TAILINGS MANAGEMENT PLAN
AMENDED DECEMBER, 2005
REVISED APRIL, 2007

FOR

SHOOTARING CANYON URANIUM PROCESSING FACILITY

Utah Department of Environmental Quality
Division of Radiation Control
Byproduct Material License No. UT0900480
Division of Water Quality
Discharge Permit Number UGW170003

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1. INTRODUCTION

This submittal of the amended Tailings Management Plan (TMP) for the Shootaring Canyon Uranium millsite is to support the conversion of the present license UT-0900480 from **Standby** to **Operational** Status. The existing TMP was previously submitted to the U. S. Nuclear Regulatory Commission (NRC) and State of Utah Department of Environmental Quality, Division of Radiation Control (DRC) in 1999. This amended plan incorporates many of the general concepts presented in the previous submittal with significant improvements in the approach to tailings management. This submittal amends the plans previously submitted to the U. S. Nuclear Regulatory Commission (NRC) and State of Utah Department of Environmental Quality, Division of Radiation Control (DRC) for the Shootaring Canyon Uranium Mill site. A map of the site and surrounding area with some of the site features is presented in Figure 1-1.

One of the primary proposed improvements in the TMP is the option for Reduced-Moisture Tailings Placement (RMTP). With the RMTP approach, a paste admixture, thickener, screw press, belt press or similar fluid extraction or slurry process equipment is used to extract a significant volume of tailings solution from the tailings slurry yielding moist or paste tailings in semi-solid/solid form and a liquid stream of tailings solution. This in turn allows handling tailings solids with the potential for stabilized placement above grade in the tailings cell(s). The solution extraction from the tailings slurry prior to delivery of the tailings to the cell also reduces the drainage from the in-place tailings and allows segregation of tailings solution for reuse in the mill or delivery to a process solution storage and/or evaporation pond. Additional advantages of this approach include increased tailings solids capacity for each tailings cell which potentially reduces the areal extent of the reclaimed tailings facility, and an enhancement of the stability of the tailings and tailings containment structures.

A seven-part liner with a drainage collection system and leak detection system is used for containment in the tailings cell(s). The proposed liner is discussed in more detail in Section 5.1 of this Tailings Management Plan.

Potentially, three distinct cells, constructed with a seven-part liner, will be used to receive tailings. The first cell to be constructed is designated as the Evaporation and Process Pond Cell (EPPC). This cell will be the repository for the existing tailings and other contaminated materials and will also contain HDPE lined evaporation and process pond(s). Cell 1 will be constructed upstream of the cross valley berm in the basin where the existing tailings are currently located. When additional tailings capacity is needed, Cell 2 will be constructed between the cross valley berm and the Shootaring Canyon Dam and will be contiguous with Cell 1. This configuration will result in a construction sequence that allows transfer of all existing tailings and contaminated material to the EPPC with the Best Available Technology (BAT) liner system prior to the construction of Cell 1.

Construction of Cell 2 can be delayed until additional tailings capacity is needed or foregone entirely if processing is discontinued prior to exhausting the capacity of Cell 1. The resulting complete tailings facility will consist of two or three contiguous cells with a continuous liner between cells.

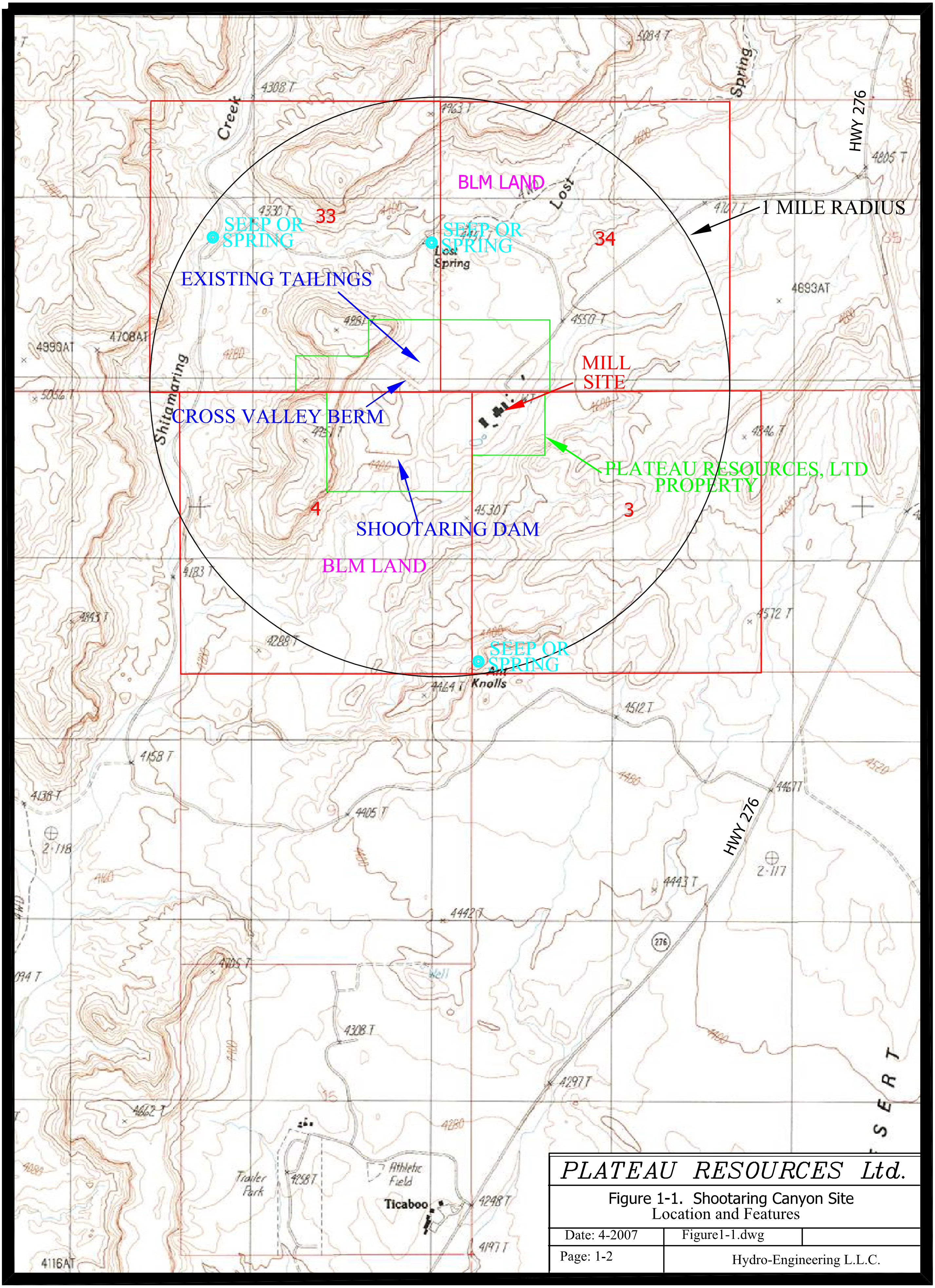

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Figure 1-1. Shootaring Canyon Site
Location and Features

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2. REGULATORY ANALYSIS AND OBJECTIVES OF THE TAILINGS MANAGEMENT AND RECLAMATION PLANS.

2.1 State and Federal Regulations

Prior to the State of Utah obtaining agreement state status in 2004, the tailings at the Shootaring site were regulated primarily by the U.S. Nuclear Regulatory Commission (NRC) pursuant to 10 CFR 40, Appendix A, and the U.S. Environmental Protection Agency (EPA) under 10 CFR 61, Subparts A and W which are administered by the State of Utah Division of Air Quality. Although this recent change has transferred primacy of regulatory authority to the State of Utah, the existing framework of regulations previously administered by the NRC still serves as a useful guideline. The State of Utah will regulate the site according to rules and regulations presented in R313 - Environmental Quality, Radiation Control. These rules include; through reference, clarification or exception; sections of 10 CFR 40 extending through Appendix A. With this in mind, the applicable state and federal regulations are referenced and described in Sections 2.1.1 through 2.1.3.

Additional, enhanced, or modified regulations developed by the State of Utah are discussed in Section 2.2.

NRC and EPA have a Memorandum of Understanding (MOU) that covers joint expectations under what was originally Subpart T of 40 CFR 61 (uranium mill tailings closure) and a generic MOU on elimination of dual regulation. The NRC regulations also incorporate other standards by reference that were promulgated by the EPA pursuant to the Uranium Mill Tailings Radiation Control Act (UMTRCA - 1978), and Section 112 of the Clean Air Act, as amended. Compliance with these regulations under the authority of the State of Utah is provided through R313 and referenced sections of 10 CFR 40.

In the following discussion, applicable state and federal regulations are summarized in **bold** lettering and the means by which this liner plan, the Tailings Management Plan and the Reclamation Plan meet these regulations are discussed immediately below the bold caption.

2.1.1 Utah DRC and NRC Regulations - Guiding Principles

- **Permanent isolation of tailings (10 CFR 40 Appendix A, Criterion 1)**

The tailings will be placed in a lined impoundment, designed and operated to meet all regulations referenced below and reclaimed with a stable cover designed according to applicable regulations, guidelines and NRC staff technical positions. The tailings facility currently exists behind a constructed dam within Shootaring Canyon in Garfield County, Utah. The site is remote, and the nearest residence is located approximately 1.5 miles to the east of the site. There is a small population in the town of Ticaboo, which is located approximately 2.6 miles south of the mill and associated tailings site (see Figure 1-1). Siting criteria were evaluated prior to construction of the existing mill and tailings facility (see Woodward-Clyde 1978a, 1978b, and 1978c).

- **No ongoing maintenance (10 CFR 40 Appendix A, Criterion 1)**

The reclamation design ensures that no ongoing maintenance will be required following reclamation. The tailings will be dewatered to mitigate seepage and tailings settlement. Cover surfaces have slopes designed to be stable under Probable Maximum Precipitation (PMP) flows and the reclaimed tailings surface will be covered with rock mulch or rock riprap to afford erosion protection. A low permeability clay cap and an overlying HDPE geomembrane will control infiltration. These are described in the Reclamation Plan dated December 2005 and subsequent revisions.

- **Tailings disposal (10 CFR 40 Appendix A, Criterion 3)**

The tailings cell(s) are located within a natural drainage behind an existing constructed dam. The cells are surrounded on the east and west sides by bluffs which protect the area from wind erosion and promote deposition. There are currently no nearby active mine pits that would serve as alternate disposal sites. Because the tailings will be contained within a structure using a Best Available Technology (BAT) liner system and will be reclaimed and covered with a multi-layer cover to include a geomembrane and erosion protection rock mulch, the proposed disposal method will minimize the potential for exposure of the tailings or dispersal of the tailings by mechanical forces.

- **Closed with 1000-year design life, and in any case at least 200 years (10 CFR 40 Appendix A, Criterion 6)**

The reclamation design complies with applicable NRC staff technical positions, guidelines and recommendations. See above.

2.1.2 Design Requirements

2.1.2.1 Siting (10 CFR 40 Appendix A, Criterion 4)

- **Upstream drainage minimized**

The tailings impoundment is in a natural drainage enclosed on the downstream end by an engineered, NRC and Utah State Engineer approved dam within a very small watershed runoff area. The total watershed area to the dam is approximately 217 acres. The upper 50 acres of this drainage area will be diverted to a different drainage and, therefore, the effective drainage area is approximately 167 acres. During operations, the runoff will collect in the impoundment and be recycled within the mill process and/or evaporated. After reclamation, runoff will be collected in channels that are located on the periphery of the tailings and diverted to the south where it is returned to the original drainage system.

- **Wind protection**

The tailings disposal basin is effectively surrounded by natural cliffs and hills. A net deposition of windborne soils is expected to occur over the impoundment area, rather than loss of covering over the tailings due to wind erosion. The reclamation plan includes rock mulch over the tailings surface, which will prevent wind erosion of the tailings cover system.

- **Erosion potential limited through flat cover slopes and designed covers**

The final tailings cover will be graded to provide sufficiently flat slopes to mitigate erosional forces but allow precipitation runoff. Rock mulch erosion protection will be included as part of the cover design for the entire tailings area. The top reclamation surface will also be configured to limit upland contributing drainage area to overland flow.

- **Conservative factors of safety attained through flat embankment slopes**

Cell embankments and sides will be designed with sufficiently flat slopes to provide conservative factors of safety.

- **Not susceptible to earthquake damage**

The cell design accounts for stresses induced by the postulated maximum credible earthquake for the Shootaring facility region based on the June 26, 1994 "Seismic Hazard Analysis of Title II Reclamation Plans" by Lawrence Livermore National Laboratory. Additional analyses have been performed including a Newmark deformation analysis requested by the State of Utah Division of Water Rights State Engineer. The slope stability analyses are included in Section 3 of this Tailings Management Plan.

- **Deposition promoted**

Where possible, final cover slopes will be flat enough to promote deposition, and in any case, to limit erosion to acceptable levels during the 1000-year stability period.

2.1.2.2 Ground Water Protection Standards (Utah Administrative Code Rule R317-6, 10 CFR 40 Appendix A, 40 CFR 192, etc.)

- **Liner that will prevent migration of wastes out of the impoundment (Utah Administrative Code Rule R317-6).**

The cells are designed with a competent liner system (double HDPE liner with leak detection and sub-clay liner) to prevent migration of wastes from the cells. The liner will be constructed of materials that have the appropriate chemical and physical properties to prevent failure per 10 CFR 40 Appendix A Criterion 5(a)(2)(a) (see Section 5.1.3.1 through 5.1.3.4 and Appendix C). The liner will be placed on a competent foundation or base pursuant to 10 CFR 40 Appendix A Criterion 5(a)(2)(b) (see Appendix I, Appendix C, and Section 5.2.5). The dikes impounding the tailings have been designed, constructed and maintained with sufficient structural integrity to prevent massive failure pursuant to 10 CFR 40 Appendix A Criterion 5(a)(5**).** The cross valley berm will be reconfigured according to criteria described in Section 3.2 and Appendix A. Site licensed activities are administered under Ground Water Quality Discharge Permit UGW 170003, and the requirements regarding potential discharges from the facility are contained therein.

- **If liner left in place following operations, wastes cannot migrate into liner during active life of facility (10 CFR 40 Appendix A, Criterion 5A(1))**

The proposed design will prevent the migration of wastes into the liner during and following operations. The operation of the leachate collection system will continue until the drainage rate is minimal. The post-closure cover system will limit infiltration to immeasurably small levels. The volume of free liquids within the cell after closure will be very small.

- **Impoundment must not be overtopped (10 CFR 40 Appendix A, Criterion 5A(4))**

Minimum impoundment freeboard to store PMP inflow and operational water as well as to allow adequate height for wave action is included in the design.

- **Leakage detection system required for synthetic liners (Utah Administrative Code Rule R317-6, BAT requirement).**

A leakage detection system will be provided, independent of any ground-water monitoring program.

- **Tailings must be dewatered by a drainage system at the bottom of the impoundment (Utah Administrative Code Rule R317-6, BAT requirement).**

A leachate collection system will be installed in the tailings cells and operated until the drainage rate approaches minimal levels.

- **Must install two or more liners and a leak collection system between such liners (Utah Administrative Code Rule R317-6, BAT requirement).**

Requirement satisfied by:

- A double synthetic liner with leak detection system will be installed over a one-foot compacted clay base as described in this TMP.

- A leachate collection system will be installed in a filter/drainage bed over the double liner and clay base.

2.1.2.3 Closure (10 CFR 40 Appendix A, Criterion 6 and as Directed by NRC Staff Technical Position [STP] for Erosion Protection covers)

- **Eliminate free liquids**

With the RMTP method, all free liquids will be diverted to a HDPE lined storage pond within the tailings area. In the event that slurry is discharged to the general tailings area (i.e. the RMTP method is not used), the volume of free liquids in the decant pool will be minimized in the tailings cells during operations by dewatering with the leachate collection system. Operation of the leachate collection system will be continued until the collection rates stabilize at levels of less than 1.5 gpm per leachate collection sump or 10% of the typical full production operational collection rate, whichever is smaller.

- **Stabilize wastes**

Tailings will be allowed to stabilize prior to placement of the reclamation cover. The method of tailings deposition will promote rapid tailings consolidation.

- **Cover the impoundment to:**

  - **Minimize long-term liquid migration**
  - **Promote drainage and minimize erosion**
  - **Accommodate settling and subsidence**
  - **Maintain effectiveness with minimum maintenance**

The final cover will be designed: (1) with a HDPE geomembrane and low permeability clay cap to minimize infiltration and radon gas flux; (2) to not require post-closure maintenance due to its conservative erosion-resistant design; (3) to promote drainage while minimizing erosion through flat slopes and/or rock protection; (4) to control run-on and drainage of waters and (5) to accommodate any tailings settlement. Further, the site is located in a geographical area where annual evaporation (greater than 70 inch/yr.) exceeds the sum of annual precipitation, (conservative estimate of 7 inch/yr.).

2.1.2.4 Radon Standards

- **Post-operations (40 CFR 61, Subpart T; currently EPA - NRC MOU):**

  - **radon emissions not to exceed 20 pCi/m^2-s**
  - **must be in compliance 7 years after ceasing to be operational**

The reclamation cover design incorporates a radon barrier capable of reducing emissions to levels below the radon standard for the required time period while reducing infiltration of surface waters into the cell.

2.1.3 EPA Regulations (40 CFR 61, National Emission Standards for Hazardous Air Pollutants [NESHAPs])

Any modifications to the existing cells shall be in accordance with 40 CFR 61. Operations, maintenance and monitoring of the facility shall comply with 40 CFR 61.

2.1.4 10 CFR 40 Appendix A Criterion 6 through Criterion 10

Criterion 6 - Closure Cover. The closure cover design is described in the "Tailings Reclamation and Decommissioning Plan for Shootaring Canyon Uranium Project – 2005, Revised: December 2006" and as subsequently revised.

Criterion 7 – Preoperational Monitoring. The mill and the major tailings impoundment structures exist at the site. Pre-construction monitoring was conducted, and the ongoing monitoring program including proposed changes is described in following sections.

Criterion 7A – Detection Monitoring. The ground-water monitoring program is discussed in detail in Section 7 and is administered under Ground Water Quality Discharge Permit UGW 170003.

Criterion 8 – Airborne Emissions. Airborne emissions related to the tailings facility are associated with dust and windblown tailings. Placement of tailings as a paste is expected to result in crusting that limits dust and windblown tailings. Commercial dust suppression agents will be applied during operations if necessary. If needed, an interim tailings cover may be used to reduce particulate emissions to ALARA levels.

Criterion 8A – Daily Inspection of Waste Retention Systems. The Standard Operating Procedure (SOP) for dam and facilities inspection is under development as described in Section 5.4.

Criterion 9 – Financial Surety. The financial surety for decontamination and decommissioning is described in the "Tailings Reclamation and Decommissioning Plan for Shootaring Canyon Uranium Project – 2005, Revised: December 2006" and as subsequently revised.

Criterion 10 – Long-Term Surveillance. The CPI adjusted long-term surveillance fee is included in the financial surety described in the "Tailings Reclamation and Decommissioning Plan for Shootaring Canyon Uranium Project – 2005, Revised: December 2006" as subsequently revised.

2.2 State of Utah Regulations

The State of Utah entered into an agreement with the NRC in 2004 that resulted in the State of Utah assuming primacy in the regulation of uranium milling and tailings facilities. With this agreement, the applicable regulations as cited in Section 2.1 and any modifications or additions are under the administration of the State of Utah.

2.2.1 Ground Water Protection (Utah Administrative Code Rule R317-6)

The administrative rule stipulates that any newly constructed facility which discharges or would probably result in a discharge of pollutants that may move directly or indirectly into the ground water must apply for a ground water discharge permit. The rule identifies a broad range of facilities to which it applies, and specifically includes facilities with waste storage piles, landfills and dumps, mining, milling and metallurgical operations. The rule also requires that any facility constructed or operated before the rule was enacted (August 1989), must submit a notice of the nature and location of any discharges to the state within 180 days of the adoption of the rule, and submit an application for a discharge permit upon notification by the state. The design of the seven-part-liner system, as outlined within this TMP, will prevent discharge of pollutants either directly or indirectly into the ground water for this milling operation. The site is administered under Ground Water Quality Discharge Permit UGW 170003.

3. TAILINGS IMPOUNDMENT STRUCTURE DESIGN FEATURES

3.1 Dam Stability Analysis

The design, construction and inspection of the existing tailings-embankment-retention system includes construction methods and hydraulic, seepage, stability, seismic and settlement analyses. Most of these items have been addressed in the following reports: *Preliminary Geotechnical Engineering Report Shootaring Canyon Uranium Project Garfield County*, Utah Woodward-Clyde, April 1978; *Tailings Management Plan and Geotechnical Engineering Studies Shootaring Canyon Uranium Project Garfield County*, Utah Woodward-Clyde, September 1978; *Stage I – Tailings Impoundment and Dam Final Design Report Shootaring Canyon Uranium Project Garfield County, Utah* Woodward-Clyde, May 1979 and *Earthwork Quality Control Overview and As-Built Drawings Construction of Stage I Tailings Impoundment and Dam Shootaring Canyon Uranium Project Garfield County, Utah* Woodward-Clyde, July 1982. Recent reviews of the seismic stability and settlement analysis have been completed and are included in this section. The consulting engineering firm of Inberg-Miller Engineers (IME) completed the analysis with results that show the tailings dam has a safety factor of 1.14 at a horizontal seismic coefficient of 0.19g. At the request of the State of Utah, Department of Natural Resources (UDNR), Division of Water Rights (State Engineer), a deformation analysis was performed for the existing dam using the Newmark method with a specified magnitude 6.5-earthquake and peak ground acceleration of 0.33g. In contrast to the seismic stability analysis by IME which determines the factor of safety with respect to structure failure, the deformation analysis predicts the potential displacement of the top of the dam under the prescribed earthquake magnitude and ground acceleration. The resulting predicted displacement from this deformation analysis is 1.9 inches, and as indicated in the IME letter reports (Appendix A: Inberg-Miller Engineers letter reports dated January 9, 1997, December 11, 1997 and January 28, 1999), is not significant to the integrity or performance of the dam. On March 8, 1999, the UDNR Division of Water Rights determined that the Shootaring Canyon Mill Tailings Dam meets the stability criteria adopted by their office (Appendix A, Section A.6, UDNR Division of Water Rights letter dated March 8, 1999).

In January 2007, IME performed an additional stability analysis of the Shootaring Canyon Dam with the maximum plausible utilization of tailings capacity. The assumptions in the analysis included: (1) the dam crest was raised 30 feet to the stage II configuration, (2) the upstream face of the dam was buttressed to flatten the slope to 3H:1V in accordance with the lined cell configuration, and, (3) the RMTP was utilized to place tailings behind the dam to a height of 25 feet above the dam crest at a slope of 5H:1V. The analysis with this configuration represents a conservative evaluation of proposed changes in the dam and Cell 2 configuration. The resulting safety factor was 1.18, which is slightly larger than the original analysis under the existing configuration. The letter report conveying the results of this analysis is included in Appendix A.

3.2 Cross Valley Berm Analysis

The cross valley berm was evaluated for stability by IME on June 14, 1999. This evaluation found that using a seismic coefficient of 0.19g, reshaping is necessary to stabilize the berm... The specifications and reshaping recommendations are provided in the Slope Stability Analysis Cross Valley Berm Letter Report (see Appendix A). During construction of Cell 1, the upstream and downstream slopes of the cross valley berm will be flattened to a steepest slope of 3H:1V. This is a more conservative and stable condition than the recommended steepest slope of 2H:1V provided in the stability analysis report. The material specifications, alignment and construction procedures for reconfiguration of the cross valley berm will meet or exceed those presented in the IME letter report of June 14, 1999 (Appendix A). The combination of construction meeting or exceeding the requirements of the IME seismic stability analysis and additional slope reduction to 3H:1V will produce a reconfigured cross valley berm having a stability analysis factor of safety that is significantly greater than one.

A deformation analysis using the Newmark method was also performed for the cross valley berm. The IME report of this analysis is included in a letter dated June 14, 1999 in Appendix A. The prescribed earthquake magnitude of 6.5 with a peak ground acceleration of 0.33g resulted in a predicted displacement of 2.8 inches. The IME letter report concluded that this displacement is not significant to the integrity or performance of the berm.

3.3 Sequence of Existing Facility Stability Analyses

The following listing indicates the sequence of analyses and reporting for the stability and deformation evaluation of the Shootaring Dam and the cross valley berm.

- January 9, 1997 - IME performs a “seismic stability analysis” of the Shootaring Canyon Dam using the program PCSTABL ver. 5M with the Bishop and Janbu methods. It was assumed a full tailings pool was present and the horizontal seismic coefficient of 0.19g was based on “Seismic Hazard Analysis of Title II Reclamation Plans” by Lawrence Livermore National Laboratory. The lowest safety factor indicated in the analysis was 1.14.

- June 14, 1999 revision of May 2, 1997 letter – IME performs a slope stability analysis of the Cross Valley Berm using the same methods used previously for the Shootaring Canyon Dam. A horizontal seismic coefficient of 0.19g was used in the analysis. Based on the analysis, IME states that the lowest calculated safety factor was 1.02 for the Cross Valley Berm that has been reconfigured according to their recommendations. The Cross Valley Berm will be reconfigured to the prescribed configuration during construction of Cell 1 as presented in the TMP.

- December 11, 1997 - IME performs an updated analysis of the Shootaring Canyon Dam stability using revised soil strength parameters of the dam core material. Other parameters and methods in the slope stability analysis remain unchanged. The results of the seismic stability analysis were unchanged from the January 9, 1997 analysis.

- July 1, 1998 - PRL receives comments from the UDNR Division of Water Rights (State Engineer) on the previously submitted dam stability analysis. One of the three comments received requires a <u>deformation analysis</u> with a magnitude 6.5 or greater event and a peak site acceleration of about 0.33g. The other two comments are related to phreatic surface and pore pressure and are rendered moot when the details of the liner system are provided to the reviewer.

- January 28, 1999 – IME performs a Newmark deformation analysis of the Shootaring Canyon Dam to calculate the potential displacement under a specific seismic event. A peak ground acceleration of 0.33g and a magnitude 6.5 event were used as required by the State Engineer. The source of the peak ground acceleration was a general regional map produced by the USGS. IME stated in the letter report that the calculated potential displacement of 1.9 inches was not significant to the integrity or performance of the dam.

- March 8, 1999 - PRL receives a letter from the State Engineer indicating that the dam stability analysis and responses to the July 1, 1998 comments were acceptable.

- June 14, 1999 – IME performs a Newmark deformation analysis of the Cross Valley Berm to calculate the potential displacement under a specific seismic event. The peak ground acceleration of 0.33g and a magnitude 6.5 event were used as required by the State Engineer for the Shootaring dam. IME stated in the letter report that the calculated potential displacement of 2.8 inches was not significant to the integrity and performance of the berm.

- January 11, 2007 - IME performs a "seismic stability analysis" of the Shootaring Canyon Dam using the program SLOPE/W with Janbu method. It was assumed that: (1) the dam was raised to full stage II height, (2) the slope of the upstream face was reduced to 3H:1V with buttressing, and (3) the tailings was placed to a height of 25 feet above the dam crest at a slope of 5H:1V. Other dam material properties and the horizontal seismic coefficient were the same as those used in the 1997 analyses. The safety factor indicated in the analysis was 1.18.

3.4 Other Structures

The EPPC will be constructed largely as a below-grade excavation in the broad swale between the mill area and the existing tailings impoundment. There will be a relatively minor berm on the southwest side of the EPPC. The upstream and downstream slopes of the berm will be at a steepest slope of 3H:1V with a crest width of 20 feet. The existing tailings and other contaminated material will be excavated and transferred to the EPPC prior to the construction of Cell 1, thus all material will be placed in the EPPC in dry form.

The EPPC will have a leachate collection system and leak detection system to maintain the tailings in a dewatered state. These systems are expected to function primarily as a contingency for minor leakage from the lined evaporation pond(s) or the very small contribution of incident precipitation on the exposed surface of the EPPC. Figure 3-1 presents a cross section through the central portion of the berm that forms the southeastern boundary of the EPPC.

There are several factors which lend stability to the structure. The small berm height in combination with the mild (3H:1V) upstream and downstream slopes results in a very stable structure. In addition, excavation of the EPPC results in a berm outslope toe that is more than eight feet above the upstream base of EPPC toe with an extended mildly sloping (approximately 100 feet at 5% slope) transition to Cell 1 beyond the western berm toe. The existing tailings and contaminated materials will be transferred to the EPPC in dry form and evaporation and process ponds will be lined with HDPE, so the material in the EPPC will be maintained in a dewatered condition by the leachate collection system. The favorable outslope toe conditions in combination with the small structure height and internal material stability yields an impoundment with little or no concern for berm displacement or failure. Cell 1 will also serve as additional containment for the area upstream of the cross valley berm, and the Shootaring dam will serve as final containment for the tailings facility.

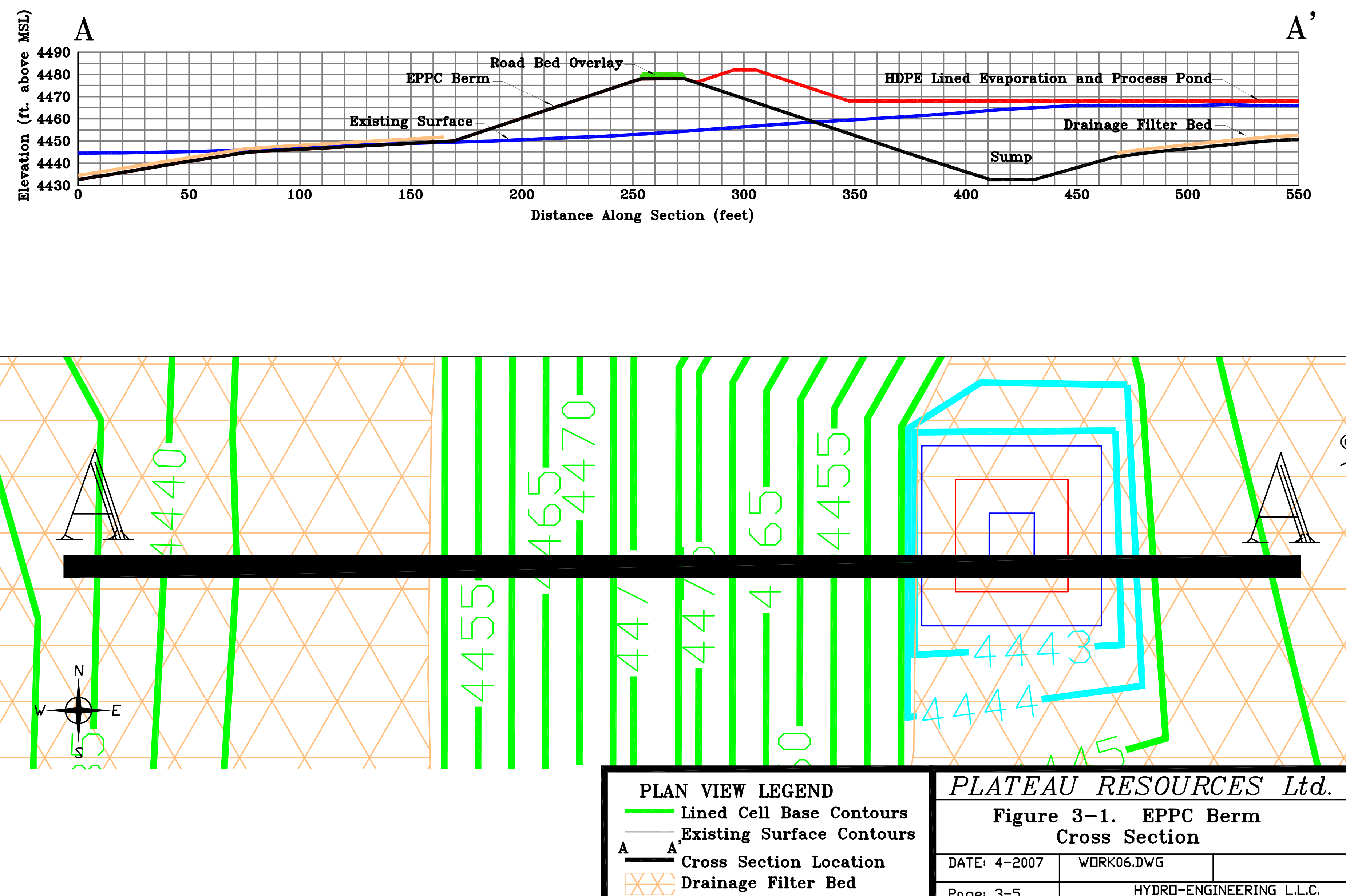

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Figure 3-1. EPPC Berm Cross Section

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4. CONTROL OF LIQUID AND SOLID EFFLUENTS

The following section discusses the above-grade retention systems used to prevent the release of liquid or solid mill-related waste to ground water and offsite areas. NRC Regulatory Guide 3.11, "Design, Construction and Inspection of Embankment Retention Systems for Uranium Mills" and the Utah Water Quality Discharge Permit served as a guide for these sections. Further details on the existing tailings impoundment system are presented in the referenced support documents.

4.1 WATER RESOURCE PROTECTION

4.1.1 Seven-Part Liner

The TMP for the Shootaring Canyon uranium project has been developed to prevent tailings related impacts to ground water. A lining system consisting of a 12" minimum clay base under a double HDPE liner with leak detection over the natural sandstone of the impoundment area will prevent seepage from the tailings impoundment into the foundation rock (see Figure 4-1). To reduce the amount of tailings liquids available for seepage from the impoundment, the tailings slurry will be processed through a belt press or other fluid extraction equipment to remove the majority of the liquid and divert it to a process storage pond or other storage vessel for recycle to the process circuit. Also, tailings liquid collected in the leachate collection system of the impoundment will be recycled to the process circuit or discharged to evaporation ponds for disposal. During initial tailings placement for a particular cell, the tailings will be placed over the base of the cell with deliberate distribution to provide access roads/points and to extend over the exposed HDPE liner on the 3H:1V slope areas where there is no drainage blanket. The initial tailings placement will occur from a constructed access point and in a maximum practical lift thickness to extend over as much of the exposed liner as possible. This initial lift will anchor the liner system and reduce the potential for lateral slippage and liner damage. After the initial lift is placed, moist tailings will then be conveyed to the tailings area in a form ranging from paste to solid and placed in a selected area in a six-inch to several feet lift.

The anticipated RMTP method is a paste technology (see Appendix G). Using paste processing equipment, various cementing and fixing agents can be added to the reduced-moisture tailings stream to produce a partially cemented and erosion resistant emplaced material. In the event that the reduced-moisture tailings are not adequately cemented or fixed after placement, a commercial dust suppression agent will be applied to areas of the tailings as required to minimize wind blown tailings. Following the cessation of tailings placement in a cell, the average moisture content in the tailings will be slightly greater than the expected long-term moisture content for the tailings. Hence, the tailings will be almost completely dewatered when the use of the cell is discontinued. At the time of reclamation, the tailings area will be dewatered of drainable water, further limiting the amount of water which may seep from the tailings impoundment.

At the project site, net evaporation from exposed water surfaces will average approximately 70 inches (177.8 cm) per year, which is equivalent to approximately 3.6 gallons (13.63 l) per minute per acre of exposed surface. Given an ore processing rate of 1,000 tons (907 mt) per day, and assuming a tailings slurry containing 49 percent solids by weight, approximately 173 gallons (655 l) per minute of tailings liquid will be delivered to the processing area where the paste processing, screw press(es), belt press(es) or other fluid extraction equipment will be located. In the event that the operation of the fluid extraction equipment is temporarily suspended or terminated, the tailings slurry will be delivered directly to the tailings cell. During normal operations, it is anticipated that the fluid extraction equipment will reduce the moisture content of the tailings solids to a target level of 20 to 40 percent by weight. Dense, settled tailings are expected to have retained long-term moisture content of 15 to 35 percent. Based on this assumption, approximately 131 to 63 gallons (496 to 238 liter) per minute will be recycled to the mill and approximately 42 to 110 gallons (159 to 416 liter) per minute of the tailings liquid will be retained in the tailings. There will be some post-placement reduction in moisture content of the tailings due to drainage and evaporation and this is expected to be equivalent to 12 to 81 gallons (45 to 307 liter) per minute when expressed as rate or fraction of the process stream.

Since the TMP provides a means for disposing of all excess tailings liquids during the project operation, no significant amount of free tailings liquid will remain in the impoundment to seep into the ground water at project termination. Also, after the project is terminated, normal evaporation from the tailings cover will help to dispose of the incident precipitation. The slope of the final reclaimed surface will help to reduce infiltration by shedding precipitation off the reclaimed facility. To prevent the “bathtub” effect from occurring, a detailed infiltration model was completed for the cover system which includes a geomembrane. This modeling indicated infiltration will be reduced to as low as achievable. The Tailings Reclamation Plan (TRP) includes a discussion of infiltration modeling and the potential accumulation of infiltration within the lined cell. Limited potential for ground water impacts from this project exists, and the requirements for surveillance of the ground water of the area will be minimal. Ground-water monitoring wells, located near the impoundment perimeter to monitor potential seepage from the basin during project operation, will be maintained and be available for future ground water monitoring.

CFR 40 Appendix A requires the use of a liner under the tailings that "is designed, constructed, and installed to prevent any migration of wastes out of the impoundment to the adjacent subsurface soil, ground water, or surface water at any time during the active life” (including the closure period) of the impoundment. The installation of the double liner system as described for of the tailings impoundment would preclude any seepage from those areas.

The double liner with leak detection system design is the Best Available Technology (BAT) and comparable to similar facilities in the industry. The design allows for verifying on a continuous basis that the ground-water protection levels are not being exceeded. The use of HDPE geomembrane material offers superior performance by maintaining the highest standards of durability and the low permeability provides assurance that the leachate will not penetrate the liner.

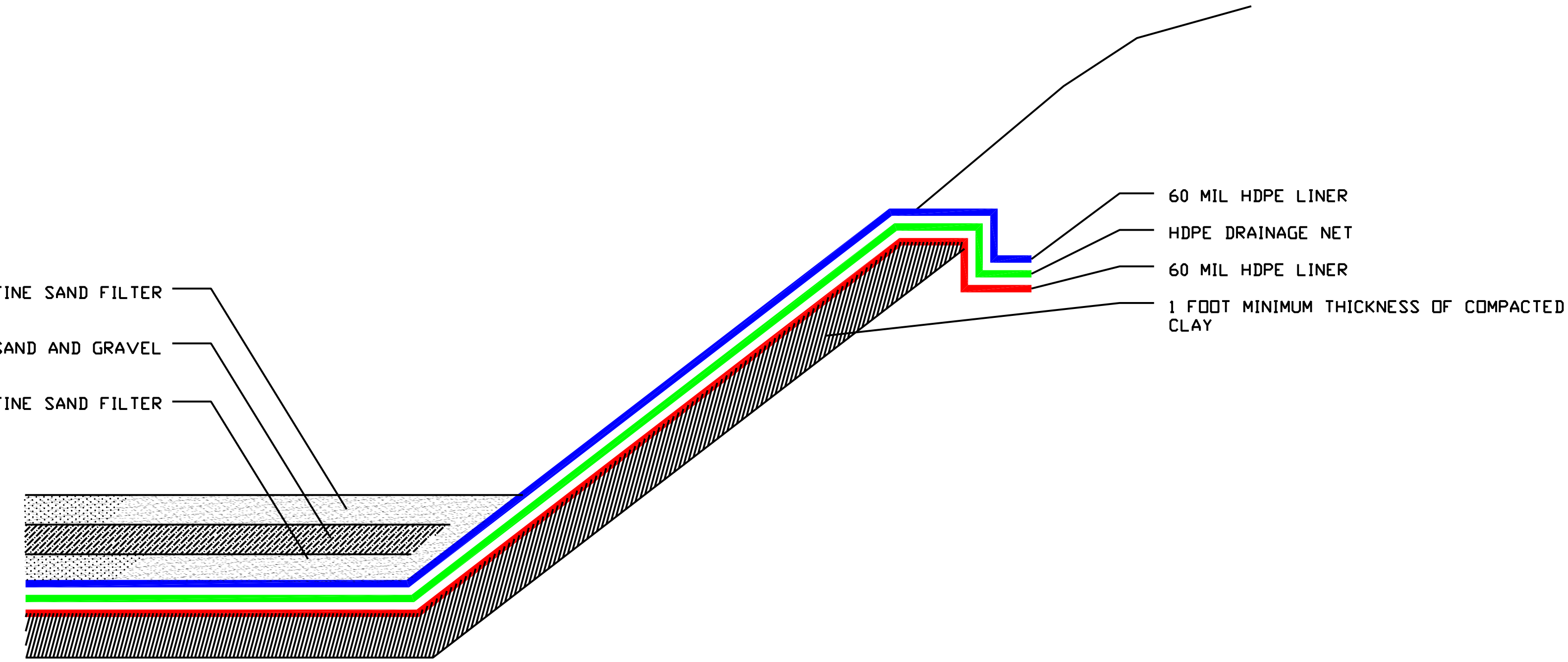
The area above the existing cross valley berm has been lined with a clay blanket of not less than two and up to ten feet thick. The clay blanket has been overlain with sandy material followed by gravel material, which is designed to collect slimes. Within the sand layer and adjacent to the clay liner are drainage pipes that drain to a collection sump.

The collection sump, located downstream of the cross valley berm, is equipped with a pump. The liquid in the sump is pumped to surface evaporation ponds or recycled back to the mill. Prior to installation of the seven-part liner in Cell 1, all tailings and associated material in this existing cell will be placed in the EPPC located adjacent to Cell 1 on the east side. The EPPC will be constructed with the seven-part liner prior to the start of construction work on Cell 1. Once the tailings and other contaminated material are removed from the existing tailings cell, Cell 1 will be constructed upstream of the cross valley berm.

Construction of Cell 1 will require reshaping and reconfiguration of the cross valley berm to a much more stable configuration with 3H:1V upstream and downstream outslopes. If the quantity of contaminated material exceeds the anticipated capacity of the EPPC below the HDPE lined fluid storage pond(s), the excess material can be stacked above grade in the EPPC or temporarily stockpiled within the existing Cell 1 area and then transferred to the lined Cell 1 after construction. The Cell 1 liner system will utilize as much of the existing clay liner as possible with attendant testing of clay thickness and quality. During construction of Cell 1, the liner system will be extended to connect Cell 1 and the EPPC and allow Cell 1 to serve as an additional containment measure for the EPPC. The extension and bridging liner between Cell 1 and EPPC will consist of a single 60 mil HDPE liner and will connect with the primary (upper) HDPE liner in each cell. The complete liner will extend from Cell 1 across the cross valley berm and connect with the seven-part liner in Cell 2 if Cell 2 is constructed. See Section 10 for more detail.

During milling activities, seepage from the ore storage pad will be minimal due to the current pad construction on a clay pad to reduce infiltration. Future ore storage pads will be constructed with a low permeability clay pad to reduce infiltration. The limited precipitation runoff from the ore stockpiles and ore storage pad is diverted into a HDPE-lined temporary holding pond for eventual transfer to the EPPC. Recent studies have determined that a clay material has been used to construct the ore pad. Tested thickness of the clay material is 12 to 14 inches with a hydraulic conductivity of 3.7 E-6 cm/sec. See Section 9 for more detailed discussion on the current ore pad.

The impoundment will be divided into two major tailings cells and the EPPC, which will all have a double liner system with leak detection placed over a 12" compacted clay base. A collection system will be installed over the double liner consisting of HDPE drainage piping placed within a filter bed. All the collection piping will attach together into one continuous drain field per sump, which will collect tailings leachate. From each sump, the liquid will be pumped to the lined Storage/Evaporation Pond for evaporation or recycling to the mill. The sumps will be used until the reclamation phase of covering the impoundment has been completed. See Section 5 for detailed design drawings of the tailings facility and liner system.



TYPICAL SECTION FOR SEVEN-PART LINER.
NOT TO SCALE.

ALL INTERIOR SLOPES WILL NOT EXCEED A MAXIMUM OF 3H:1V.

18" TOTAL THICKNESS OF FINE SAND AND SAND AND GRAVEL FILTER PLACED IN BOTTOM OF CELLS AND ON SLOPES UP TO 5H:1V.

LINER ANCHOR SYSTEM PER MANUFACTURE'S RECOMMENDATIONS.

FINE SAND FILTER SHALL CONSIST OF TYPICAL ENTRADA SAND

SAND AND GRAVEL FILTER SHALL MEET THE FOLLOWING GRADATION:

3"	100%	PASSING
NO. 4	20 - 70%	PASSING
NO 200	0 - 25%	PASSING

TYPICAL ON SITE MATERIALS WILL SATISFY THESE REQUIREMENTS

PLATEAU RESOURCES Ltd.		
Figure 4-1. Schematic of Seven-Part Liner		
DATE: 4-2007	Liner_schematic.dwg	NOT TO SCALE
Page 4-5	HYDRO-ENGINEERING L.L.C.	

5. TAILINGS DISPOSAL SYSTEM

5.1 General Design

Tailings from the ore processing operation are discharged to an impoundment, created by a dam, adjacent to the uranium mill. Cell 1 has an estimated design capacity of 1,602,000 tons with a maximum stacking height of 50 feet above the top of cell elevation of 4455 feet above MSL and an assumed emplaced tailings density of 80 pcf (dry basis). Cell 2 has an estimated ultimate capacity of 5,265,000 tons with a maximum stacking height of 70 feet above the top-of-cell elevation of 4430 feet above MSL and an assumed emplaced tailings density of 80 pcf (dry basis). A portion of the Cell 2 capacity (approximately 97,000 cu. yd.) will be reserved for tailings fluid or runoff storage. A portion of this storage may also be reserved in Cell 1. When Cell 2 approaches capacity, a drainage diversion or interior runoff storage system will be proposed to allow utilization of the full Cell 2 storage capacity unless the decision has been made to expand the cell to the Stage II configuration. At a plant throughput of 1000 tons of dry ore per day with 350 days per year operation, Cell 1 has a capacity of slightly more than 4 years of production. With full utilization of Cell 2, the capacity is sufficient for approximately 14 years of mill production. At capacity, the two tailings cells in the impoundment will cover an area of approximately 60 surface acres. The impoundment is fenced to exclude livestock and warn the general public that the facility has restricted access. Although it is not included in this submittal, the Stage II configuration includes raising the tailings dam 30 feet for an additional capacity of 2,867,000 tons.

The tailings management system for the facility was designed to meet the criteria in Regulatory Guide 3.11, 3.11.1, Appendix A of 10 CFR Part 40 and State of Utah Dam Safety Guide to Standard Operating Procedures, 1991. Stabilization will be accomplished by draining the tailings as they are placed in the impoundment. For this purpose, a leachate collection system has been installed in the bottom of the impoundment and the planned RMTP procedures will limit the segregation of fine and coarse tailings within the cells. The combination of RMTP, limited segregation of tailings fines, and the leachate collection system will maintain the tailings in a largely dewatered condition throughout operation. It will therefore be possible to reclaim the tailings disposal area in a relative short time period after it is filled to its ultimate level.

5.1.1 Existing Structures

A site selection survey (*Preliminary Site Selection Study Proposed Shootaring Canyon Uranium* Project, Utah, Woodward-Clyde Consultants, June, 1977) was completed to identify locations near the Shootaring Canyon uranium mines best suited for the safe and efficient disposal of tailings and convenient to areas suitable for an ore processing facility. A preliminary design and construction specifications (Woodward-Clyde Consultants, May, 1978) were completed for a dam and tailings impoundment facility at a candidate site identified in the earlier study. A third study, *Evaluation of Tailings Disposal Alternatives Shootaring Canyon Uranium Project, Utah*, Woodward-Clyde Consultants, December, 1978 reviewed alternative tailings disposal systems considered for the project. A supporting document, presenting the results of a tailings disposal and proposed ore processing facility performance assessment, was submitted to the NRC in June, 1978. The report included comparative data on costs and performance for the alternative methods of tailings disposal considered for the project.

Construction plans and specifications for the tailings disposal dam, impoundment area clay liner, and a final design report were submitted to the NRC in May, 1979.

Prior to construction of the tailings impoundment, the area was shaped to remove surface irregularities, unsuitable material was removed, and the surface compacted. Care was taken to ensure that the natural southwesterly slope of the area was maintained. Following the foundation dressing and compaction, clay was spread evenly over the impoundment area and compacted to 95 percent Standard Proctor Density with a sheepsfoot compactor. Water was used to wet the clay during the operation to ensure proper moisture content for compaction. Total thickness of the compacted clay liner is at least two feet in all areas. A layer of sandy material was spread over the clay liner promptly after it was placed, to preserve its integrity.

A dam key trench, about 40 feet wide and extending up the abutments above the level of the top of the dam was excavated across the natural drainage outlet from the impoundment basin. A dam about 400 feet wide at the base and 68 feet high was constructed for the first stage. The interior of the dam was constructed with a clay core placed into the key trench. Exterior slopes of the dam are not steeper than two horizontal to one vertical (2H:1V). The initial structure is expected to serve for 16 to 18 years of operations without raising the dam. Dam construction materials were obtained from local sources. Adequate quantities of all materials required for additions to the dam and any other clay usage in the impoundment have been identified and are available locally.

5.1.2 Modifications of Existing Structures

The cross valley berm will be modified to improve the stability of the structure. Both the cross valley berm and the Shootaring Dam will be modified to facilitate construction of the tailings cells, including reduction of the upstream slope of the Shootaring Dam and both the upstream and downstream slopes of the cross valley berm to a steepest slope of 3H:1V.

5.1.2.1 Cross Valley Berm

Based on the analyses presented in Section 4 and Appendix A, it is necessary to modify the cross valley berm in order to produce an acceptable level of stability. IME performed an analysis of a proposed berm section with the conclusion that stability would be acceptable provided the berm was reconfigured to a steepest inslope or outslope of 2H:1V and a raised berm crest to 4455 feet above MSL. The details of the analysis and the required modifications are provided in the IME letter report that comprises section A.3 of Appendix A. Specific materials with corresponding compaction and moisture content are required for additional fill on the reconfigured berm. Significant stability enhancement of the stability will also result from the additional fill on both the upstream and downstream faces of the berm to reduce the steepest slope to 3H:1V. This additional slope reduction is necessary for the lined cell construction and will preserve all features of the required berm modification including centerline alignment. The fill material specifications for the additional slope reduction will be the same as those required in the IME analysis.

5.1.2.2 Shootaring Dam

Modifications to the Shootaring Dam are necessary to allow construction of the Cell 2 liner system. Rock must be removed from the upstream face of the dam, and will be stockpiled for later use. Additional material will be added to the upstream face to reduce the slope to 3H:1V prior to construction of the liner.

5.1.3 Seven-Part Liner

The new seven-part liner will be placed over the prepared impoundment basin. Preparation will consist of base rock removal and/or dirt fill placement pursuant to the Construction Quality Control and Quality Assurance Plan (CQCQAP). The surface will be graded to create a smooth uniform surface prior to placement of the base clay liner. A minimum of twelve inches of clay material will serve as the base and the secondary 60 mil HDPE liner will be placed on top of the clay (see Figure 4-1). In Cell 1, portions of the existing clay will be preserved if possible. The next component in the liner system is a HDPE geonet material for leak detection and this is overlain by the primary 60 mil HDPE liner. A leachate collection system consisting of perforated and corrugated HDPE piping with a geotextile-wrapped clean-gravel envelope will be placed in a 6 inch thick layer of Entrada sand. A 6 inch thick layer of rocky sand and gravel soil will be placed on top of the Entrada sand. This will be overlain by a second 6 inch thick layer of Entrada sand for a total of 18 inches of drainage layer on the base of the cell. The drainage layer will be placed on the base of the pond and areas with a slope flatter than approximately 4H:1V. In areas where the leachate collection pipe is extended beyond the drainage layer, a filter sock will be placed around the pipe to prevent intrusion of tailings. An analysis of the hydraulic and chemical properties of the two proposed drainage layer materials was conducted with the conclusion that the proposed materials are suitable to perform the functions of: (1) guarding the HDPE liner against penetration or damage by stones or other objects; (2) conveying drainage from the tailings to the piping network; and (3) preventing intrusion of tailings into the drainage system. A synopsis of the analysis of the filter gradations and estimated hydraulic conveyance is included in Appendix B. The drainage sand and gravel materials will not be placed on the side slopes of the lined cells. This new liner system is detailed in the attached figures. Figure 5-1 presents the Cell 1, EPPC and Cell 2 configurations with contours to the top of the upper HDPE liner. The leachate that drains from the tailings will be collected in sumps and pumped to the EPPC for disposal or return to the mill process circuit.

5.1.3.1 Clay Liner

The clay liner will consist of a minimum of 12 inches of compacted clay and will be subject to gradation, compaction, and construction specifications in Appendix C. Cell 1 currently has a clay liner in place, and this will be preserved to the extent possible. After the existing tailings and other contaminated materials are removed and transferred to the EPPC, the existing clay liner will be tested for compliance with specifications in Appendix C and will be surveyed for compliance with radiological cleanup criteria described in Section 8 and other relevant sections of "Tailings Reclamation and Decommissioning Plan for Shootaring Canyon Uranium Project – 2005, Revised: December 2006" and as subsequently revised. The testing frequency and specifications for the remaining clay liner will be the same as that for newly placed clay liner as described in Appendix C. If the existing clay liner is left in place as subgrade, but is not included in the seven-part liner, it will be compacted to a minimum of 95% of Standard Proctor density in accordance with specifications for general fill soils.

5.1.3.2 HDPE Liner, Geonet, and Piping Material

The liners, geonet, and piping will be HDPE. The general specifications for the HDPE materials are included in Appendix C. In addition to the structural and strength related specifications, specifications related to UV and environmental stability, as well as chemical resistance of the HDPE are included. Many sources of chemical resistance data were consulted for the purposes of anticipating possible degradation of the liner system. Based on the review of available data, no measurable chemical degradation of the HDPE materials is expected. The identified process stream constituents that were evaluated as potentially detrimental to the liner include: sulfuric acid, sodium chlorate, and kerosene. Other constituents such as flocculants, sodium hydroxide, ammonia, tridecanol, tertiary amine, or sodium bicarbonate may be added or otherwise introduced to the process stream and eventually discharged to the tailings, but not at concentrations that are considered significant. The UV stability is related to carbon black content specifications in Appendix C.

The acidification of the process stream is considered the primary chemical alteration that has the potential to affect the liner. The estimated free acid (sulfuric) concentration in the discharge to the tailings is 5 g/liter or approximately 5%. All available chemical resistance information indicates that this concentration is not damaging to HDPE and that acid concentrations can be dramatically greater than 5% without damaging the liner. The sources of chemical resistance information include include: Poly-flex Chemical Resistance Tables; Personal communication with George Yazdani of Poly-flex, Inc.; ISCO Industries Chemical Resistance Listing, Zeus Chemical Resistance Listing; ADS Pipe Chemical Resistance Tables; Cole-Parmer Chemical Resistance Charts and others. The same sources also indicate that sodium chlorate will not damage HDPE. The expected addition of sodium chlorate to the ore stream is at a rate of approximately 1.7 lb/ton of ore feed, so concentration of the salt in the discharge stream will be very small.

Available chemical resistance information does indicate that pure kerosene will damage HDPE lining, particularly at very high temperatures (60 deg. C or 140 deg F). The anticipated kerosene loss rate from the Solvent Exchange process is 0.5 gal kerosene per 1000 gallons of process feed, which equates to a concentration of approximately 500 ppm. Kerosene is volatile and the concentration in any free solution in the tailings cell(s) will likely be smaller than that in the discharge stream leaving the mill. Ultimately, the limited amount of kerosene that remains within the tailings will become relatively immobile because of adsorption to the tailings solids. It is also possible that the kerosene will undergo a biodegradation process. Because the maximum plausible kerosene concentration in the discharge to the tailings is very small and the degree of contact with the double liner system is very limited, there is negligible potential for damage to the liner, geonet, or piping by the presence of small concentrations of kerosene.

5.1.3.3 Filter Sock and Other Synthetic Materials

The two common materials available for leachate collection pipe filter socks are nylon and polyester. Of the two materials, polyester has the superior resistance to damage by small concentrations of sulfuric acid that will be present in the tailings solution and is the preferred material for the sock. Neither material is subject to damage by the small concentrations of kerosene or sodium chlorate that may be present in the discharge stream. Other specialty materials may be considered for the leachate collection pipe socks if appropriate physical and chemical resistance properties are demonstrated.

The filter sock will have a typical opening size equivalent to or smaller than a #70 US Standard sieve. The drainage pipe will be bedded in a gravel envelope in the base of the cells and areas where the slope is flatter than 4H:1V. No filter sock will be used in these areas. Where the drainage pipe extends up slopes steeper than 4H:1V, a filter sock will be placed around the pipe. In areas where it is practical, these sections of pipe may be bedded in Entrada sand which has a grain size distribution that falls almost entirely between the #200 and #50 U.S. Standard sieves. A detailed evaluation of the drainage filter analysis is included in Appendix B. The potential for filter sock plugging within bedding in Entrada sand is limited because the Entrada sand is very fine and very uniform with very little silt and clay. Very little internal migration of fines within the Entrada sand is expected, and the structure and gradation of the bedding sand adjacent to the filter sock or geotextile is not expected to change. For those sections of pipe on the steeper slopes where the tailings will be in direct contact with the filter sock, there is very little potential for a measurable saturated depth and a corresponding hydraulic gradient across the sock because of the steep (0.33) gradient of the liner/tailings interface. In these sections, the drainage pipe is largely superfluous. The acidic nature of the tailings solution should prevent significant biological growth and bio-fouling of the filter sock.

Other synthetic materials may be used for controlling erosion, protecting the liner system during the operational phase, and as a filter and cushioning layer for the drainage pipe gravel bedding. In these circumstances, the synthetic materials may be exposed to the tailings and tailings solution. A variety of polypropylene-based geosynthetic materials are available and the chemical resistance of these materials for the required life in this application should approach that of HDPE or other polyethylene based materials. The nonwoven geotextile material that overlies the gravel in the sumps (see Figures 5-5 and 5-6) and is used to wrap the gravel leachate collection pipe gravel envelope may be polyethylene or polypropylene based and must meet specifications in Appendix C. This geotextile will serve as a filter for the leachate collection pipe gravel envelope.

5.1.3.4 Drainage Filter Materials

The two materials that will be used for the drainage filter of the leachate collection system are Entrada sand and a rocky soil sand/gravel material produced by screening the available material in the quarry area. Two column tests were conducted using these materials to evaluate the potential for adverse geochemical reactions in the presence of acidic tailings solution. A solution was produced by acidifying mill site production well water (from well WW1) to a pH of 0.85 with sulfuric acid. More than 20 pore volumes of the solution were passed through each column using a peristaltic pump. There was significant off-gassing of CO_2 as the acidic solution contacted the Entrada sand or rocky soil and the gas locking prevented column operation with a simple gravity feed system. However, manometer ports were observed during column operation, and there was no indication of a significant reduction in permeability or conveyance through the proposed drainage filter materials. The report of this testing is included in Appendix H. The general conclusion of this column testing is that there was no significant adverse geochemical reaction that would measurably reduce the permeability of the drainage filter bed.

The drainage filter materials were selected to serve three primary functions. The very fine and very uniform Entrada sand will be placed as the upper and lower layers in the drainage filter. The upper Entrada sand layer will prevent migration of even very fine tailings into the drainage filter, while, in combination with the geotextile filter, the lower Entrada sand layer will prevent movement of fines into the leachate collection piping. Because the lower Entrada sand bedding layer will be selected and/or screened to exclude debris and particles larger than coarse sand, this layer will also protect the primary HDPE liner from punctures by gravel-sized and larger stones. The third primary function of the drainage filter is to provide conveyance of the tailings solution to the leachate collection piping or directly to the sumps. The rocky soil sand/gravel filter layer is expected to have a slightly greater permeability than the Entrada sand while having a broad enough gradation to prevent intrusion of the fine sand at the layer interface. The overall permeability of the 18 inch thick drainage filter layer is expected to be moderate to high.

5.1.3.5 Pumps, Wiring, and Other Materials

Equipment such as extraction pumps, plumbing, submerged wiring, and fluid level monitors will be constructed of materials that will provide an acceptable life and degree of reliability. Selection of commercially available equipment that is exposed to tailings and tailings solution will be based on chemical resistance to the acidic solution as well as durability and economic considerations. HDPE piping with a suitable pressure rating will generally be acceptable and stainless steel will be the preferred material for pipe fittings, pump bodies, pump impellors, etc. There are two access pipes for installation of pumps in the leachate collection sump and the leakage detection sump for a total of four pumping access pipes per sump (see Figure 5-4). There will be redundant pumping systems installed within each sump, or, alternatively, at least one replacement pump of each size and configuration will be kept on hand for replacement in the event of a leachate collection or leakage detection pump failure. Fluid level monitoring equipment will also be constructed from materials that will withstand the harsh environment of the sumps.

5.1.4 Leachate Collection System and Leak Detection System

Figure 5-2 presents the layout of the leachate collection system for the tailings cells. This figure also shows the location of the below grade berms that serve to separate and isolate drainage from the cell to individual sumps. In some cases, these berms are minor extensions of natural drainage divides in the cell base. The separation berms will be constructed as a small (approximately 1 foot high) ridge in the subgrade, and will be overlain with the full thickness of liner and drainage system. Two cross sections were developed to represent the two tailings cells and the EPPC, and these are included in Figure 5-3. Figures 5-4 through 5-7 present details of the collection and leak detection sump construction. Each sump is constructed as a dual sump with separate collection areas for the leak detection discharge and the leachate collection discharge. Within each composite sump, there are two 12 inch diameter pump access pipes for pump installation within both the leak detection sump and the the leachate collection sump for a total of four pump installation pipes per sump. There is also a 4 inch diameter access pipe in the leak detection and the leachate collection portions of the sump. These access pipes will be used for installation of water level monitoring equipment. The Construction Quality Control and Quality Assurance Plan is presented in Section 5.3.

5.1.4.1 Collection Piping Capacity

A minimum of five individual trunk or branch lines is routed into each sump, but under most plausible conditions with one foot of head over the primary liner, the drainage water will be delivered to at least two sumps (except for the EPPC). There will be no slurried or moist tailings placed in the EPPC, so the required leachate collection system capacity in this cell is very small. The minimum and anticipated size of the perforated and corrugated drainage lines is specified as a 3 inch internal diameter. A 4 inch internal diameter perforated and corrugated pipe will also be acceptable. From the standpoint of capacity, the 3 inch single-wall corrugated pipe is the critical condition because it the smallest specified diameter with the greatest hydraulic roughness. Larger diameter pipes or dual wall pipes with a smoother internal surface will have a larger conveyance capacity. There are variable slope conditions for the drainage lines which results in differing individual capacities for the drainage lines. For the range of hydraulic grade lines that result from the existing slope conditions, the expected capacity of a single 3 inch diameter drain pipe ranges from approximately 50 gpm to over 150 gpm. If it is assumed that six to ten drain lines are actively contributing to sumps, the drain pipe capacity ranges from 300 gpm to over 1500 gpm. This does not include water delivered directly to the sump(s) through the 18 inch thick granular drainage layer which is estimated to range from approximately 1 gpm to over 20 gpm/100 feet of width with a saturated thickness of 12 inches. There is also a large gravel envelope around each collection pipe with a conveyance capacity that is approaching and may even exceed the conveyance capacity of the pipe.

Table 5-1. Leachate Collection Piping Capacity

Hydraulic Grade	Pipe Area	Hydraulic Radius	Manning's n	Flow Velocity	Discharge Rate	
(feet/feet)	(ft^2)	(feet)		(ft/second)	(cfs)	(gpm)
3 inch Diameter Pipe						
0.02	0.049	0.0625	0.014	2.36	0.116	52
0.05	0.049	0.0625	0.014	3.74	0.183	82
0.1	0.049	0.0625	0.014	5.29	0.259	116
0.15	0.049	0.0625	0.014	6.47	0.318	143
0.2	0.049	0.0625	0.014	7.48	0.367	165
4 inch Diameter Pipe						
0.02	0.087	0.083	0.014	2.86	0.250	112
0.05	0.087	0.083	0.014	4.53	0.395	177
0.1	0.087	0.083	0.014	6.40	0.559	251
0.15	0.087	0.083	0.014	7.84	0.684	307
0.2	0.087	0.083	0.014	9.06	0.790	355

Discharge calculated with Manning's equation: $Q=1.486/n\ A\ R^{2/3}\ S^{1/2}$

Q = discharge in cfs
n = Manning's n
A = Pipe cross sectional area in square feet
R = Hydraulic radius in feet = Area/wetted perimeter
S = Pipe slope or hydraulic grade

As described in Section 4.1.1, the required fluid conveyance capacity in the leachate collection system may vary over a large range. The limiting maximum required rate of leachate collection would coincide with the contingency of conventional slurry placement. The continuous fluid discharge rate for this contingency slurry placement is estimated at 173 gpm, from which an abstraction of approximately 60 gpm is taken for intermediate term retained water within the tailings as well as other minor losses such as evaporation. This results in a maximum required leachate collection system capacity of approximately 113 gpm in the active tailings placement area. When the conveyance capacity through the piping system and granular drainage blanket is considered, the minimum ratio of available to required capacity is over 3. When the planned fluid extraction processing is considered, the conveyance capacity of the leachate collection system will be one or more orders of magnitude greater than the anticipated leachate drainage rate. In addition, the proposed leachate collection pipe installation configuration provides an expanded gravel envelope that increases overall conveyance capacity to the sumps.

5.1.4.2 Piping Structural Design

The perforated and corrugated collection system piping will be 3 inch or 4 inch diameter HDPE with a minimum pipe stiffness of 50 psi at 5% deflection (as determined by methods described in ASTM D2412 and AASHTO M252). Standard wall perforated HDPE pipe with an equivalent or superior pipe stiffness may also be used. The pipes will be bedded at the base of a clean gravel envelope that is wrapped within a nonwoven geotextile (see Figure 5-8) meeting the specifications in Appendix C. A geotextile layer will be placed directly on top of the primary liner to cushion the geotextile-wrapped gravel envelope. The wrapping geotextile will be placed between the gravel envelope and the cushioning geotextile over a base width of approximately 6 feet. After placement of the pipe and gravel envelope, the remaining width of the geotextile roll will be folded over the gravel envelope with sufficient overlap to completely enclose the gravel envelope. The anticipated roll width for the geotextile is 15 feet, which should be sufficient to enclose a gravel envelope with 3 to 5 square feet of cross sectional area. This gravel envelope will extend to a minimum of 6 inches above the top of the pipe (see Figure 5-8). Entrada sand or the rocky soil sand/gravel will be placed directly over the top of the geotextile surrounding the gravel envelope as shown in Figure 5-8 and then compacted with small vibratory compactor on both sides of the pipe to compact materials around and over the pipe. This will produce a very dense envelope around the drainage pipes which corresponds to the desirable material Class I with compaction condition for the pipe bedding Soil Modulus (E') value. Where the pipe is extended up slopes steeper than 4H:1V beyond the drainage layers, a filter sock will be placed around the pipe and the pipe may not be bedded within imported material unless it is necessary to accommodate equipment access.

An alternative corrugated pipe installation is shown in Figure 5-9. This alternative installation configuration will only be used for segments where access for installation equipment is limited by proximity to steeper slopes or existing access roads.

The maximum anticipated overburden thickness for the leachate collection piping is approximately 128 feet. The small diameter and favorable bedding conditions for the corrugated HDPE pipe will provide a substantial load bearing capacity. A minimum of 27 inches of compacted material must be in place over the pipe (30 inches of material over the primary liner) before general equipment traffic will be allowed. Only specialized low ground pressure or other approved equipment will be allowed on areas where the cover over the pipe or primary liner is less than 27 inches or 30 inches respectively. With these restrictions on equipment traffic and live loading during the construction, the critical loading condition will be the static overburden load at maximum thickness and full cell utilization.

An analysis of the load bearing capacity of the 3 inch diameter corrugated and perforated collection pipe is included in Appendix J. The method for determining the acceptability of the pipe installation was based on the Modified Iowa Formula as presented in the "Plastic Pipe Design Manual" available on-line from Lamson Vylon Pipe. The Modified Iowa Formula is considered a conservative approach, and an alternate calculation was made with the Burns and Richard Solution using a program provided by ADS Pipe. The results of both calculations indicated that the 3 inch diameter corrugated and perforated pipe with a minimum pipe stiffness of 50 psi would withstand the maximum static overburden load of 128 feet of tailings at a moist density of 100 pcf.

5.1.4.3 Leachate Collection Operation

The nonwoven geotextile will function as a filter for the gravel envelope surrounding the leachate collection pipes. The filter area for the geotextile will range from 6 to 9 sq. ft. per foot of collection pipe run. The Entrada sand drainage layer will be placed around the geotextile and will virtually eliminate the migration of fines to the geotextile filter.

The potential for biofouling in the acidic tailings condition is also limited. The combination of large geotextile filter surface area and the Entrada sand granular filter will prevent the internal migration of fines within the leachate collection system and the plugging of the collection system.

The leachate collection pipe layout is presented in Figure 5-2. The maximum drainage distance to a collection pipe on the base of the cell(s) was limited to 100 feet or less. The pipe configuration for Cell 2 was also adapted to provide additional collection pipe length along the toe of the 3H:1V side slopes of the cell. Selected drainage pipes are extended up the side slopes of Cell 2 as a contingency, but these pipes are not expected to function because the potential for a significant saturated thickness and gradient to the pipe with a tailings/liner interface gradient of 0.33 feet/feet is very small. The collection piping system delivers leachate to one of two sumps for Cell 1, and to one of four sumps in Cell 2. There is a single sump in the EPPC. The material placed in the EPPC will be in dry form and the quantity of leachate will be very small. The leachate collection pipes will discharge to the collection sumps. The gravel envelope around the pipes will also provide substantial conveyance capacity to supplement that in the pipes or to replace that in the pipes in the event of a local pipe failure.

5.1.4.4 Leachate Collection Sumps

Details of the sump construction and design are presented in Figures 5-4 through 5-7. Each sump consists of two compartments for separate capture and containment of the leachate from the tailings, and the discharge from the leakage detection system. Both compartments of each sump will be filled with washed gravel (3/8 to 1 inch diameter) to a depth of 10 feet. Both the primary and secondary HDPE liners will be doubled in the sumps for added strength and puncture resistance. The perimeter of the sumps will be 80 feet on each side with 3H:1V side slopes to a total depth of 10 feet with a 20 foot by 20 foot sump base. The approximate volume of the leachate collection compartment of the sump assuming 35% gravel porosity is approximately 60,000 gallons. The volume of the leakage detection compartment of the sump assuming 35% gravel porosity is approximately 13,000 gallons.

5.1.4.5 Sump Access Pipes

Three access pipes will be installed in each of the two compartments of each sump and will extend to an accessible location on the cross valley berm, EPPC berm, or Shootaring Dam. The configuration of the sump access pipes are presented in Figure 5-2 and details of the sump construction and design are presented in Figures 5-4 through 5-7. Two of the access pipes for each compartment will be 12 inch HDPE SDR 9 pipe, and the third pipe will be 4 inch HDPE SDR 9. The loading conditions for the sump access pipes were evaluated and are presented in Appendix J with the conclusion that the load bearing capacity of the pipes is acceptable. The 12 inch pipes will be extended into the sump to the base of the compartment and a minimum of 5 feet across the base. The 4 inch access pipes will be extended to the base of the sump and a minimum of 5 feet across the base. The pipe alignment will be as straight as possible to allow installation of pumps or water-level monitoring equipment. The sections of the access pipes within the gravel fill in the sump will be perforated. The maximum perforation width or size will be 0.188 inch, and there will be a minimum of 5 square feet of exposed perforation area for each 12 inch access pipe. The 4 inch diameter pipes will also be perforated within the sump.

The primary purpose of the 12 inch access pipes is for pump installation. It is anticipated that 4 inch diameter submersible pumps will be installed in the 12 inch access pipes, and the inside diameter of approximately 9.75 inches is sufficient to accommodate larger diameter pumps without excessively restrictive limitations on pipe alignment. The two 12 inch access pipes allow redundant or contingency pump installation. The 4 inch access pipes will house fluid level monitoring equipment for controlling the pumps or generating an alarm signal.

The three access pipes to the leak detection compartment of the sumps will exit the sump between the primary and secondary liners. In order to extend the access pipes to crest of the cross valley berm or the Shootaring Dam where the pipes will terminate, a provision must be made to support or envelope the pipes and overlaying primary liner, or the pipes must be booted through the primary liner and extended to the top of the crest on top of the liners. If the pipes are extended to the crest between the liners, the loading conditions require that a shaped envelope be placed around the pipes. This envelope will both provide bedding support for the pipe, and provide a smooth surface over which the primary liner can be draped. For this configuration, it is necessary to eliminate voids in the shaped envelope to avoid excessive tension in the primary liner. With the continuous liner system across the top of the cross valley berm, it will still be necessary to boot the leak detection access pipes through the primary liner just below the crest of the cross valley berm. However, a boot near the crest of a pond is far less critical because it is well above the anticipated fluid level.

Three potential options for leak detection access pipe configurations are presented in Figure 5-10. The preferred option is the encasement of the three access pipes routed between the primary and secondary liners with a moderate strength flowable fill. In order to contain the flowable fill during construction, a channel will be created by seaming a cap HDPE liner strip to the secondary liner. The expected width of the channel is approximately 10 feet. The construction sequence will require layout of the pipes over the secondary liner. The channel strip will then be laid over the pipes and seamed to the secondary liner to form a smooth encasement surface. The flowable fill will then be injected to completely fill the channel and encase the pipes. It is anticipated that the flowable fill will have to be injected in intervals and allowed to solidify to avoid slumping and distortion or damage of the channel at the base of the slope.

The second proposed option shown in Figure 5-10 uses a more conventional sand envelope to surround the pipes and provide a smoothed and shaped surface for placement of the primary liner. The installation of the sand envelope will require compaction and elimination of void space in and around the pipes. This configuration with the sand envelope presents some construction challenges with installation and compaction of granular fill on a 3H:1V slope with restrictive equipment traffic limitations.

The third proposed option for the leak detection access pipe configuration includes pipe boots to extend the pipes through the primary liner just above the exit point on the sump. The elevation of the boots will be at least five vertical feet above the primary liner over the sump. The boot(s) will consist of HDPE materials conforming to the specifications in Appendix C. A commercially produced skirt or flange-type boot can be used or extrusion welding can be used to construct a skirt or flange type boot on-site. The minimum area of the skirt will be 16 square feet per pipe boot. A moderate-strength flowable fill will be applied to encase the pipe and boot of each leakage detection sump access pipe. The flowable fill must be applied in a manner to fill all voids in and around the boot to reduce local stress concentrations on the boot seam welds. Alternatives to the flowable-fill encasement will be considered if it can be demonstrated that the alternative will relieve the stress produced by the overburden loading.

A nonwoven geotextile will be placed between the sump access pipes and the primary HDPE liner for pipes running over the primary liner. All access pipes above the primary liner will also be bedded and enveloped in a minimum thickness of 2 feet of compacted Entrada sand. This sand envelope will extend from the sumps up the cell side slopes for a distance of at least 100 horizontal feet. If acceptable installation practices and equipment are demonstrated, the sand envelope will be extended to the crest of the slope. The surface of the Entrada sand envelope will then be plated with 3 to 6 inches of the sand and gravel material to reduce erosion. In addition to providing support for the piping, the sand envelope will moderate temperature changes and thermal expansion/contraction of the pipe. The six pipes may be routed in a single corridor, but the pipes within the sand envelope will be separated by a minimum distance of 1 foot to facilitate compaction of the sand envelope.

The sump access pipes will be terminated in a structure at the crest of the 3H:1V slopes. The structure will include an extendable pipe anchorage arrangement to allow thermal expansion/contraction of the pipes. A simplified conceptual cutaway structure for the Cell 1 sump access pipes terminating at the crest of the cross valley berm is shown in Figure 5-11. Other mechanisms and available commercial structures will be considered for the pipe termination.

