CELL 4A LINING SYSTEM DESIGN REPORT

FOR THE

WHITE MESA MILL BLANDING, UTAH

Prepared for:

International Uranium (USA) Corporation
White Mesa Mill
6425 S. Highway 191
P.O. Box 809
Blanding, Utah 84544
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Prepared by:

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JANUARY 2006

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1. INTRODUCTION

This report presents the results of design analyses performed in support of the Cell 4A liner construction at the White Mesa Mill Facility in Blanding, Utah (site). The San Diego office of GeoSyntec Consultants, Inc. (GeoSyntec) prepared this report for International Uranium (USA) Corporation (IUSA). This report was prepared by Ms. Jane Soule 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 the liner system and to demonstrate that the proposed Cell 4A liner system design complies with the applicable regulatory standards for the State of Utah, the U.S. 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

Cell 4A was originally constructed in 1989 and first put into service in early 1990. The entire Cell was originally lined with a 40 mil high density polyethylene (HDPE) geomembrane which was underlain by one foot of clay on the bottom of the Cell. The Cell also had a leak detection system and a slimes drain system. The liner system experienced problems from the very early days of operation. All deposits, including silt and precipitated salts from process operations, on the top of the liner system, as well as the HDPE geomembrane, were removed in early 2005 and disposed in the Cell 3 area. The Cell 4A berms and base materials were not significantly altered during the removal of the 40-mil HDPE geomembrane.

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. The capacity of these cells is diminishing and the re-construction of Cell 4A is needed to supplement evaporation/disposal capacity at the site.

The re-lined Cell 4A will 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 Information*, presents general information on the site and background information on the existing conditions at Cell 4A.

- Section 3, *Lining System Design,* presents the liner system design for Cell 4A. The design 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 4A permit documents include Design Drawings (Appendix A), a Construction Quality Assurance (CQA) Plan (Appendix B), Technical Specifications (Appendix C), Existing Berm and Clay Liner Construction Documentation (Appendix D), and engineering design calculations (Appendix E).

2. BACKGROUND AND SITE CONDITIONS

2.1 Site Location

The location of the site is shown on Sheet 1 of the Design Drawings (Appendix A). The site is located approximately 6 miles (9.5 kilometers) 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 4A is located south of the eastern half of Cell 3. The site plan is shown on Sheet 2 of the Design Drawings.

2.2 Climatology

The climate of southeastern Utah is classified as dry to arid continental. 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 (34 cm) (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 maximum temperatures are approximately 91°F in July (http://www.city-data.com/city/Blanding-Utah.html).

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 68 inches (173 cm) (NOAA, 1977), with the largest evaporation occurring in July. Values of pan coefficients range from 60% to 81%. The annual lake evaporation rate

for the site is 47.6 inches (120.9 cm) and the net evaporation rate is 34.2 inches per year (86.8 cm/yr).

2.3 Topography

The regional topography is relatively flat, with drainage traveling from the north to south in the region. The existing Cell 4A has a surface area of approximately 40 acres and is approximately 40 feet deep at the lowest corner. The existing Cell 4A bottom slopes from the northeast to the southwest at approximately one (1) percent. Cell 4A was constructed in 1989 by building compacted fills on the south and west side of the Cell and excavating the interior to gain additional volume. The existing berms are inclined at approximately 3Horizontal:1Vertical (3H:1V), with the exception of the western berm which is inclined at approximately 2H:1V on the interior slope and 3H:1V on the exterior slope. A discussion of the berm construction is provided in Section 2.4.1 of this Section.

2.4 Existing Soil Conditions

2.4.1 Soil Berms

Based on a review of the construction field reports, the existing soil berms are constructed of compacted sandy silt, and silty to clayey sand. Based on available records, compaction tests (with nuclear gauge and sand cone) were performed at a rate of approximately 1 test per 682 cubic yards (CY) of soil fill. Additional testing included Atterberg limits (1 per approximately 3,751 CY), soil gradation (1 per approximately 3,751 CY), and moisture density relations (standard Proctor, 1 per approximately 6,804 CY). Records indicate that the berm soil was compacted to meet the project specifications of 95 percent compaction of the maximum dry density per ASTM D698 (standard Proctor) at a moisture content of ±2% of optimum. Copies of the field test data are provided in Appendix D.

Since the soil berms appear to have been constructed in general accordance with the standard industry practice, there is no need to reconstruct them.

2.4.2 Clay Liner

The liner system constructed in 1989 included a 12-inch thick layer of

compacted clay on the bottom of Cell 4A. Based on construction quality assurance/quality control testing, the soil used for the compacted clay liner is a sandy lean clay. Laboratory testing during construction included sieve analyses, Atterberg limits, and compaction tests. The test results for these analyses are presented in Appendix D. This clay liner is currently exposed in the bottom of the existing Cell 4A area.

2.5 Surface Water

Surface water at the facility is diverted around all the Cells including Cell 4A. Surface water run-on into Cell 4A is limited to the perimeter access road surrounding the Cell and direct precipitation into Cell 4A.

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. No changes to the existing groundwater monitoring plan are proposed by this project.

2.7 Tailings

Cell 4A will accept process liquids, tailings, and by-products associated with on-site 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 0.023 inches (0.6 mm).

3. LINER SYSTEM DESIGN

The liner system is designed to provide a cell for disposal of by-products from the on-site processing operations while protecting the groundwater beneath the site. The liner system is designed to meet the Best Available Technology requirements of the Utah Administrative Code (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 980 acre-feet (1.6 million cubic yards) of tailings with 3-feet of freeboard. The lowest elevation in Cell 4A is the sump located in the southwest corner at an elevation of approximately 5,556 feet above mean sea level (MSL).

Sideslopes will remain at the current inclinations, ranging from approximately 3H:1V to 2H:1V. Access to the bottom of Cell 4A for construction is provided at the northeast corner, which is inclined at approximately 6H:1V. The existing, approximately 15 feet wide, access road that surrounds the entire Cell 4A will be maintained as an unpaved road.

3.2 Earthwork

Limited earthwork for the project is expected within the bottom area of the cell and for anchor trenches at the top of the perimeter berms. As discussed in Section 1.2, the existing geomembrane liner and

overlying waste materials have been removed from Cell 4A. The clay liner constructed in 1989 has been exposed and will be used as the subgrade surface for the new liner system (see Section 3.3 below). Earthwork will be limited to excavation of a 4 foot deep emergency spillway from Cell 3 to Cell 4A, excavation of the sump area in the southwest corner of the cell, minor re-grading of the bottom area of the cell for the purposes of preparing subgrade for the geosynthetic liner system installation, excavation of leak detection system trenches, and excavation of anchor trenches on top of the perimeter berms. Fill soil will consist of anchor trench backfill, as discussed in Section 3.3.5.

3.3 Liner System

A double liner system is proposed for Cell 4A, 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);
- Geonet Drainage Layer (Leak Detection System);
- 60 mil smooth HDPE geomembrane;
- Geosynthetic Clay Liner (GCL); and } (Composite Secondary Liner)
- Prepared Subgrade.

3.3.1 Slimes Drain System

A slimes drain system will be placed on top of the primary geomembrane liner 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, drainage aggregate, cushion geotextile, and geosynthetic drainage materials that will provide a means to drain the tailings disposed within Cell 4A. The slimes drain system is shown on Sheets 4, 5, and 6 of the Drawings (Appendix A).

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 E, while Appendix C,

Section 02616 provides material specifications for the pipe and drainage aggregate.

The cushion geotextile is designed to protect the underlying primary 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 E, while Appendix C, Section 02771 provides material specifications.

The Slimes Drain sump will include a sideslope 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 Drawings (Appendix A).

3.3.2 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 survive 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 Drawings (Appendix A).

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 Construction Quality Assurance (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.

3.3.3 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 200-mil thick geonet and a network of

gravel trenches throughout the bottom of the cell. 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 any seepage to the drain sump. The LDS is shown on Sheets 4, 5 and 6 of the Drawings (Appendix A).

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 E, while Appendix C, Section 02616 provides material specifications for the pipe and 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. The design analyses for the cushion geotextile are presented in Appendix E, while Appendix C, Section 02771 provides material specifications.

The LDS sump will include a sideslope 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 Drawings (Appendix A).

3.3.4 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.

3.3.4.1 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 Drawings (Appendix A).

3.3.4.2 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 $1x10^{-9}$ cm/s (Daniel and Scranton, 1996). Since 1986, GCLs have been increasingly used as an alternative to compacted clay liners (CCLs) on containment projects due to their low cost, ease of construction/placement, and resistance to freeze-thaw and wet-dry cycles. In general, the USEPA and the containment industry accepts that GCLs are hydraulically equivalent to a minimum of 2 feet of compacted clay liner consisting of 1×10^{-7} cm/sec soil materials.

To demonstrate that a secondary composite liner system consisting of a 60-mil HDPE geomembrane overlying a geosynthetic clay liner (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 compacted clay liner (CCL) having a saturated hydraulic conductivity less than 1×10^{-7} cm/s, GeoSyntec prepared an engineering analysis, which is presented in Appendix E. Based on this site specific analysis, which accounts 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 0.16in. (4 mm). 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 geotexile components of the GCL can result in protection from puncture. The geotextile components of the GCL for Cell 4A

are designed to protect the overlying secondary HDPE geomembrane from puncture due to the protrusions from the subgrade and the anticipated loading associated with the maximum height of overlying tailings. The design analyses for the geotextile components of the GCL are presented in Appendix E, while Appendix C, Section 02772 provides material specifications.

- 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 4A 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 4A 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 dessication 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 dessication, 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 4A 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.

3.3.5 Subgrade

As discussed in Section 2.4.2, the subgrade in the bottom of the Cell consists of approximately 12 inches of compacted clay from the original 1989 liner construction. Prior to placement of the secondary composite liner system, the existing clay subgrade and side slopes will be prepared by minor regrading and compacting to a smooth, consistent surface.

3.3.6 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 E. The anchor trench will be 2 feet deep and 2 feet wide and filled with compacted soil, see Sheet 5 of the Design Drawings (Appendix A).

3.4 Splash Pad

Approximately three splash pads will be constructed to allow filling of Cell 4A without damaging the liner system. The splash pad consists of an additional geomembrane placed along the sideslope 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 Design Drawings (Appendix A). The locations of the splash pads will be finalized in the field during construction, based on site operational needs.

3.5 Emergency Spillway

An emergency spillway will be constructed between Cells 3 and 4A. 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 E. 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 E. The Cell 4A liner will extend beneath the concrete as shown on Sheet 7 of the Design Drawings (Appendix A).

4. SUMMARY AND CONCLUSIONS

This report presents the design engineering evaluations for the Cell 4A Liner System at the White Mesa Mill Facility. The calculations presented in this engineering report establish the dimensions and properties of the liner system components. The design plans and details are presented in the project 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.

Gregory T. Corcoran, P.E.
Utah Registration No. 6020077-2202

5. REFERENCES

Daniel, D.E., and Scranton, H.G. (1996), "Report of 1995 Workshop of Geosynthetic Clay Liners," EPA/600/R-96/149, June, 93 pgs.

Koerner, R.M. and Daniel, D.E. (1993) "Technical Equivalency Assessment of GCLs to CCLs." "Proc. Seventy Annual GRI Seminar, Geosynthetic Research Institute, Philadelphia, PA."

Western Regional Climate Center (WRCC), 2005. Based on data from 12/8/1904 to 3/31/2005 at Blanding, Utah weather station (420738).

Appendix A

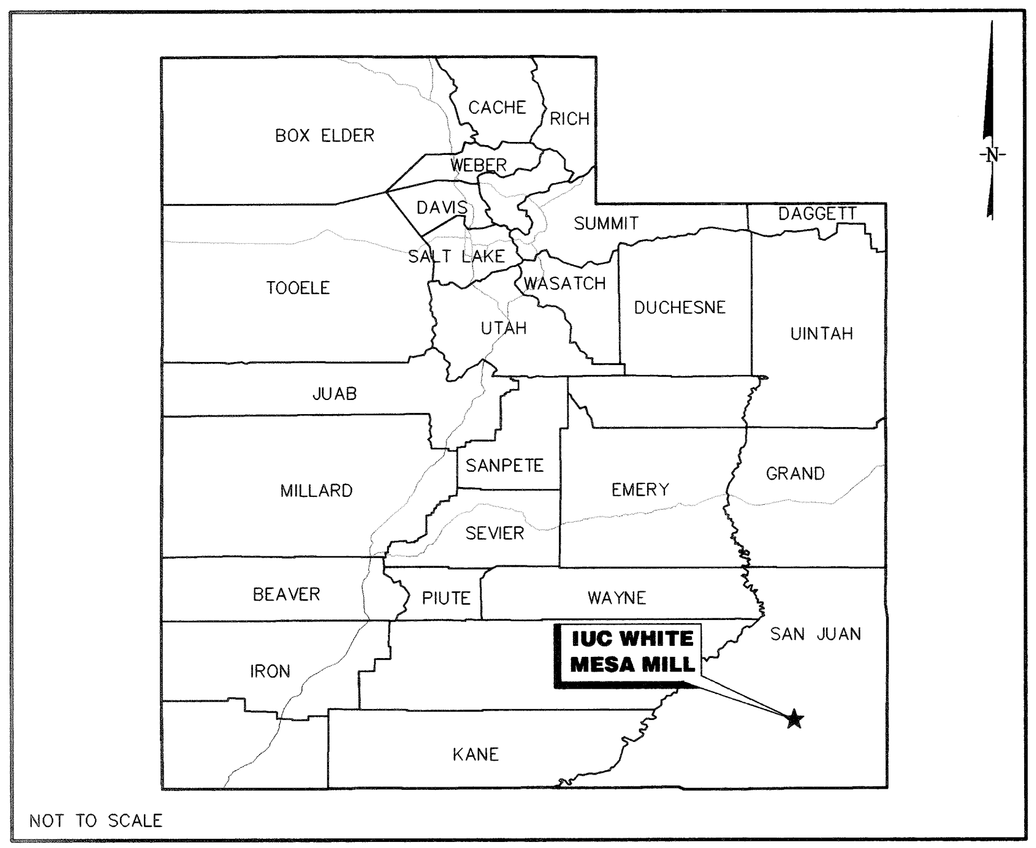
Design Drawings

PERMIT DRAWINGS - NOT FOR CONSTRUCTION

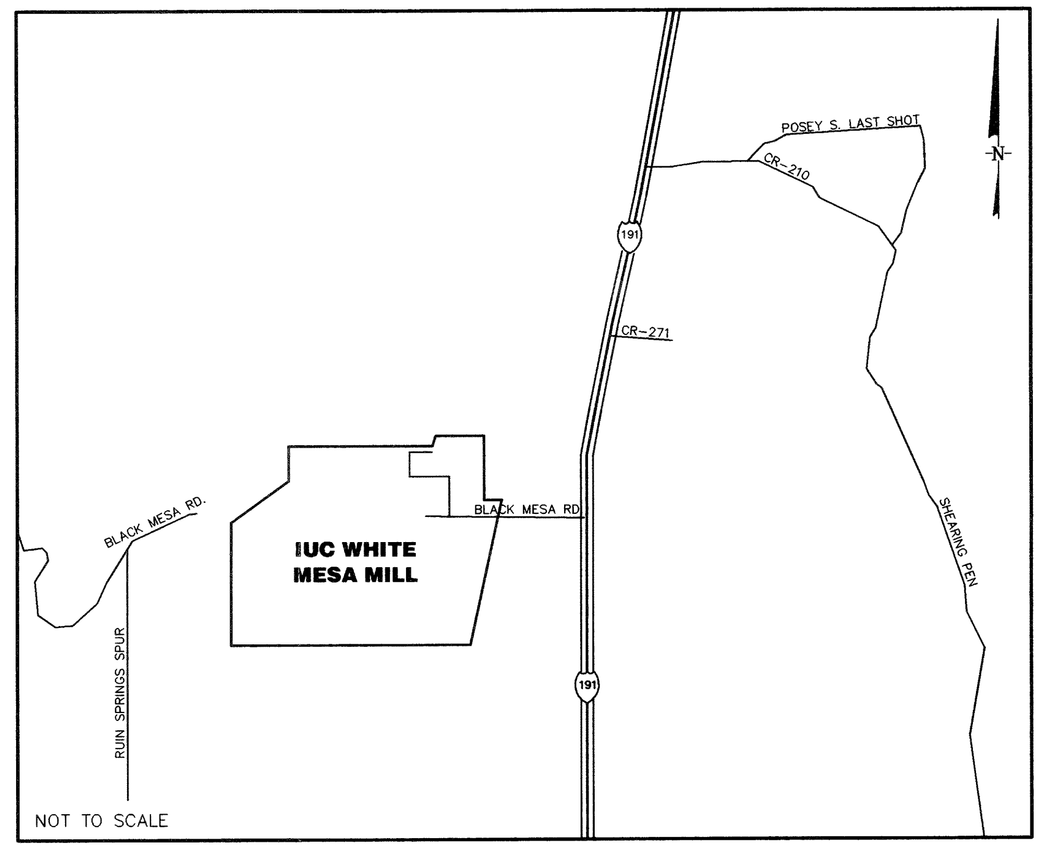
IUC WHITE MESA MILL
CELL 4A LINING SYSTEM
BLANDING, UTAH
JANUARY 2006

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



VICINITY MAP



LOCATION MAP

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BLANDING, UTAH 84511
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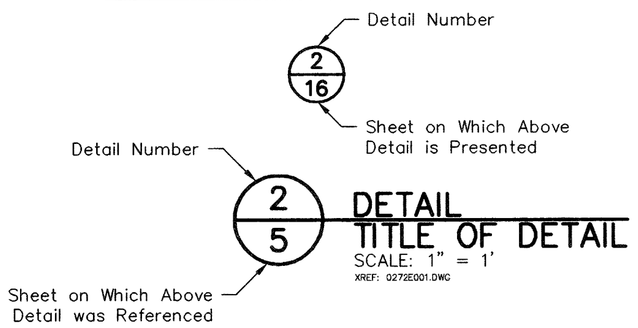
DETAIL IDENTIFICATION LEGEND



Example: Detail Number 2 Presented on Sheet No. 16 was Referenced on Sheet No. 5.

NOTES: ABOVE REFERENCING SYSTEM ALSO APPLIES TO SECTION IDENTIFICATIONS.

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LEGEND

- EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
- EXISTING FENCE
- 5560 PROPOSED BASE GRADING (2' CONTOUR)
- LIMIT OF LINER SYSTEM
- SPLASH PAD

N

250 125 0 250 500

SCALE IN FEET

NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM INTERNATIONAL URANIUM (USA) CORPORATION.
2. EXISTING WELLS, PIPING, AND OTHER SITE FEATURES SHALL BE PROTECTED IN PLACE.

PERMIT DRAWINGS
NOT FOR CONSTRUCTION

IUC

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PROJECT:
CELL 4A, WHITE MESA MILL
BLANDING, UTAH

TITLE:
SITE PLAN

MARK	DATE	REVISION	BY	APPROVED

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

SIGNATURE

DATE

SEAL

DATE:	JANUARY 2006	SCALE:	AS SHOWN
DESIGN BY:	GTC	JOB NO.:	SC0349-01
DRAWN BY:	SLB	FILE NO:	0349C002
CHECKED BY:	GTC	DOCUMENT NO:	-----
REVIEWED BY:	GTC	DRAWING NO:	2 OF 7
APPROVED BY:	GTC		

EXISTING CELL 3

EMERGENCY SPILLWAY (10/7)

CELL 4A

5560 5562 5564 5566 5568 5570 5572 5574

1%

(A/5) (5/5) (D/5) (7/5) (9/6) (6/5)

15'X10' SUMP

LIMIT OF LINER

SEE NOTE 4

LEGEND

- EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
- EXISTING FENCE
- 5560 PROPOSED BASE GRADING (2' CONTOUR)
- LIMIT OF LINER SYSTEM
- SPLASH PAD (NOTE 3)
- 1% SLOPE DIRECTION AND GRADE

N

100 50 0 100 200
SCALE IN FEET

NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM INTERNATIONAL URANIUM (USA) CORPORATION.
2. REMOVE AND REPLACE FENCE AS NECCESSARY TO CONSTRUCT ANCHOR TRENCH AND SPILLWAY.
3. SPLASH PAD LOCATION TO BE FIELD SELECTED BY OWNER.
4. ANCHOR TRENCH SHALL BE INSTALLED ON LINED SLOPE SIDE OF WELL LOCATION.

PERMIT DRAWINGS
NOT FOR CONSTRUCTION

IUC

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TELEPHONE: (858) 674-6559

PROJECT:
CELL 4A, WHITE MESA MILL
BLANDING, UTAH

TITLE:
BASE GRADING PLAN

MARK	DATE	REVISION	BY	APPROVED

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SIGNATURE

DATE

SEAL

DATE:	JANUARY 2006	SCALE:	AS SHOWN
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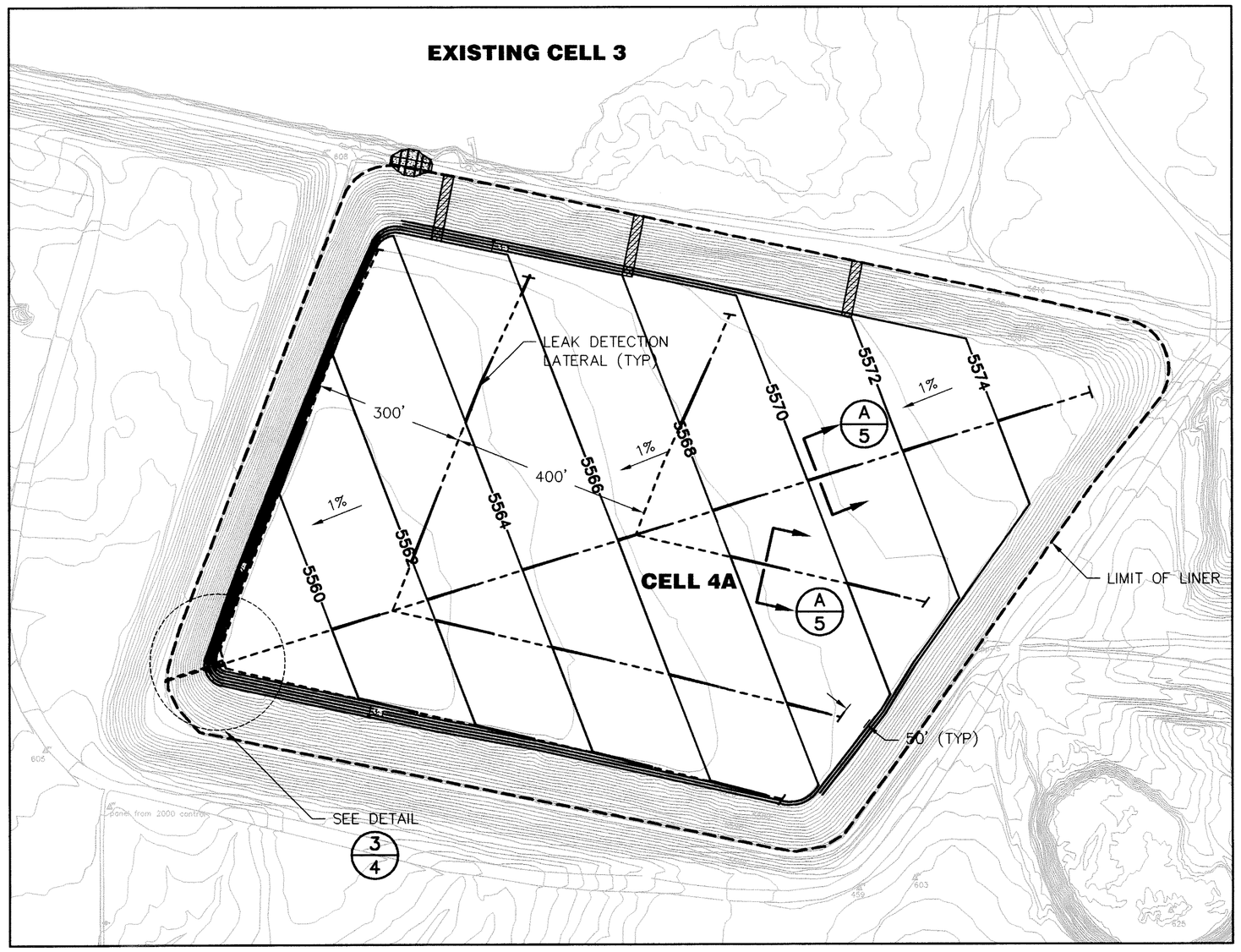
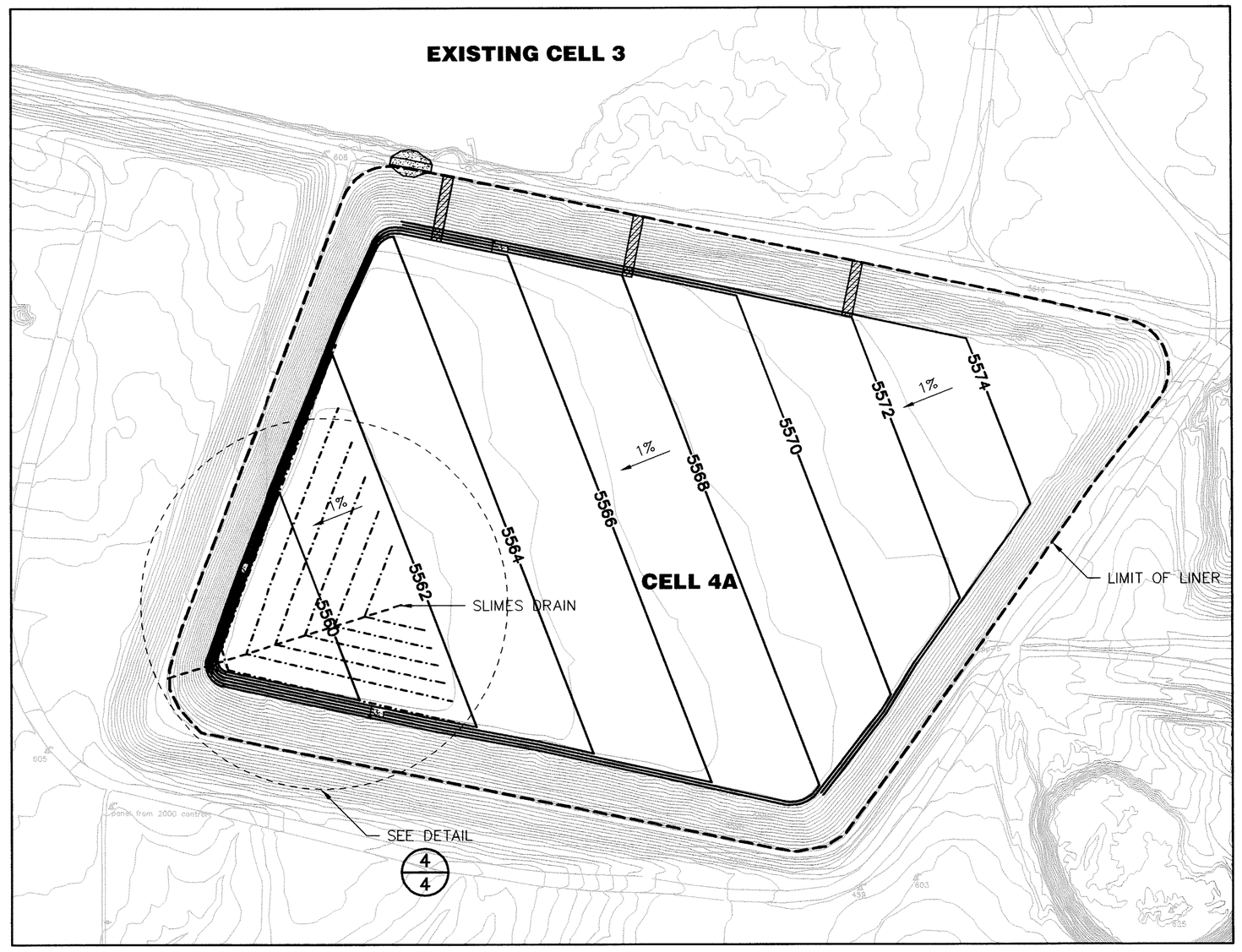


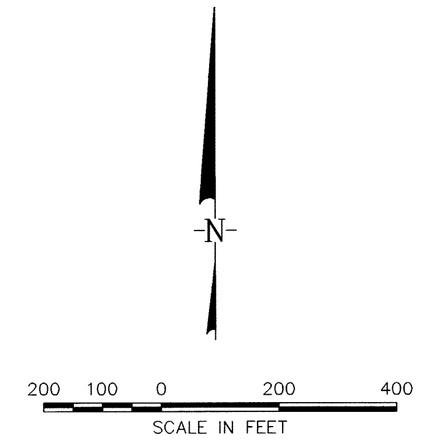
1/4 PLAN
LEAK DETECTION SYSTEM
SCALE: 1" = 200'

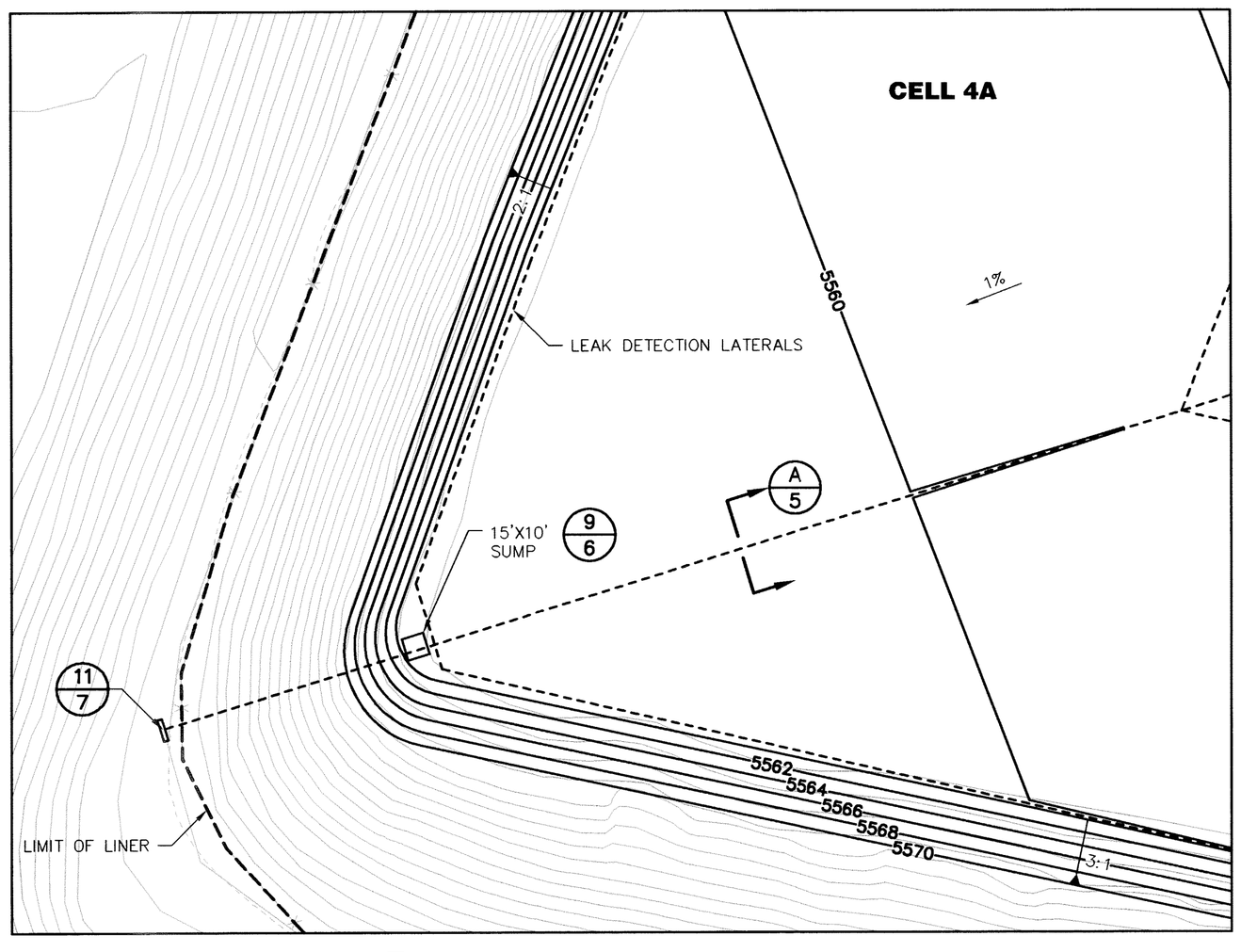


2/4 PLAN
SLIMES DRAIN SYSTEM
SCALE: 1" = 200'

LEGEND

- EXISTING TOPOGRAPHY (FEET ABOVE M.S.L.)
- EXISTING FENCE
- 5560 PROPOSED BASE GRADING (10' CONTOUR)
- 4" ø PVC PIPE
- CONTECH STRIP DRAIN 80
- LIMIT OF LINER SYSTEM
- SPLASH PAD (NOTE 3)
- 1% SLOPE DIRECTION AND GRADE





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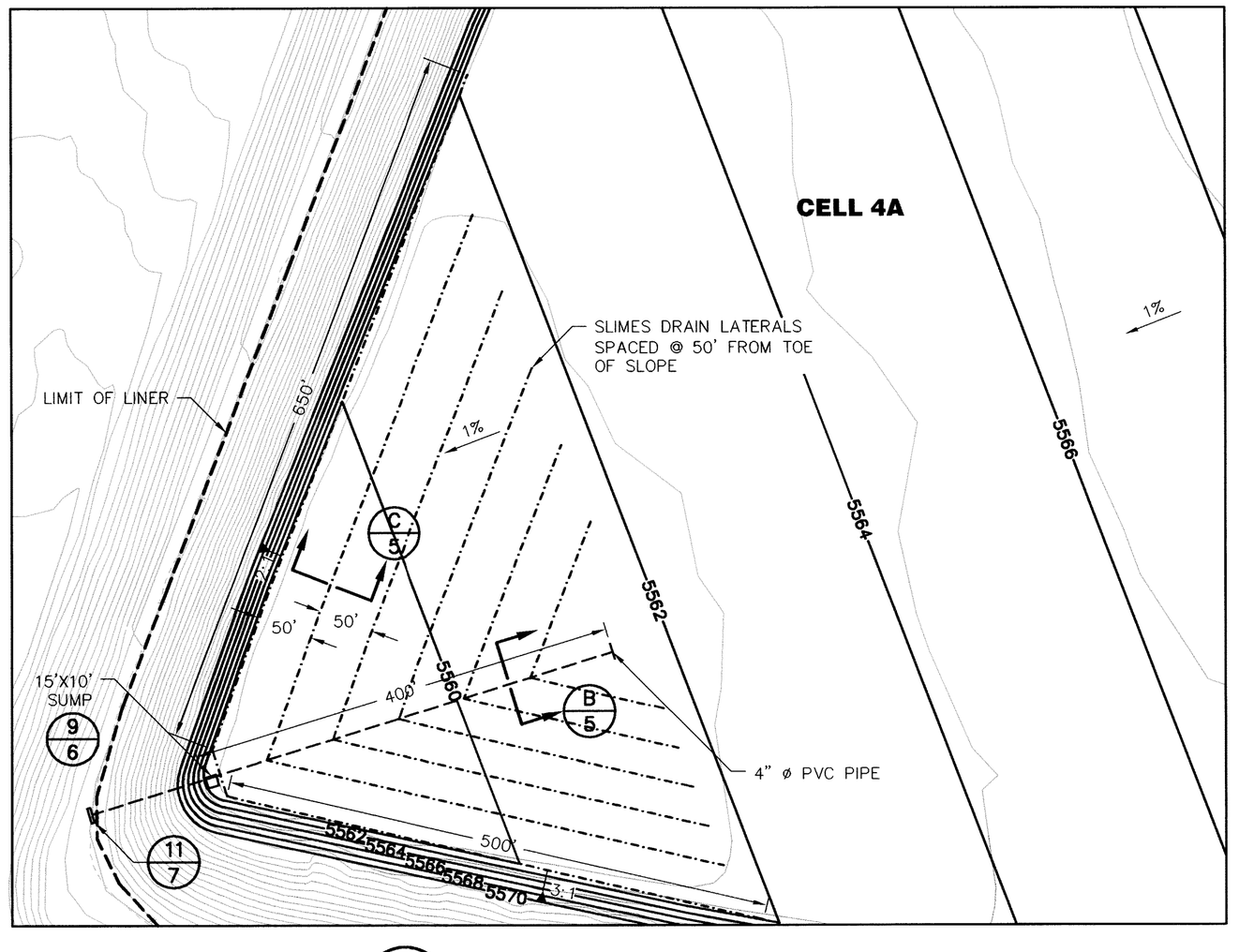

4/4 DETAIL
SLIMES DRAIN SYSTEM
SCALE: 1" = 100'
XREF: 0349X007.DWG

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NOTES:

1. EXISTING TOPOGRAPHY OBTAINED FROM INTERNATIONAL URANIUM (USA) CORPORATION.
2. REMOVE AND REPLACE FENCE AS NECCESSARY TO CONSTRUCT ANCHOR TRENCH AND SPILLWAY.
3. SPLASH PAD LOCATION TO BE FIELD SELECTED BY OWNER.

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TELEPHONE: (858) 674-6559

PROJECT:
CELL 4A, WHITE MESA MILL
BLANDING, UTAH

TITLE:
PIPE LAYOUT PLAN

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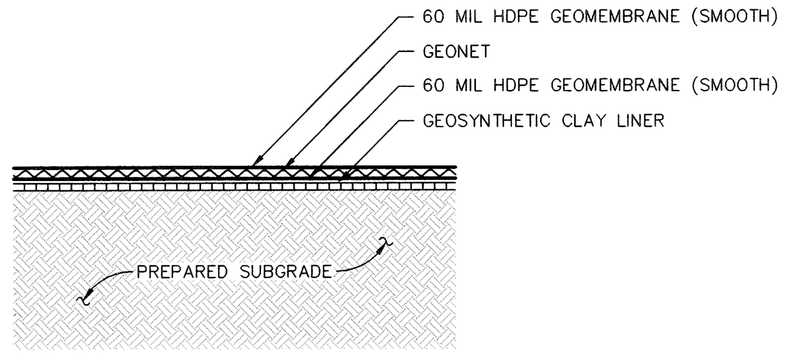
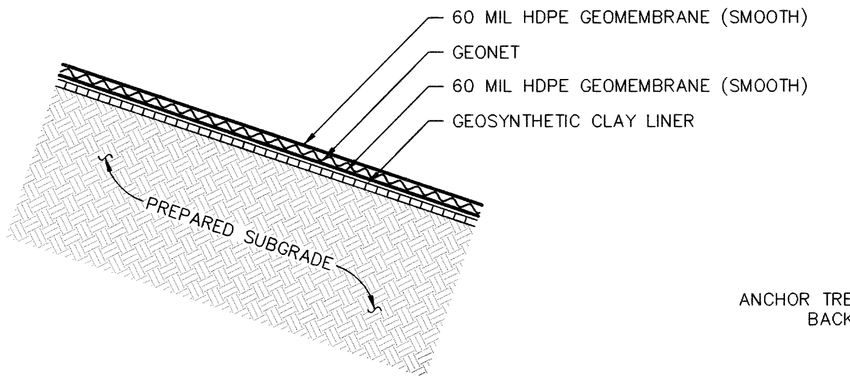


5/3 DETAIL
BASE LINER SYSTEM
SCALE: 1" = 1'
XREF: 0349X002.DWG



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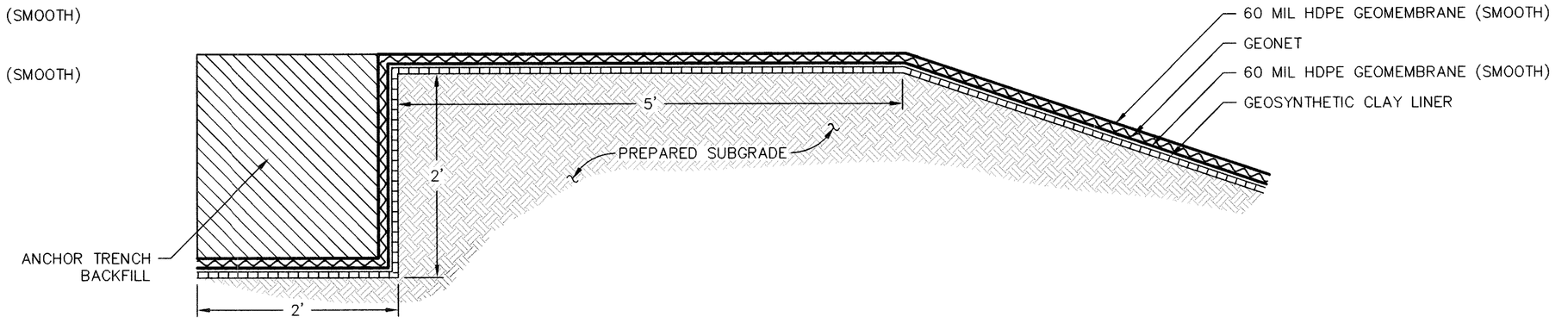
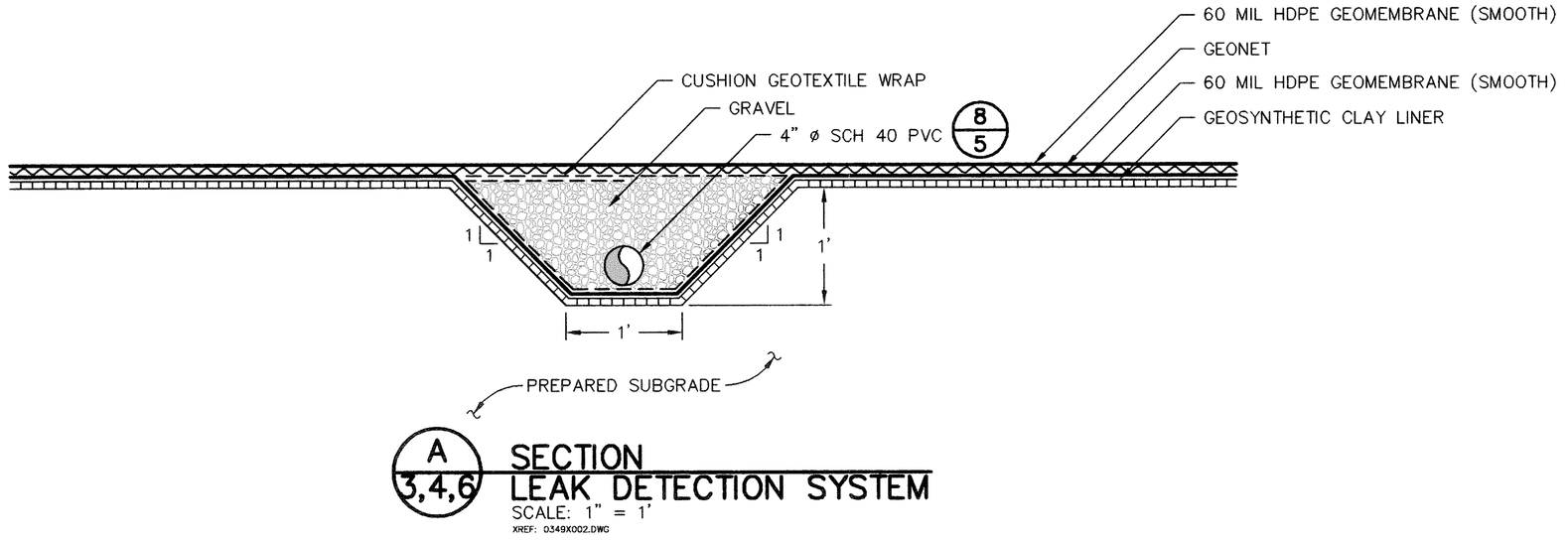


7/3,7 SECTION
ANCHOR TRENCH
SCALE: 1" = 1'
XREF: 0349X002.DWG



A/3,4,6 SECTION
LEAK DETECTION SYSTEM
SCALE: 1" = 1'
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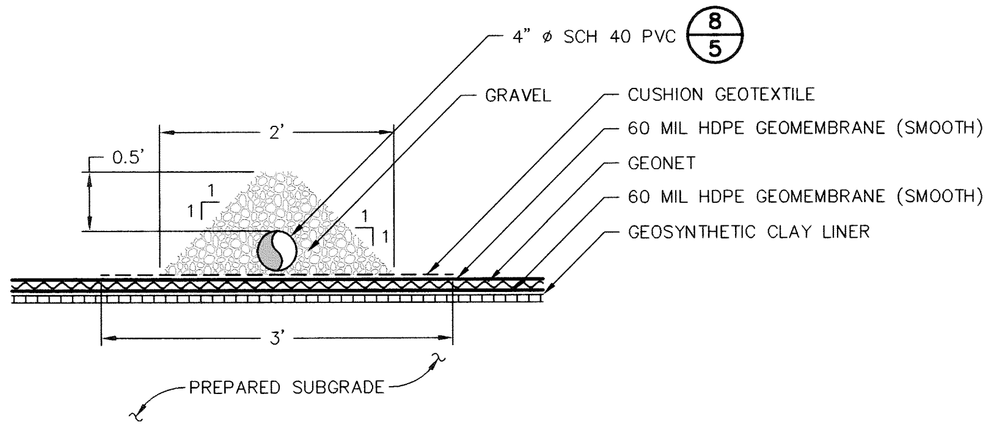

B/4,6 SECTION
SLIMES DRAIN HEADER
SCALE: 1" = 1'
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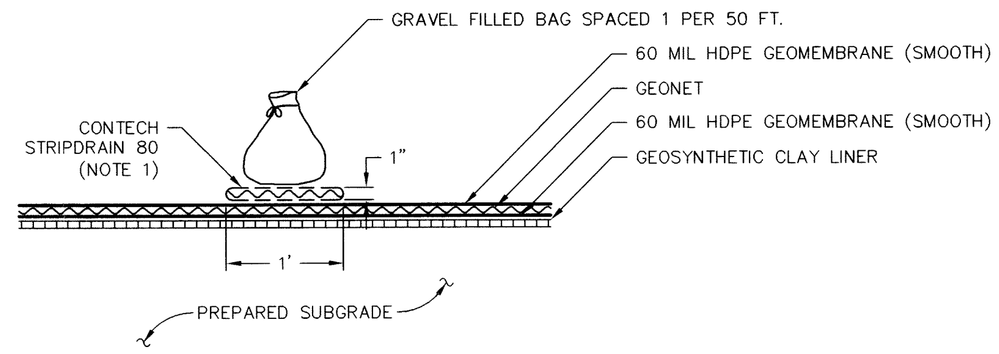

C/4,6 SECTION
SLIMES DRAIN LATERAL
SCALE: 1" = 1'
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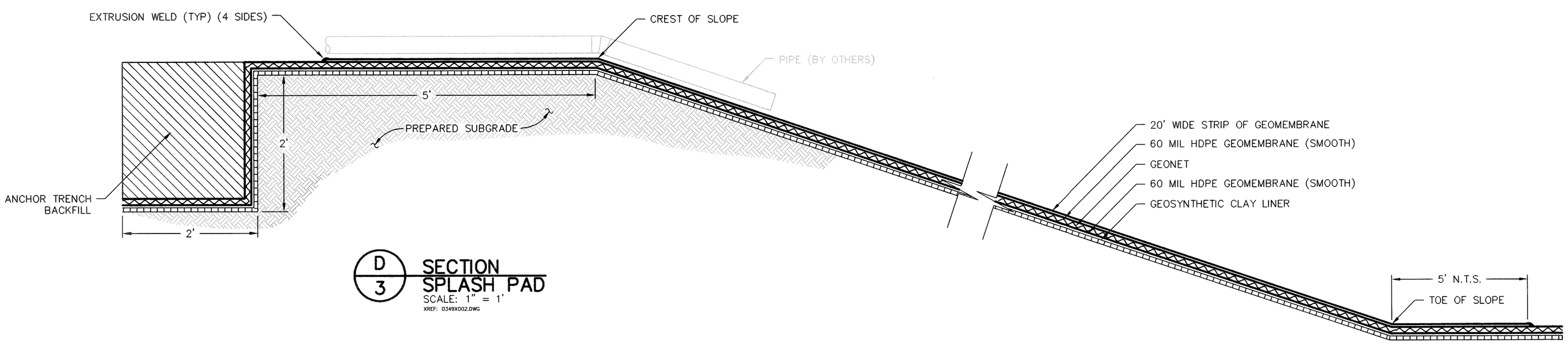
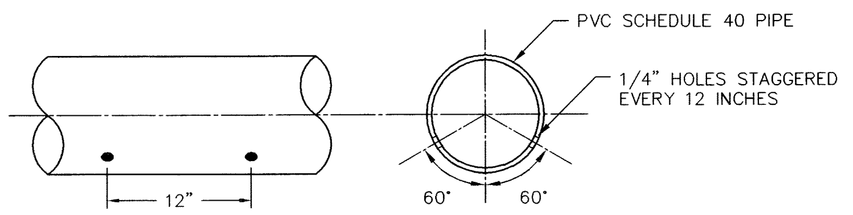


D/3 SECTION
SPLASH PAD
SCALE: 1" = 1'
XREF: 0349X002.DWG



8/5 DETAIL (TYPICAL)
PERFORTED PIPE
SCALE: N.T.S.
XREF: 0349X001.DWG

NOTES:

1. CONTECH STRIPDRAIN 80 TO CONNECT TO 4" DIAMETER PVC HEADER PIPE USING STANDARD END OUTLET FITTING. OPEN ENDS OF CONTECH STRIPDRAIN 80 TO BE TAPED CLOSED.
2. DETAILS ARE SHOWN TO SCALE INDICATED EXCEPT FOR THE GEOSYNTHETICS, WHICH ARE SHOWN AT AN EXAGGERATED SCALE FOR CLARITY. SOIL THICKNESS ARE MINIMUMS.

