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CELL 4B DESIGN REPORT

WHITE MESA MILL
BLANDING, UTAH

Prepared by

Geosyntec consultants

engineers | scientists | innovators

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Project Number SC0349

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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 will become active in spring 2008. Construction of Cell 4B is expected to begin in spring 2008 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 (°F) in January to approximately 76°F in July. Average minimum temperatures are approximately 18°F in January and average maximum temperatures are approximately 91°F 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 analysis was conducted for the final earthen berms and interim waste/tailings slopes associated with the operation of Cell 4B. Final slope stability and operational conditions are required to maintain a minimum factor of safety of approximately 1.5 for final berm slope conditions and 1.3 for interim slope conditions based on the proposed design of the cell and its liner system.

Two cross-sections were analyzed which represent typical 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 analysis of the east-west cross section estimated a factor of safety of 2.0 and 1.6 for Cell 4B for empty and half-full conditions, respectively, while Cell 4A was full for both conditions. The second cross section was modeled to analyze the stability of the southern perimeter berm when Cell 4B full with tailings. The south side of the berm was conservatively modeled with outside berm slopes inclined at 2:1, rather than the current 3:1 berm slopes, to represent future build out conditions. Results from the analysis of Section B-B' indicate a minimum static factor of safety of approximately 2.1 under these conditions. 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 lifts no greater than 12-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 ±4 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 8-inches and compacted to 90 percent of the maximum density at a moisture content of ±4 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 02221 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 lifts of no more than 12-inches and compacted to 90 percent of the maximum density at a moisture content of ±4 percent of optimum moisture content, as determined by ASTM D1557.

3.4 Liner System

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 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 3, 5, and 6 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 2225 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). The GCL will be hydrated to account for the high acidity of the tailings.

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 8.51 times greater than flow through the secondary liner system with GCL for a liquid head of 0.16 inches, which is more than the calculated Cell 4B liquid head (0.0006 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 QA project (Geosyntec 2007). Based on the results, the GCL will be hydrated to a moisture content of 50% during construction.
- 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 three 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 5 of the

Construction Drawings (Appendix A). The locations of the splash pads will be finalized in the field during construction, based on site operational needs.

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).

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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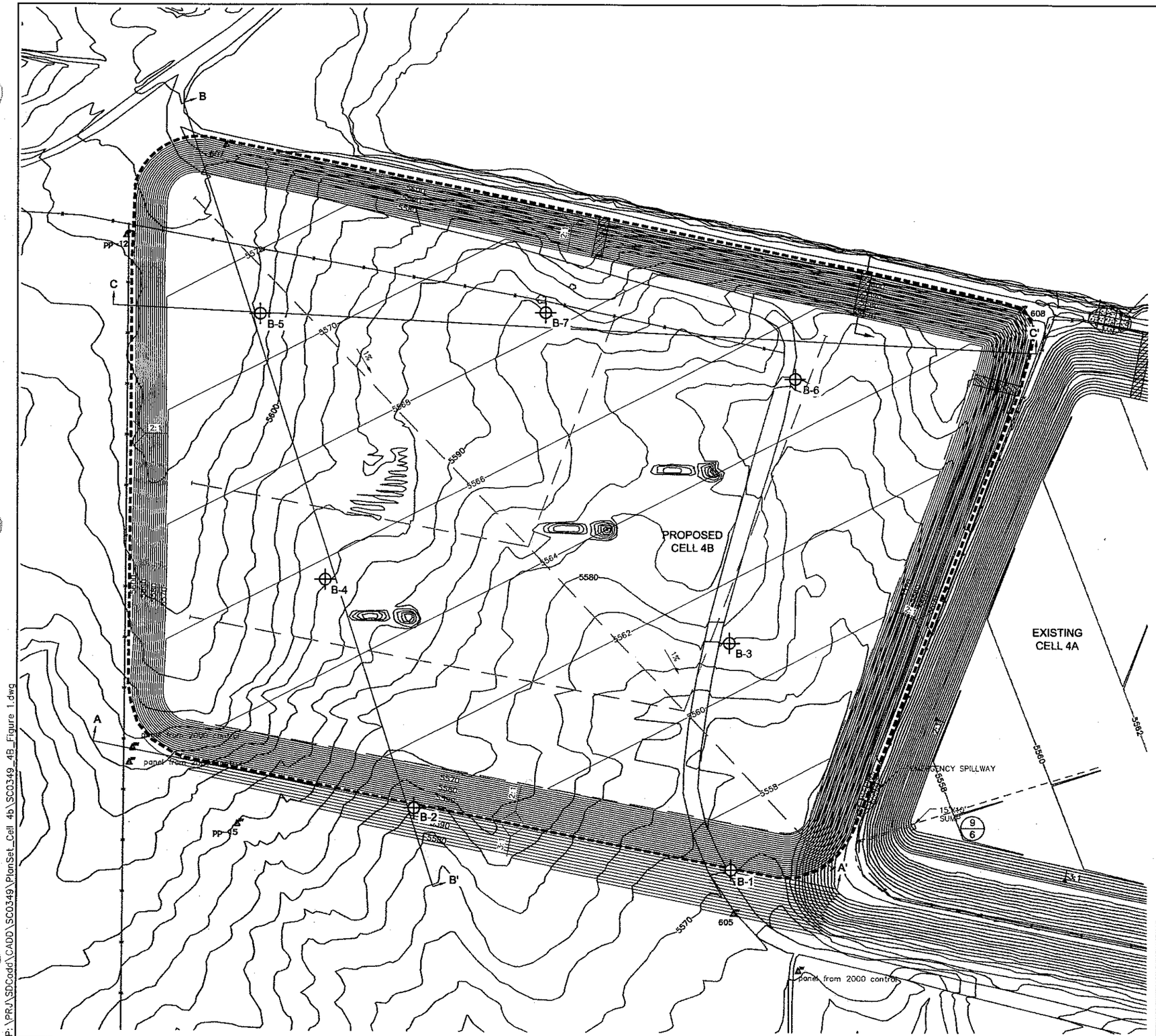
WRCC, 2007. Monthly Average Pan Evaporation Rate for Mexican Hat, Utah. Available at: www.wrcc.dri.edu/htmlfiles/westevap.final.html#utah

FIGURES



LEGEND

5560	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (2' CONTOUR)
1%	SLOPE DIRECTION AND GRADE
	EXPLORATORY BORING
A — A'	GEOLOGIC CROSS SECTION

N

200 100 0 200

SCALE IN FEET

PHASE 4B SITE PLAN
WHITE MESA MILL

Geosyntec consultants

DATE:	DECEMBER 2007	FIGURE
PROJECT NO.	SC0349	1

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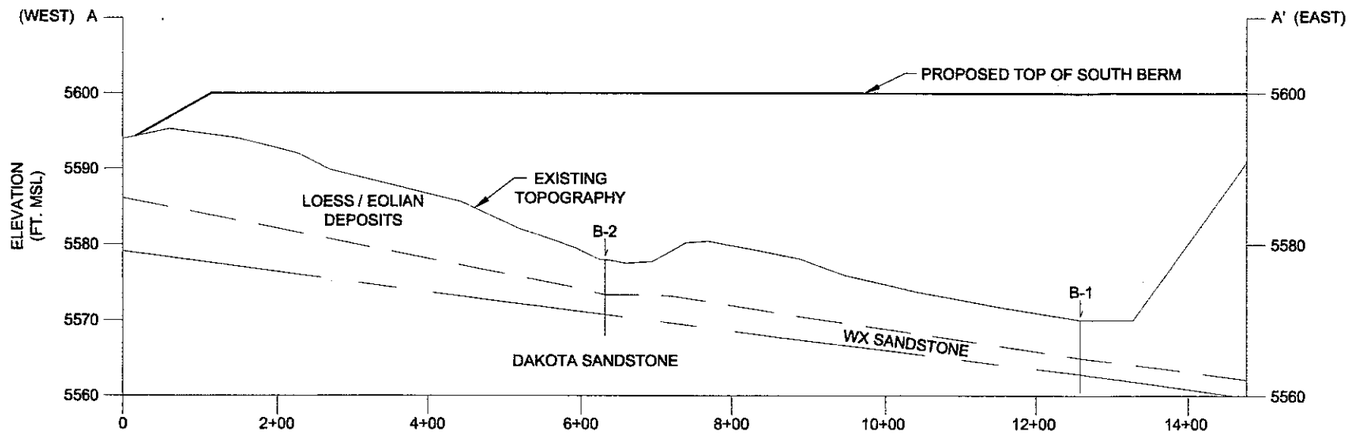

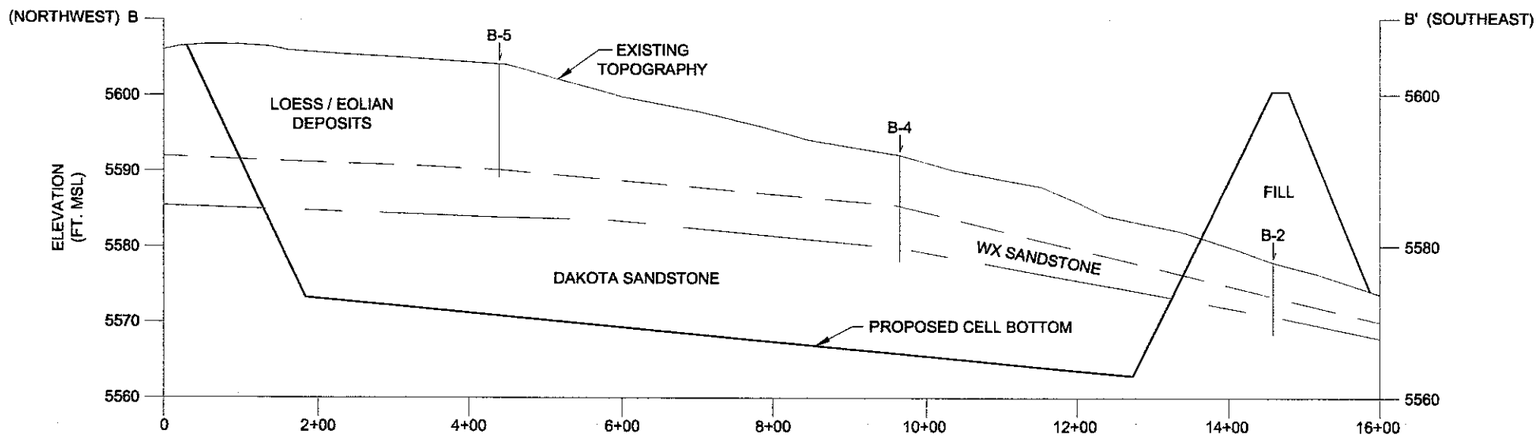

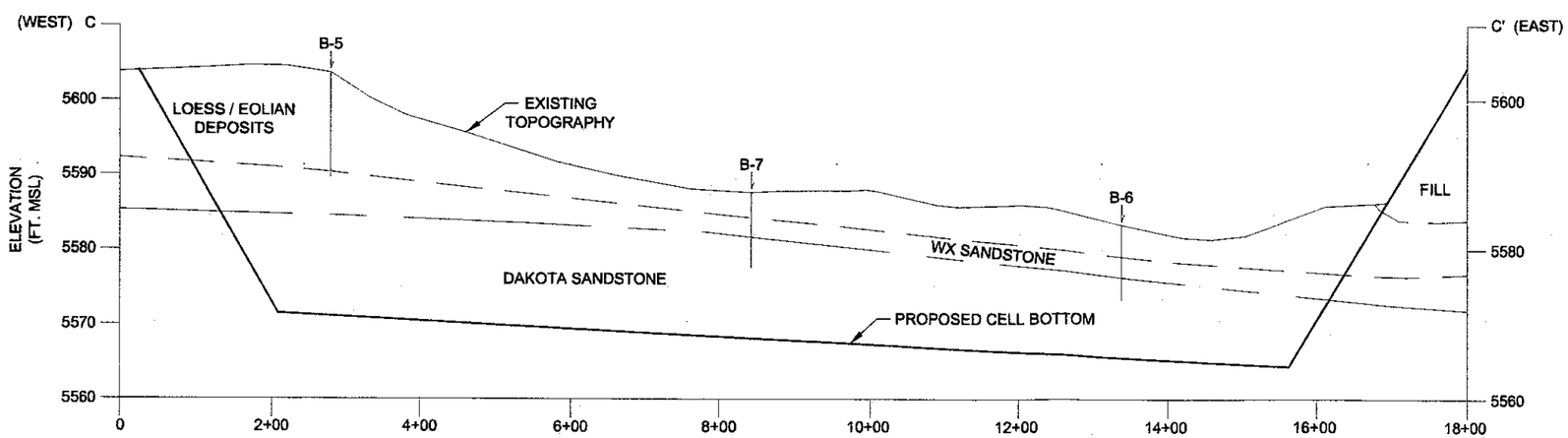

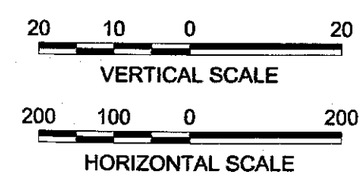

GEOLOGIC CROSS SECTION
WHITE MESA MILL

Geosyntec consultants

DATE: DECEMBER 2007		FIGURE
PROJECT NO. SC0349		2

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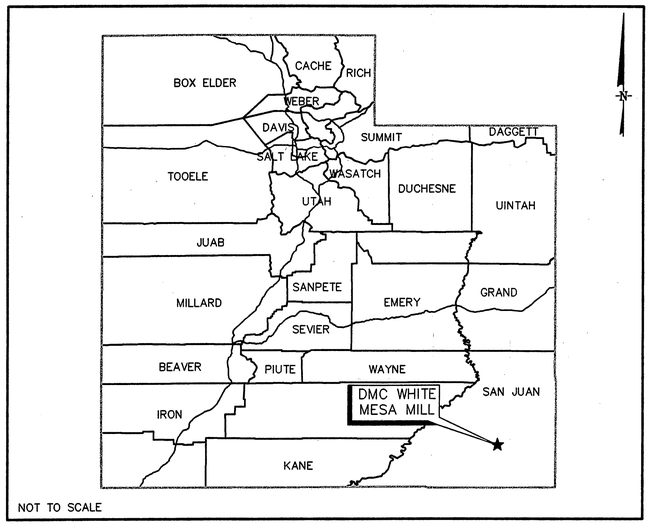
APPENDIX A

Construction Drawings

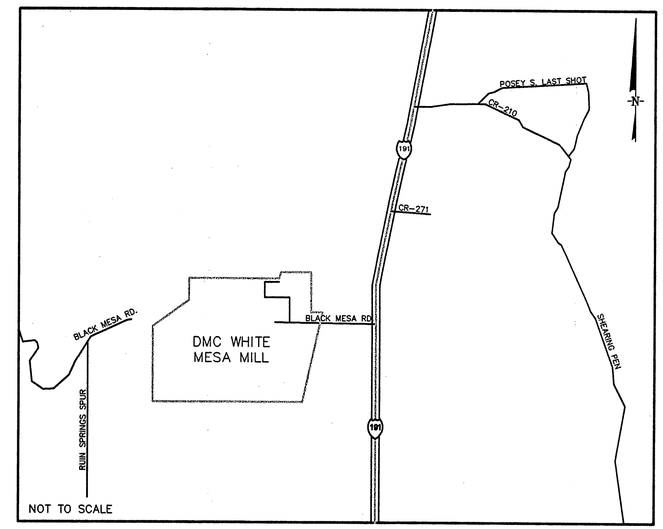
CONSTRUCTION DRAWINGS
DMC WHITE MESA MILL
CELL 4B LINING SYSTEM
BLANDING, UTAH
DECEMBER 2007



VICINITY MAP

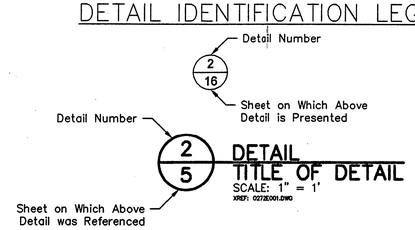
LIST OF DRAWINGS

DRAWING	DESCRIPTION
1	TITLE SHEET
2	SITE PLAN
3	BASE GRADING PLAN
4	PIPE LAYOUT PLAN AND DETAILS
5	LINING SYSTEM DETAILS I
6	LINING SYSTEM DETAILS II
7	LINING SYSTEM DETAILS III



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 (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

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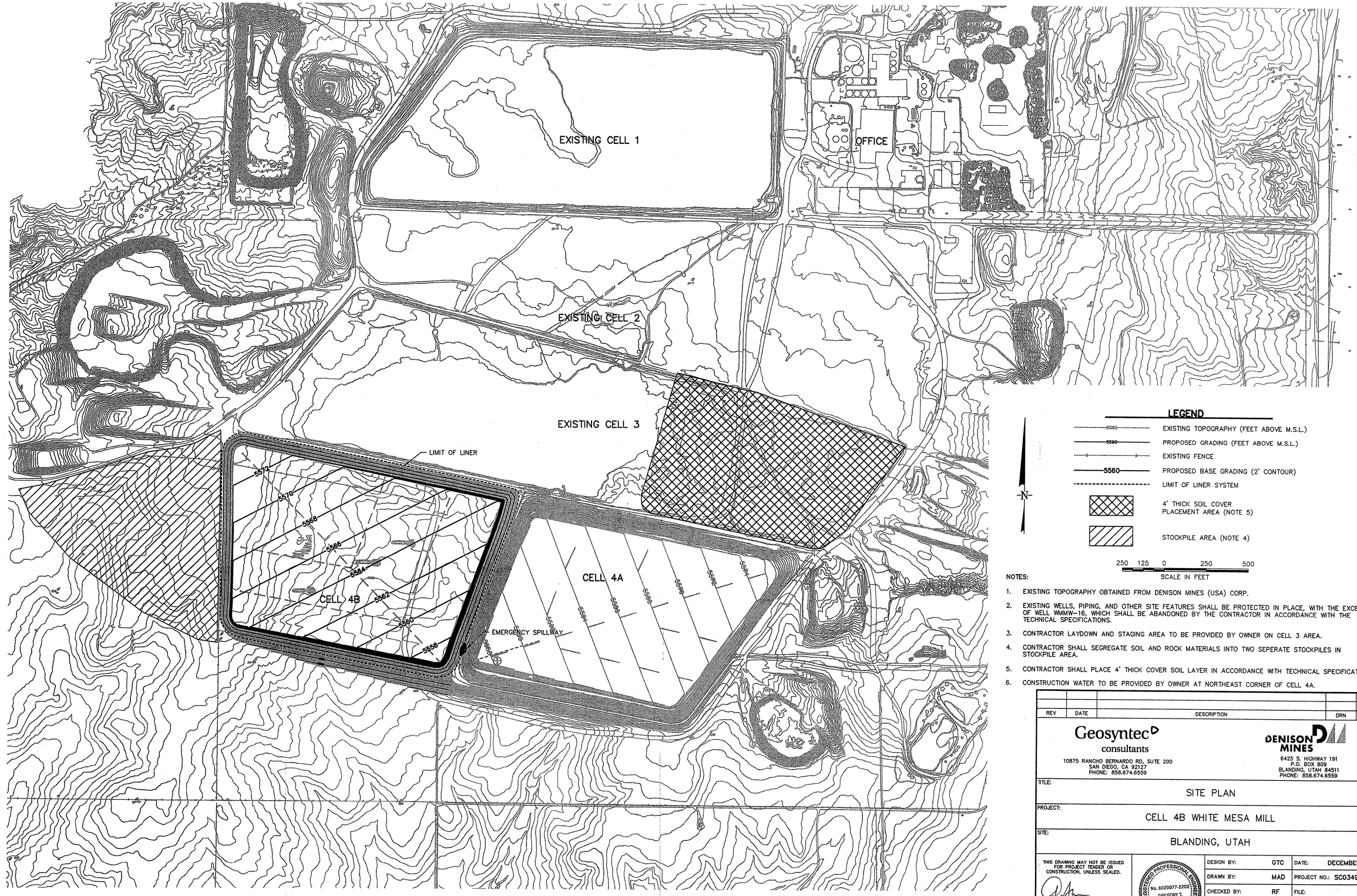

NOTES:

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. CONTRACTOR SHALL PLACE 4' THICK COVER SOIL LAYER IN ACCORDANCE WITH TECHNICAL SPECIFICATIONS.
6. CONSTRUCTION WATER TO BE PROVIDED BY OWNER AT NORTHEAST CORNER OF CELL 4A.

REV	DATE	DESCRIPTION	DRN	APP

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PHONE: 858.674.6559

TITLE: SITE PLAN

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

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SIGNATURE

12/5/07
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REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC	2 OF 7	

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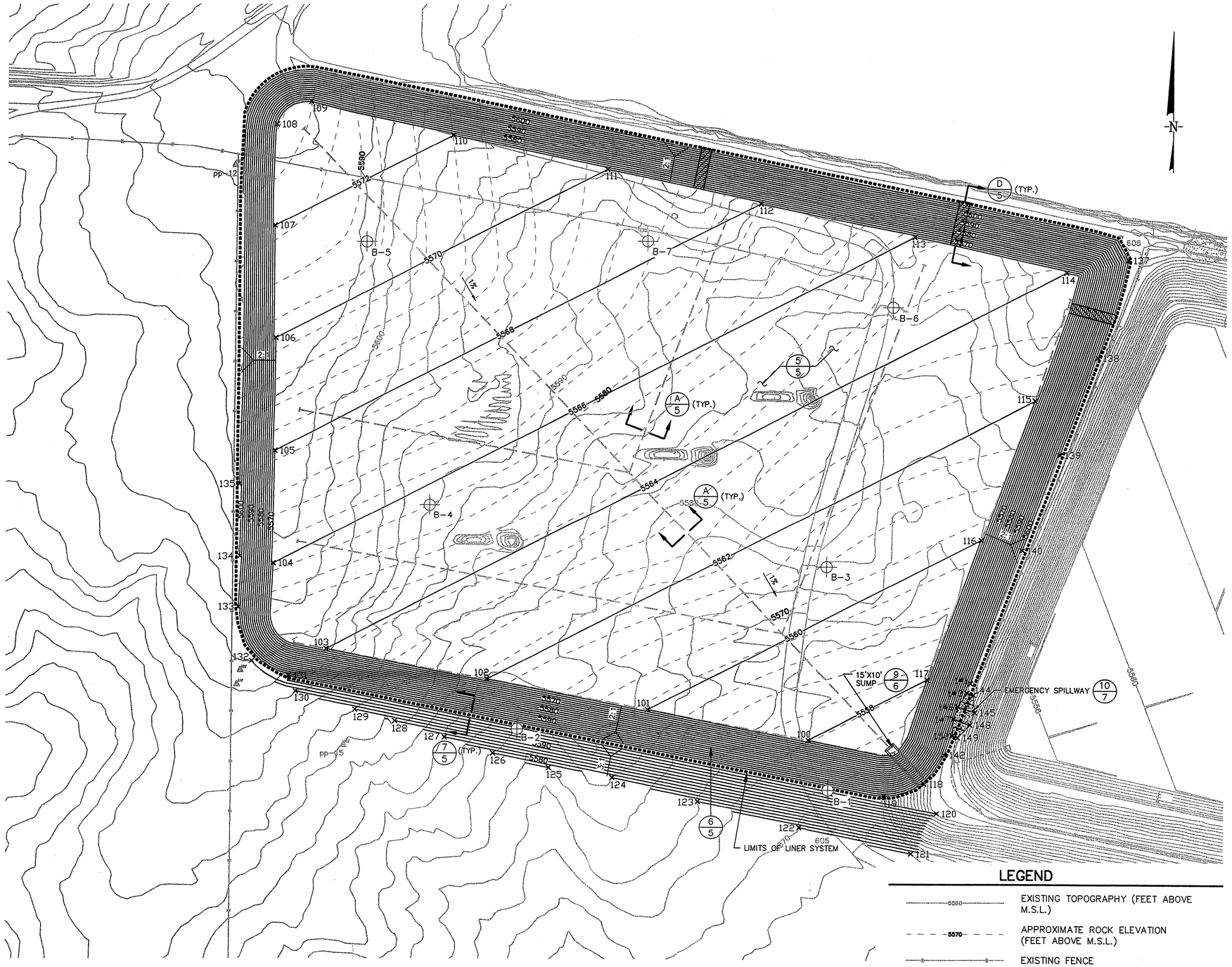

POINT TABLE			
POINT #	NORTHING	EASTING	ELEVATION
100	319447.2859	2577164.8929	5558
101	319507.2322	2576844.3805	5560
102	319567.1785	2576523.8681	5562
103	319627.1248	2576203.3557	5564
104	319797.6436	2576097.6307	5566
105	320023.8073	2576099.9958	5568
106	320249.0377	2576100.5478	5570
107	320472.5420	2576097.7471	5572
108	320675.6564	2576100.8126	5574
109	320719.8957	2576168.6204	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
146	319515.3611	2577463.1200	5596
147	319490.7627	2577454.7052	5596
148	319479.1526	2577487.4112	5596
149	319453.9250	2577462.8219	5600
150	319457.5650	2577452.5678	5600

100 50 0 100 200
SCALE IN FEET

LEGEND

- EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
- APPROXIMATE ROCK ELEVATION (FEET ABOVE M.S.L.)
- EXISTING FENCE
- PROPOSED BASE GRADING (10' CONTOUR)
- LIMIT OF LINER SYSTEM
- SPLASH PAD (NOTE 2)
- SLOPE DIRECTION AND GRADE
- APPROXIMATE SOIL BORING LOCATION
- LEAK DETECTION PIPE TRENCH

NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM DENISON MINES (USA) CORP.
2. SPLASH PAD LOCATION TO BE FIELD SELECTED BY OWNER.

REV	DATE	DESCRIPTION	DRN	APP

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BLANDING, UTAH 84511
PHONE: 858.674.6559

TITLE: BASE GRADING PLAN

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

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DATE

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APPROVED BY:	GTC		3 OF 7

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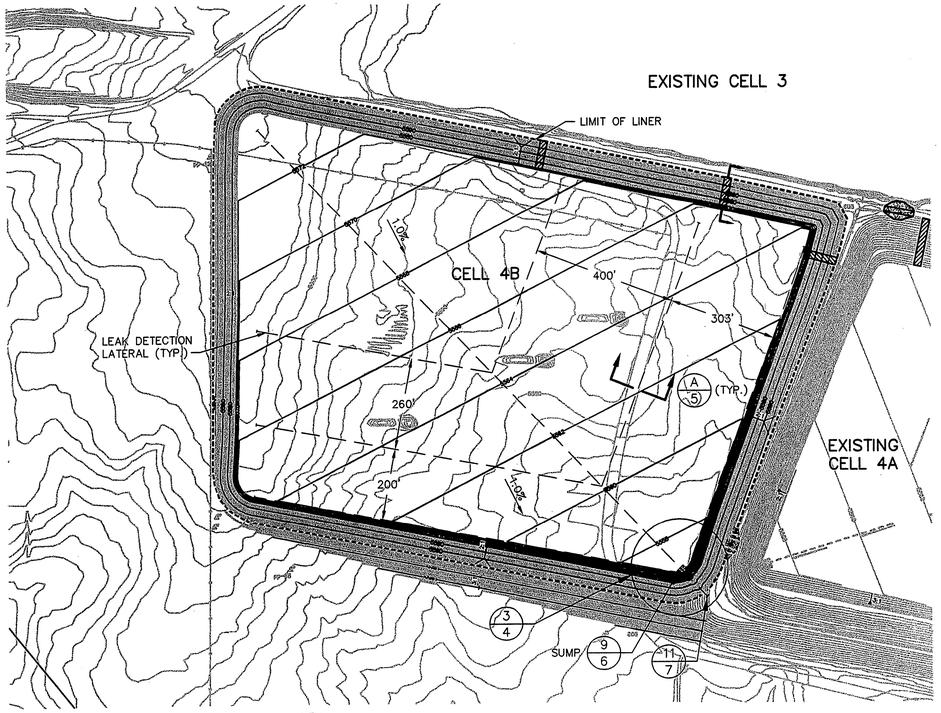
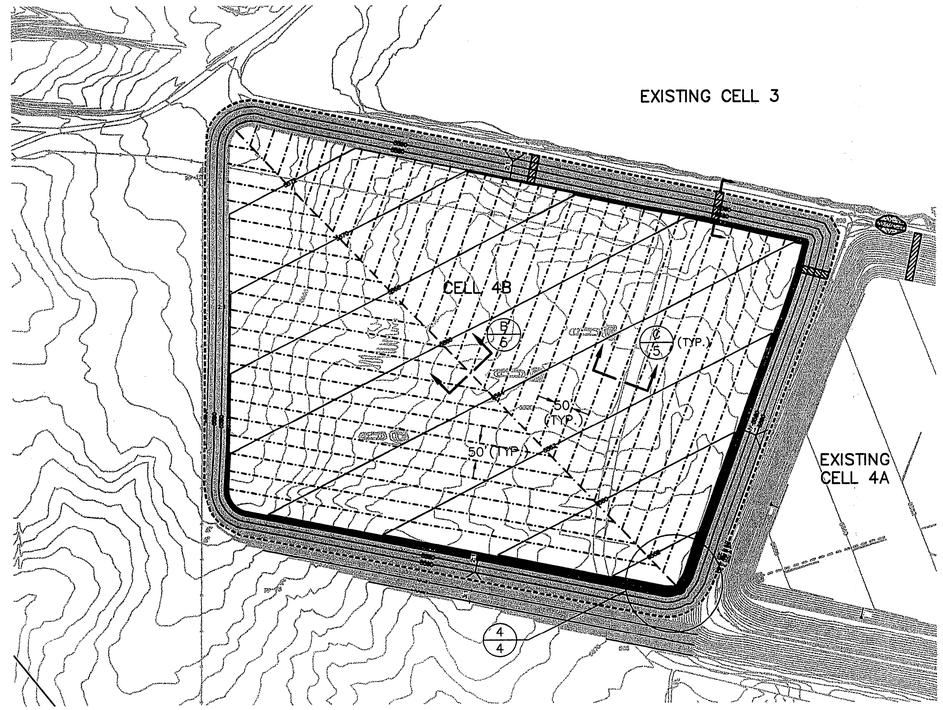


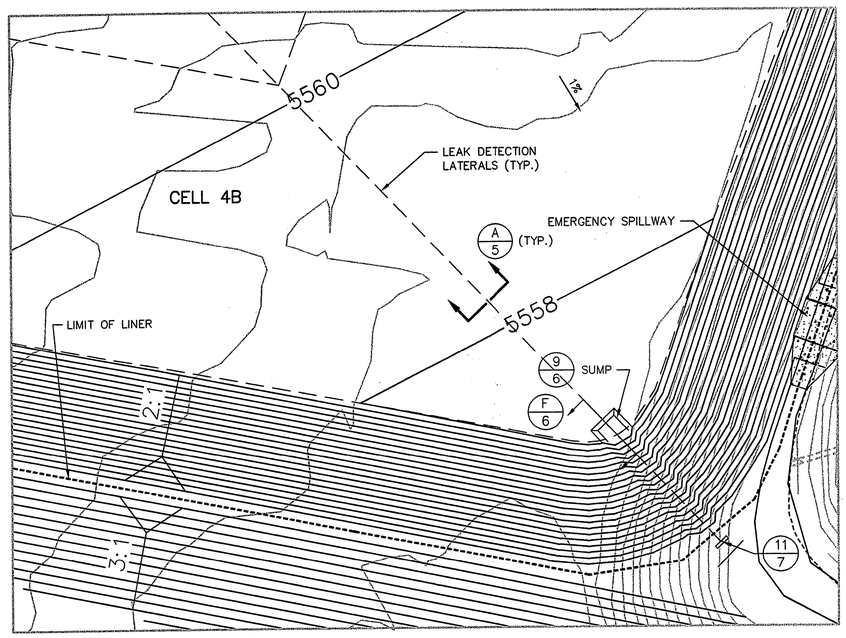
1/4 PLAN
LEAK DETECTION SYSTEM
SCALE: 1" = 200'



2/4 PLAN
SLIMES DRAIN SYSTEM
SCALE: 1" = 200'

LEGEND

- EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
- EXISTING FENCE
- 5560 PROPOSED BASE GRADING (10' CONTOUR)
- LIMIT OF LINER SYSTEM
- SPLASH PAD (NOTE 3)
- 1% SLOPE DIRECTION AND GRADE
- STRIP COMPOSITE AND SAND BAGS
- PVC PIPE AND GRAVEL
- LEAK DETECTION PIPE TRENCH



3/4 DETAIL
LEAK DETECTION SYSTEM
SCALE: 1" = 50'
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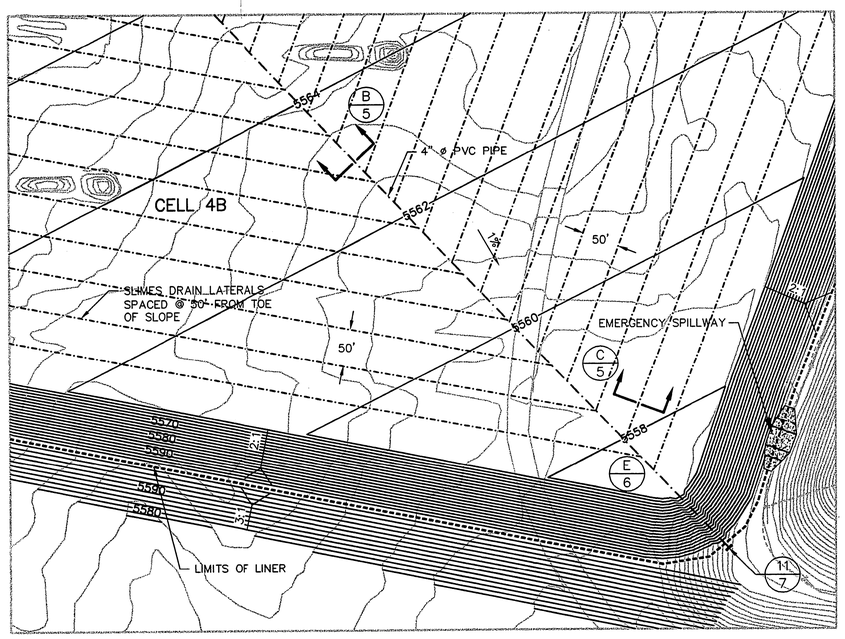

4/4 DETAIL
SLIMES DRAIN SYSTEM
SCALE: 1" = 100'
XREF: 0349X007.DWG

N

NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM DENISON MINES (USA) CORP.
2. SPLASH PAD LOCATION TO BE FIELD SELECTED BY OWNER.

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Geosyntec consultants
10875 RANCHO BERNARDO RD, SUTE 200
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PHONE: 858.674.6559

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

TITLE: PIPE LAYOUT PLAN

PROJECT: CELL 4B WHITE MESA MILL

SITE: BLANDING, UTAH

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REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC		4 OF 7

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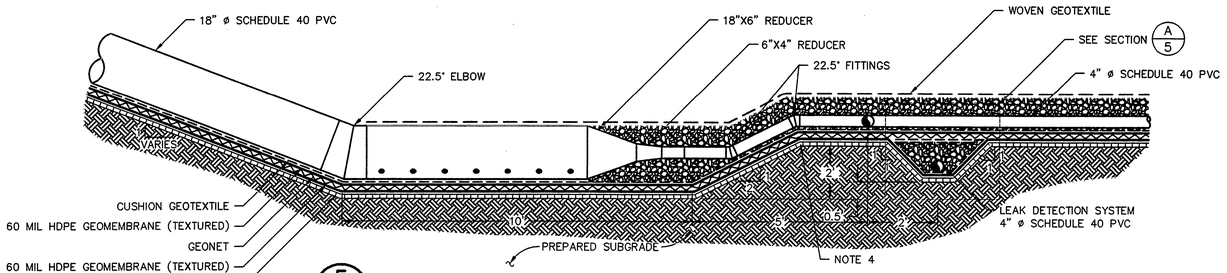
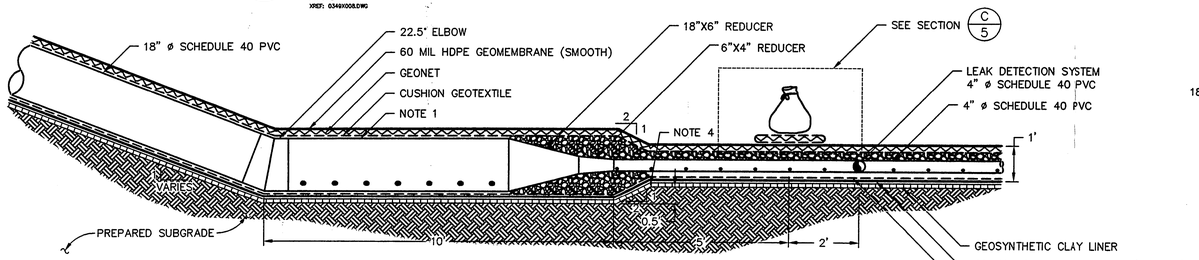


E/6 SECTION
SLIME DRAIN SYSTEM SUMP
N.T.S.
XREF: 0349X006.DWG



F/6 SECTION
LEAK DETECTION SYSTEM SUMP
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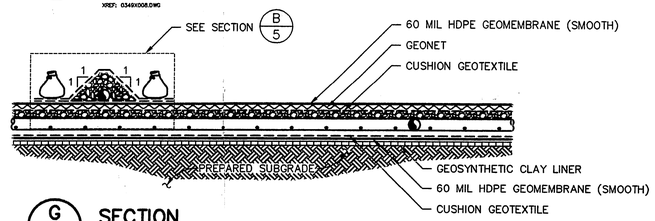

G/6 SECTION
SLIMES AND LEAK DETECTION SYSTEM SUMP
N.T.S.
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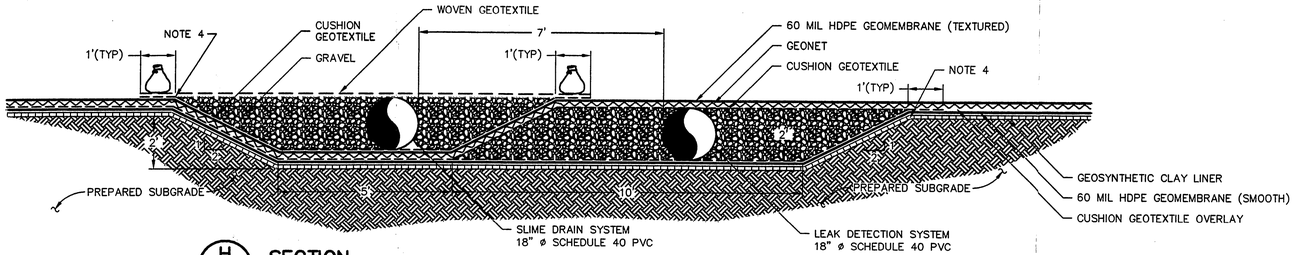

H/6 SECTION
SLIMES AND LEAK DETECTION SYSTEM SUMP
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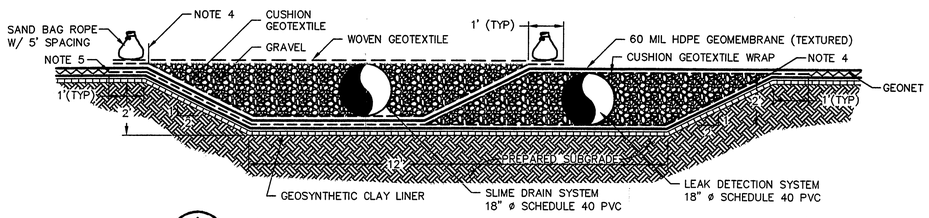
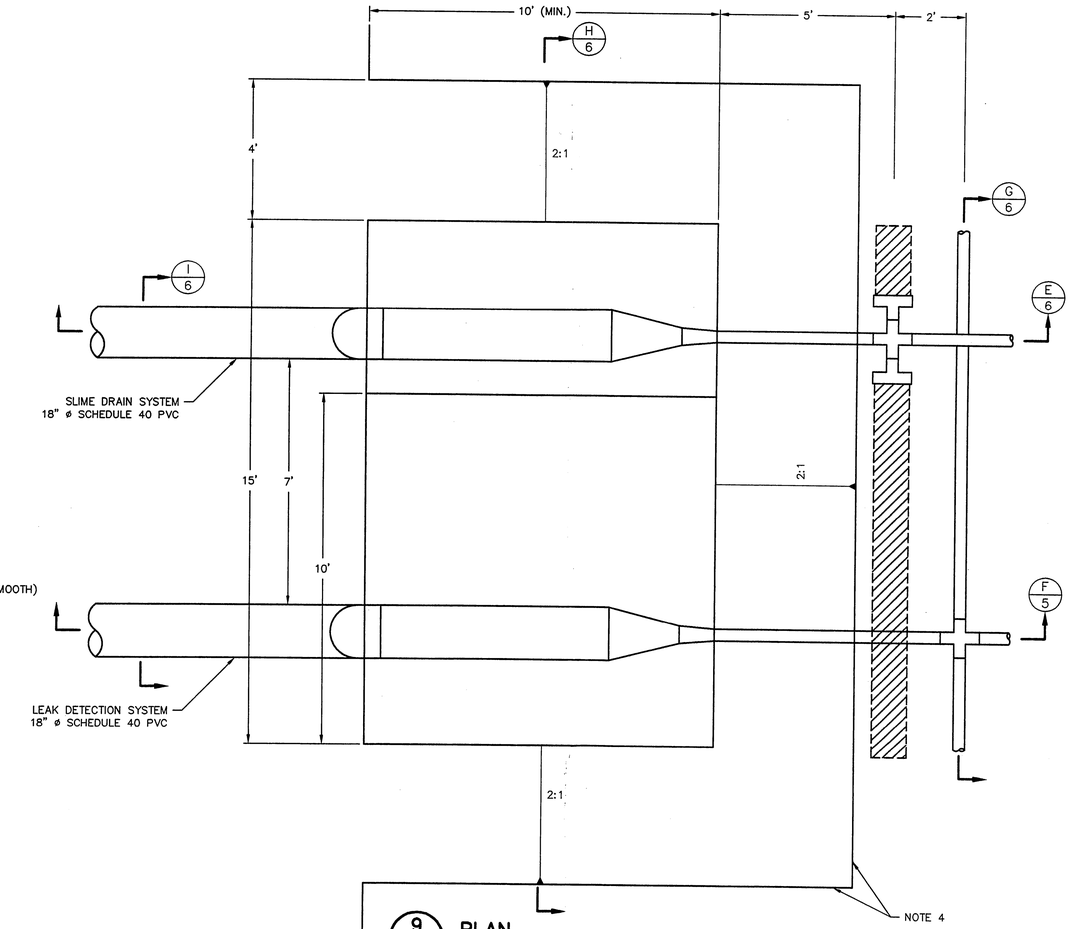


I/6 SECTION
SLIMES AND LEAK DETECTION SYSTEM SIDE SLOPE
N.T.S.
XREF: 0349X006.DWG



9/3,4 PLAN
SUMP
SCALE: 1" = 2'
XREF: 0349X006.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. CONTECH C-200, OR APPROVED EQUAL (WOVEN SLIT FILM, AOS = 40, FLOW RATE = 6 GPM/SF, GRAB STRENGTH = 200 LBS, PUNCTURE = 100 LBS.)
4. LIMIT OF TEXTURED GEOMEMBRANE.
5. LIMIT OF GEONET.

REV	DATE	DESCRIPTION	DRN	APP

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6425 S. HIGHWAY 191
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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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APPROVED BY:	GTC		

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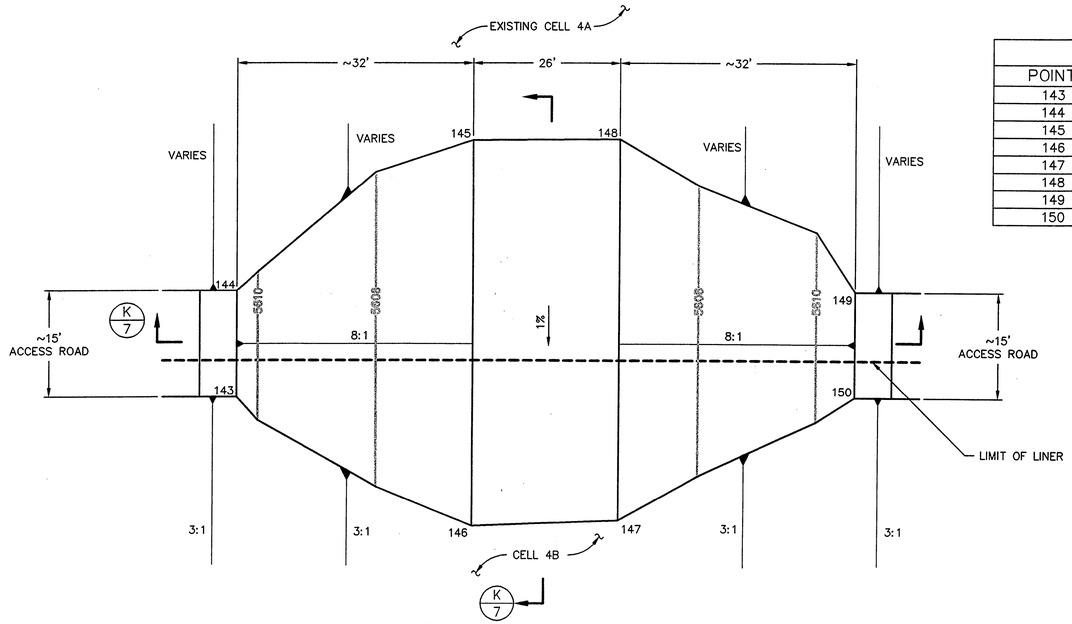
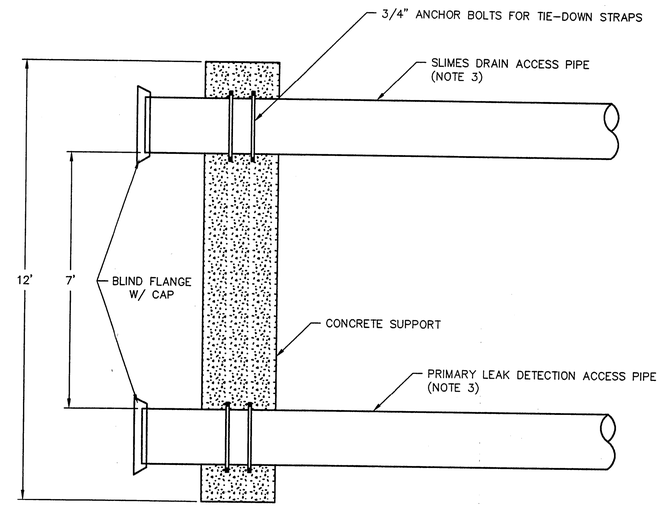
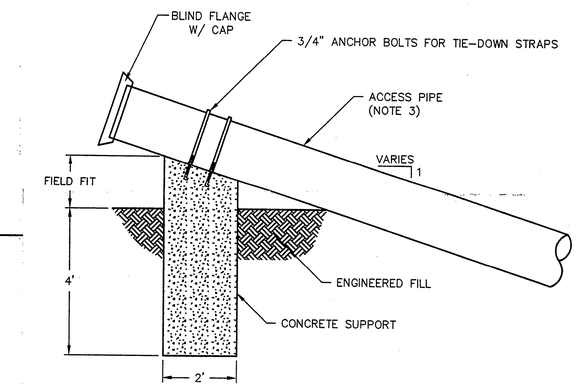


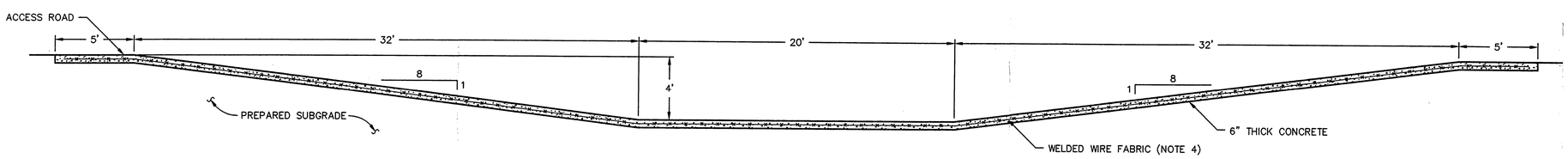
10/3 PLAN
EMERGENCY SPILLWAY
SCALE: 1" = 10'
XREF: 0349X004.DWG

POINT TABLE			
POINT #	NORTHING	EASTING	ELEVATION
143	319542.8711	2577481.2798	5600
144	319538.3476	2577494.0228	5600
145	319503.5037	2577496.5228	5569
146	319515.3611	2577463.1200	5569
147	319490.7627	2577454.7052	5569
148	319479.1526	2577487.4112	5569
149	319453.9250	2577462.8219	5600
150	319457.5650	2577452.5678	5600

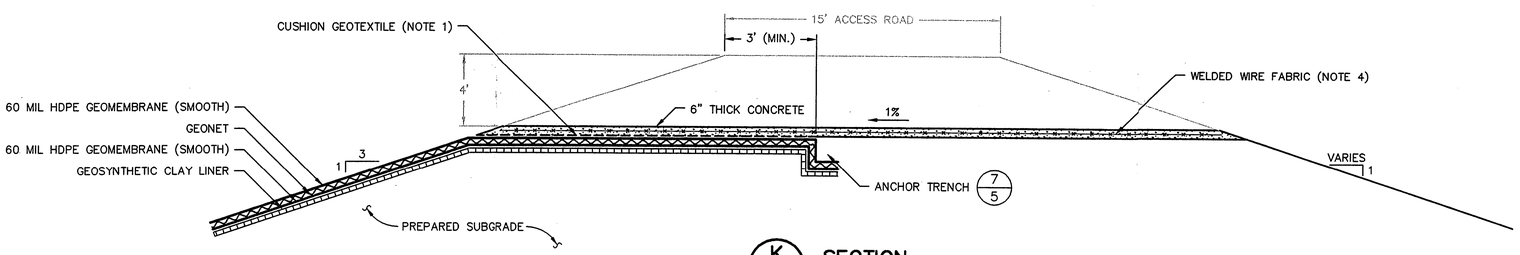




11/4 DETAIL
PIPE SUPPORT
SCALE: 1" = 2'
XREF: 0349X005.DWG



J/7 SECTION
EMERGENCY SPILLWAY
SCALE: 1" = 4'
XREF: 0349X003.DWG



K/7 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 DAMAGE.
4. WELDED WIRE FABRIC SHALL BE INSTALLED AT MIDSECTION OF CONCRETE SLAB.

REV	DATE	DESCRIPTION	DRN	APP

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6425 S. HIGHWAY 191
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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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SIGNATURE
12/5/07
DATE

REGISTERED PROFESSIONAL ENGINEER No. 6020077-2202 GREGORY T. CORCORAN STATE OF UTAH

DESIGN BY:	GTC	DATE:	DECEMBER 2007
DRAWN BY:	MAD	PROJECT NO.:	SC0349
CHECKED BY:	RF	FILE:	
REVIEWED BY:	GTC	DRAWING NO.:	
APPROVED BY:	GTC		7 OF 7

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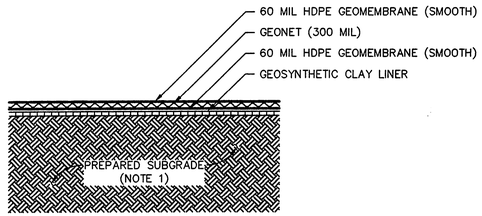
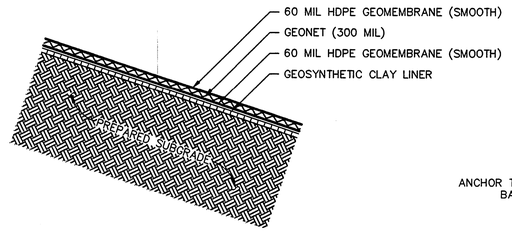


5/3 DETAIL
BASE LINER SYSTEM
SCALE: 1" = 1'
XREF: 03499X002.DWG



6/3 DETAIL
SIDE SLOPE LINER SYSTEM
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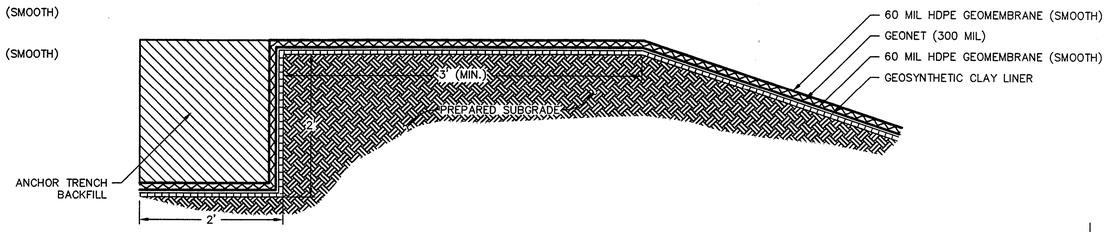

7/3,7 SECTION
ANCHOR TRENCH
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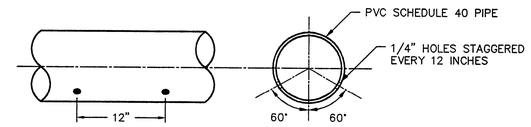
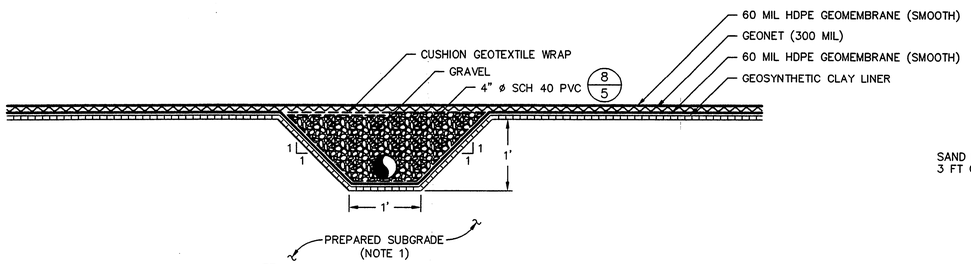


8/5 DETAIL (TYPICAL)
PERFORATED PIPE
SCALE: N.T.S.
XREF: 03499X001.DWG



A/3,4,6 SECTION
LEAK DETECTION SYSTEM
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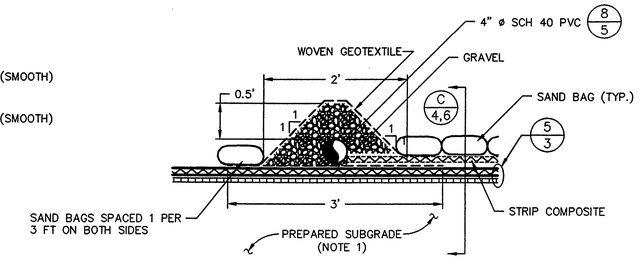

B/4,6 SECTION
SLIMES DRAIN HEADER
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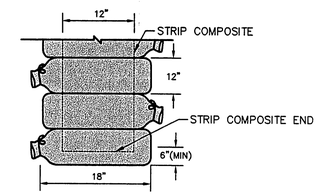
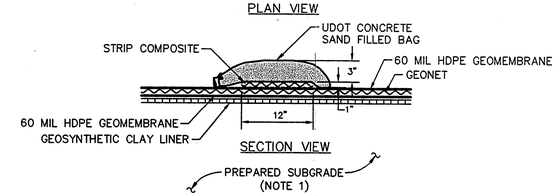


PLAN VIEW



SECTION VIEW

C/4,6 SECTION
SLIMES DRAIN LATERAL
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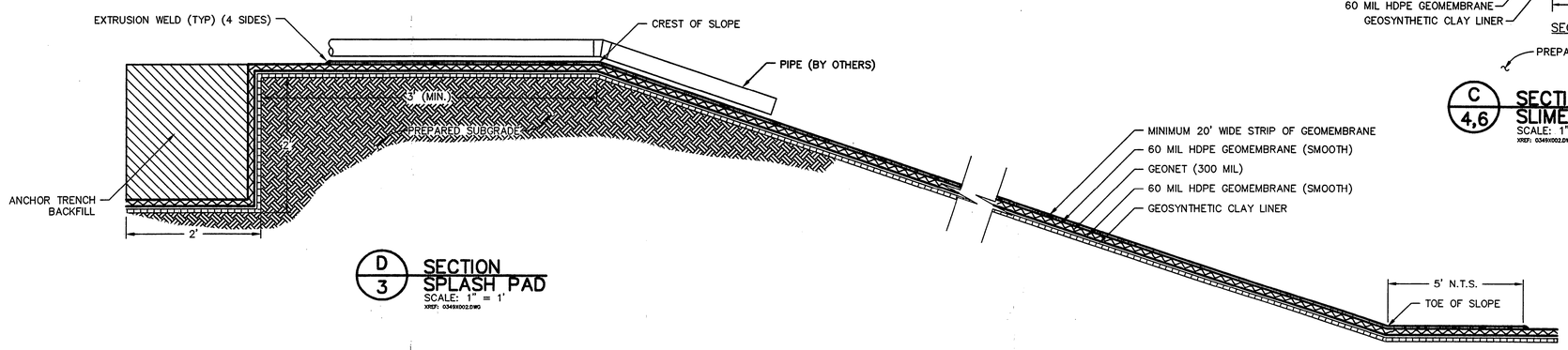
XREF: 03499X002.DWG



D/3 SECTION
SPLASH PAD
SCALE: 1" = 1'
XREF: 03499X002.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)

REV	DATE	DESCRIPTION	DRN	APP

Geosyntec consultants
10875 RANCHO BERNARDO RD, SUITE 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

12/5/07
DATE

REGISTERED PROFESSIONAL ENGINEER
No. 6020077-2202
GREGORY T. CORCORAN
STATE OF UTAH

DESIGN BY:	GTC	DATE:	DECEMBER 2007
DRAWN BY:	MAD	PROJECT NO.:	SC0349
CHECKED BY:	RF	FILE:	
REVIEWED BY:	GTC	DRAWING NO.:	5 OF 7
APPROVED BY:	GTC		

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APPENDIX B

Construction Quality Assurance 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

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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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.

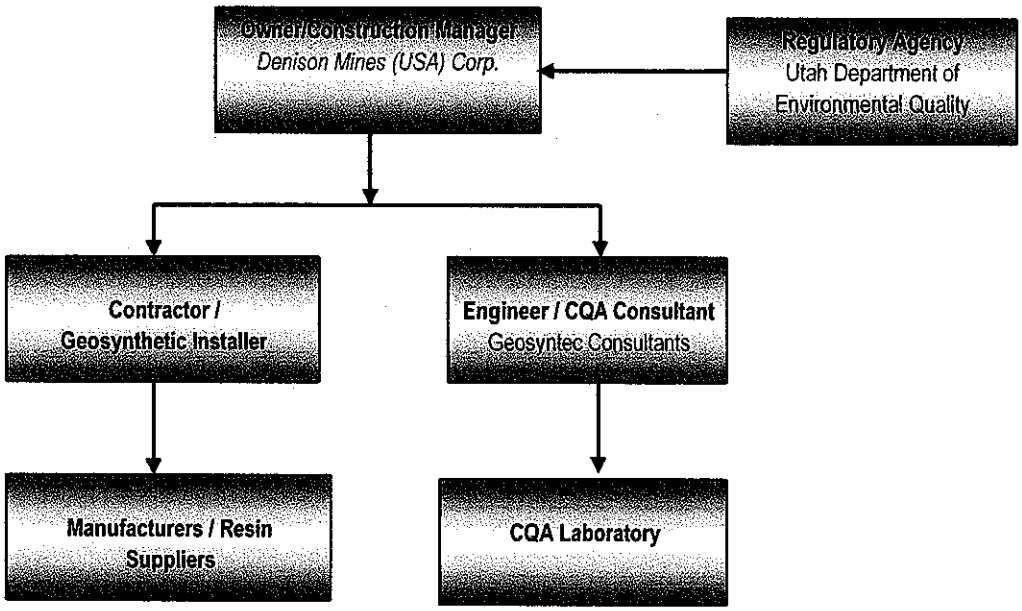
Qualifications

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 Deficiencies

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.

During GCL installation, the CQA Site Manager will deploy a small container to collect water as it is being applied to the surface of the GCL. The depth of water within the container will be measured and compared to the requirements outlined in the Technical Specifications. In addition, the CQA Site Manager will collect 6 inch square samples of the hydrated GCL for testing for moisture content. Samples will be collected once the overlying secondary geomembrane is in place and taken from within a destructive sample location. The test methods and minimum testing frequencies are indicated in Table 5.

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 GCL Delivery and Storage

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;
- monitoring that the subgrade surface is moist to within a minimum of 1 inch from the subgrade surface;
- monitoring that the GCL is hydrated prior to installation of the overlying geomembrane; 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 50 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 Densometer – In-situ Density	ASTM D 2922	1 per 1,000 yd^3 (fill) or 1 per 20,000 ft^2 (subgrade)
Nuclear Densometer – In-situ Moisture Content	ASTM D 3017	1 per 1,000 yd^3 (fill) or 1 per 20,000 ft^2 (subgrade)

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[1,2,3]	ASTM D 5321	1 per project

Notes:
1. To be performed at normal stresses of 10, 20, and 40 psi between smooth geomembrane and geonet.
2. To be performed at normal stresses of 100, 200, and 400 psf between smooth and textured geomembrane and woven side of GCL. GCL shall be hydrated for 48 hours prior to testing.
3. To be performed at normal stresses of 100, 200, and 400 psf between textured geomembrane and nonwoven geotexile.

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
Bentonite Moisture Content – Post Field Hydration	ASTM D 2216	1 test per 4 secondary geomembrane destructive samples

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 steel plates. Measurements are taken one hour after application of confining pressure..

APPENDIX C

Project 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

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

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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	Fill	CY
7	02221	Subgrade Preparation	SF
8	02221	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	02772	Geosynthetic Clay Liner	SF
14	02773	300-mil Geonet	SF
15	03400	Cast-In-Place Concrete	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; cost of bonds, both prime and subcontractors; 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]

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 Project 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 2922	Standard Test Methods for Density of Soil and Soil-Aggregate In-Place by Nuclear Density Methods (Shallow Depth)
ASTM D 3017	Standard Test Method for Water Content of Soil and Rock In-Place 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 Project 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 Project 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 Project 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 Project 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 Project Manager information regarding the source of the material. In

addition, the Contractor shall provide the Project 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 Project 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 Project 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 Project Manager. 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 cause to change the classification of the surrounding material.

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 Project 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 Project Manager shall be notified in writing prior to commencing Work. Failure to notify the Project 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 Project 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 12 inches and compacted to 90% of the maximum density at a moisture content of between -4% and +4% 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 Project 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 Project 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 Project 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) Consultant. The Contractor shall take this testing frequency into account in planning the construction schedule.

1. The Project Manager 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 2922 and ASTM D 3017, respectively. 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 Consultant 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 Project 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 Project Manager will evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the Project Manager will determine the extent of the defective area by additional tests, observations, a review of records, or other means that the Project Manager deems appropriate. If the defect is related to adverse Site conditions, such as overly wet soils or surface desiccation, the Project Manager 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 Project Manager. The Contractor shall not perform additional Work in the area until the Project Manager approves the correction of the defect.

3. Additional testing may be performed by the Project Manager 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 Project 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 Project 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 Project 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 2922	Standard Test Methods for Density of Soil and Soil-Aggregate In-Place by Nuclear Density Methods (Shallow Depth)
ASTM D 3017	Standard Test Method for Water Content of Soil and Rock In-Place 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.

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. Rock or soil particles larger than 0.5 inches in longest diameter shall be removed from the surface prior to geosynthetics placement.

D. Prior to GCL installation, the subgrade surface shall be proof-rolled with appropriate compaction equipment to confirm subgrade stability.

E. In the case additional soil is imported on the site for subgrade use, it shall be placed in loose lifts of no more than 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.

F. Subgrade soils shall be moisture conditioned prior to installation of overlying GCL to a wet but workable condition.

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 6 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 Consultant.

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 Consultant. 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 Consultant. 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%

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 Consultant 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, 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 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 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 Consultant. 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 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 Consultant adequate time to perform conformance testing on the geomembrane samples as described in Subpart 3.05 of this Section. If the CQA Consultant 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 Consultant 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 40°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 excessive winds.

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 20 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 40°F or above 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 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 Consultant, 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 Consultant 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 Consultant reserves the right to increase the sampling frequency if observations suggest an increased frequency is warranted.

b. The CQA Consultant shall mark the destructive sample locations. Samples shall be cut by the Geosynthetic Installer at the locations designated by the CQA Consultant 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 Consultant 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 Consultant 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 Consultant 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 Consultant 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; and

vi. 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 Consultant, 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 40°F or above 104°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 Consultant and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. The CQA Consultant may collect samples at the manufacturing plant or from the rolls delivered to the site. The Geosynthetic Installer shall assist the CQA Consultant 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 Consultant 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 Consultant 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 Consultant:

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 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	SPECIFIED VALUES	TEST METHOD
Physical Properties				
Thickness	Average Minimum	mils mils	60 54	ASTM D 5199
Specific Gravity	Minimum	N/A	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	ASTM D 638
Puncture	Minimum	lb	108	ASTM D 4833
Environmental Properties				
Carbon Black Content	Range	%	2-3	ASTM D 1603
Carbon Black Dispersion	N/A	none	Note 1	ASTM D 5596
Environmental Stress Crack	Minimum	hr	400	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 Consultant. 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 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 geosythetic interfaces as outlined below.

1. Geotextile and Textured HDPE Geomembrane - 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 60-mil textured HDPE geomembrane, geotextile, 60-mil textured HDPE geomembrane, geotextile, and gravel material. 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 geotextiles and a textured 60-mil HDPE geomembranes. 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 34 degrees.

2. Geotextile and Textured HDPE Geomembrane - 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 60-mil textured HDPE geomembrane, geotextile, and gravel material. 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 geotextile and a textured 60-mil HDPE geomembrane. 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 34 degrees.

2.04 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.

2.05 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 Consultant 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 Consultant, 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 Consultant 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 Consultant, at a minimum frequency of one sample per 260,000 square feet (minimum of one).

C. The CQA Consultant 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
Liner System Properties				
Interface Shear Strength	Minimum	degrees	34	ASTM D5321[1]
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

Note:

(1) Interface shear strength testing shall be performed, by the CQA Engineer, in accordance with Part 2.03 of this Section.

[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 02221 – 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 Consultant. 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 90 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 he 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 geosythetic interfaces as outlined below.

1. Hydrated GCL 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, geotextile, textured 60-mil HDPE geomembrane, geotextile, and gravel material. 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 and a textured 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 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 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, geotextile, and gravel material. 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 and a textured 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 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.

2.03 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.

B. Special care shall be taken to maintain the prepared soil surface. The subgrade shall be moisture conditioned prior to installation of the GCL. GCL subgrade shall be moisture conditioned the day before installation such that the surface is workable but not dry to a depth of more than 1 inch from subgrade surface.

C. 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. All GCL shall be hydrated by the Geosynthetic Installer once in place by direct spraying with water. Hydrated GCL shall be defined as greater than 50% moisture content when tested in accordance with ASTM D 2216. To monitor the hydration process, small, shallow, flat bottom containers shall be deployed on the GCL surface by the CQA Consultant during water spraying to measure the amount (depth) of water applied. Minimum depth of water will be 1/8-inch. During hot, dry periods, additional water may be required. Upon completion of the direct spraying with water, the GCL shall be covered with the overlying secondary geomembrane within 2 hours. Samples of the hydrated GCL will be obtained by the CQA Consultant from locations of destructive tests in the secondary geomembrane. GCL sample holes shall be repaired in accordance with Part 3.06 of this Section.

E. The 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 20 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 Consultant 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 Consultant 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 sqaure feet (90,000 square feet), then the Geosynthetic Installer shall pay the cost for all additional testing.

C. The CQA Consultant 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
Liner System Properties				
Interface Shear Strength	minimum	degrees	25	ASTM D 5321[2]
Adhesion	minimum	psf	100	ASTM D 5321[2]
GCL Properties				
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 02221 – 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 Consultant. 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 Consultant adequate time to inventory the geonet rolls and obtain additional conformance samples, if needed. The Geosynthetic Installer shall notify the CQA Consultant 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 Consultant in obtaining conformance samples, if requested. The CQA Consultant 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 Consultant 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 Consultant. 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 (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, clean, and potable, 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 Consultant. 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]

APPENDIX D

Design Calculations

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations: **Spillway Capacity Calculations**

Computations by:	Signature		11/30/07 Date
	Printed Name	Rebecca Flynn	
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		11/30/07 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Computations Checked by:	Signature		11/30/07 Date
	Printed Name	Meghan Lithgow	
	Title	Staff Engineer	
Computations backchecked by: (originator)	Signature		11/30/07 Date
	Printed Name	Rebecca Flynn	
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07 Date
	Printed Name	Gregory T. Corcoran	
	Title	Principal	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	10/15/07	Reviewed by:	G. Corcoran	Date:	
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

SPILLWAY CAPACITY CALCULATIONS

OBJECTIVE

The purpose of this calculation is to estimate the capacity of the spillway designed for Cell 4B. Cell 4B will be used for process liquids evaporation and disposal of tailings and by-products of the ore processing operations at the site. Cell 4B will also contain excess runoff from the upstream Cells 2, 3, and 4A during the Probable Maximum Precipitation (PMP); 6 hour storm event. The spillway between Cells 4A and 4B is located at the southeast corner of Cell 4A and is designed to pass excess runoff not retained in Cells 2, 3, and 4A during the PMP event.

ASSUMPTIONS

The following assumptions were used for completion of this calculation:

- The watershed areas of the upstream Cells 2, 3, and 4A are 87 acres (ac), 83 ac, and 42 ac, respectively.
- The spillway conveying flows from Cell 3 to Cell 4A was designed with a discharge of 3050 cubic feet per second (cfs). (Attachment A)
- Runoff from Cell 4A was calculated using a weighted average of the Cell 3 runoff. The area weighted discharge (Q) for Cell 3 is: 2507 cfs/170ac = 14.75 cfs. Therefore, the design Q for Cell 4B is 14.75 cfs/ac * 42 acres = 620 cfs.
- During the PMP event Cells 2, 3, and 4A are at capacity and the discharge passing through the 4B spillway is the sum of the three design flows: $Q_{Cell\ 2} + Q_{Cell\ 3} + Q_{cell\ 4A}$ = 1283 cfs + 1224 cfs + 620 cfs = 3127 cfs.
- The 4B spillway is designed with a bottom width of 26 feet, 8:1 (horizontal: vertical) side slopes, a channel slope of 1 percent, total depth of 4 feet, and finished with smooth concrete (Manning's n of 0.015). (Attachment B)

SPILLWAY CAPACITY CALCULATIONS

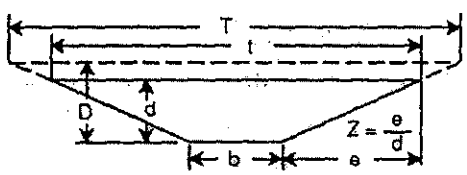
The spillway capacity is estimated using the Manning's equation:

$$Q = (1.49/n)*R^{2/3}*S^{1/2}*A$$

Where:
Q – Discharge (cfs),
n – Roughness Coefficient,
R – Hydraulic Radius (ft),
S – Channel Slope (ft/ft),
A – Flow Area (ft^2)
The capacity calculation for the spillway used the channel dimensions presented above and assumed a 0.5 feet freeboard. Therefore, the discharge at a depth of 3.5 feet is calculated to verify that the design Q could pass with spillway freeboard. The equations used to determine the area and hydraulic radius is are presented in Figure 1 (Mays, 2005).

Figure 1 – Channel Dimensions for Trapezoidal Channels

Note: Freeboard = D–d for all selection



Cross-sectional area a	Wetted perimeter, p
$bd + Zd^2$	$b + 2d\sqrt{Z^2 + 1}$

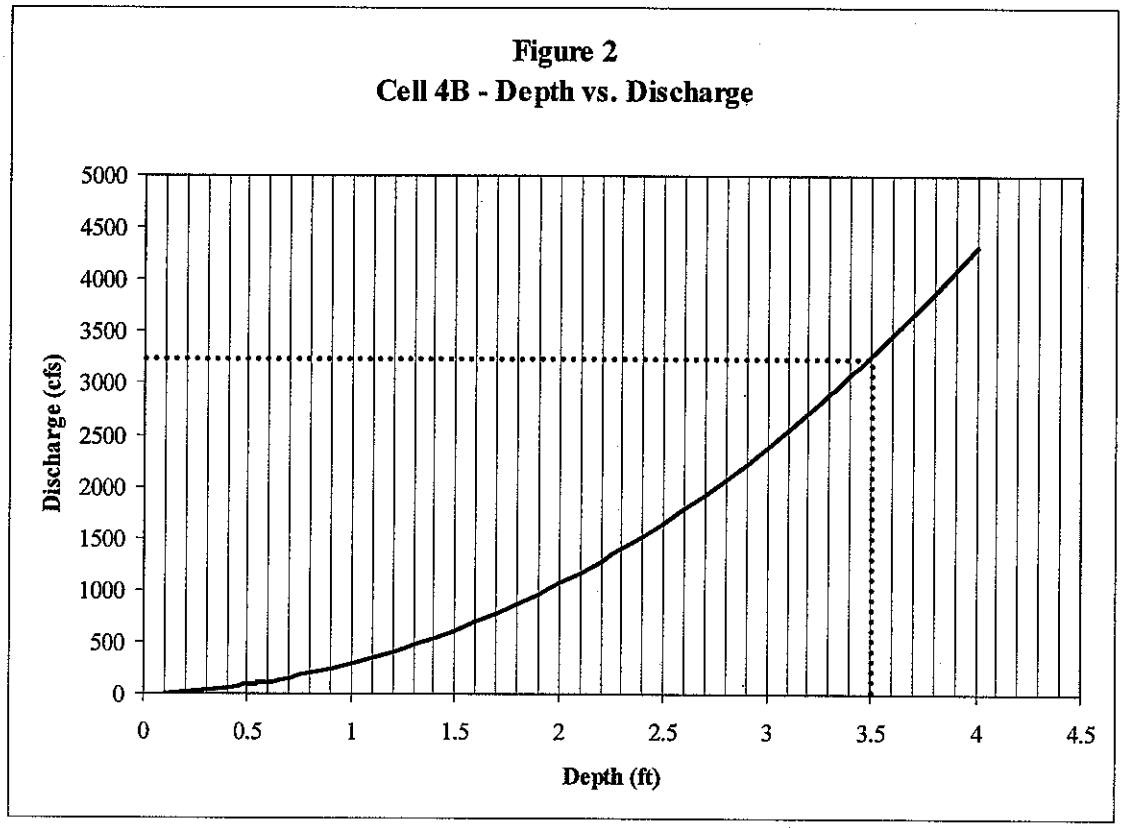
Trapezoidal cross section

Discharge with Freeboard
n = 0.015
b = 26 ft
Z = 8
d = 3.5 ft (assumed depth of flow)
$A = (26 \times 3.5) + 8 \times 3.5^2 = 189\ ft^2$
$R = A/P;\ R = ((26 \times 3.5)+(8 \times 3.5^2))/(26+(2 \times 3.5)x(8^2+1)^{0.5}) = 2.29$ ft
S = 0.01
$Qall = (1.49/0.015) \times 2.29^{2/3} x 0.01^{1/2} \times 189 = 3264$ cfs

The allowable flow for the spillway, as designed, is estimated to be 3264cfs, which is approximately equal to the flow rate for the PMP event (3,263 cfs). Iterating on the discharge depth, the rating curve (Depth vs. Q) for the spillway is presented as Figure 2.

Written by:	R. Flynn	Date:	10/15/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

Figure 2 – Rating Curve for Spillway 4B.



References

Mays, Larry W., "Water Resources Engineering, 2005 Edition," John Wiley & Sons, Inc, 2005.

Written by: Nathan Jacobsen Date: 06/ 01/ 09 (YY MM DD) Reviewed by: GTL Date: 12/3/07 (YY MM DD)

Client: IUC Project: White Mesa Mill – Cell 4A Design Project No.: SC0349 Task No.: 04

SPILLWAY CAPACITY CALCULATIONS
WHITE MESA MILL – CELL 4A
BLANDING, UTAH

OBJECTIVE

The purpose of this calculation is to estimate the capacity of the spillway designed for Cell 4A. Cell 4A will be used for process liquids evaporation and disposal of tailings and by-products of the uranium and thorium processing operations at the site. Cell 4A will also contain excess runoff from the upstream Cells 2 and 3 during the Probable Maximum Precipitation (PMP; 6 hour storm) event. The spillway between Cells 3 and 4A is located at the northwest corner of Cell 4A and is designed to pass excess runoff not retained in Cells 2 and 3 during the PMP event.

ASSUMPTIONS

The following assumptions were used for completion of this calculation:

- The watershed areas of the upstream Cells 2 and 3 are 87 acres (ac) and 83 ac, respectively.
- The spillway conveying flows from Cell 2 to Cell 3 was designed with a discharge or 1283 cubic feet per second (cfs).
- Runoff from Cell 3 was calculated using a weighted average of the Cell 2 runoff. The area weighted discharge (Q) for Cell 2 is: 1283 cfs/87ac = 14.75 cfs/ac; Therefore, the design Q for Cell 3 is 14.75 cfs/ac * 83 acres = 1224 cfs.
- During the PMP event Cells 2 and 3 are at capacity and the discharge passing through the 4A spillway is the sum of the two design flows: $Q_{Cell\ 2} + Q_{Cell\ 3}$ = 1283cfs + 1224cfs = 2507cfs (use 2510 cfs).
- The 4A spillway is designed with a bottom width of 20 feet, 8:1 (horizontal: vertical) side slopes, a channel slope of 1 percent, total depth of 4 feet, and finished with smooth concrete (Manning's n of 0.01).

SPILLWAY CAPASITY CALCULATIONS

The spillway capacity is estimated using the Manning's equation:

$Q = (1.49/n)*R^{2/3}*S^{1/2}*A$

Where:

Q – Discharge (cfs),

n – Roughness Coefficient,

R – Hydraulic Radius (ft),

S – Channel Slope (ft/ft),

A – Flow Area (ft^2)

Attachment A (1 of)

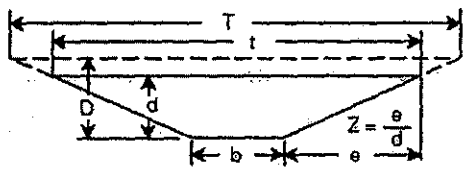
Written by: Nathan Jacobsen Date: 06/ 01/ 09 Reviewed by: GTC Date: 12/3/02
S YY MM DD YY MM DD

Client: IUC Project: White Mesa Mill – Cell 4A Design Project No.: SC0349 Task No.: 04

The capacity calculation for the spillway used the channel dimensions presented above and assumed a 1 foot freeboard. Therefore, the discharge at a depth of 3 feet is calculated to verify that the design Q could pass with spillway freeboard. The equations used to determine the area and hydraulic radius is are presented in Figure 1 (Haan, et.al, 1993).

Figure 1 – Channel Dimensions for Trapezoidal Channels

Note: Freeboard = D–d for all selection



Cross-sectional area a	Wetted perimeter, p
$bd + Zd^2$	$b + 2d\sqrt{Z^2 + 1}$

Trapezoidal cross section

Discharge with Freeboard

n = 0.01

b = 20

z = 8

d = 3 (assumed depth of flow)

$A = (20*3) + 8*3^2 = 132\ ft^2$

$R = A/P;\ R = ((20*3)+(8*3^2))/(20+(2*3)*(8^2+1)^{0.5}) = 1.93$ ft

S = 0.01

$Qall = (1.49/0.01)*1.93^{2/3}*0.01^{1/2}*132 = 3050$ cfs

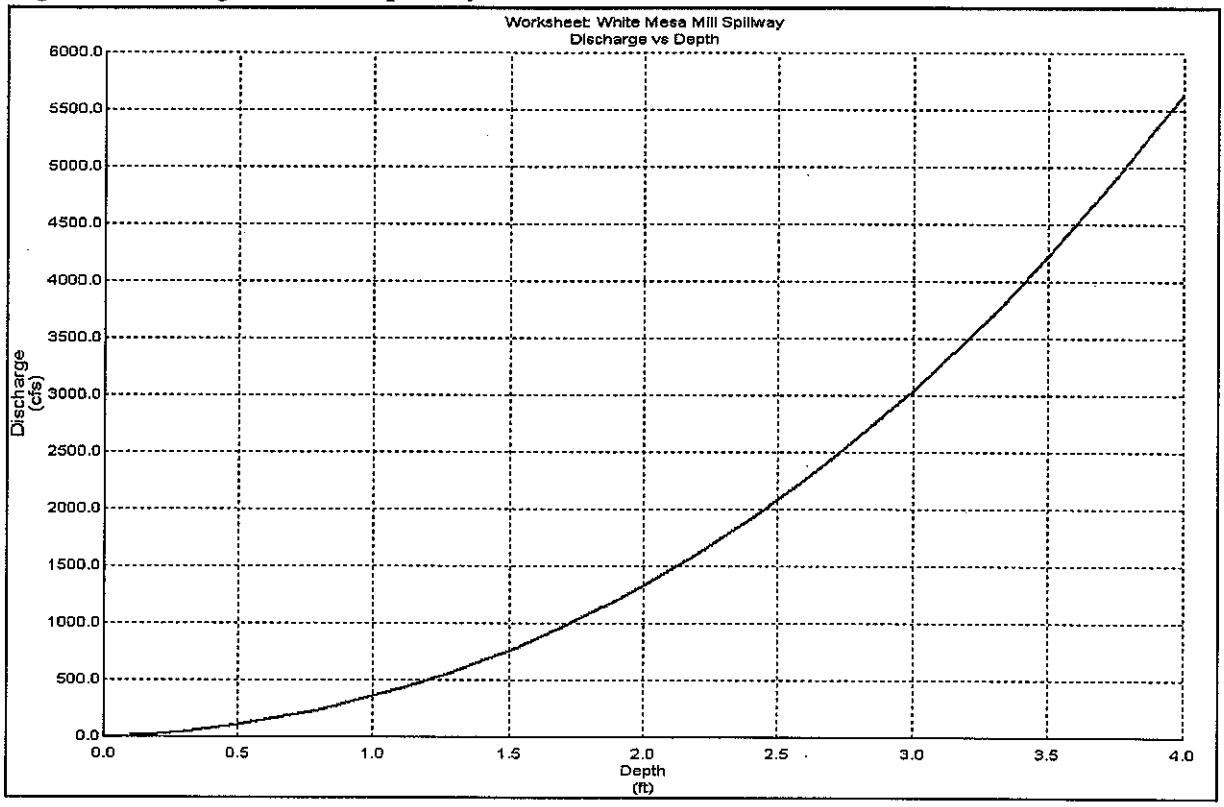
The allowable flow for the spillway, as designed, is estimated to be 3050 cfs, which exceeds the flow rate for the PMP event (2510 cfs). Iterating on the discharge depth, the rating curve (Depth vs. Q) for the spillway is presented as Figure 2. Per Figure 2, the spillway is capable of passing the required flow rate with approximately 1.26 feet of freeboard (depth of flow is 2.74 feet).