5.1.4.6 Leachate Collection Pump Capacity

As described in section 5.1.4.1, the anticipated maximum continuous rate of leachate fluid discharge to the sumps is 113 gpm. It is unlikely that all of the leachate will be delivered to a single sump for an extended period of time, and the planned moisture reduction techniques are expected to dramatically reduce the fluid delivery rate to the sumps. A preliminary leachate collection pump sizing estimate for Cell 1 includes a submersible pump with a production of 100 gpm at a Total Dynamic Head (TDH) of 110 feet, and a secondary or contingency pump with a production of 50 gpm at a TDH of 110 feet. This will allow evacuation at a rate greater than the anticipated leachate collection rate to accommodate precipitation runoff contributions and other excess recharge, while allowing evacuation at a rate as small as 50 gpm to capture leachate from tailings placed with the RMTP method. The active volume of the leachate collection compartment in the sump may be as large as 40,000 gallons, so it will be practical to set pump control levels that will produce pump run times of several hours for evacuation of the sump even under very small leachate delivery rates. Combining multiple operating sumps with two available leachate collection pumps of differing capacities in each sump, it will be possible to operate the total leachate collection system over a wide range of discharge rates.

The proposed leachate collection pump sizing for the EPPC is a 20 gpm discharge at a TDH of 50 feet. The sizing of the pumps for the Cell 2 sumps will be evaluated based on operational leachate collection rates for Cell 1.

5.1.4.7 Leakage Detection System Capacity and Action Leakage Rates

The preliminary leakage detection pump sizing is based upon anticipated maximum leakage detection rates for each sump. The specified Action Leakage Rate (ALR) from the TMP submitted in 1999 to the Utah DRC and NRC was 200 gal/day/acre and the acceptability of this ALR was confirmed with the following analysis. The "Action Leakage Rate Guideline" published by Alberta Environmental Protection presents a method for estimating leakage through the primary liner for a properly installed and functioning liner system. This method corresponds to similar or identical methods recommended by the EPA and others. Although there is a minute rate of leakage through HDPE through permeation or diffusion, the permeation rate is insignificant when contrasted with the leakage through small punctures or defects in the installed liner. The accepted ALR calculation predicts leakage through a single small hole in the liner with a corresponding assumption of the maximum acceptable number of holes for a specified area of the liner.

The assumed number of holes in the primary liner for the seven-part liner is one per acre. The recommended assumption for head above the hole with a freely-draining condition is three feet. Table 5-2 presents the formulation and calculation that indicates the predicted leakage through a 0.082 inch (2.08 mm) hole with a total head of three feet is 200 gallon per day. In combination the assumption of one hole per acre, the specified ALR is 200 gal/day/acre.

Table 5-2. Action Leakage Rate Calculation

Estimated Hole Diameter (inch)		0.082
Number of Holes per Acre		1.00
Assumed Average Liquid Depth (feet)		3
Leakage per hole - $Q = C_b(\text{area})(2\ g\ h_w)^{1/2}$		
C_b = 0.6 (default coefficient), g = 32.2 ft/sec^2		
area =	$(0.079/2/12)^2$ (3.1416) = 3.7E-05 sq.ft.	
Q =	0.0003059 ft^3/sec	
Q =	26.4 ft^3/day	
Q =	198 gal/day	
Action Leakage Rate (ALR)		
ALR = Q (number of holes/acre)		
ALR =	200 gal/day/acre	

The ALR of 200 gal/day/acre can be converted to a Sump Action Leakage Rate (SALR) by taking the product of the ALR and the area contributing to the sump. There are a total of seven sumps for the two tailings cells and the EPPC. Table 5-3 presents the maximum leakage capture area for each sump and the SALR for each sump. If the SALR is exceeded for any sump, the tailings disposal will be: shifted to the other cell, shifted to an area in the cell contributing to a different sump, or discontinued. Subsequent moist or slurried tailings disposal within the area of the cell where the exceedance of the SALR occurred will be contingent on locating and repairing/correcting the point(s) of leakage. If the point(s) of leakage cannot be located and repaired, the suspect area of the cell will be abandoned or restricted to placement of tailings in a form where there will be no significant post-placement drainage (dry form or non-draining paste).

Table 5-3. Sump Action Leakage Rate for Cell 1, Cell 2 and EPPC Sumps

Sump	Contributing Area	Action Leakage Rate	SALR	
	(acre)	(gallon/day/acre)	(gallon/day)	(gallon/minute)
EPPC	7.84	200	1570	1.09
Cell 1 East	12.15	200	2430	1.69
Cell 1 West	9.23	200	1850	1.28
Cell 2 Northeast	9.48	200	1900	1.32
Cell 2 Northwest	10.05	200	2010	1.4
Cell 2 Southeast	8.22	200	1640	1.14
Cell 2 Southwest	10.88	200	2180	1.51

Based on the SALRs presented in Table 5-3, the required pump capacity for the leak detection system is less than 5 gpm. There is a wide variety of 4-inch diameter submersible pumps available with sufficient TDH to service the evacuation of the leakage detection sump. The pumped discharge from the leakage detection sump will be metered with a combination totalizing/instantaneous meter and discharged to the EPPC pond(s). The preliminary frequency of sump evacuation for active tailings areas will be once per day with a daily record of evacuated volume. The frequency may be reduced to a weekly evacuation and recording if the total evacuated volume is less than the daily SALR for the sump. Fluid-level monitoring equipment will be installed in the leak detection sump prior to operation of the corresponding tailings cell area. The fluid-level monitoring equipment will, at a minimum, provide a measurement of the depth of fluid in the cell and an adjustable alarm level to activate a light or siren type alarm. The fluid-level monitoring equipment may also incorporate features to allow pump control. Acceptable fluid-level monitoring equipment may include suitable pressure transducers or transmitters

After a period of record for evacuation is established, level controls within the sump access pipes may be installed or existing controls adjusted to automate the pump operation and evacuation process provided an alarm system remains in place to clearly indicate excessive fluid levels. The leakage detection fluid evacuation equipment will be inspected daily after a sump is activated and this will continue as long as there is measurable discharge to either the leachate collection or leakage detection sump.

The conveyance capacity of the geonet is primarily a function of the transmissivity of the geonet and the hydraulic gradient. The minimum specified transmissivity of the geonet is 1.0E-03 m^2/sec (1.076E-02 ft^2/sec) at a loading of 2000 psf (see Table 5 in Appendix C). The capacity of the leak detection system at critical locations was calculated using Q=TiL where: T is the geonet transmissivity; i is the hydraulic gradient, and L is a flow width. The critical location for flow width is the perimeter of sump, and as a measure of conservatism, the gradient was set as the smallest average bottom slope in the approach to the sump. Table 5-4 presents a calculation of the geonet conveyance capacity for a critical location in both SI and English units.

Table 5-4. Leak Detection Geonet Capacity For Sump Entry

Specified Geonet Transmissivity (m^2/sec)	1.00E-03	**Specified Geonet Transmissivity (ft^2/sec)**	1.076E-02
Minimum Bottom Slope Near Sump	0.0143	**Minimum Bottom Slope Near Sump**	0.0143
NE Cell 2 Sump		***NE Cell 2 Sump***	
Minimum Assumed Gradient	0.0143	**Minimum Assumed Gradient**	0.0143
Assuming gradient = bottom slope		***Assuming gradient = bottom slope***	
Sump Perimeter (meter)	97.5	**Sump Perimeter (feet)**	320
Conveyance = (gradient)(transmissivity)(perimeter)		**Conveyance = (gradient)(transmissivity)(perimeter)**	
= (0.0143)(0.001)(97.5)		= (0.0143)(0.0108)(320)	
= 0.0013948 m^3/sec		= 0.0492605 ft^3/sec	
= 121 m^3/day		= 4256 ft^3/day	
= 120508 l/day		= 31836 gal/day	

The conveyance capacity of the geonet must meet or exceed the SALR for each sump. It is also necessary to have an acceptable factor of safety calculated as the ratio of calculated conveyance capacity to SALR to accommodate rib layover, compression, siltation and other mechanisms that may reduce transmissivity of the geonet. Table 5-5 presents a conservative estimate of the sump entry geonet conveyance capacity using the minimum average bottom slope of the sump approach as the hydraulic gradient. The minimum calculated factor of safety for the geonet conveyance is 16.5.

Table 5-5. Sump Entry Geonet Conveyance Capacity and SALR Factor of Safety

Sump	Contributing Area	SALR	Minimum Bottom Slope and Hydraulic Gradient	Sump Entry Conveyance Capacity	Conveyance Factor of Safety
	(acre)	(gallon/day)		(gallon/day)	(ratio)
EPPC	7.84	1570	0.03	66760	42.5
Cell 1 East	12.15	2430	0.05	111260	45.8
Cell 1 West	9.23	1850	0.15	333790	180.4
Cell 2 Northeast	9.48	1900	0.0143	31820	16.7
Cell 2 Northwest	10.05	2010	0.0149	33160	16.5
Cell 2 Southeast	8.22	1640	0.079	175790	107.2
Cell 2 Southwest	10.88	2180	0.018	40050	18.4

5.1.4.8 Liner System Perimeter Anchorage

The geometry and slope conditions for the perimeter of the two tailings cells and the EPPC are highly variable and a variety of perimeter anchorage approaches will be used. Because the placement of the granular drainage layer will be limited to slopes of 4H:1V (14 degrees) or flatter, and some narrow corridors for pipe envelopes on 3H:1V slopes, the additional tensile stress imposed on the liners by overburden loading will be dramatically reduced. In Cell 1, only small areas with short 4H:1V slope segments will be overlain by the granular drainage layers, and the more typical slope of the cell base is 6H:1V (9.5 degrees) or flatter. The anchorage for the perimeter of mildly sloping areas of the cell(s) will be done primarily with a standard trench anchor or with a horizontal (runout) anchor where liner continuity precludes trenching. For the edge of the liner at the crest of steeper side slopes, the primary anchorage will be done with a standard trench anchor. Appendix K presents sample anchor design calculations. Figure 5-12 presents locations and descriptions of liner system anchorage on the perimeter of the tailings cells and the EPPC. Temporary liner weighting systems will be required for the majority of the side slopes for Cell 2, the upstream and downstream faces of the cross valley berm and the upstream face of the Shootaring Dam. Sandbags attached to durable ropes will be deployed to temporarily ballast the liner. Pursuant to a recommendation by personnel with Colorado Lining International (personal communication), operational ballasting of the liner on the 3H:1V slopes will be provided by sand-filled ballast tubes (6 inch diameter or greater) extending from the anchor trench to the toe of the 3H:1V slopes. The spacing of these ballast tubes will not be more than 35 feet on the exposed liner. Alternative weighting methods and configurations proposed by the lining construction contractor must be approved by PRL.

5.1.4.9 Liner Interface Stability

The areas of the tailings cells where there will be cover soil or drainage layers over the primary liner are limited to slopes of 4H:1V (14 degrees) and flatter. The slope sections (with cover) steeper than 5H:1V (11.3 degrees) are limited in length and plan area. This configuration dramatically limits the cover induced tension on the liner and anchorage. Koerner (2005) lists typical peak friction angles for soil to smooth geomembrane interfaces of approximately 18 degrees. Hence, those areas in the base of the cell(s) with drainage layer are at a significantly flatter slope than the critical interface friction angle.

The liners will be extended up slopes of 3H:1V (18.4 degrees) without a cover soil. At the crest of these slopes, the liners will be anchored with a trench anchor or will have an extended linear or horizontal anchor across an access road. The angle of the interface between the clay underliner and the secondary HDPE liner is approximately equal to the critical friction angle, and the support provided by the slope crest anchor should be sufficient to prevent liner slippage in the absence of cover soil loading. The friction angle between the geombranes and the geonet can be appreciably smaller than 18 degrees, but again, the geomembranes and geonets are anchored at the crest of the 3H:1V slopes, and the downslope tensile load is limited to the self-weight of the material. With the exception of access road and ramp construction at mild slopes, the cell(s) will be filled from the bottom and progressive material loading up the slopes will be supported at the base of the cells.

There is a special case of the sump access pipe corridors where there will be small areas of 3H:1V (18.4 degrees) slope where a sand envelope around the pipes is constructed. In order to facilitate construction of these pipe envelopes, it is anticipated that additional material will be required at the toe of the envelope corridor and this will result in a much milder slope (likely 4H:1V or flatter) for the top surface of the pipe envelope. Thus there will be a large flare in the footprint of the sand envelopes at the base. This condition will approach that of a base-filled cell, and the additional downslope tensile loading will be minor. The pipes will be tensioned at the crest and this will further support the sand envelope. There will also be a geotextile between the pipes and primary liner to prevent abrasion of the liner, and the tensioning of the pipes at the crest will also provide support for this geotextile.

5.1.5 Tailings Cell Fluid Distribution and Piping

The fluid collected in the leachate collection sumps and any fluids evacuated from the leakage detection sumps will be conveyed to the EPPC. The piping system to accomplish this will consist of at least two parallel pipes from the sumps discharging to the crest of the cross valley berm, and at least two parallel pipes from the sumps discharging to the crest of the Shootaring Dam. The fluids will be discharged to one or more of the ponds within the EPPC. The pipes will be routed along the inside crest of the cross valley berm, the inside crest of the Shootaring Dam, and the eastern side slope crest of Cell 2 as shown in Figure 5-13. This will place the piping to the EPPC within the containment of the liner system. Fluid return piping to the mill will be routed within the same corridor as the tailings discharge line.

5.1.6 Tailings Area Stormwater Drainage Plan

The configuration of the Shootaring Canyon drainage basin limits the runoff contributing area to the tailings cell(s). A major diversion structure will be constructed to further reduce the total contributing drainage area, and other minor structures and features will be utilized to control runoff and reduce the stormwater contribution to the disposal cells.

5.1.6.1 North Drainage Diversion

A significant berm/channel diversion structure is planned for the area just to the northeast of the EPPC (see Figure 5-13). This structure will divert runoff from the area north of the site and west of the site access road. Detailed cross sections for the diversion are presented in the “Tailings Reclamation and Decommission Plan for Shootaring Canyon Uranium Project – 2005, Revised: December 2006” and as subsequently revised. The structure consists primarily of a downstream berm that forms the drainage divide and an excavated and riprap protected channel on the upstream side of the berm. Runoff reporting to this structure is diverted to a large topographic depression which ultimately outlets to the north and west to an adjacent drainage.

5.1.6.2 Perimeter Diversion Ditch/Berm

Minor structures will be utilized to preclude or reduce runoff delivery to the tailings disposal cells. Figure 5-13 presents the location of and a cross section for a minor diversion structure around the east and northeast sides of the EPPC. This feature can be constructed with moderately sized grading equipment and is intended to prevent runoff from entering the EPPC. The alignment of the ditch/berm will result in delivery of runoff to the far north end of the berm where it will pond. The height of the berm will be increased at the north end and the labeled runoff discharge pipe will be installed to convey runoff that exceeds the storage volume upstream of the berm into Cell 1. The upstream invert of the pipe will be approximately 1 foot below the minimum berm crest elevation as an "emergency spillway" for this minor structure. This type of minor structure may also be used for local runoff capture in other areas.

5.1.6.3 EPPC Emergency Discharge Pipe

Figure 5-13 presents the location of and a cross section for an emergency discharge pipe connecting the EPPC to Cell 1. The top of the berm separating the EPPC from Cell 1 will have an overlay to form an access road. A small depression in the entire liner system will be formed perpendicular to the alignment of the berm. The depth of this depression will be approximately 1 foot and the total width of the depression will be approximately 10 feet. The minimum elevation for the upper geomembrane on the remainder of the perimeter of the EPPC is 4778 feet above MSL. The roadbed and perimeter anchor will add between 1.5 and 2.5 feet of elevation to the surface, and this will bring the actual surface elevation of the perimeter of the EPPC to 4780 feet above MSL. The invert of the EPPC emergency discharge pipe on the upstream side of the berm will be approximately 4777 feet above MSL, so fluid that may accumulate between the single lined fluid storage ponds and the edge of the EPPC will discharge to Cell 1 before overtopping the liner containment.

5.1.6.4 Passive Runoff Exclusion

Figure 5-13 presents the location of and a cross section for an area of passive runoff exclusion. This approach simply exploits flat areas on the perimeter of Cell 1 as a minor runoff capture area. The placement of the liner and drainage layers on the northern edge of Cell 1 is expected to result in an edge of the lined cell that is modestly above the grade of the surrounding area. This forms a small depression outside of the cell in which runoff from the outer drainage area can accumulate.

5.1.6.5 External Depressional Storage

The construction of Cell 2 will require substantial fill on the west side of the cell. This fill across natural swales will leave a series of depressions on the west side of the cell (see Figure 5-13). The depressions will capture runoff from the area west of Cell 2 and allow it to infiltrate or evaporate rather then entering the cell.

5.1.6.6 Ore Pad Sediment and Runoff Pond

A HDPE lined pond will be constructed to contain runoff and sediment from the ore pad area. The ore pad has been constructed on a compacted clay base as described in Section 9.1. A perimeter ditch currently captures runoff from the ore pad and diverts it to the existing tailings area. The pond will be constructed by local excavation and fill as shown in Figure 5-14. The pond base and berm will be compacted according to specifications in Appendix C. The single HDPE liner and installation will conform to the specifications in Appendix C. The liner will be anchored at the crest with a 4 feet wide and 16 inch deep conventional trench anchor. The liner in the base of the pond will be weighted with sandbags or other approved weights. The liner will extend to an elevation of 4514 feet above MSL. The approximate capacity of the pond is presented in Table 5-6.

Table 5-6. Ore Pad Sediment and Runoff Pond Capacity

Elevation	Area	Incremental Volume		Cumulative Volume	
(ft above MSL)	(sq ft)	(cubic feet)	(acre-ft)	(cubic feet)	(acre-ft)
4505	393	0	0.000	0	0.000
4506	1824	1109	0.025	1109	0.025
4507	4304	3064	0.070	4173	0.096
4508	7832	6068	0.139	10241	0.235
4509	9088	8460	0.194	18701	0.429
4510	10426	9757	0.224	28458	0.653
4511	11847	11137	0.256	39594	0.909
4512	13351	12599	0.289	52193	1.198
4513	14936	14144	0.325	66337	1.523
4514	16605	15771	0.362	82107	1.885

The ultimate capacity of the pond at overtopping of the liner is approximately 1.9 acre-feet. This is sufficient to contain more than 6 inches of runoff from the current ore pad area. Even with the capture of runoff from small drainage areas external to the ore pad, and a possible reconfiguration of the ore pad area, the pond will have sufficient capacity to contain several inches of runoff from the contributing area. A discharge pipe will be installed in the berm on the southwest side of the sediment pond. This pipe will be installed to place the upstream invert through a boot at an elevation of the 4512 feet above MSL or two feet below the pond crest. This pipe will be an 8” or larger HDPE pipe (SDR 17 or heavier) and will convey water from the sediment pond to a storage pond in the EPPC.

The runoff from the ore pad area will be diverted to the sediment and runoff storage pond. The existing capture ditch around the ore pad will require some minor reconfiguration to discharge to the pond. If necessary, small lined capture basins with pipe outlets will be constructed to capture ore pad runoff and convey it through pipes to the sediment pond. The pond will be evacuated with a centrifugal pump after every significant runoff event. A significant runoff event is one that results in a water depth of more than two feet in the lowest portion of the sediment pond. The evacuated water will be delivered to a pond in the EPPC or to a tailings cell. The sediment collected in the pond will be periodically cleaned out with a combination of a solids handling pump and a pressurized water stream to flush sediment to the base of the pond for collection with the pump.

5.2 Proposed Construction Sequencing and Control

The construction sequencing for the tailings facility will include actions to: create a lined repository for existing tailings and contaminated material, clean up existing contaminated materials in the tailings area, construct lined Cell 1, and construct lined Cell 2 when Cell 1 is approaching capacity.

5.2.1 EPPC Construction

The EPPC will be the first tailings disposal constructed. Prior to the start of construction activities, the EPPC area will be surveyed for compliance with radiological cleanup criteria described in Section 8 and other relevant sections of "Tailings Reclamation and Decommissioning Plan for Shootaring Canyon Uranium Project – 2005, Revised: December 2006" and any subsequent revisions. Any contaminated materials in the area may be temporarily transferred to the existing tailings area. The EPPC may be constructed with local fill and borrow. The clay for the underliner will conform to specifications in Appendix C, and the seven-part liner will be constructed in the EPPC. The liner system will be extended across the top of the berm between the EPPC and Cell 1 for eventual attachment to the Cell 1 liner. The northeastern corner of the EPPC is approximately at grade and a low slope entry point at this corner will allow access. A minimum of 30 inches of material must be in place above the primary liner before unrestricted traffic is allowed in the area.

5.2.2 Contaminated Material Transfer

After construction of the EPPC is completed, the existing tailings and other contaminated materials in the existing tailings basin will be transferred to the EPPC. This includes materials on the top and outslopes of the cross valley berm. The Area F contaminated materials upstream of the Shootaring Dam will also be excavated and placed within the EPPC. All areas where the contaminated materials are collected will be surveyed for compliance with radiological cleanup criteria described in Section 8 and other relevant sections of "Tailings Reclamation and Decommissioning Plan for Shootaring Canyon Uranium Project – 2005, Revised: December 2006" and as subsequently revised.

5.2.3 Cell 1 Construction

After transfer of the contaminated materials to the EPPC and confirmation of the radiological cleanup, the remaining materials above the existing clay barrier will be excavated and utilized to the extent possible. If the material is suitable, it can be used to reconfigure the cross valley berm to the specified 3H:1V. The rocky soil material in the existing tailings basin can also be used to construct the berm for the north drainage diversion or for general fill. After grading of the subgrade surface and testing of the existing clay liner, the seven-part liner will be constructed with any necessary augmentation of the clay liner. The subgrade below the clay liner consists primarily of Entrada sandstone. The Entrada sandstone is a dense, compact and sound foundation for the lined tailings cell. Mildly sloping construction ramps (6H:1V and flatter) can be installed on the north side of Cell 1. A minimum of 30 inches of material must be in place above the primary liner before unrestricted traffic is allowed in the area.

5.2.4 Pond Construction

The single HDPE line ponds will be constructed within the perimeter of the EPPC. If the volume of material in the EPPC is acceptable, the ponds can be constructed anytime after completion of the transfer of contaminated materials to the EPPC. If there is excess contaminated material within the EPPC, it can be transferred to Cell 1 after completion of the liner and prior to the construction of ponds. Selective handling of the contaminated materials in the EPPC will be required to create a suitable subgrade for HDPE liner installation. If necessary, Entrada sand may be imported to grade the base and side slopes of the ponds.

5.2.5 Cell 2 Construction

The construction of Cell 2 will be delayed until Cell 1 has been utilized for approximately 60% of its capacity. Cell 2 will require extensive earthwork to construct the subgrade including the reduction of the upstream slope of the Shootaring Dam to 3H:1V. The subgrade of Cell 2 will consist of Entrada sand. Appendix I presents the results of Proctor compaction testing for the Entrada sand, and these results indicate that the very uniform fine sand can be compacted to a high density over a broad moisture content range. The majority of the Cell 2 base and a significant portion of the side slopes will be excavated into the native Entrada sandstone which will form a dense consolidated base. Prepared and compacted Entrada sand will be a suitable subgrade for the pond construction. In will be necessary to construct an access ramp into Cell 2 prior to construction of the liner system. After a portion of the liner system is in place, a secondary ramp can be constructed by bridging over a completed section of the liner. This will allow removal of the original access ramp. The secondary access ramp will be at a flatter slope than the 3H:1V side slopes of the cell and will be constructed with additional fill in a buttress at the toe of the side slope. The material placed directly against the HDPE liner will consist of the Entrada sand from which is free of debris and oversized (>0.5 inch) particles. The overlying fill may consist of sand and gravel rocky soil or other suitable material. A minimum of 30 inches of fill will be placed over the primary liner before general traffic is allowed.

5.3 Construction Quality Control and Quality Assurance Plan

The Construction Quality Control and Quality Assurance Plan that will be utilized in the construction of the tailings impoundment system is included in Section C.1 of Appendix C.

5.4 SOP for Tailings Dam and Facilities Inspection Program

The SOP for Tailings Dam and Facilities Inspection Program will be kept on the Shootaring Mill site. SOP HP-21 was previously presented in the 1999 submittal of the Tailings Management Plan – Amended. The previous version of the SOP was withdrawn with changes in the license status, but is currently undergoing revision and will be updated, assigned a new SOP number, and submitted to the DRC. The revised SOP for the tailings dam inspection program utilizes references of *State of Utah Dam Safety Guide to Standard Operating Procedures, 1991 and* NRC Regulatory Guide 3.11.1, *Operational Inspection and Surveillance of Embankment Retention System for Uranium Mill Tailings, 1980.*

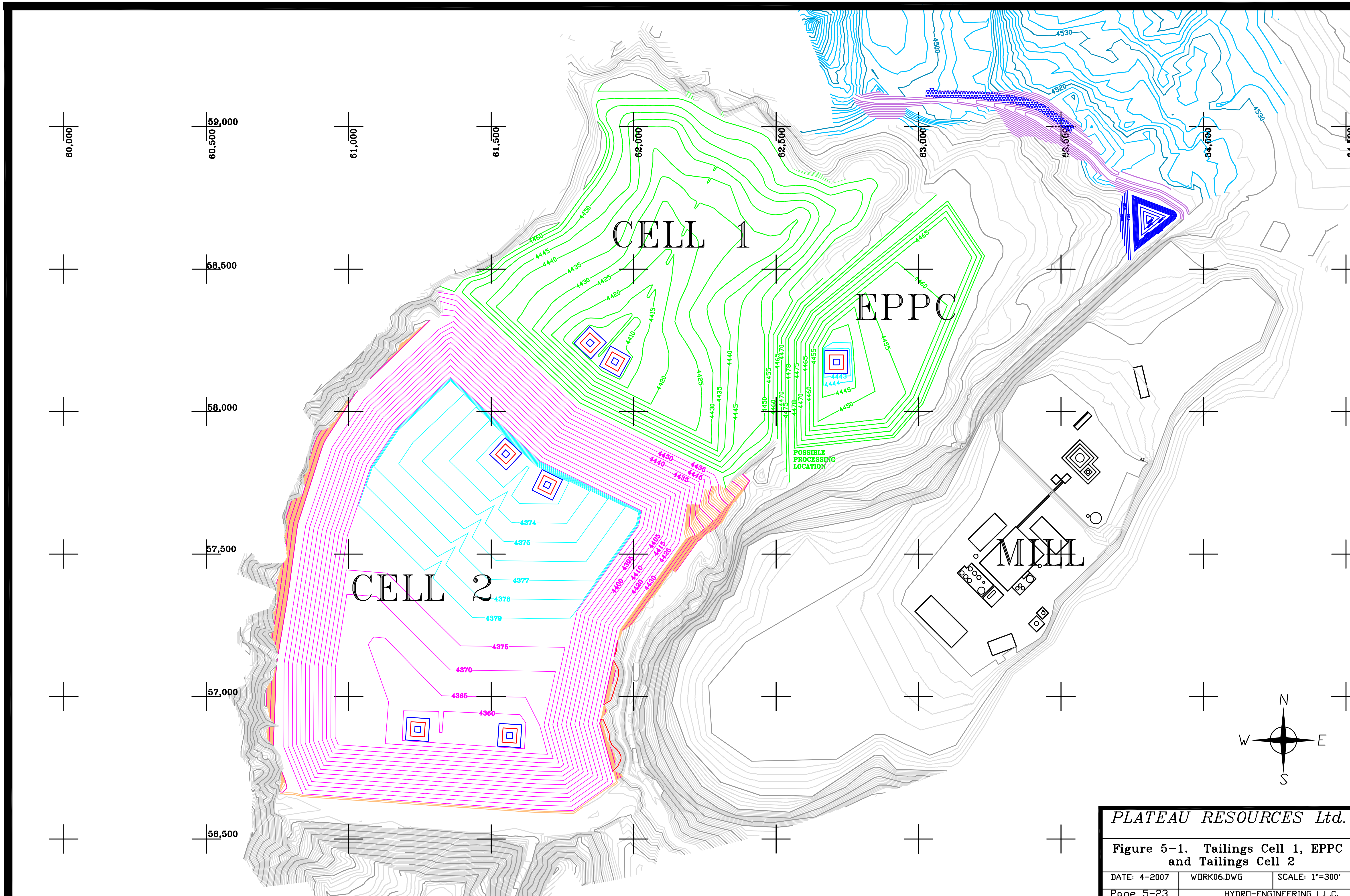


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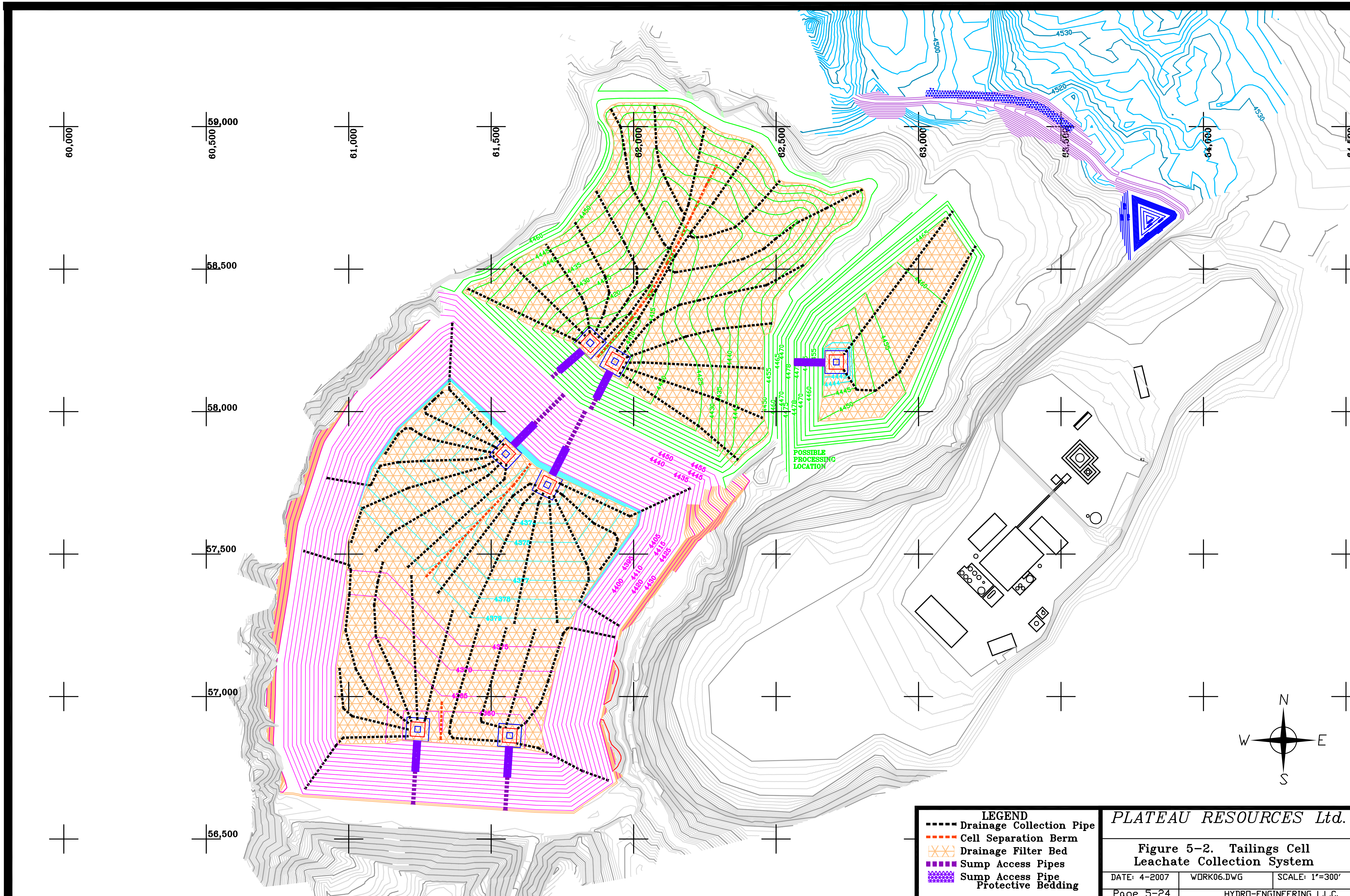
Figure 5-1. Tailings Cell 1, EPPC and Tailings Cell 2

DATE: 4-2007	WORK06.DWG	SCALE: 1"=300'
Page 5-23	HYDRO-ENGINEERING L.L.C.	



LEGEND
- Drainage Collection Pipe
- Cell Separation Berm
- Drainage Filter Bed
- Sump Access Pipes
- Sump Access Pipe Protective Bedding

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Figure 5-2. Tailings Cell Leachate Collection System

DATE: 4-2007	WORK06.DWG	SCALE: 1"=300'
Page 5-24	HYDRO-ENGINEERING L.L.C.	

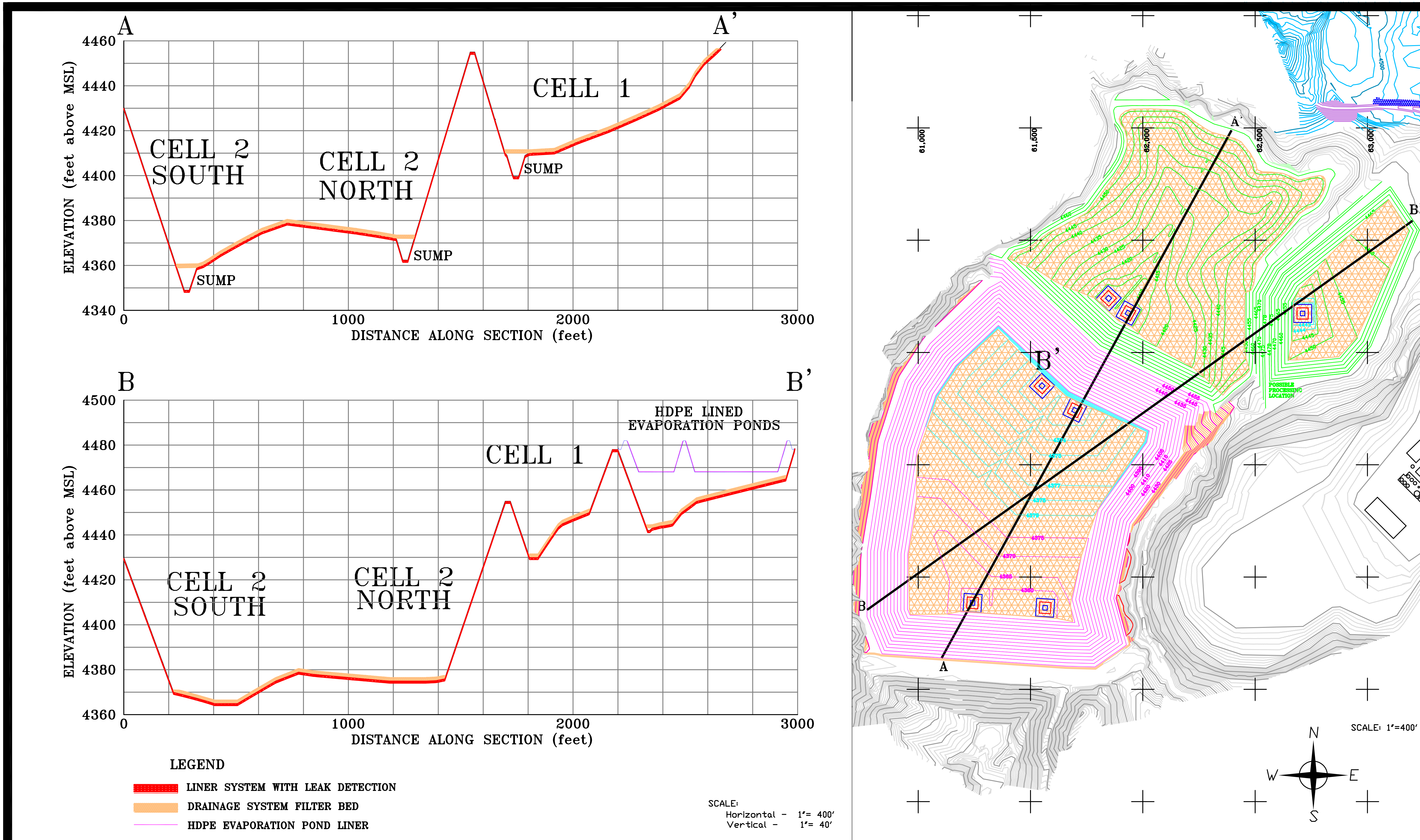

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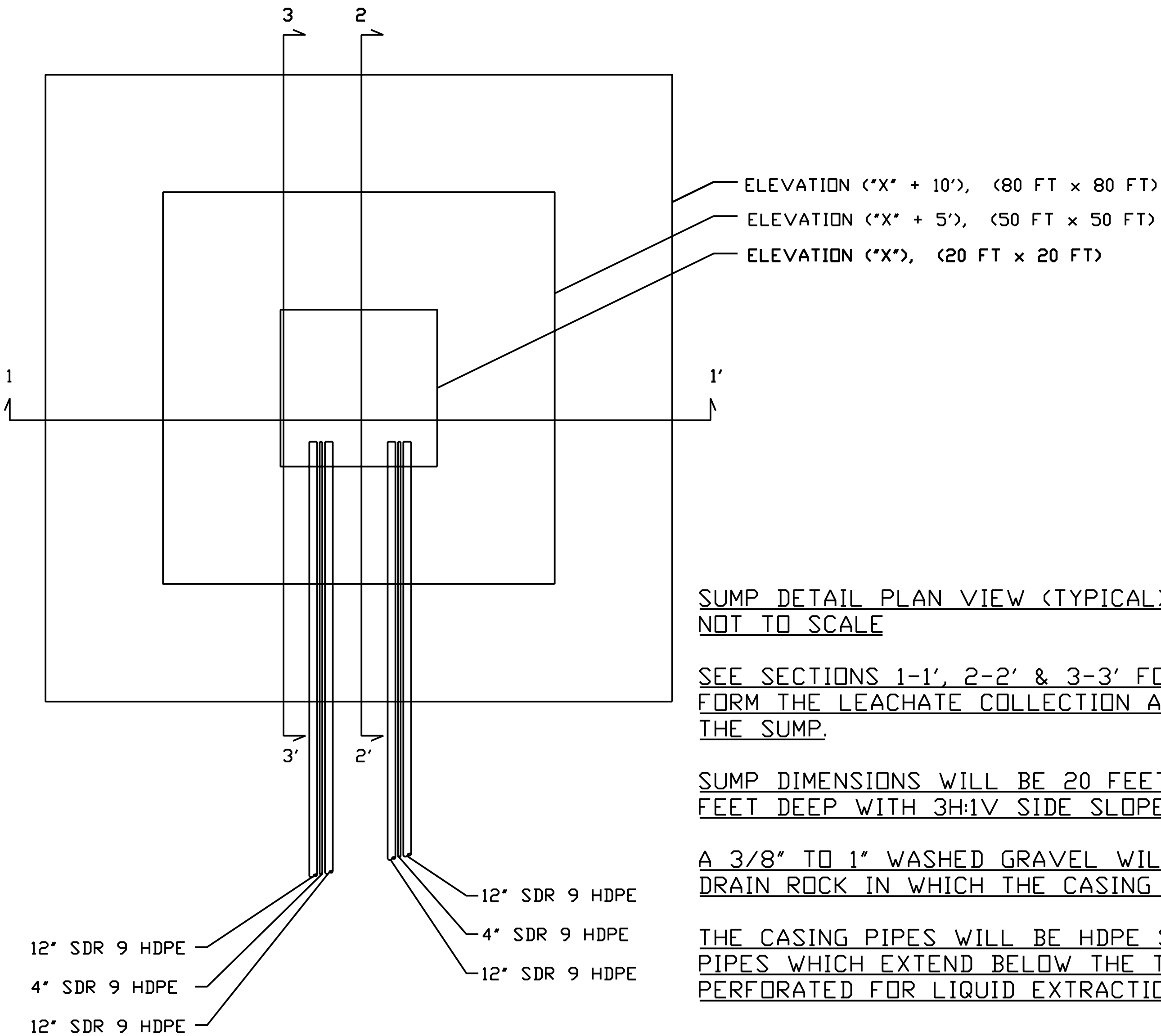
Figure 5-3. Tailings Area Cross Sections

DATE: 4-2007	WORK06.DWG
Page 5-25	HYDRO-ENGINEERING L.L.C.



SUMP DETAIL PLAN VIEW (TYPICAL)
NOT TO SCALE

SEE SECTIONS 1-1', 2-2' & 3-3' FOR LINER AND PIPE PLACEMENT TO FORM THE LEACHATE COLLECTION AND LEAK DETECTION CELLS WITHIN THE SUMP.

SUMP DIMENSIONS WILL BE 20 FEET BY 20 FEET SQUARE BOTTOM, 10 FEET DEEP WITH 3H:1V SIDE SLOPES.

A 3/8" TO 1" WASHED GRAVEL WILL BE USED IN THE SUMPS AS THE DRAIN ROCK IN WHICH THE CASING PIPES ARE PLACED.

THE CASING PIPES WILL BE HDPE SDR 9. THE PORTION OF THE CASING PIPES WHICH EXTEND BELOW THE TOP OF THE SUMP WILL BE PERFORATED FOR LIQUID EXTRACTION.

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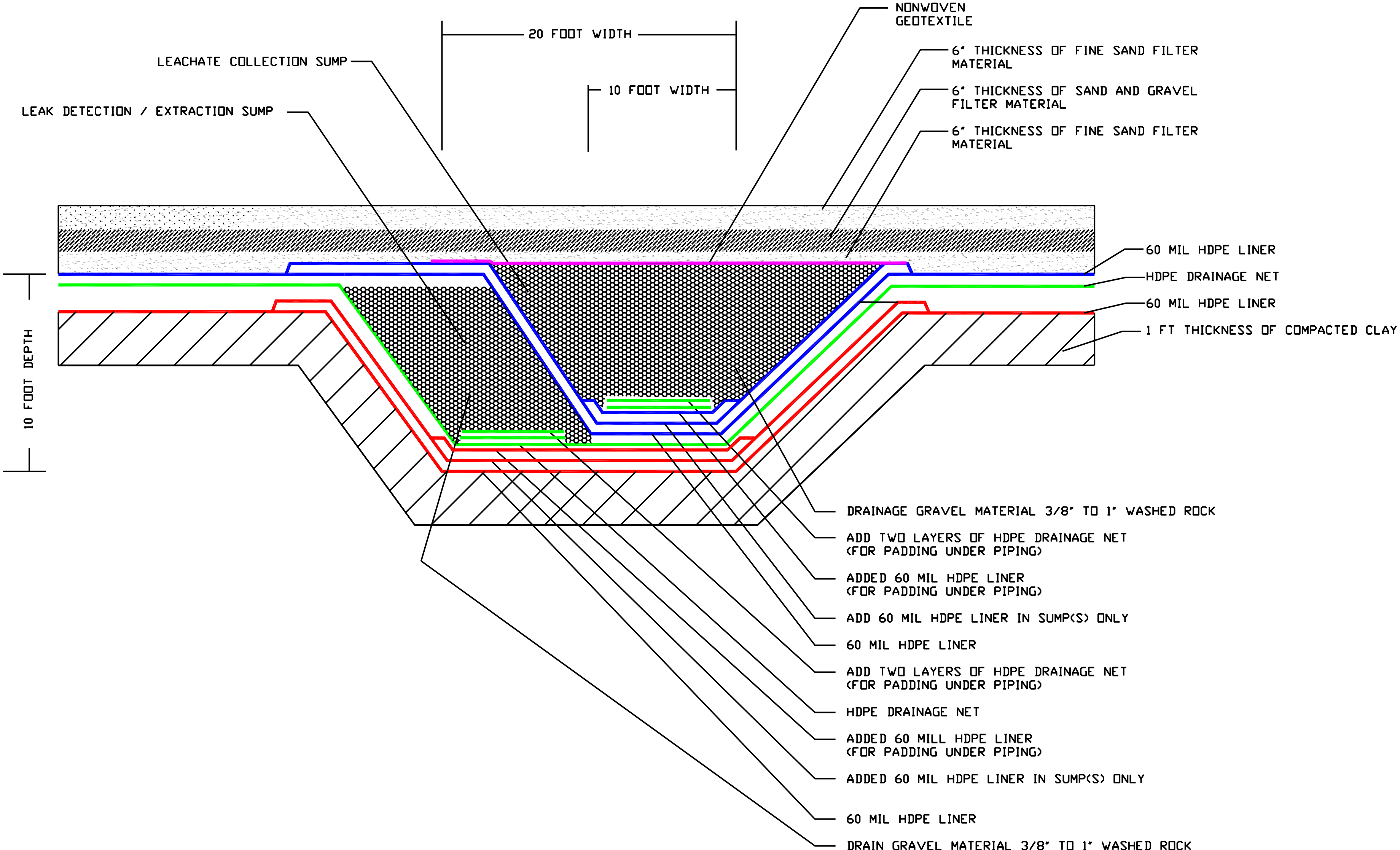
Figure 5-4. Tailings Cell Sump Detail and Cross Section Locations

DATE: 4-2007	Liner_schematic.dwg	NOT TO SCALE
Page 5-26	HYDRO-ENGINEERING L.L.C.	



SECTION 1-1' LEACHATE COLLECTION LEAK DETECTION / EXTRACTION SUMP(S)
(FRONT VIEW) NOT TO SCALE

INSTALL SUMP PUMP(S) INTO GRAVEL SUMP(S) THROUGH 12" SDR 9 HDPE PERFORATED CASING PIPE. (NOT SHOWN) NOTE: PERFORATIONS ONLY WITHIN THE SUMP(S)

SUMP PUMP(S) CONTROL SYSTEMS INSERTED INTO THE SUMP(S) THROUGH A 4" SDR 9 HDPE PERFORATED CASING PIPE. (NOT SHOWN) NOTE: PERFORATIONS ONLY WITHIN THE SUMP(S).

A RESERVE 12" SDR 9 HDPE PERFORATED CASING PIPE WILL BE INSTALLED INTO EACH SUMP(S) AS A BACKUP IF NEEDED FOR ADDITIONAL PUMPING.

SLOPES IN SUMP(S) WILL NOT EXCEED 3H:1V.

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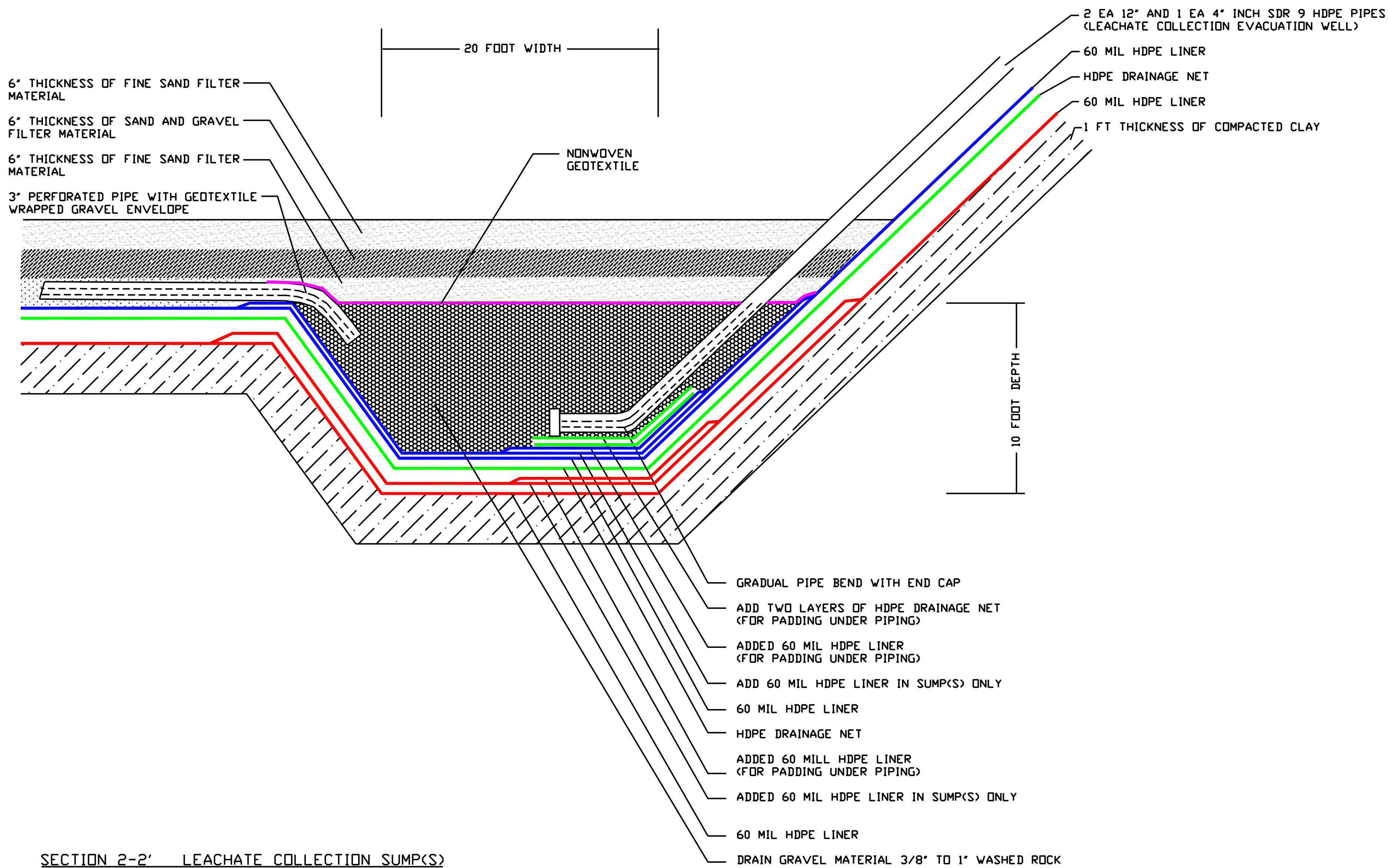
Figure 5-5. Sump Detail Cross Section 1 - 1'		
DATE: 4-2007	Liner_schematic.dwg	NOT TO SCALE
Page 5-27	HYDRO-ENGINEERING L.L.C.	



SECTION 2-2' LEACHATE COLLECTION SUMP(S)
(SIDE VIEW) NOT TO SCALE

INSTALL SUMP PUMP(S) INTO GRAVEL SUMP(S) THROUGH 12" SDR 9 HDPE PERFORATED CASING PIPE. NOTE: PERFORATIONS ONLY WITHIN THE SUMP(S).

SUMP PUMP(S) TO OPERATE ON A HIGH/LOW (ON/OFF) CONTROL SYSTEM. CONTROL SYSTEM INSERTED INTO THE SUMP(S) THROUGH A 4" SDR 9 HDPE PERFORATED CASING PIPE. NOTE: PERFORATIONS ONLY WITHIN THE SUMP(S).

A RESERVE 12" SDR 9 HDPE PERFORATED CASING PIPE WILL BE INSTALLED AS A BACKUP IF NEEDED FOR ADDITIONAL PUMPING.

SLOPES IN SUMP WILL NOT EXCEED 3H:1V.

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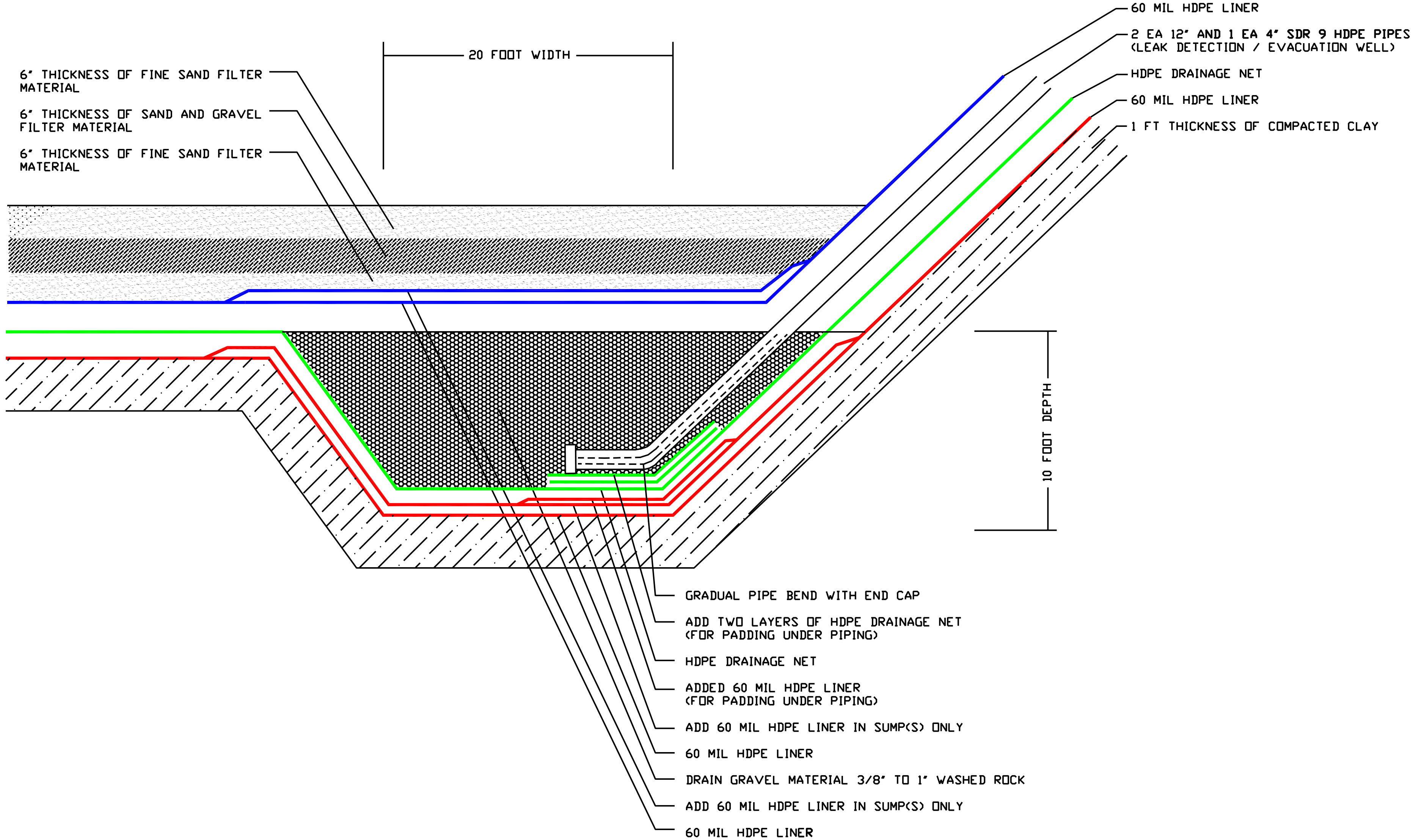
Figure 5-6. Sump Detail Cross Section 2 – 2'

DATE: 4-2007	Liner_schematic.dwg	NOT TO SCALE
Page 5-28	HYDRO-ENGINEERING L.L.C.	



SECTION 3-3' LEAK DETECTION / EVACUATION SUMP(S)
(SIDE VIEW) NOT TO SCALE

INSTALL SUMP PUMP(S) INTO GRAVEL SUMP(S) THROUGH 12" SDR 9 HDPE PERFORATED CASING PIPE. NOTE: PERFORATIONS ONLY WITHIN THE SUMP(S).

SUMP PUMP(S) TO OPERATE ON A HIGH/LOW (ON/OFF) CONTROL SYSTEM WITH HIGH WATER LIGHT SIGNAL(S). CONTROL SYSTEM INSERTED INTO THE SUMP THROUGH A 4" SDR 9 HDPE PERFORATED CASING PIPE. NOTE: PERFORATIONS ONLY WITHIN THE SUMP(S).

A RESERVE 12" SDR 9 HDPE PERFORATED CASING PIPE WILL BE INSTALLED AS A BACKUP IF NEEDED FOR ADDITIONAL PUMPING.

SLOPES IN SUMPS WILL NOT EXCEED 3H:1V.

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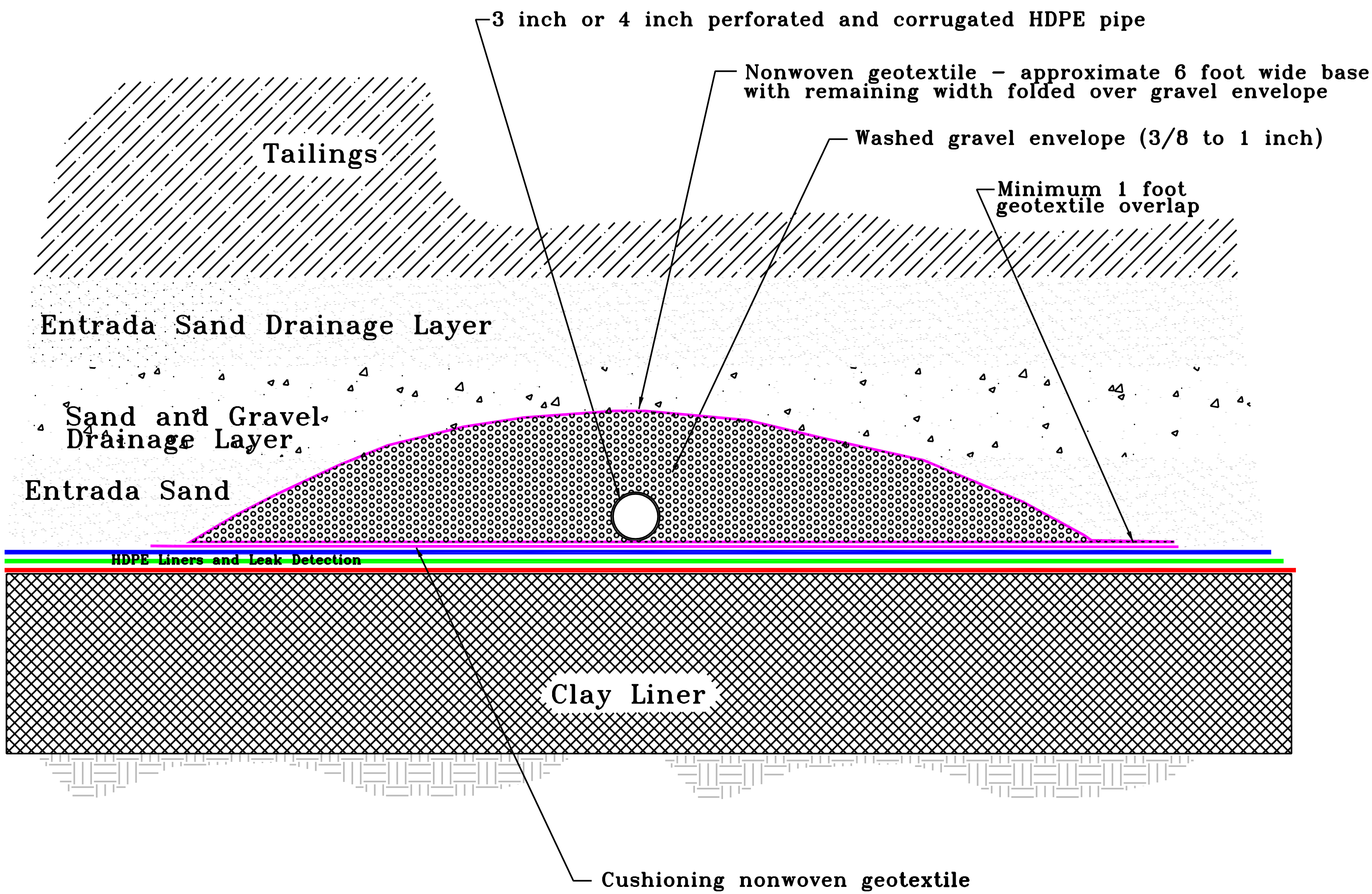
Figure 5-7. Sump Detail Cross Section 3 – 3'		
DATE: 4-2007	Liner_schematic.dwg	NOT TO SCALE
Page 5-29	HYDRO-ENGINEERING L.L.C.	



PLATEAU RESOURCES Ltd.		
Figure 5-8. Leachate Collection Pipe Installation		
REVISED: 4-2007	Fig5-8.dwg	NOT TO SCALE
Page: 5-30	HYDRO-ENGINEERING L.L.C.	

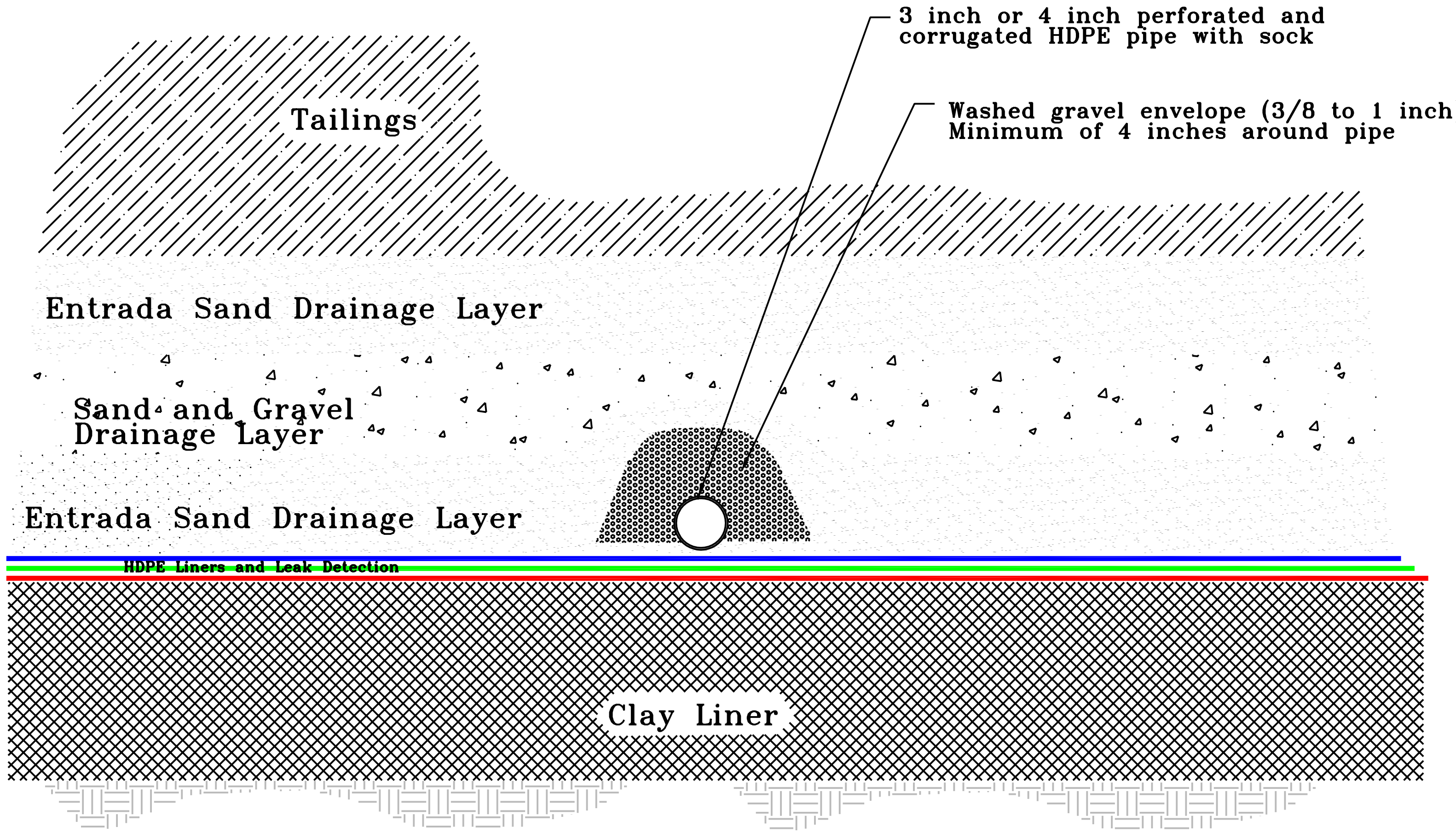


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Figure 5-9. Alternate Leachate Collection Pipe Installation

REVISED: 4-2007	Fig5-8.dwg	NOT TO SCALE
Page: 5-31	HYDRO-ENGINEERING L.L.C.	

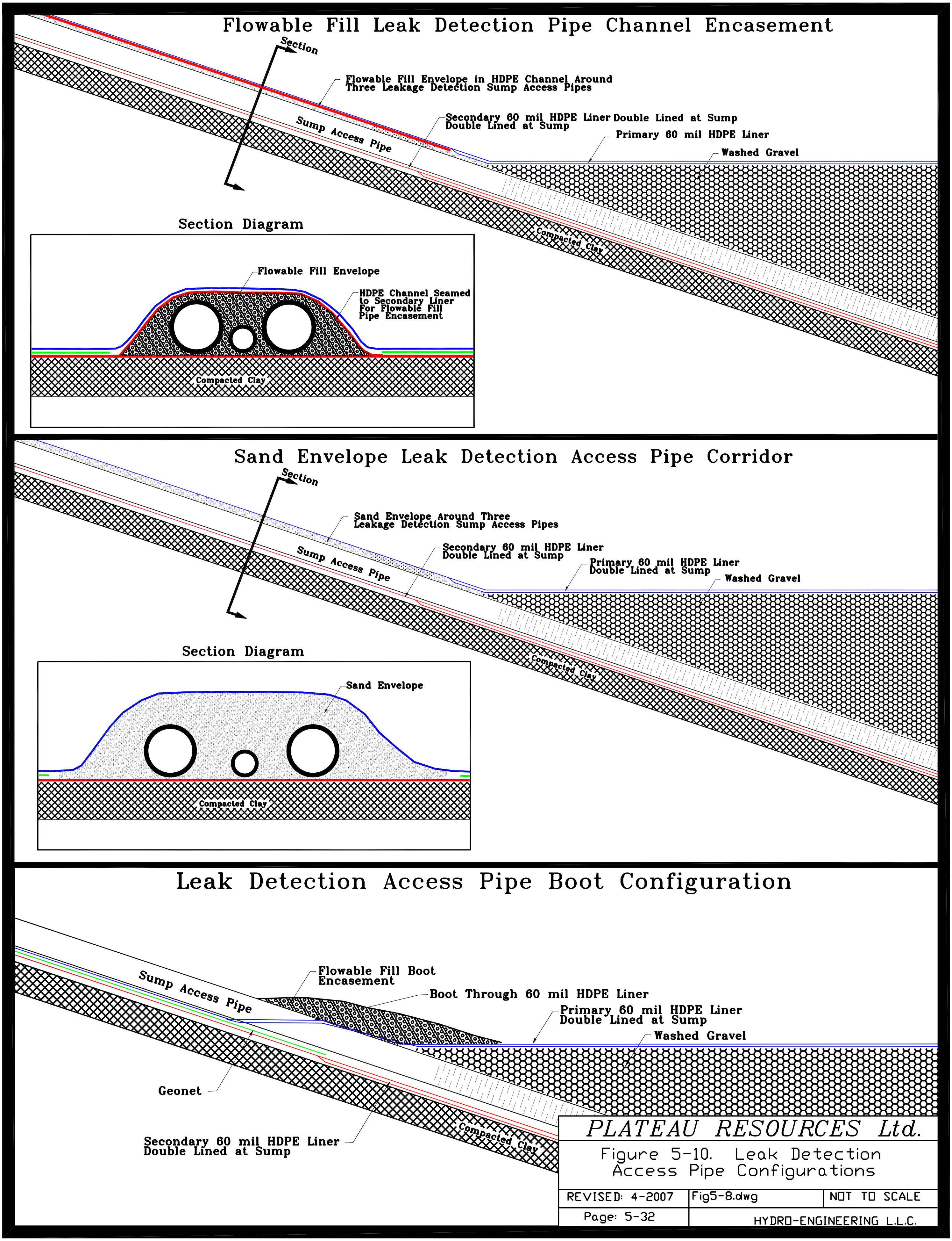
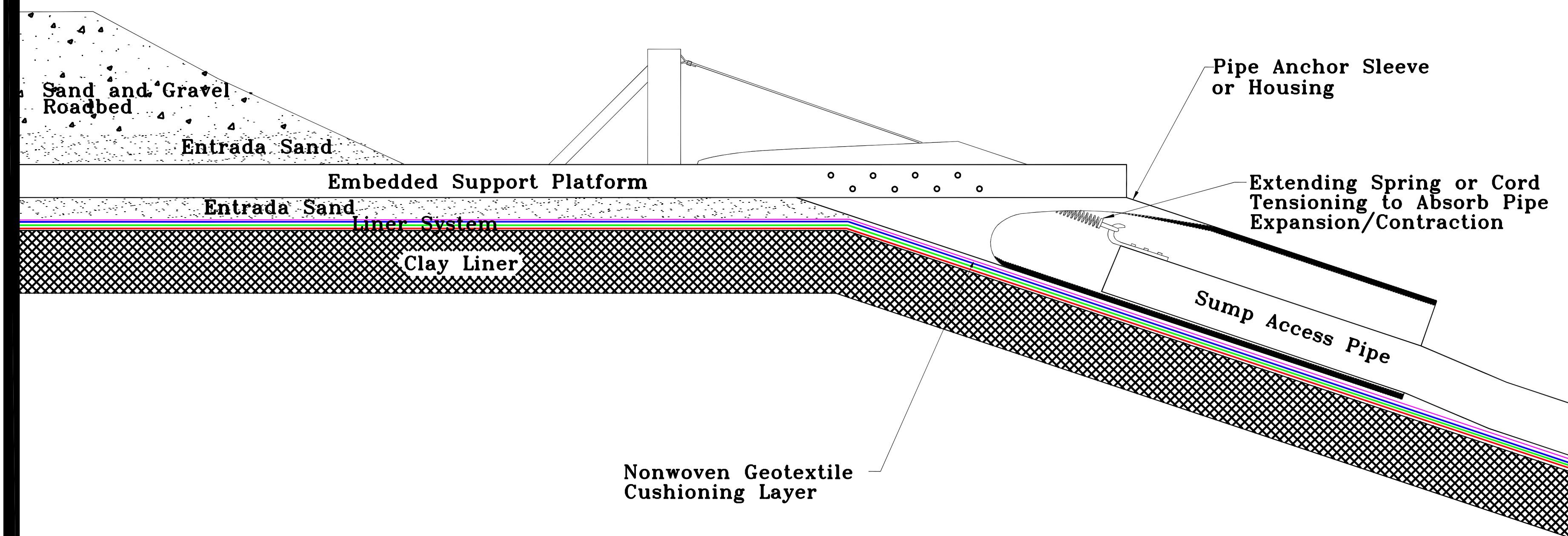


Figure 5-10. Leak Detection Access Pipe Configurations



PLATEAU RESOURCES Ltd.		
Figure 5-11. Conceptual Sump Access Pipe Termination Structure		
REVISED: 4-2007	Fig5-8.dwg	NOT TO SCALE
Page: 5-33	HYDRO-ENGINEERING L.L.C.	

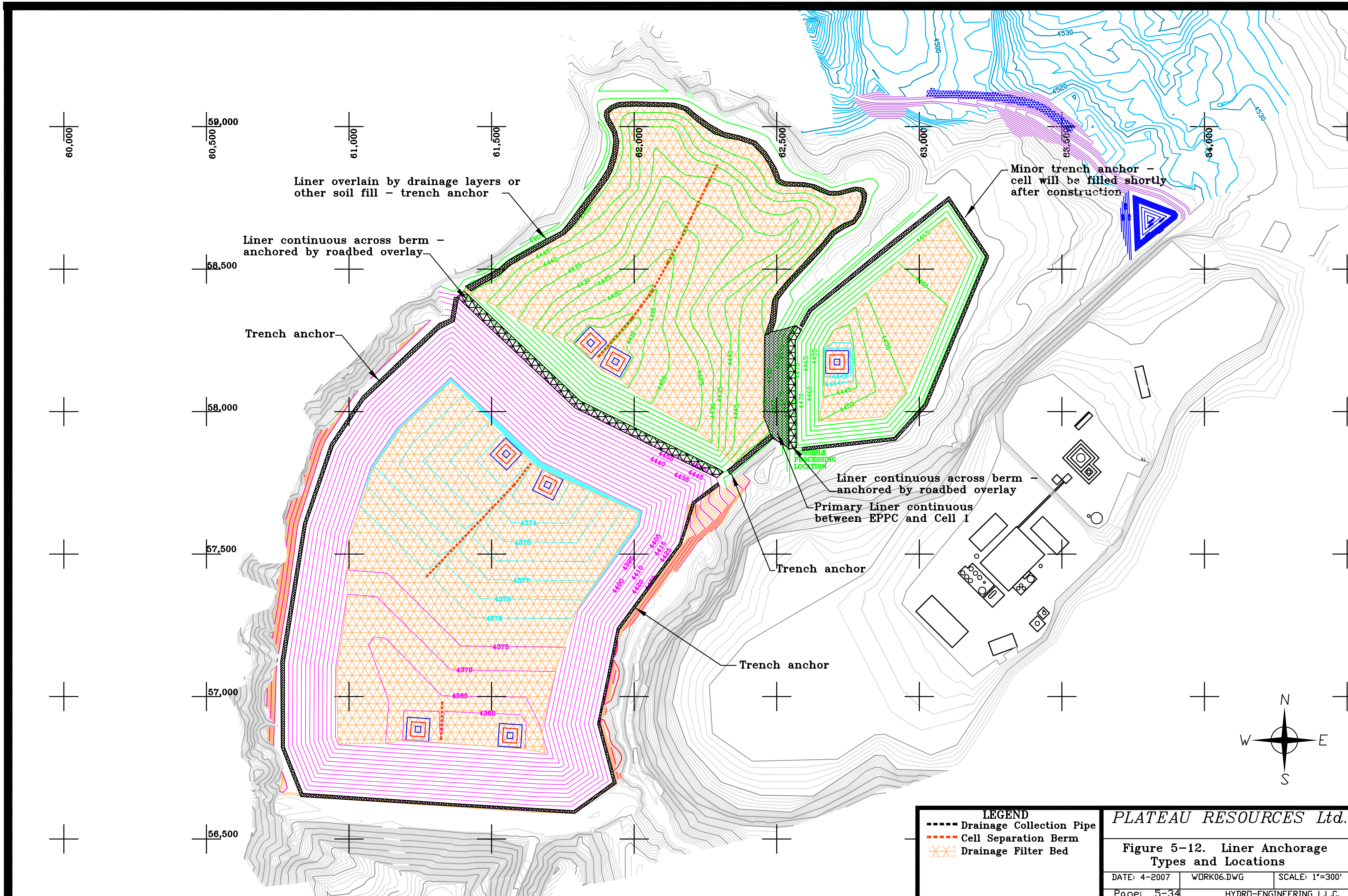


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Figure 5-12. Liner Anchorage Types and Locations

DATE: 4-2007	WORK06.DWG	SCALE: 1"=300'
Page: 5-34	HYDRO-ENGINEERING L.L.C.	