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PROJECT:
CELL 4A, WHITE MESA MILL
BLANDING, UTAH

TITLE:
LINING SYSTEM DETAILS I

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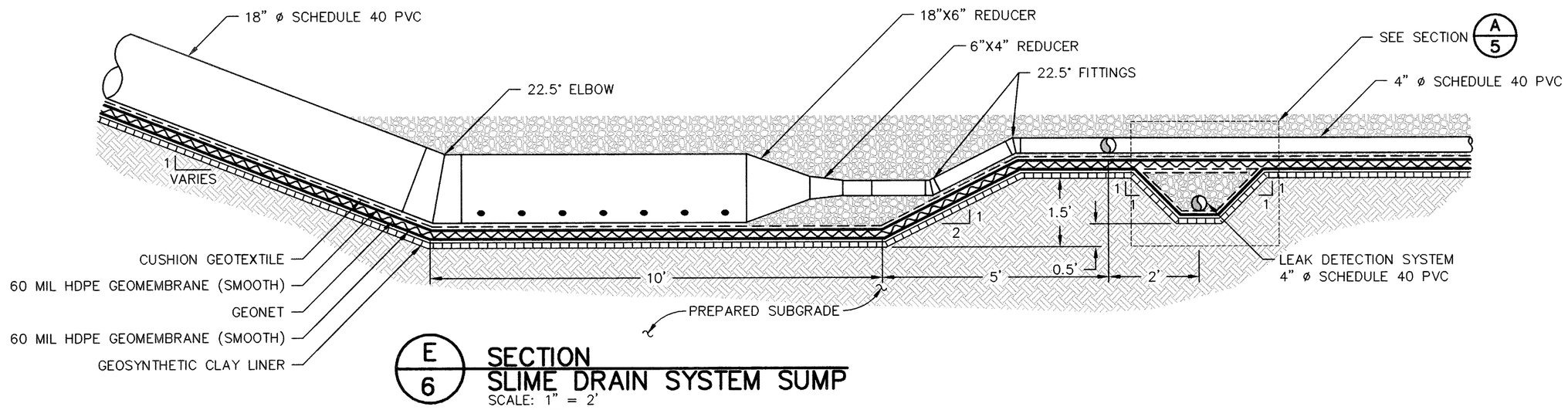
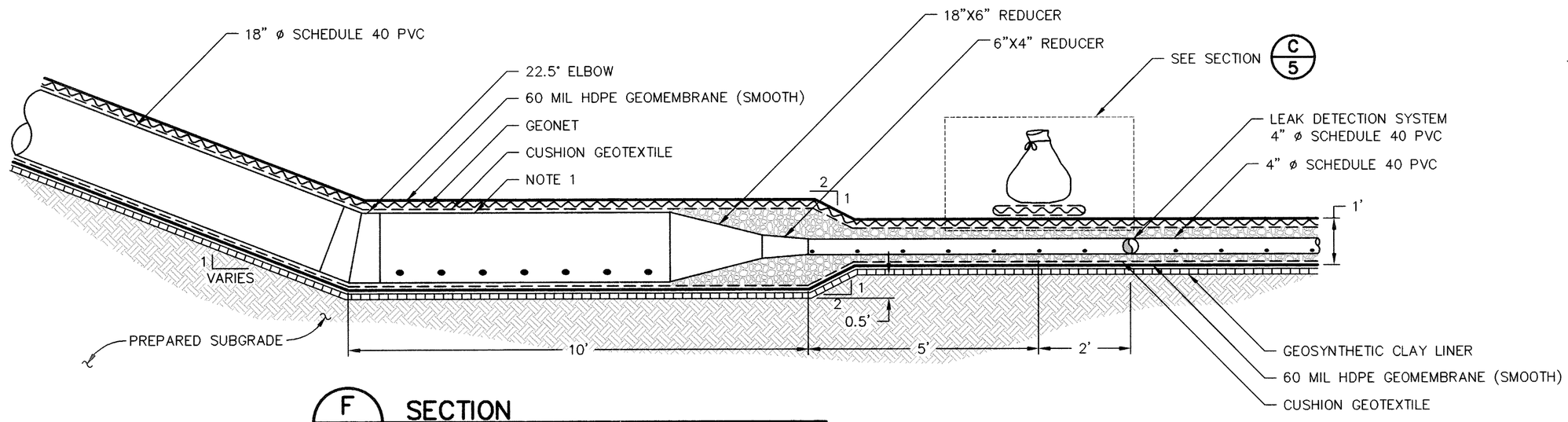
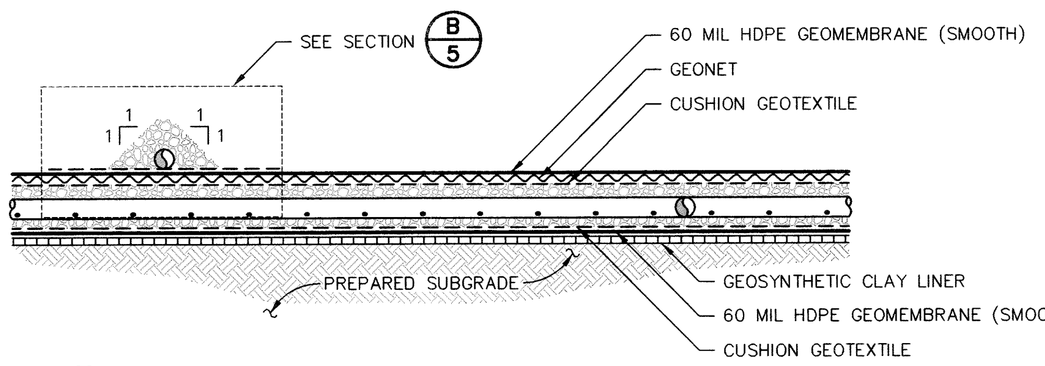


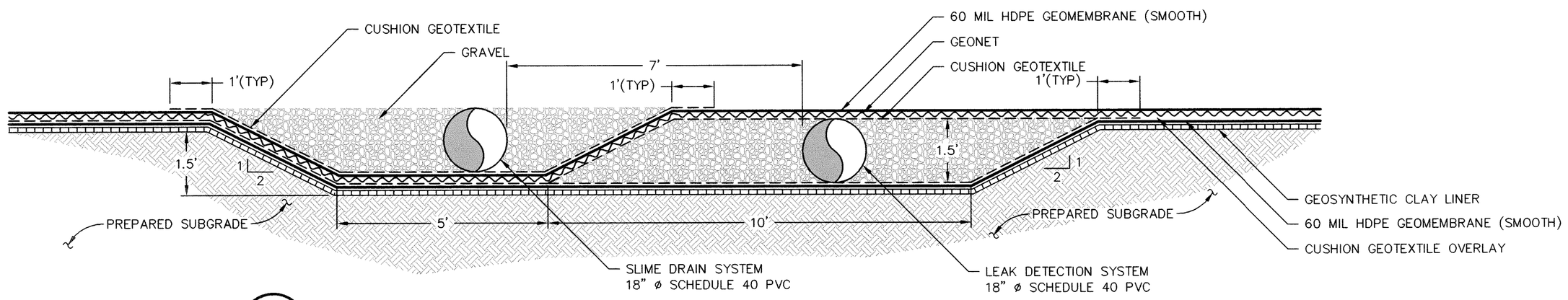
E 6 **SECTION**
SLIME DRAIN SYSTEM SUMP
SCALE: 1" = 2'
XREF: 0349X008.DWG



F 6 **SECTION**
LEAK DETECTION SYSTEM SUMP
SCALE: 1" = 2'
XREF: 0349X008.DWG



G 6 **SECTION**
SLIMES AND LEAK DETECTION SYSTEM SUMP
SCALE: 1" = 2'
XREF: 0349X008.DWG



H 6 **SECTION**
SLIMES AND LEAK DETECTION SYSTEM SUMP
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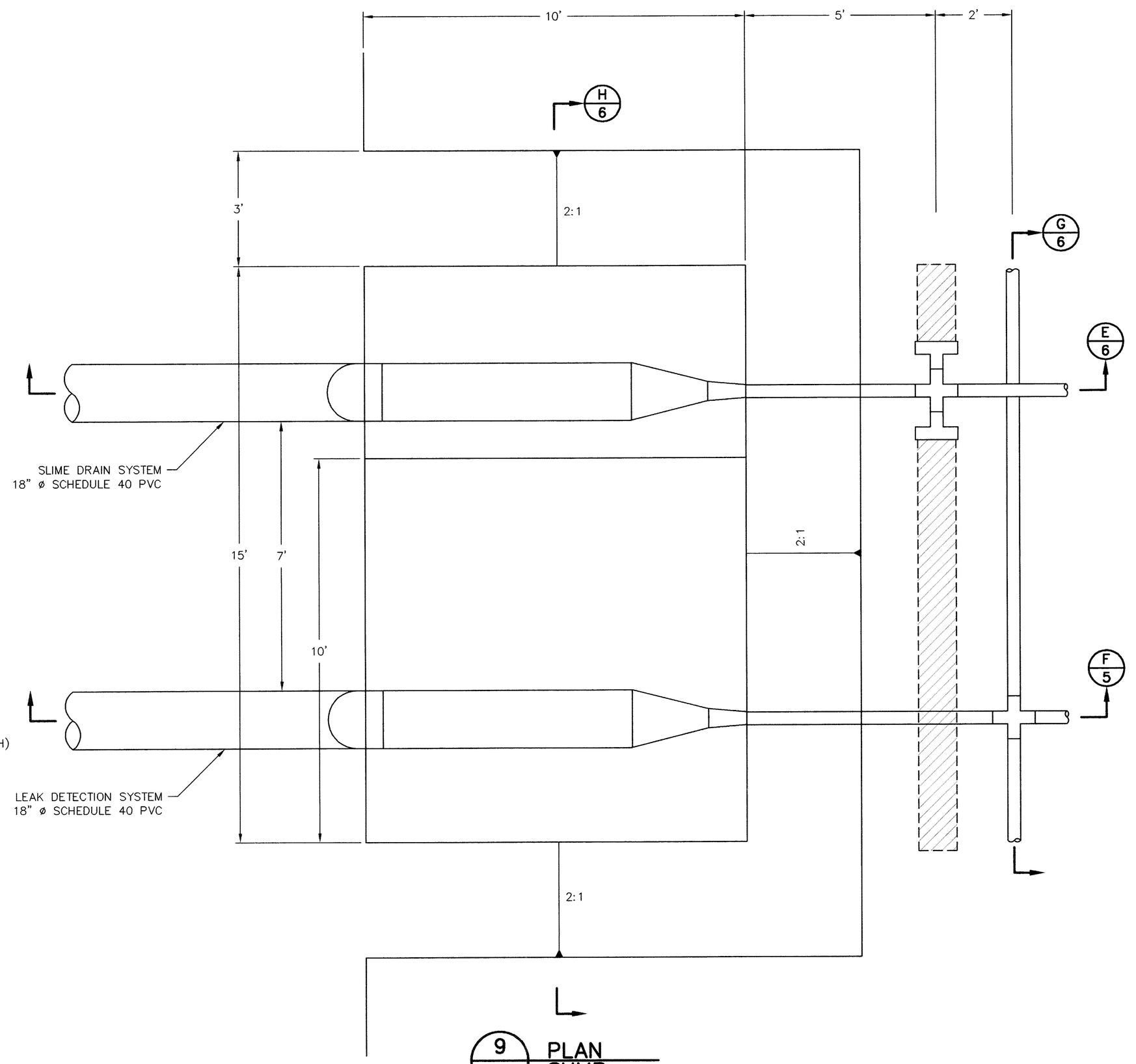

9 3,4 **PLAN**
SUMP
SCALE: 1" = 2'
XREF: 0349X008.DWG

NOTES:

1. HORIZONTAL PORTION OF 18-IN ø PVC PIPE SHALL BE PERFORATED AS INDICATED ON DETAIL 8, SHEET 5.

2. DETAILS ARE SHOWN TO SCALE INDICATED EXCEPT FOR THE GEOSYNTHETICS, WHICH ARE SHOWN AT AN EXAGGERATED SCALE FOR CLARITY, SOIL THICKNESS ARE MINIMUMS.

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BLANDING, UTAH

TITLE:
LINING SYSTEM DETAILS II

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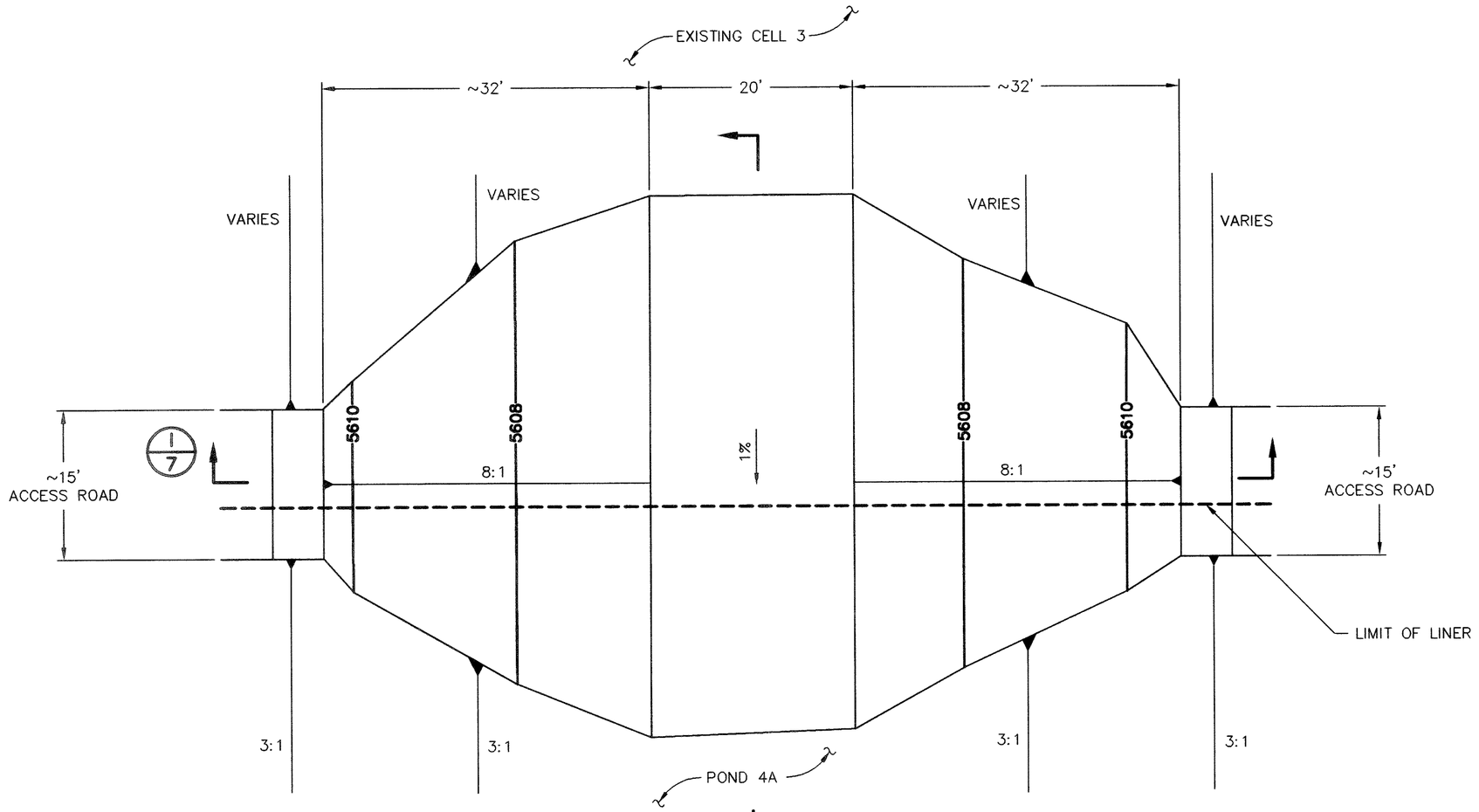
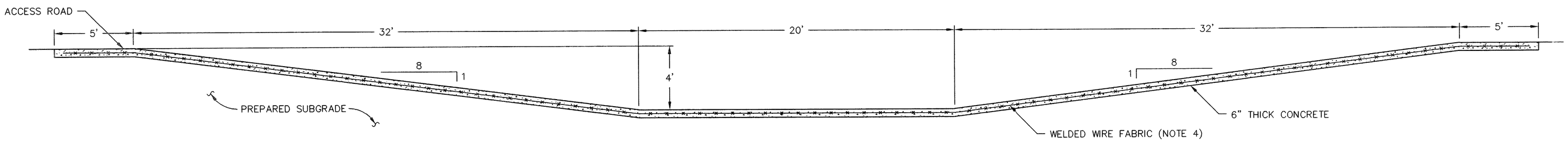


10/3 PLAN
EMERGENCY SPILLWAY
SCALE: 1" = 10'
XREF: 0349X004.DWG



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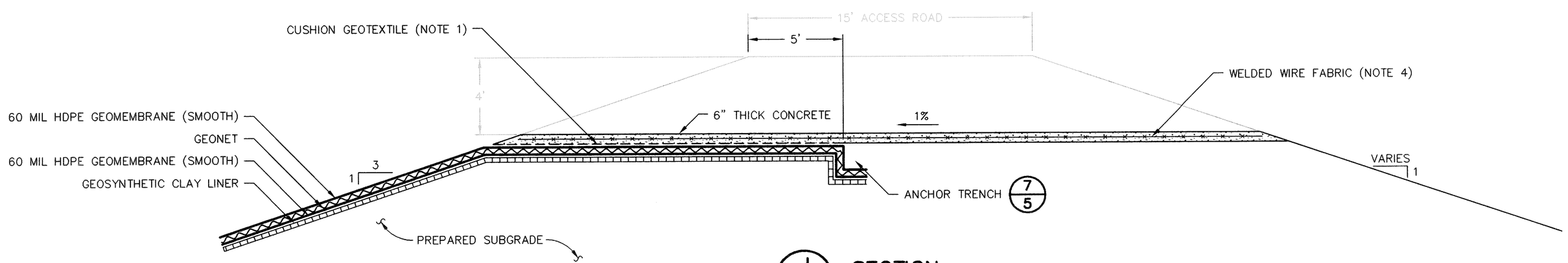

J/7 SECTION
EMERGENCY SPILLWAY
SCALE: 1" = 4'
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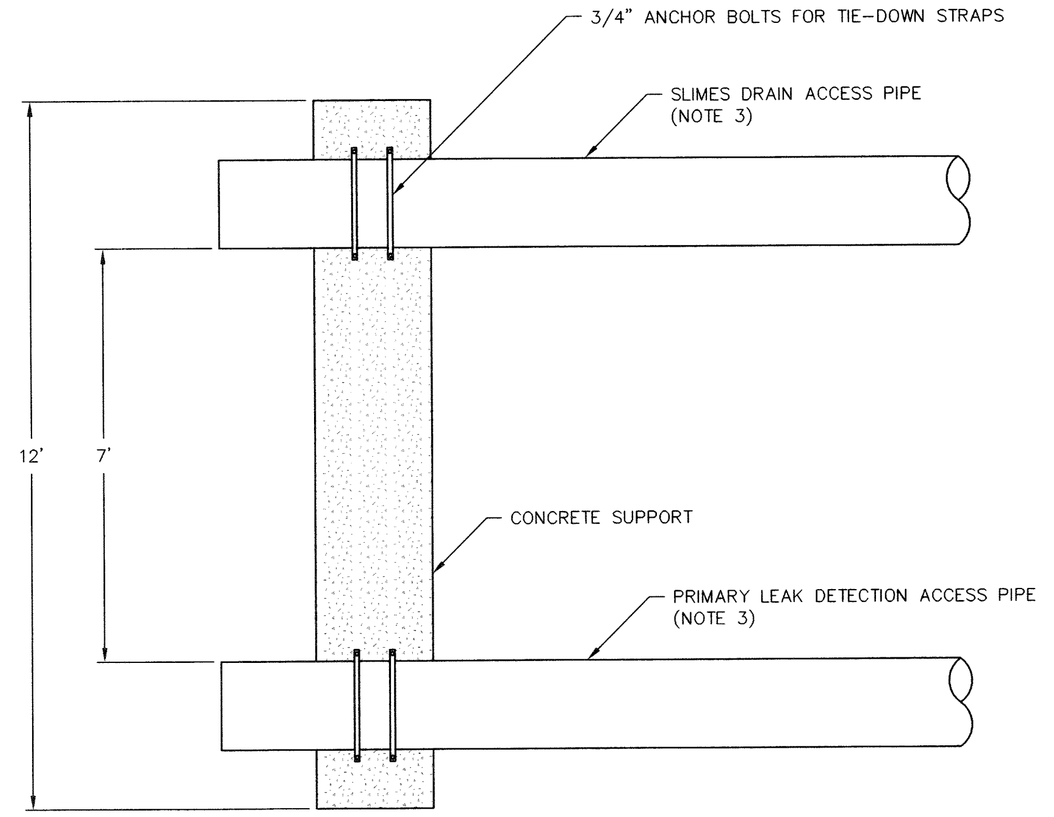

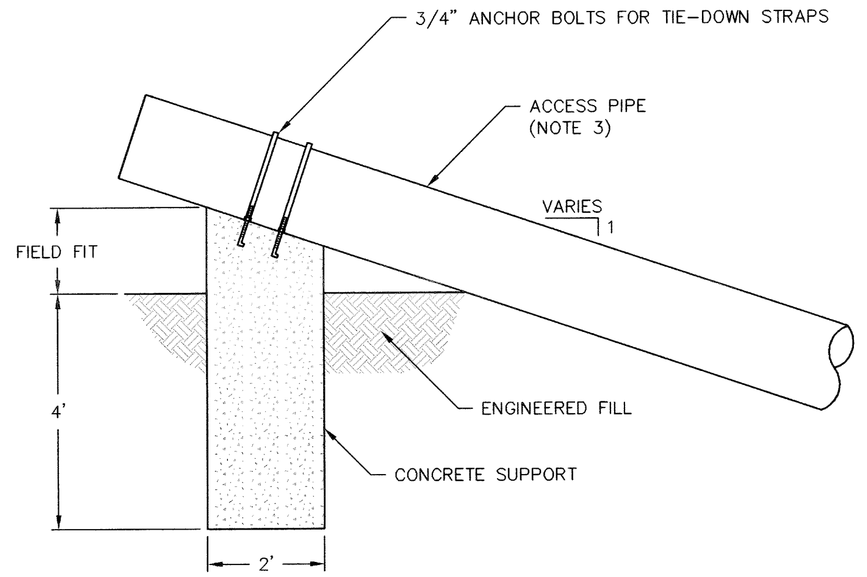

11/4 DETAIL
PIPE SUPPORT
SCALE: 1" = 2'
XREF: 0349X005.DWG

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.

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PROJECT:
CELL 4A, WHITE MESA MILL
BLANDING, UTAH

TITLE:
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Appendix B

Construction Quality Assurance Plan

CONSTRUCTION QUALITY ASSURANCE PLAN
for the Construction of
Cell 4A Lining System

IUC White Mesa Mill
Blanding, Utah

Prepared for:

International Uranium (USA) Corporation
6425 S. Highway 191
P.O. Box 809
Blanding, UT 84511
Phone: (306) 628-7798

Prepared by:

GeoSyntec Consultants
11305 Rancho Bernardo Rd.
Suite 101
San Diego, California 92127

JANUARY 2006

CERTIFICATION PAGE

CONSTRUCTION QUALITY ASSURANCE (CQA) PLAN FOR
CELL 4A LINING SYSTEM CONSTRUCTION
INTERNATIONAL URANIUM (USA) CORPORATION
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

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5 Geonet Conformance Testing Requirements

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 4A Lining System Construction at the International Uranium (IUC) Corporation White Mesa Mill Facility (site), located at 6425 S. Highway 191, Blanding, UT 84511. This CQA Plan was prepared by Mr. Chad Bird, E.I.T., of GeoSyntec Consultants (GeoSyntec), and was reviewed by Mr. Greg 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 soils and geosynthetic components of the liner system for the project. The soils, geosynthetic, and appurtenant components include 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 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 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 parties involved with the CQA;
- Section 4 describes the responsibilities of the CQA personnel;
- Section 5 describes site and project control requirements;
- Section 6 presents CQA documentation;
- Section 7 presents CQA of earthworks;
- Section 8 presents CQA of the drainage aggregates;
- 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 geonet;
- Section 13 presents CQA of the geosynthetic clay liner;
- Section 14 presents CQA surveying.

2. DEFINITIONS RELATING TO CQA

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 and/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 *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 *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 International Uranium (USA) Corporation.

2.2 Construction Manager

Responsibilities

The Construction Manager is responsible for managing the construction and implementation of the *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, *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 general accordance with this CQA Plan, the *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 general accordance with the *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 meet 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 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 who 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 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, Geosynthetic Installer, that is responsible for conducting tests in general accordance with ASTM and other applicable test standards on samples of geosynthetic materials, soil, and in the field and in either an on-site or off-site 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.

3. CQA CONSULTANTS PERSONNEL ORGANIZATION 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:

- 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, *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 on-site 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 general accordance with the Construction Documents.

3.4 CQA Site Manager

The CQA Site Manager:

- acts as the on-site 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 on-site CQA equipment;
- prepares a daily summary report for the project;
- reviews the MQC documentation;
- reviews the Geosynthetic Installer's personnel Qualifications for conformance with those pre-approved for work on site;
- notes on-site activities in daily field reports and reports to the CQA Officer and Construction Manager;
- reports unresolved deviations from the CQA Plan, *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 *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 and/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 2 working days of the meeting, draft minutes will be transmitted to representatives of parties in attendance for review and comment. Corrections and/or comments to the draft minutes shall be made within 2 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 *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 and/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 *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), and/or construction problem and solution data sheet(s).

5.5 Design and/or Specifications Changes

Design and/or specifications changes may be required during construction. In such cases, the CQA Site Manager will notify the Engineer. Design and/or

specification changes will be made with the written agreement of the Engineer and will take the form of an addendum to the *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 the Professional Engineer licensed in the State of Utah. The CQA report will acknowledge: (i) that the work has been performed in compliance with the *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 *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 general accordance with the CQA Plan and *Technical Specifications*;
- documentation that field seams were non-destructively tested using a method in general 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;
- 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. EARTHWORK

6.1 Introduction

This section prescribes the CQA activities to be performed to monitor that prepared subgrade is constructed in general accordance with *Drawings* and *Technical Specifications*. The prepared subgrade construction procedures to be monitored by the CQA Consultant, if required, shall include:

- vegetation removal;
- subgrade preparation;
- fine-grading; and
- anchor trench excavation and backfill.

6.2 CQA Monitoring Activities

6.2.1 Vegetation Removal

The CQA Site Manager will monitor and document that vegetation is sufficiently cleared and grubbed in areas where geosynthetics are to be placed. Vegetation removal shall be performed as described in the *Technical* Specification and the *Drawings*.

6.2.2 Grading

Construction of the Cell 4A liner system will require minor re-grading in certain areas. The CQA Site Manager shall monitor and document that site re-grading performed meets the requirements of the *Technical Specifications* and the *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 *Drawings*.

6.2.3 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 *Drawings*. The CQA Site Manager will monitor, at a minimum, that:

- the anchor trench is free of sharp rocks, debris and other undesirable materials and that particles are no larger than 6-inches in longest dimension;
- the anchor trench is constructed to the lines and grades shown on the *Drawings*; and
- compaction requirements are met, through visual observations, as specified in the *Technical Specifications.*

6.3 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.

6.3.1 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.

6.3.2 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.

7. DRAINAGE AGGREGATE

7.1 Introduction

This section prescribes the CQA activities to be performed to monitor that drainage aggregates are constructed in general accordance with *Drawings* and *Technical Specifications*. The drainage aggregates construction procedures to be monitored by the CQA Consultant include drainage aggregate placement.

7.2 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 general accordance with the current versions of the corresponding American Society for Testing and Materials (ASTM) test procedures. The test methods indicated in Table 1 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 aggregate testing for material qualification and material conformance will conform to the minimum frequencies presented in Table 1. The

frequency of aggregate testing shall conform to the minimum frequencies presented in Table 1. 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

With the exception of qualification samples, sampling locations will be selected by the CQA Site Manager. Conformance samples will be obtained from borrow pits and/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.

7.3 CQA Monitoring Activities

7.3.1 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 *Drawings*; and
- monitoring that the construction activities do not cause damage to underlying geosynthetic materials.

7.4 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.

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-tests when the work deficiency is to be corrected.

7.4.2 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.

8. POLYVINYL CHLORIDE (PVC) PIPE AND FITTINGS

8.1 Material Requirements

PVC pipe and fittings must conform to the requirements of the *Technical Specifications*. The CQA Consultant will document that the PVC pipe and fittings meet those requirements.

8.2 Manufacturer

8.2.1 Submittals

Prior to the installation of PVC pipe, 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.

8.3 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 general 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.

8.4 Perforations

The CQA Site Manager shall monitor and document that the perforations of the PVC pipe conform to the requirements of the *Drawings* and the *Technical Specifications*.

8.5 Joints

The CQA Monitor shall monitor and document that pipe and fittings are joined by the methods indicated in the *Technical Specifications*.

9. GEOMEMBRANE

9.1 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 *Drawings*, *Technical Specifications,* and any approved Addenda regarding this material.

9.2 Geomembrane Material Conformance

9.2.1 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.

9.2.2 Review of Quality Control

9.2.2.1 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;
- 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.

9.2.2.2 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 general 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.

9.2.3 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 2. 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 ft of material. Unless otherwise specified, samples will be 3 ft 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 2.

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.

9.3 Delivery

9.3.1 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.

9.3.2 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 on-site 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.

9.4 Geomembrane Installation

9.4.1 Introduction

The CQA Consultant will document that the geomembrane installation is carried out in general accordance with the *Drawings, Technical Specifications,* and Manufacturer's recommendations.

9.4.2 Earthwork

9.4.2.1 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.

9.4.2.2 Geosynthetic Termination

The CQA Site Manager will document that the geosynthetic terminations (Anchor Trench) have been constructed in general accordance with the *Drawings*. Backfilling above the terminations will be conducted in general accordance with the *Technical Specifications*.

9.4.3 Geomembrane Placement

9.4.3.1 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.

9.4.3.2 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 general 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), and/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 and/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 general accordance with procedures described in Section 9.4.5.

9.4.4 Field Seaming

This section details CQA procedures to document that seams are properly constructed and tested in general accordance with the Manufacturer's specifications and industry standards.

9.4.4.1 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.

9.4.4.2 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.

9.4.4.3 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."

9.4.4.4 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.

9.4.4.5 Overlapping and Temporary Bonding

The CQA Site Manager will document that:

- the panels of geomembrane have a finished overlap of a minimum of 3 in. 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.

9.4.4.6 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.

9.4.4.7 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.
- 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.
- Cleaning, overlap, and shingling requirements are maintained.

9.4.4.8 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 general accordance with Section 9.4.5.

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.

9.4.4.9 Destructive Testing

Concept

Destructive seam testing will be performed on site and at the independent CQA laboratory in general accordance with the *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. The minimum frequency is to be evaluated as an average taken throughout the entire facility.
- 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 general accordance with repair procedures described in Section 9.4.5. The continuity of the new seams in the repaired area will be tested in general accordance with Section 9.4.4.8.

Size and Distribution of Samples

The destructive sample will be 12 in. (0.3 m) wide by 42 in. (1.1 m) long with the seam centered lengthwise. The sample will be cut into three parts and distributed as follows:

- one portion, measuring 12 in. × 12 in. (0.30 cm × 30 cm), to the Geosynthetic Installer for field testing;
- one portion, measuring 12 in. × 18 in. (30 cm × 45 cm), for CQA Laboratory testing; and
- one portion, measuring 12 in. × 12 in. (30 cm × 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...). 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 ft (3 m) 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 general accordance with Section 9.4.5.

The CQA Site Manager will document actions taken in conjunction with destructive test failures.

9.4.5 Defects and Repairs

This section prescribes CQA activities to document that defects, tears, rips, punctures, damage, or failing seams shall be repaired.

9.4.5.1 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.

9.4.5.2 Evaluation

Potentially flawed locations, both in seam and non-seam areas, shall be non-destructively tested using the methods described in Section 9.4.4.8 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.

9.4.5.3 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;
- removing 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 in. (150 mm) beyond the edge of the defect, and all corners of patches will be rounded with a radius of at least 3 in. (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.

9.4.5.4 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.

9.4.5.5 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.

9.4.6 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 general accordance with the *Technical Specifications* for the project.

10. GEOTEXTILE

10.1 Introduction

This section of the CQA Plan outlines the CQA activities to be performed for the geotextile installation. The CQA Consultant will review the *Drawings*, and the *Technical Specifications*, and any approved addenda or changes.

10.2 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 3.

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.

10.3 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.

10.4 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.

10.5 Conformance Testing

10.5.1 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 3. These conformance tests will be performed in general accordance with the test methods specified in the *Technical Specifications* and will be documented by the CQA Site Manager.

10.5.2 Sampling Procedures

Samples will be taken across the width of the roll and will not include the first three feet. Unless otherwise specified, samples will be 3 ft 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 3 for geotextiles.

10.5.3 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.

10.5.4 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.

10.6 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.

10.7 Seams and Overlaps

All geotextiles will be continuously sewn in accordance with *Technical Specifications*. Geotextiles will be overlapped 12 in. 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.

10.8 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 in. (0.60 m) 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.

10.9 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.

11. GEOSYNTHETIC CLAY LINER (GCL)

11.1 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 *Drawings*, and the *Technical Specifications*, and 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 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; and
- index flux.

Quality control tests must be performed, in general accordance with the test methods specified in Table 4, 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.

11.3 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.

11.4 Shipment and Storage

During shipment and storage, the GCL will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, 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.

11.5 Conformance Testing

11.5.1 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 4.

Samples will be taken across the width of the roll and will not include the first 3 ft if the sample is cut on site. Unless otherwise specified, samples will be 3 ft long by the roll width. The CQA Consultant will mark the machine direction with an arrow and the manufacturer's roll number on each sample.

The CQA Site Manager will examine results from laboratory conformance testing and will report non-conformance to the Construction Manager.

11.5.2 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.

11.6 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.

11.7 GCL Installation

The CQA Consultant will monitor and document that the GCL is installed in general 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 general 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 GCL is not installed on a saturated subgrade or standing water and is not exposed such that it is hydrated prior to completion of the construction; 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.

12. GEONET

12.1 Introduction

This section of the CQA Plan outlines the CQA activities to be performed for the geonet installation. The CQA Consultant will review the *Drawings*, and the *Technical Specifications*, and any approved addenda or changes.

12.2 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.

12.3 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.

12.4 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.

12.5 Conformance Testing

12.5.1 Tests

The geonet material will be tested for transmissivity (ASTM D 4716) and for thickness (ASTM D 5199) at the frequencies presented in Table 5.

12.5.2 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 ft. Unless otherwise specified, samples will be 3 ft long by the roll width. The CQA Consultant will mark the machine direction on the samples with an arrow.

12.5.3 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.

12.5.4 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.

12.6 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, and/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.

12.7 Geonet Seams and Overlaps

Adjacent geonet panels will be joined in general accordance with Construction Drawings and Technical Specifications. As a minimum, the adjacent rolls will be overlapped by at least 4 in. and secured by tying, in general accordance with the Technical Specifications.

The CQA Consultant will note any noncompliance and report it to the Construction Manager.

12.8 Repair

Holes or tears in the geonet will be repaired by placing a patch extending 2 ft beyond edges of the hole or tear. The patch will be secured by tying with approved tying devices every 6 in. 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 general accordance with Section 11.7.

The CQA Site Manager will observe repairs, note non-compliances with the above requirements and report them to the Construction Manager.

13. SURVEYING

13.1 Survey Control

Survey control will be performed by the Construction Manager 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.

13.2 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.

13.3 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.

13.4 Frequency and Spacing

A line of survey points no further than 50 ft apart must be taken at the top of pipes or other appurtenances to the liner.

13.5 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 AutoCADD 2000 format or other suitable format as directed by the Construction Manager.

TABLE 1A

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 1B

MINIMUM AGGREGATE TESTING FREQUENCIES FOR CONFORMANCE TESTING

TEST	TEST METHOD	DRAINAGE AGGREGATE
Sieve Analysis	ASTM C 136	1 per 5,000 yd^3
Hydraulic Conductivity	ASTM D 2434	1 per 10,000 yd^3

TABLE 2

GEOMEMBRANE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	FREQUENCY
Specific Gravity	ASTM D 792 Method A or ASTM D 1505	100,000 ft^2
Thickness	ASTM D 5199	100,000 ft^2
Tensile Strength at Yield	ASTM D 638	100,000 ft^2
Tensile Strength at Break	ASTM D 638	100,000 ft^2
Elongation at Yield	ASTM D 638	100,000 ft^2
Elongation at Break	ASTM D 638	100,000 ft^2
Carbon Black Content	ASTM D 1603	100,000 ft^2
Carbon Black Dispersion	ASTM D 5596	100,000 ft^2

TABLE 3

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 4

GCL CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
Mass per Unit Area	ASTM D 5993	1 test per 100,000 ft^2
Index Flux	ASTM D 5887	1 test per 400,000 ft^2

Note: Hydraulic index flux testing shall be performed under an effective confining stress of 5 pounds per square inch.

TABLE 5

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 Specifications

TECHNICAL SPECIFICATIONS

For the Construction of

Cell 4A Lining System

IUC White Mesa Mill

Blanding, Utah

Prepared for:

International Uranium (USA) Corporation

6425 S. Highway 191
P.O. Box 809
Blanding, UT 84511

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JANUARY 2006

CERTIFICATION PAGE

TECHNICAL SPECIFICATIONS
CELL 4A LINING SYSTEM CONSTRUCTION
INTERNATIONAL URANIUM (USA) CORPORATION
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.

Gregory T. Corcoran, P.E.
Engineer of Record

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Section 03400	—	**Cast-In-Place Concrete**

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 Earthwork. 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 vegetation removal, re-conditioning, 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 02770 – Geomembrane

Section 02771 – Geotextile

Section 02772 – Geosynthetic Clay Liner

Section 02773 – Geonet

1.03 REFERENCES

A. Drawings

B. Site CQA Plan

1.04 QUALIFICATIONS

A. The Contractor's Site Superintendent for the earthworks operations shall have supervised the construction of at least two civil construction and earthworks projects in the last 5 years, including grading and trenching.

1.05 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 at the time of geosynthetics placement.

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. Rocks greater than 6-in. in any dimension shall not be permitted in anchor trench backfill material.

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.

3.03 TRENCH EXCAVATION

A. The Contractor shall excavate the anchor trench to the limits and grades shown on the Drawings.

B. All excavated materials not used 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 Engineer.

C. Material not suitable for anchor trench backfill shall be relocated as directed by the Engineer.

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 Part 2.02 of this Section.

C. Soil used for anchor trench backfill shall be placed in a loose lift that results in a compacted lift thickness of no greater than 6 inches. 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 Assurance and Quality Control.
- Excavation, loading, and hauling.
- Temporary haul roads.
- Dust control.
- 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 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 (in.)	Percent Finer
¾-in.	100
No. 200 Sieve	0 to 2

B. Carbonate loss shall be no greater than 10% 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, meeting the ground pressure requirements listed in Subpart 3.02.F.

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 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 1 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 1 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. 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.

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

A. Providing for and complying with the requirements set forth in this Section for PVC Pipe will be measured as in-place lineal foot (LF) 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.
- Fittings.
- Drainage aggregate.
- Joining.
- Mobilization.
- Placement.
- Rejected material.
- Rejected material removal, handling, re-testing, and repair.

[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 ft^2 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 ft^2 of polyethylene geomembrane on previous projects with similar sideslopes, 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's 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, 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, 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 ft^2 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 1 per 50,000 ft^2 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 or 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., 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.

   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' 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 in. above the geomembrane surface.

D. Overlapping:

1. 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 downslope 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 in. 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 Subpart of the approved submittal specified in Subpart 1.06 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-ft long by 1-ft wide (after seaming) with the seam centered lengthwise for fusion equipment and at least 3-ft long by 1-ft wide for extrusion equipment. Seam overlap shall be as indicated in Subpart 3.03.D of this Section.

2. Four coupon specimens, each 1-in. wide, shall be cut from the trial seam sample by the Geosynthetics Installer using a die cutter to ensure precise 1-in. 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 in./min. 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 ft of total seam length. 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-in. wide and 12 in. 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-ft wide by 3.5-ft 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-in. 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-in. wide strips tested at a rate of 2 in./min. 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 ft, 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 in. beyond the edge of the defect, and all corners of patches and holes shall be rounded with a radius of at least 3 in.; 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. No vehicles of any kind shall be driven directly on the geomembrane.

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 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 ft^2. 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 ft^2 (90,000 ft^2), 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 1.07.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 a complying with the requirements set forth in this Section for 60-mil, smooth HDPE geomembrane will be measured as in-place square feet (SF), 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	.94	ASTM D 792 Method A or ASTM D 1505
Mechanical Properties				
Tensile Properties (each direction)				
1. Tensile (Break) Strength 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 PACKING AND LABELING

A. Geotextile shall be supplied in rolls wrapped in relatively impervious and opaque protective covers.

B. Geotextile rolls shall be marked or tagged with the following information:

1. manufacturer's name;

2. product identification;

3. lot or batch number;

4. roll number; and

5. roll dimensions.

2.04 TRANSPORTATION, HANDLING, AND STORAGE

A. The Geosynthetic Manufacturer shall be liable for any damage to the geotextile incurred prior to and during transportation to the site.

B. The geotextile shall be delivered to the site at least 14 days prior to the planned deployment date to allow the CQA 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% 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 ft^2 (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 02271):

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
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}	0.7	ASTM D 4491
Grab strength	Minimum	lb	390	ASTM D 4632
Tear strength	Minimum	lb	150	ASTM D 4533
Puncture strength	Minimum	lb	250	ASTM D 4833
Ultraviolet Resistance @ 500 hours	Minimum	%	70	ASTM D 4355

[END OF SECTION]

SECTION 02772
GEOSYNTHETIC CLAY LINER

PART 1 – GENERAL

1.01 SCOPE

A. The Geosynthetic Installer shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary for installation of the geosynthetic clay liner (GCL). The work shall be carried out as specified herein and in accordance with the Drawings and Construction Quality Assurance (CQA) Plan.

B. The work shall include, but not be limited to, delivery, offloading, storage, placement, anchorage, and seaming of the GCL.

1.02 RELATED SECTIONS

Section 02220 – Subgrade Preparation

Section 02770 – Geomembrane

1.03 REFERENCES

A. Drawings

B. Site CQA Plan

C. Latest Version American Society of Testing and Materials (ASTM) Standards:

ASTM D 5887	Test Method for Measurement of Index Flux Through Saturated Geosynthetic Clay Liner Specimens using a Flexible Wall Permeameter
ASTM D 5888	Guide for Storage and Handling of Geosynthetic Clay Liners
ASTM D 5890	Test Method for Swell Index of Clay Mineral Component of Geosynthetic Clay Liners
ASTM D 5891	Test Method for Fluid Loss of Clay Component of Geosynthetic Clay Liners
ASTM D 5993	Test Method for Measuring Mass per Unit Area of Geosynthetic Clay Liners

1.04 QUALIFICATIONS

A. GCL Manufacturer:

1. The Manufacturer shall be a well-established firm with more than five (5) years of experience in the manufacturing of GCL.

