organic matter level averages greater than 1 percent, it is expected that total N levels are adequate for establishing a self-sustaining community of native species and nitrogen fertilizer will only be used if N deficiencies are observed in plant growth during the first or second growing season (see Section 2.1.2 for further discussion).

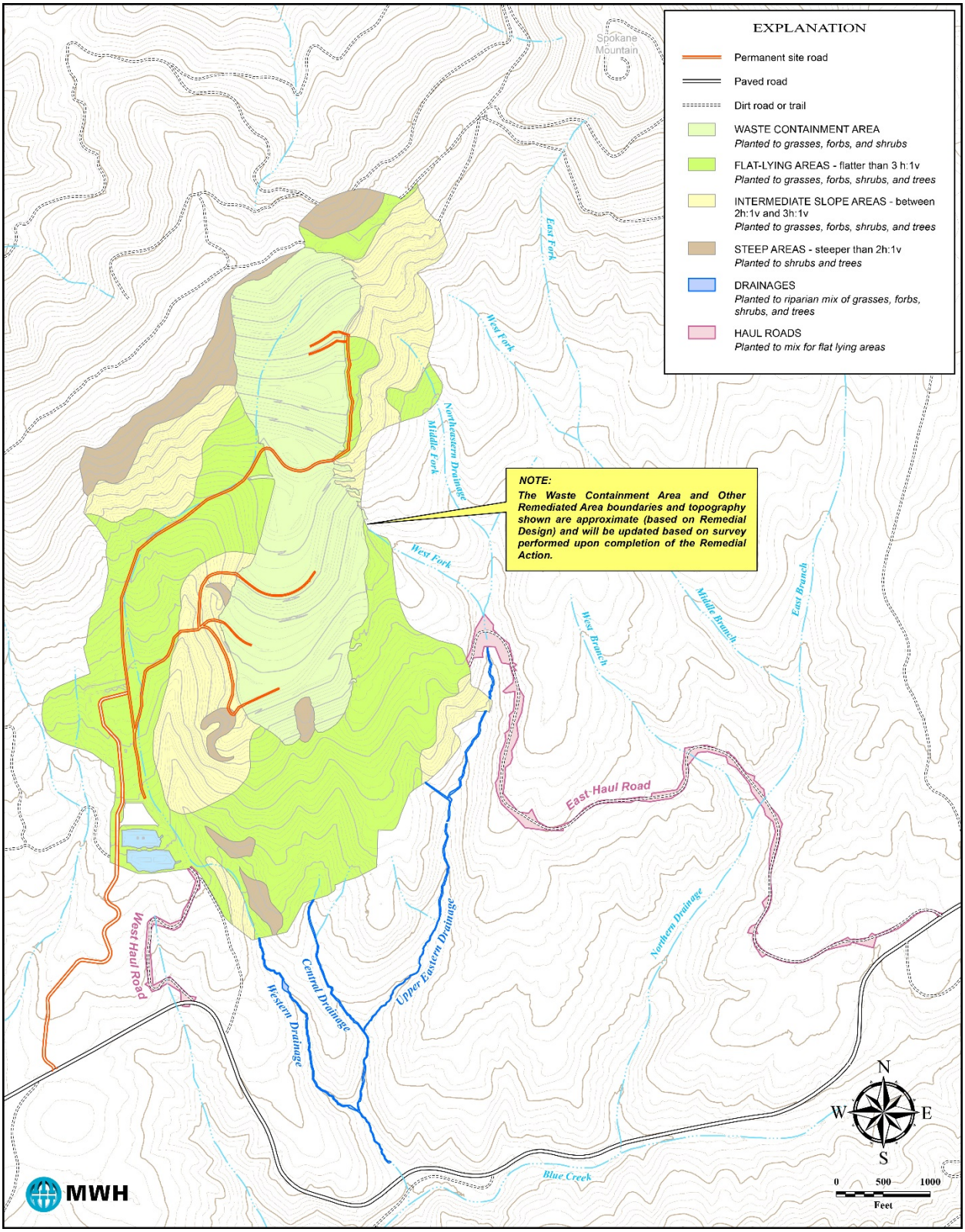


Figure 1. Revegetation Planting Plan

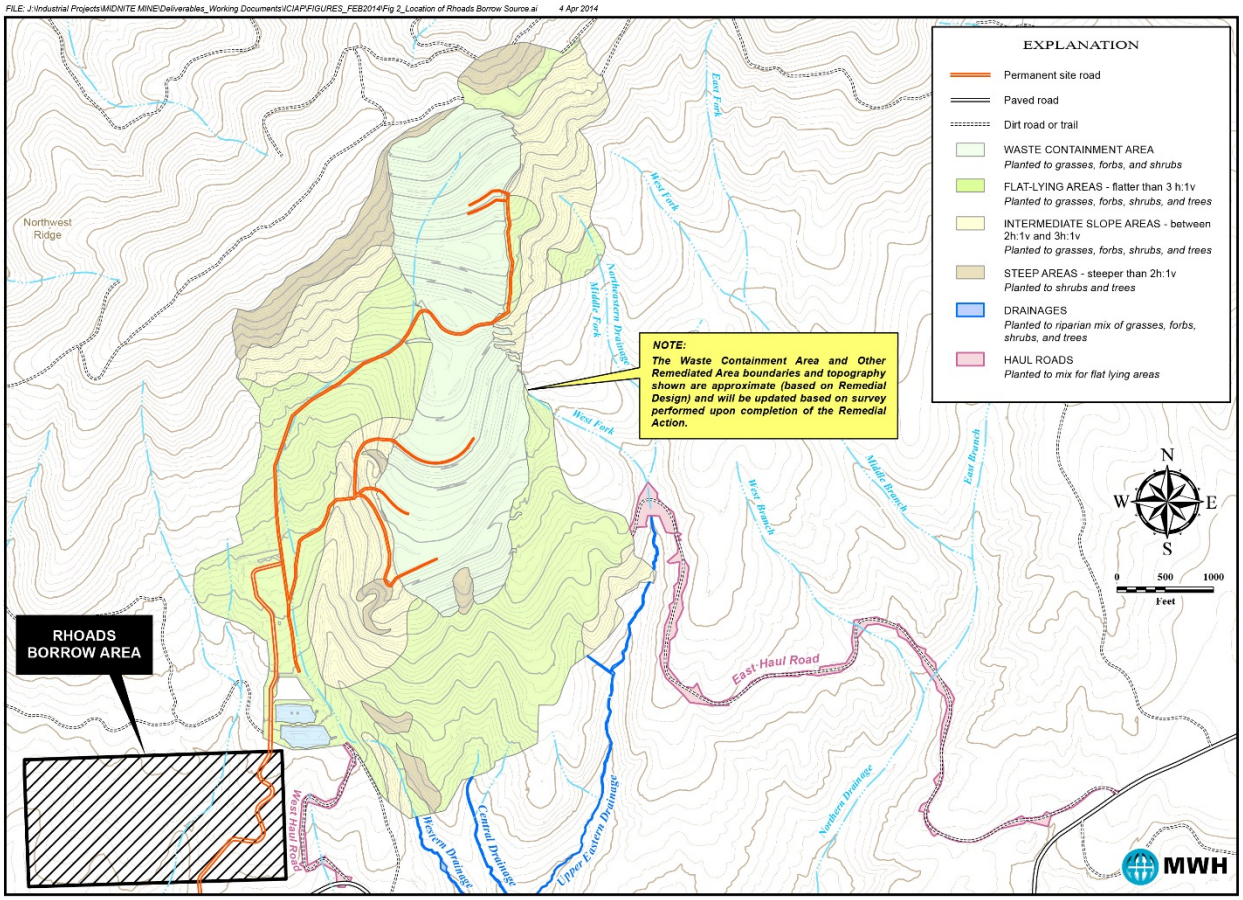


Figure 2. Location of Rhoads Property Borrow Area in Relation to Mine Site

2.0 REVEGETATION APPROACHES

2.1 WASTE CONTAINMENT AREAS

The Waste Containment Areas will be capped with a vegetated composite cover that will include a three-foot-thick soil layer from the Rhoads Property Borrow Area (see Appendix D10.0 Cover System Design for details on cover placement). This soil layer will be placed over a geocomposite cover system that will include a synthetic geomembrane, and a synthetic geocomposite drainage layer on steeper slopes. The extent of the Waste Containment Area requiring revegetation is shown in Figure 1. Care will be taken during each of the planting activities discussed below to minimize the potential for contacting and damaging the underlying geomembrane.

2.1.1 Surface Preparation

All areas that will be revegetated will be scarified to a depth of 6 to 12 inches following placement of a cover soil. Scarification will be accomplished with a chisel plow or similar agricultural implement to reduce compaction that occurs during soil placement and will provide a favorable soil medium for root development and overall plant growth. Soil will be disked to a depth of six inches as a final surface preparation treatment just prior to seeding and planting. Figure 3 provides photos of various tillage implements that may be used to reduce compaction and for seedbed preparation.

2.1.2 Amendments and Fertilization

As noted above, the only fertilization that may occur would be the addition of nitrate-N if early plant growth shows N deficiencies. If needed, nitrate-N would be surface applied to revegetated areas at a rate of 50 lbs of N/acre as ammonium nitrate or similar formulation.

2.1.3 Seeding, Planting and Mulching

Grasses and Forbs

Grasses and forbs on all Midnite Mine sites will be broadcast seeded using hydroseeding or similar type broadcast seeding equipment. In the event that hydroseeding equipment is used, hydroseeding activities will be conducted using 50 pounds per acre of cellulose wood fiber mulch to aid seed transport with the seed applications.

Completion of the seedbed preparation process will include harrowing or other agricultural methods to further scarify the planting surface. Harrowing will be accomplished following seeding with conventional agricultural drag implements which have the ability to scarify the soil surface and cover seed applications to an appropriate one-half inch. This approach also leaves the soil surface in a roughened condition to reduce erosion. Following completion of harrowing, a cellulose wood fiber mulch will be applied at 1.25 tons/acre, along with a bio-degradable tackifer. Final mulch rates will depend on aspect, grade, elevation, and soil conditions such as percent rock fragments on the soil surface. Should alternate broadcast methods be used to better accommodate site conditions, such planting methods, equipment types, and mulch types and rates will be proposed to EPA well in advance so that these revegetation techniques can be appropriately reviewed and approved. Site seedbed preparation, seeding, mulching and erosion control applications will be conducted in late fall.

Shrubs

Shrubs will be planted as mycorrhizal inoculated seedlings grown from seed or cuttings. The containerized stock will be 6 to 12 months old and plantings will take place in late fall as dormant plantings <u>after</u> the first growing season of the seeded grasses and forbs.

If planting stock is received at the Site within two weeks of planting, stock will be kept out of direct sunlight and watered to protect from desiccation. If planting stock is to be held at the Site for longer than two weeks prior to planting, stock will be kept out of direct sunlight, containers covered with mulch (e.g. wood chips) and kept moist until planting can occur.

Figure 3. Examples of Revegetation Tillage Equipment

Site preparation for transplanting would include removal of herbaceous vegetation (e.g. grasses and forbs) in a 12 to18 inch circle around the planting hole with a mattock or similar tool. This step will reduce competition with transplanted seedlings and as a result, increase survival and rate of growth.

The following procedures will be used during the transplanting process for shrubs.

1. Loose organic material such as leaves, grass, etc. will be brushed aside from the planting spot to expose mineral soil.
2. A slit will then be made in the ground using a planting bar or similar tool. The slit will be deep enough for the roots to be fully extended (see Figure 4 below). In order to avoid damage to the underlying geomembrane, excavations and planting bar insertions shall be limited to depths of 18 inches or less in the Waste Containment Area. If the roots are too long for the planting equipment, minimal pruning of small end roots may be needed to prevent J-hooking of roots, but not more than 25 percent of the total root length would be removed. After planting in the slit, the bar is re-inserted several inches away, rocked away from the plant to kick in the soil at the bottom of the roots, and then rocked toward the plant to compress the soil around the base of the plant. The soil around the plant will then be packed with foot pressure.
3. Shrubs will not be planted when the soil is frozen.
4. Shrubs will be planted in a vertical position with the root collars approximately level with or slightly below (0.5 inch or less) the existing ground line.

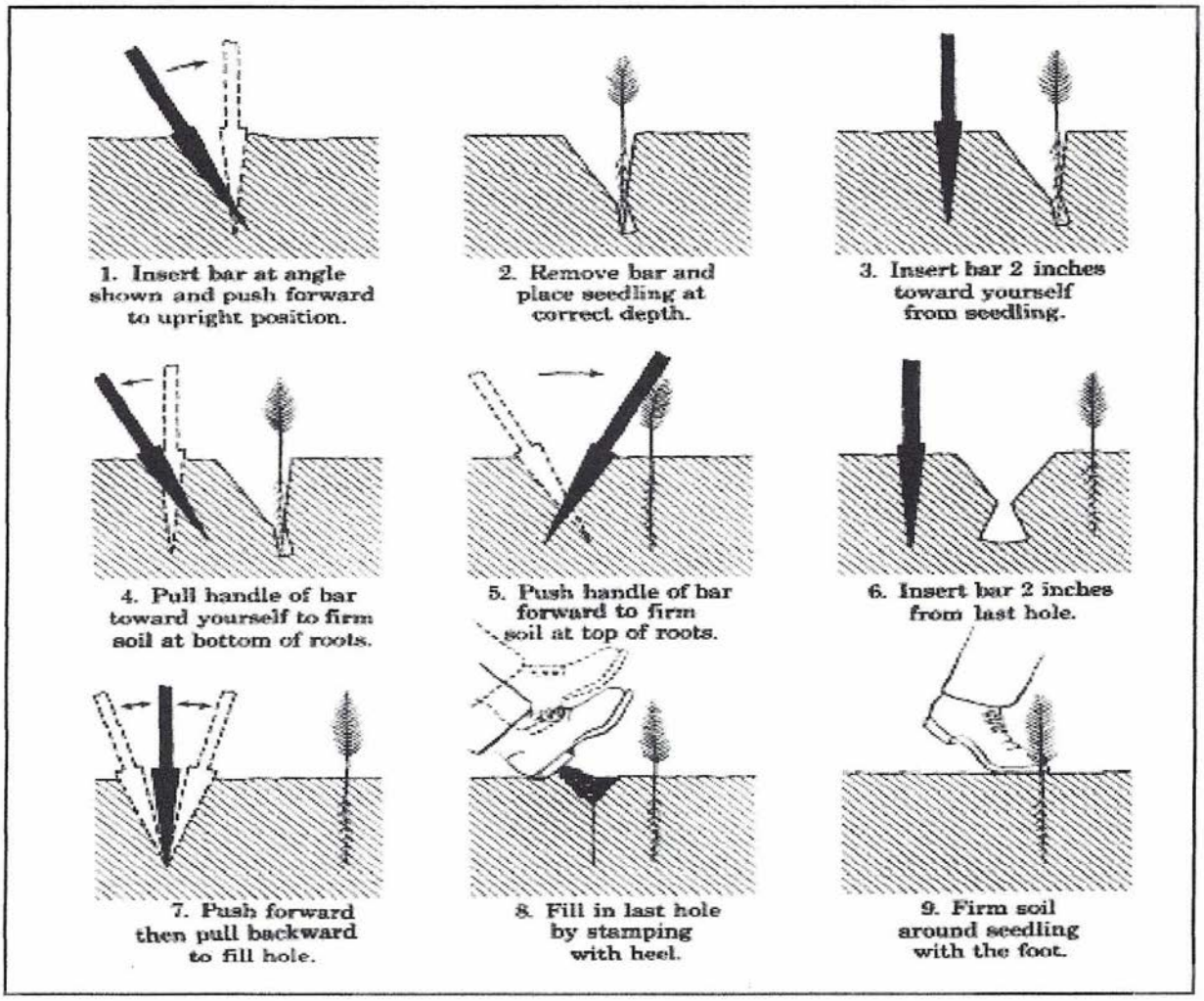


Figure 4. Planting Procedure for Transplanting Shrubs using a Planting Bar

Seedling protection tubes shall be used for all transplants to protect seedlings during the first three years from browsing damage by mice, rabbits, beaver, deer, elk and other small or large herbivores. Seedling protectors would most likely be rigid polypropylene mesh tubes that cover the entire plant and allow room for growth for a minimum of three years (Schaap and DeYoe 1986). The tubes would be held in place with bamboo stakes and removed after three years so the tubes do not cause the plants to grow in a deformed shape. A three year protection period should be adequate to ensure high survivability.

2.1.4 Species Selection

DMC/Newmont conducted a vegetation survey of the Rhoads Property Borrow Area in July 2012 and a habitat survey in May 2014 following the Spokane Tribe Habitat Evaluation Procedures (HEP) protocol. Both surveys have been used to finalize the selection of native plant species for Site revegetation along with planting density for shrub species. In addition, results from both surveys will be used to establish species diversity and cover for each vegetative strata, and help finalize post-reclamation vegetation performance criteria for the Waste Containment Areas.

Grasses and Forbs

Table 1 presents the proposed seed mixture of grasses and forbs for the Waste Containment Areas. The proposed species are native to the region and adapted to the climatic and soil conditions of the Site.

Table 1. Proposed Seed Mixture of Native Grasses and Forbs for the Waste Containment Areas

Common Name	Scientific Name	Seeding Rate in lbs of pure live seed/acre
Grasses		
Bluebunch wheatgrass (var. Goldar)	*Pseudoroegneria spicata*	4.0
Idaho fescue (var. Nespurs)	*Festuca idahoensis*	2.0
Slender wheatgrass (var. Pryor)	*Elymus trachycaulus*	4.0
Sandberg bluegrass (var. High Plains)	*Poa secunda*	2.0
Canada bluegrass (var. Ruebens)	*Poa compressa*	3.0
Indian ricegrass (var. Nezpar)	*Achnatherum hymenoides*	3.0
Green needlegrass (var. Lodorm)	*Nassella viridula*	3.0
Forbs		
Western yarrow	*Achillea millefolium*	2.0
Silvery lupine	*Lupinus argenteus*	5.0
Arrowleaf balsamroot	*Balsamorhiza sagittata*	4.0
TOTAL		32.0

Shrubs

Shrub species that will be used for transplanting in the Waste Containment Areas as seedlings have been selected using results from the 2014 HEP survey of the Rhoads Property Borrow Area. The species include: snowberry (*Symphoricarpos albus*), Woods' rose (*Rosa woodsii*), and serviceberry (*Amelanchier alnifolia*). Rooting depth was an important criterion in selecting shrub species for Waste Containment Areas to ensure that root growth does not extend beyond the soil cover. Six to 12-month old shrub seedlings will be planted at a total density of 125 seedlings/acre (on average). The most common shrub species from the HEP survey was snowberry and it would therefore be planted at the highest density. The planting rate would therefore be 75 snowberry seedlings per acre, on average, and Woods' rose and serviceberry both planted at 25 seedlings per acre, on average. Placement of seedlings would be dictated by landscape features, with some areas planted at higher than average densities and some areas planted at lower than average densities; with the entire area meeting the overall average density of 125 seedlings per acre. A more detailed revegetation specification will be developed for the revegetation contractor at a later time.

2.2 OTHER REMEDIATED AREAS - NEAR THE WASTE CONTAINMENT AREA

Other remediated areas include flat-lying areas where either native soil remains or where one foot of soil is placed over weathered bedrock following soil cleanup and excavation to weathered bedrock. In addition, there are steeper areas [as steep as 2:1 (h:v)] that are currently covered with waste rock and may require soil layer following waste rock removal. There also will be areas remaining following soil cleanup where slopes will be too steep for soil placement or operation of equipment for revegetation. Revegetation will also need to occur along drainages downstream of the Site where contaminated sediments will be removed as part

of remedial construction. Finally, revegetation will need to occur and along the existing East and West Haul Roads, as well as other existing mine site roads that might be removed during the remedial action (RA). The estimated extent of other disturbed areas requiring revegetation is shown in Figure 1. None of these other disturbed areas will be underlain by a geocomposite membrane and so there is no danger of encountering that layer during the revegetation processes.

2.2.1 Flat-Lying Areas

Flat-lying areas will be revegetated in a similar manner regardless of the presence of native soil or the placement of a soil layer, except for site preparation prior to seeding and planting. The revegetation approach for flay-lying areas will follow the procedures previously presented for the Waste Containment Areas and are briefly summarized below.

Areas that will receive a soil layer over weathered bedrock will be ripped to a depth of 18 inches with ripper shanks on the back of a track type dozer. This step will loosen the bedrock prior to soil placement to facilitate deep root penetration. A one-foot soil layer will then be placed and scarified to a depth of 6 to 12 inches to reduce compaction. Soil will be disked to a depth of six inches as a final treatment just prior to seeding and planting. Areas that have native soil remaining will be scarified to a depth of 12 to 18 inches and then disked to a depth of six inches as a final treatment just prior to seeding and planting.

Areas that receive a soil layer may be amended with nitrate-N fertilizer as discussed under Section 2.1.2. Seeding of grasses and forbs in flat-lying areas will be conducted using hydroseeding and hydromulching or other appropriate methods as previously described. Shrubs and trees will be planted with seedlings one year after seeding with the planting procedures described under Waste Containment Areas. The seed mixture in Table 1 will be used for flat-lying areas outside of the Waste Containment Areas. The same shrub species and planting rates as specified for the Waste Containment Areas would also be used for the flat-lying areas and tree species would include mycorrhizal inoculated ponderosa pine (*Pinus ponderosa*) seedlings planted at a density of 250 seedlings per acre, based on results of the 2014 HEP survey.

2.2.2 Steeper Remedial Areas

There are two types of steep slope areas that will require some form of revegetation. The first are slope areas that are currently covered with waste rock, but where slope angles will allow the placement of a soil layer [flatter than 2:1 (h:v)] following the removal of waste rock. The second are steeply sloped areas where bedrock outcrops will be exposed following removal of waste material that have slope angles too steep [steeper than 2:1 (h:v)] for placement of a soil layer.

Areas flatter than 2:1 (h:v) - Areas that receive a soil layer will be ripped to a depth of 18 inches with ripper shanks on the back of a track type dozer. Ripping will be either on the contour or up and down the slope, depending on slope angle and safety requirements. A one-foot soil layer then will be placed and the procedures described above for Flat-Lying Areas (Section 2.2.1) will be followed for soil scarification, soil disking, and soil amendments. Soil scarification and disking may need to be conducted up and down slopes, depending on slope angle and safety requirements.

The seeding method for grasses and forbs will be hydroseeding or another broadcast method using a two-step operation with the first step consisting of seed application with wood-fiber mulch to aid in seed transport and the second step consisting of seed coverage with a harrow or

similar implement. This will be followed with a mulch application such as hydromulching with a wood-fiber mulch and tackifier at a rate ranging from 1 to 2 tons/acre. Shrubs and trees will be planted as mycorrhizal inoculated seedlings grown from seed or cuttings. The stock will be 6 to 12 months old and plantings will take place in late fall as dormant plantings after the first growing season of the seeded grasses and forbs and planting procedures will follow those described under the Waste Containment Areas. The seed mixture in Table 2 would be used for areas flatter than 2:1 (h:v) to provide quick erosion protection and long-term sustainability. Shrub seedlings would include snowberry, Woods' rose and serviceberry planted at equal densities of 50 seedlings per acre for a total of 150 seedlings per acre. All three species have the ability to resprout or spread with underground rhizomes and are effective in providing long-term erosion control. Ponderosa pine seedlings would be planted at 200 seedlings per acre. All seedlings would be planted with consistent spacing to provide uniform coverage on the revegetated slopes.

Table 2. Proposed Seed Mixture of Native Grasses and Forbs for Stepper Remedial Areas

Common Name	Scientific Name	Seeding Rate in lbs of pure live seed/acre
Grasses		
Bluebunch wheatgrass (var. Goldar)	*Pseudoroegneria spicata*	8.0
Idaho fescue (var. Nespurs)	*Festuca idahoensis*	3.0
Slender wheatgrass (var. Pryor)	*Elymus trachycaulus*	8.0
Sandberg bluegrass (var. High Plains)	*Poa secunda*	3.0
Canada bluegrass (var. Ruebens)	*Poa compressa*	5.0
Green needlegrass (var. Lodorm)	*Nassella viridula*	4.0
Forbs		
Western yarrow	*Achillea millefolium*	4.0
TOTAL		35.0

Areas steeper than 2:1 (h:v) - Exposed bedrock areas that are too steep to receive a soil layer will not be seeded or planted because of inaccessible terrain and rock substrate that prohibit the planting of trees and shrubs. However, it is believed that these areas will become colonized with trees and shrubs over time as seed volunteers onto these areas from surrounding vegetation. This process of natural revegetation has occurred on the Site since mining ceased and will continue to an even greater extent in the future as sources of plant material will increase from revegetation efforts.

2.2.3 Downstream Drainages

Drainages downstream of the Site where sediment removal occurs as part of remedial construction will require revegetation of disturbances associated with these removal actions. Revegetation of these areas will occur in substrate that remains following sediment removal and likely will not require the use of a soil layer for successful remediation. However, in sediment removal areas where uncontaminated topsoil does exist and where topsoil removal is required, this topsoil will be removed and temporarily stockpiled adjacent to the disturbed area. These topsoil stockpiles will be seeded with a rapid growing annual grass [e.g. annual rye (*Secale cereal*e or Regreen (*Triticum aestivum* x *Elytrigia elongate*)] for erosion control while the topsoil is in storage.

Sediment removal sites will be revegetated using a combination of hydroseeding of grasses and forbs, hydromulching, and planting of shrub seedlings adapted to the environmental conditions that characterize these drainages. The seed mixture that will be used for sediment removal sites will be riparian adapted species based on the wetland and riparian area survey conducted for the Midnite Mine (WME 2013b) and shown in Table 3. Shrub and tree seedlings would include snowberry, Woods' rose, red osier dogwood (*Comus stolonifera*), Douglas hawthorne (*Crataegus douglasii*), blue elderberry (*Sambucus cerulean*), mountain alder (*Alnus incana*), and Rocky Mountain maple (*Acer glabrum*) planted at equal densities of 25 seedlings per acre for a total of 175 seedlings per acre. All seedlings would be planted with consistent spacing to provide uniform coverage on the revegetated drainages.

Table 3. Proposed Seed Mixture of Native Grasses and Forbs for Downstream Drainages

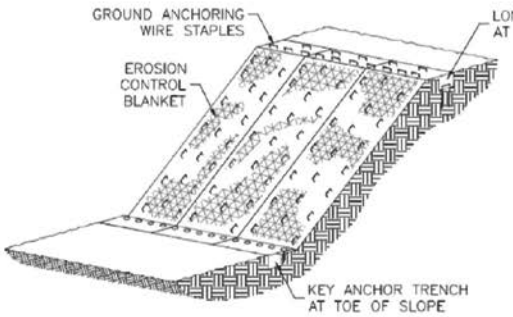
Common Name	Scientific Name	Seeding Rate in lbs of pure live seed/acre
Grasses		
Bluebunch wheatgrass (var. Goldar)	*Pseudoroegneria spicata*	5.0
Slender wheatgrass (var. Pryor)	*Elymus trachycaulus*	6.0
Thickspike wheatgrass (var. Bannock)	*Elymus lanceolatus*	8.0
Blue wildrye (var. Arlington)	*Elymus glaucus*	6.0
Mountain brome (var. Bromar)	*Bromus marginatus*	6.0
Forbs		
Western yarrow	*Achillea millefolium*	2.0
Canada milkvetch	*Astragalus canadensis*	2.0
TOTAL		35.0

A separate wetland restoration plan will be developed for downstream wetland areas that require restoration. Species that will be considered for this restoration activity include some of the following: Baltic rush (*Juncus balticus*), blue wildrye, mountain alder, and red osier dogwood. The final selection of species will be based on the wetland survey noted above (WME 2013b).

Other erosion control materials such as erosion control blankets (ECB) (including jute netting and Excelsior) will be used to stabilize steeper gradients [i.e. slopes that are 2.5:1 (h:v) or greater] and channel bottoms until vegetation establishment occurs. If ECBs are used, seeding would occur prior to the installation of the ECB. The installation of ECBs would use the following procedures (see Figure 5 for examples of ECB installation). The first step in the installation of ECBs on slopes is site preparation. The site would be graded to a relatively smooth surface that is relatively free of weeds, clods, stones, roots, sticks, rills or gullies. The second step in the installation process of ECBs on slopes is to hydroseed the area as previously described. Following seeding, an anchor trench approximately six-inches deep by six-inches wide would be excavated at the top of the slope. The ECB would be anchored in this trench with metal staples or wooden stakes and 12 inches of the terminal end of the ECB would extend beyond the upper end of the trench and later used to cover the trench after soil is replaced in the trench. Staples or stakes would be installed within the trench and across the ECB with a one-foot spacing. The trench would then be backfilled with the excavated soil, the backfilled trench would be hand seeded and the remaining 12 inches of the terminal end of the ECB would be folded over the seeded trench and staked on 12 inch centers. The use of ECBs are anticipated to be limited to

small areas along steeper gradients and in channel bottoms that will need greater erosion control practices for successful plant establishment.

Once the top of the ECB is secured, the ECB is rolled down the slope, starting at the crest. Approximately every 20 feet to 25 feet, the ECB is pulled from the bottom to remove any excess



Erosion Control Blanket
Isometric View

Placement of ECB Showing Trench Construction

Revegetated Area with Completed ECB Placement

Rolling Out ECB on Slope Reclamation

Initial Plant Growth after ECB Placement

MWH

Figure 5. Examples of Installation of Erosion Control Blankets

slack. The goal is to have the ECB in intimate contact with the soil. The edges of the adjacent ECBs are installed so that the edges butt against one another and do not allow the soil to be exposed. The edges are secured with staples and a typical installation would require one staple be placed at three to five foot intervals along the vertical length of the ECBs. Staples are staggered every 18 to 24 inches horizontally across the ECB. The choice between metal staples or wooden stakes would depend on soil conditions and how strong the anchoring system needs to be. Metal staples are typically 11 gauge wire that range from six to 24 inches in length, with a one to two-inch crown. A more aggressive anchoring system would use 6-inch long wooden stakes.

2.2.4 Existing Site Roads

Existing roads that will not be maintained for future use will be dozer ripped at least 18 inches in depth to alleviate roadway compaction and then regraded to blend in with surrounding topography following removal of any contaminated material. Contoured surfaces will be scarifed to a depth of 12 inches with a chisel plow or similar implement to reduce equipment compaction and revegetated following procedures discussed for flat lying areas. The seed mixture, along with the shrub and tree transplanting described above for flat-lying areas (i.e., Table 1) will be used for revegetating existing roads that will not be maintained for future use.

2.3 INTERIM REVEGETATION - AREAS WITH TEMPORARY DISTURBANCES

Temporary disturbances in areas such as roads, growth medium stockpiles, and staging areas will be seeded as soon as possible following disturbance using hydroseeding and hydromulching methods or other appropriate seeding methods described under the Waste Containment Areas (Section 2.1). However, these sites will not receive a growth medium cover since this is only interim stabilization. Sites would be scarified to a depth of 12 inches with a chisel plow or similar implement prior to seeding to reduce soil compaction and to provide a suitable seedbed. The seed mixtures for these areas will include rapid growing annual and perennial grasses to provide erosion protection as quickly as possible. Species to be used for this effort would be selected from the following: annual ryegrass, Regreen, Quickguard (Triticum aestivum x *Secale cereale)*, and slender wheatgrass.

3.0 LONG-TERM MONITORING AND MAINTENANCE PLAN

3.1 INSPECTIONS AND MAINTENANCE

Vegetation surveys will be conducted during each growing season following revegetation until revegetation success criteria are met. The surveys will include identification of bare ground areas, which will be defined as areas greater than 250 ft^2 containing less than 20 percent live cover and areas containing signs of uncontrolled erosion (e.g., rills and gullies, accumulations of sediment or debris). Bare ground areas exhibiting less than 20 percent live vegetative cover for two successive seasons will be re-assessed for nutrients and other conditions that could cause vegetation failure. Pending the assessment results, the affected areas will be retreated and reseeded. Observed rills and gullies will be repaired as soon as practicable, and the area reseeded and stabilized with mulch or riprap. Areas which meet the defined success criteria after three successive survey seasons will be released from future surveys. Revegetation success criteria will be established based on the 2014 HEP survey of the Rhoads Property Borrow Area and will include criteria for percent plant cover and woody plant density.

3.2 NOXIOUS WEED CONTROL PROGRAM

In addition to the above surveys, noxious weed surveys will be conducted annually, beginning with construction activities. Should noxious weeds (as defined by the Spokane Tribe) be documented at the Site, control procedures will be taken to eliminate the weed following a weed control program approved by the Tribal Department of Natural Resources.

The management of noxious weeds at the revegetated Site will follow the *Weed Management Plan (Plan)* prepared for the Rhoads Property (WME 2013a) (See Supplement D-12.1 as part of this Attachment). The *Plan* provides management practices for designated noxious weeds and will be part of the operations, maintenance and monitoring activities during and after the RA is completed. Practices include prevention and proven control methods. The intent is to incorporate those practices that are effective, timely, and technically feasible.

Although the *Plan* presents several viable weed management practices, the control methods that will be selected and implemented at the Site will be dependent on actual site conditions, weed locations with respect to site features and local soil conditions, and progress of previously implemented control methods. As a result, noxious weed management is a dynamic process that will be implemented in conjunction with stakeholder input.

3.2.1 Program Goals

The goals of weed management program at the Site are:

1) Develop and implement a comprehensive noxious weed management program using integrated management practices based upon site conditions and investigation
2) Prevent, contain, and control noxious weeds
3) Maintain healthy plant communities with native, desirable and beneficial vegetation
4) Prevent the introduction and/or the infestation of noxious weeds to neighboring lands.

The noxious weeds included in the *Plan* are a combination of weeds identified by Stevens County and identified in the *Integrated Resource Management Plan (IRMP) for the Spokane Indian Reservation* (AESE, Inc., 2008). Specific major noxious weed species noted in the IRMP are yellow starthistle (*Centaurea solstitialis*), diffuse knapweed (*Centaurea diffusa*), spotted knapweed (*Centaurea maculosa*), Russian knapweed (*Acroptilon repens*), and toadflax (*Linaria dalmatica*).

A DMC/Newmont representative will perform pre-construction and annual weed inspections to locate weed populations, map their locations, estimate the density of the populations, and record the information for use in tracking progress of the management program. The primary goals of the inspections are to identify and record noxious weed species, density, and detailed location information. Identifying the species that are on site is vital in selecting the appropriate control methods and developing a strategy for control. The selected control methods will be based on the type, size, and location of the mapped noxious weeds. The treated area(s) will be monitored and re-inspected annually for new weed introductions and to evaluate the success of the control methods.

3.2.2 Prevention and Control Measures

Weed management is divided into prevention and control measures. The prevention of noxious weed infestations will be a standard practice throughout soil disturbance and reclamation activities and weed control methods will be selected and implemented based on actual site

conditions that include such things as species, size and density of the population, associated species, soil type, and proximity to water. Prevention is the highest priority weed management practice on non-infested lands; therefore protecting weed-free plant communities is the most economical and efficient land management practice. Disturbed land provides opportunity for noxious weeds to invade. The spread of noxious weeds is most likely to occur where soil has been disturbed by activities such as roads, construction sites, the spread of gravel, road fill and or topsoil contaminated with noxious weed seed. Prevention is best accomplished by ensuring that new weed species seed or vegetative reproductive plant parts of weeds are not introduced into new areas, and by early detection of any new weed species before they begin to spread. Methods to prevent introduction or establishment of noxious weeds in areas without infestations are listed in the *Plan*.

Control methods include biological, cultural, mechanical, and chemical. The optimum method or methods for weed management will vary depending on a number of site-specific variables such as soil type and stability, grade, associated vegetation, existing and proposed land use, proximity to water, weed type and stage of growth, and severity of the weed infestation. Specific control methods will be consistent with the IRMP. The *Plan* provides a detailed description of each control method (i.e. biological, cultural, mechanical and chemical) and recommended control methods for designated weed species. The Stevens County Noxious Weed Board lists chemicals recommended for noxious weed control, which are included in the Plan. The IRMP does not specify approved chemicals to be used for weed control, but that pesticide (including herbicide) use should be consistent with to FIFRA (Federal Insecticide, Fungicide and Rodenticide Act). The recommended chemicals will only be used as approved by the Tribal Department of Natural Resources and the use of chemicals will be avoided in traditional gathering areas in order to protect cultural resources.

Selection of control methods often requires an integrated management approach to achieve success. In some cases, only one control method may be warranted, while in other cases a combination of control methods may be needed. All alternatives will be considered before selecting one or more control methods. Additional considerations will be made when selecting control treatments when specific situations arise regarding type, size, and location of weed infestations. Examples of this are perennial versus biennial, broadleaf versus grasses, noxious weeds interspersed with desirable vegetation, large monoculture patches, or small patches requiring spot treatment, infestations located near bodies of water, or on rough, steep, or nearly inaccessible terrain.

Treatment window schedules, based on the control methods chosen and the noxious weeds present, will be established for each treatment area. The best time to treat perennial noxious weeds is in the spring or fall during their active growth phase. The treatment windows selected will depend on the species present, geographic location, elevation, and control methods selected.

The final preparatory step is to determine the priority for areas to be treated. Prioritization ensures that the most important areas are dealt with at the most effective times. High priority areas to be considered are major traffic areas, road cuts and embankments. Ditches and pond embankments should be considered priority treatment areas to prevent offsite contamination by water transported seeds. Important areas of concern include areas that may transport weed seeds. These areas include ditches, streams, roadsides, trails, and land equipment storage sites. Large monoculture patches are of concern wherever they occur and will always be a high priority. Also, small patches of weeds would always be treated quickly to prevent expansion of weed populations.

Once the treatment plan is implemented, detailed records will be kept, and success or failure of treatments will be recorded so as to eliminate unsuccessful treatments. It may be necessary to experiment with different control methods to determine what works best for a specific treatment area.

4.0 REFERENCES

AESE, Inc., 2008. Integrated Resource Management Plan for the Spokane Indian Reservation--Final.

Miller Geotechnical Consultants, LLC (MGC), 2011. Borrow Source Design Investigation Report—Revision 2. Prepared for U.S. Environmental Protection Agency. Prepared by Miller Geotechnical Consultants. Fort Collins, CO.

Schaap, W. and D. DeYoe, 1986. Seedling Protectors for Preventing Deer Browsing. Forestry Research Lab, Oregon State University. Corvallis, OR.

Worthington Miller Environmental LLC (WME), 2013a. Weed Management Plan for the Rhoads Property. Prepared for Newmont USA and Dawn Mining Company. Fort Collins, CO.

Worthington Miller Environmental LLC (WME), 2013b. Wetland Delineation Midnite Mine. Prepared for U.S. Environmental Protection Agency. Fort Collins, CO.

Supplement D-12.1

Weed Management Plan for the Rhoads Property

Weed Management Plan for the Rhoads Property

November, 2013

Prepared on Behalf of:
Newmont USA, Limited and
Dawn Mining Company

Prepared By:
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1.0 INTRODUCTION

This Noxious Weed Management Plan (Plan) provides guidelines for managing noxious weeds during soil excavation and reclamation activities at the Rhoads Property, located on the Spokane Indian Reservation in Washington. This plan provides management practices for designated noxious weeds. Practices include preventive and proven control methods. The intent is to incorporate those practices that are effective, timely, and technically feasible.

This Plan is a component of the *Rhoads Property Plan of Operations* (PoO; MWH 2013), which presents details for obtaining cover soil materials from the Rhoads Property for use with the Midnite Mine Superfund Site (MM) remediation. The PoO includes the proposed reclamation practices that will be conducted to return the Rhoads Property to the identified post-operation land use(s). The PoO contains information regarding project stakeholders, planning considerations, and project schedule.

The noxious weed management practices presented in this Plan will be implemented and evaluated based on pre-construction investigations and subsequent annual inspections performed following phased soil excavation and concurrent site reclamation. Although this Plan presents several viable weed management practices, the control methods that are selected and implemented will be dependent on actual site conditions, weed locations with respect to site features and local soil conditions, and progress of previously implemented control methods. As a result, noxious weed management is a dynamic process that Newmont/DMC will implement in conjunction with stakeholder input.

1.1 Goals and Objectives

The following are the goals of this Plan:

- Develop and implement a comprehensive noxious weed management program using integrated management practices based upon site conditions and investigation.
- Prevent, contain, and control noxious weeds.
- Maintain healthy plant communities with native, desirable and beneficial vegetation.
- Prevent the introduction and/or the infestation of noxious weeds to neighboring lands.

1.2 Noxious Weeds List

The noxious weeds list included in this Plan (see Table 1) is a combination of noxious weeds identified by Stevens County and identified in the *Integrated Resource Management Plan (IRMP) for the Spokane Indian Reservation*. Stevens County is located closest to the Spokane reservation, and Table 1 lists their current noxious weeds, which include all Class A, Class B-designate and certain Class B/C noxious weeds from the 2013 Washington State Noxious Weeds List (WAC 16750) that are currently found growing or are known to have grown previously (reported sightings within the past 5 years in Steven's county), and the level of recommended control. Class A noxious weeds are those not native to the state, that are of limited distribution or are unrecorded in the state, and that pose a serious threat to the state. Eradication may be required and prevention/early detection are

high priority. Class B noxious weeds are those not native to the state, that are of limited distribution or are unrecorded in a region of the state, and that pose a serious threat to that region. Class B-designate means those Class B noxious weeds whose populations in a region or area are such that all seed production can be prevented within a calendar year, and Class C are any other noxious weeds. The complete list of noxious weeds can be found on the Stevens County Noxious Weed Control Board website:

http://www.co.stevens.wa.us/weedboard/noxious_weeds.htm.

The IRMP references the Spokane Indian Reservation Grazing Resource Inventory report, which estimated that 35% of the Reservation was infested with noxious weeds. Specific major noxious weed species noted in the IRMP are yellow starthistle, diffuse knapweed, spotted knapweed, Russian knapweed, and toadflax.

Table 1 gives a listing of the weeds from the Stevens County Designated Noxious Weed List, the State of Washington Class/List Type and the level of recommended control by the county. Also included are any species noted above in the IRMP that were not included in the Steven's county current list (i.e. Russian knapweed and toadflax).

The species listed for required control include Class A, Class B-Designate, and certain Class B/C select species. All are known to be aggressive invaders capable of spreading rapidly.

TABLE 1. STEVENS COUNTY/SPOKANE TRIBE NOXIOUS WEED LIST

Plant	Class List Type (WAC 16750)	Control Recommendation (Steven's County)
Buffalobur (*Solanum rostratum*)	Class A	May be required
Bighead knapweed (*Centaurea macrocephala*)	Class A	May be required
Clary sage (*Salvia sclarea*)	Class A	May be required
Flowering rush (*Butomus umbellatus*)	Class A	May be required
Meadow clary (*Salvia pratensis)*	Class A	May be required
Mediterranean sage (*Salvia aethiopis*)	Class A	May be required
Velvetleaf (*Abutilon theophrast*)	Class A	May be required
Annual bugloss (*Anchusa arvensis*)	Class B-designate	Required
Blueweed (viper's bugloss) (*Echium vulgare*)	Class B-designate	Required
Common bugloss (*Anchusa officinalis*)	Class B-designate	Required
Garden loosestrife (*Lysimachia valgaris*)	Class B-designate	Required
Kochia (*Kochia scoparia*)	Class B-designate	Required
Leafy Spurge (*Euphorbia esula*)	Class B-designate	Required
Musk Thistle (*Carduus nutans*)	Class B-designate	Required
Perennial Pepperweed (*Lepidium latifolium*)	Class B-designate	Required
Punturevine (*Tribulus terrestris*)	Class B-designate	Required
Purple Loosestrife (*lythrum salicaria*)	Class B-designate	Required
Saltcedar (*Tamarix ramosissima*)	Class B-designate	Required
Scotch broom (*Cytisus scoparius*)	Class B-designate	Required
Scotch thistle (*Onopordum acanthium*)	Class B-designate	Required
Tansy ragwort (*Senecio jacobaea*)	Class B-designate	Required
Wild chervil (*Anthriscus sylvestris*)	Class B-designate	Required
Yellow archangel (*Lamiastrum galeobdolon*)	Class B-designate	Required
Black Henbane (*Hyoscyamus niger*)	Class B select/C select	Required
Hoary alyssum (*Berteroa incana*)	Class B select/C select	Required
Longspine sandbur (*Cenchrus longispinus*)	Class B select/C select	Required
Butterfly bush (*Buddleja davidii*)	Class B select	Required if spreading off site
Myrtle spurge (*Euphorbia myrsinites*)	Class B select	Required if spreading off site
Plumeless Thistle (*Carduus acanthoides*)	Class B select/designate	Required
Rush skeletonweed (*Chondrilla juncea*)	Class B select/designate	Required
Yellow Starthistle (*Centaurea solstitialis*)*	Class B select/designate	Required
Diffuse knapweed (*Centaurea diffusa*)*	Class B select	A control strip of at least 40' wide must be maintained along property boundaries, including travel corridors
Spotted knapweed (*Centaurea maculosa*) *	Class B select	
Russian Knapweed (*Acroptilon repens*)*	Class B select	Not required
Dalmatian Toadflax (*Linaria dalmatica*)*	Class B select	Not required
Yellow Toadflax (*Linaria vulgaris*) *	Class C	Not required

*Weed species noted in IRMP; toadflax species was not specified

2.0 NOXIOUS WEED INSPECTIONS

A Newmont/DMC representative will perform pre-construction and annual weed inspections to locate weed populations, map their locations, estimate the density of the populations, and record the information for use in tracking progress of the management program. The primary goals of the inspections are to identify and record noxious weed species, density, and detailed location information. Identifying the species that are on site is vital in selecting the appropriate control methods and developing a strategy for control. All weed observations will be mapped in sufficient detail to re-locate for treatment and monitoring purposes. Mapping details will assist in identifying potential problems that may occur when control methods are implemented, such as damage to non-targeted species. Mapping also may eliminate certain control methods before implementation, by virtue of location. Finally, annual inspection of mapped areas will allow for evaluation of the effectiveness of the control methods.

The inspections will be performed prior to initiating any soil disturbing activities and annually after phased excavation activities and reclamation activities. Inspection forms are included in Attachment A (Form A). The initial and annual inspections will be conducted in late spring/early summer. The weed inspection forms will contain the following principle components:

- Date of inspection.
- Location and estimated density of any target weed species found on the premises.
- Additional weeds of concern found on site.
- Recommended control methods.

Each inspection (i.e. initial and annual) will identify noxious weed populations and locate these populations on a map using a set of symbols to identify species, size of the infestation, density of the population, type of surrounding vegetation, and type and degree of disturbance. The effectiveness of control methods will also be documented in each annual survey. Inspections will be conducted by a plant biologist that is familiar with the identification of noxious weeds in the area and will be completed using a grid mapping system within disturbed areas and a less intensive reconnaissance level survey within undisturbed areas.

The selected control methods will be based on the type, size, and location of the mapped noxious weeds. The treated area(s) will be monitored and re-inspected annually for new weed introductions and to evaluate the success of the control methods. Follow-up inspections will be documented using Inspection Follow-up forms (Attachment A, Form B).

3.0 WEED MANAGEMENT

Weed management is divided primarily into prevention and control. The prevention of noxious weed infestations will be a standard practice throughout soil disturbance and reclamation activities and weed control methods will be selected and implemented based on actual site conditions that include such things as species, size and density of the population, associated species, soil type, and proximity to water.

3.1 Prevention

Prevention is the highest priority weed management practice on non-infested lands; therefore protecting weed-free plant communities is the most economical and efficient land management practice. Disturbed land provides opportunity for noxious weeds to invade. The spread of noxious weeds is most likely to occur where soil has been disturbed by activities such as road and trail cuts, construction sites, the spread of gravel, road fill and topsoil contaminated with noxious weed seed, or overgrazing. Other known methods of weed introduction include contaminated hay, straw, and mulch, movement of contaminated vehicles and equipment across uncontaminated lands, wildlife fur, and human clothing.

Prevention is best accomplished by ensuring that new weed species seed or vegetative reproductive plant parts of weeds are not introduced into new areas, and by early detection of any new weed species before they begin to spread.

Methods to prevent introduction or establishment of noxious weeds in areas without infestations include:

- Identification of existing conditions, disturbances, and activities that could facilitate noxious weed invasion.
- Identification and eradication of small, new weed populations.
- Protection of weed free areas from initial introduction.
- Monitoring and control implementation to prevent recurrence.
- Timely revegetation and reclamation of disturbed sites
- The use of weed free seeds, mulch and straw.
- Prioritization of weed management along entry areas to the Rhoads Property.
- Limiting vehicle movement between weed areas and decontamination/rinsing of vehicles when feasible. (It will be a standard practice to pressure wash the exterior of any vehicles, ATV's, or mobile equipment prior to entering the Spokane Indian Reservation [refer to Section 2.2 of the PoO]).

3.2 Control

3.2.1 Control Methods

Control methods include biological, cultural, mechanical, and chemical. The optimum method or methods for weed management will vary depending on a number of site-specific variables such as soil type and stability, grade, associated vegetation, existing and proposed land use, proximity to water, availability of irrigation water, weed type and stage of growth, and severity of the weed infestation. Specific control recommendations will be included on the inspection form(s) on a case by case basis. Control methods will be consistent with the IRMP.

Biological Control

Biological weed control involves using natural enemies for the control of specific weed species. Biological control is defined as "controlling pests by interfering with their ecological status, such as introduction of a natural enemy into the environment." This can sometimes be hard to establish, and can take 5 to 10 years for partial control because it involves moving an organism to a new environment. However, once established, biological controls can be less labor intensive and more continual than other methods of control. Biological control can be used as part of an integrated pest management program to destroy the invasive plant tissues and cause stress to those plants, thus reducing competition between weeds and desirable species.

Cultural Control

Cultural control is probably the most important control to utilize in preventing weed introduction and reducing spread once weeds are present. Cultural control employs common sense to prevent formation and spread of infestations. Some forms of this control include: using weed free seed and mulch when planting; cleaning tillage and cutting/harvesting equipment before moving from a weedy area to a non-weedy area; making sure topsoil is not harboring noxious weeds before stripping, stockpiling and reapplying; and education to be able to identify noxious weeds, becoming familiar with their life cycles, and determining how an weed population got started, why it still exists, and how to remove it with the least impact to the surrounding environment.

Mechanical Control

Mechanical control is the physical removal of weeds from the soil medium and includes tilling, mowing, pulling, cutting, or burning undesirable plant species. Tillage is most effective prior to seeding and establishment of desirable vegetation. The tillage method of weed control can be effective in eliminating noxious perennial weeds when repeated at short intervals (every 1-2 weeks) throughout the growing season. Tillage has the drawback of indiscriminately impacting all vegetation interspersed with weeds in established areas and can eliminate competitive, desirable vegetation leaving behind a prime seedbed for weeds to reinvade. Mowing can be an effective method for controlling the spread of an infestation and preventing the formation and dispersal of seeds. Mowing is most effective on weeds which spread solely or primarily by seed. In order to achieve this, it must be repeated at least twice during the growing season prior to, or shortly after bloom. Also, even the most intense mowing treatment will not kill hardy perennial weeds. Burning or cutting performs essentially the same function as mowing.

Chemical Control

Chemical control consists mostly of selective and non-selective herbicides. Considerations for chemical controls include; herbicide selection, timing of application, target weed, desirable plant species being grown or that will be planted, number of applications per year and number of years a particular species will need to be treated for desired control. Also important are the health and safety factors involved, and the need to consider undesirable impacts. The use of herbicides will be in compliance with all Tribal, Federal and State laws on proper use, storage, and disposal. The chemical application will be done by a licensed contractor in accordance with all applicable laws and regulations and all label instructions will be strictly followed. Applications of herbicides would not be permitted when the instructions on the herbicide label indicate conditions that are not optimal.

Application of herbicides will be suspended if the following conditions exist:

- Wind velocity exceeds 20 miles per hour;
- Snow or ice covers the foliage of noxious weeds; or
- Precipitation is occurring or imminent.

3.2.2 Identification and Recommended Controls by Species

Attachment B provides the identification and the recommended chemical, biological, cultural and mechanical management controls for the designated weed species, courtesy of the Stevens County Noxious Weed Control Board website.

3.2.3 Approach for Selection of Controls

Selection of control methods often requires an integrated management approach to achieve success. In some cases, only one control method may be warranted, while in other cases a combination of control methods may be needed. For instance, it may be necessary to use a total vegetation control herbicide on a small area followed by reseeding to the desired vegetation. Another example of integrated management is mowing or cultivating in spring or early summer followed by chemical treatment in the fall after regrowth has occurred. Infested areas may be inaccessible with large equipment and only mechanical or chemical control by hand may be possible. All alternatives will be considered before selecting one or more control methods.

Additional considerations will be made when selecting control treatments when specific situations arise regarding type, size, and location of weed infestations. Examples of this are perennial versus biennial, broadleaf versus grasses, noxious weeds interspersed with desirable vegetation, large monoculture patches, or small patches requiring spot treatment, infestations located near bodies of water, infestations located on topsoil piles, or on rough, steep, or nearly inaccessible terrain.

Treatment window schedules, based on the control methods chosen and the noxious weeds present, will be established for each treatment area. The best time to treat perennial noxious weeds is in the spring or fall during their active growth phase. Different species will have different

optimum treatment times even with the same type of control. Perennial weeds usually grow vegetatively in the spring, flower and seed in late spring and early summer, enter dormancy during the summer and actively grow again in the fall. The treatment windows selected will depend on the species present, geographic location, elevation, and control methods selected.

The final preparatory step is to determine the priority for areas to be treated. Prioritization ensures that the most important areas are dealt with at the most effective times. High priority areas to be considered are major traffic areas, road cuts and embankments. Ditches and pond embankments should be considered priority treatment areas to prevent offsite contamination by water transported seeds. Important areas of concern include areas that may transport weed seeds. These areas include ditches, streams, roadsides, trails, land equipment storage sites. Large monoculture patches are of concern wherever they occur and should always be a high priority. Also, small patches of weeds should always be treated to prevent expansion of weed populations.

Once the treatment plan is implemented, detailed records will be kept, and success or failure of treatment will be recorded so as to eliminate unsuccessful treatments. It may be necessary to experiment with different control methods to determine what works best for a specific treatment area.

3.2.4 County and Tribe Recommendations for Chemical and Biological Control Methods

The Stevens County Noxious Weed Board lists chemicals recommended for noxious weed control (included in Attachment C). The IRMP does not specify approved chemicals to be used for weed control, but that pesticide (including herbicide) use should be consistent with to FIFRA (Federal Insecticide, Fungicide and Rodenticide Act).

The recommended chemicals will only be used as approved by the Tribal Department of Natural Resources. The IRMP states that pesticides (including herbicides) are being phased out on the reservation. The IRMP states the following previsions from the Law and Order Code regarding pesticides:

- No aerial application unless specifically authorized by the Tribal Council
- Drift must be prevented
- Pesticide (herbicide, insecticide, fungicide and rodenticide) will not be permitted within any riparian corridor, any closer than 500 feet from the center of any riparian zone, nor within 500 feet of any wellhead protection area, without special clearance through the Tribal Water Resources Program. (Riparian corridors are defined as extending 100 feet beyond the outer boundaries of a riparian zone, and 1/2 mile from a river.)

The IRMP also requires that spraying of pesticides be avoided in traditional gathering areas in order to protect cultural resources.

With the desire to reduce of the use of pesticides, the IRMP mentions implementing the use of biological controls. The Stevens County weed management website lists many biocontrol agents

that are effective for some of the major noxious weed species. Table 2 lists these species and the corresponding biocontrol(s). Details of the insects used for as biocontrols are included in Attachment D.

TABLE 2. BIOCONTROL LIST

Weed Species	Biocontrol Agent
Dalmatian Toadflax	Mecinus janthinus
Diffuse knapweed, Spotted knapweed	Cyphocleonus achates, Larinus minutus, Metzneria paucipuntella, Sphenoptera jugoslavica
Leafy Spurge	Aphthona flava
Purple Loosestrife	Galerucella pusilla
Rush skeletonweed	Cystiphora schmidti, Eriophyes chondrillae, Puccinia chondrillina
Plumeless Thistle	Rhinocyllus conicus
Yellow Starthistle	Bangasternus orientalis, Eustenopus villosus

4.0 PLAN REVISIONS AND REPORTING

This Plan will be revised as appropriate, per compliance to any revisions made to the Spokane Tribe IRMP, Stevens County Weed Management Plans or State of Washington noxious weed species lists.

Information obtained from the weed inspections including control selection and implementation will be recorded on the weed inspection and follow-up forms (Attachment A). The completed inspection forms and mapping results of any identified weeds will be provided to the Tribe DNR within 60 days of the inspection.

5.0 REFERENCES

Integrated Resource Management Plan (IRMP) for the Spokane Indian Reservation, FINAL, May 2008.

Steven's County Noxious Weed Control Board website: http://www.co.stevens.wa.us/weedboard/noxious_weeds.htm

Attachment A
Weed Inspection and Follow-Up Forms

Rhoads Borrow Property

Weed Inspection Form A

Date: _____

Inspector: _____

Page: ___ of ___

Location: Coordinates:

Species:

Estimated Plant Density:

Description (approx. size of area infested, surrounding vegetation, soil type, topography, proximity to water, disturbances etc.):

Photos:

Comments / Control measures to be implemented:

Date controls implemented:

Rhoads Borrow Property

Weed Inspection / Follow-Up Form B

Date: _____

Inspector: _____

Page: ___ of ___

Location: Coordinates:

Species Treated:

Controls applied:

Species present: yes no

If present: Approx. density of plants or size of area infested:

Photos:

Comments / Follow-up measures to be implemented:

Location: Coordinates:

Species Treated:

Controls applied:

Species present: yes no

If present: Approx. density of plants or size of area infested:

Photos:

Comments / Follow-up measures to be implemented:

Attachment B
Weed Identification and Recommended Controls

Buffalobur

Solanum rostratum Dun. Nightshade family

Key identifying traits

- Deeply lobed watermelon type leaves
- Yellow, 5-lobed flowers through out summer
- Extremely spiny plant including stems, leaves and round seed pods
- Grows up to 2 feet tall with erect stem and ascending branches

Biology and ecology

- An annual with a tap root
- Native to U.S. but not to Washington State
- Common in some western wastelands and prairies
- Serves as a host for Colorado potato beetle
- Common on sandy soils but grows in most soils
- Occasionally found in gardens in our area probably introduced as a seed contaminant
- Not a highly competitive species and may not reproduce well in our climate

Control

Prevention – Learn to identify plants; know your property; beware of unusual or unintended plants in gardens and around bird feeders
Biological – none contemplated for this U.S. native; should have natural enemies already in native areas
Cultural – Good competitive vegetative cover helps
Mechanical – Cultivation, pulling and repeated cutting all work well; be careful to avoid puncture wounds from spines
Chemical – Several work well at label rates; care must be used if in a garden site to avoid off target plant damage

Where found – To date in Stevens County, found exclusively in garden settings or the odd plant under bird feeders with one or two individual plants being reported most years. Believed to be a contaminant seed in garden seed packets and birdseed mixtures.