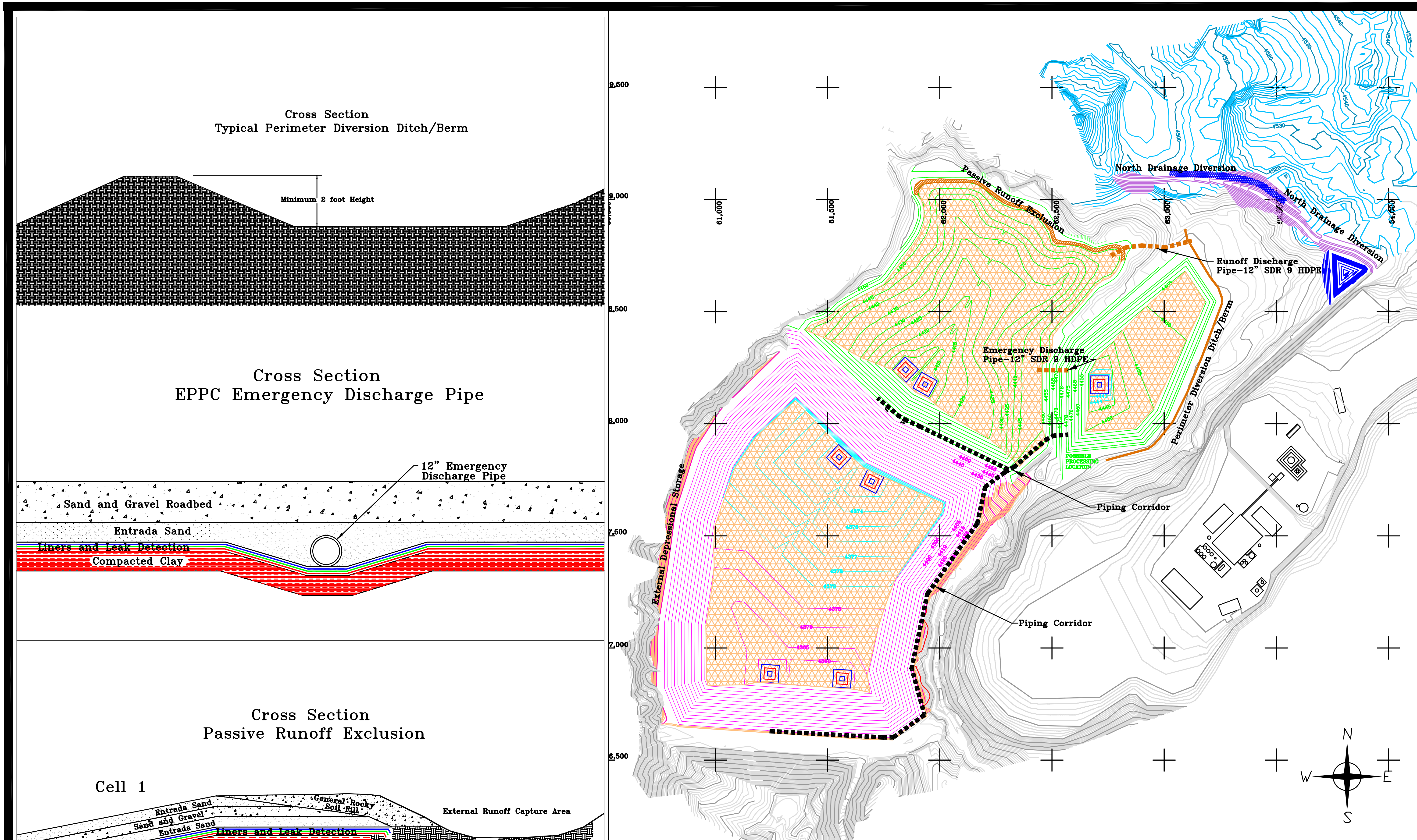

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Figure 5-13. Piping Corridors and Surface Drainage Control Features

DATE: 4-2007	WORK06.DWG	SCALE: Varies
Page: 5-35	HYDRO-ENGINEERING L.L.C.	

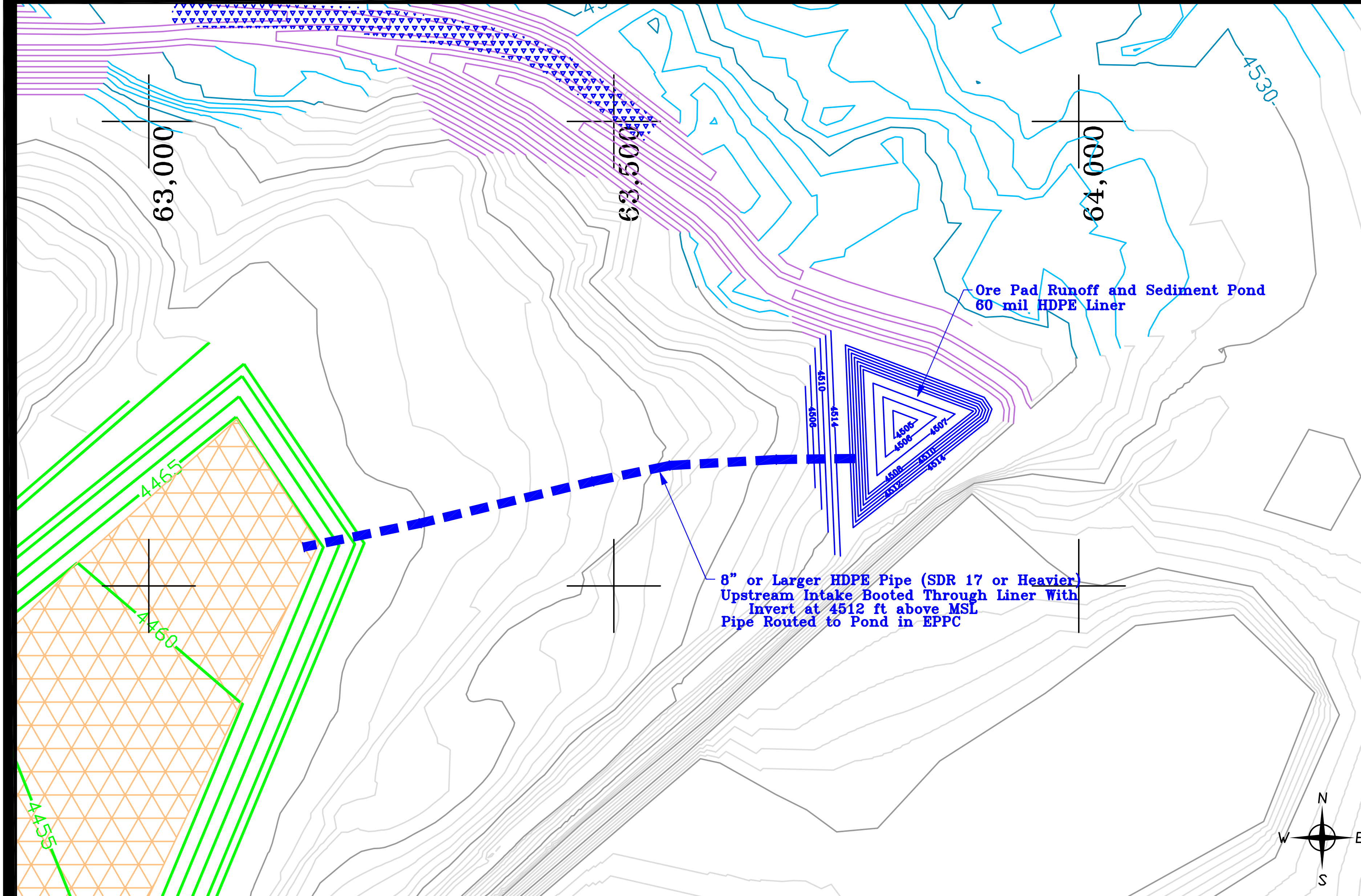

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Figure 5-14. Ore Pad Sediment and Runoff Pond

DATE: 4-2007	WORK06.DWG	SCALE: 1" = 150'
Page: 5-36	HYDRO-ENGINEERING L.L.C.	

6. TAILINGS DISPOSAL MANAGEMENT

Tailings will be transported, in the form of slurry at about 45-55 percent solids, by weight, to the fluid extraction area through a high-density polyethylene pipe. The fluid extraction area may be located adjacent to the mill, adjacent to the EPPC, or within the tailings cells. A provision will be made to allow direct discharge of the tailings slurry to the tailings cell(s) in the event of a fluid extraction failure. The discharge pipe will be supported within an HDPE-lined trench (60 mil or thicker) with a minimum depth of 12 inches, or alternatively, within an 18-inch half-round polyethylene pipe. The HDPE-lined trench or half-round pipe will contain any potential leakage from the discharge slurry pipe. This slurry pipe support will conduct any potential leakage to the impoundment by gravity flow. The fluid recycle line from the storage ponds in the EPPC will also be placed within this containment pipe or lined trench.

6.1 Tailings Cell Configuration

The tailings impoundment area has been divided into two major disposal cells and a smaller disposal cell for the existing tailings and other contaminated material. The existing cross valley berm will be reshaped and reconfigured to serve as the cell divider between Cell 1 and Cell 2. The first cell to be constructed will be the Evaporation and Process Pond Cell and this will be followed by construction of Cell 1. The use of multiple cells will allow progressive expansion of tailings capacity along with interim stabilization measures and eventually progressive reclamation of cells. The anticipated start of construction for Cell 2 will be approximately 1 to 2 years prior to reaching full capacity in Cell 1.

6.2 Fluid Extraction Processes and Tailings Placement

A thickener, belt press, or other fluid extraction equipment will be used to extract a significant portion of the fluid from the tailings slurry. This fluid will be discharged to a small HDPE-lined decant pond and subsequently delivered to the Storage/Evaporation ponds or recycled directly to the mill. All fluid storage ponds and the fluid extraction equipment will be located within the perimeter of the seven-part liner or within a constructed area near the mill. The target moisture content of the reduced-moisture tailings is 30% or less by weight. The reduced-moisture tailings solids will be delivered to the tailings cells by one of two methods. The preferred method will be a solids-handling pumping system which delivers the reduced-moisture tailings via pipeline to a distribution tower or possibly a continuously moving distribution machine which places the tailings the maximum practical lift thickness. Other possible placement alternatives include transport vehicles equipped with a hopper and conveyor unloading system or a suitable dump bed to haul the tailings to the cell.

A total lift of several feet of tailings will be placed over a large area of the base of the cell prior to placement of significant volumes of tailings within the cell to avoid load-induced displacement and damage of the liner. After the placement of the initial lift across as much of the cell base as possible, the lift thickness of subsequent tailings placement will be a function of the selected fluid extraction process. With the typical paste tailings approach, the appropriate lift thickness is relatively small (less than one foot) to facilitate rapid evaporation, stabilization and hardening. It is anticipated that a paste approach will require multiple discharge points to allow cycling or a movable discharge structure. A significant advantage of the paste placement is that, with appropriate processing and admixtures, the surface of the newly placed tailings can harden to a relatively erosion resistant crust.

With a simple belt or screw press to extract fluids from the slurry tailings, the post-extraction consistency of the tailings may range from a flowable paste material to a moist soil solid. The proposed placement methods will have to be adapted to the consistency of the tailings. If the consistency of the tailings is such that it can be pumped with a positive displacement pump, the pumping arrangement will be similar to that of a paste placement. If the tailings are in a semi-solid to solid form, the two primary alternatives are a flexible conveyor system or a transport or tram vehicle arrangement. With this arrangement, the tailings will be placed in the largest practical lift thickness to consolidate newly-placed tailings in the smallest possible area. A commercial co-polymer dust suppression agent will be applied to the newly-placed tailings when the condition at the tailings is such that there may be any wind-blown transport of tailings. During the summer months, it is anticipated that the dust suppression agent will be applied at least once a day.

The Tailings Management Plan permits a wide variation in tailings placement procedures. The duration of tailings placement in a cell may be varied and the number of points of stacking or discharge may be adjusted. These procedures may require seasonal adjustments due to the large local seasonal variations in evaporation rates. A major advantage of the planned fluid extraction process, as described, will be that most of the tailings liquid will be immediately reclaimed for reuse in the process circuit, which decreases the amount of fresh water to be consumed by the plant. Since the tailings liquid will be acidic, its recovery will have an important effect on the total acid requirements of the plant. As previously noted, tailings placement will start in Cell 1, which is located at the impoundment basin. The available tailings disposal volume in the first cell is sufficient to store the tailings from the first three to four years of plant operation.

6.3 Conventional Slurry Tailings Placement

It is likely that the fluid extraction operations will have to be temporarily suspended for maintenance or repair operations. In order to allow continued milling during these periods, a tailings slurry discharge line to the tailings will be maintained. The discharge point will be in a depression within a tailings cell where there is no potential for discharge of ponded fluids outside of the liner containment. This discharge point will have to be periodically shifted as the tailings accumulate in the cell. The leachate collection system is designed to accommodate conventional slurry placement and redundant pumps with a wide range of combination capacities will be installed to evacuate the leachate collection sumps. Therefore, the only required adjustment in tailings cell operation for a temporary suspension of fluid extraction is possibly some switching of pump sequencing. The tailings placed as slurry will not require any further processing and will be dewatered by drainage to the leachate collection system.

In the event that reduced-moisture tailings handling is suspended for an extended period of time and the conventional hydraulic slurry placement is used, tailings discharged to the cells will be located within the boundary of the lined cell with a sequential rotation of the discharge location to all the corners of each cell. Present expectations are to discharge the entire flow of tailings slurry from a single spigot at one corner of a cell. This flow may be continued for a period chosen to provide efficient cell operation before the discharge is shifted to the lowest corner of the cell. With the hydraulic placement, the sand and slime fractions of the tailings will segregate as they are discharged to the cells, with the sand depositing nearer the point of discharge and the slimes flowing to the lowest area within the cell (which will continuously be shifting in location because of the shifting discharge points). Since each layer of slimes will collect and stabilize in the lowest part of the cell and since the next tailings discharge will be from the lowest corner of that cell, each layer of slimes should be largely covered by sand. Ultimately, the central part of each cell will be filled with alternating layers of sand and slimes lying in a helical configuration. The cell perimeter will consist mainly of tailings sand. This configuration will facilitate drainage and consolidation of the slimes, and will lead to continuous burial of that part of the tailings containing most of the residual radioactivity in the processed ore.

7. COMPLIANCE MONITORING

All environmental and radiological monitoring will be in accordance to the standard operating procedures (SOP's) as detailed in the *Plateau Resources Limited Administrative Procedures, Environmental Protection Procedures and Radiation Protection Procedures, Radiological and Environmental Monitoring Program found in Table 5.5-7 and 5.5-8* (March 1, 1996 Renewal Application) and State of Utah Water Quality Discharge Permit. The tables and Discharge Permit include the ground-water monitoring schedule along with all other types of monitoring.

7.1 Ground-Water Monitoring Quality Assurance Plan

Ground water is monitored at the locations specified in Table 5.5-7, and 5.5-8 and the Discharge Permit. These locations are selected to monitor any seepage entering surface waters or ground water from the tailings impoundment during operations. Further details are provided in the Ground-Water Monitoring Quality Assurance Project Plan dated August 22, 2006.

The seventeen ground water monitoring well locations were selected using the following criteria stipulated in Regulatory Guide 4.14 and in the EPA Health and Environmental Protection Standards for Uranium Mills, 40 CFR 192, Subpart D and State of Utah Discharge Permit:

1. Ground water hydrologically down gradient and relatively close to the tailings impoundment and hydrologically up gradient, i.e., not influenced by potential seepage from tailings.
2. Criteria to be used as indicator chemical and radiological parameters for early detection of potential tailings seepage allow for simplified but efficient monitoring program.
3. No surface waters leave the mill facility or tailings facility, all drainage flows into the tailings impoundment. No monitoring of surface water is expected to be necessary at this site.

7.1.1 Location, Number and Type of Ground-Water Monitoring Wells.

Two upgradient monitoring wells and five downgradient monitoring wells, all located with respect to the uranium mill tailings impoundment, are sampled in accordance with Ground Water Quality Discharge Permit UGW 170003. The upgradient wells RM-1 and RM-12 are located immediately north of the tailings impoundment. Well RM-14 is located on the west side of the tailings impoundment while well RM-2R is located to the east. The remaining wells, RM-7, RM-18 and RM19 are located downgradient of Cell 1. A summary table of the well depths and screen locations for each of the above wells is included in Section D.3 of Appendix D. This table is duplicated from Table 3-1 of "Ground-water Hydrology of the Shootaring Canyon Tailings Site – 2005".

7.1.2 Monitored Parameters and Frequency.

Monitoring wells RM-1, RM-2R, RM-7, RM-12, RM-14, RM-18 and RM-19 will be sampled semiannually in accordance with Ground-Water Quality Discharge Permit UGW 170003 and the Ground-Water Quality Assurance Plan dated August 22, 2006.

Wells RM-23 through RM-32 will replace wells RM-7, RM-18 and RM-19 prior to the construction of Cell 2.

Ground-water surface elevation will also be measured semiannually to calculate ground- water flow rate and direction in the uppermost aquifer.

7.1.3 Sampling and Analytical Techniques

Ground-water samples will be obtained according to procedures outlined in the Ground-Water Quality Assurance Plan dated August 22, 2006. Each sample will be filtered, preserved and analyzed using EPA analytical procedures or the equivalent. The sampling results will be used to determine whether a significant increase in any constituents has occurred and to provide reasonable confidence that the migration of hazardous constituents from the tailings impoundment into and through the aquifer will be indicated.

7.1.4 Background Levels.

Background data for various constituents for the ground-water monitoring program are being collected prior to the operation of the facility. The background data will be used to define the natural range in concentration for each constituent.

Action levels for the ground-water monitoring program are based on sampling results and trend analyses. If individual sampling results exceed the upper limit of the range of natural background for ground water samples which are obtained within the restricted area of the mill, or if trends of increasing concentration with time are observed, the ERHS staff will investigate to determine the cause of the water quality changes. Corrective actions involve identification of the source of the contamination and possible mitigating measures, such as the installation of ground-water flow barriers or seepage pump-back systems. Currently, all analyses are performed by commercial laboratories. These commercial laboratories will be Utah certified. During operations, analysis may be completed by the mill laboratory if it is Utah certified and at commercial laboratories with various commercial laboratories utilized for quality assurance on an as needed basis.

7.1.5 Exceed Site Standards

Site standards have not been set for the Shootaring site. Additional background monitoring data is being collected and needs to continue as long as possible to best define the full range of natural background concentrations. Site standards will then be developed based on the historical background data set.

7.2 Leak Detection System Recording and Fluid Transfer

To insure that the primary upper liner is functioning properly, a continuous liquid detection recorder will be installed in the sump(s) which collect liquid from between the two 60 mil HDPE liners. Any indication of leakage will result in pumping the liquid into an operating tailings or evaporation cell when necessary. The pumping assembly will be connected to an alarm and light to monitor the pumping systems operation. Weekly evaluations will be made to determine the quantity of liquid, if any, due to leakage. Initial measurement and evaluation frequency may be higher until the system performance is documented.

7.3 BAT Performance Monitoring Plan Leak Detection

The quantity and rate of any leakage collected in the sump(s) will be measured at least once per week. Initial frequency of measurement may be higher until a record of system performance is developed. Any leakage that is collected will be delivered to the Storage/Evaporation Pond for disposal through evaporation or recycle through the mill. The maximum allowable leakage rate is 200 gallon per day per acre. The action leakage rates for each sump are presented in Table 5-3 and are discussed in Section 5.1.4.7. The maximum allowable head on the leak detection system is three feet above the top of the individual leak detection sump.

7.4 Other Environmental Monitoring

Tables 5.5-7 and 5.5-8 which are presented in Sections D.1 and D.2 of Appendix D present the monitoring programs for direct radiation, soil, vegetation, and meteorology. Figure 7-1 presents the monitoring locations. The operational monitoring program and interim monitoring programs were designed to meet the following criteria presented in Regulatory Guide 4.14:

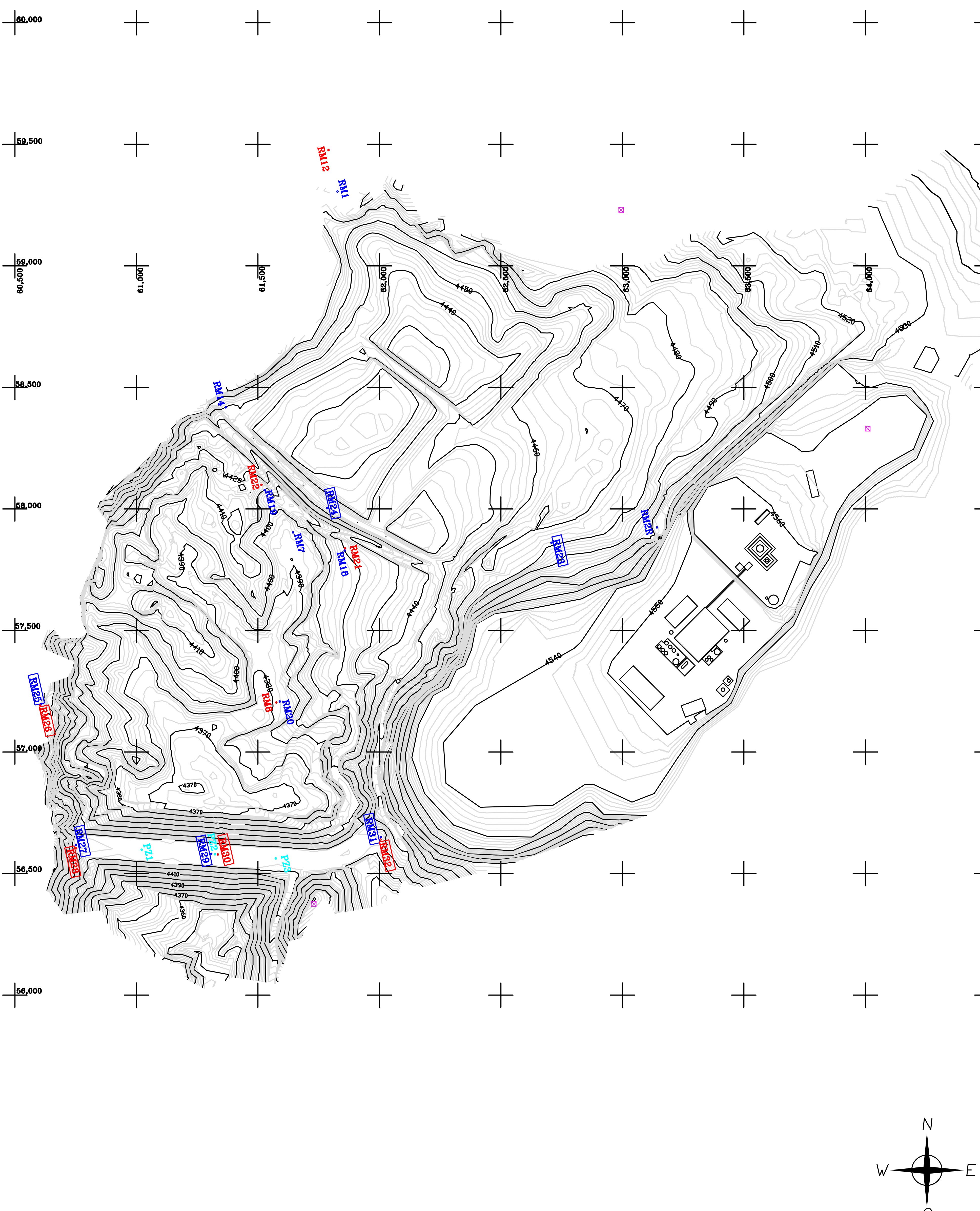
1. Sample vegetation from animal grazing areas near the mill site in the direction of the highest predicted airborne radionuclide concentrations.

2. Sample soils and measure gamma radiation at each of the locations chosen for air particulate samples.

Increasing trends for a monitored parameter will be investigated by the CRSO or his/her staff to determine the cause and identify potential corrective actions.

Meteorological monitoring during operations consists of continuous wind speed and direction measurements recorded on strip charts. Digital logging equipment may also be used for meteorological monitoring. That information is of value in the unlikely event of a puff-type release from one of the mill stacks. During the interim operational status of the mill, the monitoring program for meteorological monitoring is suspended.

Fish sampling and sediment sampling is not conducted because of the lack of streams flowing through or near the processing facility.



LEGEND

RM12 ○ UPPER ENTRADA WELL
RM1 ○ ENTRADA WELL
RM27 ○ PROPOSED UPPER ENTRADA WELL
RM28 ○ PROPOSED ENTRADA WELL
PZ1 ○ DAM PIEZOMETER
⊠ AIRBORNE PARTICULATE SAMPLE LOCATION

PLATEAU RESOURCES Ltd.		
FIGURE 7-1. LOCATION OF MONITORING WELLS		
DATE: 4-2007	MONITORING.DWG	SCALE: 1"=400'
Page 7-4	HYDRO-ENGINEERING L.L.C.	

8. CONTINGENCY PLANS

The following contingency plans are presented for the tailings facility elements. These contingency plans address plausible events that can reasonably be expected to impact the tailings facility or result in the potential release of tailings or tailings solution.

8.1 Tailings Liner – Leak Detection System

If the collection rate from the leak detection sump exceeds the allowable rate of 200 gallon per day per acre, a series of steps will be taken to reduce the rate of discharge from the leak detection system.

If the change in rate of discharge from the leak detection system is fairly abrupt and indicates a new contact with a liner puncture, recent locations of tailings placement or tailings solution ponding will be examined for liner damage. This may include excavating through recently placed tailings or evacuating ponded solution to try to expose the area of the liner where the leak is likely to be located. If a damaged section of liner is located, the liner will be repaired and tested. During this process, the location of tailings placement will be changed or the tailings placement will be suspended.

If the contributing punctures in the primary liner cannot be located, all ponded tailings solution will be pumped from the suspect area to an adjacent cell or to the most distant practical location within the cell. If the rate of discharge to the leak detection subsequently declines to acceptable levels, restrictions will be placed on the moisture content of tailings that can be placed with the area of the cell where the leak occurred. Only reduced-moisture tailings will be allowed to be placed in the section of the cell contributing to the sump where the allowable leak detection rate was exceeded. No ponding of solution will be allowed within the section of the cell contributing to the leak detection sump.

8.2 Tailings Liner – Evidence of Bottom Liner Loss of Integrity

If there is evidence of seepage from the tailings system detected in the ground water, the nature and probable location of the source of the seepage will be evaluated. All water levels in the tailings leachate collection and leak detection systems will be measured and the sumps will be continuously evacuated to the lowest possible level. If the cell or a portion of a cell can be identified as the source of the seepage, tailings placement and/or solution discharge to that area will immediately be suspended. Additional monitoring wells may be installed and a Corrective Action Program will be evaluated.

8.3 Excess Tailings Solution or Runoff Volume

Excess solution or runoff water captured within the tailings disposal cells will be transferred to the storage/evaporation pond within the EPPC if possible. If there is not sufficient capacity in the storage/evaporation pond, any fluids that cannot be evaporated in a reasonable period of time will be distributed over the tailings cell surface to increase the evaporative surface area. This distribution system may include sprinklers, sprays, and commercial fan enhanced spray units to accelerate the evaporation process.

Various stormwater control measures are specified to limit drainage area to the tailings cell(s). These measures are described in Sections 5.1.6 through 5.1.6.6. The result of implementation of these measures is that the runoff contribution of external areas to the tailings cells is expected to be relatively small. The containment of excess fluids from runoff may become a concern as the tailings cell(s) approach capacity. The sequential construction and continuous liner system between Cells 1 and 2 will limit the periods when fluid containment capacity is a concern to two intervals. At the point when Cell 1 is approaching capacity, the containment capacity in Cell 1 will be limited to available fluid storage below the cell crest liner elevation of 4455 feet above MSL. A minimum of 20 acre-feet of fluid storage will be maintained below the containment liner elevation of 4455 feet above MSL in Cell 1 until the liner is completed in Cell 2. If the decision is made to discontinue milling prior to construction of Cell 2, a stormwater containment contingency plan will be developed and submitted to the state to allow continued usage of Cell 1 and a waiver of the requirement to maintain 20 acre-feet of fluid storage. The contingency plan will likely include construction of internal ponds within the tailings along with high capacity transfer pumps.

After Cell 2 is in use, a total of 60 acre-feet of fluid storage will be maintained within Cell 2 or a combination of Cell 1 and Cell 2. When Cell 2 is approaching capacity, a stormwater containment contingency plan will be developed and submitted to the state to allow complete utilization of Cell 2 and a waiver of the requirement to maintain 60 acre-feet of fluid storage.

9. Mill Ore Pad

9.1 Geotechnical Review

A geotechnical review on the ore pad liner has been completed and submitted to the State of Utah Division of Radiation Control. The study found that there are 12 to 14 inches of clay material covering the ore pad. This clay material has a hydraulic conductivity of 3.7 E-06 cm/sec. The ore pad is currently designed to have the small surface drainage area diverted into the tailings facility. A HDPE-lined sediment pond will be constructed northwest of the ore pad, and runoff from the ore pad will be diverted to this pond. Excess water in the sediment pond will be transferred to the EPPC. With the clay pad and diverted surface drainage, seepage from the ore pad is minimal. The ore pad report is presented in Section E.1 of Appendix E.

10. Stability of Previously Deposited Tailings Material

The previously deposited tailings material and associated radiologically contaminated material will be excavated and deposited within the lined EPPC. Single HDPE-lined ponds will be constructed within the EPPC on top of the tailings and contaminated material that has been transferred to the lined EPPC. These ponds will be used for storage and evaporation of water. The stability of material transferred to the EPPC is not expected to be a concern since it is currently dewatered and will be transported and placed in dry form.

11. References

Koerner, R.M. 2005, Designing With Geosynthetics – Fifth Edition. Prentice Hall, Upper Saddle River, NJ.

Woodward-Clyde, 1978a. June 16, 1980 revision. Environmental Report, Shootaring Canyon Uranium Project, Garfield County. Prepared for Plateau Resources Limited by Woodward-Clyde Consultants.

Woodward-Clyde, 1978b. June 1978. Supplement S1 Environmental Report, Shootaring Canyon Uranium Project, Garfield County, Utah. Prepared for Plateau Resources Limited.

Woodward-Clyde, 1978c. September 1978. Supplement S2 Environmental Report, Shootaring Canyon Uranium Project, Garfield County, Utah. Prepared for Plateau Resources Limited.

APPENDIX A

TAILINGS STABILITY AND DEFORMATION ANALYSES

APPENDIX A

TABLE OF CONTENTS

A.1 Seismic Stability Analysis, Letter Report
by Inberg-Miller Engineers, January 9, 1997

INBERG-MILLER ENGINEERS
124 EAST MAIN STREET RIVERTON, WYOMING 82501-4397 307-856-8136

January 9, 1997 7664-RX

U.S. Energy
877 North 8th West
Riverton, Wyoming 82501

ATTENTION: KEN WEBBER

SEISMIC STABILITY ANALYSIS RESULTS
STAGE I - SHOOTERING CANYON DAM
GARFIELD COUNTY, UTAH

Gentlemen:

This letter summarizes the results of a seismic stability analysis we performed for Stage I of the Shootering Canyon Dam in Utah. Our services were performed in accordance with our November 11, 1996 Service Agreement and Proposal.

You provided the following documents for our review:

- "Tailings Management Plan and Geotechnical Engineering Studies - Shootering Canyon Uranium Project", by Woodward-Clyde Consultants dated September, 1978.

- "Stage I - Tailings Impoundment and Dam - Final Design Report, Shootering Canyon Uranium Project", by Woodward-Clyde Consultants dated May 1979.

- "Stage I Tailings Impoundment and Dam Field Density Test in Zone 2 Material - Shootering Canyon Uranium Project", by Woodward-Clyde Consultants dated November 13, 1980.

- "Earthwork Quality Control Overview and As-Built Drawings - Construction of Stage I Tailings Impoundment and Dam - Shootering Canyon Uranium Project" by Woodward-Clyde Consultants dated July 28, 1982.

We developed our understanding of dam geometry, geologic conditions, and engineering properties of soils which comprise the dam according to the above documents as a basis for modeling the dam for analysis.

As requested, we also reviewed the following document which is contained in our files:

- "Seismic Hazard Analysis of Title II Reclamation Plans", by Lawrence Livermore National Laboratory dated June 26, 1994.

SLOPE CONDITIONS AND PARAMETERS

We understand the Stage I of the Shootering Canyon Dam was completed in 1982. The dam is an earthen structure designed to impound uranium mill tailings. It has a crest elevation of 4433 feet and a maximum height of approximately 85 feet. The design maximum surface elevation of impounded tailings is 4420 feet. Tailings are assumed to be saturated.

In general, the dam is comprised of 3 zones as shown on Figure 5, Section C -C from July 28, 1982 Earthwork Quality Control Overview and As-Built Drawings - Construction of Stage I Tailings Impoundment and Dam - Shootering Canyon Uranium Project. Zone 1 is the core of the dam, extending from the base to the crest, which is "silty sandy clayey" soil. Zone 1 is key-in to the rock foundation at the base. Zone 3 adjoins the core on the upstream and downstream sides, also extending from the base to the crest, which is "fine sand". Zone 2 forms the upstream and downstream face of the dam outside of Zone 3, and is described as "boulders, cobbles, gravels, and sand". We also understand an additional 2.25-foot thick layer of 18" rip-rap will extend from the downstream toe up the face a height of 15 feet. Soil descriptions for each soil zone were as defined by Woodward-Clyde Consultants in their above referenced reports. A copy of Section C-C that we referred to for modeling the slope is contained in Exhibit A.

Based on information provided by Plateau Resources Limited (PRL), we understand that the tailings will be contained by a liner and collection system. The liner system will consist of a double-layer 60 mil HDPE liner with leak detection, and will extend up the upstream face of the dam to the crest. Accordingly, our slope stability analysis assumes there is no phreatic surface through the dam.

The soil properties of the different units which are part of the dam system were taken from Table C-1 for operating conditions and seismic conditions in the May 1979 Stage I - Tailings Impoundment and Dam Final Design Report. Based on the Nov. 13, 1980, letter regarding Stage I Tailings Impoundment and Dam Field Density in Zone 2 Material by Woodward-Clyde Consultants, the unit weight of Zone 2 soil was increased from 125.0 to 131.0 pcf. A copy of Table C-1 and the Nov. 13, 1980 letter is contained in Exhibit B. The soil properties we used are summarized below:

Soil Number	Description	Unit Wt. (pcf)	Cohesion (psf)	Friction Angle (ϕ)
1	Zone 1 - Silty Sandy Clayey Soil	125	1500	0
2	Zone 2 - Boulders, cobbles, gravels, sand	131	0	40
3	Zone 3 - Fine sand	125	0	32
4	Rock Foundation	140	1000	45
5.	Tailings	100	0	10

PRL requested that we use a horizontal seismic coefficient of 0.19 g based on "Seismic Hazard Analysis of Title II Reclamation Plans", by Lawrence Livermore National Laboratory. A copy of the report section, to which PRL referred us, is contained in Exhibit C.

ANALYSIS RESULTS

We performed a slope stability analysis using the computer program PCSTABL version 5M, and the parameters which were described above. Stability analyses were performed in accordance with Bishop and Janbu methods which are available as options on PCSTABL. Per PRL's request, we analyzed the downstream slope assuming a full tailings pool (surface elevation 4420 feet). No other configurations were requested or analyzed.

The lowest safety factor (1.14) was determined using the Janbu method for the downstream face of the dam. The critical failure surface determined with PCSTABL is characterized as an "infinite slope failure" which is planar and parallel to the slope face, and typical of failure surfaces in granular soil. The safety factor calculated with PCSTABL compares favorably with manual calculations for an "infinite slope" using a soil friction angle of 40 degrees and a horizontal seismic coefficient of 0.19 g. Input and plot files for the PCSTABL critical failure surface are included in Exhibit D.

INBERG-MILLER ENGINEERS

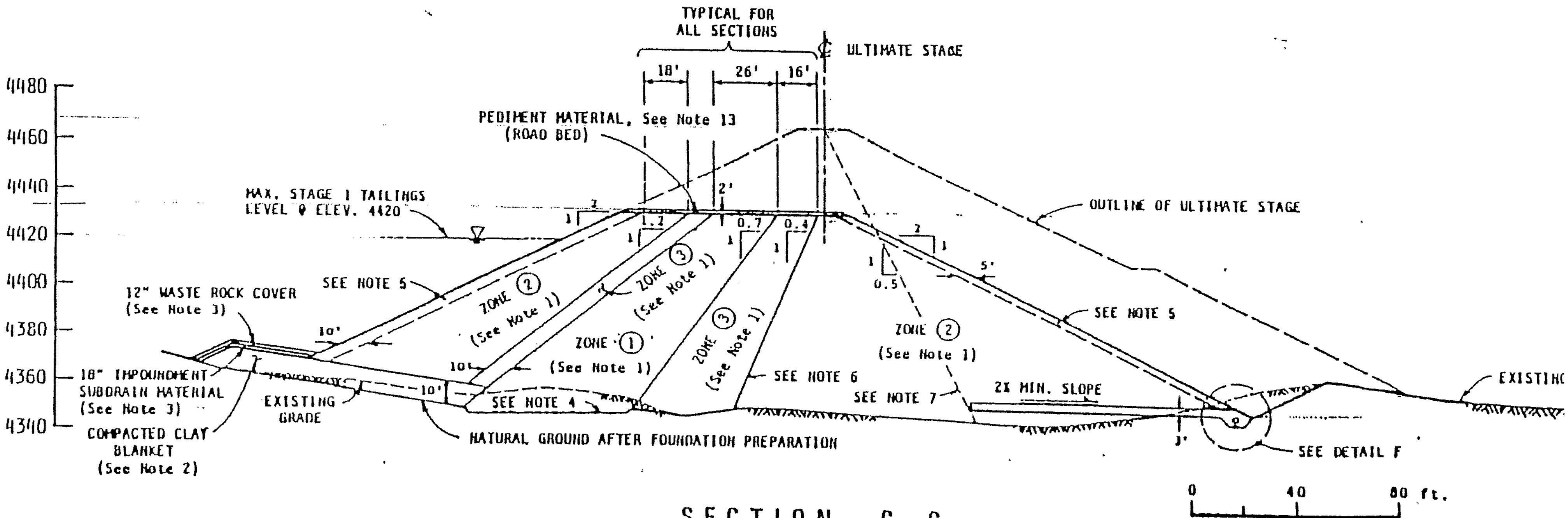
Glen M. Bobnick

Glen M. Bobnick, P.E.
Geotechnical Engineer

Exhibit A - Existing Conditions
Exhibit B - Soil Properties
Exhibit C - Seismic Hazard Analysis
Exhibit D - Stability Analysis Results

EXHIBIT A - EXISTING CONDITIONS

Figure 5, Section C -C from July 28, 1982 Earthwork Quality Control Overview and As-Built Drawings - Construction of Stage I Tailings Impoundment and Dam - Shootering Canyon Uranium Project



TYPICAL FOR ALL SECTIONS
18'
26'
16'
ULTIMATE STAGE
PEDIMENT MATERIAL, See Note 13 (ROAD BED)
2'
MAX. STAGE 1 TAILINGS LEVEL @ ELEV. 4420
OUTLINE OF ULTIMATE STAGE
1.2
0.7
0.4
0.5
5'
SEE NOTE 5
ZONE 2 (See Note 1)
ZONE 3 (See Note 1)
ZONE 1 (See Note 1)
ZONE 3 (See Note 1)
ZONE 2 (See Note 1)
SEE NOTE 5
12" WASTE ROCK COVER (See Note 3)
10'
SEE NOTE 6
SEE NOTE 7
2% MIN. SLOPE
EXISTING
18" IMPOUNDMENT SUBDRAIN MATERIAL (See Note 3)
EXISTING GRADE
SEE NOTE 4
COMPACTED CLAY BLANKET (See Note 2)
NATURAL GROUND AFTER FOUNDATION PREPARATION
SEE DETAIL F
3'
0
40
80 ft.
4480
4460
4440
4420
4400
4380
4360
4340
ELEVATION, FEET

SECTION C - C

EXHIBIT B - SOIL PROPERTIES

Table C-1 from May 1979 Stage I - Tailings Impoundment and Dam Final Design Report, and November 13, 1980 letter regarding State I Tailings Impoundment and Dam Field Density Test in Zone 2 Material

TABLE C-1

SOIL PROPERTIES USED IN STABILITY ANALYSES

Material	Soil Number (Figure D-3)	Unit Weight (pcf)	Effective Strength Parameters C'(psf)	Effective Strength Parameters Ø'(°)	Total Strength Parameter C(psf)	Total Strength Parameter Ø(°)
Zone 1*	1	125	0	26 (1)	900 (2) 2200 (4) 1500 (5)	13 (2) 0 (4) 0 (5)
Zone 2 (3)	2	125	0	40	-	-
Zone 3 (3)	3	125	0	32	-	-
Clay Blanket*	1	125	0	26 (1)	900 (2) 2200 (4) 1500 (5)	13 (2) 0 (4) 0 (5)
Tailings (3)	5	100	0	10	-	-
Rock Foundation (3)	4	140	1000	45	-	-

(1) Parameters for operating conditions - static condition

(2) Parameters for end of construction - static condition

(3) Effective strength parameters for these materials apply to all conditions

(4) Parameters for end of construction - seismic condition

(5) Parameters for operating conditions - seismic condition

* Estimates strength values to be confirmed and presented with additional stability analyses in supplemental report to be submitted by June 5, 1979.

Three Embarcadero Center, Suite 700
San Francisco, California 94111
415-956-7070

Woodward·Clyde Consultants

November 13, 1980

Project: 60255N

Plateau Resources Limited
772 Horizon Drive
Grand Junction, Colorado 81501

Attention: Mr. U.K. Gupta

Gentlemen:

STAGE 1 TAILINGS IMPOUNDMENT AND DAM
FIELD DENSITY TEST IN ZONE 2 MATERIAL
SHOOTERING CANYON URANIUM PROJECT
Garfield County, Utah

As required in Amendment No. 1 to the Source Material License SUA 1371, and as discussed during the NRC site inspection on November 5, 1980, a field density test is required in the Zone 2 material for every 50,000 cubic yards of Zone 2 material placed. Because of the wide range of grain sizes, the conventional testing being used for Zone 1 and 3 is not applicable. The first test was completed on November 6, 1980, and the results show that the Zone 2 material is being compacted to a dry density of 131 pcf. This value is well above the estimated 125 pcf used in the stability analysis for the dam (WCC Report, May, 1979), hence stability being achieved is well in excess of the minimium requirements.

Attached is a copy of the test procedures for the field density test discussed above. These procedures will also apply to the remaining density tests to be performed in the Zone 2 material. The total time required to complete the field portion of the test is about 1-1/2 hours provided all of the necessary equipment and labor is present at the onset of the test.

If you have any questions concerning the contents of this letter, please contact Mr. Bernard Gordon or the undersigned.

Sincerely yours,

Don A. Poulter

Don A. Poulter
Staff Engineer

sme

Enclosure

cc: w/Enclosure
Bill Luhrs (PRL)
PRL Field File
(c/o S. Ankrum)
R. Duncan (Garco)
D. Rose (Garco)
D. Staton (MSME)
M Brown (MSME)

ZONE 2 FILL DENSITY TESTS PROCEDURES

SHOOTERING CANYON URANIUM PROJECT

Garfield County, Utah

---

1) Select a representative area approximately six to seven feet square. The area should be approximately level or require only minimal grading.

2) Excavate a pit approximately 6ft. x 6ft. x 3ft. The corners and bottom may be rounded. As the material is excavated, carefully load it into a clean, empty truck making sure that no material is wasted or lost.

3) Trim the loose material off the sides and bottom of the pit by hand. Place this material into the truck making sure that no material is wasted or lost.

4) After all of the material has been loaded into the truck, weigh the loaded truck on calibrated scales (+10 lbs. is desired); dump the material and weigh the truck empty. If the scales are not on site, the material should be covered with a tarp to minimize moisture evaporation during travel.

5) After weighing, collect a sample (approx. 2-1/2 lbs.) of the finer material (minus 2 inch) and determine its moisture content. The sample should be representative and not contain material reduced in moisture from evaporation.

6) Line the excavated pit with a flexible sheet of plastic, approximately 10 mils thick. The linear should be loosely fitted so that it may conform to the sides of the pit as it is filled with water. The plastic should overlap the top by two or three feet.

7) Using a calibrated meter or calibrated container, carefully fill the pit with a measured volume of water. Once the water level reaches the top of the pit, stop the test and record the volume of water placed in the pit. If the top of the pit is not level, measure the unfilled portion and determine its volume. (For this reason, it is best to excavate a square or rectangular pit).

SHOOTERING CANYON URANIUM PROJECT

(Continued)

---

8) After the test is completed and all of the data are recorded, empty the pit by pumping out the water, and discharge it into an area that will not adversely affect the construction or performance of the dam.

9) Backfill test pit to original grade with material recompacted to same density.

10) Calculate the density of compacted Zone 2 material, using the attached form. A copy of the completed test form should be sent to WCC.

FIELD DENSITY TEST

ZONE 2 MATERIAL

SHOOTERING CANYON URANIUM PROJECT

Garfield County, Utah

Test No: Date:

Tested by:

Supervisor(s) Present:

Weight of Truck plus Material:

Weight of Empty Truck:

Weight of Excavated Material:

Wet Weight of Moisture Sample:

Dry Weight of Moisture Sample:

Moisture Content:

Gallons of Water:

Volume of Water:

Volume of Unfilled Portion of Pit:

Total Volume of Pit:

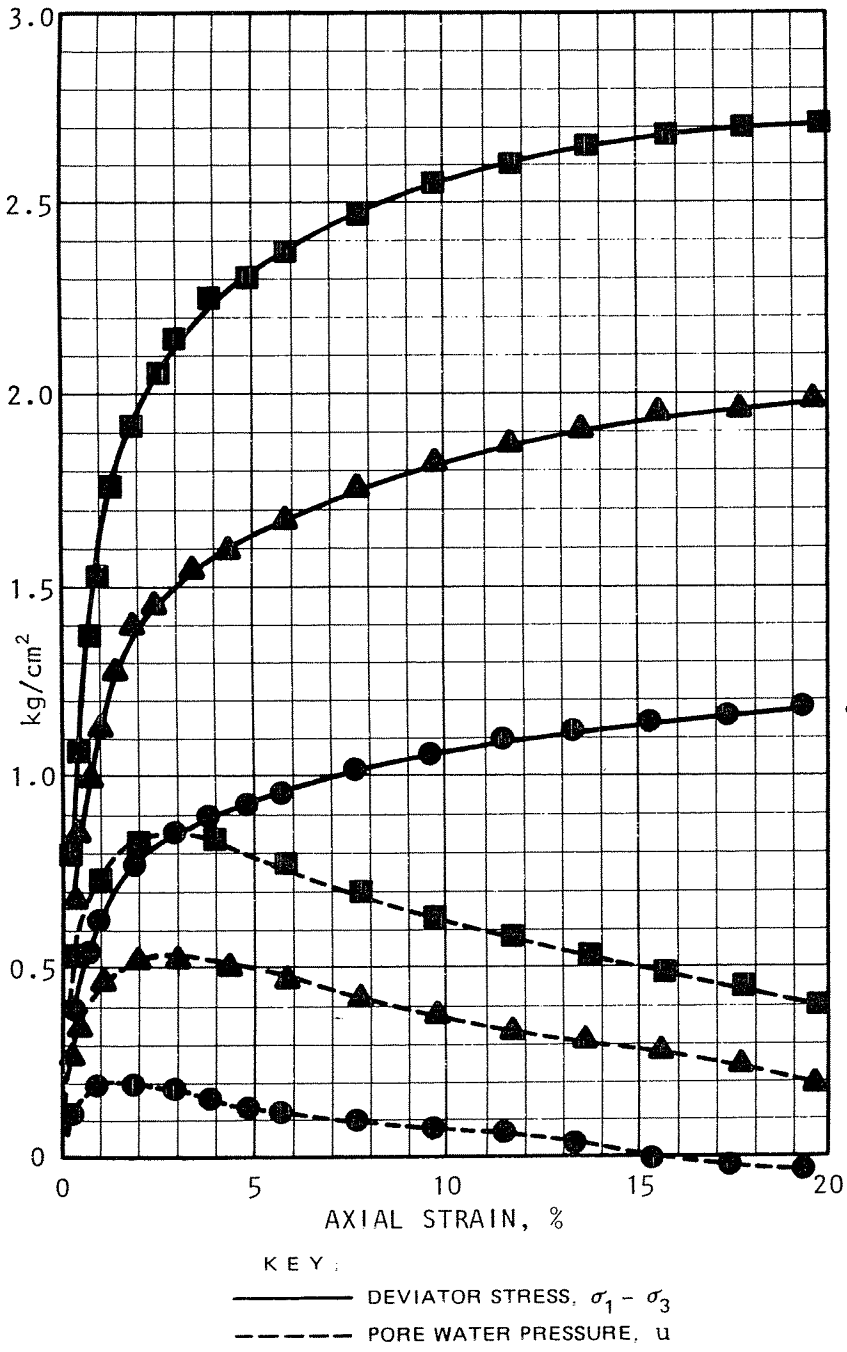
Dry Density:

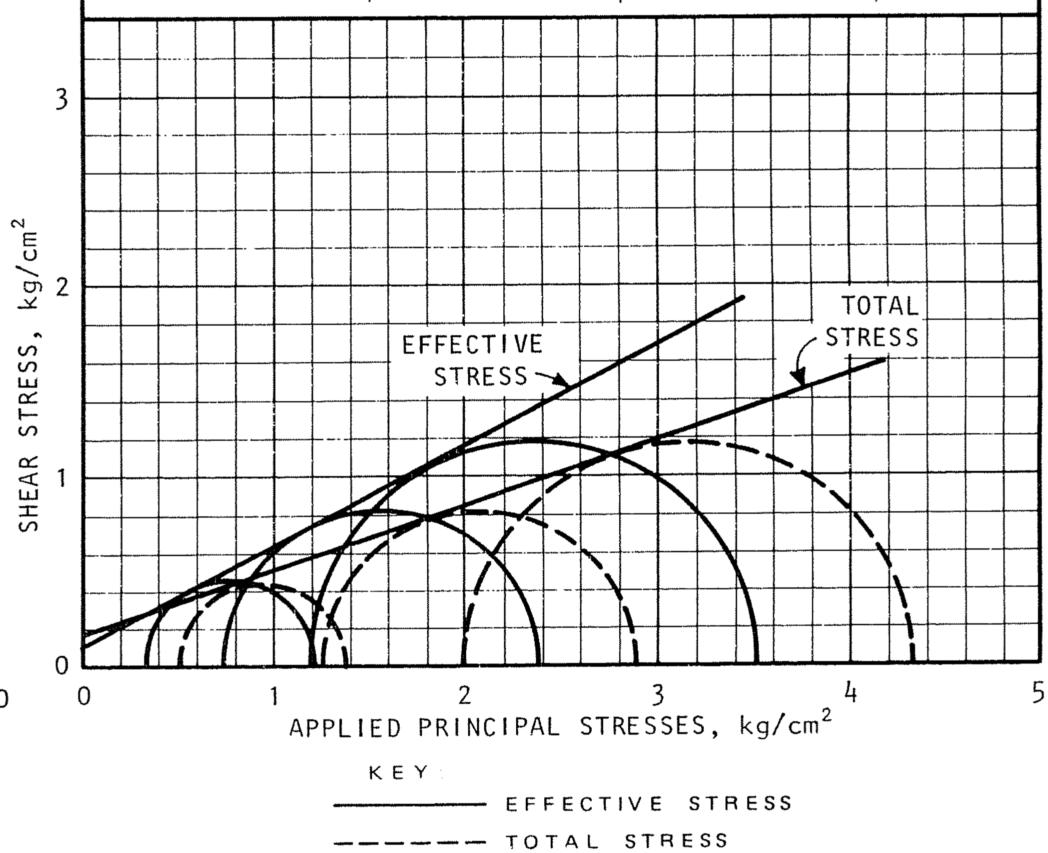
Calculations:

SPECIMEN: BORROW AREA GA - Sample No. 1
TYPE OF SPECIMEN: REMOLDED
TYPE OF TEST: CONSOLIDATED - UNDRAINED

SPECIMEN AND SYMBOL	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)	CONFINING PRESSURE (tsf)	DEVIATOR STRESS AT FAILURE (tsf)	PORE PRESSURE AT FAILURE (tsf)
(●)	27.5	93.5	0.50	0.87	0.16
(▲)	27.5	93.5	1.25	1.63	0.48
(■)	27.5	93.5	2.00	2.31	0.80

$c' = 0.10$ tsf $\phi' = 27.1°$ $c_T = 0.19$ tsf $\phi_T = 21.2°$





Project No. 60255N
Woodward-Clyde Consultants

TRIAXIAL COMPRESSION TEST
PRL - SHOOTERING CANYON URANIUM PROJECT

Figure 1

EXHIBIT C - SEISMIC HAZARD ANALYSIS

Referenced section from June 26, 1994 Seismic Hazard Analysis of Title II Reclamation Plans", by Lawrence Livermore National Laboratory larger M~5.8 event, we use the median estimate to account for its much lower probability of occurring. This leads to an estimate for PGA of 0.19g.

Fault 2

Fault 2 trends northwest hence it is favorably oriented with the stress field. The fault is approximately 10 km long. If the entire fault ruptured in a single event this could lead to a M~6.25 earthquake. If we assume only one-half of the fault ruptures, this leads to a M~5.9 earthquake. The fault is approximately 13 km from the site. The 1-sigma estimate for PGA at the site from a M~5.9 earthquake located on what we have labeled fault 2 is 0.28g. Because of its lower probability of occurrence, we use the median estimate for M_u~6.25 which is 0.19g. The median estimate for a M~5.9 event is 0.16g.

Fault 3

This fault is almost due east of the site. The fault is listed as a possible Quaternary fault by Hecker (1993) and could have some seismicity associated with it. The fault trends northeast and hence not in the most likely direction for earthquakes. Thus it is not a likely candidate for earthquakes. However, it is included in the analysis for completeness. The fault has a length of approximately 23 km and lies approximately 35 km from the site. If we assume the entire fault ruptured, this would give rise to a 6.7 earthquake. This is larger than might be expected, at least based on the historical record. However, as we pointed out in the methodology section, it is not clear that the historical record gives a good indication of the largest event that could occur because we expect that the largest possible event would be a characteristic earthquake governed by its own characteristic return interval. If we use a distance of 35 km and M = 6.7 in the Joyner - Boore model, we get 1-sigma estimate of 0.14g.

Random Earthquake Analysis

Based on the geology and pattern of seismicity around the Plateau Resources site, we selected a source zone which seemed reasonable to use to develop our recurrence model. As described in the methodology section we applied Stepp's method to try and determine the completeness of the earthquake catalog. There is no data in the catalog before 1963 for the selected zone. Stepp's method indicated that the catalog was reasonably complete for events of about magnitude 3 for the last 10 to fifteen years. The smaller events did not appear to be complete. Fig. 7.17 shows the data for the last 30 years. Also shown is the truncated exponential model that we use with M_u = 5.75. The model appears to fit the data reasonably well. The simple Richter form of the model normalized to a per year basis is

$$\log N = 2.43 - 0.92M$$

We used this recurrence model to develop the seismic hazard for the region around the Plateau Resources site as outlined in our methodology section. Fig. 7.18 gives the hazard curves for values of M_u = 5.5, 5.75, 6.25, and 7. We see from the hazard curves that at a PE level of 10^{-4} the PGA varies between 0.17g to 0.24g. As there are no major faults in the vicinity of the site our preferred choice for M_u is 5.75. This leads to 0.19g estimate for the ground motion at the site from the random earthquake at a PE level of 10^{-4}. At a PE level of $5x10^{-4}$ the PGA varies between 0.08g to 0.12g depending upon the choice of M_u with a value of 0.09g at M_u = 5.75.

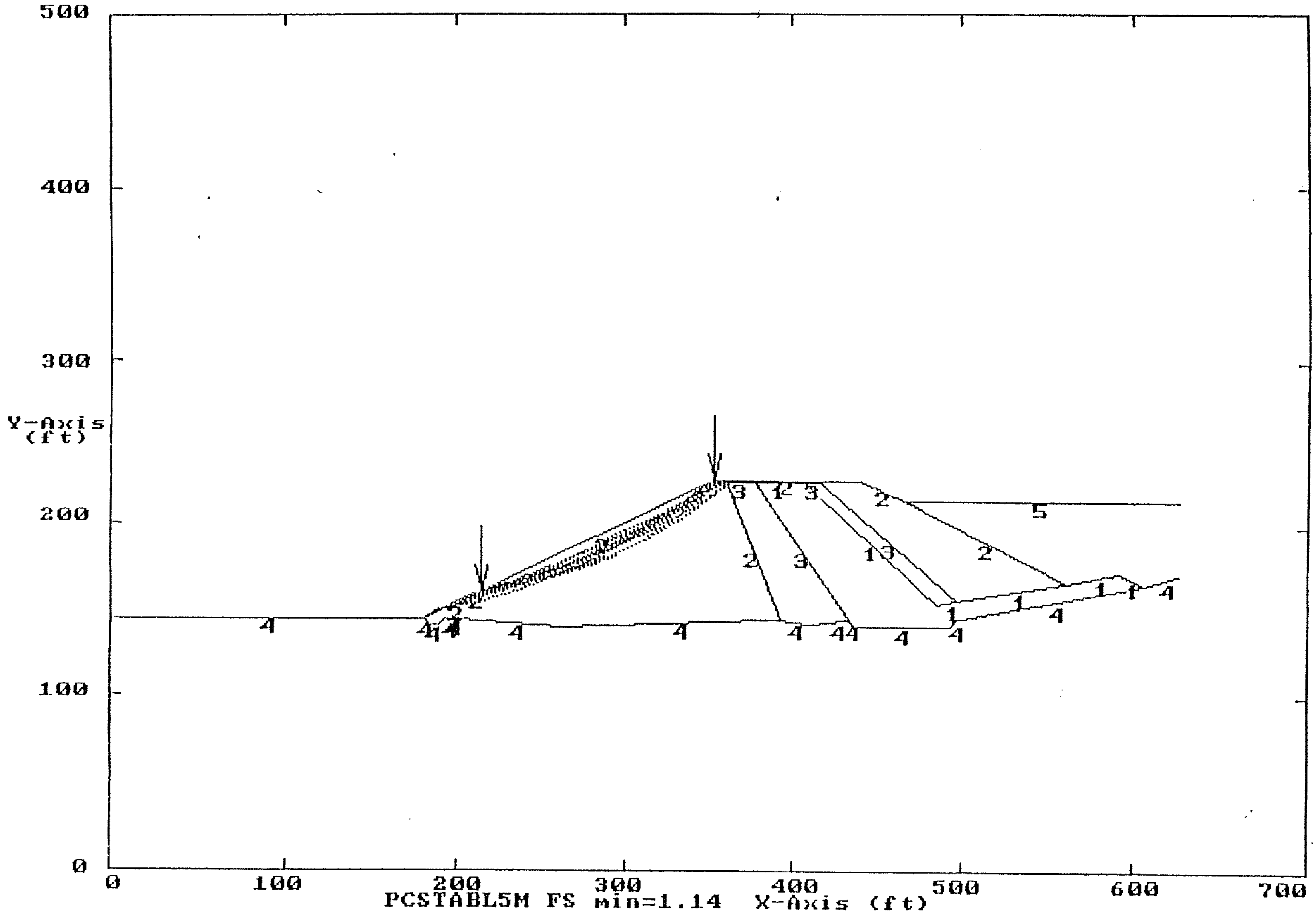
7.8.5 Conclusions

There appear to be no faults through the site that could cause problems. Our deterministic analysis lead to an estimate for PGA of 0.16g to 0.3g. The random earthquake analysis gives a lower estimate of 0.17 g to 0.24 g. There is a possibility of a larger earthquake in the vicinity of the site, which is included in the analysis for random earthquakes, however the likelihood is sufficiently low that in our opinion the M~5.5 earthquake meets our criteria.

EXHIBIT D - STABILITY ANALYSIS RESULTS

Input and Plot Files for the PCSTABL Critical Failure SurfacE

SHOOTERING CANYON DAM SEISMIC STABILITY
Ten Most Critical. C:JH19.PLT By: GMB 01-09-97 7:42am



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PROFIL         C:JU19.IN  PCSTABL Version 5M
SHOOTERING CANYON DAM SEISMIC STABILITY
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211. 160. 214. 160. 2
214. 160. 353. 228. 2
353. 228. 440. 228. 2
440. 228. 466. 216. 2
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SOIL
5
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```

```
100. 100. 0. 10. 0. 0. 0
EQUAKE
0.19 0. 0.
CIRCLE-Janbu circular, search.
0
20 20
175. 225. 350. 400. 10. 25. 0. 0.
```

A.2 Seismic Stability Analysis, Letter Report
by Inberg-Miller Engineers, December 11, 1997

RECEIVED DEC 1 2 1997

INBERG-MILLER ENGINEERS

124 EAST MAIN STREET RIVERTON, WYOMING 82501-4397 307-856-8136

December 11, 1997 7664-RX

U.S. Energy
877 North 8th West
Riverton, Wyoming 82501

ATTENTION: DAN ARIMA

SUPPLEMENTAL SEISMIC STABILITY ANALYSIS
STAGE I - SHOOTARING CANYON DAM
GARFIELD COUNTY, UTAH

Gentlemen:

This letter supplements our January 9, 1997 letter with regard to seismic stability analysis for Stage I of the Shootaring Canyon Dam. Soil strength parameters for Zone 1 (dam core) and impounded tailings were revised based on additional information provided by U.S. Energy.

Zone 1 soil used in the analysis presented in our January 9, 1997 letter was based on Table C-1 in the May 1979 Stage I - Tailings Impoundment and Dam Final Design Report. However, the strength parameters were subject to confirmation as noted on Table C-1. U.S. Energy provided test data which was contained in a June 12, 1979 letter from their consultant, Woodward-Clyde Consultants, which presented confirmation. The test data presents soil strength parameters based on consolidated-undrained shear testing for two Zone 1 soil borrow areas (H and I). A copy of the referenced test data is contained in Attachment A to this letter. Total strength parameters were used for seismic analysis. Total strength parameters for Zone 1 changed from the Table C-1 values as indicated below:

		Soil Strength Parameter Confirmation (Woodward-Clyde, June 12, 1979 Letter)	
	Table C-1	Borrow Area H	Borrow Area I
Cohesive Strength	1500 psf	400 psf	400 psf
Friction Angle	0	16.7°	18.4°

We used Borrow Area H strength parameters for this supplemental analysis since they are more conservative. We also revised the strength of impounded tailings for our analysis. Impounded tailings were modeled without any shear strength to consider conservative limit conditions. All other parameters, including the horizontal seismic coefficient of 0.19g, remained unchanged.

The results of seismic stability analysis were unchanged from our previous report. The critical faliure surface indicates a shallow slide-plane which does not intersect either Zone 1 soil (dam core) or impounded tailings. A copy of PCSTABL input, output, and plot files are contained in Attachment B.

We are pleased to be of continued service to you on this project. Please feel free to call if you have any questions.

Sincerely,

INBERG-MILLER ENGINEERS

Glen M. Bobnick

Glen M. Bobnick, P.E.
Geotechnical Engineer

GMB:jlw:gd\7664rx

Attachment A: June 12, 1979 Woodward-Clyde Consultants Shear Strength Test Data
Attachment B: PCSTABL Input, Ouput, and Plot Files

ATTACHMENT A

June 12, 1979 Woodward-Clyde Consultants

Shear Strength Test Data

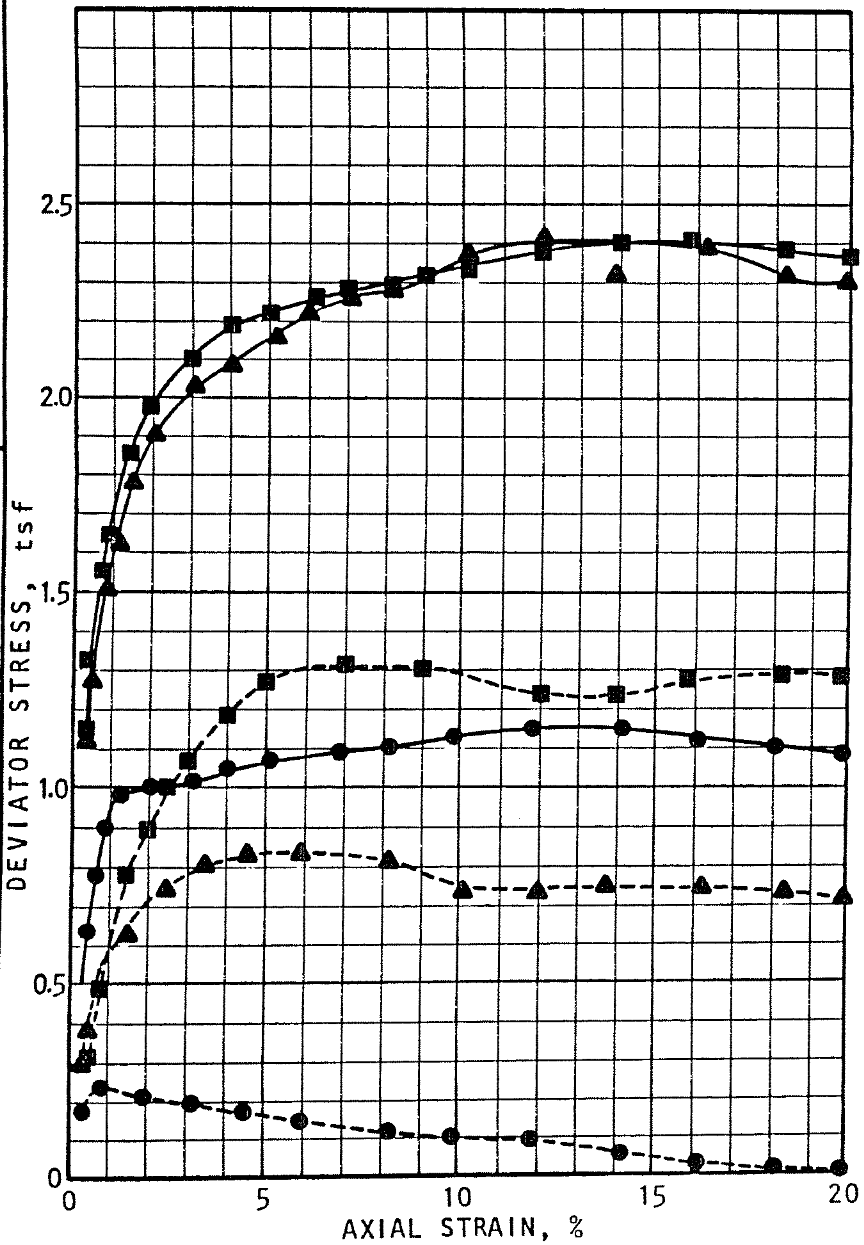
Project No. 60255J

Woodward-Clyde Consultants

TRIAXIAL COMPRESSION TEST

PRL - SHOOTERING CANYON URANIUM PROJECT

Figure 3



KEY:

——— DEVIATOR STRESS, $\sigma_1 - \sigma_3$

– – – – PORE WATER PRESSURE, u

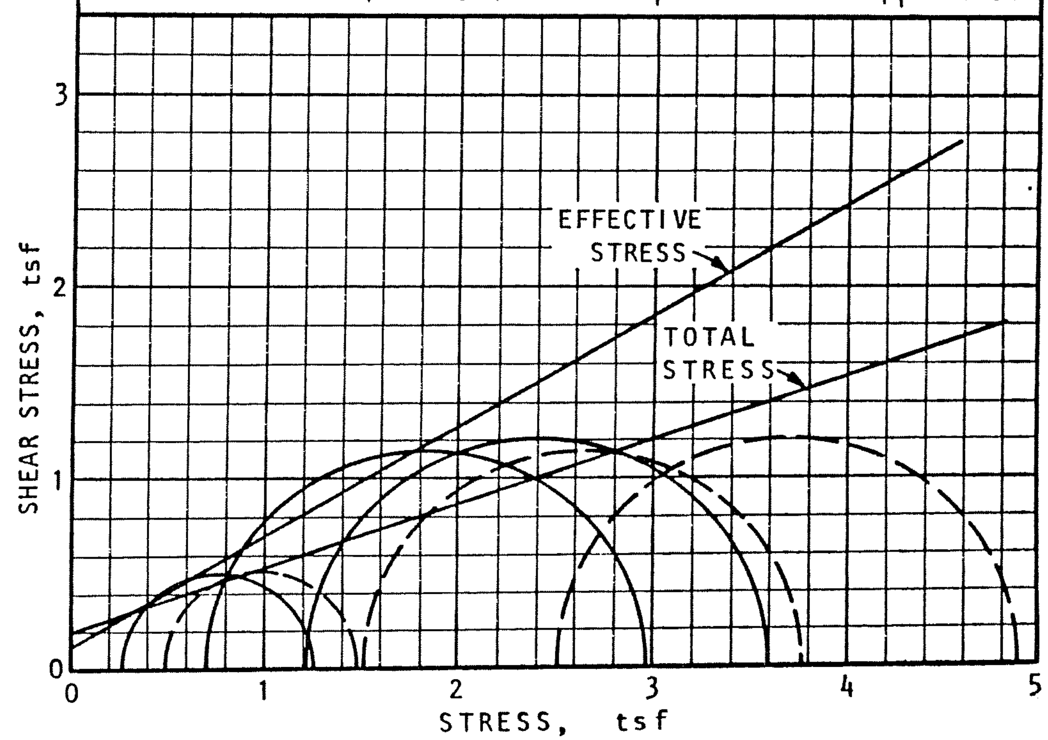
SPECIMEN: Borrow Area H - Sample #1

TYPE OF SPECIMEN: Remolded

TYPE OF TEST: Consolidated-Undrained

SPECIMEN AND SYMBOL	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)	CONFINING PRESSURE (tsf)	DEVIATOR STRESS AT FAILURE (tsf)	PORE PRESSURE AT FAILURE (tsf)
1 (●)	35	83	0.5	0.97	0.23
2 (▲)	35	83	1.5	2.28	0.81
3 (■)	35	83	2.5	2.39	1.28

$c' = 0.10$ tsf $\phi' = 29.7°$ $c_T = 0.2$ tsf $\phi_T = 18.4°$



KEY:

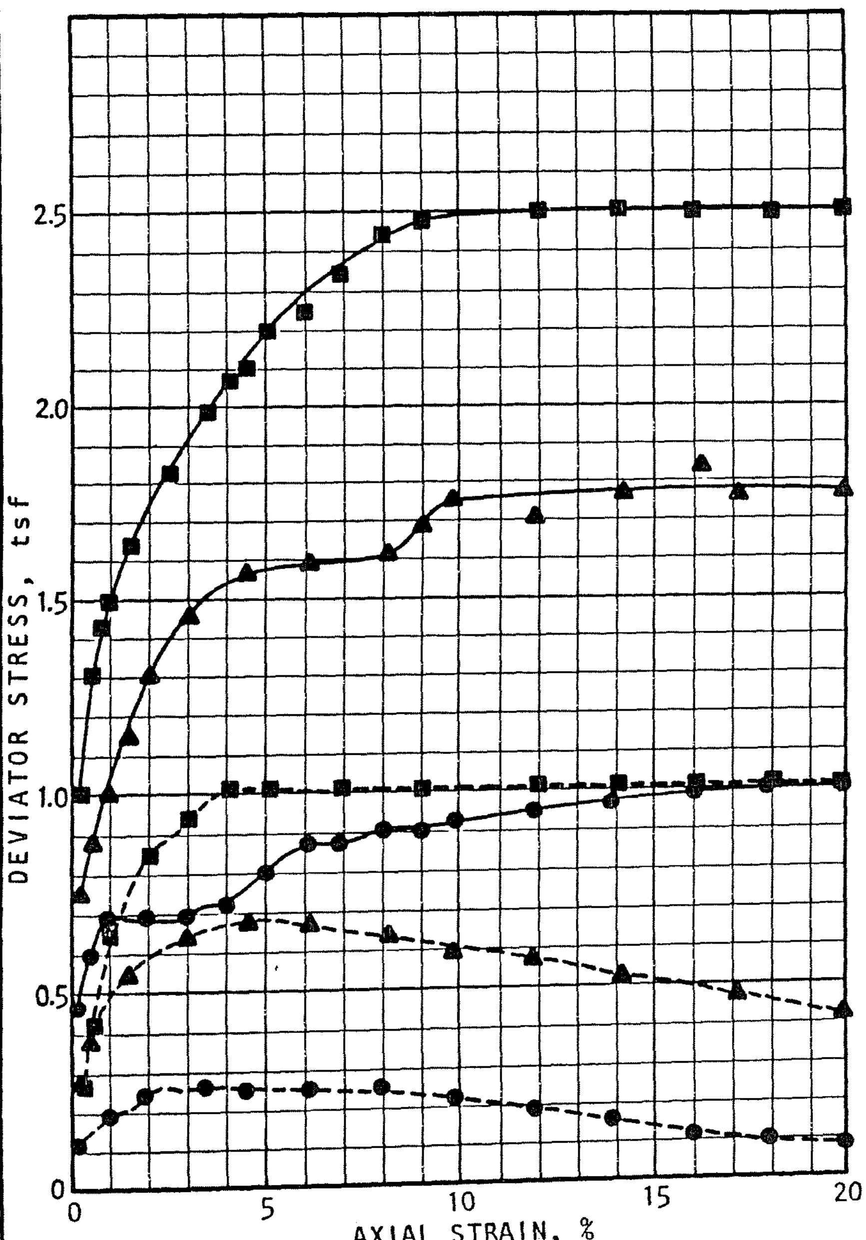
——— EFFECTIVE STRESS

– – – – TOTAL STRESS

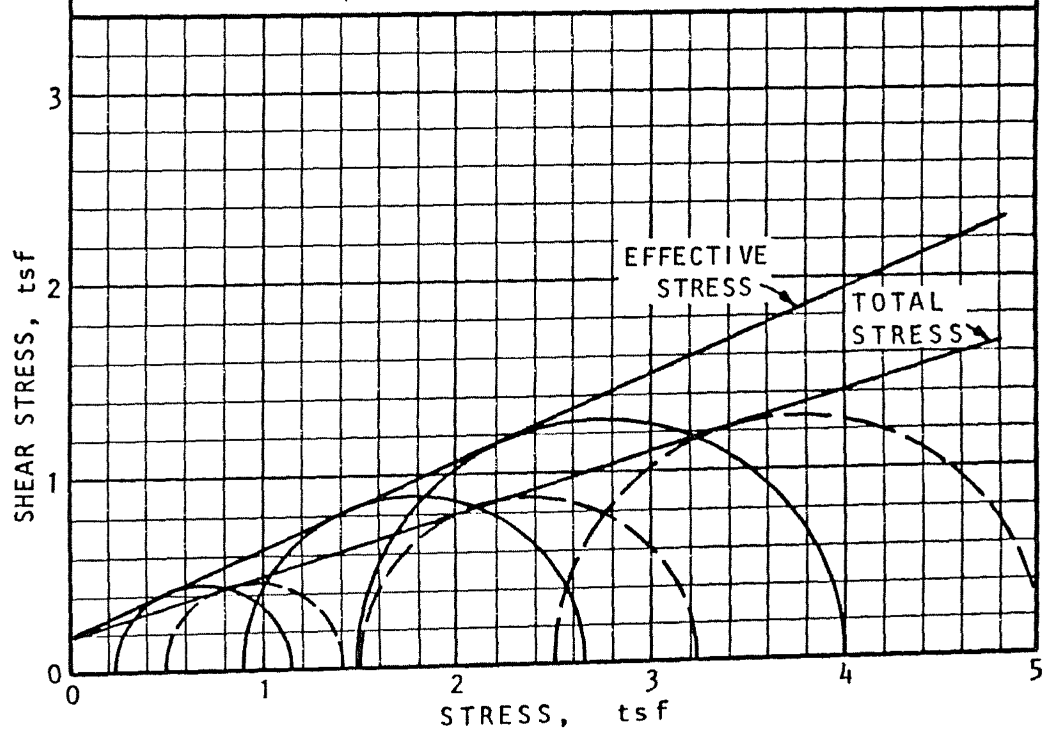
SPECIMEN: Borrow Area I - Sample #1
TYPE OF SPECIMEN: Remolded
TYPE OF TEST: Consolidated-Undrained

SPECIMEN AND SYMBOL	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)	CONFINING PRESSURE (tsf)	DEVIATOR STRESS AT FAILURE (tsf)	PORE PRESSURE AT FAILURE (tsf)
1 (●)	22	98	0.25	0.91	0.25
2 (▲)	22	98	0.91	1.74	0.59
3 (■)	22	98	1.48	2.52	1.02

$c' = 0.2$ tsf $\phi' = 23.6°$ $c_T = 0.2$ tsf $\phi_T = 16.7°$



KEY:
—— DEVIATOR STRESS, $\sigma_1 - \sigma_3$
– – – PORE WATER PRESSURE, u



KEY:
—— EFFECTIVE STRESS
– – – TOTAL STRESS

Project No. 60255J **Woodward-Clyde Consultants**	TRIAXIAL COMPRESSION TEST PRL - SHOOTERING CANYON URANIUM PROJECT	Figure 4

ATTACHMENT B

PCSTABL Input, Ouput, and Plot Files

```
PROFIL        C:JU19REV.IN  PCSTABL Version 5M
SHOOTERING CANYON DAM SEISMIC STABILITY
33 7
0. 145. 181. 145. 4
181. 145. 211. 160. 2
211. 160. 214. 160. 2
214. 160. 353. 228. 2
353. 228. 440. 228. 2
440. 228. 466. 216. 2
466. 216. 626. 216. 5
466. 216. 562. 167. 2
562. 167. 593. 173. 1
593. 173. 607. 165. 1
607. 165. 626. 171. 4
360. 226. 377. 226. 3
377. 226. 406. 226. 1
406. 226. 416. 226. 3
416. 226. 498. 156. 3
498. 156. 562. 167. 1
406. 226. 485. 154. 1
485. 154. 498. 156. 1
377. 226. 433. 144. 3
360. 226. 392. 145. 2
181. 145. 184. 141. 4
184. 141. 190. 141. 4
190. 141. 195. 144. 4
195. 144. 204. 144. 4
204. 144. 268. 140. 4
268. 140. 392. 145. 4
392. 145. 412. 142. 4
412. 142. 433. 144. 4
433. 144. 437. 141. 4
437. 141. 492. 141. 4
492. 141. 495. 145. 4
495. 145. 607. 165. 4
607. 165. 626. 171. 4
SOIL
5
125. 125. 400. 16.7 0. 0. 0
131. 131. 0. 40. 0. 0. 0
125. 125. 0. 32. 0. 0. 0
140. 140. 1000. 45. 0. 0. 0
```

```
100. 100. 0. 0. 0. 0. 0
EQUAKE
0.19 0. 0.
CIRCLE-Janbu circular, search.
0
20 20
175. 225. 350. 400. 10. 25. 0. 0.
```

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date:	12-04-97
Time of Run:	3:13pm
Run By:	GMB
Input Data Filename:	C:JU19REV.IN
Output Filename:	C:JU19REV.OUT
Plotted Output Filename:	C:JU19REV.PLT

PROBLEM DESCRIPTION   SHOOTERING CANYON DAM SEISMIC STABILITY

BOUNDARY COORDINATES

7 Top Boundaries
33 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	145.00	181.00	145.00	4
2	181.00	145.00	211.00	160.00	2
3	211.00	160.00	214.00	160.00	2
4	214.00	160.00	353.00	228.00	2
5	353.00	228.00	440.00	228.00	2
6	440.00	228.00	466.00	216.00	2

7	466.00	216.00	626.00	216.00	5
8	466.00	216.00	562.00	167.00	2
9	562.00	167.00	593.00	173.00	1
10	593.00	173.00	607.00	165.00	1
11	607.00	165.00	626.00	171.00	4
12	360.00	226.00	377.00	226.00	3
13	377.00	226.00	406.00	226.00	1
14	406.00	226.00	416.00	226.00	3
15	416.00	226.00	498.00	156.00	3
16	498.00	156.00	562.00	167.00	1
17	406.00	226.00	485.00	154.00	1
18	485.00	154.00	498.00	156.00	1
19	377.00	226.00	433.00	144.00	3
20	360.00	226.00	392.00	145.00	2
21	181.00	145.00	184.00	141.00	4
22	184.00	141.00	190.00	141.00	4
23	190.00	141.00	195.00	144.00	4
24	195.00	144.00	204.00	144.00	4
25	204.00	144.00	268.00	140.00	4
26	268.00	140.00	392.00	145.00	4
27	392.00	145.00	412.00	142.00	4
28	412.00	142.00	433.00	144.00	4
29	433.00	144.00	437.00	141.00	4
30	437.00	141.00	492.00	141.00	4
31	492.00	141.00	495.00	145.00	4
32	495.00	145.00	607.00	165.00	4
33	607.00	165.00	626.00	171.00	4

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	400.0	16.7	.00	.0	0
2	131.0	131.0	.0	40.0	.00	.0	0

3	125.0	125.0	.0	32.0	.00	.0	0
4	140.0	140.0	1000.0	45.0	.00	.0	0
5	100.0	100.0	.0	.0	.00	.0	0

A Horizontal Earthquake Loading Coefficient
Of .190 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

20 Surfaces Initiate From Each Of 20 Points Equally Spaced Along The Ground Surface Between X = 175.00 ft.
and X = 225.00 ft.