Attachment A
(2/2)

Written by: Nathan Jacobsen Date: 06/ 01/ 09 (YY MM DD) Reviewed by: _____ Date: __/__/__ (YY MM DD)

Client: IUC Project: White Mesa Mill – Cell 4A Design Project No.: SC0349 Task No.: 04

Figure 2 – Rating Curve for Spillway 4A.



Attachment B (3 of 3)

Mays, Larry W., "Water Resources Engineering, 2005 Edition," John Wiley & Sons, Inc, 2005.

Table 5.1.1 Values of the Roughness Coefficient *n (continued)*
(Boldface figures are values generally recommended in design)

Type of channel and description	Minimum	Normal	Maximum
c. Concrete			
1. Trowel finish	0.011	**0.013**	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	—
8. On irregular excavated rock	0.022	0.027	—
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	**0.013**	0.015
2. In cement mortar	0.012	**0.015**	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	—
2. Rough	0.016	0.016	—
j. Vegetal lining	0.030	—	0.500
C. Excavated or dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	**0.022**	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations **SLOPE STABILITY ANALYSIS**

Computations by:	Signature		
	Printed Name	Meghan Lithgow	Date
	Title	Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		7/23/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		7/23/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		
	Printed Name	Meghan Lithgow	Date
	Title	Staff Engineer	
Approved by: (pm or designate)	Signature		7/23/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	M. Lithgow	Date:	07/20/07	Reviewed by:	G. Corcoran	Date:	7/23/07
Client:	DMC	Project:	White Mesa Mill-Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

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

OBJECTIVE

This calculation includes static 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 and 1.3 for interim 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.

To model the various stability conditions encountered in Cell 4B during and after filling, two cross-sections were selected for analyses and are shown in 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 A-A' was modeled under two conditions; the first condition showed Cell 4A filled with tailings and Cell 4B empty and the second condition 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. 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.

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, 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).

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.

Based on subsurface investigation performed at the site and lab results from collected samples, the fill material is anticipated to be primarily a sandy lean clay with an estimated unit weight of approximately 125 pcf. Using graphical correlations from DM-7.3 and Figure C34 titled, "Gibbs-Holtz Standard Penetration Correlation," (Attachments D and E, respectively) along with results of subsurface investigations and laboratory analyses, the angle of internal friction is estimated to be approximately 35 degrees. Also, we have conservatively estimated the cohesion for this material to be 100 pounds per square foot.

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.

STATIC STABILITY RESULTS/RECOMMENDATIONS

As discussed above, two cross-sections were analyzed which represent typical 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 analysis of the east-west cross section A-A' estimated a factor of safety of 2.0 and 1.6 for Cell 4B for empty and half-full conditions (Figures 2 and 3), respectively, while Cell 4A was full for both conditions.

Section B-B' was modeled to analyze the stability of the southern perimeter berm when Cell 4B full with tailings. The south side of the berm was conservatively modeled with outside berm slopes inclined at 2:1, rather than the current 3:1 berm slopes, to represent future build out conditions. Results from the analysis of Section B-B' indicate a minimum static factor of safety of approximately 2.1 under these conditions, as shown in Figure 4.

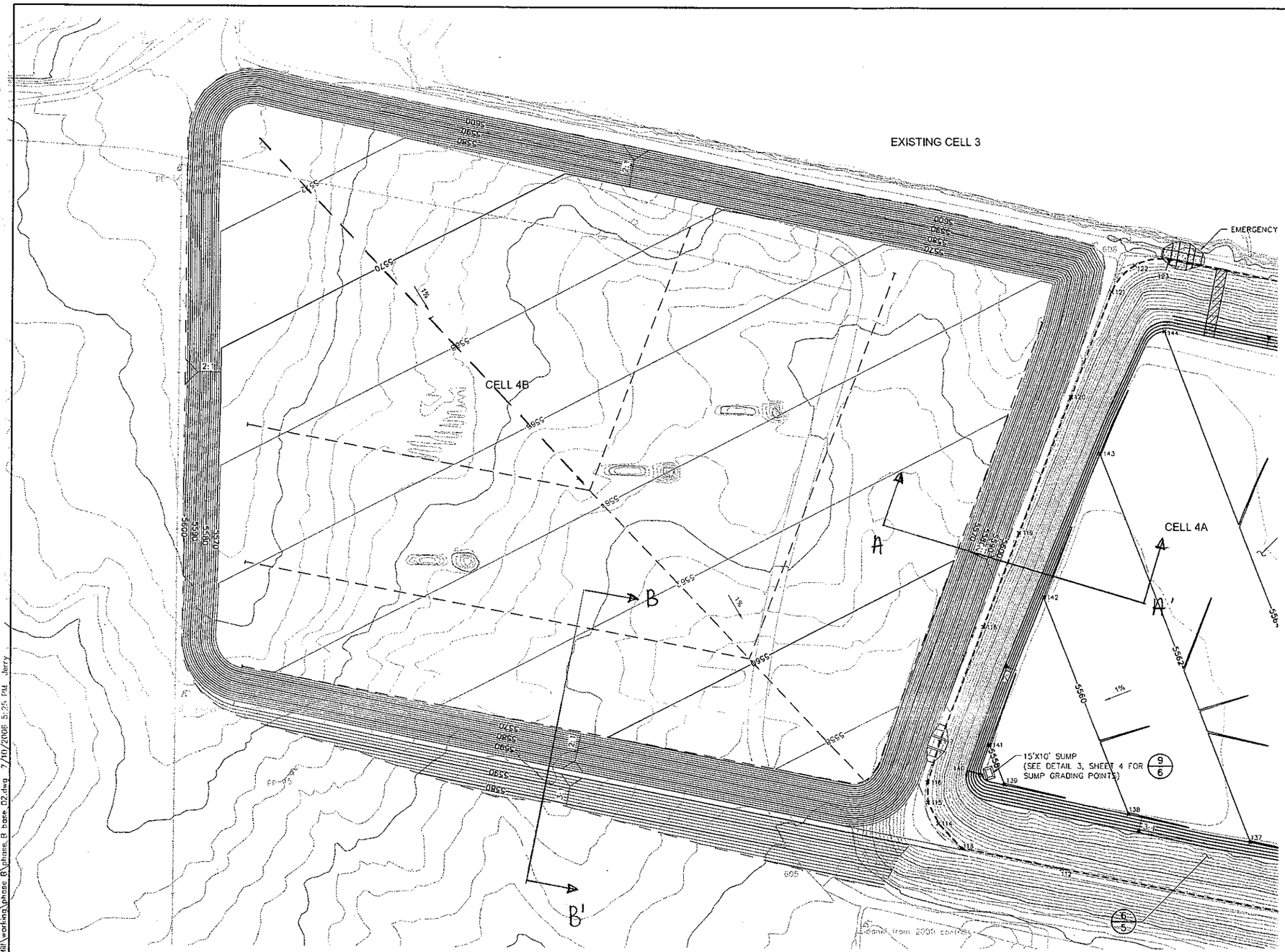
These results indicate the minimum factor of safety of 1.3 is met during and after filling operations. We recommend that interface testing of the liner system be performed during construction quality assurance to verify that the interface friction angles used in this analysis for the liner system are met or exceeded. Further, 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.

Gibbs, H.J. and W.G. Holtz, "Research on Determining the Density of Sand by Spoon Penetration Testing," Proc. International Conf. on Soil Mechanic and Found Engineering, Vol. 1, 1957.

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



LEGEND

5590	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (2' CONTOUR)
1%	SLOPE DIRECTION AND GRADE

ORIGINAL

-N-

100 50 0 100 200

SCALE IN FEET

Figure 1

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: 1
DESIGN BY:	REVIEWED BY: (PROJ. MGR.)	JOB NO.: SC0349	

P:\CAD\SC0349 - White Mesa Mill\working\phase 4\phase 4 base 02.dwg 7/10/2006 5:25 PM Jerry

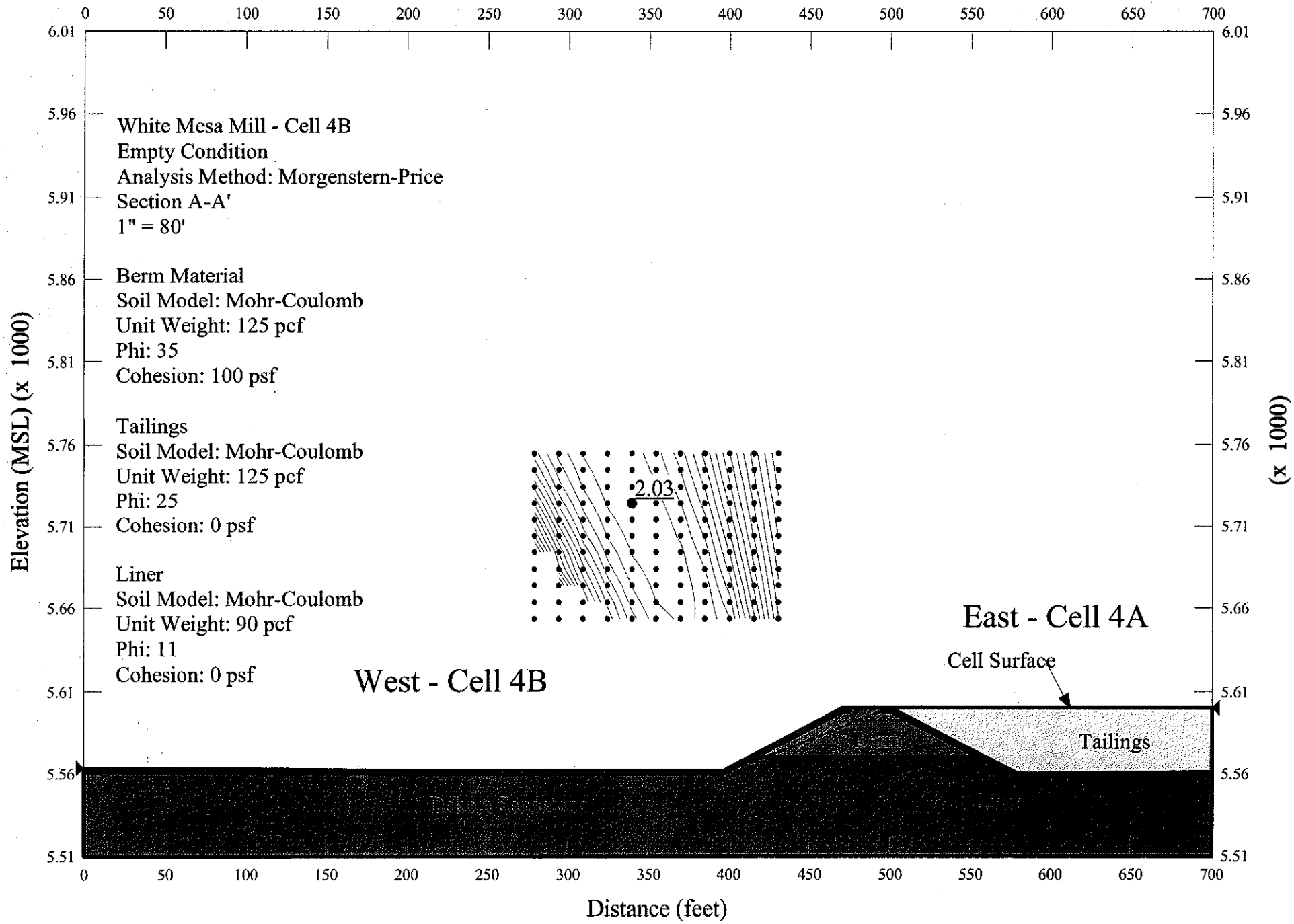

White Mesa Mill - Cell 4B
Empty Condition
Analysis Method: Morgenstern-Price
Section A-A'
1" = 80'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 35
Cohesion: 100 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
2.03
West - Cell 4B
East - Cell 4A
Cell Surface
Berm
Tailings
Dakota Sandstone
Elevation (MSL) (x 1000)
Distance (feet)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
6.01
5.96
5.91
5.86
5.81
5.76
5.71
5.66
5.61
5.56
5.51

Figure 2

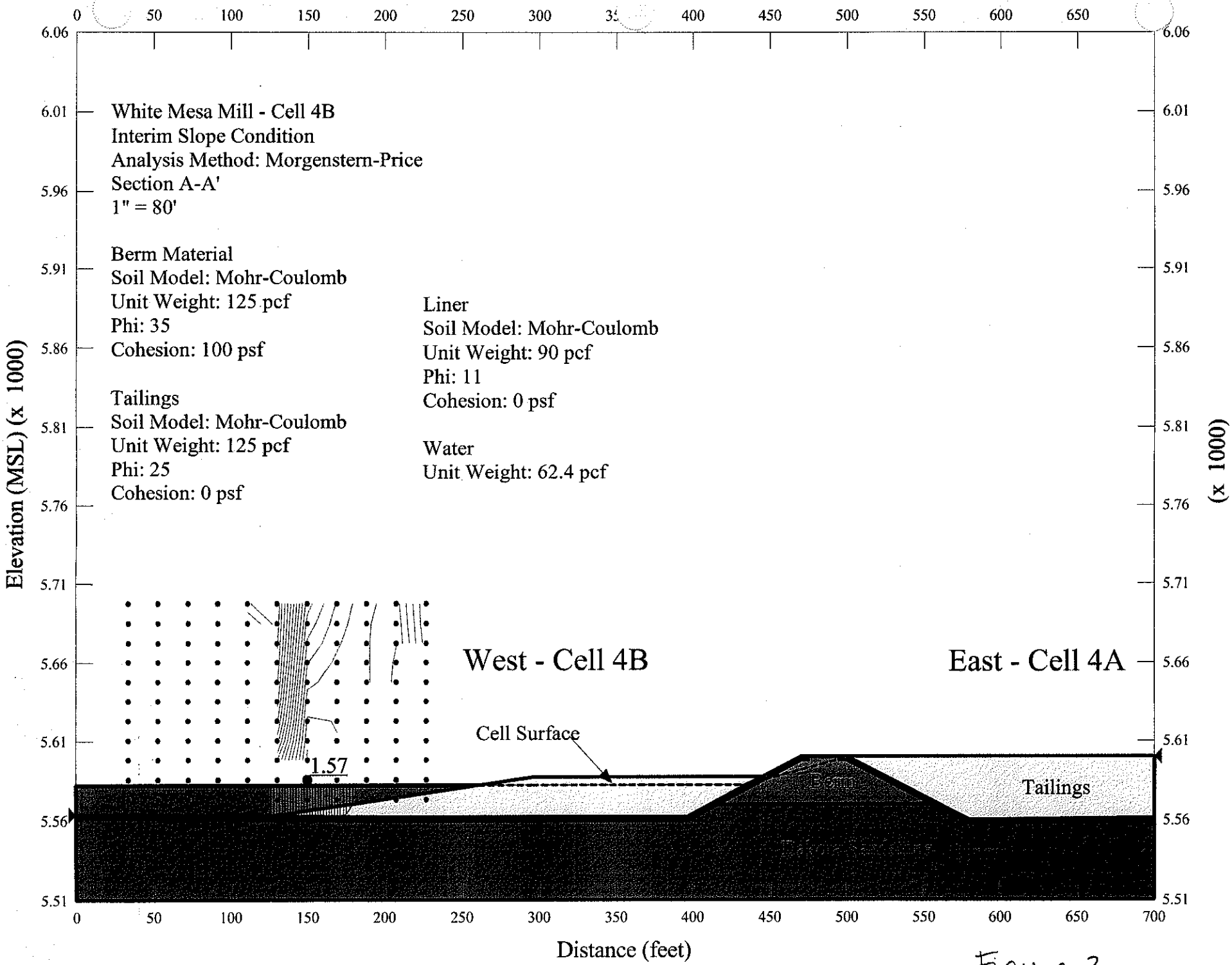

White Mesa Mill - Cell 4B
Interim Slope Condition
Analysis Method: Morgenstern-Price
Section A-A'
1" = 80'
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 35
Cohesion: 100 psf
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
Water
Unit Weight: 62.4 pcf
West - Cell 4B
East - Cell 4A
Cell Surface
1.57
Berm
Tailings
Elevation (MSL) (x 1000)
(x 1000)
Distance (feet)
6.06
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
600
650
700

Figure 3

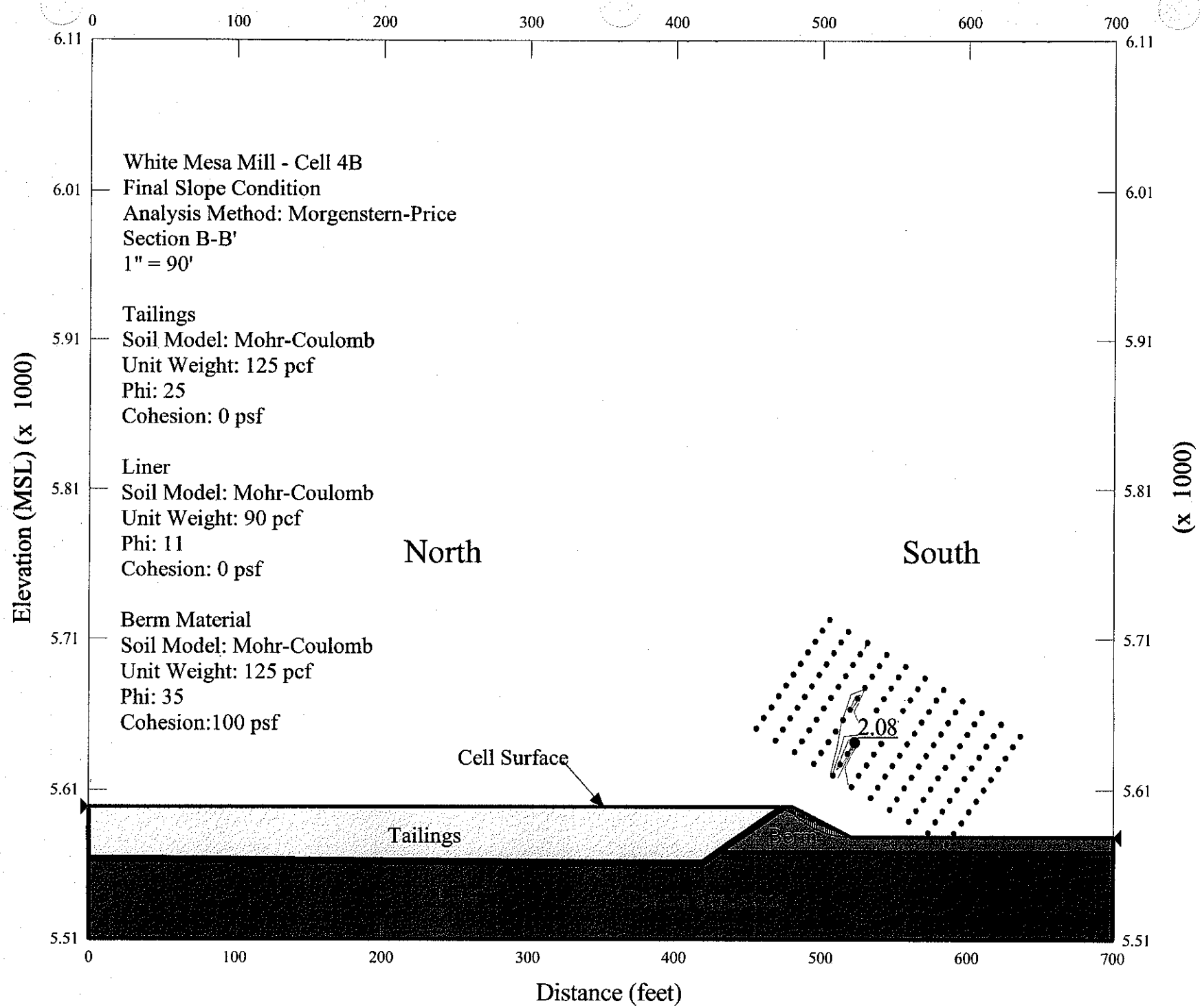
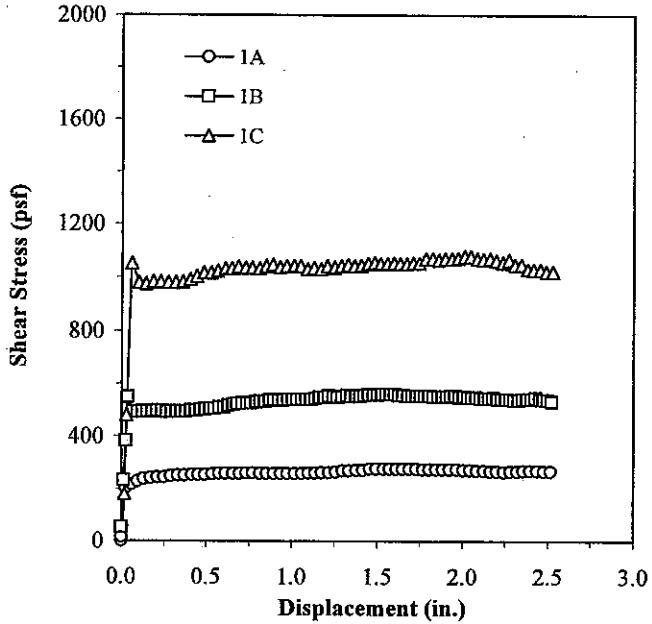
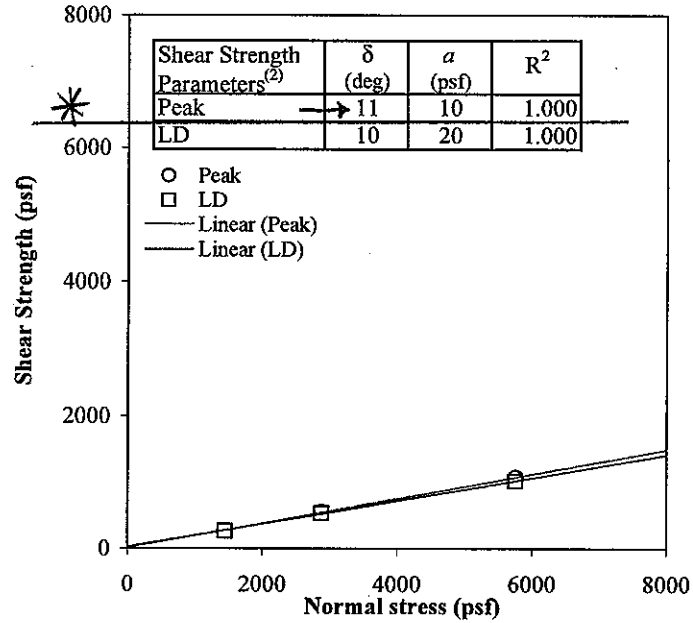

White Mesa Mill - Cell 4B
Final Slope Condition
Analysis Method: Morgenstern-Price
Section B-B'
1" = 90'
Tailings
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 25
Cohesion: 0 psf
Liner
Soil Model: Mohr-Coulomb
Unit Weight: 90 pcf
Phi: 11
Cohesion: 0 psf
Berm Material
Soil Model: Mohr-Coulomb
Unit Weight: 125 pcf
Phi: 35
Cohesion:100 psf
North
South
2.08
Cell Surface
Tailings
Berm
Elevation (MSL) (x 1000)
(x 1000)
Distance (feet)
0
100
200
300
400
500
600
700
6.11
6.01
5.91
5.81
5.71
5.61
5.51

Figure 4

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.

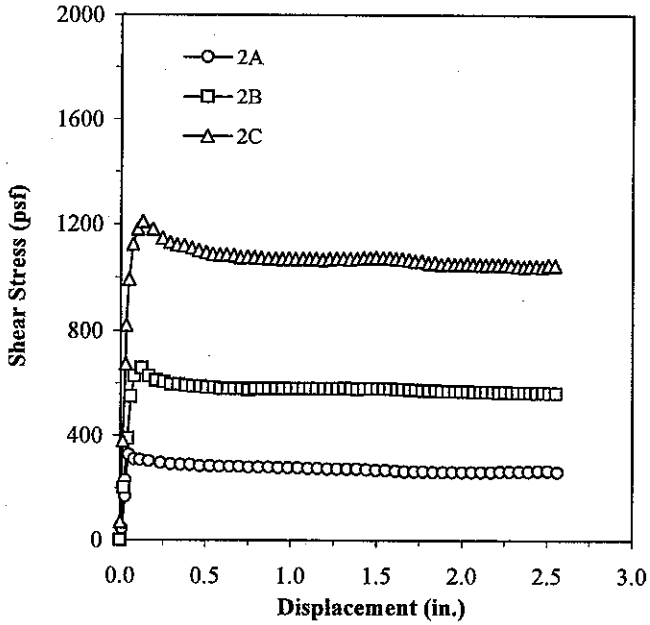
SGI TESTING SERVICES, LLC

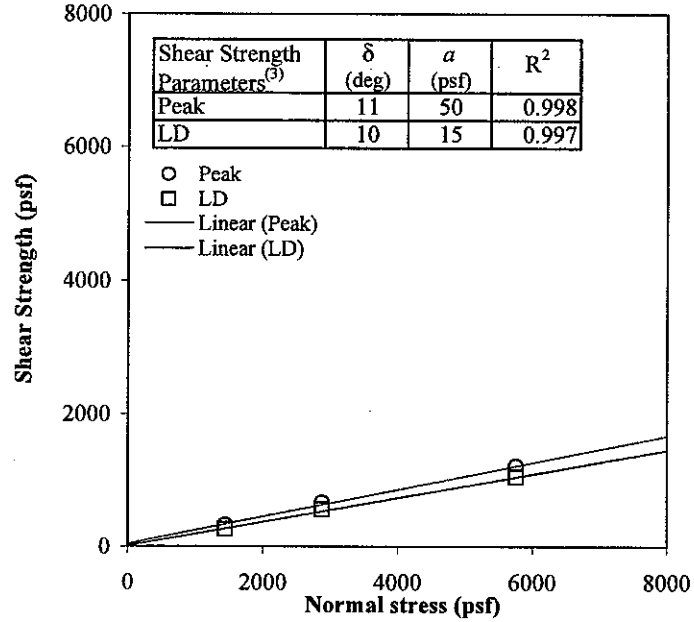
DATE OF TEST:	11/6/2006
FIGURE NO.	B-1
PROJECT NO.	SGI6014-03
DOCUMENT NO.	
FILE NO.	

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





Shear Strength Parameters[3]	δ (deg)	a (psf)	R^2
Peak	11	50	0.998
LD	10	15	0.997

Test No.	Shear Box Size (in. x in.)	Normal Stress (psf)	Shear Rate (in./min)	GCL Soaking		Consolidation[1]		Lower Soil			Upper Soil			GCL		Shear Stress		Failure Mode
				Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_d (pcf)	ω_i (%)	ω_f (%)	γ_d (pcf)	ω_i (%)	ω_f (%)	ω_i (%)	ω_f (%)	τ_P (psf)	τ_{LD} (psf)	
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

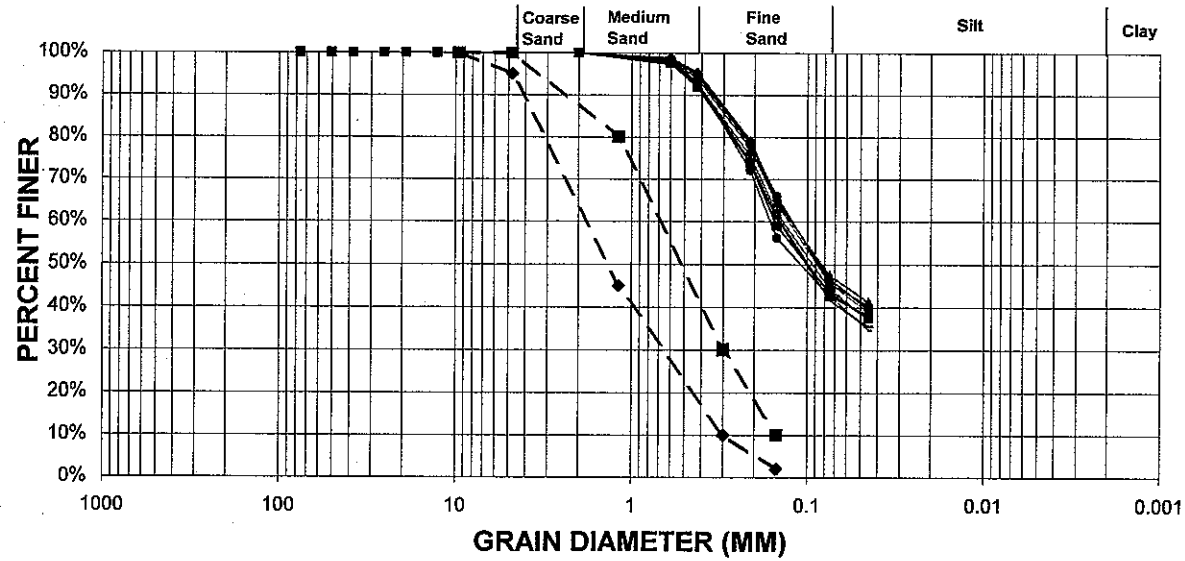
SIEVE ANALYSIS

		Grinding Test 1			Grinding Test 2A			Grinding Test 2B			Grinding Test 3A			Grinding Test 3B		
Sieve No.	Diameter (mm)	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% 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	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

		Grinding Test 6A			Grinding Test 6B		
Sieve No.	Diameter (mm)	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer
3 in.	76.2	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%
1 1/2 in.	38.1	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%
3/4 in.	19.1	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%
3/8 in.	9.530	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%
No. 10	2.000	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%
No. 40	0.425	5.2	5.2%	94.8%	4.7	4.7%	95.3%
No. 70	0.212	21.7	21.7%	78.3%	21.4	21.4%	78.6%
No. 100	0.150	34.1	34.1%	65.9%	35.9	35.9%	64.1%
No. 200	0.075	54.4	54.4%	45.6%	54.4	54.4%	45.6%
No. 325	0.045	59.7	59.7%	40.3%	61.1	61.1%	38.9%
Pan	-	-	-	-	-	-	-

3/8 in.

Grinding Test 4A			Grinding Test 4B		
Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer
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%	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%
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%	0.0	0.0%	100.0%
0.0	0.0%	100.0%	0.0	0.0%	100.0%
2.7	2.7%	97.3%	2.7	2.7%	97.3%
7.6	7.6%	92.4%	7.3	7.3%	92.7%
26.2	26.2%	73.8%	25.9	25.9%	74.1%
38.7	38.7%	61.3%	39.2	39.2%	60.8%
57.3	57.3%	42.7%	58.3	58.3%	41.7%
65.4	65.4%	34.6%	64.6	64.6%	35.4%
-	-	-	-	-	-



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

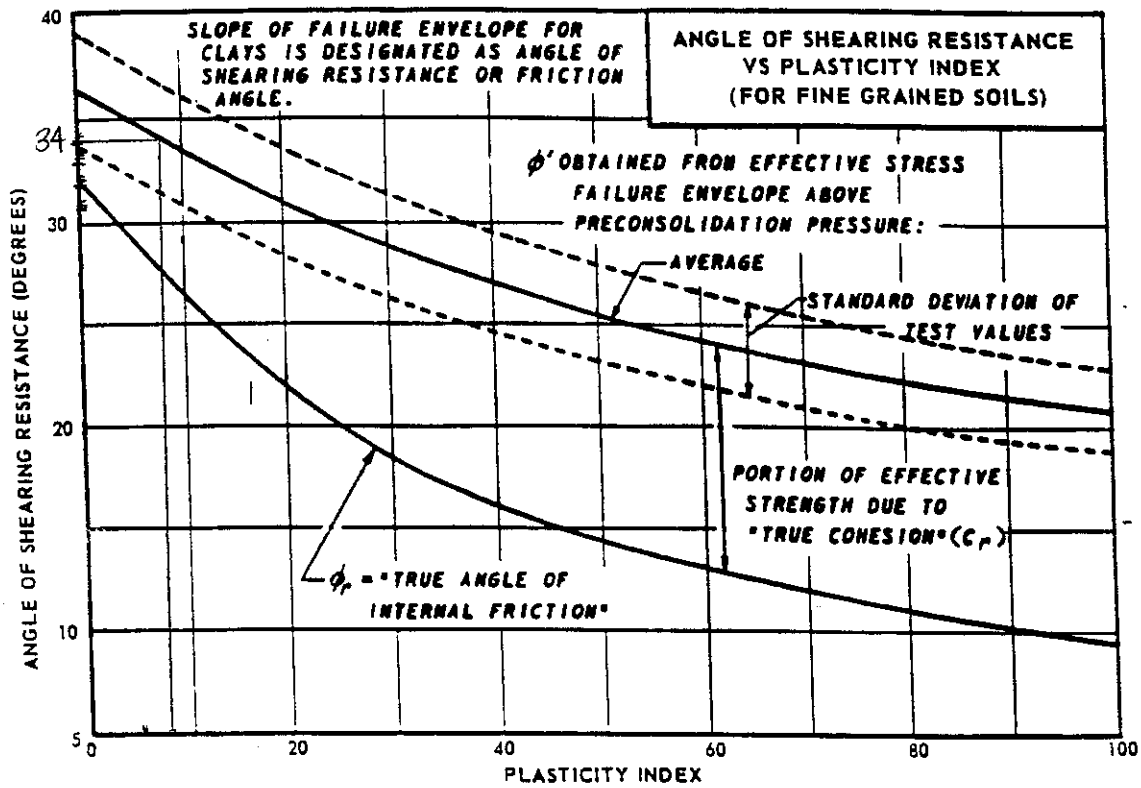
Attachment B, 1/1

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids(1)					Unit Weight(2) (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	146(3)	90	155(3)	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	148(4)	125	156(4)	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	81	125	18	62

*

Attachment C, NAVFAC DM7-01, 1986
(Navy Soil Design Manual)



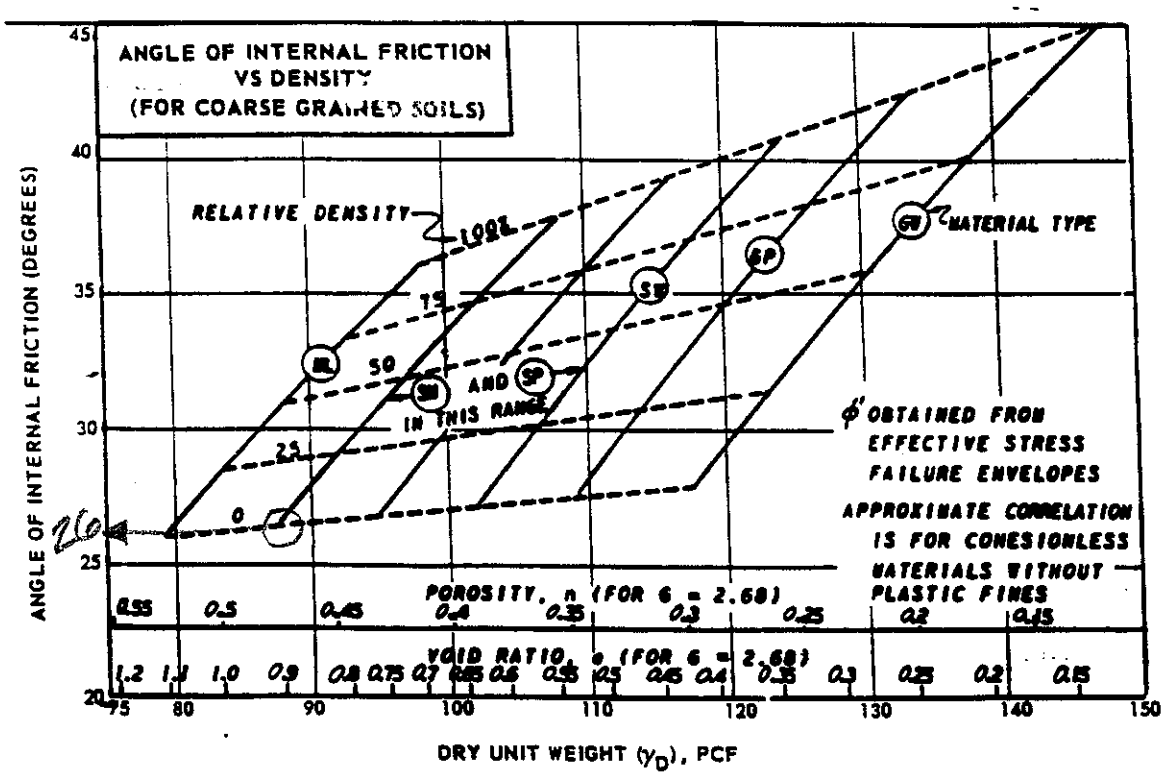


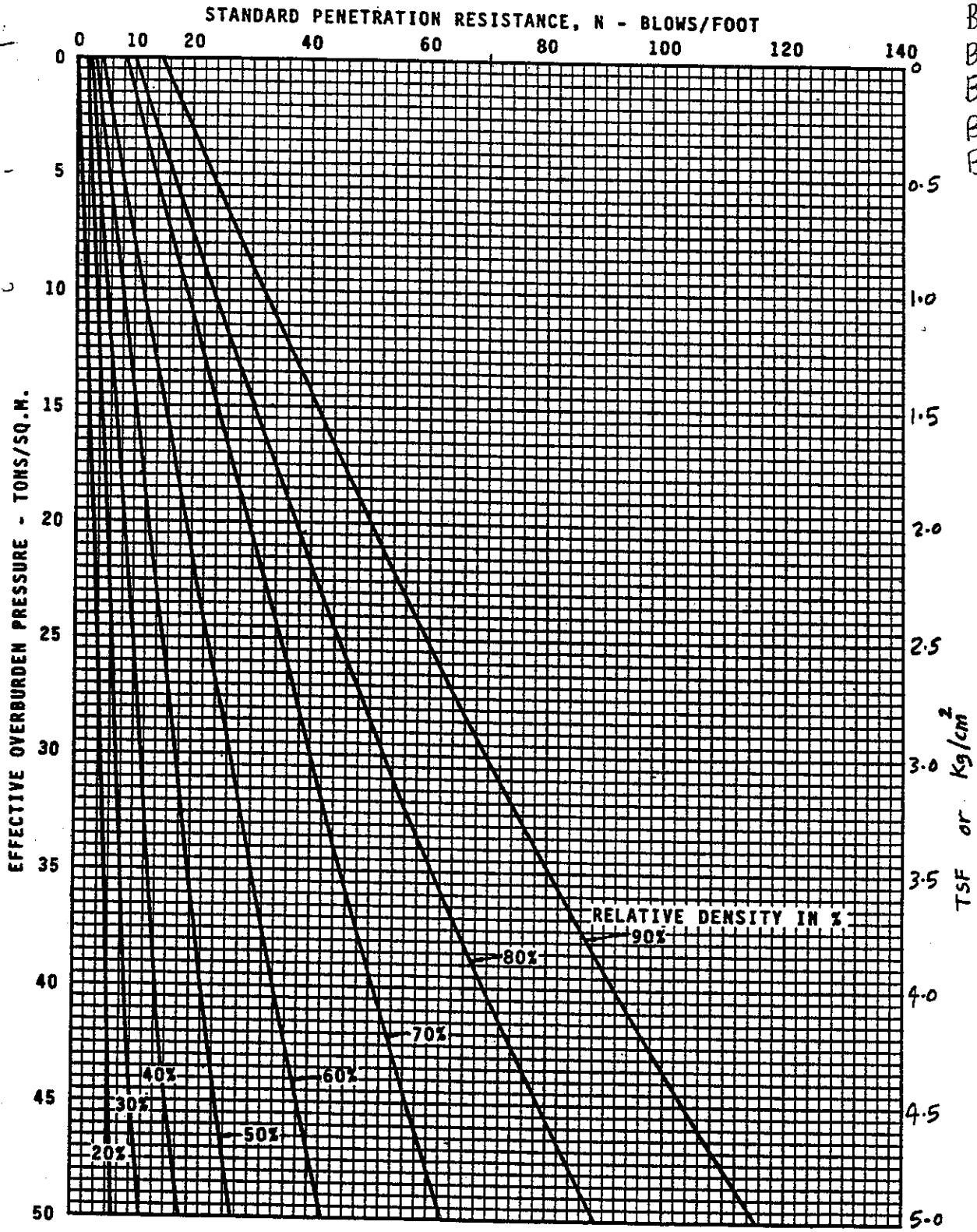
FIGURE 3-7
Correlations of Strength Characteristics

NAVFAC DM7-03, 1986

ATTACHMENT D
(1/1)

@ 5'

B-1 72
B-2 38
B3 75
B4 72
B5 85
B6 26
B7 50+



GIBBS - HOLTZ STANDARD PENETRATION CORRELATION	
Project No. G-13180 WOODWARD-CLYDE CONSULTANTS	Figure C34

ATTACHMENT E
(1/1.)

Geosyntec consultants

COMPUTATION COVER SHEET

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

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

Computations by:	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	10/31/07	Reviewed by:	G. Corcoran	Date:	12/3/07
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 compacted clay liner (CCL) having a saturated hydraulic conductivity less than 1×10^{-7} cm/s. 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):

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

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):
$k_{CCL} = 1 \times 10^{-7}$ cm/s
$t_{CCL} = 2$ ft (0.6 m)

Properties of Geosynthetic Clay Liner (GCL):
$k_{GCL} = 5 \times 10^{-9}$ cm/s (maximum hydraulic conductivity reported in manufacturer's documentation and typically allowed in technical specifications for GCLs)
$t_{GCL} = 0.20$ in. (5 mm) (minimum thickness reported in manufacturer's documentation and typically allowed in technical specifications for GCLs)

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:

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

Equation (3)

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 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):

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

Equation (4)

where:

t_0 = thickness of liquid above geomembrane, m
t_{LCL} = thickness of leak detection layer (geonet), 200 mils (0.005 m)
Q = flow rate through defect in geomembrane, $1.58 \times 10^{-5}\ m^3/sec$
k = permeability of geonet layer above secondary geomembrane

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

Attachment D, 2/3 shows a transmissivity curve for a ~~200~~ 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 hydraulic gradient is based on the longest drainage path (950 feet), slope of the geonet (1%), and height of liquid above the liner system (37 feet). Based on this information, the hydraulic gradient can be estimated as follows:

$$i = (37 \text{ ft} + 950 \text{ ft} \times 0.01) / 950 \text{ ft} = 0.049$$

Graphing the permeability data for the ~~200~~ 300 mil thick geonet under a normal stress of 5,000 psf (Attachment E, 1/1), results in the following equation of the line:

$$k = -0.2303 \ln (i) + 0.2809$$

Placing the estimated hydraulic gradient into the above equation results in a permeability of 9.76×10^{-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:

$$t_0 = \frac{0.005}{2}\left(1 + \frac{1.58x10^{-5}}{(9.76x10^{-1})(0.005^2)}\right) = 0.004 \text{ m} = 4 \text{ mm } (0.16 \text{ in.})$$

Flow rates through the CCL and GCL containing composite liner systems are evaluated using Equation (2), assuming a liquid head of 0.16 in. (4 mm). The results of the analysis and a sample calculation are presented below.

Head on secondary liner system	$q_{compCCL}/q_{compGCL}$
0.16 in. (4 mm)	8.51

Sample Calculation:

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

$$q_{comp\ CCL} / q_{comp\ GCL} = (1 \times 10^{-7} / 5 \times 10^{-9})^{0.74} [(1+0.1(0.16/24.0)^{0.95}) / (1+0.1(0.16/0.20)^{0.95}]$$

$$= 8.51$$

Thus, for a liquid head of 0.16 in. (4 mm) on the secondary geomembrane, the flow through the secondary composite liner system that includes a CCL is 8.51 times greater than the flow through the secondary liner system that includes a GCL instead of a CCL.

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 8.51 times greater than flow through the secondary liner system with GCL for a liquid head of 0.16in. (4 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.

REFERENCES

Giroud, J.P., Badu-Tweneboah, K., and Soderman, K.L. (1997), "Comparison of Leachate Flow Through Compacted Clay Liners and Geosynthetic Clay Liners in Landfill Liner Systems," Geosynthetic International, Vol. 4, No. 3-4, pp. 391-431. *(Attachment A)*

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 B)

4 FLOW THROUGH COMPOSITE LINERS

4.1 Introduction

As indicated in Section 2.8, GCLs used in landfills are always used as the low-permeability soil component of composite liners. In other words, GCLs used in landfills are always 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 permeability), 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:

$$\frac{q_{comp\,CCL}}{q_{comp\,GCL}} = \frac{0.21N\,[1 + 0.1(h/t_{CCL})^{0.95}]\,a^{0.1}\,h^{0.9}\,k_{CCL}^{0.74}}{0.21N\,[1 + 0.1(h/t_{GCL})^{0.95}]\,a^{0.1}\,h^{0.9}\,k_{GCL}^{0.74}} \tag{52}$$

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

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Discussion. It appears
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Table 7. Ratio between
and a composite liner in

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 value
following CCL characteristic
is the standard CCL defined i

ATTACHMENT A 1/2

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$$\frac{h^{0.9}\ k_{CCL}^{0.74}}{h^{0.9}\ k_{GCL}^{0.74}} \qquad (52)$$

r 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

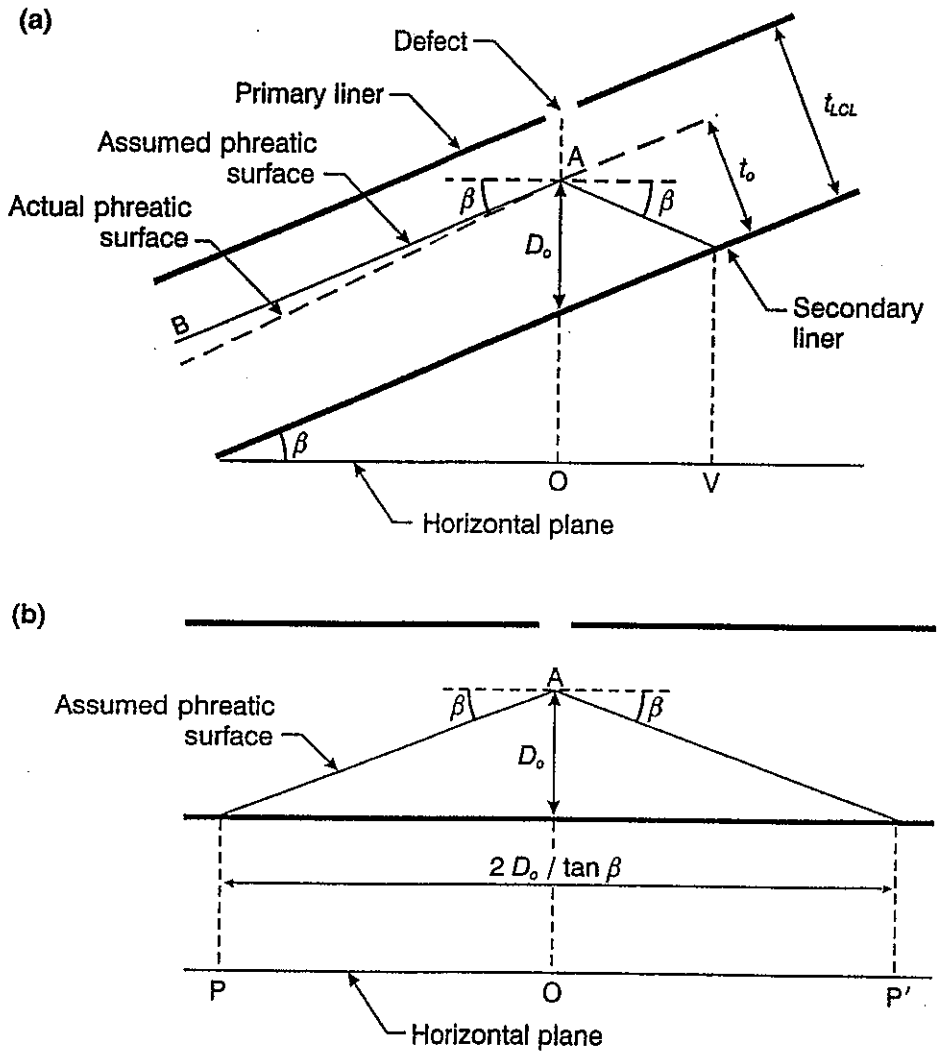

Figure 4. Assumed phreatic surface in the leakage collection layer in the case where the leakage collection layer is not filled with leachate: (a) cross section in a vertical plane along the slope and passing through the defect in the primary liner; (b) cross section in a vertical plane perpendicular 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:

$$t_{LCL} \geq t_{LCL,full} = \sqrt{\frac{Q}{k}} \tag{11}$$

where $t_{LCL,full}$ 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, $t_{LCL,full}$, 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 $t_{LCL,full}$ 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

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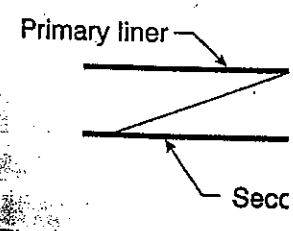

Figure 5. Vertical cross sec
layer in the case where the le
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ATTACHMENT B 3/6

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 line

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 thicknes Equation 4. The leachate thickn if $t_o > t_{LCL}$. The tabulated valu t_{LCL} and Equation 16 when t_o :

It appears from Table leachate on top of the s can convey significantly flow rates of Table 1 wit used alone (i.e. not part which is expressed as fo

$$Q = 0.6\, a \sqrt{2g}$$

where: a = defect area; = head of leachate on to

Table 2 gives rates of using Equation 20. It ap primary liners of activel provided that the geom

- a small geomembrane undetected during co per day (lpd);
- a geomembrane defec ing construction phas of granular leachate c on the order of 1000 l
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Attachment B, 4/5

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conductivity of 1×10^{-3} m/s.

ydraulic conductivity and the
et, the virtual leachate thick-
the thickness of the leachate
ı Table 1, the geonet is filled
rea being zero for $t_o = 5$ mm).
cknesses considered in Table
collection layer; therefore, in
r are not filled (or just filled)
icknesses, t_o, shown in Table

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)	(m^3/s)	(lpd)	(m^3/s)	(lpd)	(m^3/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 m^3/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:

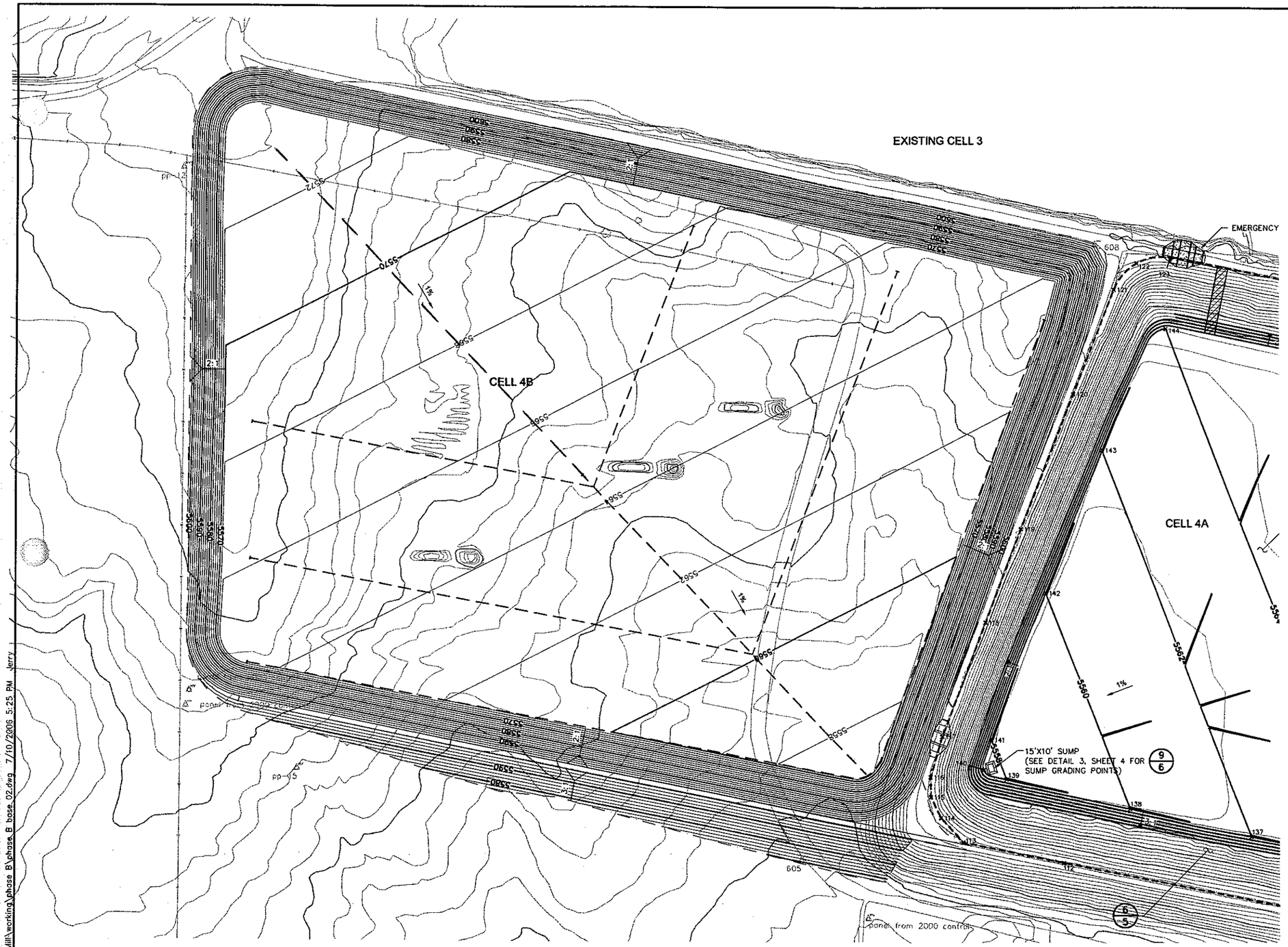
$$Q = 0.6\, a \sqrt{2\, g\, h_{prim}} = 0.6\, \pi\, (d^2/4) \sqrt{2\, g\, h_{prim}} \approx (2/3)\, d^2 \sqrt{g\, h_{prim}} \qquad (20)$$

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 m^3/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 m^3/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

N

100 50 0 100 200
SCALE IN FEET

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

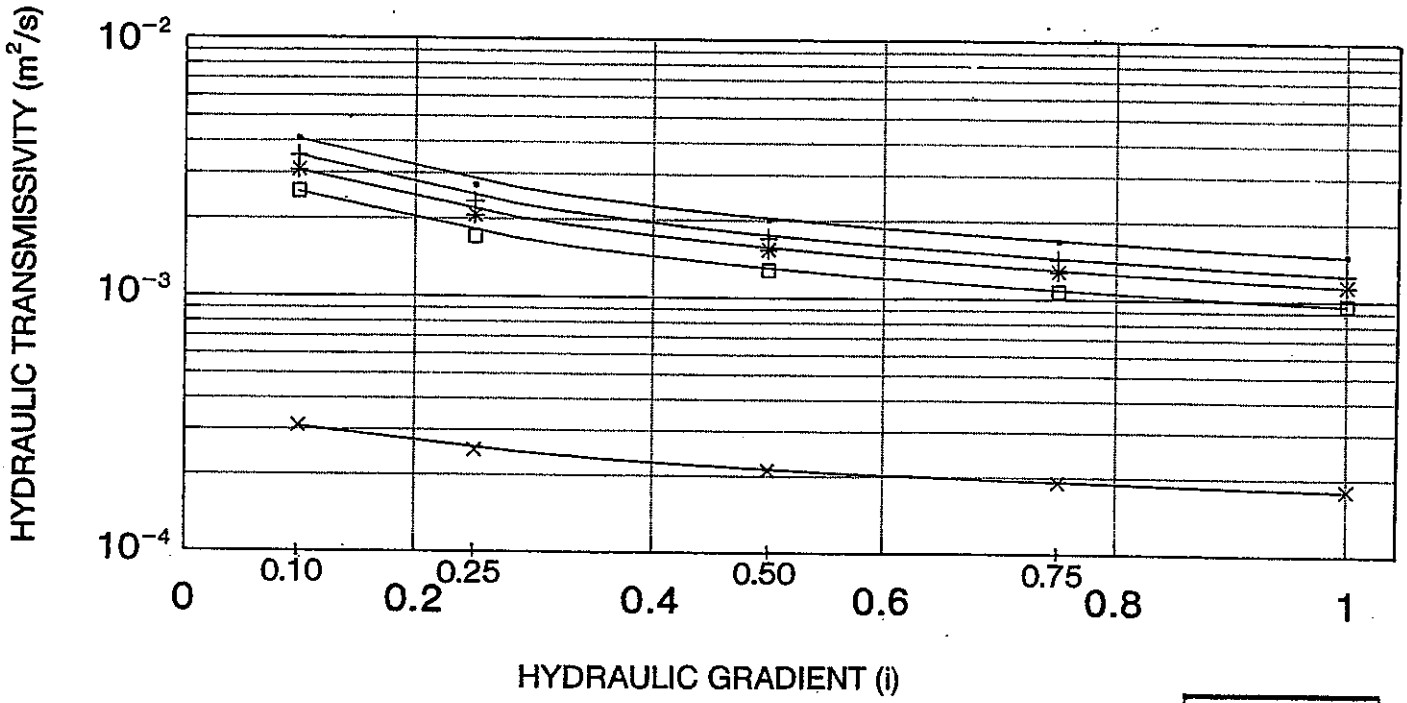
FLUID SYSTEMS, INC.

POLY-NET®
TRANSMISSIVITY CHARTS

ATTACHMENT D 1/3

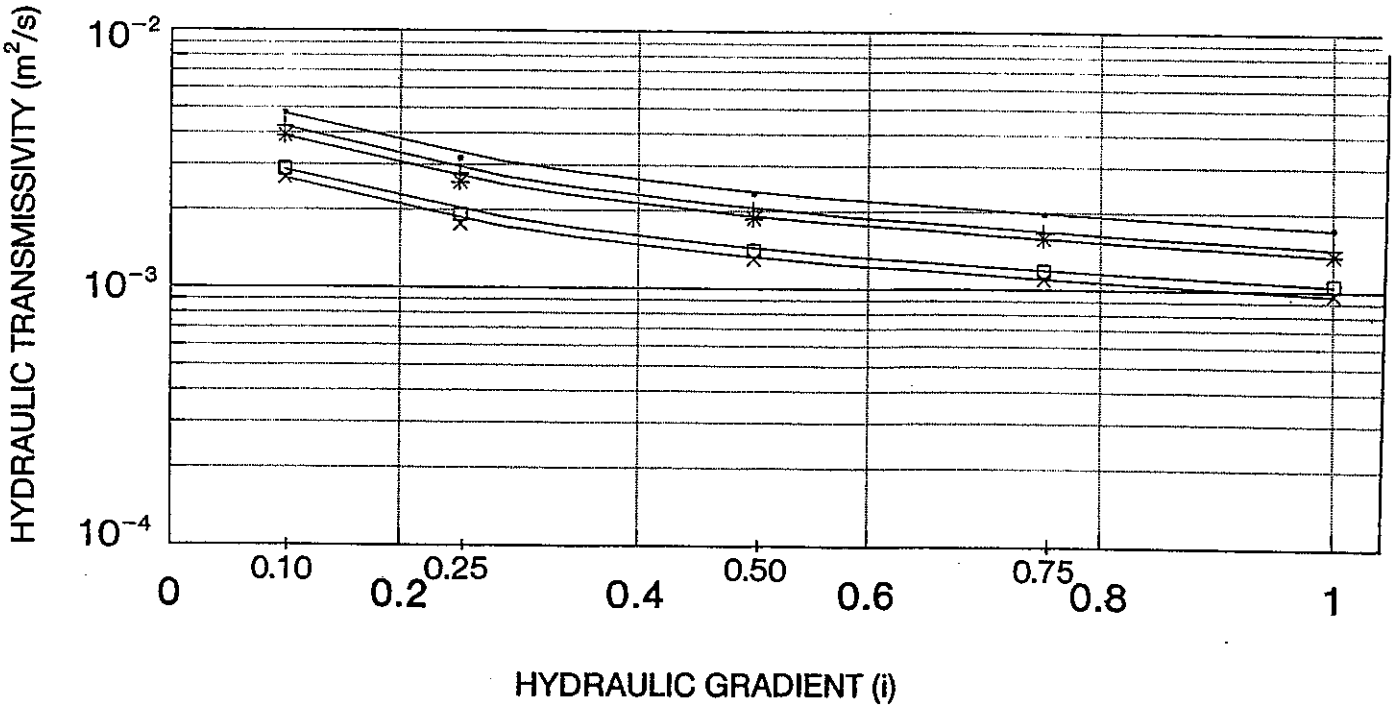
POLY-NET PN2000

HDPE/PN2000/HDPE



POLY-NET PN3000

HDPE/PN3000/HDPE



ATTACHMENT D 2/3

SPECIFICATION — Minimum Average Roll Values

Raw material	PN-2000 polyethylene	PN-3000 polyethylene	PN-3000CN* polyethylene	PN-4000* polyethylene
Weight (lbs/ft²) D-3776	0.117	0.162	0.115	0.220
Thickness (inches) D-751	0.160	0.200	0.200	0.270
Density of polymer (g/cm³) D-1505**	0.940	0.940	0.940	0.940
Tensile strength (lb/in) D-1682 (Modified)	30	42	23	44
Carbon black ASTM D-1603 (%)	2	2	2	2
Porosity (%), Nom.	83	80	76	67
Roll width (feet), Nom.	7.54	7.54 & 14.5	7.54	7.54
Standard roll length (feet), Nom.	300	300	300	220
Area per roll (ft²), Nom.	2262	2262 & 4350	2262	1659

*Foamed
**Fully compounded resin without foam

The transmissivity results listed on the preceding pages were determined in compliance with ASTM D4716-87 test procedure. The transmissivity was measured using water @ 20°C (68°F) with a seat time of one hour. Values may vary, based on dimensions of the transmissivity specimen and specific laboratory.

CONVERSION FACTORS FOR TRANSMISSIVITY UNITS

$1 m^2/s$ = 1 cubic meter/second/meter width/unit gradient
$1 m^2/s = 10^3$ liters/second/meter width/unit gradient
$1 m^2/s = 6 \times 10^4$ liters/minute/meter width/unit gradient
$1 m^2/s$ = 10.76 ft^2/second
$1 m^2/s$ = 646 ft^2/minute
$1 m^2/s$ = 4830 gallons/minute/foot width/unit gradient

1 ft^2/second = 1 cubic foot/second/foot width/unit gradient
1 ft^2/second = $9.3 \times 10^{-2} m^2/s$
1 ft^2/minute = 1 cubic foot/minute/foot width/unit gradient
1 ft^2/minute = $1.55 \times 10^{-3} m^2/s$
1 gpm/foot width/unit gradient = $2.07 \times 10^{-4} m^2/s$
1 liter/minute/meter width/unit gradient = $1.66 \times 10^{-5} m^2/s$
100 liters/minute/meter width/unit gradient = 1.07 ft^2/min.

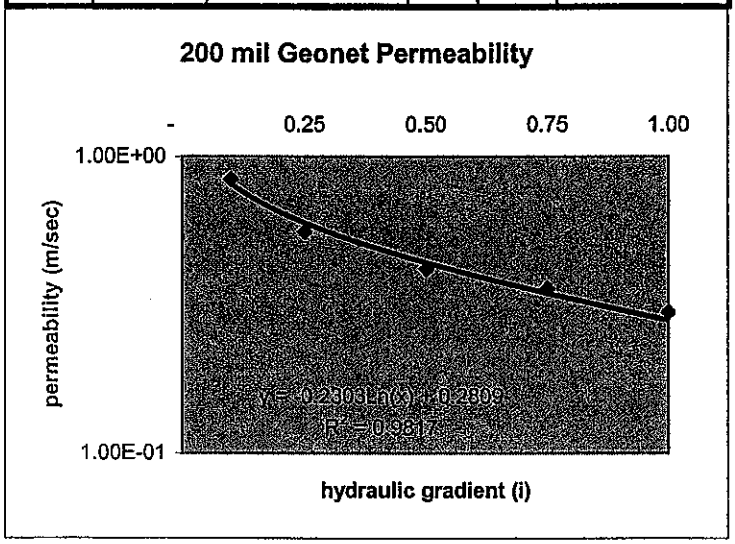
The information contained herein has been compiled by Fluid Systems, Inc. and is, to the best of our knowledge, true and accurate. All suggestions and recommendations are offered without guarantee. Final determination of suitability for use based on any information provided, is the sole responsibility of the user. There is no implied or expressed warranty of merchantability or fitness of the product for the contemplated use.

FLUID SYSTEMS, INC.
32 Triangle Park Drive, Suite 3201
Cincinnati, OH 45246
Phone: 513/771-5656
800/346-9107
Fax: 513/771-4844

ATTACHMENT D 3/3

NSC Polynet 3000 (HDPE/GN/HDPE)

i	Normal Stress (psf)	Transmissivity (m2/sec)	Thickness (mil)	Thickness (mm)	Permeability (m/sec)
0.10	5,000	4.20E-03	200	5	8.40E-01
0.25	5,000	2.80E-03	200	5	5.60E-01
0.50	5,000	2.10E-03	200	5	4.20E-01
0.75	5,000	1.80E-03	200	5	3.60E-01
1.00	5,000	1.50E-03	200	5	3.00E-01



ATTACHMENT E, 1/3

$$Q = (2/3)\, d^2\, (g\, h_{prim})^{1/2}$$

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**
200	mil	t_{LCL}	thickness of leachate collection layer
0.0051	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	m3/sec	Q	steady state rate of leachate flow in the leakage collection layer
37	ft	h_{prim}	head of leachate on top of the primary liner (max height of 40' - freeboard of 3')
950	ft	L	maximum drainage length
0.049		i	hydraulic gradient
9.76E-01	m/sec	k	permeability of geonet at hydraulic gradient in cell
t_0 =	**0.004**	**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

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations **EMERGENCY SPILLWAY CONCRETE PAVEMENT**

Computations by:	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		7/23/07
	Printed Name	Gregory T. Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		7/23/07
	Printed Name	Gregory T. Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran, P.E.	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	07/09/07	Reviewed by:	G. Corcoran	Date:	7/23/07
Client:	DMC	Project:	Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	

EMERGENCY SPILLWAY CONCRETE PAVEMENT

OBJECTIVE

An emergency spillway will be constructed as part of the Cell 4B construction at the White Mesa Mill, in Blanding, Utah for the Denison Mines (USA) Corp. (DMC). The emergency spillway is 100 feet wide with a 15-foot wide access road across the crest of the spillway. The emergency spillway is shown on Sheet 7 of the project plans "Lining System Details III, Pond 4B, White Mesa Mill, Blanding Utah," prepared by Geosyntec Consultants. A pick-up truck design loading has been assumed for the pavement of the access road.

The concrete design of the concrete slab will be performed in accordance with American Concrete Institute (ACI) Publication 318 standards (ACI 318). The objective of this design is to determine the dimensional, reinforcement, and concrete requirements necessary to withstand the applied loading.

SUMMARY OF DESIGN

Based on the assumptions and calculations presented herein, the slab on grade will be 6 inches thick and consist of concrete with compressive strength of 3,000 pounds per square inch (psi), and welded wire reinforcement (WWR) fabric sized as 6x6 – W1.4xW1.4.

ANALYSIS

Loading conditions

The loading conditions for the slab-on-grade for the emergency spillway is assumed to be a 12,000 lb loaded pick-up truck, which equates to an assumed 4,000 lb front axle loading and a maximum 8,000 lb rear axle loading. A wheel spacing of 72 inches was assumed.

To determine the required slab thickness, a Wire Reinforcement Institute (WRI) method, which simulates concentrated loads as the loading resulting from a single-axle,

will be utilized. The method is presented in a report by ACI on the design of slabs on grade (ACI 360).

Slab on Grad Design Proceedure

WRI presents a slab on grade thickness determination method based on the concentrated loads from the wheels of a forklift (ACI 360). The method accounts for the total axle load and each wheel individually. The method also takes into account the moments on the slab caused by the spacing of the wheels.

The determination of slab thickness can be made by following the example presented in Appendix A of the ACI 360. To utilize this method, some values and properties were required to be assumed. Modifications to the design may be required if the assumptions are determined to not be valid.

The design begins with the assumed values of:

- Concrete modulus (E_c) = 3,000 kips per square inch (ksi)
- Subgrade modulus (k) = 400 pounds per cubic inch (pci)
- Compressive strength of concrete (f_c') = 3,000 psi
- Modulus of rupture (MOR or f_r) = $7.5 \cdot \sqrt{f_c'} \approx 411 psi$
- Slab thickness = 6 inches

Note that 1 kip equals 1,000 lbs. The assumed subgrade modulus value is for a Sandy soil (Attachment A). The compressive concrete strength, and therefore the modulus of rupture, can be specified when ordering concrete; this assumed value will not likely require modification. The value of 6 inches for the slab thickness is an arbitrary trial "guess", which will be validated or dispelled at the end of applying the method.

Figure A2.2.1 (Attachment B) is utilized to find the relative stiffness parameter (D/k) based on the above assumed values. The first trial results in a D/k value of 1.5×10^5 in^4.

Next, the contact area for each wheel must be converted to determine the diameter of a hypothetical circle that has the same area. A tire air pressure of 80 psi was assumed for a loaded truck from Section 4.2. The following basic equations were used to determine the equivalent circle diameter:

Area of a circle:

$$A = \pi\left(\frac{D}{2}\right)^2$$

Where A is the area of the circle (based on tire pressure and tire load, Attachment E) and D is the diameter of the circle. Rearranging and solving for D:

$$D = 2\sqrt{\frac{A}{\pi}} = 2\sqrt{\frac{50\,in^2}{3.14}} = 8.0\ in$$

Therefore a circle with a 8.0-in. diameter has an area approximately equal to the contact area of one vehicle wheel (50-in^2).

Next, the distance between wheels on the axle must be incorporated into the design method. The length between the back two wheels on a pick-up truck is utilized to determine the equivalent forklift axle wheel spacing. This distance was assumed to be 72-in.

The equivalent wheel base, equivalent contact circle diameter, and the D/k value are then utilized to determine the basic bending moment in the slab (in-lb/in) that results per kip of wheel load applied. From Figure A2.2.2 (Attachment C), we see that the basic bending moment due to the two wheels is 200 plus 5 in-lb/in/kip, which results in a total moment of approximately 205 in-lb/in per kip stress. This value is multiplied by the "wheel" load to give the design moment. Based on a total vehicle operating weight of 10,000 lbs. The wheel load is:

$$"Wheel\ load" = \frac{Total\ axle\ weight}{\#\ of\ wheels} = \frac{8{,}000\,lbs}{2} = 4{,}000\frac{lbs}{wheel} = 4.0\frac{kip}{wheel}$$

Multiplying the basic moment by the "wheel load", the resulting design moment is:

$$Design\ moment = basic\ moment \times wheel\ load = \left(205\frac{\frac{in-lb}{in}}{kip}\right) \times (4.0\,kip) = 820\frac{in-lb}{in}$$

This design moment and the total allowable flexural stress are utilized to assess if the initial guess for slab thickness is valid. The total allowable flexural stress is the MOR (f_r) divided by a safety factor (SF). For concentrated loads, ACI 360 recommends a SF value between 1.7 and 2.0. For this design, the lower value of 1.7 will be utilized. The 1.7 SF value results in a total allowable tensile stress of:

$$\frac{MOR}{SF} = \frac{411\,psi}{1.7} = 242\,psi$$

Using this MOR/SF value and the design stress with Figure A2.2.3 (Attachment D), we check to see if our initial concrete thickness guess was accurate. With the values calculated above, we see that the resulting thickness is approximately 5 in. The calculations and resulting values are summarized in a spreadsheet, presented on Attachment E.

Temperature & Shrinkage Reinforcement Design

The subgrade drag equation (ACI 360) is used to determine the minimum area of steel reinforcement required to prevent temperature and shrinkage cracking:

$$A_{s,\min} = \frac{F \cdot L \cdot w}{2 \cdot f_s}$$

Where the variables are defined as follows:

$A_{s,min}$ = *minimum cross-sectional area of steel per foot width concrete*

F = subgrade friction factor, for granular subbase = 1.5 *(Section 6.3))*
L = distance between joints in slab = 15 ft
w = dead weight of slab, 12.5 lb per inch of 6-in. slab = 75 lbs
f_y = yield strength of reinforcement steel, ASTM A610 = 60,000 psi
f_s = allowable tensile strength of reinforcement (.75f_y) = 45,000 psi

Substitution of variables in the preceding equation yields:

$$A_{s,\min} = \frac{(1.5)\cdot(15\,\text{ft})\cdot(75\,\text{lb})}{(2)\cdot(45{,}000\,\text{psi})} = 0.019\,\text{in}^2/\text{ft}$$

The result indicates that a minimum reinforcement area of 0.019-in^2 must be provided per each foot of slab length and width. This value is lower than the A_s value provided by the narrowest rebar (#3) at the maximum recommended spacing (18-in.), which provides an A_s value of 0.073-in.2. (By ACI 318 standards, reinforcement spacing must not exceed 18-in. or the lesser of three times the slab thickness (also 18-in.). According to the WRI Manual of Standard Practice (WRI Manual), if welded reinforcement wire (WWR) were to be utilized, the product size denoted as 6x6 – W1.4xW1.4 would provide an A_s value of 0.028-in^2 (Attachment F). This value slightly exceeds the required $A_{s,min}$ value of 0.019-in^2, and requires considerably less steel than the minimum value provided if rebar were to be used. The WWR 6x6 – W1.4xW1.4 will be utilized.

The development length of the reinforcement steel must be checked to determine whether or not the steel has a chance to fully develop its tensile strength. According to the WRI Manual, the equation for determining the required development length (ℓ_d) for welded plain wire is as follows:

$$\ell_d = 0.27\frac{A_w}{S_w}\left(\frac{f_y}{\sqrt{f'_c}}\right)\cdot\lambda$$

Where the variables are defined as follows:

A_w = area of individual wire to be developed = 0.014-in^2
S_w = spacing of wires to be developed or spliced = 6-in

λ = concrete weight factor, normal weight concrete = 1.0
f_y = 60,000 psi
f'_c = 45,000 psi

The minimum development length shall be the greater of the calculated ℓ_d value or 8 inches. Plugging in the variables and solving the equation:

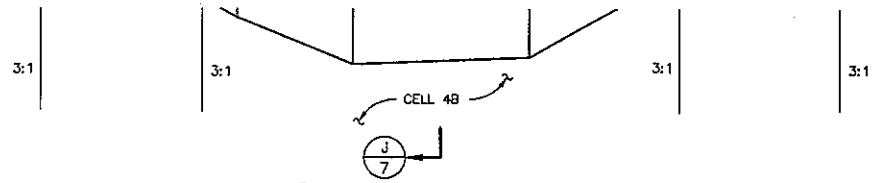
$$\ell_d = 0.27\frac{0.014}{6.0}\left(\frac{60{,}000}{\sqrt{45{,}000}}\right)\cdot 1.0 = 0.178\,in$$

This value is less than the minimum specified development length of 8-in., so at least 8-in. is required for the WWR to develop its tensile strength. Since the minimum length of the concrete slab is 15 feet, or 180 inches, the reinforcement steel will fully develop its tensile strength and function as desired.

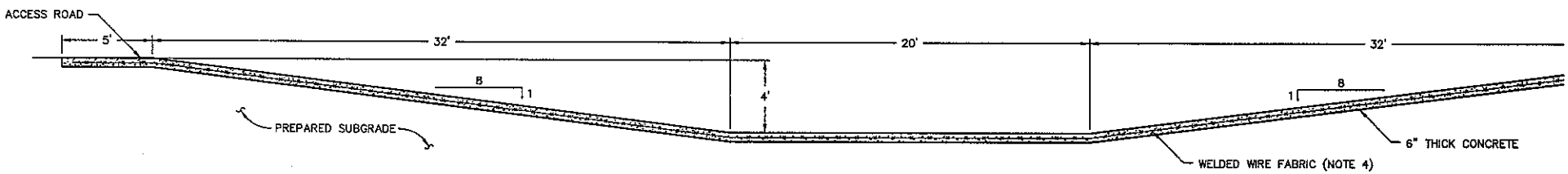
REFERENCES

American Concrete Institute, *Design of Slabs on Grade (ACI 360R-92),* ACI: Farmington Hills, Michigan 1997.

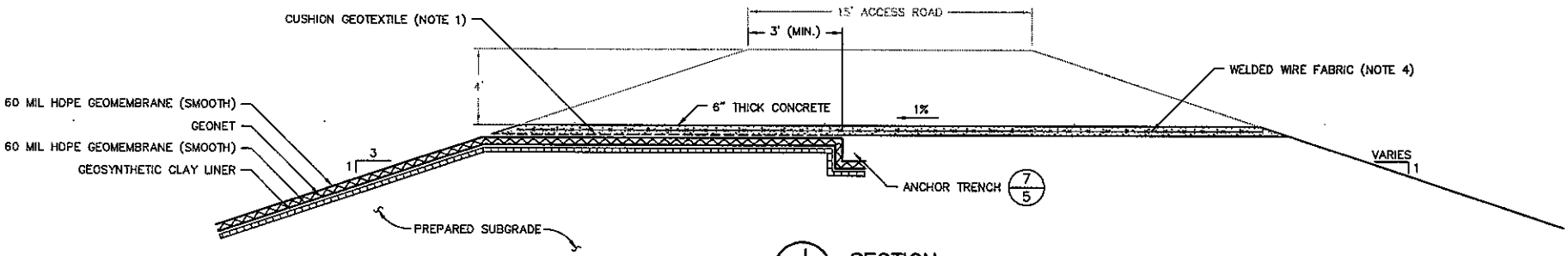
Wire Reinforcement Institute, Incorporated, *Manual of Standard Practice: Structural Welded Wire Reinforcement,* WWR-500, 6th Edition, 2001



10/3 PLAN
EMERGENCY SPILLWAY
SCALE: 1" = 10'



1/7 SECTION
EMERGENCY SPILLWAY
SCALE: 1" = 4'



J/7 SECTION
EMERGENCY SPILLWAY
SCALE: 1" = 4'

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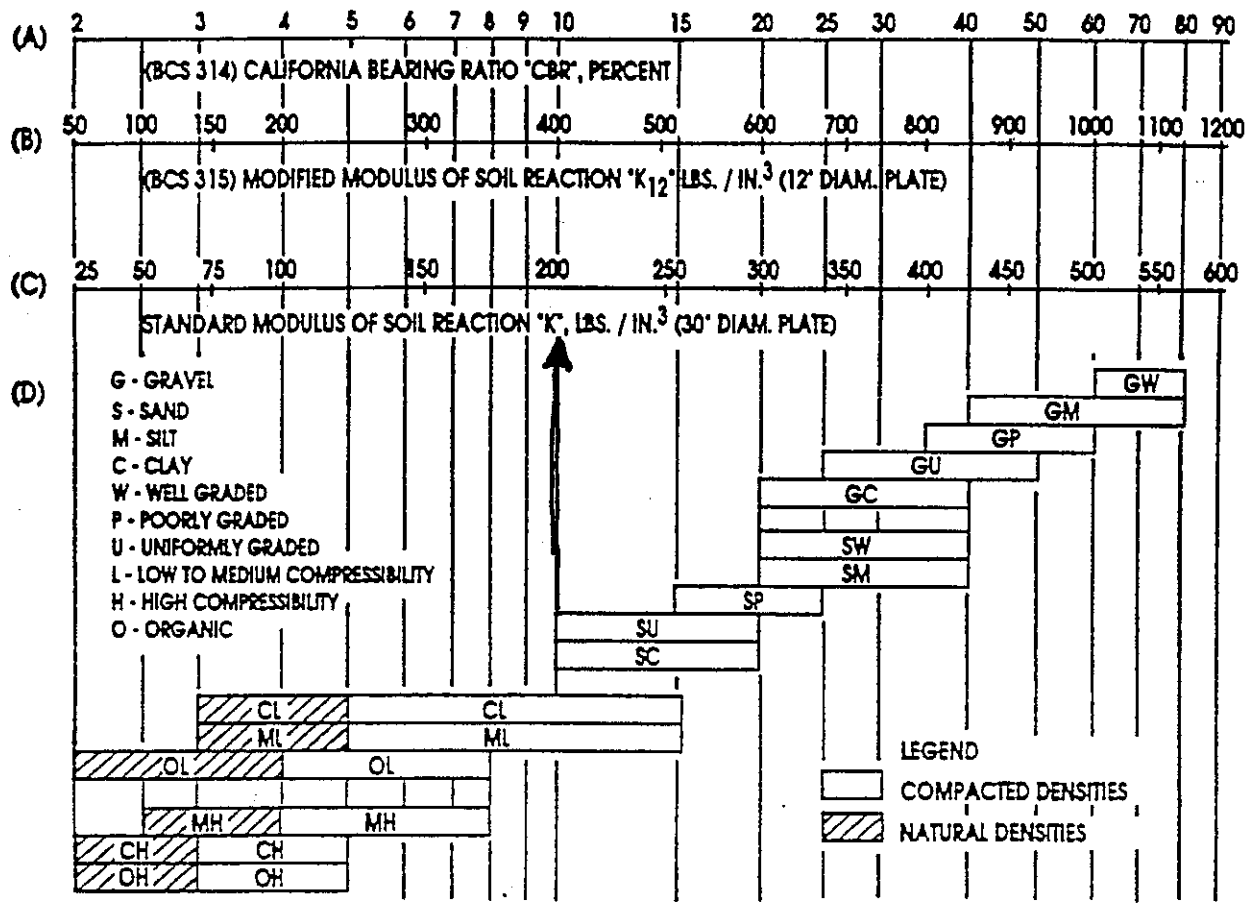
FIGURE 1



Note: Comparison of soil type to "K", particularly in the "L" and "H" Groups, should generally be made in the lower range of the soil type.