Clary sage

Salvia sclarea L. Mint family

Key identifying traits

- Has characteristic square stem of mint family
- Flower color range from white-pink-blue-purple
- Flower petals divided into arching upper lip and a lower lip, also characteristic of the mint family
- Large showy bract under each cluster of flowers
- Erect plant with many branched flower spikes
- Plant covered with gland tipped hairs with an odor
- Leaves large and mostly basal with toothed margin

Biology and ecology

- A tap rooted biennial 20 inches to 6 feet tall
- Sometimes grown as an ornamental and a traditional producer of essential oils used in perfumery, distilling and tobacco flavoring
- First year growth as a fuzzy hairy basal rosette
- Quite competitive on dry sunny sites
- Can hybridize with Meadow clary if on same site

clary sage rosettes

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area

Biological – None- the closely related Mediterranean sage in Idaho is apparently reduced by a weevil

Cultural – Good vegetative cover helps but does not totally prevent spread from adjacent disturbed areas

Mechanical – Will not withstand regular tillage; pulling and digging are effective prior to seed production; cutting may be effective if repeated

Chemical – Several effective at label rates if a penetrating surfactant is used to get through the hairs to the leaf surface

Where found – Only one rangeland site of less than one acre in south central Stevens County has a pure wild Clary sage infestation but other sites of predominately Meadow clary shows indications of hybridization of the two.

Bighead knapweed

Centaurea macrocephala Puschk. Sunflower family

Key identifying traits

- Single, 1 to 3 inch wide, yellow flower heads on erect, unbranched, leafy stems 2 to 5 feet tall
- Broadly lance-shaped leaves have sharp pointed tips, shallowly toothed edges and rough surfaces
- Leaves change from simple stalks to winged stalks to no stalk as they become smaller up the stem
- Bracts surrounding the flower head are wider than long, mostly brown with lighter margins appearing in 8 to 12 layered rows
- Seeds are $\frac{1}{4}$ inch long with pale to brown plumes

Biology and ecology

- A perennial that reproduces by seed with up to 200 seeds per head
- Sold commercially as an ornamental under the names "Lemon fluff" or "Globe centaury"
- Found only as an escaped ornamental to date but competes vigorously in heavy grass stands in areas with loam soils and 20 inches of precipitation
- Remains green in August & palatable to livestock
- Generally only one or two stems 2 to 3 feet tall with 1 1/2 inch heads when found competing with grass but as an ornamental may be five feet tall with 3 inch heads on each of multiple stems

Control

Prevention – Learn to identify plants; know your property; beware of aggressive ornamentals
Biological – None known
Cultural – Competing vegetation helps limit spread
Mechanical – Repeated tillage, digging or cutting are all effective if properly timed to prevent seeds
Chemical – Several effective at label rates

Where found – Only occasionally reported as an intentionally planted ornamental in Stevens County but is in known as an aggressive escaped ornamental in northern Pend Oreille County and in Whitman and Okanogan Counties in Washington

Velvetleaf

***Abutilon theophrasti* Medic.** Mallow Family

Key identifying traits

- Large (3-8 inch wide) heart shaped leaves grow alternately on long slender stalks
- Entire plant is soft with short velvety hairs
- Yellow to yellow-orange 5-petal flowers; ½-1 inch wide, solitary generally in upper leaf axils
- 1-8 feet tall, rather linear in appearance with branching occurring in upper portion of plant
- Distinctive circular cluster of 12-15 seed pods produce purplish brown, kidney shaped seeds

Biology and ecology

- Tap rooted, summer annual reproducing by seed
- Does not tolerate frost
- 700-17,000 hard-coated seeds per plant remain viable when buried for more than 50 years
- Native to Asia where fiber is used to make rope, bags, nets and paper-introduced in North America in 1700's as a potential fiber crop
- Serious row crop (corn/soybeans) weed in the mid-West
- Found infrequently in gardens, along fencelines and roadsides throughout Washington
- Leaves are horizontal by day, changing to nearly vertical at night

Control

Prevention – Learn to identify plants; know your property; beware of contaminated vegetable seeds and feed screenings

Biological – Some active in mid-West

Cultural – Healthy established vegetation helps, but plants can produce seed under shade of a cover crop

Mechanical – Pulling, digging and cultivating prior to bloom will kill plants; burn plants with seed pods-even immature pods will ripen after cutting

Chemical – Several are reported to control the plant but no specific data available in Pacific Northwest

Where found – Known to occur only as single plants found in gardens in Stevens County

Created by Stevens County Noxious Weed Control Board, April 2000

Mediterranean sage

Salvia aethiopis L. Mint Family

Key identifying traits

- Wooly, upright, single-stalked plant; can reach 4' tall
- 4-12" long basal leaves have irregular margins and are on short stalks; upper leaves are opposite, smaller and clasp the stem
- Flowering stock is square in cross section and develops into a multi-branched inflorescence resembling a candelabra
- Many small white flowers are found in whorls at the tips of branches; 2 upper petals form a lip like a hooked beak, 3 lower petals form 3 lobes
- Leaves and stalks have a strong sage-like odor when crushed

Biology and ecology

- Biennial or short-lived perennial with a stout taproot; spreads only through seed
- Mature plants break off just above the basal leaves allowing the seeds to be dispersed through tumbling or dragging by animals or equipment
- Grows in dryer, disturbed sites such as roadsides, and abandoned fields and seems to prefer south-facing slopes with loose, gravelly soils
- Mediterranean sage is native to southern and southeastern Europe

Control

Prevention – Learn to identify plants; can be confused with common mullein in the rosette stage-must learn to distinguish; cleaning equipment is crucial

Biological – There are some established in states where infestations are large: not appropriate here

Mechanical – Digging at least 2-3" of taproot, mowing, or cultivation prior to seed set can all be effective

Chemical –Several herbicides- picloram, clopyralid or aminopyralid with 2,4-D & surfactant added should work well

Where found – ***Found only on one site in west-central Stevens Co as of 2011.***

Meadow clary

Salvia pratensis L. Mint family

Key identifying traits

- Has characteristic square stem of mint family
- Flowers blue to violet with small green bracts
- Flower petals divided into arching upper lip and a lower lip, also characteristic of the mint family
- One to two feet tall with few flowering spikes
- Leaves have bumpy surface and pointed tip
- Plant covered by small glandular hairs
- Plants range from strong odor to no odor

Biology and ecology

- A fibrous rooted perennial
- Sometimes grown as an ornamental
- Stressed or damaged plants can send up new plants from the remaining roots
- Quite competitive on dry sunny sites and able to live on deeper, less well drained sites as well
- Can hybridize with Clary sage if on same site

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt or contaminated equip.
Biological – None; the closely related Mediterranean sage in Idaho is apparently reduced by a weevil
Cultural – Good vegetative cover helps but does not prevent infestation and spread
Mechanical – Not believed to withstand regular tillage; pulling cutting and digging are effective only if repeated with diligence to kill regrowth and new plants from roots
Chemical – Several effective at label rates if a penetrating surfactant is used to get through the hairs to the leaf surface and timing is geared to promote translocation to the roots

Where found –Less than a total of 10 acres near Waitts Lake, west of Addy and in the Kettle Falls area of Stevens County. May be hybridizing with the biennial Clary sage.

Created by Stevens County Noxious Weed Control Board, April 2000; Updated Jan 2004

Flowering rush

Butomus umbellatus Butomacea Family

Key identifying traits

- Freshwater aquatic plant resembling a large sedge
- Leaves are sword shaped, narrow and triangular in cross section
- Flowering stems are fleshy and spirally twisted
- Showy, pink flowers grow in umbels at ends of stems (but not all plants flower) June-August
- Grows as a shoreline emergent plant with upright, stiff foliage or as a wholly to partially submersed plant with flexible leaves
- Fleshy, rhizomatous roots with "bulbils"

Jenifer Parsons, DOE, holding plant. Top & root photos hers.

Biology and ecology

- Perennial plant reproducing by rhizomes, bulbils and, possibly, seeds
- Grows along muddy shores of lakes, streams, ditches, but can grow out into water as deep as 20' and be totally submersed
- It is the single member of it's family-it is not a true "rush"
- It is native to Eurasia

Control

Prevention – Learn to identify plants; looks like a large grass or sedge-must learn to distinguish

Biological – No known biological control in our area

Cultural – Does not seem to invade healthy shoreline communities as quickly as disturbed sites, but it can still invade

Mechanical – Digging and covering control trials are currently underway-it may be difficult to remove all root parts. Raking and cutting are not effective

Chemical –Plants that are mostly above water are more susceptible to foliar treatment with aquatic herbicides, but control has been inconsistent. Trials are underway in Montana, Idaho and WA.

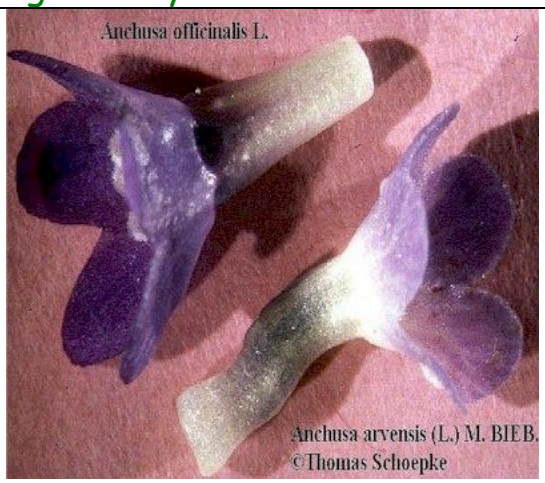
Flower Photo Leslie J. Mehrhoff, Univ. of Connecticut, Bugwood.org

Where found – Found along shore and well out into the waters of Lake Spokane in 2010. Also found in Whatcom, Benton, Yakima and Pend Oreille counties. Large infestation in Flathead Lake, MT and smaller populations elsewhere in the state and in Idaho.

Annual (small) bugloss

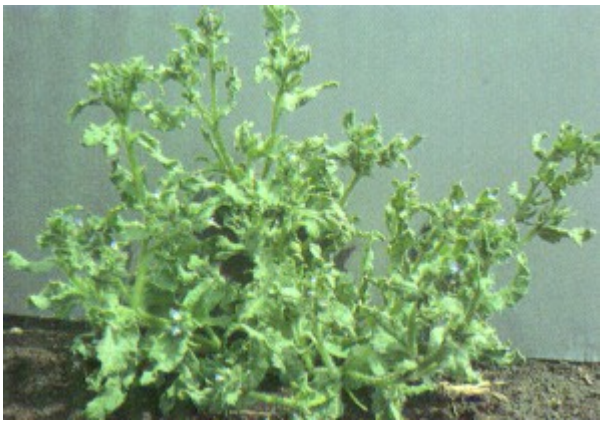
Anchusa arvensis L. Borage family

Key identifying traits



- Leafy, coiled flower stems typical of the Borage family
- Small five-lobed blue flowers with white centers
- Flower tubes white with a distinctive curve-unlike common bugloss which has a straight tube
- Bristly hairs arise from warty bumps covering plant
- Rough wrinkled lance-shaped alternate leaves
- Plant grows 6" to 24" tall with branching stems

Biology and ecology



- Annual plant-reproduces only by seed
- Member of the Borage family
- Germinates primarily in spring but some wait until fall
- Weed of waste ground and cultivated fields
- Especially in small grains both spring and fall
- Seeds viable in soil for several years

Control

Prevention – Learn to identify the plant; prevent establishment in fields by killing infestations along field borders and entrances; prevent spread from infested fields by cleaning swathers and combines

Biological – None available at this time

Cultural – Good competitive vegetation helps prevent establishment

Mechanical – Cultivation kills existing plants; pull or hoe plants in waste areas and burn them if they have produced seeds; seeds are long lived and can germinate over a period of months each year

Chemical – Some effective; best with adjuvant added to help penetrate through hairs; most effective when applied to young plants, prior to bud formation

Where found – Common in Williams valley and moving north in the Chewelah area of Stevens County. Primarily in farming areas-some sites believed to be from contaminated alfalfa seed. 2003- several roadside patches found in the north half of the county. Watch r.o.w. & fill areas.

Created by Stevens County Noxious Weed Control Board, March 2000; Updated January 2004

Blueweed *(Viper's bugloss)*

Echium vulgare Borage Family

Key identifying traits

- Blue flowers with pink stamens arranged on the upper side of short, scorpion-tail stems; as the stem straightens flowers open a few at a time, stalk to tip
- Stems and leaves are covered with stiff hairs which are swollen at the base
- Swellings have a reddish or blackish tint giving the stem a flecked appearance
- Single stalks or multiple branches 1-3 feet tall

digging mature blueweed

Biology and Ecology

- Biennial to short-lived, tap-rooted perennial
- Reproduces by seed and can regrow from crown
- Flowers June through early fall
- Seeds are produced in a four-part nutlet and are very hard and long-lived
- Thrives in sunny sites-does not tolerate shade
- Invades lawns, ornamental areas, roadsides and range land

blueweed rosette-bumpy

Control

Prevention- learn to identify plant; know your property; beware of fill dirt, hay and seed from outside your area

Biological- no known biological controls approved for Blueweed at this time

Cultural- competitive vegetation helps; keep lawns healthy (water/feed) to increase the vigor of turf and reduce open spaces for invasion

Mechanical- mowing can stop seed production in short term, but plants can re-grow and bloom shorter than the cutting blade; digging or pulling when soil conditions are moist is effective-control prior to bloom or, if after, bag or burn plants in a pile as appropriate (check local ordinances); seed bank will be present for years

Chemical- several herbicides are effective at label rates; best timing is in early spring when plants are in the rosette stage

blueweed flowers

Where found- common in the extreme southeastern portion of Stevens County, with only minor outbreaks elsewhere.

Common bugloss

Anchusa officinalis L. Borage family

Key identifying traits

- Flowers are blue to purple with white centers and straight rather than the curved tubes found in annual bugloss
- Flower stem initially coiled like a fiddleneck, but uncoils as flowers open
- Leaves are fleshy with a rough hairy surface; basal leaves to 8 inches long and oblong, stem leaves narrow and smaller
- Mature plants can have several flowering stalks
- Each flower can produce four seeds (nutlets)

Biology and ecology

- A perennial plant 1 to 2 feet tall with a tap root
- First year growth is a rosette; second year a single flower stalk, subsequently many stalks
- Flowers from May through July
- Spreads by seeds only
- Grows in dry areas including dry land alfalfa fields where it's succulent leaves cause mold in baled hay

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area

Biological – None known at this time

Cultural – Good competitive vegetation helps prevent infestation but doesn't stop it entirely

Mechanical – Cultivation, digging and pulling can all be effective if sufficient tap root is removed

Chemical – Several effective at label rates at or before bud stage with a surfactant added to penetrate the leaf hairs

Where found – Common in the Enterprise region of Stevens County and occasionally elsewhere in the southern 2/3 of the county. Large infestations in northern Spokane County; first reported in Washington state in Spokane County.

Created by Stevens County Noxious Weed Control Board, February 2001; Updated Jan 2006

Garden loosestrife

Lysimachia vulgaris L. Primrose Family

Key identifying traits

- Showy, yellow, primrose-like terminal flower clusters
- Five petals and reddish brown calyx margins
- Both the stems and leaves are softly hairy
- Lance shaped, opposite or whorled leaves
- Leaves are dotted with black or orange glands
- Erect plant growing 4 feet tall or more
- The fruit is a dry capsule

Biology and ecology

- Perennial; spreads by seeds and rhizomes
- Inhabits wetland and marshy sites
- In our county, flowering period has been observed to be between July and September
- Reported to be out-competing purple loosestrife (no relation) in western WA
- Blooms are an indication of well established plants since young plants don't appear to flower

Control

Prevention – Learn to identify plants; know your property; beware of ornamental starts and contaminated recreational equipment

Biological – None known and no research to date

Cultural – In combination with other methods, competitive vegetation & minimizing site disturbance helps

Mechanical – Small infestations can be dug, bagged and disposed of taking care not to disperse seeds; black plastic covers may be effective with small patches; limited by rhizomes and moist sites

Chemical - Broad leaf and non selective herbicides may be effective depending on site; special care, permit and license required for aquatic applications

Where found – The only recorded site in eastern WA is a limited infestation along the northern shoreline of Loon Lake in Stevens County.

Created by Stevens County Noxious Weed Control Board, February 2000; Updated Jan 2004

Kochia

Kochia scoparia L. Goosefoot family

Key identifying traits

- Many-branched erect plant 1 to 6 feet tall
- Leaves are ½-2 inches long, alternate, narrow, lance shaped with hairy margins & undersides
- Stems round, usually softly hairy & often red-tinged
- Inconspicuous flowers form dense spikes in leaf axils
- Flowers are usually surrounded by cluster of long hairs
- Short, upper flower spikes often nod

Biology and ecology

- Tap-rooted summer annual; reproduces by seed
- Seeds are generally only viable for 1 or 2 years
- Flowering and seed production from July through October; several flushes of seedlings per season
- Common in Western US in cultivated fields, gardens, roadsides, ditchbanks and waste areas
- Readily grazed by livestock although it sometimes contains high nitrate levels and can be toxic
- A serious economic problem in crops
- Drought resistant but does well under irrigation
- Old plants spread seeds while tumbling

kochia seedlings can form a solid mat

Control

Prevention – Learn to identify plants; know your property; control kochia along fencelines and roadways to reduce seed scatter by tumbling plants

Biological – No known biological controls

Cultural – Competitive vegetation helps avoid invasion and winter wheat withstands kochia better than spring wheat

Mechanical – Grazing and mowing will not stop seed production or kill the plant which will resprout from the stem; pull, hoe or cultivate to kill kochia; shallow tillage helps force seeds to sprout or decay

Chemical – Several effective at label rates, but kochia is often resistant to triazine & sulfonylurea herbicides; rotating herbicides with different modes of action helps prevent resistance development

Where found - Limited numbers of plants but widely scattered distribution in Stevens County; mainly along roadsides, railroad, parking lots, gravel pits and piles, other highly disturbed sites.

Leafy spurge

Euphorbia esula L. Spurge Family

Key identifying traits

- Showy pairs of yellow-green, heart shaped bracts (often mistaken for flowers)
- Entire plant contains a milky juice (sap)
- Roots are extensive with pink buds for new shoots or roots
- Leaves are alternate, narrow, 1 to 4 inches long
- Single stalks or multiple stems, up to 3 feet tall
- True flowers are yellow-green but inconspicuous
- Seeds are contained in a 3-celled capsule

Biology and ecology

- Perennial; reproduces by vigorous root stalks and seed; seed capsules explode when dry, projecting seeds up to 20 feet
- Sap can be an extreme irritant to man or beast
- Invades a wide variety of sites- dry hillsides, wet meadows, riparian areas, roadsides
- Once plants are well established (3 years) large nutrient reserves in the extensive root system make control difficult & expensive and eradication nearly impossible
- Forms dense stands grazed only by goats/sheep

Control

Prevention- Learn to identify the plant; know your property; beware of fill dirt, hay and seed from outside your area

Biological – Several used in combination with sheep and with herbicides to combat extensive infestations in the mountain west; trial colony only in Stevens Co

Cultural – Competitive vegetation helps but does not stop infestations

Mechanical – Persistent, almost zealous digging or cultivation is required to be at all successful

Chemical – A few are effective if applied using the correct rates, timing, mixtures, and adjuvants; most effective herbicides generally require a pesticide license. Many years required. Monitor for several years after last top growth is seen.

young plants

Where found - Small infestations found scattered around Stevens County; active ID and cost share control programs have limited the acres involved to less than 100

Created by Stevens County Noxious Weed Control Board, February 2000; Updated Jan 2004

Musk thistle

Carduus nutans Sunflower Family

Key identifying traits

- Large, 1 ½ -3 inch diameter bent over (nodding) flower heads
- Flowers are rose, violet or purple and one per stem
- Bracts surrounding flowers are broad and spine tipped
- Upper flower stems typically bare and lower stems are winged with spines
- Leaves are dark green, lobed and spiny with light green midrib
- Older plants multi-branched & grow to over 6 ft tall

Biology and ecology

- Biennial or sometimes a winter annual
- Invades pasture, range ,forest, road sides and fields
- Reproduction of musk thistle is entirely by seed
- Numerous small roots in the fall; large fleshy taproot in spring
- Only 2% of seeds have pappus and are wind borne
- Can form dense stands, discouraging animal presence and forage

Control

Prevention- Learn to identify the plant; know your property; beware of fill dirt, hay, manure and seed from outside your area

Biological – Biological controls well established in all parts of the county and dramatically reducing musk thistle stand density and vigor

Cultural – Competitive vegetation helps reduce open spaces for invasion

Mechanical – Mowing, cutting, digging, pulling or cultivation are all effective if done prior to flowering and repeated if necessary; seed bank will be present for years

Chemical – Several effective at label rates; best timing is when plants are in rosette stage or early bolt-spring and again in fall

Where found- Common in part of the Harvey Creek drainage with only minor outbreaks elsewhere in Stevens County. Population has decreased with effective biocontrol establishment

Created by Stevens County Noxious Weed Control Board, March 2000; Updated Jan 2004

Perennial Pepperweed

Lepidium Latifolium L. Mustard Family

Key identifying traits

- Small white flowers in dense clusters mainly at ends of stems
- Four flower petals, each 1/16 inch long
- Plant grows from 1 to over 3 feet tall
- Leaves are alternate, lance like, bright green to gray-green, sometimes toothed on margins
- Basal leaves have petioles (stalks) & are larger than upper leaves which are clasping
- Two small, rounded, flattened, slightly hairy reddish-brown seeds per fruit silicle (pod)

Biology and ecology

- A perennial spread by roots and seeds
- Prefers wet sites but can invade cropland- more often in pastures, ditches and roadsides
- Flowering occurs from early summer to fall
- Extensive root system and waxy layer on leaves and stems make it difficult to control
- A wide and fluctuating range of winter temperatures favors seed germination

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area- due to difficulty controlling this weed prevention and early control are important

Biological – None known at this time

Cultural – Competitive vegetation aids in control

Mechanical – Repeated hand pulling and digging are good for limited infestations-clean tillage equipment to avoid spreading roots

Chemical – Some broad leaf herbicides are effective at label rates, but must be used with care around moist sites

Where found – There are about ½ dozen known sites in Stevens County. Sites range from extremely dry gravel sites to very moist, grassy sites.

Created by Stevens County Noxious Weed Control Board, January 2001; Updated Jan 2004

Puncturevine

Tribulus terrestris L. Caltrop Family

Key identifying traits

- Usually grows prostrate, with trailing stems forming a mat from 1-10 feet in diameter
- Opposite, compound leaves are divided into 4-8 pairs of small, oval leaflets covered with fine hair
- Yellow, 5-petaled flowers are up to ½" wide and are formed at the leaf axils
- When mature and dry, the 5-part fruit breaks into hard, tack-like structures with sharp spines

Biology and ecology

- Native to southern Europe, this annual spreads only from seeds that can remain dormant 4-5 yrs
- Grows best on dry sandy soils, but tolerates most soil types. Intolerant of freezing temperatures
- Foliage is toxic to livestock, especially sheep, when consumed in quantity
- The hard, spiny burs damage wool, are objectionable in hay, can be injurious to livestock and will puncture bicycle tires and feet
- Other names: goathead, Texas sandbur, tackweed

Control

Prevention – Learn to identify plants; pay particular attention at roadside pullouts and along driveways; monitor tires, shoes, animals for burs

Biological – Two weevils, *Microlarinus lareynii* and *M. lypriformis*, have been introduced into the U.S. and have given reasonably good results

Cultural – Healthy, competitive vegetation can protect against this invader

Mechanical – Tillage is effective prior to seed production. Hand-pulling is preferred for small infestations. Mowing is ineffective due to the prostrate growth habit

Chemical – Many herbicides are reported to be effective

Where found – The only known plant in the county was found in 2004 at a pull-out off of Hwy 395, just east of the Columbia River bridge outside of Kettle Falls. Report any sightings please!

Created by Stevens County Noxious Weed Control Board, Dec 2004

Purple loosestrife

Lythrum salicaria L. Loosestrife Family

Key identifying traits

- Has long, showy, rose-purple flower spikes
- Flowers are small, numerous & have 5-7 petals
- Usually associated with moist or marshy areas
- Leaves simple, entire, and opposite or whorled
- Forms substantial root wads with many stems
- Has erect stems, often growing 6 to 10 feet tall
- Stems are stiff and four-sided
- Develops root rhizomes and abundant seeds
- Both a woody tap root and fibrous root system

Biology and ecology

- An attractive rhizomatous perennial; reproduces by seed, roots and vegetative growth
- Forms up to 2.5 million, pepper-size seeds per plant; seeds float on water and stick to animals
- Commonly displaces desirable wildlife habitat
- Only bees make good use of infested habitat
- Dense infestations can impede water flow
- Infestations often arise from ornamental use
- Sometimes confused with fireweed and spiraea

browning plants due to biocontrol damage

Control

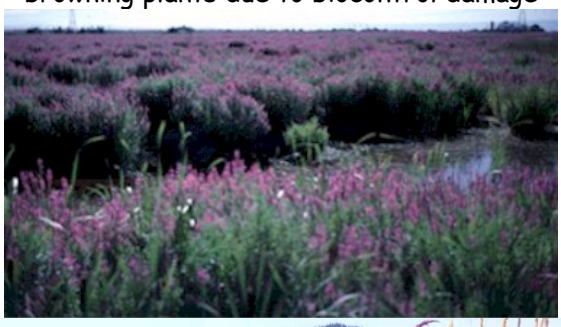
Prevention – Learn to identify plants; beware of ornamental varieties & contaminated boats/motors

Biological – Some very effective agents available and established in our county; substantial impact on plant vigor and reproduction (*Galerucella & Hylobius*)

Cultural – Competitive vegetation helps although plants can invade healthy native as well as managed introduced plant habitats

Mechanical – Small infestations can be dug, bagged and disposed of taking care not to disperse seeds

Chemical – Non-selective or broad leaf herbicides can be effectively used depending on site; if plants are in bloom, clip/bag flowers prior to spray; special care, permit and license required for aquatic applications



cutting flowering heads to bag prior to spray

Where found – Infestations at Waitts Lake, Loon Lake and Lake Spokane; a few other minor infestations have been located in pastures and along ditches elsewhere in Stevens County.

Created by Stevens County Noxious Weed Control Board, February 2000; Updated Jan 2004

Saltcedar

Tamarix ramosissima Ledeb. Tamarisk family

Key identifying traits

- Flowers are pale pink to white with 5 petals
- Many tiny flowers are in each finger-like cluster at branch tips
- Leaves look like cedar and are small and scale like on highly branched slender stems
- May have appearance of a shrub or a small tree
- Bark on saplings and stems is reddish-brown

a pretty specimen in a local yard

Biology and ecology

- Grows as deciduous or evergreen shrubs or small trees, 5 to 20 feet tall
- Both T. ramosissima and T. parviflora are used as ornamentals, but have escaped and naturalized along streams, canals, and reservoirs in the west
- Aggressive colonizer forming single species stands in riparian areas of arid landscapes
- Secretes salt inhibiting growth of other plants
- Extensive root system combined with a high evapotranspiration rate can cause lowered ground water levels and dry up springs and marshy areas

leaves are scales like on Cedar trees

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt and seed from outside your area—Be careful with introduced ornamentals
Biological – Research is currently underway and field releases in the U.S. are pending. Not appropriate for the small amount of this plant found here
Cultural – Competitive vegetative cover helps but cannot prevent spread in susceptible sites
Mechanical – Repeated digging can be effective
Chemical – Several herbicides are effective at label rates but care must be used around moist areas

flowering stem

Where found – Ornamental plantings, usually single plants, of both T. ramosissima and T. parviflora have been noted here and there throughout Stevens County in residential areas. No confirmed escapes and naturalization known at this time, but wet sites within our more arid areas are potential locations for infestation

Scotch broom

Cytisus scoparius L. Pea Family

Key identifying traits

- A woody shrub up to 10 feet tall
- Classic legume flowers are showy, yellow and abundant
- Leaves mostly three parted with small, rounded leaflets
- Forms pods that are flattened, brown or black, with white hair on the margins
- Erect branches are angled and dark green
- Winter stems are bare of leaves, but usually remain bright green

Biology and ecology

- An aggressive, deciduous, perennial shrub
- Often started as an ornamental
- A problem in pastures, forests, wasteland and roadsides
- Common pest in western WA and OR very limited infestations east of the Cascades
- Dry mature pods are often heard "popping" as they split and eject seeds several feet
- Seeds remain viable in the soil for many years

Control

Prevention – Learn to identify plants; know your property; beware of contaminated logging equipment especially from west of the Cascades
Biological – A few identified in western Washington
Cultural – Good ground cover and seeding of disturbed areas helps but doesn't block infestations
Mechanical – Digging is very effective on smaller patches if sufficient root is removed and diligent follow up is used to pull or dig seedlings
Chemical – Several effective at label rates; must exercise care near trees to avoid damage; timing varies greatly with type of chemical

Where found –In northern Stevens County a few small infestations are known on forested sites following logging; one grave site ornamental planting also in a forested setting.

Created by Stevens County Noxious Weed Control Board, February 2000; Updated Jan 2006

Scotch thistle

Onopordum acanthium L. Sunflower Family

Key identifying traits

- Very large thistle growing up to 8 feet tall
- Stems have broad spiny wings
- Leaves are spiny and covered with fine dense hair
- Hairy leaf surface causes a grayish appearance
- Flower heads numerous, 1 to 2 inches in diameter
- Flowers violet to reddish; bracts taper to a spine
- Upper leaves alternate and coarsely lobed
- Basal leaves up to 2 feet long and 1 foot wide

Biology and ecology

- Tap-rooted biennial that reproduces by seed
- Invades roadsides, range and disturbed areas
- Thrives in sunny sites – but can tolerate shade
- Up to 50,000 seeds per plant; seeds viable 6-15 years
- Repeated branching may result in plants 5' wide
- Dense stands can create barriers that restrict livestock
- Drought tolerant but can flourish along open streams

Control

Prevention- Learn to identify the plant; know your property; beware of fill dirt, hay, manure and seed from outside your area

Biological – No known biological controls

Cultural – Competitive vegetation helps reduce open spaces for invasion

Mechanical – Mowing can stop seed production in short term, but plants can regrow; digging cutting and cultivation are effective; monitor for new growth season long; seed bank will remain for years

Chemical – Several effective at label rates; best timing is in early spring when plants are in rosette stage and again in fall when more rosettes germinate

a yard stick on a Scotch thistle rosette

Where found- Scattered small infestations, particularly in old barnyards as well as some sites covering several acres over pasture/range ground in the northern half of the county.

Created by Stevens County Noxious Weed Control Board, March 2000; Updated Jan 2006

Tansy ragwort

Senecio Jacobaea L *Sunflower family*

Key identifying traits

- Small (< 1inch), golden, daisy-like flowers are arranged in clusters at ends of stems
- Single or multi-branched; 1-6' tall
- Leaves 2-8" long, alternate, deeply lobed
- Irregular, deeply cut leaf segments give "ragged" appearance
- Observed at a distance (mainly during flowering), plant structure resembles St. Johnswort

Biology and ecology

- Biennial to short-lived, tap-rooted perennial
- Reproduces by seed; flowers July to September
- High alkaloid content makes all plant parts toxic to cattle, horses and, to some extent, sheep
- Invades log landings, roads and other openings in forested areas; widespread in western WA

Control

Prevention – Learn to identify plants; know your property; beware of equipment/vehicles recently used in western WA or OR and not thoroughly cleaned

Biological – Three established agents are working well in western WA, OR and CA; not practical for the small amounts of plants found in eastern WA

Cultural – Competitive vegetation helps reduce open spaces for invasion

Mechanical – Digging or pulling when soil conditions are moist is effective; needs to be before bloom or, if after, bag or burn plants in a pile as appropriate; seed bank will be present for years

Chemical - Several effective at label rates; best timing is spring during rosette stage although fall rosettes can be treated as well; new infestations are usually identified when in flower, requiring a combination of mechanical and chemical control over more than one year

Where found - A few small, isolated infestations are found in Stevens County in disturbed openings in forest settings and along backcountry roads.

Created by Stevens County Noxious Weed Control Board, February 2000; Updated Dec 2005

Wild chervil

Anthriscus sylvestris L. Parsley Family

Key identifying traits

- White flowers arranged in umbrella like cluster
- Fernlike compound leaves have a sheathing base
- Hollow, furrowed stems with lower stem hairy
- Nodes have a fringe of longer hairs
- Each $\frac{1}{8}$ to $\frac{1}{4}$ inch white flower produces two seeds
- Two seeds are joined and have antennae like tips
- Seeds are dark, $\frac{1}{4}$ inch long, narrow, smooth, shiny
- Plant grows 1 to 4 feet tall and has a tap root
- Foliage is not aromatic

Biology and ecology

- Biennial or short lived perennial reproduces by seed
- Upright rosette first year; flowers the following May through June or July
- Found in moist pastures, forested areas and roadsides

Control

Prevention – Learn to identify the plant; be careful of British wildflower seed mixes formulated to recreate the flora of hedgerows and meadows of the British Isles.

Biological – None available at this time

Cultural – Competitive vegetation helps but can invade well managed areas

Mechanical – Cultivation generally kills plants although some can regenerate from the crowns; mowing is ineffective; digging is effective if enough of the tap root is removed

Chemical – Not much solid information available at this writing. We are experimenting with various herbicides applied in the fall and spring; because it likes moist areas, options are somewhat limited.

Where found – Limited acreage primarily in the northeastern portion of Stevens County, but known to be in neighboring Spokane Co and British Columbia

Yellow archangel

Lamiastrum galeobdolon Mint Family

Key identifying traits

- Flowers are bright yellow and bell-shaped
- Stems are purplish, hairy, and square-shaped
- Leaves are opposite, serrated & ovate-shaped with a silver or white pattern on top; the undersides are a lighter green or sometimes purplish color
- It is viny, grows prostrate & may have an unpleasant odor

Photo by Richard Old, www.xidservices.com

Biology and ecology

- A perennial plant spreading by seeds & above ground runners (stolons)
- It tolerates a wide variety of growing conditions, is frost tolerant and can grow in lightly shaded to well shaded areas
- It is a popular choice for ground cover & is also used in hanging baskets and flower beds
- This plant is native to temperate regions of Asia
- Some infestations are believed to have started by improperly disposed yard & basket waste

Photo courtesy WA State Noxious Weed Board

Control

Prevention – Learn to identify plants; start monitoring early in the season; dispose of clippings and old hanging baskets or tubs properly

Biological – No known biological control in our area

Cultural – Do not plant this as a ground cover or in sites where it can easily spread and escape

Mechanical – Plants grow back heavily if cut and although easy to pull any remaining root fragments will grow new plants

Chemical – There are no specific chemical recommendations yet in the PNW Weed Management handbook but this plant should be susceptible to dicamba, glyphosate and mixtures with triclopyr

Photo courtesy WA State Noxious Weed Board

Where found – In WA there are escaped areas in King, Kitsap, and San Juan Counties and a site in the Mt. Baker Snoqualmie National Forest; no known escaped sites in Stevens County

Black henbane

Hyoscyamus niger L. Nightshade family

Key identifying traits

- Large, coarse, abundant foliage has a fowl odor
- Soft, stalkless leaves are toothed to shallowly lobed and have sticky hairs
- Funnel-shaped fused petals are yellow-green with purple veins and center
- 1 inch long pineapple-shaped seed capsules are in two overlapping rows on one side of the stem
- Each capsule has hundreds of tiny black seeds
- Hardened, almost woody seed stalks remain standing throughout winter and following spring

rosette

flowering plant

Biology and ecology

- Can be an annual or biennial 1 to 4 feet tall with a fleshy tap root
- Reproduces by seeds
- All parts are highly poisonous but the foliage is unpalatable and seldom grazed
- Has historic medicinal use at controlled doses and has also been used as an ornamental
- Grows on roadsides, waste areas and in pastures

flowers

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area

Biological – None known at this time

Cultural – Good vegetative cover substantially reduces chance of infestation

Mechanical – Will not tolerate cultivation; pulling and cutting can be very effective if properly timed to avoid seed set; bag and dispose of seed capsules if mature

Chemical - Several effective at label rates; generally easier to control when young, before the stalk(s) elongate; refer to the PNW Weed Management Handbook for specific chemical recommendations

seed heads & new growth

Where found – Found in several small infestations in the Colville area of Stevens County. Is found throughout the US and is a common weed in many suitable habitats.

Hoary Alyssum

Berteroa incana L. Mustard Family

Key identifying traits

- Flowers are white with 4 petals, deeply divided
- Usually only branched at the top of the plant
- Leaves are alternate, blades are simple
- Fruit is oblong, with two compartments containing 2-6 seeds
- Seeds grow close to stem, are round to oblong, narrowly winged, grayish-brown and rough

Biology and ecology

- Can be an annual, winter annual, biennial, or a short-lived perennial (!); usually 1-2½' tall
- Most abundant in disturbed sites but is also found in meadows and pastures
- Starting to be an invader in alfalfa crops
- Can be toxic to horses if it comprises 30% or more of their diet
- Spreads rapidly due to the high number of seeds per plant; spreads only by seed
- Likes direct sunlight but can also tolerate shade

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed

Biological – No known biological control

Cultural – Good vegetative cover helps but does not prevent establishment or spread

Mechanical – Hand pulling or digging can be very effective for small infestations but should be done before there are seed pods. Mowing is not a good control option.

Chemical –PNW Weed Management Handbook does not have this weed listed for control recommendations, but other annual mustard recommendations may be useful. Telar and Escort are effective on most mustards. Spraying can be effective into early fall, as long as the plants are actively growing and have not dried out.

Hoary alyssum on street corner

Hoary alyssum in tree planting

Hoary alyssum on a mountain road

Where found – Largest infestations in the county are in Colville, Arden, Chewelah and Northport areas but Hoary alyssum is spreading rapidly.

Longspine sandbur

Cenchrus longispinus Grass Family

Key identifying traits

- Warm-season grass growing horizontally or upright to 3'
- 1-3" seed spikes have 10-30 pea size hard burs
- Burs are thickly set with stiff, sharp, spreading spines
- Leaf blades and leaf sheaths are flat in appearance; immature plants spread horizontally
- Frequently, old burs can be found clinging to a root of young plants which aids in distinguishing immature sandbur from other grasses

seedling with the bur attached to root-a good ID for young plants

Biology and ecology

- Summer annual that favors sandy or well-drained gravelly soils; thrives on sunny sites
- Invades lawns, roadsides, ornamental, disturbed areas
- Flowering and seed production July to September
- Mature burs cause injury to livestock mouths, noses and eyes and can reduce value of wool
- Commonly spread by animals, machinery and vehicles; burs cling to tires, clothing, soles and laces of shoes, knees, knuckles and pet hair

mature plant with ripe burs

Control

Prevention – Learn to identify the plant; know your property; beware of fill dirt, hay and seed from outside your area

Biological – None

Cultural – Competitive vegetation helps-keep lawns watered and fertilized to increase the vigor of turf and reduce open spaces

Mechanical – Mowing can stop seed production in short term, but plants can regrow and bloom shorter than the cutting blade; digging or pulling is effective; needs to be before bloom or, if after, bag or burn plants in a pile as appropriate; seed bank will be present for years

Chemical – Pre-emergence treatments are effective at label rates prior to germination but effectiveness can be lost due to leaching from rain or irrigation; post-emergence spot treatment with a non-selective glyphosate product is effective but care must be used to avoid loss of desirable plants. Broadleaf herbicides are not effective.

burs newly emerged

Where found – Limited infestations found in the sandy bench areas of Stevens County above Lake Roosevelt, particularly west of Kettle Falls.

Created by Stevens County Noxious Weed Control Board, April 2000; Updated Jan 2006

Butterfly bush

Buddleja davidii Loganiaceae Family

Key identifying traits

- Large, deciduous shrub, growing up to 10' tall
- Opposite, lance-shaped, 3-4" dark green leaves appear white or silvery underneath due to a covering of fine hairs
- Flowers are usually a shade of purple, but may be red, pink, blue, orange, yellow or white
- The fragrant, small, funnel-shaped flowers are borne in showy spikes at the ends of stems, looking and smelling very much like lilacs

"seedling" and backside of leaves

Biology and ecology

- Perennial shrub, reproducing mainly from seed, but will re-sprout from root stock when cut
- May produce up to 3 million seeds per plant
- This plant is a native to China and is a popular ornamental plant here
- Butterfly bush has escaped ornamental plantings and invaded roadsides, riparian areas & pastures

Control

Prevention – Learn to identify plants; do not allow ornamentals to escape; if you grow it in your yard, cut the spent flowers off and dispose to prevent spread of seed by birds, wind, water; choose alternate ornamental shrubs to plant

Biological – A weevil and a stem-boring beetle are currently being tested in New Zealand

Cultural – Healthy, competitive vegetation will help discourage domination, but will not stop it from getting started

Mechanical – Cutting will cause it to re-sprout; digging and pulling may cause more site disturbance, allowing more seeds to sprout

Chemical –Cut stump applications are effective as well as complete foliar coverage at the proper time

competing with invasive blackberries

Where found – No populations of escaped, untended bushes are known to be in Stevens County at this time but it could pop up almost anywhere

Myrtle spurge

Euphorbia myrsinites Spurge Family

Key identifying traits

- Fleshy, blue-green leaves form tight spirals around spreading, prostrate stems
- Bright yellow-green, showy bracts surround inconspicuous flowers in early spring
- Stems branch out from a central taproot, growing up to 18" long; plant height is only 4-6"
- White, milky sap (latex) is found when any part of the plant is broken

Biology and ecology

- A herbaceous perennial reproducing primarily and readily from seed; it has been noted that roots broken by cultivation can form new plants
- The latex is toxic and may cause severe dermatitis, burns and/or blisters in sensitive people
- This plant is a native to the Mediterranean region and was introduced to the US as an ornamental
- Prefers dry, well-drained soils in part to full sun; it is a popular rock garden ornamental
- Other names: donkey-tail and creeping spurge

Control

Prevention – Learn to identify plants; choose alternate plants for landscaping; do not allow ornamentals to escape

Biological – No known biological control

Cultural – Keeping desirable vegetation healthy and thick will help keep invaders out; don't let plants go to seed

Mechanical – Pulling/digging small patches can work; important to get the root; wear protective clothing

Chemical-Due to the waxy cuticle, must use an appropriate surfactant with the chosen herbicide; 2.4-D, dicamba and glyphosate should be effective

Where found – No "wild/escaped" sites are known to be in Stevens County at this time, but it has been seen in many ornamental landscapes.

Plumeless thistle

Carduus acanthoides L. Sunflower Family

Key identifying traits

- Abundant upper branching gives "top heavy" appearance to mature plants or old dead plants
- Spiny wings extend entire length of stems, right up to flower heads
- Flowers about 1 inch across and purple (generally)
- Bracts surrounding the flower are narrow, appearing as sharp spines
- Stem leaves are alternate and blend into stems
- Older rosettes have deeply lobed, wavy leaves with yellow spines along white leaf margins

Biology and ecology

- A winter annual or biennial with a tap root
- Reproduces by seed only
- Found in pastures, fields and roadsides
- Mature plants normally grow 3-6 feet tall but are known to grow as tall as 8 feet in our area
- Flowering occurs June to August
- Can dominate sites to exclusion of livestock
- Can hybridize with Musk thistle

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area

Biological – Some established and effective in the county reducing seed production and plant density

Cultural – Good vegetative cover tends to inhibit establishment but it can invade healthy sites

Mechanical – Regular cultivation, digging or cutting are effective because of seed only spread

Chemical –Several effective at label rates when applied at vegetative stage either in the first or second year of lifecycle (spring/fall)

Where found – Large infestations occur in valleys and along roads in the northern half of Stevens County with small outbreaks south of Highway 20.

Created by Stevens County Noxious Weed Control Board, February 2000; Updated Jan 2004

Rush skeletonweed

Chondrilla juncea L. Sunflower Family

Key identifying traits

- Early season rosette resembles dandelion leaves
- Inconspicuous stem leaves cause skeleton-like plant appearance and hence the common name
- Course hairs on lower 4 to 6 inches of stem
- Yellow, $\frac{3}{4}$ inch flowers are inconspicuous and widely scattered along length of stems
- Cut stems and leaves exude milky juice (sap)
- Flower petals number from 7-15 and are squared across the end with tiny lobes or teeth

Biology and ecology

- Perennial, multi-stemmed, 1 to 4 feet tall
- Reproduces by seed and by lateral root buds
- Parachute like seeds easily spread by wind
- Extensive deep roots make control difficult
- Inhabits well drained, light textured soils
- Found in roadsides, fields, range and pasture
- Soil disturbance aids establishment
- Flowers mid July through frost

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area

Biological – Gall midge, Gall mite and rust all well established in Stevens County and effective in limiting density and seed production

Cultural – Good vegetation inhibits establishment

Mechanical – Repeated effort required because of extensive roots and re-growth from roots and seeds

Chemical – Some are effective at label rates, particularly if applied to young growth and repeated on regrowth

Where found - Primarily in the southern 1/3 of Stevens County with the heaviest infestations along the HW 291 corridor from Spokane County to HW 231.

Created by Stevens County Noxious Weed Control Board, February 2000; Updated Jan 2006

Yellow starthistle

Centaurea solstitialis L. Sunflower Family

Key identifying traits

- Bright yellow flower head
- Sharp spines or thorns ¾"-1" long at base of flower heads
- Plant grows 2-3 feet tall, has grayish-green foliage and small "wings" on the stems
- Flower heads are borne singly on ends of branches
- Old plants or last year's skeletons have a cottony white tuft where flowers were
- Seedlings resemble a dandelion with deep lobed leaves

Biology and Ecology

- Annual – reproduces only by seed
- Invades rangelands, roadsides, and other disturbed areas
- Thrives in sunny sites – does not tolerate shade
- Toxic to horses in large amounts
- Flowers June through October
- Can have several flushes of plants per year

Control

Prevention - Learn to identify the plant; know your property; beware of fill dirt, hay and seed from outside your area; and confirm identification of any possible sightings

Biological – Over the years, several agents have been released in our county; no visible impact to date

Cultural – Competitive vegetation helps although plants can invade well managed areas

Mechanical – Killed by cultivation and tillage or by pulling; needs to be before bloom or, if after, bag or burn plants in a pile as appropriate; seed bank will be present for years

Chemical – Several effective at label rates when applied pre bud

Where found – Common in the Gold Hill area near Kettle Falls with smaller infestations found on open south and west slopes elsewhere in Stevens County.

Created by Stevens County Noxious Weed Control Board, February 2000; Updated Jan 2006

Diffuse & Spotted knapweed

Centaurea diffusa Lam. & Centaurea biebersteinii Lam. Sunflower Family

Key identifying traits

Diffuse & Spotted KW are hybridizing in our county; many plants have characteristics of both

- Many spreading branches with divided lower leaves; upper leaves are smaller and mostly undivided
- Diffuse kw flower bracts are tipped with definite slender spines
- Spotted kw flower bracts are dark tipped
- Bract margins divided like the teeth of a comb
- Leaves are grayish green, hairy and bitter
- Flowers are numerous; white to rose to purple

typical diffuse

typical spotted

Biology and ecology

- Annual, biennial(usually) or short lived perennials
- Tap rooted and growing 1 to 4 feet tall
- Infest roadsides, waste areas and rangelands
- Highly competitive and invasive on dry sites
- Flowering occurs from early July to September
- Old DK plants break off, tumble and scatter seed. SK flower heads open and disperse most seed within a 3 to 4 foot radius of the plant

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area

Biological – Several established and common in this area and having some effect on density and vigor when found working together on roots and seeds- especially *Larinus minutus*

Cultural – Competitive vegetation helps but can be displaced; fertilizer and irrigation generally favor grasses and burning reduces seed dispersal

Mechanical – Tillage, digging, pulling & cutting will reduce or eliminate plants if repeated frequently, thoroughly & prior to seed production

Chemical – Several effective at label rates with rosette and early bolt best stages for treatment

Where found – Widely distributed in Stevens County and surrounding areas

Russian knapweed

Acroptilon repens Sunflower Family

Key identifying traits

- Black to brown scaly roots produce stem buds
- Cone shaped heads with light pink to purple flowers
- Bracts surrounding each flower are paper like
- Forms dense colonies from root shoots
- Stems and leaves covered with fine hair, giving a blue or gray –green color
- Stems are erect, openly branched, 18" to 36"

Biology and ecology

- Tap-rooted perennial with limited seed production
- Spreads mainly by underground rootstocks
- Flowers June through September
- Invades cultivated fields, pastures and roadsides
- Roots penetrate to a depth of over 8 feet
- Ingestion of substantial amounts can be fatal to horses
- Grazed readily and without ill effects by other livestock

Control

Prevention- Learn to identify the plant; know your property; beware of fill dirt, hay and seed from outside your area

Biological – A gall forming nematode established in Washington may eventually assist control

Cultural – Competitive vegetation helps avoid initial infestation, but Russian knapweed can invade healthy plant communities

Mechanical – Mowing, pulling and cultivation are ineffective controls because of root spread

Chemical – Several effective at label rates, but timing is critical with bud stage and after a light frost being the two best times

Where found – Scattered sites along roadsides, pastures and cultivated fields in most regions of Stevens County.

Dalmatian toadflax

Linaria dalmatica Figwort family

Key identifying traits

- Waxy, blue green heart shaped leaves
- Dense alternate lower leaves clasp stem
- Yellow snapdragon type flowers have long spurs and an orange bearded throat
- Long slender flower/seed stalks develop throughout season with pea size 2-celled seed capsules
- Several stems from mature root crown

flowers and clasping leaves

Biology and ecology

- Perennial with extensive root system
- Spreads by both roots and seeds
- Grows to 3' tall, often in dense patches
- Flowers mid-summer until early fall
- Prefers dry sandy or gravely sites
- Aggressively invades roadsides and range

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area

Biological – A promising agent has been released county wide (2001-2002) with a few sites showing considerable damage. Potential for substantial impact to the toadflax population is strong

Cultural – Good ground cover helps avoid initial infestation; can invade healthy dry sites

Mechanical – Will not stand regular cultivation but occasional tillage or cutting and pulling are not sufficient due to extensive root system

Chemical – A few herbicides are effective if proper timing to promote translocation to the roots is combined with a good surfactant to penetrate the waxy leaves; refer to the PNW Weed Management Handbook for specific chemical recommendations

young plant

Where found – Widely distributed in Stevens County on well drained sites, particularly near some old cemeteries where it was introduced as an ornamental.

Yellow toadflax

Linaria vulgaris Mill. Figwort family

Key identifying traits

- Abundant yellow snapdragon type flowers
- Flowers 1" long with a bearded, orange throat
- Leaves are pale green, numerous, narrow, pointed at both ends and 2 $\frac{1}{2}$" long
- Fruit is round, $\frac{1}{4}$" in diameter and brown with 2 cells having many seeds
- Seeds are dark brown to black, 1/12" diameter, flattened with a papery wing

why it's sometimes called "butter & eggs"

Biology and ecology

- A perennial spreading from seeds and roots
- Tends to grow in patches as it spreads by roots
- Introduced and spread as an ornamental; also known as "butter and eggs"
- 1-2' tall; shorter than Dalmatian toadflax
- Extensive root system makes control difficult
- Aggressive invader of rangeland; able to displace desirable grasses
- Also found along roadsides, waste areas and fields

Yellow toadflax

Dalmatian toadflax

Control

Prevention – Learn to identify plants; know your property; beware of fill dirt, hay and seed from outside your area

Biological – Several agents have been found on yellow toadflax, but with no substantial impact yet

Cultural – Good vegetative cover helps but does not prevent establishment or spread

Mechanical – Usually will not stand regular cultivation – regular cutting and digging can weaken infestations but must be repeated to be effective

Chemical – Several effective but usually requires repeat treatment to kill extensive root systems; refer to the PNW Weed Management Handbook for specific chemical recommendations

another color variant

Where found – Found in scattered locations throughout Stevens County. Occasionally in fields but more often in waste areas and rangeland.

Attachment C
Recommended Chemical Controls

Chemical Recommendations for Noxious Weed Control

The following information is provided only as a general guide to weed control. Size of infestations, current or projected land use and site conditions will determine what herbicide or other control method will best suit your needs.

For individualized plans, please consult the Noxious Weed Control Board or WSU Cooperative Extension Agent in your area.

Always read and follow all label instructions on the products you select and use!

HERBICIDES

Most of the products listed below are . To purchase and apply restricted-use herbicides, a pesticide license is required. Contact the WA Dept. of Ag. or the WSU Cooperative Extension Office for information on how to obtain a license.

2,4-D: This product is sold under many names and formulations **(*Amine 4®, Hi-Dep®, Weedar 64®*** etc**).** 2,4-D is a selective, broadleaf herbicide, it will last 2-4 weeks in the soil after spraying, but nearly all of its effect is on the foliage.

Aminopyralid: On the market since **2006**. It is for use on broadleaf weeds, applied to foliage and moving down into the root system. This product can be sprayed up to the edge of water and can also be used on "seasonally dry" wetland. There are no grazing restrictions. The only brand name of this product is ***Milestone®.*** One drawback is that it is not very effective for the control of mustards or toadflax. This product has a very low use rate at a maximum rate of 7oz/acre.

Aminopyralid + 2,4-D: **New in 2008 Forefront R&P®** A mixture is for use in range and grass pastures, CRP lands, and permanent grass pastures. It controls a wide spectrum of broadleaf weeds and the 2,4-D aids in the burn down of plants.

Aminopyralid + Metsulfuron: **New in 2009 Chaparral®** This mixture is for use in range and pastures, CRP lands, and permanent grass pastures. The metsulfuron will help in the control of mustards and members of the borage family (tarweeds, etc.).

Dicamba: This is the active ingredient in ***Banvel®.*** It is also a selective, broadleaf herbicide and can last 2-4 months in the soil. Most of its activity is through the foliage but will provide some residual control of new seedlings. Usually provides quick burn down of plants.

Dicamba + 2,4-D: This is a combination often recommended to landowners with noxious weeds in a pasture or lawn setting. Some of the trade names include ***Weedmaster®, Range Star®, and Outlaw®.*** It controls a wide variety of broadleaf weeds.

Clopyralid: A broadleaf herbicide for use in pastures and non-crop areas. ***Transline®*** is recommended when weeds are growing among conifers.

Clopyralid + 2,4-D: A mix for pasture and rangeland, ***Curtail®*** is very good for use on thistles and other broadleaf weeds.

Clopyralid + Triclopyr: ***Redeem R&P®*** Another herbicide for pastures, rangeland, and non-crop areas.

Chlorsulfuron: A selective broadleaf herbicide for use in non-crop settings. ***Telar®*** is a trade name for this chemical. Now available in a smaller amount.

Metsulfuron: Another broadleaf herbicide used for weed and brush control in pastures, rangeland, and non-crop areas, the trade name is ***Escort®.***

Metsulfuron + 2,4-D, + Dicamba: Trade name is ***Cimarron Max®*** or **an** alternative is to buy the Dicamba + 2,4-D mix *(Weedmaster®, Outlaw®,* etc.) and add metsulfuron *(Escort®*). This is an excellent mix for a wide variety of weeds and brush. For use in pasture, rangeland, and CRP areas. In adverse conditions it can yellow or temporarily stunt grasses.

Picloram: ***Tordon®*** is the trade name for this herbicide and it is a long lasting herbicide for broadleaf plants. A license is required to purchase this product in any amount. It is a selective herbicide that will remain in the soil controlling broadleaf plants an average of 2-3 years depending on soil type. Picloram has both foliar and root activity.

Glyphosate: Most commonly known as ***Roundup®*****,** this active ingredient can be found in many herbicides. Glyphosate only works on foliage and has ***no*** soil activity. Particular caution should be used when applying glyphosate as it is non-selective (works on grasses and broadleaf plants); leaving bare ground for new weeds to get established unless reseeding is done.

*These are just a few of the many herbicides on the market today. The Pacific Northwest Weed Management Handbook has a much larger list of herbicides and their recommendations. The book can be purchased through the WSU Extension or accessed on line at http://weeds.ippc.orst.edu/pnw/weeds or find herbicides and label information at cdms.net

Backpack Calibrations
(per 1 gallon of water)

2,4-D	2 oz
Hi-Dep	2 oz
WeedMaster	1 oz
Roundup	2 oz
Curtail	3 oz
Redeem	1/2 oz
Tordon	2 oz

3-way mix per acre rate:

½ ounce Metsulfuron (Escort)

8 ounces Dicamba (Banvel)

1 quart 2,4-D

Surfactants

Surfactants are an important key to maximizing an herbicide's effectiveness.

Surfactants are formulated to improve the performance of herbicides in spray solutions. They do this by modifying one or more of the following:

1. Mixing (emulsifying and dispersing oil and water-soluble particles)
2. Coverage (spreading and wetting leaves)
3. Spray retention (sticking to the leaves)
4. Absorption (penetrating properties)

There are 5 classes of surfactants: nonionic, crop oil concentrates, nitrogen-surfactant blends, esterified seed oils, and organo-silicone surfactants.

Not all herbicides behave the same on or in the plant. Most labels will recommend the type of surfactant that is best for that particular herbicide. Also there must be consideration of plant characteristics of whatever plant you are spraying, such as waxy leaves, plant hairs, lack of leaves, and leaf arrangement.