Each Surface Terminates Between X = 350.00 ft.
and X = 400.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 10.00 ft.

25.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial

Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	214.47	160.23
2	238.61	166.73
3	262.22	174.98
4	285.15	184.92
5	307.30	196.51
6	328.55	209.69
7	348.78	224.38
8	353.07	228.00

*** 1.139 ***

Individual data on the 8 slices

ce .	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
	24.1	8394.5	.0	.0	.0	.0	1595.0	.0	.0
	23.6	21519.1	.0	.0	.0	.0	4088.6	.0	.0
	22.9	27793.8	.0	.0	.0	.0	5280.8	.0	.0
	22.2	27600.5	.0	.0	.0	.0	5244.1	.0	.0
	21.2	21553.8	.0	.0	.0	.0	4095.2	.0	.0
	20.2	10494.4	.0	.0	.0	.0	1992.0	.0	.0

4.2	448.1	.0	.0	.0	.0	85.1	.0	.0
.1	.3	.0	.0	.0	.0	.1	.0	.0

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	198.68	153.84
2	222.31	162.01
3	245.65	170.99
4	268.66	180.75
5	291.32	191.30
6	313.62	202.62
7	335.51	214.69
8	356.98	227.50
9	357.75	228.00

*** 1.141 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	198.68	153.84
2	222.86	160.20
3	246.63	167.94
4	269.92	177.03
5	292.65	187.45
6	314.74	199.15
7	336.12	212.10
8	356.73	226.27
9	358.96	228.00

*** 1.152 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	188.16	148.58
2	212.58	153.92
3	236.56	161.00
4	259.96	169.78
5	282.68	180.22
6	304.59	192.25
7	325.58	205.84
8	345.54	220.90
9	353.67	228.00

*** 1.156 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	198.68	153.84
2	223.06	159.40
3	247.00	166.61
4	270.39	175.44
5	293.12	185.85
6	315.09	197.78
7	336.19	211.13

8	356.33	226.00
9	358.69	228.00

*** 1.157 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.53	147.26
2	208.88	156.20
3	232.05	165.57
4	255.05	175.38
5	277.86	185.61
6	300.47	196.28
7	322.88	207.36
8	345.07	218.87
9	361.91	228.00

*** 1.159 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	182.90	145.95
2	206.44	154.34
3	229.78	163.30
4	252.90	172.81

5	275.79	182.87
6	298.43	193.48
7	320.81	204.63
8	342.91	216.30
9	363.82	228.00

*** 1.164 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	217.11	161.52
2	241.85	165.06
3	266.05	171.33
4	289.41	180.25
5	311.62	191.72
6	332.43	205.58
7	351.56	221.68
8	357.56	228.00

*** 1.172 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	214.47	163.23
2	238.71	166.35

3	262.54	173.93
4	285.86	182.93
5	308.60	193.32
6	330.67	205.07
7	351.98	218.13
8	366.09	228.00

*** 1.176 ***

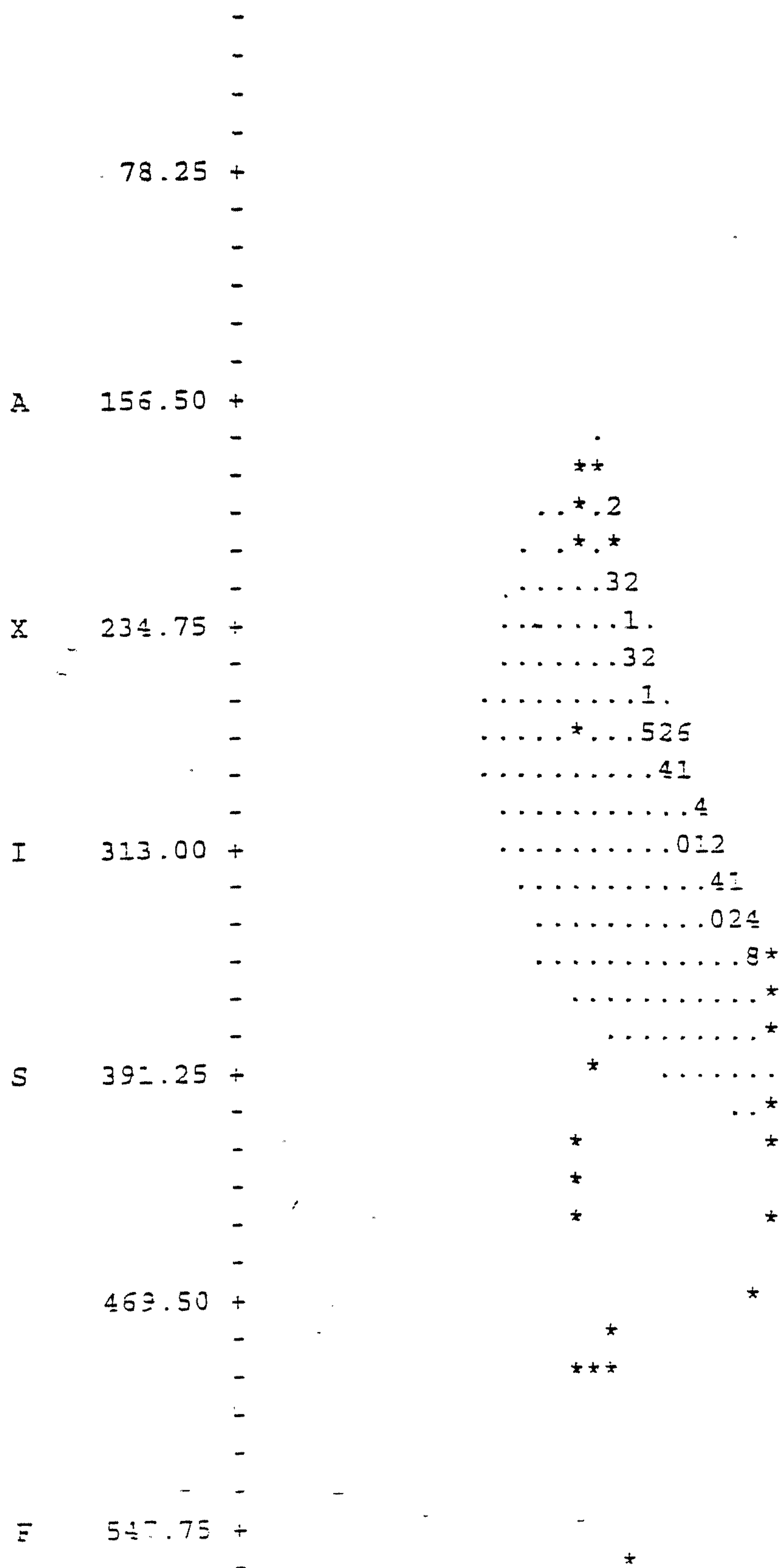
Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	217.11	161.52
2	241.77	165.61
3	265.94	172.01
4	289.40	180.64
5	311.95	191.44
6	333.38	204.30
7	353.51	219.13
8	363.46	228.00

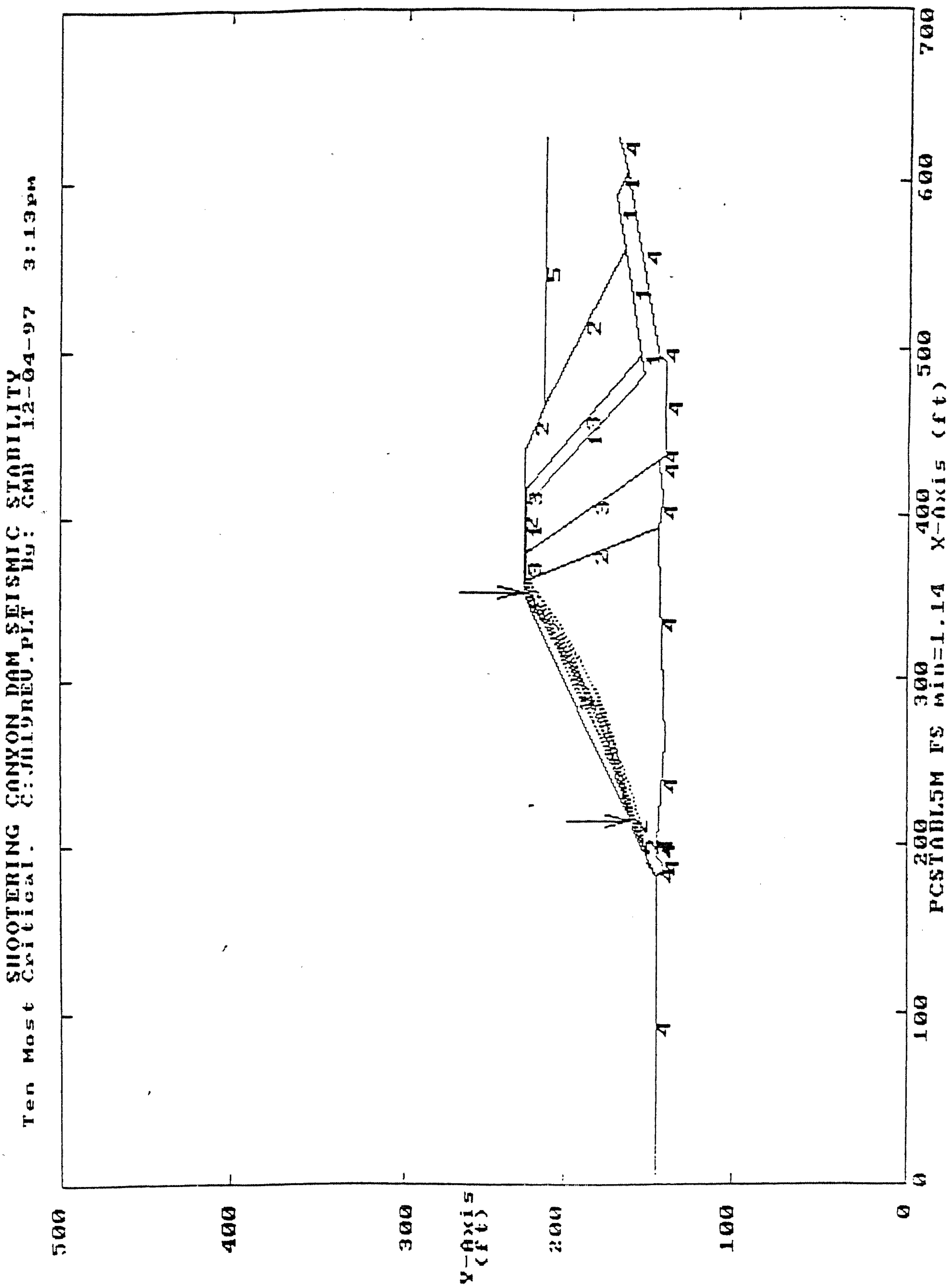
*** 1.182 ***

```
        Y            A       X       I       S          F       T

           .00     78.25    156.50   234.75   313.00   391.25
 X      .00 +---------+---------*+---------------------+-----------+
            -
```


78.25
A 156.50
X 234.75
I 313.00
S 391.25
469.50
F 547.75

```
                   -
                   -                    *
                   -
                   -                   *
T       626.00     +                    *     *
```


SHOOTERING CANYON DAM SEISMIC STABILITY
Ten Most Critical.
By: GMB 12-04-97 3:13pm
Y-Axis (ft)
X-Axis (ft)
PCSTABL5M FS min=1.14
0
100
200
300
400
500
600
700

A.3 Slope Stability Analysis Cross Valley Berm, Letter Report
by Inberg-Miller Engineers, June 14, 1999

INBERG-MILLER ENGINEERS
124 EAST MAIN STREET RIVERTON, WYOMING 82501-4397 307-856-8136

May 2, 1997
Revised June 14, 1999

7664-RX

U.S. Energy Corporation
877 N. 8th West
Riverton, Wyoming 82501

ATTENTION: FRED CRAFT

RE: SLOPE STABILITY ANALYSIS
CROSS VALLEY BERM
SHOOTARING CANYON URANIUM PROJECT, UTAH

Dear Sir:

This letter summarizes the results of a slope stability analysis that we performed for the Cross Valley Berm at the proposed Shootaring Canyon Uranium Project located in southeast Utah. A summary of project background, basis for analysis, slope stability analysis results, and recommendations for berm earthwork are presented herein.

BACKGROUND

Inberg-Miller Engineers performed a slope stability analysis for the existing sedimentation dam at the Shootaring Canyon Uranium Project, the results of which are contained in our January 9, 1997 letter report. You subsequently requested we also perform a slope stability analysis for an existing cross valley berm located upgradient of the sedimentation dam. We understand the cross valley berm is a temporary tailings impoundment which will ultimately be covered by tailings as the pool elevation rises behind the sedimentation dam.

We understand the cross valley berm was installed in about 1981. You provided an April 17, 1997 topographic map of the berm, a subsequent updated topographic berm map dated March 18, 1999 typical berm cross section, and a copy of compaction test dated for soil which comprises the berm. Based on information you provided, we understand the existing berm has the following geometry at maximum berm height:

Crest Elevation:	4448
Upstream Toe Elevation:	4430
Downstream Toe Elevation:	4408
Upstream Slope:	3 horizontal: 1 vertical
Downstream Slope:	Varies from 1.1: 1 at the toe to 3: 1 at the crest
Crest Width:	14 feet
Base Width:	145 feet

You indicated that the berm would be reworked to adjust the upstream and downstream slope to a minimum of 1 horizontal to 1 vertical. You also indicated that the crest elevation would be raised 10 feet. The downstream berm toe would remain at its current location to maintain an existing drainage system. The crest and upstream berm toe would be relocated upstream of the

BACKGROUND, Continued

current position to accommodate the increase in crest elevation. Reworking the slopes would involve removal of soil from the downstream slope and placement of fill on the upstream slope. An illustration of proposed berm modifications is presented in Attachment A.

BASIS FOR ANALYSIS

The basis for slope stability analysis includes analytical method, soil parameters, groundwater conditions, and seismic conditions. A discussion of each of these items follows.

Analytical Method

Slope stability analysis was performed using the computer program PCSTABL. The slope was analyzed using the Bishop and Janbu methods. The slope was modeled based on the proposed geometry (e.g. 10-foot higher crest elevation and 2:1 outslopes). We assumed the tailings pool is at the proposed crest elevation. Separate analyses were performed for static and seismic conditions. Refer to our January 9, 1997 report for further discussion of the basis for slope stability analysis.

Soil Parameters

According to information you provided, the berm is substantially comprised of compacted sand, similar to the material described as Zone 3 – Fine Sand modeled in the sedimentation dam analysis. We understand the sand was compacted to a minimum of 95% of the maximum density determined in accordance with ASTM D698. The berm is founded on compacted clay subgrade which is the same soil described as Zone 1 – Silty Sandy Clayey Soil used in the sedimentation dam analysis. Compacted clay is immediately underlain by native "rock foundation" which is the same material modeled in the sedimentation dam analysis. A topographic map and berm cross-section is presented in Attachment A. Based on the topographic maps you provided, the berm slopes are actually steeper than indicated on the cross-section. Actual slopes for the maximum section are described under the background section above. Soil numbers assigned to each of the units identified above are as follows:

Soil 1:	Sand
Soil 2:	Clay
Soil 3:	Rock Foundation
Soil 4:	Tailings

The soil properties used for sedimentation dam stability analysis as documented in our January 9, 1997 letter report also apply to the cross valley berm except for Soil 1. You provided Inberg-Miller Engineers with a sample of the sand representing Soil 1 and requested we perform a direct shear test to determine strength parameters. The sample was tested based on remolding

BASIS FOR ANALYSIS, Continued

Soil Parameters, Continued

specimens to 95% of the maximum dry density of 110.2 pounds per cubic foot as reported on the compaction test date you provided. Direct shear test results of Soil 1 are presented in Attachment B.

Engineering properties we used for this slope stability analysis are tabulated below.

Soil No.	Description	Moist Density (pcf)	Effective Friction Angle	Effective Cohesive Strength (psf)
1	Sand	115.0	38	0
2	Clay	125.0	0	1,500
3	Rock Foundation	140.0	45	1,000
4	Tailings	100.0	10	0

Groundwater Conditions

We understand that the impoundment will be lined with an HDPE liner, therefore, we have assumed for the analysis that no phreatic surface will develop through the berm.

Seismic Conditions

The basis for seismic conditions is the same as described in our January 9, 1997 letter report. In general, a horizontal seismic coefficient of 0.19g was used for this analysis.

RESULTS

Slope stability analysis that were performed using the Janbu method typically produced the lowest safety factors. We calculated a minimum safety factor of 1.02 for seismic conditions and 1.56 under static conditions. A copy of our PCSTABL input files and plots of the critical failure surfaces for static and seismic conditions is presented in Attachment C. Filenames are REMCVBEQ and REMCVB for seismic and static conditions, respectively.

RECOMMENDATIONS

You requested recommendations for fill placement on the upstream slope face which will provide soil conditions that would result in slope safety factors with at least the values estimated above. We anticipate a portion of the fill on the upstream slope will originate from cut on the

RECOMMENDATIONS, Continued

downstream slope and will consist of the Sand modeled as Soil 1. You indicated that other fill soil may be collected from areas located in the proposed impoundment area. Other fill soil could include clay, sand and rock. We recommend the following:

1) Separate fill soils as much as practical based on soil type and moisture condition. Clay, sand, and rock should not be mixed together. Soils with a moisture content over 3% plus or minus of optimum should also be kept separate to permit moisture conditioning before placement as fill.

2) Remove loose soil, debris, and vegetation from the upstream face of the existing berm. The exposed subgrade should be compacted to a minimum of 95% of the maximum density determined in accordance with ASTM D698 (Standard Proctor).

3) Sand excavated from the downstream slope should be classified by a qualified geotechnical engineer to verify that it is consistent with Soil 1 modeled in our analysis. If downstream soils are not consistent with Soil 1, we should be contacted immediately to discuss other options.

4) Provided that sand excavated from the downstream face is consistent with Soil 1 and it has a moisture content within plus or minus 3% of the optimum moisture content, it should be placed before other fill on the upstream slope. If moisture content is outside the recommended range, it should be wetted and mixed, or loosened and air dried, as applicable. Fill should be compacted to a minimum of 95% of the maximum density determined in accordance with ASTM D698 (Standard Proctor).

5) Fill other than Soil 1 needed to achieve a final upstream slope of 2:1 should be approved by a geotechnical engineer. Fill which meets the requirements of Envelop A (See Attachment D) which is compacted as described in Item 4 above may be suitable for use as fill subject to approval of the geotechnical engineer. Fill not meeting Envelope A requirements may have properties not consistent with those used for our slope stability analysis. Fill requirements for non-Envelope A soil should be established on a case-by-case basis by a geotechnical engineer as sources are identified and classified. Coarse soil such as gravel, cobbles and boulders may be subject to placement at the slope toe and protection from infiltration into voids by finer-grained overlying soil by enclosing within

RECOMMENDATIONS, Continued

filter fabric. For fine soil, such as silts and clays, it may not be practical to achieve high enough strength to provide a stable slope at 2:1. If fine soil fill is proposed, we recommend that strength testing be performed to verify whether or not it has sufficient strength to be used on the proposed slope.

Please feel free to call if you have any questions. We are pleased to be of continued service to you on this project.

Sincerely,

INBERG-MILLER ENGINEERS

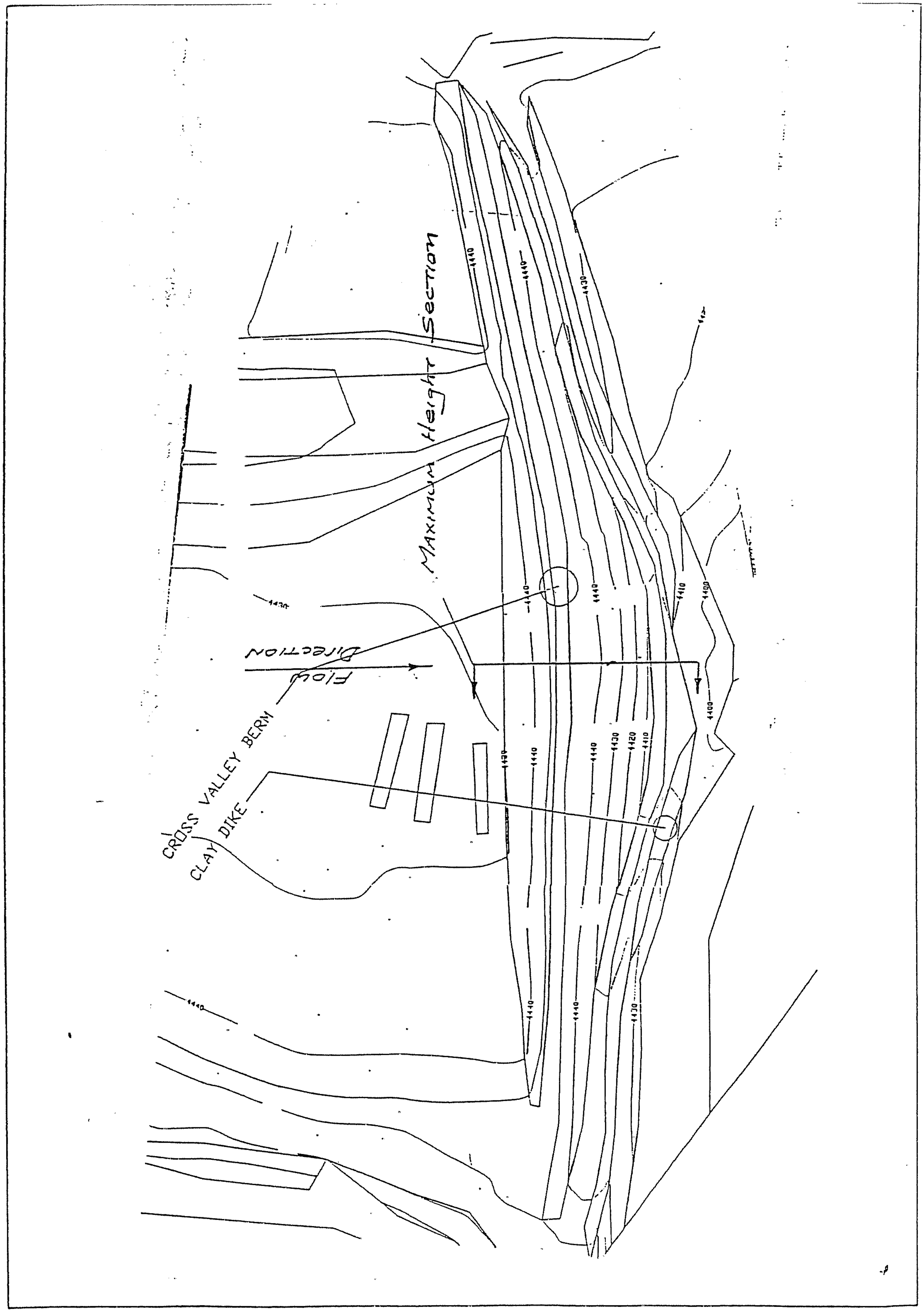
Glen M. Bobnick

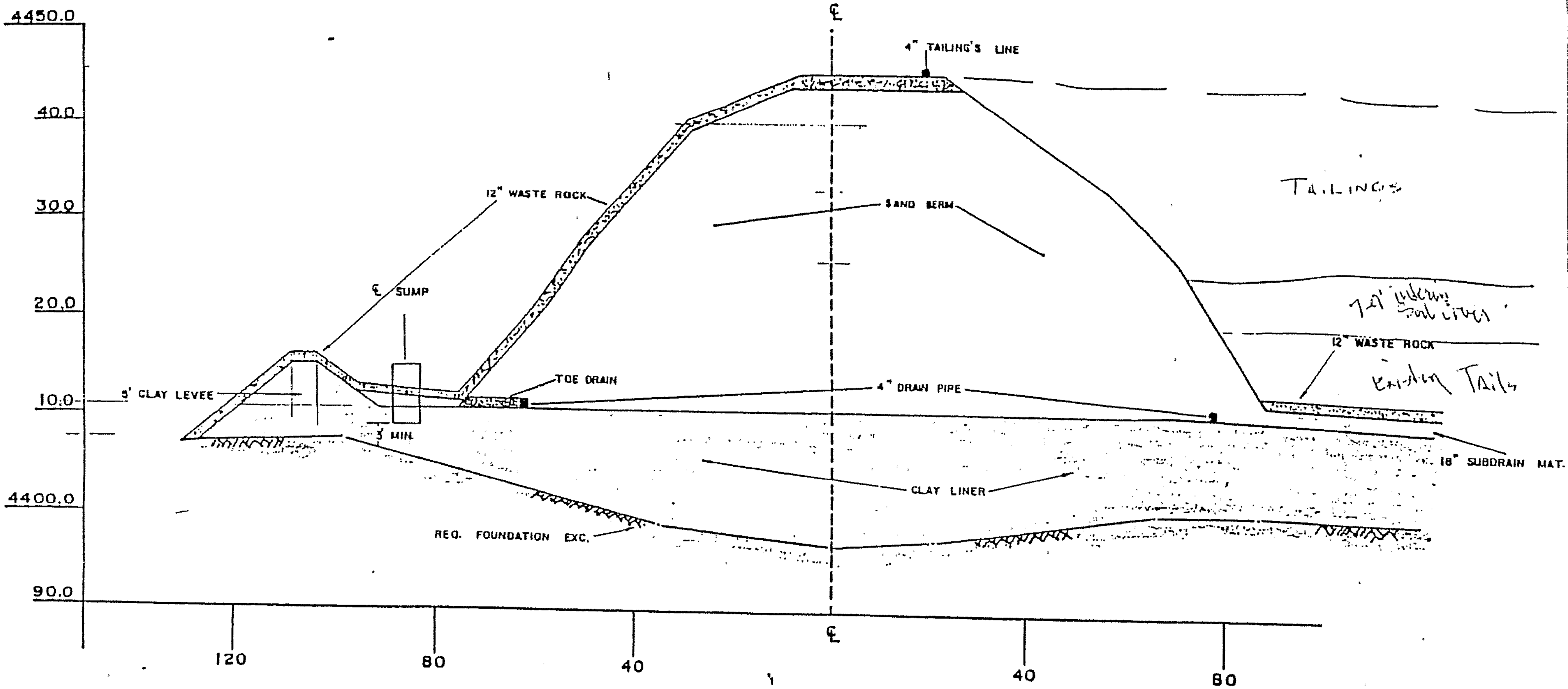
Glen M. Bobnick, P.E.
Geotechnical Engineer

GMB:jlw:client letters\7664-RX-summary letter

Enclosures:	Attachment A -	Typical Berm Plan and Cross-Sections
	Attachment B -	Laboratory Test Results
	Attachment C -	PCSTABL Input and Critical Section Plots

Attachment A
Typical Berm Cross-Section


Maximum Height Section
Flow Direction
CROSS VALLEY BERM
CLAY DIKE
4440
4430
4420
4410
4400

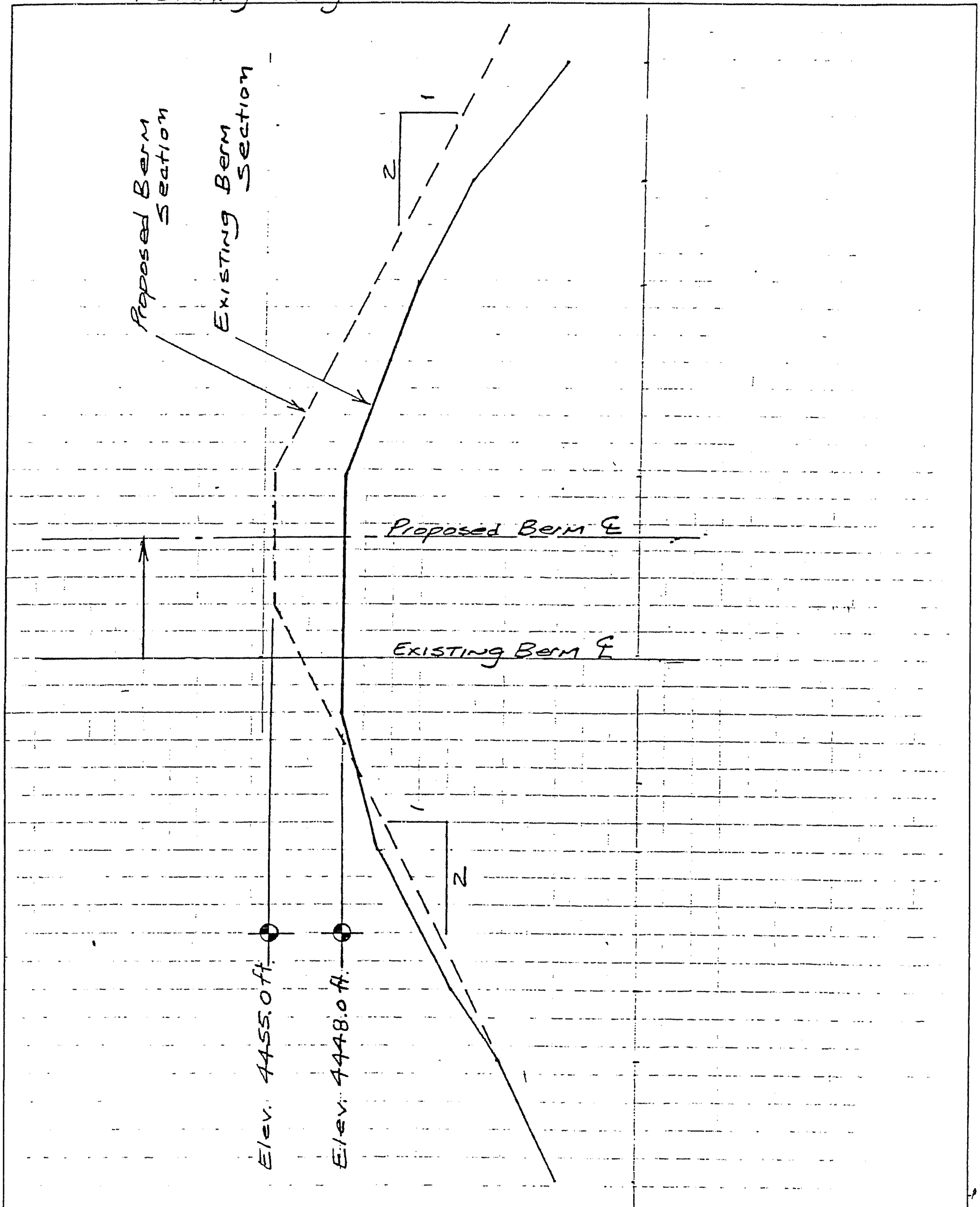


DRAWN		PLATEAU RESC'S LTD.	DRAWING TITLE	mountain states enghrs
CHECKED		URANIUM FACILITY	TYPICAL SECTION	TUCSON ARIZONA
JOB ENGR:		SHOOTERING CANYON	CROSS VALLEY	SCALE 1" = 10 V 1" = 20' H — REV. NO
DEPT HD:		UTAH	BERM	II-6
CONST MGR:				JOB NO. 435 — 3-15-81
PROJ MGR:				
3-17-81				

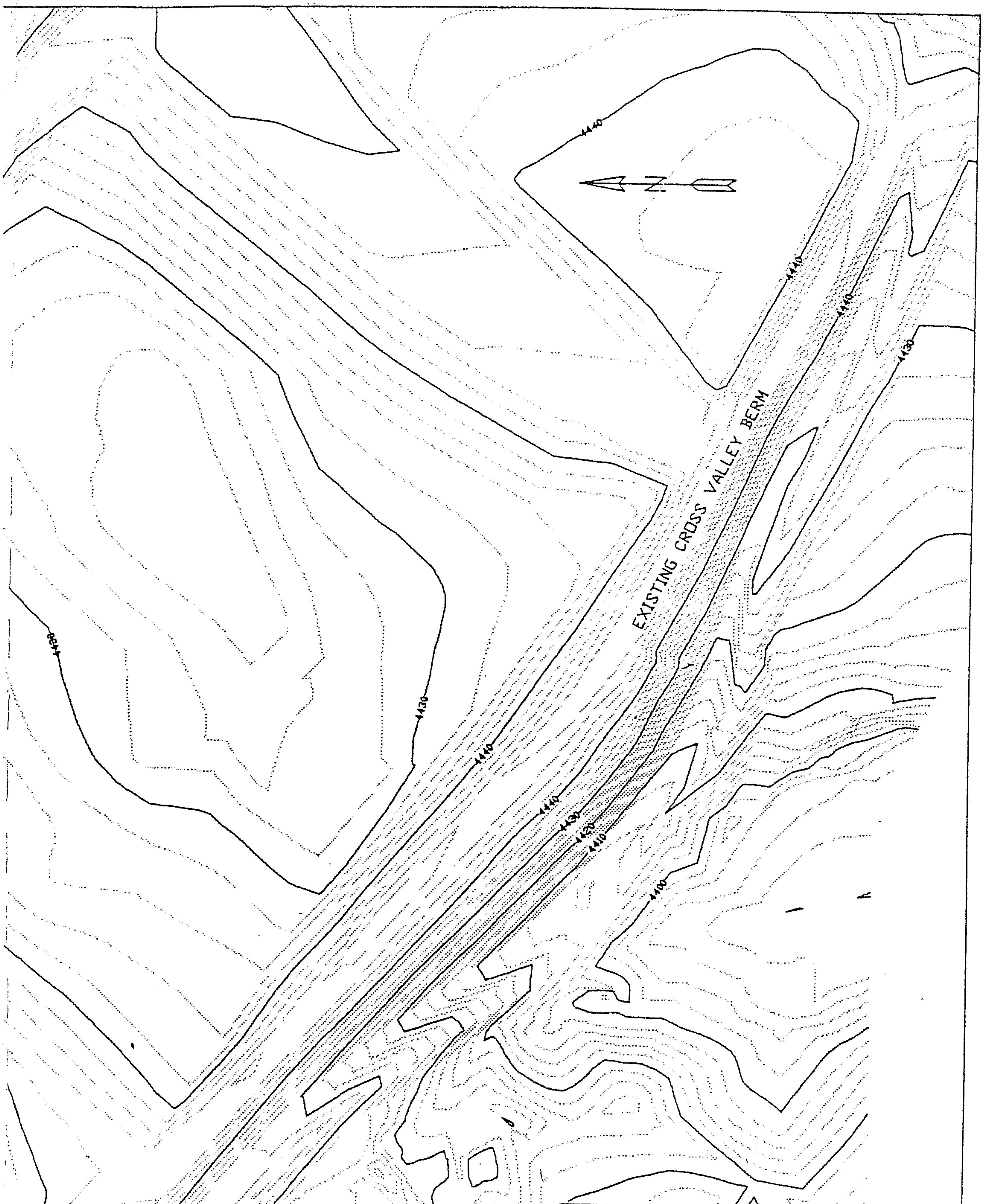
SUBJECT Berm Modifications PROJECT NO. 7664RX PAGE 1/1

CLIENT US Energy DATE 6-11-99 BY GMB

PROJECT Shootaring Canyon CHECKED BY



INBERG-MILLER ENGINEERS



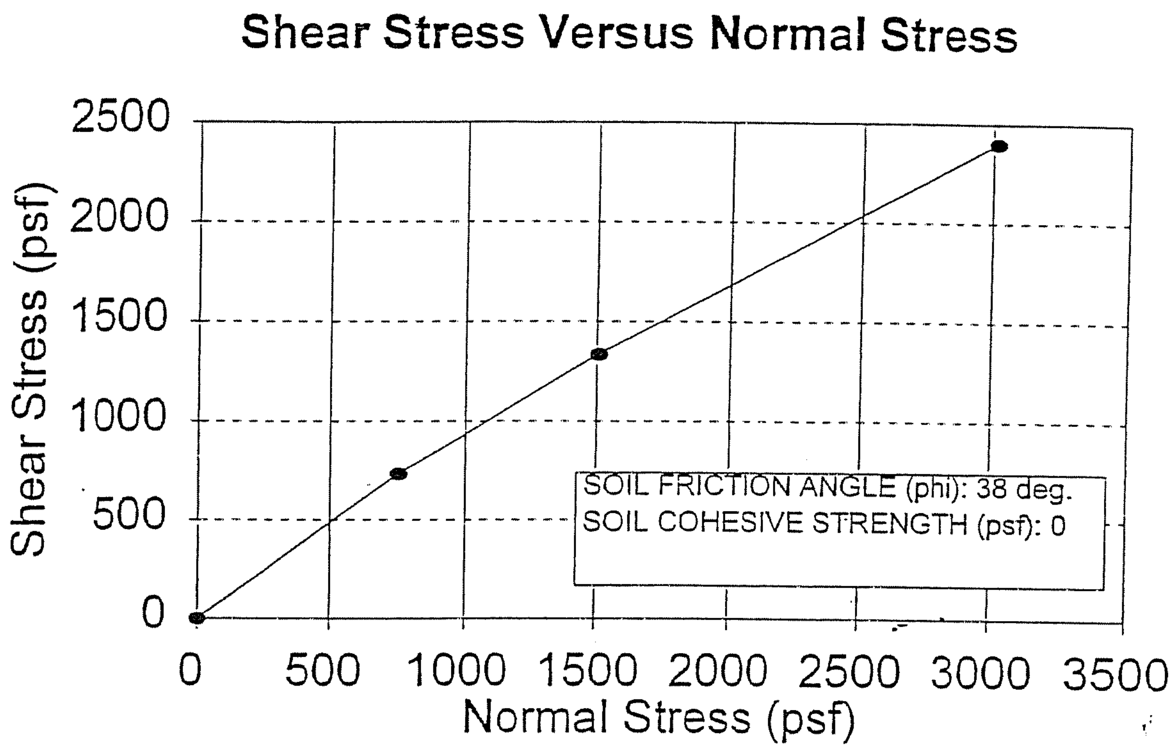
PLATEAU RESOURCES LIMITED
SHOOTARING CANYON URANIUM MILL
GARFIELD COUNTY, UTAH
EXISTING CROSS VALLEY BERM
3-18-99 SCALE 1'' = 100'

Attachment B

Laboratory Test Results

PROJECT: Shootering Canyon Dam

CLIENT: U.S. Energy Corporation



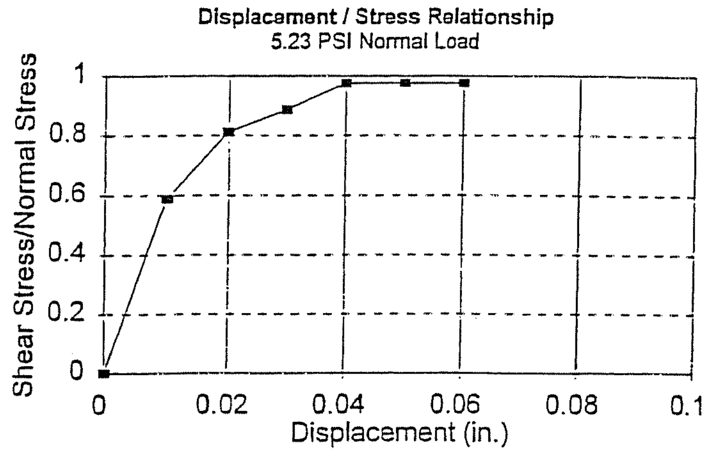
TEST DATE: 4/17/97

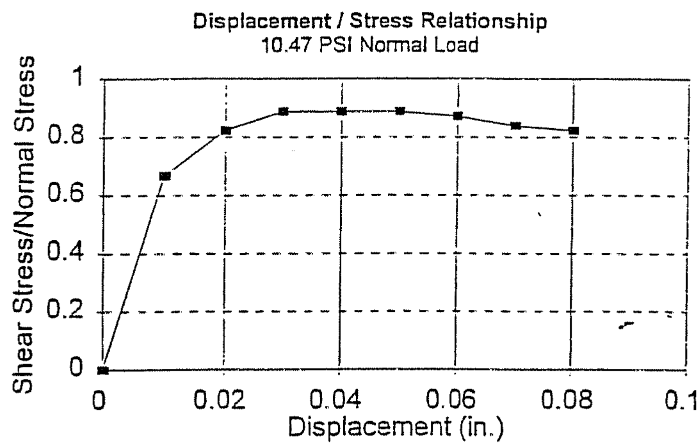
SAMPLE DESCRIPTION: Light Brown Fine Grained Sand

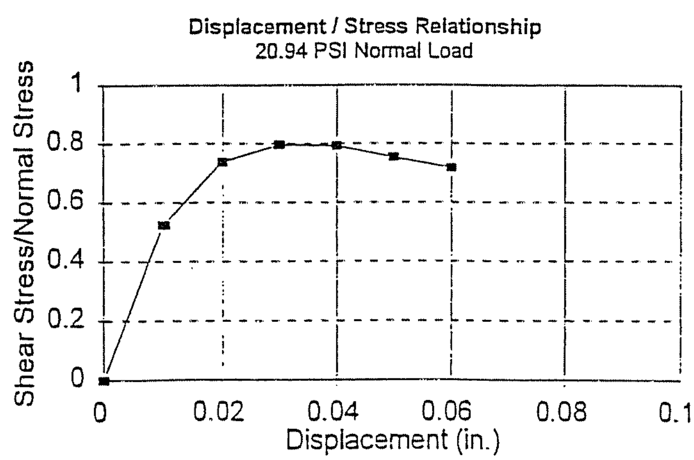
SOURCE: Cross Valley Berm

TECHNICIAN: DSD/GLM

PROJECT: Shootering Canyon Dam CLIENT: U.S. Energy Corporati







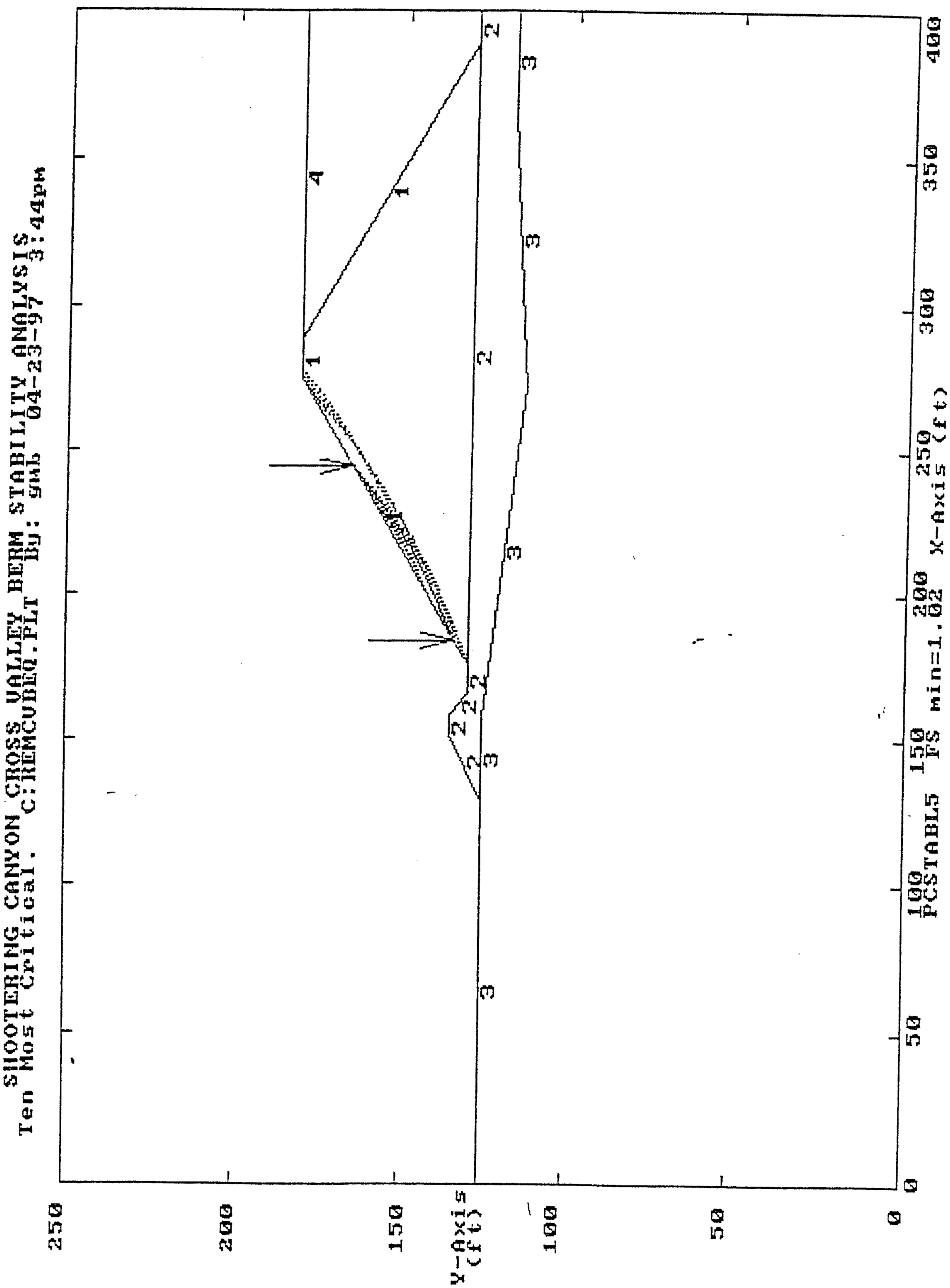
TEST DATE: 4/17/97 TECHNICIAN: DSD/GLM

SAMPLE DESCRIPTION: Light Brown Fine Grained Sand

SOURCE: Cross Valley Berm

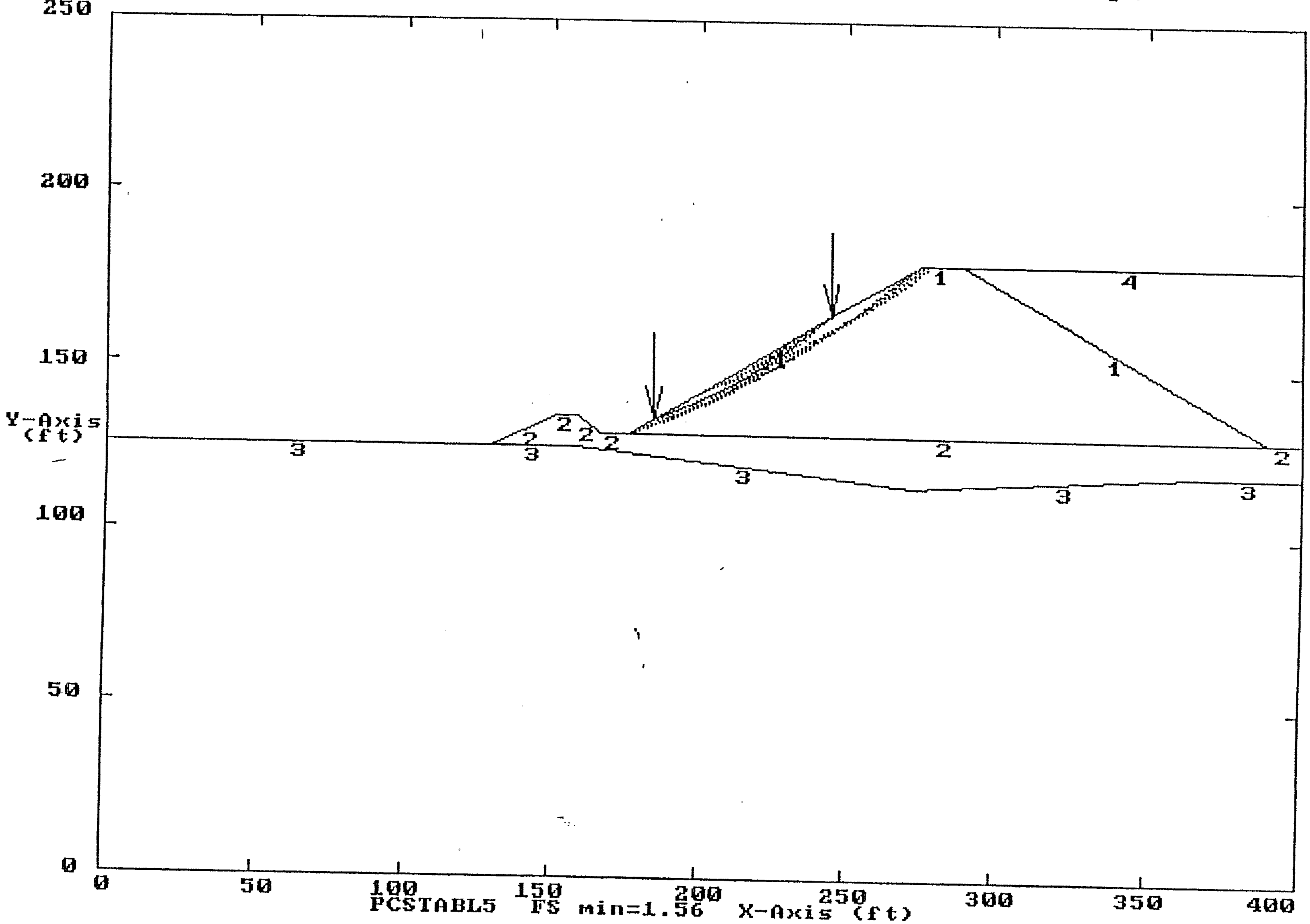
Attachment C
PCSTABL Input and Critical Section Plots

```
PROFIL        C:REMCVBEQ.IN  PCSTABL Version 6
SHOOTERING CANYON                          CROSS VALLEY BERM STABILITY AN
SIS
15 8
0. 126. 129. 126. 3
129. 126. 151. 135. 2
151. 135. 158. 135. 2
158. 135. 166. 130. 2
166. 130. 176. 130. 2
176. 130. 275. 180. 1
275. 180. 289. 180. 1
289. 180. 400. 180. 4
289. 180. 390. 130. 1
176. 130. 390. 130. 2
390. 130. 400. 130. 2
129. 126. 158. 126. 3
158. 126. 274. 114. 3
274. 114. 368. 119. 3
368. 119. 400. 119. 3
SOIL
4
115. 115. 0. 38. 0. 0. 0
125. 125. 1500. 0. 0. 0. 0
140. 140. 1000. 45. 0. 0. 0
100. 100. 0. 10. 0. 0. 0
EQUAKE
0.19 0. 0.
CIRCLE-Janbu circular, search.
20 20
150. 200. 230. 280. 0. 15. 0. 0.
```


SHOOTERING CANYON CROSS VALLEY BERM STABILITY ANALYSIS
Ten Most Critical. C:REMCUBEQ.PLT By: gwb 04-23-97 3:44pm
PCSTABL5 FS min=1.02
X-Axis (ft)
Y-Axis (ft)
0
50
100
150
200
250
300
350
400

```
PROFIL        C:REMCVB.IN  PCSTABL Version 6
SHOOTERING CANYON                            CROSS VALLEY BERM STABILITY AN
SIS
15 8
0. 126. 129. 126. 3
129. 126. 151. 135. 2
151. 135. 158. 135. 2
158. 135. 166. 130. 2
166. 130. 176. 130. 2
176. 130. 275. 180. 1
275. 180. 289. 180. 1
289. 180. 400. 180. 4
289. 180. 390. 130. 1
176. 130. 390. 130. 2
390. 130. 400. 130. 2
129. 126. 158. 126. 3
158. 126. 274. 114. 3
274. 114. 368. 119. 3
368. 119. 400. 119. 3
SOIL
4
115. 115. 0. 38. 0. 0. 0
125. 125. 1500. 0. 0. 0. 0
140. 140. 1000. 45. 0. 0. 0
100. 100. 0. 10. 0. 0. 0
CIRCLE-Janbu circular, search.
20 20
150. 200. 230. 280. 0. 15. 0. 0.
```

SHOOTERING CANYON CROSS VALLEY BERM STABILITY ANALYSIS
Ten Most Critical. C:REMCUB.PLT By: gmb 04-23-97 3:45pm



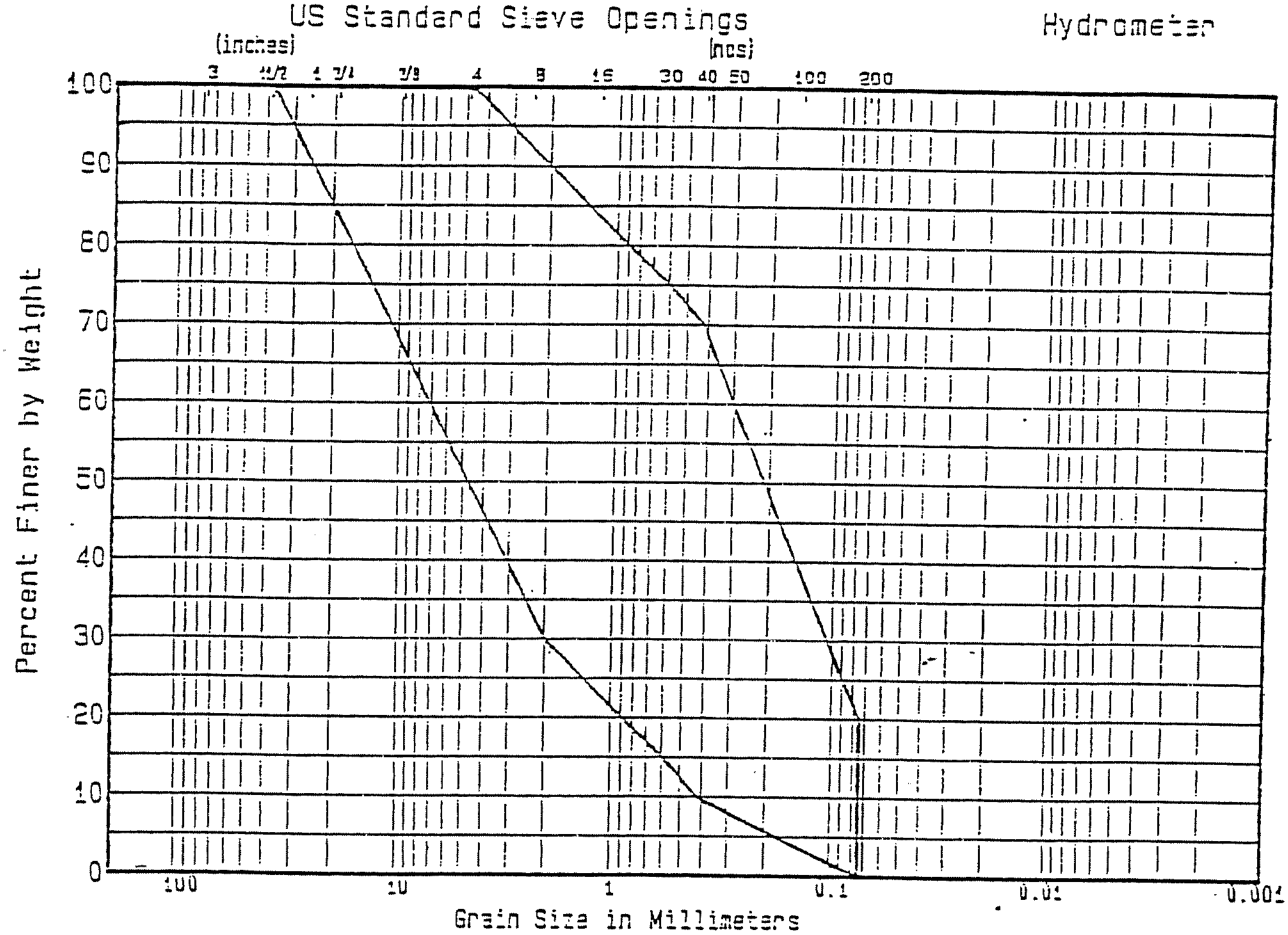
Attachment D
Envelope A

GRADATION ENVELOPE A

Project: Shootering Canyon

Client: U.S. Energy Corp.

Job No.: 7664 RX



Cobbles	Coarse Gravel	Fine Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt	Clay

Unified Soil Classification System (ASTM D2487)

Material Description (Envelope A): GRANULAR FILL

Recommended Gradation :

Sieve Size	% Passing
1.5 in.	100
# 4	50 - 100
# 10	30 - 90
# 30	15 - 75
# 40	10 - 70
# 200	0 - 20

A.4 Deformation Analysis, Letter Report
by Inberg-Miller Engineers, January 28, 1999

INBERG-MILLER ENGINEERS

124 EAST MAIN STREET RIVERTON, WYOMING 82501-4397 307-856-8136

January 28, 1999 7664-RX

U.S. Energy Corporation
877 N. 8th West
Riverton, Wyoming 82501

ATTENTION: FRED CRAFT

RE: NEWMARK ANALYSIS
SHOOTARING CANYON DAM (UT00417)

Dear Sir;

This letter summarizes the results of a deformation analysis that we performed for the above referenced project pursuant to the July 1, 1998 letter of review comments by the State of Utah.. We understand that an evaluation of seismic deformation based on a magnitude 6.5 earthquake with a peak ground acceleration of 0.33g is required. These services are in addition to previous slope stability analyses that we performed for the above project.

SUMMARY OF APPROACH

The deformation analysis method is described in the following reference:

> N.M. Newmark, 1965, "Effects of Earthquakes on Dams and Embankments" *Geotechnique, Vol. 15, Issue 2, pp. 139-160*

We understand that the deformation analysis estimates ground displacement due to seismic forces. The above reference suggests methodology for evaluating cumulative displacement and resultant deformation of sloping soils exposed to repetitive forces, as in the case of an earth dam experiencing seismic shaking.

Displacement is estimated according to the following equation:

$$(V^2/(2gN)) \times (A/N)$$

where:

V = velocity of ground motion
g = acceleration due to gravity
N = $((\tan \phi/\tan \theta) - 1) \sin \theta$
ϕ = Internal soil friction angle
θ = Embankment slope angle
A = Percent of peak acceleration of ground motion

We are able to readily establish the basis for all of the above parameters except velocity.

GROUND MOTION VELOCITY

Velocity of ground motion for the subject site is apparently not available. We spoke with Robert Smith of the University of Utah-Geology Department and Dave Perkins of the U.S. Geological Survey, and neither were aware of any recordings of strong ground motion within an applicable distance of the project site where velocity could be determined. Based on information provided in the above referenced publication (Newmark, 1965), velocities between 8.3 and 13.7 in/sec were recorded in Pacific Coast states.

In order to establish the basis for an appropriate velocity, we reviewed the following document:

> David J. Leeds, 1992, "Recommended Accelerograms for Earthquake Ground Motions", *Miscellaneous Paper S-73-1, Report 28, prepared for Department of the Army*

The above document provides recommended ground motion velocity based on the parameters of earthquake magnitude, distance from the epicenter, focal depth, and whether the site is hard or soft.

As a basis for evaluating parameters for the subject site, we referenced the following map:

> "Earthquakes in Utah, 1889-1985", *United States Geological Survey – National Earthquake Information Center, 1990*

According to the above map, the closest epicenter to the site for the range of earthquake magnitudes in Utah are listed below:

Mag 6.0	165 km
Mag 4.9	34 km
Mag 3.9	6 km

The map indicates that focal depth for all earthquake data is less than 25 km. Based on our knowledge of site geology, the site is underlain by sedimentary bedrock that meets the definition of a "hard site".

For the purpose of establishing a conservative velocity for use in analysis, we utilized the following parameters in conjunction with Figure 20c of "Recommended Accelerograms for Earthquake Ground Motions" (see attached):

Earthquake Magnitude	6.5
Distance to Epicenter	10 km
Focal Depth	<19 km
Site Conditions	Hard

Accordingly, a velocity of 50 cm/sec (20 in/sec) for mean velocity plus one standard deviation appears conservative and appropriate.

DEFORMATION ANALYSIS RESULTS

Values used for each parameter for deformation analysis are summarized as follows:

$V = 20$ in/sec
$g = 386.4$ in/sec^2 (32.2 ft/sec^2)
$N = 0.30$
$\phi = 40^\circ$ (as previously established for this site)
$\theta = 26.6^\circ$ (for 2 H: 1V dam face)
$A = 0.33$ (per the attached USGS Peak Acceleration Map)

We calculate a displacement of 1.9 inches based on the above parameters and references. In our opinion, the displacement as indicated does not appear significant to the integrity and performance of the subject dam.

Please feel free to call if you have any questions.

Sincerely,

INBERG-MILLER ENGINEERS

Glen M. Bobnick, P.E.
Geotechnical Engineer

GMB:jlw:geotech\7664-RX

Enclosures as stated

– from "Recommended Accelerograms for
– Earthquake Ground Motions"

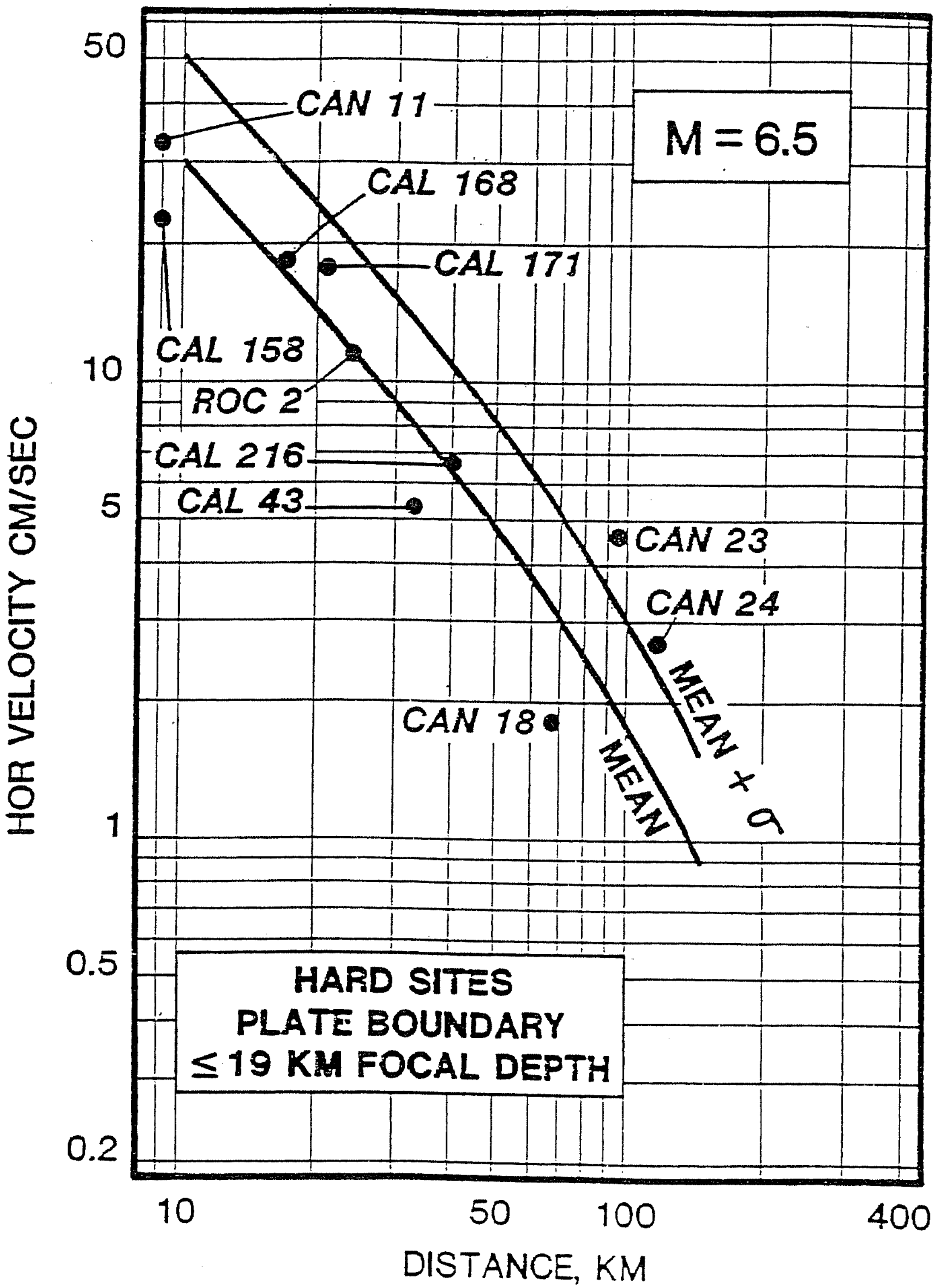


Figure 20c. Accelerograms for velocity, M = 6.5, and distance from source for shallow earthquakes at hard sites. (See Table 20c.)

no fault-specific sources; site: NEHRP B-C boundary

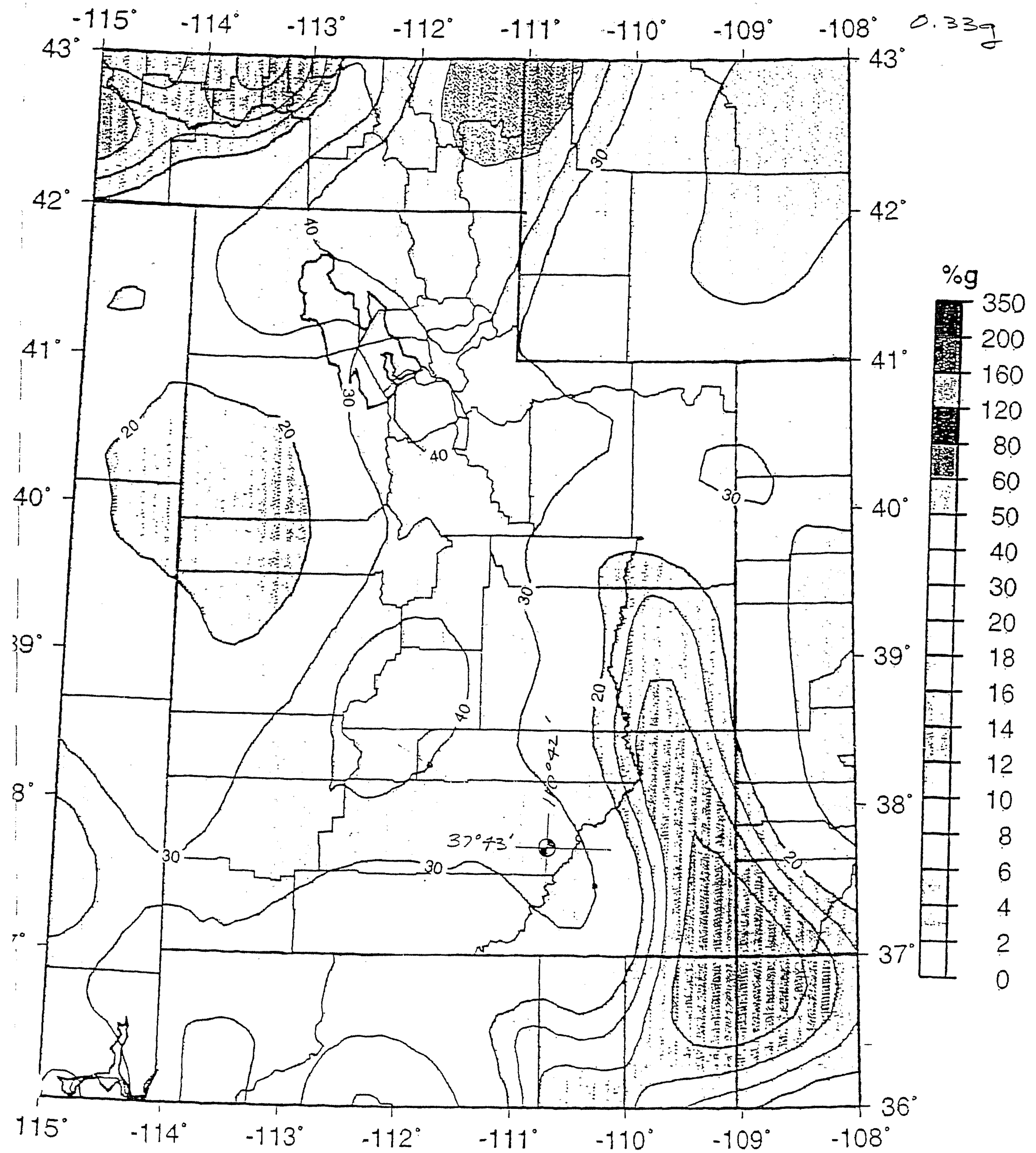
U.S. Geological Survey

National Seismic Hazard Mapping Project

$\frac{23}{77} = 0.3$

$0.3 \times 10 = 3$

0.33g



A.5 Newmark Analysis, Letter Report
by Inberg-Miller Engineers, June 14, 1999

I N B E R G - M I L L E R   E N G I N E E R S
124 EAST MAIN STREET   RIVERTON, WYOMING 82501-4397   307-856-8136

June 14, 1999   7664-RX

U.S. Energy Corporation
877 N. 8th West
Riverton, Wyoming 82501

ATTENTION: FRED CRAFT

RE: NEWMARK ANALYSIS
CROSS VALLEY BERM
SHOOTARING CANYON URANIUM PROJECT, UTAH

Dear Sir:

This letter summarizes the results of a deformation analysis that we performed for the above referenced project pursuant to your request. We understand that an evaluation of seismic deformation based on a magnitude 6.5 earthquake with a peak ground acceleration of 0.33g is required. These services are in addition to previous slope stability analyses that we performed for the above project.

SUMMARY OF APPROACH

The deformation analysis method is described in the following reference:

N.M. Newmark, 1965, "Effects of Earthquakes on Dams and Embankments" *Geotechnique, Vol. 15, Issue 2, pp. 139-160*

We understand that the deformation analysis estimates ground displacement due to seismic forces. The above reference suggests methodology for evaluating cumulative displacement and resultant deformation of sloping soils exposed to repetitive forces, as in the case of an earth dam experiencing seismic shaking.

Displacement is estimated according to the following equation:

$$(V^2/(2gN)) \times (A/N)$$

where:

V = velocity of ground motion
g = acceleration due to gravity
N = $((\tan \phi/\tan \theta) - 1) \sin \theta$
ϕ = Internal soil friction angle
θ = Embankment slope angle
A = Percent of peak acceleration of ground motion

We are able to readily establish the basis for all of the above parameters except velocity.

GROUND MOTION VELOCITY

Velocity of ground motion for the subject site is apparently not available. We spoke with Robert Smith of the University of Utah-Geology Department and Dave Perkins of the U.S. Geological Survey, and neither were aware of any recordings of strong ground motion within an applicable distance of the project site where velocity could be determined. Based on information provided in the above referenced publication (Newmark, 1965), velocities between 8.3 and 13.7 in/sec were recorded in Pacific Coast states.

In order to establish the basis for an appropriate velocity, we reviewed the following document:

> David J. Leeds, 1992, "Recommended Accelerograms for Earthquake Ground Motions", *Miscellaneous Paper S-73-1, Report 28, prepared for Department of the Army*

The above document provides recommended ground motion velocity based on the parameters of earthquake magnitude, distance from the epicenter, focal depth, and whether the site is hard or soft.

As a basis for evaluating parameters for the subject site, we referenced the following map:

> "Earthquakes in Utah, 1889-1985", *United States Geological Survey – National Earthquake Information Center, 1990*

According to the above map, the closest epicenter to the site for the range of earthquake magnitudes in Utah are listed below:

Mag 6.0	165 km
Mag 4.9	34 km
Mag 3.9	6 km

The map indicates that focal depth for all earthquake data is less than 25 km. Based on our knowledge of site geology, the site is underlain by sedimentary bedrock that meets the definition of a "hard site".

For the purpose of establishing a conservative velocity for use in analysis, we utilized the following parameters in conjunction with Figure 20c of "Recommended Accelerograms for Earthquake Ground Motions" (see attached):

Earthquake Magnitude	6.5
Distance to Epicenter	10 km
Focal Depth	<19 km
Site Conditions	Hard

Accordingly, a velocity of 50 cm/sec (20 in/sec) for mean velocity plus one standard deviation appears conservative and appropriate.