Fig. 3.3.5—Interrelationship of soil classifications and strengths (from Reference 23)

sand or gravel fill, or use the existing material in its in-situ condition.

Normally there is a wide range of soils across the site. The soil support system is rarely uniform. Therefore, some soil work is generally required to provide a more uniform surface to support the slab. The extent of this work, such as the degree of compaction or the addition of a sand-gravel base, is generally a problem of economics. Selection of soils in the wellgraded gravel (GW) and poorly graded gravel (GP) groups as a base material may appear costly. However, the selection of these materials has distinct advantages. Not only do they provide a superior modulus of subgrade reaction, but they also tend to speed construction during inclement weather.

3.4.2 *Economics and simplified design*—Certainly not all projects will require all of the data discussed above. On projects where the slab performance is not critical, engineering judgement should be exercised to reduce costs. A prime prerequisite for the proper design of a slab support system is soils identification. Without this knowledge, the modulus of subgrade reaction is unknown and potential volume change cannot be determined. With knowledge of soil classification, the engineer can select an appropriate k value and design for the specific soil conditions.

For small projects, it may be advantageous to assume a low k factor and add a selected thickness of crushed stone to enhance the safety factor rather than performing an expensive soil analysis. Use of the modified modulus of subgrade reaction test rather than the standard modulus test can also reduce costs. Risk of slab failure at an earlier age increases as the design is rationalized but there are occasions where the simplified design approach is justified. These decisions are a matter of engineering judgment and economics.

Compounding safety factors is a common error. Inclusion of safety factors in the modulus of subgrade reaction, the applied loads, the compressive strength of the concrete, the flexural strength of the concrete and the number of load repetitions will produce an expensive design. The safety factor is normally contained in the flexural strength of the concrete and is a function of the number of load repetitions (see Sec. 4.9).

3.5—Site preparation

3.5.1 *Introduction*—Prior to soil compaction, the top

ATTACHMENT A

CHAPTER A2—SLAB THICKNESS DESIGN BY WRI METHOD

A2.1—Introduction

The following two examples show the determination of thickness for a slab on grade intended to have mild steel reinforcement for shrinkage and temperature stresses. The amount of steel is commonly selected using the subgrade drag theory presented in Chapter 6 and discussed in Reference 53.

The design charts are for a single axle loading with two single wheels and for the controlling moment in an aisle with uniform loading on either side. The first situation is controlled by tension on the bottom of the slab and the second is controlled by tension on the top of the slab. Both procedures start with use of a relative stiffness term D/k, and require the initial assumption of the concrete modulus of elasticity E and slab thickness H, as well as selecting the allowable tensile unit stress and the appropriate subgrade modulus k.

A2.2—WRI thickness selection for single-axle wheel load

This procedure selects the concrete slab thickness for a single axle with wheels at each end of the axle, using Fig. A2.2.1, A2.2.2, and A2.2.3. The procedure starts with Fig. A2.2.1 where a concrete modulus of elasticity E and slab thickness H, and modulus of subgrade reaction k are assumed or known. For example, taking

E = 3000 ksi
Thickness = 8 in. (trial value)
Subgrade modulus k = 400 pci

Fig. A.2.2.1 gives the relative stiffness parameter D/k = 3.4 x 10^5 in.^{4} The procedure then uses Fig. A2.2.2.
Wheel Contact Area = 28 sq in.

Diameter of equivalent circle = $\sqrt{[28 \times 4]/\pi}$
= 6 in.

Wheel spacing = 45 in.

This gives the basic bending moment of 265 in.-lb/in. of width/kip of wheel load for the wheel load using the larger design chart in Fig. A2.2.2. The smaller chart in the figure gives the additional moment due to the other wheel as 16 in.-lb per in. of width per kip of wheel load.
Moment = 265 + 16 = 281 in.-lb/in./kip
(Note that in.-lb/in. = ft-lb/ft)

Axle Load = 14.6 kips
Wheel Load = 7.3 kips

Design Moment = 281 x 7.3 = 2051 ft-lb/ft

Then from Fig. A2.2.3:
Allowable tensile stress = 190 psi

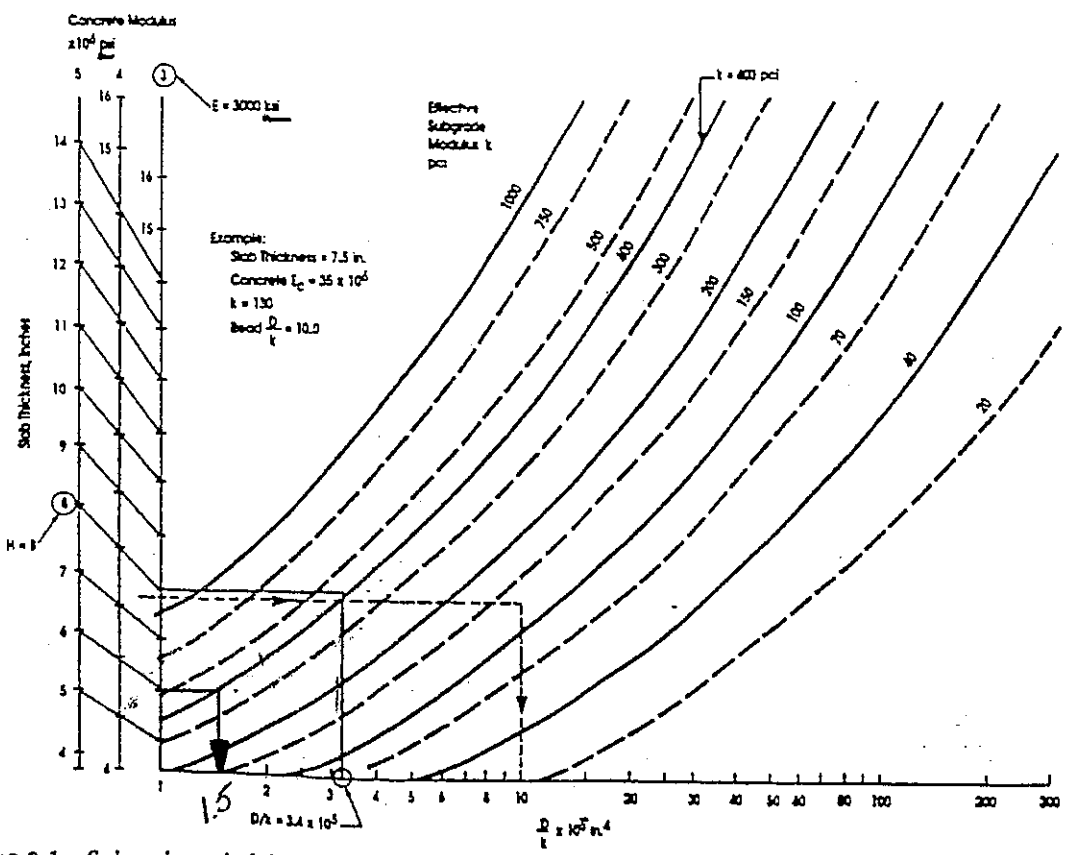


Fig. A2.2.1—Subgrade and slab stiffness relationship, used with WRI design procedure

ATTACHMENT B

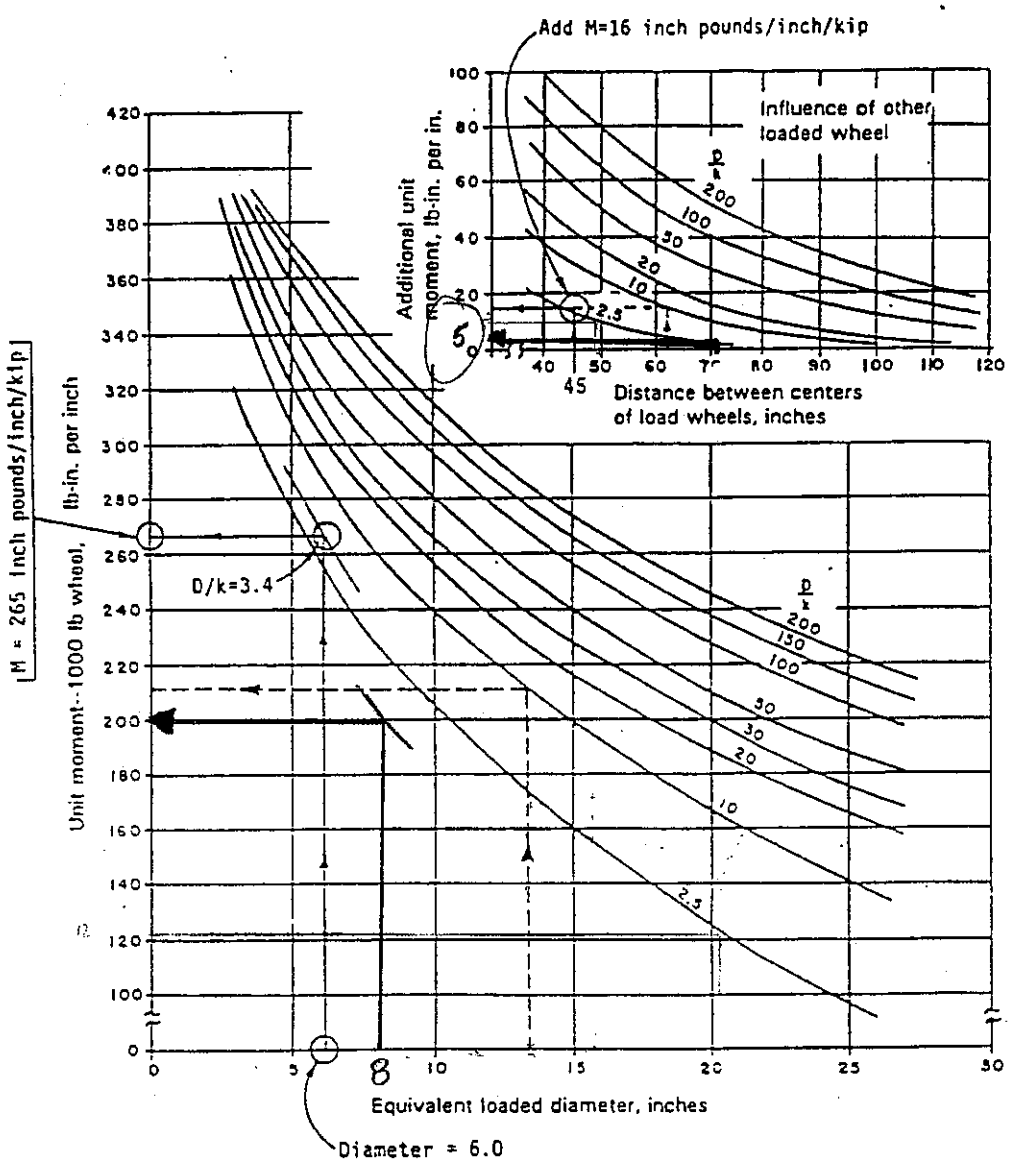


Fig. A2.2.2—Wheel loading design chart used with WRI procedure

Solution:
Slab thickness (H) = 7 7/8 in.

If the design thickness differs substantially from the assumed thickness, the procedure is repeated with a new assumption of thickness.

A2.3—WRI thickness selection for aisle moment due to uniform loading

The procedure for the check of tensile stress in the top of the concrete slab due to this loading uses Fig. A2.2.1 and A2.3. Note that Fig. A2.2.3 is a part of Fig. A2.3., separated here for clarity of procedure.

The procedure starts as before with determination of the term $D/k = 3.4 \times 10^5$ in.^{4} It then goes to Fig. A2.3 as follows:
Aisle width = 10 ft = 120 in.
Uniform load = 2500 psf = 2.5 ksf
Allowable tension = MOR/SF = 190 psi

The solution is found by plotting up from the aisle width to D/k, then to the right-hand plot edge, then down through the uniform load value to the left-hand edge of the next plot, then horizontally to the allow-able stress and down to the design thickness.
Solution: Thickness = 8.0 in.

Again, if the design thickness differs substantially from the assumed value, the process should be repeated until reasonable agreement is obtained.

A2.4—Comments

These procedures assume the use of conventional steel reinforcement in the concrete slab. The applied moments from the loads are not used in selecting the steel reinforcement except in the case of a Type F structurally reinforced slab.

ATTACHMENT C

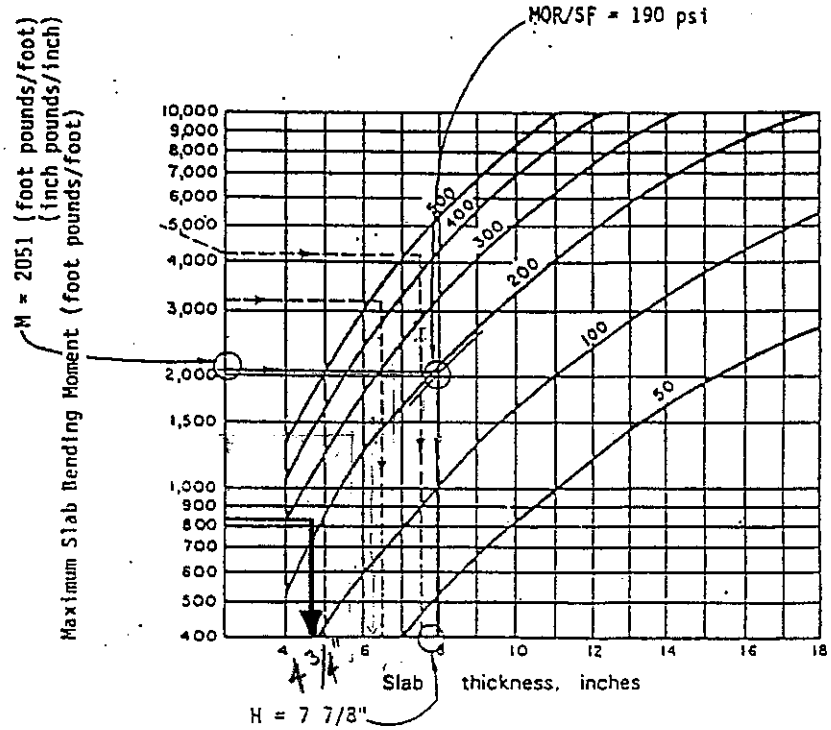


Fig. A2.2.3—Slab tensile stress charts used with WRI design procedure

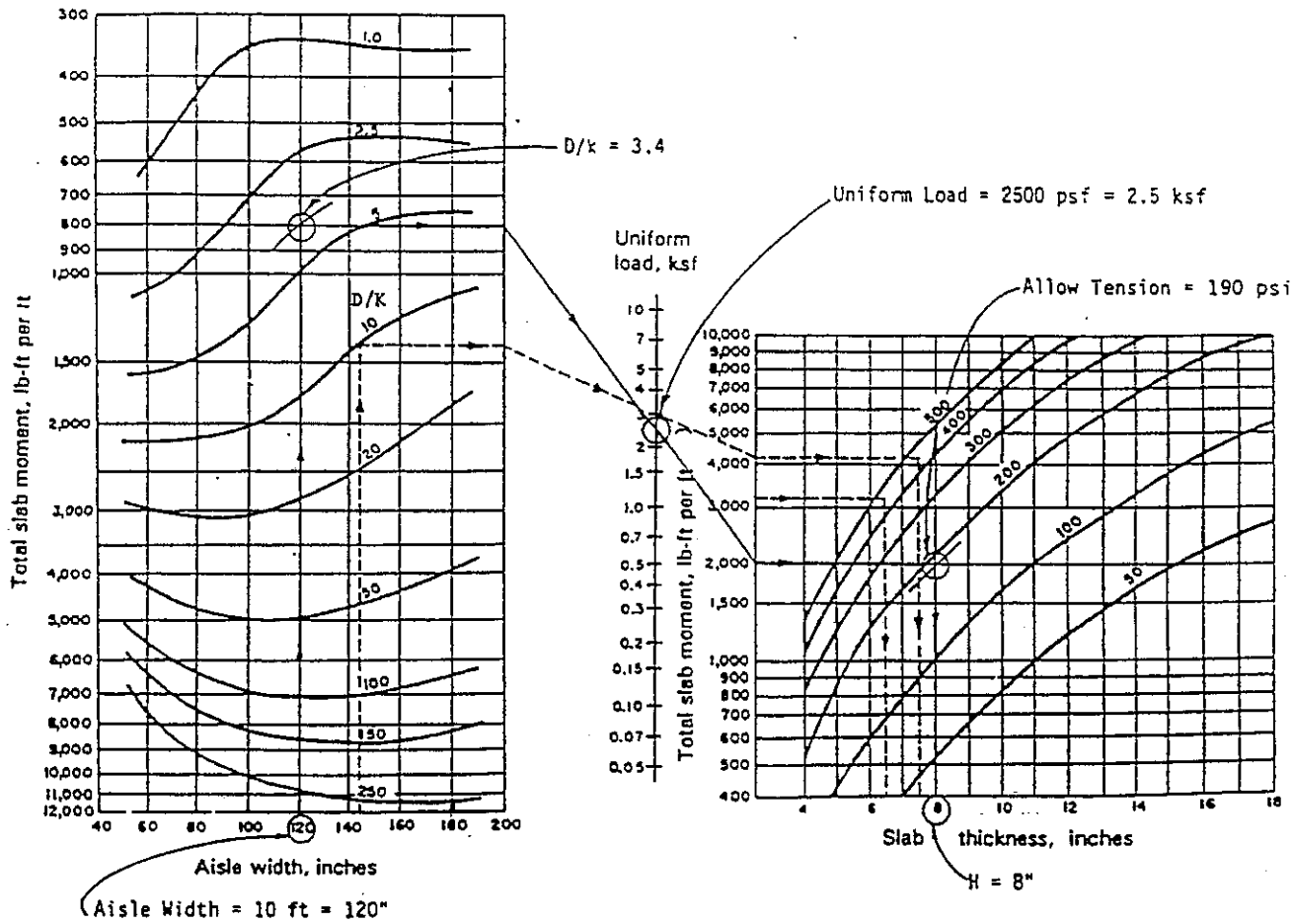


Fig. A2.3—Uniform load design and slab tensile stress charts used with WRI design procedure

ATTACHMENT D

Slab Design Thickness Determination

	Trial 1	Trial 2	Trial 3		
Wheel spacing =	**60**	**60**	**60**	**in**	*distance between wheels on same axle
Total working vehicle weight =	12,000	12,000	12,000	lb	Assume Pick-up Truck Loading
Rear Axle Load =	8,000	8,000	8,000	lb	Assume Pick-up Truck Loading
"Axle" load =	8.0	8.0	8.0	kip	rear axle
"Wheel" load =	4.0	4.0	4.0	kip	half of axle load
Wheel Tire Pressure =	80.0	80.0	80.0	psi	80 to 120 psi for pneumatic tires
Wheel Contact area =	50.0	50.0	50.0	sq in	
Wheel Equivalent Diameter =	8.0	8.0	8.0	in	
Concrete modulus (E) =	3,500	3,500	3,500	ksi	
Compressive strength of concrete (*f'c*) =	3,000	3,500	4,000	psi	
Subgrade Modulus (k) =	400	400	400	pci	Fig 3.3.5
Concrete Modulus of Rupture (MOR, f_r) =	411	444	474	psi	
Safety Factor (SF) =	1.7	1.7	1.7	---	
Allowable tensile stress (MOR/SF) =	242	261	279	psi	

WRI Method - Single Axle Wheel Load

		Trial			
	Units	*1*	*2*	*3*	
Trial Thickness =	in	6.0	6.0	6.0	
Stiffness parameter (D/k) =	x 10^{-5} in^4	1.5	1.5	1.5	(from Fig A2.2.1)
Basic bending moment per kip stress =	in-lb/in/kip	200	200	200	(from Fig A2.2.2)
Moment due to other wheel =	in-lb/in/kip	5	5	5	(from Fig A2.2.2)
Total moment per kip stress =	in-lb/in/kip	205	205	205	
Design moment =	in-lb/in	820	820	820	
SLAB THICKNESS =	**in**	**5.0**	**4.5**	**4.5**	(from Fig. A2.2.3)

**Highlighted values are calculated from other entered values.*

6-in. is an acceptable design slab thickness.

ATTACHMENT E

Slab Design Thickness Determination

	Trial 1	Trial 2	Trial 3		
Wheel spacing =	**72**	**72**	**72**	**in**	*distance between wheels on same axle
Total working vehicle weight =	12,000	12,000	12,000	lb	Assume Pick-up Truck Loading
Rear Axle Load =	8,000	8,000	8,000	lb	Assume Pick-up Truck Loading
"Axle" load =	8.0	8.0	8.0	kip	rear axle
"Wheel" load =	4.0	4.0	4.0	kip	half of axle load
Wheel Tire Pressure =	80.0	80.0	80.0	psi	80 to 120 psi for pneumatic tires
Wheel Contact area =	50.0	50.0	50.0	sq in	
Wheel Equivalent Diameter =	8.0	8.0	8.0	in	
Concrete modulus (E) =	3,500	3,500	3,500	ksi	
Compressive strength of concrete (*f'c*) =	3,000	3,500	4,000	psi	
Subgrade Modulus (k) =	400	400	400	pci	Fig 3.3.5
Concrete Modulus of Rupture (MOR, f_r) =	411	444	474	psi	
Safety Factor (SF) =	1.7	1.7	1.7	---	
Allowable tensile stress (MOR/SF) =	242	261	279	psi	

WRI Method - Single Axle Wheel Load

		Trial			
	Units	*1*	*2*	*3*	
Trial Thickness =	in	6.0	6.0	6.0	
Stiffness parameter (D/k) =	x 10^{-5} in^4	1.5	1.5	1.5	(from Fig A2.2.1)
Basic bending moment per kip stress =	in-lb/in/kip	200	200	200	(from Fig A2.2.2)
Moment due to other wheel =	in-lb/in/kip	5	5	5	(from Fig A2.2.2)
Total moment per kip stress =	in-lb/in/kip	205	205	205	
Design moment =	in-lb/in	820	820	820	
SLAB THICKNESS =	**in**	**5.0**	**4.5**	**4.5**	(from Fig. A2.2.3)

**Highlighted values are calculated from other entered values.*

6-in. is an acceptable design slab thickness.

ATTACHMENT E

Sectional Areas of Welded Wire Reinforcement

TABLE 5 Customary Units

Wire Size Number		Nominal Diameter	Nominal Weight	As - Square Inch Per Linear Feet *Center to Center Spacing*								
Plain	*Deformed*	*Inches*	*Lbs./Lin. Ft.*	2"	3"	4"	6"	8"	10"	12"	16"	18"
W45	D45	0.757	1.530	2.70	1.80	1.35	.909	.68	.54	.45	.34	.30
W31	D31	0.628	1.054	1.86	1.24	.93	.62	.47	.37	.31	.23	.21
W20	D20	0.505	.680	1.20	.80	.60	.40	.30	.24	.20	.15	.13
W18	D18	0.479	.612	1.08	.72	.54	.36	.27	.216	.18	.14	.12
W16	D16	0.451	.544	.96	.64	.48	.32	.24	.192	.16	.12	.11
W14	D14	0.422	.476	.84	.56	.42	.28	.21	.168	.14	.11	.09
W12	D12	0.391	.408	.72	.48	.36	.24	.18	.144	.12	.09	.08
W11	D11	0.374	.374	.66	.44	.33	.22	.165	.132	.11	.08	.07
W10.5		0.366	.357	.63	.42	.315	.21	.157	.126	.105	.08	.07
W10	D10	0.357	.340	.60	.40	.30	.20	.15	.12	.10	.08	.07
W9.5		0.348	.323	.57	.38	.285	.19	.142	.114	.095	.07	.06
W9	D9	0.338	.306	.54	.36	.27	.18	.135	.108	.09	.07	.06
W8.5		0.329	.289	.51	.34	.255	.17	.127	.102	.085	.06	.06
W8	D8	0.319	.272	.48	.32	.24	.16	.12	.096	.08	.06	.05
W7.5		0.309	.255	.45	.30	.225	.15	.112	.09	.075	.056	.05
W7	D7	0.299	.238	.42	.28	.21	.14	.105	.084	.07	.053	.047
W6.5		0.288	.221	.39	.26	.195	.13	.097	.078	.065	.048	.043
W6	D6	0.276	.204	.36	.24	.18	.12	.09	.072	.06	.045	.04
W5.5		0.265	.187	.33	.22	.165	.11	.082	.066	.055	.041	.037
W5	D5	0.252	.170	.30	.20	.15	.10	.075	.06	.05	.038	.033
W4.5		0.239	.153	.27	.18	.135	.09	.067	.054	.045	.034	.03
W4	D4	0.226	.136	.24	.16	.12	.08	.06	.048	.04	.03	.027
W3.5		0.211	.119	.21	.14	.105	.07	.052	.042	.035	.026	.023
W3		0.195	.102	.18	.12	.09	.06	.045	.036	.03	.023	.02
W2.9		0.192	.098	.174	.116	.087	.058	.043	.035	.029	.022	.019
W2.5		0.178	.085	.15	.10	.075	.05	.037	.03	.025	.019	.017
W2.1		0.161	.070	.13	.084	.063	.042	.032	.025	.021	.016	.014
W2		0.160	.068	.12	.08	.06	.04	.03	.024	.02	.015	.013
W1.4		0.134	.049	.084	.056	.042	.028	.028	.017	.014	.011	.009

Note: For other available wire sizes other than those listed, contact your nearest WWR manufacturer.

ATTACHMENT F

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations **SETTLEMENT ANALYSIS OF BERMS**

Computations by:	Signature		11/30/07
	Printed Name	Meghan Lithgow	Date
	Title	Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		
	Printed Name	Steven M. Fitzwilliam	Date
	Title	Associate	
Computations Checked by:	Signature		
	Printed Name	Steven M. Fitzwilliam	Date
	Title	Associate	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Meghan Lithgow	Date
	Title	Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran, P.E.	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	M. Lithgow	Date:	07/12/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	Cell 4B	Project No.:	SC0349	Task No.:	03

SETTLEMENT EVALUATION OF BERMS

OBJECTIVE

The objective of this calculation is to evaluate the differential settlement under the loading from the berms at the perimeter of the cells to assess the potential effect on the system.

SUMMARY OF DESIGN

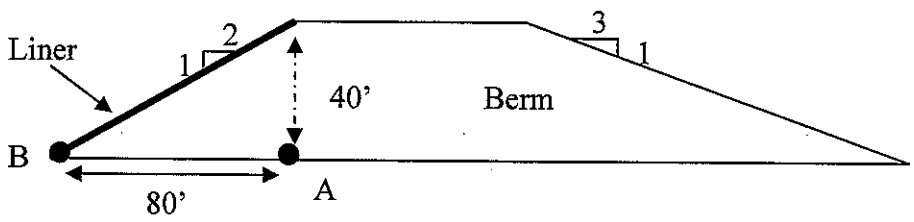
Based on the assumptions and calculations presented herein, the differential settlement of the berms will be approximately 0.16 inches at the toe of the slope and approximately 0.27 inches at the top of the slope, causing a strain of approximately 0.01% of the liner, when the cell is empty. When the cell is full, the differential settlement of the berms will be approximately 2.48 inches at the toe of the slope and approximately 2.51 inches at the top of the slope, causing a strain of approximately 0.003% of the liner.

ANALYSIS

Task 1: Evaluate the settlement of the berm in empty conditions.

Evaluate the settlement under the center of the berm versus at the toe of the berm.

Based on the proposed grading for cell 4B (Attachment A), the following cross-section can be evaluated for the highest berm (the south berm).



Using data from the native soil index property compaction tests performed for Cell 4B (Attachment B), $\gamma = 125\ pcf$

$$P, load = \gamma \times H = (125\,pcf)(40\,ft) = 5{,}000\ psf$$

Assuming the foundation for the berm consists of formational soil consisting of silty and clayey coarse to fine sandstone with layered shale (Attachment C), and based on the dense nature of the soil and underlying sandstone, the conservative stress-strain modulus is assumed to be as follows:

E_s = 1500 MPa (Boyles, 5th Ed., Attachment D)

$$E_S = 1500\ MPa \times \frac{145\frac{lb}{in^2}}{1\ MPa} \times \frac{144 in^2}{1 ft^2} \times \frac{1 kip}{1000 lb} = 31{,}320\ ksf$$

The settlement, S, is calculated in Table 1 using the elastic theory (Attachment E,1/2).

Elastic Theory: $\Delta = \frac{PL}{AE_E}$

Where:
- Δ = deformation
- P = force (F)
- L = length of material column (L)
- A = area of material column (L^2)
- E_E = Modulus of Elasticity (F/L^2)

For this analysis, the Boussinesq case for influence under a triangular load will be used, as presented in DM7.1-171. (Attachment E, 1/2). Translating the terms from Elastic Theory to apply to the Boussinesq approach to settlement calculation, the following equation is derived:

$$S = \frac{(4I \times P)(\Delta H)}{(A)(Es)} \times \left(\frac{12\ in}{1\ ft}\right) \text{ (Attachment E, 2/2)}$$

Where:
- S = Settlement (Δ in Elastic Theory)
- P = 5000 lb

A = 1 ft^2
ΔH = incremental depth, ft (L in Elastic Theory)
E_S = Soil Modulus, = E_E in Elastic Theory
I = Boussinesq zone of influence, obtained from Figure 7 on page DM7.1-171 using m and n parameters (Attachment F, 1/2).

To Find I:

n = B/z (Attachment F)
m = L/z (Attachment F)
B = 80 ft, distance from outer edge of berm
L = 1,000 ft, length along berm edge
z = depth, ft
σz = 4I*P, ksf

Table 1. Settlement due to load under the embankment at the top of the slope, using Figure 7, "Beneath Corner O" chart, (Point A):

Under top of berm - Point A								
z (ft)	n = B/z	m = L/z	I	4I	σz=4I*P (ksf)	E_s (ksf)	ΔH (ft)	S (in)
5	16.0	200.0	0.240	0.96	4.8	31,320	5	0.009
10	8.0	100.0	0.235	0.94	4.7	31,320	5	0.009
15	5.3	66.7	0.230	0.92	4.6	31,320	5	0.009
20	4.0	50.0	0.220	0.88	4.4	31,320	5	0.008
40	2.0	25.0	0.200	0.80	4.0	31,320	20	0.031
60	1.3	16.7	0.175	0.70	3.5	31,320	20	0.027
80	1.0	12.5	0.158	0.63	3.2	31,320	20	0.024
100	0.8	10.0	0.140	0.56	2.8	31,320	20	0.021
150	0.5	6.7	0.108	0.43	2.2	31,320	50	0.041
200	0.4	5.0	0.085	0.34	1.7	31,320	50	0.033
250	0.3	4.0	0.046	0.18	0.9	31,320	50	0.018
300	0.3	3.3	0.042	0.17	0.8	31,320	50	0.016
350	0.2	2.9	0.028	0.11	0.6	31,320	50	0.011
400	0.2	2.5	0.028	0.11	0.6	31,320	50	0.011
							Total S, in.	0.268

Settlement under A (S_A), the center of the berm, is approximately **0.27 inches**.

The calculation is repeated in Table 2 for settlement at the toe of the berm also using Figure 7, but the influence chart is used is "Beneath Corner Q" chart.

Table 2. Settlement due to load at toe of the slope (Point B):

Under top of berm - Point B								
z (ft)	n = B/z	m = L/z	I	4I	σz=4I*P (ksf)	E_s (ksf)	ΔH (ft)	S (in)
5	16.0	200.0	0.080	0.32	1.6	31,320	5	0.003
10	8.0	100.0	0.080	0.32	1.6	31,320	5	0.003
15	5.3	66.7	0.080	0.32	1.6	31,320	5	0.003
20	4.0	50.0	0.080	0.32	1.6	31,320	5	0.003
40	2.0	25.0	0.080	0.32	1.6	31,320	20	0.012
60	1.3	16.7	0.080	0.32	1.6	31,320	20	0.012
80	1.0	12.5	0.080	0.32	1.6	31,320	20	0.012
100	0.8	10.0	0.077	0.31	1.5	31,320	20	0.012
150	0.5	6.7	0.061	0.24	1.2	31,320	50	0.023
200	0.4	5.0	0.053	0.21	1.1	31,320	50	0.020
250	0.3	4.0	0.042	0.17	0.8	31,320	50	0.016
300	0.3	3.3	0.041	0.16	0.8	31,320	50	0.016
350	0.2	2.9	0.029	0.12	0.6	31,320	50	0.011
400	0.2	2.5	0.028	0.11	0.6	31,320	50	0.011
							Total S, in.	0.158

Settlement under point B (S_B), the toe of the berm, is approximately **0.16 inches**.

<u>Evaluate the elastic strain of the side slope liner system related to settlement.</u>

ΔS = 0.27-0.16 = 0.11 inches.

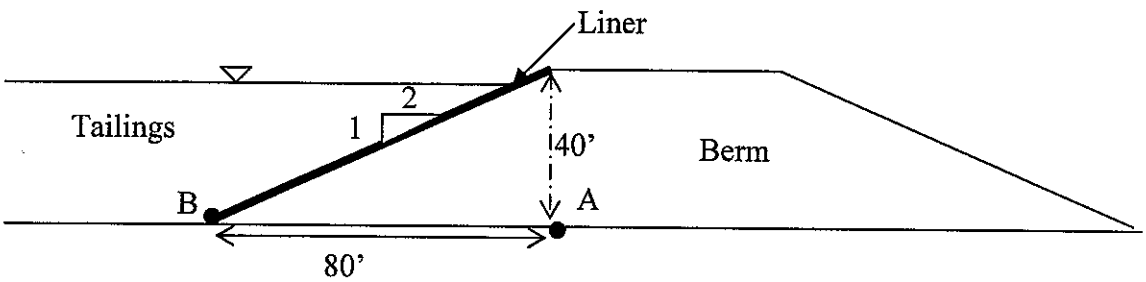
The liner is $\sqrt{[(80ft)^2 + (40ft)^2]} = 89.4\ ft$ in length.

Therefore, the strain in the liner is $\dfrac{0.11\ in}{89.4\ ft \times \dfrac{12in}{1ft}} = 0.01\%$

Written by:	M. Lithgow	Date:	07/12/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	Cell 4B	Project No.:	SC0349	Task No.:	03

Task 2: Evaluate the settlement under the berm upon filling of the Cell.

Upon filling the Cell, the foundation may show differential settlement between Point A and Point B on the liner.



Assume the following conditions:

Berm Material:	$\gamma = 125\ psf$
Tailing Material:	$\gamma = 125\ psf$
Foundation Soil:	$E_s = 31{,}320$ ksf
Length	L = 1,300 ft
Width	B = 1,000 ft

Load over Point A:

$P_A = 5{,}000\ psf$ at a distance of 80 ft

Load over Point B:

$P_B = (125\ psf)(40\ ft) = 5{,}000\ psf$

Calculations are performed using the equation defined in the previous section. Calculations are shown in Table 3 use influence values obtained from Figure 3 on page 7.1-167 of DM 7.1 (Attachment F, 2/2) for infinitely long footing.

Written by: M. Lithgow Date: 07/12/07 Reviewed by: G. Corcoran Date: 12/3/07

Client: DMC Project: Cell 4B Project No.: SC0349 Task No.: 03

Table 3. Settlement due to load under the embankment (filled condition)

Under toe of slope - Point A (80/1000 = 0.08B from center)						
z (ft)	z/B	I	σz=I*P (ksf)	E_s (ksf)	ΔH (ft)	S (in)
5	0.01	1.00	5.00	31,320	5	0.010
10	0.01	1.00	5.00	31,320	5	0.010
15	0.02	1.00	5.00	31,320	5	0.010
20	0.02	1.00	5.00	31,320	5	0.010
40	0.04	1.00	5.00	31,320	20	0.038
60	0.06	1.00	5.00	31,320	20	0.038
80	0.08	1.00	5.00	31,320	20	0.038
100	0.10	0.97	4.85	31,320	20	0.037
150	0.15	0.92	4.60	31,320	50	0.088
200	0.20	0.90	4.50	31,320	50	0.086
300	0.30	0.89	4.45	31,320	100	0.170
400	0.40	0.85	4.25	31,320	100	0.163
500	0.50	0.75	3.75	31,320	100	0.144
600	0.60	0.70	3.50	31,320	100	0.134
700	0.70	0.66	3.30	31,320	100	0.126
800	0.80	0.61	3.05	31,320	100	0.117
900	0.90	0.58	2.90	31,320	100	0.111
1000	1.00	0.53	2.65	31,320	100	0.102
1100	1.10	0.50	2.50	31,320	100	0.096
1200	1.20	0.49	2.45	31,320	100	0.094
1300	1.30	0.44	2.20	31,320	100	0.084
1400	1.40	0.40	2.00	31,320	100	0.077
1500	1.50	0.39	1.95	31,320	100	0.075
1600	1.60	0.37	1.85	31,320	100	0.071
1700	1.70	0.35	1.75	31,320	100	0.067
1800	1.80	0.34	1.70	31,320	100	0.065
1900	1.90	0.32	1.60	31,320	100	0.061
2000	2.00	0.30	1.50	31,320	100	0.057
2100	2.10	0.29	1.45	31,320	100	0.056
2200	2.20	0.28	1.40	31,320	100	0.054
2300	2.30	0.27	1.35	31,320	100	0.052
2400	2.40	0.26	1.30	31,320	100	0.050
2500	2.50	0.25	1.25	31,320	100	0.048
2600	2.60	0.24	1.20	31,320	100	0.046
					Total	2.484

Settlement under A (S_A), the center of the berm, is **2.48 inches**.

Written by: M. Lithgow Date: 07/12/07 Reviewed by: G. Corcoran Date: 12/3/07

Client: DMC Project: Cell 4B Project No.: SC0349 Task No.: 03

For point B, assume uniform loading (aerial fill) and B = 1000 feet for infinitely long footing, again using Figure 3 on page DM7.1-167 (Attachment D, 2/2). The calculation of settlement for this condition is shown in Table 4.

Table 4. Settlement due to load under the toe of the slope (filled condition)

Under top of berm - Point B						
z (ft)	z/B	I	σz=I*P (ksf)	E_s (ksf)	ΔH (ft)	S (in)
5	0.01	1.00	5.00	31,320	5	0.010
10	0.01	1.00	5.00	31,320	5	0.010
15	0.02	1.00	5.00	31,320	5	0.010
20	0.02	1.00	5.00	31,320	5	0.010
40	0.04	1.00	5.00	31,320	20	0.038
60	0.06	1.00	5.00	31,320	20	0.038
80	0.08	0.99	4.95	31,320	20	0.038
100	0.10	0.98	4.90	31,320	20	0.038
150	0.15	0.95	4.75	31,320	50	0.091
200	0.20	0.92	4.60	31,320	50	0.088
300	0.30	0.90	4.50	31,320	100	0.172
400	0.40	0.83	4.15	31,320	100	0.159
500	0.50	0.75	3.75	31,320	100	0.144
600	0.60	0.71	3.55	31,320	100	0.136
700	0.70	0.69	3.45	31,320	100	0.132
800	0.80	0.61	3.05	31,320	100	0.117
900	0.90	0.60	3.00	31,320	100	0.115
1000	1.00	0.55	2.75	31,320	100	0.105
1100	1.10	0.52	2.60	31,320	100	0.100
1200	1.20	0.49	2.45	31,320	100	0.094
1300	1.30	0.46	2.30	31,320	100	0.088
1400	1.40	0.41	2.05	31,320	100	0.079
1500	1.50	0.39	1.95	31,320	100	0.075
1600	1.60	0.37	1.85	31,320	100	0.071
1700	1.70	0.35	1.75	31,320	100	0.067
1800	1.80	0.34	1.70	31,320	100	0.065
1900	1.90	0.32	1.60	31,320	100	0.061
2000	2.00	0.31	1.55	31,320	100	0.059
2100	2.10	0.29	1.45	31,320	100	0.056
2200	2.20	0.28	1.40	31,320	100	0.054
2300	2.30	0.27	1.35	31,320	100	0.052
2400	2.40	0.26	1.30	31,320	100	0.050
2500	2.50	0.25	1.25	31,320	100	0.048
2600	2.60	0.24	1.20	31,320	100	0.046
					Total	2.513

Settlement under point B (S_B), the toe of the berm, is **2.51 inches**.

Evaluate for strain

$\Delta S = 2.51 - 2.48 = 0.03$ inches.

The liner is $\sqrt{[(80\,ft)^2 + (40\,ft)^2]} = 89.4\ ft$ in length.

Therefore, the strain in the liner is $\dfrac{0.03\ in}{89.4\ ft \times \dfrac{12in}{1ft}} = 0.003\%$

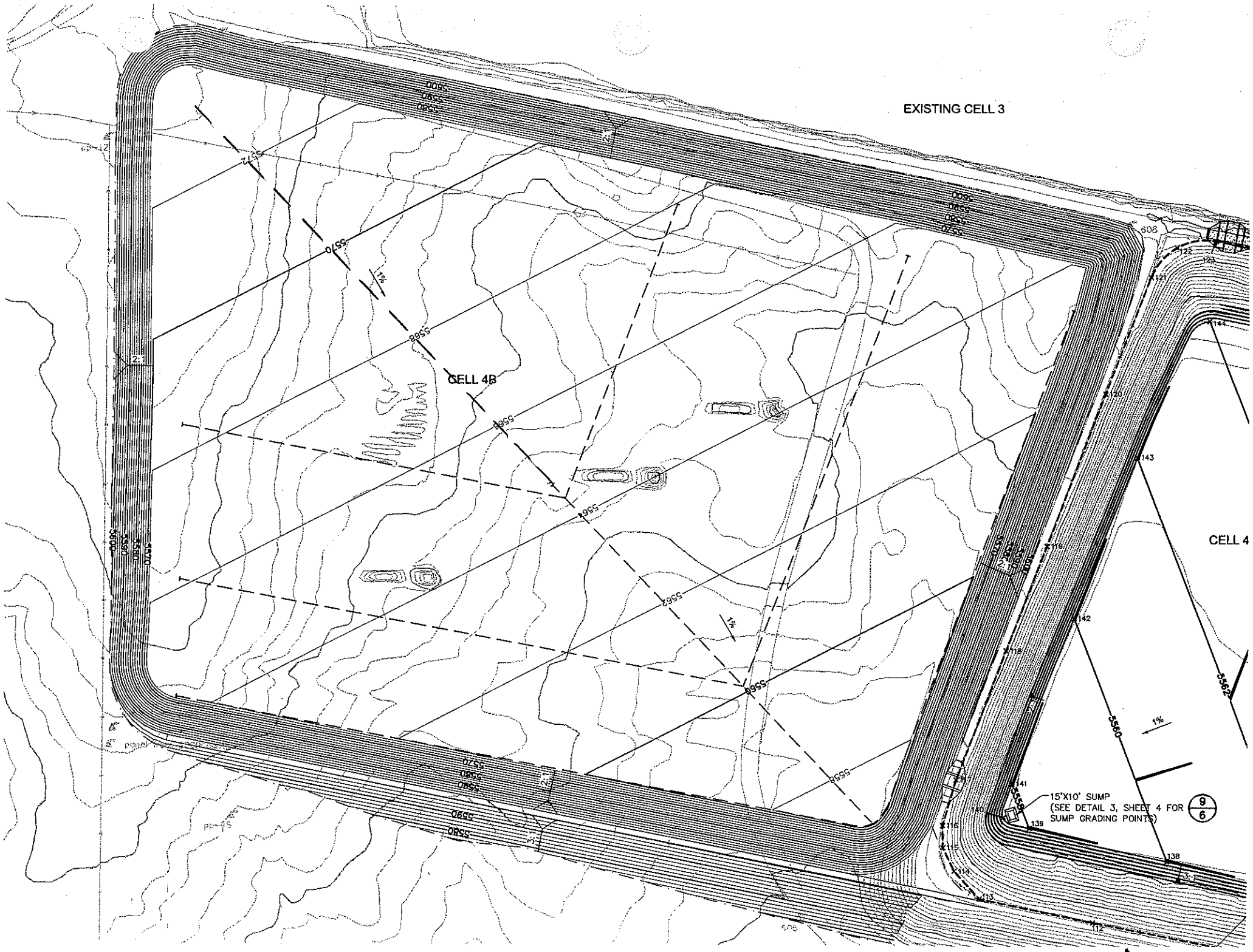
REFERENCES

Bowles, Joseph E., *Foundation Analysis and Design, 5th Edition,* Mc-Graw-Hill, 1996.

Holtz, Robert D. and Kovacs, William D., *An Introduction to Geotechnical Engineering,* Prentice- Hall International, 1981.

Navy Design Manual DM-7.1

PEEL Environmental Services, *WMMW-16 Boring and Well Construction Log,* December, 1992.


EXISTING CELL 3
CELL 4B
CELL 4
15'X10' SUMP
(SEE DETAIL 3, SHEET 4 FOR 9/6
SUMP GRADING POINTS)
1%
5570
5568
5566
5564
5562
5560
5580
5590
5600
5570
5572
5574
5576
5578
5580
5582
5584
5586
5588
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Geosyntec
White Mesa Mall - Cell 4B, Job No. SC0349-03

May 23, 2007
GF12204

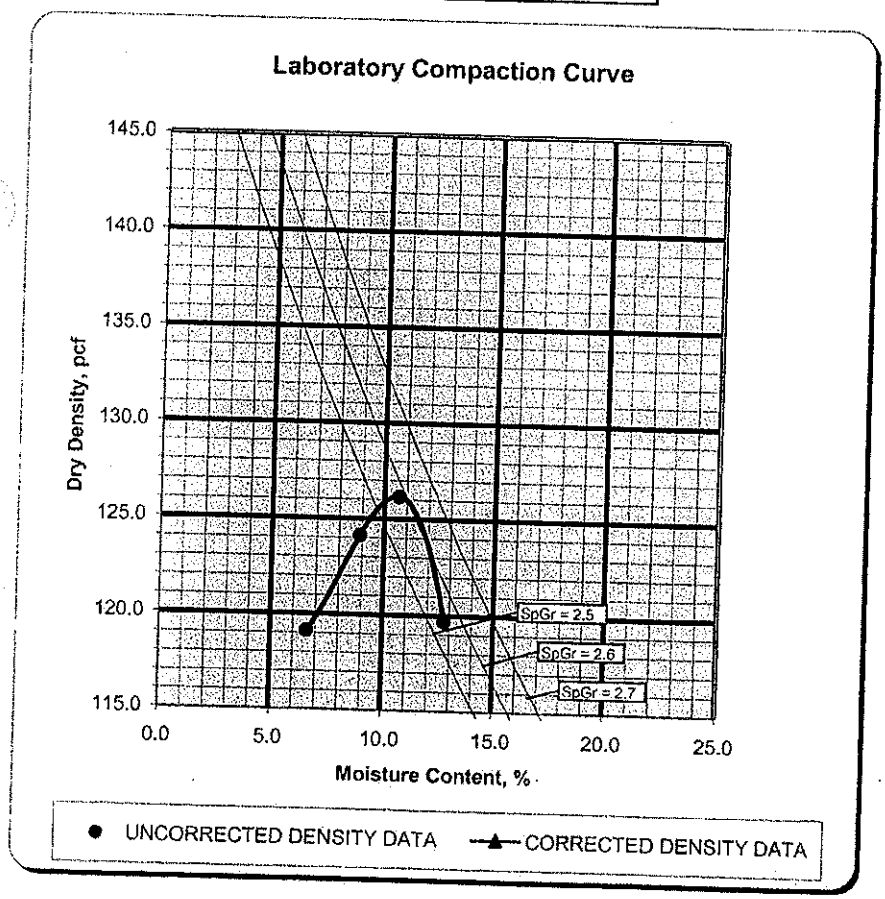
LABORATORY COMPACTION CURVE

G Force Lab No.: **5224**
Sample Location: **B-2**
Soil Description: **Light brown sandy lean clay**
Source of Soil: **Not submitted**

Depth, ft.: **0 - 6'**

Test Designation: **ASTM D1557** Method **A**
% +3/4" **0** % +3/8" **0** % +#4 **0**
Oversize Correction Applied? **No**
Method of Sample Preparation: **Wet**
Type of Rammer Used: **Manual**

M/D Curve No. 1



Test Results

Maximum Density, pcf	**126.0**
Optimum Moisture, %	**10.5**

Oversize Corrected Results

Maximum Density, pcf	
Optimum Moisture, %	

Checked by: John Inlow, Lab Manager

Reviewed by: Scott L. Larson 5/23/07
Scott Larson, RCE, 42432

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8788 Balboa Avenue ♦ San Diego, CA 92123 ♦ 619-583-6633 ♦ Fax 619-583-6654
3536 Concours Avenue, Suite 110 ♦ Ontario, CA 91764 ♦ 909-481-6833 ♦ Fax 909-481-4642
www.gforceca.com

Attachment B(1/2)

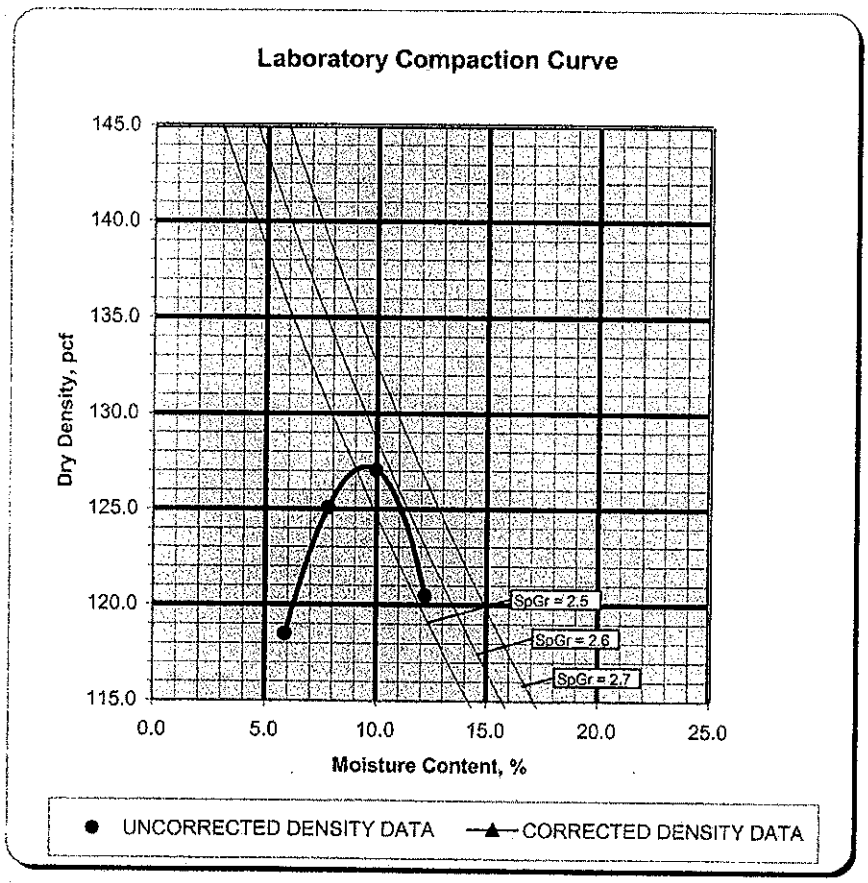
Geosyntec **May 23, 2007**
White Mesa Mall - Cell 4B, Job No. SC0349-03 **GF12204**

LABORATORY COMPACTION CURVE

G Force Lab No.: **5225**
Sample Location: **B-5** Depth, ft.: **0 - 5'**
Soil Description: **Light brown sandy lean clay**
Source of Soil: **Not submitted**

Test Designation: **ASTM D1557** Method **A**
% +3/4" **0** % +3/8" **0** % +#4 **0**
Oversize Correction Applied? **No**
Method of Sample Preparation: **Wet**
Type of Rammer Used: **Manual**

M/D Curve No. 2



Test Results

Maximum Density, pcf	**127.0**
Optimum Moisture, %	**9.5**

Oversize Corrected Results

Maximum Density, pcf	
Optimum Moisture, %	

Checked by: John Inlow, Lab Manager

Reviewed by: Scott L. Larson 5/23/07
Scott Larson, RCE, 42432

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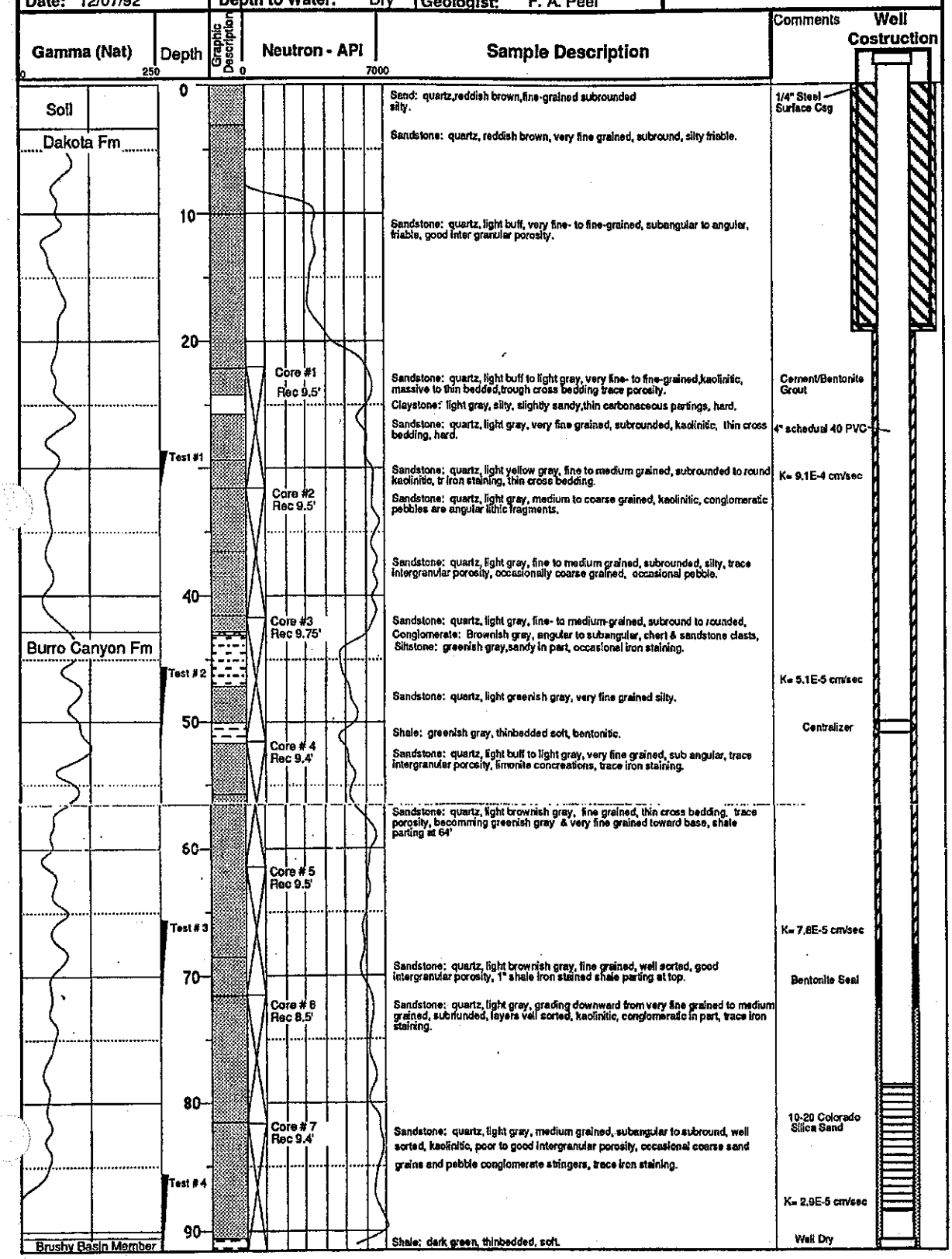
Attachment B (2/2

PEEL Environmental Services
Arvada, CO (303)422-5116

Bore Hole No. : WMMW-16

UMETCO Minerals Corporation

Project: White Mesa	Surface Elev. 5588.18	T. D. = 91.5	
Date: 12/07/92	Depth to Water: Dry	Geologist: F. A. Peel	



Attachment C (1/1)

TABLE 2-8
Value range* for the static stress-strain modulus E_s for selected soils (see also Table 5-6)

Field values depend on stress history, water content, density, and age of deposit

Soil	E_s, MPa
Clay	
Very soft	2–15
Soft	5–25
Medium	15–50
Hard	50–100
Sandy	25–250
Glacial till	
Loose	10–150
Dense	150–720
Very dense	500–1440
Loess	15–60
Sand	
Silty	5–20
Loose	10–25
Dense	50–81
Sand and gravel	
Loose	50–150
Dense	100–200
Shale	150–5000 ← use 1500 MPa
Silt	2–20

*Value range is too large to use an "average" value for design.

in situ, it is reasonable for confined compression tests to produce better "elastic" parameters. Although it is difficult to compare laboratory and field E_s values, there is some evidence that field values are often four to five times larger than laboratory values from the unconfined compression test. For this reason, current practice tends to try to obtain "field" values from in situ testing whenever possible. This topic will be taken up in more detail in the next chapter.

Table 2-8 gives a range of E_s values that might be obtained. Note that the range is very large, owing to the foregoing factors as well as those factors given on the table. With this wide range of values the reader should not try to use "averaged" values from this table for design.

If laboratory test plots similar to Fig. 2-43*a* are used, it is most common to use the initial tangent modulus to compute the stress-strain modulus E_s for the following reasons:

1. Soil is elastic only near the origin.
2. There is less divergence between all plots in this region.
3. The largest values are obtained—often three to five times larger than a tangent or secant modulus from another point along the curve.

Bowles, Joseph E. Foundation Analysis and Design, 5th Ed., 1996.
Attachment D (1/1)

	Definition
	ı index (Eq. 8-8)
	(Eq. 8-15); C_E and C_s are sometimes used
	on index
	ıetric modulus (Eq. 8-6)
	ratio
	ı soil layer (Eq. 8-3)
	f a soil layer (Eq. 8-3)
	8-30)
	(Eq. 8-23)
	(Eq. 8-20)
	width to depth (Eqs. 8-28 and 8-29)
	change (Eq. 8-6)
	8-25)
	8-34)
	io (Eq. 8-2)
	4)
	ss (Eq. 8-27)
	om load to a point (Eq.8-24)
	ent (Eq. 8-1)
	ın settlement (Eq. 8-1)
	n (Eq. 8-1)
	8-1)
	ıore water pressure
	8-
	1)
	33)
	ss (Eq. 8-22)
	olidation stress
	ıtress or maximum past
	(Eq. 8-2); p'_c and σ'_{om}
	rburden stress (Eq. 8-2)
	ı z (Eq. 8-22)

MENT

ample by a structure or a
otal vertical deformation at
tlement. The movement may
ard (called *swelling*) with a

decrease in load. Temporary construction excavations and permanent excavations such as highway cuts will cause a reduction in the stress, and swelling may result. As shown in Chapter 7, a lowering of the water table will also cause an increase in the effective stresses within the soil, which will lead to settlements. Another important aspect about settlements of especially fine-grained soils is that they are often time-dependent.

In the design of foundations for engineering structures, we are interested in how much settlement will occur and how fast it will occur. Excessive settlement may cause structural as well as other damage, especially if such settlement occurs rapidly. The total settlement, s_t, of a loaded soil has three components, or

$$s_t = s_i + s_c + s_s \tag{8-1}$$

where s_i = the *immediate*, or *distortion*, settlement,
s_c = the *consolidation* (time-dependent) settlement, and
s_s = the *secondary* compression (also time-dependent).

The immediate, or distortion, settlement although not actually elastic is usually estimated by using elastic theory. The equations for this component of settlement are in principle similar to the deformation of a column under an axial load P, where the deformation is equal to PL/AE. In most foundations, however, the loading is usually three dimensional, which causes some distortion of the foundation soils. Problems arise concerning the proper evaluation of a compression modulus and the volume of soil that is stressed. Immediate settlements must be considered in the design of shallow foundations, and procedures for dealing with this problem can be found in textbooks on foundation engineering.

The consolidation settlement is a time-dependent process that occurs in saturated fine-grained soils which have a low coefficient of permeability. The rate of settlement depends on the rate of pore water drainage. Secondary compression, which is also time-dependent, occurs at constant effective stress and with no subsequent changes in pore water pressure. Settlement computations are discussed in this chapter; the time rate of consolidation and secondary compression are discussed in Chapter 9.

8.3 COMPRESSIBILITY OF SOILS

Assume for the time being that the deformations of our compressible soil layer will occur in only one dimension. An example of one-dimensional compression would be the deformation caused by a fill covering a very large area. Later on we shall discuss what happens when a structure of finite size loads the soil and produces deformation.

Holtz, Robert D. and Kovacs, William M., An Introduction to Geotechnical Engineering, 1981.

Attachment E (1/2)

b. Find the vertical stress under the center of the footing at a depth of 2 m.

c. Compare results with Fig. Ex. 8.17a.

Solution:

a. $x = 3$ m

$y = 4$ m

$z = 2$ m: therefore from Eqs. 8-28 and 8-29,

$$m = \frac{x}{z} = \frac{3}{2} = 1.5$$

$$n = \frac{y}{z} = \frac{4}{2} = 2$$

From Fig. 8.21, find $I = 0.223$. From Eq. 8-30,

$$\sigma_z = q_o I$$
$$= 117 \times 0.223$$
$$= 26 \text{ kPa}$$

b. To compute the stress under the center, it is necessary to divide the 3×4 m rectangular footing into four sections of 1.5×2 m in size. Find the stress under one corner and multiply this value by 4 to take into account the four quadrants of the uniformly loaded area. We can do this because, for an elastic material, superposition is valid.

$x = 1.5$ m

$y = 2$ m

$z = 2$ m; then

$$m = \frac{x}{z} = \frac{1.5}{2} = 0.75$$

$$n = \frac{y}{z} = \frac{2}{2} = 1$$

The corresponding value of I from Fig. 8.21 is 0.159. From Eq. 8-30,

$$\sigma_z = 4q_o I = 4 \times 117 \times 0.159 = 74 \text{ kPa}$$

Thus the vertical stress under the center for this case is about three times that under the corner. This seems reasonable since the center is loaded from all sides but under the corner it is not.

c. At a depth of 2 m below the 3×4 m footing, the vertical stress according to the 2:1 theory is 47 kPa (see Fig. Ex. 8.17b). This value represents the *average* stress beneath the footing at -2 m. The average of the corner and center stress by elastic theory is $(26 + 74.2)/2 = 50.1$ kPa.

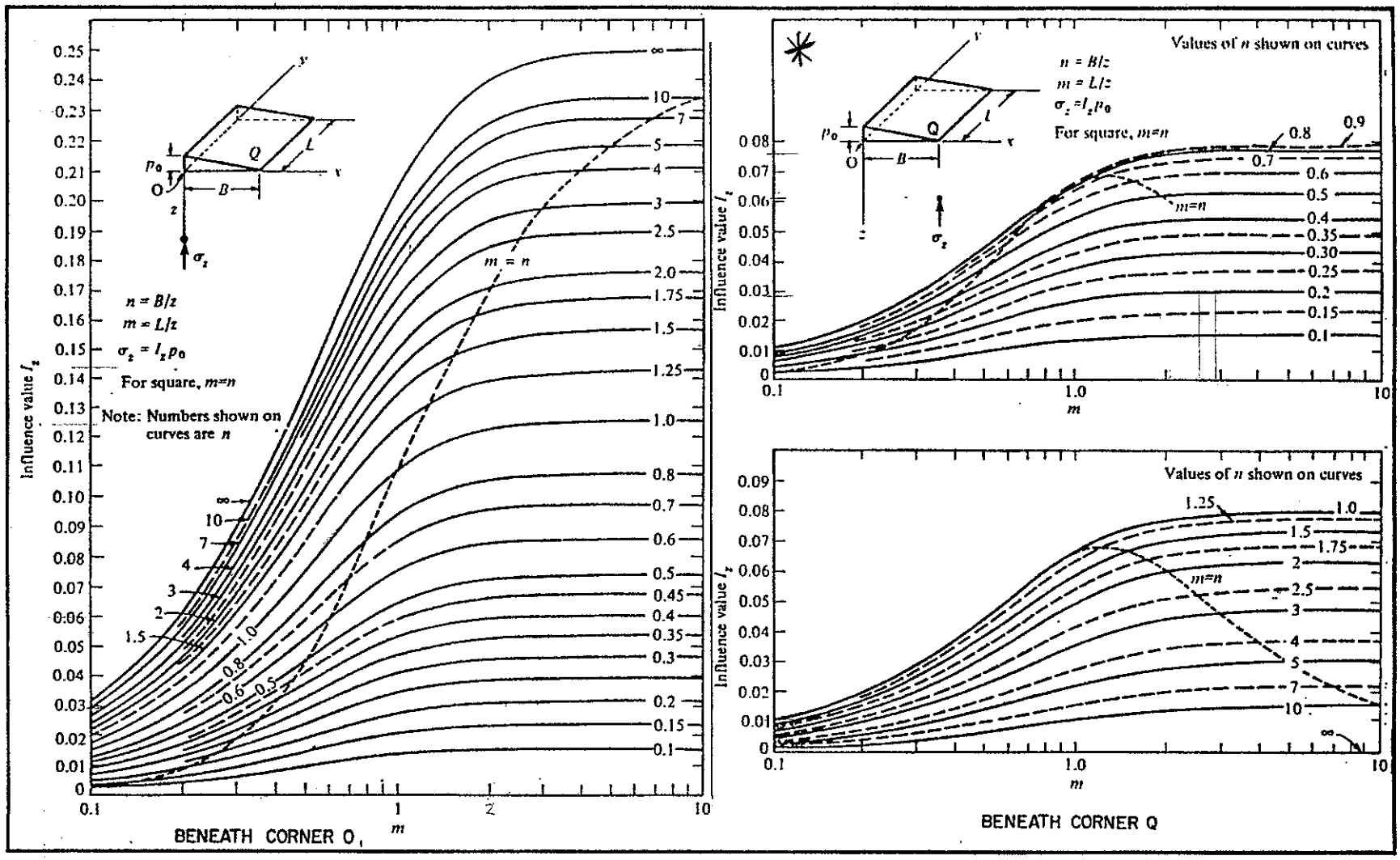
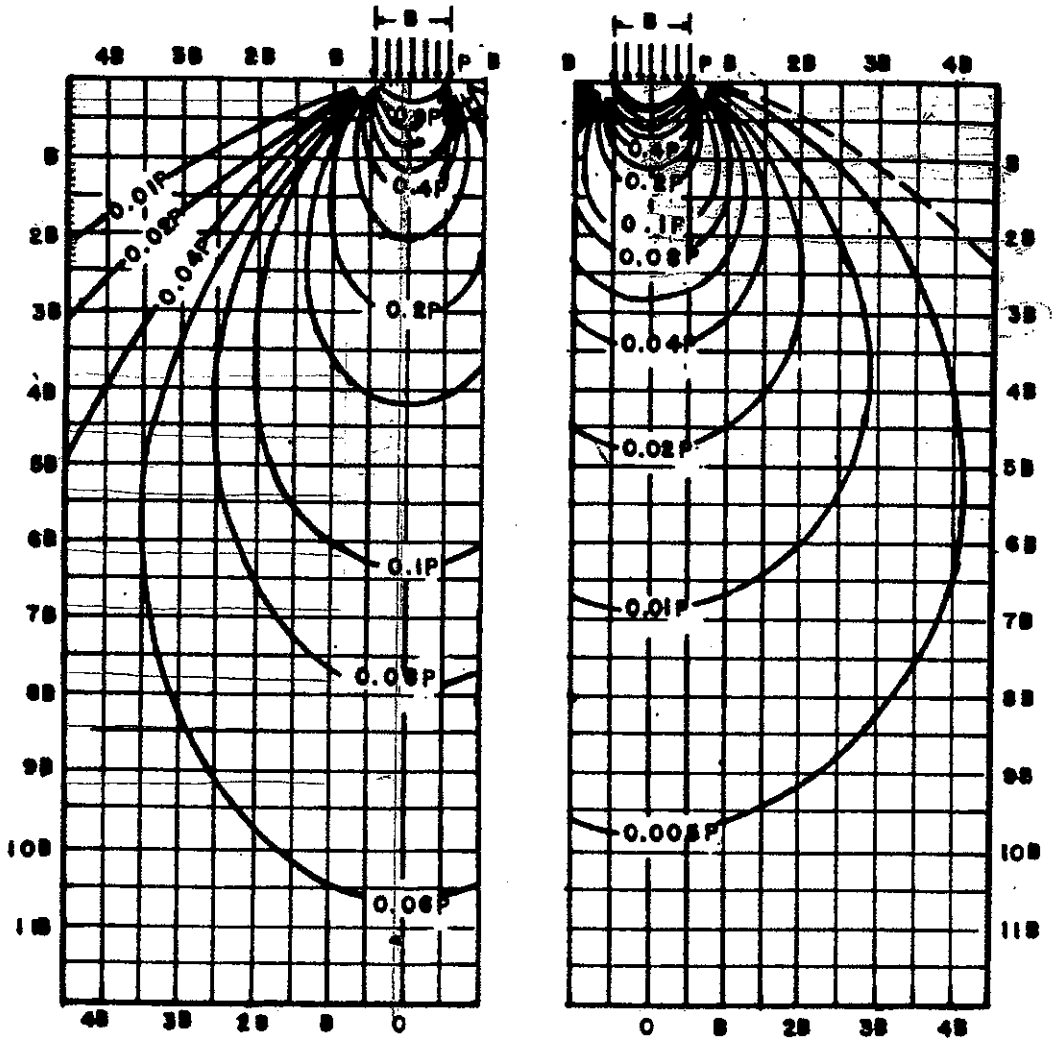


FIGURE 7
Influence Value for Vertical Stress Beneath Triangular Load
(Boussinesq Case)

ATTACHMENT B 1/2



a. INFINITELY LONG FOOTING

b. SQUARE FOOTING

SQUARE FOOTING

GIVEN

FOOTING SIZE = 20'X 20'

UNIT PRESSURE P=2TSF

FIND

PROFILE OF STRESS INCREASE BENEATH CENTER OF FOOTING DUE TO APPLIED LOAD

B = 20' P = 2 TSF

Z (FT)	Z/B	σ_z TSF
10	0.5	0.70 X 2 = 1.4
20	1	0.38 X 2 = 0.76
30	1.5	0.19 X 2 = 0.38
40	2.0	0.12 X 2 = 0.24
50	2.5	0.07 X 2 = 0.14
60	3.0	0.05 X 2 = 0.10

2/B 0.5

FIGURE 3
Stress Contours and Their Application

ATTACHMENT E 2/2

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations: ACTION LEAKAGE RATE

Computations by:	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		12/3/07
	Printed Name	Gregory T. Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran, P.E.	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	07/23/07	Reviewed by:	G. Corcoran	Date:	12/3/07
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:

$$Q = (2/3)(0.0015m)^2\sqrt{(9.81)(11.3m)} \quad = 1.58 \times 10^{-5}\ m^3/sec$$
$$= 1.36\ m^3/day$$
$$= 359.27\ gal/day$$

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, m^3/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:	07/23/07	Reviewed by:	G. Corcoran	Date:	12/3/07
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 (730 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:

$$i = (31\ \text{ft} + 730\ \text{ft} \times 0.01) / 730\ \text{ft} = 0.053$$

Placing the estimated hydraulic gradient of 0.053 into Equation 3 results in a hydraulic conductivity of 0.83 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:

$$\mathbf{k} = (0.83\ \text{m/sec}) / (1.0 \times 1.4 \times 2.0 \times 1.0) = \mathbf{2.96 \times 10^{-1}\ m/sec}$$

Placing the geonet hydraulic conductivity and thickness into Equation 2 results in the following:

$\mathbf{Q_{full}}$ = (0.296 m/sec) x $(0.0076\ m)^2$ = **1.71 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 = (1.71 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

$$= \frac{(1.36 m^3/day)(73)}{45 \text{ acres}} = 2.2 \ m^3/day/acre$$

= 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; *730 ft, 223 m*
- k = hydraulic conductivity of the geonet; *0.296 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) (223 m) / (0.296 m/sec) (sin 0.573) (cos 0.573) = 60,270 sec = 16.7 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} m^3/sec*

k = hydraulic conductivity of geonet layer above secondary geomembrane; *2.96×10^{-1} m/sec from above*

i = hydraulic gradient in leakage collection layer; *0.053 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:

$$t_{avgworst} = \frac{(73)(1.58x10^{-5})}{(2.96x10^{-1}\,\mathrm{m/sec})(0.053)(506\mathrm{m})} \qquad t_{avgworst} = 0.00015\ \mathrm{m} = 0.15\ \mathrm{mm}$$

The head on the secondary does not exceed 0.15 mm (0.006 inches), much less than the required 12 inch (1 foot) maximum.

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.

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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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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:

$$t_{LCL} \geq t_{LCL,full} = \sqrt{\frac{Q}{k}} \qquad (11)$$

where $t_{LCL,full}$ 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:

$$Q \leq Q_{full} = k\, t_{LCL}^2 \qquad (12)$$

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, $t_{LCL,full}$, 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 $t_{LCL,full}$ 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 Leachate Flow When the Leachate Collection Layer is Fu

If the thickness of the leakage collection layer is less than $t_{LCL,full}$ 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 perpenc 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:

$$Q = k\, t_{LCL}(2t_o - t_{LCL})$$

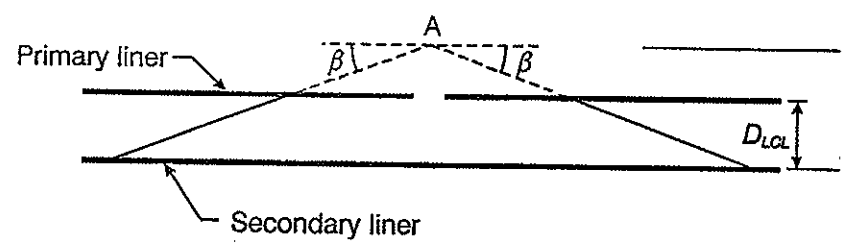


Figure 5. Vertical cross section of the assumed phreatic surface in the leakage co layer in the case where the leakage collection layer is filled with leachate in a certa around the primary liner defect.

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:

$$t_o = \frac{t_{LCL}}{2}\left(1 + \frac{Q}{k\, t_{LCL}^2}\right) \tag{17}$$

ollowing equation can be derived from Equations 13 and 16:

$$t_{LCL} = t_o\left(1 - \sqrt{1 - \frac{Q}{k\, t_o^2}}\right) \tag{18}$$

ion 18 is valid only if the following condition is met:

$$Q \le k\, t_o^2 \tag{19}$$

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

g 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 ı 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 ıchate 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) ıchate, and for these two materials the leachate thicknesses, t_o, shown in Table ctual (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)	(m^3/s)	(lpd)	(m^3/s)	(lpd)	(m^3/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 m^3/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:

$$Q = 0.6\, a \sqrt{2\, g\, h_{prim}} = 0.6\pi (d^2/4) \sqrt{2\, g\, h_{prim}} \approx (2/3)\, d^2 \sqrt{g\, h_{prim}} \tag{20}$$

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 m^3/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 determined in Sections 4.4.3 and 4.4.4, and the average head will be determined in Sections 1 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 consided: 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 overing 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),

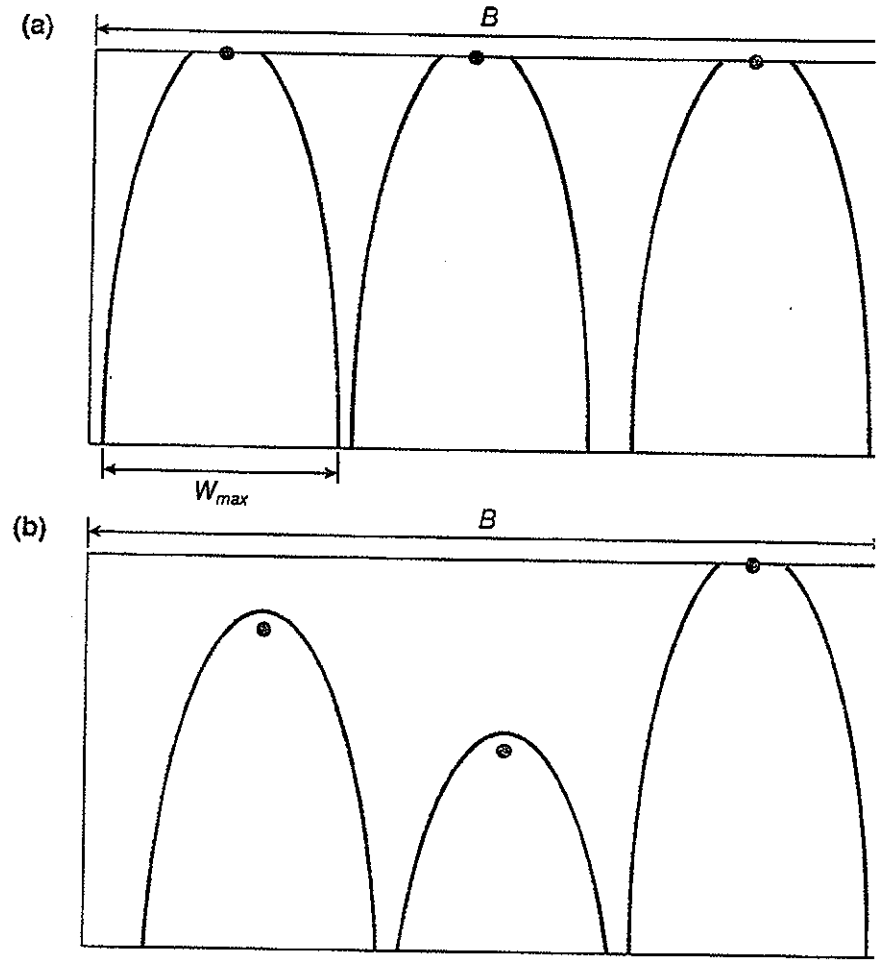


Figure 9. Leakage collection layer zones wetted by leachate migrating through se defects in the primary liner, assuming no overlapping of wetted zones: (a) scenario where all the defects are located at the high end of the leachate collection slope; (b) random scenario where the defects are randomly distributed.

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 scen 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 9b). In both scenarios it is assumed that the frequency of defects

$$\frac{t_{avg\,rand}\,\lambda_{rand}}{t_{avg\,worst}\,\lambda_{worst}} = \frac{2\,\mu}{9} + \frac{x_{rand}}{L} = \frac{\mu^{5/3}}{\left(10\sqrt{2}\right)^{2/3}}\left[\left(1+\frac{2}{\mu}\right)^{5/2} - 2\right]^{2/3} - \frac{5\mu}{18} \qquad (192)$$

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- l, 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 ations 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- 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- e 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 n 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 ives 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- ns 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 sing the leachate thickness determined as indicated in Sections 5.2.2 ltiplied 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.