2. The GCL Manufacturer shall be responsible for the production of GCL rolls and shall have sufficient production capacity and qualified personnel to provide material meeting the requirements of this Section and the construction schedule for this project.

B. GCL Installer:

1. The Geosynthetic Installer shall install the GCL and shall meet the requirements of Section 02770 Subpart 1.04. B and this Section.

2. The Geosynthetics Installer shall be responsible and shall provide sufficient resources for field handling, deploying, temporarily restraining (against wind), and other aspects of the deployment and installation of the GCL and other geosynthetic components of the project.

1.05 SUBMITTALS

A. At least 7 days before transporting any GCL to the site, the Manufacturer shall provide the following documentation to the Engineer for approval.

1. list of material properties, including test methods utilized to analyze / confirm properties.

2. GCL samples.

3. projected delivery dates for this project.

4. Manufacturing quality control certificates for each shift's production for which GCL for the project was produced, signed by responsible parties employed by the Manufacturer (such as the production manager).

5. Manufacturer Quality Control (MQC) certificates, including:

a. roll numbers and identification; and

b. MQC results, including description of test methods used, outlined in Subpart 2.01 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.

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 have a combined mass per unit area of 9 oz./SY.

H. The GCL shall be needle punched.

2.02 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 1 per 50,000 ft2 and 1 per 200,000 ft2, respectively.

G. The Manufacturer shall comply with the certification and submittal requirements of this Section.

2.03 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.

2.04 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 3.08 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.

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 and so that it does not become hydrated (greater than 30% moisture content in accordance with ASTMD 5993) prior to or during installation.

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 3.07 by the Geosynthetic Installer, at the expense of the Geosynthetic Installer.

D. The GCL shall not be installed on an excessively moist subgrade or on standing water. The GCL shall be installed in a way that prevents hydration of the GCL prior to completion of construction of the liner system.

E. The GCL shall not be installed during precipitation or other conditions that may cause hydration of the GCL.

F. All hydrated GCL shall be removed and replaced by the Geosynthetic Installer at the expense of the Geosynthetic Installer. Hydrated GCL shall be defined as a moisture content greater than 30%.

3.04 OVERLAPS

A. On slopes steeper than 10 horizontal to 1 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. Geomembrane installation shall immediately follow the GCL installation. All GCL that is placed during a day's work shall be covered with geomembrane before the Geosynthetic Installer leaves the site at the end of the day. The edges of GCL placement should be covered each day and protected from hydration due to storm water run-on.

B. No material shall be placed on GCL that is hydrated.

C. Installation of other components of the liner system shall be carefully performed to avoid damage to the GCL.

D. No equipment shall be driven directly on the GCL.

E. 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 ft^2: 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 and 400,000 ft^2: 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 ft^2 (90,000 ft^2), 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

A. Providing for and complying with the requirements set forth in this Section for GCL will be measured as in-place square feet (SF) to the limits shown on the Drawings, and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

- Submittals.
- Quality Control.
- Shipping, handling and storage.
- Overlaps and seaming.
- Layout survey.
- Mobilization.
- Rejected material.
- Rejected material removal, handling, re-testing, and repair.
- Overlaps and seaming.
- Temporary anchorage.
- Visqueen.

TABLE 02772-1
REQUIRED GCL PROPERTY VALUES

PROPERTIES	QUALIFIERS	UNITS	SPECIFIED[1] VALUES	TEST METHOD
Bentonite Content[3]	minimum	lb/ft^2	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 Flux	minimum	m^3/m^2-s	1×10^{-8}	ASTM D 5887[2]
Moisture Content (Bentonite)	maximum	percent	30	ASTM D 5993

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) Hydraulic flux testing shall be performed under an effective confining stress of 5 pounds per square inch.
(3) Measured at a moisture content of 0 percent.

[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 02225 – Drainage Aggregate

Section 02220 – Subgrade Preparation

Section 02616 – PVC Pipe

Section 02270 – 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 02073-1, 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 ft^2 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 02073-1 need not be performed at a frequency of one per 100,000 ft^2. 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 ft^2 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 ft^2 (180,000 ft^2), 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 horizontal to 1 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. The geonet shall be handled in such a manner as to ensure it is not damaged in any way.

C. Precautions shall be taken to prevent damage to underlying layers during placement of the geonet.

D. The geonet shall be installed in a manner that minimizes wrinkles.

E. 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 in. 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 ft. along the geonet roll edges and 2 ft. 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 ft. 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 in. 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

A. Providing for and complying with the requirements set forth in this Section for geonet will be measured as in-place square feet (SF) 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	200	ASTM D5199
Transmissivity[2]	Minimum	m^2 / sec	2×10^{-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 steel plates. 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 in.

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 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 degrees F while curing. Cover concrete and provide sufficient heat to maintain 50 degrees F minimum adjacent to both the formwork and the structure while curing. Limit the rate of cooling to 5 degrees F in any 1 hour and 50 degrees 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 five feet.

E. When placing of concrete is temporarily halted or delayed, provide construction joints.

F. Concrete shall not be dropped a distance greater than five feet.

G. Place concrete with aid of internal mechanical vibrator equipment capable of 9,000 cycles/min. 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 2.03).

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 2.03).

C. Maintain concrete with minimal moisture loss at relatively constant temperature for period necessary for hydration of cement and hardening of concrete (Subpart 2.03).

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 ft 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 ft x 15 ft grid spacing along the slab unless otherwise approved by the Engineer. Control joints shall be no greater than 1 ½ inch 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

Existing Berm and Clay Liner Construction Documentation

Dike Construction

Umetco Minerals Corporation

WHITE MESA MILL • P.O. BOX 669 • BLANDING, UTAH 84511
☎ (801) 678-2221

May 17, 1989
REPORT NO. 1

Mr. Ed Hawkins, Director
U. S. Nuclear Regulatory Commission
Region IV
Uranium Recovery Field Office
Box 25325
Denver, CO 80225

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period May 9, 1989 to May 12, 1989, 4 days of construction activity occurred.

These activities consisted of: Stripping.

Testing activities included:

0 In place density tests (Nuclear)

0 In place density tests (sand cone)

5 ASTM D-698 Moisture Density Relations

5 ASTM D 1140-54 Soils Finer than No. 200 Sieve

5 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed None

failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

WHITE MESA MILL • P.O. BOX 669 • BLANDING, UTAH 84511
☎ (801) 678-2221

May 17, 1989

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

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Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

WHITE MESA MILL • P.O. BOX 669 • BLANDING, UTAH 84511
☎ (801) 678-2221

May 23, 1989
REPORT NO. 2

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period May 15, 1989 to May 19, 1989, 5 days of construction activity occurred.

These activities consisted of: Stripping, proof rolling, placing fill in south dike, removal and stockpiling of calcareous material from cell.

Testing activities included:

11 In place density tests (Nuclear)

3 In place density tests (sand cone)

1 ASTM D-698 Moisture Density Relations

0 ASTM D 1140-54 Soils Finer than No. 200 Sieve

0 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed Two in-place density tests indicated moisture contents that failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

WHITE MESA MILL • P.O. BOX 669 • BLANDING, UTAH 84511
☎ (801) 678-2221

May 30, 1989
REPORT NO.3

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period May 22, 1989 to May 26, 1989, 5 days of construction activity occurred.

These activities consisted of: Placing fill in south dike, removal and stockpiling of calcareous material from cell.

Testing activities included:

25 In place density tests (Nuclear)

6 In place density tests (sand cone)

2 ASTM D-698 Moisture Density Relations

4 ASTM D 1140-54 Soils Finer than No. 200 Sieve

4 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed Five in-place density tests indicated moisture contents that failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

WHITE MESA MILL • P.O. BOX 669 • BLANDING, UTAH 84511
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June 7, 1989
REPORT NO.4

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period May 30, 1989 to June 2, 1989, 4 days of construction activity occurred.

These activities consisted of: Placing fill in south dike, removal and stockpiling of calcareous material and clay material from the cell.

Testing activities included:

26 In place density tests (Nuclear)

8 In place density tests (sand cone)

4 ASTM D-698 Moisture Density Relations

4 ASTM D 1140-54 Soils Finer than No. 200 Sieve

4 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed Two in-place density tests indicated moisture content that failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

WHITE MESA MILL • P.O. BOX 669 • BLANDING, UTAH 84511
☎ (801) 678-2221

June 20, 1989
REPORT NO.5

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period June 5, 1989 to June 9, 1989, 5 days of construction activity occurred.

These activities consisted of: Placing fill in south and west dikes and removal and stockpiling of clay material from the cell.

Testing activities included:

49 In Place Density Tests (Nuclear)

6 In Place Density Tests (Sand Cone)

4 ASTM D-698 Moisture Density Relations

7 ASTM D 1140-54 Soils Finer Than No. 200 Sieve

7 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed Six in-place density tests indicated moisture content that failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

Frequency of Testing Activities
Stated as Tests Per Cubic Yard

This Period	Cumulative	
6/5/89 to 6/9/89	5/17/89 to 6/9/89	
1:747	1:795	In Place Densities And Moisture Content
1:5866	1:4959	Atterberg Limits Tests
1:5866	1:4959	Soil Gradations
1:10265	1:6943	Moisture Density Relations

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

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☎ (801) 678-2221

June 23, 1989
REPORT NO. 6

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period June 12, 1989 to June 16, 1989, 5 days of construction activity occurred.

These activities consisted of: Placing fill in south and west dikes and removal and stockpiling of clay material from the cell.

Testing activities included:

55 In Place Density Tests (Nuclear)

11 In Place Density Tests (Sand Cone)

5 ASTM D-698 Moisture Density Relations

12 ASTM D 1140-54 Soils Finer Than No. 200 Sieve

12 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed Two in-place density tests indicated moisture content that failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

Frequency of Testing Activities
Stated as Tests Per Cubic Yard

This Period	Cumulative	
6/12/89 to 6/16/89	5/17/89 to 6/16/89	
1:790	1:801	In Place Densities And Moisture Content
1:4345	1:4596	Atterberg Limits Tests
1:4345	1:4596	Soil Gradations
1:10428	1:8225	Moisture Density Relations

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

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☎ (801) 678-2221

June 28, 1989
REPORT NO. 7

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period June 19, 1989 to June 23, 1989, 5 days of construction activity occurred.

These activities consisted of: Placing fill in south and west dikes and removal and stockpiling of clay and random material from the cell.

Testing activities included:

63 In Place Density Tests (Nuclear)

16 In Place Density Tests (Sand Cone)

7 ASTM D-698 Moisture Density Relations

12 ASTM D 1140-54 Soils Finer Than No. 200 Sieve

12 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed Ten in-place density tests indicated moisture content that failed to meet specification requirements, and 2 in-place density tests indicated densities that failed to meet specification requirements

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

Frequency of Testing Activities
Stated as Tests Per Cubic Yard

This Period	Cumulative	
6/19/89 to 6/23/89	5/17/89 to 6/23/89	
1:569	1:735	In Place Densities And Moisture Content
1:3750	1:4193	Atterberg Limits Tests
1:3750	1:4193	Soil Gradations
1:6429	1:7189	Moisture Density Relations

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

WHITE MESA MILL • P.O. BOX 669 • BLANDING, UTAH 84511
☎ (801) 678-2221

July 7, 1989
REPORT NO. 8

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period June 26, 1989 to June 30, 1989, 5 days of construction activity occurred.

These activities consisted of: Placing fill in west dike and removal and stockpiling of clay material from the cell.

Testing activities included:

39 In Place Density Tests (Nuclear)

9 In Place Density Tests (Sand Cone)

5 ASTM D-698 Moisture Density Relations

11 ASTM D 1140-54 Soils Finer Than No. 200 Sieve

11 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed 5 in-place density tests indicated moisture content that failed to meet specification requirements, and 1 in-place density tests that indicated densities that failed to meet specification requirements

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

Frequency of Testing Activities
Stated as Tests Per Cubic Yard

This Period	Cumulative	
6/26/89 to 6/30/89	5/17/89 to 6/30/89	
1:796	1:744	In Place Densities And Moisture Content
1:3475	1:4059	Atterberg Limits Tests
1:3475	1:4059	Soil Gradations
1:7644	1:7258	Moisture Density Relations

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

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☎ (801) 678-2221

July 14, 1989
REPORT NO. 9

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period July 5, 1989 to July 8, 1989, 4 days of construction activity occurred.

These activities consisted of: Placing fill in south, east and west dikes and removal and stockpiling of clay material from the cell, and material classified as random.

Testing activities included:

26 In Place Density Tests (Nuclear)

9 In Place Density Tests (Sand Cone)

4 ASTM D-698 Moisture Density Relations

8 ASTM D 1140-54 Soils Finer Than No. 200 Sieve

8 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed 7 in-place density tests indicated moisture content that failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

Frequency of Testing Activities
Stated as Tests Per Cubic Yard

This Period	Cumulative	
7/5/89 to 7/8/89	5/17/89 to 7/8/89	
1:653	1:735	In Place Densities And Moisture Content
1:2858	1:3916	Atterberg Limits Tests
1:2858	1:3916	Soil Gradations
1:5715	1:7091	Moisture Density Relations

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H Sampson / G L Swanson

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

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July 20, 1989
REPORT NO. 10

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period July 10, 1989 to July 14, 1989, 5 days of construction activity occurred.

These activities consisted of: Placing fill in south, east and west dikes and removal and stockpiling of clay material from the cell, and material classified as random. Also placing 3' of random cover on Tailings Cell No. 2.

Testing activities included:

33 In Place Density Tests (Nuclear)

14 In Place Density Tests (Sand Cone)

5 ASTM D-698 Moisture Density Relations

10 ASTM D 1140-54 Soils Finer Than No. 200 Sieve

10 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed 2 in-place density tests indicated moisture content that failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

Frequency of Testing Activities
Stated as Tests Per Cubic Yard

This Period	Cumulative	
7/10/89 to 7/14/89	5/17/89 to 7/14/89	
1:602	1:719	In Place Densities And Moisture Content
1:2828	1:3775	Atterberg Limits Tests
1:2828	1:3775	Soil Gradations
1:5656	1:6920	Moisture Density Relations

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson / L. S. Swanson

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

Umetco Minerals Corporation

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☎ (801) 678-2221

July 31, 1989
REPORT NO. 11

Mr. Robert L. Morgan, P.E.
State Engineer
State of Utah
Department of Natural Resources
Division of Water Rights
Dam Safety
1636 West North Temple, Suite 220
Salt Lake City, Utah 84116-3156

Re: Cell 4A Tailings Pond Construction

Gentlemen:

During the period July 17, 1989 to July 24, 1989, 6 days of construction activity occurred.

These activities consisted of: Topping out fill on south, east & west dikes and removal and stockpiling of clay material from the cell, and material classified as random. Also placing 3' of random cover on Tailings Cell No. 2.

Testing activities included:

20 In Place Density Tests (Nuclear)

2 In Place Density Tests (Sand Cone)

1 ASTM D-698 Moisture Density Relations

1 ASTM D 1140-54 Soils Finer Than No. 200 Sieve

1 ASTM D 4318-84 Atterberg Limits Tests.

Of the tests performed 1 in-place density tests indicated moisture content that failed to meet specification requirements.

All of the material that failed to meet specified requirements was reworked to meet specifications or was removed from the structure.

Only 1,460 cu. yds. of fill material was placed during this 6 day period to fill in low spots and bring all dikes to the specified elevations.

Frequency of Testing Activities
Stated as Tests Per Cubic Yard

This Period	Cumulative	
7/17/89 to 7/24/89	5/17/89 to 7/24/89	
1:66	1:682	In Place Densities And Moisture Content
1:1460	1:3751	Atterberg Limits Tests
1:1460	1:3751	Soil Gradations
1:1460	1:6804	Moisture Density Relations

All individual materials tests and inspection reports are filed in the project files on site.

Sincerely,

Henry H. Sampson / A D Swanson

Henry H. Sampson, Jr. P. E.
Q A Consultant
Umetco Minerals Corporation
White Mesa Mill

xc: J. S. Hamrick

F-3 FILL OBSERVATION & TESTING

Project No. C-4A-WM

Technician T. Krake / R. Dix

Date 5/18/89

Approved By [signature]

Daily Report No. 8

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
1	Sta 20+00 80' Lt of CL	5563	110.7	15.2	106.3	12.3	96	Sandy Silt
2	Sta 19+15 60' Lt of CL	5565	114.0	13.5	115.4	11.8	100	Sandy Silt SAND CONE
3	Sta 20+20 On CL (Sandcone)	5566	114.0	12.1 ~~13.5~~ *	111.4	12.1	98	Sandy Silt
4	Sta 19+75 70' Rt of E	5571	114.0	12.1 ~~13.5~~ *	109.2	11.0	96	Sandy Silt
5	Sta 18+80 70' Lt of E	5566	114.0	12.1 ~~13.5~~ *	112.0	11.0 *	98	Sandy Silt
6	Sta 20+80 20' Rt of E	5565	114.0	12.1 ~~13.5~~ *	109.7	15.0 *	96	Sandy Silt
5R	Retest of test # 5 (Sand cone)	5566	114.0	12.1 ~~13.5~~	110.1	13.1	97	Sandy Silt

SPECIFICATION COMPACTION & MATERIAL

Min 95%, ± 2% moisture *

TYPE AND NUMBER OF EARTH MOVING UNITS 2 - [illegible]

TYPE AND NUMBER OF COMPACTION UNITS 1 - Sheepsfoot

NUMBER OF PASSES THICKNESS OF LIFT 8→12"

METHOD OF ADDING MOISTURE Spray

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVATIC & TESTING

Project No. C-4A-WM

Technician T. Kruke / T. Day

Date 5-18-89

Approved By S L Swanson

Daily Report No. 8

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY MAXIMUM DRY DENSITY (pcf)	LABORATORY OPTIMUM MOISTURE CONTENT (%)	FIELD DRY DENSITY (pcf)	FIELD MOISTURE CONTENT (%)	PERCENT COMPACTION	SOIL TYPE
7	Sta 18+60 on ℄	5567	114.0	12.1 * ~~13.5~~	113.4	12.9	99	Sandy Silt
8	Sta 20+50 40' Rt of ℄	5565	114.0	12.1 * ~~13.5~~	111.0	13.0	97	Sandy Silt *

SPECIFICATION COMPACTION & MATERIAL

Min 95%, ±2% Moisture

TYPE AND NUMBER OF EARTH MOVING UNITS 2 - Scrapers

TYPE AND NUMBER OF COMPACTION UNITS 1-Sheep Foot 1-Grader

NUMBER OF PASSES THICKNESS OF LIFT 8-12"

METHOD OF ADDING MOISTURE Spray

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

~~* Serves as retest of Moisture on Test #6~~

F-3 FILL OBSERVATI & TESTING

Project No. C-4A-NM

Technician T. Krake / R. Day

Date 6/1/89

Approved By H. Stevenson

Daily Report No. 17A + B

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
54	Sta 22+40 50'Rt of ℄ (Sandcone)	5574	112.0	14.7	114.2	12.9	100	Sandy Silt
55	Sta 19+00 30'Lt of ℄	5575	112.0	14.7	108.3	12.7	97	Sandy Silt
56	Sta 16+00 40' Lt of ℄	5577	112.0	14.7	111.0	16.7	99	Sandy Silt
57	Sta 21+70 65' Lt of ℄ (Sandcone)	5573	112.0	14.7	110.0	13.9	100	Sandy Silt
58	Sta 19+00 ON ℄	5575	112.0	14.7	108.4	15.4	97	Sandy Silt
59	Sta 24+40 40' Rt of ℄	5573	112.0	14.7	110.8	14.2	99	Sandy Silt
60	Sta 27+00 ON ℄	5572	112.0	14.7	109.2	13.1	98	Sandy Silt

SPECIFICATION COMPACTION & MATERIAL

Min Den - 95%, ±2% Moisture

TYPE AND NUMBER OF EARTH MOVING UNITS 2 - Scrapers

TYPE AND NUMBER OF COMPACTION UNITS 2 - Compactors

NUMBER OF PASSES THICKNESS OF LIFT 8-12"

METHOD OF ADDING MOISTURE Spray

- ☑ FILL TESTED MEETS SPECIFICATIONS.
- ☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.
- ☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVATI & TESTING

Project No. C-9A-Wm

Technician R. Day

Date 6-1-89

Approved By A. L. Sherman

Daily Report No. 178

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
61	Sta 25+60 30' Lt. of ℄	5573	112.0	14.7	111.0	12.8	99	Sandy Silt
62	Sta 18+40 50' Rt. of ℄	5575	112.0	14.7	107.9	13.8	96	Sandy Silt

SPECIFICATION COMPACTION & MATERIAL
min. Den 95%, ± 2% moisture

TYPE AND NUMBER OF EARTH MOVING UNITS 2- Scrapers

TYPE AND NUMBER OF COMPACTION UNITS 1- Compactor

NUMBER OF PASSES THICKNESS OF LIFT 8-12"

METHOD OF ADDING MOISTURE Spray

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S)
AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVATI[illegible] & TESTING

Project No. C-4A-WM

Technician T. Krake

Date 6/15/89

Approved By [illegible]

Daily Report No. 27A, 27B

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY MAXIMUM DRY DENSITY (pcf)	LABORATORY OPTIMUM MOISTURE CONTENT (%)	FIELD DRY DENSITY (pcf)	FIELD MOISTURE CONTENT (%)	PERCENT COMPACTION	SOIL TYPE
169	Sta 16+50 30' Rt	5580	113.3	13.6	109.0	12.2	96	Silty Sand
170	Sta 28+20 20' Rt	5580	113.3	13.6	116.2	13.2	100	Silty Sand
171	Sta 27+50 40' Lt	[illegible]	[illegible]	[illegible]	107.4	13.0	100	Clayey Sand
172	Sta 26+50 50' Rt	[illegible]	113.3	13.6	109.2	11.8	96	Silty Sand
172	STA 2[illegible]	5581	113.3	13.6	114.7	11.7	100	Silty Sand
173	STA 29+60 30' LT of CL	5579	113.3	13.6	119	12.7	100	Silty Sand
174	STA 31+50 20' LT of CL	5577	113.3	13.6	118.2	12.2	100	Silty Sand

SPECIFICATION COMPACTION & MATERIAL
Min 15% Pen, ± 2% moisture

TYPE AND NUMBER OF EARTH MOVING UNITS [illegible], 2 sheepsfoot, 1-blade, 2 waterwagons

TYPE AND NUMBER OF COMPACTION UNITS 2 sheepsfoot

NUMBER OF PASSES THICKNESS OF LIFT

METHOD OF ADDING MOISTURE [illegible]

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVA[illegible] & TESTING

Project No. C-4A-WM

Technician J Sanchez

Date [illegible]-15-89

Approved By [illegible]

Daily Report No. 27B

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY MAXIMUM DRY DENSITY (pcf)	LABORATORY OPTIMUM MOISTURE CONTENT (%)	FIELD DRY DENSITY (pcf)	FIELD MOISTURE CONTENT (%)	PERCENT COMPACTION	SOIL TYPE
175	STA 27+10 30' LT of CL (SC)	5582	113.3	13.6	118.7	[illegible]	100	Silty Sand
176	STA 27+25 30' LT of CL	5582	113.3	13.6	108.[illegible]	15.2	95	Silty Sand
177	STA 29+00 30' LT of CL	5581	113.3	13.6	115.5	11.6	100	Silty Sand
178	STA 32+00 ON ~~[illegible]~~ of CL	5580	113.3	13.6	118.5	13.4	100	Silty Sand
179	STA 34+50 ON CL	5580	113.3	13.6	115.9	12.5	100	Silty Sand
180	STA 30+20 10' RT of CL	5581	113.3	13.6	112.8	12.0	99	Silty Sand
181	Sta 27+00 on CL (Sand Core)	5582	113.3	13.6	105.9	11.1*	94	Silty Sand

SPECIFICATION COMPACTION & MATERIAL
Min 95% Den, ±2% moisture

TYPE AND NUMBER OF EARTH MOVING UNITS 2 scrapers, 1 blade

TYPE AND NUMBER OF COMPACTION UNITS 2 sheep foot
2 waterwagon

NUMBER OF PASSES THICKNESS OF LIFT

METHOD OF ADDING MOISTURE Spray

☐ FILL TESTED MEETS SPECIFICATIONS.
☑ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) 181
AND SHOULD BE REMOVED OR REWORKED.
☑ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVA N & TESTING

Project No. C-4A-WM

Technician T. Krake

Date 7/5/89

Approved By Daryl Swanson

Daily Report No. 3917

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
314	Sta 26+00 20'R+ F9	5589	116.0	13.2	110.8	15.1	96	Silty Sand w/sandstone
315	Sta 22+00 20 R+ F8	5590	116.0	13.2	111.1	13.9	96	"
316	Sta 24+00 on CL F7	5591	116.0	13.2	110.1	11.9	95	"
317	Sta 21+90 on CL F8	5590	116.0	13.2	112.0	16.4 *	97	"
318	Sta 19+90 on CL F9	5589	116.0	13.2	109.9	11.9	95	"
317R	Retest of F317	5590	116.0	13.2	114.1	14.6	98	"
319	Sta 16+10 20'Rt F10	5588	116.0	13.2	120.0	11.3	100+	"

SPECIFICATION COMPACTION & MATERIAL
Min Den - 95%, ± 2% Moisture

TYPE AND NUMBER OF EARTH MOVING UNITS 5 - Scrapers

TYPE AND NUMBER OF COMPACTION UNITS 2 - Compactors

NUMBER OF PASSES THICKNESS OF LIFT

METHOD OF ADDING MOISTURE Spray

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S)
AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERV[illegible]ON & TESTING

Project No. C-4A-WM

Technician H. Kuebler

Date 7/5/89

Approved By [signature]

Daily Report No. 39A, 39B

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY MAXIMUM DRY DENSITY (pcf)	LABORATORY OPTIMUM MOISTURE CONTENT (%)	FIELD DRY DENSITY (pcf)	FIELD MOISTURE CONTENT (%)	PERCENT COMPACTION	SOIL TYPE
320	Sta 18+00 20' Rt F6	5592	116.0	13.2	115.4	12.0 ✓	99 ✓	silty sand w/sandstone
321	Sta 25+50 20' L/CL Sand Cone F6	5592	116.0	13.2	124.3	12.9 ✓	100+ ✓	"
322	Sta 16+00 10 L/CL Sand Cone F10	5588	116.0	13.2	116.2	12.5 ✓	100 ✓	"
323	Sta 24+75 CL Sand Cone F6	5592			109.2	7.6 *		Silt
324R	Retest of 323 Sand Cone F6	5592			120.3	8.4 *		Silt
325	Sta 15+75 20' L/CL F10 Sand Cone	5588	116.0	13.2	113.3	14.9 ✓	98 ✓	silty sand w/sandstone

SPECIFICATION COMPACTION & MATERIAL

Min Den 95%, ±2% M/C

TYPE AND NUMBER OF EARTH MOVING UNITS 3 Scrapers

TYPE AND NUMBER OF COMPACTION UNITS 1 Sheepfoot

NUMBER OF PASSES

THICKNESS OF LIFT

METHOD OF ADDING MOISTURE Spray

☐ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

* = approved by Henry Samson (SS) [initials]

F-3 FILL OBSERVATION & TESTING

Project No. C-49-WM

Technician T. Krake | H Kuebler

Date 7/14/89

Approved By L. L. Swanson

Daily Report No. 47A

TEST NO.	LOCATION		DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
				MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
388	Sta 5+00 on CL	SC	5595	114.6	13.5	116.6	11.6	100	clayey sand
389	Sta 6+00 5' Rt	SC	5596	114.6	13.5	117.2	12.2	100	"
390	Sta 7+00 5' Lt	SC	5596	114.6	13.5	118.9	12.1	100	"
391	Sta 8+10 on CL		5595	114.6	13.5	113.8	14.5	99	"
392	Sta 9+00 5' Rt		5595	114.6	13.5	115.4	11.6	100	"
393	Sta 6+50 5' Lt	F-1	5597	114.6	13.5	112.8	15.4	98	"
394	Sta 8+50 on CL	F-1	5597	114.6	13.5	115.8	13.2	100	"

SPECIFICATION COMPACTION & MATERIAL

Min Den - 95%, ±2% Moisture

TYPE AND NUMBER OF EARTH MOVING UNITS 2 - Scrapers

TYPE AND NUMBER OF COMPACTION UNITS 1 - Compactor

NUMBER OF PASSES THICKNESS OF LIFT

METHOD OF ADDING MOISTURE Spray

- ☑ FILL TESTED MEETS SPECIFICATIONS.
- ☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.
- ☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVATION & TESTING

Project No. C-4A-WM

Technician H Kuebler

Date 7-14-89

Approved By K.L. Swanson

Daily Report No. 47-B

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
395	Sta 12+75 on CL F-1	5597	114.6	13.5	122.6	12.6	100	Clayey/Sand

SPECIFICATION COMPACTION & MATERIAL

TYPE AND NUMBER OF EARTH MOVING UNITS

TYPE AND NUMBER OF COMPACTION UNITS

NUMBER OF PASSES THICKNESS OF LIFT

METHOD OF ADDING MOISTURE

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

Clay Liner Construction

F-3 FILL OBSERVATION & TESTING

Project No. C-4A-WM

Technician T Krake

Date 7/1/89

Approved By [signature]

Daily Report No. 56A

TEST NO.	LOCATION	DEPTH OR ELEVATION* (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
416	Sta 26+00, Sta 30+00 B1	100	111.4	15.7	106.0	11.3*	95	Lean Clay
417	Sta 26+25, Sta 32+00 D1	100	111.4	15.7	109.7	11.0*	98	Lean Clay
418	Sta 26+00, Sta 33+00 E1	100	111.4	15.7	110.5	7.0*	99	Lean Clay
419	Sta 26+00, Sta 35+00 G1	100	111.4	15.7	112.1	7.4*	100	Lean Clay
420								

SPECIFICATION COMPACTION & MATERIAL

Den - 95% min, Moisture -1 to +3%*

TYPE AND NUMBER OF EARTH MOVING UNITS 1-Scraper

TYPE AND NUMBER OF COMPACTION UNITS 1-Sheepsfoot

NUMBER OF PASSES THICKNESS OF LIFT 1'

METHOD OF ADDING MOISTURE Spray

☐ FILL TESTED MEETS SPECIFICATIONS.

☒ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) 416, 417, 418, 419, AND SHOULD BE REMOVED OR REWORKED.

☒ CONTRACTOR ADVISED

OBSERVATIONS

* 100 = Top of clayey Material

F-3 FILL OBSERVATION & TESTING

Project No. C-4A-WM

Technician T Krake

Date 9/6/89

Approved By D L Swanson

Daily Report No. 58

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
416R	Retest of 416	100	111.4	15.7	110.0	15.7	99	Lean Clay
417R	Retest of 417	100	111.4	15.7	108.8	16.7	98	Lean Clay
418R	Retest of 418	100	111.4	15.7	111.3	14.8	100	Lean Clay

SPECIFICATION COMPACTION & MATERIAL

Min. Den. 95%, Moisture -1 → +3%

TYPE AND NUMBER OF EARTH MOVING UNITS 2 - Scrapers

TYPE AND NUMBER OF COMPACTION UNITS 1 - Compactor

NUMBER OF PASSES THICKNESS OF LIFT 12"

METHOD OF ADDING MOISTURE Spray

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S)
AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVATI[illegible] & TESTING

Project No. C-4A-WM

Technician T. Krake / H Kuebler

Date 10/11/89

Approved By [illegible]

Daily Report No. 83

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY MAXIMUM DRY DENSITY (pcf)	LABORATORY OPTIMUM MOISTURE CONTENT (%)	FIELD DRY DENSITY (pcf)	FIELD MOISTURE CONTENT (%)	PERCENT COMPACTION	SOIL TYPE
455	C4 E. Side S.C.	5563.5	111.4	15.7	113.4	16.2 ✓	100 ✓	Lean Clay
456	E4 E. Side S.C.	5564.5	111.4	15.7	110.3	16.6 ✓	99 ✓	Lean Clay
457	J4 S.C.	5566.5	111.4	15.7	117.6	14.8 ✓	100 ✓	Lean Clay
458	J5 Center	5567.5	111.4	15.7	105.9	16.8 ✓	95 ✓	Lean Clay
459	J6 N. Side	5568.5	111.4	15.7	108.9	17.3 ✓	98 ✓	Lean Clay
460	J7 S. Side	5569.0	111.4	15.7	108.0	15.6 ✓	97 ✓	Lean Clay
461	J8 Center	5570.0	111.4	15.7	111.8	15.3 ✓	100 ✓	Lean Clay

SPECIFICATION COMPACTION & MATERIAL

Min Den - 95%, Moisture -1% to +3%

TYPE AND NUMBER OF EARTH MOVING UNITS 1 - Scraper

TYPE AND NUMBER OF COMPACTION UNITS 1 - Compactor

NUMBER OF PASSES THICKNESS OF LIFT 12"

METHOD OF ADDING MOISTURE

Spray

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVATION & TESTING

Project No. C-4A-WM

Technician T. Krake / H. Kuebler

Date 11/1/89

Approved By G. Swanson

Daily Report No. 100

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
549	I 14 Center (S.C)	5574	101.0	20.2	103.0	22.1 ✓	100 ✓	lean clay 102
550	H 14 W. side (S.C)	5573	101.0	20.2	107.5	20.9 ✓	100	lean clay 106.4
551	G 14 Center	5572.5	101.0	20.2	97.2	21.1 ✓	96 ✓	lean clay
552	F 14 E. Side	5572.5	101.0	20.2	99.1	20.5 ✓	98 ✓	lean clay
553	E 14 Center	5571.5	101.0	20.2	101.0	20.9 ✓	100 ✓	lean clay
554	I 15 Center	5574.5	101.0	20.2	98.4	23.0 ✓	97 ✓	lean clay
555	H 15 Center	5574	101.0	20.2	101.5	23.0 ✓	100 ✓	lean clay 101.5

SPECIFICATION COMPACTION & MATERIAL

Min Den - 95%, Moisture -1% to +3%

TYPE AND NUMBER OF EARTH MOVING UNITS 1 - blade

TYPE AND NUMBER OF COMPACTION UNITS 1 - compactor

NUMBER OF PASSES THICKNESS OF LIFT 12"

METHOD OF ADDING MOISTURE Spray

☑ FILL TESTED MEETS SPECIFICATIONS.

☐ FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S)
AND SHOULD BE REMOVED OR REWORKED.

☐ CONTRACTOR ADVISED

OBSERVATIONS

F-3 FILL OBSERVATI & TESTING

Project No. C-49-WM

Technician T. Krake / H Knebler

Date 11/1/89

Approved By D. L. Swanson

Daily Report No. 100

Small Section of dike in Northeast Corner to allow anchoring of Liner

TEST NO.	LOCATION	DEPTH OR ELEVATION (FEET)	LABORATORY		FIELD		PERCENT COMPACTION	SOIL TYPE
			MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)		
556	Sta 1+25 on CL	5605	101.0	20.2	98.4	22.0	97	Lean Clay
557	Sta 0+75 5' Rt	5606	101.0	20.2	98.6	21.0	98	Lean Clay

SPECIFICATION COMPACTION & MATERIAL

Min Den - 95%, Moisture ±2%

TYPE AND NUMBER OF EARTH MOVING UNITS 1 - Scraper

TYPE AND NUMBER OF COMPACTION UNITS 1 - Compactor

NUMBER OF PASSES THICKNESS OF LIFT 12"

METHOD OF ADDING MOISTURE Spray

- [x] FILL TESTED MEETS SPECIFICATIONS.
- [ ] FILL TESTED DOES NOT MEET SPECIFICATIONS AS INDICATED BY TEST NO.(S) AND SHOULD BE REMOVED OR REWORKED.
- [ ] CONTRACTOR ADVISED

OBSERVATIONS

F-4 SOIL SAMPLING LOG

SAMPLE NO. 78

PROJECT NO. C-4A-WM

DATE 7/13/89

DELIVERED TO LABORATORY

SAMPLED BY J Krake
Q.C. G.L. Swanson

DATE 7/13/89

LOCATION Sta 33+00 off grade
(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk
(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Silty Sand

INTENDED USE Dike Construction
(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, P.I.
(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

T-10 GRADATION ANALYSIS

WORKSHEET

TECHNICIAN: T. Kraky
APPROVED BY: D. L. Swanson
PROJECT NO: C-4A-WM
DATE 7/13/89

SAMPLE NO. 28
VISUAL DESCRIPTION: Silty Sand

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED					16.7	51.2
DRY WT. PASSING					1007.9	973.4
% OF TOTAL PASSING					98.4	95.0
	W% = _____					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	1127.1	1024.0
RETAINED ON NO. 4	_____	_____
PASSING NO. 4	_____	_____

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	85.0 / 91.9	938.8 / 932.7	91.7 / 91.0
16	103.7	920.9	90.
30 (40)	128.1 / 165.6	896.5 / 859	88 / 84
50	268.4	756.2	74
100	513.7	510.9	50
200	751.7	272.9	27
PAN			—
TOTAL			—

FACTOR = $\frac{W\%}{W}$ = _____ = _____

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH	675.5			
WT. DRY SOIL AND DISH	614.2			
WT. DISH				
WT. OF DRY SOIL	614.2	_____ = W		
% MOISTURE	10.0			

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT: _____
DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB. _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ
	START MIX	—	—	—	—
	STOP MIX	—	—	—	—
	0.5 min				
	1.0 min				
	4.0 min				
	19 min				
	60 min				
	7h 15 min				
	25h 45 min				

FACTOR X CORRECTED READING = % OF TOTAL PASSING

% OF TOTAL PASSING	PARTICLE DIAMETER	
_____	_____	
_____	_____	
	0.050 mm	
	0.037 mm	
	0.019 mm	
	0.009 mm	
	0.005 mm	
	0.002 mm	
	0.001 mm	

GRAVEL _____ % SAND _____ % CLAY-SLIT _____ %. STORAGE LOCATION _____

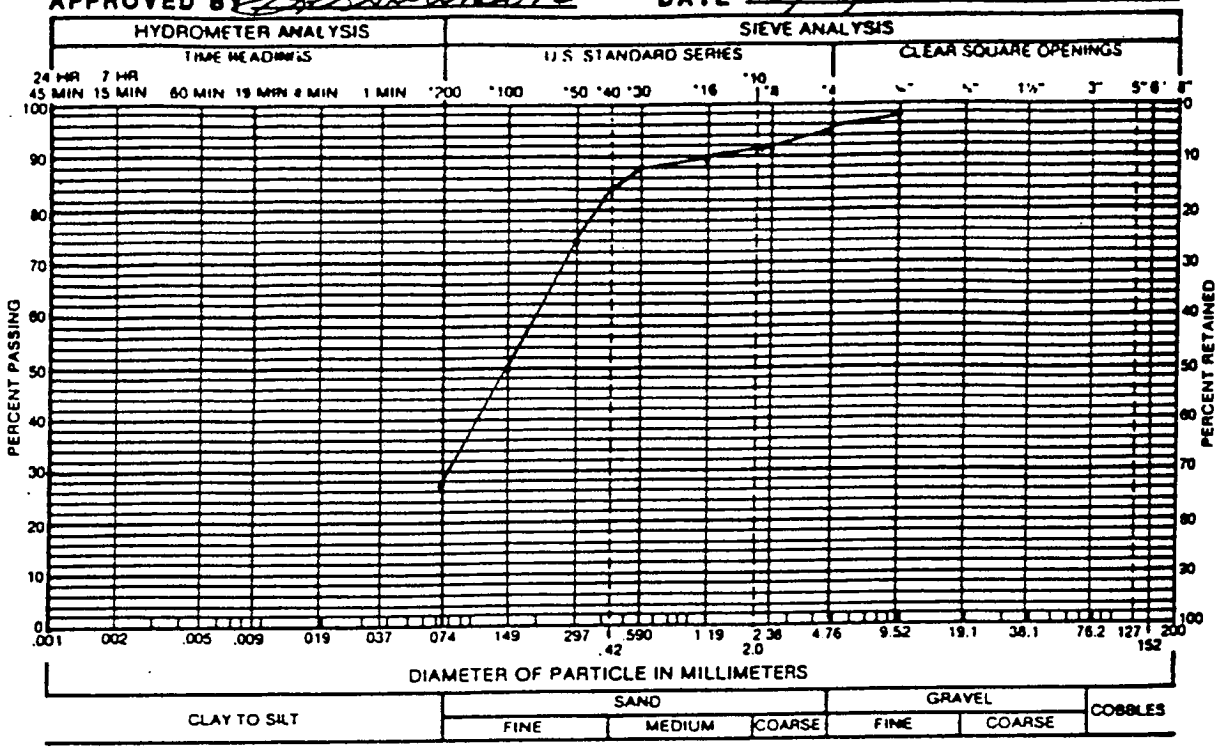
* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

-2-79

F-12 GRADATION TEST RESULTS

TECHNICIAN T. Kraky PROJECT NO. C-4A-WM

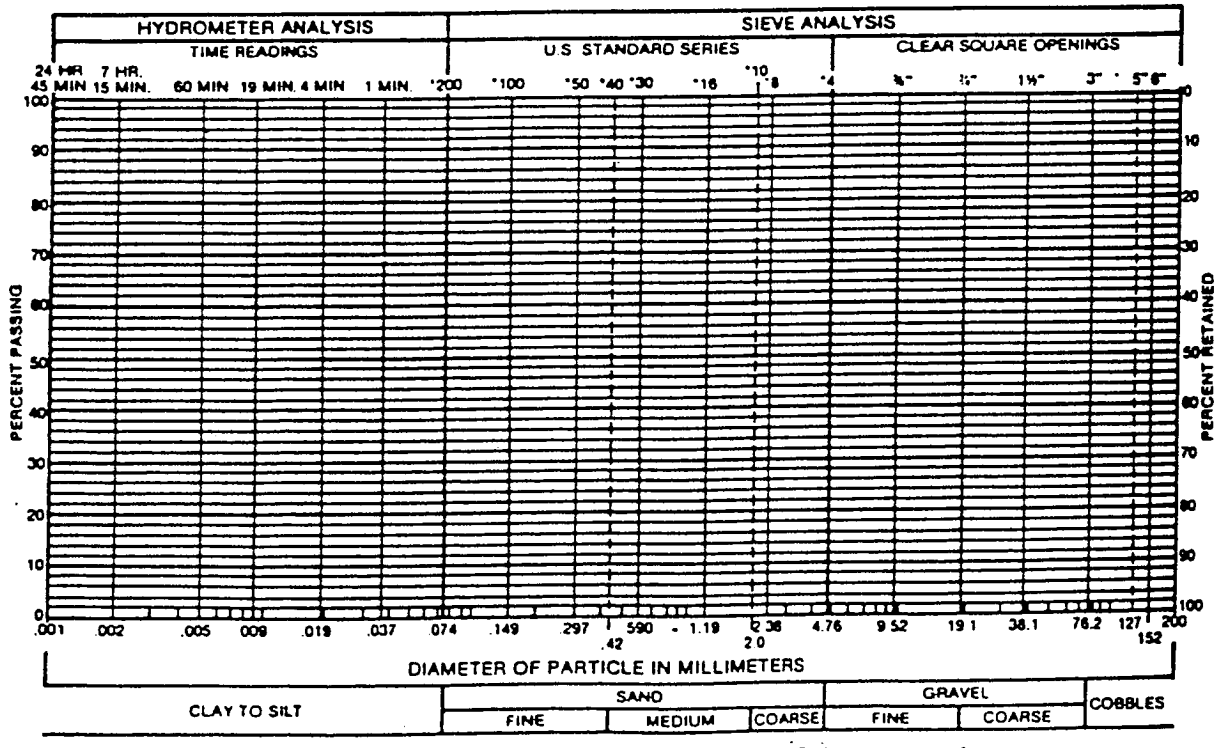
APPROVED BY H. L. Swanson DATE 7/13/89



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Silty Sand FROM Sta 33+00



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN H. Kuebler PROJECT NO. C-4A-WM
APPROVED BY G.L. Swanson DATE 7/13/89

SAMPLE NO. 78
SAMPLE DESCRIPTION Silty Sand COLOR

ATTERBERG LIMITS	PL	LL
PREP. DISH	RUN BY	
NO. OF BLOWS		
DISH NO.		
WT. OF WET SOIL & DISH		
WT. OF DRY SOIL & DISH		
WT. OF DISH		
WT. OF WATER		
WT. OF DRY SOIL		
WATER CONTENT		

Cannot be rolled

10 taps before sides touch

-200	
RUN BY	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

LIQUID LIMIT, LL
PLASTIC INDEX, PI ← Nonplastic

PERCENT -200 %

MOISTURE CONTENT	
RUN BY	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

DENSITY	
RUN BY	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

MOISTURE CONTENT %

DRY DENSITY PCF

REMARKS:

F-4 SOIL SAMPLING LOG

SAMPLE NO. 60

PROJECT NO. C-4A-WM

DATE 7/5/89

DELIVERED TO LABORATORY

SAMPLED BY T Krake
P.C. G Swanson

DATE 7/5/89

LOCATION Sta 22+00 off grade

(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk

(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Silty Clayey Sand

INTENDED USE Dike Construction

(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, PI, Proctor

(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

SOIL/AGGREGATE - MOISTURE DENSITY RELATIONS

#60

Job No. C-4A-WM

Lab./Invoice No.