DEFORMATION ANALYSIS RESULTS

Values used for each parameter for deformation analysis are summarized as follows:

$V = 20$ in/sec
$g = 386.4$ in/sec^2 (32.2 ft/sec^2)
$N = 0.25$
$\phi = 38^{\circ}$ (as previously established for this site)
$\theta = 26.6^{\circ}$ (for 2 H: 1V dam face)
$A = 0.33$ (per the attached USGS Peak Acceleration Map)

We calculate a displacement of 2.7 inches based on the above parameters and references. In our opinion, the displacement as indicated does not appear significant to the integrity and performance of the subject dam.

Please feel free to call if you have any questions.

Sincerely,

INBERG-MILLER ENGINEERS

Glen M. Bobnick, P.E.
Geotechnical Engineer

GMB:jlw:client letters\7664-RX

Enclosures as stated

_ from "Recommended Accelerograms for
_ Earthquake Ground Motions"

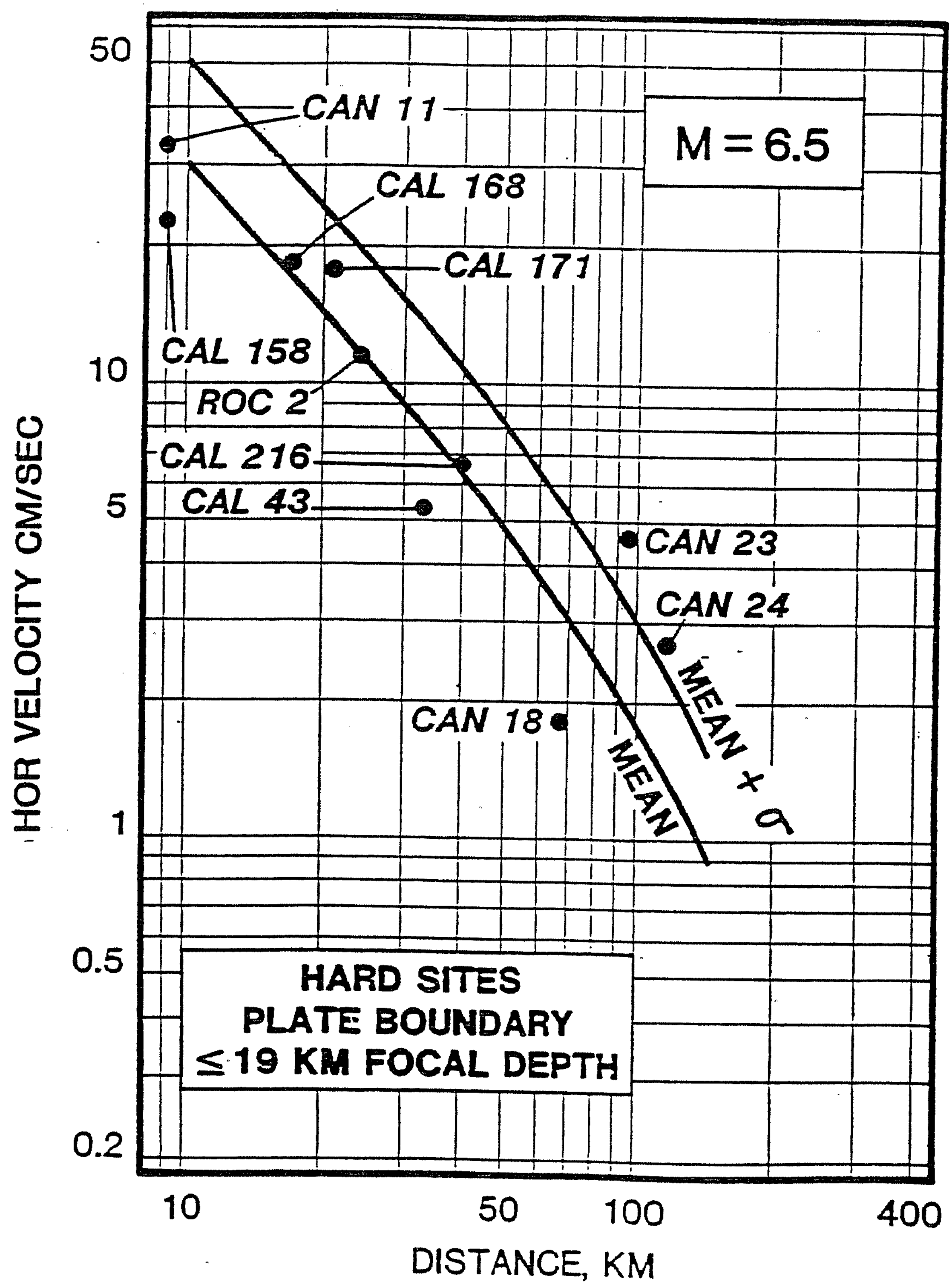


Figure 20c. Accelerograms for velocity, M - 6.5, and distance from source for shallow earthquakes at hard sites. (See Table 20c.)

no fault-specific sources; site: NEHRP B-C boundary

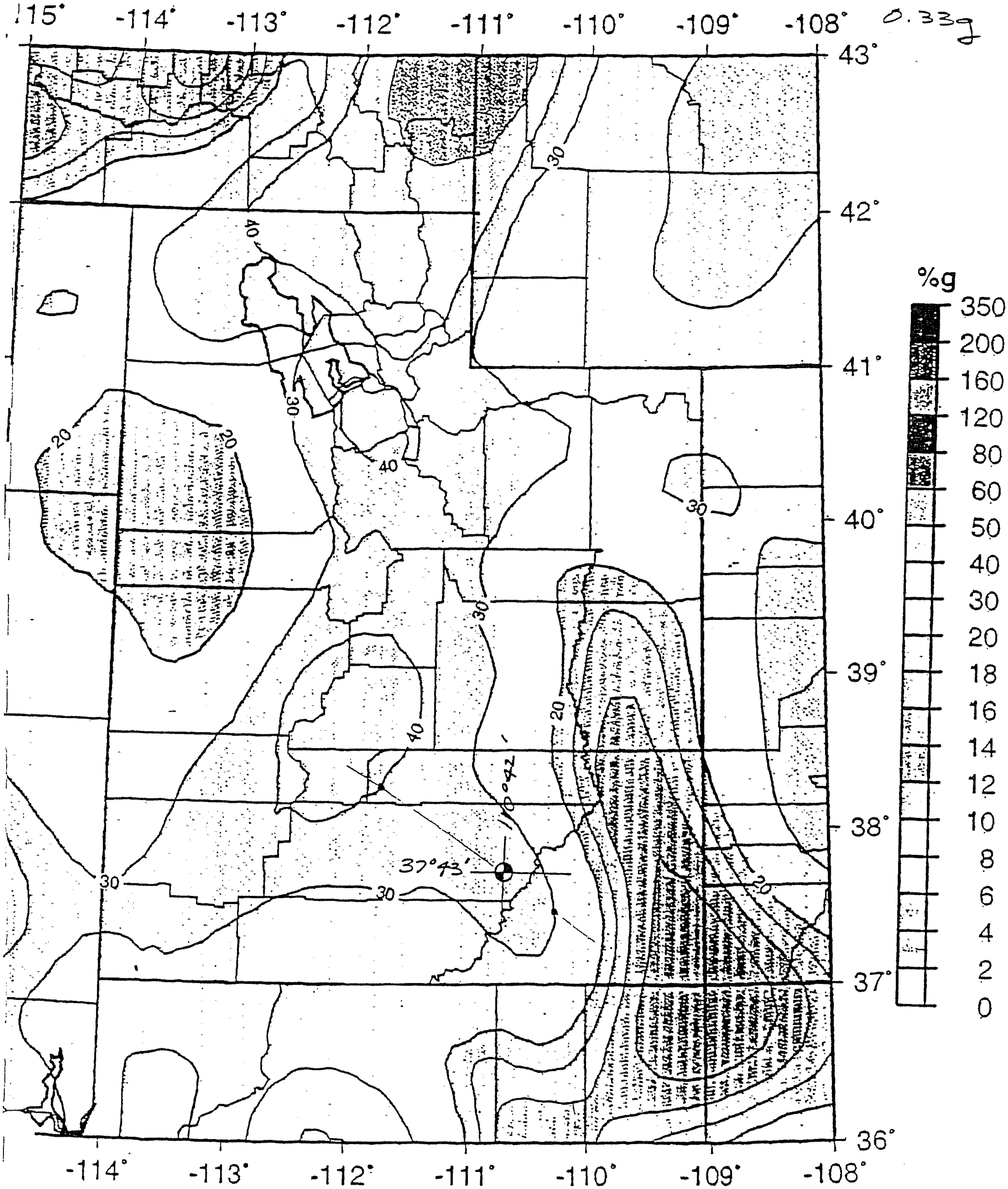
U.S. Geological Survey

National Seismic Hazard Mapping Project

$\frac{23}{77} = 0.3$

$0.3 \times 10 = 3$

0.33g



A.6 Tailings Dam Stability Approval Letter **from State of Utah Department of Natural Resources Division of Water Rights, State Engineer, March 8, 1999**

State of Utah
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF WATER RIGHTS

Michael O. Leavitt
Governor
Ted Stewart
Executive Director
Robert L. Morgan
State Engineer

1594 West North Temple, Suite 220
Box 146300
Salt Lake City, Utah 84114-6300
801-538-7240
801-538-7467 (Fax)

RECEIVED MAR 1 1 1999

March 8, 1999

F.R. Craft
Plateau Resources LTD.
877 North 8th West
Riverton, WY 82501

Re: Shootaring Canyon Mill Tailings Dam/UT00417 - Stability Analysis

We have completed our review of the information submitted with your letter of June March 4, 1998. Based on our review, we find the explanations and analyses to be acceptable, and the Shootaring Canyon Mill Tailings Dam meets the stability criteria adopted by this office.

If you have any questions concerning the preceding information, please feel free to contact Richard Hall, (801) 538-7373 of our Dam Safety Section.

Sincerely,

FOR Robert L. Morgan, P.E.
State Engineer

RLM/rbh/jm

pc: Mark Page - Regional Engineer

A.7 Ultimate Dam Stage Seismic Stability Analysis, Letter Report **by Inberg-Miller Engineers, January 11, 2007**

INBERG-MILLER ENGINEERS

QUALITY SOLUTIONS THROUGH TEAMWORK

January 11, 2007 7664-RX

JAN 15 2007

Mr. Fred Craft
U. S. Energy Corporation
877 North 8th West
Riverton, WY 82501

RE: SEISMIC STABILITY ANALYSIS RESULTS
SHOOTARING CANYON DAM – ULTIMATE STAGE
GARFIELD COUNTY, UTAH

Dear Mr. Craft:

This letter summarizes the results of our seismic stability analysis performed for the Ultimate Stage of the Shootaring Canyon Dam located in Garfield County, Utah. The work described in this letter has been performed per Amendment No. 4 dated January 3, 2007 of our Service Agreement and Appendix A dated November 21, 1996.

BACKGROUND

We understand that Stage I of the Shootaring Canyon Dam was completed in 1982 and consists of a zoned earthen dam constructed to impound uranium mill tailings. Inberg-Miller Engineers performed a slope stability analysis for the existing dam configuration, the results of which are contained in our January 9, 1997 letter report. We understand that construction of the Ultimate Stage of the tailings dam will include modifications to the existing dam to increase overall capacity. These modifications include:

1. Placing a buttress on the upstream face of the dam to flatten the slope from 2:1 (H:V) to 3:1 (H:V).

2. Raising the dam crest elevation approximately 30 feet.

3. Using "PASTE" technology to place tailings behind the dam at a 5:1 (H:V) slope extending above the raised dam crest elevation approximately 25 feet.

SOIL PARAMETERS

The dam is comprised of three zones, which are described in our January 9, 1997 letter report. For the stability analysis, the interior zones (Zones 1 and 3) were extended to the proposed Ultimate Stage dam crest elevation. A summary of the engineering properties used for the slope stability analysis is tabulated below.

Soil No.	Description	Moist Unit Weight (pcf)	Cohesion (psf)	Friction Angle (ϕ°)
1	Zone 1 – Silty, Sandy, Clay	125	1,500	0
2	Zone 2 – Boulders, Cobbles, Gravels, Sand	131	0	40
3	Zone 3 – Fine Sand	125	0	32
4	Rock Foundation	140	1,000	45
5	Tailings	100	0	10

GROUNDWATER CONDITIONS

We understand that the impoundment will be lined with an impermeable HDPE liner. Consequently, the analysis assumed that no phreatic surface will develop through the earthen dam.

SEISMIC CONDITIONS

A horizontal seismic coefficient of 0.19g was used for the stability analysis. The basis for seismic conditions is the same as described in our January 9, 1997 letter report.

ANALYSIS RESULTS

Slope stability analysis was performed for the proposed Ultimate Stage dam configuration using the computer program "SLOPE/W". Based on the design parameters described above, a calculated minimum safety factor of 1.18 was determined using the Janbu method. The critical failure surface is characterized as an "infinite slope failure" which is planar and parallel to the downstream slope face. This failure type was also determined for the original Stage I slope stability analysis using the same downstream slope of 2:1 (H:V). Consequently, the calculated factor of safety of 1.18 is comparable to the Stage I safety factor of 1.14 presented in our January 9, 1997 letter report. Results of the Ultimate Stage Slope Stability Analysis are attached.

CLOSURE

We appreciate participating in your project. Please call at (307) 856-8136 if you have any questions regarding the services performed.

Sincerely,

INBERG-MILLER ENGINEERS

Travis E. Guthrie
Travis E. Guthrie, E.I.T.
Geotechnical Engineer

REVIEWED BY:

Glen M. Bobnick
Glen M. Bobnick, P.E.
Geotechnical Engineer

TEG:GMB:bjh\7664 stability ltr

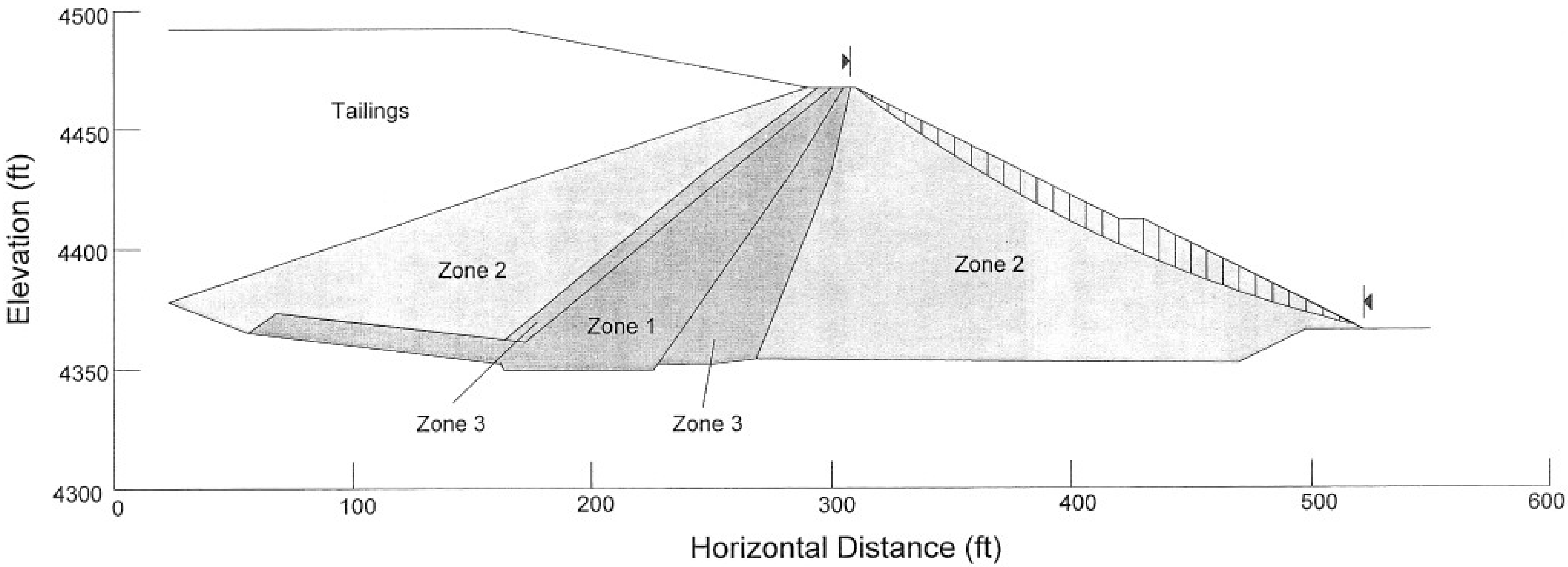
Client: U.S. Energy Corporation
Project: Shootaring Canyon Tailings Dam Stability
IME No: 7664-RX
Date: 01/05/07
Analysis Method: Bishop (with Ordinary & Janbu)
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
Seismic Coefficient: 0.19
Minimum Safety Factor: 1.183

Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 10 deg

Zone 1
Sandy, Silty, Clay
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 1500 psf
Phi: 0 deg

Zone 2
Boulders, Cobbles, Gravels, Sands
Soil Model: Mohr-Coulomb
Unit Weight: 131 pcf
Cohesion: 0 psf
Phi: 40 deg

Zone 3
Fine Sand
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 32 deg



APPENDIX B

DRAINAGE FILTER ANALYSIS

APPENDIX B

TABLE OF CONTENTS

Page Number

FIGURES

APPENDIX B

Drainage Filter Analysis

B.0 Introduction

A three-layer drainage filter will be installed in the tailings cells as a primary component of the drainage collection system. This drainage filter will protect the HDPE liner and serve as a means of conveying drainage from the tailings to a collection pipe for eventual discharge to a collection sump. The properties of the drainage filter layers are specified to provide both the necessary filtration and conveyance functions.

B.1 Drainage Filter Configuration

The planned drainage system includes a perforated pipe network that is installed within a three-layer drainage blanket that will be installed over the pond base. The bottom layer of the drainage blanket will consist of six (6) inches of Entrada sand. One of the primary purposes of this bottom Entrada sand layer is to protect the upper HDPE liner from puncture by stones within the middle layer which will consist of a six (6) in thick sand and gravel material produced from the quarry area. The uppermost drainage blanket layer will consist of six (6) inches of Entrada sand. In addition to providing a protective layer for the HDPE liner, the use of two distinct materials has the advantage of providing a more robust drainage blanket. The sand and gravel material from the quarry area is generally slightly coarser and should have a somewhat greater permeability, and the presence of the upper Entrada sand layer should prevent intrusion of tailings fines into the coarser middle layer. The use of two materials with differing mineralogy also reduces the potential for degradation of the entire drainage blanket by an adverse geochemical process.

The two major functions of the three layer drainage blanket are:

> To convey tailings solution to the drainage pipe network or directly to the sump and thereby prevent the accumulation of excess head over the HDPE liner.
>
> To prevent excessive intrusion of the tailings into the drainage blanket or piping system. Intrusion of fines into the blanket could eventually result in plugging of the blanket and drain system.

Underground drainage system filter/envelope design criteria were used in evaluating the suitability of the proposed materials. These criteria are presented in “Drainage of Agricultural Land” which is published by the Water Information Center Inc. The criterion which limits the fine fraction to no more than 10% passing a No. 60 sieve is waived because a geotextile wrapped gravel envelope or fabric sock will be used to restrict movement of fines into the piping system. Chapter 26 (“Gradation Design of

Sand and Gravel Filters") of the USDA-NRCS National Engineering Handbook also presents relevant design criteria that were considered in the evaluation of the proposed filter materials.

B.2 Entrada Sand and Possible Tailings Properties

Sieve analysis was conducted on two Entrada sand samples during evaluation of the existing tailings facility. The results of this analysis are presented in Figure B-1 along with gradations for three tailings samples. Entrada sand is a very uniform fine sand with only a very small silt and clay fraction. In contrast, the gradation of uranium tailings can range from a slime with more than 85% passing the #200 screen, to a medium to coarse sand with a relatively small fines fraction. The coarsest of the tailings samples in Figure B-1 was taken from the existing tailings at the Shootaring site. The other two samples were taken from a uranium tailings facility in central Wyoming. The three tailings samples generally span the expected range of tailings gradations.

The Entrada sand will be used as the lower and upper layers of the drainage filter system. Because the Entrada sand is free of stones and other debris, this lower layer will serve to guard the upper HDPE liner. The upper drainage layer of Entrada sand should be very effective in preventing the intrusion of tailings into the drainage layer.

From the standpoint of penetration of fines into the drainage layer and piping collection system, the critical tailings material is fine-grained slime tailings. Entrada sand is very uniform and there is no concern for a gap-graded material, so the applicable filter criterion is related to the maximum D_{15} of the Entrada sand. According to the criteria described in Chapter 26 of the USDA-NRCS National Engineering Handbook for a fine silt and clay base soil, the maximum D_{15} of the filter is less than or equal to 9 x d_{85} of the slime tailings base soil. Based on the gradations presented in Figure B-1, the D_{15} of the Entrada sand is suitable for tailings with a d_{85} as small as 0.01 mm. The minimum D_{15} is a function of the desired permeability of the filter material. Harr (1962) lists typical permeabilities of fine sand ranging from 0.001 to 0.05 cm/sec. Because the gradation of Entrada sand is very uniform, the permeability is likely 0.01 cm/sec or greater and is assumed to be approximately 0.05 cm/sec. Therefore, the properties of Entrada sand represent a reasonable compromise between filtration of fine tailings and the conveyance of drainage to the collection system.

B.3 Sand and Gravel Filter Properties

The middle layer of the drainage filter will consist of a processed material from the rocky soil in the quarry area near the mill site. There are large stones present in this rocky soil so the processing will necessarily include screening to remove stones larger than approximately three (3) inches in diameter. Because there will be a protective Entrada sand layer between the sand and gravel filter and the synthetic liners, the presence of coarse gravel-sized stones is acceptable. However, the size of the individual stones in the sand and gravel filter will be limited to approximately three (3) inches to facilitate placement within a six (6) inch thick layer. There will also be a layer of Entrada sand above the sand and gravel filter, so there is no concern for penetration of tailings into the sand and gravel filter. The primary function of the sand and gravel filter is to provide lateral and vertical conveyance of the drainage from the tailings to the drainage collection system.

Figure B-2 presents a comparison of the Entrada sand gradation with three gradations of potential sand and gravel filters.   The Quarry Fines sample was taken as the less than ½ inch fraction from the QU3 sample taken during a 2002 evaluation of the site. This gradation is generally coarser than the Entrada sand, and represents the finest material that would be considered for the sand and gravel filter. The Screened Rocky Soil gradation was generated by a virtual recombining of the Quarry Fines with the material between ½ inch and 3.25 inches from the original QU3 sample. This reflects the expected product that will result from a single screening operation the removes the larger than 3 inch fraction. The third sand and filter gradation (Double Screened Rocky Soil) represents the expected product when the quarry material is processed through a double screen to remove the larger than 3 inch fraction and a significant portion of the smaller than ¼ inch fraction. Since it is not necessary to remove all fines from sand and gravel filter, and the presence of some fine to coarse sand is desirable, it was assumed that the screening operation would be operated at a feed rate that resulted in the removal of 70% by weight of the less than ¼ inch fraction.

The gradations for the Screened Rocky Soil and Double Screened Rocky Soil represent the target range for the sand and gravel filter. This material is significantly coarser than the Entrada sand, which should result in a greater permeability. However, the presence of even a very small sand fraction within the screened quarry material will keep the D_{15}/d_{85} ratio generally in the range of 0.8 to 5. Significant intrusion of the Entrada sand into the sand and gravel filter is unlikely, but minor intrusion at the interface to the internal filter layer will not adversely affect the filter system performance. Depending on the processing operations, the proposed sand and gravel filter may be slightly gap-graded. However, it is the internal layer in a three layer filter system, and will be placed at a thickness of approximately six (6) inches, which should allow easy detection and correction of placement operations that result in segregation or other adverse placement conditions.

It would be possible to eliminate any gap grading from the sand and gravel filter by more aggressive screening to remove sand, silt and clay from the quarry area rocky soil. Figure B-3 presents a possible gradation for such a highly processed material. This material is generally less desirable as a sand and gravel filter material because the differential in size when compared with Entrada sand is so great that the intrusion of Entrada sand into the middle filter layer will be dramatic. If the Entrada sand does dramatically intrude into the gravel filter layer, the resulting filter system would likely be less permeable than the situation where the separation of the layers is maintained. However, the gradation shown in Figure B-3 does indicate that it may be possible to produce a gravel material for the collection sumps with additional processing of the quarry area material.

The conveyance capacity of the drainage layer will be a composite of the conveyance capacities of the Entrada sand and sand and gravel layers. With a gradation that falls in the range of the Screened Rocky Soil and Double Screened Rocky Soil shown in Figure B-2, the hydraulic conductivity of the sand and gravel layer is expected to be approximately 0.5 cm/sec or greater (estimated from a tabulation in Harr (1962)). Considering only the 12 inches of material directly above the liner, and estimating the hydraulic conductivity of the Entrada sand at 0.05 cm/sec, the composite hydraulic conductivity of the lower 12 inches of granular material is approximately 0.28 cm/sec.

B.4 Discussion

The combination of Entrada sand and a processed rocky soil material for a three layer filter results in a drainage filter system that should meet all performance objectives. The Entrada sand upper and lower filter layers will: prevent intrusion of tailings into the drainage collection system, guard the HDPE liner, and provide sufficient permeability to convey drainage to the collection system. The screened sand and gravel filter adds: enhanced permeability to rapidly convey drainage to the collection system, and multiple materials in the filter system to avoid compromising the entire system in the event of unforeseen chemical or physical degradation of a particular material.

B.5 References

Harr, M.E., 1962, “Groundwater and Seepage”, McGraw-Hill, New York.

USDA - NRCS, 1994, Part 633 – National Engineering Handbook, Chapter 26 - Gradation Design of Sand and Gravel Filters, U.S. Department of Agriculture, Washington D.C.

USDA - SCS, 1973, “Drainage of Agricultural Land”, Water Information Center, Inc. Port Washington, New York.

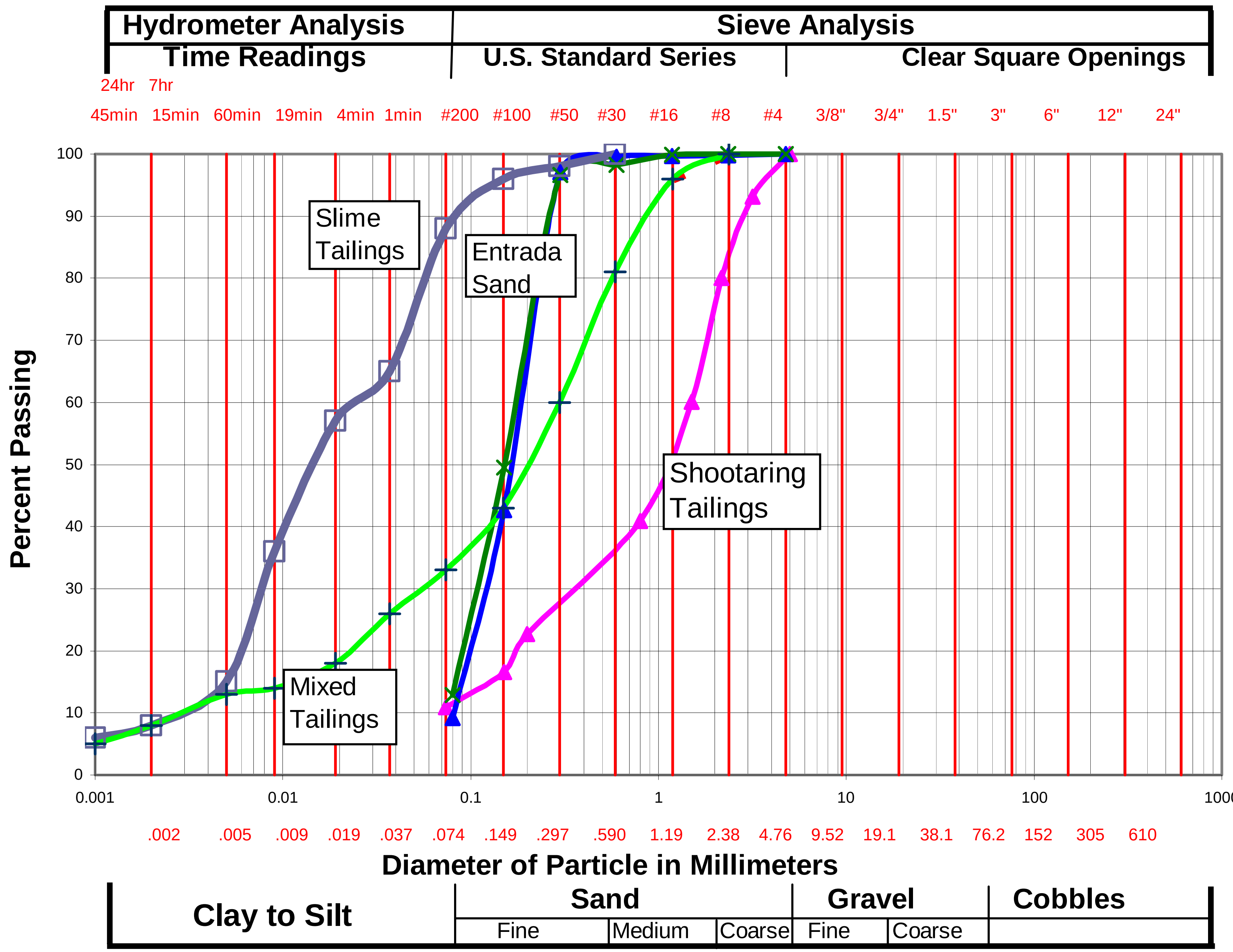


FIGURE B-1. ENTRADA SAND AND TAILINGS GRADATIONS

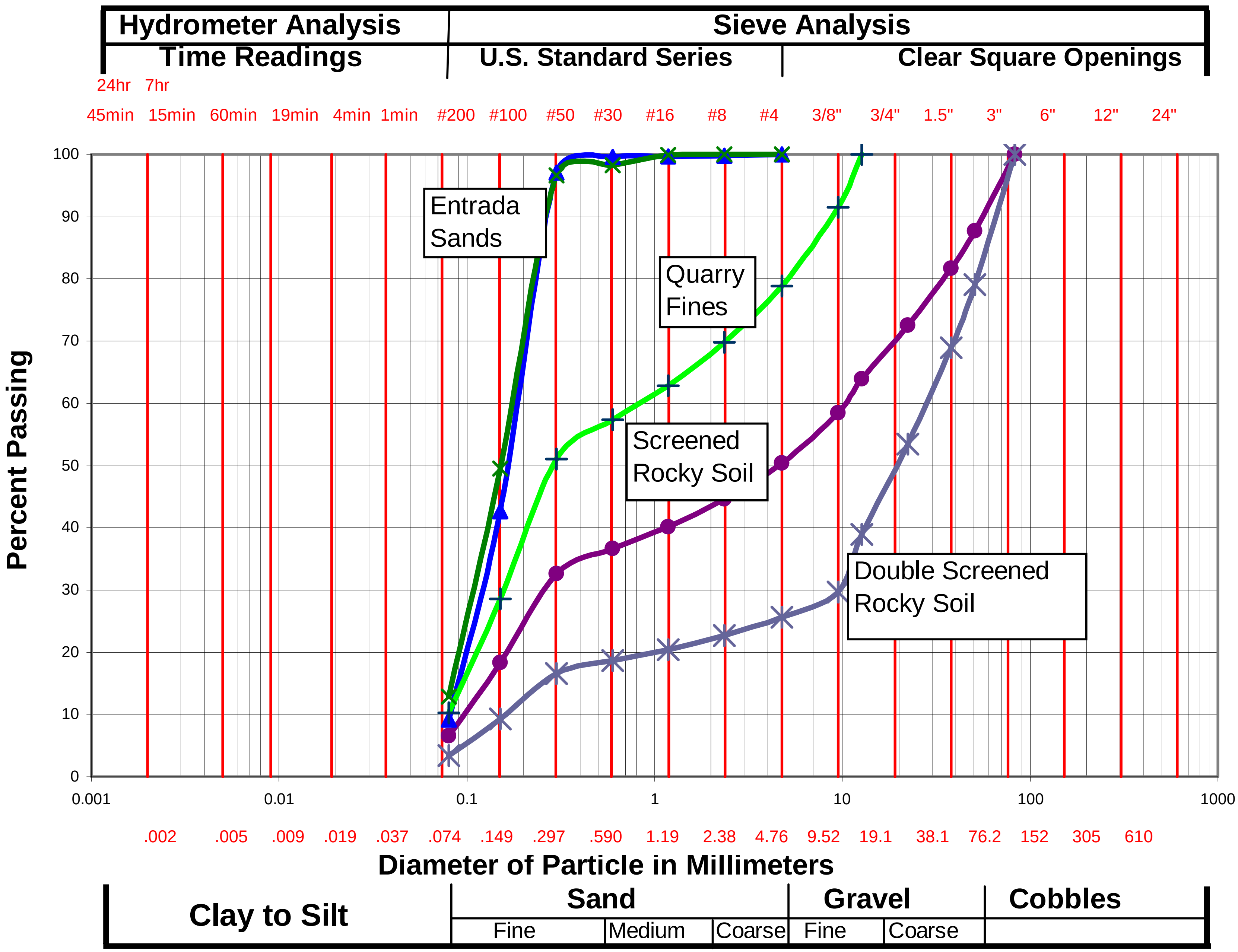


FIGURE B-2. ENTRADA SAND AND SAND AND GRAVEL FILTER GRADATIONS

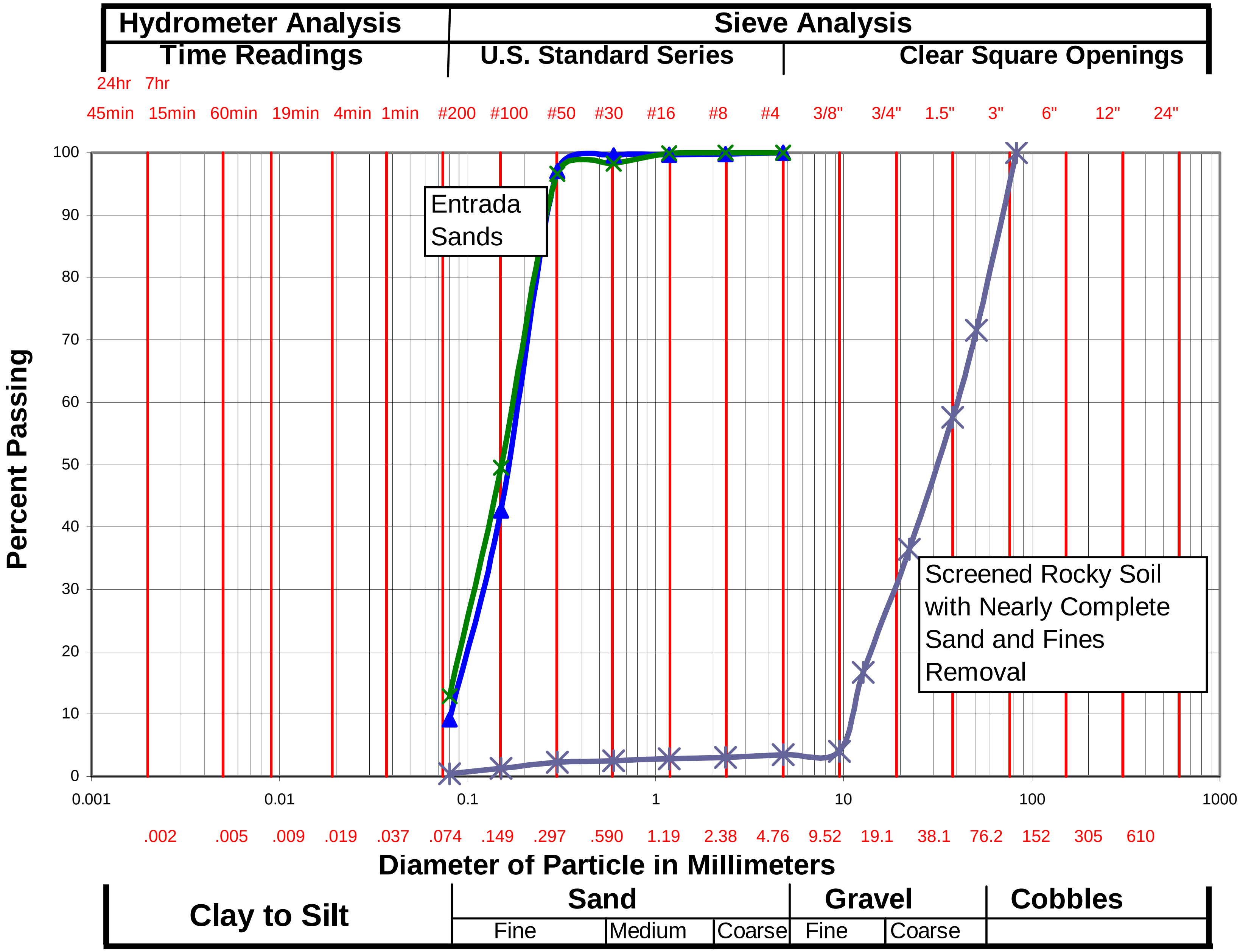


FIGURE B-3. HIGHLY PROCESSED SAND AND GRAVEL FILTER GRADATION

APPENDIX C

TAILINGS CONSTRUCTION CONTROL AND QUALITY ASSURANCE

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ATTACHMENTS

1.0 SCOPE OF QUALITY PLAN

The Quality Plan for the Tailings Impoundment Liner construction hereinafter referred to as the Quality Plan describes the implementation of the Construction Quality Control/Quality Assurance (QC/QA) methods and procedures. The Quality Plan shall be comprised of the following:

- Surveys, Inspections, Sampling and Testing
- Changes and Corrective Actions
- Documentation Requirements
- Construction Verification Program
- Quality Control Procedures

2.0 QUALITY PLAN OBJECTIVES

The main objectives of the Quality Plan for this project are to effectively control the quality of work performed, to verify that any and all construction activities are performed in accordance with the Plans and Specifications and to provide cross checks and audits to assure proper implementation of the quality control activities. Proper implementation of these objectives will provide detailed documentation of the project and assure that construction activities have been truly performed as specified in the Plans and Specifications.

3.0 DEFINITIONS

Compliance Report: A report prepared by the Quality Control Officer upon completion of a Construction Segment. Any subsequent Construction Segment that is dependent upon successful completion of a specific Construction Segment cannot not be initiated until a Compliance Report is prepared and approved for the previous dependent Construction Segment. The Compliance Report requires approval by the Design Engineer and the Site Manager. Compliance Reports are to be completed on Form No. PR-20.

Construction Task: A feature of the Construction Project involving a specific construction activity.

Construction Segment: An essential construction component consisting of one or more Construction Tasks of the Project. Upon completion of a Construction Segment, a Compliance Report is required to verify that this project component was constructed in accordance with the Final Plans and Specifications.

Construction Project: The total authorized/approved project, as defined in the Plans and Specifications, that requires several Construction Segments to complete.

Design Change: Any change made in the Construction Project that alters or changes the intent of the Plans and Specifications. Design changes require approval from the Design Engineer and the Site Manager or his designated representative. Design Changes are to be reported on Form No. PR-22.

Field Change: Changes made during construction to fit field conditions that do not alter the intent of the Final Plans and Specifications. Field changes require approval from the Site Manager or his designated representative. Field changes are to be reported on Form No. PR-21.

Final Construction Report: A report prepared by the Design Engineer or his designated representative upon completion of the construction project. This report shall contain "as-built" drawings, material tests, summaries, Compliance Reports and photographs of the construction activities associated with the Construction Project.

Quality Assurance: A planned system of activities and audits that establishes and exercises control over the reliability of any data produced, in terms of precision, accuracy, completeness and comparability.

Quality Control: A planned system of activities, tests and inspections by the designated Quality Control Officer or representative(s), used to directly monitor and control the quality of construction activities set forth in the Plans and Specifications.

4.0 QUALITY CONTROL/QUALITY ASSURANCE

4.1 Methodology

4.1.1 <u>Flow of Activities</u>

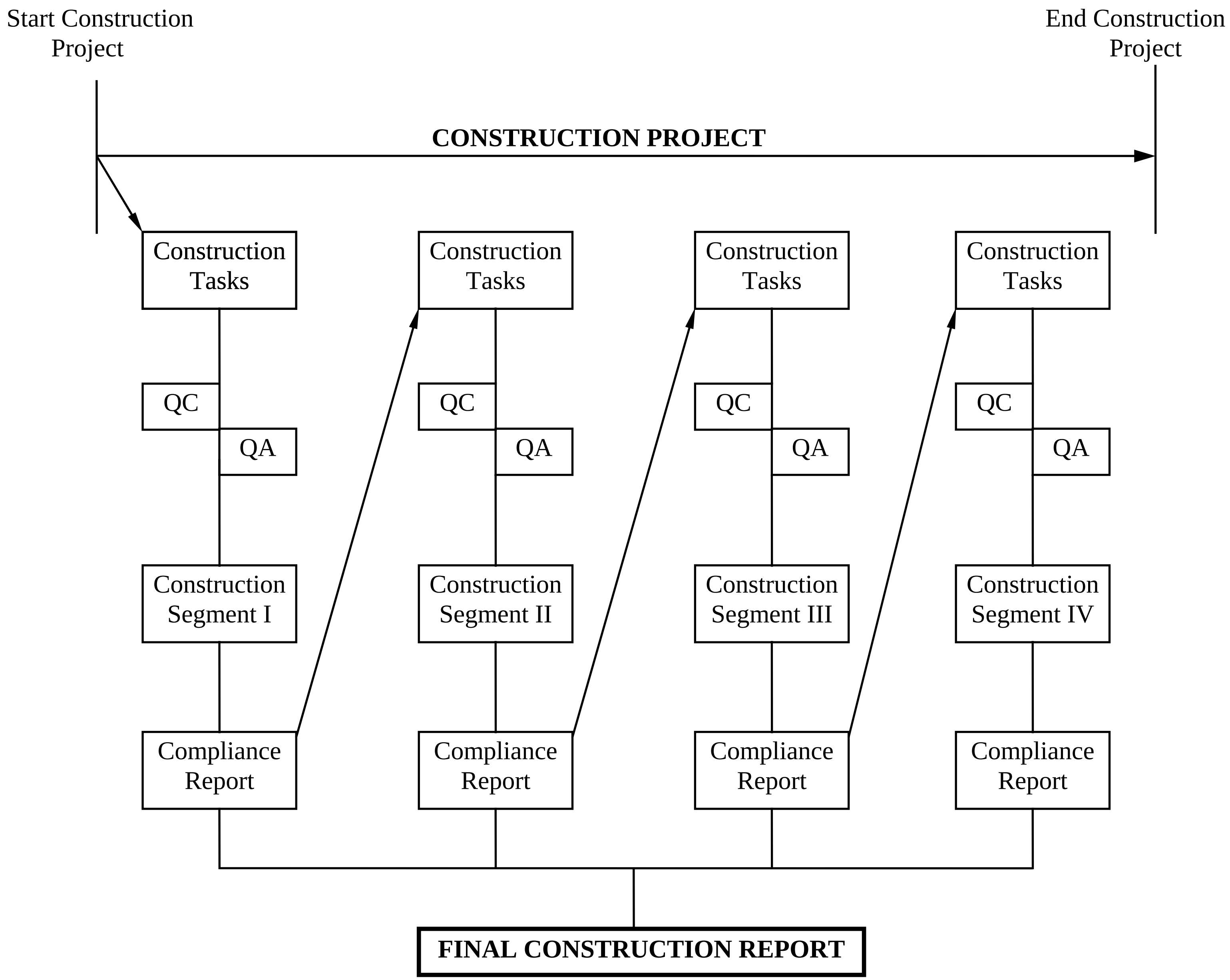
Figure 1 illustrates the general relationship between Quality Control and Quality Assurance activities and construction elements for any given project. The Quality Control activities, implemented with standardized Quality Control procedures provide the necessary tests and observations for construction monitoring and sampling. Quality Assurance audits and data validation will provide independent oversight of the Quality Control activities.

4.1.2 <u>Compliance Reports</u>

The Quality Plan requires a Compliance Report to be submitted upon the successful completion of a Construction Segment. The Construction Tasks that make up any Construction Segment shall be determined to be in compliance with the Plans and Specifications by the Quality Control Officer (hereinafter referred to as QC Officer). A Compliance Report along with all applicable support data will be prepared by the QC Officer and submitted to the Design Engineer and the Site Manager for approval before the next phase of construction can begin.

Upon completion of the Construction Project, a Final Construction Report shall be prepared by the Design Engineer or his designee for submittal to the proper Regulatory Agencies.

FIGURE 1 - TYPICAL FLOW CHART for CONSTRUCTION QUALITY CONTROL and ASSURANCE



4.2 Quality Control

4.2.1 General

Quality Control (QC) will be conducted under the direction of the QC Officer or his designee. The QC Officer will implement and administer the QC Program. The QC Officer may be an employee of the company or a Consultant, providing all qualifications are met.

4.2.2 Duties of the Quality Control Officer

The Quality Control Officer shall be responsible for the overall implementation and management of the Quality Control Program. He shall supervise field and laboratory Quality Control Technicians and control documentation of construction, quality control and quality assurance activities. He shall have specific authority and responsibility to reject any work or materials, to stop work, to require removal or replacement of unsatisfactory workmanship or materials, to specify and require appropriate corrective action if it is determined that the personnel, instructions, controls, tests or records are not in conformance to the Quality Control Program. The Quality Control Officer's signature shall be required on all Compliance Reports, inspections and tests.

The Quality Control Officer shall be familiar with the existing facilities and acceptable Quality Control/Quality Assurance methodologies. As Quality Control Officer, his responsibilities shall include the following:

- Conduct inspections and quality control testing to verify and document compliance with the Plans and Specifications.
- Must be familiar with all documents, requirements, equipment, and procedures relating to the project construction.
- Provide and document Quality Control Technician training.
- Prepare Compliance Reports.
- Arrange consultation with staff, the QA Officer, Site Manager, and/or Design Engineer to resolve problems or needs in order to keep the project running smoothly and on track.
- Identify invalid, unacceptable or unusable data.
- Take corrective action if Quality Control inspections and testing indicate that construction is not meeting the Plans and Specifications.
- Assure all documentation is complete, accurate and up-to-date.
- Interact and cooperate with construction and QA personnel.

4.2.3 Quality Control Technicians

The QC Technicians shall be classified as follows:

- Field Technicians

- Laboratory Technicians

Quality Control Technicians may be qualified for and perform the duties required for field, laboratory or both upon approval of the QC Officer.

The QC Officer shall supervise or appoint a supervisor for each classification to provide scheduling, to verify equipment calibrations and to assure documentation of the field observations and laboratory tests. The number of technicians in each classification will depend on project needs as the work progresses. The Quality Control Technicians shall satisfactorily complete a training program or demonstrate knowledge of construction testing and receive on-the job training as required under the direction of the QC Officer.

4.2.4 Quality Control Activities

Quality Control activities are presented in Section 7 of the Quality Plan. A verification program will assure that the construction activities are inspected and documented in a logical organized manner so that any or all data and results are easily retrievable.

The Quality Control activities will be implemented with standardized Quality Control Procedures. These Quality Control Procedures include field sampling, testing, laboratory testing procedures, observation and monitoring procedures. The Quality Control Procedures are included in the Quality Plan.

4.3 Quality Assurance

4.3.1 General

The effectiveness of the QC program will be verified by the Quality Assurance Officer (hereinafter referred to as the QA Officer) by means of internal audits on the sampling and testing equipment, calculations, documentation and personnel qualifications.

The QA Officer shall review all areas of deficiency identified within the QC activities and the subsequent corrective actions taken. QA audit reports will be prepared by the QA Officer and submitted to the Design Engineer. These audit reports will be kept in the project files and made available for review.

4.3.2 Duties of the Quality Assurance Officer

The Quality Assurance Officer shall implement the Quality Assurance functions that include pre-qualification of QC personnel, verification of test procedures and results, equipment checks and review calculations, documentation and Compliance Reports. The QA Officer will be appointed by the Design Engineer. Responsibilities of the QA Officer will include the following:

- Be familiar with all documents, requirements, equipment and procedures relating to the project.
- Certify that the QC Officer is qualified to conduct the various test and monitoring procedures and observations.
- Review calculations and documentation of all Quality Control testing and determine reliability of data produced in terms of precision, accuracy, completeness, and comparability.
- Shall conduct thorough spot checks, re-tests, equipment checks and review of calculations and documentation. Verify that testing procedures, monitoring and observations are being performed correctly and accurately in accordance with the Specifications.
- Consult with QC Officer, Site Manager and Design Engineer to resolve any problems or deficiencies that arise.
- Prepare QA audit reports for review by the Design Engineer.

5.0 CHANGES AND CORRECTIVE ACTIONS

5.1 Scope

This section deals with methods or means of changes and corrective actions.

5.2 Authority of Personnel

The Site Manager, Design Engineer and/or the Quality Control Officer has the authority to reject material or work, to require removal or replacement, to specify and require appropriate actions if it is determined that the Quality Control/Quality Assurance, personnel, instructions, controls, test, records are not conforming to the Specifications.

5.3 Methodology

5.3.1 <u>Field and Design Changes</u>

Any changes in locations or alignments of construction features that do not alter design features or concepts shall be approved by the Design Engineer or his designated representative. These changes will require a Field Change Order (Form PR-21).

Should a change in design be necessary, (any change that alters or changes the intent of the Plans and Specifications) approval from the Design Engineer and Site Manager shall be required. These changes will be documented on a Design Change Order (Form PR-22).

All changes will be recorded in the Final Construction Report including the “as-built” drawings of the project.

5.3.2 Nonconformance and Corrective Actions

Nonconformances will be identified and verified by the QC Officer or his designee. The Construction Task or Segment shall stop work until specific corrective action is performed to alleviate the problem(s) that has evolved. The QA Officer or other qualified person can and may be contacted as needed to identify the importance of the nonconformance and issue the necessary corrective action to be taken if required.

The designated corrective action will be implemented before additional related work is permitted. The QC Officer will verify the corrective action appropriate by measurements, tests and/or other permanent documentation.

6.0 DOCUMENTATION

6.1 Scope

Documentation requirements shall include the following:

- Identify the person who has authority to provide for the submittal and/or storage of all survey, test and inspection reports.
- Shall provide a description of record keeping to document construction methods and results, surveys, sampling, testing and inspection of the project.

6.2 Document Control

Sampling, test inspections and construction records shall be maintained in the project files. A list of required reports are listed on Table 1.

A Construction Activity Report, recording quantities, thickness and locations of fill placed shall be maintained daily. Any significant events or conditions that affect placement or properties of the fill placed shall also be recorded on the daily Construction Activities Report. Each QC Technician shall complete a Construction Activities Report for each day’s work. Forms shall contain all pertinent and important events of that day relating to the construction project. The minimum data required on all forms and/or notebooks shall include the project number, date, technician’s signature and the signature of the QC Officer or his designee, indicating the work was reviewed and approved.

Table 2 lists titles of forms to be used for the Quality Control procedures. Examples of forms to be used during the construction project are attached to the appropriate Quality Control procedure. Similar forms may be substituted with approval from the QC Officer.

TABLE 1 - REQUIRED REPORTS

Report Type	Frequency	Originator	Approval
Construction Activities	Daily during construction	QC Technician	QC Officer
Field sampling and laboratory testing	Report for each respective test as required by the test procedure	QC Technician	QC Officer
Compliance Report	Upon Construction Segment Completion	QC Officer	Site Manager Design Engineer
Final Construction Report	After completion of the Construction project	Design Engineer	Regulatory Agency

TABLE 2 - LIST of FORMS

Form No.	Title
PR-1	Construction Activities Report
PR-2	Soil Sampling Log
PR-3	Gradation Analysis Worksheet
PR-4	Gradation Analysis with Hydrometer Worksheet
PR-5	Gradation Test Results
PR-6	Moisture Content Worksheet
PR-7	Atterberg Limits Worksheet
PR-8	Laboratory Compaction Test Worksheet
PR-9	Rock and Moisture Correction Calculations
PR-10	Moisture Density Relationship
PR-11	Summary of Laboratory Test Results
PR-12	Nuclear Density Test Data
PR-13	Field Density Tests (Sand Cone)
PR-14	Panel Placement Log
PR-15	Geomembrane Field Trial Log
PR-16	Geomembrane Seaming Record
PR-17	Geomembrane Seam Air Pressure Test Log
PR-18	Repair Log
PR-19	Geomembrane Seam Destructive Sample Log
PR-20	Compliance Report
PR-21	Field Change Order
PR-22	Design Change Order

7.0 CONSTRUCTION INSPECTION AND TESTING

7.1 General

This section describes the minimum engineering practices, testing, inspection and record keeping controls considered satisfactory for implementation of the Quality Control Plan. Acceptable construction shall be verified by means of visual examination, measurements and testing. The extent of the inspection and testing programs shall be sufficient to provide adequate quality control, to satisfy all requirements of the Plans and Specifications and to furnish necessary permanent records. It is also essential that all personnel performing the inspection and testing are qualified, defined by training and experience, to perform this professional job.

The QC Officer will be responsible for establishing and maintaining the inspection and testing program. He will also assure that the inspection and testing activities are properly documented and are conducted in accordance with the Plans and Specifications.

Construction activities involved during construction of the tailings impoundment and the attendant Compliance Reports for construction are as follows:

Construction Activity	Compliance Report
1. Earthwork - Excavation and Placement	PR-TP-CR1
2. Leak Detection/Leachate Removal System	PR-TP-CR2
3. Clay Soil Liner	PR-TP-CR3
4. Synthetic Liner System	PR-TP-CR4

7.2 Performance Standards for Earthwork Construction Activities

The following QC/QA program shall be implemented for all earthwork including: preparation of the foundation, excavation and placement of materials during any phase of construction (i.e. construction of embankments, backfilling trenches, finish grading). The minimum standards for Earthwork Construction are as follows:

- Clearing, grubbing and stripping of the area shall be accomplished prior to construction of the tailings cell. After removal of the organic materials, the area will be bladed with a motorgrader or equivalent piece of equipment, to create a relatively smooth surface, free of rocks and sharp angular edges.

- Prior to placing the first layer of fill on the foundation, a final inspection of the subgrade shall be performed to assure there are no cavities, separations, or irregularities. The QC Officer shall ensure the foundation has been prepared by leveling, moistening, and compaction so the surface materials of the foundation are stable and provide a satisfactory bonding surface with the first layer of fill to be placed.

- Assure that excavations are made to the lines, grades and dimensions shown on the Drawings. Documentation of any measurements and surveys shall be reviewed by the QC Officer.

- Placement of all fill materials shall be performed in accordance with the Specifications. Items including soil uniformity, lift thickness, compaction equipment, compactive effort and production of materials placed will be continuously observed and documented. Any soils placed with scrapers, trucks or equivalent pieces of equipment are not placed in lifts exceeding eight (8) inches prior to compaction. Distribution and gradations of each material shall be, as far as practicable, free of lenses, pockets, streaks or layers of materials differing substantially in texture, gradation or moisture content from surrounding materials or subsequent lifts. Fill soils placed beneath the synthetic liners and in areas immediately adjacent to the lined cells will be compacted to at least 95 percent of the Standard Proctor maximum density (ASTM D698) at a moisture content between plus four and minus two percent of the Optimum Moisture Content (ASTM D2216). Compaction can be obtained by tamping foot (sheepsfoot) roller or by splitting tracks with rubber-tired equipment or other approved methods. If the compacted surface of any layer of fill is too dry or smooth to bond properly with the layer of material to be placed thereon, it will be moistened and/or re-worked with a harrow, disk, scarifier or other suitable equipment to provide a relatively uniform moisture content and satisfactory bonding surface prior to placing the next layer of fill. If the compacted surface is too wet for proper compaction of the fill material to be placed thereon, it will be allowed to dry or be re-worked with a harrow, disk, scarifier or other suitable piece of equipment to reduce the moisture content to an allowable level. The re-conditioned layers/lifts shall all be re-compacted and re-tested to the specified requirements.

- No fill material shall be placed under adverse weather conditions, including freezing temperatures, or during or immediately after heavy precipitation events. Authorized personnel or the QC Officer shall determine when these adverse conditions exist.

7.2.1 Quality Control Procedures and Frequencies

Quality Control procedures to be utilized during construction are attached. A list of the tests and the procedures required for any Earthwork Excavation and Placement and the testing frequencies are presented below.

Procedure	Procedure No.
Field Inspection	QC-PR-1
Sampling of Soils and Aggregates	QC-PR-2
Particle Size Analysis	QC-PR-3
Moisture Content of Soils	QC-PR-4
Atterberg Limits	QC-PR-5
Soil Classification for Engineering Purposes	QC-PR-6

Laboratory Compaction Tests	QC-PR-7
In-place Density Tests	QC-PR-8
Compacted Soil Layer Thickness	QC-PR-9

- Field density and moisture tests shall be not less than one test for every 500 cubic yards of fill placed and in accordance with ASTM D1556, ASTM D2922, ASTM D3017, and/or ASTM D4643. There will be, at a minimum, a field density test and moisture test for each lift of material placed and for every full shift of compaction operations.

- During construction, one-point Proctor tests shall be taken at a frequency of one test for every five (5) field density tests to ensure that the correct laboratory Standard Proctor is being used.

- Gradations and Atterberg limits of compacted materials shall be performed at a frequency of not less than each 1,000 cubic yards of placed fill in accordance with ASTM D422, ASTM D2216, ASTM D4318, and/or ASTM D4643.

- The frequencies for laboratory Standard Proctor compaction tests will be such that maximum densities are determined for the entire range of materials being placed during construction, however, the frequency for compaction tests shall not be less than one test for each 5,000 cubic yards of compacted fill in accordance with ASTM D698 and/or ASTM D1557 as applicable.

- If the nuclear density gauge is used for field density and moisture content determination, a correlation test shall be taken for every ten (10) nuclear gauge tests. The Sand Cone method (ASTM D1556) shall be used for correlation for density determination and the Oven Drying method (ASTM D2216) for moisture content. Alternate methods may be used, such as, the Rubber Balloon method (ASTM D2167) for density correlation and the Microwave Oven method (ASTM D4643) for moisture content with approval by the QC Officer or Design Engineer. Density and moisture correlations shall be evaluated in accordance with the method as described in USBR 7230, Section 9.

7.3 Performance Standards for Installation of the Leak Detection/Leachate Removal System

The following QC/QA program shall be implemented for excavation and installation of each component for the Leak Detection/Leachate Removal System. Backfilling of the trenches/ditches will be monitored to be in accordance with Earthwork Construction quality procedures (Section 7.2). The minimum standards for installation of the Leak Detection/Leachate Removal System are as follows:

- Verify that materials to be utilized for installation satisfy the specified requirements. The QC Officer shall document on the proper form and transmittal sheets, acceptance of the materials or reasoning for non-acceptance.

- Ensure that excavations of the leak detection drains are made to the lines, grades, and dimensions shown on the Drawings. Documentation of any measurements and surveys shall be reviewed by the QC Officer prior to placement of pipe or drainage materials.

- Check that the installation of the drain pipe and sump are in conformance with the Specifications. Any pipe used for the system shall be joined together in accordance with the manufacturer's recommendation.

- Verify that the correct type of drainage material with the specified gradations is placed. The placed material should be clean and free of unsuitable material, placed in a manner that minimizes segregation and placed to the lines and grades as designated in the Specifications and on the Drawings.

- The bottom layer of the granular material drainage layers will consist of Entrada sand consistent with the properties described in Appendix B. The Entrada sand will be free of debris, excessive clay and silt fraction, and the maximum particle size is one inch. The material will be placed in a single lift of not less than four inches.

- The middle layer of the granular material drainage layers will consist of processed sand and gravel rocky soil. The gradation of the sand and gravel drainage layer will be similar to the Screened Rocky Soil through Double Screened Rocky Soil as presented in Figure B-2 of Appendix B. The maximum particle size is three inches, and a minimum of 50% by weight must be retained on a #8 U.S. Standard Sieve. No more than 15% by weight shall pass a #200 U.S. Standard sieve. The sand and gravel will be free of debris. The material will be placed in a single lift of not less than six inches, and the total thickness of the middle and lower layers of the granular drainage layers will not be less than 12 inches.

- The top layer of the granular material drainage layers will consist of Entrada sand consistent with the properties described in Appendix B. The Entrada sand will be free of debris, excessive clay and silt fraction, and the maximum particle size is one inch. The material will be placed in a single lift of not less than four inches and the total thickness of the three granular drainage layers will not be less than 18 inches.

- The leachate collection pipes will be installed in a configuration consistent with that described in Section 5.1.4.2 and Figure 5-8 of the Tailings Management Plan. The pipes will be installed in a clean gravel envelope that is wrapped in a geotextile that conforms with specifications in Section 7.7 of this appendix.

- The leachate collection pipes will have a minimum pipe stiffness of 50 psi as determined by methods described in ASTM D2412.

- The gravel envelope material for the leachate collection pipe will consist of 3/8 inch to 1 inch clean crushed rock or gravel. No more than 10% by weight of the gravel envelope may pass a #8 U.S. Standard Sieve.

- The granular drainage layers directly over and immediately adjacent to the leachate collection pipes will be compacted with a vibratory compactor or other approved method.

7.3.1 Quality Control Procedures

Quality Control Procedures to be utilized during construction are attached. A summary of the tests and the procedures required for installation of the Leak Detection/Leachate Removal System are listed below:

Procedure	Procedure No.
Field Inspection	QC-PR-1
Sampling Aggregate and Soils	QC-PR-2
Particle Size Analysis	QC-PR-4

Any backfilling of the trenches/ditches shall be inspected and tested in accordance with the Earthwork Construction procedures and frequencies.

7.4 Performance Standards and Specifications for Construction of Clay Liner

The following QC/QA program shall be implemented for excavation, conditioning, placement and compaction of the clay liner system. The minimum standards for construction of the Clay Liner are as follows:

- Ensure that final grading and preparation of the subgrade has been performed in accordance with the Specifications and to the lines and grades shown on the Drawings. The QC Officer shall review the documentation of any measurements and surveys prior to clay liner placement.

- A final inspection of the foundation is to be performed to assure that it has no deterioration due to frost action, erosion, rutting, areas of subsidence, or drying out of the surface. The inspection shall also verify that the foundation material has been moistened, but there is no standing water on the surface. Any unacceptable surface material will either be removed or re-compacted to the Specifications.

- Laboratory tests shall be conducted on the materials obtained from the borrow site to ensure the materials are within the limits specified in the Specifications. Clay soils used for construction of the clay liner shall classify as CL, CH, or SC by the

Unified Classification System and conform to the following physical requirements:

1. At least 30 percent passing the No. 200 sieve.

2. Maximum particle size of 1 inch.

3. Liquid limit of the material shall be at least 25 percent with a minimum plasticity index of 10 in accordance with ASTM D4318.

4. Maximum hydraulic conductivity of 1E-7 cm/sec when compacted to 95 percent of Standard Proctor maximum dry density within the specified moisture range as determined by ASTM D698 and ASTM D2216.

- As far as practicable, the soils will be brought to the proper moisture content prior to placement. Conditioning of the clay can be achieved by disking and adding water in a stockpile, processing with a "pug mill" or any other similar method approved by the QC Officer.

- Clay placement shall be performed in accordance with the Specifications. Items including soil uniformity, lift thickness, compaction equipment, compactive effort and production of materials placed shall be observed and documented. Lifts shall not exceed eight (8) inches prior to compaction. Distribution shall be, as far practicable, free of lenses, pockets, streaks or layers differing substantially in moisture content from subsequent lifts. The clay will be compacted to at least 95 percent of the Standard Proctor maximum dry density in accordance with ASTM D698, at a moisture content between minus two (-2) and plus four (4) of the optimum moisture content as determined by ASTM D2216 or ASTM D4643 (if approved by the QC Officer). Compaction can be obtained by tamping foot (sheepsfoot) rollers or equivalent types of equipment. After placing the clay, maintenance of the moisture content must be addressed at all times.

- Placement of the clay shall be accomplished in a manner to alleviate loss of moisture. Once the first lift has been placed over an area, and been compacted and tested, the subsequent lift should be placed directly over that area that has passed the compaction and moisture specifications. The entire clay liner system shall be constructed by alternating the first and final lifts in areas sufficient in size to minimize congestion between equipment placing and compacting the clay liner. This method or an approved alternate should be performed throughout the placement of the clay liner system. After the final lift has been placed, the clay shall be kept moist by application of water from a water truck or water wagon. Continuous visual monitoring of the placed clay shall be performed. Any areas that are suspected to have dried will be re-tested and a moisture content shall be obtained with either a nuclear density gauge (ASTM D3017) or a sample obtained for a laboratory test (ASTM D2216 or ASTM D4643). Documentation of any re-testing is mandatory. The Lining Contractor should be scheduled so that commencement of the synthetic liner installation begins as soon as possible after the clay liner has been constructed.

- No disking will be allowed on the first lift of placed clay. It will be necessary to remove the dried clay and re-condition it off of the floor or slope of the cell. Disking or scarifying the initial lift could allow mixing of the clay with the foundation materials altering the permeability coefficient of the clay materials. If any of the compacted lifts, other than the first, are too dry or smooth to bond properly with the next layer to be placed thereon, it will be moistened and/or re-worked with a harrow, disk, scarifier or other equivalent piece of equipment to provide a relatively uniform moisture and satisfactory bonding surface prior to placing the next layer of clay. If any of the compacted lifts, other than the first, are too wet for proper compaction of the clay to be placed thereon, it will be allowed to dry or be re-worked with a harrow, disk, scarifier or other piece of suitable piece of equipment to reduce the moisture content to an allowable level. That layer or lift will then be re-compacted and re-tested to the specified requirements. The final lift of clay shall be graded and compacted with a smooth-drum roller in order to prepare a smooth surface for the installation of the geomembrane liner.

- In areas where the existing clay liner will be preserved and used for the clay liner for the newly constructed cell, the upper one foot thickness of clay will conform to the same specifications as newly placed clay. Disking or scarifying and in-place moisture conditioning will be allowed provided there is no penetration of the clay or other mixing with unsuitable material.

- No clay shall be placed under adverse weather conditions, including freezing temperatures or immediately or during heavy precipitation events. Authorized personnel or the QC Officer shall determine when these adverse conditions exist.

7.4.1 Quality Control Procedures and Frequencies

The following discussion contains the Quality Control procedures to be utilized during construction of the clay liner. A list of the tests and procedures required during this phase of construction and the testing frequencies are presented below.

Procedure	Procedure No.
Field Inspection	QC-PR-1
Soil Sampling Log	QC-PR-2
Particle Size Analysis	QC-PR-3
Moisture Content of Soils	QC-PR-4
Atterberg Limits	QC-PR-5
Soil Classification for Engineering Purposes	QC-PR-6
Laboratory Compaction Tests	QC-PR-7
In-place Density Tests	QC-PR-8
Compacted Soil Layer Thickness	QC-PR-9

- Field density and moisture tests shall be not less than one test for every 500 cubic yards of clay placed and in accordance with ASTM D1556, ASTM D2922, ASTM D3017, and/or ASTM D4643. There will be at the minimum at least one field density test and moisture test for each lift of material placed and for every full shift of compaction operations.

- During construction, one-point Proctor tests shall be taken at a frequency of one test for every five (5) field density tests to ensure that the correct laboratory Standard Proctor is being used.

- Gradations and Atterberg limits of compacted materials shall be performed at a frequency of not less than each 1,000 cubic yards of placed fill in accordance with ASTM D422, ASTM D2216, ASTM D4318, and/or ASTM D4643.

- The frequencies for laboratory Standard Proctor compaction test will be such that maximum densities are determined for the entire range of materials being placed during construction, however, the frequency for compaction tests shall not be less than one test for each 5,000 cubic yards of compacted fill in accordance with ASTM D698 and/or ASTM D1557 as applicable.

- If the nuclear density gauge is used for field density and moisture content determination, a correlation test shall be taken for every ten (10) nuclear gauge tests. The Sand Cone method (ASTM D1556) shall be used for correlation for density determination and the Oven Drying method (ASTM D2216) for moisture content. Alternate methods may be used, such as, the Rubber Balloon method (ASTM D2167) for density correlation and the Microwave Oven method (ASTM D4643) for moisture content with approval of the QC Officer or Design Engineer. Density and moisture correlations shall be evaluated in accordance with the method as described in USBR 7230, Section 9.

For every 10,000 cubic yards of clay placed, clay liner composite samples of the placed clay shall be collected and tested for hydraulic conductivity. These samples shall re-molded and compacted to 95 percent of the Standard maximum dry density at a moisture content between minus 2 (-2) and plus four (4) as determined by ASTM D698 and ASTM D2216 respectively. Permeability testing will be by falling head permeameter (ASTM D5084) or other approved method.

7.5 Performance Standards and Specifications for Installation of the Synthetic Liner System

The following QC/QA program shall be implemented during installation of the synthetic liner system. The minimum standards are as follows:

- The Lining Contractor shall use adequate numbers of skilled workmen whom are thoroughly trained and experienced in the necessary skills and methods for placement of the liner system. At least one seaming operator "Master Welder", shall have a minimum of 10,000,000 square feet of geomembrane seaming experience using the same type of seaming apparatus to be utilized for the project. The "Master Welder" shall provide direct supervision, as required, over less experienced operators. No seaming operations will be permitted if the Contractor's quality control and supervisory personnel are not onsite to direct and/or observe production welding. Other seaming operators shall have seamed a minimum of 1,000,000 square feet of geomembrane. Apprentice seamers shall be qualified by completion of at least two successful geomembrane test seams performed under similar weather conditions and seaming procedures used for production seaming. These tests must be witnessed by the QC Officer or his representative.

- Prior to installation of the lining system, the Lining Contractor shall provide written approval verifying the subgrade has been properly prepared and is acceptable for lining installation. If any deficiencies are noted, arrangements to correct the deficiencies, to the satisfaction of the Lining Contractor shall be administered. The area on which the liner is to be placed shall be smooth and free of projections or depressions that may cause puncturing or stretching.

- The synthetic liner material shall be new, first quality product manufactured for the purpose of liquid containment. The materials shall be free of holes, blisters, undispersed raw materials or contamination by any foreign material. Geomembrane material shall be shipped and delivered in rolls free of seams. Delivery of the geomembrane must be made in the original wrappings indicating the name of the manufacturer, product identification, roll number, roll thickness, roll dimensions, resin type and date of manufacture. The Lining Contractor also shall submit proper certification from the manufacturer that all synthetic materials meet or exceed all the physical property criteria for the intended application. The QC Officer or his designee shall verify shipment of all materials and ensure Roll Numbers match the Invoice or Bill of Lading.

- Sand bags will be utilized to hold the liner in place during installation. On-site materials may be used to fill the bags as long as the materials are free of rocks or other sharp particles that could puncture the lining. The QC Officer shall ensure that there are adequate provisions on-site to protect the synthetic materials from wind displacement during installation.

- Anchor trenches shall be excavated just prior to installation of the liner system. The anchor trenches shall be excavated to the lines and grades shown on the Drawings or as modified by the QC Officer in the field. Backfilling of the anchor trenches shall not be allowed until the liner has been through several expansion/contraction cycles. The Lining Contractor shall be responsible for securing the lining system in the anchor trench with an adequate number of sandbags or other approved method by the QC Officer until the anchor trench can be backfilled. Rounded edges shall be provided in the anchor trenches where the geomembrane enters into the anchor trench to provide subgrade support and to avoid sharp bends in the geomembrane. The geomembrane shall be seamed completely to the ends of the panels to minimize the potential of tearing along the seams.

- Prior to installation, the Lining Contractor shall provide the QC Officer with a panel layout indicating the general panel configuration intended. Panels shall be oriented perpendicular to the line of the slope crest (i.e. down and not across the slope).

- The method and equipment used to deploy the liner shall not damage the material to be installed, the already installed materials or the subgrade in any way. Geomembrane shall be unrolled using methods that will not damage, stretch, or crimp the geomembrane and protect the underlying subsurface from damage. Personnel walking on the liner shall not engage in activities or wear any types of shoes that could damage the liner. Vehicular traffic such as cars, truck, ATV's, etc. directly on the liner will not be permitted. Equipment shall not damage the geomembrane by handling, trafficking, leaking of any hydrocarbons or any other means. The geomembrane shall not be utilized as a work or storage area. If needed, a protective cover may be spread out as a work or storage area on the liner. <u>Smoking is strictly prohibited when on the liner</u>.

- The primary and secondary liners shall consist of a geomembrane of High Density Polyethylene (HDPE) with a typical thickness of 60 mils. The geomembrane manufacturer shall be listed by the National Sanitation Foundation as having met Standard 54 requirements for flexible liners. Resin used to manufacture the geomembrane shall be formulated to be resistant to chemical and ultraviolet degradation. The geomembrane material shall be free of any plasticizers or other leachable additives. Material properties for the geomembranes are presented in Table 3.

- Double wedge fusion welding (hot shoe) will be the primary means of welding. Seaming methods other than the method specified above will require prior approval by the QC Officer. The acceptance or rejection shall be based on data submitted by the Lining Contractor and shall include recommendations from the manufacturer, case history and laboratory testing. Double wedge fusion welding shall be performed in accordance with these Specifications and the manufacturer’s recommendations. The two sheets of geomembrane to be joined together, shall be properly positioned so that a minimum overlap of 4 inches and a maximum of 6 inches exist. “Fishmouths” or wrinkles at seam overlaps shall be cut to achieve a flat overlap. The cut “fishmouths” or wrinkles shall be either extrusion welded if the cut is less than 3 inches in length or patched with a cap if the area cut is longer than 3 inches. If a sudden change in temperature should occur, readjustment of the panel to the acceptable overlap limits must be accomplished. The exact width of overlap is dependent on the width of the wedge element being used. All cutting and preparation of odd shaped sections or small fitted areas must be completed at least 50 feet ahead of the seaming operation in order to allow the seaming operation to proceed with as few interruptions as possible. Overlapped sheets ready for seaming must be completely free of moisture and dirt in the area of the seam. No seaming shall be allowed during rain or snow unless proper precautions are made to allow seaming on dry materials within an enclosure or shelter. Ambient temperatures shall between 32° F (0° C) and rising up to 104° F (40° C) when measured two feet above the surface of the liner. Seaming will not be allowed on frozen or saturated subgrade without taking proper corrective actions approved by the QC Officer.

- Extrusion welding will be used only for repairs and detail work such as around pipes and sumps. All extrusion fillet seams shall be in accordance with the Specifications and the manufacturer’s recommendations. Prior to extrusion welding, all surfaces shall be clean and dry. A hot air device or hot air wedge (Lyster) shall be used to “tack” the two sheets together. This tacking procedure is not intended to be the primary seam but, simply creates a light bond between the two sheets, securing their position. Grind marks should not be deeper than approximately 5 percent of the geomembrane thickness. The main purpose for grinding is the removal of oxide layers and dirt from the liner surfaces and to roughen the interface for the extrudate. Grinding marks shall not extend beyond 1/4 inch of either side of the extrudate after its placement. Any grinding marks appearing more than 1/4 inch beyond the extrudate will require repair by placement of a cap over the entire seam or patch where the excessive grinding occurred. Seaming must take place no more than 10 minutes after grinding to ensure the surface oxide layers do not reappear to the area prepared for the extrudate. The welding rod shall be made from the same resin and free from dirt, dust, moisture and tangles at all times. The extrusion welder’s barrel shall be purged of heat-degraded extrudate for approximately 30 seconds before beginning to seam. This must be done every time the extruder is restarted after two or more minutes of inactivity. The purged extrudate shall be disposed of properly, not on the surface of placed liner or on the subgrade, where it could damage the liner in any way. The bottom portion of the welding die must stay in contact with the sheet surface and conform to the various seam angles and configurations. The placed extrudate should be approximately twice the specified sheet thickness, measured from the top of the bottom sheet to the top or “crown” of the extrudate. Excessive squeeze-out is acceptable, only if it is equal on both sides and does interfere with subsequent vacuum box testing. However, if the extrudate can be pulled off of the seam by the squeeze-out, the weld is considered unacceptable. If the seaming process is interrupted during mid-seaming, the extrudate should trail off gradually and not in a large mass of solidified extrudate. Where such welds are abandoned long enough to cool, a new patch strip shall be placed over the entire existing patch. No extrusion welds will be permitted over the top of another extrusion weld or side-by-side of another weld. The only cases that extrudate will be allowed over the top of another weld is for "T" or "Y" shaped seams after the existing weld has been ground. In the event an extrusion weld cannot be tested with a vacuum box, provisions must be provided for the seam to be spark tested according to the spark tester manufacturer's procedures.