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:

$$\bar{t}_{req} > \frac{nx}{k\sin\beta\cos\beta} + \frac{2nQ^{1/2}}{9\sin^2\beta\cos\beta\,k^{3/2}} \tag{203}$$

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:

$$\bar{t}_{req} > \frac{nx}{k\sin\beta\cos\beta} \tag{204}$$

Equation 204 may be written as follows:

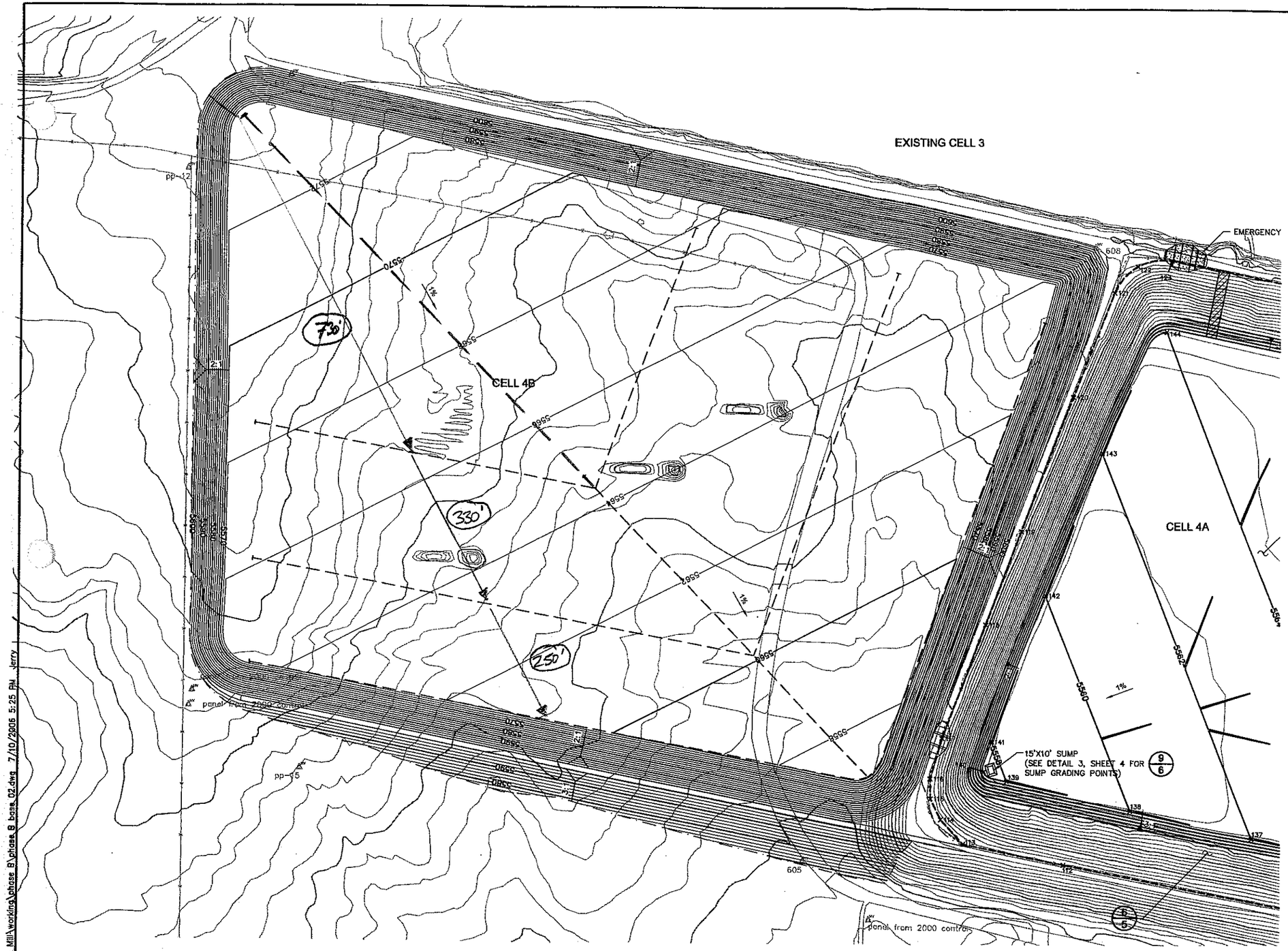
$$\bar{t}_{req} > \frac{x/\cos\beta}{k\sin\beta/n} \tag{205}$$

Combining Equations 8 and 205 gives:

$$\bar{t}_{req} > \frac{x/\cos\beta}{ki/n} \tag{206}$$

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:

$$\bar{t}_{req} > \bar{t}_{travel} = \frac{nx}{k\sin\beta\cos\beta} \tag{207}$$



LEGEND

5560	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (2' CONTOUR)
1%	SLOPE DIRECTION AND GRADE

N

100 50 0 100 200
SCALE IN FEET

ATTACHMENT B

GEOSYNTEC CONSULTANTS
3980 OLD TOWN AVENUE, SUITE B-101
SAN DIEGO, CALIFORNIA 92110
TELEPHONE: (619) 297-1530

PROJECT: WHITE MESA MILL

TITLE: PHASE 4B GRADING

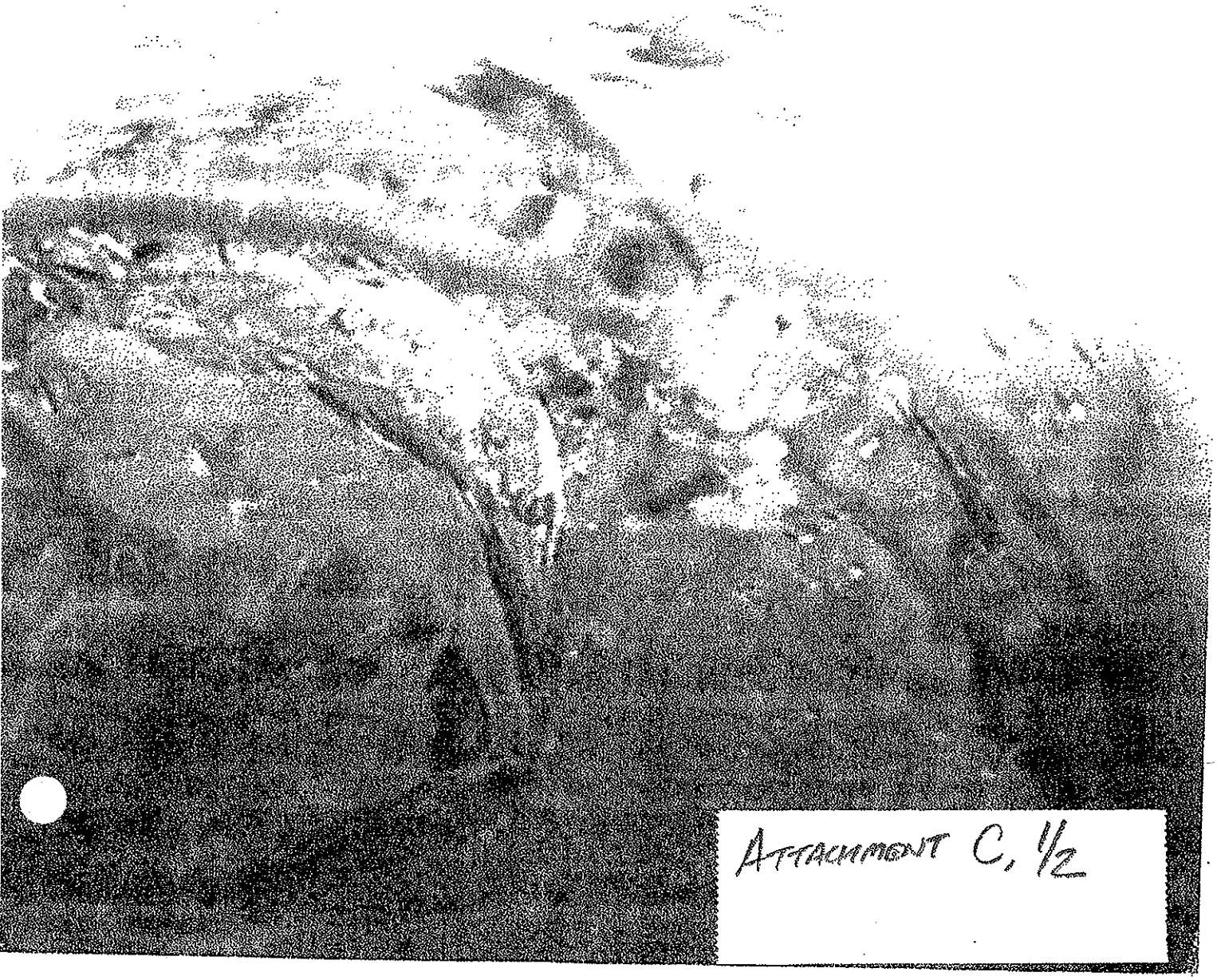
DATE: OCT 2006	CHECKED BY:	SCALE: 1"= 100'	FIGURE NO:
DESIGN BY:	REVIEWED BY: (PROJ. MGR.)	JOB NO.: SC0349	

P:\CAD\SC0349 - White Mesa Mill\working\phase B\phase 8 base 02.dwg 7/10/2006 5:25 PM Jerry Amos

GSE®

The GSE Drainage Design Manual

Robert Bachus • Dhani Narejo • Richard Thiel • Te-Yang Soong



ATTACHMENT C, 1/2

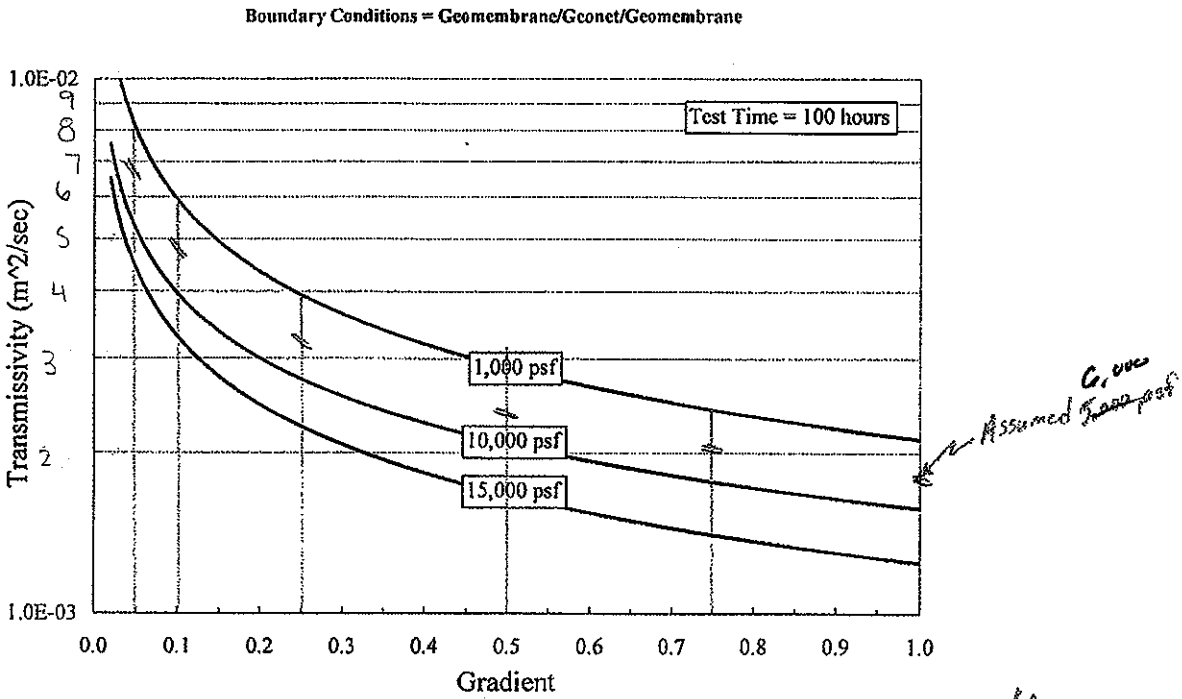


Figure A-7 100-hour transmissivity data for a 300 mil biplanar geonet between two geomembranes.

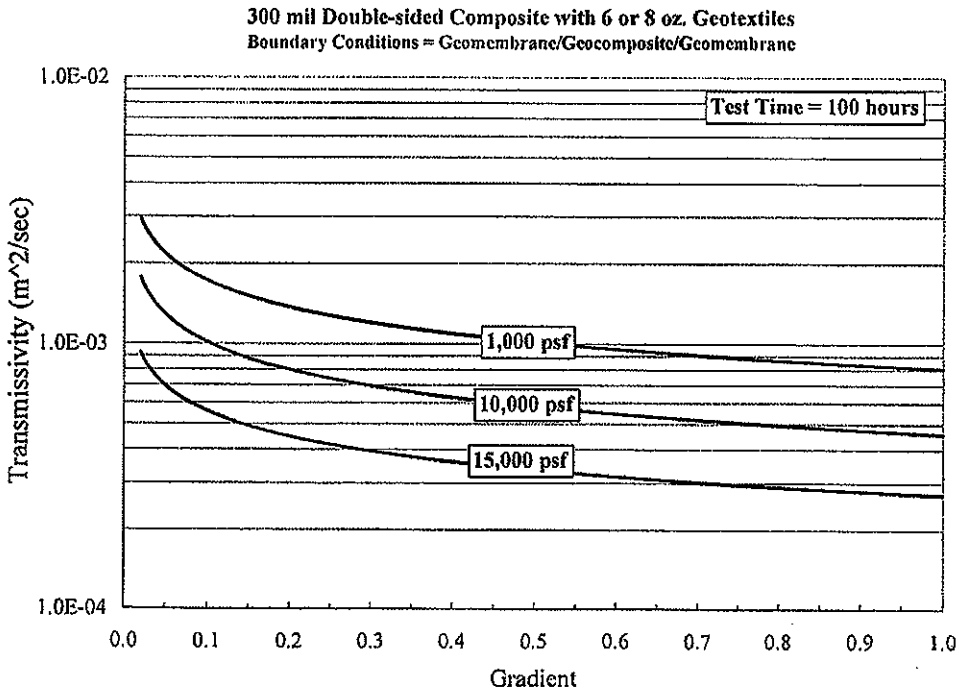
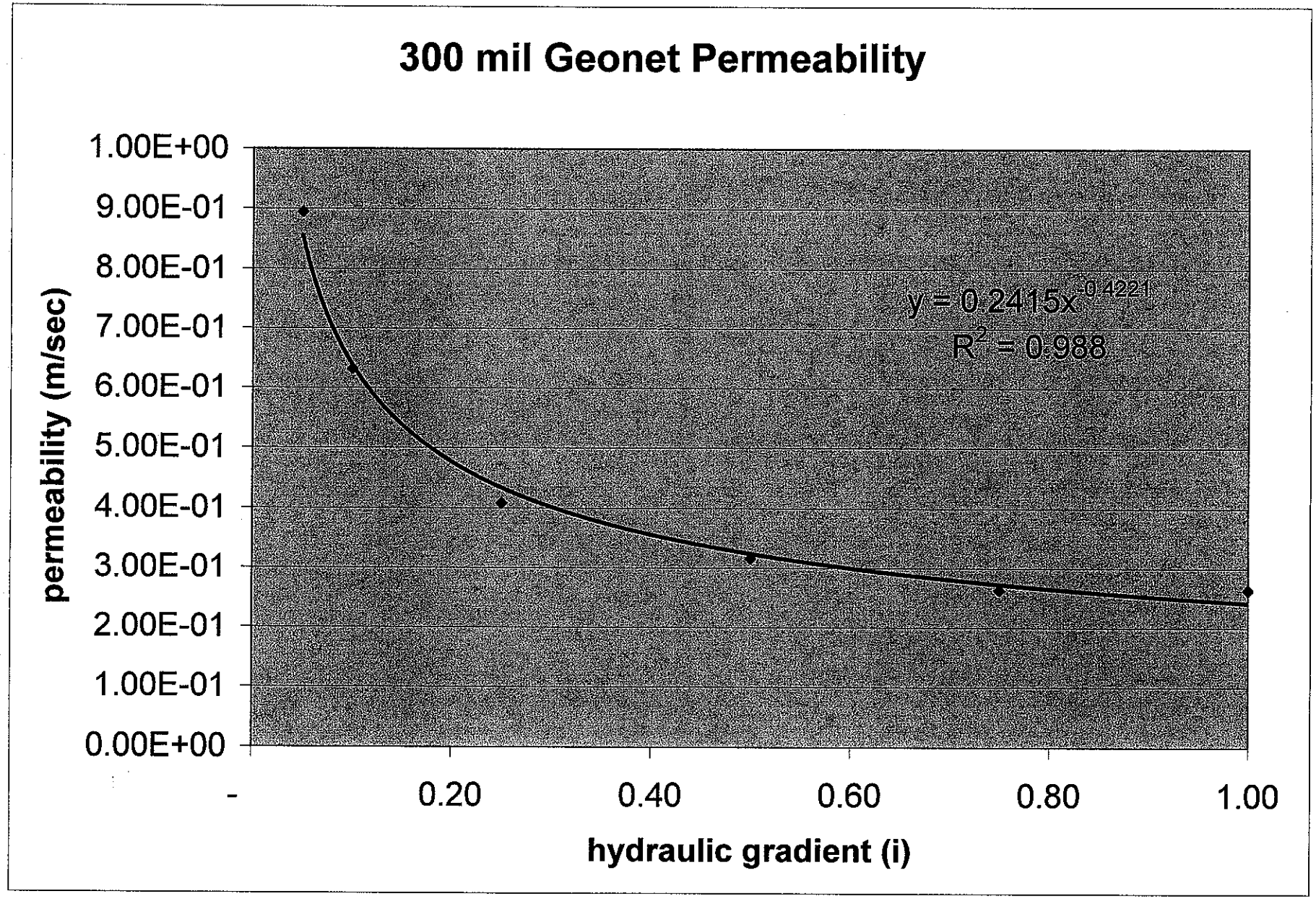


Figure A-8 100-hour transmissivity data for a 300 mil biplanar geonet geocomposite between two geomembranes.

Attachment C 2/2

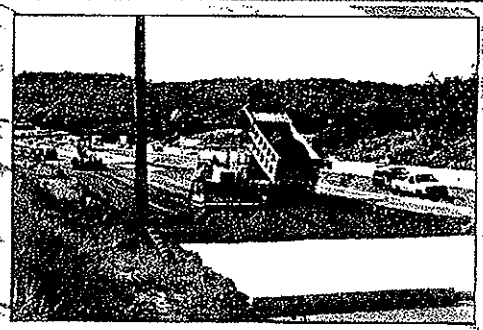


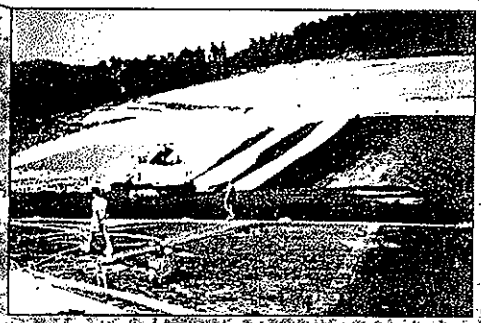
leak detection system geonet20070717.xls

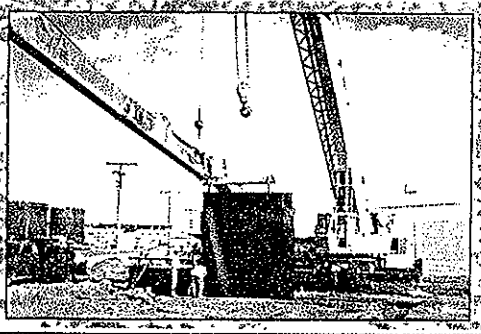
ATTACHMENT D(1/1)

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.

$$\mathrm{FS} = \frac{q_{\mathrm{allow}}}{q_{\mathrm{reqd}}} \tag{4.3}$$

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.

$$\mathrm{FS} = \frac{\theta_{\mathrm{allow}}}{\theta_{\mathrm{reqd}}} \tag{4.4}$$

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,

$$q_{\mathrm{allow}} < q_{\mathrm{ult}}$$

One way of doing this is to ascribe reduction factors on each of the items not adequately assessed in the laboratory test. For example,

$$q_{\mathrm{allow}} = q_{\mathrm{ult}}\left[\frac{1}{\mathrm{RF}_{IN} \times \mathrm{RF}_{CR} \times \mathrm{RF}_{CC} \times \mathrm{RF}_{BC}}\right] \tag{4.5}$$

or if all of the reduction factors are considered together.

$$q_{\mathrm{allow}} = q_{\mathrm{ult}}\left[\frac{1}{\Pi\mathrm{RF}}\right] \tag{4.6}$$

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,

Sec. 4.1

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ATTACHMENT E, 2/3

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(4.6)

S 12958 for short-
ansported 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, 3/3

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations **GEOMEMBRANE TENSION DUE TO WIND UPLIFT**

Computations by:	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	07/06/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

GEOMEMBRANE TENSION DUE TO WIND UPLIFT
WHITE MESA MILL
BLANDING, UTAH

OBJECTIVE

The project includes the installation of a double composite liner system within Cell 4B at the White Mesa Mill in Blanding, Utah. The proposed liner system is shown in Attachment A. The objective of this calculation is to evaluate tension in the primary geomembrane on the exposed side slopes due to wind uplift. The method outlined by Giroud, et al (1995) will be employed herein. Tension generated by wind uplift will be used to design the anchor trench capacity (see companion calculation package titled, *Anchor Trench Capacity Calculations*)

SITE CONDITIONS

The side slope liner system considered in the wind uplift calculation consists (from top to bottom) of:

- 60-mil (1.5 mm) HDPE geomembrane;
- Geonet (leak detection system);
- 60-mil HDPE geomembrane;
- Geosynthetic clay liner (GCL); and
- Prepared subgrade.

The capacity of the anchor trench is determined in a separate calculation package.

ANALYSIS

The analysis will follow the method outlined by Giroud, et al., in "Uplift of Geomembrane by Wind" (Attachment B). Giroud et al. offer the following equation for estimating the effective suction on a geomembrane (Attachment B):

Written by:	R. Flynn	Date:	07/06/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

$$S_e = 0.050\lambda V^2 e^{-[1.252\times10^{-4}]z} - 9.81\mu_{GM}$$ (Attachment B, 1/6)

where: S_e = effective suction (Pa)

λ = suction factor (dimensionless)

V = wind velocity (km/h)

z = altitude above sea level (m)

μ_{GM} = mass per unit area of geomembrane (kg/m^2)

Evaluate Variables

λ Suction factor = 0.70 for the entire side slope being considered (Attachment B, page 2)

V wind velocity = 25 mph = 40.2 km/h (IUC, 2003, see Attachment C)

z altitude above sea level (m)

A minimum elevation for the base side slopes is approximately 5,560 ft = 1694.7 meters

μ_{GM} mass per unit area of geomembrane (kg/m^2)

μ_{GM} = 1.41 kg/m^2 (Attachment B, page 3)

Evaluate Suction

$$S_e = 0.050(0.70)(40.2)^2 e^{-(1.252\times10^{-4})1694.7} - 9.81(1.41) = 31.9\,\text{Pa}$$

The maximum height of the exposed slope (2H:1V) is approximately 41 vertical feet, so the total length of exposed slope, L, is $L = \sqrt{41^2 + (2(41))^2} = 91.7\,\text{ft} = 27.9\,\text{m}$. See Attachment A for the conceptual base grading plan.

$$S_eL = 31.9\frac{N}{m^2}(27.9\,m)\times\frac{1kN}{1000N} = 0.89\frac{kN}{m}$$

Geomembrane Properties

The geomembrane properties needed for the calculations herein are tensile stiffness and strain. These values are chosen from manufacturer data for 60-mil HDPE smooth geomembrane (Attachment D). The tensile strength at yield is 126 ppi (22 kN/m) and the elongation at yield is 12%.

EVALUATION OF TENSION IN GEOMEMBRANE

The objective of this analysis is to evaluate wind induced tension in the geomembrane. Assuming the geomembrane tension strain curve is linear, the tension can be expressed as:

$T = J\varepsilon$ (Attachment B, page 5)

where: T = Tension

J = Stiffness

ε = Strain

To evaluate tension, we need to first evaluate stiffness and strain.

Stiffness, J

For strains less than 10 percent, the stress strain curve for geomembranes is approximately linear for HDPE geomembranes. Therefore, stiffness can be approximated as:

$J = Et_{GM}$

where: E = Elastic Modulus

= 450 MPa, this modulus value corresponds to wide-width tension values, according to Koerner (1998, Attachment E)

t_{GM} = Geomembrane Thickness

= 1.5×10^{-3} m (60 mil)

Therefore:

$J = (450\ \text{MPa})(0.0015\ \text{m}) = 675\ \text{kN/m}$

Strain, ε

The strain on the geomembrane induced by wind uplift loading can be estimated using Table 4 (Attachment B, 6/6):

$$\frac{J}{S_e L} = \frac{675}{0.89} = 758.4,$$ and from Table 4 (Attachment B, pg. 6), $\varepsilon = 0.42\%$

The maximum anticipated strain in the geomembrane that occurs is 0.42%, which is less than 10%; hence the assumption of a linear stress strain relationship for HDPE geomembrane is valid.

Therefore, the tension in the geomembrane is:

$$T = J\varepsilon = 675\frac{\text{kN}}{\text{m}}(0.0053) = 3.58\frac{\text{kN}}{\text{m}} = 20.4\ \text{ppi}$$

CONCLUSIONS

Based on the calculation performed herein, the geomembrane is acceptable for a wind speed of 25 mph (40.2km/h) with a slope length of approximately 92 ft (28 m). The tension in the geomembrane under the design conditions is 20.4 ppi (3.58 kN/m).

The capacity of the anchor trench is determined in a separate calculation package.

REFERENCES

Giroud, J.P., Pelte., Bathurst, R.J. 1995. *Uplift of Geomembranes by Wind*, Geosynthetic International, Vol. 2, No. 6, pg. 897-952.

(Attachment B)

International Uranium (USA) Corporation (IUC). 2003. *Environmental Report*. June 20, 2003, page 3-3.

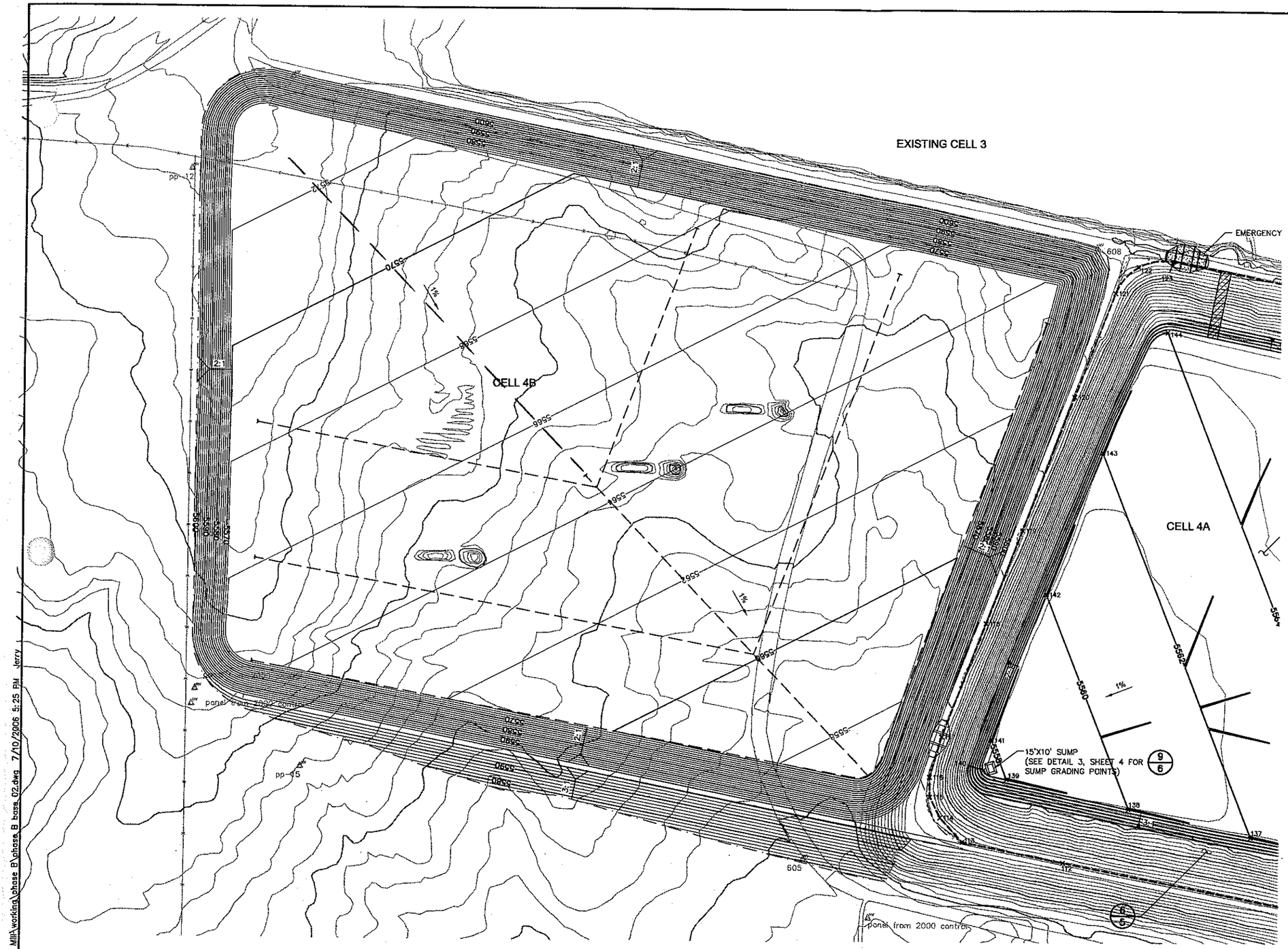
(Attachment C)

Geosynthetic Research Institute. 2003. GRI Test Method GM13, Standard Specification for "Test Properties, Testing Frequency and Recommended Warranty for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes." Revision 5: May 15, 2003.

(Attachment D)

Koerner, R.M. 1998. *Designing with Geosynthetics*, 4th Edition. Prentice-Hall: Upper Saddle River, NJ.

(Attachment E)



LEGEND

5560	EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
	EXISTING FENCE
5560	PROPOSED BASE GRADING (2' CONTOUR)
1%	SLOPE DIRECTION AND GRADE

N

100 50 0 100 200

SCALE IN FEET

ATTACHMENT A

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	

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GTC 11/11/05

• At altitude z above sea level:

$$S_e = 0.6465\lambda V^2 e^{-(1.252 \times 10^{-4})z} - 9.81\mu_{GM} \tag{40}$$

with S_e(Pa), V(m/s), z(m), μ_{GM}(kg/m^2)

$$S_e = 0.050\lambda V^2 e^{-(1.252 \times 10^{-4})z} - 9.81\mu_{GM} \tag{41}$$

with S_e(Pa), V(km/h), z(m), μ_{GM}(kg/m^2)

3.3 Determination of Geomembrane Tension and Strain

According to Equation 36, the effective suction results from two components: a component due to the wind-generated suction, which is normal to the geomembrane; and a component due to the geomembrane mass per unit area, which is not normal to the geomembrane. The component due to the geomembrane mass per unit area is generally small compared to the component due to the wind-generated suction. Therefore, the effective suction is essentially normal to the geomembrane. Since the effective suction is taken as normal to the geomembrane and has been assumed to be uniformly distributed over the length L of geomembrane, and since the problem is considered to be two-dimensional (see Section 3.2.2), the cross section of the uplifted geomembrane has a circular shape (Figure 9). As a result, the resultant F of the applied effective suction is equal to the effective suction multiplied by the length of chord AB, i.e. L:

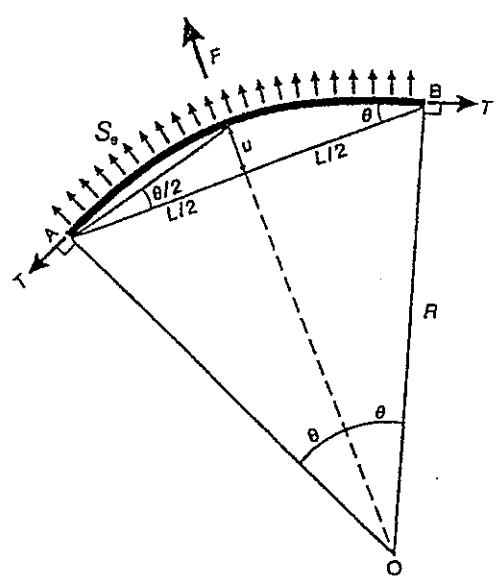

Figure 9. Schematic representation of uplifted geomembrane used for developing equations.

 Attachment B 1/6

CTC 11/11/05

- A leeward slope experiences a suction over its entire length. The suction on the leeward slope ranges between 45% of the reference pressure variation at the toe of the slope and 75% at the top of the slope, with an average value of 60%, i.e. $0.45 \leq \lambda \leq 0.75$ with an average value of 0.6.
- Large portions of the reservoir bottom are subjected to a suction ranging between 20% and 40% of the reference pressure variation ($0.2 \leq \lambda \leq 0.4$).

The above conclusions result from modeling in a wind tunnel where the wind velocity is constant. In reality, there are gusts of wind that may cause suctions greater than those indicated above, in localized areas for short periods of time.

Considering the conclusions from wind tunnel tests presented above and the need for extra safety due to gusts of wind, the following values of the suction factor, λ, are recommended for design of any slope based on the critical leeward slope:

- $\lambda = 1.00$ if the crest only is considered;
- $\lambda = 0.70$ if an entire side slope is considered;
- $\lambda = 0.85$ for the top third, $\lambda = 0.70$ for the middle third, and $\lambda = 0.55$ for the bottom third for a slope decomposed in three thirds by intermediate benches or anchor trenches as shown in Figure 7c and 7d; and
- $\lambda = 0.40$ at the bottom.

These recommendations are summarized in Figure 5. According to Equation 13, the suction factor, λ, is to be multiplied by Δp_R to obtain the suction S. The reference pressure variation, Δp_R, can be calculated using Equations 7 to 11.

It should be emphasized that the recommendations made above and used in the remainder of this paper rely entirely on the results of small-scale wind tunnel tests reported by Dedrick (1973, 1974a, 1974b, 1975). Nevertheless, the tests can be deemed representative of most practical situations because they were carried out on a wide range of dike cross section geometries and alignments typically associated with reservoir structures. However, a review of data for other shapes including obstacles with sinusoidal or smooth curve geometry can result in suction factors as great as $\lambda = 1.30$. Therefore, for unusual geometries, the designer may elect to increase the values of the suction factor, λ, given in Figure 5 by up to 30%. Also, for unusual geometries or large projects for which wind-induced damage of exposed geomembranes may have large financial consequences, wind tunnel tests of reduced-scale models or numerical simulation may be warranted.

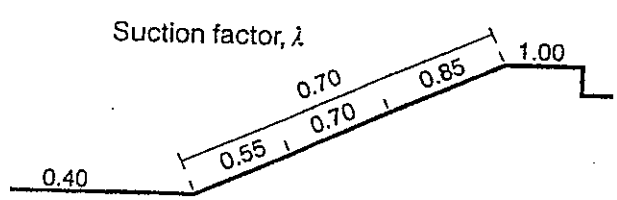

Figure 5. Recommended values of the suction factor for design of any slope based on the critical leeward slope.

Attachment B 2/6

GTC 11/11/05

Table 1. Typical density, thickness and mass per unit area for geomembranes, and relationship between mass per unit area and minimum uplift wind velocity.

Type of geomembrane	Geomembrane density ϱ_{GM} (kg/m^3)	Geomembrane thickness t_{GM} (mm)	Geomembrane mass per unit area μ_{GM} (4) (kg/m^2)	Minimum uplift wind velocity V_{upmin} (5) (km/h)
PVC (1)	1250 (2)	0.5	0.63	11.1
		1.0	1.25	15.7
HDPE (1)	940	1.0	0.94	13.6
		1.5	1.41	16.7
		2.0	1.88	19.2
		2.5	2.35	21.5
CSPE-R (1)	(3)	0.75	0.9	13.3
		0.90	1.15	15.0
		1.15	1.5	17.2
EIA-R (1)	(3)	0.75	1.0	14.0
		1.0	1.3	16.0
Bituminous	(3)	3	3.5	26.2
		5	6	34.3

Notes: (1) PVC = polyvinyl chloride; HDPE = high density polyethylene; CSPE-R = chlorosulfonated polyethylene-reinforced (commercially known as Hypalon); and EIA-R = ethylene interpolymer alloy-reinforced (commercially known as XR5). (2) PVC geomembranes have densities ranging typically from 1200 to 1300 kg/m^3. An average value has been used in this table. (3) These geomembranes consist of several plies of different materials with different densities. (4) The relationship between density, thickness and mass per unit area is expressed by Equation 16. (5) Calculated using Equation 27 which is applicable to a geomembrane located at sea level and subjected to a suction equal to the reference pressure variation. Values tabulated in the last column can be found in Figure 6 on the curve for $z = 0$.

$$\mu_{GM} \geq \mu_{GMreq} = 0.0659\lambda V^2 e^{-(1.252 \times 10^{-4})z} \text{ with } \mu_{GMreq}(\text{kg/m}^2),\ V(\text{m/s}) \text{ and } z(\text{m}) \quad (20)$$

$$\mu_{GM} \geq \mu_{GMreq} = 0.005085\lambda V^2 e^{-(1.252 \times 10^{-4})z} \text{ with } \mu_{GMreq}(\text{kg/m}^2),\ V(\text{km/h}) \text{ and } z(\text{m}) \quad (21)$$

Figure 6 gives the relationship between the geomembrane mass per unit area, μ_{GM}, and the wind velocity, V, as a function of the altitude above sea level, z, for the case $\lambda = 1$, corresponding to the case where the geomembrane is subjected to a suction equal to the reference pressure variation ($S = \Delta p_R$). Figure 6 shows that typical polymeric geomembranes, with masses per unit area ranging between 0.5 and 2 kg/m^2, can resist uplift at sea level by winds with velocities ranging between 10 and 20 km/h, whereas bituminous geomembranes, with masses per unit area ranging between 3.5 and 6 kg/m^2, can resist uplift at sea level by winds with velocities ranging between 25 and 35 km/h.

Example 1. A 1.5 mm thick HDPE geomembrane is located at the bottom of a reservoir. The altitude of the reservoir is 450 m. Would this geomembrane be uplifted by a wind with a velocity of 30 km/h?

Attachment B 3/6

GTC 11/11/05

3.2.2 *Mechanical Behavior of the Geomembrane*

The problem is assumed to be two-dimensional. Therefore, the geomembrane is assumed to be characterized by its tension-strain curve measured in a tensile test that simulates plane-strain conditions. A wide-width tensile test provides a satisfactory approximation of this case. If only results of a uniaxial tensile test are available, the tensile characteristics under plane-strain conditions can be derived from the tensile characteristics under uniaxial conditions as indicated by Soderman and Giroud (1995).

Essential characteristics of geomembranes for use in design are the allowable tension, T_{all}, and strain, ε_{all}. Typical tension-strain curves are shown in Figure 8:

- If the geomembrane tension-strain curve has a peak (Curve 1), the allowable tension and strain correspond to the values of T and ε at the peak (as shown in Figure 8) or before the peak if a margin of safety is required.
- If the geomembrane tension-strain curve has a plateau (Curve 2), the allowable tension and strain correspond to the values of T and ε at the beginning of the plateau (as shown in Figure 8) or before if a margin of safety is required.
- If the geomembrane tension-strain curve has neither peak nor plateau (Curve 3), the allowable tension and strain correspond to the values of T and ε at the end of the curve, i.e. at break (as shown in Figure 8), or before if a margin of safety is required.

In all three cases, values of T_{all} and ε_{all} that are less than the values given above can be selected for any appropriate reasons (i.e. to meet regulatory requirements, to limit deformations, etc.).

* In some cases, the geomembrane tension-strain curve, or a portion of it, is assumed to be linear. Then, the following relationship exists:

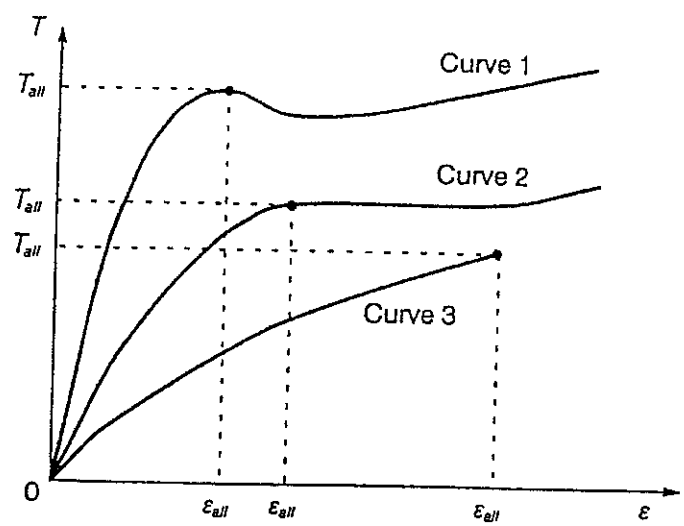

Figure 8. Typical tension-strain curves of geomembranes.

Attachment B 4/6

GTC 11/1/05

$$T = J\varepsilon \tag{34}$$

where: T = geomembrane tension; J = geomembrane tensile stiffness; and ε = geomembrane strain. The case of geomembranes with a linear tension-strain curve will be further discussed in Section 3.5.

It is important to note that geomembranes that are not reinforced with a fabric, for example PVC and PE geomembranes, have tensile characteristics that are highly dependent on temperature. Extensive data on the influence of temperature on the tensile characteristics of HDPE geomembranes are provided by Giroud (1994). The influence of temperature will be further discussed in Section 3.6.

3.2.3 *Suction Due to Wind*

In the subsequent analysis, the suction applied by the wind is assumed to be uniform over the entire length L. In reality, the suction due to the wind is not uniformly distributed as shown in Figure 4. Therefore, the design engineer using the method presented in this paper must exercise judgment in selecting the value of the length L and the value of the ratio λ defined by Equation 13.

In accordance with the discussions presented in Sections 2.3 and 2.4, the suction that effectively uplifts the geomembrane is:

$$S_e = S - \mu_{GM}\, g \tag{35}$$

where S_e is the "effective suction".

Combining Equations 2, 13 and 35 gives:

$$S_e = \lambda \varrho V^2/2 - \mu_{GM}\, g \tag{36}$$

Combining Equations 3 and 36 gives:

$$S_e = \lambda \varrho_o (V^2/2) e^{-\varrho_o g z/p_o} - \mu_{GM}\, g \tag{37}$$

Using the values of ϱ_o and p_o given in Section 2.1 and g = 9.81 m/s^2, Equation 37 gives:

- At sea level:

$$S_e = 0.6465 \lambda V^2 - 9.81 \mu_{GM} \tag{38}$$

with S_e(Pa), V(m/s), μ_{GM}(kg/m^2)

$$S_e = 0.050 \lambda V^2 - 9.81 \mu_{GM} \tag{39}$$

with S_e(Pa), V(km/h), μ_{GM}(kg/m^2)

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Table 4. Relationship between the strain of the geomembrane uplifted by the wind and the normalized tensile stiffness of the geomembrane for the case where the geomembrane has a linear tension-strain curve (Equation 57).

ε (%)	$\frac{J}{S_e L}$	ε (%)	$\frac{J}{S_e L}$	ε (%)	$\frac{J}{S_e L}$	ε (%)	$\frac{J}{S_e L}$
0	∞	3.6	31.347	7.2	11.607	10.8	6.607
0.1	6463.688	3.7	30.124	7.3	11.384	10.9	6.525
0.2	2288.342	3.8	28.981	7.4	11.168	11.0	6.443
0.3	1247.294	3.9	27.910	7.5	10.959	11.1	6.365
0.4	811.232	4.0	26.905	7.6	10.757	11.2	6.291
0.5	581.251	4.1	25.960	7.7	10.561	11.3	6.212
0.6	442.767	4.2	25.071	7.8	10.372	11.4	6.138
0.7	351.834	4.3	24.233	7.9	10.189	11.5	6.065
0.8	288.358	4.4	23.442	8.0	10.010	11.6	5.994
0.9	241.983	4.5	22.694	8.1	9.839	11.7	5.925
1.0	206.885	4.6	21.987	8.2	9.671	11.8	5.857
1.1	179.565	4.7	21.316	8.3	9.508	11.9	5.790
1.2	157.804	4.8	20.680	8.4	9.351	12.0	5.724
1.3	140.137	4.9	20.076	8.5	9.198	12.1	5.660
1.4	125.562	5.0	19.502	8.6	9.049	12.2	5.598
1.5	113.368	5.1	18.956	8.7	8.905	12.3	5.537
1.6	103.044	5.2	18.435	8.8	8.765	12.4	5.477
1.7	94.212	5.3	17.939	8.9	8.628	12.5	5.418
1.8	86.586	5.4	17.465	9.0	8.495	12.6	5.359
1.9	79.947	5.5	17.013	9.1	8.365	12.7	5.302
2.0	74.125	5.6	16.580	9.2	8.240	12.8	5.247
2.1	68.985	5.7	16.167	9.3	8.118	12.9	5.192
2.2	64.421	5.8	15.771	9.4	7.998	13.0	5.138
2.3	60.345	5.9	15.392	9.5	7.882	13.1	5.086
2.4	56.688	6.0	15.027	9.6	7.769	13.2	5.035
2.5	53.391	6.1	14.678	9.7	7.658	13.3	4.984
2.6	50.407	6.2	14.342	9.8	7.551	13.4	4.934
2.7	47.696	6.3	14.020	9.9	7.446	13.5	4.885
2.8	45.223	6.4	13.710	10.0	7.344	13.6	4.837
2.9	42.960	6.5	13.412	10.1	7.243	13.7	4.790
3.0	40.885	6.6	13.126	10.2	7.146	13.8	4.743
3.1	38.973	6.7	12.849	10.3	7.051	13.9	4.698
3.2	37.209	6.8	12.582	10.4	6.958	14.0	4.653
3.3	35.577	6.9	12.325	10.5	6.867	14.1	4.609
3.4	34.064	7.0	12.078	10.6	6.779	14.2	4.566
3.5	32.657	7.1	11.838	10.7	6.692	14.3	4.524

Attachment B 6/6

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The weather in the Blanding area is typified by warm summers and cold winters. The mean annual temperature in Blanding is about 50°F (10°C). January is usually the coldest month and July is usually the warmest month.

Winds are usually light to moderate in the area during all seasons, although occasional stronger winds may occur in the late winter and spring. The predominant winds are from the north through north-east (approximately 30 percent of the time) and from the south through south-west (about 25 percent of the time). Winds are generally less than 15 mph, with wind speeds faster than 25 mph occurring less than one percent of the time. The National Weather Service Station in Blanding, Utah is located about 6.25 miles (10km) north of the Mill. Data from the station is considered representative of the local weather conditions (1978 ER, Section 2.7.2).

Further description of local and regional weather and climate data are given in the 1978 ER (Section 2.7) and in the FES (Section 2.1).

3.3.1.2 On Site

On-site meteorological monitoring at the Mill was initiated in early 1977 and continues today. The original purpose of the meteorological monitoring program was to document the regional atmospheric baseline and to provide data to assist in assessing the potential air quality and radiological impacts arising from the operation of the Mill.

After the Mill construction was completed, the monitoring programs were modified to facilitate the assessment of Mill operations. The current meteorological monitoring program includes data collection for wind speed, wind direction, atmospheric stability according to the standard Pasquill scheme (via measurements of deviations in wind direction, referred to as sigma-theta), and precipitation as either rain or snow. The meteorological data are reported on a semi-annual basis. The details of these meteorological monitoring programs and the results are described in semi-annual reports prepared for IUSA and maintained at the Mill. Figure 3.3-1 shows windroses for the Mill site for January – December 2001.

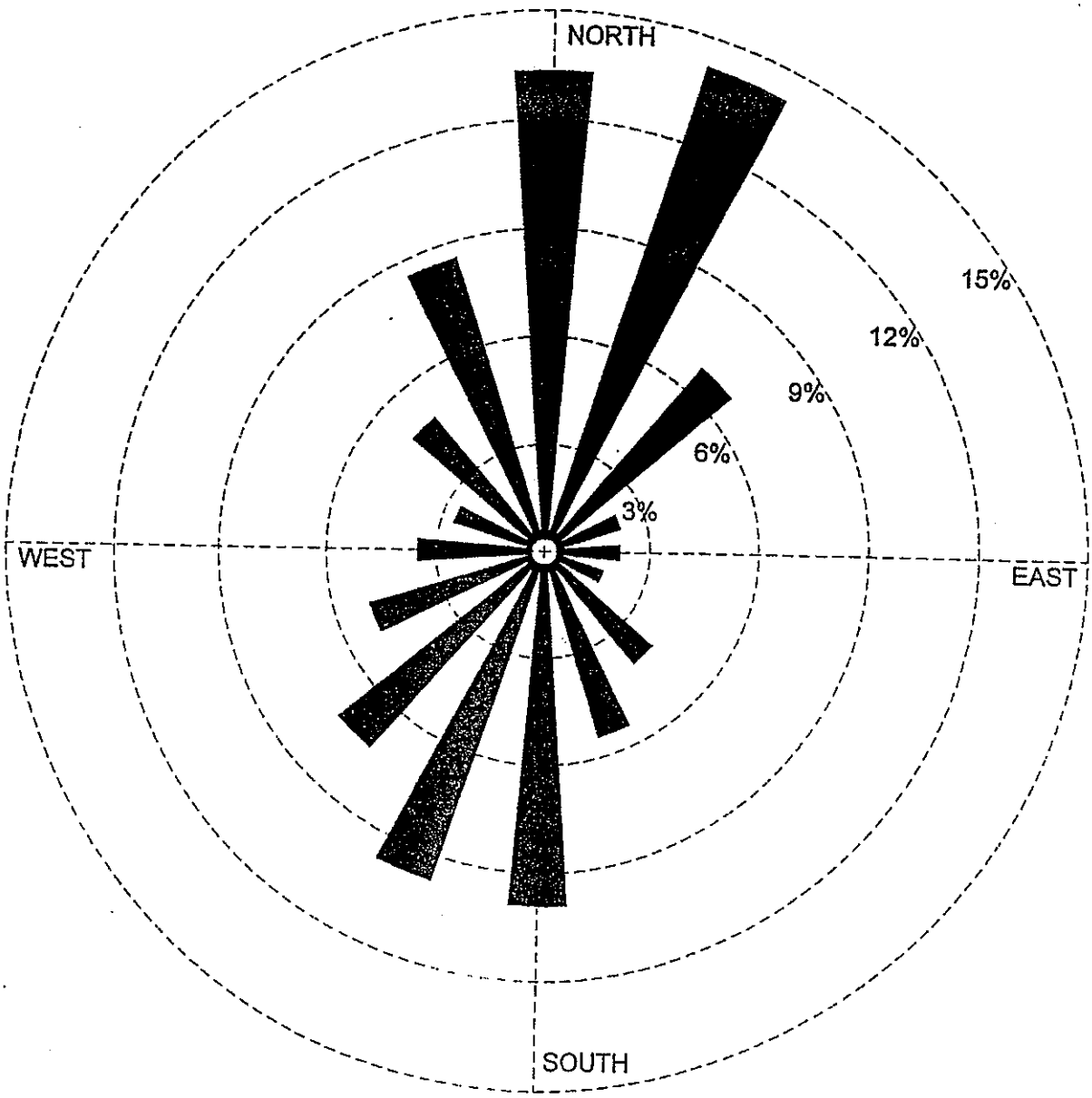
3.3.2 Baseline Air Quality

3.3.2.1 FES Evaluation

At the time of the 1978 ER and FES, the Four Corners Air Quality Control Region which encompasses parts of Colorado, Arizona, New Mexico and Utah and within which the Mill site is located had a priority IA rating, signifying a violation of federal air standards, for particulate matter and sulfur dioxide due to emissions from fossil-fueled power plants located within the region (1978 ER, Sect. 2.7.4.2). This was an important consideration at the time since the original proposal was to use coal and oil as the source of process and building heat. Thus, much of the discussion of potential air quality effects of the Mill arose from discussions of the potential

Attachment C

ETC 11/1/05



January through December 2001

International Uranium (USA) Corporation

Project	WHITE MESA MILL
REVISIONS	County: / State: UT
Date / By	Location:
	Figure 3.3-1 Wind Speed Direction (blowing from) For All Hours
	Scale: AS SHOWN / Date: June 2003 / figure 3.3-1.dwg
	Author: HRR / Drafted By: BM

ENGLISH UNITS

Table 1(a) – High Density Polyethylene (HDPE) Geomembrane -Smooth

Properties	Test Method	Test Value							Testing Frequency (minimum)
		30 mils	40 mils	50 mils	60 mils	80 mils	100 mils	120 mils	
Thickness (min. ave.)	D5199	nom.	Nom.	Nom.	Nom.	Nom.	Nom.	Nom.	Per roll
• lowest individual of 10 values		-10%	-10%	-10%	-10%	-10%	-10%	-10%	
Density mg/l (min.)	D 1505/D 792	0.940 g/cc	0.940 g/cc	0.940 g/cc	0.940 g/cc	0.940 g/cc	0.940 g/cc	0.940 g/cc	200,00 lb
Tensile Properties *(1)* (min. ave.)	D 638 Type IV								20,000 lb
• yield strength		63 lb/in.	84 lb/in.	105 lb/in.	126 lb/in.	168 lb/in.	210 lb/in.	252 lb/in.	
• break strength		114 lb/in.	152 lb/in.	190 lb/in.	228 lb/in.	304 lb/in.	380 lb/in.	456 lb/in.	
• yield elongation		12%	12%	12%	12%	12%	12%	12%	
• break elongation		700%	700%	700%	700%	700%	700%	700%	
Tear Resistance (min. ave.)	D 1004	21 lb	28 lb	35 lb	42 lb	56 lb	70 lb	84 lb	45,000 lb
Puncture Resistance (min. ave.)	D 4833	54 lb	72 lb	90 lb	108 lb	144 lb	180 lb	216 lb	45,000 lb
Stress Crack Resistance *(2)*	D5397 (App.)	300 hr.	300 hr.	300 hr.	300 hr.	300 hr.	300 hr.	300 hr.	per GRI-GM10
Carbon Black Content (range)	D 1603 *(3)*	2.0-3.0%	2.0-3.0%	2.0-3.0%	2.0-3.0%	2.0-3.0%	2.0-3.0%	2.0-3.0%	20,000 lb
Carbon Black Dispersion	D 5596	note *(4)*	note *(4)*	note *(4)*	note *(4)*	note *(4)*	note *(4)*	note *(4)*	45,000 lb
Oxidative Induction Time (OIT) (min. ave.) *(5)*									200,000 lb
(a) Standard OIT	D 3895	100 min.	100 min.	100 min.	100 min.	100 min.	100 min.	100 min.	
— or —									
(b) High Pressure OIT	D 5885	400 min.	400 min.	400 min.	400 min.	400 min.	400 min.	400 min.	
Oven Aging at 85°C *(5), (6)*	D 5721								
(a) Standard OIT (min. ave.) - % retained after 90 days	D 3895	55%	55%	55%	55%	55%	55%	55%	per each formulation
— or —									
(b) High Pressure OIT (min. ave.) - % retained after 90 days	D 5885	80%	80%	80%	80%	80%	80%	80%	
UV Resistance *(7)*	GM 11								
(a) Standard OIT (min. ave.)	D 3895	N.R. *(8)*	N.R. *(8)*	N.R. *(8)*	N.R. *(8)*	N.R. *(8)*	N.R. *(8)*	N.R. *(8)*	per each formulation
— or —									
(b) High Pressure OIT (min. ave.) - % retained after 1600 hrs *(9)*	D 5885	50%	50%	50%	50%	50%	50%	50%	

(1) Machine direction (MD) and cross machine direction (XMD) average values should be on the basis of 5 test specimens each direction.
Yield elongation is calculated using a gage length of 1.3 inches
Break elongation is calculated using a gage length of 2.0 in.

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

(3) Other methods such as D 4218 (muffle furnace) or microwave methods are acceptable if an appropriate correlation to D 1603 (tube furnace) can be established.

(4) Carbon black dispersion (only near spherical agglomerates) for 10 different views:
9 in Categories 1 or 2 and 1 in Category 3

(5) The manufacturer has the option to select either one of the OIT methods listed to evaluate the antioxidant content in the geomembrane.

(6) It is also recommended to evaluate samples at 30 and 60 days to compare with the 90 day response.

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

(8) Not recommended since the high temperature of the Std-OIT test produces an unrealistic result for some of the antioxidants in the UV exposed samples.

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

Attachment D

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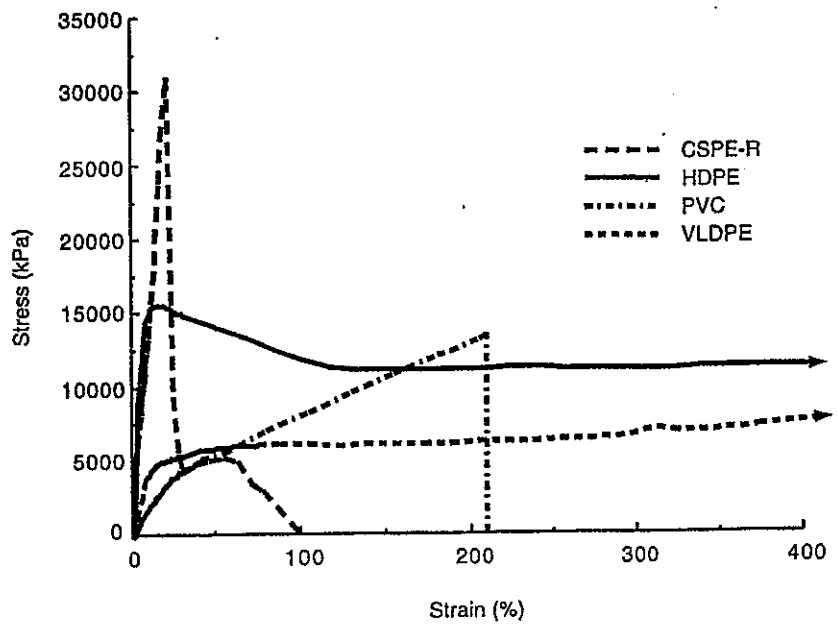


Figure 5.3 Tensile test results on 200 mm wide-width specimens of commonly used geomembranes using ASTM D4885 test method.

formation beneath a geomembrane is such a case. This type of behavior could well be anticipated for a geomembrane used in a landfill cover placed over differentially subsiding solid-waste material. The situation can be modeled by placing the geomembrane in an empty container, as shown in Figure 5.4. An appropriate seal is made with the cover section and water is introduced above the geomembrane. Pressure is mobilized until the failure of the test specimen occurs. Beginning with Stefan [6], a number of variations of this test have been made. It is currently formalized as ASTM D5716.

TABLE 5.5b TENSILE BEHAVIOR PROPERTIES OF HDPE, VLDPE, PVC, AND CSPE-R

		Wide-Width Tension Tests (Figure 5.3)			
Test Property	Unit	HDPE	VLDPE	PVC	CSPE-R
Maximum stress and	(kPa)	15,900	7,600	13,800	31,000
corresponding strain	(%)	15	400+	210	23
Modulus	(MPa)	450	69	20	300
Ultimate stress and	(kPa)	11,000	7,600	13,800	2,800
corresponding strain	(%)	400+	400+	210	79

Nom. thicknesses are: HDPE 1.5 mm, VLDPE 1.0 mm, PVC 0.75 mm, CSPE-R 0.91 mm.
Abbreviations: + = did not fail

Attachment E

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations **PIPE STRENGTH CALCULATIONS**

Computations by:	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	07/23/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	WMM – Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

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 Poly Vinyl Chloride (PVC) pipe will be buried under a maximum of 45 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 39.1 pounds per square inch (psi). Assuming a maximum allowable ring deflection of 7.5 percent, a schedule 40 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 45 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.

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

Written by:	R. Flynn	Date:	07/23/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	WMM – Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

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 × 45 ft = 5, 625 psf = 39.1 psi

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 D, 2/2)

t Minimum pipe wall thickness = 0.237 in. (Attachment D, 2/2)

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

E Modulus of elasticity of pipe = 400,000 psi

(for Class 12454-B rigid PVC pipe; Attachment E, 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:

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

$$= \frac{1.0(0.1)(39.1) + 0.1(0)}{\left[\frac{2(400{,}000)}{3(19.0-1)^3}\right] + 0.061(3{,}000)} = 1.7\%$$

Since the calculated ring deflection (1.7%) 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 D, 2/2)

and;

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

where:

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

A 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 (39.1 psi) provides:

FS_{WC} = 1,129/39.1

= 28.9

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

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 E, 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} = 137/39.1

= 3.5

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

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 39.1 psi will only cause a ring deflection of 1.7 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 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

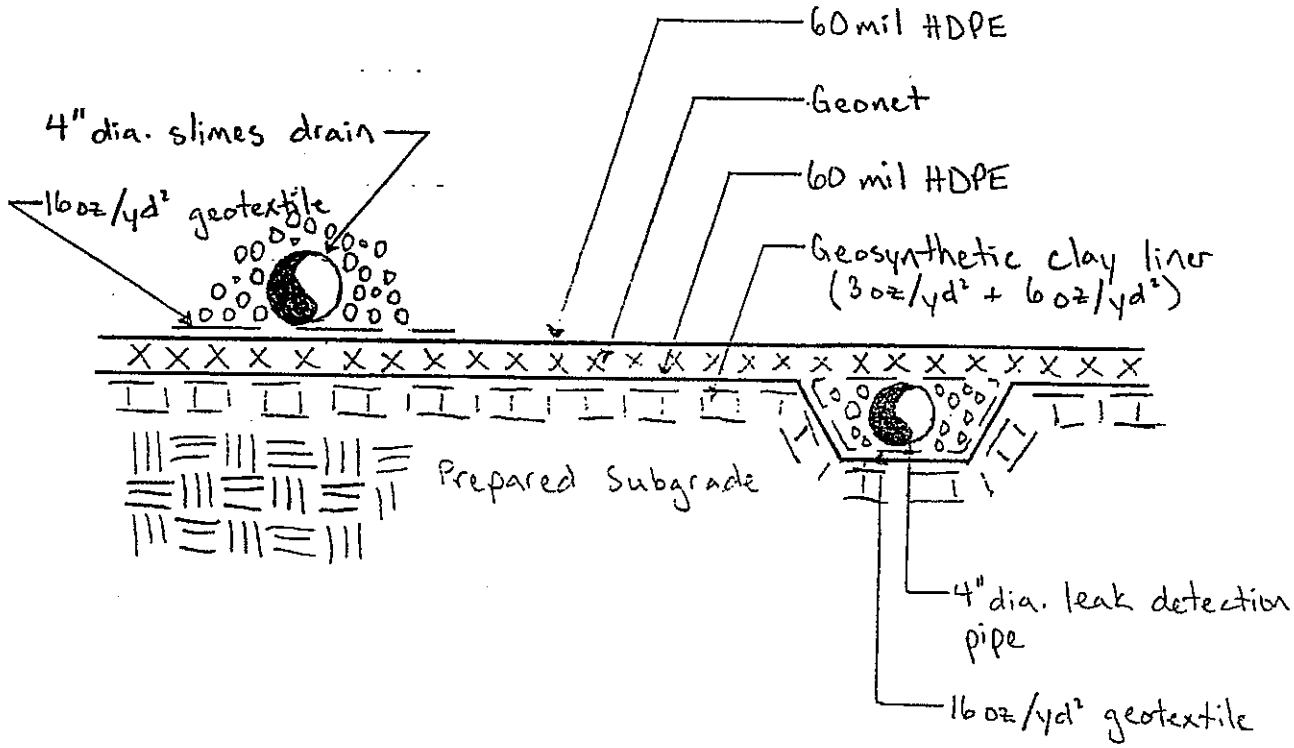
ASTM D 3034 (1997), "Standard Specification for Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings" ASTM Annual Method of Standards - Plastics

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

GEOSYNTEC CONSULTANTS

Written by: Jennifer Ferguson Date: 05/11/08 (YY/MM/DD) Reviewed by: GTC Date: 05/11/23 (YY/MM/DD)

Client: IUC Project: Tailings Cell 4A Project/Proposal No.: SC0349 Task No.: 03



NOT TO SCALE

Attachment A

GTC 11/23/05

ATTACHMENT B, 1/8

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

TABLE 6.3 - Continued

Height of Cover (ft)	Soil Unit Weight (lb/ft³) 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:

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

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 ($I_f = 0.766 - 0.133H$; $0 \le I_f \le 0.50$) * $1+I_f$ = 1.5 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 traffic

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DESIGN OF BURIED PVC PIPE

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

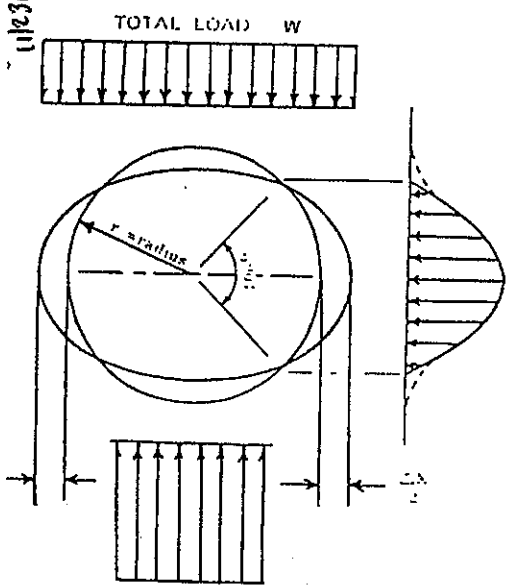
The resulting PS values for various dimension ratios and E values of PVC pipe are as shown in Table 7.1.

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, 3/3

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/ΔX
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

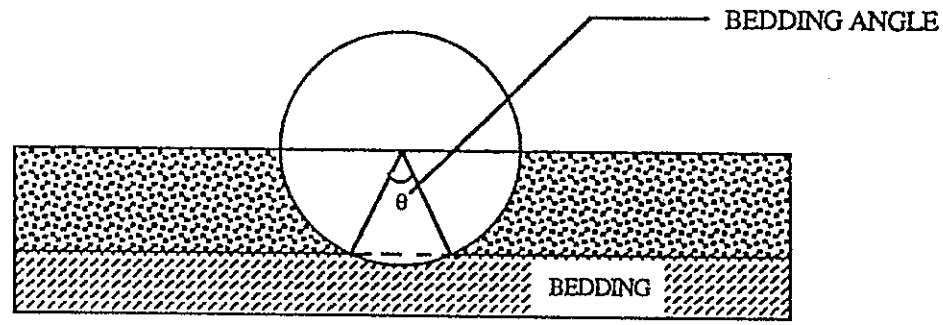


TABLE 7.2
VALUES OF BEDDING CONSTANT, K

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'. ires 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 C. Howard of the United States Bureau of Reclamation. Howard both laboratory and field data from many sources. Using information over 100 laboratory and field tests, he compiled a table of aver- alues 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 al deflection was then compared with actual measurements. By as- e 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 eel 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.

GRC 11/23/05

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:

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

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

6/23

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 pro 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 defined 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 t Long-term deflection tests were run at Utah State University by imposi given soil load which was held constant throughout the duration of the PVC pipe material creep properties have little influence on deflection but soil properties such as density exert great influence.

of up to 18 kip axle. 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-and 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-us 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.

gitudinal 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 itions - 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 , 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

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

or for solid wall pipes

EQUATION 7.16

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

d in this installation.

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

:

DR 35 PVC sewer pipe with a 400,000 psi modulus of elasticity ifined in a saturated soil providing E' = 800 psi, what height (H) of rated 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:

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

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

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

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

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

imum 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- 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:

ATTACHMENT B 8/8

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.

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

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

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

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

[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.

Current edition approved Dec. 10, 1996 and May 10, 1997. Published November 1997. Originally published as D 3034 – 72. Last previous edition D 3034 – 96.

[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			SDR-35			SDR-26			SDR 23.5		
	Average Inside Diameter	Base Inside Diameter[A]	7½ % Deflection Mandrel	Average Inside Diameter	Base Inside Diameter[A]	7½ % Deflection Mandrel	Average Inside Diameter	Base Inside Diameter[A]	7½ % Deflection Mandrel	Average Inside Diameter	Base Inside Diameter[A]	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.

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

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

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.	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

GTC
11/23/06

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

[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 D 1/2

GC 11/23/05

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)	...	0.020 (0.51)
1	1.315 (33.40)	±0.005 (±0.13)	...	0.020 (0.51)
1¼	1.660 (42.16)	±0.005 (±0.13)	...	0.024 (0.61)
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)	...	0.030 (0.76)
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)	0.060 (1.52)
6	6.625 (168.28)	±0.011 (±0.28)	0.100 (2.54)	0.070 (1.78)
8	8.625 (219.08)	±0.015 (±0.38)	0.150 (3.81)	0.090 (2.29)
10	10.750 (273.05)	±0.015 (±0.38)	0.150 (3.81)	0.100 (2.54)
12	12.750 (323.85)	±0.015 (±0.38)	0.150 (3.81)	0.120 (3.05)
14	14.000 (355.60	±0.015 (±0.38)	0.200 (5.08)	...
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 (3.55)	+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 D, 2/2

GTC 11/23/05

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 Insulatir 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

[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 E 1/9

GTC 11/23/06

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

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 E, 2/2

ATTACHMENT T

P 1/1 GTC 11/23/05

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			
x 10⁻⁵ cm/cm °C		5.2	9.9	6.2
x 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-0, 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/10¹⁴		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 F, 1/1

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations **EVALUATION OF LINER SYSTEM ANCHOR TRENCH CAPACITY**

Computations by:	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	10/31/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	WMM- Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

EVALUATION OF LINER SYSTEM ANCHOR TRENCH CAPACITY

WHITE MESA MILL

BLANDING, UTAH

OBJECTIVE

The project includes the installation of a double composite liner system within Cell 4B at the White Mesa Mill in Blanding, Utah. The proposed liner system and anchor trench is shown in Attachment A. The objective of this calculation package is to evaluate the tensile strength capacity for anchorage of the liner system at termination locations of the liner system with respect to wind uplift forces on the geomembrane. The anchor capacity presented herein is applicable to geomembrane and geonet pullout.

PROPOSED ANCHOR DETAILS

The proposed design for Cell 4B is an anchor trench at the top of slope. The trench will be located a minimum of 3 ft from the crest of the slope. The width and depth of the trench is 2 ft and 2 ft, respectively. The geosynthetic anchorage is presented in Attachment A.

Method of analysis

Anchor trench capacity is evaluated using methods and equations presented by Koerner (1998) and are included as Attachment B.

Koerner (1998) presents design equations developed from static equilibrium to evaluate the allowable geosynthetic tension from an anchor trench (see Attachment B). The equation considers frictional resistance due to (i) overburden pressures, (ii) anchor trench side slopes, and (iii) base of the anchor trench. The proposed design equation for determination of the allowable geomembrane tension from an anchor trench is:

$$T_{ult} = F_U + F_L + F_{AT\text{-}SIDE1} + F_{AT\text{-}SIDE2} + F_{AT\text{-}BASE1} + F_{AT\text{-}BASE2} \quad (3)$$

Where: T_{ult} = Ultimate tensile force in the geomembrane;

F_U = Friction force above the geosynthetics;

F_L = Friction force below the geosynthetics;

$F_U, F_L = q \tan\delta(L_{RO})$

q = surcharge pressure due to soil overburden

= depth of soil in anchor trench × unit weight

$\delta_{1,2}$ = minimum friction angle between liner system interfaces and the soil

L_{RO} = Runout length subjected to overburden

$F_{AT\text{-}SIDE}$ = Friction force due to the side of the anchor trench at each interface;

$F_{AT\text{-}SIDE} = (\sigma_h)_{ave} \tan\delta\ (d_{AT})$

$(\sigma_h)_{ave}$ = average horizontal stress in the anchor trench = $K_o(\sigma_v)_{ave}$

K_o = coefficient of earth pressure at rest = $1\text{-}\sin\varphi$

φ = effective stress envelope

$(\sigma_v)_{ave}$ = vertical overburden stress (depth of soil at mid-point of trench (plus additional overburden) multiplied by the soil unit weight (γ))

d_{AT} = depth of the anchor trench

$F_{AT\text{-}BASE}$ = Friction force due to the base of the anchor trench at each interface;

$F_{AT\text{-}BASE} = q \tan\delta_{1,2}(L_{AT})$

L_{AT} = width of the anchor trench

For this site, overburden will not be placed on top of the liner beyond the anchor trench, therefore $L_{RO} = 0$. So the equation for the allowable geomembrane tension from an anchor trench now becomes:

$$T_{ult} = F_{AT\text{-}SIDE1} + F_{AT\text{-}SIDE2} + F_{AT\text{-}BASE1} + F_{AT\text{-}BASE2}$$

ANALYSIS

Evaluating Variables

Since tension may develop in the geomembrane (see the calculation package Tension due to Wind Uplift) due to wind uplift forces and thermal forces, frictional forces will be mobilized along the geosynthetic and soil interfaces on the side and base of the anchor trench. The maximum load due to wind uplift is 20.4 lb/in (245 lb/ft) (see the calculation package Tension due to Wind Uplift). For the analysis presented herein, the following two interfaces are evaluated:

1. A friction angle of 18 degrees will be used to represent the interface friction value between the anchor trench backfill and the smooth geomembrane (Attachment C).
2. A friction angle of 11 degrees will be used to represent the interface friction value between the geomembrane and the GCL (Attachment D).

For determination of the surcharge due to soil overburden, q, and the vertical and horizontal overburden stresses, σ_h and σ_v, a unit weight of overburden soil of 125 pounds per cubic foot (pcf) was used. For evaluation of the effective horizontal overburden stress based on the coefficient of earth pressure at rest, a friction angle of 35 degrees was used for the soil. See Slope Stability calculation package for material parameter assumptions.

From Equation (3):

$T_{ult} = F_{AT\text{-}SIDE1} + F_{AT\text{-}SIDE2} + F_{AT\text{-}BASE1} + F_{AT\text{-}BASE2}$

$$F_{AT-SIDE1} = (\sigma_h)_{ave} \tan\delta_1 (d_{AT})$$
$$= K_o (\sigma_v)_{ave} (\gamma) \tan\delta (d_{AT})$$
$$= (1 - \sin 35°)\left(\frac{1}{2}(2\text{ ft})\right)(125\text{ pcf})\tan 18°(2\text{ ft})$$
$$= 34.6\text{ lb/ft}$$

$$F_{AT-SIDE2} = (\sigma_h)_{ave} \tan\delta_2 (d_{AT})$$
$$= K_o (\sigma_v)_{ave} (\gamma) \tan\delta (d_{AT})$$
$$= (1 - \sin 35°)\left(\frac{1}{2}(2\text{ ft})\right)(125\text{ pcf})\tan 11°(2\text{ ft})$$
$$= 20.7\text{ lb/ft}$$

$$F_{AT-BASE1} = q\tan\delta_1 (L_{AT})$$
$$= 2\text{ ft}(125\text{ pcf})\tan 18°(2\text{ ft})$$
$$= 162.5\text{ lb/ft}$$

$$F_{AT-BASE2} = q\tan\delta_2 (L_{AT})$$
$$= 2\text{ ft}(125\text{ pcf})\tan 11°(2\text{ ft})$$
$$= 97.2\text{ lb/ft}$$

$$T_{ult} = F_{AT-SIDE1} + F_{AT-SIDE2} + F_{AT-BASE1} + F_{AT-BASE2}$$
$$= 34.6 + 20.7 + 162.5 + 97.2$$
$$\mathbf{T_{ult} = 315\text{ lb/ft} > 245\text{ lb/ft} \quad OK}$$

CONCLUSIONS

The tensile capacity of the anchorage system as calculated herein exceeds the expected wind uplift tensile loads (from the calculation package entitled *Evaluation of Tension due to Wind Uplift*). The expected tensile load due to wind uplift was evaluated to be

245 lb/ft. The capacity of the anchor trench is 315 lb/ft. Therefore, the anchorage design for the geomembrane is adequate.

Based on the methods employed herein, results of analysis indicate that the design anchorage evaluated provides adequate tensile capacity to resist geomembrane tension induced by wind uplift forces.

NOTES TO PROJECT DOCUMENTS

The anchor trench shall be 2 ft deep and 2 ft wide during periods of wind uplift potential. The anchor trench shall be located at least 3 ft from the crest of the slope.

REFERENCES

Koerner, R.M. (1998), "*Designing with Geosynthetics*," 4th Edition. Prentice-Hall Inc.: Upper Saddle River, NJ.

Attachment B

GSE Lining Technology. "*GSE FrictionFlex Application Data.*" Technical Note.

Attachment C

SGI Testing Services, LLC. Direct Shear Testing per ASTM D5321, provided to Geosyntec Consultants, 2006.

Attachment D

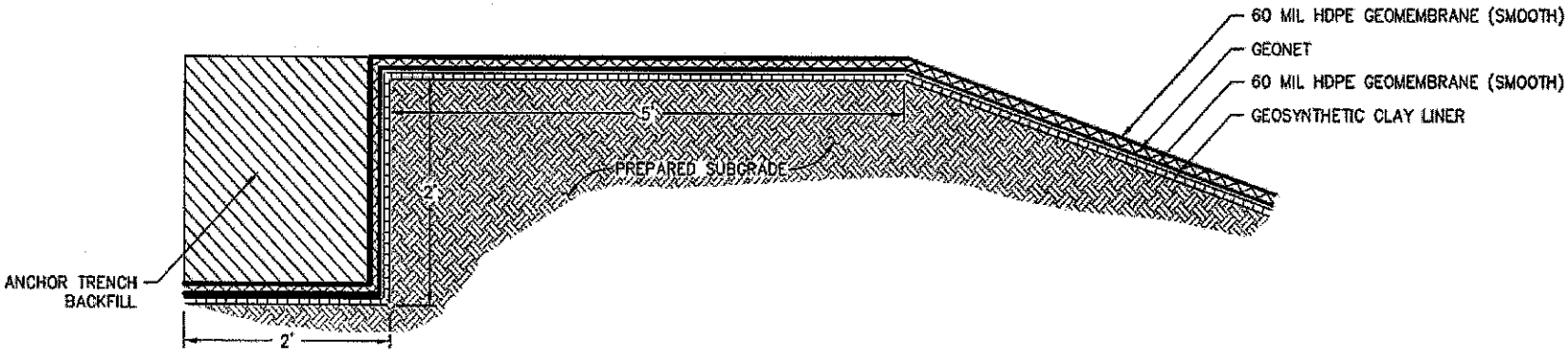
Hunt, Roy E. (1983). *Geotechnical Engineering Investigation Manual*, McGraw-Hill: New York.

Attachment E

Geosynthetic Research Institute, *"GRI Test Method GM13 – Standard Specification for "Test Methods, Test Properties, and Testing Frequency for High Density Polyethylene (HDPE) Smooth and Texture Geomembranes."* Adopted June 17, 1997, Revised July 10, 2006.

Attachment F

Geosyntec consultants



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

Attachment A, 1/1 (Geosyntec Consultants)

GTC
11/23/05

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

5.3.6 Runout and Anchor Trench Design

As shown in Figure 5.21 and the subsequent profile sections of geomembrane-lined reservoirs the liner comes up from the bottom of the excavation, covers the side slopes, and then runs over the top a short distance. It often terminates vertically down into an anchor trench. This anchor trench is typically dug by a small backhoe or trenching machine; the liner is draped over the edge, and then the trench is backfilled with the same soil that was there originally. The backfilled soil should be compacted in layers as the backfilling proceeds. Although concrete has been used as an anchorage block, it is rarely justified, at least on the basis of calculations, as will be seen in this section.

Regarding design, two separate cases will be analyzed: one with geomembrane runout only and no anchor trench at all (as is often used with canal liners), and the other as described above, with both runout and anchor trench considerations (as with reservoirs and landfills). Figure 5.30 defines the first situation, together with the forces and stresses involved. Note that the cover soil applies normal stress due to its weight, but does not contribute frictional resistance above the geomembrane. This is due to the fact that the soil moves along with the geomembrane as it deforms and undoubtedly cracks, thereby losing its integrity.