Environmental conditions must also be a consideration. When there is high humidity and moisture, plants tend to have thin relatively permeable cuticles making it easier for the herbicide to penetrate the plants. When there is lower humidity the plant cuticles are thicker and harder to penetrate. During low humidity periods crop oil concentrates and esterified seed oils can pass through the cuticles better than nonionic surfactants.

If the label calls for 1 quart of surfactant per 100 gallons; the equation would be
32(converting quart to ounces) divided by 100.
Answer = .32 oz per gal.
So multiply .32 X number of gallons of water you are using.

If the label calls for a .5% solution;
Multiply the gallons of water you are using by .005 (converting .5% into a decimal)
Using 3 gallons as an example
3 × .005 = .015 gal
Then multiply by 128 to convert to ounces (128 oz in a gallon)
.015 × 128 = 1.9 oz for the 3 gallons

HERBICIDE MIXING ORDER

Unless the label states otherwise, add the herbicide to the water or fertilizer in the following order:

1. wettable powders or dispersible granules
2. flowable or aqueous liquids (solutions)
3. emulsifiable concentrates
4. crop oil concentrates

Spray tanks should be at least half filled with the carrier (water or fertilizer) before the herbicides are added. If the mixture foams excessively, separates, or becomes syrupy, do not apply. Tank mixes usually require constant agitation and should be applied promptly.

WEED	GROWTH CYCLE	EARLY SPRING When plants start actively growing	MID-SUMMER Bud to bloom	FALL Rosettes/ Perennials
Bull, Musk, Plumeless, and Scotch thistles	Biennial	2,4-D® Curtail® Redeem R&P® Tordon® Weedmaster® Milestone®	Same herbicides: Apply before plants flower	Same herbicides for fall rosette treatment
Canada thistle Dalmatian or yellow toadflax Russian knapweed	Perennial	Herbicide treatment at this time is not as effective	Curtail® Redeem R&P® Tordon® Weedmaster® Milestone® Telar®	Same herbicides: Apply after first frost
Fiddleneck tarweeds, Bugloss & other borage family	Annual, biennial & perennial	3-way mix* Telar® Redeem R&P® Escort + 2,4-D	Effective but should be done before plants flower	Same herbicides: Apply to perennials & rosettes
Hawkweeds	Perennial	Herbicide treatment at this time is not as effective	Banvel® Curtail® Redeem R&P® Milestone® Weedmaster®	Plants are hard to find but it may be effective
Knapweeds including yellow starthistle	Biennial & Perennial	2,4-D Milestone® Curtail® Redeem R&P® Tordon® Roundup®	Same herbicides: Apply before flowering	Same herbicides for fall rosette treatment
Kochia-	Annual- Usually starts germinating later than most annuals, but seeds keep germinating into the fall season. Should be treated as soon as it is growing.		Weedmaster® Vista® Roundup®-The smaller the plants the more effective herbicide will be. Big plants may develop resistance.	
Mustards	Annuals & Biennials	Escort or 3-way* Weedmaster® Telar®	Same herbicides: Apply before flowering	Same herb. for rosette treatment
Rush skeletonweed	Perennial	If spot spraying plants can be difficult to see early in the year	Tordon®+ 2,4-D Milestone® 3-way mix*	Same herbicides for fall rosette treatment
St Johnswort	Perennial	3-way mix* Weedmaster® Tordon®+ 2,4-D	3-way mix* Weedmaster® Tordon®+ 2,4-D	Not as effective in the fall

Reminder: 3-way mix is Metsulfuron, Dicamba, & 2,4-D aka Cimarron Max®

Attachment D
Biocontrol Information Sheets

Aphthona flava

Flea beetle

Weed(s) Attacked: Leafy Spurge

SCNWCB February 2006

GENEALOGY

Original sources for U.S. release was Italy and Hungary. First U.S. releases made in 1985. Now established in numerous states, especially Montana.

LIFE CYCLE

Eggs are deposited in June through early fall on the plant stem at or just under the soil surface. Larvae hatch and actively feed on roots from July through the winter, and into the following spring. Larvae are so small that a microscope is generally needed to see them. Depending on the climate of a particular site, adults may emerge from June through early fall. Adults are about 4mm long, and will jump when disturbed.

EFFECT

Both adult and larvae damage the plant. Adults feed on leaves and flowers. Larvae feed in and on roots hairs and young roots.

REDISTRIBUTION

This agent is not yet established in Stevens County.

COMMENTS

Aphthona flava was the first flea beetle to be released against Leafy Spurge in the United States. Around Bozeman Montana it's effect has been spectacular. There are numerous species of the Aphthona flea beetle. All have similarities in life cycle and habit, but some are better adapted for certain soil type, moisture regime, plant community, and exposure.

The most heavily infested Leafy Spurge areas in Stevens County are found on rugged, dry, hot, sandy sites in southern Stevens County. Efforts to establish this agent there started in 2001. Each year large numbers (a mix of species) have been released, but have thus far failed to establish in Stevens County. Efforts will continue.

Bangasternus orientalis

Yellow Starthistle Bud Weevil

Weed(s) Attacked: Yellow Starthistle

SCNWCB February 2006

GENEALOGY

Original source for U.S. release was northern Greece. First U.S. releases were made in 1985. Now established in California, Idaho, Oregon, and Washington. During 1990 at least 4,600 adult Bangasternus were released in Stevens County. How well the agent has established is not known.

LIFE CYCLE

Overwintered females lay as many as 470 eggs in late spring and early summer. Eggs are laid singly at the tips of unopened flower heads. Hatched larvae tunnel into the flower head and begin feeding on immature seeds and receptacle tissue. Larvae pupate into adults within a case constructed of chewed seeds. Adults about 6 mm long emerge from their individual seed head case in late summer and overwinter outside the plant.

EFFECT

Yellow Starthistle reproduces by seed alone. It is known that a single Bangasternus larva can destroy 60% of the seeds in a seed head.

REDISTRIBUTION

Shake adults off plant into a funnel assembly or pan in late spring and early summer. If possible, release 500 at new sites.

COMMENTS

This agent (along with Eustenopus villosus) has greatly reduced the Yellow Starthistle population at a site near Colfax WA over the past decade. In Stevens County Eustenopus villosus is more established and visible, and is more the focus of BioControl for Yellow Starthistle.

Cyphocleonus achates

Knapweed Root Weevil

Weed(s) Attacked: Diffuse and Spotted Knapweed

SCNWCB February 2006

GENEALOGY

Original sources for U.S. release was Austria, Greece, Hungary, and Romania. First U.S. releases made in 1988. Now established in Colorado, Montana, Oregon, and Washington. There are populations established in Stevens County.

LIFE CYCLE

In Stevens county, overwintered adults emerge from the knapweed roots mid to late July and into September. Mature adults are up to 20 mm long. They feed on younger leaves if available, leaves, and tissue from the stems. Females place a single egg in a chewed out hole on the root crown just below soil level. She may produce more than 100 eggs in her lifetime. Adults die before winter. When the eggs hatch the larvae begin tunneling into the knapweed roots. The larva overwinters in the root.

EFFECT

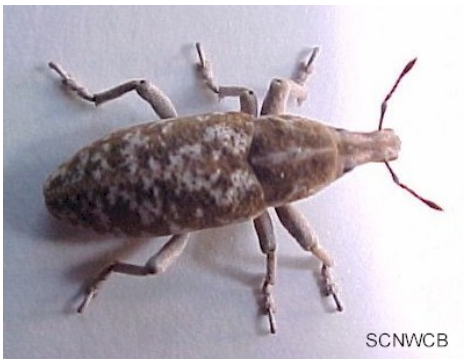
The larval root tunneling severely damages the weed. Adults eat above ground plant parts.

REDISTRIBUTION

This agent does not fly and spreads slowly if not manually redistributed. Not easy to spot in the field due to it's mottled color and habit of sitting perfectly still. When disturbed they often drop to the ground. Best chance to see and collect them is in the heat of the day. To collect, quickly bend the entire plant over into a collection net or pan. It is harder to hand pick them. They have a powerful grip but do not bite. For making a new release 50 adults is considered a workable number. A new colony stands the best chance of survival and establishment at new sites which are dominated by Diffuse Knapweed, spaced enough (1-2 feet) to allow the soil to become hot and dry. South facing slopes and/or large open fields of knapweed are best. A site free of disturbance (including development, traffic, herbicides, and regular grazing) increases the chances of establishment.

COMMENTS

The year 2000 marked the beginning of an effort to distribute this agent in Stevens County. Although Cyphocleonus achates is an excellent agent once established, it's inability to fly to new weed patches is a drawback. Therefore, the seed head weevil Larinus minutus (a strong flyer) has proven to be a more valuable natural enemy against Diffuse and Spotted Knapweed.



Cystiphora schmidti

Skeletonweed Gall Midge

Weed(s) Attacked: Rush Skeletonweed

SCNWCB February 2006

GENEALOGY

Original sources for U.S. release was Greece. First U.S. releases made in 1975. Established in the Pacific Northwest, including Idaho, Oregon, and Washington. Established in Stevens County.

LIFE CYCLE

There are as many as five generations per year. Eggs are injected into leaves and stems. . Larva hatch and begin to feed on the plant tissue around them. This causes the plant to form swollen purplish galls which encase the larva. Most larva pupate inside the gall. Some mature larva leave the gall, fall to the ground litter, and pupate. Both Larva and pupa stages overwinter, most inside the gall, some in the ground litter. The generation that winter catches finish their development into adults the following spring.

EFFECT

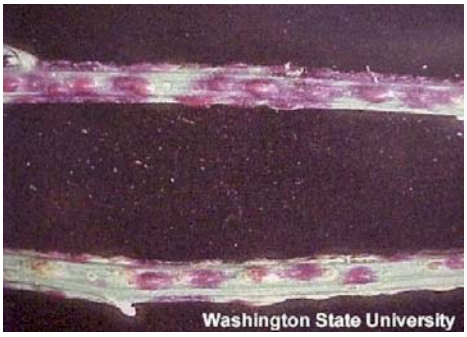
The larva is the destructive stage.

REDISTRIBUTION

Collect infected plants from July through September. Bind the galled plants together into "Teepees". Wedge these bundles into plants that are not infected. Some of the immature midges will complete their development within the collected galls, emerge, and attack the new plant. Cooler weather, evening releases, and high humidity helps the chances for establishment.

COMMENTS

The midge has become prey of native predators and parasitoids. The usefulness of this agent is therefore diminishing. It is still an important natural enemy of Rush Skeletonweed however.



Eriophyes chondrillae

Skeletonweed Gall Mite

Weed(s) Attacked: Rush Skeletonweed

SCNWCB February 2006

GENEALOGY

Original source for U.S. release was Italy. First U.S. releases made in 1977. Established in the Pacific Northwest, including Idaho, Oregon, and Washington. Established in Stevens County.

LIFE CYCLE

Eggs are microscopic (0.04mm). Adults usually cannot be seen with the naked eye. Each of the tiny females may deposit 60-100 eggs within the gall they share. After hatching hundreds of the minute worm like nymphs feed on the green gall material around them. The nymphs pass through several developmental stages to become adults. Whenever the gall dries out there is a mass exodus of all mobile stages.

In the Pacific Northwest the female mites form a dark brown overwintering stage called a deutogyne. The deutogynes move down the stem in fall to crevices in the plant at or below the soil surface where they become quiescent until spring.

When the weed starts to bolt in the spring the mites invade the shoot buds. Adults live up to 4 weeks. The mites feed and continue to reproduce on a roughly 10 day cycle until fall.

EFFECT

Mite feeding causes the buds to contort into galls which look like miniature cauliflower. Both nymphal and adult stages damage the plant. Mite feeding decreases plant vigor, helps reduce or eliminate seed production, hinders formation of rosettes, reduces shoot production, and sometimes results in the death of the plant.

REDISTRIBUTION

Collect infected plants from July to mid October. Bind the galled plants together into "Teepees". Wedge these bundles into plants that are not infected. Cooler weather, evening releases, and high humidity helps the chances for establishment.

COMMENTS

This mite disperses well within the weed patch, and is known to colonize any biotype of Rush Skeletonweed found in our area. It is considered the most effective agent released on Rush Skeletonweed to date in our area. The agent is well established in Stevens County.

Eustenopus villosus

Yellow Starthistle Hairy Weevil

Weed(s) Attacked: Yellow Starthistle

SCNWCB February 2006

GENEALOGY

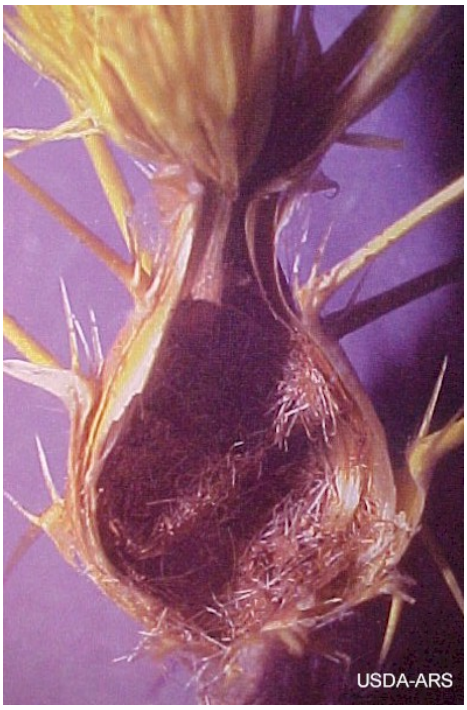
Original source for U.S. release was northern Greece. First U.S. releases were made in 1990. Now established in California, Idaho, Oregon, and Washington. First known release in Stevens County was made in 1996 on Gold Hill above Kettle Falls.

LIFE CYCLE

Adults which have overwintered outside the host plant began egg laying activity for the new year late in spring and early summer. These overwintered adults feed on and destroy many of the smaller developing flower heads. Females chew holes in the larger unopened flower heads and lay their eggs singly. They then cover the hole with chewed material. In about 3 days the eggs hatch and the larvae begin feeding on the developing seeds and receptacle tissue. After about two weeks the larvae fashion a pupal chamber from chewed seed head content. In another two weeks adults emerge and probably feed on late developing, small flower heads before overwintering begins.

EFFECT

Both the adult and larval stage are destructive. Feeding activity of the adults destroy a high percentage of the smaller unopened flower heads. Larvae feeding inside the larger flower heads often destroy 100% of the developing seeds.



REDISTRIBUTION

In Stevens County July is a good time to collect these agents. Easiest collection method is to shake the agents off into a collection pan. If possible release 250-500 at a new site.

COMMENTS

This agent has excellent potential. It has proven effective at sites in Washington and Idaho. In Stevens County the agent is well established on Gold Hill above Kettle Falls and is present at other sites as well. Starting in 2006 this agent will be the focus of July collections and redistribution.

Galerucella pusilla

Golden Loosestrife Beetle

Weed(s) Attacked: Purple Loosestrife

SCNWCB February 2006

GENEALOGY

Original source for U.S. release was northern Germany. First U.S. releases were made in 1992. Now established in Idaho, Maryland, Minnesota, Montana, New York, Oregon, Pennsylvania, Utah, Virginia, and Washington. This agent is well established at sites in Stevens county.

LIFE CYCLE

Overwintered adults emerge from the soil and soil litter, feed on buds and leaves, mate, and begin to lay eggs in May and June. A distinctive line of frass is often placed on top of the egg. When the eggs hatch larvae feed on leaves and other plant parts before moving down into the soil or litter to pupate into adults. These new adults emerge in July and August and continue feeding before hibernating for the winter. If the female emerges before mid-July she often lays eggs before going to ground for the winter. Presumably, some of these eggs hatch, and if the larvae find enough remaining food to develop, will also descend to the ground to pupate into adults and overwinter.

EFFECT

Both the adult and larvae are destructive to Purple Loosestrife. Plants that are attacked are severely defoliated. They turn brown and are easy to spot.

REDISTRIBUTION

A good time to collect these agents is from mid May to mid June. Collect these agents by shaking the plant inside a sweep net. This agent and it's close relative Galerucella calmariensis readily establish at new sites which are not flooded year-round. If possible release 250-500 at a new site.

COMMENTS

About 70-80% of the Galerucella releases in the U.S. are Galerucella pusilla. The remainder are Galerucella calmariensis. They are very similar. The photo of the eggs and larva are Galerucella calmariensis, but Galerucella pusilla eggs and larvae look the same. These agents are a success story almost everywhere they are found. Within a few years after being released at Purple Loosestrife sites in Stevens county that weed has been heavily impacted. Occasional releases are still being made in Stevens County, but the agent seems to be doing well on it's own.

Agriculture and Agri-Food Canada

Agriculture and Agri-Food Canada

Agriculture and Agri-Food Canada

NY Cooperative Fish & Wildlife Research Unit

Larinus minutus

Knapweed Flower/Seedhead Weevil

Weed(s) Attacked: Diffuse and Spotted Knapweed

SCNWCB February 2006

GENEALOGY

Original source for U.S. release was Greece. First U.S. releases were made in 1991. Now established in Montana, Oregon, and Washington. In Stevens County, starting in 2000, this has been the most heavily collected and redistributed agent of all time. It is now well established throughout the county, and needs little assistance.

LIFE CYCLE

Overwintered adults emerge from the ground litter in late May or June and begin feeding. Females lay clusters of eggs in open flowers. When the eggs hatch the larvae start feeding on flower parts and immature seeds. The larvae are aggressive and kill one another and other species within the seed head. The surviving larvae feed and go through numerous changes. Pupation into an adult takes place inside a cocoon made of chewed seed and flower parts and is attached to the flower receptacle. The new adults emerge from their cocoon in July and August. These adults feed before going into the ground litter to hibernate.

EFFECT

Both adult and larvae are destructive to Diffuse and Spotted Knapweed. Adults feed on young leaves in spring, and leaves and flowers later on. A larva often destroys all the seeds in it's seedhead.

REDISTRIBUTION

June and early July are the best times to collect this agent. The females have more egg laying ability at these times. A good collection method is to bend the entire plant over into a sweep net or container and shake the agents off. Place bugs in a paper sack with a little food (knapweed) and some water (a clean moistened sponge) for the trip to the new release site. Keep the bugs cool during their captivity. New release sites should closely resemble the environment from which the insects were collected. Generally this will be open, sunny knapweed patches, where plants are spaced enough for the ground between to be hot and dry. Release about 250-500 adults for each new site.

COMMENTS

The year 2000 marked the beginning of an effort to widely establish this agent throughout Stevens County. Over the years they have proven to be the best available BioAgent for Diffuse and Spotted Knapweed in our area. The insects are probably now established everywhere they are going to establish in Stevens County. Their impact is visible throughout numerous areas. They need little further assistance. As their populations increase over the coming years their impact should become significant throughout the county.

Larinus minutus adult

Larinus minutus adults

Larinus minutus on knapweed

Mecinus janthinus

Stem-boring Weevil

Weed(s) Attacked: Dalmatian Toadflax

SCNWCB February 2006

GENEALOGY

Original source for Canadian release was Yugoslavia and Italy. The agent is well established in British Columbia. Mecinus janthinus was approved for release in the U.S. in 1995. It migrated from southern British Columbia into northern Stevens County and was first documented in 2000. The first known U.S. collections and releases took place in Stevens County WA. Field Insectaries are being established throughout the county.

LIFE CYCLE

Starting in June females lay a single egg per chewed out cavity in the toadflax stems and seal the hole with a cap of chewed material. When the eggs hatch the larvae begins tunneling within the stem, increasing the size and length of the tunnel as they grow. After about a month the larvae stop tunneling and over several more weeks pupate into a new adult. These new adults remain inside the stem during the winter. Snow cover helps the dormant adults inside the stems survive the freezing temperatures of winter. Starting in May these new adults chew their way out of the stem, feed, and in June start laying eggs.

EFFECT

Adult feeding does substantial damage to the leaves. An attacked plant exhibits leaves with "buckshot" holes. The larval stem tunneling disrupts the food transport system between the roots and the leaves. At the initial collection sites in northern Stevens County the impact of this agent has been remarkable.

REDISTRIBUTION

This agent can be collected in late May and June. It is a focus of collections and redistribution in Stevens County. The population of this agent is on the increase in the county.

COMMENTS

At the initial collection sites in northern Stevens County this agent has proven highly destructive to Dalmatian Toadflax. At numerous new sites the population is increasing and damage is evident. A county wide grid of release sites has been selected and will receive the bulk of the insects collected over the coming years. It is expected that the populations of the insect will increase at these undisturbed "Field Insectaries" and move out into all areas of the county where Dalmatian Toadflax is found, which is almost everywhere.

Mecinus janthinus adult

Mecinus janthinus breeding

Mecinus janthinus damage

Metzneria paucipunctella

Knapweed Seed Head Moth

Weed(s) Attacked: Diffuse Knapweed, and Spotted Knapweed

SCNWCB February 2006

GENEALOGY

Original source for U.S. release was Switzerland. First U.S. releases were made in 1980. Now established in Idaho, Montana, Oregon, and Washington. In Stevens county 10,600 adults were released between 1986 and 1990. This agent is established in Stevens county, but is not very important.

LIFE CYCLE

Larvae which have overwintered inside the seed head pupate into adults in May. Adults are small moths about 7 mm long. They only fly at dusk and are rarely seen. Starting in June females lay between 60 and 100 eggs on the flower head. Eggs are reddish-brown, oval, less than 1mm long, and turn yellowish as they mature. Hatched larvae crawl into the opening flower head and begin to consume the contents. Larvae are aggressive and kill each another, and other larva in the seedhead. Generally, only one larva survives per seed head to start overwintering. Good snow cover increases the chances of larval survival for the winter.

EFFECT

In a single seedhead a young larva will consume a few seeds and reduce the viability of surviving seeds as it mines the seedhead receptacle. The older larva will web seeds together, preventing dispersal.

REDISTRIBUTION

Adult moths in the field are seldom seen or collected. The better method is to collect seed heads in early spring from sites where you have confirmed the presence of pupa and/or larva.

Metzneria paucipunctella pupa (left) and larva (right)

COMMENTS

Although Metzneria larvae kill one another, and other larval species in the seed head, it is thought the best possible impact occurs when Metzneria shares the knapweed patch with two gall fly species: Urophora affinis and Urophora quadrifasciata. Adult Urophora gall flies are everywhere in Stevens county. The distribution and density of Metzneria in the county is unclear. It is likely they are being supplanted by the much more important agent Larinus minutus. If other knapweed agents are absent, the impact of Metzneria and the two gall flies are minimal.

Puccinia chondrillina

Rush Skeletonweed Rust Fungus

Weed(s) Attacked: Rush Skeletonweed

SCNWCB February 2006

GENEALOGY

Original sources for U.S. release was Italy. First U.S. releases made in 1977. Established in the Pacific Northwest, including Idaho, Oregon, and Washington. Established in Stevens County.

LIFE CYCLE

The fungus forms infective spores within brownish pustules (uredia) on all above ground parts of the plant. These infective spores are released from the pustules from spring to fall to continue infection. Lesions (telia) at the base of flowering shoots are another source of rust spores. The overwintering stage is composed of dormant spores. In the spring the overwintered spores germinate on rosette leaves, which ultimately leads to the production of rust pustules (uredia) on all plant parts, and lesions (telia) at the base of the flowering shoots.

EFFECT

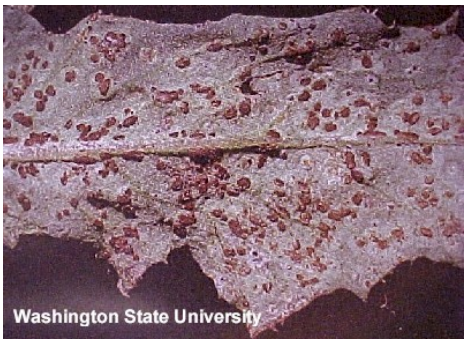
The pustules (uredia) and the lesions (telia) are destructive. Infected seedlings may be killed outright. Maturing plants have reduced vigor, reduced photosynthetic surfaces, reduced root vigor, and their ability to form viable seeds is hampered. Effects of the rust on Rush Skeletonweed is diminished at hot and dry sites.

REDISTRIBUTION

Infected rosettes can be collected and placed among uninfected plants during the spring and fall. During the summer collect rusted plants and bind them into "Teepees". Wedge these bundles into plants that are not infected. Cooler weather, evening releases, and high humidity helps the chances for establishment.

COMMENTS

This was the first plant pathogen released for the control of a noxious weed in North America.



Rhinocyllus conicus

Thistle Head Weevil

Weed(s) Attacked: Musk Thistle, Plumeless Thistle, Canada Thistle

SCNWCB February 2006

GENEALOGY

Original source for U.S. release was numerous sites in Europe. First U.S. releases made in 1969. Now well established in the Northwest, northern plains states, and Stevens county north of Hwy 20 (where most of our Plumeless, Musk, and Scotch Thistle are).

LIFE CYCLE

Starting in May overwintered adults mate, lay eggs, and then die. A single female may lay close to 200 eggs. Eggs are laid on the flower bud bracts and stem, and covered with chewed plant material. This covering turns tan and looks like warts. When the eggs hatch the larvae tunnel into the flower bud and feed in the receptacle and on developing seeds. Larvae in the seed head fashion individual hard protective pupal chambers from chewed plant material and feces, larvae in the stems do not. Development from egg to emerged adult takes anywhere from 40 to 60 days. Adults averaging about 6mm long, emerge from inside the seed head by chewing a hole in their pupa chamber, and then the receptacle face. These new adults look like they are covered in pollen, but this soon wears off and they are almost black to the eye. When they are not feeding, the new adults hide. These new adults do not lay eggs until the following spring. Adults overwinter in sheltered places.

EFFECT

Adults may slightly defoliate the Thistle. The larval stage is more destructive. This is an effective agent against Musk Thistle. It is much less effective against Plumeless and Canada Thistle, even though there may be many larva in a particular seed head.

REDISTRIBUTION

In Stevens County collect the egg laying adults in May and June. Shake the agents off the plant into a sweep net or deep sided plastic dish pan. A funnel assembly can also be used. Overwintered adults need to be released as soon as possible. Some new adults can be collected in the summer and fall if you can find them when they come out of hiding to feed. Release 250-500 per new site. This agent requires sites with less heat and some moisture. Hot dry places are not good.

COMMENTS

This agent is present at most thistle sites in Stevens county. Redistribution will be undertaken only where the need is clear. In Stevens County the agents impact on Plumeless Thistle is helpful, but is not providing control.

Sphenoptera jugoslavica

Bronze Knapweed Root-borer

Weed(s) Attacked: Diffuse Knapweed (preferred), and Spotted Knapweed

SCNWCB February 2006

GENEALOGY

Original source for U.S. release was Greece. First U.S. releases were made in 1980. Now established in California, Idaho, Montana, Oregon, and Washington. First documented release in Stevens County was in 1986.

LIFE CYCLE

The larvae overwinter in the root. The whitish larva are distinctive in that their head is noticeably wider than the rest of their body. Pupation into an adult occurs inside the root in late May and June. The peak adult emergence coincides with flowering, usually in July. Adults are somewhat flat, metallic bronze in color, and about 10 mm long. Females need 5 hot days (86 degrees Fahrenheit or more) before they lay eggs. The flat white eggs are placed between tightly packed rosette leaves in July and August. When the eggs hatch the larvae tunnel into the plant and down into the roots. Usually only one larva develops per root. The larval activity causes a rather large swollen gall on the root. Dry soil conditions favor larval survival.

EFFECT

Larva damage the plant by tunneling within the roots. Surviving plants are stunted and produce fewer flowers. Adults do a lesser degree of damage by feeding on leaves.

REDISTRIBUTION

Use a sweep net or deep sided plastic dish pan to collect adults in July-August an hour or so before dusk.

Comments

The population of this agent remains low in Stevens County. The reason may be predation of the larva. Outside of Stevens county there are sites where the collapse of knapweed population is attributed to this agent. Sites that are dry, and have some bare ground between plants spaced one to two feet apart are said to favor establishment. Though an excellent agent where established this agent appears to play a minor role in Stevens County.

Attachment D-13

Cover Deformation Analysis

ATTACHMENT D-13

MIDNITE MINE REMEDIAL ACTION COVER DEFORMATION ANALYSIS

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*9-Jul-2014*	*90% Design*	*Christine Weber and Brad Sick*	*P. Boodagh and Melanie Davis*	*18-Jul-2014*	*Tom Kelley*
1	*3-Jun-2014*	*100% Design*	*Christine Weber, Brad Sick, and Stephen Goodwin*	*Melanie Davis*	*11-Jun-2015*	*Tom Kelley*

Location and Format

Electronic copies of these calculations are located in the project files system at:

\\usden1s01\projects\NRII\Clients_I-P\Newmont\1011322 Midnite Mine\Technical\Calculations\Cover Settlement Analysis

\\usftc2s01\Projects\Newmont\Midnite Mine_2011\6.0 Studies & Reports\6.2 Technical\Remedial Action Cover Design\Cover Deformation

The following calculations were generated using the following software:

PLAXIS 2D
Microsoft Office Professional Plus 2010 (Microsoft Excel)

1.0 BACKGROUND AND PURPOSE

Stress-deformation analyses were performed to evaluate the potential cover settlement of the mine waste that will be placed in Pit 3 and Pit 4 as part of the Remedial Action (RA) at the Midnite Mine Superfund Site (the Site). The purpose of the analyses is to estimate the amount of settlement that the cover could potentially experience during construction and post-construction creep settlement of the mine waste in Pit 3 and Pit 4 (i.e., over an extended period of time). The horizontal (lateral) displacements of the pit dewatering sump risers, as well as the lateral displacements and strains in of the top cover geomembrane were also estimated.

Based on the calculated potential post-construction creep settlement, additional analyses were performed to evaluate changes in slope and potential drainage issues that may develop over time on the cover system drainage-bench channels

This attachment presents the methods, assumptions, and material properties used in these analyses, as well as the results and conclusions.

2.0 SETTLEMENT ANALYSIS

2.1 METHODS AND ASSUMPTION

The settlement analysis was performed using Plaxis 2D software (Plaxis, 2014). Plaxis 2D is a two-dimensional (2D) finite element program that is used to perform deformation analysis for various types of geotechnical applications. Plaxis 2D includes advanced constitutive models that allow for the simulation of non-linear, time-dependent, and anisotropic behavior of soils and/or rock.

The analysis was performed for initial (occurring during construction) settlement and long-term (post-cover-construction) settlement. The initial construction settlement is used to provide initial configuration and stress-state information for the analyses of post-construction settlement. The post-construction settlement provides an estimate of deformations that may occur in the cover system due to long-term creep. Deformation of the cover system may have impacts on the long-term performance of the cover system, particularly with respect to liner strain and drainage. The lateral displacements of the pit dewatering sump risers due to construction and long-term creep deformations of the backfill also were estimated. In addition, the lateral displacements of the top cover surface due to long-term creep deformations of the backfill were estimated along with the corresponding maximum lateral strain on the cover geomembrane.

2.2 LOADING STAGES

Seven loading stages were used for the analysis and are as follows:

- Initial Condition: The initial condition was the empty pit, with only gravity loading of the bedrock being considered in order to establish initial stress conditions.
- First Loading Stage: The first loading stage included the underdrain and the first layer of waste rock and only considered the displacements associated with gravity loading from these layers.

- Second Loading Stage: The second loading stage included the gravity loading of the second layer of waste rock, with additional displacements of the previous (first layer) of waste rock being modeled using the Hardening Soil constitutive model.
- Third and Fourth Loading Stages: The third and fourth loading stages were similar to the second loading stage, with deformations and stress distributions of previous loaded layers being modeled using the Hardening Soil constitutive model.
- Final Construction Stage: In the final construction stage, the placement of the waste rock in the pit is complete and all of the layers of waste rock are modeled using the Hardening Soil constitutive model.
- Post-Construction Stage: After construction loading is completed, a time-dependent settlement analysis performed to evaluate the settlement after 50 years. During this post-construction stage, the waste rock is modeled using the Soft Soil Creep constitutive model.

In the model, gravity loads associated with pit backfilling in Pit 3 and Pit 4 were applied in four stages, referred to as the First through Fourth Loading Stages above. For the First through Fourth Loading Stages, gravity loads were applied and analyzed in increments that represent sequential waste rock placement in thicknesses of 75 to 100 feet, depending on the section being analyzed.

Given the coarse nature of the waste rock, it was assumed that pore pressures do not develop during placement of the waste rock in the Pits and during the long-term creep phase of the analysis. Therefore, all materials were assigned drained/effective stress parameters.

2.3 CONSTITUTIVE MODELS

Four constitutive (stress-strain response) models were used in the analyses of cover settlement to represent the responses of the different materials under specific loading conditions: elastic, Mohr-Coulomb, Hardening Soil, and Soft Soil Creep. These models are described in detail in the Plaxis Material Models Manual (Plaxis, 2014). The general input parameters required for each constitutive model are provided in the following subsections.

2.3.1 Linear Elastic Model

The linear-elastic model is the simplest constitutive model in Plaxis and is based on Hooke's Law for isotropic linear elastic behavior. The linear elastic model requires a Young's Modulus (E) and Poisson's ratio (ν) as input.

The linear elastic constitutive model was used to calculate initial stresses due in the pit bedrock layers prior to backfilling.

2.3.2 Mohr-Coulomb Model

The Mohr-Coulomb model is a linear-elastic perfectly-plastic constitutive model. For stress states that fall within the fixed yield surface (i.e. do not exceed the material strength), the material stress-strain behavior is defined by a linear-elastic behavior and the strains are reversible. However, once the stress at a given point reaches the yield surface (i.e. the strength is exceeded), plastic flow occurs and irreversible strains occur. For the Mohr-Coulomb constitutive model, the yield surface is defined by a friction angle (ϕ) and cohesion (c) which are

defined in the same manner as the failure envelop in traditional limit-equilibrium (e.g. slope stability) analyses. Since a linear elastic constitutive model is used to define the stress-strain response in the sub-yielding stress region, a Young's Modulus (*E*) and Poisson's ratio (ν) as described in Section 2.3.1 are also required as input.

The Mohr-Coulomb Constitutive Model was used to calculate stresses and strains in the bedrock and underdrain layers during backfill (construction) loading.

2.3.3 Hardening Soil Model

The Hardening Soil constitutive model is a more advanced constitutive model appropriate for many different types of soils. In the hardening soil constitutive model, when the soil is subjected to primary deviatoric (related to shear) loading, it shows a decreasing stiffness and simultaneously develops irreversible plastic strains (Plaxis, 2014). Unlike the Mohr-Coulomb model, the yield surface for the Hardening Soil model is not fixed in principal stress space, but can expand (harden) as plastic straining occurs. The Hardening Soil constitutive model is similar to the more familiar Duncan and Chang (1970) Hyperbolic model, but with some improvements that more accurately capture important aspects of the behavior of geologic materials during loading. These improvements include: 1) the Hardening Soil model uses the theory of plasticity rather than the theory of elasticity, 2) the Hardening Soil model includes soil dilatancy, and 3) the Hardening Soil model includes a yield cap (Plaxis, 2014).

The basic characteristics of the Hardening Soil model are (Plaxis, 2014):

- Stress dependent stiffness defined by a power law with an associated input parameter: *m*
- Plastic straining due to primary deviatoric loading – associated input parameter: the secant stiffness in a standard drained triaxial test (E_{50})
- Plastic straining due to primary compression – associated input parameter: the tangent stiffness for primary oedometer loading (E_{oed})
- Elastic unloading/reloading – associated input parameters: unloading/reloading stiffness (E_{ur}) and unloading/reloading Poisson's ratio (ν_{ur})
- Failure according to the Mohr-Coulomb failure criterion – associated input parameters: *c*, ϕ, and the angle of dilatancy (ψ).

The Hardening Soil Constitutive Model was used to calculate stresses and strains in the waste rock backfill during construction loading.

2.3.4 Soft Soil Creep Model

The Soft Soil Creep model was used to predict the long-term, post-construction time-dependent behavior of the waste rock under a nearly-constant state of stress. The basic characteristics of the model are (Plaxis, 2014):

- Stress-dependent stiffness (logarithmic compression behavior)
- Distinction between primary loading and unloading-reloading
- Secondary (time-dependent) compression
- Memory of pre-consolidation stress
- Failure behavior according to the Mohr-Coulomb criterion

The basic stiffness parameters for the Soft Soil Creep model include the modified swelling index (κ^*), modified compression index (λ^*), and modified creep index (μ^*). These values can be obtained from either an isotropic compression test or an oedometer test. The modified compression index is the slope of the normal consolidation line when the logarithm of stress is plotted as a function of strain. Similarly, the modified swelling index is the slope of the recompression line. The modified creep index can be estimated by plotting the long-term volumetric strain against the logarithm of time. Alternatively, the more traditional compression and recompression indices (C_c and C_r, respectively) can be entered into Plaxis, as well as the secondary compression index (C_α).

The Soft-Soil Creep Constitutive Model was used to long-term stresses and strains in the mine waste backfill layers after construction has been completed.

2.4 GEOMETRY

Two-dimensional sections of both Pit 3 and Pit 4 were evaluated for deformation, with two sections at critical locations analyzed for each pit. The locations of the sections used in the deformation analyses of Pit 3 and Pit 4 are shown on Figure 1 and Figure 2, respectively. Section A and Section B through Pit 3 are shown on Figure 3 and Section C and Section D at Pit 4 are shown on Figure 4. The sections are oriented to include the underdrain sump, and sump risers in each pit. The underdrain material was included in the model as a distinct material, however, the sump risers were not modelled as a separate material. The design of the well risers include the incorporation of friction sleeves and slip layers around the stainless steel well risers which will result in very little load transfer from the waste rock as it settles, to the steel riser pipes. In addition, the riser pipes will have a large amount of lateral flexibility. As such, the sump risers are not expected significantly influence the waste rock displacement patterns. The geometry used in modelling the sections in Plaxis is shown on Figure 5 through Figure 8.

The bedrock in the analysis (assigning the appropriate geologic units) was defined based on the engineering geology maps generated by geologic mapping performed as part of the Midnite Mine Pre-Design Investigations prepared by Miller Geotechnical Consultants (MGC, 2011).

2.5 MATERIAL PROPERTIES

The strength and deformation parameters for the materials used in the analysis were developed based on laboratory data, field test data, and previous experience with similar materials, as well as a one-dimensional calibration to rockfill settlement data from published literature sources as described in Section 4.1.1. The values selected and the basis of these values is provided in the following subsections. A summary of the material properties used in the analysis is provided in Table 1 and are discussed in detail in the following subsections.

Table 1. Summary of Material Properties

Material	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)	Poisson's Ratio	Young's Modulus (psf)	Rock Deformation Modulus (psf)
Waste Rock & Existing Waste	119	0	38	0.3	8.01×10^5	---
Bedrock - Quartz Monzonite	145	57,200	35.8	0.25	---	3.12×10^7
Bedrock - Schist/Phyllite	145	42,300	27.6	0.25	---	3.83×10^7
Bedrock - Calc-Silicate	145	78,900	32	0.25	---	7.37×10^7
Underdrain (Hillside Dump)	108	0	35	0.3	8.01×10^5	---

2.5.1 Waste Rock Backfill

The waste rock backfill that will be placed in the Pit 3 and Pit 4 currently is located in existing waste rock piles throughout the Site. Field and laboratory tests were performed in the South Spoils and Hillside Dump waste rock piles as part of an investigation performed by URS (URS 2002). The investigation included both excavation of test pits and drilling of test holes in each of the waste rock piles. In-situ density tests were performed in the test pits and samples of waste rock were obtained for gradation and other index property testing. Bulk samples (samples from the test pits with similar index properties) were used for triaxial shear testing.

It should be noted that it appears that only the gravel-sized (3-inch) and finer materials were included in the laboratory test specimens. A review of test pit logs from the 2002 investigation indicates that a significant percentage (in some cases greater than 50 percent) of the waste rock material was larger than 3 inches in size and was excluded from the test specimens. Due to the exclusion of a significant amount of coarser-grained material the compressibility and strength parameters estimated from these laboratory test results will be conservative and result in higher estimated settlements.

General Properties. The unit weight of the waste rock was estimated from the results of the in-situ density tests. The measured moist unit weight ranged from 95 pcf to 124 pcf, with an average value of approximately 119 pcf. The average value was selected for use in this analysis. These in-situ density tests did include coarser fractions of the waste rock that were excluded from the laboratory samples, and are considered representative of the whole waste rock density that will be achieved in the waste rock backfill.

The consolidated-undrained (CU) triaxial tests were performed on the bulk samples of waste rock compacted for 90 percent of the standard proctor maximum dry density at the natural moisture content. The CU tests resulted in a range of effective friction angles from about 33 to 44 degrees and a range of cohesion of 0 to 650 psf. A friction angle of 38 degrees with zero cohesion was used for the waste rock in stability analyses performed as part of the current design activities. This value is consistent with the laboratory data and was used in the settlement analysis in the Mohr-Coulomb model.

The Young's modulus, or elastic modulus, for the waste rock was estimated using both the laboratory data (triaxial stress-strain curves) and field data (SPT N-values) since both types of

data were available and neither type of data was considered more accurate than the other. The average value of Young's Modulus obtained from the triaxial test data was approximately 1.78×10^6 psf. Correlations provided by Bowles (1996) and Schmertmann (1970) were used to estimate the Young's Modulus from SPT N-values collected in the field by URS (2002). The average $N_{1,60}$ value estimated from the SPTs performed in the waste rock is 26 and the 33rd percentile value is estimated at approximately 14. Both field and laboratory test values were used to estimate the Young's Modulus. The Young's Modulus values calculated from empirical correlations with SPTs range from 2.51×10^5 to 8.02×10^5 psf. A value of 3.6×10^5 psf was selected for the SPT-based estimate of Young's Modulus. The geometric mean of the laboratory-based and SPT-based estimates of Young's Modulus was used in the analysis and is 8.01×10^5 psf.

The Poisson's ratio for the waste rock was selected based on typical values for similar materials. Poisson's ratio typically ranges from 0.2 to 0.3 for cohesionless soils and from 0.3 to 0.4 for clayey soils. A value of 0.3 was used for the waste rock in the settlement analyses.

Properties for Constitutive Models. The input parameters for the Hardening Soil and Soft Soil Creep constitutive models, both of which were used for the waste rock in the settlement analysis, are discussed in in the following subsections. The values selected for the waste rock are summarized in Table 2 and are discussed in more detail below.

Table 2. Summary of Waste Rock Constitutive Model Input Parameters

Property	Value
Hardening Soil Model	
Secant Stiffness in standard drained triaxial test, E_{50}	1.15×10^6 psf
Tangent stiffness for primary oedometer loading, E_{oed}	1.08×10^6 psf
Unloading/reloading stiffness, E_{ur}	3.46×10^6 psf
Stress dependency, *m*	0.5
Dilatancy Angle	8 degrees
Soft Soil Creep Model	
C_c	0.08
C_r	0.008
C_α	0.003 – 0.006*

** Range of values considered in one-dimensional calibration model.*

Waste Rock - Hardening Soil Model. The secant stiffness in a standard drained triaxial test (E_{50}) was estimated using the triaxial test data for the waste rock. The secant stiffness was calculated from the stress-strain curves obtained from the triaxial tests based on the definition provided in the Plaxis Material Model Manual (Plaxis, 2014), which is shown graphically in Figure 9. The average E_{50} estimated from the nine CU triaxial tests performed on the waste rock from the South Spoils, which was used in the settlement analysis, is 1.33×10^6 psf. This estimated value is an undrained modulus. To estimate the drained modulus from the undrained modulus, the following equation was used:

$$E'_{50} = E_{50} \frac{2(1+\nu)}{3}$$

where:

E'_{50} – Drained secant stiffness
E_{50} – Undrained secant stiffness

ν – Poisson's Ratio

The estimated drained secant stiffness (E'_{50}) is approximately 1.15×10^6 psf. This is the value that was used in the settlement analysis.

The tangent stiffness for primary oedometer loading (E_{oed}) was calculated using the following equation:

$$E_{oed} = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}$$

where:

E – Young's Modulus
ν – Poisson's Ratio

The value of E_{oed} calculated using the above equation, a Young's Modulus of 8.01×10^5 psf, and a Poisson's ratio of 0.3 is 1.08×10^6 psf.

The unloading/reloading stiffness (E_{ur}) was estimated using the secant stiffness (E_{50}). As the triaxial testing did not include an unload/reload cycle, the default value in Plaxis of three times E_{50} (3.99×10^6 psf) was assumed for E_{ur} in the settlement analysis.

The exponent in the Hardening Soil constitutive model, *m*, represents the stiffness dependence on stress-level. As discussed in the Plaxis (2014), Janbu (1963) reported values of *m* around 0.5 for sands and silts. This value was assumed as a conservative estimate for the waste rock in the settlement analysis.

The Mohr-Coulomb parameters ϕ and c used in the Hardening Soil model are the same values as discussed in the General Properties section. The angle of dilatancy (ψ) was estimated based on information provided in Plaxis (2014). The value of ψ can be estimated for cohesionless soils with friction angles greater than 30 degrees as $\psi \approx \phi - 30°$. As the friction angle (ϕ) of 38 degrees is being used for the waste rock in the settlement analysis, the angle of dilatancy (ψ) is estimated to be 8 degrees.

Waste Rock - Soft Soil Creep Model. The compression index (C_c) for the waste rock was estimated based on previous experience with similar materials and void ratios. A C_c of 0.08 was used in the settlement analysis.

The recompression index, which is referred to as the swelling index in Plaxis (C_s), was estimated to be 10 percent of C_c, or 0.008.

The secondary compression index (C_α) was initially estimated using data from the consolidation phase of the triaxial testing. An average value of 0.003 was calculated from the consolidation vertical displacement versus time curves. A C_α value of 0.006 was obtained from previous experience for a similar material. A one-dimensional (1D) analysis was performed for the waste rock to calibrate the settlements obtained from Plaxis to measured rockfill settlements by changing the secondary compression index. This 1D calibration analysis is discussed in more detail in the Section 4.1.1.

The required Mohr-Coulomb input parameters for the Soft Soil Creep model are the same as those used for the Hardening Soil model.

2.5.2 Underdrain

As currently planned, the underdrain will be constructed from material processed from the Hillside Dump. The unit weight of the underdrain material was estimated from in-situ density tests performed in the test pits at the Hillside Dump (URS 2002). An average value of 108 psf was used in the settlement analysis. One set of triaxial tests was performed on material from the Hillside Dump, which resulted in a friction angle of about 35 degrees and a cohesion of 400 psf. A friction angle of 35 degrees with zero cohesion was used for the underdrain in the settlement analysis. The Young's Modulus for the underdrain material was assumed to be the same as for the waste rock material for the purposes of these analyses. The underdrain was modeled using the Mohr-Coulomb constitutive model.

2.5.3 Bedrock

As mentioned above, there are three main units of bedrock at Pit 3 and Pit 4: quartz monzonite, schist/phyllite, and calc-silicate. The rock strengths were estimated for each of the units, as discussed below.

The unit weight for all units of bedrock was estimated from laboratory density tests performed on rock cores obtained during a site investigation performed by as part of the Midnite Mine Storage Ponds Investigation (MWH, 2012). A conservative value of 145 pcf was selected for the bedrock. Also, a typical Poisson's ratio for rock of 0.25 was assumed for all bedrock units in Pit 3 and Pit 4.

The bedrock was modeled using the Mohr-Coulomb constitutive model. The input parameters for the Mohr-Coulomb model are provided below.

Bedrock - Quartz Monzonite. The friction angle, cohesion, and rock deformation modulus for quartz monzonite were estimated as part of the Midnite Mine Pre-Design Investigations (MGC 2011). These values were obtained using RocLab (RocScience, 2002) and the following estimated Hoek-Brown parameters: GSI = 37, UCS = 1000 ksf, m = 29, E_i = 240,000 ksf. The estimated friction angle and cohesion of 35.8 degrees and 57,200 psf, respectively. The estimated rock deformation modulus was 3.12×10^7 psf.

Bedrock - Schist/Phyllite. The strength properties for the schist/phyllite rock were also estimated as part of the Midnite Mine Pre-Design Investigations (MGC, 2011). The Hoek-Brown parameters used to estimate the strength and deformation properties were: GSI = 40, UCS = 1000 ksf, m = 10, E_i = 240,000 ksf. The estimated friction angle and cohesion based on the Hoek-Brown parameters, were 27.6 degrees and 42,300 psf, respectively. The rock deformation modulus was estimated to be about 3.83×10^7 psf.

Bedrock - Calc-Silicate. The strength and deformation properties for the calc-silicate rock were also estimated as part of the Midnite Mine Pre-Design Investigations (MGC, 2011). The Hoek-Brown parameters estimated for the calc-silicate are: GSI = 50, UCS = 1500 ksf, m = 12, E_i = 240,000 ksf. The estimated friction angle and cohesion were 32 degrees and 78,900 psf, respectively. The rock deformation modulus was estimated to be approximately 7.37×10^7 psf.

2.5.4 In-Place Waste Rock

In Section D at Pit 4, the waste rock will be placed over existing in-place waste rock in Area 5. In addition, in the toe Area of Pit 3 and in the Backfilled Pit Area (BPA) the existing waste rock will be left in place and regraded. The in-place waste rock was assumed to have the same material properties as the waste rock backfill and was assigned the same Hardening Soil constitutive model parameters. The existing waste was included in the initial stage gravity loading along with the bedrock (backfilling of Pit 4 has not begun).

3.0 DIFFERENTIAL SETTLEMENT

The results of the Plaxis long-term creep settlement analyses were used to calculate differential settlement in the waste cover system. The initial construction settlement associated with backfill loading is expected to occur very rapidly and will be largely complete prior to final grading of the waste surface and construction of the cover system. Therefore, only the long-term, creep-related settlement was included in the differential settlement analysis.

Three-dimensional (3D) deformations in the cover systems were estimated from the results of the 2D settlement analysis using the relationship developed between the vertical creep settlement and the thickness of the waste fill material (shown in Figure 20). Computer-aided design (CAD) software was used to compare the as constructed (i.e., pre-creep) cover elevation surfaces to the existing grade surfaces in order to calculate thickness of waste fill material at each point across the two surfaces. The relationship between the vertical creep settlement and the thickness of the waste fill material was then used to create creep settlement surfaces for Pit 3 and Pit 4. The creep settlement surfaces were subtracted from the end-of-construction (i.e., pre-creep) surfaces to generate final settled cover surfaces.

Because the cover bench channels are designed to be constructed with fairly shallow slopes, post-creep elevation profiles were created for each bench channel in order to identify areas where ponding may occur due to slope reversal as a result of creep settlement. The post-settlement bench channel slopes were compared with the recommended minimum permissible design slope for bench channels provided in Appendix F (Supplement F-3.5). Post-settlement ground surface profiles were created using the final settled cover surfaces.

4.0 RESULTS

4.1 SETTLEMENT ANALYSIS

4.1.1 1D Calibration Analysis

As mentioned previously, a one-dimensional analysis was performed for the waste rock to calibrate the settlements obtained from Plaxis to measured rockfill settlements presented in the literature by varying the secondary compression index. The measured settlements for rockfill dams are reported by Oldecop and Alonso (2007). The settlement results obtained from the Plaxis calibration runs are presented, along with the measured waste rock settlements, on Figure 10.

For the one-dimensional calibration model, a 300-ft tall column (representing Pit 4) and a 400-foot tall column (representing Pit 3) of waste rock material were generated in Plaxis. The waste rock was first loaded by gravity and assigned the Hardening Soil model with the associated material properties. The waste rock material was then assigned the Soft Soil Creep model and allowed to deform over a period of 30 years (the period of time shown in Figure 1 of Oldecop and Alonso). The percent settlement (settlement divided by the height of the column and multiplied by 100) was plotted versus time and compared with the datasets presented in Figure 1 in Oldecop and Alonso (2007). The results of the 1D calibration runs are shown in Figure 10.

The calibration analysis was performed for a range of values of the secondary compression index values, C_α. The initial analysis was performed assuming a C_α of 0.006. Additional analyses were performed assuming C_α values of 0.004 and 0.003. A C_α of 0.004 provided results similar to the Rivera de Gata and Beliche Dams and was considered a conservative estimate of the secondary compression index, as the value provides settlements on the higher end of values measured for rockfill dams. A C_α value of 0.004 was used in the subsequent 2D cover settlement analyses.

4.1.2 2D Deformation Analysis

The 2D deformation analysis was performed for four sections (two for Pit 3 and two for Pit 4) for construction and post-construction conditions (i.e. Figures 3 and 4 for Pits 3 and 4, respectively). Vertical settlement contours for the end of construction (EOC) and post-construction (Creep) are shown in Figure 11 through Figure 18. A summary of the settlement, shown in the contours for the two stages (EOC and Creep), is provided in Table 3.

Table 3. Summary of Maximum Calculated Settlements in Waste Rock

Section	End of Construction (ft)	50 years (Creep) (ft)	Total (ft)
Pit 3 - A	4.6	6.4	11
Pit 3 - B	5.7	7.2	13
Pit 4 - C	2.2	3.7	5.9
Pit 4 - D	2.8	4.5	7.3

The Creep settlements are relevant for estimating deformations in the cover system over time. The relationship between calculated creep settlements and time are shown for each section in Figure 19. As can be seen on Figure 19, creep settlement initially occurs at a rapid rate, and although creep settlement continues indefinitely, the rate slows considerably after about 10 years. As a result, subsequent estimates of long-term creep settlement have been based on the settlement that occurs within the first 50 years of completion of construction.

In order to estimate the settlement patterns across the cover systems from the results of the analyses of the 2D sections, a relationship was developed between the vertical creep settlement and the thickness of the waste fill material at ten locations, with varying fill thicknesses, in each 2D section analyzed. The results of the 2D analyses were used to develop the relationship between creep settlement and fill thickness shown in Figure 20. As shown in Figure 20, there is a linear trend (represented by the line on the drawing) to the relationship between creep

settlement and fill thickness, and a linear regression was used to develop a best fit line. The equation of the line then was used to estimate cover settlement at other off-section locations within each backfilled pit.

In addition to estimating the vertical settlements that would affect the cover system, the 2D settlement analysis was used to estimate the lateral displacements at the sump risers both during construction due to backfill loading and after construction. The estimated distribution of the lateral displacements at the location of the sump risers is shown for each section in Figure 21 through Figure 28. A summary of the maximum lateral displacements for each section at the sump risers is provided in Table 4. In addition, the total estimated horizontal displacement for the sump risers in each pit was calculated as the resultant of the estimated displacements vectors from the two approximately perpendicular analysis sections in each pit, as well as the resulting deviation from vertical are also summarized in Table 4.

Table 4. Summary of Maximum Calculated Lateral Displacements at the Sump Risers

	In-Section Displacement			Resultant Displacement			
Section	EOC (ft)	50 years (Creep) (ft)	Total (ft)	EOC (ft)	50 years (Creep) (ft)	Total (ft)	Deviation from Vertical
Pit 3 - A	0.96	1.7	2.7	0.97	1.9	2.8	0.6%
Pit 3 - B	0.13	0.78	0.9				
Pit 4 - C	0.10	0.31	0.4	0.42	0.42	1.3	0.4%
Pit 4 - D	0.41	0.82	1.2				

The lateral displacements of the top cover surface due to long-term creep deformations were also estimated based on the 2D deformation analyses. The results were used to evaluate potential effects of lateral displacements on the cover geomembrane, and the non-welded cover geomembrane overlap at the drainage benches. The average lateral displacements and associated lateral strains were calculated at approximately 100-ft intervals along the sections that were analyzed as part of the 2D deformation analysis as shown on Figures 29 and 30. Calculation sheets are provided in Supplement D-13.1. Note that negative strains indicate compression along a particular 100-foot section, which is not of consequence in terms of either geomembrane yielding or loss of geomembrane continuity at non-welded geomembrane overlaps. Positive (tensile) strains have potential consequences in terms of geomembrane yielding and loss of overlap continuity, and were used for evaluation of the effects of creep deformation on cover performance.

The maximum potential geomembrane extension along off-section (i.e. not located along 2d deformation analysis sections) locations was conservatively estimated using the maximum average lateral tensile strain calculated for any 100-ft spacing along an interbench slope (1.06 percent) multiplied by a representative maximum interbench slope length (approximately 250 feet). The locations of the maximum average lateral strain and representative interbench slope length are indicated on Figure 29. Although a higher strain (1.96) was calculated at on location farther to the east of the location selected, it did not occur along a cover slope as can be seen on Figure 29, and was not considered relevant to the analyses of non-welded cover overlap areas.

Using the estimated maximum lateral down-slope strain of 1.06 percent, the calculated maximum lateral extension along the 250-foot slope length is approximately 2.7 feet. This maximum lateral extension will result in both stretching (tensile straining) of the cover system LLDPE geomembrane as well as sliding of the geomembrane along the non-welded geomembrane overlap at the drainage benches. This sliding will occur over that portion of the free end of the geomembrane in the overlap area that does not have sufficient embedment to resist the tensile forces imparted by the lateral extension. The length of the free end of the geomembrane that may be subject to sliding can be calculated based on the required development length needed to resist the tensile forces imparted on the geomembrane by the creep-related lateral extension. The maximum tensile forces that may be developed in the geomembrane can be calculated as:

$$T = \sigma_t A$$

Where:

T = maximum imparted tensile force within the geomembrane
σ_t = imparted tensile force within the geomembrane
A = Cross sectional area of geomembrane = 0.48 in^2/ft for 40-mill LLDPE
$\sigma_t = E\varepsilon_t$
E = Modulus of LLDPE geomembrane = 60,000 psi (typical, 2 percent strain)
ε_t = maximum tensile Strain = 0.0106

Based upon the above considerations, the maximum tensile force imparted on the geomembrane due to creep deformations is expected to be approximately 305 lbs/ft. Likewise, the required development length needed at the free end to resist movement can be calculated based on the interface shear resistance on the top and bottom surfaces of the geomembrane as:

$$R = \sigma_v \tan\delta_U + \sigma_v \tan\delta_L$$

Where:

σ_v = vertical stress due to soil cover loading = d*γ_s
d = thickness of soil cover = 3 ft nominal, actually somewhat thicker at outer edge of drainage benches
γ_s = unit weight of soil cover = 110 pcf as described in Attachment D-7
So:
σ_v = 330 psf
δ_U = frictional resistance along upper geomembrane/soil cover interface = 22.7 degrees as described in Attachment D-7.
δ_L = frictional resistance along lower geomembrane/GDL interface = 25.4 degrees as described in Attachment D-7.