TABLE 3 - Material Properties for HDPE Geomembrane

Property	Test Method	Minimum Requirement
Thickness (mils minimum ± 10%)	ASTM D 5199	60
Specific Gravity (g/cc minimum)	ASTM D 1505/D 792	0.94
Carbon Black Content (%)	ASTM D 1603	2 - 3
Carbon Black Dispersion	ASTM D 5596	Note 1
Minimum Tensile Strength (each direction) 1. Tensile strength yield (lb/in. width) 2. Tensile strength break (lb/in. width) 3. Elongation at yield (%) 4. Elongation at break (%)	ASTM D 6693	 126 228 12 700
Tear Strength (lb.)	ASTM D 1004	42
Puncture Resistance	FTMS 101 - 2065 ASTM D 4833	80 108
Stress Crack Resistance[2] (hrs)	ASTM D 5397	300
Oxidative Induction Time (OIT) (minutes)	ASTM D 3895	100
Oven Aging at 85 ºC Standard OIT- % retained after 90 days	ASTM D 5721 ASTM D 3895	55
UV Resistance[3] High Pressure OIT[4]- % retained after 1600 hrs	GRI GM11 ASTM D 5885	50

(1) Carbon black dispersion for 10 different views: Nine in Categories 1 and 2 with one allowed in Category 3.

(2) The yield stress used to calculate the applied load for the SP-NCTL test should be the mean value via MQC testing.

(3) The condition of the test should be 20 hr. UV cycle at 75 ºC followed by 4 hr. condensation at 60 ºC.

(4) UV resistance is based on percent retained value regardless of original HP-OIT value.

7.5.1 Quality Control Procedures and Frequencies

Quality Control of the geomembrane liner placement shall be furnished by the Lining Contractor. PRL shall monitor and maintain that liner deployment is in accordance with the Specifications through its Quality Assurance Program. A list of the tests and procedures required during this phase of construction and the testing frequencies are presented below.

Procedure	Procedure No.
Field Inspection	QC-PR-1
HDPE Liner Seam Integrity	QC-PR-10

- The Lining Contractor shall qualify each seaming apparatus (double wedge fusion and/or extrusion welder) and operator at the start of each day or shift of seaming, and at least once every 4 hours thereafter. A representative seam fabricated from the same sheet material and using the same seaming procedure to be utilized for production welding shall be submitted to the QC Officer or his representative. The start-up seam shall be a minimum of 12 inches wide by 10 feet in length with the seam being centered lengthwise within the strip. Five specimens shall be obtained from each end of the strip. A tensiometer will be utilized to test five of the specimens for shear and five specimens for peel. Shear and peel tests shall result in Film Tearing Bond (FTB) as defined by NSF Standard 54, which is a failure in ductile mode of one of the bonded sheets by tearing prior to complete separation in the bonded area. Should any seam fail to meet the Specifications, the seaming device and/or seamer shall not be accepted and will not be used for any seaming until the deficiencies are corrected and two successful start-up seams have been accepted. The Lining Contractor's quality control officer/technician shall initial each test seam submitted, indicating the start-up seam has been inspected and tested for peel and shear. Every submitted test seam will marked with the time, date, operator's initials, welding machine number and welding temperature and speed. Minimum values for shear and peel tests are presented in Table 4.

Table 4 - Field Seam Requirements

Property	Test Method	Minimum Requirement
Shear Strength (lb/in. width)	ASTM D4437 (1)	120
Peel Strength (lb/in. width)	ASTM D4437 (1)	78 (2)

(1) As modified in Annex A, NSF 54
(2) Minimum recorded stress required in conjunction with Film Tear Bond (FTB) for acceptance

- Daily visual inspection of the seaming and testing process shall be performed by the QC Officer or his representative. All testing procedures shall be periodically monitored to ensure proper procedures are adhered to. If the QC Officer or his designee witnesses a vacuum test or air pressure test, they will initial, date and check that the information that was written in reference to the test results is correct.

- All seams created by the double wedge fusion weld shall be checked by the Air Pressure Testing method in the following manner:

  1. Seal one end of the seam to be tested.
  2. Insert a needle or other approved device to supply pressure through one end of the sealed channel end created by the double wedge fusion weld.
  3. Apply pressure to the device to ensure unobstructed passage of air through the channel.
  4. Seal off the opposite end of the channel.
  5. Insert a pressure between 25 and 30 psi, and allow 2 minutes for the injected air to come to an equilibrium in the channel. The channel shall sustain pressure for 5 minutes.
  6. At the end with the pressure gauge, write down the date, time test started, time test ended, air pressure reading at the beginning of the test and air pressure reading after the minimum 5 minute time period, whether the test failed or passed and the initials of the inspector.
  7. If the pressure loss exceeds 2 psi, or if the pressure never stabilizes, the defective area must be located and repaired with a cap.
  8. If the test passes after 5 minutes, the seal shall be removed from the opposite end of the pressure gauge. The air channel should deflate immediately indicating the entire length of the seam was tested.
  9. All repair welds and welds to seal the air insert holes will be tested by the vacuum box as described below.

- All extrusion welds shall be tested with a vacuum box. The vacuum box assembly shall consist of a rigid housing with a transparent viewing window on the top, a soft rubber gasket fixed to the bottom, valve assembly and a vacuum gauge. The testing procedure shall be as follows:

  1. Wet a strip of the extrusion weld approximately 12 inches wide by the length of the box with a soapy solution.
  2. Place the box over the wetted surface and compress.
  3. Create a vacuum of 3 to 5 psi.
  4. Make sure the seal between the box and the geomembrane is tight.
  5. Examine the geomembrane for about 15 seconds looking for animated bubbles or bubbles that increase in size while under pressure.
  6. If no animated bubbles appear: close the vacuum valve, open the bleed valve, and move the box over the next adjoining weld to be tested with a minimum of 3 inches of overlap before repeating the process.
  7. After completing the test on the extrusion welds, the inspector shall write on the liner the date, time, whether the test passed or failed and initials.

- If an extrusion weld can not be tested by the vacuum box method, the seams shall be spark tested according to the spark tester manufacturer's specifications and procedures.

- Destructive seam testing shall be minimized to help preserve the integrity of the liner. The Lining Contractor shall provide the QC Officer or his representative with a destructive test sample for approximately every 500 feet of production. As far as practical, these samples shall be cut above the proposed high water level of the cell or on the flat surface of the cell bottom. All samples will be a minimum of 12 inches wide by 36 inches long with the seam centered lengthwise. The sample will then be divided into three equal pieces, one to be tested by the Lining Contractor and two to be given to the QC Officer or his designee. The Contractor shall test ten, 1 inch wide specimens, five for shear strength and five for peel strength in accordance with Table 4. <u>Seam failure is defined as failure of any one of these specimens by shear or peel.</u> <u>For peel adhesion, the minimum strength value must be obtained in combination with Film Tear Bond (FTB) for acceptance.</u> <u>For shear strength, the geomembrane specimens must exhibit at least 50 percent elongation prior to failure.</u> The location, seam number, seaming apparatus number, operator, date and time of each cut-out shall be recorded on the each segment of the 36 inch specimen. All holes resulting from the destructive testing shall be patched as soon as possible and tested.

- In the event of a laboratory test failure for a field seams, additional destructive seam testing will be conducted by taking seam samples adjacent to and on both sides of the failed sample location. If the additional destructive seam tests result in failure, the entire seam represented by the samples will be repaired with a cap strip. The cap strip will be vacuum box tested.

7.6 Performance Standards and Specifications for Installation of Geonet System

The following QC/QA program shall be implemented during installation of the geonet system. The minimum standards are as follows:

- Only after the bottom (secondary) liner has been deployed, seamed, tested and approved by the QC Officer or his representative, shall deployment of the geonet commence. The Lining Contractor shall present all test results, as-built drawings and repair logs of the secondary liner for approval.

- The geonet shall be NSC, POLY-NET 2000 or an approved equal. The geonet shall conform with the minimum values and tolerances as listed in Table 5.

Table 5 - Material Properties for Geonet

Property	Test Method	Qualifier	Value
Resin Density (g/cm^3)	ASTM D 1505	minimum	0.94
Carbon Black Content (%)	ASTM D 1603	minimum	2.0
Thickness (inches)	ASTM D 1777	minimum	.160
Mass per Unit Area (lbs/ft^2)	ASTM D 3776	minimum	.117
Transmissivity[1] (m^2/sec)	ASTM D 4716	minimum	1×10^{-3}
Standard Width X Length (feet)	----	----	14 X 300

(1) Hydraulic gradient i = 1. Normal pressure = 10,000 psf.

- The geonet drainage material shall be manufactured by extruding two sets of polyethylene strands to form a dimensional structure allowing planar flow. All geonet materials shall be manufactured of new first quality products. The QC Officer or his designee shall ensure that delivery is made in the original wrappings showing the name of the manufacturer, product identification, lot number and roll dimensions.

- During deployment of the geonet, the Contractor shall at all times keep the geonet clean and free from debris prior to and during installation. Storage of the geonet shall be in accordance with the manufacturer's recommendations and in a location that will keep the material from damage. Installed geonet that is permitted to become filled with accumulations of debris or blowing dirt and sand shall be removed, cleaned and reinstalled following cleanup of the geomembrane secondary liner's surface.

- The geonet rolls shall be overlapped at least 4 inches and secured together by plastic ties no more than 5 feet apart. Plastic ties shall be white or any other bright color for ease of inspection. Metallic ties such as wire will not be permitted.

7.7 Performance Standards for Installation of Geotextile Materials

The following QC/QA program shall be implemented during installation of any geotextile materials to be placed. The minimum standards are as follows:

- Geotextile fabric shall be nonwoven fabric with a minimum fabric weight of 8 oz/yd^2 like AMOCO 4508 or approved equal. Material properties of the nonwoven geotextile shall conform to the minimum values and tolerances presented in Table 6.

Table 6 - Material Properties for Nonwoven Geotextile

Property	Test Method	Minimum Values	Typical Physical Properties
Unit Weight (oz/yd^2)	ASTM D 3776	8.0	----
Grab Tensile (lbs)	ASTM D 4632	200	270-275
Grab Elongation (%)	ASTM D 4632	50	65
Mullen Burst (psi)	ASTM D 3786	450	575
Puncture (lbs.)	ASTM D 4833	130	170
Trapezoid Tear (lbs)	ASTM D 4533	80	120-140
Apparent Open Size (US Sieve No.)	ASTM D 4751	100	100-200
Permittivity (sec^{-1})	ASTM D 4491	1.5	----
Permeability (cm/sec)	ASTM D 4491	.2	.27
Thickness (mils)	ASTM D 1777	90	115
U.V. Resistance[1] (%)	ASTM D 4355	70	---

(1) Percent of minimum grab tensile strength after 500 hours.

- Delivery of geotextile fabric shall be made in original wrappings showing the name of the manufacturer and product weight. Storage of the geotextile material must be in accordance with the manufacturer's recommendations and in a location that will keep the fabric clean and protected from damage.

- The geotextile fabric shall be placed in the Leak Detection system in accordance with the Specifications and Drawings. During installation, the fabric will be rejected if it has any defects, rips, holes, flaws, deterioration or damage incurred during manufacture, transportation and/or storage.

- The area on which the fabric is placed shall be smooth and free of projections or depressions that may cause the puncturing or stretching of the fabric. Care shall be taken to remove all sharp rocks, stones and any other sharp objects. Geotextile fabric shall be placed without stretching and shall lie smoothly in contact with the prepared surface. The adjacent ends of the fabric shall be placed with seams overlapped four to six inches. The geotextile fabric seam shall be overlapped according to Plans and Specifications for geotextile-wrapped gravel envelope.

7.8 Quality Control Reports

Test reports, resin batch test results, material properties and manufacturer's quality control as required by these Specifications shall be submitted by the Lining Contractor to the QC Officer for review prior to installation of any of the synthetic lining system.

7.8.1 <u>Field Installation Reports</u>

The Contractor shall submit to the QC Officer daily reports documenting the following:

- Changes in layout and Drawings (panel placement).

- Production data, indicating materials placed and seams welded along with batch and roll numbers.

- Non-destructive test results.

- Destructive test results.

- Areas of deficiency and corrective actions taken.

7.8.2 <u>As-built Drawings</u>

Upon completion of the project, the Contractor shall provide a reproducible original of the "as-built" drawings illustrating panel location, seam location, seam numbers, repair locations and the locations of destructive test samples with corresponding test sample numbers.

8.0 GEOMEMBRANE REPAIR PROCEDURE

Any portion of the geomembrane exhibiting a flaw or failing destructive or non-destructive quality control test must be repaired. The repair of any of these faults shall be in accordance with these Specifications and the manufacturer's recommendations. All repair procedures, materials and techniques shall be approved in advance of the specific repair by the QC Officer.

9.0 GEOMEMBRANE WARRANTY

The Lining Contractor shall guarantee the synthetic lining system and geomembrane to be free of defects for a period of 20 years after installation. These warranties shall be provided to the Owner upon completion of the project.

10.0 ACCEPTANCE

Acceptance of the lining system will be accepted by PRL when:

1. The installation has been completed in accordance with the Plans and Specifications and to the satisfaction of the QC Officer and Design Engineer.
2. All quality control documentation has been submitted.
3. As- built drawings have been completed and submitted to the QC Officer.
4. Warranties have been received by PRL.

QUALITY CONTROL PROCEDURE

FOR

FIELD INSPECTIONS
QC-PR-1

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

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1.0 METHODOLGY

1.1 Scope

The procedure for field inspections is to be used to monitor construction activities during construction by visual observation and measurement and to record and compare these observations and measurements with the Specifications.

1.2 Procedure

The field inspection activities set forth in the Quality Plan shall be documented for earthwork, construction materials, surveys and sampling. Observations shall be recorded on Form PR-1 or an approved alternate. Items to be documented include, but are not limited to, locations, dimensions, quantities, slopes or grades of excavation and placement. Areas to receive compacted fill shall be observed and the condition of the surfaces prior to fill placement shall be noted. During placement, lift thickness, lift uniformity, compactive effort and other construction details shall be monitored in accordance with the appropriate Specification. Construction materials, surveys, and sampling shall also be observed and documented to verify compliance with the applicable Specifications.

1.3 Frequency of Observations

Observations of fill placement shall be conducted on-going during any phase of the construction process according to the Specifications.

1.4 References

1. *Annual Book of ASTM Standards,* Volumes 04.08 and 04.09

2. *Earth Manual - A Water Resources Technical Publication (Third Edition),* Part 2, 1990, U.S. Department of Interior

2.0 REPORTING

2.1 Forms

The following form or approved equivalent shall be used to record observations of all construction activities.

- PR-1 Construction Activities Report

2.2 Records

The original of the construction activities report shall be maintained in a Project File. Copies shall be available upon request.

PR-1 CONSTRUCTION ACTIVITIES REPORT

Technician: ______________________________

Approved By: ______________________________

Project No.:______________________________

Date: ______________________________

Daily Report No.:________________ Sheet________of__________

Weather Conditions and Temperature: ______________________________

Equipment: ______________________________

____

Construction Activities and Observations: ______________________________

QUALITY CONTROL PROCEDURE

FOR

SAMPLING of SOILS and AGGREGATES
QC-PR-2

BY

PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

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1.0 METHODOLGY

1.1 Scope

Determine a procedure to provide standard sampling procedures for obtaining samples of soils, aggregates and/or soil-aggregate mixtures from stockpiles, truck loads, borrow areas and at the construction site. This procedure shall include a visual-manual method for describing and identifying the different sample types.

1.2 Procedure

All soil, aggregate, soil-aggregate sampling shall be done in accordance with standardized procedures as described in the latest version of ASTM D75. Description and identification of soils using visual-manual methods shall be done in accordance with standard procedure described in the most recent version of ASTM D2488.

1.3 References

1. *Annual Book of ASTM Standards*, Volume 04.03 and Volume 04.08

2.0 REPORTING

2.1 Forms

The following form or approved equivalent shall be used for all sampling activities associated with this procedure.

- PR-2 Soil Sampling Log

2.2 Records

The original of the sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-2 SOIL SAMPLING LOG

Sample No. ______________________________ **Report No.** ______________________________

Date ____________________________________ **Sheet** _______ **of** ______________

Sampled By _____________________________ **Reviewed By** _____________________________
QC Officer

Location (Stockpile, Test Pit, Fill, Borrow Area, Truck, etc.) ______________________________

Depth of Sample ______________________________

Sample Type (Large bulk, Undisturbed, Grab, Composite, etc.) ______________________________

Visual Classification (Color, Grain size, Texture, etc.) ______________________________

Intended Use (Fill material, Clay Liner, etc.) ______________________________

Testing Program (Standard Density, Atterberg, etc.) ______________________________

Note - A copy of this form must be attached with all laboratory tests performed on the sample.

QUALITY CONTROL PROCEDURE

FOR

PARTICLE SIZE ANALYSIS
QC-PR-3

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

<u>INDEX</u>

1.0 METHODOLGY

1.1 Scope

These procedures are to be used to quantitatively determine the distribution of particle sizes of soils, aggregates and soil-aggregate mixtures. The distribution of particle sizes larger than a No. 200 sieve are determined by screening and particle sizes smaller than a No. 200 sieve are to be determined by hydrometer analysis.

1.2 Procedure

Preparation of soil samples to be analyzed for particle size shall be in accordance with the most current version of ASTM D421. For particle sizes greater than the No. 200 sieve procedures from the most current version of ASTM D422 shall be adhered to. The latest version of ASTM D1140 shall be used to analyze for particle sizes smaller than the No. 200 sieve.

1.3 References

1. *Annual Book of ASTM Standards*, Volume 04.08

2.0 REPORTING

2.1 Forms

The following forms or approved equivalents shall be used for all sampling activities associated with this procedure.

- PR-3 Gradation Analysis Worksheet
- PR-4 Gradation Analysis with Hydrometer Worksheet
- PR-5 Gradation Test Results
- PR-11 Summary of Laboratory Test Results

2.2 Records

The original of the sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-3 GRADATION ANALYSIS WORKSHEET

Technician____________________________________ Project No. _______________________________

Approved By __________________________________ Date ___________________________________

Sample No.		Sample No.		Sample No.		Sample No.		Sample No.
Description		Description		Description		Description		Description
Run by		Run by		Run by		Run by		Run by

Dish No.		Dish No.		Dish No.		Dish No.		Dish No.
Dry Soil & Dish		Dry Soil & Dish		Dry Soil & Dish		Dry Soil & Dish		Dry Soil & Dish
+200 Soil & Dish		+200 Soil & Dish		+200 Soil & Dish		+200 Soil & Dish		+200 Soil & Dish
Dish Weight		Dish Weight		Dish Weight		Dish Weight		Dish Weight
Dry Soil Weight		Dry Soil Weight		Dry Soil Weight		Dry Soil Weight		Dry Soil Weight

Sieve Size		Cum. Wt. Ret.	% Pass		Sieve Size		Cum. Wt. Ret.	% Pass		Sieve Size		Cum. Wt. Ret.	% Pass		Sieve Size		Cum. Wt. Ret.	% Pass		Sieve Size		Cum. Wt. Ret.	% Pass
5					5					5					5					5			
3					3					3					3					3			
1 1/2					1 1/2					1 1/2					1 1/2					1 1/2			
3/4					3/4					3/4					3/4					3/4			
3/8					3/8					3/8					3/8					3/8			
#4					#4					#4					#4					#4			
#8					#8					#8					#8					#8			
#16					#16					#16					#16					#16			
#30					#30					#30					#30					#30			
#50					#50					#50					#50					#50			
#100					#100					#100					#100					#100			
#200					#200					#200					#200					#200			
Pan					Pan					Pan					Pan					Pan			

% Gravel		% Gravel		% Gravel		% Gravel		% Gravel
% Sand		% Sand		% Sand		% Sand		% Sand
% Silt & Clay		% Silt & Clay		% Silt & Clay		% Silt & Clay		% Silt & Clay
Remarks:		Remarks:		Remarks:		Remarks:		Remarks:

PR-4 GRADATION ANALYSIS with HYDROMETER WORKSHEET

Technician ___________________________________ Project No. ______________________
Approved By __________________________________ Date ____________________________
Sample No. ________________ Visual Description ____

Ran by________________________________ **Sample Preparation** Sieve Time _____________________

Sieve Size		3”	1 1/2”	3/4”	3/8”	No. 4	Sample Weights		
Sample No and Pan No.								Wet	Dry
Weight of Pan							Total Sample	___________	___________
Dry Weight Retained									
Dry Weight Passing		XXX	XXX	XXX	XXX	XXX	Retained on No.4	___________	___________
%of Total Passing									
					w % =	_____	Passing No. 4	___________	___________

Ran By ________________________________ **Sieve & Hydrometer Analysis** Sieve Time ______________

Factor = $\frac{w\%}{w}$ = ___________ = __________

Sieve No.	Weight Retained	Weight Passing	% of Total Passing	MOISTURE DETERMINATION				
8 (10)		XXXXX						
16		XXXXX		Dish No.	+4 Material	-4 Material	Hygro. Moisture	Hydro. Sample
30 (40)		XXXXX		Wt. Wet Soil & Dish				
50		XXXXX		Wt. Dry Soil & Dish				
100		XXXXX		Wt. Dish				
200				Wt. of Dry Soil				
PAN			XXXXXX	Wt. of Water		_____ = w		
Total			XXXXXX	% Moisture				

Ran By ________________________________ **Hydrometer Analysis** Sieve Time _____________

Cylinder No. ______________ Specific Gravity ________________ Dispersing Agent _____________________

Dish No. ________________ Date ________________ Amount _______________ml Date Calibrated ___________

Clock Time	Test Time	Temp. °C	Hyd. Read	Hyd.* Corr.	Corr. Read		% of Total Passing	Particle Diameter	
	Start Mix	XXXXX	XXXXX	XXXXX	XXXXX		XXXXX	XXXXX	
	Stop Mix	XXXXX	XXXXX	XXXXX	XXXXX		XXXXX	XXXXX	
	0.5 Min.								
	1.0 Min.								
	4.0 Min.								
	19 Min.								
	60 Min.								
	7 Hr 15 Min.								
	25 Hr 45 Min.								

Gravel ___________ % Sand __________ % Silt-Clay ___________ % Storage Location ___________________

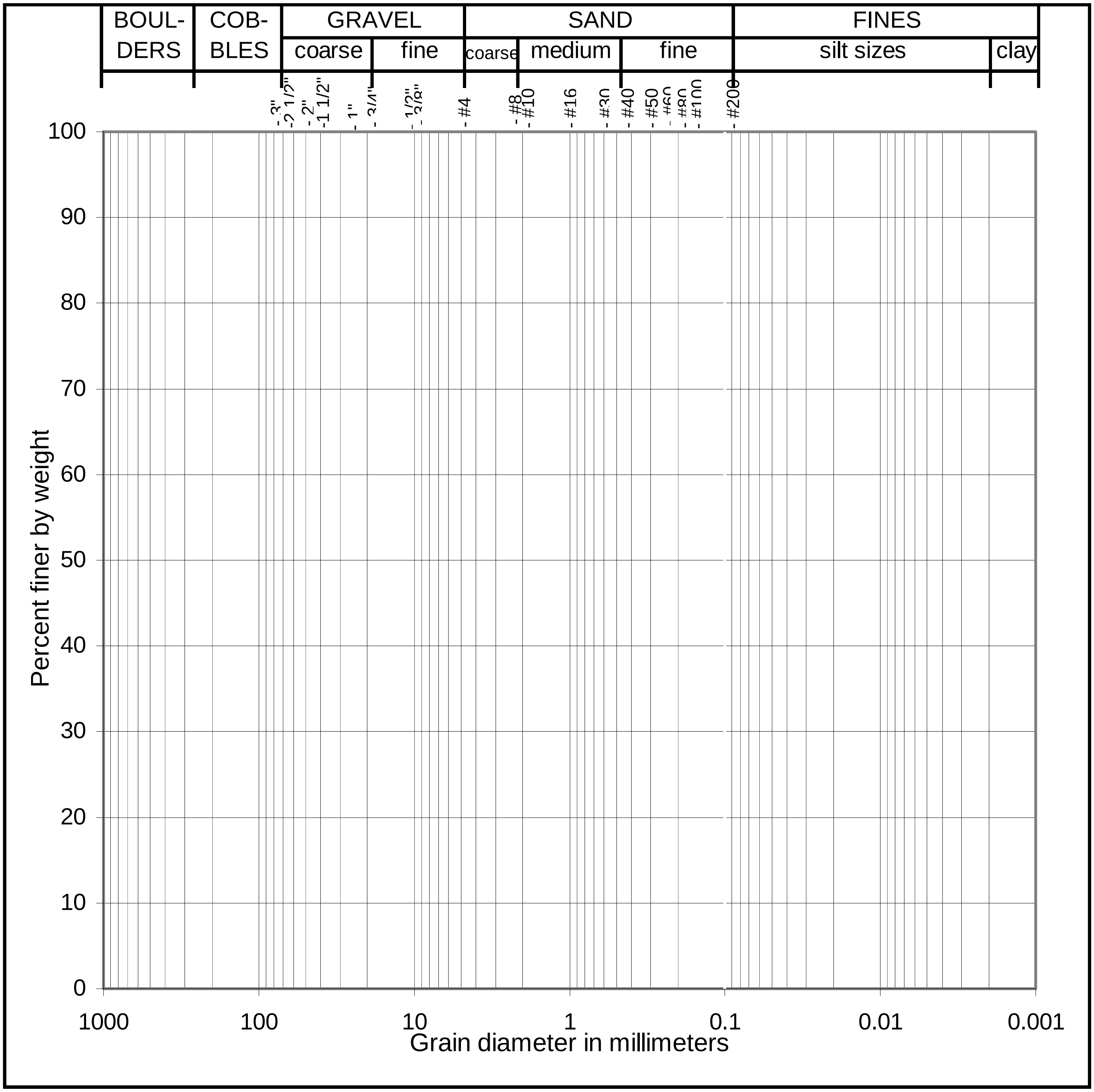
* Correction includes temperature, meniscus and de-flocculent

PR-5 GRADATION TEST RESULTS

Technician ______________________ Project No. ______________________

Sample Id. ______________________ Date Screened ______________________

Approved By ______________________ Date ______________________



Remarks ____

____

____

PR-11 SUMMARY of LABORATORY TEST RESULTS

Technician____ Project No. ____________________________________

Approved By ______________________________________ Date ____

Material Tested ____

Sample Number	Moisture Content %	Dry Density pcf	**Gradation**		% Passing No. 200 Sieve	**Atterberg Limits**		Soil Type	Comments
			Gravel %	Sand %		Liquid Limit	Plasticity Index		

QUALITY CONTROL PROCEDURE

FOR

LABORATORY MOISTURE CONTENT of SOILS
QC-PR-4

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

INDEX

1.0 METHODOLGY

1.1 Scope

These procedures are to be used to determine the laboratory moisture content in soils, aggregates and soil-aggregate mixtures.

1.2 Procedure

The moisture contents shall be in accordance with the procedures described in the most recent version of ASTM D2216. ASTM D4643 may be utilized after a correlation factor has been established between the two methods.

1.3 References

1. *Annual Book of ASTM Standards,* Volume 04.08

2.0 REPORTING

2.1 Forms

The following forms or approved equivalents shall be used for all sampling activities associated with this procedure.

- PR-6 Moisture and Density Worksheet
- PR-11 Summary of Laboratory Test Results

2.2 Records

The original of the sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-6 MOISTURE CONTENT WORKSHEET

Technician____ Project No. ____________
Approved By ______________________________________ Date _________________

ASTM D2216 ☐	ASTM D4643 ☐
Sample No.	
Dish No.	
Wt. of Dish & Wet Soil	
Wt. of Dish & Dry Soil	
Wt. of Dish	
Wt. of Water	
Wt. of Dry Soil	
% Moisture	

ASTM D2216 ☐	ASTM D4643 ☐
Sample No.	
Dish No.	
Wt. of Dish & Wet Soil	
Wt. of Dish & Dry Soil	
Wt. of Dish	
Wt. of Water	
Wt. of Dry Soil	
% Moisture	

ASTM D2216 ☐	ASTM D4643 ☐
Sample No.	
Dish No.	
Wt. of Dish & Wet Soil	
Wt. of Dish & Dry Soil	
Wt. of Dish	
Wt. of Water	
Wt. of Dry Soil	
% Moisture	

ASTM D2216 ☐	ASTM D4643 ☐
Sample No.	
Dish No.	
Wt. of Dish & Wet Soil	
Wt. of Dish & Dry Soil	
Wt. of Dish	
Wt. of Water	
Wt. of Dry Soil	
% Moisture	

ASTM D2216 ☐	ASTM D4643 ☐
Sample No.	
Dish No.	
Wt. of Dish & Wet Soil	
Wt. of Dish & Dry Soil	
Wt. of Dish	
Wt. of Water	
Wt. of Dry Soil	
% Moisture	

ASTM D2216 ☐	ASTM D4643 ☐
Sample No.	
Dish No.	
Wt. of Dish & Wet Soil	
Wt. of Dish & Dry Soil	
Wt. of Dish	
Wt. of Water	
Wt. of Dry Soil	
% Moisture	

Remarks ____

PR-11 SUMMARY of LABORATORY TEST RESULTS

Technician______________________________ Project No. ______________________________

Approved By ______________________________ Date ______________________________

Material Tested ______________________________

Sample Number	Moisture Content %	Dry Density pcf	Gradation		% Passing No. 200 Sieve	Atterberg Limits		Soil Type	Comments
			Gravel %	Sand %		Liquid Limit	Plasticity Index		

QUALITY CONTROL PROCEDURE

FOR

ATTERBERG TESTS
QC-PR-5

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

INDEX

1.0 METHODOLGY

1.1 Scope

These procedures are to be used to determine liquid limit, plastic limit and the plasticity index of fine-grained soils.

1.2 Procedure

The tests shall be performed in accordance with the procedure described in the most current version of ASTM D4318.

1.3 References

1. *Annual Book of ASTM Standards*, Volume 04.08

2.0 REPORTING

2.1 Forms

The following forms or approved equivalents shall be used for all sampling activities associated with this procedure.

- PR-7 Atterberg Limits Worksheet
- PR-11 Summary of Laboratory Test Results

2.2 Records

The original of the sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-7 ATTERBERG LIMITS WORKSHEET

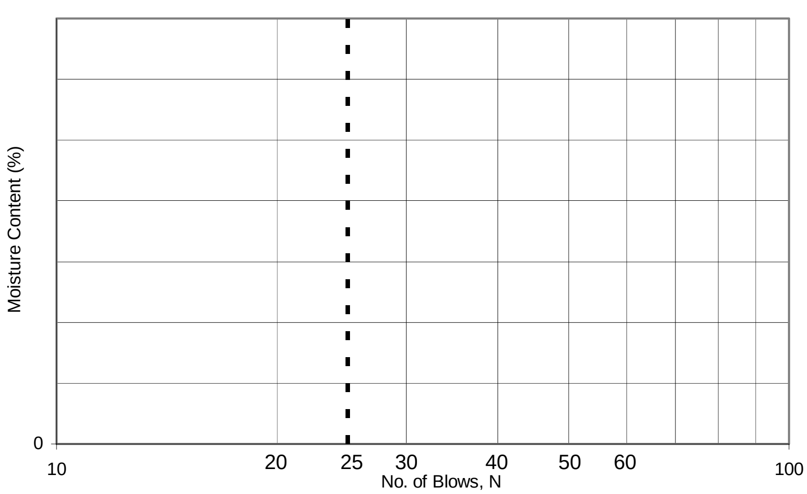
Technician______________________________ Project No. ____________
Approved By ______________________________ Date ____________
Material Sampled ____

Plastic Limit

Test Number	1	2	3	4	5	6
Dish No.						
Wt. of Dish & Wet Soil (g)						
Wt. of Dish & Dry Soil (g)						
Wt. of Dish (g)						
Wt. of Dry Soil (g)						
Wt. of Water (g)						
% Moisture						

Liquid Limit

Test Number	1	2	3	4	5	6
Dish No.						
No. of Blows						
Wt. of Dish & Wet Soil (g)						
Wt. of Dish & Dry Soil (g)						
Wt. of Dish (g)						
Wt. of Dry Soil (g)						
Wt. of Water (g)						
% Moisture						



Liquid Limit Factors from
Water Content & No. of Drops Causing Closure

N No. of Drops	k Liquid Limit Factor
20	0.974
21	0.979
22	0.985
23	0.990
24	0.995
25	1.000
26	1.005
27	1.009
28	1.014
29	1.018
30	1.022

Results	
LL	
PL	
PI	

PR-11 SUMMARY of LABORATORY TEST RESULTS

Technician______________________________ Project No. ______________________________

Approved By ______________________________ Date ______________________________

Material Tested ____

Sample Number	Moisture Content %	Dry Density pcf	Gradation		% Passing No. 200 Sieve	Atterberg Limits		Soil Type	Comments
			Gravel %	Sand %		Liquid Limit	Plasticity Index		

QUALITY CONTROL PROCEDURE

FOR

SOIL CLASSIFICATION for ENGINEERING PURPOSES
QC-PR-6

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

<u>INDEX</u>

1.0 METHODOLGY

1.1 Scope

The procedure is to determine the classification of soils for engineering purposes in accordance with the Unified Soils Classification System based on particle size and Atterberg Limits (liquid limit and plasticity index) of the soil.

1.2 Procedure

Classification of soils shall be performed in accordance with the most current version of ASTM D2487. Quality Control Procedures QC-PR-3 and QC-PR-5 shall be used to determine the classification parameters necessary to classify the materials according to the Unified Soil Classification System.

1.3 References

1. *Annual Book of ASTM Standards*, Volume 04.08

2.0 REPORTING

2.1 Forms

The soils classification shall be recorded on Form PR-11, Summary of Laboratory Test Results or approved equivalent.

2.2 Records

The original of the sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-11 SUMMARY of LABORATORY TEST RESULTS

Technician____________________________________ Project No. ______________________________

Approved By __________________________________ Date ___________________________________

Material Tested ____

Sample Number	Moisture Content %	Dry Density pcf	**Gradation**		% Passing No. 200 Sieve	**Atterberg Limits**		Soil Type	Comments
			Gravel %	Sand %		Liquid Limit	Plasticity Index		

QUALITY CONTROL PROCEDURE

FOR

LABORATORY COMPACTION CHARACTERISTICS of SOIL
QC-PR-7

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

INDEX

1.0 METHODOLGY

1.1 Scope

The procedure is to be used to determine the relationship between water content and dry unit weight of soils. (compaction curve).

1.2 Procedure

The procedure for performing this test shall be in accordance with the most current version of ASTM D698. Correction for unit weight and water content of soils containing oversize particles shall be determined in accordance with the current version of ASTM D4718.

1.2.1 One-point Proctors will be obtained and used as tool to determine whether the proctor being used for calculation of field compaction is representative of the material(s) being tested. If the dry density of the one-point is within ± 3 percent of the Proctor value being used, this provides adequate confirmation of the field compaction. If the dry density is greater than ± 3 percent of the proctor value, recalculation of the field compaction test will be required using a new Proctor value as established in Section 1.2. One-point Proctors will be performed in accordance with ASTM D698 also.

1.3 References

1. *Annual Book of ASTM Standards,* Volume 04.08

2.0 REPORTING

2.1 Forms

The following forms or approved equivalent shall be used to record test results of laboratory compaction tests:

- PR-8 Laboratory Compaction Test Worksheet
- PR-9 Rock and Moisture Correction Calculation Worksheet
- PR-10 Moisture Density Relationship
- PR-11 Summary of Laboratory Test Results

2.2 Records

The original sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-8 LABORATORY COMPACTION TEST WORKSHEET

Technician ________________________________ Project No. ____________________
Approved by _______________________________ Date __________________________
Material Sampled ____

GRADATION for COMPACTION METHOD SELECTION

Sieve Size	3/4”	3/8”	#4	-#4	Total
Weight Retained					
% Retained					
Cumulative % Retained					

Sample No. ____

Sample Description ____

ASTM D698 ☐ Method: A B C Other___________

TEST DATA

Point Number	1	2	3	4	5	6	7
Amt. Of Water Added, Vol.							
Wt. of Mold and Wet Soil							
Wt. of Mold							
Wt. of Wet Soil							
Wet Density, pcf							

Dish Number							
Weight of Dish & Wet Soil							
Weight of Dish & Dry Soil							
Weight of Dish							
Weight of Water							
Weight of Dry Soil							

Moisture Content, %							
Dry Density, pcf							

Remarks ____

____

PR-9 ROCK and MOISTURE CORRECTION CALCULATIONS

Technician ______________________________ Project No. ________________

Approved By ______________________________ Date ____________________

Sample No. __________ Material ______________________________________

Field Unit Dry Weight ______________________________
Field Moisture Content ______________________________
Total Wet Weight of Correction Sample ______________________________
Wet Weight of Oversized Fraction ______________________________
Wet Weight of Finer Fraction ______________________________
Specific Gravity of Oversized Material ______________________________
SSD Moisture Content of Oversized Material ______________________________
Laboratory Max. Dry Density (Finer Fraction) ______________________________
Optimum Moisture Content (Finer Fraction) ______________________________

% Wet Oversize Fraction ______________________________
% Wet Finer Fraction ______________________________
% Dry Oversize Fraction ______________________________
% Dry Finer Fraction ______________________________
Corrected Moisture Content (Finer Fraction) ______________________________
Corrected Dry Unit Weight (Finer Fraction) ______________________________
Corrected % Compaction ______________________________
Deviation from Optimum Moisture Content ______________________________

Remarks ____
____
____
____
____

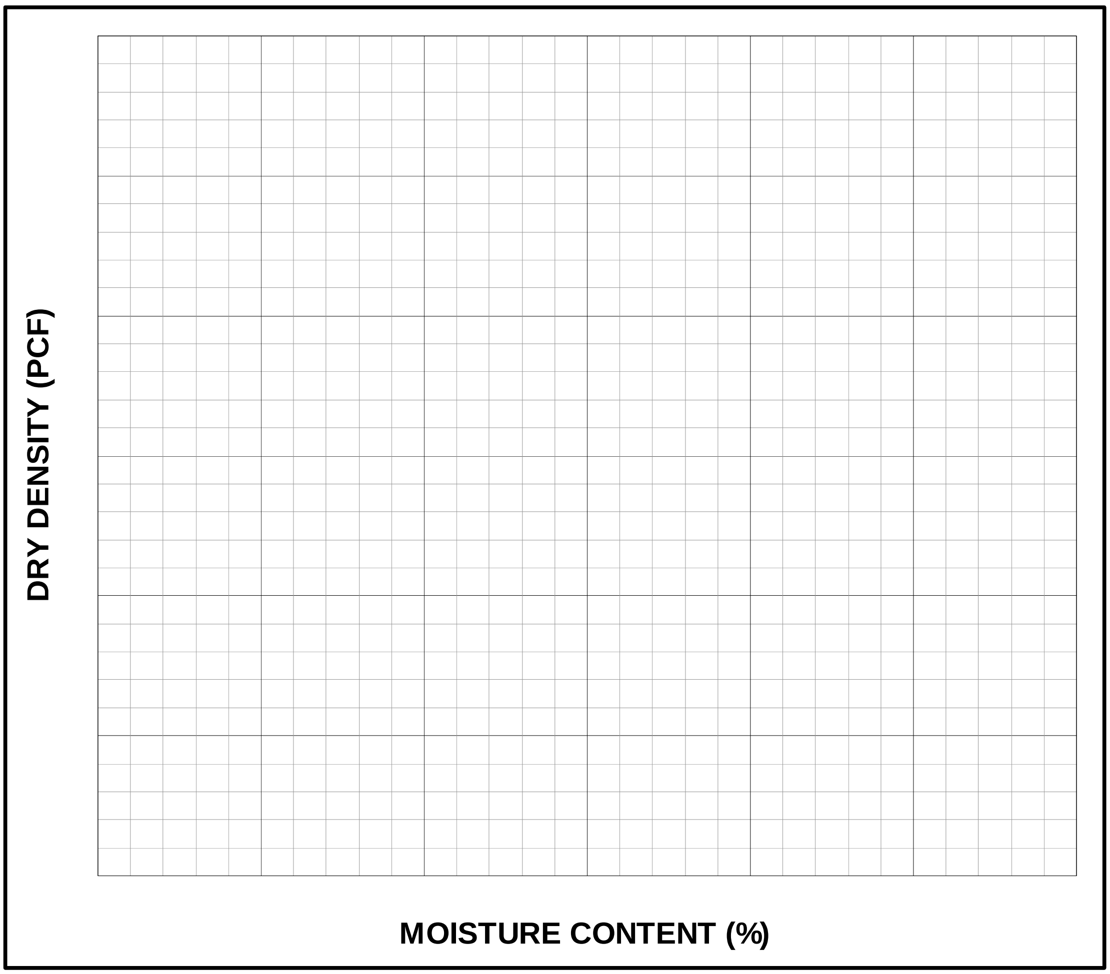
PR-10 MOISTURE DENSITY RELATIONSHIP

Technician ______________________________ Project No. ________________

Approved By ______________________________ Date ____________________

Sample No. ___________ Material ______________________________________

Method ______________ % Oversized ___________ Specific Gravity __________



Maximum Dry Density ________ pcf Opt. Moisture Content ________%

Corrected Max. Density * ______ pcf Corrected Opt. Moisture Content * _______ %

Liquid Limit ________________ Plasticity Index ___________________

Gravel ___________ % Sand ___________ % Silt & Clay ____________ %

* Corrected Density and Moisture by ASTM D4718

PR-11 SUMMARY of LABORATORY TEST RESULTS

Technician________________________ Project No. ________________________

Approved By ________________________ Date ________________________

Material Tested ____

Sample Number	Moisture Content %	Dry Density pcf	**Gradation**		% Passing No. 200 Sieve	**Atterberg Limits**		Soil Type	Comments
			Gravel %	Sand %		Liquid Limit	Plasticity Index		

QUALITY CONTROL PROCEDURE

FOR

IN-PLACE DENSITY TESTS
QC-PR-8

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

INDEX

1.0 METHODOLGY

1.1 Scope

The test procedures are to be used to determine the density of in-place soils, aggregates or a combination of these materials.

1.2 Procedure

Tests shall be conducted in accordance with the most recent version of the Standard ASTM test procedures referenced below. Compaction shall be based on the percent of field maximum dry density versus the laboratory maximum dry density as established in Procedure QC-PR-7 for the correlative material type. All compaction tests shall be performed for each material type and at frequencies in accordance with the Specifications.

1.2.1 Nuclear Gauge Method

- ASTM D2922; Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow depth).
- ASTM D3017; Water Content of Soil and Rock in Place by Nuclear Methods (Shallow depth).

1.2.2 Sand Cone Method

- ASTM D1556; Density and Unit Weight of Soil in Place by the Sand-Cone Method.
- ASTM D2216; Laboratory Determination of Water (Moisture) Content of Soil and Rock.
- ASTM D4643; Determination of Water (Moisture) Content of Soil by the Microwave Oven Method.

1.3 References

1. *Annual Book of ASTM Standards,* Volume 04.08

2.0 REPORTING

2.1 Forms

The following forms or approved equivalent shall be used to record test data and results.

- PR-12 Nuclear Density Test Data
- PR-13 Field Density Tests (Sand Cone)

2.2 Records

The original sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-12 NUCLEAR DENSITY TEST DATA

Technician ______________________________ Project No. __________________________ Date ________________

Approved By __________________________________ Material ____

Standard Count - Density _____________________________ Moisture _____________________________

Test No.	1	2	3	4	5	6	7	8	9	10
Station										
Offset										
Elevation										
Mode & Depth										
Moisture Count										
Density Count										
Wet Density										
Dry Density										
% Compaction										
Moisture										
% Moisture										
Standard Density (max.)										
Optimum Moisture										
Moisture Correction										
Moisture Variation ± from Optimum										
Specified Degree of Compaction										

Remarks ____

____

PR-13 FIELD DENSITY TESTS (SAND CONE)

Technician ______________________________ Project No. ____________

Approved By ______________________________ Date ________________

Material ____

Ground Surface Calibration

Weight of Jar ( ) Full of Sand	____________
Weight of Jar ( ) After Surface Calibration	____________
Weight of Sand Used, *Gs*	____________

Soil Density

Weight of Soil + Can ( )	____________
Weight of Can ( )	____________
Weight of Soil, *W*	____________
Weight of Jar ( ) before use, *WJ1*	____________
Weight of Jar ( ) after use, *WJ2*	____________
Weight of Sand Used, (*WJ1 - WJ2) = SU*	____________
Weight of Sand in Cone, *Gs*	____________
Weight of Sand in Hole, (*SU - Gs*) = *SW*	____________
Density of Standard Sand, *Gamma (pcf)*	____________
Volume of Hole, (*SW / Gamma*) = *Vh*	____________
Wet Density, (*W / Vh*) = G_{wet}	____________
Dry Density, (G_{wet} / (1+ *%W*)) = G_{dry}	____________

Moisture Content

Weight of Wet Soil + Pan ( )	____________
Weight of Dry Soil + Pan ( )	____________
Weight of Pan ( )	____________
Weight of Water, *Ww*	____________
Weight of Dry Soil, *Wd*	____________
Water Content, (*Ww / Wd*) = *%W*	____________

QUALITY CONTROL PROCEDURE

FOR

COMPACTED SOIL LAYER THICKNESS
QC-PR-9

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

INDEX

1.0 METHODOLGY

1.1 Scope

The procedure is to be used to determine the thickness of compacted soil layers during dam construction and clay placement.

1.2 Procedure

Continuous monitoring and surveying of placed materials using standard survey methods during construction shall be the preliminary means of verifying that lifts are being placed in accordance with the Specifications. The thickness of compacted soil layers will be checked at random locations by either drilling or excavation pits to verify survey data. After the layer of clay or fill has been placed and compacted, a hole will be drilled or a pit excavated. Lift thickness shall be measured by taping the distance from a straight-edge placed across the top of the hole or pit to the bottom of the cavity. All measurements shall be made to the nearest 100^{th} of a foot. Prior to placing the straight-edge across the top, all loose surface soils shall be removed to expose a firm base.

1.3 References

1. *Surveying Theory and Practice,* Sixth Edition, 1981, Davis, Foote, Anderson and Mikhail

2.0 REPORTING

2.1 Forms

Field data notebooks containing raw survey data shall be maintained in the Project Files. Direct measurement data shall by systematically recorded and incorporated in the construction verification program using PR-1 or an approved equivalent.

2.2 Records

The original sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-1 CONSTRUCTION ACTIVITIES REPORT

Project No.:______________________________

Technician: ______________________________

Date: ______________________________

Approved By: ______________________________

Daily Report No.:____________ Sheet______of________

Weather Conditions and Temperature: ______________________________

Equipment: ______________________________

Construction Activities and Observations: ______________________________

QUALITY CONTROL PROCEDURE

FOR

HDPE LINER SEAM INTEGRITY
QC-PR-10

BY
PLATEAU RESOURCES, LTD.

877 N. 8th W.
Riverton, WY 82501

Revision No.	Issue Date	Approved By:

INDEX

1.0 METHODOLGY

1.1 Scope

The procedure is used to determine the integrity of field seams used in joining sheets of High Density Polyethylene (HDPE) liners by destructive and nondestructive testing.

1.2 Procedure

1. All testing shall be in accordance with the standardized procedures described in the most current version of ASTM D4437. Preparation of test samples shall be in accordance with ASTM D618.
2. Destructive testing will include peel and sheer tests as presented in the above-referenced standards. In either case, a failed test occurs when the weld fails and a passing test occurs when the fabric fails first.
3. All field seams will be continuously inspected visually.
4. All field seams shall be tested by The Vacuum Box test or Air Pressure test, dependent on the welding method.
5. Any seams or weld found to be defective by destructive, nondestructive testing or visually, shall be marked and repaired in accordance with the manufacturer's recommendations.
6. All repairs shall be tested.

1.3 Reference

Annual Book of ASTM Standards, Volumes 04.08 and 04.09

2.0 REPORTING

2.1 Forms

The following forms or approved equivalent shall be used to record test data and results.

- PR-1 Construction Activities Report
- PR-14 Panel Placement Log
- PR-15 Geomembrane Field Trial Log
- PR-16 Geomembrane Seaming Record
- PR-17 Geomembrane Seam Air Pressure Test Log
- PR-18 Repair Log
- PR-19 Geomembrane Seam Destructive Sample Log

2.2 Records

The original sampling reports shall be maintained in the Project File. Copies shall be available upon request.

PR-1 CONSTRUCTION ACTIVITIES REPORT

Project No.:______________________________

Technician: ______________________________

Date: ______________________________

Approved By: ______________________________

Daily Report No.:____________ Sheet______of________

Weather Conditions and Temperature: ______________________________

Equipment: ______________________________

Construction Activities and Observations: ______________________________

PR-14 PANEL PLACEMENT LOG

Technician ______________________________ Project No. ____________________

Approved By ______________________________ Date __________________________

Liner Type ____________________ Primary ☐ Secondary ☐ Page ________of ________

Panel No.	Date	Time	Roll No.	Width (ft.)	Length (ft.)	Area (Sq. Ft.)

PR-15 GEOMEMBRANE FIELD TRIAL LOG

Technician ______________________________________ Project No. ____________

Approved By ____________________________________ Date _________________

Liner Type __________________ Primary ☐ Secondary ☐ Page ________of ________

Project Seam Requirements: Fusion - Peel __________ ppi Shear __________ ppi

Project Seam Requirements: Extrusion - Peel __________ ppi Shear __________ ppi

Sample	Date	Time	Amb.	Welder Id.		Wedge	Extruder	Seam Strength		Pass or	Ins p	Remarks
			Temp.	Mach.	Oper.	Temp./ Speed	Temp./ Pre-Heat	Peel ppi IN/OUT	Shear ppi	Fail	Id.	

PR-16 GEOMEMBRANE SEAMING RECORD

Technician ______________________________________ Project No. ____________

Approved By ______________________________________ Date _________________

Liner Type __________________ Primary ☐ Secondary ☐ Page ________of ________

Seam =	Seaming Record							
Panel No./ Panel No.	Length Welded	Date Time	Welder Operator	Machine No.	Temperature		Machine Speed	Nondestructive Test Pass/Fail
					Machine	Ambient		

C:\ed\projects\2007-50\TMP\TEXT\QCP 10.doc

April 2007

PR-17 GEOMEMBRANE SEAM AIR PRESSURE TEST LOG

Technician ______________________________________ Project No. ____________

Approved By ______________________________________ Date __________________

Liner Type __________________ Primary ☐ Secondary ☐ Page ________of ________

Seam = Panel No./ Panel No.	Start Location	End Location	Pressure Test			
			Pressure	Tester Id.	Pass/Fail	Date/Time Tested

PR-18 REPAIR LOG

Technician ______________________________ Project No. ____________

Approved By ______________________________ Date ________________

Liner Type ________________ Primary ☐ Secondary ☐ Page _______ of _______

Repair No.	Defect Code	Repair Date	Location / Panels	Approx. Time	Repair Type	Approx. Dimensions	Repair Tech.	Inspector	Vacuum Test P/F	Vacuum Test Date

PR-19 GEOMEMBRANE SEAM DESTRUCTIVE SAMPLE LOG

Technician ____ Project No. ____________

Approved By ______________________________________ Date __________________

Liner Type __________________ Primary ☐ Secondary ☐ Page ________of ________

Project Seam Requirements: Fusion - Peel __________ ppi Shear __________ ppi

Project Seam Requirements: Extrusion - Peel __________ ppi Shear __________ ppi

	Seam =			Field	Test	Results	Lab Test	
Sample	Panel No. / Panel No.	Date Removed	Inspector Id.	Peel, ppi IN/OUT	Shear, ppi	Pass/ Fail	Results Pass/Fail	Remarks
Describe	Sample	Location						
Describe	Sample	Location						
Describe	Sample	Location						

APPENDIX D

MONITORING

APPENDIX D

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D.1 Table 5.5-7 Radiological Environmental Monitoring Program – Operational

Table 5.5-7					
RADIOLOGICAL ENVIRONMENTAL MONITORING PROGRAM - OPERATIONAL					
Type of Sample	**SAMPLE COLLECTION AND MEASUREMENT**				
	No.	**Location**	**Method and Frequency**	**Test Frequency**	**Type of Measurement**
Air stack particulates	1	Ore dump point stack	Semi-annual grab sample	Semiannually	Natural uranium Th-230, Ra-226, Pb-210 and flow rate
	1	Yellowcake Dryer and packaging stack	Isokinetic sample	Quarterly	Natural uranium, Th-230, Ra-226, and Pb-210
				Quarterly	Flow rate
Environmental particulates	3	At site boundaries & in different sectors having highest predicted concentrations	Continuous; weekly filter change or as required by dust loading	Quarterly composited	Natural uranium, Th-230, Ra-226, and Pb-210
	1	At nearest residence - Ticaboo	Continuous; weekly filter change or as required by loading	Quarterly composited	Natural uranium Th-230, Ra-226, and Pb-210
	1	Control location	Continuous; weekly filter change or as required by dust loading	Quarterly composited	Natural uranium, Th-230, Ra-226, Pb-210
Radon	5	Same as for air particulates	Continuous Track Etch	Quarterly	Rn-222
Groundwater	4, (*11)	Down-groundwater-flow gradient monitoring wells (RM-2R, RM-7, RM-14, RM-18, RM-19) (*RM-23 through RM-32)	Semiannually	Semiannually	Natural uranium, As, Cl, Se, pH
	1	Groundwater under tailings	Annually	Annually	Rate and direction of flow
	1	up-gradient control well (RM-1, RM-12)	Semiannually	Semiannually	Natural uranium, As, Cl, Se, pH
Surface water	None	N/A	N/A	N/A	N/A
Direct radiation	5	Same as for air particulate samples	TLDs	Quarterly	Gamma
Vegetation	1	Animal grazing areas	Annual grab sample in	Hold sample	Th-230, Ra-226,

Table 5.5-7					
RADIOLOGICAL ENVIRONMENTAL MONITORING PROGRAM - OPERATIONAL					
Type of Sample	**SAMPLE COLLECTION AND MEASUREMENT**				
	No.	**Location**	**Method and Frequency**	**Test Frequency**	**Type of Measurement**
		downwind of mill	spring growing season	for 1 yr; Analyze only if required	Pb-210
Soil	5	Same as for air particulate samples	Annual grab samples	Annually	Natural uranium Th-230, Ra-226
Instrument calibration	All instruments in use	N/A	Semiannually or at mfg's suggested intervals, whichever is sooner	Voltage plateau[1] Pulse Source	Instrument response
Instrument calibrations	Environmental air samplers	N/A	Quarterly	Quarterly	Flow rate
Surface Evaluations	N/A	Tailings Impoundment	Daily, Monthly, Quarterly, Per SOP	N/A	Examination Measurements Surveys
Meteorology	1		Continuously; wind speed & direction	N/A	N/A
Trend analyses	Routine monitoring programs	N/A	Annually	N/A	N/A
Reports	1	N/A	Semiannually effluent monitoring report	N/A	N/A
Quality assurance audit	N/A	N/A	Semiannually	N/A	N/A
Wildlife	N/A	Tailings Impoundment	Daily Visual	N/A	Record Observations
Security	N/A	Mill & Tailing Facility	Inspection	24 hr.	Visual

* = Wells to replace wells RM-7, RM-18 and RM-19 prior to construction of Cell 2.

[1]Where electrodes are accessible

D.2 Table 5.5-8 Interim Environmental Monitoring Program
(Mill not operational for 30 days or more)

Table 5.5-8					
INTERIM ENVIRONMENTAL MONITORING PROGRAM (Mill not operational for 30 days or more)					
Type of Sample	**Sample Collection and Measurement**				
	No.	**Location**	**Method and Frequency**	**Test Frequency**	**Type of Measurement**
Air particulates	1	Downwind of impoundment and ore stockpiles	20 hrs/quarter	Semiannually composited	Natural uranium and Ra-226
Radon	None	N/A	N/A	N/A	N/A
Water - Groundwater	4, (*11)	Down-groundwater-flow gradient monitoring wells (RM-2R, RM-7, RM-14, RM-18, RM-19) (*RM-23 through RM-32)	Semiannually	Semiannually	Natural uranium, As, Cl Se, pH
Water - Surface Water (Seeps)	None	N/A	N/A	N/A	N/A
Direct Radiation	None	N/A	N/A	N/A	N/A
Soil	None	N/A	N/A	N/A	N/A
Vegetation	None	N/A	N/A	N/A	N/A
Instrument calibrations	All instruments in use	N/A	Semiannually or at mfg's suggested intervals, whichever is sooner	Voltage plateau[2] Pulse Source	
Surface Evaluations	N/A	Tailings Impoundment	Monthly & Yearly Per SOP	N/A	Examination Measurement Surveys
	N/A	Ore stockpiles	Monthly	N/A	N/A
Meteorology	None		N/A	N/A	N/A
Trend analyses	Routine monitoring program	N/A	Annually	N/A	N/A
Reports	1	N/A	Semiannually effluent monitoring report	N/A	N/A
Audit	1	N/A	Annually ALARA	N/A	
Security	N/A	Mill & Tailing Facility	Inspection	Daily	Visual
* = Wells to replace wells RM-7, RM-18 and RM-19 prior to construction of Cell 2.					

[2]Where electrodes are accessible.

D.3 Table 3-1 Basic Data for the Shootaring Wells and Piezometers

TABLE 3-1. BASIC DATA FOR THE SHOOTARING WELLS AND PIEZOMETERS.

WELL NAME	NORTH. COORD.	EAST. COORD	CASING DIAMETER (in)	TOTAL DEPTH (ft-mp)	STICKUP (ft)	MP ELEV. (ft-msl)	WATER LEVEL DEPTH (ft-mp)	WATER LEVEL ELEVATION (ft-msl)	SLOTTED CASING (ft-lsd)	SAND PACK (ft-lsd)	PUMP INTAKE (ft-mp)
						WELLS					
OW1A	57140	63730	1.0	300.0	0.2	4472.53	239.40	4233.13	200-300	-	---
OW1B	57140	63730	1.0	798.0	1.9	4474.23	449.73	4024.50	648-798	-	---
OW2	57094	63667	1.0	300.0	0.2	4470.70	228.50	4242.20	200-300	-	---
OW3	57046	63659	1.0	798.0	2.3	4470.78	452.85	4017.93	650-798	-	---
OW4	57035	63707	1.0	570.0	2.3	4472.54	230.48	4242.06	435-570	-	---
RM1	59307	61827	3.0	487.0	2.2	4449.20	176.50	4272.70	220-480	157-487	225
* RM2	57731	63040	3.0	520.0	1.6	4519.76	258.25	4261.51	260-520	250-520	---
RM2R	57924	63142	5.0	300.0	1.2	4504.86	243.40	4261.46	250-300	242-300	273
* RM3	57193	60647	6.0	540.0	1.8	4461.32	214.80	4246.52	230-540	190-540	246
* RM4	56472	61099	3.0	500.0	3.5	4395.50	155.80	4239.70	190-490	115-500	176
* RM4R	56358	61086	5.0	160.0	1.0	4368.32	128.60	4239.72	110-160	105-160	157
* RM5	56416	61286	3.0	440.0	3.6	4379.12	140.30	4238.82	150-430	130-440	172
* RM6	56348	61481	3.0	460.0	2.3	4374.57	136.50	4238.07	175-455	110-460	174
RM7	57904	61645	3.0	219.5	2.2	4395.86	140.30	4255.56	187-217	177-217	200
RM8	57204	61576	3.0	79.1	3.1	4381.77	58.10	4323.67	57-77	47-77	75
* RM9	56767	61363	3.0	82.8	1.2	4369.31	61.30	4308.01	62-82	52-82	80
* RM10	56286	61272	5.0	99.0	2.0	4343.57	95.30	4248.27	57-97	53-97	---
* RM11	56594	60769	5.0	240.0	2.0	4436.14	184.70	4251.44	140-180	5-180	220
									180-240#	-	
RM12	59477	61791	5.0	157.0	1.3	4415.95	142.90	4273.05	117-157	110-157	156
* RM13	56648	61996	5.0	270.0	2.0	4434.81	189.60	4245.21	140-180	5-180	219
									180-270#	-	
RM14	58419	61368	5.0	260.0	1.5	4450.84	191.30	4259.54	134-174	127-174	253
									174-260#	-	
* RM15	56311	61354	5.0	460.0	1.9	4343.75	107.70	4236.05	379-459	95-459	157
* RM16	56615	60772	5.0	296.0	1.2	4434.95	194.60	4240.35	246-296	240-296	225
* RM17	56636	61993	5.0	290.0	0.7	4433.58	190.00	4243.58	240-290	235-290	218
RM18	57833	61851	5.0	243.3	1.3	4421.56	163.80	4257.76	162-242	149-242	232
RM19	58077	61524	5.0	236.3	1.3	4409.50	152.30	4257.20	155-235	139-235	219
RM20	57208	61592	5.0	212.6	1.6	4380.83	129.70	4251.13	131-211	120-212	201
RM21	57843	61851	5.0	141.3	1.3	4421.64	Dry	4280.34	110-140	100-140	---
RM22	58088	61513	5.0	120.8	0.8	4410.52	Dry	4289.72	90-120	80-120	---
WW1	57144	63677	6.0	870.0	-2.8	4454.79	---	---	635-870#	-	---
WW2	56562	63086	6.0	1000.0	-3.4	4471.61	---	---	602-1000#	-	---
						TAILINGS WELLS					
T4	58456	61953	2.0	20.0	1.2	4431.20	Dry	4411.20	12.9-17.9	10-18	---
T5	58371	61891	2.0	10.0	2.5	4425.00	Dry	4415.00	2.5-7.5	0.7-8	---
T6	58133	61801	2.0	11.7	2.9	4429.00	Dry	4417.30	3.8-8.8	1-9	---
						PIEZOMETERS					
PZ1	56598	61022	1.0	87.0	1.8	4434.51	---	---	75-85	2-85	---
PZ2	56580	61327	1.0	88.0	1.7	4434.74	---	---	76-86	3-86	---
PZ3	56564	61575	1.0	88.0	1.9	4435.34	---	---	76-86	3-86	---
* PZ4	56271	61383	1.0	25.0	1.7	4347.17	Dry	4320.92	13-23	2-23	---
* PZ5	56301	61275	1.0	25.0	1.8	4344.79	Dry	4318.49	13-23	1-23	---

TABLE 3-1. BASIC DATA FOR THE SHOOTARING WELLS AND PIEZOMETERS.

WELL NAME	NORTH. COORD.	EAST. COORD	CASING DIAMETER (in)	TOTAL DEPTH (ft-mp)	STICKUP (ft)	MP ELEV. (ft-msl)	WATER LEVEL DEPTH (ft-mp)	WATER LEVEL ELEVATION (ft-msl)	SLOTTED CASING (ft-lsd)	SAND PACK (ft-lsd)	PUMP INTAKE (ft-mp)
* PZ6	56332	61167	1.0	25.0	1.6	4362.50	Dry	4336.90	13-23	2-23	---

NOTE: Wells RM1 through RM6, RM15 through RM17, OW1A and OW2 are completed in the Entrada Aquifer
Wells RM2R, RM4R, RM7 through RM14 and PZ4 through PZ6 are completed in the Upper Entrada Sandstone
Wells WW1, WW2, OW1B and OW3 are completed in the Navajo Aquifer
Well OW4 is completed in the Carmel Aquitard
Piezometers PZ1 through PZ3 are Dam Piezometers
mp = measuring point; lsd = land surface datum; msl = mean sea level
# = open hole
* = Abandoned Well
Above data compiled from physical measurements, records and site surveys.

APPENDIX E

ORE PAD LINER

APPENDIX E

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E.1 Ore Pad Study, December 11, 1998

plateau resources LIMITED

Corporate Offices:
877 North 8th West, Riverton, WY 82501

Tel: (307) 856-9271
Fax: (307) 857-3050

Shootaring Operations:
Box 2111, Ticaboo, Lake Powell, UT 84533

Tel: (801) 788-2120
Fax: (801) 788-2118

December 11, 1998

State of Utah
Department of Environmental Quality
Division of Radiation Control
168 North 1950 West
Salt Lake City, Utah 84114-4850
Attn. Mr. Rob Herbert

Re: Shootaring Canyon Uranium Mill, Ore Pad Study

Dear Rob:

As per your request, Plateau conducted a study of the surface material on the Ore Pad. This study was to determine the hydraulic conductivity of the surface material. On September 17, 1998 three samples were collected and composited into one sample for testing. The samples were collected at a depth of two to twelve inches. The composite sample was sent to Inberg-Miller Engineers for laboratory testing. On December 7 additional data was collected as to the total depth of the surface material. Five areas on the Ore Pad were tested and found to have a depth of 12 to 14 inches of clay material on a couple inch gravel base.

The sample was prepared to optimum moisture of 95 % maximum density. The 95 % density was used as it will be most representative of ore pad operating conditions. Ore pad operating conditions will include the use of heavy equipment and water for dust control. The compaction effect of these operating conditions will produce at least the 95 % maximum density used in the laboratory test.

The use of any other ore pad surface material, such as, concrete or asphalt, is not easy to maintain and Plateau will continue to use the clay prepared base for the ore pad. Attached is the November 19, 1998 laboratory report. The results are; optimum moisture of 12.5 percent, maximum density of 94.4 percent and hydraulic conductivity of 3.7 x E-6 cm/sec.

Should you have any questions please contact me at the Riverton office.

Sincerely,
Plateau Resources Limited

F. R. Craft

Enclosure
xc: File

Procedure for Collection of Soil Samples from Ore Pad

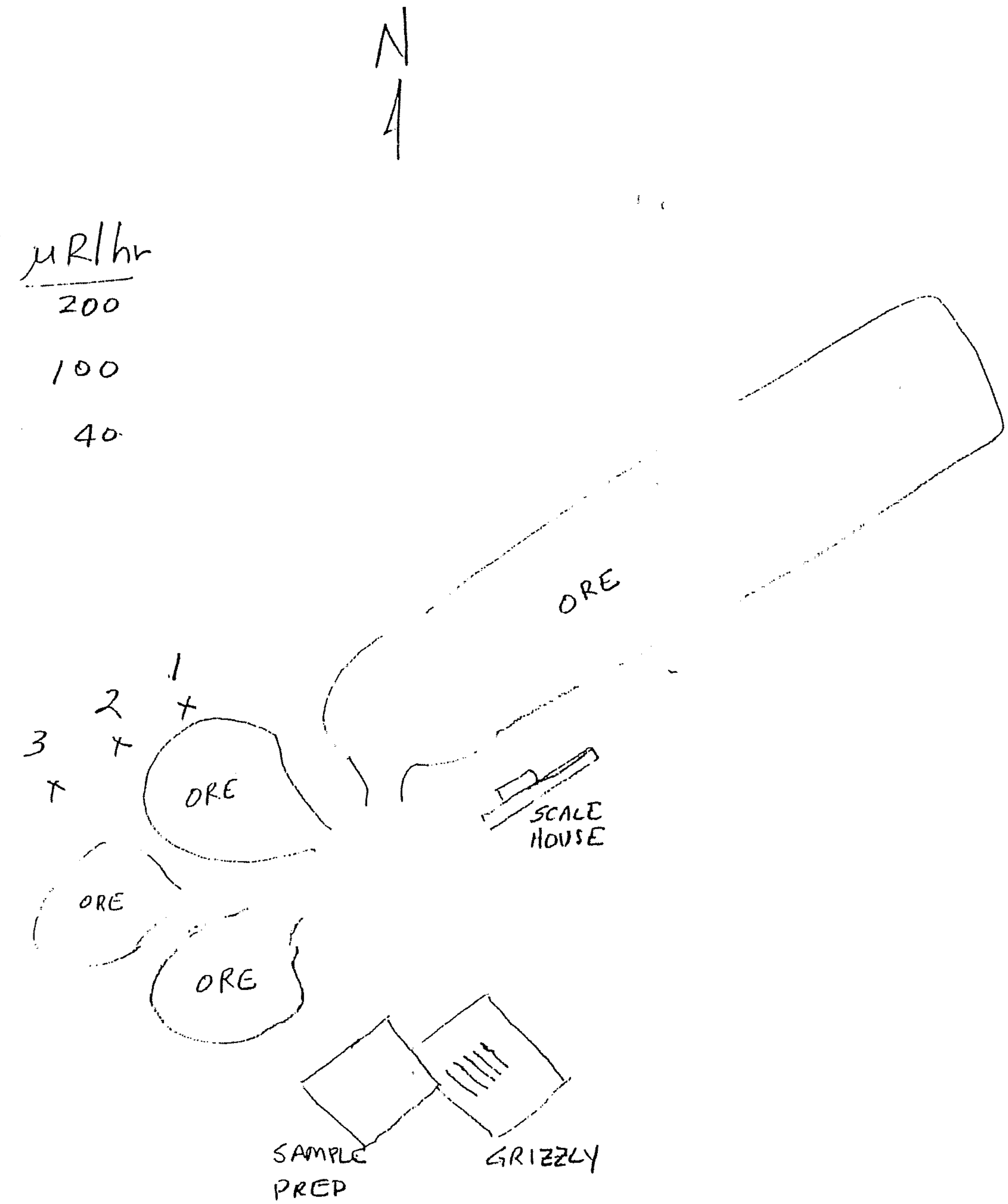
Locations:

See attached sketch.
Holes 1 and 2 were opposites water supply nozzles.

Method:

Used a round point shovel to dig down approximately two inches and then started placing soils into a five gallon plastic bucket. Continued placing soil into the bucket until a depth of approximately twelve inches was reached.
This was repeated at two additional sampling locations. The bucket at completion was three fourts full.
A gamma probe was suspended into each hole and a gamma field reading was recorded. Those results are indicated on the sketch.
The samples were taken on September 17, 1998.

Hole #	µR/hr
1	200
2	100
3	40



NOT TO SCALE

Ore Pad Sample Locations
Sampled 9-17-98

INBERG-MILLER ENGINEERS

124 EAST MAIN STREET RIVERTON, WYOMING 82501-4397 307-856-8136

November 19, 1998 7664-RX

U.S. Energy Corp./Crested Corp.
877 N. 8th West
Riverton, WY 82501

ATTENTION: FRED CRAFT

RE: MOISTURE-DENSITY RELATIONSHIP
AND PERMEABILITIES
ORE PAD
SHOOTARING CANYON DAM SITE

Gentlemen:

This letter transmits the results of moisture-density relationships and hydraulic conductivity (permeability) testing that we performed in accordance with our November 21, 1996, Service Agreement and Proposal and Amendment No. 1 dated January 2, 1998.

Samples were collected from three locations on the northwest areas of the ore pad at the above project site. Each sample, approximately 1/6 of a cubic foot, was collected at depths between 2 and 12 inches below the ground surface. Samples were collected by U.S. Energy personnel and delivered to Inberg-Miller Engineers for testing. Two laboratory soil tests performed:

1. Moisture-Density Relationship (ASTM D698)
2. Measurement of Hydraulic Conductivity (ASTM D5084) Method B

Measurement of the hydraulic conductivity included preparing a sample with an initial diameter of 2.432 inches and length of 2.994 inches, a dry density of 108.6 pounds per cubic foot (pcf) and a moisture content of 11.9 percent. A graphic representation of the moisture-density relationship as determined by a Standard Proctor analysis is included with this letter. The results indicate an optimum moisture content of 12.5 percent and a maximum dry density of 115.0 pcf. Laboratory tests resulted in a hydraulic conductivity of 3.7×10^{-6} cm/sec. Accordingly, the sample density is 94.4 percent of the ASTM D698 maximum density.

We appreciate the opportunity to participate on this project. If you have any questions with the contents of this letter or enclosures or if we can be of additional assistance, please contact us.

Sincerely,

INBERG-MILLER ENGINEERS

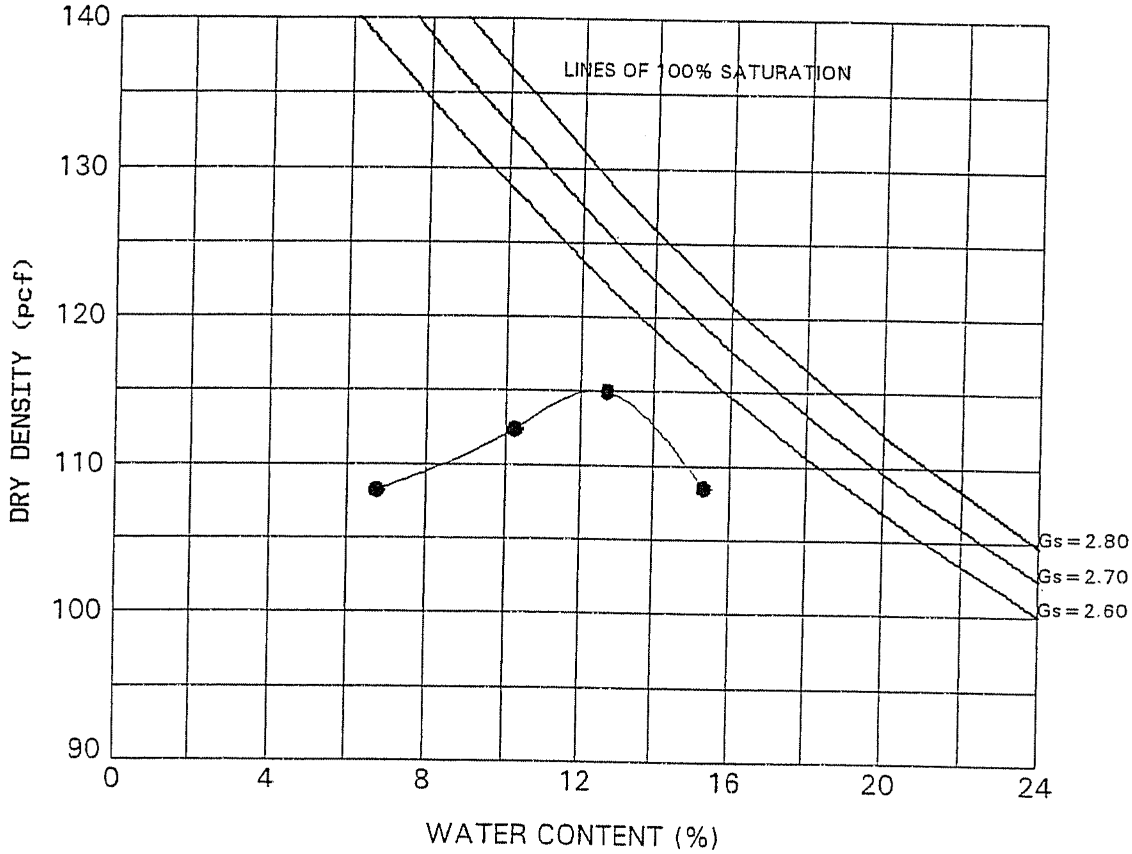
Jeffrey A. Young
Civil Engineer

JAY:cāg:ltr\7664-rx.ltr

Enclosure as stated

MOISTURE-DENSITY ANALYSIS

PROJECT:	SHOOTERING DAM	TEST DATE:	9-28-98
JOB NO.:	7664 RX	TESTED BY:	JMR
CLIENT:	U.S. ENERGY	TEST METHOD:	STANDARD PROCTOR



SAMPLE NO.:	1	SOIL DESCRIPTION:	Red to Brown, Silty, Fine Sand
SAMPLED BY:	CLNT		
DEPTH:	0.00	SOURCE:	On Site

PASSING #200 SIEVE:	%		
LIQUID LIMIT:		OPTIMUM WATER CONTENT:	12.5 %
PLASTICITY INDEX:		MAXIMUM DRY DENSITY:	115.0 pcf

INBERG-MILLER ENGINEERS

APPENDIX F

CLAY BORROW MATERIAL

APPENDIX F

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F.1 Permeability, Atterbert Limits, Gradation and Moisture-Density for the Alternate Clay Source

by Inberg-Miller Engineers, September 20, 2005

RIVERTON . CASPER . CHEYENNE
POWELL . GREEN RIVER

INBERG-MILLER ENGINEERS

QUALITY SOLUTIONS THROUGH TEAMWORK

September 20, 2005

10223-RM

RECEIVED
SEP 21 2005

Mr. Fred Craft
U.S. Energy Corporation
877 North 8th West
Riverton, WY 82501

RE: SOIL TEST RESULTS
SHOOTERING CANYON MILL PROJECT

Dear Fred:

This letter transmits the results of laboratory testing that we performed on a sample of claystone that you submitted to our Riverton, Wyoming laboratory.