From Figure 5.30, the following horizontal force summation results, which leads to the appropriate design equation.

$$\Sigma F_x = 0$$

$$T_{\text{allow}} \cos \beta = F_{U\sigma} + F_{L\sigma} + F_{LT}$$

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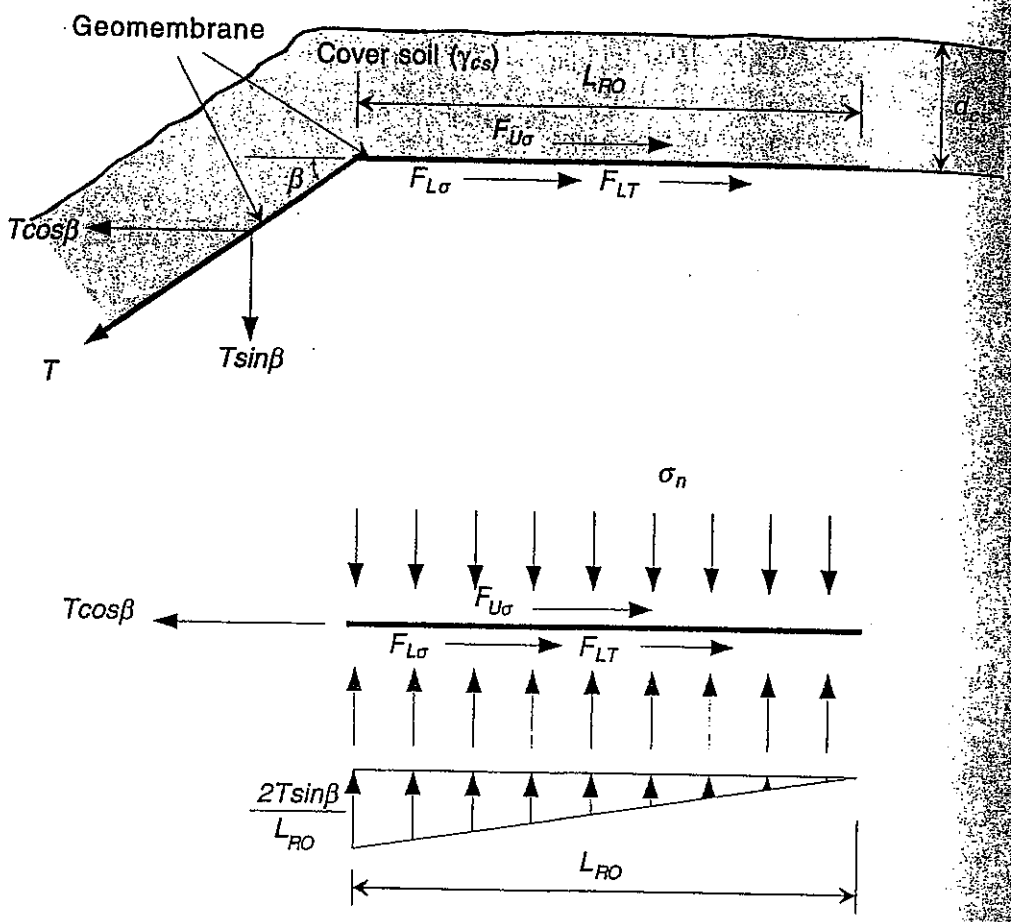

Figure 5.30 Cross section of geomembrane runout section and related stresses and forces involved.

$$= \sigma_n \tan \delta_U (L_{RO}) + \sigma_n \tan \delta_L (L_{RO}) + 0.5\left(\frac{2T_{\text{allow}} \sin \beta}{L_{RO}}\right)(L_{RO}) \tan \delta_L$$

$$L_{RO} = \frac{T_{\text{allow}}(\cos \beta - \sin \beta \tan \delta_L)}{\sigma_n(\tan \delta_U + \tan \delta_L)} \tag{5.25}$$

where

T_{allow} = allowable force in geomembrane stress = $\sigma_{\text{allow}} t$, where
σ_{allow} = allowable stress in geomembrane, and
t = thickness of geomembrane;
β = side slope angle;
$F_{U\sigma}$ = shear force above geomembrane due to cover soil (note that for thin cover soils tensile cracking will occur and this value will be negligible)

Attachment B 2/3

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

GEC 11/23/03

$F_{L\sigma}$ = shear force below geomembrane due to cover soil;
F_{LT} = shear force below geomembrane due to vertical component of T_{allow};
σ_n = applied normal stress from cover soil;
δ = angle of shearing resistance between geomembrane and adjacent material (i.e., soil or geotextile); and
L_{RO} = length of geomembrane runout.

Example 5.13 illustrates the use of the concept and the equations just developed.

Example 5.13

Consider a 1.0 mm thick VLDPE geomembrane with a mobilized allowable stress of 7000 kPa, which is on a 3(H) to 1(V) side slope. Determine the required runout length to resist this stress without use of a vertical anchor trench. In this analysis use 300 mm of cover soil weighing 16.5 kN/m³ and a friction angle of 30° with the geomembrane.

Solution: From the design equations just presented,

$$T_{\text{allow}} = \sigma_{\text{allow}} t$$

$$= (7000)(0.001)$$

$$T_{\text{allow}} = 7.0 \text{ kN/m}$$

and

$$L_{RO} = \frac{T_{\text{allow}}(\cos\beta - \sin\beta \tan\delta_L)}{\sigma_n(\tan\delta_U + \tan\delta_L)}$$

$$= \frac{(7.0)[\cos 18.4 - (\sin 18.4)(\tan 30)]}{(16.5)(0.30)[\tan 0 + \tan 30]}$$

$$= \frac{5.37}{2.86}$$

$$L_{RO} = 1.9 \text{ m}$$

Note that this value is strongly dependent on the value of mobilized allowable stress used in the analysis. To mobilize the failure strength of the geomembrane would require a longer runout length or embedment in an anchor trench. This, however, might not be desirable. Pullout without geomembrane failure might be a preferable phenomenon. It is a site-specific situation.

The situation with an anchor trench at the end of the runout section is illustrated in Figure 5.31. The configuration requires some important assumptions regarding the state of stress within the anchor trench and its resistance mechanism. In order to provide lateral resistance, the vertical distance within the anchor trench has lateral forces acting upon it. More specifically, an active earth pressure (P_A) is tending to destabilize the situation, whereas a passive earth pressure (P_P) is tending to resist pullout. As will

Attachment B 3/8

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

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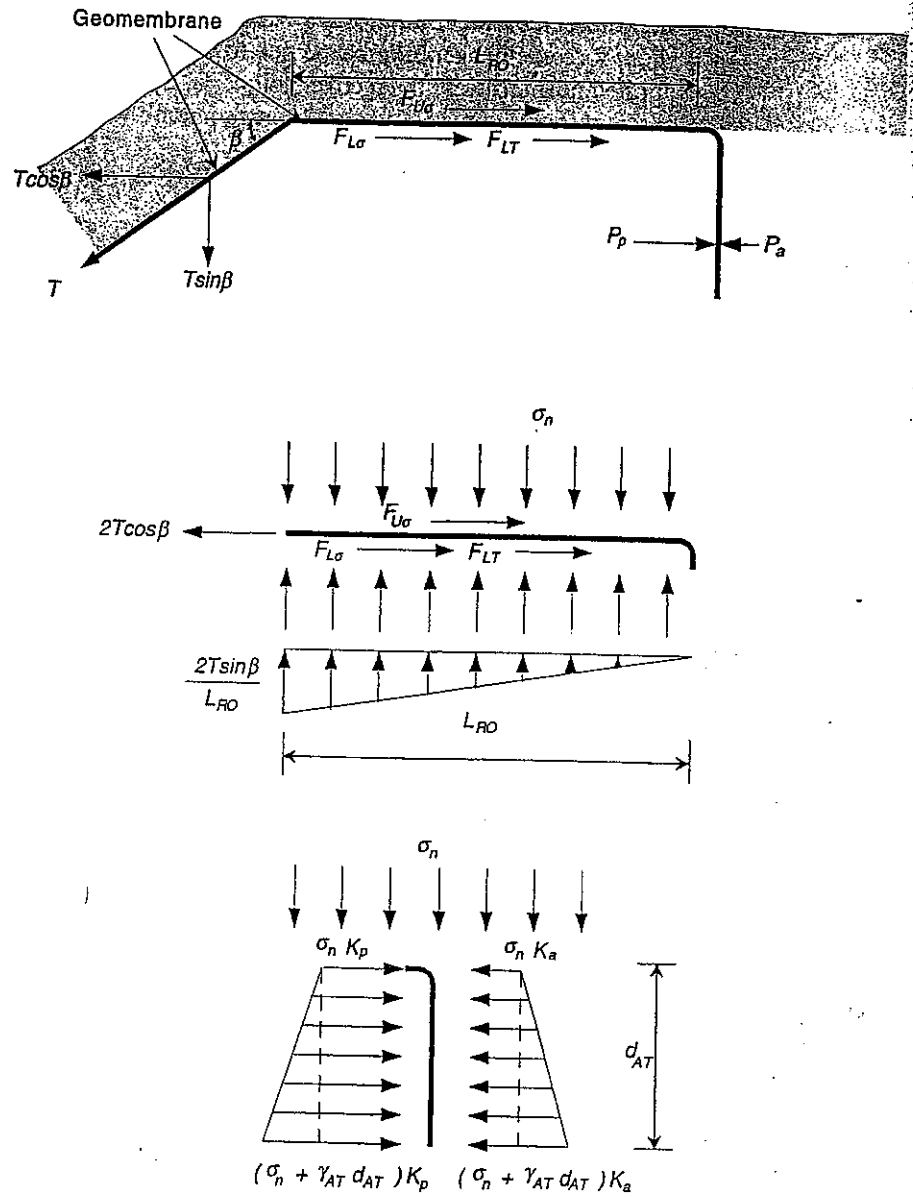

Figure 5.31 Cross section of geomembrane runout section with anchor trench and related stresses and forces involved.

be shown, this passive earth pressure is very effective in providing a resisting force (see Holtz and Kovacs [44]). Using the free-body diagram in Figure 5.31,

$$\Sigma F_x = 0$$

$$T_{\text{allow}} \cos\beta = F_{U\sigma} + F_{L\sigma} + F_{LT} - P_A + P_P \tag{5.26}$$

Attachment B 4/8

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

GCC 11/23/05

where

T_{allow} = allowable force in geomembrane = $\sigma_{allow}t$, where
σ_{allow} = allowable stress in geomembrane, and
t = thickness of geomembrane;
β = side slope angle;
$F_{U\sigma}$ = shear force above geomembrane due to cover soil (note that for thin cover soils, tensile cracking will occur, and this value will be negligible);
$F_{L\sigma}$ = shear force below geomembrane due to cover soil;
F_{LT} = shear force below geomembrane due to vertical component of T_{allow};
P_A = active each pressure against the backfill side of the anchor trench; and
P_P = passive earth pressure against the in-situ side of the anchor trench.

The values of $F_{U\sigma}$, $F_{L\sigma}$, and F_{LT} have been defined previously. The values of P_A and P_P require the use of lateral earth pressure theory.

$$P_A = \tfrac{1}{2}(\gamma_{AT}d_{AT})K_A d_{AT} + (\sigma_n)K_A d_{AT}$$

$$P_A = (0.5\gamma_{AT}d_{AT} + \sigma_n)K_A d_{AT} \tag{5.27}$$

$$P_P = (0.5\gamma_{AT}d_{AT} + \sigma_n)K_P d_{AT} \tag{5.28}$$

where

γ_{AT} = unit weight of soil in anchor trench,
d_{AT} = depth of the anchor trench,
σ_n = applied normal stress from cover soil,
K_A = coefficient of active earth pressure = $\tan^2(45 - \phi/2)$,
K_P = coefficient of passive earth pressure = $\tan^2(45 + \phi/2)$, and
ϕ = angle of shearing resistance of respective soil.

This situation results in one equation with two unknowns; thus a choice of either L_{RO} or d_{AT} is necessary to calculate the other. As with the previous situation, the factor of safety is placed on the geomembrane force T, which is used as an allowable value, T_{allow}. Example 5.14 illustrates the procedure.

Example 5.14

Consider a 1.5 mm thick HDPE geomembrane extending out of a facility as shown in Figure 5.31. What depth anchor trench is needed if the runout distance is constrained to 1.0 m? In the solution, use a geomembrane allowable stress of 16,000 kPa on a 3(H) to 1(V) side slope. There are 300 mm of cover soil at 16.5 kN/m³ placed over the geomembrane runout and anchor trench (this is also the unit weight of the anchor trench soil). The friction angle of the geomembrane to the soil is 30° (although assume 0° for the top of the geomembrane under a soil-cracking assumption) and the soil itself is 35°.

Attachment B 5/8

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

GTC 11/23/05

Solution: Using the previously developed design equations based on Figure 5.31:

$$T_{\text{allow}} = \sigma_{\text{allow}} t$$
$$= 16000(0.0015)$$
$$= 24.0 \text{ kN/m}$$

and

$$F_{U\sigma} = \sigma_n \tan \delta_U (L_{RO})$$
$$= (0.3)(16.5) \tan 0 (L_{RO})$$
$$= 0$$

$$F_{L\sigma} = \sigma_n \tan \delta_L (L_{RO})$$
$$= (0.3)(16.5) \tan 30 (L_{RO})$$
$$= 2.86 L_{RO}$$

$$F_{LT} = T_{\text{allow}} \sin \beta \tan \delta_L$$
$$= (24.0) \sin 18.4 \tan 30$$
$$= 4.37 \text{ kN/m}$$

$$P_A = (0.5\gamma_{AT} d_{AT} + \sigma_n) K_A d_{AT}$$
$$= [(0.5)(16.5) d_{AT} + (0.3)(16.5)] \tan^2 (45 - 35/2)\, d_{AT}$$
$$= [8.25 d_{AT} + 4.95](0.271) d_{AT}$$
$$= 2.24 d_{AT}^2 + 1.34 d_{AT}$$

$$P_P = (0.5\gamma_{AT} d_{AT} + \sigma_n) K_P d_{AT}$$
$$= [(0.5)(16.5) d_{AT} + (0.3)(16.5)] \tan^2 (45 + 35/2)\, d_{AT}$$
$$= [8.25 d_{AT} + 4.95](3.69) d_{AT}$$
$$= 30.4 d_{AT}^2 + 18.3 d_{AT}$$

This is substituted into the general force equation [Eq. (5.26)] to arrive at the solution in terms of the two variables L_{RO} and d_{AT}.

$$T_{\text{allow}} \cos \beta = F_{U\sigma} + F_{L\sigma} + F_{LT} - P_A + P_P$$
$$(24.0) \cos 18.4 = 0 + 2.86 L_{RO} + 4.37 - 2.24 d_{AT}^2 - 1.34 d_{AT} + 30.4 d_{AT}^2 + 18.3 d_{AT}$$
$$18.4 = 2.86 L_{RO} + 17.0 d_{AT} + 28.2 d_{AT}^2$$

Attachment B 6/8

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

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Since $L_{RO} = 1.0$ m, the equation can be solved for the unknown d_{AT}.

$$d_{AT} = 0.50 \text{ m}$$

Using this formulation we can develop a design chart for a wide range of geomembranes and thicknesses as characterized by different values of T_{allow}. For the specific conditions of Example 5.14,

$$\beta = 18.4°, \text{ which is } 3(H) \text{ to } 1(V)$$

$$\sigma_n = d_{cs}\gamma_{cs}$$

$$= (0.30)(16.5)$$

$$= 4.95 \text{ kN/m}^2$$

$$\delta_U = 0°$$

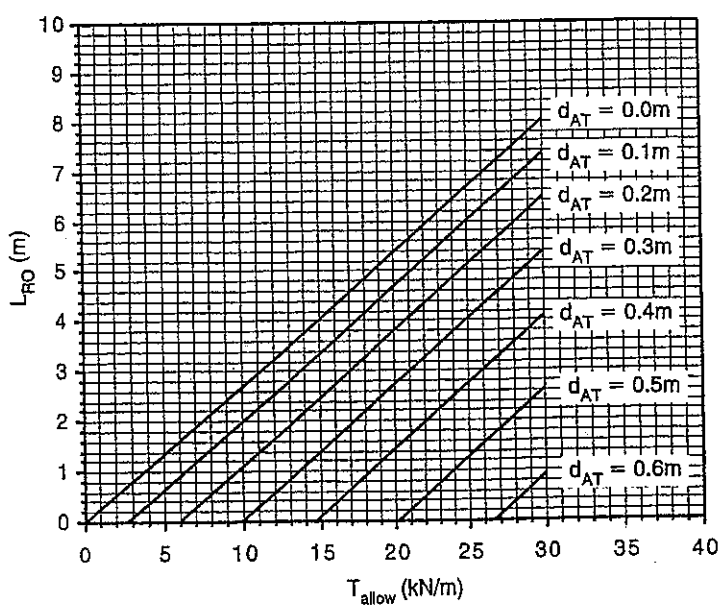
$$\delta_L = 30°$$

$$\phi = 35°$$

$$\gamma_{AT} = 16.5 \text{ kN/m}^3$$

$$\delta_{AT} = 30°$$

the response in terms of the two unknowns L_{RO} and d_{AT} is given in the following figure. Using this figure, Example 5.14 with the 1.5 mm thick HDPE at 24.0 kN/m gives an anchor trench depth of 0.50 m for an assumed runout length of 1.0 m. Other values can be readily selected.



Attachment B 7/8

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

GM 11/23/05

It should be noted that many manufacturers specify 500 mm deep anchor trenches and 1000 mm long runout sections. As seen above, this is very simplistic, for each membrane type and thickness requires its own analysis. By using a model as presented here, any set of conditions can be used to arrive at a solution. Even situations in which geotextiles and/or geonets are used in conjunction with the geomembrane (under, over, or both) and brought into the anchor trench can be analyzed in a similar manner.

5.3.7 Summary

Projects involving liquid containment using geomembranes are often extremely large. With large size come some inherent advantages over smaller projects. Foremost of these advantages is that most parties involved take the project seriously and approve of and enter into a planned and sequential design procedure. This section was laid out with this in mind, so that the design process proceeded step by step. Each element of design that is made leads to a new issue, which is followed by a new design element. Eventually, the quantitative process is concluded and details, often qualitative by nature, must be attended to. These details, such as seam type, seam layout, piping layout, and appurtenance details, are extremely important. They are, however, common to all geomembrane projects and therefore will be handled in Sections 5.10 and 5.11.

Although such large projects obviously warrant a careful design procedure, it does not follow that smaller projects do not deserve the same attention. Indeed, failures of small liner systems can be significant. Many warrant a design effort comparable to that of large projects, as illustrated in this section.

With this section behind us, we can now focus on other applications involving geomembranes. Where a similar analysis is called for, reference will be made back to this section. Thus only new and/or unique features of geomembrane projects will form the basis of the sections to follow.

5.4 COVERS FOR RESERVOIRS

Geomembrane covers are often used above the liquid surface of storage reservoirs. They are of fixed, floating, or suspended types.

5.4.1 Overview

There are a number of important reasons why liquid containment structures should be covered. These include: losses due to evaporation (up to 84% per year; see Cooley [45]), savings on chlorine treatment (for water reservoirs), savings on algae control chemicals (for water reservoirs), reduced air pollution (for reservoirs holding chemicals), reduced need for drainage and cleaning, increased safety against accidental drowning, protection from natural pollution entering the reservoir (e.g., animal excretion), and protection from intentional pollution (i.e., sabotage).

Obviously, a rigid roof structure could be constructed over the reservoir, but the costs involved are usually prohibitively high. At a far lower cost, both during initial construction or in a retrofitted system, is the use of an impermeable liner. All the materials

Attachment B 8/8

Source: *Koerner, R.M. (1998). "Designing with Geosynthetics," 4th Edition. Prentice-Hall: Upper Saddle River, NJ.*

TECHNICAL NOTE

GSE 11/23/06

*For environmental lining solutions... the world comes to GSE.**

GSE FrictionFlex* Application Data

GSE's FrictionFlex process provided the geomembrane industry's first textured liner. It is the only geomembrane texturing process ever to be granted a U.S. Patent. The FrictionFlex process begins with smooth GSE membrane that is manufactured to stringent industry standards. After the smooth surfaced sheet passes all GSE's standard quality assurance testing, texturing is added to one or both sides as required. The patented manufacturing process enables GSE to produce a textured liner exhibiting the outstanding mechanical and chemical properties demanded of GSE's premium grades of smooth geomembrane liners.

GSE geomembranes textured by the FrictionFlex process can be utilized to improve the factor of safety on steep slopes. This can increase facility design capacity, service life and ultimately, total revenue potential. GSE's textured geomembranes can be used to improve a number of applications.

GSE FrictionFlex geomembranes have an approximate six inch (15 cm) wide edge that remains smooth. This smooth edge means that GSE's seaming procedures are the same for FrictionFlex textured geomembranes and smooth geomembranes therefore requiring no changes in field quality control.

FrictionFlex liner has many performance benefits when in contact with soils and synthetics:

- High coefficient of friction with soils
- High coefficient of friction with other geosynthetic materials
- Premium grade mechanical and chemical properties
- Excellent narrow *and* wide width tensile elongation

The table below shows typical comparative data for smooth and FrictionFlex textured geomembranes. Testing was performed according to ASTM D 5321. GSE recommends that specific data be developed for all application designs. Shearbox testing of the specific geosynthetic and natural components of the composite is necessary to establish an appropriate design basis. GSE will be pleased to provide material samples for such purposes.

Friction Angle Comparison - Smooth vs. FrictionFlex Textured Geomembranes

	Smooth Geomembrane	GSE FrictionFlex Textured Liner Materials	
Material	Friction Angle (deg.)	Adhesion (lb/ft²)	Friction Angle (deg.)
Sandy Glacial Till	20	27	36
* Sandy Clay	18 *	65	35
Smooth Clay	16	39	32
Ottawa Sand	19	21	30
Non-woven Geotextile	12	133	33

This information is provided for reference purposes only and is not intended as a warranty or guarantee. GSE assumes no liability in connection with the use of this information.

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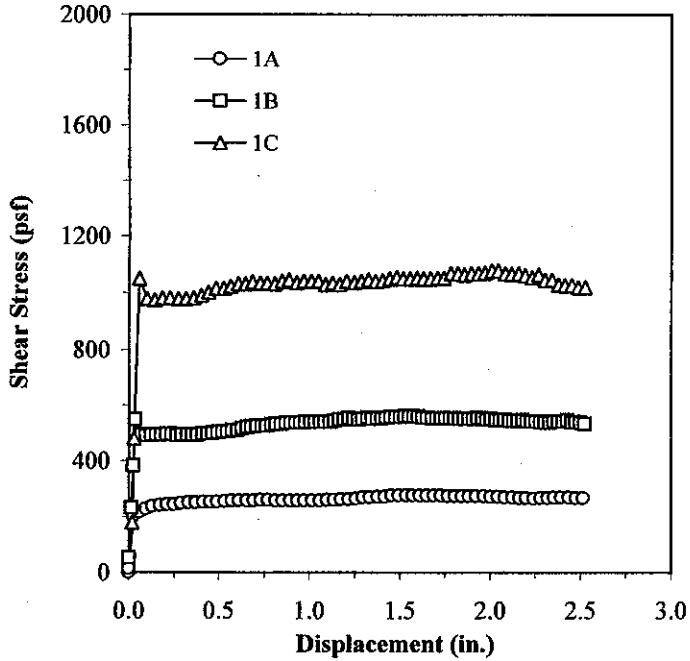
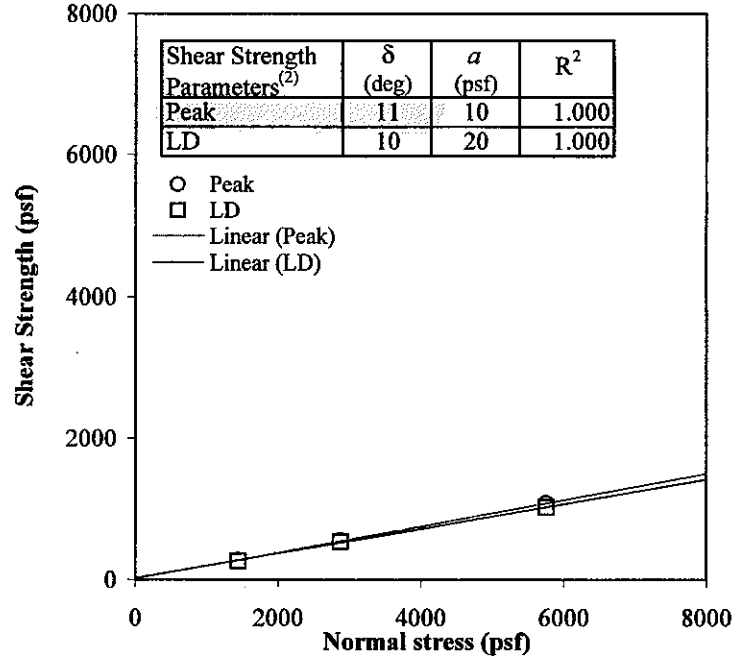
Represented by:

Attachment C

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		Consolidation		Lower Soil			Upper Soil			GCL		Shear Stress		Failure Mode
				Stress (psf)	Time (hour)	Stress (psf)	Time (hour)	γ_d (pcf)	ω_i (%)	ω_f (%)	γ_d (pcf)	ω_i (%)	ω_f (%)	ω_i (%)	ω_f (%)	τ_P (psf)	τ_{LD} (psf)	
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

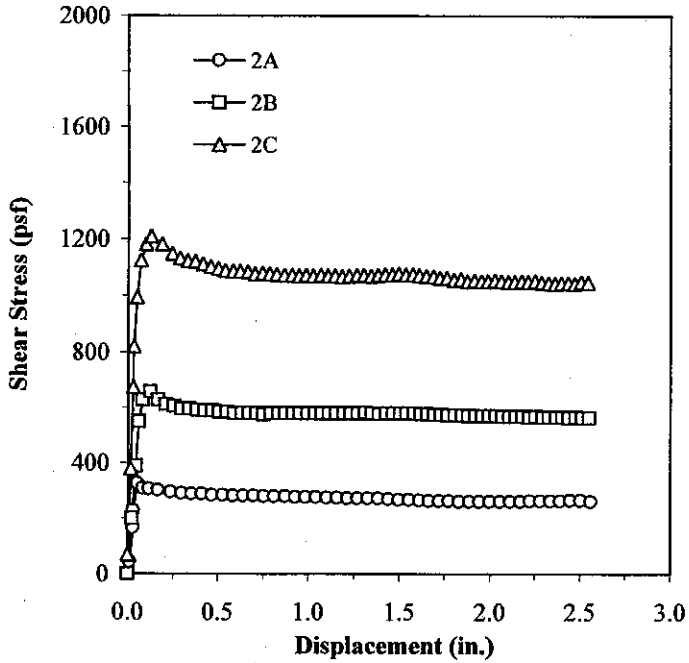
DATE OF TEST:	11/6/2006
FIGURE NO.	B-1
PROJECT NO.	SGI6014-03
DOCUMENT NO.	
FILE NO.	

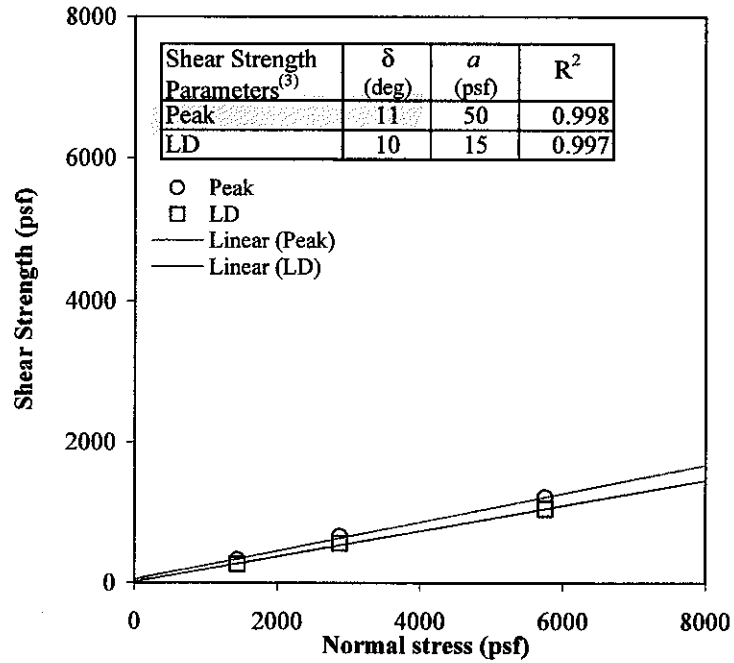
Attachment D, 1/2

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





Shear Strength Parameters[3]	δ (deg)	a (psf)	R^2
Peak	11	50	0.998
LD	10	15	0.997

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 D, 2/2

TABLE 5.3
TYPICAL PROPERTIES OF COMPACTED SOILS*

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, %	Typical value of compression: Percent of original height: At 1.4 tsf (20 psi)	At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted), psf	Cohesion (saturated), psf	Effective stress envelope ϕ, degrees	tan ϕ	Typical coefficient of permeability, ft/min	Range of CBR values	Range of subgrade modulus k_s, lb/in³
GW	Well-graded clean gravels, gravel-sand mixtures	125-135	11-8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40-80	300-500
GP	Poorly graded clean gravels, gravel-sand mix	115-125	14-11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30-60	250-400
GM	Silty gravels, poorly graded gravel-sand silt	120-135	12-8	0.5	1.1	...	...	>34	>0.67	$>10^{-6}$	20-60	100-400
GC	Clayey gravels, poorly graded gravel-sand-clay	115-130	14-9	0.7	1.6	...	...	>31	>0.60	$>10^{-7}$	20-40	100-300
SW	Well-graded clean sands, gravelly sands	110-130	16-9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20-40	200-300
SP	Poorly-graded clean sands, sand-gravel mix	100-120	21-12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10-40	200-300
SM	Silty sands, poorly graded sand-silt mix	110-125	16-11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10-40	100-300
SM-SC	Sand-silt clay mix with slightly plastic fines	110-130	15-11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	...	
SC	Clayey sands, poorly graded sand-clay mix	105-125	19-11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5-20	100-300
ML	Inorganic silts and clayey silts	95-120	24-12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100-200
ML-CL	Mixture of inorganic silt and clay	100-120	22-12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	..	
CL	Inorganic clays of low to medium plasticity	95-120	24-12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50-200
OL	Organic silts and silt-clays, low plasticity	60-100	33-21	.	...	...	.	...		...	5 or less	50-100
MH	Inorganic clayey silts, elastic silts	70-95	40-24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50-100
CH	Inorganic clays of high plasticity	75-105	36-19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50-150
OH	Organic clays and silty clays	65-100	45-21	..	...	...	...	...	...	...	5 or less	25-100

...TEAC Manual DM 7 (1982).[5] All properties are for condition of "standard Proctor" maximum density, except ... strength characteristics are for effective

Attachment E

Hunt, Roy E
"Geotechnical Engineering Investigation Manual" 1984

GEC 11/23/05

Table 2(a) – High Density Polyethylene (HDPE) Geomembrane - Textured

Properties	Test Method	Test Value							Testing Frequency (minimum)
		30 mils	40 mils	50 mils	60 mils	80 mils	100 mils	120 mils	
Thickness mils (min. ave.) • lowest individual for 8 out of 10 values • lowest individual for any of the 10 values	D 5994	nom. (-5%) -10% -15%	nom. (-5%) -10% -15%	nom. (-5%) -10% -15%	nom. (-5%) -10% -15%	nom. (-5%) -10% -15%	nom. (-5%) -10% -15%	nom. (-5%) -10% -15%	per roll
Asperity Height mils (min. ave.) *(1)*	GM 12	10 mil	10 mil	10 mil	10 mil	10 mil	10 mil	10 mil	every 2nd roll *(2)*
Density (min. ave.)	D 1505/D 792	0.940 g/cc	0.940 g/cc	0.940 g/cc	0.940 g/cc	0.940 g/cc	0.940 g/cc	0.940 g/cc	200,000 lb
Tensile Properties (min. ave.) *(3)* • yield strength • break strength • yield elongation • break elongation	D 6693 Type IV	63 lb/in. 45 lb/in. 12% 100%	84 lb/in. 60 lb/in. 12% 100%	105 lb/in. 75 lb/in. 12% 100%	126 lb/in. 90 lb/in. 12% 100%	168 lb/in. 120 lb/in. 12% 100%	210 lb/in. 150 lb/in. 12% 100%	252 lb/in. 180 lb/in. 12% 100%	20,000 lb
Tear Resistance (min. ave.)	D 1004	21 lb	28 lb	35 lb	42 lb	56 lb	70 lb	84 lb	45,000 lb
Puncture Resistance (min. ave.)	D 4833	45 lb	60 lb	75 lb	90 lb	120 lb	150 lb	180 lb	45,000 lb
Stress Crack Resistance *(4)*	D 5397 (App.)	300 hr.	300 hr.	300 hr.	300 hr.	300 hr.	300 hr.	300 hr.	per GRI GM10
Carbon Black Content (range)	D 1603 *(5)*	2.0-3.0 %	2.0-3.0 %	2.0-3.0 %	2.0-3.0 %	2.0-3.0 %	2.0-3.0 %	2.0-3.0 %	20,000 lb
Carbon Black Dispersion	D 5596	note *(6)*	note *(6)*	note *(6)*	note *(6)*	note *(6)*	note *(6)*	note *(6)*	45,000 lb
Oxidative Induction Time (OIT) (min. ave.) *(7)* (a) Standard OIT — or — (b) High Pressure OIT	 D 3895 D 5885	 100 min. 400 min.	 100 min. 400 min.	 100 min. 400 min.	 100 min. 400 min.	 100 min. 400 min.	 100 min. 400 min.	 100 min. 400 min.	200,000 lb
Oven Aging at 85°C *(7), (8)* (a) Standard OIT (min. ave.) - % retained after 90 days — or — (b) High Pressure OIT (min. ave.) - % retained after 90 days	D 5721 D 3895 D 5885	 55% 80%	 55% 80%	 55% 80%	 55% 80%	 55% 80%	 55% 80%	 55% 80%	per each formulation
UV Resistance *(9)* (a) Standard OIT (min. ave.) — or — (b) High Pressure OIT (min. ave.) - % retained after 1600 hrs *(11)*	GM11 D 3895 D 5885	 N.R. *(10)* 50%	 N.R. *(10)* 50%	 N.R. *(10)* 50%	 N.R. *(10)* 50%	 N.R. *(10)* 50%	 N.R. *(10)* 50%	 N.R. *(10)* 50%	per each formulation

(1) Of 10 readings; 8 out of 10 must be ≥ 7 mils, and lowest individual reading must be ≥ 5 mils; also see Note 6.
(2) Alternate the measurement side for double sided textured sheet
(3) Machine direction (MD) and cross machine direction (XMD) average values should be on the basis of 5 test specimens each direction.
Yield elongation is calculated using a gage length of 1.3 inches
Break elongation is calculated using a gage length of 2.0 inches
(4) P-NCTL test is not appropriate for testing geomembranes with textured or irregular rough surfaces. Test should be conducted on smooth edges of textured rolls or on smooth sheets made from the same formulation as being used for the textured sheet materials.
The yield stress used to calculate the applied load for the SP-NCTL test should be the manufacturer's mean value via MQC testing.
(5) Other methods such as D 4218 (muffle furnace) or microwave methods are acceptable if an appropriate correlation to D 1603 (tube furnace) can be established.
(6) Carbon black dispersion (only near spherical agglomerates) for 10 different views:
9 in Categories 1 or 2 and 1 in Category 3
(7) The manufacturer has the option to select either one of the OIT methods listed to evaluate the antioxidant content in the geomembrane.
(8) It is also recommended to evaluate samples at 30 and 60 days to compare with the 90 day response.
(9) The condition of the test should be 20 hr. UV cycle at 75°C followed by 4 hr. condensation at 60°C.
(10) Not recommended since the high temperature of the Std-OIT test produces an unrealistic result for some of the antioxidants in the UV exposed samples.
(11) UV resistance is based on percent retained value regardless of the original HP-OIT value.

Attachment F, GRI Institute, 2006.

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations: **CUSHION FABRIC CALCULATIONS**

Computations by:	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory Corcoran, P.E.	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	07/23/07	Reviewed by:	G. Corcoran	Date:	12/3/07
Client:	DMC	Project:	Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	03

CUSHION FABRIC CALCULATIONS
WHITE MESA MILL
BLANDING, UTAH

OBJECTIVE

The project involves placement of a double composite 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. The objective of this calculation is to evaluate the maximum particle sizes of soil/aggregate materials adjacent to the geomembrane that will not puncture or damage the geomembrane.

SUMMARY OF ANALYSIS

The analyses suggest that the following maximum particle sizes and geotextile mass per unit areas will be required:

Component of Liner	**Maximum Particle Size (in)**	**Maximum Protrusion Height (in)**	***Cushion Material***
Slimes drain system over geomembrane	¾	N/A	16 oz/yd^2
Leak detection system (LDS) over geomembrane	¾	N/A	16 oz/yd^2
Geosynthetic clay liner (GCL) over prepared subgrade	N/A	½	3 oz/yd^2 + 6 oz/yd^2

SITE CONDITIONS

The proposed double composite liner system will be comprised of the following components, from top to bottom:

- 60-mil HDPE primary geomembrane;
- Geonet (LDS);
- 60-mil HDPE secondary geomembrane; and
- GCL.

The slimes drain will be placed on top of the primary geomembrane, surrounded with gravel which will then be wrapped in a 16 oz/yd^2 geotextile. The LDS will be installed between the primary geomembrane and the secondary geomembrane, and will consist of a PVC pipe surrounded by aggregate and wrapped in a 16 oz/yd^2 geotextile. The GCL will be installed on the prepared subgrade.

The tailings deposits are anticipated to be similar to silt with an average maximum wet unit weight of 125 pounds per cubic foot (pcf) (See slope stability calculation for this value). For conservatism, we have assumed that a maximum of 42 ft of tailing deposits may be present. Therefore, the design overburden pressure is 42 ft × 125 pcf = 5,250 pounds per square foot (psf) or 251 kilopascals (kPa).

APPROACH

Wilson-Fahmy, Narejo, and Koerner have evaluated puncture protection of geomembranes in a series of three papers. These papers are:

1) Wilson-Fahmy, R.F., Narejo, D., and Koerner, R.M (1996) "Puncture Protection of Geomembranes Part I: Theory", Geosynthetics International, Vol. 3, No. 5, pp. 605-628

2) Narejo, D., Koerner, RM. and Wilson-Fahmy, R.F. (1996) "Puncture Protection of Geomembranes Part II: Experimental", Geosynthetics International, Vol. 3, No. 5, pp. 629-653

3) Koerner, R.M., Wilson-Fahmy, R.F. and Narejo, D. (1996) "Puncture Protection of Geomembranes Part III: Examples", Geosynthetics International, Vol. 3, No. 5, pp. 655-675

These papers present an evaluation of geomembrane puncture theory, the results of a laboratory experimental program, and design examples in regards to puncture protection of geomembranes. The design methods and conclusions of these papers were used for the analysis herein.

According to these papers, the important parameters that affect the puncture protection of geomembranes are: overlying pressure, mass per area of the geotextile, and the particle size and shape of the material overlying the geotextile. For the analysis herein, the overlying pressure and the mass per unit area of the geotextile are given and the maximum particle size is evaluated for the two types of geotextile.

ANALYSES

Narejo et al. (1996, Attachment B) present the following equation for evaluating geotextile puncture protection of 60 mil (1.5 mm) HDPE geomembrane:

$$H^2 = \frac{450 M_A}{P_{allow}} \qquad \text{(Attachment B)}$$

where:

H = cone height (mm), which corresponds to predicted effective protrusion height, which equals one-half maximum stone size.

Case I: M_A = mass per unit area geotextile (g/m^2)

= 16 oz/yd^2 = 542 g/m^2 (slimes drain and LDS)

Case II: M_A = 9 oz/yd^2 = 305 g/m^2 (GCL overlying prepared subgrade)

Written by:	R. Flynn	Date: 07/23/07	Reviewed by:	G. Corcoran	Date: 12/3/07
Client: DMC	Project: Cell 4B		Project/ Proposal No.:	SC0349	Task No.: 03

P_{allow} = maximum long term allowable pressure

where: $P_{allow} = P'_{allow}(MF_S \times MF_{PD} \times MF_A)(FS_{CR} \times FS_{CBD})$ (Attachment B)

where: MF_S, MF_{PD}, MF_A = modification factors (discussed below)

FS_{CR}, FS_{CBD} = partial factor of safety values (discussed below)

P'_{allow} = allowable pressure based on field conditions (Attachment B)

$= (FS)(P_{actual\ field\ pressure})$

where: FS = global factor of safety, 3.0 (Attachment B)

$P_{actual\ field\ pressure}$ = 251 kPa

P'_{allow} = (251)(3) = 753 kPa

MF_S = shape factor: (Attachment B)

1.0 (assume angular particles)

MF_{PD} = packing density: (Attachment B)

1.0 (assume isolated protrusions)

MF_A = soil arching: (Attachment B)

1.0 (assume none)

FS_{CR} = partial factor of safety for creep: (Attachment B)

for H>12 mm, FS_{CR} = 1.3

FS_{CBD} = partial factor of safety for chemical and biological degradation: (Attachment B)

2.0 (based on aggressive environment for polypropylene geotextiles in LDS and slimes drain)

1.0 (GCL on prepared subgrade)

Solving for P_{allow} provides:

Case I: $P_{allow} = (753)\,(1.0 \times 1.0 \times 1.0)\,(1.3 \times 2.0)$

$P_{allow} = 1{,}958$ kPa

Case II: $P_{allow} = (753)\,(1.0 \times 1.0 \times 1.0)\,(1.3 \times 1.0)$

$P_{allow} = 979$ kPa

Solving for H, the predicted effective protrusion height, provides:

Case I:

$$H^2 = \frac{450M_A}{P_{allow}}$$

$$H_{cushion} = \left(\frac{450(542)}{1{,}958}\right)^{1/2} = 11.2\text{ mm} = 0.4\text{ in}$$

Case II:

$$H_{cushion} = \left(\frac{450(305)}{979}\right)^{1/2} = 11.8\text{ mm} = 0.5\text{ in}$$

The predicted effective protrusion height equals one half the maximum stone size. Therefore, the maximum stone size for the gravel to be placed around the slimes drain

and in the LDS is 2 × 0.4 inches, or 0.8 inches. We recommend the maximum particle size for construction be ¾ inch for the slimes drain and LDS.

NOTE TO TECHNICAL SPECIFICATIONS

For practical construction and CQA purposes, the calculated maximum particle sizes and protrusion heights of the soil components of the liner are rounded down to a convenient magnitude. The subgrade will be rolled and compacted; therefore, the maximum protrusion height (instead of maximum particle size) is required for the technical specifications. The specifications should reflect the following information:

Soil Component of Liner	Maximum Protrusion Height (in.)	Maximum Particle Size (in.)
Drainage aggregate	N/A	¾
Prepared subgrade	½	N/A

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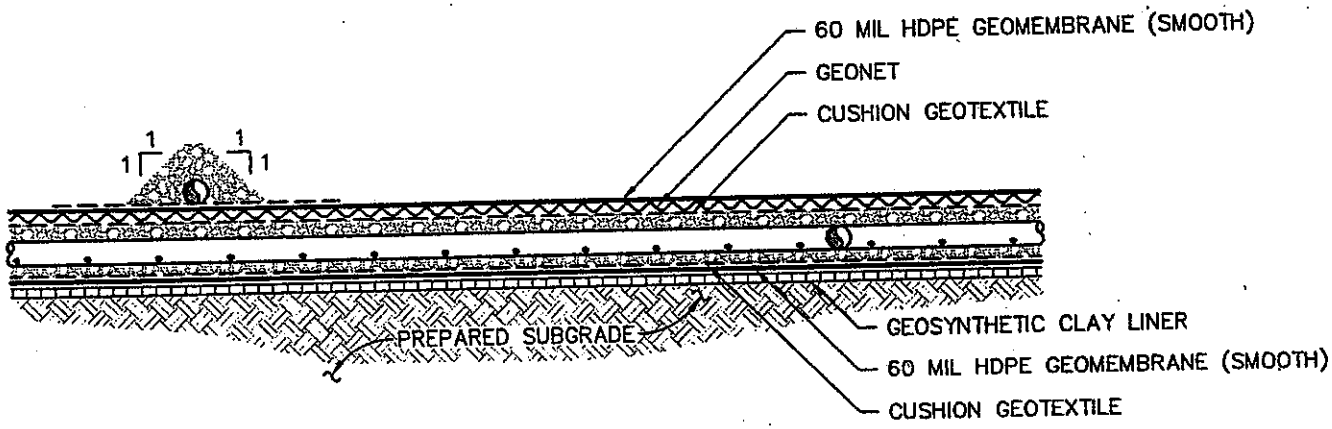

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Attachment

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SIMPLIFIED DESIGN CHARTS FOR GEOMEMBRANE CUSHIONS

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DERON N. AUSTIN, P.E. – SYNTHETIC INDUSTRIES, INC. (USA)

ABSTRACT

Recent and ongoing research indicates that use of a properly selected nonwoven, needle-punched geotextile cushion adjacent to (above and/or below) a geomembrane can effectively protect it from construction and operational damage. The current practice selects an appropriate geotextile cushion using the Geosynthetic Research Institute (GRI) method (Koerner, et. al. 1996). This method was used to develop simplified design charts allowing quick, conservative selection of an appropriate geotextile cushion. Charts are provided for typical applications including solid waste landfills and liquid impoundments with varying load, subgrade and cover/subgrade soil conditions. In addition, a brief discussion of the design procedure is provided with completed numerical examples.

INTRODUCTION

Most solid and hazardous waste landfills, lagoons and reservoirs built today incorporate geomembranes to contain liquids. Although these low permeability liners have demonstrated excellent performance, they are susceptible to damage when drainage stone or alternate drainage media (such as shredded tires, crushed glass, etc.) are placed over them (Figure 1). In addition, geomembranes are prone to damage from isolated protrusions present in the subgrade onto which they are deployed.

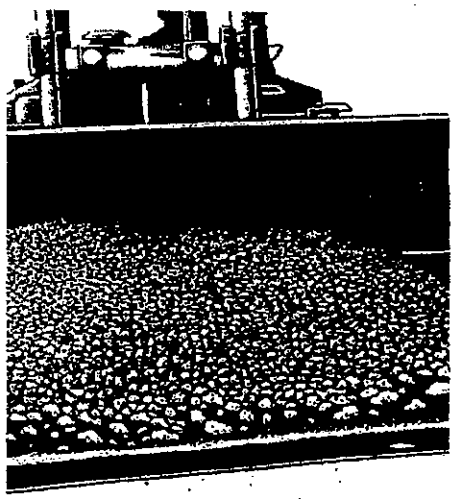

Figure 1. Stone Placement over a Geomembrane

Figure 2 illustrates the typical components of modern landfill liner system and Figure 3 represents a typical liquid impoundment liner system. Of these components, the geomembrane is the most prone to

Attachment – 1/16

damage. Protecting the geomembrane from tearing or puncturing during construction and operation is critical. Recent and ongoing research indicates that deployment of a properly selected nonwoven, needle-punched geotextile cushion adjacent to (above and/or below) a geomembrane provides effective protection against damage.

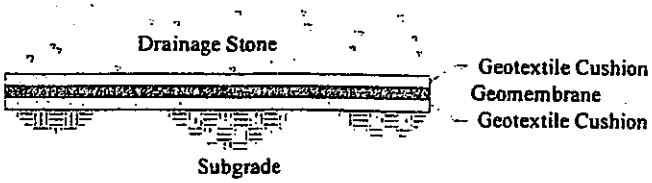

Figure 2. Typical Municipal Landfill Liner System

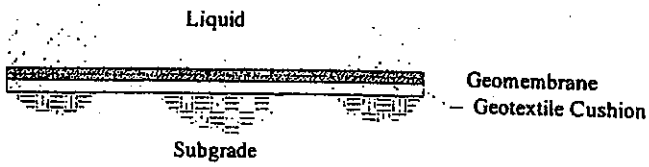

Figure 3. Typical Liquid Impoundment Liner System

STATE OF GEOMEMBRANE CUSHION DESIGN PRACTICE

State of geomembrane cushion design practice suggests using the generalized procedure developed by Koerner, et. al (1996) at the Geosynthetic Research Institute (GRI). The GRI method couples theoretical (Wilson-Fahmy, et. al., 1996) and empirical (Narejo, et. al., 1996) puncture protection analysis through use of a global factor of safety. The method directly applies to 1.5 mm (60 mil), smooth, high density polyethylene (HDPE) geomembranes protected by virgin polymer, nonwoven, needle-punched geotextiles. However, early work by Hullings and Koerner (1991) and field research by Richardson and Johnson (1998) indicate that the method may be conservative for geomembranes manufactured from more flexible polymers. Koerner, et. al. (1996) also suggest that the GRI method may be extended to other types of cushion materials.

The governing equation (Equation 1) incorporates several simplifying assumptions. For extrapolation to field design, (at least partially) subjective modification/reduction factors are required. In addition, laboratory testing used to develop the model did not incorporate dynamic loading. Therefore, the GRI method is considered adequate in cases where uniform, normal, static loading controls the design (i.e. moderate to high waste fills and most liquid impoundments) and may be under-conservative in cases where construction (dynamic) loading controls (i.e. shallow waste fills, poor construction practices, etc.). Based on field evaluation of geosynthetic cushions under construction loading, Richardson (1996) recommends modification of the GRI method such that the minimum nonwoven geotextile cushion mass selected is 405 g/m^2 (12 oz/yd^2) for 2.5 cm (1 in) maximum gravel over smooth HDPE geomembranes. This recommendation was later extended to 1.3 cm (0.5 in) gravel through additional field testing (Richardson and Johnson, 1998). Reddy et. al. (1996) performed similar field evaluations and concluded that a lighter 270 g/m^2 (8 oz/yd^2) geotextile is capable of providing adequate protection against construction loading. Based on laboratory testing, Reddy and Saichek (1998) also concluded that a 270 g/m^2 (8 oz/yd^2) may provide acceptable long-term protection under specific conditions.

Although a comprehensive review of previous research is beyond the scope of this paper, the reader is encouraged to read and understand the referenced literature prior to application or modification of the GRI method. This methodology (that forms the basis for the simplified design charts presented later in this paper) is summarized in the following steps.

Step 1: Estimate the Allowable Pressure on the Geomembrane (in terms of M_A)

$$P'_{allow} = \left(450 \cdot \frac{M_A}{H^2}\right)\left(\frac{1}{MF_S \cdot MF_{PC} \cdot MF_A}\right)\left(\frac{1}{FS_{CR} \cdot FS_{CBD}}\right) \qquad \text{(Equation 1)}$$

Where:

P'_{allow} = Allowable pressure on geomembrane (kPa)
450 = Empirical constant ($kPa \cdot mm^2/(g/m^2)$)
M_A = Required mass per unit area of nonwoven, needle-punched geotextile (g/m^2)
H = Effective height of protrusion (mm)
MF_S = Modification factor for protrusion shape (dimensionless)
MF_{PC} = Modification factor for protrusion configuration (dimensionless)
MF_A = Modification factor for overburden arching effect (dimensionless)
FS_{CR} = Factor of safety for geotextile creep (dimensionless)
FS_{CBD} = Factor of safety for geotextile chemical/biological degradation (dimensionless)

Equation 1 should be solved in terms of M_A. The effective height of protrusion (H) presents the maximum height of any object in contact with the geomembrane extends relative to the overlying/underlying media. In cases where protection is sought from uniformly packed stones (such as landfill leachate collection/drainage media), H may be estimated as one-half the maximum particle diameter of the stones. However, when protection is sought from isolated protrusions (such as stones encountered in a hastily prepared subgrade), H may be estimated as the maximum particle diameter of the protrusions. In the later case, the value of H may be based on observed conditions, or specified by restricting the largest particle size allowed to remain on the prepared subgrade during geosynthetic deployment. Modification Factors for protrusion shape, protrusion configuration, and overburden arching effect may be selected based on guidelines presented by Narejo, et. al (1996):

Table 1. Recommended Modification Factor for Protrusion Shape
(Adapted from Narejo, et. al., 1996, page 647)

Protrusion Shape	Modification Factor, MF_S
Angular	1.00
Subrounded	0.50
Rounded	0.25

Table 2. Recommended Modification Factor for Protrusion Configuration
(Adapted from Narejo, et. al., 1996, page 647)

Protrusion Configuration	Modification Factor, MF_{PC}
Isolated Protrusions	1.00
Uniformly Packed Surface	0.50

Table 3. Recommended Modification Factor for Overburden Arching Effect
(Adapted from Narejo, et. al., 1996, page 648)

Anticipated Arching Effect	Modification Factor, MF_A
None (i.e. Liquid Overburden)	1.00
Moderate	0.50
Maximum	0.25

Through limited creep testing, Narejo, et. al. (1996) indicated that geotextile cushion creep is primarily a function of H and M_A. Since M_A is unknown at this point, Equation 1 may be solved by assuming a reasonable value for FS_{CR} based on the anticipated M_A required. Following completion of the required calculations, the assumed value of FS_{CR} must be checked for validity. Table 4 provides recommended FS_{CR} values in the form of unique linear equations for several commonly available nonwoven, needle-punched geotextiles. It is interesting to note that the recommended upper limit with respect to H, is in general agreement (probably by coincidence) with construction limits established by Richardson (1996) and Reddy and Saichek (1996). The equations for FS_{CR} and their range of validity were interpolated/extrapolated from available geotextile cushion creep test data (Narejo, et. al., 1996).

Table 4. Factor of Safety for Geotextile Creep
(Adapted from Narejo, et. al., 1996, page 644 - 648)

Nonwoven, Needle-punched Geotextile Mass per Unit Area	Factor of Safety, FS_{CR}
270 g/m^2 (N/R for H > 12 mm)	$\approx 0.0417 \cdot H + 1.25$
405 g/m^2 (N/R for H > 19 mm)	$\approx 0.0292 \cdot H + 1.18$
540 g/m^2 (N/R for H > 25 mm)	$\approx 0.0166 \cdot H + 1.11$
675 g/m^2 (N/R for H > 29 mm)	$\approx 0.0139 \cdot H + 1.08$
745 g/m^2 (N/R for H > 31 mm)	$\approx 0.0129 \cdot H + 1.07$
810 g/m^2 (N/R for H > 32 mm)	$\approx 0.0119 \cdot H + 1.06$
945 g/m^2 (N/R for H > 35 mm)	$\approx 0.0100 \cdot H + 1.03$
1015 g/m^2 (N/R for H > 36 mm)	$\approx 0.0089 \cdot H + 1.02$
1080 g/m^2 (N/R for H > 38 mm)	$\approx 0.0080 \cdot H + 1.00$

NOTE: N/R = Not recommended

The factor of safety for chemical and biological degradation (FS_{CBD}) should be selected based on the aggressiveness of the anticipated chemical environment and the geotextile polymer composition. Table 5 provides general recommendations:

Table 5. Recommended Factor of Safety for Chemical and Biological Degradation (based on Koerner, 1994, page 151 and Synthetic Industries, 1997)

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Chemical Environment	Factor of Safety for Chem/Bio Degradation, FS_{CBD}	
	Polyester (PET) Geotextiles	Polypropylene (PP) Geotextiles
Normal (i.e. 3 < pH < 10)	1.0	1.0
Aggressive (pH <3 or pH > 10)	1.5 – 2.0	1.0 – 1.5

Step 2: Estimate the Anticipated Pressure on the Geomembrane

$$P_{actual} = \gamma \cdot h \qquad \text{(Equation 2)}$$

Where:

γ = Unit weight of overburden material or liquid (kN/m^3)
h = Design height of overburden material or liquid depth (m)
P_{actual} = Estimated maximum pressure on geomembrane (kPa)

The parameters required to complete Equation 2 may be assumed or specified based on site specific considerations. The unit weight of typical municipal solid waste may be estimated to equal 12.56 kN/m^3 (80 lb/ft^3) in the absence of site specific data. Likewise, the unit weight of most liquids can be approximated by the unit weight of water, 9.81 kN/m^3 (62.4 lb/ft^3).

In some cases (i.e. shallow waste fills, poor construction practices, etc.), the dynamic orces associated with construction loading may exceed those associated with long-term static loading. The exact point at which this occurs is dependent on multiple variables and difficult (if not impossible) to estimate. Therefore, caution should be exercised in selection of a geotextile cushion having a mass per unit area less than 405 g/m^2 (12 oz/yd^2), the construction limit recommended by Richardson and Johnson (1998).

Step 3: Calculate the Required Mass per Unit Area of the Cushion Geotextile

$$P'_{allow} \geq FS_{gmin} \cdot P_{actual} \qquad \text{(Equation 3)}$$

Where:

P'_{allow} = Allowable pressure on geomembrane in terms of M_A (Equation 1)
FS_{gmin} = Global Factor of Safety (dimensionless)

Equation 3 may be solved for M_A through substitution (Equation 1 and 2 results) and algebraic manipulation. The global factor of safety (FS_{gmin}) should be selected based on the protrusion configuration and H. Recommendations are provided in Table 6.

Table 6. Recommended Global Factor of Safety
(Adapted from Koerner, et. al., 1996, page 648)

Protrusion Configuration	Global Factor of Safety, FS_{gmin}
Isolated Protrusions	$= 0.22 \cdot H + 1.77$ (≥ 3.0)
Uniformly Packed Surface	3.0

Step 4: Select Appropriate Geotextile Cushion

Select a nonwoven, needle-punched geotextile having a minimum average roll value (MARV) M_A greater than or equal to that calculated in Step 3. It should be noted that the method presented herein is based on limited testing (Narejo, et. al, 1996) using virgin polymer, nonwoven, needle-punched geotextile and may not apply to all types of geotextiles and cushion materials.

Step 5: Check Assumed Value of FS_{CR} and Construction Limits

In Step 1, FS_{CR} was assumed to allow solution of Equation 1. Check Table 4 to ensure that the assumed value is valid for the geotextile selected in Step 4 (If not, revise FS_{CR} and repeat Steps 1 through 4).

In cases where solid material (i.e. rock, solid waste, etc.) will be placed on top of the geomembrane with heavy equipment, construction loading must be considered. Based on field experimentation, the minimum M_A geotextile should be between 270 g/m^2 (8 oz/yd^2) (Reddy, et. al., 1996) and 405 g/m^2 (12 oz/yd^2) (Richardson and Johnson, 1998) to prevent construction damage. The reader should review and understand both documents prior to selecting a geotextile having M_A less than 405 g/m^2.

SIMPLIFIED GEOMEMBRANE CUSHION SELECTION CHARTS

A series of simplified design charts have been developed for the most common geomembrane cushioning applications based on the methodology presented. These charts allow the user to quickly and conservatively select an appropriate virgin polymer, nonwoven, needle-punched geotextile cushion. The applicability and assumptions associated with these charts are provided in the notes section of each figure. In addition, the reader is encouraged to review and understand the limitations of the GRI method (discussed in the referenced literature) prior to application the charts on the following pages. Figures 4 through 7 present charts for landfill applications while Figures 8 and 9 relate to liquid impoundment applications.

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EXAMPLES

The following simple design examples illustrate application of the charts and GRI ethod to three common geomembrane cushion applications. Examples 1 and 2 illustrate lection of a geotextile cushion using Figures 4 through 9. Example 3 depicts selection of a geotextile cushion for conditions other than those represented by the charts.

Example 1: Municipal Landfill Liner Cushion

A municipal solid waste (MSW) landfill cell is to be constructed over a carefully prepared subgrade (no significant isolated protrusions). The leachate collection media (to be placed above the geomembrane) is angular crushed stone with a maximum diameter of 38 mm (1.5 in). The maximum design height of the cell is 80 m (262.5 ft). Select an appropriate geotextile to protect the geomembrane.

Solution 1:

Using the design charts in Figures 4 or 5 select a needle-punched, nonwoven, polypropylene geotextile having a MARV M_A of at least 540 g/m^2 (16 oz/yd^2).

Example 2: Liquid Impoundment Liner Cushion

A liquid impoundment is to be constructed over a subgrade containing isolated, angular stone protrusions. The impoundment is to be lined with a geomembrane underlain by a 540 /m^2 (16.0 oz/yd^2) needle-punched, nonwoven, polypropylene geotextile for protection against e subgrade stones. No stone or other solid material will be placed on the geomembrane. Therefore, construction loading is not a concern. It is anticipated that the maximum liquid depth will be 20 m (65.6 ft). For specification purposes, determine the largest stone which may safely remain on the subgrade without damaging the geomembrane.

Solution 2:

Based on the design charts in Figures 8 or 9, stones larger than 23 mm (0.9 in) in diameter might damage the geomembrane. Thus, the construction specification could be written to require removal of all protruding subgrade stones larger than approximately 25 mm (1 in).

Example 3: Industrial Landfill Liner Cushion

A portion of the cell described in Example 1 is to be used as a monofill for automobile shredder fluff (average unit weight equal to 10.2 kN/m^3 (65 lb/ft^3)). This portion of the cell is design to be filled to a height of 25 m (82 ft). In addition, a finer 25 mm (1 in) angular, crushed stone will be used for leachate collection media. Assuming all other liner components (except the cushion) remain unchanged, select an appropriate geotextile to protect the geomembrane.

Solution 3:

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The design charts are not applicable to this problem since $\gamma \neq 12.6$ kN/m^3 (80 lb/ft^3). In addition, construction loading may control geotextile selection given the relatively shallow fill height and low unit weight of waste. Consequently, the problem must be solved by equation.

A. Determine P'_{allow} in terms of M_A, where:

H = ½ of maximum overlying particle diameter = 12.5 mm
MF_S = 1.0 (Table 1 - angular stone)
MF_{PD} = 0.5 (Table 2 - uniformly packed surface)
MF_A = 0.75 (Table 3 - moderate arching of waste materials)
FS_{CR} = 1.6 (assumed, corresponds to 270 g/m^2 - to be checked against Table 4)
FS_{CBD} = 1.2 (Table 5 - polypropylene geotextile in waste application)

$$P'_{allow} = \left(450 \cdot \frac{M_A}{12.5^2}\right)\left(\frac{1}{1.0 \cdot 0.5 \cdot 0.75}\right)\left(\frac{1}{1.6 \cdot 1.2}\right) = 4.0 \cdot M_A$$

B. Determine anticipated pressure on geomembrane, where:

γ = 10.2 kN/m^3 (given)
h = 25 m (given)

$P_{actual} = 10.2 \cdot 25 = 255 \cdot kPa$ (Equation 2)

C. Solve for minimum geotextile M_A through manipulation of Equation 3, where:

FS_{gmin} = 3.0 (Table 6 – uniform packed stones, no isolated subgrade protrusions)
P'_{allow} = $4.0 \cdot M_A$ (Equation 1)

$4.0 \cdot M_A \geq 3.0 \cdot 255$ (Equation 3) or: $M_A \geq \frac{3.0 \cdot 255}{4.0}$

Thus, $M_A \geq 191.3$ g/m^2 (5.7 oz/yd^2)

D. Check result against Creep limits established in Table 4 and Construction Limits:

From Table 4, the minimum acceptable M_A = 405 g/m^2 (12 oz/yd^2). Coincidentally, this agrees with the construction limits recommended by Richardson and Johnson (1998). Thus, select a nonwoven, needle-punched geotextile having a MARV M_A of at least 405 g/m^2. Although, FS_{CR} was selected based on a 270 g/m^2 (8 oz/yd^2) geotextile, the problem need not be reevaluated in this case since a 405 g/m^2 (12 oz/yd^2) geotextile is the minimum acceptable material based on creep limits (Table 4).

SUMMARY AND APPLICABILITY

The design charts and methodology provided herein are intended to provide a quick and nservative method to select an appropriate geomembrane cushion. Prior to applying the sign charts or method, the reader should review and understand the limitations and ssumptions discussed in the referenced literature. In circumstances where site specific conditions deviate significantly from the research forming the basis for the charts and GRI method, it is recommended that a project specific testing program be conducted and evaluated by a qualified professional. Geosynthetic materials, testing parameters, etc. should be modeled after anticipated field conditions.

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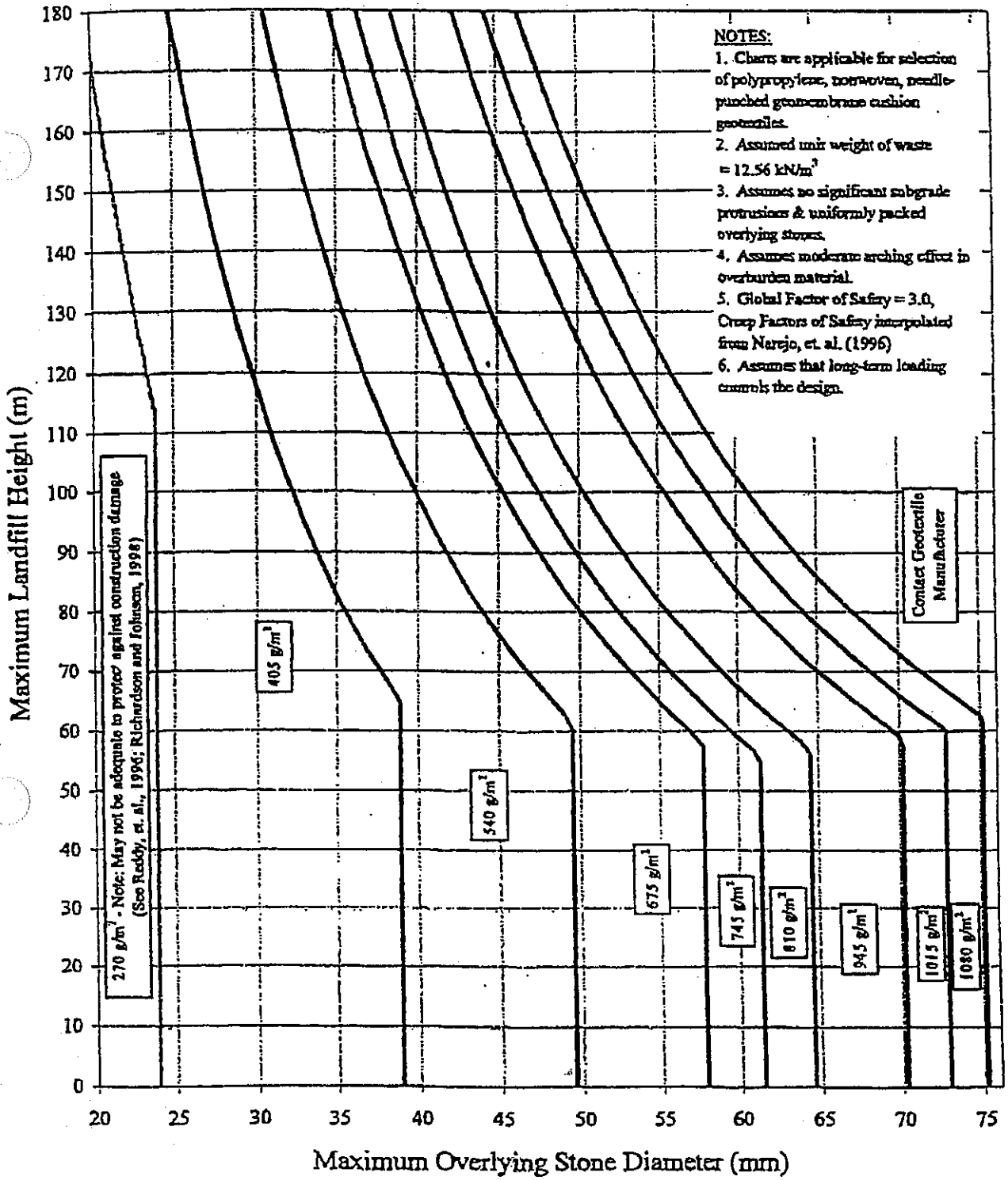

Figure 4. Geomembrane Cushion Selection Chart - Landfill Application, Rounded Overlying Stones (SI Units)

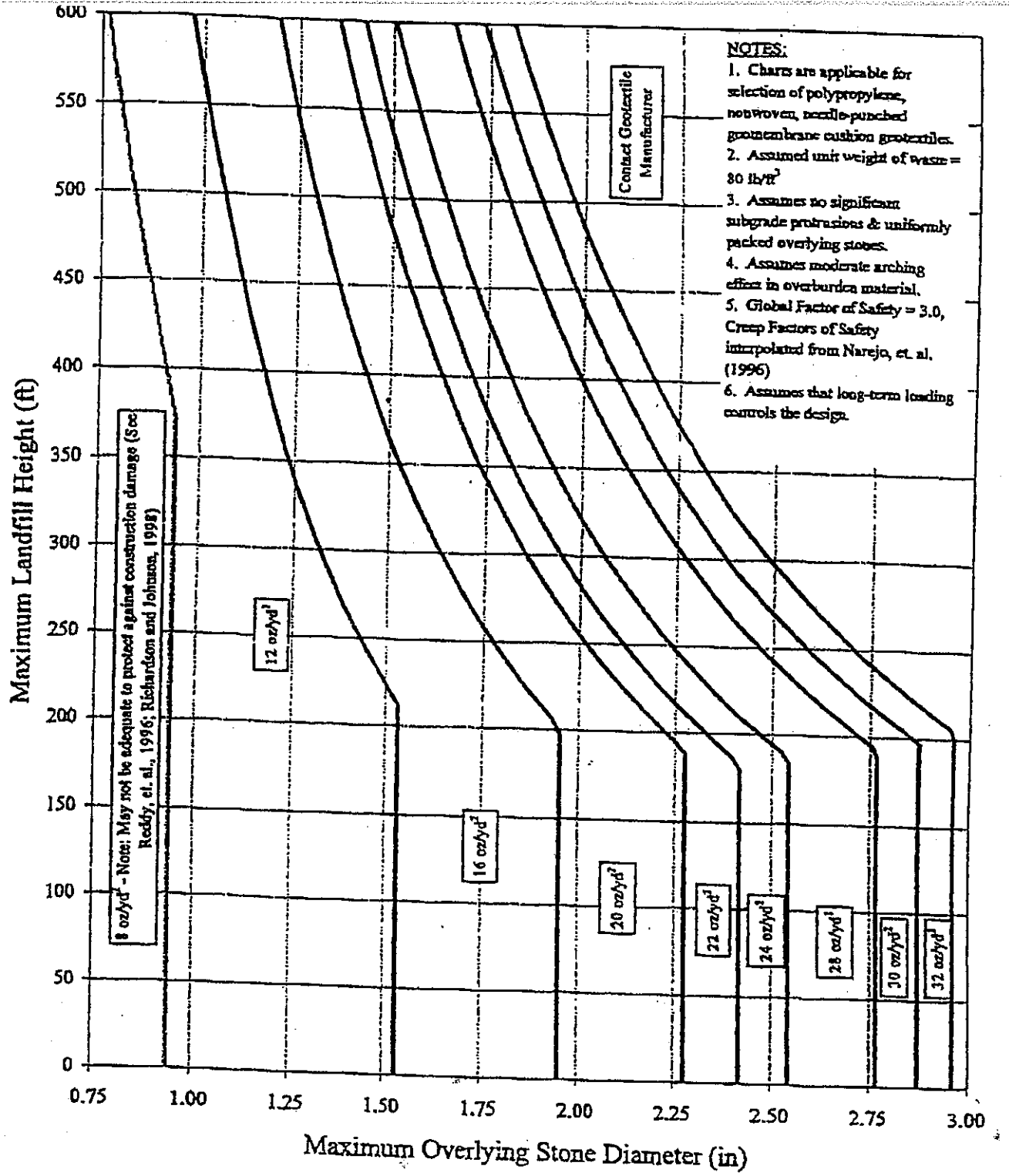

Figure 5. Geomembrane Cushion Selection Chart - Landfill Application, Rounded Overlying Stones (US Units)

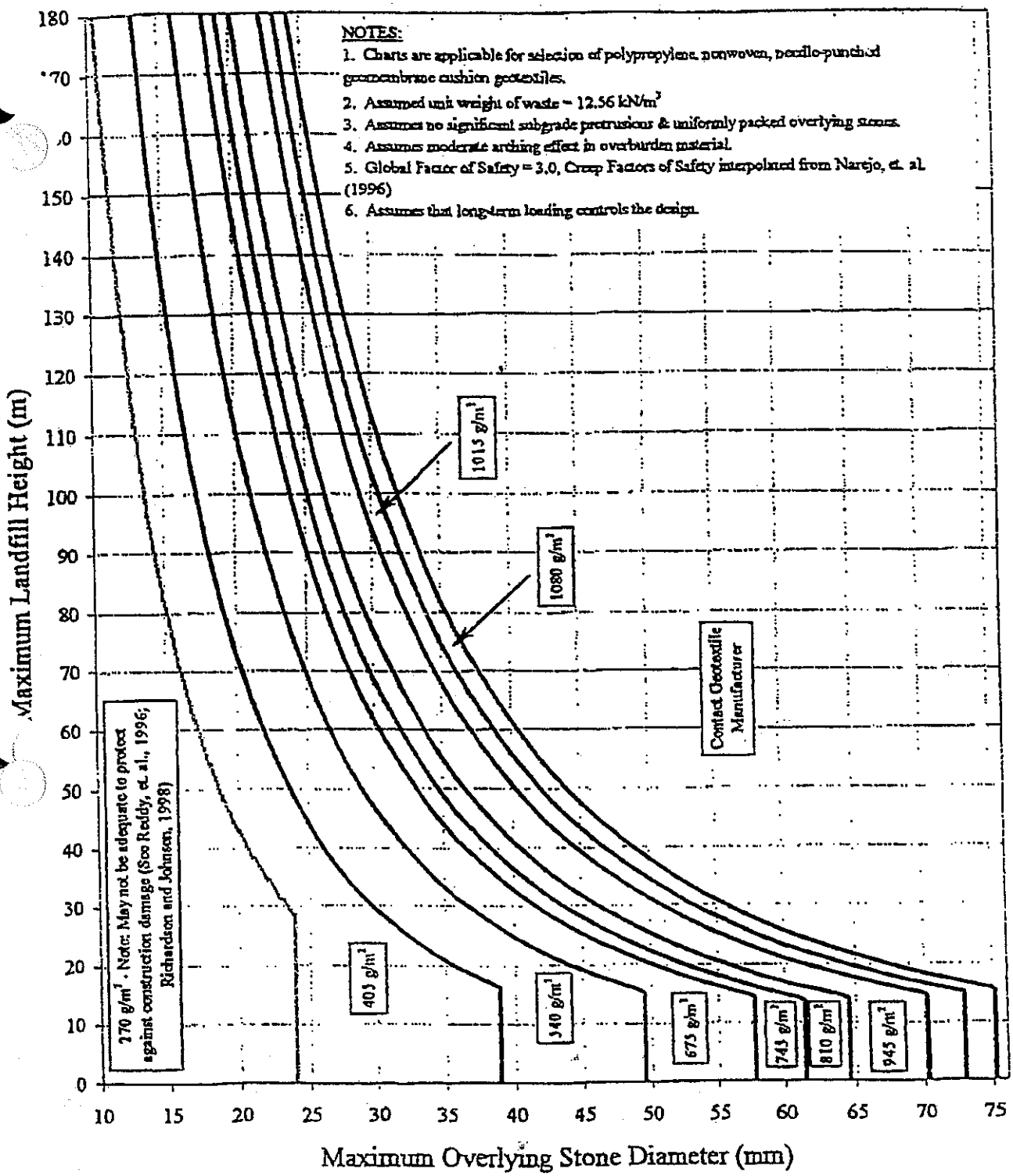

Figure 6. Geomembrane Cushion Selection Chart - Landfill Application, Angular Overlying Stones (SI Units)

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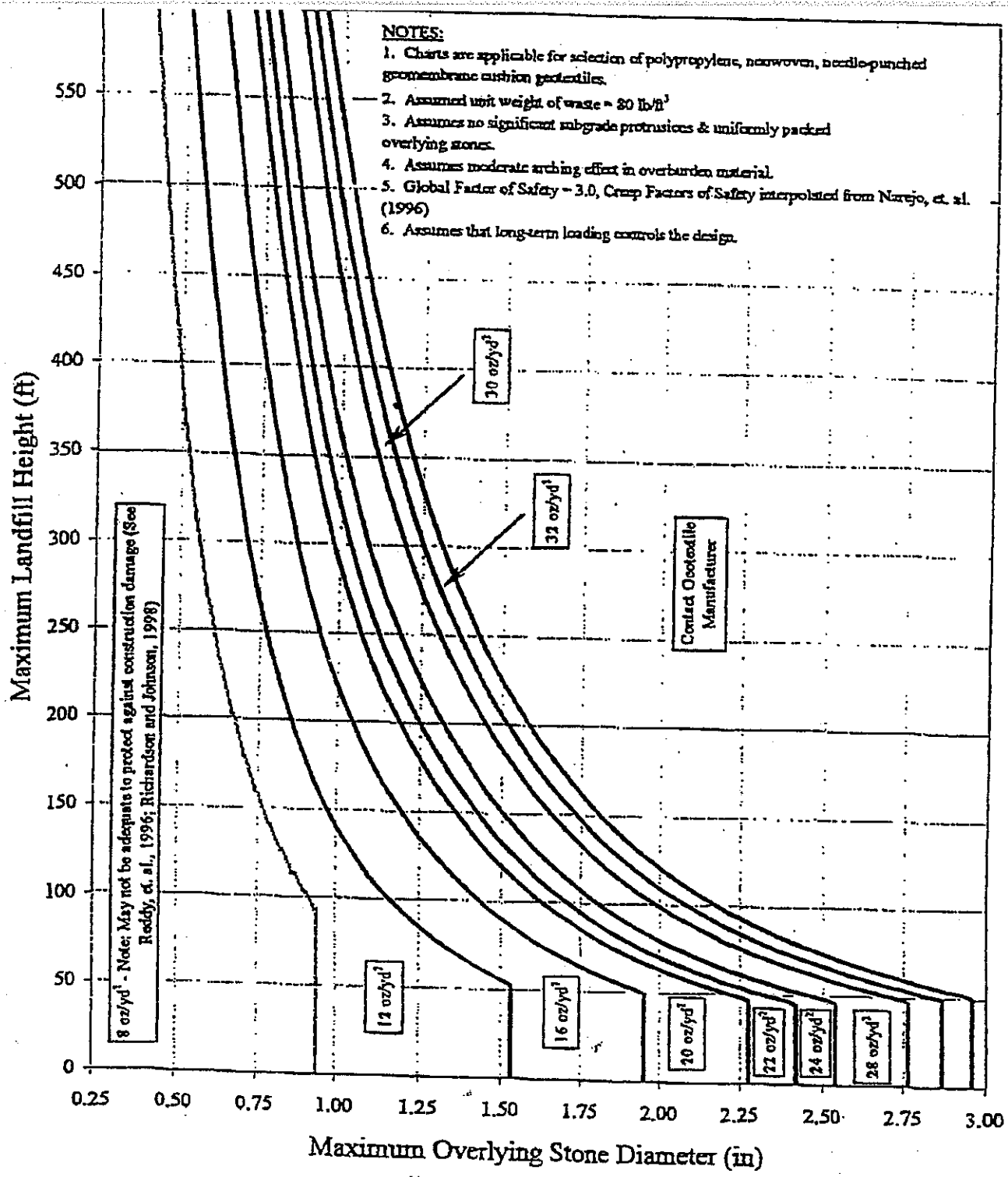

Figure 7. Geomembrane Cushion Selection Chart - Landfill Application, Angular Overlying Stones (US Units)

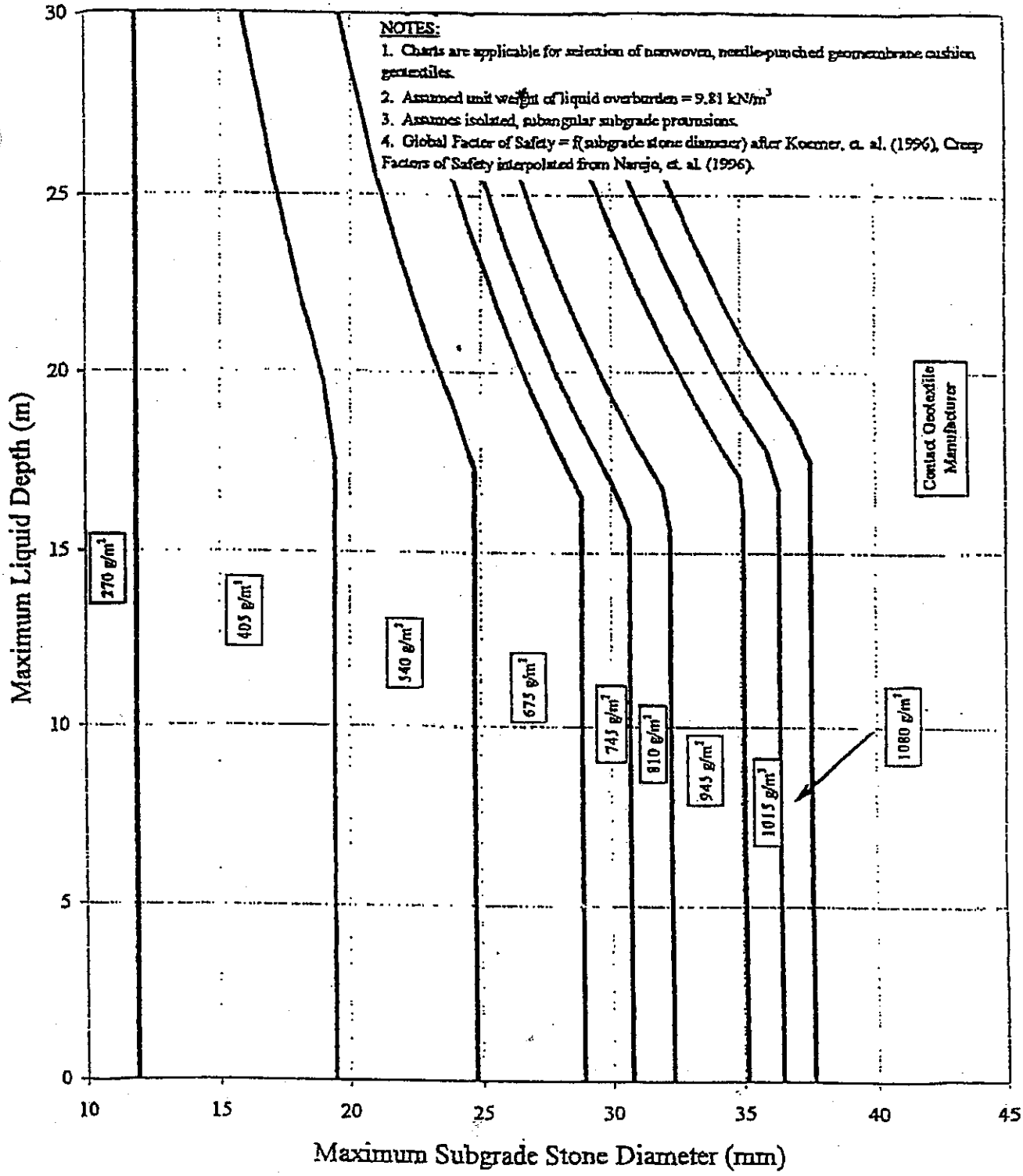

Figure 8. Geomembrane Cushion Selection Chart - Liquid Impoundment Application, Subangular Subgrade Stones (SI Units)

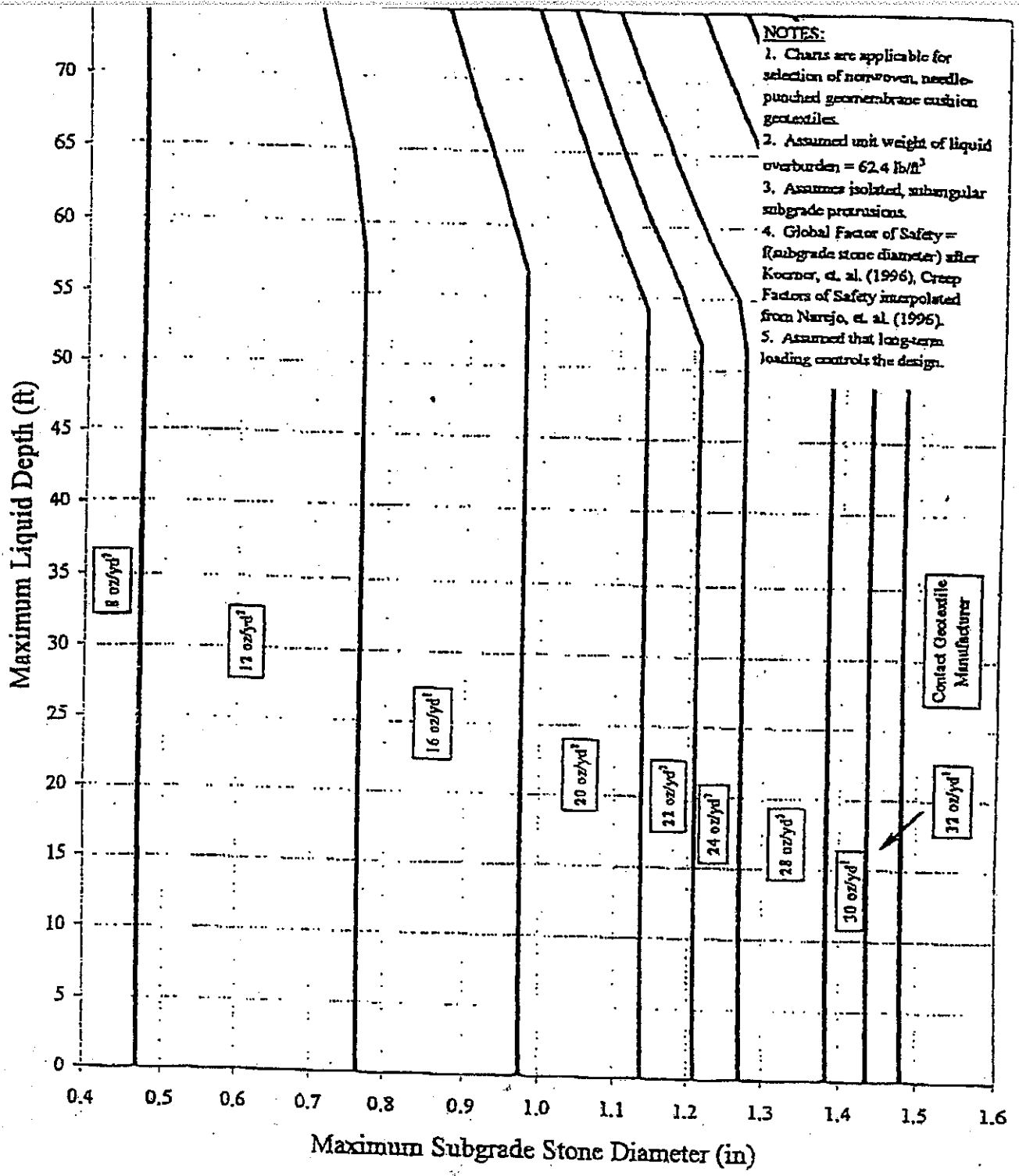

Figure 9. Geomembrane Cushion Selection Chart - Liquid Impoundment Application, Subangular Subgrade Stones (US Units)

Geosyntec consultants

COMPUTATION COVER SHEET

Client:	Denison Mines	Project:	White Mesa Mill – Cell 4B	Project/ Proposal No.:	SC0349-01
				Task No.	04

Title of Computations: **ANALYSIS OF SLIMES DRAIN**

Computations by:	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal	
Computations Checked by:	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal	
Computations backchecked by: (originator)	Signature		11/30/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/3/07
	Printed Name	Gregory T. Corcoran	Date
	Title	Principal	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

PURPOSE AND METHOD OF ANALYSIS

The purpose of this calculation package is to demonstrate that the proposed "slimes drain system" will dewater the tailings at the site within a reasonable time.