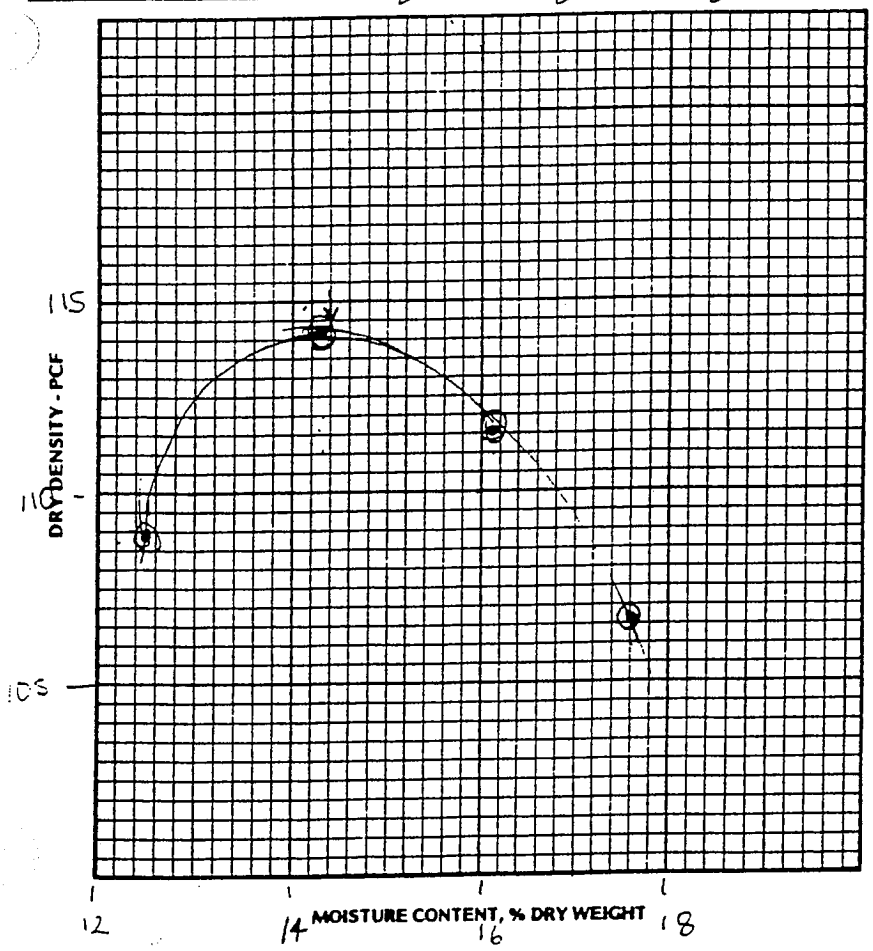
Type of Material Silty Clayey Sand — Sampled By T.K. — Date 7-5-89

Source of Material Sta 22+00 — Submitted By — Date

Test Procedure D698 — Tested/Calc. By H Kubler — Date 7-6-89

Reviewed By G.C. Swanson — Date 7/7/89

Trial No.	1	2	3	4	5	6	7
Water, estimated %							
Water, cc	50	100	0	150			
Wt. Sample + Mold	6200	6188	6078	6124			
Wt. Mold	4227	4227	4227	4227			
Wt. Wet Sample, gm	1973	1961	1851	1897			
Wt. Wet Sample, lbs.	4.35	4.32	4.08	4.18			
Wet Density, pcf	130.5	129.6	122.4	125.4			
Moisture Sample, wet	304.0	267.0	268.2	312.4			
Moisture Sample, Dry	266.0	230.0	238.5	265.6			
Wt. Moisture	38.0	37.0	29.7	46.8			
Moisture, %	14.3	16.1	12.5	17.6			
Dry Density, pcf	114.2	111.6	108.8	106.6			



Max. Dry density, pcf 114.3

Optimum Moisture Content, % 14.4

Diameter of Mold, in. 4inch

Height of Mold, in. 4.584

No. of Layers 3

Blows per Layer 25

Wt. of Hammer, lbs. 5.5.

Height of Drop 12inch

Material Used -#4 material

WESTERN TECHNOLOGIES INC.

WORKSHEET

TECHNICIAN: T. Krake
APPROVED BY: L. Swanson

PROJECT NO: C-4A-WM
DATE 7/5/85

SAMPLE NO. 60
VISUAL DESCRIPTION: Silty Clayey Sand

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED						17.4
DRY WT. PASSING						378.0
% OF TOTAL PASSING						98.1 ✓
	W% = _____					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	1010.0	895.4
RETAINED ON NO. 4	_____	_____
PASSING NO. 4	_____	_____

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	48.1 / 54.2	874.3 / 841.2	99.6 ✓ / 93.9 ✓
16	67.6	827.8	92.5 ✓
30 (40)	87.7 / 110.0	807.7 / 731.7	90.2 ✓ / 88 ✓
50	152.9	742.5	83 ✓
100	253.7	641.7	72 ✓
200	461.1	434.3	49 ✓
PAN			—
TOTAL			—

FACTOR = $\frac{W\%}{W}$ = _____ = _____

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH	330.4			
WT. DRY SOIL AND DISH	780.5			
WT. DISH				
WT. OF DRY SOIL		= W		
% MOISTURE	12.8 ✓			

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT _____
DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ
	START MIX	—	—	—	—
	STOP MIX	—	—	—	—
	0.5 min				
	1.0 min				
	4.0 min				
	19 min				
	60 min				
	7h 15 min				
	25h 45 min				

FACTOR X CORRECTED READING = % OF TOTAL PASSING

% OF TOTAL PASSING	PARTICLE DIAMETER	
—	—	
—	—	
	0.050 mm	
	0.037 mm	
	0.019 mm	
	0.009 mm	
	0.005 mm	
	0.002 mm	
	0.001 mm	

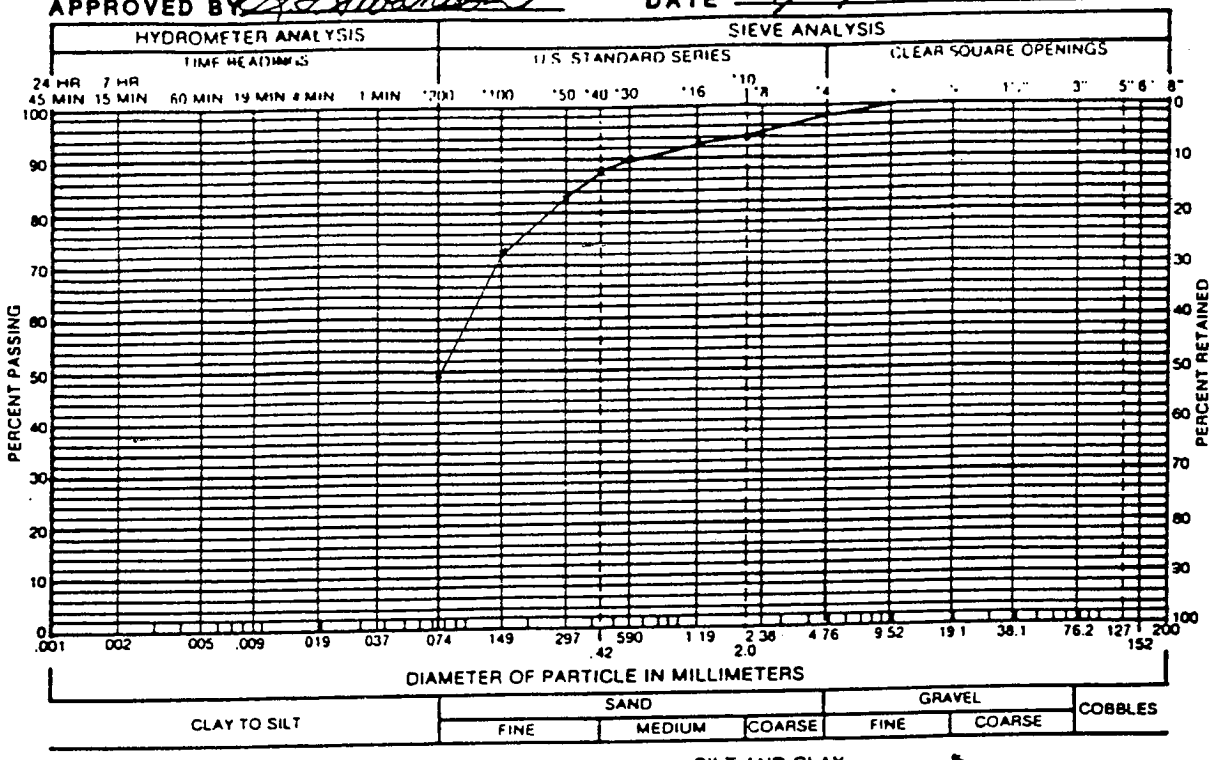
GRAVEL _____ % SAND _____ % CLAY-SLIT _____ %. STORAGE LOCATION _____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

F-12 GRADATION TEST RESULTS

TECHNICIAN. T. Krake PROJECT NO. C-48-WM

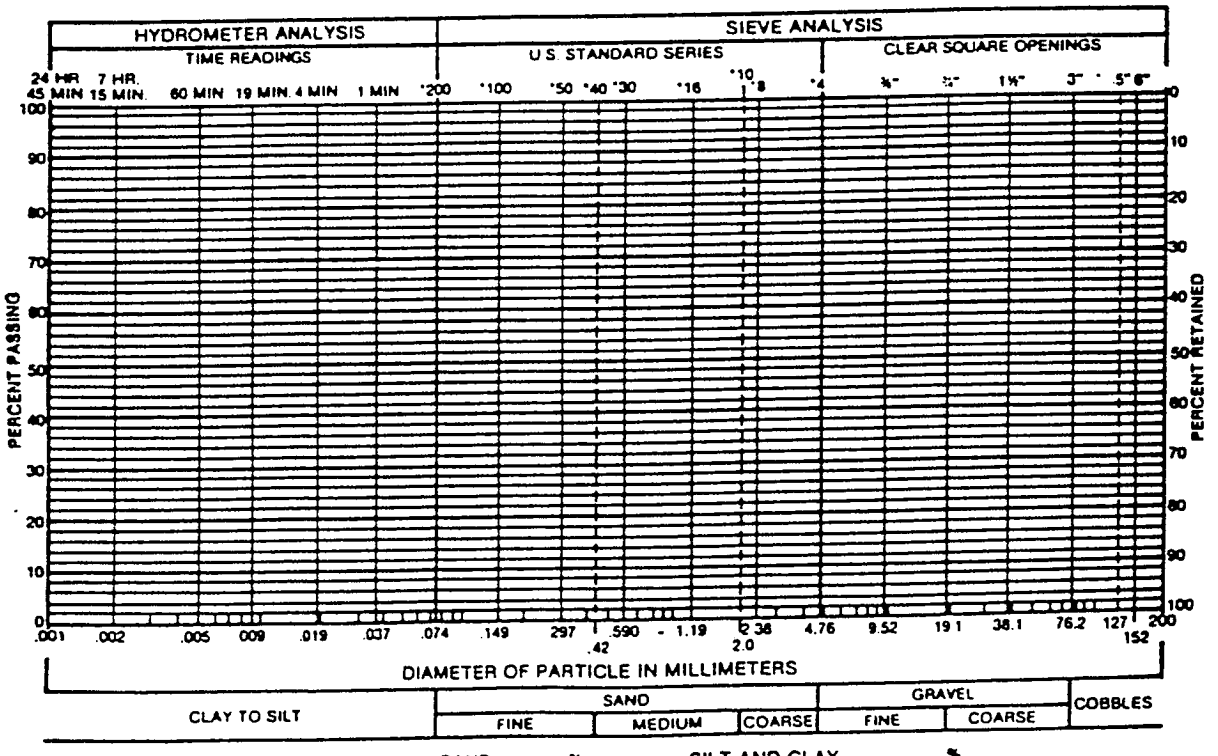
APPROVED BY H.L. Swanson DATE 7/5/89



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Silty Clayey Sand FROM Sta 22+00



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN H Knabler PROJECT NO. C-4A-WM

APPROVED BY [signature] DATE 7/5/89

SAMPLE NO. 60

SAMPLE DESCRIPTION Silty Clayey Sand COLOR Red-tan

ATTERBERG LIMITS	PL	LL
PREP. DISH	RUN BY	
NO. OF BLOWS	—	20
DISH NO.	17	19
WT. OF WET SOIL & DISH	11.00	20.90
WT. OF DRY SOIL & DISH	10.61	19.10
WT. OF DISH	7.60	10.79
WT. OF WATER	.39	1.80
WT. OF DRY SOIL	3.01	8.31
WATER CONTENT	13.0	21.7

-200	
RUN BY	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

LIQUID LIMIT, LL 21

PLASTIC INDEX, PI 8 OK

PERCENT -200 _____ %

MOISTURE CONTENT	
RUN BY	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

DENSITY	
RUN BY	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

MOISTURE CONTENT _____ %

DRY DENSITY _____ PCF

REMARKS: _____

F-4 SOIL SAMPLING LOG

SAMPLE NO. 33

PROJECT NO. C-4A-WM

DATE 6/15/89

DELIVERED TO LABORATORY

SAMPLED BY T Krake
Q.C. L. Swanson

DATE 6/15/89

LOCATION Sta 27+00 off grade
(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk
(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Clayey Sand

INTENDED USE Dike Construction
(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, P.E, L.L.
(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

P-10 GRADATION ANALYSIS

WORKSHEET

TECHNICIAN: J. Sanchez PROJECT NO: C-4A-WM

APPROVED BY: [signature] DATE: 6/15/89

SAMPLE NO. 33

VISUAL DESCRIPTION: Clayey Sand

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN					0.0	0.0
DRY WT. RETAINED					37.0	9%
DRY WT. PASSING					849.1	788.1
% OF TOTAL PASSING					95.8	89
	W% = _____					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	957.9	886.1
RETAINED ON NO. 4	_____	_____
PASSING NO. 4	_____	_____

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	108 / 149	778.1 / 737.1	88 / 83
16	173	713.1	80
30 (40)	249 / 275	637.1 / 611.1	72 / 69
50	306	580.1	65
100	377	509.1	57
200	577	309.1	35
PAN			—
TOTAL			—

FACTOR = $\frac{W\%}{W}$ = _____ = _____

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH	1287.5			
WT. DRY SOIL AND DISH	1191.4			
WT. DISH	0.0			
WT. OF DRY SOIL	1191.4	= W		
% MOISTURE	3.1			

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT _____

DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ
	START MIX	—	—	—	—
	STOP MIX	—	—	—	—
	0.5 min				
	1.0 min				
	4.0 min				
	19 min				
	60 min				
	7h 15 min				
	25h 45 min				

FACTOR X CORRECTED READING = % OF TOTAL PASSING

% OF TOTAL PASSING	PARTICLE DIAMETER	
_____	_____	
_____	_____	
	0.050 mm	
	0.037 mm	
	0.019 mm	
	0.009 mm	
	0.005 mm	
	0.002 mm	
	0.001 mm	

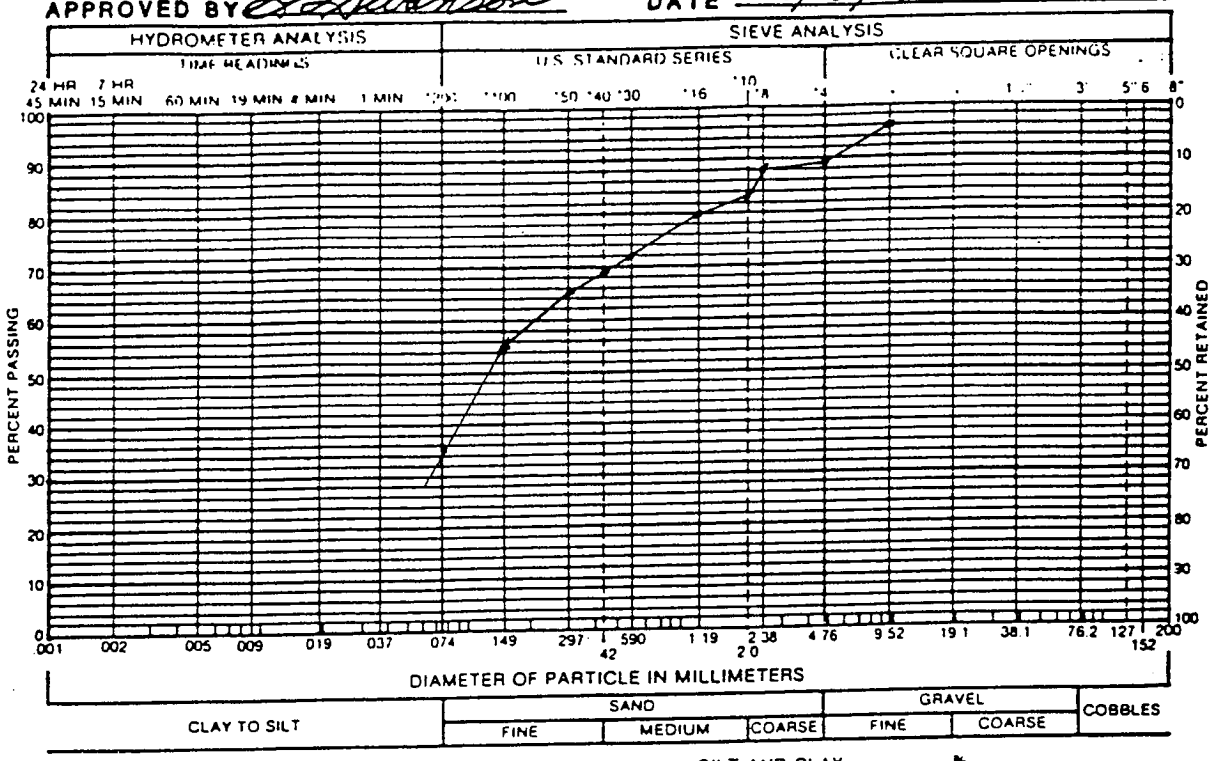
GRAVEL _____ % SAND _____ % CLAY-SLIT _____ %. STORAGE LOCATION _____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

F-12 GRADATION TEST RESULTS

TECHNICIAN T. Krake PROJECT NO. C-4A-WM

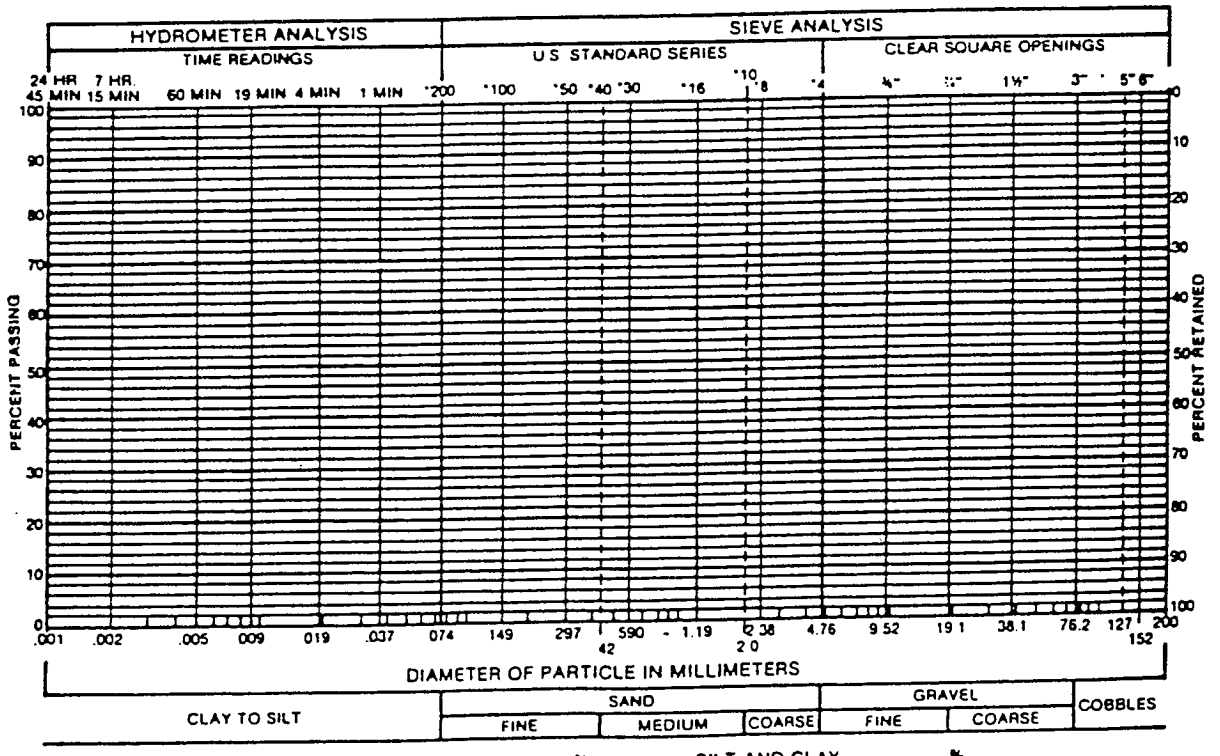
APPROVED BY S.O. Swanson DATE 6/15/89



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Clayey Sand FROM Sta 27+00



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN T Krahe PROJECT NO. C-4A-WM
APPROVED BY [signature] DATE 6/15/89

SAMPLE NO. 33
SAMPLE DESCRIPTION Clayey Sand COLOR

ATTERBERG LIMITS	PL	LL
PREP. DISH	RUN BY	
NO. OF BLOWS	—	20
DISH NO.	14	
WT. OF WET SOIL & DISH	18.21	41.72
WT. OF DRY SOIL & DISH	17.60	39.19
WT. OF DISH	13.03	29.31
WT. OF WATER	.61	2.53
WT. OF DRY SOIL	4.57	9.88
WATER CONTENT	13.3	25.6

LIQUID LIMIT, LL 25
PLASTIC INDEX, PI 12

-200	
RUN BY	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 _____ %

MOISTURE CONTENT	
RUN BY	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

MOISTURE CONTENT _____ %

DENSITY	
RUN BY	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY _____ PCF

REMARKS:

F-4 SOIL SAMPLING LOG

SAMPLE NO. 15 **PROJECT NO.** C-4A-WM

DATE 6/1/89 **DELIVERED TO LABORATORY**

SAMPLED BY T. Krake **DATE** 6/1/89

P. C. Swenson

LOCATION Sta 20+00 off grade

(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk

(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Sandy Silt

INTENDED USE Dike Construction

(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Proctor, Sieve, P.I.

(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

SOIL/AGGREGATE - MOISTURE DENSITY RELATIONS

C-4A-WM
#15

Job No. _____

Lab./Invoice No. _____

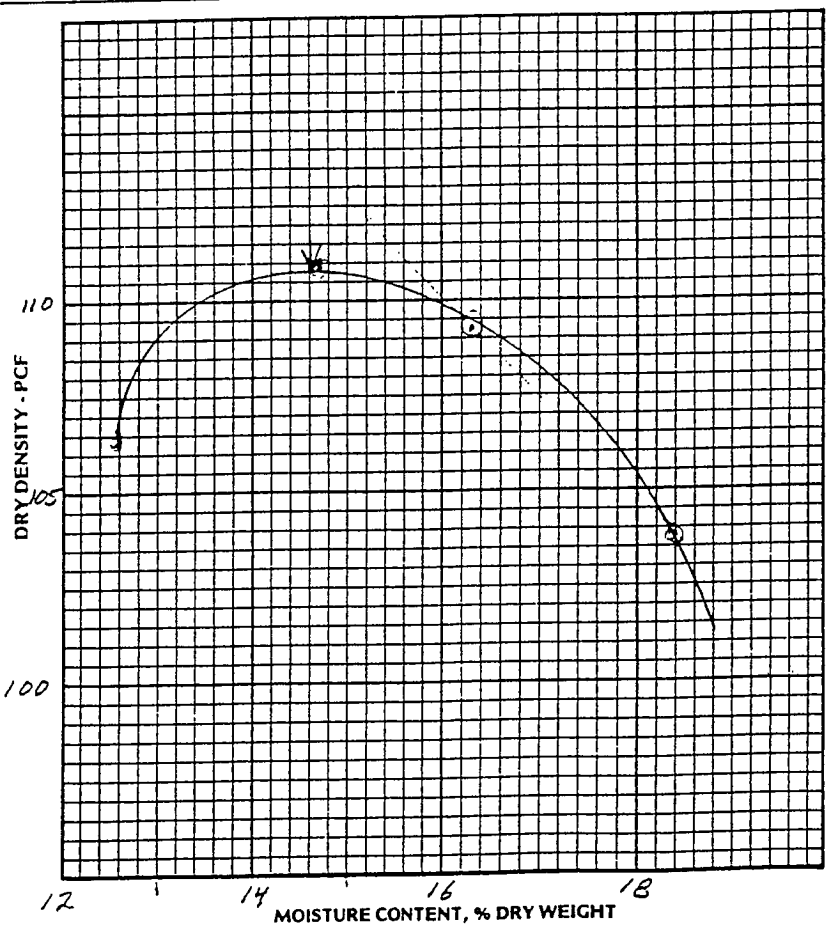
Type of Material Sandy Silt — Sampled By _____ Date _____

Source of Material Sta 20+00 — Submitted By _____ Date _____

Test Procedure D-698 A — Tested/Calc. By Randy Day Date 6-1-89

Reviewed By H.Swanson Date 6/6/89

Trial No.	1	2	3	4	5	6	7
Water, estimated %							
Water, cc	50	100	150	0			
Wt. Sample + Mold	6151	6148	6086	6038			
Wt. Mold	4227	4227	4227	4227			
Wt. Wet Sample, gm	1924	1921	1859	1811			
Wt. Wet Sample, lbs.	4.242	4.235	4.098	3.992			
Wet Density, pcf	127.2	127.1	122.9	119.8			
Moisture Sample, wet	324.5	344.0	352.5	316.4			
Moisture Sample, Dry	283.0	295.9	297.6	281.1			
Wt. Moisture	41.5	48.1	54.9	35.3			
Moisture, %	14.7	16.3	18.4	12.6			
Dry Density, pcf	110.9	109.3	103.8	106.4			



Max. Dry density, pcf 110.9

Optimum Moisture Content, % 14.6

Diameter of Mold, in. 4

Height of Mold, in. 4.584

No. of Layers 3

Blows per Layer 25

Wt. of Hammer, lbs. 5.5

Height of Drop 12

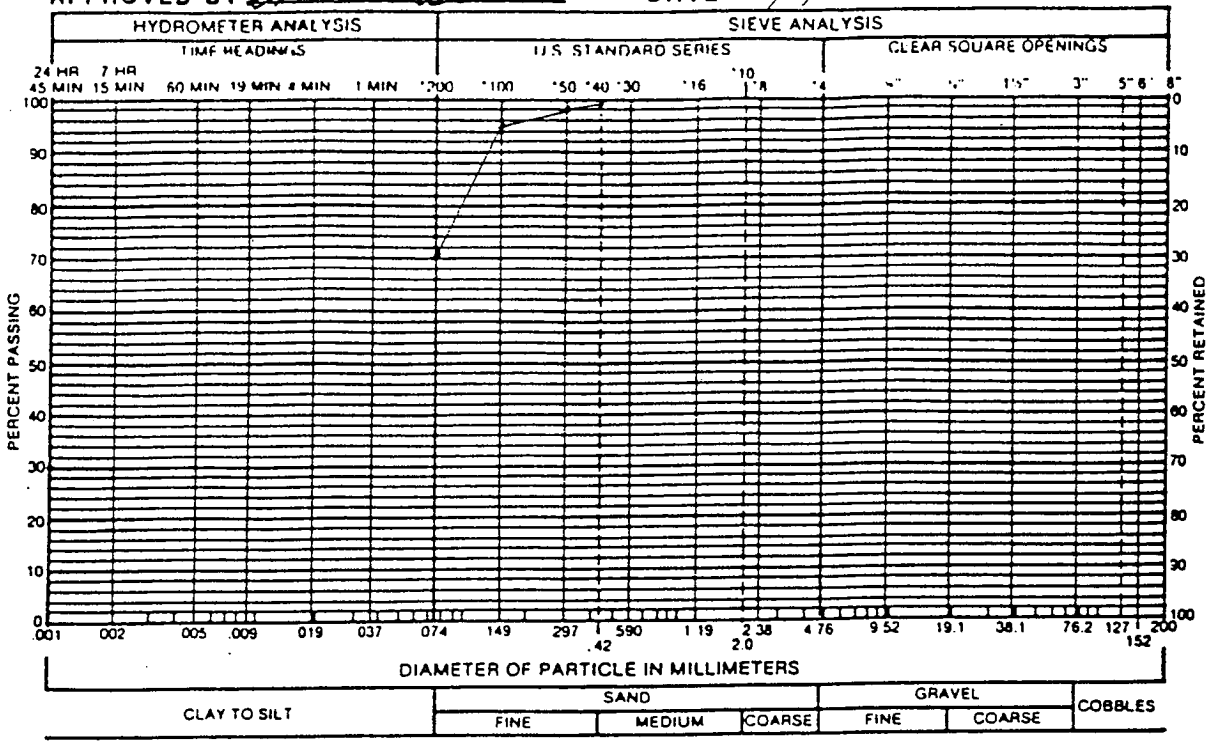
Material Used - #4

WESTERN TECHNOLOGIES INC.

F-12 GRADATION TEST RESULTS

TECHNICIAN R. Day PROJECT NO. C-4A-WM

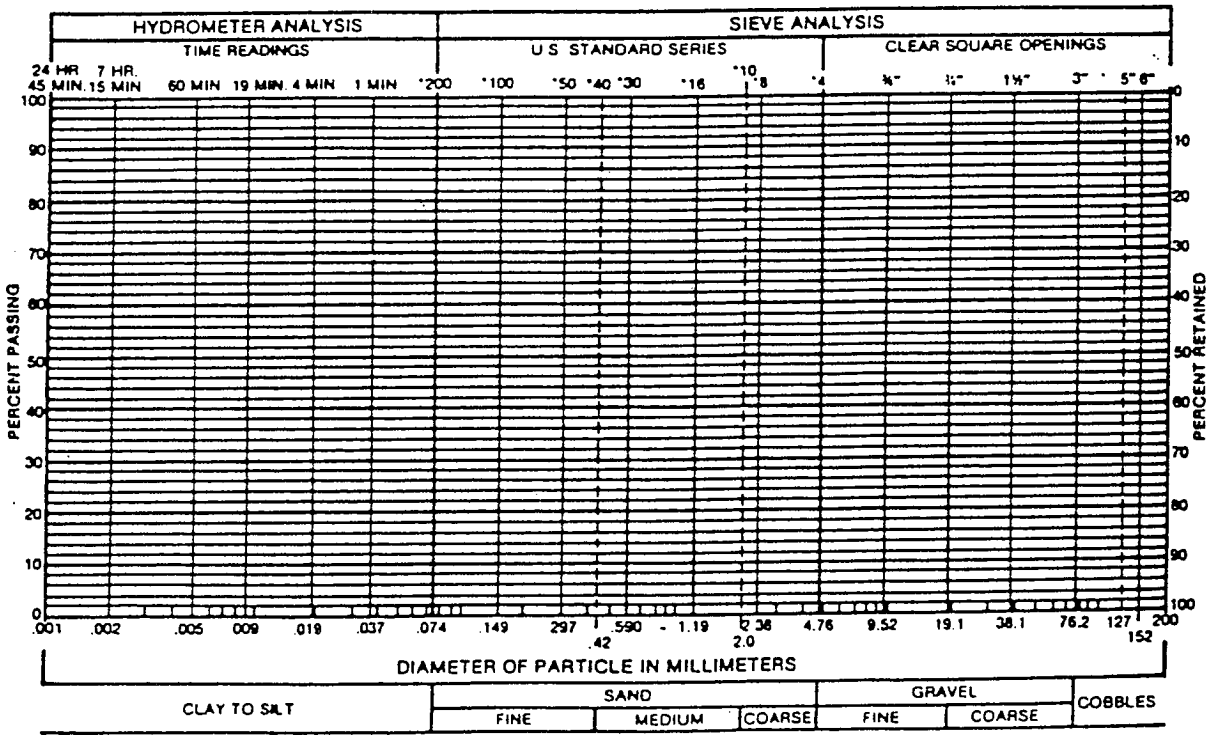
APPROVED BY [signature] DATE 6/1/99



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Sandy Silt FROM Sta 20+00



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

WORKSHEET

TECHNICIAN: R. Day
APPROVED BY: [signature]
PROJECT NO: C-4A-WM
DATE 6/1/89

SAMPLE NO. 15
VISUAL DESCRIPTION: Sandy Silt

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED				0	0	0
DRY WT. PASSING				766.6	766.6	766.6
% OF TOTAL PASSING				100	100	100
	W% = _____					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	878.5	766.6
RETAINED ON NO. 4	_____	_____
PASSING NO. 4	_____	_____

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	1.1 / 2.0	765.5 / 764.6	99.8 / 99.7
16	3.3	763.3	99.6
30 (40)	7.0 / 8.9	759.6 / 757.7	99.1 / 98.8
50	12.7	753.9	98.3
100	33.1	733.5	95.7
200	224.5	542.1	71
PAN			—
TOTAL			—

FACTOR = W% / W = _____ = _____

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH	674.4			
WT. DRY SOIL AND DISH	588.3			
WT. DISH	0.0			
WT. OF DRY SOIL	588.3	= W		
% MOISTURE	14.6			

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT _____
DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—				
	STOP MIX	—	—	—	—				
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

GRAVEL _____ % SAND _____ % CLAY-SLIT _____ %. STORAGE LOCATION _____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN Drake PROJECT NO. C-4A-WM

APPROVED BY [signature] DATE 4/1/89

SAMPLE NO. 15

SAMPLE DESCRIPTION Sandy Silt COLOR Red

ATTERBERG LIMITS	PL	LL
PREP. DISH	RUN BY	
NO. OF BLOWS	—	25
DISH NO.		19
WT. OF WET SOIL & DISH		30.01
WT. OF DRY SOIL & DISH		27.05
WT. OF DISH		10.79
WT. OF WATER	NP	2.96
WT. OF DRY SOIL		16.26
WATER CONTENT		18.2

LIQUID LIMIT, LL 18

PLASTIC INDEX, PI NP

-200	
RUN BY	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 _____ %

MOISTURE CONTENT	
RUN BY	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

MOISTURE CONTENT _____ %

DENSITY	
RUN BY	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY _____ PCF

REMARKS: _____

F-4 SOIL SAMPLING LOG

SAMPLE NO. 7

PROJECT NO. C-4A-WM

DATE 5/22/89

DELIVERED TO LABORATORY

SAMPLED BY JK
Q.C. G. Swanson

DATE

LOCATION Sta 17+00. Off Grade Approx elv 5570
(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk
(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Sandy Silt

INTENDED USE Dike Construction
(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, P.I
(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

WORKSHEET

TECHNICIAN: T. Krake
PROJECT NO: C-49-WM
APPROVED BY: L. L. Swanson
DATE: 2/22/89

SAMPLE NO. 7
VISUAL DESCRIPTION: Sandy Silt

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED					0	0
DRY WT. PASSING					620.0	620.0
% OF TOTAL PASSING					100 ✓	100 ✓
	W% = _____					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	—	620.0
RETAINED ON NO. 4	_____	_____
PASSING NO. 4	_____	_____

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	0 / 0	620.0 / 620.0	100 ✓ / 100 ✓
16	1.1	618.9	99.8 ✓
30 (40)	1.5 / 10.1	618.5 / 609.9	99.8 ✓ / 98.4 ✓
50	17.6	602.4	97.2 ✓
100	55.7	564.3	91.0 ✓
200	292.3	327.7	53 ✓
PAN			—
TOTAL			—

FACTOR = $\frac{W\%}{W}$ = _____ = _____

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH				
WT. DRY SOIL AND DISH				
WT. DISH				
WT. OF DRY SOIL		= W		
% MOISTURE				

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT _____
DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB. _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—		—	—	
	STOP MIX	—	—	—	—		—	—	
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

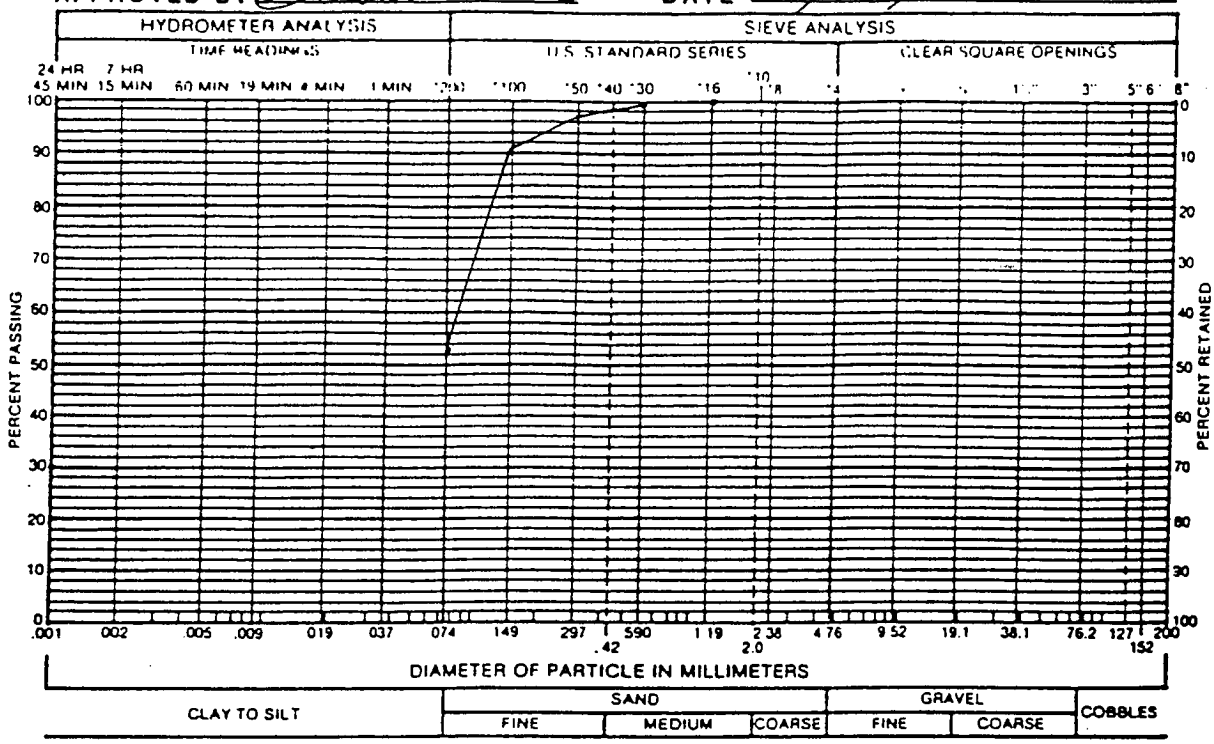
GRAVEL _____ % SAND _____ % CLAY-SLIT _____ %. STORAGE LOCATION _____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

F-12 GRADATION TEST RESULTS

TECHNICIAN J Krake PROJECT NO. C-4A-WM

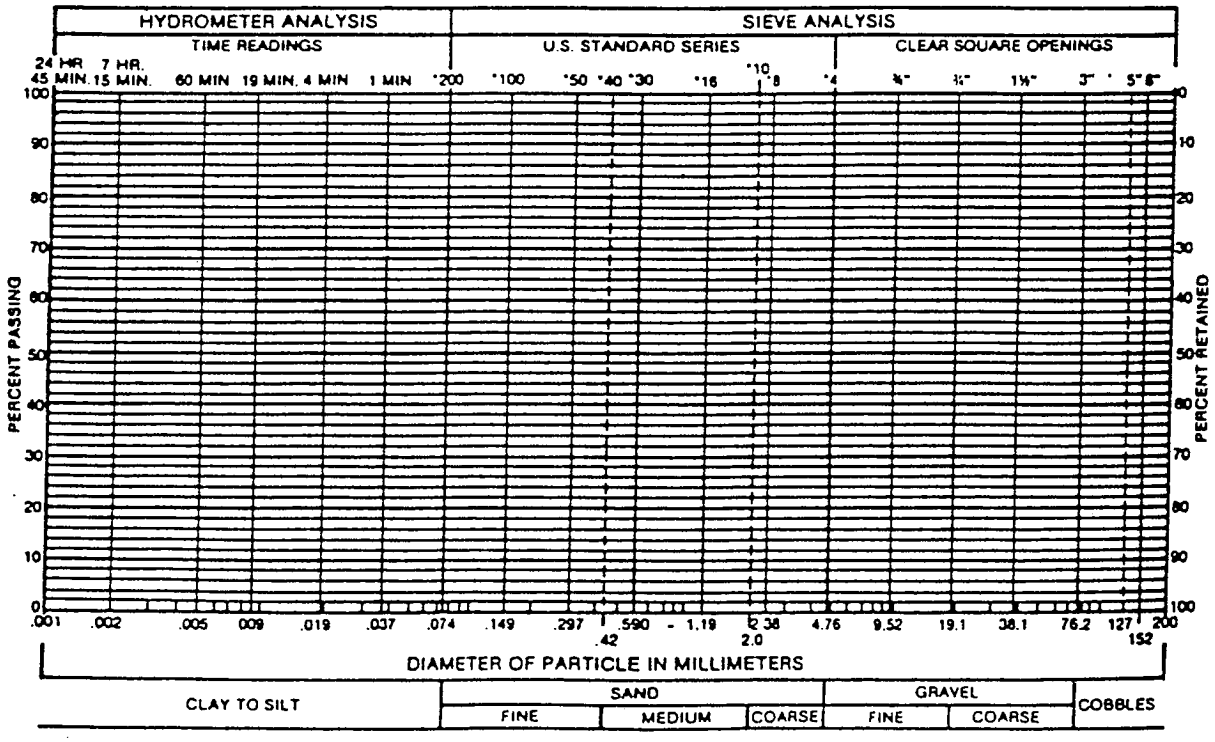
APPROVED BY S Swanson DATE 5/22/89



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT 21 % PLASTICITY INDEX NP %

SAMPLE OF Sandy Silt FROM Sta 46 17+00



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN: T. Krake PROJECT NO. C-4A-WM

APPROVED BY _____ DATE 5/22/89

SAMPLE NO. 7

SAMPLE DESCRIPTION _____ COLOR _____

ATTERBERG LIMITS	PL	LL
PREP. DISH _____ RUN BY _____		
NO. OF BLOWS	———	30
DISH NO.	19	10
WT. OF WET SOIL & DISH		28.77
WT. OF DRY SOIL & DISH		25.79
WT. OF DISH	10.82	11.54
WT. OF WATER	NP	2.98
WT. OF DRY SOIL		14.25
WATER CONTENT		20.9

LIQUID LIMIT, LL 21

PLASTIC INDEX, PI NP

-200	
RUN BY _____	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 _____ %

MOISTURE CONTENT	
RUN BY _____	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

MOISTURE CONTENT _____ %

DENSITY	
RUN BY _____	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY _____ PCF

REMARKS: _____

F-4 SOIL SAMPLING LOG

SAMPLE NO. 6

DATE 5/19/87

SAMPLED BY T. Krake
Q.C. G.L. Swanson

PROJECT NO. C-4A-W1.7

DELIVERED TO LABORATORY

DATE 5/19/87
Daily Report No. 8

LOCATION Sta 13+00 on CL as dumped

(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-6"

SAMPLE TYPE Bulk

(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Sandy Silt

INTENDED USE Dike Construction

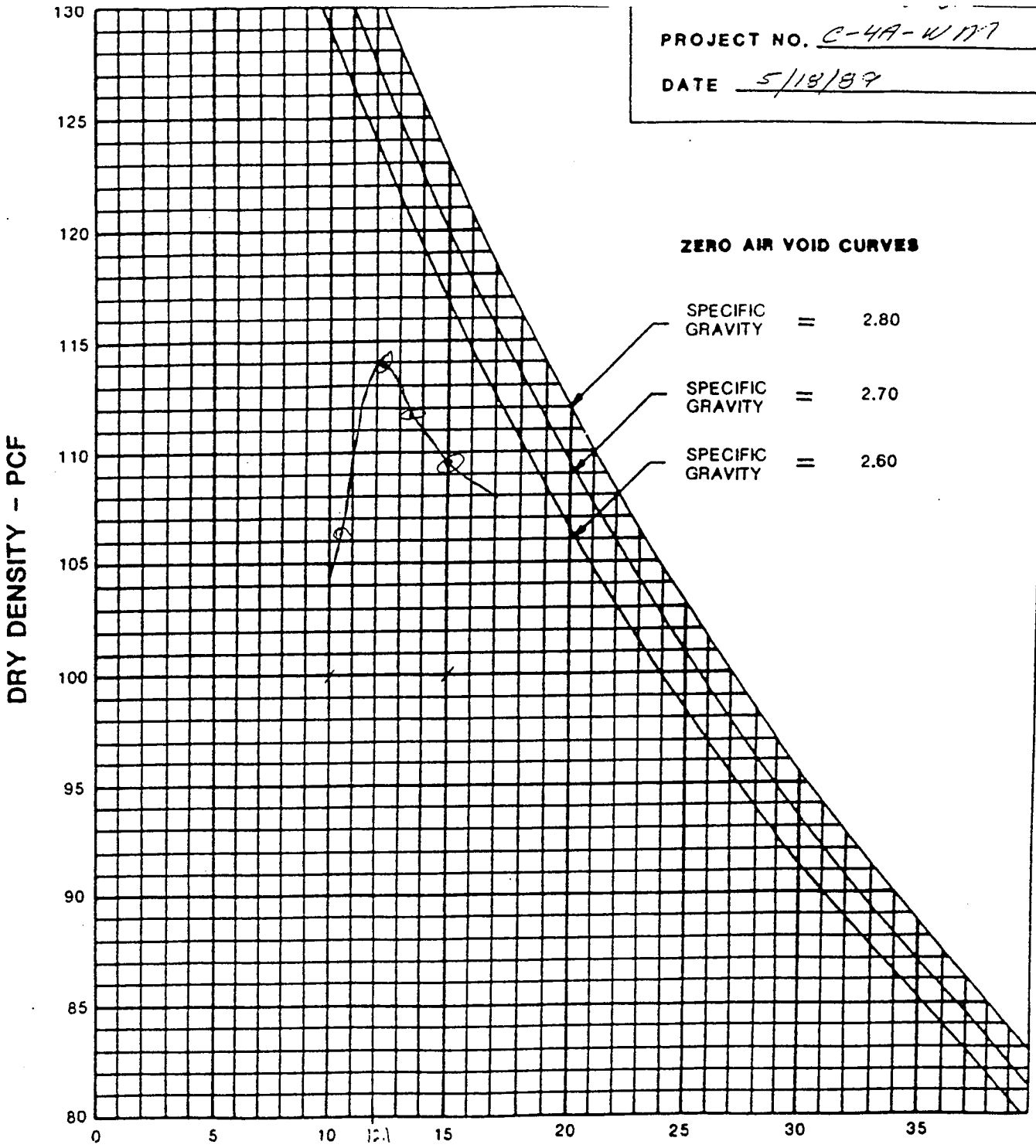
(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Proctor

(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

PROJECT NO. C-4A-W177

DATE 5/18/87



LOCATION : Sta 18+00 ON CL	F-8 MOISTURE-DENSITY RELATIONSHIPS - 1
SAMPLE NO:	
MATERIAL DESCIPTION: Sandy SILT (12.1)	
MAX. DRY DENSITY : 114.0 PCF OPT. MOIST. CONTENT : 13.5 %	PROCEDURE: D 698 A
LIQUID LIMIT : PLASTICITY INDEX :	TECHNICIAN: TM
GRAVEL : % SAND : % SILT AND CLAY (-200) : %	APPROVED BY: H. Thomson

TECHNICIAN. T Krake PROJECT NO. C-414-4111

APPROVED BY H Swanson DATE 5/12/83

SAMPLE NO. Sta 18+00 on ℄ as dumped

SAMPLE DESCRIPTION Sandy Silt COLOR Lt Pk L

ASTM D 698-78 [✓] ASTM D 1557-78 [ ] METHOD: (A) B C D OTHER _____

TEST DATA							
POINT NUMBER	1	2	3	4	5	6	7
WT. OF SOIL AND TARE							
AMOUNT OF ADDED WATER, WT.							
WT.OF SOIL, WATER & TARE							
AMOUNT OF WATER ADDED, VOL.	200	150	100				
WT. OF MOLD AND WET SOIL	13.54	13.57	13.23	13.51			
WT. OF MOLD	9.31						
WT.OF WET SOIL	4.23	4.26	3.92	4.20			
WET DENSITY PCF	126.9	127.8	117.8	126.0			
DISH NUMBER							
WT. OF DISH AND WET SOIL							
WT. OF DISH AND SOIL							
WT. OF DISH							
WT. OF WATER							
WT. OF DRY SOIL							
MOISTURE CONTENT %	13.5	12.1	10.6	15.0			
DRY DENSITY PCF	111.8	114.0	106.3	109.6			

AT WHICH POINT WAS SOIL PUMPING ? #1

AT WHICH POINT WAS SOIL BLEEDING ? _____

OPTIMUM MOISTURE CONTENT 12.5 %

MAXIMUM DRY DENSITY 114.0 pcf

ROCK CORRECTION DATA

(WEIGHTS ARE PER POINT)

+#4 _____ % = _____ lb.

