Prepared for

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ROUND 1 – INTERROGATORY RESPONSE FOR THE CELL 4B DESIGN REPORT

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VIA E-MAIL AND OVERNIGHT DELIVERY

Mr. Dane L. Finerfrock
Executive Secretary
Division of Radiation Control
Department of Environmental Quality
168 North 1950 West
P.O Box 144850
Salt Lake City, UT 84114-4850

Re: Cell 4B Lining System Design Report, Response to Division of Radiation Control ("DRC") Request for Additional Information – Round 1 Interrogatory, Cell 4B Design.

Dear Mr. Finerfrock:

We are responding to your May 29, 2008 letter, requesting additional information regarding the Cell 4B Design Report. We have delayed submittal of our comments to the earlier request until the final approval of Cell 4A was received. The Cell 4B design now incorporates the final approved changes to the Cell 4A design.

For ease of review, the DRC's questions are repeated below in italics with Denison Mines (USA) Corp.'s ("DMC's") responses following each question.

1. *<u>Dike Integrity</u> – Please provide a revised slope stability evaluation that identifies the critical Cell 4B slopes. The evaluation needs to consider Cell 4B interior slopes of the joint berms between Cells 3 and 4B and 4A and 4B, the potential conditions associated with these berms, and the impacts of Cell 3 and 4A on the berm stability. The parameters and conditions used in the evaluation need to be identified and their values justified. If they vary from the previous relevant evaluations (i.e., those provided in support of Cell 4A), the reasons for the difference need to be presented and justified. The evaluation also needs to address potential impacts from seismic concerns, ground motions resulting from blasting operations to remove bedrock, potential operational loading, and other conditions that may impact the stability of the berms (between Cell 3 and Cell 4B and between Cell 4A and 4B) during the proposed cut back work to obtain the desired 2H:1V slopes.*

   Geosyntec performed a geotechnical investigation at the Cell 4B site consisting of advancing 7 solid-stem auger borings and performing laboratory testing on the collected soil samples from the borings. In addition, Geosyntec reviewed previous geotechnical investigation for the site performed by others, including a memorandum from MFG, Inc. (MFG) dated 13 June 2006 and a follow-up letter dated 7 July 2006. MFGs follow-up letter described their geotechnical investigation at the site, which included an exploratory boring through the existing berm between Cell 4A and 4B and a triaxial compression test on the recovered soil samples. Based on our geotechnical investigation and our review of the existing data, we have selected material properties for the fill material in the berm consistent with the properties used by MFG in their previous slope stability analyses for the embankment between Cell 4A and 4B (Unit Weight = 137 pcf, phi = 26, cohesion = 900 psf). As the embankment fill material will be derived from the on-site soil, similar to the embankment between Cell 4A and 4B, the same berm material properties were used for the embankment fills. As the boring performed by MFG through the embankment between Cell 4A and 4B did not indicate a sand layer beneath the sandy clay layer, one soil type was used for the berm fill material.

The existing soil to be left in place for the cut slopes has been assumed to be similar to the eolian/loess materials encountered in the seven geotechnical borings performed as part of the Geosyntec geotechnical investigation for the project. As such, soil properties based on laboratory testing were selected for the eolian/loess materials (Unit Weight = 135 pcf, phi = 24, cohesion = 1,000 psf).

Material properties selected for the sandstone, the soil tailings, and the geosynthetic liner for Geosyntec's previous geotechnical investigation have been used again for our reevaluation of the slope stability of the embankment slopes.

Slope stability analyses were performed for the critical slopes for each of the four embankments surrounding Cell 4B. The critical slopes analyzed are indicated on Figure 1 in Attachment A as Sections A-A', B-B'. C-C', and D-D'. In addition, slope stability analyses were performed on a typical cross section of the interim waste/tailings slopes. Slope stability analyses were performed for the final earthen berms and the interim waste/tailings slopes using the computer program SLOPE/W from GEOSLOPE International (Slope/W, 2008). Analyses were performed for both static and pseudo-static conditions as well as addressing construction loading. Numerous potential failure surfaces were performed to evaluate various slip surface geometries and to identify the critical slip surface for each cross section and condition evaluated. Output files from SLOPE/W for these analyses are provided as Exhibit A. The results of the slope stability analyses are summarized in Table 1 of the attached Slope Stability Analysis Calculation Package (Exhibit A).

Construction loading was considered in the slope stability analyses by applying a point load to the top of the slope. The point load selected corresponds to an AASHTO HS-20 loading. The point load was applied 2 feet from the top of the slope. Surcharge loading from a soil stockpile with side slope of 2H:1V was also evaluated for Cross Section C-C' where the contractor stockpile has been designated. A stockpile height of 20 feet was assumed based on the area of the stockpile and the quantity of material to be stockpiled. The stockpile was kept 20 feet from the top of slope to allow for the haul roads required to access the stockpile area.

Geosyntec also performed an analysis that evaluated the seismically-induced permanent deformation of the embankment slopes for Cell 4B. The analysis used the Makdisi and Seed method to estimate permanent seismic deformations based on yield accelerations determined from pseudo-static limit equilibrium analyses, design earthquake motions determined from documented sources, and design charts (Makdisi and Seed, 1978). This calculation and output files are provided as Exhibit B. Results from the analysis indicated that the expected seismically-induced permanent deformation is expected to be minimal (less than 1 centimeter), and significantly less than the design criterion of 6 inches.

Blasting operations on site to facilitate excavation of the sandstone will be performed within the footprint of Cell 4B. Charges for explosives will be controlled to maintain a peak particle velocity (PPV) of 5 inches per second (IPS) at the base of the embankment slopes between the existing cells (Cell 3 and 4A) and Cell 4B. Seismic waves from construction blasting are typically at a frequency of about 10 to 40 Hertz (Hz) for a PPV of 5 IPS as compared with a frequency of approximately 0.1 to 2 Hz from earthquake loading. Because of the relatively high frequency of the seismic waves from construction blasting, the effect of construction blasting on the global stability of earthen slopes is not typically analyzed. Limitations on the blasting with respect to PPV and displacement of the soil or rock are generally considered sufficient measures to ensure the global stability of earthen slopes.

A double liner system is being installed in Cell 4A and should prevent the build-up of groundwater in the embankment between Cell 4A and 4B from leakage. However, as suggested, the condition of the liner system in Cell 3 is unknown. In order to account for possible leaks or breaches in the liner systems, a piezometric level was assumed in the embankment berms between Cell 4A and 4B and between Cell 3 and 4B that extends from the top of the embankment berm on the side of Cell 3 or Cell 4A to the toe of the slope on the Cell 4B side. This piezometric level is considered conservative because of the limited chance of complete failure of the liner system, which would be required to produce a piezometric level of this elevation. Results are presented in the Slope Stability Analysis Calculation Package (Exhibit A).

Please justify the use of factors of safety equal to 1.5 and 1.3.

Final slope stability and operational conditions were performed to a minimum factor of safety of 1.5 and 1.3, respectively. Minimum factor of safety values of 1.5 and 1.3 are standard of practice values and are recommended for slope stability in the Naval Facility Engineering Command (NAVFAC) Design Manual (DM) 7.1. A copy of the pages (318 and 329) describing the minimum factors of safety are provided in Exhibit A.

Please provide more detail on the construction drawings and in the design report on the proposed construction of the access road. This detail should include the layout of the road, its materials and means of construction, surface water flow, erosion controls and infiltration controls to preclude adversely impacting the integrity of the Cell 4B liner system and berms. Additional detail should also demonstrate that the road is adequate to handle the proposed traffic loading.

The access road proposed around the perimeter of the tailings cells are only intended for light pickup truck traffic from DMC operations staff and are therefore not designed as main thoroughfares around the site. As part of the stability analyses, however, Geosyntec modeled loading conditions caused by a HS-20 truck driving on the access road approximately two feet from the crest of the berm. The calculation and output files are provided in Exhibit A.

The access roads are narrow areas on the top of the perimeter dikes and will be impacted only by direct precipitation (no run on). Therefore, the surface water impact will be minimal and the small amount of water will sheet flow off of the access road, as has occurred previously at the site on the existing access roads.

2. <u>Slimes Drain System and Sideslope Risers for Slime Drain Pipe and Leak Detection Pipe</u>

Refer to the Design Report. *Please indicate in the Design Report (Section 3.4.1) that the slimes drain system has been designed to be compliant with the following performance standards which were also specified in Part 1.D.6 of the Groundwater Discharge Permit) for Cell 4A, which, at a minimum include:*

…"c. Slimes Drain Monthly and Annual Average Recovery Head Criteria – after the Permittee initiates pumping conditions in the slimes drain layer in Cell 4A, the Permittee will provide continuous declining fluid heads in the slimes drain layer, in a manner equivalent to the requirements found in Part I.D.3(b) [of the Groundwater Discharge Permit]."

Geosyntec has amended Section 3.4.1 of the design report to include this statement. The revised section of the design report is attached to this letter as Exhibit I.

Please demonstrate that the strength of the specified perforated PVC pipes will be sufficient to ensure that the pipes will perform satisfactorily given the Cell 4B design.

Geosyntec has revised the pipe strength calculation that considers the 0.25 inch diameter perforations staggered 12 inches along the length of the PVC pipe. The calculation package is attached to this letter as Revised Pipe Strength Calculations (Exhibit C). Environmental Protection Agency (EPA), Manual SW-8, "Lining of Waste Impoundment and Disposal Facilities," describes a design method that increases the design's total vertical stress to account for the perforations. The calculation demonstrates the effect of perforations is minimal on the pipe strength and the pipe exceeds design requirements for this project.

Please modify the design of the PVC pipe used in the slimes drain access pipe on the sideslopes to include measures to protect against damage of the PVC pipe due to prolonged exposure to the atmosphere, including sunlight (e.g., UV radiation) or, alternatively, provide piping material that is resistant to damage from such long-term exposure conditions.

PVC pipe is typically used in environmental applications due to its excellent resistance to aggressive environments. Its resistance to natural ultraviolet (UV) radiation has been documented by the Uni-Bell, who performed a two year study to determine changes in the pipe's mechanical properties after exposing pipe to some of the worst aging conditions found in North America (Uni-Bell, 1997). One of the results from the study concluded that,

> *"…any opaque shield, no manner how thin, will effectively prevent UV degradation. As a result, above-ground pipe systems may be painted, wrapped or coated to stop UV degradation."* (Uni-Bell, 1997)

Please see Note 4 of Construction Drawing 5 that states, "Exposed PVC pipe shall be painted to minimize damage due to UV."

Refer to the Design Report and Design Calculation: Action Leakage Rate*. Please provide additional information to justify the selection of a maximum flow length (longest drainage path) of 730 feet (p. 3 of the calculation and Attachment B to the calculation).*

The maximum flow length was selected based on the longest path a drop of water could flow between a point on the liner and a drain location (simple geometry related to flow paths perpendicular to topographic contour lines). We have determined that a flow length of 780 feet is the longest path. A revised copy of this calculation package is provided as Exhibit E.

Refer to Construction Drawing 5, Section D/3. *The use of a textured HDPE splash pad needs to be considered. The textured HDPE provides for a friction surface and traction if there is need to access this area, it would also provide for a more stable base for the discharge pipe.*

Section D/3 is now located on Construction Drawing 6. This detail has been revised to reflect the use of textured HDPE in the splash pad design. This drawing is provided in Exhibit H.

Construction Drawing 6, the call out for Section F/6 is shown on Plan Detail 9/3,4 as F/5. Please correct. *Also on this drawing, please correct the depiction of the sand bags shown to be consistent with those on Construction Drawing 5, provide the detail to scale. In order to ensure the proper design and constructability of these pipes and sump systems, please provide these sections and details to scale.*

Section F/4,7 (formerly F/6) and Plan Detail 9/3,4 are now located on Construction Drawing 7. The call outs for Section F/4,7 shown on Plan Detail 9/3,4 have been corrected. The symbol depicting the sandbags has been changed so that they are consistent throughout the drawing set. This drawing is provided in Exhibit H.

Refer to Construction Drawing 6, Section I/6 and Drawing 7, Section 11/4*. Please provide revised construction drawings showing how all the components (i.e., GCL, HDPE membranes, geotextile, geonet, and aggregate) of the slimes drain and leak detection side slope are finished at the top of the side slope/berm and how do the slimes drain pipe and leak detection pipe penetrate these components? Additional detail is required in Drawings 6 and 7 to illustrate how these components are finished at the top of the side slope/berm in relationship with the slimes drain pipe and leak detection pipe penetrations. Demonstrate that the slimes drain pipe and leak detection pipe at the top of the side slope/berm will not allow water to seep around the liner components, including flow around the points where the pipes penetrate the liner components, such that water does not flow down the side slope into the sump below.*

Details 10 and 11 have been added to Construction Drawing 5 to illustrate how the slimes drain pipe and leak detection pipe penetrate the geosynthetic components of the liner system. Underneath the pipe, the geosynthetic components of the system terminate in an anchor trench in the same manner as the geosynthetic termination around the perimeter of the cell preventing water from migrating into the individual liner components. At the penetration of the header pipes through the geosynthetic components of the liner, the flexible HDPE geomembrane will be booted to the pipe approximately one to two feet beyond the edge of the berm crest and secured with a clamp. The liner at the crest is welded on all sides to the underlying geomembrane to prevent the migration of liquids into the liner components.

Refer to Construction Drawing 7, Detail 11/4*. Please clarify why the cross-section shows the pipes exiting the ground surface at a slope which varies, given that the inside slope of the berm is reported to be 2:1? Demonstrate that the tie down straps and anchor bolts provide sufficient strength to prevent movement of the pipes. Demonstrate that the concrete header/foundation used to support the pipes provides sufficient strength without the use of rebar, as identified in Construction Drawing 7. Please provide additional detail how the concrete header/foundation will be constructed in relationship with the liner components (i.e., GCL, HDPE membranes, geotextile, geonet, and drainage aggregate) at the top of the side slope/berm.*

Details 10 and 11 on Construction Drawing 5 illustrate a 3:1 slope in the corner where the leak detection system and slimes drain system pipes penetrate the liner. Please refer to details 10 and 11 on Construction Drawing 5 that depict the placement of the concrete header wall in reference to the liner system. The pipes are supported by a concrete header wall that bears on the soil above the anchor trench outside the limits of the liner system. Rebar is not necessary in the design because the concrete header wall has a maximum height of approximately three and a half feet and the weight of the 18" diameter header pipe is sufficient to keep the pipe in place. The tie down straps are a design redundancy because the U-shaped groove in the concrete is sufficient to restrict lateral movement and the weight of the pipe will not allow it to move in the vertical direction.

Refer to Construction Drawing 6, Section I/6*. It provides details for sand bags and rope with 5' spacing placed on top of the slimes drain and leak detection system side slope. Is this currently being used successfully for Cell 4A? If so, demonstrate how the ropes will be secured around the sand bags such that rope does not become loose from the sand bag; demonstrate how the ropes are secured at the top of the berm; and demonstrate that the rope and will be resistant to UV light, weathering, low pH caused by the tailings, and other environmental operating conditions at Cell 4B.*

The woven geotextile on the sideslope under the pipe will be sewn to the underlying cushion geotextile using a bonded polypropylene thread. Bonded polypropylene thread is typically used in outdoor and automotive uses and is commonly used to sew automobile trim, carpets, industrial fabrics, medical, and hygienic products. Bonded polypropylene has a resin coating that makes it run smoothly through a sewing machine and improves its abrasion resistance. This material is resistant to strong acids, alkalis, and common solvents. Bonded polypropylene thread is commercially available from multiple retailers (Cole Parmer, 2008).

For all Construction Drawings ensure that the following key components are identified, and drawn to scale:

- *Liner system component layer surface elevations.*
- *Slimes drain piping and sump invert elevations and horizontal coordinates at terminations, changes in direction or grade, and connection points (at fittings).*
- *Leak Detection System drain piping and sump invert elevations and horizontal coordinates at terminations, changes in direction or grade, and connection points (at fittings).*
- *Elevations and horizontal coordinates at all liner system changes in grade such at key transition locations including but not limited to from the cell bottom to the side slopes and top of berms and in the sump area.*

Surface elevations have been added to Construction Drawing 4, which is provided in Exhibit H.

Please provide additional information/data demonstrating that components of the sideslope system for the slimes drain system and leak detection system riser pipes (including the tie down straps for the pipes onto the concrete header, the concrete header, and other components (e.g., ropes and sand bags) can operate in the low pH operating conditions and other operating environmental conditions.

Details 10 and 11 on Construction Drawing 5 shows the concrete headwall and tie down straps are located outside of the liner system and are not exposed to the low pH environment.

Please provide the justification for not including cleanouts for both the slimes drain and the leak detection piping (refer to Sheet 4). Include the methods proposed to maintain these pipes so they function as designed.

The slimes drain and LDS piping can be cleaned from the sump riser piping. Typical clean out equipment can extend up to 1,000 feet into a pipe. Because solution in the LDS and slimes drain piping will most likely be low pH with high dissolved solids content, flushing the pipe with fresh water would most likely neutralize the solution and cause dissolved solids to precipitate out and plug the piping system.

3. <u>Spillway Capacity Design/Calculation and Surface Water Runoff</u>

Please include the flows from the mill operation to these calculations and apply the results to design (as needed).

The Emergency Spillway is designed to pass the flows from the PMP storm event from one Cell to the next. The Cell must be operated within the approved freeboard limits during normal mill operations. The freeboard capacity of each of the cells is designed to handle the PMP storm volumes and the flows from mill operations are not allowed to consume any of the freeboard volume, therefore the mill flows are not included in the calculation for the PMP flows through the Emergency Spillway.

The discharge inlet/outlet elevations need to be identified on the Construction Drawings to identify how the flow occurs serially from Cell 2, 3, 4A, to 4B.

The elevations for the emergency spillway between Cell 4A and 4B are indicated on Construction Drawing 4, which is provided in Exhibit H. The requested emergency spillway elevations were provided to the UDEQ in the O&M Plan and the DMT Plan for Cell 4A. These plans were approved in a letter from the UDEQ to DMC dated 17 September 2008.

Please include a demonstration that if the operational requirements for freeboard in each cell are maintained (i.e., freeboard elevations maintained) the complete cell system has the capacity to contain stormwater from the PMP combined with the water and tailings from anticipated mill processing. The response to this request should include reference to the original determination of discharge from Cells 1, 2, 3, and 4A. It should also include a demonstration on where the overflow discharge from Cell 4B would go (if it occurred when all the other cells are full), and how the overflow water would be handled such that an uncontrolled release of tailings does not occur, or erosion and failure of the cells berms does not occur?

DMC submitted revised freeboard calculations for Cell 3 and Cell 4A to DRC on December 11, 2008. The freeboard limit for Cell 4A was established at 5593.7 feet above mean sea level ("fmsl"). This provides for the PMP events from Cell 2, Cell 3 and Cell 4A areas. Once Cell 4B is operational the freeboard limit will initially be set at 3 feet below the top of liner, which is more than adequate to handle the 10 inches of precipitation from the PMP event, plus wave runup. At the point the remaining pool area in Cell 4A is insufficient to handle the PMP runoff from Cell 2, Cell 3 and Cell 4A, then the freeboard limit in Cell 4B will be revised, with approval from the Executive Secretary, to store the PMP from Cell 2, Cell 3 and Cell 4A, as well as Cell 4B. The freeboard calculation is very conservative and Cell 4B is therefore designed as a zero discharge facility.

As stated above, it is not necessary to include the volumes from mill processing in the freeboard calculation.

At the point of discharge from Cell 4A into Cell 4B from the emergency spillway, please demonstrate how the design has incorporated features to prevent damage from occurring to the liner system and slimes drain piping by debris which may be entrained with the discharge water.

No debris is expected to be entrained within the discharge water and tailings consist of silts, sands, and clays. Prior to Denison performing a tailings discharge, the cell will be filled with process solution to a level with a minimum level of 2 feet. Tailings will be deposited in the liquid-filled cell and the sand, silt, and clay particles that comprise the tailings will gently fall out of solution in a manner that will not damage the liner system or its underlying components. In addition, a textured geomembrane splash pad has been added to the liner system below the splash pad to provide an additional level of protection to the liner system.

Section 2.5 of the Design Report states that "surface water at the facility is diverted around the Cells including Cell 4B." Please provide a drawing(s) that show how surface water runoff is diverted around Cell 4B, including runoff from adjacent cells which are either closed or in the process of being closed (i.e., placement of fill material as a cover), such that outside slopes of Cell 4B do not erode and lead to potential failure. Include the design components which allow the surface water to divert around Cell 4B. Also show the entire site surface water drainage flows, and explain how Cell 4B is incorporated into this overall facility drainage. This needs to include how contact stormwater that is or may be contaminated is discriminated from uncontaminated or non-contact stormwater. The relocation of the existing access road due to the construction of Cell 4B also needs to be considered.

The attached drawing, "Figure 1, Mill Site Drainage Basins" shows the various drainage basins for the entire mill site and the diversion structures where needed. The drainage basin boundaries delineate the topographic high point for each of the basins, or series of basins. Precipitation falling within the drainage basin boundary is retained within that area, and precipitation falling outside the boundary is diverted away from the area. Drainage basins "C", "D" and "E" represent Cells 2, 3 and 4A, with all of the runoff from those basins contained in Cell 4A, (see response to Comment 3, above). Drainage basin "F" is the Cell 4B area. The basin boundaries are the Cell 3 dike on the north, the Cell 4A dike on the east, and the Cell 4B dikes on the south and west. These boundaries represent the topographic high point on all sides of the basin. Precipitation falling within the drainage basin boundary is retained within that area, and is considered to be contaminated water, and precipitation falling outside the boundary is diverted away from the area, and is considered to be uncontaminated water.

Refer to Construction Drawing 7, Section 10/3. Please clarify why there are two Sections labeled "K/7"?

Section 12/3 (formerly Section 10/3) is located on Construction Drawing 8 and has been revised to show one of the two Sections as J/8 and the other as Section K/8. This drawing is presented in Exhibit H.

Please clarify why Construction Drawing 7, Section 10/3 and Section K/7 identify the inside slope of Cell 4B as 3:1, when other portions of the Design Report states that the inside slope of Cell 4B is 2:1. Also clarify and justify why the Construction Drawing 7, Section 10/3 and Section K/7 identify the inside slope of Cell 4A as varies.

Section 10/3 (formerly Section 10/3) and K/8 (formerly K/7) are now located on Construction Drawing 8. This drawing has been revised with the correct slopes and is presented in Exhibit H.

Resolve the conflict between Design Calculations for the Emergency Spillway (spillway width is 100 feet) and Construction Drawing 7, Section J/(width as 94 feet).

Detail J/8 on Construction Drawing 8 has been revised with the dimensions described in the Design Calculations. This drawing is presented in Exhibit H.

4. *<u>Monitoring Well WMMW-16</u> Please identify the location of WMMW-16 on the Construction Drawings. Please also provide a well construction diagram for WMMW-16. This diagram*

needs to be appended to Specification Section 02070, Well Abandonment), which is currently missing from that Section. Please also submit a well plugging and abandonment (well decommissioning) plan for Well WMMW-16.

DMC has surveyed Monitoring Well WMMW-16 and revised Construction Drawing 3 to reflect its location within the footprint of Cell 4B (Exhibit H). The well construction diagram has also been appended to Specification Section 02070 and is provided in Exhibit F. Well plugging and abandonment (well decommissioning) is at the expense and responsibility of the Contractor to comply with regulatory requirements as outlined in Section 02070, Part 1.04, Subpart B and Section 02070, Part 3.01, Subpart A.

5. *Splash Pads – Please describe what the operational criteria are which will determine the selection of the splash pads locations, and why would these criteria become apparent during construction, as opposed to during the design phase?*

The locations of the splash pads have been added to Drawing 3 and are presented in Exhibit H.

6. *Construction Drawing 5, Section D/3 suggests that a pipe located at the upper portion of the splash pad will be the mechanism by which tailings will be placed into Cell 4B. Provide an overview how the tailings will be introduced and fed through the pipe (i.e., operations related to input of tailings into Cell 4B) such that the liner system is not damaged by movement/handling of the pipe.*

Solution Discharge

Cell 4B will initially be used for storage and evaporation of process solutions from the Mill operations. These process solutions will be from the uranium/vanadium solvent extraction circuit, or transferred from Cell 1 evaporation pond or the free water surface from Cell 4A. The solution will be pumped to Cell 4B through 6 inch or 8 inch diameter HDPE pipelines. The initial solution discharge will be in the southeast corner of the Cell. The discharge pipe will be routed down the Splash Pad provided in the corner of the Cell to protect the pipeline running from the solution reclaim barge. The solution will be discharged in the bottom of the Cell, away from any sand bags or other installation on the top of the primary geomembrane. Building the solution pool from the low end of the Cell will allow the solution pool to gradually rise around the slimes drain strips, eliminating potential damage to the strip drains or the sand bag cover due to solution flowing past the drainage strips. The solution will eventually be discharged along the dike between Cell 3 and Cell 4B, utilizing Splash Pads. The subsequent discharge of process solutions will be near the floor of the pond, through a discharge header designed to discharge through multiple points, thereby reducing the potential to damage the Splash Pads or the Slimes Drain system. At no time will the solution be discharged into less than 2 feet of solution. As the cell begin to fill with solution the discharge point will be pull back up the Splash Pad and allowed to continue discharging at or near the solution level.

Initial Solids Discharge

Once Cell 4B is needed for storage for tailings solids the slurry discharge from No. 8 CCD thickener will be pumped to the cell through 6 inch or 8 inch diameter HDPE pipelines. The pipelines will be routed along the dike between Cell 3 and Cell 4B, with discharge valves and drop pipes extending down the Splash Pads to the solution level. One or all of the discharge points can be used depending on operational considerations. Solids will settle into a cone, or mound, of material under the solution level, with the courser fraction settling out closer to the discharge point. The valves are 6” or 8” stainless steel knife-gate valves. The initial discharge of slurry will be at or near the toe of the Cell slope and then gradually moved up the slope, continuing to discharge at or near the water surface. Because of the depth of Cell 4B, each of the discharge points will be utilized for an extended period of time before the cone of material is above the maximum level of the solution. The discharge location will then be moved further to the interior of the Cell allowing for additional volume of solids to be placed under the solution level. The solution level in the Cell will vary depending on the operating schedule of the Mill and the seasonal evaporation rates. The tailings slurry will not be allowed to discharge directly on to the Splash Pads, in order to further protect the primary geomembrane. The tailings slurry will discharge directly in to the solution contained in the Cell, onto an additional protective sheet, or on to previously deposited tailings sand.

Reclaim Water System

A pump barge and solution recovery system will be installed in the southeast corner of the cell to pump solution from the cell for water balance purposes or for re-use in the Mill process. The pump barge will be constructed and maintained to ensure that the primary geomembrane liner is not damaged during the initial filling of the Cell or subsequent operation and maintenance activities. The condition of the pump barge and access walkway will be noted during the weekly Cell inspections.

Demonstrate how the tailings will flow, settle, and enter Cell 4B at critical time periods over the operational life of Cell 4B and will not damage components (i.e., movement of sandbags, displacement of gravel and geotextiles) of slimes drain, strip drains, leak detection system, and liner system present in the bottom of Cell 4B.

See response to Comment 6, above.

Demonstrate that the dimension of the protective HDPE geomembrane (20' wide and 5' extension from the toe of the berm) will resist the influent pressure and scour flow rate of the tailings (in all directions, width of the side slope and extension from the toe of the berm).

The purpose of the protective HDPE geomembrane is to provide an extra protection layer for the pipe resting above the underlying geosynthetic components. The pressure used to pump the tailings into the pond is not large enough to scour or damage the liner.

Update the Project Technical Specifications to include the requirements for the construction of the protective HDPE geomembrane at splash pad locations and update the Construction Quality Assurance Plan to include procedures which will be followed to ensure that the protective HDPE geomembrane at splash pads is properly installed.

The HDPE geomembrane at splash pad locations will be installed in accordance with Technical Specification Section 02770, Part 3 – Geomembrane Installation. Detail D/3 on Construction Drawing 6 notes the splash pad will be extrusion welded on 4 sides.

7. *Subgrade Preparation and Earthwork – Demonstrate how the construction process for the earthwork movement of soil between Cell 4B and Cell 3 will not cause cross-contamination of impacted soil to clean areas. Please note as presented in Interrogatory 09, it must be demonstrated that the levels of radiation (contamination) in Cell 4B subgrade are acceptable before a construction permit can be issued and the liner system installed.*

Soil and rock excavated during earthwork for the construction of Cell 4B will be placed onto Cell 3 in a manner that precludes vehicles from operating directly on the existing Cell 3 surface. This will be accomplished by placing soil material at the edge of Cell 3 and pushing it over the existing surface of Cell 3. Dozers will be used on top of this material to continually push material out in front of the dozer, thereby allowing the dozer to always be on top of Cell 4B soil material and not contact the Cell 3 materials.

Technical Specification Section 02200, Paragraph 3.05 and Construction Drawing No. 2 detail the requirements for stockpiling excavated soil. No limit is placed on the height of the stockpiled soil in the Technical Specifications. How does the height (i.e., weight) of the stockpiled soil affect slope stability of the cut (i.e., West Berm slope)? Demonstrate that soil stockpile slopes will be stable under foreseeable future conditions. How does stockpiling of soils (loading) just west of the West Berm and subsequent removal of that soil affect the performance of the West Berm?

The maximum height of the stockpiled soil to be approximately 20 feet. This condition is modeled in stability analyses for the west slope of the cell, as referenced in the interrogatory response to *Dike Stability*. Stability analyses indicate the cut slope through the western perimeter has a factor of safety that exceeds the minimum required factor of safety equal to 1.5. These analyses are provided in Exhibit A. Drawings have been updated to include notes related to stockpile construction maximum height, maximum side slope, and distance from the top of the Cell 4B slope.

Please provide technical specifications on how each of the cut slope surfaces will be completed (i.e., compacted,) to ensure strength and stability of the slopes for Cell 4B's operation.

Cut slope surfaces will be constructed in accordance with Section 02200 (Earthwork) and Section 02220 (Subgrade Preparation) of the Technical Specifications. Slopes will cut from existing compacted or native materials and will be shaped in accordance with safe slope conditions determined in the slope stability calculations.

Demonstrate how the outside slope of the south berm of Cell 4B and the upgradient portion of the west berm of Cell 4B will be completed to prevent excessive erosion and potential slope failure.

There is no outside exposed slope on the west berm of Cell 4B because it is a cut slope below existing grades. The outside slope of the south berm of Cell 4B will be completed in accordance with Technical Specifications Section 02200 (Earthwork). The construction of the south berm of Cell 4A, which has minimal erosion on the outside, was performed using similar methods to those outlined in Project Documents for Cell 4B.

Provide specifications for drilling and ripping to support any blasting the Contractor might perform. Demonstrate what level of blasting will be required to remove rock to the grades/elevations for Cell4B as indicated in the Drawings and how the blasting will affect the stability of the surrounding berms in place, effect the functionality of the surrounding berms which will be cut to serve as the side slopes for Cell 4B, and effect any other components of Cell 4B and adjacent Cells. Demonstrate the effect blasting will have on the effective permeability and speed of water travel through underlying material. The Design should demonstrate that removal of the rock by blasting does not compromise the design and functionality of Cell 4B and other Cells. The current design and Technical Specification Section 02200 places a requirement on the Contractor that blasting shall not cause damage. Please define "damage" both in terms of nearby dike stability, but also foundation permeability under Cell 4B.

Please provide detail what level of blasting is necessary to construct Cell 4B without causing damage and specific points of compliance the Contractor should be expected to meet such that damage is not caused.

The effects of blasting are described in the response to the first interrogatory question *Dike Integrity.*

Please define "Project Manager" as used in the Technical Specifications. Section 02200 of the Technical Specifications uses the term "Project Manager". The role of Project Manager is not defined in the Technical Specifications.

Technical Specifications have been revised to remove instances of "Project Manager" and replace these occurrences with the appropriate personnel. A revision of the technical specification is provided as Exhibit F.

Please revise Subsections 2.01 (A through C) and 3.02 (A through F) of Section 02220 (Subgrade Preparation) and Section 7.3.3 of the Construction Quality Assurance Plan to incorporate applicable requirements contained in ASTM D 6102-06.

Section 02220 of the Technical Specifications has been revised to include the applicable requirements contained in ASTM D 6102-06 and is provided as Exhibit F.

The provision in Paragraph 2.01B of the Technical Specification Section 02220 (Subgrade Preparation) stating that desiccation cracks less than or equal to ¼-inch in width in the subgrade prior to liner construction are acceptable is not supported and is apparently inconsistent with the requirements of ASTM D 6102-06. Please demonstrate that desiccation cracks of ¼ inch width or less are acceptable or remove this permissible crack width value from the specifications. The specifications should detail how any desiccation cracks observed in the subgrade will be remedied. A requirement that the subgrade surface be checked for cracks and such cracks be remedied should be included in specifications and/or in the Construction Quality Assurance Plan as applicable.

The crack width has been removed from Section 02220. The revised Section 02220 is provided as Exhibit F.

The provision in Technical Specification Section 02220 (Subgrade Preparation) Paragraph 2.01C stating that subgrade soil shall consist of on-site soils that are free of particles greater than 3 inches in longest dimension is apparently inconsistent with typical GCL and/or FML manufacturer's recommendations for subgrade soil used for GCL and FML installation applications (e.g., see GSE 2008, Section 4.5). Please demonstrate that such a large maximum particle dimension is acceptable or revise this requirement to be consistent with typical GCL / FML manufacturer's recommendations. Please also indicate that such soil shall be well graded material (to be consistent with additional typical GCL / FML manufacturer's recommendations for subgrade soil) or provide justification for not including this requirement in the specifications.

It is stated in Technical Specification Section 2.01 Paragraph A that, "Subgrade surface be free of protrusions larger than 0.5 inches. Any such observed particles shall be removed prior to placement of geosynthetics." The three inch particle refers to soil particles, whereas the 0.5 inch protrusion height refers to rock protruding more than 0.5 inches through the surface of the subgrade. The revised section is consistent with manufacturer's recommendations for subgrade soil used for GCL and FML installation applications.

Please define "fill" as used in Subsection 2.01 of the Section 02200 (Earthwork) and "subgrade soil" as used in Subsection 2.01 (C) of Section 02220 (Subgrade Preparation) of the Technical Specifications and clearly distinguish between these two types of fill material. Subsection 2.01 of the Section 02200 (Earthwork) states that fill will consist of material free from rock larger than 6-inches. Subsection 2.01 (C) of Section 02220 (Subgrade Preparation) states that subgrade soil shall be free of particles greater than 3-inches in longest dimension. If "fill" is referring to the material that is suitable for use in constructing the berms, and "subgrade soil" is select fill material suitable for use in constructing/developing the subgrade surface, then define them as such.

The requirements for the soil used in earthwork components such as berm construction, subgrade preparation, and anchor trench construction, are specifically described in the Technical Specifications under each appropriate heading. Material and placement requirements of fill soils and subgrade soils are described in revised Section 02200 and Section 02220, respectively.

Subsection 3.04 (D) of Section 02200 (Earthwork) of the specifications calls for the fill to be compacted in lifts no greater than 12-inches, to 90% of maximum density and to +/- 4% of optimum moisture content (per ASTM 1557). Subsections 3.03 (E) and 3.04 (C) of Section 02220 (Subgrade Preparation) call for fill to be compacted in lifts no greater than 8-inches, to 90% of maximum density and to +/- 3% of optimum moisture content (per ASTM 1557). Due the critical nature of the fill placement for the slopes and the subgrade fill placement for the subgrade, the DRC judges that all the fill placed needs to be compacted in lifts no greater than 8-inches, to 90% of maximum density and to +/- 3% of optimum moisture content (per ASTM 1557). Please revise the specifications accordingly. Note that the compaction requirements cited in Section 3.3.4 of the design report are inconsistent with the Technical Specifications that call for 8-inch lifts and +/- 3% of optimum moisture. Please resolve this contradictory information.

Compaction requirements have been revised to reflect soil should be placed in either a "12-inch loose lift" or "a loose lift that results in a compacted lift thickness no greater than 8-inches," with a relative compaction of 90% and a moisture content $^{+}$/- 3% of optimum moisture content as determined by ASTM D 1557.

8. <u>*Cell 4B Aggregates Backfill and Compatibility of Materials -*</u> *The Design Report states that the pH range for the tailings is between 1 and 2. The Design Report also states that the aggregate backfill materials and sand (in sand bags) shall have a carbonate content loss of no more than 10 percent by weight based on UDOT standard specifications. Please provide the basis for determining the requirement of the aggregate and sand. Demonstrate how much the specified aggregate and sand will deteriorate under a pH of 1 to 2 over the design life of Cell 4B, including the change in permeability of the aggregate and sand with time and how the change in permeability will affect the drainage of liquids in the slimes drain (both the header and strip drains) with time; and how the head on the primary liner and secondary liner is affected over time due to the change in aggregate and sand permeability.*

Aggregate and sand requirements were selected based on their use in Cell 4A for an identical slimes drain system design. The UDEQ previously approved the use of these materials in the design approval letter for Cell 4A issued on 25 June 2007.

Carbonate loss from the aggregate material is expected to have negligible effect on the slimes drain system because the lost carbonate will dissolve into solution instead of clogging pore space in the aggregate or sand. Even though clogging of the slimes drain aggregate is not anticipated, the calculation to compute the amount of time needed to drain liquids from Cell 4B after it is filled with water and tailings (*Slimes Drain Calculation*) includes multiple built-in safety factors attributed to chemical clogging, installation defects, and creep.

On Construction Drawing 6, Section I/6, the side slope riser system for liquid removal from slimes drain and leak detection includes aggregate bedding. Demonstrate how the slope stability (i.e., resistance to sliding) of the aggregate is affected by the low pH environment. This is of particular concern if the risers and bedding are placed on a 2H:1V slope.

Construction Drawings have been revised to reflect the southeast corner of the cell containing the sideslope riser pipes shall be graded at a 3:1 slope (Exhibit H). In the case that the aggregate bedding reduces in volume due to carbonate loss, settlement of the pipe within the aggregate layer would occur very slowly over a long period of time, and will not affect the stability of the side slope riser system. The weight of the tailings deposited over the pipe will also weight the pipe to the sideslope, preventing sliding or other movement.

9. <u>GCL, Primary Liner, Secondary Liner, and Leak Detection System</u>

Refer to the Design Report: *Please indicate and justify in the Design Report (Section 3.4.3) that the leak detection system has been designed to be compliant with the following performance standards (the same as or equivalent to those that were also specified Part I.D.6. of the Groundwater Discharge Permit) for Cell 4A, which at a minimum, included):*

a. *"Leak Detection System (LDS) Maximum Allowable Daily Head – the fluid head in the LDS shall not exceed 1 foot above the lowest point in the lower membrane liner.*
b. *LDS Maximum Allowable Daily Leak Rate – shall not exceed some specified number of gallons/day." [value used here should equal to the maximum flow rate to the Cell 4B sump as determined in the final (approved) Action Leakage Rate calculation for Cell 4A, e.g. ≤ 24,160 gal/day]*

Part a. The Action Leakage Rate (ALR) calculation shows that the maximum head on the secondary liner does not exceed 0.15 mm, which is much less than the required 12 inch (1 foot) maximum. This information is already stated in the second paragraph of Section 3.4.3 of the design report.

Part b. As noted by URS in the interrogatory question, the ALR computed for Cell 4B is different from the ALR computed for Cell 4A. An identical design approach and safety factors were used in the calculation for Cell 4A and Cell 4B. However, because the cells have different dimensions, the ALR is not the same for both cells.

Refer to the Design Calculation: Comparison of Flow through a Compacted Clay Liner (CCL) and Geosynthetic Clay Liner (GCL). *In the definition of input variables for equation (4), t_{LCL} is 200 mils. Subsequent value for t_{LCL} is corrected to 300 mils, but in actual calculations 200 mils is used. Please clarify what is the correct value for t_{LCL} and how does the head and flow rate for the GCL change; and how does the change in head and flow rate compare with a compacted clay liner?*

The correct value of the leak detection system (geonet) thickness, t_{LCL}, is 300 mils and the supporting calculation has been edited to reflect this inconsistency in the analysis. A revised version of this calculation is provided to UDEQ as the attached Revised Comparison of Flow Through Compacted Clay Liner and Geosynthetic Clay Liner Calculation Package (Exhibit D).

The increase of geonet thickness from 200 to 300 mil did not change the previously calculated amount of liquid head on the secondary geomembrane. The revised calculations show that the amount of flow through the secondary liner system with a CCL is 4.74 times greater than the flow through the secondary liner system with GCL for a liquid head of 0.20 inches.

Refer to the Design Report, Construction Quality Assurance Plan, and the Design Calculation: Action Leakage Rate*. Please demonstrate that a low factor of safety of 1.1 for flow in the geonet is acceptable, since long-term degradation of the installed geonet's flow capacity (e.g., through gradual partial degradation of the geonet core as a result of long-term exposure to the acidic solutions contained in the cell) could significantly lower this factor of safety, thus resulting in a higher head on the secondary liner. Also, the possibility exists that the geonet might become damaged during/following installation or the installation methods otherwise result a reduced geonet capacity. Possible means whereby the geonet might become damaged during or following installation or the installation methods otherwise result a reduced geonet capacity are described below. Please revise Section 13 (Geonet) of the Construction Quality Assurance Plan to include measures to minimize/preclude damage to the geonet so as to minimize the potential for reduced geonet function occurring as a result of geonet and primary liner installation activities.*

The overall factor of safety equal to 1.1 for flow in the geonet is acceptable because it has multiple built-in factors of safety to the calculation. These safety factors account for the geonet's reduced flow capacity due to intrusion, biological clogging, chemical clogging, and creep, resulting in an overall factor of safety of 3.1. The measures and precautions outlined in Section 13, "Geonet" of the CQA Plan are specified to ensure minimal installation damage and defects. Furthermore, the geonet is comprised of a high density polyethylene polymer, which is resistant to acidic environments.

Refer to the Design Report and the Cell 4B Technical Specifications*: Please revise the Design Report and the technical specifications as needed to include a description of the measures that need to be taken during installation to ensure the GCL is properly hydrated prior to covering it with geomembrane material.*

To evaluate the performance of the GCL, an analysis of the anticipated flow through the GCL with time will be discussed. The hydraulic conductivity data presented herein was submitted to the UDEQ in a letter report, dated 31 August 2007, presenting the results of a Geosyntec study of hydrated GCL used in the Cell 4A liner system. Based on results of the field and laboratory study, the UDEQ approved the hydration of the GCL to 50% moisture content.

Since the construction and final acceptance of Cell 4A by the UDEQ, Denison has performed additional GCL hydraulic conductivity testing with a high pH solution. Results from the permeability testing are presented in Exhibit J. The high pH solution was created in the laboratory, as was done for the testing performed previously for Cell 4A. The laboratory has used this solution to perform permeability tests on the GCL at a 17% moisture content. Results from this laboratory study demonstrate GCL with a 17% moisture content have a lower permeability than GCL samples hydrated to 50%.

A permeant time of travel analysis using the permeability data obtained from laboratory permeability testing is presented below.

For this analysis, the following equation will be used:

$$Q = kiA \qquad \text{(Equation 1)}$$

Where:

Q = flow through the GCL (cm^3/sec)

k = permeability of the GCL, from test data (cm/sec)

i = hydraulic gradient

A = area (cm^2), use 1

Based on the Action Leakage Rate calculation package provided in the Cell 4B Design Report, the quantity of liquids passing through the primary geomembrane into the leak detection system will result in a very small head (0.17 mm) on the secondary geomembrane. Conservatively assuming that the secondary geomembrane is non-existent and the liquid cannot drain laterally, the small head would act to drive the liquid vertically down into the GCL. Therefore, the head on the GCL will be 0.017 cm.

Given a thickness of the GCL as 0.3 inches, or 0.76 cm, the hydraulic gradient can be estimated as follows:

i = 0.017 cm / 0.76 cm = 0.022

Placing the hydraulic gradient, area, and the permeability into Equation 1, results in a flow rate. Permeability of the GCL was measured after permeation of 0.25, 0.50, 0.75, 1, 1.5, and 2.0 pore volumes of tailings solution. After permeation of 0.25 pore volumes through the GCL at a moisture content of 17%, the resulting permeability of approximately 1.0 x 10^{-8} cm/sec can be inserted into the equation, resulting in the following:

Q = (1.0 x 10^{-8} cm/sec) x (0.022) x (1 cm^2) = 2.2 x 10^{-10} cm^3/sec = 6.94 x 10^{-3} cm^3/year

Based on a typical GCL thickness of 0.76 cm and a porosity of 0.70, ¼ pore volume per square centimeter can be estimated as follows:

Pore volume = 0.25 x 0.76 cm x 1 cm^2 x 0.70 = 0.13 cm^3

The time for ¼ of a pore volume of pH 1 liquid to permeate through the GCL hydrated to a moisture content of 17% can be estimated as follows:

$T = V_p / Q$

T = 0.13 cm^3 / 6.94 x $10^{-3}$$cm^3$/year = 19 years

This calculation is repeated in incremental steps for the permeation of 0.5, 0.75, 1.0, 1.5, and 2.0 pore volumes and is illustrated in the spreadsheet presented in Exhibit J. The calculation of the time required to pass fractions of a pore volume is repeated for each increment and the number of years required for each permeation event are summed. Based on the boundary conditions and test data presented above, the permeant would require approximately 270 years to permeate 2 pore volumes of pH 1 liquid through the GCL at a moisture content of 17%.

Based on results of the above analysis using a GCL at a 17% moisture content, we have concluded that the amount of time required for 2 pore volume of liquid to pass through the GCL is greater than the design life required to meet Best Available Technology (BAT) requirements. The laboratory test results demonstrated that a GCL with a moisture content of 17% will have a lower permeability than a GCL with a moisture content of 50%. Therefore, GCL with a moisture content 17% or greater is sufficient for use in this project. Based on Geosyntec's experience with GCL material and manufacturer data (CETCO, 2008), the as-received moisture content of a GCL is typically greater than 17% and will be acceptable for deployment without additional hydration.

10. Radiation Survey to Demonstrate Acceptable Subgrade Conditions Prior to Liner System Construction

Please provide an evaluation that demonstrates that the existing soil subgrade has radiation and contamination levels that are acceptable. One possible scenario to minimize contamination and meet Best Available Technology (BAT) requirements is to base the design of the liner system for Cell 4B on a clean and stable subgrade, i.e., one with background soil levels. Therefore, demonstration of the absence of wind-blown contamination from other nearby sources of uranium tailings at the proposed Cell 4B site is important before construction begins. Another scenario is to demonstrate that the levels of any soil contamination left under the new liner design will have no adverse impact on local groundwater quality or the environment. In either case, the applicant needs to demonstrate and justify that any soil concentration level proposed as a cleanup standard has both technical and regulatory justification. Consequently, it is imperative that this evaluation be submitted to the DRC and is approved prior to issuance of the Construction Permit. Also, if the implementation of the plan results in modifications to the proposed subgrade and liner system, the respective modifications will need to be submitted to the DRC for review and concurrence prior to liner construction.

The area proposed for the construction of Cell 4B has not been previously used as a radioactive material storage location. No impacts to the soil below the existing ground surface of the area are anticipated. The Cell 4B area is generally up wind of the existing tailings cells and has exhibited no impact from windblown tailings. A recent survey of the Cell 4B area indicates no areas of radiological contamination. It is highly unlikely that any windblown material, if present, could migrate through 20+ feet of undisturbed soil and rock, and cause contamination of the base of the liner system. No cleanup or justification of residual contamination of the liner subgrade is warranted.

If you have any additional questions please feel free to contact me at (303) 389-4160.

Yours very truly,

DENISON MINES (USA) CORP.

Harold R. Roberts
Executive Vice President – U. S. Operations

cc: Ron F. Hochstein, DMC
Gregory T. Corcoran, Geosyntec

Attached:
- Exhibit A – Slope Stability Analysis Calculation Package
- Exhibit B – Seismic Deformation Analysis Calculation Package
- Exhibit C – Revised Pipe Strength Analysis Calculation Package
- Exhibit D – Revised Comparison of Flow Through Compacted Clay Liner and Geosynthetic Clay Liner Calculation Package
- Exhibit E – Revised Action Leakage Rate Calculation Package

DENISON MINES

Exhibit F – Revised Technical Specifications
Exhibit G – Revised Construction Quality Assurance (CQA) Plan
Exhibit H – Revised Construction Drawings
Exhibit I – Revised Cell 4B Design Report
Exhibit J – GCL Permeability Test Results and Calculations
Figure 1 – Mill Site Drainage Basins

References:

CETCO, 2008. Technical data available from website at www.cetco.com

GeoStudio 2004. SLOPE/W, version 6.22. Computer Modeling Software.

Uni-Bell PVC Pipe Association, (1997), "UNI-TR-5-97 – *The Effects of Ultraviolet Aging on PVC Pipe."*

EXHIBIT A

SLOPE STABILITY ANALYSIS CALCULATION PACKAGE

Geosyntec consultants

COMPUTATION COVER SHEET

Client: **DMC** Project: **White Mesa Mill – Cell 4B** Project/ Proposal No.: **SC0349** Task No. **02**

Title of Computations **SLOPE STABILITY ANALYSIS**

Computations by:	Signature		07/01/08 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		7/2/08 Date
	Printed Name	Steven Fitzwilliam, P.E.	
	Title	Associate	
Computations Checked by:	Signature		7/2/08 Date
	Printed Name	Steven Fitzwilliam, P.E.	
	Title	Associate	
Computations backchecked by: (originator)	Signature		07/01/08 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Approved by: (pm or designate)	Signature		7/28/08 Date
	Printed Name	Gregory Corcoran, P.E.	
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	M. Lithgow	Date:	07/02/08	Reviewed by:	S. Fitzwilliam	Date:	07/08/08
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

SLOPE STABILITY ANALYSES
WHITE MESA MILL, CELL 4B
BLANDING, UTAH

OBJECTIVE

This calculation includes static and seismic slope stability analyses for the final earthen berms and interim waste/tailings slopes associated with operation of Cell 4B at the White Mesa Mill facility located in Blanding, Utah.

The purpose of the stability analyses is to evaluate final slope stability and operational conditions required to maintain a minimum factor of safety of approximately 1.5 for final berm slope conditions, 1.3 for interim and temporary slope conditions, and 1.1 for seismically-loaded slope conditions based on the proposed design of the cell and its liner system.

METHODOLOGY

Two-dimensional static slope stability analyses were performed using the computer program SLOPE/W 2004 (Version 6.17) developed by Geo-Slope International Ltd. (2004). The results of the slope stability analyses are based on Spencer's Method of Slices for moment and force equilibrium by assuming a constant interslice shear force function. The analyzed slopes were kinematically modeled using either circular or linear/circular sliding surfaces.

For each condition analyzed, the program will search for the sliding surface that produces the lowest factor of safety. Factors of safety are defined as the ratio of the shear forces/moments resisting movement along a sliding surface to the forces/moments driving the instability.

Cross sections were selected on each of the four (4) perimeter berms to represent the critical slope conditions on each slope. The location of each cross section within Cell 4B is shown on Figure 1. The first cross section, Section A-A', is a west-east cross section that spans Cells 4A and 4B, with berm slopes inclined at approximately 2:1 (Horizontal:Vertical) and a base grade sloping southeast at approximately 1 percent. Section B-B' is a north-south cross section through the highest portion of the southern embankment of Cell 4B with berm slopes inclinded approximately 2:1 on the inside of Cell 4B and 3:1 outside of the cell. Section B-B' depicts the conditions with the cell full of tailings, and the analysis was performed for the embankment slope outside of Cell 4B. Section C-C' is also a north-south section through the northern embankment that spans Cells 3 and 4B. The slope within Cell 4B will be steepened from an inclination of approximately 3:1 to an inclination of approximately 2:1. The embankment slope within Cell 3 is inclined at an inclination of approximately 3:1. Section D-D' is an east-west cross section through the western embankment slope. Section D-D' will be a cut slope into the native soil and bedrock at an inclination of approximately 2:1. Section D-D' will have a surcharge load from the stockpile at the top of the slope. The surcharge load will be set back from the top of the slope at least 20 feet to allow for access along the top of the western slope. The stockpile has been modeled as a 20-foot high embankment with a slope inclination of 2:1.

Sections A-A', B-B', C-C', and D-D' were modeled for four conditions. These four conditions included static analyses for the final as-built condition for embankments for Cell 4B, pseudo-static evaluation of the seismic loading conditions, evaluation of the yield acceleration, and interim construction loading.

Pseudo-static evaluations for slope stability were performed for a seismic acceleration of 0.1g in accordance with the Cell 4 Design Report (UMETCO, 1988) as referenced by MFG, Inc. in a letter to International Uranium Corporation (presently Denison Mines) dated 27 November 2006.

A phreatic surface was assumed for Sections A-A', B-B', and C-C' within the embankment berm, the liner system, and in the tailings. Through the embankment for Sections A-A' and C-C' the phreatic surface extends from the top of the contact with the tailings within Cells 3 and 4A to the toe of the slope inside of Cell 4B. For Section B-B' the phreatic surface extends from the top of the contact with the tailings within Cell 4B (assumed filled) to the toe of the outside slope of the southern embankment of Cell 4B. This condition was considered as the conservative case, which would occur if the double liner system within the cells failed, which is not anticipated.

Interim loading was considered for the four cross sections from construction and maintenance vehicle traffic on the access roads and haul roads on top of the embankment berms. A conservative AASHTO H 20 loading was assumed for the interim construction and maintenance vehicle loading. The 16 kip load (32 kips on the back axle with half the load applied at each side of the axle) was applied 2 feet and 12 feet from the top of the slope.

One final cross section was modeled that showed Cell 4A filled with tailings and Cell 4B partially filled during interim filling conditions. This cross-section was modeled to show interim-conditions to determine the steepest slope at which the tailings could be placed.

Cell 4B will be constructed with the following liner system on both the bottom area and side slopes:

- Slimes Drain System (Cell bottom only);
- 60 mil smooth HDPE geomembrane (Primary Liner);
- Geonet Drainage Layer (Leak Detection System);
- 60 mil smooth HDPE geomembrane;
- Geosynthetic Clay Liner (GCL); and } (Composite Secondary Liner)
- Prepared Subgrade.

During operations, tailings/waste deposits are expected to be pumped into the cell below the water surface where the tailings will settle out creating a gradual build-up of solids along the base of the cell. The tailings will be pumped into the cell from north to south, beginning at the splash pad locations located along the northern slope of the cell. Based on a review of existing operations in the existing cells at the facility, we have assumed the pond will be filled to approximately one-half of full height with liquids (approximately 20 feet) and that tailings may extend up to approximately 5 feet above water levels in the cell. In the modeling for the interim tailings evaluation, the phreatic surface (water surface) is assumed to apply to only the waste and liner materials, since the composite liner system minimizes infiltration of liquids into the underlying subgrade/foundation.

MATERIAL PARAMETERS

Based on site specific direct shear testing of the liner system components for Cell 4A, the interface shear strength between the geonet and smooth geomembrane is approximately 11 degrees (Attachment A).

Written by:	M. Lithgow	Date:	07/02/08	Reviewed by:	S. Fitzwilliam	Date:	07/08/08
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

Based on existing operations at the site, tailings/waste deposits are anticipated to be primarily fine sands with silt and some clay (Attachment B). We have conservatively estimated a total unit weight of 125 pounds per cubic foot (pcf) for this material based on Table 6 from the Naval Design Manual for Soil Mechanics DM7-01, (Attachment C). The value selected is based on the minimum wet weight (under loose placement to simulate the tailings settling underwater) for a similar type of material. Based on Figure 3.7 (Attachment D) for a 0% relative density silty sand, a friction angle of 26 degrees could be expected. We have conservatively estimated a friction angle of 25 degrees, with no cohesion, for these materials.

Geosyntec reviewed previous geotechnical investigation for the site performed by others, including a memorandum from MFG, Inc. (MFG) dated 7 June 2006 and a follow-up letter dated 13 July 2006 (Attachments E and F). MFGs follow-up letter described their geotechnical investigation at the site, which included an exploratory boring through the existing berm between Cell 4A and 4B and a triaxial compression test on the recovered soil samples. Based on our geotechnical investigation and our review of the existing data, we have selected material properties for the fill material in the berm consistent with the properties used by MFG in their previous slope stability analyses for the embankment between Cell 4A and 4B (Unit Weight = 137 pcf, phi = 26, cohesion = 900 psf). As the embankment fill material will be derived from the on-site soil, similar to the embankment between Cell 4A and 4B, the same berm material properties were used for the embankment fills.

Construction loading was modeled using AASHTO H 20 loading for each cross section to model construction traffic (Attachment G).

The existing soil to be left in place for the cut slopes has been assumed to be similar to the eolian/loess materials encountered in the seven geotechnical borings performed as part of the Geosyntec geotechnical investigation for the project. As such, soil properties based on laboratory testing were selected for the eolian/loess materials (Unit Weight = 135 pcf, phi = 24, cohesion = 1,000 psf) (Attachment H).

Due to the low interface strength of the liner system, failures are not anticipated to extend beneath the liner system into the foundation (Dakota Sandstone). As such, the foundation system has been modeled as bedrock within the slope stability program (i.e., impenetrable) to allow slip surfaces within the liner system.

The soil material to make up the 20-foot high stockpile will be derived from the same material as that used to make up the fill for the embankment berms. As the required compaction for the stockpile will be less than that of the embankment berms, soil properties for the stockpile have been reduced from those for the embankment berms. The selected material properties for the stockpile are Unit Weight = 130 pcf, phi = 26, cohesion = 200 psf.

SLOPE STABILITY RESULTS/RECOMMENDATIONS

As discussed above, four cross-sections were analyzed which represent critical conditions for Cell 4B.

Numerous potential failure surfaces were performed to evaluate various slip surface geometries and to identify the critical slip surface for each cross-section and conditions. The results of the slope stability analyses for Cross Sections A-A', B-B', C-C', and D-D', and the interim tailings slope are presented in the Table 1.

The slope stability analyses results are presented as Figures 2 through 18.

For the cross sections evaluated to assess the yield acceleration of the slope, the critical failure surface tends to recede from the slope face with respect to the static analyses for the cross section. For these conditions the computer program was allowed to search for the critical failure surface with the lowest factor of safety provided that the base of the failure surface remained within the berm. If allowed to search for the critical failure surface with the absolute lowest factor of safety for the cross sections analyzed, the critical failure surface would extend down onto the liner of the adjacent cell. As this is not a kinematically feasible condition for the cross sections analyzed in these analyses, the base of the critical failure surface was fixed to remain within the berm to evaluate the yield acceleration of the slopes.

These results indicate the minimum factor of safety of 1.3 is met during and after filling operations. We recommend that operations at the site limit the tailings/waste deposits slope to inclinations of 7H:1V or flatter.

REFERENCES

GeoSlope International, LTD (2004) SLOPE/W Version 6.17.

U.S. Navy. 1986. Soil mechanics. NAVFAC Design Manual DM-7. Washington, DC.

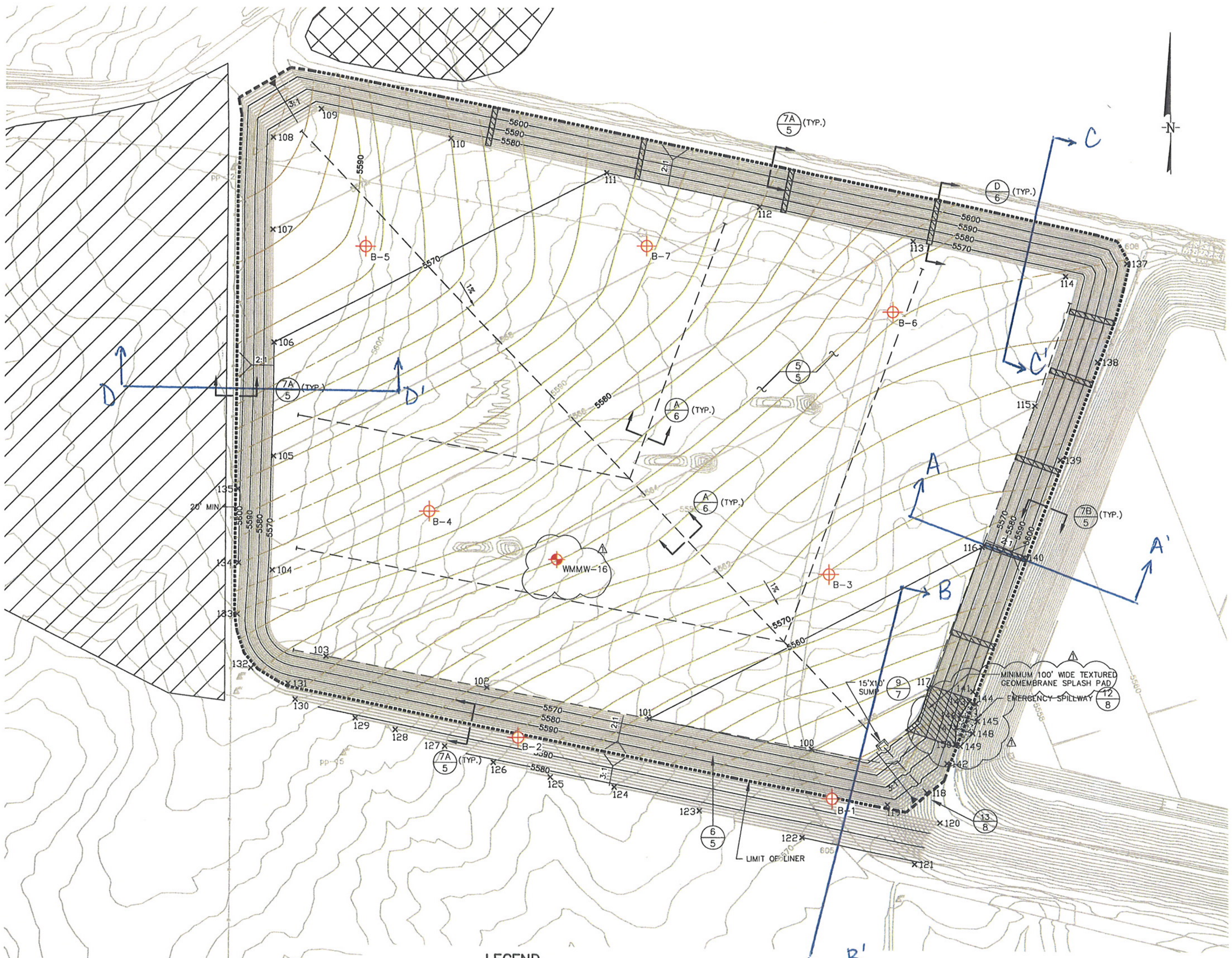
MFG , Inc. 2006. "Technical Memorandum: White Mesa Stability Analysis," dated 7 June 2006.

MFG, Inc. 2006. "Draft, Soil Property Verification and Slope Stability Analyses, Earthen Embankment between Cells 4A and 4B, IUC White Mesa Project, Blanding, Utah," dated 13 July 2006.

MFG, Inc. 2006. "White Mesa Uranium Facility, Cell 4 Seismic Study, Blanding, Utah," dated 27 November 2006.

TABLE 1
SUMMARY OF SLOPE STABILITY ANALYSES
Denison Mines - White Mesa Mill, Cell 4B
Blanding, Utah

Cross Section	Condition	Slope Inclinations	Yield Acceleration	Minimum Factor of Safety	Calculated Factor of Safety
A-A'	Static - Final Slope Configuration	2:1 (inside both Cell 4B and 4A)	--	1.5	2.9
A-A'	Static with Construction Loading		--	1.3	2.3
A-A'	Seismic Loading (0.1g)		--	1.1	2.2
A-A'	Yield Acceleration		0.48	1.0	1.0
B-B'	Static - Final Slope Configuration	2:1 (inside Cell 4B), 3:1 (outside Cell 4B)	--	1.5	3.1
B-B'	Static with Construction Loading		--	1.3	2.2
B-B'	Seismic Loading (0.1g)		--	1.1	2.3
B-B'	Yield Acceleration		0.47	1.0	1.0
C-C'	Static - Final Slope Configuration	2:1 (inside Cell 4B), 3:1 (inside Cell 3)	--	1.5	2.7
C-C'	Static with Construction Loading		--	1.3	2.2
C-C'	Seismic Loading (0.1g)		--	1.1	2.2
C-C'	Yield Acceleration		0.52	1.0	1.0
D-D'	Static - Final Slope Configuration	2:1 (inside)	--	1.5	3.5
D-D'	Static with Construction Loading		--	1.3	2.1
D-D'	Seismic Loading (0.1g)		--	1.1	2.7
D-D'	Yield Acceleration		0.65	1.0	1.0
Tailings Slope	Interim Tailings Slope	7:1 (tailings slope)	--	1.3	1.3



100 50 0 100 200

SCALE IN FEET

POINT TABLE			
POINT #	NORTHING	EASTING	ELEVATION
100	319447.2859	2577164.8929	5558
101	319507.2322	2576844.3805	5560
102	319567.1785	2576523.8681	5562
103	319627.1248	2576203.3557	5564
104	319797.6436	2576097.6307	5566
105	320023.8073	2576099.9958	5568
106	320249.0377	2576100.5478	5570
107	320472.5420	2576097.7471	5572
108	320655.2385	2576098.0345	5574
109	320711.5023	2576192.6462	5574
110	320654.7732	2576451.7313	5572
111	320588.0182	2576759.0177	5570
112	320520.2894	2577064.4125	5568
113	320453.5570	2577371.7429	5566
114	320384.0551	2577673.6935	5564
115	320127.7348	2577612.7493	5562
116	319848.5293	2577507.3503	5560
117	319569.1802	2577401.6727	5558
118	319361.7901	2577396.4633	5598
119	319337.6725	2577316.9812	5598
120	319302.0250	2577421.5901	5598
121	319220.3604	2577371.2400	5568
122	319271.8283	2577146.1490	5570
123	319324.7464	2576944.2404	5574
124	319371.2425	2576774.2086	5578
125	319390.5815	2576647.8346	5576
126	319419.1551	2576536.3841	5578
127	319450.7637	2576440.2498	5582
128	319482.9638	2576341.1792	5586
129	319505.2211	2576261.0810	5588
130	319541.3069	2576142.7230	5592
131	319572.6925	2576128.7102	5598
132	319602.9949	2576055.3321	5596
133	319709.7865	2576028.4338	5598
134	319811.6839	2576031.0071	5600
135	319957.9848	2576030.0557	5602
136	320449.2987	2577774.1998	5608
137	320409.2490	2577796.0277	5608
138	320214.1456	2577737.3159	5506
139	320020.3153	2577662.5848	5604
140	319828.0217	2577589.1862	5602
141	319562.1406	2577487.9916	5600
142	319419.5433	2577436.4654	5598
143	319542.8711	2577481.2798	5600
144	319538.3476	2577494.0228	5600
145	319503.5037	2577496.5228	5596.3
146	319515.3611	2577463.1200	5596
147	319490.7627	2577454.7052	5596
148	319479.1526	2577487.4112	5596.3
149	319453.9250	2577462.8219	5600
150	319457.5650	2577452.5678	5600

LEGEND

5560	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
5570	APPROXIMATE ROCK ELEVATION (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (10' CONTOUR)
	LIMIT OF LINER
	MONITORING WELL
	STOCKPILE AREA (NOTE 2)
	SPLASH PAD
1%	SLOPE DIRECTION AND GRADE
B-1	APPROXIMATE SOIL BORING LOCATION
	LEAK DETECTION PIPE TRENCH
	4' THICK SOIL COVER PLACEMENT AREA (NOTE 3)

NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM DENISON MINES (USA) CORP.
2. CONTRACTOR SHALL SEGREGATE SOIL AND ROCK MATERIALS INTO TWO SEPERATE STOCKPILES IN STOCKPILE AREA.
3. STOCKPILE TO BE CONSTRUCTED AT SLOPES NO STEEPER THAN 2H:1V AND A MINIMUM OF 20 FT FROM THE CREST OF THE SLOPE. STOCKPILE WITHIN 100 FT OF CREST OF SLOPE SHALL NOT EXCEED 20 FT IN HEIGHT.

REV	DATE	DESCRIPTION	DRN	APP
1	01/09/09	INTERROGATORY ROUND 1	MD	GTC

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
SAN DIEGO, CA 92127
PHONE: 858.674.6559

DENISON MINES
6425 S. HIGHWAY 191
P.O. BOX 809
BLANDING, UTAH 84511
PHONE: 858.674.6559

TITLE: BASE GRADING PLAN

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.

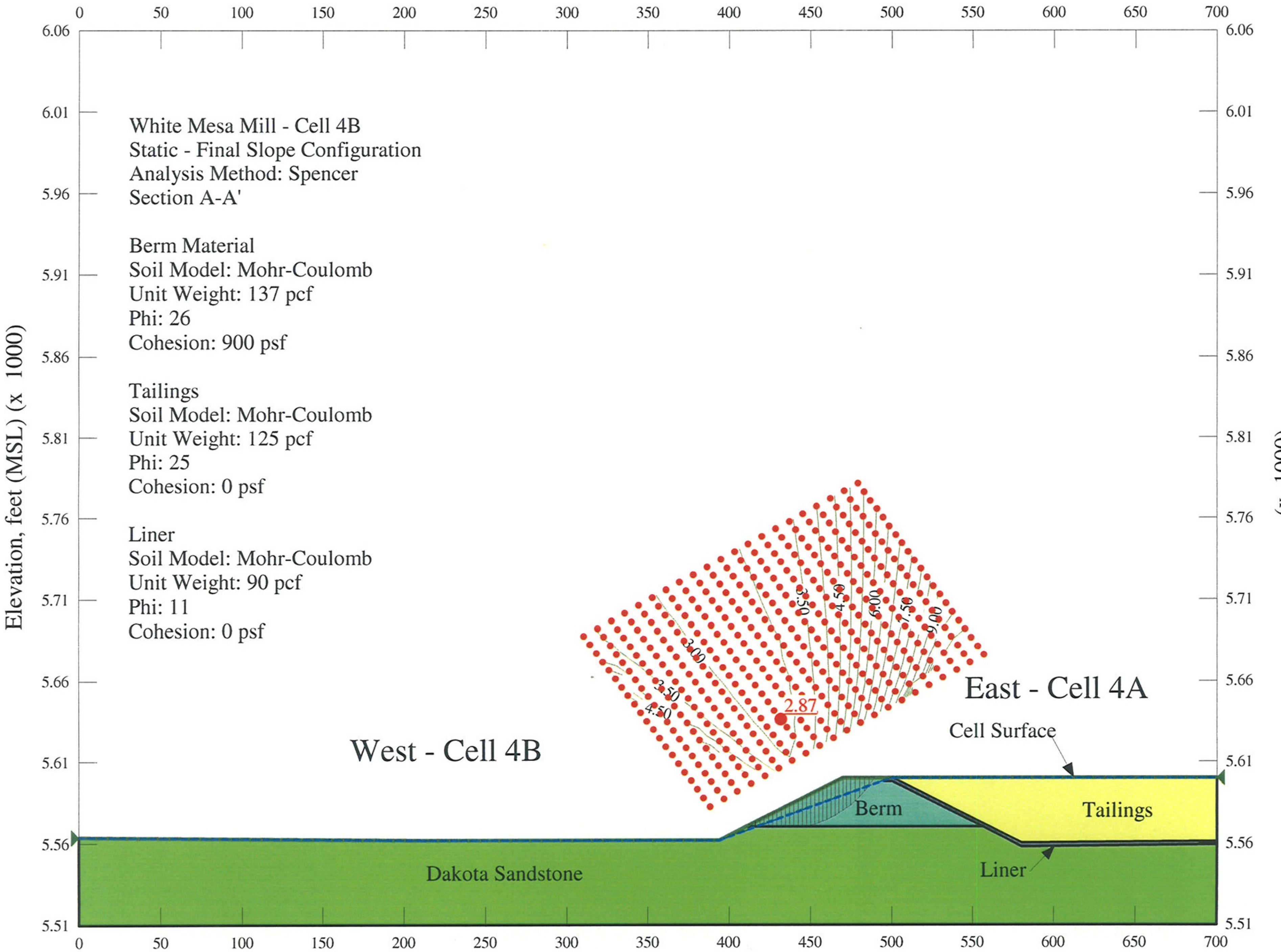
SIGNATURE

DATE

DESIGN BY:	GTC	DATE:	JANUARY 2009
DRAWN BY:	MAD	PROJECT NO.:	SC0349
CHECKED BY:	RF	FILE:	
REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC	3 OF 8	

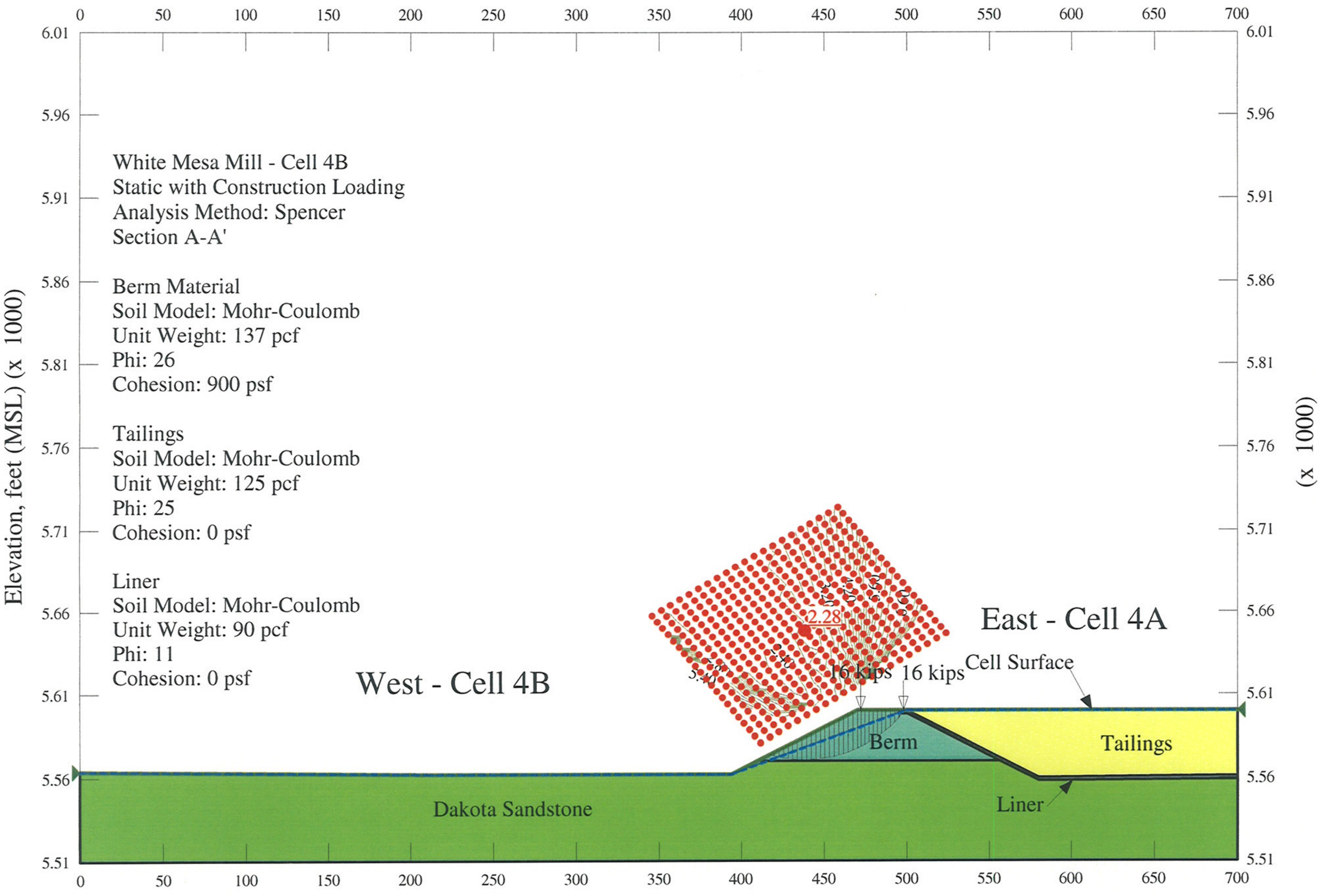
P:\PRJ\SDCadd\CADD\SC0349\SC0349_12-30-08\PlanSet_Cell 4b\0349C003.dwg

FIGURE 1



File Name: A-A'-1.gsz

FIGURE 2



File Name: A-A'-2.gsz

FIGURE 3

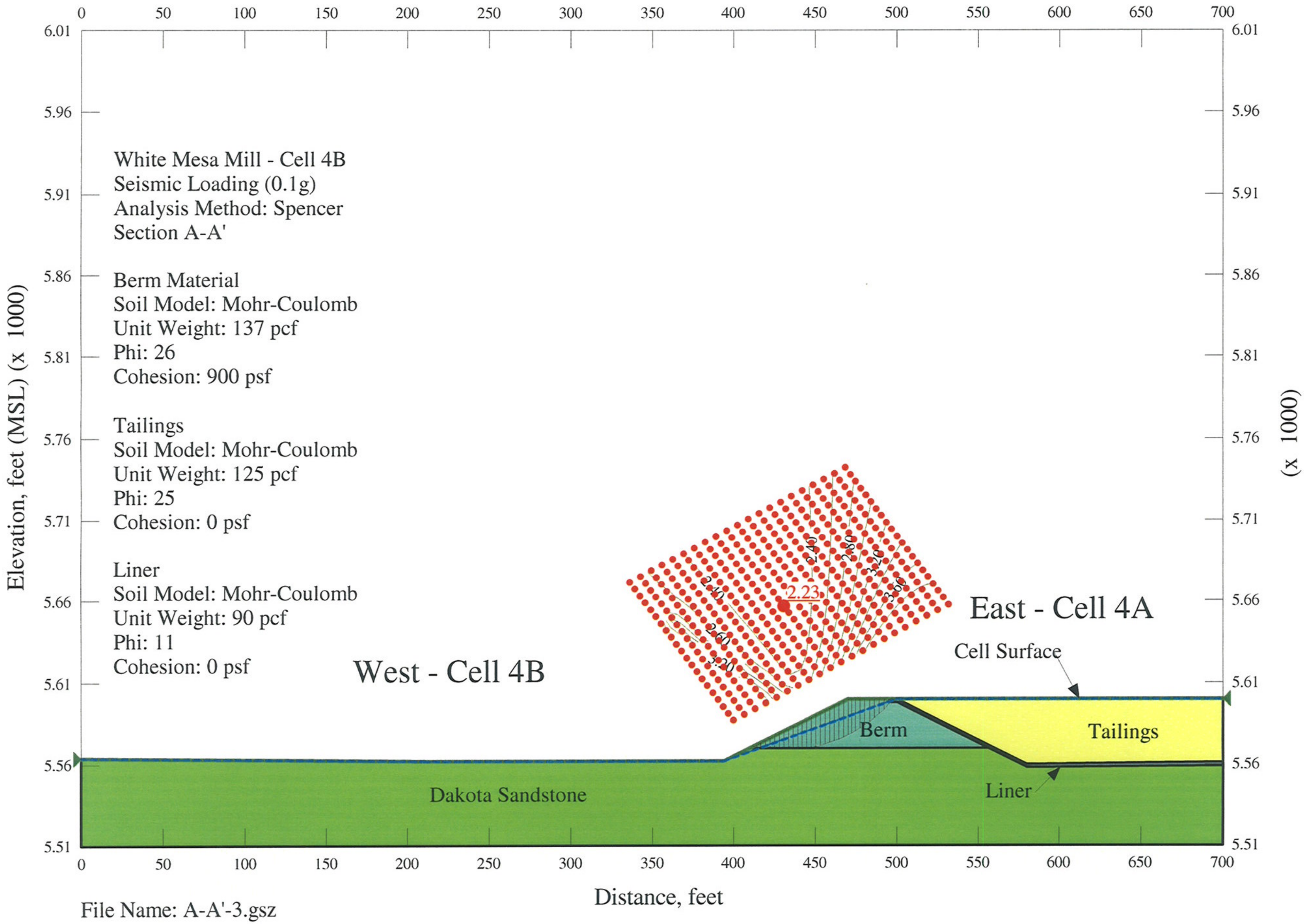
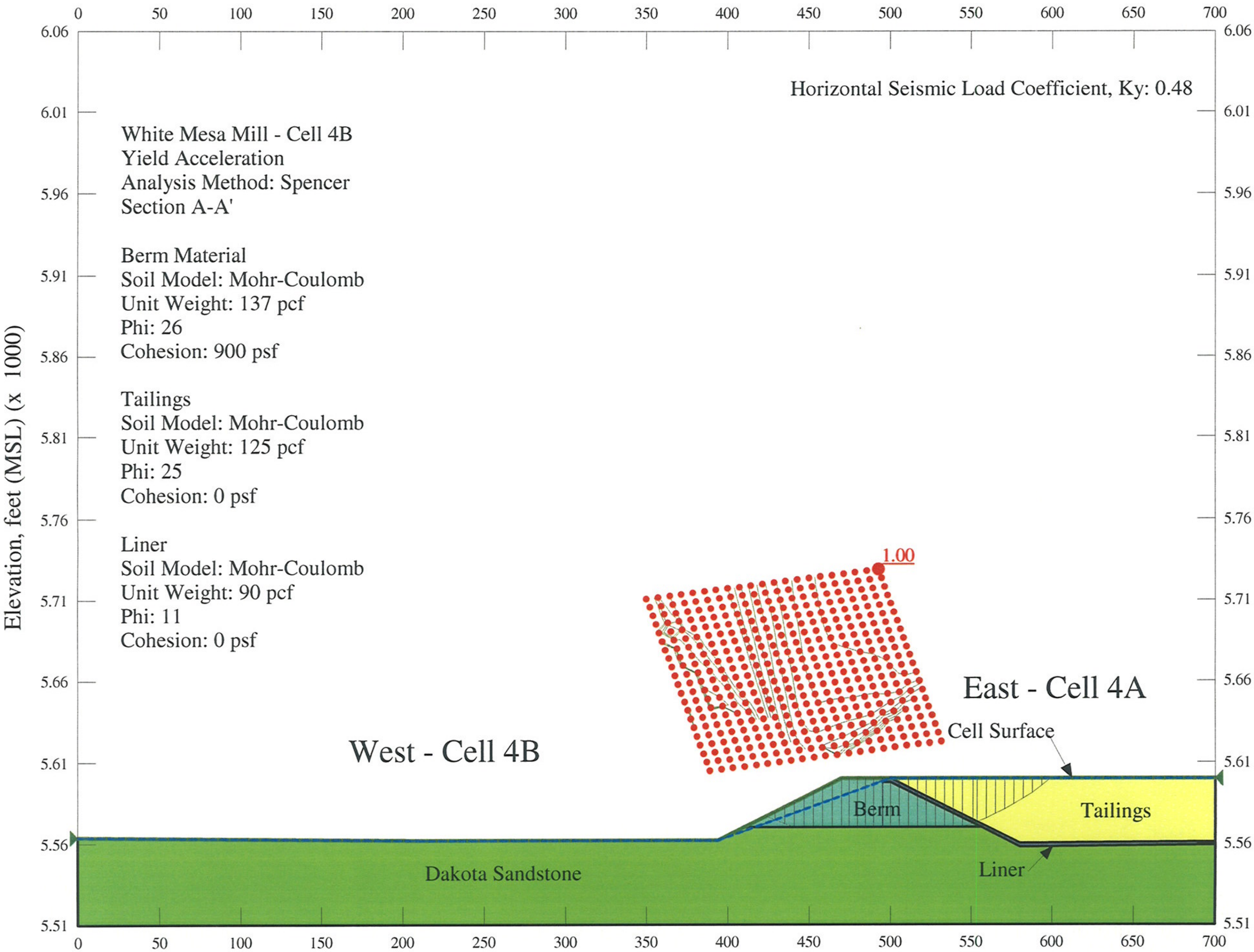

White Mesa Mill - Cell 4B
Seismic Loading (0.1g)
Analysis Method: Spencer
Section A-A'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 137 pcf
Phi: 26
Cohesion: 900 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
West - Cell 4B
East - Cell 4A
Cell Surface
2.23
Berm
Tailings
Liner
Dakota Sandstone
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
File Name: A-A'-3.gsz

FIGURE 4



File Name: A-A'-4.gsz

FIGURE 5

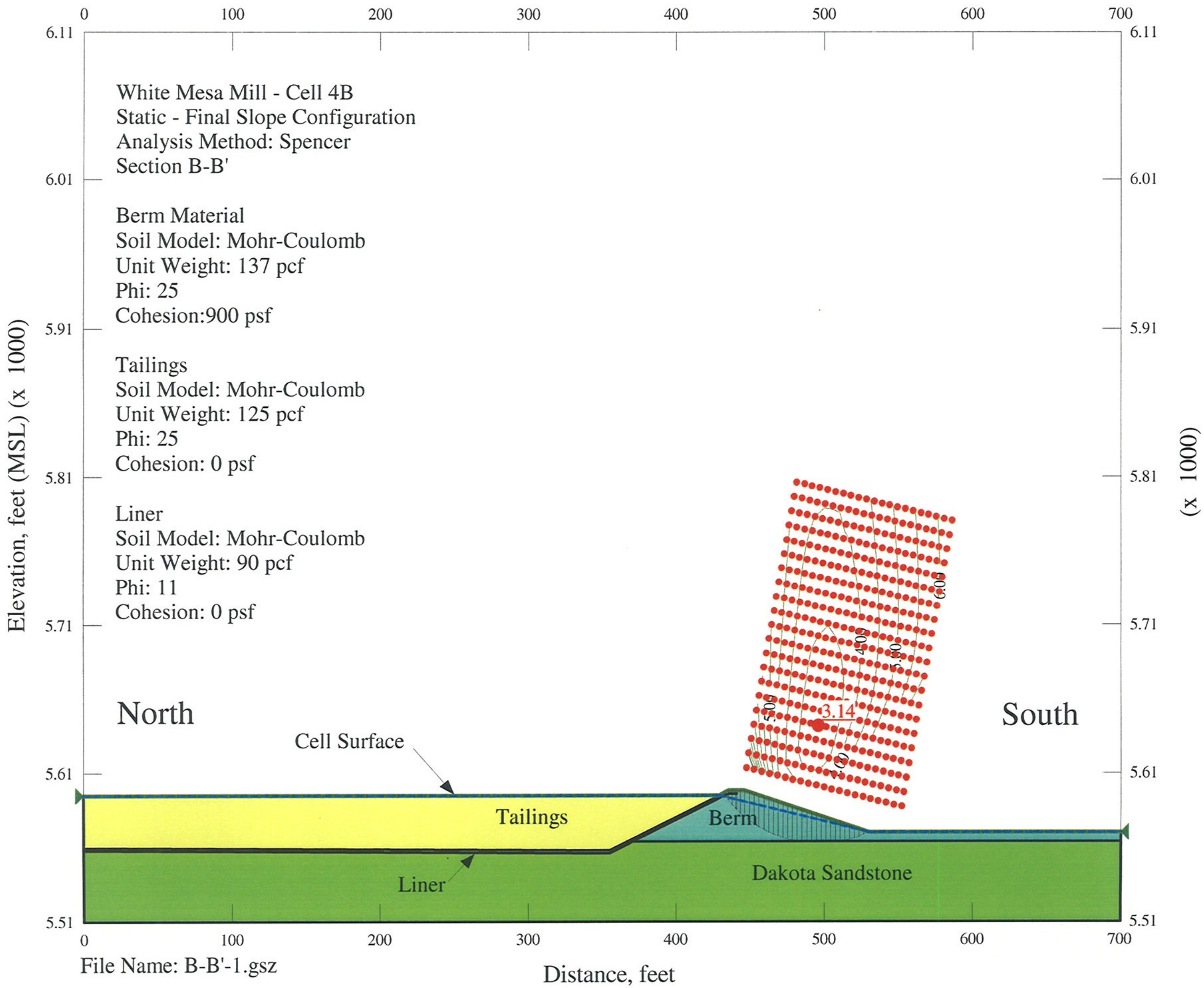

White Mesa Mill - Cell 4B
Static - Final Slope Configuration
Analysis Method: Spencer
Section B-B'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 137 pcf
Phi: 25
Cohesion:900 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
North
South
Cell Surface
Tailings
Berm
Liner
Dakota Sandstone
3.14
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
File Name: B-B'-1.gsz
0
100
200
300
400
500
600
700
6.11
6.01
5.91
5.81
5.71
5.61
5.51

FIGURE 6

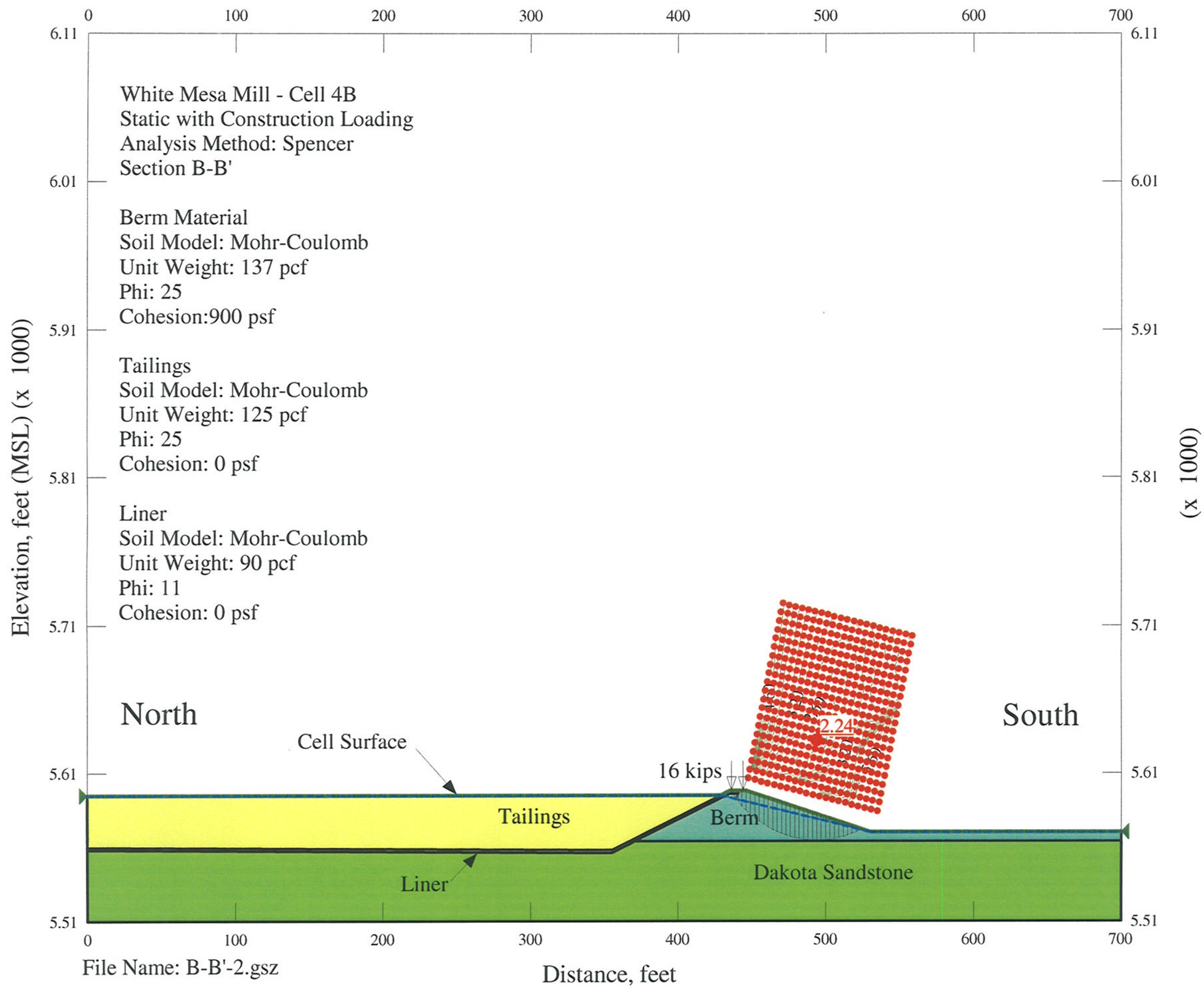

White Mesa Mill - Cell 4B
Static with Construction Loading
Analysis Method: Spencer
Section B-B'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 137 pcf
Phi: 25
Cohesion:900 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
North
South
Cell Surface
16 kips
2.24
Tailings
Berm
Liner
Dakota Sandstone
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
File Name: B-B'-2.gsz
0
100
200
300
400
500
600
700
6.11
6.01
5.91
5.81
5.71
5.61
5.51

FIGURE 7

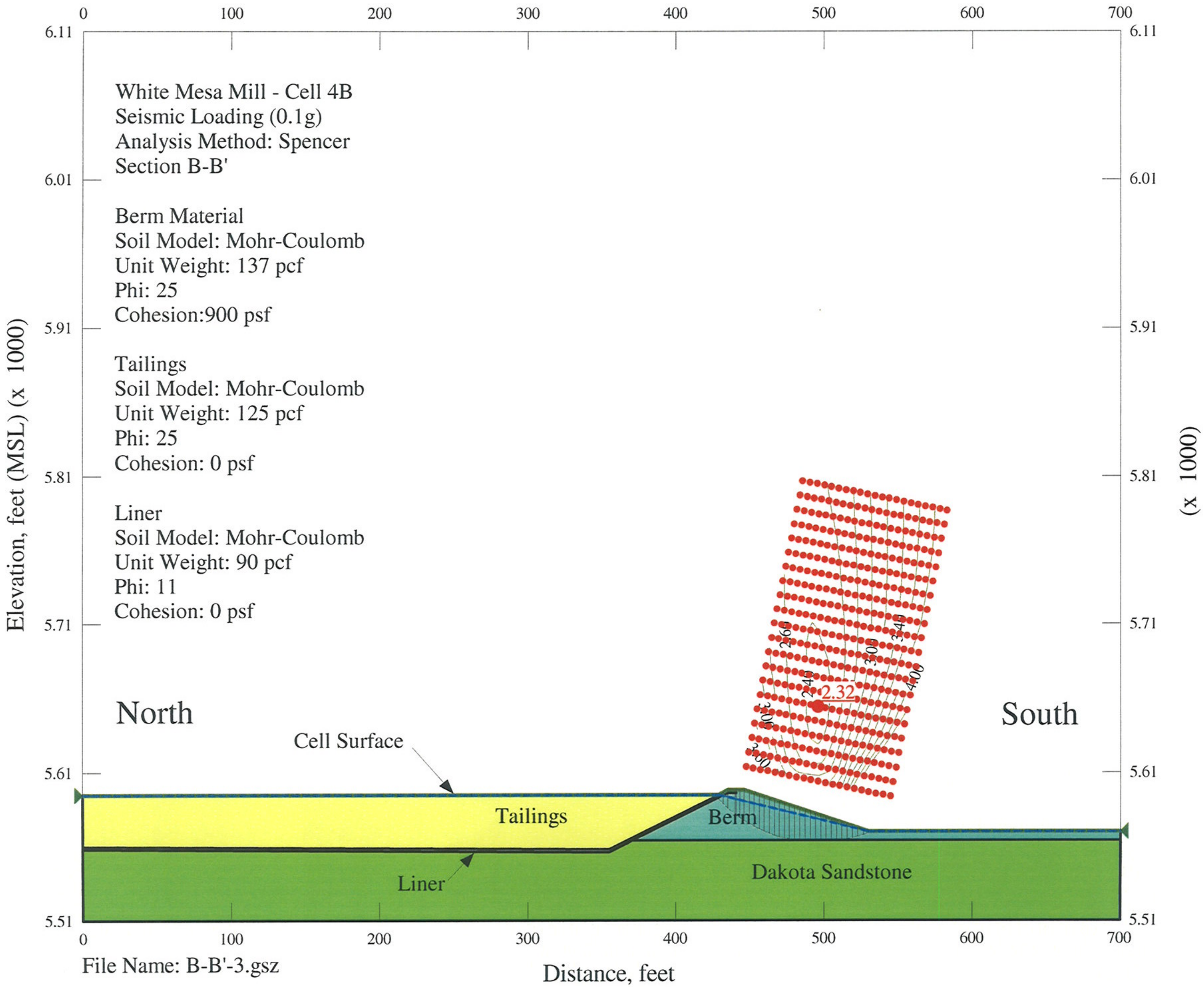

White Mesa Mill - Cell 4B
Seismic Loading (0.1g)
Analysis Method: Spencer
Section B-B'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 137 pcf
Phi: 25
Cohesion:900 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
North
South
Cell Surface
Tailings
Berm
Liner
Dakota Sandstone
2.32
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
File Name: B-B'-3.gsz

FIGURE 8

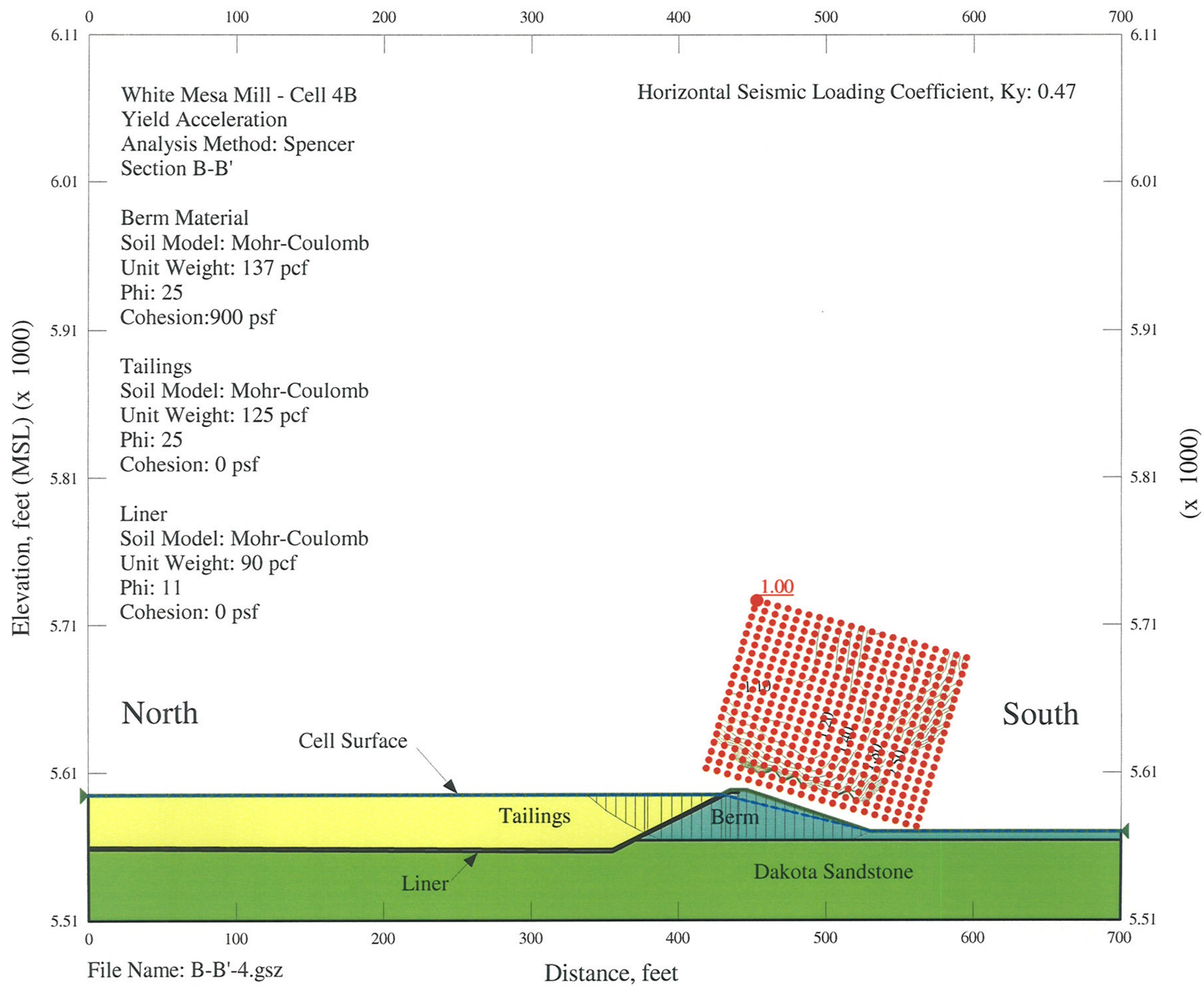
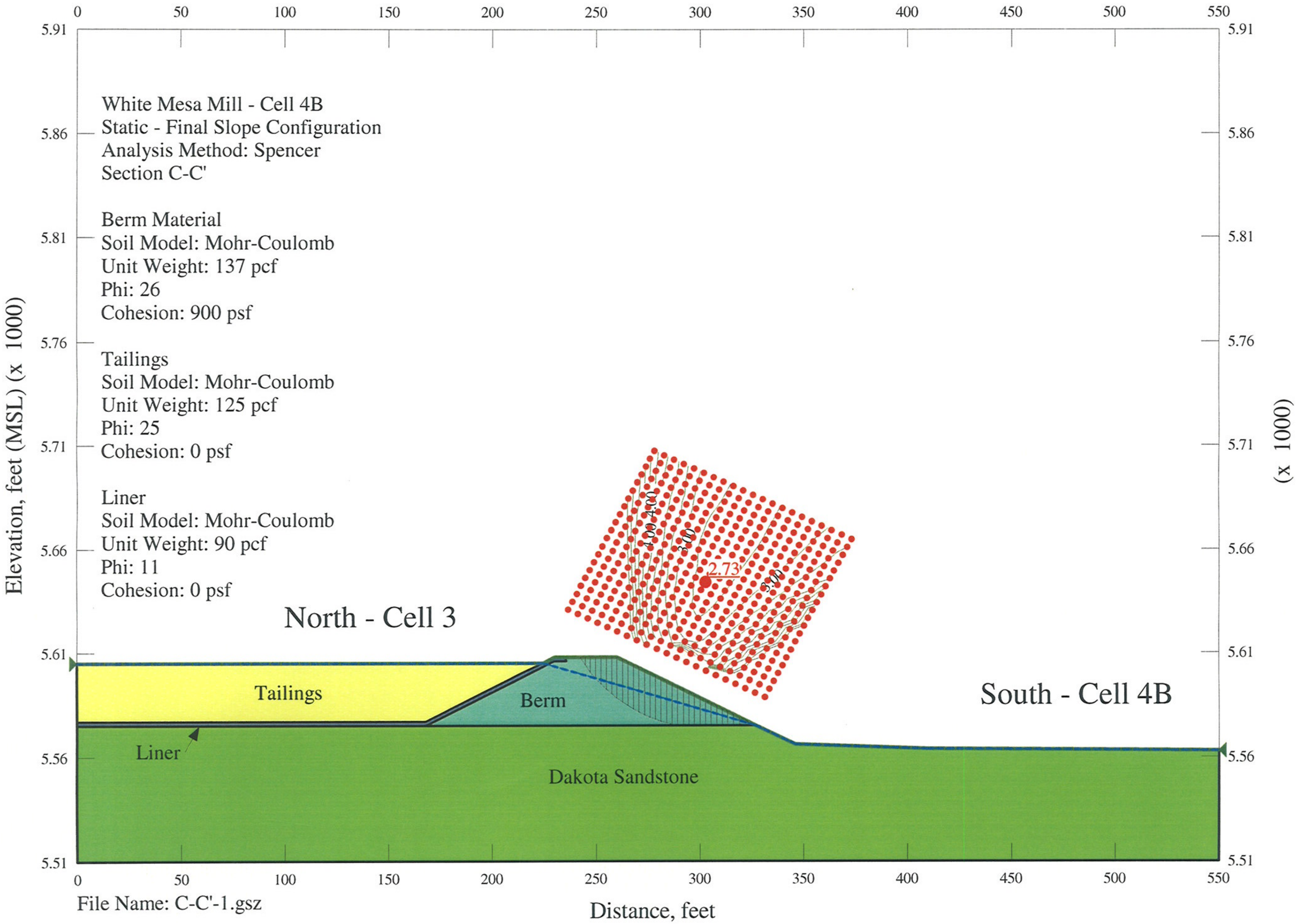
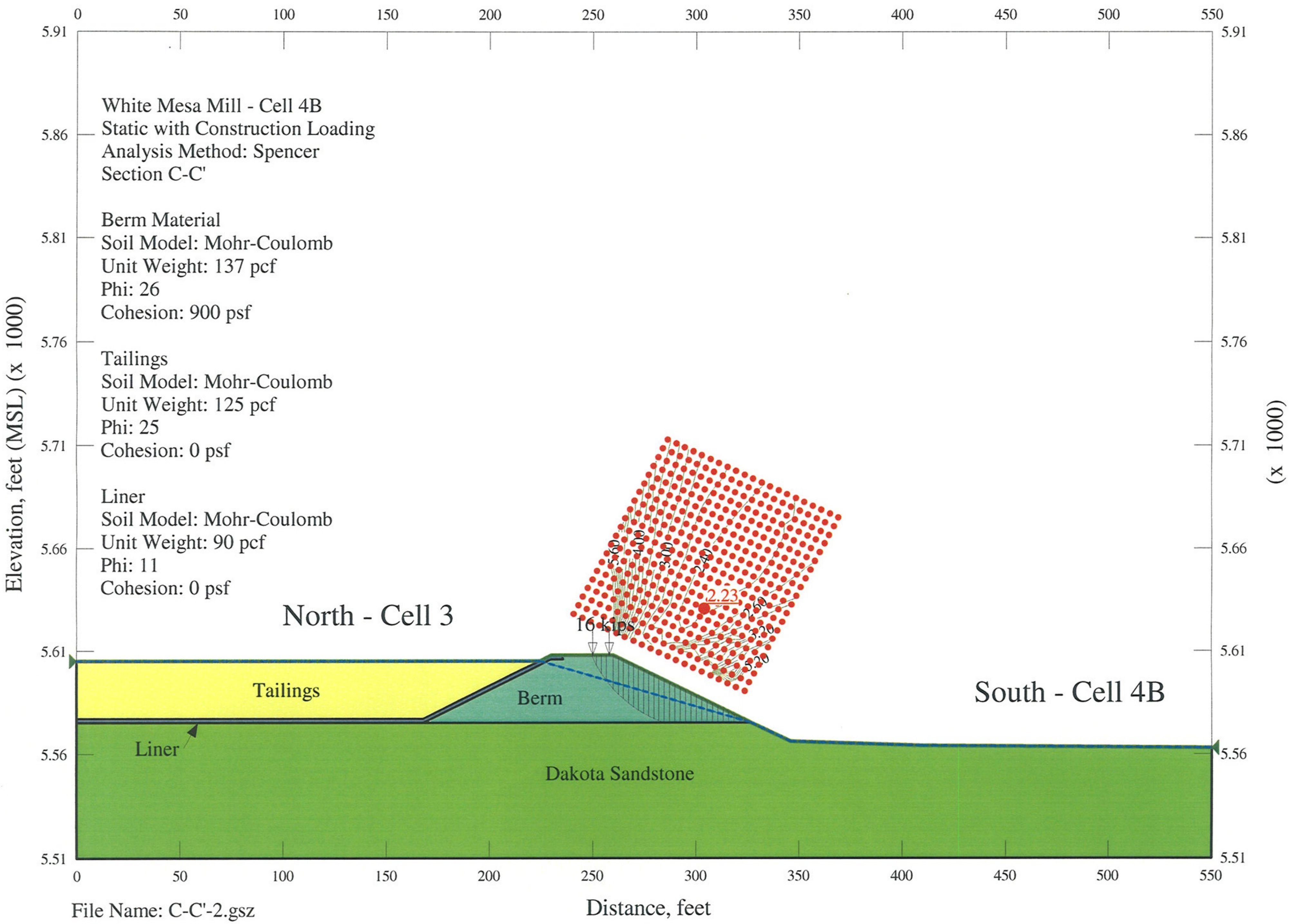

White Mesa Mill - Cell 4B
Yield Acceleration
Analysis Method: Spencer
Section B-B'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 137 pcf
Phi: 25
Cohesion:900 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
Horizontal Seismic Loading Coefficient, Ky: 0.47
1.00
North
South
Cell Surface
Tailings
Berm
Liner
Dakota Sandstone
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
File Name: B-B'-4.gsz

FIGURE 9



File Name: C-C'-1.gsz

FIGURE 10


White Mesa Mill - Cell 4B
Static with Construction Loading
Analysis Method: Spencer
Section C-C'

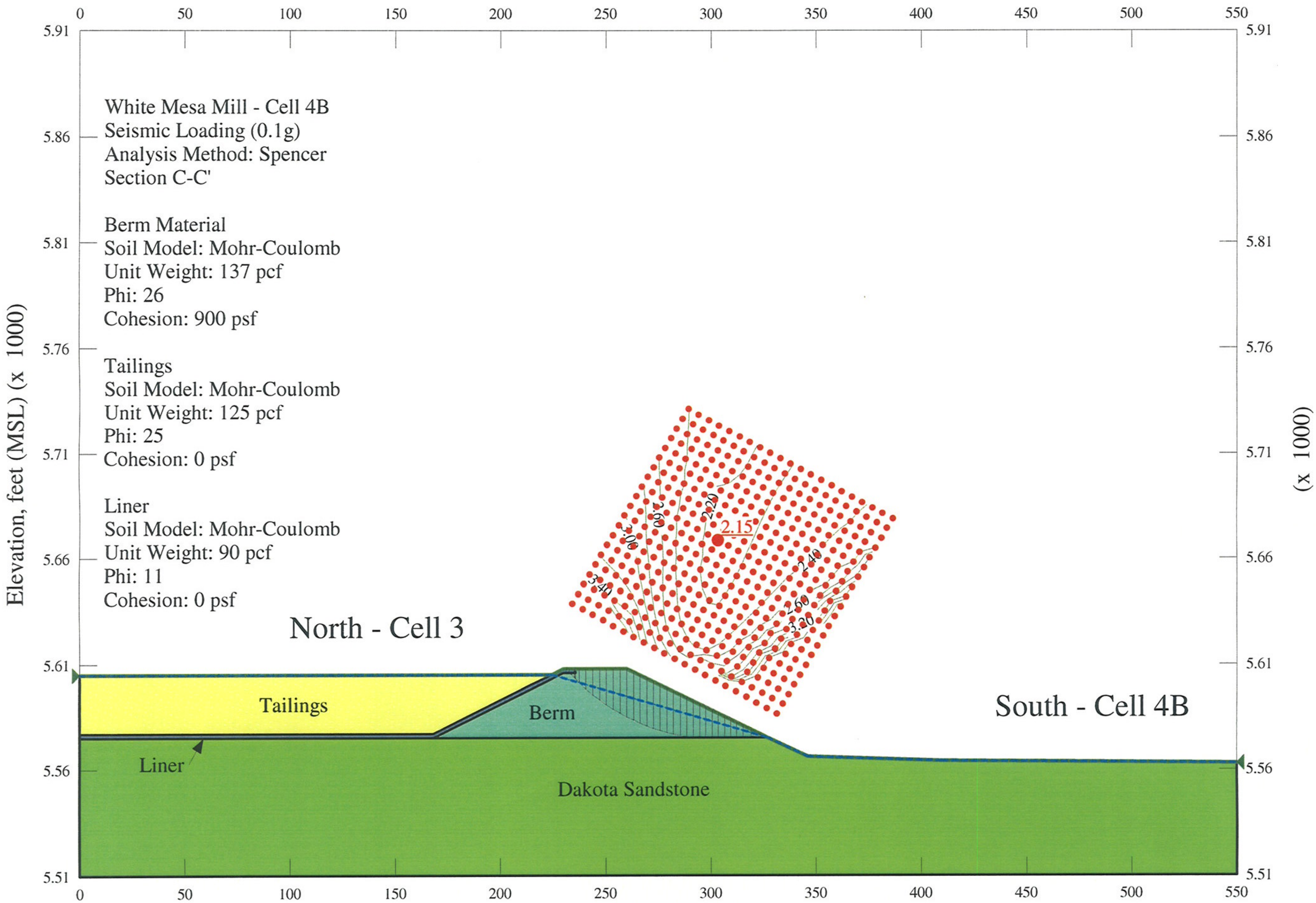
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 137 pcf
Phi: 26
Cohesion: 900 psf

Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf

Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf

North - Cell 3
South - Cell 4B
16 kips
2.23
Tailings
Berm
Liner
Dakota Sandstone
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
File Name: C-C'-2.gsz

FIGURE 11



File Name: C-C'-3.gsz

FIGURE 12

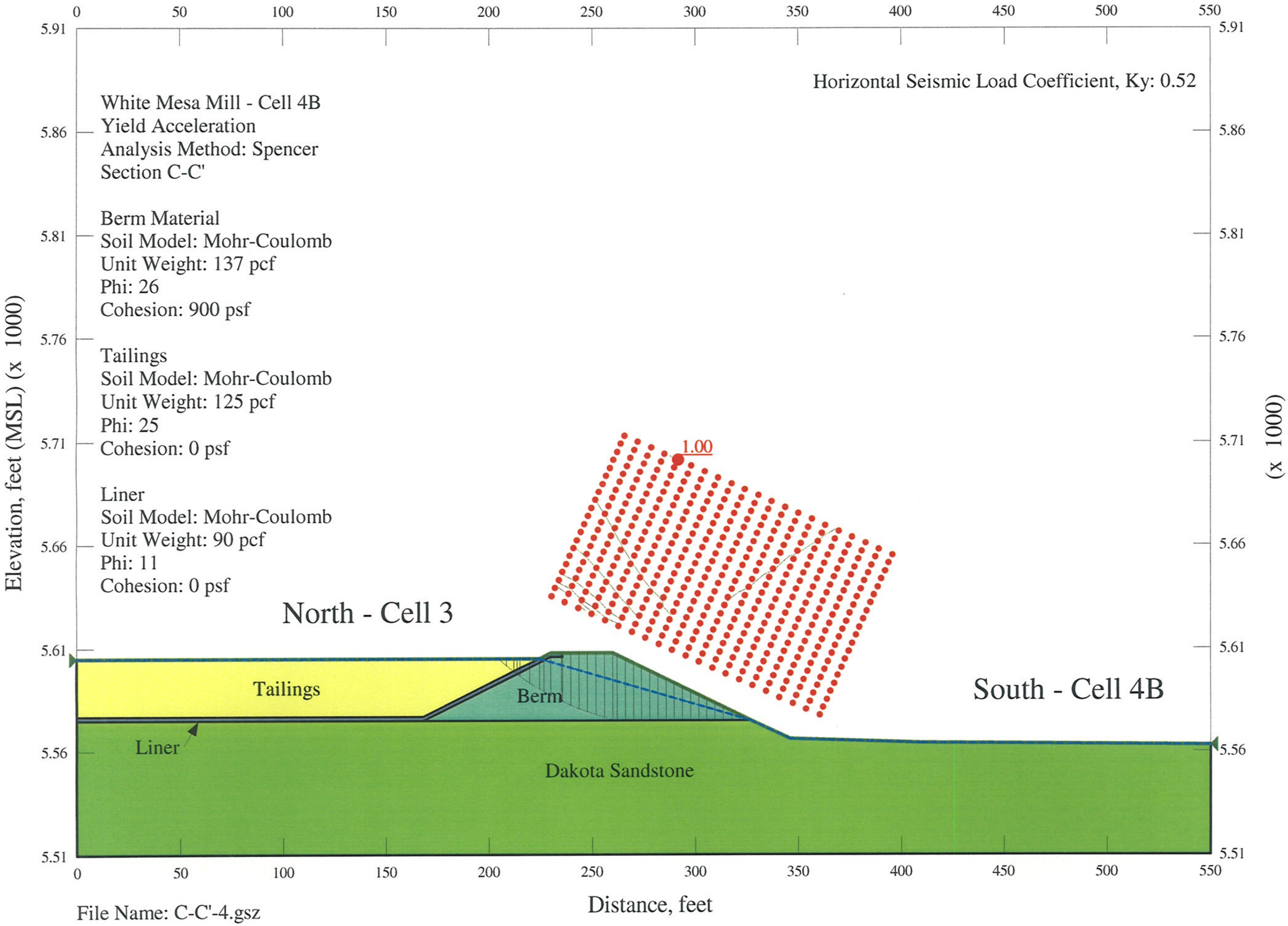
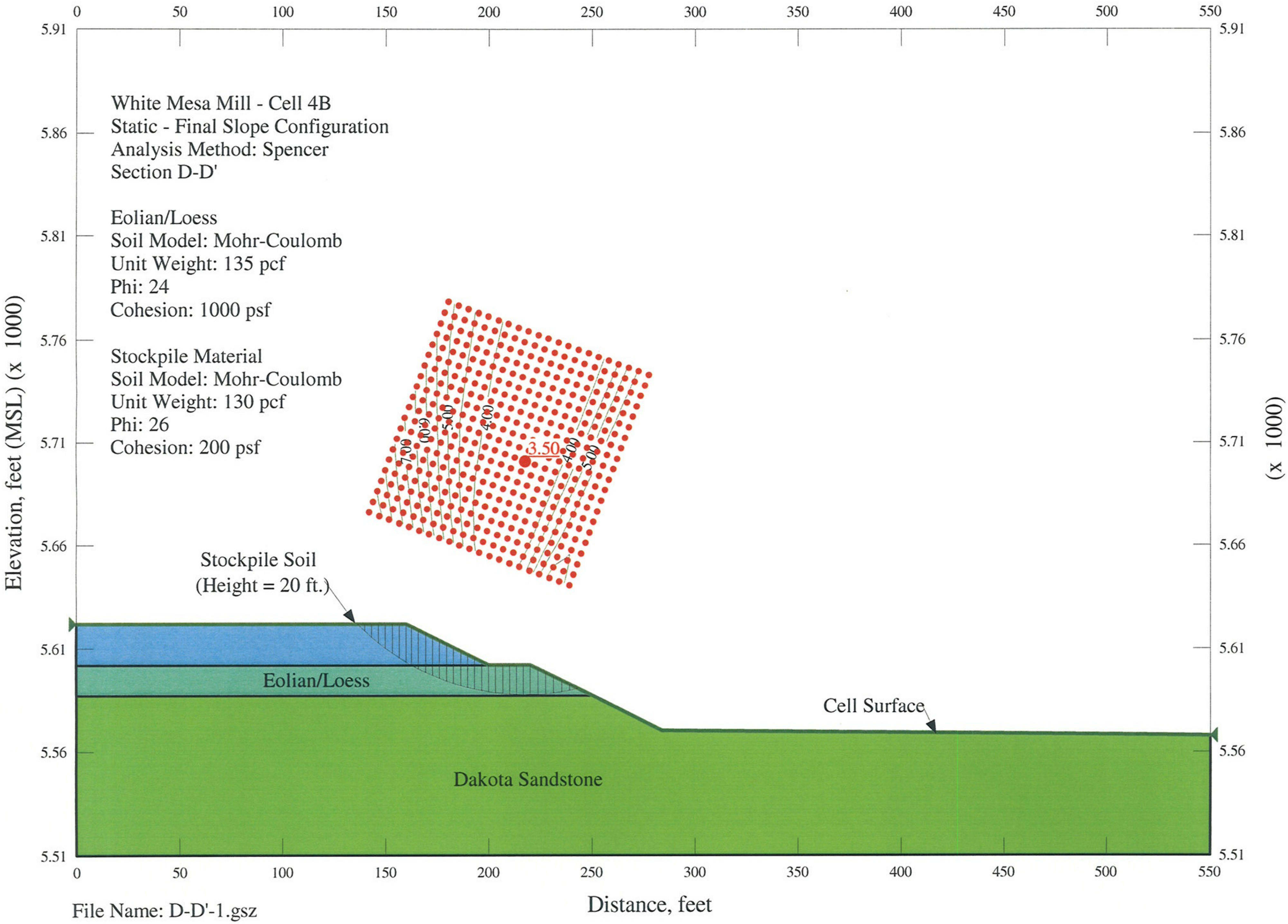
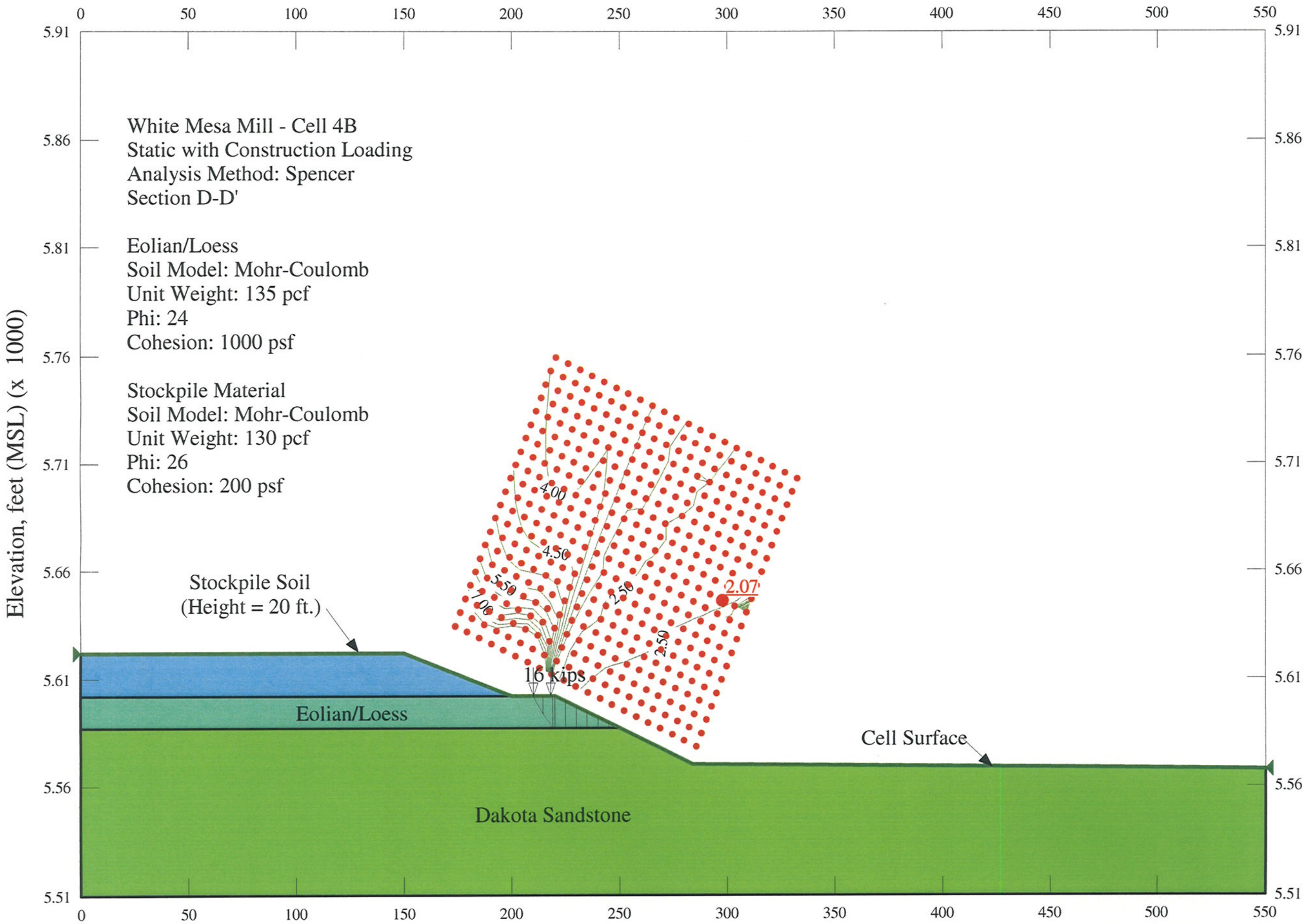

Horizontal Seismic Load Coefficient, Ky: 0.52
White Mesa Mill - Cell 4B
Yield Acceleration
Analysis Method: Spencer
Section C-C'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 137 pcf
Phi: 26
Cohesion: 900 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
North - Cell 3
South - Cell 4B
1.00
Tailings
Berm
Liner
Dakota Sandstone
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
0
50
100
150
200
250
300
350
400
450
500
550
5.91
5.86
5.81
5.76
5.71
5.66
5.61
5.56
5.51
File Name: C-C'-4.gsz

FIGURE 13



File Name: D-D'-1.gsz

FIGURE 14



File Name: D-D'-2.gsz

FIGURE 15

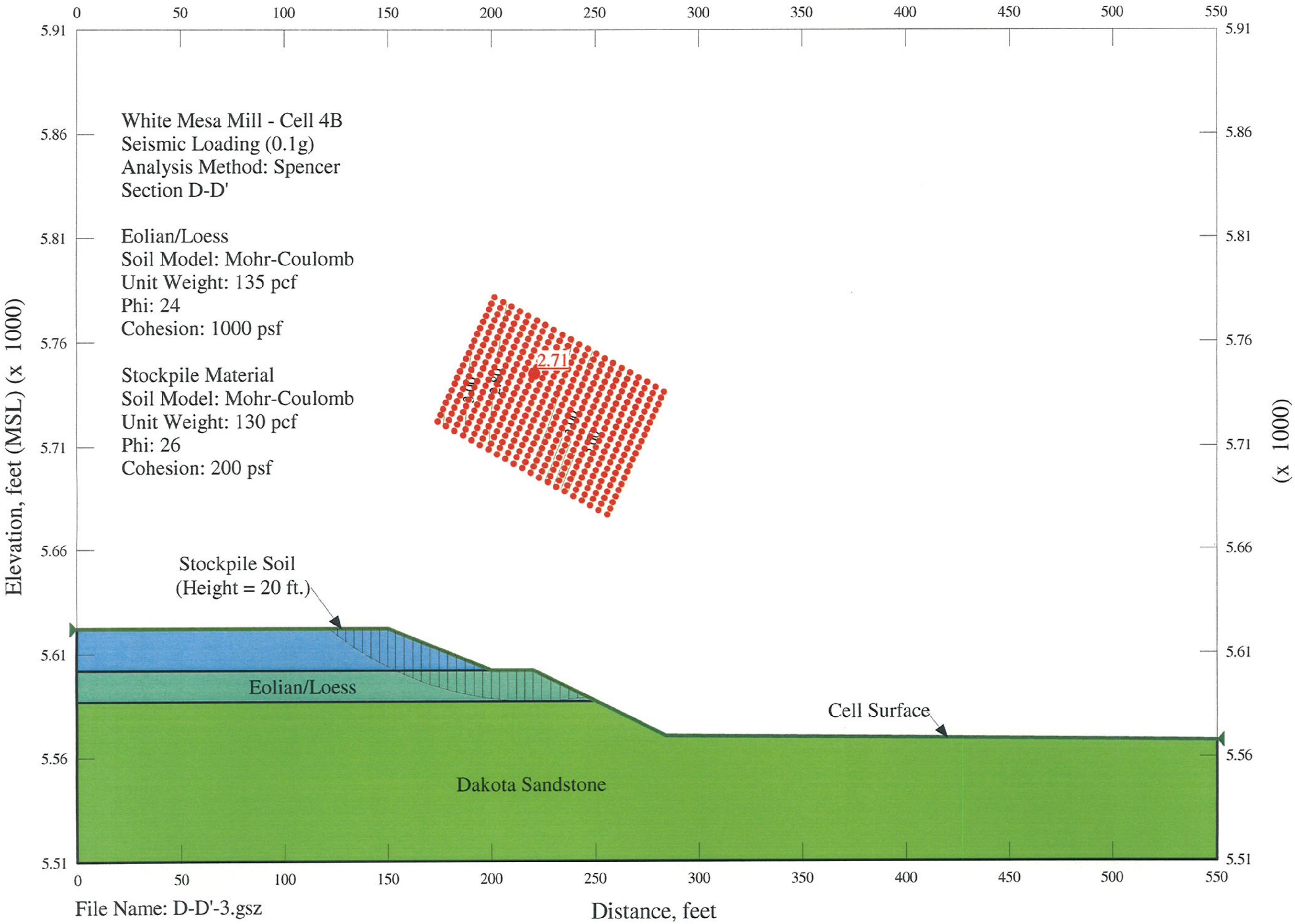

White Mesa Mill - Cell 4B
Seismic Loading (0.1g)
Analysis Method: Spencer
Section D-D'
Eolian/Loess
Soil Model: Mohr-Coulomb
Unit Weight: 135 pcf
Phi: 24
Cohesion: 1000 psf
Stockpile Material
Soil Model: Mohr-Coulomb
Unit Weight: 130 pcf
Phi: 26
Cohesion: 200 psf
2.71
Stockpile Soil
(Height = 20 ft.)
Eolian/Loess
Cell Surface
Dakota Sandstone
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
File Name: D-D'-3.gsz

FIGURE 16

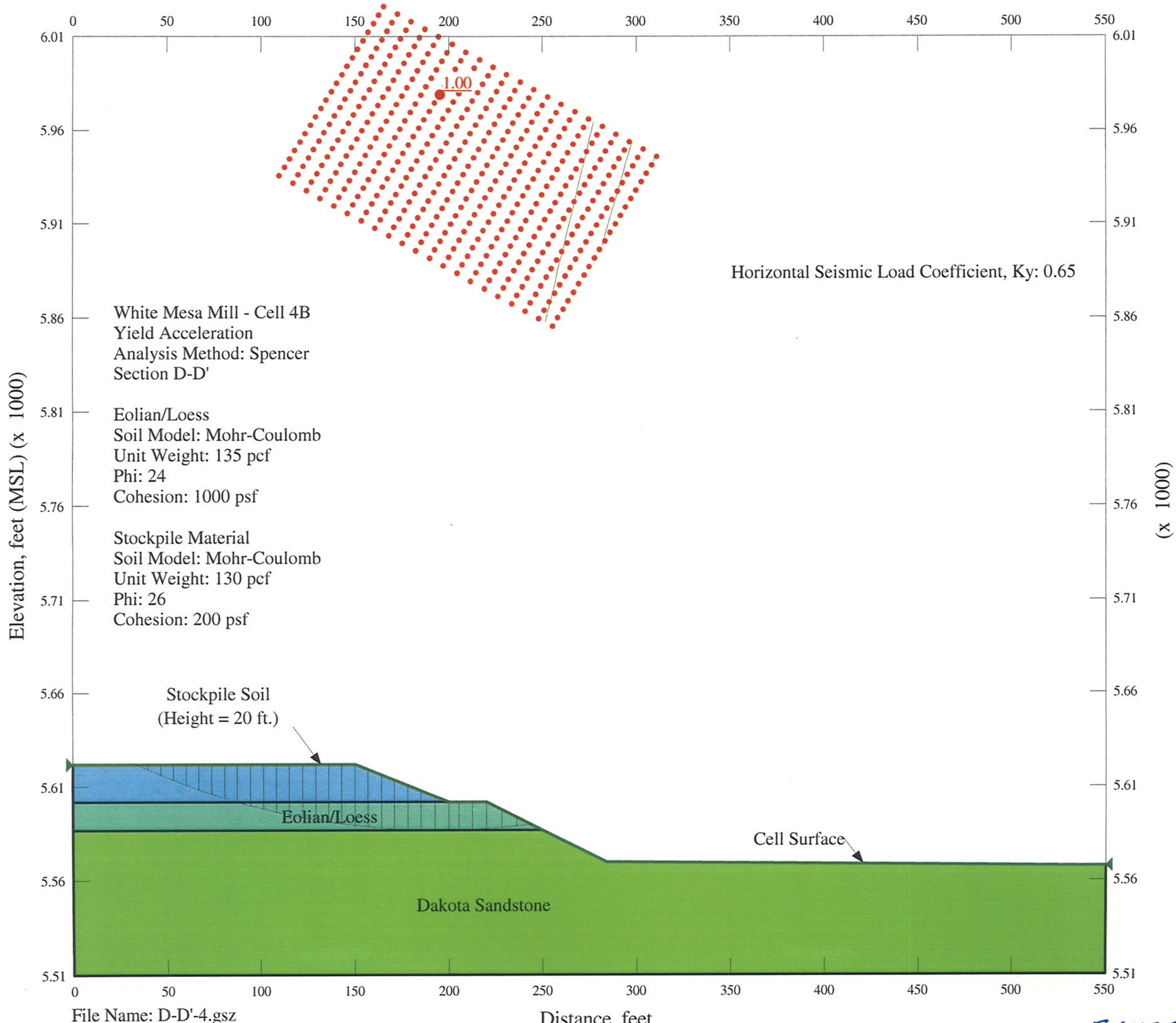

White Mesa Mill - Cell 4B
Yield Acceleration
Analysis Method: Spencer
Section D-D'
Eolian/Loess
Soil Model: Mohr-Coulomb
Unit Weight: 135 pcf
Phi: 24
Cohesion: 1000 psf
Stockpile Material
Soil Model: Mohr-Coulomb
Unit Weight: 130 pcf
Phi: 26
Cohesion: 200 psf
Horizontal Seismic Load Coefficient, Ky: 0.65
1.00
Stockpile Soil
(Height = 20 ft.)
Eolian/Loess
Cell Surface
Dakota Sandstone
Elevation, feet (MSL) (x 1000)
(x 1000)
6.01
5.96
5.91
5.86
5.81
5.76
5.71
5.66
5.61
5.56
5.51
0
50
100
150
200
250
300
350
400
450
500
550
Distance, feet
File Name: D-D'-4.gsz

FIGURE 17

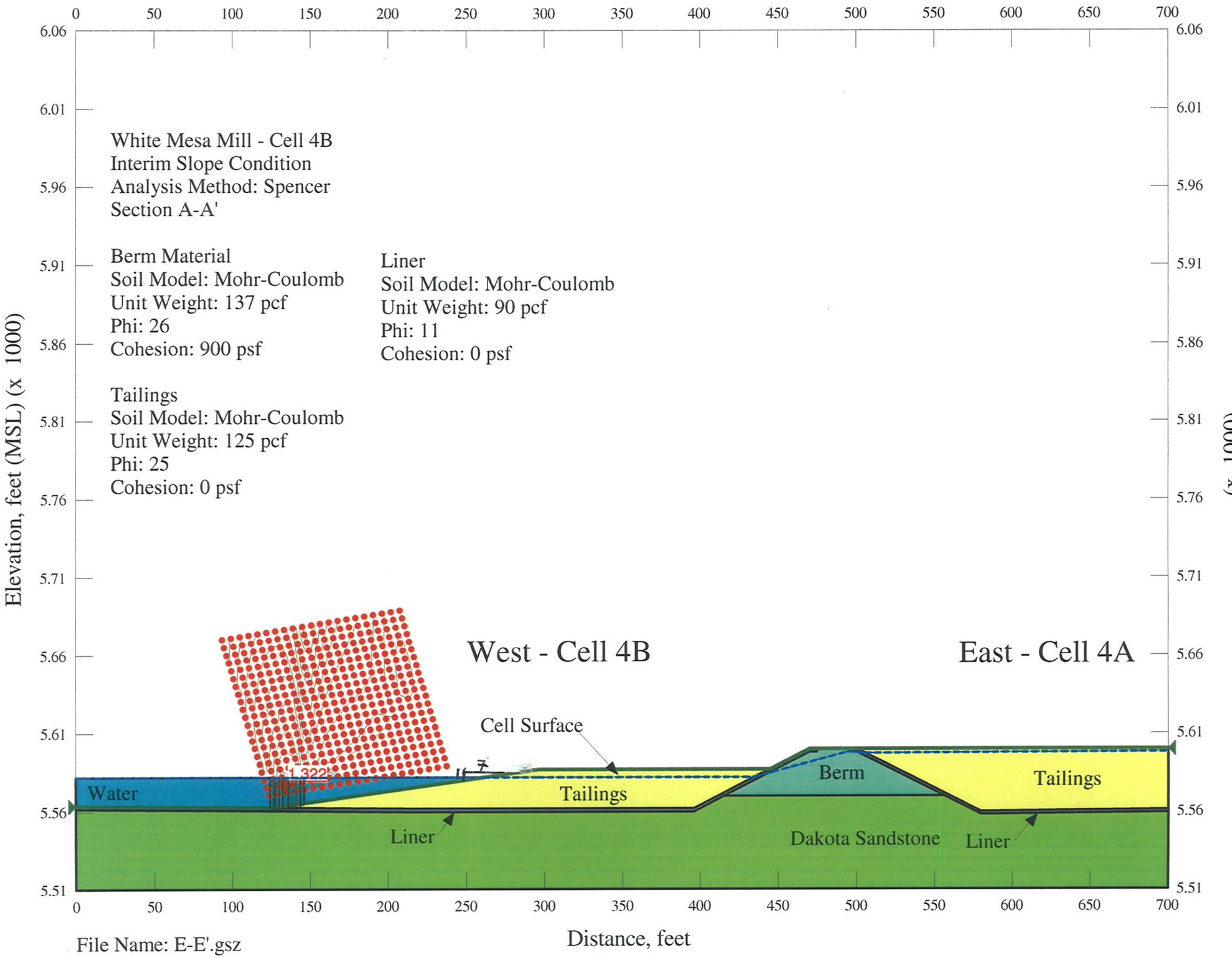
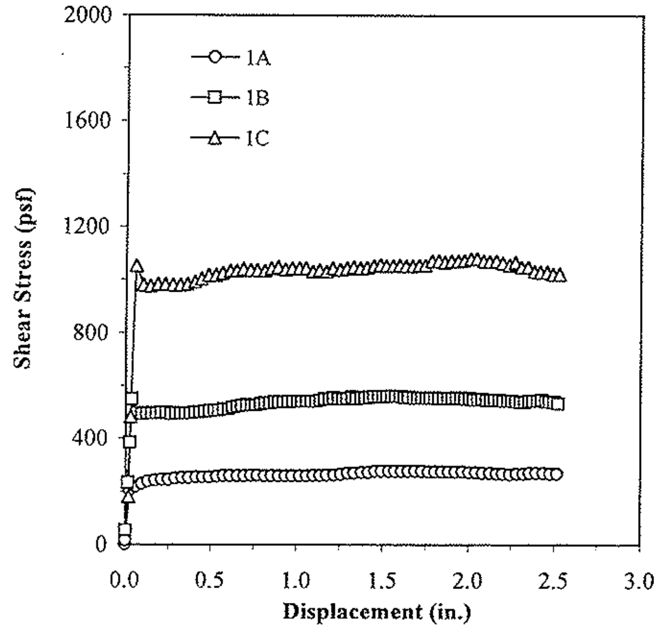
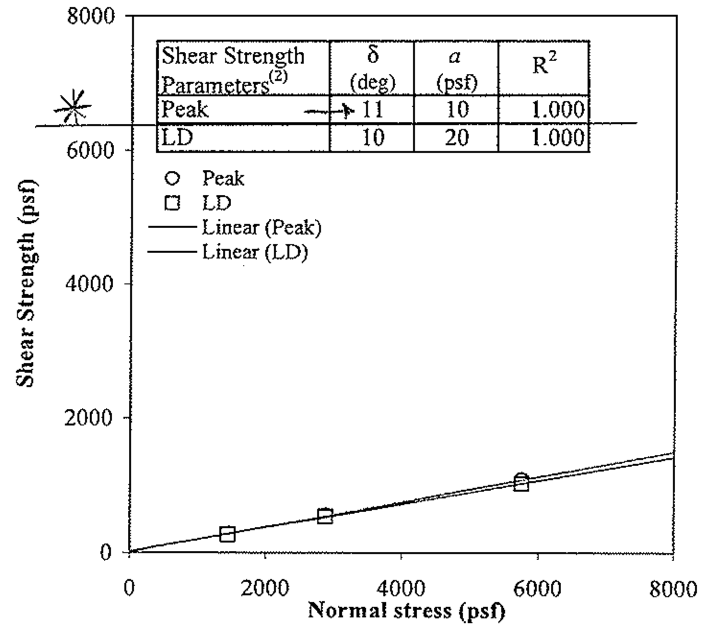

White Mesa Mill - Cell 4B
Interim Slope Condition
Analysis Method: Spencer
Section A-A'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 137 pcf
Phi: 26
Cohesion: 900 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
West - Cell 4B
East - Cell 4A
Cell Surface
Water
Tailings
Berm
Tailings
Liner
Dakota Sandstone
Liner
1.322
Elevation, feet (MSL) (x 1000)
(x 1000)
Distance, feet
File Name: E-E'.gsz

FIGURE 18

GEOSYNTEC CONSULTANTS - INTERNATIONAL URANIUM CORP PROJECT
DIRECT SHEAR TESTING (ASTM D 5321)

Test Series Number 1: SKAPS TN 330 geonet in machine direction against white side of GSE 60-mil black/white smooth HDPE geomembrane (Roll # 105130507) in machine direction under wetted conditions





Shear Strength Parameters[(2)]	δ (deg)	a (psf)	R^2
Peak	11	10	1.000
LD	10	20	1.000

Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	Soaking Stress (psf)	Soaking Time (hour)	Consolidation Stress (psf)	Consolidation Time (hour)	Lower Soil γ_d (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_d (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_P (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
1A	12 x 12	1440	0.04	-	-	-	-	-	-	-	-	-	-	-	-	276	266	(1)
1B	12 x 12	2880	0.04	-	-	-	-	-	-	-	-	-	-	-	-	549	533	(1)
1C	12 x 12	5760	0.04	-	-	-	-	-	-	-	-	-	-	-	-	1079	1022	(1)

NOTES:

(1) Sliding (i.e., shear failure) occurred at intended test interface in each test.

(2) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

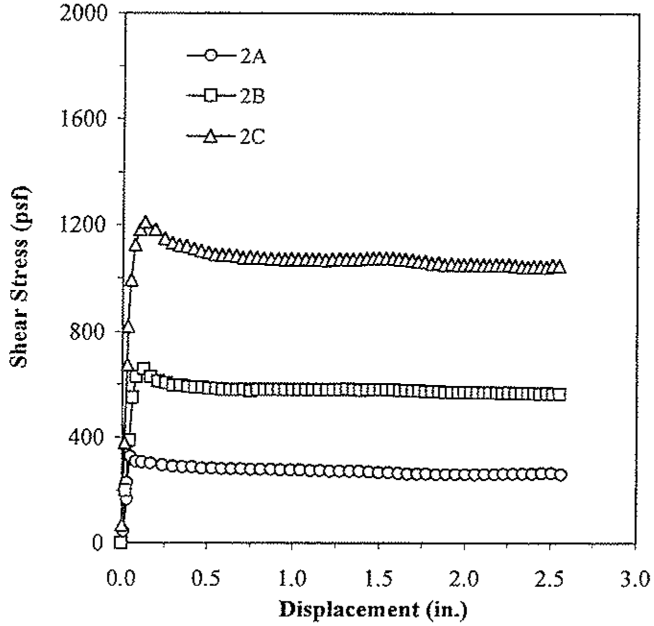
SGI TESTING SERVICES, LLC

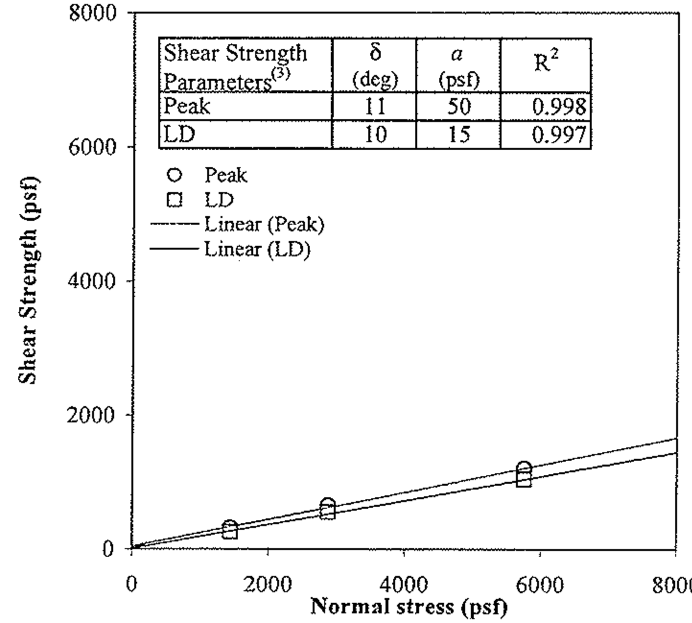
DATE OF TEST:	11/6/2006
FIGURE NO.	B-1
PROJECT NO.	SGI6014-03
DOCUMENT NO.	
FILE NO.	

ATTACHMENT A (1/2)

GEOSYNTEC CONSULTANTS - INTERNATIONAL URANIUM CORP PROJECT
DIRECT SHEAR TESTING (ASTM D 5321)

Test Series Number 2: Woven side of Bentomat ST GCL (Lot # 200640LO/Roll #6397) in machine direction against black side of GSE 60-mil black/white smooth HDPE geomembrane (Roll # 105130507) in machine direction under soaked and consolidated conditions





Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	GCL Soaking Stress (psf)	GCL Soaking Time (hour)	Consolidation[1] Stress (psf)	Consolidation[1] Time (hour)	Lower Soil γ_d (pcf)	Lower Soil ω_i (%)	Lower Soil ω_f (%)	Upper Soil γ_d (pcf)	Upper Soil ω_i (%)	Upper Soil ω_f (%)	GCL ω_i (%)	GCL ω_f (%)	Shear Stress τ_P (psf)	Shear Stress τ_{LD} (psf)	Failure Mode
2A	12 x 12	1440	0.04	1440	24	1440	24	-	-	-	-	-	-	17.9	68.8	325	259	(2)
2B	12 x 12	2880	0.04	2880	24	2880	24	-	-	-	-	-	-	17.9	55.8	657	562	(2)
2C	12 x 12	5760	0.04	5760	24	5760	24	-	-	-	-	-	-	17.9	48.7	1211	1046	(2)

NOTES:

(1) The hydrated GCL specimen was placed on the geomembrane and consolidated together under each test normal stress for 24 hours prior to shearing. The test specimens were not submerged in water during consolidation.

(2) Sliding (i.e., shear failure) occurred at intended test interface in each test.

(3) The reported total-stress parameters of friction angle and adhesion were determined from a best-fit line drawn through the test data. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series. The large-displacement (LD) shear strength was calculated using the shear force measured at the end of the test.

SGI TESTING SERVICES, LLC

DATE OF TEST:	11/3 to 11/7/2006
FIGURE NO.	B-2
PROJECT NO.	SGI6014-03
DOCUMENT NO.	
FILE NO.	

ATTACHMENT A
(2)2

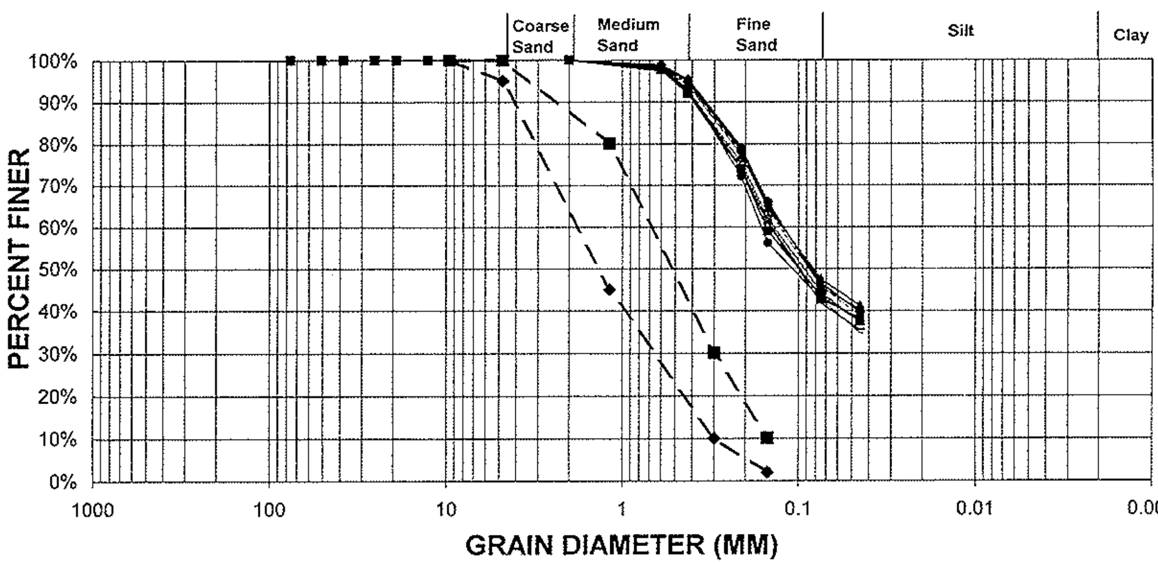
Ta.

DSM Screen Undersize Gradation

SIEVE ANALYSIS

Sieve No.	Diameter (mm)	Grinding Test 1 Wt. Retained (grams)	Grinding Test 1 % Retained	Grinding Test 1 % Finer	Grinding Test 2A Wt. Retained (grams)	Grinding Test 2A % Retained	Grinding Test 2A % Finer	Grinding Test 2B Wt. Retained (grams)	Grinding Test 2B % Retained	Grinding Test 2B % Finer	Grinding Test 3A Wt. Retained (grams)	Grinding Test 3A % Retained	Grinding Test 3A % Finer	Grinding Test 3B Wt. Retained (grams)	Grinding Test 3B % Retained	Grinding Test 3B % Finer
3 in.	76.2	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
2 in.	50.8	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
1 1/2 in.	38.1	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
1 in.	25.4	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
3/4 in.	19.1	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
1/2 in.	12.7	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
3/8 in.	9.530	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
No. 4	4.750	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
No. 10	2.000	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
No. 30	0.600	1.2	1.2%	98.8%	2.0	2.0%	98.0%	1.7	1.7%	98.3%	2.4	2.4%	97.6%	1.9	1.9%	98.1%
No. 40	0.425	4.6	4.6%	95.4%	7.3	7.3%	92.7%	6.0	6.0%	94.0%	8.1	8.1%	91.9%	6.9	6.9%	93.1%
No. 70	0.212	20.8	20.8%	79.2%	24.5	24.5%	75.5%	22.6	22.6%	77.4%	26.2	26.2%	73.8%	27.9	27.9%	72.1%
No. 100	0.150	34.8	34.8%	65.2%	38.1	38.1%	61.9%	35.5	35.5%	64.5%	41.0	41.0%	59.0%	43.9	43.9%	56.1%
No. 200	0.075	53.4	53.4%	46.6%	55.7	55.7%	44.3%	52.5	52.5%	47.5%	56.6	56.6%	43.4%	57.4	57.4%	42.6%
No. 325	0.045	60.5	60.5%	39.5%	62.7	62.7%	37.3%	58.8	58.8%	41.2%	62.5	62.5%	37.5%	61.9	61.9%	38.1%
Pan	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Sieve No.	Diameter (mm)	Grinding Test 6A Wt. Retained (grams)	Grinding Test 6A % Retained	Grinding Test 6A % Finer	Grinding Test 6B Wt. Retained (grams)	Grinding Test 6B % Retained	Grinding Test 6B % Finer	Grinding Test 4A Wt. Retained (grams)	Grinding Test 4A % Retained	Grinding Test 4A % Finer	Grinding Test 4B Wt. Retained (grams)	Grinding Test 4B % Retained	Grinding Test 4B % Finer
3 in.	76.2	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
2 in.	50.8	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
1 1/2 in.	38.1	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
1 in.	25.4	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
3/4 in.	19.1	0.0	0.0%	100.0%	0.0	0.0%	100.0% 3/8 in.	0.0	0.0%	100.0%	0.0	0.0%	100.0%
1/2 in.	12.7	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
3/8 in.	9.530	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
No. 4	4.750	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
No. 10	2.000	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%	0.0	0.0%	100.0%
No. 30	0.600	1.3	1.3%	98.7%	1.0	1.0%	99.0%	2.7	2.7%	97.3%	2.7	2.7%	97.3%
No. 40	0.425	5.2	5.2%	94.8%	4.7	4.7%	95.3%	7.6	7.6%	92.4%	7.3	7.3%	92.7%
No. 70	0.212	21.7	21.7%	78.3%	21.4	21.4%	78.6%	26.2	26.2%	73.8%	25.9	25.9%	74.1%
No. 100	0.150	34.1	34.1%	65.9%	35.9	35.9%	64.1%	38.7	38.7%	61.3%	39.2	39.2%	60.8%
No. 200	0.075	54.4	54.4%	45.6%	54.4	54.4%	45.6%	57.3	57.3%	42.7%	58.3	58.3%	41.7%
No. 325	0.045	59.7	59.7%	40.3%	61.1	61.1%	38.9%	65.4	65.4%	34.6%	64.6	64.6%	35.4%
Pan	-	-	-	-	-	-	-	-	-	-	-	-	-



Average	
Med Sand	6.4%
Fine Sand	49.1%
Silt	44.4%

Colorado School of Mines
Research Institute
Grinding Reports
5 June 1978
"DSM Screen Undersize"

%FINER = 100 - Σ%RETAINED

7/20/2007

Attachment B, 1/1

(1) Safety factor no less than 1.5 for permanent or sustained loading conditions.

*

(2) For foundations of structures, a safety factor no less than 2.0 is desirable to limit critical movements at foundation edge. See DM-7.2, Chapter 4 for detailed requirements for safety factors in bearing capacity analysis.

(3) For temporary loading conditions or where stability reaches a minimum during construction, safety factors may be reduced to 1.3 or 1.25 if controls are maintained on load application.

*

(4) For transient loads, such as earthquake, safety factors as low as 1.2 or 1.15 may be tolerated.

6. EARTHQUAKE LOADING. Earthquake effects can be introduced into the analysis by assigning a disturbing force on the sliding mass equal to kW where W is the weight of the sliding mass and k is the seismic coefficient. For the analyses of stability shown in Figure 9a, k+s,W is assumed to act parallel to the slope and through the center of mass of the sliding mass. Thus, for a factor of safety of 1.0:

Wb + k+s,Wh = FR

The factor of safety under an earthquake loading then becomes

```
          FR
F+Se,= ))))))))
       Wb + k+s,Wh
```

To determine the critical value of the seismic efficient (k+cs,) which will reduce a given factor of safety for a stable static condition (F+So,) to a factor of safety of 1.0 with an earthquake loading (F+Se, = 1.0), use

```
         b
k+cs, =  )  (F+So, - 1) = (F+So, -1) sin [theta]
         h
```

If the seismic force is in the horizontal direction and denoting such force as k+ch, W, then k+ch, = (F+So,-1) tan[theta].

For granular, free-draining material with plane sliding surface (Figure 9b): F+So, = tan[phi]/tan[theta], and k+cs, = (F+So, -1)sin[theta].

Based on several numerical experiments reported in Reference 7, Critical Acceleration Versus Static Factor of Safety in Stability Analysis of Earth Dams and Embankments, by Sarma and Bhave, k+ch, may be conservatively represented as k+ch, [approximately] (F+So, -1)0.25.

The downslope movement U may be conservatively predicted based on Reference 8, Effect of Earthquakes on Dams and Embankments, by Newmark as:

```
       V.2-                       A
U  =  )))))))))) [multiplied by] )))))
      2g k+cs,                   k+cs,
```

NAVFAC DM 7.1 (1986)

ATTACHMENT C (1/1)

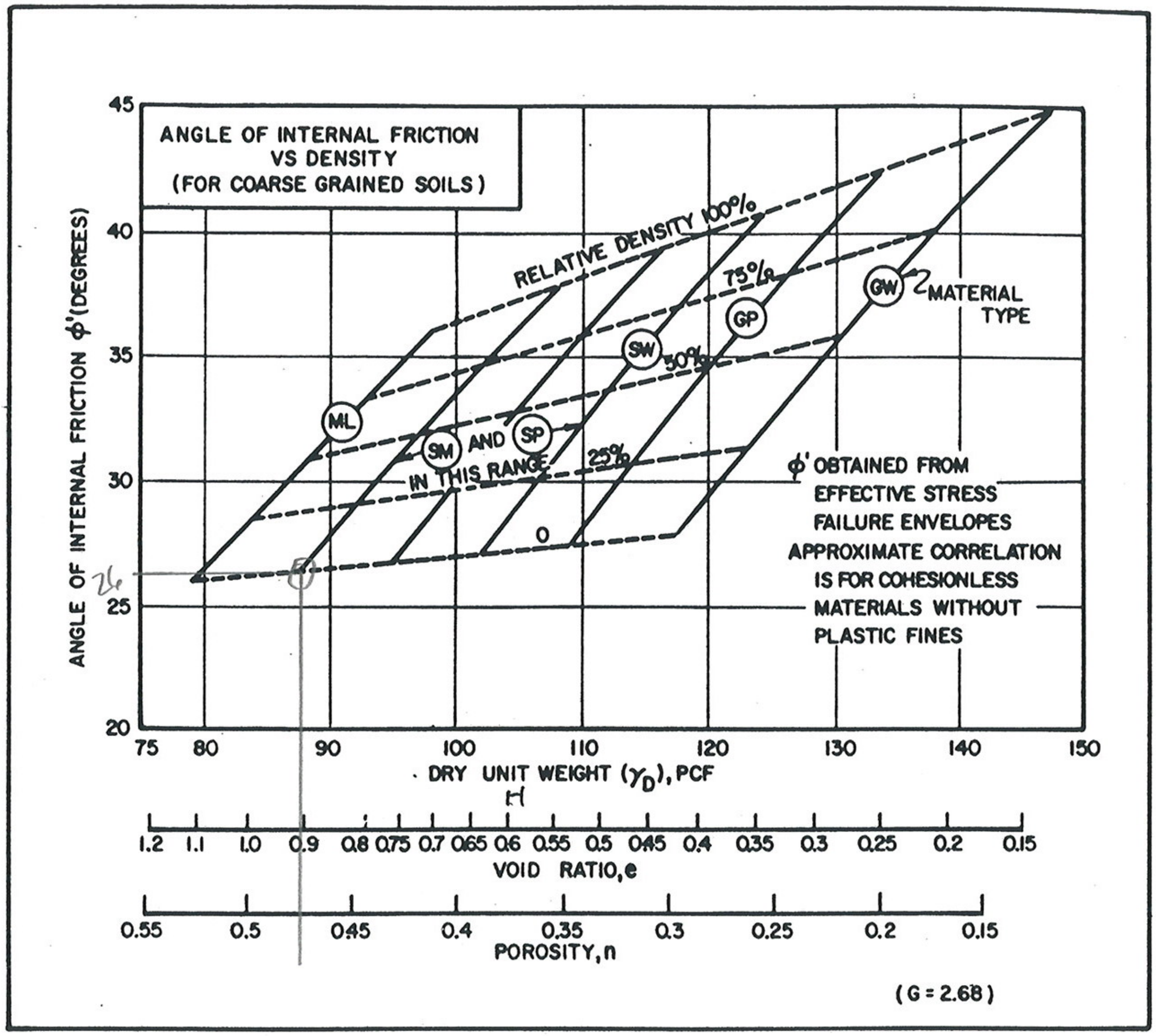


FIGURE 7
Correlations of Strength Characteristics for Granular Soils

NAVFAC DM7.1 (1986)

ATTACHMENT D (1/1)

M
F
G
consulting
scientists and
engineers

TECHNICAL MEMORANDUM

To: JoAnn Tischler, TetraTech EMI, Denver **MFG Project: 181413x**

FROM: Tom Chapel

Date: June 7, 2006

Subject: White Mesa Stability Analysis

This memorandum presents details and results of slope stability analyses performed for an earthen embankment at the White Mesa Project near Blanding, Utah. The embankment was designed in approximately 1988 by Umetco Minerals Corporation, with details described in a report titled "*Cell 4 Design, Tailings Management System, White Mesa Project, Blanding, Utah*". The text of that report, excluding appendices was provided for our review, as were Sheet C4-1 and Sheet C4-2, plans prepared by Western Engineers, Inc. and dated January 17, 1989. Sheet C4-1 shows the location of Cell 4 and other facilities; and Sheet C4-2 shows cross sections at specified locations. The locations and configuration of the section used in our analyses are described later in this memorandum. In addition to the design report and plan sheets, we received a packet titled *Dike Construction, Soil Properties*, and one titled *Dike Construction, Compaction Test*. These documents are copies of laboratory and field tests characterizing the site soils from tests performed during design and construction of the embankment.

We understand the International Uranium (IUSA) Corporation is considering using Cell 4 to impound water and tailings. As part of the permitting process, IUSA has been requested to evaluate the stability of the 2h:1v embankment slope that was constructed on the Cell 4-A side of an embankment constructed between Cells 4-A and 4-B. Tetra Tech has evaluated the stability of the 2h:1v embankment slope. Our methodology, results, conclusions, and opinions are presented in the following paragraphs.

The design report indicates Cell 4-A and Cell 4-B are adjacent cells of a tailing impoundment, each approximately 1150 acre feet with final surface areas of 40 acres each. The tailings will be impounded on the upstream side of a homogenous earth dike. The embankment that is the subject of our investigation is a homogenous earthen embankment constructed between Cell 4-A and Cell 4-B. The general site layout and location are shown on Figure 1.

Several geotechnical investigations were conducted at the site between 1978 and 1981 and results are described in the design report. The embankment was constructed of on-site soils classified as CL and/or ML according to the Unified Soil Classification method (USCS). In the vicinity of Cell 4, bedrock is reported to be sandstone of the Dakota Formation that was encountered at depths of 3.5 to 13 feet. The bedrock is described as including discontinuous lenses of claystone and siltstone. Groundwater was found at depths of 70 and 110 feet below the ground surface in the vicinity of Cell 4.

According to the design report, the embankment base was prepared by removing topsoil, then compacting and proof-rolling the base to identify soft areas, which were removed and replaced with suitable soils.

The embankment was constructed using 12 inch loose layers compacted and tested. Test results provided to us support the methods described in the design report.

The design report included a slope stability analysis performed on the Cell 4-B side of the separating embankment using a STABR computer model, the Ordinary Method of slices, and Bishops modified Method of analysis. That analysis indicated a minimum factor of safety of 1.5 for a 25 foot high embankment and a 3h:1v slope, assuming a saturated, steady state condition in which water was impounded to a level 2 feet below the crest of the embankment. The section was also analyzed using a 0.1g lateral load and a minimum factor of safety of 1.1 was calculated.

Tetra Tech modeled the slope using Cell 4 cross section D-D' shown on Sheet C4-2. We assumed a maximum crest elevation of 5608 feet, a crest width of 18 feet, a side slope of 2h:1v on the Cell 4-A side of the embankment, and a side slope of 3h:1v on the Cell 4-B side of the embankment. This resulted in a maximum embankment height of 46 feet, including 28 feet of man-placed, fine, silty sand fill over seven feet of natural silty sand, over sandstone bedrock. Where the excavation penetrated the bedrock we assumed a one foot thick layer was processed to a sand soil condition and recompacted in place. IUSA indicated a minimum 3 foot freeboard will be maintained. The soil parameters used in our analysis were taken from Figure 3.4-1 of the design report, and are shown in Table 1 below:

Table 1. Soil Properties

Unit	Description	Phi (degrees)	Cohesion, c (psf)	Total unit weight (pcf)
1	water	0	0	62.4
2	Compacted fine, silty sand	30	0	123
3	Natural silty sand	28	0	120
4	bedrock	-	-	-

We evaluated the embankment stability with Slope/W software by Geoslope International, using Spencers method, Bishops modified method, and the Ordinary method of slices. We evaluated a steady state condition under static conditions and using a 0.1g seismic loading. IUSA requested we model the slope in a submerged condition assuming a no-strength fluid (water) as one alternative; and in a submerged condition with an impermeable synthetic liner/barrier as a second alternative. We understand that rapid draw down conditions are not applicable for this application. Figures 2 and 3 show the slope conditions and minimum factors of safety for the static and seismic conditions and the steady state, saturated condition. Figures 4 and 5 show the slope conditions and minimum factors of safety for static and seismic conditions assuming an impenetrable barrier between the water and the soil. Minimum safety factors are summarized in Table 2, below:

E (2/8)

Table 2. Minimum Factors of Safety

Figure	Condition	Calculated Minimum Factor of Safety
2	Unlined alternative, static, steady state	1.42
3	Unlined Alternative, 0.1g seismic	0.93
4	Lined Alternative, static	1.88
5	Lined Alternative, 0.1g seismic	1.37

The Slope/W software includes a feature called "safety mapping" which plots variable numbers of slip surfaces in addition to the critical failure surface. These radii can be seen in Figures 2 and 3 and show primary failure planes are generally more deep seated, but the slope has a much higher factor of safety against the larger failure planes. A similar plot is included in Figures 4 and 5, however the slip surfaces (including the critical radius) are very small and occur near the crest of the embankment.

The results of our analysis indicate the minimum factors of safety for the unlined alternative are lower than recommended standards. A factor of safety of 1.0 indicates an unstable condition. However, these scenarios assumed an unlined saturated, condition and are therefore not representative of the planned construction. We understand the planned construction is with double synthetic liners with a drain medium and solution recovery system between the liners. The unlined alternative is not a valid analysis if the Cell is completed according to the reported plans. The lined alternative had minimum safety factors greater than commonly accepted standards for both the static and seismic conditions. The impoundment should not be used in an unlined condition unless additional analyses are performed that indicate acceptable performance, but if the construction is completed as described then the dike between Cell 4-A and 4-B with the side slope of 2h:1v meets or exceeds recommended standards for stability and safety factors.

We assumed that as-constructed soil conditions are as indicated in the design report and according to data from tests performed during the actual construction, and significant changes have not occurred since the time of construction. These analyses and results should be considered valid only for the conditions described herein.

We understand the soil/liner stability issues will be addressed by others. If you have any questions regarding our analysis or this memorandum, please contact the undersigned.

Thomas A. Chapel, CPG, PE

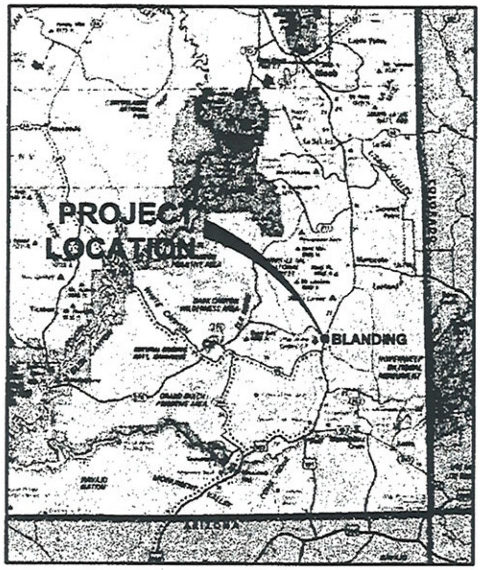
Senior Geotechnical Engineer

Reviewed by:

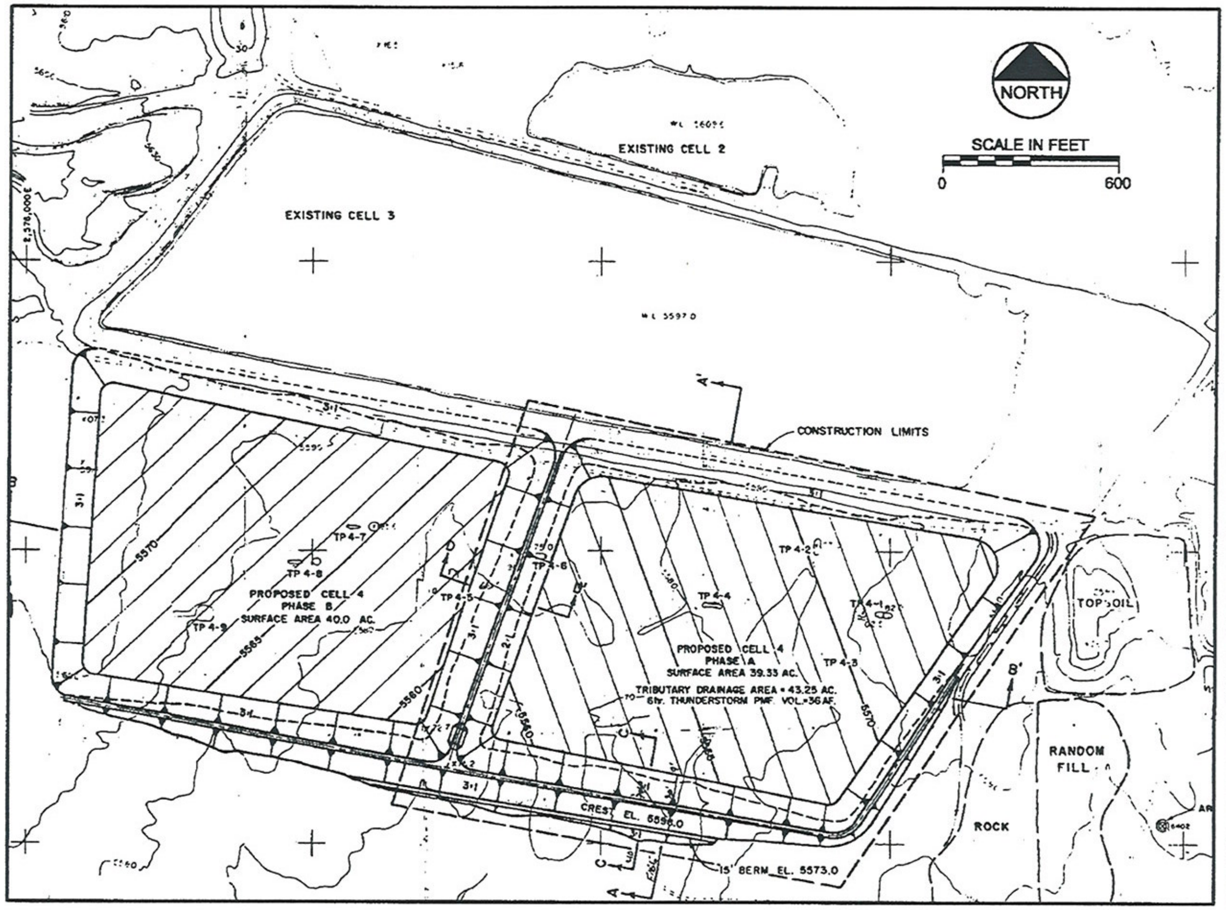
Daniel D. Overton, PE

Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
No. 262501
DANIEL D. OVERTON
12/31/06
STATE OF UTAH



VICINITY MAP



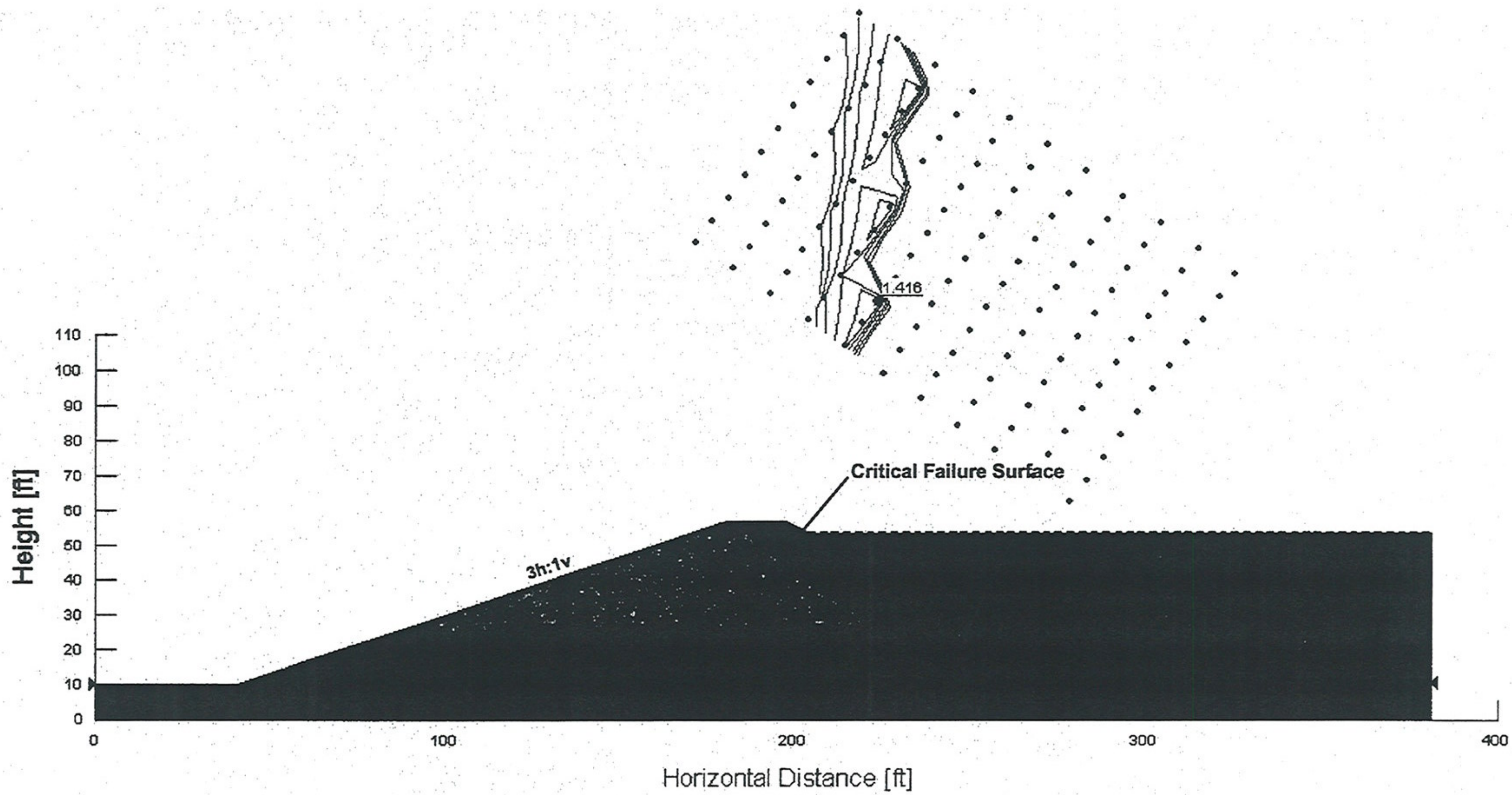
PROJECT AREA

TETRA TECH, INC.

FIGURE 1
SITE LOCATION MAP

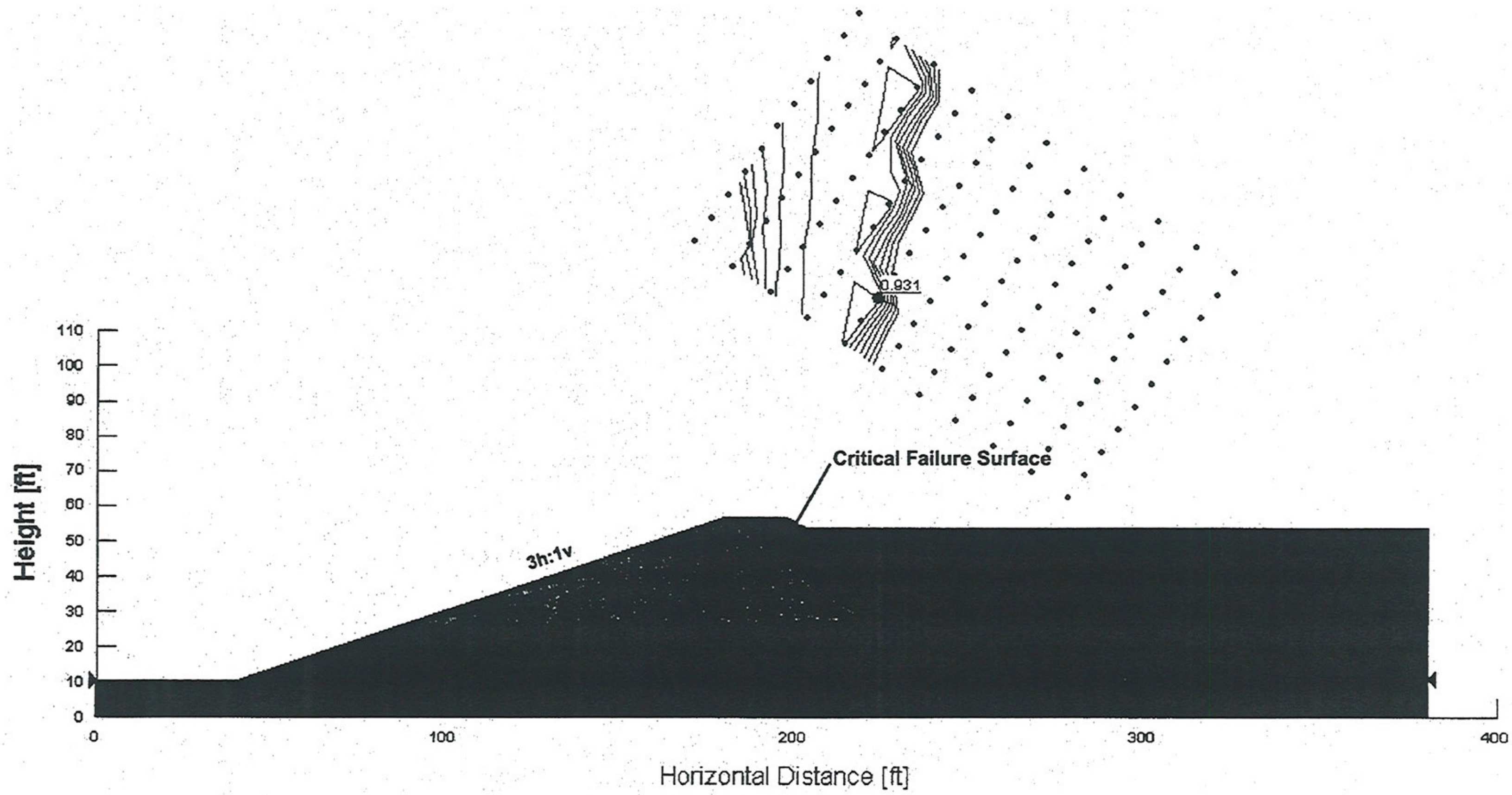
Date:	JUNE 2006
Project:	181413X
File:	LOCATION.DWG

E (4/8)



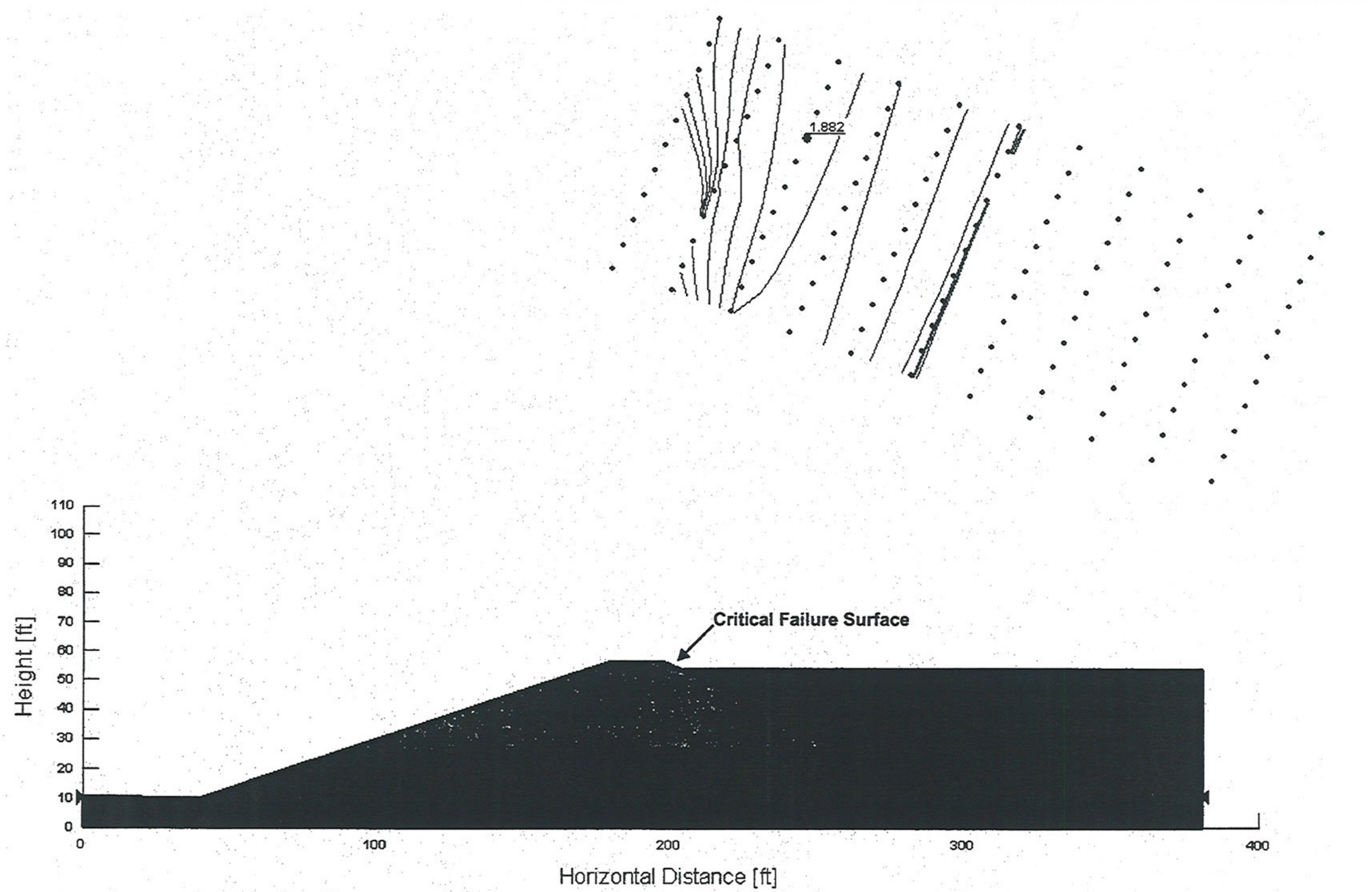
TetraTech, Inc.	FIGURE 2 Unlined Alternative Static Condition	International Uranium Corp. Project: White Mesa (181413x) 06/01/06

E (5/8)



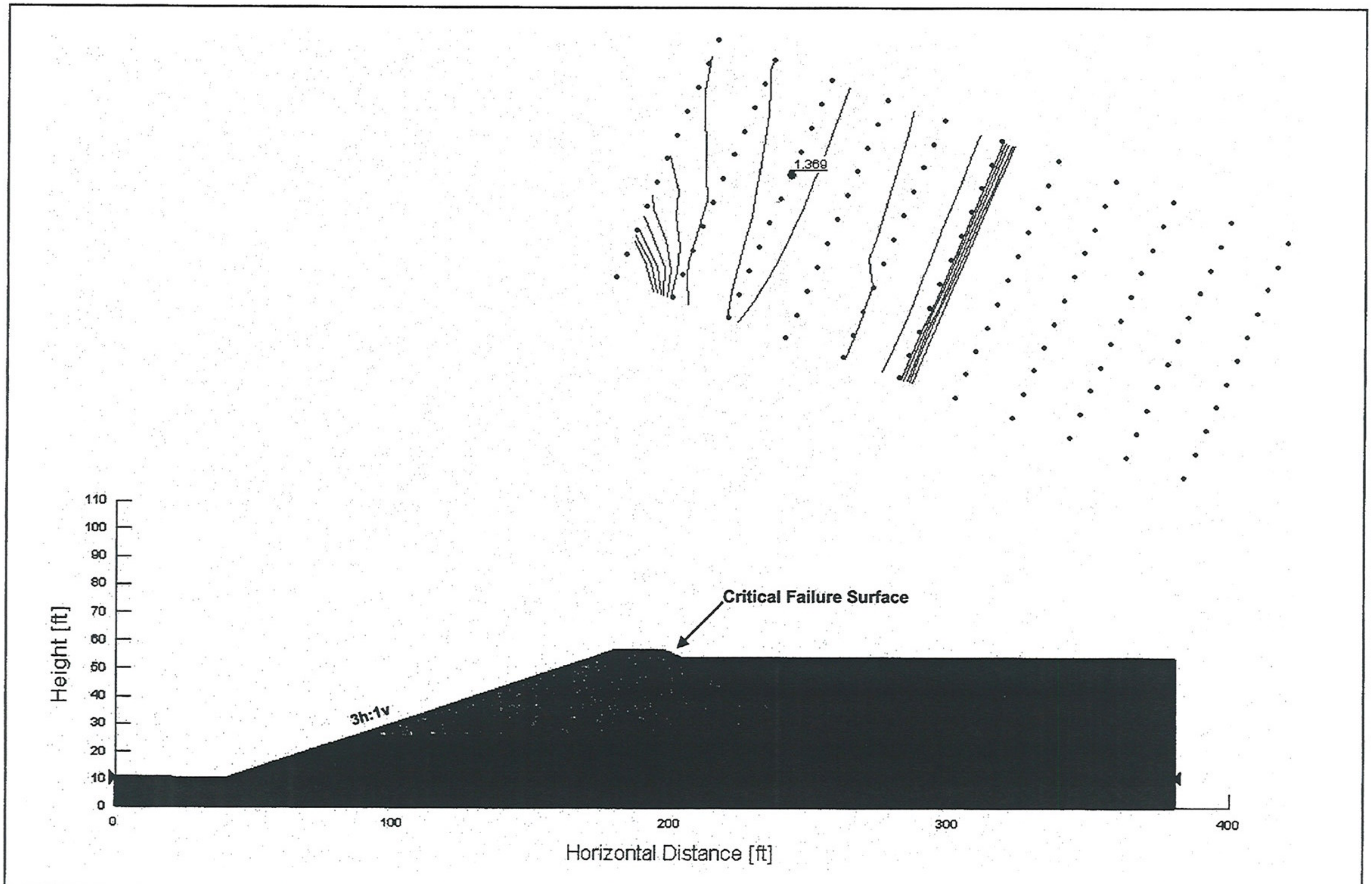
TetraTech, Inc.	FIGURE 3 Unlined Alternative Seismic Condition	International Uranium Corp. Project: White Mesa (181413x) 06/01/06

E(6/8)



TetraTech, Inc.	FIGURE 4 Lined Alternative Static Condition	International Uranium Corp. Project: White Mesa (181413x) 06/01/06

E(7/8)



TetraTech, Inc.	FIGURE 5 Lined Alternative Seismic Condition	International Uranium Corp. Project: White Mesa (181413x) 06/01/06

E (8/8)

M
F
G
consulting
scientists and
engineers

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A TETRA TECH COMPANY
Fort Collins Office
3801 Automation Way, Suite 100
Fort Collins, CO 80525
970.223.9600
Fax: 970.223.7171

July 13, 2006

Tetra Tech EM, Inc.
950 17th Street, 22nd Floor
Denver, Colorado 80202

MFG Project No. 181413x

Attn: Ms. JoAnn Tischler

Draft

Subject: Soil Property Verification and
Slope Stability Analyses
Earthen Embankment between Cells 4A and 4B,
IUC White Mesa Project
Blanding, Utah

Tetra Tech MFG prepared a technical memorandum dated June 7, 2006, and a letter dated June 9, 2006 describing slope stability analyses, assumptions, and recommendations for verification of soil properties for an earthen embankment at the International Uranium (USA) Corporation, White Mesa Project near Blanding, Utah.

On June 15, 2006, Tetra Tech drilled an exploratory boring in the embankment between Cell 4A and Cell 4B at the approximate location shown on Figure 1 (attached). Descriptions of soils encountered in the boring are shown on the Borehole log (also attached). The boring was drilled to a depth of 30 feet and sampled at 5 foot intervals using a 2 inch diameter California sampler driven into the soil by a 140 pound weight dropped 30 inches (a Standard Penetration Test, SPT). Samples were examined by a geotechnical engineer in our soils laboratory. Samples were selected and tested for moisture and density and Atterberg Limits to determine their classification and similarity to properties identified in previous geotechnical reports for the project. A triaxial test was performed to compare the angle of internal friction and cohesion of the in-place soil with the values determined by the original designers in 1981.

The moisture and density of the samples tested are shown in Table 1 below:

ATTACHMENT F (1/7)

Table 1. Soil Properties

Depth	Description	Wet Density (pcf)	Dry Density (pcf)	Moisture content (%)
10	Silty sand	136.5	125.0	9.2
20	Silty sand	140.5	126.3	11.3
25	Silty sand	134.7	122.6	9.9
-	Average	137.2	124.6	10.1

Atterberg limits tests indicate a liquid limit of 25, and a Plasticity Index of 13, with 50 percent silt and clay sized particles (passing the number 200 sieve). Triaxial testing indicated an effective angle of internal friction of 26.5 degrees and a drained cohesion of 957.5 psf.

These test results indicate although the samples were visually classified as silty sand, laboratory tests indicate the embankment soils tested are a very sandy clay rather than sand and silty sand as reported by others and assumed in our initial analysis.

We performed additional slope stability analyses using the following soil properties: an average moist unit weight of 137 pcf, an angle of internal friction of 26 degrees, and an effective cohesion of 900 psf. We calculated the minimum factors of safety shown in Table 2.

Table 2. Revised Minimum Factors of Safety

Condition	Calculated Minimum Factor of Safety
Unlined alternative, static, steady state	2.45
Unlined Alternative, 0.1g seismic	1.67
Lined Alternative, static	4.61
Lined Alternative, 0.1g seismic	3.21

Therefore the factors of safety calculated and presented in our June 2 Technical Memorandum are conservative. In fact, analyses using the measured soil properties indicate that the embankment exceeds typical minimum acceptable safety factors even in the event leakage were to occur from the liner and produce a saturated condition as shown in Figure 3 of our previous memorandum.

If you have any questions regarding our analysis, our previous correspondence, or this letter, please contact the undersigned.

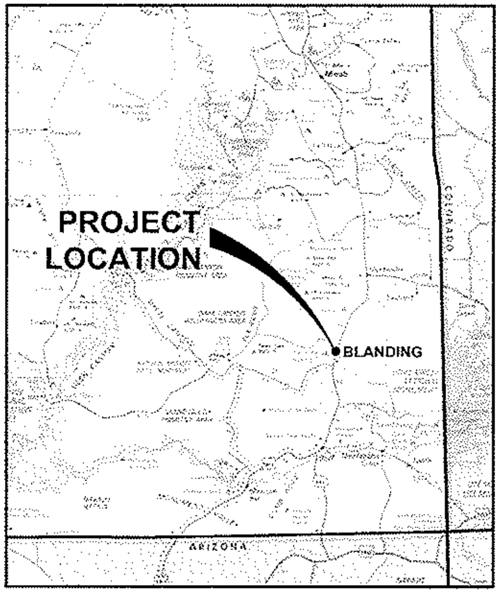
Respectfully submitted,

Tetratech MFG, Inc.

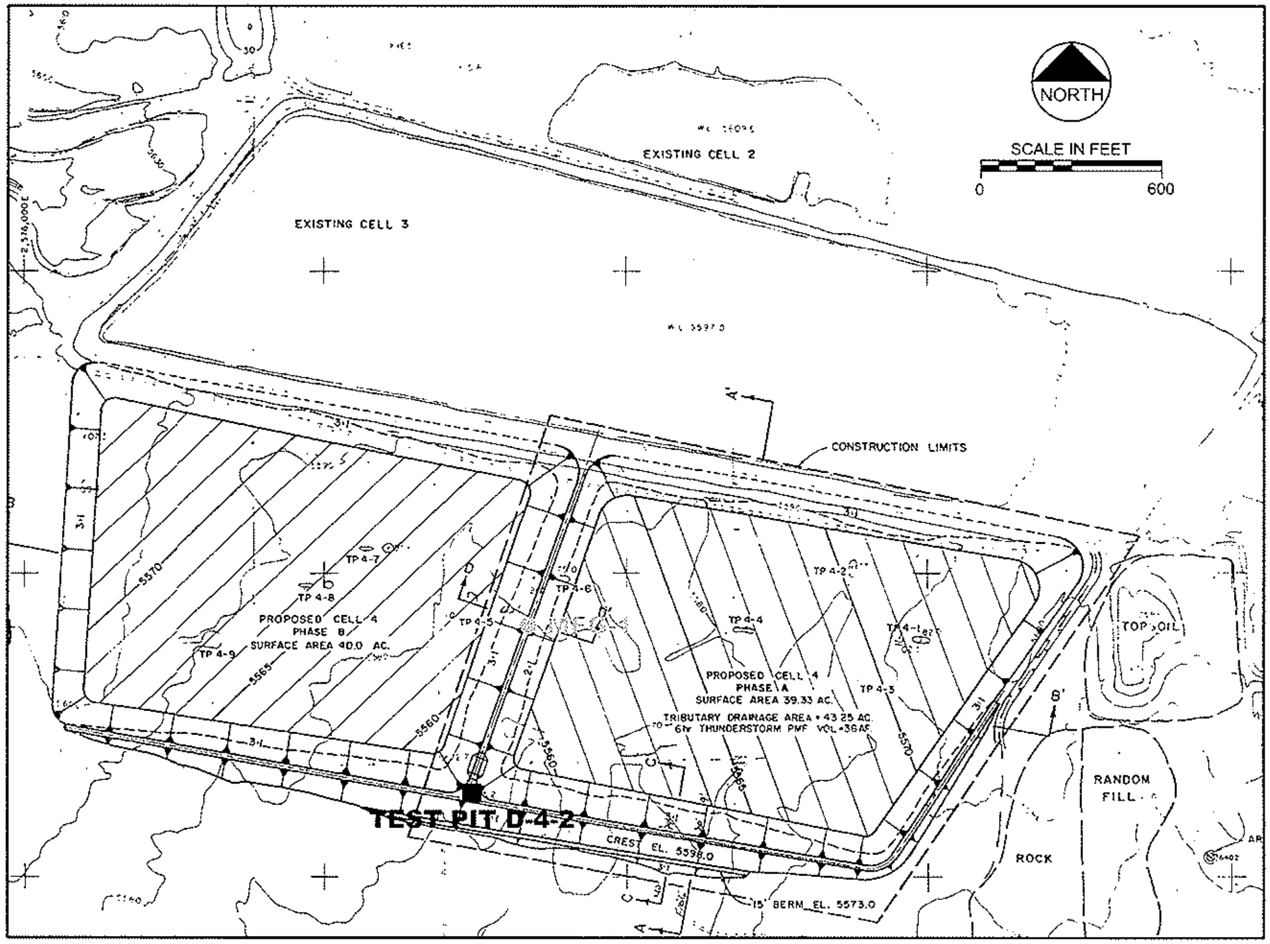
Thomas A. Chapel, CPG, PE
Senior Geotechnical Engineer

2 copies sent

F (3/7)



VICINITY MAP



PROJECT AREA

● 4-INCH SOLID AUGER TO 30 FT. SAMPLE AT 5 FT. INTERVALS BACKFILL WITH BENTONITE

TETRA TECH, INC.

FIGURE 1
SITE LOCATION MAP

Date:	JUNE 2006
Project:	181413X
File:	LOCATION.DWG

F (4/7)

BOREHOLE LOG

MFG, Inc.
consulting scientists and engineers

PAGE: 1 OF 3

DATE: 6/15/06

BOREHOLE NO.: **MFG-1**

PROJECT INFORMATION

PROJECT: WHITE MESA

PROJECT NO.: 181413X

CLIENT: TETRA TECH EMI

OWNER: INTERNATIONAL URANIUM (IUSA) CORPORATION

LOCATION: BLANDING, UTAH

BOREHOLE LOCATION

SEE FIGURE 1

FIELD INFORMATION

DATE & TIME ARRIVED: 6/15/06 9:00AM

BOREHOLE LOGGED BY: NMT

VISITORS: NONE

WEATHER: PARTLY CLOUDY, SLIGHT BREEZE, APPROX. 80°

DRILLING INFORMATION

DRILLING COMPANY: DA SMITH DRILLING

START TIME: 11:10AM

BORING DEPTH: APPROX. 31' BORING DIA.: 6"

DRILLING METHOD: CME 75 SOLID STEM AUGER

SAMPLING METHOD: 2-IN CA SAMPLES

TIME DRILLING COMPLETE: 12:50PM

BOREHOLE COMPLETION / ABANDONMENT INFORMATION

START TIME: 12:50PM COMPLETE TIME: 1:10PM

INSTRUMENTATION: NONE BACKFILL: BENTONITE

GROUNDWATER CONDITIONS

GROUNDWATER WAS NOT ENCOUNTERED DURING DRILLING

FOLLOWING FIELD WORK

TIME OF CLEAN-UP COMPLETE: 1:10PM TIME LEFT SITE: 1:50PM

NOTES:

F(5/7)

MFG, Inc.
consulting scientists and engineers

BOREHOLE LOG

PROJECT: *WHITE MESA* **PAGE:** *2 OF 3*

PROJECT NO.: *181413X* **DATE:** *6/15/06*

BOREHOLE NO.: **MFG-1**

DEPTH (FT)	CORE RECOV.	DRIVE SAMPLES: SAMPLE TYPE	DRIVE SAMPLES: BLOWS (PER 6")	DRIVE SAMPLES: RECOV.	ADD'L SAMPLES	LITHOLOGY GRAPHIC	SOIL DESCRIPTION
0							**COAL COVER AT SURFACE (APPROX. 0.25')**
0–5.5							**SILTY CLAY (0 TO APPROX. 5.5')** SLIGHTLY MOIST, LIGHT OLIVE BROWN (2.5Y 5/3), VERY STIFF SILTY CLAY FILL, TRACE SAND, TRACE PEBBLES, WHITE PRECIPITATE, ZONES OF COLOR CHANGE TO RED (2.5YR 4/6). APPROX. 0.5' - MOIST.
5–6.5		CA B A	11 19 33	17"			**SILTY SAND (APPROX. 5.5' TO APPROX. 30')** SLIGHTLY MOIST, RED (2.5YR 5/6), VERY DENSE SILTY SAND, FINE TO MEDIUM GRAIN, TRACE TO SOME CLAY, WHITE PRECIPITATE. APPROX. 6.5' - SANDSTONE FRAGMENTS, DRY, PINK (5YR 8/3), VERY DENSE, MEDIUM CEMENTATION, FINE GRAIN.
10–11.5		CA B A	15 32 43	13"			
15–16.5		CA B A	13 18 36	18"			APPROX. 15' - ZONES OF SANDY CLAY VARIOUS COLORS, MOIST.
20							

F(6/7)

MFG, Inc. *consulting scientists and engineers*	**BOREHOLE LOG** **PROJECT:** *WHITE MESA* **PAGE:** *3 OF 3* **PROJECT NO.:** *181413X* **DATE:** *6/15/06*	**BOREHOLE NO.:** **MFG-1**

DEPTH (FT)	CORE RECOV.	DRIVE SAMPLES: SAMPLE TYPE	DRIVE SAMPLES: BLOWS (PER 6")	DRIVE SAMPLES: RECOV.	ADD'L SAMPLES	LITHOLOGY GRAPHIC	SOIL DESCRIPTION
20							**SILTY SAND (APPROX. 5.5' TO APPROX. 30')** SEE DESCRIPTION ON PREVIOUS PAGE.
20–21.5		CA B A	15 29 50/6"	18"			
24							APPROX. 24' - SLIGHTLY MOIST.
25–26.5		CA B A	12 13 20	13"			
30							**SANDSTONE (APPROX. 30' TO E.O.B.)** SLIGHTLY MOIST, PINK (2.5YR 8/3), VERY DENSE SANDSTONE, FINE TO MEDIUM CEMENTATION, FINE GRAIN.
30–31		CA B A	38 50/5"	13"			
31							E.O.B. = 31.0'

Depth scale: 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40

F(7/7)

Caltrans (Equivalent Loading to AASHTO)

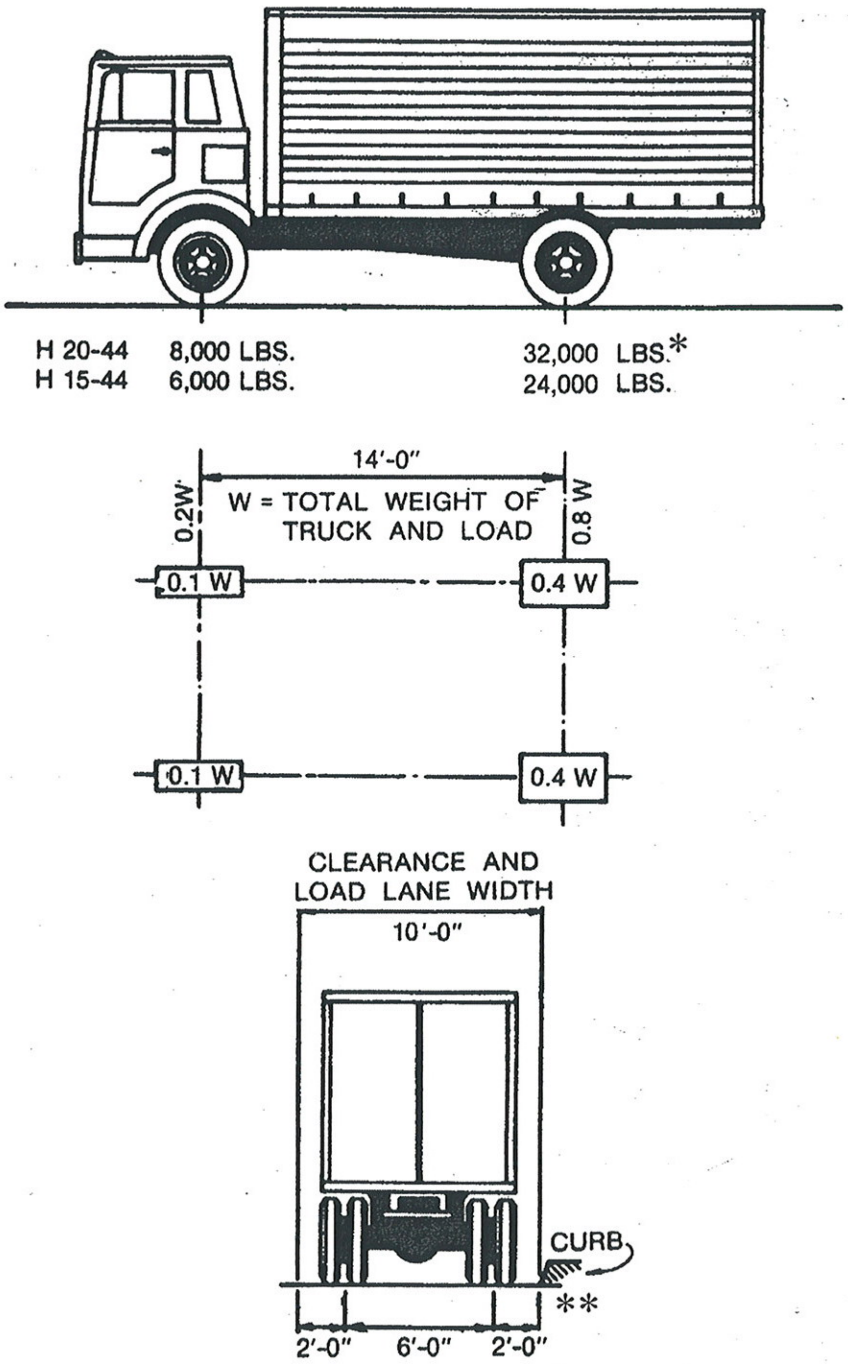
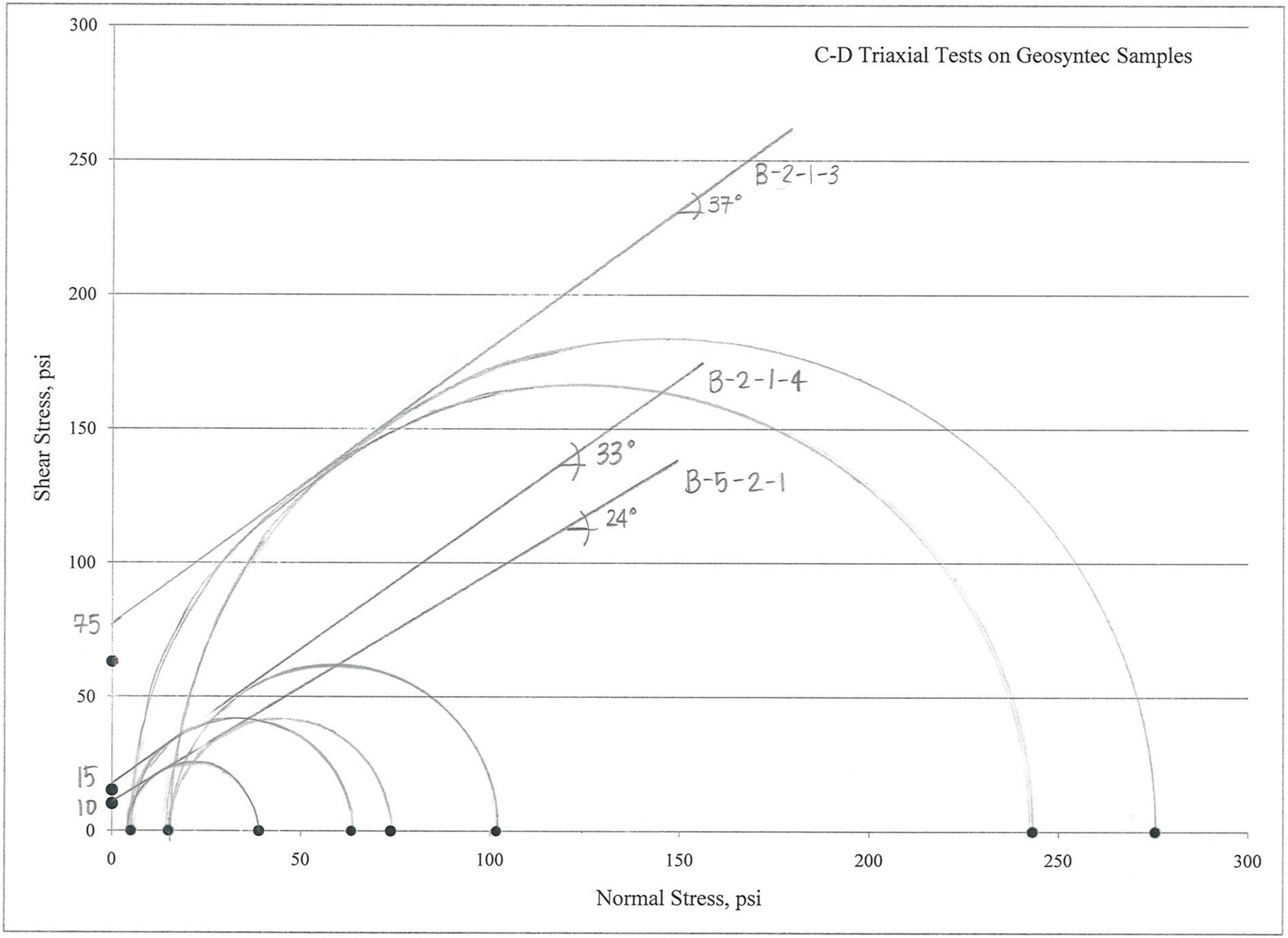


FIGURE 3.7.6A Standard H Trucks

* In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H 20 loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

** For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2)

ATTACHMENT G (1/1)

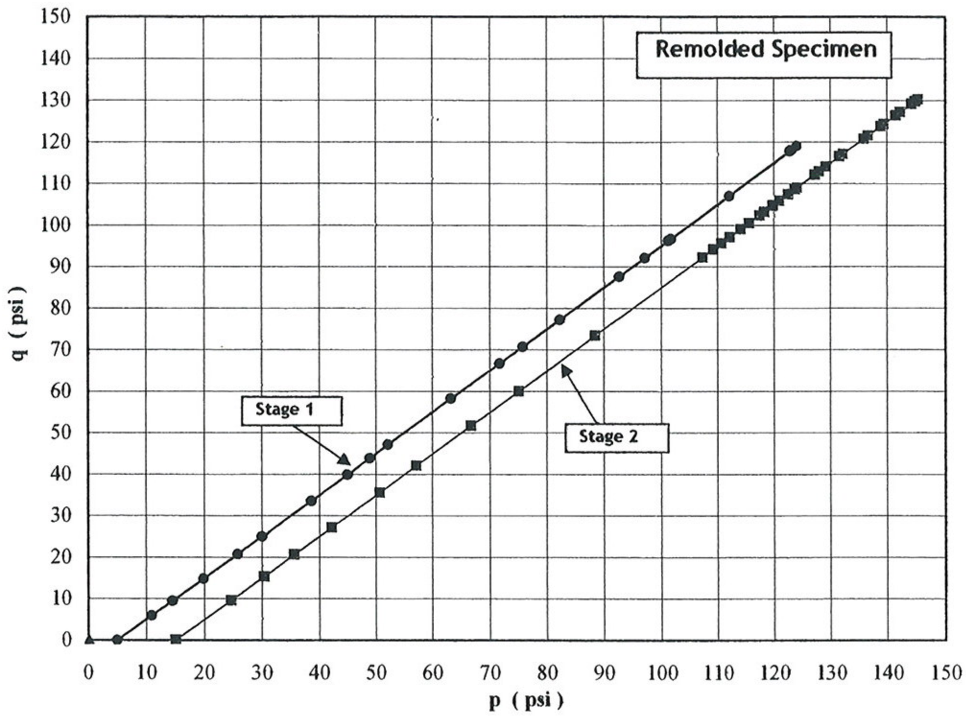


ATTACHMENT H (1/7)

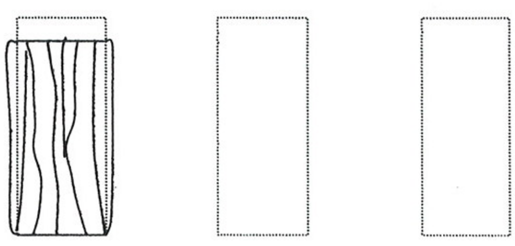
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
941 Forrest Street, Roswell, Georgia 30075
Tel: (770) 650 1666 Fax: (770) 650 5786

Project Name:	White Mesa Mill - Cell 4 A
Project No:	246
Site Sample ID:	B-2-1-3
Lab Sample No:	E100

CONSOLIDATED-DRAINED (CD) TRIAXIAL TEST



Test Specimen No.	Initial Conditions							Strain Rate	Sample Quality Bad to Good
	Height (in .)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	u_i (psi)	σ'_c (psi)	(% / min)	(1 to 10)
1	3.03	1.92		120.8	NA	NA	5.0	0.297	9
1	3.03	1.92		120.8	NA	NA	15.0	0.297	9



Specimen No.1 Specimen No. 2 Specimen No. 3

Notes:

u_i = Initial pore pressure,(psi)

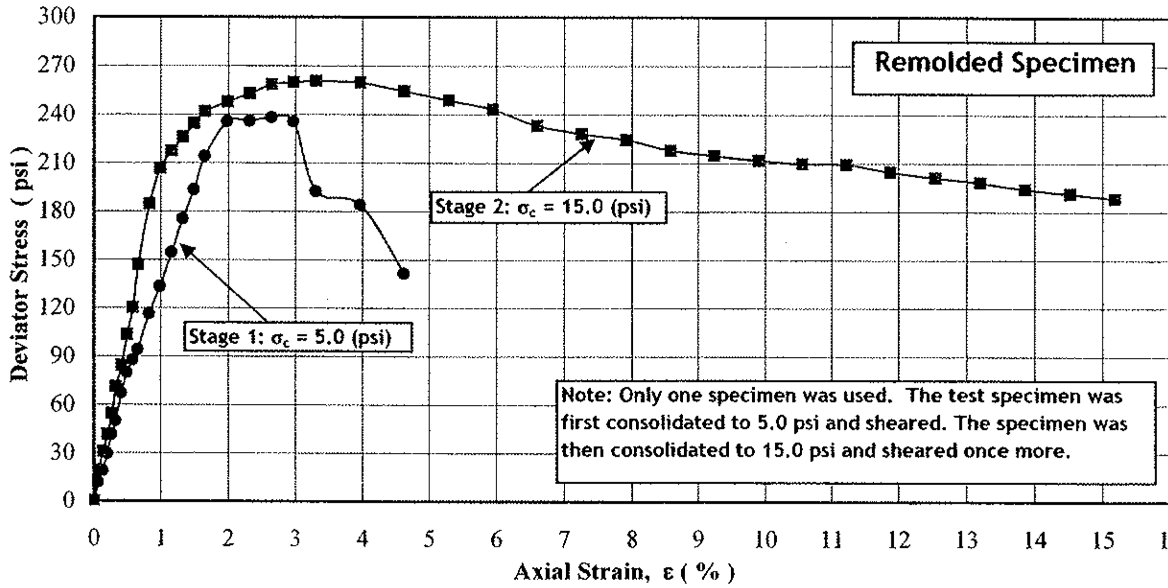
σ'_c= Consolidation pressure, (psi)

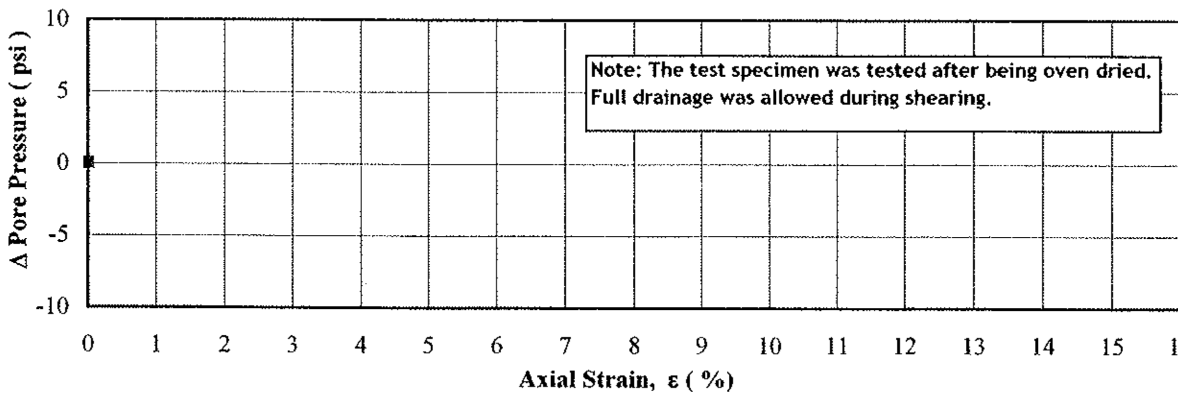
Remolded specimen was formed by tamping loose/broken down soil, at a moisture content of app. 20%, in one-centimeter-thick layers utilizing very high tamping energy. The remolded test specimen was then dried, while still in the mold, in the oven over night and then tested.

H (2/7)

Excel Geotechnical Testing, Inc. "Excellence in Testing" 941 Forrest Street, Roswell, Georgia 30075 Tel: (770) 650 1666 Fax: (770) 650 5786	**Project Name:** White Mesa Mill - Cell 4 A **Project No:** 246 **Site Sample ID:** B-2-1-3 **Lab Sample No:** E100

CONSOLIDATED-DRAINED (CD) TRIAXIAL TEST





Test Specimen No.	Maximum Strength				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	238.1	243.1	5.0	NA	2.6
1	260.5	275.5	15.0	NA	3.3

Test Specimen No.	Strength at End of Test				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	141.5	146.5	5.0	NA	4.6
2	188.1	203.1	15.0	NA	15.2

Notes:

σ'_c = Consolidation pressure, (psi) u_i = Initial pore pressure,(psi)

σ'_1 = Effective axial stress, (psi) σ'_3 = Effective radial stress (confining pressure), (psi) $\sigma'_1 - \sigma'_3$ = Deviator stress, (psi)

u = Pore pressure,(psi) ε_a = Axial strain, (%)

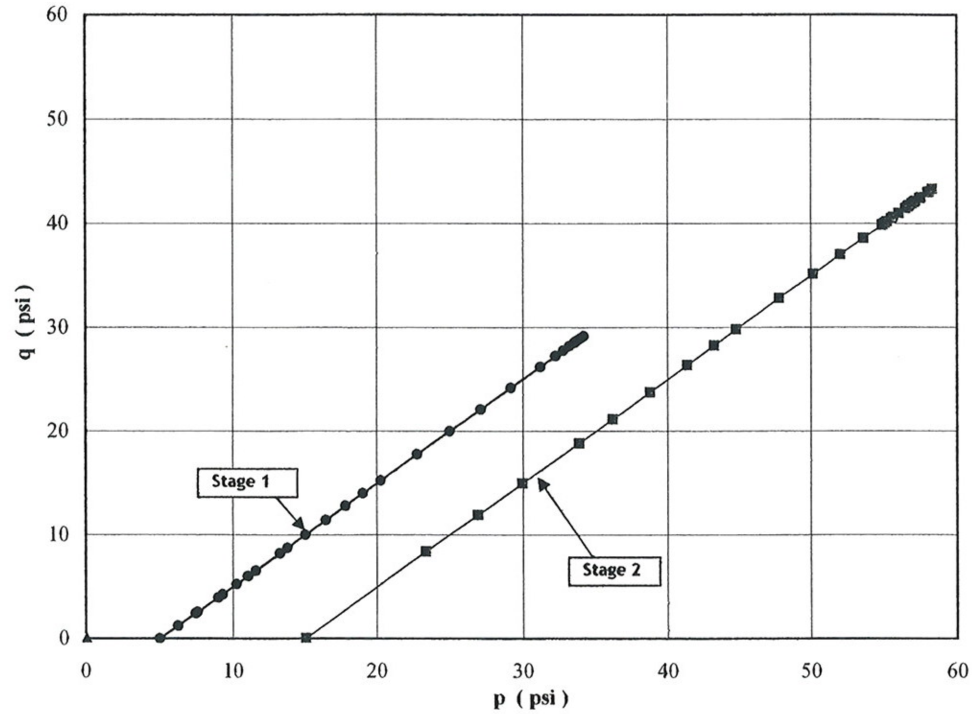
Remolded specimen was formed by tamping loose/broken down soil, at a moisture content of app. 20%, in one-centimeter-thick layers utilizing very high tamping energy. The remolded test specimen was dried, while still in the mold, in the oven over night and then tested.