Fluid flow rate in porous media will be evaluated using Darcy's law.

ASSUMPTIONS

- This project involves the construction of a 42 acre double lined tailings cell (Cell 4B) that is approximately 42 feet deep at its deepest point and 31 feet deep at the shallowest point with an average depth of 35 feet. The liquids level in the cell will be kept a minimum of 3 feet below the top of the berm (free-board). Therefore, the maximum depth of liquid in the cell will be 39 feet at the start of dewatering.
- The cell will be filled with -28 mesh (US No. 30 sieve) tailings, largely consisting of fine sands and silts, with some clay. Results of grinding test sieve analyses, which are reported based on Tyler Mesh sieve sizes, are presented in Table 1. The grinding test data report is presented in Attachment A. Sieve to Tyler Mesh conversions are presented in Attachment B.
- The tailings will be placed within the cell in a slurry form under the surface of the free liquid contained within the cell. This placement methodology is anticipated to result in a low density (no compaction) soil structure. Therefore, saturated hydraulic conductivity and total porosity are anticipated to be higher than similar soils that are compacted.
- Based on the grinding report (Attachment A), tailings are comprised of approximately 6% medium sand, 49% fine sand, and 45% silt and clay size particles (Table 1).
- Based on the gradation of the tailings (Table 1) from the grinding report (Attachment A), the tailings would be classified as silty sand (SM) by the unified soil classification system (USCS). According to the Hydrologic Evaluation of Landfill Performance (HELP) Model Engineering Documentation (Attachment C), **low density** SM soils would exhibit saturated hydraulic

conductivities of between $1.7x10^{-3}$ cm/sec and $5.2x10^{-4}$ cm/sec and **low density** silt (ML) and sandy clay (SC) would exhibit saturated hydraulic conductivities of between $3.7x10^{-4}$ cm/sec and $1.2x10^{-4}$ cm/sec. The geomean of these two groups of soils, which are gradationally similar to the tailings, is $4.74x10^{-4}$ cm/sec (Table 2). According to Cedergren (Attachment D), under a normal stress of 2 tons per square foot (approximate normal stress on deeper tailings in the cell), medium sand, fine sand, silt, and silty clay would exhibit a saturated hydraulic conductivities of approximately $2x10^{-2}$ cm/sec, $1x10^{-2}$ cm/sec, $1x10^{-4}$ cm/sec $5x10^{-7}$ cm/sec, respectively. The geomean of these three soil types, where are gradationally similar to the tailings, is $3.31x10^{-4}$ cm/sec. The more conservative, lower hydraulic conductivity of $3.31x10^{-4}$ cm/sec, will be used in this analysis.

- Based on the gradation of the tailings from the grinding report, the tailings would be classified as silty sand (SM) by the unified soil classification system (USCS). According to the HELP Model Engineering Documentation (Attachment C), **low density** SM soils would exhibit drainable porosity of between 0.251 and 0.332 and **low density** silt (ML) and sandy clay (SC) would exhibit drainable porosity of between 0.154 and 0.231. The average of these two groups of soils, which are gradationally similar to the tailings, is 0.253 (Table 2). According to the HELP Model Engineering Documentation, medium sand, fine sand, silt, and silty clay would exhibit drainable porosity values of 0.35, 0.29, 0.14, and 0.11, respectively. The average of these three soil types, where are gradationally similar to the tailings, is 0.22. Since the average drainable porosity of 0.22 corresponds to the lower hydraulic conductivity (higher density, lower permeability, lower porosity) selected above, this value will be used in this analysis.
- The permeability of the tailings is isotropic.
- Darcy's law will be used to compute groundwater flow velocities.
- The proposed slimes drain system will consist of a series of strip drains (geotextile wrapped HDPE core, 1" thick, 12" wide, with a transmissivity of 29 (gal/min/ft), which connect to a perforated 4" diameter PVC header pipe that

is bedded in drainage aggregate and wrapped in a woven geotextile. The PVC pipe will convey the liquid to the sump for removal.

- The slimes drain spacing will be 50' and will be continuous across the base of the cell (Figure 1).

CALCULATIONS

The flow geometry for the average depth of liquid within the cell is illustrated on Figure 2 and used to compute the emptying time for the proposed slimes drain system.

Calculate the flow into a unit length of strip drain for the various hydraulic gradient conditions.

At the start of cell dewatering, the maximum depth of liquid will vary between 31 feet at the shallow end and 39 feet at the deep end, with an average depth of approximately 35 feet. As the water level drops within the cell, the length of the longest flow path and the associated hydraulic gradient will continually change with time.

The total volume to be drained by a unit length of strip, Q, can be calculated using Darcy's law as follows:

$$Q = kiA$$

where:

k = hydraulic conductivity of tailings = 3.31×10^{-4} cm/sec = 6.51×10^{-4} ft/min

i = gradient along flowpath = $\frac{dh}{dl} = \frac{35}{40.6} = 0.86$ (see Figure 2)

A = area of strip drain where flow will pass =1.17 ft^2/ft (see Figure 3)

$$Q = (6.51 \times 10^{-4} \frac{ft}{\min})(0.86)(1.17\ ft^2)$$

$$Q = 6.55x10^{-4} \frac{ft^3}{\min} \times 7.48 \frac{gal}{ft^3} = 4.9x10^{-3} \frac{gal}{\min}$$

For each one foot incremental drop in fluid elevation within the cell, the total volume to be drained by a unit length of strip drain is as follows:

V = 1 ft unit length x 1ft depth x 50 ft width x 0.022 (drainable porosity) = 11 ft^3 of free liquid

Therefore, the time to drain the first one foot of liquid within the cell can be estimated as follows:

$t = V/Q = 11\ ft^3 / 6.55x10^{-4}\ ft^3/min = 16{,}793$ minutes = 11.66 days

Tables 3, 4, and 5 depict the calculations for the maximum (39 feet), average (35 feet), and minimum (31 feet) cell liquid depth, respectively. The results of the maximum depth calculations indicate that the proposed slimes drain system will allow the tailings contained in Cell 4B to drain within approximately 5.45 years.

Calculate the design flow rate of the strip drains.

For this calculation we will assume that the strip drains have a flow rate of 29 gallon per minute per foot (Attachment E, GDE Multi-Flow, 2006), a width of 12" and that flow is occurring under a gradient of 0.01.

Design Flow rate of strip drains:

$$q = \Theta i$$

where:

q = flowrate per unit width

$$i = \frac{dh}{dl} = 0.01$$

Θ = transmissivity = 29 gpm/ft

To account for detrimental effects on the geonet such as chemical clogging, biological clogging, installation defects, and creep, partial factors of safety were used to reduce the strip drain transmissivity. Using recommended partial factor of safety values from Koerner (1999) (Attachment F, 2/4), the reduced transmissivity is calculated as follows:

$$\Theta_{allow} = \Theta_{ult}[\frac{1}{FS_{IN} \times FS_{CR} \times FS_{CC} \times FS_{BC}}]$$

where:

Θ_{allow} = allowable flow

$\Theta_{ultimate}$ = calculated value of flow

FS_{IN} = factor of safety for installation, 1.5 (CQA performed during installation)

FS_{CR} = factor of safety for creep, 2.0

FS_{CC} = factor of safety for chemical clogging, 2.0

FS_{BC} = factor of safety for biological clogging, 1.0 (low pH precludes biological activity)

The factors of safety are used to calculate the allowable transmissivity:

$$\Theta_{allow} = 29\frac{gpm}{ft}[\frac{1}{1.5 \times 2.0 \times 2.0 \times 1.0}] = 4.83\frac{gpm}{ft}$$

Using this transmissivity value, the average factor of safety for flow in the strip composite is estimated to be as follows:

$$FS = \frac{Q_D}{Q_R} = \frac{4.83\ gpm}{0.0049\ gpm} = 986 \quad \text{(Acceptable)}$$

The average allowable flow rate is much larger than the average maximum flow rate, even with the built-in partial factors of safety. Furthermore, as indicated on Tables 3, 4, and 5, the calculated flow rate within the strip drain decreases with time, which further increases the factor of safety.

Calculate the minimum required AOS and permittivity for filtration geotextile component of strip drain

The geotextile serves as a filter between the strip composite core and the tailings material. The geotextile minimizes fine particles of the tailings material from migrating into the strip composite, yet allows water to penetrate. Migration of fine particles would have the adverse effect of decreasing the transmissivity of the strip composite layer.

To be conservative in these calculations, the tailings material soil is assumed to consist of more than 20 percent clay.

The retention requirements for geotextiles can be evaluated using the chart entitled "Soil Retention Criteria for Steady-State Flow Conditions" developed by Luettich et al., (1991) (Attachment G, 1/3). This chart uses soil properties to evaluate the required apparent opening size (AOS or O_{95}) of the geotextile. Using the Soil Retention Chart, the AOS of the filter fabrics shall be:

$O_{95} < 0.21$ mm, which corresponds to sieve No. 70.

The permeability of the filter fabric must be evaluated to allow flow through the filter fabric. The following equation can be used to evaluate the minimum allowable geotextile permeability:

$k_g > i_s k_s$ (Luettich et al. (1991), Att. G, 2/3)

where: k_g = permeability of geotextile (cm/s)
i_s = hydraulic gradient (dimensionless)
k_s = permeability of the tailings material (cm/s)

Hydraulic Gradient, i: Attachment G, page 3/3 from Luettich et al. (1991) lists typical hydraulic gradients for various geotextile drainage applications. In this attachment, a hydraulic gradient of 10 for liquid impoundment applications is recommended.

Soil Permeability, k_s: A permeability of 3.31×10^{-4} cm/s was assumed for the tailings material, as previously defined.

Therefore,

$k_g > \quad i_s k_s = (10)(3.31 \times 10^{-4}\ cm/s)$
$k_g > \quad 3.31 \times 10^{-3}\ cm/s$

Koerner (1999) suggests applying partial factors of safety to the ultimate flow capacity of the geotextile to account for clogging of the geotextile. Using recommendations given in Table 2.12 on p. 150 of Koerner (1999) (Attachment F, 1/4), the following partial safety values were applied:

soil clogging and blinding:	10 (5 – 10)
creep reduction of voids:	2.0 (1.5 – 2.0)
intrusion into voids:	1.2 (1.0 – 1.2)
chemical clogging:	1.5 (1.2 – 1.5)
biological clogging (low pH precludes biological activity):	1.0 (2 – 10)

Therefore,

$k_g > \quad (3.31 \times 10^{-3})(10)(2)(1.2)(1.5)(1)$
$k_g > \quad 0.12\ cm/s$

The thickness of a typical nonwoven needled punched 4 oz/yd^2 (135 g/m^2) geotextile is approximately 40 mils (0.10 cm), see Attachment H. Dividing the permeability by the thickness of the geotextile results in a required minimum permittivity of 1.2 sec^{-1}. The geotextile used in this project has a permittivity of 2.0 sec^{-1}, which is greater than the required permittivity.

Check Pipe Flow Rate

Based on calculations from previous sections, the maximum daily flow rate to the sump is estimated to be 144 gpm (0.32 cfs) (Table 3). The capacity of the pipe is calculated based on Manning's equation for gravity flow as follows:

$$Q = \frac{1.486}{n} R_h^{2/3} S^{1/2} A$$

Where

n = 0.010 (Koerner (1999), Attachment F, 4/4)
S = Slope of liner (ft/ft) = 1.0 %

R_h = hydraulic radius, ft
Q = flow rate, cubic feet per second, ft^3/s
A = flow area, ft^2

Assuming 4-inch pipe:

$A = \pi D^2/4 = 12.6$ sq. inches $= 0.088\ ft^2$
R_h = Area ($\pi D^2/4$)/Wetted Perimeter (π D)
= D/4 = 1 in = 0.083 ft

$$Q = \frac{1.486}{0.010} 0.083^{2/3} 0.01^{1/2} 0.088\ \text{ft}^2 = 0.25\ \frac{\text{ft}^3}{s} = 112\ gpm$$

Since 112 gpm is less than the maximum required 144 gpm, this calculation shows that the 4-inch diameter slimes drain pipe is the limiting factor for dewatering the tailings in the early phase of dewatering (high flow rates). However, it does not mean that the pipe will be unable to handle this flow, but rather the pipe will require additional time to drain. The additional time needed is computed in the following section.

Effect of Maximum Pipe Capacity on Drainage Time

The maximum capacity of the pipe is 112 gpm, as computed above. Assuming the cell's total lateral length of strip drain is 27,550 feet, the flow rate, per foot of strip drain is calculated to be:

$$\text{Flow Rate} = \frac{112\ gallon}{\text{min}} * \frac{60\ \text{min}}{1\ hr} * \frac{24\ hr}{1\ day} * \frac{1 ft^3}{7.48\ gallon} * \frac{1}{29{,}977\ feet} = 0.72\ \frac{ft^3}{day}$$

The time needed to de-water first layer is:

$$Time = \frac{Volume}{Drain\ \ length\ \times flow\ rate} = \frac{(50\ x\ 1\ x\ 1\ x\ 0.22)\ ft^3}{1\ ft \times 0.72\ \frac{ft^3}{day}} = 15.27\ days$$

The difference between the maximum daily flow rate drainage time and the maximum daily flow the pipe is able to deliver for the first foot is:

15.27 day – 11.93 day (first row of Table 3) = 3.34 days.

Therefore, the first layer will require an additional 3.34 days to drain. The calculation is repeated until the pipe's allowable flow capacity of 112 gpm is equal to the maximum flow rate from the cell (Table 3). The additional drainage time needed for each layer is added to the original drainage time of 5.45 years. The results of this analysis are shown in Table 3.

The total additional drainage time occurs over the first 12 layers and adds 23 days (0.06 years) to the computed drainage time. Including the effects of the maximum pipe capacity, the cell will take an estimated 5.51 years to drain.

Effect of Precipitation on Drainage Time

To account for the effect of precipitation added to the tailings cell, the HELP Model was used to estimate the average annual leakage through a 3 foot thick (tailings above the liquid) layer of silty sand material (Attachment I). HELP Model default parameters were used along with a maximum 16 inch evaporative zone (conservative for dry climate) and weather data from Grand Junction, Colorado. The model was performed for a 10 year period and included precipitation events ranging from 5.83 to 10.36 inches per year.

The results of this analysis suggest that a maximum average annual percolation through the 3 foot soil layer above the liquid will be approximately 12 ft^3 per acre or 504 ft^3 (3,770 gal.) for the entire Cell 4B area of 42 acres.

The average flow rate during Cell 4B dewatering, as calculated from Table 3 is equal to 78 gpm (112,320 gallon/day).

The time required to drain the additional volume of precipitation in the tailing is computed using the following equation:

$$Time = \frac{Volume}{FlowRate} = \frac{3{,}770\ gal}{112{,}320\ \frac{gal}{day}} = 0.03\ days$$

The additional time that the pond will require to empty due to precipitation is insignificant.

Therefore, the estimated time to dewater Cell 4B will be 5.45 years (baseline) + 0.06 years (pipe limitations) + 0.03 years (precipitation) = 5.54 years.

REFERENCES

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(Attachment F)

Luettich, S.M., Giroud, J.P., and Bachus, R.C., (1991), "Geotextile Filter Design Manual, report prepared for Nicolon Corporation, Norcross, GA.
(Attachment G)

Amoco Fabrics and Fibers Company, (1991), "*Amoco Waste Related Geotextiles.*"
(Attachment H)

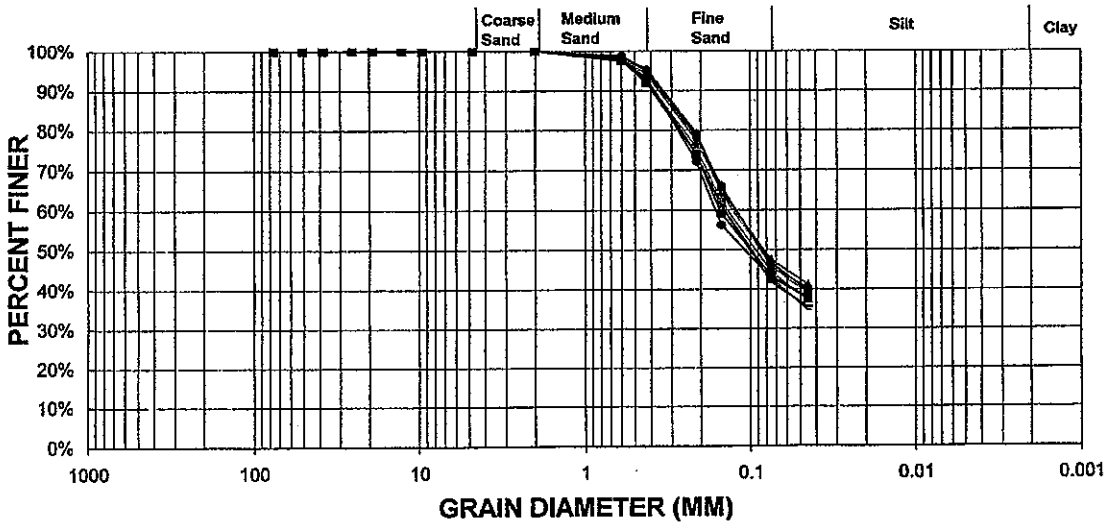
Table 1
DSM Screen Undersize Gradation

SIEVE ANALYSIS

		Grinding Test 1			Grinding Test 2A			Grinding Test 2B			Grinding Test 3A			Grinding Test 3B		
Sieve No.	Diameter (mm)	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% 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	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

		Grinding Test 6A			Grinding Test 6B		
Sieve No.	Diameter (mm)	Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer
3 in.	76.2	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%
1 1/2 in.	38.1	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%
3/4 in.	19.1	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%
3/8 in.	9.530	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%
No. 10	2.000	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%
No. 40	0.425	5.2	5.2%	94.8%	4.7	4.7%	95.3%
No. 70	0.212	21.7	21.7%	78.3%	21.4	21.4%	78.6%
No. 100	0.150	34.1	34.1%	65.9%	35.9	35.9%	64.1%
No. 200	0.075	54.4	54.4%	45.6%	54.4	54.4%	45.6%
No. 325	0.045	59.7	59.7%	40.3%	61.1	61.1%	38.9%
Pan	-	-	-	-	-	-	-

Grinding Test 4A			Grinding Test 4B		
Wt. Retained (grams)	% Retained	% Finer	Wt. Retained (grams)	% Retained	% Finer
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%	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%
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%	0.0	0.0%	100.0%
0.0	0.0%	100.0%	0.0	0.0%	100.0%
2.7	2.7%	97.3%	2.7	2.7%	97.3%
7.6	7.6%	92.4%	7.3	7.3%	92.7%
26.2	26.2%	73.8%	25.9	25.9%	74.1%
38.7	38.7%	61.3%	39.2	39.2%	60.8%
57.3	57.3%	42.7%	58.3	58.3%	41.7%
65.4	65.4%	34.6%	64.6	64.6%	35.4%
-	-	-	-	-	-



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

%FINER = 100 - Σ%RETAINED

5/12/2007

Table 2
Tailings Parameters

Soil	Permeability[1] (cm/sec)	Drainable Porosity[2] (vol./vol.)
med sand	2.00E-02	0.35
fine sand	1.00E-02	0.29
silt	1.00E-04	0.14
silty clay	6.00E-07	0.11
average	7.53E-03	0.22
geomean	3.31E-04	0.20

Soil	Permeability[3] (cm/sec)	Drainable Porosity[3] (vol./vol.)
SM (LS)	1.70E-03	0.332
SM (LFS)	1.00E-03	0.326
SM (SL)	7.20E-04	0.263
SM (FSL)	5.20E-04	0.251
ML (L)	3.70E-04	0.231
ML (SiL)	1.90E-04	0.217
SC (SCL)	1.20E-04	0.154
average	6.60E-04	0.253
geomean	4.74E-04	0.246

Notes:

(1) Source - "Seepage, Drainage, and Flow Nets", Cedergren, H. R., 1989.

(2) Source - The Hydrologic Evaluation of Landfill Performance (HELP) Model, Version 3, EPA, 1994 - Figure 2 - Soil texture vs. Moisture Retention.

(3) Source - The Hydrologic Evaluation of Landfill Performance (HELP) Model, Version 3, EPA, 1994 - Table 1 - Low Density Soil Characteristics.

TABLE 3
White Mesa Mill
Cell 4B Slimes Drain
Maximum Liquid Depth

Permeability (cm/sec)	Permeability (ft/min)	Drainage Path Length (ft.)	Thickness (VF)	Q (cfm/ft)	Volume of Liquid (CF/ft)	Time to Dewater (min/VF/ft)	Time to Dewater (days/VF/ft)	Total Flow Rate (gpm)	Volume Removed (gal)	Pipe Limitation (days)
3.31E-04	6.51E-04	46.3	39	6.40E-04	11	17,185	11.93	143.54	2,466,685	3.34
3.31E-04	6.51E-04	45.8	38	6.31E-04	11	17,446	12.12	141.39	2,466,685	3.15
3.31E-04	6.51E-04	45.4	37	6.19E-04	11	17,761	12.33	138.88	2,466,685	2.94
3.31E-04	6.51E-04	45.0	36	6.08E-04	11	18,094	12.57	136.33	2,466,685	2.70
3.31E-04	6.51E-04	44.6	35	5.96E-04	11	18,446	12.81	133.73	2,466,685	2.46
3.31E-04	6.51E-04	44.2	34	5.85E-04	11	18,818	13.07	131.08	2,466,685	2.20
3.31E-04	6.51E-04	43.8	33	5.73E-04	11	19,213	13.34	128.39	2,466,685	1.93
3.31E-04	6.51E-04	43.5	32	5.59E-04	11	19,677	13.66	125.36	2,466,685	1.61
3.31E-04	6.51E-04	43.2	31	5.45E-04	11	20,172	14.01	122.28	2,466,685	1.26
3.31E-04	6.51E-04	43.0	30	5.30E-04	11	20,748	14.41	118.89	2,466,685	0.86
3.31E-04	6.51E-04	42.8	29	5.15E-04	11	21,363	14.84	115.46	2,466,685	0.43
3.31E-04	6.51E-04	42.6	28	4.99E-04	11	22,023	15.29	112.00	2,466,685	(0.02)
3.31E-04	6.51E-04	42.4	27	4.84E-04	11	22,731	15.79	108.51	2,466,685	
3.31E-04	6.51E-04	42.3	26	4.67E-04	11	23,550	16.35	104.74	2,466,685	
3.31E-04	6.51E-04	42.2	25	4.50E-04	11	24,434	16.97	100.95	2,466,685	
3.31E-04	6.51E-04	42.1	24	4.33E-04	11	25,392	17.63	97.14	2,466,685	
3.31E-04	6.51E-04	42.1	23	4.15E-04	11	26,496	18.40	93.10	2,466,685	
3.31E-04	6.51E-04	42.1	22	3.97E-04	11	27,700	19.24	89.05	2,466,685	
3.31E-04	6.51E-04	42.1	21	3.79E-04	11	29,019	20.15	85.00	2,466,685	
3.31E-04	6.51E-04	42.2	20	3.60E-04	11	30,543	21.21	80.76	2,466,685	
3.31E-04	6.51E-04	42.3	19	3.41E-04	11	32,226	22.38	76.54	2,466,685	
3.31E-04	6.51E-04	42.5	18	3.22E-04	11	34,178	23.73	72.17	2,466,685	
3.31E-04	6.51E-04	42.6	17	3.03E-04	11	36,273	25.19	68.00	2,466,685	
3.31E-04	6.51E-04	42.8	16	2.84E-04	11	38,721	26.89	63.70	2,466,685	
3.31E-04	6.51E-04	43.1	15	2.64E-04	11	41,592	28.88	59.31	2,466,685	
3.31E-04	6.51E-04	43.3	14	2.46E-04	11	44,770	31.09	55.10	2,466,685	
3.31E-04	6.51E-04	43.6	13	2.27E-04	11	48,548	33.71	50.81	2,466,685	
3.31E-04	6.51E-04	44.0	12	2.07E-04	11	53,076	36.86	46.47	2,466,685	
3.31E-04	6.51E-04	44.3	11	1.89E-04	11	58,296	40.48	42.31	2,466,685	
3.31E-04	6.51E-04	44.7	10	1.70E-04	11	64,704	44.93	38.12	2,466,685	
3.31E-04	6.51E-04	45.1	9	1.52E-04	11	72,537	50.37	34.01	2,466,685	
3.31E-04	6.51E-04	45.6	8	1.33E-04	11	82,509	57.30	29.90	2,466,685	
3.31E-04	6.51E-04	46.0	7	1.16E-04	11	95,123	66.06	25.93	2,466,685	
3.31E-04	6.51E-04	46.5	6	9.81E-05	11	112,183	77.90	21.99	2,466,685	
3.31E-04	6.51E-04	47.1	5	8.07E-05	11	136,357	94.69	18.09	2,466,685	
3.31E-04	6.51E-04	47.6	4	6.39E-05	11	172,255	119.62	14.32	2,466,685	
3.31E-04	6.51E-04	48.2	3	4.73E-05	11	232,569	161.51	10.61	2,466,685	
3.31E-04	6.51E-04	48.8	2	3.11E-05	11	353,196	245.27	6.98	2,466,685	
3.31E-04	6.51E-04	49.4	1	1.54E-05	11	715,076	496.58	3.45	2,466,685	
						days	1,989.58		96,200,703	22.86
						years	5.45			

Average Soil Porosity	0.22	
Geomean Soil Permeability	3.31E-04	cm/sec
Distance Between Drains	50	ft
Thickness of Unit	1	ft
Maximum Depth	39	ft
Length of Strip Drain	29,977	ft

TABLE 4
White Mesa Mill
Cell 4B Slimes Drain
Average Liquid Depth

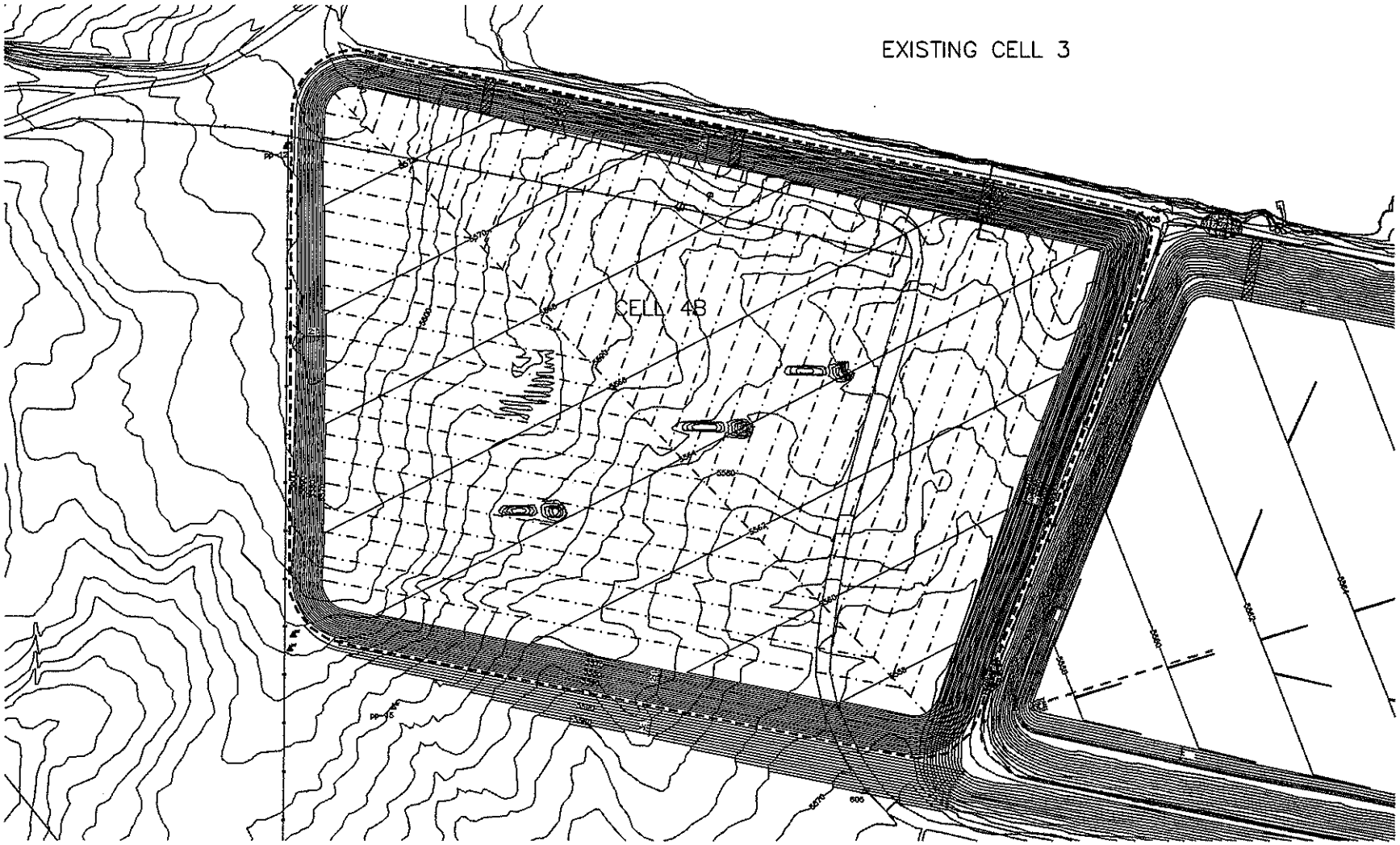
Permeability (cm/sec)	Permeability (ft/min)	Drainage Path Length (ft.)	Thickness (VF)	Q (cfm/ft)	Volume of Liquid (CF/ft)	Time to Dewater (min/VF/ft)	Time to Dewater (days/VF/ft)	Total Flow Rate (gpm)	Volume Removed (gal)
3.31E-04	6.51E-04	43.0	35	6.19E-04	11	17,784	12.35	138.70	2,466,685
3.31E-04	6.51E-04	42.6	34	6.07E-04	11	18,137	12.59	136.01	2,466,685
3.31E-04	6.51E-04	42.3	33	5.93E-04	11	18,555	12.89	132.94	2,466,685
3.31E-04	6.51E-04	42.0	32	5.79E-04	11	18,999	13.19	129.83	2,466,685
3.31E-04	6.51E-04	41.7	31	5.65E-04	11	19,472	13.52	126.68	2,466,685
3.31E-04	6.51E-04	41.4	30	5.51E-04	11	19,976	13.87	123.48	2,466,685
3.31E-04	6.51E-04	41.2	29	5.35E-04	11	20,565	14.28	119.95	2,466,685
3.31E-04	6.51E-04	41.0	28	5.19E-04	11	21,196	14.72	116.38	2,466,685
3.31E-04	6.51E-04	40.9	27	5.02E-04	11	21,927	15.23	112.49	2,466,685
3.31E-04	6.51E-04	40.8	26	4.84E-04	11	22,715	15.77	108.59	2,466,685
3.31E-04	6.51E-04	40.7	25	4.67E-04	11	23,566	16.37	104.67	2,466,685
3.31E-04	6.51E-04	40.7	24	4.48E-04	11	24,548	17.05	100.49	2,466,685
3.31E-04	6.51E-04	40.7	23	4.29E-04	11	25,615	17.79	96.30	2,466,685
3.31E-04	6.51E-04	40.7	22	4.11E-04	11	26,779	18.60	92.11	2,466,685
3.31E-04	6.51E-04	40.8	21	3.91E-04	11	28,123	19.53	87.71	2,466,685
3.31E-04	6.51E-04	40.9	20	3.72E-04	11	29,602	20.56	83.33	2,466,685
3.31E-04	6.51E-04	41.1	19	3.51E-04	11	31,312	21.74	78.78	2,466,685
3.31E-04	6.51E-04	41.3	18	3.31E-04	11	33,213	23.06	74.27	2,466,685
3.31E-04	6.51E-04	41.5	17	3.11E-04	11	35,337	24.54	69.81	2,466,685
3.31E-04	6.51E-04	41.8	16	2.91E-04	11	37,817	26.26	65.23	2,466,685
3.31E-04	6.51E-04	42.1	15	2.71E-04	11	40,627	28.21	60.72	2,466,685
3.31E-04	6.51E-04	42.4	14	2.51E-04	11	43,839	30.44	56.27	2,466,685
3.31E-04	6.51E-04	42.7	13	2.31E-04	11	47,546	33.02	51.88	2,466,685
3.31E-04	6.51E-04	43.1	12	2.12E-04	11	51,990	36.10	47.45	2,466,685
3.31E-04	6.51E-04	43.6	11	1.92E-04	11	57,375	39.84	42.99	2,466,685
3.31E-04	6.51E-04	44.0	10	1.73E-04	11	63,691	44.23	38.73	2,466,685
3.31E-04	6.51E-04	44.5	9	1.54E-04	11	71,572	49.70	34.46	2,466,685
3.31E-04	6.51E-04	45.0	8	1.35E-04	11	81,423	56.54	30.29	2,466,685
3.31E-04	6.51E-04	45.5	7	1.17E-04	11	94,089	65.34	26.22	2,466,685
3.31E-04	6.51E-04	46.1	6	9.89E-05	11	111,218	77.23	22.18	2,466,685
3.31E-04	6.51E-04	46.7	5	8.14E-05	11	135,199	93.89	18.24	2,466,685
						days	898.47		76,467,226
						years	2.46		

Average Soil Porosity	0.22	
Geomean Soil Permeability	3.31E-04	cm/sec
Distance Between Drains	50	ft
Thickness of Unit	1	ft
Average Depth	35	ft
Length of Strip Drain	29,977	ft

TABLE 5
White Mesa Mill
Cell 4B Slimes Drain
Minimum Liquid Depth

Permeability (cm/sec)	Permeability (ft/min)	Drainage Path Length (ft.)	Thickness (VF)	Q (cfm/ft)	Volume of Liquid (CF/ft)	Time to Dewater (min/VF/ft)	Time to Dewater (days/VF/ft)	Total Flow Rate (gpm)	Volume Removed (gal)
3.31E-04	6.51E-04	39.8	31	5.92E-04	11	18,584	12.91	132.73	2,466,685
3.31E-04	6.51E-04	39.6	30	5.76E-04	11	19,107	13.27	129.10	2,466,685
3.31E-04	6.51E-04	39.4	29	5.59E-04	11	19,666	13.66	125.43	2,466,685
3.31E-04	6.51E-04	39.2	28	5.43E-04	11	20,265	14.07	121.72	2,466,685
3.31E-04	6.51E-04	39.1	27	5.25E-04	11	20,962	14.56	117.67	2,466,685
3.31E-04	6.51E-04	39.0	26	5.07E-04	11	21,713	15.08	113.60	2,466,685
3.31E-04	6.51E-04	38.9	25	4.88E-04	11	22,523	15.64	109.52	2,466,685
3.31E-04	6.51E-04	38.9	24	4.69E-04	11	23,462	16.29	105.14	2,466,685
3.31E-04	6.51E-04	39.0	23	4.48E-04	11	24,545	17.05	100.50	2,466,685
3.31E-04	6.51E-04	39.0	22	4.29E-04	11	25,661	17.82	96.13	2,466,685
3.31E-04	6.51E-04	39.2	21	4.07E-04	11	27,020	18.76	91.29	2,466,685
3.31E-04	6.51E-04	39.3	20	3.87E-04	11	28,444	19.75	86.72	2,466,685
3.31E-04	6.51E-04	39.5	19	3.66E-04	11	30,093	20.90	81.97	2,466,685
3.31E-04	6.51E-04	39.8	18	3.44E-04	11	32,006	22.23	77.07	2,466,685
3.31E-04	6.51E-04	40.1	17	3.22E-04	11	34,145	23.71	72.24	2,466,685
3.31E-04	6.51E-04	40.4	16	3.01E-04	11	36,550	25.38	67.49	2,466,685
3.31E-04	6.51E-04	40.8	15	2.79E-04	11	39,373	27.34	62.65	2,466,685
3.31E-04	6.51E-04	41.2	14	2.58E-04	11	42,599	29.58	57.91	2,466,685
3.31E-04	6.51E-04	41.6	13	2.37E-04	11	46,321	32.17	53.25	2,466,685
3.31E-04	6.51E-04	42.1	12	2.17E-04	11	50,784	35.27	48.57	2,466,685
3.31E-04	6.51E-04	42.6	11	1.96E-04	11	56,059	38.93	44.00	2,466,685
3.31E-04	6.51E-04	43.1	10	1.76E-04	11	62,388	43.33	39.54	2,466,685
3.31E-04	6.51E-04	43.7	9	1.57E-04	11	70,285	48.81	35.10	2,466,685
						days	536.50		56,733,748
						years	1.47		

Average Soil Porosity	0.22	
Geomean Soil Permeability	3.31E-04	cm/sec
Distance Between Drains	50	ft
Thickness of Unit	1	ft
Maximum Depth	31	ft
Length of Strip Drain	29,977	ft



300 150 0 300

SCALE IN FEET

SLIMES DRAIN LAYOUT
CELL 4B
BLANDING, UTAH

Geosyntec consultants

DATE: SEPTEMBER 2007

PROJECT NO. SC0349

FIGURE 1

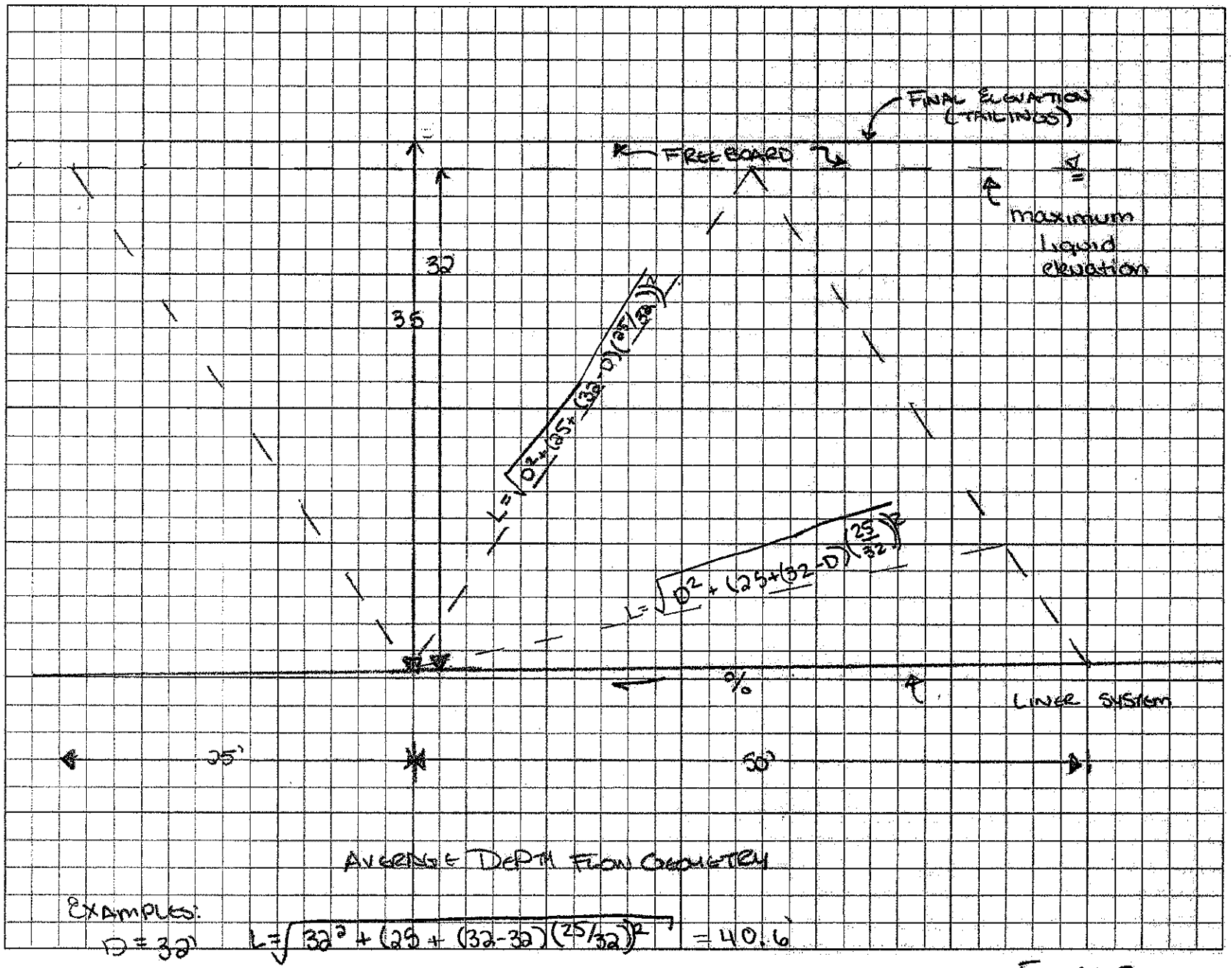
P:\PRJ\SDCadd\CADD\SC0349\PlanSet_Cell 4b\0349C004.dwg

Geosyntec consultants

Page ___ of ___

Written by: ___ Date: DD / MM / YY ___ Reviewed by: ___ Date: DD / MM / YY ___

Client: ___ Project: ___ Project/Proposal No. ___ Task No. ___



EXAMPLES:

$D=32'$ $L=\sqrt{32^2+(25+(32-32)(25/32))^2}=40.6'$

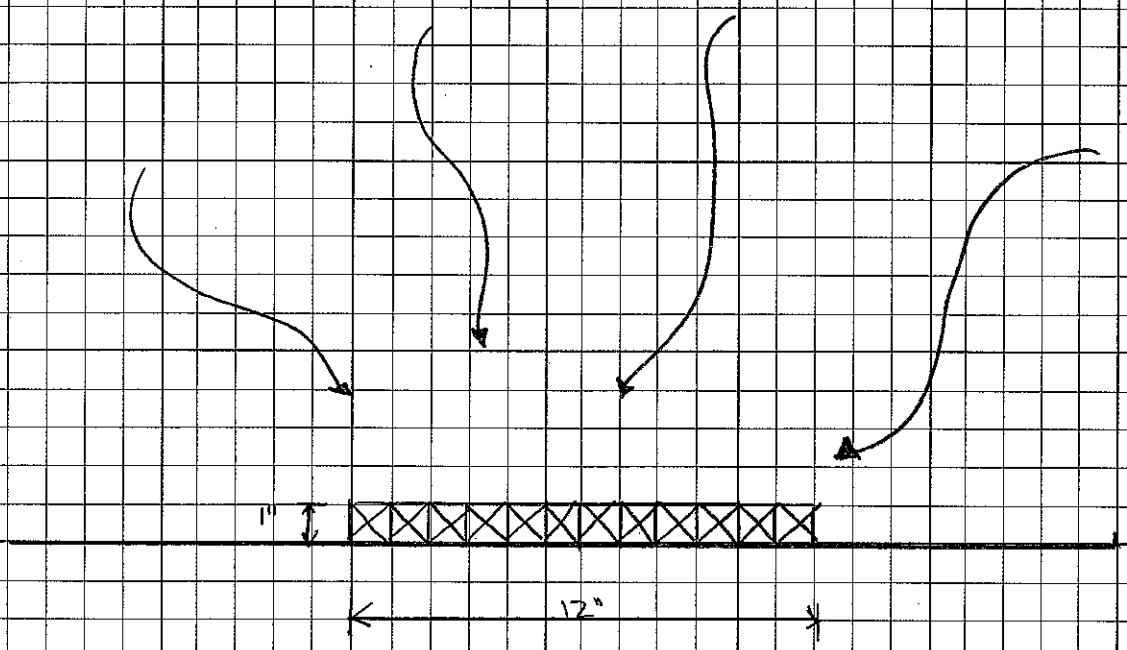
FIGURE 2

$D=10'$ $L=\sqrt{10^2+(25+(32-10)(25/32))^2}=43.4'$

Geosyntec consultants

Written by: RF Date: 30/08/07 (DD MM YY) Reviewed by: _____ Date: __/__/__ (DD MM YY)

Client: DMC Project: WMM-Cell4B Project/Proposal No. SC0349 Task No. _____



FLOW AREA - STRIP COMPOSITE

$$A = \frac{14''}{12''/1} = 1.17 \text{ ft}^2/\text{ft}$$

FIGURE 3

EXHIBIT 1

SAMPLE DESCRIPTION AND PREPARATION

CSMRI Sample 1

Sponsor's Designation of Sample: Run-of-mine.

Date Received at Institute: June 5, 1978.

Sample Weight: 100,520 lb.

Sample Container: Two truckloads.

Sample Description: Mine ore -- estimate 5% +10-in. material. Largest boulder -- 48 in. x 24 in. x 14 in. Only two or three rocks were greater than 36 in.

Method of Preparation: All +10-in. material broken to -10 in. by sledgehammer and jackhammer. The sample was screened at 6 in. and 1-1/2 in. with the +6 in. fraction, put in barrels, and the -1/2 in. fraction piled. The -6 in. +1-1/2 in. material was screened at 4 in. and 1-1/2 in. with the -6 in. +4 in. and -4 in. +1-1/2 in. fractions barreled. The additional -1-1/2 in. fraction was piled with the previous -1-1/2 in. fraction. A screen size analysis of the entire quantity of mill feed material is presented in Exhibit 3. A summary screen size analysis of the ore is as follows:

Screen Product in.		Weight %
Head (calculated)		100.00
-10	+6	2.92
-6	+4	9.48
-4	+1-1/2	15.30
-1-1/2		72.30

EXHIBIT 1

CSMRI Sample 2

Sponsor's Designation of Sample:	Crushed ore.
Date Received at Institute:	June 5, 1978.
Sample Weight:	47,380 lb.
Sample Container:	One truckload.
Sample Description:	Ore previously crushed to -3 in., maximum particles approximately 2-1/2 in.
Method of Preparation:	The ore was used as received.

EXHIBIT 2

GRINDING TESTS

Grinding Test 1, Autogenous

Date:	June 13, 1978
Feed Rate, stph:	2
Ore:	Run-of-mine
DSM Screen, in. width:	12
DSM Screen Opening, mm:	1.27
Measured Mill Power Tare (empty mill), kw:	2.06
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Meter Reading watt-hr	Mill-Bearing Oil Temp. °F	Ore Feed Rate (as received)(1) -1-1/2 in. lb/hr	-4 in. +1-1/2 in. lb/hr	-6 in. +4 in. lb/hr	-10 in. +6 in. lb/hr	Mill Discharge Solids %	Mill Discharge Solids lb/hr	Sweco Screen Oversize Solids %	Sweco Screen Oversize Solids lb/hr	DSM Screen Overflow Solids %	DSM Screen Overflow Solids lb/hr	DSM Screen Underflow Solids %	DSM Screen Underflow Solids lb/hr	Mill Water Meter %	Mill Water Rate lb/hr	Mill Load Volume %	Remarks
0910	0	--	--	104	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Start mill.
0915	5	12.2	12,964	--	3,150	612	380	116	63	8,335	--	--	--	--	--	--	90	2,858	--	--
1005	55	8.7	--	--	2,880	612	380	116	62	--	90	506	60	3,348	57	2,616(2)	90	2,858	--	--
1030	80	6.8	--	105	2,835	612	380	116	69	--	90	304	70	3,591	58	710(2)	90	2,858	--	--
1100	110	6.5	12,977	106	2,993	612	380	116	66	--	--	--	69	4,223	58	679(2)	80	2,540	--	--
1135	145	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Mill down, elevator plugged.
1142	145	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Start mill.
1150	153	6.2	--	109	2,993	612	380	116	69	12,420	90	1,114	70	5,544	56	2,583	--	--	--	--
1230	193	6.0	12,988	111	2,903	612	380	116	64	10,829	90	405	69	6,955	60	4,388	75	2,382	--	--
1300	223	6.2	--	112	3,319	612	380	116	65	11,232	90	365	70	6,048	60	3,861	81	2,572	--	--
1345	238	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Pump plugged. DSM feed.
1400	253	6.4	--	112	3,128	612	380	116	65	11,700	90	122	69	3,229	60	3,996	80	2,540	--	Sample
1415	268	6.3	13,004	112	2,970	612	380	116	65	9,945	90	547	71	3,515	59	2,907	79	2,509	15	Sample.
Average					3,019	612	380	116	65	10,744	90	480	69	4,557	59	3,547	83	2,640		

(1) Moisture: -1-1/2 in., 2.8%; -4 in. +1-1/2 in., 1.0%; -6 in. +4 in., 0.8%; -10 in. +6 in., 0.7%. Average dry ore feed rate: -1-1/2 in., 2,934.5 lb/hr; -4 in. +1-1/2 in., 605.9 lb/hr; -6 in. +4 in., 376.8 lb/hr; -10 in. +6 in., 115.0 lb/hr; total, 4,032.2 lb/hr, 2.016 dry stph. Mill volume end of test: 15%.
(2) Excluded from average.

Feed Rate, stph dry:	2.016
Ball Charge:	None
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Instantaneous Gross Power (meter reading) kwhr	Instantaneous Corrected Power (from input-output curve) kwhr	Power Consumption Gross kwhr/st	Power Consumption Net kwhr/st	Circulating Load Weight % of Feed(1)	Mill Discharge Solids %	Remarks
0910	0	--	--	--	--	--	--	--	--
0915	5	12.2	4.25	2.64	1.31	1.01	--	63	--
1005	55	8.7	5.96	4.25	2.11	1.81	--	62	--
1030	80	6.8	7.62	5.80	2.88	2.58	--	69	--
1100	110	6.5	7.97	6.10	3.03	2.73	--	66	--
1135	145	--	--	--	--	--	--	--	Unplug bucket elevator.
1150	153	6.2	8.36	6.47	3.21	2.91(2)	162.0	69	--
1230	193	6.0	8.64	6.73	3.34	3.04(2)	183.0	64	--
1300	223	6.2	8.36	6.47	3.21	2.91(2)	145.0	65	--
1345	238	--	--	--	--	--	--	--	Unplug DSM feed pump.
1400(3)	253	6.4	8.10	6.23	2.09	2.79(2)	79.0	65	--
1415(3)	268	6.3	8.23	6.35	3.15	2.85(2)	100.0	65	--
Average						2.90	133.8		

(1) Calculated: Sum of Sweco oversize and DSM oversize as percentage of dry mill feed.
(2) Average for power (last five readings): 2.90 kwhr/st.
(3) Sample run.

EXHIBIT 2

Grinding Test 1 -- continued

Procedure: Sample was wet screened on a 325M screen, products dried, and the +325M material dry screened using a Ro-Tap for 30 min.

Test Product	Screen Size Analysis DSM Screen Undersize
Sample Time:	1415
Sample Weight, g:	4,630.5
Screen Product (Tyler) Mesh	**Weight %**
Head (calculated)	100.0
+28	1.2
-28 +35	3.4
-35 +65	16.2
-65 +100	14.0
-100 +200	18.6
-200 +325	7.1
-325	39.5

US SIEVE

GTC 5/10/07

No. 30
No. 40
No. 70
No. 100
No. 200
No. 325

EXHIBIT 2

Grinding Test 2

Date:	June 14, 1978
Feed Rate, stph:	2.0
Ore:	Run-of-mine
Ball Charge:	Total: 301.8 lb; 2% mill volume
-1-1/2 in. +1 in. Balls, lb:	114.5
-2 in. +1-1/2 in. Balls, lb:	151.3
3 in. Balls, lb:	36.0
DSM Screen, in. width:	12
DSM Screen Openings, mm:	1.27
Measured Mill Power Tare (empty mill), kw:	2.06
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Meter Reading watt-hr	Mill-Bearing Oil Temp. °F	Ore Feed Rate (as received)(1) -1-1/2 in. lb/hr	-4 in. +1-1/2 in. lb/hr	-6 in. +4 in. lb/hr	-10 in. +6 in. lb/hr	Mill Discharge Solids %	Mill Discharge Solids lb/hr	Sweco Screen Oversize Solids %	Sweco Screen Oversize Solids lb/hr	DSM Screen Overflow Solids %	DSM Screen Overflow Solids lb/hr	DSM Screen Underflow Solids %	DSM Screen Underflow Solids lb/hr	Mill Water Meter %	Mill Water Rate lb/hr	Mill Load Volume %	Remarks
1040	0	8.7	13,004	102	--	612	380	116	--	--	--	--	--	--	--	--	95	3,017	-	Start mill.
1110	30	5.2	--	104	--	612	380	116	--	--	--	--	--	--	--	--	83	2,636	-	--
1130	50	5.3	--	106	3,060	612	380	116	62	8,147	50	248	74	1,565	54	2,989(2)	84	2,668	-	--
1200	80	5.0	--	108	2,846	612	380	116	63	6,577	67	653	71	1,150	--	--	82	2,604	-	--
1230	110	4.8	13,023	111	3,105	612	380	116	64	8,467	64	605	73	1,281	--	--	82	2,604	-	--
1300	140	4.8	--	112	3,139	612	380	116	63	6,917	62	391	73	2,102	57	3,694	81	2,572	-	--
1330	170	4.8	--	113	3,263	612	380	116	66	8,494	63	595	69	3,571	56	3,881	81	2,572	-	--
1400	200	4.9	--	113	2,981	612	380	116	66	9,029	64	624	71	2,939	58	3,680	81	2,572	-	--
1415	215	5.0	--	113	2,869	612	380	116	66	10,098	64	547	70	3,119	58	3,811	79	2,509	-	Sample.
1430	230	5.0	13,044	113	2,993	612	380	116	65	8,483	64	557	71	3,259	57	3,565	79	2,509	9	Sample. End of test.
Average					3,032	612	380	116	65	8,277	62	528	72	2,373	57	3,726	83	2,626		

(1) Moisture: -1-1/2 in., 2.8%; -4 in. +1-1/2 in., 1.0%; -6 in. +4 in., 0.8%; -10 in. +6 in., 0.7%. Average dry ore feed rate: -1-1/2 in., 2,947.0 lb/hr; -4 in. +1-1/2 in., 605.9 lb/hr; -6 in. +4 in., 376.8 lb/hr; -10 in. +6 in., 115.0 lb/hr; total: 4,044.7 lb/hr, 2.022 dry stph. Mill volume end of test: 9%.
(2) Excluded from average.

Feed Rate, stph dry:	2.022
Ball Charge:	301.8 lb, 2% mill volume
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Instantaneous Gross Power (meter reading) kwhr	Instantaneous Corrected Power (from input-output curve) kwhr	Power Consumption Gross kwh/st	Power Consumption Net kwh/st	Circulating Load Weight % of Feed(1)	Mill Discharge Solids %
1040	0	8.7	5.96	4.22	2.09	1.79	--	--
1110	30	5.2	9.97	7.93	3.92	3.63	--	--
1130	50	5.3	9.78	7.78	3.85	3.55	--	62
1200	80	5.0	10.36	8.25	4.08	3.78	--	63
1230	110	4.8	10.80	8.63	4.27	3.97	--	64
1300	140	4.8	10.80	8.63	4.27	3.97	59.0(4)	63
1330	170	4.8	10.80	8.63	4.27	3.97	95.0	66
1400	200	4.9	10.58	8.44	4.17	3.88	87.0	66
1415(3)	215	5.0	10.36	8.25	4.08	3.78(2)	92.0	66
1430(3)	230	5.0	10.36	8.25	4.08	3.78(2)	93.0	65
Average						3.78	91.8	

(1) Calculated: Sum of Sweco oversize and DSM oversize as a percentage of dry mill feed.
(2) Average for power (last two readings): 3.78 kwhr/st.
(3) Sample run.
(4) Omitted from average.

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EXHIBIT 2

Grinding Test 2 -- continued

Procedure: Samples were wet screened on a 325M screen, products dried, and the +325M material dry screened using a Ro-Tap for 30 min.

Test Product	Screen Size Analysis								
	Mill Discharge		Sweco Screen Oversize		DSM Screen Oversize		DSM Screen Undersize		Circulating Load
Sample Time	1415	1430	1415	1430	1415	1430	1415	1430	
Sample Weight, g:	1,058.8	1,206.6	669.3	979.0	915.6	1,106.8	888.1	932.3	
Screen Product (Tyler) Mesh	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %
Head (calculated)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
+28	23.8	21.6	65.5	71.8	40.4	37.6	2.0	1.7	43.4
-28 +35	6.8	6.4	2.5	1.6	8.4	9.9	5.3	4.3	8.1
-35 +65	13.5	13.3	4.2	3.6	8.8	12.0	17.2	16.6	9.4
-65 +100	9.4	10.2	3.2	3.0	4.7	7.6	13.6	12.9	5.7
-100 +200	11.9	13.4	5.0	5.0	7.3	10.3	17.6	17.0	8.3
-200 +325	4.2	5.9	3.0	2.1	1.6	4.7	7.0	6.3	3.1
-325	30.4	29.2	16.6	12.9	28.8	17.9	37.3	41.2	22.0

EXHIBIT 2

Grinding Test 3

Date:	June 15, 1978
Feed Rate, stph:	3.0
Ore:	Run-of-mine
Ball Charge:	Total: 301.8 lb, 2% mill volume
-1-1/2 in. +1 in. Balls, lb:	114.5
-2 in. +1-1/2 in. Balls, lb:	151.3
3 in. Balls, lb:	36.0
DSM Screen, in. width:	12
DSM Screen Openings, mm:	1.27
Measured Mill Power Tare (empty mill), kw:	2.06
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Meter Reading watt-hr	Mill-Bearing Oil Temp. °F	Ore Feed Rate (as received)(1) -1-1/2 in. lb/hr	-4 in. +1-1/2 in. lb/hr	-6 in. +4 in. lb/hr	-10 in. +6 in. lb/hr	Mill Discharge Solids %	Mill Discharge Solids lb/hr	Sweco Screen Oversize Solids %	Sweco Screen Oversize Solids lb/hr	DSM Screen Overflow Solids %	DSM Screen Overflow Solids lb/hr	DSM Screen Underflow Solids %	DSM Screen Underflow Solids lb/hr	Mill Water Meter %(2)	Mill Water Rate lb/hr	Mill Load Volume %	Remarks
1050	0	5.0	13,045	93	--	918	570	174	--	--	--	--	--	--	--	--	--	--	--	Start mill.
1135	45	4.5	--	--	--	918	570	174	--	--	--	--	--	--	--	--	--	--	--	--
1200	70	4.4	--	99	4,350	918	570	174	65	13,631	68	857	70	6,237	58	5,090	105	3,350	--	--
1207	77	--	--	--	--	918	570	174	--	--	--	--	--	--	--	--	--	--	--	Shutdown, rock jammed in feeder.
1230	77	--	--	--	--	918	570	174	--	--	--	--	--	--	--	--	--	--	--	Start mill.
1300	107	4.9	--	109	3,435	918	570	174	65	10,530	63	808	73	3,679	55	4,430	106	3,366	--	--
1330	137	4.8	--	108	4,815	918	570	174	66	11,642	64	878	72	5,508	61	5,408	104	3,303	--	--
1400	167	4.9	--	110	4,275	918	570	174	67	11,095	58	639	73	5,059	61	5,545	104	3,303	--	--
1430	197	4.7	--	111	4,590	918	570	174	67	11,156	65	761	72	5,573	61	4,804	103	3,271	--	Sample.
1445	212	4.8	13,085	112	5,040	918	570	174	67	15,135	67	1,010	71	6,646	62	5,692	104	3,303	--	Sample.
1500	242	--	--	--	--	918	570	174	--	--	--	--	--	--	--	--	--	--	25	Shut down.
Average					4,417	918	570	174	66	12,198	64	826	72	5,450	60	5,162	104	3,316		

(1) Moisture: -1-1/2 in., 2.8%; -4 in. +1-1/2 in., 1.0%; -6 in. +4 in., 0.8%, -10 in. +6 in., 0.7%. Average dry ore feed rate: -1-1/2 in., 4,293.8 lb/hr, -4 in. +1-1/2 in., 908.8 lb/hr; -6 in. +4 in., 565.4 lb/hr; -10 in. +6 in., 172.8 lb/hr; total, 5,940.8 lb/hr, 2.970 dry stph. Mill volume end of test: 25%.
(2) Auxilliary water line used -- measured twice, averaged, and added as percentage of regular water meter.

Feed Rate, stph dry:	2.970 stph dry
Ball Charge:	301.8 lb, 2% of mill volume
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Instantaneous Gross Power (meter reading) kwhr	Instantaneous Corrected Power (from input-output curve) kwhr	Power Consumption Gross kwhr/st	Power Consumption Net kwhr/st	Circulating Load Weight % of Feed(1)	Mill Discharge Solids %	Remarks
1050	0	5.0	10.36	8.26	2.78	2.58	--	--	--
1135	45	4.5	11.52	9.24	3.11	2.91	--	--	--
1200	70	4.4	11.78	9.45	3.18	2.98	118.0(4)	65	--
1207	77	--	--	--	--	--	--	--	Rock jammed in feeder.
1230	77	--	--	--	--	--	--	--	--
1300	107	4.9	10.58	8.43	2.84	2.64	88.0	65	--
1330	137	4.8	10.80	8.62	2.90	2.70(2)	99.0	66	--
1400	167	4.9	10.58	8.43	2.84	2.64(2)	96.0	67	--
1430(3)	197	4.7	11.03	8.82	2.97	2.77(2)	101.0	67	--
1445(3)	212	4.8	10.80	8.62	2.90	2.70(2)	114.0	67	--
1500	242								
Average						2.70	99.6		

(1) Calculated: Sum of Sweco oversize and DSM oversize as a percentage of dry mill feed.
(2) Average for power (last four readings): 2.70 kwhr/st.
(3) Sample run.
(4) Omitted from average.

EXHIBIT 2

Grinding Test 3

Procedure: Samples were wet screened on a 325M screen, products dried, and the +325M material dry screened using a Ro-Tap for 30 min.

Test Product	Screen Size Analysis								
	Mill Discharge		Sweco Screen Oversize		DSM Screen Oversize		DSM Screen Undersize		Circulating Load
Sample Time	1430	1445	1430	1445	1430	1445	1430	1445	--
Sample Weight, g:	1,174.9	1,310.3	1,365.7	1,223.1	1,183.4	1,245.5	850.1	962.4	--
Screen Product (Tyler) Mesh	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %
Head (calculated)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
+28	27.8	25.1	65.0	67.5	47.4	33.3	2.4	1.9	43.7
-28 +35	6.5	7.1	1.8	2.0	9.1	7.9	5.7	5.0	7.6
-35 +65	12.8	14.6	3.7	4.0	12.4	13.2	18.1	21.0	11.7
-65 +100	9.2	9.0	3.1	3.4	6.5	8.5	14.8	16.0	7.0
-100 +200	11.4	13.5	5.4	5.5	8.9	9.9	15.6	13.5	8.9
-200 +325	4.8	3.4	3.4	3.3	1.6	3.3	5.9	4.5	2.5
-325	27.5	27.3	17.6	14.3	14.1	23.9	37.5	38.1	18.6

EXHIBIT 2

Grinding Test 4

Date:	June 16, 1978
Feed Rate, stph:	2.5
Ore:	Crushed
Ball Charge:	Total: 301.8 lb, 2% mill volume
-1-1/2 in. +1 in. Balls, lb:	114.5
-2 in. +1-1/2 in. Balls, lb:	151.3
3 in. Balls, lb:	36.0
DSM Screen, in. width:	12
DSM Screen Openings, mm:	1.27
Measured Mill Power Tare (empty mill), kw:	2.06
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Meter Reading watt-hr	Mill-Bearing Oil Temp. °F	Ore Feed Rate (as received)(1) -3 in. lb/hr	Mill Discharge Solids %	Mill Discharge Solids lb/hr	Sweco Screen Oversize Solids %	Sweco Screen Oversize Solids lb/hr	DSM Screen Overflow Solids %	DSM Screen Overflow Solids lb/hr	DSM Screen Underflow Solids %	DSM Screen Underflow Solids lb/hr	Mill Water Meter %(1)	Mill Water Rate lb/hr	Mill Load Volume %	Remarks
1010	0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	9	Start mill.
1030	20	6.6	13,094	96	--	--	--	--	--	--	--	--	--	90	2,858	--	--
1100	50	6.3	--	97	5,130	63	7,598	67	362	74	1,931	61	5,243	87	2,763	--	--
1130	80	5.9	--	99	5,350	62	8,091	64	418	72	2,398	60	4,482	82	2,604	--	--
1200	110	5.9	--	99	4,995	65	12,519	66	535	70	3,717	61	3,953	80	2,540	--	--
1215	125	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Feed off (feed belt jammed).
1218	128	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Start mill.
1230	137	6.0	--	100	4,770	62	5,692	62	288	71	2,077	58	3,628	80	2,540	--	--
1300	167	6.0	--	100	5,423	65	6,786	62	326	71	1,885	60	4,428	80	2,540	--	--
1320	187	--	--	--	--	--	--	--	--	--	--	60	4,316(3)	--	--	--	--
1330	197	5.8	--	102	4,826	65	6,728	65	449	69	2,298	59	4,806	79	2,509	--	--
1400	227	5.7	--	104	4,635	64	6,797	62	260	72	1,134	60	4,617	79	2,509	--	Sample.
1415	242	5.7	13,128	104	6,793	63	6,010	64	230	70	819	59	4,328	79	2,509	15	Sample.
1500	257	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	
Average					5,240	64	7,528	64	359	71	2,032	60	4,422	82	2,597	--	

(1) Moisture: -3 in., 4.3%. Average dry ore feed rate: -3 in., 5,015 lb/hr, 2.508 dry stph. Mill volume end of test: 15%.
(2) Auxilliary water line used -- measured twice, averaged, and added as percentage of regular water meter.
(3) 55-gal drum timed sample.

Feed Rate, stph:	2.508
Ball Charge:	301.8 lb, 2% of mill volume
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Instantaneous Gross Power (meter reading) kwhr	Instantaneous Corrected Power (from input-output curve) kwhr	Power Consumption Gross kwhr/st	Power Consumption Net kwhr/st	Circulating Load Weight % of Feed(1)	Mill Discharge Solids %	Remarks
1010	0	--	--	--	--	--	--	--	--
1030	20	6.6	7.85	6.00	2.39	2.15	--	--	--
1100	50	6.3	8.23	6.35	2.53	2.29	--	63	--
1130	80	5.9	8.78	6.87	2.74	2.50	50.0(4)	62	--
1200	110	5.9	8.78	6.87	2.74	2.50	81.0(4)	65	--
1215	125	--	--	--	--	--	--	--	Feed belt jammed.
1230	137	6.0	8.64	6.73	2.68	2.44	48.0	62	--
1300	167	6.0	8.64	6.73	2.68	2.44	39.0	65	--
1320	187	--	--	--	--	--	--	--	--
1330	197	5.8	8.93	7.00	2.79	2.55(2)	54.0	65	--
1400(3)	227	5.7	9.09	7.13	2.84	2.60(2)	29.0	64	--
1415(3)	242	5.7	9.09	7.13	2.84	2.60(2)	14.0	63	--
Average						2.58	36.8		

(1) Calculated; Sum of Sweco oversize and DSM oversize as a percentage of dry mill feed.
(2) Average for power (last three readings): 2.58 kwhr/st.
(3) Sample run.
(4) Omitted from average.

EXHIBIT 2

Grinding Test 4 -- continued

Procedure: Samples were wet screened on a 325M screen, products dried, and the +325M material dry screened using a Ro-Tap for 30 min.

	Screen Size Analysis								
Test Product	Mill Discharge		Sweco Screen Oversize		DSM Screen Oversize		DSM Screen Undersize		Circulating Load
Sample Time	1140	1415	1400	1415	1400	1415	1400	1415	--
Sample Weight, g:	1,139.4	886.7	715.4	726.2	1,152.9	1,020.0	763.8	769.4	--
Screen Product (Tyler) Mesh	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %
Head (calculated)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
+28	15.3	13.1	86.5	91.8	39.1	43.1	2.7	2.7	55.5
-28 +35	5.8	5.2	0.3	0.3	8.9	7.6	4.9	4.6	5.9
-35 +65	17.8	17.9	0.9	0.5	14.9	12.7	18.6	18.6	9.9
-65 +100	11.1	11.8	0.7	0.3	6.8	6.3	12.5	13.3	4.7
-100 +200	15.8	16.7	1.6	0.7	8.8	8.9	18.6	19.1	6.6
-200 +325	7.7	6.4	0.9	0.4	3.3	4.1	8.1	6.3	2.8
-325	26.5	28.9	9.1	6.0	18.2	17.3	34.6	35.4	14.6

EXHIBIT 2

Grinding Test 5

Date:	June 19, 1978
Feed Rate, stph:	2.0
Ore:	Crushed
Ball Charge:	Total 301.8 lb, 2% mill volume
-1-1/2 in. Balls, lb:	114.5
-2 in. +1-1/2 in. Balls, lb:	151.3
3 in. Balls, lb:	36.0
DSM Screen, in. width:	12
DSM Screen Openings, mm:	1.27
Measured Mill Power Tare (empty mill), kw:	2.06
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Meter Reading watt-hr	Mill-Bearing Oil Temp. °F	Ore Feed Rate (as received)(1) -3 in. lb/hr	Mill Discharge		Sweco Screen Oversize		DSM Screen Overflow		DSM Screen Underflow		Mill Water		Mill Load Volume %	Remarks
						Solids %	Solids lb/hr	Solids %	Solids lb/hr	Solids %	Solids lb/hr	Solids %	Solids lb/hr	Meter %	Rate lb/hr		
0840	0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	7	Start mill.
0910	30	6.7	13,136	90	3,623	--	--	--	--	--	--	--	--	75	2,382	--	--
0930	50	6.3	--	91	3,960	67	8,744	48	356	67	3,558	60	2,970	71	2,255	--	--
1000	80	6.2	--	92	3,803	66	6,663	45	324	70	2,079	60	4,077	68	2,159	--	--
1030	110	6.5	--	91	--	56	3,578	15	68	70	347	59	3,452	66	2,096	--	Shut down -- out of feed.
1035	115	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
1040	115	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Start mill.
1100	135	6.5	--	94	4,230	66	4,990	38	182	75	346	62	4,241	68	2,159	--	--
1130	165	6.6	--	96	4,298	66	5,049	42	239	72	729	62	4,101	69	2,191	--	--
1155	190	--	--	--	--	--	--	--	--	--	--	--	--	--	--	13	--
1200	195	6.7	--	97	4,320	63	3,856	37	200	75	405	61	3,870	69	2,191	--	--
1230	225	6.7	--	100	3,533	62	3,894	27	101	73	394	58	3,445	64	2,032	--	--
1300	255	6.6	--	103	4,016	66	4,693	29	111	70	851	61	3,870	68	2,159	--	--
1330	285	6.3	--	104	4,005	68	9,058	34	173	68	3,672	64	3,744	61	1,937	--	--
1345	300	6.5	--	104	3,645	63	4,139	32	134	71	250	59	3,452	68	2,159	--	Sample.
1400	315	6.1	--	104	4,005	64	4,781	34	143	72	238	57	3,104	69	2,191	--	Sample.
1430	345	6.1	--	105	4,140	63	4,820	33	193	69	598	59	3,505	69	2,191	--	--
1445	360	6.0	--	106	3,713	62	4,018	38	182	71	423	56	2,696	69	2,191	--	Sample.
1500	375	5.7	13,184	107	4,028	63	4,139	36	151	70	1,323	56	2,696	69	2,191	--	Sample.
1510	380	--	--	--	--	--	--	--	--	--	--	--	--	--	--	15	Shut down.
1513	388	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Collecting mill discharge sample.
1522	397	--	--	--	3,690	--	--	--	--	--	--	--	--	--	--	--	Second barrel.
1529	404	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Third barrel.
1536	411	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Hopper went empty.
1537	412	--	--	--	--	--	--	--	--	--	--	--	--	--	--	15	Shut down mill.
Average	--	--	--	--	3,934	64	5,173	35	183	71	1,087	59	3,516	68	2,165	--	--

(1) Moisture: -3 in., 2.0%. Average dry ore feed rate: -3 in., 3,855 lb/hr, 1.928 dry stph. Mill volume end of test: 15%.

EXHIBIT 2

Grinding Test 5 -- continued

Feed Rate, stph (dry): 1.928
Ball Charge: 301.8 lb, 2% of mill charge
Corrected Mill Power Tare (empty mill), kw: 0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Instantaneous Gross Power (meter reading) kwhr	Instantaneous Corrected Power (from input-output curve) kwhr	Power Consumption Gross kwhr/st	Power Consumption Net kwhr/st	Circulating Load Weight % of Feed(1)	Mill Discharge Solids %	Remarks
0840	0	--	--	--	--	--	--	--	--
0910	30	6.7	7.73	5.89	3.05	2.74	--	--	--
0930	50	6.3	8.23	6.35	3.29	2.98	--	67	--
1000	80	6.2	8.36	6.47	3.36	3.04	--	66	--
1030	110	6.5	7.97	6.10	3.16	2.85	--	56	--
1035	115	--	--	--	--	--	--	--	Ran out of ore.
1100	135	6.5	7.97	6.10	3.16	2.85	12.0(4)	66	--
1130	165	6.6	7.85	6.00	3.11	2.80	23.0(4)	66	--
1155	190	--	--	--	--	--	--	--	Check mill volume.
1200	195	6.7	7.73	5.89	3.05	2.74	14.0	63	--
1230	225	6.7	7.73	5.89	3.05	2.74	14.0	62	--
1300	255	6.6	7.85	6.00	3.11	2.80	24.0	66	--
1330	285	6.3	8.23	6.35	3.29	2.98	96.0(4)	68	--
1345(3)	300	6.5	7.97	6.10	3.16	2.85	11.0	63	--
1400(3)	315	6.1	8.50	6.60	3.42	3.11(2)	10.0	64	--
1430	345	6.1	8.50	6.60	3.42	3.11(2)	19.0	63	--
1445(3)	360	6.0	8.64	6.73	3.49	3.18(2)	16.0	62	--
1500(3)	375	5.7	9.09	7.13	3.70	3.37	37.0	63	--
1510	385	--	--	--	--	--	--	--	Check mill load level.
1513	388	--	--	--	--	--	--	--	Start filling No. 1 mill discharge sample barrel.
1522	397	--	--	--	--	--	--	--	Start filling No. 2 mill discharge sample barrel.
1529	404	--	--	--	--	--	--	--	Start filling No. 3 mill discharge sample barrel.
1536	411	--	--	--	--	--	--	--	End filling No. 3 mill discharge sample barrel.
1537	412	--	--	--	--	--	--	--	End of test.
Average						3.13	18.0		

(1) Calculated: Sum of Sweco oversize and DSM oversize as a percentage of dry mill feed.
(2) Average for power (three readings, omitted reading at 1,500 from average): 3.13 kwhr/st.
(3) Sample run.
(4) Omitted from average.

Procedure: Samples were wet screened on a 325M screen, products dried, and the +325M material dry screened using a Ro-Tap for 30 min.