-#4 _____ % = _____ lb.

100% = _____ lb.

COMMENTS: _____

F-4 SOIL SAMPLING LOG

SAMPLE NO. 1

PROJECT NO. C-4A-WM

DATE 5/10

DELIVERED TO LABORATORY

SAMPLED BY JK

DATE 5/10/89

LOCATION sta 15+00 180'N of S. Dike

(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 1' below original ground

SAMPLE TYPE Bulk sample

(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION SM [illegible]

INTENDED USE Dike Construction

(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, P.I., [illegible]

(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

WORKSHEET

TECHNICIAN. Tom Krake PROJECT NO. C-4A-WM
APPROVED BY [signature] DATE 5/15/89

SAMPLE NO. Sta 15+00 180'N of South Dike
SAMPLE DESCRIPTION Sandy Silty Clay COLOR
ASTM D 698-78 ☑ ASTM D 1557-78 ☐ METHOD: (A) B C D OTHER

TEST DATA

POINT NUMBER	1	2	3	4	5	6	7
WT. OF SOIL AND TARE							
AMOUNT OF ADDED WATER, WT.							
WT.OF SOIL, WATER & TARE							
AMOUNT OF WATER ADDED, VOL.	150	200	250	300			
WT. OF MOLD AND WET SOIL	6020.5	6097.9	6152.3	6219.1			
WT. OF MOLD	4221.0						
WT.OF WET SOIL	1799.5	1873.9	1931.3	1898.1			
WET DENSITY PCF	119.1	124.2	127.8	125.4			
DISH NUMBER							
WT. OF DISH AND WET SOIL	237.5	253.0	218.7	207.7			
WT. OF DISH AND SOIL	213.1	223.0	189.5	176.7			
WT. OF DISH ? →	—						
WT. OF WATER	24.4	30.0	29.2	30.3			
WT. OF DRY SOIL ? →	213.1	223.0	189.5	176.7			
MOISTURE CONTENT %	11.5	13.5	15.4	17.4			
DRY DENSITY PCF	106.8	109.4	110.7	106.8			

AT WHICH POINT WAS SOIL PUMPING ? _____

AT WHICH POINT WAS SOIL BLEEDING ? _____

OPTIMUM MOISTURE CONTENT 15.4 %

MAXIMUM DRY DENSITY 110.7 pcf

ROCK CORRECTION DATA
(WEIGHTS ARE PER POINT)

+#4 _____ %= _____ lb.

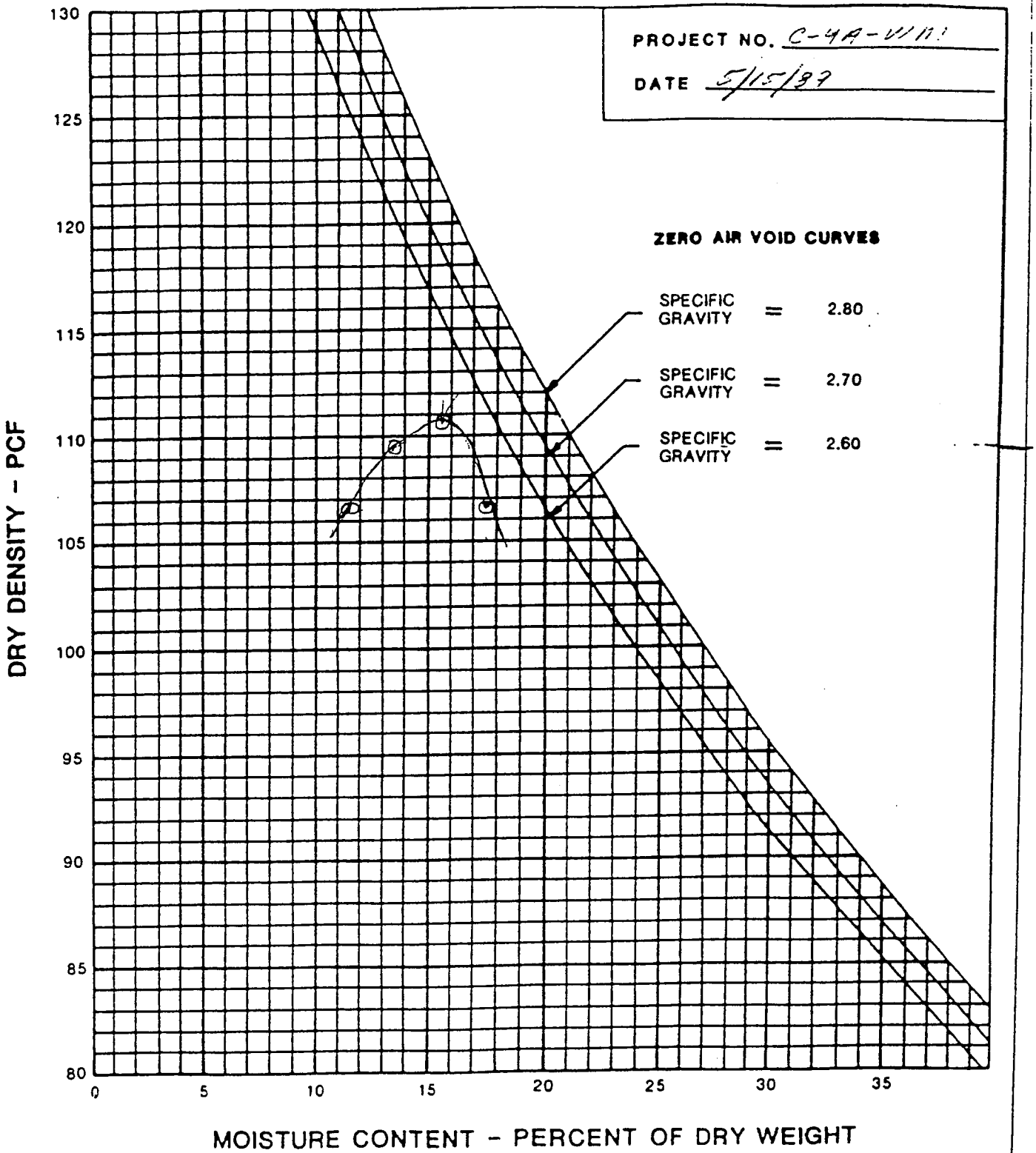
-#4 _____ %= _____ lb.

100%= _____ lb.

COMMENTS:

PROJECT NO. C-4A-VII.1

DATE 5/15/87



LOCATION : Sta 15+00 180' N of South Lake

SAMPLE NO:

MATERIAL DESCIPTION: Sandy Silty Clay

MAX. DRY DENSITY 110.7 PCF OPT. MOIST. CONTENT : 15.4 %

LIQUID LIMIT : 26 PLASTICITY INDEX : 8

GRAVEL : % SAND : % SILT AND CLAY (-200) : %

F-8 MOISTURE-DENSITY RELATIONSHIPS - 1

PROCEDURE: D698 A

TECHNICIAN: Tom Krake

APPROVED BY:

WORKSHEET

TECHNICIAN: Tom Rieke PROJECT NO: C-4A-WM

APPROVED BY: [signature] DATE 5/10/89

SAMPLE NO. Sta 15+00 180'N of South Dike

VISUAL DESCRIPTION: Silty Sandy Clay

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE					0	0
WT. OF PAN					0	0
DRY WT. RETAINED					0	0
DRY WT. PASSING						
% OF TOTAL PASSING					100	100
	W% = 5.6					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	959.2	909.3
RETAINED ON NO. 4		
PASSING NO. 4		

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	0 / 0.3	100 / 908.0	100 / 100
16	1.6	906.7	99.8
30 (40)	5.6 / 8.9	901.7 / 899.4	99.4 / 99
50	13.2	895.1	98.5
100	32.4	876.9	96.4
200	285.8	622.5	68.5 ✓
PAN			—
TOTAL			—

FACTOR = $\frac{W\%}{W}$ = _____ = _____

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH				
WT. DRY SOIL AND DISH				
WT. DISH				
WT. OF DRY SOIL		= W		
% MOISTURE				

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT _____

DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB. _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—		—	—	
	STOP MIX	—	—	—	—		—	—	
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

GRAVEL _____ % SAND _____ % CLAY-SLIT _____ %. STORAGE LOCATION _____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

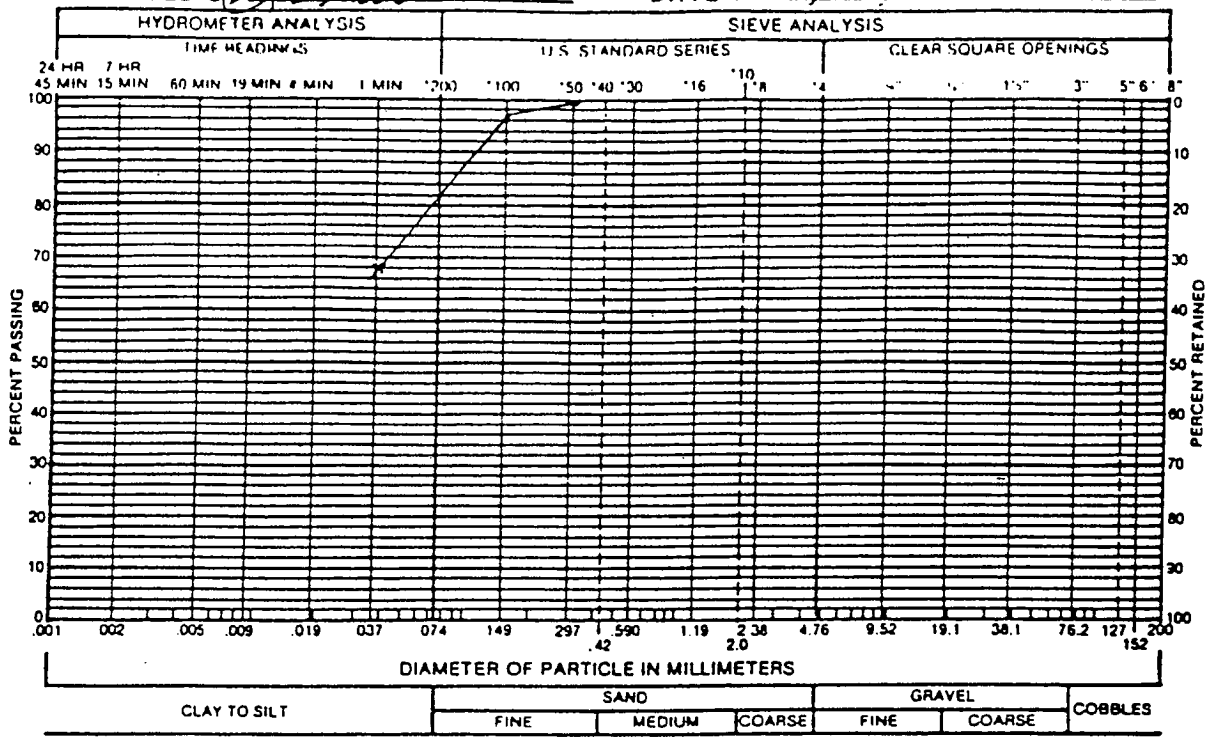
CA-2-79

F-12 GRADATION TEST RESULTS

MISS TLO
.200

TECHNICIAN Tom Krake PROJECT NO. C-4A-WM

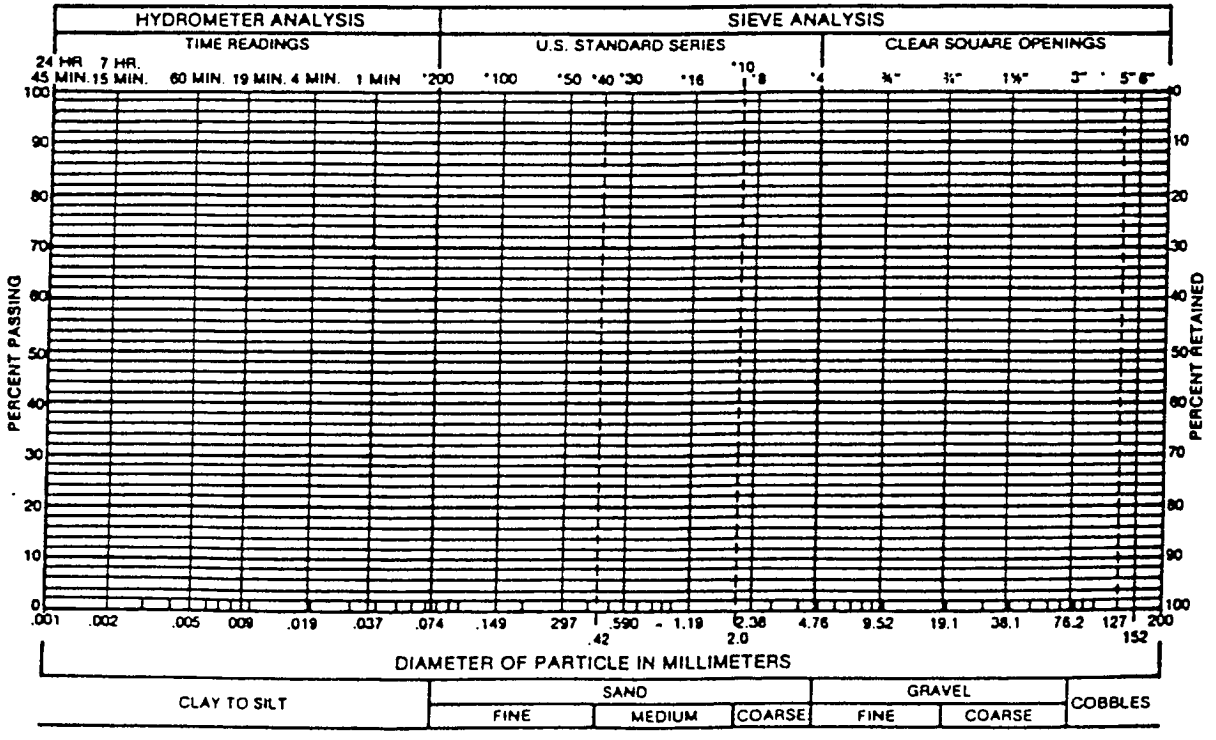
APPROVED BY [signature] DATE 5/15/79



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Silty Sandy Clay FROM Sta 15+00



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

OK

TECHNICIAN: Tom Kraake PROJECT NO. C-4A-WM
APPROVED BY: [signature] DATE: 5/10/89

SAMPLE NO. Sta 15+00 180' N of South Dike
SAMPLE DESCRIPTION Sand Silty Clay COLOR Lt Red

ATTERBERG LIMITS	PL	LL
PREP. DISH	RUN BY LW	
NO. OF BLOWS	—	26
DISH NO.	19	18
WT. OF WET SOIL & DISH	16.99	26.88
WT. OF DRY SOIL & DISH	16.04	23.80
WT. OF DISH	10.79	11.71
WT. OF WATER	0.95	3.08
WT. OF DRY SOIL	5.25	12.09
WATER CONTENT ? →		

-200	
RUN BY	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

LIQUID LIMIT, LL 26 ✓
PLASTIC INDEX, PI 8 ✓

PERCENT -200 _____ %

MOISTURE CONTENT	
RUN BY	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

DENSITY	
RUN BY	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

MOISTURE CONTENT _____ %

DRY DENSITY _____ PCF

MARKS:

SOIL/AGGREGATE - MOISTURE DENSITY RELATIONS

Job No. 3149J037

Lab./Invoice No. 31490318

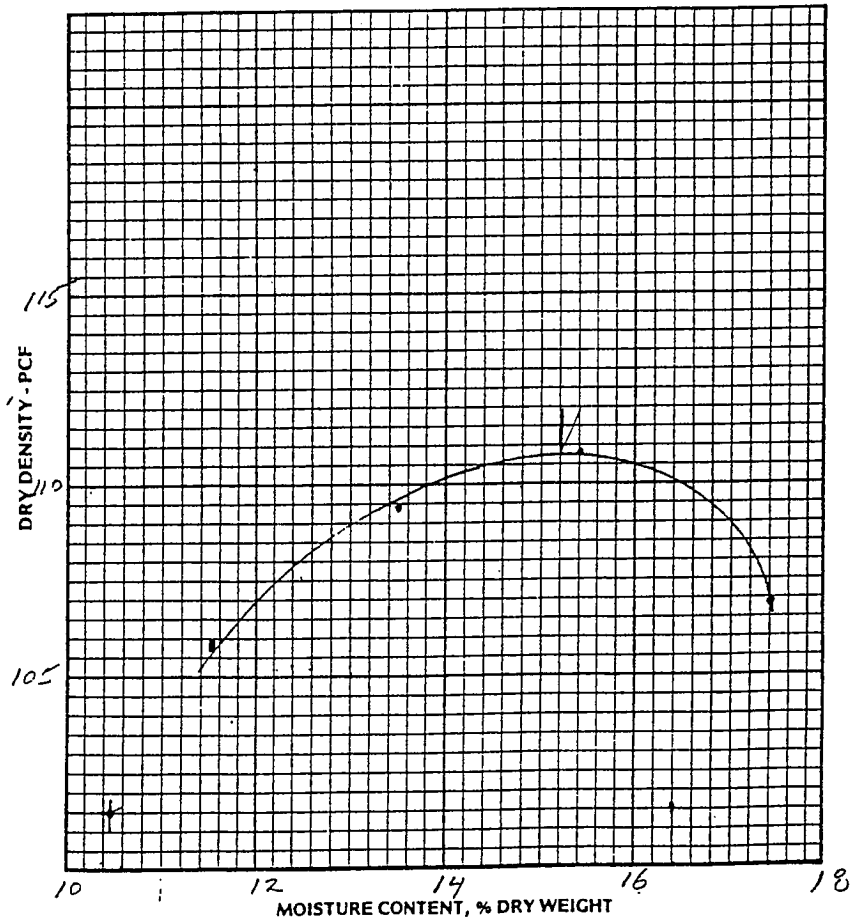
Type of Material _____ Sampled By TK Date 5/10/89

Source of Material Sta 15+00 180'N of S. Dike Submitted By ↓ Date ↓

Test Procedure 0698 A Tested/Calc. By C. Smalkanyen Date 5-15-89

_____ Reviewed By _____ Date _____

Trial No.	1	2	3	4	5	6	7
Water, estimated %							
Water, cc	150	[illegible]	[illegible]	[illegible]			
Wt. Sample + Mold	[illegible]	[illegible]	[illegible]	6,119.1			
Wt. Mold	[illegible]	[illegible]	[illegible]	[illegible]			
Wt. Wet Sample, gm	1799.5	1878.9	1931.3	1898.1			
Wt. Wet Sample, lbs.	3.97	4.14	4.26	4.18			
Wet Density, pcf	119.1	124.2	127.8	125.4			
Moisture Sample, wet	[illegible]	[illegible]	[illegible]	[illegible]			
Moisture Sample, Dry	213.	[illegible]	[illegible]	[illegible]			
Wt. Moisture	24.4	30.0	29.2	30.8			
Moisture, %	11.5	13.5	15.4	17.4			
Dry Density, pcf	106.8	109.4	110.7	106.8			



Max. Dry density, pcf 110.7

Optimum Moisture Content, % 15.2

Diameter of Mold, in. _____

Height of Mold, in. [illegible]

No. of Layers [illegible]

Blows per Layer [illegible]

Wt. of Hammer, lbs. [illegible]

Height of Drop 12

Material Used _____

WESTERN TECHNOLOGIES INC.

Type of Mate: 2-2L Sandy Silty Clay Job No. 3149J037

Source of Material Sta 15+00 Lab./Invoice No. 31490318

Sieve	Weight Retained	% Retained	% Pass Accum.	Specs.
4				
3				
2				
1½				
1½				
1				
¾				
½				
⅜	0	0	100	
¼.3				
#4	0	0	100	
Ret #4				Tech.
Wet		Wet Weight Before Wash	959.2	
Pass #4 Dry		Dry Weight Before Wash	908.3	
Total Dry		Weight After Wash		
Initial Total		Elutriation		
8	0	0	100	
10	0.3	0	100	
16	1.6	.2	99.8	
30	5.6	.6	99.4	
40	8.9	1.0	99	
50	13.2	1.5	98.5	
100	32.4	3.6	96.4	
200	285.8	32	68	
Finer Than 200				
Total				

Sampled By TK Date 5/10/87

Submitted By ↓ Date

Test/Calc. By Date

Reviewed By Date

Classification:

Test Procedure

☐ Aggregate

Sieve ASTM C136-

-200 ASTM C117-

☐ Soil

Sieve ASTM D422-

-200 ASTM D1140-

☐

☐

Special Instructions:

M/c: wet 244.4
231.5
5.6%

WESTERN TECHNOLOGIES INC.

ASTM D4318-

Type of Material [illegible] Job No. 3. 5037

Source of Material Sta. 15+00 180'W of S. Lk. Lab./Invoice No. 31490318

Sampled By TK Date 5/10/87

Submitted By ↓ Date ↓

Test/Calc. By Date

Reviewed By Date

LIQUID LIMIT

Taps	26	
Container Identification	18	
Wet Weight + Container	26.88	
Dry Weight + Container	− 23.80	
Weight of Water	= 3.08	
Dry Weight + Container	23.80	
Weight of Container	− 11.71	
Weight of Dry Soil	= 12.09	
Weight of Water / Weight of Dry Soil × 100 = Liquid Limit	= 25.5	
Liquid Limit at 25 Taps	= 26	

PLASTIC LIMIT

Container Identification	19	
Wet Weight + Container	− 16.99	
Dry Weight + Container	= 16.04	
Weight of Water	.95	
Dry Weight + Container	= 16.04	
Weight of Container	− 10.79	
Weight of Dry Soil	= 5.25	
Weight of Water / Weight of Dry Soil × 100 = Plastic Limit	= 18	

PLASTICITY INDEX ASTM D4318- = 8

WESTERN TECHNOLOGIES INC.

Clay Liner Construction

F-4 SOIL SAMPLING LOG

SAMPLE NO. 110

PROJECT NO. C-4A-WM

DATE 10/31/89

DELIVERED TO LABORATORY

SAMPLED BY H Kuebler, Q. C. L. Swanson

DATE 10/31/89

LOCATION Cell Bottom

(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1

SAMPLE TYPE Bulk

(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Sandy lean Clay

INTENDED USE Clay base material

(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, PI, Proctor

(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

SOIL/AGGREGATE - MOISTURE DENSITY RELATIONS

Job No. C-4A WM

Lab./Invoice No.

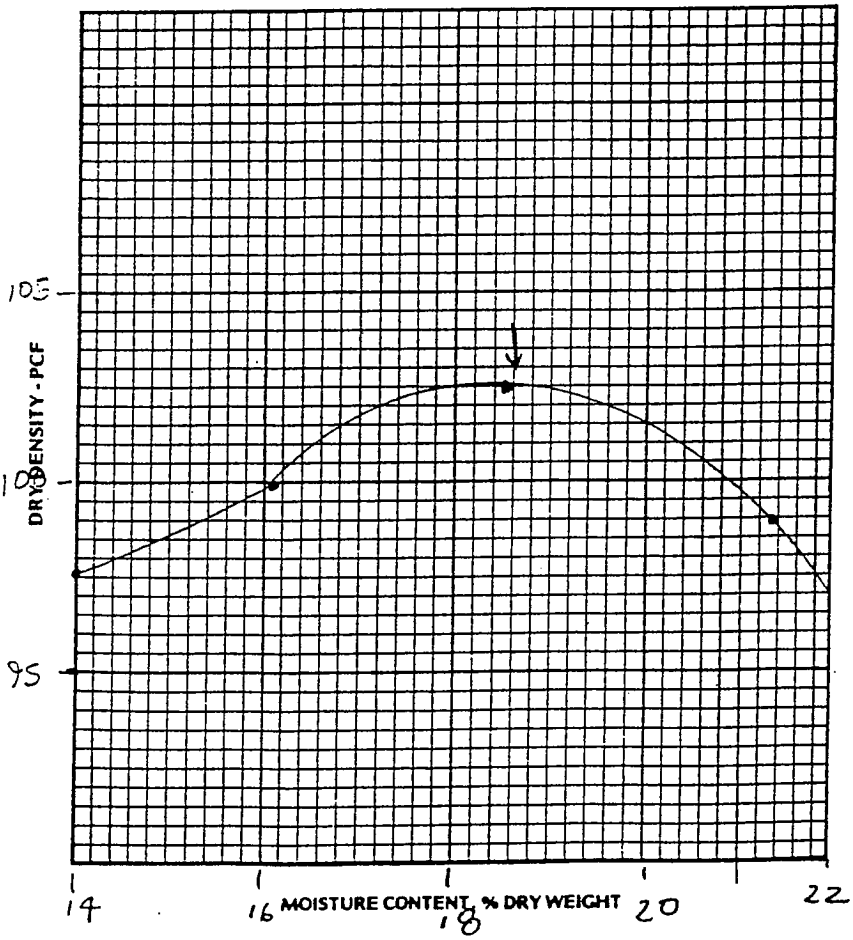
Type of Material Sandy Lean Clay — Sampled By H Kubler — Date 11-1-89

Source of Material Cell Bottom — Submitted By — Date

Test Procedure ASTM 698A — Tested/Calc. By — Date

Reviewed By L Swanson — Date 11/2/89

Trial No.	1	2	3	4	5	6	7
Water, estimated %							
Water, cc	300	250	350	200			
Wt. Sample + Mold	6066	5982	6044	5912			
Wt. Mold	4228	→	→				
Wt. Wet Sample, gm	1838	1754	1816	1684			
Wt. Wet Sample, lbs.	4.05	3.87	4.00	3.71			
Wet Density, pcf	121.5	116.1	120.0	111.3			
Moisture Sample, wet	273.5	343.7	288.1	412.0			
Moisture Sample, Dry	230.8	296.1	237.6	361.5			
Wt. Moisture	42.7	47.6	50.5	50.5			
Moisture, %	18.5	16.1	21.3	14.0			
Dry Density, pcf	102.5	100.0	98.9	97.6			



Max. Dry density, pcf 102.5

Optimum Moisture Content, % 18.6

Diameter of Mold, in. 4inch

Height of Mold, in. 4.584

No. of Layers 3

Blows per Layer 25

Wt. of Hammer, lbs. 5.5

Height of Drop 12inch

Material Used -4material

WESTERN TECHNOLOGIES INC.

WORKSHEET

TECHNICIAN: _____ PROJECT NO: C-4-A-WM

APPROVED BY: L. Swanson DATE: 11-1-89

SAMPLE NO. 110

VISUAL DESCRIPTION: Sandy lean Clay

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED				0	6.1	49.0
DRY WT. PASSING				804	797.9	755.0
% OF TOTAL PASSING				100	99	94
	W% = _____					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	904.5	804.0
RETAINED ON NO. 4	_____	_____
PASSING NO. 4	_____	_____

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	98.0 / 105.8	706 / 698.2	88 / 87
16	131.3	672.7	84
30 (40)	153.2 / 162.0	650.8 / 642	81 / 80
50	175.0	629.0	78
100	219.3	584.7	73
200	289.4	514.6	64
PAN			—
TOTAL			—

FACTOR = $\frac{W\%}{W}$ = _____ = _____

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH				
WT. DRY SOIL AND DISH				
WT. DISH				
WT. OF DRY SOIL		_____ = W		
% MOISTURE				

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT _____

DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—		—	—	
	STOP MIX	—	—	—	—		—	—	
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

GRAVEL _____ % SAND _____ % CLAY-SLIT _____ %. STORAGE LOCATION _____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

F-14 **ATTERBERG, -200, MOISTURE & DENSITY**
WORKSHEET

TECHNICIAN H Kubly PROJECT NO. C-4-AWM
APPROVED BY H Swanson DATE

SAMPLE NO. #110
SAMPLE DESCRIPTION Sandy lean Clay COLOR Brw

ATTERBERG LIMITS	PL	LL
PREP. DISH		RUN BY
NO. OF BLOWS	—	21
DISH NO.	17	18
WT. OF WET SOIL & DISH	15.73	27.35
WT. OF DRY SOIL & DISH	14.82	22.57
WT. OF DISH	10.78	11.76
WT. OF WATER	.91	4.78
WT. OF DRY SOIL	4.04	10.81
WATER CONTENT	23	44.2

LIQUID LIMIT, LL 43
PLASTIC INDEX, PI 20

-200	
RUN BY	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 _____ %

MOISTURE CONTENT	
RUN BY	
DISH NO.	
WT. OF DISH & WET SOIL	735.0
WT. OF DISH & DRY SOIL	652.5
WT. OF DISH	—
WT. OF WATER	82.5
WT. OF DRY SOIL	12.6

MOISTURE CONTENT _____ %

DENSITY	
RUN BY	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY _____ PCF

REMARKS: _____

Western
Appro
Thomas S. Krake

F-4 SOIL SAMPLING LOG

SAMPLE NO. 100

PROJECT NO. C-4A-WM

DATE 10/4/89

DELIVERED TO LABORATORY

SAMPLED BY H. Kuebler
Q.C. L.B. Swanson

DATE 10/4/89

LOCATION Stockpile #2
(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk
(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Lean clay w/shale

INTENDED USE Clay Base Material
(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, PI
(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

#10

SOIL/AGGREGATE - MOISTURE DENSITY RELATIONS

Job No. C4AWM

Lab./Invoice No.

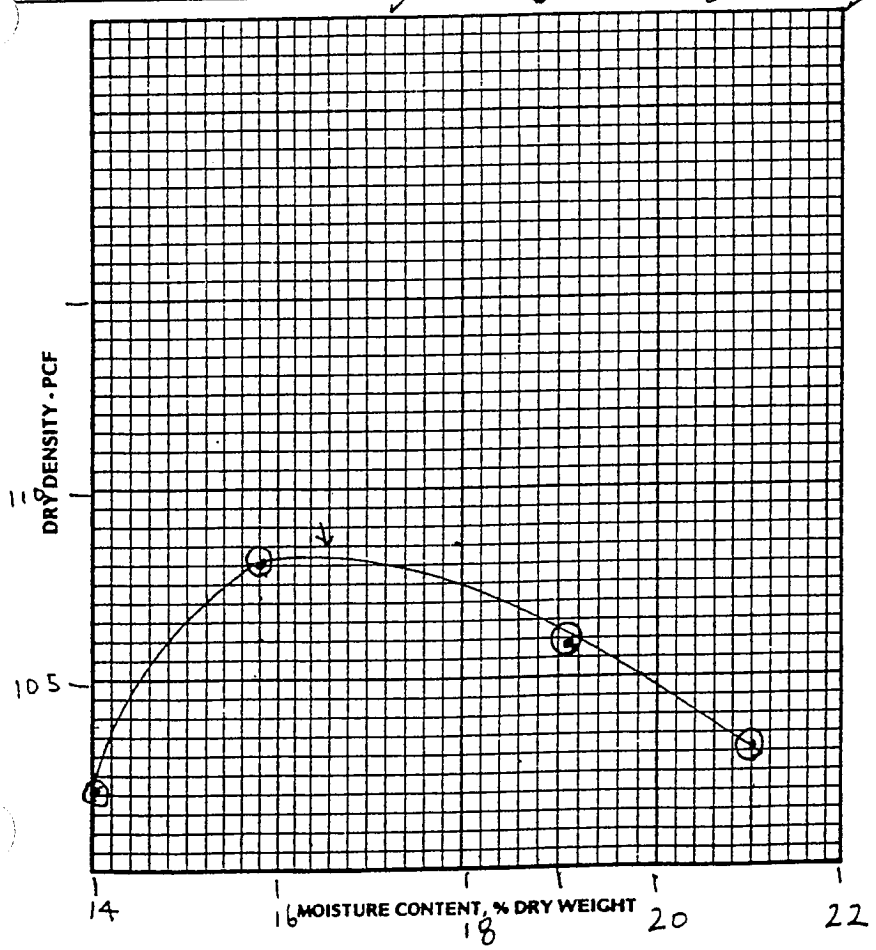
Type of Material Clay with shale — Sampled By H Kuebler — Date 10-4-89

Source of Material stockpile #2 — Submitted By — Date

Test Procedure ASTM 698 - A — Tested/Calc. By — Date 10-6-89

Reviewed By — Date 10/9/89

Trial No.	1	2	3	4	5	6	7
Water, estimated %							
Water, cc	250	300	200	150			
Wt. Sample + Mold	6134	6115	6122	5988			
Wt. Mold	4230	4230	4230	4230			
Wt. Wet Sample, gm	1904	1885	1892	1758			
Wt. Wet Sample, lbs.	4.20	4.16	4.17	3.88			
Wet Density, pcf	126.0	124.8	125.1	116.4			
Moisture Sample, wet	278.9	221.0	193.8	190.1			
Moisture Sample, Dry	234.1	182.5	167.4	166.8			
Wt. Moisture	44.8	38.5	26.4	23.3			
Moisture, %	19.1	21.0	15.8	14.0			
Dry Density, pcf	105.8	103.1	108.0	102.1			



Max. Dry density, pcf 108.3

Optimum Moisture Content, % 16.6

Diameter of Mold, in. 4 inch

Height of Mold, in. 4.584

No. of Layers 3

Blows per Layer 25

Wt. of Hammer, lbs. 5.5

Height of Drop 12

Material Used #4 material

Western Tech
Approved
Thomas S. Krake
WESTERN TECHNOLOGIES INC.

F-10 GRADATION ANALYSIS ...

WORKSHEET

TECHNICIAN: H. Kuebler
APPROVED BY: [signature]
PROJECT NO: C-4A-WM
DATE 10/4/89

SAMPLE NO. #100
VISUAL DESCRIPTION: Clay with Shale

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED					0	1.2
DRY WT. PASSING					549.3	548.1
% OF TOTAL PASSING					100	100
					W% =	

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	589.9	549.3
RETAINED ON NO. 4		
PASSING NO. 4		

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

FACTOR = W%/W = _____ = _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	10.0 / 15.6	539.3 / 533.1	98 / 98
16	45.8	503.5	92
30 (40)	90.2 / 112.2	459.1 / 437.1	84 / 80
50	138.4	410.9	75
100	210.0	339.3	62
200	289.1	260.2	47.
PAN			—
TOTAL			—

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH	720.0			
WT. DRY SOIL AND DISH	670.3			
WT. DISH	0.0			
WT. OF DRY SOIL	670.3	_____ = W		
% MOISTURE	7.4			

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT _____
DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—		—	—	
	STOP MIX	—	—	—	—		—	—	
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

GRAVEL _____ % SAND _____ % CLAY-SLIT _____ %. STORAGE LOCATION _____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

Western Tech
Approved
Thomas S. Krake

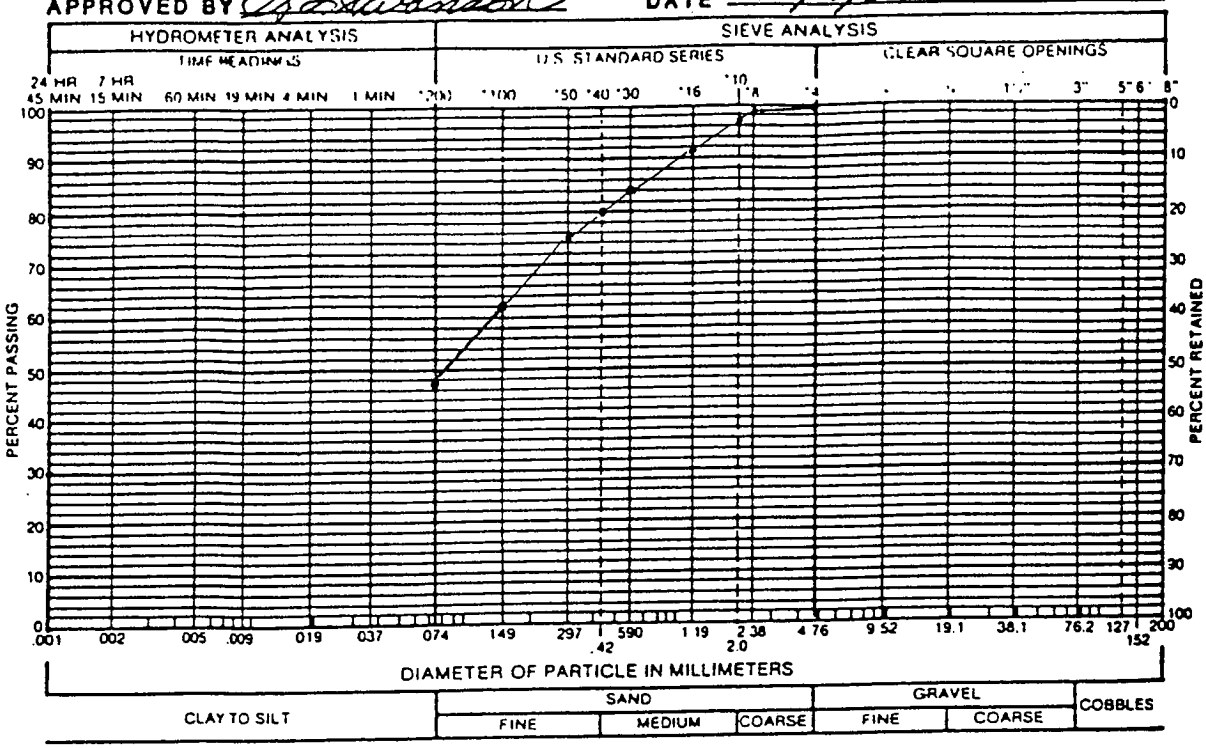
2-79

#100
-0X-XXI

F-12 GRADATION TEST RESULTS

TECHNICIAN H Kuetler PROJECT NO. C-4A-W10

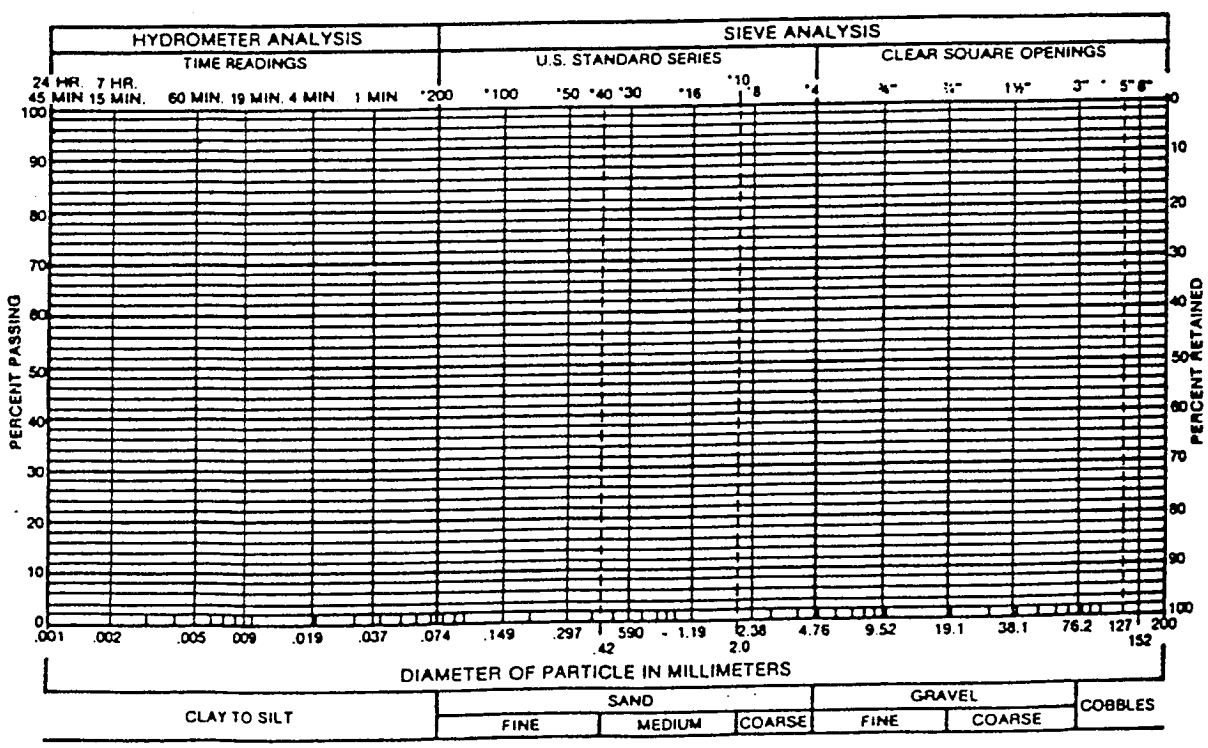
APPROVED BY [signature] DATE 10/4/89



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT 35 % PLASTICITY INDEX 13 %

SAMPLE OF FROM stockpile #2



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

Western Tech
Approved
Thomas S. Krake

#100
OK-XXX

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN: H Kuehler PROJECT NO. C-4A-WM
APPROVED BY: [signature] DATE 10/4/89

SAMPLE NO. 100
SAMPLE DESCRIPTION Lean Clay w/shale COLOR Gray-Green

ATTERBERG LIMITS	PL	LL
PREP. DISH ____ RUN BY ____		
NO. OF BLOWS	—	22
DISH NO.	17	18
WT. OF WET SOIL & DISH	12.73	24.0
WT. OF DRY SOIL & DISH	11.80	20.77
WT. OF DISH	7.60	11.73
WT. OF WATER	.93	3.23
WT. OF DRY SOIL	4.20	9.04
WATER CONTENT	22	35.7

35 LIQUID LIMIT, LL 35
PLASTIC INDEX, PI 13

-200	
RUN BY ____	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 ____ %

MOISTURE CONTENT	
RUN BY ____	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

MOISTURE CONTENT ____ %

DENSITY	
RUN BY ____	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY ____ PCF

ARKS:

Western Tech
Approved
Thomas S. Krake

F-4 SOIL SAMPLING LOG

SAMPLE NO. 96

PROJECT NO. C-449-W177

DATE 9/14/89

DELIVERED TO LABORATORY

SAMPLED BY T Krake
Q.C. G.D. Swanson

DATE 9/14/89

LOCATION Stockpile

(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk

(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Lean Clay

INTENDED USE Clay Base

(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, PI

(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

F-10 GRADATION ANALYSIS WITH HYDROMETER

WORKSHEET

TECHNICIAN: T. Krake
PROJECT NO. C-4A-WRA
APPROVED BY: [signature]
DATE 9/14/87

SAMPLE NO. 96
VISUAL DESCRIPTION: Lean Clay

SAMPLE PREPERATION

RUN BY _____ SIEVING TIME _____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED					0	26.5
DRY WT. PASSING					883.0	856.5
% OF TOTAL PASSING					100	97
					W% =	

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	965.4	883.0
RETAINED ON NO. 4		
PASSING NO. 4		

SIEVE AND HYDROMETER ANALYSIS

RUN BY _____ SIEVING TIME _____

FACTOR = W%/W = _____ = _____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	35.3 / 53.0	847.7 / 830.0	96 / 94
16	83.3	794.7	90
30 (40)	123.6 / 132.4	759.4 / 750.6	86 / 85
50	229.6	653.4	74
100	264.9	618.1	70
200	335.5	547.5	62
PAN			—
TOTAL			—

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH	492.6			
WT. DRY SOIL AND DISH	451.5			
WT. DISH	0.0			
WT. OF DRY SOIL	451.5	= W		
% MOISTURE	9.1			

HYDROMETER ANALYSIS

RUN BY _____

CYLINDER NO. _____ SPECIFIC GRAVITY _____ DISPERSING AGENT _____
DISH NO. _____ DATE _____ AMOUNT _____ ml DATE CALIB. _____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—		—	—	
	STOP MIX	—	—	—	—		—	—	
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

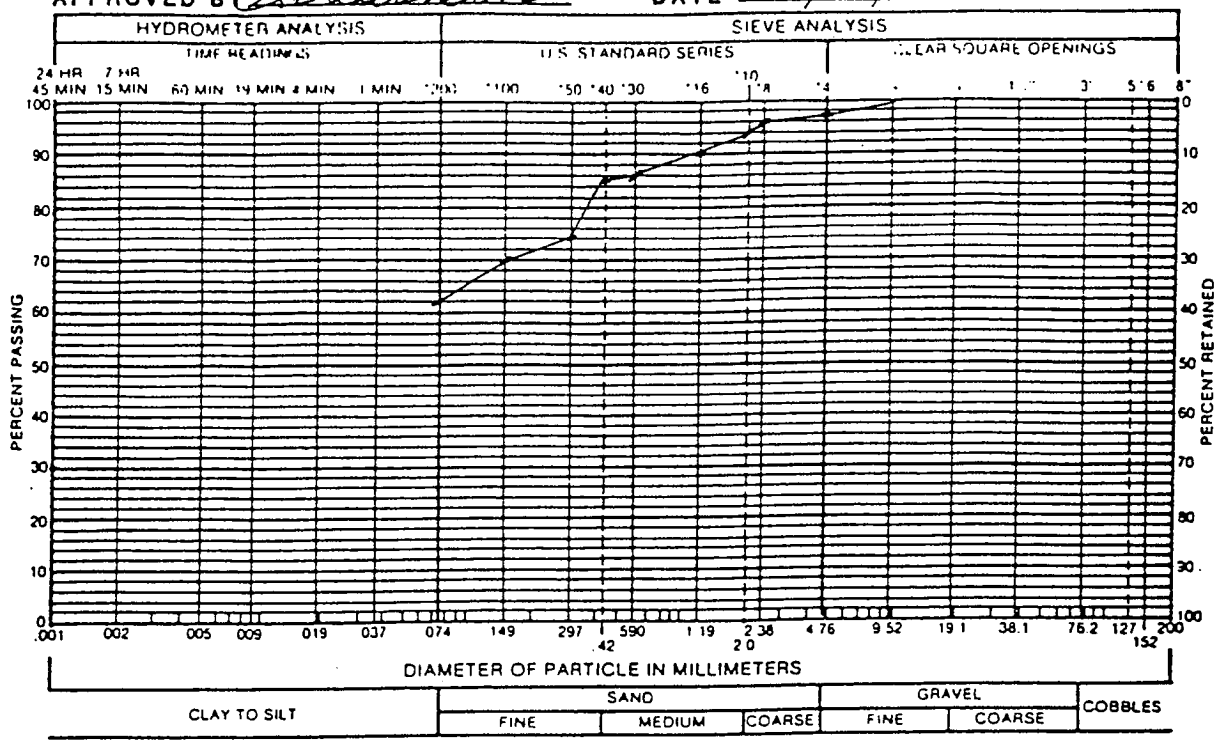
GRAVEL _____ % SAND _____ % CLAY-SLIT _____ % STORAGE LOCATION _____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