H (3/7)

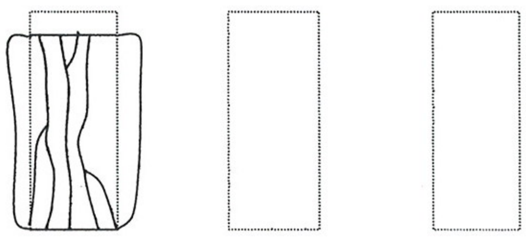
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
941 Forrest Street, Roswell, Georgia 30075
Tel: (770) 650 1666 Fax: (770) 650 5786

Project Name:	White Mesa Mill - Cell 4 A
Project No:	246
Site Sample ID:	B-2-1-4
Lab Sample No:	E101

CONSOLIDATED-DRAINED (CD) TRIAXIAL TEST



Test Specimen No.	Initial Conditions							Strain Rate	Sample Quality Bad to Good
	Height (in .)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	u_i (psi)	σ'_c (psi)	(% / min)	(1 to 10)
1	2.98	1.90	9.5	102.3	NA	NA	5.0	0.302	3
1	2.98	1.90	9.5	102.3	NA	NA	15.0	0.302	3



Specimen No.1 Specimen No. 2 Specimen No. 3

Notes:

u_i = Initial pore pressure,(psi)

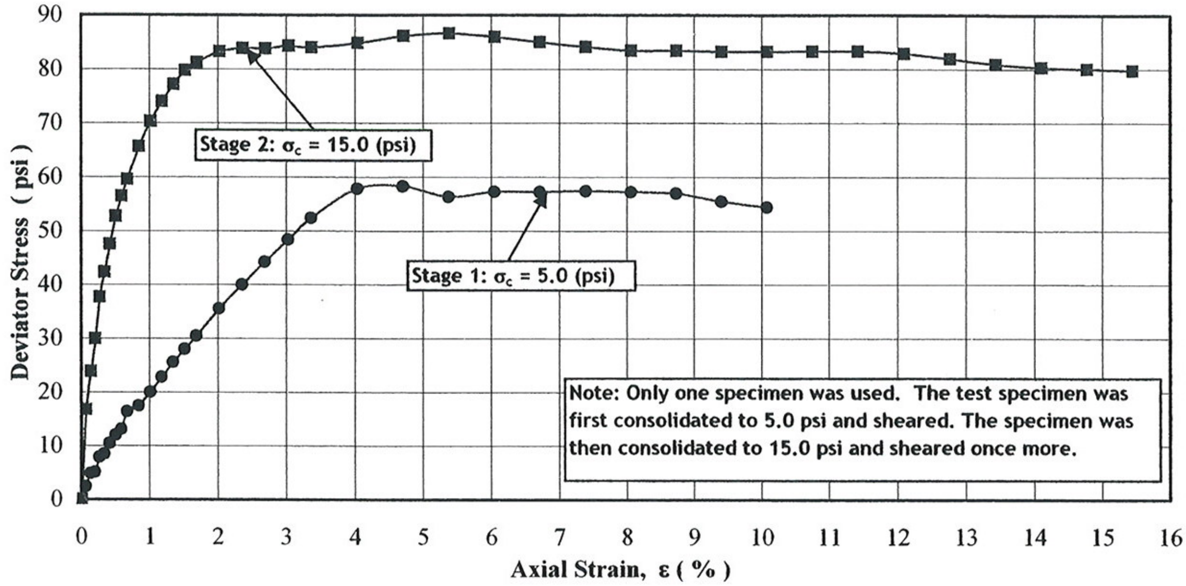
σ'_c= Consolidation pressure, (psi)

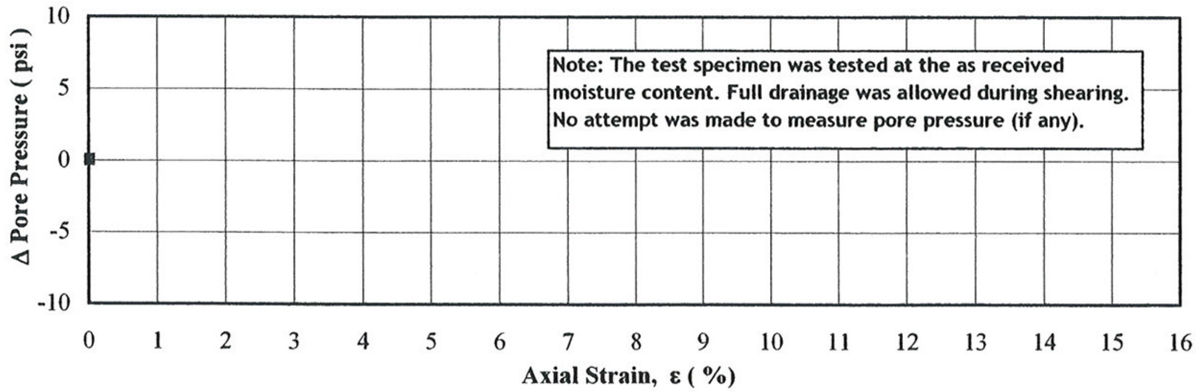
H (4/7)

Excel Geotechnical Testing, Inc.
"Excellence in Testing"
941 Forrest Street, Roswell, Georgia 30075
Tel: (770) 650 1666 Fax: (770) 650 5786

Project Name: White Mesa Mill - Cell 4 A
Project No: 246
Site Sample ID: B-2-1-4
Lab Sample No: E101

CONSOLIDATED-DRAINED (CD) TRIAXIAL TEST





Test Specimen No.	Maximum Strength				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	58.3	63.3	5.0	NA	4.7
1	86.6	101.6	15.0	NA	5.4

Test Specimen No.	Strength at End of Test				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	54.5	59.5	5.0	NA	10.1
2	79.7	94.7	15.0	NA	15.4

Notes:

σ'_c = Consolidation pressure, (psi)
σ'_1 = Effective axial stress, (psi)
u = Pore pressure,(psi)
u_i = Initial pore pressure,(psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
ε_a = Axial strain, (%)
$\sigma'_1 - \sigma'_3$ = Deviator stress, (psi)

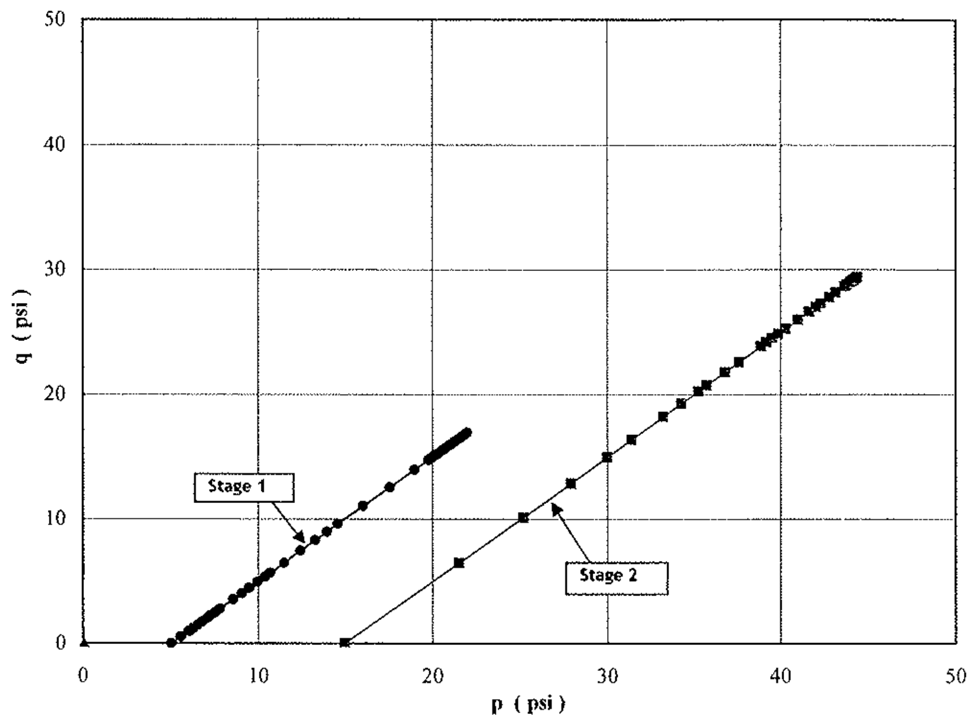
H (5/7)

Excel Geotechnical Testing, Inc.	
"Excellence in Testing"	
941 Forrest Street, Roswell, Georgia 30075	
Tel: (770) 650 1666 Fax: (770) 650 5786	

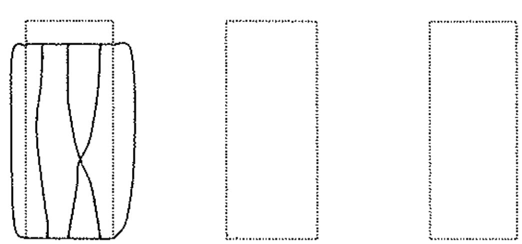
Project Name: White Mesa Mill - Cell 4 A
Project No: 246
Site Sample ID: B-5-2-3
Lab Sample No: E102

ASTM D ----

CONSOLIDATED-DRAINED (CD) TRIAXIAL TEST



Test Specimen No.	Initial Conditions							Strain Rate	Sample Quality Bad to Good
	Height (in .)	Diameter (in.)	Moisture Content (%)	Dry Unit Weight (pcf)	B Parameter (-)	u_i (psi)	σ'_c (psi)	(% / min)	(1 to 10)
1	3.45	1.90	13.4	117.2	NA	NA	5.0	0.261	4
1	3.32	1.90	13.4	121.8	NA	NA	15.0	0.271	4



Specimen No.1 Specimen No. 2 Specimen No. 3

Notes:

u_i = Initial pore pressure.(psi)

σ'_c= Consolidation pressure. (psi)

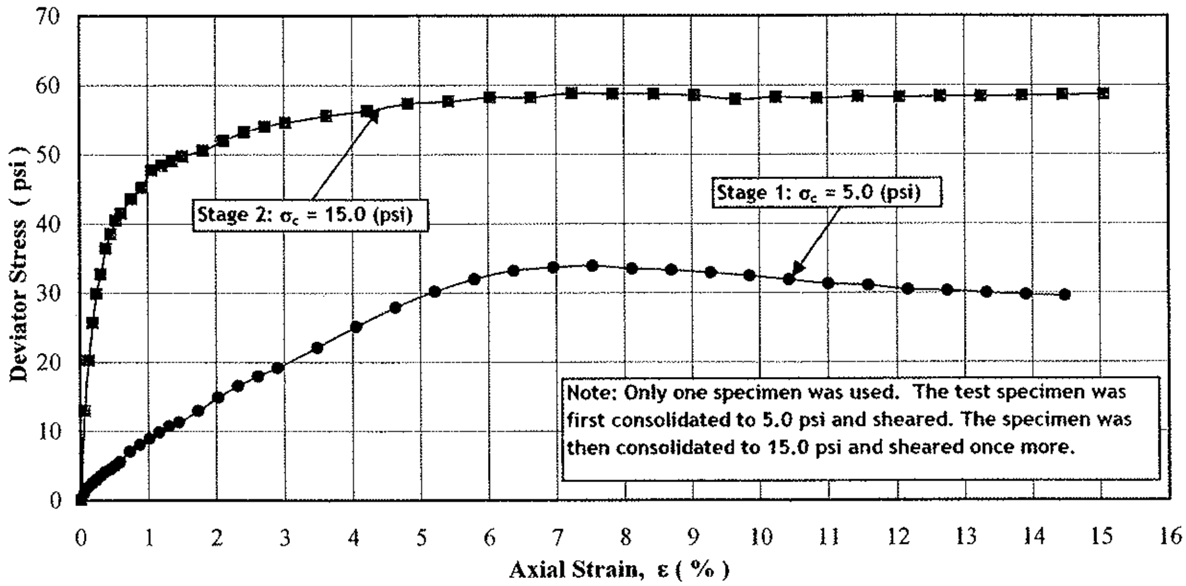
H(6/7)

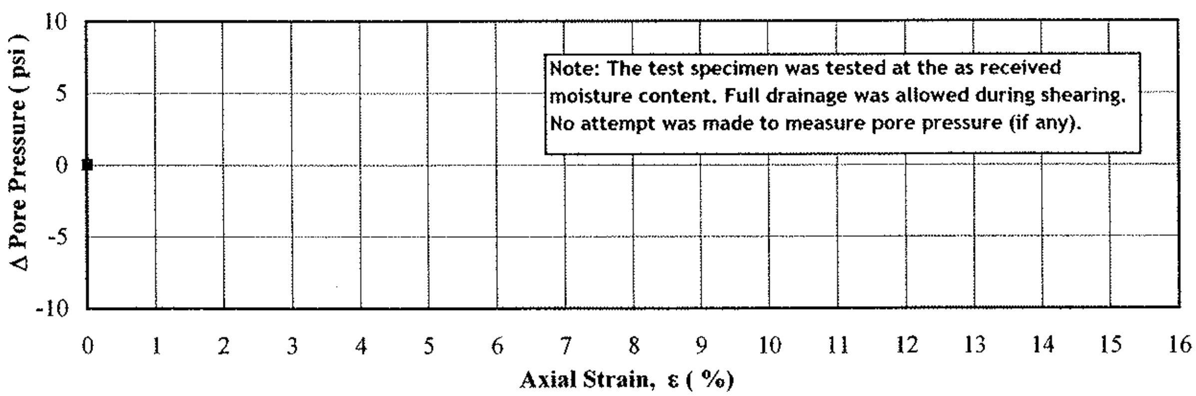
Excel Geotechnical Testing, Inc.
"Excellence in Testing"
941 Forrest Street, Roswell, Georgia 30075
Tel: (770) 650 1666 Fax: (770) 650 5786

Project Name: White Mesa Mill - Cell 4 A
Project No: 246
Site Sample ID: B-5-2-3
Lab Sample No: E102

ASTM D ----

CONSOLIDATED-DRAINED (CD) TRIAXIAL TEST





Test Specimen No.	Maximum Strength				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	33.9	38.9	5.0	NA	7.5
1	58.8	73.8	15.0	NA	7.2

Test Specimen No.	Strength at End of Test				
	$\sigma'_1 - \sigma'_3$ (psi)	σ'_1 (psi)	σ'_3 (psi)	u (psi)	ε_a (%)
1	29.6	34.6	5.0	NA	14.5
2	58.6	73.6	15.0	NA	15.1

Notes:

σ'_c = Consolidation pressure, (psi)
u_i = Initial pore pressure,(psi)
σ'_1 = Effective axial stress, (psi)
σ'_3 = Effective radial stress (confining pressure), (psi)
$\sigma'_1 - \sigma'_3$ = Deviator stress, (psi)
u = Pore pressure,(psi)
ε_a = Axial strain, (%)

H (7/7)

EXHIBIT B

SEISMIC DEFORMATION ANALYSIS CALCULATION PACKAGE

Geosyntec
consultants

COMPUTATION COVER SHEET

Client:	**DMC**	Project:	**White Mesa Mill Facility – Cell 4B**	Project/ Proposal No.: Task No.	**SC0349**

Title of Computations: **SEISMIC DEFORMATION ANALYSIS**

Computations by:	Signature	Meghan Lithgow	07/01/08 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature	Steven Fitzwilliam	7/1/08 Date
	Printed Name	Steven Fitzwilliam	
	Title	Associate	
Computations Checked by:	Signature	Steven Fitzwilliam	7/1/08 Date
	Printed Name	Steven Fitzwilliam	
	Title	Associate	
Computations backchecked by: (originator)	Signature	Meghan Lithgow	07/01/08 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Approved by: (pm or designate)	Signature	Greg Corcoran	7/2/08 Date
	Printed Name	Greg Corcoran, P.E.	
	Title	Principal	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	M. Lithgow	Date:	07/01/08	Reviewed by:	G. Corcoran	Date:	7/02/08
Client: DMC		Project:	White Mesa Mill Facility – Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

CELL 4B EMBANKMENT SLOPES – PERMANENT SEISMIC DEFORMATION
WHITE MESA MILL FACILITY – CELL 4B
BLANDING, UTAH

OBJECTIVE

The objective of this analysis is to evaluate the seismically-induced permanent deformation of the cut slopes for Cell 4B at the White Mesa Mill Facility located in Blanding, Utah. The final slopes of the containment cell consist of 2H:1V (horizontal:vertical) cut slopes.

METHOD OF ANALYSIS

Seismic deformation is a function of average acceleration of the sliding mass and the yield acceleration. Geosyntec used the Makdisi and Seed (Attachment A) method to estimate permanent seismic deformations, based on yield accelerations determined from pseudo-static limit equilibrium analyses, design earthquake motions determined from documented sources, and the attached design charts (Makdisi and Seed, 1978).

Two cross sections depicting embankment slopes created by the construction of Cell 4B were previously analyzed for static stability as part of the design report (Geosyntec, 2007). Two additional cross sections also depicting embankment slopes created by the construction of Cell 4B were analyzed for static stability as part of the response to Utah Department of Environmental Quality (UDEQ) Interrogatory Round 1. The four cross sections are provided as Attachment B.

Cross sections were selected on each of the four (4) perimeter berms to represent the critical slope conditions on each slope. The first cross section, Section A-A', is a west-east cross section that spans Cells 4A and 4B, with berm slopes inclined at approximately 2H:1V and a base grade sloping southeast at approximately 1 percent. The most critical condition (modeled herein) for Section A-A' is the operational condition; Cell 4A is filled with tailings and Cell 4B is empty. The second cross section, Section B-B', is a north-south cross section that spans the southern perimeter of Cell 4B, with berm slopes inclined at approximately 2:1. Section B-B' was modeled under the conditions that Cell 4B is filled with tailings (operational condition, prior to closure). The third cross section, Section C-C', is a north-south cross section that spans Cells 3 and 4B, with berm slopes inclined at approximately 2H:1V. The most critical condition (modeled herein) for Section C-C' is the pre-operational condition; Cell 3 is filled with tailings and Cell 4B is empty. The fourth cross section, Section D-D' is a west-east cross section that spans the western berm of Cell 4B, with berm slopes inclined at approximately 2H:1V and a base grade sloping southeast at approximately 1 percent. Section D-D' was modeled under the conditions that Cell 4B is empty (pre-operational condition) and a 20 foot stockpile of soil excavated from Cell 4B is located 20 feet from the edge of the cell.

DESIGN CRITERION

In accordance with the current state of practice, acceptable seismically-induced permanent deformations are less than 6 to 12 inches for waste mass configurations (Seed and Bonaparte, 1992). To evaluate seismically-induced permanent deformations at Cell 4B, Geosyntec established a maximum seismically-induced deformation of 6 inches as the design criterion.

The maximum average acceleration (k_{max}) of the sliding mass was previously evaluated in the Cell 4 Design Report (UMETCO, 1988) as referenced by MFG, Inc. in a letter to International Uranium Corporation (presently Denison Mines) dated 27 November 2006 (Attachment C). The design report indicates that the maximum acceleration at the site is 0.10 g, representing a 2 percent probability of exceedance within 50 years (approximate return period of 2,500 years). The report states that this design acceleration is suitable for operational conditions at the site. The report also indicates that the maximum credible earthquake (MCE) to produce the 0.10 g ground motion is an MCE of 6.4.

DEFORMATION ANALYSES

Estimating the seismically-induced deformations includes the following steps. The results are summarized in Table 1.

1. Perform pseudostatic slope stability analyses to evaluate the yield acceleration (k_y) resulting in a factor of safety of 1.0 for the critical cross sections. The results of the pseudostatic slope stability evaluation for each cross section

are provided in Table 1. These values were determined using the computer software SLOPE/W (GeoSlope, 2004).

2. Estimate k_{max} (the maximum average acceleration for a potential sliding mass extending to a specified depth y) using the upper bound for observed motions at earth dams reported by Harder (Harder, 1991), through the following two steps:

   a. Estimate value of acceleration at the top of the embankment, $\ddot{u}_{max}$ based on the Harder (1991) curve (included in Attachment D), the acceleration at the crest of the berm, $\ddot{u}_{max}$, is estimated to be 0.35 g;

   b. Calculate k_{max} as 0.35 times $\ddot{u}_{max}$ based on the Makdisi and Seed curve in Attachment A (Figure 7 in Makdisi and Seed, 1978).

3. Calculate the ratio of k_y/k_{max} for each cross section and compute resulting deformations based on the Makdisi and Seed Simplified Method (see Figure 10 in Attachment A). Because the ratio of k_y/k_{max} exceeds 1.0, seismically-induced deformations are estimated to be minimal (less than 1 centimeter or 0.4 inches).

Table 1: Seismic Deformation Analyses Results

Cross Section	MHA (g)	k_y	$\ddot{u}_{max}$	k_{max}	k_y/k_{max}	δ (cm)
A-A’	0.1	0.48g	0.35g	0.12g	1.8	<1
B-B’	0.1	0.47g	0.35g	0.12g	3.3	<1
C-C’	0.1	0.52g	0.35g	0.12g	4.3	<1
D-D’	0.1	0.65g	0.35g	0.12g	5.3	<1

RESULTS AND CONCLUSIONS

Results of the permanent deformation analysis indicate that the expected seismically-induced permanent deformation is expected to be minimal, and significantly less than the design criterion of 6 inches.

Written by:	**M. Lithgow**	Date:	**07/01/08**	Reviewed by:	**G. Corcoran**	Date:	**7/02/08**
Client:	**DMC**	Project:	**White Mesa Mill Facility – Cell 4B**	Project/ Proposal No.:	**SC0349**	Task No.:	**02**

REFERENCES

Geosyntec [2007], "Cell 4B Design Report, White Mesa Mill, Blanding, Utah," submitted December 2007.

Geo-Slope, SLOPE/W, 2007. Slope Stability Modeling Software.

Harder, L.F., Jr. [1991], "Performance of Earth Dams During the Loma Prieta Earthquake," Proc. Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri, Rolla, pp. 11-15.

Makdisi and Seed [1978], "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformation," *Journal of the Geotechnical Engineering Division,* ASCE, Vol 104, No. GT7, pp 849-867.

MFG, Inc. [2006], "White Mesa Uranium Facility, Cell 4 Seismic Study, Blanding, Utah," letter to International Uranium (USA) Corporation, dated 27 November 2006.

Seed, H.B., and Bonaparte, R., [1992], "Seismic Analysis and Design of Lined Waste Fills: Current Practice," Proceedings of ASCE Specialty Conference on Stability and Performance of Slopes and Embankments – II, pp. 1521 – 1545.

JOURNAL OF THE
GEOTECHNICAL
ENGINEERING DIVISION

Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations

1973

By Faiz I. Makdisi,[1] A. M. ASCE and H. Bolton Seed,[2] F. ASCE

Introduction

In the past decade major advances have been achieved in analyzing the stability of dams and embankments during earthquake loading. Newmark (13) and Seed (18) proposed methods of analysis for predicting the permanent displacements of dams subjected to earthquake shaking and suggested this as a criterion of performance as opposed to the concept of a factor of safety based on limit equilibrium principles. Seed and Martin (26) used the shear beam analysis to study the dynamic response of embankments to seismic loads and presented a rational method for the calculation of dynamic seismic coefficients for earth dams. Ambraseys and Sarma (1) adopted the same procedure to study the response of embankments to a variety of earthquake motions.

Later the finite element method was introduced to study the two-dimensional response of embankments (5,7) and the equivalent linear method (21) was used successfully to represent the strain-dependent nonlinear behavior of soils. In addition the nature of the behavior of soils during cyclic loading has been the subject of extensive research (10,20,23,29). Both the improvement in the analytical tools to study the response of embankments and the knowledge of material behavior during cyclic loading led to the development of a more rational approach to the study of stability of embankments during seismic loading. Such an approach was used successfully to analyze the Sheffield Dam failure during the 1925 Santa Barbara earthquake (24) and the behavior of the San Fernando Dams during the 1971 earthquake (25). This method has since been used extensively in the design and analysis of many large dams in the State of California and elsewhere.

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From the study of the performance of embankments during strong earthquakes, two distinct types of behavior may be discerned: (1) That associated with loose to medium dense sandy embankments, susceptible to rapid increases in pore pressure due to cyclic loading resulting in the development of pore pressures equal to the overburden pressure in large portions of the embankment, associated reductions in shear strength, and potentially large movements leading to almost complete failure; and (2) the behavior associated with compacted cohesive clays, dry sands, and some dense sands; here the potential for buildup of pore pressures is much less than that associated with loose to medium dense sands, the resulting cyclic strains are usually quite small, and the material retains most of its static undrained shearing resistance so that the resulting post-earthquake behavior is a limited permanent deformation of the embankment.

The dynamic analysis procedure proposed by Seed, et al. (25) has been used to predict adequately both types of embankment behavior using the "Strain Potential" concept. Procedures for integrating strain potentials to obtain the overall deformation of an embankment have been proposed by Seed, et al. (25), Lee (9), and Serff, et al. (27).

The dynamic analysis approach has been recommended by the Committee on Earthquakes of the International Commission on Large Dams (3): "high embankment dams whose failure may cause loss-of-life or major damage should be designed by the conventional method at first, followed by a dynamic analysis in order to investigate any deficiencies which may exist in the pseudo-statical design of the dam." For low dams in remote areas the Committee recommended the use of conventional pseudostatic methods using a constant horizontal seismic coefficient selected on the basis of the seismicity of the area. However, the inadequacy of the pseudostatic approach to predict the behavior of embankments during earthquakes has been clearly recognized and demonstrated (19,24,25,26, 28). Furthermore in the same report (3) the Commission refers to the conventional method as follows: "There is a need for early revision of the conventional method since the results of dynamic analyses, model tests and observations of existing dams show that the horizontal acceleration due to earthquake forces varies throughout the height of the dam . . . in several instances, this method predicts a safe condition for dams which are known to have had major slides."

It is this need for a simple yet rational approach to the seismic design of small embankments that prompted the development of the simplified procedure described herein.

This approximate method uses the concept originally proposed by Newmark (13) for calculating permanent deformations but it is based on an evaluation of the dynamic response of the embankment as proposed by Seed and Martin (26) rather than rigid body behavior. It assumes that failure occurs on a well-defined slip surface and that the material behaves elastically at stress levels below failure but develops a perfectly plastic behavior above yield. The method involves the following steps:

1. A yield acceleration, i.e., an acceleration at which a potential sliding surface would develop a factor of safety of unity is determined. Values of yield acceleration are a function of the embankment geometry, the undrained strength of the material (or the reduced strength due to shaking), and the location of the potential sliding mass.

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2. Earthquake induced accelerations in the embankment are determined using dynamic response analyses. Finite element procedures using strain-dependent soil properties can be used for calculating time histories of acceleration, or simpler one-dimensional techniques might be used for the same purpose. From these analyses, time histories of average accelerations for various potential sliding masses can be determined.

3. For a given potential sliding mass, when the induced acceleration exceeds the calculated yield acceleration, movements are assumed to occur along the direction of the failure plane and the magnitude of the displacement is evaluated by a simple double integration procedure.

The method has been applied to dams with heights in the range of 100 ft–200 ft (30 m–60 m), and constructed of compacted cohesive soils or very dense cohesionless soils, but may be applicable to higher embankments. A similar approach has been proposed by Sarma (16) using the assumption of a rigid block on an inclined plane rather than a deformable earth structure that responds with differential motions to the imposed base excitation.

In the following sections the steps involved in the analyses will be described in detail and design curves prepared on the basis of analyzed cases will be presented, together with an example problem to illustrate the use of the method. Note, however, that the method is an approximate one and involves simplifying assumptions. The design curves are averages based on a limited number of cases analyzed and should be updated as more data become available and more cases are studied.

Determination of Yield Acceleration

The yield acceleration, k_y, is defined as that average acceleration producing a horizontal inertia force on a potential sliding mass so as to produce a factor of safety of unity and thus cause it to experience permanent displacements.

For soils that do not develop large cyclic strains or pore pressures and maintain most of their original strength after earthquake shaking, the value of k_y can be calculated by stability analyses using limiting equilibrium methods. In conventional slope stability analyses the strength of the material is defined as either the maximum deviator stress in an undrained test, or the stress level that would cause a certain allowable axial strain, say 10%, in a test specimen. However, the behavior of the material under cyclic loading conditions is different than that under static conditions. Due to the transient nature of the earthquake loading, an embankment may be subjected to a number of stress pulses at levels equal to or higher than its static failure stress that simply produce some permanent deformation rather than complete failure. Thus the yield strength is defined, for the purpose of this analysis, as that maximum stress level below which the material exhibits a near elastic behavior (when subjected to cyclic stresses of numbers and frequencies similar to those induced by earthquake shaking) and above which the material exhibits permanent plastic deformation of magnitudes dependent on the number and frequency of the pulses applied. Fig. 1 shows the concept of cyclic yield strength. The material in this case has a cyclic yield strength equal to about 90% of its static undrained strength and as shown in Fig. 1(*a*) the application of 100 cycles of stress amounting to 80%

of the undrained strength resulted in essentially an elastic behavior with very little permanent deformation. On the other hand, the application of 10 cycles of stress level equal to 95% of the static undrained strength led to substantial permanent strain as shown in Fig. 1(*b*). On loading the material monotonically to failure after the series of cyclic stress applications, the material was found to retain the original undrained strength. This type of behavior is associated with various types of soils that exhibit small increases in pore pressure during cyclic loading. This would include clayey materials, dry or partially saturated cohesionless soils, or very dense saturated cohesionless materials that will not undergo significant deformations, even under cyclic loading conditions, unless the undrained static strength of the soil is exceeded.

Seed and Chan (20) conducted cyclic tests on samples of undisturbed and compacted silty clays and found that for conditions of no stress reversal and for different values of initial and cyclic stresses, the total stress required to produce large deformations in 10 cycles and 100 cycles ranged between 90%–110% of the undrained static strength.

Sangrey, et al. (15) investigated the effective stress response of clay under repeated loading. They tested undisturbed samples of clay (LL = 28, PI = 10) and found that the cyclic yield strength of this material was of the order of 60% of its static undrained strength.

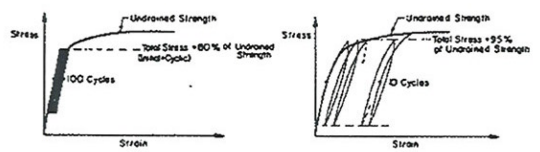

FIG. 1.—Determination of Dynamic Yield Strength

Rahman (14) performed similar tests on remolded samples of a brittle silty clay (LL = 91, PI = 49) and found that the cyclic yield strength was a function of the initial effective confining pressure. For practical ranges of effective confining pressures the cyclic yield strength for this material ranged between 80%–95% of its static undrained strength. At cyclic stress levels below the yield strength, in all cases, the material reached equilibrium and assumed an elastic behavior at strain levels less than 2% irrespective of the number of stress cycles applied.

Thiers and Seed (28) performed tests on undisturbed and remolded samples of different clayey materials to determine the reduction in static undrained strength due to cyclic loading. Their results are summarized in Fig. 2 which shows the reduction in undrained strength after cyclic loading as a function of the ratio of the "maximum cyclic strain" to the "static failure strain." These results were obtained from strain controlled cyclic tests; after the application of 200 cycles of a certain strain amplitude, the sample was loaded to failure monotonically at a strain rate of 3%/min. Thus from Fig. 2 it could be argued that if a clay is subjected to 200 cycles of strain with an amplitude less than half its static failure strain, the material may be expected to retain at least 90% of its original static undrained strength.

Andersen (2), on the basis of cyclic simple shear tests on samples of Drammen clay, determined that the reduction in undrained shear strength was found to be less than 25% as long as the cyclic shear strain was less than ±3% even after 1,000 cycles. Some North Sea clays, however, have shown a strength reduction of up to 40% for the same level of cyclic loading.

On the basis of the experimental data reported previously and for values

TABLE 1.—Maximum Cyclic Shear Strains Calculated from Dynamic Finite Element Response Analyses

Magnitude (1)	Embankment height, in feet (2)	Slope, $H:V$ (3)	Maximum base acceleration, g (4)	Maximum shear strain, as a percentage (5)
6-1/2 (Caltech record)	75	2:1	0.5	0.2–0.4
6-1/2 (Caltech record)	150	2:1	0.2	0.1–0.15
6-1/2 (Caltech record)	150	2:1	0.5	0.2–0.3
6-1/2 (Lake Hughes record)	150	2:1	0.2	0.1–0.15
6-1/2 (Caltech record)	150	2-1/2:1	0.5	0.2–0.3
7-1/2 (Taft record)	150	2:1	0.5	0.2–0.5
7-1/2 (Taft record)	150	2:1	0.2	0.1–0.2
8-1/4 (S-I record)	150	2:1	0.75	0.4–1.0
8-1/4 (S-I record)	135	—	0.4	0.2–0.5

Note: 1 ft = 0.305 m.

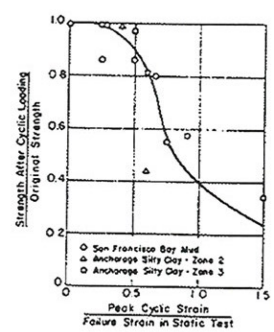

FIG. 2.—Reduction in Static Undrained Strength Due to Cyclic Loading (29)

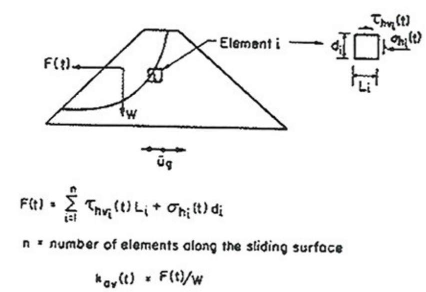

FIG. 3.—Calculation of Average Acceleration from Finite Element Response Analysis

of cyclic shear strains calculated from earthquake response analyses, the value of cyclic yield strength for a clayey material can be estimated. In most cases this value would appear to be 80% or more of the static undrained strength. This value in turn may be used in an appropriate method of stability analysis to calculate the corresponding yield acceleration.

Finite element response analyses (as will be described later) have been carried out to calculate time histories of crest acceleration and average acceleration

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for various potential sliding masses. The method of analysis employs the equivalent linear technique with strain-dependent modulus and damping. The ranges of calculated maximum shear strains, for different magnitude earthquakes and different embankment characteristics, are presented in Table 1. It can be seen from Table 1 that the maximum cyclic shear strain induced during the earthquakes ranged between 0.1% for a magnitude 6-1/2 earthquake with a base acceleration of 0.2 *g* and 1% for a magnitude 8-1/4 earthquake with a base acceleration of 0.75 *g*. For the compacted clayey material encountered in dam embankments "static failure strain" values usually range between 3%–10%, depending on whether the material was compacted on the dry or wet side of the optimum moisture content. Thus in both instances the ratio of the "cyclic strain" to "static failure strain" is less than 0.5.

It seems reasonable, therefore, to assume that for these compacted cohesive soils, very little reduction in strength may be expected as a result of strong earthquake loading of the magnitude described previously.

Once the cyclic yield strength is defined, the calculation of the yield acceleration can be achieved by using one of the available methods of stability analysis. In the present study the ordinary method of slices has been used to calculate the yield acceleration for circular slip surfaces using a pseudostatic analysis. As an alternative one of the writers (18) has suggested a method of combining both effective and total stress approaches, where the shear strength on the failure plane during the earthquake is considered to be a function of the initial effective normal stress on that same plane before the earthquake. This method is applicable to noncircular slip surfaces and the horizontal inertia force resulting in a factor of safety of unity can readily be calculated.

Having determined the yield acceleration for a certain location of the slip surface, the next step in the analysis is to determine the time history of earthquake-induced average accelerations for that particular sliding mass. This will be treated in the following section.

Determination of Earthquake Induced Acceleration

In order for the permanent deformations to be calculated for a particular slip surface, the time history of earthquake induced average accelerations must first be determined.

Two-dimensional finite element procedures using equivalent linear strain-dependent properties are available (6) and have been shown to provide response values in good agreement with measured values (8) and with closed-form one-dimensional wave propagation solutions (17).

For most of the case studies of embankments used in the present analysis, the response calculation was performed using the finite element computer program QUAD-4 (6) with strain-dependent modulus and damping. The program uses the Rayleigh damping approach and allows for variable damping to be used in different elements.

To calculate the time history of average acceleration for a specified sliding mass, the method described by Chopra (4) was adopted in the present study. The finite element calculation provides time histories of stresses for every element in the embankment. As shown in Fig. 3, at each time step the forces acting along the boundary of the sliding mass are calculated from the corresponding

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normal and shear stresses of the finite elements along that boundary. The resultant of these forces divided by the weight of the sliding mass would give the average acceleration, $k_{av}(t)$, acting on the sliding mass at that instant in time. The process is repeated for every time step to calculate the entire time history of average acceleration.

For a 150-ft (46-m) high dam subjected to 30 sec of the Taft earthquake record scaled to produce a maximum base acceleration of 0.2 *g*, the variation

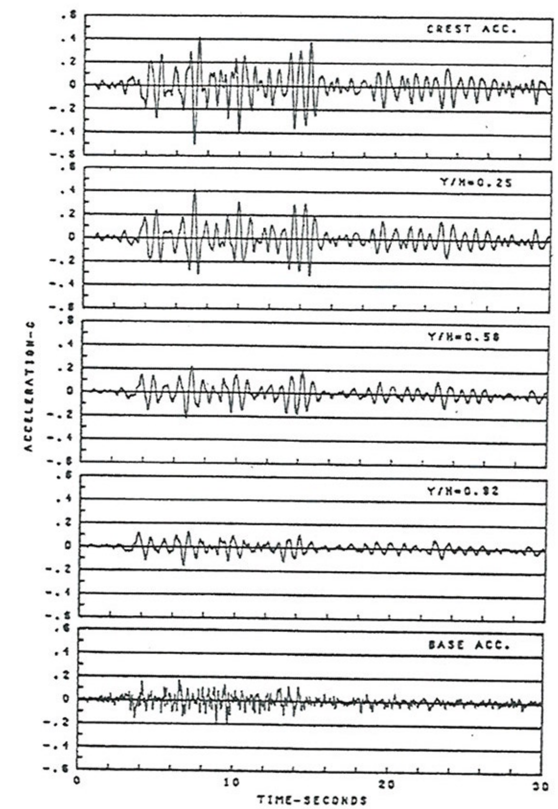

FIG. 4.—Time Histories of Average Acceleration for Various Depths of Potential Sliding Mass

of the time history of k_{av} with the depth of the sliding mass within the embankment, together with the time history of crest accelerations, is shown in Fig. 4.

Comparing the time history of crest acceleration with that of the average acceleration for different depths of the potential sliding mass, the similarity in the frequency content is readily apparent (it generally reflects the first natural period of the embankment), while the amplitudes are shown to decrease as the depth of the sliding mass increases towards the base of the embankment. The maximum crest acceleration is designated by $\ddot{u}_{max}$, and k_{max} is the maximum average acceleration for a potential sliding mass extending to a specified depth, y.

It would be desirable to establish a relationship showing the variation of the maximum acceleration ratio, $k_{max}/\ddot{u}_{max}$, with depth for a range of embankments and earthquake loading conditions. It would then be sufficient, for design purposes, to estimate the maximum crest acceleration in a given embankment due to a specified earthquake and use this relationship to determine the maximum average acceleration for any depth of the potential sliding mass. A simplified procedure to estimate the maximum crest acceleration and the natural period

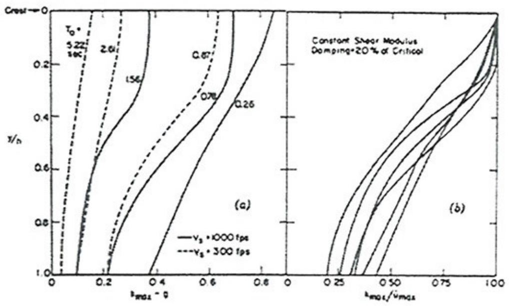

FIG. 5.—El Centro Record (12): (*a*) Variation of Maximum Average Acceleration with Depth of Sliding; (*b*) Variation of Ratio of Average Acceleration to Maximum Crest Acceleration with Depth of Sliding Surface

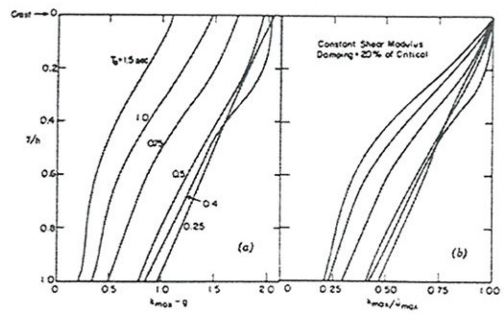

FIG. 6.—Average of Eight Strong Motion Records (1): (*a*) Variation of Maximum Average Acceleration with Depth of Sliding Mass; (*b*) Variation of Ratio of Maximum Average Acceleration to Maximum Crest Acceleration with Depth of Sliding Surface

of an embankment subjected to a given base motion is described in Appendix A of Ref. 11.

To determine the variation of maximum acceleration ratio with depth, use was made of published results of response computations using the one-dimensional shear slice method with visco-elastic material properties (1,26). Martin (12) calculated the response of embankments ranging in height between 100 ft–600 ft (30 m–180 m) and with shear wave velocities between 300 fps–1,000 fps (92 m/s–300 m/s). Using a constant shear modulus and a damping factor of 0.2,

the average acceleration histories for various levels were computed for embankments subjected to ground accelerations recorded in the El Centro earthquake of 1940. The variation of the maximum average acceleration, k_{max}, with depth for these embankments with natural periods ranging between 0.26 sec–5.22 sec is presented in Fig. 5(*a*). The maximum average acceleration in Fig. 5(*a*) is normalized with respect to the maximum crest acceleration and the ratio, $k_{max}/\ddot{u}_{max}$, plotted as a function of the depth of the sliding mass is presented in Fig. 5(*b*).

Ambraseys and Sarma (1) used essentially the same method reported by Seed and Martin (26) and calculated the response of embankments with natural periods ranging between 0.25 sec and 3.0 sec. They presented their results in terms of average response for eight strong motion records. The variation of maximum average acceleration with depth based on the results reported by Ambraseys and Sarma (1) is shown in Fig. 6(*a*) and that for the maximum acceleration ratio, $k_{max}/\ddot{u}_{max}$, is shown in Fig. 6(*b*). A summary of the results obtained

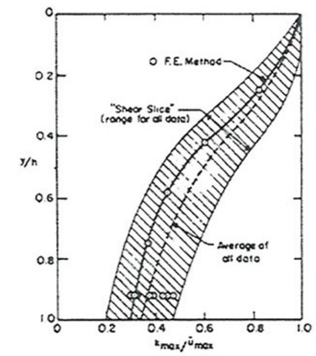

FIG. 7.—Variation of Maximum Acceleration Ratio with Depth of Sliding Mass

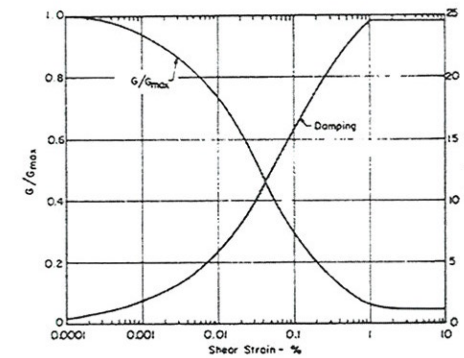

FIG. 8.—Shear Modulus and Damping Characteristics Used in Response Computations

from the different shear slice response calculations mentioned previously is presented in Fig. 7 together with results obtained from finite element calculations made in the present study. As can be seen from Fig. 7 the shape of the curves obtained using the shear slice method and the finite element method are very similar. The dashed curve in Fig. 7 is an average relationship of all data considered. The maximum difference between the envelope of all data and the average relationship ranges from ±10% to ±20% for the upper portion of the embankment and from ±20% to ±30% for the lower portion of the embankment.

Considering the approximate nature of the proposed method of analysis, the use of the average relationship shown in Fig. 7 for determining the maximum average acceleration for a potential sliding mass based on the maximum crest acceleration is considered accurate enough for practical purposes. For design computations where a conservative estimate of the accelerations is desired the upper bound curve shown in Fig. 7 may be used leading to values that are 10%–30% higher than those estimated using the average relationship.

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Calculation of Permanent Deformations

Once the yield acceleration and the time history of average induced acceleration for a potential sliding mass have been determined, the permanent displacements can readily be calculated.

By assuming a direction of the sliding plane and writing the equation of

TABLE 2.—Embankment Characteristics for Magnitude 6-1/2 Earthquake

Case number (1)	Embankment description (2)	Height, in feet (3)	Base acceleration, g (4)	T_o, in seconds (5)[a]	k_{max}, g (6)[b]	Symbol[c] (7)
1	Example slope = 2:1 k_{2max} = 60	150	0.2 (Caltech record)	0.8	(1) 0.31 (2) 0.12	● ■
2	Example slope = 2:1 k_{2max} = 60	150	0.5 (Caltech record)	1.08	(1) 0.4 (2) 0.18	○ □
3	Example slope = 2:1 k_{2max} = 80	150	0.5 (Lake Hughes record)	0.84	(1) 0.33 (2) 0.16	⊙ △
4	Example slope = 2-1/2:1 k_{2max} = 80	150	0.5 (Caltech record)	0.95	(1) 0.49 (2) 0.22	◇ ▽
5	Example slope = 2:1 k_{2max} = 60	75	0.5 (Caltech record)	0.6	(1) 0.86 (2) 0.26	◑ ◩

[a]Calculated first natural period of the embankment.
[b]Maximum value of time history of: (1) Crest acceleration; and (2) average acceleration for sliding mass extending through full height of embankment.
[c]Legend used in Fig. 9(*a*).
Note: 1 ft = 0.305 m.

motion for the sliding mass along such a plane, the displacements that would occur any time the induced acceleration exceeds the yield acceleration may be evaluated by simple numerical integration. For the purposes of the soil types considered in this study, the yield acceleration was assumed to be constant throughout the earthquake.

The direction of motion for a potential sliding mass once yielding occurs

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was assumed to be along a horizontal plane. This mode of deformation is not uncommon for embankments subjected to strong earthquake shaking, and is manifested in many cases in the field by the development of longitudinal cracks along the crest of the embankment. However studies made for other directions of the sliding surface showed that this factor had little effect on the computed displacements (11).

To calculate an order of magnitude of the deformations induced in embankments due to strong shaking a number of cases have been analyzed during the course of this study. The height of embankments considered ranged between 75 ft–150 ft (23 m–46 m) with varying slopes and material properties. The embankments were subjected to ground accelerations representing three different earthquake magnitudes: 6-1/2, 7-1/2, and 8-1/4.

The method used for calculating the response, as mentioned earlier, is a time-step finite element analysis using the equivalent linear method. The strain-dependent modulus and damping relations for the soils used in this study are

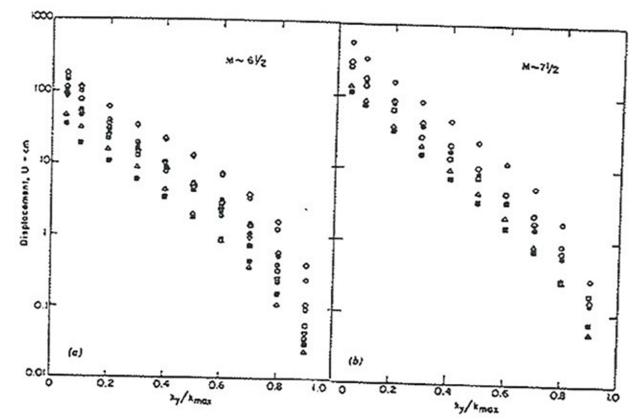

FIG. 9.—Variation of Permanent Displacement with Yield Acceleration: (*a*) Magnitude 6-1/2 Earthquake; (*b*) Magnitude 7-1/2 Earthquake

presented in Fig. 8. The response computation for each base motion was repeated for a number of iterations (mostly 3–4) until strain compatible material properties were obtained. In each case both time histories of crest acceleration and the average acceleration for a potential sliding mass extending through almost the full height of the embankment were calculated, together with the first natural period of the embankment. In one case however, time histories of average acceleration for sliding surfaces at five different levels in the embankment were obtained (see Fig. 4), and the corresponding permanent deformations for each time history were calculated for different values of yield acceleration. It was found that for the same ratio of yield acceleration to maximum average acceleration at each level, the computed deformations varied uniformly between a maximum value obtained using the crest acceleration time history to a minimum value obtained using the time history of average acceleration for a sliding mass extending through the full height of the embankment. Thus it was considered sufficient for the remaining cases to compute the deformations only for these two levels.

Table 2 shows details of the embankments analyzed using ground motions representative of a magnitude 6-1/2 earthquake. The two rock motions used were those recorded at the Cal Tech Seismographic Laboratory (S90W Component) and at Lake Hughes Station No. 12 (N12E) during the 1971 San Fernando earthquake, with maximum accelerations scaled to 0.2 g and 0.5 g. The computed natural periods and maximum values of the acceleration time histories are also presented in Table 2. The computed natural periods ranged between a value of 0.6 sec for the 75-ft (23-m) high embankment to a value of 1.08 sec for the 150-ft (46-m) high embankment. Because of the nonlinear strain-dependent

TABLE 3.—Embankment Characteristics for Magnitude 7-1/2 Earthquake

Case number (1)	Embankment description (2)	Height, in feet (3)	Base acceleration, g (4)	T_o, in seconds (5)[a]	k_{max}, g (6)[b]	Symbol[c] (7)
1	Example slope = 2:1 k_{2max} = 60	150	0.2 (Taft record)	0.86	(1) 0.41 (2) 0.13	● ■
2	Example slope = 2:1 k_{2max} = 60	150	0.5 (Taft record)	1.18	(1) 0.54 (2) 0.21	○ □
3	Example slope = 2-1/2:1 k_{2max} = 80	150	0.2 (Taft record)	0.76	(1) 0.46 (2) 0.15	⊙ △

[a]Calculated first natural period of the embankment.
[b]Maximum value of time history of: (1) Crest acceleration; and (2) average acceleration for sliding mass extending through full height of embankment.
[c]Legend used in Fig. 9(*b*).
Note: 1 ft = 0.305 m.

behavior of the material, the response of the embankment is highly dependent on the amplitude of the base motion. This is clearly demonstrated in the first two cases in Table 2, where the same embankment was subjected to the same ground acceleration history but with different maximum accelerations for each case. In one instance, for a base acceleration of 0.2 g the calculated maximum crest accelerations was 0.3 g with a magnification of 1.5 and a computed natural period of the order of 0.8 sec. In the second case, for a base acceleration of 0.5 g the computed maximum crest acceleration was 0.4 g with an attenuation of 0.8 and a computed natural period of 1.1 sec.

From the time histories of induced acceleration calculated for all the cases

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described in Table 2 and for various ratios of yield acceleration to maximum average acceleration, k_y/k_{max}, the permanent deformations were calculated by numerical double integration. The results are presented in Fig. 9(*a*) which shows that for relatively low values of yield acceleration, k_y/k_{max} of 0.2 for example, the range of computed permanent displacements was of the order of 10 cm–70 cm (4 in.–28 in.). However, for larger values of k_y/k_{max}, say 0.5 or more, the calculated displacements were less than 12 cm (4.8 in.). It should be emphasized that for very low values of yield accelerations (in this case $k_y/k_{max} \leq 0.1$) the basic assumptions used in calculating the response by the finite element

TABLE 4.—Embankment Characteristics of Magnitude 8-1/4 Earthquake

Case number (1)	Embankment description (2)	Height, in feet (3)	Base acceleration, g (4)	T_o, in seconds (5)[a]	k_{max}, g (6)[b]	Symbol[c] (7)
1	Chabot Dam (average properties)	135	0.4 (S-I Synth. record)	0.99	(1) 0.57	○
	Chabot Dam (Lower bound)	135	0.4 (S-I Synth. record)	1.07	(1) 0.53	△
	Chabot Dam (Upper bound)	135	0.4	0.83	(1) 0.68	□
2	Example slope = 2:1 k_{2max} = 60	150	0.75	1.49	(1) 0.74 (2) 0.34	● ■

[a]Calculated first natural period of the embankment.
[b]Maximum value of time history of: (1) Crest acceleration; and (2) average acceleration for sliding mass extending through full height of embankment.
[c]Legend used in Fig. 10(*a*).
Note: 1 ft = 0.305 m.

method, i.e., the equivalent linear behavior and the small strain theory, become invalid. Consequently, the acceleration time histories calculated for such a case do not represent the real field behavior and the calculated displacements based on these time histories may not be realistic.

The procedure described previously was repeated for the case of a magnitude 7-1/2 earthquake. The base acceleration time history used for this analysis was that recorded at Taft during the 1952 Kern County earthquake and scaled to maximum accelerations of 0.2 g and 0.5 g. The details of the three cases analyzed are presented in Table 3 and the results of the computations of the permanent displacements are shown in Fig. 9(*b*). For a ratio of k_y/k_{max} of 0.2 the calculated displacements in this case ranged between 30 cm–200 cm (12 in.–80 in.), and for ratios greater than 0.5 the displacements were less than 25 cm (0.8 ft).

In the cases analyzed for the 8-1/4 magnitude earthquake, an artificial accelerogram proposed by Seed and Idriss (21) was used with maximum base accelerations of 0.4 *g* and 0.75 *g*. Two embankments were analyzed in this case and their calculated natural periods ranged between 0.8 sec and 1.5 sec. Table 4 shows the details of the calculations and in Fig. 10(*a*) the results of the permanent displacement computations are presented. As can be seen from Fig. 10(*a*) the permanent displacements computed for a ratio of k_y/k_{max} of 0.2 ranged between 200 cm–700 cm (80 in.–28 in.), and for ratios higher than 0.5 the values were less than 100 cm (40 in.). Note in this case that values of deformations calculated for a yield ratio less than 0.2 may not be realistic.

An envelope of the results obtained for each of the three earthquake loading

RIGHT SCALE SHIFTED

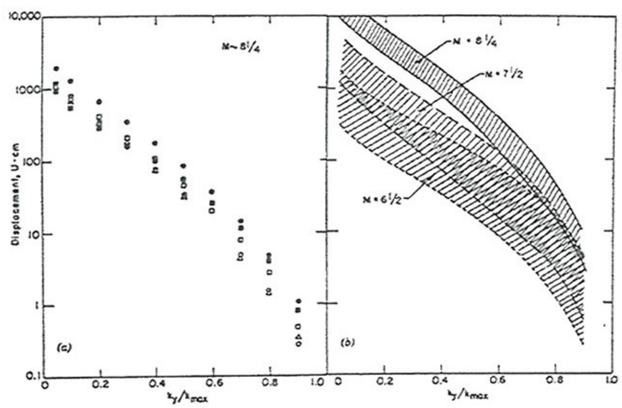

FIG. 10.—Variation of Permanent Displacement with Yield Acceleration: (*a*) Magnitude 8-1/4 Earthquake; (*b*) Summary of All Data

conditions is presented in Fig. 10(*b*) and reveals a large scatter in the computed results reaching, in the case of the magnitude 6-1/2 earthquake, about one order of magnitude.

It can reasonably be expected that for a potential sliding mass with a specified yield acceleration, the magnitude of the permanent deformation induced by a certain earthquake loading is controlled by the following factors: (1) The amplitude of induced average accelerations, which is a function of the base motion, the amplifying characteristics of the embankment, and the location of the sliding mass within the embankment; (2) the frequency content of the average acceleration time history, which is governed by the embankment height and stiffness characteristics, and is usually dominated by the first natural frequency of the embankment; and (3) the duration of significant shaking, which is a function of the magnitude of the specified earthquake.

Thus to reduce the large scatter exhibited in the data in Fig. 10(*b*), the permanent

Attachment A, 89

displacements for each embankment were normalized with respect to its calculated first natural period, T_0, and with respect to the maximum value, k_{max}, of the average acceleration time history used in the computation. The resulting normalized permanent displacements for the three different earthquakes are presented in Fig. 11(*a*). It may be seen that a substantial reduction in the scatter of the data is achieved by this normalization procedure as evidenced by comparing the results in Figs. 10(*b*) and 11(*a*). This shows that for the ranges of embankment heights considered in this study [75 ft–150 ft (50 m–65 m)] the first natural period of the embankment and the maximum value of acceleration time history may be considered as two of the parameters having a major influence on the calculated permanent displacements. Average curves for the normalized permanent displacements based on the results in Fig. 11(*a*) are presented in Fig. 11(*b*). Although some scatter still exists in the results as shown in Fig. 11(*a*), the average curves presented in Fig. 11(*b*) are considered adequate to provide an order of magnitude of the induced permanent displacements for different

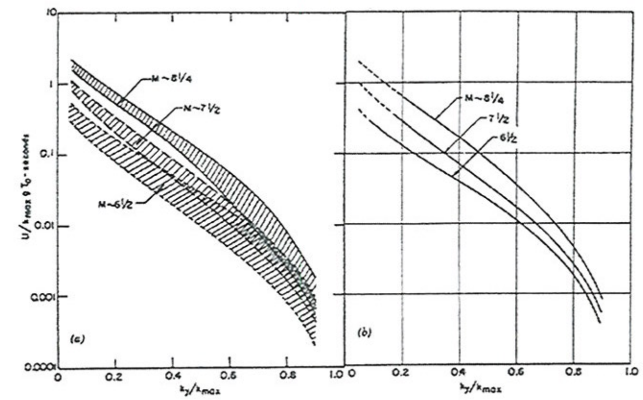

FIG. 11.—Variation of Yield Acceleration with: (*a*) Normalized Permanent Displacement—Summary of All Data; and (*b*) Average Normalized Displacement

magnitude earthquakes. At yield acceleration ratios less than 0.2 the average curves are shown as dashed lines since, as mentioned earlier, the calculated displacements at these low ratios may be unrealistic.

Thus, to calculate the permanent deformation in an embankment constructed of a soil that does not change in strength significantly during an earthquake, it is sufficient to determine its maximum crest acceleration, $\ddot{u}_{max}$, and first natural period, T_0, due to a specified earthquake. Then by the use of the relationship presented in Fig. 7, the maximum value of average acceleration history, k_{max}, for any level of the specified sliding mass may be determined. Entering the curves in Fig. 11(*b*) with the appropriate values of k_{max} and T_0, the permanent displacements can be determined for any value of yield acceleration associated with that particular sliding surface.

It has been assumed earlier in this paper that in the majority of embankments, permanent deformations usually occur due to slip of a sliding mass on a horizontal failure plane. For those few instances where sliding might occur on an inclined strength of the material and in estimating the maximum accelerations in the embankment, the calculated deformations for this 135-ft (40-m) clayey embankment ranged between 0.1 ft–1.5 ft (0.3 m–0.46 m). These approximate displacement values are in good accord with the actual performance of the embankment during the earthquake.

Whereas the method described herein provides a rational approach to the design of embankments and offers a significant improvement over the conventional pseudostatic approach, the nature of the approximations involved requires that it be used with caution and good judgment especially in determining the soil characteristics of the embankment to which it may be applied.

For large embankments, for embankments where failure might result in a loss of life or major damage and property loss, or where soil conditions cannot be determined with a significant degree of accuracy to warrant the use of the method, the more rigorous dynamic method of analysis described earlier might well provide a more satisfactory alternative for design purposes.

Acknowledgment

The study described in this paper was conducted under the sponsorship of the National Science Foundation (Grant ENV 75-21875). The support of the National Science Foundation is gratefully acknowledged.

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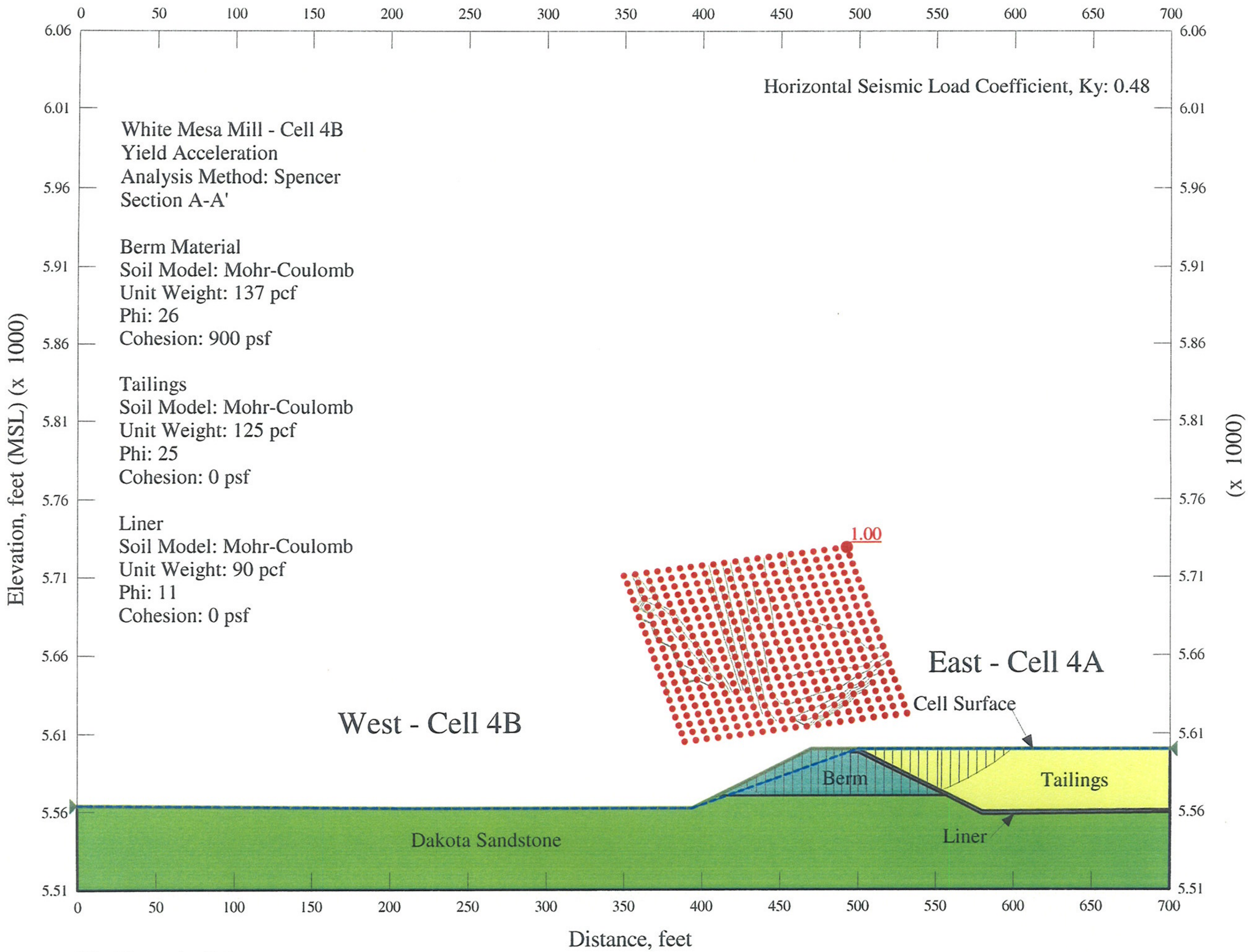

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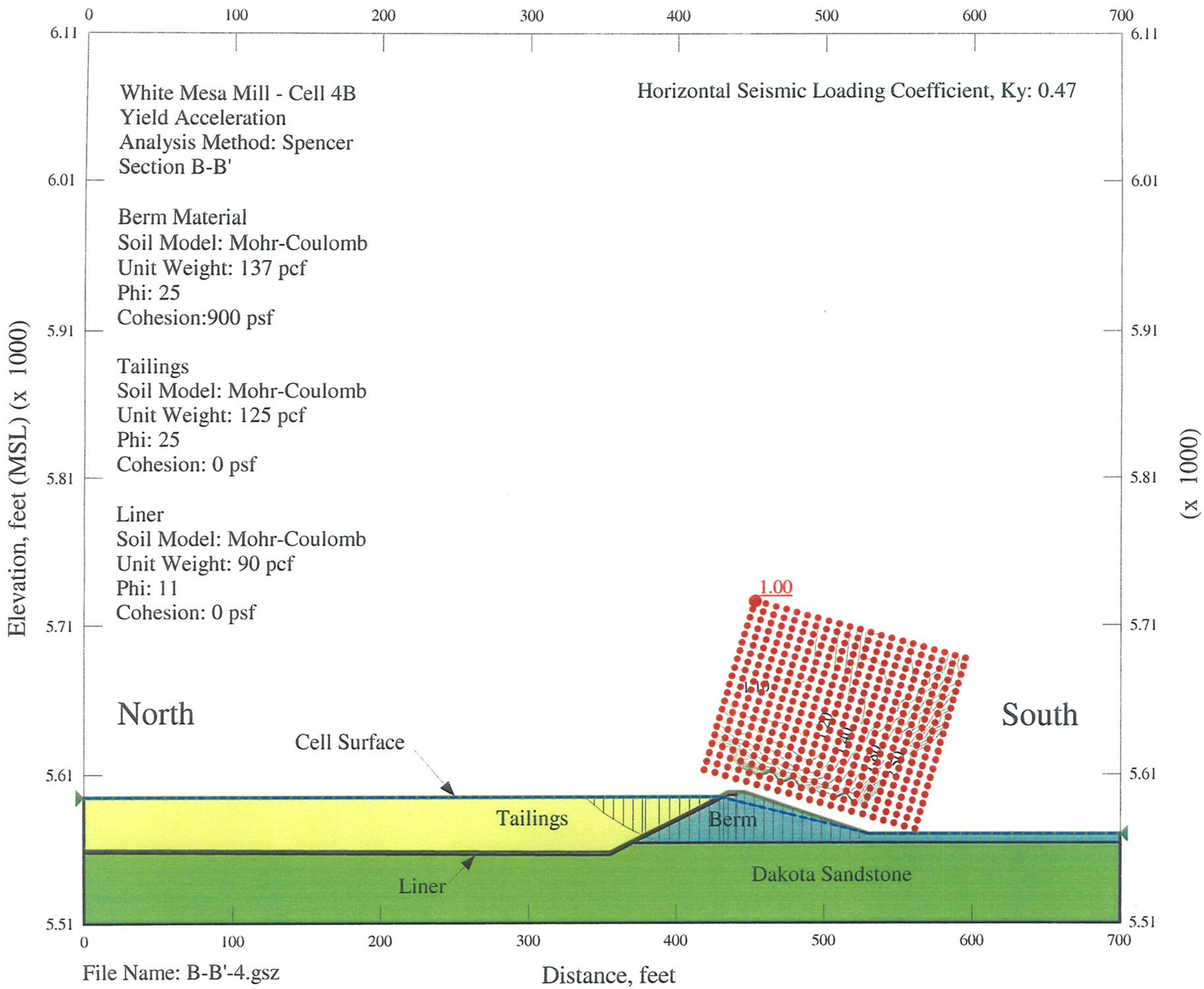
Attachment B, 1/4



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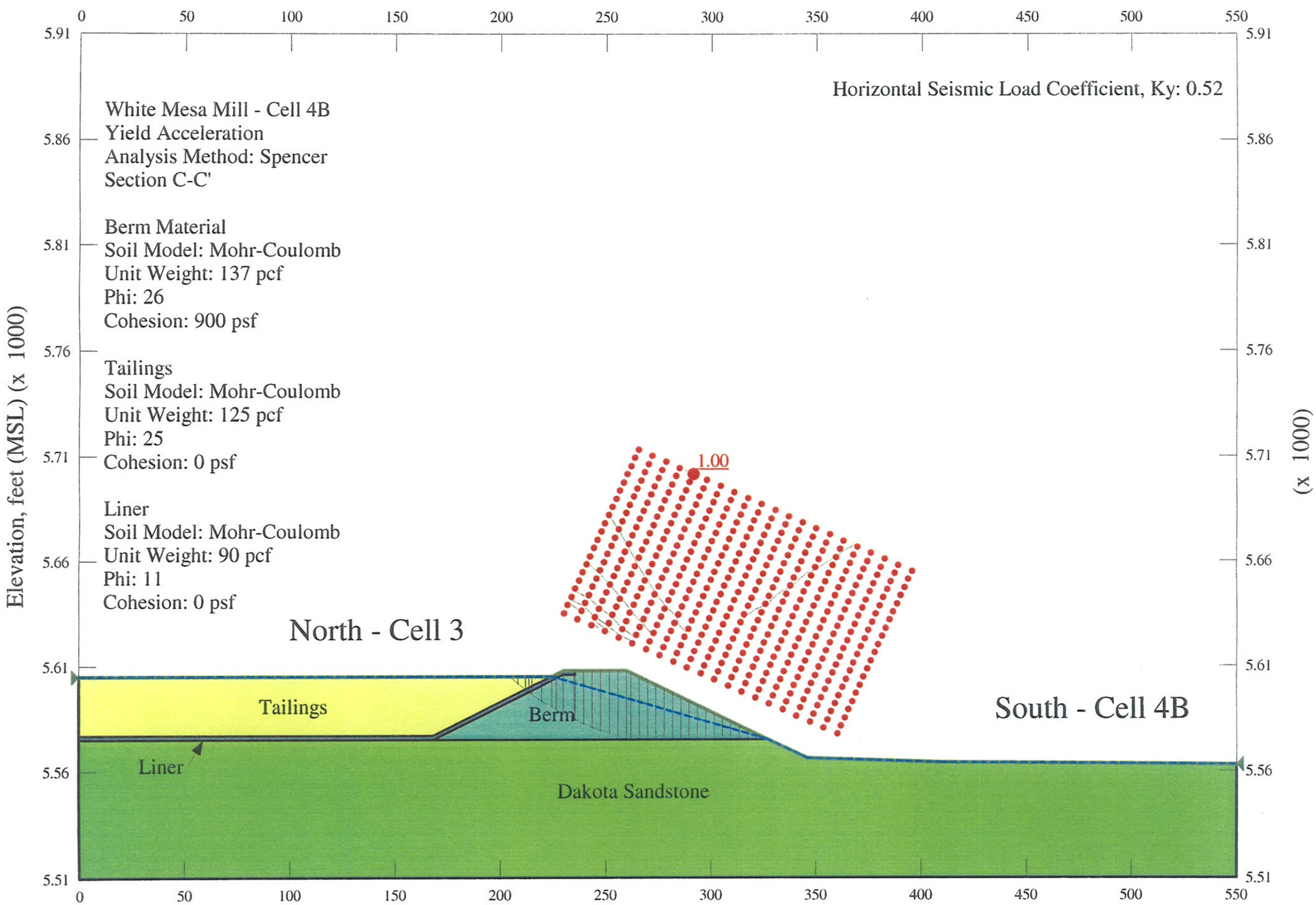

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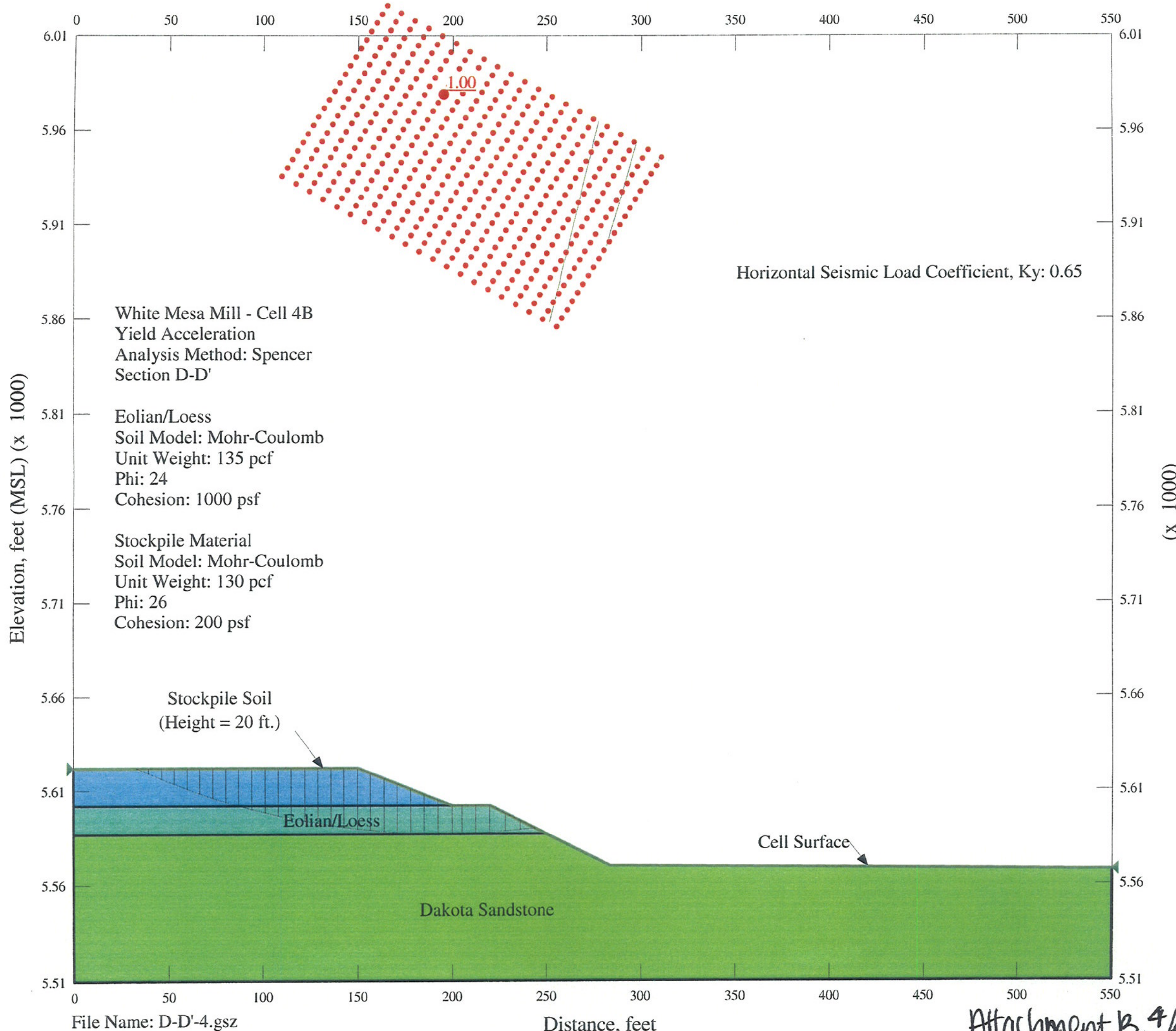

Attachment B, 4/4

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November 27, 2006

MFG Project No. 181413x.102

Mr. Harold R. Roberts
International Uranium (USA) Corporation
1050 Seventeenth Street, Suite 950
Denver, CO 80265

Subject: **White Mesa Uranium Facility**
Cell 4 Seismic Study
Blanding, Utah

Dear Mr. Roberts:

This document has been prepared to examine the seismicity of the White Mesa site and to recommend a design peak ground acceleration (PGA) to be incorporated in the Cell 4A design. This letter addresses concerns brought forth in comments by Utah Department of Environmental Quality (UDEQ) as documented in Interrogatory IUC R313-24-4-05/05: Dike Integrity.

Comments in Interrogatory IUC R313-24-4-05/05

Comments from UDEQ state that the seismic loading used (0.10 g) for stability analysis of the Cell 4A slopes is based on an outdated seismic analysis presented in the 1988 Cell 4 Design Report (UMETCO), and that updated seismic hazard analysis should be performed. As stated in the Interrogatory 05, it is not thought that there is any new information on active faults that would impact the hazard at White Mesa. However, UDEQ requested ground motion attenuation relationships be updated to reflect current evaluation methods.

Original Design Basis for Cell 4

This original design report for Cell 4 (UMETCO, 1988), characterized the geologic conditions at the site. Section 1.3.4 identified potential earthquake hazards to the project. The specified hazards include minor random earthquakes not associated with a known seismic structure, and an unnamed fault located 57 km north of the project site (north of Monticello), with a fault length well defined for 3 km, and possibly as long as 11 km. The fault is considered a suspected Quaternary fault, but does not have strong evidence for Quaternary movement. Estimates of the maximum credible earthquake (MCE) associated with this fault were estimated to have a magnitude of 6.4 based on relationships developed by Slemmons in 1977. Ground motions at the project site were estimated using attenuation curves established in 1982 by Seed and Idriss. Peak horizontal accelerations at the site from the fault were estimated to be 0.07 g.

Attachment C, 1/4

Updated attenuation relationships
A search of the Quaternary Fault and Fold Database (USGS 2006) lists Shay graben faults as a Class B (suspected) Quaternary fault. No other faults within 50 km of the site are included in the database. Shay graben faults were included in the Lawrence Livermore National Laboratory (LLNL) report. Other faults considered as possible seismic sources include the unnamed fault north of Monticello that was the design basis of the design accelerations in the 1988 report.

Many attenuation relationships have been developed within the last ten years and are currently being used to estimate ground motions. Three relationships are used in this report to estimate the peak ground motion at the White Mesa site. Abrahamson and Silva (1997) is a well accepted relationship used for shallow crustal earthquakes in Western North America. In addition, Spudich et al. (1999) is used because it has been specifically developed for extensional tectonic regimes, such as those encountered in the area of the site. Campbell and Bozorgnia (2003), is also examined as a current, applicable model, which accounts for normal faulting. In all cases, mean values plus one standard deviation are reported. A comparison of the three methods can be found in Table 1.

Design Peak Ground Acceleration for Cell 4
The above discussion is based on the PGA associated with MCE predicted for a known tectonic feature, and as such, cannot be correlated to a specific return period. 10 CFR 100 Appendix A and 10 CFR 40 Appendix A of Nuclear Regulatory Commission (NRC) regulations are interpreted to apply to long-term, reclaimed impoundments. A distinction should be made between seismic conditions that apply to operational conditions versus long-term conditions. Disposal areas are required to demonstrate closure performance that provides control of radiological hazards to be effective for one thousand years, to the extent reasonably achievable, and, in any case, for at least 200 years. However, this standard should not apply to the operational time-period of the disposal cell. In 2002, the USGS updated the National Seismic Hazard Maps (NSHM), which show peak ground and spectral accelerations at 2 percent and 10 percent probability of exceedance in 50 years. From these maps, the PGA for the White Mesa site is shown to be 0.090 g with a 2 percent probability of exceedance in 50 years. The probability of exceedance can be represented by the following equation:

$$PE = 1 - e^{-(n/T)}$$

Where PE = probability of exceedance, n = time period, in years, and T = return period, in years.

It can be shown that the return period associated with a PGA of 0.090 g is equivalent to 2,475 years, and if the life of the project is conservatively taken to be 100 years, the probability of exceedance of 0.090 g is approximately 4 percent. Therefore, the PGA taken from the USGS maps is an appropriate design acceleration to use for operational conditions of the disposal cell.

Conclusions
The seismic loading of 0.1 g used in analysis of the Cell 4A dikes exceeds the PGA associated with a 2 percent probability of exceedance within 50 years, and is appropriate for the operational life of the disposal cell. At the time when design of closure is implemented, design PGA based on the MCE associated with known or suspected Quaternary features and the background seismicity of the area should be incorporated into the design long-term seismic loading.

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If we can be of further assistance, please do not hesitate to contact the undersigned.

Sincerely,

TETRA TECH COMPANY
MFG, INC.

Roslyn Stern

Roslyn Stern
Senior Staff Geotechnical Engineer

Reviewed by:

Thomas A. Chapel, CPG, PE
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
No. 6232722-2202
THOMAS A. CHAPEL
11-27-06
STATE OF UTAH

cc: Tetra Tech EM
Ms. JoAnn Tischler

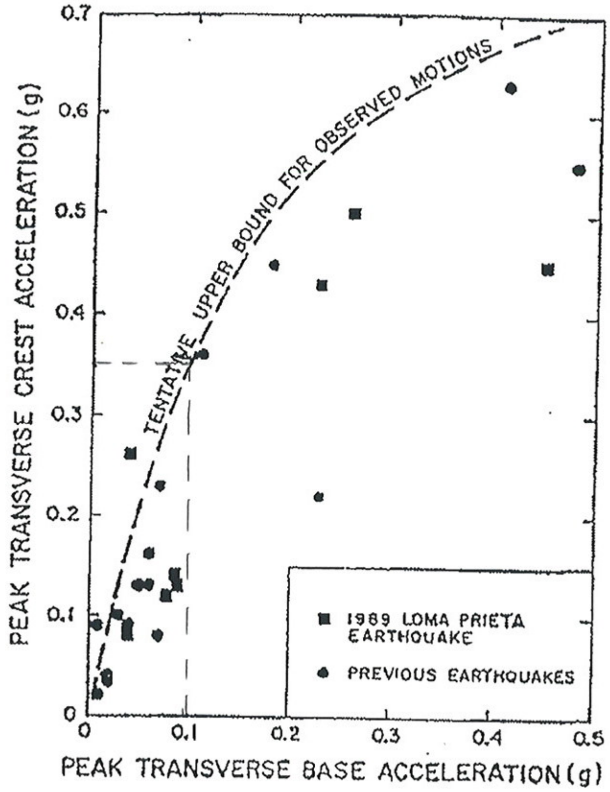
Attachment(s)

Table 1: Peak Ground Accelerations – White Mesa

Name	Fault Length (km)	Fault Type[1]	Site Class[2]	Distance from site (km)	MCE (Wells and Coppersmith, 1994)	PGA Mean plus 1 SD (Spudich et al., 1999)	PGA Mean plus 1 SD (Abrahamson and Silva, 1997)	PGA Mean plus 1 SD, Campbell-Bozorgnia 2003	PGA Mean plus 1 SD average
unnamed fault north of Monticello, defined length	3.0	N	R	57.4	5.49	0.034	0.027	0.037	0.032
unnamed fault north of Monticello, possible total length	11.0	N	R	57.4	6.23	0.050	0.059	0.055	0.055
unnamed fault north of Monticello, 1/2 total rupture	5.5	N	R	57.4	5.84	0.041	0.039	0.044	0.041
Shay graben faults (Class B)	40.0	N	R	44.6	6.97	0.096	0.116	0.113	0.108

[1]Fault Type: N = Normal
[2]Site Class: R =Rock or shallow soils

Att C, 4/4



Source: Harder [1991]

GEOSYNTEC CONSULTANTS

PEAK TRANSVERSE CREST ACCELERATION VERSUS PEAK TRANSVERSE BASE ACCELERATION	FIGURE NO.
	PROJECT NO. SC0349
	DATE: April 2008

Attachment D

1/1

EXHIBIT C

REVISED PIPE STRENGTH ANALYSIS CALCULATION PACKAGE

Geosyntec consultants

COMPUTATION COVER SHEET

Client: **DMC** Project: **White Mesa Mill – Cell 4B** Project/ Proposal No.: **SC0349**
Task No. **03**

Title of Computations **REVISED PIPE STRENGTH CALCULATIONS**

Computations by:	Signature		07/01/08 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		7/27/08 Date
	Printed Name	Gregory Corcoran, P.E.	
	Title	Principal Engineer	
Computations Checked by:	Signature		7/3/08 Date
	Printed Name	Rebecca Flynn	
	Title	Senior Staff Engineer	
Computations backchecked by: (originator)	Signature		07/01/08 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Approved by: (pm or designate)	Signature		7/27/08 Date
	Printed Name	Gregory Corcoran, P.E.	
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	M. Lithgow	Date:	06/25/08	Reviewed by:	G. Corcoran	Date:	
Client:	DMC	Project:	WMM – Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

REVISED PIPE STRENGTH CALCULATIONS
WHITE MESA MILL
BLANDING, UTAH

OBJECTIVE

The project involves placement of a double liner system for the base of Cell 4B at the White Mesa Mill in Blanding, Utah. The proposed liner system is shown in Attachment A. A 4-in diameter schedule 40 perforated Poly Vinyl Chloride (PVC) pipe will be buried under a maximum of 40 ft of tailing deposits. This calculation will evaluate if the pipe will remain structurally intact with the maximum load placed above the buried pipe.

SUMMARY OF ANALYSIS

The maximum possible load on the buried pipe is evaluated to be 41.7 pounds per square inch (psi). Assuming a maximum allowable ring deflection of 7.5 percent, a schedule 40 perforated PVC pipe diameter of 4-in will remain structurally intact.

SITE CONDITIONS

The construction components pertinent to this analysis are, from top to bottom:

- Maximum of 40 ft of silt-like deposits with assumed maximum wet unit weight of 125 pounds per cubic foot (pcf);
- 4-in diameter schedule 40 PVC pipe, embedded in coarse aggregate for slimes drain;
- 60-mil HDPE geomembrane;
- Geonet and 4-in diameter schedule 40 PVC pipe, embedded in coarse aggregate for leak detection system (LDS);
- 60-mil HDPE geomembrane; and
- Geosynthetic clay liner (GCL).

A cross-section of the site conditions is presented as Attachment A.

Written by:	M. Lithgow	Date:	06/25/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	WMM – Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

ANALYSIS

In the analysis herein, the allowable ring deflection and the factor of safety values against pipe wall crushing and buckling will be evaluated.

Ring Deflection

Ring deflection is the change in the vertical diameter of the pipe as the pipe/bedding aggregate system deforms under the external vertical pressure. Ring deflection can be evaluated using Spangler's Modified Iowa Formula, as follows:

$$\frac{\Delta}{D} = \frac{D_L KP + KW'}{\left[\frac{2E}{3(DR-1)^3}\right] + 0.061E'}$$ (Attachment. B, 6/8)

where:

Δ	Pipe deflection or change in diameter, in.
D	Pipe diameter, in.
P	Prism soil load, psi
K	Bedding constant
W′	Live load, psi
DR	Standard dimension ratio (SDR)
E	Modulus of elasticity of pipe, psi
E′	Modulus of soil reaction, psi
D_L	Deflection lag factor

Evaluate Variables

Δ/D	The allowable ring deflection for PVC pipe is 7.5 percent based on a factor of safety of 4	(Attachment C, 2/2)
P	Prism soil load = 125 pcf × 40 ft = 5,000 psf = 34.7 psi	

Written by:	**M. Lithgow**	Date:	**06/25/08**	Reviewed by:	**G. Corcoran**	Date:	7/27/08
Client:	**DMC**	Project:	**WMM – Cell 4B**	Project/ Proposal No.:	**SC0349**	Task No.:	**03**

Effect of Perforations

The effects of the perforations in the pipe should be checked to ensure they will not significantly reduce the pipe strength. The frequency of perforations in the pipe will be 2 perforations per every 12 lineal inches of the pipe (Attachment H, 1/1). The perforations are anticipated to be 0.25 inches in diameter. According to EPA, Manual SW-8, "Lining of Waste Impoundment and Disposal Facilities," the cumulative length of perforations (l_p) in the pipe should be determined per foot of pipe (Attachment D, 1/1). This value is determined by:

$$l_p = \left(\frac{length}{perforation}\right) \cdot (perforations) = \left(\frac{0.25\ in}{perforation}\right) \cdot (2 perforations) = 0.50\ in$$

The total vertical stress to be utilized for pipe design calculations should be adjusted according to the following equation:

$$P_T = \left(\frac{12in}{12in - l_p}\right) \cdot \left(P_{T''}\right) = \left(\frac{12in}{12in - 0.75\ in}\right) \cdot \left(34.7 psi\right) = 36.2 psi = 5{,}214\ psf$$

K	Bedding constant = 0.1	(typical value, Attachment B, 5/8)
W′	Live load = 0 (no live loads are expected for the site)	
DR	Standard dimension ratio = $\frac{D_o}{t}$	(Attachment B, 3/8)

where:

D_o	Outside diameter of pipe = 4.500 in.	(Attachment E, 2/2)
t	Minimum pipe wall thickness = 0.237 in.	(Attachment E, 2/2)

so, $DR = \frac{4.500}{0.237} = 19.0$

E	Modulus of elasticity of pipe = 400,000 psi

Written by:	M. Lithgow	Date:	06/25/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	WMM – Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

(for Class 12454-B rigid PVC pipe; Attachment F, 2/2)

E' Modulus of soil reaction = 3,000 psi

(for crushed rock, Attachment B, 5/8)

D_L = 1.0 (Attachment B, 5/8)

Solve for the deflection provides:

$$\frac{\Delta}{D} = \frac{D_L KP + KW'}{\left[\frac{2E}{3(DR-1)^3}\right] + 0.061E'}$$

$$= \frac{1.0(0.1)(36.2) + 0.1(0)}{\left[\frac{2(400{,}000)}{3(19.0-1)^3}\right] + 0.061(3{,}000)} = 1.6\%$$

Since the calculated ring deflection (1.6%) is lower than the maximum allowable ring deflection (7.5%), the schedule 40 PVC pipe with 4-in will be suitable for the anticipated loading conditions.

Wall Crushing

Wall crushing can occur when the stress in the pipe wall, due to external vertical pressure, exceeds the compressive strength of the pipe material. Wall crushing can be calculated using the following equation:

$$T = \frac{P_y D_o}{2}$$ (Attachment B, 8/8)

where:

T Wall thrust, lbs/in.

P_y Vertical pressure, psi

D_o Outside diameter of pipe = 4.500 in (Attachment E, 2/2) and;

$$\sigma_c \quad = \frac{T}{A}$$ (Attachment B, 8/8)

where:

σ_c Compressive stress = 9,600 psi (Attachment G, 1/1)

Cross sectional area of the pipe wall per unit length

$$= \frac{\pi}{4}\left(4.500^2 - \left(4.500 - 2(0.237)\right)^2\right) = 3.174\ \text{in}^2/12\ \text{in} = 0.265\ \text{in}^2/\text{in}$$

Combining Equations and solving for P_y provides:

$$P_y = \frac{2\sigma_c A}{D_o}$$

Substituting the variables into the above equation provides:

$$P_y = \frac{2(9{,}600)(0.265)}{4.500} = 1{,}129\ \text{psi}$$

Comparing the above estimated value to the maximum loading allowed under ring deflection criteria (36.2 psi) provides:

$$FS_{WC} = 1{,}129/36.2$$

$$= 31.2$$

This value is greater than the acceptable factor of safety of 2.

Written by:	M. Lithgow	Date:	06/25/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	WMM – Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

Wall Buckling

Wall buckling, a longitudinal wrinkling in the pipe wall, can occur when the external vertical pressure exceeds the critical buckling pressure of the pipe/bedding aggregate system. Wall buckling can be calculated using the following equation:

$$P_{cr} = \frac{2E}{(DR-1)^3}$$ (Attachment B, 7/8)

where:

P_{cr} Buckling pressure, psi

E Modulus of elasticity = 400,000 psi (Attachment F, 2/2)

DR Standard dimension ratio $= \frac{D_o}{t} = \frac{4.500}{0.237} = 19.0$

Therefore,

$$P_{cr} = \frac{2(400{,}000)}{(19.0-1)^3} = 137 \text{ psi}$$

Comparing the above estimated value to the maximum loading allowed under ring deflection criteria (39.1 psi) provides:

FS_{WC} = 137/36.2

= 3.8

This value is greater than the acceptable factor of safety of 2.

Written by:	M. Lithgow	Date:	06/25/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	WMM – Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

SUMMARY AND CONCLUSIONS

Using the Modified Iowa Formula as outlined in the Uni-Bell Plastic Pipe Association Handbook on PVC Pipe, the maximum load on the buried pipe assumed to be 36.2 psi will only cause a ring deflection of 1.6 percent, which is below the acceptable ring deflection of 7.5 percent. Acceptable factor of safety values against wall crushing and wall buckling were also evaluated using methods outlined in Uni-Bell Plastic Pipe Association Handbook on PVC Pipe. Therefore, schedule 40 PVC pipe with 4-in diameter and 0.25 inch perforations staggered every 12 inches on each side is suitable for this application.

REFERENCES

ASTM D 1784 (1993), "Standard Specification for Rigid Poly (Vinyl Chloride) (PVC) Compounds and Chlorinated Poly (Vinyl Chloride) (CPVC) Compounds" ASTM Annual Method of Standards - Plastics

ASTM D 1785 (1996), "Standard Specification for Poly (Vinyl Chloride) (PVC) Plastic Pipe, Schedules 40, 80, and 120" ASTM Annual Method of Standards - Plastics

ASTM D 3034 (1997), "Standard Specification for Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings" ASTM Annual Method of Standards - Plastics

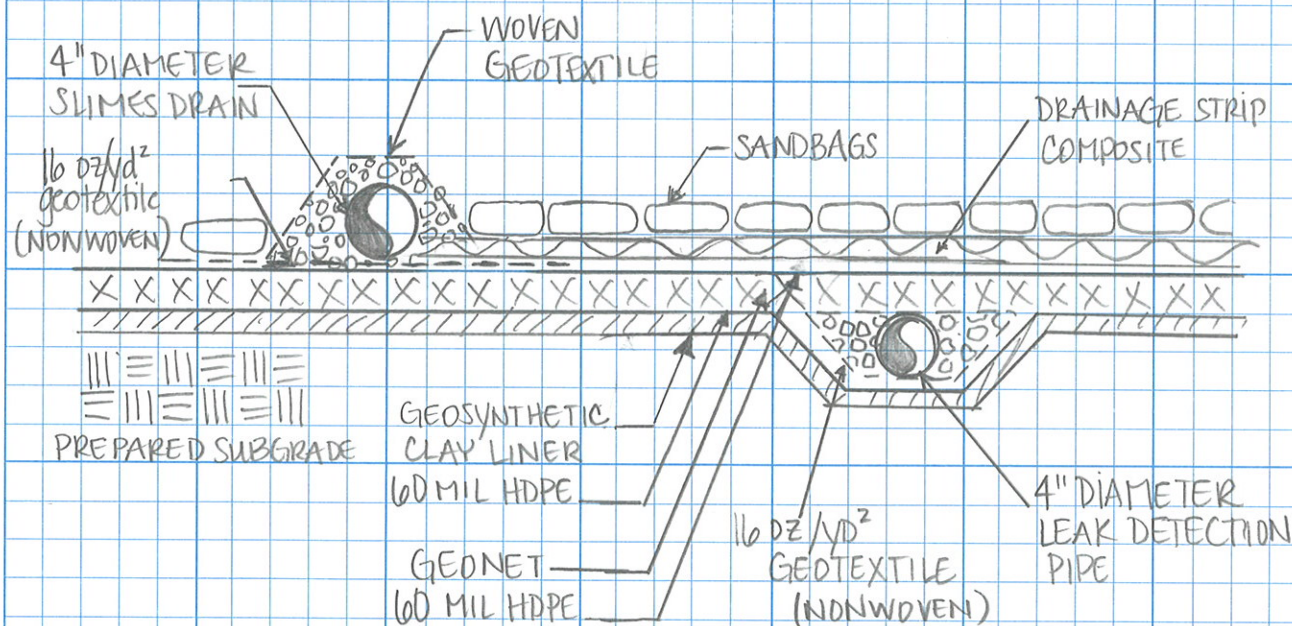
The Uni-Bell Plastic Pipe Association, "Handbook of PVC Pipe, Design and Construction," Dallas, Texas, 214-243-3902

USEPA, "Lining of Waste Impoundment and Disposal Facilities," National Service Center for Environmental Publications P.O. Box 42419 Cincinnati, OH 45242-2419

Geosyntec consultants

Written by: M. Lithgow Date: 06/04/08 (DD MM YY) Reviewed by: GTC Date: 27/7/08 (DD MM YY)

Client: DMC Project: Cell 4B Project/Proposal No. SC0349 Task No.



NOT TO SCALE

(ATTACHMENT A 1/1)

The Uni-Bell PVC Pipe Association

Handbook of PVC Pipe

Design and Construction

Uni-Bell PVC Pipe Association

2655 Villa Creek Drive, Suite 155

Dallas, Texas 75234

$40.00

Attachment B, 1/8

TABLE 6.3 - Continued

Height of Cover (ft)	Soil Unit Weight (lb/ft^3) 100	110	120	125	130
36	25.00	27.50	30.00	31.25	32.50
37	25.69	28.26	30.83	32.12	33.40
38	26.39	29.03	31.67	32.99	34.31
39	27.08	29.79	32.50	33.85	35.21
40	27.78	30.56	33.33	34.72	36.11
41	28.47	31.32	34.17	35.59	37.01
42	29.17	32.08	35.00	36.46	37.92
43	29.86	32.85	35.83	37.33	38.82
44	30.56	33.61	36.67	38.19	39.72
45	31.25	34.38	37.50	39.06	40.63
46	31.94	35.14	38.33	39.93	41.53
47	32.64	35.90	39.17	40.80	42.43
48	33.33	36.67	40.00	41.67	43.33
49	34.03	37.43	40.83	42.53	44.24
50	34.72	38.19	41.67	43.40	45.14

Tables 6.1, 6.2 and 6.3 assume a typical range for H and w. The table limits do not imply application limits.

Live Loads: Underground PVC pipe may also be subjected to live loads from different sources such as highways and railways. Live loads have little effect on pipe performance except at shallow burial depths.

Several methods exist for calculating these live loads. The design approach presented here is taken from the American Water Works Association standard for fiberglass pipe (AWWA C950).

Based on the Boussinesq formula for a point load at the surface of a semi-infinite elastic soil:

$$W_L = \frac{C_L P(1 + I_f)}{12} \quad *$$

Where: W_L = live-load on pipe, in pounds per inch
C_L = live-load coefficient, per foot of effective length
P = wheel load, in pounds
I_f = impact factor, dimensionless ($I_f = 0.766 - 0.133H$; $\underline{0 \le I_f \le 0.50}$) * $1+I_f = \boxed{1.5 \text{ max}}$

Note: ...ndard AASHTO H20 ...

Tables 6.4 and 6.5 give the live load coefficient C_L for a single wheel load and for two passing trucks, respectively. The design appr[...] taken in these tables conservatively represents a wheel load as a point l[...] Analytical expressions for C_L are given below the tables in terms of the diameter or radius and the height of cover.

TABLE 6.4
LIVE-LOAD COEFFICIENTS FOR SINGLE-WHEEL LOA[...]

Pipe Diameter in.	Height of Cover Over Pipe H -- ft: 2	4	6	8	10	12	14	16	
	Live-Load Coefficient C_L								
8	0.056	0.020	0.010	0.006	0.004	0.003	0.002	0.001	0.
10	0.069	0.025	0.012	0.007	0.004	0.003	0.002	0.002	0.
12	0.081	0.029	0.014	0.008	0.005	0.004	0.003	0.002	0.
14	0.091	0.034	0.016	0.009	0.006	0.004	0.003	0.002	0.
16	0.103	0.038	0.018	0.010	0.007	0.005	0.004	0.003	0.
18	0.115	0.042	0.020	0.012	0.008	0.005	0.004	0.003	0.
20	0.124	0.046	0.022	0.013	0.008	0.006	0.004	0.003	0.
24	0.141	0.055	0.026	0.015	0.010	0.007	0.005	0.004	0.
30	0.167	0.066	0.032	0.019	0.012	0.007	0.006	0.005	0.
36	0.183	0.076	0.038	0.022	0.015	0.010	0.008	0.006	0.
42	0.196	0.085	0.044	0.026	0.017	0.012	0.009	0.007	0.
48	0.205	0.094	0.049	0.029	0.019	0.014	0.010	0.008	0.

NOTE 1: An effective length of 3.0 ft of pipe is assumed.
NOTE 2:

$$C_L = \frac{1}{3} - \frac{2}{3\pi}\text{ARCSIN}\left[H\sqrt{\frac{R^2 + H^2 + 1.5^2}{(R^2 + H^2)(H^2 + 1.5^2)}}\right] + \frac{RH\left[\left(\frac{1}{R^2 + H^2} + \frac{1}{H^2 + 1.5^2}\right)\right]}{\pi\sqrt{R^2 + H^2 + 1.5^2}}$$

WHERE: H = earth cover, in feet; R = pipe radius, in feet; ARCSIN must be in radians.

As mentioned previously, the influence of live loads on the performa[...] of PVC pipe is only significant in shallow depths, usually 4 feet (1.2[...] and less for highway loads. This is graphically demonstrated by the gr[...] in Figure 6.7. Both show the total load calculated on a pipe exposed to loads and earth loads for highway and for railway traffi[...]

Attachment 3 (2/8)

xible pipe may be defined as a conduit that will deflect at least two without any sign of structural distress such as injurious cracking. nduit to behave as a flexible pipe when buried, it is required that the nore yielding than the embedment soil surrounding it.

xible pipe derives its soil load carrying capacity from its flexibility. oil load, the pipe tends to deflect, thereby developing passive soil at the sides of the pipe. At the same time, the ring deflection re- e pipe of the major portion of the vertical soil load which is then y the surrounding soil through the mechanism of an arching action pipe. (See Chapter VI.)

effective strength of the pipe-soil system is remarkably high. For , tests at Utah State University indicate that a rigid pipe with a three- ring strength of 3300 lb/ft (48.15 kN/m) buried in Class C bedding with a soil load of 5000 lb/ft (72.95 kN/m). However, under the soil conditions and loading, PVC sewer pipe with a minimum pipe of 46 psi deflects only 5 percent. This deflection is far below that ould cause damage to the PVC pipe wall. Thus, in this example, the e has failed but the flexible pipe has performed successfully.

ourse, in flat plate or three-edge loading, the rigid pipe will support ore than the flexible pipe. This anomaly tends to mislead many e flexible pipe users because they relate low flat plate supporting for flexible pipe to the in-soil load capacity. Flat plate or three-edge is an appropriate measure of load bearing strength for rigid pipes or flexible pipes.

Stiffness: The inherent strength of flexible pipe is called pipe which is measured, according to ASTM D 2412 Standard Test for External Loading Properties of Plastic Pipe by Parallel-Plate , at an arbitrary datum of 5 percent deflection. Pipe stiffness is de-

EQUATION 7.1

$$PS = F/\Delta Y = \frac{EI}{0.149r^3} = \frac{6.71EI}{r^3}$$

l wall pipes Equation 7.1 can be rewritten as:

$$PS = F/\Delta Y = \frac{6.71Et^3}{12r^3} = 0.559E\left[\frac{t}{r}\right]^3$$

Where:
- PS = Pipe Stiffness, lbf/in/in. or psi
- F = Force, lbs./Lin.
- ΔY = Vertical deflection, in.
- E = Modulus of elasticity, psi
- I = Moment of inertia of the wall cross-section per unit length of pipe, $in^4/Lin. = in^3$
- r = Mean radius of pipe, in.
- t = wall thickness, in.

For solid wall PVC pipe with outside diameter controlled dimensions (rather than I.D.) Equation 7.2 can be further simplified:

EQUATION 7.3

$$PS = 4.47\frac{E}{(DR - 1)^3}$$

Where: $DR = \frac{D_o}{t}$

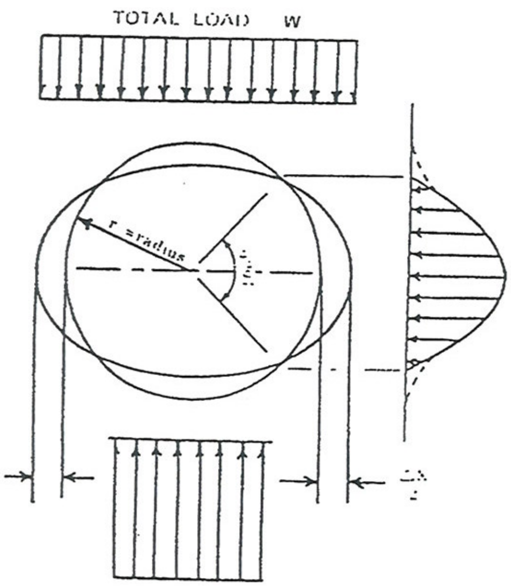
The resulting PS values for various dimension ratios and E values of PVC pipe are as shown in Table 7.1.

In addition to altering the "I" value by changing the DR, alternative shapes can be employed. It is this option of more efficient shapes that has resulted in a variety of profile wall gravity PVC pipe products for sanitary and drain applications. Users are afforded the economy of a higher stiffness than a DR product of the same raw material quantity and strength.

Equation 7.1 shows that the pipe stiffness increases as the moment of inertia of the wall cross section increases. For a solid wall pipe the moment of inertia is equal to $\frac{t^3}{12}$ $in^4/Lin.$, with the center of gravity being at the mid-point of the pipe wall.

Attachment B (3/8)

SIS OF SPANGLER'S DERIVATION OF THE IOWA
)RMULA FOR DEFLECTION OF BURIED PIPES



(EQUATION 7.6)

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061er^4}$$

THE IOWA FORMULA

e	=	$2h/\Delta X$
2r	=	D = Mean Pipe diameter
K	=	Bedding constant
D_L	=	Deflection lag factor
EI	=	Stiffness factor (related to pipe stiffness)

EQUATION 7.9

$$\Delta X = D_L \frac{K W_c r^3}{EI + 0.061er^4}$$

e:

D_L	=	Deflection lag factor
K	=	Bedding constant
W_c	=	Marston's load per unit length of pipe, lb/Lin.
r	=	Mean radius of the pipe, in.
E	=	Modulus of elasticity of the pipe material, psi
I	=	Moment of inertia of the pipe wall per unit length, $in^4/Lin = in^3$
e	=	Modulus of passive resistance of the side fill, $lb/in^2/in$.
ΔX	=	Horizontal deflection or change in diameter, in.

n 7.9 can be used to predict deflections of buried pipe if the three
onstants K, D_L and e are known. The bedding constant, K, ac-

commodates the response of the buried flexible pipe to the opposite and equal reaction to the load force derived from the bedding under the pipe. The bedding constant varies with the width and angle of the bedding achieved in the installation. The bedding angle is shown in Figure 7.4. Table 7.2 contains a list of bedding factors, K, dependent upon the bedding angle. These were determined theoretically by Spangler and published in 1941. As a general rule, a value of K = 0.1 is assumed. *

FIGURE 7.4

BEDDING ANGLE

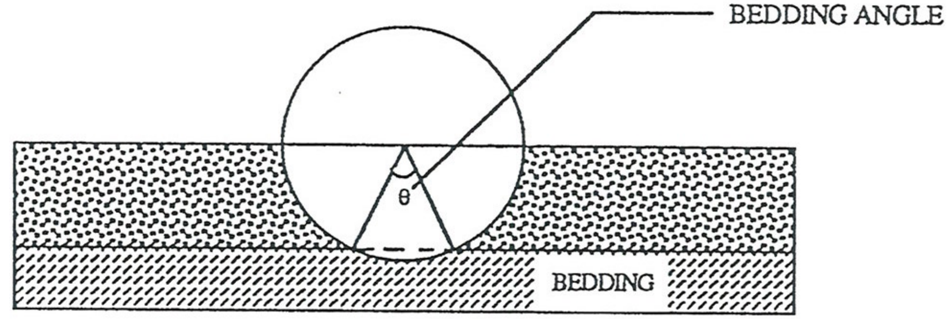


TABLE 7.2

VALUES OF BEDDING CONSTANT, K

BEDDING ANGLE (DEGREES)	K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

In 1955, Reynold K. Watkins, a graduate student of Spangler, was investigating the modulus of passive resistance through model studies and examined the Iowa Formula dimensionally. The analysis determined that e could not possibly be a true property of the soil in that its dimensions are not those of a true modulus. As a result of Watkins' effort, another soil parameter was defined. This was the modulus of soil reaction, E' = er.

Attachment B (4/8)

EQUATION 7.10

$$\Delta X = D_L \frac{KW_c r^3}{EI + 0.061E'r^3}$$

other observations from Watkins' work are of particular note. little point in evaluating E' by a model test and then using the to predict ring deflection; the model gives ring deflection directly. ection may not be the only performance limit.

y research efforts have attempted to measure E' without success. t useful method has involved the measure of deflections for a pipe hich other conditions were known followed by back-calculation the Modified Iowa Formula to determine the correct value of E'. iires assumptions regarding the load, bedding factor and deflection : to be used and has led to a variation in reported values of E'.

attempt to acquire information on values of E' was conducted by K. Howard of the United States Bureau of Reclamation. Howard both laboratory and field data from many sources. Using infor- om over 100 laboratory and field tests, he compiled a table of aver- lues for various soil types and densities (see Table 7.3). He was o this by assuming values of E', K and W_c and then using the Iowa Formula to calculate a theoretical value of deflection. This l deflection was then compared with actual measurements. By as- ie E' values of Table 7.3, a bedding constant K = 0.1 and deflec- actor D_L = 1.0, Howard was able to correlate the theoretical and results to within ± 2 percent deflection when he used the prism

This means that if theoretical deflections using Table 7.3 were ately 5 percent, measured deflection would range between 3 and 7 Although the vast majority of data from this study was taken from teel and reinforced plastic mortar pipe with diameters greater than , it does provide some useful information to guide designers of all ipe including PVC pipe since it helps to give an understanding of ied Iowa Deflection Formula.

AVERAGE VALUES OF MODULUS OF SOIL REACTION, E'
(For Initial Flexible Pipe Deflection)

Soil type-pipe bedding material (Unified Classification System[a]) (1)	E' for Degree of Compaction of Bedding, in pounds per square inch: Dumped (2)	Slight, <85% Proctor, <40% relative density (3)	Moderate, 85%-95% Proctor, 40%-70% relative density (4)	High, >95% Proctor, >70% relative density (5)
Fine-grained Soils (LL > 50)[b] Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer; Otherwise use E' = 0			
Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with more than 25% coarse-grained particles; Coarse-grained Soils with Fines GM, GC, SM, SC[c] contains more than 12% fines	100	400	1,000	2,000
Coarse-grained Soils with Little or no Fines GW, GP, SW, SP[c] contains less than 12% fines	200	1,000	2,000	3,000
Crushed Rock	1,000	3,000	3,000	3,000
Accuracy in Terms of Percentage Deflection[d]	±2	±2	±1	±0.5

[a]ASTM Designation D 2487, USBR Designation E-3.
[b]LL = Liquid limit.
[c]Or any borderline soil beginning with one of these symbols (i.e. GM-GC, GC-SC).
[d]For ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Note: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate Deflection Lag Factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/cu ft (598,000 J/m³) (ASTM D 698, AASHTO T-99, USBR Designation E-11). 1 psi = 6.9 kPa.

SOURCE: "Soil Reaction for Buried Flexible Pipe" by Amster K. Howard, U.S. Bureau of Reclamation, Denver, Colorado. Reprinted with permission from American Society of Civil Engineers.

Attachment B (5/8)

TABLE 7.4

CALCULATED DEFLECTIONS OF BURIED AWWA C900 PVC PIPE
Deflection (Percent) for Highway H20 and Railway E80 Loads

of Cover →	2'		4'		6'		8'		10'		12'		14'		16'		18'		20'	
Load →	H20	E80	H20	E80	H20	E80	H20	E80	H20	E80	H20	E80	H20	E80	H20	E80	H20	E80	H20	E80
E' Value	DR 14																			
50	0.58	2.26	0.49	1.75	0.51	1.66	0.59	1.43	0.67	1.28	0.80	1.25	0.94	1.27	1.07	1.35	1.21	1.49	1.34	1.51
200	0.54	2.10	0.45	1.63	0.48	1.55	0.55	1.34	0.62	1.19	0.75	1.16	0.87	1.19	1.00	1.26	1.12	1.33	1.25	1.40
400	0.50	1.93	0.42	1.49	0.44	1.42	0.50	1.22	0.57	1.09	0.69	1.07	0.80	1.09	0.91	1.15	1.03	1.22	1.14	1.29
1000	0.40	1.54	0.34	1.19	0.35	1.13	0.40	0.98	0.46	0.87	0.55	0.85	0.64	0.87	0.73	0.92	0.82	0.97	0.91	1.03
2000	0.30	1.15	0.25	0.89	0.26	0.85	0.30	0.73	0.34	0.66	0.41	0.64	0.48	0.65	0.55	0.69	0.62	0.73	0.68	0.77
E' Value	DR 18																			
50	1.26	4.89	1.07	3.79	1.11	3.60	1.28	3.10	1.45	2.79	2.09	2.71	2.04	2.76	2.33	2.93	2.62	3.10	2.91	3.27
200	1.09	4.22	0.92	3.28	0.97	3.11	1.12	0.69	1.25	2.40	1.51	2.34	1.76	2.38	2.01	2.53	2.26	2.67	2.51	2.82
400	0.92	3.58	0.78	2.77	0.82	2.63	0.93	2.27	1.06	2.03	1.28	1.98	1.48	2.01	1.69	2.14	1.91	2.26	2.12	2.38
1000	0.63	2.44	0.53	1.89	0.56	1.79	0.63	1.55	0.72	1.38	0.87	1.35	1.01	1.37	1.16	1.46	1.30	1.54	1.45	1.63
2000	0.41	1.59	0.35	1.23	0.36	1.17	0.42	1.01	0.47	0.91	0.57	0.88	0.66	0.90	0.76	0.95	0.85	1.01	0.95	1.06
E' Value	DR 25																			
50	3.24	12.56	2.73	9.73	2.86	9.23	3.29	7.96	3.73	7.15	4.48	6.97	5.22	7.09	5.97	7.52	6.71	7.96	7.46	8.39
200	2.30	8.91	1.94	6.96	2.05	6.60	2.34	5.71	2.66	5.10	3.21	4.97	3.71	5.03	4.23	5.34	4.76	5.65	5.29	5.95
400	1.65	6.47	1.41	4.97	1.47	4.75	1.68	4.10	1.91	3.66	2.30	3.57	2.67	3.63	3.05	3.85	3.43	4.07	3.82	4.29
1000	0.90	3.51	0.76	2.71	0.80	2.58	0.91	2.23	1.04	1.99	1.25	1.94	1.45	1.97	1.66	2.09	1.87	2.21	2.08	2.34
2000	0.51	1.99	0.43	1.54	0.45	1.46	0.52	1.26	0.59	1.13	0.71	1.10	0.83	1.12	0.94	1.19	1.06	1.26	1.18	1.33

(EQUATION 7.12)

Deflection Calculated by:

$$\frac{\Delta Y}{D} = \frac{D_L KP + KW'}{[2E/(3(DR - 1)^3)] + 0.061E'}$$

*

Where:

- P = Prism Load, psi
- K = Bedding constant, 0.1
- W' = Live load, psi
- DR = Dimension ratio
- E = 400,000 psi
- E' = Modulus of soil reaction, psi
- D_L = Deflection lag factor, 1.0

NOTE: Calculation based on soil weight (w) = 120 lb/ft³

Deflection Lag and Creep: The length of time that a buried flex pipe will continue to deflect after the maximum imposed load is realize limited and is a function of soil density in the pipe zone. As soil densit the sides of the pipe increases, the time during which the pipe will conti to deflect decreases, and the total deflection in response to load decrease:

In fact, after the trench load reaches a maximum, the pipe-soil sys continues to deflect only as long as the soil around the pipe is in the proc of consolidation. Once the embedment soil has reached the density requ to support the load, the pipe will not continue to deflect.

The full load on any buried pipe is not reached immediately after ins lation unless the final backfill is compacted to a high density. For a p with good flexibility, the long-term load will not exceed the prism lo The increase in load with time is the largest contribution to increasing flection. Therefore, for design, the prism load should be used, thus ef tively compensating for the increased trench consolidation load with t and resulting increased deflection. When deflection calculations are ba on prism loads, the deflection lag factor, D_L, should be 1.0.

Creep is normally associated with the pipe material and is define continuing deformation with time when the material is subjected to a c stant load. Most plastics exhibit creep. As temperature increases, the cr rate under a given load increases. Also, as stress increases, the creep for a given temperature increases. As PVC creeps, it also relaxes with ti Stress relaxation is defined as the decrease in stress, with time, in a mate held in constant deformation.

Figure 7.5 shows stress relaxation curves for PVC pipe samples hel a constant deflection condition. It is evident that PVC pipe does re stresses with time. The highest stress in a buried PVC non-pressure pip encountered at the equilibrium deflection condition. The behavior dem strated in Figure 7.5 results in a decrease in the actual stress in the pip that deflection.

Figure 7.6 shows long-term data for PVC pipe buried in a soil b Long-term deflection tests were run at Utah State University by imposi given soil load which was held constant throughout the duration of the t PVC pipe material creep properties have little influence on deflection but soil properties such as density exert great influence.

Attachment B (6/8)

ui up iu io kip axie. Under light to medium aircraft loads of up to 00 pounds gross weight, a minimum burial depth of 2 feet is recom- ed.

is recommended that special attention be given to the selection, place- ınd compaction of backfill material with shallow burial flexible pipe, s PVC pipe underneath rigid pavement to prevent injurious cracking road surface.

reverse curvature performance limit for flexible steel pipe was estab- shortly after publication of the Iowa Formula. It was determined that ated steel pipe would begin to reverse curvature at a deflection of 20 percent. Design at that time called for a limit of 5 percent deflec- ıus providing a structural safety factor of 4.0. From this early design eration, an arbitrary design value of 5.0 percent deflection was se-

ried PVC sewer pipe (ASTM D 3034, DR 35), when deflecting in re- to external loading, may develop recognizable reversal of curvature flection of 30 percent. This level of deflection has been commonly ated as a conservative performance limit for PVC sewer pipe. Re- at Utah State University has demonstrated that the load carrying ca- of PVC sewer pipe continues to increase even when deflections in- substantially beyond the point of reversal of curvature. With consid- of this performance characteristic of PVC sewer pipe, engineers lly consider the 7.5 percent deflection limit recommended by ASTM (Appendixes) to provide a very conservative factor of safety against al failure.

ıgitudinal bending of a pipeline is usually indicative of less than sat- y installation conditions. Unlike "rigid pipes," PVC pipe will not n flexure but will bend. Usually such bending does not significantly a pipeline's performance. Only short radius bends can be considered nance limiting for PVC pipe. (See Chapter VIII - Special Design ıtions - Longitudinal Bending.)

buckling phenomenon may govern design of flexible pipes under ons of internal vacuum, sub-aqueous installations or loose soil f the external load exceeds the compressive strength of the pipe ma- For a circular ring subjected to a uniform external pressure or internal ı, the critical buckling pressure (P_{cr}) is defined by Timoshenko as:

*

$$P_{cr} = \frac{3EI}{r^3} = 0.447\ PS$$

Where:
- r = Mean pipe radius, in.
- I = Pipe wall moment of inertia (in^4/in)
- PS = Pipe stiffness
- E = Modulus of elasticity, psi

With the moment of inertia (I) defined as $t^3/12$ for solid wall pipes, Equation 7.13 becomes:

EQUATION 7.14

$$P_{cr} = \frac{2E}{\left[\frac{D_o - t}{t}\right]^3} = \frac{2E}{(DR - 1)^3}$$

Where:
- E = Modulus of elasticity, psi
- DR = Dimension ratio
- D_o = Outside pipe diameter, in.
- t = Pipe wall thickness, in.