Which results in a calculated pullout resistance of R = 330tan(22.7) + 330tan(25.4) = 295 psf. The required geomembrane development length required to resist the tensile force imparted by the creep-related extension can then be calculated as:

$$L_d = T/R = 305 \text{ (lb/ft)} / 295 \text{ (psf)} = 1.03 \text{ ft.}$$

Thus, it can be concluded that only the first one foot of LLDPE geomembrane nearest the free end of the overlap will potentially be subjected to slippage along the overlap, whereas the majority of the geomembrane along the 250-long slope can be expected to experience stretching. The actual amount of slippage along the one-foot portion of geomembrane at the free end is expected to be less than one inch. Thus, the proposed 5-foot non-welded geomembrane overlap at each drainage bench is expected to be more than sufficient to avoid separating due to post-construction, creep-related deformations.

The synthetic geomembrane proposed for use in the cover system is specified as 40-mil GSE UltraFlex Textured LLDPE Geomembrane or equivalent. The maximum percent elongation at break for this material is 250 percent. The maximum calculated average lateral strain (based on an approximately 100-ft spacing) developed in the geomembrane due to lateral displacement (1.96 percent in a cross-slope direction) is two-and-a-half orders of magnitude lower than this specified break strain. Thus the proposed LLDPE geomembrane is expected to have sufficient flexibility to accommodate post-construction, creep-related deformations.

4.1.3 Three-Dimensional (3D) Settlement Analysis

3D deformations in the cover systems were estimated as described in Section 3 using the relationship developed as described in Section 4.1.2 between the vertical creep settlement and the thickness of the waste fill material. Creep settlement contours created using this relationship are presented in Figures 31 through Figure 34. The creep settlement contours were subtracted from the end-of-construction (i.e., pre-creep) contours to create final settled cover contours, as presented in Figures 35 and 36 for Pit 3 and Pit 4, respectively.

4.2 DIFFERENTIAL SETTLEMENT

Post-creep settlement of the cover was calculated over the entire waste containment area cover as described in Section 3.0. Based on the estimated final settled cover contours, profiles were created along the flow line of each drainage bench channel on the cover system in order to identify areas of potential ponding that may develop with long-term creep settlement. Plan views of bench channel profile lines are presented on Figure 37 and Figure 43 for Pit 3 and Pit 4, respectively. Profiles for each bench channel are presented on Figures 38 through 42 for Pit 3 and on Figures 44 through 47 for Pit 4. All bench channels maintain a down-gradient slope greater than 0.5 percent after long term creep settlement. This slope is greater than the minimum permissible design slope of 0.4 percent for the bench channels (see Supplement F-3.5 of Appendix F, Calculation Brief Addendum, WCA Bench Channel Design Update).

5.0 CONCLUSIONS

The results of settlement analysis presented herein were used to evaluate the design of the cover system and the potential for lateral displacement of the sump risers due to backfill loading and long-term creep. The magnitude of the settlements is considered reasonable for the evaluated loading conditions. The use of a relationship to estimate the vertical settlement

based on fill thickness represents an estimation of the three-dimensional behavior of the waste rock in the backfilled mine pits, but cannot fully account for the actual three-dimensional effects that may occur during and after construction. Generally, these three-dimensional effects are expected to result in slightly lower maximum settlements than would occur under two dimensional conditions due to out-of-plane effects of the relatively rigid pit walls.

Long-term maximum creep settlements of approximately 7.2 feet in Pit 3 and 4.5 feet in Pit 4 are anticipated, however the differential settlement between adjacent points will be much lower. Overall, the estimated long-term creep settlements will not result in significant changes to the cover geometry or flow directions due to the relatively steep grades of the majority of the as-designed cover surface.

The evaluation of estimated long-term creep settlement along bench channels indicates that all bench channels will maintain a down-gradient slope greater than the minimum permissible design slope of 0.4 percent after settlement. Though ponded areas are not expected, the drainage benches will be monitored as described in the long-term OM&M plan and will be regraded as necessary if long-term creep settlement leads to conditions where drainage is not occurring as designed.

The amounts of lateral displacement of the sump riser pipes due to construction loadings, and from long-term creep settlement were also estimated. It is estimated that total lateral displacements due to both construction loading and long-term creep will result in deflections of the sump risers between 0.4 percent (at Pit 4) and 0.6 percent (at Pit 3). These deviations from vertical are not sufficient to represent problems for the functioning of the dewatering risers as the predicted deviations from vertical are relatively minor and occur in a uniform manner with fill height.

The lateral extensions of the cover geomembrane between drainage benches due to long-term creep deformation of the backfill is estimated to be a maximum average of 2.7 feet. Due to the flexibility and interface frictional resistance of the textured LLDPE geomembrane proposed for the cover system, the vast majority of the geomembrane will experience stretching, rather than interface sliding due to the predicted lateral extensions. Only the outermost free edge of the non-welded geomembrane overlap at each drainage bench is expected to experience interface slippage, with the actual displacement along the interface expected to be less than one inch. Thus, the design overlap of 5 feet is more than sufficient to maintain geomembrane continuity.

The estimated tensile strains that will develop within the cover geomembrane due to long-term differential settlement are significantly less than the maximum strain the geomembrane is able to withstand. The maximum calculated longitudinal strain developed in the geomembrane due to differential settlement is two-and-a-half orders of magnitude lower than the specified break strain for the LLDPE geomembrane. As such, the longitudinal strains induced in the geomembrane liner by creep settlement are considered acceptable and will not cause failure of the liner.

6.0 REFERENCES

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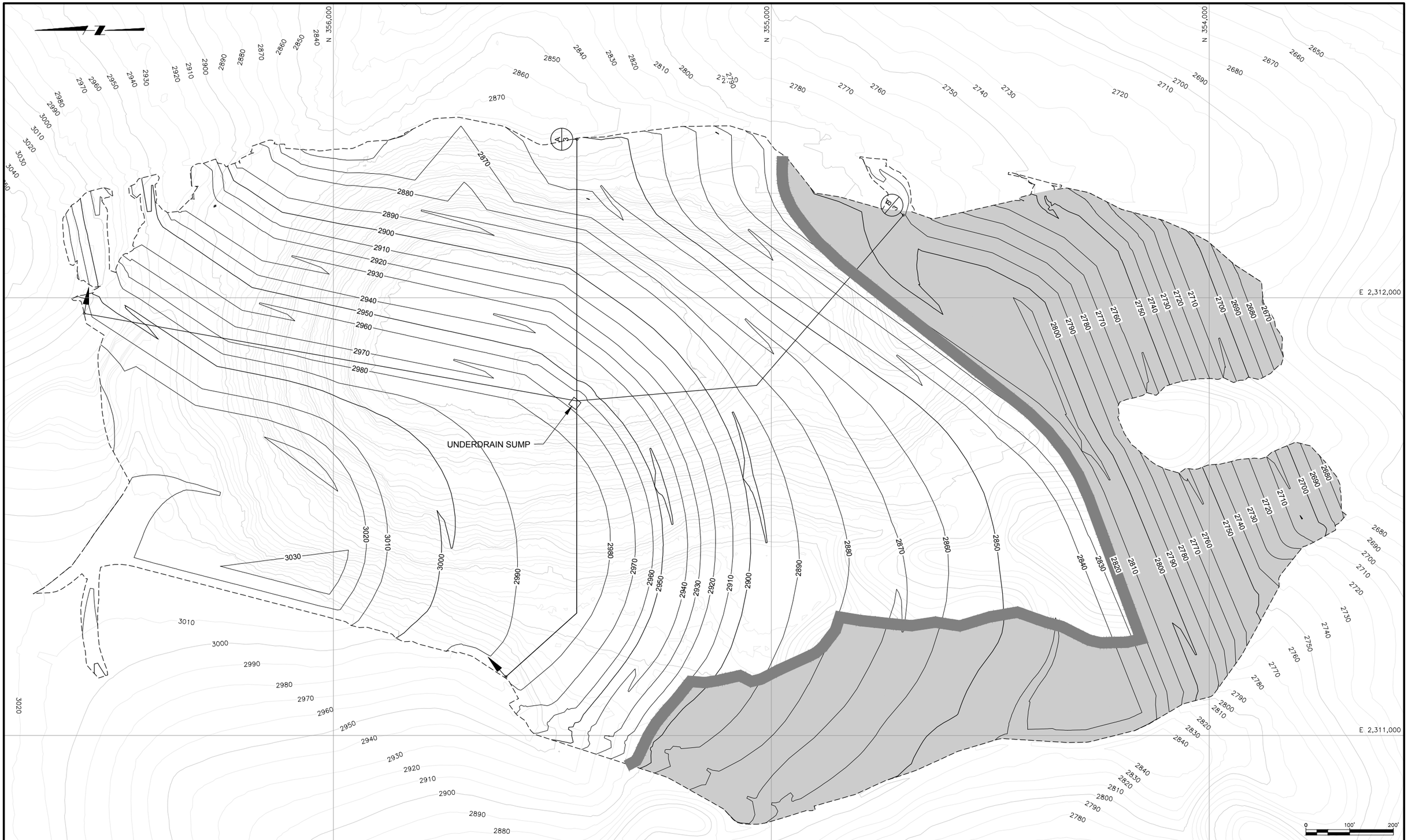
FIGURES



LEGEND:

5605	EXISTING GROUND SURFACE ELEVATION, FEET
5605	FINISHED GROUND SURFACE ELEVATION, FEET
	PHASE 2 ANCHOR TRENCH
	EXTENT OF PHASE 2 COVER (REMAINING AREA OF PIT 3 WILL BE COVERED IN PHASE 3)

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS SECTION LOCATIONS
FIGURE 1	REVISION 0
FILE NAME 1011322 SLOC	DATE JULY 2014

MWH

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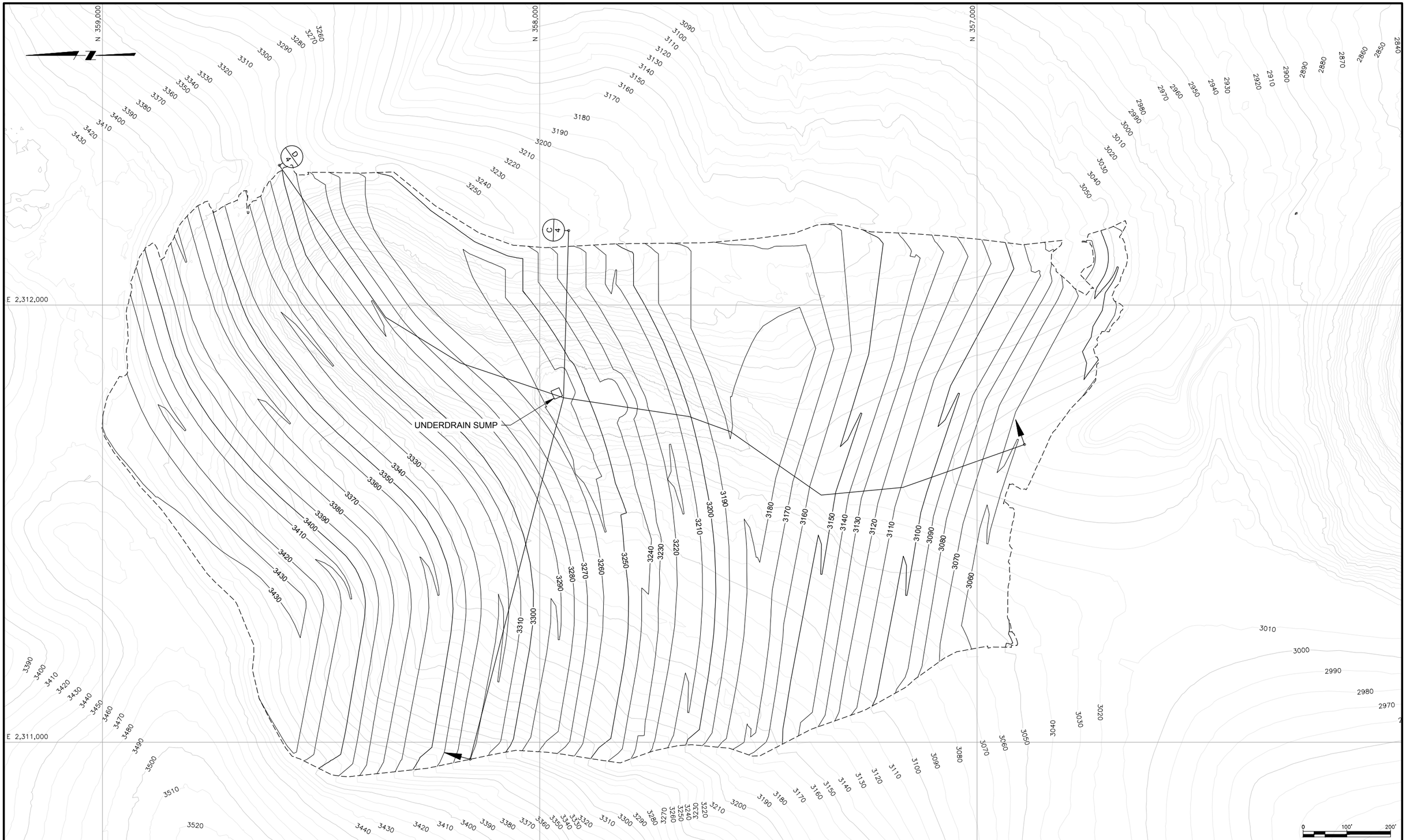

LEGEND:

5605 — EXISTING GROUND SURFACE ELEVATION, FEET

5605 — FINISHED GROUND SURFACE ELEVATION, FEET

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS SECTION LOCATIONS
FIGURE 2	REVISION 0
FILE NAME 1011322 SLOC	DATE JULY 2014

MWH

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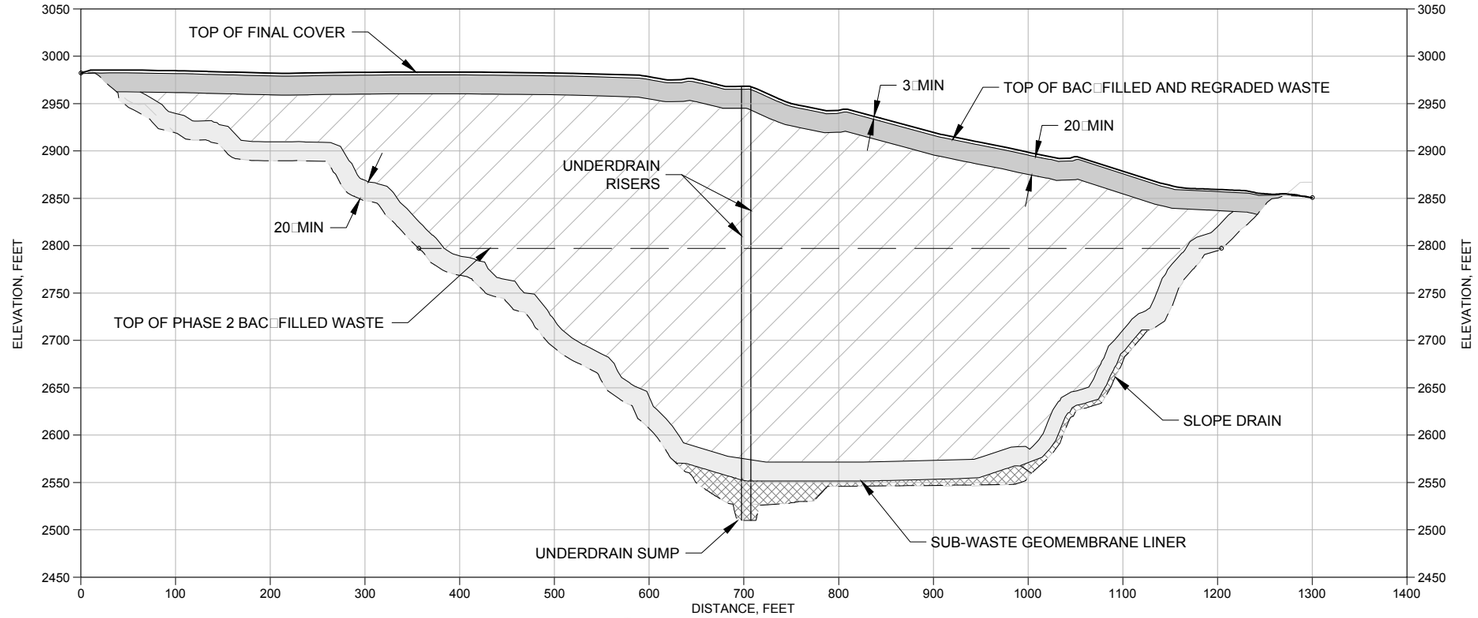
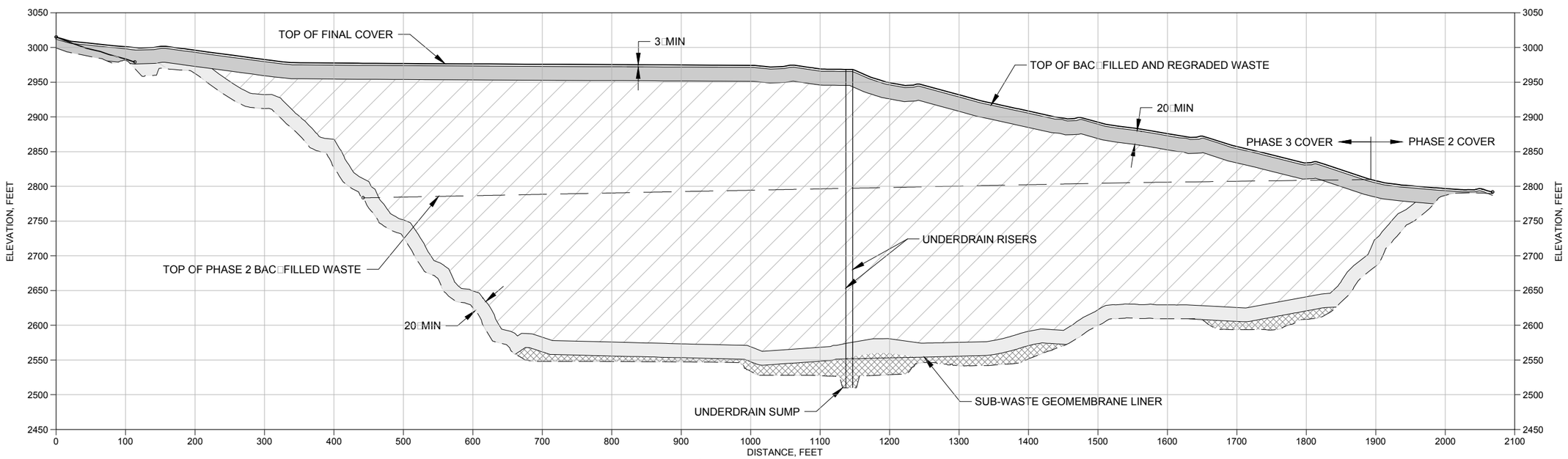


A
1

SECTION A

0 80' 160'



B
1

SECTION B

0 80' 160'

LEGEND:

- MINE SITE SUBGRADE
- EXISTING GROUND SURFACE
- REGRADED PIT BOTTOM
- UNCLASSIFIED WASTE
- LOW–REACTIVITY WASTE
- DRAIN GRAVEL
- LOW–ACTIVITY WASTE

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS SECTIONS

MWH

FIGURE 3	REVISION 0
FILE NAME 1011322 □SLOC	DATE JULY 2014

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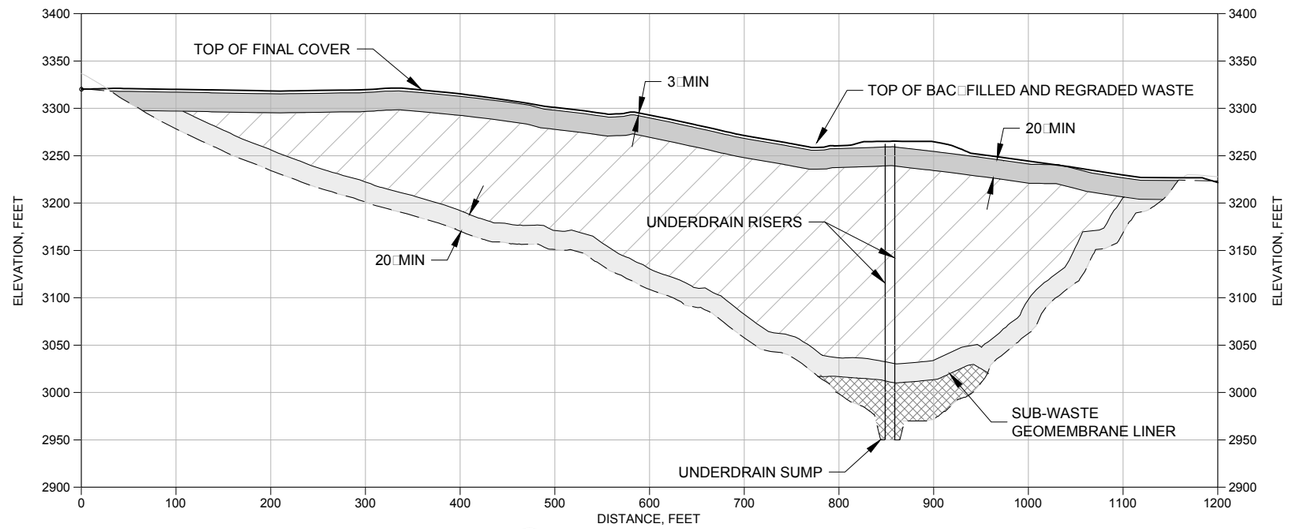


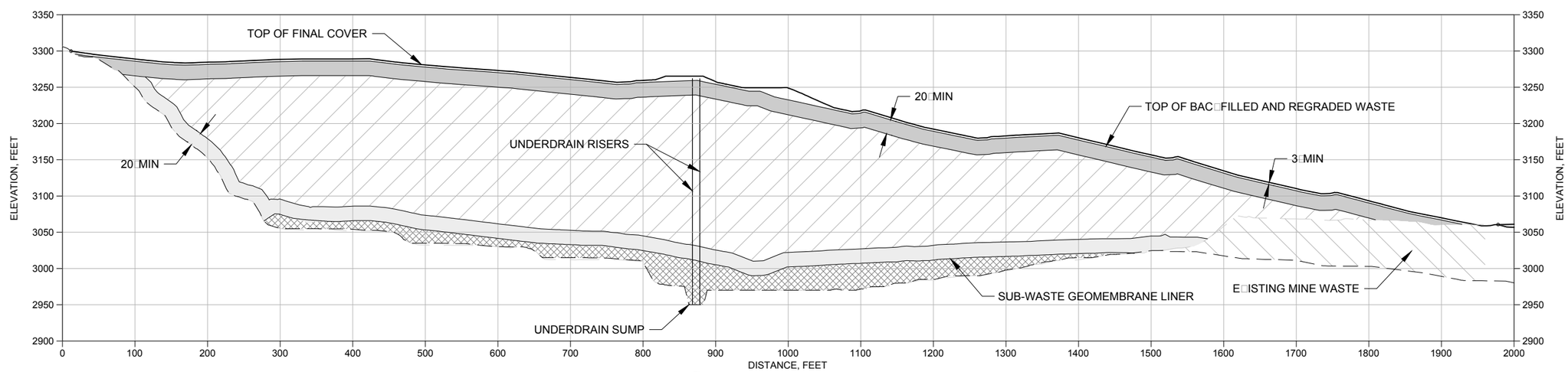
C
2

SECTION C

0 80' 160'

LEGEND:

- MINE SITE SUBGRADE
- EXISTING GROUND SURFACE
- REGRADED PIT BOTTOM
- UNCLASSIFIED WASTE
- LOW–REACTIVITY WASTE
- DRAIN GRAVEL
- LOW–ACTIVITY WASTE



D
2

SECTION D

0 80' 160'

0 80' 160'

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS SECTIONS

MWH

FIGURE 4	REVISION 0
FILE NAME 1011322 □SLOC	DATE JULY 2014

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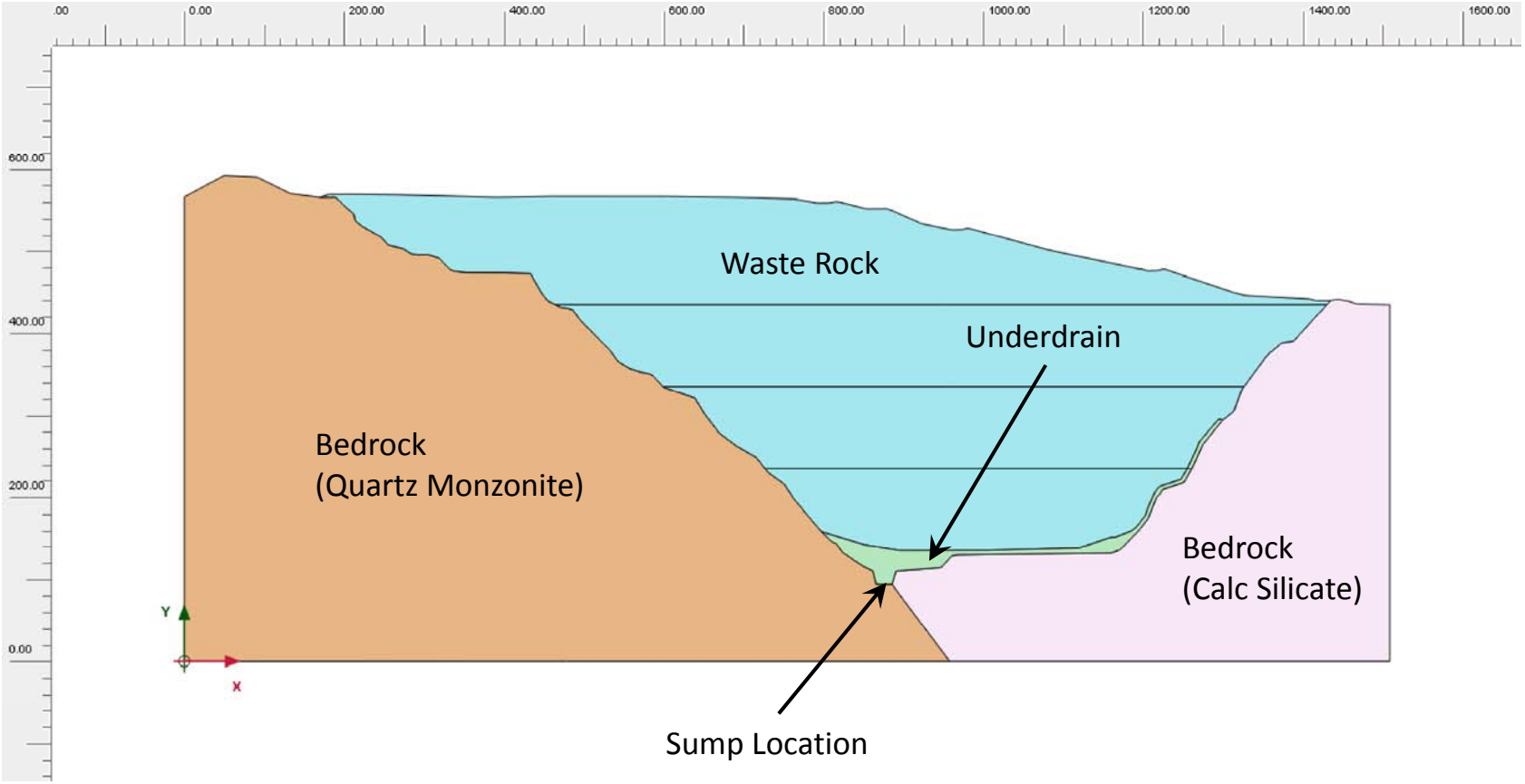
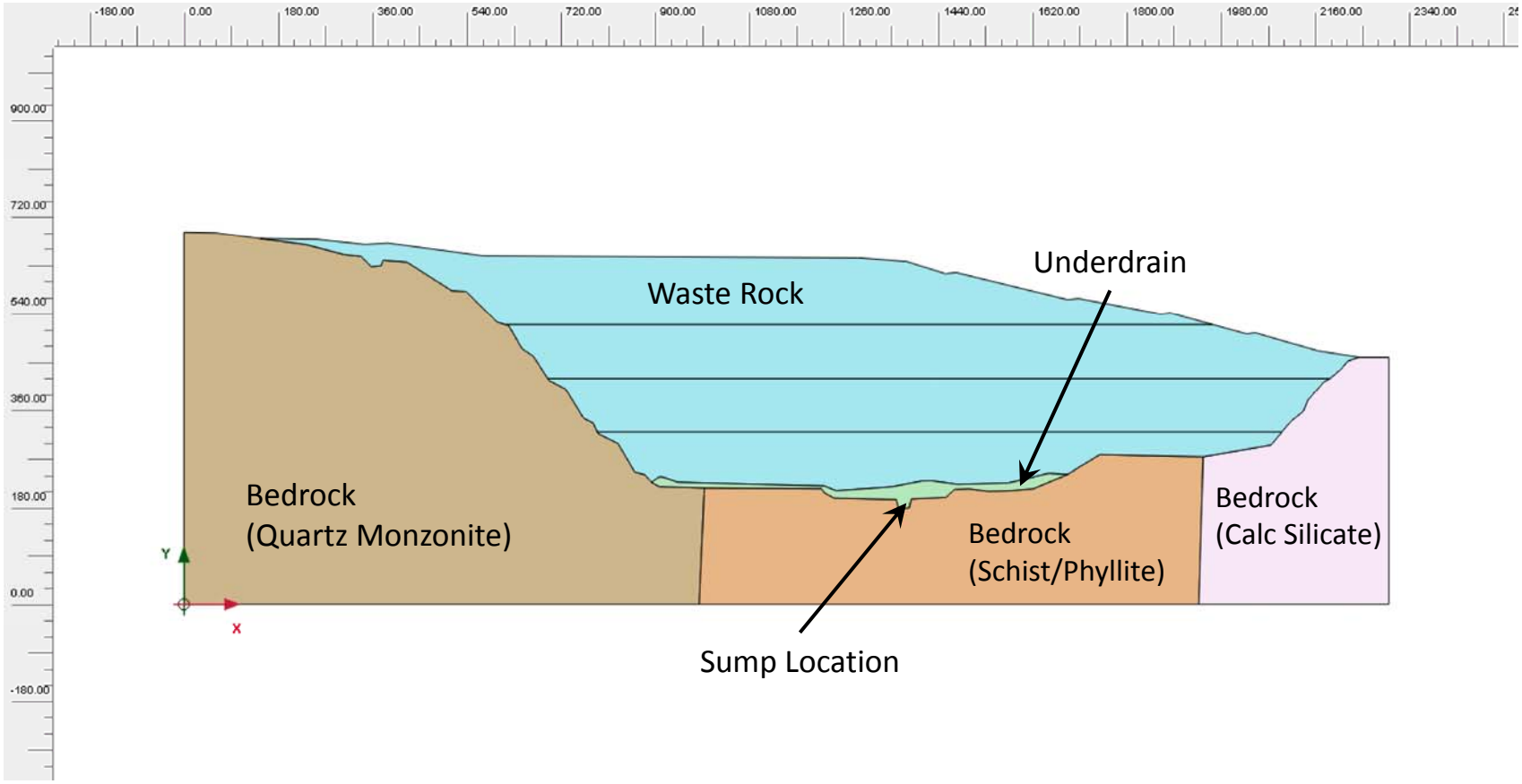
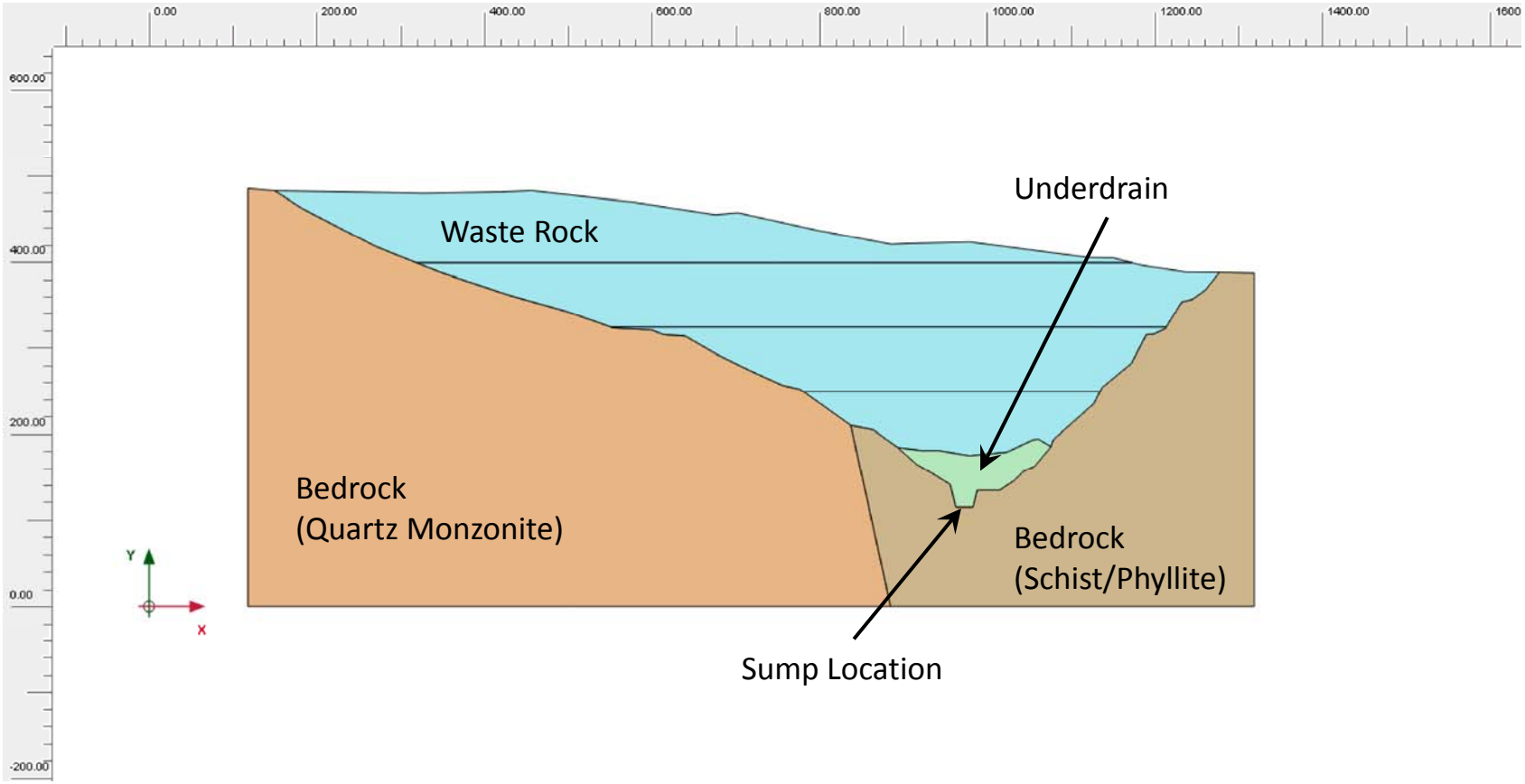


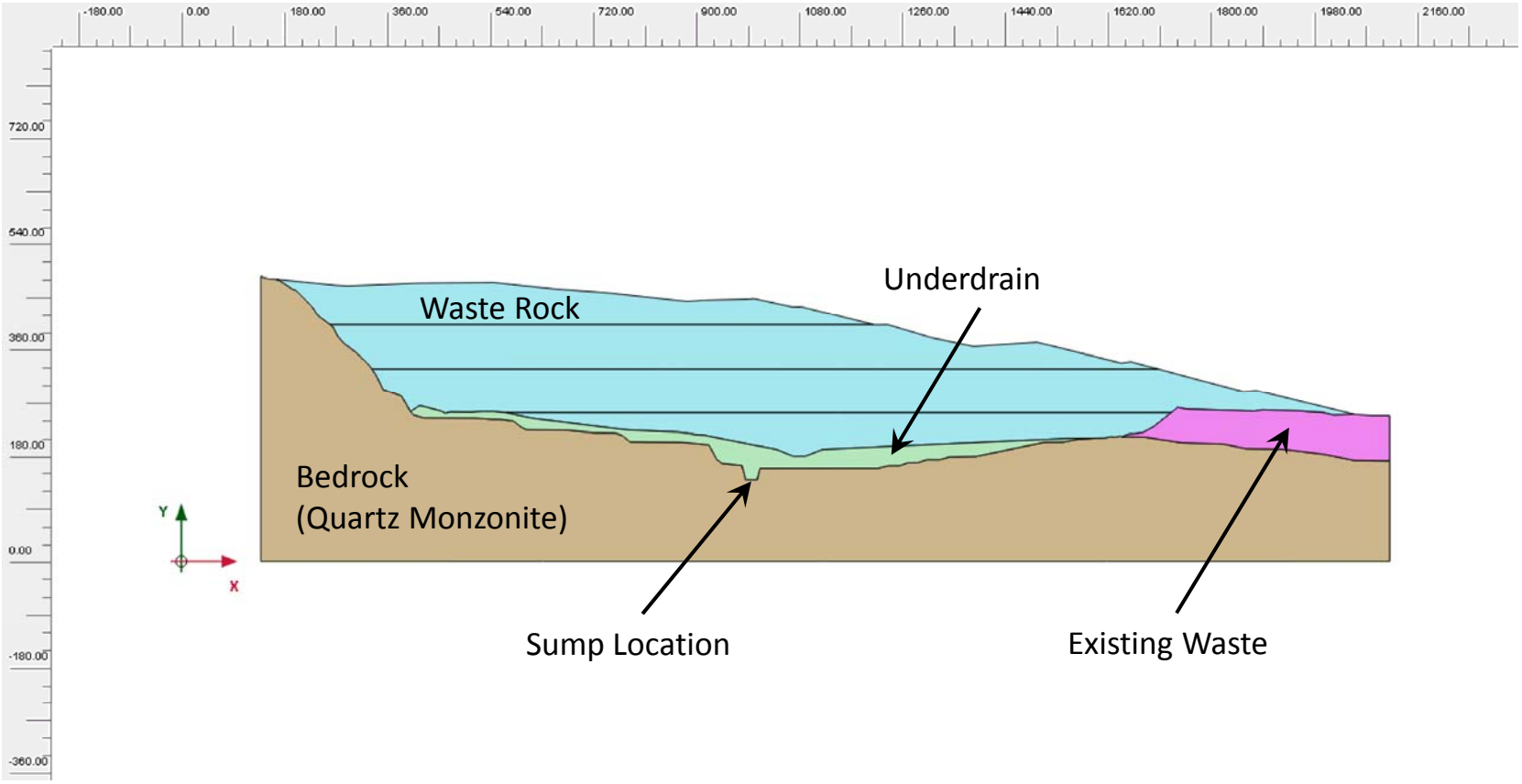
NEWMONT	PROJECT: MIDNITE MINE	MWH	
	TITLE: PIT 3 – SECTION A PLAXIS MODEL GEOMETRY	DATE: July 2014	FIGURE 5
		FILENAME	



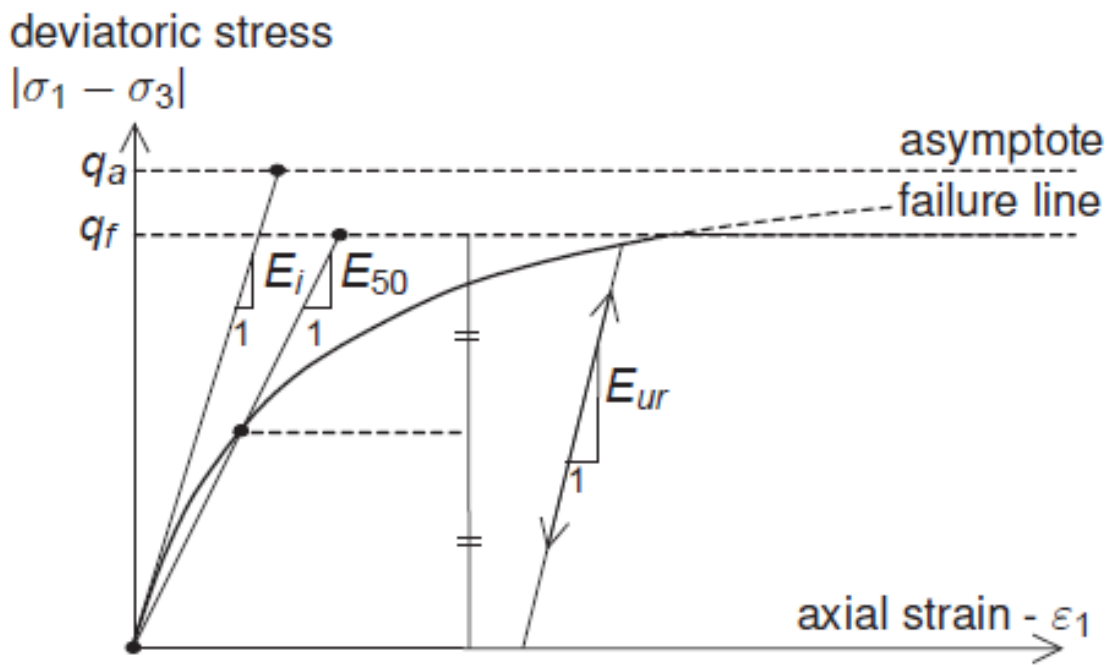
NEWMONT	PROJECT: MIDNITE MINE	MWH	
	TITLE: PIT 3 – SECTION B PLAXIS MODEL GEOMETRY	DATE: July 2014	FIGURE 6
		FILENAME	



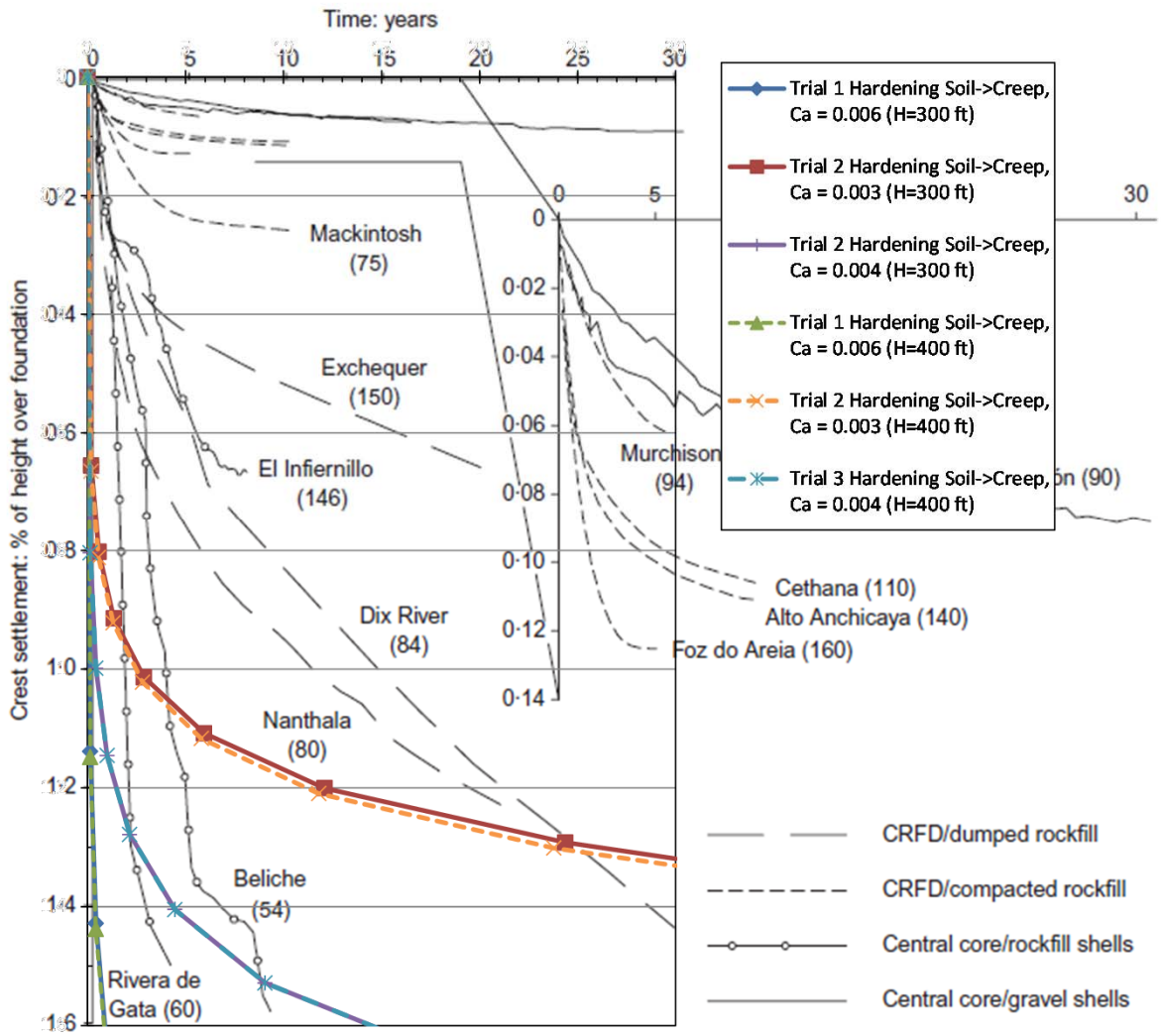
NEWMONT	PROJECT: MIDNITE MINE	MWH	
	TITLE: PIT 4 – SECTION C PLAXIS MODEL GEOMETRY	DATE: July 2014	FIGURE 7
		FILENAME	



NEWMONT	PROJECT: MIDNITE MINE	MWH	
	TITLE: PIT 4 – SECTION D PLAXIS MODEL GEOMETRY	DATE: July 2014	FIGURE 8
		FILENAME	



NEWMONT	PROJECT: MIDNITE MINE	MWH	
	TITLE: HYPERBOLIC STRESS-STRAIN RELATIONSHIP IN PRIMARY LOADING (PLAXIS 2014)	DATE: July 2014	FIGURE 9
		FILENAME	



NEWMONT

PROJECT: MIDNITE MINE

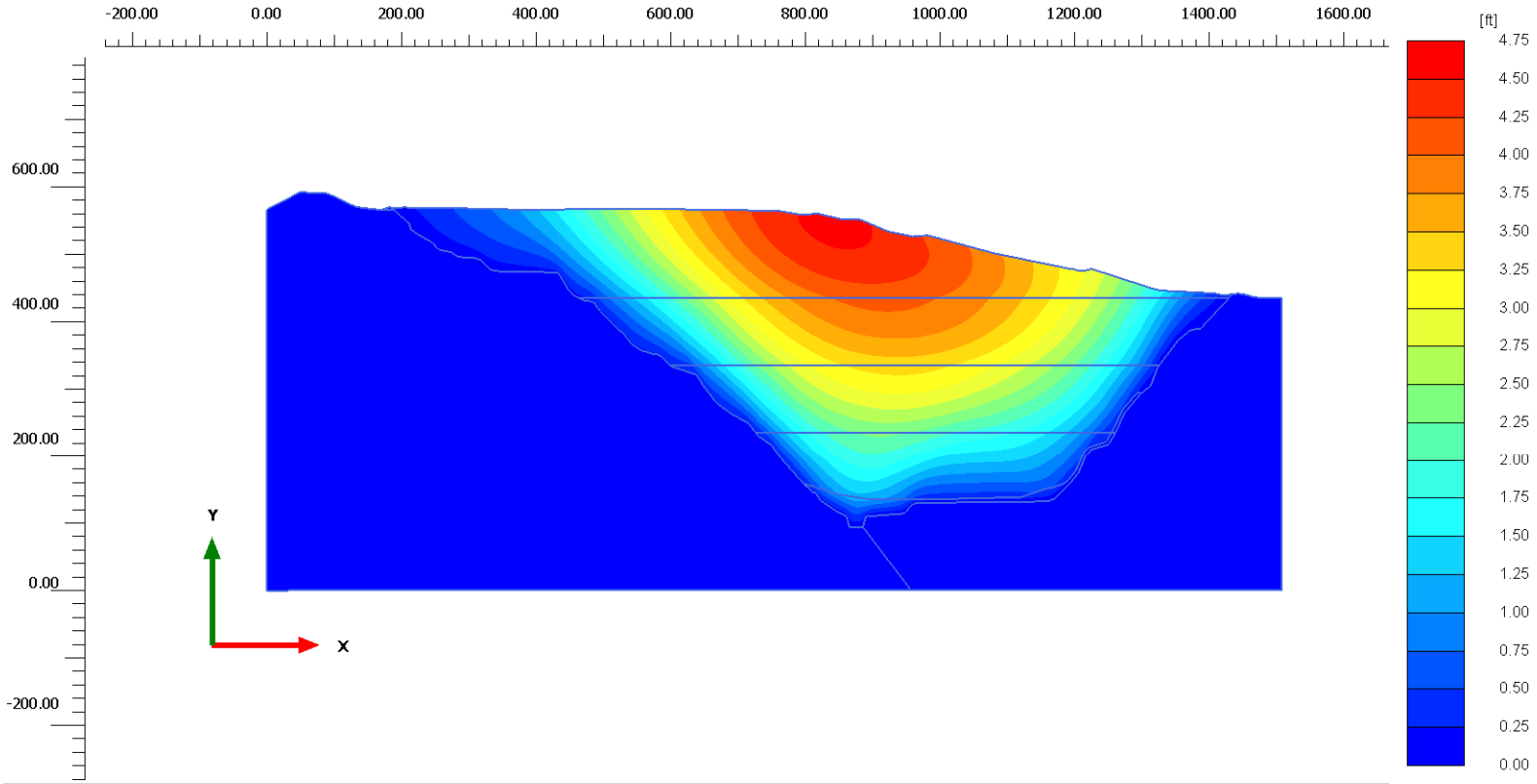
TITLE: 1D CALIBRATION RESULTS (AFTER OLDECOP AND ALONSO 2007)

MWH

DATE: July 2014

FIGURE 10

FILENAME



Total displacements |u|

Maximum value = 4.560 ft (Element 32 at Node 18222)

NEWMONT

PROJECT: MIDNITE MINE

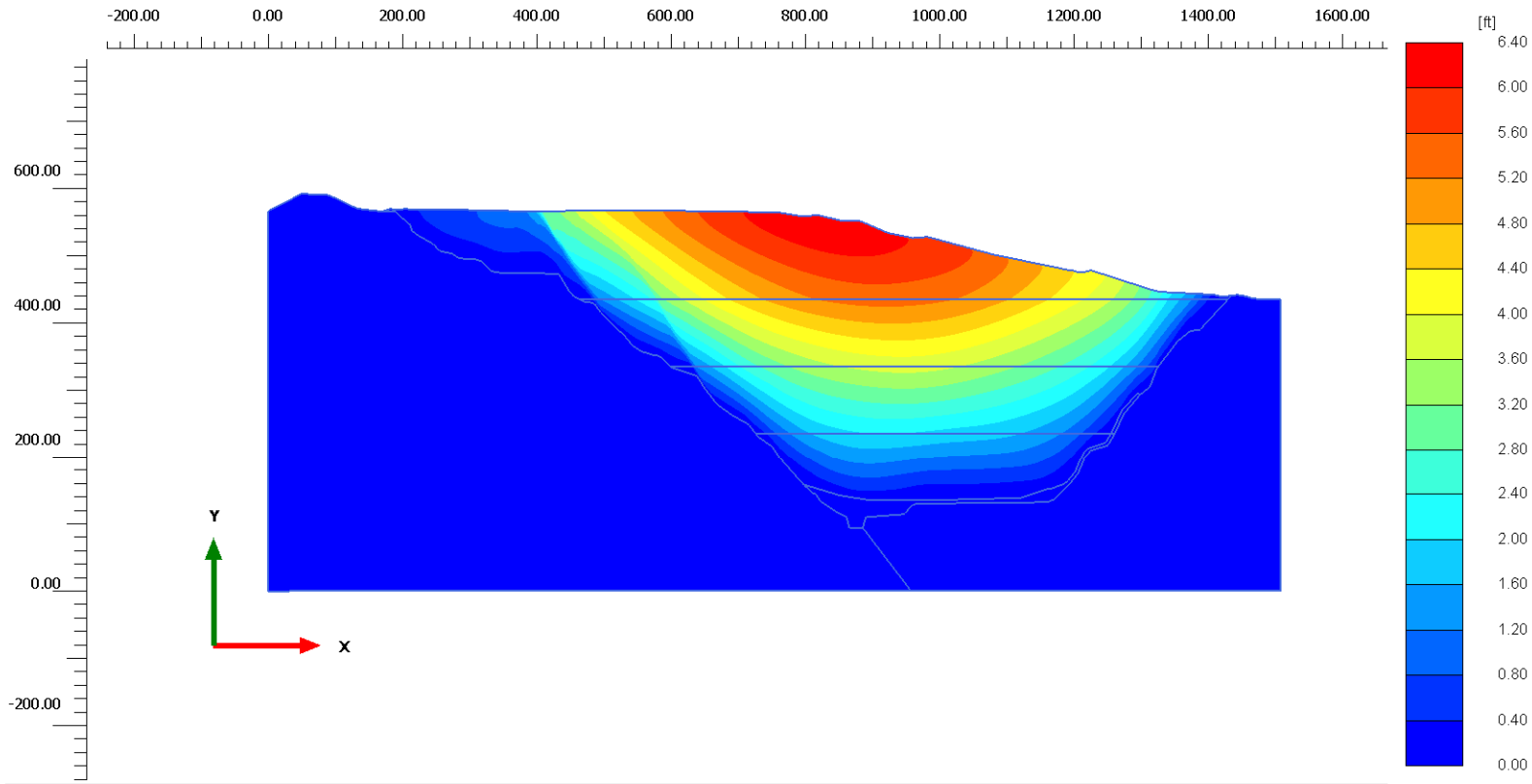
TITLE: PIT 3 – SECTION A
END OF CONSTRUCTION
DISPLACEMENT CONTOURS

MWH

DATE: July 2014

FIGURE 11

FILENAME



Total displacements |u|

Maximum value = 6.350 ft (Element 127 at Node 18855)

NEWMONT

PROJECT: MIDNITE MINE

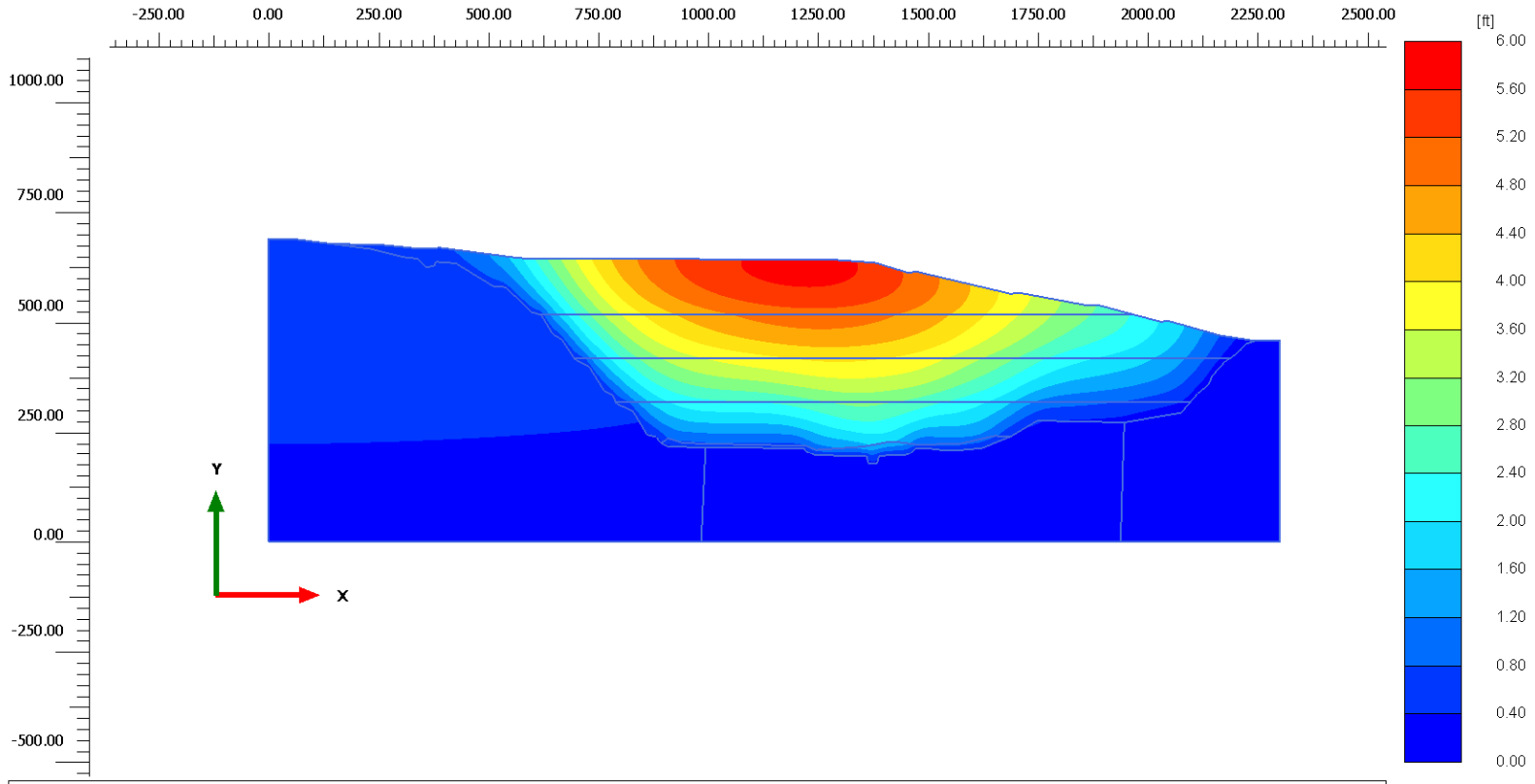
TITLE: PIT 3 – SECTION A
POST-CONSTRUCTION (CREEP)
DISPLACEMENT CONTOURS

MWH

DATE: July 2014

FIGURE 12

FILENAME



Total displacements |u|

Maximum value = 5.737 ft (Element 431 at Node 8735)