F-12 GRADATION TEST RESULTS

TECHNICIAN T. Krajko PROJECT NO. C-48-WM

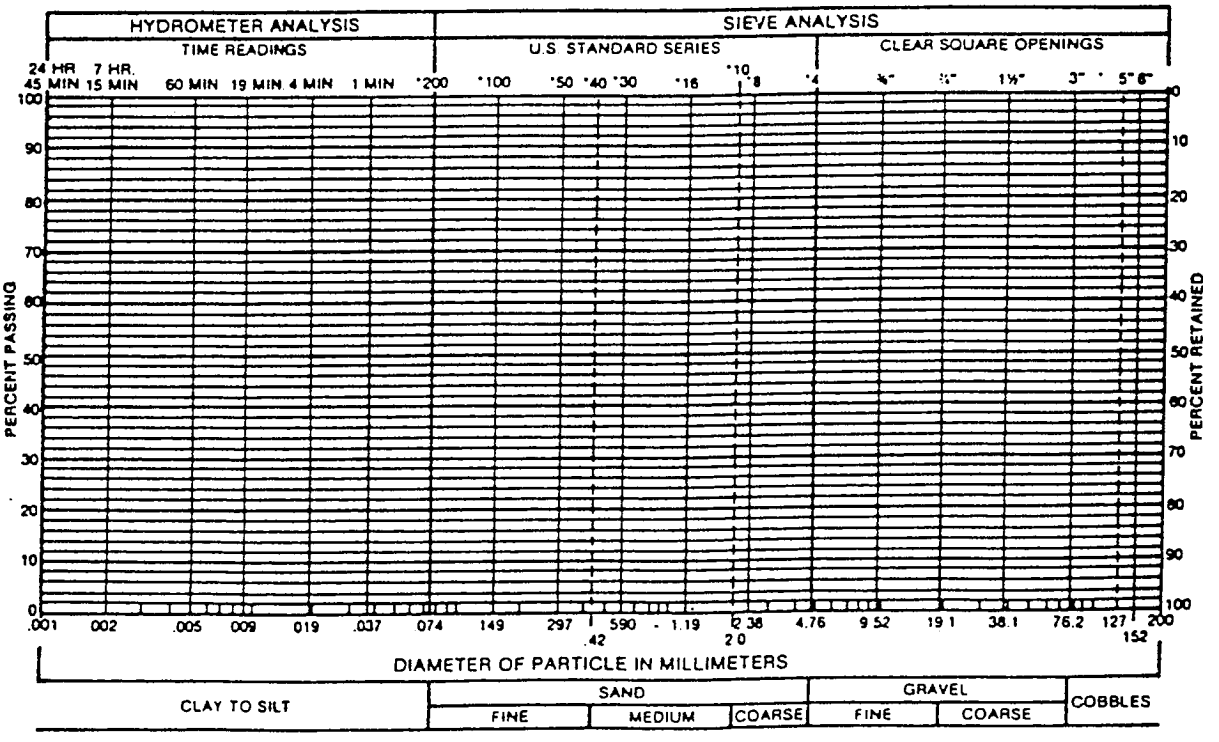
APPROVED BY [signature] DATE 9/14/89



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Lean Clay FROM Stockpile



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN T. Krak PROJECT NO. C-4A-WM

APPROVED BY [signature] DATE 9/14/89

SAMPLE NO. 96

SAMPLE DESCRIPTION Lean Clay COLOR Green

ATTERBERG LIMITS	PL	LL
PREP. DISH	RUN BY	
NO. OF BLOWS	—	24
DISH NO.	C	E
WT. OF WET SOIL & DISH	15.03	24.14
WT. OF DRY SOIL & DISH	13.96	20.44
WT. OF DISH	5.67	8.43
WT. OF WATER	1.07 ✓	3.70 ✓
WT. OF DRY SOIL	8.29 ✓	12.01 ✓
WATER CONTENT	12.9 ✓	30.8 ✓

LIQUID LIMIT, LL 31 ✓

PLASTIC INDEX, PI 18 ✓ OK

-200	
RUN BY	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 _____ %

MOISTURE CONTENT	
RUN BY	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

MOISTURE CONTENT _____ %

DENSITY	
RUN BY	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY _____ PCF

REMARKS: _____

F-4 SOIL SAMPLING LOG

SAMPLE NO. 90

PROJECT NO. C-4A-WM

DATE 9/7/89

DELIVERED TO LABORATORY

SAMPLED BY T Krake

DATE 9/7/89

Q.C. S. Swanson

LOCATION Stockpile

(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk

(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Lean Clay

INTENDED USE Clayey Liner Material

(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, PI

(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

WORKSHEET

TECHNICIAN: T. Krake — PROJECT NO: C-4A-WM

APPROVED BY: [illegible] Swanson — DATE 9/7/89

SAMPLE NO. 90

VISUAL DESCRIPTION: Lean Clay

SAMPLE PREPERATION

RUN BY ____ SIEVING TIME ____

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED					0	1.5
DRY WT. PASSING					796.2	794.7
% OF TOTAL PASSING					100	100
	W% = ____					

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	817.7	796.2
RETAINED ON NO. 4	____	____
PASSING NO. 4	____	____

SIEVE AND HYDROMETER ANALYSIS

RUN BY ____ SIEVING TIME ____

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	17.5 / 19.6	778.7 / 776.6	98 / 98
16	22.1	774.1	97
30 (40)	26.2 / 28.9	770.0 / 767.3	97 / 96
50	35.9	760.3	95
100	75.0	721.2	91
200	142.3	653.9	82
PAN			—
TOTAL			—

FACTOR = W% / W = ____ = ____

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH	457.4			
WT. DRY SOIL AND DISH	445.2			
WT. DISH				
WT. OF DRY SOIL		= W		
% MOISTURE	2.7			

HYDROMETER ANALYSIS

RUN BY ____

CYLINDER NO. ____ SPECIFIC GRAVITY ____ DISPERSING AGENT: ____

DISH NO. ____ DATE ____ AMOUNT ____ ml DATE CALIB. ____

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—		—	—	
	STOP MIX	—	—	—	—		—	—	
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

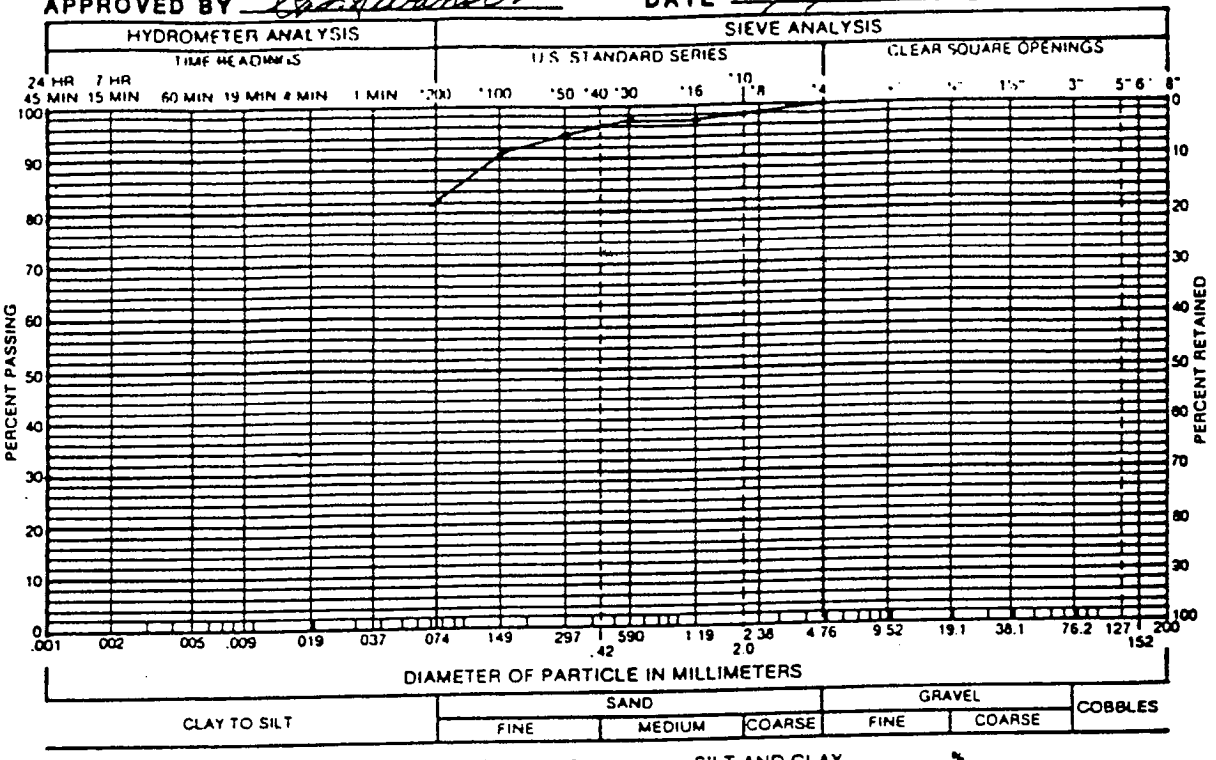
GRAVEL ____ % SAND ____ % CLAY-SLIT ____ %. STORAGE LOCATION ____

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

F-12 GRADATION TEST RESULTS

TECHNICIAN T. Brake PROJECT NO. C-4A-WM

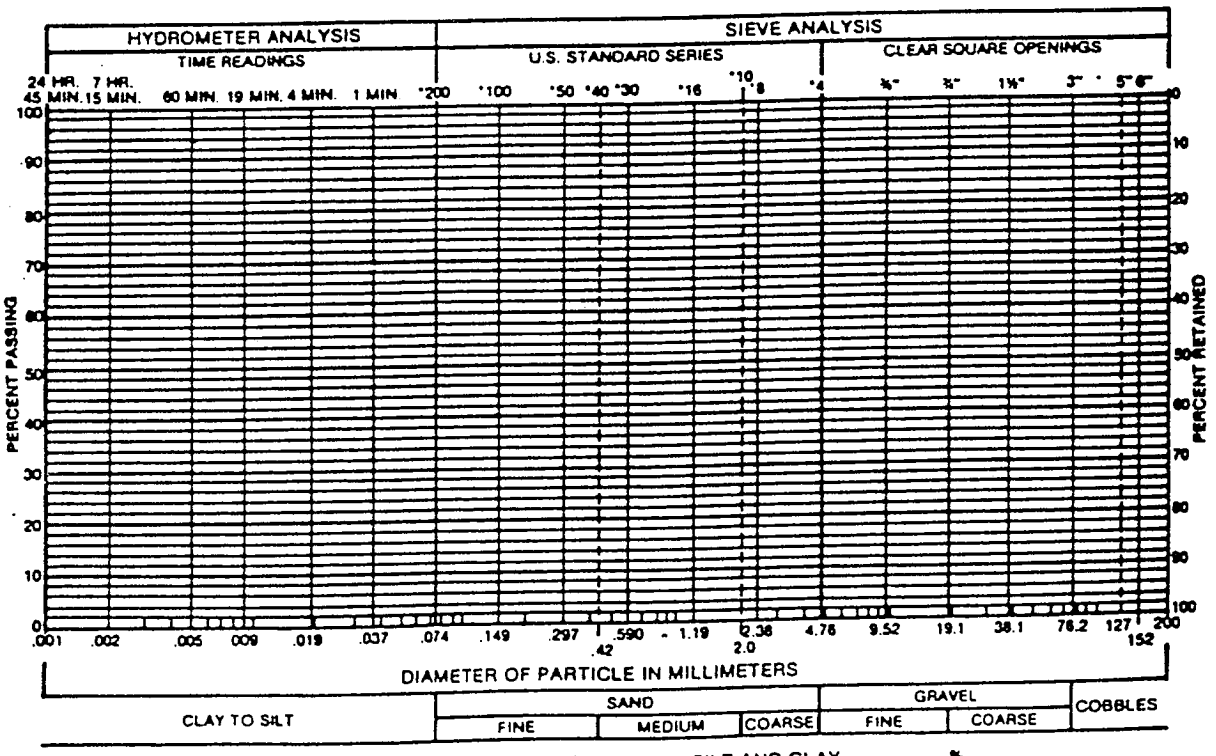
APPROVED BY S. Swanson DATE 9/7/88



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Lean Clay FROM Stockpile



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN J Krake / PROJECT NO. C-4A-WM

APPROVED BY [illegible] DATE 9/2/89

SAMPLE NO. 20

SAMPLE DESCRIPTION Lean Clay COLOR Green

ATTERBERG LIMITS	PL	LL
PREP. DISH ____ RUN BY ____		
NO. OF BLOWS	____	20
DISH NO.	J	19
WT. OF WET SOIL & DISH	8.39	35.02
WT. OF DRY SOIL & DISH	7.36	28.65
WT. OF DISH	2.21	10.86
WT. OF WATER	1.03 ✓	6.37 ✓
WT. OF DRY SOIL	5.15 ✓	17.79 ✓
WATER CONTENT	20.0 ✓	35.8 ✓

LIQUID LIMIT, LL 35 ✓

PLASTIC INDEX, PI 15 ✓

-200	
RUN BY ____	
DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 ____ %

MOISTURE CONTENT	
RUN BY ____	
DISH NO.	
WT. OF DISH & WET SOIL	
WT. OF DISH & DRY SOIL	
WT. OF DISH	
WT. OF WATER	
WT. OF DRY SOIL	

MOISTURE CONTENT ____ %

DENSITY	
RUN BY ____	
LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY ____ PCF

REMARKS: ____

Appendix E

Design Calculations

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: International Uranium Corporation **Project:** White Mesa Mill, Pond 4A
Project/Proposal #: SC 0349 **Task #:** 1

Title of Computations: Comparison of Flow Through Compacted Clay Liner and Geosynthetic Clay Liner

Computations By:
SIGNATURE — DATE: 1/25/06
Gregory T. Corcoran, Associate
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE — DATE: 1/25/06
Steven M. Fitzwilliam, Sr. Project Engineer
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE — DATE: 1/25/06
Steven M. Fitzwilliam, Sr. Project Engineer
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):
SIGNATURE — DATE: 1/25/06
Gregory T. Corcoran, Associate
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE — DATE: 1/25/06
Gregory T. Corcoran, Associate
PRINTED NAME AND TITLE

Approval Notes: _____

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Greg Corcoran Date: 24 January 2006 Reviewed by: SF Date: 1/25/06

Client: International Uranium (USA) Corporation Project: Cell 4A Project No.: SC0349 Task No.: 01

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

OBJECTIVE

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 x 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

Written by: Greg Corcoran Date: 24 January 2006 Reviewed by: SF Date: 1/25/06

Client: International Uranium (USA) Corporation Project: Cell 4A Project No.: SC0349 Task No.: 01

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:

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

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 4A (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:

Written by: Greg Corcoran Date: 24 January 2006 Reviewed by: SF Date: 1/25/06

Client: International Uranium (USA) Corporation Project: Cell 4A Project No.: SC0349 Task No.: 01

$Q = (2/3)(0.0015m)^2\sqrt{(9.81)(11.3m)}$ $= 1.58 \times 10^{-5}$ 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, 1.58×10^{-5} m^3/sec
k = permeability of geonet layer above secondary geomembrane

Attachment D, 2/3 shows a transmissivity curve for a 200 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 ft + 950 ft x 0.01) / 950 ft = 0.049

Graphing the permeability data for the 200 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 x 10^{-1} m/sec.

Written by: Greg Corcoran Date: 24 January 2006 Reviewed by: SF Date: 1/25/06

Client: International Uranium (USA) Corporation Project: Cell 4A Project No.: SC0349 Task No.: 01

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:

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

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.

Written by: Greg Corcoran Date: 24 January 2006 Reviewed by: SF Date: 1/25/06

Client: International Uranium (USA) Corporation Project: Cell 4A Project No.: SC0349 Task No.: 01

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:

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

where: $q_{comp\ CCL}$ = unit rate of flow through a composite liner where the soil component is a CCL; $q_{comp\ GCL}$ = unit rate of flow through a composite liner where the soil component is a GCL; t_{CCL} = thickness of the CCL in the composite liner; t_{GCL} = thickness of the GCL

in the composite liner;
simplification, Equatio

$$\frac{q_c}{q_c}$$

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_{GCL}	(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 values
following CCL characteristics
is the standard CCL defined ir

ATTACHMENT A 1/2

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$$k^{0.74} \qquad (51)$$

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will be considered in Equa-

e flow through a composite
follows:

$$\frac{0.1\ h^{0.9}\ k_{CCL}^{0.74}}{0.1\ h^{0.9}\ k_{GCL}^{0.74}} \qquad (52)$$

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

part, which is unsaturated. Then, when the leachate reaches the saturated portion of the leakage collection layer, it first flows in all directions (Figure 1). It is therefore logical to assume that the leachate phreatic surface in the leakage collection layer is a cone with its apex at Point A located vertically beneath the defect in the primary liner (Figure 4). Furthermore, for leachate to flow in all directions, the hydraulic gradient must be

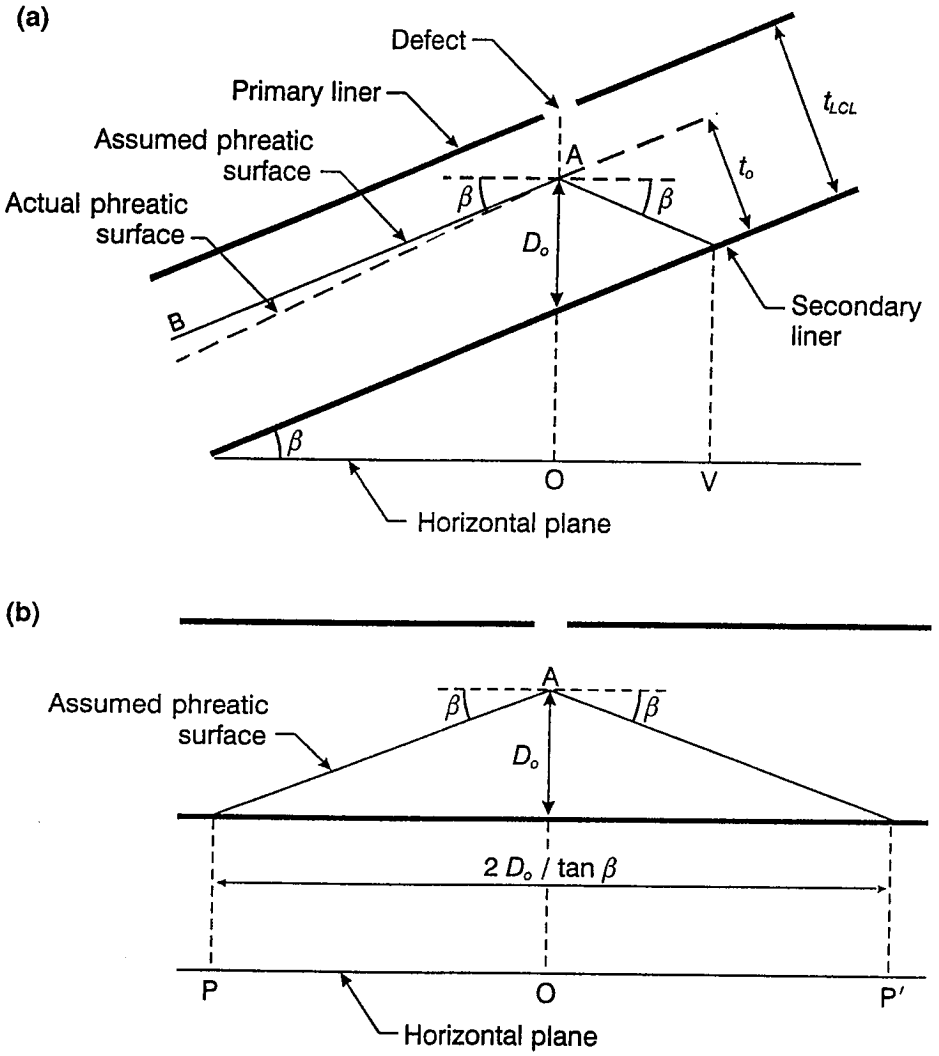

Figure 4. Assumed phreatic surface in the leakage collection layer in the case where the leakage collection layer is not filled with leachate: (a) cross section in a vertical plane along the slope and passing through the defect in the primary liner; (b) cross section in a vertical plane perpendicular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

ATTACHMENT B, 2/5

It appears that, when the leakage collection layer is not full, there is an extremely simple relationship between the rate of leachate migration through the primary liner defect, Q, and the thickness of leachate in the leakage collection layer beneath the defect, t_o. It is interesting to note that this relationship does not depend on the size of the defect in the primary liner or on the slope of the leakage collection layer.

An approximation that was made to establish Equations 9 and 10 was to assume that the downslope flow line from A (i.e. AB in Figure 4a) is parallel to the liner. This assumption is close to reality as discussed in Section 2.2. However, the actual flow line from A is below Line AB as the flow thickness decreases in the downslope direction, as discussed at the end of Section 5.1.2. Therefore, t_o should only be regarded as the flow thickness at a primary liner defect, and it is the maximum flow thickness.

Since the simple relationship expressed by Equations 9 and 10 was demonstrated for the case when the leakage collection layer is not full, the condition expressed by Equation 1 must be met for Equations 9 and 10 to be valid. Combining Equations 1 and 10 gives the following equation, which is another way to express the condition that should be met to ensure that the leakage collection layer is not full:

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

where $t_{LCLfull}$ is the *minimum* thickness that a leakage collection layer with a hydraulic conductivity k should have to contain, without being full at any location, the leachate flow which results from a defect in the primary liner.

The following equation, derived from Equation 11, is another way to express the condition that should be met to ensure that the leakage collection layer is not full:

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

where Q_{full} is the *maximum* steady-state rate of leachate migration through a defect in the primary liner that a leakage collection layer, with a thickness t_{LCL} and a hydraulic conductivity k, can accommodate without being filled with leachate.

It is important to remember that the subscript *full* corresponds to a *minimum* thickness of the leakage collection layer and to a *maximum* rate of leachate migration (which is also the *maximum* flow rate in the leakage collection layer). It is noteworthy that the minimum thickness of the leakage collection layer, $t_{LCLfull}$, and the maximum flow rate, Q_{full}, which are required to ensure that the leakage collection layer can contain, without being full, the flow that results from a defect in the primary liner, do not depend on the slope of the leakage collection layer.

It is not impossible to design a leakage collection layer with a thickness less than the value $t_{LCLfull}$ given by Equation 11, i.e. where the flow rate is greater than Q_{full} defined by Equation 12. In this case, the leakage collection layer is filled with leachate in a certain area around the defect of the primary liner (i.e. "the leachate collection layer is full"). This case is discussed in Section 3.2.

3.2 Rate of Leachat

If the thickness of the l
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Q_{full} expressed by Equati
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leakage collection layer i
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The surface area of the v
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$$S = \frac{D_o^2}{\tan}$$

where D_{LCL} is the depth of
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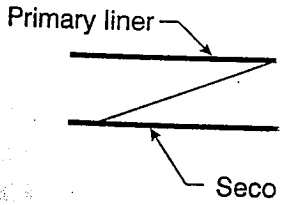

**Figure 5. Vertical cross sect
layer in the case where the le
around the primary liner defe**

ATTACHMENT B 3/5

Knowing (or assuming) the leachate head, h_o, on top of the secondary liner vertically beneath the primary liner defect, one may derive the virtual leachate thickness, t_o, using Equation 4. Then, knowing t_o, t_{LCL} and k, one may use Equation 16 to calculate the rate of leachate flow through a defect that the leakage collection layer can convey.

The following equation can be derived from Equation 16:

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

The following equation can be derived from Equations 13 and 16:

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

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

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

It should be noted that if $t_{LCL} = t_o$, i.e. if the leakage collection layer is filled with leachate at only one point, i.e. at the location of the primary liner defect, Equation 16 is equivalent to Equation 9.

3.3 Parametric Study

Using the equations presented in Sections 3.1 and 3.2 it is possible to compare the flow capacity of different leakage collection layers in case of a defect in the primary liner. In Table 1, three different leakage collection layers are compared:

- a geonet with a thickness of 5 mm and a hydraulic transmissivity resulting in a hydraulic conductivity (obtained by dividing the hydraulic transmissivity by the thickness) of 1×10^{-1} m/s;
- a gravel layer with a thickness of 300 mm and a hydraulic conductivity of 1×10^{-1} m/s; and
- a sand layer with a thickness of 300 mm and a hydraulic conductivity of 1×10^{-3} m/s.

The first two leakage collection layers have the same hydraulic conductivity and the last two have the same thickness. In the case of the geonet, the virtual leachate thickness, t_o, considered in Table 1 is greater than, or equal to, the thickness of the leachate collection layer, t_{LCL}; therefore, in all cases considered in Table 1, the geonet is filled with leachate over a certain area around the defect (this area being zero for t_o = 5 mm). In the case of the gravel and sand layers, the leachate thicknesses considered in Table 1 are less than, or equal to, the thickness of the leakage collection layer; therefore, in all cases considered in Table 1, the gravel and sand layer are not filled (or just filled) with leachate, and for these two materials the leachate thicknesses, t_o, shown in Table 1 are actual (not virtual) thicknesses.

Table 1. Rate of leachat defect in the primary line

Leachate thickness (actual or virtual)		
t_o (m)	(mm)	
0.005	5	2
0.01	10	7
0.05	50	4.
0.1	100	9.
0.3	300	2.9

Notes: The leachate thickness 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 se can convey significantly flow rates of Table 1 with used alone (i.e. not part which is expressed as fo

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

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

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

- a small geomembrane undetected during cons per day (lpd);
- a geomembrane defect ing construction phase of granular leachate col on the order of 1000 lp
- a large geomembrane c special circumstances,

ATTACHMENT B, 4/5

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Table 1. Rate of leachate flow in three different leachate collection layers resulting from a defect in the primary liner.

Leachate thickness (actual or virtual)		Leakage collection layer material					
		Geonet t_{LCL} = 5 mm $k = 1 \times 10^{-1}$ m/s		Gravel t_{LCL} = 300 mm $k = 1 \times 10^{-1}$ m/s		Sand t_{LCL} = 300 mm $k = 1 \times 10^{-3}$ m/s	
t_o		Q		Q		Q	
(m)	(mm)	(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:

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

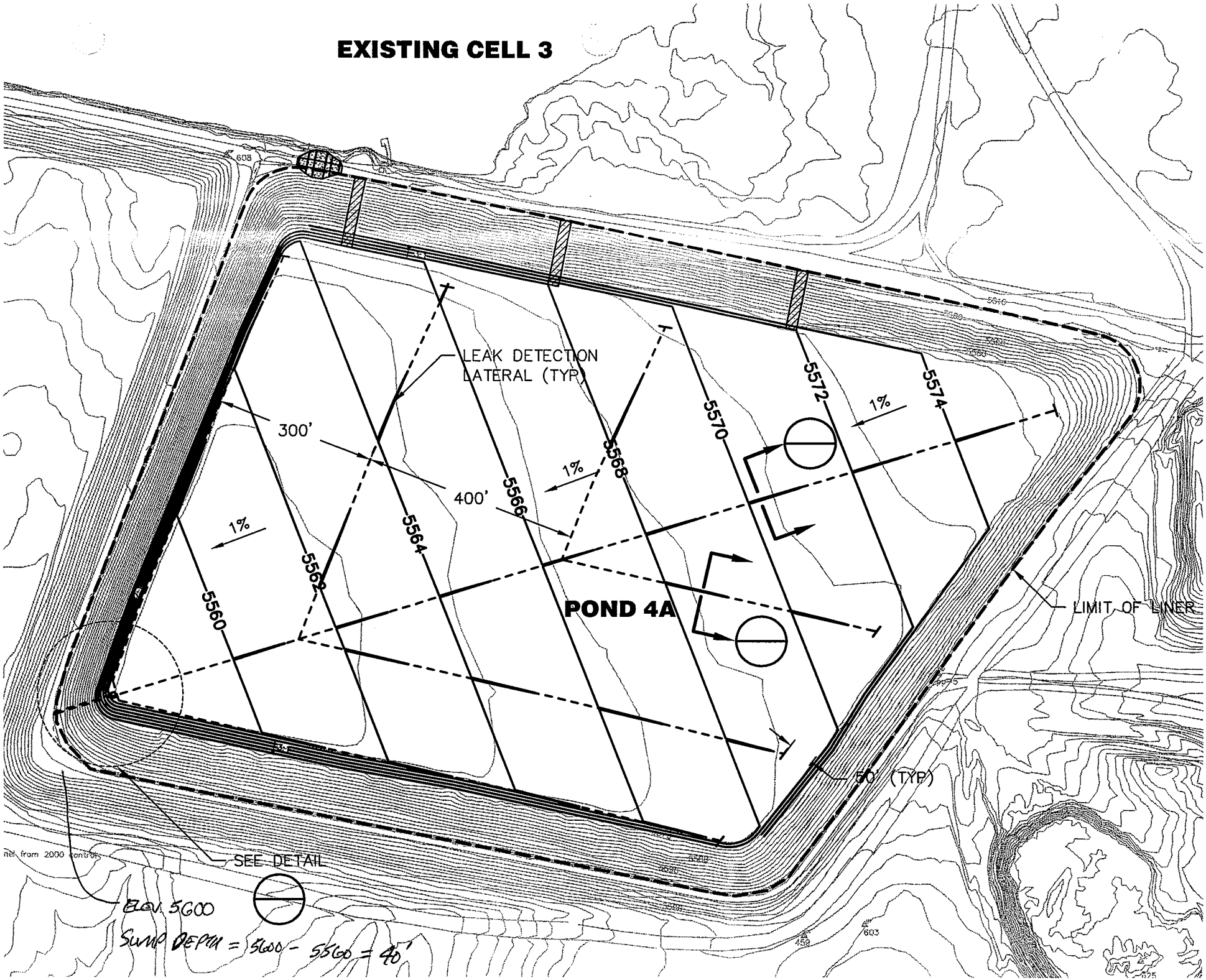
where: a = defect area; d = defect diameter; g = acceleration due to gravity; and h_{prim} = head of leachate on top of the primary liner.

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

EXISTING CELL 3



Attachment C 1/

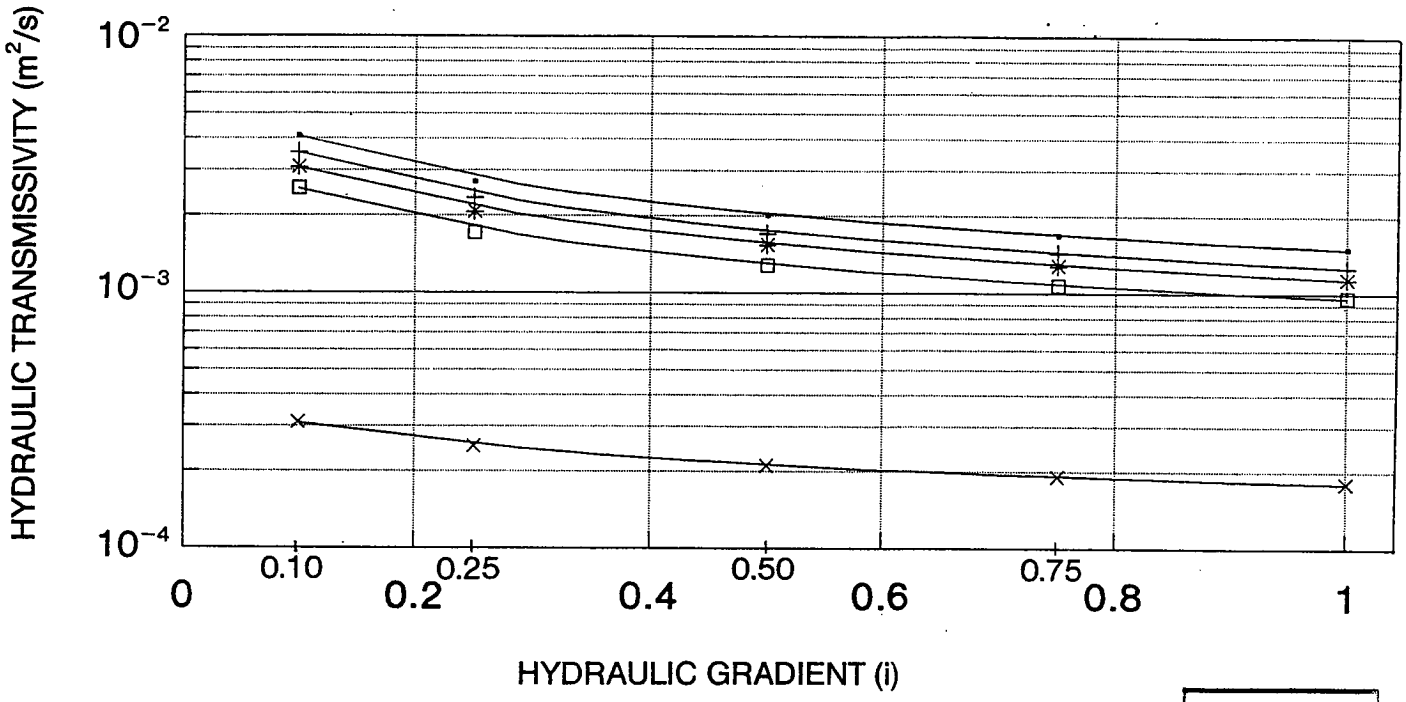
FLUID SYSTEMS, INC.

POLY-NET® TRANSMISSIVITY CHARTS

ATTACHMENT D 1/3

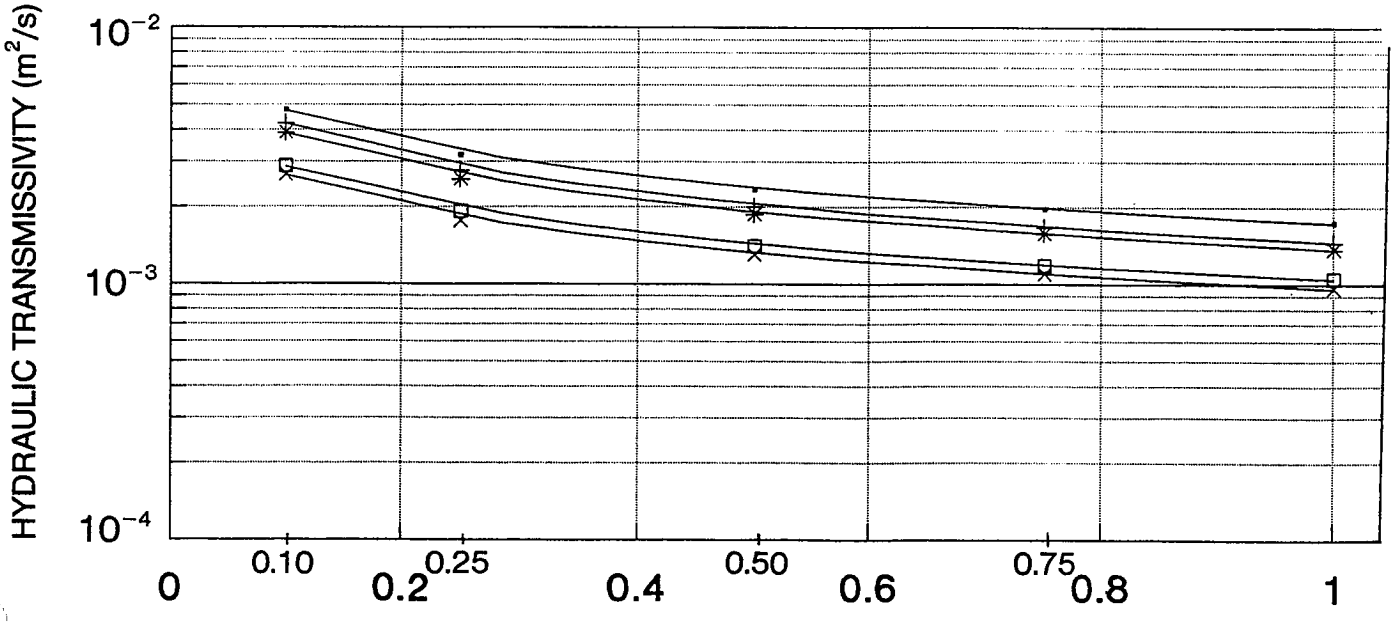
POLY-NET PN2000

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
$1 m^2/s = 10.76$ ft²/second
$1 m^2/s = 646$ ft²/minute
$1 m^2/s = 4830$ gallons/minute/foot width/unit gradient

1 ft²/second = 1 cubic foot/second/foot width/unit gradient
1 ft²/second = $9.3 \times 10^{-2} m^2/s$
1 ft²/minute = 1 cubic foot/minute/foot width/unit gradient
1 ft²/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 ft²/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.

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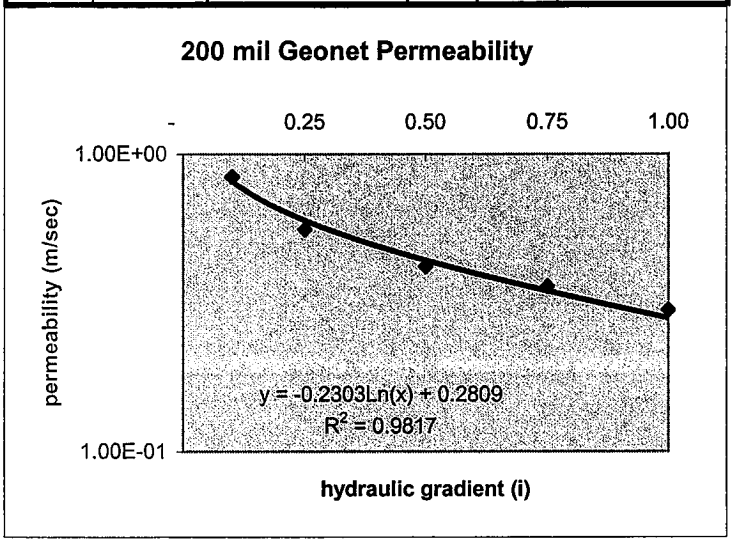
32 Triangle Park Drive, Suite 3201
Cincinnati, OH 45246
Phone: 513/771-5656
800/346-9107
Fax: 513/771-4844

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

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: IUC **Project:** Tailings Cell 4A **Project/Proposal #:** SC0349 **Task #:** 03

Title of Computations: Cushion Fabric Calculations

Computations By:
SIGNATURE — 05/11/08 DATE
Jennifer Ferguson, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE — 11/23/05 DATE
GREG CORCORAN - ASSOCIATE
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE — 11/23/05 DATE
GREG CORCORAN - ASSOCIATE
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Computations Backchecked By (Originator):
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Jennifer Ferguson, Staff Engineer
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Approved By (PM or Designate):
SIGNATURE — 11/23/05 DATE
GREG CORCORAN - ASSOCIATE
PRINTED NAME AND TITLE

Approval Notes: _____

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (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

CUSHION FABRIC CALCULATIONS
WHITE MESA MILL
BLANDING, UTAH

OBJECTIVE

The project involves placement of a double composite liner system for the base of Cell 4A 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

Written by: Jennifer Ferguson Date: 05/10/27 (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

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 unit weight of 120 pounds per cubic foot (pcf) (Attachment B). For conservatism, we have assumed that a maximum of 45 ft of tailing deposits may be present. Therefore, the design overburden pressure is 45 ft × 120 pcf = 5,400 pounds per square foot (psf) or 258.5 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

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (YY MM DD) Reviewed by: GC Date: 05 / 11 / 23 (YY MM DD)

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

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 C) present the following equation for evaluating geotextile puncture protection of 60 mil (1.5 mm) HDPE geomembrane:

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

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)

P_{allow} = maximum long term allowable pressure

where: $$P_{allow} = P'_{allow}\left(MF_S \times MF_{PD} \times MF_A\right)\left(FS_{CR} \times FS_{CBD}\right) \qquad \text{(Attachment C)}$$

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

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (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

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

$P_{actual\ field\ pressure}$ = 258.5 kPa

P'_{allow} = (258.5)(3) = 775.5 kPa

MF_S = shape factor: (Attachment C)
1.0 (assume angular particles)

MF_{PD} packing density: (Attachment C)
1.0 (assume isolated protrusions)

MF_A soil arching: (Attachment C)
1.0 (assume none)

FS_{CR} partial factor of safety for creep: (Attachment C)
for H>12 mm, FS_{CR} = 1.3

FS_{CBD} partial factor of safety for chemical and biological degradation: (Attachment C)
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} = (775.5) (1.0 × 1.0 × 1.0) (1.3 × 2.0)
P_{allow} = 2016 kPa

Case II: P_{allow} = (775.5) (1.0 × 1.0 × 1.0) (1.3 × 1.0)
P_{allow} = 1008 kPa

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

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

$$H_{cushion} = \left(\frac{450(542)}{2016}\right)^{1/2} = 11.0\ mm = 0.4\ in$$

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (YY MM DD) Reviewed by: GC Date: 05 / 11 / 23 (YY MM DD)

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

Case II: $H_{cushion} = \left(\frac{450(305)}{1008}\right)^{1/2} = 11.7 \text{ mm} = 0.5 \text{ in}$

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

7. REFERENCES

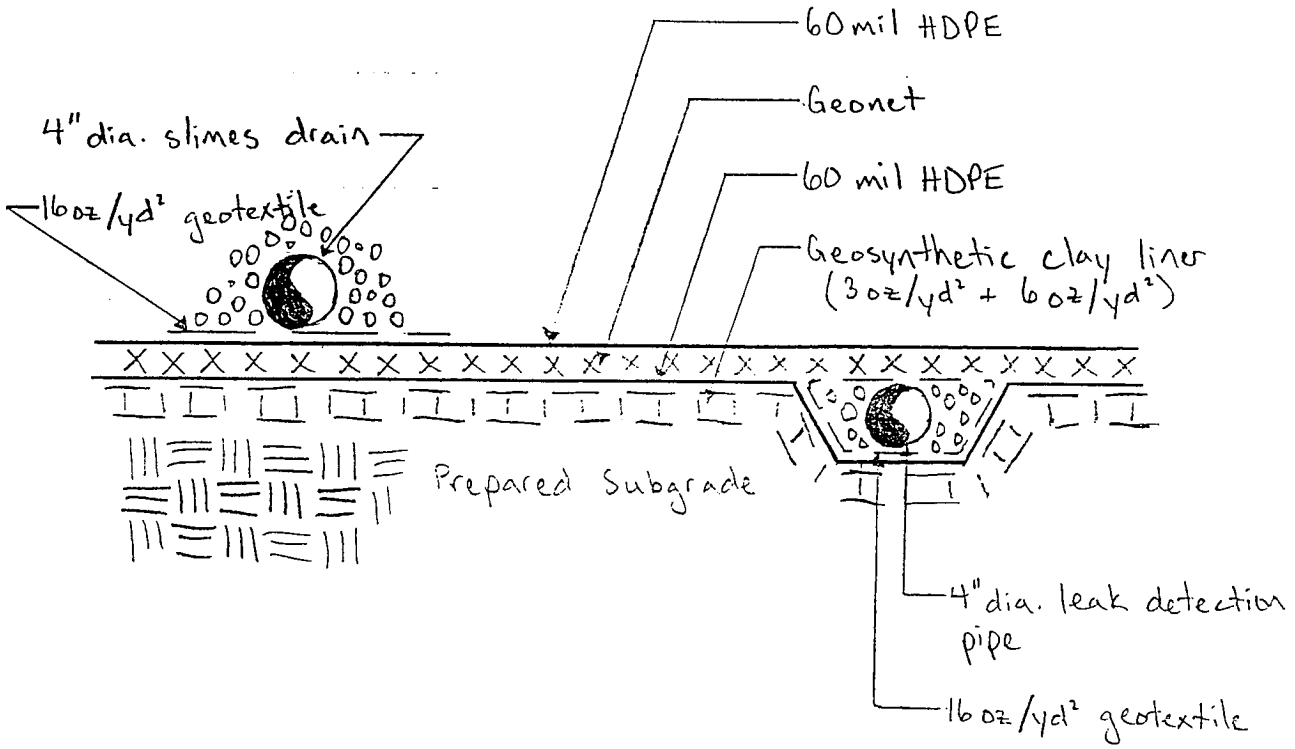
Koerner, R.M., Wilson-Fahmy, R.F. and Narejo, D. (1996) "Puncture Protection of Geomembranes Part III: Examples", Geosynthetics International, Vol. 3, No. 5, pp. 655-675.

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

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.

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

Client: IUC Project: Tailings Cell 4A Project/Proposal No.: SL0349 Task No.: 03



NOT TO SCALE

Attachment A

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)	Typical value of compression: Percent of original height, At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted), psf	Typical strength characteristics: Cohesion (saturated), psf	Typical strength characteristics: Effective stress envelope ϕ, degrees	Typical strength characteristics: tan ϕ	Typical coefficient of permeability, ft/min	Range of CBR values	Range of subgrade modulus k_s, lb/in^3
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	80–100	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

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

GTC
11/23/05

As previously explained, the logic behind the formulation is to first determine the failure pressure based on the short term hydrostatic truncated cone test data. A series of modification factors are then applied to correlate the truncated cone data to actual field conditions. The modification factors consider the stone shape, arrangement and soil arching. All of these modification factors have a magnitude of 1.0 or less since the experiments were conducted on a worst-case basis. Partial factors of safety are then incorporated into the design equations to account for creep and chemical/biological degradation. These partial factors of safety are equal to 1.0 or greater since longer periods of time are typically required for these factors to have an effect. Finally, a global factor of safety is applied to account for uncertainties in the formulation. The above described empirical formulation is presented in a step-by-step manner in order to emphasize the various factors involved.