For long tubes such as pipelines under combined stress, E is replaced by $E/(1 - v^2)$ and the critical buckling pressure is:

EQUATION 7.15

$$P_{cr} = \frac{3EI}{(1 - v^2)r^3} = \frac{0.447\ PS}{(1 - v^2)}$$

or for solid wall pipes

EQUATION 7.16

$$P_{cr} = \frac{2E}{(1 - v^2)(DR - 1)^3} = \frac{2E}{(1 - v^2)}\left[\frac{t}{D_o - t}\right]^3$$

Attachment B(7/8)

·d in this installation.

$$= \frac{2E}{(1-v^2)(DR-1)^3} = \frac{2(400{,}000)}{[1 - (0.38)^2]\ (18-1)^3} = 190.3 \text{ psi}$$

ɜ:

DR 35 PVC sewer pipe with a 400,000 psi modulus of elasticity ıfined in a saturated soil providing E' = 800 psi, what height (H) of ırated soil which weighs 120 lbs/ft^3 (w) would cause buckling? eight will be limited so deflection does not exceed 7.5 percent.

$$P_{cr} = \frac{2(400{,}000)}{[1 - (0.38)^2]\ (35 - 1)^3} = 23.8 \text{ psi}$$

$$P_b = 1.15\sqrt{23.8\,(800)} = 158.7 \text{ psi} = 22{,}850 \text{ psf}$$

$$H = P/w = 22{,}850/120 = 190 \text{ feet}$$

deflection to 7.5 percent:

$$\Delta = \frac{KP_e}{.149\ PS + .061E'} \times 100$$

$$P_e = \frac{\Delta(.149\ PS + .061E')}{K}$$

$$= \frac{0.075\ [.149(46) + .061(800)]}{0.11}$$

$$P_e = 37.9 \text{ psi} = 5{,}464 \text{ psf}$$

$$H \text{ (to limit deflection)} = 5{,}464/120 = 45.5 \text{ ft.}$$

ːimum cover is limited by the allowable deflection not by buckling. ·efore, the safety factor for the critical failure mode by buckling of ·VC pipe is ample.

·arch has established that flexible steel pipe walls can buckle at de- ː considerably less than 20 percent if the load is large and the soil ling the pipe is extremely compacted. Based on these observations, the design of buried flexible pipes. This theory assumed that the backfill was highly compacted, that deflection would be negligible and that the performance limit was *wall crushing*. The design concept is expressed by:

EQUATION 7.21

$$T = P_y \times \frac{D_o}{2}$$

Where: P_y = Vertical soil pressure, psi
D_o = Outside diameter, in.
T = Wall Thrust, pounds/in.

EQUATION 7.22

$$\sigma_c = \frac{T}{A}$$

Where: σ_c = Compressive stress, psi
A = Area of the pipe wall, in.^{2}/in.

Example: A profile wall PVC pipe (D_o = 19.15 in., A = 2.503 in.^{2}/ft.) is concrete cradled. At what vertical soil pressure or depth of cover could one expect failure by ring compression? (w = 120 lbs./ft.^{3})

$$\sigma_c = \frac{T}{A} \qquad P_y = wH$$

Conservatively assume σ_c = hydrostatic design basis or hoop tensile = 4000 psi.

$$P_y = \frac{\sigma_c 2A}{D_o} = \frac{4000(2)(2.503/12)}{19.15}$$

$$P_y = 87.1 \text{ psi} = wH$$

$$H = \frac{P_y}{w} = \frac{87.1 \text{ psi}}{120 \text{ lbs/ft.}^3} \times 144 \text{ in}^2/\text{ft}^2$$

Attachment B (8/8)

Designation: D 3034 – 97

Standard Specification for Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings[1]

This standard is issued under the fixed designation D 3034; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This specification has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This specification covers requirements and test methods for materials, dimensions, workmanship, flattening resistance, impact resistance, pipe stiffness, extrusion quality, joining systems and a form of marking for type PSM poly(vinyl chloride) (PVC) sewer pipe and fittings.

1.2 Pipe and fittings produced to this specification should be installed in accordance with Practice D 2321.

1.3 The text of this specification references notes, footnotes, and appendixes which provide explanatory material. These notes and footnotes (excluding those in tables and figures) shall not be considered as requirements of the specification.

1.4 The values stated in inch-pound units are to be regarded as the standard. The values given in parentheses are for information only.

1.5 The following precautionary caveat pertains only to the test methods portion, Section 8, of this specification: *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 618 Practice for Conditioning Plastics and Electrical Insulating Materials for Testing[2]

D 1600 Terminology for Abbreviated Terms Relating to Plastics[2,3]

D 1784 Specification for Rigid Poly(Vinyl Chloride) (PVC) Compounds and Chlorinated Poly(Vinyl Chloride) (CPVC) Compounds[3]

D 2122 Test Method for Determining Dimensions of Thermoplastic Pipe and Fittings[3]

D 2152 Test Method for Degree of Fusion of Extruded Poly(Vinyl Chloride) (PVC) Pipe and Molded Fittings by Acetone Immersion[3]

D 2321 Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications[3]

D 2412 Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading[3]

D 2444 Test Method for Impact Resistance of Thermoplastic Pipe and Fittings by Means of a Tup (Falling Weight)[3]

D 2564 Specification for Solvent Cements for Poly(Vinyl Chloride) (PVC) Plastic Piping Systems[3]

D 2749 Symbols for Dimensions of Plastic Pipe Fittings[3]

D 2855 Practice for Making Solvent-Cemented Joints with Poly(Vinyl Chloride) (PVC) Pipe and Fittings[3]

D 3212 Specification for Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals[3]

F 412 Terminology Relating to Plastic Piping Systems[3]

2.2 *Federal Standard:*[4]

Fed. Std. No. 123 Marking for Shipment (Civil Agencies)

2.3 *Military Standard:*[4]

MIL-STD-129 Marking for Shipment and Storage

3. Terminology

3.1 *Definitions*— Definitions are in accordance with Terminology F 412, and abbreviations are in accordance with Terminology D 1600, unless otherwise specified. The abbreviation of poly(vinyl chloride) plastics is PVC.

3.1.1 The term PSM is not an abbreviation but rather an arbitrary designation for a product having certain dimensions.

4. Significance and Use

4.1 The requirements of this specification are intended to provide pipe and fittings suitable for non-pressure drainage of sewage and surface water.

NOTE 1—Industrial waste disposal lines should be installed only with the specific approval of the cognizant code authority since chemicals not commonly found in drains and sewers and temperatures in excess of 60°C (140°F) may be encountered.

5. Materials

5.1 *Basic Materials*—The pipe shall be made of PVC plastic having a cell classification of 12454-B or 12454-C or 12364-C or 13364-B (with minimum tensile modulus of

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[1] This specification is under the jurisdiction of ASTM Committee F-17 on Plastic Piping Systems and is the direct responsibility of Subcommittee F17.62 on Sewer.

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[2] *Annual Book of ASTM Standards*, Vol 08.01.

[3] *Annual Book of ASTM Standards*, Vol 08.04.

[4] Available from Standardization Documents Order Desk, Bldg. 4 Section D, 700 Robbins Ave., Philadelphia, PA 19111-5094, Attn: NPODS.

ATTACHMENT C, 1/2

TABLE X1.1 Base Inside Diameters and 7½ % Deflection Mandrel Dimension

Nominal Size, in.	SDR-41 Average Inside Diameter	SDR-41 Base Inside Diameter[A]	SDR-41 7½ % Deflection Mandrel	SDR-35 Average Inside Diameter	SDR-35 Base Inside Diameter[A]	SDR-35 7½ % Deflection Mandrel	SDR-26 Average Inside Diameter	SDR-26 Base Inside Diameter[A]	SDR-26 7½ % Deflection Mandrel	SDR 23.5 Average Inside Diameter	SDR 23.5 Base Inside Diameter[A]	SDR 23.5 7½ % Deflection Mandrel
						in.						
6	5.951	5.800	5.37	5.893	5.742	5.31	5.764	5.612	5.19	5.713	5.562	5.14
8	7.966	7.740	7.16	7.891	7.665	7.09	7.715	7.488	6.93	...	...	...
9	8.952	8.691	8.04	...	...	...	...	...	...	...	...	...
10	9.958	9.657	8.93	9.864	9.563	8.84	9.644	9.342	8.64	...	...	...
12	11.854	11.478	10.62	11.737	11.361	10.51	11.480	11.102	10.27	...	...	...
15	14.505	14.029	12.98	14.374	13.898	12.86	14.053	13.575	12.56	...	...	...
						mm						
6	151.16	147.32	136.3	149.68	145.85	134.9	146.41	142.54	131.8	145.11	141.27	130.6
8	202.34	196.60	181.8	200.43	194.69	180.1	195.96	190.20	175.9	...	...	...
9	227.38	220.75	204.2	...	...	...	...	...	...	...	...	...
10	252.93	245.29	226.9	250.54	242.90	224.7	244.96	237.29	219.5	...	...	...
12	301.09	291.54	269.7	298.12	288.57	266.9	291.59	281.99	260.9	...	...	...
15	368.43	356.34	329.6	365.10	353.01	326.5	356.95	344.80	318.9	...	...	...

[A] Base inside diameter is a minimum pipe inside diameter derived by subtracting a statistical tolerance package from the pipe's average inside diameter. The tolerance package is defined as the square root of the sum of squared standard manufacturing tolerances.

$$\text{Average inside diameter} = \text{average outside diameter} - 2(1.06t)$$

$$\text{Tolerance package} = \sqrt{A^2 + 2B^2 + C^2}$$

where:

t = minimum wall thickness (Table 1),
A = outside diameter tolerance (Table 1),
B = excess wall thickness tolerance = $0.06t$, and
C = out-of-roundness tolerance.

The values for C were derived statistically from field measurement data and are given as follows for various sizes of pipe:

Nominal Size, in.	Value for C, in.	Value for C, mm
6	0.150	3.81
8	0.225	5.72
9	0.260	6.60
10	0.300	7.62
12	0.375	9.52
15	0.475	12.06

X2. RECOMMENDED LIMIT FOR INSTALLED DEFLECTION[5]

X2.1 Design engineers, public agencies, and others who have the responsibility to establish specifications for maximum allowable limits for deflection of installed PVC sewer pipe have requested direction relative to such a limit.

X2.2 PVC sewer piping made to this specification and installed in accordance with Practice D 2321 can be expected to perform satisfactorily provided that the internal diameter of the barrel is not reduced by more than 7½ % of its base inside diameter when measured not less than 30 days following completion of installation.

[5] Supporting data can be obtained from ASTM Headquarters. Request RR:F-17-1009.

ATTACHMENT C, 2/2

be small compared to the pressure due to the fill, the vertical pressure on the top of the pipe can be assumed to be equal to the unit weight of the refuse fill multiplied by the distance from top of fill to top of pipe, thus:

$$\sigma_v = (\omega_f)(H_f).$$

V.2.2.3 Perforated Pipe

Perforations will reduce the effective length of pipe available to carry loads and resist deflection. The effect of perforations can be taken into account by using an increased load per nominal unit length of the pipe. If l_p equals the cumulative length in inches of perforations per foot of pipe, the increased vertical stress to be used equals:

$$(\sigma_v)\text{design} = \frac{12}{12-l_p} \times (\sigma_v)\text{actual}$$

REFERENCE: EPA, MANUAL SW-8
"LINING OF WASTE IMPOUNDMENT
AND DISPOSAL FACILITIES".
SEPTEMBER 1980

Attachment D 1/1

Designation: D 1785 – 96b

An American National Standard

Standard Specification for Poly(Vinyl Chloride) (PVC) Plastic Pipe, Schedules 40, 80, and 120[1]

This standard is issued under the fixed designation D 1785; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This specification covers poly(vinyl chloride) (PVC) pipe made in Schedule 40, 80, and 120 sizes and pressure-rated for water (see Appendix). Included are criteria for classifying PVC plastic pipe materials and PVC plastic pipe, a system of nomenclature for PVC plastic pipe, and requirements and test methods for materials, workmanship, dimensions, sustained pressure, burst pressure, flattening, and extrusion quality. Methods of marking are also given.

1.2 The text of this specification references notes, footnotes, and appendixes which provide explanatory material. These notes and footnotes (excluding those in tables and figures) shall not be considered as requirements of the specification.

1.3 The values stated in inch-pound units are to be regarded as the standard. The values given in parentheses are for information only.

1.4 The following safety hazards caveat pertains only to the test methods portion, Section 8, of this specification: *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.* A specific precautionary statement is given in Note 7.

NOTE 1—CPVC plastic pipes, Schedules 40 and 80, which were formerly included in this specification, are now covered by Specification F 441.

NOTE 2—The sustained and burst pressure test requirements, and the pressure ratings in the Appendix, are calculated from stress values obtained from tests made on pipe 4 in. (100 mm) and smaller. However, tests conducted on pipe as large as 24-in. (600-mm) diameter have shown these stress values to be valid for larger diameter PVC pipe.

NOTE 3—PVC pipe made to this specification is often belled for use as line pipe. For details of the solvent cement bell, see Specification D 2672 and for details of belled elastomeric joints, see Specifications D 3139 and D 3212.

2. Referenced Documents

2.1 *ASTM Standards:*

D 618 Practice for Conditioning Plastics and Electrical Insulating Materials for Testing[2]

D 1598 Test Method for Time-to-Failure of Plastic Pipe Under Constant Internal Pressure[3]

D 1599 Test Method for Short-Time Hydraulic Failure Pressure of Plastic Pipe, Tubing, and Fittings[3]

D 1600 Terminology for Abbreviated Terms Relating to Plastics[2]

D 1784 Specification for Rigid Poly(Vinyl Chloride) (PVC) Compounds and Chlorinated Poly(Vinyl Chloride) (CPVC) Compounds[2]

D 2122 Test Method for Determining Dimensions of Thermoplastic Pipe and Fittings[3]

D 2152 Test Method for Degree of Fusion of Extruded Poly(Vinyl Chloride) (PVC) Pipe and Molded Fittings by Acetone Immersion[3]

D 2672 Specification for Joints for IPS PVC Pipe Using Solvent Cement[3]

D 2837 Test Method for Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials[3]

D 3139 Specification for Joints for Plastic Pressure Pipes Using Flexible Elastomeric Seals[3]

D 3212 Specification for Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals[3]

F 412 Terminology Relating to Plastic Piping Systems[3]

F 441 Specification for Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Pipe, Schedules 40 and 80[3]

2.2 *Federal Standard:*

Fed. Std. No. 123 Marking for Shipment (Civil Agencies)

2.3 *Military Standard:*

MIL-STD-129 Marking for Shipment and Storage[4]

2.4 *NSF Standards:*

Standard No. 14 for Plastic Piping Components and Related Materials[5]

Standard No. 61 for Drinking Water System Components—Health Effects[5]

3. Terminology

3.1 *Definitions*—Definitions are in accordance with Terminology F 412 and abbreviations are in accordance with Terminology D 1600, unless otherwise specified. The abbreviation for poly(vinyl chloride) plastic is PVC.

3.2 *Descriptions of Terms Specific to This Standard:*

3.2.1 *hydrostatic design stress*—the estimated maximum

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[1] This specification is under the jurisdiction of ASTM Committee F-17 on Plastic Piping Systems and is the direct responsibility of Subcommittee F17.25 on Vinyl Based Pipe.

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[2] *Annual Book of ASTM Standards*, Vol 08.01.

[3] *Annual Book of ASTM Standards*, Vol 08.04.

[4] Available from Standardization Documents Order Desk, Bldg. 4 Section D, 700 Robbins Ave., Philadelphia, PA 19111-5094, Attn: NPODS.

[5] Available from the National Sanitation Foundation, P.O. Box 1468, Ann Arbor, MI 48106.

ATTACHMENT E 1/2

TABLE 1 Outside Diameters and Tolerances for PVC Plastic Pipe Schedules 40, 80, and 120, in. (mm)

Nominal Pipe Size	Outside Diameter	Tolerances		
		Average	Maximum Out-of-Roundness (maximum minus minimum diameter)	
			Schedule 40 sizes 3½ in. and over; Schedule 80 sizes 8 in. and over	Schedule 40 sizes 3 in. and less; Schedule 80 sizes 6 in. and less; Schedule 120 sizes all
⅛	0.405 (10.29)	±0.004 (±0.10)	...	0.016 (0.41)
¼	0.540 (13.72)	±0.004 (±0.10)	...	0.016 (0.41)
⅜	0.675 (17.14)	±0.004 (±0.10)	...	0.016 (0.41)
½	0.840 (21.34)	±0.004 (±0.10)	...	0.016 (0.41)
¾	1.050 (26.67)	±0.004 (±0.10)	...	0.016 (0.41)
1	1.315 (33.40)	±0.005 (±0.13)	...	0.020 (0.51)
1¼	1.660 (42.16)	±0.005 (±0.13)	...	0.020 (0.51)
1½	1.900 (48.26)	±0.006 (±0.15)	...	0.024 (0.61)
2	2.375 (60.32)	±0.006 (±0.15)	...	0.024 (0.61)
2½	2.875 (73.02)	±0.007 (±0.18)	...	0.024 (0.61)
3	3.500 (88.90)	±0.008 (±0.20)	...	0.030 (0.76)
3½	4.000 (101.60)	±0.008 (±0.20)	0.100 (2.54)	0.030 (0.76)
4	4.500 (114.30)	±0.009 (±0.23)	0.100 (2.54)	0.030 (0.76)
5	5.563 (141.30)	±0.010 (±0.25)	0.100 (2.54)	0.030 (0.76)
6	6.625 (168.28)	±0.011 (±0.28)	0.100 (2.54)	0.060 (1.52)
8	8.625 (219.08)	±0.015 (±0.38)	0.150 (3.81)	0.070 (1.78)
10	10.750 (273.05)	±0.015 (±0.38)	0.150 (3.81)	0.090 (2.29)
12	12.750 (323.85)	±0.015 (±0.38)	0.150 (3.81)	0.100 (2.54)
14	14.000 (355.60	±0.015 (±0.38)	0.200 (5.08)	0.120 (3.05)
16	16.000 (406.40)	±0.019 (±0.48)	0.320 (8.13)	...
18	18.000 (457.20)	±0.019 (±0.48)	0.360 (9.14)	...
20	20.000 (508.00)	±0.023 (±0.58)	0.400 (10.2)	...
24	24.000 (609.60)	±0.031 (±0.79)	0.480 (12.2)	...

TABLE 2 Wall Thicknesses and Tolerances for PVC Plastic Pipe, Schedules 40, 80, and 120,[A,B] in. (mm)

Nominal Pipe Size	Wall Thickness[A]					
	Schedule 40		Schedule 80		Schedule 120	
	Minimum	Tolerance	Minimum	Tolerance	Minimum	Tolerance
⅛	0.068 (1.73)	+0.020 (+0.51)	0.095 (2.41)	+0.020 (+0.51)	...	...
¼	0.088 (2.24)	+0.020 (+0.51)	0.119 (3.02)	+0.020 (+0.51)	...	...
⅜	0.091 (2.31)	+0.020 (+0.51)	0.126 (3.20)	+0.020 (+0.51)	...	...
½	0.109 (2.77)	+0.020 (+0.51)	0.147 (3.73)	+0.020 (+0.51)	0.170 (4.32)	+0.020 (+0.51)
¾	0.113 (2.87)	+0.020 (+0.51)	0.154 (3.91)	+0.020 (+0.51)	0.170 (4.32)	+0.020 (+0.51)
1	0.133 (3.38)	+0.020 (+0.51)	0.179 (4.55)	+0.021 (+0.53)	0.200 (5.08)	+0.024 (+0.61)
1¼	0.140 (3.56)	+0.020 (+0.51)	0.191 (4.85)	+0.023 (+0.58)	0.215 (5.46)	+0.026 (+0.66)
1½	0.145 (3.68)	+0.020 (+0.51)	0.200 (5.08)	+0.024 (+0.61)	0.225 (5.72)	+0.027 (+0.68)
2	0.154 (3.91)	+0.020 (+0.51)	0.218 (5.54)	+0.026 (+0.66)	0.250 (6.35)	+0.030 (+0.76)
2½	0.203 (5.16)	+0.024 (+0.61)	0.276 (7.01)	+0.033 (+0.84)	0.300 (7.62)	+0.036 (+0.91)
3	0.216 (5.49)	+0.026 (+0.66)	0.300 (7.62)	+0.036 (+0.91)	0.350 (8.89)	+0.042 (+1.07)
3½	0.226 (5.74)	+0.027 (+0.68)	0.318 (8.08)	+0.038 (+0.96)	0.350 (8.89)	+0.042 (+1.07)
4	0.237 (6.02)	+0.028 (+0.71)	0.337 (8.56)	+0.040 (+1.02)	0.437 (11.10)	+0.052 (+1.32)
5	0.258 (6.55)	+0.031 (+0.79)	0.375 (9.52)	+0.045 (+1.14)	0.500 (12.70)	+0.060 (+1.52)
6	0.280 (7.11)	+0.034 (+0.86)	0.432 (10.97)	+0.052 (+1.32)	0.562 (14.27)	+0.067 (+1.70)
8	0.322 (8.18)	+0.039 (+0.99)	0.500 (12.70)	+0.060 (+1.52)	0.718 (18.24)	+0.086 (+2.18)
10	0.365 (9.27)	+0.044 (+1.12)	0.593 (15.06)	+0.071 (+1.80)	0.843 (21.41)	+0.101 (+2.56)
12	0.406 (10.31)	+0.049 (+1.24)	0.687 (17.45)	+0.082 (+2.08)	1.000 (25.40)	+0.120 (+3.05)
14	0.437 (11.10)	+0.053 (+1.35)	0.750 (19.05)	+0.090 (+2.29)	...	...
16	0.500 (12.70)	+0.060 (+1.52)	0.843 (21.41)	+0.101 (+2.57)	...	...
18	0.562 (14.27)	+0.067 (+1.70)	0.937 (23.80)	+0.112 (+2.84)	...	...
20	0.593 (15.06)	+0.071 (+1.80)	1.031 (26.19)	+0.124 (+3.15)	...	...
24	0.687 (17.45)	+0.082 (+2.08)	1.218 (30.94)	+0.146 (+3.71)	...	...

[A] The minimum is the lowest wall thickness of the pipe at any cross section. The maximum permitted wall thickness, at any cross section, is the minimum wall thickness plus the stated tolerance. All tolerances are on the plus side of the minimum requirement.

[B] These dimensions conform to nominal IPS dimensions, with the exception that Schedule 120 wall thickness for pipe sizes ½ to 3½ in. (12.5 to 87.5 mm), inclusive, are special PVC plastic pipe sizes.

ATTACHMENT E, 2/2

Designation: D 1784 – 92

An American National Stan

Standard Specification for Rigid Poly(Vinyl Chloride) (PVC) Compounds and Chlorinated Poly(Vinyl Chloride) (CPVC) Compounds[1]

This standard is issued under the fixed designation D 1784; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This specification has been approved for use by agencies of the Department of Defense. Consult the DoD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

1. Scope

1.1 This specification covers rigid PVC and CPVC compounds intended for general purpose use in extruded or molded form, including piping applications involving special chemical and acid resistance or heat resistance, composed of poly(vinyl chloride), chlorinated poly(vinyl chloride), or vinyl chloride copolymers containing at least 80 % vinyl chloride, and the necessary compounding ingredients. The compounding ingredients may consist of lubricants, stabilizers, non-poly(vinyl chloride) resin modifiers, pigments and inorganic fillers.

NOTE 1—Selection of specific compounds for particular end uses or applications requires consideration of other characteristics such as thermal properties, optical properties, weather resistance, etc. Specific requirements and test methods for these properties shall be by mutual agreement between the purchaser and the seller.

1.2 Rigid PVC compounds intended for pipe, fittings and other piping appurtenances are covered in Specifications D 3915 and D 4396.

1.3 Rigid PVC compounds intended for building product applications are covered in Specification D 4216.

1.4 The values stated in SI units are to be regarded as the standard. The values given in parentheses are for information only.

1.5 The following safety hazards caveat pertains only to the test methods portion, Section 11, of this specification: *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

NOTE 2—This specification is similar in content (but not technically equivalent) to ISO 1163-1:1985 and ISO 1163-2:1980.[2]

2. Referenced Documents

2.1 *ASTM Standards:*

D 256 Test Methods for Impact Resistance of Plastics and Electrical Insulating Materials[3]

D 471 Test Method for Rubber Property—Effect Liquids[4]

D 543 Test Method for Resistance of Plastics to Chemic Reagents[3]

D 618 Practice for Conditioning Plastics and Electric Insulating Materials for Testing[3]

D 635 Test Method for Rate of Burning and/or Extent a Time of Burning of Self-Supporting Plastics in a Ho zontal Position[3]

D 638 Test Method for Tensile Properties of Plastics[3]

D 648 Test Method for Deflection Temperature of Plasti Under Flexural Load[3]

D 790 Test Methods for Flexural Properties of Unrei forced and Reinforced Plastics and Electrical Insulatin Materials[3]

D 883 Terminology Relating to Plastics[3]

D 1600 Terminology for Abbreviated Terms Relating Plastics[3]

D 1898 Practice for Sampling of Plastics[5]

D 1921 Test Methods for Particle Size (Sieve Analysis) Plastic Materials[5]

D 3892 Practice for Packaging/Packing of Plastics[5]

D 3915 Specification for Poly(Vinyl Chloride) (PVC) an Related Plastic Pipe and Fitting Compounds for Pre sure Applications[7]

D 4216 Specification for Rigid Poly(Vinyl Chloride) (PVC and Related Plastic Building Products Compounds[7]

D 4396 Specification for Rigid Poly(Vinyl Chloride) (PVC and Related Plastic Compounds for Non-Pressure Pip ing Products[7]

D 5260 Classification for Chemical Resistance of Poly (Vinyl Chloride) (PVC) Homopolymer and Copolyme Compounds and Chlorinated Poly(Vinyl Chloride (CPVC) Compounds[6]

3. Terminology

3.1 *Definitions*—Definitions are in accordance with Defi nitions D 883 and abbreviations with Terminology D 160 unless otherwise indicated.

4. Classification

4.1 Means for selecting and identifying rigid PVC com

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[1] This specification is under the jurisdiction of ASTM Committee D-20 on Plastics and is the direct responsibility of Subcommittee D20.15 on Thermoplastic Materials.

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[2] Available from American National Standards Institute, 11 W. 42nd St., 13th Floor, New York, NY 10036.

[3] *Annual Book of ASTM Standards*, Vol 08.01.

[4] *Annual Book of ASTM Standards*, Vol 09.01.

[5] *Annual Book of ASTM Standards*, Vol 08.02.

[6] *Annual Book of ASTM Standards*, Vol 08.03.

[7] *Annual Book of ASTM Standards*, Vol 08.04.

ATTACHMENT F 1/2

TABLE 1 Class Requirements for Rigid Poly(Vinyl Chloride) Compounds

NOTE—The minimum property value will determine the cell number although the maximum expected value may fall within a higher cell

Designation Order No.	Property and Unit	Cell Limits								
		0	1	2	3	4	5	6	7	8
1	Base resin	unspecified	poly(vinyl chloride) homo-polymer	chlorinated poly(vinyl chloride)	vinyl copolymer					
2	Impact strength (Izod) min:									
	J/m of notch	unspecified	<34.7	34.7	80.1	266.9	533.8	800.7		
	ft·lb/in. of notch		<0.65	0.65	1.5	5.0	10.0	15.0		
3	Tensile strength, min:									
	MPa	unspecified	<34.5	34.5	41.4	48.3	55.2			
	psi		<5 000	5 000	6 000	7 000	8 000			
4	Modulus of elasticity in tension, min:									
	MPa	unspecified	<1930	1930	2206	2482	2758	3034		
	psi		<280 000	280 000	320 000	360 000	400 000	440 000		
5	Deflection temperature under load, min, 1.82 MPa (264 psi):									
	°C	unspecified	<55	55	60	70	80	90	100	110
	°F		<131	131	140	158	176	194	212	230
	Flammability	A	A	A	A	A	A	A	A	A

[A] All compounds covered by this specification when tested in accordance with Method D 635 shall yield the following results: average extent of burning of <25 mm; average time of burning of <10 s.

pounds are provided in Tables 1 and 2. The properties enumerated in Table 1 and the tests defined are expected to provide identification of the compounds selected. They are not necessarily suitable for direct application in design because of differences in shape of part, size, loading, environmental conditions, etc.

4.2 Classes are designated by the cell number for each property in the order in which they are listed in Table 1 including a suffix letter specifying the requirements for chemical resistance, as shown in Table 2.

NOTE 3—The chemical resistance requirements in Table 2 are included to provide identification of the compounds selected. They are not necessarily suitable for rating of application chemical resistance.

NOTE 4—The manner in which selected materials are identified by this classification system is illustrated by a Class 12454-B rigid PVC compound having the following requirements (see Tables 1 and 2):

Class	1 2 4 5 4 B
Identification:	
Poly(vinyl chloride) homopolymer	1
Property and Minimum Value:	
Impact strength (Izod) (34.7 J/m (0.65 ft·lbf/in.))	2
Tensile stength (48.3 MPa (7000 psi))	4
Modulus of elasticity in tension (2758 MPa (400 000 psi))	5
Deflection temperature under load (70°C (158°F))	4
Chemical resistance (meets the requirements of Suffix B in Table 2)	B

NOTE 5—The cell-type format provides the means for identification and close characterization and specification of material properties, alone or in combination, for a broad range of materials. This type format, however, is subject to possible misapplication since unobtainable property combinations can be selected if the user is not familiar with commercially available materials. The manufacturer should be consulted.

4.3 Type and grade number designations have been widely used to define the minimum physical properties and chemical resistance requirements of certain commercial classes of rigid PVC compounds. Table X1.1 in the Appendix lists these type and grade numbers and the corresponding class numbers selected from Table 1 and 2. The classes for previous types and grades of poly(vinyl chloride vinyl acetate) compounds are listed in Table X2.1 in the Appendix.

4.4 Product application chemical resistance when specified shall be classified according to the Classification Section of Classification D 5260.

5. Ordering Information

5.1 The purchase order, or inquiry for these materials, shall state the specification number and identify the class selected, for example, D 1784, Class 12454-B.

5.2 Further definition, as may be required for the fol-

TABLE 2 Suffix Designation for Chemical Resistance

Solution	A	B	C	D
H_2SO_4 (93 %)—14 days immersion at 55 ± 2°C:				
Change in weight:				
Increase, max, %	1.0[A]	5.0[A]	25.0	NA[B]
Decrease, max, %	0.1[A]	0.1[A]	0.1	NA
Change in flexural yield strength:				
Increase, max, %	5.0[A]	5.0[A]	5.0	NA
Decrease, max, %	5.0[A]	25.0[A]	50.0	NA
H_2SO_4 (80 %—30 days immersion at 60 ± 2°C:				
Change in weight:				
Increase, max, %	NA	NA	5.0	15.0
Decrease, max, %	NA	NA	5.0	0.1
Change in flexural yield strength:				
Increase, max, %	NA	NA	15.0	25.0
Decrease, max, %	NA	NA	15.0	25.0
ASTM Oil No. 3—30 days immersion at 23°C:				
Change in weight:				
Increase, max, %	0.5	1.0	1.0	10.0
Decrease, max, %	0.5	1.0	1.0	0.1

[A] Specimens washed in running water and dried by an air blast or other mechanical means shall show no sweating within 2 h after removal from the acid bath.

[B] NA = not applicable.

ATTACHMENT F, 2/2

Physical Properties of Harvel Rigid PVC & CPVC Pipe

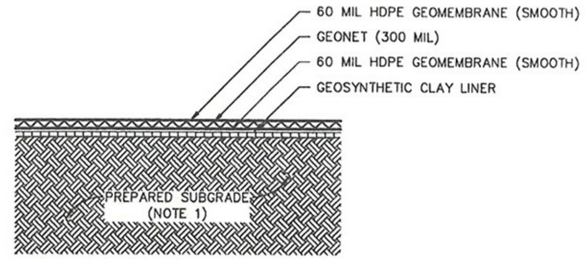
Properties	ASTM Test Method	PVC 1120 (Normal Impact)	PVC 2110 (HI Impact)	Harvel CPVC 4120
Mechanical				
Specific Gravity, g/cm³	D792	1.40 ± .02	1.37 ± .02	1.55 ± .02
Tensile Strength at 73° F psi	D638	7,450	6,400	8,000
Modulus Elasticity In Tension, psi at 73° F	D638	420,000	385,000	360,000
Compressive Strength, psi at 73° F	D695	9,600	8,600	9,000
Flexural Strength at 73° F psi	D790	14,450	11,650	15,100
Izod Impact, ft. lb./in. notch at 73° F	D256	.75	10.9	1.5
Hardness Durometer D	D2240	82 ± 3	78 ± 3	—
Hardness Rockwell R	D785	110 - 120	—	119
Thermal				
Coefficient of Thermal Conductivity $\frac{(Cal.)(cm)}{(cm^2)(sec.)(°C)} \times 10^{-4}$	C177	3.5	4.5	0.96
Coefficient of Linear Expansion	D696			
$\times 10^{-5}$ cm/cm °C		5.2	9.9	6.2
$\times 10^{-5}$ in/in °F		2.9	5.5	3.4
Heat Distortion Temperature, °F at 264 psi	D648	170	146	217
Specific Heat, Cal./°C/gm	D2766	0.25	0.25	—
Upper Service Temp. Limit °F		140	140	200
Flammability				
Average Time of Burning (sec.)	D635	<5	<5	<5
Average Extent of Burning (mm)		<10	<15	<10
Flame Spread Index	E162	<10	—	<10
Flame Spread	E84	10-25	—	4-18
Flash Ignition		730°F	—	900°F
Smoke Developed*		600-1000	—	9-169
Flammability (.062")	UL-94	V-O	—	V-O, 5VB, 5VA
Softening Starts, approx. °F		250	—	295
Material Become Viscous, °F		350	—	395
Material Carbonizes, °F		425	—	450
Limiting Oxygen Index (LOI)				60
Electrical				
Dielectric Strength, volts/mil	D149	1,413	1,085	1,250
Dielectric Constant	D150			
60 cps at 30°C		3.70	3.90	—
1000 cps at 30°C		3.62	3.31	—
Power Factor %	D150			
60 cps at 30°C		1.25	2.85	—
1000 cps at 30°C		2.82	3.97	—
Volume Resistivity at 95°C, ohms/cm/10^{12}		1.2	2.4	—
Harvel Rigid Pipe is non-electrolytic.				
Other Properties				
Water Absorption, % Increase— 24hrs. at 25°C	D570	0.05	0.10	0.03
Light Transmission	E308	Opaque	Opaque	—
Light Stability		Excellent	Excellent	—
Effect of Sunlight		Slight Darkening	Slight Darkening	—
Color (Standard)		Dark Grey	Light Grey	Medium Grey
Material Cell Classification				
ASTM D1784		12454-B	16334-D	23447-B
ASTM D3915		12452-4	14341-1	23444-4

ASTM D1784 and D3915 refer to similar compounds. The major difference is that the alphabetical sixth place designation refers to corrosion resistance under ASTM D1784, and the sixth place designation under D3915 refers to the hydrostatic design stress. In addition, D3915 also places upper limits for values in the second through the fifth place designations.

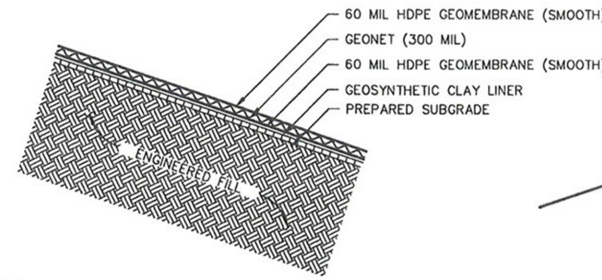
*Tests performed on pipe sizes 3/4" - 4" with a single pipe exposed each test. Some of the CPVC pipes were water filled and these resulted in the lower smoke development values.
NOTE: Harvel CPVC pipe is extruded from Corzan™ CPVC compounds manufactured by BF Goodrich Specialty Polymers and Chemicals Division.

HARVEL PLASTICS MANUFACTURER DOCUMENTATION

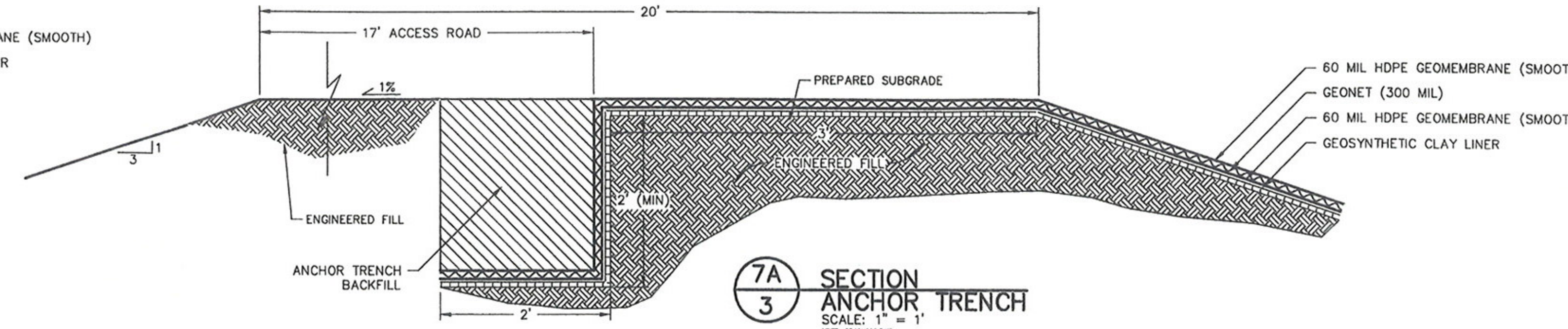
ATTACHMENT G, 1/1



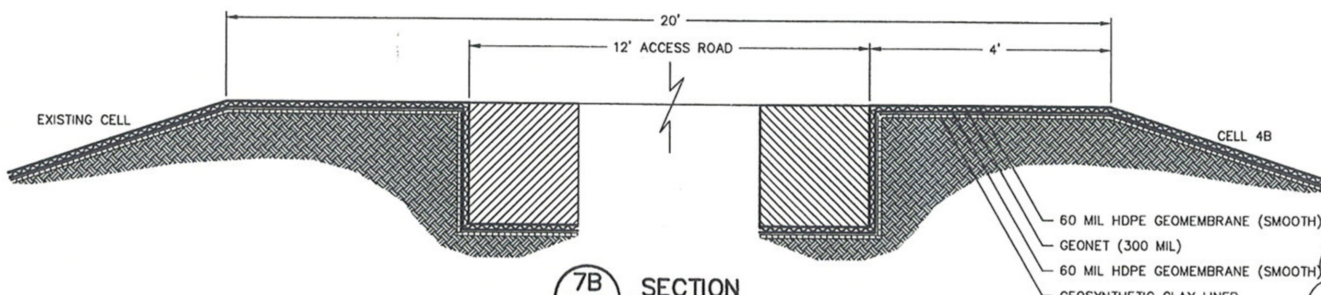
5/3 DETAIL
BASE LINER SYSTEM
SCALE: 1" = 1'



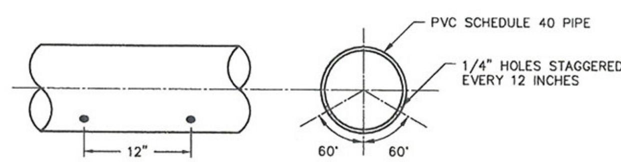
6/3 DETAIL
SIDE SLOPE LINER SYSTEM
SCALE: 1" = 1'



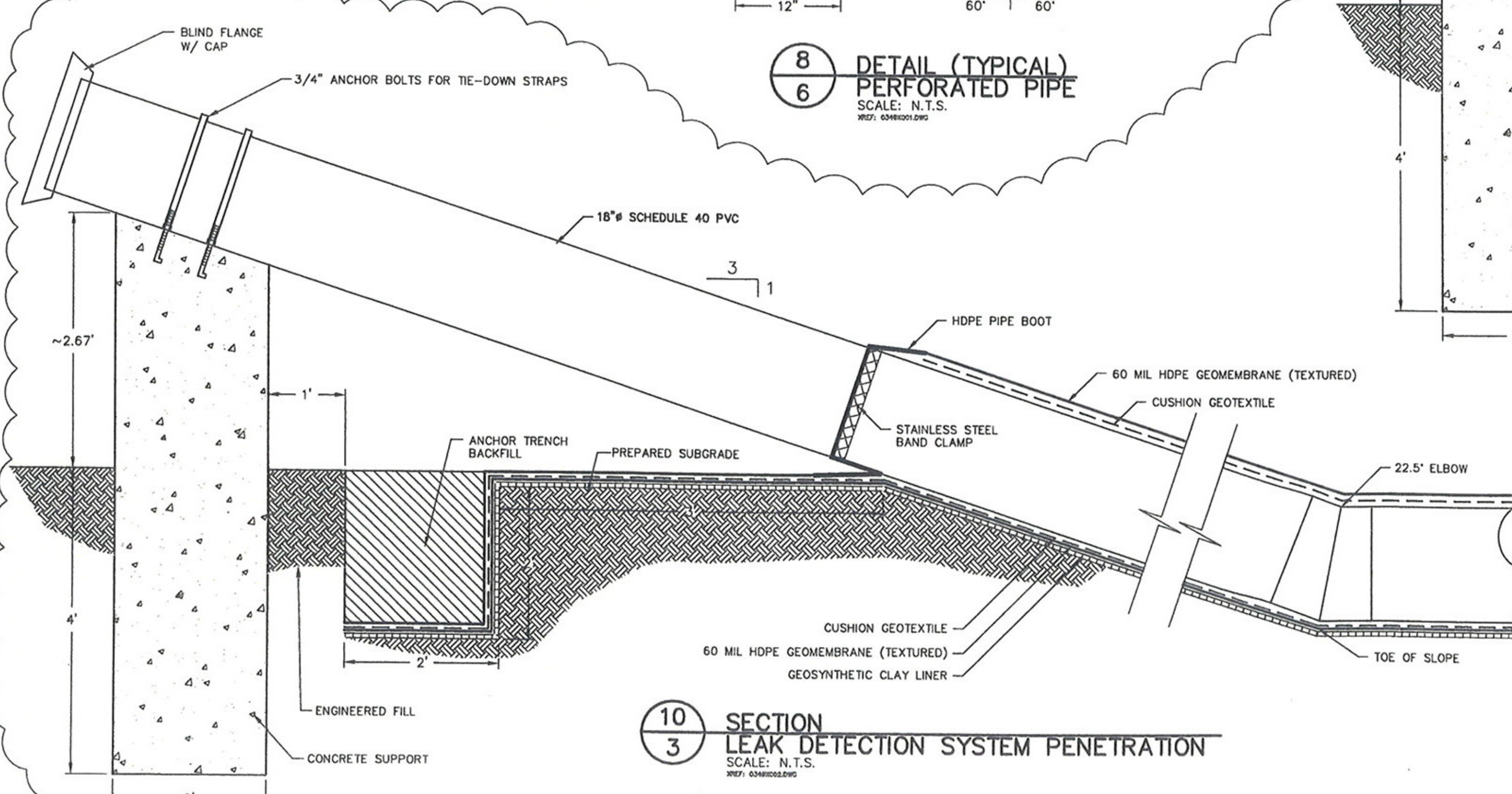
7A/3 SECTION
ANCHOR TRENCH
SCALE: 1" = 1'



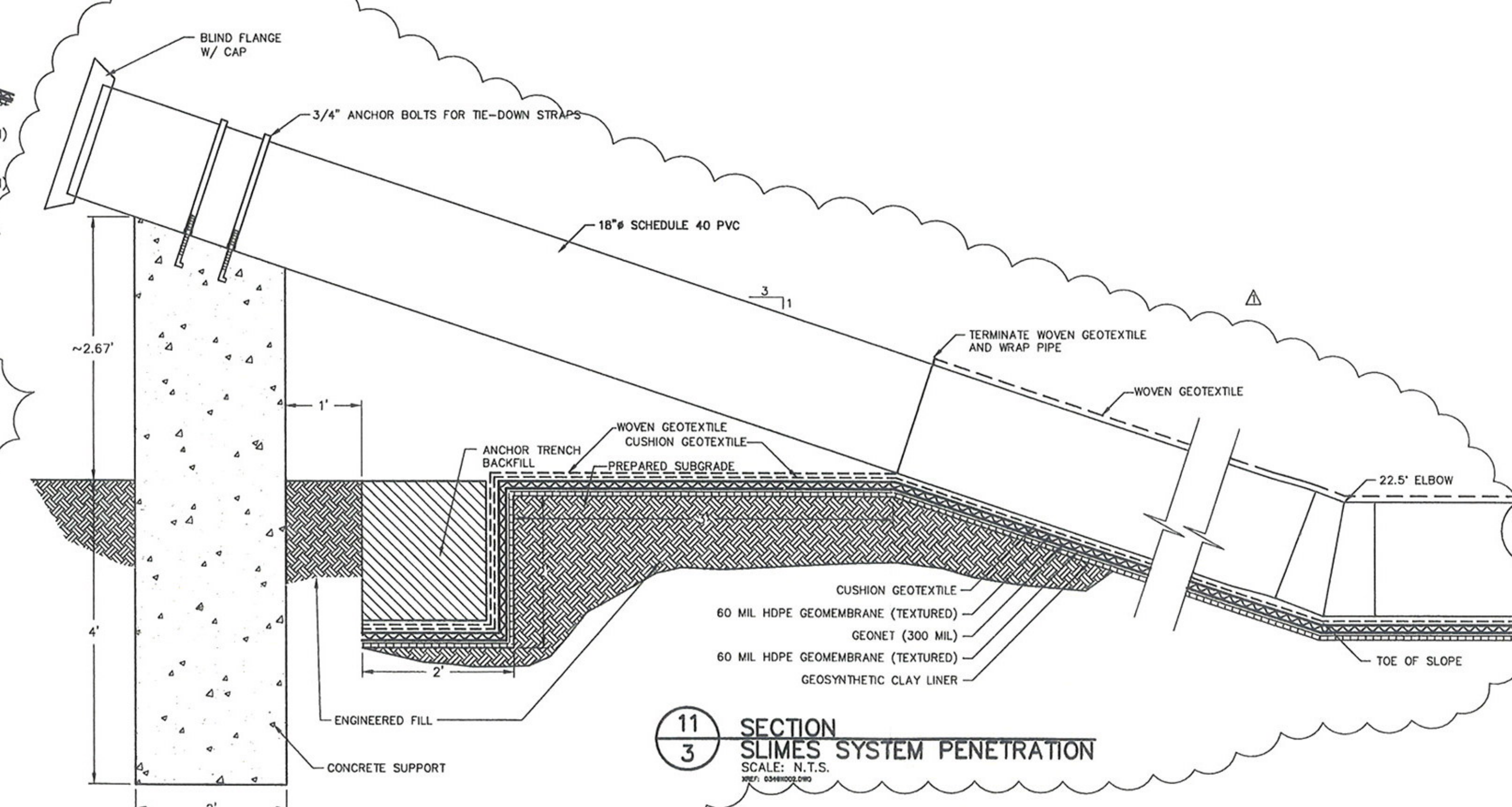
7B/3 SECTION
ACCESS ROAD
SCALE: N.T.S.



8/6 DETAIL (TYPICAL)
PERFORATED PIPE
SCALE: N.T.S.



10/3 SECTION
LEAK DETECTION SYSTEM PENETRATION
SCALE: N.T.S.



11/3 SECTION
SLIMES SYSTEM PENETRATION
SCALE: N.T.S.

NOTES:

1. PREPARED SUBGRADE AT CELL BASE SHALL CONSIST OF AT LEAST 6-INCHES OF FILL OVERLYING SANDSTONE IN ACCORDANCE WITH SECTIONS 02200 AND 02220 OF THE TECHNICAL SPECIFICATIONS.
2. DETAILS ARE SHOWN TO SCALE INDICATED EXCEPT FOR THE GEOSYNTHETICS, WHICH ARE SHOWN AT AN EXAGGERATED SCALE FOR CLARITY SOIL THICKNESS ARE MINIMUMS.
3. WOVEN GEOTEXTILE SHALL BE PROPEX 200 ST, SKAPS W-200, OR APPROVED EQUAL (WOVEN SLIT FILM, AOS = 40, FLOW RATE = 4 GPM/SF, GRAB STRENGTH = 200 LBS, PUNCTURE = 100 LBS)
4. EXPOSED PVC PIPE SHALL BE PAINTED TO MINIMIZE DAMAGE DUE TO UV.

REV	DATE	DESCRIPTION	DRN	APP
1	01/09/09	INTERROGATORY ROUND 1	MD	GTC

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
SAN DIEGO, CA 92127
PHONE: 858.674.6559

DENISON MINES
6425 S. HIGHWAY 191
P.O. BOX 809
BLANDING, UTAH 84511
PHONE: 858.674.6559

TITLE: LINING SYSTEM DETAILS I

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.

SIGNATURE

DATE

DESIGN BY:	GTC	DATE:	DECEMBER 2007
DRAWN BY:	MAD	PROJECT NO.:	SC0349
CHECKED BY:	RF	FILE:	
REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC	5 OF 8	

P:\PRJ\SDCadd\CADD\SC0349\PlanSet_Cell 4b\SC0349_12-30-08\0349C005.dwg

(Attachment H, 1/1)

EXHIBIT D

REVISED COMPARISON OF FLOW THROUGH COMPACTED CLAY LINER AND GEOSYNTHETIC CLAY LINER CALCULATION PACKAGE

Geosyntec consultants

COMPUTATION COVER SHEET

Client: **DMC**	Project: **White Mesa Mill – Cell 4B**	Project/ Proposal No.:	**SC0349**
		Task No.	**02**

Title of Computations: **REVISED - COMPARISON OF FLOW THROUGH COMPACTED CLAY LINER AND GEOSYNTHETIC CLAY LINER.**

Computations by:	Signature		07/01/08 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		7/27/08 Date
	Printed Name	Gregory T. Corcoran	
	Title	Principal Engineer	
Computations Checked by:	Signature		7/2/08 Date
	Printed Name	Rebecca Flynn	
	Title	Senior Staff Engineer	
Computations backchecked by: (originator)	Signature		07/01/08 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Approved by: (pm or designate)	Signature		7/27/08 Date
	Printed Name	Gregory T. Corcoran	
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	M. Lithgow	Date:	06/27/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

COMPARISON OF FLOW THROUGH COMPACTED CLAY LINER AND GEOSYNTHETIC CLAY LINER.

PURPOSE

Evaluate the use of a secondary composite liner system consisting of a 60-mil HDPE geomembrane overlying a geosynthetic clay liner (GCL), to demonstrate equivalent or better fluid migration characteristics when compared with a secondary composite liner system consisting of a 60-mil HDPE geomembrane overlying a 2 ft thick compacted clay liner (CCL) having a saturated hydraulic conductivity less than 1 x 10^{-7} cm/s (USEPA, 1998). The method outlined by Giroud, et al. (1997) will be employed to compare the fluid migration characteristics.

ANALYSIS

Liquid migration through a composite liner occurs essentially through defects in the geomembrane. According to Giroud, et al. (1997) (see Attachment A, p. 1/2), the rate of liquid migration through a defect in the geomembrane component of a composite liner is given by the following semi-empirical equation:

$$Q = 0.21[1+0.1(h/t)^{0.95}]\ a^{0.1}\ h^{0.9}\ k^{0.74} \qquad \text{Equation (1)}$$

where:

Q = flow rate through one geomembrane defect, m^3/s
h = head of liquid above the geomembrane, m
t = thickness of soil component of composite liner, m
a = defect area, m^2
k = hydraulic conductivity of the soil component of composite liner, m/s

Using equation (1), the ratio between the rate of leachate flow through the GCL composite liner system and the CCL composite liner system can be compared as follows (see Attachment A, p. 2/2):

Written by:	M. Lithgow	Date:	06/27/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

$$\frac{q_{compCCL}}{q_{compGCL}} = \left(\frac{k_{CCL}}{k_{GCL}}\right)^{0.74} \frac{1+0.1(h/t_{CCL})^{0.95}}{1+0.1(h/t_{GCL})^{0.95}} \qquad \text{Equation (2)}$$

where:

$q_{comp\ CCL}$ = unit rate of flow through a composite liner where soil component is a CCL

$q_{comp\ GCL}$ = unit rate of flow through a composite liner where soil component is a GCL

EVALUATE FLOW THROUGH COMPOSITE LINER SYSTEMS

The values for the parameters in Equation (2) are discussed below:

Properties of Compacted Clay Liner (CCL):

k_{CCL} = 1 x 10^{-7} cm/s (maximum saturated hydraulic conductivity permitted by the USEPA for compacted clay liners) (USEPA, 1998)

t_{CCL} = 2 ft (0.6 m) (minimum required thickness permitted by the USEPA for compacted clay liners) (USEPA, 1998)

Properties of Geosynthetic Clay Liner (GCL):

k_{GCL} = 1.2 x 10^{-8} cm/s (approximate permeability of GCL with moisture content of 50% after one pore volume of pH 1 liquid)

t_{GCL} = 0.20 in. (5 mm) (minimum thickness reported in manufacturer's documentation and typically allowed in technical specifications for GCLs)

The permeability of the GCL, k_{GCL}, was obtained through laboratory testing conducted by TRI Environmental as part of a field study. The field study was requested by the Utah Department of Environmental Quality (UDEQ) in interrogatory rounds for the approval of Cell 4A. A discussion of the field and laboratory results of the GCL hydration and permeation with a pH 1 liquid is described in a letter reported dated 31 August 2007 (Geosyntec, 2007). Based on the results of this testing, the GCL moisture content of 50% was approved by UDEQ in a letter to Denison Mines (USA) Corp., dated 28 September 2007 (UDEQ, 2007).

Written by:	M. Lithgow	Date:	06/27/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

Head Above Liner, (h):

The maximum liquid head on the secondary composite liner system is derived by assuming that a hole in the primary geomembrane liner system exists to allow liquids to migrate through the primary geomembrane to the leak detection system and create potential head on the secondary geomembrane liner.

According to Giroud, et al. (1997a) (see Attachment B, p. 5/5), the rate of liquid migration through a defect in the geomembrane is given by the following:

$$Q = (2/3)d^2\sqrt{gh_{prim}} \qquad \text{Equation (3)}$$

where:

Q = flow rate through one geomembrane defect, m^3/s
d = defect diameter, 60 mil = 0.0015 m
g = acceleration due to gravity, 9.81 m/sec^2
h_{prim} = head of liquid on top of primary liner, m

According to the EPA, common practice is to assume that the diameter of a leak in the geomembrane is equal to the thickness of the geomembrane (i.e. 60 mil, 0.0015 m).

Based on the proposed grading for Cell 4B (Attachment C, 1/1) and the operational constraint of maintaining 3 feet of freeboard within the cell, the maximum height of liquids above the primary geomembrane will be approximately 37 feet (11.3 m).

Placing the above values into Equation 3 results in the following maximum flow rate:

$$Q = (2/3)(0.0015m)^2\sqrt{(9.81)(11.3m)} \qquad = 1.58 \times 10^{-5}\ m^3/sec$$

Knowing the maximum potential flow rate through a specific defect in the primary geomembrane allows for the calculation of the liquid head build-up on the secondary geomembrane using the following equation from Giroud, et al. (1997a) (see Attachment B, p. 4/5):

Written by:	M. Lithgow	Date:	06/27/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	White Mesa Mill- Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

$$t_0 = \frac{t_{LCL}}{2}\left(1 + \frac{Q}{kt_{LCL}^2}\right)$$ Equation (4)

where:

t_0 = thickness of liquid above geomembrane, m
t_{LCL} = thickness of leak detection layer (geonet), 300 mils (0.0076 m)
Q = flow rate through defect in geomembrane, 1.58×10^{-5} m^3/sec
k = permeability of geonet layer above secondary geomembrane

Attachment D, 2/3 shows a transmissivity curve for a 300 mil thick geonet sandwiched between two HDPE geomembranes. Based on the transmissivity and the thickness of the geonet, a permeability can be estimated for a variety of normal stresses and hydraulic gradient conditions.

Based on the site grading (Attachment C, 1/1), a maximum thickness of waste material (tailings/slimes) of 40 feet will be placed above the liner system. Assuming a unit weight of 125 pcf, a normal stress of approximately 5,000 psf will be exerted on the geonet.

The use of the maximum normal stress (5,000 psf), maximum head (37 feet), and the flow rate (1.58×10^{-5} m^3/s) is conservative for this calculation as the maximum drainage path is located in the shallower end of the cell (31 ft versus 37 feet) where the normal stress, head, and flow rate are lower.

The hydraulic gradient is based on the longest drainage path (780 feet), slope of the geonet (1%), and height of liquid above the liner system at the upgradient end of the flow path (31 feet). Based on this information, the hydraulic gradient can be estimated as follows:

$$i = (31 \text{ ft} + 780 \text{ ft} \times 0.01) / 780 \text{ ft} = 0.050$$

Graphing the permeability data for the 300 mil thick geonet under a normal stress of 5,000 psf (Attachment E, 1/3), results in the following equation of the line:

Written by:	M. Lithgow	Date:	06/27/08	Reviewed by:	G. Corcoran	Date:	7/27/08
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

$k = 0.2415\ i^{-0.422}$ Attachment E

Placing the estimated hydraulic gradient into the above equation results in an estimated permeability of 8.55×10^{-1} m/sec.

Placing the estimated geonet permeability, flow rate through the defect in the primary geomembrane, and the thickness of the leak detection layer geonet into Equation 4 results in the following:

$$t_0 = \frac{0.0076}{2}(1 + \frac{1.58x10^{-5}}{8.55x10^{-1}(0.0076^2)}) = 0.005 \text{ m} = 5 \text{ mm } (0.20 \text{ in.})$$

Flow rates through the CCL and GCL containing composite liner systems are evaluated using Equation (2), assuming a liquid head of 0.20 in. (5 mm). The results of the analysis and a sample calculation are presented below.

Head on secondary liner system	$q_{compCCL}/q_{compGCL}$
0.20 in. (5 mm)	4.37

Sample Calculation:
Plugging the above values for case 1, equation (2) becomes as follows:

$$\frac{q_{compCCL}}{q_{compGCL}} = \left(\frac{k_{CCL}}{k_{GCL}}\right)^{0.74} \frac{1 + 0.1(h / t_{CCL})^{0.95}}{1 + 0.1(h / t_{GCL})^{0.95}}$$
$$= (\frac{1x10^{-7}}{1.2x10^{-8}})^{0.74} \frac{1 + 0.1(0.20 / 24.0)^{0.95}}{1 + 0.1(0.20 / 0.20)^{0.95}}$$
$$= 4.37$$

Thus, for a liquid head of 0.20 in. (5 mm) on the secondary geomembrane, the flow through the secondary composite liner system that includes a CCL is 4.37 times greater than the flow through the secondary liner system that includes a GCL instead of a CCL.

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Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

SUMMARY AND CONCLUSIONS

- Using the method outlined by Giroud, et al. (1997), the flow rate through the secondary liner system with CCL was evaluated to be greater than the flow rate through the proposed secondary liner system (with GCL instead of CCL).

- The amount of flow through the secondary liner system with CCL was evaluated to be 4.37 times greater than flow through the secondary liner system with GCL for a liquid head of 0.20 in. (5 mm).

- In terms of limiting fluid flow through the composite secondary liner system, the liner system containing a GCL performs better than the liner system containing a CCL.

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4 FLOW THROUGH COMPOSITE LINERS

4.1 Introduction

As indicated in Section 2.8, GCLs used in landfills are always used as the low-perme-bility soil component of composite liners. In other words, GCLs used in landfills are ways associated with a geomembrane. The cases discussed in Section 3 were only relevant to the extreme design scenario where the geomembrane is ignored, and to other containment structures where GCLs may be used without a geomembrane.

In Section 4, the geomembrane is not ignored and the effectiveness of composite liners constructed with CCLs and GCLs is compared.

4.2 Rate of Leachate Migration Through Composite Liners With CCL and GCL

Development of Equation. As indicated by Giroud and Bonaparte (1989), liquid migration through a composite liner occurs essentially through defects of the geomembrane. According to Giroud (1997), the rate of liquid migration through a defect in the geomembrane component of a composite liner is given by the following semi-empirical equation:

$$Q = 0.21\,[1 + 0.1(h/t)^{0.95}]\,a^{0.1}\,h^{0.9}\,k^{0.74} \tag{51}$$

where: Q = flow rate through one geomembrane defect; h = head of liquid above the geomembrane; t = thickness of the soil component of the composite liner; a = defect area; and k = hydraulic conductivity of the soil component of the composite liner. It is important to note that Equation 51 can only be used with the following units: a (m^2), h (m), t (m), k (m/s).

As discussed in Sections 2.5 and 2.6, there are cases where it is prescribed by regulations, or simply envisioned by design engineers, to place a GCL on a layer of soil with a low hydraulic conductivity such as 1×10^{-8} or 1×10^{-7} m/s. An important conclusion from Section 3, is that, if a GCL is placed on a soil layer (even a soil layer with low ermeability), the soil layer has no influence on leachate advective flow and only the GCL should be considered in leachate flow calculations. The same conclusion applies to the soil component of a composite liner. Accordingly, if, in a composite liner, a GCL is placed on a layer of low-permeability soil, only the GCL will be considered in Equation 51.

Using Equation 51, the ratio between the rate of leachate flow through a composite liner with a CCL and a composite liner with a GCL is as follows:

$$\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = \frac{0.21\,N\,[1 + 0.1(h/t_{CCL})^{0.95}]\,a^{0.1}\,h^{0.9}\,k_{CCL}^{0.74}}{0.21\,N\,[1 + 0.1(h/t_{GCL})^{0.95}]\,a^{0.1}\,h^{0.9}\,k_{GCL}^{0.74}} \tag{52}$$

where: $q_{comp\ CCL}$ = unit rate of flow through a composite liner where the soil component is a CCL; $q_{comp\ GCL}$ = unit rate of flow through a composite liner where the soil component is a GCL; t_{CCL} = thickness of the CCL in the composite liner; t_{GCL} = thickness of the GCL

in the composite liner;
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Discussion. It appear
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outperforms the standa
7 m depending on the
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Table 7. Ratio betweer and a composite liner ir

GCL characteristics: Thickness, t_{GCL} (mm) Hydraulic conductivity, k_{GC}	
	(m)
Leachate head on top of the liner, h	0
	0.01
	0.05
	0.1
	0.3
	0.6
	1.0
	3.0
	5.0
	7.0
	10
	∞

Notes: The tabulated valu
following CCL characteristic
is the standard CCL defined

ATTACHMENT A 1/2

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will be considered in Equa-

e flow through a composite
follows:

$$\frac{{}^{0.1}\ h^{0.9}\ k_{CCL}^{0.74}}{{}^{0.1}\ h^{0.9}\ k_{GCL}^{0.74}} \qquad (52)$$

er where the soil component
er where the soil component
; t_{GCL} = thickness of the GCL

in the composite liner; and N = number of geomembrane defects per unit area. After simplification, Equation 52 becomes:

$$\frac{q_{comp\ CCL}}{q_{comp\ GCL}} = \left(\frac{k_{CCL}}{k_{GCL}}\right)^{0.74} \frac{1 + 0.1(h / t_{CCL})^{0.95}}{1 + 0.1(h / t_{GCL})^{0.95}} \qquad (53)$$

Discussion. It appears that the leachate flow rate ratio expressed by Equation 53 does not depend on the number and the size of defects. Numerical values of $q_{comp\ CCL}/q_{comp\ GCL}$ calculated using Equation 53 are presented in Table 7. It appears that, for leachate heads typically encountered in landfills (i.e. heads smaller than 0.3 m, and generally smaller than 0.1 m), the calculated advective flow control performance of a composite liner which consists of a geomembrane on a GCL is significantly better than the calculated advective flow control performance of the standard composite liner which consists of a geomembrane on the standard CCL (i.e. a CCL with a thickness of 0.6 m and a hydraulic conductivity of 1×10^{-9} m/s). Table 7 also shows that a composite liner with a GCL outperforms the standard composite liner for leachate heads up to approximately 1 to 7 m depending on the GCL hydraulic conductivity; such large heads should be a very rare occurrence in a landfill since they would correspond to a major malfunction of the leachate collection and removal system.

Table 7. Ratio between rates of advective flow through a composite liner including a CCL and a composite liner including a GCL, $q_{comp\ CCL}/q_{comp\ GCL}$.

GCL characteristics: Thickness, t_{GCL} (mm) Hydraulic conductivity, k_{GCL} (m/s)			5 5×10^{-12}	7 1×10^{-11}	9 5×10^{-11}
	(m)	(mm)	$q_{comp\ CCL}/q_{comp\ GCL}$	$q_{comp\ CCL}/q_{comp\ GCL}$	$q_{comp\ CCL}/q_{comp\ GCL}$
Leachate head on top of the liner, h	0	0	50.44	30.20	9.18
	0.01	10	42.36	26.54	8.28
	0.05	50	26.92	18.50	6.14
	0.1	100	18.87	13.66	4.71
	0.3	300	9.01	6.98	2.54
	0.6	600	5.31	4.23	1.58
	1.0		3.59	2.89	1.09
	3.0		1.65	1.35	0.52
	5.0		1.23	1.01	0.39
	7.0		1.04	0.85	0.33
	10		0.90	0.74	0.28
	∞		0.53	0.44	0.17

Notes: The tabulated values of the advective flow rate ratio were calculated using Equation 53 with the following CCL characteristics: thickness, t_{CCL} = 0.6 m; and hydraulic conductivity, $k_{CCL} = 1 \times 10^{-9}$ m/s. (This is the standard CCL defined in Section 2.5.) The characteristics of the GCL are from Table 2.

ATTACHMENT A. 2/2

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Technical Paper by J.P. Giroud, B.A. Gross, R. Bonaparte and J.A. McKelvey

Leachate Flow in Leakage Collection Layers Due to Defects in Geomembrane Liners

ABSTRACT: This paper provides analytical and graphical solutions related to the flow of leachate in a leakage collection layer due to defects in the overlying liner (i.e. the primary liner of a double liner system). The defects are assumed to be small (e.g. holes in geomembrane liners). It is shown that leachate flows in a zone of the leakage collection layer (the wetted zone) that is limited by a parabola. A simple relationship is established between the rate of leachate migration through the defect and the maximum thickness of leachate in the leakage collection layer; this relationship depends on the hydraulic conductivity (but not on the slope) of the leakage collection layer. Equations are provided to calculate the average head of leachate on top of the liner underlying the leakage collection layer (i.e. the secondary liner of a double liner system), which is useful for calculating the rate of leachate migration through that liner. Finally, the case of several leaks randomly distributed is considered, and equations for the surface area of the wetted zone and the average head are given for this case. Parametric analyses and design examples provide useful comparisons between the three types of materials used in leakage collection layers: gravel, sand and geonets.

KEYWORDS: Geomembrane, Defect, Leachate migration, Leachate collection, Leakage, Leakage collection, Liner system, Double liner, Geosynthetic leakage collection layer.

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ATTACHMENT B 1/5

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part, which is unsaturated. Then, when the leachate reaches the saturated portion of the leakage collection layer, it first flows in all directions (Figure 1). It is therefore logical to assume that the leachate phreatic surface in the leakage collection layer is a cone with its apex at Point A located vertically beneath the defect in the primary liner (Figure 4). Furthermore, for leachate to flow in all directions, the hydraulic gradient must be

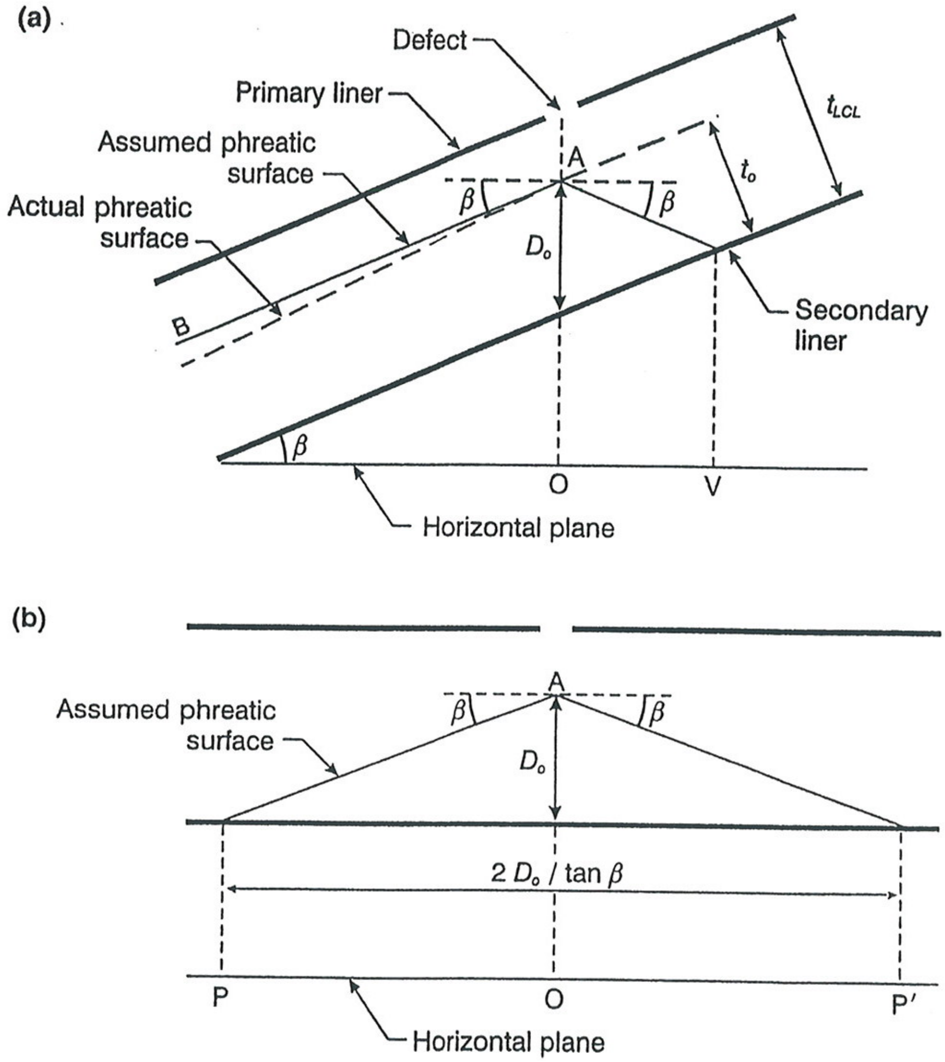

Figure 4. Assumed phreatic surface in the leakage collection layer in the case where the leakage collection layer is not filled with leachate: (a) cross section in a vertical plane along the slope and passing through the defect in the primary liner; (b) cross section in a vertical plane perpendicular to the plane of the preceding cross section and passing through the defect.

ATTACHMENT B, 2/5

It appears that, when the leakage collection layer is not full, there is an extremely simple relationship between the rate of leachate migration through the primary liner defect, Q, and the thickness of leachate in the leakage collection layer beneath the defect, t_o. It is interesting to note that this relationship does not depend on the size of the defect in the primary liner or on the slope of the leakage collection layer.

An approximation that was made to establish Equations 9 and 10 was to assume that the downslope flow line from A (i.e. AB in Figure 4a) is parallel to the liner. This assumption is close to reality as discussed in Section 2.2. However, the actual flow line from A is below Line AB as the flow thickness decreases in the downslope direction, as discussed at the end of Section 5.1.2. Therefore, t_o should only be regarded as the flow thickness at a primary liner defect, and it is the maximum flow thickness.

Since the simple relationship expressed by Equations 9 and 10 was demonstrated for the case when the leakage collection layer is not full, the condition expressed by Equation 1 must be met for Equations 9 and 10 to be valid. Combining Equations 1 and 10 gives the following equation, which is another way to express the condition that should be met to ensure that the leakage collection layer is not full:

$$t_{LCL} \geq t_{LCLfull} = \sqrt{\frac{Q}{k}} \tag{11}$$

where $t_{LCLfull}$ is the *minimum* thickness that a leakage collection layer with a hydraulic conductivity k should have to contain, without being full at any location, the leachate flow which results from a defect in the primary liner.

The following equation, derived from Equation 11, is another way to express the condition that should be met to ensure that the leakage collection layer is not full:

$$Q \leq Q_{full} = k\, t_{LCL}^2 \tag{12}$$

where Q_{full} is the *maximum* steady-state rate of leachate migration through a defect in the primary liner that a leakage collection layer, with a thickness t_{LCL} and a hydraulic conductivity k, can accommodate without being filled with leachate.

It is important to remember that the subscript *full* corresponds to a *minimum* thickness of the leakage collection layer and to a *maximum* rate of leachate migration (which is also the *maximum* flow rate in the leakage collection layer). It is noteworthy that the minimum thickness of the leakage collection layer, $t_{LCLfull}$, and the maximum flow rate, Q_{full}, which are required to ensure that the leakage collection layer can contain, without being full, the flow that results from a defect in the primary liner, do not depend on the slope of the leakage collection layer.

It is not impossible to design a leakage collection layer with a thickness less than the value $t_{LCLfull}$ given by Equation 11, i.e. where the flow rate is greater than Q_{full} defined by Equation 12. In this case, the leakage collection layer is filled with leachate in a certain area around the defect of the primary liner (i.e. "the leachate collection layer is full"). This case is discussed in Section 3.2.

3.2 Rate of Leachat

If the thickness of the l
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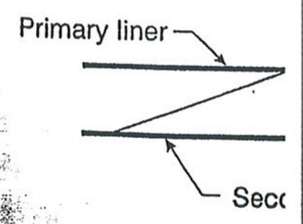

Figure 5. Vertical cross sec
layer in the case where the le
around the primary liner de

ATTACHMENT B 3/5

Knowing (or assuming) the leachate head, h_o, on top of the secondary liner vertically beneath the primary liner defect, one may derive the virtual leachate thickness, t_o, using Equation 4. Then, knowing t_o, t_{LCL} and k, one may use Equation 16 to calculate the rate of leachate flow through a defect that the leakage collection layer can convey.

The following equation can be derived from Equation 16:

$$t_o = \frac{t_{LCL}}{2}\left(1 + \frac{Q}{k\, t_{LCL}^2}\right) \tag{17}$$

The following equation can be derived from Equations 13 and 16:

$$t_{LCL} = t_o\left(1 - \sqrt{1 - \frac{Q}{k\, t_o^2}}\right) \tag{18}$$

Equation 18 is valid only if the following condition is met:

$$Q \le k\, t_o^2 \tag{19}$$

It should be noted that if $t_{LCL} = t_o$, i.e. if the leakage collection layer is filled with leachate at only one point, i.e. at the location of the primary liner defect, Equation 16 is equivalent to Equation 9.

3.3 Parametric Study

Using the equations presented in Sections 3.1 and 3.2 it is possible to compare the flow capacity of different leakage collection layers in case of a defect in the primary liner. In Table 1, three different leakage collection layers are compared:

- a geonet with a thickness of 5 mm and a hydraulic transmissivity resulting in a hydraulic conductivity (obtained by dividing the hydraulic transmissivity by the thickness) of 1×10^{-1} m/s;
- a gravel layer with a thickness of 300 mm and a hydraulic conductivity of 1×10^{-1} m/s; and
- a sand layer with a thickness of 300 mm and a hydraulic conductivity of 1×10^{-3} m/s.

The first two leakage collection layers have the same hydraulic conductivity and the last two have the same thickness. In the case of the geonet, the virtual leachate thickness, t_o, considered in Table 1 is greater than, or equal to, the thickness of the leachate collection layer, t_{LCL}; therefore, in all cases considered in Table 1, the geonet is filled with leachate over a certain area around the defect (this area being zero for $t_o = 5$ mm). In the case of the gravel and sand layers, the leachate thicknesses considered in Table 1 are less than, or equal to, the thickness of the leakage collection layer; therefore, in all cases considered in Table 1, the gravel and sand layer are not filled (or just filled) with leachate, and for these two materials the leachate thicknesses, t_o, shown in Table 1 are actual (not virtual) thicknesses.

Table 1. Rate of leachat
defect in the primary lin

Leachate thickness (actual or virtual)		
t_o		
(m)	(mm)	
0.005	5	
0.01	10	
0.05	50	4
0.1	100	9
0.3	300	2

Notes: The leachate thickne
Equation 4. The leachate thick
if $t_o > t_{LCL}$. The tabulated val
t_{LCL} and Equation 16 when t_o

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$$Q = 0.6\, a \sqrt{2}$$

where: a = defect area;
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Table 2 gives rates o
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Attachment B, 4/5

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Table 1. Rate of leachate flow in three different leachate collection layers resulting from a defect in the primary liner.

Leachate thickness (actual or virtual)		Leakage collection layer material					
		Geonet t_{LCL} = 5 mm $k = 1 \times 10^{-1}$ m/s		Gravel t_{LCL} = 300 mm $k = 1 \times 10^{-1}$ m/s		Sand t_{LCL} = 300 mm $k = 1 \times 10^{-3}$ m/s	
t_o		Q		Q		Q	
(m)	(mm)	(m³/s)	(lpd)	(m³/s)	(lpd)	(m³/s)	(lpd)
0.005	5	2.5×10^{-6}	216	2.5×10^{-6}	216	2.5×10^{-8}	2.16
0.01	10	7.5×10^{-6}	648	1.0×10^{-5}	864	1.0×10^{-7}	8.64
0.05	50	4.75×10^{-5}	4,104	2.5×10^{-4}	21,600	2.5×10^{-6}	216
0.1	100	9.75×10^{-5}	8,424	1.0×10^{-3}	86,400	1.0×10^{-5}	864
0.3	300	2.975×10^{-4}	25,704	9.0×10^{-3}	777,600	9.0×10^{-5}	7,776

Notes: The leachate thickness, t_o, can be derived from the leachate head on top of the secondary liner using Equation 4. The leachate thickness, t_o, is the actual leachate thickness if $t_o < t_{LCL}$ and a virtual leachate thickness if $t_o > t_{LCL}$. The tabulated values of the rate of leachate flow, Q, were calculated using Equation 9 when $t_o < t_{LCL}$ and Equation 16 when $t_o > t_{LCL}$. Units: 1 m³/s = 86,400,000 liters per day (lpd).

It appears from Table 1, that for a given value of t_o; i.e. a given value of the head of leachate on top of the secondary liner, h_o (see Equation 4), the gravel and the geonet can convey significantly more leachate than the sand. It is interesting to compare the flow rates of Table 1 with rates of leachate migration through defects of geomembranes used alone (i.e. not part of a composite liner) calculated using Bernoulli's equation, which is expressed as follows:

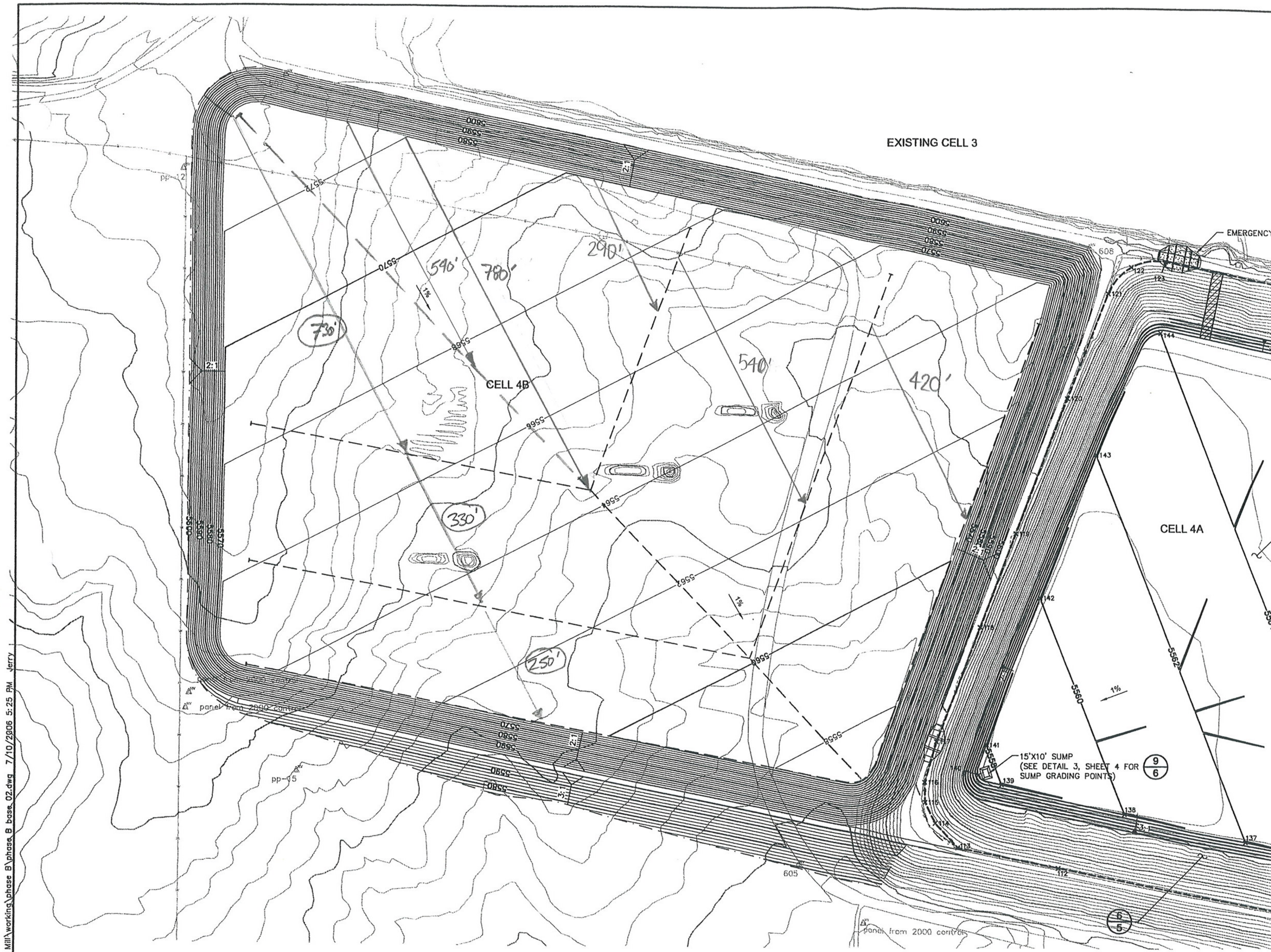
$$Q = 0.6\,a\sqrt{2\,g\,h_{prim}} = 0.6\,\pi\,(d^2/4)\,\sqrt{2\,g\,h_{prim}} \approx (2/3)\,d^2\,\sqrt{g\,h_{prim}} \qquad (20)$$

where: a = defect area; d = defect diameter; g = acceleration due to gravity; and h_{prim} = head of leachate on top of the primary liner.

Table 2 gives rates of leachate migration through geomembrane defects calculated using Equation 20. It appears that, with the leachate heads that typically exist on the primary liners of actively operating landfills (i.e. landfills that are receiving waste), and provided that the geomembrane is used alone (i.e. is not part of a composite liner):

- a small geomembrane defect (e.g. 1 to 2 mm diameter), which may occasionally be undetected during construction, results in a rate of leakage on the order of 100 liters per day (lpd);
- a geomembrane defect (e.g. 3 to 5 mm diameter), which may occasionally occur during construction phases where defect detection may not be possible (e.g. placement of granular leachate collection material on geomembrane), results in a rate of leakage on the order of 1000 lpd (1 m³/day); and
- a large geomembrane defect (e.g. 10 mm diameter or more), which may occur under special circumstances, results in a rate of leakage of 10,000 lpd (10 m³/day) or more.

ATTACHMENT B 5/5



LEGEND

5560	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (2' CONTOUR)
1%	SLOPE DIRECTION AND GRADE

* Most critical path = longest path

* 780' is longest path

N

100 50 0 100 200
SCALE IN FEET

ATTACHMENT C

GEOSYNTEC CONSULTANTS
3990 OLD TOWN AVENUE, SUITE B-101
SAN DIEGO, CALIFORNIA 92110
TELEPHONE: (619) 297-1530

PROJECT: WHITE MESA MILL			
TITLE: PHASE 4B GRADING			
DATE: OCT 2006	CHECKED BY:	SCALE: 1"= 100'	FIGURE NO:
DESIGN BY:	REVIEWED BY: (PROJ. MGR.)	JOB NO.: SC0349	

P:\CAD\SC0349 – White Mesa Mill\working\phase B\phase_B_base_02.dwg 7/10/2006 5:25 PM Jerry Amos

The GSE Drainage Design Manual
Second Edition

Co-Authors:
Dhani Narejo, Ph.D., P.E., Caro Engineering
Robert Bachus, Ph.D., P.E., GeoSyntec Consultants
Richard Thiel, P.E., Thiel Engineering
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Mengjia Li, Ph.D., P.E., GSE Lining Technology, Inc.

June 2007

Attachment D, 1/3

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Chapter 2

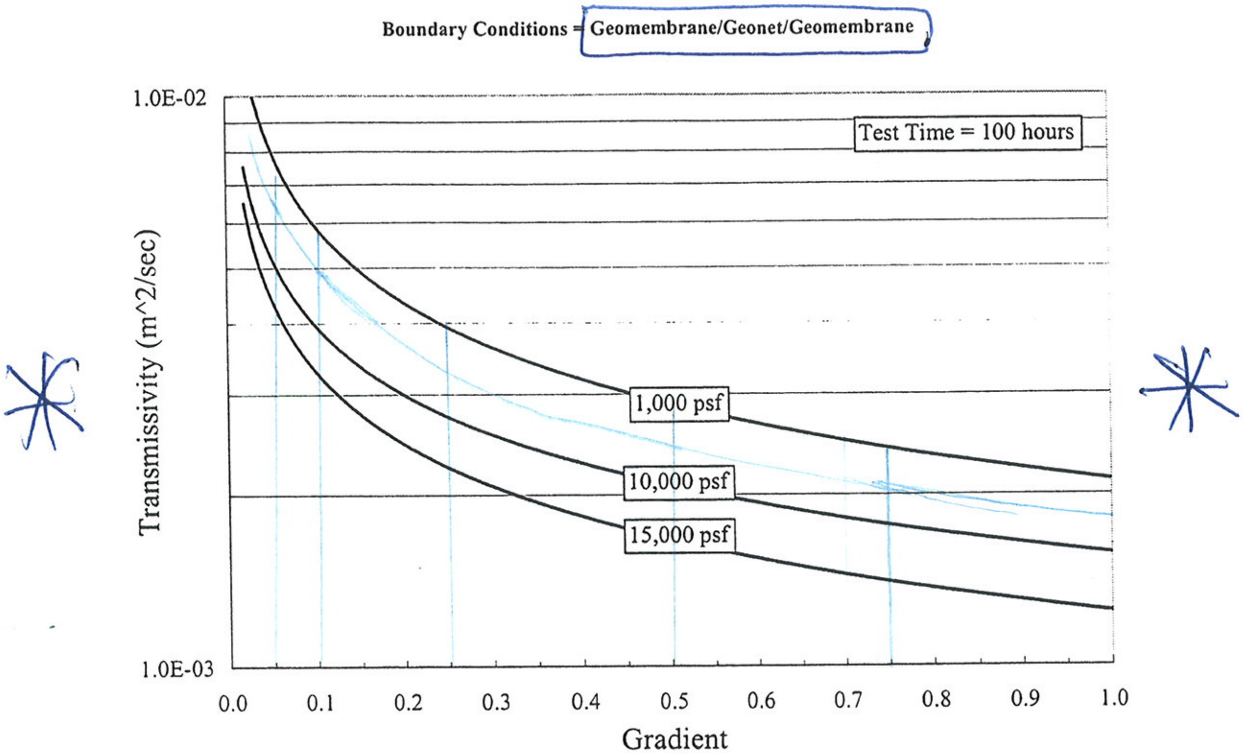


Figure A-7. Performance Transmissivity of a 300 mil GSE HyperNet UF geonet.

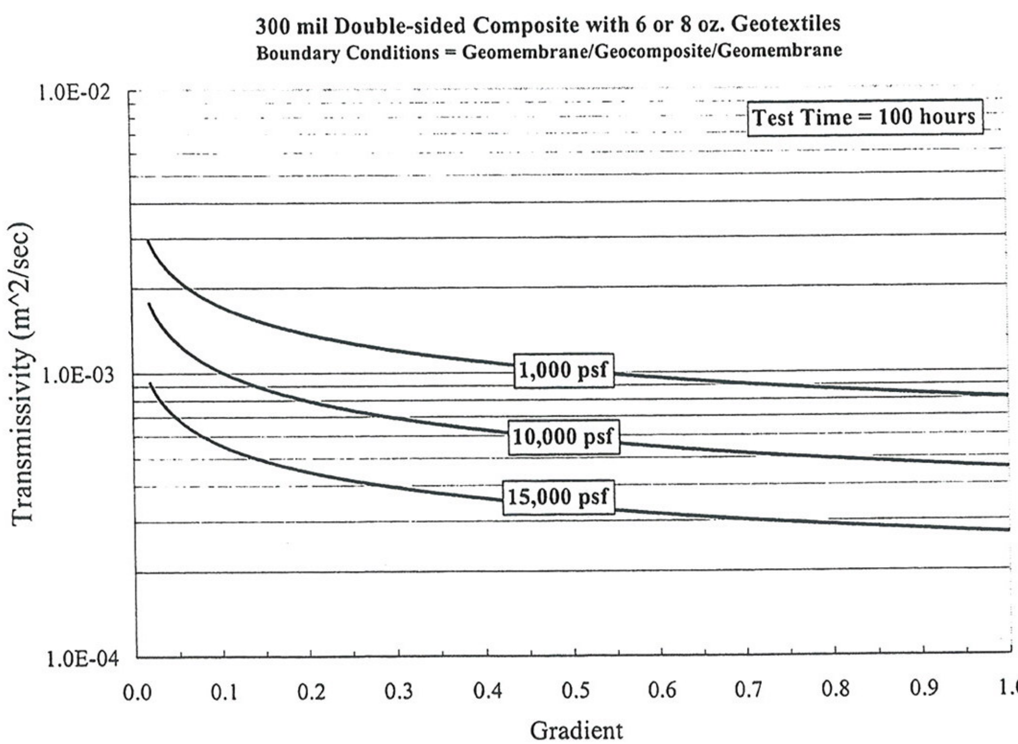


Figure A-8. Performance Transmissivity of a 300 mil GSE FabriNet UF geocomposite between Plates.

(Attachment D, 2/3)

GSE®

GSE STANDARD PRODUCTS

Product Data Sheet

GSE HyperNet, HF, HS and UF Geonets

GSE HyperNet geonets are synthetic drainage materials manufactured from a premium grade high density polyethylene (HDPE) resin. The structure of the HyperNet geonet is formed specifically to transmit fluids uniformly under a variety of field conditions. HDPE resins are inert to chemicals encountered in most of the civil and environmental applications where these materials are used. GSE geonets are formulated to be resistant to ultraviolet light for time periods necessary to complete installation. GSE HyperNet geonets are available in standard, HF, HS, and UF varieties.

The table below provides index physical, mechanical and hydraulic characteristics of GSE geonets. Contact GSE for information regarding performance of these products under site-specific load, gradient, and boundary conditions.

Product Specifications

TESTED PROPERTY	TEST METHOD	FREQUENCY	MINIMUM AVERAGE ROLL VALUE[b]			
			HyperNet	HyperNet HF	HyperNet HS	HyperNet UF
Product Code			XL4000N004	XL5000N004	XL7000N004	XL8000N004
Transmissivity[a], gal/min/ft (m^2/sec)	ASTM D 4716	1/540,000 ft^2	9.66 (2 x 10^{-3})	14.49 (3 x 10^{-3})	28.98 (6 x 10^{-3})	38.64 (8 x 10^{-3})
Thickness, mil (mm)	ASTM D 5199	1/50,000 ft^2	200 (5)	250 (6.3)	275 (7)	* 300 (7.6)
Density, g/cm^3	ASTM D 1505	1/50,000 ft^2	0.94	0.94	0.94	0.94
Tensile Strength (MD), lb/in (N/mm)	ASTM D 5035	1/50,000 ft^2	45 (7.9)	55 (9.6)	65 (11.5)	75 (13.3)
Carbon Black Content, %	ASTM D 1603*/4218	1/50,000 ft^2	2.0	2.0	2.0	2.0
Roll Width[c], ft (m)			15 (4.6)	15 (4.6)	15 (4.6)	15 (4.6)
Roll Length[c], ft (m)			300 (91)	250 (76)	220 (67)	200 (60)
Roll Area, ft^2 (m^2)			4,500 (418)	3,750 (348)	3,300 (305)	3,000 (278)

NOTES:

- [a]Gradient of 0.1, normal load of 10,000 psf, water at 70° F (20° C), between steel plates for 15 minutes.
- [b]These are MARV values that are based on the cumulative results of specimens tested by GSE.
- [c]Roll widths and lengths have a tolerance of ±1%.
- *Modified.

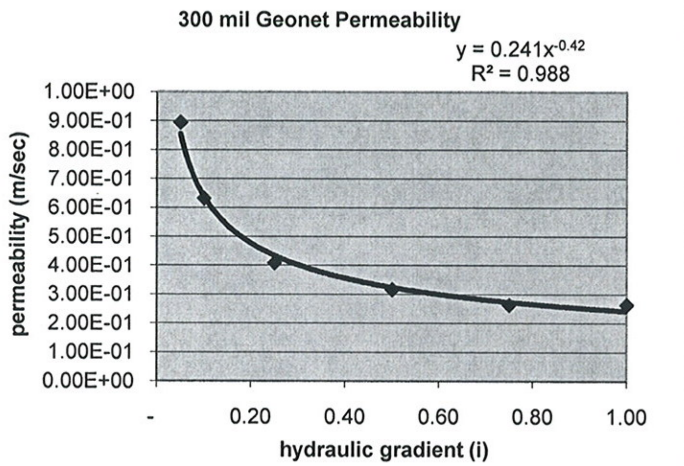
DS017 HyperNet R01/07/08

North America	GSE Lining Technology, Inc.	Houston, Texas	800.435.2008	281.443.8564	Fax: 281.230.6739
South America	GSE Lining Technology Chile S.A.	Santiago, Chile		56.2.595.4200	Fax: 56.2.595.4290
Asia Pacific	GSE Lining Technology Company Limited	Bangkok, Thailand		66.2.937.0091	Fax: 66.2.937.0097
Europe & Africa	GSE Lining Technology GmbH	Hamburg, Germany		49.40.767420	Fax: 49.40.7674234
Middle East	GSE Lining Technology-Egypt	The 6th of October City, Egypt		20.2.828.8888	Fax: 20.2.828.8889

www.gseworld.com

Attachment D, 3/3

GSE 300 mil Geonet (HDPE/GN/HDPE)					
i	Normal Stress (psf)	Transmissivity (m2/sec)	Thickness (mil)	Thickness (mm)	Permeability (m/sec)
0.05	5,000	6.80E-03	300	7.6	8.95E-01
0.10	5,000	4.80E-03	300	7.6	6.32E-01
0.25	5,000	3.10E-03	300	7.6	4.08E-01
0.50	5,000	2.40E-03	300	7.6	3.16E-01
0.75	5,000	2.00E-03	300	7.6	2.63E-01
1.00	5,000	1.80E-03	300	7.6	2.63E-01



$$Q = (2/3)\, d^2\, (g\, h_{prim})^{1/2}$$

Leachate Flow Through Geomembrane Defect			
Value	**Units**	**Variable**	**Definition**
60	mil	d	defect diameter (EPA HELP Model assumes equal to thickness of geomembrane)
1.524	mm	d	defect diameter
0.0015	m	d	defect diameter
9.8067	m/s2	g	acceleration due to gravity
37	ft	h_{prim}	head of leachate on top of the primary liner (max height of 40' - freeboard of 3')
11.2776	m	h_{prim}	head of leachate on top of the primary liner
Q =	**1.63E-05**	**m3/sec**	

[ ] = Input Value

ATTACHMENT E, 2/3

$$t_0 = (t_{LCL}/2)\ (1 + (Q/(k * t_{LCL}^2)))$$

Head Above Secondary Liner

Value	Units	Variable	Definition
300	mil	t_{LCL}	thickness of leachate collection layer
0.0076	m	t_{LCL}	thickness of leachate collection layer
1	%	β	angle of slope of the leachage collection layer
0.01	degrees	β	angle of slope of the leachage collection layer
1.63E-05	m^3/sec	Q	steady state rate of leachate flow in the leakage collection layer
31	ft	h_{prim}	head of leachate on top of the primary liner (max height of 40' - freeboard of 3')
780	ft	L	maximum drainage length
0.050		i	hydraulic gradient
8.57E-01	m/sec	k	permeability of geonet at hydraulic gradient in cell
t_0 =	**0.005**	**m**	maximum thickness of leachate in the leakage collection layer
$t_{LCL\ Full}$ =	**0.004**	**m**	maximum thickness of collection layer

= Input Value

(Attachment E, 3/3)

EXHIBIT E

REVISED ACTION LEAKAGE RATE CALCULATION PACKAGE

Geosyntec consultants

COMPUTATION COVER SHEET

Client:	DMC	Project:	White Mesa Mill – Cell 4B	Project/ Proposal No.: Task No.	SC0349

Title of Computations: **REVISED ACTION LEAKAGE RATE**

Computations by:	Signature	Meghan Lithgow	Date 9/22/08
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature	Gregory T. Corcoran	Date 9/23/08
	Printed Name	Gregory T. Corcoran, P.E.	
	Title	Principal Engineer	
Computations Checked by:	Signature	Gregory T. Corcoran	Date 9/23/08
	Printed Name	Gregory T. Corcoran, P.E.	
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature	Meghan Lithgow	Date 9/22/08
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Approved by: (pm or designate)	Signature	Gregory T. Corcoran	Date 9/23/08
	Printed Name	Gregory T. Corcoran, P.E.	
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	09/22/08	Reviewed by:	G. Corcoran	Date:	9/23/08
Client:	DMC	Project:	Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	

CALCULATION OF ACTION LEAKAGE RATE THROUGH THE LEAKAGE DETECTION SYSTEM UNDERLYING A GEOMEMBRANE LINER.

OBJECTIVE

In accordance with Part 254.302 of the USEPA Code of Federal Regulations, determine the action leakage rate (ALR) that a leak detection system (LDS) can remove, and not allow the maximum fluid head on the bottom liner to exceed 1 foot. The ALR shall be given as an average daily flow rate in gallons per day per acre for each sump associated with the LDS. The calculation shall include a margin of safety sufficient to allow for design uncertainties, operational changes, and material characteristics. The LDS shall have a surface area of approximately 45 acres, and consist of a 300-mil thick geonet and a network of gravel trenches that contain 4-inch diameter perforated PVC pipe, drainage aggregate, and a cushion geotextile. There shall be one sump associated with the LDS. The primary liner shall consist of a smooth 60-mil HDPE geomembrane.

The method outlined by Giroud, et al. (1997) will be employed to calculate the ALR and confirm the maximum expected head.

ANALYSIS

Liquid flow through defect in primary geomembrane

Liquid migration through a liner occurs essentially through defects in the geomembrane. According to Giroud, et al. (1997) (see Attachment A, 3/6) the rate of liquid migration through a defect in the geomembrane is given by the following:

$$Q = (2/3)d^2\sqrt{gh_{prim}} \qquad \text{Equation (1)}$$

where:

- Q = flow rate through one geomembrane defect, m^3/s
- d = defect diameter, m
- g = acceleration due to gravity, 9.81 m/sec^2
- h_{prim} = head of liquid on top of primary liner, m

According to the EPA, common practice is to assume that the diameter of a leak in the geomembrane is equal to the thickness of the geomembrane (i.e. 60 mil, 0.0015 m).

Based on the proposed grading for Cell 4B (Attachment B, 1/1) and the operational constraint of maintaining 3 feet of freeboard within the cell, the maximum height of liquids above the primary geomembrane will be approximately 37 feet (11.3 m). Placing the above values into Equation 1 results in the following maximum flow rate per defect:

$$Q = (2/3)(0.0015m)^2\sqrt{(9.81)(11.3m)} = 1.58 \times 10^{-5} \text{ m}^3/\text{sec}$$
$$= 1.36 \text{ m}^3/\text{day}$$
$$= 359.27 \text{ gal/day}$$

Maximum flow rate within geonet

According to Giroud, et al. (1997) (see Attachment A, p. 2/6) the maximum flow rate within the leak detection layer geonet is given by the following:

$$Q_{full} = k\, t_{LCL}^2 \qquad \text{Equation (2)}$$

Where:

Q_{full} = maximum flow rate within the geonet; *to be determined, m³/sec*
k = hydraulic conductivity of geonet; *see below, m/sec*
t_{LCL} = thickness of leak detection layer; *300 mil, 0.0076 m*

Hydraulic conductivity of Geonet, k

Attachment C, 2/2 shows a transmissivity curve for a 300 mil thick geonet sandwiched between two HDPE geomembranes tested for a duration of 100 hours. Based on the transmissivity and the thickness of the geonet, a hydraulic conductivity can be estimated for a variety of normal stresses and hydraulic gradient conditions.

Written by:	R. Flynn	Date:	09/22/08	Reviewed by:	G. Corcoran	Date:	9/23/08
Client: DMC	Project: Cell 4B	Project/ Proposal No.:	SC0349	Task No.:			

Based on the site grading (Attachment B, 1/1), a maximum thickness of waste material (tailings/slimes) and final cover system of 45 feet will be placed above the liner system. Assuming a unit weight of 125 pounds per cubic feet (pcf), a normal stress of approximately 5,625 pounds per square foot (psf) will be exerted on the geonet.

Graphing the permeability data for the 300 mil thick geonet under a normal stress of approximately 6,000 psf (Attachment D, 1/1), results in the following equation of the line:

$$k = 0.2415\ i^{(-0.4221)} \qquad \text{Equation (3)}$$

The hydraulic gradient is based on the longest drainage path (780 feet), slope of the geonet (1%), and height of liquid above the liner system at the deepest point along the flow path (5,597-5566)= 31 feet, which accounts for the 3 foot freeboard). Based on this information, the hydraulic gradient can be estimated as follows:

$$i = (31 \text{ ft} + 780 \text{ ft} \times 0.01) / 780 \text{ ft} = 0.050$$

Placing the estimated hydraulic gradient of 0.050 into Equation 3 results in a hydraulic conductivity of 0.86 m/sec.

Accounting for intrusion (RF_{IN}), creep (RF_{CR}), chemical clogging (RF_{CC}), and biological clogging (RF_{BC}), Koerner (Attachment E, 3/3) suggests the following partial factor of safety values for secondary leak detection systems:

RF_{IN}	1.5 to 2.0	use 1.0 (no geotextiles on either side to intrude, test data accounts for geomembrane intrusion)
RF_{CR}	1.4 to 2.0	use 1.4 (low normal stress, 100 hour transmissivity data)
RF_{CC}	1.5 to 2.0	use 2.0 (very low pH)
RF_{BC}	1.5 to 2.0	use 1.0 (very low pH should preclude biological activity)

Applying these values to the hydraulic conductivity results in the following:

$\mathbf{k} = (0.86 \text{ m/sec}) / (1.0 \times 1.4 \times 2.0 \times 1.0) = \mathbf{3.10 \times 10^{-1}\ m/sec}$

Written by:	R. Flynn	Date:	09/22/08	Reviewed by:	G. Corcoran	Date:	9/23/08
Client:	DMC	Project:	Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	

Placing the geonet hydraulic conductivity and thickness into Equation 2 results in the following:

Q_{full} = (0.310 m/sec) x (0.0076 m)^2 = **1.79 x 10^{-5} m^3/sec**

Based on the anticipated flow through defects in the primary geomembrane and the allowable maximum flow rate within the geonet, the following overall factor of safety results:

FS = (1.79 x 10^{-5} m^3/sec) / (1.58 x 10^{-5} m^3/sec) = **1.1**

Therefore, the proposed 300 mil thick geonet leak detection layer can accommodate the anticipated flow through defects in the primary geomembrane.

Action Leakage Rate (ALR)

The number of defects in a geomembrane is given by Giroud, et al (Attachment A, 4/6), as the following:

$N = (F)(A_{LCL})$ Equation (4)

where:

N = number of defects
F = frequency of defects (per m^2 of geomembrane)
A_{LCL} = area of leakage collection layer; *45 acres, 182,109 m^2*

Using an assumed F = 1/2,500 m^2 (Attachment A, 4/6), the number of defects assumed in the primary geomembrane is as follows:

$$N = \frac{1 \text{ defect}}{2{,}500 \text{ m}^2} \times (182{,}109 \text{m}^2) = 73 \text{ (rounded up to nearest whole number)}$$

ALR = $(Q)(N)$/acre

Written by:	R. Flynn	Date:	09/22/08	Reviewed by:	G. Corcoran	Date:	9/23/08
Client:	DMC	Project:	Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	

$$= \frac{(1.36\text{m}^3/\text{day})(73)}{45 \text{ acres}} \qquad = 2.2 \text{ m}^3/\text{day/acre}$$

$$\mathbf{= 581\ gal/day/acre}$$

Maximum flow rate to sump

Based on the area of the Cell 4B liner system, the following maximum flow rate to the sump is anticipated:

Q_{sump} = (581 gal/day/acre) (45 acres) = 26,145 gal/day = 18.1 gpm

A sump pump capable of a minimum flow rate of 20 gallons per minute at the head conditions present (approximately 42 vertical feet plus piping losses) will be utilized to remove liquids from the LDS.

Time of travel

According to Giroud, et al. (1997) (see Attachment A, 6/6) the travel time for the liquid to reach the LDS piping system from the defect in the primary geomembrane is given by the following:

$t_{travel} = (nx) / (k \sin \beta \cos \beta)$ Equation (5)

where:

t_{travel} = time for liquid to travel from defect in primary geomembrane to the LDS piping; *to be determined, sec*

n = porosity of geonet; *0.8*

x = distance from defect in primary geomembrane to LDS piping; *780 ft, 238 m*

k = hydraulic conductivity of the geonet; *0.311 m/sec from above*

β = slope of floor; *1%, 0.573 degrees*

Substituting the values into equation 5 results in the following:

t_{travel} = (0.8) (238 m) / (0.310 m/sec) (sin 0.573) (cos 0.573) = 61,419 sec = 17.1 hours

Therefore, the leak detection system geonet will allow for timely detection of liquids.

Head Above Liner, (h):

Knowing the maximum potential flow rate through a specific defect in the primary geomembrane, and assuming a worst case condition where all primary liner defects are located at the higher end of the leakage collection layer slope, liquid head build-up on the secondary geomembrane is calculated using the following equation from Giroud, et al. (1997) (see Attachment A, 5/6):

$$t_{avgworst} = \frac{NQ}{kiB} \qquad \text{Equation (6)}$$

where:

$t_{avg\ worst}$ = average thickness of liquid above secondary (bottom) geomembrane under worst case scenario; *to be determined, m*

N = total number of defects in primary geomembrane; *73 from above*

Q = flow rate through one defect in primary geomembrane; *1.58×10^{-5} m³/sec*

k = hydraulic conductivity of geonet layer above secondary geomembrane; *3.10×10^{-1} m/sec from above*

i = hydraulic gradient in leakage collection layer; *0.050 from above*

B = width of leakage collection layer; *1,660 feet, 506 m (Attachment B, 1/1)*

Placing the estimated geonet hydraulic conductivity, average thickness of liquid in the LDS, and the thickness of the leak detection layer geonet into Equation 6 results in the following:

$$t_{avgworst} = \frac{(73)(1.58x10^{-5})}{(3.10x10^{-1}\,\text{m/sec})(0.050)(506\text{m})} \qquad t_{avgworst} = 0.00015 \text{ m} = 0.15 \text{ mm}$$

The head on the secondary does not exceed 0.15 mm (0.006 inches), much less than the required 12 inch (1 foot) maximum.

Written by:	**R. Flynn**	Date:	**09/22/08**	Reviewed by:	**G. Corcoran**	Date:	9/23/08
Client:	**DMC**	Project:	**Cell 4B**	Project/ Proposal No.:	**SC0349**	Task No.:	

SUMMARY AND CONCLUSIONS

- Using the method outlined by Giroud, et al. (1997), and an N = 73, the ALR was calculated to be 581 gal/day/acre.
- Liquids entering the geonet LDS layer will take less than one day to travel from the leak to the LDS piping system.
- Assuming a worst case scenario under which all the primary geomembrane defects are located at the high end of the leakage collection layer slope, the liquid head on the secondary liner does not exceed 1-inch, well below the required maximum limit of 1-foot.
- The geonet provides sufficient flow rate to accommodate the ALR.

REFERENCES

Bachus, Bob; Narejo, Dhani; Thiel, Richard; Soong, Te-Yang (2004), The GSE Drainage Design Manual, June 2004. *(Attachment C)*

Giroud, J.P., Gross, B.A., Bonaparte, R., and McKelvey, J.A. (1997), "Leachate Flow in Leakage Collection Layers Due to Defects in Geomembrane Liners," Geosynthetic International, Vol. 4, No. 3-4, pp. 215-292. *(Attachment A)*

Koerner, R. M., Designing with Geosynthetics, Prentice Hall, Upper Saddle River, NJ, 1998. *(Attachment E)*

vledge the support of GeoSyntec Consultants for the preparation of and express my gratitude to its Chief Executive Officer and Presi- :, and its Principals, T.R. Sanglerat, J.F. Beech, R.C. Bachus, T.N. a, R.J. Dunn, E. Kavazanjian and D.M. Hendron. I also want to say 1 of this issue would not have been possible without the contribution T.D. King, from Tesoro Petroleum Company, and the others from tants: K. Badu-Tweneboah (2 papers), R. Bonaparte, B.A. Gross, T. '. Khire (3 papers), J.A. McKelvey (3 papers), N.S. Rad, T.R. Sang- erman (3 papers). It is certainly very rewarding for me to work with is caliber. Finally, I am grateful to S.L. Berdy who produced the ex- K. Holcomb who ensured flawless word processing, with the help N. Pierce for several of the papers. I also want to acknowledge the ie by the unknown soldiers of *Geosynthetics International*, the anon- ,ho reviewed the papers and provided so many valuable comments. I express my gratitude to T.S. Ingold, Editor, and R.J. Bathurst, Co- *hetics International* for giving me the opportunity of grouping these same issue published as a Special Issue, and to K. Labinaz, Produc- *synthetics International*, who provided two rounds of editing, with , to ensure not only the correctness of each paper, but, also, the con- cial Issue.

J.P.G.

ATTACHMENT A, 1/6

Technical Paper by J.P. Giroud, B.A. Gross, R. Bonaparte and J.A. McKelvey

Leachate Flow in Leakage Collection Layers Due to Defects in Geomembrane Liners

ABSTRACT: This paper provides analytical and graphical solutions related to the flow of leachate in a leakage collection layer due to defects in the overlying liner (i.e. the primary liner of a double liner system). The defects are assumed to be small (e.g. holes in geomembrane liners). It is shown that leachate flows in a zone of the leakage collection layer (the wetted zone) that is limited by a parabola. A simple relationship is established between the rate of leachate migration through the defect and the maximum thickness of leachate in the leakage collection layer; this relationship depends on the hydraulic conductivity (but not on the slope) of the leakage collection layer. Equations are provided to calculate the average head of leachate on top of the liner underlying the leakage collection layer (i.e. the secondary liner of a double liner system), which is useful for calculating the rate of leachate migration through that liner. Finally, the case of several leaks randomly distributed is considered, and equations for the surface area of the wetted zone and the average head are given for this case. Parametric analyses and design examples provide useful comparisons between the three types of materials used in leakage collection layers: gravel, sand and geonets.

KEYWORDS: Geomembrane, Defect, Leachate migration, Leachate collection, Leakage, Leakage collection, Liner system, Double liner, Geosynthetic leakage collection layer.

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DATES: Original manuscript received 1 March 1997 and accepted 19 April 1997. Discussion open until 1 March 1998.