NEWMONT

PROJECT: MIDNITE MINE

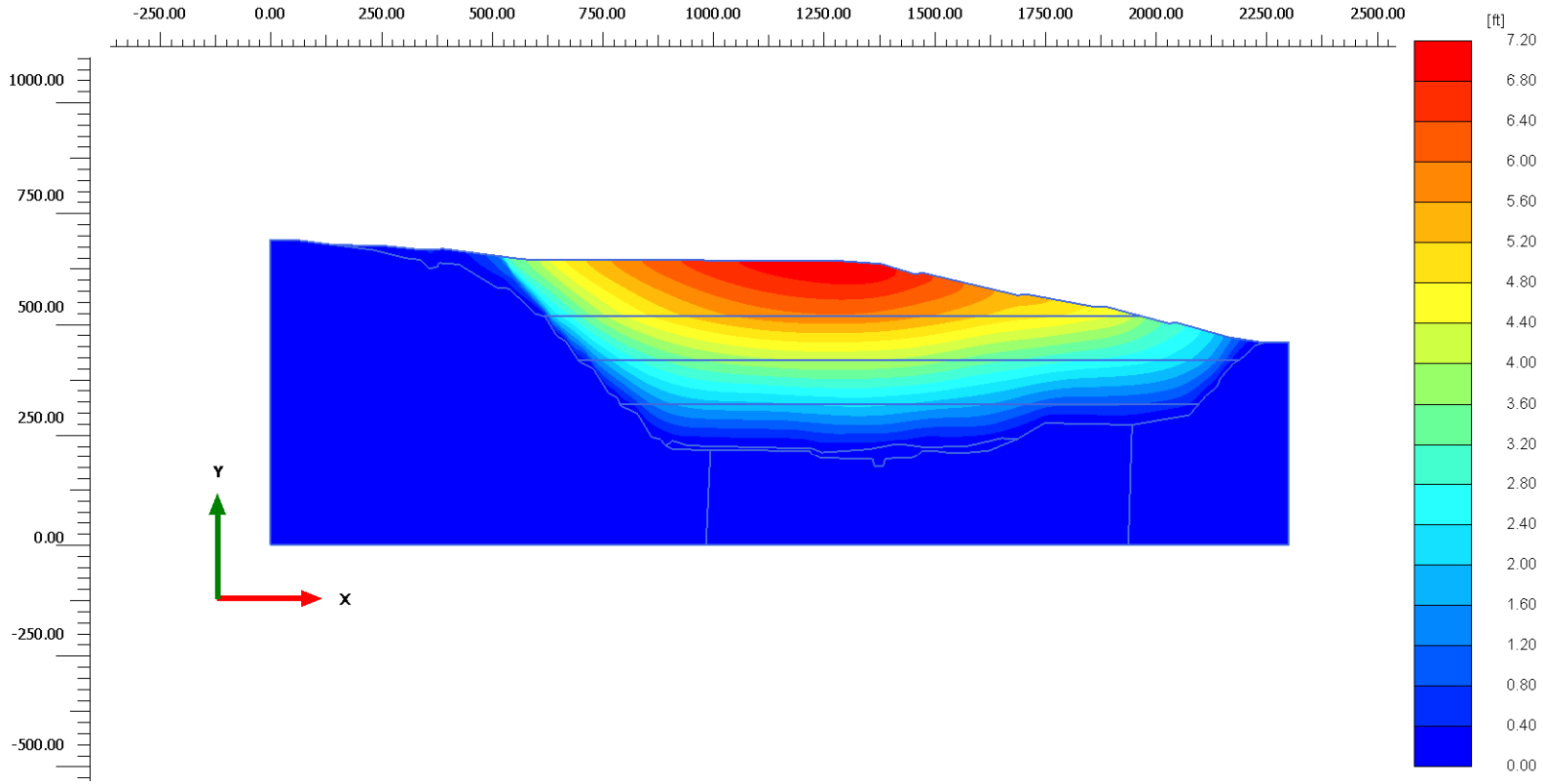
TITLE: PIT 3 – SECTION B
END OF CONSTRUCTION
DISPLACEMENT CONTOURS

MWH

DATE: July 2014

FIGURE 13

FILENAME



Total displacements |u|

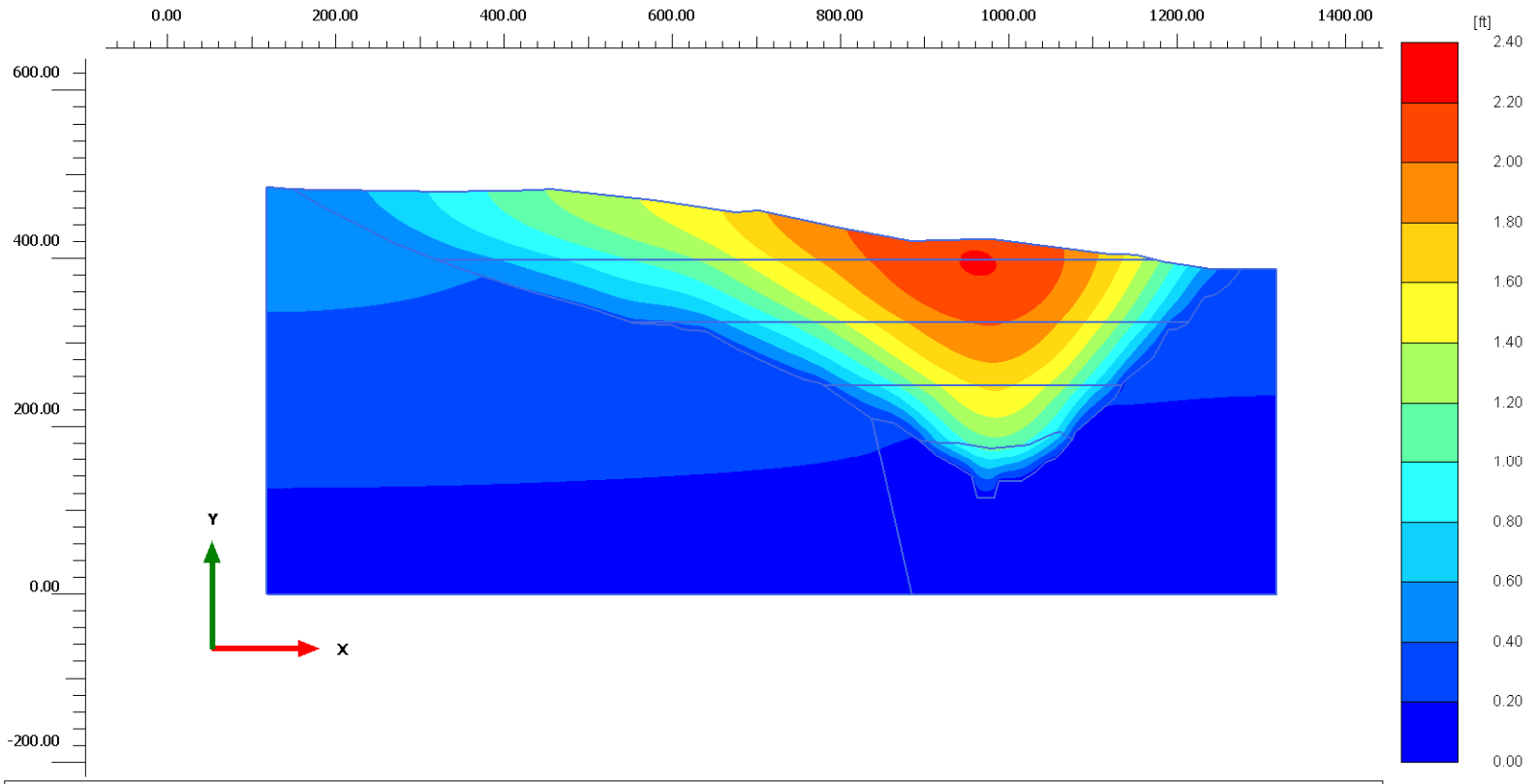
Maximum value = 7.166 ft (Element 455 at Node 10262)

NEWMONT

PROJECT	MIDNITE MINE
TITLE	PIT 3 – SECTION B POST-CONSTRUCTION (CREEP) DISPLACEMENT CONTOURS
DATE	July 2014
FILENAME	

MWH

FIGURE 14



Total displacements |u|

Maximum value = 2.209 ft (Element 557 at Node 11316)

NEWMONT

PROJECT: MIDNITE MINE

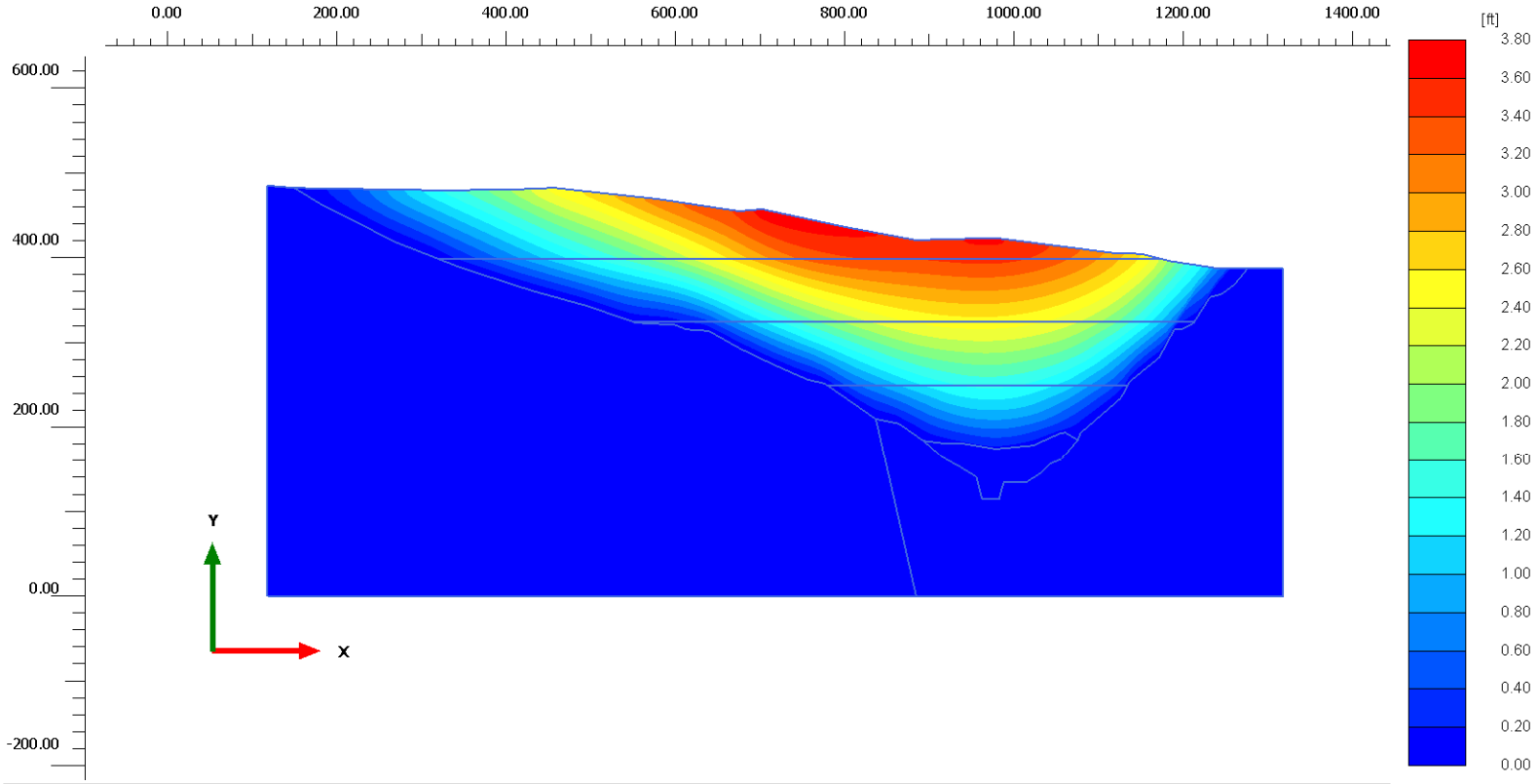
TITLE: PIT 4 – SECTION C
END OF CONSTRUCTION
DISPLACEMENT CONTOURS

MWH

DATE: July 2014

FIGURE 15

FILENAME



Total displacements |u|

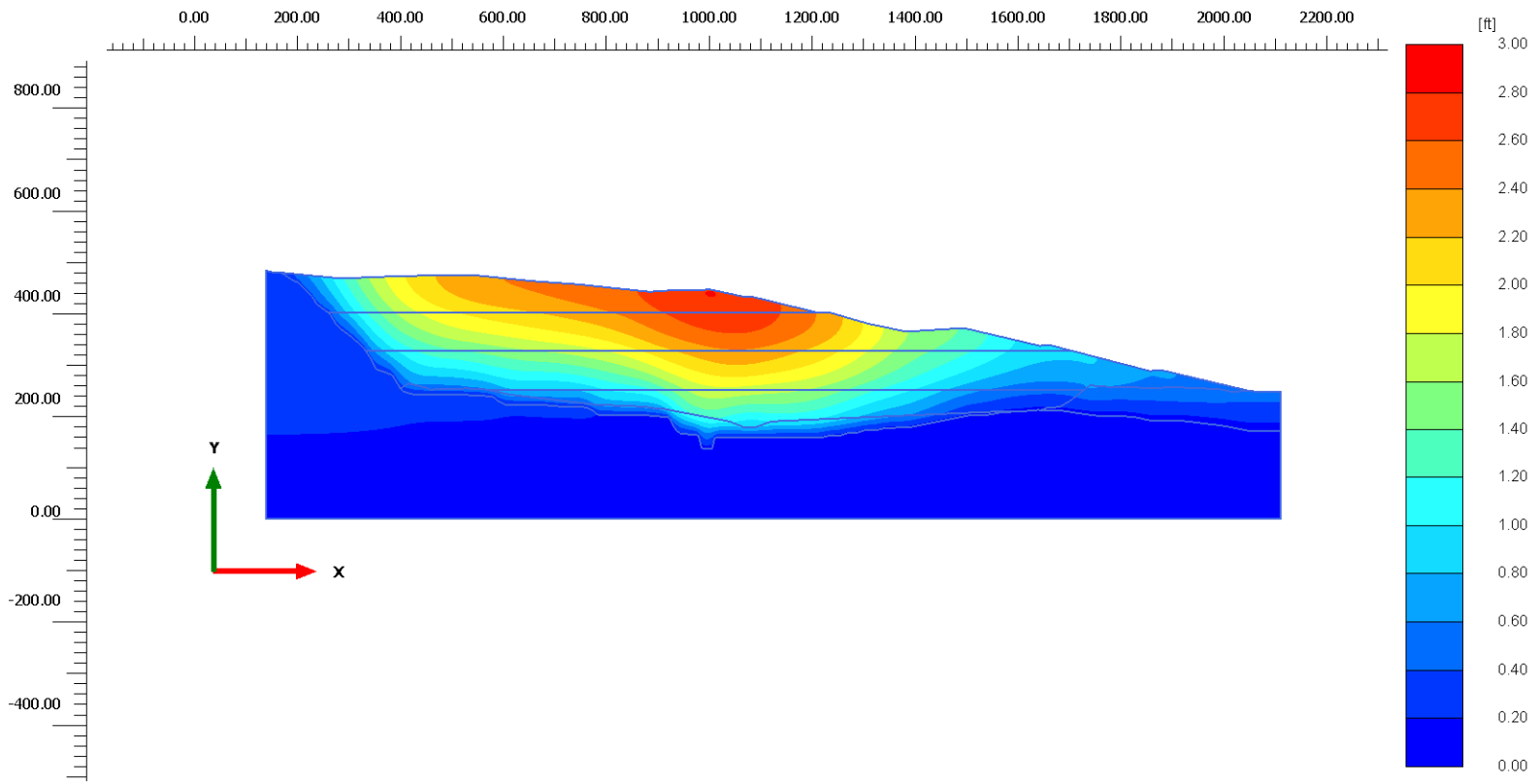
Maximum value = 3.680 ft (Element 15 at Node 5969)

NEWMONT

PROJECT	MIDNITE MINE
TITLE	PIT 4 – SECTION C POST-CONSTRUCTION (CREEP) DISPLACEMENT CONTOURS
DATE	July 2014
FILENAME	

MWH

FIGURE 16



Total displacements |u|

Maximum value = 2.802 ft (Element 12 at Node 21673)

NEWMONT

PROJECT: MIDNITE MINE

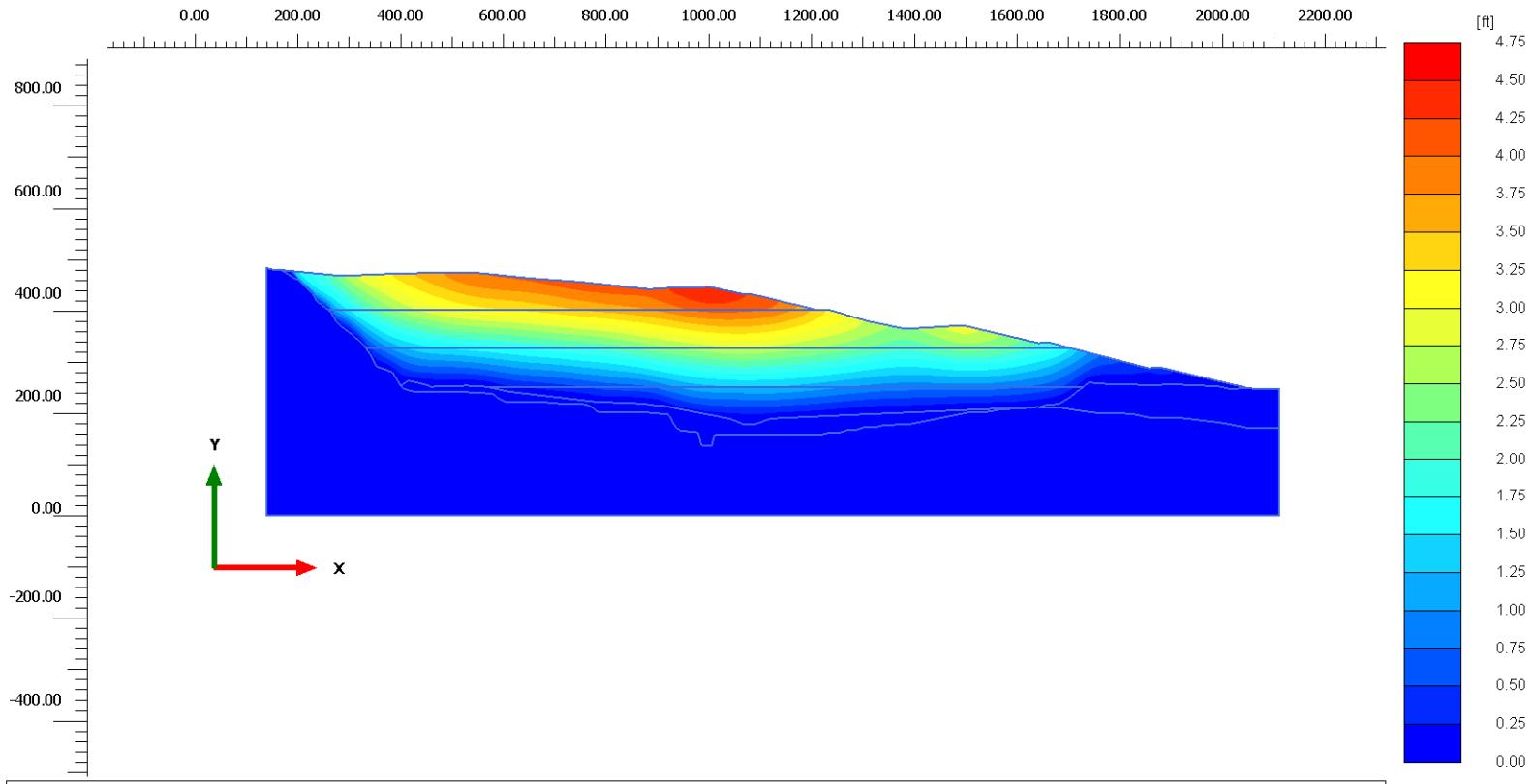
TITLE: PIT 4 – SECTION D
END OF CONSTRUCTION
DISPLACEMENT CONTOURS

MWH

DATE: July 2014

FIGURE 17

FILENAME



Total displacements |u|

Maximum value = 4.497 ft (Element 12 at Node 21697)

NEWMONT

PROJECT: MIDNITE MINE

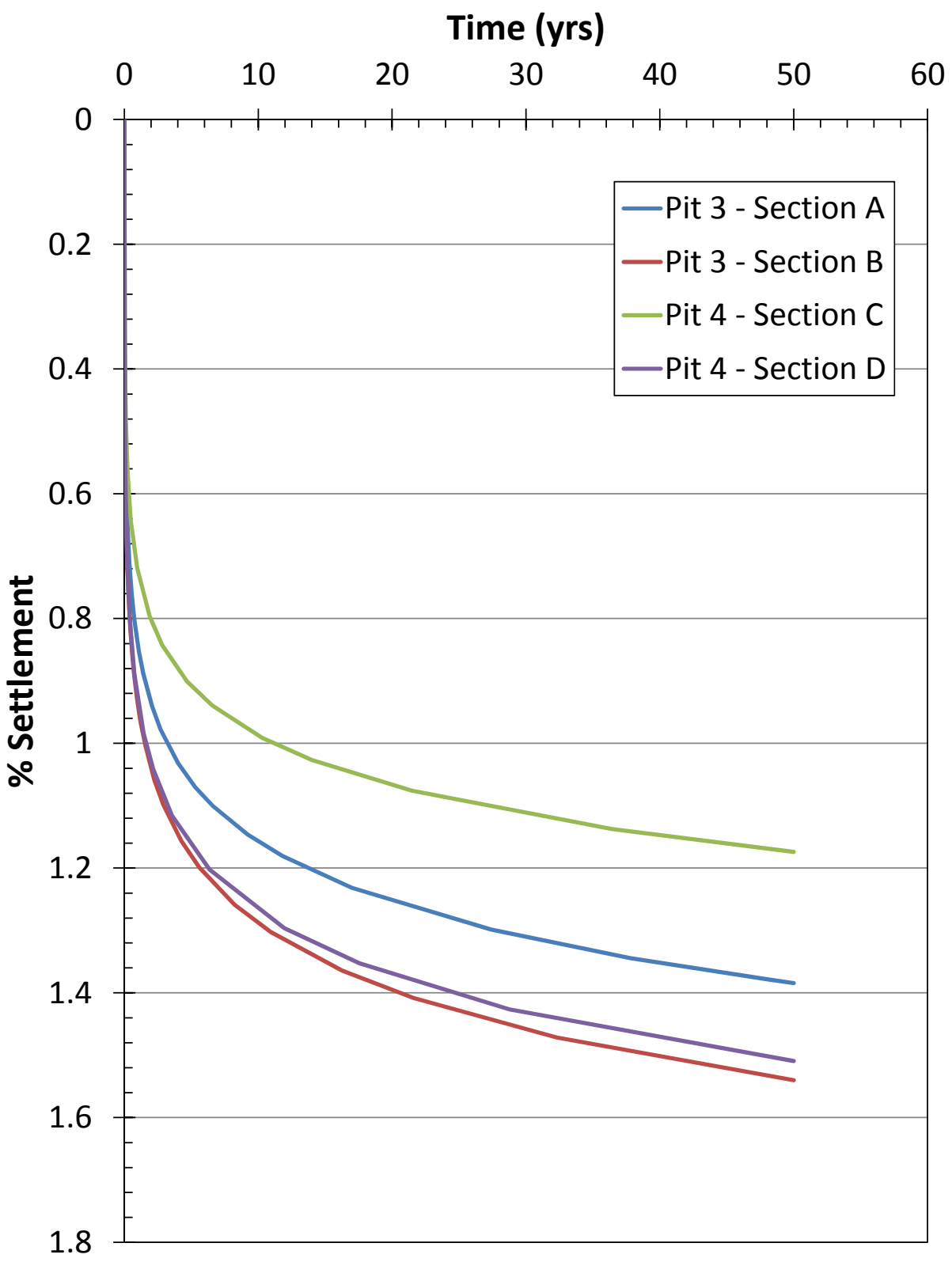
TITLE: PIT 4 – SECTION D
POST-CONSTRUCTION (CREEP)
DISPLACEMENT CONTOURS

MWH

DATE: July 2014

FIGURE 18

FILENAME



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NEWMONT

PROJECT
MIDNITE MINE

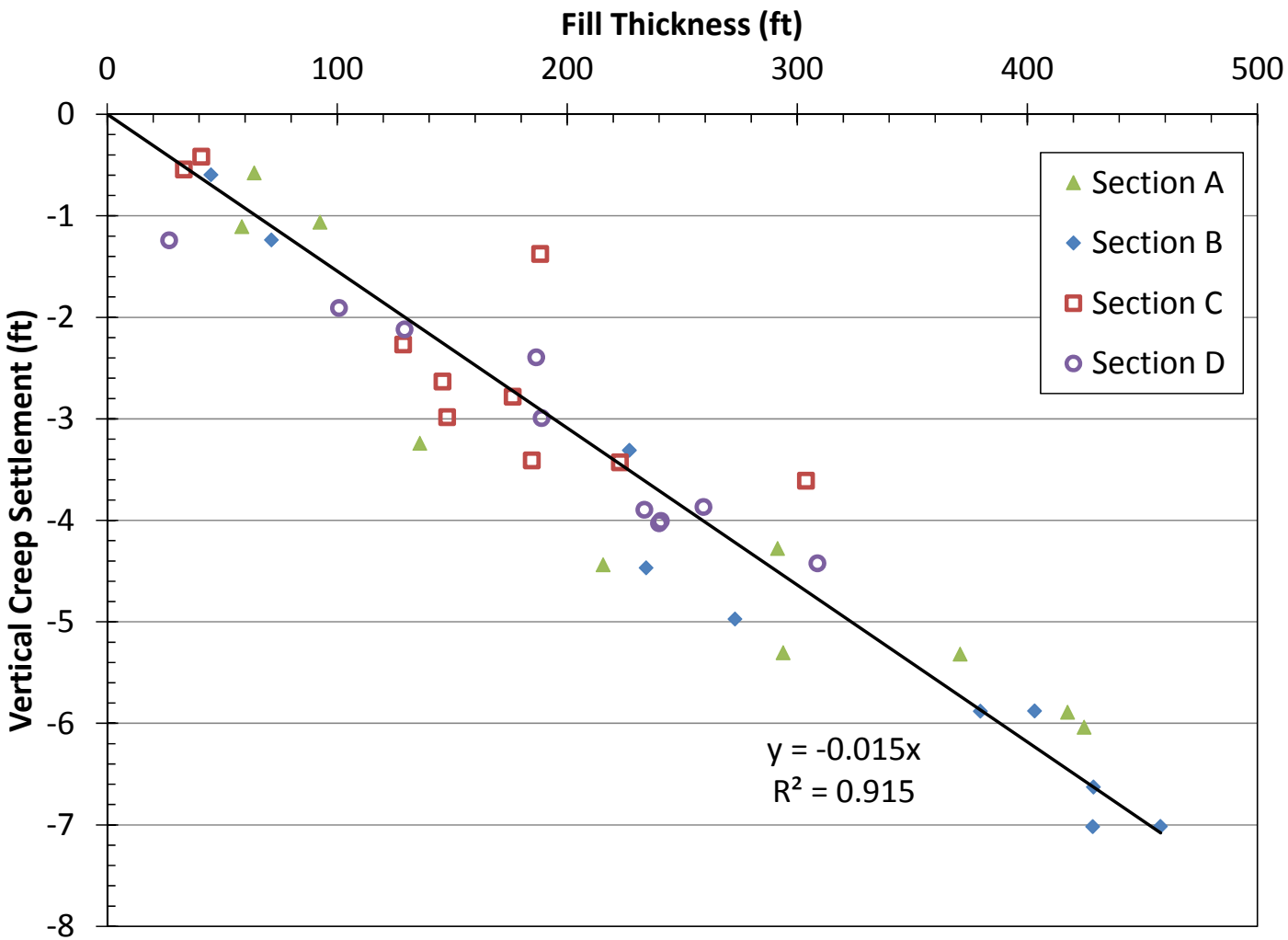
TITLE
CREEP SETTLEMENT OVER 50 YEARS

MWH

DATE
July 2014

FIGURE 19

FILENAME



NEWMONT

PROJECT: MIDNITE MINE

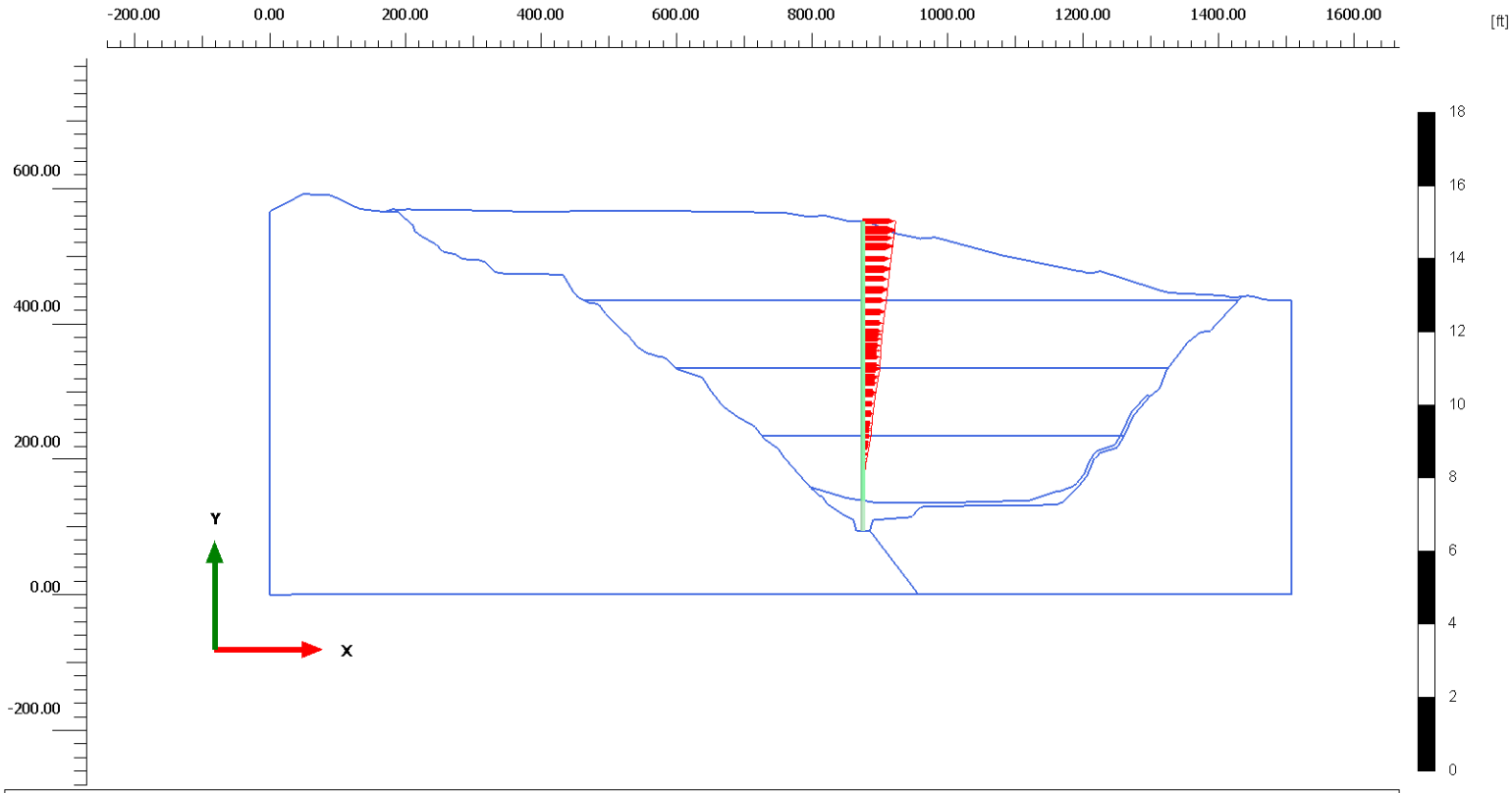
TITLE: FILL THICKNESS VS VERTICAL CREEP SETTELMENT

MWH

DATE: July 2014

FIGURE 20

FILENAME



Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.9634 ft

Minimum value = -0.04839 ft

NEWMONT

PROJECT: MIDNITE MINE

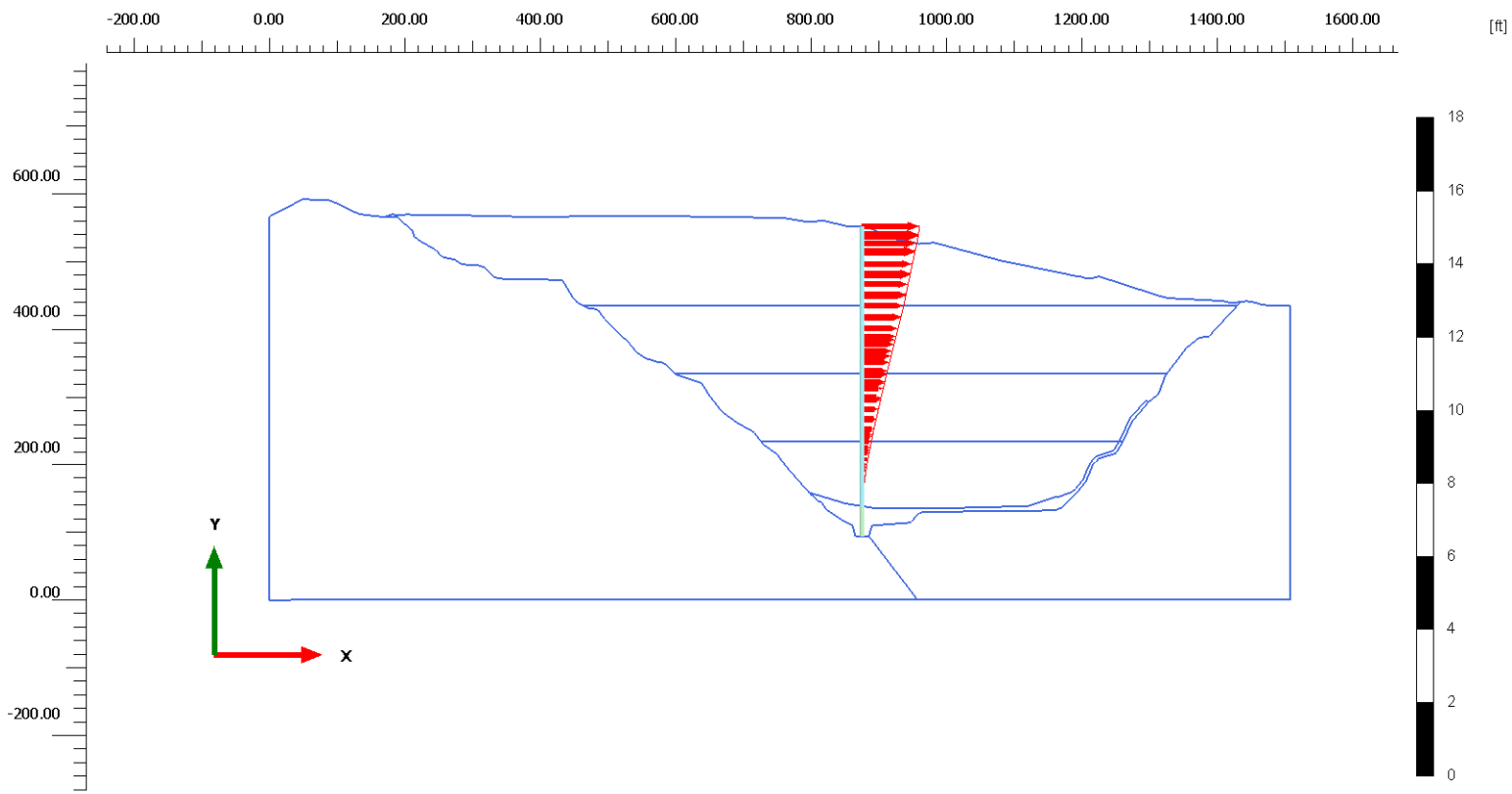
TITLE: PIT 3 – SECTION A
END OF CONSTRUCTION
LATERAL DISPLACEMENTS AT SUMP RISERS

MWH

DATE: July 2014

FIGURE 21

FILENAME



Total displacements u_x (scaled up 50.0 times)

Maximum value = 1.702 ft

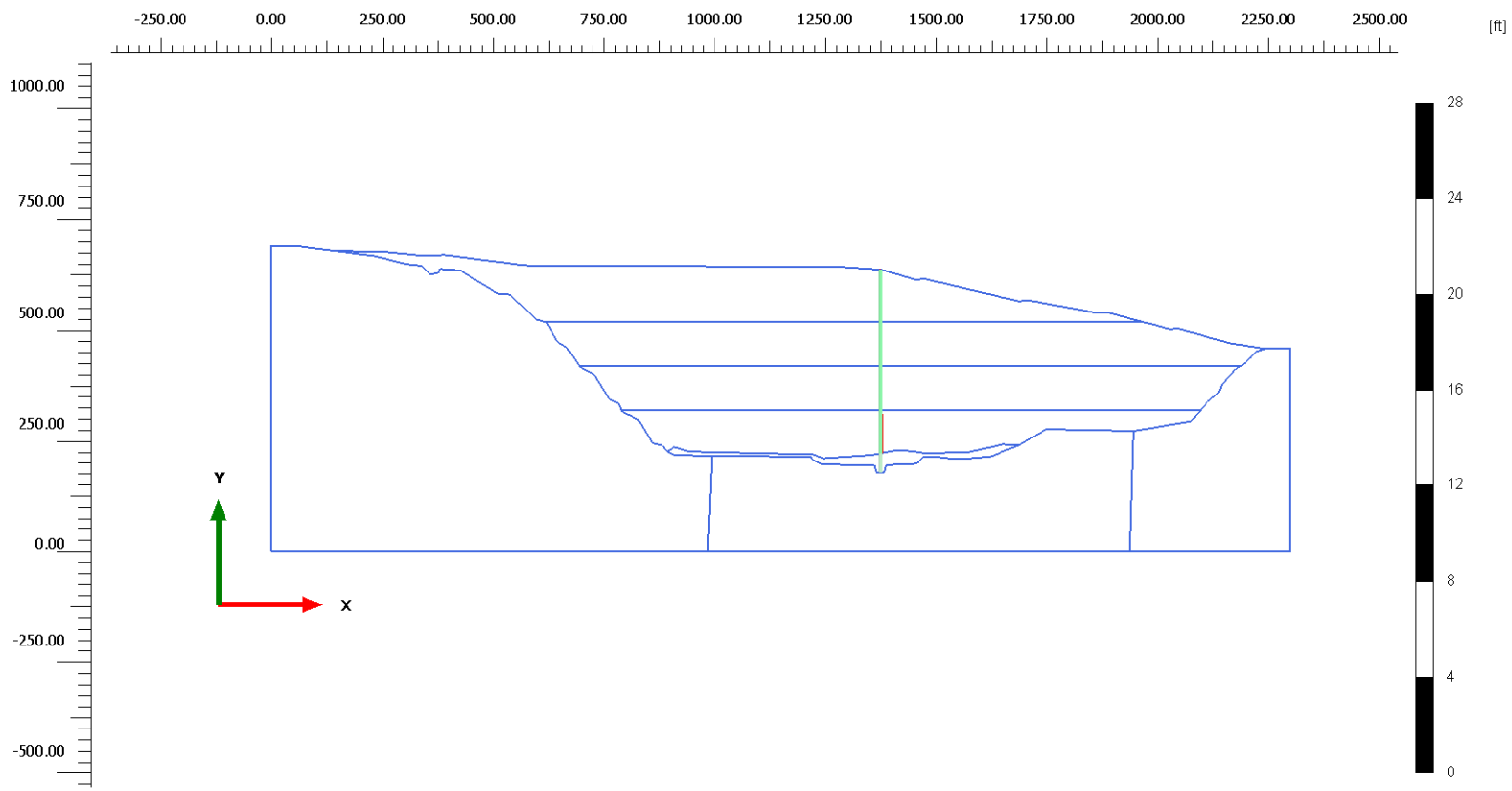
Minimum value = -0.01017 ft

NEWMONT

PROJECT	MIDNITE MINE
TITLE	PIT 3 – SECTION A POST-CONSTRUCTION (CREEP) LATERAL DISPLACEMENTS AT SUMP RISERS
DATE	July 2014
FILENAME	

MWH

FIGURE 22



Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.1294 ft

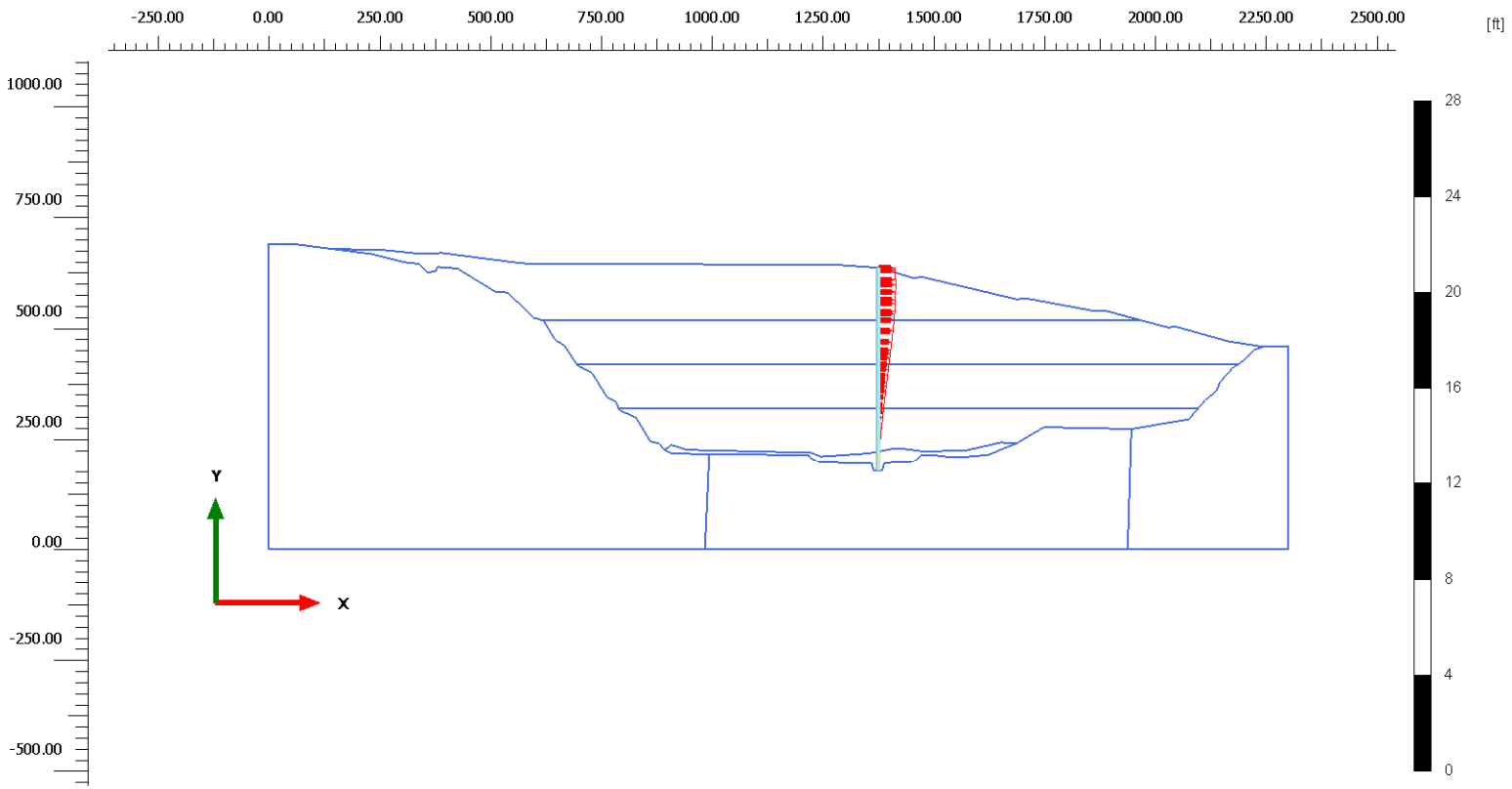
Minimum value = -0.08054 ft

NEWMONT

PROJECT	MIDNITE MINE
TITLE	PIT 3 – SECTION B END OF CONSTRUCTION LATERAL DISPLACEMENTS AT SUMP RISERS
DATE	July 2014
FILENAME	

MWH

FIGURE 23



Total displacements u_x (scaled up 50.0 times)

Maximum value = 0.7754 ft

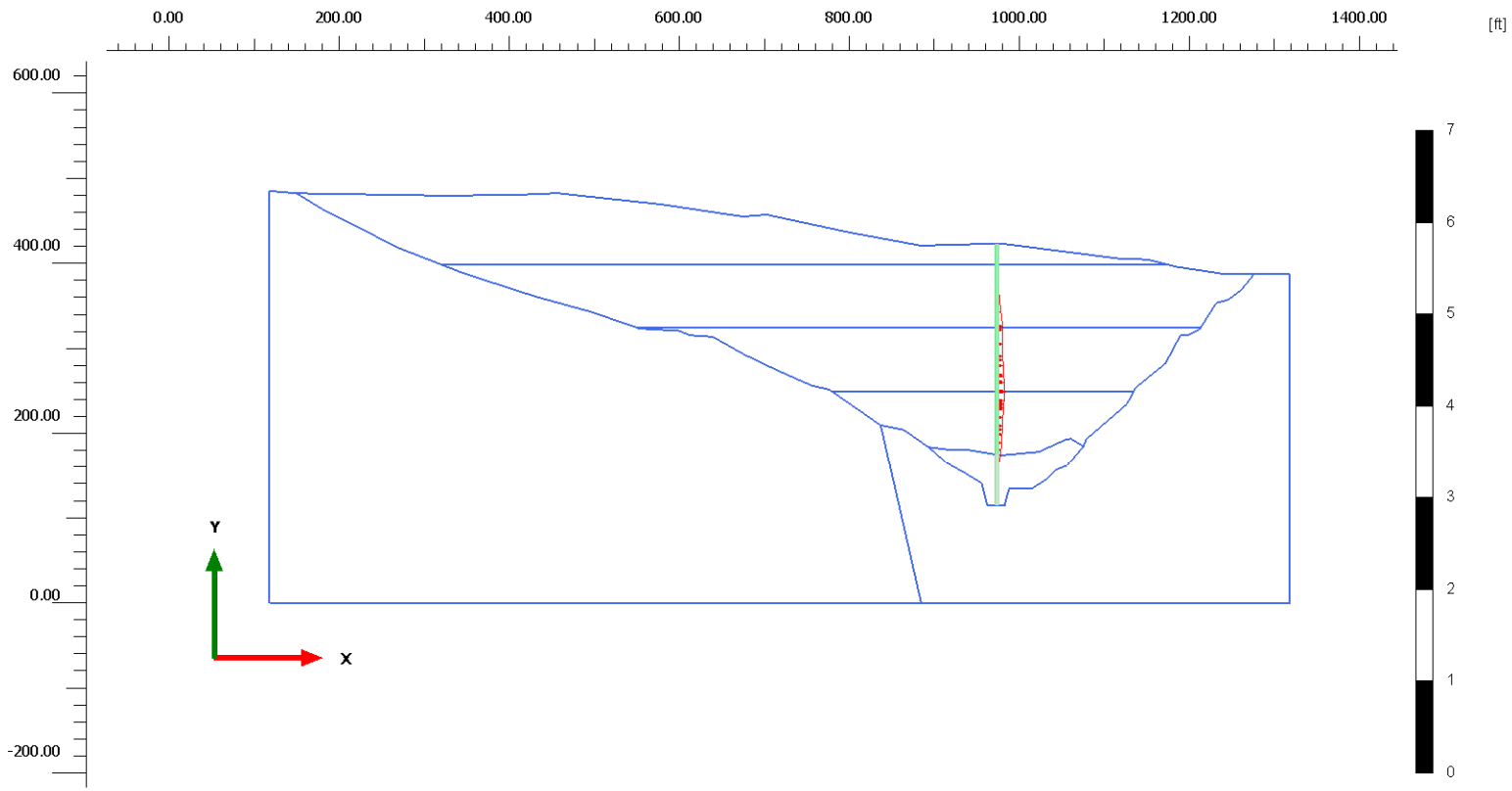
Minimum value = $3.208*10^{-3}$ ft

NEWMONT

PROJECT	MIDNITE MINE
TITLE	PIT 3 – SECTION B POST-CONSTRUCTION (CREEP) LATERAL DISPLACEMENTS AT SUMP RISERS
DATE	July 2014
FILENAME	

MWH

FIGURE 24



Total displacements u_x (scaled up 100 times)

Maximum value = 0.1005 ft

Minimum value = -0.01389 ft

NEWMONT

PROJECT: MIDNITE MINE

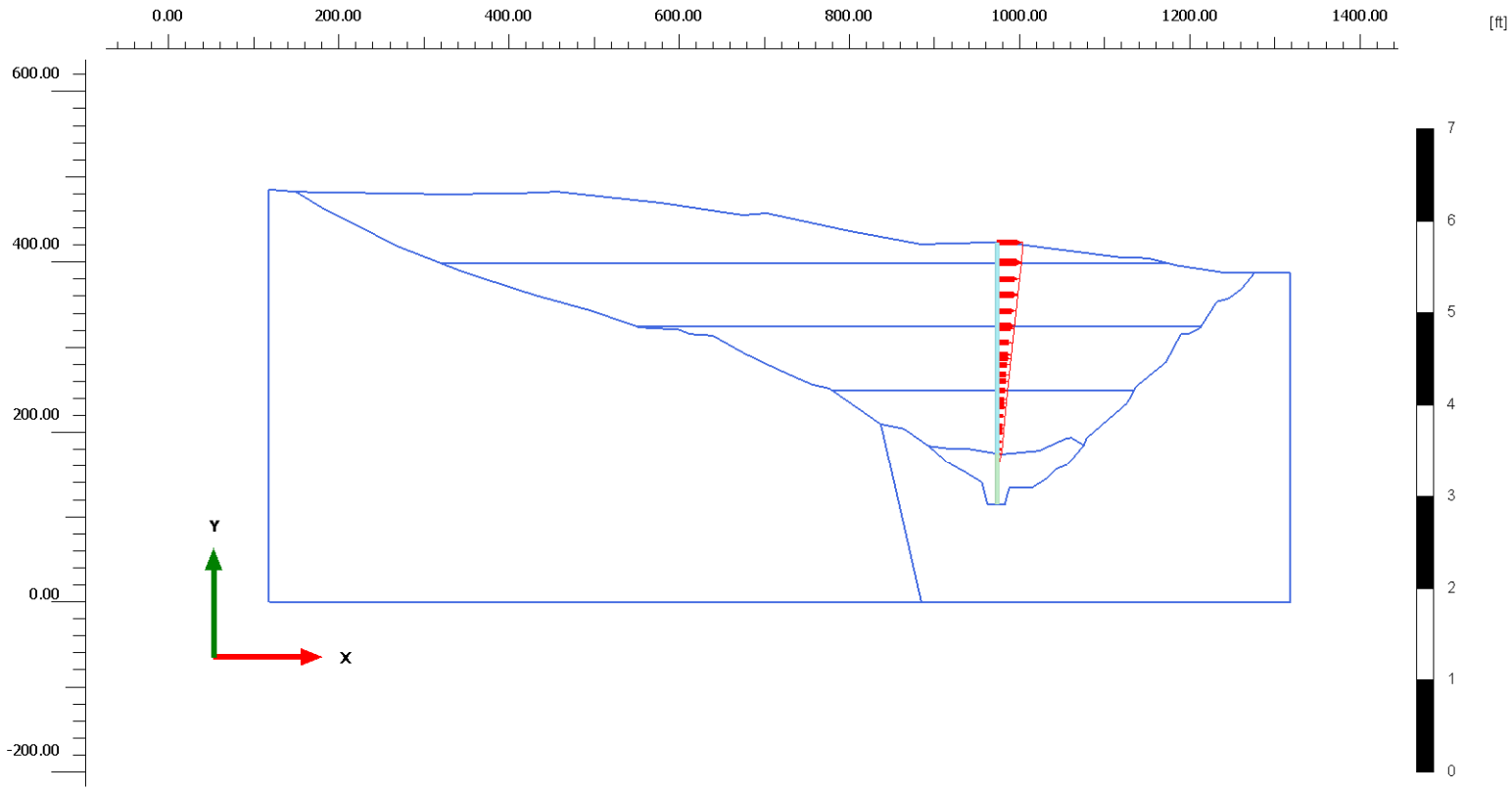
TITLE: PIT 4 – SECTION C
END OF CONSTRUCTION
LATERAL DISPLACEMENTS AT SUMP RISERS

MWH

DATE: July 2014

FIGURE 25

FILENAME

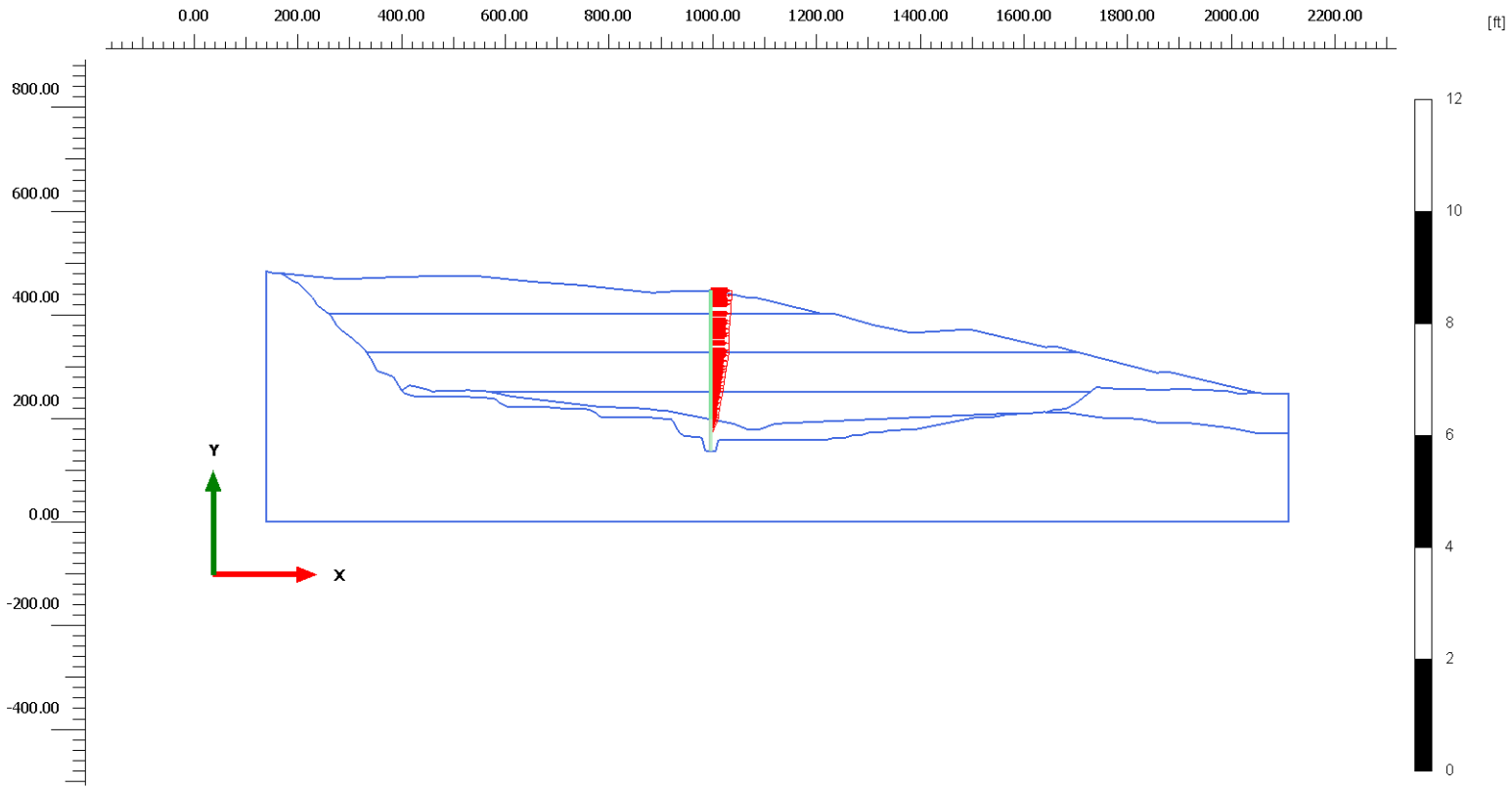


Total displacements u_x (scaled up 100 times)

Maximum value = 0.3075 ft

Minimum value = $-0.3257*10^{-3}$ ft

NEWMONT	PROJECT: MIDNITE MINE	MWH	
	TITLE: PIT 4 – SECTION C POST-CONSTRUCTION (CREEP) LATERAL DISPLACEMENTS AT SUMP RISERS	DATE: July 2014	FIGURE 26
		FILENAME	



Total displacements u_x (scaled up 100 times)