Test Product	Screen Size Analysis: Mill Discharge				Sweco Screen Oversize				DSM Screen Oversize				DSM Screen Underflow				Circulating Load
Sample Time	1345	1400	1445	1500	1345	1400	1445	1500	1345	1400	1445	1500	1345	1400	1445	1500	--
Sample Weight, g:	1,058.6	1,062.1	911.3	859.1	442.5	300.3	282.2	381.8	1,065.9	713.5	478.8	920.6	817.4	757.0	743.7	787.8	--
Screen Product (Tyler) Mesh	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %
Head (calculated)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
+28	12.0	11.5	10.2	10.8	78.4	82.9	81.4	87.5	67.0	54.5	51.9	32.0	1.8	2.0	1.9	1.6	58.1
-28 +35	3.7	3.7	2.7	2.9	1.5	0.8	1.0	0.4	5.0	4.6	4.5	3.9	3.1	3.1	2.8	2.3	3.7
-35 +65	15.3	16.3	12.9	13.4	4.1	1.9	3.0	1.1	6.2	7.4	6.9	10.9	16.8	16.3	15.8	14.2	6.7
-65 +100	12.3	13.4	12.8	12.7	2.4	1.2	1.9	0.8	3.4	5.2	5.2	9.1	14.7	14.6	14.2	14.5	4.8
-100 +200	19.1	18.5	21.3	20.6	4.1	2.7	3.7	1.6	5.3	8.2	9.3	14.2	20.5	20.5	21.7	21.8	8.0
-200 +325	8.0	6.6	9.0	8.6	1.1	1.0	1.1	1.2	1.6	2.8	4.0	5.3	8.1	8.4	7.4	7.4	2.9
-325	29.6	30.0	31.1	31.0	8.4	9.5	7.9	7.4	11.5	17.3	18.2	24.6	35.0	35.1	36.2	38.2	15.8

EXHIBIT 2

Grinding Test 6

Date:	June 20, 1978
Feed Rate, stph:	2.5
Ore:	Run-of-mine
Ball Charge:	Total: 301.8 lb, 2% mill volume
-1-1/2 in. +1 in. Balls, lb:	114.5
-2 in. +1-1/2 in. Balls, lb:	151.3
3 in. Balls, lb:	36.0
DSM Screen, in. width:	12
DSM Screen Openings, mm:	1.27
Measured Mill Power Tare (empty mill), kw:	2.06
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Meter Reading watt-hr	Mill-Bearing Oil Temp. °F	Ore Feed Rate (as received)(1) -1-1/2 in. lb/hr	-4 in. +1-1/2 in. lb/hr	-6 in. +4 in. lb/hr	-10 in. -6 in. lb/hr	Mill Discharge Solids %	Mill Discharge Solids lb/hr	Sweco Screen Oversize Solids %	Sweco Screen Oversize Solids lb/hr	DSM Screen Overflow Solids %	DSM Screen Overflow Solids lb/hr	DSM Screen Underflow Solids %	DSM Screen Underflow Solids lb/hr	Mill Water Meter %	Mill Water Rate lb/hr	Mill Load Volume %	Remarks
0820	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--(Grind out)--		Start mill.
0925	0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	Start feed.
0930	5	6.8	13,195	82	--	768	474	219	--	--	--	--	--	--	--	--	80	2,540	--	--
1000	35	5.9	--	80	--	768	474	219	66	11,286	60	662	71	4,090	61	5,737	85	2,699	--	--
1030	65	5.3	--	82	3,713	768	474	219	66	9,742	54	535	68	4,896	61	3,486	84	2,668	--	--
1100	95	5.2	--	83	3,825	768	474	219	67	10,492	60	608	68	4,651	61	4,255	85	2,699	--	--
1135	130	5.2	--	84	3,510	768	474	219	66	7,960	59	597	68	3,733	61	4,255	84	2,668	25	--
1200	155	5.2	--	87	3,758	768	474	219	68	10,588	57	487	68	4,651	60	3,699	85	2,699	--	--
1230	185	5.1	--	88	3,420	768	474	219	68	10,037	55	545	69	3,974	60	4,104	89	2,826	--	--
1245	200	5.1	--	88	3,420	768	474	219	67	9,950	52	714	68	4,223	59	4,275	89	2,826	--	Sample.
1300	215	5.0	--	89	3,600	768	474	219	67	11,759	62	781	68	6,487	62	3,627	85	2,699	--	Sample.
1330	245	5.0	12,236	92	--	768	474	219	67	8,924	60	1,337	68	4,039	60	3,780	88	2,795	--	--
1337	252	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	27	Shut down.
Average					3,607	768	474	219	67	10,082	58	696	68	4,527	61	4,135	85	2,712		

(1) Moisture: -1-1/2 in., 2.3%; -4 in. +1-1/2 in., 1.0%; -6 in. +4 in., 0.8%; -10 in. +6 in., 0.7%. Average dry ore feed rate: -1-1/2 in., 3,524 lb/hr; -4 in. +1-1/2 in., 760.3 lb/hr; -6 in. +4 in., 470.2 lb/hr; -10 in. +6 in., 217.5 lb/hr; Total: 4,972 lb/hr; 2.486 dry stph. Mill volume end of test: 27%.

Feed Rate, stph (dry):	2.486
Ball Charge:	301.8 lb, 2% of mill volume
Corrected Mill Power Tare (empty mill), kw:	0.6

Clock Time	Running Time min	Disc Revolutions sec/rev	Instantaneous Gross Power (meter reading) kwhr	Instantaneous Corrected Power (from input-output curve) kwhr	Power Consumption Gross kwhr/st	Power Consumption Net kwhr/st	Circulating Load Weight % of Feed(1)	Mill Discharge Solids %	Remarks
0820	--	--	--	--	--	--	--	--	Grind out.
0925	--	--	--	--	--	--	--	--	Start feed.
0930	5	6.8	7.62	5.80	2.33	2.09	--	--	--
1000	35	5.9	8.78	6.87	2.76	2.52	--	66	--
1030	65	5.3	9.78	7.78	3.13	2.92	105.0	66	--
1100	95	5.2	9.97	7.92	3.18	2.94	99.0	67	--
1135	130	5.2	9.97	7.92	3.18	2.94	87.0	66	--
1200	155	5.2	9.97	7.92	3.18	2.94	98.0	68	--
1230	185	5.1	10.16	8.09	3.25	3.01	93.0	68	--
1245(3)	200	5.1	10.16	8.09	3.25	3.01	101.0	67	--
1300(3)	215	5.0	10.36	8.26	3.32	3.08(2)	144.0	67	--
1330	245	4.0	10.36	8.26	3.32	3.08(2)	--	67	--
1337	252	--	--	--	--				End of test.
Average						3.08	103.9		

(1) Calculated: Sum of Sweco oversize and DSM oversize as a percentage of dry mill feed.
(2) Average for power (two readings): 3.08 kwhr/st.
(3) Sample run.

EXHIBIT 2

Grinding Test 6 -- continued

Procedure: Samples were wet screened on a 325M screen, products dried, and the +325M material dry screened using a Ro-Tap for 30 min.

	Screen Size Analysis								
Total Product	Mill Discharge		Sweco Screen Oversize		DSM Screen Oversize		DSM Screen Undersize		Circulating Load
Sample Time	1245	1300	1245	1300	1245	1300	1245	1300	
Sample Weight, g:	1,258.8	1,237.7	673.8	642.6	1,361.9	1,079.3	832.1	918.1	--
Screen Product (Tyler) Mesh	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %	Weight %
Head (calculated)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
+28	21.0	18.4	64.8	70.7	32.9	23.1	1.3	1.0	32.9
-28 +35	6.4	6.5	1.9	1.2	9.4	8.5	3.9	3.7	8.1
-35 +65	13.9	15.1	3.8	2.7	12.8	14.3	16.5	16.7	12.2
-65 +100	10.5	11.4	3.2	2.2	8.8	8.6	12.4	14.5	8.0
-100 +200	13.3	14.2	5.4	5.0	11.8	14.2	20.3	18.5	12.0
-200 +325	5.5	5.6	3.1	2.2	4.8	3.7	5.3	6.7	4.1
-325	29.4	28.8	17.8	16.0	19.5	27.6	40.3	38.9	22.7

Sediment Description and Classification Background

U.S. Standard Sieves

Note that the same size mesh can be a differing sieve number depending on the Sieve manufacturer (Tyler vs. ASTM)

Mesh Size (microns)	TYLER	ASTM-E11	BS-410	DIN-4188
µm	Mesh	No.	Mesh	mm
5	2500		2500	0.005
10	1250		1250	0.010
15	800		800	0.015
20	625		625	0.020
22				0.022
25	500		500	0.025
28				0.028
32				0.032
36				0.036
38	400	400	400	
40				0.040
45	325	325	350	0.045
50				0.050
53	270	270	300	
56				0.056
63	250	230	240	0.063
71				0.071
75	200	200	200	
80				0.080
90	170	170	170	0.090
100				0.100
106	150	140	150	
112				0.112
125	115	120	120	0.125
140				0.140
150	100	100	100	

Attachment B, 1/10

160				0.160
180	80	80	85	0.180
200				0.200
212	65	70	72	
250	60	60	60	0.250
280				0.280
300	48	50	52	
315				0.315
355	42	45	44	0.355
400				0.400
425	35	40	36	
450				0.450
500	32	35	30	0.500
560				0.560
600	28	30	25	
630				0.630
710	24	25	22	0.710
800				0.800
850	20	20	18	
900				0.900
1000	16	18	16	1.0
1120				1.12
1180	14	16	14	
1250				1.25
1400	12	14	12	1.4
1600				1.6
1700	10	12	10	
1800				1.8
2000	9	10	8	2.0
2240				2.24
2360	8	8	7	
2500				2.5
2800	7	7	6	2.8
3150				3.15
3350	6	6	5	
3550				3.55
4000	5	5	4	4.0
4500				4.5

4750	4	4	3.5	
5000				5.0

Sediment Classification based on Grain Size:

Unified Soil Classification System (USCS)

Sediment Name	**Diameter (mm)**	**Sieve No.**
Cobble	greater than 75 mm	
Gravel	4.75 to 75 mm	4
Sand	0.075 to 4.75 mm	200
Fines (silt and clay)	less than 0.075 mm	

USCS Division of Sands

Sediment Name	Diameter Range (mm)	Passes through Sieve No.	Retained on Sieve No.
Coarse Sand	2.0 - 4.8	4	10
Medium Sand	0.43 - 2.0	10	40
Fine Sand	0.075 - 0.43	40	200

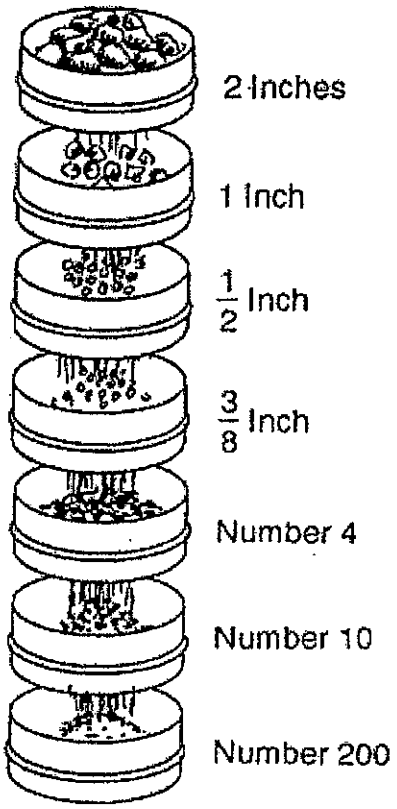


Figure 4-3. Dry sieve analysis.

USCS Classification System

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	DESCRIPTIONS
COARSE GRAINED SOILS More Than Half Retained on 200 Sieve	GRAVELS More Than Half Coarse Fraction Retained on No. 4 Sieve	Clean Gravels (Little or no Fines)	GW	Well Graded Gravels, Gravel - Sand Mixtures, Little or no Fines
			GP	Poorly Graded Gravels, Gravel - Sand Mixtures, Little or no Fines
		Gravels With Fines (Appreciable Fines)	GM	Silty Gravels, Gravel-Sand-Silt Mixtures
			GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
	SANDS More Than Half Coarse Fraction Passes a No. 4 Sieve	Clean Sands (Little or no Fines)	SW	Well Graded Sands, Gravelly Sands, Little or no Fines
			SP	Poorly Graded Sands, Gravelly Sands, Little or no Fines
		Sands With Fines (Appreciable Fines)	SM	Silty Sands, Sand - Silt Mixtures
			SC	Clayey Sands, Sand - Clay Mixtures
FINE GRAINED SOILS More Than Half Passes 200 Sieve	SILTS and CLAYS Liquid Limit Less Than 50		ML	Inorganic Silts & Very Fine Sands, Silty or Clayey Fine Sands, Clayey Silts
			CL	Inorganic Clays of Low to Medium Plasticity, Lean Clays
			OL	Organic Silts & Organic Silty Clays of Low Plasticity
	SILTS and CLAYS Liquid Limit Greater Than 50		MH	Inorganic Silts, Fine Sand or Silty Soils, Elastic Silts
			CH	Inorganic Clays of High Plasticity, Fat Clays
			OH	Organic Clays of Medium to High Plasticity, Organic Silts
Highly Organic Soils			PT	Peat and Other Highly Organic Soils

Visual logging of sediments entails estimating percentages of gravels, sands and fines (silt and clays). Practice and the use of the Geotechnical Gage will increase your confidence and ability in visually logging sediments.

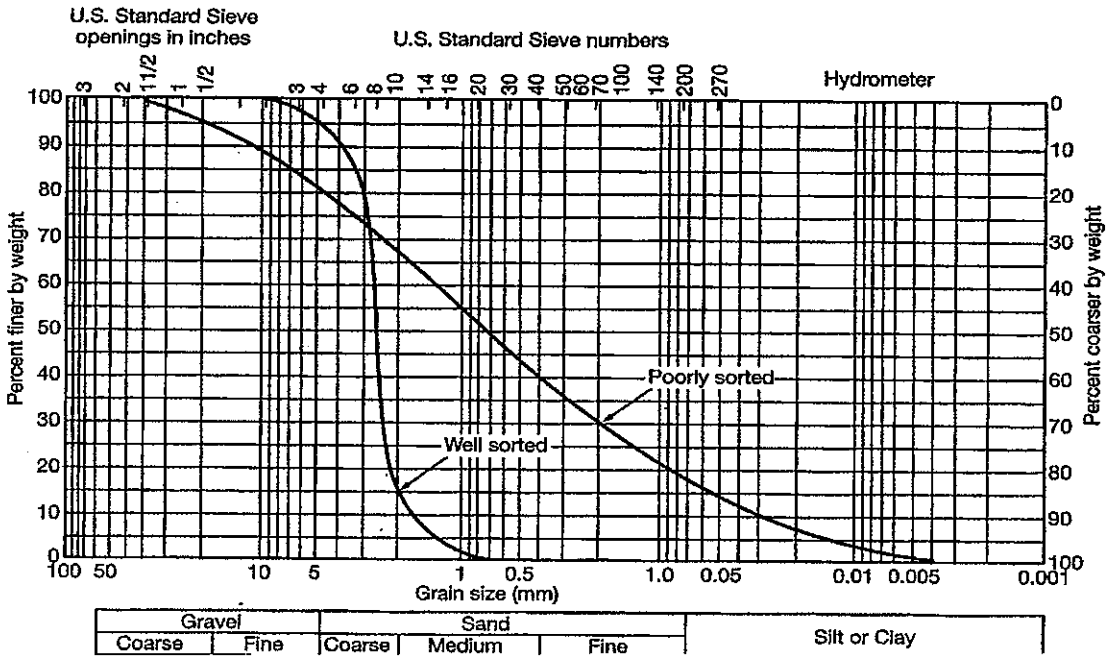
Read: Visual Exam Test

Read: Field Identification Guidelines

Ultimately, sediment samples may undergo grain size analysis through sieves. Graphing the cumulative weight percent retained/passing by sieve no. or grain size will result in the sediment grain-size distribution curve. The grain-size distribution curve is used to quantitatively classify the sediment type (your visual identification is a qualitative classification).

Read: Grain Size Distribution Measurement

Grain Size Distribution Curve



The grain-size distribution curve is used with the USCS classification chart to classify the sediment type. Other measures used to describe the sediment are the sorting or gradation of the sediment. As can be seen in the above chart, a well-sorted sediment has a small range of sediment grain sizes while a poorly sorted sediment has a large range of sediment grain sizes. In the USCS classification scheme, the gradation of the sediment is used instead of the sorting. A well-graded sediment has a large range of grain sizes while a poorly or uniformly graded sediment has a small range of grain sizes.

Figure 4-6. Well-graded soil.

POORLY SORTED SEDIMENT = WELL GRADED SEDIMENT

Figure 4-7. Uniformly graded soil.

WELL-SORTED SEDIMENT = POORLY OR UNIFORMLY GRADED SEDIMENT

Figure 4-8. Gap-graded soil.

After sieve analysis, the data are tabulated showing the weight of sediment retained on each sieve. The cumulative weight retained is calculated starting from the largest sieve size and adding subsequent sediment weights from the smaller size sieves (see table below). The percent retained is calculated from the weight retained and the total weight of the sample. [Don't get confused by the graph - it is individual percent retained in Column 16 and cumulative percent passing in Column 17]. The cumulative percent passing in Column 17 of the table below is calculated by sequentially subtracting percent retained from 100 %. In table below, cumulative percent passing 1/4 inch sieve = 100 - 16 = 84; cumulative percent passing #4 sieve = 84-5.2 = 78.8; etc.

SIEVE ANALYSIS DATA

1. DATE STARTED: 22 FEB 91
2. PROJECT: BRAVO AIRFIELD
3. EXCAVATION: 1+00
4. DATE COMPLETED: 28 FEB 91
5. SAMPLE DESCRIPTION: LIGHT BROWN SANDY SOIL
6. SAMPLE NUMBER: 1A
7. PREWASHED (if one): XX YES / NO
8. ORIGINAL SAMPLE WEIGHT: 2459
9. + #200 SAMPLE WEIGHT: 2359
10. −#200 SAMPLE WEIGHT: 100

11. SIEVE SIZE	12. WEIGHT OF SIEVE	13. WEIGHT OF SIEVE+ SAMPLE	14. WEIGHT RETAINED	15. CUMULATIVE WEIGHT RETAINED	16. PERCENT RETAINED	17. PERCENT PASSING
1½	202					
1	231					
½	210	210	0	0	0	100.0
¼	230	624	394	394	16.0	84.0
#4	205	332	127	521	5.2	78.8
#8	225	691	466	987	19.0	59.8
#20	215	612	397	1384	16.2	43.6
#60	235	581	346	1730	14.1	29.5
#100	250	612	362	2092	14.7	14.8
#200	260	515	255	2347	10.4	4.4

18. TOTAL WEIGHT RETAINED IN SIEVES (from Column 14): 2347
19. ERROR (8 - 23): 2459-2457 = 2
20. WEIGHT SIEVED THROUGH #200 (weight in pan): 270-260 — 10
21. WASHING LOSS (18 - 19 + 20): 2459-(2359+100) — 0
22. TOTAL WEIGHT PASSING #200 (19 + 10): 10+100 — 110
23. TOTAL WEIGHT OF FRACTIONS (18 + 22): 2457

24. REMARKS
USCS SP
PERCENT-G 21.2
PERCENT-S 74.4
PERCENT-F 4.4

25. ERROR (Percent): $\frac{\text{ERROR (19)}}{\text{ORIGINAL WT (8)}} \times 100 = \frac{2}{2459} \times 100 = .08$

26. TECHNICIAN: Joe Blob PV2
27. COMPUTED BY (Signature): Joe Blob PV2
28. CHECKED BY (Signature): Fred Jones SSG

DD Form 1206, DEC 86 — Previous editions are obsolete

Figure 4-4. Data sheet, example of dry sieve analysis.

The cumulative percent passing is plotted on the grain-size distribution graph. The percentage passing the No. 4 and 200 sieves is used to classify the sediments as gravels (G), sands (S) or fines (must use plasiticity index to differentiate between silts and clays).

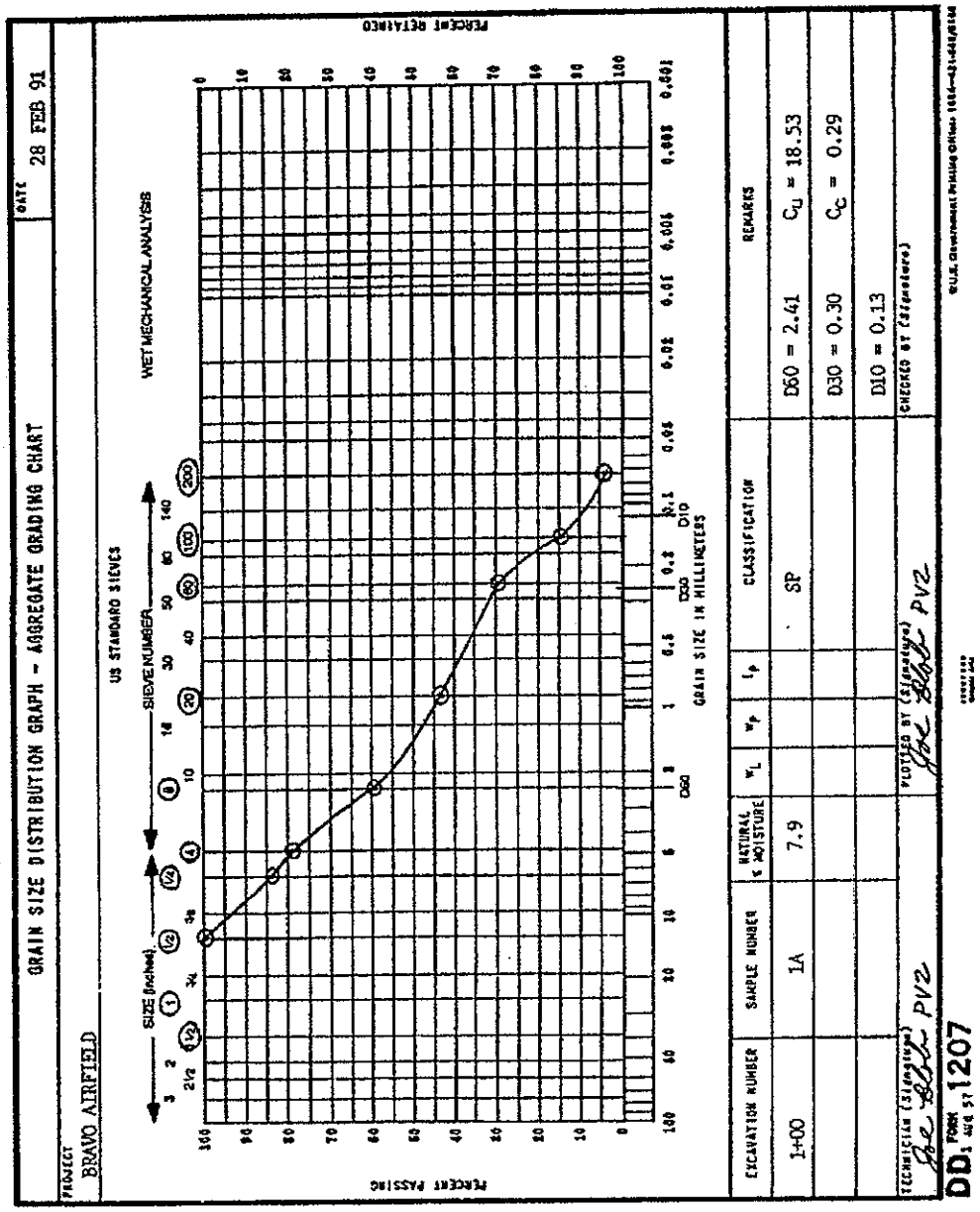


Figure 4-5. Grain-size distribution curve from sieve analysis.

The grain-size distribution graph is used to read off the grain size at which 10% of the sample passed (D_{10}), 30% of the sample passed (D_{30}) and 60% of the sample passed (D_{60}). These numbers are used to calculate several coefficients:

Hazen's effective size, D_{10}, which will be used to estimate permeability

Uniformity Coefficient, C_u = D_{60}/D_{10}

In the above graph,

$$D_{60} = 2.4 \text{ mm and } D_{10} = 0.13 \text{ mm}$$

$$\text{then} \quad C_u = 2.4/0.13 = 18.5$$

The uniformity coefficient is used to judge gradation.

Coefficient of Curvature, C_c

$$C_c = \frac{(D_{30})^2}{(D_{60} \times D_{10})}$$

In the above graph,

$$D_{30} = 0.3 \text{ mm}$$

$$\text{and } C_c = \frac{(0.3)^2}{(2.4)(0.13)} = .29$$

In the graph below, well-graded soils (GW and SW) are long curves spanning a wide range of sizes with a constant or gently varying slope. Uniformly graded soils (SP) are steeply sloping curves spanning a narrow range of sizes. For a gap-graded soil (GP), the curve flattens out in the area of the grain-size deficiency or gap.

The USCS criteria for well-graded gravels (GW) and sands (SW) are:

1. Less than 5% finer than No. 200 sieve
2. Uniformity coefficient greater than 4
3. Coefficient of curvature between 1 and 3

If Criterion 1 is met, but not Criteria 2 and 3, the gravels are gap-graded or uniform gravels (GP) or sands (SP)

If you are interested in more information: Gradation and Bearing Capacity

United States
Environmental Protection
Agency

Office of Research and
Development
Washington DC 20460

EPA/600/R-94/168b
September 1994

EPA

The Hydrologic Evaluation of Landfill Performance (HELP) Model

Engineering Documentation for Version 3

Attachment C, 1/3

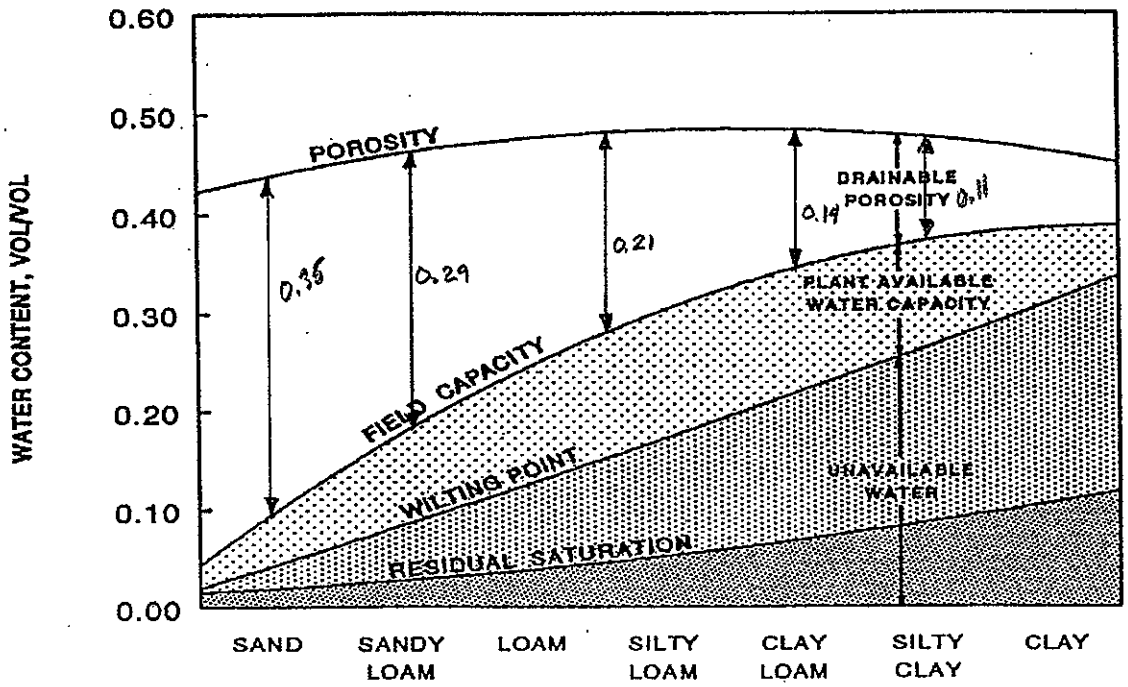


Figure 2. Relation Among Moisture Retention Parameters and Soil Texture Class

are not specified, the program assumes values near the steady-state values (allowing no long-term change in moisture storage) and runs a year of simulation to initialize the moisture contents closer to steady state. The soil water contents at the end of this year are substituted as the initial values for the simulation period. The program then runs the complete simulation, starting again from the beginning of the first year of data. The results of the volumetric water content initialization period are not reported in the output.

3.3.2 Unsaturated Hydraulic Conductivity

Darcy's constant of proportionality governing flow through porous media is known quantitatively as hydraulic conductivity or coefficient of permeability and qualitatively as permeability. Hydraulic conductivity is a function of media properties, such as particle size, void ratio, composition, fabric, degree of saturation, and the kinematic viscosity of the fluid moving through the media. The HELP program uses the saturated and unsaturated hydraulic conductivities of soil and waste layers to compute vertical drainage, lateral drainage and soil liner percolation. The vapor diffusivity for geomembranes is specified as a saturated hydraulic conductivity to compute leakage through geomembranes by vapor diffusion.

Attachment C, 2/3

TABLE 1. DEFAULT LOW DENSITY SOIL CHARACTERISTICS

Soil Texture Class			A Total Porosity vol/vol	B Field Capacity vol/vol	Wilting Point vol/vol	Saturated Hydraulic Conductivity cm/sec	A-B DRAINABLE POROSITY vol/vol
HELP	USDA	USCS					
1	CoS	SP	0.417	0.045	0.018	1.0×10^{-2}	
2	S	SW	0.437	0.062	0.024	5.8×10^{-3}	
3	FS	SW	0.457	0.083	0.033	3.1×10^{-3}	
4	LS	SM	0.437	0.105	0.047	1.7×10^{-3}	0.332
5	LFS	SM	0.457	0.131	0.058	1.0×10^{-3}	0.326
6	SL	SM	0.453	0.190	0.085	7.2×10^{-4}	0.263
7	FSL	SM	0.473	0.222	0.104	5.2×10^{-4}	0.251
8	L	ML	0.463	0.232	0.116	3.7×10^{-4}	0.231
9	SiL	ML	0.501	0.284	0.135	1.9×10^{-4}	0.217
10	SCL	SC	0.398	0.244	0.136	1.2×10^{-4}	0.154
11	CL	CL	0.464	0.310	0.187	6.4×10^{-5}	
12	SiCL	CL	0.471	0.342	0.210	4.2×10^{-5}	
13	SC	SC	0.430	0.321	0.221	3.3×10^{-5}	
14	SiC	CH	0.479	0.371	0.251	2.5×10^{-5}	
15	C	CH	0.475	0.378	0.251	2.5×10^{-5}	
21	G	GP	0.397	0.032	0.013	3.0×10^{-1}	

a = constant representing the effects of various fluid constants and gravity, 21 cm^3/sec

ϕ = total porosity, vol/vol

θ_r = residual volumetric water content, vol/vol

ψ_b = bubbling pressure, cm

λ = pore-size distribution index, dimensionless

A more detailed explanation of Equation 11 can be found in Appendix A of the HELP program Version 3 User's Guide and the cited references.

ATTACHMENT C, 3/3

ered that when well-graded mixtures of sand and gravel contained as little as 5% of fines (sizes smaller than a No. 200 sieve) high compactive efforts reduced the effective porosities nearly to zero and the permeabilities to less than 0.01% of those at moderate densities. These tests explain one of the reasons that blends of sand and gravel often used for drains are virtually useless as drainage aggregates if they contain more than insignificant amounts of fines.

In the preceding paragraphs variations in the permeability of remolded materials caused by variable compaction were discussed. Any factor that densifies soils reduces permeability. Studies of the rate of consolidation of clay and peat foundations are sometimes made by using initial coefficients of permeability of compressible formations. While the consolidation process is going on in foundations their permeabilities are becoming less. Generally, decreases in the permeabilities of clay foundations are rather moderate, but they can be large in highly compressible organic silts and clays and in peats. Modified calculation methods utilizing the changing permeability are needed in the analysis of highly compressible foundations. Some typical variations in permeability caused by consolidation are given in Fig. 2.10, a plot of consolidation pressure versus permeability.

$\sigma_n \simeq 31' \times 130\,pcf.$
$\simeq 4000\,psf$
$\simeq 2\,Tsf$

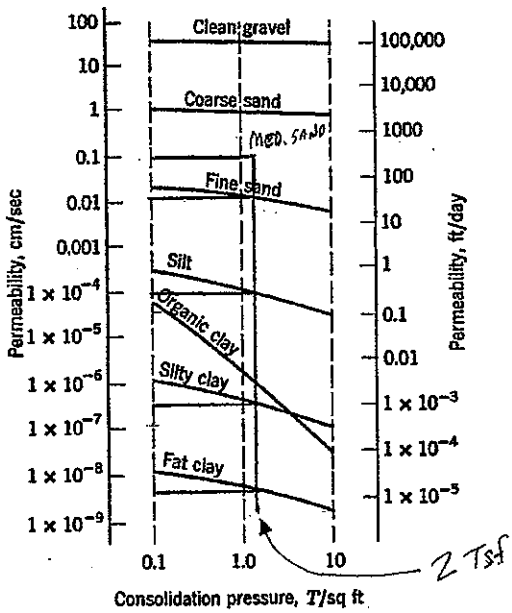


FIG. 2.10 Permeability versus consolidation pressure.

"Seepage, Drainage, and Flow Nets"
3rd Edition, Cedergren, H.R. 1989

Attachment D 1/2

$$k = \frac{Q}{iAt} \tag{2.2}$$

Darcy's *discharge velocity* multiplied by the entire cross-sectional area, including voids e and solids 1, gives the seepage quantity Q under a given hydraulic gradient $i = \Delta h/\Delta l$ or h/L. It is an imaginary velocity that does not exist anywhere. The average *seepage velocity* v_s of a mass of water progressing through the pore spaces of a soil is equal to the discharge velocity $(v_d = ki)$ multiplied by $(1 + e)/e$ or the discharge velocity divided by the effective porosity n_e; hence permeability is related to seepage velocity by the expression

$$k = \frac{v_s n_e}{i} \tag{2.3}$$

For any seepage condition in the laboratory or in the field in which the *seepage quantity*, the area perpendicular to the direction of flow, and the hydraulic gradient are known the coefficient of permeability can be calculated. Likewise, for any situation where the *seepage velocity* is known at a point at which the hydraulic gradient and soil porosity also are known, permeability can be calculated.

Experimentally determined coefficients of permeability can be combined with prescribed hydraulic gradients and discharge areas in solving practical problems involving seepage quantities and velocities. When a coefficient of permeability has been properly determined, it furnishes a very important factor in the analysis of seepage and in the design of drainage features for engineering works.

The coefficient of permeability as used in this book and in soil mechanics in general should be distinguished from the physicists' coefficient of permeability K, which is a more general term than the engineers' coefficient and has units of centimeters squared rather than a velocity; it varies with the porosity of the soil but is independent of the viscosity and density of the fluid. The transmissibility factor T represents the capability of an aquifer to discharge water and is the product of permeability k and aquifer thickness t.

The engineers' coefficient, which is used in practical problems of seepage through masses of earth and other porous media, applies only to the flow of water and is a simplification introduced purely from the standpoint of convenience. It has units of a velocity and is expressed in centimeters per second, feet per minute, feet per day, or feet per year, depending on the habits and personal preferences of individuals using the coefficient. In standard soil mechanics terminology k is expressed in centimeters per second.

Although coefficient of permeability is often considered to be a constant for a given soil or rock, it can vary widely for a given material, depending on a number of factors. Its absolute values depend, first of all, on the properties of water, of which viscosity is the most important. For individual materials

Attachment D, 2/2

Cedergren, "Seepage, Drainage, and Flow Nets", 3rd Ed. 1989

GDE Control Products, Inc.

Multi-FLO

Technical Properties

Multi-Flow

Product Information

Applications

Fittings
Accessories

Technical

Backfill

Installation
Drainage Guide
FAQ's

Drainage Core

Property	Test Method	Value
Thickness, inches	ASTM D-1777	1.0
Flow Rate, gpm/ft*	ASTM D-4716	29
Compressive Strength	ASTM D-1621	6000

Geotextile Filter

Property	Test Method	Value
Weight, oz/sq yd2	ASTM D-3776	4.0
Tensile Strength, lb.	ASTM D-4632	100
Elongation, %	ASTM D-4632	50
Puncture, lb.	ASTM D-4833	50
Mullen Burst, psi	ASTM D-3786	200
Trapezoidal Tear, lb.	ASTM D-4533	42
Coeffecient of Perm,cm/sec	ASTM D-4491	0.1
Flow Rate, gpm/ft2	ASTM D-4491	100
Permittivity, 1/sec	ASTM D-4491	1.8
A.O.S Max US Std Sieve	ASTM D 4751	70
UV Stability, 500 hrs., %	ASTM D-4355	70
Seam Strength, lb./ft	ASTM D-4595	100
Fungus	ASTM G-21	No Growth

* Horizontal Installation , gradient = 0.01, compressive force = 10 psi for 1(

All values given represent minimum average roll values

GDE Control Products, Inc. Laguna Hills, CA. 949-305-7117

GDE, Multi-Flow
<http: www.gdecontrol.com/Multi-Flow5.html> Attachment E 1/1

TABLE 2.12 RECOMMENDED REDUCTION FACTOR VALUES FOR USE IN EQ. (2.25a)

Application	Range of Reduction Factors				
	Soil Clogging and Blinding*	Creep Reduction of Voids	Intrusion into Voids	Chemical Clogging†	Biological Clogging
Retaining wall filters	2.0 to 4.0	1.5 to 2.0	1.0 to 1.2	1.0 to 1.2	1.0 to 1.3
Underdrain filters	5.0 to 10	1.0 to 1.5	1.0 to 1.2	1.2 to 1.5	2.0 to 4.0
Erosion-control filters	2.0 to 10	1.0 to 1.5	1.0 to 1.2	1.0 to 1.2	2.0 to 4.0
Landfill filters	5.0 to 10	1.5 to 2.0	1.0 to 1.2	1.2 to 1.5	5 to 10‡
Gravity drainage	2.0 to 4.0	2.0 to 3.0	1.0 to 1.2	1.2 to 1.5	1.2 to 1.5
Pressure drainage	2.0 to 3.0	2.0 to 3.0	1.0 to 1.2	1.1 to 1.3	1.1 to 1.3

*If stone riprap or concrete blocks cover the surface of the geotextile, use either the upper values or include an additional reduction factor.

†Values can be higher particularly for high alkalinity groundwater.

‡Values can be higher for turbidity and/or for microorganism contents greater than 5000 mg/l.

$$q_{\text{allow}} = q_{\text{ult}}\left(\frac{1}{\Pi\text{RF}}\right) \tag{2.25b}$$

where

q_{allow} = allowable flow rate,
q_{ult} = ultimate flow rate,
RF_{SCB} = reduction factor for soil clogging and blinding,
RF_{CR} = reduction factor for creep reduction of void space,
RF_{IN} = reduction factor for adjacent materials intruding into geotextile's void space,
RF_{CC} = reduction factor for chemical clogging,
RF_{BC} = reduction factor for biological clogging, and
ΠRF = value of cumulative reduction factors.

As with Eqs. (2.24) for strength reduction, this flow-reduction equation could also have included additional site-specific terms, such as blocking of a portion of the geotextile's surface by riprap or concrete blocks.

2.5 DESIGNING FOR SEPARATION

Application areas for geotextiles used for the separation function were given in Section 1.3.3. There are many specific applications, and it could be said, in a general sense, that geotextiles always serve a separation function. If they do not also serve this function, any other function, including the primary one, will not be served properly. This should not give the impression that the geotextile function of separation always plays a secondary role. Many situations call for separation only, and in such cases the geotextiles serve a significant and worthwhile function.

ATTACHMENT F 1/4

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.

$$q_{allow} = q_{ult}\left[\frac{1}{IIRF}\right] \tag{4.6}$$

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

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
IIRF = 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.

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The above formula can be readily converted to flow rate, Q, by multiplying the velocity by the cross-sectional area A of the pipe.

For pipelines that are either flowing full or flowing partially full, the *Manning equation* is generally used.

$$V = \frac{1}{n} R_H^{0.66} S^{0.5} \tag{7.10}$$

where

V = velocity of flow (m/s),
R_H = hydraulic radius (m),
S = slope or gradient of pipeline (m/m), and
n = coefficient of roughness (see Table 7.7) (dimensionless).

Note that plastic pipe of the type discussed in this chapter, with a *smooth interior*, has a Manning coefficient from 0.009 to 0.010. Plastic pipe with a *profiled or corrugated interior* has a Manning coefficient ranging from 0.018 to 0.025.

Eqs. (7.9) and (7.10) are generally used in the form of charts or nomographs to determine pipe sizes, flow velocity or discharge flow rates (see Figures 7.6 and 7.7). For each chart we include an example from Hwang [7], illustrated on the respective nomographs by heavy lines. Note that both nomographs are for pipes flowing full.

Example 7.1

A 100 m long pipe with D = 200 mm and C = 120 carries a discharge of 30 l/s. Determine the head loss in the pipe. (See the Hazen-Williams chart in Figure 7.6.)

Solution: Applying the conditions given to the solution chart in Figure 7.6, the energy gradient is obtained.

$$S = 0.0058 \text{ m/m}$$

TABLE 7.7 VALUES OF MANNING ROUGHNESS COEFFICIENT, *N*, FOR REPRESENTATIVE SURFACES

Type of Pipe Surface	Representative Roughness
* Lucite, glass, or plastic*	0.010
Wood or finished concrete	[illegible]
Unfinished concrete, well-laid brickwork, concrete or cast iron pipe	[illegible]
Riveted or spiral steel pipe	[illegible]
Smooth, uniform earth channel	[illegible]
Corrugated flumes, typical canals, river free from large stones and heavy weeds	[illegible]
Canals and rivers with many stones and weeds	[illegible]

*The table does not distinguish between different types of plastic, or between smooth wall [illegible] pipes with perforations.

Source: After Fox and McDonald [9].

Koerner, R.M., "Designing with Geosynthetics," 4th Ed., 1999.

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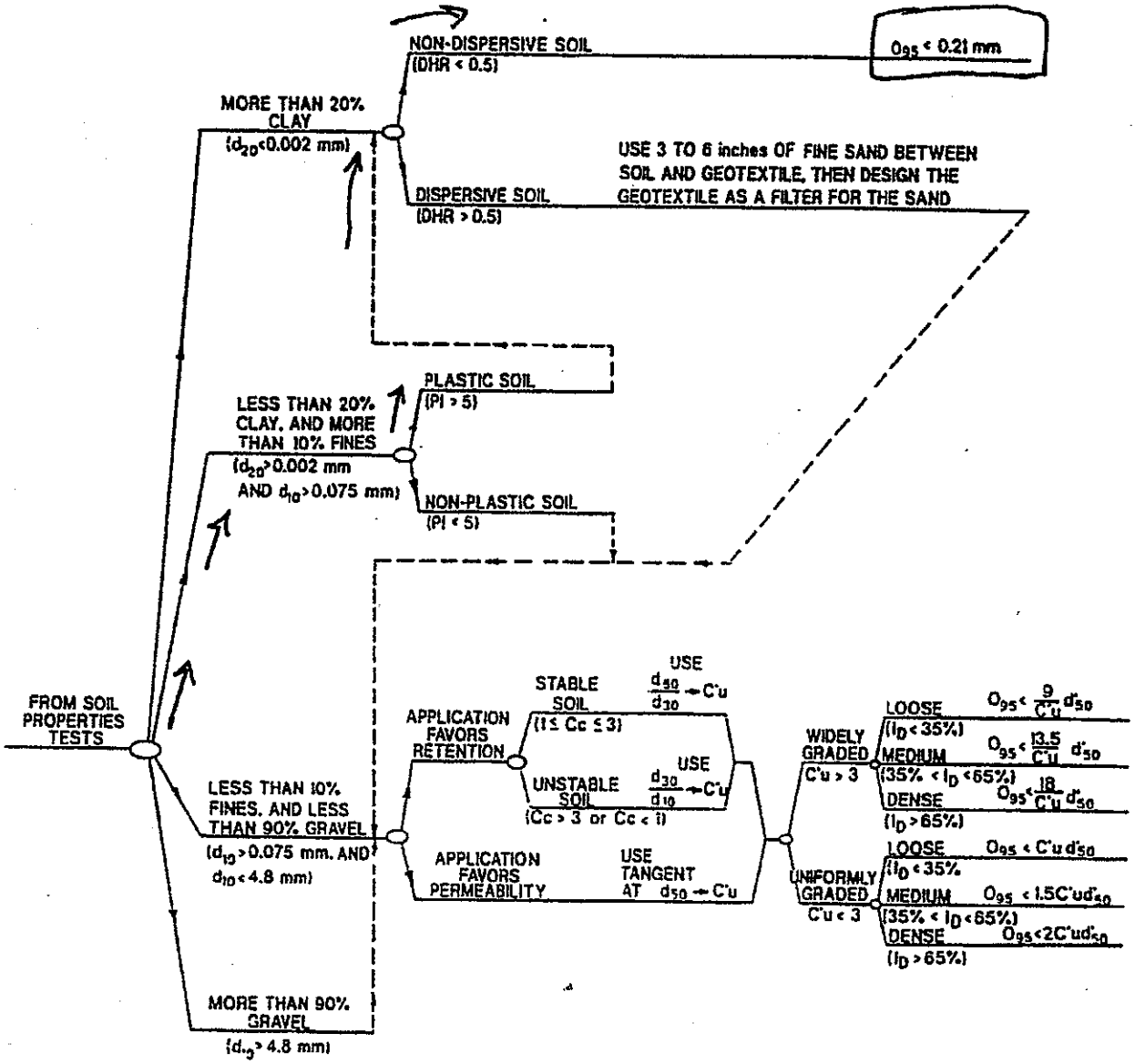
CHART 1
SOIL RETENTION CRITERIA
FOR STEADY-STATE FLOW CONDITIONS



NOTES:

d_x is the particle size of which x percent is smaller

$C'u = \sqrt{\frac{d'_{100}}{d'_0}}$ where: d'_{100} and d'_0 are the extremities of a straight line drawn through the particle-size distribution, as directed above; and d'_{50} is the midpoint of this line.

$Cc = \frac{(d_{30})^2}{d_{50} \times d_{10}}$

I_D is the relative denisty of the soil
PI is the plasticity index of the soil
DHR is the double-hydrometer ratio of the soil
Portions of this flow chart modified from Giroud (1988)

Source: Luettich, S.M., Giroud, J.P., and Bachus, R.C. (1991). "Geotextile Filter Design Manual". Report prepared for Nicolon Corporation, Norcross, Georgia.

Attachment G 1/3

4.2 Define the Hydraulic Gradient for the Application (i_s)

The hydraulic gradient will vary depending on the application of the filter. Anticipated hydraulic gradients for various applications may be estimated using Figure 3.

4.3 Determine the Minimum Allowable Geotextile Permeability (k_g)

After determining the soil hydraulic conductivity and the hydraulic gradient, the following equation can be used to determine the minimum allowable geotextile permeability [Giroud, 1988]:

$$k_g > i_s k_s$$

The hydraulic conductivity (permeability) of the geotextile can be calculated from the permittivity test method ASTM D 4491; this value can often be obtained from the manufacturer's literature as well. The geotextile permeability is defined as the product of the permittivity, ψ, and the geotextile thickness, t_g:

$$k_g > \varphi t_g$$

STEP 5. DETERMINE ANTI-CLOGGING REQUIREMENTS

To minimize the risk of clogging, the following criteria should be met:

- Use the largest opening size (O_{95}) that satisfies the retention criteria.
- For nonwoven geotextiles, use the largest porosity available, but not less than 30 percent.
- For woven geotextiles, use the largest percent open area available, but not less than 4 percent.

Source: Luettich, S.M., Giroud, J.P., and Bachus, R.C. (1991). "Geotextile Filter Design Manual". Report prepared for Nicolon Corporation, Norcross, Georgia.

Attachment
G 2/3

Table 4-5

Typical Hydraulic Gradients[a]

DRAINAGE APPLICATION	TYPICAL HYDRAULIC GRADIENT
Standard Dewatering Trench	1.0
Vertical Wall Drain	1.5
Pavement Edge Drain	1[b]
Landfill LCDRS	1.5
Landfill LCRS	1.5
Landfill SWCRS	1.5
Inland Channel Protection	1[b]
Shoreline Protection	10[b]
Dams	10[b]
Liquid Impoundments	10[b]

NOTES: [a] Table developed after Giroud [1988].

[b] Critical applications may require designing with higher gradients than those given.

Attachment G
3/3

AMOCO WASTE RELATED GEOTEXTILES

MINIMUM PHYSICAL PROPERTIES
(Minimum Average Roll Values)

Property	Test Method	Units	4504	4506	4508	4510	4512	4516
Unit Weight	ASTM D-3776	Oz./yd.²	4.0 *	6.0	8.0	10.0	12.0	16.0
Grab Tensile	ASTM D-4632	lbs.	95	150	200	235	275	350
Grab Elongation	ASTM D-4632	%	50	50	50	50	50	50
Mullen Burst	ASTM D-3787	psi	225	350	450	560	650	750
Puncture	ASTM D-4833	lbs.	55	90	130	165	185	220
Trapezoid Tear	ASTM D-4533	lbs.	35	65	80	95	115	130
Apparent Opening Size	ASTM D-4751	US Sieve Number	70	70	100	100	100	100
Permittivity	ASTM D-4491	gal/min/ft² sec⁻¹	100 2.0	90 1.7	80 1.5	70 1.1	60 0.9	50 0.7
Permeability	ASTM D-4491	cm/sec	.2	.2	.2	.2	.2	.2
Thickness	ASTM D-1777	mils	40 *	65	90	110	130	175
U.V. Resistance	ASTM D-4355[1]	%[2]	70	70	70	70	70	70

1. Fabric conditioned per ASTM D-4355 2. Percent of minimum grab tensile after conditioning.

TYPICAL PHYSICAL PROPERTIES

Property	Test Method	Units	4504	4506	4508	4510	4512	4516
Grab Tensile	ASTM D-4632	lbs.	130/115	225/200	275/270	315/310	410/370	510/470
Grab Elongation	ASTM D-4632	%	75	65	65	65	65	65
Mullen Burst	ASTM D-3786	psi	285	410	575	650	825	920
Puncture	ASTM D-4833	lbs.	75	120	170	190	210	270
Trapezoid Tear	ASTM D-4533	lbs.	60/50	100/80	140/120	160/140	185/155	220/180
Apparent Opening Size	ASTM D-4751	US Sieve Number	70/120	70/140	100/200	100+	100+	100+
Permittivity	ASTM D-4491	gal/min/ft² sec⁻¹	150 3.1	110 2.0	100 1.8	80 1.5	70 1.3	60 1.0
Permeability	ASTM D-4491	cm/sec	.35	.31	.27	.26	.25	.23
Thickness	ASTM D-1777	mils	50	75	115	130	160	195

PACKAGING

Dimensions		4504	4506	4508	4510	4512	4516
Roll Width	ft.	15	15	15	15	15	15
Roll Length	ft.	1200	900	600	600	450	300
Gross Weight	lbs.	500	550	500	600	550	500
Area	sq. yds.	2000	1500	1000	1000	750	500

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Amoco Fabrics and Fibers Company 900 Circle 75 Parkway, Suite 300 Atlanta, Georgi

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Code #19100512;28.060;6/91

Amoco Fabrics & Fibers Company, 1991. "Amoco Waste Related Geotextiles" H 1/1

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******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     C:\HLP3\IUC\IUC30.D4
TEMPERATURE DATA FILE:       C:\HLP3\IUC\IUC30.D7
SOLAR RADIATION DATA FILE:   C:\HLP3\IUC\IUC30.D13
EVAPOTRANSPIRATION DATA:     C:\HLP3\IUC\IUC30.D11
SOIL AND DESIGN DATA FILE:   C:\HLP3\IUC\Cell4B.D10
OUTPUT DATA FILE:            C:\HLP3\IUC\Cell4B.OUT


TIME:  12:18     DATE:   8/30/2007


******************************************************************************

     TITLE:  IUC Slimes Drain Analysis Cell 4B

******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                  LAYER  1
                                  --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER   0
          THICKNESS                    =     36.00   INCHES
          POROSITY                     =      0.4700 VOL/VOL
          FIELD CAPACITY               =      0.2220 VOL/VOL
          WILTING POINT                =      0.1000 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.1980 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.520000001000E-03 CM/SEC