Specifically, you requested that we perform classification tests consisting of Moisture-Density Relationship (Standard Proctor), Atterberg-Limits, and Particle Size Analysis on 2 sub-samples of the claystone that you submitted. Further, you requested permeability testing on 3 specimens re-molded from the claystone.

The claystone as submitted was hard, dry, and shale-like. The claystone rapidly softened when submerged in water. The tests were performed on the claystone after it was softened to a soil-like consistency.

Refer to the attached test results. Note that the progress of permeability testing was slow due to the low permeability of the remolded claystone (which had been remolded to 95 percent of the ASTM D698 maximum dry density). The permeability tests were terminated when the volume of water measured passing through the sample was determined to represent permeability on the order of 10^{-8} centimeters/second or less.

Please call if you have any questions or require further information.

Sincerely,

INBERG-MILLER ENGINEERS

Glenn M. Bobnick

Glenn M. Bobnick, P.E.
Geotechnical Engineer
Riverton Office

Enclosures as stated

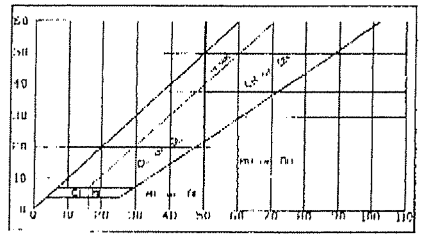
124 East Main Street, Riverton, WY 82501, 307-856-8136, 307-856-3851 (fax), riverton@inberg-miller.com
1120 East "C" Street, Casper, WY 82601, 307-577-0806, 307-472-4402 (fax), casper@inberg-miller.com
350 Parsley Boulevard, Cheyenne, WY 82007, 307-635-6827, 307-635-2713 (fax), cheyenne@inberg-miller.com
428 Alan Road, Powell, WY 82435, 307-754-7170, 307-754-7088 (fax), powell@inberg-miller.com
520 Wilkes Drive, Suite 13, Green River, WY 82935, 307-875-4394, 307-875-4395 (fax), greenriver@inberg-miller.com

ATTERBERG LIMITS TEST

INBERG-MILLER ENGINEERS

ASTM D4318

CLIENT:	U.S. Energy
PROJECT:	Shootering Canyon Mill
JOB NO.:	10223 RM
TEST DATE:	6-3-05
TESTED BY:	DAL
SAMPLE NOS:	Sec. 16, A & C
SAMPLED BY:	Client
SOURCE:	Site Soil



SAMPLE NO. A	Plastic (Y/N)?	Y

PLASTIC LIMIT INFORMATION			LIQUID LIMIT INFORMATION			TEST RESULTS	
TRIAL NO.:	1	2	1	2	3	LIQUID LIMIT:	90
Tare (Pan) No.:	2P		3L			PLASTIC LIMIT:	29
Tare (Pan) Wt.:	13.83		23.58			PLASTIC INDEX:	61
Tare + Wet Soil Wt.:	17.81		45.5			USCS CLASSIFICATION:	CH
Tare + Dry Soil Wt.:	16.92		35.14			ERROR MESSAGES	
No. of Blows:			25				
PERCENT MOISTURE:	28.80%		89.62%				
AVERAGE MOISTURE:	28.80%		89.62%				

SAMPLE NO. C	Plastic (Y/N)?	Y

PLASTIC LIMIT INFORMATION			LIQUID LIMIT INFORMATION			TEST RESULTS	
TRIAL NO.:	1	2	1	2	3	LIQUID LIMIT:	90
Tare (Pan) No.:	6		6L			PLASTIC LIMIT:	28
Tare (Pan) Wt.:	14.35		23.76			PLASTIC INDEX:	62
Tare + Wet Soil Wt.:	17.51		48.1			USCS CLASSIFICATION:	CH
Tare + Dry Soil Wt.:	16.82		36.64			ERROR MESSAGES	
No. of Blows:			28				
PERCENT MOISTURE:	27.94%		90.20%				
AVERAGE MOISTURE:	27.94%		90.20%				

SAMPLE NO.	Plastic (Y/N)?	

PLASTIC LIMIT INFORMATION			LIQUID LIMIT INFORMATION			TEST RESULTS	
TRIAL NO.:	1	2	1	2	3	LIQUID LIMIT:	
Tare (Pan) No.:						PLASTIC LIMIT:	
Tare (Pan) Wt.:						PLASTIC INDEX:	
Tare + Wet Soil Wt.:						USCS CLASSIFICATION:	
Tare + Dry Soil Wt.:						ERROR MESSAGES	
No. of Blows:							
PERCENT MOISTURE:							
AVERAGE MOISTURE:							

SAMPLE NO.	Plastic (Y/N)?	

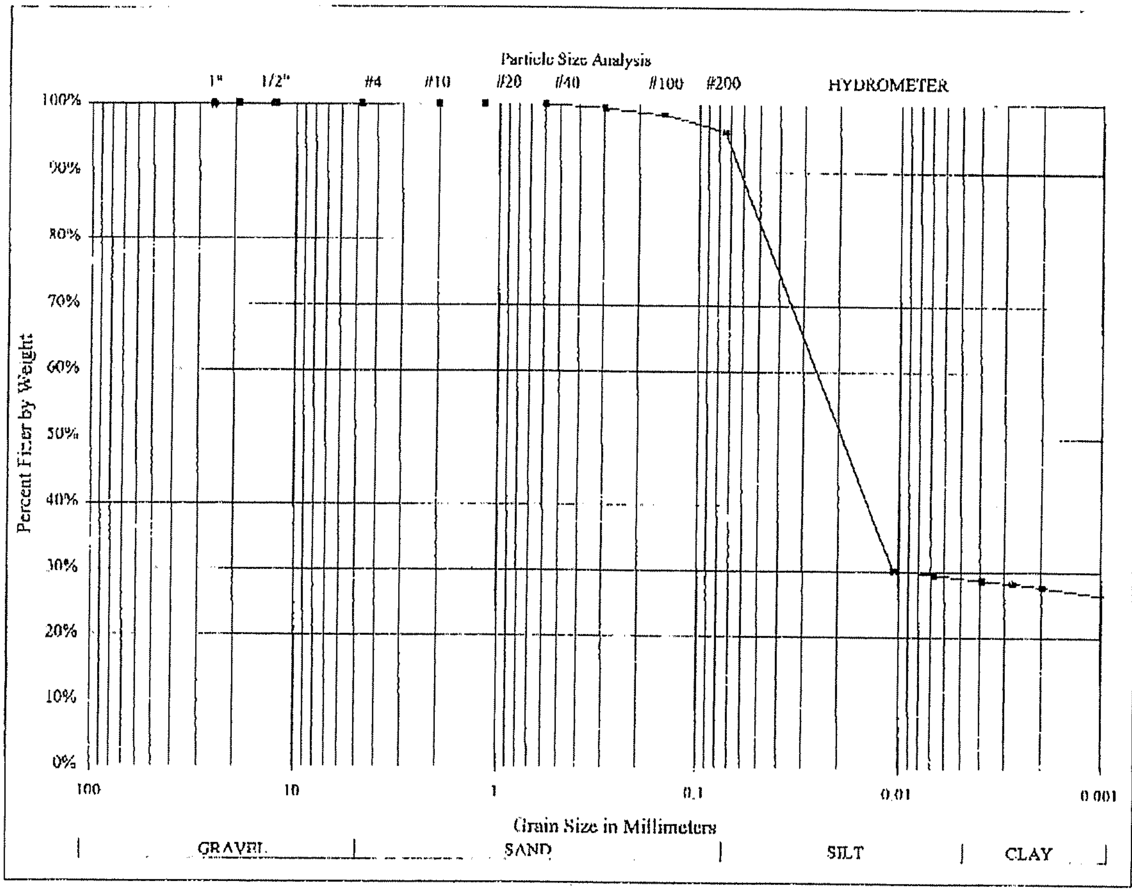
PLASTIC LIMIT INFORMATION			LIQUID LIMIT INFORMATION			TEST RESULTS	
TRIAL NO.:	1	2	1	2	3	LIQUID LIMIT:	
Tare (Pan) No.:						PLASTIC LIMIT:	
Tare (Pan) Wt.:						PLASTIC INDEX:	
Tare + Wet Soil Wt.:						USCS CLASSIFICATION:	
Tare + Dry Soil Wt.:						ERROR MESSAGES	
No. of Blows:							
PERCENT MOISTURE:							
AVERAGE MOISTURE:							

SAMPLE NO.	Plastic (Y/N)?	

PLASTIC LIMIT INFORMATION			LIQUID LIMIT INFORMATION			TEST RESULTS	
TRIAL NO.:	1	2	1	2	3	LIQUID LIMIT:	
Tare (Pan) No.:						PLASTIC LIMIT:	
Tare (Pan) Wt.:						PLASTIC INDEX:	
Tare + Wet Soil Wt.:						USCS CLASSIFICATION:	
Tare + Dry Soil Wt.:						ERROR MESSAGES	
No. of Blows:							
PERCENT MOISTURE:							
AVERAGE MOISTURE:							

SIEVE & HYDROMETER TEST ASTM D422

IME SAMPLE NO :	C	DATE RECEIVED:	5/6/1998
CLIENT:	U.S. Energy	TYPE OF SAMPLE	Bulk
CLIENT SAMPLE NO.:	Sec. 16 Site Soil		
SOIL DESCRIPTION:	Shale/Clay		

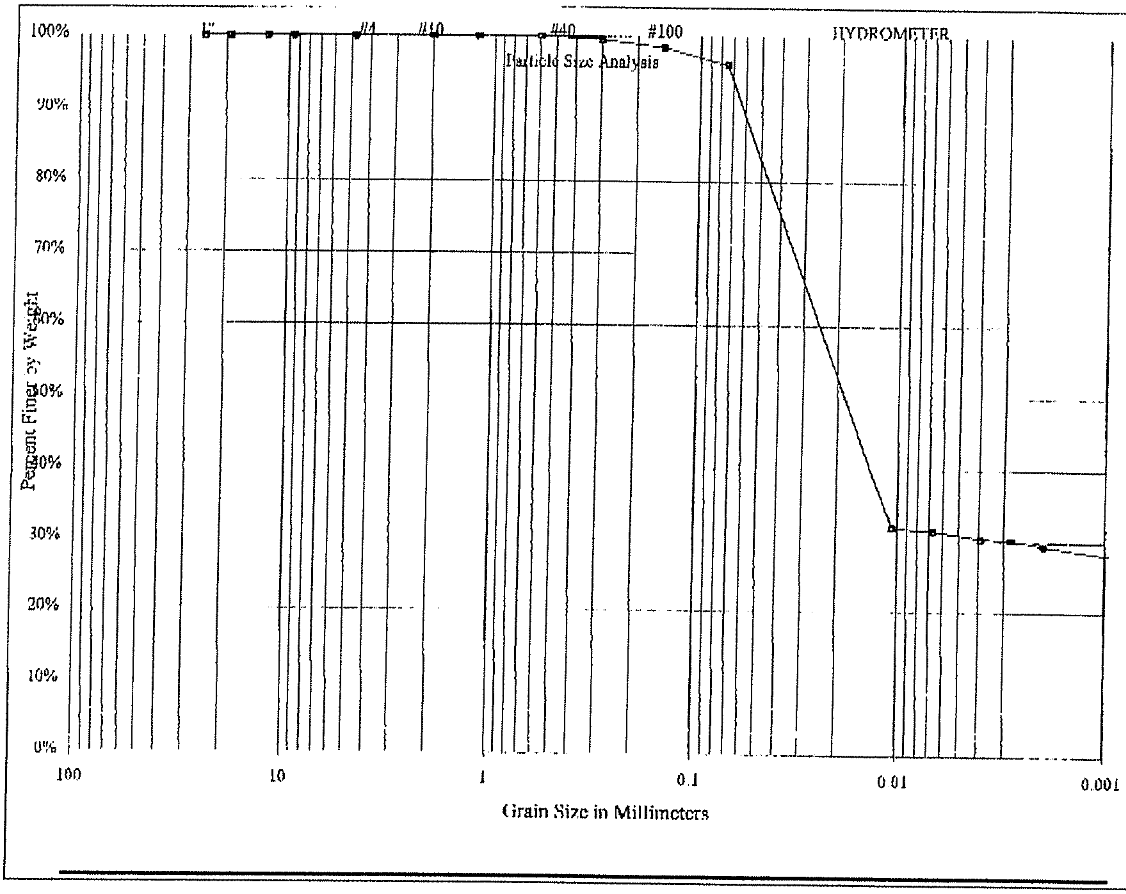


Sieve Size	PARTICLE SIZE (mm)	PERCENT FINER
1"	25.4000	100.0%
3/4"	19.1000	100.0%
1/2"	12.7000	100.0%
3/8"	9.5200	100.0%
NO. 4	4.7600	100.0%
NO.10	2.0000	100.0%
NO. 16	1.1900	100.0%
NO. 30	0.5900	100.0%
NO. 50	0.2970	99.4%
NO. 100	0.1490	98.4%
NO. 200	0.0740	95.9%
Hydrometer Range	0.0106	30.2%
	0.0067	29.5%
	0.0039	28.7%
	0.0028	28.3%
	0.0020	27.7%
	0.0010	26.4%
	0.0004	26.0%

Inberg-Miller Engineers
270 North American Road
Cheyenne, WY 82007

SIEVE & HYDROMETER TEST ASTM D422

IME SAMPLE NO.:	A	DATE RECEIVED:	5/6/1998
CLIENT:	U.S. Energy	TYPE OF SAMPLE	Bulk
CLIENT SAMPLE NO :	Sec. 16 Site Soil		
SOIL DESCRIPTION:	Shale/Clay		



Sieve Size	PARTICLE SIZE (mm)	PERCENT FINER
1"	25.4000	100.0%
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3/8"	9.5200	100.0%
NO. 4	4.7600	100.0%
NO.10	2.0000	100.0%
NO. 16	1.1900	100.0%
NO. 30	0.5900	100.0%
NO. 50	0.2970	99.6%
NO. 100	0.1490	98.5%
NO. 200	0.0740	96.0%
Hydrometer Range	0.0105	31.8%
	0.0067	31.5%
	0.0039	30.5%
	0.0028	30.3%
	0.0020	29.4%
	0.0010	28.1%
	0.0004	27.5%

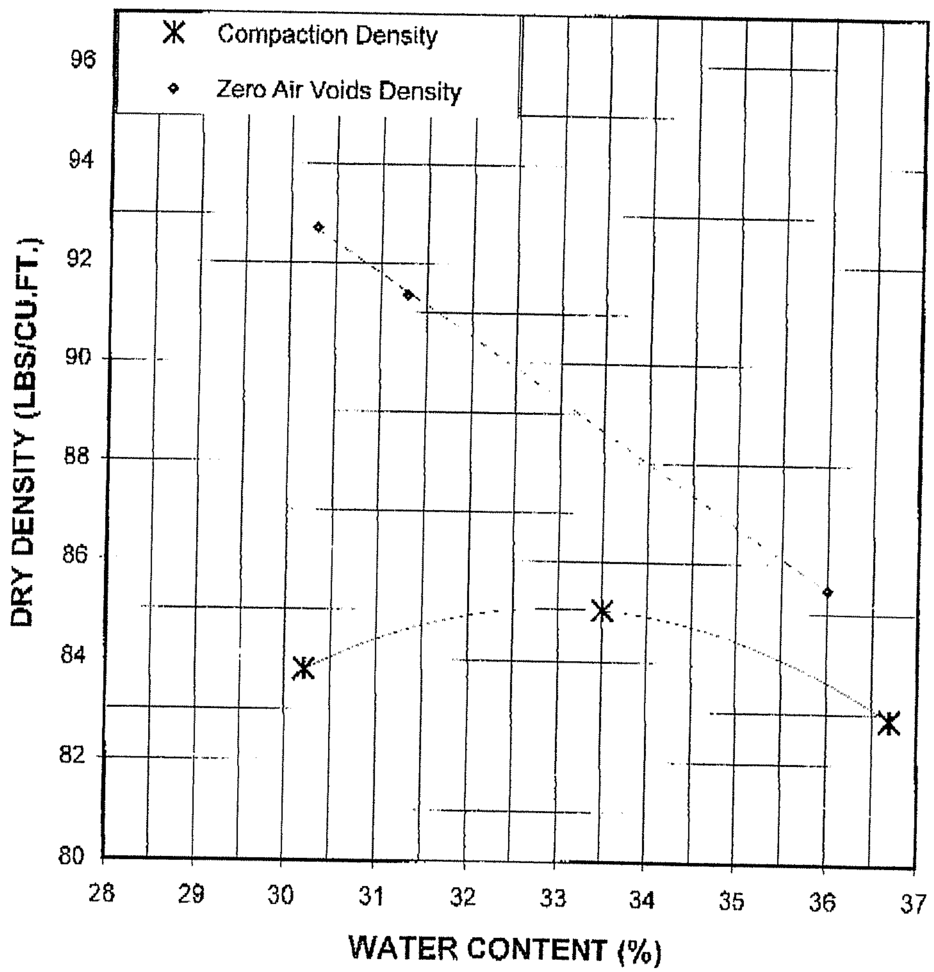
Inberg Miller Engineers
270 North American Road
Cheyenne, WY 82007

MOISTURE-DENSITY ANALYSIS

INBERG-MILLER ENGINEERS

CLIENT: U.S. Energy
PROJECT: Shootering Canyon Mill
JOB NO. 10223 RM
TEST DATE: 6-3-05
SOURCE: Site Soil
DESCRIPTION: Shale/Clay

SAMPLE NO.: Sec. 16, #C
SAMPLED BY: Client
TESTED BY : BJC
TEST METHOD: D 698-A



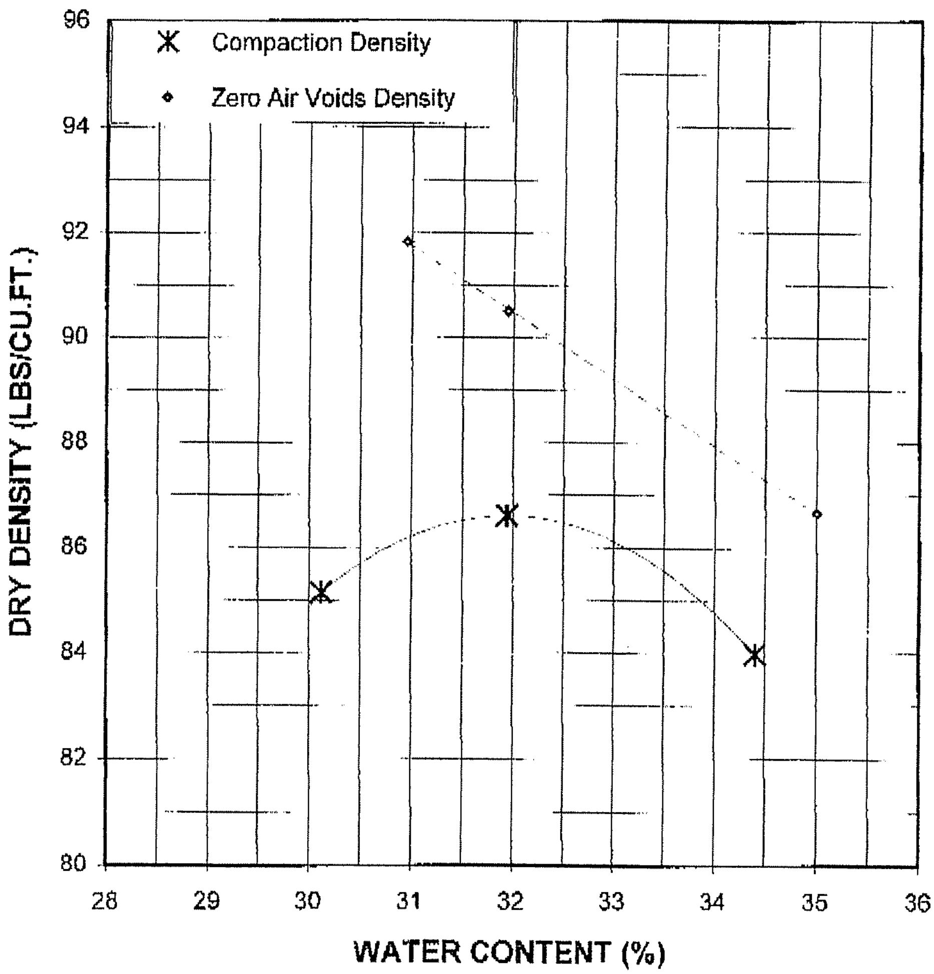
OPTIMUM WATER CONTENT (%): 31.3
MAXIMUM DRY DEN. (LBS/CU. FT): 85.7

MOISTURE-DENSITY ANALYSIS

INBERG-MILLER ENGINEERS

CLIENT: U.S. Energy
PROJECT: Shootering Canyon Mill
JOB NO. 10223 RM
TEST DATE: 6-3-05
SOURCE: Site Soil
DESCRIPTION: Shale/Clay

SAMPLE NO.: Sec. 16, #A
SAMPLED BY: Client
TESTED BY : BJC
TEST METHOD: D 698-A



OPTIMUM WATER CONTENT (%): 32.0
MAXIMUM DRY DEN. (LBS/CU. FT): 86.6

F.2 Discussion of Alternate Source Clay Properties
by Inberg-Miller Engineers, December 7, 2005

December 7, 2005 10223-RM

Mr. Fred Craft
U.S. Energy
877 North 8th West
Riverton, WY 82501

RECEIVED
DEC - 8 2005

RE: SEPTEMBER 19, 2005 SOIL TESTING
SHOOTERING CANYON MILL PROJECT

Dear Fred:

This letter summarizes our observations of the claystone soil sample you submitted for laboratory testing, the results of which were reported on September 19, 2005.

As mentioned in our test report, the sample (as originally submitted) appeared shale-like, but softened rapidly upon inundation with water. Subsequently, moisture-density relationship, particle size analysis including hydrometer analysis, Atterberg Limits and permeability tests were performed. You and your consultant, Hydro Engineers, noted that the particle size analysis test indicated the fine fraction (minus 200 sieve) appeared to be substantially silt-size particles, and that the particle size analysis does not corroborate the relatively low permeability for the sample which was more representative of clay.

There are two observations that we make with regard to your note as presented above:

1. As stated, the sample was processed from hard shale-rock fragments to an apparent relatively soft soil through the addition of water. While the majority of the sample was soil when tested, the disintegration from silt to clay was likely incomplete based on visual and manual observations of variable texture.

2. Hydraulic permeability is controlled by pore size and pore volume of the soil mass through which water flows. Although a substantial portion of a certain soil may include silt through gravel-sized particles, if the soil particles are well graded and there is sufficient clay-sized particles to close the pore space of the larger soil particle fraction (soil matrix), soil pore size and pore volume may be reduced to that of the clay and render clay-like permeability test results.

Based on the above observations, it is our opinion the hydrometer analysis is not a good indicator of hydraulic permeability for the subject sample.

Please feel free to call if you have questions or require further information.

Sincerely,

INBERG-MILLER ENGINEERS

Glen M. Bobnick, P.E.
Geotechnical Engineer
Riverton Office

GMB:bjh:10223/10223 test observ. Ltr 12-07-05

APPENDIX G

REDUCED-MOISTURE TAILINGS EVALUATION

APPENDIX G

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Golder Paste Technology Ltd.

1010 Lorne Street
Sudbury, Ontario, Canada P3C 4R9
Telephone: (705) 524-5533
Fax: (705) 524-9636

REPORT ON

CONCEPTUAL STUDY FOR SURFACE DISPOSAL OF URANIUM TAILINGS AT THE SHOOTARING CANYON MILL GARFIELD COUNTY, UTAH

Submitted to:

Mr. Fredrick R. Craft, V.P. Engineering
Plateau Resources Limited
877 N, 8th St. W.
Riverton, WY
82501
USA

DISTRIBUTION:

1	Copy	- Plateau Resources Limited, Riverton, WY, USA
1	Electronic Copy	- Plateau Resources Limited, Riverton, WY, USA
2	Copies	- Golder Paste Technology Ltd., Sudbury, Ontario

February 22, 2007 06-1900-042 - B

EXECUTIVE SUMMARY

Plateau Resources Limited (Plateau) has retained Golder Paste Technology Ltd. (PasteTec) to investigate, at a conceptual level, the use of paste technology to produce Reduced Moisture Tailings (RMT) for the surface disposal of uranium tailings at the Shootaring Canyon Mill.

The Shootaring processing facility, designed for 1,000 tons of ore per day, has been in suspended operational mode since 1982. However, Plateau is in the process of moving forward with the Utah Division of Radiation Control, under a Memorandum of Agreement to transfer the Shootaring Mill license from "reclamation" to "full operational" status.

Since no material was available for testing, it was assumed that the Shootaring tailings will make a paste. Depending on the properties of the uranium tailings and the required deposition parameters, two process options, each producing a different wt% solid paste product, are available for the handling of the Shootaring Uranium Mill tailings using paste technology:

(1) Paste via thickening; and

(2) Paste via filtering.

Based on PasteTec's experience with other tailings, having similar properties as described to us by Plateau, paste technology is a viable option here; however, it is recommended that material laboratory testing be completed in the next phase in order to ascertain the ability of the Shootaring tailings to make paste and be transportable by pipeline.

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1.0 INTRODUCTION

Plateau Resources Limited (Plateau) has retained Golder Paste Technology Ltd. (PasteTec) to investigate the use of paste technology for the surface disposal of uranium tailings at the Shootaring Canyon Mill. Based on conversations with Plateau, PasteTec will explore, at a conceptual level, the opportunities offered by paste technology for allowing Reduced Moisture Tailings (RMT) placement on surface.

Plateau is in the process of moving forward with the Utah Division of Radiation Control, under a Memorandum of Agreement to transfer the Shootaring Mill license from "reclamation" to "full operational" status. The property has been in suspended operational mode since 1982.

The Shootaring Canyon Uranium Processing Facility is located in Garfield County in Southeastern Utah. The processing facility is designed to process 1,000 tons of ore per day. The ore processed is principally sandstone, obtained from various mines in the area. The property includes a mill processing facility and a designated Tailings Management Area (TMA).

2.0 BACKGROUND INFORMATION

2.1 Operational Status

Plateau began commercial operations at the Shootaring Mill in 1982, but, due to the decline in the market for uranium concentrate, suspended operations after having operated for only a few months. The facility was then placed on a standby basis and cleanup operations completed.

A recent market analysis by Plateau indicated that due to the current favorable uranium market, it is feasible to resume operations as soon as approval of the License Amendment is obtained from the State of Utah and the mill and associated equipment and facilities are fully restored and functional.

2.2 Mill Process Description

The processing facility is designed to process approximately 1,000 tons of ore per day. The ore to be processed is principally sandstone obtained from various mines in the region. The ore will be ground to sand-size particles. The uranium minerals will be leached from the ore by a conventional sulfuric acid leach process. Soluble uranium is recovered with the decanted liquid in countercurrent decantation (CCD) tanks. The decanted liquid proceeds to the solvent extraction circuit to concentrate the uranium. The uranium rich organic solvent is advanced to the stripping operation. This stripped pregnant solution is directed to the precipitation circuit which produces the yellow cake (U_3O_8). This uranium concentrate is then dried and packaged.

2.3 Tailings Management Area

The tailings are the slurry discharged from the CCD system. The disposal of the tailings is by permanent storage in a lined cell that utilizes a natural depression enclosed by a dam and located adjacent to the plant site. This envisaged tailings management focuses on reduced moisture tailings placement. The scenarios currently envisaged by Plateau include:

- The use of a belt filter or similar liquid extraction equipment located at or near the TMA. This would yield moist tailings solids for placement in the lined cells and a tailings solution stream which would be recycled back to the mill circuit, used for dust control in the tailings facility, and/or evaporated; and
- Another tailings liquid separation method being considered is to filter the tailings slurry at the CCD circuit and transport the tailings solids to the lined cells. The tailings solution would then either be recycled or transported to the solution storage/evaporation pond.

3.0 ASSUMPTIONS AND LIMITATIONS

In completing this conceptual study, PasteTec has relied on information provided by Plateau and information from our database of similar projects. It should be noted that:

- At the time of this report, no information is available on the material properties of the ore that will make up the feed to the Shootaring Mill;
- It is assumed at this point that the Shootaring tailings will make a paste, however, no material was available for testing so this must be verified at a later stage;
- The information regarding size distribution, flocculation and settling rates were derived from a 1980 report “Amenability of the Tony M Ore To a Two-Stage Acid Leaching Solvent Extraction Process”;
- The Tony M deposit is adjacent to the Shootaring Mill, however, it may not be part of the blend making up the mill feed; and
- Plateau has indicated that the design thickening process decided upon may be strongly influenced by environmental and regulatory factors.

4.0 ENVIRONMENTAL AND REGULATORY REQUIREMENTS

Based on discussion and review of existing documentation, there are a number of environmental and regulatory factors that need to be considered when assessing the suitability of paste technology for the disposal of the uranium tailings from the Shootaring Canyon Mill. These are listed below and will form part of the basis for the evaluation.

- The tailings produced must be non-segregating with little water bleed. This is to minimize drainage collection and maximize the use of recycled water while at the same time limiting the future consolidation of the tailings; and
- The tailings must be confined within the tailings management facility. Considering the local climate (i.e. semi arid conditions with the occasional major storm event), the deposition method must minimize the real or perceived carryover of radioactive material as a result of wind and/or water erosion.

5.0 PASTE PLANT

The main component of the study consists of the conceptual paste plant design, although some mention will be made of the deposition methodology.

Two (2) options are available in terms of process design for the handling of the Shootaring Uranium Mill tailings using paste technology:

(1) Paste via thickening; and

(2) Paste via filtering.

The required deposition parameters, based on environmental and regulatory requirements, will help drive the product requirements from the paste plant.

5.1 Design Criteria

A summary of the basic design criteria developed by PasteTec and Plateau can be found in Table 1 and the complete list can be found in Appendix A.

TABLE 1
OPERATIONAL AND SITE DATA

SHOOTARING CANYON MILL	
MILL FEED (ORE BLEND SIX DIFFERENT MINING DISTRICTS	
Ore (Specific Gravity)	Sandstone Matrix
Composition	Silica ~ (25 to 35%)
	Clay ~ (5 to 30%)
MILL (TAILINGS PRODUCTION)	
Design Daily Production Rate	1,000 tons/day
Design Hourly Throughput	42 t/hr
Design Annual Throughput	350,000 tons/yr
Yearly Operating Days	350 days
Daily Operating Hours	24 hrs

SHOOTARING CANYON MILL	
TAILINGS PROPERTIES	
Size Distribution	
- Out of Grinding Circuit	100% Passing 600 microns
- Fines Content	~20 % Passing 38 microns
Discharge CCD	49 wt % solids
Discharge High Rate Thickener	65 wt % solids (PasteTec)
Discharge Paste Thickener	70 wt % solids (PasteTec)
Discharge From Filtration	85 wt % solids (PasteTec)
Tailings Solution	pH 1.5 to 3.0
SETTLING AND THICKENING CHARACTERISTICS	
Flocculants	Percol 351 / Superfloc 127
First Stage CCD	
- Flocculant Dosage	0.11 lb/ton (55 g/tonne)
- Wt % Solids	Feed (~22%) Discharge (~60%)
- Unit Area	2.2 – 2.5 ft2/tpd (0.225 – 0.256 m^2/tonne/d)
Second Stage CCD	
- Flocculant Dosage	0.24 lb/ton (120 g/tonne)
- Wt % Solids	Feed (~24%) Discharge (~59%)
- Unit Area	1.8 to 2.4 ft2/tpd (0.184 – 0.246 m^2/tonne/d)
PROPOSED PLANT LOCATION	
Elevation (Above Sea Level)	
- Mill	4,550 Feet (1,387 m)
- Tailings Stack Final Elevation	4,510 Feet (1,375 m)

5.2 Site Location

It is understood that the proposed paste plant would be located adjacent to the existing mill. This location would take advantage of the existing consolidated base and would limit the distances to the mill for services such as water, electricity and allow coverage of the paste plant with little or no additional manpower requirement.

Once the material properties of the tailings are known it will be worthwhile to revisit and trade-off the capital and operating cost associated with the location of the paste plant. Having the paste plant closer to the tailings area would shorten the pumping distance and could greatly reduce the capital and operating costs associated with pumping the tailings to the TMA (centrifugal versus PD pumps).

5.3 Paste Plant

As described earlier in the report, there are two different alternatives for the paste plant design; paste production via thickening or filtering. The difference between the options is the final end product. A description of the processes can be found in the following sections:

5.3.1 Option 1: Paste via Thickening

The mill tailings will be delivered as a slurry to the paste plant and fed to an agitated surge tank. The surge tank is required in case the flow from the mill is inconsistent in terms of volume flowrate and % solids.

From the agitated surge tank, the tailings would be pumped into a paste thickener. Flocculant would be added to aid in the settling of the solids. The overflow from the paste thickener will be sent to a process water tank to be recycled in the mill. The thickener underflow would be gravity fed to a Positive Displacement (PD) pump which would then pump the material out to the TMA.

The key element of this option is the paste thickener, which can achieve, with the addition of the right flocculants, rapid settling and a denser slurry material compared to other types of thickeners. This is due in part to the shape of the thickener and the storage mechanism that promotes self weight consolidation and dewatering channels to relieve the excess pressure in the settled solids allowing the reaching of greater slurry densities in the range of 70% solids by weight.

5.3.2 Option 2: Paste via Filtering

The mill tailings will be delivered as a slurry to the paste plant and fed to an agitated surge tank. The surge tank and flocculant addition remain as described above.

From the agitated surge tank, the tailings will be pumped into a high rate thickener. The overflow will be sent to the process water tank to be recycled in the mill. From the thickener, the underflow would flow to the filter feed tank.

From the filter feed tank, the underflow material will be fed into the disc filter and the filter cake will drop onto a reversible conveyor. The conveyor will transport the cake to a continuous mixer where the filter cake is re-pulped to the appropriate density for pump/pipeline transport. From the continuous mixer, the paste is gravity fed to a gob hopper which feeds a positive displacement pump which then pumps the material out to the TMA.

In the case of operational difficulties or maintenance issues, the filter cake can be stockpiled off of the filter conveyor and hauled by truck to the TMA.

5.4 Deposition Methodology

It is understood by PasteTec that the tailings deposition will occur within two lined cells. It is also understood that the deposition areas have a finite footprint and stacking angles of the tailings material, in order to increase storage capacity, will be a key driver in the option chosen by Plateau. While the TMA deposition design is outside of the scope of this study, typically paste tailings are deposited in thin layers to promote desiccation. The slope angle that can be obtained using thin layer deposition is material and process dependent and will need to be determined by laboratory testing. In this case, Option 2 paste via filtering will likely produce the densest material which could result in greater stacking angles.

6.0 ADVANTAGES OF THICKENED TAILINGS OR PASTE

Listed below are the advantages that paste technology is currently achieving at other facilities, and pending successful testwork, could be realized by Plateau for its tailings management.

- Little or no ponded water on top of tailings;
- Less tailing solution to manage; recycling can take place at the mill;
- Little or no liquid / solid separation and no segregation;
- Higher placement density than conventional slurry deposition;
- Greater chemical stability, improved erosion resistance;
- Lower seepage potential;
- Higher potential stacking angle of tailings;
- Potential of encapsulating other materials such as process water salt precipitates in the paste;
- More accessible to foot traffic and equipment; and
- Facilitates progressive closure.

Appendix B provides additional information about paste and process and equipment selection. Also included within Appendix B is a Table that summarizes some of the potential benefits that paste technology could provide in light of the regulatory and environmental requirements listed in Section 4 for different tailing slurry densities. Options D and E of the Table are respectively Options 1 and 2 referred to within this report. Any potential benefit would need to by verified by laboratory testing of the tailings material keeping in mind that in the results obtained in the field can be affected by a wide range of factors including, effective operation of the paste plant, weather conditions, temperature variations, shear effects during pumping, rate of deposition, and the nature of the material upon which the material is deposited.

7.0 CONCLUSIONS

Option 1 (Paste via Thickening) is likely the preferred option as it would minimize capital and operating costs. The key element of this option is the possibility of obtaining the required material densities directly from the paste thickener underflow.

Due to the importance of maximizing the deposition angles should the desired material properties not be achievable with Option 1, Option 2 (Paste via Filtering), is presented as an alternative. By filtering part or all of the thickener underflow a higher density material can be achieved.

It is not known at this point, because of the lack of material availability, which will prove to be the most appropriate technical solution for the Shootaring tailings.

In future stages of work, material testing must be completed in order to characterize the tailings in terms of their performance as a paste and the potential for pipeline transport and deposition in the current TMA.

8.0 CLOSURE

We trust that you find this report satisfactory. Should you have any questions, please do not hesitate to contact the undersigned.

GOLDER PASTE TECHNOLOGY LTD.

P. Primeau

Pierre Primeau, P.Eng.
Project Engineer

David Landriault, P.Eng.
Principal/President

PP/BF/FP/ns

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Golder Associates PasteTec

APPENDIX A

DESIGN CRITERIA

Project No.	06-1900-042	**APPENDIX A** **SHOOTARING CANYON MILL** **CONCEPTUAL STUDY SURFACE DISPOSAL** **BASIC DESIGN CRITERIA**	Golder Associates PasteTec
Date:	Feb-07		
Revision Number:	0		

DEFINITION: Nominal values are based on the annual planned mill throughput averaged over 365 days per year. The nominal values are used to size the disposal facility. The design values are larger and take into account the utilization/availability of the mill and a general design factor. The design values are used to size pipelines and pumping systems with an appropriate factor of safety.

General Information

Name of mine	Shootaring Canyon Mill
U/g Backfill or Surface Disposal	Surface Disposal
Type: Open Pit or Underground or Both	n/a
Mineral(s):	Uranium

	Value	Units	Source	Equivalent Units
Mill Production				
Days per year	350	days/yr	Plateau	
Days per week	7	days/wk	Plateau	
Hours per day	24	hrs/day	Plateau	
Mill design tonnage or capacity	1000	tons/day	Plateau	
Mill availability (*% of the year that the mill is available to operate - usually 90 to 95%*)	95	%	Golder	
Mill utilization (*% of the availability that the mill is actually running - usually 90-95%*)	95	%	Golder	
Tailings Production				
Tailings / ore ratio (*the difference between the ore and the tailings is concentrate*)	1	-	Golder	
Total Tailings Production	350000	tons/yr	Plateau	
Discharge slurry density from the mill (*S1*) (*% solids in total mass of solids and water*)	49	wt.% solids	Plateau	
Discharge paste via thickening (*S2*) (*% solids in total mass of solids and water*)	70	wt.% solids	Golder	
Discharge paste via filtration (*S3*) (*% solids in total mass of solids and water*)	80	wt.% solids	Golder	
Tailings Properties				
Specific gravity of tailings solids (*sg*)	2.2	-	Plateau	
Assumed void ratio of deposited tailings at final deposition (*vol. of voids / vol. of solids*)		-		
Elevations				
Mill elevation	4550	ft ASL	Plateau	
Paste plant elevation	4550	ft ASL	Plateau	
Paste pipeline discharge elevation **(Final Reclamation Cover Height)**	4510	ft ASL	Plateau	
Excess water pipeline discharge elevation	4510	ft ASL	Plateau	
Distances				
Tailings pipeline to Cell 1	1400	ft	Golder	
Tailings pipeline to Cell 2	1800	ft	Golder	
Process water to mill	150	ft	Golder	
Miscellaneous flows impacting the mill water balance				
Moisture content of the ore going into the mill (*w2*) (*% of dry mass of ore*)	4	%	Plateau	
Moisture in concentrate leaving the mill				
If by truck - moisture content (*w3*) (*% of dry mass of concentrate*)		% water		
OR				
If by pipeline - slurry density (*S2*) (*% solids in total mass*)		% solids		
Fresh make-up water total	107	gpm	Plateau	
Fresh make-up water used in the mill for reagent mixing etc. (*m^3 / ton of ore milled*)	7	gal / ton	Plateau	
Estimated water lost in the mill to evaporation and spillage (*m^3 / ton of ore milled*)		gal / ton		
Miscellaneous flows that could impact the tailings pond flow model				
Water used for dust control (*taken from the pond +* ***partly using potable water***)	25	gpm	Plateau	
Other (*such as mine water that is discharged to the tailings facility*)		gpm		
Evaporation / Precipitation	70 / 7	in/year	Plateau	

Notes:
1) The sources of the information could be either the owner, contractors, Golder or other consultants.
2) Information is only required in the shaded cells (data input cells).

APPENDIX B

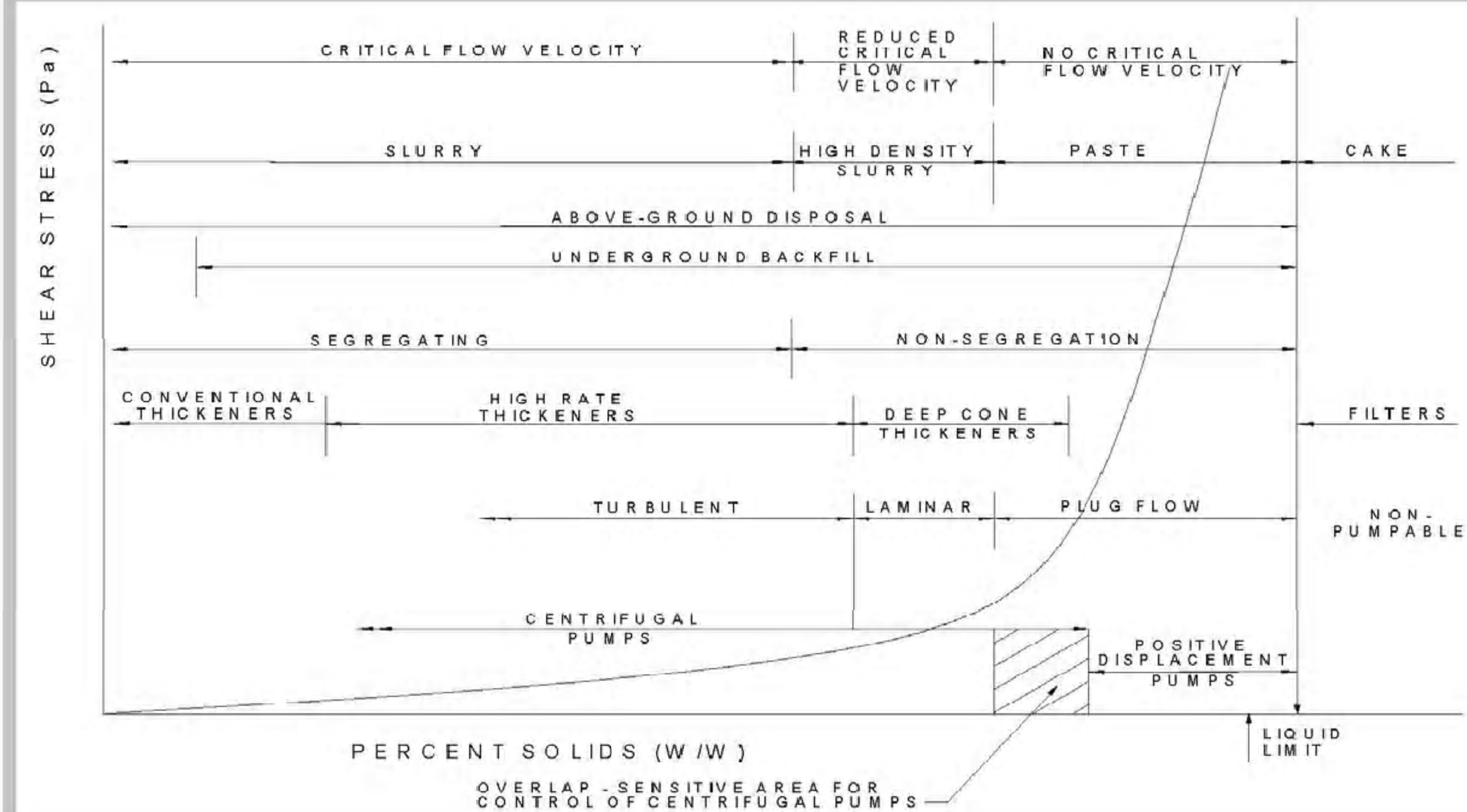
PASTE DEFINITIONS AND EQUIPMENT SELECTION CRITERIA

APPENDIX B
PASTE DEFINITION

Paste is a densified uniform material of such mineralogical and size makeup, that it will bleed only minor quantities of water when at rest, experience minimum segregation and can be moved in a pipeline at line velocities well below that of critical velocities for similar sized materials at lower pulp densities. Paste can remain sitting in a pipeline for extended periods of time when no cementitious material is present, and its slump can be measured. The slump is normally measured using an ASTM 12 inch slump cone, a standard tool used in the concrete industry. Paste can generally be produced from materials with a wide range of size distributions; however, they usually contain a minimum of 15% by weight of minus 635 mesh (20µm) material. Mineralogical makeup is very important, as not all materials within the outlined size distribution may make paste. Hence rheological testing is required.

Figure 1 illustrates an equipment selection guide with pulp wt% solids versus shear strength.

Figure 1: Process and Pumping Equipment Selection Guide based on Pulp wt% solids versus Shear Strength



Project No. 06-1900-042-B
Date: February 2007
Revision Number: 1

APPENDIX B
SHOOTARING CANYON MILL
CONCEPTUAL STUDY USING PASTE TECHNOLOGY FOR SURFACE COMPARISON FOR DIFFERENT TAILING SLURRY DENSITIES [1]

Assessment Criteria →	Cost Consideration	Operating Consideration: Thickened Tailings With Little Bleed	Operating Consideration: Erosion and Wind Blown material	Provides Required Storage Capacity (Slope of Material)
OPTION A Mill (CCD) Slurry 49% Solids by Weight	N/A	Conventional tailings would require large amount of water to be handled in cells.	Material segregation; fines as they dry would be susceptible to wind erosion.	Material cannot be stacked. Would require larger containment dams. (1^o to 2^o)[3]
OPTION B Thickened Slurry 60%[2] Solids by Weight	Conventional thickener with centrifugal pumps. ($)	There would be some water bleed, volume of water greatly reduced.	Material will segregate, fines could become prone to wind erosion.	Material cannot be stacked. Again would require larger structures for containment. (2^o to 3^o)[3]
OPTION C Non-Segregating Thickened Slurry 65%[2] Solids by Weight	High compression thickener with centrifugal pumps. ($$)	Considering climatic conditions there may be little water bleed.	Non-segregating material, crust may form that can be resistant to erosion.	Based on climatic conditions with thin layer deposition could get some stacking. (3^o to 5^o)[3]
OPTION D (Option 1 in Report) Paste Via Thickening 70%[2] Solids by Weight	Paste thickener with positive displacement pumps. ($$$)	Considering climatic conditions there would be little or no water bleed.	Non-segregating material, crust may form that can be resistant to erosion.	Based on climatic conditions with thin layer deposition could pot. get good slope. [4] (5^o to 7^o)[3]
OPTION E (Option 2 in Report) Paste Via Filtering 80%[2] Solids by Weight	High rate thickener with disk filter with positive displacement pumps. ($$$$)	Considering climatic conditions there would be very little or no water bleed.	Non-segregating material, crust may form that can be resistant to erosion.	Based on climatic conditions with thin layer deposition could pot. get good slope. [4] (5^o to 10^o)[3]
OPTION F Filtered Product 85%[2] Solids by Weight	Thickener plus disk filter / belt filter and mobile equipment handling. ($$$$$)	No water bleed.	Material would need to be handled by mobile equipment and would be be prone to erosion. Material may need to be wetted. QA/QC during placement would be a key issue.	Material can be stacked and placed as required to meet final topography. (>10^o)[3]

Notes:
[1] Potential Benefits; [2] Percent Solids; [3] Slope Angle: Actual results can vary significantly since they are highly dependent on a number of factors including:
the actual physical and paste properties of the tailings, the operation and management of the thickening plant and tailings management area and on the climatic conditions at the site.
[4] May increase storage capacity of existing tailings facility and/or limit the need for future expansion

APPENDIX H

COLUMN TESTING LABORATORY REPORT

APPENDIX H

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ENERGY LABORATORIES, INC. • 2393 Salt Creek Highway (82601) • P.O. Box 3258 • Casper, WY 82602
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From: R. A. Garling
Energy Laboratories, Inc. – Casper
To: Fred Craft
Plateau Resources, Ltd.
Date: July 27, 2005
Subject: Column Leach Study Results

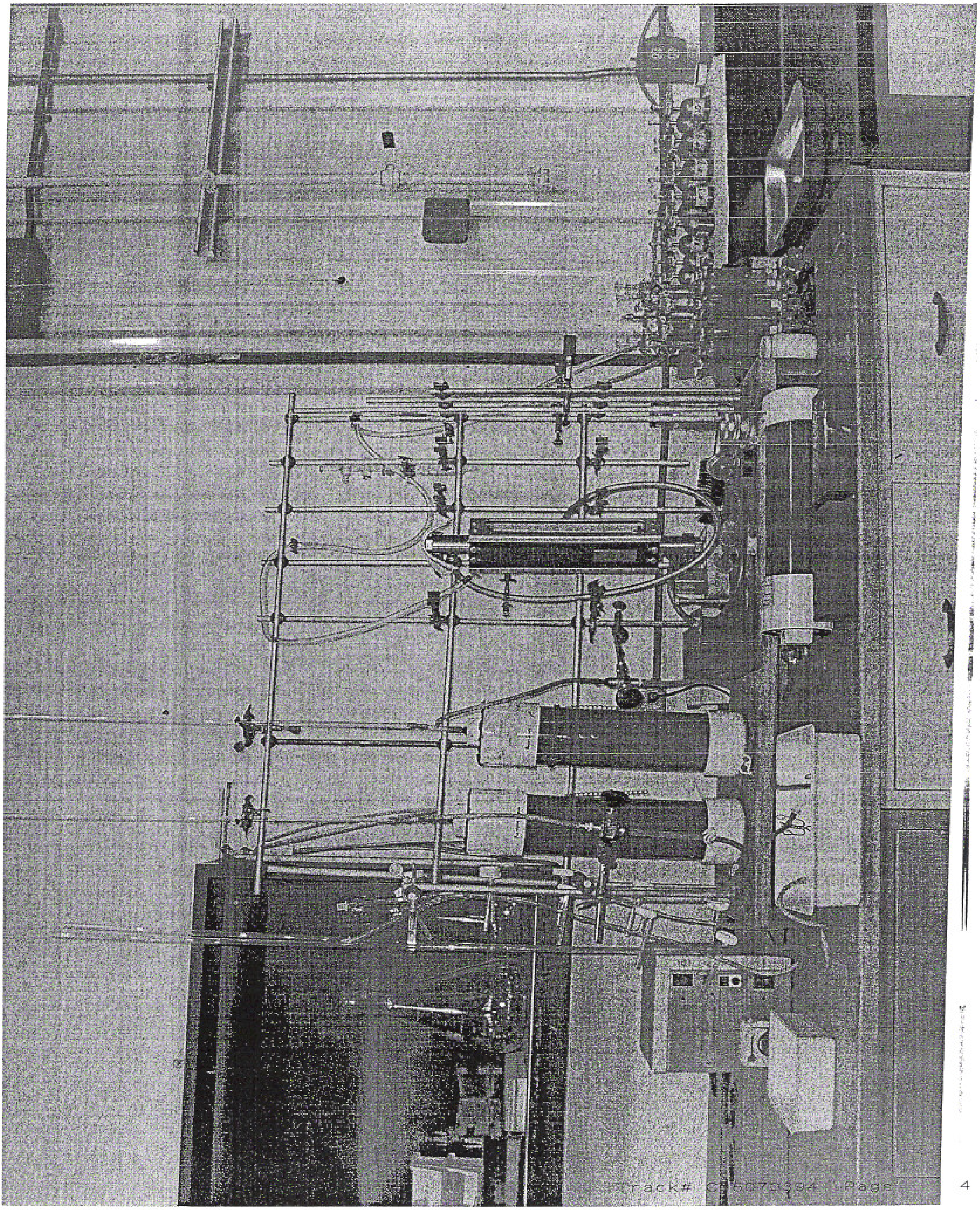
Introduction:

During June 2005 Plateau Resources Ltd. retained Energy Laboratories, Inc.-Casper to perform column leach tests to support development of the Shootaring Canyon Mill tailings regime. Water collected from the mill supply well and samples of fine Entrada sand and rocky soil were delivered to the Casper ELI facility. The column tests were intended to determine the effect of a sulfuric acid synthetic tailings solution on physical and chemical characteristics of the two soil types. Flow of unaltered process water was initiated June 13, 2005 and 4 to 5 pore volumes of effluent were collected prior to starting the sulfuric acid solution flow. Twenty plus pore volumes of the sulfuric acid effluent were collected from 6/27 to 6/30 and analyzed for pH and electrical conductivity. Further analysis on the feedstock, the soil samples and selected acid effluent samples was conducted and is attached to this report.

Test Conditions and Results

1. Water collected from the Shootaring Mill supply well was used both unaltered and adjusted to ~95 g/L H_2SO_4 as the column feed solutions.
2. The two 4" diameter clear PVC columns were loaded with the Entrada sand (called Plateau soil or PS) and the Rocky soil material (called RS). Twelve inches of material was tamped to an estimated 40% porosity which equated to a pore volume of 980 ml. The base of each column was packed with glass wool to assure equal flow and minimized plugging. The two soil types were tested for EC, Lime, pH and PSA/texture and results are attached.
3. The columns were operated upflow to prevent channeling and for flow regulation purposes. Initially, a constant head reservoir was used to provide the flow of unaltered water. Four to five pore volumes of discharge were collected in this fashion. On initiating the 95 g/L sulfuric flow, however, it became impossible to control the flow by head alone. The reason for this was CO_2 offgassing as the acid neutralized the calcite entrained in the soil. Gas plugging required the use of a peristaltic pump to control the feed flow and therefore the column discharge rates. Manometers were installed in the column feed line to monitor the potential of column plugging, one of the reasons for the testing (potential of creating precipitates which would retard fluid flow). Backpressures of 1 to 3 inches of water were observed through the acid feed portion of the test on both columns. The backpressure readings were highly variable due to the effect of the gas generation.

4. Once the peristaltic pump was installed, the column tests operated continuously from 6/27/05 to 6/30/05. All flow was collected using an automated system which sequentially retained ~1 pore volume samples. Average flow for the test was 4.22 ml/min or 6.2 pore volumes/day. The 20+ pore volume samples were analyzed for sulfate, pH and conductivity. Based on the initial analytical results, four samples (two from each column) were collected and analyzed for an expanded suite of analytes.
5. Chemical Analysis Results: The soil analysis revealed that both soils were >75% sand, had significant lime/carbonate concentrations (6-15%), slightly basic pH (probably due to the lime%), and relatively low salt loading. The unadjusted feed water was good quality with TDS of ~230 mg/L. Two samples of column effluent taken prior to the acid addition were analyzed for basic water quality parameters. These showed that the soil in the two columns had no appreciable effect on the feed water quality. The Rocky Soil column was relatively quick to equilibrate with the feed solution showing full sulfate, EC, and pH breakthrough by the fifth pore volume. This may have been due to the lower levels of carbonate in the soil. There was also a mineral present that leached throughout the entire 23-pore volume test and increased sulfate and EC levels above that of the feed solution. Conversely, the Plateau Soil column had not fully equilibrated to the feed solution by the 21st pore volume. Sulfate and EC values remained suppressed and pH values continued to show neutralization. This may have been due to the ~15% lime/carbonate concentration in the soil. All data reports are attached to this report.



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ANALYTICAL SUMMARY REPORT

July 29, 2005

Fred Craft
Plateau Resources (US Energy)
877 North 8th St
Riverton, WY 82501-

Workorder No.: C05070394

Project Name: Column Tests

Energy Laboratories Inc. received the following 4 samples from Plateau Resources (US Energy) on 7/12/2005 for analysis.

Sample ID	Client Sample ID	Collect Date	Receive Date	Matrix	Test
C05070394-001	PS Acid 4		07/12/05	Aqueous	Metals by ICP/ICPMS, Dissolved Alkalinity QA Calculations Conductivity Fluoride Nitrogen, Ammonia Nitrogen, Nitrite Nitrogen, Nitrate + Nitrite pH Solids, Total Dissolved
C05070394-002	PS Acid 21		07/12/05	Aqueous	Same As Above
C05070394-003	RS Acid 5		07/12/05	Aqueous	Same As Above
C05070394-004	RS Acid 20		07/12/05	Aqueous	Same As Above

There were no problems with the analyses and all data for associated QC met EPA or laboratory specifications except where noted in the Case Narrative or Report.

If you have any questions regarding these tests results, please call.

Report Approved By:

LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Workorder: C05060266

Report Date: 06/17/05
Date Received: 06/07/05

	Analysis	EC, SatPst	Lime as CaCO3	pH, SatPst	Texture	Sand	Silt	Clay	Moisture
	Units	mmhos/cm	%	s_u_		%	%	%	%
Sample ID	Client Sample ID	Results	Results	Results	Results	Results	Results	Results	Results
C05060266-001	Rocky Soil	0.46	6.4	7.9	SL	75	19	6	7.1
C05060266-002	Plateau Soil	0.28	14.9	8.0	LS	83	9	8	4.8

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Shootaring Column Leach Test
Lab ID: C05060184-001
Client Sample ID: Raw Water Subsampled

Report Date: 06/13/05
Collection Date: 06/03/05 16:00
Date Received: 05/19/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
MAJOR IONS							
Carbonate as CO3	ND	mg/L		1		A2320 B	06/08/05 20:47 / slb
Bicarbonate as HCO3	215	mg/L		1		A2320 B	06/08/05 20:47 / slb
Calcium	31.7	mg/L		0.5		E200.7	06/06/05 15:18 / cp
Chloride	4	mg/L		1		A4500-Cl B	06/06/05 14:41 / sl
Magnesium	23.0	mg/L		0.5		E200.7	06/06/05 15:18 / cp
Nitrogen, Nitrate+Nitrite as N	0.1	mg/L		0.1		E353.2	06/07/05 09:51 / jal
Potassium	5.9	mg/L		0.5		E200.7	06/06/05 15:18 / cp
Sodium	30.1	mg/L		0.5		E200.7	06/06/05 15:18 / cp
Sulfate	65	mg/L	D	1		A4500-SO4 E	06/06/05 14:03 / jal
PHYSICAL PROPERTIES							
Conductivity	469	umhos/cm		1.0		A2510 B	06/06/05 08:15 / th
pH	8.20	s.u.		0.01		A4500-H B	06/06/05 08:25 / th
Solids, Total Dissolved TDS @ 180 C	229	mg/L		10		A2540 C	06/07/05 16:43 / th
METALS - TOTAL							
Iron	0.03	mg/L		0.03		E200.7	06/06/05 15:18 / cp
DATA QUALITY							
A/C Balance (± 5)	-0.663	%				Calculation	06/09/05 16:13 / smd
Anions	5.00	meq/L				Calculation	06/09/05 16:13 / smd
Cations	4.93	meq/L				Calculation	06/09/05 16:13 / smd
Solids, Total Dissolved Calculated	266	mg/L				Calculation	06/09/05 16:13 / smd
TDS Balance (0.80 - 1.20)	0.860	dec. %				Calculation	06/09/05 16:13 / smd

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
D - RL increased due to sample matrix interference.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

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LABORATORY ANALYTICAL REPORT

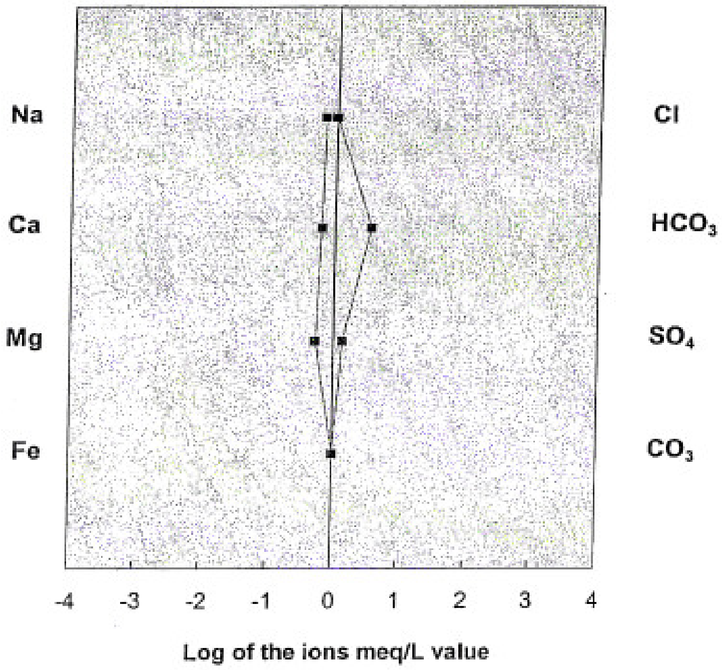
Company:	Plateau Resources (US Energy)	Report Date:	June 13, 2005
Project:	Shootaring Column Leach Test	Collection Date:	06/03/05 16:00
Location:	NOT GIVEN	Date Received:	05/19/05 00:00
Laboratory ID:	C05060184-001	Matrix:	Water
Sample ID:	Raw Water Subsampled		

CATIONS	mg/L	meq/L	ANIONS	mg/L	meq/L
Calcium	32	1.58	Bicarbonate	215	3.53
Magnesium	23	1.89	Carbonate	<1	<0.03
Potassium	6	0.15	Hydroxide	NA	NA
Sodium	30	1.31	Chloride	4	0.11
Iron, Total	0.03	0.00	Sulfate	65	1.36
Barium, Total	NA	NA			
Strontium, Total	NA	NA			
SUM +	91	4.93	SUM -	284	5.00

Cation/Anion Balance, % difference -0.80

Solids			Sample Conditions		
Total Dissolved Solids @ 180°C	229	mg/L	TDS, Calculated	268	mg/L
Total Solids, NaCl equivalents	197	mg/L	pH, s.u.	8.2	s.u.
Chloride as NaCl	6	mg/L	Accuracy	0.43	Sigma
NaCl % of Total Dissolved Solids	0.0	%			
Other Properties					
Calcium Hardness as $CaCO_3$	79	mg/L	Ionic Strength	0.007	μ
Magnesium Hardness as $CaCO_3$	95	mg/L	Specific Gravity, Calc	1.000	g/cc
Total Hardness as $CaCO_3$	174	mg/L	Resistivity, 68°F	23.431	ohm meter

Water Analysis Pattern



Probable Mineral Residue, Dry

Compound	mg/L
Mg(HCO3)2	138
Ca(HCO3)2	128
Na2SO4	104
KCl	8
NaHCO3	4

Plot Above for Na includes K

NOTE: NA indicates not analyzed

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Test
Lab ID: C05060578-004
Client Sample ID: P.S.-4 Raw GW

Report Date: 07/08/05
Collection Date: 06/14/05
Date Received: 06/14/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
MAJOR IONS							
Carbonate as CO3	ND	mg/L		1		A2320 B	06/21/05 13:18 / sl
Bicarbonate as HCO3	199	mg/L		1		A2320 B	06/21/05 13:18 / sl
Calcium	30.2	mg/L		0.5		E200.7	06/15/05 15:45 / cp
Chloride	4	mg/L		1		E200.7	06/15/05 15:45 / cp
Magnesium	20.7	mg/L		0.5		E200.7	06/15/05 15:45 / cp
Potassium	3.4	mg/L		0.5		E200.7	06/15/05 15:45 / cp
Sodium	39.2	mg/L		0.5		E200.7	06/15/05 15:45 / cp
Sulfate	73	mg/L		1		E200.7	06/15/05 15:45 / cp
PHYSICAL PROPERTIES							
Conductivity	463	umhos/cm		1.0		A2510 B	06/14/05 15:57 / th
pH	8.25	s.u.		0.01		A4500-H B	06/15/05 09:31 / th
Solids, Total Dissolved TDS @ 180 C	310	mg/L		10		A2540 C	06/14/05 16:28 / th
METALS - TOTAL							
Iron	ND	mg/L		0.03		E200.7	06/15/05 15:45 / cp
DATA QUALITY							
A/C Balance (± 5)	1.26	%				Calculation	06/22/05 22:24 / smd
Anions	4.88	meq/L				Calculation	06/22/05 22:24 / smd
Cations	5.01	meq/L				Calculation	06/22/05 22:24 / smd
Solids, Total Dissolved Calculated	268	mg/L				Calculation	06/22/05 22:24 / smd
TDS Balance (0.80 - 1.20)	1.16	dec. %				Calculation	06/22/05 22:24 / smd

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

LABORATORY ANALYTICAL REPORT

Company:	**Plateau Resources (US Energy)**	Report Date:	**July 8, 2005**
Project:	**Column Test**	Collection Date:	**06/14/05**
Location:	**NOT GIVEN**	Date Received:	**06/14/05 15:00**
Laboratory ID:	**C05060578-004**	Matrix:	**Water**
Sample ID:	**PS-4 Raw GW**		

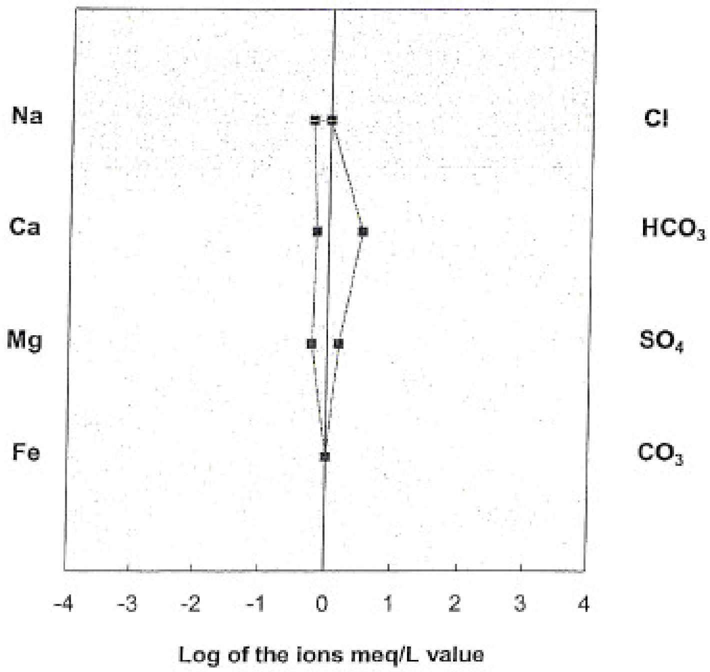
CATIONS	mg/L	meq/L	ANIONS	mg/L	meq/L
Calcium	30	1.51	**Bicarbonate**	199	3.26
Magnesium	21	1.70	**Carbonate**	<1	<0.03
Potassium	3	0.09	**Hydroxide**	NA	NA
Sodium	39	1.70	**Chloride**	4	0.10
Iron, Total	<0.03	<0.01	**Sulfate**	73	1.52
Barium, Total	NA	NA			
Strontium, Total	NA	NA			
SUM +	**94**	**5.00**	**SUM -**	**276**	**4.88**

Cation/Anion Balance, % difference **1.21**

Solids			**Sample Conditions**		
Total Dissolved Solids @ 180°C	310	mg/L	TDS, Calculated	270	mg/L
Total Solids, NaCl equivalents	200	mg/L	pH, s.u.	8.25	s.u.
Chloride as NaCl	6	mg/L	Accuracy	-0.66	Sigma
NaCl % of Total Dissolved Solids	2.0	%			

Other Properties					
Calcium Hardness as $CaCO_3$	75	mg/L	Ionic Strength	0.007	μ
Magnesium Hardness as $CaCO_3$	85	mg/L	Specific Gravity, Calc	1.000	g/cc
Total Hardness as $CaCO_3$	161	mg/L	Resistivity, 68°F	23.734	ohm meter

Water Analysis Pattern



Probable Mineral Residue, Dry

Compound	mg/L
Na2SO4	125
Mg(HCO3)2	125
Ca(HCO3)2	122
NaCl	6
NaHCO3	4

Plot Above for Na includes K

NOTE: NA indicates not analyzed

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Test
Lab ID: C05060578-009
Client Sample ID: R.S.-5 Raw GW

Report Date: 07/08/05
Collection Date: 06/15/05
Date Received: 06/14/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
MAJOR IONS							
Carbonate as CO3	3	mg/L		1		A2320 B	06/21/05 13:28 / sl
Bicarbonate as HCO3	199	mg/L		1		A2320 B	06/21/05 13:28 / sl
Calcium	33.0	mg/L		0.5		E200.7	06/20/05 14:21 / ts
Chloride	7	mg/L		1		E200.7	06/20/05 14:21 / ts
Magnesium	23.0	mg/L		0.5		E200.7	06/20/05 14:21 / ts
Potassium	6.6	mg/L		0.5		E200.7	06/20/05 14:21 / ts
Sodium	38.0	mg/L		0.5		E200.7	06/20/05 14:21 / ts
Sulfate	66	mg/L		1		E200.7	06/20/05 14:21 / ts
PHYSICAL PROPERTIES							
Conductivity	470	umhos/cm		1.0		A2510 B	06/16/05 09:46 / th
pH	8.37	s.u.		0.01		A4500-H B	06/16/05 10:30 / th
Solids, Total Dissolved TDS @ 180 C	286	mg/L		10		A2540 C	06/16/05 00:00 / th
METALS - TOTAL							
Iron	ND	mg/L		0.03		E200.7	06/20/05 14:21 / ts
DATA QUALITY							
A/C Balance (± 5)	4.69	%				Calculation	06/25/05 22:32 / smd
Anions	4.88	meq/L				Calculation	06/25/05 22:32 / smd
Cations	5.36	meq/L				Calculation	06/25/05 22:32 / smd
Solids, Total Dissolved Calculated	292	mg/L				Calculation	06/25/05 22:32 / smd
TDS Balance (0.80 - 1.20)	0.980	dec. %				Calculation	06/25/05 22:32 / smd

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

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LABORATORY ANALYTICAL REPORT

Company:	**Plateau Resources (US Energy)**	Report Date:	**July 8, 2005**
Project:	**Column Test**	Collection Date:	**06/15/05**
Location:	**NOT GIVEN**	Date Received:	**06/14/05 15:00**
Laboratory ID:	**C05060578-009**	Matrix:	**Water**
Sample ID:	**R.S.-5 Raw GW**		

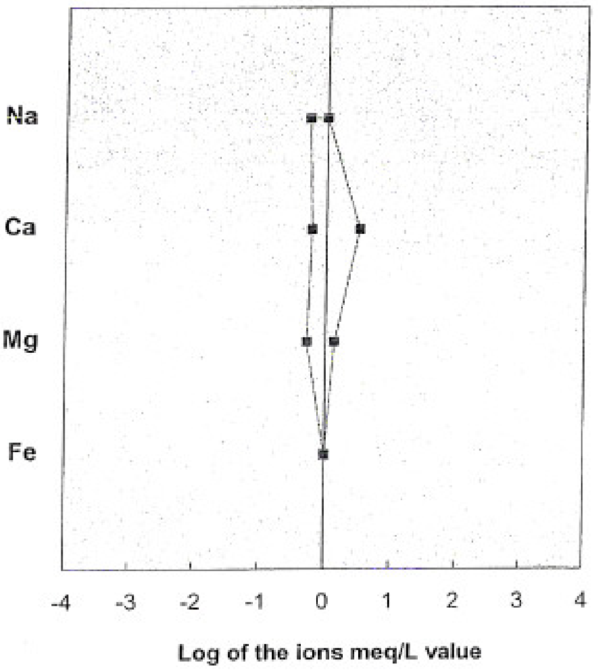
CATIONS	mg/L	meq/L	ANIONS	mg/L	meq/L
Calcium	33	1.65	**Bicarbonate**	199	3.26
Magnesium	23	1.89	**Carbonate**	3	0.09
Potassium	7	0.17	**Hydroxide**	NA	NA
Sodium	38	1.65	**Chloride**	7	0.18
Iron, Total	<0.03	<0.01	**Sulfate**	66	1.37
Barium, Total	NA	NA			
Strontium, Total	NA	NA			
SUM +	**101**	**5.36**	**SUM -**	**274**	**4.90**

Cation/Anion Balance, % difference 4.48

Solids			Sample Conditions		
Total Dissolved Solids @ 180°C	286	mg/L	TDS, Calculated	272	mg/L
Total Solids, NaCl equivalents	206	mg/L	pH, s.u.	8.37	s.u.
Chloride as NaCl	11	mg/L	Accuracy	-2.52	Sigma
NaCl % of Total Dissolved Solids	3.8	%			

Other Properties					
Calcium Hardness as $CaCO_3$	82	mg/L	Ionic Strength	0.008	μ
Magnesium Hardness as $CaCO_3$	95	mg/L	Specific Gravity, Calc	1.000	g/cc
Total Hardness as $CaCO_3$	177	mg/L	Resistivity, 68°F	23.381	ohm meter

Water Analysis Pattern



Probable Mineral Residue, Dry

Compound	mg/L
Ca(HCO3)2	133
Mg(HCO3)2	118
Na2SO4	90
MgSO4	17
NaCl	11

Plot Above for Na includes K

NOTE: NA indicates not analyzed

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05060578-010
Client Sample ID: Acid Feed

Report Date: 07/08/05
Collection Date: 06/22/05
Date Received: 06/14/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
MAJOR IONS							
Carbonate as CO3	ND	mg/L		1		A2320 B	06/23/05 16:25 / sl
Bicarbonate as HCO3	ND	mg/L		1		A2320 B	06/23/05 16:25 / sl
Calcium	29.0	mg/L		0.5		E200.7	06/23/05 15:34 / cp
Chloride	4	mg/L		1		E200.7	06/23/05 15:34 / cp
Magnesium	18.8	mg/L		0.5		E200.7	06/23/05 15:34 / cp
Potassium	5.8	mg/L		0.5		E200.7	06/23/05 15:34 / cp
Sodium	28.8	mg/L		0.5		E200.7	06/23/05 15:34 / cp
Sulfate	23300	mg/L	D	300		A4500-SO4 E	06/23/05 14:20 / jal
PHYSICAL PROPERTIES							
Conductivity	108000	umhos/cm		1.0		A2510 B	06/23/05 11:55 / th
pH	0.85	s.u.		0.01		A4500-H B	06/23/05 15:51 / th
Solids, Total Dissolved TDS @ 180 C	23400	mg/L		10		A2540 C	06/28/05 15:13 / th
METALS - TOTAL							
Iron	0.03	mg/L		0.03		E200.7	06/23/05 15:34 / cp

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
D - RL increased due to sample matrix interference.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

LABORATORY ANALYTICAL REPORT

Company:	**Plateau Resources (US Energy)**	Report Date:	**July 8, 2005**
Project:	**Column Tests**	Collection Date:	**06/22/05**
Location:	**NOT GIVEN**	Date Received:	**06/14/05 15:00**
Laboratory ID:	**C05060578-010**	Matrix:	**Water**
Sample ID:	**Acid Feed**		