Maximum value = 0.4128 ft

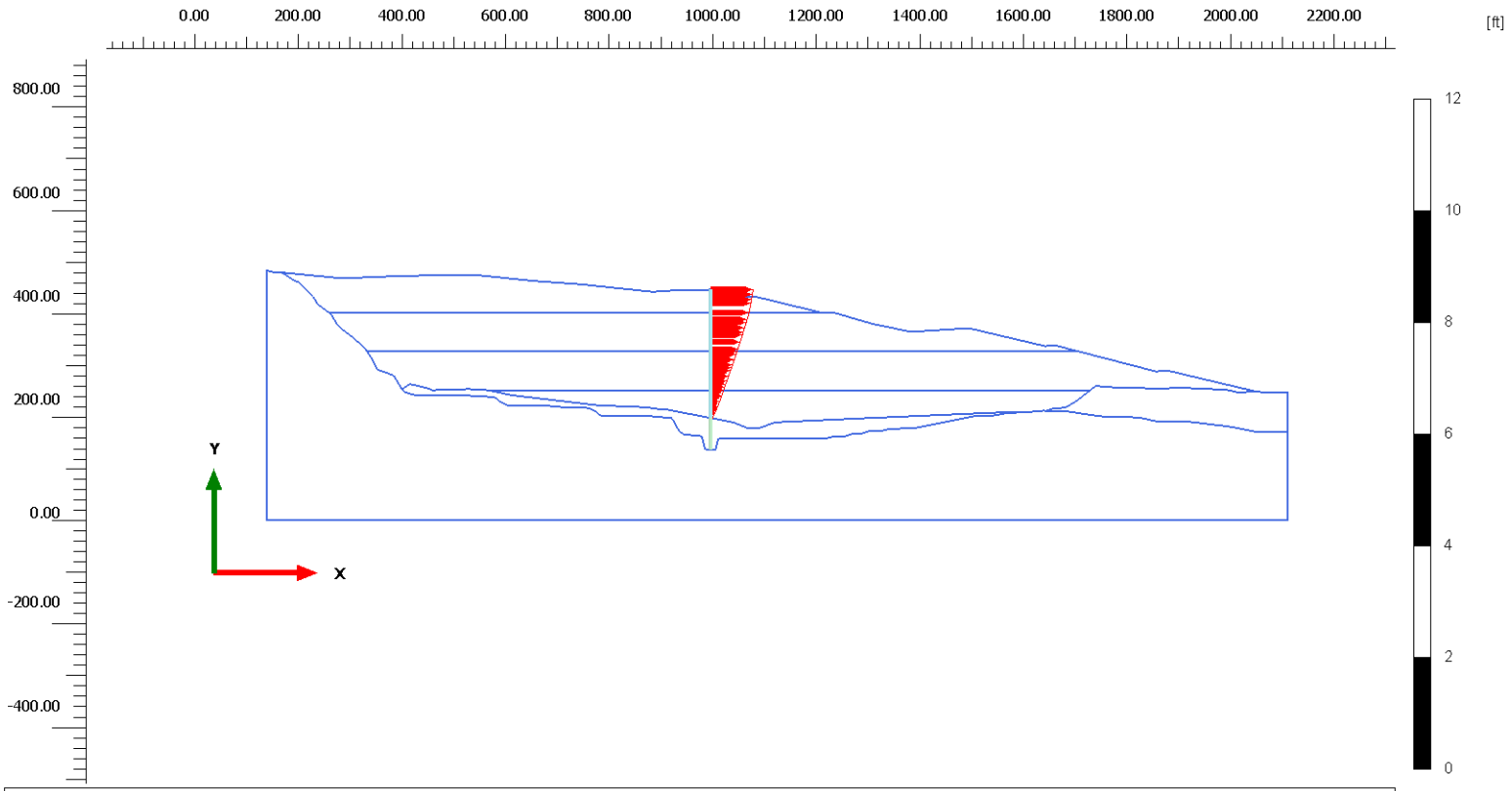
Minimum value = $0.3965*10^{-3}$ ft

NEWMONT

PROJECT	MIDNITE MINE
TITLE	PIT 4 – SECTION D END OF CONSTRUCTION LATERAL DISPLACEMENTS AT SUMP RISERS
DATE	July 2014
FILENAME	

MWH

FIGURE 27



Total displacements u_x (scaled up 100 times)

Maximum value = 0.8244 ft

Minimum value = $0.5470*10^{-3}$ ft

NEWMONT

PROJECT: MIDNITE MINE

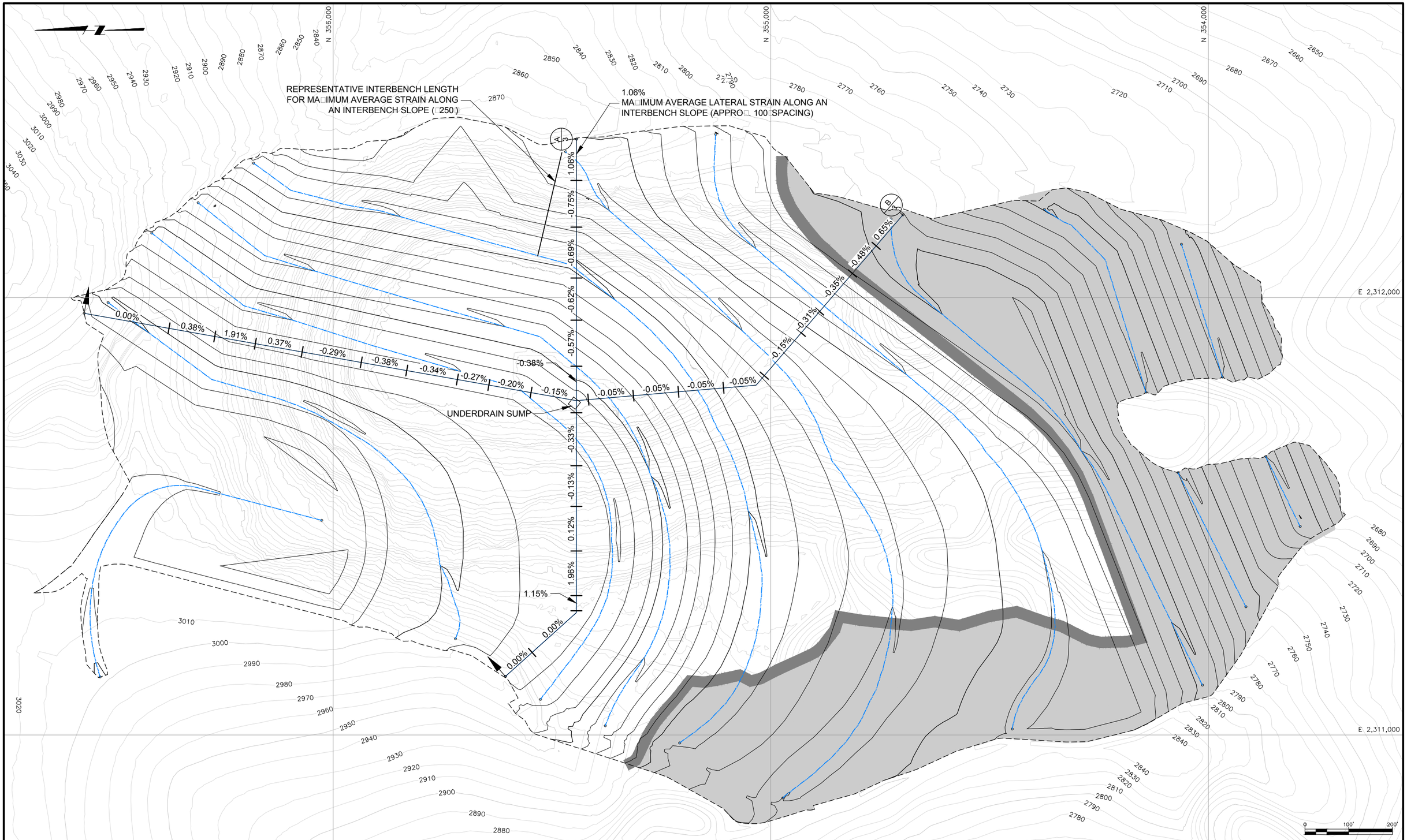
TITLE: PIT 4 – SECTION D
POST-CONSTRUCTION (CREEP)
LATERAL DISPLACEMENTS AT SUMP RISERS

MWH

DATE: July 2014

FIGURE 28

FILENAME



LEGEND:

5605	EXISTING GROUND SURFACE ELEVATION, FEET
5605	FINISHED GROUND SURFACE ELEVATION, FEET
	PHASE 2 ANCHOR TRENCH
	EXTENT OF PHASE 2 COVER (REMAINING AREA OF PIT 3 WILL BE COVERED IN PHASE 3)
	DRAIANGE CHANNEL FLOWLINES

NEWMONT

MWH

PROJECT LOCATION: WELLPINIT, WASHINGTON

PROJECT: MIDNITE MINE

TITLE: PIT 3 COVER AVERAGE LATERAL STRAIN (APPRO□. 100□SPACING)

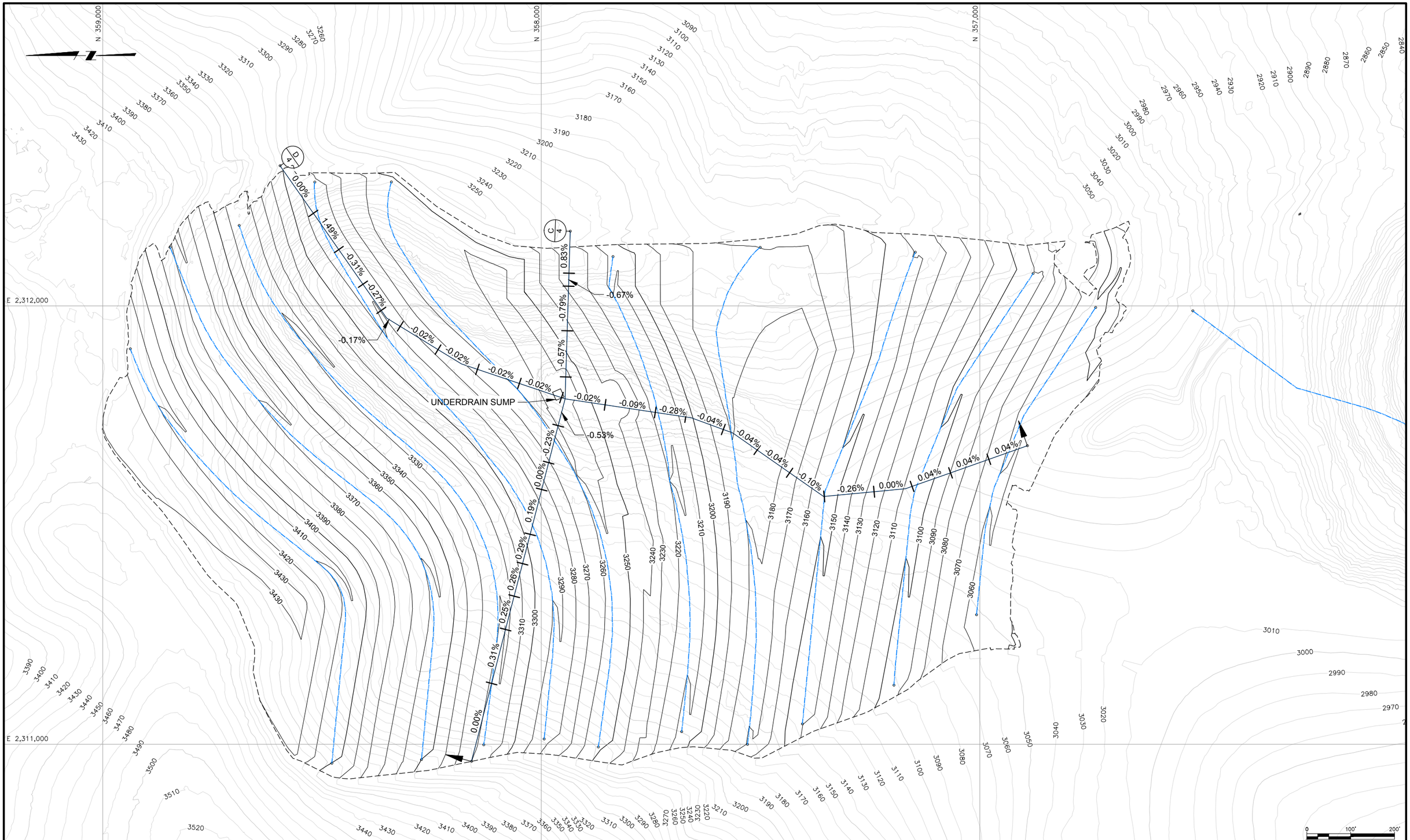
FIGURE 29

REVISION: 0

FILE NAME: 1011322 □SLOC

DATE: JUNE 2015

L:\Design-Drafting\Clients-I-P\NEWMONT\1011322 MIDNITE MINE\013-Sheet Set\FIGURES\2014-07-31 SETTLEMENT ANALYSIS\1011322 XSLOC



LEGEND:

5605 — EXISTING GROUND SURFACE ELEVATION, FEET

5605 — FINISHED GROUND SURFACE ELEVATION, FEET

— · · — DRAINAGE CHANNEL FLOWLINES

NEWMONT

MWH

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER AVERAGE LATERAL STRAIN (APPRO□. 100□SPACING)
FIGURE 30	REVISION 0
FILE NAME 1011322 □SLOC	DATE JUNE 2015

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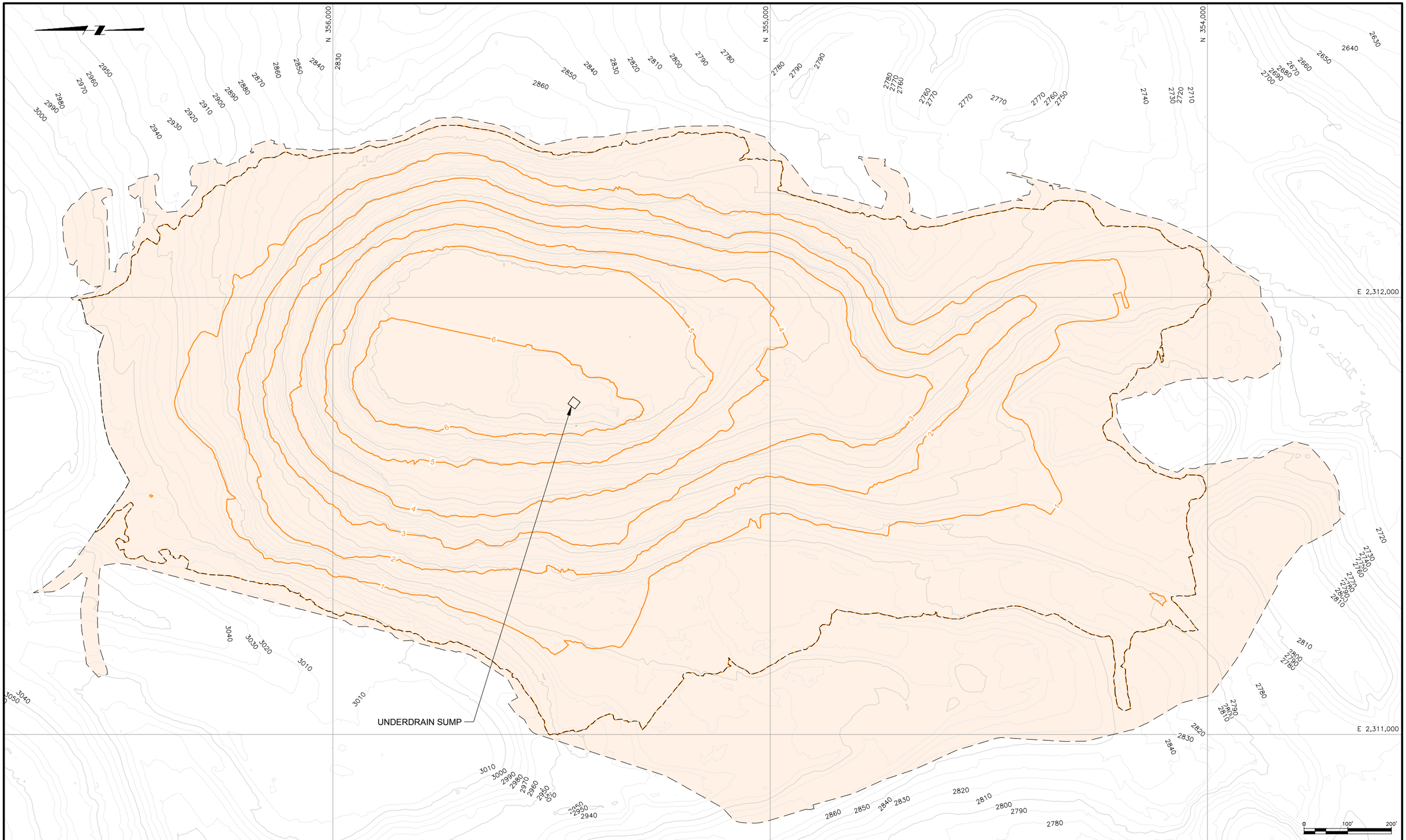


LEGEND:

3490 — EXISTING GROUND SURFACE ELEVATION, FEET

4 — CREEP SETTLEMENT CONTOUR, FEET

— — — — APPROXIMATE EXTENT OF COVER

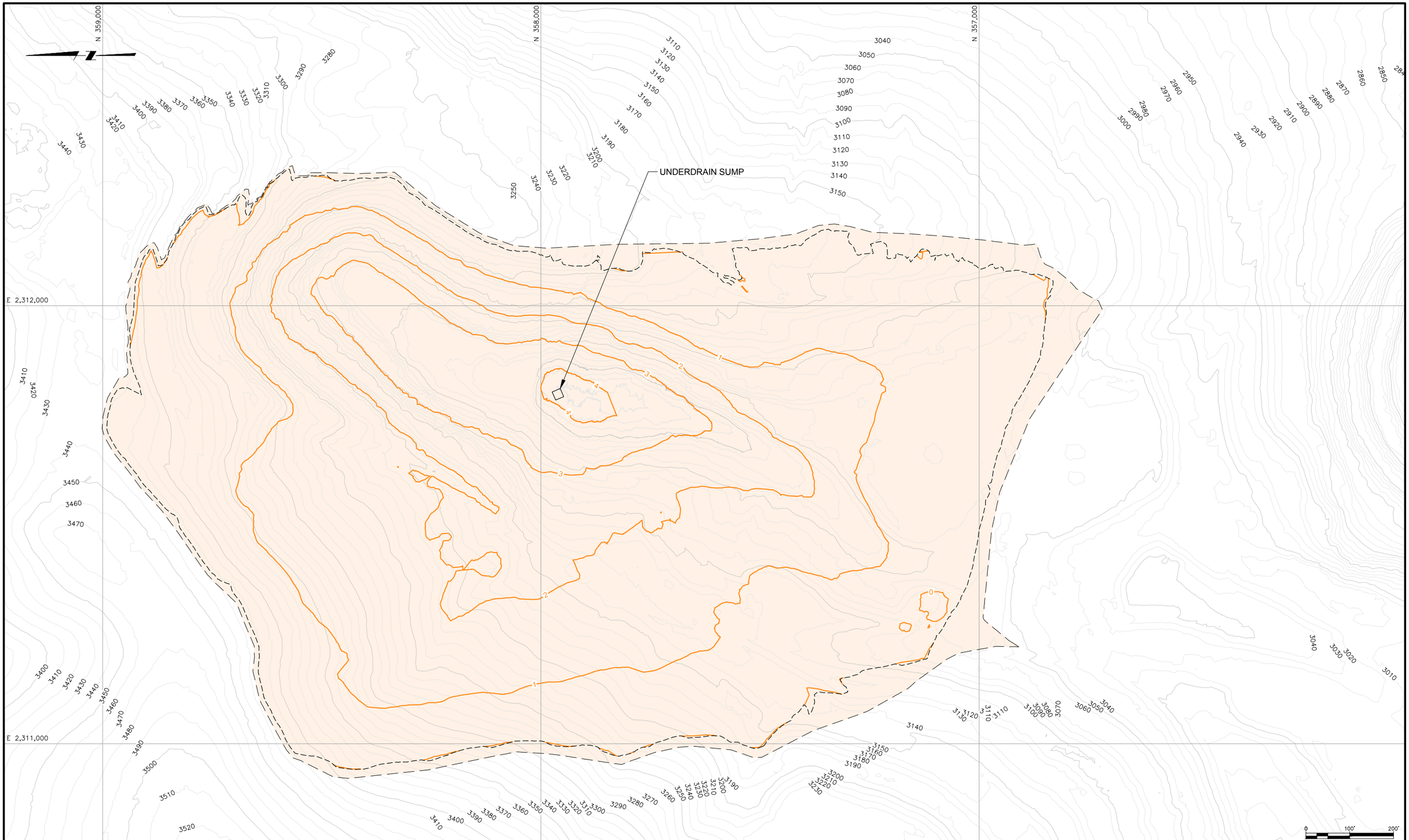
— — — — — — EDGE OF FILL

NEWMONT

MWH

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS CREEP SETTLEMENT CONTOURS
FIGURE 31	REVISION 1
FILE NAME PLAN VIEW	DATE JUNE 2015

L:\Design-Drafting\Clients-I-P\NEWMONT\101322 MIDNITE MINE\013-Sheet Set\FIGURES\2014-10-22 SETTLEMENT ANALYSIS\PLAN_VIEW



LEGEND:

3490 — EXISTING GROUND SURFACE ELEVATION, FEET

4 — CREEP SETTLEMENT CONTOUR, FEET

— — — — APPROXIMATE EXTENT OF COVER

- - - - - - - EDGE OF FILL

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS CREEP SETTLEMENT CONTOURS
FIGURE 32	REVISION 1
FILE NAME PLAN_VIEW	DATE JUNE 2015

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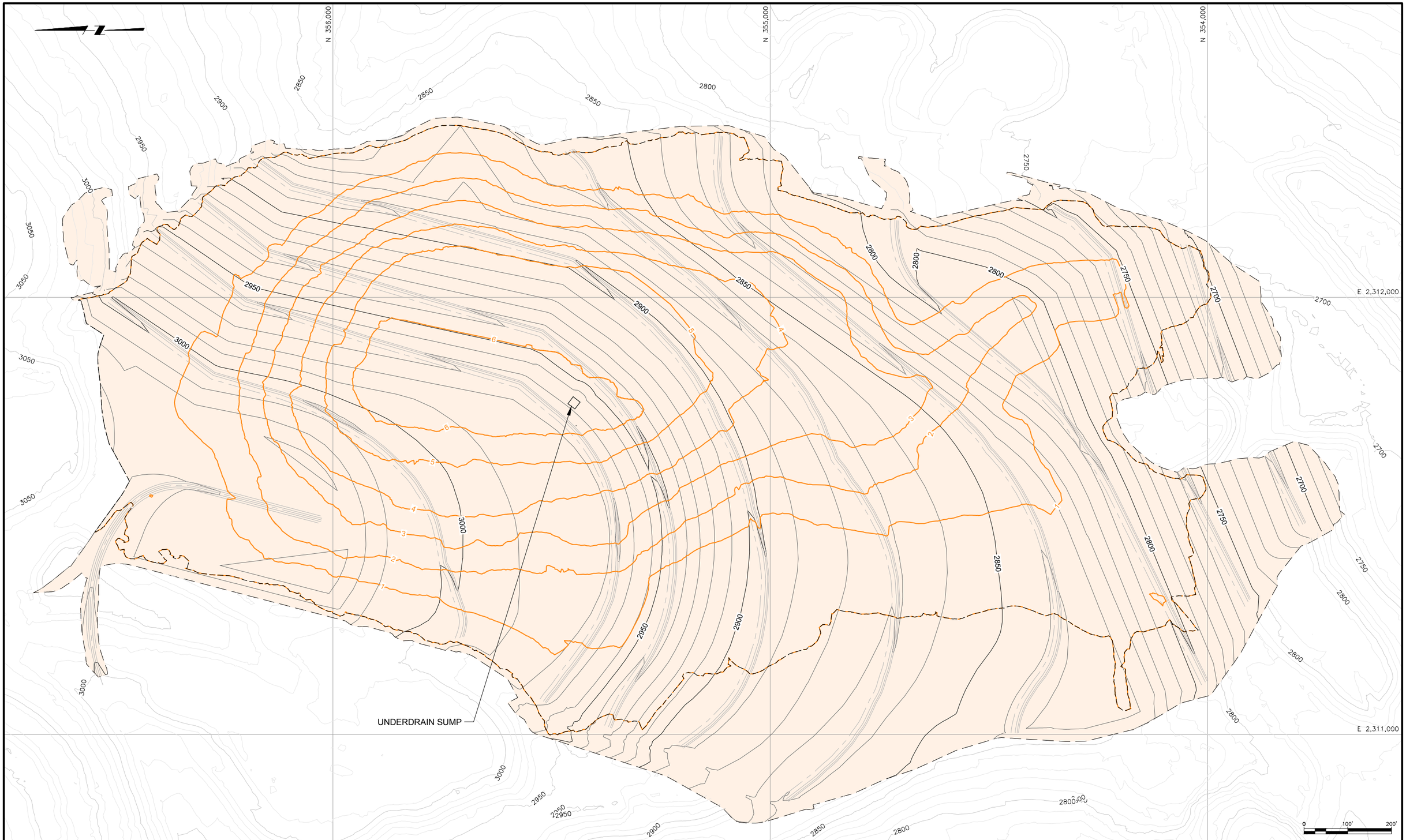

LEGEND:

- 3510 — EXISTING GROUND SURFACE ELEVATION, FEET
- 4 — CREEP SETTLEMENT CONTOUR ELEVATION, FEET
- 3430 — COVER CONTOUR GROUND SURFACE ELEVATION, FEET
- APPROXIMATE EXTENT OF COVER
- EDGE OF FILL

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS SETTLEMENT COVER CONTOURS
FIGURE 33	REVISION 1
FILE NAME PLAN VIEW	DATE JUNE 2015

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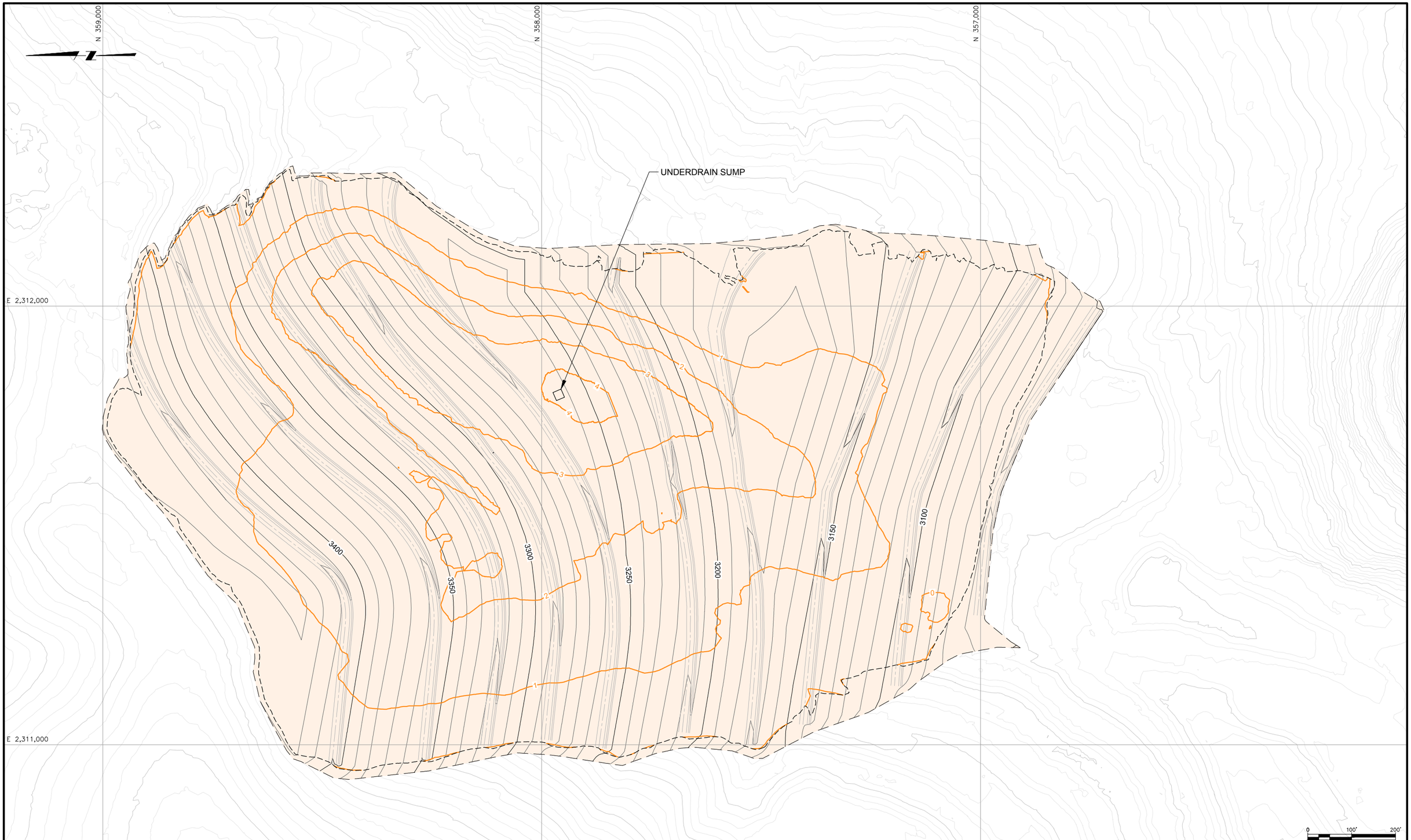

LEGEND:

- 3510 — EXISTING GROUND SURFACE ELEVATION, FEET
- 4 — CREEP SETTLEMENT CONTOUR ELEVATION, FEET
- 3430 — COVER CONTOUR GROUND SURFACE ELEVATION, FEET
- APPROXIMATE EXTENT OF COVER
- EDGE OF FILL

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS SETTLEMENT COVER CONTOURS
FIGURE 34	REVISION 1
FILE NAME	PLAN_VIEW
DATE	JUNE 2015

MWH

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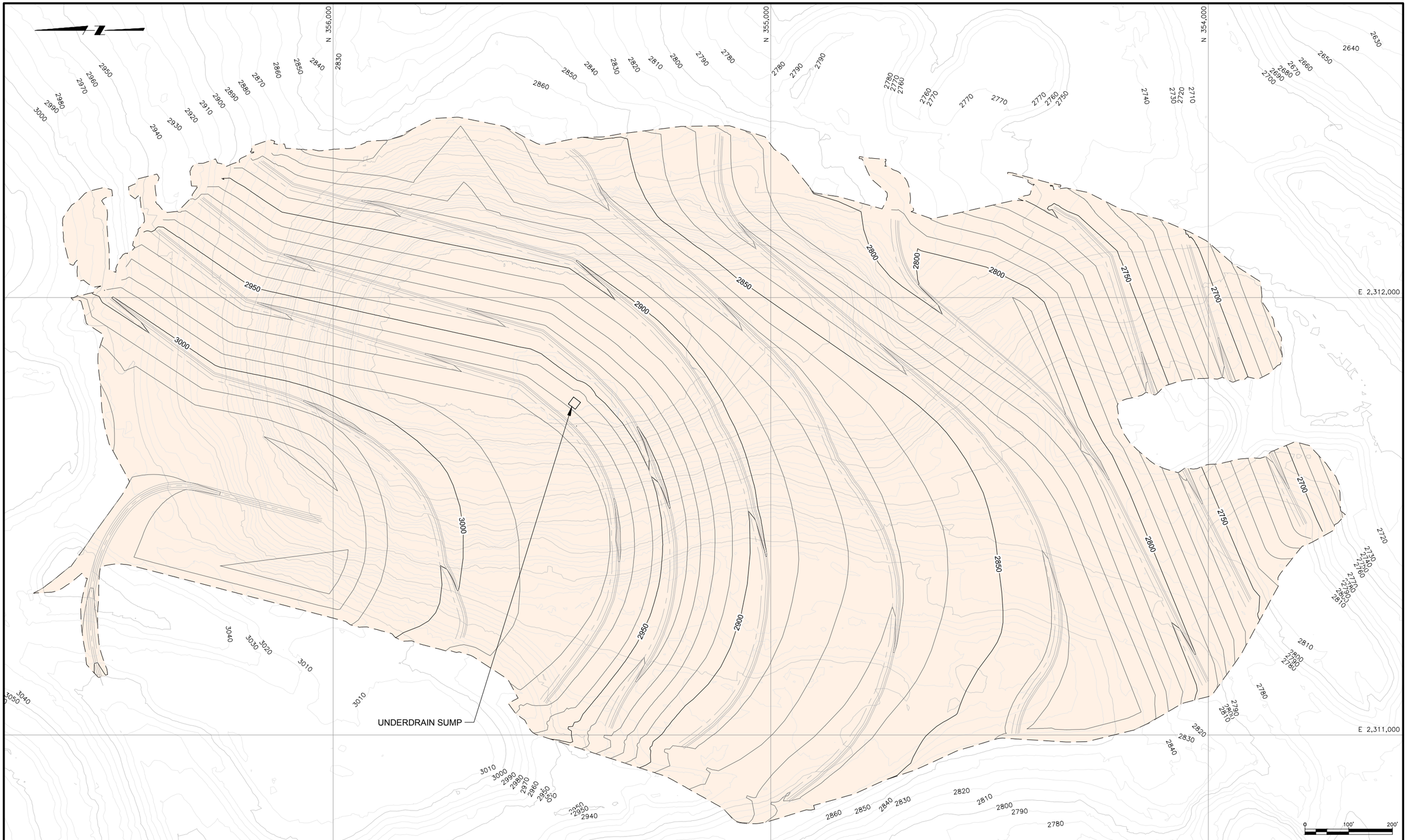

LEGEND:

3510 — EXISTING GROUND SURFACE ELEVATION, FEET

3430 — FINAL SETTLED COVER CONTOUR GROUND SURFACE ELEVATION, FEET

— — — — APPROXIMATE EXTENT OF COVER

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS SETTLEMENT COVER CONTOURS
FIGURE 35	REVISION 1
FILE NAME PLAN_VIEW	DATE JUNE 2015

MWH

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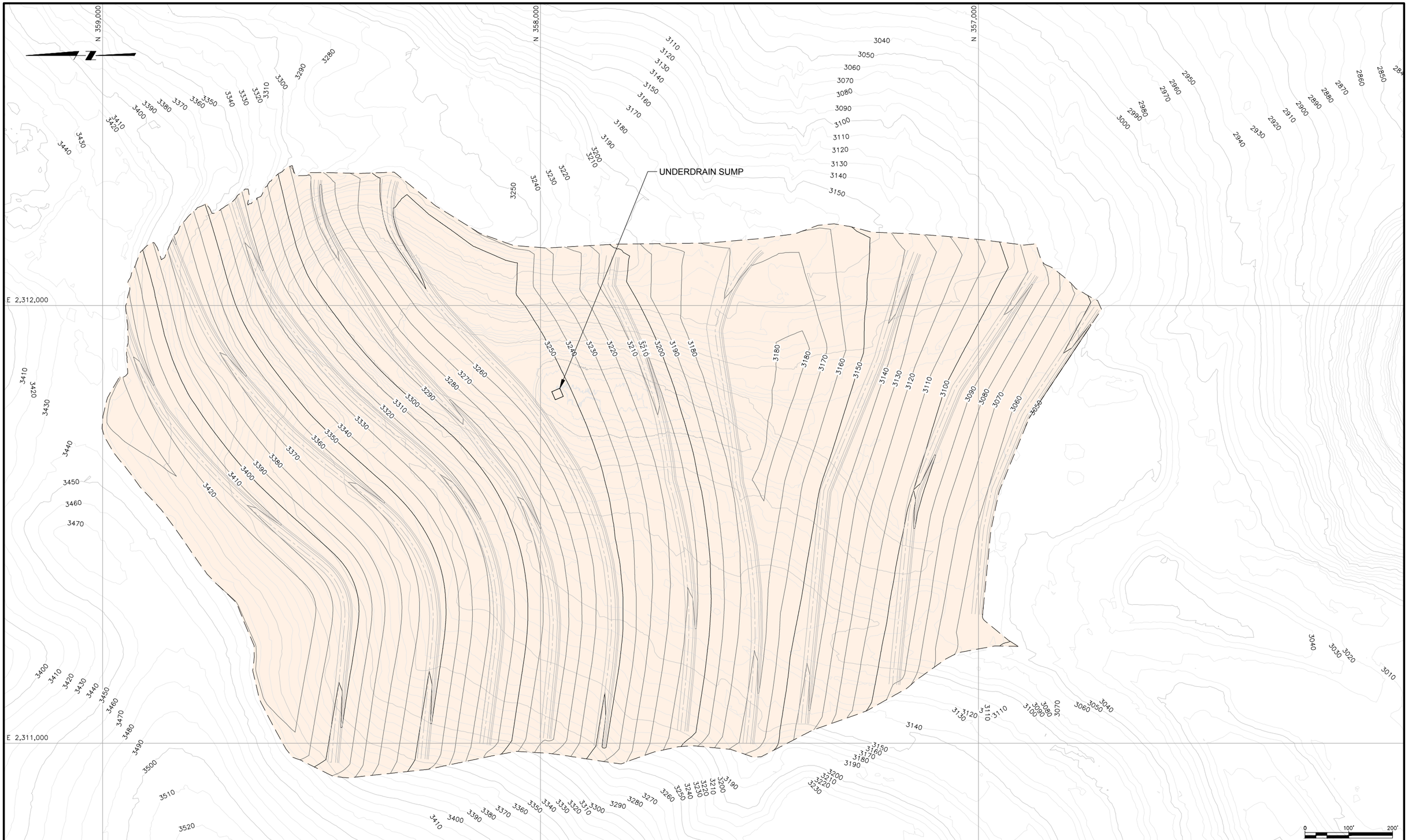

LEGEND:

3510 — EXISTING GROUND SURFACE ELEVATION, FEET

— — — — APPROXIMATE EXTENT OF COVER

3430 — FINAL SETTLED COVER CONTOUR GROUND SURFACE ELEVATION, FEET

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS SETTLEMENT COVER CONTOURS
FIGURE 36	REVISION 1
FILE NAME PLAN_VIEW	DATE JUNE 2015

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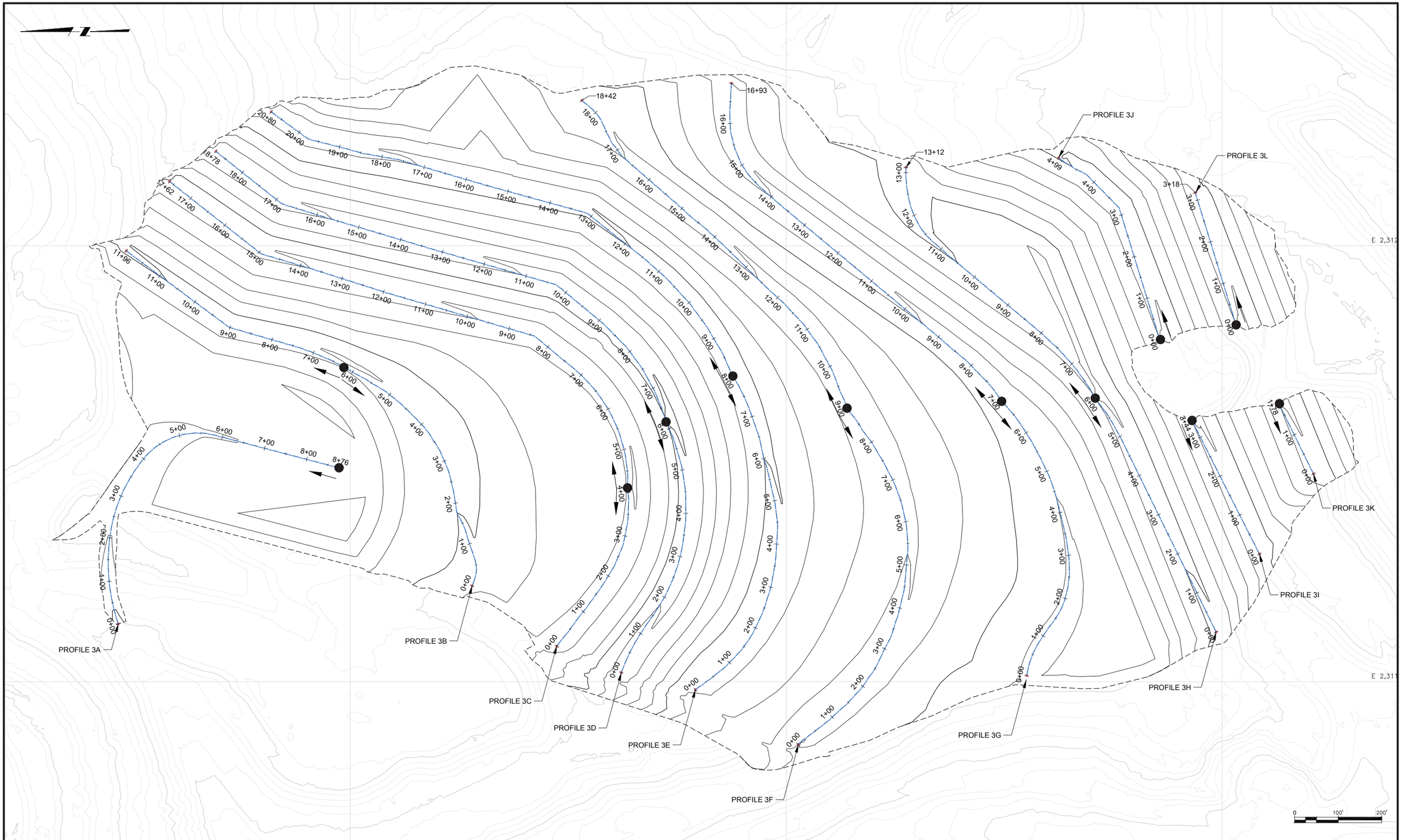

LEGEND:

3490 EXISTING GROUND SURFACE ELEVATION, FEET

3430 SETTLEMENT CONTOUR GROUND SURFACE ELEVATION, FEET

FLOW DIRECTION
HIGH POINT

APPROXIMATE EXTENT OF COVER

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS PROFILE LOCATIONS
SHEET	FIGURE 37
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

MWH

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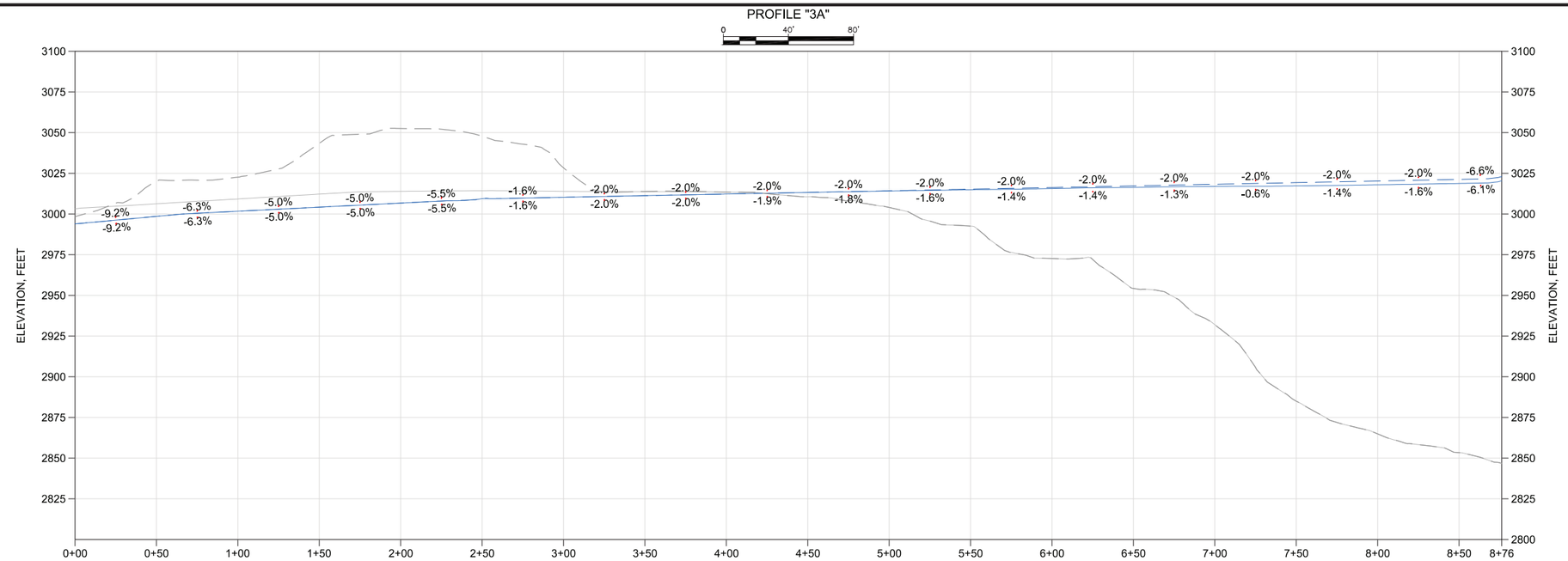

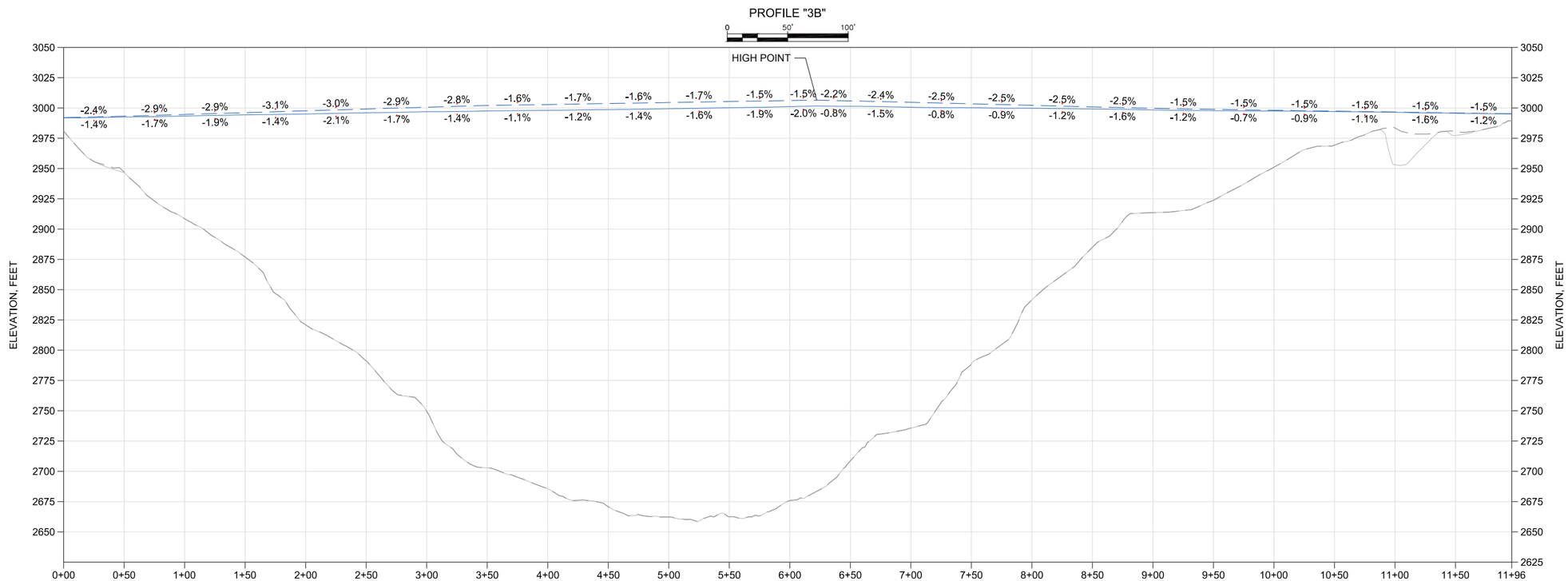

LEGEND:

-2.1% — SLOPE OF DESIGN GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

DESIGN GROUND SURFACE

-1.7% — SETTLEMENT GROUND SURFACE

SLOPE OF SETTLED GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

EXISTING GROUND SURFACE

MINE SITE SUBGRADE SURFACE

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS PROFILES A THROUGH B
SHEET	FIGURE 38
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

MWH

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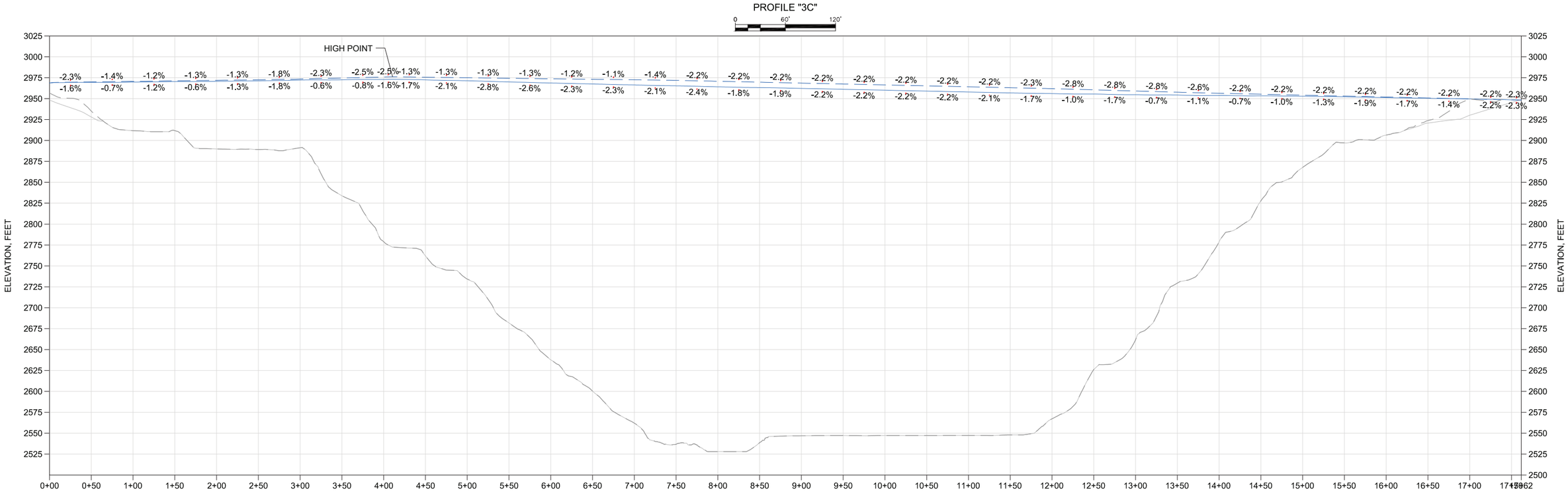

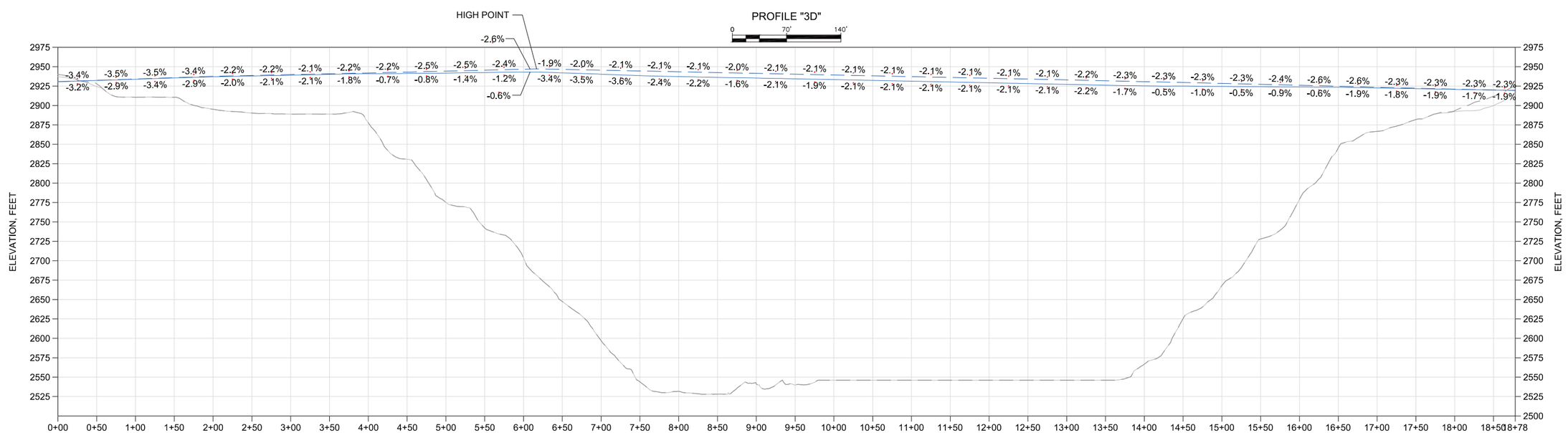

LEGEND:

-2.1% — SLOPE OF DESIGN GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

DESIGN GROUND SURFACE

-1.7% — SETTLEMENT GROUND SURFACE

SLOPE OF SETTLED GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

EXISTING GROUND SURFACE

MINE SITE SUBGRADE SURFACE

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS PROFILES C THROUGH D
SHEET	FIGURE 39
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

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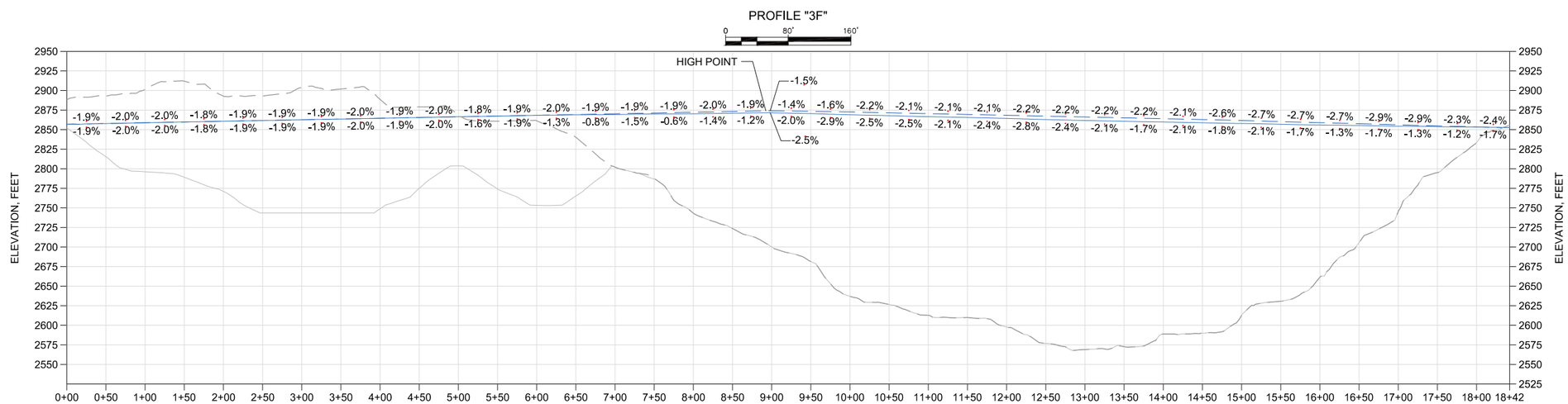


LEGEND:

-2.1% — SLOPE OF DESIGN GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

DESIGN GROUND SURFACE

-1.7% — SETTLEMENT GROUND SURFACE

SLOPE OF SETTLED GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

EXISTING GROUND SURFACE

MINE SITE SUBGRADE SURFACE

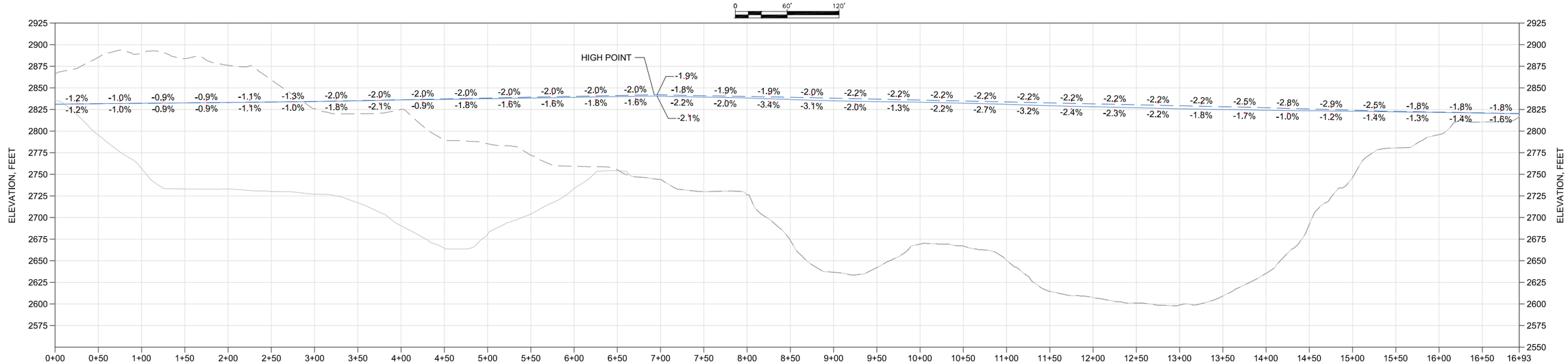
NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS PROFILES E THROUGH F
SHEET	FIGURE 40
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

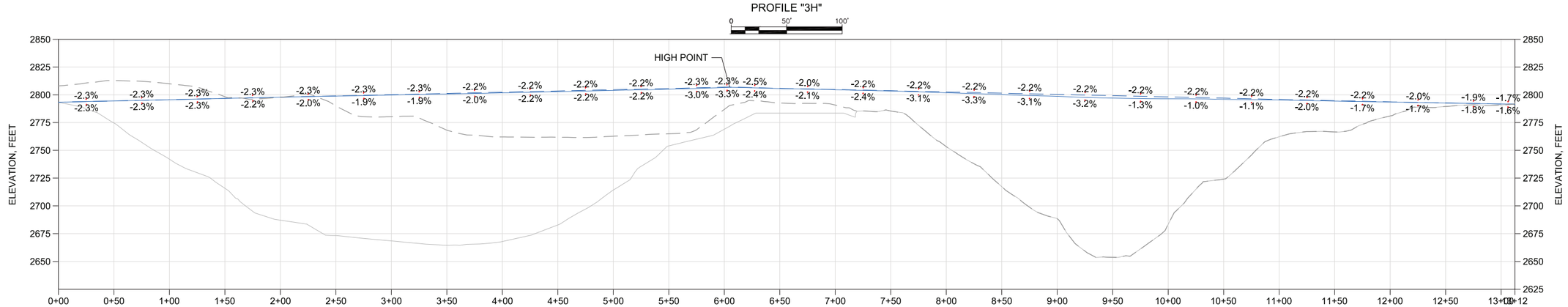
MWH

L:\design-drafting\clients-i-p\NEWMONT\101322 midnite mine\013-sheet set\FIGURES\2014-10-22 settlement analysis\PROFS AND LOCATIONS LAYOUT:Prof_3E_3F PLOT DATE:10/31/2014