6.2 Basic Design Equation

The formulation for predicting geomembrane failure pressure, p, is based on Figure 3 where it is seen that for each cone height, the failure pressure varies linearly with respect to the mass per unit area of the geotextile. Note that this failure pressure from the experiments is assumed to be the maximum allowable design pressure with an implied global factor of safety of 1.0. Thus, the maximum allowable pressure can be expressed as follows:

$$p_{allow} = d \times M_A \tag{1}$$

where: p_{allow} = maximum allowable pressure (with an implied factor of safety of 1.0); M_A = mass per unit area of the protection geotextile (g/m^2); and d = constant. From Figure 3, it is found that the parameter d can be related to the cone height, H, according to the following equation:

$$d = \frac{450}{H^2} \tag{2}$$

where H is in millimeters.

Combining Equations 1 and 2, the failure pressure can be determined in terms of the cone height and mass per unit area of the protection geotextile as follows (a minimum pressure of 50 kPa is imposed which conservatively corresponds to the failure pressure of the 1.5 mm thick HDPE geomembrane without any protection material):

$$p_{allow} = 450 \frac{M_A}{H^2} \geq 50 \text{ kPa} \tag{3}$$

The accuracy of the above equation is depicted in Figure 6 which shows the relationship between the measured failure pressure and the failure pressure predicted using Equation 3. The data in Figure 6 are for polyester geotextiles made from continuous filaments, and polypropylene geotextiles made of staple fibers. Hence, Equation 3 applies to essentially all of the polymer and fiber types used in the nonwoven needle-punched geotextiles.

Attachment C 1/3

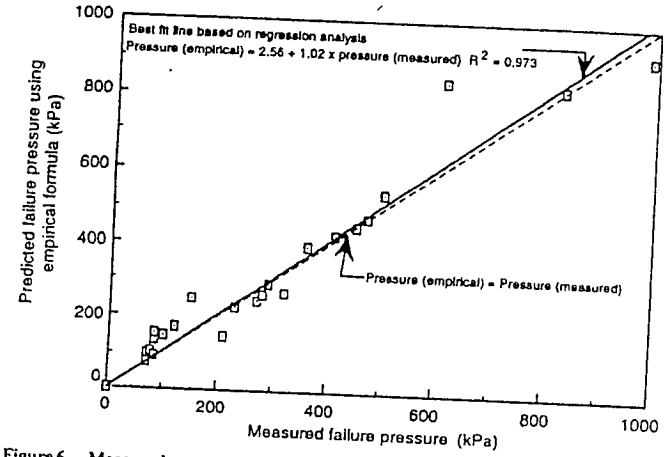


Figure 6. Measured versus empirically predicted failure pressures using Equation 3 for all nonwoven needle-punched geotextiles evaluated with a 1.5 mm thick HDPE geomembrane.
Note: R = correlation coefficient.

6.3 Modification Factors

A series of modification factors is now sequentially applied to Equation 3 in order to arrive at a pressure representing field conditions. The modified pressure will be referred to as p'_{allow}.

6.3.1 *Modification Factor for the Protrusion Shape*

It was previously shown that the failure pressure depends on the protrusion shape. Rounded stones gave the highest failure pressure followed by subrounded stones. The lowest failure pressure is associated with angular stones and is approximately equal to the failure pressure of truncated cones. In order to account for the effect of stone shape, a modification factor is introduced into Equation 3 as follows:

$$p'_{allow} = p_{allow}\left(\frac{1}{MF_S}\right) \tag{4}$$

where MF_S is the modification factor for the protrusion shape. Hereafter, p'_{allow} refers to the empirically modified value of p_{allow} as is illustrated in Figure 6.

Based on the analysis of the data presented in Section 5.2.1, the modification factors for different stone shapes are presented in Table 9.

Table 9. Modification factor for protrusion shape.

Stone shape	Modification factor, MF_S
Angular	1.00
Subrounded	0.50
Rounded	0.25

6.3.2 *Modification Factor for Packing Density*

It is shown in Section 5.2.2 that the allowable pressure for packed stones is much higher than for isolated stones. Unfortunately, within the capacity of the experimental device, no failure could be achieved with the packed stones, and hence, no direct correlation with isolated stones could be made. However, using the theoretical analysis presented in Part I of this series of papers (Wilson-Fahmy et al. 1996), the pressure at yield for packed stones ($R_o/H = 2$) could be compared with the pressure at yield for isolated stones ($R_o/H = 4$) where R_o is the horizontal distance from a undeformed geomembrane point of tangency with the protrusion tip to the undeformed geomembrane point of tangency with the soil subgrade. The analysis was performed for geomembranes with and without protection. Based on the results, a modification factor of 0.5 is suggested which provides a conservative estimate of the effect of packing density. Thus, Equation 4 can be rewritten after introducing a modification factor for packing density as follows:

$$p'_{allow} = p_{allow}\left(\frac{1}{MF_S \times MF_{PD}}\right) \tag{5}$$

where MF_{PD} is the modification factor for packing density. The modification values presented in Table 10 can be used for isolated protrusions and packed stone arrangements.

6.3.3 *Modification Factor for Soil Arching*

Equation 5 can be further modified as follows to include the effect of soil arching:

$$p'_{allow} = p_{allow}\left(\frac{1}{MF_S \times MF_{PD} \times MF_A}\right) \tag{6}$$

where MF_A is the modification factor for soil arching.

Table 10. Modification factors for packing density.

Protrusion arrangement	Modification factor, MF_{PD}
Isolated protrusions	1.00
Packed stones	0.50

It is shown in Section 5.2.3 that geostatic loading can lead to an increase in failure pressure by a factor of six in comparison with hydrostatic loading. This corresponds to a modification factor of 0.17. It may be noted, however, that the effect of soil arching on the pressure at yield may not be as great as the effect on the failure pressure. The deformations of the geomembrane up to yield may not be large enough to mobilize the soil arching effect; therefore, caution must be exercised when using the data in Table 7 for design. It is recommended that the values in Table 11 be used when soil arching is anticipated.

6.4 Partial Factors of Safety

After introducing the various modification factors (all of which are 1.0 or less), several partial factors of safety should be applied in order to determine the allowable pressure on the geomembrane. The partial factors of safety are equal to 1.0 or greater. Two factors are considered below, a partial factor of safety for long term creep and a partial factor of safety to account for long term chemical/biological degradation of the materials involved.

6.4.1 *Partial Factor of Safety for Creep*

A partial factor of safety for creep is incorporated into Equation 6, and the allowable pressure is now calculated as follows:

$$p'_{allow} = p_{allow}\left(\frac{1}{MF_S \times MF_{PD} \times MF_A}\right)\left(\frac{1}{FS_{CR}}\right) \qquad (7)$$

where FS_{CR} is the partial factor of safety for creep. Based on the creep data presented in Table 8, the recommended partial factors of safety for creep are given in Table 12.

Table 11. Modification factors for soil arching.

Soil arching effect	Modification factor, MF_A
None	1.00
Moderate	0.75
Maximum	0.50

Table 12. Partial factors of safety for creep.

Geotextile mass per unit area (g/m^2)	Partial factors of safety for creep			
	Protrusion height (mm)			
	38	25	12	6
No geotextile	N/R	N/R	N/R	>>1.5
270	N/R	N/R	>1.5	1.5
550	N/R	1.5	1.3	1.2
1100	1.3	1.2	1.1	1.0
>1100	~1.2	~1.1	~1.0	1.0

Note: N/R = not recommended.

It may be noted that the above partial factors of safety values for creep are relatively low in comparison with the factors of safety found in the literature for creep of geotextiles in tension. This may be explained by the fact that, in the puncture mode, the geomembrane and its protection material will conform more to the subgrade as they creep and hence the unsupported length will decrease with time. It was shown in Part I of this series of papers (Wilson-Fahmy et al. 1996) that for the same applied pressure the maximum stress mobilized at the protrusion tip will decrease as the unsupported length decreases. Thus, a decrease in stress in the geomembrane and its protection material is expected with time. Accordingly, a lower factor of safety for creep is required for the puncture mode in comparison to the stress mode in which the material is subjected to a constant tensile stress.

6.4.2 *Partial Factor of Safety for Chemical/Biological Degradation*

The partial factor of safety against chemical/biological degradation, FS_{CBD}, is included in Equation 7 as follows:

$$p'_{allow} = p_{allow}\left(\frac{1}{MF_S \times MF_{PD} \times MF_A}\right)\left(\frac{1}{FS_{CR} \times FS_{CBD}}\right) \qquad (8)$$

Although not assessed in this study, the value of FS_{CBD} is felt to range between 1.0 and 2.0 with an average value of 1.5; see Koerner (1994) for discussion and details.

6.5 Global Factor of Safety

After determining an allowable pressure that is suitably adjusted for modification factors and partial factors of safety (Equation 8), a global factor of safety is determined by dividing the allowable pressure by the required pressure as follows:

$$FS = \frac{p'_{allow}}{p_{reqd}} \qquad (9)$$

where: p_{reqd} = maximum stress required on the geomembrane; and FS = desired global factor of safety for uncertainties related to site specific conditions.

It is felt that the global factor of safety should never be less than 3.0. Higher values may be used depending on site specific conditions. For example, a high factor of safety should be used in situations where large isolated stones are frequently encountered on the subgrade. Also, a tightly installed geomembrane may also require a larger global factor of safety compared to a geomembrane installed with slack. Furthermore, no modification has been included for in situ temperatures different from the test procedure temperature, i.e. ≃ 20°C. More definitive recommendations for the global factor of safety are made in Part III of this series of papers (Koerner et al. 1996).

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: IUC **Project:** Tailings Cell 4A **Project/Proposal #:** SC0349 **Task #:** 03

Title of Computations: Geomembrane Tension Due to Wind Uplift

Computations By:
SIGNATURE — DATE: 05/11/08
Jennifer Ferguson, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE — DATE: 11/23/05
GREG CORCORAN ASSOCIATE
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE — DATE: 11/23/05
GREG CORCORAN ASSOCIATE
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):
SIGNATURE — DATE: 05/11/08
Jennifer Ferguson, Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE — DATE: 11/23/05
GREG CORCORAN ASSOCIATE
PRINTED NAME AND TITLE

Approval Notes: _____

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (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

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 4A 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):

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

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (YY MM DD) Reviewed by: GTC Date: 05 / 11 / 23 (YY MM DD)

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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 m

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 (3H:1V) is approximately 41 vertical feet, so the total length of exposed slope, L, is $L = \sqrt{41^2 + (3(41))^2} = 129.6\,\text{ft} = 39.5\,\text{m}$. See Attachment A for the conceptual base grading plan.

$$S_e L = 31.9\frac{N}{m^2}(39.5m)\times\frac{1kN}{1000N} = 1.26\frac{kN}{m}$$

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%.

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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):

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$$\frac{J}{S_e L} = \frac{675}{1.26} = 535.7, \quad \text{and from Table 4 (Attachment B, pg. 6), } \epsilon = 0.53\%$$

The maximum anticipated strain in the geomembrane that occurs is 0.53%, 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:

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

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 130 ft (39.5 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

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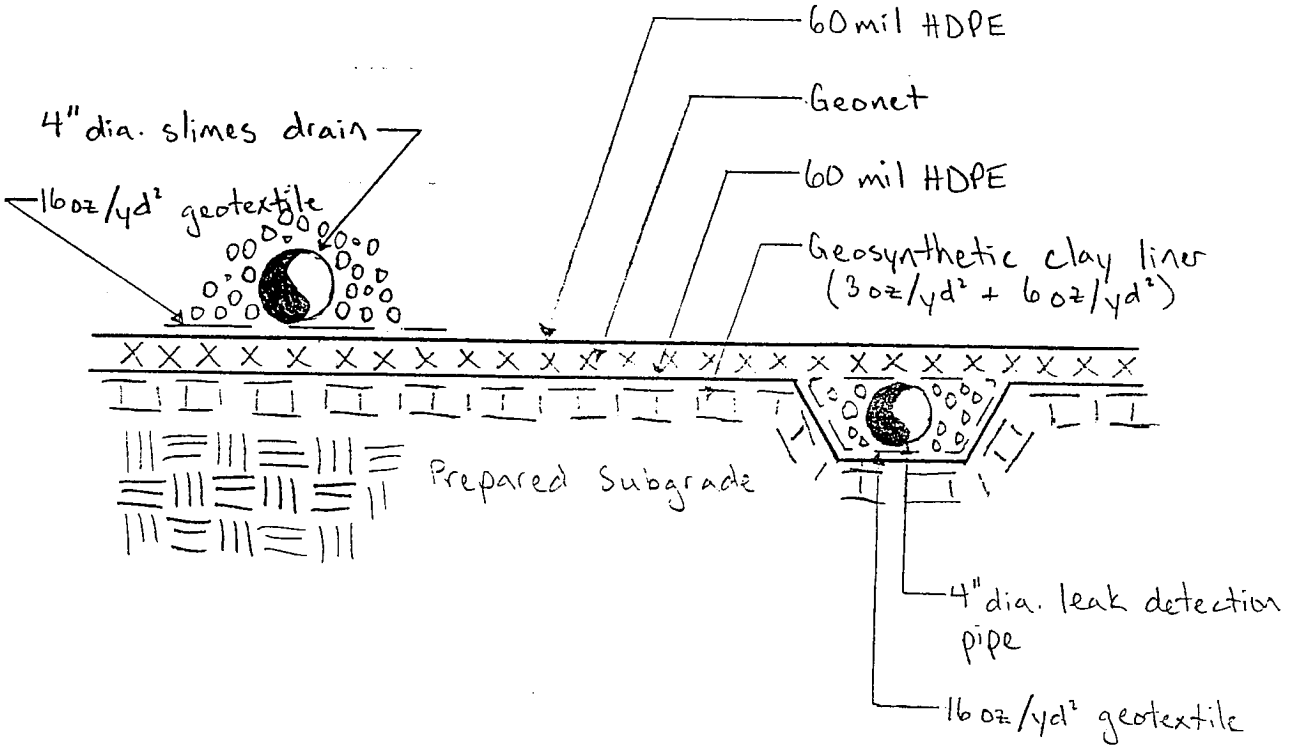
Attachment C

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

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

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



NOT TO SCALE

Attachment A

GTC
11/11/05

• At altitude z above sea level:

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

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

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

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:

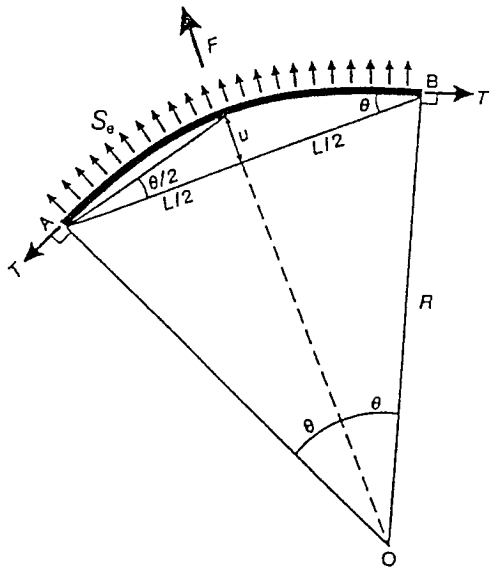


Figure 9. Schematic representation of uplifted geomembrane used for developing equations.

 Attachment B 1/6

- 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.

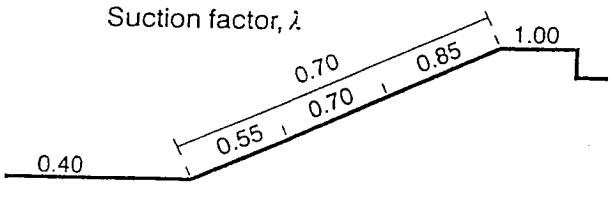


Figure 5. Recommended values of the suction factor for design of any slope based on the critical leeward slope.

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?

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:

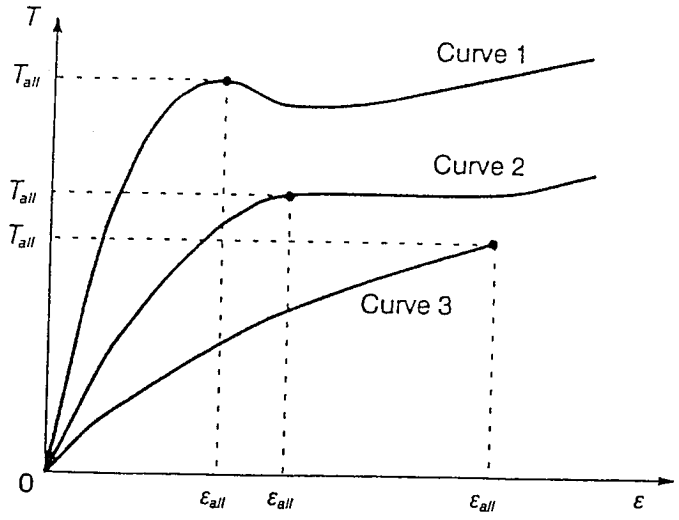


Figure 8. Typical tension-strain curves of geomembranes.

GTC 10/1/95

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

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)

Attachment B 5/6

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.

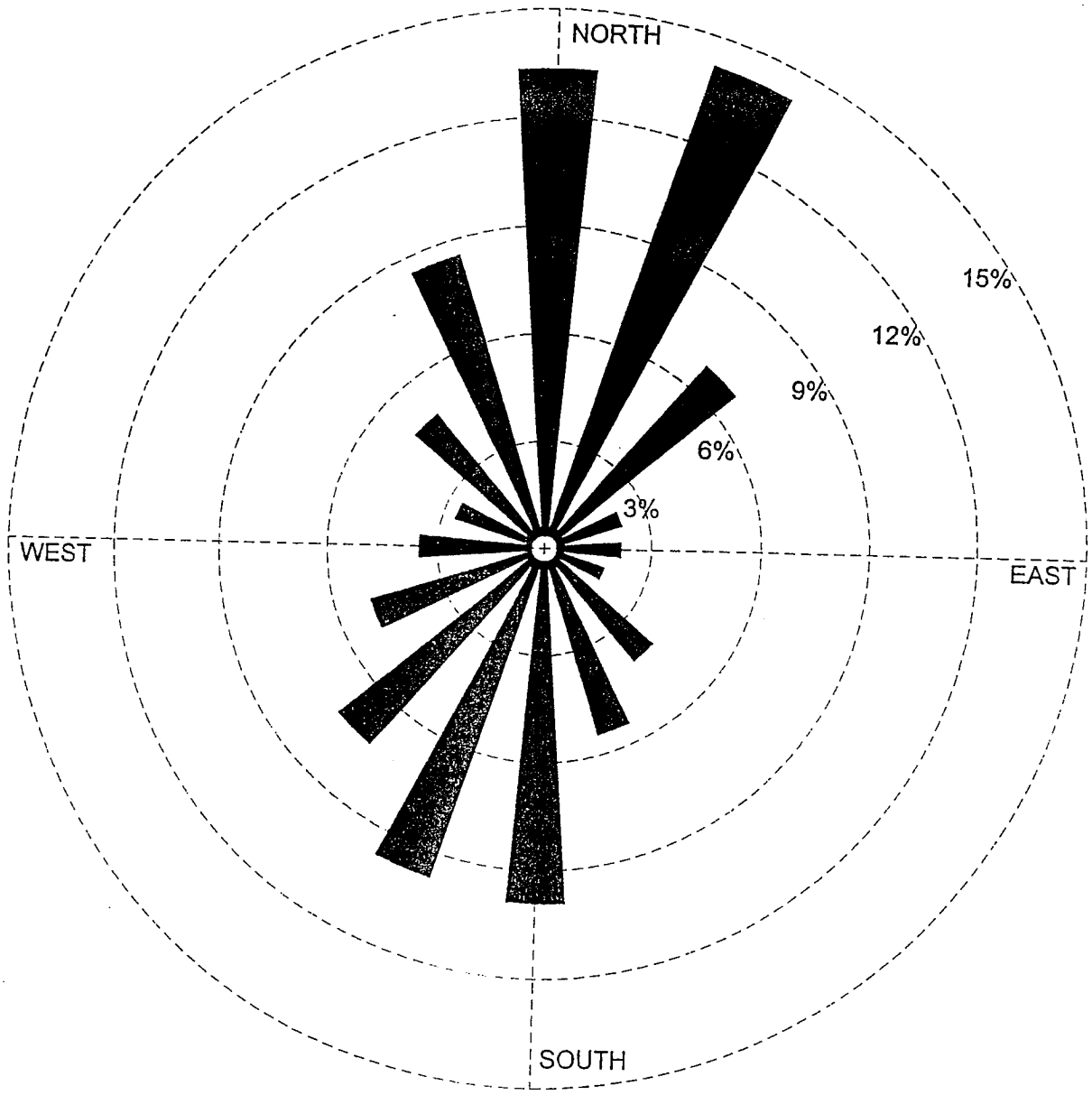
After the Mill construction was completed, the monitoring programs were modified to facilitate the assessment of Mill operations. The current meteorological monitoring program includes data collection for wind speed, wind direction, atmospheric stability according to the standard Pasquill scheme (via 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 ½



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.

GRI Update

Attachment D

GRC 1/11/05

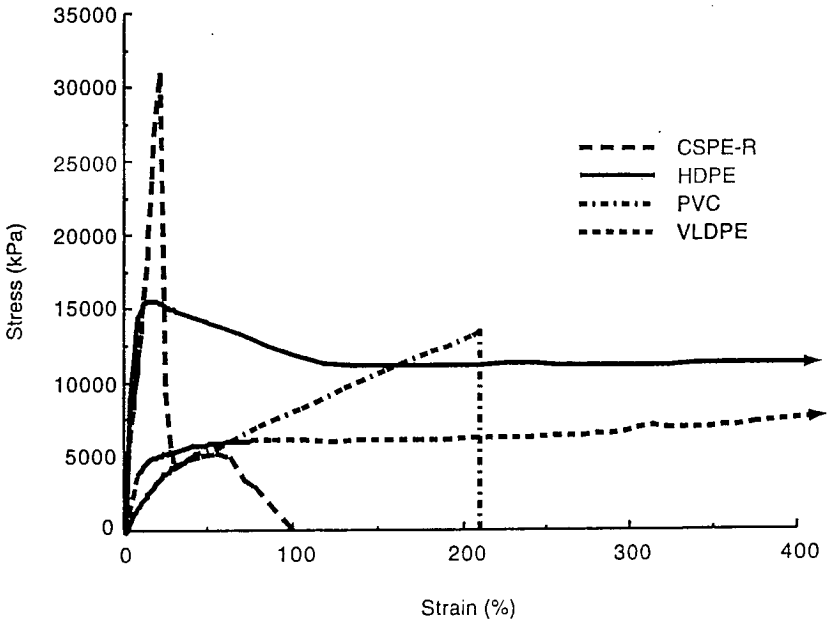


Figure 5.3 Tensile test results on 200 mm wide-width specimens of commonly used geomembranes using ASTM D-4885 test method.

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

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: IUC **Project:** Tailings Cell 4A **Project/Proposal #:** SC0349 **Task #:** 03

Title of Computations: Evaluation of Liner System Anchor Trench Capacity

Computations By:
SIGNATURE — DATE: 05/11/08
Jennifer Ferguson, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE — DATE: 11/23/05
GREG CORCORAN ASSOCIATE
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Approval Notes: _____

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (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

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 4A 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 4A 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

δ = 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-sinϕ

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;

$F_{AT\text{-}BASE} = q \tan\delta (L_{AT})$

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*

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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 13 degrees will be used to represent the interface friction value between the geomembrane and the geonet (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 minimum unit weight of overburden soil of 105 pounds per cubic foot (pcf) was assumed (Attachment E). For evaluation of the effective horizontal overburden stress based on the coefficient of earth pressure at rest, a friction angle of 31 degrees was assumed for the soil (Attachment E).

From Equation (3):

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

$$\begin{aligned} F_{AT\text{-}SIDE1} &= (\sigma_h)_{ave} \tan\delta(d_{AT}) \\ &= K_o(\sigma_v)_{ave}(\gamma)\tan\delta(d_{AT}) \\ &= (1-\sin 31°)\left(\frac{1}{2}(2\text{ ft})\right)(105\text{ pcf})\tan 18°(2\text{ ft}) \\ &= 33.1\text{ lb/ft} \end{aligned}$$

$$\begin{aligned} F_{AT\text{-}SIDE2} &= (\sigma_h)_{ave} \tan\delta(d_{AT}) \\ &= K_o(\sigma_v)_{ave}(\gamma)\tan\delta(d_{AT}) \\ &= (1-\sin 31°)\left(\frac{1}{2}(2\text{ ft})\right)(105\text{ pcf})\tan 13°(2\text{ ft}) \\ &= 23.5\text{ lb/ft} \end{aligned}$$

$$\begin{aligned} F_{AT\text{-}BASE1} &= q\tan\delta(L_{AT}) \\ &= 2\text{ ft}(105\text{ pcf})\tan 18°(2\text{ ft}) \\ &= 136.5\text{ lb/ft} \end{aligned}$$

$$\begin{aligned} F_{AT\text{-}BASE2} &= q\tan\delta(L_{AT}) \\ &= 2\text{ ft}(105\text{ pcf})\tan 13°(2\text{ ft}) \\ &= 97.0\text{ lb/ft} \end{aligned}$$

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$$T_{ult} = F_{AT\text{-}SIDE1} + F_{AT\text{-}SIDE2} + F_{AT\text{-}BASE1} + F_{AT\text{-}BASE2}$$
$$= 33.1 + 23.5 + 136.5 + 97.0$$
$$\mathbf{T_{ult} = 290\ lb/ft > 245\ lb/ft \quad OK}$$

Evaluation of Maximum Backfill Density

The maximum anchor trench size was evaluated. If the geomembrane is placed in tension, the optimum failure mode should be designed for anchor trench pullout rather than yielding of the geomembrane. The yield strength of 60-mil textured geomembrane is 126 lb/in, or 1512 lb/ft (Attachment F). Therefore, the capacity of the anchor trench cannot be greater than 1512 lb/ft. The capacity of the anchor trench designed above (2 ft deep, 2 ft wide) has a capacity of 290 lb/ft based on anchor trench backfill unit weight of 105 pcf. The maximum anticipated unit weight of anchor trench backfill is 125 pcf (Attachment E). The following calculation evaluates the trench capacity with the maximum unit weight of the backfill.

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

$$F_{AT\text{-}SIDE1} = (\sigma_h)_{ave}\tan\delta(d_{AT})$$
$$= K_o(\sigma_v)_{ave}(\gamma)\tan\delta(d_{AT})$$
$$= (1-\sin 31°)\left(\frac{1}{2}(2\text{ ft})\right)(125\text{ pcf})\tan 18°(2\text{ ft})$$
$$= 39.4\text{ lb/ft}$$

$$F_{AT\text{-}SIDE2} = (\sigma_h)_{ave}\tan\delta(d_{AT})$$
$$= K_o(\sigma_v)_{ave}(\gamma)\tan\delta(d_{AT})$$
$$= (1-\sin 31°)\left(\frac{1}{2}(2\text{ ft})\right)(125\text{ pcf})\tan 13°(2\text{ ft})$$
$$= 28.0\text{ lb/ft}$$

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

$$F_{AT\text{-}BASE2} = q\tan\delta(L_{AT})$$
$$= 2\text{ ft}(125\text{ pcf})\tan 13°(2\text{ ft})$$
$$= 115.4\text{ lb/ft}$$

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$T_{ult} = F_{AT\text{-}SIDE1} + F_{AT\text{-}SIDE2} + F_{AT\text{-}BASE1} + F_{AT\text{-}BASE2}$
$= 39.4 + 28.0 + 162.5 + 115.4$
T_{ult} = 345 lb/ft < 1512 lb/ft OK

Therefore, the 2 feet by 2 feet anchor trench is sufficiently wide such that a pullout failure will occur before the geomembrane yields.

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 290 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

GRI. (1990), "*Proceedings of the 4th GRI Seminar on the Topic of Landfill Closures*," December 14, 1990.
Attachment D

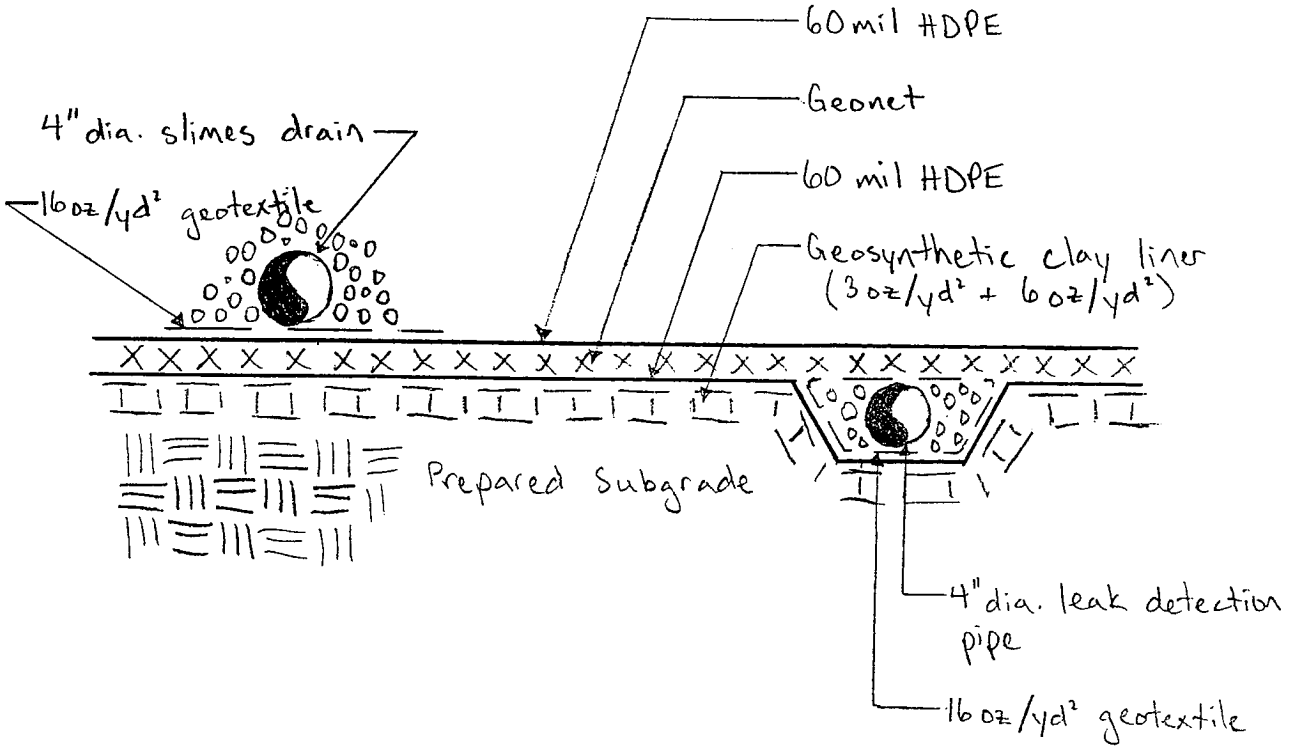
Written by: Jennifer Ferguson Date: 05 / 10 / 27 (YY / MM / DD) Reviewed by: GTC Date: 05 / 11 / 23 (YY / MM / DD)

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

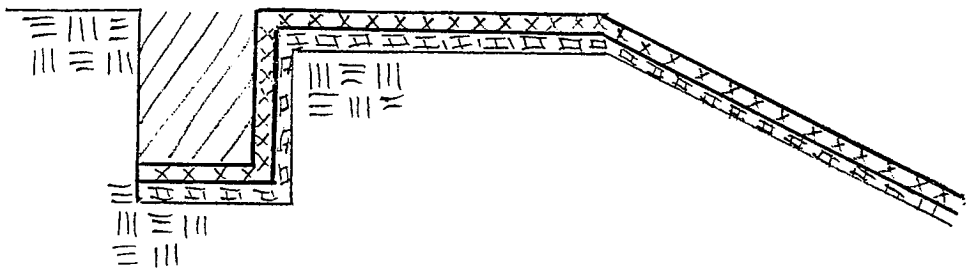
Hunt, Roy E. (1984), "*Geotechnical Engineering Investigation Manual.*"
Attachment E

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

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



ANCHOR TRENCH (dimensions: 2ft x 2ft)



NOT TO SCALE

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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.

$$\Sigma F_x = 0$$

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

Attachment B 1/8

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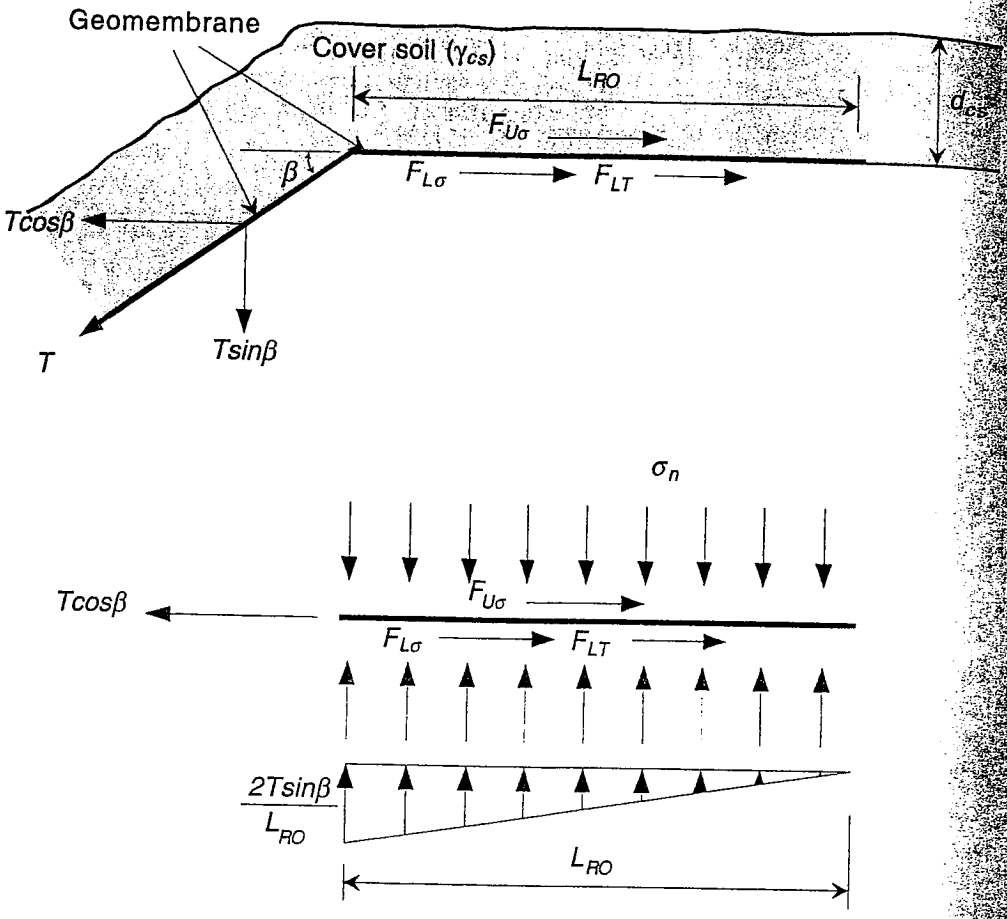

Figure 5.30 Cross section of geomembrane runout section and related stresses and forces involved.

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

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

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.*

$F_{L\sigma}$ = 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/m^3 and a friction angle of 30° with the geomembrane.

Solution: From the design equations just presented,

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

$$= (7000)(0.001)$$

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

and

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

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

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

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

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

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

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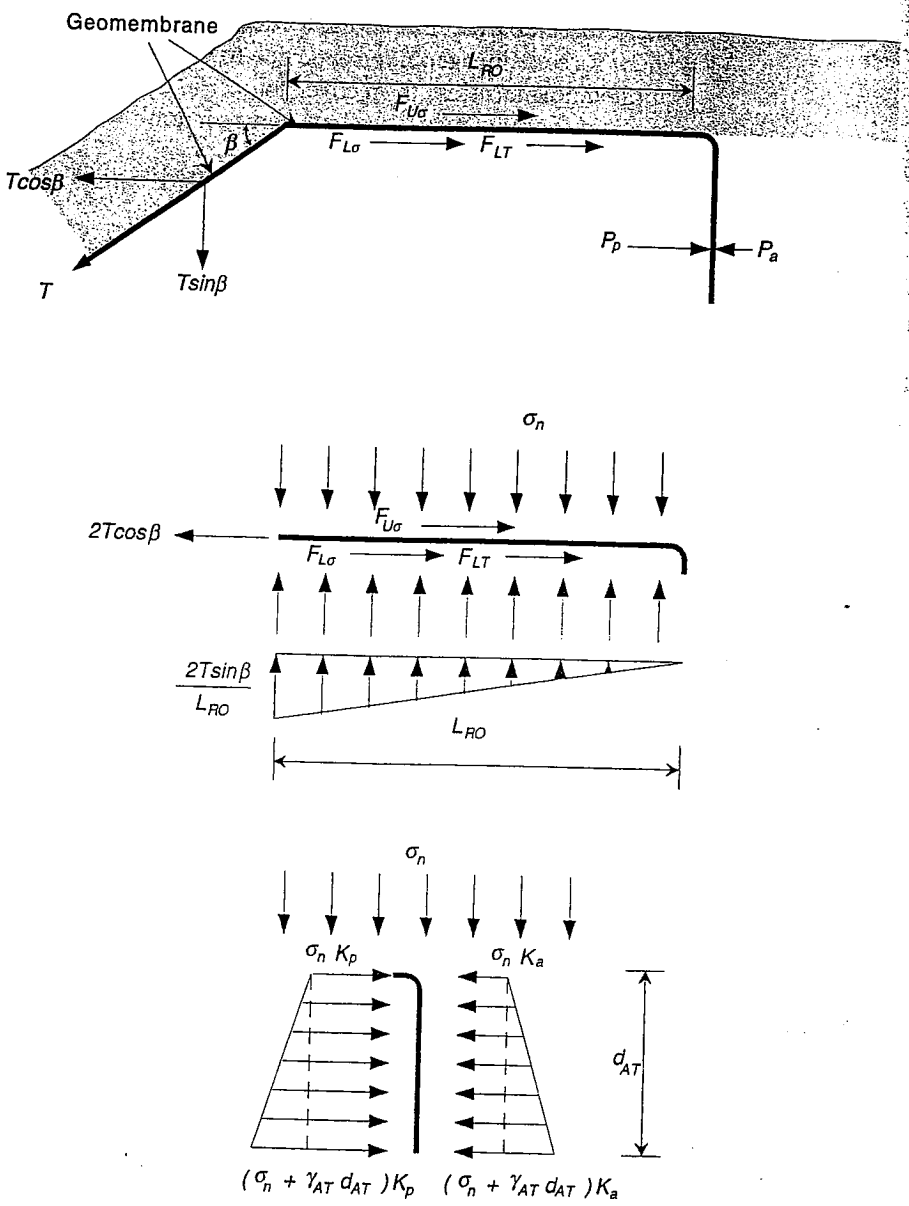

Figure 5.31 Cross section of geomembrane runout section with anchor trench and related stresses and forces involved.

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.*

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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/m^3 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.

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

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

and

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

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

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

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

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

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}.

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

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,

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

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

$$= (0.30)(16.5)$$

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

$$\delta_U = 0°$$

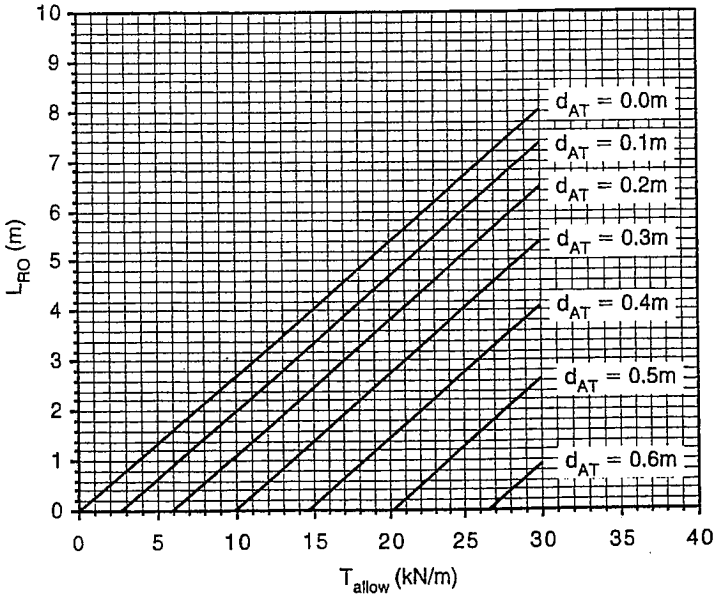
$$\delta_L = 30°$$

$$\phi = 35°$$

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

$$\delta_{AT} = 30°$$

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.*

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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.*

GSE*

TECHNICAL NOTE

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*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

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Attachment C

FROM - PROCEEDINGS OF THE 4th GRI SEMINAR ON THE TOPIC OF LANDFILL CLOSURES
GRI DEC 14, 1990

GTC 11/23/05

TABLE 4
GEONET VS. GEOSYNTHETICS INTERFACE FRICTION

NONWOVEN, NEEDLEPUNCHED	19°
NONWOVEN, HEAT BONDED	17° (.2 PSI)
MONOFILAMENT	9° (.2 PSI)
MULTIFILAMENT	18°
SLIT FILM	14° (.25 PSI)
HDPE - SMOOTH	13°
HDPE - ROUGH	19°
CSPE	24°
VLDPE	10° (.2 PSI)
PVC	13° (16° @ 3 PSI)

TABLE 5
GEOTEXTILE VS. GEOMEMBRANE INTERFACE FRICTION

GEOTEXTILE INTERFACE	HDPE (SMOOTH)	HDPE (ROUGH)	PVC	CSPE	VLDPE
WOVEN, SLIT FILM	8° (.375)	13° (.2)	18°	25°	11° (.25)
WOVEN, MONOFILAMENT	7° (.25)	9° (.3)	18°	23° (.5)	11° (.25)
WOVEN, MULTIFILAMENT	8° (.375)	17° (.6)	23°	26°	10° (.375)
NONWOVEN, HEATBONDED	9° (.25)	16°	21°	26°	12° (.25)
NONWOVEN, NEEDLEPUNCHED	10° (.4)	15° (2.5)	26°	28°	11° (.4)

The adhesive intercept is in parenthesis and in psi units.

Attachment D

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)	Typical value of compression: Percent of original height, At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted), psf	Typical strength characteristics: Cohesion (saturated), psf	Typical strength characteristics: Effective stress envelope ϕ, degrees	Typical strength characteristics: tan ϕ	Typical coefficient of permeability, ft/min	Range of CBR values	Range of subgrade modulus k_s, lb/in^3
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	80–100	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

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

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.

11/23/05

Attachment F

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: IUC **Project:** Tailings Cell 4A **Project/Proposal #:** SC0349 **Task #:** 03

Title of Computations: Pipe Strength Calculations

Computations By:
SIGNATURE — DATE: 05/11/08
Jennifer Ferguson, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE — DATE: 11/23/05
GREGORY T CORCORAN ASSOCIATE
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE — DATE: 11/23/05
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):
SIGNATURE — DATE: 05/11/08
Jennifer Ferguson, Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE — DATE: 11/23/05
GREGORY T CORCORAN ASSOCIATE
PRINTED NAME AND TITLE

Approval Notes: _____

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (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

PIPE STRENGTH CALCULATIONS
WHITE MESA MILL
BLANDING, UTAH

OBJECTIVE

The project involves placement of a double composite liner system for the base of Cell 4A 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 37.5 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 dry unit weight of 120 pounds per cubic foot (pcf);
- 4-in diameter schedule 40 PVC pipe, embedded in coarse aggregate for slimes drain;
- 60-mil HDPE geomembrane;
- Geonet and 4-in diameter schedule 40 PVC pipe, embedded in coarse aggregate for leak detection system (LDS);
- 60-mil HDPE geomembrane; and
- Geosynthetic clay liner (GCL).

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

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (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

ANALYSIS

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

Ring Deflection

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

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

where:

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

Evaluate Variables

Δ/D The allowable ring deflection for PVC pipe is 7.5 percent based on a factor of safety of 4 (Attachment C, 2/2)

P Prism soil load = 120 pcf × 45 ft = 5,400 psf = 37.5 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)

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 23 (YY MM DD)

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

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:

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

$$= \frac{1.0(0.1)(37.5) + 0.1(0)}{\left[\frac{2(400000)}{3(19.0-1)^3}\right] + 0.061(3000)} = 1.6\%$$

Since the calculated ring deflection is lower than the maximum allowable ring deflection, 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)

Written by: Jennifer Ferguson Date: 05 / 10 / 27 (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

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:

$$P_y = \frac{2(9600)(0.265)}{4.500} = 1129\,\text{psi}$$

Comparing the above estimated value to the maximum loading allowed under ring deflection criteria (37.5 psi) provides:

FS_{WC} = 1129/37.5
= 30.1

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

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

Wall Buckling

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

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

where:

P_{cr} Buckling pressure, psi

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

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

Therefore,

$$P_{cr} = \frac{2(400000)}{(19.0-1)^3} = 137 \text{ psi}$$

Comparing the above estimated value to the maximum loading allowed under ring deflection criteria (136 psi) provides:

FS_{WC} = 137/37.5
= 3.6

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 37.5 psi will only cause a ring deflection of 3.5 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.