REFERENCE: Giroud, J.P., Gross, B.A., Bonaparte, R. and McKelvey, J.A., 1997, "Leachate Flow in Leakage Collection Layers Due to Defects in Geomembrane Liners", *Geosynthetics International*, Vol. 4, Nos. 3-4, pp. 215-292.

It appears that, when the leakage collection layer is not full, there is an extremely sim-
le relationship between the rate of leachate migration through the primary liner defect,
), and the thickness of leachate in the leakage collection layer beneath the defect, t_o.
t is interesting to note that this relationship does not depend on the size of the defect
n the primary liner or on the slope of the leakage collection layer.

An approximation that was made to establish Equations 9 and 10 was to assume that
he downslope flow line from A (i.e. AB in Figure 4a) is parallel to the liner. This as-
umption is close to reality as discussed in Section 2.2. However, the actual flow line
rom A is below Line AB as the flow thickness decreases in the downslope direction,
s discussed at the end of Section 5.1.2. Therefore, t_o should only be regarded as the flow
hickness at a primary liner defect, and it is the maximum flow thickness.

Since the simple relationship expressed by Equations 9 and 10 was demonstrated for
he case when the leakage collection layer is not full, the condition expressed by Equa-
ion 1 must be met for Equations 9 and 10 to be valid. Combining Equations 1 and 10
ives the following equation, which is another way to express the condition that should
e met to ensure that the leakage collection layer is not full:

$$t_{LCL} \geq t_{LCLfull} = \sqrt{\frac{Q}{k}} \qquad (11)$$

vhere $t_{LCLfull}$ is the *minimum* thickness that a leakage collection layer with a hydraulic
onductivity k should have to contain, without being full at any location, the leachate
low which results from a defect in the primary liner.

The following equation, derived from Equation 11, is another way to express the con-
lition that should be met to ensure that the leakage collection layer is not full:

$$\underline{Q \leq Q_{full} = k\, t_{LCL}^2} \qquad (12)$$

vhere Q_{full} is the *maximum* steady-state rate of leachate migration through a defect in
he primary liner that a leakage collection layer, with a thickness t_{LCL} and a hydraulic
onductivity k, can accommodate without being filled with leachate.

It is important to remember that the subscript *full* corresponds to a *minimum* thickness
of the leakage collection layer and to a *maximum* rate of leachate migration (which is
also the *maximum* flow rate in the leakage collection layer). It is noteworthy that the
minimum thickness of the leakage collection layer, $t_{LCLfull}$, and the maximum flow rate,
Q_{full}, which are required to ensure that the leakage collection layer can contain, without
being full, the flow that results from a defect in the primary liner, do not depend on the
slope of the leakage collection layer.

It is not impossible to design a leakage collection layer with a thickness less than the
value $t_{LCLfull}$ given by Equation 11, i.e. where the flow rate is greater than Q_{full} defined
by Equation 12. In this case, the leakage collection layer is filled with leachate in a cer-
ain area around the defect of the primary liner (i.e. "the leachate collection layer is
full"). This case is discussed in Section 3.2.

2/

3.2 Rate of Leachate Flow When the Leachate Collection Layer is Fu

If the thickness of the leakage collection layer is less than $t_{LCLfull}$ expressed
tion 11 (or if the rate of leachate migration through a primary liner defect is gre
Q_{full} expressed by Equation 12, which is equivalent), the leakage collection
filled with leachate in a certain area around the defect. Following the appr
scribed in Section 2.2, it may then be assumed that the leachate phreatic surfa
leakage collection layer is a truncated cone (Figure 5). The virtual apex of the t
cone, A′, is above the leakage collection layer (i.e. above the primary liner,
the upper boundary of the leakage collection layer). The virtual leachate depth
the virtual leachate thickness, t_o, are related to the actual leachate head, h_o
Equation 4, and the virtual leachate thickness t_o is greater than the thickness o
chate collection layer:

$$t_o > t_{LCL}$$

The surface area of the vertical cross section of the flow in the leakage collect
(Figure 5) is expressed by:

$$S = \frac{D_o^2}{\tan\beta} - \frac{(D_o - D_{LCL})^2}{\tan\beta} = \frac{D_{LCL}(2D_o - D_{LCL})}{\tan\beta}$$

where D_{LCL} is the depth of the leakage collection layer.

The depth is measured vertically whereas the thickness is measured perpend
to the slope, hence, in accordance with Equation 3:

$$t_{LCL} = D_{LCL}\cos\beta$$

Using the demonstration presented in Section 2.2, i.e. combining Equations
8, 14 and 15, gives:

$$Q = k\, t_{LCL}(2t_o - t_{LCL})$$

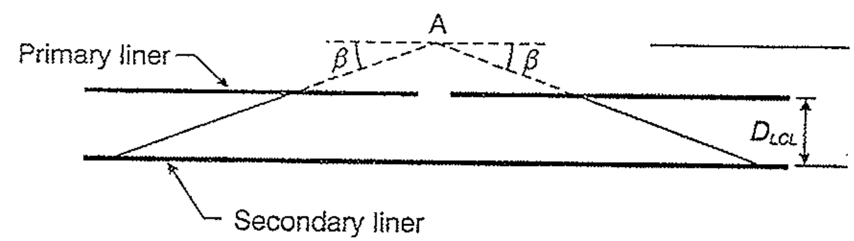


**Figure 5. Vertical cross section of the assumed phreatic surface in the leakage co
layer in the case where the leakage collection layer is filled with leachate in a certa
around the primary liner defect.**

ring (or assuming) the leachate head, h_o, on top of the secondary liner vertically the primary liner defect, one may derive the virtual leachate thickness, t_o, using n 4. Then, knowing t_o, t_{LCL} and k, one may use Equation 16 to calculate the rate ate flow through a defect that the leakage collection layer can convey. ollowing equation can be derived from Equation 16:

$$t_o = \frac{t_{LCL}}{2}\left(1 + \frac{Q}{k\, t_{LCL}^2}\right) \quad (17)$$

ollowing equation can be derived from Equations 13 and 16:

$$t_{LCL} = t_o\left(1 - \sqrt{1 - \frac{Q}{k\, t_o^2}}\right) \quad (18)$$

ion 18 is valid only if the following condition is met:

$$Q \le k\, t_o^2 \quad (19)$$

uld be noted that if $t_{LCL} = t_o$, i.e. if the leakage collection layer is filled with lea- only one point, i.e. at the location of the primary liner defect, Equation 16 is ent to Equation 9.

Parametric Study

; the equations presented in Sections 3.1 and 3.2 it is possible to compare the pacity of different leakage collection layers in case of a defect in the primary Table 1, three different leakage collection layers are compared:

net with a thickness of 5 mm and a hydraulic transmissivity resulting in a hy- ic conductivity (obtained by dividing the hydraulic transmissivity by the thick- of 1×10^{-1} m/s;

vel layer with a thickness of 300 mm and a hydraulic conductivity of 1×10^{-1} and

l layer with a thickness of 300 mm and a hydraulic conductivity of 1×10^{-3} m/s.

irst two leakage collection layers have the same hydraulic conductivity and the have the same thickness. In the case of the geonet, the virtual leachate thick- , considered in Table 1 is greater than, or equal to, the thickness of the leachate on layer, t_{LCL}; therefore, in all cases considered in Table 1, the geonet is filled chate over a certain area around the defect (this area being zero for $t_o = 5$ mm). ase of the gravel and sand layers, the leachate thicknesses considered in Table ss than, or equal to, the thickness of the leakage collection layer; therefore, in s considered in Table 1, the gravel and sand layer are not filled (or just filled) chate, and for these two materials the leachate thicknesses, t_o, shown in Table tual (not virtual) thicknesses.

Table 1. Rate of leachate flow in three different leachate collection layers resulting from a defect in the primary liner.

Leachate thickness (actual or virtual)		Leakage collection layer material					
		Geonet t_{LCL} = 5 mm $k = 1 \times 10^{-1}$ m/s		Gravel t_{LCL} = 300 mm $k = 1 \times 10^{-1}$ m/s		Sand t_{LCL} = 300 mm $k = 1 \times 10^{-3}$ m/s	
t_o		Q		Q		Q	
(m)	(mm)	(m³/s)	(lpd)	(m³/s)	(lpd)	(m³/s)	(lpd)
0.005	5	2.5×10^{-6}	216	2.5×10^{-6}	216	2.5×10^{-8}	2.16
0.01	10	7.5×10^{-6}	648	1.0×10^{-5}	864	1.0×10^{-7}	8.64
0.05	50	4.75×10^{-5}	4,104	2.5×10^{-4}	21,600	2.5×10^{-6}	216
0.1	100	9.75×10^{-5}	8,424	1.0×10^{-3}	86,400	1.0×10^{-5}	864
0.3	300	2.975×10^{-4}	25,704	9.0×10^{-3}	777,600	9.0×10^{-5}	7,776

Notes: The leachate thickness, t_o, can be derived from the leachate head on top of the secondary liner using Equation 4. The leachate thickness, t_o, is the actual leachate thickness if $t_o < t_{LCL}$ and a virtual leachate thickness if $t_o > t_{LCL}$. The tabulated values of the rate of leachate flow, Q, were calculated using Equation 9 when $t_o < t_{LCL}$ and Equation 16 when $t_o > t_{LCL}$. Units: 1 m³/s = 86,400,000 liters per day (lpd).

It appears from Table 1, that for a given value of t_o, i.e. a given value of the head of leachate on top of the secondary liner, h_o (see Equation 4), the gravel and the geonet can convey significantly more leachate than the sand. It is interesting to compare the flow rates of Table 1 with rates of leachate migration through defects of geomembranes used alone (i.e. not part of a composite liner) calculated using Bernoulli's equation, which is expressed as follows:

$$Q = 0.6\, a \sqrt{2\, g\, h_{prim}} = 0.6\, \pi (d^2/4) \sqrt{2\, g\, h_{prim}} \approx (2/3)\, d^2 \sqrt{g\, h_{prim}} \quad (20)$$

where: a = defect area; d = defect diameter; g = acceleration due to gravity; and h_{prim} = head of leachate on top of the primary liner.

Table 2 gives rates of leachate migration through geomembrane defects calculated using Equation 20. It appears that, with the leachate heads that typically exist on the primary liners of actively operating landfills (i.e. landfills that are receiving waste), and provided that the geomembrane is used alone (i.e. is not part of a composite liner):

- a small geomembrane defect (e.g. 1 to 2 mm diameter), which may occasionally be undetected during construction, results in a rate of leakage on the order of 100 liters per day (lpd);
- a geomembrane defect (e.g. 3 to 5 mm diameter), which may occasionally occur during construction phases where defect detection may not be possible (e.g. placement of granular leachate collection material on geomembrane), results in a rate of leakage on the order of 1000 lpd (1 m³/day); and
- a large geomembrane defect (e.g. 10 mm diameter or more), which may occur under

4 Wetted Fraction

4.1 *Scope of Section 4.4*

To calculate the rate of leakage through the secondary liner, it is useful to know what ıction of the total surface area of the secondary liner is wetted and what is the average ad of leachate over this fraction of the secondary liner. The wetted fraction is deter- ined in Sections 4.4.3 and 4.4.4, and the average head will be determined in Sections I and 5.2.

In the preceding sections, only one defect in the primary liner was considered. This no longer the case in Section 4.4 because the wetted fraction depends on the number defects per unit area. In Section 4.4, two scenarios of defect location will be consid- ed: a scenario where the defects are located to give the maximum wetted fraction, and scenario where the defects are at random.

In Section 4.4, a leakage collection layer whose length in the direction of the flow s a horizontal projection L, and whose width in the direction perpendicular is B, is nsidered (Figure 9). The projected surface area of this leakage collection layer is erefore:

$$A_{LCL} = L\,B \tag{98}$$

4.2 *Definitions*

etted Fraction. The wetted fraction, R_w, is defined as the ratio between the surface ea of the total wetted zone and the surface area of the leakage collection layer:

$$R_w = \frac{\sum_{n=1}^{n=N} A_w}{A_{LCL}} \tag{99}$$

As shown by the numerator of the fraction, the surface area of the total wetted zone the sum of the surface areas of the wetted zones that correspond to every defect in e primary liner, the number of defects being N.

efect Frequency. The frequency of defects, F, in the primary liner (i.e. the liner over- ing the leakage collection layer) is defined as the ratio of the total number of defects, in the liner and the surface area of the liner, which is equal to the surface area of the akage collection layer:

$$F = \frac{N}{A_{LCL}} \tag{100}$$

In typical design calculations the frequency of the defects in the primary liner, F, is sumed to be known. For example, if there are four defects per hectare (10,000 m^2),

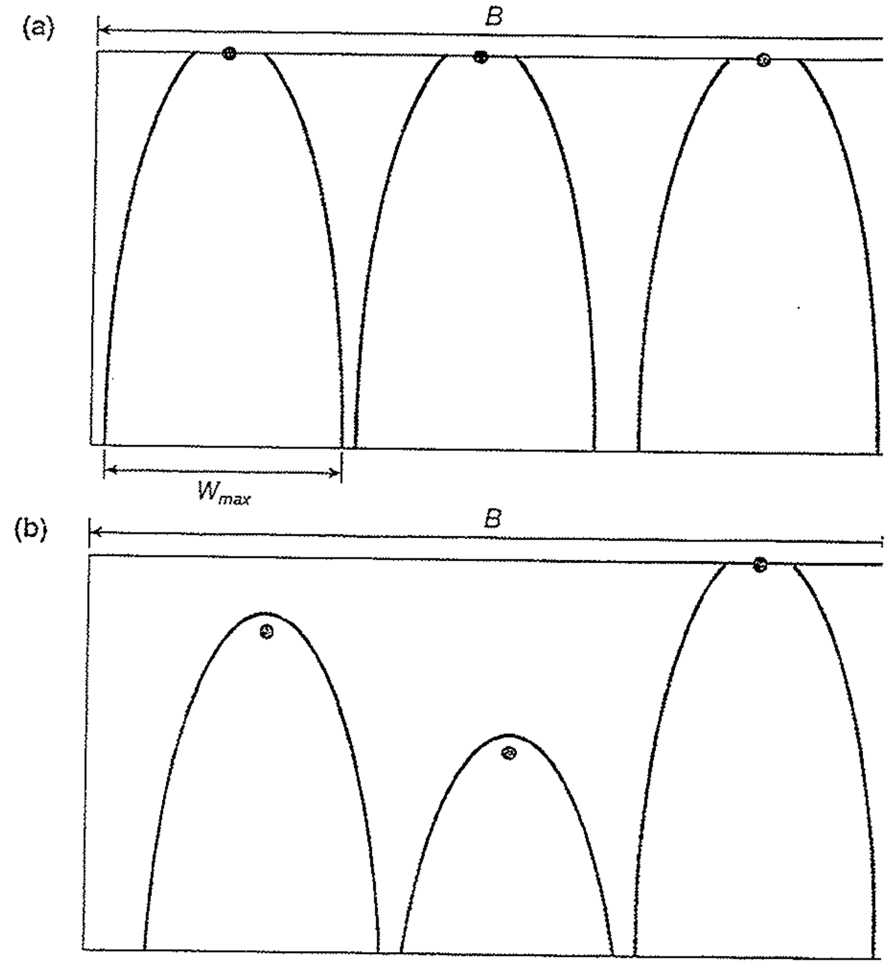


Figure 9. Leakage collection layer zones wetted by leachate migrating through se defects in the primary liner, assuming no overlapping of wetted zones: (a) scenario where all the defects are located at the high end of the leachate collection slope; (b) random scenario where the defects are randomly distributed.

Notes: L is the horizontal projection of the length of the leakage collection layer in the direction of the and B is the width of the leakage collection layer. The dots represent the horizontal projection of the loca of the primary liner defects.

Scenarios. Two defect location scenarios will be considered: (i) the worst sce where all of the defects are at the high end of the leakage collection layer slope (Fi 9a); and (ii) the random scenario where the defects are randomly distributed (Fi

$$\frac{t_{avg\,rand}\,\lambda_{rand}}{t_{avg\,worst}\,\lambda_{worst}} = \frac{2\,\mu}{9} + \frac{x_{rand}}{L} = \frac{\mu^{5/3}}{\left(10\sqrt{2}\right)^{2/3}}\left[\left(1+\frac{2}{\mu}\right)^{5/2} - 2\right]^{2/3} - \frac{5\mu}{18} \qquad (192)$$

Equation 192 is valid only for $\mu \leq 1.0696$. Values calculated using Equation 192 are given in Table 6. Values of $t_{avg\,rand}\,\lambda_{rand}/(t_{avg\,worst}\,\lambda_{worst})$ given in Table 6 for $\mu > 1.0696$ were calculated from numerical values of $\lambda_{worst}/\lambda_{rand}$ given in Table 4 and numerical values of $t_{avg\,rand}/t_{avg\,worst}$ given in Table 6.

5.2.4 *Average Leachate Thickness When Wetted Zones Overlap*

The values of the average leachate thickness given in Sections 5.2.2 and 5.2.3 are valid only if there is no overlapping of different wetted zones, i.e. if, as shown in Section 4.4.5:

$$R_{w\,worst} \leq \mathrm{Crit}\,(R_{w\,worst}) \qquad (193)$$

$$R_{w\,rand} \leq \mathrm{Crit}\,(R_{w\,rand}) \qquad (194)$$

If the conditions expressed by Equations 193 and 194 are not satisfied, there is overlapping between adjacent wetted zones. In this case, the best approach, from a practical standpoint, is to assume that the entire area of the leakage collection layer is wetted. Again, the worst scenario and the random scenario are considered. These two scenarios are defined in Section 4.4.2.

Worst Scenario. In the worst scenario all of the primary liner defects are located at the higher end of the leakage collection layer slope. Since the wetted zones have been assumed to overlap, it is approximately correct to consider that the entire leakage collection layer area is wetted. As a result, the leachate thickness is approximately uniform over the entire leakage collection layer area provided that the defects are uniformly distributed at the high end of the leakage collection layer slope. The average leachate thickness is then derived using the classical Darcy's equation, resulting in:

$$t_{avg\,worst} = \frac{N\,Q}{k\,i\,B} \qquad (195)$$

where: N = total number of defects in the primary liner; Q = rate of leachate migration through one defect of the primary liner, all defects being assumed identical and subjected to the same leachate head over the entire surface area of the primary liner; k = hydraulic conductivity of the leakage collection layer material; i = hydraulic gradient in the leakage collection layer; and B = width of the leakage collection layer.

Combining Equations 8 and 195 gives:

$$t_{avg\,worst} = \frac{N\,Q}{k\,B\,\sin\beta} \qquad (196)$$

Combining Equations 98, 100 and 197 gives:

$$t_{avg\,worst} = \frac{F\,L\,Q}{k\,\sin\beta}$$

Equations 195 to 197 are valid only if the leakage collection layer is not the condition expressed by Equation 11 (or Equation 12 which is equivalent) case where the leakage collection layer is *full over its entire surface area* i (i) Equations 16 to 18, which where established for the case where the leak tion layer is *full in a limited area* around the primary liner defect, are not : and (ii) assuming that the virtual thickness of leachate is a constant (t_{avg}) ove area of the leakage collection layer allows Darcy's equation to be written :

$$N\,Q = k\,B\,t_{LCL}\,\sin\beta$$

which shows that there is no relationship between Q and t_{avg}. In other words. indeterminate. Therefore, no solution is proposed for the average leachate virtual thickness) for the case where the leakage collection layer is filled wit

Random Scenario. In the random scenario, the primary liner defects are at random. In the case where there are enough defects to assume that the ent collection layer area is wetted, the design of a leakage collection layer becor to the design of a leachate collection layer subjected to a uniform rate of lea eration. As shown by Giroud and Houlihan (1995), in most practical cases, value of the leachate thickness is :

$$\frac{t_{avg}}{L} = \frac{\sum Q/(L\,B)}{2\,k\,\sin\beta}$$

With the notations used in this paper, Equation 199 becomes:

$$t_{avg\,rand} = \frac{N\,Q}{2\,k\,B\,\sin\beta}$$

Combining Equations 98, 100 and 200 gives:

$$t_{avg\,rand} = \frac{F\,L\,Q}{2\,k\,\sin\beta}$$

Comparing Equations 197 and 200 shows that the average leachate thickne greater in the worst scenario than in the random scenario. (It should be remen it has been assumed that, in both cases, the entire surface area of the leakage layer is wetted.)

Equations 199 to 201 are valid only if the leakage collection layer is not the condition is expressed by Equation 11 (or Equation 12 which is equivale Also, for the reasons indicated after Equation 197, no solution is proposed f where the leakage collection layer is full.

...e of Primary Liner Defect Frequency on Average Leachate Thickness

...difference between Sections 5.2.2 and 5.2.3 on one hand, and Section ...er hand should be noted. Equations for $t_{avg\,worst}$ and $t_{avg\,rand}$ do not depend ...equency, F, in Sections 5.2.2 and 5.2.3, whereas they depend on F in ...The reason for that is the following:

...2.2 and 5.2.3, the wetted zones, that correspond to various defects in ...ner, do not overlap. The average leachate thickness is the same in any ...idual wetted zones and it is calculated for any of them. Consequently, ...achate thickness does not depend on the frequency of defects. However, ...of defects governs the wetted fraction (i.e. the ratio between the total ...f all wetted zones and the surface area of the leakage collection layer). ...4, it is assumed that the entire surface area of the leakage collection ...l. In other words, it is assumed that the wetted fraction is equal to one. ...average leachate thickness is a function of all of the defects in the pri...d, consequently, is a function of the defect frequency.

...to note that, when the wetted fraction exceeds the critical value (Sec... ...lesign engineer must assume that the individual wetted zones (i.e. the ...t correspond to the individual defects in the primary liner) overlap and ...iations given in Section 5.2.4 to calculate the average leachate thick... ...when the wetted fraction does not exceed the critical value, the design ...her use the equations given in Section 5.2.4 or use the equations given ...and 5.2.3. The approach described in Section 5.2.4 is simpler: it con...g that the entire leakage collection layer area is wetted. The approach ...tions 5.2.2 and 5.2.3 is more complex but closer to reality: only a frac...ge collection layer is wetted and, in addition to calculating the average ...ss as shown in Sections 5.2.2 and 5.2.3, it is necessary to determine the ...on using equations provided in Section 4.4. The use of both approaches ...Example 6 in Section 6.1.
...aches give values of the leachate thickness (and head) that are different ...etted zones do not overlap, only the approach described in Sections ...gives a correct value of the leachate thickness (or head). However, in ...age leachate thickness is only calculated as a first step in the calcula... ...leakage through the secondary liner. In this case, both approaches are ...pproach described in Section 5.2.4 gives a leachate thickness that is ...nly distributed over the entire secondary liner, while the approach de...as 5.2.2 and 5.2.3 gives a greater leachate thickness in the wetted area ...utside the wetted area. The leakage rates calculated using the leachate ...ined as indicated in Section 5.2.4 are conservative (i.e. greater than ...using the leachate thickness determined as indicated in Sections 5.2.2 ...ultiplied by the wetted fraction) because leakage rates typically vary ...the head to a power less than one. This will be illustrated quantitative... after Example 6.

6

5.3 Time Required to Reach Steady-State Flow Conditions

5.3.1 *Equations*

The volume of liquid in a porous medium is less than the volume of porous medium that contains the liquid. As indicated by Equation 143, the volume of leachate in the leakage collection layer is equal to the volume of the leakage collection layer that contains the leachate multiplied by the porosity, n, of the leakage collection layer material. The time required for such a volume to pass through the primary liner defect, $\bar{t}_{req}$, gives a lower boundary of the time required to reach steady-state flow conditions, hence:

$$\bar{t}_{req} > \frac{nV}{Q} \tag{202}$$

Combining Equations 10, 153 and 202 gives the following equation for the case where the leakage collection layer is not full:

$$\bar{t}_{req} > \frac{n\,x}{k \sin\beta \cos\beta} + \frac{2\,n\,Q^{1/2}}{9 \sin^2\beta \cos\beta \; k^{3/2}} \tag{203}$$

The last term is generally negligible, because it represents the time required to fill the volume of the leakage collection layer that contains leachate between axes Oy and VY (Figure 6). This volume is either small or reduced by truncation (Figure 8). Therefore:

$$\bar{t}_{req} > \frac{n\,x}{k \sin\beta \cos\beta} \tag{204}$$

Equation 204 may be written as follows:

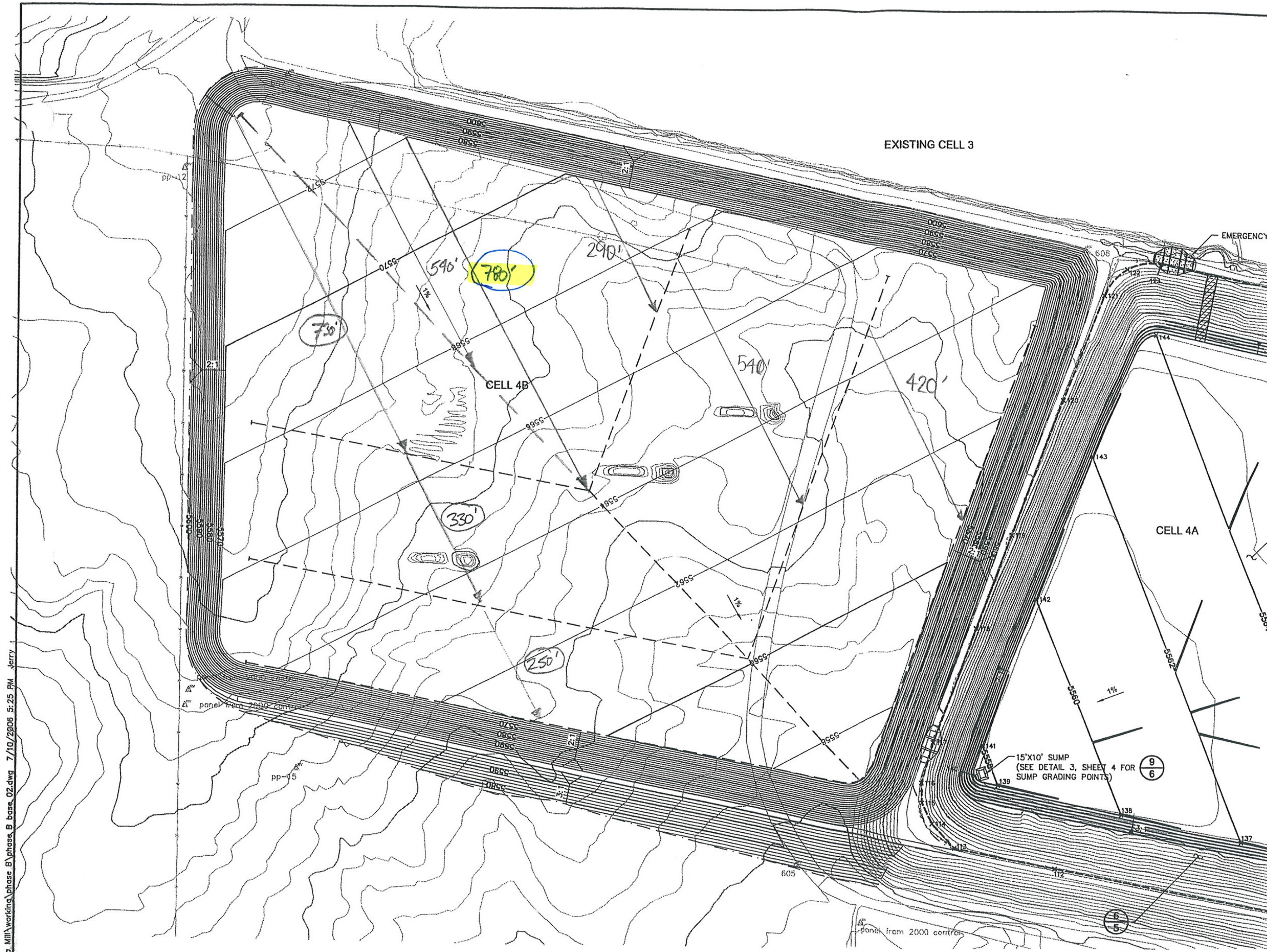
$$\bar{t}_{req} > \frac{x / \cos\beta}{k \sin\beta / n} \tag{205}$$

Combining Equations 8 and 205 gives:

$$\bar{t}_{req} > \frac{x / \cos\beta}{k\,i / n} \tag{206}$$

where the numerator is the distance between the primary liner defect and the low end of the leakage collection layer slope, and the denominator is the actual liquid velocity derived from Darcy's equation. Therefore, the right hand member of Equation 204 is the travel time, $\bar{t}_{travel}$, i.e. the time required by a drop of leachate to travel from the primary liner defect to the low end of the leakage collection layer, assuming that flow is not hampered by capillarity in the leakage collection layer:

$$\bar{t}_{req} > \bar{t}_{travel} = \frac{n\,x}{k \sin\beta \cos\beta} \tag{207}$$



LEGEND

5560	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (2' CONTOUR)
1%	SLOPE DIRECTION AND GRADE

* Most critical path = longest path

* 780' is longest path

N

100 50 0 100 200
SCALE IN FEET

ATTACHMENT B

GEOSYNTEC CONSULTANTS
3990 OLD TOWN AVENUE, SUITE B-101
SAN DIEGO, CALIFORNIA 92110
TELEPHONE: (619) 297-1530

PROJECT: WHITE MESA MILL

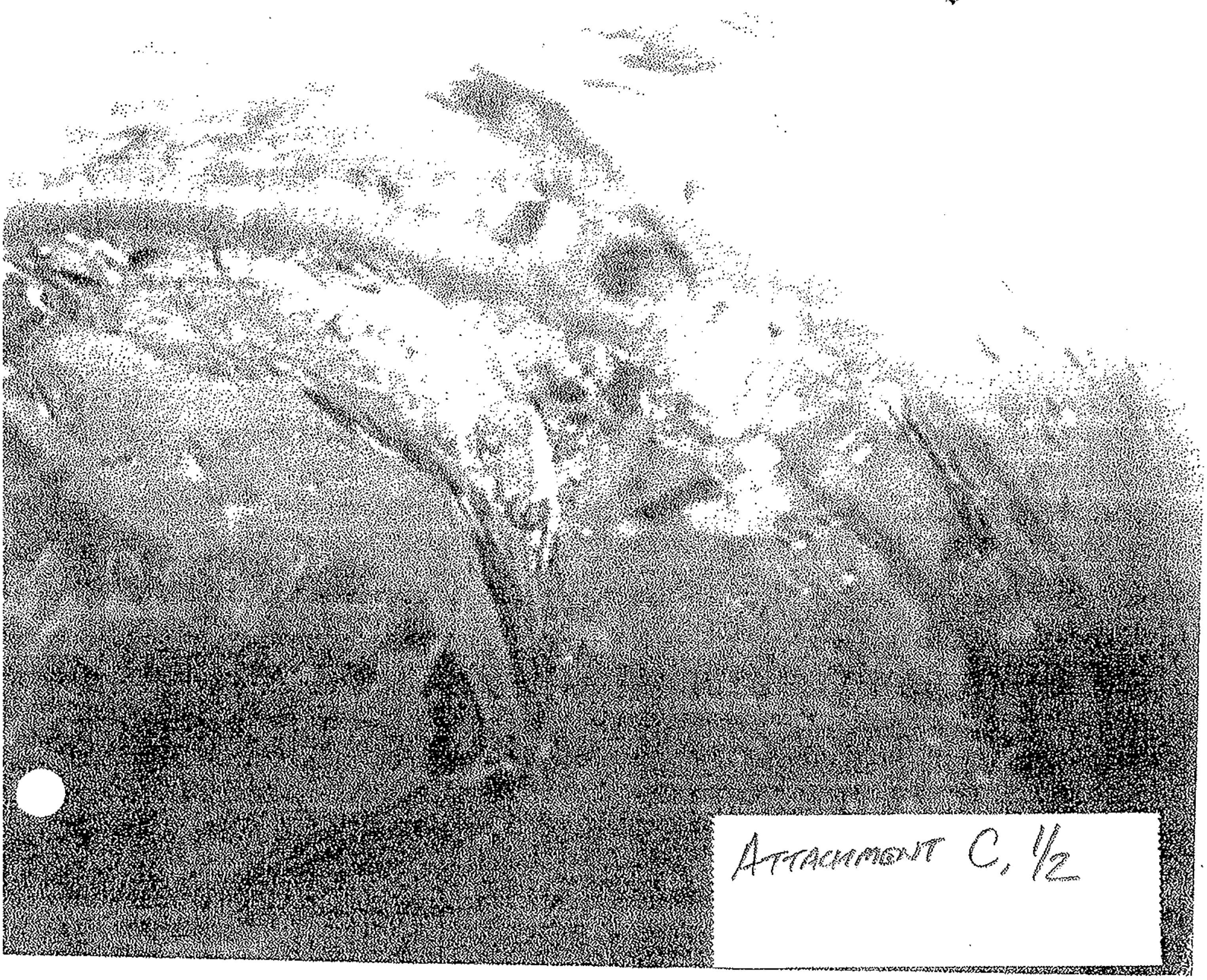
TITLE: PHASE 4B GRADING

DATE: OCT 2006	CHECKED BY:	SCALE: 1"= 100'	FIGURE NO:
DESIGN BY:	REVIEWED BY: (PROJ. MGR.)	JOB NO.: SC0349	

P:\CAD\SC0349 – White Mesa Mill\working\phase B\phase_B_base_02.dwg 7/10/2006 5:25 PM Jerry

The GSE Drainage Design Manual

Robert Bachus • Dhani Narejo • Richard Thiel • Te-Yang Soong



ATTACHMENT C, 1/2

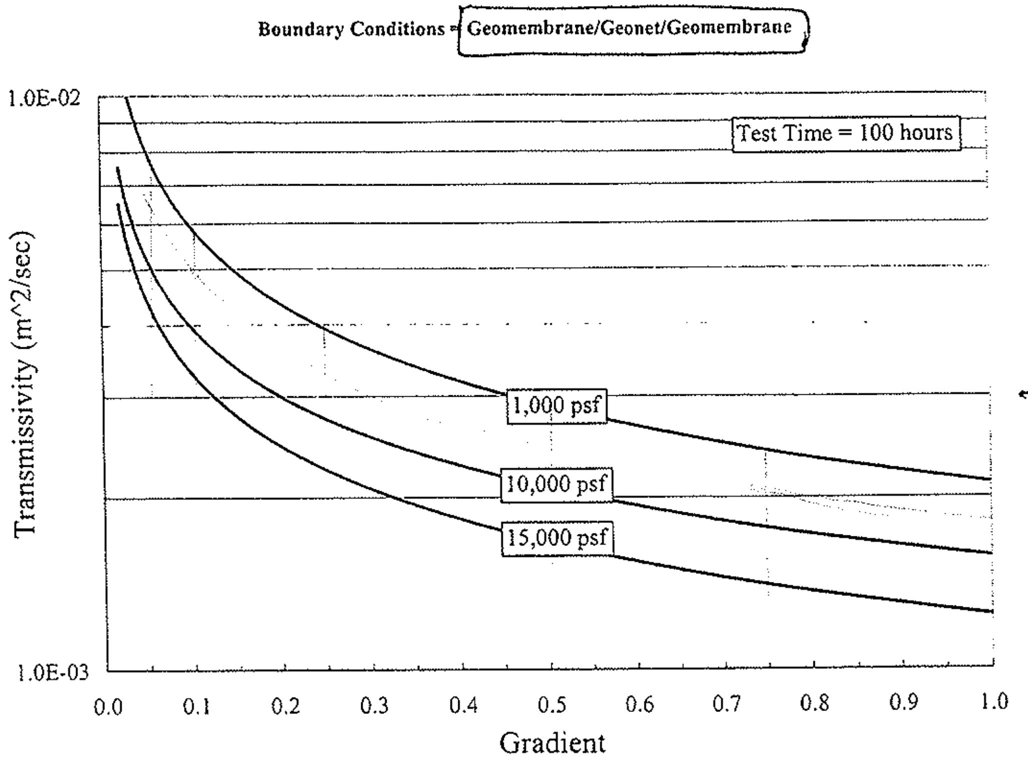


Figure A-7. Performance Transmissivity of a 300 mil GSE HyperNet UF geonet.

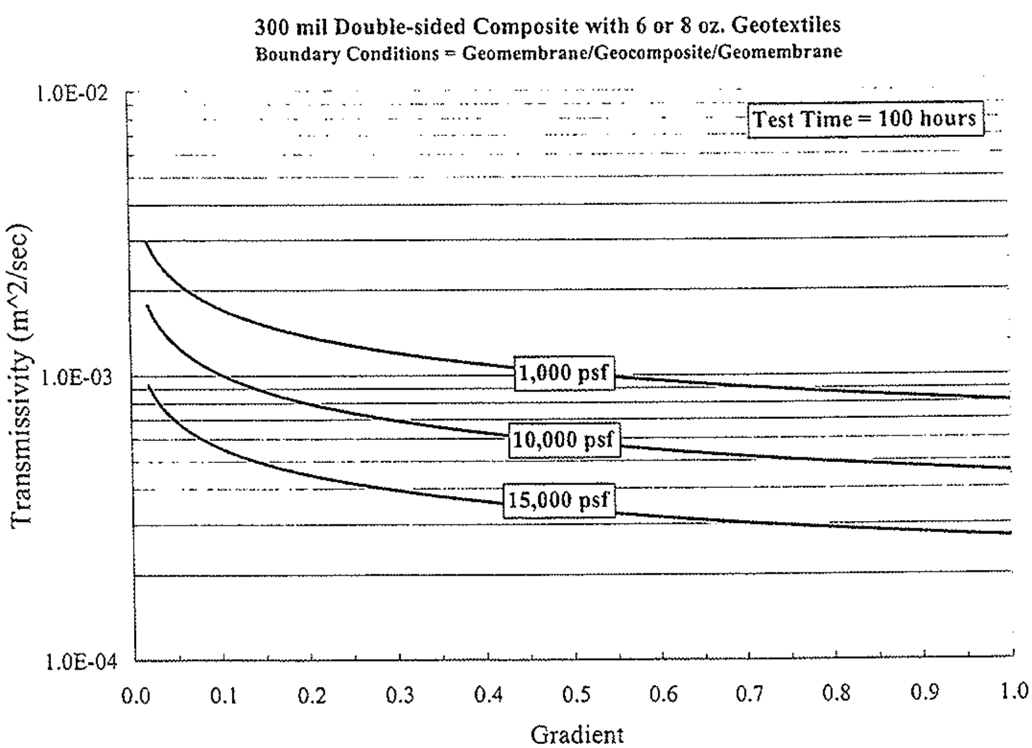
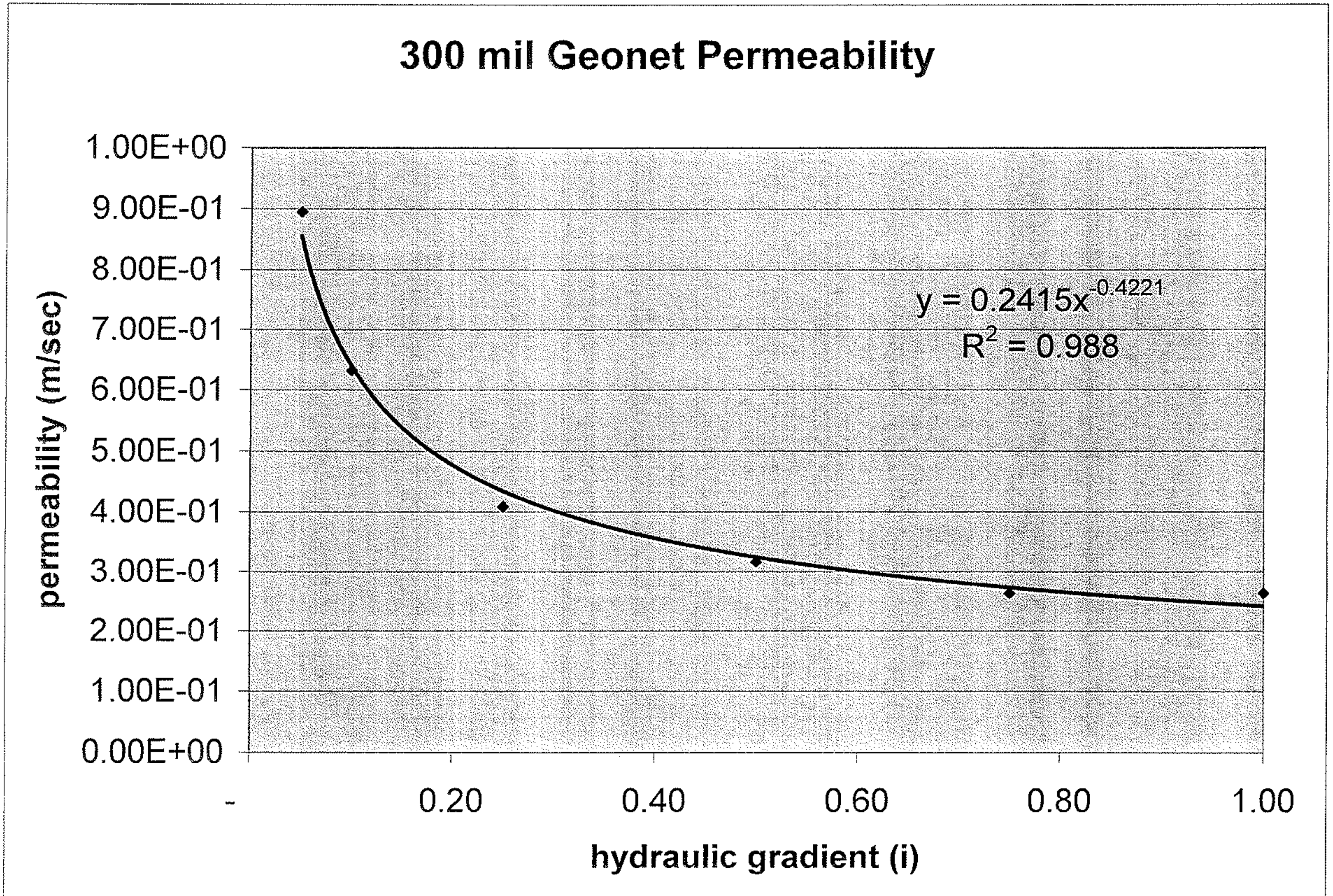


Figure A-8. Performance Transmissivity of a 300 mil GSE FabriNet UF geocomposite between Plates.

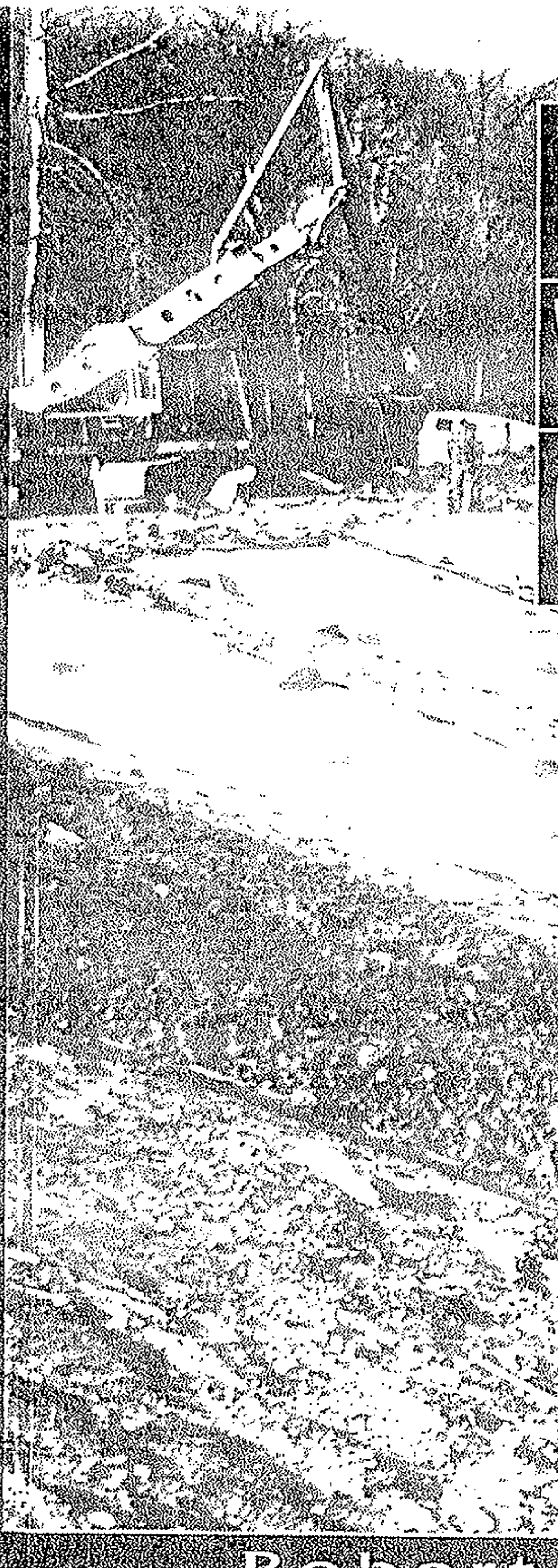
(Attachment C, 2/2)

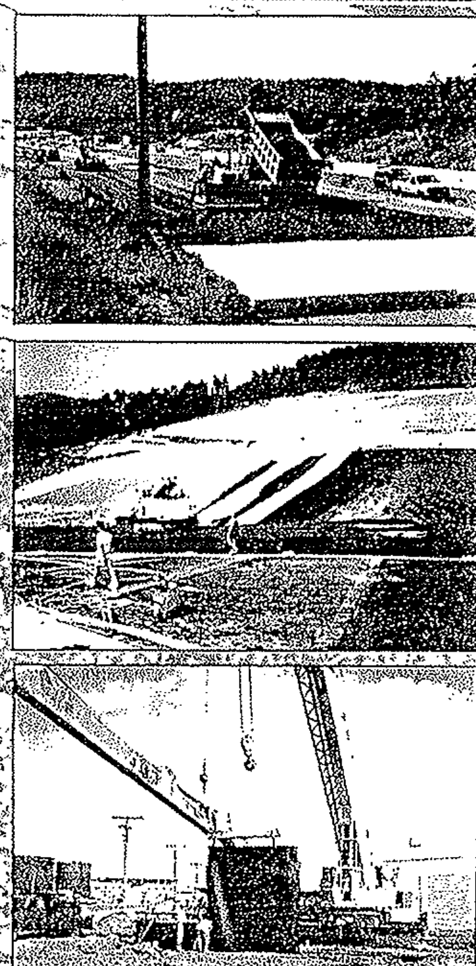


leak detection system geonet20070717.xls

ATTACHMENT D(1/1)

DESIGNING WITH GEOSYNTHETICS





Fourth Edition

Robert M. Koerner

ATTACHMENT E, 1/3

4.1.6 Allowable Flow Rate

As described previously, the very essence of the design-by-function concept is the establishment of an adequate factor of safety. For geonets, where flow rate is the primary function, this takes the following form.

$$FS = \frac{q_{allow}}{q_{reqd}} \tag{4.3}$$

where

FS = factor of safety (to handle unknown loading conditions or uncertainties in the design method, etc.),
q_{allow} = allowable flow rate as obtained from laboratory testing, and
q_{reqd} = required flow rate as obtained from design of the actual system.

Alternatively, we could work from transmissivity to obtain the equivalent relationship.

$$FS = \frac{\theta_{allow}}{\theta_{reqd}} \tag{4.4}$$

where θ is the transmissivity, under definitions as above. As discussed previously, however, it is preferable to design with flow rate rather than with transmissivity because of nonlaminar flow conditions in geonets.

Concerning the allowable flow rate or transmissivity value, which comes from hydraulic testing of the type described in Section 4.1.3, we must assess the realism of the test setup in contrast to the actual field system. If the test setup does not model site-specific conditions adequately, then adjustments to the laboratory value must be made. This is usually the case. Thus the laboratory-generated value is an ultimate value that must be reduced before use in design; that is,

$$q_{allow} < q_{ult}$$

One way of doing this is to ascribe reduction factors on each of the items not adequately assessed in the laboratory test. For example,

$$q_{allow} = q_{ult}\left[\frac{1}{RF_{IN} \times RF_{CR} \times RF_{CC} \times RF_{BC}}\right] \tag{4.5}$$

or if all of the reduction factors are considered together.

$$q_{allow} = q_{ult}\left[\frac{1}{\Pi RF}\right] \tag{4.6}$$

where

q_{ult} = flow rate determined using ASTM D4716 or ISO/DIS 12958 for short-term tests between solid platens using water as the transported liquid under laboratory test temperatures,

ATTACHMENT E, 2/3

io ncept is the es-
ɔw rate is the primary

(4.3)

ons or uncertainties

ing, and
tual system.

ıivalent relationship.

(4.4)

sed previously, how-
ımissivity because of

, which comes from
ıssess the realism of
does not model site-
/alue must be made.
ı ıte value that

the items not ade-

(4.5)

(4.6)

12958 for short-
ınsported liquid

q_{allow} = allowable flow rate to be used in Eq. (4.3) for final design purposes,
RF_{IN} = reduction factor for elastic deformation, or intrusion, of the adjacent geosynthetics into the geonet's core space,
RF_{CR} = reduction factor for creep deformation of the geonet and/or adjacent geosynthetics into the geonet's core space,
RF_{CC} = reduction factor for chemical clogging and/or precipitation of chemicals in the geonet's core space,
RF_{BC} = reduction factor for biological clogging in the geonet's core space, and
ΠRF = product of all reduction factors for the site-specific conditions.

Some guidelines for the various reduction factors to be used in different situations are given in Table 4.2. Please note that some of these values are based on relatively sparse information. Other reduction factors, such as installation damage, temperature effects, and liquid turbidity, could also be included. If needed, they can be included on a site-specific basis. On the other hand, if the actual laboratory test procedure has included the particular item, it would appear in the above formulation as a value of unity. Examples 4.2 and 4.3 illustrate the use of geonets and serve to point out that high reduction factors are warranted in critical situations.

Example 4.2

What is the allowable geonet flow rate to be used in the design of a capillary break beneath a roadway to prevent frost heave? Assume that laboratory testing was done at the proper design load and hydraulic gradient and that this testing yielded a short-term between-rigid-plates value of 2.5×10^{-4} m²/s.

Solution: Since better information is not known, average values from Table 4.2 are used in Eq. (4.5).

TABLE 4.2 RECOMMENDED PRELIMINARY REDUCTION FACTOR VALUES FOR EQ. (4.5) FOR DETERMINING ALLOWABLE FLOW RATE OR TRANSMISSIVITY OF GEONETS

Application Area	RF_{IN}	RF_{CR}*	RF_{CC}	RF_{BC}
Sport fields	1.0 to 1.2	1.0 to 1.5	1.0 to 1.2	1.1 to 1.3
Capillary breaks	1.1 to 1.3	1.0 to 1.2	1.1 to 1.5	1.1 to 1.3
Roof and plaza decks	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2	1.1 to 1.3
Retaining walls, seeping rock, and soil slopes	1.3 to 1.5	1.2 to 1.4	1.1 to 1.5	1.0 to 1.5
Drainage blankets	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2
Surface water drains for landfill covers	1.3 to 1.5	1.1 to 1.4	1.0 to 1.2	1.2 to 1.5
* Secondary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0
Primary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0

*These values are sensitive to the density of the resin used in the geonet's manufacture. The higher the density, the lower the reduction factor. Creep of the covering geotextile(s) is a product-specific issue.

ATTACHMENT E, 2/3

EXHIBIT F

REVISED TECHNICAL SPECIFICATIONS

Prepared for

Denison Mines (USA) Corp.
6425 S. Highway 191
P.O. Box 809
Blanding, UT 84511

TECHNICAL SPECIFICATIONS FOR THE CONSTRUCTION OF CELL 4B LINING SYSTEM

WHITE MESA MILL BLANDING, UTAH

Prepared by

Geosyntec consultants

engineers | scientists | innovators

10875 Rancho Bernardo Rd., Suite 200
San Diego, CA 92127

Project Number SC0349

December 2007
Revised January 2009

CERTIFICATION PAGE

TECHNICAL SPECIFICATIONS
CELL 4B LINING SYSTEM CONSTRUCTION
DENISON MINES (USA) CORP.
WHITE MESA MILL
BLANDING, UTAH

The Engineering material and data contained in these Technical Specifications were prepared under the supervision and direction of the undersigned, whose seal as a registered Professional Engineer is affixed below.

REGISTERED PROFESSIONAL ENGINEER
No. 6020077-2202
GREGORY T.
CORCORAN
STATE OF UTAH

Gregory T. Corcoran, P.E.
Engineer of Record

TABLE OF CONTENTS

SECTION 01010
SUMMARY OF WORK

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. The Work generally involves the placement and compaction of fill, preparation of subgrade, installation of geosynthetic liner system, and associated piping.

B. The Work will generally consist of:

1. Initial topographic survey;
2. Mass excavation and fill placement and compaction;
3. Subgrade preparation;
4. Anchor trench and leak detection system trench excavation;
5. Installation of needle-punched geosynthetic clay liner (GCL) consisting of woven and nonwoven geotextiles;
6. Installation of 60-mil high density polyethylene (HDPE) secondary geomembrane;
7. Installation of leak detection system 4-inch and 18-inch polyvinyl chloride (PVC) pipe and fittings;
8. Installation of aggregate within leak detection system pipe trench and sump;
9. Installation of 300-mil geonet;
10. Installation of 60-mil HDPE primary geomembrane;
11. Installation of 16 oz./SY nonwoven geotextile cushion;
12. Installation of slimes drain 4-inch and 18-inch PVC pipe and fittings;
13. Installation of aggregate around slimes drain and within sump; and
14. Installation of strip composite drainage layer.

1.02 CONTRACTOR’S RESPONSIBILITIES

A. Start, layout, construct, and complete the construction of the Cell 4B lining system (the Project) in accordance with the Technical Specifications, CQA Plan, and Drawings (Contract Documents).

B. Provide a competent site superintendent, capable of reading and understanding the Construction Documents, who shall receive instructions from the Construction Manager.

C. Establish means, techniques, and procedures for constructing and otherwise executing the Work.

D. Establish and maintain proper Health and Safety practices for the duration of the Project.

E. Except as otherwise specified, furnish the following and pay the cost thereof:

1. Labor, superintendent, and products.
2. Construction supplies, equipment, tools, and machinery.
3. Water, electricity, and other utilities required for construction.
4. Other facilities and services necessary to properly execute and complete the Work.

5. A Registered Land Surveyor, licensed in the State of Utah, to survey and layout the Work, and to certify as-built Record Drawings.

F. Pay cost of legally required sales, consumer, and use taxes and governmental fees.

G. Perform Work in accordance with codes, ordinances, rules, regulations, orders, and other legal requirements of governmental bodies and public agencies bearing on performance of the Work.

H. Forward submittals and communications to the Construction Manager. Where applicable, the Construction Manager will coordinate submittals and communications with the representatives who will give approvals and directions through the Construction Manager.

I. Maintain order, safe practices, and proper conduct at all times among Contractor's employees. The Owner, and its authorized representative, may require that disciplinary action be taken against an employee of the Contractor for disorderly, improper, or unsafe conduct. Should an employee of the Contractor be dismissed from his duties for misconduct, incompetence, or unsafe practice, or combination thereof, that employee shall not be rehired for the duration of the Work.

J. Coordinate the Work with the utilities, private utilities, and/or other parties performing work on or adjacent to the Site. Eliminate or minimize delays in the Work and conflicts with those utilities or contractors. Coordinate activities with the Construction Manager. Schedule private utility and public utility work relying on survey points, lines, and grades established by the Contractor to occur immediately after those points, lines, and grades have been established.

K. Coordinate activities of the several trades, suppliers, and subcontractors, if any, performing the Work.

1.03 NOTIFICATION

A. The Contractor shall notify the Construction Manager in writing if he elects to subcontract, sublet, or reassign any portion of the Work. This shall be done at the time the bid is submitted. The written statement shall describe the portion of the Work to be performed by the Subcontractor and shall include an indication, by reference if desired by the Construction Manager, that the Subcontractor is particularly experienced and equipped to perform that portion of the Work. No portion of the Work shall be subcontracted, sublet, or reassigned without written permission of the Construction Manager. Consent to subcontract, sublet, or reassign any portion of the Work by the Construction Manager shall not be considered as a testimony of the Construction Manager as to the qualifications of the Subcontractor and shall not be construed to relieve the Contractor of any responsibilities for completion of the Work.

1.04 CONFORMANCE

A. Work shall conform to the Technical Specifications, Construction Quality Assurance (CQA) Plan, and Drawings that form a part of these Contract Documents.

B. Omissions from the Technical Specifications, CQA Plan, and Drawings or the misdescription of details of the Work which are necessary to carry out the intent of the Contract Documents, are customarily performed and shall not relieve the Contractor from performing such omitted or misdescribed details of the Work, but they shall be performed as if fully and correctly set forth and described in the Technical Specifications, CQA Plan, and Drawings.

1.05 DEFINITIONS

A. **OWNER** – The term Owner means the Denison Mines (USA) Corp. for whom the Work is to be provided.

B. **CONSTRUCTION MANAGER** – The term Construction Manager means the firm responsible for project administration and project documentation control. All formal documents will be submitted to the Construction Manager for proper distribution and/or review. During the period of Work the Construction Manager will act as an authorized representative of the Owner.

C. **DESIGN ENGINEER** – The term Design Engineer means the firm responsible for the design and preparation of the Construction Documents. The Engineer is responsible for approving all design changes, modifications, or clarifications encountered during construction. The Design Engineer reports directly to the Owner.

D. **CQA ENGINEER –** The term CQA Engineer refers to the firm responsible for CQA related monitoring and testing activities. The CQA Engineer's authorized personnel will include CQA Engineer-of-Record and Lead CQA Monitor. The CQA Engineer may also perform construction quality control (CQC) work as appropriate. The CQA Engineer reports directly to the Owner.

E. **CONTRACTOR** – The term Contractor means the firm that is responsible for the Work. The Contractor's responsibilities include the Work of any and all of the subcontractors and suppliers. The Contractor reports directly to the Construction Manager. All subcontractors report directly to the Contractor.

F. **SURVEYOR** – The term Surveyor means the firm that will perform the survey and provide as-built Record Drawings for the Work. The Surveyor shall be a Registered Land Surveyor, licensed to practice in the State of Utah. The Surveyor is employed by and reports directly to the Contractor.

G. **SITE –** The term Site refers to all approved staging areas, and all areas where the Work is to be performed, both public and private owned.

H. **WORK** – The term Work means the entire completed construction, or various separately identifiable parts thereof, required to be furnished under the Contract Documents. Work includes any and all labor, services, materials, equipment, tools, supplies, and facilities required by the Contract Documents and necessary for the completion of the project. Work is the result of performing services, furnishing labor, and furnishing and incorporating materials and equipment into the construction, all as required by the Contract Documents.

I. **DAY –** A calendar day on which weather and other conditions not under the control of the Contractor will permit construction operations to proceed for the major part of the day with the normal working force engaged in performing the controlling item or items of Work which would be in progress at that time.

J. **CONTRACT DOCUMENTS** – Contract Documents consist of the Technical Specifications, CQA Plan, and Drawings.

1.06 CONTRACT TIMES

A. The time stated for completion and substantial completion shall be in accordance with the Contract Times specified in the Agreement. Extensions to the Contract Time of performance shall be granted for those days when the Contractor is unable to work due to adverse weather conditions or as a result of abnormal conditions. Extension of time of performance based on adverse weather conditions shall be granted when requested by the Contractor and reviewed in writing by the Construction Manager. All requests for extensions of time by the Contractor based on adverse weather conditions must be submitted in writing to the Construction Manager within five (5) working days of the time in question. No claims for damages shall be made by the Contractor for delays.

B. Contractor shall adhere to the schedule provided in the Contract. Unapproved extensions to the schedule will result in the Contractor paying liquidated damages in the amount of $4,000 per day to cover costs associated with Construction Management and construction oversight.

1.07 CONTRACTOR USE OF WORK SITE

A. Confine Site operations to areas permitted by law, ordinances, permits, and the Contract Documents. The Contractor shall ensure that all persons under his control (including Subcontractors and their workers and agents) are kept within the boundaries of the Site and shall be responsible for any acts of trespass or damage to property by persons who are under his control. Consider the safety of the Work, and that of people and property on and adjacent to work Site, when determining amount, location, movement, and use of materials and equipment on work Site.

B. The Contractor shall be responsible for protecting private and public property including pavements, drainage culverts, electricity, highway, telephone, and similar property and shall make good of, or pay for, all damage caused thereto. Control of erosion throughout the project is of prime importance and is the responsibility of the Contractor. The Contractor shall provide and maintain all necessary measures to control erosion during progress of the Work to the satisfaction of the Construction Manager and all applicable laws and regulations, and shall remove such measures and collected debris upon completion of the project. All provisions for erosion and sedimentation control apply equally to all areas of the Work.

C. The Contractor shall promptly notify the Construction Manager in writing of any subsurface or latent physical conditions at the Site that differ materially from those indicated or referred to in the Contract Documents. Construction Manager will promptly review those conditions and advise Owner in writing if further investigations or tests are necessary. If the Construction Manager finds that the results of such investigations or tests indicate that there are subsurface and latent physical conditions which differ materially from those intended in the Contract Documents, and which could not reasonably have been anticipated by Contractor, a Change Order shall be issued incorporating the necessary revisions.

D. At no time shall the Contractor interfere with operations of businesses on or in the vicinity of the Site. Should the Contractor need to work outside the regular working hours, the Contractor is required to submit a written request and obtain approval by the Construction Manager.

1.08 PRESERVATION OF SCIENTIFIC INFORMATION

A. Federal and State legislation provides for the protection, preservation, and collection of data having scientific, prehistoric, historical, or archaeological value (including relics and specimens) that might otherwise be lost due to alteration of the terrain as a result of any construction work. If evidence of such information is discovered during the course of the Work, the Contractor shall notify the Construction Manager immediately, giving the location and nature of the findings. Written confirmation shall be forwarded within two (2) working days.

B. The Contractor shall exercise care so as not to damage artifacts uncovered during excavation operations, and shall provide such cooperation and assistance as may be necessary to preserve the findings for removal or other disposition by the Construction Manager or Government agency.

C. Where appropriate, by reason of a discovery, the Construction Manager may order delays in the time of performance, or changes in the Work, or both. If such delays, or changes, or both, are ordered, the time of performance and contract price shall be adjusted in accordance with the applicable clauses of the Contract.

1.09 MEASUREMENT AND PAYMENT

A. Measurement for Work will be according to the work items listed in Section 01025 of these Specifications.

1.10 EXISTING UTILITIES

A. The Contractor shall be responsible for locating, uncovering, protecting, flagging, and identifying all existing utilities encountered while performing the Work. The Contractor shall request that Underground Service Alert (USA) locate and identify the existing utilities. The request shall be made 48 hours in advance.

B. Costs resulting from damage to utilities shall be borne by the Contractor. Costs of damage shall include repair and compensation for incidental costs resulting from the unscheduled loss of utility service to affected parties.

C. The Contractor shall immediately stop work and notify the Construction Manager of all utilities encountered and damaged. The Contractor shall also Survey the exact location of any utilities encountered during construction.

1.11 CONTRACTOR QUALIFICATIONS

A. The Contractor, and all subcontractors, shall be licensed at the time of bidding, and throughout the period of the Contract, by the State of Utah to do the type of work required under terms of these Contract Documents. By submitting a bid, the Contractor certifies that he is skilled, competent, and knowledgeable on the nature, extent and inherent conditions of the Work to be performed and has been regularly engaged in the general class and type of work called for in these Contract Documents and meets the qualifications required in these Specifications.

B. The Construction Manager shall disqualify a bidder that either cannot provide references, or if the references cannot substantiate the Contractor's qualifications.

C. By submission of a bid for this Project, the Contractor acknowledges that he is thoroughly familiar with the Site conditions.

1.12 INTERPRETATION OF TECHNICAL SPECIFICATIONS, CQA PLAN, AND DRAWINGS

A. Should it appear that the Work to be done or any matters relative thereto are not sufficiently detailed or explained in the Technical Specifications, CQA Plan, and/or Drawings, the Design Engineer will further explain or clarify, as may be necessary. In the event of any questions arising respecting the true meaning of the Contract Documents, the matter shall be referred to the Design Engineer, whose decision thereon shall be final.

1.13 HEALTH AND SAFETY

A. The Contractor shall be responsible for health and safety of its own crew, subcontractors, suppliers, and visitors. The Contractor shall adhere to the Contractor Safety Rules for the Site.

1.14 GENERAL REQUIREMENTS

A. SURVEYING – The Surveyor shall be responsible for all surveying required to layout and control the Work. Surveying shall be conducted such that all applicable standards required by the State of California.

B. PERMITS – The Contractor shall be required to obtain permits in accordance with construction of the facility.

C. SEDIMENTATION, EROSION CONTROL, AND DEWATERING – Contractor shall comply with all laws, ordinances, and permits for controlling erosion, water pollution, and dust emissions resulting from construction activities; the Contractor shall be responsible for any fines imposed due to noncompliance. The Contractor shall perform work in accordance with the Storm Water Pollution Prevention Plan (SWPPP) provided by the Owner. The Contractor shall pump all water generated from dewatering into Cell 3, as directed by the Construction Manager.

D. PROTECTION OF EXISTING SERVICES AND WELLS – The Contractor shall exercise care to avoid disturbing or damaging the existing monitor wells, electrical poles and lines, permanent below-ground utilities, permanent drainage structures, and temporary utilities and structures. When the Work requires the Contractor to be near or to cross locations of known utilities, the Contractor shall carefully uncover, support, and protect these utilities and shall not cut, damage, or otherwise disturb them without prior authorization from the Construction Manager. All utilities or wells damaged by the Contractor shall be immediately repaired by the Contractor to the satisfaction of the Construction Manager at no additional cost.

E. BURNING – The use of open fires for any reason is prohibited.

F. TEMPORARY ROADS – The Contractor shall be responsible for constructing and maintaining all temporary roads and lay down areas that the Contractor may require in the execution of the Work.

G. CONSTRUCTION WATER – The Contractor shall obtain water from the Owner for construction and dust control. The Contractor shall not add substances (such as soap) to construction water.

H. COOPERATION – The Contractor shall cooperate with all other parties engaged in project-related activities to the greatest extent possible. Disputes or problems should be referred to the Construction Manager for resolution.

I. FAMILIARIZATION – The Contractor is responsible for becoming familiar with all aspects of the Work prior to performing the Work.

J. SAFEGUARDS – The Contractor shall provide and use all personnel safety equipment, barricades, guardrails, signs, lights, flares, and flagmen as required by Occupational Safety and Health Administration (OSHA), state, or local codes and ordinances. No excavations deeper than 4 feet with side slopes steeper than 2:1 (horizontal:vertical) shall be made without the prior approval of the Design Engineer and the Construction Manager. When shoring is required, the design and inspection of such shoring shall be the Contractor's responsibility and shall be subject to the review of the Design Engineer and Construction Manager prior to use. No personnel shall work within or next to an excavation requiring shoring until such shoring has been installed, inspected, and approved by an engineer registered in the State of Utah. The Contractor shall be responsible for any fines imposed due to violation of any laws and regulations relating to the safety of the Contractor's personnel.

K. CLEAN-UP – The Contractor shall be responsible for general housekeeping during construction. Upon completion of the Work, the Contractor shall remove all of his equipment, facilities, construction materials, and trash. All disturbed surface areas shall be re-paved, re-vegetated, or otherwise put into the pre-existing condition before performing the Work, or a condition satisfactory to the Construction Manager.

L. SECURITY – The Contractor is responsible for the safety and condition of all of his tools and equipment.

M. ACCEPTANCE OF WORK – The Contractor shall retain ownership and responsibility for all Work until accepted by Construction Manager. Construction Manager will accept ownership and responsibility for the Work: (i) when all Work is completed; and (ii) after the Contractor has submitted all required documentation, including manufacturing quality control documentation and manufacturing certifications.

PART 2 – PRODUCTS

NOT USED.

PART 3 – EXECUTION

NOT USED.

PART 4 – MEASUREMENT AND PAYMENT

NOT USED.

[END OF SECTION]

SECTION 01025
MEASUREMENT AND PAYMENT

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. This section covers measurement and payment criteria applicable to the Work performed under lump sum and unit price payment methods, and non-payment for rejected work.

1.02 RELATED SECTIONS

A. This section relates to all other sections of the contract.

1.03 AUTHORITY

A. Measurement methods delineated in the individual specification sections are intended to complement the criteria of this section. In the event of conflict, the requirements of the individual specification section shall govern.

B. A surveyor, licensed in the State of Utah, hired by the Contractor will take all measurements and compute quantities accordingly. All measurements, cross-sections, and quantities shall be stamped and certified by the licensed surveyor and submitted to the Construction Manager. The Construction Manager maintains the right to provide additional measurements and calculation of quantities to verify measurements and quantities submitted by the Contractor.

1.04 UNIT QUANTITIES SPECIFIED

A. Quantities and measurements indicated in the Bid Schedule are for bidding and contract purposes only. Quantities and measurements supplied or placed in the Work and verified by the Construction Manager shall determine payment. If the actual work requires more or fewer quantities than those quantities indicated, the Contractor shall provide the required quantities at the lump sum and unit prices contracted unless modified elsewhere in these Contract Documents.

B. Utah sales tax shall be included in each bid item as appropriate.

1.05 MEASUREMENT OF QUANTITIES

A. Measurement by Volume: Measurement shall be by the cubic dimension using mean lengths, widths, and heights or thickness, or by average end area method as measured by the surveyor. All measurement shall be the difference between the original ground surface and the design ("neat-line") dimensions and grades.

B. Measurement by Area: Measurement shall be by the square dimension using mean lengths and widths and/or radius as measured by the surveyor. All measurement shall be the difference between the original ground surface and the design ("neat-line") dimensions and grades.

C. Linear Measurement: Measurement shall be by the linear dimension, at the item centerline or mean chord. All measurement shall be the difference between the original ground surface and the design ("neat-line") dimensions and grades.

D. Stipulated Lump Sum Measurement: Items shall be measured as a percentage by weight, volume, area, or linear means or combination, as appropriate, of a completed item or unit of Work.

1.06 PAYMENT

A. Payment includes full compensation for all required labor, products, tools, equipment, transportation, services, and incidentals; erection, application, or installation of an item of the Work; and all overhead and profit. Final payment for Work governed by unit prices will be made on the basis of the actual measurements and quantities accepted by the Construction Manager multiplied by the unit price for Work which is incorporated in or made necessary by the Work.

B. A monthly progress payment schedule will be used to compensate the Contractor for the Work. The monthly amount to be paid to the Contractor is calculated as the percent of completed work for each bid item multiplied by the total anticipated work for that bid item minus a 10 percent retainer.

C. When the Contractor has completed all Work associated with completion of the project, the remaining 10 percent retainer of the contract amount will be paid to the Contractor after filing the Notice of Completion.

1.07 NON-PAYMENT FOR REJECTED PRODUCTS

A. Payment shall not be made for any of the following:

1. Products wasted or disposed of in a manner that is not acceptable.
2. Products determined as unacceptable before or after placement.
3. Products not completely unloaded from the transporting vehicle.
4. Products placed beyond the design lines, dimensions, grades, and levels of the required Work.
5. Products remaining on hand after completion of the Work.
6. Loading, hauling, and disposing of rejected Products.
7. Products rejected because of contamination (i.e. soil residues, fuel spills, solvents, etc.).

1.08 BID ITEMS

A. The following bid items shall be used by the Owner and by the Contractor to bid the Work described in these bid documents.

BID ITEM	SECTION	DESCRIPTION	UNITS
1	01500	Construction Facilities	LS
2	01505	Mobilization / Demobilization	LS
3	02070	Well Abandonment	LF
4	02200	Soil Excavation	CY
5	02200	Rock Excavation	CY
6	02200	Engineered Fill	CY
7	02220	Subgrade Preparation	SF
8	02220	Anchor Trench	LF

BID ITEM	SECTION	DESCRIPTION	UNITS
9	02616	4-inch PVC Pipe and Fittings	LF
10	02616	18-inch PVC Pipe and Fittings	LF
11	02616	Strip Drain Composite	LF
12	02770	60-mil Smooth HDPE Geomembrane	SF
13	02770	60-mil Textured HDPE Geomembrane	SF
14	02772	Geosynthetic Clay Liner	SF
15	02773	300-mil Geonet	SF
16	03400	Cast-In-Place Concrete	LS
17	01505	Performance Bond	LS

PART 2 – PRODUCTS

NOT USED.

PART 3 – EXECUTION

NOT USED.

PART 4 – MEASUREMENT AND PAYMENT

NOT USED.

[END OF SECTION]

SECTION 01300
SUBMITTALS

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. This section contains requirements for administrative and work-related submittals such as construction progress schedules, Shop Drawings, test results, operation and maintenance data, and other submittals required by Contract Documents.

B. Submit required materials to the Construction Manager for proper distribution and review in accordance with requirements of the Contract Documents**.**

1.02 CONSTRUCTION PROGRESS SCHEDULES

A. The Contractor shall prepare and submit two (2) copies of the construction progress Schedule to the Construction Manager for review within five (5) days after the effective date of Contract.

B. Schedules shall be prepared in the form of a horizontal bar chart. The schedule shall include the following items.

1. A separate horizontal bar for each operation.
2. A horizontal time scale, which identifies the first workday of each week.
3. A scale with spacing to allow space for notations and future revisions.
4. Listings arranged in order of start for each item of the Work.

C. The Construction Progress Schedule for construction of the Work shall include the following items where applicable.

1. Submittals: dates for beginning and completion of each major element of construction and installation dates for major items. Elements shall include, but not be limited to, the following items which are applicable:

    a. Mobilization schedule
    b. Demobilization schedule.
    c. Final site clean-up.
    d. Show projected percentage of completion for each item as of first day of each week.
    e. Show each individual Bid Item.

D. Schedule Revisions:

1. Bi-weekly to reflect changes in progress of Work.
2. Indicate progress of each activity at submittal date.
3. Show changes occurring since the previous schedule submittal. Changes shall include the following.

    a. Major changes in scope.
    b. Activities modified since previous submittal.

c. Revised projections of progress and completion.

d. Other identifiable changes.

4. Provide narrative report as needed to define:

a. Problem areas, anticipated delays, and impact on schedule.

b. Recommended corrective action and its effect.

1.03 CONSTRUCTION WORK SCHEDULE

A. The Contractor shall submit an updated 14-day work schedule at the beginning of each week by Monday morning at 8:00 a.m. The schedule shall address applicable line items from the construction project schedule with a refined level of detail for special activities.

1.04 SHOP DRAWINGS AND SAMPLES

A. Shop Drawings, product data, and samples shall be submitted as required in individual Sections of the Specifications.

B. The Contractor's Responsibilities:

1. Review Shop Drawings, product data, and samples prior to submittal.

2. Determine and verify:

a. Field measurements.

b. Field construction criteria.

c. Catalog numbers and similar data.

d. Conformance with Specifications.

3. Coordinate each submittal with requirements of the Work and Contract Documents.

4. Notify the Construction Manager in writing, at the time of the submittal, of deviations from requirements of Contract Documents.

5. Begin no fabrication or Work pertaining to required submittals until return of the submittals with appropriate approval.

6. Designate dates for submittal and receipt of reviewed Shop Drawings and samples in the construction progress schedule.

C. Submittals shall contain:

1. Date of submittal and dates of previous submittals.

2. Project title and number.

3. Contract identification.

4. Names of:

a. The Contractor.

b. Supplier.

c. Manufacturer.

5. Summary of items contained in the submittal.

6. Identification of the product with identification numbers and the Drawing and Specification section numbers.
7. Clearly identified field dimensions.
8. Details required on the Drawings and in the Specifications.
9. Manufacturer, model number, dimensions, and clearances, where applicable.
10. Relation to adjacent or critical features of the Work or materials.
11. Applicable standards, such as ASTM or Federal Specification numbers.
12. Identification of deviations from Contract Documents.
13. Identification of revisions on re-submittals.
14. 8-inch by 3-inch blank space for the Contractor’s and proper approval stamp.
15. The Contractor’s stamp, signed, certifying review of the submittal, verification of the products, field measurements, field construction criteria, and coordination of information within the submittal with requirements of Work and Contract Documents.

D. Re-submittal Requirements:

1. Re-submittal is required when corrections or changes in submittals are required by the Construction Manager, Design Engineer, or CQA Engineer. Re-submittals are required until all comments by the Construction Manager, Design Engineer, or CQA Engineer is addressed and the submittal is approved.
2. Shop Drawings and Product Data:
    a. Revise initial drawings or data and resubmit as specified for initial submittal.
    b. Indicate changes made other than those requested by the Construction Manager, Design Engineer, or CQA Engineer.

E. Distribute reproductions of Shop Drawings and copies of product data which have been accepted by the Construction Manager to:

1. Job site file.
2. Record documents file.

F. Construction Manager’s Duties:

1. Verify that review comments are technically correct and are consistent with technical and contractual requirements of the work.
2. Return submittals to the Contractor for distribution or re-submittal.

G. Design Engineer’s Duties:

1. Review submittals promptly for compliance with contract documents and in accordance with the schedule.
2. Affix stamp and signature, and indicate either the requirements for re-submittal or no comments.
3. Return submittals to the Construction Manager.

H. CQA Engineer's Duties:

1. Review submittals promptly for compliance with contract documents and in accordance with the schedule.
2. Affix stamp and signature, and indicate either the requirements for re-submittal or no comments.
3. Return submittals to the Construction Manager.

1.05 TEST RESULTS AND CERTIFICATION

A. Results of tests conducted by the Contractor on materials or products shall be submitted for review.

B. Certification of products shall be submitted for review.

1.06 SUBMITTAL REQUIREMENTS

A. Provide complete copies of required submittals as follows.

1. Construction Work Schedule:

   a. Two copies of initial schedule.
   b. Two copies of each revision.

2. Construction Progress Schedule:

   a. Two copies of initial schedule.
   b. Two copies of each revision.

3. Shop Drawings: Two copies.
4. Certification Test Results: Two copies.
5. Other Required Submittals:

   a. Two copies, if required, for review.
   b. Two copies, if required, for record.

B. Deliver the required copies of the submittals to the Construction Manager.

PART 2 – PRODUCTS

NOT USED.

PART 3 – EXECUTION

NOT USED.

PART 4 – MEASUREMENT AND PAYMENT

NOT USED.

[END OF SECTION]

SECTION 01400
QUALITY CONTROL

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. Monitor quality control over suppliers, Manufacturers, products, services, Site conditions, and workmanship, to produce Work of specified quality.

B. Comply with Manufacturers' instructions, including each step in sequence.

C. Should Manufacturers' instructions conflict with Technical Specifications, request clarification from Design Engineer before proceeding.

D. Comply with specified standards as minimum quality for the Work except where more stringent tolerances, codes, or specified requirements indicate higher standards or more precise workmanship.

E. Perform Work by persons qualified to produce workmanship of specified quality.

1.02 TOLERANCES

A. Monitor tolerance control of installed products to produce acceptable Work. Do not permit tolerances to accumulate.

B. Comply with Manufacturers' tolerances. Should Manufacturers' tolerances conflict with Technical Specifications, request clarification from Design Engineer before proceeding.

C. Adjust products to appropriate dimensions; position before securing products in place.

1.03 REFERENCES

A. For products or workmanship specified by association, trade, or other consensus standards, complies with requirements of the standard, except when more rigid requirements are specified or are required by applicable codes.

B. Conform to reference standard by date of current issue on date of Notice to Proceed with construction, except where a specific date is established by code.

C. Obtain copies of standards where required by product Specification sections.

1.04 INSPECTING AND TESTING SERVICES

A. The CQA Engineer will perform construction quality assurance (CQA) inspections, tests, and other services specified in individual Sections of the Specification.

B. The Contractor shall cooperate with CQA Engineer; furnish samples of materials, design mix, equipment, tools, storage, safe access, and assistance by incidental labor as requested.

C. CQA testing or inspecting does not relieve Contractor, subcontractors, and suppliers from their requirements to perform quality control Work as indicated in the Technical Specifications.

PART 2 – PRODUCTS

NOT USED.

PART 3 – EXECUTION

NOT USED.

PART 4 – MEASUREMENT AND PAYMENT

NOT USED.

[END OF SECTION]

SECTION 01500
CONSTRUCTION FACILITIES

PART 1 – GENERAL

1.01 SECTION INCLUDES

A. Construction facilities include furnishing of all equipment, materials, tools, accessories, incidentals, labor, and performing all work for the installation of equipment and for construction of facilities, including their maintenance, operation, and removal, if required, at the completion of the Work under the Contract.

1.02 DESCRIPTION OF WORK

A. Construction facilities include, but are not limited to, the following equipment, materials, facilities, areas, and services:

1. Parking Areas.
2. Temporary Roads.
3. Storage of Materials and Equipment.
4. Construction Equipment.
5. Temporary Sanitary Facilities.
6. Temporary Water.
7. First Aid Facilities.
8. Health and Safety.
9. Security.

B. Construct/install, maintain, and operate construction facilities in accordance with the applicable federal, state, and local laws, rules, and regulations, and the Contract Documents.

1.03 GENERAL REQUIREMENTS

A. Contractor is responsible for furnishing, installing, constructing, operating, maintaining, removing, and disposing of the construction facilities, as specified in this Section, and as required for the completion of the Work under the Contract.

B. Contractor shall maintain construction facilities in a clean, safe, and sanitary condition at all times until completion of the Work.

C. Contractor shall minimize land disturbances related to the construction facilities to the greatest extent possible and restore land, to the extent reasonable and practical, to its original contours by grading to provide positive drainage and by seeding the area to match with existing vegetation or as specified elsewhere.

1.04 TEMPORARY ROADS AND PARKING AREAS

A. Temporary roads and parking areas are existing roads that are improved or new roads constructed by Contractor for convenience of Contractor in the performance of the Work under the Contract.

B. Contractor shall coordinate construction with Construction Manager.

C. If applicable, coordinate all road construction activities with local utilities, fire, and police departments.

D. Keep erosion to a minimum and maintain suitable grade and radii of curves to facilitate ease of movement of vehicles and equipment.

E. Furnish and install longitudinal and cross drainage facilities, including, but not limited to, ditches, structures, pipes and the like.

F. Clean equipment so that mud or dirt is not carried onto public roads. Clean up any mud or dirt transported by equipment on paved roads both on-site and off-site.

1.05 STORAGE OF MATERIALS AND EQUIPMENT

A. Make arrangements for material and equipment storage areas. Locations and configurations of approved facilities are subject to the acceptance of the Construction Manager.

B. Confine all operations, including storage of materials, to approved areas. Store materials in accordance with these Technical Specifications and the Construction Drawings.

C. Store construction materials and equipment within boundaries of designated areas. Storage of gasoline or similar fuels must conform to state and local regulations and be limited to the areas approved for this purpose by the Construction Manager.

1.06 CONSTRUCTION EQUIPMENT

A. Erect, equip, and maintain all construction equipment in accordance with all applicable statutes, laws, ordinances, rules, and regulations or other authority having jurisdiction.

B. Provide and maintain scaffolding, staging, hoists, barricades, and similar equipment required for performance of the Work. Provide hoists or similar equipment with operators and signals, as required.

C. Provide, maintain, and remove upon completion of the Work, all temporary rigging, scaffolding, hoisting equipment, debris boxes, barricades around openings and excavations, fences, ladders, and all other temporary work, as required for all Work hereunder.

D. Construction equipment and temporary work must conform to all the requirements of state, county, and local authorities, OSHA, and underwriters that pertain to operation, safety, and fire hazard. Furnish and install all items necessary for conformity with such requirements, whether or not called for under separate Sections of these Technical Specifications.

1.07 TEMPORARY SANITARY FACILITIES

A. Provide temporary sanitary facilities for use by all employees and persons engaged in the Work, including subcontractors, their employees and authorized visitors, and the Construction Manager.

B. Sanitary facilities include enclosed chemical toilets and washing facilities. These facilities must meet the requirements of local public health standards.

C. Locate sanitary facilities as approved by Construction Manager, and maintain in a sanitary condition during the entire course of the Work.

1.08 TEMPORARY WATER

A. Make all arrangements for water needs from the Owner.

B. Provide drinking water for all personnel at the site.

1.09 FIRST AID FACILITIES

A. Provide first aid equipment and supplies to serve all Contractor personnel at the Site.

1.10 HEALTH AND SAFETY

A. Provide necessary monitoring equipment and personal protective equipment in accordance with Contractor prepared Site Health and Safety Plan.

1.11 SECURITY

A. Make all necessary provisions and be responsible for the security of the Work and the Site until final inspection and acceptance of the Work, unless otherwise directed by the Construction Manager.

1.12 SHUT-DOWN TIME OF SERVICE

A. Do not disconnect or shut down any part of the existing utilities and services, except by express permission of Construction Manager.

1.13 MAINTENANCE

A. Maintain all construction facilities, utilities, temporary roads, and the like in good working condition as required by the Construction Manager during the term of the Work.

1.14 STATUS AT COMPLETION

A. Upon completion of the Work, or prior thereto, when so required by Construction Manager:

1. Repair damage to roads caused by or resulting from the Contractor's work or operations.
2. Remove and dispose of all construction facilities. Similarly, all areas utilized for temporary facilities shall be returned to near original, natural state, or as otherwise indicated or directed by the Construction Manager.

PART 2 – PRODUCTS

NOT USED**.**

PART 3 – EXECUTION

NOT USED.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for Construction Facilities as lump sum (LS) and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

1. Mobilization.
2. Temporary roadways and parking areas.
3. Temporary sanitary facilities.
4. Decontamination of equipment.
5. Security.
6. Demobilization.

[END OF SECTION]

SECTION 01505
MOBILIZATION / DEMOBILIZATION

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. Mobilization consists of preparatory work and operations, including but not limited to those necessary for the movement of personnel and project safety; including: adequate personnel, equipment, supplies, and incidentals to the project Site; establishment of facilities necessary for work on the project; premiums on bond and insurance for the project and for other work and operations the Contractor must perform or costs the Contractor must incur before beginning work on the project, which are not covered in other bid items.

B. Demobilization consists of work and operations including, but not limited to, movement of personnel, equipment, supplies, and incidentals off-site.

PART 2 – PRODUCTS

NOT USED.

PART 3 – EXECUTION

NOT USED.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section as lump sum (LS) and payment will be based on the unit price provided on the Bid Schedule.

B. The Contract Price for Mobilization/Demobilization shall include the provision for movement of equipment onto the job site; removal of all facilities and equipment at the completion of the project; permits; preparation of a Health and Safety Plan; all necessary safety measures; and all other related mobilization and demobilization costs. Price bid for mobilization shall not exceed 10 percent of the total bid for the Project. Fifty percent of the mobilization bid price, less retention, will be paid on the initial billing provided all equipment and temporary facilities are in place and bond fees paid. The remaining 50 percent of the mobilization bid price will be paid on satisfactory removal of all facilities and equipment on completion of the project.

[END OF SECTION]

SECTION 01560
TEMPORARY CONTROLS

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. Temporary Controls required during the term of the Contract for the protection of the environment and the health and safety of workers and general public.

B. Furnishing all equipment, materials, tools, accessories, incidentals, and labor, and performing all work for the installation of equipment and construction of facilities, including their maintenance and operation during the term of the Contract.

C. Temporary Controls include:

1. Dust Control.
2. Pollution Control.
3. Traffic and Safety Controls.

D. Perform Work as specified in the Technical Specifications and as required by the Construction Manager. Maintain equipment and accessories in clean, safe, and sanitary condition at all times until completion of the Work.

1.02 DUST CONTROL

A. Provide dust control measures in-accordance with the Technical Specifications. Dust control measures must meet requirements of applicable laws, codes, ordinances, and permits.

B. Dust control consists of transporting water, furnishing required equipment, testing of equipment, additives, accessories and incidentals, and carrying out proper and efficient measures wherever and as often as necessary to reduce dust nuisance, and to prevent dust originating from construction operations throughout the duration of the Work.

1.03 POLLUTION CONTROL

A. Pollution of Waterways:

1. Perform Work using methods that prevent entrance or accidental spillage of solid or liquid matter, contaminants, debris, and other objectionable pollutants and wastes into watercourses, flowing or dry, and underground water sources.
2. Such pollutants and wastes will include, but will not be limited to, refuse, earth and earth products, garbage, cement, concrete, sewage effluent, industrial waste, hazardous chemicals, oil and other petroleum products, aggregate processing tailings, and mineral salts.

B. Dispose of pollutants and wastes in accordance with applicable permit provisions or in a manner acceptable to and approved by the Construction Manager.

C. Storage and Disposal of Petroleum Product:

1. Petroleum products covered by this Section include gasoline, diesel fuel, lubricants, and refined and used oil. During project construction, store all petroleum products in such a way as to prevent contamination of all ground and surface waters and in accordance with local, state, and federal regulations.

2. Lubricating oil may be brought into the project area in steel drums or other means, as the Contractor elects. Store used lubricating oil in steel drums, or other approved means, and return them to the supplier for disposal. Do not burn or otherwise dispose of at the Site.

3. Secondary containment shall be provided for products stored on site, in accordance with the Owner provided Storm Water Pollution Prevention Plan.

1.04 TRAFFIC AND SAFETY CONTROLS

A. Post construction areas and roads with traffic control signs or devices used for protection of workmen, the public, and equipment. Signs and devices must conform to the American National Standards Institute (ANSI) Manual on Uniform Traffic Control Devices for Streets and Highways.

B. Remove signs or traffic control devices after they have finished serving their purpose. It is particularly important to remove any markings on road surfaces that under conditions of poor visibility could cause a driver to turn off the road or into traffic moving in the opposite direction.

C. Provide flag persons, properly equipped with International Orange protective clothing and flags, as necessary, to direct or divert pedestrian or vehicular traffic. A full-time flag person shall be required for the duration of importation of fill.

D. Barricades for protection of employees must conform to the portions of the ANSI Manual on Uniform Traffic Control Devices for Streets and Highways, relating to barricades.

E. Guard and protect all workers, pedestrians, and the public from excavations, construction equipment, all obstructions, and other dangerous items or areas by means of adequate railings, guard rails, temporary walks, barricades, warning signs, sirens, directional signs, overhead protection, planking, decking, danger lights, etc.

F. Construct and maintain fences, planking, barricades, lights, shoring, and warning signs as required by local authorities and federal and state safety ordinances, and as required to protect all property from injury or loss and as necessary for the protection of the public, and provide walks around any obstructions made in a public place for carrying out the Work covered in this Contract. Leave all such protection in place and maintained until removal is authorized by the Construction Manager.

1.05 MAINTENANCE

A. Maintain all temporary controls in good working conditions during the term of the Contract for the safe and efficient transport of equipment and supplies, and for construction of permanent works.

1.06 STATUS AT COMPLETION

A. Upon completion of the Work, or prior thereto as approved by the Construction Manager, remove all temporary controls and restore disturbed areas.

PART 2 – PRODUCTS

NOT USED.

PART 3 – EXECUTION

NOT USED.

PART 4 – MEASUREMENT AND PAYMENT

4.01 TEMPORARY CONTROLS

A. Temporary Controls: the measurement and payment of temporary controls shall be in accordance with and as a part of Mobilization/Demobilization, as outlined in Section 01505.

[END OF SECTION]

SECTION 01700
CONTRACT CLOSEOUT

PART 1 – GENERAL

1.01 CLOSEOUT PROCEDURES

A. Contractor shall submit written certification that the Technical Specifications, CQA Plan, and Drawings have been reviewed, Work has been inspected, and that Work is complete and in-accordance with the Technical Specifications, CQA Plan, and Drawings and ready for Owner's inspection.

1.02 FINAL CLEANING

A. Contractor shall execute final cleaning prior to final inspection.

B. Contractor shall clean equipment and fixtures to a sanitary condition.

C. Contractor shall remove waste and surplus materials, rubbish, and construction facilities from the construction Site.

1.03 PROJECT RECORD DOCUMENTS

A. Maintain on Site, one set of the following record documents and record actual revisions to the Work.

1. Drawings.
2. Specifications.
3. Addenda.
4. Change Orders and other Modifications to the Contract.
5. Reviewed Shop Drawings, product data, and samples.

B. Store Record Documents separate from documents used for construction.

C. Record information concurrent with construction progress.

D. Specifications: Legibly mark and record at each product Section a description of actual products installed, including the following:

1. Manufacturer's name and product model and number.
2. Product substitutions or alternates utilized.
3. Changes made by Addenda and Modifications.

E. Record Documents and Shop Drawings: Legibly mark each item to record actual construction including:

1. Measured horizontal and vertical location of underground utilities and appurtenances referenced to permanent surface features.
2. Measured locations of internal utilities and appurtenances concealed in construction, referenced to visible, accessible, and permanent features of the Work.
3. Field changes of dimension and detail.

4. Details not shown on original Construction Drawings.

F. Submit record documents to the Construction Manager.

PART 2 – PRODUCTS

NOT USED.

PART 3 – EXECUTION

NOT USED.

PART 4 – MEASUREMENT AND PAYMENT

4.01 CONTRACT CLOSEOUT

A. Contract Closeout: the measurement and payment of contract close out shall be in accordance with and as part of Mobilization/Demobilization, as outlined in Section 01505.

[END OF SECTION]

SECTION 02070
WELL ABANDONMENT

PART 1 — GENERAL

1.01 DESCRIPTION OF WORK

A. Supply all equipment, materials, and labor needed to abandon one (1) 4-inch diameter polyvinyl chloride (PVC) casing groundwater monitoring well as specified herein and as indicated on the Drawings.

B. Well abandonment shall be accomplished under the direct supervision of a currently licensed water well driller who shall be responsible for verification of the procedures and materials used.

1.02 RELATED SECTIONS

Section 01025 – Measurement and Payment

Section 01300 – Submittals

Section 01400 – Quality Control

1.03 REFERENCES

A. Drawings.

B. Construction Quality Assurance (CQA) Plan

C. Latest version of the American Society for Testing and Materials (ASTM) standards:

ASTM C-150 Standard Specification for Portland Cement.

D. Latest version of the American Petroleum Institute (API) standards:

API - 13A Specification for Drilling-Fluid Materials

1.04 SUBMITTALS

A. The Contractor shall keep detailed drilling logs for all wells abandoned, including drilling procedures, total depth of abandonment, depth to groundwater (if applicable), final depth of boring, and well destruction details, including the depths of placement of all well abandonment materials. The Contractor shall provide a minimum of 7 days advance notice prior to beginning drilling and shall submit a list of the type and quantity of materials used for well abandonment.

B. The Contractor shall acquire all necessary permits and prepare and file a well abandonment report as required by the State of Utah, Division of Water Rights.

PART 2 — PRODUCTS

2.01 BENTONITE

A. Bentonite shall be Volclay (powdered sodium bentonite API-13A) or as otherwise approved by the Engineer.

2.02 WATER

A. Water used in the grout mixture shall be potable water or disinfected in accordance with R655-4-9.6.5 Utah Administrative Code (UAC).

2.03 CEMENT

A. Cement shall be Portland Type I (ASTM C-150).

PART 3 — EXECUTION

3.01 GENERAL

A. The Contractor is responsible for obtaining all permits for the abandonment of wells and shall be responsible for following all regulatory requirements as outlined in the Administraive Rules for Water Well Drillers R655-4 UAC.

B. The Contractor shall be responsible for reviewing the well construction boring log for the groundwater well to be abandoned. The original construction boring logs for the well to be abandoned are attached to the end of this Section, as Exhibit I.

3.02 DRILLING

A. The Contractor shall sound and record the total depth of the well casing, depth to groundwater (if encountered), and depth of the over boring.

B. Each well shall be over bored to a diameter 3 inches greater than the well casing diameter to a depth of 10 feet below the proposed Cell 4B base elevation. The exact depth of the wells shall be in accordance with the Contract Documents and as determined by the Design Engineer.

3.03 CEMENT-BENTONITE GROUT

A. A cement-bentonite grout, shall be mixed for each well. The cement-bentonite grout shall have approximately 2% by weight bentonite (i.e. one 94-lbs sack of cement and two lbs. of bentonite) and be mixed with approximately 6.5 gallons of water. The cement-bentonite grout shall be mixed using a recirculating pump to form a homogeneous mixture free from lumps.

B. Immediately after removing all well materials and recording the over bored depth, the slurry shall be pressure grouted into the well borehole to 10 feet below ground surface (bgs).

C. The uppermost 10 feet of the abandoned well shall consist of neat cement grout or sand cement grout.

D. The Contractor shall monitor the mass, volume, and level of cement-bentonite grout placed in each well borehole. These quantities shall be reported to the Engineer during the abandonment process.

E. The cement grout or sand cement grout shall be allowed to settle. Cement grout or sand cement grout shall be added, as necessary, until the elevation of the cured and settled cement grout or sand cement grout conforms to the surface topography at the time of abandonment.

PART 4 — MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements for well abandonment set forth in this Section will be measured as each well; and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

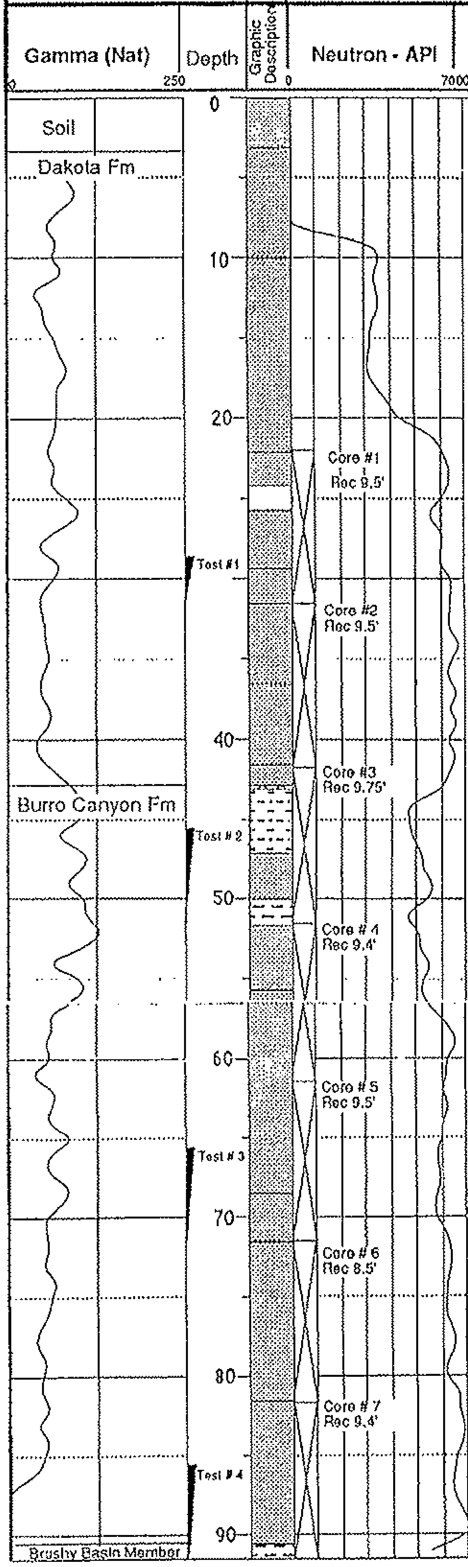
1. Submittals.
2. Bentonite.
3. Water.
4. Cement.
5. Well permits.
6. Mobilization.
7. Decontamination of well abandonment equipment.
8. Disposal of decontamination materials.
9. Disposal of drill cuttings.

[END OF SECTION]

PEEL Environmental Services
Arvada, CO (303)422-5116

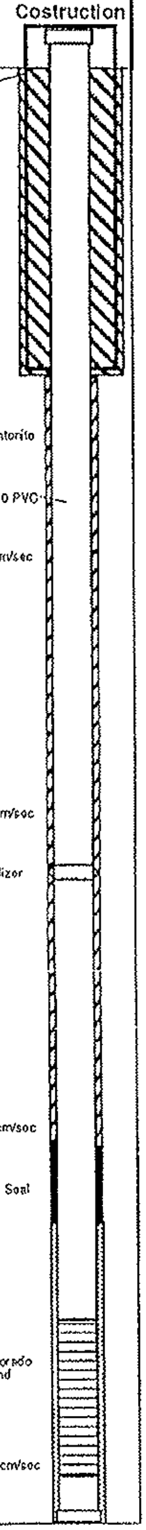
Project: White Mesa	Surface Elev. 5588.18	T. D. = 91.5	UMETCO Minerals Corporation
Date: 12/07/92	Depth to Water: Dry	Geologist: F. A. Peel	

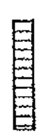
Bore Hole No. : WMMW-16



Sample Description	Comments
Sand: quartz,reddish brown,fine-grained subrounded silty.	1/4" Steel Surface Csg
Sandstone: quartz, reddish brown, very fine grained, subround, silty friable.	
Sandstone: quartz, light buff, very fine- to fine-grained, subangular to angular, friable, good inter granular porosity.	
Sandstone: quartz, light buff to light gray, very fine- to fine-grained,kaolinitic, massive to thin bedded,rough cross bedding trace porosity.	Cement/Bentonite Grout
Claystone: light gray, silty, slightly sandy,thin carbonaceous partings, hard.	
Sandstone: quartz, light grey, very fine grained, subrounded, kaolinitic, thin cross bedding, hard.	4" schedule 40 PVC
Sandstone: quartz, light yellow gray, fine to medium grained, subrounded to round kaolinitic, tr iron staining, thin cross bedding.	K= 9.1E-4 cm/sec
Sandstone: quartz, light gray, medium to coarse grained, kaolinitic, conglomeratic pebbles are angular lithic fragments.	
Sandstone: quartz, light gray, fine to medium grained, subrounded, silty, trace intergranular porosity, occasionally coarse grained, occasional pebble.	
Sandstone: quartz, light grey, fine- to medium-grained, subround to rounded. Conglomerate: Brownish gray, angular to subangular, chert & sandstone clasts, Siltstone: greenish gray,sandy in part, occasional iron staining.	
	K= 5.1E-5 cm/sec
Sandstone: quartz, light greenish gray, very fine grained silty.	
Shale: greenish gray, thinbedded soft, bentonitic.	Centralizer
Sandstone: quartz, light buff to light gray, very fine grained, sub angular, trace intergranular porosity, limonite concretions, trace iron staining.	
Sandstone: quartz, light brownish gray, fine grained, thin cross bedding, trace porosity, becomming greenish grey & very fine grained toward base, shale parting at 64'	
	K= 7.0E-5 cm/sec
Sandstone: quartz, light brownish gray, fine grained, well sorted, good intergranular porosity, 1" shale iron stained shale parting at top.	Bentonite Seal
Sandstone: quartz, light grey, grading downward from very fine grained to medium grained, subrounded, layers well sorted, kaolinitic, conglomeratic in part, trace iron staining.	
Sandstone: quartz, light gray, medium grained, subangular to subround, well sorted, kaolinitic, poor to good intergranular porosity, occasional coarse sand grains and pebble conglomerate stingers, trace iron staining.	10-20 Colorado Silica Sand
	K= 2.0E-5 cm/sec
Shale: dark green, thinbedded, soft.	Well Dry

Well Costruction

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SECTION 02200
EARTHWORK

PART 1 — GENERAL

1.01 DESCRIPTION OF WORK

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary to perform all Earthwork. The Work shall be carried out as specified herein and in accordance with the Drawings.

B. The Work shall include, but not be limited to excavating, blasting, ripping, trenching, hauling, placing, moisture conditioning, backfilling, compacting and grading. Earthwork shall conform to the dimensions, lines, grades, and sections shown on the Drawings or as directed by the Construction Manager.

1.02 RELATED SECTIONS

Section 02220 – Subgrade Preparation

1.03 REFERENCES

A. Drawings

B. Latest version of American Society for Testing and Materials (ASTM) standards:

ASTM D 422	Standard Method for Particle-Size Analysis of Soils
ASTM D 1557	Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lb-ft/ft^3 (2,700 kN-m/m^3))
ASTM D 6938	Standard Test Method for In-Place Density and Water Content of Soil-Aggregate by Nuclear Methods (Shallow Depth)

1.04 QUALIFICATIONS

A. The Contractor's Site superintendent for the earthworks operations shall have supervised the construction of at least two earthwork construction projects in the last 5 years.

1.05 SUBMITTALS

A. The Contractor shall submit to the Construction Manager a description of equipment and methods proposed for excavation, and fill placement and compaction construction at least 14 days prior to the start of activities covered by this Section.

B. If rock blasting is the chosen rock removal technique, the Contractor shall submit to the Construction Manager a blast plan describing blast methods to remove rock to proposed grade. The blast plan shall include a pre-blast survey, blast schedule, seismic monitoring records, blast design and diagrams, and blast safety. The Contractor shall submit the plan to the Construction Manager at least 21 days prior to blast.

C. If the Work of this Section is interrupted for reasons other than inclement weather, the Contractor shall notify the Construction Manager a minimum of 48 hours prior to the resumption of Work.

D. If foreign borrow materials are proposed to be used for any earthwork material on this project, the Contractor shall provide the Construction Manager information regarding the source of the material. In addition, the Contractor shall provide the Construction Manager an opportunity to obtain samples for conformance testing 14 days prior to delivery of foreign borrow materials to the Site. If conformance testing fails to meet these Specifications, the Contractor shall be responsible for reimbursing the Owner for additional conformance testing costs.

E. The Contractor shall submit as-built Record Drawing electronic files and data, to the Construction Manager, within 7 days of project substantial completion, in accordance with this Section.

1.06 QUALITY ASSURANCE

A. The Contractor shall ensure that the materials and methods used for Earthwork meet the requirements of the Drawings and this Section. Any material or method that does not conform to these documents, or to alternatives approved in writing by the Construction Manager will be rejected and shall be repaired, or removed and replaced, by the Contractor at no additional expense to the Owner.

B. The Contractor shall be aware of and accommodate all monitoring and field/laboratory conformance testing required by the Contract Documents. This monitoring and testing, including random conformance testing of construction materials and completed Work, will be performed by the CQA Engineer. If nonconformances or other deficiencies are found in the materials or completed Work, the Contractor will be required to repair the deficiency or replace the deficient materials at no additional cost to the Owner.

PART 2 — PRODUCTS

2.01 MATERIAL

A. Fill material shall consist of on-site soil obtained from excavation or owner provided stockpile and shall be free from rock larger than 6 inches, organic or other deleterious material.

B. Rock shall consist of all hard, compacted, or cemented materials that require blasting or the use of ripping and excavating equipment larger than defined for common excavation. The excavation and removal of isolated boulders or rock fragments larger than 1 cubic yard encountered in materials otherwise conforming to the definition of common excavation shall be classified as rock excavation. The presence of isolated boulders or rock fragments larger than 1 cubic yard is not in itself sufficient to cause to change the classification of the surrounding material.

C. Rippable Soil and Rock: Material that can be ripped at more than 250 cubic yards per hour for each Caterpillar D9 dozer (or equivalent) with a single shank ripper attachment.

2.02 EQUIPMENT

A. The Contractor shall furnish, operate, and maintain compaction equipment as is necessary to produce the required in-place soil density and moisture content.

B. The Contractor shall furnish, operate and maintain tank trucks, pressure distributors, or other equipment designed to apply water uniformly and in controlled quantities.

C. The Contractor shall furnish, operate, and maintain miscellaneous equipment such as earth excavating equipment, earth hauling equipment, and other equipment, as necessary for Earthwork construction.

D. The Contractor shall be responsible for cleaning up all fuel, oil, or other spills, at the expense of the Contractor, and to the satisfaction of the Construction Manager.

PART 3 — EXECUTION

3.01 FAMILIARIZATION

A. Prior to implementing any of the Work in this Section, the Contractor shall become thoroughly familiar with the Site, the Site conditions, and all portions of the Work falling within this and other related Sections.

B. Inspection:

1. The Contractor shall carefully inspect the installed Work of all other Sections and verify that all Work is complete to the point where the installation of the Work specified in this Section may properly commence without adverse impact.

2. If the Contractor has any concerns regarding the installed Work of other Sections, the Construction Manager shall be notified in writing prior to commencing Work. Failure to notify the Construction Manager, or commencement of the Work of this Section, will be construed as Contractor's acceptance of the related Work of all other Sections.

3.02 SOIL EXCAVATION

A. The Contractor shall excavate materials to the limits and grades shown on the Drawings.

B. The Contractor shall rip, blast, and mechanically remove rock 6-inches below final grades shown on the Drawings.

C. All excavated material not used as fill shall be stockpiled as shown on the Drawings and in accordance with Subpart 3.05 of this Section.

3.03 ROCK EXCAVATION

A. The Contractor shall remove rock by ripping, drilling, or blasting, or as approved by Construction Manager.

B. Requirements for Blasting:

1. The Contractor shall arrange for a pre-blast survey of nearby buildings, berms, or other structures that may potentially be at risk from blasting damage. The survey method used shall be acceptable to the Contractor’s insurance company. The Contractor shall be responsible for any damage resulting from blasting. The preblast survey shall be made available for review three weeks before any blasting begins. Pre-blast surveys shall be completed by a practicing civil engineer registered in the State of Utah, who has experience in rock excavation and geotechnical design.

2. The Contractor shall submit for review the proposed methods and sequence of blasting for rock excavations. The Contractor shall identify the number, depth, and spacing of holes; stemming and number and type of delays; methods of controlling overbreak at excavation limits, procedures for monitoring the shots and recording information for each shot; and other data that may be required to control the blasting.

3. Blasting shall be done in accordance with the federal, state, or local regulatory requirements for explosives and firing of blasts. Such regulations shall not relieve the Contractor of any responsibility for damages caused by them or their employees due to the work of blasting. All blasting work must be performed or supervised by a licensed blaster who shall at all times have a license on their person and shall permit examination thereof by the Engineer or other officials having jurisdiction.

4. The Contractor shall develop a trial blasting technique that identifies and limits the vibrations and damage at varying distances from each shot. This trial blasting information shall be collected and recorded by beginning the work at points farthest from areas to remain without damage. The Contractor can vary the hole spacing, depths and orientations, explosive types and quantities, blasting sequence, and delay patterns to obtain useful information to safeguard against damage at critical areas.

5. Establish appropriate maximum limit for peak particle velocity for each structure or facility that is adjacent to, or near blast sites. Base maximum limits on expected sensitivity of each structure or facility to blast induced vibrations and federal, state, or local regulatory requirements.

6. The Contractor shall discontinue any method of blasting which leads to overshooting or is dangerous to the berms surrounding the existing pond structures.

7. The Contractor shall install a blast warning sign to display warning signals. Sign shall indicate the following:

    a. Five (5) minutes before blast: Three (3) long sounds of airhorn or siren

    b. Immediately before blast: Three (3) short sounds of airhorn or siren

    c. All clear signal after blast: one (1) long sound of airhorn or siren

3.04 FILL

A. Prior to fill placement, areas to receive fill shall be cleared and grubbed.

B. The fill material shall be placed to the lines and grades shown on the Drawings.

C. Soil used for fill shall meet the requirements of Subpart 2.01 of this Section.

D. Soil used for fill shall be placed in a loose lift that results in a compacted lift thickness of no greater 8 inches and compacted to 90% of the maximum density at a moisture content of between -3% and +3% of optimum moisture content, as determined by ASTM D 1557.

E. The Contractor shall utilize compaction equipment suitable and sufficient for achieving the soil compaction requirements.

F. During soil wetting or drying, the material shall be regularly disced or otherwise mixed so that uniform moisture conditions in the appropriate range are obtained.

3.05 STOCKPILING

A. Soil suitable for fill and excavated rock that is required to be stockpiled shall be stockpiled, separately, in areas as shown on the Drawings or as designated by the Construction Manager, and shall be free of incompatible soil, clearing debris, or other objectionable materials.

B. Stockpiles shall be no steeper than 2H:1V (Horizontal:Vertical) or other slope approved by the Engineer, graded to drain, sealed by tracking parallel to the slope with a dozer or other means approved by the Construction Manager, and dressed daily during periods when fill is taken from the stockpile. The Contractor shall employ temporary erosion and sediment control measures (i.e. silt fence) as directed by the Construction Manager around stockpile areas.