PROFILE "3G"



PROFILE "3H"



LEGEND:

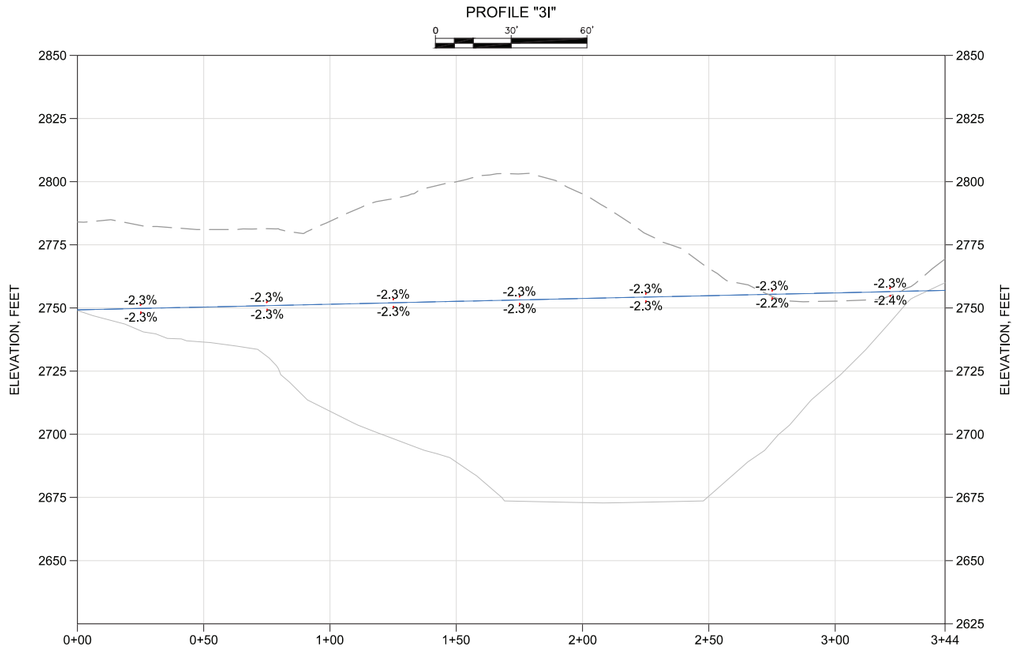
- -2.1% — SLOPE OF DESIGN GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)
- DESIGN GROUND SURFACE
- -1.7% — SETTLEMENT GROUND SURFACE
- SLOPE OF SETTLED GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)
- EXISTING GROUND SURFACE
- MINE SITE SUBGRADE SURFACE

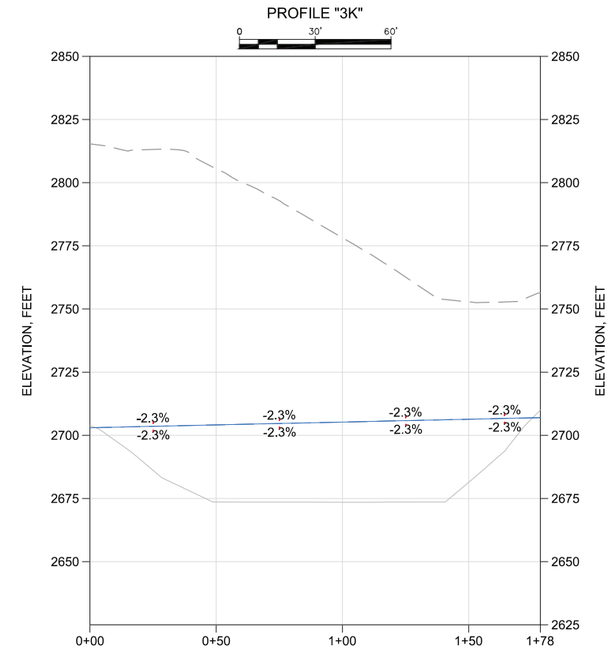
NEWMONT

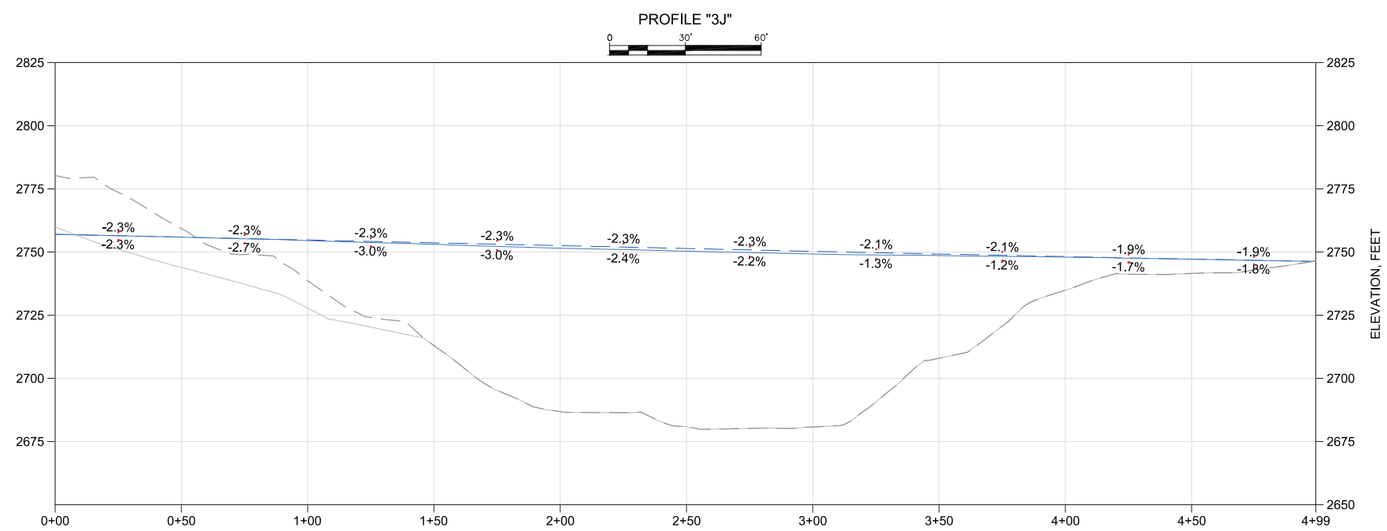
PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS PROFILES G THROUGH H
SHEET	FIGURE 41
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

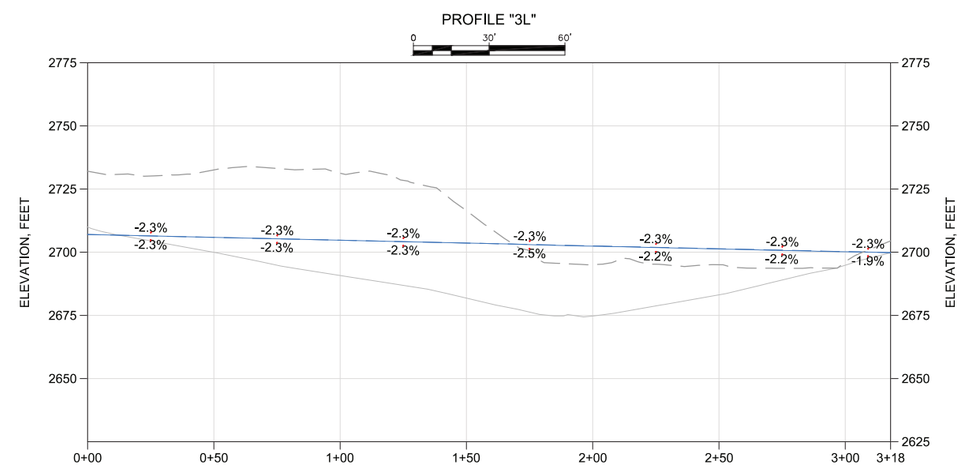
MWH

L:\design-drafting\clients-p\NEWMONT\011322 midnite mine\013-sheet set\FIGURES\2014-10-22 settlement analysis\PROFS AND LOCATIONS LAYOUT:Prof_3G_3H PLOT DATE:10/31/2014









LEGEND:

- -2.1% — SLOPE OF DESIGN GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)
- DESIGN GROUND SURFACE
- -1.7% — SETTLEMENT GROUND SURFACE
- SLOPE OF SETTLED GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)
- EXISTING GROUND SURFACE
- MINE SITE SUBGRADE SURFACE

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 3 COVER SETTLEMENT ANALYSIS PROFILES I THROUGH L
SHEET	FIGURE 42
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

MWH

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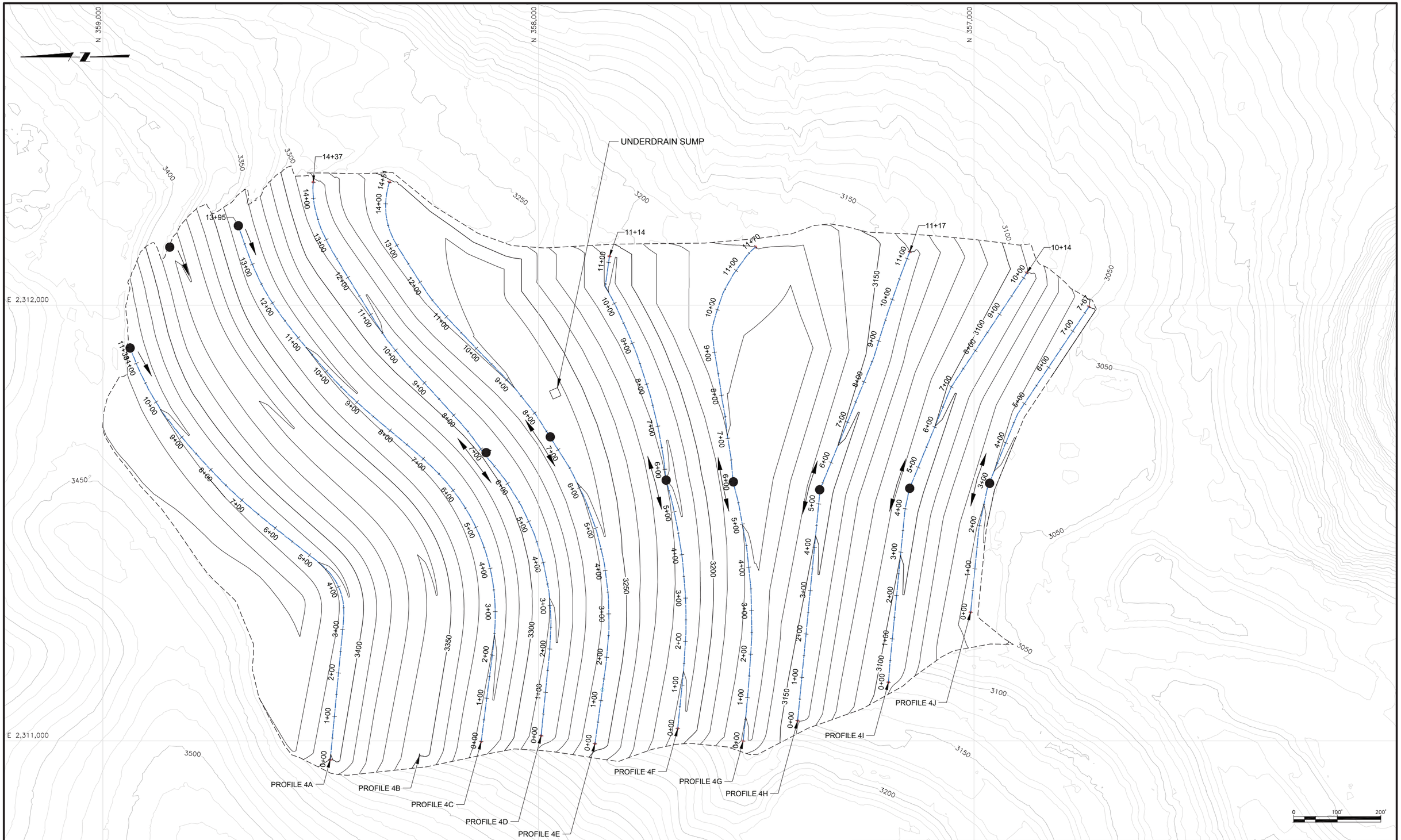


LEGEND:

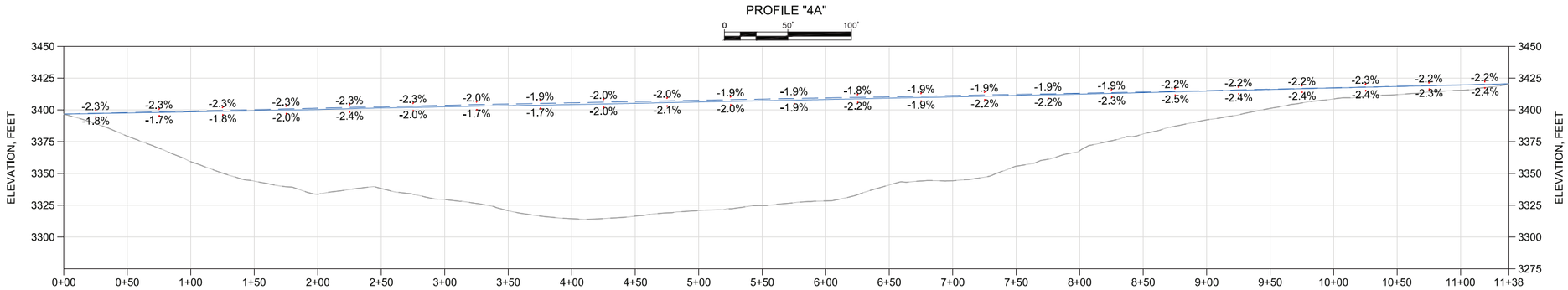
- 3490 — EXISTING GROUND SURFACE ELEVATION, FEET
- 3430 — SETTLEMENT CONTOUR GROUND SURFACE ELEVATION, FEET
- FLOW DIRECTION
- HIGH POINT
- APPROXIMATE EXTENT OF COVER

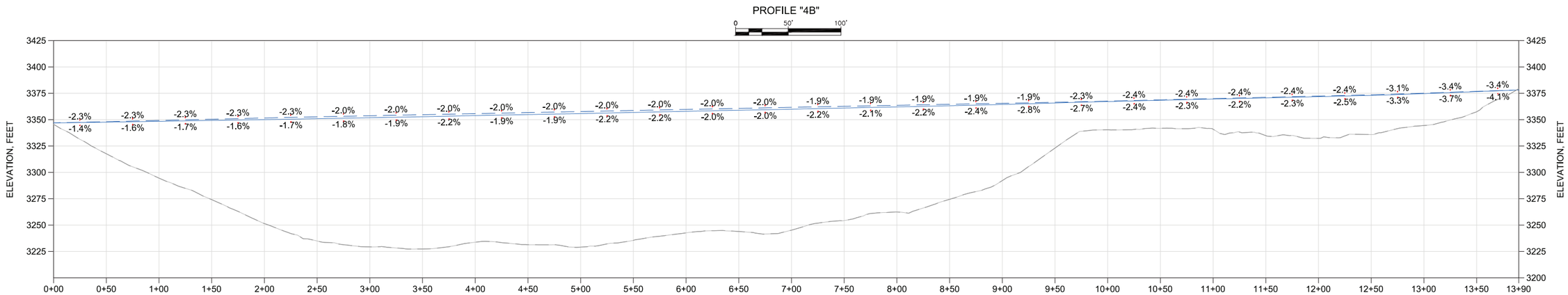
NEWMONT

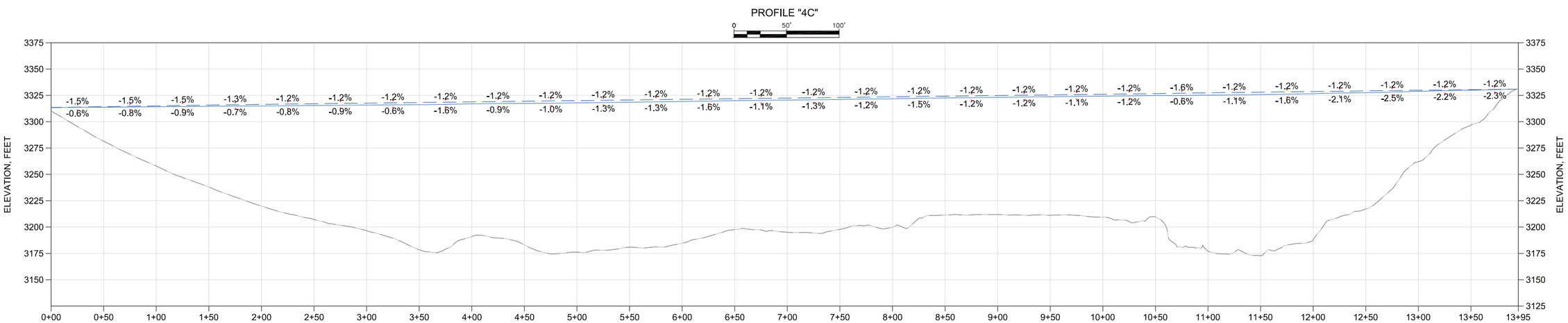
PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS PROFILE LOCATIONS
SHEET	FIGURE 43
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

MWH

L:\design-drafting\clients-i-p\NEWMONT\1013322 midnite mine\013-sheet set\FIGURES\2014-10-22 settlement analysis\PROFS AND LOCATIONS LAYOUT:Plan Pit 4 PLOT DATE:10/31/2014







LEGEND:

-2.1% — SLOPE OF DESIGN GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

DESIGN GROUND SURFACE

-1.7% — SETTLEMENT GROUND SURFACE

SLOPE OF SETTLED GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

EXISTING GROUND SURFACE

MINE SITE SUBGRADE SURFACE

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS PROFILES A THROUGH C
SHEET	FIGURE 44
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

MWH

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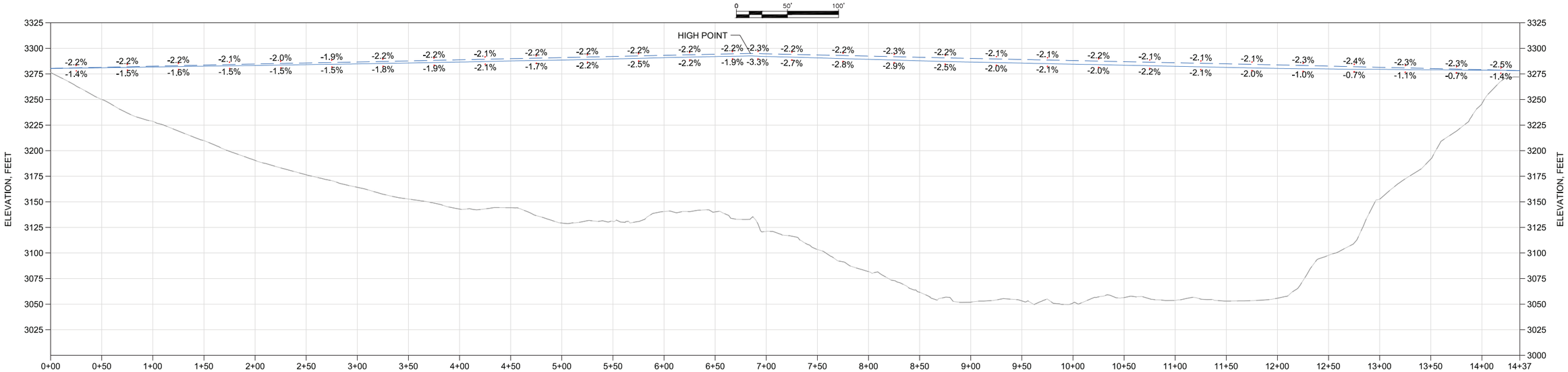
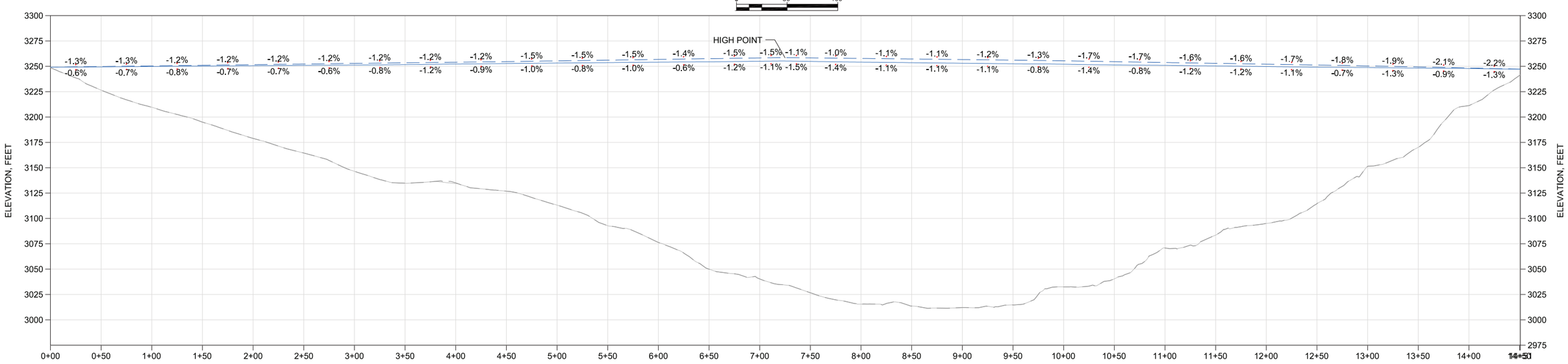
PROFILE "4D"



PROFILE "4E"



LEGEND:

-2.1% — SLOPE OF DESIGN GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

DESIGN GROUND SURFACE

-1.7% — SETTLEMENT GROUND SURFACE

SLOPE OF SETTLED GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)

EXISTING GROUND SURFACE

MINE SITE SUBGRADE SURFACE

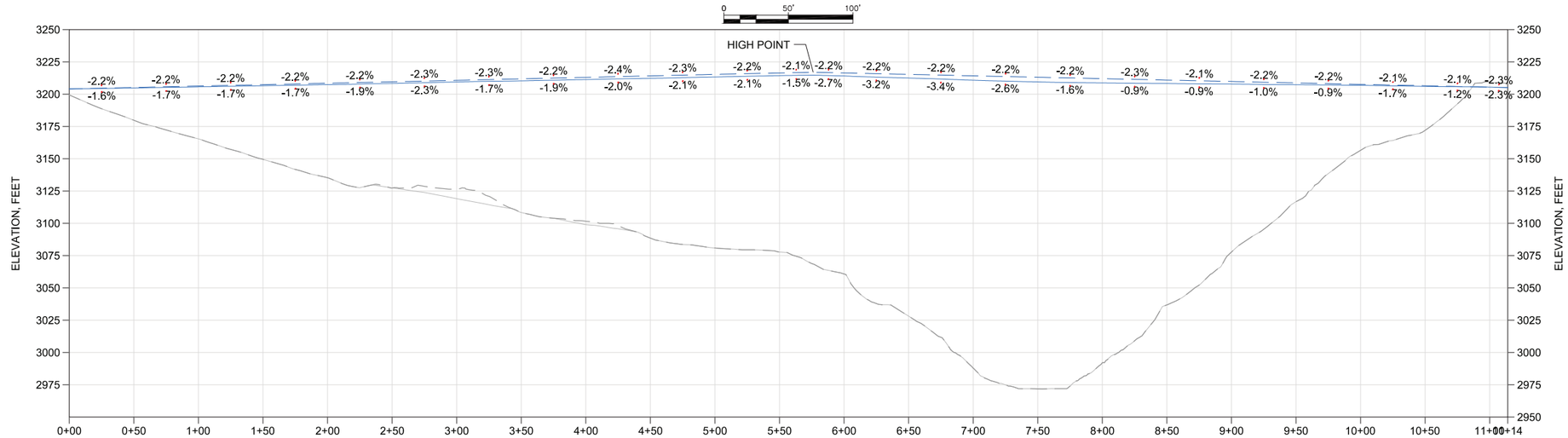
NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS PROFILES D THROUGH E
SHEET	FIGURE 45
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

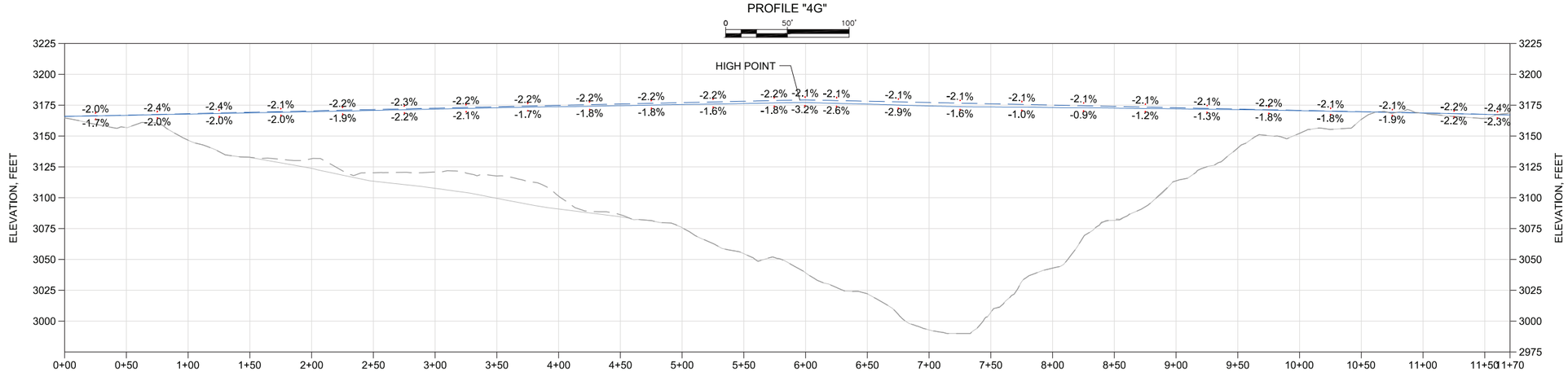
MWH

L:\design-drafting\clients-i-p\NEWMONT\1011322 midnite mine\013-sheet set\FIGURES\2014-10-22 settlement analysis\PROFS AND LOCATIONS LAYOUT:Prof_4D_4E PLOT DATE:10/31/2014

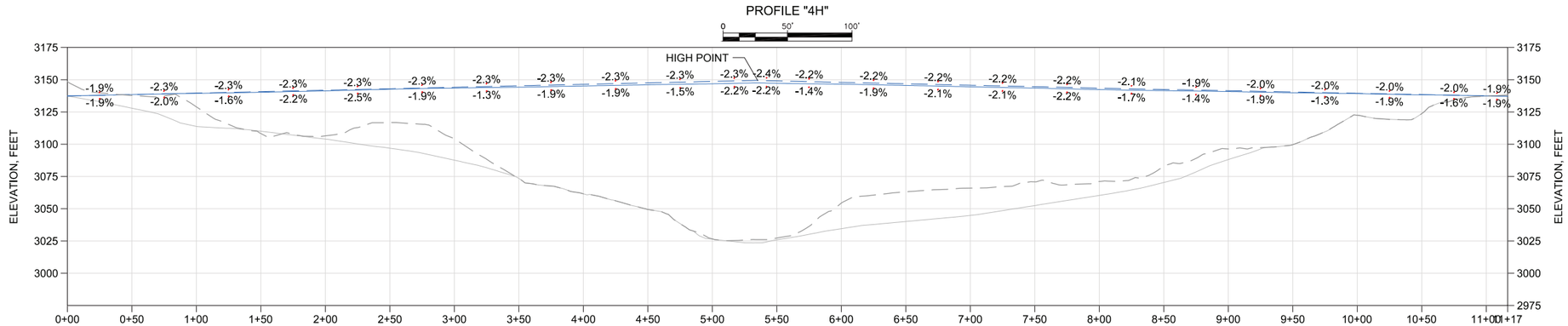
PROFILE "4F"



PROFILE "4G"



PROFILE "4H"



LEGEND:

- SLOPE OF DESIGN GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW) (-2.1%)
- DESIGN GROUND SURFACE
- SETTLEMENT GROUND SURFACE (-1.7%)
- SLOPE OF SETTLED GROUND SURFACE (NEGATIVE SIGN INDICATES DOWNGRADIENT FLOW)
- EXISTING GROUND SURFACE
- MINE SITE SUBGRADE SURFACE

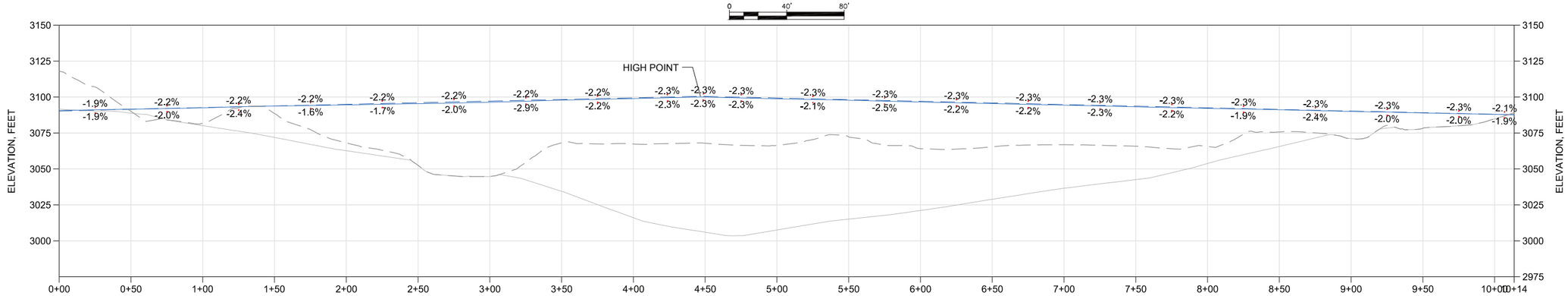
NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS PROFILES F THROUGH H
SHEET	FIGURE 46
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

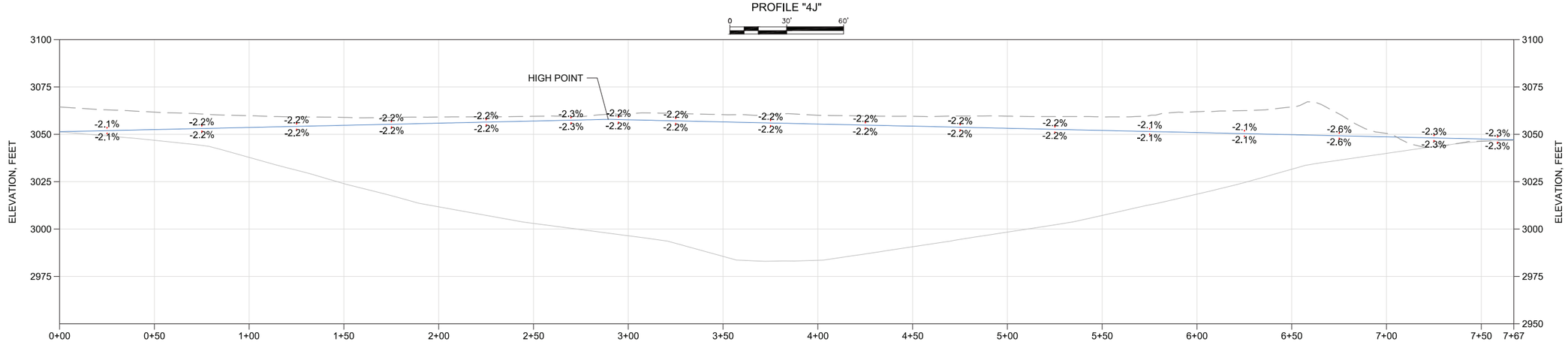
MWH

L:\design-drafting\clients-p\NEWMONT\011322 midnite mine\013-sheet set\FIGURES\2014-10-22 settlement analysis\PROFS AND LOCATIONS LAYOUT:Prof_4F_4G_4H PLOT DATE:10/31/2014

PROFILE "4I"



PROFILE "4J"



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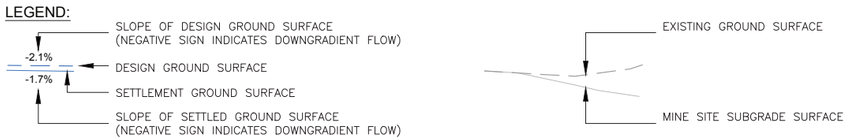

NEWMONT

PROJECT LOCATION	WELLPINIT, WASHINGTON
PROJECT	MIDNITE MINE
TITLE	PIT 4 COVER SETTLEMENT ANALYSIS PROFILES I THROUGH J
SHEET	FIGURE 47
REVISION	1
FILE NAME	PROFS AND LOCATIONS
DATE	OCT 2014

MWH

L:\design-drafting\clients-p\NEWMONT\011322 midnite mine\013-sheet set\FIGURES\2014-10-22 settlement analysis\PROFS AND LOCATIONS LAYOUT:Prof_4I_4J PLOT DATE:10/31/2014

Supplement D-13.1

Cover Geomembrane Lateral Displacement Calculations

Client: Newmont Mining USA
Project: Midnite Mine
Description: Pit 3 and Pit 4 Cover Lateral Displacement and Strain
Detail: Evaluate Effects of Differential Lateral Displacement on Cover Geomembrane Overlaps at Drainage Berms and Estimate Lateral Strain on Geomembrane

Job No.: 1011322
Date: 6/3/2015
Computed By: S.Goodwin
Checked By: M.Davis

Purpose: Evaluate the cover geomembrane overlap needed at drainage berms due to effects of lateral displacements of the top surface due to long-term creep deformations of the backfill.
Evaluate the strain on the cover geomembrane due to lateral displacements of the cover due to long-term creep deformations of the backfill.

Objective: Estimate maximum cover geomembrane overlap needed at drainage berms to account for lateral displacements of the top surface. Estimate the maximum lateral strain on the geomembrane due to lateral displacements.

Summary of Results:

Design cover geomembrane overlap at drainage benches is 5 feet which is greater than the calculated maximum overlap of 2.7 feet needed to account for lateral displacement of the cover.

Maximum average lateral strain (approx. 100' spacing) along an interbench slope for Pit 3 and Pit 4 (see Figure 29) =	1.06%
Maximum interbench slope length representative of maximum average lateral displacement along an interbench slope (see Figure 29) =	250 ft
Maximum geomembrane overlap needed at drainage berms to account for lateral displacement of the cover =	2.7 ft

Calculated as maximum average lateral strain (approx. 100' spacing) along an interbench slope multiplied by the maximum interbench slope length representative of the maximum average lateral displacement along an interbench slope.

Maximum lateral strain on geomembrane=	1.96%

Supporting Calculations:

Pit 3 Section A

Data from Plaxis Deformation Analysis					Average Lateral Strain for Approx. 100ft Spacing			
X	Cumulative Lateral Displacement (ft)	Final X	Original Horizontal Length (ft)	Final Horizontal Length (ft)	Original Horizontal Length (ft)	Final Horizontal Length (ft)	Incremental Lateral Displacement (ft)	Average Lateral Strain for approx. 100ft spacing (%)
201.769	0	201.769						
342.126	0	342.126	140.36	140.36	140.36	140.36	0.00	0.00%
352.973	0.2	353.173	10.85	11.05	34.76	35.16	0.40	1.15%
376.883	0.4	377.283	23.91	24.11				
396.586	0.6	397.186	19.70	19.90	101.84	103.84	2.00	1.96%
399.464	0.8	400.264	2.88	3.08				
401.899	1	402.899	2.44	2.63				
404.556	1.2	405.756	2.66	2.86				
406.769	1.4	408.169	2.21	2.41				
423.152	1.6	424.752	16.38	16.58				
439.091	1.8	440.891	15.94	16.14				
451.71	2	453.71	12.62	12.82				
463	2.2	465.2	11.29	11.49				
478.719	2.4	481.119	15.72	15.92				
580.776	2.522	583.298	102.06	102.18	102.06	102.18	0.12	0.12%
673.774	2.4	676.174	93.00	92.88	93.00	92.88	-0.12	-0.13%
742.83	2.2	745.03	69.06	68.86	121.13	120.73	-0.40	-0.33%
794.906	2	796.906	52.08	51.88				
849.246	1.8	851.046	54.34	54.14	106.09	105.69	-0.40	-0.38%
900.999	1.6	902.599	51.75	51.55				
928.723	1.4	930.123	27.72	27.52	105.61	105.01	-0.60	-0.57%
960.425	1.2	961.625	31.70	31.50				
1006.607	1	1007.607	46.18	45.98				
1041.049	0.8	1041.849	34.44	34.24	96.08	95.48	-0.60	-0.62%
1072.75	0.6	1073.35	31.70	31.50				
1102.691	0.4	1103.091	29.94	29.74				
1134.784	0.2	1134.984	32.09	31.89	116.44	115.64	-0.80	-0.69%
1167.073	0	1167.073	32.29	32.09				
1195.252	-0.2	1195.052	28.18	27.98				
1219.126	-0.4	1218.726	23.87	23.67				
1262.57	-0.6	1261.97	43.44	43.24	106.82	106.02	-0.80	-0.75%
1288.988	-0.8	1288.188	26.42	26.22				
1311.883	-1	1310.883	22.90	22.69				
1325.944	-1.2	1324.744	14.06	13.86				
1340.562	-1.336	1339.226	14.62	14.48	94.05	95.05	1.00	1.06%
1355.708	-1.2	1354.508	15.15	15.28				
1374.905	-1	1373.905	19.20	19.40				
1385.649	-0.8	1384.849	10.74	10.94				
1393.222	-0.6	1392.622	7.57	7.77				
1400.795	-0.4	1400.395	7.57	7.77				
1419.992	-0.2	1419.792	19.20	19.40				

Pit 3 Section B

Data from Plaxis Deformation Analysis					Average Lateral Strain for Approx. 100ft Spacing			
X	Cumulative Lateral Displacement (ft)	Final X	Original Horizontal Length (ft)	Final Horizontal Length (ft)	Original Horizontal Length (ft)	Final Horizontal Length (ft)	Incremental Lateral Displacement (ft)	Average Lateral Strain for approx. 100ft spacing (%)
360.775	0	360.775						
405.607	0	405.607	44.83	44.83	106.30	106.70	0.40	0.38%
431.85	0.2	432.05	26.24	26.44				
467.075	0.4	467.475	35.23	35.43				
490.718	0.6	491.318	23.64	23.84	94.07	95.87	1.80	1.91%
513.864	0.8	514.664	23.15	23.35				
519.65	1	520.65	5.79	5.99				
521.8	1.2	523	2.15	2.35				
523.618	1.4	525.018	1.82	2.02				
525.437	1.6	527.037	1.82	2.02				
528.909	1.8	530.709	3.47	3.67				
539.655	2	541.655	10.75	10.95				
561.148	2.2	563.348	21.49	21.69				
576.855	2.4	579.255	15.71	15.91	108.95	109.35	0.40	0.37%
592.561	2.6	595.161	15.71	15.91				
631.248	2.756	634.004	38.69	38.84				
670.101	2.6	672.701	38.85	38.70				
747.311	2.4	749.711	77.21	77.01	137.06	136.66	-0.40	-0.29%
807.16	2.2	809.36	59.85	59.65				
860.231	2	862.231	53.07	52.87	105.65	105.25	-0.40	-0.38%
912.806	1.8	914.606	52.58	52.38				
968.853	1.6	970.453	56.05	55.85	117.39	116.99	-0.40	-0.34%
1030.191	1.4	1031.591	61.34	61.14				
1103.432	1.2	1104.632	73.24	73.04	73.24	73.04	-0.20	-0.27%
1201.143	1	1202.143	97.71	97.51	97.71	97.51	-0.20	-0.20%
1333.863	0.8	1334.663	132.72	132.52	132.72	132.52	-0.20	-0.15%
1747.091	0.6	1747.691	413.23	413.03	413.23	413.03	-0.20	-0.05%
1876.556	0.4	1876.956	129.47	129.27	129.47	129.27	-0.20	-0.15%
1940.931	0.2	1941.131	64.38	64.18	64.38	64.18	-0.20	-0.31%
2006.308	0	2006.308	65.38	65.18	114.83	114.43	-0.40	-0.35%
2055.765	-0.2	2055.565	49.46	49.26				
2103.795	-0.4	2103.395	48.03	47.83	82.98	82.58	-0.40	-0.48%
2138.747	-0.6	2138.147	34.95	34.75				
2171.797	-0.7234	2171.0736	33.05	32.93	92.26	92.86	0.60	0.65%
2189.155	-0.6	2188.555	17.36	17.48				
2202.47	-0.4	2202.07	13.31	13.51				
2216.974	-0.2	2216.774	14.50	14.70				
2231.003	0	2231.003	14.03	14.23				

Client: Newmont Mining USA
Project: Midnite Mine
Description: Pit 3 and Pit 4 Cover Lateral Displacement and Strain
Detail: Evaluate Effects of Differential Lateral Displacement on Cover Geomembrane Overlaps at Drainage Berms and Estimate Lateral Strain on Geomembrane

Job No.: 1011322
Date: 6/3/2015
Computed By: S.Goodwin
Checked By: M.Davis

Pit 4 Section C

Data from Plaxis Deformation Analysis					Average Lateral Strain for Approx. 100ft Spacing			
X	Cumulative Lateral Displacement (ft)	Final X	Original Horizontal Length (ft)	Final Horizontal Length (ft)	Original Horizontal Length (ft)	Final Horizontal Length (ft)	Incremental Lateral Displacement (ft)	Average Lateral Strain for approx. 100ft spacing (%)
201.805	0.2	202.005						
252.521	0.4	252.921	50.72	50.92				
328.841	0.6	329.441	76.32	76.52	127.04	127.44	0.40	0.31%
407.354	0.8	408.154	78.51	78.71	78.51	78.71	0.20	0.25%
484.161	1	485.161	76.81	77.01	76.81	77.01	0.20	0.26%
553.165	1.2	554.365	69.00	69.20	69.00	69.20	0.20	0.29%
658.744	1.4	660.144	105.58	105.78	105.58	105.78	0.20	0.19%
688.979	1.464	690.443	30.24	30.30				
721.896	1.4	723.296	32.92	32.85	63.15	63.15	0.00	0.00%
809.188	1.2	810.388	87.29	87.09	87.29	87.09	-0.20	-0.23%
857.954	1	858.954	48.77	48.57				
889.896	0.8	890.696	31.94	31.74				
922.326	0.6	922.926	32.43	32.23	113.14	112.54	-0.60	-0.53%
956.462	0.4	956.862	34.14	33.94				
994.012	0.2	994.212	37.55	37.35				
1027.661	0	1027.661	33.65	33.45	105.34	104.74	-0.60	-0.57%
1057.652	-0.2	1057.452	29.99	29.79				
1085.205	-0.4	1084.805	27.55	27.35				
1108.125	-0.6	1107.525	22.92	22.72				
1129.338	-0.8	1128.538	21.21	21.01	101.68	100.88	-0.80	-0.79%
1159.086	-1	1158.086	29.75	29.55	29.75	29.55	-0.20	-0.67%
1182.25	-1.055	1181.195	23.16	23.11	96.07	96.87	0.80	0.83%
1202	-1	1201	19.75	19.81				
1221.506	-0.8	1220.706	19.51	19.71				
1233.21	-0.6	1232.61	11.70	11.90				
1244.914	-0.4	1244.514	11.70	11.90				
1255.155	-0.2	1254.955	10.24	10.44				

Pit 4 Section D

Data from Plaxis Deformation Analysis					Average Lateral Strain for Approx. 100ft Spacing			
X	Cumulative Lateral Displacement (ft)	Final X	Original Horizontal Length (ft)	Final Horizontal Length (ft)	Original Horizontal Length (ft)	Final Horizontal Length (ft)	Incremental Lateral Displacement (ft)	Average Lateral Strain for approx. 100ft spacing (%)
173.115	0	173.115						
180.71	0	180.71	7.60	7.60				
183.449	0.1	183.549	2.74	2.84				
185.468	0.2	185.668	2.02	2.12				
186.91	0.3	187.21	1.44	1.54				
187.583	0.4	187.983	0.67	0.77				
188.256	0.5	188.756	0.67	0.77				
189.025	0.6	189.625	0.77	0.87				
189.698	0.7	190.398	0.67	0.77				
190.418	0.8	191.218	0.72	0.82				
191.139	0.9	192.039	0.72	0.82				
205.27	1	206.27	14.13	14.23				
209.356	1.1	210.456	4.09	4.19				
221.324	1.2	222.524	11.97	12.07				
235.935	1.3	237.235	14.61	14.71				
248.432	1.4	249.832	12.50	12.60				
274.05	1.5	275.55	25.62	25.72	100.94	102.44	1.50	1.49%
286.354	1.532	287.886	12.30	12.34				
295.246	1.5	296.746	8.89	8.86				
313.078	1.4	314.478	17.83	17.73				
339.321	1.3	340.621	26.24	26.14				
370.61	1.2	371.81	31.29	31.19	96.56	96.26	-0.30	-0.31%
404.495	1.1	405.595	33.89	33.79				
443.427	1	444.427	38.93	38.83	72.82	72.62	-0.20	-0.27%
501.056	0.9	501.956	57.63	57.53	57.63	57.53	-0.10	-0.17%
1006.576	0.8	1007.376	505.52	505.42	505.52	505.42	-0.10	-0.02%
1122.086	0.7	1122.786	115.51	115.41	115.51	115.41	-0.10	-0.09%
1174.775	0.6	1175.375	52.69	52.59				
1192.273	0.5	1192.773	17.50	17.40	70.19	69.99	-0.20	-0.28%
1464.274	0.4	1464.674	272.00	271.90	272.00	271.90	-0.10	-0.04%
1560.708	0.3	1561.008	96.43	96.33	96.43	96.33	-0.10	-0.10%
1616.363	0.2	1616.563	55.66	55.56				
1643.723	0.1	1643.823	27.36	27.26				
1674.223	0	1674.223	30.50	30.40	113.52	113.22	-0.30	-0.26%
1698.444	-0.1	1698.344	24.22	24.12				
1707.19	-0.1546	1707.0354	8.75	8.69				
1734.551	-0.1	1734.451	27.36	27.42				
1762.135	0	1762.135	27.58	27.68	87.91	87.91	0.00	0.00%
2040.887	0.1	2040.987	278.75	278.85	278.75	278.85	0.10	0.04%

Attachment D-14

Boyd Pit and Pit 2 Dewatering

ATTACHMENT D-14

MIDNITE MINE REMEDIAL ACTION
BOYD PIT AND PIT 2 DEWATERING

Revisioning						
Rev.	**Date**	**Description**	**By**	**Checked**	**Date**	**Reviewed**
0	*11-Jun-2015*	*100% Design*	*Tom Kelley and Jim Finley*	*Vance Drain*	*12-Jun-2015*	*Vance Drain*

Location and Format
Electronic copies of these calculations are located in the project files system at: *\\usslc1s01\ifo\Industrial Projects\MIDNITE MINE\Deliverables_Working Documents\Basis of Design Rpt\Appendix D - Mine Waste Excavation and Containment\100% Submittal\Att D-14 BPA Pumping*

1.0 INTRODUCTION

Pit 2 and the Boyd Pit are part of the Backfilled Pit Area (BPA) at the Midnite Mine (the Site) and were backfilled during mining operations. The BPA also includes two smaller pits, Pit 2 West and the Adit Pit, which are currently dry. Removal of groundwater in Pit 2 and the Boyd Pit is necessary to meet Performance Standards established in the Consent Decree that prescribe, "Water in the BPA shall be removed using wells or other methods approved by the EPA during RD, to elevations determined during RD which minimize head, scaling, and fouling." This attachment is a review of data generated during initial implementation of BPA groundwater pumping from dewatering wells through the end of 2014. The objective of the initial pumping was to obtain data that could be used to define the configuration of, and water levels to be maintained by, the long-term BPA dewatering system. While the focus of this evaluation is on data collected from the start of pumping in September 2013 to the present, some consideration has been given to historic data to reconcile differences in water surface elevations related primarily to variations in reported Tops of Casings (TOCs) as groundwater measuring points.

Groundwater in Pit 2 and the Boyd Pit at the Site is being pumped in accordance with the *Backfilled Pits Area Pumping Plan* (*BPA Work Plan*, WME, 2013a). The current configuration of the BPA, including groundwater monitoring wells in the immediate vicinity, is shown on Figure 1. The estimated configuration of the BPA prior to backfilling with waste rock is shown on Figure 2, with cross sections through the BPA shown on Figure 3.

The conceptual groundwater flow model for the BPA is summarized in the *BPA Pumping Plan*. This conceptual groundwater flow model suggests that in the absence of active pumping, groundwater accumulating in the backfilled Pit 2 flows to the south and reports to the Boyd Pit. Again in the absence of active pumping of the Boyd Pit, groundwater will eventually accumulate to the point where it flows through a buried alluvial channel south of the Boyd Pit at the waste rock/native soil contact and reports to the Pollution Control Pond (PCP). In addition, some of the groundwater flow in this buried alluvial channel may be redirected toward the Western Drainage by remnants of a buried mine haul road that can be seen in historical aerial photographs, but is not reflected in the pre-backfill topography shown on Figure 2.

Groundwater pumping began in the Boyd Pit in September 2013 via groundwater well GW-54 with the objective of lowering the pit groundwater surface in stages until a final target elevation of 2,660 ft amsl was reached. Once the final target elevation in the Boyd Pit was reached, groundwater pumping began in Pit 2 (in March 2014). Groundwater levels at monitoring and dewatering wells within the BPA are measured by pressure transducers and/or manually. Samples of groundwater also were collected from the dewatering wells (GW-54 in Boyd Pit and GW-58 in Pit 2) for chemical analysis.

2.0 DATA

2.1 METHODS AND ASSUMPTION

Early in 2014, a survey of groundwater well locations (NAD83 horizontal coordinate system) and pertinent elevations (e.g., TOC) (NAVD88) was completed, and all subsequent water surface elevations for this evaluation were calculated using the new survey information and a consistent (NAD83/NAVD88) reference datum. The survey information was transmitted by letter dated

January 27, 2014 from Walter O. Dale (Benthin and Associates) to Rob Noble (Worthington Miller Environmental, LLC [WME]).

Monthly reports detailing the status of Site activities associated with the Consent Decree were prepared and submitted by WME. The data contained in the monthly reports was used for this evaluation. Measurements of depth to groundwater were obtained through a combination of manual readings and pressure transducer readings. An examination of the raw data was first made to identify potential inconsistencies and verify the water level data obtained from pressure transducers are internally consistent with the data collected manually.

The evaluation presented herein was conducted using the background information provided in the *BPA Work Plan* and available from the BPA Pumping Reports (WME, 2013b, 2014a, 2014b, 2014c) through the end of 2014. Data are not available for the entire period for all monitoring locations. However, pumping of the BPA will continue into the future and the evaluation of Pit 2 and Boyd Pit dewatering should continue and be revised as necessary to incorporate the additional, pertinent information as it becomes available.

3.0 RESULTS

3.1 BOYD PIT

Figure 4 provides a summary of water-level measurements obtained from the BPA pumping campaign that started in September 2013. As expected, the groundwater level in GW-54 (the pumping well) began to decline as soon as pumping started, and subsequently responded accordingly with changes in the pumping rate. For most of October and November the pumping rate remained at a relatively constant rate ranging from 33 to 34 gallons per minute (gpm), until the groundwater level reached the first target elevation at the level of the estimated Boyd Pit overflow (approximately 2679 ft amsl). The pumping rate then was reduced to approximately 7.5 gpm in order to maintain the Boyd Pit groundwater at this first target elevation.

After maintaining the groundwater elevation at approximately 2679 ft amsl for two weeks, the pumping rate was once again increased to draw the Boyd Pit groundwater level down to the second target elevation. The pumping rate during the second drawdown stage was approximately equal to the pumping rate during the initial drawdown stage of the campaign (i.e., rate ranging from 33 to 34 gpm). The rate of the groundwater level decline increased slightly during the second drawdown stage, which is consistent with the influence of a low-hydraulic conductivity hydrogeologic boundary (i.e., the pit walls) that decreases in radius with depth; this change is also consistent with the interpreted elevation of the Boyd Pit overflow.

Once the second target elevation was reached (at approximately 2,673 ft amsl) in mid-January 2014, the pumping rate was decreased to an average pumping rate of 7.2 gpm in order to maintain the Boyd Pit groundwater level at this second target elevation. After maintaining the second target groundwater level in the Boyd Pit for approximately two weeks, the third drawdown stage commenced by increasing the pumping rate to approximately 30 gpm in early February 2014. A noticeable increase in the rate of groundwater level decline relative to previous stages was observed in the third drawdown stage and indicates the influence of a distinct low-permeability hydrogeologic boundary condition, again to be expected given the limited extent of waste rock backfill surrounded by porphyritic quartz monzonite bedrock. Once the third target elevation of approximately 2,664 ft amsl was reached, the pumping rate was

decreased to an average pumping rate of 6.7 gpm in order to maintain the Boyd Pit groundwater level at this elevation.

The final (fourth) drawdown stage began after maintaining the third Boyd Pit target groundwater level for approximately two weeks. The fourth target elevation of approximately 2,662 was achieved within approximately one week by pumping at an average rate of 13.4 gpm. This fourth target level was maintained for approximately two weeks by pumping at approximately 6.6 gpm prior to commencing with dewatering of Pit 2 by pumping from well GW-58.

After dewatering of Pit 2 started using GW-58, the Boyd Pit groundwater level continued to decline even though the pumping rate was reduced. This response was predicted based on the conceptual flow model because the flow contribution from Pit 2 to the Boyd would be reduced as the water level in Pit 2 declined. By late May/early June 2014, a relatively constant Boyd Pit groundwater level was being maintained at an approximate elevation of 2,660 by pumping from GW-54 at a rate of 4 gpm. For the balance of 2014, a groundwater level between elevation 2,659 and 2,665 was maintained in the Boyd Pit by pumping from GW-54 at an average rate of approximately 2.4 gpm. The pumping rates at GW-54 that were required to maintain the various target groundwater elevations in the Boyd Pit are summarized in Table 1. To the extent possible, average pumping rates shown in Table 1 were calculated at points in time when the measured water level in GW-54 were approximately equal in order to negate the effects of changes in water storage due to fluctuating water levels.

Table 1 Summary of Boyd Pit (GW-54) Water Levels and Pumping Rates

Stage	Elevation Range (ft – amsl)	Date Range	Average Pumping Rate (gpm)
1	2,678.6 to 2,678.7	Dec. 2, 2013 to Dec. 16, 2013	7.5
2	2,672.6 to 2,672.6	Jan. 13, 2014 to Jan 21, 2014	7.2
3	2,664.4 to 2,664.6	Feb. 10, 2014 to Feb 24, 2014	6.7
4	2,661.8 to 2,661.9	March 4, 2014 to March 25, 2014(1)	6.6(1)
5	2,659.6 to 2,660.4	May 12, 2014 to June 9, 2014	4.0
6	2,659.3 to 2,665.4	July 14, 2014 to Dec 29, 2014	2.4

Note: (1) Dewatering of Pit 2 commenced on March 20, 2014 by pumping from GW-58.

Two additional groundwater monitoring wells are located in or adjacent to the Boyd Pit. Groundwater monitoring well GW-57 is located in the Boyd Pit about 76 feet northeast of the pumping well GW-54 and is completed in bedrock (Figures 1 and 2). The second groundwater monitoring well is GW-56, which is located outside of the lowest point of the Boyd Pit and completed in waste rock in the shallow topographic gradient area that was the ramp into the Boyd Pit during operations, which also constitutes the Boyd Pit overflow (at approximately 2,679 ft amsl elevation).

As shown in Figure 4, the groundwater level measured in GW-57 indicates that there is a hydraulic gradient from the bedrock below Boyd Pit into the waste rock. The hydraulic connection between the bedrock measured in GW-57 and the pumping well GW-54 is good because the response measured in GW-57 nearly parallels the response observed in GW-54. The essential observation from the groundwater data collected at GW-57 is that there is a hydraulic gradient from the bedrock groundwater system to the backfilled Boyd Pit at all measured pit water levels, and that pumping from GW-54 appears to increase the hydraulic gradient from the bedrock groundwater system toward the Boyd Pit.

Groundwater levels measured in GW-56 track the decline in groundwater levels in GW-54 almost exactly until the groundwater levels decline below the elevation of the effective screen bottom in GW-56 (Figure 4). Waste rock along the hydrologic flow path between GW-54 and GW-56 does not impart significant resistance to hydraulic stress induced by pumping GW-56 as indicated by the close correspondence between measured groundwater levels. To the extent that the hydraulic properties of waste rock in the Boyd Pit are highly permeable and relatively homogeneous, controlling groundwater flow from the Boyd Pit by pumping GW-54 appears to be effective. Furthermore, this analysis of the available data indicates that a target water level between 2,659 and 2,665 ft amsl is sustainable as a groundwater level within this range was maintained for approximately 9 months during the BPA dewatering program in 2014.

Both the average groundwater pumping rate and daily precipitation are shown in Figure 4. The average groundwater pumping rate was determined based on the cumulative flow meter readings. Note that the one value of greater than 60 gpm is probably not reflective of actual conditions in the backfill, but rather is likely reflective of the replacement of the flow meter that occurred in the same time period. As for precipitation, the main thing to note is that precipitation during February and March 2014 contributed 5.2 inches of water to the overall system, in addition to snowmelt that occurred during the same period.

Groundwater level data collected from mid-1999 through October 2002 were also reviewed after adjusting calculated water surface elevations using the survey data from January 2014 (Figure 5). Overall the water levels are comparable and consistent with the more recent information. Furthermore, there is a similar response of groundwater levels as was observed during the recent pumping campaign. That is, the groundwater levels measured in GW-56 track very closely the groundwater levels measured in GW-54 until the effective bottom elevation of the well screen is reached, while the groundwater level measured in GW-57 parallels, but lags the groundwater levels in GW-56 and GW-54 and remains at a somewhat higher elevation than in the pit backfill, indicating a gradient from the surrounding bedrock toward the backfilled pit (Figure 5).