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

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

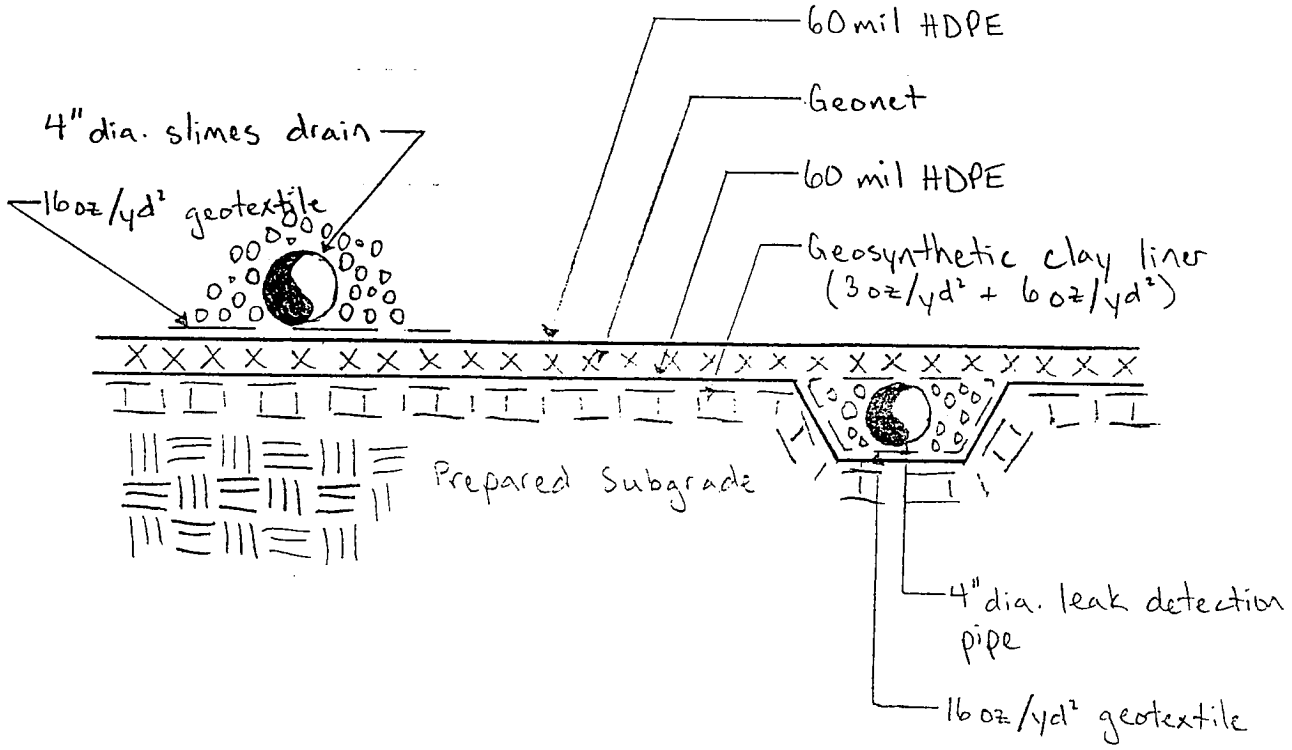
ASTM D 1785 (1996), "Standard Specification for Poly (Vinyl Chloride) (PVC) Plastic Pipe, Schedules 40, 80, and 120" ASTM Annual Method of Standards - Plastics

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

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

The Uni-Bell PVC Pipe Association

Handbook of PVC Pipe

Design and Construction

Uni-Bell PVC Pipe Association

2655 Villa Creek Drive, Suite 155

Dallas, Texas 75234

$40.00

Attachment B, 1/8

TABLE 6.3 - Continued

Height of Cover (ft)	Soil Unit Weight (lb/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 ($I_f = 0.766 - 0.133H$; $0 \le I_f \le 0.50$)

Note: ...andard 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.

CHAPTER VII

DESIGN OF BURIED PVC PIPE

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

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

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

course, in flat plate or three-edge loading, the rigid pipe will support nore than the flexible pipe. This anomaly tends to mislead many be flexible pipe users because they relate low flat plate supporting n 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 for flexible pipes.

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

EQUATION 7.1

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

id wall pipes Equation 7.1 can be rewritten as:

EQUATION 7.2

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

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

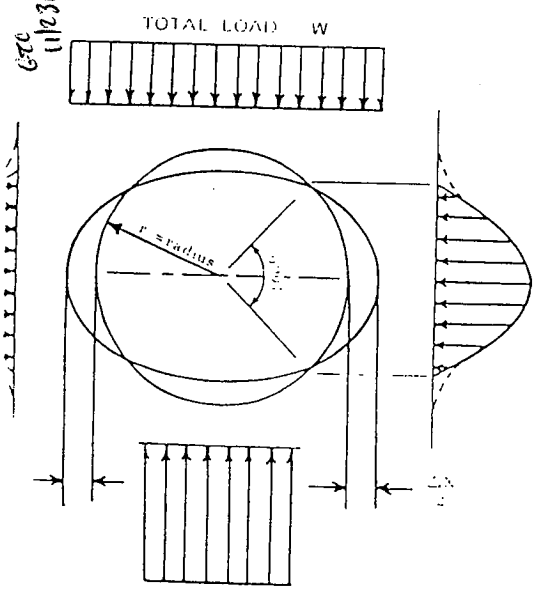
The resulting PS values for various dimension ratios and E values of PVC pipe are as shown in Table 7.1.

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 midpoint of the pipe wall.

FIGURE 7.3

ASIS OF SPANGLER'S DERIVATION OF THE IOWA ORMULA 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

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

ere:
- 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.

tion 7.9 can be used to predict deflections of buried pipe if the three constants 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

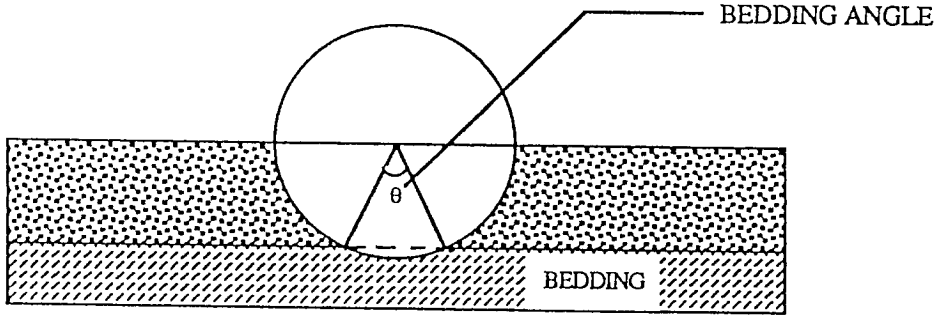


TABLE 7.2

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.

fiently, a new formula called the Modified Iowa Formula was writ-

EQUATION 7.10

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

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

ny research efforts have attempted to measure E' without success. ost useful method has involved the measure of deflections for a pipe vhich other conditions were known followed by back-calculation the Modified Iowa Formula to determine the correct value of E'. uires assumptions regarding the load, bedding factor and deflection or to be used and has led to a variation in reported values of E'.

attempt to acquire information on values of E' was conducted by K. Howard of the United States Bureau of Reclamation. Howard d both laboratory and field data from many sources. Using infor- from over 100 laboratory and field tests, he compiled a table of aver- values for various soil types and densities (see Table 7.3). He was do this by assuming values of E', K and W_c and then using the d Iowa Formula to calculate a theoretical value of deflection. This cal deflection was then compared with actual measurements. By as- the E' values of Table 7.3, a bedding constant K = 0.1 and deflec- factor D_L = 1.0, Howard was able to correlate the theoretical and al results to within ± 2 percent deflection when he used the prism l. This means that if theoretical deflections using Table 7.3 were nately 5 percent, measured deflection would range between 3 and 7

Although the vast majority of data from this study was taken from steel and reinforced plastic mortar pipe with diameters greater than s, it does provide some useful information to guide designers of all pipe including PVC pipe since it helps to give an understanding of lified Iowa Deflection Formula.

TABLE 7.3

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/m^3) (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.

GTC 11/23/05

TABLE 7.4

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

Depth of Cover →	2'		4'		6'		8'		10'		12'		14'		16'		18'		20'	
Live 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

6/9

Deflection Lag and Creep: The length of time that a buried flexi
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 decreases

In fact, after the trench load reaches a maximum, the pipe-soil syst
continues to deflect only as long as the soil around the pipe is in the proc
of consolidation. Once the embedment soil has reached the density requi
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 eff
tively compensating for the increased trench consolidation load with ti
and resulting increased deflection. When deflection calculations are ba
on prism loads, the deflection lag factor, D_L, should be 1.0.

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

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

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

...nches is recommended for PVC (DR 35) pipe subjected to highway
: 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-
led.

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

reverse curvature performance limit for flexible steel pipe was estab-
l shortly after publication of the Iowa Formula. It was determined that
gated 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-
hus providing a structural safety factor of 4.0. From this early design
leration, an arbitrary design value of 5.0 percent deflection was se-

uried PVC sewer pipe (ASTM D 3034, DR 35), when deflecting in re-
e to external loading, may develop recognizable reversal of curvature
eflection of 30 percent. This level of deflection has been commonly
nated as a conservative performance limit for PVC sewer pipe. Re-
ı 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-
n of this performance characteristic of PVC sewer pipe, engineers
ılly consider the 7.5 percent deflection limit recommended by ASTM
4 (Appendixes) to provide a very conservative factor of safety against
ıral failure.

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

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

EQUATION 7.13

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

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

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

EQUATION 7.14

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

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

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

EQUATION 7.15

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

or for solid wall pipes

EQUATION 7.16

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

ure were to exist on the inside of the pipe? The pipe will not be
ted in this installation.

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

ple:

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

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

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

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

nit deflection to 7.5 percent:

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

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

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

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

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

aximum cover is limited by the allowable deflection not by buckling. herefore, the safety factor for the critical failure mode by buckling of PVC pipe is ample.

search has established that flexible steel pipe walls can buckle at de- ns considerably less than 20 percent if the load is large and the soil nding the pipe is extremely compacted. Based on these observations,

H. E. White and J. P. Layer proposed the "Ring Compression Theory" for 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:

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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.

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

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

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

ATTACHMENT C, 1/2

GTC 11/23/05

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.

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

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

where:

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

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

Nominal Size, in.	Value for C	
	in.	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

Designation: D 1785 – 96b

An American National Standard

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[4]

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 (3.56)	+0.020 (+0.51)	0.191 (4.85)	+0.023 (+0.58)	0.215 (5.46)	+0.026 (+0.66)
1½	0.145 (3.68)	+0.020 (+0.51)	0.200 (5.08)	+0.024 (+0.61)	0.225 (5.72)	+0.027 (+0.68)
2	0.154 (3.91)	+0.020 (+0.51)	0.218 (5.54)	+0.026 (+0.66)	0.250 (6.35)	+0.030 (+0.76)
2½	0.203 (5.16)	+0.024 (+0.61)	0.276 (7.01)	+0.033 (+0.84)	0.300 (7.62)	+0.036 (+0.91)
3	0.216 (5.49)	+0.026 (+0.66)	0.300 (7.62)	+0.036 (+0.91)	0.350 (8.89)	+0.042 (+1.07)
3½	0.226 (5.74)	+0.027 (+0.68)	0.318 (8.08)	+0.038 (+0.96)	0.350 (8.89)	+0.042 (+1.07)
4	0.237 (6.02)	+0.028 (+0.71)	0.337 (8.56)	+0.040 (+1.02)	0.437 (11.10)	+0.052 (+1.32)
5	0.258 (6.55)	+0.031 (+0.79)	0.375 (9.52)	+0.045 (+1.14)	0.500 (12.70)	+0.060 (+1.52)
6	0.280 (7.11)	+0.034 (+0.86)	0.432 (10.97)	+0.052 (+1.32)	0.562 (14.27)	+0.067 (+1.70)
8	0.322 (8.18)	+0.039 (+0.99)	0.500 (12.70)	+0.060 (+1.52)	0.718 (18.24)	+0.086 (+2.18)
10	0.365 (9.27)	+0.044 (+1.12)	0.593 (15.06)	+0.071 (+1.80)	0.843 (21.41)	+0.101 (+2.56)
12	0.406 (10.31)	+0.049 (+1.24)	0.687 (17.45)	+0.082 (+2.08)	1.000 (25.40)	+0.120 (+3.05)
14	0.437 (11.10)	+0.053 (+1.35)	0.750 (19.05)	+0.090 (+2.29)	...	...
16	0.500 (12.70)	+0.060 (+1.52)	0.843 (21.41)	+0.101 (+2.57)	...	...
18	0.562 (14.27)	+0.067 (+1.70)	0.937 (23.80)	+0.112 (+2.84)	...	...
20	0.593 (15.06)	+0.071 (+1.80)	1.031 (26.19)	+0.124 (+3.15)	...	...
24	0.687 (17.45)	+0.082 (+2.08)	1.218 (30.94)	+0.146 (+3.71)	...	...

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

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

ATTACHMENT D, 2/2

GTC 11/23/05

Designation: D 1784 – 92

An American National Standard

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 of Liquids[4]

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

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

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

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

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

D 790 Test Methods for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials[3]

D 883 Terminology Relating to Plastics[3]

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

D 1898 Practice for Sampling of Plastics[5]

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

D 3892 Practice for Packaging/Packing of Plastics[7]

D 3915 Specification for Poly(Vinyl Chloride) (PVC) and Related Plastic Pipe and Fitting Compounds for Pressure 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 Piping Products[7]

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

3. Terminology

3.1 *Definitions*—Definitions are in accordance with Definitions D 883 and abbreviations with Terminology D 1600 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 11/23/05

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 F

P 1/1 GCC 11/28/05

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	11,850	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 (Cal.) (cm) / (cm²) (sec.) (°C) x 10⁻⁴	C177	3.5	4.5	0.96
Coefficient of Linear Expansion	D696	5.2	9.9	6.2
x 10⁻⁵ cm/cm °C		2.9	5.5	3.4
x 10⁻⁵ in/in °F				
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-(), 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

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: International Uranium Corporation **Project:** White Mesa Mill, Pond 4A
Project/Proposal #: sc0349 **Task #:** 1

Title of Computations: Emergency Spillway Concrete Pavement

Computations By: _____ SIGNATURE — 1/6/06 DATE
Steven M. Fitzwilliam, Sr. Project Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer): _____ SIGNATURE — 1/6/06 DATE
Gregory T. Corcoran, Associate
PRINTED NAME AND TITLE

Computations Checked By: _____ SIGNATURE — 1/6/06 DATE
Gregory T. Corcoran, Associate
PRINTED NAME AND TITLE

Computations Backchecked By (Originator): _____ SIGNATURE — 1/6/06 DATE
Steven M. Fitzwilliam, Sr. Project Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate): _____ SIGNATURE — 1/6/06 DATE
Gregory T. Corcoran, Associate
PRINTED NAME AND TITLE

Approval Notes: _____

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Steve Fitzwilliam Date: 05/01/06 (YY MM DD) Reviewed by: Greg Corcoran GTC Date: 06/01/06 (YY MM DD)

Client: IUC Project: White Mesa Mill, Pond 4A Project/Proposal No.: SC0349 Task No.:

Emergency Spillway Concrete Pavement

OBJECTIVE

An emergency spillway will be constructed as part of the Pond 4A construction at the White Mesa Mill, in Blanding, Utah for the International Uranium Corporation (IUC). The emergency spillway is 94 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 4A, 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 60 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 GRADE DESIGN PROCEDURE**

Written by: Steve Fitzwilliam Date: 05/01/06 (YY/MM/DD) Reviewed by: Greg Corcoran GC Date: 06/01/06 (YY/MM/DD)

Client: IUC Project: White Mesa Mill, Pond 4A Project/Proposal No.: SC0349 Task No.:

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:

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

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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 60-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 165 plus 5 in-lb/in/kip, which results in a total moment of approximately 170 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:

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

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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 4.5 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.

Written by: Steve Fitzwilliam Date: 05/01/06 (YY MM DD) Reviewed by: Greg Corcoran GTC Date: 06/01/06 (YY MM DD)

Client: IUC Project: White Mesa Mill, Pond 4A Project/Proposal No.: SC0349 Task No.:

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

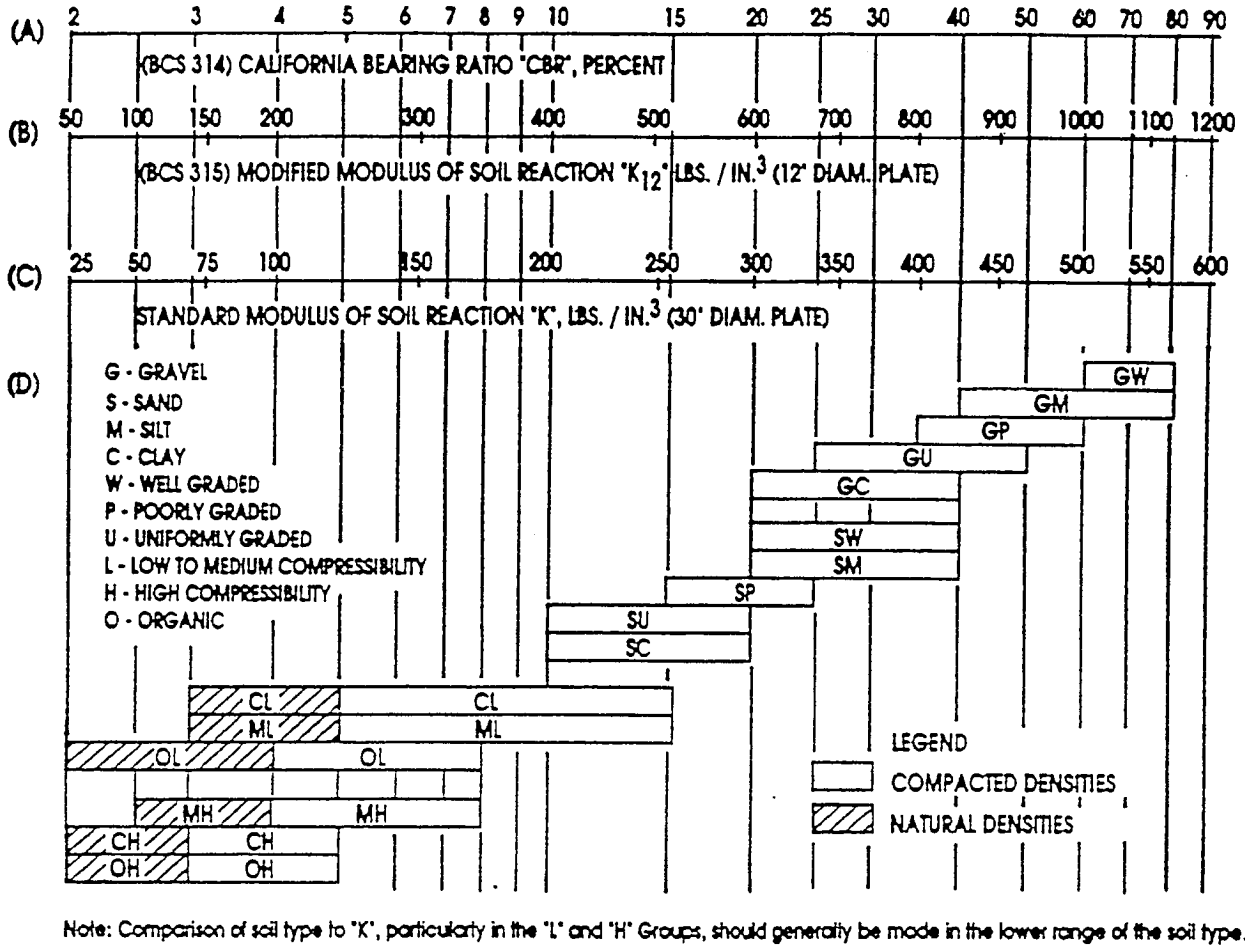


Fig. 3.3.5—Interrelationship of soil classifications and strengths (from Reference 23)

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

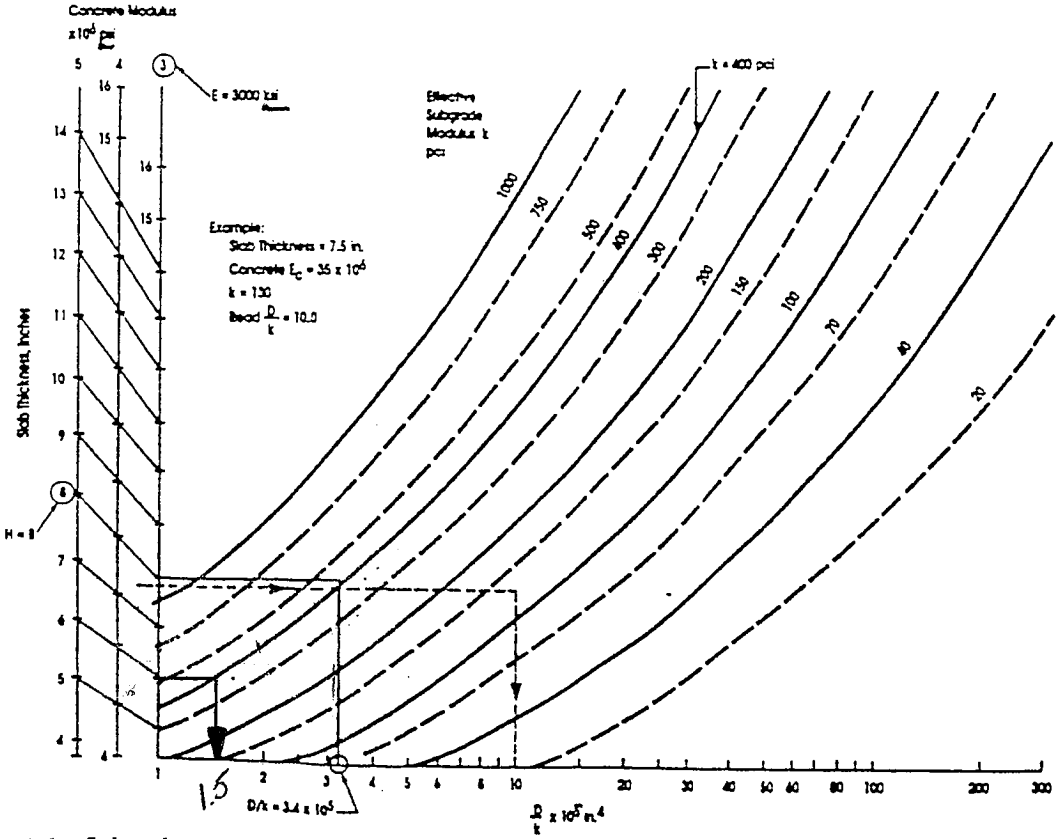


Fig. A2.2.1—Subgrade and slab stiffness relationship, used with WRI design procedure

ATTACHMENT B

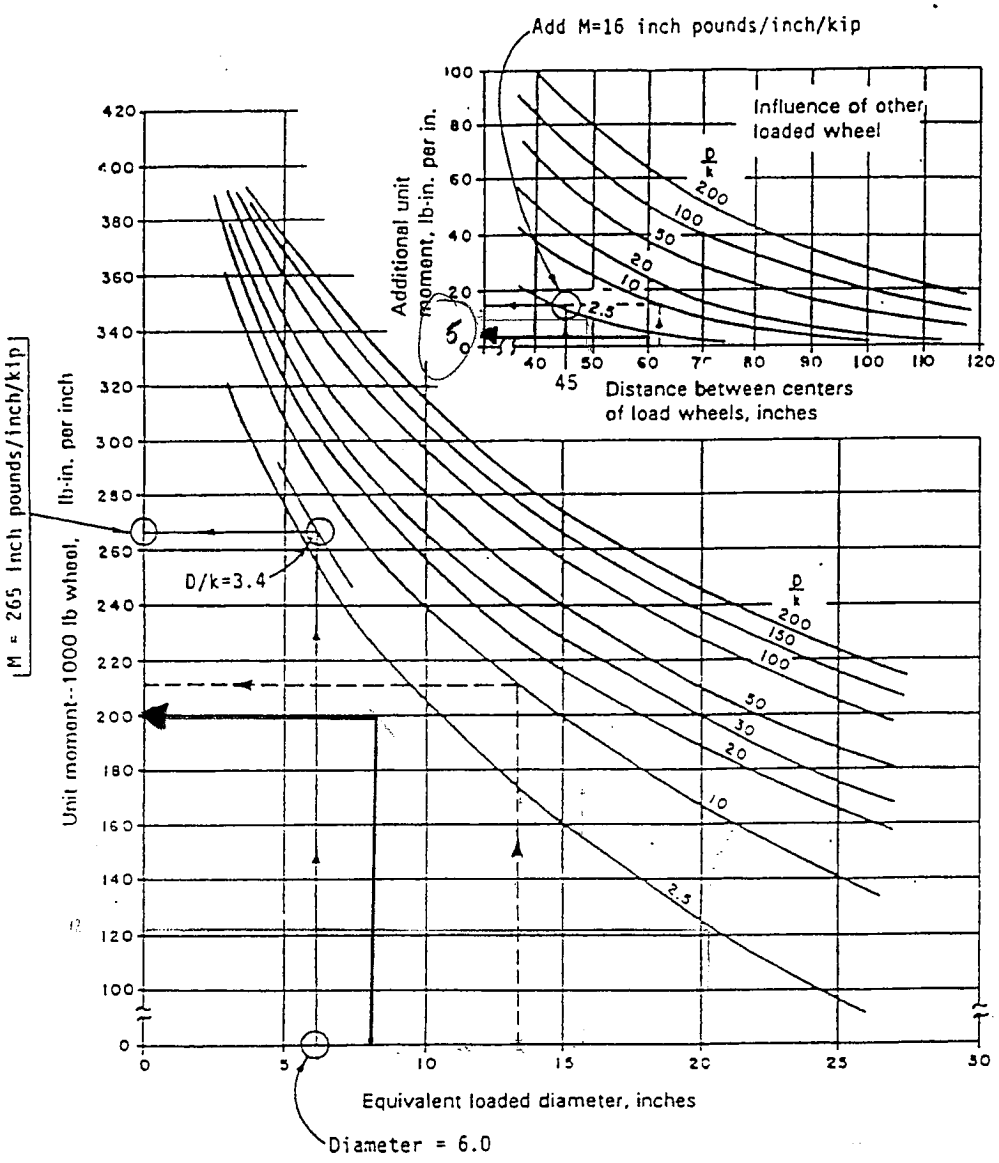


Fig. A2.2.2—Wheel loading design chart used with WRI procedure

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

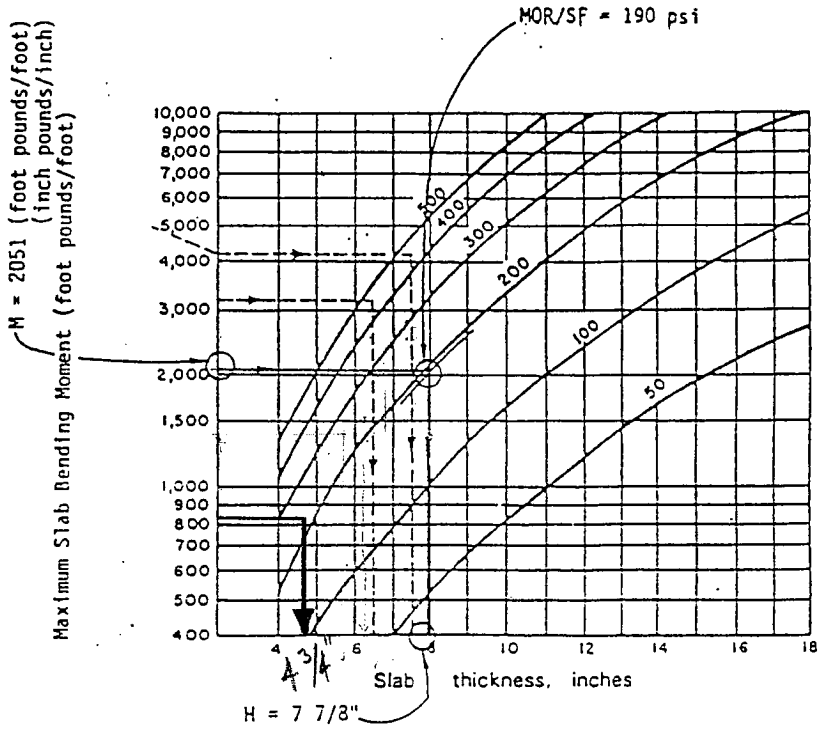


Fig. A2.2.3—Slab tensile stress charts used with WRI design procedure

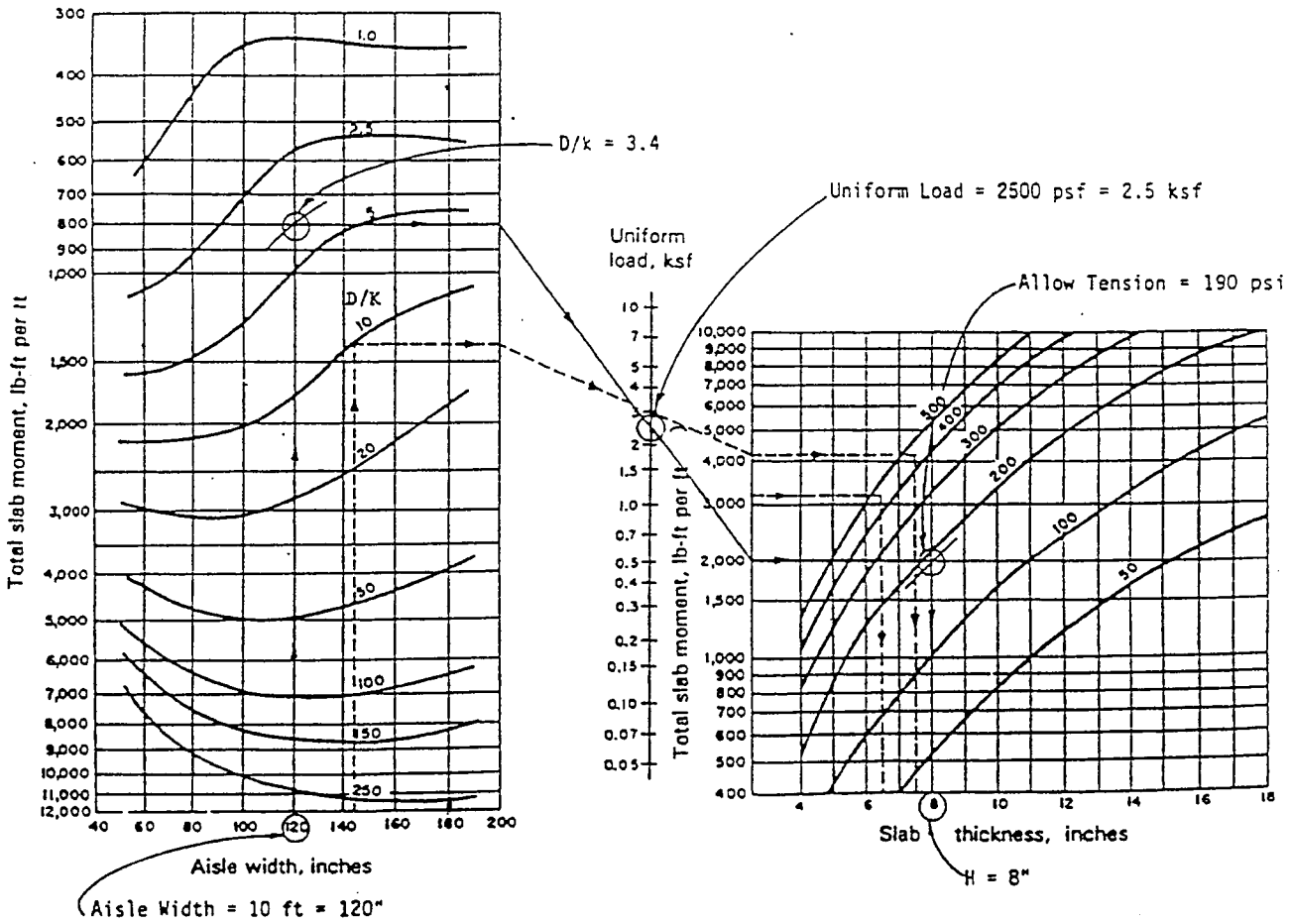


Fig. A2.3—Uniform load design and slab tensile stress charts used with WRI design procedure

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

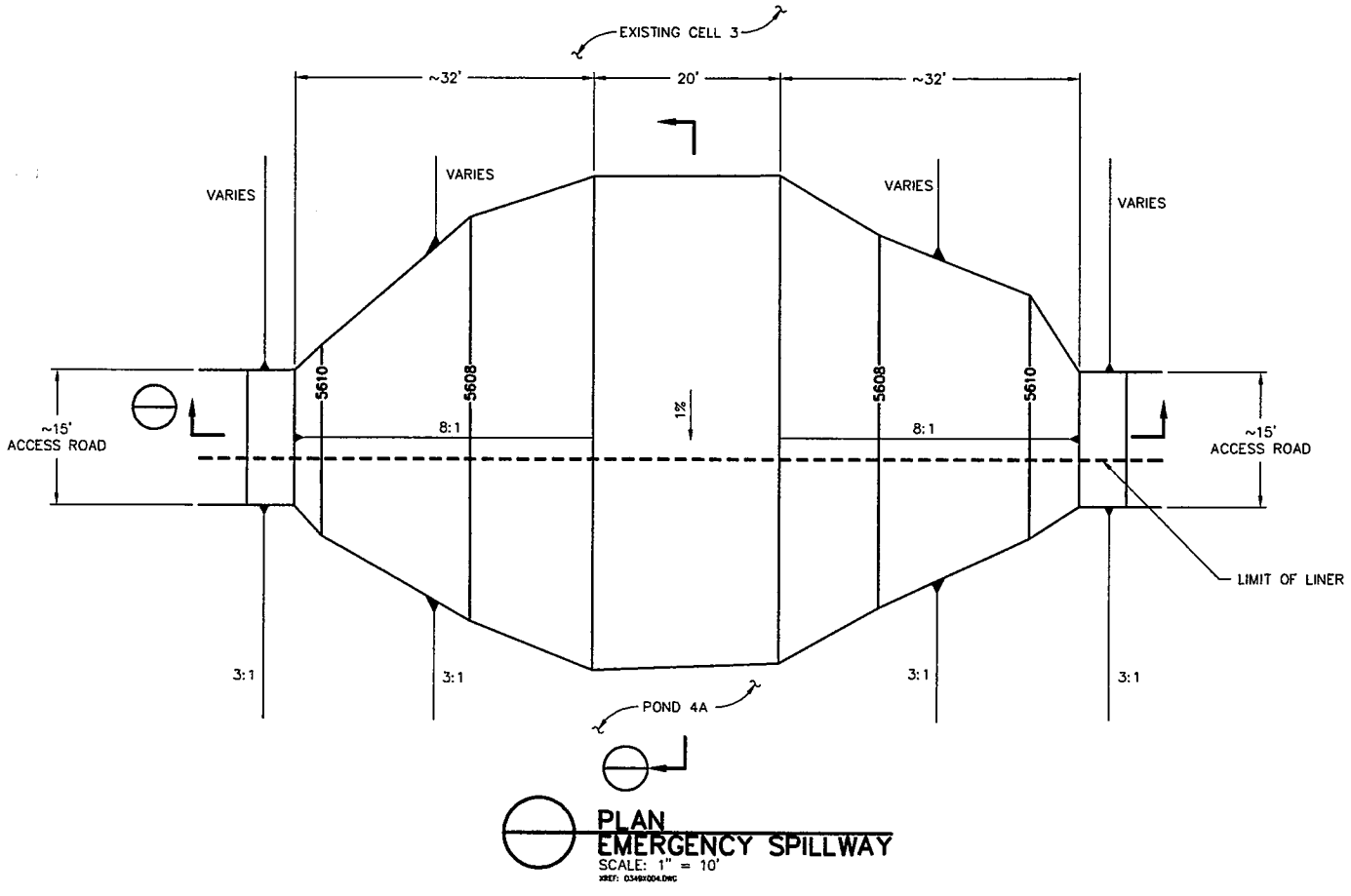
Sectional Areas of Welded Wire Reinforcement

TABLE 5 Customary Units

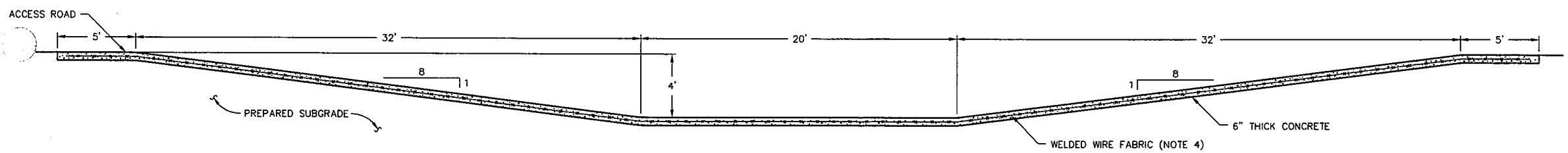
Wire Size Number		Nominal Diameter	Nominal Weight	As - Square Inch Per Linear Feet *Center to Center Spacing*								
Plain	*Deformed*	*Inches*	*Lbs./Lin. Ft.*	2"	3"	4"	6"	8"	10"	12"	16"	18"
W45	D45	0.757	1.530	2.70	1.80	1.35	.909	.68	.54	.45	.34	.30
W31	D31	0.628	1.054	1.86	1.24	.93	.62	.47	.37	.31	.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



PLAN
EMERGENCY SPILLWAY
SCALE: 1" = 10'
XREF: 0349I004.DWG



SECTION
EMERGENCY SPILLWAY
SCALE: 1" = 4'
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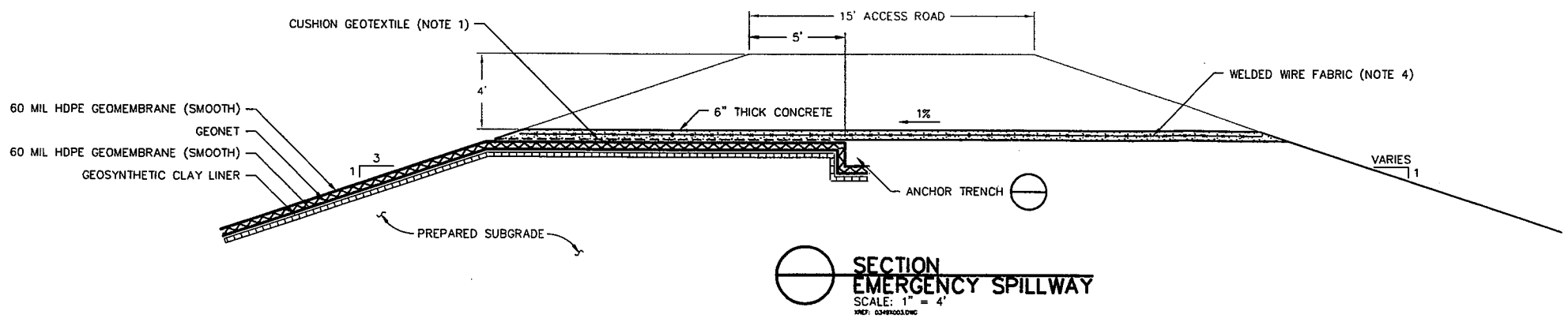

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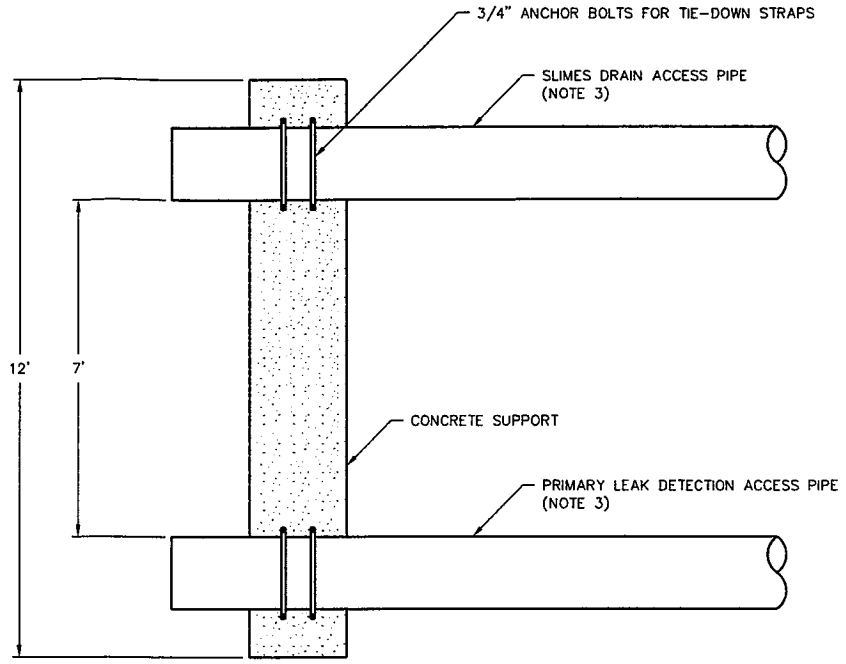

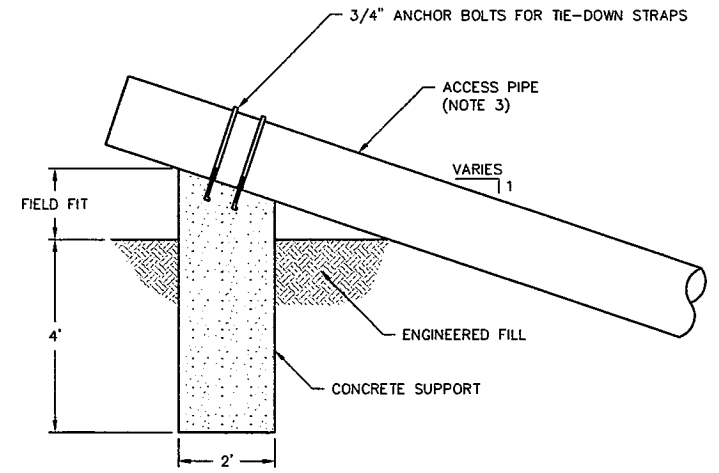

DETAIL
PIPE SUPPORT
SCALE: 1" = 2'
XREF: 0349I005.DWG

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 DUES TO UV DAMAGE.
4. WELDED WIRE FABRIC SHALL BE INSTALLED AT MIDSECTION OF CONCRETE SLAB.

INTERNATIONAL URANIUM (USA) CORPORATION
GEOSYNTEC CONSULTANTS
11305 RANCHO BERNARDO ROAD, SUITE 101
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT:
POND 4A, WHITE MESA MILL
BLANDING, UTAH

TITLE:
LINING SYSTEM DETAILS III

MARK	DATE	REVISION	BY	APPROVED

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

SIGNATURE

DATE

SEAL

DATE:	NOVEMBER 2005	SCALE:	AS SHOWN
DESIGN BY:	GTC	JOB NO.:	SC0349-01
DRAWN BY:	SLB	FILE NO:	0349C007
CHECKED BY:	GTC	DOCUMENT NO:	-----
REVIEWED BY:	GTC	DRAWING NO:	7 OF 7
APPROVED BY:	GTC		

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GeoSyntec Consultants

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COMPUTATION COVER SHEET

Client: IUC **Project:** White Mesa Mill – Cell 4A Design **Project/Proposal #:** SC0349 **Phase #:** 04

Title of Computations: Cell 4A Spillway Capacity Calculations

Computations By:

SIGNATURE — 01/09/06 DATE

Nathan Jacobsene, Project Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE — 1/9/06 DATE

Gregory Corcoran, Associate
PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE — 1/9/06 DATE

Gregory Corcoran, Associate
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):

SIGNATURE — 01/09/06 DATE

Nathan Jacobsen, Project Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):

SIGNATURE — 1/9/06 DATE

Gregory Corcoran, Associate
PRINTED NAME AND TITLE

Approval Notes: _____

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

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

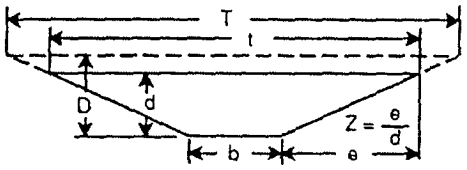
Written by: Nathan Jacobsen Date: 06/01/09 Reviewed by: GTC Date: 06/01/09
S YY MM DD YY MM DD

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).

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

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

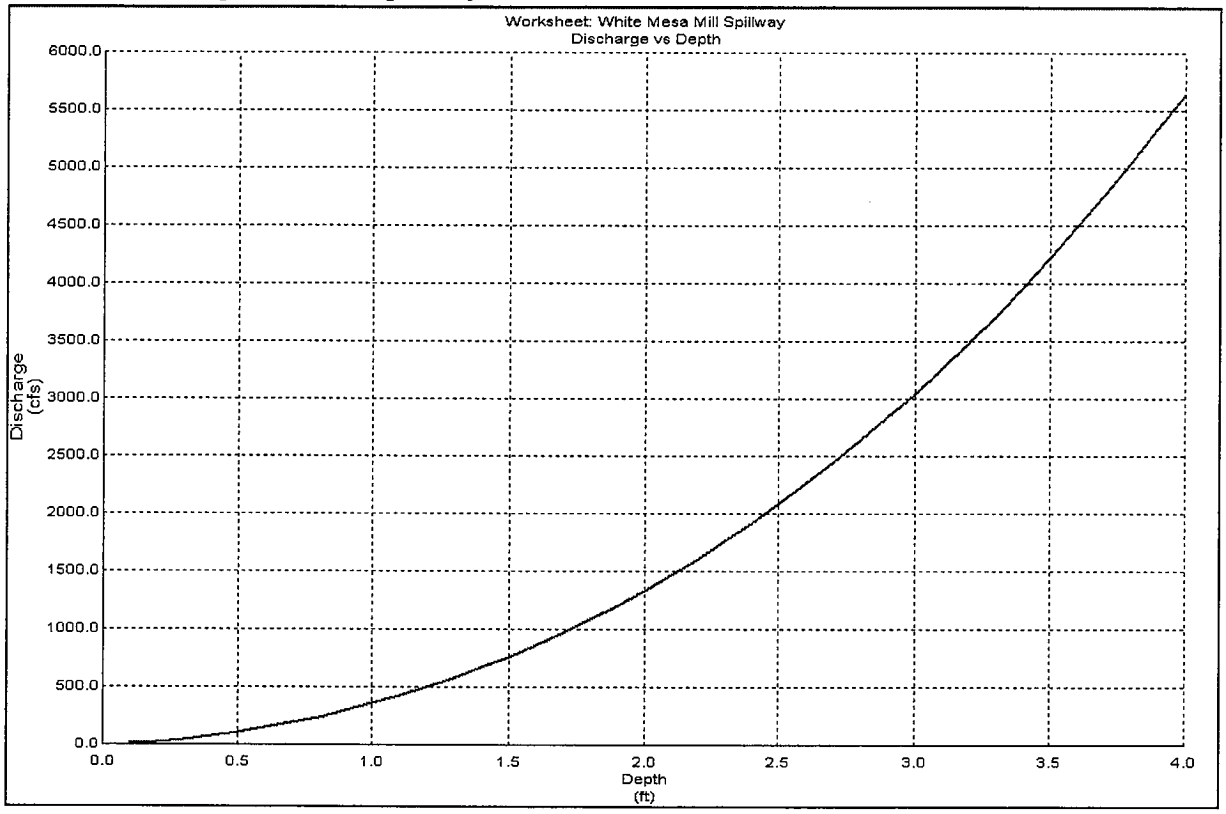



Figure 2 – Rating Curve for Spillway 4A.