C. There are no compaction requirements for stockpiled materials.

3.06 FIELD TESTING

A. The minimum frequency and details of quality control testing for Earthwork are provided below. This testing will be performed by the Construction Quality Assurance (CQA) Engineer. The Contractor shall take this testing frequency into account in planning the construction schedule.

1. The CQA Engineer will perform conformance tests on placed and compacted fill to evaluate compliance with these Specifications. The dry density and moisture content of the soil will be measured in-situ with a nuclear moisture-density gauge in accordance with ASTM D 6938. The frequency of testing will be one test per 500 cubic yards of soil place.

2. A special testing frequency will be used by the CQA Engineer when visual observations of construction performance indicate a potential problem. Additional testing will be considered when:

   a. The rollers slip during rolling operation;
   b. The lift thickness is greater than specified;
   c. The fill is at improper and/or variable moisture content;
   d. Fewer than the specified number of roller passes are made;
   e. Dirt-clogged rollers are used to compact the material;
   f. The rollers do not have optimum ballast; or
   g. The degree of compaction is doubtful.

3. During construction, the frequency of testing will be increased by the Construction Manager in the following situations:

   a. Adverse weather conditions;
   b. Breakdown of equipment;
   c. At the start and finish of grading;
   d. If the material fails to meet Specifications; or
   e. The work area is reduced.

B. Defective Areas:

1. If a defective area is discovered in the Earthwork, the CQA Engineer will evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Engineer will determine the extent of the defective area by additional tests, observations, a review of records, or other means that the Construction Manager deems appropriate. If the defect is related to adverse Site conditions, such as overly wet soils or surface desiccation, the CQA Engineer shall define the limits and nature of the defect.

2. Once the extent and nature of a defect is determined, the Contractor shall correct the deficiency to the satisfaction of the CQA Engineer. The Contractor shall not perform additional Work in the area until the Construction Manager approves the correction of the defect.

3. Additional testing may be performed by the CQA Engineer to verify that the defect has been corrected. This additional testing will be performed before any additional Work is allowed in the area of deficiency. The cost of the additional Work and the testing shall be borne by the Contractor.

3.07 SURVEY CONTROL

A. The Contractor shall perform all surveys necessary for construction layout and control.

3.08 CONSTRUCTION TOLERANCE

A. The Contractor shall perform the Earthwork construction to within ±0.1 vertical feet of elevations on the Drawings.

3.09 AS-BUILT SURVEY

A. For purposes of payment on Earthwork quantities, the Contractor shall conduct a comprehensive as-built survey that complies with this Section.

B. The Contractor shall produce complete electronic as-built Record Drawings in conformance with the requirements set forth in this Section. This electronic file shall be provided to the Construction Manager for verification.

C. The Contractor shall produce an electronic boundary file that accurately conforms to the project site boundary depicted on the plans or as modified during construction by approved change order. The electronic file shall be provided to the Construction Manager for verification prior to use in any earthwork computations or map generation.

D. As-built survey data shall be collected throughout the project as indicated in these Specifications. This data shall be submitted in hard-copy and American Standard Code for Information Interchange (ASCII) format. ASCII format shall include: point number, northing and easting, elevations, and descriptions of point. The ASCII format shall be as follows:

1. PPPP,NNNNNN.NNN,EEEEEE.EEE,ELEV.XXX,Description

a. Where:

- P – point number
- N- Northing
- E – Easting
- ELEV.XXX – Elevation
- Description – description of the point

3.10 PROTECTION OF WORK

A. The Contractor shall use all means necessary to protect completed Work of this Section.

B. At the end of each day, the Contractor shall verify that the entire work area is left in a state that promotes drainage of surface water away from the area and from finished Work. If threatening weather conditions are forecast, soil surfaces shall be seal-rolled at a minimum to protect finished Work.

C. In the event of damage to Work, the Contractor shall make repairs and replacements to the satisfaction of the Construction Manager, at the expense of the Contractor.

PART 4 — MEASUREMENT AND PAYMENT

4.01 GENERAL

A. All earthwork quantities shall be independently verified by the Engineer prior to approval. The independent verification by the Engineer shall utilize the same basic procedures as those used by the Contractor.

B. Any interim or soon-to-be buried (or otherwise obstructed) earthwork shall be surveyed and quantified as the project progresses to enable timely verification by the engineer.

C. Providing for and complying with the requirements set forth in this Section for Soil Excavation will be measured as in-place cubic yards (CY), prior to the excavation, and payment will be based on the unit price provided on the Bid Schedule.

D. Providing for and complying with the requirements set forth in this Section for Rock Excavation will be measured as in-place cubic yards (CY), prior to the excavation, and payment will be based on the unit price provided on the Bid Schedule.

E. Providing for and complying with the requirements set forth in this Section for Fill will be measured as compacted and moisture conditioned cubic yards (CY), and payment will be based on the unit price provided on the Bid Schedule.

F. The following are considered incidental to the work:

- Submittals.
- Quality Control.
- Material samples, sampling, and testing.
- Excavation.
- Blasting, ripping, and hammering.
- Loading, and hauling.
- Scarification.
- Screening.
- Layout survey.
- Rejected material removal, retesting, handling, and repair.
- Temporary haul roads.
- Erosion control.
- Dust control.
- Spill cleanup.
- Placement, compaction, and moisture conditioning.
- Stockpiling.
- Record survey.

[END OF SECTION]

SECTION 02220
SUBGRADE PREPARATION

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary to perform all Subgrade Preparation. The Work shall be carried out as specified herein and in accordance with the Drawings and the Construction Quality Assurance (CQA) Plan.

B. The Work shall include, but not be limited to placement, moisture conditioning, compaction, and grading of subgrade soil and construction of geosynthetics anchor trench. Earthwork shall conform to the dimensions, lines, grades, and sections shown on the Drawings or as directed by the Engineer.

1.02 RELATED SECTIONS

Section 02200 – Earthwork

Section 02772 – Geosynthetic Clay Liner

1.03 REFERENCES

A. Drawings

B. Site CQA Plan

C. Latest version of American Society for Testing and Materials (ASTM) standards:

ASTM D 422	Standard Method for Particle-Size Analysis of Soils
ASTM D 1557	Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lb_f/ft^3 (2,700 kN-m/m^3))
ASTM D 6938	Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)

1.04 QUALITY ASSURANCE

A. The Contractor shall ensure that the materials and methods used for subgrade preparation meet the requirements of the Drawings and this Section. Any material or method that does not conform to these documents, or to alternatives approved in writing by the Design Engineer will be rejected and shall be repaired, or removed and replaced, by the Contractor at no additional expense to the Owner.

PART 2 – PRODUCTS

2.01 SUBGRADE SOIL

A. Subgrade surface be free of protrusions larger than 0.5 inches. Any such observed particles shall be removed prior to placement of geosynthetics.

B. Subgrade surface shall be free of large desiccation cracks (ie, larger than ¼ inch) at the time of geosynthetics placement.

C. Subgrade soil shall consist of on-site soils that are free of particles greater than 3 inches in longest dimension, deleterious, organic, and/or other soil impacts that can damage the overlying liner system.

D. The subgrade surface shall be firm and unyielding, with no abrupt elevation changes, ice, or standing water.

E. The subgrade surface shall be smooth and free of vegetation, sharp-edged rock, stones, sticks, construction debris, and other foreign matter that could contact the GCL.

F. At a minimum, the subgrade surface shall be rolled with a smooth-drum compactor of sufficient weight to remove any excessive wheel ruts greater than 1-inch or other abrupt grade changes.

2.02 ANCHOR TRENCH BACKFILL

A. Anchor trench backfill is the soil material that is placed in the anchor trench, as shown on the Drawings.

B. Where rocks are included in the anchor trench backfill, they shall be mixed with suitable excavated materials to eliminate voids.

C. Material removed during trench excavation may be utilized for anchor trench backfill, provided that all organic material, rubbish, debris, and other objectionable materials are first removed.

2.03 EQUIPMENT

A. The Contractor shall furnish, operate, and maintain grading and compaction equipment as is necessary to produce smooth surfaces for the placement of geosynthetics and acceptable in-place soil density in the anchor trenches.

B. The Contractor shall furnish, operate, and maintain tank trucks, pressure distributors, or other equipment designed to apply water uniformly and in controlled quantities for dust control and for moisture conditioning soils to be placed as trench backfill.

C. The Contractor shall be responsible for cleaning up all fuel, oil, or other spills, at the expense of the Contractor, and to the satisfaction of the Engineer.

PART 3 – EXECUTION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work in this Section, the Contractor shall become thoroughly familiar with the Site, the Site conditions, and all portions of the work falling within this and other related Sections.

B. The Contractor shall provide for the protection of work installed in accordance with other Sections. In the event of damage to other work, the Contractor shall make repairs and replacements to the satisfaction of the Engineer, at the expense of the Contractor.

3.02 SUBGRADE SOIL

A. The Contractor shall remove vegetation and roots to a minimum depth of 4-inches below ground surface in all areas where geosynthetic materials are to be installed.

B. Contractor shall grade subgrade soil to be uniform in slope, free from ruts, mounds, or depressions.

C. Prior to GCL installation, the subgrade surface shall be proof-rolled with appropriate compaction equipment to confirm subgrade stability.

D. In the case additional soil is imported on the site for subgrade use, it shall be placed in loose lifts of no more than 12 inches and compacted to 90% of the maximum density at a moisture content of between -3% and +3% of optimum moisture content, as determined by ASTM D 1557.

3.03 TRENCH EXCAVATION

A. The Contractor shall excavate the anchor trench to the limits and grades shown on the Drawings.

B. Excavated anchor trench materials shall be returned as backfill for the anchor trench and compacted.

C. Excavated materials not suitable for anchor trench backfill shall be stockpiled in an area as shown on the Drawings in accordance with Subpart 3.05 of this Section, or as designated by the Owner.

D. Material not suitable for anchor trench backfill shall be relocated as directed by the Owner.

3.04 TRENCH BACKFILL

A. The anchor trench backfill shall be placed to the lines and grades shown on the Drawings.

B. Soil used for anchor trench backfill shall meet the requirements of Subpart 2.02 of this Section.

C. Soil used for anchor trench backfill shall be placed in loose lifts of no more than 12 inches and compacted to 90% of maximum dry density per ASTM D 1557. Backfill shall be within -3% to +3% of optimum moisture content. The maximum permissible pre-compaction soil clod size is 6 inches.

D. The Contractor shall compact each lift of anchor trench backfill to the satisfaction of the CQA Engineer.

E. The Contractor shall utilize compaction equipment suitable and sufficient for achieving the soil compaction requirements.

F. During soil wetting or drying, the material shall be regularly disked or otherwise mixed so that uniform moisture conditions are obtained in the appropriate range.

3.05 STOCKPILING

A. Soil and rock materials suitable for earthworks that are required to be stockpiled shall be stockpiled in areas as shown on the Drawings or as designated by the Engineer, and shall be free of incompatible soil, clearing debris, vegetation, trash, large rocks, or other objectionable materials.

B. Stockpiles shall be no steeper than 2H:1V (Horizontal:Vertical) or other slope approved by the Engineer, graded to drain, sealed by tracking parallel to the direction of the slope with a dozer or other means approved by the Engineer, and dressed daily during periods when fill is taken from the stockpile. The Contractor shall employ temporary erosion and sediment control measures (i.e. silt fence) as directed by the Engineer around all temporary stockpile areas.

C. There are no compaction requirements for stockpiled materials.

3.06 SURVEY CONTROL

A. The Contractor shall perform all surveys necessary for construction layout and control.

B. The Contractor shall perform as-built surveys for all completed surfaces for purposes of Record Drawing preparation. At a minimum, survey points shall be obtained at grade breaks, top of slope, toe of slope, and limits of material type.

3.07 PROTECTION OF WORK

A. The Contractor shall protect completed work of this Section.

B. At the end of each day, the Contractor shall verify that the entire work area is left in a state that promotes drainage of surface water away from the area and from finished work.

C. In the event of damage to Work, the Contractor shall make repairs and replacements to the satisfaction of the CQA Engineer, at the expense of the Contractor.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements for subgrade preparation will be measured on a square foot (SF) basis and payment will be based on the unit price as provided on the Bid Schedule.

B. Providing for and complying with the requirements for anchor trench excavation and backfill shall be measured on a lineal foot (LF) basis and payment will be based on the unit price as provided on the Bid Schedule.

C. The following are considered incidental to the work:

- Submittals.
- Quality Control.
- Material samples.
- Screening.
- Excavation, loading, and hauling.
- Temporary haul roads.
- Layout survey.
- Rejected material removal, testing, hauling, and repair.
- Erosion Control
- Dust control.
- Spill Clean-up
- Placement, compaction, and moisture conditioning.
- Stockpiling.
- Record survey.

[END OF SECTION]

SECTION 02225
DRAINAGE AGGREGATE

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary for the installation of Drainage Aggregate. The work shall be carried out as specified herein and in accordance with the Drawings and the site Construction Quality Assurance (CQA) Plan.

B. The work shall include, but not be limited to, delivery, offloading, storage, and placement of Drainage Aggregate (aggregate).

1.02 RELATED SECTIONS

Section 02616 – PVC Pipe

Section 02770 – Geomembrane

Section 02771 – Geotextile

Section 02773 – Geonet

1.03 REFERENCES

A. Drawings

B. Site Construction Quality Assurance (CQA) Plan

C. Latest Version of American Society for Testing and Materials (ASTM) Standards:

ASTM C 33	Standard Specification for Concrete Aggregates
ASTM C 136	Test Method for Sieve Analysis of Fine and Coarse Aggregates
ASTM D 2434	Test Method for Permeability of Granular Soils (Constant Head)
ASTM D 3042	Standard Test Method for Insoluble Residue in Carbonate Aggregates

1.04 SUBMITTALS

A. The Contractor shall submit to the Engineer for approval, at least 7 days prior to the start of construction, Certificates of Compliance for proposed aggregate materials. Certificates of Compliance shall include, at a minimum, typical gradation, insoluable residue content, representative sample, and source of aggregate materials.

B. The Contractor shall submit to the Engineer a list of equipment and technical information for equipment proposed for use in placing the aggregate material in accordance with this Section.

1.05 CONSTRUCTION QUALITY ASSURANCE (CQA) MONITORING

A. The Contractor shall be aware of and accommodate all monitoring and field/laboratory conformance testing required by the CQA Plan. This monitoring and testing, including random conformance testing of construction materials and completed work, will be performed by the CQA Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Contractor will be required to repair the deficiency or replace the deficient materials.

PART 2 – PRODUCTS

2.01 MATERIALS

A. Aggregate shall meet the requirements specified in ASTM C 33 and shall not contain limestone. Aggregate shall have a minimum permeability of 1×10^{-1} cm/sec when tested in accordance with ASTM D 2434. The requirements of the Aggregate are presented below:

Maximum Particle Size	Percent Finer
¾-inch	100
No. 200 Sieve	0 to 2

B. Carbonate loss shall be no greater than 10 percent by dry weight basis when tested in accordance with ASTM D 3042.

2.02 EQUIPMENT

A. The Contractor shall furnish, operate, and maintain hauling, placing, and grading equipment as necessary for aggregate placement.

PART 3 – EXECUTION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work in this Section, the Contractor shall become thoroughly familiar with the site, the site conditions, and all portions of the work falling within this and other related Sections.

B. Inspection:

1. The Contractor shall carefully inspect the installed work of all other Sections and verify that all work is complete to the point where the installation of the work specified in this Section may properly commence without adverse impact.

2. If the Contractor has any concerns regarding the installed work of other Sections, the Engineer shall be notified in writing prior to commencing work. Failure to notify the Engineer or commencement of the work of this Section will be construed as Contractor's acceptance of the related work of all other Sections.

3.02 PLACEMENT

A. Place after underlying geosynthetic installation is complete, including construction quality control (CQC) and CQA work.

B. Place to the lines, grades, and dimensions shown on the Drawings.

C. The subgrade of the aggregate consists of a geotextile overlying a geomembrane. The Contractor shall avoid creating large wrinkles (greater than 6-inches high), tearing, puncturing, folding, or damaging in any way the geosynthetic materials during placement of the aggregate material.

D. Damage to the geosynthetic liner system caused by the Contractor or his representatives shall be repaired by the Geosynthetic Installer, at the expense of the Contractor.

E. No density or moisture requirements are specified for placement of the aggregate material.

3.03 FIELD TESTING

A. The minimum frequency and details of conformance testing are provided below. This testing will be performed by the CQA Engineer. The Contractor shall take this testing frequency into account in planning the construction schedule.

1. Aggregates conformance testing:

a. particle-size analyses conducted in accordance with ASTM C 136 at a frequency of one test per 5,000 yd^3, minimum one per project; and

b. permeability tests conducted in accordance with ASTM D 2434 at a frequency of one test per 10,000 yd^3, minimum one per project.

3.04 SURVEY CONTROL

A. The Contractor shall perform all surveys necessary for construction layout, control, and Record Drawings.

3.05 PROTECTION OF WORK

A. The Contractor shall use all means necessary to protect all work of this Section.

B. In the event of damage, the Contractor shall make repairs and replacements to the satisfaction of the Engineer at no additional cost to the Owner.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for Drainage Aggregate will be incidental to the PVC pipe, and payment will be based on the unit price for PVC pipe provided on the Bid Schedule.

B. The following are considered incidental to the work:

- Submittals.
- Quality Control.
- Material samples, sampling, and testing.
- Excavation, loading, and hauling.
- Placing and grading.
- Layout survey.
- Rejected material.
- Rejected material removal, re-testing, handling, and repair.
- Mobilization.

[END OF SECTION]

SECTION 02616
POLYVINYL CHLORIDE (PVC) PIPE

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, and equipment necessary to install perforated and solid wall polyvinyl chloride (PVC) Schedule 40 pipe and fittings, as shown on the Drawings and in accordance with the Construction Quality Assurance (CQA) Plan.

1.02 RELATED SECTIONS

Section 02225 – Drainage Aggregate

Section 02270 – Geomembrane

Section 02771 – Geotextile

Section 02772 – Geonet

1.03 REFERENCES

A. Drawings.

B. Site CQA Plan.

C. Latest version of the American Society for Testing and Materials (ASTM) standards:

ASTM D 1784 Standard Specification for Rigid Poly (Vinyl Chloride) (PVC) Compounds and chlorinated Poly (Vinyl Chloride) (CPVC) Compounds.

ASTM D 1785 Poly (Vinyl Chloride) (PVC) Plastic Pipe, Schedules 40, 80 and 120.

ASTM D 2466 Standard Specification for Poly (Vinyl Chloride) (PVC) Plastic Pipe Fittings, Schedule 40.

ASTM D 2564 Standard Specification for Solvent Cements for Poly (Vinyl Chloride) (PVC) Plastic Pipe and Fittings.

ASTM D 2774 Practice for Underground Installation of Thermoplastic Pressure Piping.

ASTM D 2855 Standard Practice for Making Solvent-Cemented Joints with Poly (Vinyl Chloride) (PVC) Pipe and Fittings.

ASTM F 656 Standard Specification for Primers for Use in Solvent Cement Joints of Poly (Vinyl Chloride) (PVC) Plastic Pipe and Fittings.

1.04 SUBMITTALS

A. The Contractor shall submit to the Engineer for approval, at least 7 days prior to installation of this material, Certificates of Compliance for the pipe and fittings to be furnished. Certificates of Compliance shall consist of a properties sheet, including specified properties measured using test methods indicated herein.

B. The Contractor shall submit to the Engineer, Record Drawings of the installed piping at a frequency of not less than once per every 50 feet of installed pipe and strip composite. Record Drawings shall be submitted within 7 days of completion of the record survey.

1.05 CQA MONITORING

A. The Contractor shall ensure that the materials and methods used for PVC pipe and fittings installation meet the requirements of the Drawings and this Section. Any material or method that does not conform to these documents, or to alternatives approved in writing by the Engineer, will be rejected and shall be repaired or replaced by the Contractor at no additional cost to the Owner.

PART 2 – MATERIALS

2.01 PVC PIPE & FITTINGS

A. PVC pipe and fittings shall be manufactured from a PVC compound which meets the requirements of Cell Classification 12454 polyvinyl chloride as outlined in ASTM D 1784.

B. PVC pipe shall meet the requirements of ASTM D 1784 and ASTM D 1785 for Schedule 40 PVC pipe.

C. PVC fittings shall meet the requirements of ASTM D 2466.

D. Clean rework or recycle material generated by the Manufacturer's own production may be used so long as the pipe or fittings produced meet all the requirements of this Section.

E. Pipe and fittings shall be homogenous throughout and free of visible cracks, holes, foreign inclusions, or other injurious defects, being uniform in color, capacity, density, and other physical properties.

F. PVC pipe and fitting primer shall meet the requirements of ASTM F 656 and solvent cements shall meet the requirements of ASTM D 2564.

2.02 PVC PERFORATED PIPE

A. Perforated pipe shall meet the requirements listed above for solid wall pipe, unless otherwise approved by the Engineer. PVC pipe perforations shall be as shown on the Drawings.

2.03 STRIP COMPOSITE

A. Strip composite shall be comprised of high density polyethylene core Multi-Flow Drainage Systems 12-inch product, or Engineer approved equal. Consideration for equality will involve chemical resistance, compressive strength, and flow capacity. Strip composite shall be installed as shown on the Drawings.

B. Sand bags used to continuously cover the strip composite shall be comprised of woven geotextile capable of allowing liquids to pass and shall have a minimum length of 18-inches.

C. Sand bags shall contain Utah Department of Transportation (UDOT) concrete sand having a carbonate loss of no greater than 10 percent by dry weight basis when tested in accordance with ASTM D 3042 and meeting the following gradation.

Sieve Size	Percent Passing
3/8 inch	100%
No. 4	95% to 100%
No. 16	45% to 80%
No. 50	10% to 30%
No. 100	2% to 10%

D. Contractor shall monitor that sand bags shall not be overfilled to the extent that the underlying strip composite is visible.

PART 3 – PART 3 EXECUTION

3.01 PVC PIPE HANDLING

A. When shipping, delivering, and installing pipe, fittings, and accessories, do so to ensure a sound, undamaged installation. Provide adequate storage for all materials and equipment delivered to the site. PVC pipe and pipe fittings shall be handled carefully in loading and unloading so as not to damage the pipe, fittings, or underlying materials.

3.02 PVC PIPE INSTALLATION

A. PVC pipe installation shall conform to these Specifications, the Manufacturer’s recommendations, and as outlined in ASTM D 2774.

B. PVC perforated and solid wall pipe shall be installed as shown on the Drawings.

C. PVC pipe shall be inspected for cuts, scratches, or other damages prior to installation. Any pipe showing damage, which in the opinion of the CQA Engineer will affect performance of the pipe, must be removed from the site. Contractor shall replace any material found to be defective at no additional cost to the Owner.

3.03 JOINING OF PVC PIPES

A. PVC pipe and fittings shall be joined by primer and solvent-cements per ASTM D 2855.

B. All loose dirt and moisture shall be wiped from the interior and exterior of the pipe end and the interior of fittings.

C. All pipe cuts shall be square and perpendicular to the centerline of the pipe. All burrs, chips, etc., from pipe cutting shall be removed from pipe interior and exterior.

D. Pipe and fittings shall be selected so that there will be as small a deviation as possible at the joints, and so inverts present a smooth surface. Pipe and fittings that do not fit together to form a tight fit will be rejected.

3.04 PROTECTION OF WORK

A. The Contractor shall use all means necessary to protect all work of this Section.

B. In the event of damage, the Contractor shall make all repairs and replacements necessary, to the satisfaction of the Engineer.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for 4-inch PVC Pipe will be measured as in-place linear foot (LF) to the limits shown on the Drawings, and payment will be based on the unit price provided on the Bid Schedule.

B. Providing for and complying with the requirements set forth in this Section for 18-inch PVC Pipe will be measured as in-place LF to the limits shown on the Drawings, and payment will be based on the unit price provided on the Bid Schedule.

C. Providing for and complying with the requirements set forth in this Section for Strip Drain, including connectors and sand bags, will be measured as in-place LF to the limits shown on the Drawings, and payment will be based on the unit price provided on the Bid Schedule.

D. The following are considered incidental to the Work:

- Submittals.
- Quality Control.
- Shipping, handling and storage.
- Fittings.
- Drainage aggregate.
- Joining.
- Mobilization.
- Placement.
- Rejected material.
- Rejected material removal, handling, re-testing, and repair.
- Gravel and sand bags.

[END OF SECTION]

SECTION 02770
GEOMEMBRANE

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary for the installation of smooth and textured high-density polyethylene (HDPE) geomembrane, as shown on the Drawings. The work shall be performed as specified herein and in accordance with the Drawings and the site Construction Quality Assurance (CQA) Plan.

B. The work shall include, but not be limited to, delivery, offloading, storage, placement, anchorage, and seaming of the geomembrane.

1.02 RELATED SECTIONS

Section 02225 – Drainage Aggregate

Section 02771 – Geotextile

Section 02773 – Geonet

1.03 REFERENCES

A. Drawings

B. Site CQA Plan

C. Latest version of the American Society for Testing and Materials (ASTM) standards:

ASTM D 638	Standard Test Method for Tensile Properties of Plastics
ASTM D 792	Standard Test Methods for Specific Gravity (Relative Density) and Density of Plastics by Displacement
ASTM D 1505	Standard Test Methods for Density of Plastics by Density-Gradient Technique
ASTM D 1603	Standard Test Method for Carbon Black in Olefin Plastics
ASTM D 4439	Terminology for Geosynthetics
ASTM D 4833	Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products
ASTM D 5199	Standard Test Method for Measuring the Nominal Thickness of Geosynthetics
ASTM D 5397	Test Method for Evaluation of Stress Crack Resistance of Polyolefin Geomembranes Using Notched Constant Tensile Load Test
ASTM D 5596	Recommended Practice for Microscopical Examination of Pigment Dispersion in Plastic Compounds
ASTM D 5641	Practice for Geomembrane Seam Evaluation by Vacuum Chamber
ASTM D 5820	Practice for Pressurized Air Channel Evaluation of Dual Seamed Geomembranes

ASTM D 6365 Standard Test Method for the Non-destructive Testing of Geomembrane Seams using the Spark Test.

ASTM D 6392 Standard Test Method for Determining the Integrity of Non-reinforced Geomembrane Seams Produced using Thermo-Fusion Methods.

1.04 QUALIFICATIONS

A. Geomembrane Manufacturer:

1. The Geomembrane Manufacturer shall be responsible for the production of geomembrane rolls from resin and shall have sufficient production capacity and qualified personnel to provide material meeting the requirements of this Section and the construction schedule for this project.

2. The Geomembrane Manufacturer shall have successfully manufactured a minimum of 20,000,000 square feet of polyethylene geomembrane.

B. Geosynthetics Installer:

1. The Geosynthetics Installer shall be responsible and shall provide sufficient resources for field handling, deploying, seaming, temporarily restraining (against wind), and other aspects of the deployment and installation of the geomembrane and other geosynthetic components of the project.

2. The Geosynthetics Installer shall have successfully installed a minimum of 20,000,000 square feet of polyethylene geomembrane on previous projects with similar side slopes, bench widths, and configurations.

3. The installation crew shall have the following experience.

a. The Superintendent shall have supervised the installation of a minimum of 10,000,000 square feet of polyethylene geomembrane on at least ten (10) different projects.

b. At least one seamer shall have experience seaming a minimum of 2,000,000 square feet of polyethylene geomembrane using the same type of seaming apparatus to be used at this Site. Seamers with such experience will be designated "master seamers" and shall provide direct supervision over less experienced seamers.

c. All other seaming personnel shall have seamed at least 100,000 square feet of polyethylene geomembrane using the same type of seaming apparatus to be used at this site. Personnel who have seamed less than 100,000 square feet shall be allowed to seam only under the direct supervision of the master seamer or Superintendent.

1.05 WARRANTY

A. The Geosynthetic Manufacturer shall furnish the Owner a 20-year written warranty against defects in materials. Warranty conditions concerning limits of liability will be evaluated by, and must be acceptable to, the Owner.

B. The Geosynthetic Installer shall furnish the Owner with a 1-year written warranty against defects in workmanship. Warranty conditions concerning limits of liability will be evaluated by, and must be acceptable to, the Owner.

1.06 SUBMITTALS

A. The Geosynthetic Installer shall submit the following documentation on the resin used to manufacture the geomembrane to the Engineer for approval 14 days prior to transporting any geomembrane to the Site.

1. Copies of quality control certificates issued by the resin supplier including the production dates, brand name, and origin of the resin used to manufacture the geomembrane for the project.

2. Results of tests conducted by the Geomembrane Manufacturer to verify the quality of the resin used to manufacture the geomembrane rolls assigned to the project.

3. Certification that no reclaimed polymer is added to the resin during the manufacturing of the geomembrane to be used for this project, or, if recycled polymer is used, the Manufacturer shall submit a certificate signed by the production manager documenting the quantity of recycled material, including a description of the procedure used to measure the quantity of recycled polymer.

B. The Geosynthetic Installer shall submit the following documentation on geomembrane roll production to the Engineer for approval 14 days prior to transporting any geomembrane to the site.

1. Quality control certificates, which shall include:

   a. roll numbers and identification; and

   b. results of quality control tests, including descriptions of the test methods used, outlined in Subpart 2.02 of this Section.

2. The manufacturer warranty specified in Subpart 1.05 of this Section.

C. The Geosynthetic Installer shall submit the following information to the Engineer for approval 14 days prior to mobilization.

1. A Panel Layout Drawing showing the installation layout and identifying geomembrane panel configurations, dimensions, details, locations of seams, as well as any variance or additional details that deviate from the Drawings. The Panel Layout Drawing shall be adequate for use as a construction plan and shall include dimensions, details, etc. The Panel Layout Drawing, as modified and/or approved by the Engineer, shall become Subpart of these Technical Specifications.

2. Installation schedule.

3. Copy of Geosynthetic Installer's letter of approval or license by the Geomembrane Manufacturer.

4. Installation capabilities, including:

   a. information on equipment proposed for this project;

   b. average daily production anticipated for this project; and

   c. quality control procedures.

5. A list of completed facilities for which the Geosynthetic Installer has installed a minimum of 20,000,000 square feet of polyethylene geomembrane, in accordance with Subpart 1.04 of this Section. The following information shall be submitted to the Engineer for each facility:

a. the name and purpose of the facility, its location, and dates of installation;

b. the names of the owner, Engineer, and geomembrane manufacturer;

c. name of the supervisor of the installation crew; and

d. thickness and surface area of installed geomembrane.

6. In accordance with Subpart 1.04 of this Section, a resume of the Superintendent to be assigned to this project, including dates and duration of employment, shall be submitted at least 7 days prior to beginning geomembrane installation.

7. In accordance with Subpart 1.04 of this Section, resumes of all personnel who will perform seaming operations on this project, including dates and duration of employment, shall be submitted at least 7 days prior to beginning geomembrane installation.

D. A Certificate of Calibration less than 12 months old shall be submitted for each field tensiometer prior to installation of any geomembrane.

E. During installation, the Geosynthetic Installer shall be responsible for the timely submission to the Engineer of:

1. Quality control documentation; and

2. Subgrade Acceptance Certificates, signed by the Geosynthetic Installer, for each area to be covered by geosynthetic materials.

F. Upon completion of the installation, the Geosynthetic Installer shall be responsible for the submission to the Engineer of a warranty from the Geosynthetic Installer as specified in Subpart 1.05.B of this Section.

G. Upon completion of the installation, the Geosynthetic Installer shall be responsible for the submission to the Engineer of a Record Drawing showing the location and number of each panel and locations and numbers of destructive tests and repairs.

H. The Geosynthetic Installer shall submit samples and material property cut-sheets on the proposed geomembrane to the Engineer at least 7 days prior to delivery of this material to the site.

I. The Geosynthetic Installer shall submit the following documentation on welding rod to the Engineer for approval 14 days prior to transporting welding rod to the Site:

1. Quality control documentation, including lot number, welding rod spool number, and results of quality control tests on the welding rod.

2. Certification that the welding rod is compatible with the geomembrane and this Section.

1.07 CONSTRUCTION QUALITY ASSURANCE (CQA) MONITORING

A. The Geosynthetic Installer shall be aware of and accommodate all monitoring and conformance testing required by the CQA Plan. This monitoring and testing, including random conformance testing of construction materials and completed work, will be performed by the CQA Engineer. If nonconformances or other deficiencies are found in the Geosynthetic Installer's materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 – PRODUCTS

2.01 GEOMEMBRANE PROPERTIES

A. The Geomembrane Manufacturer shall furnish white-or off-white-surfaced (upper side only), smooth and textured geomembrane having properties that comply with the required property values shown in Table 02770-1.

B. In addition to the property values listed in Table 02770-1, the geomembrane shall:

1. Contain a maximum of 1 percent by weight of additives, fillers, or extenders (not including carbon black and titanium dioxide).

2. Not have striations, pinholes (holes), bubbles, blisters, nodules, undispersed raw materials, or any sign of contamination by foreign matter on the surface or in the interior.

2.02 MANUFACTURING QUALITY CONTROL (MQC)

A. Rolls:

1. The Geomembrane Manufacturer shall continuously monitor geomembrane during the manufacturing process for defects.

2. No geomembrane shall be accepted that exhibits any defects.

3. The Geomembrane Manufacturer shall measure and report the geomembrane thickness at regular intervals along the roll length.

4. No geomembrane shall be accepted that fails to meet the specified thickness.

5. The Geomembrane Manufacturer shall sample and test the geomembrane at a minimum of once every 50,000 square feet to demonstrate that its properties conform to the values specified in Table 02770-1. At a minimum, the following tests shall be performed:

Test	Procedure
Thickness	ASTM D 5199
Specific Gravity	ASTM D 792 Method A or ASTM D 1505
Tensile Properties	ASTM D 638
Puncture Resistance	ASTM D 4833
Carbon Black	ASTM D 1603
Carbon Black Dispersion	ASTM D 5596

6. Tests not listed above but listed in Table 02770-1 need not be run at the one per 50,000 square feet frequency. However, the Geomembrane Manufacturer shall certify that these tests are in compliance with this Section and have been performed on a sample that is identical to the geomembrane to be used on this project. The Geosynthetic Installer shall provide the test result documentation to the Engineer.

7. Any geomembrane sample that does not comply with the requirements of this Section will result in rejection of the roll from which the sample was obtained and will not be used for this project.

8. If a geomembrane sample fails to meet the quality control requirements of this Section, the Geomembrane Manufacturer shall sample and test, at the expense of the Manufacturer, rolls manufactured in the same resin batch, or at the same time, as the failing roll. Sampling and testing of rolls shall continue until a pattern of acceptable test results is established to bound the failed roll(s).

9. Additional testing may be performed at the Geomembrane Manufacturer's discretion and expense, to isolate and more closely identify the non-complying rolls and/or to qualify individual rolls.

B. The Geomembrane Manufacturer shall permit the Engineer to visit the manufacturing plant for project specific visits. If possible, such visits will be prior to or during the manufacturing of the geomembrane rolls for the specific project. The Engineer may elect to collect conformance samples at the manufacturing facility to expedite the acceptance of the materials.

2.03 LABELING

A. Geomembrane rolls shall be labeled with the following information.

1. thickness of the material;
2. length and width of the roll;
3. name of Geomembrane Manufacturer;
4. product identification;
5. lot number; and
6. roll number.

2.04 TRANSPORTATION, HANDLING, AND STORAGE

A. The Geosynthetic Manufacturer shall be liable for any damage to the geomembrane incurred prior to and during transportation to the site.

B. Handling and care of the geomembrane at the site prior to and following installation shall be the responsibility of the Geosynthetic Installer. The Geosynthetic Installer shall be liable for all damage to the materials incurred prior to final acceptance of the liner system by the Owner.

C. Geosynthetic Installer shall be responsible for storage of the geomembrane at the site. The geomembrane shall be protected from excessive heat or cold, dirt, puncture, cutting, or other damaging or deleterious conditions. Any additional storage procedures required by the Geomembrane Manufacturer shall be the Geosynthetic Installer's responsibility. Geomembrane rolls shall not be stored or placed in a stack of more than two rolls high.

D. The geomembrane shall be delivered at least 14 days prior to the planned deployment date to allow the CQA Engineer adequate time to perform conformance testing on the geomembrane samples as described in Subpart 3.05 of this Section. If the CQA Engineer performed a visit to the manufacturing plant and performed the required conformance sampling, geomembrane can be delivered to the site within the 14 days prior to the planned deployment date as long as there is sufficient time for the CQA Engineer to complete the conformance testing and confirm that the rolls shipped to the site are in compliance with this Section.

PART 3 – GEOMEMBRANE INSTALLATION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work described in this Section, the Geosynthetic Installer shall become thoroughly familiar with all portions of the work falling within this Section.

B. Inspection:

1. The Geosynthetic Installer shall carefully inspect the installed work of all other Sections and verify that all work is complete to the point where the work of this Section may properly commence without adverse effect.

2. If the Geosynthetic Installer has any concerns regarding the installed work of other Sections, he shall notify the Engineer in writing prior to the start of the work of this Section. Failure to inform the Engineer in writing or commencing installation of the geomembrane will be construed as the Geosynthetic Installer's acceptance of the related work of all other Sections.

C. A pre-installation meeting shall be held to coordinate the installation of the geomembrane with the installation of other components of the liner system.

3.02 GEOMEMBRANE DEPLOYMENT

A. Layout Drawings:

1. The Geosynthetic Installer shall deploy the geomembrane panels in general accordance with the Panel Layout Drawing specified. The Panel Layout Drawing must be approved by the Engineer prior to installation of any geomembrane.

B. Field Panel Identification:

1. A geomembrane field panel is a roll or a portion of roll cut in the field.

2. Each field panel shall be given a unique identification code (number or letter-number). This identification code shall be agreed upon by the Engineer and Geosynthetic Installer.

C. Field Panel Placement:

1. Field panels shall be installed, as approved or modified, at the location and positions indicated on the Panel Layout Drawing.

2. Field panels shall be placed one at a time, and each field panel shall be seamed immediately after its placement.

3. Geomembrane shall not be placed when the ambient temperature is below 32°F or above 122°F, as measured in Subpart 3.03.C.3 in this Section, unless otherwise authorized in writing by the Engineer.

4. Geomembrane shall not be placed during any precipitation, in the presence of excessive moisture (e.g., fog, dew), in an area of ponded water, or in the presence of wind speeds greater than 20 mph.

5. The Geosynthetic Installer shall ensure that:

   a. No vehicular traffic is allowed on the geomembrane with the exception of all terrain vehicles with a contact pressures at or lower than that exhibited by foot traffic.

b. Equipment used does not damage the geomembrane by handling, trafficking, or leakage of hydrocarbons (i.e., fuels).

c. Personnel working on the geomembrane do not smoke, wear damaging shoes, bring glass onto the geomembrane, or engage in other activities that could damage the geomembrane.

d. The method used to unroll the panels does not scratch or crimp the geomembrane and does not damage the supporting soil or geosynthetics.

e. The method used to place the panels minimizes wrinkles (especially differential wrinkles between adjacent panels). The method used to place the panels results in intimate contact between the geomembrane and adjacent components.

f. Temporary ballast and/or anchors (e.g., sand bags) are placed on the geomembrane to prevent wind uplift. Ballast methods must not damage the geomembrane.

g. The geomembrane is especially protected from damage in heavily trafficked areas.

h. Any rub sheets to facilitate seaming are removed prior to installation of subsequent panels.

6. Any field panel or portion thereof that becomes seriously damaged (torn, twisted, or crimped) shall be replaced with new material. Less serious damage to the geomembrane may be repaired, as approved by the Engineer. Damaged panels or portions of damaged panels that have been rejected shall be removed from the work area and not reused.

D. If the Geosynthetic Installer intends to install geomembrane between one hour before sunset and one hour after sunrise, he shall notify the Engineer in writing prior to the start of the work. The Geosynthetic Installer shall indicate additional precautions that shall be taken during these installation hours. The Geosynthetic Installer shall provide proper illumination for work during this time period.

3.03 FIELD SEAMING

A. Seam Layout:

1. In corners and at odd-shaped geometric locations, the number of field seams shall be minimized. No horizontal seam shall be constructed along a slope with an inclination steeper than 10 percent. Horizontal seams shall be considered as any seam having an alignment exceeding 30 degrees from being perpendicular to the slope contour lines, unless otherwise approved by the Engineer. No seams shall be located in an area of potential stress concentration.

2. Seams shall not be allowed within 5 feet of the top or toe of any slope. Horizontal seams can be placed on benches, as long as they are not within 5 feet of the top or toe of slope.

B. Personnel:

1. All personnel performing seaming operations shall be qualified as indicated in Subpart 1.04 of this Section. No seaming shall be performed unless a "master seamer" is present on-site.

C. Weather Conditions for Seaming:

1. Unless authorized in writing by the Engineer, seaming shall not be attempted at ambient temperatures below 32°F or above 122°F. If the Geosynthetic Installer wishes to use methods that may allow seaming at ambient temperatures below 32°F or above 122°F, the procedure must be approved by the Engineer.

2. A meeting will be held between the Geosynthetic Installer and Engineer to establish acceptable installation procedures. In all cases, the geomembrane shall be dry and protected from wind damage during installation.

3. Ambient temperatures, measured by the CQA Engineer, shall be measured between 0 and 6 inches above the geomembrane surface.

D. Overlapping:

1. The geomembrane shall be cut and/or trimmed such that all corners are rounded.

2. Geomembrane panels shall be shingled with the upslope panel placed over the down slope panel.

3. Geomembrane panels shall be sufficiently overlapped for welding and to allow peel tests to be performed on the seam. Any seams that cannot be destructively tested because of insufficient overlap shall be treated as failing seams.

E. Seam Preparation:

1. Prior to seaming, the seam area shall be clean and free of moisture, dust, dirt, debris of any kind, and foreign material.

2. If seam overlap grinding is required, the process shall be completed according to the Geomembrane Manufacturer's instructions within 20 minutes of the seaming operation and in a manner that does not damage the geomembrane. The grind depth shall not exceed ten percent of the geomembrane thickness.

3. Seams shall be aligned with the fewest possible number of wrinkles and "fishmouths."

F. General Seaming Requirements:

1. Fishmouths or wrinkles at the seam overlaps shall be cut along the ridge of the wrinkle to achieve a flat overlap, ending the cut with circular cut-out. The cut fishmouths or wrinkles shall be seamed and any portion where the overlap is insufficient shall be patched with an oval or round patch of geomembrane that extends a minimum of 6 inches beyond the cut in all directions.

2. Any electric generator shall be placed outside the area to be lined or mounted in a manner that protects the geomembrane from damage due to the weight and frame of the generator or due to fuel leakage. The electric generator shall be properly grounded.

G. Seaming Process:

1. Approved processes for field seaming are extrusion welding and double-track hot-wedge fusion welding. Only equipment identified as part of the approved submittal specified in Subpart 1.06 of this Section shall be used.

2. Extrusion Equipment and Procedures:

   a. The Geosynthetics Installer shall maintain at least one spare operable seaming apparatus on site.

b. Extrusion welding apparatuses shall be equipped with gauges giving the temperatures in the apparatuses.

c. Prior to beginning an extrusion seam, the extruder shall be purged until all heat-degraded extrudate has been removed from the barrel.

d. A smooth insulating plate or fabric shall be placed beneath the hot welding apparatus after use.

3. Fusion Equipment and Procedures:

a. The Geosynthetic Installer shall maintain at least one spare operable seaming apparatus on site.

b. Fusion-welding apparatus shall be automated vehicular-mounted devices equipped with gauges giving the applicable temperatures and speed.

c. A smooth insulating plate or fabric shall be placed beneath the hot welding apparatus after use.

H. Trial Seams:

1. Trial seams shall be made on fragment pieces of geomembrane to verify that seaming conditions are adequate. Trial seams shall be conducted on the same material to be installed and under similar field conditions as production seams. Such trial seams shall be made at the beginning of each seaming period, typically at the beginning of the day and after lunch, for each seaming apparatus used each day, but no less frequently than once every 5 hours. The trial seam sample shall be a minimum of 5 feet long by 1 foot wide (after seaming) with the seam centered lengthwise for fusion equipment and at least 3 feet long by 1 foot wide for extrusion equipment. Seam overlap shall be as indicated in Subpart 3.03.D of this Section.

2. Four coupon specimens, each 1-inch wide, shall be cut from the trial seam sample by the Geosynthetics Installer using a die cutter to ensure precise 1-inch wide coupons. The coupons shall be tested, by the Geosynthetic Installer, with the CQA Monitor present, in peel (both the outside and inside track) and in shear using an electronic readout field tensiometer in accordance with ASTM D 6392, at a strain rate of 2 inches/minute. The samples shall not exhibit failure in the seam, i.e., they shall exhibit a Film Tear Bond (FTB), which is a failure (yield) in the parent material. The required peel and shear seam strength values are listed in Table 02770-2. At no time shall specimens be soaked in water.

3. If any coupon specimen fails, the trial seam shall be considered failing and the entire operation shall be repeated. If any of the additional coupon specimens fail, the seaming apparatus and seamer shall not be accepted and shall not be used for seaming until the deficiencies are corrected and two consecutive successful trial seams are achieved.

I. Nondestructive Seam Continuity Testing:

1. The Geosynthetic Installer shall nondestructively test for continuity on all field seams over their full length. Continuity testing shall be carried out as the seaming work progresses, not at the completion of all field seaming. The Geosynthetic Installer shall complete any required repairs in accordance with Subpart 3.03.K of this Section. The following procedures shall apply:

a. Vacuum testing in accordance with ASTM D 5641.

b. Air channel pressure testing for double-track fusion seams in accordance with ASTM D 5820 and the following:

i. Insert needle, or other approved pressure feed device, from pressure gauge and inflation device into the air channel at one end of a double track seam.

ii. Energize the air pump and inflate air channel to a pressure between 25 and 30 pounds per square inch (psi). Close valve and sustain the pressure for not less than 5 minutes.

iii. If loss of pressure exceeds 3 psi over 5 minutes, or if the pressure does not stabilize, locate the faulty area(s) and repair seam in accordance with Subpart 3.03.K of this Section.

iv. After 5 minutes, cut the end of air channel opposite from the end with the pressure gauge and observe release of pressure to ensure air channel is not blocked. If the channel does not depressurize, find and repair the portion of the seam containing the blockage per Subpart 3.03.K of this Section. Repeat the air pressure test on the resulting segments of the original seam created by the repair and the ends of the seam.. Repeat the process until the entire length of seam has successfully passed pressure testing or contains a repair. Repairs shall also be non-destructively tested per Subpart 3.03.K.5 of this Section.

v. Remove needle, or other approved pressure feed device, and seal repair in accordance with Subpart 3.03.K of this Section.

c. Spark test seam integrity verification shall be performed in accordance with ASTM D 6365 if the seam cannot be tested using other nondestructive methods.

J. Destructive Testing:

1. Destructive seam tests shall be performed on samples collected from selected locations to evaluate seam strength and integrity. Destructive tests shall be carried out as the seaming work progresses, not at the completion of all field seaming.

2. Sampling:

a. Destructive test samples shall be collected at a minimum average frequency of one test location per 500 feet of total seam length. If after a total of 50 samples have been tested and no more than 1 sample has failed, the frequency can be increased to one per 1,000 feet. Test locations shall be determined during seaming, and may be prompted by suspicion of excess crystallinity, contamination, offset seams, or any other potential cause of imperfect seaming. The CQA Engineer will be responsible for choosing the locations. The Geosynthetic Installer shall not be informed in advance of the locations where the seam samples will be taken. The CQA Engineer reserves the right to increase the sampling frequency if observations suggest an increased frequency is warranted.

b. The CQA Engineer shall mark the destructive sample locations. Samples shall be cut by the Geosynthetic Installer at the locations designated by the CQA Engineer as the seaming progresses in order to obtain laboratory test results before the geomembrane is covered by another material. Each sample shall be numbered and the sample number and location identified on the Panel Layout Drawing. All holes in the geomembrane resulting from the destructive seam sampling shall be immediately repaired in accordance with the repair procedures described in Subpart 3.03.K of this Section. The continuity of the new seams associated with the repaired areas shall be tested according to Subpart 3.03.I of this Section.

c. Two coupon strips of dimensions 1-inch wide and 12-inches long with the seam centered parallel to the width shall be taken from any side of the sample location. These samples shall be tested in the field in accordance with Subpart 3.03.J.3 of this Section. If these samples pass the field test, a laboratory sample shall be taken. The laboratory sample shall be at least 1-foot wide by 3.5-feet long with the seam centered along the length. The sample shall be cut into three parts and distributed as follows:

   i. One portion 12-inches long to the Geosynthetic Installer.

   ii. One portion 18-inches long to the Geosynthetic CQA Laboratory for testing.

   iii. One portion 12-inches long to the Owner for archival storage.

3. Field Testing:

   a. The two 1-inch wide strips shall be tested in the field tensiometer in the peel mode on both sides of the double track fusion welded sample. The CQA Engineer has the option to request an additional test in the shear mode. If any field test sample fails to meet the requirements in Table 02770-2, then the procedures outlined in Subpart 3.03.J.5 of this Section for a failing destructive sample shall be followed.

4. Laboratory Testing:

   a. Testing by the Geosynthetics CQA Laboratory will include "Seam Strength" and "Peel Adhesion" (ASTM D 6392) with 1-inch wide strips tested at a rate of 2 inches/minute. At least 5 specimens will be tested for each test method (peel and shear). Four of the five specimens per sample must pass both the shear strength test and peel adhesion test when tested in accordance with ASTM D 6392. The minimum acceptable values to be obtained in these tests are indicated in Table 02770-2. Both the inside and outside tracks of the dual track fusion welds shall be tested in peel.

5. Destructive Test Failure:

   a. The following procedures shall apply whenever a sample fails a destructive test, whether the test is conducted by the Geosynthetic CQA's laboratory, the Geosynthetic Installer laboratory, or by a field tensiometer. The Geosynthetic Installer shall have two options:

      i. The Geosynthetic Installer can reconstruct the seam (e.g., remove the old seam and reseam) between any two laboratory-passed destructive test locations created by that seaming apparatus. Trial welds do not count as a passed destructive test.

      ii. The Geosynthetic Installer can trace the welding path in each direction to an intermediate location, a minimum of 10 feet from the location of the failed test, and take a small sample for an additional field test at each location. If these additional samples pass the field tests, then full laboratory samples shall be taken. These full laboratory samples shall be tested in accordance with Subpart 3.03.J.4 of this Section. If these laboratory samples pass the tests, then the seam path between these locations shall be reconstructed and nondestructively (at a minimum) tested. If a sample fails, then the process shall be repeated, i.e. another destructive sample shall be obtained and tested at a distance of at least 10 more feet in the seaming path from the failed sample. The seam path between the ultimate passing sample locations shall be reconstructed and nondestructively (at a minimum) tested. In cases where repaired seam lengths exceed 150 feet, a destructive sample shall be taken from the repaired seam and the above procedures for destructive seam testing shall be followed.

b. Whenever a sample fails destructive or non-destructive testing, the CQA Engineer may require additional destructive tests be obtained from seams that were created by the same seamer and/or seaming apparatus during the same time shift.

K. Defects and Repairs:

1. The geomembrane will be inspected before and after seaming for evidence of defects, holes, blisters, undispersed raw materials, and any sign of contamination by foreign matter. The surface of the geomembrane shall be clean at the time of inspection. The geomembrane surface shall be swept or washed by the Installer if surface contamination inhibits inspection.

2. At observed suspected flawed location, both in seamed and non-seamed areas, shall be nondestructively tested using the methods described Subpart 3.03.I of this Section, as appropriate. Each location that fails nondestructive testing shall be marked by the CQA Engineer and repaired by the Geosynthetic Installer.

3. When seaming of a geomembrane is completed (or when seaming of a large area of a geomembrane is completed) and prior to placing overlying materials, the CQA Engineer shall identify all excessive geomembrane wrinkles. The Geosynthetic Installer shall cut and reseam all wrinkles so identified. The seams thus produced shall be tested as per all other seams.

4. Repair Procedures:

a. Any portion of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, shall be repaired by the Geosynthetic Installer. Several repair procedures are acceptable. The final decision as to the appropriate repair procedure shall be agreed upon between the Engineer and the Geosynthetic Installer. The procedures available include:

i. Patching – extrusion welding a patch to repair holes larger than 1/16 inch, tears, undispersed raw materials, and contamination by foreign matter;

ii. Abrading and reseaming – applying an extrusion seam to repair very small sections of faulty extruded seams;

iii. Spot seaming – applying an extrusion bead to repair minor, localized flaws such as scratches and scuffs;

iv. Capping – extrusion welding a geomembrane cap over long lengths of failed seams; and

v. Strip repairing – cutting out bad seams and replacing with a strip of new material seamed into place on both sides with fusion welding.

b. In addition, the following criteria shall be satisfied:

i. surfaces of the geomembrane that are to be repaired shall be abraded no more than 20 minutes prior to the repair;

ii. all surfaces must be clean and dry at the time of repair;

iii. all seaming equipment used in repair procedures must be approved by trial seaming;

iv. any other potential repair procedures shall be approved in advance, for the specific repair, by the Engineer;

v. patches or caps shall extend at least 6 inches beyond the edge of the defect, and all corners of patches and holes shall be rounded with a radius of at least 3 inches;

vi. extrudate shall extend a minimum of 3 inches beyond the edge of the patch; and

vii. any geomembrane below large caps shall be appropriately cut to avoid water or gas collection between the two sheets.

5. Repair Verification:

a. Repairs shall be nondestructively tested using the methods described in Subpart 3.03.I of this Section, as appropriate. Repairs that pass nondestructive testing shall be considered acceptable repairs. Repairs that failed nondestructive or destructive testing will require the repair to be reconstructed and retested until passing test results are observed. At the discretion of the CQA Engineer, destructive testing may be required on any caps.

3.04 MATERIALS IN CONTACT WITH THE GEOMEMBRANE

A. The Geosynthetic Installer shall take all necessary precautions to ensure that the geomembrane is not damaged during its installation. During the installation of other components of the liner system by the Contractor, the Contractor shall ensure that the geomembrane is not damaged. Any damage to the geomembrane caused by the Contractor shall be repaired by the Geosynthetic Installer at the expense of the Contractor.

B. Soil and aggregate materials shall not be placed over the geomembranes at ambient temperatures below 32°F or above 122°F, unless otherwise specified.

C. All attempts shall be made to minimize wrinkles in the geomembrane.

D. Construction loads permitted on the geomembrane are limited to foot traffic and all terrain vehicles with a contact pressures at or lower than that exhibited by foot traffic.

3.05 CONFORMANCE TESTING

A. Samples of the geomembrane will be removed by the CQA Engineer and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. The CQA Engineer may collect samples at the manufacturing plant or from the rolls delivered to the site. The Geosynthetic Installer shall assist the CQA Engineer in obtaining conformance samples from any geomembrane rolls sampled at the site. The Geosynthetic Installer and Contractor shall account for this sampling and testing requirement in the installation schedule, including the turnaround time for laboratory results. Only materials that meet the requirements of Subpart 2.02 of this Section shall be installed.

B. Samples will be selected by the CQA Engineer in accordance with this Section and with the procedures outlined in the CQA Plan.

C. Samples will be taken at a minimum frequency of one sample per 100,000 square feet. If the Geomembrane Manufacturer provides material that requires sampling at a frequency (due to lot size, shipment size, etc.) resulting in one sample per less than 90 percent of 100,000 square feet (90,000 square feet), then the Geosynthetic Installer shall pay the cost for all additional testing.

D. The CQA Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Subpart 2.02 of this Section.

E. The following tests will be performed by the CQA Engineer:

Test	Test Method
Specific Gravity	ASTM D 792 or D 1505
Thickness	ASTM D 5199
Tensile Properties	ASTM D 638
Carbon Black Content	ASTM D 1603
Carbon Black Dispersion	ASTM D 5596

F. Any geomembrane that is not certified in accordance with Subpart 1.06.C of this Section, or that conformance testing indicates does not comply with Subpart 2.02 of this Section, shall be rejected. The Geosynthetic Installer shall replace the rejected material with new material.

3.06 GEOMEMBRANE ACCEPTANCE

A. The Geosynthetic Installer shall retain all ownership and responsibility for the geomembrane until accepted by the Owner.

B. The geomembrane will not be accepted by the Owner before:

1. the installation is completed;
2. all documentation is submitted;
3. verification of the adequacy of all field seams and repairs, including associated testing, is complete; and
4. all warranties are submitted.

3.07 PROTECTION OF WORK

A. The Geosynthetic Installer and Contractor shall use all means necessary to protect all work of this Section.

B. In the event of damage, the Geosynthetic Installer shall make all repairs and replacements necessary, to the satisfaction of the Engineer.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for 60-mil, smooth and textured HDPE geomembrane will be measured as in-place square feet (SF), as measured by the surveyor, including geomembrane in the anchor trench to the limits shown on the Drawings, and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

- Submittals.
- Quality Control.
- Shipping, handling and storage.
- Deployment.
- Layout survey.
- Mobilization.
- Rejected material.
- Rejected material removal, handling, re-testing, and repair.
- Overlaps and seaming.
- Temporary anchorage.
- Pipe boots.
- Cleaning seam area.

TABLE 02770-1
REQUIRED HDPE GEOMEMBRANE PROPERTIES

PROPERTIES	QUALIFIERS	UNITS	SMOOTH HDPE SPECIFIED VALUES	TEXTURED HDPE SPECIFIED VALUES	TEST METHOD
Physical Properties					
Thickness	Average Minimum	mils mils	60 54	60 54	ASTM D 5199
Specific Gravity	Minimum	N/A	0.94	0.94	ASTM D 792 Method A or ASTM D 1505
Mechanical Properties Tensile Properties (each direction) 1. Tensile (Break) Strength 2. Elongation at Break 3. Tensile (Yield) Strength 4. Elongation at Yield	Minimum	lb/in % lb/in %	228 700 126 12	90 100 126 12	ASTM D 638
Puncture	Minimum	lb	108	90	ASTM D 4833
Environmental Properties					
Carbon Black Content	Range	%	2-3	2	ASTM D 1603
Carbon Black Dispersion	N/A	none	Note 1	Note 1	ASTM D 5596
Environmental Stress Crack	Minimum	hr	300	300	ASTM D 5397

Notes: (1) Minimum 9 of 10 in Categories 1 or 2; 10 in Categories 1, 2, or 3.

TABLE 02770-2
REQUIRED GEOMEMBRANE SEAM PROPERTIES

PROPERTIES	QUALIFIERS	UNITS	SPECIFIED VALUES(3)	TEST METHOD
Shear Strength(1)				
Fusion	minimum	lb/in	120	ASTM D 6392
Extrusion	minimum	lb/in	120	ASTM D 6392
Peel Adhesion				
FTB(2)				Visual Observation
Fusion	minimum	lb/in	91	ASTM D 6392
Extrusion	minimum	lb/in	78	ASTM D 6392

Notes: (1) Also called “Bonded Seam Strength”.
(2) FTB = Film Tear Bond means that failure is in the parent material, not the seam. The maximum seam separation is 25 percent of the seam area.
(3) Four of five specimens per destructive sample must pass both the shear and peel strength tests.

[END OF SECTION]

SECTION 02771
GEOTEXTILE

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary for the installation of the geotextile. The work shall be carried out as specified herein and in accordance with the Drawings and the Construction Quality Assurance (CQA) Plan.

B. The work shall include, but not be limited to, delivery, offloading, storage, placement, and seaming of the various geotextile components of the project.

C. Geotextile shall be used between the Drainage Aggregate and Geomembrane as shown on the Drawings.

1.02 RELATED SECTIONS

Section 02200 – Earthwork

Section 02225 – Drainage Aggregate

Section 02770 – Geomembrane

Section 02773 – Geonet

1.03 REFERENCES

A. Drawings

B. Site CQA Plan

C. Latest version of American Society for Testing and Materials (ASTM) standards:

ASTM D 4355 Standard Test Method for Deterioration of Geotextile from Exposure to Ultraviolet Light and Water

ASTM D 4439 Terminology for Geosynthetics

ASTM D 4491 Standard Test Method for Water Permeability of Geotextile by Permittivity

ASTM D 4533 Standard Test Method for Trapezoid Tearing Strength of Geotextile

ASTM D 4632 Standard Test Method for Breaking Load and Elongation of Geotextile (Grab Method)

ASTM D 4751 Standard Test Method for Determining Apparent Opening Size of a Geotextile

ASTM D 4833 Standard Test Method for Index Puncture Resistance of Geotextile, Geomembranes, and Related Products

ASTM D 5261 Standard Test Method for Measuring Mass Per Unit Area of Geotextile

1.04 SUBMITTALS

A. The Contractor shall submit the following information regarding the proposed geotextile to the Engineer for approval at least 7 days prior to geotextile delivery:

1. manufacturer and product name;

2. minimum property values of the proposed geotextile and the corresponding test procedures;

3. projected geotextile delivery dates; and

4. list of geotextile roll numbers for rolls to be delivered to the site.

B. At least 7 days prior to geotextile placement, the Contractor shall submit to the Engineer the Manufacturing Quality Control (MQC) certificates for each roll of geotextile. The certificates shall be signed by responsible parties employed by the geotextile manufacturer (such as the production manager). The MQC certificates shall include:

1. lot, batch, and/or roll numbers and identification;

2. MQC test results, including a description of the test methods used; and

3. Certification that the geotextile meets or exceeds the required properties of the Drawings and this Section.

1.05 CQA MONITORING

A. The Contractor shall be aware of and accommodate all monitoring and conformance testing required by the CQA Plan. This monitoring and testing, including random conformance testing of construction materials and completed work, will be performed by the CQA Engineer. If nonconformances or other deficiencies are found in the Contractor's materials or completed work, the Contractor will be required to repair the deficiency or replace the deficient materials at no additional expense to the Owner.

PART 2 – PRODUCTS

2.01 GEOTEXTILE PROPERTIES

A. The Geotextile Manufacturer shall furnish materials that meet or exceed the criteria specified in Table 02771-1 in accordance with the minimum average roll value (MARV), as defined by ASTM D 4439.

B. The geotextile shall be nonwoven materials, suitable for use in filter/separation and cushion applications.

2.02 MANUFACTURING QUALITY CONTROL (MQC)

A. The geotextile shall be manufactured with MQC procedures that meet or exceed generally accepted industry standards.

B. The Geotextile Manufacturer shall sample and test the geotextile to demonstrate that the material conforms to the requirements of these Specifications.

C. Any geotextile sample that does not comply with this Section shall result in rejection of the roll from which the sample was obtained. The Contractor shall replace any rejected rolls.

D. If a geotextile sample fails to meet the MQC requirements of this Section the Geotextile Manufacturer shall additionally sample and test, at the expense of the Manufacturer, rolls manufactured in the same lot, or at the same time, as the failing roll. Sampling and testing of rolls shall continue until a pattern of acceptable test results is established to define the bounds of the failed roll(s). All the rolls pertaining to the failed rolls shall be rejected.

E. Additional sample testing may be performed, at the Geotextile Manufacturer's discretion and expense, to identify more closely the extent of non-complying rolls and/or to qualify individual rolls.

F. Sampling shall, in general, be performed on sacrificial portions of the geotextile material such that repair is not required. The Geotextile Manufacturer shall sample and test the geotextile to demonstrate that the geotextile properties conform to the values specified in Table 02771-1.

1. At a minimum, the following MQC tests shall be performed on the geotextile (results of which shall meet the requirements specified in Table 02271):

Test	Procedure	Frequency
Grab strength	ASTM D 4632	130,000 ft^2
Mass per Unit Area	ASTM D 5261	130,000 ft^2
Tear strength	ASTM D 4533	130,000 ft^2
Puncture strength	ASTM D 4833	130,000 ft^2
Permittivity	ASTM D 4491	540,000 ft^2
A.O.S.	ASTM D 4751	540,000 ft^2

G. The Geotextile Manufacturer shall comply with the certification and submittal requirements of this Section.

2.03 PACKING AND LABELING

A. Geotextile shall be supplied in rolls wrapped in relatively impervious and opaque protective covers.

B. Geotextile rolls shall be marked or tagged with the following information:

1. manufacturer's name;
2. product identification;
3. lot or batch number;
4. roll number; and
5. roll dimensions.

2.04 TRANSPORTATION, HANDLING, AND STORAGE

A. The Geosynthetic Manufacturer shall be liable for any damage to the geotextile incurred prior to and during transportation to the site.

B. The geotextile shall be delivered to the site at least 14 days prior to the planned deployment date to allow the CQA Engineer adequate time to perform conformance testing on the geotextile samples as described in Subpart 3.06 of this Section.

C. Handling, unloading, storage, and care of the geotextile at the site, prior to and following installation, are the responsibility of the Contractor. The Contractor shall be liable for any damage to the materials incurred prior to final acceptance by the Owner.

D. The Contractor shall be responsible for offloading and storage of the geotextile at the site.

E. The geotextile shall be protected from sunlight, puncture, or other damaging or deleterious conditions. The geotextile shall be protected from mud, dirt, and dust. Any additional storage procedures required by the geotextile Manufacturer shall be the responsibility of the Contractor.

PART 3 – EXECUTION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work described in this Section, the Contractor shall become thoroughly familiar with the site, the site conditions, and all portions of the work falling within this Section.

B. If the Contractor has any concerns regarding the installed work of other Sections or the site, the Engineer shall be notified, in writing, prior to commencing the work. Failure to notify the Engineer or commencing installation of the geotextile will be construed as Contractor's acceptance of the related work of all other Sections.

3.02 PLACEMENT

A. Geotextile installation shall not commence over other materials until CQA conformance evaluations, by the CQA Engineer, of underlying materials are complete, including evaluations of the Contractor's survey results to confirm that the previous work was constructed to the required grades, elevations, and thicknesses. Should the Contractor begin the work of this Section prior to the completion of CQA evaluations for underlying materials or this material, this shall be at the risk of removal of these materials, at the Contractor’s expense, to remedy the non-conformances. The Contractor shall account for the CQA conformance evaluations in the construction schedule.

B. The Contractor shall handle all geotextile in such a manner as to ensure it is not damaged in any way.

C. The Contractor shall take any necessary precautions to prevent damage to underlying materials during placement of the geotextile.

D. After unwrapping the geotextile from its opaque cover, the geotextile shall not be left exposed for a period in excess of 15 days unless a longer exposure period is approved in writing by the Geotextile Manufacturer.

E. The Contractor shall take care not to entrap stones, excessive dust, or moisture in the geotextile during placement.

F. The Contractor shall anchor or weight all geotextile with sandbags, or the equivalent, to prevent wind uplift.

G. The Contractor shall examine the entire geotextile surface after installation to ensure that no foreign objects are present that may damage the geotextile or adjacent layers. The Contractor shall remove any such foreign objects and shall replace any damaged geotextile.

3.03 SEAMS AND OVERLAPS

A. On slopes steeper than 10 horizontal to 1 vertical, geotextiles shall be continuous down the slope; that is, no horizontal seams are allowed. Horizontal seams shall be considered as any seam having an alignment exceeding 20 degrees from being perpendicular to the slope contour lines, unless otherwise approved by the Engineer. No horizontal seams shall be allowed within 5 feet of the top or toe of the slopes.

B. Geotextile shall be overlapped a minimum of 12-inches.

3.04 REPAIR

A. Any holes or tears in the geotextile shall be repaired using a patch made from the same geotextile. If a tear exceeds 50 percent of the width of a roll, that roll shall be removed and replaced.

3.05 PLACEMENT OF SOIL MATERIALS

A. The Contractor shall place soil materials on top of the geotextile in such a manner as to ensure that:

1. the geotextile and the underlying materials are not damaged;
2. minimum slippage occurs between the geotextile and the underlying layers during placement; and
3. excess stresses are not produced in the geotextile.

B. Equipment shall not be driven directly on the geotextile.

3.06 CONFORMANCE TESTING

A. Conformance samples of the geotextile materials will be removed by the CQA Engineer after the material has been received at the site and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section and the CQA Plan. This testing will be carried out, in accordance with the CQA Plan, prior to the start of the work of this Section.

B. Samples of each geotextile will be taken, by the CQA Engineer, at a minimum frequency of one sample per 260,000 square feet (minimum of one).

C. The CQA Engineer may increase the frequency of sampling in the event that test results do not comply with requirements of Subpart 2.01 of this Section until passing conformance test results are obtained for all material that is received at the site. This additional testing shall be performed at the expense of the Contractor.

D. The following conformance tests will be performed (results of which shall meet the requirements specified in Table 02771):

Test	Procedure
Grab strength	ASTM D 4632
Mass per Unit Area	ASTM D 5261
Puncture strength	ASTM D 4833
Permittivity	ASTM D 4491
A.O.S.	ASTM D 4751

E. Any geotextile that is not certified in accordance with Subpart 1.04 of this Section, or that conformance testing results do not comply with Subpart 2.01 of this Section, will be rejected. The Contractor shall replace the rejected material with new material. All other rolls that are represented by failing test results will also be rejected, unless additional testing is performed to further determine the bounds of the failed material.

3.07 PROTECTION OF WORK

A. The Contractor shall protect all work of this Section.

B. In the event of damage, the Contractor shall make repairs and replacements to the satisfaction of the Engineer at the expense of the Contractor.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for Geotextile will be incidental to PVC Pipe, and payment will be based on the unit price provided for PVC Pipe on the Bid Schedule.

B. The following are considered incidental to the work:

- Submittals.
- Quality Control.
- Shipping, handling, and storage.
- Layout survey.
- Mobilization.
- Rejected material.
- Overlaps and seaming.
- Rejected material removal, handling, re-testing, and repair.
- Temporary anchorage.

TABLE 02771-1
REQUIRED PROPERTY VALUES FOR GEOTEXTILE

PROPERTIES	QUALIFIERS	UNITS	SPECIFIED VALUES	TEST METHOD
Physical Properties				
Mass per unit area	Minimum	oz/yd^2	16	ASTM D 5261
Apparent opening size (O_{95})	Maximum	mm	0.21	ASTM D 4751
Permittivity	Minimum	s^{-1}	0.5	ASTM D 4491
Grab strength	Minimum	lb	390	ASTM D 4632
Tear strength	Minimum	lb	150	ASTM D 4533
Puncture strength	Minimum	lb	240	ASTM D 4833
Ultraviolet Resistance @ 500 hours	Minimum	%	70	ASTM D 4355

[END OF SECTION]

SECTION 02772
GEOSYNTHETIC CLAY LINER

PART 1 – GENERAL

1.01 SCOPE

A. The Geosynthetic Installer shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary for installation of the geosynthetic clay liner (GCL). The work shall be carried out as specified herein and in accordance with the Drawings and Construction Quality Assurance (CQA) Plan.

B. The work shall include, but not be limited to, delivery, offloading, storage, placement, anchorage, and seaming of the GCL.

1.02 RELATED SECTIONS

Section 02220 – Subgrade Preparation

Section 02770 – Geomembrane

1.03 REFERENCES

A. Drawings

B. Site CQA Plan

C. Latest Version American Society of Testing and Materials (ASTM) Standards:

ASTM D 5887	Test Method for Measurement of Index Flux Through Saturated Geosynthetic Clay Liner Specimens using a Flexible Wall Permeameter
ASTM D 5888	Guide for Storage and Handling of Geosynthetic Clay Liners
ASTM D 5890	Test Method for Swell Index of Clay Mineral Component of Geosynthetic Clay Liners
ASTM D 5891	Test Method for Fluid Loss of Clay Component of Geosynthetic Clay Liners
ASTM D 5993	Test Method for Measuring Mass per Unit Area of Geosynthetic Clay Liners

1.04 QUALIFICATIONS

A. GCL Manufacturer:

1. The Manufacturer shall be a well-established firm with more than five (5) years of experience in the manufacturing of GCL.

2. The GCL Manufacturer shall be responsible for the production of GCL rolls and shall have sufficient production capacity and qualified personnel to provide material meeting the requirements of this Section and the construction schedule for this project.

B. GCL Installer:

1. The Geosynthetic Installer shall install the GCL and shall meet the requirements of Section 02770 Subpart 1.04. B and this Section.

2. The Geosynthetics Installer shall be responsible and shall provide sufficient resources for field handling, deploying, temporarily restraining (against wind), and other aspects of the

deployment and installation of the GCL and other geosynthetic components of the project.

1.05 SUBMITTALS

A. At least 7 days before transporting any GCL to the site, the Manufacturer shall provide the following documentation to the Engineer for approval.

1. list of material properties, including test methods utilized to analyze/confirm properties.

2. GCL samples.

3. projected delivery dates for this project.

4. Manufacturing quality control certificates for each shift's production for which GCL for the project was produced, signed by responsible parties employed by the Manufacturer (such as the production manager).

5. Manufacturer Quality Control (MQC) certificates, including:

   a. roll numbers and identification; and

   b. MQC results, including description of test methods used, outlined in Subpart 2.02 of this Section.

6. Certification that the GCL meets all the properties outlined in Subpart 2.01 of this Section.

1.06 CONSTRUCTION QUALITY ASSURANCE (CQA) MONITORING

A. The Geosynthetic Installer shall be aware of all monitoring and conformance testing required by the CQA Plan. This monitoring and testing, including random conformance testing of construction materials and completed work, will be performed by the CQA Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials at no additional cost to the Owner.

PART 2 – PRODUCTS

2.01 MATERIAL PROPERTIES

A. The flux of the bentonite portion of the GCL shall be no greater than 1×10^{-8} m^3/m^2-sec, when measured in a flexible wall permeameter in accordance with ASTM D 5887 under an effective confining stress of 5 pounds per square inch (psi).

B. The GCL shall have the following minimum dimensions:

1. the minimum roll width shall be 15 feet; and

2. the linear length shall be long enough to conform with the requirements specified in this Section.

C. The bentonite used to fabricate the GCL shall be comprised of at least 88 percent sodium montmorillonite.

D. The bentonite component of the GCL shall be applied at a minimum concentration of 0.75 pound per square foot (psf), when measured at a water content of 0 percent.

E. The GCL shall meet or exceed all required property values listed in Table 02772-1.

F. The bentonite will be adhered to the backing material(s) in a manner that prevents it from being dislodged when transported, handled, and installed in a manner prescribed by the Manufacturer. The method used to hold the bentonite in place shall not be detrimental to other components of the lining system.

G. The geotextile components of the GCL shall be woven and nonwoven and have a combined mass per unit area of 9 ounces per square yard (oz./SY).

H. The GCL shall be needle punched.

2.02 INTERFACE SHEAR TESTING

A. Interface Shear test(s) shall be performed on the proposed geosynthetic and soil components in accordance with ASTM D 5321. Tests shall be performed on several geosynthetic interfaces as outlined below.

1. Hydrated GCL and Cushion Geotextile to textured HDPE Geomembrane interface - the GCL shall be underlain by prepared subgrade compacted to 90% of the maximum dry density (ASTM D 1557) at the optimum moisture content and overlain by a textured 60-mil HDPE geomembrane and cushion geotextile. The geosynthetic components of the liner system shall be allowed to “float” (i.e., not fixed) such that the failure surface can occur between any of the interfaces.

   a. The test shall evaluate the interface between the woven GCL or cushion geotextile and a textured HDPE geomembrane. Before shearing, the GCL shall be hydrated under for 48 hours. The test shall be performed at normal stresses of 100, 200, and 300 psf at a shear rate of no more than 0.04 in./min. (1 mm/min.).

   b. The results of this test shall have a peak apparent friction angle in excess of 25 degrees.

2. Hydrated GCL and geonet to smooth geomembrane interface - the GCL shall be underlain by prepared subgrade compacted to 90% of the maximum dry density (ASTM D 1557) at the optimum moisture content and overlain by a smooth 60-mil HDPE geomembrane and geonet. The geosynthetic components of the liner system shall be allowed to “float” (i.e., not fixed) such that the failure surface can occur between any of the interfaces.

   a. The test shall evaluate the interface between the woven GCL or geonet and a smooth HDPE geomembrane. Before shearing, the GCL shall be hydrated under a loading of 250 psf for 48 hours. The test shall be performed at normal stresses of 10, 20, and 30 psi at a shear rate of no more than 0.04 in./min. (1 mm/min.).

   b. The results of this test shall have a peak apparent friction angle in excess of 10 degrees.

2.03 MANUFACTURING QUALITY CONTROL (MQC)

A. The GCL shall be manufactured with quality control procedures that meet or exceed generally accepted industry standards.

B. The Manufacturer shall sample and test the GCL to demonstrate that the material complies with the requirements of this Section.

C. Any GCL sample that does not comply with this Section will result in rejection of the roll from which the sample was obtained. The Manufacturer shall replace any rejected rolls.

D. If a GCL sample fails to meet the quality control requirements of this Section, the Engineer will require that the Manufacturer sample and test, at the expense of the Manufacturer, rolls manufactured in the same lot, or at the same time, as the failing roll. Sampling and testing of rolls shall continue until a pattern of acceptable test results is established to determine the bounds of the failed roll(s). All rolls pertaining to failed tests shall be rejected.

E. Additional sample testing may be performed, at the Manufacturer's discretion and expense, to more closely identify the extent of any non-complying rolls and/or to qualify individual rolls.

F. Sampling shall, in general, be performed on sacrificial portions of the GCL material such that repair is not required. The Manufacturer shall sample and test the GCL to demonstrate that its properties conform to the requirements stated herein. At a minimum, the following (MQC) tests shall be performed by the Manufacturer: dry mass per unit area (ASTM D5993) and index flux at frequencies of at least one per 50,000 square feet and one per 200,000 square feet, respectively.

G. The Manufacturer shall comply with the certification and submittal requirements of this Section.

2.04 PACKING AND LABELING

A. GCL shall be supplied in rolls wrapped in impervious and opaque protective covers.

B. GCL shall be marked or tagged with the following information:

1. Manufacturer's name;
2. product identification;
3. lot number;
4. roll number; and
5. roll dimensions.

2.05 TRANSPORTATION, HANDLING AND STORAGE

A. The Geosynthetic Manufacturer shall be liable for any damage to the GCL incurred prior to and during transportation to the site.

B. Handling, storage, and care of the GCL at the site prior to and following installation, are the responsibility of the Geosynthetic Installer, until final acceptance by the Owner.

C. The GCL shall be stored and handled in accordance with ASTM D 5888.

D. The Geosynthetic Installer shall be liable for all damage to the materials incurred prior to and during transportation to the site including hydration of the GCL prior to placement.

E. The GCL shall be on-site at least 14 days prior to the scheduled installation date to allow for completion of conformance testing described in Subpart 3.07 of this Section.

PART 3 – EXECUTION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work described in this Section, the Geosynthetic Installer shall carefully inspect the installed work of all other Sections and verify that all work is complete to the point where the installation of this Section may properly commence without adverse impact.

B. If the Geosynthetic Installer has any concerns regarding the installed work of other Sections, he should notify the Engineer in writing prior to commencing the work. Failure to notify the

Engineer or commencing installation of the GCL will be construed as Geosynthetic Installer's acceptance of the related work of all other Sections.

C. A pre-installation meeting shall be held to coordinate the installation of the GCL with the installation of other components of the lining system.

3.02 SURFACE PREPARATION

A. The Geosynthetics Installer shall provide certification in writing that the surface on which the GCL will be installed is acceptable. This certification of acceptance shall be given to the Engineer's representative prior to commencement of geosynthetics installation in the area under consideration. Special care shall be taken to maintain the prepared soil surface..

B. No GCL shall be placed onto an area that has been softened by precipitation or that has cracked due to desiccation. The soil surface shall be observed daily to evaluate the effects of desiccation cracking and/or softening on the integrity of the prepared subgrade.

3.03 HANDLING AND PLACEMENT

A. The Geosynthetic Installer shall handle all GCL in such a manner that it is not damaged in any way.

B. In the presence of wind, all GCL shall be sufficiently weighted with sandbags to prevent their movement.

C. Any GCL damaged by stones or other foreign objects, or by installation activities, shall be repaired in accordance with Subpart 3.06 by the Geosynthetic Installer, at the expense of the Geosynthetic Installer.

D. CQA EngineerCQA EngineerThe GCL shall be installed with the woven geotextile facing up (against the overlying geomembrane).

3.04 OVERLAPS

A. On slopes steeper than 10:1 (horizontal:vertical), all GCL shall be continuous down the slope, i.e., no horizontal seams shall be allowed on the slope. Horizontal seams shall be considered as any seam having an alignment exceeding 30 degrees from being perpendicular to the slope contour lines, unless otherwise approved by the Engineer.

B. All GCL shall be overlapped in accordance with the Manufacturer's recommended procedures. At a minimum, along the length (i.e., the sides) of the GCL placed on slopes steeper than 10:1 (horizontal:vertical), the overlap shall be 12 inches, and along the width (i.e., the ends) the overlap shall be 24 inches.

C. At a minimum, along the length (i.e., the sides) of the GCL placed on non-sloped areas (i.e. slopes no steeper than 10:1), the overlap shall be 6-inches, and along the width (i.e., the ends) the overlap shall be 12-inches.

3.05 MATERIALS IN CONTACT WITH THE GCL

A. Installation of other components of the liner system shall be carefully performed to avoid damage to the GCL.

B. Engineer approved low ground pressure equipment may be driven directly on the GCL.

C. Installation of the GCL in appurtenant areas, and connection of the GCL to appurtenances shall be made according to the Drawings. The Geosynthetic Installer shall ensure that the GCL is not damaged while working around the appurtenances.

3.06 REPAIR

A. Any holes or tears in the GCL shall be repaired by placing a GCL patch over the defect. On slopes steeper than 10 percent, the patch shall overlap the edges of the hole or tear by a minimum of 2 feet in all directions. On slopes 10 percent or flatter, the patch shall overlap the edges of the hole or tear by a minimum of 1 foot in all directions. The patch shall be secured with a Manufacturer recommended water-based adhesive.

B. Care shall be taken to remove any soil, rock, or other materials, which may have penetrated the torn GCL.

C. The patch shall not be nailed or stapled.

3.07 CONFORMANCE TESTING

A. Samples of the GCL will be removed by the CQA Engineer and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section and the CQA Plan. The Geosynthetic Installer shall assist the CQA Engineer in obtaining conformance samples. The Geosynthetic Installer shall account for this testing in the installation schedule.

B. At a minimum, the following conformance tests will be performed at a minimum frequency rate of one sample per 100,000 square feet: mass per unit area (ASTM D 5993) and bentonite moisture content (ASTM D 5993). At a minimum, the following conformance tests will be performed at a frequency of one sample per 400,000 square feet: index flux (ASTM D 5887). If the GCL Manufacturer provides material that requires sampling at a frequency (due to lot size, shipment size, etc.) resulting in one sample per less than 90 percent of 100,000 square feet (90,000 square feet), then the Geosynthetic Installer shall pay the cost for all additional testing.

C. The CQA Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Subpart 2.01 of this Section until passing conformance test results are obtained for all material that is received at the site. This additional testing shall be performed at the expense of the Geosynthetic Installer.

D. Any GCL that is not certified by the Manufacturer in accordance with Subpart 1.05 of this Section or that does not meet the requirements specified in Subpart 2.01 shall be rejected and replaced by the Geosynthetic Installer, at the expense of the Geosynthetic Installer.

3.08 PROTECTION OF WORK

A. The Geosynthetic Installer shall protect all work of this Section.

B. In the event of damage, the Geosynthetic Installer shall immediately make all repairs and replacements necessary to the approval of the Engineer, at the expense of the Geosynthetic Installer.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for GCL will be measured as in-place square feet (SF), as measured by the surveyor, to the limits shown on the Drawings, and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

- Submittals.
- Quality Control.
- Shipping, handling and storage.

- Overlaps and seaming.
- Layout survey.
- Mobilization.
- Rejected material.
- Rejected material removal, handling, re-testing, and repair.
- Overlaps and seaming.
- Temporary anchorage.
- Visqueen.

TABLE 02772-1
REQUIRED GCL PROPERTY VALUES

PROPERTIES	QUALIFIERS	UNITS	SPECIFIED[(1)] VALUES	TEST METHOD
Bentonite Content[4]	minimum	lb/ft^3	0.75	ASTM D 5993
Bentonite Swell Index	minimum	mL/2g	24	ASTM D 5890
Bentonite Fluid Loss	maximum	mL	18	ASTM D 5891
Hydraulic Index Flux	maximum	m^3/m^2-s	1×10^{-8}	ASTM D 5887[3]

Notes: (1) All values represent minimum average roll values (i.e., any roll in a lot should meet or exceed the values in this table).
(2) Interface shear strength testing shall be performed, by the CQA Engineer, in accordance with Part 2.02 of this Section.
(3) Hydraulic flux testing shall be performed under an effective confining stress of 5 pounds per square inch.
(4) Measured at a moisture content of 0 percent; also known as mass per unit area

[END OF SECTION]

SECTION 02773
GEONET

PART 1 – GENERAL

1.01 SCOPE

A. The Geosynthetic Installer shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary for installation of the geonet. The work shall be carried out as specified herein and in accordance with the Drawings and Construction Quality Assurance (CQA) Plan.

B. The work shall include, but not be limited to, delivery, offloading, storage, placement, anchorage, and seaming of the geonet.

1.02 RELATED SECTIONS

Section 02220 – Subgrade Preparation

Section 02225 – Drainage Aggregate

Section 02616 – Polyvinyl Chloride (PVC) Pipe

Section 02770 – Geomembrane

Section 02771 – Geotextile

1.03 REFERENCES

A. Drawings

B. Site CQA Plan

C. Latest Version American Society of Testing and Materials (ASTM) Standards:

ASTM D792	Standard Test Methods for Specific Gravity and Density of Plastics by Displacement
ASTM D1505	Standard Test Method for Density of Plastics by the Density-Gradient Technique
ASTM D1603	Standard Test Method for Carbon Black in Olefin Plastics
ASTM D4218	Standard Test Method for Determination of Carbon Black Content in Polyethylene Compounds by Muffle-Furnace Technique
ASTM D4716	Standard Test Method for Constant Head Hydraulic Transmissivity (In-Place Flow) of Geotextiles and Geotextile Related Products
ASTM D5199	Standard Test Method for Measuring Nominal Thickness of Geosynthetics

1.04 QUALIFICATIONS

A. Geonet Manufacturer:

1. The Manufacturer shall be a well-established firm with more than five (5) years of experience in the manufacturing of geonet.

2. The Manufacturer shall be responsible for the production of geonet rolls and shall have sufficient production capacity and qualified personnel to provide material meeting the requirements of this Section and the construction schedule for this project.

B. Geonet Installer:

1. The Geosynthetic Installer shall meet the requirements of Subpart 1.04. B of Section 02770, and this Section.

2. The Geosynthetics Installer shall be responsible and shall provide sufficient resources for field handling, deploying, temporarily restraining (against wind and re-curling), and other aspects of the deployment and installation of the geonet and other geosynthetic components of the project.

1.05 SUBMITTALS

A. At least 7 days before transporting any geonet to the site, the Manufacturer shall provide the following documentation to the Engineer for approval.

1. list of material properties, including test methods utilized to analyze/confirm properties.

2. geonet samples.

3. projected delivery dates for this project.

4. Manufacturing Quality Control (MQC) certificates for each shift's production for which geonet for the project was produced, signed by responsible parties employed by the Manufacturer (such as the production manager). MQC certificates shall include:

a. roll numbers and identification; and

b. MQC results, including description of test methods used, outlined in Subpart 2.01 of this Section.

c. Certification that the geonet meets all the properties outlined in Subpart 2.01 of this Section.

1.06 CONSTRUCTION QUALITY ASSURANCE (CQA)

A. The Geosynthetic Installer shall ensure that the materials and methods used for producing and handling the geonet meet the requirements of the Drawings and this Section. Any material or method that does not conform to these documents, or to alternatives approved in writing by the Engineer, will be rejected and shall be repaired or replaced, at the Geosynthetic Installer’s expense.

B. The Geosynthetic Installer shall be aware of all monitoring and conformance testing required by the CQA Plan. This monitoring and testing, including random conformance testing of construction materials and completed work, will be performed by the CQA Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials at now additional cost to the Owner.

PART 2 – PRODUCTS

2.01 GEONET PROPERTIES

A. The Manufacturer shall furnish geonet having properties that comply with the required property values shown on Table 02773-1.

B. In addition to documentation of the property values listed in Table 02773-1, the geonet shall contain a maximum of one percent by weight of additives, fillers, or extenders (not including carbon black) and shall not contain foaming agents or voids within the ribs of the geonet.

2.02 MANUFACTURING QUALITY CONTROL (MQC)

A. The geonet shall be manufactured with MQC procedures that meet or exceed generally accepted industry standards.

B. Any geonet sample that does not comply with the Specifications will result in rejection of the roll from which the sample was obtained. The Geonet Manufacturer shall replace any rejected rolls at no additional cost to Owner.

C. If a geonet sample fails to meet the MQC requirements of this Section, then the Geonet Manufacturer shall sample and test each roll manufactured, in the same lot, or at the same time, as the failing roll. Sampling and testing of rolls shall continue until a pattern of acceptable test results is established.

D. Additional sample testing may be performed, at the Geonet Manufacturer's discretion and expense, to more closely identify any non-complying rolls and/or to qualify individual rolls.

E. Sampling shall, in general, be performed on sacrificial portions of the geonet material such that repair is not required. The Manufacturer shall sample and test the geonet, at a minimum, once every 100,000 square feet to demonstrate that its properties conform to the values specified in Table 02773-1.

F. At a minimum, the following MQC tests shall be performed:

Test	**Procedure**
Density	ASTM D 792 or D 1505
Thickness	ASTM D 5199
Carbon Black Content	ASTM D 1603

G. The hydraulic transmissivity test (ASTM D 4716) in Table 02773-1 need not be performed at a frequency of one per 100,000 square feet. However, the Geonet Manufacturer will certify that this test has been performed on a sample of geonet identical to the product that will be delivered to the Site. The Geonet Manufacturer shall provide test results as part of MQC documentation.

H. The Geonet Manufacturer shall comply with the certification and submittal requirements of this Section.

2.03 LABELING

A. Geonet shall be supplied in rolls labeled with the following information:

1. manufacturer's name;
2. product identification;
3. lot number;
4. roll number; and
5. roll dimensions.

2.04 TRANSPORTATION

A. Transportation of the geonet shall be the responsibility of the Geonet Manufacturer. The Geonet Manufacturer shall be liable for all damages to the materials incurred prior to and during transportation to the site.

B. Geonet shall be delivered to the site at least 7 days before the scheduled date of deployment to allow the CQA Engineer adequate time to inventory the geonet rolls and obtain additional conformance samples, if needed. The Geosynthetic Installer shall notify the CQA Engineer a minimum of 48 hours prior to any delivery.

2.05 HANDLING AND STORAGE

A. The Geosynthetic Manufacturer shall be responsible for handling, off-loading, storage, and care of the geonet prior to and following installation at the Site. The Geosynthetic Installer shall be liable for all damages to the materials incurred prior to final acceptance of the geonet drainage layer by the Owner.

B. The geonet shall be stored off the ground and out of direct sunlight, and shall be protected from mud and dirt. The Geosynthetic Installer shall be responsible for implementing any additional storage procedures required by the Geonet Manufacturer.

2.06 CONFORMANCE TESTING

A. Conformance testing, if required, shall be performed in accordance with the CQA Plan. The Geosynthetics installer shall assist the CQA Engineer in obtaining conformance samples, if requested. The CQA Engineer has the option of collecting samples at the manufacturing facility.

B. Passing conformance testing results, if applicable, are required before any geonet is deployed.

C. Samples shall be taken at a minimum frequency of one sample per 200,000 square feet with a minimum of one sample per lot. If the Geonet Manufacturer provides material that requires sampling at a frequency (due to lot size, shipment size, etc.) resulting in one sample per less than 90 percent of 200,000 square feet (180,000 square feet), then the Geosynthetic Installer shall pay the cost for all additional testing.

D. The CQA Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Subpart 2.01 of this Section until passing conformance test results are obtained for all material that is received at the Site. This additional testing shall be performed at the expense of the Geosynthetic Installer.

E. Any geonet that are not certified in accordance with Subpart 1.05 of this Section, or that conformance testing indicates do not comply with Subpart 2.01 of this Section, will be rejected by the CQA Engineer. The Geonet Manufacturer shall replace the rejected material with new material at no additional cost to the Owner.

PART 3 – EXECUTION

3.01 HANDLING AND PLACEMENT

A. On slopes steeper than 10:1 (horizontal:vertical), all geonet shall be continuous down the slope, i.e., no horizontal seams shall be allowed on the slope. Horizontal seams shall be considered as any seam having an alignment exceeding 20 degrees from being perpendicular to the slope contour lines, unless otherwise approved by the Engineer.

B. Geonet shall be placed with the machine direction perpendicular to the contour intervals (i.e. placed with machine direction in line with the direction of flow).

C. The geonet shall be handled in such a manner as to ensure it is not damaged in any way.

D. Precautions shall be taken to prevent damage to underlying layers during placement of the geonet.

E. The geonet shall be installed in a manner that minimizes wrinkles.

F. Care shall be taken during placement of geonet to prevent dirt or excessive dust in the geonet that could cause clogging and/or damage to the adjacent materials.

3.02 JOINING AND TYING

A. Adjacent panels of geonet shall be overlapped by at least 4 inches These overlaps shall be secured by tying with nylon ties.

B. Tying shall be achieved by plastic fasteners or polymer braid. Tying devices shall be white or yellow for easy inspection. Metallic devices shall not be used.

C. Tying shall be performed at a minimum interval of every 5 feet along the geonet roll edges and 2 feet along the geonet roll ends.

3.03 REPAIR

A. Any holes or tears in the geonet shall be repaired by placing a patch extending 1 foot. beyond the edges of the hole or tear. The patch shall be placed under the panel and secured to the original geonet by tying every 6 inches with approved tying devices. If the hole or tear width across the roll is more than 50 percent of the width of the roll, then the damaged area shall be cut out and the two portions of the geonet shall be joined in accordance with the requirements of Subpart 3.02 of this Section.

3.04 PRODUCT PROTECTION

A. The Geosynthetics Installer shall use all means necessary to protect all prior work, and all materials and completed work of other Sections.

B. In the event of damage to the geonet, the Geosynthetic Installer shall immediately make all repairs per the requirements of this Section.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for geonet will be measured as in-place square feet (SF), as measured by the surveyor, to the limits shown on the Drawings, and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

- Submittals.
- Quality Control.
- Shipping, handling, and storage.
- Overlaps and seaming.
- Layout survey.
- Offloading.
- Mobilization.
- Rejected material.
- Rejected material removal, handling, re-testing, and repair.
- Temporary anchorage.

TABLE 02773-1
REQUIRED GEONET PROPERTY VALUES

PROPERTIES	QUALIFIERS	UNITS	SPECIFIED[1] VALUES	TEST METHOD
Resin Density	Minimum	g/cc	0.94	ASTM D792 or D1505
Carbon Black Content	Range	%	2.0 – 3.0	ASTM D1603 or D4218
Thickness	Minimum	Mils	300	ASTM D5199
Transmissivity[2]	Minimum	m^2 / sec	8×10^{-3}	ASTM D4716

Notes: (1) All values (except transmissivity) represent average roll values.
(2) Transmissivity shall be measured using water at 68°F with a gradient of 0.1 under a confining pressure of 7,000 lb/ft^2. The geonet shall be placed in the testing device between 60-mil HDPE smooth geomembrane. Measurements are taken one hour after application of confining pressure.

[END OF SECTION]

SECTION 03400
CAST-IN-PLACE CONCRETE

PART 1 – GENERAL

1.01 DESCRIPTION OF WORK

A. The Contractor shall furnish all labor, materials, tools, transportation and equipment necessary to construct a cast-in-place spillway crossing as shown on the Drawings and as specified herein.

B. The Work shall include, but not be limited to, procurement, delivery, subgrade preparation, formwork, concrete placement, control joints, surface treatment, and curing.

1.02 RELATED SECTIONS

None.

1.03 REFERENCES

A. Drawings

B. Construction Quality Assurance (CQA) Plan

C. Latest version of American Concrete Institute (ACI) standards:

ACI 117	Tolerances for Concrete Construction and Materials
ACI 211.1	Selecting Proportions for Normal, Heavyweight, and Mass Concrete
ACI 301	Structural Concrete for Buildings
ACI 304R	Measuring, Mixing, Transporting, and Placing Concrete
ACI 308	Standard Practice for Curing Concrete
ACI 318	Building Code Requirements for Reinforced Concrete
ACI 347R	Formwork for Concrete

D. Latest version of the American Society for Testing and Materials (ASTM) standards:

ASTM A 615	Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
ASTM C 33	Concrete Aggregates
ASTM C 39	Compressive Strength of Cylindrical Concrete Specimens
ASTM C 94	Ready- Mixed Concrete
ASTM C 127	Specific Gravity and Adsorption of Coarse Aggregate
ASTM C 128	Specific Gravity and Adsorption of Fine Aggregate
ASTM C 143	Slump of Hydraulic Cement Concrete
ASTM C 150	Portland Cement

ASTM C 171	Sheet Materials for Curing Concrete
ASTM C 192	Making and Curing Concrete Test Specimens in the Laboratory
ASTM C 309	Liquid Membrane - Forming Compounds for Curing Concrete
ASTM C 403	Time of Setting of Concrete Mixtures by Penetration Resistance
ASTM C 494	Chemical Admixtures for Concrete
ASTM C 618	Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete

1.04 SUBMITTALS

A. At least 7 days prior to construction of the concrete, Contractor shall submit a mix design for the type of concrete. Submit a complete list of materials including types, brands, sources, amount of cement, fly ash, pozzolans, retardants, and admixtures, and applicable reference specifications for the following:

1. Slump design based on total gallons of water per cubic yard.

2. Type and quantity of cement.

3. Brand, type, ASTM designation, active chemical ingredients, and quantity of each admixture.

4. Compressive strength based on 28-day compression tests.

B. Delivery Tickets:

1. Provide duplicate delivery tickets with each load of concrete delivered, one for Contractor's records and one for Engineer, with the following information:

a. Date and serial number of ticket.
b. Name of ready-mixed concrete plant, operator, and job location.
c. Type of cement, admixtures, if any, and brand name.
d. Cement content, in bags per cubic yard (CY) of concrete, and mix design.
e. Truck number, time loaded, and name of dispatcher.
f. Amount of concrete (CY) in load delivered.
g. Gallons of water added at job, if any, and slump of concrete after water was added.

C. Delivery

1. The Concrete Manufacturer shall be liable for all damage to the materials incurred prior to and during transportation to the Site.

1.05 MANUFACTURER QUALITY CONTROL (MQC)

A. Aggregates shall be sampled and tested in accordance with ASTM C 33.

B. Concrete test specimens shall be made, cured, and stored in conformity with ASTM C 192 and tested in conformity with ASTM C 39.

C. Slump shall be determined in accordance with ASTM C 143.

1.06 LIMITING REQUIREMENTS

A. Unless otherwise specified, each concrete mix shall be designed and concrete shall be controlled within the following limits:

1. Concrete slump shall be kept as low as possible, consistent with proper handling and thorough compaction. Unless otherwise authorized by the Engineer, slump shall not exceed 5 inches.

2. The admixture content, batching method, and time of introduction to the mix shall be in accordance with the manufacturer's recommendations for minimum shrinkage and for compliance with this Section. A water-reducing admixture may be included in concrete.

PART 2 – PRODUCTS

2.01 PROPORTIONING AND DESIGN MIXES

A. Concrete shall have the following properties.

1. 3,000 pounds per square inch (psi), 28-day compressive strength.

2. Slump range of 1 to 5 inches.

3. Coarse Aggregate Gradation, ASTM C 33, Number 57 or 67.

B. Retarding admixture in proportions recommended by the manufacturer to attain additional working and setting time from 1 to 5 hours.

2.02 CONCRETE MATERIALS

A. Cement shall conform to ASTM C 150 Type II.

B. Water shall be fresh and clean, free from oils, acids, alkalis, salts, organic materials, and other substances deleterious to concrete.

C. Aggregates shall conform to ASTM C 33. Aggregates shall not contain any substance which may be deleteriously reactive with the alkalis in the cement, and shall not possess properties or constituents that are known to have specific unfavorable effects in concrete.

D. The Contractor may use a water reducing chemical admixture. The water reducing admixture shall conform to ASTM C 494, Type A. The chemical admixture shall be approved by the Engineer.

2.03 REINFORCING STEEL

A. The reinforcing steel shall be Grade 60 in accordance with ASTM A 615.

B. Unless otherwise noted on the Drawings, all reinforcement bars shall be No. 3 (3/8-inch diameter) in accordance with ASTM A 615 and welded wire fabric shall be sized as 6 x 6, W1.4 x W1.4.

PART 3 – EXECUTION

3.01 BATCHING, MIXING, AND TRANSPORTING CONCRETE

A. Batching shall be performed according to ASTM C 94, ACI 301, and ACI 304R, except as modified herein. Batching equipment shall be such that the concrete ingredients are consistently measured within the following tolerances: 1 percent for cement and water, 2 percent for aggregate, and 3 percent for admixtures. Concrete Manufacturer shall furnish mandatory batch ticket information for each load of ready mix concrete.

B. Machine mixing shall be performed according to ASTM C 94 and ACI 301. Mixing shall begin within 30 minutes after the cement has been added to the aggregates. Concrete shall be placed within 90 minutes of either addition of mixing water to cement and aggregates or addition of cement to aggregates. Additional water may be added, provided that both the specified maximum slump and water-cement ratio are not exceeded. When additional water is added, an additional 30 revolutions of the mixer at mixing speed is required. Dissolve admixtures in the mixing water and mix in the drum to uniformly distribute the admixture throughout the batch.

C. Transport concrete from the mixer to the forms as rapidly as practicable. Prevent segregation or loss of ingredients. Clean transporting equipment thoroughly before each batch. Do not use aluminum pipe or chutes. Remove concrete which has segregated in transporting and dispose of as directed.

3.02 SUBGRADE PREPARATION

A. Subgrade shall be graded to the lines and elevations as shown on the Drawings.

B. Standing water, mud, debris, and foreign matter shall be removed before concrete is placed.

3.03 PLACING CONCRETE

A. Place concrete in accordance with ACI 301, ACI 318, and ACI 304R. Place concrete as soon as practicable after the forms and the reinforcement have been approved by the CQA Engineer. Do not place concrete when weather conditions prevent proper placement and consolidation, in uncovered areas during periods of precipitation, or in standing water. Prior to placing concrete, remove dirt, construction debris, water, snow, and ice from within the forms. Deposit concrete as close as practicable to the final position in the forms. Place concrete in one continuous operation from one end of the structure towards the other

B. Ensure reinforcement is not disturbed during concrete placement.

C. Do not allow concrete temperature to decrease below 50 °F while curing. Cover concrete and provide sufficient heat to maintain 50 °F minimum adjacent to both the formwork and the structure while curing. Limit the rate of cooling to 5 °F in any 1 hour and 50 °F per 24 hours after heat application.

D. Do not spread concrete with vibrators. Concrete shall be placed in final position without being moved laterally more than 5 feet.

E. When placing of concrete is temporarily halted or delayed, provide construction joints.

F. Concrete shall not be dropped a distance greater than 5 feet.

G. Place concrete with aid of internal mechanical vibrator equipment capable of 9,000 cycles/minute. Transmit vibration directly to concrete.

H. Hot Weather:

1. Comply with ACI 304R.

2. Concrete temperature shall not exceed 90°F.

3. At air temperatures of 80°F or above, keep concrete as cool as possible during placement and curing. Cool forms by water wash.

4. Evaporation reducer shall be used in accordance with manufacturer recommendations (Subpart 2.02).

3.04 CURING AND PROTECTION

A. Immediately after placement, protect concrete from premature drying, excessively hot or cold temperatures, and mechanical injury in accordance with ACI 308.

B. Immediately after placement, protect concrete from plastic shrinkage by applying evaporation reducer in accordance with manufacturer recommendations (Subpart 2.02).

C. Maintain concrete with minimal moisture loss at relatively constant temperature for period necessary for hydration of cement and hardening of concrete (Subpart 2.02).

D. Protect from damaging mechanical disturbances, particularly load stresses, heavy shock, and excessive vibration.

E. Membrane curing compound shall be spray applied at a coverage of not more than 300 square feet per gallon. Unformed surfaces shall be covered with curing compound within 30 minutes after final finishing. If forms are removed before the end of the specified curing period, curing compound shall be immediately applied to the formed surfaces before they dry out.

F. Curing compound shall be suitably protected against abrasion during the curing period.

G. Film curing will not be allowed.

3.05 FORMS

A. Formwork shall prevent leakage of mortar and shall conform to the requirements of ACI 347R.

B. Do not disturb forms until concrete is adequately cured.

C. Form system design shall be the Contractor’s responsibility.

3.06 CONTROL JOINTS

A. Control joints shall consist of plastic strips set flush with finished surface or ¼-inch wide joints formed with a trowel immediately after pouring or cut with a diamond saw within 12 hours after pouring.

B. Control joints shall be installed in a 15 foot by 15 foot grid spacing along the slab unless otherwise approved by the Engineer. Control joints shall be no greater than 1 ½ inches below the surface.

3.07 SLAB FINISHES

A. Unformed surfaces of concrete shall be screeded and given an initial float finish followed by additional floating, and troweling where required.

B. Concrete shall be broom finished.

3.08 SURVEY

A. The Surveyor shall locate the features of the concrete structure. The dimensions, locations and elevations of the features shall be presented on the Surveyor’s Record Drawings.

PART 4 – MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for Cast-In-Place Concrete will be measured as lump sum (LS) and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the work:

- Mobilization.
- Submittals.
- Quality Control.
- Excavation.
- Subgrade preparation.
- Concrete batching, mixing, and delivery.
- Layout and as-built Record Survey.
- Subgrade preparation.
- Reinforcing steel.
- Formwork.
- Concrete placement and finishing.
- Sawcutting and control joints.
- Rejected material removal, handling, re-testing, repair, and replacement.

[END OF SECTION]

EXHIBIT G

REVISED CONSTRUCTION QUALITY ASSURANCE (CQA) PLAN

Prepared for

Denison Mines (USA) Corp.
6425 S. Highway 191
P.O. Box 809
Blanding, UT 84511

CONSTRUCTION QUALITY ASSURANCE PLAN FOR THE CONSTRUCTION OF CELL 4B LINING SYSTEM

WHITE MESA MILL
BLANDING, UTAH

Prepared by

Geosyntec consultants

engineers | scientists | innovators

10875 Rancho Bernardo Rd., Suite 200
San Diego, CA 92127

Project Number SC0349

December 2007
Revised January 2009

CERTIFICATION PAGE

CONSTRUCTION QUALITY ASSURANCE (CQA) PLAN FOR
CELL 4B LINING SYSTEM CONSTRUCTION
DENISON MINES (USA) CORP.
WHITE MESA MILL
BLANDING, UTAH

The Engineering material and data contained in this CQA Plan were prepared under the supervision and direction of the undersigned, whose seal as a registered Professional Engineer is affixed below.

Gregory T. Corcoran, P.E.
Engineer of Record

REGISTERED PROFESSIONAL ENGINEER
No. 6020077-2202
GREGORY T.
CORCORAN
STATE OF UTAH

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TABLES

1. INTRODUCTION

1.1 Terms of Reference

Geosyntec Consultants (Geosyntec) has prepared this Construction Quality Assurance (CQA) Plan for the construction of liner systems associated with the Cell 4B Lining System Construction at the Denison Mines (USA) Corp. (DMC) White Mesa Mill Facility (site), located at 6425 South Highway 191, Blanding, Utah 84511. This CQA Plan was prepared by Ms. Rebecca Flynn, E.I.T., of Geosyntec, and was reviewed by Mr. Gregory T. Corcoran, P.E., also of Geosyntec, in general accordance with the peer review policies of the firm.

1.2 Purpose and Scope of the Construction Quality Assurance Plan

The purpose of the CQA Plan is to address the CQA procedures and monitoring requirements for construction of the project. The CQA Plan is intended to: (i) define the responsibilities of parties involved with the construction; (ii) provide guidance in the proper construction of the major components of the project; (iii) establish testing protocols; (iv) establish guidelines for construction documentation; and (v) provide the means for assuring that the project is constructed in conformance to the *Technical Specifications*, permit conditions, applicable regulatory requirements, and *Construction Drawings*.

This CQA Plan addresses the earthwork and geosynthetic components of the liner system for the project. The earthwork, geosynthetic, and appurtenant components include excavation, fill, prepared subgrade, geosynthetic clay liner (GCL), geomembrane, geotextile, geonet, drainage aggregate, and polyvinyl chloride (PVC) pipe. It should be emphasized that care and documentation are required in the placement of aggregate and in the production and installation of the geosynthetic materials installed during construction. This CQA Plan delineates procedures to be followed for monitoring construction utilizing these materials.

The CQA monitoring activities associated with the selection, evaluation, and placement of drainage aggregate are included in the scope of this plan. The CQA protocols applicable to manufacturing, shipping, handling, and installing all geosynthetic materials are also included. However, this CQA Plan does not specifically address either installation specifications or specification of soils and geosynthetic materials as these requirements are addressed in the *Technical Specifications*.

1.3 References

The CQA Plan includes references to test procedures in the latest editions of the American Society for Testing and Materials (ASTM).

1.4 Organization of the Construction Quality Assurance Plan

The remainder of the CQA Plan is organized as follows:

- Section 2 presents definitions relating to CQA;
- Section 3 describes the CQA personnel and duties;
- Section 4 describes site and project control requirements;
- Section 5 presents CQA documentation;
- Section 6 presents CQA of well abandonment;
- Section 7 presents CQA of earthwork;
- Section 8 presents CQA of the drainage aggregate;
- Section 9 presents CQA of the pipe and fittings;
- Section 10 presents CQA of the geomembrane;
- Section 11 presents CQA of the geotextile;
- Section 12 presents CQA of the geosynthetic clay liner;
- Section 13 presents CQA of the geonet;
- Section 14 presents CQA of the concrete spillway; and
- Section 15 presents CQA surveying.

2. DEFINITIONS RELATING TO CONSTRUCTION QUALITY ASSURANCE

This CQA Plan is devoted to Construction Quality Assurance. In the context of this document, Construction Quality Assurance and Construction Quality Control are defined as follows:

Construction Quality Assurance (CQA) - A planned and systematic pattern of means and actions designed to assure adequate confidence that materials or services meet contractual and regulatory requirements and will perform satisfactorily in service. CQA refers to means and actions employed by the CQA Consultant to assure conformity of the project "Work" with this CQA Plan, the *Construction Drawings*, and the *Technical Specifications*. CQA testing of aggregate, pipe, and geosynthetic components is provided by the CQA Consultant.

Construction Quality Control (CQC) - Actions which provide a means to measure and regulate the characteristics of an item or service in relation to contractual and regulatory requirements. Construction Quality Control refers to those actions taken by the Contractor, Manufacturer, or Geosynthetic Installer to verify that the materials and the workmanship meet the requirements of this CQA Plan, the *Construction Drawings*, and the *Technical Specifications*. In the case of the geosynthetic components and piping of the Work, CQC is provided by the Manufacturer, Geosynthetic Installer, and Contractor.

2.1 Owner

The Owner of this project is Denison Mines (USA) Corp..

2.2 Construction Manager

Responsibilities

The Construction Manager is responsible for managing the construction and implementation of the *Construction Drawings* and *Technical Specifications* for the project work. The Construction Manager is selected/appointed by the Owner.

2.3 Engineer

Responsibilities

The Engineer is responsible for the design, *Construction Drawings*, and *Technical Specifications* for the project work. In this CQA Plan, the term "Engineer" refers to Geosyntec.

Qualifications

The Engineer of Record shall be a qualified engineer, registered as required by regulations in the State of Utah. The Engineer should have expertise, which demonstrates significant familiarity with piping, geosynthetics and soils, as appropriate, including design and construction experience related to liner systems.

2.4 Contractor

Responsibilities

In this CQA Plan, Contractor refers to an independent party or parties, contracted by the Owner, performing the work in accordance with this CQA Plan, the *Construction Drawings*, and the *Technical Specifications*. The Contractor will be responsible for the installation of the soils, pipe, drainage aggregate, and geosynthetic components of the liner systems. This work will include subgrade preparation, anchor trench excavation and backfill, placement of drainage aggregate for the slimes drain and the leak detection system, installation of PVC piping, placement of cast-in-place concrete, and coordination of work with the Geosynthetic Installer and other subcontractors.