3.2 PIT 2

As discussed in the previous section, dewatering in Pit 2 began in March 2014 by pumping from GW-58 after the final target groundwater elevation of 2,662 had been reached and maintained for approximately 2 weeks in the Boyd Pit. The response of groundwater levels to pumping from GW-58 is shown in Figure 6. Prior to initiation of groundwater pumping, there was a gradual steady decline in groundwater levels in all groundwater monitoring wells that ranged from a decline of 1 to 6 feet from September 2013 until March 2014. Although some of this decline may have been a response to dewatering in the Boyd Pit, it should be noted that a gradual decline in water levels was evident in many of the Pit 2 wells prior to the onset of pumping of the Boyd Pit on September 12, 2013. In addition, groundwater levels in many of the Pit 2 wells during that period were at elevations greater than the estimated spill elevation for Pit 2 (2,759 ft amsl). Thus it appears likely that the gradual decline seen in the Pit 2 monitoring well water levels prior to the onset of pumping from GW-58 was the result of seasonal fluctuations Pit 2 groundwater levels.

As noted previously, precipitation during February and early March of 2014 totaled 5.2 inches in addition to any snowmelt induced by the rain-on-snow event. There was a correspondent increase in groundwater levels in all monitoring wells, though the degree of response was different for different groundwater wells. The bedrock groundwater monitoring wells, BOM-2S and BOM-2D, each showed an increase in groundwater level of about 7 feet; whereas the

measured response in other groundwater wells (BOM-12S, BOM-12D, and GW-53) was on the order of 1 foot or so. The bedrock groundwater monitoring wells at BOM-2 are located on the north wall of Pit 2 (Figure 2) and the response observed is the response expected for a fractured bedrock groundwater system with good fracture connection to the surface. The other groundwater monitoring wells in Pit 2 are all located in the deeper part of the pit and the subdued response is reflective of a reduced hydraulic connection to the driving event (e.g., infiltration of precipitation).

There is the potential that the large amount of precipitation coupled with enhanced snowmelt (i.e., rain on snow event) resulted in a significant recharge event to the Pit 2 groundwater system. Groundwater level data were not available from GW-58 during this period, possibly because the transducer may have been undergoing maintenance. Given that the precipitation event occurred over the entire project area, a similar response might have been seen in groundwater levels in the Boyd Pit; however because the Boyd Pit was being dewatered at the time, a similar response on the order of a foot of groundwater level increase would have easily been obscured by the effect of ongoing pumping from GW-54.

Groundwater levels within Pit 2 began to drop with initiation of the pumping in GW-58 on March 20, 2014 (Figure 6). Because the water level in the GW-58 pumping well was already below the first target water elevation (as identified in the BPA Pumping Plan) at the estimated spill elevation for Pit 2 (2,759 ft amsl) prior to pumping, the first target groundwater elevation was skipped. The first drawdown stage rate involved pumping at rates of up to 11 gpm, until the groundwater level reached the target elevation of approximately 2753 ft amsl. Once this target groundwater elevation was reached, the pumping rate was decreased to an average rate of 4.5 gpm in order to maintain the groundwater level in GW-58 at this elevation.

After maintaining this target groundwater elevation in GW-58 for approximately two weeks (i.e., 2753 ft amsl), the second drawdown stage commenced by increasing the pumping rate until the second target groundwater elevation of approximately 2,743 was reached. This second target groundwater elevation was then maintained in GW-58 for approximately two weeks by pumping at an average rate of 3.2 gpm prior to commencing with the third drawdown stage.

For the balance of 2014, a groundwater level between elevation 2,740 and 2,744 was maintained in Pit 2 by pumping from GW-58 at an average rate of approximately 2 gpm. The pumping rates at GW-58 needed to maintain the various target Pit 2 groundwater elevations are summarized in Table 2. As with the analyses of GW-54 pumping rates, average pumping rates shown in Table 2 were calculated at points in time when the measured water level in GW-58 were approximately equal, to the extent possible in order to negate the effects of changes in water storage due to fluctuating water levels.

Table 2 Summary of Pit 2 (GW-58) Water Levels and Pumping Rates

Stage	Elevation Range (ft amsl)	Date Range	Average Pumping Rate (gpm)
1	2,752.42 to 2,752.8	April 8, 2014 to April 21, 2014	4.5
2	2,742.5 to 2,743.4	June 16, 2014 to June 30, 2014	3.2
3	2,739.8 to 2,743.7	June 30, 2014 to Dec 22, 2014	2.0

Groundwater levels in BOM-12S closely tracked groundwater levels measured in GW-58; whereas the deeper bedrock groundwater monitoring well BOM-12D responded hydraulically to the stress imposed by pumping in GW-58, but always indicated higher groundwater levels and corresponding hydraulic gradients toward the shallow bedrock and waste rock zones.

Groundwater levels in the other bedrock monitoring well series showed similar declines in water level with the onset of pumping in both the shallow (BOM-2S) and deep (BOM-2D) well installations, the groundwater water levels in both wells remained well above the groundwater levels in the other Pit 2 wells (Figure 6). Note that for the majority of the time period considered, BOM-2S was dry.

In contrast to the strong hydraulic connection between GW-54 and GW-56 in the Boyd Pit, GW-53 in Pit 2 showed a dampened response to groundwater pumping from GW-58. The lack of response in groundwater levels in GW-53 indicates that the backfill material in Pit 2 is likely more heterogeneous and/or of lower hydraulic conductivity than the backfill material in Boyd Pit; however, because there are only two groundwater wells in Pit 2 that were completed in the waste rock, there is the possibility that GW-53 was completed in a zone of fine-grained material. Another possibility is that previous pumping at GW-53 affected the well screen and/or filter pack by scale formation such that the groundwater levels measured in GW-53 are not reflective of the overall hydraulic response of the Pit 2 backfill to groundwater pumping at GW-58.

Groundwater pumping from GW-58 in Pit 2 induces a hydraulic response from both the bedrock aquifer and the shallower backfill material aquifer. Groundwater levels declined below the Pit 2 spill elevation for all but the BOM-2 bedrock wells. Based on the observed groundwater levels in bedrock wells in Pit 2, an upward hydraulic gradient exists from the deeper bedrock to the backfilled Pit 2. It should be noted that pumping from GW-58 did not appear to significantly affect the direction of hydraulic gradients in the vicinity of Pit 2. This is likely due to the hydraulic control provided by overflow to the Boyd Pit once a groundwater level above approximately 2,759 is reached in Pit 2.

Groundwater levels measured during the period 1999 until 2002 were also evaluated based on the most recent survey of top of casing for the groundwater monitoring well system (Figure 7). During this period, groundwater pumping was conducted in GW-53 and the overall slow response from other groundwater monitoring wells is well demonstrated. However, there may well be a threshold of pit dewatering after which groundwater levels were much more responsive to pumping from GW-53 (Figure 7). Note that the data plotted for BOM-2D and BOM-2S show opposite responses from the data collected more recently; there is potential that the early data were recorded under the wrong name. Also, there is evidence of previous hydraulic events as was observed in Pit 2 in 2014 where a fairly dramatic increase in groundwater level was observed, again with the sharpest response observed in BOM-2 wells and a more subdued response in the wells completed deeper in the Pit 2 area.

3.3 OTHER LOCATIONS

Bedrock groundwater monitoring wells at BOM-3 are located adjacent to the conceptual groundwater flow path that follows from the Boyd Pit to the southeast as shown in Figure 2. There are three nested groundwater wells at BOM-3 and the screen interval in the mid-depth well, BOM-3M, is at an elevation just about the same as the bottom elevation of the Boyd Pit pumping well GW-54.

Measured groundwater surface elevations are shown in Figure 8 for the recent period beginning August 2013 and for the period August 1999 to May 2001. Large changes in groundwater surface elevation are observed with the onset of pumping of the Boyd Pit in the mid-level and deep wells; whereas in the shallow well, there is essentially no fluctuation in groundwater surface elevation with time (Figure 8). Moreover, the measured groundwater levels in the mid-depth and deep wells are very similar and track each other closely indicating the mid-depth and

deep wells are monitoring the same potentiometric groundwater system. The shallow groundwater monitoring well at BOM-3 probably is indicative of a shallow alluvial groundwater system that is not connected to the fracture system(s) intersected by the mid-depth and deep groundwater monitoring wells.

Overall, the changes in groundwater elevation for BOM-3M and BOM-3D are similar for the recent period as compared to the earlier time period with about 25 feet of change that occurred over the course of almost a year. A fractured bedrock groundwater system should transmit hydraulic changes fairly rapidly due to the small storage capacity in the bedrock system, that is, the bulk of the porosity and storage in a fractured bedrock system is contained in the fractures and not the rock. If so, then the steady decline in groundwater elevation, even over the course of the snowmelt period when infiltration/recharge should be the greatest, suggests a different process affecting the hydraulic conditions in the bedrock aquifer. For the most recent period, groundwater was pumped only from the Boyd Pit from September 12, 2013 until March 20, 2014 during which both BOM-3M and BOM-3D experienced approximately 15 feet of drawdown.

Pumping also commenced from Pit 2 on March 20, 2014 and no discernable change in the drawdown plot in Figure 8 can be seen due to this additional pumping. During the earlier (1999 – 2001) BPA dewatering episode, pumping commenced from both Pit 2 and the Boyd Pit on December 22, 1999. Pumping ceased in the Boyd Pit on June 14, 2000 and although pumping continued in Pit 2 until November 11, 2000 the water levels in both BOM-3M and BOM-3D began to rise very soon after pumping in the Boyd Pit had ceased. Pumping in both Pits commenced again on January 31, 2001 and the water levels in BOM-3M and BOM-3D began declining again almost immediately after pumping resumed.

Comparison of the drawdown curves with the pumping history from both BPA dewatering events suggests that although the bedrock groundwater system in the vicinity of BOM-3M and BOM-3D are affected by pumping of the Boyd Pit, the response due to pumping from Pit 2 is very weak, or even non-existent. The much greater influence of pumping from the Boyd Pit on the response of the BOM-3 series bedrock wells is not surprising due to the close proximity of the Boyd Pit to these wells, and due to the fact that even at the final drawdown stage, the water level in Pit 2 was approximately 50 feet higher than the static (pre-pumping) water levels in the BOM-3 series bedrock wells.

Groundwater monitoring wells completed in bedrock and waste rock at BOM-4 (shallow and deep) and GW-43, respectively, are located well down gradient of the Boyd Pit along a buried alluvial channel that is, according the conceptual hydrogeologic model, the source of the submerged seep at the PCP as shown in Figures 1 and 2. Measured groundwater levels in the paired BOM-4 series wells are shown in Figure 9 and indicate an upward hydraulic gradient from bedrock to the waste rock, similar to the other locations. Although a decrease in groundwater levels in all wells occurred during recent BPA dewatering activities, the data indicate that this trend was ongoing prior to commencement of pumping on September 12, 2013 and likely unrelated to BPA dewatering. Likewise, data presented in Figure 9 from the previous (1999 to 2001) BPA dewatering program show that groundwater levels in all three wells fluctuate seasonally, with an increase in water level occurring during springtime of 2000 despite pumping from both Pit 2 and the Boyd Pit. While there is probably no direct hydraulic connection between the pit dewatering system and the groundwater at BOM-4 and GW-43, dewatering of the Boyd Pit below its spill elevation will reduce the overall groundwater supply, which should result in a reduction in groundwater levels and contaminant loading in the bedrock and shallow alluvial aquifers.

4.0 POTENTIAL FOR SCALING

The potential for scaling of well screens, pumps and pipelines was identified as a concern based on previous pumping in the BPA. The performance objectives identified minimization of scaling and fouling as one of the criteria on which the pumping from the BPA would be designed. The current pumping program indicated that while there was some scaling in pumps and pipelines, routine maintenance was sufficient to address this issue. However, it is recognized that future conditions could change and that the distance over which the water from the BPA must be pumped will be greater after Pit 3 is no longer available to receive and store water from the BPA. Therefore, an evaluation to understand theoretical potential for scaling using existing water quality was performed.

The water being pumped from both pits has elevated concentrations of TDS, calcium, magnesium, sulfate, and aluminum. Recorded pH values are below 4. The water chemistry is consistent with the interaction of infiltrating meteoric water with oxidized sulfide-bearing waste rock, with the result that the percolating water leaches oxidation products (mainly iron and sulfate) from the waste rock. Calcium, magnesium and aluminum are also leached from the host rock reflecting more the chemical reactivity of minerals in the Togo Formation than in the porphyritic quartz monzonite.

The potential for scaling or fouling of the dewatering system is influenced by the ambient chemical state of the pumped water with regard to potential solid phases that could precipitate from the groundwater. In the field of geochemistry, the saturation index (*SI*) is used to evaluate the potential for the formation of solids from a given water. The metric is a ratio of the effective concentration (or activity) of constituents in the water of interest to the theoretical concentrations of constituents in thermodynamic equilibrium with a given mineral phase. For example, to evaluate the potential for gypsum ($CaSO_4 \cdot 2H_2O$) formation:

$$SI_{gypsum} = \log \frac{IAP}{K_{sp}} = \log \frac{a_{Ca^{2+}} a_{SO_4^{2-}}}{K_{sp}}$$

Where: IAP = ion activity product (obtained from analysis of water samples)
K_{sp} = theoretical solubility product (i.e., effectively concentrations at equilibrium)
a_x = activity or effective concentration of cation/anion (x) in water

If the product of the effective concentrations measured in the water are the same as the theoretical concentrations (i.e., $IAP = K_{sp}$), the *SI* will be zero and the solution is said to be in equilibrium with the mineral. If the $IAP > K_{sp}$, the *SI* will be positive and the water is said to be oversaturated with respect to the mineral. Conversely, if the $IAP < K_{sp}$, the *SI* will be a negative value and the water is unsaturated with respect to the mineral phase. Details of the calculation of *SI*s to account for all potential minerals phases is well suited to computer programming, and the computer program PHREEQC, which is maintained and supported by the U.S. Geological Survey and is regularly updated to reflect the most recent research (Parkhurst and Appelo, 2013). The geochemical condition of recent groundwater samples from GW-54 and GW-58 was evaluated using PHREEQC. Table 3 is a listing of the water chemistry data from GW-54 and GW-58 used in the geochemical modeling.

Table 3 Recent Water Chemistry Data from GW-54 and GW-58

Location	Date	SO_4	Al	Ca	Cu	Fe	Mg	Mn	Na	Zn	pH
GW-54	9/12/13	10600	631	374	0.169	209	1159	682	39.6	32.0	3.73
GW-54	12/18/13	9200	511	363	0.344	66.1	1019	566	36.6	28.0	3.55
GW-54	1/28/14	8570	394	370	0.375	88.3	905	505	36.3	21.9	3.76
GW-54	2/25/14	7530	413	424	0.785	62.4	901	519	39.9	20.4	3.65
GW-54	3/20/14	6770	352	407	1.39	29.8	848	444	38.0	17.0	3.73
GW-54	4/16/14	8010	309	389	2.02	16.8	735	434	42.3	17.9	3.73
GW-54	5/14/14	7120	312	393	2.59	17.8	696	426	32.9	18.0	3.73
GW-54	6/11/14	7140	333	388	3.01	19.7	718	418	39.7	17.7	3.61
GW-54	7/24/14	7570	431	445	8.08	9.39	888	482	35.3	20.9	3.54
GW-58	3/20/14	6700	219	376	0.397	114	817	412	39.7	8.12	3.76
GW-58	4/16/14	6640	183	359	0.357	129	723	398	43.0	7.71	3.85
GW-58	6/11/14	5270	155	342	0.336	122	645	357	39.6	6.11	3.58
GW-58	7/24/14	4900	136	389	0.186	126	662	332	32.2	5.71	3.85

Notes: all concentration in mg/L except pH

Analysis of the water chemistry as indicated by chemical analysis of samples collected from GW-54 and GW-58 indicate that the groundwater is in equilibrium with regard to gypsum. However, the elevated concentrations of aluminum result in oversaturation with regard to the aluminum hydroxysulfate mineral jurbanite [$Al(SO_4)(OH)\cdot 5H_2O$] and basaluminite [$Al_4(SO_4)(OH)_{10})\cdot 5H_2O$]. It is common to calculate oversaturation for mineral phases based on the measured water chemistry and never see the oversaturated mineral phase; this condition is usually due to reaction kinetic limitations that prevent the formation of the mineral phase. However, both basaluminite and jurbanite are viable mineral phases that form in low pH, aluminum-bearing, acid rock drainage (Nordstrom, 1982). If the groundwater is exposed to atmospheric conditions, i.e., oxygen, then iron sulfate (e.g., jarosite [$NaFe_3(SO_4)_2(OH)_6$] or ferrihydrite [$Fe(OH)_3(a)$]) minerals are oversaturated and would likely precipitate from solution.

Based on results of the geochemical analysis of the measured groundwater chemistry in the Boyd Pit and in Pit 2, there is good potential for there to be scaling and fouling associated with the dewatering system. Routine preventative maintenance, initially on a quarterly basis, of the pumps and piping is recommended for incorporation in the operating plan to ensure efficient operation of the dewatering system.

5.0 CONCLUSIONS

Operation of the BPA dewatering system has generated enough data to allow an evaluation of the ability of the BPA dewatering system to meet the performance standard as outlined in the Consent Decree. The following is a summary of the qualitative observations related to the BPA dewatering system:

- The water levels in wells surrounding the Boyd Pit and Pit 2 indicate the pits act as hydraulic sinks when dewatered. In addition, water levels indicate Pit 2 acts as a hydraulic sink even without active pumping
- Long-term base inflow rates into the Boyd Pit and Pit 2 appear to be lower than previously estimated. The long-term late season (2014) inflow rates were measured to be approximately 2.5 gpm into the Boyd Pit, and the 2.0 gpm into Pit 2.

- Effective hydraulic control of the BPA groundwater system can be accomplished by pumping groundwater from the Boyd Pit only. The volumetric rate of water capture when operating only the Boyd Pit pumping well (GW-54) at the later drawdown stages (6.6 gpm at el 2,662) was similar to the rate of capture when both the Boyd Pit and Pit 2 wells were operating, indicating little or no increase in capture efficiency by operating both well systems. This is consistent with the conceptual flow model based on BPA pre-backfill topography, which indicates water flowing from Pit 2 to the Boyd Pit.
- Pumping from the Pit 2 (GW-58) dewatering well did not significantly affect the direction of hydraulic gradients in the vicinity of Pit 2. Evaluation of paired deep and shallow wells in Pit 2 indicates flow from the deeper bedrock aquifer toward Pit 2 whether or not GW-58 was operational. This is likely due to the water-level control provided by the hydraulic connection to the Boyd Pit once a groundwater level above approximately 2,759 is reached in Pit 2.
- There is connection between the groundwater levels measured in bedrock wells BOM-3M/BOM-3D and the dewatering of the Boyd Pit. However, additional pumping from Pit 2 did not have a noticeable effect on groundwater levels in BOM-3M/BOM-3D. Neither pumping from the Boyd Pit, nor Pit 2 had a measureable effect on the shallow well BOM-3S.
- Groundwater levels down gradient of the Boyd Pit at BOM-4 and GW-43 declined during the recent pumping campaign. Although it is possible this decline may be due to a decrease in the supply groundwater in the shallow buried alluvial channel down gradient of the Boyd Pit after the groundwater level in the pit dropped below the overflow elevation (2,679 ft amsl); it may also be the result of normal seasonal variations. Additional pumping from Pit 2 during the later stages of the BPA dewatering program did not affect the rate of decline in groundwater levels at BOM-4 and GW-43.
- The measured water chemistry at GW-54 and GW-58 indicate a good potential for the formation of scale and potential fouling of the dewatering system (pumps and pipes). Therefore, routine maintenance of the dewatering system is recommended to ensure efficient operation.
- Based on the above observations and conclusions above, it appears that hydraulic control of the BPA can be effectively accomplished by pumping only from the downgradient Boyd Pit and that pumping from Pit 2 can be discontinued. It is recommended that an additional (redundant) dewatering well be installed in the Boyd Pit in the vicinity (within 20 to 25 feet) of the existing GW-54 dewatering well to provide a backup, and to limit the length of disruptions to operation of the dewatering system in the event of a catastrophic well failure.
- The target operating water level range for the Boyd Pit dewatering system should be between 2,660 and 2,665 ft amsl, which is sustainable as a groundwater level within this range was maintained for approximately 9 months during the BPA dewatering program in 2014.
- It is estimated that the long-term, late-season dewatering rate (prior to capping of the BPA) will be approximately 4.4 gpm based upon the current combined flow rates from both the Boyd Pit and Pit 2.

6.0 REFERENCES

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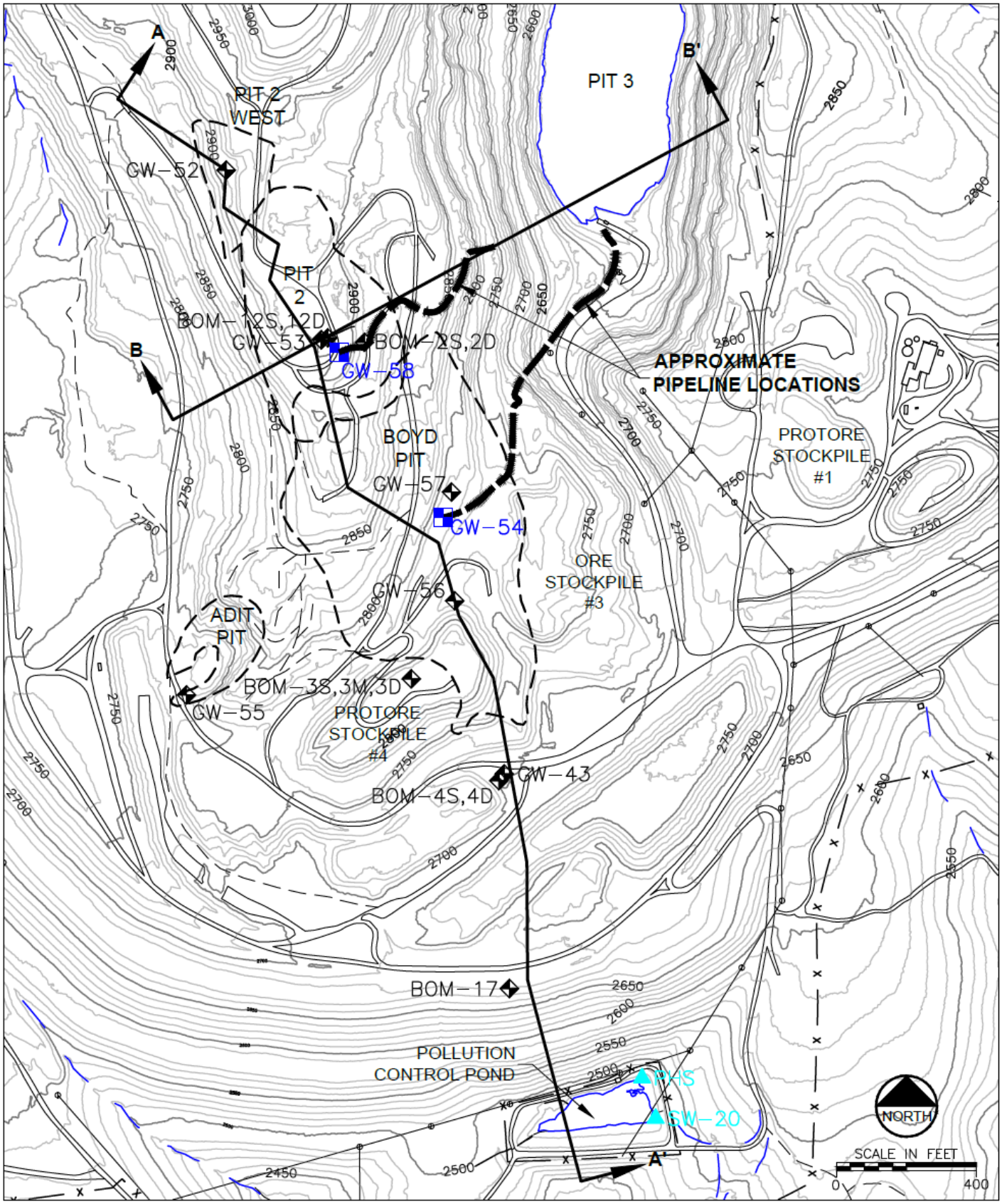
FIGURES



MONITORING WELLS PUMPING WELLS SURFACE WATER MONITORING LOCATIONS

NOTE: CONTOURS SHOWN ARE RELATIVE TO 1983/88 GEODETIC DATUM.

from WME, 2013a

NEWMONT

PROJECT: MIDNIGHT MINE

TITLE: Backfilled Pits Area

MWH

DATE: June 2015

FIGURE #: 1

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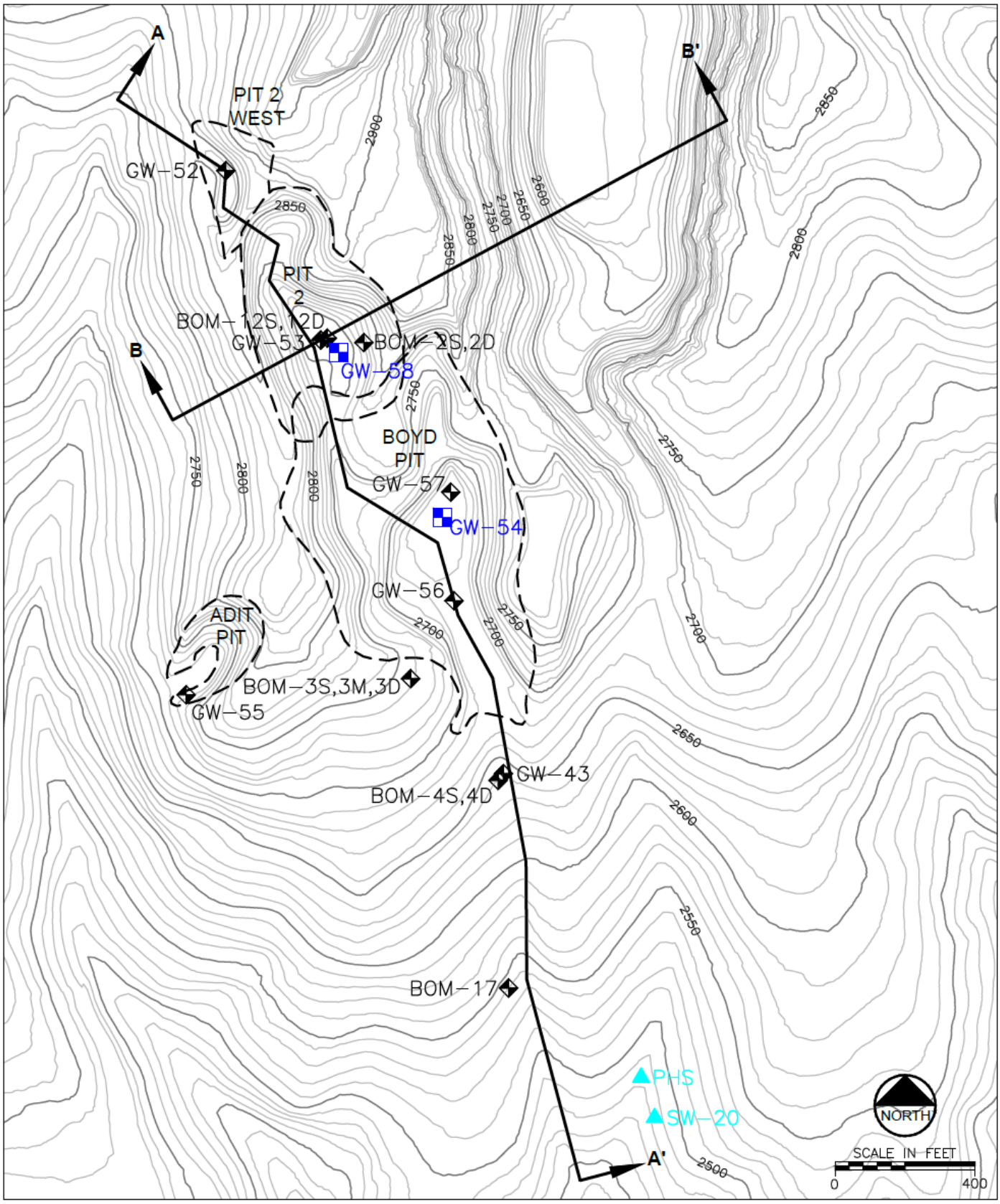

MONITORING WELLS PUMPING WELLS SURFACE WATER MONITORING LOCATIONS

NOTE: CONTOURS SHOWN ARE RELATIVE TO 1983/88 GEODETIC DATUM.

from WME, 2013a

NEWMONT

PROJECT: MIDNIGHT MINE

TITLE: Backfilled Pits Area with Pre-Backfill Topography

MWH

DATE: June 2015

FIGURE #: 2

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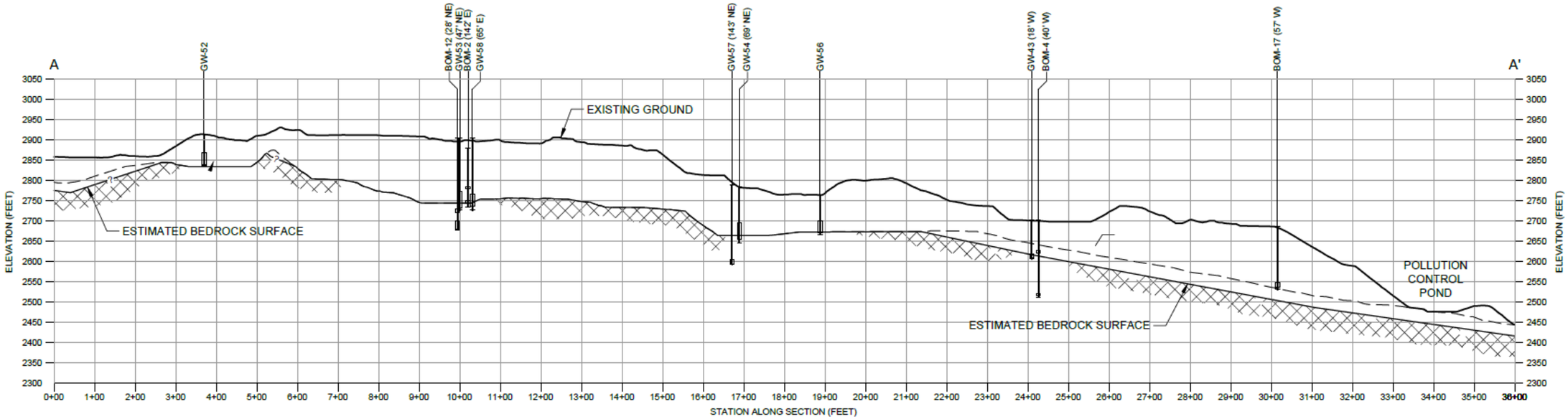

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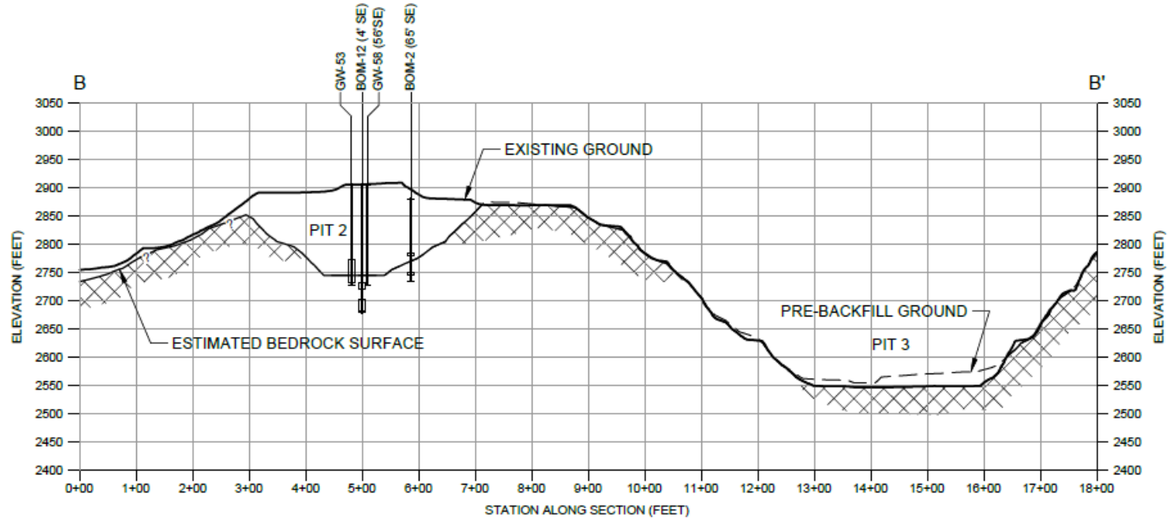

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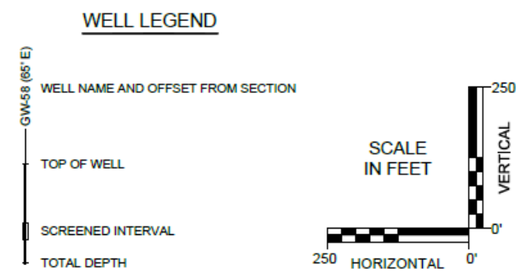

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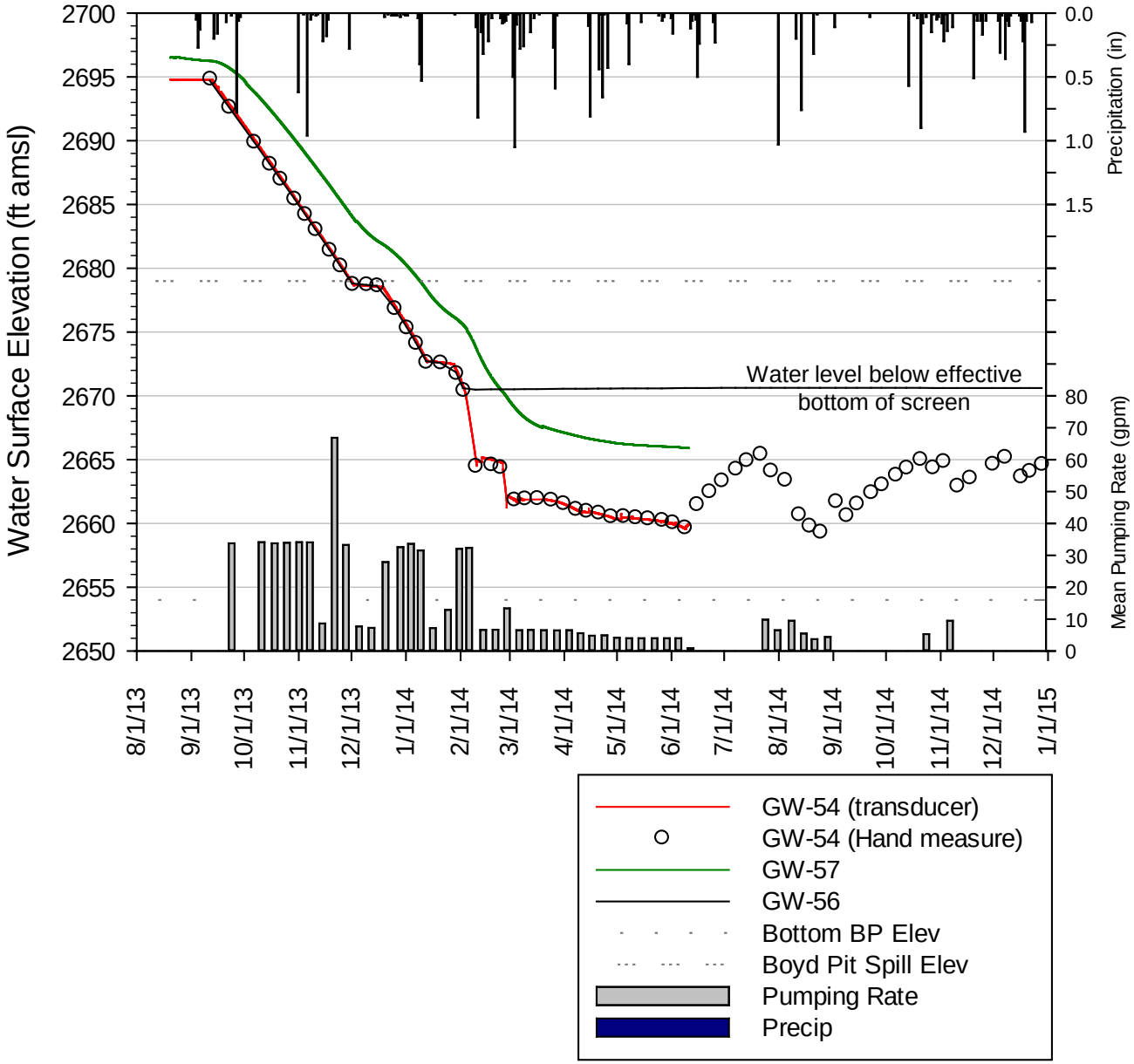

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PROJECT
MIDNIGHT MINE

TITLE
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DATE
June 2015

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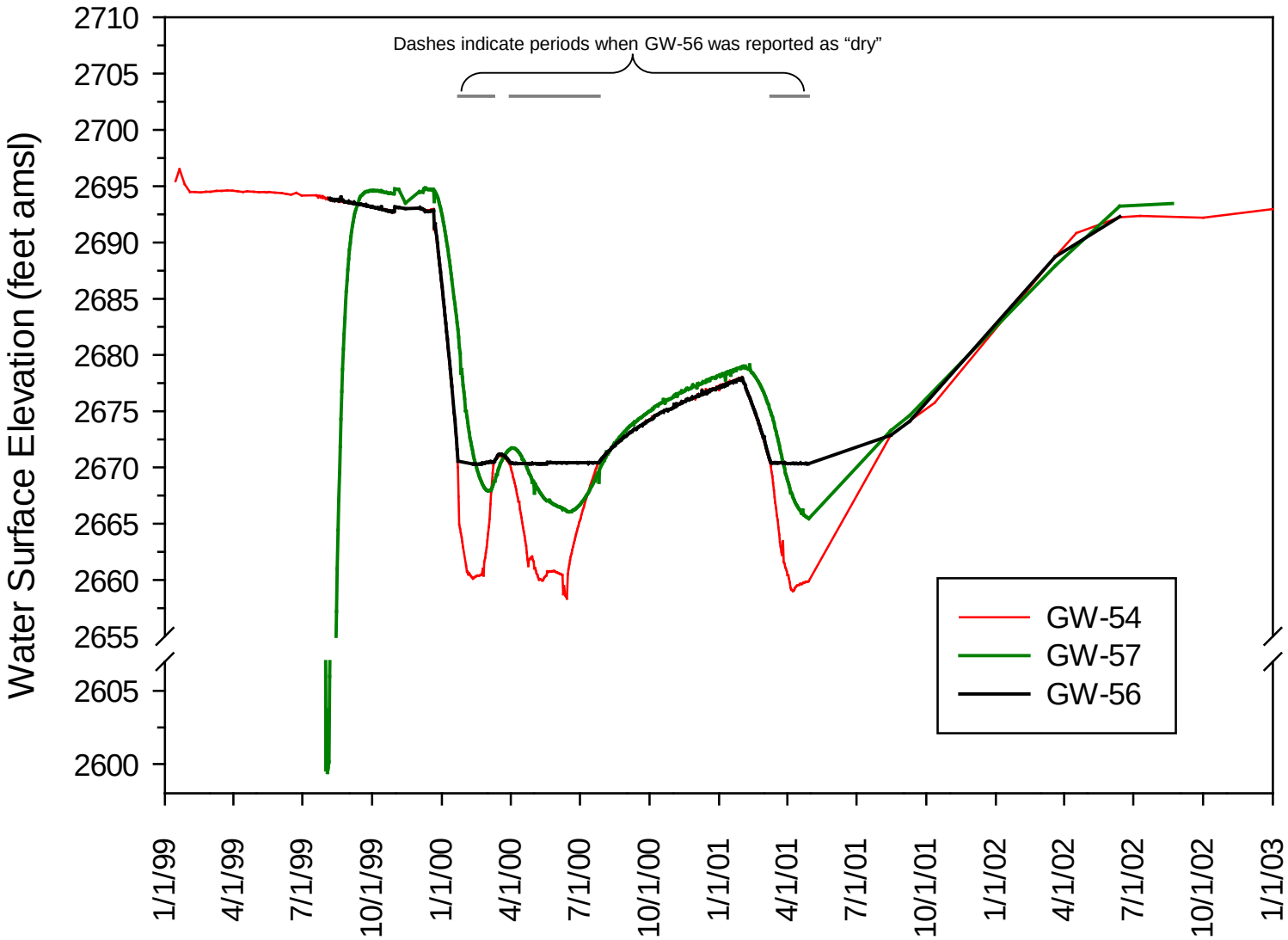

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PROJECT
MIDNIGHT MINE

TITLE
HISTORIC BOYD PIT
GROUNDWATER LEVELS

MWH

DATE
June 2015

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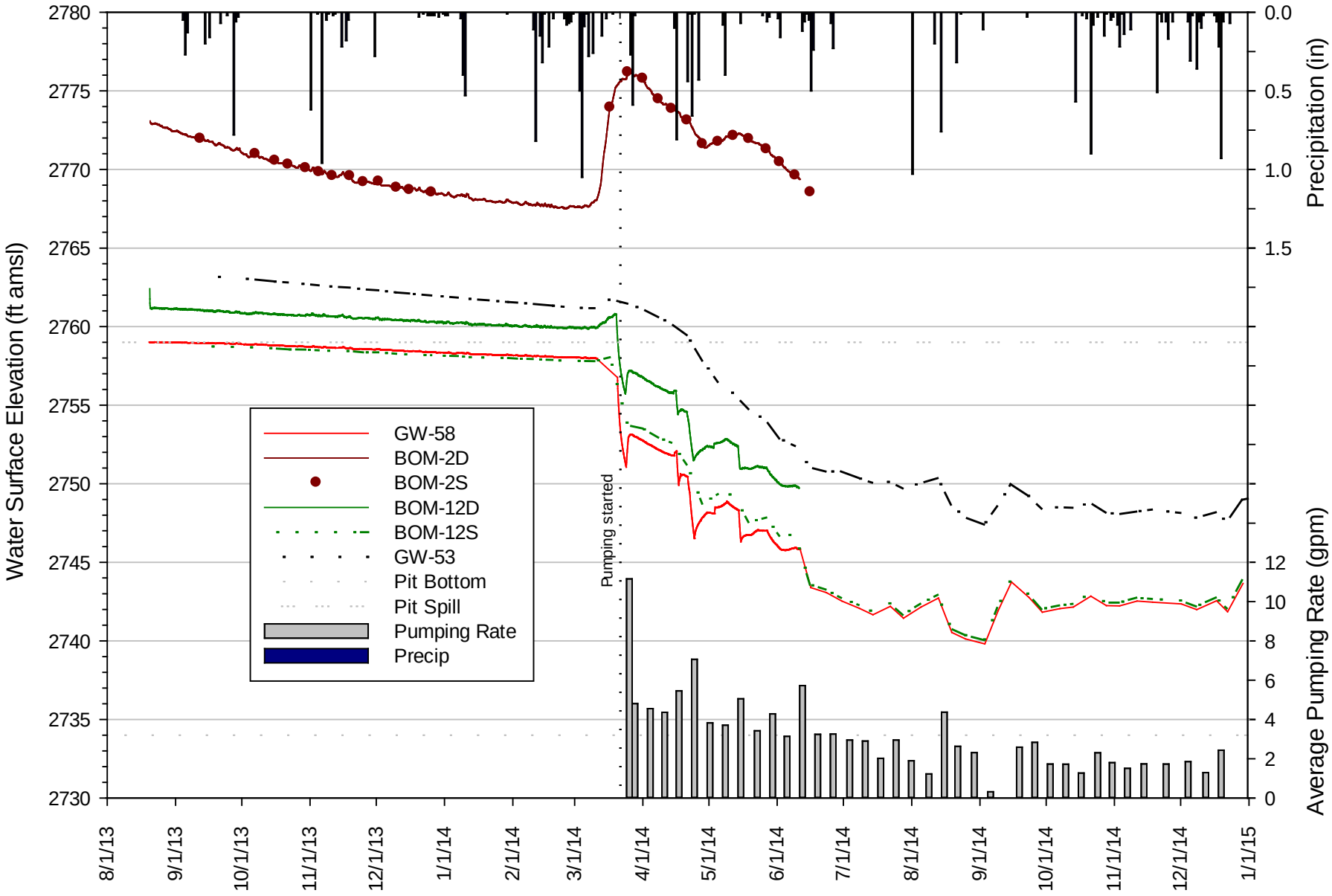


NEWMONT

PROJECT
MIDNIGHT MINE

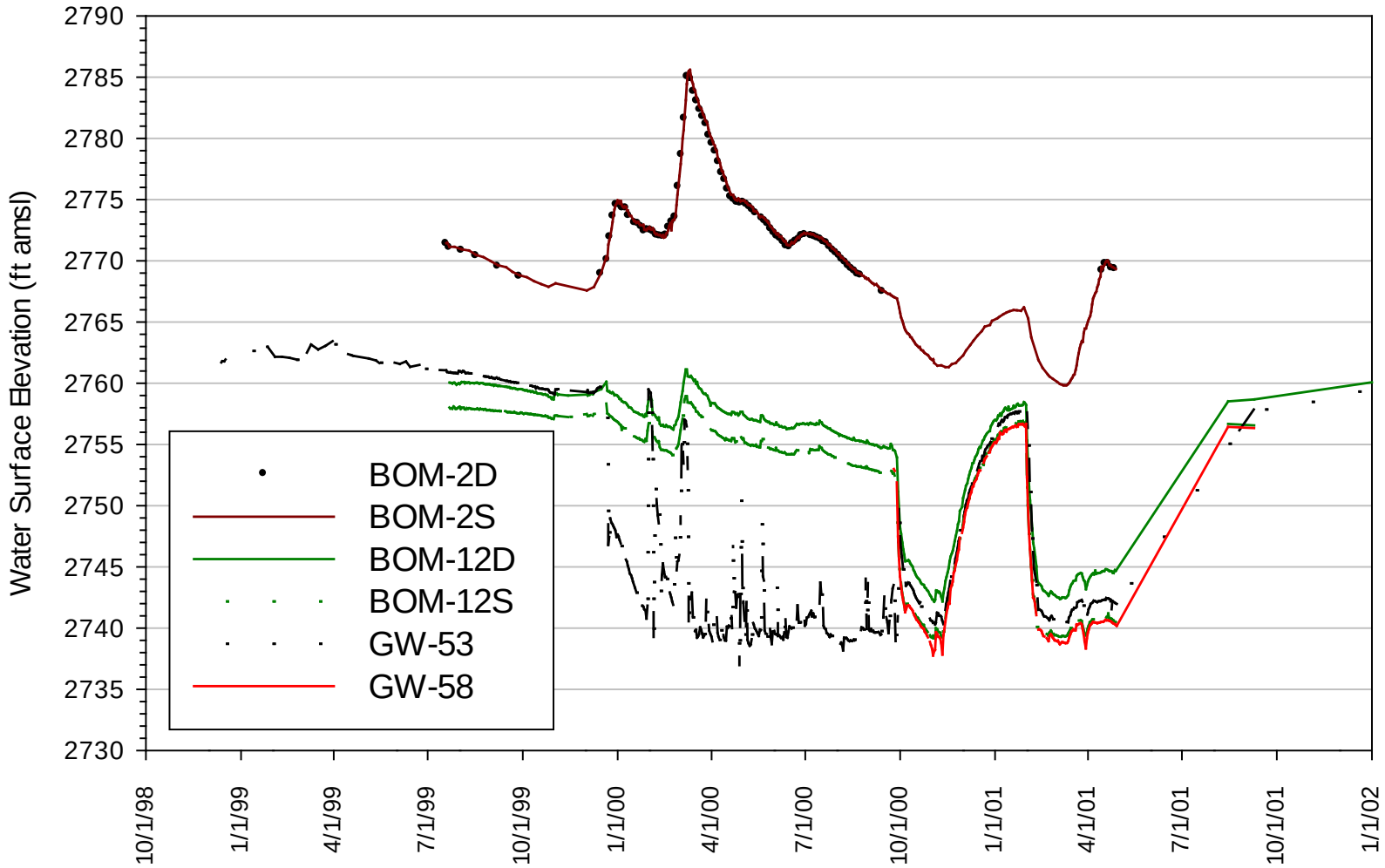
TITLE
PIT 2 GROUNDWATER LEVELS
AND PUMPING RATES 2013& 2014

MWH

DATE
June 2015

6

FILENAME
PitDewateringFigs_I.pptx



Note: BOM-2D and BOM-2S are probably switched based on recent information

NEWMONT

PROJECT
MIDNIGHT MINE

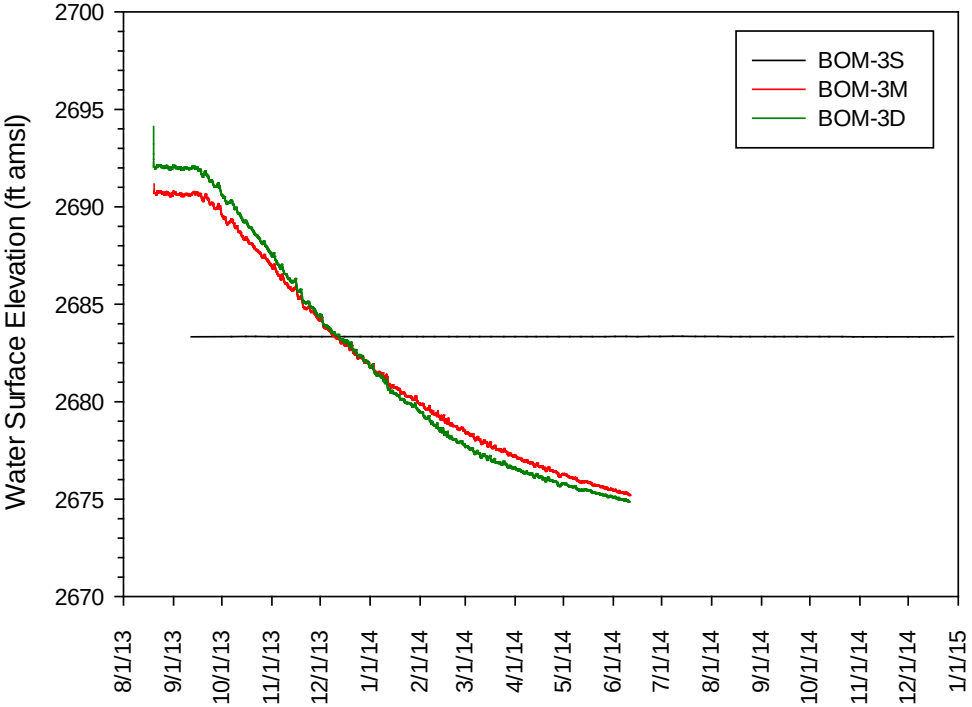
TITLE
HISTORIC PIT 2
GROUNDWATER LEVELS

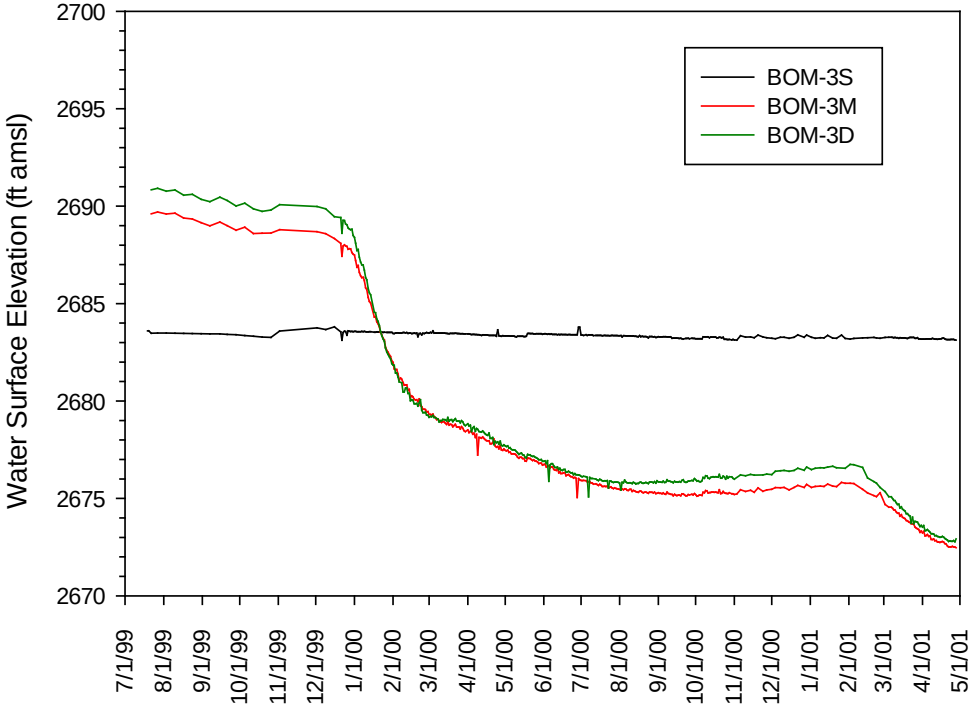
MWH

DATE
June 2015

7

FILENAME
PitDewateringFigs_I.pptx





NEWMONT

PROJECT
MIDNIGHT MINE

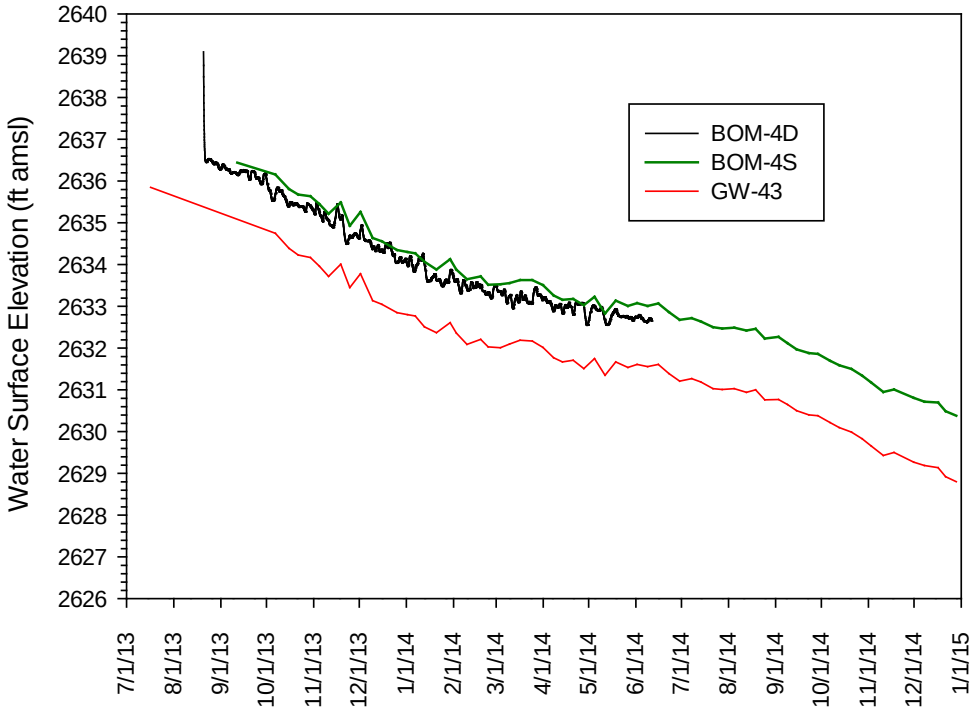
TITLE
RECENT AND HISTORIC GROUNDWATER LEVELS IN BOM-3 WELLS

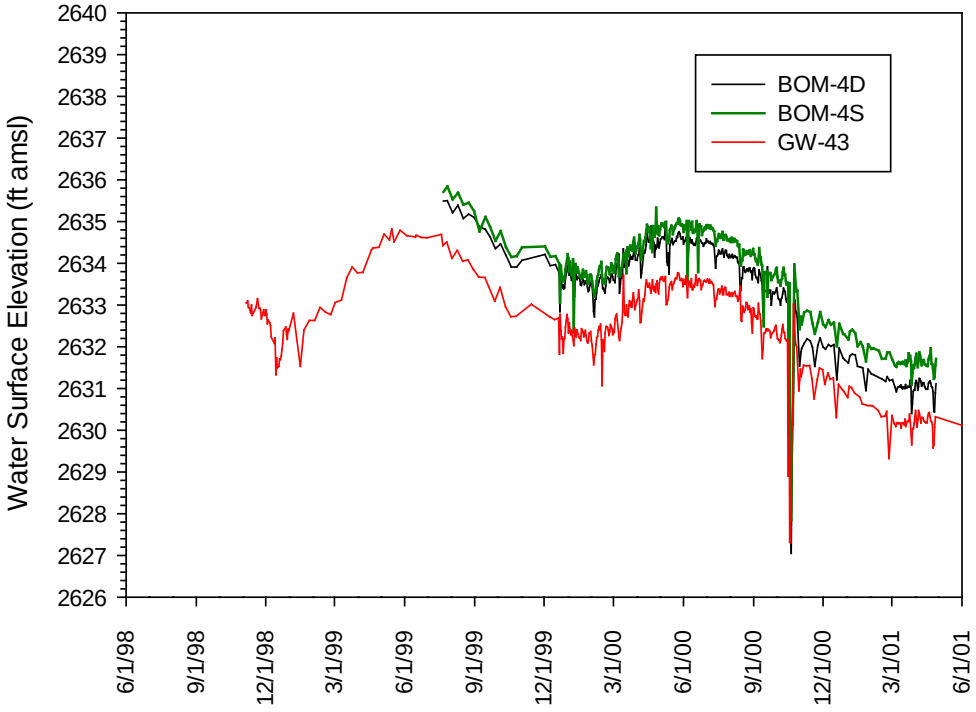
MWH

DATE
June 2015

8

FILENAME
PitDewateringFigs_I.pptx





NEWMONT

PROJECT
MIDNIGHT MINE

TITLE
RECENT AND HISTORIC GROUNDWATER LEVELS IN BOM-4 AND GW-43

MWH

DATE
June 2015

FILENAME
PitDewateringFigs_I.pptx