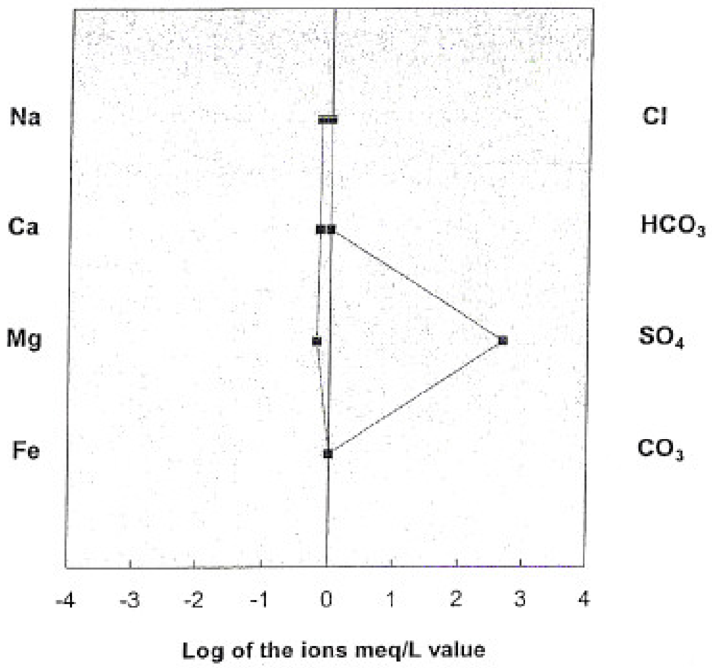
CATIONS	mg/L	meq/L	ANIONS	mg/L	meq/L
Calcium	29	1.45	**Bicarbonate**	<1	<0.02
Magnesium	19	1.55	**Carbonate**	<1	<0.03
Potassium	6	0.15	**Hydroxide**	NA	NA
Sodium	29	1.25	**Chloride**	4	0.12
Iron, Total	0.03	0.00	**Sulfate**	23,297	485.04
Barium, Total	NA	NA			
Strontium, Total	NA	NA			
SUM +	**82**	**4.40**	**SUM -**	**23,301**	**485.16**

Cation/Anion Balance, % difference **-98.20**

Solids			Sample Conditions		
Total Dissolved Solids @ 180°C	23,400	mg/L	TDS, Calculated	23,384	mg/L
Total Solids, NaCl equivalents	11,729	mg/L	pH, s.u.	0.85	s.u.
Chloride as NaCl	7	mg/L	Accuracy	63.04	Sigma
NaCl % of Total Dissolved Solids	0.0	%			

Other Properties					
Calcium Hardness as $CaCO_3$	72	mg/L	Ionic Strength	0.489	μ
Magnesium Hardness as $CaCO_3$	77	mg/L	Specific Gravity, Calc	1.016	g/cc
Total Hardness as $CaCO_3$	150	mg/L	Resistivity, 68°F	0.102	ohm meter

Water Analysis Pattern



Probable Mineral Residue, Dry

Compound	mg/L
Na2SO4	104
CaSO4	98
MgSO4	93
KCl	9

Plot Above for Na includes K

NOTE: NA indicates not analyzed

tae: r:\clients2005\plateau_resources\roa\c05060578-010r.xls

LABORATORY ANALYTICAL REPORT

Client:	Plateau Resources (US Energy)	**Report Date:**	07/08/05
Project:	Column Test	**Date Received:**	06/14/05
Workorder:	C05060578		

	Analysis	SO4	EC	pH
	Units	mg/L	umhos/cm	s_u_
Sample ID	**Client Sample ID**	**Results**	**Results**	**Results**
C05060578-001	P.S.-1 Raw GW		470	8.22
C05060578-002	P.S.-2 Raw GW		467	8.18
C05060578-003	P.S.-3 Raw GW		465	8.23
C05060578-004	P.S.-4 Raw GW		463	8.25
C05060578-010	Acid Feed	23300	108000	0.85
C05060578-012	P.S. Acid 1	347	1400	6.64
C05060578-015	P.S. Acid 2	3110	7510	2.10
C05060578-036	P.S. Acid 3	4900	16500	1.68
C05060578-037	P.S. Acid 4	11200	48300	1.25
C05060578-038	P.S. Acid 5	11500	50000	1.23
C05060578-039	P.S. Acid 6	12000	51600	1.18
C05060578-040	P.S. Acid 7	13100	55000	1.13
C05060578-041	P.S. Acid 8	14000	55600	1.15
C05060578-042	P.S. Acid 9	13700	61500	1.11
C05060578-043	P.S. Acid 10	13400	55200	1.12
C05060578-044	P.S. Acid 11	13600	61700	1.23
C05060578-045	P.S. Acid 12	13500	65300	1.24
C05060578-046	P.S. Acid 13	14600	68900	1.21
C05060578-047	P.S. Acid 14	14400	67700	1.17
C05060578-048	P.S. Acid 15	15100	69700	1.17
C05060578-049	P.S. Acid 16	16900	83600	1.11
C05060578-050	P.S. Acid 17	17400	82000	1.15
C05060578-051	P.S. Acid 18	18000	83400	1.10
C05060578-052	P.S. Acid 19	17700	85100	1.08
C05060578-053	P.S. Acid 20	18600	87300	1.09
C05060578-054	P.S. Acid 21	19900	91500	1.09

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LABORATORY ANALYTICAL REPORT

Client:	Plateau Resources (US Energy)	**Report Date:**	07/08/05
Project:	Column Test	**Date Received:**	06/14/05
Workorder:	C05060578		

	Analysis	SO4	EC	pH
	Units	mg/L	umhos/cm	s_u_
Sample ID	**Client Sample ID**	**Results**	**Results**	**Results**
C05060578-005	R.S.-1 Raw GW		483	8.22
C05060578-006	R.S.-2 Raw GW		478	8.29
C05060578-007	R.S.-3 Raw GW		484	8.39
C05060578-008	R.S.-4 Raw GW		479	8.34
C05060578-009	R.S.-5 Raw GW		470	8.37
C05060578-010	Acid Feed	23300	108000	0.85
C05060578-011	R.S. Acid 1	1460	3480	2.55
C05060578-013	R.S. Acid 2	7650	28400	1.40
C05060578-014	R.S. Acid 3	14500	61200	1.10
C05060578-016	R.S. Acid 4	15700	74700	1.03
C05060578-017	R.S. Acid 5	22200	106000	0.93
C05060578-018	R.S. Acid 6	22400	117000	0.90
C05060578-019	R.S. Acid 7	23500	116000	0.88
C05060578-020	R.S. Acid 8	23900	120000	0.91
C05060578-021	R.S. Acid 9	24300	123000	0.93
C05060578-022	R.S. Acid 10	24900	121000	0.94
C05060578-023	R.S. Acid 11	25000	121000	0.90
C05060578-024	R.S. Acid 12	24900	120000	0.92
C05060578-025	R.S. Acid 13	25400	121000	0.93
C05060578-026	R.S. Acid 14	25800	116000	0.93
C05060578-027	R.S. Acid 15	26000	115000	0.91
C05060578-028	R.S. Acid 16	26600	118000	0.91
C05060578-029	R.S. Acid 17	25600	118000	0.92
C05060578-030	R.S. Acid 18	26400	120000	0.95
C05060578-031	R.S. Acid 19	26300	115000	0.91
C05060578-032	R.S. Acid 20	26200	116000	0.90
C05060578-033	R.S. Acid 21	26100	118000	0.92
C05060578-034	R.S. Acid 22	27300	121000	0.93
C05060578-035	R.S. Acid 23	26200	120000	0.92

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05070394-001
Client Sample ID: PS Acid 4

Report Date: 07/21/05
Collection Date: Not Provided
Date Received: 07/12/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
MAJOR IONS							
Alkalinity, Total as CaCO3	ND	mg/L		1		A2320 B	07/14/05 16:56 / sl
Carbonate as CO3	ND	mg/L		1		A2320 B	07/14/05 16:56 / sl
Bicarbonate as HCO3	ND	mg/L		1		A2320 B	07/14/05 16:56 / sl
Calcium	564	mg/L		1		E200.7	07/18/05 12:06 / ts
Chloride	96	mg/L	D	8		E200.7	07/18/05 12:06 / ts
Fluoride	2.7	mg/L		0.1		A4500-F C	07/13/05 08:48 / sl
Magnesium	323	mg/L		1		E200.7	07/18/05 12:06 / ts
Nitrogen, Ammonia as N	0.97	mg/L		0.05		A4500-NH3 G	07/13/05 16:32 / jal
Nitrogen, Nitrate+Nitrite as N	ND	mg/L		0.1		E353.2	07/13/05 13:06 / jal
Nitrogen, Nitrite as N	ND	mg/L		0.1		A4500-NO2 B	07/13/05 13:06 / jal
Potassium	14	mg/L		1		E200.7	07/18/05 12:06 / ts
Silica	179	mg/L	D	0.8		E200.7	07/18/05 12:06 / ts
Sodium	56	mg/L		1		E200.7	07/18/05 12:06 / ts
Sulfate	11200	mg/L	D	8		E200.7	07/18/05 12:06 / ts
PHYSICAL PROPERTIES							
Conductivity	46800	umhos/cm		1.0		A2510 B	07/12/05 13:40 / th
Hydrogen Ion	65	mg/L				A4500-H B	07/12/05 15:19 / th
pH	1.19	s.u.		0.01		A4500-H B	07/12/05 15:19 / th
Solids, Total Dissolved TDS @ 180 C	11800	mg/L		10		A2540 C	07/12/05 14:36 / th
METALS - DISSOLVED							
Aluminum	40.7	mg/L		0.1		E200.8	07/13/05 14:24 / bws
Arsenic	0.05	mg/L	D	0.02		E200.8	07/13/05 14:24 / bws
Barium	0.2	mg/L		0.1		E200.8	07/13/05 14:24 / bws
Boron	0.9	mg/L		0.1		E200.7	07/18/05 12:06 / ts
Cadmium	ND	mg/L	D	0.02		E200.8	07/13/05 14:24 / bws
Chromium	ND	mg/L		0.05		E200.8	07/13/05 14:24 / bws
Copper	0.26	mg/L		0.01		E200.8	07/13/05 14:24 / bws
Iron	18.8	mg/L	D	0.04		E200.7	07/18/05 12:06 / ts
Lead	ND	mg/L		0.05		E200.8	07/13/05 14:24 / bws
Manganese	12.5	mg/L		0.01		E200.8	07/13/05 14:24 / bws
Mercury	ND	mg/L		0.001		E200.8	07/13/05 14:24 / bws
Molybdenum	ND	mg/L		0.1		E200.8	07/13/05 14:24 / bws
Nickel	0.26	mg/L		0.05		E200.8	07/13/05 14:24 / bws
Selenium	ND	mg/L	D	0.04		E200.8	07/13/05 14:24 / bws
Uranium	0.036	mg/L	D	0.004		E200.8	07/13/05 14:24 / bws
Vanadium	0.2	mg/L		0.1		E200.8	07/13/05 14:24 / bws
Zinc	0.23	mg/L	D	0.03		E200.8	07/13/05 14:24 / bws

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
D - RL increased due to sample matrix interference.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05070394-001
Client Sample ID: PS Acid 4

Report Date: 07/21/05
Collection Date: Not Provided
Date Received: 07/12/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
DATA QUALITY							
A/C Balance (± 5)	-26.7	%				Calculation	07/19/05 16:13 / ks
Anions	223	meq/L				Calculation	07/19/05 16:13 / ks
Cations	129	meq/L				Calculation	07/19/05 16:13 / ks

Report Definitions:
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MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05070394-002
Client Sample ID: PS Acid 21

Report Date: 07/21/05
Collection Date: Not Provided
Date Received: 07/12/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
MAJOR IONS							
Alkalinity, Total as CaCO3	ND	mg/L		1		A2320 B	07/14/05 16:58 / sl
Carbonate as CO3	ND	mg/L		1		A2320 B	07/14/05 16:58 / sl
Bicarbonate as HCO3	ND	mg/L		1		A2320 B	07/14/05 16:58 / sl
Calcium	604	mg/L		1		E200.7	07/18/05 12:13 / ts
Chloride	36	mg/L	D	8		E200.7	07/18/05 12:13 / ts
Fluoride	3.2	mg/L		0.1		A4500-F C	07/13/05 08:50 / sl
Magnesium	262	mg/L		1		E200.7	07/18/05 12:13 / ts
Nitrogen, Ammonia as N	1.42	mg/L		0.05		A4500-NH3 G	07/13/05 16:34 / jal
Nitrogen, Nitrate+Nitrite as N	0.5	mg/L		0.1		E353.2	07/13/05 13:08 / jal
Nitrogen, Nitrite as N	ND	mg/L		0.1		A4500-NO2 B	07/14/05 10:26 / jal
Potassium	15	mg/L		1		E200.7	07/18/05 12:13 / ts
Silica	188	mg/L	D	0.8		E200.7	07/18/05 12:13 / ts
Sodium	47	mg/L		1		E200.7	07/18/05 12:13 / ts
Sulfate	19900	mg/L	D	40		E200.7	07/18/05 12:16 / ts
PHYSICAL PROPERTIES							
Conductivity	83000	umhos/cm		1.0		A2510 B	07/12/05 13:44 / th
Hydrogen Ion	107	mg/L				A4500-H B	07/12/05 15:21 / th
pH	0.97	s.u.		0.01		A4500-H B	07/12/05 15:21 / th
Solids, Total Dissolved TDS @ 180 C	17400	mg/L		10		A2540 C	07/12/05 14:36 / th
METALS - DISSOLVED							
Aluminum	43.3	mg/L		0.1		E200.8	07/13/05 14:31 / bws
Arsenic	0.06	mg/L	D	0.02		E200.8	07/13/05 14:31 / bws
Barium	0.2	mg/L		0.1		E200.8	07/13/05 14:31 / bws
Boron	1.8	mg/L		0.1		E200.7	07/18/05 12:13 / ts
Cadmium	ND	mg/L	D	0.01		E200.8	07/13/05 14:31 / bws
Chromium	ND	mg/L		0.05		E200.8	07/13/05 14:31 / bws
Copper	0.14	mg/L		0.01		E200.8	07/13/05 14:31 / bws
Iron	24.1	mg/L	D	0.04		E200.7	07/18/05 12:13 / ts
Lead	ND	mg/L		0.05		E200.8	07/13/05 14:31 / bws
Manganese	17.8	mg/L		0.01		E200.8	07/13/05 14:31 / bws
Mercury	ND	mg/L		0.001		E200.8	07/13/05 14:31 / bws
Molybdenum	ND	mg/L		0.1		E200.8	07/13/05 14:31 / bws
Nickel	0.23	mg/L		0.05		E200.8	07/13/05 14:31 / bws
Selenium	ND	mg/L	D	0.04		E200.8	07/13/05 14:31 / bws
Uranium	0.032	mg/L	D	0.004		E200.8	07/13/05 14:31 / bws
Vanadium	0.2	mg/L		0.1		E200.8	07/13/05 14:31 / bws
Zinc	1.02	mg/L	D	0.03		E200.8	07/13/05 14:31 / bws

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
D - RL increased due to sample matrix interference.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05070394-002
Client Sample ID: PS Acid 21

Report Date: 07/21/05
Collection Date: Not Provided
Date Received: 07/12/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
DATA QUALITY							
A/C Balance (± 5)	-21.9	%				Calculation	07/19/05 16:15 / ks
Anions	368	meq/L				Calculation	07/19/05 16:15 / ks
Cations	236	meq/L				Calculation	07/19/05 16:15 / ks

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05070394-003
Client Sample ID: RS Acid 5

Report Date: 07/21/05
Collection Date: Not Provided
Date Received: 07/12/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
MAJOR IONS							
Alkalinity, Total as CaCO3	ND	mg/L		1		A2320 B	07/14/05 17:00 / sl
Carbonate as CO3	ND	mg/L		1		A2320 B	07/14/05 17:00 / sl
Bicarbonate as HCO3	ND	mg/L		1		A2320 B	07/14/05 17:00 / sl
Calcium	580	mg/L		1		E200.7	07/18/05 12:19 / ts
Chloride	32	mg/L	D	8		E200.7	07/18/05 12:19 / ts
Fluoride	3.3	mg/L		0.1		A4500-F C	07/13/05 08:52 / sl
Magnesium	238	mg/L		1		E200.7	07/18/05 12:19 / ts
Nitrogen, Ammonia as N	0.66	mg/L		0.05		A4500-NH3 G	07/13/05 16:36 / jal
Nitrogen, Nitrate+Nitrite as N	0.1	mg/L		0.1		E353.2	07/13/05 13:10 / jal
Nitrogen, Nitrite as N	ND	mg/L		0.1		A4500-NO2 B	07/14/05 10:26 / jal
Potassium	15	mg/L		1		E200.7	07/18/05 12:19 / ts
Silica	132	mg/L	D	0.8		E200.7	07/18/05 12:19 / ts
Sodium	45	mg/L		1		E200.7	07/18/05 12:19 / ts
Sulfate	22200	mg/L	D	40		E200.7	07/18/05 12:22 / ts
PHYSICAL PROPERTIES							
Conductivity	92800	umhos/cm		1.0		A2510 B	07/12/05 13:47 / th
Hydrogen Ion	123	mg/L				A4500-H B	07/12/05 15:22 / th
pH	0.91	s.u.		0.01		A4500-H B	07/12/05 15:22 / th
Solids, Total Dissolved TDS @ 180 C	20400	mg/L		10		A2540 C	07/12/05 14:36 / th
METALS - DISSOLVED							
Aluminum	41.4	mg/L		0.1		E200.8	07/13/05 14:37 / bws
Arsenic	0.09	mg/L	D	0.02		E200.8	07/13/05 14:37 / bws
Barium	0.2	mg/L		0.1		E200.8	07/13/05 14:37 / bws
Boron	0.7	mg/L		0.1		E200.7	07/18/05 12:19 / ts
Cadmium	ND	mg/L	D	0.01		E200.8	07/13/05 14:37 / bws
Chromium	0.10	mg/L		0.05		E200.8	07/13/05 14:37 / bws
Copper	0.13	mg/L		0.01		E200.8	07/13/05 14:37 / bws
Iron	72.0	mg/L	D	0.04		E200.7	07/18/05 12:19 / ts
Lead	ND	mg/L		0.05		E200.8	07/13/05 14:37 / bws
Manganese	15.5	mg/L		0.01		E200.8	07/13/05 14:37 / bws
Mercury	ND	mg/L		0.001		E200.8	07/13/05 14:37 / bws
Molybdenum	ND	mg/L		0.1		E200.8	07/13/05 14:37 / bws
Nickel	0.20	mg/L		0.05		E200.8	07/13/05 14:37 / bws
Selenium	ND	mg/L	D	0.04		E200.8	07/13/05 14:37 / bws
Uranium	0.025	mg/L	D	0.004		E200.8	07/13/05 14:37 / bws
Vanadium	0.3	mg/L		0.1		E200.8	07/13/05 14:37 / bws
Zinc	0.51	mg/L	D	0.03		E200.8	07/13/05 14:37 / bws

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
D - RL increased due to sample matrix interference.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05070394-003
Client Sample ID: RS Acid 5

Report Date: 07/21/05
Collection Date: Not Provided
Date Received: 07/12/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
DATA QUALITY							
A/C Balance (± 5)	-21.9	%				Calculation	07/19/05 16:16 / ks
Anions	404	meq/L				Calculation	07/19/05 16:16 / ks
Cations	259	meq/L				Calculation	07/19/05 16:16 / ks

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

ENERGY LABORATORIES, INC. • 2393 Salt Creek Highway (82601) • P.O. Box 3258 • Casper, WY 82602
Toll Free 888.235.0515 • 307.235.0515 • Fax 307.234.1639 • casper@energylab.com • www.energylab.com

LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05070394-004
Client Sample ID: RS Acid 20

Report Date: 07/21/05
Collection Date: Not Provided
Date Received: 07/12/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
MAJOR IONS							
Alkalinity, Total as CaCO3	ND	mg/L		1		A2320 B	07/14/05 17:02 / sl
Carbonate as CO3	ND	mg/L		1		A2320 B	07/14/05 17:02 / sl
Bicarbonate as HCO3	ND	mg/L		1		A2320 B	07/14/05 17:02 / sl
Calcium	602	mg/L		1		E200.7	07/18/05 12:25 / ts
Chloride	19	mg/L	D	8		E200.7	07/18/05 12:25 / ts
Fluoride	2.7	mg/L		0.1		A4500-F C	07/13/05 08:54 / sl
Magnesium	158	mg/L		1		E200.7	07/18/05 12:25 / ts
Nitrogen, Ammonia as N	0.41	mg/L		0.05		A4500-NH3 G	07/13/05 16:38 / jal
Nitrogen, Nitrate+Nitrite as N	ND	mg/L		0.1		E353.2	07/13/05 13:13 / jal
Nitrogen, Nitrite as N	ND	mg/L		0.1		A4500-NO2 B	07/13/05 13:13 / jal
Potassium	13	mg/L		1		E200.7	07/18/05 12:25 / ts
Silica	143	mg/L	D	0.8		E200.7	07/18/05 12:25 / ts
Sodium	40	mg/L		1		E200.7	07/18/05 12:25 / ts
Sulfate	26200	mg/L	D	40		E200.7	07/18/05 12:28 / ts
PHYSICAL PROPERTIES							
Conductivity	113000	umhos/cm		1.0		A2510 B	07/12/05 13:48 / th
Hydrogen Ion	141	mg/L				A4500-H B	07/12/05 15:23 / th
pH	0.85	s.u.		0.01		A4500-H B	07/12/05 15:23 / th
Solids, Total Dissolved TDS @ 180 C	23400	mg/L		10		A2540 C	07/12/05 14:37 / th
METALS - DISSOLVED							
Aluminum	41.5	mg/L		0.1		E200.8	07/13/05 14:44 / bws
Arsenic	0.07	mg/L	D	0.02		E200.8	07/13/05 14:44 / bws
Barium	0.2	mg/L		0.1		E200.8	07/13/05 14:44 / bws
Boron	0.3	mg/L		0.1		E200.7	07/18/05 12:25 / ts
Cadmium	ND	mg/L	D	0.01		E200.8	07/13/05 14:44 / bws
Chromium	0.06	mg/L		0.05		E200.8	07/13/05 14:44 / bws
Copper	0.07	mg/L		0.01		E200.8	07/13/05 14:44 / bws
Iron	71.5	mg/L	D	0.04		E200.7	07/18/05 12:25 / ts
Lead	ND	mg/L		0.05		E200.8	07/13/05 14:44 / bws
Manganese	9.51	mg/L		0.01		E200.8	07/13/05 14:44 / bws
Mercury	ND	mg/L		0.001		E200.8	07/13/05 14:44 / bws
Molybdenum	ND	mg/L		0.1		E200.8	07/13/05 14:44 / bws
Nickel	0.08	mg/L		0.05		E200.8	07/13/05 14:44 / bws
Selenium	ND	mg/L	D	0.04		E200.8	07/13/05 14:44 / bws
Uranium	0.017	mg/L	D	0.004		E200.8	07/13/05 14:44 / bws
Vanadium	0.3	mg/L		0.1		E200.8	07/13/05 14:44 / bws
Zinc	0.39	mg/L	D	0.03		E200.8	07/13/05 14:44 / bws

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
D - RL increased due to sample matrix interference.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

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LABORATORY ANALYTICAL REPORT

Client: Plateau Resources (US Energy)
Project: Column Tests
Lab ID: C05070394-004
Client Sample ID: RS Acid 20

Report Date: 07/21/05
Collection Date: Not Provided
Date Received: 07/12/05
Matrix: Aqueous

Analyses	Result	Units	Qual	RL	MCL/ QCL	Method	Analysis Date / By
DATA QUALITY							
A/C Balance (± 5)	-7.70	%				Calculation	07/19/05 16:16 / ks
Anions	467	meq/L				Calculation	07/19/05 16:16 / ks
Cations	400	meq/L				Calculation	07/19/05 16:16 / ks

Report Definitions:
RL - Analyte reporting limit.
QCL - Quality control limit.
MCL - Maximum contaminant level.
ND - Not detected at the reporting limit.

Date: *29-Jul-05*

CLIENT: Plateau Resources (US Energy)
Project: Column Tests
Sample Delivery Group: C05070394

CASE NARRATIVE

THIS IS THE FINAL PAGE OF THE LABORATORY ANALYTICAL REPORT

LABORATORY COMMENTS
Work orders C05060184, C05060266, C05060578, and C05070394 have been consolidated into one report package.

BRANCH LABORATORY LOCATIONS
eli-b - Energy Laboratories, Inc. - Billings, MT
eli-f - Energy Laboratories, Inc. - Idaho Falls, ID
eli-g - Energy Laboratories, Inc. - Gillette, WY
eli-h - Energy Laboratories, Inc. - Helena, MT
eli-r - Energy Laboratories, Inc. - Rapid City, SD
eli-t - Energy Laboratories, Inc. - College Station, TX

ORIGINAL SAMPLE SUBMITTAL(S)
All original sample submittals have been returned with the data package. A copy of the submittal(s) has been included and tracked in the data package.

SUBCONTRACTING ANALYSIS
Subcontracting of sample analyses to an outside laboratory may be required. If so, ENERGY LABORATORIES will utilize its branch laboratories or qualified contract laboratories for this service. Any such laboratories will be indicated within the Laboratory Analytical Report.

SAMPLE TEMPERATURE COMPLIANCE: 4°C (±2°C)
Temperature of samples received may not be considered properly preserved by accepted standards. Samples that are hand delivered immediately after collection shall be considered acceptable if there is evidence that the chilling process has begun.

ENERGY LABORATORIES, INC. - CASPER,WY certifies that certain method selections contained in this report meet requirements as set forth by NELAC. Some client specific reporting requirements may not require NELAC reporting protocol. NELAC Certification Number E87641.

ELI appreciates the opportunity to provide you with this analytical service. For additional information and services visit our web page www.energylab.com.

The total number of pages of this report are indicated by the page number located in the lower right corner.

APPENDIX I

ENTRADA SAND PROCTOR TEST RESULTS

APPENDIX I

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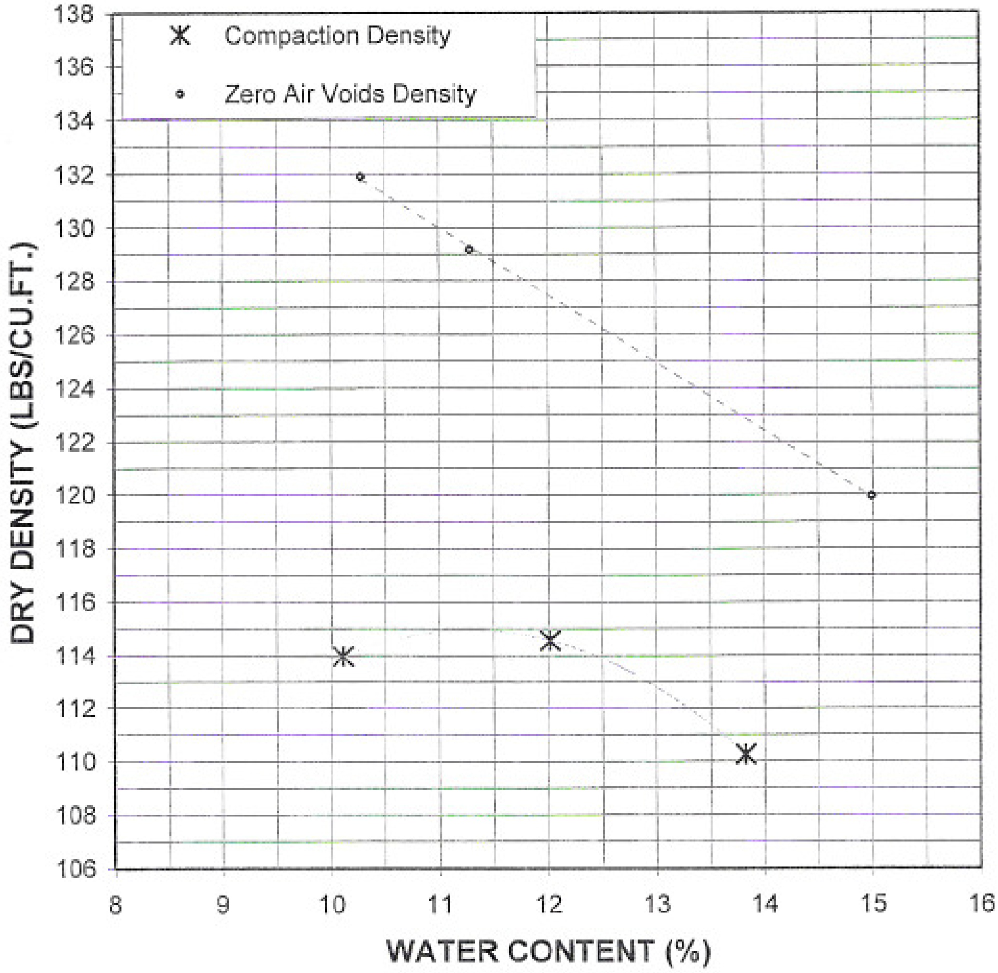
MOISTURE-DENSITY ANALYSIS

INBERG-MILLER ENGINEERS

CLIENT: U.S. Energy
PROJECT: General Testing
JOB NO. 12870-RM
TEST DATE: 12-1-06
SOURCE: Site Material
DESCRIPTION: Red Silty Fine Sand

SAMPLE NO.: 1
SAMPLED BY: Client
TESTED BY : TEG
TEST METHOD: ASTM D1557



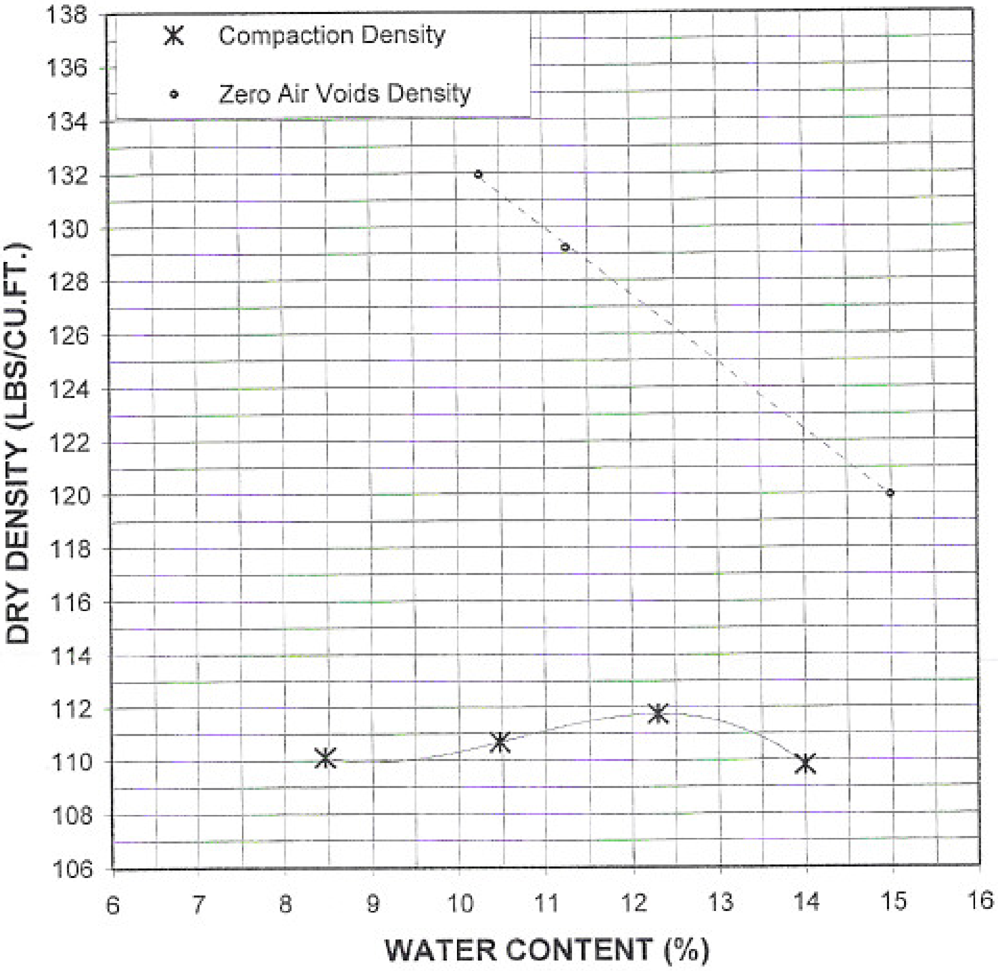
OPTIMUM WATER CONTENT (%): 11.3
MAXIMUM DRY DEN. (LBS/CU. FT): 115.0

MOISTURE-DENSITY ANALYSIS

INBERG-MILLER ENGINEERS

CLIENT: U.S. Energy
PROJECT: General Testing
JOB NO. 12870-RM
TEST DATE: 12-1-06
SOURCE: Site Material
DESCRIPTION: Red Silty Fine Sand

SAMPLE NO.: 1
SAMPLED BY: Client
TESTED BY : TEG
TEST METHOD: ASTM D698



OPTIMUM WATER CONTENT (%): 12.3
MAXIMUM DRY DEN. (LBS/CU. FT): 111.7

APPENDIX J

BURIED PIPE LOADING

APPENDIX J

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APPENDIX J

Buried Pipe Loading

J.0 Introduction

The load bearing capacity of the piping that is installed as a component in the leachate collection and recovery system and the sump access pipes must be sufficient to withstand the load imposed by up to 128 feet of tailings above the pipes. The leachate collection pipes are specified as a commercially available 3 inch internal diameter corrugated and perforated HDPE pipe. Perforated standard wall HDPE pipe with equivalent or superior pipe stiffness and sectional properties contributing to increased load bearing capacity may be substituted for the corrugated pipe. The sump access pipes are specified as 4 inch or 12 inch diameter SDR 9 HDPE pipe. Two methods were used to evaluate the deflection and potential buckling or crushing of the pipe under the imposed loads. These included the Modified Iowa Formula as presented in the “Plastic Pipe Design Manual” available on-line from Lamson Vylon Pipe and the Burns and Richard Solution using a program provided by ADS Pipe.

J.1 Modified Iowa Formula

J.1.1 Deflection

The Modified Iowa Formula is used to predict the deflection of a flexible pipe. The equation is:

$$\Delta = \left[\frac{D_L \cdot K \cdot P_y}{(0.149 \cdot PS) + (0.061 \cdot E')} \right] \cdot 100$$

where:

Δ	=	Deflection in %
D_L	=	Deflection Lag Factor
K	=	Bedding Constant
P_y	=	Prism Load, in psi
PS	=	Pipe Stiffness in psi
E’	=	Soil Modulus in psi

The deflection lag factor (D_L) is set to unity when the prism load is used to calculate deflection. The bedding constant (K) ranges from 0.083 to 0.110 for bedding angles ranging from 180 degrees to 0 degrees. The prism load is calculated as the sum of the static (dead) load and any live load. The soil modulus (E’) is generally determined from tabulated values based on the gradation and degree of compaction for the backfill around the pipe. The pipe stiffness (PS) can be a measured value or can be calculated using:

$$PS = \frac{6.71 \cdot E \cdot I}{r^3}$$

where:

PS	=	Pipe Stiffness in psi
E	=	Modulus of Elasticity in psi
I	=	Moment of Inertia in cubic inches
r	=	Mean Pipe Radius in inches

J.1.2 Unconfined Buckling Pressure

The calculation of unconfined buckling pressure is ultimately used to determine the maximum thickness of cover or overburden that the pipe can sustain. It does not incorporate the support provided to the pipe by the surrounding soil. The equation is:

$$P_{cr} = \frac{0.447 \cdot PS}{(1 - v^2)}$$

where:

P_{cr}	=	Unconfined Buckling Pressure in psi
PS	=	Pipe Stiffness in psi
v	=	Poisson's Ratio (approx. 0.4 for HDPE)

J.1.3 Confined Buckling Pressure

The calculation of confined buckling pressure is ultimately used to determine the maximum thickness of cover or overburden that the pipe can sustain and includes the support provided by the bedding surrounding the pipe. The equation is:

$$P_b = 1.15 \sqrt{P_{cr} \cdot E'}$$

where:

P_b	=	Confined Buckling Pressure in psi
P_{cr}	=	Unconfined Buckling Pressure in psi
E'	=	Soil Modulus in psi

J.1.4 Hydrostatic Buckling Pressure

For the conditions that will be present in the tailings cell(s) the contribution of hydrostatic force to the pipe buckling is considered negligible.

J.1.5 Buckling Resistance

With the total confined buckling pressure and the hydrostatic pressure, the maximum height (thickness) of cover can be calculated as:

$$H = \frac{P_b}{\gamma} \cdot 144$$

where:

H	=	Thickness of Cover in feet
P_b	=	Confined Buckling in psi
γ	=	Soil Unit Weight in pcf

J.1.6 Wall Crushing

The wall crushing calculation is basically a comparison of the allowable compressive stress in the pipe wall with the "ring" compressive stress imposed by the loading. The compressive stress is determined by:

$$\sigma = \frac{T}{A}$$

where:

σ	=	Compressive Stress in psi
T	=	Wall Thrust in lb/inch
A	=	Area of Pipe Wall in square inches/inch

The wall thrust is calculated as:

$$T = \frac{P_y \cdot D_o}{2}$$

where:

T	=	Wall Thrust in lb/inch
P_y	=	Vertical Soil Pressure in psi
D_o	=	Outside Diameter in inches

J.2 Leachate Collection Pipe – Modified Iowa Method

The leachate collection pipe is specified as a 3 inch internal diameter corrugated and perforated HDPE pipe with a minimum pipe stiffness (PS) of 50 psi. A pipe diameter of 4 inches is considered an option because of better commercial availability. The pipe crushing calculation is evaluated for a 4 inch internal diameter corrugated and perforated HDPE pipe as a measure of conservatism. The outside diameter (D_o) of a commercially available dual wall 4 inch pipe is 4.67 inches and the pipe wall area (A) is approximately 0.086 in^2/in. The typical Poisson's Ratio for HDPE is 0.40. On the base of the tailings cell(s), the leachate collection pipe will be bedded in washed gravel which results in a soil modulus (E') of 3000 psi (crushed rock with slight to high compaction). Other relevant properties of the pipe, installation, and loading conditions include: a maximum static load of 128 feet of overburden to the base of the reclamation cover at an assumed moist density of 100 pcf, a typical deflection lag factor of 1.0, and an intermediate bedding constant.

J.2.1 Deflection

The predicted deflection in the leachate collection pipe is:

$$\Delta = [\frac{D_L \cdot K \cdot P_y}{(0.149 \cdot PS) + (0.061 \cdot E')}] \cdot 100$$

The maximum prism load (P_y) is estimated as:

$$P_y = \frac{100 \cdot 128}{144} = 89\ psi$$

$$\Delta = [\frac{1 \cdot 0.1 \cdot 89}{(0.149 \cdot 50) + (0.061 \cdot 3000)}] \cdot 100 = 4.7\%$$

The predicted deflection is slightly smaller than the generally accepted 5% deflection limit for deep burial of flexible pipe. Some sources list 7.5% as an acceptable degree of deflection for HDPE pipes, and some testing has indicated that deflection can approach 20% before the pipe is compromised. Therefore, the predicted deflection under the maximum loading condition is acceptable.

J.2.2 Unconfined Buckling Pressure

The unconfined buckling pressure is calculated as:

$$P_{cr} = \frac{0.447 \cdot PS}{(1 - v^2)}$$

$$P_{cr} = \frac{0.447 \cdot 50}{(1 - 0.4^2)} = 26.6 \ psi$$

J.2.3 Confined Buckling Pressure

The confined buckling pressure is calculated as:

$$P_b = 1.15 \sqrt{P_{cr} \cdot E'}$$

$$P_b = 1.15 \sqrt{26.6 \cdot 3000} = 325 \ psi$$

J.2.4 Hydrostatic Buckling Pressure

For the conditions that will be present in the tailings cell(s), the contribution of hydrostatic force to the pipe buckling is considered negligible.

J.2.5 Buckling Resistance

The total confined buckling pressure can be used to calculate the maximum height (thickness) of cover as:

$$H = \frac{P_b}{\gamma} \cdot 144$$

$$H = \frac{325}{100} \cdot 144 = 468 \ feet$$

J.2.6 Wall Crushing

The wall thrust for a 4 inch inside diameter pipe is calculated as:

$$T = \frac{P_y \cdot D_o}{2}$$

$$T = \frac{89 \cdot 4.67}{2} = 208 \ lb/in$$

The tabulated allowable compressive stress in the HDPE pipe wall is approximately 3000 psi. The predicted compressive stress is calculated as:

$$\sigma = \frac{T}{A}$$

$$\sigma = \frac{208}{0.086} = 2419 \ psi$$

The compressive stress is less than the allowable short term stress of 3000 psi. If a similar configuration of 3 inch inside diameter pipe is used, the outside diameter will be reduced to approximately 3.67 inches while the sectional area of the pipe will be similar to that of the 4 inch inside diameter pipe. For this configuration the pipe thrust is calculated as:

$$T = \frac{89 \cdot 3.67}{2} = 163 \ lb / in$$

The corresponding compressive stress is calculated as:

$$\sigma = \frac{163}{0.086} = 1895 \ psi$$

J.3 12 inch Sump Access Pipes – Modified Iowa Method

The primary sump access pipes are specified as a 12 inch SDR 9 HDPE pipe. The outside diameter (D_o) of a 12 inch SDR 9 pipe is 12.75 inches and the wall thickness is approximately 1.417 inches. The pipe wall area (A) is approximately 1.417 in^2/in. A typical Poisson's Ratio for HDPE is 0.40. The sump access pipes are routed up 3H:1V slopes so it is not practical to install the pipes in a permanent compacted bedding up the complete length of the slope. However, the Entrada Sand will be used to form a compacted bed around the access pipes from the sump to a distance of at least 100 feet up the slope to surround, anchor, and protect these access pipes. The surface of the Entrada Sand may be plated with sand and gravel to reduce the erodibility. For the purposes of calculating load bearing capacity, it was assumed that the maximum static load of 100 feet of material is applied to the well-bedded lower section of the access pipe with a soil modulus (E') of 2000 psi. The load bearing capacity and deflection for the upper section of the pipe will be calculated with the reduced overburden thickness of 51 feet and a weaker soil with a modulus (E') of 200 psi. Other relevant properties of the pipe, installation, and loading conditions include: an assumed moist density of 100 pcf for the tailings over the pipe, a typical deflection lag factor of 1.0, an intermediate bedding constant, an HDPE modulus of elasticity (E) or 133000, a effective pipe radius of 5.67 inches and a 12 inch pipe moment of inertia (I) of 0.237 in^3.

J.3.1 Deflection

The predicted deflection in the primary sump access pipe is:

$$\Delta = [\frac{D_L \cdot K \cdot P_y}{(0.149 \cdot PS) + (0.061 \cdot E')}] \cdot 100$$

The maximum prism load (P_y) for the well bedded lower pipe section is estimated as:

$$P_y = \frac{100 \cdot 100}{144} = 69\ psi$$

The maximum prism load (P_y) for the upper pipe section is estimated as:

$$P_y = \frac{51 \cdot 100}{144} = 35.4\ psi$$

The pipe stiffness is estimated as:

$$PS = \frac{6.71 \cdot E \cdot I}{r^3}$$

$$PS = \frac{6.71 \cdot 133000 \cdot 0.237}{5.67^3} = 1160\ psi$$

The deflection for the lower pipe section is estimated as:

$$\Delta = [\frac{1 \cdot 0.1 \cdot 69}{(0.149 \cdot 1160) + (0.061 \cdot 2000)}] \cdot 100 = 2.3\%$$

The deflection for the upper pipe section is estimated as:

$$\Delta = [\frac{1 \cdot 0.1 \cdot 35.4}{(0.149 \cdot 1160) + (0.061 \cdot 200)}] \cdot 100 = 1.9\%$$

The predicted deflection is smaller than the generally accepted 5% deflection limit for deep burial of flexible pipe. The predicted deflection for both conditions is also smaller than the recommended maximum deflection of 2.5% as presented in a Plexco pipe Application Note.

J.3.2 Unconfined Buckling Pressure

The unconfined buckling pressure for the lower pipe section is calculated as:

$$P_{cr} = \frac{0.447 \cdot PS}{(1 - v^2)}$$

$$P_{cr} = \frac{0.447 \cdot 1160}{(1 - 0.4^2)} = 617 \ psi$$

J.3.3 Confined Buckling Pressure

The confined buckling pressure for the lower pipe section is calculated as:

$$P_b = 1.15 \sqrt{P_{cr} \cdot E'}$$

$$P_b = 1.15 \sqrt{617 \cdot 2000} = 1277 \ psi$$

The confined buckling pressure for the upper pipe section is calculated as:

$$P_b = 1.15 \sqrt{617 \cdot 200} = 404 \ psi$$

J.3.4 Hydrostatic Buckling Pressure

For the conditions that will be present in the tailings cell(s) the contribution of hydrostatic force to the pipe buckling is considered negligible.

J.3.5 Buckling Resistance

The total confined buckling pressure can be used to calculate the maximum height (thickness) of cover for the lower pipe section as:

$$H = \frac{P_b}{\gamma} \cdot 144$$

$$H = \frac{1277}{100} \cdot 144 = 1839\ feet$$

The maximum height (thickness) of cover for the upper pipe section is:

$$H = \frac{404}{100} \cdot 144 = 582\ feet$$

J.3.6 Wall Crushing

The maximum wall thrust for the 12 inch pipe is calculated as:

$$T = \frac{P_y \cdot D_o}{2}$$

$$T = \frac{69 \cdot 12.75}{2} = 880\ lb/in$$

The tabulated allowable compressive stress in the HDPE pipe wall is approximately 3000 psi. The predicted compressive stress is calculated as:

$$\sigma = \frac{T}{A}$$

$$\sigma = \frac{880}{1.417} = 621\ psi$$

J.4 4 inch Sump Access Pipes – Modified Iowa Method

The secondary sump access pipes are specified as a 4 inch SDR 9 HDPE pipe. The outside diameter (D_o) of a 4 inch SDR 9 pipe is 4.5 inches and the wall thickness is approximately 0.50 inches. The pipe wall area (A) is approximately 0.50 in^2/in. A typical Poisson's Ratio for HDPE is 0.40. Like the primary sump access pipes, the secondary access pipes are routed up 3H:1V slopes so it is not practical to install the pipes in a permanent compacted bedding up the complete length of the slope. However, the Entrada Sand will be used to form a compacted bed around the access pipes from the sump to a distance of at least 100 feet up the slope to surround, anchor, and protect these access pipes. The surface of the Entrada Sand may be plated with sand and gravel to reduce the erodibility. For the purposes of calculating load bearing capacity, it was assumed that the maximum static load of 100 feet of material is applied to the well-bedded lower section of the access pipe with a soil modulus (E') of 2000 psi. The load bearing capacity and deflection for the upper section of the pipe will be calculated with the reduced overburden thickness of 51 feet and a weaker soil with a modulus (E') of 200 psi. Other relevant properties of the pipe, installation, and loading conditions include: an assumed moist density of 100 pcf for the tailings over the pipe, a typical deflection lag factor of 1.0, an intermediate bedding constant, an HDPE modulus of elasticity (E) or 133000, a effective pipe radius of 2.0 inches and a 4 inch pipe moment of inertia (I) of 0.0104 in^3.

J.4.1 Deflection

The predicted deflection in the primary sump access pipe is:

$$\Delta = [\frac{D_L \cdot K \cdot P_y}{(0.149 \cdot PS) + (0.061 \cdot E')}] \cdot 100$$

The maximum prism load (P_y) for the well bedded lower pipe section is estimated as:

$$P_y = \frac{100 \cdot 100}{144} = 69\ psi$$

The maximum prism load (P_y) for the upper pipe section is estimated as:

$$P_y = \frac{51 \cdot 100}{144} = 35.4\ psi$$

The pipe stiffness is estimated as:

$$PS = \frac{6.71 \cdot E \cdot I}{r^3}$$

$$PS = \frac{6.71 \cdot 133000 \cdot 0.0104}{2.0^3} = 1160\ psi$$

The deflection for the lower pipe section is estimated as:

$$\Delta = [\frac{1 \cdot 0.1 \cdot 69}{(0.149 \cdot 1160) + (0.061 \cdot 2000)}] \cdot 100 = 2.3\%$$

The deflection for the upper pipe section is estimated as:

$$\Delta = \left[\frac{1 \cdot 0.1 \cdot 35.4}{(0.149 \cdot 1160) + (0.061 \cdot 200)} \right] \cdot 100 = 1.9\%$$

The predicted deflection is smaller than the generally accepted 5% deflection limit for deep burial of flexible pipe. The predicted deflection for both conditions is also smaller than the recommended maximum deflection of 2.5% as presented in Plexco Pipe Application Note 1.

J.4.2 Unconfined Buckling Pressure

The unconfined buckling pressure for the lower pipe section is calculated as:

$$P_{cr} = \frac{0.447 \cdot PS}{(1 - v^2)}$$

$$P_{cr} = \frac{0.447 \cdot 1160}{(1 - 0.4^2)} = 617 \ psi$$

J.4.3 Confined Buckling Pressure

The confined buckling pressure for the lower pipe section is calculated as:

$$P_b = 1.15 \sqrt{P_{cr} \cdot E'}$$

$$P_b = 1.15 \sqrt{617 \cdot 2000} = 1277 \ psi$$

The confined buckling pressure for the upper pipe section is calculated as:

$$P_b = 1.15 \sqrt{617 \cdot 200} = 404 \ psi$$

J.4.4 Hydrostatic Buckling Pressure

For the conditions that will be present in the tailings cell(s) the contribution of hydrostatic force to the pipe buckling is considered negligible.

J.4.5 Buckling Resistance

The total confined buckling pressure can be used to calculate the maximum height (thickness) of cover for the lower pipe section as:

$$H = \frac{P_b}{\gamma} \cdot 144$$

$$H = \frac{1277}{100} \cdot 144 = 1839\,feet$$

The maximum height (thickness) of cover for the upper pipe section is:

$$H = \frac{404}{100} \cdot 144 = 582\,feet$$

J.4.6 Wall Crushing

The maximum wall thrust for the 4 inch pipe is calculated as:

$$T = \frac{P_y \cdot D_o}{2}$$

$$T = \frac{69 \cdot 4.5}{2} = 155\,lb/in$$

The tabulated allowable compressive stress in the HDPE pipe wall is approximately 3000 psi. The predicted compressive stress is calculated as:

$$\sigma = \frac{T}{A}$$

$$\sigma = \frac{155}{0.50} = 310\ psi$$

J.5 Leachate Collection Pipe – Burns and Richards Method

ADS Pipe provides a Technical Note 2.130 (Goddard et al., 2003) that describes the Burns and Richards solution. ADS Pipe also provides a spreadsheet based solution of the Burns and Richards equation on their website (www.ads-pipe.com). This spreadsheet was used to evaluate the load bearing capacity of the 4 inch diameter corrugated and perforated leachate collection pipe. This pipe is considered the critical application because the preceding calculations using the Modified Iowa Formula indicated the greatest degree of deflection and more critical wall thrust for the leachate collection pipes. Table J-1 presents the results of the calculation by the Burns and Richard method using the spreadsheet provided by ADS Pipe.

The inputs in the calculation were adjusted to produce the same loading conditions and soil modulus as those used in the Modified Iowa formula, and the predicted deflection of 4.34% is similar to that of the Modified Iowa formula. The predicted compressive stress was also similar to that from the Modified Iowa formula.

Table J-1. Burns and Richard Solution for 4 inch Leachate Collection Pipe.

PIPE PARAMETERS - AASHTO M294, Type C	
effective radius (in), R =	2.18
outside diameter (in), D =	4.71
thickness (in), t =	0.34
unit area of wall (in^2/in), A =	0.081
unit moment of inertia (in^4/in), I =	0.0010
flexural modulus (psi), E_f =	110,000
ring compression modulus (psi), E_{rc} =	110,000
flexural stiffness (psi), $K_f = 6E_fI/R^3$ =	63
ring compression stiffness (psi), $K_{rc} = E_{rc}A/R$ =	4,080
distance from inner wall to n.a. (in), c =	0.169
SOIL PARAMETERS - good granular soil	
mod of soil reaction at 5' of cover (psi), E'_5 =	1130
modulus of soil reaction (psi), E' =	3,002
Poisson's ratio, u =	0.30
constr mod (psi), M*=E*(1-u)/((1+u)(1-2u))=	4041.41
lateral stress ratio = K = u/(1-u) =	0.429
sym lateral stress ratio = B = (1/2)(1+K) =	0.714
antisym lat stress ratio = C = (1/2)(1-K) =	0.286
SOIL/STRUCTURE PARAMETERS (full slippage)	
ring flexibility ratio, UF =(1+K)M*/K_{rc} =	1.42
bending flexibility ratio, VF = (1-K)M*/K_f =	36.5
STRESS FUNCTION COEFFICIENTS	
constant term, a_0* =	0.106
cos(2*theta), a_2** =	0.963
sin(2*theta), b_2** =	0.945
LOAD PARAMETERS	
unit weight of soil (lb/ft^3) =	100
height of fill above crown (ft) =	128.0
surcharge pressure (psi), P =	88.9

RESPONSE OF PIPE WALL

deg c.c.w. from horiz	radial soil press P_r(psi)	radial defl w(in)	tang defl v(in)	circum wall thrust N(#/in)	wall bend mom(M) (#-lb/in)	ring comp stress (psi)	inner bend stress (psi)	outer bend stress (psi)	total stress inner (psi)	total stress outer (psi)
0	54.0	-0.034	0.000	126	5	-1556	-872	882	-2427	-674
10	54.1	-0.030	0.011	126	5	-1554	-826	836	-2380	-718
20	54.6	-0.019	0.021	126	4	-1550	-695	704	-2245	-846
30	55.4	-0.002	0.028	125	3	-1543	-495	501	-2038	-1042
40	56.3	0.019	0.032	124	1	-1535	-249	252	-1784	-1283
50	57.2	0.042	0.032	124	0	-1526	12	-13	-1514	-1539
60	58.2	0.063	0.028	123	-2	-1518	258	-261	-1260	-1779
70	58.9	0.080	0.021	122	-3	-1511	459	-464	-1053	-1975
80	59.4	0.091	0.011	122	-3	-1507	589	-596	-917	-2103
90	59.6	0.095	0.000	122	-4	-1505	635	-642	-870	-2148
100	59.4	0.091	-0.011	122	-3	-1507	589	-596	-917	-2103
110	58.9	0.080	-0.021	122	-3	-1511	459	-464	-1053	-1975
120	58.2	0.063	-0.028	123	-2	-1518	258	-261	-1260	-1779
130	57.2	0.042	-0.032	124	0	-1526	12	-13	-1514	-1539
140	56.3	0.019	-0.032	124	1	-1535	-249	252	-1784	-1283
150	55.4	-0.002	-0.028	125	3	-1543	-495	501	-2038	-1042
160	54.6	-0.019	-0.021	126	4	-1550	-695	704	-2245	-846
170	54.1	-0.030	-0.011	126	5	-1554	-826	836	-2380	-718
180	54.0	-0.034	0.000	126	5	-1556	-872	882	-2427	-674

CALCULATION OF RING SHORTENING

deg c.c.w. from horiz	ring comp stress (psi)	ring comp strain (in/in)	ring shortening (in)
0	-1556	-0.014142	-0.0054
10	-1554	-0.0141	-0.0054
20	-1550	-0.014088	-0.0054
30	-1543	-0.014027	-0.0053
40	-1535	-0.013952	-0.0053
50	-1526	-0.013873	-0.0053
60	-1518	-0.013798	-0.0053
70	-1511	-0.013737	-0.0052
80	-1507	-0.013697	-0.0052
90	-1505	-0.013684	-0.0052
100	-1507	-0.013697	-0.0052
110	-1511	-0.013737	-0.0052
120	-1518	-0.013798	-0.0053
130	-1526	-0.013873	-0.0053
140	-1535	-0.013952	-0.0053
150	-1543	-0.014027	-0.0053
160	-1550	-0.014088	-0.0054
170	-1554	-0.0141	-0.0054
180	-1556	-0.014142	-0.0054
		SUM (I/2 circle) =	-0.1008

COMMENTS

1. This is 4" diameter corrugated pipe

2. Flexural and compressive modulus are taken as 110,000 psi.

3. Typical E'_5 values (in psi) for various soils are listed in the table below:

Type of soil	Standard AASHTO Relative Compaction 85%	90%	95%
Fine-grained soils with less than 25% sand (CL, ML, DL-ML)	500	700	1000
Coarse-grained soils with fines (SM, SC)	600	1000	1200
Coarse-grained soils with little or no fines (SP, SW, GP, GW)	700	1000	1600

MISC CALCS	
Vertical deflection (%) =	4.34
Horizontal deflection (%) =	-3.12
Critical Buckling Pressure (psi), P_{cr}=	166.9
Radial Soil Pressure at Crown (psi), P_{act}=	59.6
Arc length of each sector (in) =	0.3812
CIRCUMFERENCE SHORTENS=	-0.20 inches

Max. Compressive Stress	Max. Tensile Stress	Circumfence Shortening % (2% Max)	
-2427.3 OK (< -3000)	-673.61 OK (< 1000)	-0.0137	OK

J.6 References

BRSOLUT.xls, Spreadsheet implementation of Burns and Richard Solution, available on-line from www.ads-pipe.com, ADS Pipe, Hilliard, Ohio.

Goddard, J.B, N.E. Kampbell and D.P. Kozman (2003). *Structural Performance of Corrugated PE Pipe Using the Burns and Richard Solution,* Technical Note 2.130, available on-line from www.ads-pipe.com, ADS Pipe, Hilliard, Ohio.

Plastic Pipe Design Manual, available on-line from www.vylonpipe.com, Lamson Vylon Pipe, Cleveland, Ohio.

Plexco Application Note No. 1, *Pipe Behavior Under Earth Loading*, Chevron Plexco Piping Systems.

APPENDIX K

LINER SYSTEM ANCHORAGE

APPENDIX K

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APPENDIX K

Liner System Anchorage

K.0 Introduction

The required anchorage for the Cell 1 and Cell 2 liner system varies dramatically with the slope conditions on the perimeter of the cell and the coverage by the granular drainage layers. The granular drainage layers will be placed on the base of the cells on slopes up to 4H:1V. The majority of the Cell 1 will be covered by the granular drainage layers and a typical slope on the anchored periphery for these drainage layer covered areas is 5.5H:1V. The upstream and downstream slopes of the cross valley berm and the upstream slope of the Shootaring Dam will be at a 3H:1V slope and there will not be any cover soils placed on these slopes. In addition, the side slopes of Cell 2 will be at a slope of 3H:1V and no granular drainage layers will be placed on these slopes.

The proposed liner anchor mechanisms include: a conventional trench or L anchor, a runout (also horizontal or linear) anchor, and a default linear anchor to connect and provide a continuous liner across the cross valley berm.

The two general anchor failure modes include an anchor pullout or an HDPE liner failure. Within the tailings facility, the anchor pullout will be considered the controlling condition. An anchor pullout will generally be an observable occurrence, while there may be no evidence of a tension failure of one or both of the liners. The tensile strength of one liner will be considered the critical (maximum) anchorage tension. The following methods of evaluating and designing liner anchorage are presented in Koerner (2005).

K.1 Runout Anchor

A runout anchor relies on the normal force created by a cover soil load on a horizontal liner section to produce a frictional resistance to liner pullout. The two adjustable variables in a runout design are the thickness of the cover soil and the length of the runout.

K.1.1 Summation of Forces

Koerner (2005) presents a summation of horizontal forces for a runout liner pullout as:

$$\Sigma Fx = 0$$

$$T_{allow} \cos\beta = F_{U\sigma} + F_{L\sigma} + F_{LT}$$

$$T_{allow} \cos\beta = \sigma_n \tan\delta_u \left(L_{RO}\right) + \sigma_n \tan\delta_L \left(L_{RO}\right) + 0.5\left(\frac{2T_{allow} \sin\beta}{\left(L_{RO}\right)}\right)\left(L_{RO}\right)\tan\delta_L$$

where:

T_{allow} = allowable force in geomembrane = σ_{allow} t, where
σ_{allow} = allowable stress in geomembrane, and
t = thickness of geomembrane;
β = side slope angle;
$F_{U\sigma}$ = shear force above geomembrane due to cover soil;
$F_{L\sigma}$ = shear force below geomembrane due to cover soil;
F_{LT} = shear force below geomembrane due to vertical component of T_{allow};
σ_n = applied normal stress from cover soil;
δ = angle of shearing resistance between geomembrane and adjacent material; and
L_{RO} = Length of geomembrane runout.

K.1.2 Length of Runout

As presented in Koerner (2005) a rearrangement of the previous summation of forces equations presents a summation of horizontal forces for a runout liner pullout as:

$$L_{RO} = \left(\frac{T_{allow}\,(\cos\beta - \sin\beta \tan\delta_L)}{\sigma_n(\tan\delta_u + \tan\delta_L)} \right)$$

K.2 Trench Anchor

A trench anchor typically includes a runout section with a terminating trench with the liner(s) folded over the edge of the trench prior to backfill. The depth of the anchor trench then introduces another variable into the design process. The formulation of the governing equation is very to similar to that of a runout anchor with the addition of the earth pressures in the trench.

K.2.1 Summation of Forces

Koerner (2005) presents a summation of horizontal forces for an anchor trench liner pullout as:

$$\Sigma Fx = 0$$

$$T_{allow}\cos\beta = F_{U\sigma} + F_{L\sigma} + F_{LT} - P_A + P_P$$

where the variables are as previously defined with the addition of:

P_A = active earth pressure against the backfill side of the anchor trench; and
P_P = passive earth pressure against the inside of the anchor trench.

K.2.2 Earth Pressure

The additional forces resisting liner pullout are the imposed by the passive and active earth pressure within the anchor trench. Koerner (2005) presents the calculation of these forces as:

$$P_A = (0.5\gamma_{AT} d_{AT} + \sigma_n) K_A d_{AT}$$

$$P_P = (0.5\gamma_{AT} d_{AT} + \sigma_n) K_P d_{AT}$$

where:

γ_{AT} = unit weight of soil in anchor trench,
d_{AT} = depth of the anchor trench,
σ_n = applied normal stress from cover soil,
K_A = coefficient of active earth pressure = $\tan^2(45 - \varphi/2)$,
K_P = coefficient of passive earth pressure = $\tan^2(45 + \varphi/2)$, and
φ = angle of shearing resistance of respective soil.

The resulting equation for determining liner pullout resistance has the design variables of cover thickness, length of runout and trench depth. Since the equation can only be solved for one variable, the cover thickness and length of runout are generally established as constants and the equation is solved for the depth of the trench

K.3 Top of Berm Runout Anchor Design

A runout anchor will be employed across the top of cross valley berm and the berm separating the EPPC from Cell 1, as well as other selected locations. The horizontal runout section across the top of the berms will be approximately 20 feet to extend completely across the berm and the cover layer will consist of a protective sand layer with a roadbed sand and gravel overlay. The total cover thickness is estimated at two feet. The interior slopes on the berm will be 3H:1V. The desired condition for a failure of one of the liners is to have the anchor pull out before liner rupture. Since the length of runout is basically fixed for the top of berm runout, the required length of runout to result in a tensile failure will be calculated. This length of runout will then be compared with the fixed berm width runout to determine likely controlling failure mode and the utilization of the allowable tensile force in one of the two liners.

K.3.1 Length of Runout Calculation

The inputs for the calculation are as follows:

σ_{allow} = 2100 psi
t = 0.060 inch
T_{allow} = σ_{allow} t = 126 lb/in
β = 18.4 degrees
σ_n = cover thickness x unit weight of soil = 2 ft. x 100 lb/ft^3 = 200 lb/ft^2
= 1.39 psi
δ_L = 11 degrees
δ_U = 0 degrees

The maximum length of runout that will result in reaching allowable liner tension at liner pullout is estimated as:

$$L_{RO} = \left(\frac{T_{allow}\ (\cos\beta - \sin\ \beta \tan\delta_L)}{\sigma_n(\tan\delta_u + \tan\delta_L)} \right)$$

$$L_{RO} = \left(\frac{126\ (\cos(18.4) - \sin\ (18.4)\tan(11))}{1.39(\tan(0) + \tan(11))} \right) = 414\ inches = 34.5\ \ feet$$

The calculated liner runout of 34 feet is greater than the berm width of approximately 20 feet. Therefore, the liner will pull out prior to liner tensile failure. However, the actual runout is a large enough fraction of calculated runout that available tensile strength of the liner will be largely utilized prior to pullout. Figure K-1 presents a diagram of the runout anchor.

K.4 Trench Anchor Design

A trench anchor will be used as the runout anchor will be employed as the typical anchor on perimeter areas where the liner is not extended to connect with an adjacent cell. In many areas on the perimeter of Cell 1, the liner terminates with a very mild slope and coverage by the drainage layers. In these areas, the anchor runout and trench is unnecessary, but these areas will be used as the bounding condition for establishing the minimum runout length of four feet. This allows a minimum anchorage width on the perimeter for those areas where the side slopes are very mild and the covering drainage layers are present. For areas where the liners terminate at the crest of 3H:1V side slope, the minimum runout length will be four feet, but this may be increased for ease of construction. The general thickness of cover is assumed to be 18 inches with a unit weight of 100 lb/ft^3. In order to limit the potential for a tensile failure in the liner, the pullout force will be limited to one-half of the allowable tension.

K.4.1 Trench Anchor Calculation

The inputs for the calculation are as follows:

σ_{allow} = 2100 psi
t = 0.060 inch
T_{allow} = σ_{allow} t/2 = 126/2 = 63 lb/in
β = 18.4 degrees
σ_n = cover thickness x unit weight of soil = 1.5 ft. x 100 lb/ft^3 = 150 lb/ft^2 = 1.04 psi
δ_L = 11 degrees
δ_U = 0 degrees
L_{RO} = 4 feet = 48 inches
γ_{AT} = 100 lb/ft^3 = 0.0579 lb/in^3
φ = conservatively assumed to be 32 degrees for fine uniform sand.
K_A = $\tan^2(45 - \varphi/2) = \tan^2(45 - 32/2) = 0.307$
K_P = $\tan^2(45 + \varphi/2) = \tan^2(45 + 32/2) = 3.255$

The required depth of anchor trench is calculated according to:

$$T_{allow} \cos\beta = F_{U\sigma} + F_{L\sigma} + F_{LT} - P_A + P_P$$

$$F_{U\sigma} = \sigma_n \tan\delta_u \left(L_{RO}\right) = (1.04)\tan(0)\left(L_{RO}\right) = 0$$
$$F_{L\sigma} = \sigma_n \tan\delta_L \left(L_{RO}\right) = (1.04)\tan(11)\left(48\right) = 9.7\ lb/in$$
$$F_{U\sigma} = T_{allow} \sin\beta \tan\delta_L = (63)\sin(18.4)\tan(11) = 3.87\ lb/in$$

$$P_A = (0.5\gamma_{AT} d_{AT} + \sigma_n)K_A d_{AT} = (0.5(0.0579)d_{AT} + 1.04)(0.307)d_{AT}$$
$$P_A = 0.00889\, d_{AT}^{\ 2} + 0.319\, d_{AT}$$

$$P_P = (0.5\gamma_{AT} d_{AT} + \sigma_n)K_P d_{AT} = (0.5(0.0579)d_{AT} + 1.04)(3.255)d_{AT}$$
$$P_P = 0.09423\, d_{AT}^{\ 2} + 3.385\, d_{AT}$$

$$T_{allow} \cos\beta = 63\cos(18.4) = 59.8\ lb/in$$

$$59.8 = 0 + 9.7 + 3.86 - (0.00889\, d_{AT}{}^{2} + 0.319\, d_{AT}) + 0.09423\, d_{AT}{}^{2} + 3.385\, d_{AT}$$

$$0 = 0.0853\, d_{AT}{}^{2} + 3.066\, d_{AT} - 46.24$$

Using the quadratic equation solution, the depth of the trench is determined to be:

$$d_{AT} = 11.4\ inches$$

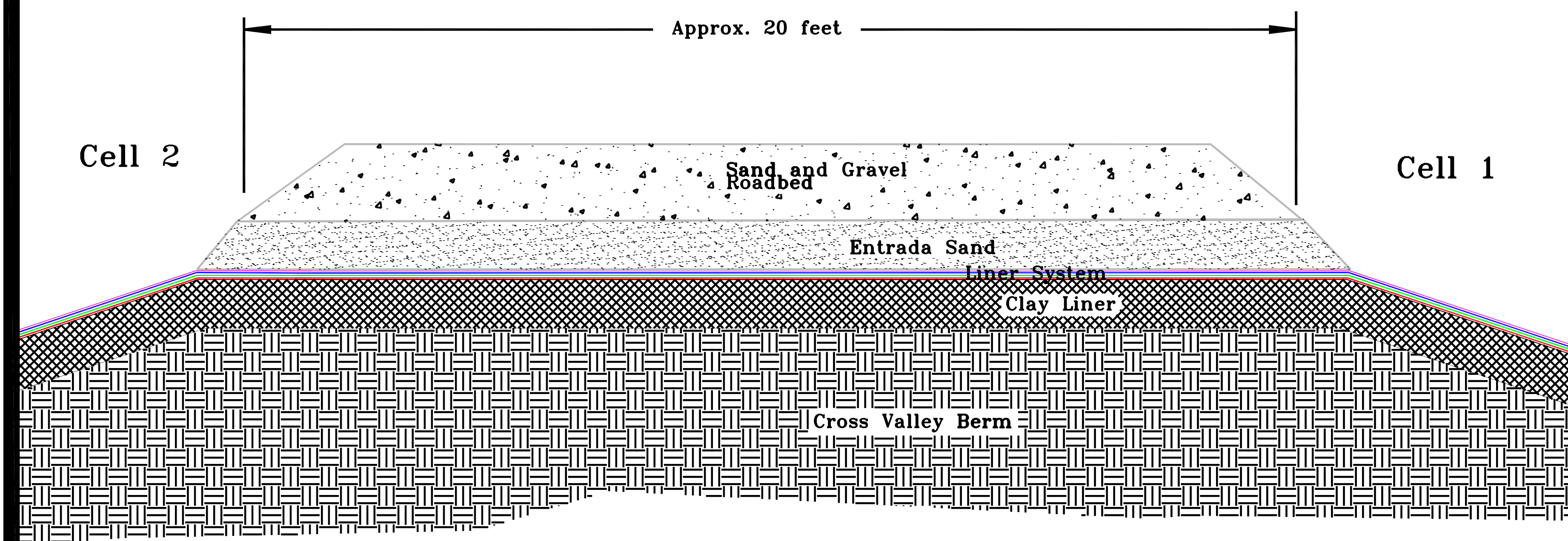
A specified trench depth of 16 inches with a minimum runout of 48 inches is sufficient to utilize one-half or more of the available tensile strength for a single HDPE liner. Figure K-2 presents a diagram of the trench anchor.

K.5 Summary and Conclusions

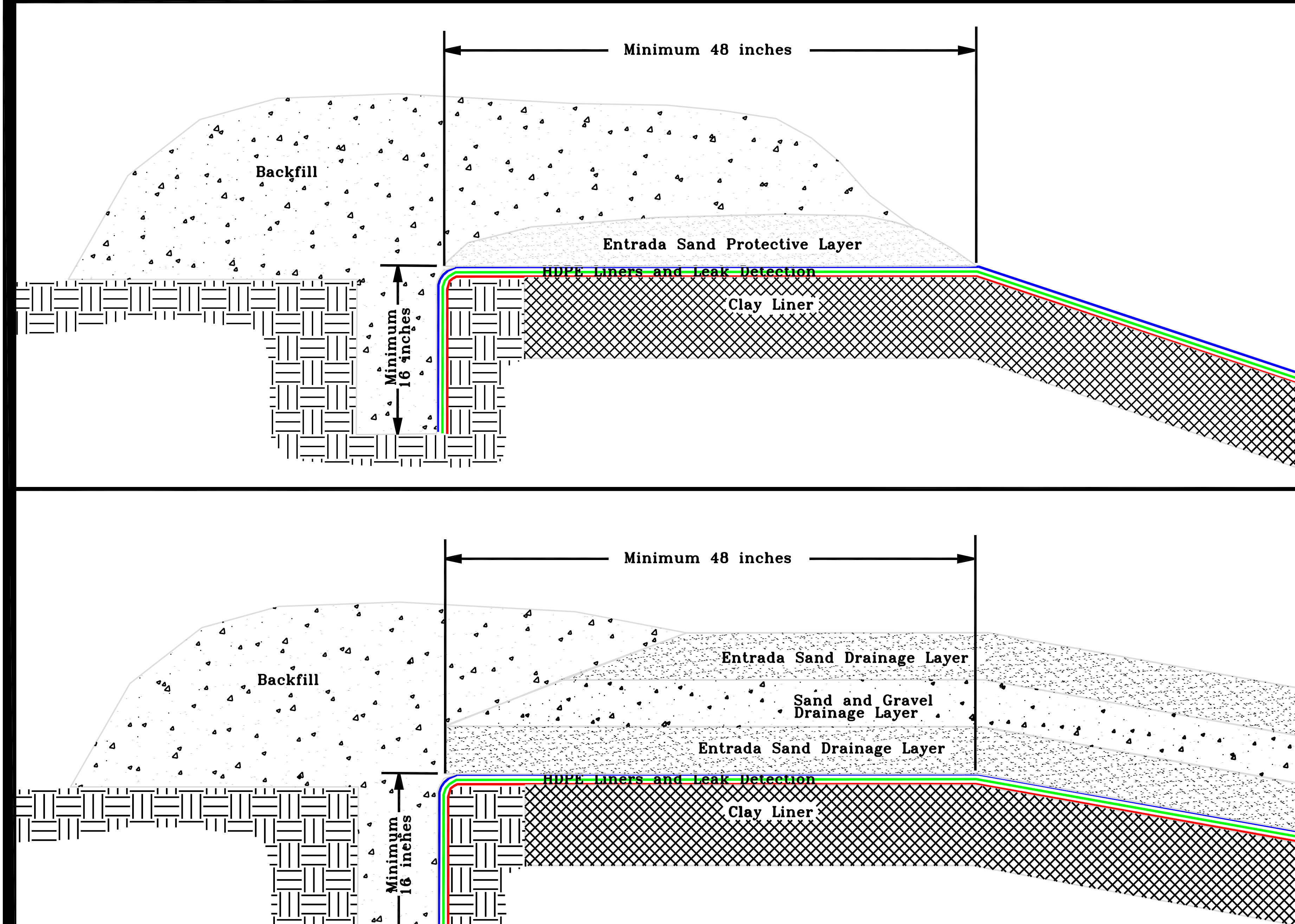
The runout anchor specified for the crest of the cross valley berm and the berm between the EPPC and Cell 1 is sufficient to resist pullout for forces that approach, but do not exceed, the allowable tensile stress in one of the two HDPE liners in the liner system. The runout anchor would generally be sufficient for mildly sloping areas on the perimeter of Cell 1, but a trench anchor is specified in the interest of uniformity of anchor construction. The liner trench anchor will be used as the on the remaining perimeter of the liner(s). The specified minimum runout for the trench anchor is 48 inches, and the trench depth will be 16 inches or more. This is sufficient for the critical areas of anchorage on the perimeter of the cells.

K.6 References

Koerner, R.M. 2005, Designing With Geosynthetics – Fifth Edition. Prentice Hall, Upper Saddle River, NJ.



PLATEAU RESOURCES Ltd.		
Figure K-1. Runout Liner Anchor Schematic		
REVISED: 4-2007	FigK-1-K-2.dwg	NOT TO SCALE
Page: K-7	HYDRO-ENGINEERING L.L.C.	



PLATEAU RESOURCES Ltd.		
Figure K-2. Trench Liner Anchor Schematic		
REVISED: 4-2007	FigK-1-K-2.dwg	NOT TO SCALE
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