The Contractor will be responsible for constructing the liner system and appurtenant components in accordance with the *Construction Drawings* and complying with the quality control requirements specified in the *Technical Specifications*.

Qualifications

Qualifications of the Contractor are specific to the construction contract. The Contractor should have a demonstrated history of successful earthworks, piping, and liner system construction and shall maintain current state and federal licenses as appropriate.

2.5 Resin Supplier

Responsibilities

The Resin Supplier produces and delivers the resin to the Geosynthetics Manufacturer.

Qualifications

Qualifications of the Resin Supplier are specific to the Manufacturer's requirements. The Resin Supplier will have a demonstrated history of providing resin with consistent properties.

2.6 Manufacturers

Responsibilities

The Manufacturers are responsible for the production of finished material (geomembrane, geotextile, geosynthetic clay liner, geonet, and pipe) from appropriate raw materials.

Qualifications

The Manufacturer(s) will be able to provide sufficient production capacity and qualified personnel to meet the demands of the project. The Manufacturer(s) must be a well established firm(s) that meets the requirements identified in the *Technical Specifications*.

2.7 Geosynthetic Installer

Responsibilities

The Geosynthetic Installer is responsible for field handling, storage, placement, seaming, ballasting or anchoring against wind uplift, and other aspects of the geosynthetic material installation. The Geosynthetic Installer may also be responsible for specialized construction tasks (i.e., including construction of anchor trenches for the geosynthetic materials).

Qualifications

The Geosynthetic Installer will be trained and qualified to install the geosynthetic materials of the type specified for this project. The Geosynthetic Installer shall meet the qualification requirements identified in the *Technical Specifications*.

2.8 CQA Consultant

Responsibilities

The CQA Consultant is a party, independent from the Owner, Contractor, Manufacturer, and Geosynthetic Installer, who is responsible for observing, testing, and documenting activities related to the CQC and CQA of the earthwork, piping, and geosynthetic components used in the construction of the Project as required by this CQA Plan and the *Technical Specifications*. The CQA Consultant will also be responsible for issuing a CQA report at the completion of the Project construction, which documents construction and associated CQA activities. The CQA report will be signed and sealed by the CQA Officer who will be a Professional Engineer registered in the State of Utah.

Qualifications

The CQA Consultant shall be a well established firm specializing in geotechnical and geosynthetic engineering that possess the equipment, personnel, and licenses necessary to conduct the geotechnical and geosynthetic tests required by the project plans and *Technical Specifications*. The CQA Consultant will provide qualified staff for the project, as necessary, which will include, at a minimum, a CQA Officer and a CQA Site Manager. The CQA Officer will be a professionally licensed engineer as required by State of Utah regulations.

The CQA Consultant will be experienced with earthwork and installation of geosynthetic materials similar to those materials used in construction of the Project. The CQA Consultant will be experienced in the preparation of CQA documentation including CQA Plans, field documentation, field testing procedures, laboratory testing procedures, construction specifications, construction drawings, and CQA reports.

The CQA Site Manager will be specifically familiar with the construction of earthworks, piping, and geosynthetic lining systems. The CQA Site Manager will be trained by the CQA Consultant in the duties as CQA Site Manager.

2.9 Surveyor

Responsibilities

The Surveyor is a party, independent from the Contractor, Manufacturer, and Geosynthetic Installer, that is responsible for surveying, documenting, and verifying the location of all significant components of the Work. The Surveyor's work is coordinated and employed by the Contractor. The Surveyor is responsible for issuing *Record Drawings* of the construction.

Qualifications

The Surveyor will be a well established surveying company with at least 3 years of surveying experience in the State of Utah. The Surveyor will be a licensed professional as required by the State of Utah regulations. The Surveyor shall be fully equipped and experienced in the use of total stations and the recent version of AutoCAD. All surveying will be performed under the direct supervision of the Contractor.

2.10 CQA Laboratory

Responsibilities

The CQA Laboratory is a party, independent from the Contractor, Manufacturer, and Geosynthetic Installer, that is responsible for conducting tests in accordance with ASTM and other applicable test standards on samples of geosynthetic materials and soil in either an onsite or offsite laboratory.

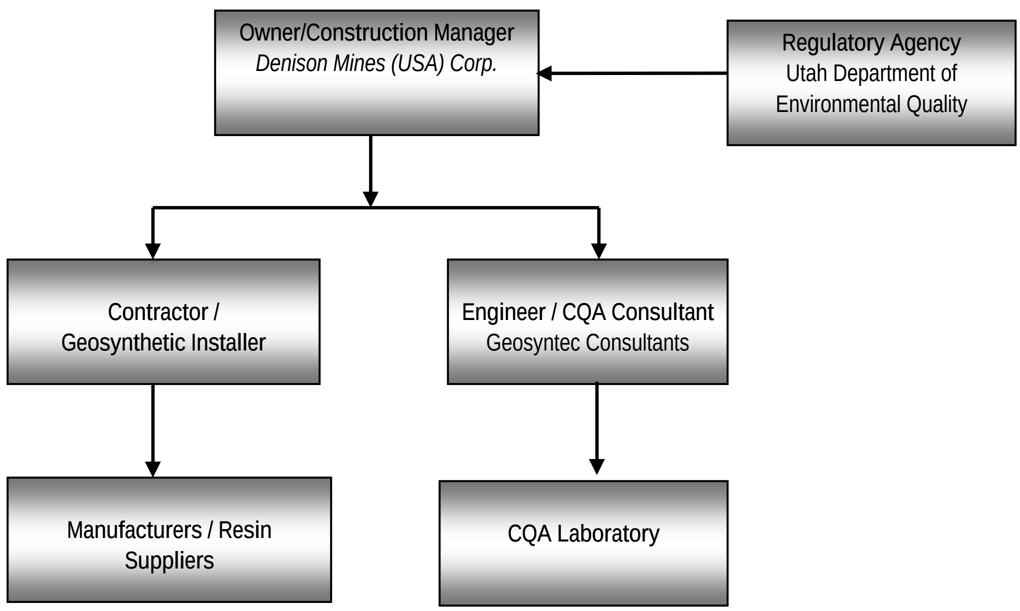
Qualifications

The CQA Laboratory will have experience in testing soils and geosynthetic materials and will be familiar with ASTM and other applicable test standards. The CQA Laboratory will be capable of providing test results within a maximum of seven days of receipt of samples and will maintain that capability throughout the duration of earthworks construction and geosynthetic materials installation. The CQA Laboratory will also be capable of transmitting geosynthetic destructive test results within 24 hours of receipt of samples and will maintain that capability throughout the duration of geosynthetic material installation.

2.11 Lines of Communication

The following organization chart indicates the lines of communication and authority related to this project.

Project Organization Chart
Denison Mines (USA) Corp.
White Mesa Mill Cell 4B



2.12 Deficiency Identification and Rectification

If a defect is discovered in the work, the CQA Engineer will evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Engineer will determine the extent of the deficient area by additional tests, observations, a review of records, or other means that the CQA Engineer deems appropriate.

After evaluating the extent and nature of a defect, the CQA Engineer will notify the Construction Manager and schedule appropriate re-tests when the work deficiency is corrected by the Contractor.

The Contractor will correct the deficiency to the satisfaction of the CQA Engineer. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the CQA Engineer will develop and present to the Design Engineer suggested solutions for approval. Major modification to the *Construction Drawings*, *Technical Specifications*, or this CQA Plan must be provided to the regulatory agency for review prior to implementation.

Defect corrections will be monitored and documented by CQA personnel prior to subsequent work by the Contractor in the area of the deficiency.

3. CQA CONSULTANT'S PERSONNEL AND DUTIES

3.1 Overview

The CQA Officer will provide supervision within the scope of work of the CQA Consultant. The scope of work for the CQA Consultant includes monitoring of construction activities including the following:

- earthwork;
- subgrade preparation;
- installation of geosynthetic clay liner;
- installation of geomembrane;
- installation of geonet;
- installation of drainage aggregate;
- installation of piping; and
- installation of geotextile.

Duties of CQA personnel are discussed in the remainder of this section.

3.2 CQA Personnel

The CQA Consultant's personnel will include:

- the CQA Officer, who works from the office of the CQA Consultant and who conducts periodic visits to the site as required; and
- the CQA Site Manager, who is located at the site.

3.3 CQA Officer

The CQA Officer shall supervise and be responsible for monitoring and CQA activities relating to the construction of the earthworks, piping, and installation of the geosynthetic materials of the Project. Specifically, the CQA Officer:

- reviews the project design, this CQA Plan, *Construction Drawings*, and *Technical Specifications*;

- reviews other site-specific documentation; unless otherwise agreed, such reviews are for familiarization and for evaluation of constructability only, and hence the CQA Officer and the CQA Consultant assume no responsibility for the liner system design;
- reviews and approves the Geosynthetic Installer's Quality Control (QC) Plan;
- attends Pre-Construction Meetings as needed;
- administers the CQA program (i.e., provides supervision of and manages onsite CQA personnel, reviews field reports, and provides engineering review of CQA related activities);
- provides quality control of CQA documentation and conducts site visits;
- reviews the *Record Drawings*; and
- with the CQA Site Manager, prepares the CQA report documenting that the project was constructed in accordance with the Construction Documents.

3.4 CQA Site Manager

The CQA Site Manager:

- acts as the onsite representative of the CQA Consultant;
- attends CQA-related meetings (e.g., pre-construction, daily, weekly (or designates a representative to attend the meetings));
- oversees the ongoing preparation of the *Record Drawings*;
- reviews test results provided by Contractor;
- assigns locations for testing and sampling;
- oversees the collection and shipping of laboratory test samples;
- reviews results of laboratory testing and makes appropriate recommendations;
- reviews the calibration and condition of onsite CQA equipment;
- prepares a daily summary report for the project;
- reviews the Manufacturer's Quality Control (MQC) documentation;
- reviews the Geosynthetic Installer's personnel Qualifications for conformance with those pre-approved for work on site;

- notes onsite activities in daily field reports and reports to the CQA Officer and Construction Manager;
- reports unresolved deviations from the CQA Plan, *Construction Drawings*, and *Technical Specifications* to the Construction Manager; and
- assists with the preparation of the CQA report.

4. SITE AND PROJECT CONTROL

4.1 Project Coordination Meetings

Meetings of key project personnel are necessary to assure a high degree of quality during installation and to promote clear, open channels of communication. Therefore, Project Coordination Meetings are an essential element in the success of the project. Several types of Project Coordination Meetings are described below, including: (i) pre-construction meetings; (ii) progress meetings; and (iii) problem or work deficiency meetings.

4.1.1 Pre-Construction Meeting

A Pre-Construction Meeting will be held at the site prior to construction of the Project. At a minimum, the Pre-Construction Meeting will be attended by the Contractor, the Geosynthetic Installer's Superintendent, the CQA Consultant, and the Construction Manager.

Specific items for discussion at the Pre-Construction Meeting include the following:

- appropriate modifications or clarifications to the CQA Plan;
- the *Construction Drawings* and *Technical Specifications*;
- the responsibilities of each party;
- lines of authority and communication;
- methods for documenting and reporting, and for distributing documents and reports;
- acceptance and rejection criteria;
- protocols for testing;
- protocols for handling deficiencies, repairs, and re-testing;
- the time schedule for all operations;
- procedures for packaging and storing archive samples;
- panel layout and numbering systems for panels and seams;
- seaming procedures;
- repair procedures; and

- soil stockpiling locations.

The Construction Manager will conduct a site tour to observe the current site conditions and to review construction material and equipment storage locations. A person in attendance at the meeting will be appointed by the Construction Manager to record the discussions and decisions of the meeting in the form of meeting minutes. Copies of the meeting minutes will be distributed to all attendees.

4.1.2 Progress Meetings

Progress meetings will be held between the CQA Site Manager, the Contractor, Construction Manager, and other concerned parties participating in the construction of the project. This meeting will include discussions on the current progress of the project, planned activities for the next week, and revisions to the work plan or schedule. The meeting will be documented in meeting minutes prepared by a person designated by the CQA Site Manager at the beginning of the meeting. Within two working days of the meeting, draft minutes will be transmitted to representatives of parties in attendance for review and comment. Corrections or comments to the draft minutes shall be made within two working days of receipt of the draft minutes to be incorporated in the final meeting minutes.

4.1.3 Problem or Work Deficiency Meeting

A special meeting will be held when and if a problem or deficiency is present or likely to occur. The meeting will be attended by the Contractor, the Construction Manager, the CQA Site Manager, and other parties as appropriate. If the problem requires a design modification, the Engineer should either be present at, consulted prior to, or notified immediately upon conclusion of this meeting. The purpose of the work deficiency meeting is to define and resolve the problem or work deficiency as follows:

- define and discuss the problem or deficiency;
- review alternative solutions;
- select a suitable solution agreeable to all parties; and
- implement an action plan to resolve the problem or deficiency.

The Construction Manager will appoint one attendee to record the discussions and decisions of the meeting. The meeting record will be documented in the form of meeting minutes and copies will be distributed to all affected parties. A copy of the minutes will be retained in facility records.

5. DOCUMENTATION

5.1 Overview

An effective CQA Plan depends largely on recognition of all construction activities that should be monitored and on assigning responsibilities for the monitoring of each activity. This is most effectively accomplished and verified by the documentation of quality assurance activities. The CQA Consultant will document that quality assurance requirements have been addressed and satisfied.

The CQA Site Manager will provide the Construction Manager with signed descriptive remarks, data sheets, and logs to verify that monitoring activities have been carried out. The CQA Site Manager will also maintain, at the job site, a complete file of *Construction Drawings* and *Technical Specifications*, a CQA Plan, checklists, test procedures, daily logs, and other pertinent documents.

5.2 Daily Recordkeeping

Preparation of daily CQA documentation will consist of daily field reports prepared by the CQA Site Manager which may include CQA monitoring logs and testing data sheets. This information may be regularly submitted to and reviewed by the Construction Manager. Daily field reports will include documentation of the observed activities during each day of activity. The daily field reports may include monitoring logs and testing data sheets. At a minimum, these logs and data sheets will include the following information:

- the date, project name, location, and other identification;
- a summary of the weather conditions;
- a summary of locations where construction is occurring;
- equipment and personnel on the project;
- a summary of meetings held and attendees;
- a description of materials used and references of results of testing and documentation;
- identification of deficient work and materials;
- results of re-testing corrected “deficient work;”
- an identifying sheet number for cross referencing and document control;

- descriptions and locations of construction monitored;
- type of construction and monitoring performed;
- description of construction procedures and procedures used to evaluate construction;
- a summary of test data and results;
- calibrations or re-calibrations of test equipment and actions taken as a result of re-calibration;
- decisions made regarding acceptance of units of work or corrective actions to be taken in instances of substandard testing results;
- a discussion of agreements made between the interested parties which may affect the work; and
- signature of the respective CQA Site Manager.

5.3 Construction Problems and Resolution Data Sheets

Construction Problems and Resolution Data Sheets, to be submitted with the daily field reports prepared by the CQA Site Manager, describing special construction situations, will be cross-referenced with daily field reports, specific observation logs, and testing data sheets and will include the following information, where available:

- an identifying sheet number for cross-referencing and document control;
- a detailed description of the situation or deficiency;
- the location and probable cause of the situation or deficiency;
- how and when the situation or deficiency was found or located;
- documentation of the response to the situation or deficiency;
- final results of responses;
- measures taken to prevent a similar situation from occurring in the future; and
- signature of the CQA Site Manager and a signature indicating concurrence by the Construction Manager.

The Construction Manager will be made aware of significant recurring nonconformance with the *Construction Drawings*, *Technical Specifications*, or CQA Plan. The cause of the nonconformance will be determined and appropriate changes in procedures or specifications will be recommended. These changes will be submitted to the Construction Manager for approval. When this type of evaluation is made, the results will be documented and any revision to procedures or specifications will be approved by the Contractor and Engineer.

A summary of supporting data sheets, along with final testing results and the CQA Site Manager's approval of the work, will be required upon completion of construction.

5.4 Photographic Documentation

Photographs will be taken and documented in order to serve as a pictorial record of work progress, problems, and mitigation activities. These records will be presented to the Construction Manager upon completion of the project. Photographic reporting data sheets, where used, will be cross-referenced with observation and testing data sheet(s), or Construction Problem and Resolution Data Sheet(s).

5.5 Design or Specifications Changes

Design or specifications changes may be required during construction. In such cases, the CQA Site Manager will notify the Engineer. Design or specification changes will be made with the written agreement of the Engineer and will take the form of an addendum to the *Construction Drawings* and *Technical Specifications*.

5.6 CQA Report

At the completion of the Project, the CQA Consultant will submit to the Owner a CQA report signed and sealed by a Professional Engineer licensed in the State of Utah. The CQA report will acknowledge: (i) that the work has been performed in compliance with the *Construction Drawings* and *Technical Specifications*; (ii) physical sampling and testing has been conducted at the appropriate frequencies; and (iii) that the summary document provides the necessary supporting information. At a minimum, this report will include:

- MQC documentation;
- a summary report describing the CQA activities and indicating compliance with the *Construction Drawings* and *Technical Specifications* which is signed and sealed by the CQA Officer;

- a summary of CQA/CQC testing, including failures, corrective measures, and retest results;
- Contractor and Installer personnel resumes and qualifications as necessary;
- documentation that the geomembrane trial seams were performed in accordance with the CQA Plan and *Technical Specifications*;
- documentation that field seams were non-destructively tested using a method in accordance with the applicable test standards;
- documentation that nondestructive testing was monitored by the CQA Consultant, that the CQA Consultant informed the Geosynthetic Installer of any required repairs, and that the CQA Consultant monitored the seaming and patching operations for uniformity and completeness;
- records of sample locations, the name of the individual conducting the tests, and the results of tests;
- *Record Drawings* as provided by the Surveyor; and
- daily field reports.

The *Record Drawings* will include scale drawings depicting the location of the construction and details pertaining to the extent of construction (e.g., plan dimensions and appropriate elevations). *Record Drawings* and required base maps will be prepared by a qualified Professional Land Surveyor registered in the State of Utah. These documents will be reviewed by the CQA Consultant and included as part of the CQA Report.

6. WELL ABANDONMENT

6.1 Introduction

This section of the CQA Plan outlines the CQA activities to be performed for well abandonment. The CQA Engineer will review and become familiar with the *Construction Documents* and any approved addenda or changes that pertain to work completed under this section.

The CQA Engineer will monitor well abandonment operations. The CQA Engineer will review the contractor's submittals pertaining to CQA and provide recommendations to the Design Engineer. Monitored abandonment activities will be documented, as will deviations from the *Construction Drawings* and the *Technical Specifications*. Any non-conformance identified by the CQA Engineer will be reported to the Construction Manager.

6.2 CQA Monitoring Activities

6.2.1 Materials

CQA activities provided for storing and handling of materials shall meet the requirements set forth in Section 02070 of the *Technical Specifications*.

6.2.2 Well Abandonment

The well to be abandoned is indicated on the *Project Drawings*. Well abandonment shall be observed by the CQA Engineer. Observed well abandonment activities shall be documented in daily field reports. The CQA Engineer shall keep a detailed log for the abandoned well, including drilling procedure, total depth of abandonment, depth to groundwater (if applicable), final depth of boring, and well destruction details, including the depth of placement and quantities of all well abandonment materials.

6.3 Deficiencies

If a defect is discovered in the well abandonment, the CQA Engineer will evaluate the extent and nature of the defect. The CQA Engineer will determine the extent of the deficient area by observations, a review of records, or other means that the CQA Engineer deems appropriate.

6.3.1 Notification

After observing a defect, the CQA Engineer will notify the Construction Manager and schedule appropriate re-evaluation after the work deficiency is corrected by the Contractor.

6.3.2 Repairs and Re-testing

The Contractor will correct the deficiency to the satisfaction of the CQA Engineer. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the CQA Engineer will develop and present to the Design Engineer suggested solutions for approval.

7. EARTHWORK

7.1 Introduction

This section prescribes the CQA activities to be performed to monitor that earthwork is constructed in accordance with *Construction Drawings* and *Technical Specifications*. The earthwork construction procedures to be monitored by the CQA Consultant, if required, shall include:

- vegetation removal;
- subgrade preparation;
- fill placement, moisture conditioning, and compaction; and
- anchor trench excavation and backfill.

7.2 Earthwork Testing Activities

Testing of earthwork to be used for fill, will be performed for material conformance. The CQA Laboratory will perform the conformance testing and CQC testing. Soil testing will be conducted in accordance with the current versions of the corresponding ASTM test procedures. The test methods indicated in Tables 1A and 1B are those that will be used for this testing unless the test methods are updated or revised prior to construction. Revisions to the test methods will be reviewed and approved by the Engineer and the CQA Site Manager prior to their usage.

7.2.1 Sample Frequency

The frequency of subgrade soil testing for material qualification and material conformance will conform to the minimum frequencies presented in Table 1A. The frequency of soil testing shall conform to the minimum frequencies presented in Table 1B. The actual frequency of testing required will be increased by the CQA Site Manager, as necessary, if variability of materials is noted at the site, during adverse conditions, or to isolate failing areas of the construction.

7.2.2 Sample Selection

Sampling locations will be selected by the CQA Site Manager. Conformance samples will be obtained from borrow pits or stockpiles of material. The Contractor must plan the work and make soil available for sampling in a timely and organized manner so that the test results can be obtained before the material is installed. The CQA Site Manager must document sample locations so that failing areas can be immediately isolated. The CQA Site Manager will follow standard sampling procedures to obtain representative samples of the proposed soil materials.

7.3 CQA Monitoring Activities

7.3.1 Vegetation Removal

The CQA Site Manager will monitor and document that vegetation is sufficiently cleared and grubbed in areas where fill is to be placed. Vegetation removal shall be performed as described in the *Technical Specifications* and the *Construction Drawings*.

7.3.2 Fill

During construction, the CQA Site Manager will monitor fill placement and compaction to confirm it is consistent with the requirements specified in the *Technical Specifications* and the *Construction Drawings*. The CQA Site Manager will monitor, at a minimum, that:

- the fill material is free of debris and other undesirable materials and that particles are no larger than 6-inches in longest dimension;
- the fill is constructed to the lines and grades shown on the *Construction Drawings*; and
- fill compaction requirements are met as specified in the *Technical Specifications.*

7.3.3 Subgrade Soil

During construction, the CQA Site Manager will monitor the subgrade soil placement and compaction methods are consistent with the requirements specified in the *Technical Specifications* and the *Construction Drawings*. The CQA Site Manager will monitor, at a minimum, that:

- the subgrade soil is free of protrusions larger than 0.5-inches and particles are to be no larger than 3-inches in longest dimension;
- the subgrade soil is constructed to the lines and grades shown on the *Construction Drawings*; and

- compaction requirements are met as specified in the *Technical Specifications.*

7.3.4 Fine Grading

The CQA Site Manager shall monitor and document that site re-grading performed meets the requirements of the *Technical Specifications* and the *Construction Drawings*. At a minimum, the CQA Site Manager shall monitor that:

- the subgrade surface is free of sharp rocks, debris, and other undesirable materials;
- the subgrade surface is smooth and uniform by visually monitoring proof rolling activities; and
- the subgrade surface meets the lines and grades shown on the *Construction Drawings*.

7.3.5 Anchor Trench Construction

During construction, the CQA Site Manager will monitor the anchor trench excavation and backfill methods are consistent with the requirements specified in the *Technical Specifications* and the *Construction Drawings*. The CQA Site Manager will monitor, at a minimum, that:

- the anchor trench is free of debris and other undesirable materials;
- the anchor trench is constructed to the lines and grades shown on the *Construction Drawings*; and
- compaction requirements are met, through visual observations, as specified in the *Technical Specifications.*

7.4 <u>Deficiencies</u>

If a defect is discovered in the earthwork product, the CQA Site Manager will immediately determine the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Site Manager will determine the extent of the defective area by additional tests, observations, a review of records, or other means that the CQA Site Manager deems appropriate. If the defect is related to adverse site conditions, such as overly wet soils or non-conforming particle sizes, the CQA Site Manager will define the limits and nature of the defect.

7.4.1 Notification

After evaluating the extent and nature of a defect, the CQA Site Manager will notify the Construction Manager and Contractor and schedule appropriate re-evaluation when the work deficiency is to be corrected.

7.4.2 Repairs and Re-Testing

The Contractor will correct deficiencies to the satisfaction of the CQA Site Manager. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the CQA Site Manager will develop and present to the Construction Manager suggested solutions for his approval.

Re-evaluations by the CQA Site Manager shall continue until it is verified that defects have been corrected before any additional work is performed by the Contractor in the area of the deficiency.

8. DRAINAGE AGGREGATE

8.1 Introduction

This section prescribes the CQA activities to be performed to monitor that drainage aggregates are constructed in accordance with *Construction Drawings* and *Technical Specifications*. The drainage aggregates construction procedures to be monitored by the CQA Consultant include drainage aggregate placement.

8.2 Testing Activities

Aggregate testing will be performed for material qualification and material conformance. These two stages of testing are defined as follows:

- Material qualification tests are used to evaluate the conformance of a proposed aggregate source with the *Technical Specifications* for qualification of the source prior to construction.
- Aggregate conformance testing is used to evaluate the conformance of a particular batch of aggregate from a qualified source to the *Technical Specifications* prior to installation of the aggregate.

The Contractor will be responsible for submitting material qualification test results to the Construction Manager and to the CQA Site Manager for review. The CQA Laboratory will perform the conformance testing and CQC testing. Aggregate testing will be conducted in accordance with the current versions of the corresponding ASTM test procedures. The test methods indicated in Tables 2A and 2B are those that will be used for this testing unless the test methods are updated or revised prior to construction. Revisions to the test methods will be reviewed and approved by the Engineer and the CQA Site Manager prior to their usage.

8.2.1 Sample Frequency

The frequency of aggregate testing for material qualification and material conformance will conform to the minimum frequencies presented in Table 2A. The frequency of aggregate testing shall conform to the minimum frequencies presented in Table 2B. The actual frequency of testing required will be increased by the CQA Site Manager, as necessary, if variability of materials is noted at the site, during adverse conditions, or to isolate failing areas of the construction.

8.2.2 Sample Selection

With the exception of qualification samples, sampling locations will be selected by the CQA Site Manager. Conformance samples will be obtained from borrow pits or stockpiles of material. The Contractor must plan the work and make aggregate available for sampling in a timely and organized manner so that the test results can be obtained before the material is installed. The CQA Site Manager must document sample locations so that failing areas can be immediately isolated. The CQA Site Manager will follow standard sampling procedures to obtain representative samples of the proposed aggregate materials.

8.3 CQA Monitoring Activities

8.3.1 Drainage Aggregate

The CQA Site Manager will monitor and document the installation of the drainage aggregates. In general, monitoring of the installation of drainage aggregate includes the following activities:

- reviewing documentation of the material qualification test results provided by the Contractor;
- sampling and testing for conformance of the materials to the *Technical Specifications*;
- documenting that the drainage aggregates are installed using the specified equipment and procedures;
- documenting that the drainage aggregates are constructed to the lines and grades shown on the *Construction Drawings*; and
- monitoring that the construction activities do not cause damage to underlying geosynthetic materials.

8.4 Deficiencies

If a defect is discovered in the drainage aggregates, the CQA Site Manager will evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Site Manager will determine the extent of the deficient area by additional tests, observations, a review of records, or other means that the CQA Site Manager deems appropriate.

8.4.1 Notification

After evaluating the extent and nature of a defect, the CQA Site Manager will notify the Construction Manager and Contractor and schedule appropriate re-tests when the work deficiency is to be corrected.

8.4.2 Repairs and Re-testing

The Contractor will correct the deficiency to the satisfaction of the CQA Site Manager. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the CQA Site Manager will develop and present to the Construction Manager suggested solutions for approval.

Re-tests recommended by the CQA Site Manager shall continue until it is verified that the defect has been corrected before any additional work is performed by the Contractor in the area of the deficiency. The CQA Site Manager will also verify that installation requirements are met and that submittals are provided.

9. POLYVINYL CHLORIDE (PVC) PIPE AND STRIP COMPOSITE

9.1 Material Requirements

PVC pipe, fittings, and strip composite must conform to the requirements of the *Technical Specifications*. The CQA Consultant will document that the PVC pipe, fittings, and strip composite meet those requirements.

9.2 Manufacturer

9.2.1 Submittals

Prior to the installation of PVC pipe and strip composite, the Manufacturer will provide to the CQA Consultant:

- a properties' sheet including, at a minimum, all specified properties, measured using test methods indicated in the *Technical Specifications*, or equivalent; and

The CQA Consultant will document that:

- the property values certified by the Manufacturer meet the *Technical Specifications*; and
- the measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

9.3 Handling and Laying

Care will be taken during transportation of the pipe such that it will not be cut, kinked, or otherwise damaged. Ropes, fabric, or rubber-protected slings and straps will be used when handling pipes. Chains, cables, or hooks inserted into the pipe ends will not be used. Two slings spread apart will be used for lifting each length of pipe. Pipe or fittings will not be dropped onto rocky or unprepared ground.

Pipes will be handled and stored in accordance with the Manufacturer's recommendation. The handling of joined pipe will be in such a manner that the pipe is not damaged by dragging it over sharp and cutting objects. Slings for handling the pipe will not be positioned at joints. Sections of the pipes with deep cuts and gouges will be removed and the ends of the pipe rejoined.

9.4 Perforations

The CQA Site Manager shall monitor and document that the perforations of the PVC pipe conform to the requirements of the *Construction Drawings* and the *Technical Specifications*.

9.5 Joints

The CQA Monitor shall monitor and document that pipe and fittings are joined by the methods indicated in the *Technical Specifications*.

9.6 Strip Composite

The CQA Site Monitor shall monitor and document that the strip composite and sandbags meet and are installed in accordance with the requirements outlined on the drawings and in the *Technical Specifications.*

10. GEOMEMBRANE

10.1 General

This section discusses and outlines the CQA activities to be performed for high density polyethylene (HDPE) geomembrane installation. The CQA Site Manager will review the *Construction Drawings*, *Technical Specifications*, and any approved Addenda regarding this material.

10.2 Geomembrane Material Conformance

10.2.1 Introduction

The CQA Site Manager will document that the geomembrane delivered to the site meets the requirements of the *Technical Specifications* prior to installation. The CQA Site Manager will:

- review the manufacturer's submittals for compliance with the *Technical Specifications*;
- document the delivery and proper storage of geomembrane rolls; and
- conduct conformance testing of the rolls before the geomembrane is installed.

The following sections describe the CQA activities required to verify the conformance of geomembrane.

10.2.2 Review of Quality Control

10.2.2.1 Material Properties Certification

The Manufacturer will provide the Construction Manager and the CQA Site Manager with the following:

- property data sheets, including, at a minimum, all specified properties, measured using test methods indicated in the *Technical Specifications*, or equivalent; and
- sampling procedures and results of testing.

The CQA Site Manager will document that:

- the property values certified by the Manufacturer meet all of the requirements of the *Technical Specifications*; and
- the measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

10.2.2.2 Geomembrane Roll MQC Certification

Prior to shipment, the Manufacturer will provide the Construction Manager and the CQA Site Manager with MQC certificates for every roll of geomembrane provided. The MQC certificates will be signed by a responsible party employed by the Geomembrane Manufacturer, such as the production manager. The MQC certificates shall include:

- roll numbers and identification; and
- results of MQC tests; as a minimum, results will be given for thickness, specific gravity, carbon black content, carbon black dispersion, tensile properties, and puncture resistance evaluated in accordance with the methods indicated in the *Technical Specifications* or equivalent methods approved by the Construction Manager.

  The CQA Site Manager will document that:

- that MQC certificates have been provided at the specified frequency, and that the certificates identify the rolls related to the roll represented by the test results; and
- review the MQC certificates and monitor that the certified roll properties meet the specifications.

10.2.3 Conformance Testing

The CQA Site Manager shall obtain conformance samples (at the manufacturing facility or site) at the specified frequency and forward them to the Geosynthetics CQA Laboratory for testing to monitor conformance to both the *Technical Specifications* and the list of properties certified by the Manufacturer. The test procedures will be as indicated in Table 3. Where optional procedures are noted in the test method, the requirements of the *Technical Specifications* will prevail.

Samples will be taken across the width of the roll and will not include the first linear 3 feet of material. Unless otherwise specified, samples will be 3 feet long by the roll width. The CQA Site Manager will mark the machine direction on the samples with an arrow along with the date and roll number. The required minimum sampling frequencies are provided in Table 3.

The CQA Site Manager will examine results from laboratory conformance testing and will report any non-conformance to the Construction Manager and the Geosynthetic Installer. The procedures prescribed in the *Technical Specifications* will be followed in the event of a failing conformance test.

10.3 Delivery

10.3.1 Transportation and Handling

The CQA Site Manager will document that the transportation and handling does not pose a risk of damage to the geomembrane.

Upon delivery of the rolls of geomembrane, the CQA Site Manager will document that the rolls are unloaded and stored on site as required by the *Technical Specifications*. Damage caused by unloading will be documented by the CQA Site Manager and the damaged material shall not be installed.

10.3.2 Storage

The Geosynthetic Installer will be responsible for the storage of the geomembrane on site. The Contractor will provide storage space in a location (or several locations) such that onsite transportation and handling are optimized, if possible, to limit potential damage.

The CQA Site Manager will document that storage of the geomembrane provides adequate protection against sources of damage.

10.4 Geomembrane Installation

10.4.1 Introduction

The CQA Consultant will document that the geomembrane installation is carried out in accordance with the *Construction Drawings, Technical Specifications,* and Manufacturer's recommendations.

10.4.2 Earthwork

10.4.2.1 Surface Preparation

The CQA Site Manager will document that:

- the prepared subgrade meets the requirements of the *Technical Specifications* and has been approved; and
- placement of the overlying materials does not damage, create large wrinkles, or induce excessive tensile stress in any underlying geosynthetic materials.

The Geosynthetic Installer will certify in writing that the surface on which the geomembrane will be installed is acceptable. The Certificate of Acceptance, as presented in the *Technical Specifications*, will be signed by the Geosynthetic Installer and given to the CQA Site Manager prior to commencement of geomembrane installation in the area under consideration.

After the subgrade has been accepted by the Geosynthetic Installer, it will be the Geosynthetic Installer's responsibility to indicate to the Construction Manager any change in the subgrade soil condition that may require repair work. If the CQA Site Manager concurs with the Geosynthetic Installer, then the CQA Site Manager shall monitor and document that the subgrade soil is repaired before geosynthetic installation begins.

At any time before and during the geomembrane installation, the CQA Site Manager will indicate to the Construction Manager locations that may not provide adequate support to the geomembrane.

10.4.2.2 Geosynthetic Termination

The CQA Site Manager will document that the geosynthetic terminations (Anchor Trench) have been constructed in accordance with the *Construction Drawings*. Backfilling above the terminations will be conducted in accordance with the *Technical Specifications*.

10.4.3 Geomembrane Placement

10.4.3.1 Panel Identification

A field panel is the unit area of geomembrane which is to be seamed in the field, i.e., a field panel is a roll or a portion of roll cut in the field. It will be the responsibility of the CQA Site Manager to document that each field panel is given an "identification code" (number or letter-number) consistent with the Panel Layout Drawing. This identification code will be agreed upon by the Construction Manager, Geosynthetic Installer and CQA Site Manager. This field panel identification code will be as simple and logical as possible. Roll numbers established in the manufacturing plant must be traceable to the field panel identification code.

The CQA Site Manager will establish documentation showing correspondence between roll numbers and field panel identification codes. The field panel identification code will be used for all CQA records.

10.4.3.2 Field Panel Placement

Location

The CQA Site Manager will document that field panels are installed at the location indicated in the Geosynthetic Installer's Panel Layout Drawing, as approved or modified by the Construction Manager.

Installation Schedule

Field panels may be installed using one of the following schedules:

- all field panels are placed prior to field seaming in order to protect the subgrade from erosion by rain;
- field panels are placed one at a time and each field panel is seamed after its placement (in order to minimize the number of unseamed field panels exposed to wind); and

- any combination of the above.

If a decision is reached to place all field panels prior to field seaming, it is usually beneficial to begin at the high point area and proceed toward the low point with "shingle" overlaps to facilitate drainage in the event of precipitation. It is also usually beneficial to proceed in the direction of prevailing winds. Accordingly, an early decision regarding installation scheduling should be made if and only if weather conditions can be predicted with reasonable certainty. Otherwise, scheduling decisions must be made during installation, in accordance with varying conditions. In any event, the Geosynthetic Installer is fully responsible for the decision made regarding placement procedures.

The CQA Site Manager will evaluate every change in the schedule proposed by the Geosynthetic Installer and advise the Construction Manager on the acceptability of that change. The CQA Site Manager will document that the condition of the subgrade soil has not changed detrimentally during installation.

The CQA Site Manager will record the identification code, location, and date of installation of each field panel.

Weather Conditions

Geomembrane placement will not proceed unless otherwise authorized when the ambient temperature is below 40°F or above 122°F. In addition, wind speeds and direction will be monitored for potential impact to geosynthetic installation. Geomembrane placement will not be performed during any precipitation, in the presence of excessive moisture (e.g., fog, dew), or in an area of ponded water.

The CQA Site Manager will document that the above conditions are fulfilled. Additionally, the CQA Site Manager will document that the subgrade soil has not been damaged by weather conditions. The Geosynthetics Installer will inform the Construction Manager if the above conditions are not fulfilled.

Method of Placement

The CQA Site Manager will document the following:

- equipment used does not damage the geomembrane by handling, trafficking, excessive heat, leakage of hydrocarbons or other means;

- the surface underlying the geomembrane has not deteriorated since previous acceptance, and is still acceptable immediately prior to geomembrane placement;
- geosynthetic elements immediately underlying the geomembrane are clean and free of debris;
- personnel working on the geomembrane do not smoke, wear damaging shoes, or engage in other activities which could damage the geomembrane;
- the method used to unroll the panels does not cause scratches or crimps in the geomembrane and does not damage the supporting soil;
- the method used to place the panels minimizes wrinkles (especially differential wrinkles between adjacent panels); and
- adequate temporary loading or anchoring (e.g., sand bags, tires), not likely to damage the geomembrane, has been placed to prevent uplift by wind (in case of high winds, continuous loading, e.g., by adjacent sand bags, is recommended along edges of panels to minimize risk of wind flow under the panels).

The CQA Site Manager will inform the Construction Manager if the above conditions are not fulfilled.

Damaged panels or portions of damaged panels that have been rejected will be marked and their removal from the work area recorded by the CQA Site Manager. Repairs will be made in accordance with procedures described in Section 9.4.5.

10.4.4 Field Seaming

This section details CQA procedures to document that seams are properly constructed and tested in accordance with the Manufacturer's specifications and industry standards.

10.4.4.1 Requirements of Personnel

All personnel performing seaming operations will be qualified by experience or by successfully passing seaming tests, as outlined in the *Technical Specifications*. The most experienced seamer, the "master seamer", will provide direct supervision over less experienced seamers.

The Geosynthetic Installer will provide the Construction Manager and the CQA Site Manager with a list of proposed seaming personnel and their experience records. These documents will be reviewed by the Construction Manager and the Geosynthetics CQA Manager.

10.4.4.2 Seaming Equipment and Products

Approved processes for field seaming are fillet extrusion welding and double-track fusion welding.

Fillet Extrusion Process

The fillet extrusion-welding apparatus will be equipped with gauges giving the temperature in the apparatus.

The Geosynthetic Installer will provide documentation regarding the extrusion welding rod to the CQA Site Manager, and will certify that the extrusion welding rod is compatible with the *Technical Specification*, and in any event, is comprised of the same resin as the geomembrane.

The CQA Site Manager will log apparatus temperatures, ambient temperatures, and geomembrane surface temperatures at appropriate intervals.

The CQA Site Manager will document that:

- the Geosynthetic Installer maintains, on site, the number of spare operable seaming apparatus decided at the Pre-construction Meeting;
- equipment used for seaming is not likely to damage the geomembrane;
- the extruder is purged prior to beginning a seam until all heat-degraded extrudate has been removed from the barrel;
- the electric generator is placed on a smooth base such that no damage occurs to the geomembrane;
- a smooth insulating plate or fabric is placed beneath the hot welding apparatus after usage; and
- the geomembrane is protected from damage in heavily trafficked areas.

Fusion Process

The fusion-welding apparatus must be automated vehicular-mounted devices. The fusion-welding apparatus will be equipped with gauges giving the applicable temperatures and pressures.

The CQA Site Manager will log ambient, seaming apparatus, and geomembrane surface temperatures as well as seaming apparatus speeds.

The CQA Site Manager will also document that:

- the Geosynthetic Installer maintains on site the number of spare operable seaming apparatus decided at the Pre-construction Meeting;
- equipment used for seaming is not likely to damage the geomembrane;
- for cross seams, the edge of the cross seam is ground to a smooth incline (top and bottom) prior to welding;
- the electric generator is placed on a smooth cushioning base such that no damage occurs to the geomembrane from ground pressure or fuel leaks;
- a smooth insulating plate or fabric is placed beneath the hot welding apparatus after usage; and
- the geomembrane is protected from damage in heavily trafficked areas.

10.4.4.3 Seam Preparation

The CQA Site Manager will document that:

- prior to seaming, the seam area is clean and free of moisture, dust, dirt, debris, and foreign material; and
- seams are aligned with the fewest possible number of wrinkles and “fishmouths.”

10.4.4.4 Weather Conditions for Seaming

The normally required weather conditions for seaming are as follows unless authorized in writing by the Engineer:

- seaming will only be approved between ambient temperatures of 40°F and 122°F.

If the Geosynthetic Installer wishes to use methods that may allow seaming at ambient temperatures below 40°F or above 122°F, the Geosynthetic Installer will demonstrate and certify that such methods produce seams which are entirely equivalent to seams produced within acceptable temperature, and that the overall quality of the geomembrane is not adversely affected.

The CQA Site Manager will document that these seaming conditions are fulfilled and will advise the Geosynthetics Installer if they are not.

10.4.4.5 Overlapping and Temporary Bonding

The CQA Site Manager will document that:

- the panels of geomembrane have a finished overlap of a minimum of 3 inches for both extrusion and fusion welding;
- no solvent or adhesive bonding materials are used; and
- the procedures utilized to temporarily bond adjacent panels together does not damage the geomembrane.

The CQA Site Manager will log appropriate temperatures and conditions, and will log and report non-compliances to the Construction Manager.

10.4.4.6 Trial Seams

Trial seams shall be prepared with the procedures and dimensions as indicated in the *Technical Specifications*. The CQA Site Manager will observe trial seam procedures and will document the results of trial seams on trial seam logs. Each trial seam samples will be assigned a number. The CQA Site Manager, will log the date, time, machine temperature(s), seaming unit identification, name of the seamer, and pass or fail description for each trial seam sample tested.

Separate trial seaming logs shall be maintained for fusion welded and extrusion welded trial seams.

10.4.4.7 General Seaming Procedure

Unless otherwise specified, the general production seaming procedure used by the Geosynthetic Installer will be as follows:

- fusion-welded seams are continuous, commencing at one end to the seam and ending at the opposite end;
- cleaning, overlap, and shingling requirements shall be maintained;
- if seaming operations are carried out at night, adequate illumination will be provided at the Geosynthetic Installer's expense; and
- seaming will extend to the outside edge of panels to be placed in the anchor trench.

The CQA Site Manager shall document geomembrane seaming operations on seaming logs. Seaming logs shall include, at a minimum:

- seam identifications (typically associated with panels being joined);
- seam starting time and date;
- seam ending time and date;
- seam length;
- identification of person performing seam; and
- identification of seaming equipment.

Separate logs shall be maintained for fusion and extrusion welded seams. In addition, the CQA Site Manager shall monitor during seaming that:

- fusion-welded seams are continuous, commencing at one end of the seam and ending at the opposite end; and
- cleaning, overlap, and shingling requirements are maintained.

10.4.4.8 Nondestructive Seam Continuity Testing

Concept

The Geosynthetic Installer will non-destructively test field seams over their length using a vacuum test unit, air pressure test (for double fusion seams only), or other method approved by the Construction Manager. The purpose of nondestructive tests is to check the continuity of seams. It does not provide information on seam strength. Continuity testing will be carried out as the seaming work progresses, not at the completion of field seaming.

The CQA Site Manager will:

- observe continuity testing;
- record location, date, name of person conducting the test, and the results of tests; and
- inform the Geosynthetic Installer of required repairs.

The Geosynthetic Installer will complete any required repairs in accordance with Section 10.4.5.

The CQA Site Manager will:

- observe the repair and re-testing of the repair;
- mark on the geomembrane that the repair has been made; and
- document the results.

The following procedures will apply to locations where seams cannot be non-destructively tested:

All such seams will be cap-stripped with the same geomembrane.

- If the seam is accessible to testing equipment prior to final installation, the seam will be non-destructively tested prior to final installation.
- If the seam cannot be tested prior to final installation, the seaming and cap-stripping operations will be observed by the CQA Site Manager and Geosynthetic Installer for uniformity and completeness.

The seam number, date of observation, name of tester, and outcome of the test or observation will be recorded by the CQA Site Manager.

Vacuum Testing

Vacuum testing shall be performed utilizing the equipment and procedures specified in the Technical Specifications. The CQA Site Manager shall observe the vacuum testing procedures and document that they are performed in accordance with the *Technical Specifications*. The result of vacuum testing shall be recorded on the CQA seaming logs. Results shall include, at a minimum, the personnel performing the vacuum test and the result of the test (pass or fail), and the test date. Seams failing the vacuum test shall be repaired in accordance with the procedures listed in the *Technical Specifications*. The CQA Site Manager shall document seam repairs in the seaming logs.

Air Pressure Testing

Air channel pressure testing shall be performed on double-track seams created with a fusion welding device, utilizing the equipment and procedures specified in the *Technical Specifications*. The CQA Site Manager shall observe the vacuum testing procedures and document that they are performed in accordance with the *Technical Specifications*. The result of air channel pressure testing shall be recorded on the CQA seaming logs. Results shall include, at a minimum, personnel performing the air pressure test, the starting air pressure and time, the final air pressure and time, the drop in psi during the test, and the result of the test (pass or fail). Seams failing the air pressure test shall be repaired in accordance with the procedures listed in the *Technical Specifications*. The CQA Site Manager shall document seam repairs in the seaming logs.

10.4.4.9 Destructive Testing

Concept

Destructive seam testing will be performed on site and at the independent CQA laboratory in accordance with the *Construction Drawings* and the *Technical Specifications*. Destructive seam tests will be performed at selected locations. The purpose of these tests is to evaluate seam strength. Seam strength testing will be done as the seaming work progresses, not at the completion of all field seaming.

Location and Frequency

The CQA Site Manager will select locations where seam samples will be cut out for laboratory testing. Those locations will be established as follows.

- The frequency of geomembrane seam testing is a minimum of one destructive sample per 500 feet of weld. If after a total of 50 samples have been tested and no more than one sample has failed, the frequency can be increased to one per 1,000 feet.
- A minimum of one test per seaming machine over the duration of the project.
- Additional test locations may be selected during seaming at the CQA Site Manager's discretion. Selection of such locations may be prompted by suspicion of excess crystallinity, contamination, offset welds, or any other potential cause of imperfect welding.

The Geosynthetic Installer will not be informed in advance of the locations where the seam samples will be taken.

Sampling Procedure

Samples will be marked by the CQA Site Manager following the procedures listed in the *Technical Specifications*. Preliminary samples will be taken from either side of the marked sample and tested before obtaining the full sample per the requirements of the *Technical Specifications*. Samples shall be obtained by the Geosynthetic Installer. Samples shall be obtained as the seaming progresses in order to have laboratory test results before the geomembrane is covered by another material. The CQA Site Manager will:

- observe sample cutting and monitor that corners are rounded;
- assign a number to each sample, and mark it accordingly;
- record sample location on the Panel Layout Drawing; and
- record reason for taking the sample at this location (e.g., statistical routine, suspicious feature of the geomembrane).

Holes in the geomembrane resulting from destructive seam sampling will be immediately repaired in accordance with repair procedures described in Section 10.4.5. The continuity of the new seams in the repaired area will be tested in accordance with Section 10.4.4.8.

Size and Distribution of Samples

The destructive sample will be 12 inches (0.3 meters) wide by 42 inches (1.1 meters) long with the seam centered lengthwise. The sample will be cut into three parts and distributed as follows:

- one portion, measuring 12 inches by 12 inches (30 centimeters (cm) by 30 cm), to the Geosynthetic Installer for field testing;
- one portion, measuring 12 inches by 18 inches (30 cm by 45 cm), for CQA Laboratory testing; and
- one portion, measuring 12 inches by 12 inches (30 cm by 30 cm), to the Construction Manager for archive storage.

Final evaluation of the destructive sample sizes and distribution will be made at the Pre-Construction Meeting.

Field Testing

Field testing will be performed by the Geosynthetic Installer using a gauged tensiometer. Prior to field testing the Geosynthetic Installer shall submit a calibration certificate for gauge tensiometer to the CQA Consultant for review. Calibration must have been performed within one year of use on the current project. The destructive sample shall be tested according to the requirements of the *Technical Specifications*. The specimens shall not fail in the seam and shall meet the strength requirements outlined in the *Technical Specifications*. If any field test specimen fails, then the procedures outlined in *Procedures for Destructive Test Failures* of this section will be followed.

The CQA Site Manager will witness field tests and mark samples and portions with their number. The CQA Site Manager will also document the date and time, ambient temperature, number of seaming unit, name of seamer, welding apparatus temperatures and pressures, and pass or fail description.

CQA Laboratory Testing

Destructive test samples will be packaged and shipped, if necessary, under the responsibility of the CQA Site Manager in a manner that will not damage the test sample. The Construction Manager will be responsible for storing the archive samples.

This procedure will be outlined at the Pre-construction Meeting. Samples will be tested by the CQA Laboratory. The CQA Laboratory will be selected by the CQA Site Manager with the concurrence of the Engineer.

Testing will include "Bonded Seam Strength" and "Peel Adhesion." The minimum acceptable values to be obtained in these tests are given in the *Technical Specifications*. At least five specimens will be tested for each test method. Specimens will be selected alternately, by test, from the samples (i.e., peel, shear, peel, shear, and so on). A passing test will meet the minimum required values in at least four out of five specimens.

The CQA Laboratory will provide test results no more than 24 hours after they receive the samples. The CQA Site Manager will review laboratory test results as soon as they become available, and make appropriate recommendations to the Construction Manager.

Geosynthetic Installer's Laboratory Testing

The Geosynthetic Installer's laboratory test results will be presented to the Construction Manager and the CQA Site Manager for comments.

Procedures for Destructive Test Failure

The following procedures will apply whenever a sample fails a destructive test, whether that test conducted by the CQA Laboratory, the Geosynthetic Installer's laboratory, or by gauged tensiometer in the field. The Geosynthetic Installer has two options:

- The Geosynthetic Installer can reconstruct the seam between two passed test locations.
- The Geosynthetic Installer can trace the welding path to an intermediate location at 10 feet (3 meters) minimum from the point of the failed test in each direction and take a small sample for an additional field test at each location. If these additional samples pass the test, then full laboratory samples are taken. If these laboratory samples pass the tests, then the seam is reconstructed between these locations. If either sample fails, then the process is repeated to establish the zone in which the seam should be reconstructed.

Acceptable seams must be bounded by two locations from which samples passing laboratory destructive tests have been taken. Repairs will be made in accordance with Section 10.4.5.

The CQA Site Manager will document actions taken in conjunction with destructive test failures.

10.4.5 Defects and Repairs

This section prescribes CQA activities to document that defects, tears, rips, punctures, damage, or failing seams shall be repaired.

10.4.5.1 Identification

Seams and non-seam areas of the geomembrane shall be examined by the CQA Site Manager for identification of defects, holes, blisters, undispersed raw materials and signs of contamination by foreign matter. Because light reflected by the geomembrane helps to detect defects, the surface of the geomembrane shall be clean at the time of examination.

10.4.5.2 Evaluation

Potentially flawed locations, both in seam and non-seam areas, shall be non-destructively tested using the methods described in Section 10.4.4.8 as appropriate. Each location that fails the nondestructive testing will be marked by the CQA Site Manager and repaired by the Geosynthetic Installer. Work will not proceed with any materials that will cover locations which have been repaired until laboratory test results with passing values are available.

10.4.5.3 Repair Procedures

Portions of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, will be repaired. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure will be at the discretion of the CQA Consultant with input from the Construction Manager and Geosynthetic Installer. The procedures available include:

- patching, used to repair large holes, tears, undispersed raw materials, and contamination by foreign matter;

- grinding and re-welding, used to repair small sections of extruded seams;
- spot welding or seaming, used to repair small tears, pinholes, or other minor, localized flaws;
- capping, used to repair large lengths of failed seams; and
- removing a bad seam and replacing with a strip of new material welded into place (used with large lengths of fusion seams).

In addition, the following provisions will be satisfied:

- surfaces of the geomembrane which are to be repaired will be abraded no more than 20 minutes prior to the repair;
- surfaces must be clean and dry at the time of the repair;
- all seaming equipment used in repairing procedures must be approved;
- the repair procedures, materials, and techniques will be approved in advance by the CQA Consultant with input from the Engineer and Geosynthetic Installer;
- patches or caps will extend at least 6 inches (150 millimeters (mm)) beyond the edge of the defect, and all corners of patches will be rounded with a radius of at least 3 inches (75 mm);
- cuts and holes to be patched shall have rounded corners; and
- the geomembrane below large caps should be appropriately cut to avoid water or gas collection between the two sheets.

10.4.5.4 Verification of Repairs

The CQA Monitor shall monitor and document repairs. Records of repairs shall be maintained on repair logs. Repair logs shall include, at a minimum:

- panel containing repair and approximate location on panel;
- approximate dimensions of repair;
- repair type, i.e. fusion weld or extrusion weld
- date of repair;
- seamer making the repair; and

- results of repair non-destructive testing (pass or fail).

Each repair will be non-destructively tested using the methods described herein, as appropriate. Repairs that pass the non-destructive test will be taken as an indication of an adequate repair. Large caps may be of sufficient extent to require destructive test sampling, per the requirements of the *Technical Specifications*. Failed tests shall be redone and re-tested until passing test results are observed.

10.4.5.5 Large Wrinkles

When seaming of the geomembrane is completed (or when seaming of a large area of the geomembrane liner is completed) and prior to placing overlying materials, the CQA Site Manager will observe the geomembrane wrinkles. The CQA Site Manager will indicate to the Geosynthetic Installer which wrinkles should be cut and re-seamed. The seam thus produced will be tested like any other seam.

10.4.6 Lining System Acceptance

The Geosynthetic Installer and the Manufacturer(s) will retain all responsibility for the geosynthetic materials in the liner system until acceptance by the Construction Manager.

The geosynthetic liner system will be accepted by the Construction Manager when:

- the installation is finished;
- verification of the adequacy of all seams and repairs, including associated testing, is complete;
- all documentation of installation is completed including the CQA Site Manager’s acceptance report and appropriate warranties; and
- CQA report, including “as built” drawing(s), sealed by a registered professional engineer has been received by the Construction Manager.

The CQA Site Manager will document that installation proceeded in accordance with the *Technical Specifications* for the project.

11. GEOTEXTILE

11.1 Introduction

This section of the CQA Plan outlines the CQA activities to be performed for the geotextile installation. The CQA Consultant will review the *Construction Drawings*, and the *Technical Specifications,* and any approved addenda or changes.

11.2 Manufacturing

The Manufacturer will provide the Construction Manager with a list of guaranteed "minimum average roll value" properties (defined as the mean less two standard deviations), for each type of geotextile to be delivered. The Manufacturer will also provide the Construction Manager with a written quality control certification signed by a responsible party employed by the Manufacturer that the materials actually delivered have property "minimum average roll values" which meet or exceed all property values guaranteed for that type of geotextile.

The quality control certificates will include:

- roll identification numbers; and
- results of MQC testing.

The Manufacturer will provide, as a minimum, test results for the following:

- mass per unit area;
- grab strength;
- tear strength;
- puncture strength;
- permittivity; and
- apparent opening size.

MQC tests shall be performed at the frequency listed in the *Technical Specifications*. CQA tests on geotextile produced for the project shall be performed according to the test methods specified and frequencies presented in Table 4.

The CQA Site Manager will examine Manufacturer certifications to evaluate that the property values listed on the certifications meet or exceed those specified for the particular type of geotextile and the measurements of properties by the Manufacturer are properly documented, test methods acceptable and the certificates have been provided at the specified frequency properly identifying the rolls related to testing. Deviations will be reported to the Construction Manager.

11.3 Labeling

The Manufacturer will identify all rolls of geotextile with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

The CQA Site Manager will examine rolls upon delivery and deviation from the above requirements will be reported to the Construction Manager.

11.4 Shipment and Storage

During shipment and storage, the geotextile will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting, or any other damaging or deleterious conditions. To that effect, geotextile rolls will be shipped and stored in relatively opaque and watertight wrappings.

Protective wrappings will be removed less than one hour prior to unrolling the geotextile. After the wrapping has been removed, a geotextile will not be exposed to sunlight for more than 15 days, except for UV protection geotextile, unless otherwise specified and guaranteed by the Manufacturer.

The CQA Site Manager will observe rolls upon delivery at the site and deviation from the above requirements will be reported to the Geosynthetic Installer.

11.5 Conformance Testing

11.5.1 Tests

Upon delivery of the rolls of geotextiles, the CQA Site Manager will obtain conformance samples and forward to the Geosynthetics CQA Laboratory for testing to evaluate conformance to *Technical Specifications*. Required test and testing frequency for the geotextiles are presented in Table 4. These conformance tests will be performed in accordance with the test methods specified in the *Technical Specifications* and will be documented by the CQA Site Manager.

11.5.2 Sampling Procedures

Samples will be taken across the width of the roll and will not include the first 3 feet. Unless otherwise specified, samples will be 3 feet long by the roll width. The CQA Site Manager will mark the machine direction on the samples with an arrow.

Unless otherwise specified, samples will be taken at a rate as indicated in Table 4 for geotextiles.

11.5.3 Test Results

The CQA Site Manager will examine results from laboratory conformance testing and will report non-conformance with the *Technical Specifications* and this CQA Plan to the Construction Manager.

11.5.4 Conformance Sample Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of geotextile that is in nonconformance with the *Technical Specifications* with a roll(s) that meets *Technical Specifications*; or
- The Geosynthetic Installer will remove conformance samples for testing by the CQA Laboratory from the closest numerical rolls on both sides of the failed roll. These two samples must conform to the *Technical Specifications*. If either of these samples fail, the numerically closest rolls on the side of the failed sample will be tested by the CQA Laboratory. These samples must conform to the *Technical Specifications*. If any of these

samples fail, every roll of geotextile on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*. This additional conformance testing will be at the expense of the Manufacturer.

The CQA Site Manager will document actions taken in conjunction with conformance test failures.

11.6 Handling and Placement

The Geosynthetic Installer will handle all geotextiles in such a manner as to document they are not damaged in any way, and the following will be complied with:

- In the presence of wind, all geotextiles will be weighted with sandbags or the equivalent. Such sandbags will be installed during placement and will remain until replaced with earth cover material.
- Geotextiles will be cut using an approved geotextile cutter only. If in place, special care must be taken to protect other materials from damage, which could be caused by the cutting of the geotextiles.
- The Geosynthetic Installer will take all necessary precautions to prevent damage to underlying layers during placement of the geotextile.
- During placement of geotextiles, care will be taken not to entrap in the geotextile stones, excessive dust, or moisture that could damage the geotextile, generate clogging of drains or filters, or hamper subsequent seaming.
- A visual examination of the geotextile will be carried out over the entire surface, after installation, to document that no potentially harmful foreign objects, such as needles, are present.

The CQA Site Manager will note non-compliance and report it to the Construction Manager.

11.7 Seams and Overlaps

All geotextiles will be continuously sewn in accordance with *Technical Specifications*. Geotextiles will be overlapped 12 inches prior to seaming. No horizontal seams will be allowed on side slopes (i.e. seams will be along, not across, the slope), except as part of a patch.

Sewing will be done using polymeric thread with chemical and ultraviolet resistance properties equal to or exceeding those of the geotextile.

11.8 Repair

Holes or tears in the geotextile will be repaired as follows:

- On slopes: A patch made from the same geotextile will be double seamed into place. Should a tear exceed 10 percent of the width of the roll, that roll will be removed from the slope and replaced.
- Non-slopes: A patch made from the same geotextile will be spot-seamed in place with a minimum of 6 inches (0.60 meters) overlap in all directions.

Care will be taken to remove any soil or other material that may have penetrated the torn geotextile.

The CQA Site Manager will observe any repair, note any non-compliance with the above requirements and report them to the Construction Manager.

11.9 Placement of Soil or Aggregate Materials

The Contractor will place all soil or aggregate materials located on top of a geotextile, in such a manner as to document:

- no damage of the geotextile;
- minimal slippage of the geotextile on underlying layers; and
- no excess tensile stresses in the geotextile.

Non-compliance will be noted by the CQA Site Manager and reported to the Construction Manager.

12. GEOSYNTHETIC CLAY LINER (GCL)

12.1 Introduction

This section of the CQA Plan outlines the CQA activities to be performed for the geosynthetic clay liner (GCL) installation. The CQA Consultant will review the *Construction Drawings*, *Technical Specifications*, and approved addenda or changes.

12.2 Manufacturing

The Manufacturer will provide the Construction Manager with a list of guaranteed "minimum average roll value" properties (defined as the mean less two standard deviations), for the GCL to be delivered. The Manufacturer will also provide the Construction Manager with a written quality control certification signed by a responsible party employed by the Manufacturer that the materials actually delivered have property "minimum average roll values" which meet or exceed all property values guaranteed for that GCL.

The quality control certificates will include:

- roll identification numbers; and
- results of quality control testing.

The Manufacturer will provide, as a minimum, test results for the following:

- mass per unit area (bentonite content); and
- index flux.

Quality control tests must be performed, in accordance with the test methods specified in Table 5, on GCL produced for the project.

The CQA Site Manager will examine Manufacturer certifications to verify that the property values listed on the certifications meet or exceed those specified for the GCL and the measurements of properties by the Manufacturer are properly documented, test methods acceptable and the certificates have been provided at the specified frequency properly identifying the rolls related to testing. Deviations will be reported to the Construction Manager.

12.3 Labeling

The Manufacturer will identify all rolls of GCL with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

The CQA Site Manager will examine rolls upon delivery and deviation from the above requirements will be reported to the Construction Manager.

12.4 Shipment and Storage

During shipment and storage, the GCL will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, and cutting or any other damaging or deleterious conditions. To that effect, GCL rolls will be shipped and stored in relatively opaque and watertight wrappings.

The CQA Site Manager will observe rolls upon delivery at the site and any deviation from the above requirements will be reported to the Construction Manager.

12.5 Conformance Testing

12.5.1 Tests

CQA personnel will sample the GCL either during production at the manufacturing facility or after delivery to the construction site. The samples will be forwarded to the Geosynthetics CQA Laboratory for testing to assess conformance with the *Technical Specifications*. The test methods and minimum testing frequencies are indicated in Table 5.

Samples will be taken across the width of the roll and will not include the first 3 ft if the sample is cut on site. Unless otherwise specified, samples will be 3 ft long by the roll width. The CQA Consultant will mark the machine direction with an arrow and the manufacturer's roll number on each sample.

The CQA Site Manager will examine results from laboratory conformance testing and will report non-conformance to the Construction Manager.

12.5.2 Conformance Sample Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of GCL that is in nonconformance with the *Technical Specifications* with a roll(s) that meets *Technical Specifications*; or
- The Geosynthetic Installer will remove conformance samples for testing by the CQA Laboratory from the closest numerical rolls on both sides of the failed roll. These two samples must conform to the *Technical Specifications*. If either of these samples fail, the numerically closest rolls on the side of the failed sample will be tested by the CQA Laboratory. These samples must conform to the *Technical Specifications*. If any of these samples fail, every roll of GCL on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*. This additional conformance testing will be at the expense of the Manufacturer.

The CQA Site Manager will document actions taken in conjunction with conformance test failures.

12.6 <u>GCL Delivery and Storage</u>

Upon delivery to the site, the CQA Consultant will check the GCL rolls for defects (e.g., tears, holes) and for damage. The CQA Consultant will report to the Construction Manager and the Geosynthetics Installer:

- any rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and
- any rolls which include minor repairable flaws.

The GCL rolls delivered to the site will be checked by the CQA Consultant to document that the roll numbers correspond to those on the approved Manufacturer's quality control certificate of compliance.

12.7 GCL Installation

The CQA Consultant will monitor and document that the GCL is installed in accordance with the *Drawings* and the *Technical Specifications*. The Geosynthetics Installer shall provide the CQA Consultant a certificate of subgrade acceptance prior to the installation of the GCL as outlined in the *Technical Specifications*. The GCL installation activities to be monitored and documented by the CQA Consultant include:

- monitoring that the GCL rolls are stored and handled in a manner which does not result in any damage to the GCL;
- monitoring that the GCL is not exposed to UV radiation for extended periods of time without prior approval;
- monitoring that the GCL are seamed in accordance with the *Technical Specifications* and the Manufacturer's recommendations;
- monitoring and documenting that the GCL is installed on an approved subgrade, free of debris, protrusions, or uneven surfaces; and
- monitoring that any damage to the GCL is repaired as outlined in the *Technical Specifications*.

The CQA Site Manager will note non-compliance and report it to the Construction Manager.

13. GEONET

13.1 Introduction

This section of the CQA Plan outlines the CQA activities to be performed for the geonet installation. The CQA Consultant will review the *Construction Drawings*, *Technical Specifications*, and any approved addenda or changes.

13.2 Manufacturing

The Manufacturer will provide the CQA Consultant with a list of certified "minimum average roll value" properties for the type of geonet to be delivered. The Manufacturer will also provide the CQA Consultant with a written certification signed by a responsible representative of the Manufacturer that the geonet actually delivered have "minimum average roll values" properties which meet or exceed all certified property values for that type of geonet.

The CQA Consultant will examine the Manufacturers' certifications to document that the property values listed on the certifications meet or exceed those specified for the particular type of geonet. Deviations will be reported to the Construction Manager.

13.3 Labeling

The Manufacturer will identify all rolls of geonet with the following:

- Manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

The CQA Site Manager will examine rolls upon delivery and deviation from the above requirements will be reported to the Construction Manager.

13.4 Shipment and Storage

During shipment and storage, the geonet will be protected from mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions. The CQA Site Manager will observe rolls upon delivery to the site and deviation from the above requirements will be reported to the Construction Manager. Damaged rolls will be rejected and replaced.

The CQA Site Manager will observe that geonet is free of dirt and dust just before installation. The CQA Site Manager will report the outcome of this observation to the Construction Manager, and if the geonet is judged dirty or dusty, they will be cleaned by the Geosynthetic Installer prior to installation.

13.5 Conformance Testing

13.5.1 Tests

The geonet material will be tested for transmissivity (ASTM D 4716) and for thickness (ASTM D 5199) at the frequencies presented in Table 6.

13.5.2 Sampling Procedures

Upon delivery of the geonet rolls, the CQA Site Manager will document that samples are obtained from individual rolls at the frequency specified in this CQA Plan. The geonet samples will be forwarded to the CQA Laboratory for testing to evaluate conformance to both the *Technical Specifications* and the list of physical properties certified by the Manufacturer.

Samples will be taken across the width of the roll and will not include the first 3 linear feet. Unless otherwise specified, samples will be 3 feet long by the roll width. The CQA Consultant will mark the machine direction on the samples with an arrow.

13.5.3 Test Results

The CQA Site Manager will examine results from laboratory conformance testing and compare results to the *Technical Specifications*. The criteria used to evaluate acceptability are presented in the *Technical Specifications*. The CQA Site Manager will report any nonconformance to the Construction Manager.

13.5.4 Conformance Test Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of geonet that is in nonconformance with the *Technical Specifications* with a roll that meets specifications; or
- The Geosynthetic Installer will remove conformance samples for testing by the CQA Laboratory from the closest numerical rolls on both sides of the failed roll. These two samples must conform to the *Technical Specifications*. If either of these samples fail, the numerically closest rolls on the side of the failed sample that is not tested, will be tested by the CQA Laboratory. These samples must conform to the *Technical Specifications*. If any of these samples fail, every roll of geonet on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*.

The CQA Site Manager will document actions taken in conjunction with conformance test failures.

13.6 Handling and Placement

The Geosynthetic Installer will handle all geonet in such a manner as to document they are not damaged in any way. The Geosynthetic Installer will comply with the following:

- If in place, special care must be taken to protect other materials from damage, which could be caused by the cutting of the geonet.
- The Geosynthetic Installer will take any necessary precautions to prevent damage to underlying layers during placement of the geonet.
- During placement of geonet, care will be taken to prevent entrapment of dirt or excessive dust that could cause clogging of the drainage system, or stones that could damage the adjacent geomembrane. If dirt or excessive dust is entrapped in the geonet, it should be cleaned prior to placement of the next material on top of it. In this regard, care should be taken with the handling or sandbags, to prevent rupture or damage of the sandbag.

- A visual examination of the geonet will be carried out over the entire surface, after installation to document that no potentially harmful foreign objects are present.

The CQA Site Manager will note noncompliance and report it to the Construction Manager.

13.7 Geonet Seams and Overlaps

Adjacent geonet panels will be joined in accordance with *Construction Drawings* and *Technical Specifications*. As a minimum, the adjacent rolls will be overlapped by at least 4 inches and secured by tying, in accordance with the *Technical Specifications*.

The CQA Consultant will note any noncompliance and report it to the Construction Manager.

13.8 Repair

Holes or tears in the geonet will be repaired by placing a patch extending 2 feet beyond edges of the hole or tear. The patch will be secured by tying with approved tying devices every 6 inches If the hole or tear width across the roll is more than 50 percent of the width of the roll, the damaged area will be cut out and the two portions of the geonet will be joined in accordance with Section 13.7.

The CQA Site Manager will observe repairs, note non-compliances with the above requirements and report them to the Construction Manager.

14. CONCRETE SPILLWAY

14.1 Introduction

This section prescribes the CQA activities to be performed to monitor that the concrete spillway is constructed in accordance with *Construction Drawings* and *Technical Specifications*. The concrete spillway construction procedures to be monitored by the CQA Consultant, if required, shall include:

- subgrade preparation;
- liner system and cushion geotextile installation;
- welded wire reinforcement installation; and
- concrete placement and finishing.

14.2 CQA Monitoring Activities

14.2.1 Subgrade Preparation

The CQA Site Manager will monitor and document that the subgrade is prepared in accordance with the *Technical Specifications* and the *Construction Drawings*.

14.2.2 Liner System and Cushion Geotextile Installation

The CQA Site Manager shall monitor and document that the liner system components, along with the anchor trench and cushion geotextile, are installed in accordance with the requirements of the *Technical Specifications* and the *Construction Drawings*.

14.2.3 Welded Wire Reinforcement Installation

The CQA Site Manager shall monitor and document that the welded wire fabric reinforcement is installed in accordance with the requirements of the *Technical Specifications* and the *Construction Drawings*.

14.2.4 Concrete Installation

The CQA Site Manager shall test, monitor, and document that the concrete is installed in accordance with the requirements of the *Technical Specifications* and the *Construction Drawings*. At a minimum, the CQA Site Manager shall review the concrete tickets prior to installing the concrete to monitor that the concrete meets the requirements outlined in the *Technical Specifications*.

14.2.5 Conformance Testing

The Contractor shall facilitate the CQA Site Manager in the collection of samples required for testing. Compression test specimens shall be prepared by the CQA Site Manager by the following method:

- compression test cylinders from fresh concrete in accordance with ASTM C 172 and C 31.

Compression testing shall be completed on one cylinder at 7 days, one cylinder at 14 days, and two (2) cylinders at the 28 day strength. The CQA Site Manager will examine results from laboratory conformance testing and will report any non-conformance with the requirements outlined in the *Technical Specifications* to the Construction Manager.

14.3 Deficiencies

If a defect is discovered in the concrete spillway, the CQA Site Manager will immediately determine the extent and nature of the defect. The CQA Site Manager will determine the extent of the defective area by additional observations, a review of records, or other means that the CQA Site Manager deems appropriate.

14.3.1 Notification

After evaluating the extent and nature of a defect, the CQA Site Manager will notify the Construction Manager and Contractor and schedule appropriate re-evaluation when the work deficiency is to be corrected.

14.3.2 Repairs

The Contractor will correct deficiencies to the satisfaction of the CQA Site Manager. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the CQA Site Manager will develop and present to the Construction Manager suggested solutions for his approval.

Re-evaluations by the CQA Site Manager shall continue until the defects have been corrected before any additional work is performed by the Contractor in the area of the deficiency.

15. SURVEYING

15.1 Survey Control

Survey control will be performed by the Surveyor as needed. A permanent benchmark will be established for the site(s) in a location convenient for daily tie-in. The vertical and horizontal control for this benchmark will be established within normal land surveying standards.

15.2 Precision and Accuracy

A wide variety of survey equipment is available for the surveying requirements for these projects. The survey instruments used for this work should be sufficiently precise and accurate to meet the needs of the projects.

15.3 Lines and Grades

The following structures will be surveyed to verify and document the lines and grades achieved during construction of the Project:

- geomembrane terminations; and
- centerlines of pipes.

15.4 Frequency and Spacing

A line of survey points no further than 100 feet apart must be taken at the top of pipes or other appurtenances to the liner.

15.5 Documentation

Field survey notes should be retained by the Land Surveyor. The findings from the field surveys should be documented on a set of Survey *Record Drawings*, which shall be provided to the Construction Manager in AutoCAD 2000 format or other suitable format as directed by the Construction Manager.

TABLE 1A

TEST PROCEDURES FOR THE EVALUATION OF EARTHWORK

TEST METHOD	DESCRIPTION	TEST STANDARD
Sieve Analysis	Particle Size Distribution	ASTM D 422
Modified Proctor	Moisture Density Relationship	ASTM D 1557

TABLE 1B

MINIMUM EARTHWORK TESTING FREQUENCIES

TEST	TEST METHOD	FILL
Sieve Analysis	ASTM D 422	1 per 20,000 CY or 1 per material type
Modified Proctor	ASTM D 1557	1 per 20,000 CY or 1 per material type
Nuclear Densiometer – In-situ Moisture/Density	ASTM D 6938	1 per 500 yd^3

TABLE 2A

TEST PROCEDURES FOR THE EVALUATION OF AGGREGATE

TEST METHOD	DESCRIPTION	TEST STANDARD
Sieve Analysis	Particle Size Distribution of Fine and Coarse Aggregates	ASTM C 136
Hydraulic Conductivity (Rigid Wall Permeameter)	Permeability of Aggregates	ASTM D 2434

TABLE 2B

MINIMUM AGGREGATE TESTING FREQUENCIES FOR CONFORMANCE TESTING

TEST	TEST METHOD	DRAINAGE AGGREGATE
Sieve Analysis	ASTM C 136	1 per project
Hydraulic Conductivity	ASTM D 2434	1 per project

TABLE 3

GEOMEMBRANE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	FREQUENCY
Specific Gravity	ASTM D 792 Method A or ASTM D 1505	200,000 ft^2
Thickness	ASTM D 5199	200,000 ft^2
Tensile Strength at Yield	ASTM D 638	200,000 ft^2
Tensile Strength at Break	ASTM D 638	200,000 ft^2
Elongation at Yield	ASTM D 638	200,000 ft^2
Elongation at Break	ASTM D 638	200,000 ft^2
Carbon Black Content	ASTM D 1603	200,000 ft^2
Carbon Black Dispersion	ASTM D 5596	200,000 ft^2
Interface Shear Strength[1,2]	ASTM D 5321	1 per project

Notes:
1. To be performed at normal stresses of 10, 20, and 30 psi between smooth geomembrane and underlying woven side of GCL and overlying geonet. GCL shall be hydrated for 48 hours under a normal stress of 250 psf prior to testing.
2. To be performed at normal stresses of 100, 200, and 300 psf between textured geomembrane and underlying woven side of GCL and overlying cushion geotextile. GCL shall be hydrated for 48 hours prior to testing.

TABLE 4

GEOTEXTILE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
Mass per Unit Area	ASTM D 5261	1 test per 260,000 ft^2
Grab Strength	ASTM D 4632	1 test per 260,000 ft^2
Puncture Resistance	ASTM D 4833	1 test per 260,000 ft^2
Permittivity	ASTM D 4491	1 test per 260,000 ft^2
Apparent Opening Size	ASTM D 4751	1 test per 260,000 ft^2

TABLE 5

GCL CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
Mass per Unit Area	ASTM D 5993	1 test per 100,000 ft^2
Index Flux	ASTM D 5887	1 test per 400,000 ft^2

Note: Hydraulic index flux testing shall be performed under an effective confining stress of 5 pounds per square inch.

TABLE 6

GEONET CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
Thickness	ASTM D 5199	1 test per 200,000 ft^2
Hydraulic Transmissivity	ASTM D 4716	1 test per 400,000 ft^2

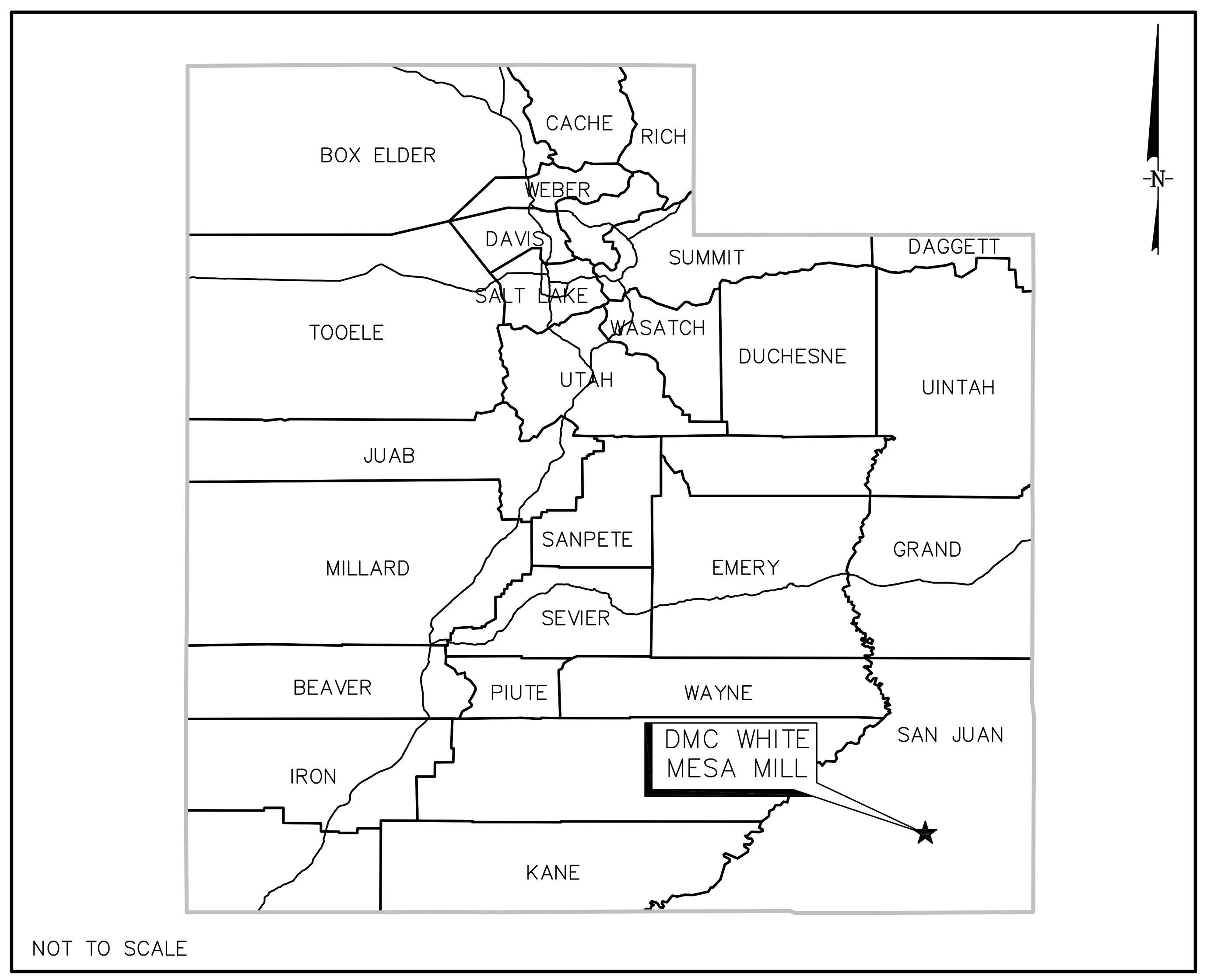
Note: Transmissivity shall be measured using water at 68°F with a gradient of 0.1 under a confining pressure of 7,000 lb/ft^2. The geonet shall be placed in the testing device between 60-mil smooth geomembrane. Measurements are taken one hour after application of confining pressure.

EXHIBIT H

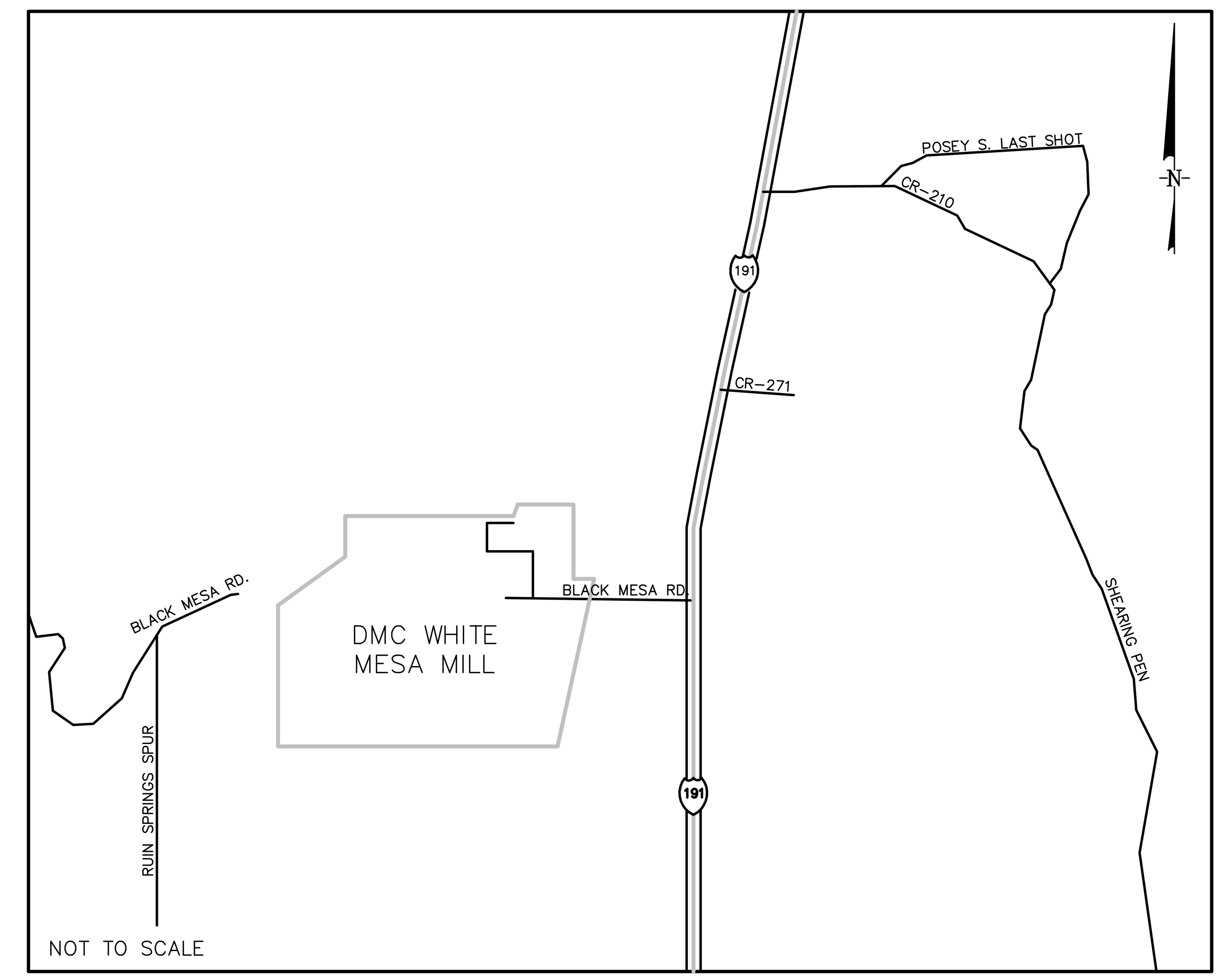
REVISED CONSTRUCTION DRAWINGS

CONSTRUCTION DRAWINGS
DMC WHITE MESA MILL
CELL 4B LINING SYSTEM
BLANDING, UTAH
SEPTEMBER 2008

LIST OF DRAWINGS

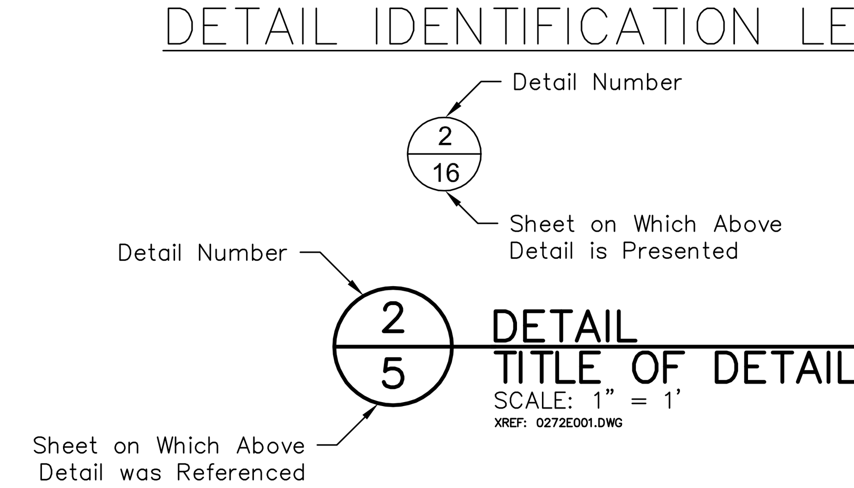


VICINITY MAP



LOCATION MAP

DETAIL IDENTIFICATION LEGEND



Example: Detail Number 2 Presented on Sheet No. 16 was Referenced on Sheet No. 5.

NOTES: ABOVE REFERENCING SYSTEM ALSO APPLIES TO SECTION IDENTIFICATIONS.

PREPARED FOR:

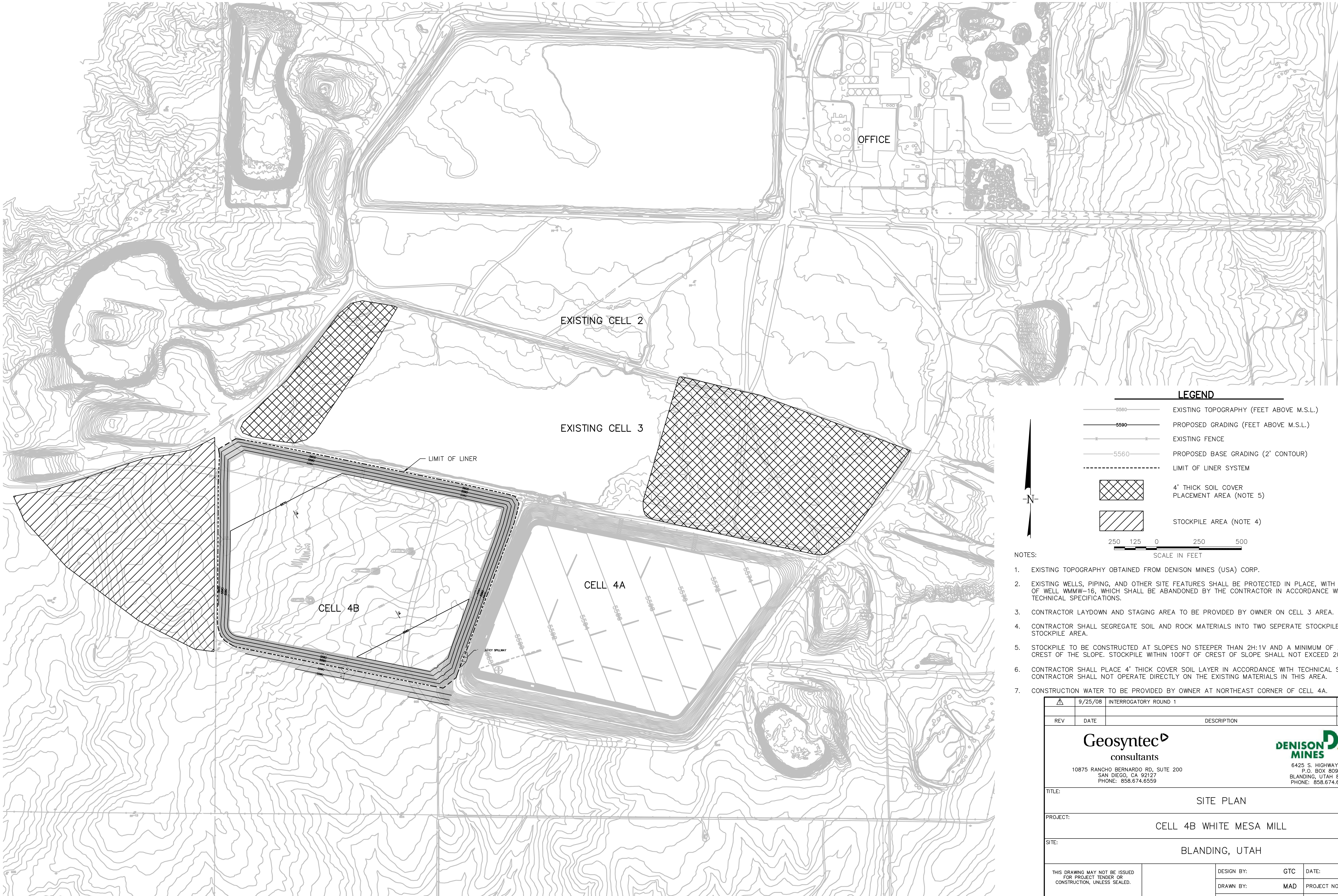
DENISON MINES

DENISON MINES (USA) CORP.
6425 S. HIGHWAY 191
P.O. BOX 809
BLANDING, UTAH 84511
(306) 628-7798

PREPARED BY:

Geosyntec consultants

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
(858) 674-6559



LEGEND

- EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
- PROPOSED GRADING (FEET ABOVE M.S.L.)
- EXISTING FENCE
- PROPOSED BASE GRADING (2' CONTOUR)
- LIMIT OF LINER SYSTEM
- 4' THICK SOIL COVER PLACEMENT AREA (NOTE 5)
- STOCKPILE AREA (NOTE 4)

250 125 0 250 500
SCALE IN FEET

NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM DENISON MINES (USA) CORP.
2. EXISTING WELLS, PIPING, AND OTHER SITE FEATURES SHALL BE PROTECTED IN PLACE, WITH THE EXCEPTION OF WELL WMMW-16, WHICH SHALL BE ABANDONED BY THE CONTRACTOR IN ACCORDANCE WITH THE TECHNICAL SPECIFICATIONS.
3. CONTRACTOR LAYDOWN AND STAGING AREA TO BE PROVIDED BY OWNER ON CELL 3 AREA.
4. CONTRACTOR SHALL SEGREGATE SOIL AND ROCK MATERIALS INTO TWO SEPERATE STOCKPILES IN STOCKPILE AREA.
5. STOCKPILE TO BE CONSTRUCTED AT SLOPES NO STEEPER THAN 2H:1V AND A MINIMUM OF 20FT FROM THE CREST OF THE SLOPE. STOCKPILE WITHIN 100FT OF CREST OF SLOPE SHALL NOT EXCEED 20FT IN HEIGHT.
6. CONTRACTOR SHALL PLACE 4' THICK COVER SOIL LAYER IN ACCORDANCE WITH TECHNICAL SPECIFICATIONS. CONTRACTOR SHALL NOT OPERATE DIRECTLY ON THE EXISTING MATERIALS IN THIS AREA.
7. CONSTRUCTION WATER TO BE PROVIDED BY OWNER AT NORTHEAST CORNER OF CELL 4A.

REV	DATE	DESCRIPTION	DRN	APP
1	9/25/08	INTERROGATORY ROUND 1	MD	GTC

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
SAN DIEGO, CA 92127
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DENISON MINES
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PHONE: 858.674.6559

TITLE: SITE PLAN

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.

SIGNATURE

DATE

DESIGN BY:	GTC	DATE:	DECEMBER 2007
DRAWN BY:	MAD	PROJECT NO.:	SC0349
CHECKED BY:	RF	FILE:	
REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC	2 OF 8	

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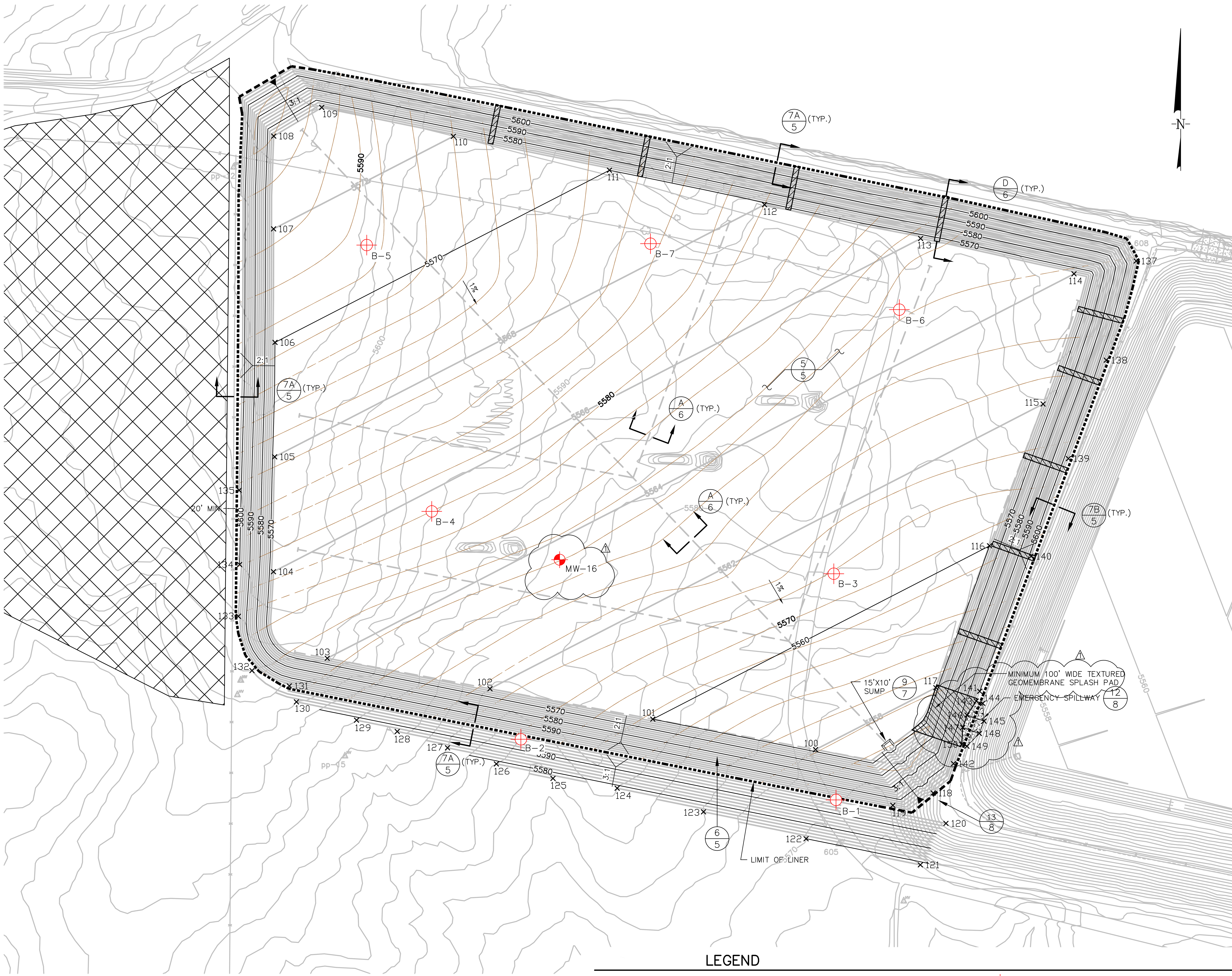

POINT TABLE			
POINT #	NORTHING	EASTING	ELEVATION
100	319447.2859	2577164.8929	5558
101	319507.2322	2576844.3805	5560
102	319567.1785	2576523.8681	5562
103	319627.1248	2576203.3557	5564
104	319797.6436	2576097.6307	5566
105	320023.8073	2576099.9958	5568
106	320249.0377	2576100.5478	5570
107	~~320472.5420~~	~~2576097.7471~~	5572
108	320655.2385	2576098.0345	5574
109	320711.5023	2576192.6462	5574
110	~~320654.7732~~	~~2576451.7313~~	5572
111	320588.0182	2576759.0177	5570
112	320520.2894	2577064.4125	5568
113	320453.5570	2577371.7429	5566
114	320384.0551	2577673.6935	5564
115	320127.7348	2577612.7493	5562
116	319848.5293	2577507.3503	5560
117	319569.1802	2577401.6727	5558
118	319361.7901	2577396.4633	5598
119	319337.6725	2577316.9812	5598
120	319302.0250	2577421.5901	5598
121	319220.3604	2577371.2400	5568
122	319271.8283	2577146.1490	5570
123	319324.7464	2576944.2404	5574
124	319371.2425	2576774.2086	5578
125	319390.5815	2576647.8346	5576
126	319419.1551	2576536.3841	5578
127	319450.7637	2576440.2498	5582
128	319482.9638	2576341.1792	5586
129	319505.2211	2576261.0810	5588
130	319541.3069	2576142.7230	5592
131	319572.6925	2576128.7102	5598
132	319602.9949	2576055.3321	5596
133	319709.7865	2576028.4338	5598
134	319811.6839	2576031.0071	5600
135	319957.9848	2576030.0557	5602
136	320449.2987	2577774.1998	5608
137	320409.2490	2577796.0277	5608
138	320214.1456	2577737.3159	5506
139	320020.3153	2577662.5848	5604
140	319828.0217	2577589.1862	5602
141	319562.1406	2577487.9916	5600
142	319419.5433	2577436.4654	5598
143	319542.8711	2577481.2798	5600
144	319538.3476	2577494.0228	5600
145	319503.5037	2577496.5228	5596.3
146	319515.3611	2577463.1200	5596
147	319490.7627	2577454.7052	5596
148	319479.1526	2577487.4112	5596.3
149	319453.9250	2577462.8219	5600
150	319457.5650	2577452.5678	5600

LEGEND

5560	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
5570	APPROXIMATE ROCK ELEVATION (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (10' CONTOUR)
	LIMIT OF LINER
	MONITORING WELL
	STOCKPILE AREA (NOTE 2)
	SPLASH PAD
1%	SLOPE DIRECTION AND GRADE
B-1	APPROXIMATE SOIL BORING LOCATION
	LEAK DETECTION PIPE TRENCH

NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM DENISON MINES (USA) CORP.
2. STOCKPILE TO BE CONSTRUCTED AT SLOPES NO STEEPER THAN 2H:1V AND A MINIMUM OF 20FT FROM THE CREST OF THE SLOPE. STOCKPILE WITHIN 100FT OF CREST OF SLOPE SHALL NOT EXCEED 20FT IN HEIGHT.

REV	DATE	DESCRIPTION	DRN	APP
1	9/25/08	INTERROGATORY ROUND 1	MD	GTC

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
SAN DIEGO, CA 92127
PHONE: 858.674.6559

DENISON MINES
6425 S. HIGHWAY 191
P.O. BOX 809
BLANDING, UTAH 84511
PHONE: 858.674.6559

TITLE: BASE GRADING PLAN

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.

SIGNATURE

DATE

DESIGN BY:	GTC	DATE:	DECEMBER 2007
DRAWN BY:	MAD	PROJECT NO.:	SC0349
CHECKED BY:	RF	FILE:	
REVIEWED BY:	GTC	DRAWING NO.:	3 OF 8
APPROVED BY:	GTC		

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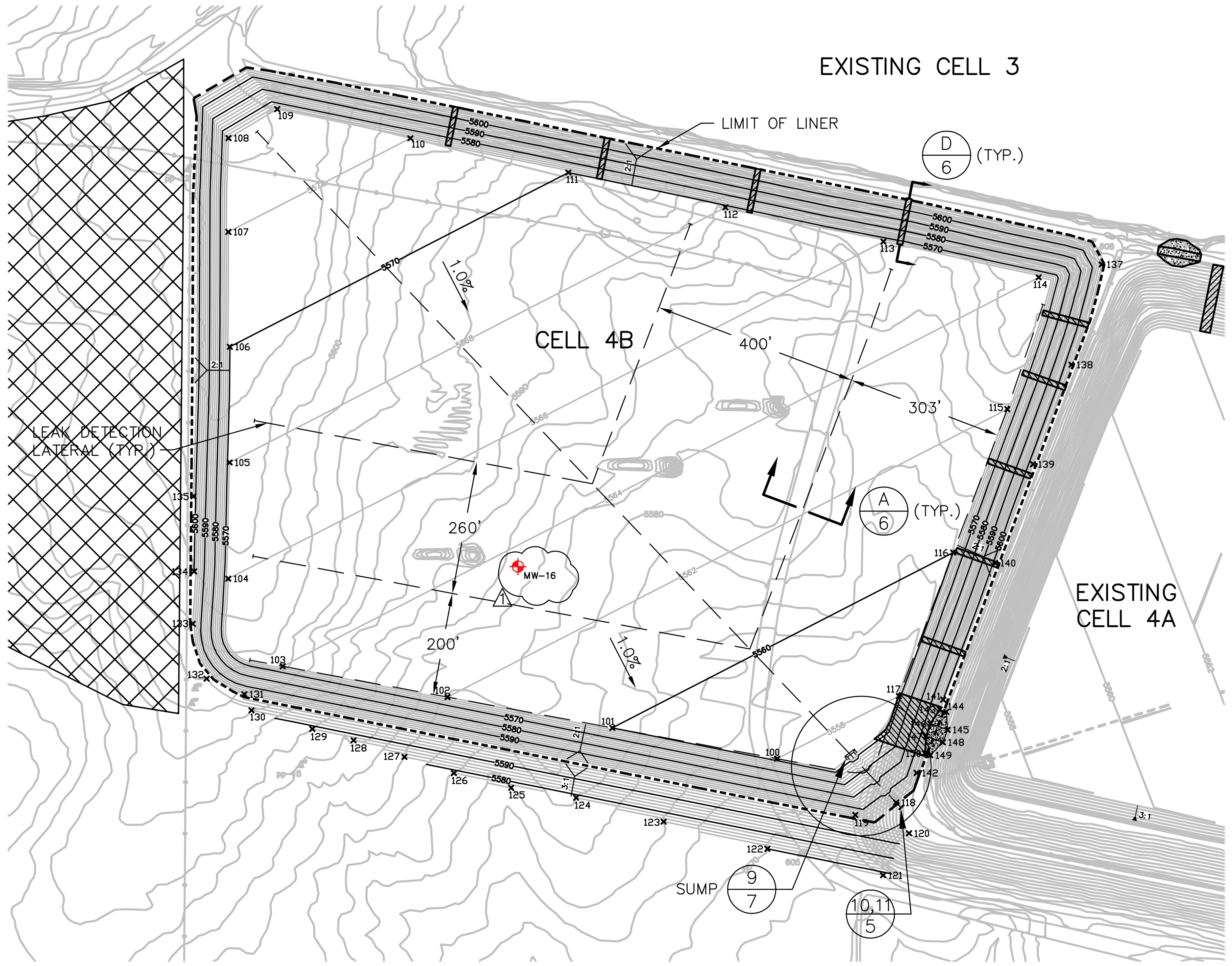
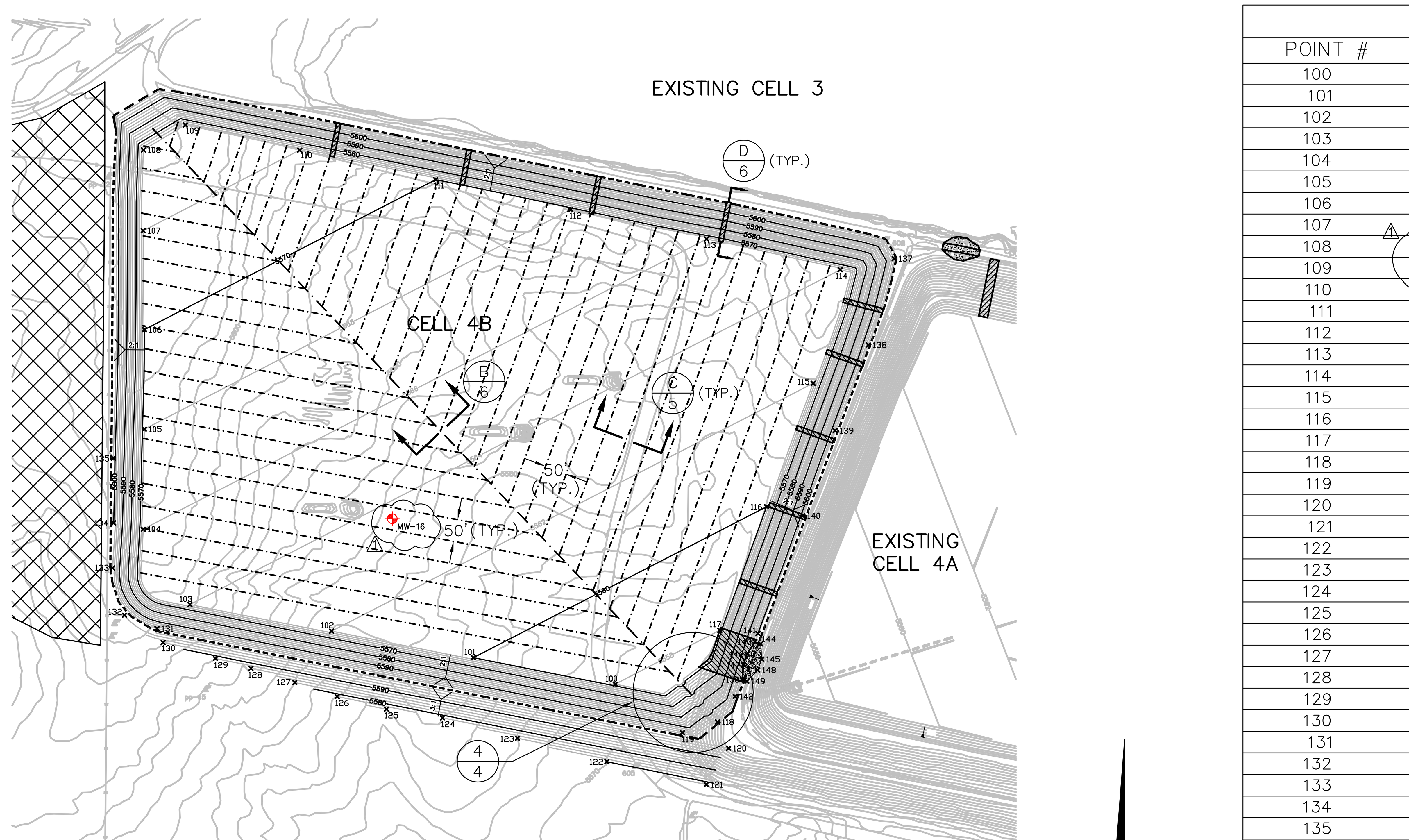
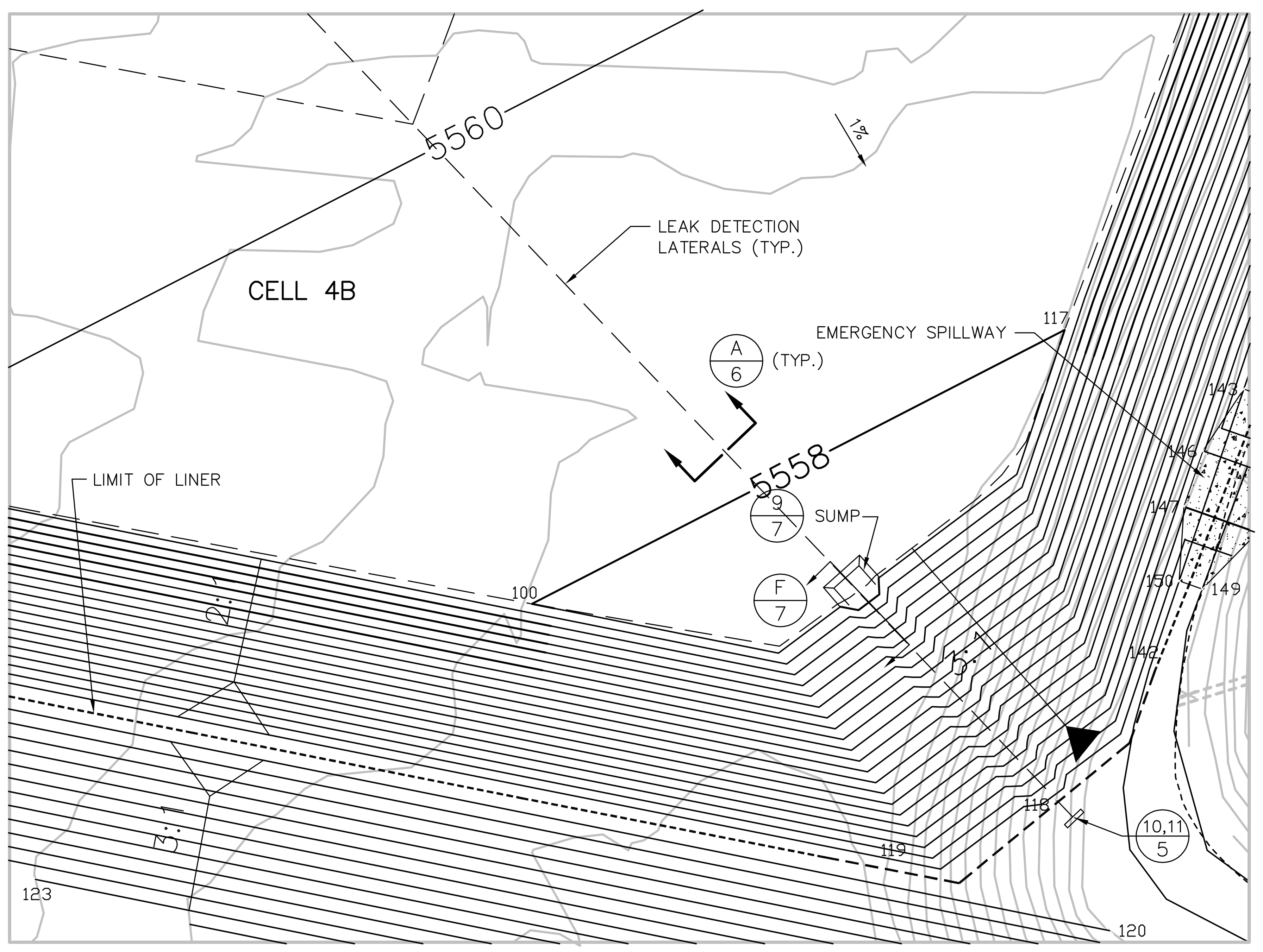


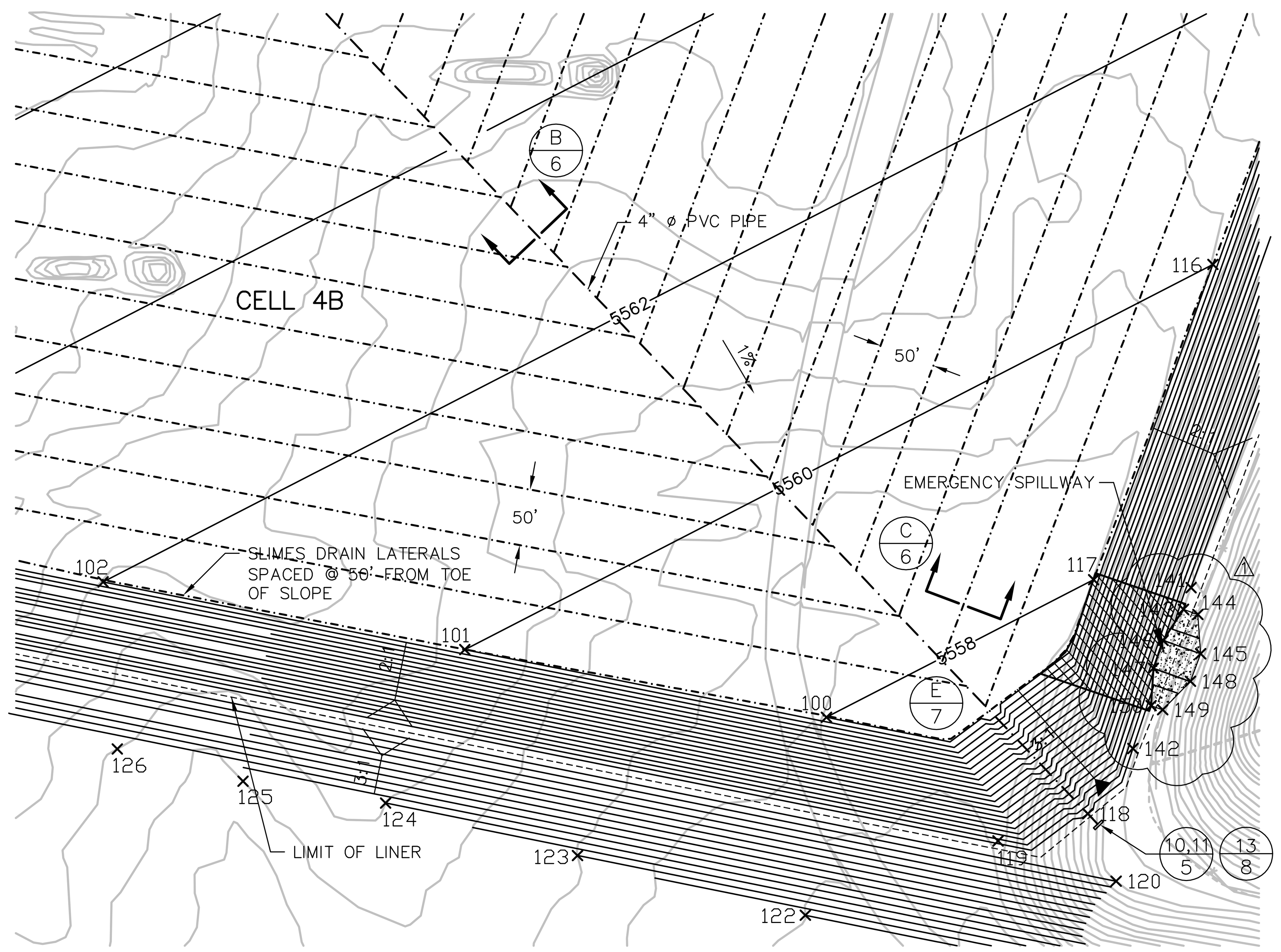
1
4
PLAN
LEAK DETECTION SYSTEM
SCALE: 1" = 200'



2
4
PLAN
SLIMES DRAIN SYSTEM
SCALE: 1" = 200'



3
4
DETAIL
LEAK DETECTION SYSTEM
SCALE: 1" = 50'
XREF: 0349X006.DWG



4
4
DETAIL
SLIMES DRAIN SYSTEM
SCALE: 1" = 100'
XREF: 0349X007.DWG

N

POINT TABLE			
POINT #	NORTHING	EASTING	ELEVATION
100	319447.2859	2577164.8929	5558
101	319507.2322	2576844.3805	5560
102	319567.1785	2576523.8681	5562
103	319627.1248	2576203.3557	5564
104	319797.6436	2576097.6307	5566
105	320023.8073	2576099.9958	5568
106	320249.0377	2576100.5478	5570
107	320472.5420	2576097.7471	5572
108	320655.2385	2576098.0345	5574
109	320711.5023	2576192.6462	5574
110	320654.7732	2576451.7313	5572
111	320588.0182	2576759.0177	5570
112	320520.2894	2577064.4125	5568
113	320453.5570	2577371.7429	5566
114	320384.0551	2577673.6935	5564
115	320127.7348	2577612.7493	5562
116	319848.5293	2577507.3503	5560
117	319569.1802	2577401.6727	5558
118	319361.7901	2577396.4633	5598
119	319337.6725	2577316.9812	5598
120	319302.0250	2577421.5901	5598
121	319220.3604	2577371.2400	5568
122	319271.8283	2577146.1490	5570
123	319324.7464	2576944.2404	5574
124	319371.2425	2576774.2086	5578
125	319390.5815	2576647.8346	5576
126	319419.1551	2576536.3841	5578
127	319450.7637	2576440.2498	5582
128	319482.9638	2576341.1792	5586
129	319505.2211	2576261.0810	5588
130	319541.3069	2576142.7230	5592
131	319572.6925	2576128.7102	5598
132	319602.9949	2576055.3321	5596
133	319709.7865	2576028.4338	5598
134	319811.6839	2576031.0071	5600
135	319957.9848	2576030.0557	5602
136	320449.2987	2577774.1998	5608
137	320409.2490	2577796.0277	5608
138	320214.1456	2577737.3159	5506
139	320020.3153	2577662.5848	5604
140	319828.0217	2577589.1862	5602
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145	319503.5037	2577496.5228	5596.3
146	319515.3611	2577463.1200	5596
147	319490.7627	2577454.7052	5596
148	319479.1526	2577487.4112	5596.3
149	319453.9250	2577462.8219	5600
150	319457.5650	2577452.5678	5600

1

LEGEND

5560	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (10' CONTOUR)
	LIMIT OF LINER SYSTEM
	SPLASH PAD
1%	SLOPE DIRECTION AND GRADE
	STRIP COMPOSITE AND SAND BAGS
	PVC PIPE AND GRAVEL
	LEAK DETECTION PIPE TRENCH
	TOE OF SLOPE

NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM DENISON MINES (USA) CORP.