                                  LAYER  2
                                  --------

                     TYPE 2 - LATERAL DRAINAGE LAYER
```

MATERIAL TEXTURE NUMBER 0

THICKNESS	=	6.00	INCHES
POROSITY	=	0.4700	VOL/VOL
FIELD CAPACITY	=	0.2220	VOL/VOL
WILTING POINT	=	0.1040	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2220	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.520000001000E-03	CM/SEC
SLOPE	=	1.00	PERCENT
DRAINAGE LENGTH	=	75.0	FEET

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 7 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 1.% AND A SLOPE LENGTH OF 75. FEET.

SCS RUNOFF CURVE NUMBER	=	88.80	
FRACTION OF AREA ALLOWING RUNOFF	=	0.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	16.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.689	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	7.520	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.600	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	8.458	INCHES
TOTAL INITIAL WATER	=	8.458	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM GRAND JUNCTION COLORADO

STATION LATITUDE	=	39.07	DEGREES
MAXIMUM LEAF AREA INDEX	=	1.00	
START OF GROWING SEASON (JULIAN DATE)	=	109	
END OF GROWING SEASON (JULIAN DATE)	=	293	
EVAPORATIVE ZONE DEPTH	=	16.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.10	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	60.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	36.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	36.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	57.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR GRAND JUNCTION COLORADO

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC

0.64	0.54	0.75	0.71	0.76	0.44
0.47	0.91	0.70	0.87	0.63	0.58

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR GRAND JUNCTION COLORADO

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
25.50	33.50	41.90	51.70	62.10	72.30
78.90	75.90	67.10	54.90	39.60	28.30

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR GRAND JUNCTION COLORADO
AND STATION LATITUDE = 39.07 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.44	0.44	0.65	0.81	0.75	0.52
	0.39	1.08	0.58	1.00	0.94	0.54
STD. DEVIATIONS	0.23	0.30	0.31	0.44	0.53	0.63
	0.30	0.48	0.44	0.63	0.52	0.31
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.441	0.542	0.627	0.714	0.941	1.152
	0.511	0.980	0.481	0.737	0.589	0.454
STD. DEVIATIONS	0.213	0.272	0.280	0.352	0.544	0.560
	0.398	0.510	0.395	0.637	0.250	0.224
PERCOLATION/LEAKAGE THROUGH LAYER 2						
TOTALS	0.0000	0.0000	0.0009	0.0011	0.0002	0.0008
	0.0003	0.0000	0.0000	0.0000	0.0010	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0020	0.0027	0.0008	0.0019

0.0009	0.0000	0.0000	0.0000	0.0032	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	INCHES		CU. FEET	PERCENT
PRECIPITATION	8.16	(1.320)	29628.1	100.00
RUNOFF	0.000	(0.0000)	0.00	0.000
EVAPOTRANSPIRATION	8.168	(1.5732)	29651.33	100.079
PERCOLATION/LEAKAGE THROUGH LAYER 2	0.00442	(0.00727)	16.060	0.05420
CHANGE IN WATER STORAGE	-0.011	(0.7860)	-39.33	-0.133

****

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	0.86	3121.800
RUNOFF	0.000	0.0000
PERCOLATION/LEAKAGE THROUGH LAYER 2	0.003089	11.21440
SNOW WATER	0.72	2615.3926

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.2271

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1000

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****

FINAL WATER STORAGE AT END OF YEAR 10

LAYER	(INCHES)	(VOL/VOL)
1	6.9017	0.1917
2	1.3320	0.2220

SNOW WATER 0.116

****
****

Geosyntec consultants

COMPUTATION COVER SHEET

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

Title of Computations: Side Slope Riser Trench Stability Evaluation- Veneer Stability

Computations by:	Signature		12/5/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		12/5/07
	Printed Name	Gregory T Corcoran	Date
	Title	Principal Engineer	
Computations Checked by:	Signature		12/5/07
	Printed Name	Gregory T Corcoran	Date
	Title	Principal Engineer	
Computations backchecked by: (originator)	Signature		12/5/07
	Printed Name	Rebecca Flynn	Date
	Title	Senior Staff Engineer	
Approved by: (pm or designate)	Signature		12/5/07
	Printed Name	Gregory T Corcoran	Date
	Title	Principal Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	R. Flynn	Date:	12/04/07	Reviewed by:	G. Corcoran	Date:	12/4/07
Client:	DMC	Project:	WMM- Cell 4B	Project/ Proposal No.:	SC0349	Task No.:	02

SIDE SLOPE RISER TRENCH STABILITY EVALUATION
VENEER STABILITY OF GEOSYNTHETIC-SOIL LINED SLOPES

OBJECTIVE

To evaluate the tension developed within the geosynthetic-soil layered side slope riser trench liner system of the tailings pond liner system of the White Mesa Mill Cell 4B in Blanding, Utah.

SUMMARY OF RESULTS

The calculations suggest that a minimum peak geosynthetic interface friction angle of 31 degrees and 0 psf peak adhesion is required to prevent the development of tension in the geosynthetic components of the side slope riser trench liner system. A review of the literature indicates that achieving shear strength consistent with the design basis outlined in the following table is obtainable.

Peak Shear Strength

Friction angle (degrees)	Adhesion (psf)
31	0
26	25
20	50
14	75
8	100
0	135

METHOD OF ANALYSIS

The stability analysis of the geosynthetic-soil layered systems was carried out using the approach outlined by Koerner and Soong [1998] (Attachment A). This approach calculates the driving force of an active soil wedge along a geosynthetic-soil layered side slope and compares it to the resisting force of the complementary passive

soil wedge to evaluate the overall factor of safety against failure. The method presented by Koerner and Soong [1998] allows for the consideration of a uniform depth soil layer and the influence of dynamic equipment loading.

SIDE SLOPE RISER TRENCH LINER SYSTEM

The side slope liner system consists of, from top to bottom (Figure 1):

- 2 ft gravel;
- Cushion Geotextile;
- 60-mil HDPE Geomembrane, textured;
- Cushion Geotextile;
- 60-mil HDPE Geomembrane, textured;
- Geosynthetic clay liner (GCL) consisting of bentonite sandwiched between a top woven geotextile and a bottom nonwoven geotextile needle-punched together; and
- Prepared subgrade.

The side slope inclination is a maximum of 2.0H:1.0V. The maximum height of the side slope riser trench is 40 vertical feet, located in the southeast corner of Cell 4B.

MATERIAL PARAMETERS

Based on a review of potential liner interfaces, the likely critical interface has been identified as the textured HDPE geomembrane and the woven component of the GCL (in accordance with the technical specifications, the woven side of the GCL is to be installed against the overlying secondary geomembrane while the nonwoven component is installed against the underlying subgrade). Based on our experience and available laboratory data (see Attachment B), this interface has been estimated to have a shear strength of approximately 25 degrees and 100 psf adhesion.

Geosynthetic Interface:

The geosynthetic interface friction angle was varied to evaluate the minimum allowable value to obtain no tension at a minimum factor of safety value of 1.3. A literature review was performed to evaluate if the calculated minimum allowable interface friction value is achievable.

REVIEW OF REPORTED INTERFACE STRENGTHS IN THE LITERATURE

The following values for the interface friction between the geosynthetic and soil components of the liner system represent values reported in the literature:

Gravel (GP)	38	NAVFAC(1), 0 psf cohesion (Attachment C)
NW geotextile to Textured HDPE	34	4th GRI Seminar (Attachment D)
Textured HDPE to Hydrated GCL (woven)	25	Bentomat(3), 100 psf adhesion (Attachment B)
Internal GCL	35	Bentomat(4), 350 psf adhesion (Attachment B)
Hydrated GCL to Subgrade	28	Bentomat(5), 186 psf adhesion (Attachment B)

1. NAVFAC (1982) lists typical shear strength values for various soils based on 100 percent standard Proctor compaction. Actual construction materials would likely be placed at 90 percent of the modified Proctor compaction. To be conservative, a value of phi = 32 degree was used.

2. Values of peak friction angle reported for interface shear strength between 60 mil textured HDPE geomembrane and woven geotextile side of CETCO Bentomat GCL under hydrated conditions at a shearing rate of 0.04 in/min. Normal stresses of 1.0, 3.0, and 6.0 psi yielded a friction angle of 25 degrees, 100 psf adhesion. A normal stress of 15 psi yielded a friction angle of 19 degrees.

3. Values of peak friction angle reported for internal shear strength of CETCO Bentomat GCL under hydrated conditions at a shearing rate of 0.04 in/min. Normal stresses of 0.7, 1.4, and 2.8 psi yielded a friction angle of 35 degrees, 350 psf adhesion.

4. Values of peak friction angle reported for interface shear strength between clayey sand subgrade soil and nonwoven geotextile side of CETCO Bentomat GCL under hydrated conditions at a shearing rate of 0.04 in/min. Normal stresses of 1.5, 3.5, and 7.0 psi yielded a friction angle of 27.8 degrees, 186 psf adhesion.

DESIGN CRITERION

For the geosynthetic-soil lined slopes of the Cell 4B liner system it was desired to evaluate inclination which would introduce no geosynthetic tension. Subsequently, zero geosynthetic tension was established as the design criterion for veneer stability of the geosynthetic-soil lined side slopes.

In consideration of the significance of no tension in the geosynthetic liner system and consistent with current practice, Geosyntec will adopt a factor of safety (FS) equal to or greater than 1.3 for veneer stability.

ANALYSIS

According to the Koerner and Soong approach, a soil veneer on a side slope is stable when the resultant driving force on the passive wedge (E_P) is equal to the resultant resistant force on the active wedge (E_A). The following equations represent the resultant resistance and active forces, respectively:

$$E_A = \frac{FS(W_A - N_A \cos\beta) - (N_A \tan\delta + C_a)\sin\beta}{\sin\beta(FS)}$$ (Attachment A, 2 of 6)

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta \tan\phi}$$ (Attachment A, 2 of 6)

The variables will be defined in a subsequent section.

By setting E_A=E_P, the resulting equation may be arranged in the form of the quadratic equation $ax^2+bx+c=0$. Considering FS as the variable of interest, the resulting equation is as follows:

$$a(FS)^2 + b(FS) + c = 0$$ (Attachment A, 2 of 6)

The factor of safety may be obtained from the solution of the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$ (Attachment A, 2 of 6)

Where the constants are defined, as in Attachment A, 6 of 6:

$$a = (W_A - N_A \cos\beta)\cos\beta \quad (1)$$

$$b = -[(W_A - N_A \cos\beta)\sin\beta \tan\phi + (N_A \tan\delta + C_a)\sin\beta\cos\beta + \sin\beta(C + W_P \tan\phi)] \quad (2)$$

$$c = (N_A \tan\delta + C_a)\sin^2\beta \tan\phi \quad (3)$$

In which the variables are indicated on Figure 1 and defined as follows:

$$C = \frac{c \cdot h}{\sin\beta} \quad (4)$$ (Attachment A, 2 of 6)

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) \quad (5)$$ (Attachment A, 2 of 6)

$$W_A = \gamma \cdot h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) \quad (6)$$ (Attachment A, 2 of 6)

$$N_A = W_A \cos\beta \quad (7)$$ (Attachment A, 2 of 6)

$$W_P = \frac{\gamma \cdot h^2}{\sin 2\beta} \quad (8)$$ (Attachment A, 2 of 6)

β = soil slope angle beneath the geomembrane, **26.6°** for 2H:1V slope
δ = minimum interface friction angle of side slope liner system, **25°, 100 psf adhesion**
ϕ = friction angle of gravel, **38°**
γ = unit weight of the gravel, **127 pcf**
C = cohesive force along the failure plane of the passive wedge
c = cohesion of the gravel, **0 psf**
C_a = adhesive force
c_a = adhesion between the GCL and the geomembrane, **100 psf**
h = thickness of the gravel, **2 feet**
L = length of slope measured along the geomembrane beneath gravel, **89.4 feet**
N_A = effective force normal to the failure plane of the active wedge
W_A = total weight of the active wedge
W_P = total weight of the passive wedge

Substituting the variables and solving equations (4)-(8), above:

$$C = \frac{c \cdot h}{\sin\beta} = \frac{0 \cdot 2}{\sin(26.6)} = \mathbf{0\ lbs/ft} \quad (4)$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) = 100\left(89.4 - \frac{2}{\sin 26.6}\right) = \mathbf{8496\ lbs/ft} \quad (5)$$

$$W_A = \gamma \cdot h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) = 127 \cdot 2^2\left(\frac{89.4}{2} - \frac{1}{\sin 26.6} - \frac{\tan 26.6}{2}\right) = \mathbf{21{,}452\ lbs/ft} \quad (6)$$

$$N_A = W_A \cos\beta = 21{,}452 \text{ x } \cos 26.6 = \mathbf{19{,}187\ lbs/ft} \quad (7)$$

$$W_P = \frac{\gamma \cdot h^2}{\sin 2\beta} = \mathbf{635\ lbs/ft} \quad (8)$$

Next, substituting the solutions to equations (4)-(8) into equations (1)-(3):

a= (21,452-19,187 x cos26.6) = **3,839 lbs/ft** (1)

b= -[(21,452 – 19,187 x cos 26.6) sin 26.6 tan 38
+ (21,452 tan 19 + 100) sin 26.6 cos 26.6
+ sin 26.6 (0 + 635 tan 38) = **-8,700 lbs/ft** (2)

c = (21,452 tan 19 + 100) $\sin^2$ 26.6 tan 38 = **2,727 lbs/ft** (3)

Finally, inserting the solutions to (1), (2) and (3) and solving the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} = \frac{-6836 + \sqrt{6836^2 - 4 \cdot 3023 \cdot 2141}}{2 \cdot 3023} = \mathbf{1.89}$$

Therefore, the factor of safety for the veneer stability of the operations layer on a sideslope composite liner system is 1.89. This factor of safety satisfies the stability criterion of 1.3, as previously described.

RESULTS AND CONCLUSIONS

The results suggest that the proposed side slope riser trench liner system satisfies the design criteria of no geosynthetic tension development:

Soil Cover Layer Inclination (H:V)	Equipment Loading	Tension FS
2.0H:1V	NONE	1.89

Results of veneer stability analyses presented herein indicate that an apparent internal friction or interface friction angle (residual) of 19 degrees with an apparent peak cohesion of 100 psf for any component of the composite liner system is the minimum allowable value providing for a static factor of safety that satisfies that design criteria of no tension in the geosynthetic liner system.

Based on the analyses herein, results of interface shear tests on the actual materials proposed for use in the composite liner system must indicate that the weakest apparent residual friction angle of the composite liner is equal to the following:

Peak Shear Strength

Friction angle (degrees)	Adhesion (psf)
31	0
26	25
20	50
14	75
8	100
0	135

The results suggest that the following conditions satisfy the design criteria of no geosynthetic tension development:

- 2H:1V side slopes;
- 2 feet thickness of gravel on side slopes, placed to a vertical height of 40 feet; and
- Acceptable factor of safety for interim slope stability is 1.3.

REFERENCES

CETCO, 2000, Bentomat/Claymax Technical Manual, September. (*Attachment B*)

Koerner, R. K. and Soong, T.Y., [1998], "Analysis and Design of Veneer Cover Soils", Proceedings of the Sixth International Conference on Geosynthetics, Atlanta, Georgia, Vol. I, 1998. *(Attachment A)*

NAVFAC (1982), "Foundations and Earth Structures, Design Manual - 7.2," Department of Navy, Naval Facilities Engineering Command, May 1982 *(Attachment C)*

Proceedings of the 4th GRI Seminar on the Topic of Landfill Closures. GRI – Dec 14, 1990. (*Attachment D*)

Geosyntec consultants

Written By: Rebecca Flynn Date: 5-Dec-07 Reviewed by: _____ Date: _____

Client: Denison Mines (USA) Corp Project: WMM Cell 4B Project No.: SC0349

CALCULATION OF VENEER STABILITY

(1) Project Variables

Unit weight of rock (g):	127	pcf
Rock thickness at crest (hc):	2	ft
Slope Height (H):	40.0	ft
Length of Slope	89.43	ft
Friction angle of rock (f):	38	degrees
	0.66	radians
Cohesion of rock (c):	0	psf
Rock and Liner inclination (nH:1V):	2	
Slope angle	0.46	radians
Slope angle	26.6	degrees

(2) Geosynthetic Properties

Minimum friction angle :	25	degrees
	0.44	radians
Adhesion Intercept (ca)	100	psf
Product name :	**Geocomposite/Geomembrane**	
Allowable tensile strength:	90	ppi

(3) Equipment Loading

Equipment Type:	NONE	
Width of placement track, b	0.0	inches
Influence Factor, I	0.00	(see Figure 7, Attachment A)
Length of placement track, w	0.0	inches
Actual weight of equipment, Wb	0	lbs
Acceleration of equipment, a	0.00	g

(4) Analysis Output

Cohesion Force along failure plane of passive wedge (C)	0	ppf
Adhesive Force b/w operations layer of active wedge and geomembrane (Ca)	8496	ppf
Total weight of active wedge (Wa)	21452	ppf
Effective force normal to failure plane of the active wedge (Na)	19187	ppf
Total weight of passive wedge (Wp)	635	ppf
Surface stress (q)	NA	psf
Vertical force of equipment (We)	NA	ppf
Dynamic Equipment Loading, parallel to slope (Fe)	NA	ppf
Equiment force Normal to to Active Wedge (Ne)	NA	ppf

Factor of Safety (FS) 1.89

Reference:
Koerner, R. M., Soong, T. [1998]. "Analysis and Design of Veneer Cover Soils", Proceedings of the Sixth International Conference on Geosynthetics.
Programmed by: J. Soule, 2005

Geosyntec consultants

Written By: Rebecca Flynn Date: 5-Dec-07 Reviewed by: _____ Date: _____

Client: Denison Mines (USA) Corp Project: WMM Cell 4B Project No.: SC0349

CALCULATION OF VENEER STABILITY

(1) Project Variables

Unit weight of rock (g):	127	pcf
Rock thickness at crest (hc):	2	ft
Slope Height (H):	40.0	ft
Length of Slope	89.43	ft
Friction angle of rock (f):	38	degrees
	0.66	radians
Cohesion of rock (c):	0	psf
Rock and Liner inclination (nH:1V):	2	
Slope angle	0.46	radians
Slope angle	26.6	degrees

(2) Geosynthetic Properties

Minimum friction angle :	31	degrees
	0.54	radians
Adhesion Intercept (ca)	0	psf
Product name :	**Geocomposite/Geomembrane**	
Allowable tensile strength:	90	ppi

(3) Equipment Loading

Equipment Type:	NONE	
Width of placement track, b	0.0	inches
Influence Factor, I	0.00	(see Figure 7, Attachment A)
Length of placement track, w	0.0	inches
Actual weight of equipment, Wb	0	lbs
Acceleration of equipment, a	0.00	g

(4) Analysis Output

Cohesion Force along failure plane of passive wedge (C)	0	ppf
Adhesive Force b/w operations layer of active wedge and geomembrane (Ca)	0	ppf
Total weight of active wedge (Wa)	21452	ppf
Effective force normal to failure plane of the active wedge (Na)	19187	ppf
Total weight of passive wedge (Wp)	635	ppf
Surface stress (q)	NA	psf
Vertical force of equipment (We)	NA	ppf
Dynamic Equipment Loading, parallel to slope (Fe)	NA	ppf
Equiment force Normal to to Active Wedge (Ne)	NA	ppf

Factor of Safety (FS) 1.28

Reference:
Koerner, R. M., Soong, T. [1998]. "Analysis and Design of Veneer Cover Soils", Proceedings of the Sixth International Conference on Geosynthetics.
Programmed by: J. Soule, 2005

Geosyntec consultants

Written By: Rebecca Flynn Date: 5-Dec-07 Reviewed by: _____ Date: _____

Client: Denison Mines (USA) Corp Project: WMM Cell 4B Project No.: SC0349

CALCULATION OF VENEER STABILITY

(1) Project Variables

Unit weight of rock (g):	127	pcf
Rock thickness at crest (hc):	2	ft
Slope Height (H):	40.0	ft
Length of Slope	89.43	ft
Friction angle of rock (f):	38	degrees
	0.66	radians
Cohesion of rock (c):	0	psf
Rock and Liner inclination (nH:1V):	2	
Slope angle	0.46	radians
Slope angle	26.6	degrees

(2) Geosynthetic Properties

Minimum friction angle :	0	degrees
	0.00	radians
Adhesion Intercept (ca)	135	psf
Product name :	**Geocomposite/Geomembrane**	
Allowable tensile strength:	90	ppi

(3) Equipment Loading

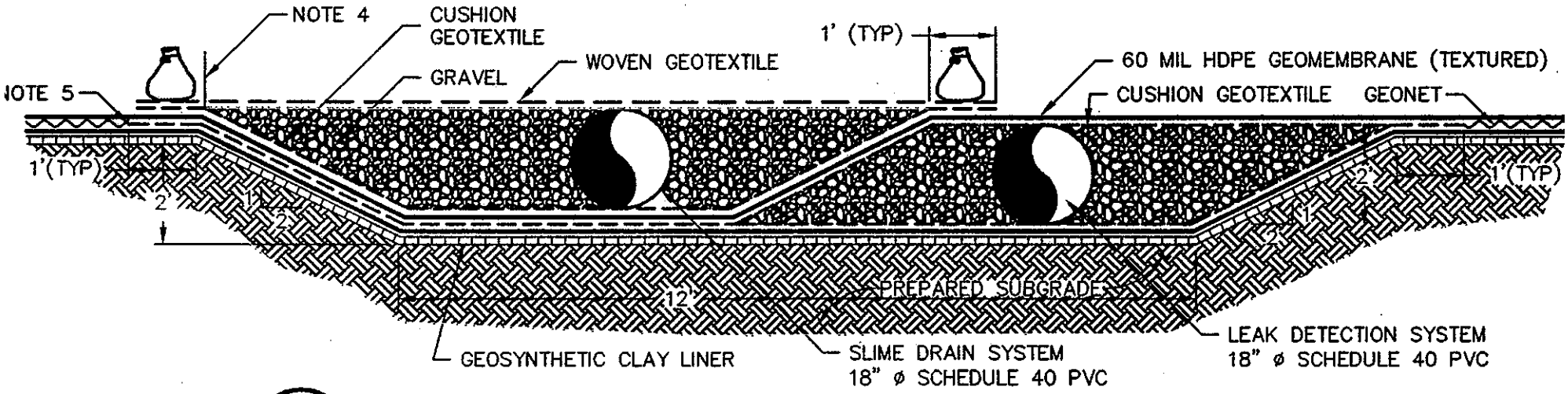
Equipment Type:	NONE	
Width of placement track, b	0.0	inches
Influence Factor, I	0.00	(see Figure 7, Attachment A)
Length of placement track, w	0.0	inches
Actual weight of equipment, Wb	0	lbs
Acceleration of equipment, a	0.00	g

(4) Analysis Output

Cohesion Force along failure plane of passive wedge (C)	0	ppf
Adhesive Force b/w operations layer of active wedge and geomembrane (Ca)	11469	ppf
Total weight of active wedge (Wa)	21452	ppf
Effective force normal to failure plane of the active wedge (Na)	19187	ppf
Total weight of passive wedge (Wp)	635	ppf
Surface stress (q)	NA	psf
Vertical force of equipment (We)	NA	ppf
Dynamic Equipment Loading, parallel to slope (Fe)	NA	ppf
Equiment force Normal to to Active Wedge (Ne)	NA	ppf

Factor of Safety (FS) 1.28

Reference:
Koerner, R. M., Soong, T. [1998]. "Analysis and Design of Veneer Cover Soils", Proceedings of the Sixth International Conference on Geosynthetics.
Programmed by: J. Soule, 2005



1/6 SECTION
SLIMES AND LEAK DETECTION SYSTEM SIDE SLOPE
N.T.S.
XREF: 0349X008.DWG

Figure 1

- The issue of appropriate normal stress is greatly complicated if gas pressures are generated in the underlying waste. These gas pressures will counteract some (or all) of the gravitational stress of the cover soil. The resulting shear strength, and subsequent stability, can be significantly decreased. See Liu et al (1997) for insight into this possibility.
- Shear rates necessary to attain drained conditions (if this is the desired situation) are extremely slow, requiring long testing times.
- Deformations necessary to attain residual strengths require large relative movement of the two respective halves of the shear box. So as not to travel over the edges of the opposing shear box sections, devices should have the lower shear box significantly longer than 300 mm. However, with a lower shear box longer than the upper traveling section, new surface is constantly being added to the shearing plane. This influence is not clear in the material's response or in the subsequent behavior.
- The attainment of a true residual strength is difficult to achieve. ASTM D5321 states that one should "run the test until the applied shear force remains constant with increasing displacement". Many commercially available shear boxes have insufficient travel to reach this condition.
- The ring torsion shearing apparatus is an alternative device to determine true residual strength values, but is not without its own problems. Some outstanding issues are the small specimen size, nonuniform shear rates along the width of the specimen, anisotropic shearing with some geosynthetics and no standardized testing protocol. See Stark and Poeppel (1994) for information and data using this alternative test method.

2.3 Various Types of Loadings

There are a large variety of slope stability problems that may be encountered in analyzing and/or designing final covers of engineered landfills, abandoned dumps and remediation sites as well as leachate collection soils covering geomembranes beneath the waste. Perhaps the most common situation is a uniformly thick cover soil on a geomembrane placed over the soil subgrade at a given and constant slope angle. This "standard" problem will be analyzed in the next section. A variation of this problem will include equipment loads used during placement of cover soil on the geomembrane. This problem will be solved with equipment moving up the slope and then moving down the slope.

Unfortunately, cover soil slides have occurred and it is felt that the majority of the slides have been associated with seepage forces. Indeed, drainage above a geomembrane (or other barrier material) in the cover soil cross section must be accommodated to avoid the possibility of seepage forces. A section will be devoted to this class of slope stability problems.

Lastly, the possibility of seismic forces exists in earthquake prone locations. If an earthquake occurs in the vicinity of an engineered landfill, abandoned dump or remediation site, the seismic wave travels through the solid waste mass reaching the upper surface of the cover. It then decouples from the cover soil materials, producing a horizontal force which must be appropriately analyzed. A section will be devoted to the seismic aspects of cover soil slope analysis as well.

All of the above actions are destabilizing forces tending to cause slope instability. Fortunately, there are a number of actions that can be taken to increase the stability of slopes.

Other than geometrically redesigning the slope with a flatter slope angle or shorter slope length, a designer can add soil mass at the toe of the slope thereby enhancing stability. Both toe berms and tapered soil covers are available options and will be analyzed accordingly. Alternatively, the designer can always use geogrids or high strength geotextiles within the cover soil acting as reinforcement materials. This technique is usually referred to as veneer reinforcement. Cases of both intentional and nonintentional veneer reinforcement will be presented.

Thus it is seen that a number of strategies influence slope stability. Each will be described in the sections to follow. First, the basic gravitational problem will be presented followed by those additional loading situations which tend to decrease slope stability. Second, various actions that can be taken by the designer to increase slope stability will be presented. The summary will contrast the FS-values obtained in the similarly crafted numeric examples.

3 SITUATIONS CAUSING DESTABILIZATION OF SLOPES

This section treats the standard veneer slope stability problem and then superimposes upon it a number of situations, all of which tend to destabilize slopes. Included are gravitational, construction equipment, seepage and seismic forces. Each will be illustrated by a design graph and a numeric example.

3.1 Cover Soil (Gravitational) Forces

Figure 3 illustrates the common situation of a *finite* length, uniformly thick cover soil placed over a liner material at a slope angle "β". It includes a passive wedge at the toe and has a tension crack of the crest. The analysis that follows is after Koerner and Hwu (1991), but comparable analyses are available from Giroud and Beech (1989), McKelvey and Deutsch (1991), Ling and Leshchinsky (1997) and others.

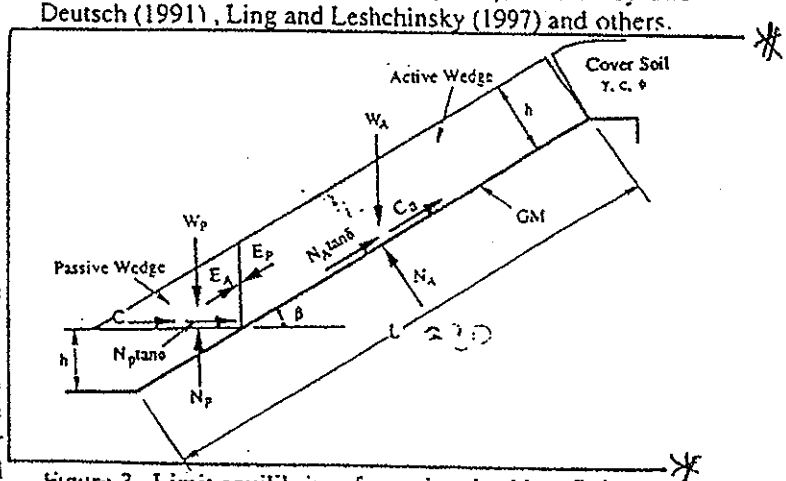


Figure 3. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil.

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The symbols used in Figure 3 are defined below.

W_A = total weight of the active wedge
W_P = total weight of the passive wedge
N_A = effective force normal to the failure plane of the active wedge
N_P = effective force normal to the failure plane of the passive wedge
γ = unit weight of the cover soil
h = thickness of the cover soil
L = length of slope measured along the geomembrane
β = soil slope angle beneath the geomembrane
ϕ = friction angle of the cover soil
δ = interface friction angle between cover soil and geomembrane
C_a = adhesive force between cover soil of the active wedge and the geomembrane
c_a = adhesion between cover soil of the active wedge and the geomembrane
C = cohesive force along the failure plane of the passive wedge
c = cohesion of the cover soil
E_A = interwedge force acting on the active wedge from the passive wedge
E_P = interwedge force acting on the passive wedge from the active wedge
FS = factor of safety against cover soil sliding on the geomembrane

The expression for determining the factor of safety can be derived as follows:

Considering the active wedge,

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) \quad (3)$$

$$N_A = W_A \cos\beta \quad (4)$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) \quad (5)$$

By balancing the forces in the vertical direction, the following formulation results:

$$E_A \sin\beta = W_A - N_A \cos\beta - \frac{N_A \tan\delta + C_a}{FS}\sin\beta \quad (6)$$

Hence the interwedge force acting on the active wedge is:

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)} \quad (7)$$

The passive wedge can be considered in a similar manner:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} \quad (8)$$

$$N_P = W_P + E_P \sin\beta \quad (9)$$

$$C = \frac{(c)(h)}{\sin\beta} \quad (10)$$

By balancing the forces in the horizontal direction, the following formulation results:

$$E_P \cos\beta = \frac{C + N_P \tan\phi}{FS} \quad (11)$$

Hence the interwedge force acting on the passive wedge is:

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} \quad (12)$$

By setting $E_A = E_P$, the resulting equation can be arranged in the form of the quadratic equation $ax^2 + bx + c = 0$ which in our case, using FS-values, is:

$$a(FS)^2 + b(FS) + c = 0 \quad (13)$$

where

$$a = (W_A - N_A\cos\beta)\cos\beta$$

$$b = -[(W_A - N_A\cos\beta)\sin\beta\tan\phi + (N_A\tan\delta + C_a)\sin\beta\cos\beta + \sin\beta(C + W_P\tan\phi)]$$

$$c = (N_A\tan\delta + C_a)\sin^2\beta\tan\phi \quad (14)$$

The resulting FS-value is then obtained from the solution of the quadratic equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \quad (15)$$

When the calculated FS-value falls below 1.0, sliding of the cover soil on the geomembrane is to be anticipated. Thus a value of greater than 1.0 must be targeted as being the minimum factor of safety. How much greater than 1.0 the FS-value should be, is a design and/or regulatory issue. The issue of minimum allowable FS-values under different conditions will be assessed at the end of the paper. In order to better illustrate the implications of Eqs. 13, 14 and 15, typical design curves for various FS-values as a function of slope angle and interface friction angle are given in Figure 4. Note that the curves are developed specifically for the variables stated in the legend of the figure. Example 1 illustrates the use of the curves in what will be the standard example to which other examples will be compared.

Example 1:
Given a 30 m long slope with a uniformly thick 300 mm cover soil at a unit weight of 18 kN/m³. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The cover soil is placed directly on a geomembrane as shown in Figure 3. Direct shear testing has resulted in a interface friction angle between the cover soil and geomembrane of 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg?

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Solution:

Substituting Eq. 14 into Eq. 15 and solving for the FS-value results in the following which is seen to be in agreement with the curves of Figure 4.

$$\left.\begin{aligned} a &= 14.7\,kN/m \\ b &= -21.3\,kN/m \\ c &= 3.5\,kN/m \end{aligned}\right\} FS = 1.25$$

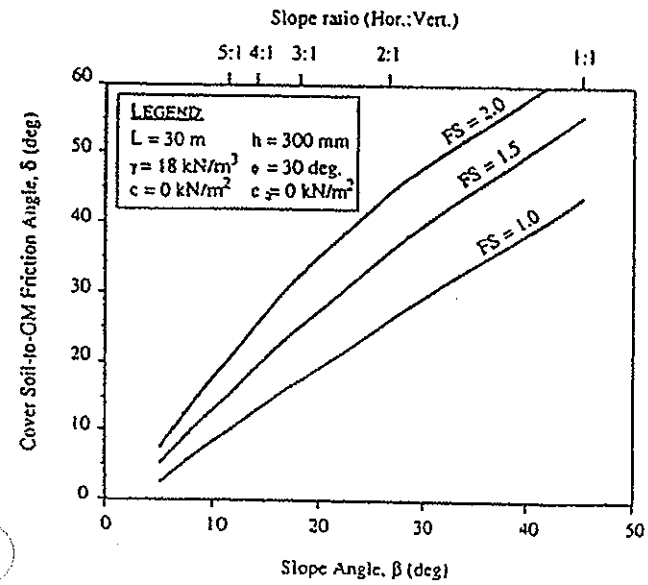


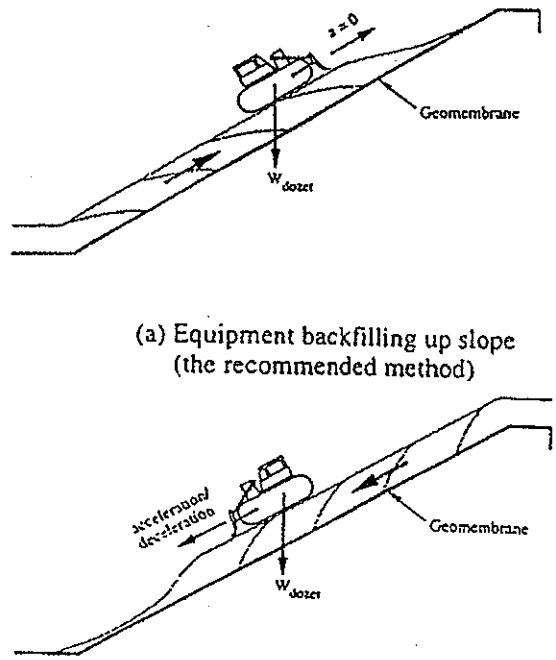
Figure 4. Design curves for stability of uniform thickness cohesionless cover soils on linear failure planes for various global factors-of-safety.

Comment:
In general, this is too low of a value for a final cover soil factor-of-safety and a redesign is necessary. While there are many possible options of changing the geometry of the situation, the example will be revisited later in this section using toe berms, tapered cover soil thickness and veneer reinforcement. Furthermore, this general problem will be used throughout the main body of this paper for comparison purposes to other cover soil slope stability situations.

3.2 Tracked Construction Equipment Forces

The placement of cover soil on a slope with a relatively low shear strength inclusion (like a geomembrane) should always be from the toe upward to the crest. Figure 5a shows the recommended method. In so doing, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever present passive wedge and stable lower-portion beneath the active wedge. While it is necessary to specify low ground pressure equipment to place the soil, the reduction of the FS-value for this situation of equipment working up the slope will be seen to be relatively small.

For soil placement down the slope, however, a stability analysis cannot rely on toe buttressing and also a dynamic stress should be included in the calculation. These conditions decrease the FS-value and in some cases to a great extent. Figure 5b shows this procedure. Unless absolutely necessary, it is not recommended to place cover soil on a slope in this manner. If it is necessary, the design must consider the unsupported soil mass and the dynamic force of the specific type of construction equipment and its manner of operation.



(a) Equipment backfilling up slope (the recommended method)

(b) Equipment backfilling down slope (method is not recommended)

Figure 5. Construction equipment placing cover soil on slopes containing geosynthetics.

For the first case of a bulldozer pushing cover soil up from the toe of the slope to the crest, the analysis uses the free body diagram of Figure 6a. The analysis uses a specific piece of tracked construction equipment (like a bulldozer characterized by its ground contact pressure) and dissipates this force or stress through the cover soil thickness to the surface of the geomembrane. A Boussinesq analysis is used, see Poulos and Davis (1974). This results in an equipment force per unit width as follows:

$$W_e = q w I \qquad (16)$$

where

W_e = equivalent equipment force per unit width at the geomembrane interface

$q = W_b / (2 \times w \times b)$

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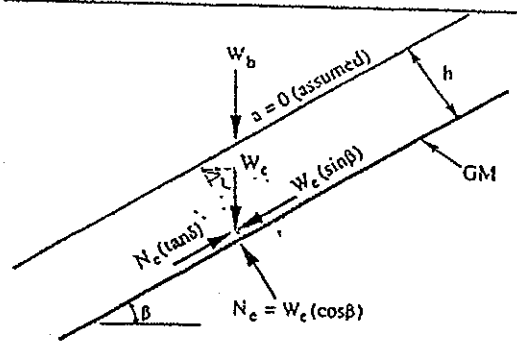
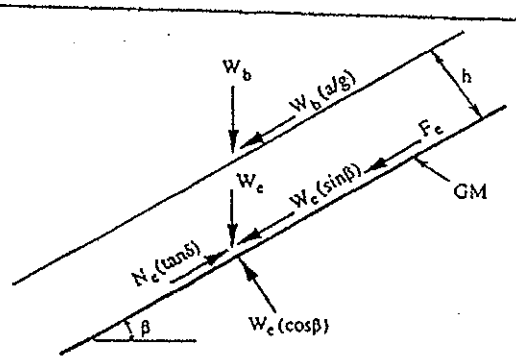
W_b = actual weight of equipment (e.g., a bulldozer)
w = length of equipment track
b = width of equipment track
I = influence factor at the geomembrane interface see Figure 7



(a) Equipment moving up slope
(load with no assumed acceleration)



(b) Equipment moving down slope
(load plus acceleration or deceleration)

Figure 6. Additional (to gravitational forces) limit equilibrium forces due to construction equipment moving on cover soil (see Figure 3 for the gravitational soil force to which the above forces are added).

Upon determining the additional equipment force at the cover soil-to-geomembrane interface, the analysis proceeds as described in Section 3.1 for gravitational forces only. In essence, the equipment moving up the slope adds an additional term, W_e, to the W_A-force in Eq. 3. Note, however, that this involves the generation of a resisting force as well. Thus, the net effect of increasing the driving force as well as the resisting force is somewhat neutralized insofar as the resulting FS-value is concerned. It should also be noted that no acceleration/deceleration forces are included in this analysis which is somewhat optimistic. Using these concepts (the same equations used in Section 3.1 are used here), typical design curves for various FS-values as a function of equivalent ground contact equipment pressures and cover soil thicknesses are given in Figure 8. Note that the curves are developed specifically for the variables stated in the legend. Example 2a illustrates the use of the formulation.

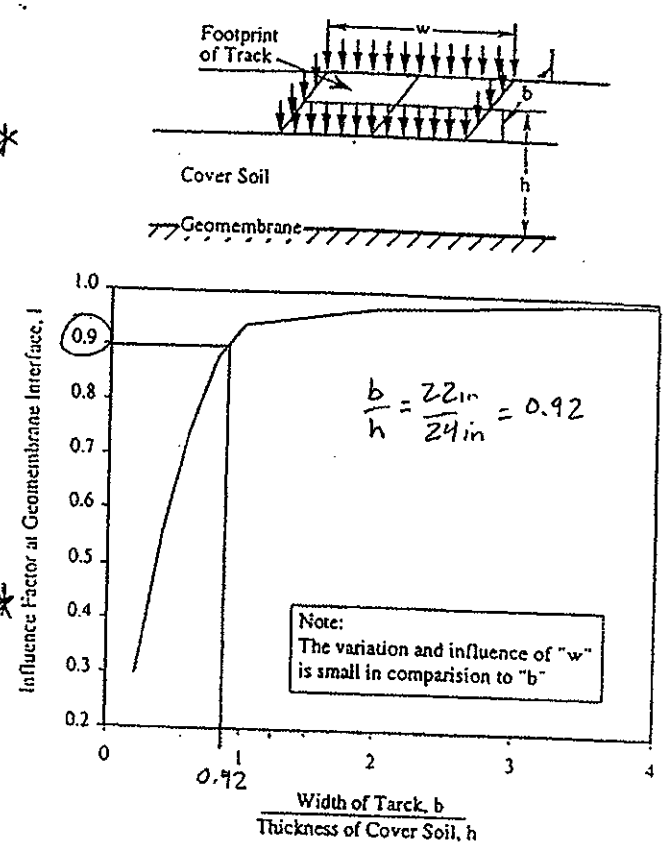


Figure 7. Values of influence factor, "I", for use in Eq. 16 to dissipate surface force of tracked equipment through the cover soil to the geomembrane interface, after Poulos and Davis (1974).

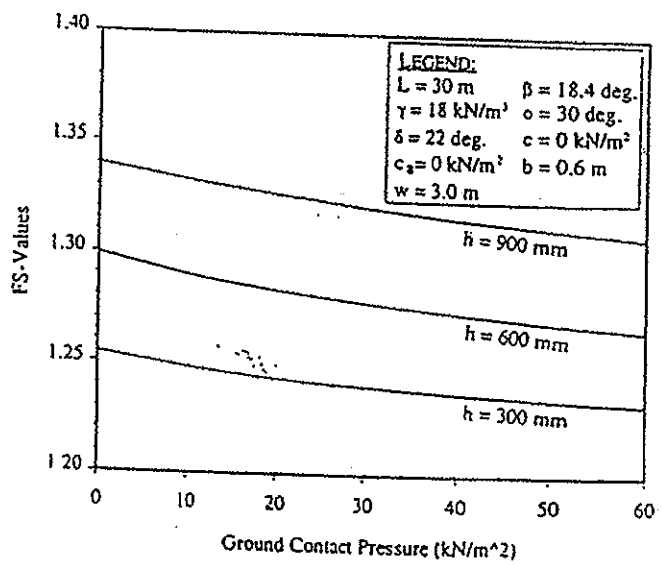


Figure 8. Design curves for stability of different thickness of cover soil for various values of tracked ground contact pressure construction equipment.

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Example 2a:

Given 30 m long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m³. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. It is placed on the slope using a bulldozer moving from the toe of the slope up to the crest. The bulldozer has a ground pressure of 30 kN/m² and tracks that are 3.0 m long and 0.6 m wide. The cover soil to geomembrane friction angle is 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.

Solution:
This problem follows Example 1 exactly except for the addition of the bulldozer moving up the slope. Using the additional equipment load Eq. 16, substituted into Eqs. 14 and 15 results in the following.

$$\left.\begin{aligned} a &= 73.1\,\text{kN/m} \\ b &= -104.3\,\text{kN/m} \\ c &= 17.0\,\text{kN/m} \end{aligned}\right\} \quad FS = 1.24$$

Comment:
While the resulting FS-value is low, the result is best assessed by comparing it to Example 1, i.e., the same problem except without the bulldozer. It is seen that the FS-value has only decreased from 1.25 to 1.24. Thus, in general, a low ground contact pressure bulldozer placing cover soil up the slope with negligible acceleration/deceleration forces does not significantly decrease the factor-of-safety.

For the second case of a bulldozer pushing cover soil down from the crest of the slope to the toe as shown in Figure 5b, the analysis uses the force diagram of Figure 6b. While the weight of the equipment is treated as just described, the lack of a passive wedge along with an additional force due to acceleration (or deceleration) of the equipment significantly changes the resulting FS-values. This analysis again uses a specific piece of construction equipment operated in a specific manner. It produces a force parallel to the slope equivalent to W_b (a/g), where W_b = the weight of the bulldozer, a = acceleration of the bulldozer and g = acceleration due to gravity. Its magnitude is equipment operator dependent and related to both the equipment speed and time to reach such a speed, see Figure 9. A similar behavior will be seen for deceleration.

The acceleration of the bulldozer, coupled with an influence factor "I" from Figure 7, results in the dynamic force per unit width at the cover soil to geomembrane interface, "F_e". The relationship is as follows:

$$F_e = W_e\left(\frac{a}{g}\right) \qquad (17)$$

where

F_e = dynamic force per unit width parallel to the slope at the geomembrane interface,

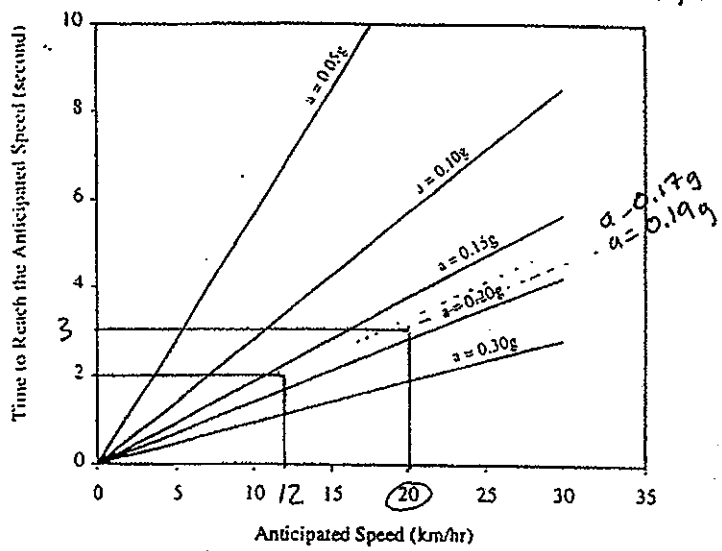

Figure 9. Graphic relationship of construction equipment speed and rise time to obtain equipment acceleration.

W_e = equivalent equipment (bulldozer) force per unit width at geomembrane interface, recall Eq. 16.
β = soil slope angle beneath geomembrane
a = acceleration of the bulldozer
g = acceleration due to gravity

Using these concepts, the new force parallel to the cover soil surface is dissipated through the thickness of the cover soil to the interface of the geomembrane. Again, a Boussinesq analysis is used, see Poulos and Davis (1974). The expression for determining the FS-value can now be derived as follows:

Considering the active wedge, and balancing the forces in the direction parallel to the slope, the following formulation results:

$$E_A + \frac{(N_e + N_A)\tan\delta + C_a}{FS} = (W_A + W_e)\sin\beta + F_e \qquad (18)$$

where

N_e = effective equipment force normal to the failure plane of the active wedge
= $W_e \cos\beta$ (19)

Note that all the other symbols have been previously defined.

The interwedge force acting on the active wedge can down be expressed as:

$$E_A = \frac{(FS)\left[(W_A + W_e)\sin\beta + F_e\right]}{FS} - \frac{\left[(N_e + N_A)\tan\delta + C_a\right]}{FS} \qquad (20)$$

The passive wedge can be treated in a similar manner. The following formulation of the interwedge force acting on the passive wedge results:

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$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} \qquad (21)$$

By setting $E_A = E_P$, the following equation can be arranged in the form of Eq. 13 in which the "a", "b" and "c" terms are as follows:

$$a = \left[(W_A + W_e)\sin\beta + F_e\right]\cos\beta$$

$$b = -\left\{\left[(N_e + N_A)\tan\delta + C_a\right]\cos\beta + \left[(W_A + W_e)\sin\beta + F_e\right]\sin\beta\tan\phi + (C + W_P\tan\phi)\right\}$$

$$c = \left[(N_e + N_A)\tan\delta + C_a\right]\sin\beta\tan\phi \qquad (22)$$

Finally, the resulting FS-value can be obtained using Eq. 15. Using these concepts, typical design curves for various FS-values as a function of equipment ground contact pressure and equipment acceleration can be developed, see Figure 10. Note that the curves are developed specifically for the variables stated in the legend. Example 2b illustrates the use of the formulation.

Example 2b:

Given a 30 m long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. It is placed on the slope using a bulldozer moving from the crest of the slope down to the toe. The bulldozer has a ground contact pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The estimated equipment speed is 20 km/hr and the time to reach this speed is 3.0 sec. The cover soil to geomembrane friction angle is 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.

Solution:

Using the design curves of Figure 10 along with Eqs. 22 substituted into Eq. 15 the solution can be obtained:

- From Figure 9 at 20 km/hr and 3.0 sec. the bulldozer's acceleration is 0.19g.
- From Eq. 22 substituted into Eq. 15 we obtain

$$\left.\begin{aligned} a &= 88.8\,\text{kN/m} \\ b &= -107.3\,\text{kN/m} \\ c &= 17.0\,\text{kN/m} \end{aligned}\right\} \quad FS = 1.03$$

Comment:

This problem solution can now be compared to the previous two examples:

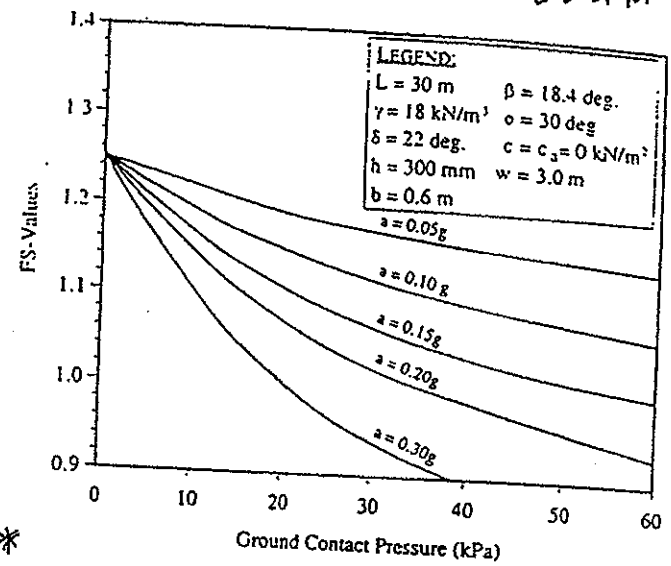


Figure 10. Design curves for stability of different construction equipment ground contact pressure for various equipment accelerations.

Ex. 1:	cover soil alone with no bulldozer loading	FS = 1.25
Ex. 2a:	cover soil plus bulldozer moving up slope	FS = 1.24
Ex. 2b:	cover soil plus bulldozer moving down slope	FS = 1.03

The inherent danger of a bulldozer moving down the slope is readily apparent. Note, that the same result comes about by the bulldozer decelerating instead of accelerating. The sharp breaking action of the bulldozer is arguable the more severe condition due to the extremely short times involved when stopping forward motion. Clearly, only in unavoidable situations should the cover soil placement equipment be allowed to work down the slope. If it is unavoidable, an analysis should be made of the specific stability situation and the construction specifications should reflect the exact conditions made in the design. The maximum allowable weight and ground contact pressure of the equipment should be stated along with suggested operator movement of the cover soil placement operations. Truck traffic on the slopes can also give as high, or even higher, stresses and should be avoided unless adequately designed. Additional detail is given in McKelvey (1994). The issue of access ramps is a unique subset of this example and one which deserves focused attention due to the high loads and decelerations that often occur.

3.3 Consideration of Seepage Forces

The previous sections presented the general problem of slope stability analysis of cover soils placed on slopes under different conditions. The tacit assumption throughout was that either permeable soil or a drainage layer was placed above the barrier layer with adequate flow capacity to efficiently remove permeating water safely way from the cross section. The amount of water to be removed is obviously a site specific situation. Note that in extremely

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GA	09-04-92	W/60-mil sm. HDPE	0.5 - 1 - 2 - 4 - 10	Hydrated	0.02	8	7	0	
		W/60-mil text. HDPE	0.5 - 1 - 2 - 4 - 10	Hydrated	0.02	28	28 / 6	29	bi-modal residual
GSC	12-08-94	W/60-mil text. HDPE	7.5 - 15 - 30	Hydrated	0.04	18	17	175	Diff. membrane manufacturers
		W/60-mil text. HDPE	7.5 - 15 - 30	Hydrated	0.04	16	12	345	
* GSC	12-16-94	W/60-mil text. HDPE	1 -3 - 6 - (15)	Hydrated	0.04	25 (19) ← low normal stress	10	100	(lower d at 15
AGP	07-12-95	W/80-mil Text. HDPE	14 - 28 - 69 - 104	Hydrated	0.04	18	8	192	
		W/80-mil Text. HDPE	14 - 28 - 69 - 104	Dry	0.04	30	14	0	
AGP	11-30-95	W/ Text. HDPE	10 - 26 - 38	Dry	0.08	30.2	13.3	0	
GSC	03-12-96	W/ 30 mil PVC	2 - 4 - 6	Hydrated	0.04	17	17	24	
GSC	05-29-96	NW/80mil Text. HDPE	140	Hydrated	0.04	19	5	475	Consol 24 hrs @140 psi
AGP	11-08-96	NW/60mil Text. HDPE	1.4 - 3.5 - 7.0	Hydrated	0.04	34.8	22.7	149	
		W/60 mil Text. HDPE	1.4 - 3.5 - 7.0	Hydrated	0.04	28.8	20.4	83	
GSC	01-08-97	NW/60mil Text. HDPE	14 - 40 - 70	Hydrated	0.04	17	9	255	
TRI	4-15-97	NW/60mil Text. HDPE	14 – 28 – 56	Hydrated	0.04	21.9	10.8	722	
Emcon	6-16-97	NW/40mil Text. LLPE	1.0 – 2.0 – 3.5	Hydrated	0.04	32.0	18.5	111	
		NW/40mil Text. LLPE	1.0 – 2.0 – 3.5	Hydrated	0.04	37.5	27.1	118	
TRI	10-15-97	NW/ 60mil Text.HDPE	3.5 – 7 – 14	Hydrated	0.04	20.3	18.6	278	
		NW/ 60mil Text.HDPE	.35 - .7 – 1.4	Hydrated	0.04	36.6	25.8	2	
TRI	12-01-97	NW/60mil Text. HDPE	0.35 – 0.7 – 1.4	Hydrated	0.001	25.6	23.3	54	Residual @ 4"
		NW/60mil Text. HDPE	3.5 – 7 – 14	Hydrated	0.001	23.2	17.8	85	"
Emcon	04-06-98	NW/60mil Text. HDPE	14 – 28 – 70	Hydrated	0.04	24.1	12.2	290	

Attachment B 1/3

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERNAL

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GRI	04-18-91	Internal ST	.5 - 1 - 5 - 10 - 20	Dry	0.035	42	ND	288	
		Internal ST	.1 - .5 - 1 - 5 - 10	Hydrated	0.035	37	"	115	
		Internal ST	.1 - .5 - 1 - 5 - 10	Hydrated	0.035	39	"	173	Hydrated in leachate
UTA	08-12-91	Internal	6 - 9 - 14 - 19	Hydrated	0.000131	26	ND	619	
GA	09-04-92	Internal ST	0.5 - 1 - 2 - 4 - 10	Hydrated	0.0025	27	8	43	Slow shear rate
GSC	12-07-93	Internal ST	0.35 - 1 - 2 - 3.5	Dry	0.04	57	ND	370	
		Internal ST	0.5 - 1 - 2 - 4 - 10	Hydrated	0.04	59	"	265	
GSC	02-02-94	Internal ST	14 - 70 - 142	Hydrated	0.04	24	7	515	
GSC	07-15-94	Internal ST	0.35 - 1 - 2 - 3.5	Hydrated	0.04	59	31	271	cross machine dir.
CETCO	07-11-95	Internal	14 - 28 - 50	Hydrated	0.04	31	2.2	626	Hydrated 72 hours
GSC	03-12-96	Internal ST	2 - 4 - 6	Hydrated	0.04	34	ND	423	
GSC	07-03-96	Internal ST	0.7 - 1.4 - 2.0	Hydrated	0.04	43	ND	220	
GSC	08-26-96	Internal ST	7 - 8.7 - 10.5	Hydrated	0.04	34	10	755	
		Internal ST	7 - 8.7 - 10.5	Hydrated	0.04	37	11	630	
		Internal ST	7 - 8.7 - 10.5	Hydrated	0.04	38	8	685	
GSC	11-08-96	Internal ST	0.7 - 1.4 - 2.8	Hydrated	0.04	45	ND	339	
	*	Internal ST	0.7 - 1.4 - 2.8	Hydrated	0.04	35	ND	~~350~~	
		Internal ST	0.7 - 1.4 - 2.8	Hydrated	0.04	48	ND	277	
		Internal ST	0.7 - 1.4 - 2.8	Hydrated	0.04	51	ND	273	
		Internal ST	0.7 - 1.4 - 2.8	Hydrated	0.04	47	ND	270	
GSC	01-09-97	Internal DN	0.7 - 1.4 - 2.8	Hydrated	0.04	44	ND	242	
GSC	02-20-97	Internal ST	0.35 - 1.4 - 3.5	Hydrated	0.04	34	ND	292	

Attachment B 2/3

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ SOIL

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GSC	03-09-95	NW/ Soaked GC Soil	7.5 - 15 - 30	Hydrated	0.04	33	31	385	
CETCO	08-29-95	W/Silty Sand	1 - 2 - 3	Hydrated	0.04	32.3	ND	127	
GSC	03-12-96	NW/ Soaked Soil	2 - 4 - 6	Hydrated	0.04	29	29	70	
STS	05-30-96	NW/Silty Sandy Gravel	0.7 – 2 – 3.5	Hydrated	0.04	28	ND	222	
CETCO	07-08-96	NW/ Sandy Silty Gravel	0.7 - 2.0 - 3.5	Hydrated	0.04	40.4	ND	238	
	07-31-96	W/ Sandy Silty Gravel	0.7 - 2.0 - 3.5	Hydrated	0.04	37.8	ND	194	
AGP	11-08-96	NW/ SC Soil	1.4 - 3.5 - 7.0	Hydrated	0.04	27.8	ND	186	*
		W/ SC Soil	1.4 - 3.5 - 7.0	Hydrated	0.04	23.5	22.1	187	
GSC	01-08-97	NW/ Soaked Soils	14 - 40 - 70	Hydrated	0.04	18	18	155	
TRI	09-23-97	NW/ Subbaase Soil	3.5 – 7 – 14	Hydrated	0.001	24.0	20.7	4	
		NW/ Subbase Soil	.35 - .7 – 1.4	Hydrated	0.001	35.5	37.2	88	
GSC	01-22-98	CLS/ SM soil	5 - 25 - 45	Hydrated	0.04	17	13	70	
CETCO	02-10-98	CLS/ SC Soil	0.5 – 1 – 1.5	Hydrated	0.04	9.5	ND	46	
VE	03-03-98	W/ Cover soil	1 - 2.1 – 4.9	Hydrated	0.04	37	33	50	
CETCO	08-11-98	W/ SC soil	2.1 – 4.2 – 6.3	Hydrated	0.04	33.6	35.2	183	
		NW/ Non-Pag Quartz Monzonite Waste	2.1 – 4.2 – 6.3	Hydrated	0.04	39.8	37.5	190	
TRI	09-22-98	NW/ soil	16 – 32 – 73	Hydrated	0.04	17.5	16.8	505	Residual @ 3"
CETCO	11-3-98	CLS/SM soil	0.5 – 1 – 1.5	Hydrated	0.04	27	27	80	
		CLS/SP soil	0.5 – 1 – 1.5	Hydrated	0.04	15.2	15.2	55	
GSC	6-24-00	CLS/drainage soil	139	Hydrated	0.04	24	0	13	Angle w/ origin

Attachment B 3/3

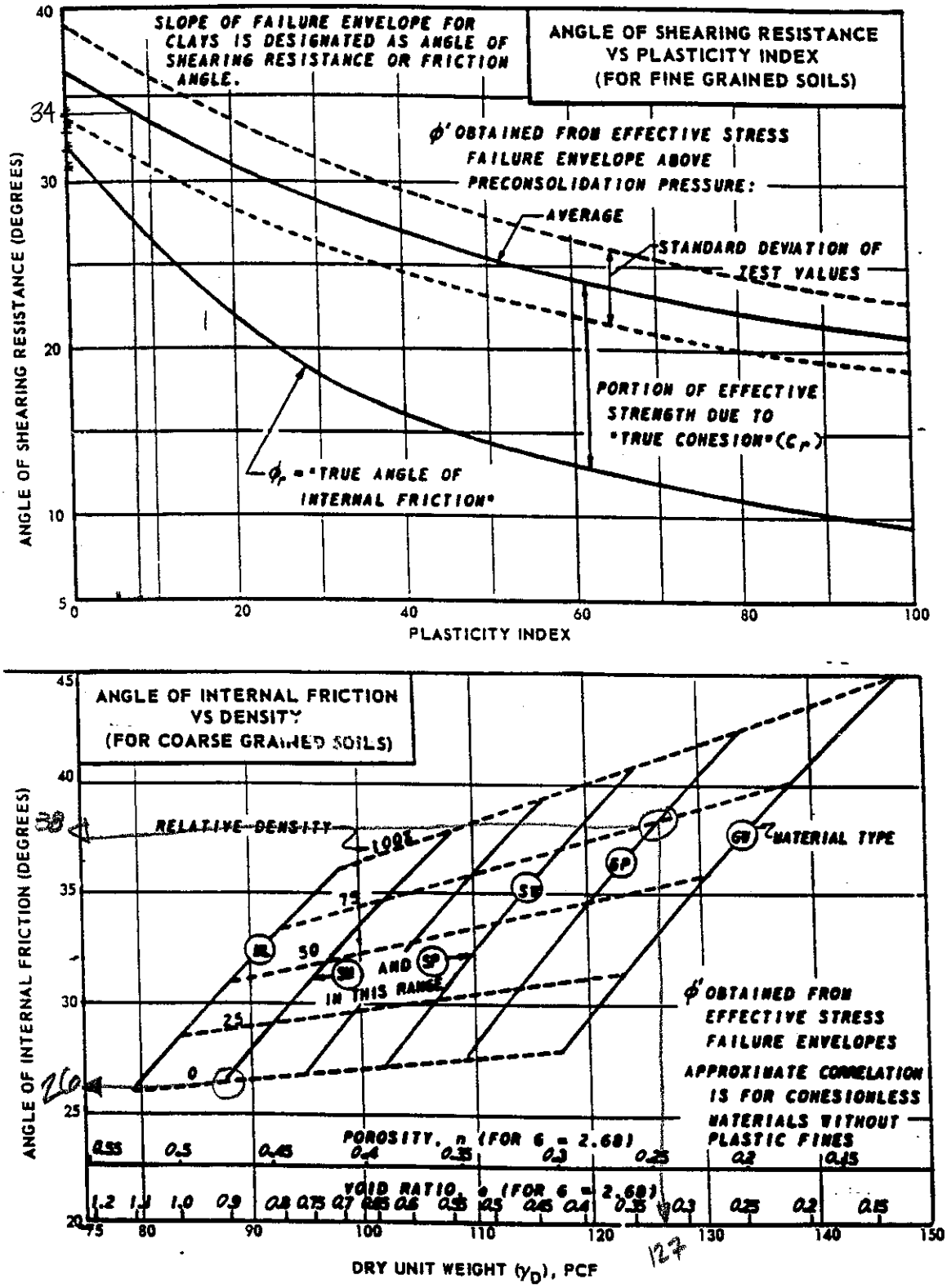


FIGURE 3-7
Correlations of Strength Characteristics

NAVFAC DM7-03, 1986

ATTACHMENT C
(1/1)

FROM - PROCEEDINGS OF THE 4th GRI SEMINAR ON THE TOPIC OF LANDFILL CLOSURES
GRI - DEC. 14, 1990

TABLE 6
TMI COEFFICIENT OF FRICTION TEST RESULTS

GEONET VS. GEOSYNTHETICS INTERFACE FRICTION

NONWOVEN, NEEDLEPUNCHED	20°
NONWOVEN, HEATBONDED	17°
MONOFILAMENT	14°
MULTIFILAMENT	20°
SLIT FILM	14°
HDPE - SMOOTH	16°
HDPE - ROUGH	18°
CSPE	24°
VLDPE	20°
PVC	17°

GEOTEXTILE VS. GEOMEMBRANE INTERFACE FRICTION

GEOTEXTILE INTERFACE	HDPE	HDPE (ROUGH)	PVC	CSPE	VLDPE
WOVEN, SLIT FILM	18°	17°	23°	25°	22°
WOVEN, MONOFILAMENT	21°	14°	22°	23°	22°
WOVEN, MULTIFILAMENT	21°	25°	29°	27°	22°
NONWOVEN, HEATBONDED	23°	23°	19°	23°	19°
NONWOVEN, NEEDLEPUNCHED	19°	34°	22°	21°	20°

Attachment D 1/1

APPENDIX E

Boring Logs and Geotechnical Laboratory Results

Geosyntec consultants
10875 Rancho Bernardo Rd, Suite 200
San Diego, CA 92127
Tel: (858) 674-6559
Fax: (858) 674-6586

PROJECT Denison Mines - White Mesa Mill
PROJECT LOCATION Cell 4B
PROJECT NUMBER SC0349

KEY SHEET - CLASSIFICATIONS AND SYMBOLS

GS FORM: KEY 09/99

EMPIRICAL CORRELATIONS WITH STANDARD PENETRATION RESISTANCE N VALUES *

	N VALUE * (BLOWS/FT)	CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT)
FINE GRAINED SOILS	0 - 2	VERY SOFT	<0.25
	3 - 4	SOFT	0.25 - 0.50
	5 - 8	FIRM	0.50 - 1.00
	9 - 15	STIFF	1.00 - 2.00
	16 - 30	VERY STIFF	2.00 - 4.00
	31 - 50	HARD	>4.00
	>50	VERY HARD	

	N VALUE * (BLOWS/FT)	RELATIVE DENSITY
COARSE GRAINED SOILS	0 - 4	VERY LOOSE
	5 - 10	LOOSE
	11 - 30	MEDIUM DENSE
	31 - 50	DENSE
	>50	VERY DENSE

* ASTM D 1586; NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 IN. O.D., 1.4 IN. I.D. SAMPLER ONE FOOT.

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

MAJOR DIVISIONS			SYMBOLS	DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	GRAVELS WITH FINES APPRECIABLE AMOUNT OF FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
MORE THAN 50% OF MATERIAL COARSER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO.4 SIEVE	SANDS WITH FINES APPRECIABLE AMOUNT OF FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL FINER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENT

NOTE: DUAL SYMBOLS USED FOR BORDERLINE CLASSIFICATIONS

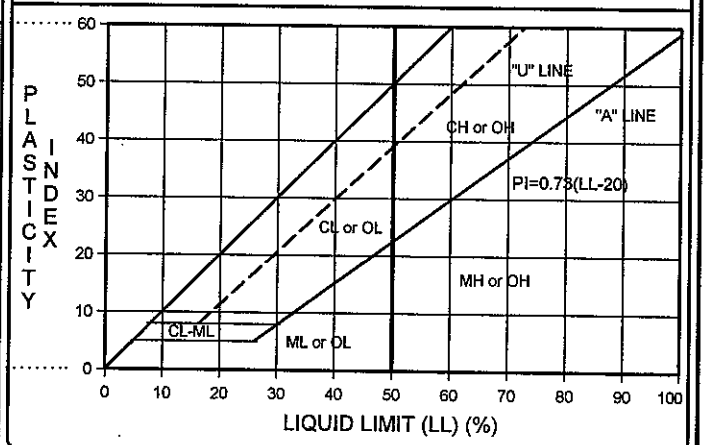
PARTICLE SIZE IDENTIFICATION

BOULDERS	>300 mm
COBBLES	75 - 300 mm
GRAVEL: COARSE	19.0 - 75 mm
GRAVEL: FINE	4.75 - 19 mm
SAND: COARSE	2.00 - 4.75 mm
SAND: MEDIUM	0.425 - 2.00 mm
SAND: FINE	0.075 - 0.425 mm
SILT	0.075 - 0.002 mm
CLAY	<0.002 mm

WELL GRADED - HAVING WIDE RANGE OF GRAIN SIZES AND APPRECIABLE AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZES

POORLY GRADED - PREDOMINANTLY ONE GRAIN SIZE, OR HAVING A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING

PLASTICITY CHART



OTHER MATERIAL SYMBOLS

Siltstone
Sandstone
Siltstone/Claystone
Claystone
Shale
Siltstone/Sandstone
Conglomerate
Granitic
Sand
Silt
Silty Sand
Evaporite
Artificial Fill
Debris Fill
Asphalt
Concrete

WELL SYMBOLS

HYDRATED GRANULAR BENTONITE
BENTONITE CEMENT GROUT
FILTER PACK
CONCRETE
NATIVE/ SLOUGH
CENTRAL-IZER

SAMPLER AND OTHER SYMBOLS

BULK SAMPLE
CORE SAMPLE
GRAB SAMPLE
HAND AUGER
DRIVE SAMPLE
GROUNDWATER SAMPLE

Water Level at Time Drilling, or as Shown
Static Water Level
MSL: Mean Sea Level
AGS: Above Ground Surface
BGS: Below Ground Surface
BTOC: Below Top of Casing
HSA: Hollow Stem Auger

Geosyntec consultants
10875 Rancho Bernardo Rd, Suite 200
San Diego, CA 92127
Tel: (858) 674-6559
Fax: (858) 674-6586

GS FORM: BORE 1/99

BOREHOLE RECORD

BORING B-1 SHEET 1 OF 1
START DATE 30 Apr 07 ELEVATION 5570 FT MSL
FINISH DATE 30 Apr 07
PROJECT Denison Mines - White Mesa Mill
LOCATION Cell 4B
PROJECT NUMBER SC0349

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	RECOVERY (%)	SO_2 Reading (ppm)	PID READING (ppm)	TIME	COMMENTS
	Eolian/Loess: Stiff, moist, yellowish red (5YR - 5/6), sandy clay (CL).			1-1						
5			5565	1-2						
	Weathered Sandstone: Dense, moist, pale yellow (5Y - 8/2), weathered sandstone fragments, possible boulder.									
	Dakota Sandstone: Very dense, moist, white to pale yellow (5Y- 8/1 to 8/2), fine sandstone with little silt (SM/SP).			1-3						
	Total Depth = 9' 1" bgs									

CONTRACTOR DA Smith
EQUIPMENT Solid Stem Auger
DRILL MTHD
DIAMETER 6"
LOGGER AG REVIEWER GC

NORTHING
EASTING
ANGLE Vertical
BEARING ——
PRINTED 15 Oct 07

REMARKS:

COORDINATE SYSTEM: MSL
SEE KEY SHEET FOR SYMBOLS AND ABBREVIATIONS

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Fax: (858) 674-6586

GS FORM: BORE 1/99

BOREHOLE RECORD

BORING **B-2** SHEET **1 OF 1**
START DATE 30 Apr 07 ELEVATION 5578 FT MSL
FINISH DATE 30 Apr 07
PROJECT Denison Mines - White Mesa Mill
LOCATION Cell 4B
PROJECT NUMBER SC0349

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	RECOVERY (%)	SO_2 Reading (ppm)	PID READING (ppm)	TIME	COMMENTS
	Eolian/Loess: Stiff, moist, yellowish red (5YR - 5/6), sandy clay (CL).		5575	2-1						Bulk sample collected 0 - 6 ft bgs
5	Weathered Sandstone Dense, moist, white to pale yellow (5Y - 8/1 to 8/2), weathered sandstone fragments.			2-2						
	Dakota Sandstone: Very dense, moist, very pale brown (10YR - 8/2 to 8/3), fine sandstone with silt (SM).		5570	2-3						
	Total Depth = 9' 3" bgs									

CONTRACTOR DA Smith
EQUIPMENT Solid Stem Auger
DRILL MTHD
DIAMETER 6"
LOGGER AG REVIEWER GC

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GS FORM: BORE 1/99

BOREHOLE RECORD

BORING B-3 SHEET 1 OF 1
START DATE 30 Apr 07 ELEVATION 5578 FT MSL
FINISH DATE 30 Apr 07
PROJECT Denison Mines - White Mesa Mill
LOCATION Cell 4B
PROJECT NUMBER SC0349

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	RECOVERY (%)	SO_2 Reading (ppm)	PID READING (ppm)	TIME	COMMENTS
	Eolian/ Loess: Stiff, moist, yellowish red (5YR - 4/6), sandy clay (CL).			3-1						
			5575							
	Weathered Sandstone Very dense, moist, pinkish white (7.5YR - 8/2) silty fine to very fine sand (SM).			3-2						
5										
	Dakota Sandstone: Very dense, moist, very pale brown (10YR - 8/3), silty very fine sand (SM).		5570							
				3-3						
	Total Depth = 9' 7" bgs									

CONTRACTOR DA Smith
EQUIPMENT Solid Stem Auger
DRILL MTHD
DIAMETER 6"
LOGGER AG REVIEWER GC

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GS FORM: BORE 1/99

BOREHOLE RECORD

BORING B-4 SHEET 1 OF 1
START DATE 30 Apr 07 ELEVATION 5592 FT MSL
FINISH DATE 30 Apr 07
PROJECT Denison Mines - White Mesa Mill
LOCATION Cell 4B
PROJECT NUMBER SC0349

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	RECOVERY (%)	SO_2 Reading (ppm)	PID READING (ppm)	TIME	COMMENTS
	Eolian/Loess: Stiff, moist, yellowish red (5YR - 4/6), sandy clay (CL).			4-1						
			5590							
				4-2						
5	Becomes very dense, reddish yellow (5YR - 6/6) with irregular zones of pinkish white (5YR - 8/2).									
	Weathered Sandstone: Dense, moist, pink (5YR - 8/3), silty fine sand (SM)		5585							
				4-3						
10										
	Dakota Sandstone: Very dense, moist, white (5Y - 8/1) fine sandstone with trace silt (SP/SM)		5580							
				4-4						
	Total Depth = 14' bgs									

CONTRACTOR DA Smith
EQUIPMENT Solid Stem Auger
DRILL MTHD
DIAMETER 6"
LOGGER AG REVIEWER GC

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GS FORM: BORE 1/99

BOREHOLE RECORD

BORING B-5 SHEET 1 OF 1
START DATE 30 Apr 07 ELEVATION 5603 FT MSL
FINISH DATE 30 Apr 07
PROJECT Denison Mines - White Mesa Mill
LOCATION Cell 4B
PROJECT NUMBER SC0349

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	RECOVERY (%)	SO_2 Reading (ppm)	PID READING (ppm)	TIME	COMMENTS
	Eolian/Loess: Firm, moist, yellowish red (5YR - 4/6), sandy clay (CL).			5-1						Bulk sample collected 0 - 5 ft bgs
			5600							
5	Becomes very dense and reddish yellow (5YR - 6/6) with irregular zones of pinkish white (5YR - 8/2).			5-2						
			5595							
10				5-3						
			5590	5-4						
	Dakota Sandstone: Very dense, moist, pink (5YR - 8/3), fine sandstone (weathered) with silt (SM).									
	Total Depth = 14.5' bgs									

CONTRACTOR DA Smith
EQUIPMENT Solid Stem Auger
DRILL MTHD
DIAMETER 6"
LOGGER AG REVIEWER GC

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BOREHOLE RECORD

BORING B-6 SHEET 1 OF 1
START DATE 30 Apr 07 ELEVATION 5582 FT MSL
FINISH DATE 30 Apr 07
PROJECT Denison Mines - White Mesa Mill
LOCATION Cell 4B
PROJECT NUMBER SC0349

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	RECOVERY (%)	SO_2 Reading (ppm)	PID READING (ppm)	TIME	COMMENTS
	Eolian/Loess: Firm, moist, yellowish red (5YR - 4/6), sandy clay (CL).		5580	6-1						
5	Weathered Sandstone, medium dense, moist, pinkish white (5YR - 8/2), to very pale brown (10YR - 8/2), silty fine to very fine sand (SM).		5575	6-2						
	Dakota Sandstone: Very dense, moist, white to very pale brown (10YR - 8/1 to 8/2) fine sandstone with trace silt (SM/SP).									
10				6-3						
	Total Depth = 10' 2" bgs									

CONTRACTOR DA Smith
EQUIPMENT Solid Stem Auger
DRILL MTHD
DIAMETER 6"
LOGGER AG REVIEWER GC

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EASTING
ANGLE Vertical
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BOREHOLE RECORD

BORING B-7 SHEET 1 OF 1
START DATE 30 Apr 07 ELEVATION 5588 FT MSL
FINISH DATE 30 Apr 07
PROJECT Denison Mines - White Mesa Mill
LOCATION Cell 4B
PROJECT NUMBER SC0349

DEPTH (ft)	MATERIAL DESCRIPTION	SYMBOLIC LOG	ELEVATION (ft)	SAMPLES NUMBER	TYPE	RECOVERY (%)	SO_2 Reading (ppm)	PID READING (ppm)	TIME	COMMENTS
	Eolian/Loess: Firm, moist, yellowish red (5YR - 4/6), sandy clay (CL).			7-1						
			5585							
	Weathered Sandstone Very dense, moist, pinkish white (5YR - 8/2) to very pale brown (10YR - 8/2), silty fine to very fine sand (SM).			7-2						
5										
	Dakota Sandstone: Very dense, moist, very pale brown (10YR - 8/2), fine sandstone with trace silt (SP/SM).									
			5580							
				7-3						
	Total Depth = 9' 2" bgs									

CONTRACTOR DA Smith
EQUIPMENT Solid Stem Auger
DRILL MTHD
DIAMETER 6"
LOGGER AG REVIEWER GC

NORTHING
EASTING
ANGLE Vertical
BEARING —
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REMARKS:

COORDINATE SYSTEM: MSL
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8788 Balboa Avenue, San Diego, CA 92123
Special Inspections for Construction Projects
DVBE □ SB (619) 583.6633 Fax (619) 583.6654

June 1, 2007 "G" Force Project No. GF12204

Geosyntec Consultants
Steve Fitzwilliam
10875 Rancho Bernardo Road, Suite 200
San Diego, CA 92127

RE: INSPECTION and/or TESTING REPORTS
White Mesa Mall - Cell 4B
Job #SC0349-03

In reference to the above project, please find copies of the field inspector/technician's daily reports and/or original laboratory test results enclosed for your records.

If you should have any questions regarding this project or we can be of further assistance to you, please don't hesitate to contact our office at (619) 583-6633.

Respectfully submitted,
"G"-Force

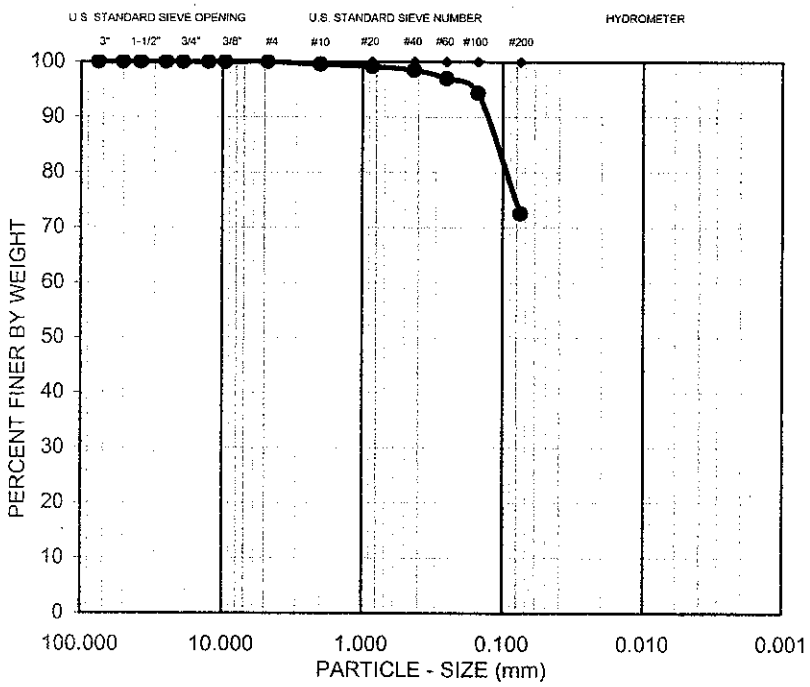
Teresa Jones
Operations Coordinator

Sieve Analysis

(ASTM C422)

G Force Lab No. 5224
Date Sampled: 5/1/2007 By: Client
Date Submitted: 5/3/2007 By: Client
Sample Location: B-2 @ 0 - 6'
Sample Description: Sandy lean clay

Gravel		Sand			Fines
CRS	Fine	CR	Med	Fine	Clay and / or Silt



Sieve Size	% Passing	Specification Low	Specification High	X = Out of Spec
3"	100			
2"	100			
1-1/2"	100			
1"	100			
3/4"	100			
1/2"	100			
3/8"	100			
#4	100			
#10	100			
#20	99			
#40	99			
#60	97			
#100	94			
#200	72			

Liquid Limit			
Plasticity Index			

Checked by: John Inlow, Lab Manager

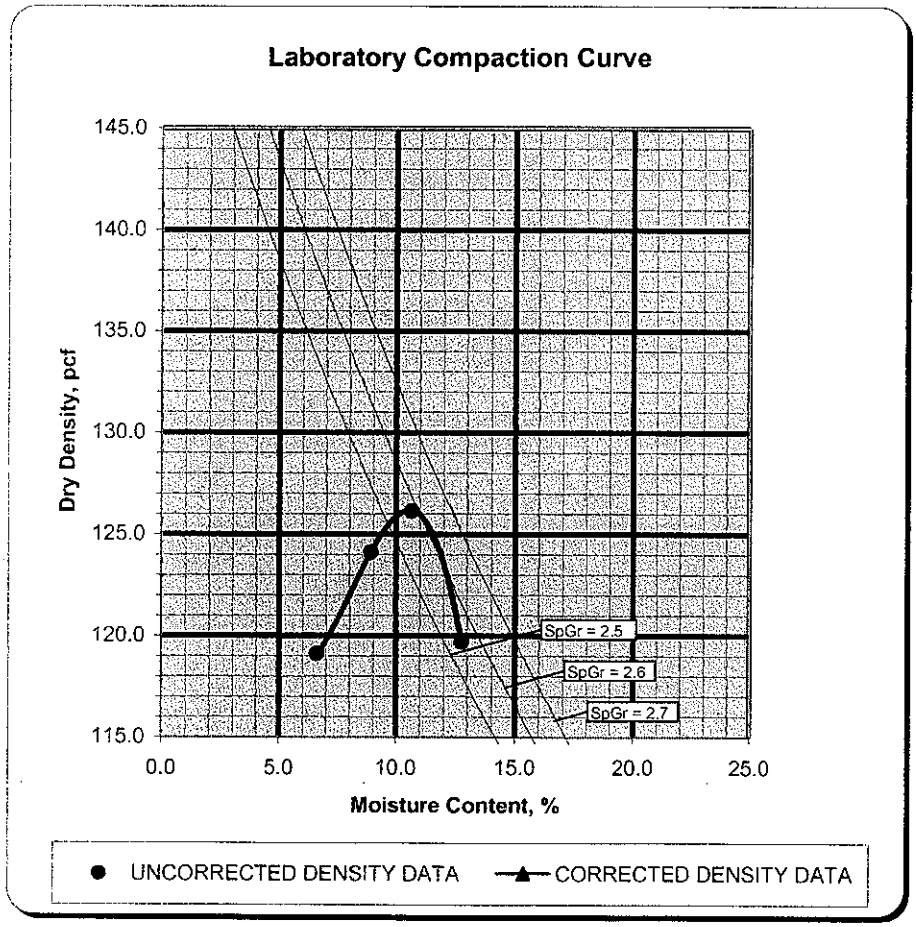
Reviewed by: Scott L. Larson 5/23/07
Scott Larson, RCE, 42432

LABORATORY COMPACTION CURVE

G Force Lab No.: **5224**
Sample Location: **B-2** Depth, ft.: **0 - 6'**
Soil Description: **Light brown sandy lean clay**
Source of Soil: **Not submitted**

Test Designation: **ASTM D1557** Method **A**
% +3/4" **0** % +3/8" **0** % +#4 **0**
Oversize Correction Applied? **No**
Method of Sample Preparation: **Wet**
Type of Rammer Used: **Manual**

M/D Curve No. 1



Test Results

Maximum Density, pcf	**126.0**
Optimum Moisture, %	**10.5**

Oversize Corrected Results

Maximum Density, pcf	
Optimum Moisture, %	

Checked by: John Inlow, Lab Manager

Reviewed by: Scott L. Larson 5/23/07
Scott Larson, RCE, 42432

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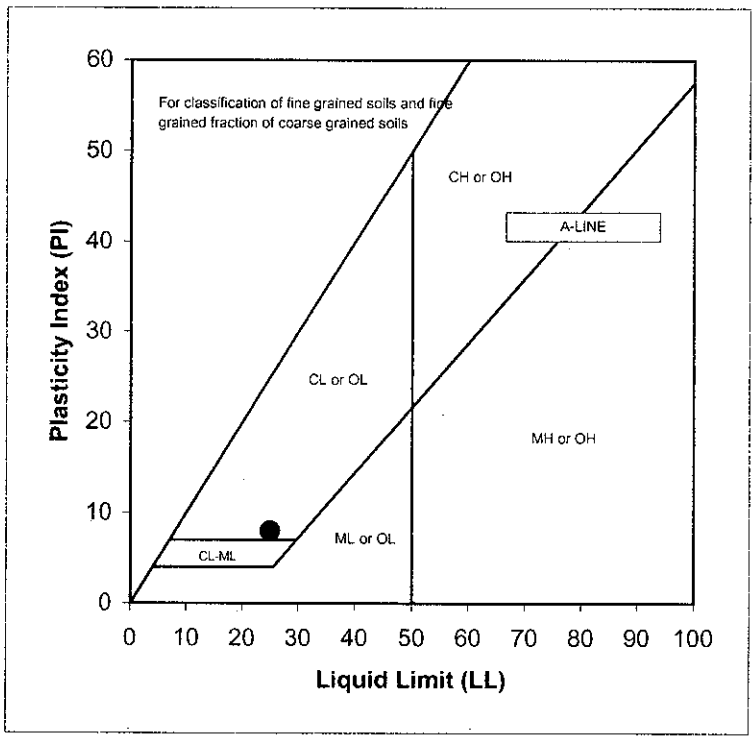
www.gforceca.com

Atterberg Limits

(ASTM D4316)

G Force Lab No.	5225		
Date Sampled:	5/1/2007	By:	Client
Date Submitted:	5/3/2007	By:	Client
Sample Location:	B5 @ 0 - 5'		
Sample Description:	Sandy lean clay		

Liquid Limit	25	
Plastic Limit	17	Specification
Plasticity Index	**8**	
Classification	**CL**	



Checked by: John Inlow, Laboratory Manager

Reviewed by: Scott L. Larson 5/23/07
Scott Larson, RCE, 42432

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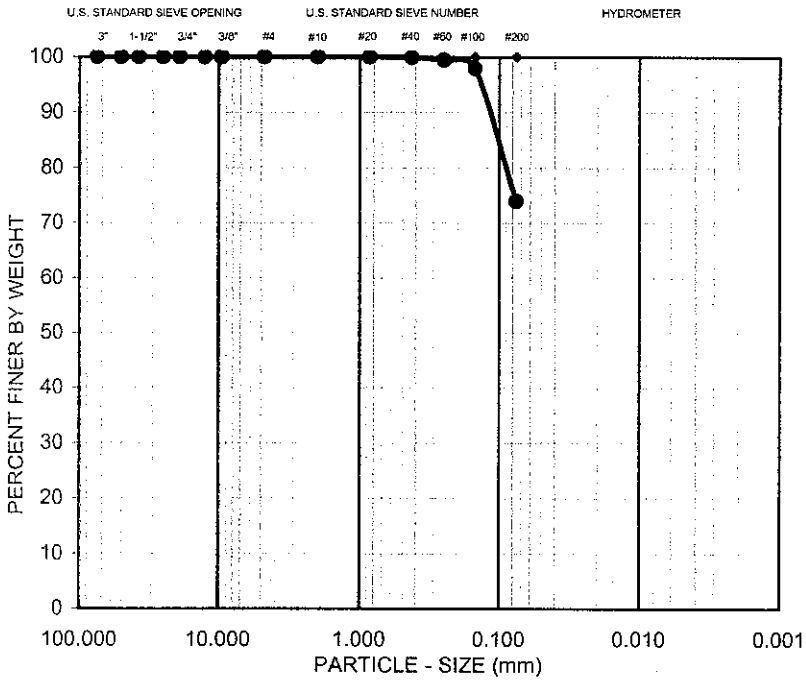
Sieve Analysis

(ASTM C422)

G Force Lab No. 5225
Date Sampled: 5/1/2007 By: Client
Date Submitted: 5/3/2007 By: Client
Sample Location: B-5 @ 0 - 5'
Sample Description: Sandy lean clay

Gravel		Sand			Fines
CRS	Fine	CR	Med	Fine	Clay and / or Silt



Sieve Size	% Passing	Specification Low	Specification High	X = Out of Spec
3"	100			
2"	100			
1-1/2"	100			
1"	100			
3/4"	100			
1/2"	100			
3/8"	100			
#4	100			
#10	100			
#20	100			
#40	100			
#60	99			
#100	98			
#200	74			

Liquid Limit	25		
Plasticity Index	8		

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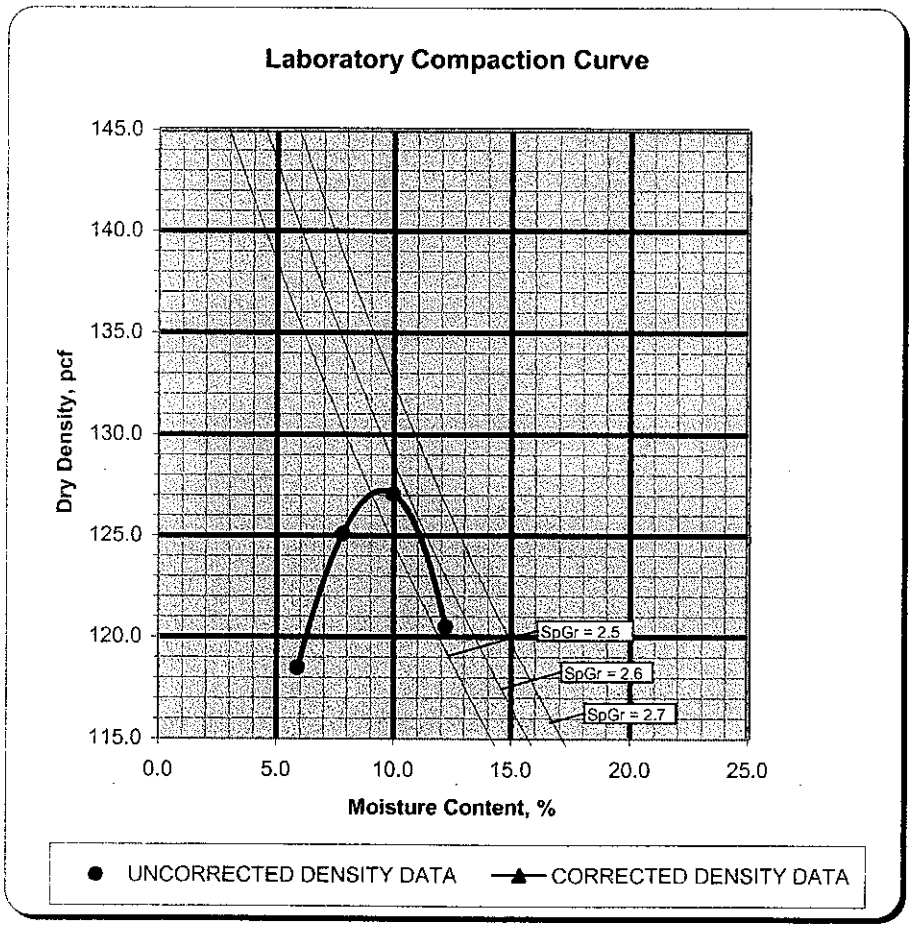
LABORATORY COMPACTION CURVE

G Force Lab No.: **5225**
Sample Location: **B-5** Depth, ft.: **0 - 5'**
Soil Description: **Light brown sandy lean clay**
Source of Soil: **Not submitted**

Test Designation: **ASTM D1557** Method **A**
% +3/4" **0** % +3/8" **0** % +#4 **0**
Oversize Correction Applied? **No**
Method of Sample Preparation: **Wet**
Type of Rammer Used: **Manual**

M/D Curve No. 2



Test Results

Maximum Density, pcf	**127.0**
Optimum Moisture, %	**9.5**

Oversize Corrected Results

Maximum Density, pcf	
Optimum Moisture, %	

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Soil Moisture Content

ASTM D2216

Lab No.	5224	5225				
Boring No.	B-2	B-5				
Depth, ft.	0 - 6'	0 - 5'				
Moisture Content, %	4.8	4.2				
Dry Density, pcf	N/A	N/A				

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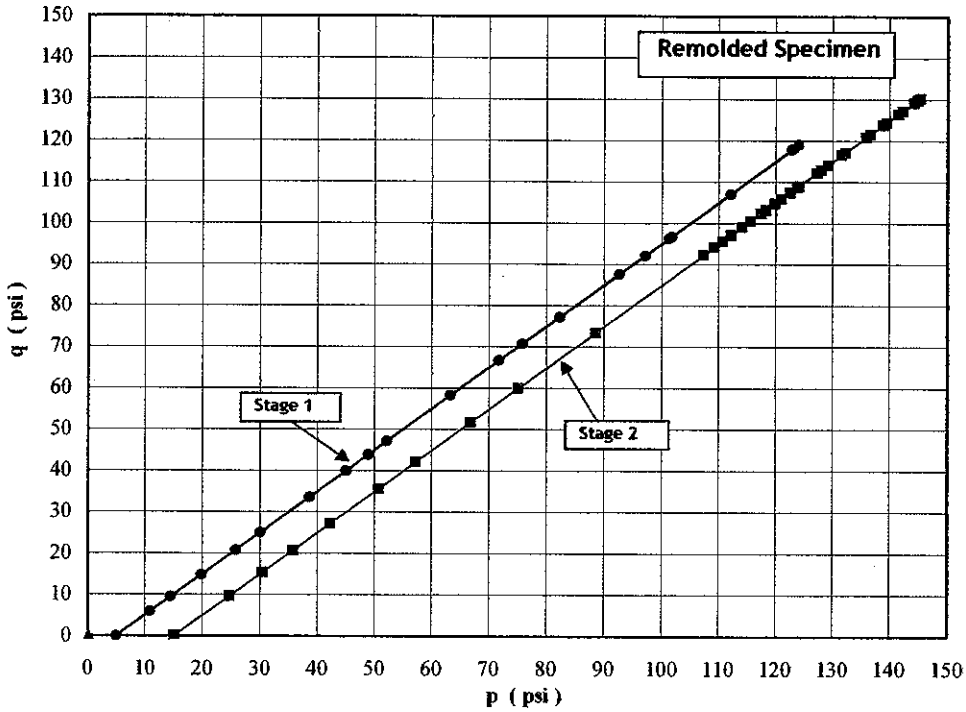
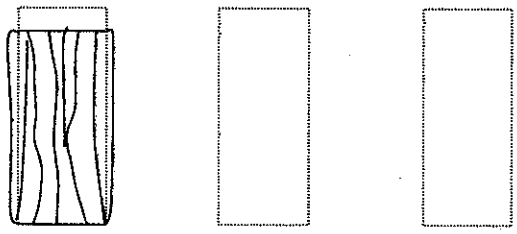
941 Forrest Street, Roswell, Georgia 30075
Tel: (770) 650 1666 Fax: (770) 650 5786

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.

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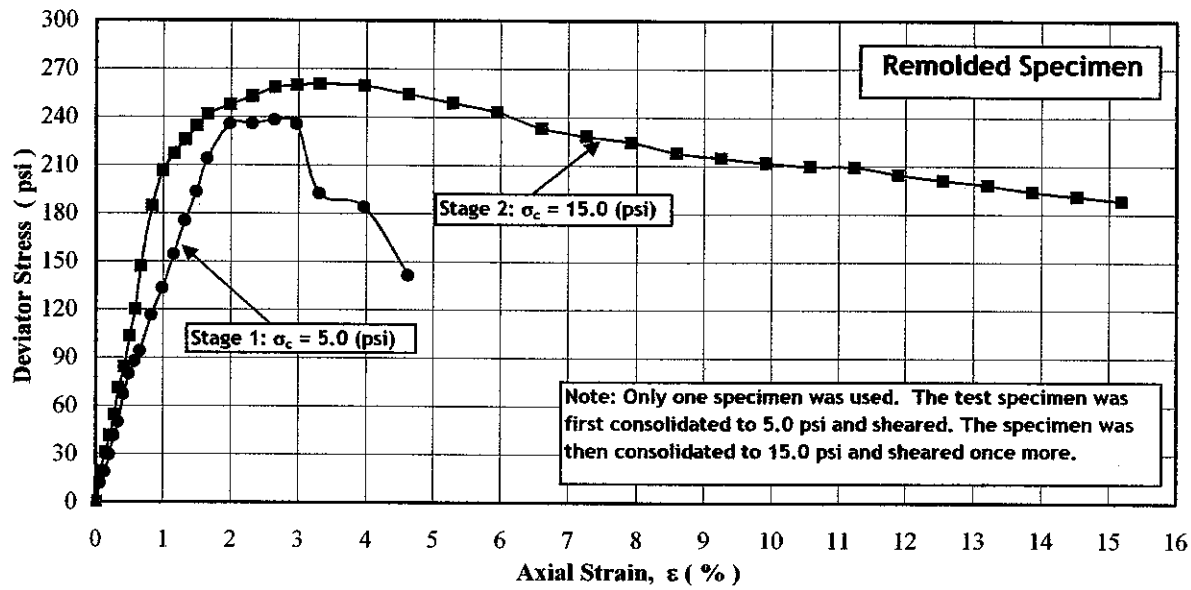
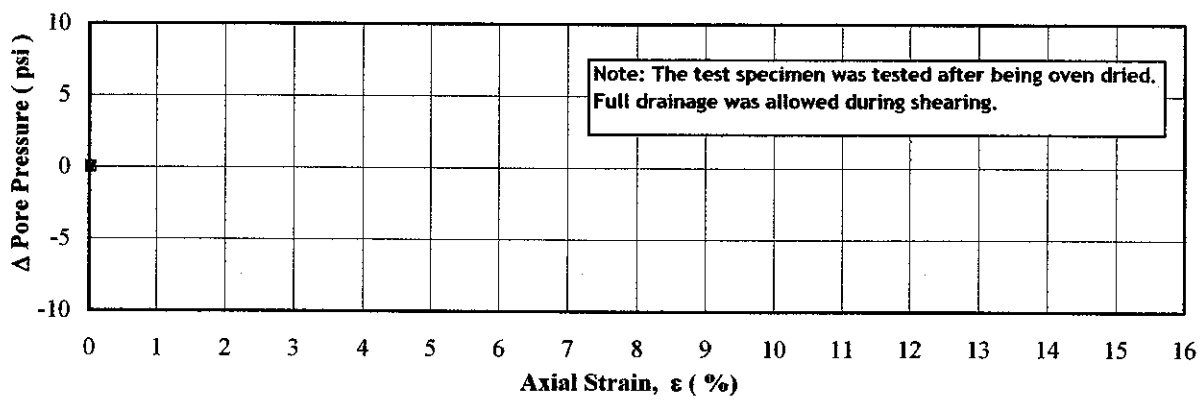
Project Name: White Mesa Mill - Cell 4 A
Project No: 246
Site Sample ID: B-2-1-3
Lab Sample No: E100

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.

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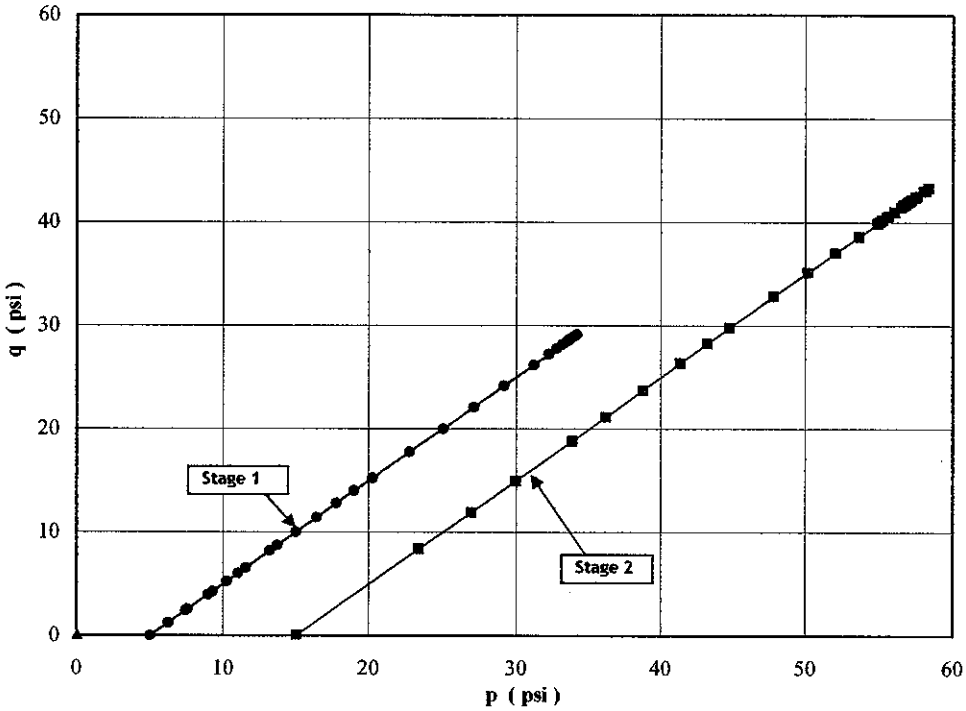
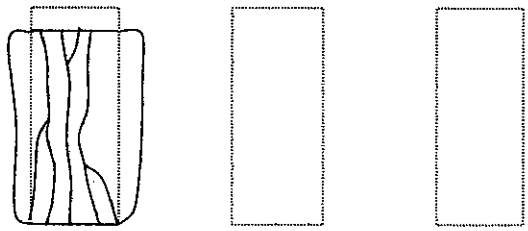
Project Name:	White Mesa Mill - Cell 4 A
Project No:	246
Site Sample ID:	B-2-1-4
Lab Sample No:	E101

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)

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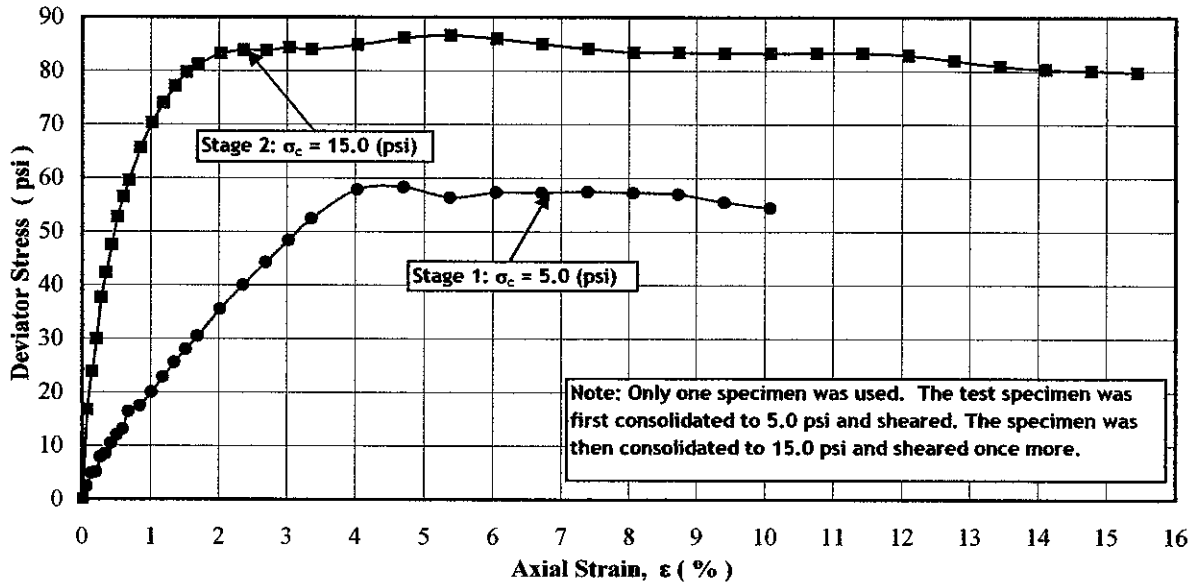
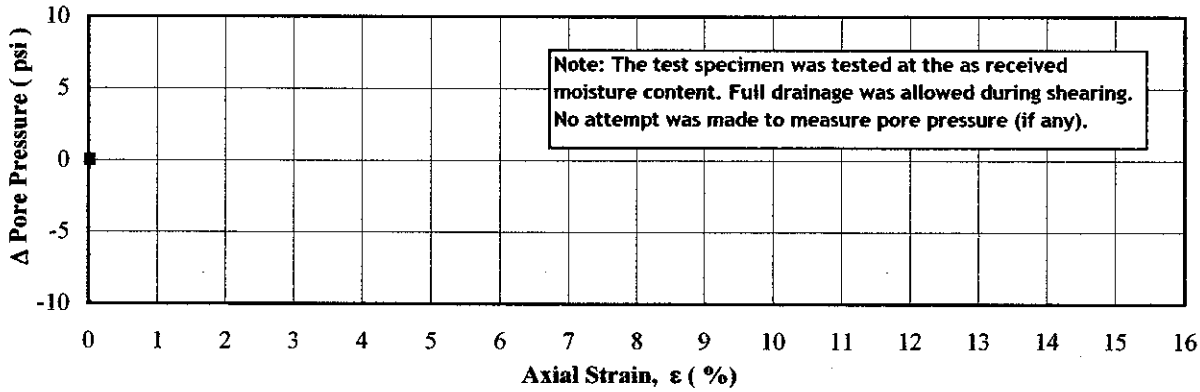
Project Name: White Mesa Mill - Cell 4 A
Project No: 246
Site Sample ID: B-2-1-4
Lab Sample No: E101

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)
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, (%)

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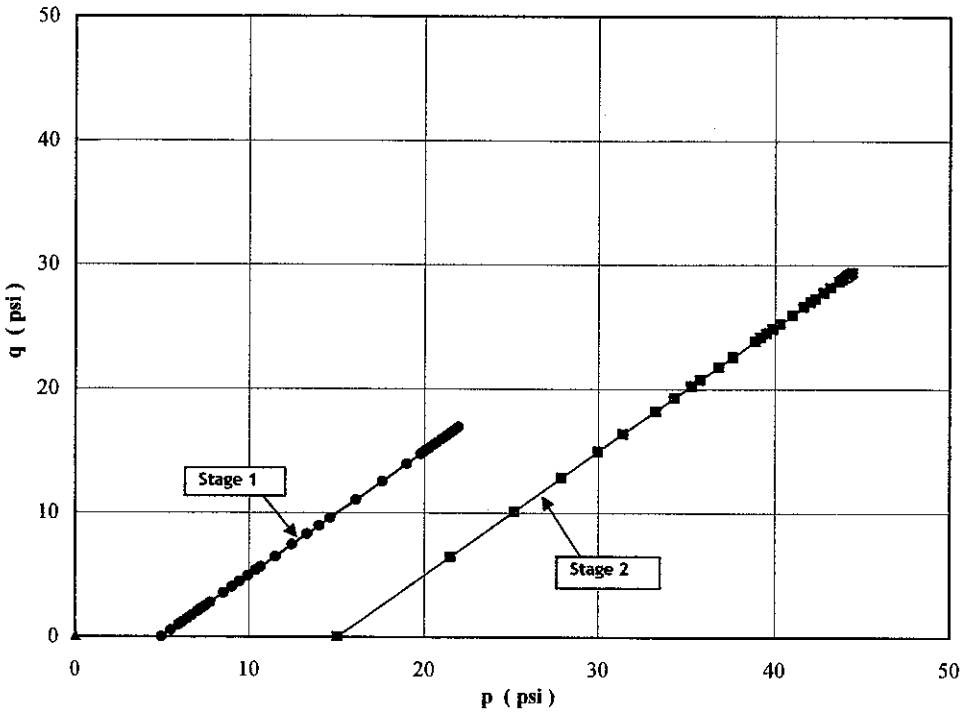
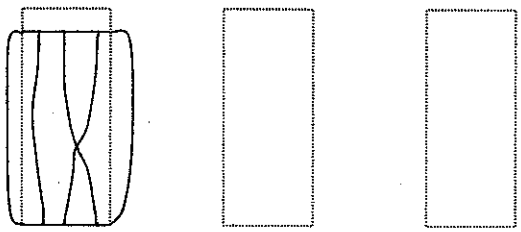
Project Name:	White Mesa Mill - Cell 4 A
Project No:	246
Site Sample ID:	B-5-2-3
Lab Sample No:	E102

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)

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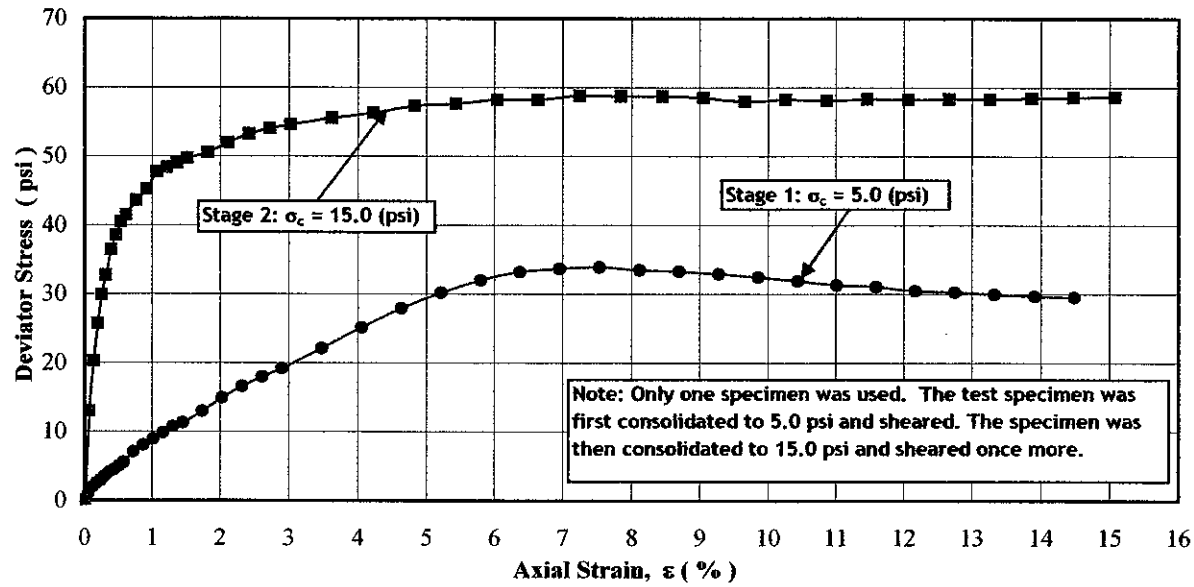
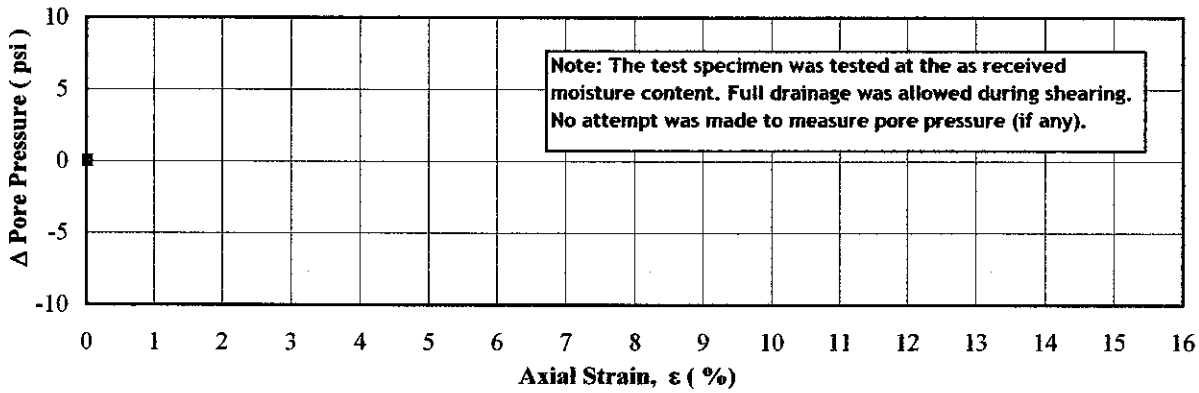
Project Name: White Mesa Mill - Cell 4 A
Project No: 246
Site Sample ID: B-5-2-3
Lab Sample No: E102

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, (%)