REV	DATE	DESCRIPTION	DRN	APP
1	9/25/08	INTERROGATORY ROUND 1	MD	GTC

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
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DENISON MINES
6425 S. HIGHWAY 191
P.O. BOX 809
BLANDING, UTAH 84511
PHONE: 858.674.6559

TITLE: PIPE LAYOUT PLAN AND DETAILS

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

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REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC	4 OF 8	

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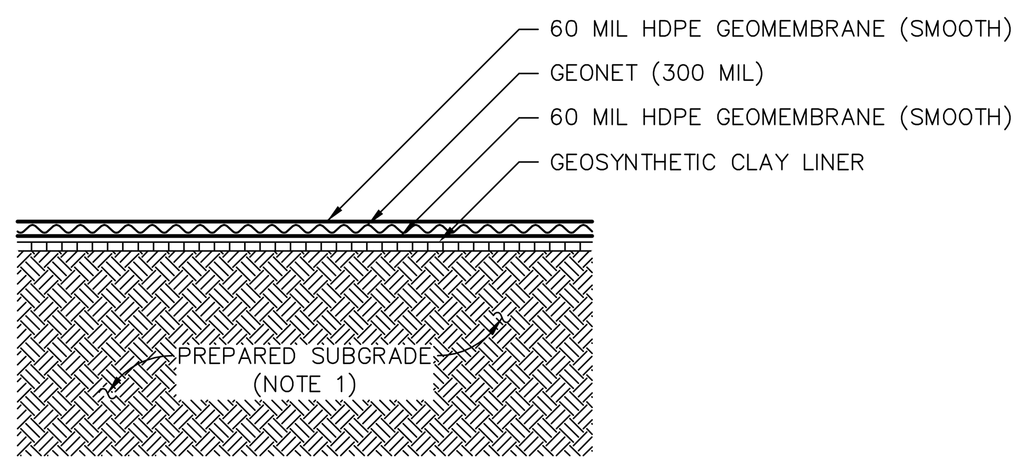
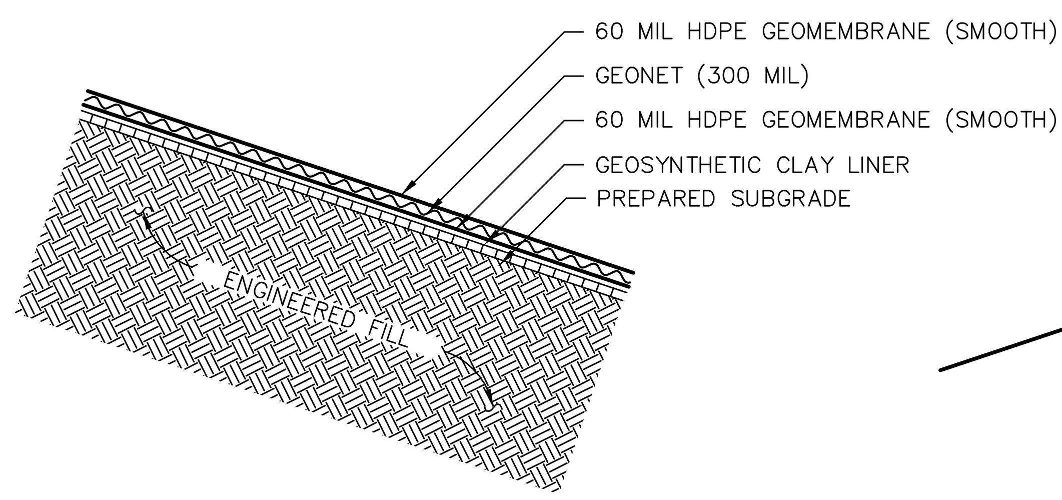
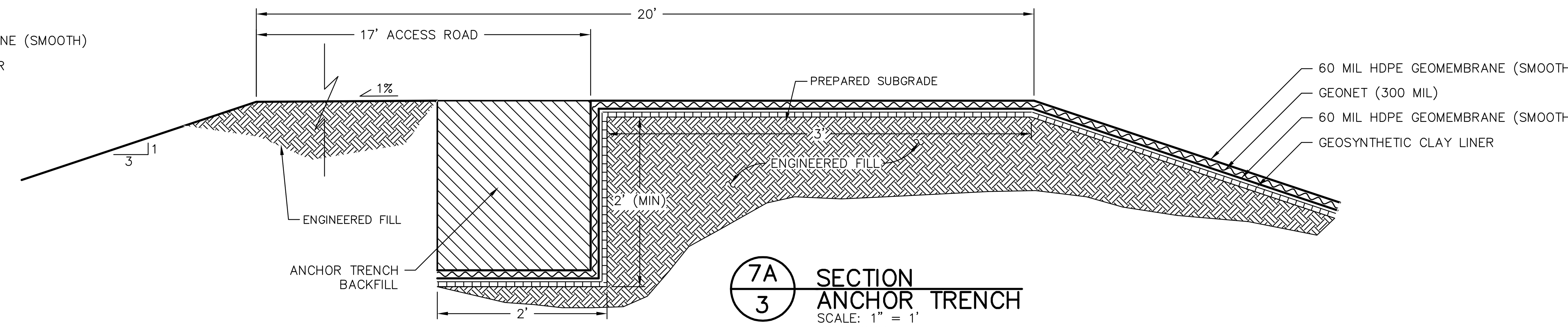


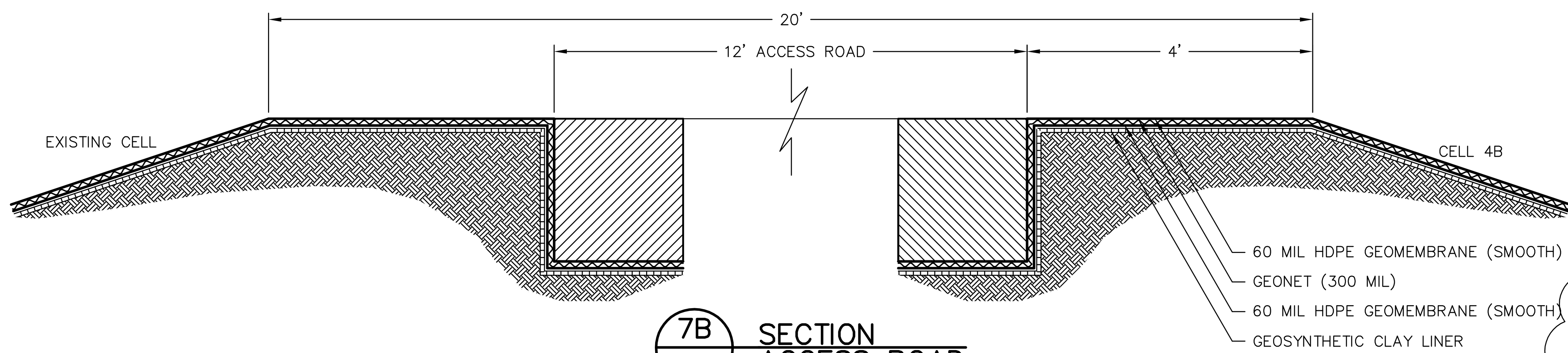
5/3 DETAIL
BASE LINER SYSTEM
SCALE: 1" = 1'
XREF: 0349X002.DWG



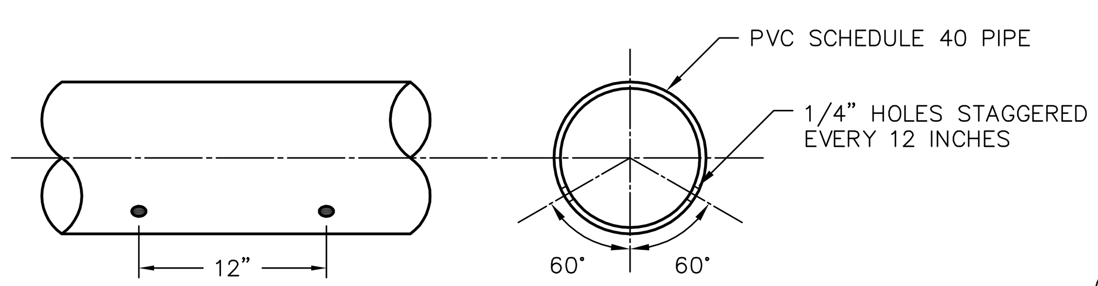
6/3 DETAIL
SIDE SLOPE LINER SYSTEM
SCALE: 1" = 1'
XREF: 0349X002.DWG



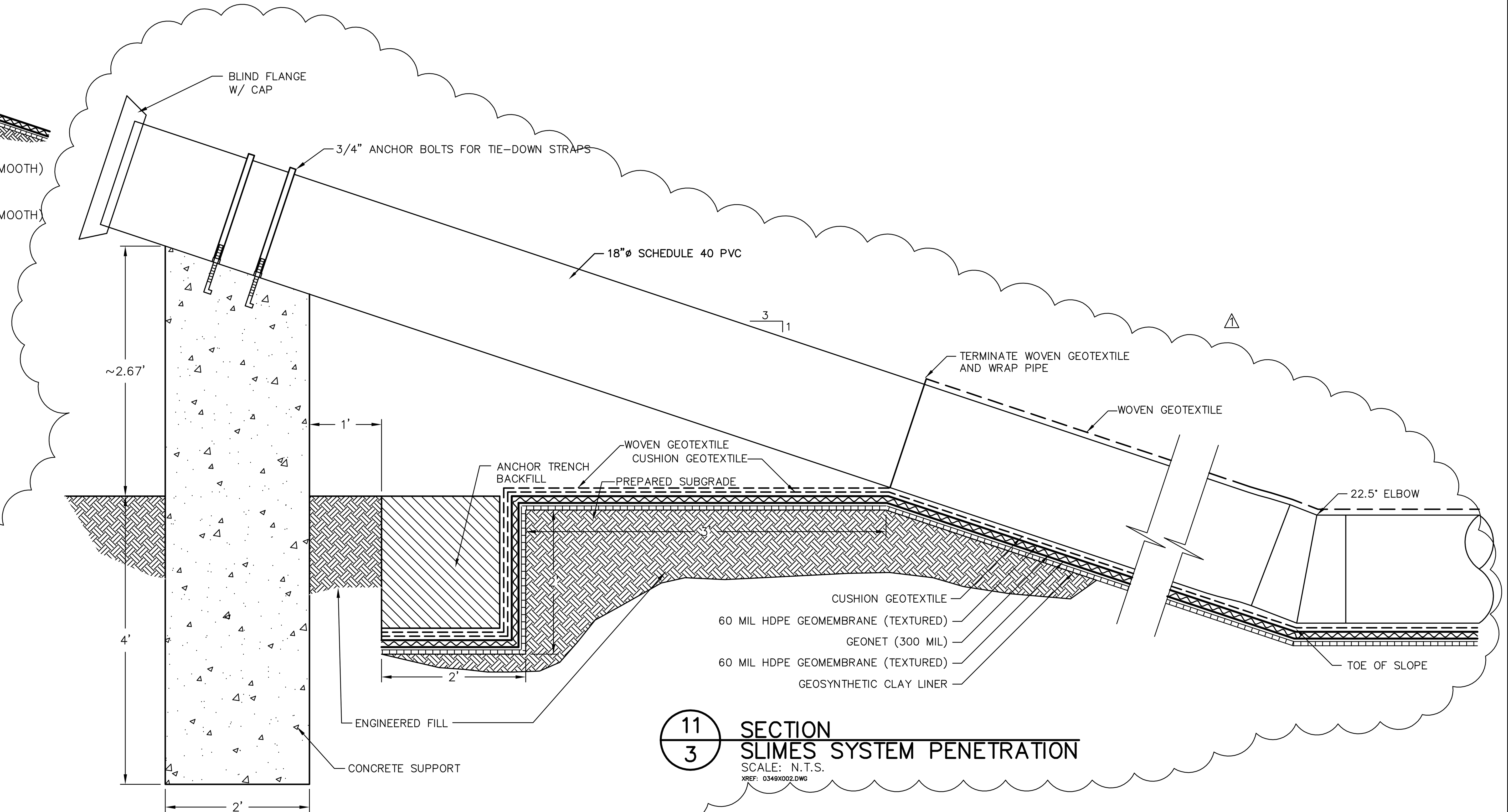
7A/3 SECTION
ANCHOR TRENCH
SCALE: 1" = 1'
XREF: 0349X002.DWG



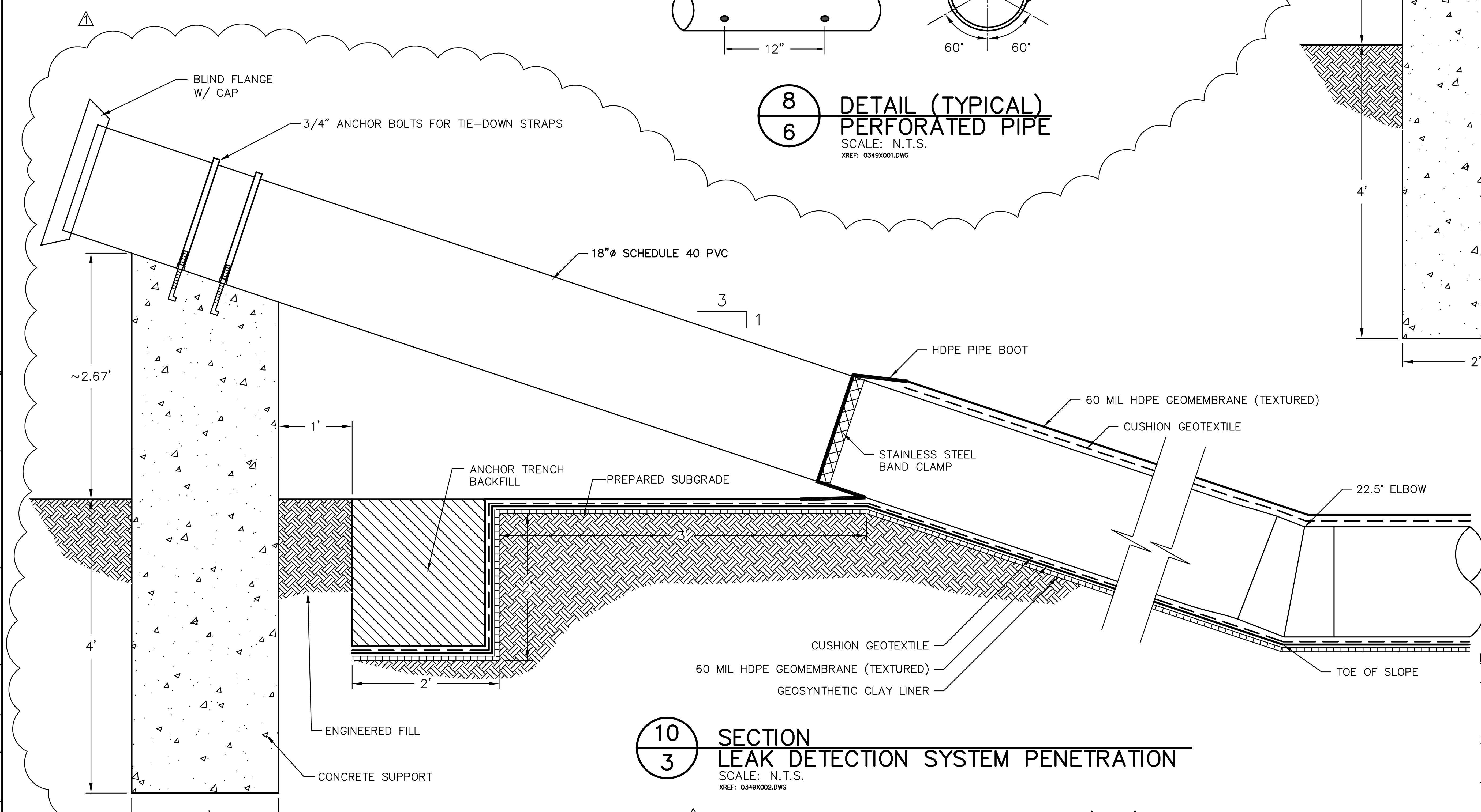
7B/3 SECTION
ACCESS ROAD
SCALE: N.T.S.
XREF: 0349X002.DWG



8/6 DETAIL (TYPICAL)
PERFORATED PIPE
SCALE: N.T.S.
XREF: 0349X001.DWG



11/3 SECTION
SLIMES SYSTEM PENETRATION
SCALE: N.T.S.
XREF: 0349X002.DWG



10/3 SECTION
LEAK DETECTION SYSTEM PENETRATION
SCALE: N.T.S.
XREF: 0349X002.DWG

NOTES:

1. PREPARED SUBGRADE AT CELL BASE SHALL CONSIST OF AT LEAST 6-INCHES OF FILL OVERLYING SANDSTONE IN ACCORDANCE WITH SECTIONS 02200 AND 02220 OF THE TECHNICAL SPECIFICATIONS.
2. DETAILS ARE SHOWN TO SCALE INDICATED EXCEPT FOR THE GEOSYNTHETICS, WHICH ARE SHOWN AT AN EXAGGERATED SCALE FOR CLARITY SOIL THICKNESS ARE MINIMUMS.
3. WOVEN GEOTEXTILE SHALL BE PROPEX 200 ST, SKAPS W-200, OR APPROVED EQUAL (WOVEN SLIT FILM, AOS = 40, FLOW RATE = 4 GPM/SF, GRAB STRENGTH = 200 LBS, PUNCTURE = 100 LBS)
4. EXPOSED PVC PIPE SHALL BE PAINTED TO MINIMIZE DAMAGE DUE TO UV.

REV	DATE	DESCRIPTION	DRN	APP
△	9/25/08	INTERROGATORY ROUND 1	MD	GTC

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
SAN DIEGO, CA 92127
PHONE: 858.674.6559

DENISON MINES
6425 S. HIGHWAY 191
P.O. BOX 809
BLANDING, UTAH 84511
PHONE: 858.674.6559

TITLE: LINING SYSTEM DETAILS I

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

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SIGNATURE

DATE

DESIGN BY:	GTC	DATE:	DECEMBER 2007
DRAWN BY:	MAD	PROJECT NO.:	SC0349
CHECKED BY:	RF	FILE:	
REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC	5 OF 8	

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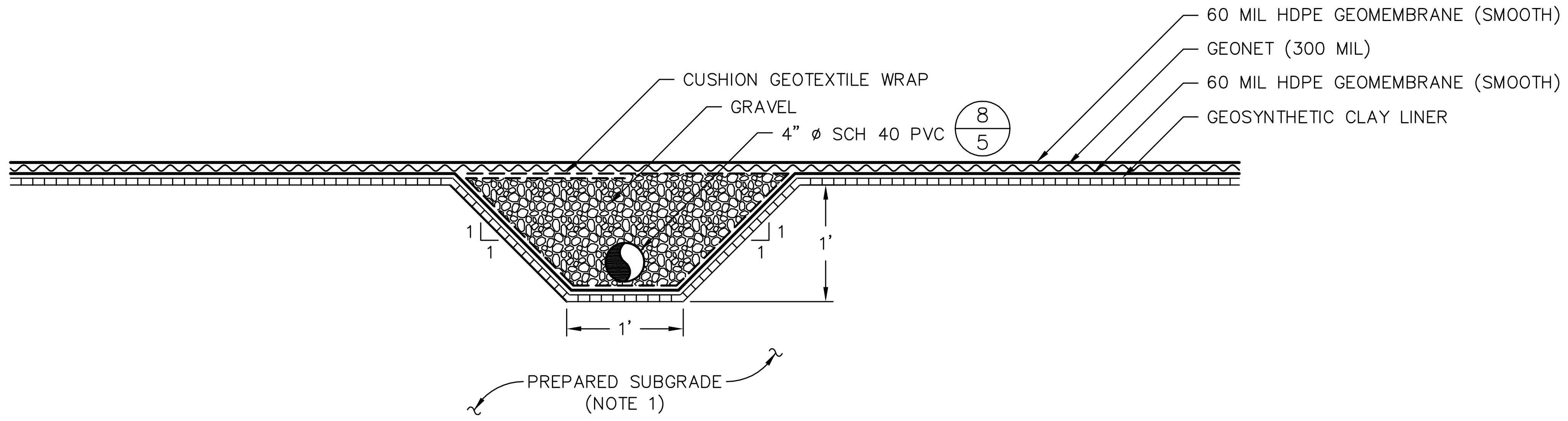
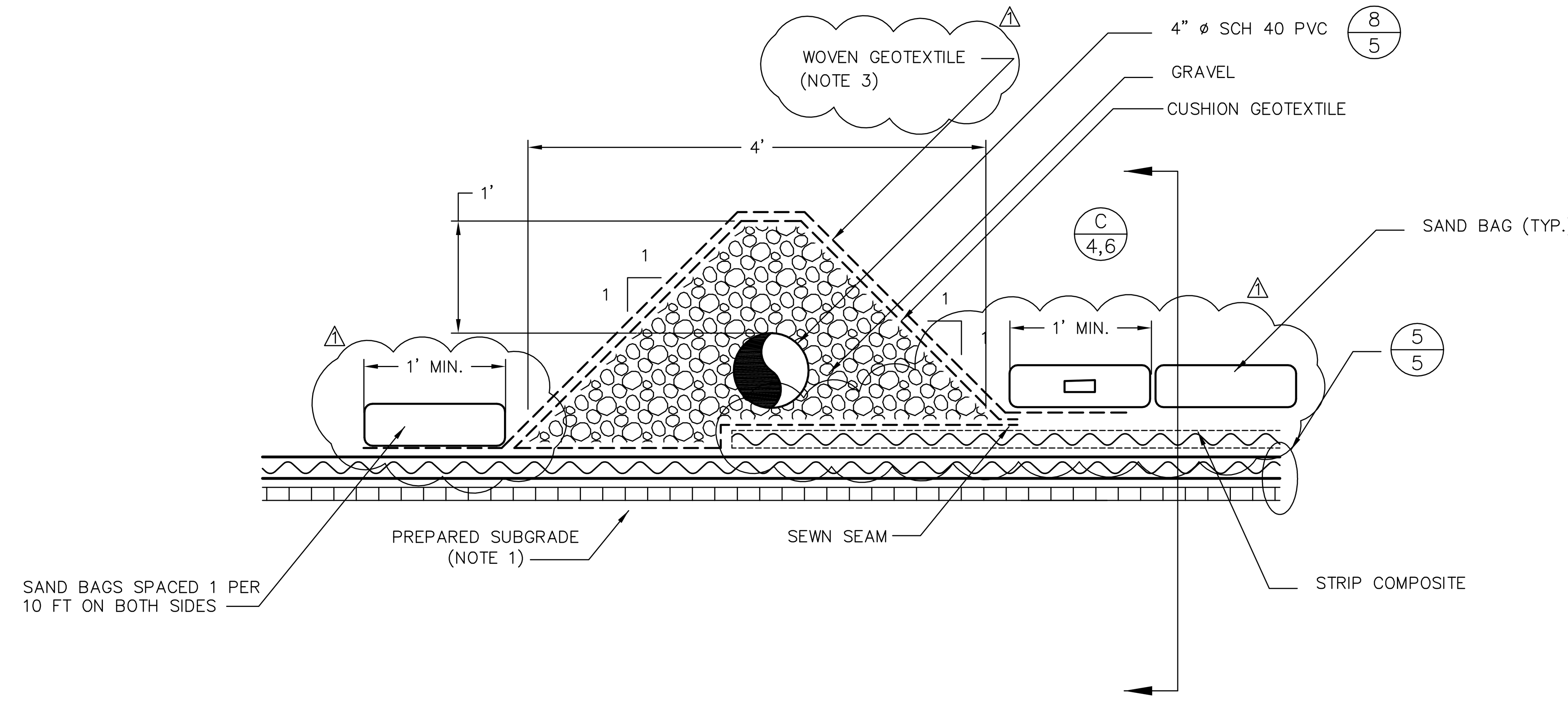
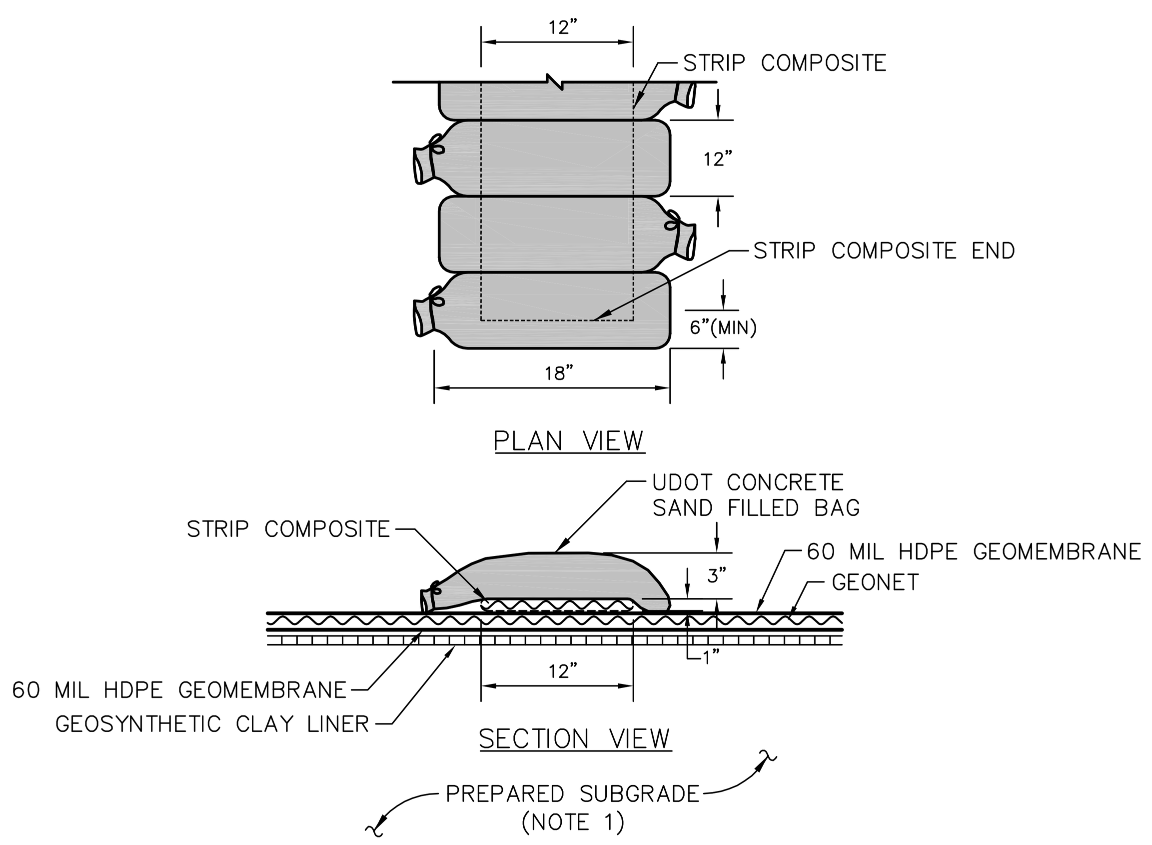


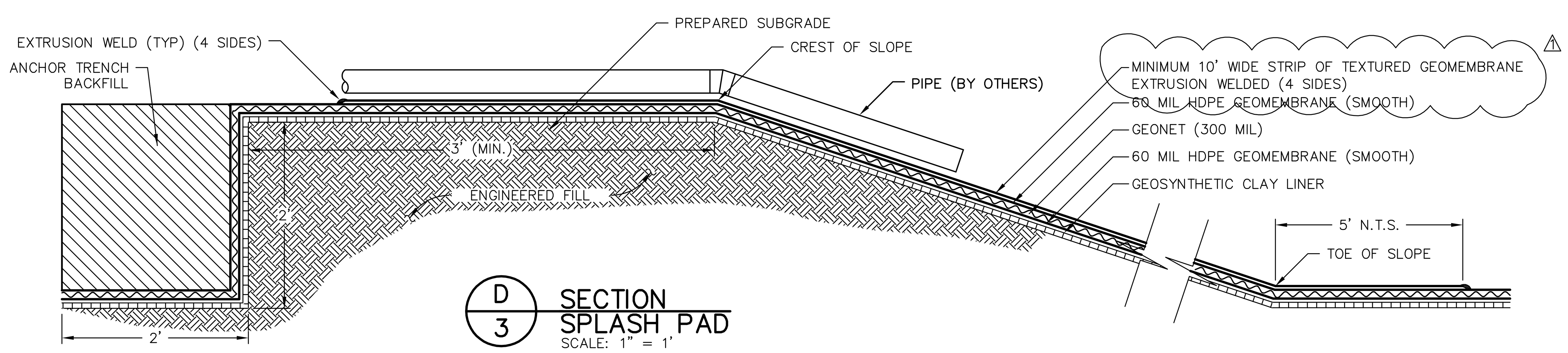
A
3,4
SECTION
LEAK DETECTION SYSTEM
SCALE: 1" = 1'
XREF: 0349X002.DWG



B
7
SECTION
SLIMES DRAIN HEADER
SCALE: 1" = 1'
XREF: 0349X002.DWG



C
4,7
SECTION
SLIMES DRAIN LATERAL
SCALE: 1" = 1'
XREF: 0349X002.DWG



D
3
SECTION
SPLASH PAD
SCALE: 1" = 1'
XREF: 0349X002.DWG

NOTES:

1. PREPARED SUBGRADE AT CELL BASE SHALL CONSIST OF AT LEAST 6-INCHES OF FILL OVERLYING SANDSTONE IN ACCORDANCE WITH SECTIONS 02200 AND 02220 OF THE TECHNICAL SPECIFICATIONS.
2. DETAILS ARE SHOWN TO SCALE INDICATED EXCEPT FOR THE GEOSYNTHETICS, WHICH ARE SHOWN AT AN EXAGGERATED SCALE FOR CLARITY SOIL THICKNESS ARE MINIMUMS.
3. WOVEN GEOTEXTILE SHALL BE PROPEX 200 ST. CONTECH C-200, OR APPROVED EQUAL (WOVEN SLIT FILM, AOS = 40, FLOW RATE = 6 GPM/SF, GRAB STRENGTH = 200 LBS, PUNCTURE = 100 LBS)
4. EXPOSED PVC PIPE SHALL BE PAINTED TO MINIMIZE DAMAGE DUE TO UV.

REV	DATE	DESCRIPTION	DRN	APP
1	9/25/08	INTERROGATORY ROUND 1	MD	GTC

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
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DENISON MINES
6425 S. HIGHWAY 191
P.O. BOX 809
BLANDING, UTAH 84511
PHONE: 858.674.6559

TITLE: LINING SYSTEM DETAILS II

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

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REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC	6 OF 8	

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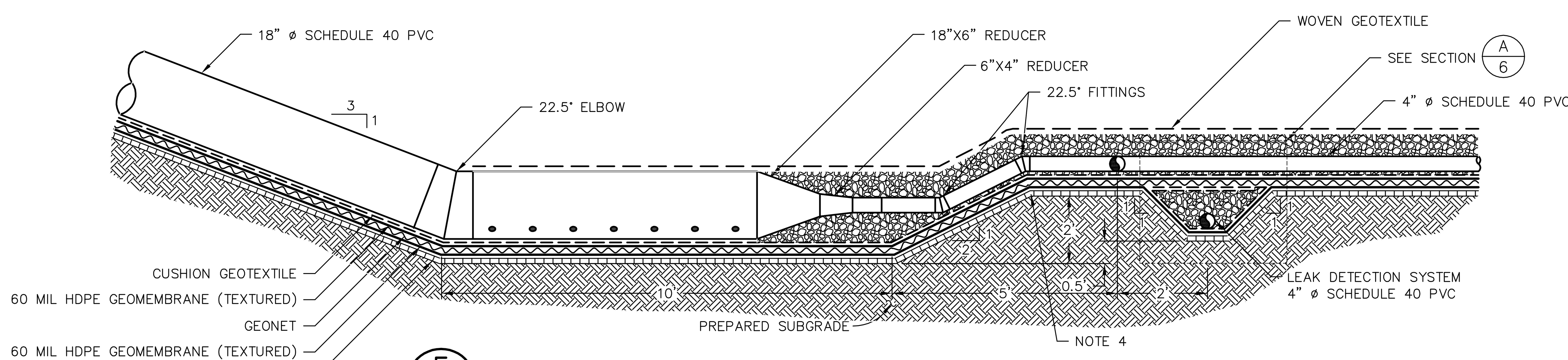
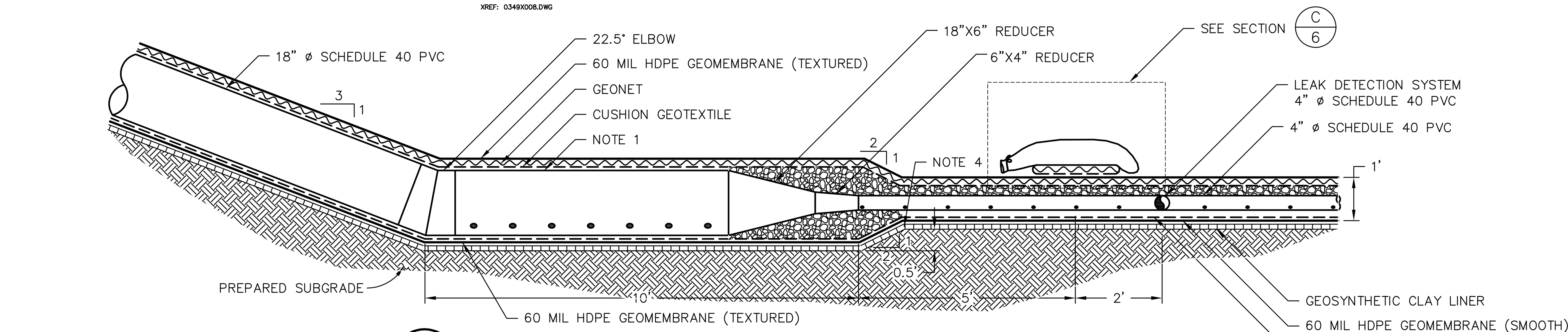
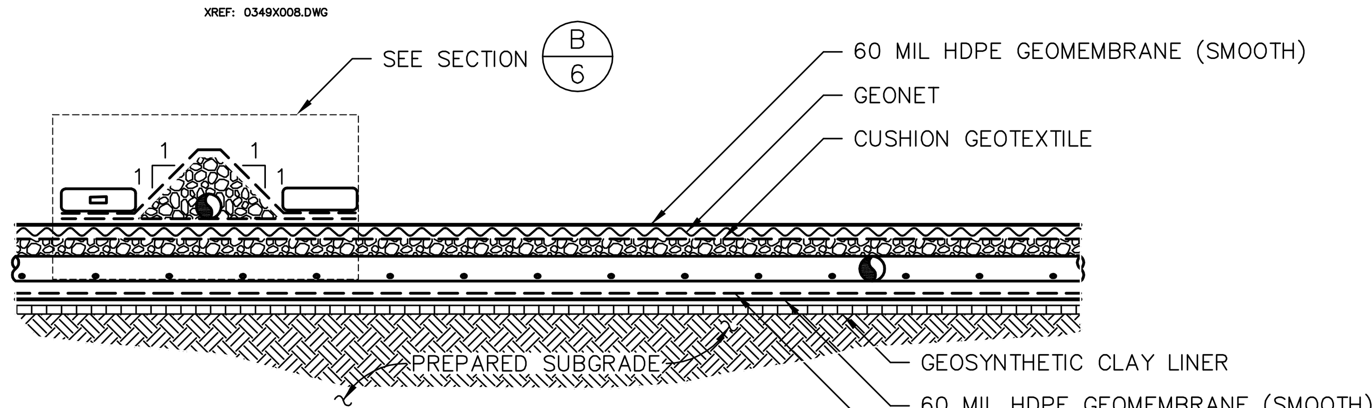


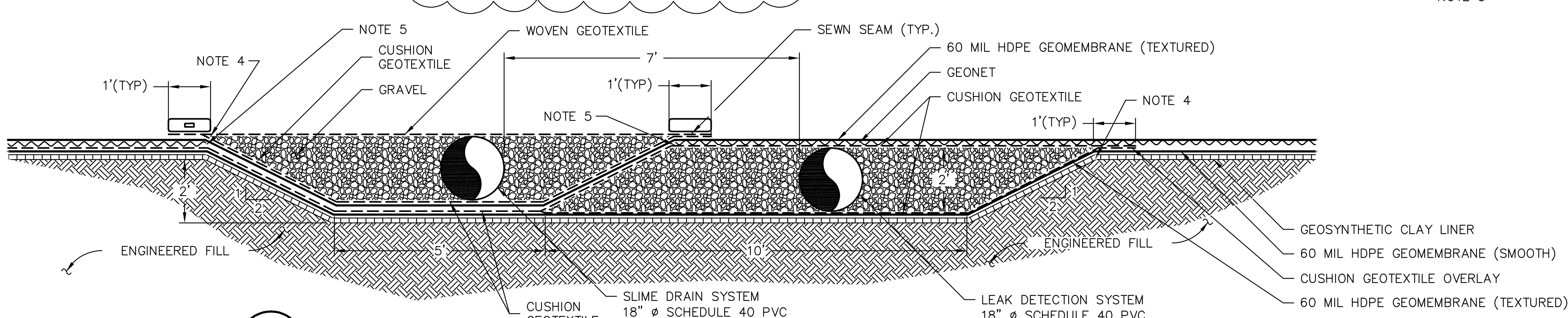
E / 4,7 SECTION
SLIME DRAIN SYSTEM SUMP
N.T.S.
XREF: 0349X008.DWG



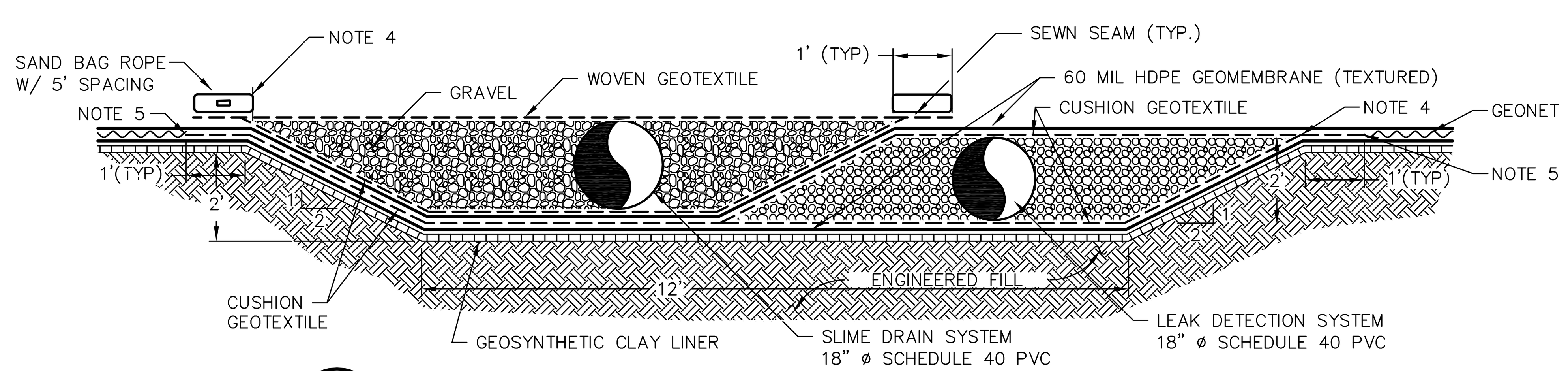
F / 4,7 SECTION
LEAK DETECTION SYSTEM SUMP
N.T.S.
XREF: 0349X008.DWG



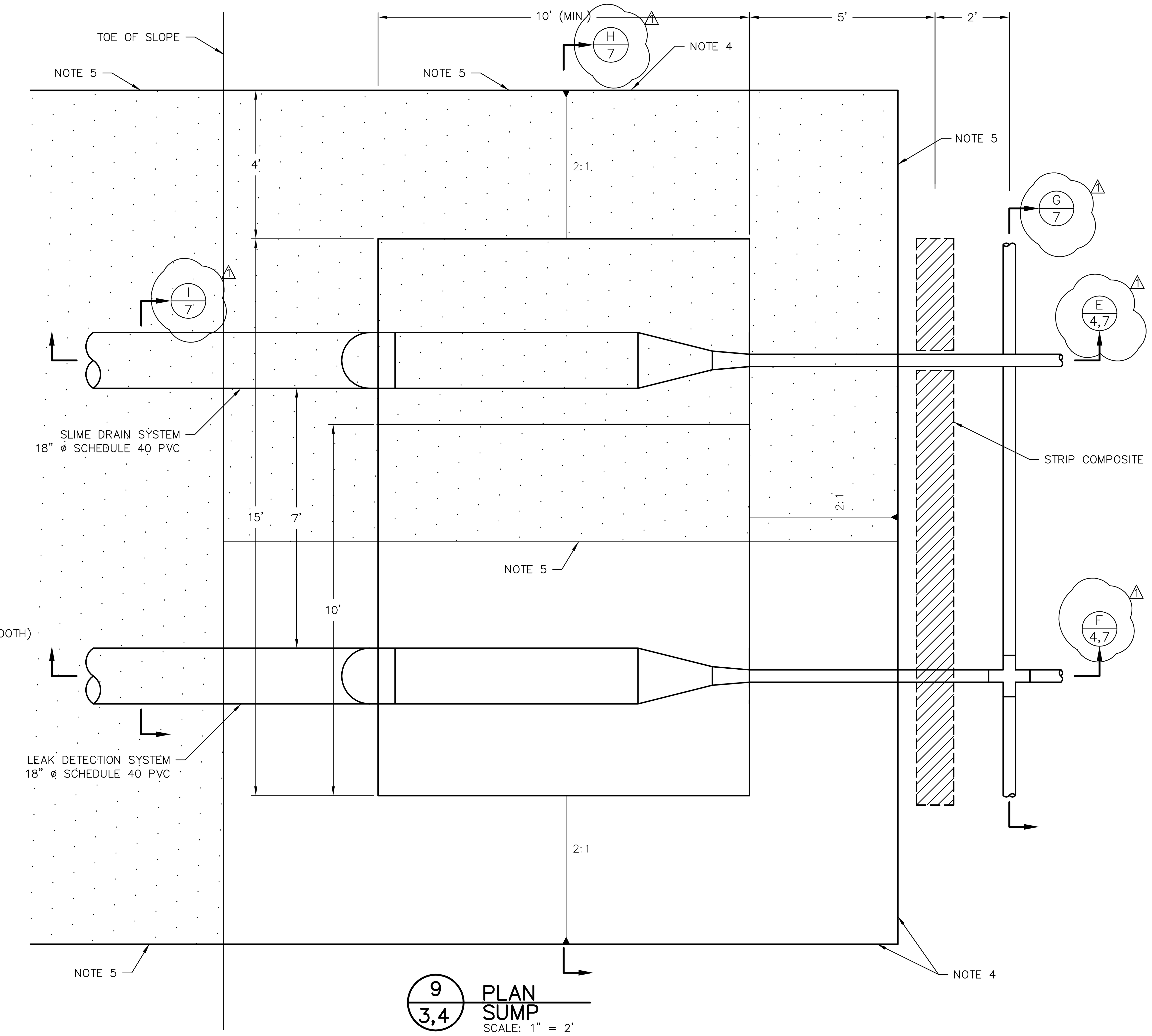
G / 7 SECTION
SLIMES DRAIN AND LDS HEADER PIPES
N.T.S.
XREF: 0349X008.DWG



H / 7 SECTION
SLIMES AND LEAK DETECTION SYSTEM SUMP
N.T.S.
XREF: 0349X008.DWG



I / 7 SECTION
SLIMES AND LEAK DETECTION SYSTEM SIDE SLOPE
N.T.S.
XREF: 0349X008.DWG



9 / 3,4 PLAN
SUMP
SCALE: 1" = 2'
XREF: 0349X008.DWG

NOTES:

1. PREPARED SUBGRADE AT CELL BASE SHALL CONSIST OF AT LEAST 6-INCHES OF FILL OVERLYING SANDSTONE AS PER SECTIONS 02200 AND 02221 OF THE TECHNICAL SPECIFICATIONS.
2. DETAILS ARE SHOWN TO SCALE INDICATED EXCEPT FOR THE GEOSYNTHETICS, WHICH ARE SHOWN AT AN EXAGGERATED SCALE FOR CLARITY SOIL THICKNESS ARE MINIMUMS.
3. WOVEN GEOTEXTILE SHALL BE SYNTHETIC INDUSTRIES 200 ST, SKAPS W 200, OR APPROVED EQUAL (WOVEN SLIT FILM, AOS = 40, FLOW RATE = 4 GPM/SF, GRAB STRENGTH = 200 LBS, PUNCTURE = 100 LBS.)
4. LIMIT OF TEXTURED GEOMEMBRANE.
5. LIMIT OF GEONET.

REV	DATE	DESCRIPTION	DRN	APP
1	6/18/08	INTERROGATORY ROUND 1	MD	GTC

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
SAN DIEGO, CA 92127
PHONE: 858.674.6559

DENISON MINES
6425 S. HIGHWAY 191
P.O. BOX 809
BLANDING, UTAH 84511
PHONE: 858.674.6559

TITLE: LINING SYSTEM DETAILS III

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

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DATE

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DRAWN BY:	MAD	PROJECT NO.:	SC0349
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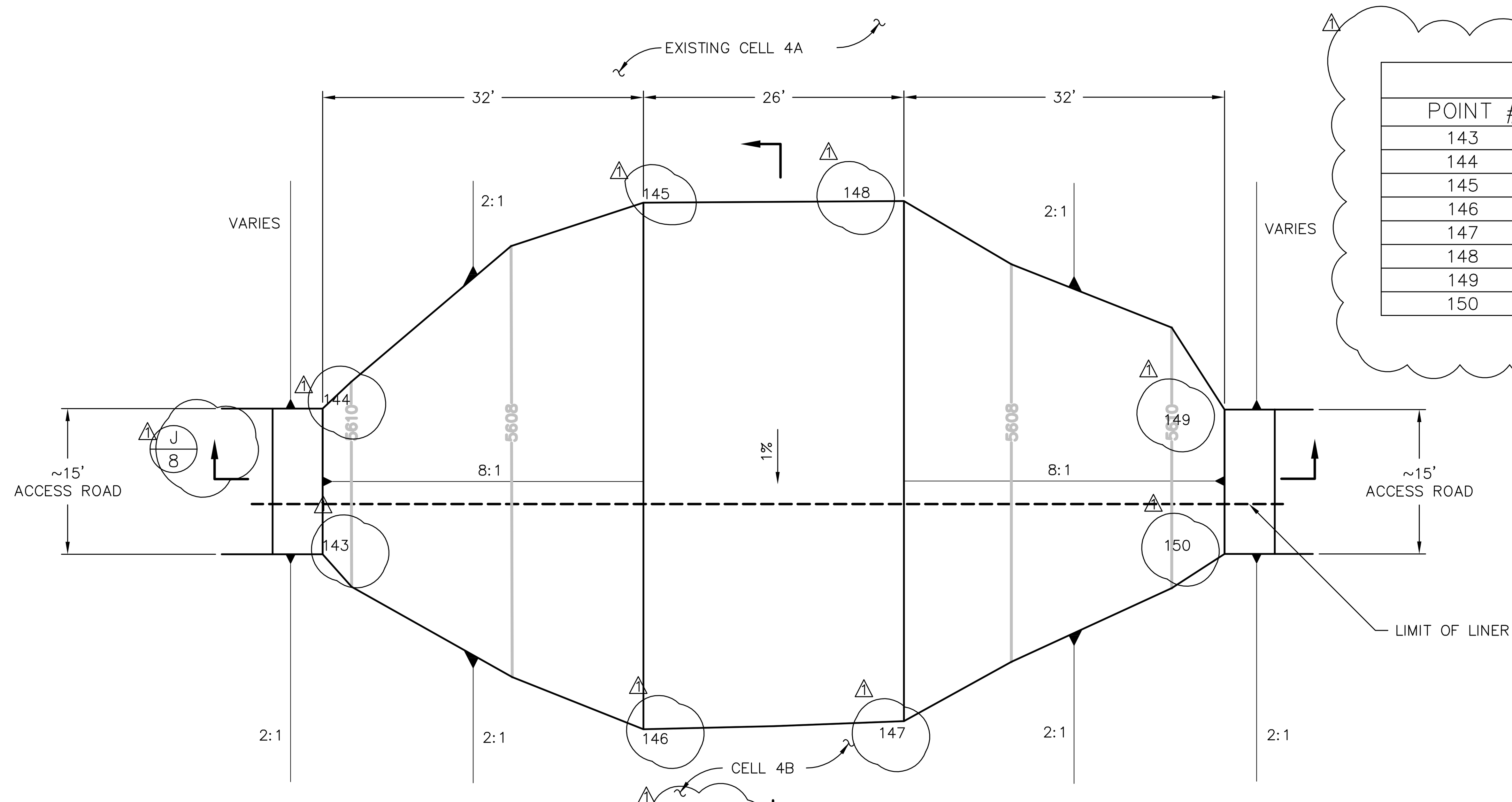
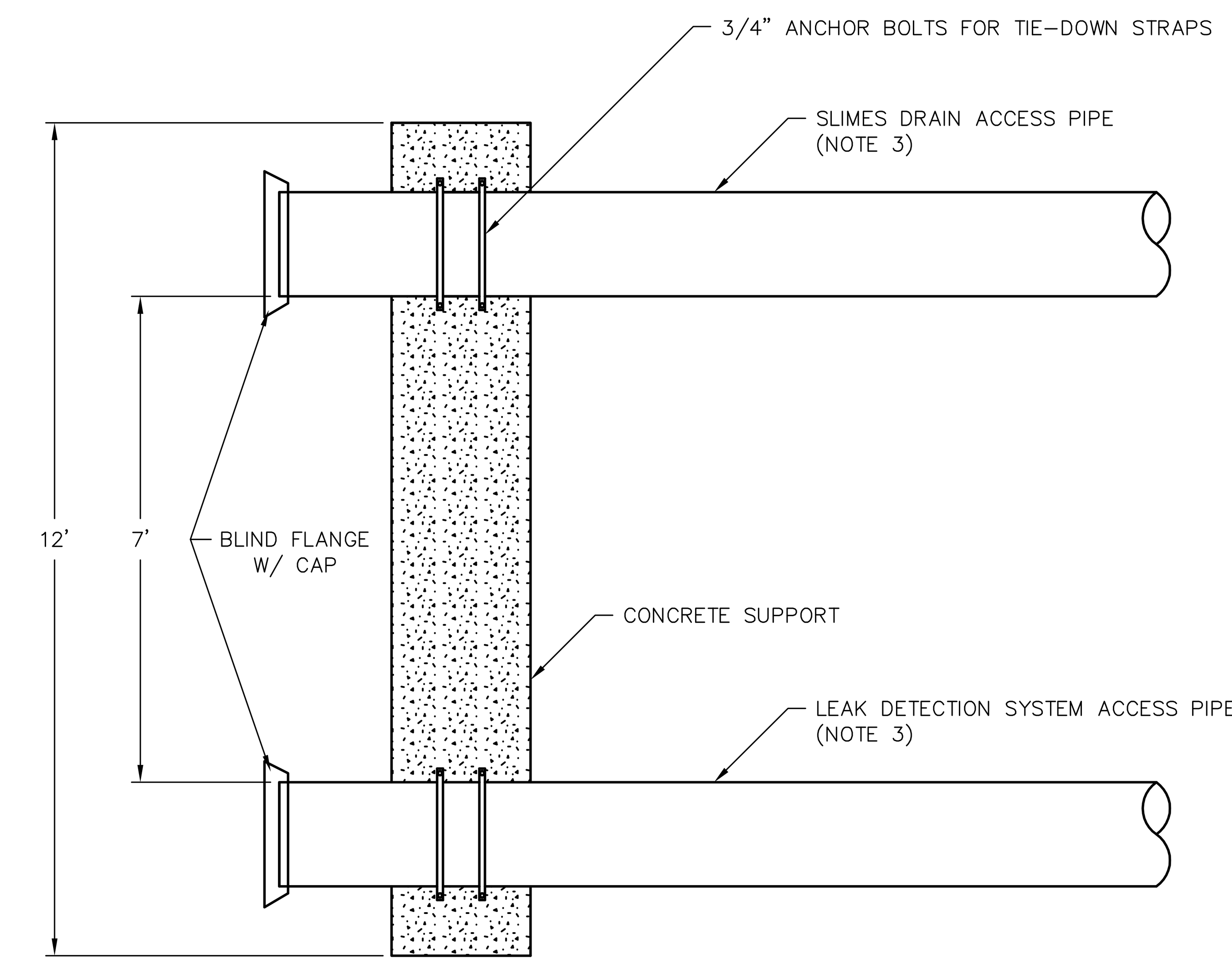
REVIEWED BY:	GTC	DRAWING NO.:	7 OF 8
APPROVED BY:	GTC		

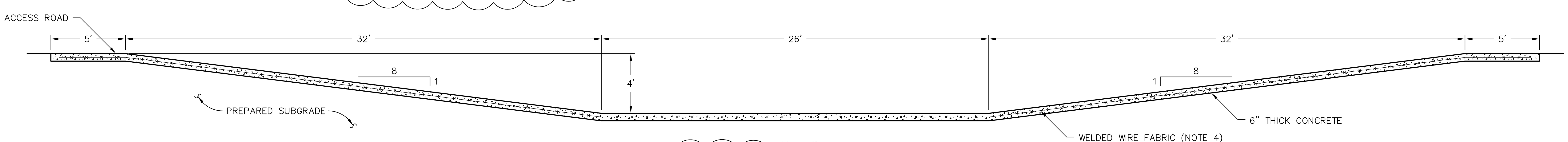


POINT TABLE			
POINT #	NORTHING	EASTING	ELEVATION
143	319542.8711	2577481.2798	5600
144	319538.3476	2577494.0228	5600
145	319503.5037	2577496.5228	5596.3
146	319515.3611	2577463.1200	5596
147	319490.7627	2577454.7052	5596
148	319479.1526	2577487.4112	5596.3
149	319453.9250	2577462.8219	5600
150	319457.5650	2577452.5678	5600

12/3 PLAN
EMERGENCY SPILLWAY
SCALE: NTS
XREF: 0349X004.DWG



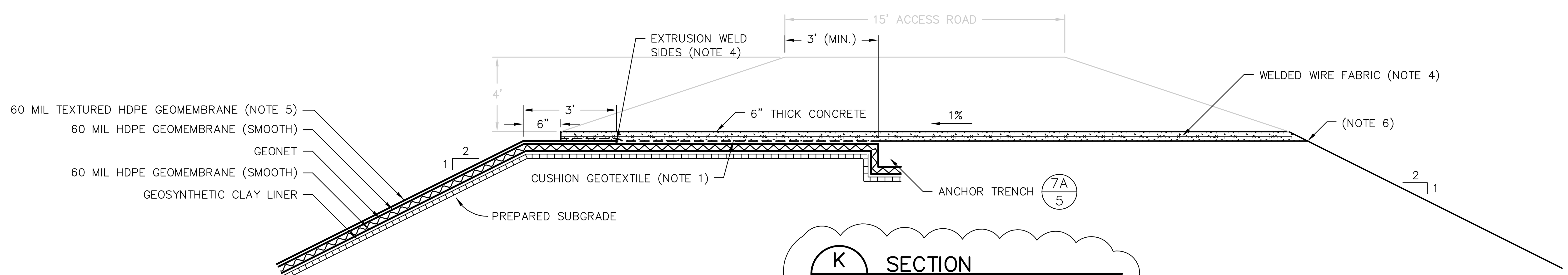
13/3,4 PLAN
PIPE SUPPORT
SCALE: 1" = 2'
XREF: 0349X004.DWG



J/8 SECTION
EMERGENCY SPILLWAY
SCALE: 1" = 4'
XREF: 0349X003.DWG

NOTES:

1. CUSHION GEOTEXTILE SHALL BE PLACED OVERLYING PRIMARY GEOMEMBRANE WHERE CONCRETE IS INSTALLED.
2. DETAILS ARE SHOWN TO SCALE INDICATED EXCEPT FOR THE GEOSYNTHETICS, WHICH ARE SHOWN AT AN EXAGGERATED SCALE FOR CLARITY SOIL THICKNESS ARE MINIMUMS.
3. EXPOSED PVC PIPE SHALL BE PAINTED TO MINIMIZE DAMAGE DUE TO UV.
4. WELDED WIRE FABRIC SHALL BE INSTALLED AT MIDSECTION OF CONCRETE SLAB.
5. SPLASH PAD AT SPILLWAY SHALL BE 100' WIDE, SHALL EXTEND 5' ONTO THE FLOOR AND BE EXTERIOR WELDED ON ALL FOUR (4) SIDES TO PRIMARY MEMBRANE.
6. CUT EXISTING GEOSYNTHETIC LINER SYSTEM AND FOLD BACK UNDER CONCRETE SPILLWAY.



K/8 SECTION
EMERGENCY SPILLWAY
SCALE: 1" = 4'
XREF: 0349X003.DWG

REV	DATE	DESCRIPTION	DRN	APP
1	6/18/08	INTERROGATORY ROUND 1	MD	GTC

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TITLE: LINING SYSTEM DETAILS IV

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.

SIGNATURE

DATE

DESIGN BY:	GTC	DATE:	DECEMBER 2007
DRAWN BY:	MAD	PROJECT NO.:	SC0349
CHECKED BY:	RF	FILE:	
REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC	8 OF 8	

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EXHIBIT I

REVISED CELL 4B DESIGN REPORT

Prepared for

Denison Mines (USA) Corp.
6425 S. Highway 191
P.O. Box 809
Blanding, UT 84511

CELL 4B DESIGN REPORT

WHITE MESA MILL
BLANDING, UTAH

Prepared by

Geosyntec consultants

engineers | scientists | innovators

10875 Rancho Bernardo Rd., Suite 200
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Project Number SC0349

December 2007
revised January 2009

Geosyntec consultants

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1. INTRODUCTION

This report presents the results of design analyses performed in support of the Cell 4B construction at the White Mesa Mill Facility in Blanding, Utah (site). The San Diego office of Geosyntec Consultants, Inc. (Geosyntec) prepared this report for Denison Mines (USA) Corp. (DMC). This report was prepared by Ms. Rebecca Flynn of Geosyntec. Mr. Gregory Corcoran, P.E. of Geosyntec was in responsible charge and provided senior peer review of the work presented herein in accordance with the internal peer review policy of the firm.

1.1 Objective

The objective of this report is to present the components of Cell 4B and to demonstrate that the proposed Cell 4B design complies with the applicable regulatory standards for the State of Utah, the United States Nuclear Regulatory Commission, and the Federal Environmental Protection Agency (USEPA). In particular, the design is in accordance with the Utah Administrative Code (UAC) R317-6, and the Best Available Technology requirements mandated by Part I.D. of existing site Ground Water Discharge Permit No. UGW370004.

1.2 Background

Current site operations utilize Cells 1 and 3 for process liquids evaporation and disposal of tailings and by-products from the processing operations at the site. Adjacent to the proposed Cell 4B is Cell 4A which began construction in July 2007 and became active in 2008. Construction of Cell 4B is expected to begin in spring 2009 to provide additional capacity for site operations. Cell 4B will similarly be used as a tailings disposal cell for evaporation of process liquids and final storage of solids contained in the tailings and by-products from processing operations at the site.

1.3 Report Organization

The remainder of this design report is organized into the following sections:

- Section 2, *Background and Site Conditions*, presents general information on the site and background information on the existing conditions at Cell 4B.
- Section 3, *Design*, presents the design for Cell 4B. The Construction Drawings are presented in Appendix A.
- Section 4, *Summary and Conclusions*, presents the summary, conclusions, and limitations of this technical design report.

In addition to this report, Cell 4B permit documents include Construction Drawings (Appendix A), a Construction Quality Assurance (CQA) Plan (Appendix B), Technical Specifications (Appendix C), engineering design calculations (Appendix D), and boring logs and geotechnical laboratory data (Appendix E).

2. BACKGROUND AND SITE CONDITIONS

2.1 Site Location

The location of the site is shown on Sheet 1 of the Construction Drawings (Appendix A). The site is located approximately 6 miles south of Blanding, Utah on Highway 191. Per the Universal Transverse Mercator (UTM) Coordinate System, the site is located at 4,159,100 meters Northing and 634,400 meters Easting.

The Mill is located on a parcel of fee land, State of Utah lease property and associated mill site claims, covering approximately 5,415 acres. The site mill operations are limited to approximately 50 acres located directly east of Cell 1. The existing tailings disposal Cells (Cells 1 through 3) are approximately 370 acres. Cell 4B is located south of the western half of Cell 3 and west of Cell 4A. The site plan is shown on Sheet 2 of the Construction Drawings.

2.2 Climatology

The climate of southeastern Utah is classified as dry to arid. Although varying somewhat with elevation and terrain, the climate in the vicinity of the site can be considered as semi-arid with normal precipitation of about 13.4 in (WRCC, 2005). Most precipitation is in the form of rain with snowfall accounting for about 30 percent of the annual precipitation total. There are two separate rainfall seasons in the region, the first in late summer and early autumn (August to October) and the second during the winter months (December to March).

The average temperature in Blanding ranges from approximately 30 degrees Fahrenheit (ºF) in January to approximately 76ºF in July. Average minimum temperatures are approximately 18ºF in January and average maximum temperatures are approximately 91ºF in July (City-Data.com, 2007).

The mean annual relative humidity is about 44 percent and is normally highest in January and lowest in July. The average annual Class I pan evaporation rate is 86 inches (WRCC, 2007), with the largest evaporation occurring in July. Values of pan coefficients range from 60 percent to 81 percent. The annual lake evaporation rate for the site is 47.6 inches and the net evaporation rate is 34.2 inches per year.

2.3 Topography

The existing topography within the Cell 4B area consists of a gently sloping grade (approximately 2 percent) from the northwestern portion of Cell 4B to the southeastern portion of Cell 4B. Existing Cell 3 south and Cell 4A west berms within the proposed Cell 4B are inclined at a slope of approximately 3 horizontal : 1 vertical (3H:1V).

2.4 Existing Soil Conditions

2.4.1 Surface Conditions

Currently, the proposed 4B Cell is undeveloped, with the exception of an unimproved access road, and covered by native low grass and shrub vegetation. The site is bordered to the north by the existing Cell 3, to the east by the existing Cell 4A, and to the south and west by undeveloped lands.

The existing ground surface within the area of the proposed Cell 4B slopes gently from northwest to south-southeast from respective elevations of approximately 5606 feet to 5570 feet, above Mean Sea Level (MSL).

2.4.2 Soil Berms

Soil berms exist on the eastern (Cell 4A) and northern (Cell 3) perimeters of the proposed Cell 4B. These berms were constructed previously of engineering fill.

2.4.3 Subsurface Conditions

Geosyntec performed a geotechnical investigation within the proposed limits of the Cell 4B (Figure 1). The geotechnical investigation consisted of a site reconnaissance, solid stem auger drilling, soil sampling, and geotechnical laboratory analysis of soil samples collected.

Soils encountered during drilling operations were consistent with formations in Southern Utah. Within the limits of the explorations, the site is underlain by surficial windblown loess and eolian deposits and variably weathered deposits of the Dakota Sandstone.

Loess and eolian deposits were encountered at the ground surface across the site extending to approximate depths of 4 to 13.5 feet. The deposit is thickest along the western portion of the site and thins to the east and southeast (Figure 2). The loess and eolian deposits are generally homogeneous across the site consisting of firm to stiff, yellowish red sandy clay (Unified Soil Classification System Classification CL). Boring logs and geotechnical laboratory results are presented in Appendix E.

The Dakota Sandstone underlies the surficial deposits at depth across the entire site area. The deposit generally exhibits a weathering rind approximately 0 to 5.5 feet thick consisting of dense to very dense, pale yellow to pink, silty fine sandstone with irregular zones of caliche accumulation. The unweathered Dakota Sandstone is encountered at approximately 6 to 15 feet below the ground surface. The deposit generally consists of very dense, very pale brown to white, fine grained sandstone with little silt.

2.5 Surface Water

Surface water at the facility is diverted around the Cells including Cell 4B. Surface water run-on into Cell 4B is limited to the perimeter access road surrounding the Cell and direct precipitation into Cell 4B.

The site has implemented a Storm Water Best Management Practices Plan in accordance with the facility permit. All site construction activities will be performed in accordance with the site Storm Water Best Management Practices Plan.

2.6 Groundwater

Groundwater is located at a depth of approximately 50 to 80 feet at the site. Monitoring well WMMW-16 is currently located within the proposed Cell 4B; therefore, during construction, WMMW-16 will be abandoned in accordance with the UAC R655-4-12. No additional changes to the existing groundwater monitoring plan are proposed by this project.

2.7 Tailings

Cell 4B will accept process liquids, tailings, and by-products associated with onsite processing operations. The liquids are typically highly acidic with a pH generally between 1 and 2. Tailings are generally comprised of ore that is ground to a maximum grain size of approximately 28 Mesh (US #30 Sieve) (0.023 inches (0.6 millimeters)), resulting in a fine sand and silt material.

3. DESIGN

The liner system is designed to provide a Cell for disposal of by-products from the onsite processing operations while protecting the groundwater beneath the site. The liner system is designed to meet the Best Available Technology requirements of the UAC R317-6, which require that the facility be designed to achieve the maximum reduction of a pollutant achievable by available processes and methods taking into account energy, public health, environmental and economic impacts, and other costs. The liner system includes the following primary components, from top to bottom:

- Slimes drain system;
- Primary geomembrane liner;
- Leak detection system;
- Secondary geomembrane liner; and
- Geosynthetic clay liner.

These components and related design considerations are discussed below.

3.1 Cell Capacity and Geometry

The cell has been designed to accommodate storage of up to approximately 1155 acre-feet (1.9 million cubic yards) of tailings with 3-feet of freeboard. The lowest elevation in Cell 4B is the sump located in the southeast corner at an elevation of approximately 5,556 feet above MSL.

Interior side slopes of Cell 4B will be constructed with 2H:1V inclinations. This will require re-grading of the western berm of Cell 4A and the southern berm of Cell 3, which currently have exterior side slopes of 3H:1V. The proposed southern berm of Cell 4B will have 2H:1V interior slopes and 3H:1V exterior slopes. A 15-foot wide unpaved access road is proposed to surround Cell 4B. Cell layout is shown on Construction Drawing Sheet 2, Site Plan.

3.2 Slope Stability

Static slope stability analyses were performed for the critical slopes for each of the four embankments surrounding Cell 4B. Analyses were performed for both static and pseudo-static conditions as well as addressing construction loading. In addition, slope stability analyses were performed on a typical cross section of the interim waste/tailings slopes. Final slope stability and operational conditions are required to maintain a minimum factor of safety of approximately 1.5 for final berm slope conditions, 1.3 for interim slope conditions, and 1.1 for seismically-loaded slope conditions based on the proposed design of the cell and its liner system. Numerous potential failure surfaces were performed to evaluate various slip surface geometries and to identify the critical slip surface for each cross-section and conditions.

Slope stability analyses indicate the factor of safety for each of the loading cases was met or exceeded in the analyses performed on the four embankment slopes. The complete calculation is presented in Appendix D.

3.3 Earthwork

Earthwork will consist of excavation, blasting, ripping, trenching, hauling, placing, moisture conditioning, backfilling, compacting, and grading. The requirements for earthwork for Cell 4B construction is provided in Appendix C, Section 02200 of the Technical Specifications.

3.3.1 Excavation

Prior to excavating soils and rock for Cell 4B, vegetation will be cleared and grubbed and surficial unsuitable materials will be removed. Excavation will proceed with the removal of in-situ soils for placement as fill for the construction of the Cell 4B south berm. Excess soils will be placed on Cell 3 as part of partial final closure or stockpiled to the west of the proposed limits of Cell 4B.

Rock will be ripped, blasted, or mechanically removed and stockpiled west of Cell 4B in a separate stockpile from the excess soil stockpile. Rock will be excavated a minimum of 6-inches below final grade and fill will be placed, moisture conditioned, compacted, and graded to provide a surface on which the geosynthetic liner system components will be installed.

Leak detection system and anchor trenches will be excavated as shown on the Construction Drawings (Appendix A).

3.3.2 Fill Placement

Along the southern perimeter of the proposed Cell 4B, a berm will be constructed of fill with 2H:1V inside slopes and 3H:1V outer slopes. Settlement analyses have been performed to evaluate the potential settlement of the berm and potential associated strain that could develop in the liner system components (Appendix D). The results of the conservative analyses indicate a maximum stress in the liner due to potential differential settlement of 0.01 percent, which is much less than the liner components can tolerate and is therefore acceptable.

Construction materials used for fill will consist of onsite soils placed in compacted lifts no greater than 8-inches and compacted to 90 percent of maximum dry density per American Society for Testing and Materials (ASTM) standard D1557 (Modified Proctor) at a moisture content of ±3 percent of optimum. Fill soil used in construction of the berm will consist of onsite soils with maximum particle size of 6-inches.

3.3.3 Subgrade Preparation

Subgrade preparation includes placement, moisture conditioning, compaction, and grading of subgrade soil. The subgrade will consist of a minimum of 6-inches of soil material with a maximum particle size of 3-inches compacted above the rock. Subgrade fill will be placed in loose lifts of no more than 12-inches and compacted to 90 percent of the maximum density at a moisture content of ±3 percent of optimum moisture content, as determined by ASTM D1557. The surface of the subgrade will have protrusions no greater than 0.5-inches. Section 02220 of the Technical Specifications, in Appendix C, provides the requirements for subgrade for Cell 4B construction.

3.3.4 Anchor Trench

The liner system will be anchored at the top of the slope with an anchor trench. The anchor trench was sized to resist anticipated maximum wind uplift forces, see Anchor Trench Capacity Calculations provided in Appendix D. The anchor trench will be 2 feet deep and 2 feet wide and filled with compacted soil, see Sheet 5 of the Construction Drawings (Appendix A). Anchor trench backfill will be placed in compacted lifts of no more than 8-inches and compacted to 90 percent of the maximum density at a moisture content of ±3 percent of optimum moisture content, as determined by ASTM D1557.

3.4 <u>Liner System</u>

A double liner system is proposed for Cell 4B, including a primary liner, leak detection system, and composite secondary liner. The liner system, for both the bottom area and side slopes, consists of (from top to bottom):

- Slimes Drain System (Cell bottom only);
- 60 mil smooth HDPE geomembrane (Primary Liner);
- 300 mil Geonet Drainage Layer (Leak Detection System);
- 60 mil smooth HDPE geomembrane;
- Geosynthetic Clay Liner (GCL); and } (Composite Secondary Liner)

- Prepared Subgrade.

Stability analyses were conducted to evaluate the various slip surface geometries and to identify the critical slip surfaces for two cross-sections and conditions. The analysis determined the minimum factor of safety of 1.3 will be met during and after filling operations. The complete calculation is located in Appendix D.

3.4.1 Slimes Drain System

A slimes drain system will be placed on top of the primary geomembrane liner in the bottom of the cell to facilitate dewatering of the tailings prior to final reclamation of the cell. The slimes drain system is designed to meet the performance standards in Part I.D.6 of the Groundwater Discharge Permit. The slimes drain system will consist of perforated 4-inch diameter schedule 40 polyvinyl chloride (PVC) pipe, concrete sand filled sand bags, drainage aggregate, cushion geotextile, filter geotextile, and strip composite that will provide a means to drain the tailings disposed within Cell 4B. The slimes drain system is shown on Sheets 4, 5, 6, and 7 of the Construction Drawings (Appendix A).

The slimes drain system is designed to remove the liquids within Cell 4B in a reasonable time. Based on the calculations presented in Appendix D, the slimes drain is expected to drain the tailings in approximately 5.5 years. A sump pump capable of pumping 18.1 gallons per minute (gpm) will be required upon start-up of the slimes drain system. The pumping rate is anticipated to decrease with time as the head within Cell 4B decreases.

The perforated PVC pipe is designed to resist crushing and wall buckling due to the anticipated loading associated with the maximum height of overlying tailings. The design analyses for the pipe are presented in Appendix D, while Appendix C, Section 02616 provides material specifications for the pipe and strip composite and Section 02225 provides material specifications for the drainage aggregate. The strip composite will be comprised of a 1-inch thick by 12-inch wide high density polyethylene, or equivalent acid resistant material, wrapped in a nonwoven polypropylene geotextile. The drainage aggregate will consist of a crushed rock that has a carbonate content loss of no more than 10 percent by weight.

A continuous row of sand bags filled with a concrete sand meeting Utah Department of Transportation (UDOT) standard specifications for Portland Cement Concrete will overlie the strip composite laterals to act as an additional filter layer above the geotextile component of the strip composite. The proposed UDOT concrete sand will be placed in sand bags consisting of woven geotextile capable of allowing liquids to pass. When placed overlying the strip composite, the sand bags will have an approximate length of 18 inches, width of 12 inches, and a height of 3 inches. This results in a sand bag that is approximately 30 to 35 pounds and will provide sufficient coverage over the width and ends of the strip composite to act as an additional filter layer. The UDOT concrete sand will consist of sand that has a carbonate content loss of no more than 10 percent by weight.

The cushion geotextile that is to be installed beneath the drainage aggregate surrounding the PVC pipe is designed to protect the underlying primary high density polyethylene (HDPE) geomembrane from puncture due to the drainage aggregate and the anticipated loading associated with the maximum height of overlying tailings. The design analyses for the cushion geotextile are presented in Appendix D, while Appendix C, Section 02771 provides material specifications. Overlying the drainage aggregate will be a woven geotextile, as shown on the Construction Drawings (Appendix A), that will serve to separate the tailings and the drainage aggregate.

The Slimes Drain sump will include a side slope riser pipe to allow installation of a submersible pump for manual collection of liquids in the sump. The sump and riser pipes are shown on Sheet 6 of the Construction Drawings (Appendix A).

3.4.2 Primary Liner

The primary liner will consist of a smooth 60-mil HDPE geomembrane. The geomembrane will have a white surface that will limit geomembrane movement and the creation of wrinkles due to temperature variations. HDPE geomembrane was selected due to its high resistance to chemical degradation and ability to retain durability in an acidic environment. The limit of the liner system (both primary and secondary) and details are shown on Sheets 2, 3, and 4 of the Construction Drawings (Appendix A).

Tension due to wind up lift was analyzed for the 60-mil HDPE geomembrane. Based on the analysis, the geomembrane anchor trench has been sized to accommodate the loading associated with a wind speed of 25 miles per hour and a slope length of approximately 92 feet. The design analyses for the HDPE liner are presented in Appendix D.

The HDPE geomembrane will be constructed in accordance with the current standard of practice for geomembrane liner installation, as outlined in the site Technical Specifications (Appendix C, Section 02770) and the site CQA Plan (Appendix B). Seams will be welded to provide a continuous geomembrane liner. Testing during construction will include both non-destructive and destructive testing, as outlined in the

Technical Specifications and CQA Plan. Upon completion of construction, the geomembrane manufacturer will provide a 20-year warranty for the geomembrane.

3.4.3 Leak Detection System

The leak detection system (LDS) will underlie the primary liner and is designed to collect potential leakage through the primary liner and convey the liquid to the sump for manual detection through monitoring of sump levels. The LDS consists of a 300-mil thick geonet and a network of gravel trenches throughout the bottom of Cell 4B. The trenches will contain a 4-inch diameter perforated schedule 40 PVC pipe, drainage aggregate, and a cushion geotextile, which will drain to a sump located in the southwest corner of the cell. The trenches will aid in rapidly conveying leakage to the LDS sump. The LDS is shown on Sheets 4, 5 and 6 of the Construction Drawings (Appendix A).

The Action Leakage Rate (ALR) was calculated for the LDS in accordance with Part 254.302 of the USEPA Code of Federal Regulations. The ALR was calculated to be 581 gallons per day per acre and the total travel time for liquids entering the geonet LDS layer to travel from the leak to the LDS piping system was estimated to be approximately 17 hours. Assuming a worst case scenario under which all the primary geomembrane defects are located at the high end of the leakage collection layer slope, the liquid head on the secondary liner does not exceed 0.006 inches (0.15 mm), well below the required maximum limit of 12 inches (1-foot). The geonet provides sufficient flow rate to accommodate the ALR. The complete ALR calculation is located in Appendix D and Section 02773 of Appendix C provides material specifications for the geonet.

The perforated PVC pipe is designed to resist crushing and wall buckling due to the anticipated loading associated with the maximum height of overlying tailings. Pipe strength analysis indicated the 4-inch PVC pipe with a maximum allowable deflection of 7.5 percent will have the ability to resist the anticipated maximum load associated with a tailing deposit height of 45 feet. The design analysis for the pipe is presented in Appendix D, while Appendix C, Section 02616 provides material specifications for the pipe and Section 02225 provides material specifications for the drainage aggregate.

The cushion geotextile is designed to protect the underlying secondary HDPE geomembrane from puncture due to the drainage aggregate and the anticipated loading associated with the maximum height of overlying tailings. Puncture analysis indicated a 16 ounce per square yard (oz./yd^2) cushion geotextile and ¾-inch maximum particle size would provide puncture protection for the 60-mil HDPE smooth geomembrane.

The design analyses for the cushion geotextile are presented in Appendix D, while Appendix C, Section 02771 provides material specifications.

The LDS sump will include a side slope riser pipe and submersible pump to allow for manual collection of liquids in the LDS sump. The LDS sump and riser pipes are shown on Sheet 6 of the Construction Drawings (Appendix A).

3.4.4 Secondary Composite Liner System

The primary purpose of the secondary liner is to provide a flow barrier so that potential leakage through the primary liner will collect on top of the secondary liner then flow through the LDS to the LDS sump for manual collection. The secondary liner also provides an added hydraulic barrier against leakage to the subsurface soils and groundwater. The secondary liner consists of a composite liner that includes a 60-mil HDPE geomembrane overlying a GCL.

3.4.4.1 Secondary Geomembrane Liner

The geomembrane component of the secondary liner system will consist of a smooth 60-mil HDPE geomembrane and will meet the same criteria as the primary liner geomembrane (Section 3.3.2). The limit of the liner system (both primary and secondary) and details are shown on Sheets 3, 5, and 6 of the Construction Drawings (Appendix A).

3.4.4.2 Secondary GCL Liner

The GCL component of the secondary liner system consists of bentonite sandwiched between two geotextile layers that are subsequently needle-punched together to form a single composite hydraulic barrier material. The GCL is approximately 0.2-inches thick with a hydraulic conductivity on the order of 1×10^{-9} cm per second (cm/s) (Daniel and Scranton, 1996).

Since 1986, GCLs have been increasingly used as an alternative to compacted clay liners (CCLs) on containment projects due to their low cost, ease of construction/placement, and resistance to freeze-thaw and wet-dry cycles. In general, the USEPA and the containment industry accept that GCLs are hydraulically equivalent to a minimum of 2 feet of compacted clay liner consisting of 1×10^{-7} cm/s soil materials.

For Cell 4A design, Geosyntec demonstrated that a secondary composite liner system consisting of a 60-mil HDPE geomembrane overlying a GCL has equivalent or better fluid migration characteristics when compared with a secondary composite liner system consisting of a 60-mil HDPE geomembrane overlying a CCL having a saturated hydraulic conductivity less than 1×10^{-7} cm/s (Geosyntec, 2006). This analysis accounted for the loading conditions and anticipated liquid head on the secondary liner system, the amount of flow through the secondary liner system with CCL was evaluated to be 4.37 times greater than flow through the secondary liner system with GCL for a liquid head of 0.20 inches, which is more than the calculated Cell 4B liquid head (0.006 inches). Therefore, in terms of limiting fluid flow through the composite secondary liner system, the secondary liner system containing a GCL performs better than the secondary liner system containing a CCL.

The following site specific conditions must be considered prior to use of a GCL in place of CCL (Koerner and Daniel, 1993):

- Puncture Resistance: While CCLs naturally provide greater puncture resistance than GCLs due to their inherent thickness, proper subgrade preparation and design of the geotextile components of the GCL can result in protection from puncture. The geotextile components of the GCL for Cell 4B are designed to protect the overlying secondary HDPE geomembrane from puncture due to protrusions from the subgrade and the anticipated loading associated with the maximum height of overlying tailings. The puncture protection analysis of the GCL indicated that a 3 oz/yd^2 geotextile and 6 oz/yd^2 geotextile above and below (respectively) the GCL and a maximum subgrade protrusion height of ½-inch will provide puncture protection for the secondary HDPE geomembrane. The design analyses for the geotextile components of the GCL are presented in Appendix D, while Appendix C, Section 02772 provides material specifications.
- Hydraulic Conductivity: Due to the acidic nature of the fluid to be stored in the cell, Geosyntec conducted hydraulic conductivity testing on hydrated specimens of GCL for the Cell 4A project (Geosyntec 2007) and Cell 4B project. Based on the results, the GCL will not be hydrated prior to deployment of the overlying secondary geomembrane.
- Chemical Adsorption Capacity: Due to the thickness of a CCL, the chemical adsorption capacity of a CCL is greater than that of a GCL. However, adsorption capacity is only relevant in the short term and not considered a parameter for steady-state analyses.

- Stability: The internal strength of a GCL can be significantly lower than that of a CCL, especially at high confinement stresses. This reduced strength can have significant effects on stability, especially at disposal facilities with high waste slopes and the potential for seismic activity. Strength of the GCL and its effects on stability are not a concern at Cell 4B due to the low confining stresses expected and geometry of the cell. Waste deposits will not be placed above the elevation of the perimeter road. Since no above grade slopes will be present, there are no long term destabilizing forces on the liner system.

- Construction Issues: For the Cell 4B liner system, GCLs may be considered superior to the CCLs with respect to construction issues. Construction of GCLs is typically much quicker and is more easily placed than a CCL, which requires moisture conditioning and compaction for placement. Further, CQA testing for a GCL is much simpler and less affected by interpretation of field staff than that for a CCL, which requires careful control of material type, moisture conditions, clod size, maximum particle size, lift thickness, etc.

- Physical/Mechanical Issues: Physical and mechanical issues include items such as the effect of freeze/thaw and wetting/drying cycles. CCLs may undergo significant increases in hydraulic conductivity as a result of freeze/thaw. Existing laboratory data suggests that GCLs do not undergo increases in hydraulic conductivity as a result of freeze/thaw. CCLs are also known to form desiccation cracks upon drying which can result in significant increases in hydraulic conductivity. This increase drastically jeopardizes the effectiveness of the CCL as a barrier layer. Available laboratory data on GCLs indicates that upon re-hydration after desiccation, GCLs swell and the cracks developed during drying cycles are 'self-healed'. Due to the arid environment at the site, GCL performance in the Cell 4B liner system with respect to physical and mechanical issues is expected to be superior to that of a CCL.

Based on review of the above site-specific considerations, a GCL is considered superior to a CCL for use in the secondary composite liner system.

3.5 Splash Pad

Approximately ten splash pads will be constructed to allow filling of Cell 4B without damaging the liner system. The splash pads consist of an additional geomembrane placed along the side slope of the Cell extending a minimum of 5 feet from the toe of the slope. The geomembrane will protect the underlying liner system from contact with the inlet pipes. A cross section of a typical splash pad is shown on Sheet 6 of the Construction Drawings (Appendix A).

3.6 Emergency Spillway

An emergency spillway will be constructed between Cells 4A and 4B. The spillway will be approximately 4 feet deep with 8H:1V approach pads that will allow traffic moving along the top of the berm to pass through the spillway (when dry). The spillway will consist of a 6-inch thick reinforced concrete pad, designed to withstand loadings from pick-up truck traffic, see Concrete Calculations provided in Appendix D. The spillway is designed to handle the Probable Maximum Precipitation (PMP) for a 6 hour storm event for the site, see Spillway Calculations provided in Appendix D. The Cell 4B liner will extend beneath the concrete as shown on Sheet 7 of the Construction Drawings (Appendix A). In addition, a 100' wide splash pad will be installed overlying the primary geomembrane liner beneath the emergency spillway exit.

4. SUMMARY AND CONCLUSIONS

This report presents the engineering design evaluations for Cell 4B at the White Mesa Mill Facility. The calculations presented in this Design Report establish the dimensions and properties of the liner system components (Appendix D). The design plans and details are presented in the Construction Drawings (Appendix A), recommended construction quality testing and observation requirements are provided in the CQA Plan (Appendix B), and material requirements are provided in the project Technical Specifications (Appendix C).

4.1 Limitations

The professional opinions and recommendations expressed in this report are made in accordance with generally accepted standards of geotechnical practice. This warranty is in lieu of any other warranty either express or implied. We are responsible for the conclusions and recommendations contained in this report based on the data relating only to the specific project and location discussed herein. We are not responsible for use of the information contained in this report for purposes other than those expressly stated in this report. In the event that there are changes in the design or location of this project that do not conform to the project as described herein, we will not be responsible for these changes unless given the opportunity to review them and concur with them in writing. We are not responsible for any conclusions or recommendations made by others based upon the data or conclusions contained herein unless given the opportunity to review them and concur with them in writing.

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EXHIBIT J

GCL PERMEABILITY TEST RESULTS AND CALCULATIONS

EXHIBIT J - GCL Permeability Results
White Mesa Mill Facility - Cell 4B

GCL Moisture Content	Pore Volumes	Permeability	GCL Thickness	Head Overlying GCL	Hydraulic Gradient	Area	Q	Q	Bentonite Porosity	Pore volume	time	time to pass 1 pore volume
(%)		(cm/sec)	(cm)	(cm)		(cm^2)	(cm^3/sec)	(cm^3/year)		(cm^3/cm^2)	(years)	
17	0.25	1.00E-08	0.76	0.017	0.022	1	2.20E-10	6.94E-03	0.7	0.13	18.74	269.72
17	0.50	5.00E-09	0.76	0.017	0.022	1	1.10E-10	3.47E-03	0.7	0.13	37.48	
17	0.75	3.20E-09	0.76	0.017	0.022	1	7.04E-11	2.22E-03	0.7	0.13	58.56	
17	1.00	4.00E-09	0.76	0.017	0.022	1	8.80E-11	2.78E-03	0.7	0.13	46.84	
17	1.50	6.00E-09	0.76	0.017	0.022	1	1.32E-10	4.16E-03	0.7	0.27	64.86	
17	2.00	9.00E-09	0.76	0.017	0.022	1	1.98E-10	6.24E-03	0.7	0.27	43.24	

TRI/ENVIRONMENTAL, INC.
A Texas Research International Company

October 28, 2008

Gregory T Corcoran, P.E.
Geosyntec Consultants, Inc.
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Suite 200
San Diego, California 92078
Phone: 858.674.6559
GCorcoran@Geosyntec.com

Subject: Results for permeability of the Bentomat ST GCL for the Denison Mines Project, (TRI Log #: E2308-67-06)

Dear Mr. Corcoran,

The intent of letter is to provide you with the results for the compatibility of the Bentomat ST GCL with the hydrochloric acid solution for the Denison Mines Project. Representative specimens of the Bentomat ST GCL from roll number 0046, were selected for permeability testing per ASTM D 6766, Scenario 1. Specimens were pre-hydrated with de-ionized water to target bentonite moisture contents of 17, 30 and 50%. Specimens were allowed to equilibrate prior to mounting in the triaxial permeameters.

Upon mounting in the permeameter the specimen was immediately tested for permeability without back pressure saturating the sample at the client's request. The cell pressure was 80 psi, the head water pressure was 77 psi and the tail water pressure was 75 psi. The specimens were permeated with a hydrochloric (HCl) acid solution with a pH of 1. Permeability with time and pore volumes is presented in the attached figures for each of the hydration conditions. Individual specimen data are presented in Table 1.

Table 1 GCL Specimen Data

Parameter	Specimens		
Target Moisture Content	17%	30%	50%
Initial Diameter, mm	101.6	101.6	101.6
Final Diameter, mm	101.6	101.6	101.6
Initial Thickness, mm	7.6	7.6	7.6
Final Thickness, mm	6.6	6.9	6.9
ω_i, %	17.1	30.0	50.0
ω_f, %	115.9	130.5	134.0
i_{avg}	227	221	217
n	0.70	0.70	0.71
Pore Volume, ml	37.2	39.3	39.3
K_f, (cm/sec)	2.4×10^{-8}	2.3×10^{-8}	2.7×10^{-8}

Sincerely,

John M. Allen, E.I.T.
Director of the Geosynthetics Interaction Laboratory
TRI/Environmental, Inc.

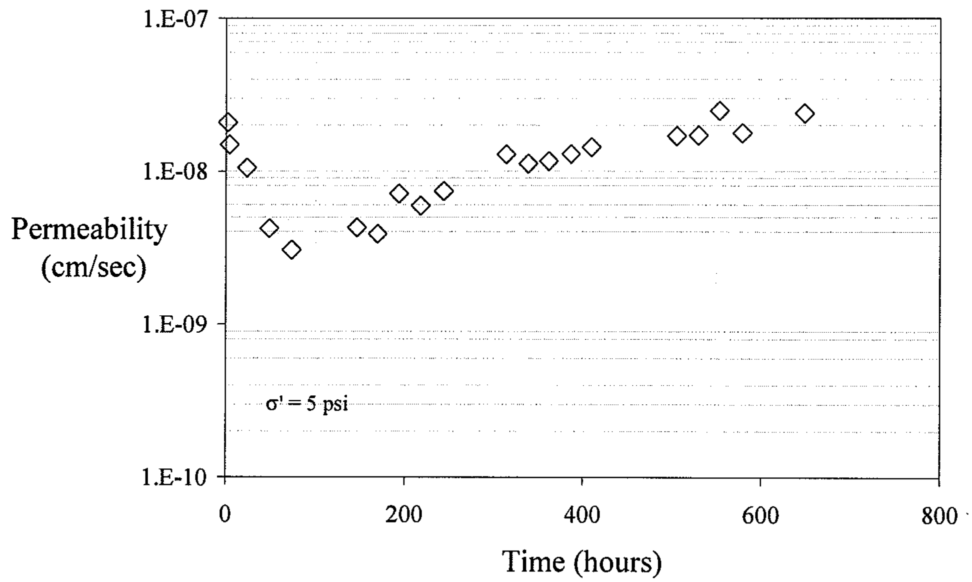


Figure 1 Permeability with time for 17% moisture content specimen with pH of 1 HCL Solution

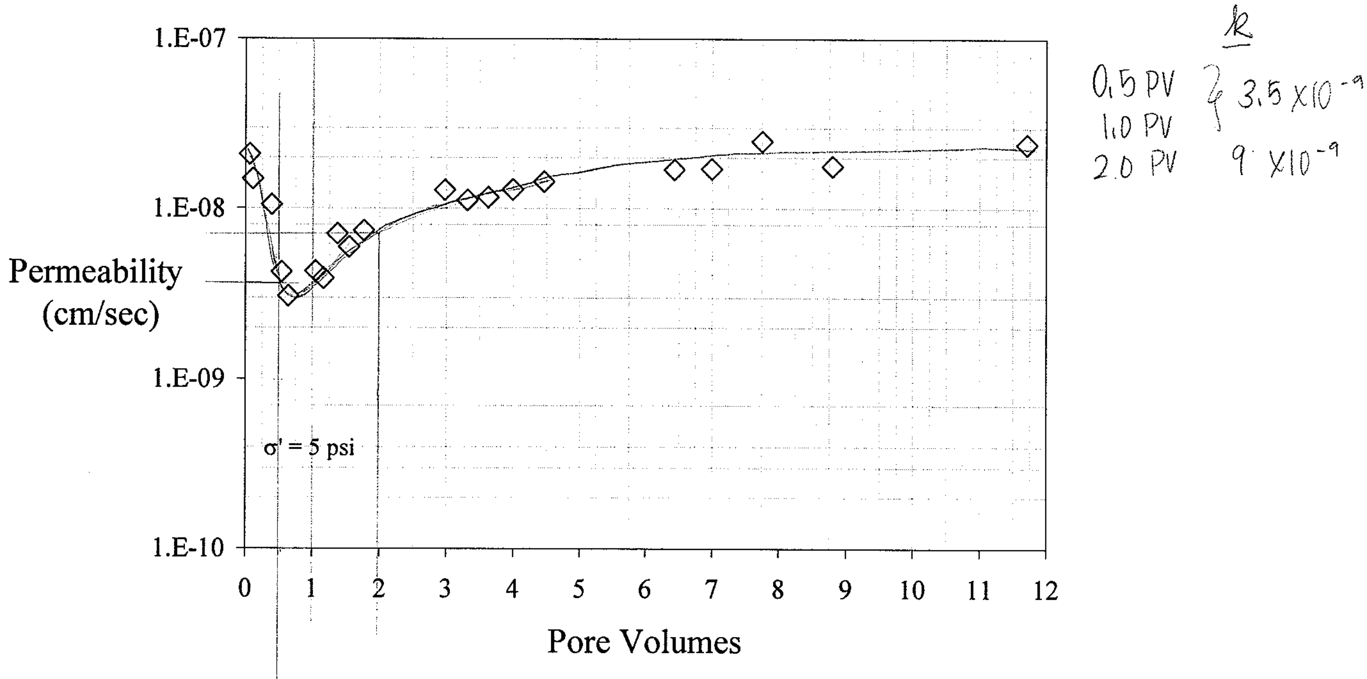


Figure 2 Permeability with pore volumes for 17% moisture content specimen with pH of 1 HCL Solution

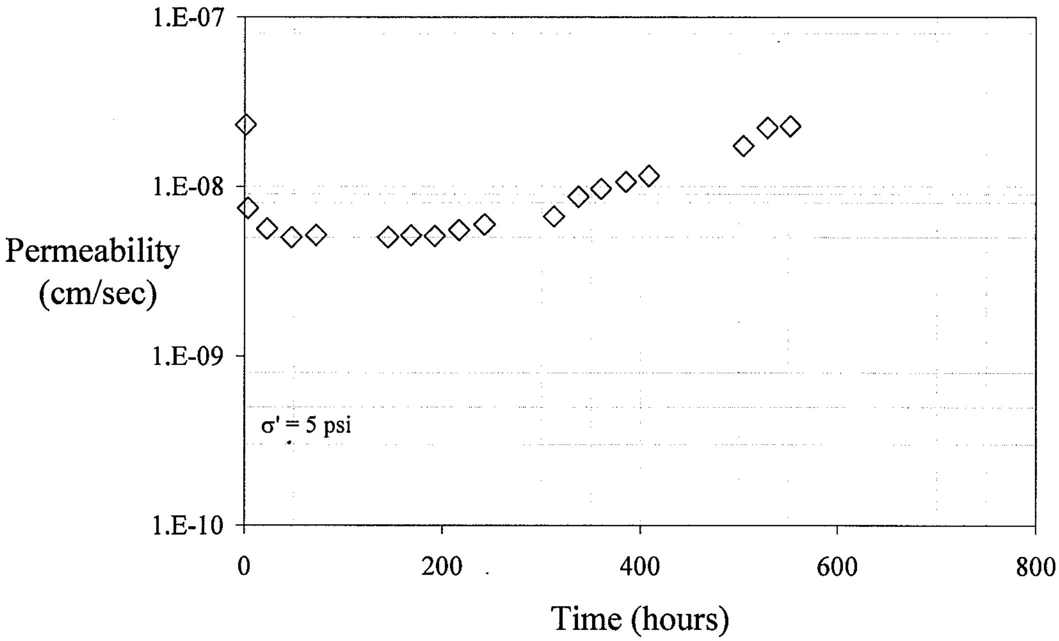
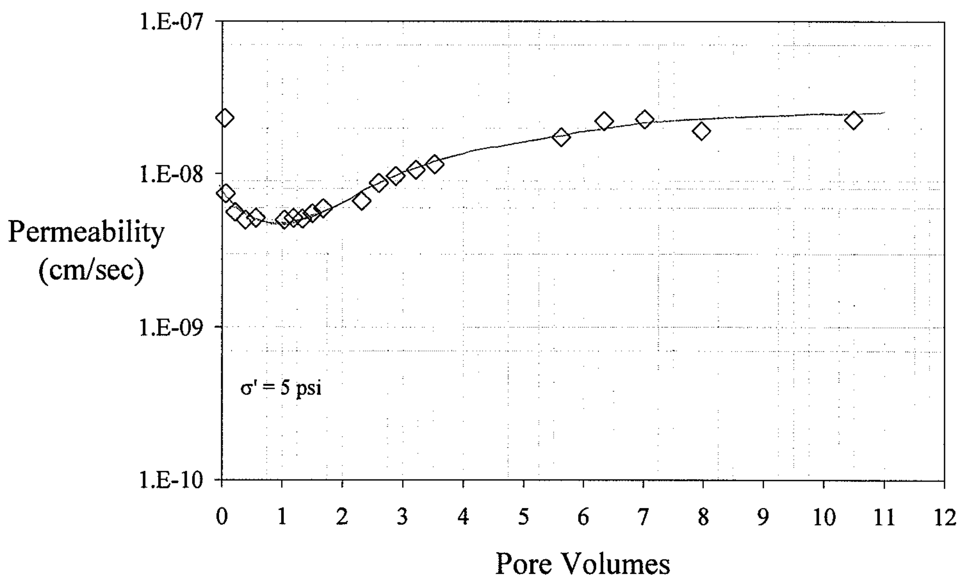


Figure 3 Permeability with time for 30% moisture content specimen with pH of 1 HCL Solution



5×10^{-9} 0.5 & 1

6×10^{-9} 2

Figure 4 Permeability with pore volumes for 30% moisture content specimen with pH of 1 HCL Solution

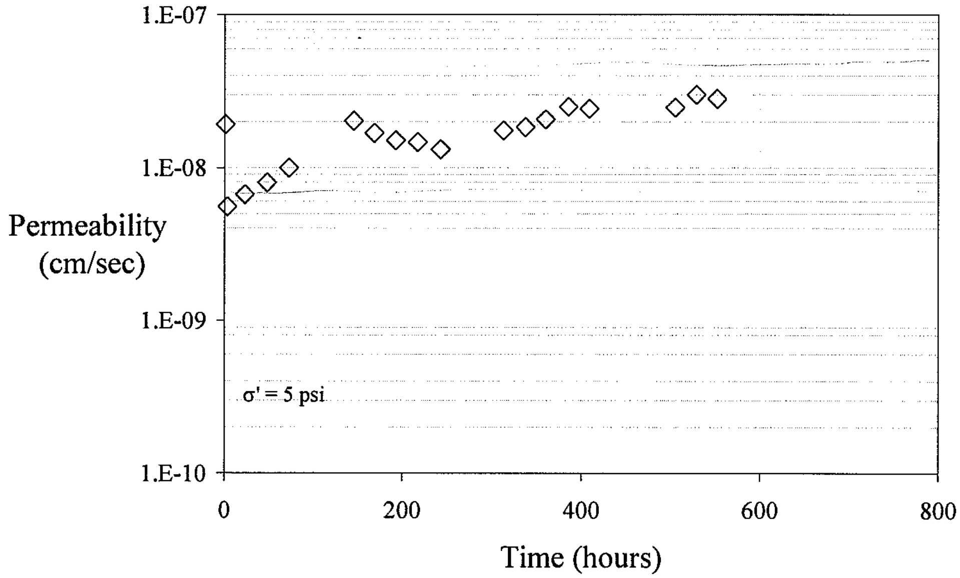


Figure 5 Permeability with time for 50% moisture content specimen with pH of 1 HCL Solution

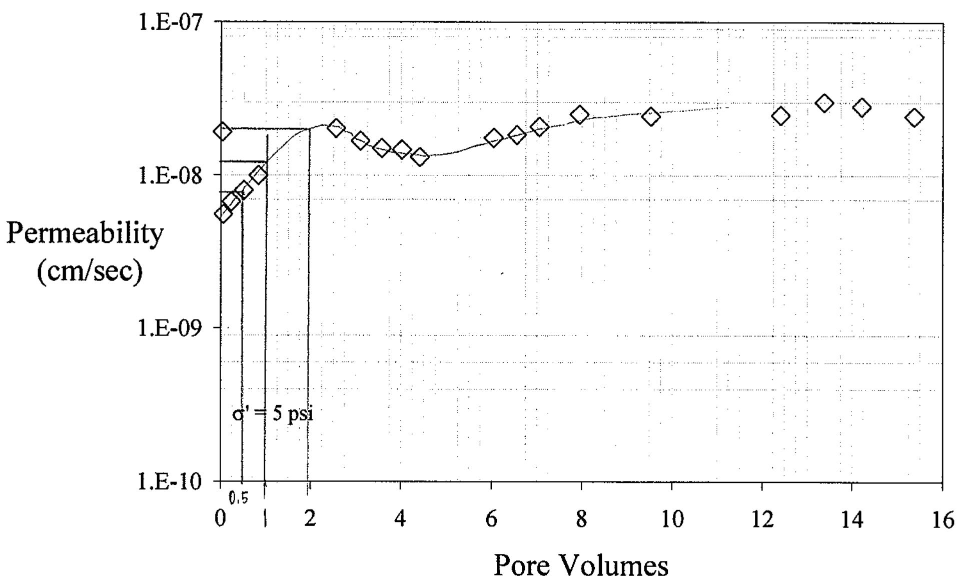
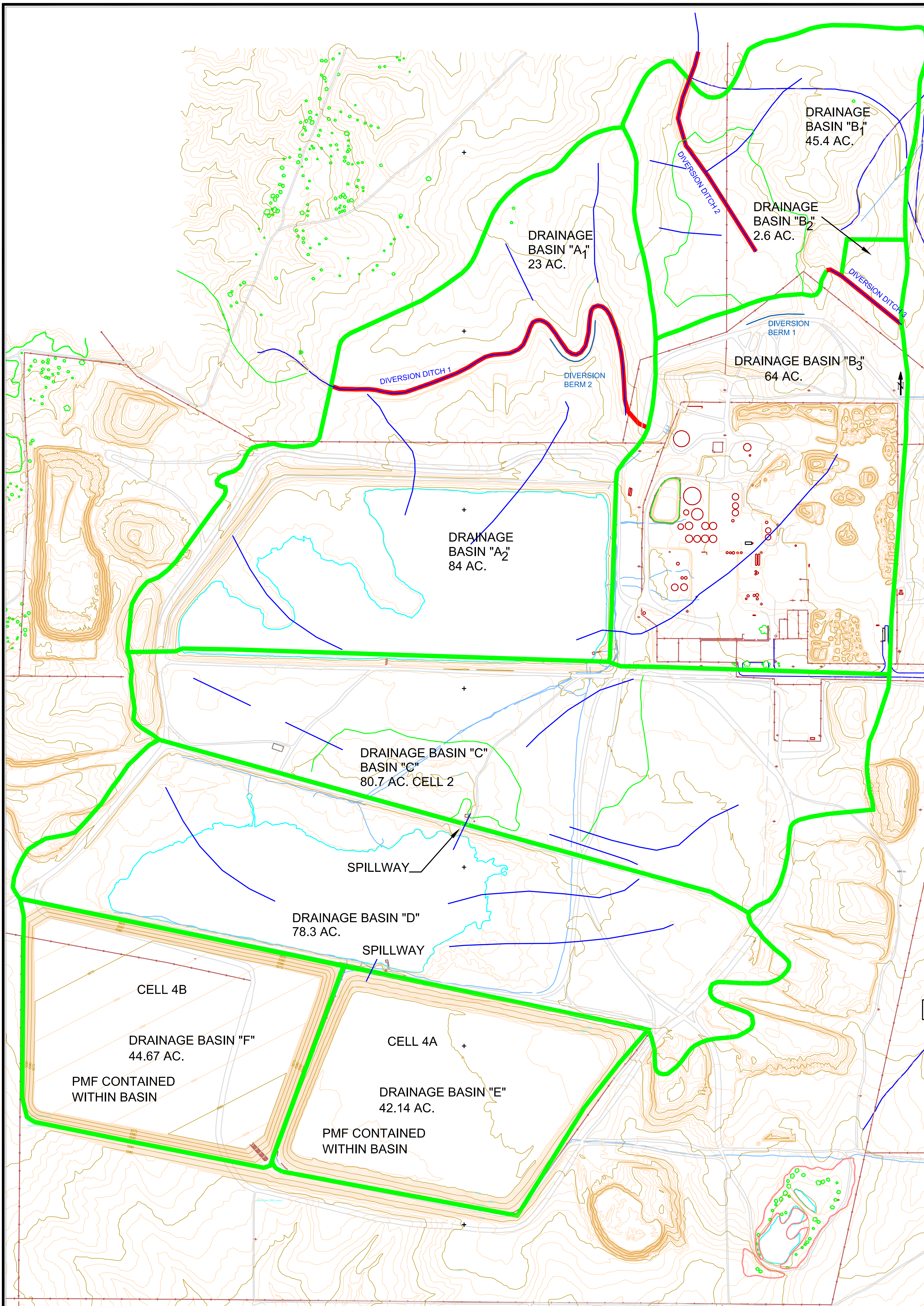
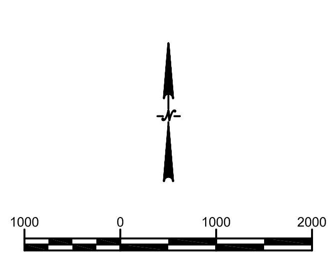


Figure 6 Permeability with pore volumes for 50% moisture content specimen with pH of 1 HCL Solution





Sufrace Water Flow
Drainage Basins
Diversion Ditches
Diversion Berm

Denison Mines (USA) Corp.

DENISON MINES

Project: WHITE MESA MILL

REVISIONS	
Date	By
2/15/07	BM
10/24/07	BM
05/16/08	BM
06/11/08	BM
12/9/08	DLS
1/7/09	BM

County: San Juan
State: Utah
Location:

Mill Site Drainage Basins

Scale: 1"=2000ft
Date: 2005
figure 2_1-6-09.dwg
Author: HRR
Drafted By: unknown

Figure 1